The British Dam Society

BDS Biennial Conference 2024

Managing Risks for Dams and Reservoirs – Papers

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A review of the applicability of the EA reservoir flood mapping specification for reservoir risk assessments

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SYNOPSIS In 2021, the Environment Agency (EA) published new reservoir flood maps of all the statutory reservoirs in England. These maps are intended to be used for a range of purposes related to flood risk and planning. In parallel, the EA also undertook to assess the Average Societal Life Loss (ASLL) associated with a breach for each reservoir, although this information has not been made publicly available.

The new reservoir flood maps (and ASLL figures) developed by the EA were assessed following the guidance provided in the EA's Reservoir Flood Mapping (RFM) Specification (EA, 2019). This was a nationwide exercise and therefore some broad assumptions had to be adopted so the methodology could be applied to all the reservoirs.

This paper presents a review of the EA's RFM Specification and associated technical papers to understand where there is the potential for conservatism in the assumptions made when developing EA Breach hydrographs and ASLL figures. This will equip reservoir undertakers with an understanding of the applicability of the data for use in assessing the societal risk posed by a reservoir.

INTRODUCTION

EA RFM Specification Background

Reservoir flood maps are used to inform people about areas at risk of flooding in the event of a dam or reservoir failure and sudden uncontrolled escape of water. In 2007, Sir Michael Pitt recommended creating national flood maps for reservoir failure, to enable Local Resilience Forums to assess risks and plan for contingency, warning and evacuation. The Reservoir Inundation Mapping (RIM) Specification, now known as Reservoir Flood Mapping (RFM) Specification, was established in 2009 (EA, 2009) and used to produce a total of 2,232 reservoirs flood maps in England and Wales.

In 2021, the Environment Agency (EA) published new reservoir flood maps for all the statutory reservoirs in England. These updated maps were produced following the methodology of an updated RFM Specification that had been published in 2016 and revised in 2019. The new specification superseded the 2009 RIM Specification. The review process was informed by an improved understanding of flood risk, incorporating advancements in modelling, analysis, and legislative considerations.

Managing Risks for Dams and Reservoirs

The principal changes in the RFM Specification (EA, 2019) included:

- Change in the terminology of the dam failure scenario from "credible upper case" to "reasonable worst case" for consistency with wider fields.
- Introduction of a new "dry day" scenario which represents a dam failure when the reservoir level is at top water level and there is no associated river flooding.
- For the "wet day" scenario (already present in the 2009 RIM Specification), explicit modelling with and without dam failure to assess the incremental effect of a dam break over and above the river flooding.
- More realistic representation of the flooding downstream by using 1 in 1000 chance per year (0.001% Annual Exceedance Probability) fluvial flood event to represent the "wet day" scenario.
- The calculation of peak flow during a breach in an embankment uses the Xu and Zhang (2009) formulation rather than Froehlich (1995), and time to peak uses Froehlich (2008) rather than simple multiplier on height.
- Revision of the water levels and volume at time of breach for the "wet day".

The detailed flood maps were produced for emergency planning and are key components of the on-site emergency plans that have recently been prepared to meet 2021 Flood Plan Ministerial Direction.

As part of the 2021 reservoir flood mapping exercise, the EA also calculated the Average Societal Life Loss (ASLL) and damages for all scenarios (dry day, wet day, fluvial only) associated with the worst breach location for each reservoir.

Applicability of the EA RFM to reservoir risk assessment

The Guide to Risk Assessment for Reservoir Safety Management (RARS) (EA, 2013) states that the societal risk posed by the presence of a reservoir can be classed as "Tolerable", "ALARP (as low as reasonably practicable)" or "Unacceptable". Two components inform the tolerability of the risk posed by a reservoir: the probability of dam failure and the consequences of failure. The consequences of failure are quantified in terms of the ASLL. A typical F-N chart which is a graphical representation of the level of societal risk generated by some activity is shown in Figure 1.

If a reservoir falls within the "Unacceptable" zone, the undertaker will likely need to take measures to reduce the risk and/or consequences of reservoir failure. If a reservoir falls within the ALARP zone, as a minimum, the undertaker will likely be required to perform a proportionality assessment between the cost of risk reduction measures and the expected reduction in risks.

For Tier 2 and Tier 3 (quantitative) risk assessments, RARS states that existing dam break maps can be used where available and of an appropriate standard. The EA reservoir flood maps are readily available for statutory reservoirs in England so prove a convenient source of data on the consequences of dam failure for undertakers. Furthermore, although not publicly available, reservoir undertakers have access to the consequence metrics for the worst-case breach scenario at each reservoir, which includes the ASLL. This gives undertakers a value for the consequences of failure, which can be input directly into an F-N chart, without the need for further time consuming, costly analyses.

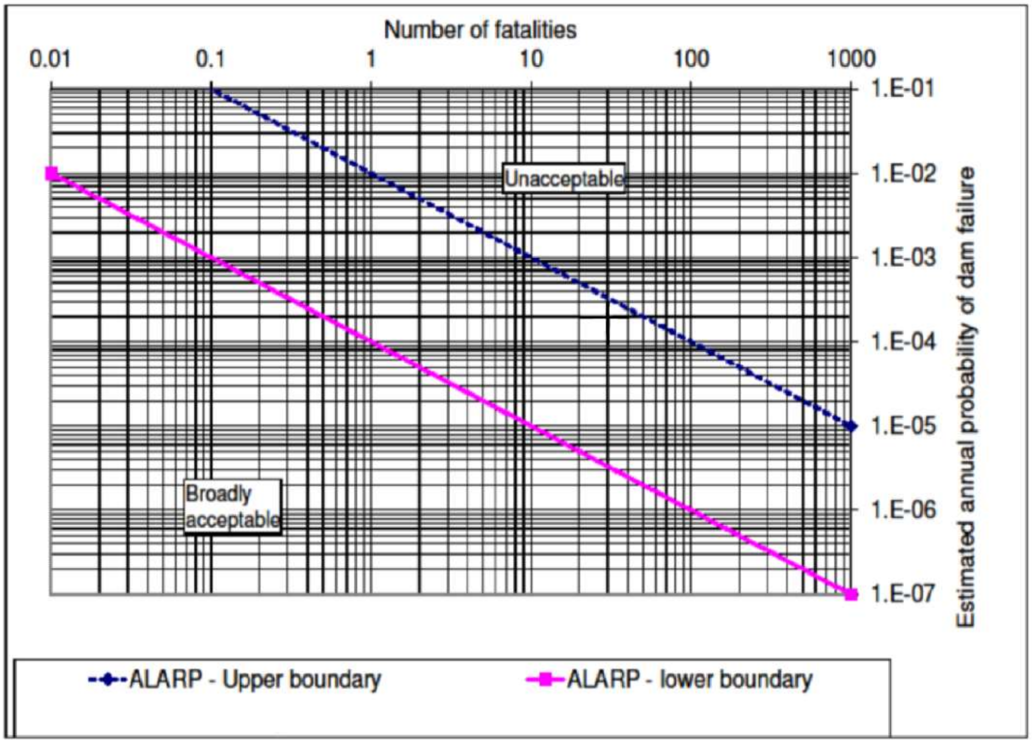

Figure 1. F-N Chart with ALARP limits (taken from RARS (EA, 2013))

However, and as shown later in this paper, the RFM methodology adopts some conservative assumptions which could result in higher ASLL figures and consequently also a higher risk profile for the reservoir. This could ultimately lead the undertaker to carry out works to reduce the probability of dam failure. Therefore, the EA's ASLL values should be used with caution if being applied to the assessment of the societal risk posed by a reservoir.

This paper discusses the conservatisms in the EA RFM Specification methodology for creating reservoir flood maps and calculating the fatality rate, both of which inform the ASLL. The review highlights the assumptions that can impact the accuracy of the breach modelling and ASLL values. This could be useful to reservoir owners if, following an initial assessment of societal risk using the EA's ASLL value, the reservoir falls within the "Unacceptable" zone, but relatively close to the ALARP zone. Review of the site-specific data against the EA adopted values could provide an indication of whether a more detailed analysis may lead to a lower ASLL value.

REVIEW OF THE EA RESERVOIR FLOOD MAPS

The RFM Specification is a national specification that applies to all reservoirs in England and therefore the assumptions and values used are not bespoke to individual reservoirs.

The methodology proposes deriving the dam breach hydrograph outside of the hydraulic model using the empirical equation for peak flow proposed by Xu and Zhang (2009), and the equation for time to failure proposed by Froehlich (2008), both shown in Table 1.

Table 1. Xu and Zhang (2009) and Froehlich (2008) equations to calculate peak outflow and failure time respectively.

Equation	**Coefficients**
Peak Outflow Rate (Qxz) $$\frac{Q_{xz}}{\sqrt{gV_w^{\frac{5}{3}}}} = 0.133\left(\frac{V_w^{\frac{1}{3}}}{H_w}\right)^{-1.276} e^{C4}$$	$C_4=b_4+b_5$ $b_4=-0.788$ and -1.232 for overtopping and seepage erosion/piping $b_5=-0.089$, -0.498, and -1.433 for high, medium and low dam erodibility
Failure time or time of breach formation (T_f) $$T_f = 63.36\sqrt{\frac{V_w}{gH_d^2}}$$	

The parameters that shape the hydrograph; peak flow (Q_p), time to peak flow (T_p) and time to end of hydrograph (T_e), are subsequently derived from the ratio between peak flow (Q_{xz}), failure time (T_f) and the escapable reservoir volume (V_w), as presented in Table 2.

Table 2. Hydrograph parameter (Source: RFM Specification 2019)

$Q_{xz}T_f/V_w$	Q_p	T_p	T_e
< 2	$Q_p=Q_{xz}$	$T_P = T_f/2$	$T_e= 2V_w/Q_{xz}$
> 2 and <5	$Q_p=Q_{xz}$	$T_p= V_w/Q_{xz}$	$T_e= 2V_w/Q_{xz}$
> 5	$Q_p= 5V_w/T_f$	$T_P = T_f/5$	$T_e = 2T_f/5$

The peak flow equation presented in RFM Specification assumes high erodibility dams, whereas the original Xu and Zhang equation (Table 1), allows for three levels of erodibility: high, medium and low, through the inclusion of a coefficient (b_5).

As well as the erodibility of the dam (b_5), the Xu and Zhang equation also takes into account the type of the dam failure through a coefficient (b_4), the escapable reservoir volume at the time of failure (V_w) and the height of water above the breach bottom (H_w), while the Froehlich equation also considers the height of the dam (H_d).

The various parameters of the breach hydrograph are discussed in the following sub-sections.

Embankment erodibility

The new reservoir flood maps were developed under the assumption of a reservoir with a high erodibility dam, due to lack of readily available data to determine the actual erodibility of the dam as outlined in the RFM Specification. Should this assumption be changed to medium erodibility, then the peak outfall rate would decrease by 30%, and the failure time would increase by 40%, when compared to the results considering high erodibility. This change would significantly affect the calculation of the breach hydrographs, as illustrated in Figure 2.

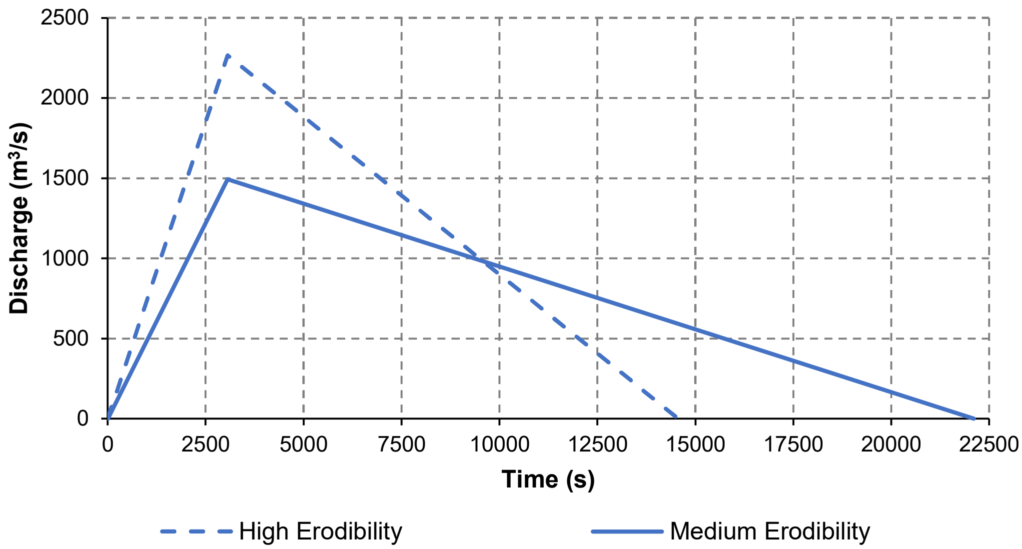

Figure 2. Example of breach hydrograph comparison between high and medium dam erodibility keeping all other parameters consistent.

The erodibility of an embankment can be reviewed against the erosion categories proposed in Briaud (2008) and reproduced in Figure 3.

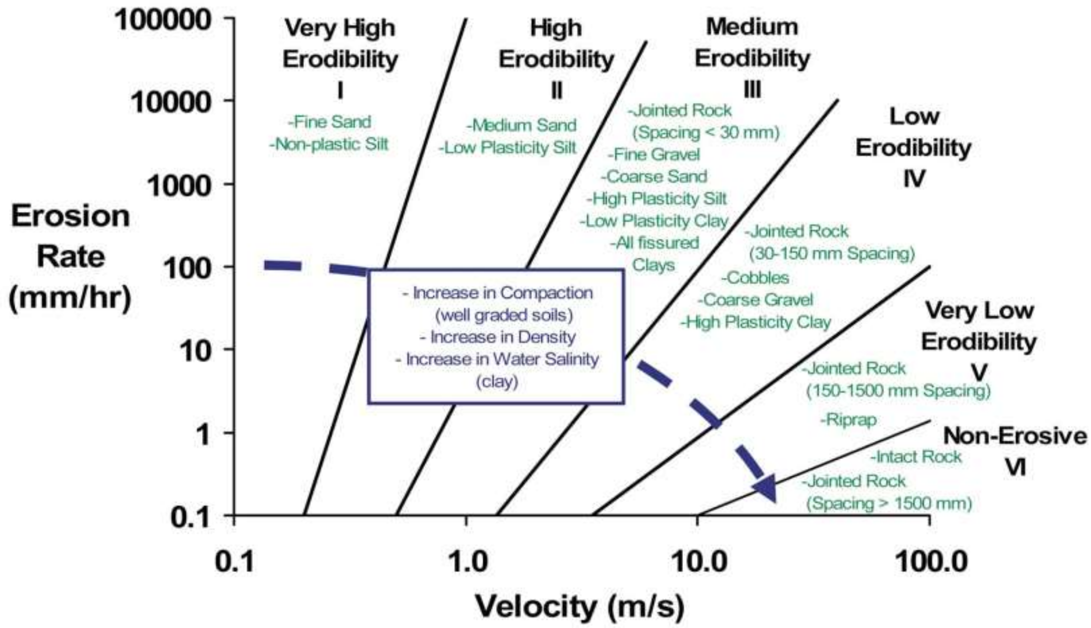

Figure 3. Erosion categories for soils and rocks (Source: Briaud, 2008).

The particle size distribution and Atterberg limits of the embankment materials of a couple of reservoirs located in the south of England, where historical ground information was available, were reviewed. The review showed that these embankments were mostly formed by medium erodibility materials, with the presence of some low erodibility materials (high plasticity clays in the core). For these embankments, the high erodibility assumption was found to be conservative.

Small embankment dams and a minority of larger embankment dams have a homogenous impermeable embankment typically formed by clay, which would fit in the medium erodibility category. In larger embankment dams, the core is supported by earthfill shoulders. The nature of the shoulders' materials and therefore their erodibility can vary significantly.

Geometrical parameters

The height of the breach (H_b), which is related to the height of the dam at the location of the breach (H_d), varies along the length of the embankment. In addition, depending on the relationship between the lowest level within the reservoir and the downstream ground level at the breach location, the escapable volume (Vw) will also vary.

The worst breach location of impounding reservoirs is often at the highest section of the embankment, however, for non-impounding reservoirs formed by perimeter embankments, the worst breach location will not be so obvious.

The 2021 EA reservoir flood maps of non-impounding reservoirs considered different possible locations for the breach, conservatively assuming the same worst parameters (dam height and escapable volume) for each location being studied. This approach can be refined by reviewing the available topographic data at each breach location to determine the height of the embankment and to reassess the escapable volume considering the downstream ground levels. In many cases, these reservoirs were built with materials from the reservoir area and the bottom of the reservoir is below the surrounding ground levels, which means that the escapable volume might be smaller than the total volume stored in the reservoir.

BREACH SCENARIOS

The new reservoir flood maps were produced for the "dry day" scenario and the "wet day" scenario. The "wet day" scenario corresponds to an overflow failure, whilst the "dry day" scenario accounts for other possible failure modes, such as internal erosion. The type of failure mode is accounted for in the peak outflow equation by the b_4 coefficient (Table 1).

When using the reservoir flood maps to determine the risk profile of the reservoir, it is important to use the ASLL value from the scenario associated with the failure mode that dictates the probability of failure of the reservoir. For instance, the probability of failure of a non-impounding reservoir is generally dictated by internal erosion and therefore the ASLL figures for the dry day scenario should be used.

FATALITY RATE

The outputs of the breach inundation mapping (flow depth and velocity) are used to calculate the individual fatality rate for each receptor (property) using the "no warning" relationship originally developed in the *Interim guide to Quantitative Risk Assessment for UK Reservoirs* (Brown and Gosden, 2004) based on DSO-99-06 (USBR, 1999). The sum product of the fatality rate and maximum occupancy at each receptor then provides a value of the ASLL.

Reclamation's Consequence Estimating Methodology (RCEM) (USBR, 2015) replaced DSO-99-06 in 2015. The RCEM 2015 fatality rates are based on case history data which was expanded from DSO-99-06. This expansion of case history data helped to strengthen the empirical relationships from which the fatality rate estimates are derived.

RCEM 2015 upper and lower limits of the suggested range for the fatality rate for little or no warning are plotted in Figure 4 together with case history data and the relationship proposed in the RFM Specification. RCEM 2015 quantifies the flood intensity in terms of maximum DV (depth multiplied by velocity), while DSO-99-06 used discharge/flooded width.

For DV values greater than 1, the RFM proposed relationship between DV and fatality rate fits reasonably well with the upper limit curve of the RCEM suggested range for fatality rate values. However, there is almost no data for DV values smaller than 1 to support the alignment of the lower leg of the RFM relationship.

The Interim guide (Brown & Gosden, 2004) mentions that the population at risk may be taken as the population in the areas where both DV is greater than 0.5m²/s and the depth above external ground is greater than 0.5m.

The alignment of the lower leg is particularly important for non-impounding reservoirs where the reservoir breach floods highly populated areas with flat topography. We have tested the sensitivity of the results in one of these reservoirs where more than 90% of the properties were in areas with DV smaller than 1. It was found that by increasing the no risk threshold from 0.1 m²/s to 0.5m²/s, the ASLL reduced by 40%.

The assessment of the fatality rate in areas with DV smaller than 1m²/s (or with fatality rates smaller than 1%) is one aspect of the RFM methodology that would benefit from further research. FD2701 (Defra, 2020) suggests that data from fluvial events with fatalities could be used. Limited Llynmouth data (only two points) were included in the original chart prepared for the Interim Guide.

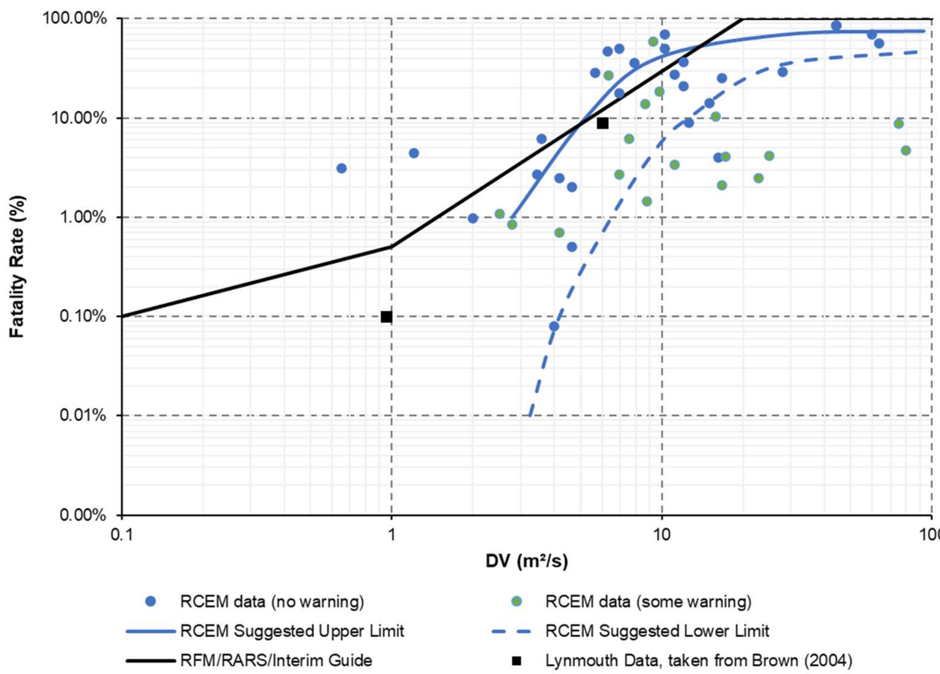

Figure 4. Fatality Rate vs DV

CONCLUSIONS

The new EA reservoir flood maps represent a significant improvement over the previous reservoir inundation maps. However, due to the lack of detailed information readily available for all reservoirs in England, certain potentially conservative assumptions regarding embankment erodibility and geometric characteristics had to be adopted for this mapping exercise to be feasible on a national scale. ASLL values calculated as part of this exercise were naturally influenced by these conservatisms.

The EA ASLL values are available to reservoir undertakers and can prove a convenient source of consequence data for reservoir risk assessments, particularly for those undertakers with limited resources to carry out detailed analyses. However, use of these values can result in the reservoir having a higher risk profile, which could lead to undertakers carrying out costly works to reduce the probability of dam failure.

This paper has outlined some of the key conservatisms with the EA RFM Specification methodology, highlighting the need for the available ASLL values to be used with caution if being applied to the assessment of the societal risk posed by a reservoir. Furthermore, with an understanding of these conservatisms, if the reservoir risk profile sits relatively close to ALARP boundary lines, the undertaker can review the site-specific data (embankment erodibility and geometrical parameters) against the data used to develop the EA reservoir flood maps, to obtain an initial indication as to whether a more detailed analysis could move the reservoir to a lower risk zone.

Outputs from the breach inundation mapping are used to calculate the fatality rate at each receptor, which is in turn used to calculate the ASLL across the inundated area. Calculation of the fatality rate considers a relationship with discharge/flood width, based on empirical equations presented in DS-99-06 (USBR, 1999). A review of more recent publications and guidance has indicated that the RFM methodology would benefit from further research into this relationship for low discharge/flood width values ($<1m^2/s$) as there is no empirical data in the USBR guides to support the alignment of the lower leg of the proposed RFM fatality rate graph. The refinement of this relationship would be particularly important for non-impounding reservoirs where the breach could flood highly populated areas with flat topography, resulting in a high ASLL value due to a large number of receptors with very low (<1%) fatality rates.

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Risk Informed Decision Making for Dinas Dam

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SYNOPSIS Dinas dam is located on the Afon Rheidol and forms part of the Rheidol Hydro Scheme owned and operated by Statkraft Energy Ltd (hereby known as the undertaker). It is a 27m-high concrete arch gravity dam that went into operation in 1962. The dam has several well documented historical issues, mainly associated with development of alkali-aggregate reaction (AAR) within sections of the dam, that was first identified during the 1980s and continued to develop for the next 30 years. The identification, monitoring and evaluation of the AAR was largely overseen by an All Reservoirs Panel Engineer (ARPE).

The aim of this paper is to provide a brief background on how the historic issues at Dinas dam have been managed to date from a risk perspective and to describe the methods and techniques used during a Quantitative Risk Assessment (QRA) workshop. The paper will also provide details of how the learnings from the workshop will assist with making risk-based decisions regarding Dinas dam, that will enable effective planning for future management / works to ensure the longevity of the dam. Lastly, the paper will discuss how this type of workshop can be used as a tool for information sharing and knowledge transfer.

INTRODUCTION

Risk informed decision making is fundamental to the undertaker's core principle of being 'Safe and Prepared.' To ensure optimal operation and maintenance of its assets and to capitalise on opportunities, the undertaker must ensure that risks are identified, analysed, and evaluated sufficiently early to establish mitigating actions. This is achieved by conducting risk assessments for all disciplines in all operating units and larger projects. The criticality of assets is evaluated and measures to address uncertainties are established. The use of cost-benefit analysis tools is well established in the organisation and together with the risk assessment, the organisation action plans, maintenance plans, and reinvestment plans are developed. When actions are completed, the residual uncertainty is evaluated, and assessments are updated.

The undertaker's processes for assessing dam safety risks have proved more challenging. This is largely because the statistical probability of a dam failing is so low and yet the consequences could be so significant that it is has proved difficult to quantify these risks and to establish measures to address the uncertainty. The undertaker has looked to address this issue in recent years by updating its governance documents and ensuring that all dams have an updated qualitative risk assessment of the major risks in the catchments where the dams are located. This includes an assessment of design and spillway capacity of each dam and the ability of the dam to endure overtopping. This approach has allowed the undertaker to assess their full global dam portfolio of 363 dams in 11 countries, in a consistent manner and has highlighted which dams have the highest risks associated with them, and which dams may need additional measures or further investigation to properly manage these risks. Based on this screening exercise, dams have been identified that require a quantitative risk assessment to better manage their risks. Dinas dam, which is part of the Rheidol Hydropower Scheme located in the Ceredigion area in Mid-Wales, was one of the dams pre-screened during the qualitative risk assessment that would benefit from a full Quantitative Risk Assessment (QRA) study.

ALKALI AGGREGATE REACTION

Alkali-aggregate reaction (AAR) is the broader term describing the chemical reaction between certain specific mineralogical types of aggregates and alkali of cement in the presence of moisture. AAR of the type evidenced at Dinas is often also referred to as alkali-silica reaction or slow-late reaction, during which the alkalis in the cement react with certain reactive silica-containing aggregates (Charlwood, 2009: 4). Alkalis may also come from other constituents in the concrete mix, such as water or adjuvant (admixtures or additives), and they may also penetrate the concrete from the environment. The alkalis are dissolved in the mixing water during the mixing of concrete and in the pore water. The resultant alkaline solution reacts chemically with the reactive silica-containing aggregates (Saouma & Perotti, 2006: 194). The silica minerals (especially the poorly crystallised ones) are transformed into an alkali-silica gel which is hygroscopic in nature. This causes a swelling action on the microscopic level which causes the aggregate to develop cracks, leading to the expansion and cracking of the surrounding cement paste (Charlwood *et al.*, 2013. It usually takes some time for the reaction to progress to a stage where it is evident on a macroscopic level and therefore evidence of the reaction is typically noticed about 10 years after initial construction (Mason, 2011).

HISTORY OF DINAS

Dinas dam is located on the Afon Rheidol and forms part of the Rheidol Hydro Scheme which is a 56MW scheme consisting of three dams (Nant-y-Moch, Dinas and Cwm Rheidol) and is the largest hydropower scheme in England and Wales. Dinas dam is a 27m-high central arch gravity concrete dam, flanked on either side by gravity dam blocks, impounding a 850,000m^3 reservoir. The dam was constructed between 1957 to 1962 with commissioning and operation starting in 1962. During the first decade there were no significant issues identified and in the 1972 Inspection Report the condition of the dam was recorded as being in a broadly good condition. In the early 1980s the condition of the dam started to deteriorate with cracks and calcite staining forming on the concrete surface; this drew the first suspicions that alkali-aggregate reaction (AAR) was developing. By the mid-1980s a major horizontal crack had developed on the downstream face of the central arched section of the dam at elevation +245.56mOD. Between 1987 and 1988 cores were taken from the dam and tested and this confirmed AAR.

In the 1992 Inspection report it was noted that water was no longer flowing uniformly over the full length of the overflow weir but concentrated over a small section at the centre of the spillway. This indicated that the geometric properties of the dam had altered and from 1993 a network of survey points was established to monitor vertical, radial and lateral movements of the overflow and dam crest. The hypothesis was that concrete expansion had caused the principal linear dimensions of the dam to increase; however, since the dam was restrained both by the foundation and abutments, the expansion had resulted in the dam effectively bowing in the upstream direction with an upstream tilt and rising crest levels. Reinforcement in the upper part of the dam had helped restrain some of the stresses in the downstream face, which resulted in reduced cracking. At the elevation +245.56mOD, the reinforcement stops, and this is where the large crack first appeared in the mid-1980s. Finite element analyses for Dinas dam were carried out by Gibb Ltd. in 1997. The results for the AAR case showed considerably increased compressive stresses throughout the dam and principal compressive stress directions along the central downstream base of the dam having changed to dominantly horizontal, arch action. The results appeared to support the visual observations and survey results.

Based on this developing situation in 1998 and on the recommendation of the Inspecting Engineer, a system of cables was installed around the upstream face of Dinas dam crest to reduce the risk of the upper dam toppling upstream during a seismic event.

For the next ten years the dam was under close supervision from a Supervising and Inspecting Engineer with quarterly surveys taken and shape surveys developed every three years. Although the dam was frequently inspected, no further testing was undertaken. In 2009 ownership of the Rheidol Hydro Scheme, including Dinas dam, changed and this prompted the next phase in how the dam was to be managed going forward. In 2011 the Inspecting Engineer, who had overseen the installation of the restraining cables, was reappointed. With this reappointment, recommendations were given to investigate the extent of AAR development in the preceding years. A summary of the further investigations since 2011 has been provided below:

- Further concrete testing was carried out in 2012 to establish the state of the concrete and the degree of deterioration since the last set of tests in the 1980s.
- A new survey system was established in 2013. New base stations were established and correlated with the previous survey, and new prism-based survey stations established along the dam crest. The results of the new surveys appeared to largely support the 2012 concrete testing results in that there is now little residual capacity in the concrete for AAR, and that the potential for future expansion has, for all intents and purposes, stopped.
- In situ stress tests were carried out prior to the 2017 Section 10 inspection, which confirmed that stresses within the dam remain within acceptable levels.

In the 2017 Inspection Report, an in-depth review of the previous years' testing, as summarised above, was given with the conclusion that concrete expansion from AAR had either ceased or was now negligible, and stresses within the dam were within acceptable limits. Options were provided to the undertaker on how best to manage the dam going forward, including the installation of stress measurement sensors. The Inspecting Engineer also discussed the potential to increase the longevity of the dam by waterproofing the upstream face.

DEVELOPMENT OF QRA IN THE UNDERTAKER'S ORGANISATION

In 2021, the undertaker commissioned the Norwegian Geotechnical Institute (NGI) to develop an international version of the handbook 'Risk Assessment and Management for Dams' (Lacasse, 2022). The handbook was to provide the personnel who are responsible for managing dam safety in the undertaker's organisation with a guide that presented various established risk assessment methods, and detailed how they can be utilised to complete an in-depth risk assessment (qualitative or quantitative analyses) to assist in making risk-informed decisions. The undertaker has used the assessment methods introduced in the handbook to deliver several QRAs on the most high-risk dams in their global portfolio. The assessment method that the undertaker has used for the majority of these QRAs is Event Tree Analysis (ETA). This method has proved useful when working with a large group of workshop participants to best utilise the diverse expertise of the people within the team.

ETA is a method used to evaluate the probability of a failure mode based on a triggering event/mechanism i.e. extreme flood, landslide, earthquake etc. The triggering event/ mechanism should have a time element e.g. a flood with a 1000-year return period. The event tree then describes in graphical format the logical sequence of events that could lead to a dam failure. Since each step (node) in the sequence could have more than one outcome, branches are then formed which should be continued through to either a dam failure or no dam failure. Probabilities can then be assigned to each node and the risk of failure for the triggering event/mechanism can be calculated.

The event tree analysis uses a nine-step procedure (Lacasse, 2022).

1. Site visit and inspection of the dam including geology, siting, and site conditions.
2. Overview of observations, earlier events, and other observations.
3. Brainstorming and screening of triggers and failure modes and prioritisation of plausible scenarios. This step is called 'failure mode screening'.
4. Discussion and agreement on scales to describe uncertainties and probability estimates.
5. Construction of event trees and estimates of probabilities at each node, and continuation of each sequence of events until failure (or non-failure).
6. Calculation of probabilities for each scenario (tree branch) leading to a failure.
7. Iteration of some or all of the event trees.
8. Calculation of total failure probability for the dam (or system of dams).
9. Evaluation of failure probabilities obtained and consequences.

DINAS DAM RISK WORKSHOP

The Dinas dam safety risk assessment workshop took place at the Rheidol Hydro Scheme in October 2023. The timing of the workshop took place between the Inspecting Engineer's periodic safety inspection and the issuing of the Section 10(3) report. This scheduling meant the results of the QRA could inform the recommendations in the Inspecting Engineer's report. This collaborative approach helped the undertaker to achieve one of its governing principles of using risk to inform their decision-making process and is a good example of how a reservoir undertaker can meet their own internal goals while satisfying the local statutory requirements.

Workshop Participants

The following participants took part in the workshop.

- Rheidol O&M team (Site Manager, HSSE Manager, Civil and Mechanical Technician, Production and Maintenance Planner and the Team Leader of the Control Centre).
- Those working in dam safety within the undertaker's organisation, including representatives from Albania, Brazil, Germany, Norway and Sweden.
- Preceding Inspecting Engineer from 1996 to 1998 and 2012 to 2021.
- Current All Reservoirs Panel Engineer (since 2021) (also Inspecting Engineer at the time).
- Current Supervising Engineer (since 2023).

Participants were chosen based on their familiarity with Dinas dam and Rheidol Hydro Scheme, local knowledge, expertise in concrete dams, and familiarity with the undertaker's QRA process. The undertaker also uses these workshops for knowledge sharing for those working within dam safety in the undertaker's organisation. The undertaker's Dam Safety Officer (DSO) for the UK facilitated the workshop and the Supervising Engineer acted as meeting secretary.

Pre-Workshop Meetings, Site Visit, and Introductory Presentation

The QRA process started pre-workshop through online meetings between the DSO, the Inspecting Engineers, and the Supervising Engineer. The primary purpose of these meetings was to identify the main failure modes that were to be assessed during the workshop and to highlight the main issues associated with the dam, with the aim to streamline the physical workshop and ensure the participants were supplied with the relevant information.

The actual workshop started with the participants visiting the Dinas dam site and reservoir. This gave those unfamiliar to the site the opportunity to question the O&M team and the Inspecting and Supervising Engineers. Following the site visit, the participants reconvened at the conference room at Rheidol Hydro Scheme main office. The room was set up with a conference table, a large whiteboard, and a projector. The meeting started with a short presentation from the facilitator about the agenda for the workshop, an introduction to the process of failure mode identification and screening, and how event tree analysis (ETA) would be used to apply annual probabilities for the failure modes identified. The use of F-N charts to plot the results from the QRA was also introduced as a means of evaluating if the risk associated with the failure mode would be acceptable to society.

Background Presentation by the Inspecting Engineer

Following this, a presentation was given by the preceding Inspection Engineer who had been involved on and off with Dinas dam for nearly three decades. This presentation provided a concise overview of the development of AAR within the dam and the issues that the expansion of the concrete has caused related to the safety and security of the dam. Explanation was given on how these issues have been managed and the risks mitigated through testing, monitoring and close supervision. Commentary was also provided on the status of the AAR development and the continuing impact on the dam.

Brainstorming Session for Trigger Events and Potential Failure Modes

The next phase of the workshop was the potential failure mode screening; this involved identifying potential ways the dam could fail and what would trigger these events. The failure modes were not to be considered in great detail at this stage, as the purpose of this activity was to determine if the failure mode was plausible and therefore required further assessment. The team represented a diverse group of disciplines with varying experience, and this added value to this type of exercise, as the aim was to be creative with potential failure modes and promote a 'blue-sky' thought process. Visiting the dam site assisted with this activity, as seeing the dam in-situ helped the team gain a greater understanding of the challenges faced in operating and maintaining the site.

In the pre-workshop meeting between the DSO and the Inspecting and Supervising Engineers, a similar failure mode exercise had taken place. These results were now shared with all participants and compared with the workshop failure mode screening exercise. The combined results were assessed in terms of credibility and perceived ranking (Table 1), and a plan was then formed on how the workshop would proceed with the ETA.

Table 1. Trigger Events and Viable Failure Modes

Ranking	Trigger Event	Failure Mode(s)
1	Earthquake	Toppling of upper (reinforced) part of spillway (dam) into the reservoir. Toppling of upper (reinforced) part of spillway (dam) downstream (into the spilling basin). General dam stability (e.g. abutment failures).
2	Flooding	Toppling of upper (reinforced) part of spillway (dam) downstream (into the spilling basin). Erosion of central rock stilling basin. Erosion of downstream abutments and foundation (overtopping of training walls). Breaking up of concrete steps.
3	Concrete Deterioration	Expansive concrete causing lifting of arch, with load shedding into abutments. Increased cracking / seepages leading to decreased concrete strength. Reduced concrete mass due to spalling concrete in RC section.
4	Cascade Failure (failure at upstream Nant-y-Moch dam)	Nant-y-Moch failure. Note: although this is considered a viable failure mode, a separate workshop for Nant-y-Moch will take place in the future and the cascade failure will be considered then.
5	Impact from plane	The Dinas reservoir is part of the training route for the Royal Air Force, with flybys at low altitude a common practice. In the 1980s a plane crashed into the Nant-y-Moch reservoir.
6	Destructive Investigation	Deliberate or accidental sabotage. Note: Over-tensioning of the spillway cables could potentially cause the top section of the spillway to topple. Further discussion deemed this improbable, and the group decided to discount this failure mode.

Assessment of Downstream Consequences

The 2009 Reservoir Inundation Map (RIM) for Dinas Reservoir was used when assessing the downstream consequence of a Dinas dam breach. The local knowledge of the operations team also assisted in defining potential at risk properties. Due to the limited quality of information available, only a high-level assessment was possible, with the area most at risk identified as a small village downstream of the dam site. Further downstream, flatter floodplains and the potential ability to provide sufficient warning reduced the number estimated to be at risk. It was concluded that a range of 10 to 100 people would be used in the F-N charts but accepted that further work was required to refine this estimate.

Event Tree Analyses

The following event trees were developed with the trigger event shown in brackets:

- *Spillway crest toppling upstream (earthquake)*
- *Spillway crest toppling downstream (flood)*
- Overtopping of spillway training walls (flood)
- Erosion of spillway stilling basin (flood)
- Break-up of left flank spillway steps (flood)
- *Deterioration of concrete from increased seepage through the dam, also considering the effect of Alkali Silica Reaction (seepage)*
- Direct hit on dam by low flying plane (impact)

This paper will only consider the event trees which are directly impacted by the historic issues associated with AAR which are shown in *italics* above.

Event Trees Sensitivity Analysis

Verbal descriptors were used to assign probabilities (p) to nodes on the event trees. The verbal descriptors used with their associated probabilities were as follows (probability range shown in brackets):

- Virtually impossible, p = 0.001 (0 – 0.005)
- Very unlikely, p = 0.01 (0.005 – 0.02)
- Unlikely, p = 0.10 (0.02 – 0.33)
- As likely as not, p = 0.50 (0.33 – 0.66)
- Likely, p = 0.9 (0.66 – 0.98)
- Very likely, p = 0.99 (0.98 – 0.995)
- Virtually certain, p = 0.999 (0.995 – 1.0)

When compiling the event trees during the workshop, a single value within the given ranges was allocated for each of the relevant nodes. Post-workshop, a Monte Carlo analysis was completed for each event tree, which would randomly select a number within the range of the verbal descriptor at each node, the process was completed 10,000 times. These Monte Carlo analyses were then used to calculate the maximum, minimum and average probabilities associated with each failure mode. This range was plotted on the risk diagram. Occasionally the workshop participants could not agree on a single verbal descriptor – for these situations it was agreed that the Monte Carlo analysis would be run using a range decided during the workshop.

ETA - Spillway Crest Toppling Upstream (Earthquake)

The annual probability of failure of the upper part of the dam during an earthquake event (i.e. upper section of the dam toppling upstream), was calculated to be 2.35 x 10^{-5} /yr (see Figure 1 for ETA). The sensitivity analysis based on the ranges of the verbal descriptors was calculated to be between 1.3 x 10^{-5} /yr to 2.1 x 10^{-7} /yr with a mean of 2.8 x 10^{-6} /yr (plotted on F-N risk diagram - see Figure 2). Fundamental to controlling this risk is the capacity of the restraint cables that have been installed on the upstream face. The cables were installed as mitigation for this failure mode in 1998 with a design life of 25 years. Although visual inspection appears to suggest the cables are in relatively good condition, the workshop concluded that their structural capacity should be confirmed through further investigation.

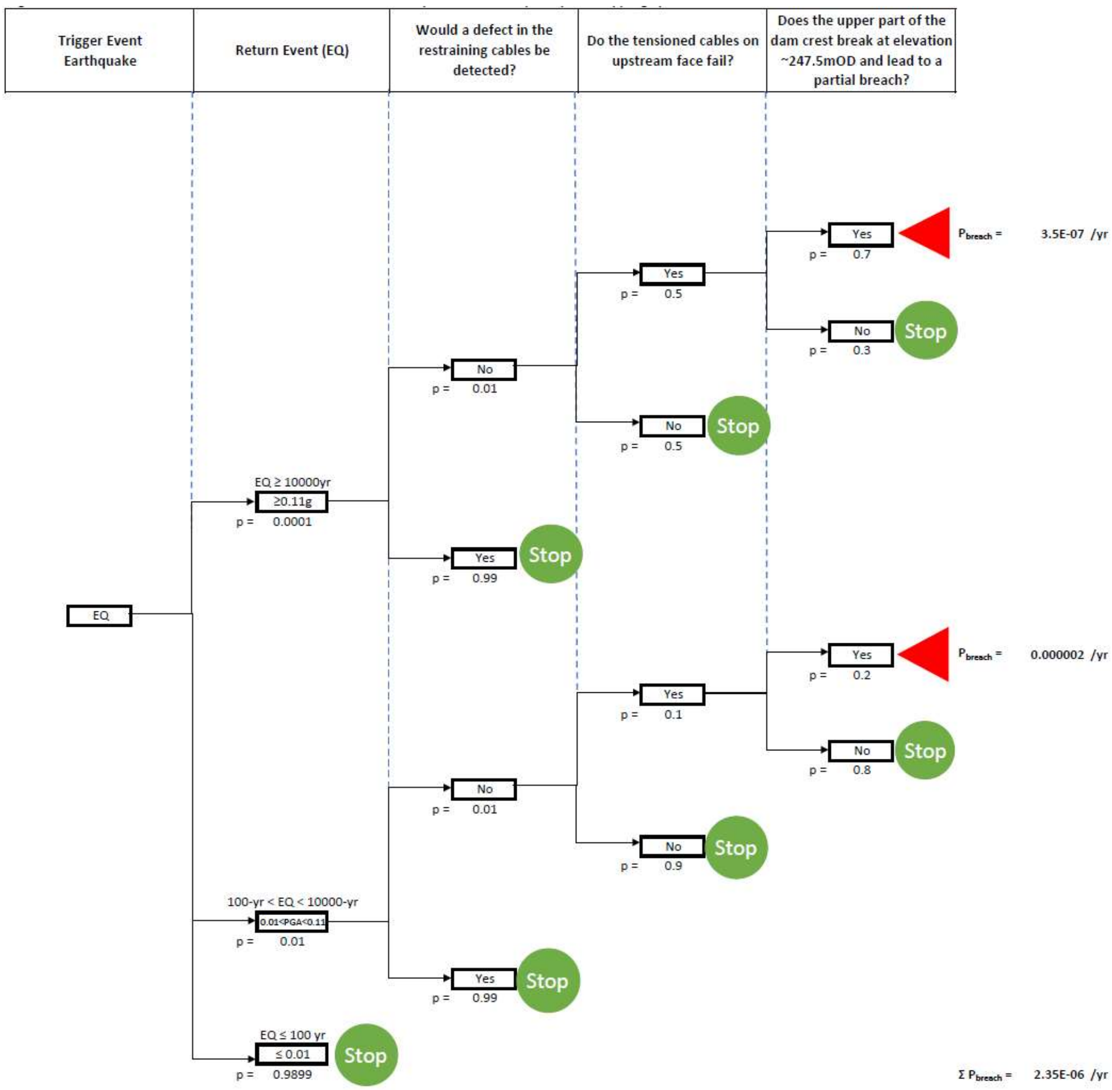

Figure 1. ETA - Spillway Crest Toppling Upstream (Earthquake)

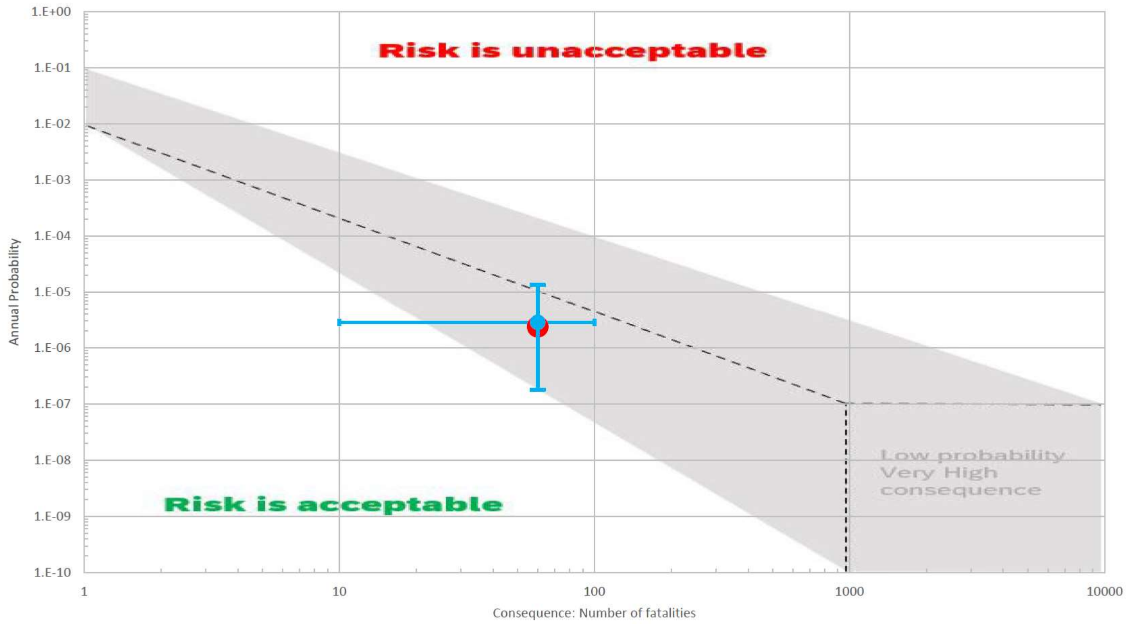

Figure 2. F-N Risk Diagram - Spillway Crest Toppling Upstream (Earthquake)

ETA - Spillway Crest Toppling Downstream (Flood)

The annual probability of failure of the upper part of the dam during a flood event (i.e. upper section of the dam toppling downstream), was calculated to be 2.65 x 10^{-5} /yr (Figure 3). The sensitivity analysis based on the ranges of the verbal descriptors was calculated to be between 1.1 x 10^{-4} /yr to 2.5 x 10^{-5} /yr with a mean of 6.6 x 10^{-5} /yr (plotted on F-N risk diagram - Figure 4). This relatively high probability of annual failure did not really correspond to the general feeling within the room, which felt this mode of failure was unlikely. The event tree developed was potentially over-simplified as the single question 'Does the construction joint fail in shear?' largely governed the end result. Although consideration was given to the impact that detection and monitoring may have on this failure mode, it was not included in the event tree.

This failure mode requires further evaluation. Finite Element modelling of the dam was discussed as an option to better understand the dam's current and possible future behavioural patterns. Due to issues with AAR in the dam, there is further information to be collected from in-situ testing, alongside with ongoing survey monitoring data, both of which should then be incorporated into the model.

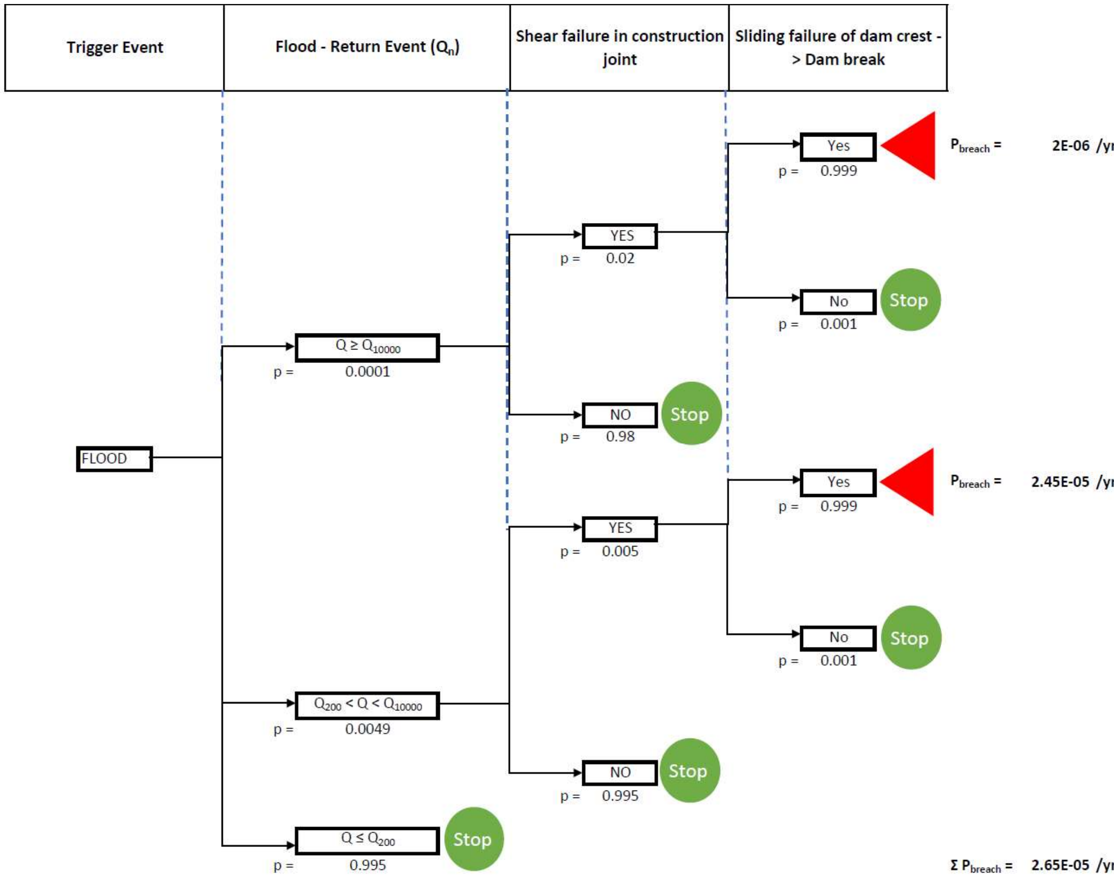

Figure 3. ETA - Spillway Crest Toppling Downstream (Flood)

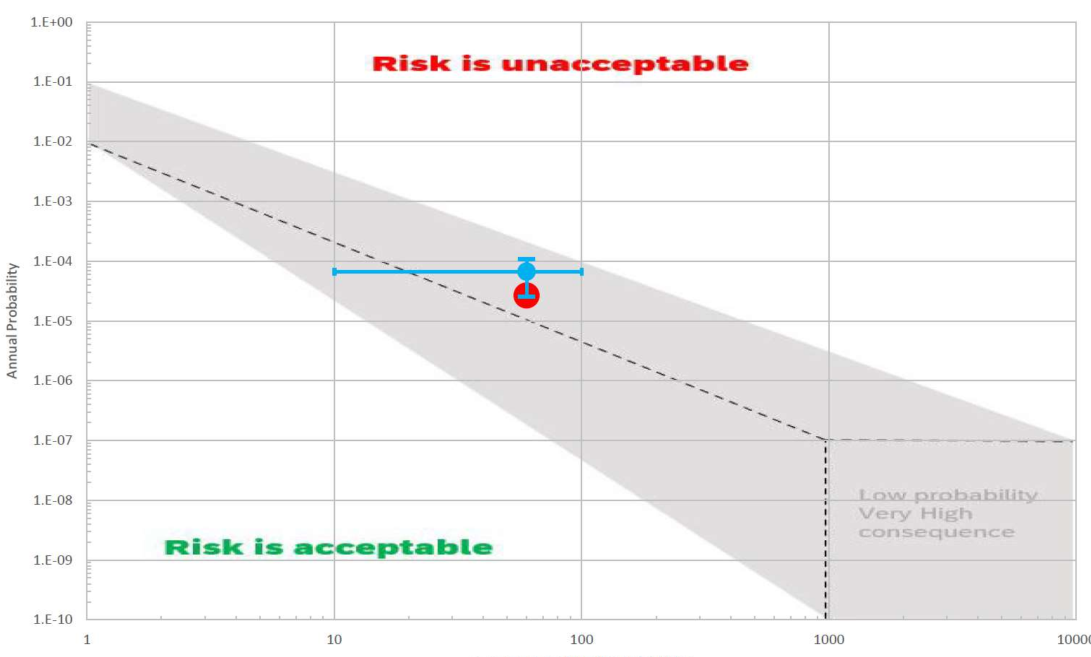

Figure 4. F-N Risk Diagram - Spillway Crest Toppling Downstream (Flood)

ETA - Deterioration of Concrete from Increased Seepage through the Dam, also considering the effect of Alkali Silica Reaction (Seepage)

Based on the current condition of the dam and the reduced impact of the AAR, it was determined that seepage through the concrete dam was not a current day risk; however, if left unabated continued leaching of the concrete paste could cause degradation to an extent that internal stresses will exceed the capacity of the concrete. To assess this risk the condition of the concrete assuming continued leaching was considered over a mid to long-term timeframe (20, 50 and 100 years). The risk assessment demonstrated that in between 20 and 50 years the probability of failure of the dam becomes unacceptable and control measures would likely need to be implemented (see Figure 5 for the ETA developed for predicted condition of the dam in 20 years).

If measures are applied, i.e. sealing the upstream face, this failure mode is no longer considered viable. The same exercise was repeated assuming re-activation of the AAR, but assuming shorter time-frames to account for the further degradation of the concrete caused by the resulting expansion (15, 40 and 80 years). It was assessed that in between 15 and 40 years it was likely that the probability of failure would be unacceptable. The risk diagram for this failure mode is shown in Figure 6 (note: Monte Carlo sensitivity analysis was not included for this assessment, since the main purpose was to demonstrate that the probability of this failure occurring increases with time).

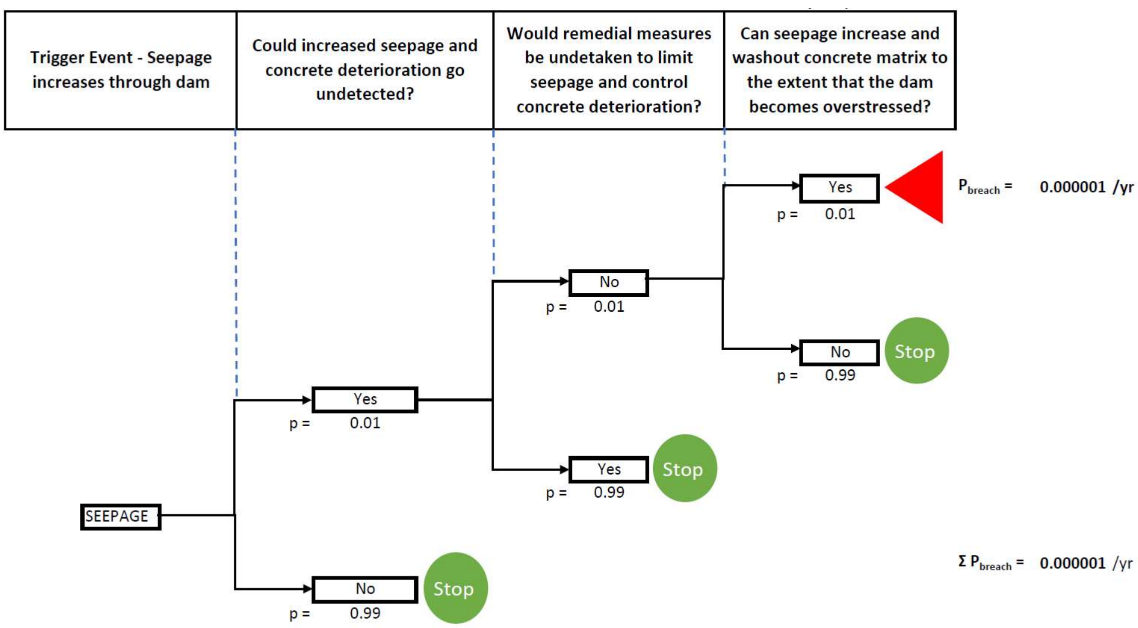

Figure 5. ETA - Deterioration of Concrete from Increased Seepage through the Dam After 20 Years

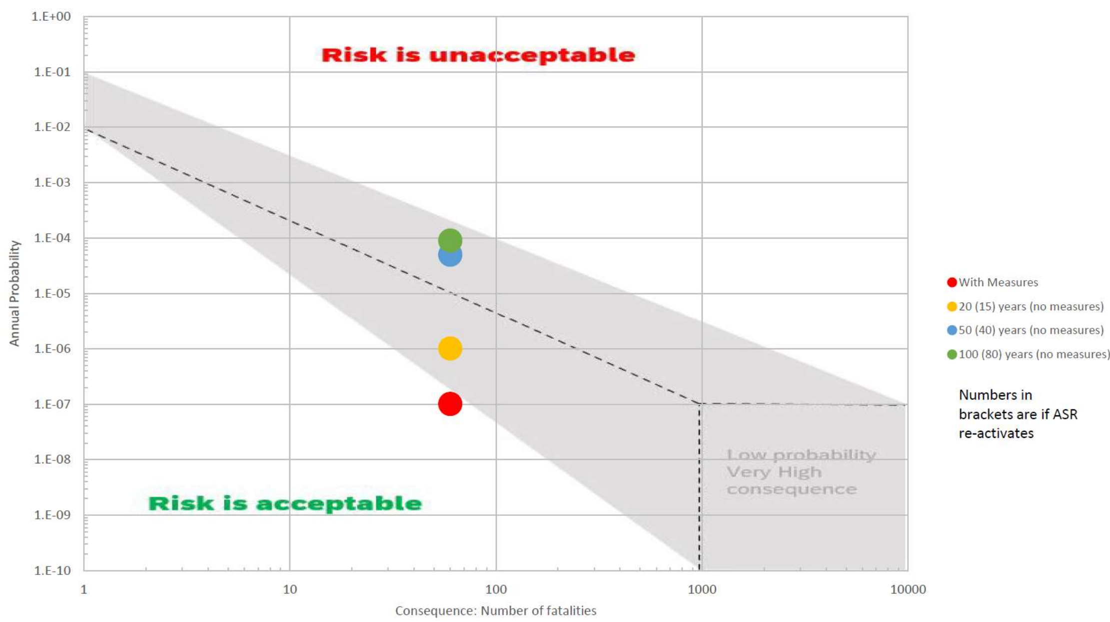

Figure 6. F-N Risk Diagram - Deterioration of Concrete from Increased Seepage through the Dam

QRA Results and Recommendations

The ETAs produced during the QRA workshop are based on a simple method of assigning annual probabilities of failure for the different failure modes identified. It is recognised that these probabilities are subjective and rely on having quality data to inform the decisions. For the different failure modes, it is not a simple case of evaluating whether the risk is acceptable or not, as many risks will fall in a 'grey' area which indicates that not enough information is known to comprehensively evaluate the risk. Inadequate assessment of consequence (as was the case for this study) also promotes indecision in the process as judgements will tend to err on the side of caution, reflected in less certainty applied to verbal descriptors. However, this method of QRA is useful at highlighting where there is potential information lacking. Therefore, the main recommendations that came from the Dinas QRA are associated with the need to obtain better information to inform future decisions.

The main recommendations are as follows:

- Detailed dam break analysis should be completed to better assess the number of receptors at risk in the downstream catchment. Since Dinas reservoir and dam are part of a cascade scheme it would be prudent to assess the Rheidol Hydro Scheme reservoirs as a whole, with assessments for both the individual failure of Dinas, Nant-Y-Moch and Cwm Rheidol, as well as a cascade failure associated with Nant-Y-Moch.
- A new finite element model should be developed which better represents the current condition of the dam.
- The current condition/capacity of the restraining cables on the upstream face of the dam should be investigated.

Alongside the above recommendations, further testing and monitoring of the dam was recommended as follows:

- Carbonation depth tests to determine what remaining pH protection cover is available to the embedded rebar.
- Laser scanning and point cloud surveys to better track deformation changes in the dam.
- Crack mapping with AI to define and track changes to the surface of the dam.
- Ultrasonic Pulse Velocities (UPV) survey to indicate localised concrete strength and soundness. This can be routinely repeated to indicate changes in the concrete.
- PPM tests of seepage flows through cracks in dam to indicate level of concrete leaching.
- New cores to be taken in future years for strength comparison with earlier results.
- In situ stress tests and the installation of stress cells to measure the stress changes in the concrete over time.

Ultimately through the information collected from the studies/monitoring activities detailed above, a strategic plan can be developed for prolonging the longevity of Dinas dam. The majority of the above recommendations have been captured in the recent Section 10 report that was finalised and issued following the workshop.

INFORMATION SHARING AND KNOWLEDGE TRANSFER

A major benefit of running a QRA workshop is that it provides an excellent opportunity to share information about the dam, the QRA process and promote knowledge transfer. Those appointed to the roles of Inspecting and Supervising Engineers have changed in recent years and with reappointment of these vital roles, there is always a danger that important knowledge of the dam gets lost. By inviting the previous Inspecting Engineer to participate in the workshop a wealth of knowledge gained over three decades of involvement with Dinas dam helped drive discussions and added real value to the process.

Although many of the local O&M personnel who took part in the workshop, are not necessarily directly involved in the dam safety programme for Dinas dam, they have knowledge that pre-dates the current undertaker’s ownership. Their insights into the local area and events over the past decades helped to inform the process and at times provided eye-witness accounts. Post-workshop feedback from the O&M team also suggested they had gained a greater understanding of the dam, which will inform their own future O&M activities.

The undertaker’s own dam safety team used this opportunity to improve processes that have been in development for nearly 5 years. The undertaker is delivering a global dam safety program and providing opportunities for the dam safety personnel to interact and discuss dam safety issues, helps promote a unified approach to dam safety. Selecting participants with a range of experience and knowledge helps promote the process of continuing improvement which can then be refined in future workshops.

CONCLUSION

The main goal for the undertaker in running a QRA type of risk assessment workshop is to gain a greater understanding about the dams that they own and operate and their residual risk. By better understanding the viable failure modes and how these are triggered, and can develop, means mitigation measures can be identified early, planned, and executed. This allows a proactive approach to the management of the dam and with decisions based on risk, resources can be used with greater efficiency.

Managing Risks for Dams and Reservoirs

The timing of the QRA workshop was also critical as it preceded the issuing of the newly appointed Inspecting Engineer's first Inspection Report for Dinas Reservoir. The recommendations outlined in the workshop were largely included in the proceeding Inspection Report and therefore the owner, Inspecting Engineer and Supervising Engineer were all in agreement with regard to the future strategy for the dam.

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Improving the quality of flood modelling studies for reservoir safety assessment

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SYNOPSIS Flood modelling studies underpin decision making on reservoir spillway capacities and dam freeboard allowances. Flood modelling is a specialist subject with many methodological decisions and assumptions that can significantly affect outcomes. It is often undertaken by third party consultants on behalf of the reservoir operator.

This paper describes the work undertaken by the Environment Agency over the past two years to improve the quality of flood modelling studies undertaken for reservoirs owned and operated by the Environment Agency. This has included developing standardised modelling scopes, reporting templates, and quality assurance procedures. We have also sought to improve guidance and accessibility of tools for undertaking calculations. Within the Environment Agency, we have introduced training materials and led webinars on flood modelling for reservoir safety studies to improve understanding across modelling and engineering professions. Together, these are improving the quality assurance of our flood modelling studies. However, there have been challenges, including the difficulties of completion of work within MIOS deadlines, and tensions between the role of modelling technical assurer and the role of the panel engineer. We make recommendations for collaborative ways of working to overcome these challenges.

INTRODUCTION

The Environment Agency has multiple roles in reservoir safety management, both as a regulator and as an undertaker. The Environment Agency also has a role as a statutory consultee in land use planning.

Regulatory roles:

- Under the Reservoirs Act (1975) the Environment Agency is the regulator responsible for managing and implementing reservoir safety regulations in England, and for enforcing safety requirements if needed.
- The Environment Agency prepares and publishes reservoir flood maps that show where water may go in the unlikely event of a dam or reservoir failure.

Managing Risks for Dams and Reservoirs

Undertaker roles:

- The Environment Agency is the owner and operator of 218 reservoirs, the majority of which are flood storage reservoirs.
- The Environment Agency designs and constructs new flood storage reservoirs through the Flood and Coastal Erosion Risk Management capital programme of work.

Statutory consultee roles:

- The Environment Agency is a statutory consultee for land use planning applications, including planning applications for constructing new reservoirs or altering existing reservoirs. The Environment Agency reviews the Flood Risk Assessment and any associated flood modelling and may object to planning applications on the grounds of flood risk impacts.

The Regulator, Undertaker and Statutory Consultee roles within the Environment Agency are kept functionally separate. This paper is presented from the perspective of the Environment Agency as the owner and operator of reservoirs (undertaker role), working with panel engineers to manage reservoir safety and undertake statutory inspections.

Reservoir modelling studies play an essential role in good reservoir management. For Section 10 inspections, the Environment Agency commissions and undertakes flood modelling studies to improve understanding of reservoir spillway capacities and dam freeboard allowances. Estimating the water levels and flows that may occur under flood conditions allows the assessment of risk of failure and design of appropriate management solutions. This may lead to remedial works as part of any recommendations for measures in the interest of safety (MIOS). However, if the modelling and supporting data and assumptions are not fit-for-purpose, the risk may not be appropriately managed.

Flood modelling is a specialist subject with many methodological decisions, assumptions and uncertainties that can significantly affect outcomes. The Environment Agency commissions many hundreds of flood modelling studies every year for different purposes, including flood zone mapping, appraisal and design of flood risk management schemes, flood warning improvements, and reservoir flood risks. Quality assurance procedures are already well established for many of these applications. These procedures are also applied to the several hundred flood models submitted as part of Flood Risk Assessments supporting planning applications, for which the Environment Agency is a statutory consultee under land use planning regulations.

This paper describes the work undertaken by the Environment Agency over the past three years to extend our quality assurance procedures to flood modelling studies undertaken for reservoirs owned and operated by the Environment Agency. This has included:

- Developing standardised modelling scopes, reporting templates, and quality assurance procedures.
- Improving guidance and accessibility of tools for undertaking calculations.
- Improving general and specialist reservoir modelling skills, including improving communication and understanding between technical and non-technical teams.

However, there have been challenges, including the difficulties of completion of work within MIOS legally binding deadlines, and tensions between the role of the modelling technical assurer and the role of the panel engineer. We make recommendations for collaborative ways of working to overcome these challenges.

THE AQUA BOOK GUIDANCE ON QUALITY ANALYSIS

The government's approach to quality assurance is set out in the Aqua Book (H M Treasury, 2015). This sets out the following principles for quality assurance:

- Proportionality of response: The extent of assurance should be proportionate to the risks, including financial, legal, operational, and reputational impacts.
- Assurance through development: Quality assurance should be considered throughout the life cycle of analysis and not just at the end. Effective communication is crucial when understanding the problem, designing the approach, conducting the analysis, and reporting the outputs.
- Verification and validation: Quality assurance is more than just checking the analysis is error-free and satisfies the specification (verification). It must also check the analysis is appropriate and fit for the intended purpose (validation).
- Analysis with RIGOUR: Quality analysis needs to be:
    - Repeatable,
    - Independent,
    - Grounded in reality,
    - Objective,
    - Understanding and managing uncertainty,
    - Robustly answering the initial question.

These principles have been used to develop our quality assurance procedures for flood modelling studies. The goal of our quality assurance is to demonstrate that the flood modelling study is fit-for-purpose prior to its use in decision making, through robust and independent peer review.

THE BENEFITS OF STANDARDISATION

Standardisation of processes and tasks leads to well-known benefits across all industries. These include reduced ambiguity about what the task is and how to perform it, and reduced inefficiencies due to avoidable reworking, leading to faster higher quality and consistent outputs.

We have introduced new standardised documents and procedures for:

- Commissioning reservoir modelling studies. A standardised technical modelling scope (LIT 72263: Reservoir Modelling Scope), based on the NEC4 Professional Service Contract, has been developed. The scope includes standard clauses and requirements, as well as optional clauses which can be chosen depending on the study being undertaken. The standard scope saves time as project managers no longer draft scopes from the beginning. It provides consistency between modelling studies and ensures no aspects are overlooked at the scoping stage.

Managing Risks for Dams and Reservoirs

- Reporting hydrology calculations for reservoir modelling studies (LIT 65993: Flood Estimation for Reservoir Safety Calculation Record; and LIT 65996: Accompanying Notes). This provides a record of the hydrological context, the method statement, the calculations, the decisions made, and the results of flood estimation. It includes the estimation of the inflow hydrograph and its routing through the reservoir to calculate the discharge hydrograph and maximum stillwater level at the spillway. The report template aims to ensure that full calculation details, decisions, assumptions and limitations are reported, to provide a complete audit trail for quality assurance. This saves time as reviewers are provided with all details needed for checking, avoiding the need to request additional information. It provides consistency in reporting standards and clarity to consultants on expectations for reporting.
- Reviewing hydrology calculations for reservoir modelling studies (LIT TBC: Flood Estimation for Reservoir Safety Calculation Review Template). The review spreadsheet provides a record of what has been checked by the reviewer and any comments. The comments are categorised into "OK" (no change needed), "Green" (change request with negligible impacts on outcomes), "Amber" (change request with medium impacts on outcomes) and "Red" (change request with significant impacts on outcomes). It would be expected that all "Amber" and "Red" comments are addressed for the study to pass quality assurance. Additional columns provide space for consultant responses, and second and third rounds of review. This review spreadsheet directly follows the structure of the reporting template. This saves time as reviewers do not have to search a document to find the information for each item to check. It removes ambiguity over what will be checked during the review process and provides an audit trail of decisions during the review process.
- Quality assurance standard procedures (LIT TBC: Quality assurance of reservoir modelling studies). These standard procedures are aimed at internal Environment Agency staff and explain the reasons for undertaking quality assurance, how to request support from the appropriate Environment Agency technical team, at what stages quality assurance should be undertaken, communication and liaison, handling sensitive information and resolving conflicts. The standard procedures provide a common shared understanding of quality assurance within the Environment Agency. This ensures appropriate time and resource for quality assurance are included in project planning, reducing ambiguity about what quality assurance is needed at what stage of the project.

THE ROLE OF GUIDANCE AND OPEN TOOLS

Guidance leads to better quality analysis through a shared understanding of the correct decisions to make on data, methods, calculations, and analysis. Open tools shared by all support this by removing ambiguity over algorithm differences. They allow calculations to be shared for review without any software licensing restrictions. The Environment Agency is committed to improving accessibility and openness of tools and methods wherever possible (see Flood Hydrology Improvements Programme report (EA, 2024)).

One area of ambiguity in probable maximum flood (PMF) estimation for reservoir safety modelling has been the estimation of snowmelt. This is frequently underestimated. In February 2022 we issued a briefing note to raise awareness of the impacts of snowmelt on the PMF. This was followed in April 2022 with a full worked example of the PMF calculation procedure and an Excel spreadsheet tool (LIT 58205: Probable Maximum Flood calculation spreadsheet) that includes snowmelt. This spreadsheet is freely and openly available on request from the Environment Agency, and the code can be adapted for other uses (e.g. batch applications).

In December 2022 we updated our Flood Estimation Guidelines and extended Chapter 6.5 on flood estimation for reservoir safety. These guidelines complement the recommendations in the Floods and Reservoir Safety 4th Edition. The guidelines were openly published on gov.uk in November 2023 to increase accessibility (EA, 2023)

In June 2024, we continued our commitment to providing open tools for calculations wherever possible, by releasing an Excel spreadsheet tool (LIT 72757: FSR-FEH and Pumped Rainfall Runoff spreadsheet) for applying the FSR/FEH rainfall-runoff method which may be used to estimate the 1 in 10,000 flood hydrograph for comparison with other methods. This spreadsheet is also freely and openly available on request from the Environment Agency.

SKILLS AND TRAINING

Quality assurance should be carried out by an independent reviewer who is not directly involved in the modelling project or programme. Good quality assurance relies on well trained and experienced staff who are able to efficiently review work and appropriately identify any concerns. Reviewers must have suitable training, qualifications, experience and supervision to carry out quality assurance. We have undertaken a programme of training to develop technical specialists within our pool of modelling staff. This has included webinars, recorded training videos and worked examples for self-led learning, attendance at external training courses, and mentoring on projects by more experienced staff. Our aspiration is to share this training more widely beyond the Environment Agency to improve skills across the industry.

In addition, we have sought to improve knowledge and understanding of flood modelling and quality assurance for project managers and engineers involved in reservoir studies who are not modelling specialists. This has improved understanding of the timescales, assumptions, risks and procedures for flood modelling and quality assurance.

CHALLENGES FOR QUALITY ASSURANCE

The principles of the quality assurance that we are now applying to reservoir modelling studies are not new. Similar procedures have been in place for many years for other types of modelling studies. Nevertheless, the extension of more rigorous quality assurance to reservoir modelling studies has led to some unexpected challenges that are unique to this application.

Firstly, reservoir modelling studies commissioned to fulfil recommendations made for measures in the interest of safety (MIOS) are legally required and must be carried out by the date given by the Inspecting Engineer, which is often 12 months. This timescale can be challenging when allowing for scoping and commissioning a study, collecting data such as survey information, undertaking analysis, and completing quality assurance. Within the Environment Agency, staff availability for quality assurance can also be affected by other high priority or statutory duties such as flood incident response and statutory consultation on land use planning applications. If quality assurance is left to the final few weeks of the programme, it is unlikely to be satisfactorily completed by the MIOS deadline if reworking is necessary.

Secondly, the Reservoirs Act places a legal duty on panel engineers to supervise and decide upon the safety of reservoirs. This legal duty overrides any comments or recommendations made by technical staff during the quality assurance procedure. The quality assurance procedure itself is not legally binding, and the comments made in the review are advice rather than instruction. The final decision on whether to accept the modelling study rests firstly with the project manager and finally with the panel engineer. If the panel engineer accepts the modelling study before the quality assurance process has been completed, this can create conflicting messages for the project team and have commercial implications for the contract. Flood modelling includes a number of subjective decisions around data and methods, and there may need to be a number of iterations as different solutions are tested and assumptions explored.

To overcome these issues, we recommend close collaborative working between the commissioning project manager, the consultant, the panel engineer, and the technical reviewer undertaking the quality assurance. This should occur throughout the project and not be limited to a single quality assurance review when calculations are already completed. Quality assurance discussions and actions should take place at the following stages in the project:

- **Scoping and commissioning.** The technical reviewer should be given notice of the upcoming study and can assist in reviewing the modelling scope and discussing any suggested edits or additional information needed.
- **Project inception.** An inception meeting between the reviewer, the modelling team and the panel engineer will provide an opportunity for the scope to be discussed and any questions raised. The quality assurance process should be explained, and work approaches agreed. Project timeframes should be reviewed so that work can be programmed and any constraints identified.
- **Method statement.** The consultant should submit a method statement for the hydraulic and hydrological modelling, which describes the catchment, the reservoir, the available data, and the proposed methods. This allows any questions over methodological approach to be addressed before calculations are completed.
- **Full calculations: first draft**. A first draft of the full calculations, model and report should be provided by the consultant for review, including all model files and details to allow calculations to be reproduced and the model to be re-run. The reviewer will check the calculations and provide comments and suggestions using the standard review template.
- **Full calculations: final version.** There are typically several rounds of review and discussions before the quality assurance process is completed. This allows consultants the opportunity to respond to comments and suggestions and where appropriate make edits to calculations and reporting. The review process therefore may take a number of weeks to complete. When ready, a final set of calculations, model files and the report should be agreed by the reviewer.

Good communication between the consultant and reviewer is essential. The review process is intended to be constructive and collaborative, rather than critical. Written text can be open to misinterpretation and therefore meetings between the reviewer and consultant are encouraged to discuss the comments, suggestions and proposed actions.

The panel engineer should also play an active role in the quality assurance process by attending meetings and reading review comments. Their knowledge of the reservoir and catchment should be shared with the reviewer and consultant to improve the local representation of the model. The panel engineer should not accept the modelling study until the quality assurance process is complete. Where new works are proposed, there is an even greater need for close collaboration between the engineer, modeller, and reviewer to test solutions iteratively and explore assumptions.

CONCLUSIONS

This paper has described work undertaken by the Environment Agency to extend existing flood modelling quality assurance procedures to studies undertaken for reservoirs owned and operated by the Environment Agency. The procedures are intended to promote quality and consistency across modelling studies carried out by various consultancies through different procurement routes, where the Environment Agency is the client.

Flood modelling is a specialist and technical subject, and the outcomes of erroneous modelling may lead to incorrect assumptions and decisions over risk management. Reworking of incorrect modelling costs time and money, and delays improvements to reservoir safety. To promote quality analysis that is of a higher standard, we have introduced new standardised modelling scopes, reporting templates, quality assurance procedures, guidance, and accessible tools for undertaking calculations. We have introduced new training procedures and mentoring to increase the skills of technical reviewers, and we have also improved general knowledge of flood modelling amongst reservoir engineers and project managers. Many of the materials we have produced can be shared externally for use by other reservoir owners and operators to aid their own quality assurance procedures.

The challenges unique to reservoir applications include legally binding deadlines for MIOS studies, and the potential for conflict between the panel engineer and technical review process. Both challenges can be overcome by purposefully promoting a collaborative and communicative approach to quality assurance from the earliest stage of the project. It is hoped that a more rigorous approach to this particularly uncertain area of reservoir flood risk assessment will add confidence to our estimates. This should in turn make for better overall decision making for safety and sustainability of the chosen solutions.

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EA, (2023) *Flood Estimation Guidelines.* Environment Agency, Bristol, UK. https://www.gov.uk/government/publications/flood-estimation-guidelines (Accessed 2 July 2024)

Environment Agency documents available on request:

LIT 58205: Probable Maximum Flood calculation spreadsheet

LIT 65993: Flood Estimation for Reservoir Safety Calculation Record

LIT 65996: Flood Estimation for Reservoir Safety Calculation Record Accompanying Notes

LIT 72263: Reservoir Modelling Scope

LIT 72757: FSR-FEH and Pumped Rainfall Runoff spreadsheet

LIT TBC: Flood Estimation for Reservoir Safety Calculation Review Template

LIT TBC: Quality Assurance of Reservoir Modelling Studies

The British Dam Society

Developing a new hazard classification for Reservoir Safety in England and Wales

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SYNOPSIS Following the major incident at Toddbrook Reservoir in August 2019 and a subsequent independent review, the Department for Environment, Food and Rural Affairs (Defra) and the Environment Agency (EA) have established the Reservoir Safety Reform Programme (RSRP). Workstream 1 of the RSRP is developing a new reservoir classification system that will underpin new legislation in England and Wales.

This paper presents the main elements of work completed to date which include reviews of international reservoir legislation and other UK high-risk industries, and development and testing of potential classification options. A new approach that uses a multi-criteria classification system is proposed and this will now be taken forwards for further refinement ahead of wider public consultation and drafting of new legislation.

INTRODUCTION

In August 2019 a major incident occurred at Toddbrook Reservoir, located upstream of the town of Whaley Bridge in Derbyshire. Following significant heavy rainfall, part of the spillway collapsed. The embankment dam did not breach, but as a precaution, some 1,500 people in Whaley Bridge were temporarily evacuated while the dam was made safe (Wilson, 2020).

After the incident, the Secretary of State (SoS) for Environment, Food and Rural Affairs commissioned an independent review to evaluate the effectiveness of reservoir legislation and regulations concerning Toddbrook specifically, and the reservoir stock in England as a whole. The initial Part A Review focussed on the Toddbrook incident and identified systemic weaknesses, rather than isolated issues, in the safety regime and poor safety management practices (Balmforth, 2020). The Part B Review considered the safety regime across the reservoir sector and the legislation governing it - the Reservoirs Act 1975 (HMG, 1975). It made 15 strategic recommendations for improving the safety regime, including establishing a new risk/hazard-based safety regime, where safety requirements are in proportion to risks (Balmforth, 2021).

In response to the recommendations made in the Part B report, the Department for Environment, Food and Rural Affairs (Defra) and the Environment Agency (EA) have established the Reservoir Safety Reform Programme (RSRP). The vision of the RSRP is "to create a safety regime for reservoir dams in England which protects our communities, by making us ready for and resilient to climate change – today, tomorrow and the future" (Defra and EA, 2024). This will be delivered in a phased way over several years.

The reform programme comprises six main workstreams (Figure 1). In collaboration with the Welsh Government and Natural Resources Wales (NRW), Defra and the EA have commissioned the Workstream 1 project to develop a new hazard classification system for reservoir safety in England and Wales. The new classification system will apply to reservoirs with a capacity of 10,000m^3 or more above natural ground. It will form the core of a new safety framework that builds on, modernises, and improves the current safety regime, ensuring risks are managed to as low as reasonably practical (ALARP). The project will inform the evidence base for new legislation on reservoir safety.

This paper presents the work undertaken to date to develop and test practical options for the new classification system that is proposed for use in England and Wales.

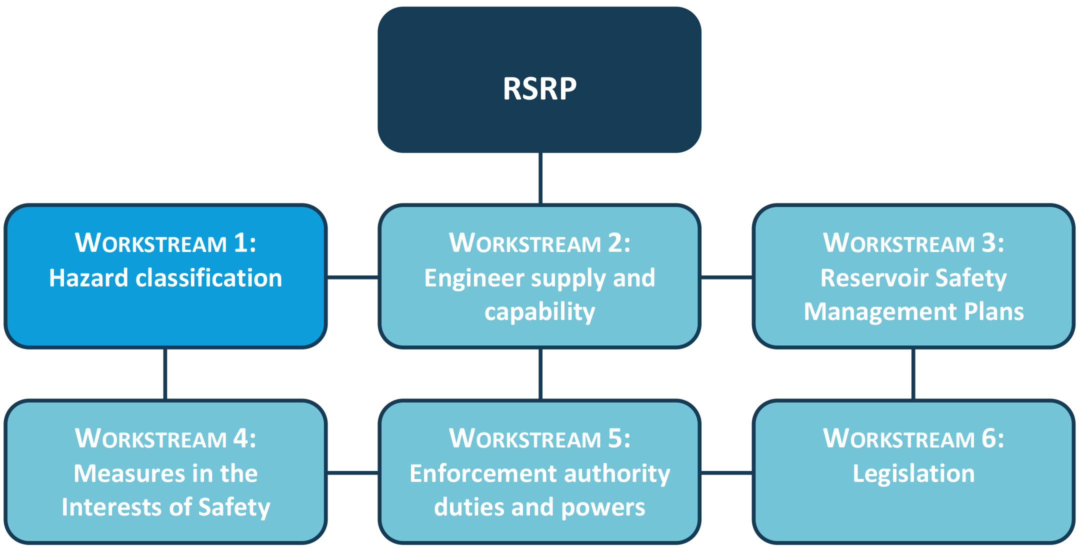

Figure 1. Structure of the RSRP. Workstream 1 is the focus of this paper.

BACKGROUND

Reservoir safety legislation in England and Wales

Calls for the introduction of reservoir safety legislation in Great Britain emerged after several major dam failures in the 1800s (Wright, 1994). However, it was the Dolgarrog dam disaster in November 1925, which claimed 16 lives in North Wales, that finally prompted the introduction of legislation (Charles, Tedd, & Warren, 2011). Since then, there have been no fatalities resulting from dam failures, although major incidents have occurred. For example, the EA recorded a total of 108 major reservoir incidents in the 16 years from 2004-2020.

Since 1930, reservoir safety in Great Britain has been regulated by Acts of Parliament. The Reservoirs (Safety Provisions) Act 1930 (HMG, 1930) required the owners of reservoirs with a capacity of more than five million gallons (22,700m^3) above the natural level of any part of the surrounding ground, to provide for their inspection by a qualified civil engineer who was a member of a panel of civil engineers. The Reservoirs Act 1975 went beyond the provisions of the earlier Act in a number of ways. Local authorities were designated as enforcement authorities, being required to keep registers of all raised reservoirs (defined as those with a capacity greater than 25,000m^3) and to ensure that undertakers, usually the owners, complied with the requirements of the Act. The duties of undertakers, enforcement authorities and engineers appointed to the various panels were laid down in the 1975 Act or set out in regulations.

A major change in reservoir safety occurred on 1 October 2004 when responsibility for the enforcement of safety legislation in England and Wales was transferred from 136 local authorities to the Environment Agency (later Natural Resources Wales in Wales) under the provisions of the Water Act (HMG, 2003), thereby ensuring a uniform application of safety legislation across the country. The Reservoirs Act 1975 was subsequently amended in July 2013 by the Flood and Water Management Act (FWMA) (HMG, 2010). FWMA made provisions for lowering the regulatory threshold from 25,000m^3 to 10,000m^3. To date this has only been enacted in Wales.

Existing categorisation of reservoirs

Registered reservoirs in England and Wales are currently assigned three different classifications:

1) **FWMA 2010 risk designation.** Reservoirs are classified as either 'high-risk' or 'not high-risk' based on whether an uncontrolled release of water could endanger human life. 'High-risk' reservoirs require regular inspection and reporting, while 'not high-risk' reservoirs have reduced requirements due to their lower hazard.
2) **Dam category – Flood safety.** During each Section 10 inspection, typically every ten years, reservoirs are classified by the Inspecting Engineer based on potential consequences of a catastrophic and uncontrolled release of water. Inspecting Engineers follow the standards-based consequence categories given in the Institution of Civil Engineers' (2015) 'Floods and Reservoir Safety, 4th Edition' (FRS4) guide:
   - Category A: Breach could endanger lives in a community (10+ persons affected).
   - Category B: Breach could endanger lives not in a community or cause extensive damage including infrastructure disruption.
   - Category C: Breach poses negligible risk to life and limited damage.
   - Category D: No foreseeable loss of life and very limited additional flood damage.
3) **Dam category – Earthquake safety.** Inspecting Engineers assess dam adequacy under seismic loading, typically using the 'Engineering Guide to Seismic Risks to Dams in the UK' (Charles et al., 1991) and the associated Application Note (Institution of Civil Engineers, 1998). This follows the categorisation in ICOLD Bulletin 72 (ICOLD, 1989).

Reservoirs in England and Wales

According to public register information, there are 2,136 registered reservoirs in England and 402 in Wales. A large proportion of these (80% and 66%, respectively) are designated as 'high risk', meaning they could endanger human life if the dam fails and causes an uncontrolled release of water. The regulation of reservoirs, which aims to keep the likelihood of failure low, protects over two million people and one million households, properties and businesses.

Managing Risks for Dams and Reservoirs

Over three quarters of the registered reservoirs are impounded by embankment-type dams. Reservoir ownership varies from large to small organisations and includes water companies, private landowners and trusts, farmers, flood risk authorities, central and local government, and many private owners.

CLASSIFICATION PROJECT SET-UP

The main project team for Workstream 1 comprises a consortium of JBA Consulting, Ove Arup and Partners Ltd, Risktec, and Paul Sayers and Partners Ltd. A Project Board has been formed to ensure that the overall aims and objectives of the project are met, providing healthy challenge in an open and transparent manner to achieve a successful outcome. The Board consists of representatives from Defra, the EA, NRW, and the Welsh Government. In addition, a High-Level Engagement Group (HLEG) has been established. The purpose of the HLEG is to enable the views and experience of professionals and representative groups, working in the dams and reservoirs sector, to help shape and test approaches for the application of a new risk-based management regime for reservoir safety. This is a broad group representing various stakeholders, including the Reservoir Research and Advisory Group, Defra, the EA, NRW, the UK Panel Engineer Committee, the Major Reservoirs Owners Group, and small owners.

PROJECT SUCCESS FACTORS

Ten key project success measures were established at the inception of the project, specifying that the new classification system should:

1. Cover the whole (existing) reservoir stock from 10,000m^3 upwards.
2. Be robust and transparent.
3. Support a continuous safety improvement culture.
4. Enable reservoirs to be classified in a proportionate and straightforward way.
5. Distinguish between mandatory actions and good practice for owners.
6. Work with the reservoir flood risk mapping data and modelling.
7. Be flexible and responsive to changing risks.
8. Be understood and accepted by stakeholders.
9. Be proportionate in cost/benefit terms.
10. Take into account regulation under other regimes.

EVIDENCE REVIEW

Global reservoir industry

Evidence was collected from other countries about the nature of the dams, dam safety regulation and classification systems. Existing material including ICOLD Bulletin 167 Regulation of Dam Safety: An Overview of current practice worldwide, and ICOLD European Club – Working Group on Safety of Existing dams Report was supplemented by the responses to questionnaires sent to survey contacts from ten countries. The technical review of reservoir safety regimes in other countries was focused on the classification systems used in each country and the criteria/thresholds adopted for regulation. It considered the scope of structures included in the classification, the hazards, risks, and consequences considered, the thresholds for these and how they are determined, and how the criteria and thresholds are applied. The key findings from the review were:

- No international regulatory framework was found which has been prepared for a similar reservoir stock to that of the UK (England, Wales, Scotland and Northern Ireland).
- The majority of countries have regulations that apply to dam safety, although there are some provinces and territories that rely on guidelines of other bodies. A mixture of regulation supported by industry guidance and best practice is therefore common.
- Most countries with reservoir regulations use both height of dam and reservoir volume to define which dams are legislated, i.e. a hazard-based approach to designation.
- The methods by which regulated dams are classified vary. Examples include systems being based on physical properties of dams, consequence of dam failure, risk of dam failure and a combination of these criteria. The most common method of classification is consequence of failure, including impacts on population, environment, property, economy, infrastructure, cultural heritage and public services.
- The number of categories used for classification varies between three and seven, with thresholds varying. Some systems are defined in detail in regulations whilst others are less defined and additional industry guidance is provided to allow categorisation.

Other high-risk UK industries

Evidence was gathered about approaches taken by other high-risk UK industries including nuclear, oil and gas, chemical, rail and aviation. The regulations are typically goal-based, rather than prescriptive, and focus on reducing risk to levels that are ALARP. The key findings were that all sectors studied:

- have a strong regulator to enforce regulations, funded by the duty holder. They require the preparation of formal safety cases, which are produced by or on behalf of the duty holder, with clear identification of safety-critical elements.
- employ some form of hazard screening to reduce the burden on resources. At the highest level, screening can be built into regulation (e.g. Control of Major Accident Hazards (COMAH) regulations (HSE, 2015)), and/or can apply defined levels of unmitigated hazard frequency, unmitigated hazard consequences or unmitigated risk below which no further assessment is warranted.
- employ the concept of proportionality to reduce the burden on resources. This takes the form of applying effort in proportion to risk in respect of risk assessment and ALARP assessment. All sectors employ the ALARP principle in their decision-making process (for tolerable risks). Cost benefit analysis may be used but should not be the sole basis for discounting any option.
- require the preparation and implementation of an effective Safety Management System. This includes emergency planning for incidents and accidents.

Managing Risks for Dams and Reservoirs

Criteria Review

Drawing on evidence from overseas reservoir safety practices and other high-risk UK industries, potential criteria for classifying dams in England and Wales were explored. Table 1 provides an overview of the criteria considered, including evidence of their application elsewhere and whether the criteria were taken forward into option development.

Table 1. Overview of potential criteria for classifying dams

Criteria / Characteristic	Thresholds / Examples used by others	Taken forward to option development?
Volume of water stored	>10,000m³ (Norway) to >1Mm³ (Italy, with height)	Yes
Dam height	>2m (France, Norway) to >15m (Italy, with volume)	Yes
Type of dam construction	Used in parts of Canada	No
Type of liquid/material stored	Most exclude mine wastes due to separate legislation	No
Uses of water in reservoir	No examples found in other countries	No
Age of key structures	Used in Canada for risk assessment and classification	Yes
Condition of key structures	Used in Canada for risk assessment and classification	Yes
Consequences of dam failure	Common criterion, varies by country (quantitative, qualitative)	Yes
Simpler measure of societal risk	Uses potential consequences (e.g. >100 fatalities)	Yes
Environmental consequences	Used with other criteria in matrices	Yes
Other consequences	Includes property, economy, infrastructure, cultural heritage	Yes
Risk of dam failure	Used in Canada	Yes
Categories from FRS4	Four categories based on population at risk	Yes
Categories from Building Research Establishment (BRE) Seismic Guide	Four categories using hazard and consequence criteria	Yes

A comprehensive dataset of consequence metrics exists for reservoirs registered under the Reservoirs Act 1975. These metrics, derived through 2D reservoir flood modelling, include the maximum population at risk (MPAR), the average societal life loss (ASLL), and property damages. ASLL quantifies the potential overall impact on society of a dam failure (Bowles, et al., 2013). For instance, an ASLL of 1.0 does not refer to a specific individual but indicates that, on average, multiple people each have a certain probability of death due to dam failure (e.g., two people each with a 50% chance of death). ASLL stands out as a strong candidate for a classification criterion because it represents the most critical consequence of dam failure. As a result, it features prominently in several of the long- and short-list options.

However, these consequence metrics are currently unavailable for the 1,500 or so smaller reservoirs in England with volumes ranging from 10,000m^3 to 25,000m^3. To address this gap, the feasibility of a simplified classification method that does not require reservoir flood modelling has been explored. To date, the focus has primarily been on dam height and reservoir volume, which dominate in dam classifications in other countries. These parameters have been evaluated both individually and in combination.

Although there are meaningful positive correlations between dam height, reservoir volume, and ASLL, substantial variability exists in ASLL values among dams with similar height and/or volume, as illustrated in Figure 2. For instance, the Coefficient of Variation (CV) for ASLL values exceeds 200% across dams of comparable height. This variability underscores the critical influence of receptor location (i.e., the proximity of people and property to the dam) on ASLL. A simplified classification method based solely on the physical characteristics of dams fails to capture this critical aspect, potentially leading to a disproportionate regulatory burden.

However, there is potential to use physical characteristics as part of a multicriteria approach to classification and for pre-screening. Of particular interest is the relationship used in the classification of dams in French regulations which use dam height, H (m) and reservoir volume, V (m^3) in the equation $H^2 \times V^{0.5}$.

This formula aligns with energy-related principles in physics and demonstrates a stronger correlation with ASLL for reservoirs in England and Wales than either dam height or reservoir volume alone.

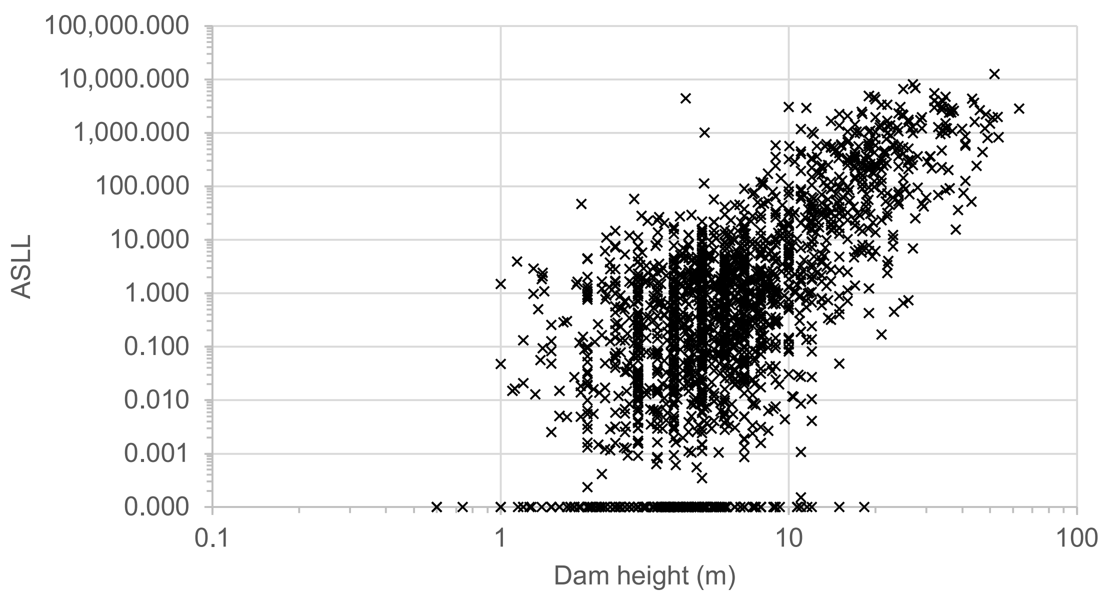

Figure 2. Variation in ASLL with dam height

OPTIONS DEVELOPMENT

Initial Options List

From the research conducted, a total of 15 options were developed as the ‘long list’. These included: deregulation of all reservoirs; maintaining the existing approach; adapting existing classification systems such as FRS4 and seismic classification; options based on different consequence thresholds; consideration of wider consequences beyond loss of life such as economic, environmental and heritage damages; risk-based classification; and the use of surrogate for consequence such as dam height or stored volume.

Stakeholder Consultation

Stakeholder engagement has been central to the development and refinement of the options. Throughout the project, meetings and workshops have been conducted with the Project Board and the HLEG. Draft reports have been issued to both groups for review and feedback. The preferred option will be subject to public consultation at the project's conclusion. Key themes emerging from the engagement to date regarding the new classification system include:

- **Hazard versus risk:** While a risk-based classification is desirable, practical implementation poses significant challenges. For instance, many dams built before the 1900s lack detailed construction records, making it difficult to assess their probability of failure. Stakeholders generally agree that classification should be consequence-based, with risk managed through a comprehensive safety management framework.
- **Type of consequences:** There is a general consensus that classification should primarily focus on the potential loss of human life. While the inclusion of wider consequences, such as environmental impacts, heritage, and critical infrastructure, is important, incorporating these within the classification itself could introduce unnecessary complexity. Instead, it is recommended that these be considered outside the classification but within the overall safety management regime. Additionally, it was suggested that the benefits of reservoirs, such as amenity value and water supply, be considered alongside the consequences of failure to provide a balanced perspective.
- **Proportionate regulation:** Aligning with Balmforth's (2021) recommendations and a primary objective of the project, stakeholders have emphasised that regulation should be proportionate to the hazard and risk posed by reservoirs. Setting appropriate thresholds between hazard classes, supported by evidence where available, is crucial.
- **Support for small owners:** Stakeholders have particularly stressed the importance of proportionate regulation that does not overburden small reservoir owners. Recognising that these owners may lack the resources and expertise to comply with detailed regulations, there is a need for clear guidance and support.
- **Classification structure and terminology:** Stakeholders have called for distinct naming conventions to avoid confusion with existing categories. Additionally, there should be transparency in how the new system maps across from current dam categories.
- **ASLL and other consequence metrics:** There is a general consensus that the derivation of ASLL and other consequence metrics through reservoir flood modelling should be conducted by the enforcement authorities to ensure consistency. As techniques and understanding evolve over time, the approach to reservoir flood modelling is expected to undergo refinement.
- **Pre-screening and exclusions:** Emphasising the importance of efficiency, there is a need for a straightforward screening process to identify reservoirs that do not pose significant hazards. This would enable early exclusion of structures that do not require regulation, ensuring that regulatory efforts are focused on higher-hazard reservoirs.

Options Shortlist

Considering stakeholder feedback, the options were shortlisted to:

A. Existing high-risk reservoirs are divided into three classes depending on the hazard created by the reservoir. Class 1: 100 ≤ ASLL; Class 2 : 1 ≤ ASLL < 100; Class 3: ASLL < 1; Class 4: Existing 'not high risk'

B. Class 1: 1,000 ≤ ASLL; Class 2: 100 ≤ ASLL < 1,000; Class 3: 1 < ASLL < 100; Class 4: ASLL < 1; Class 5: ASLL < 0.1.

C. Class 1: 100 ≤ ASLL; Class 2 : 0.1 ≤ ASLL < 100; Class 3: ASLL < 0.1; Class 4: minimal hazard assessment based on volume, height, peak outflow and containment of a breach in the downstream channel.

D. As Options A to C with additional consideration of infrastructure, economic impacts, societal impacts

E. High risk – 1 ≤ ASLL; Medium risk – ASLL <1; Low risk – existing 'not high risk'

F. Classification based on the BRE seismic guide with scoring for capacity, dam height, population at risk and potential downstream damage.

SAFETY MANAGEMENT PRACTICES

A Reservoir Safety Management System is proposed, informed by the evidence review. The key components are:

- **Reservoir Safety Case (RSC):** Serving as the cornerstone of risk-informed reservoir management under the new system, it sets out the key safety elements of the reservoir's design and operation and includes a risk assessment to identify risks and failure modes.
- **Reservoir Safety Management Plan (RSMP):** Describes how the owner will operate and manage their reservoir safely.
- **Panel Engineer Supervision:** Similar to the current Supervising Engineer role, with annual reporting that includes a review of compliance with the RSMP.
- **Periodic Safety Review:** Similar to the current Inspecting Engineer role, involving a physical inspection of the reservoir plus a detailed review of the RSC and RSMP.

OPTIONS TESTING

The evaluation of the shortlisted options has comprised three primary components: categorising the existing reservoir stock into hazard classes for each option, estimating the costs to reservoir undertakers of implementing the safety management system, and analysing the performance of each option using ten case study reservoirs. Due to the unavailability of metrics related to the wider consequences of dam failure, Option D has not been tested.

Allocation of existing reservoir stock

Over 2,000 existing reservoirs were classified under each proposed option. This revealed significant differences in distribution between the current legislation and the shortlisted options, particularly Options A, B, and F. Under the current system, 81% of the dataset is classified as 'high risk', with 14% classified as 'not high risk' and the remaining 5% yet to be designated. In contrast, Options A, B, and F distribute the reservoirs across multiple classes, with the majority in lower hazard categories.

Option E, which splits the current high-risk category in two, does not perform as well, categorising 41% of reservoirs in the highest hazard class (ASLL ≥ 1). Similarly, Option C results in a large proportion (52%) falling into Class 2 due to its wide ASLL range (0.1 ≤ ASLL < 100).

Only a limited number of reservoirs in the dataset have an ASLL value exceeding 1,000 (Class 1 under Option B). This relatively small proportion in the highest hazard class raises security concerns by making individual extreme-consequence reservoirs more identifiable. Therefore, setting an upper threshold of ASLL > 100 is more desirable, as it would include a significantly larger proportion of reservoirs and therefore reduce security risks. Additionally, this threshold aligns with upper thresholds in other countries and safety regulations in other UK industries.

Costing of options

Industry advice was used to establish cost estimates for the various safety management practices. These were used to assess national-scale costs to reservoir undertakers of implementing the proposed safety management system under each shortlisted option.

Figure 3, based on the existing reservoir stock, illustrates that Options A and F are the most expensive, followed by Option E. The costs of Options B and C are comparable to the current regime. Option A incurs higher costs primarily because it retains the 'not high risk' category, resulting in nearly 50% fewer reservoirs in the lowest hazard class compared to Options B and F. In other words, Option A subjects more reservoirs to stringent regulation under higher hazard classes compared to Options B and F. Option F's elevated costs are due to its scoring system, which does not consistently align with the ASLL threshold values used in other options. As a result, the scoring for Option F has undergone further refinement.

Case study analysis

To further evaluate the performance of the shortlisted classification options, ten case studies suggested by the HLEG and Project Board were analysed. These case studies have a variety of attributes including dam type, reservoir volume, dam height, and existing classifications.

A key finding from this analysis is that having more hazard classes for reservoirs with lower consequences provides clearer step changes in safety management requirements and hence costs for undertakers. Specifically, Options B, C, and F lower costs for some case study reservoirs compared to the current regime by creating distinct classes for ASLL values below 1, with no mandatory safety management practices proposed except for incident reporting.

The case study analysis has also highlighted the need to refine the scoring system in Option F to optimise threshold settings and achieve more proportionate regulation.

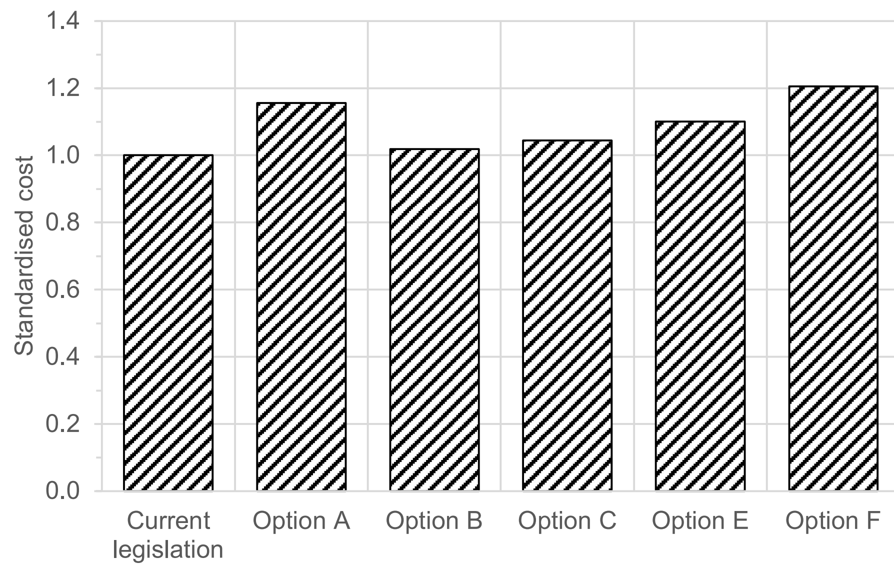

Figure 3. Standardised national costs relative to the current regime for reservoir safety management by undertakers over a 20-year period (based on existing reservoir stock)

Narrowing down and refining options

Following detailed analysis and extensive stakeholder feedback, the following conclusions regarding the shortlisted options were made:

- **Reject Option C:** The ASLL range of 0.1 to 100 in Class 2 is too wide, failing to achieve the desired proportionality.
- **Reject Option D:** This option introduces unnecessary complexity and lacks sufficient stakeholder support.
- **Reject Option E:** The high-risk category is too broad, failing to achieve the desired proportionality and is not favoured by stakeholders.
- **Recommend combination of Options A and B (Option AB):** A hybrid approach combining the strengths of Options A and B, with an upper ASLL threshold of 100 and the elimination of the existing 'not high risk' category. It performs well in meeting project success measures.
- **Recommend Option F with a refined scoring system:** This option employs a multi-criteria classification, enhancing robustness. It shows strong performance in meeting project success measures. The inclusion of property damages in the total classification factor adds valuable information for a more nuanced classification and thus more proportionate regulation. A refined version of Option F is presented in Table 3.

Table 3. Refined version of Option F (subject to further refinement)

a) **Component classification factors**

Classification factor	Value range	Weighting points
$H^2 \times V^{0.5}$	≥ 100,000	6
	8,000 to < 100,000	4
	356 to < 8,000	2
	< 356*	0
ASLL	≥ 100	16
	10 to < 100	12
	1 to < 10	8
	0.1 to < 1	4
	< 0.1	0
Potential downstream damage	Extreme	12
	High	8
	Moderate	4
	Low	0

b) **Total classification factor (TCF, sum of weighting points)**

Hazard class	Threshold values
Class 1	28 ≤ TCF
Class 2	14 ≤ TCF < 28
Class 3	6 ≤ TCF < 14
Class 4	TCF < 6

*The lower threshold of 356 is based on a dam height of 1.5 m and reservoir volume of 25,000 m3

PREFERRED OPTION AND NEXT STEPS

The RSR Programme Board has reviewed the findings and concluded that both Options AB and F are viable for the new classification system. However, they have chosen to advance Option F for further development because its multi-criteria approach offers greater robustness.

Two versions of Option F will be considered: one based on reservoir flood modelling utilising ASLL, and one less dependent on modelling and using Population at Risk. The latter is intended to allow for preliminary classification before consequence metrics from standard modelling are available. Option F, along with the reservoir safety management system, will be further refined in collaboration with the Project Board and HLEG.

Upon the project's conclusion in Autumn 2024, the preferred option will undergo public consultation ahead of the UK Government bringing forward new legislation for England and Wales in the late 2020s.

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Risk-based approach for safety review of tailings dams

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SYNOPSIS The aim of this paper is to show, by way of a case study, how the risk-based approach to the safety review of dams can help dam owners prioritise upgrade options.

Risk-based assessment is a powerful tool to assess the safety of a dam by focusing on credible failure scenarios which will help identify risks, prioritise the required actions and eventually mitigate the risks in an efficient and cost-effective way. The main advantage of this method compared to the traditional standards-based assessment is the prioritisation of the risk mitigation options based on the risks associated with different failure modes.

This paper is based on a risk-based safety assessment that was carried out for an existing tailings dam, referred to herein as a Tailings Storage Facility (TSF). The objective of the study was to ensure that the risks to society are tolerable and to suggest several practicable risk mitigation options. As a result, the estimated risks for all loadings and failure modes were expressed as F-N plots representing the level of societal risk.

Although the risk profile of the TSF was determined to be in the risk-tolerable area, efficient risk mitigation options were evaluated which could reduce the risk significantly; however, due to the marginal initial risk of the project it was concluded that the project is satisfying ALARP at this stage of the construction.

INTRODUCTION

This paper evaluates the associated risk posed by a Tailing Storage Facility (TSF). SLR Consulting undertook the quantitative risk assessment and the required studies to evaluate the level of "life safety" risk and determine whether the facility meets the tolerable risk criteria outlined in the ANCOLD Guidelines on Risk Assessment (ANCOLD, 2022).

Standards-based assessments for this project were undertaken previously including geotechnical investigations and interpretation, stability assessments, numerical (finite element method) seepage and deformation analyses, monitoring and instrumentation, detailed design of the facilities, etc. The existing information above combined with Failure Modes and Effects Analysis (FMEA), Dam Break Assessment (DBA) and Consequence Category Assessment (CCA) of the facility was used for the purpose of the risk assessment associated with the failure of the main embankment.

The risk assessment process adopted is illustrated in Figure 1.

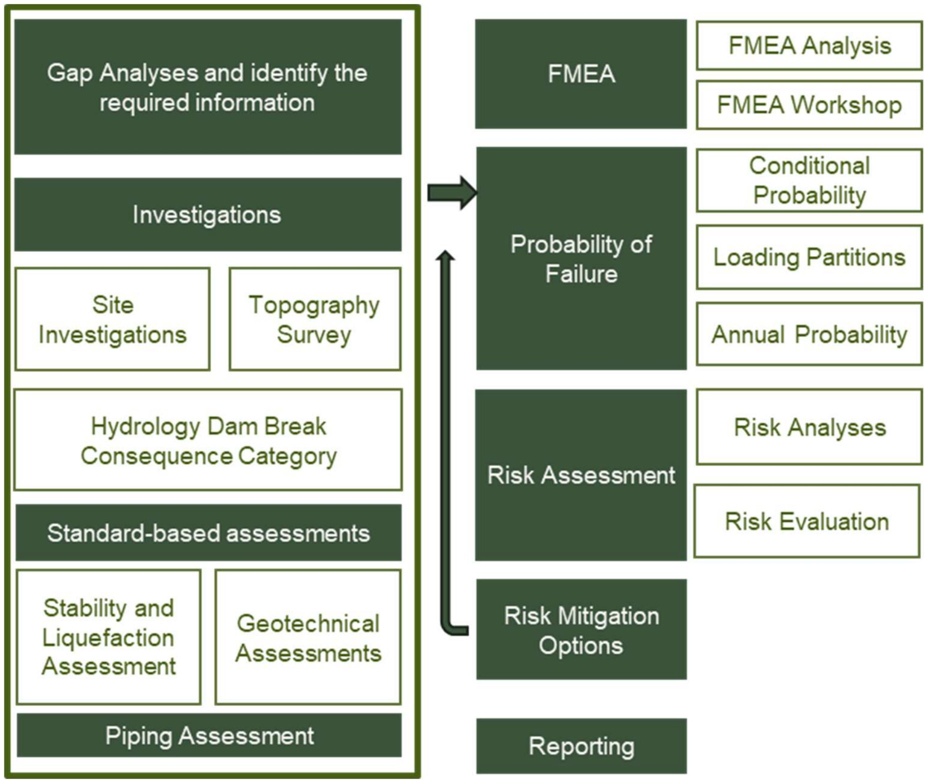


Figure 1. The process of the risk-based safety assessment

DAM BREAK AND CONSEQUENCE ASSESSMENT OUTCOME

Table 1 summarises the potential loss of life (PLL) estimated for each of the breach cases. Estimates of PLL have been developed by applying estimated fatality rates to the population at risk (PAR) for both a flood-induced dam break scenario and a no-dam break scenario for the same magnitude flood event. Incremental PLL is calculated as the increase in PLL between these two scenarios.

Table 1. Population at risk (PAR) and potential loss of life (PLL)

Scenario	Incremental PAR	Incremental PLL
Dam Break – Sunny day	6	6
Dam Break - 1:100 year	7	7
Dam Break - 1:1000 year	7	7
Dam Break - 1:10000 year	180	10

Note: The population at risk for the first three rows are mine workers who are working at the downstream toe of the embankment

Consequence Classification Assessment (CCA)

The facility has been assessed to be a High A consequence category facility, in accordance with the ANCOLD Guidelines on Tailings Dams (ANCOLD, 2019).

FAILURE MODES AND EFFECTS ANALYSES (FMEA)

A failure mode is defined as the way that a failure can occur, describing how an element or component failure must occur to cause loss of the sub-system or system function, and should form an essential part of a risk assessment.

During FMEA workshops, the following credible failure modes (FMs) were identified for further assessment as part of the quantitative risk assessment (QRA). (Table 2)

Table 2. Credible failure modes for the purpose of risk assessment

FM	Initiating	Failure Mode (FM)
ST1E	Earthquake	Instability of the embankment due to liquefaction of the tailings
ST2F	Normal/Flood	Downstream embankment slope instability due to flood loading
IM7F	Normal/Flood	Piping initiated by transverse cracking in the embankment crest due to desiccation by drying (IM7-Piping Toolbox)
IM14F	Normal/Flood	Piping initiated by continuous high permeability layer in the embankment (IM14-Piping Toolbox)
OVF	Normal/Flood	Failure due to scour erosion of the crest because of overtopping

ASSESSMENT OF THE CONDITIONAL PROBABILITIES OF FAILURE

The evaluation of the probabilities of failure was based on the event tree approach. An event tree consists of a series of linked nodes and branches. Each node represents an uncertain event or condition, while each branch represents one possible outcome of the event or one possible state that a condition may assume (i.e., the system response).

Potential of Failure Due to Instability of the Embankment

The stability evaluations were performed for the embankment for long-term, short-term and post-liquefied conditions.

SLOPE/W software (part of GeoStudio 21 R2) was used to evaluate embankment stability by applying the Morgenstern-Price method of slices to the section. The results are summarised in Table 3. The safety factors have been improving since 2022 due to the ongoing construction of a rockfill buttress at the embankment toe.

Table 3. Slope stability safety factors (FoS) obtained from the analyses

Analyses	FoS
Drained	2.5
Undrained	1.4
Post-Liquefied	1.0

Conditional Probability of Failure Due to Instability of the Embankment (ST2F)

Event tree probabilities for this failure mode due to non-seismic failure of the embankment are tabulated in Table 4 and the system response curve is presented in Figure 2.

Table 4. The conditional probability of failure due to instability of the embankment –ST2F

Flood levels (mAHD)	Probabilities				
	Slope instability (1)	Tailings overtopping(2)	Scour erosion(2)	Breach	Conditional Probability
809.00 (F1)	1.00E-04	1.00E-02	1.00E+00	1.00E-01	1.00E-07
810.00 (F2)	1.00E-04	1.00E-01	1.00E+00	1.00E-01	1.00E-06
810.41 (F3)	1.00E-04	1.00E-01	1.00E+00	5.00E-01	5.00E-06
810.94 (F4)	1.00E-04	1.00E+00	1.00E+00	1.00E+00	1.00E-04
811.40 (F5)	5.00E-03	1.00E+00	1.00E+00	1.00E+00	5.00E-03
>811.4 (F6)	1.00E-02	1.00E+00	1.00E+00	1.00E+00	1.00E-02

Notes:

1. These probabilities are based on the safety factors obtained from stability assessment and the system response curve based on reliability theory

2. These probabilities are based on the mapping scheme linking the description of likelihood to quantitative probability adopted by Barnie et al 1996

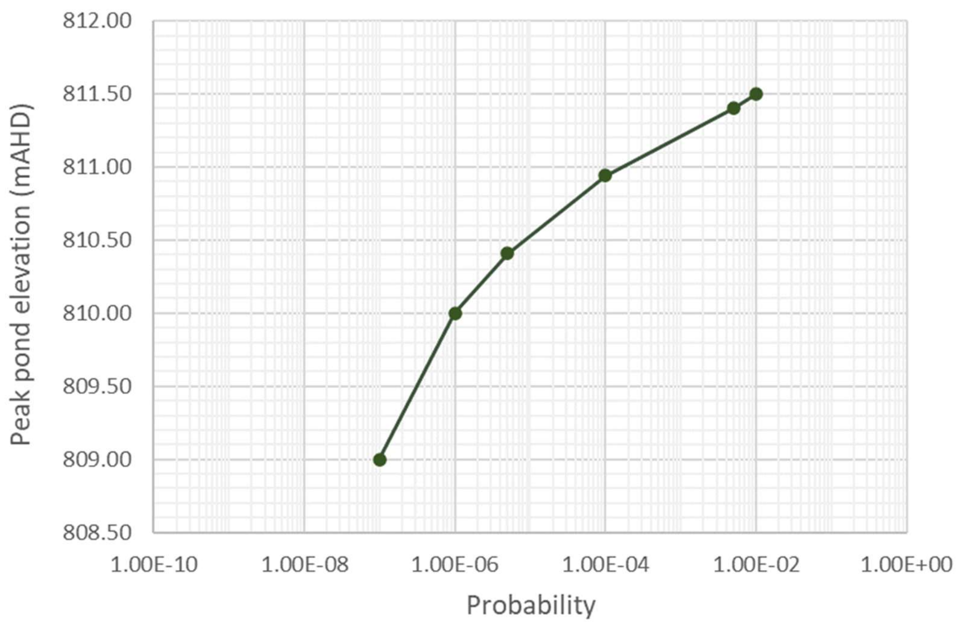


Figure 2. System response curve - failure due to instability of the embankment – ST2F

Conditional Probability of Failure Due to Instability of the Embankment (ST1E)

Event tree probabilities for this failure mode due to post-seismic liquefaction of the tailings material are tabulated in Table 5 and the event tree chart is presented in Figure 3. The resultant system response curve is shown in Figure 4.

Table 5. Conditional probability of failure due to instability of the embankment

Representative PGA for load partition (g)	Probabilities				
	Liquefaction of tailings(1)	Post seismic instability(2)	Tailings overtopping(3)	Uncontrolled release(3)	Conditional Probability
S1- 0.05	5.0E-04	4.0E-02	1.0E-03	1.0E+00	2.0E-08
S2- 0.08	4.0E-02	4.0E-02	1.0E-02	1.0E+00	1.6E-05
S3- 0.15	1.1E-01	4.0E-02	1.0E-01	1.0E+00	4.6E-04
S4- 0.23	8.5E-01	4.0E-02	5.0E-01	1.0E+00	1.7E-02
S5- 0.23<ag	9.9E-01	4.0E-02	1.0E+00	1.0E+00	4.0E-02

Notes:

1. These probabilities are based on the methodology recommended by Robertson (Robertson and Cabal, 2022)

2. These probabilities are based on the safety factors obtained from stability assessment and the system response curve based on reliability theory

3. These probabilities are based on the mapping scheme linking the description of likelihood to quantitative probability adopted by Barnie et al 1996

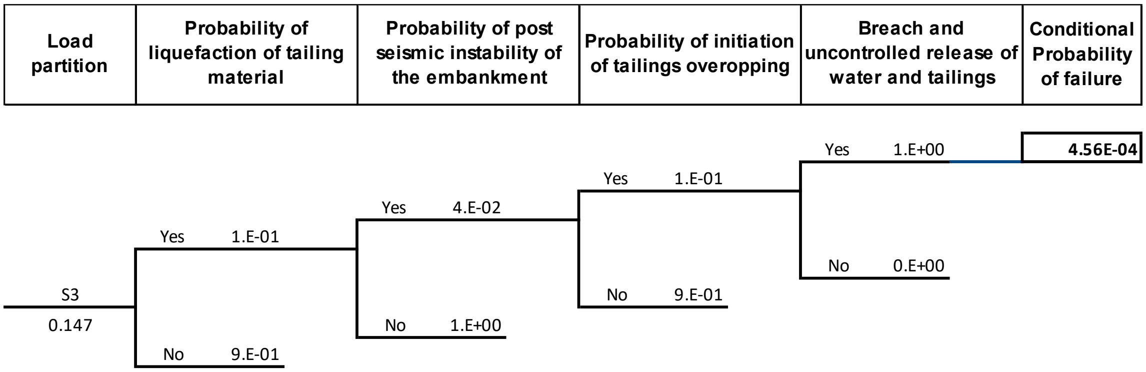


Figure 3. Event tree - failure due to instability of the embankment

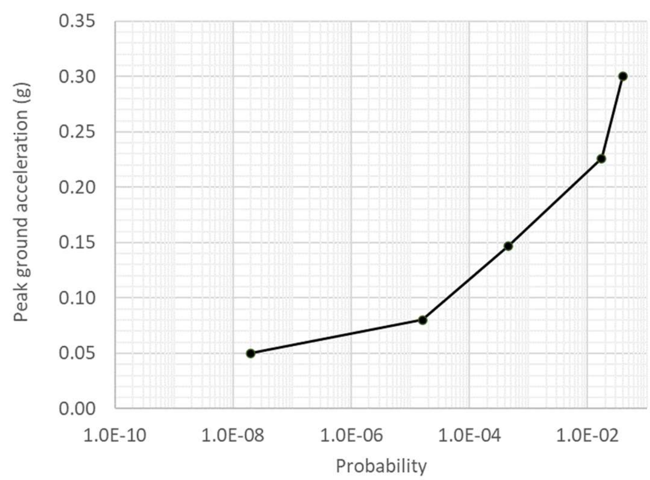


Figure 4. System response curve - failure due to instability of the embankment

Potential for Internal Erosion and Piping

Assessment of the potential for internal erosion and piping has followed the procedures of the Piping Toolbox[1] which is a systematic approach based on event tree analyses and includes the following five steps. A schematic showing different steps of the piping toolbox approach is demonstrated in Figure 6.

In order to apply the Piping Toolbox to a TSF the outer tailings zone is assumed to represent the core of the embankment dam and the upstream zone closer to the pond the upstream shoulder and the material properties for both the core and upstream shoulder are similar.

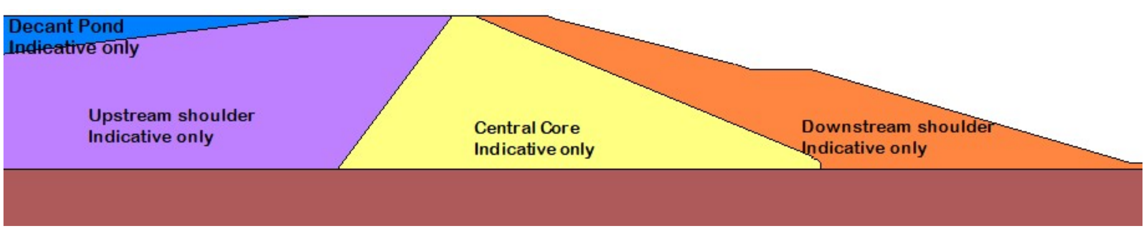


Figure 5. Schematic sketch of the zones for the purpose of piping assessment

[1] Piping Toolbox is a Unified Method for Estimating Probabilities of Failure of Embankment Dams by Internal Erosion and Piping Guidance Document developed by the University of NSW, URS, US Army Corps of Engineers, and US Bureau of Reclamation (Gilbert and UNSW, 2009).

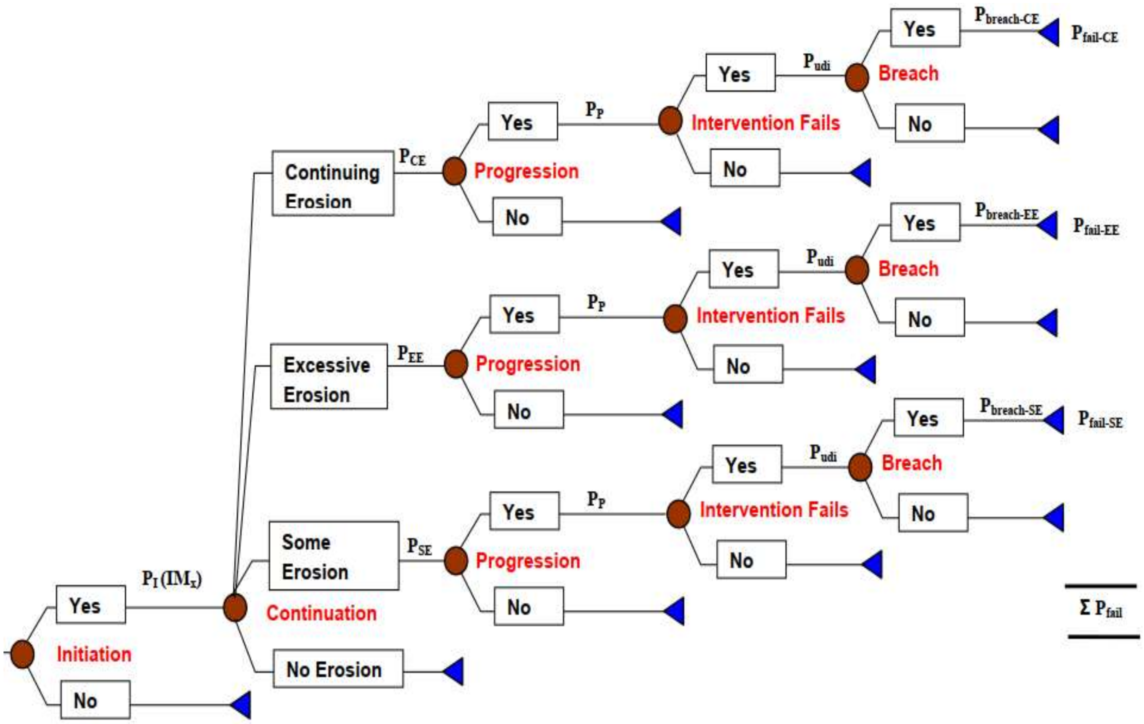


Figure 6. Event tree structure following piping toolbox 2008

Piping Failure because of Cracking in the Crest Due to Desiccation by Drying (IM7F)

Desiccation cracks are tensile cracks that occur because of the combination of drying and an increase in suction forces developed in the materials forming the crest. The system response curve associated with this FM is demonstrated in Figure 7.

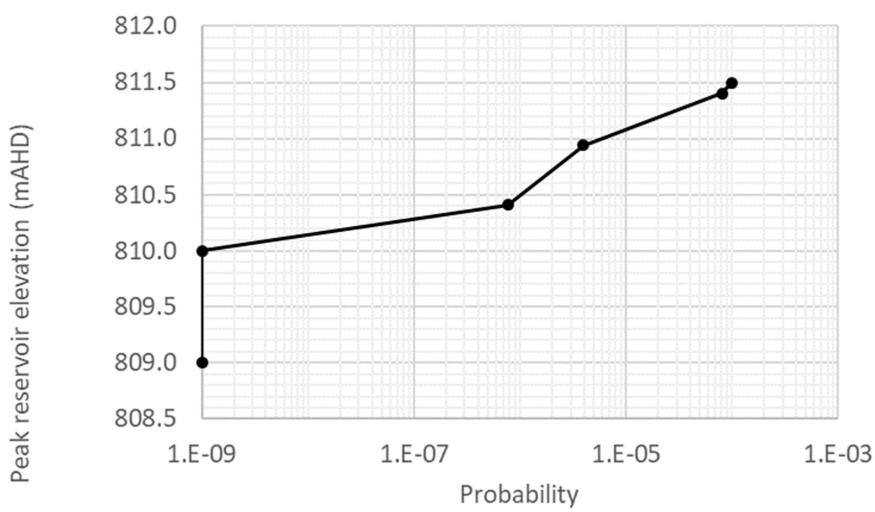


Figure 7. System response curve for piping failure (IM7F)

Piping Failure Because of High Permeability Zone in the Embankment (IM14F)

The crack initiation and propagation mechanism is dependent on soil compaction, mineralogy, initial moisture content, etc. Considering the tailings are hydraulically deposited and generally loose, it was assumed that all layers are poorly compacted. The system response curve associated with this FM is demonstrated in Figure 8.

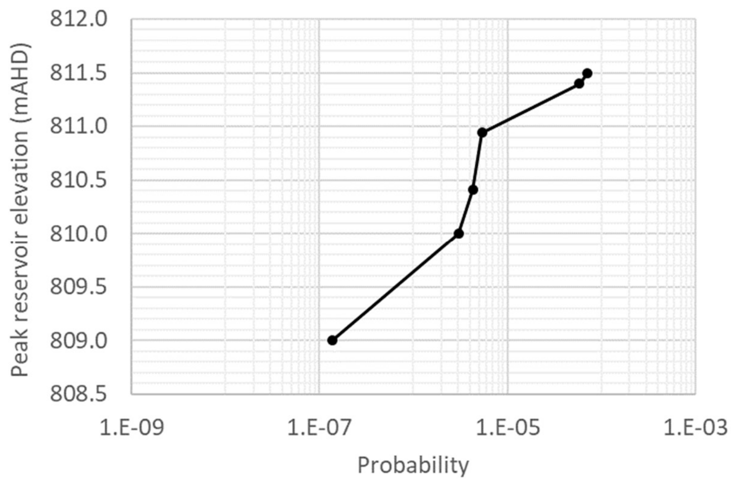


Figure 8. System response curve for piping failure (IM14F)

ASSESSMENT OF THE ANNUAL PROBABILITY OF FAILURE

This section includes estimating annual probabilities of various loading ranges (e.g. flood or earthquake) and estimating the annual probability of failure of the facility by combining the annual probabilities of the loading partitions with the conditional probabilities of the credible failure scenarios.

Load Partitioning

The resultant flood and seismic frequency curves for the TSF provide relevant frequency data for preparing a peak water level and peak ground acceleration (PGA) versus annual exceedance probability (AEP), which was used for the calculation of the annual probability of failure, risk assessment and risk evaluation.

Based on the frequency curves mentioned above, flood levels and PGAs were partitioned into a number of loading partitions as summarised in Table 6. Conditional probabilities of failure are estimated for each of the flood partitions (F1 to F6) and seismic partitions (S1 to S5), and for each credible failure mode identified from the FMEA.

Table 6. Flood loading partitions for annual probability assessment

Event	Group name	Event	Description	Representative Level (mAHD)/PGA(g)	Annual Probability of occurrence
Flood	F1	Flood	Mean decant pond level	809.0	5.50E-01
Flood	F2	Flood	1/100 Flood event	810.0	4.40E-01
Flood	F3	Flood	1/1000 Flood event	810.4	9.00E-03
Flood	F4	Flood	Spillway sill level	811.0	9.00E-04
Flood	F5	Flood	Crest at 111.4 mAHD	811.4	1.00E-04
Flood	F6	Flood	Above the crest	811.4<Flood level	5.00E-07
Seismic	S1	Seismic	Below 500 years event	0.05	9.98E-01
Seismic	S2	Seismic	500 years event	0.08	1.50E-03
Seismic	S3	Seismic	2000 years event	0.15	3.00E-04
Seismic	S4	Seismic	5000 years event	0.23	1.00E-04
Seismic	S5	Seismic	10000 years event	$0.23<a_g$	1.00E-04

Managing Risks for Dams and Reservoirs

Dam Failure Due to Slope Instability

The annual probabilities of breach due to these failure modes have been estimated by combining the conditional probabilities with the annual probabilities of the load partitions. (Table 7)

Table 7. Annual probability of failure due to slope instability

Load partitioning	Partition likelihood	Representative flood level (mAHD)	Annual Probability	Total Annual Probability
F1	5.50E-01	809.00	5.5E-08	1.13E-06
F2	4.40E-01	810.00	4.4E-07	
F3	9.00E-03	810.41	4.5E-08	
F4	9.00E-04	811.00	9.0E-08	
F5	1.00E-04	811.40	5.0E-07	
F6	5.00E-07	>811.4	5.0E-09	
S1 to S5	NA	809.00	5.85E-06	5.85E-06

Note: Seismic-related failure probabilities will be included in F1 load partitioning.

Probability of Failure Due to Piping

The annual probabilities of breach due to these failure modes have been estimated by combining the conditional probabilities mentioned above with the annual probabilities of the loading partitions and are presented in Table 8.

Table 8. Annual probability of failure due to piping (IM7F)

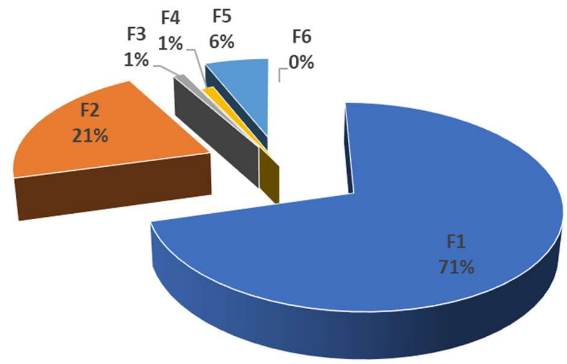
Loading partition	Representative level mAHD	Partition probability	Annual Failure probability (IM7F)	Annual Failure probability
F1	809.00	5.50E-01	5.50E-10	7.52E-08
F2	810.00	4.40E-01	4.40E-10	1.33E-06
F3	810.41	9.00E-03	6.87E-09	3.88E-08
F4	811.00	9.00E-04	3.55E-09	4.83E-09
F5	811.40	1.00E-04	8.01E-09	5.83E-09
F6	>811.4	5.00E-07	5.05E-11	3.57E-11
Total			**1.95E-08**	**1.46E-06**

Contribution of Each Loading Partition to the Annual Probability of Failure

Table 9 presents a summary of the estimated annual probabilities of failure of the TSF and contribution of each loading partition to the total failure.

Table 9. Estimated annual probabilities of failure for different loading partitions

Loading Partition	Annual Probability	Contribution (%)
F1	6.0E-06	70.69%
F2	1.8E-06	20.98%
F3	9.1E-08	1.07%
F4	9.8E-08	1.16%
F5	5.1E-07	6.04%
F6	5.1E-09	0.06%
	8.5E-06	**100.00%**

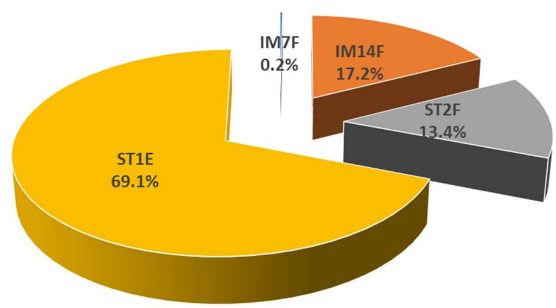


Contribution of Each Failure Mode to the Annual Probability of Failure

Table 10 presents a summary of the estimated annual probabilities of failure of the TSF due to the different failure modes that were assessed. From this table, failure due to slope instability contributes to around 82.5% of the total annual failure probability of 8.5×10^{-6} whereas failure due to piping contributes around 17.5%.

Table 10. Estimated annual probabilities of failure for each failure mode

FM Number.	Annual Probability	Contribution (%)
IM7F	1.9E-08	0.23%
IM14F	1.5E-06	17.24%
ST2F	1.1E-06	13.38%
ST1E	5.9E-06	69.14%
Total	**8.5E-06**	**100.00%**



RISK ASSESSMENT

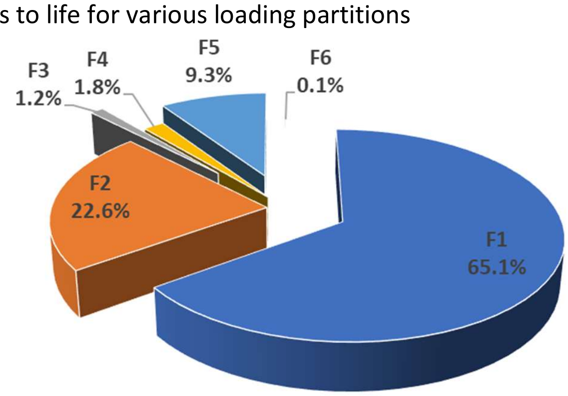
This section presents the result of the risk assessment of a failure through the TSF embankment and compares the estimated risks to the risk tolerability criteria for existing dams specified in the ANCOLD Guidelines on Risk Assessment (ANCOLD, 2022).

Estimation of Societal Risk

Table 11 summarises the estimated annual risks to life due to a failure of the TSF and the contribution of different loading partitions to the total risk.

Table 11. Annual risks to life for various loading partitions

Loading Partition	Risk	Contribution
F1	3.6E-05	65.14%
F2	1.2E-05	22.55%
F3	6.3E-07	1.15%
F4	9.8E-07	1.79%
F5	5.1E-06	9.28%
F6	5.1E-08	0.09%
Total	**5.5E-05**	**100.00%**



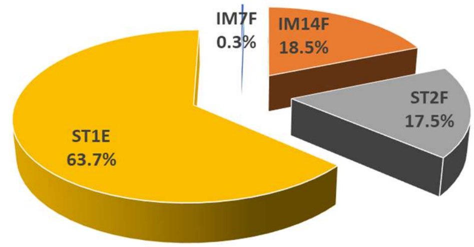
Managing Risks for Dams and Reservoirs

Estimation of Societal Risk

Table 12 summarises the estimated annual risks to life due to a failure of the TSF for the various failure modes assessed and their contributions to the total risk.

Table 12. Annual risks to life for various failure modes

Failure mode	Risk	Contribution
IM7F	1.7E-07	0.31%
IM14F	1.0E-05	18.46%
ST2F	9.7E-06	17.52%
ST1E	3.5E-05	63.71%
Total Risk	**5.5E-05**	**100.0%**



RISK EVALUATION

The plot position of the F-N curve presented as Figure 9 indicates the level of societal risk posed to the public. The diagonal line represents the safety threshold for societal risk associated with existing dams as recommended by ANCOLD (2022). The F-N plot shows that the level of societal risk posed by the TSF is below the specified safety threshold by around two orders of magnitude.

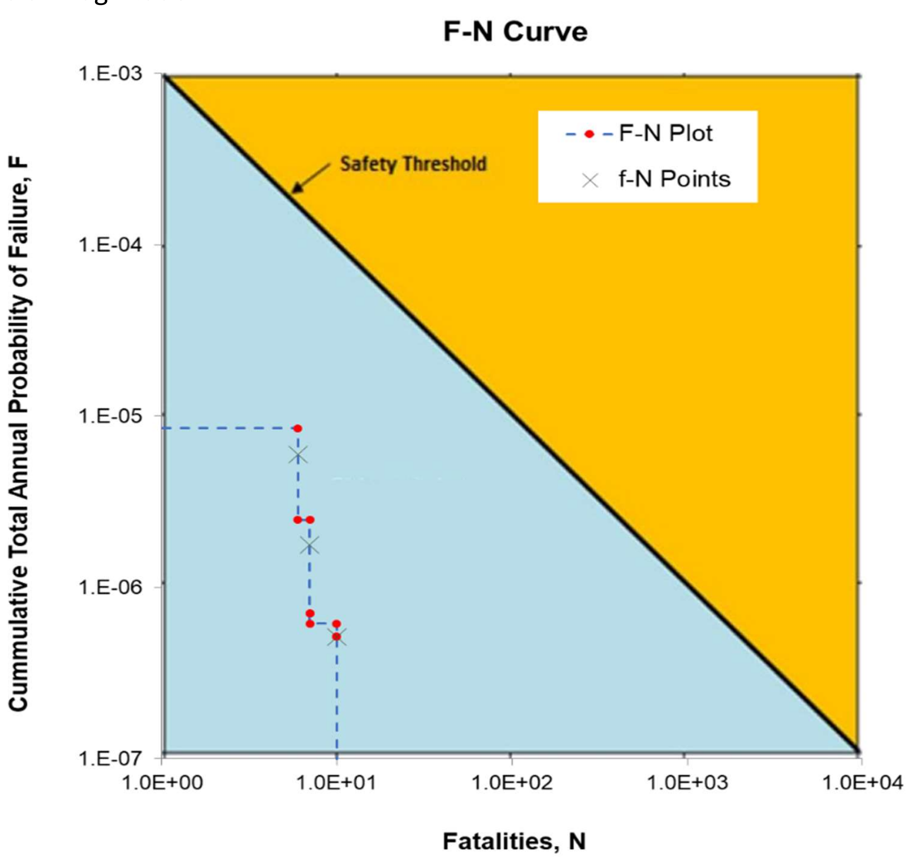


Figure 9. F-N plot showing the level of societal risk (Figure 7.4 ANCOLD 2022)

The estimated individual risk associated with TSF failure is 4.23x10^{-6} per annum, which is lower than the safety threshold of 10^{-4} per annum for an existing dam as recommended by ANCOLD (2022).

ALARP PRINCIPLES AND RISK MITIGATION OPTIONS

Both ANCOLD (2022) and the Global Industry Standard on Tailings Management (GISTM) (GTR, 2020) require risk reduction measures to be implemented to reduce risks for each credible failure mode to a level that the risk is as low as possible while the mitigation option is reasonably practicable, known as ALARP.

The most appropriate of the identified risk reduction measures would need to be further developed to determine basic definition and costing. The following options were initially considered:

1) Enhanced emergency evacuation procedures
2) Relocation of mining personnel most at risk
3) Installation of geotextile on the upstream embankment and adjacent tailings beach
4) Further buttressing of the downstream shoulder of the embankment

Among the items above, the first two items were considered to be administrative control and will mitigate the risk by managing the consequence, while the remaining two items will focus on reducing the probability of failure.

Residual Risk due to Enhanced emergency evacuation procedures, and the Relocation of mining personnel at risk in the immediate vicinity of the downstream toe

Considering the high fatality rate in the immediate vicinity of the TSF embankment, it was recommended to relocate mining personnel to another location with a lower risk. Assuming the total number of personnel in the area, including contractors and dam operators, can be reduced to 50% this will reduce the societal risk to 2.8×10^{-5}, which is 50% of the original risk. Considering that the initial risk is relatively low and although the risk reduction option would reduce the risk significantly, the cost of saving a statistical life (CSSL) is much more than the value of a statistical life (VSL) in Australia.

Residual Risk after Construction of Downstream Buttress

Construction of a downstream buttress is ongoing and it can be assumed at each stage it will improve the embankment stability safety factors by 10%. Based on this assumption, the residual risk is estimated to be 47% of the primary risk. Again, considering the initial risk is relatively low and although the risk reduction option would reduce the risk considerably, the CSSL is much more than the VSL in Australia.

Residual Risk after Applying a Geotextile Cover on the Upstream Embankment

The inclusion of embankment upstream geotextile protection was considered which may provide a risk reduction, add resiliency and/or improve facility operation. The residual risk is estimated to be 82% of the primary risk which is less effective than the other mitigation measures.

CONCLUSION

- Risk-based assessment of the safety status of dams and tailings facilities will enable us to understand the actual risks associated with different components of the project.
- Defining the risk profile of the project will help dam owners to proceed with the best upgrade option to mitigate the risk more efficiently.

Managing Risks for Dams and Reservoirs

- In certain instances, managing the dam failure consequences to reduce risk is more efficient, economical and quicker than reducing the probability of failure. This can be defined relatively simply by undertaking a risk-based safety review.
- When the initial risks are well below accepted safety thresholds (e.g. those provided by ANCOLD (2022)), the justification for risk reduction becomes more challenging. therefore, the justification to satisfy the ALARP principles will be more straightforward.

ACKNOWLEDGEMENTS

Thanks to our colleagues in SLR (globally) who provided insight and expertise that greatly assisted in the development of this paper.

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The British Dam Society

The development of reservoir safety management plans

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SYNOPSIS The national reservoir safety review in 2021 indicated a need for high-risk statutory reservoirs to have in place a reservoir safety management plan (RSMP). The intent was that this plan should set out what surveillance, monitoring and maintenance is required at a reservoir and how it is to be operated, together with the frequency of each element, how it is to be delivered and by whom. Drawing on research for the Environment Agency, the paper discusses:

- The basic requirements of a RSMP in line with international practices and guidance
- How the scope of the RSMP for any given reservoir might be informed through legislation and site-specific studies
- The responsibilities of stakeholders in preparing and managing the plan
- How development of RSMP's might change the current provisions for reservoir record-keeping.

INTRODUCTION

The Reservoir Review (the Review) in 2021 (Balmforth, 2021) proposed that high-risk statutory reservoirs in England should have a Reservoir Safety Management Plan (RSMP). The Review stated:

> *"All high risk reservoirs should have a reservoir safety management plan (RSMP) in place. This should set out what surveillance, monitoring and maintenance is required at a reservoir and how it is to be operated, together with the frequency of each element, how it is to be delivered and by whom. It would be in addition to and sit alongside an on-site emergency plan, and be appended with a record of all surveillance, operational and maintenance activities together with associated data, measurements and other information, which should be kept up to date."*

The Review also stated that the Supervising Engineer should review and approve the RSMP annually and to certify that the owner's actions have been carried out in accordance with the plan.

Mott MacDonald was commissioned by the Environment Agency (EA) to consider the elements required of a RSMP, how its development would affect the current statutory safety records (Prescribed Form of Record), the responsibilities of key stakeholders in managing the

RSMP and how the scope might vary according to new hazard class designations. The development of RSMPs will be driven through further research and development and industry consultations so the final arrangements, terms and details brought into legislation and guidance are likely to differ from those presented in this paper.

INTERNATIONAL GUIDANCE ON RESERVOIR SAFETY MANAGEMENT SYSTEMS

There are several ICOLD bulletins which cover the key elements of good reservoir safety management. These include Bulletin 138 (ICOLD, 2009), Bulletin 154 (ICOLD, 2014), Bulletin 158 (ICOLD, 2019), Bulletin 168 (ICOLD, 2017) and Bulletin 180 (ICOLD, 2018). As part of the research, consultations were made with several leading UK and international dam operators to capture examples of the best reservoir safety management practices.

Some of the key points of guidance from the research relevant to the requirements of a sound reservoir safety management system are as follows:

1. Records should include all basic physical information for the reservoir.
2. Dam owners have a 'duty of memory' to fulfil, i.e. information on the original design and construction, studies, investigations, surveys, monitoring, remediation and improvement records, incidents and matters of concern should be carefully preserved and passed on to new owners when appropriate.
3. Effective safety management should include for performance assessment through visual inspections (surveillance), monitoring and the testing of safety-critical equipment such as gates and valves.
4. Monitoring and surveillance activities should be tailored to reflect failure modes and the history of the reservoir.
5. Monitoring equipment must be maintained through planned activities (i.e. pro-actively) as well as through corrective maintenance.
6. Surveillance should be carried out continuously and in a professional manner by trained personnel.
7. The scope and frequency of monitoring, surveillance and testing should be adaptable to the life cycle of a dam.
8. Special activities should be carried out following unusual events to verify performance.
9. Management systems should include an emergency action plan.

DEVELOPMENTS FOR A NEW REGULATORY DAM SAFETY MANAGEMENT SYSTEM

Defra and the EA are currently leading research into many of the recommendations of the Review including how reservoirs might be re-designated and how the regulatory controls for individual reservoirs might be introduced according to the hazard posed by each reservoir. A significant issue for the UK is that high hazard reservoirs are owned and operated by a very diverse group of reservoir owners ranging from water companies through to private individuals. Accordingly, there is considerable diversity in the knowledge, skills and financial resources that can be directed to the promotion of reservoir safety. All reservoir owners have a responsibility for the safety of the downstream communities and those communities have a right not to be disadvantaged on account of the financial and technical resources of the reservoir owner. Upholding this principle without placing intolerable requirements on reservoir owners poses one of the greatest challenges in developing new legislation and guidance.

The recent research by Mott MacDonald proposed a safety management system as shown in Figure 1.

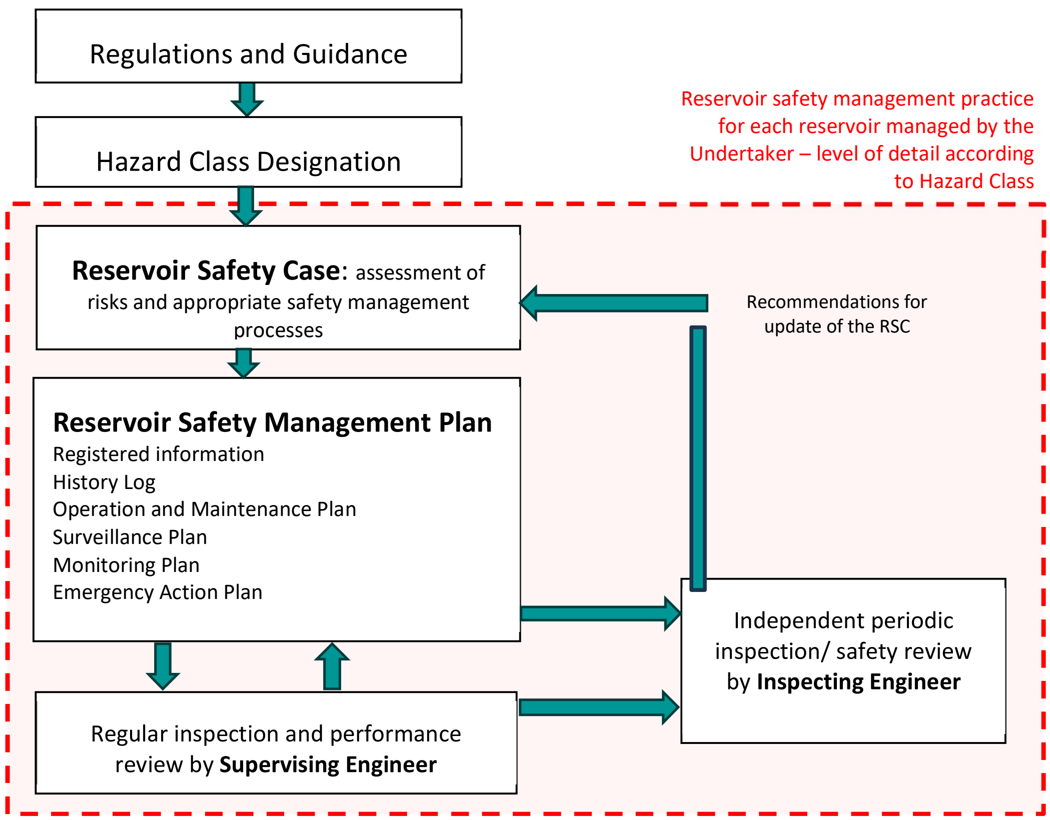

Figure 1. Outline of a new reservoir safety management system.

New regulations would define the hazard classes. All statutory reservoirs will need to be re-designated according to the new class definitions. To set out the minimum requirements for specific reservoirs it is proposed that there would be a 'reservoir safety case' (RSC). The scope of the RSC, how it would be completed and by whom has yet to be developed. Responsibility for preparing the RSC is likely to be with the Undertaker but clearly many Undertakers will require professional assistance in preparing it, as has been the case with Flood Plans. The RSC should evaluate the form and function of the reservoir structures and safety-related equipment (dams, tunnels, gated outlets etc) and consider the history of the reservoir and the modes of failure. It should aim to set out the minimum provisions for the reservoir in terms of surveillance, monitoring, competencies and training. It will therefore shape the reservoir safety management processes which are then developed further in the various parts of the RSMP. The minimum requirements for the RSC could reflect the reservoir hazard category. It would appear appropriate that the RSC should be reviewed periodically by the inspecting engineer who could make recommendations for amendments.

Managing Risks for Dams and Reservoirs

The RSMP would replace the current provisions set out in the Prescribed Form of Record and Flood Plan (emergency on-site plan). It is proposed that the RSMP should have six key elements which are described in the sections below.

The role of the Supervising Engineer and Construction/Inspecting Engineer is not proposed to substantially change in scope. These roles and responsibilities will need to be adjusted but fundamentally they would be very similar to the current provisions.

PRESCRIBED FORM OF RECORD

Currently the reservoir safety records comprise sixteen parts of the Prescribed Form of Record (PFR) and the Flood Plan. The research proposed that the PFR need not be retained in future legislation as all the information can be captured by the RSMP parts described below. The research also proposed renaming the 'Flood Plan' as an 'Emergency Action Plan' which better aligns with international terminology.

REGISTERED INFORMATION

A digital database should be made available by the enforcement authority to store basic information on the reservoir, typically including the information contained in Parts 5-8 of the PFR. Some information is already stored by the EA. It is essential that there is common agreement on key data pertaining to the reservoir and that the EA maintains an overview of national data on reservoirs.

The Review recommended that the public should have greater accessibility to information on reservoirs that may affect their safety. Accordingly, it is possible that the public would have access to some parts or all the digital database. The information available could potentially include details of whether the Undertaker is in receipt of any directions to make improvements in relation to their statutory obligations. This would help to make Undertakers more accountable to downstream communities and bring greater transparency to the workings of the legislation.

HISTORY LOG

The PFR provides for recording past statutory reports, unusual events, drawings available etc. Greater emphasis should be placed on the value of retaining and recording key information relating to the history of a reservoir in terms of its design, construction, performance, remediation and improvement. ICOLD Bulletin 138 (2009) refers to owners having a 'duty of memory'. Drawings, investigation reports, changes in equipment, incidents etc can be lost over time and information may not be fully transferred when the ownership of a reservoir changes. It is critical to the safety of reservoirs that all reasonable steps are taken to preserve information which can be used to effectively review failure modes, investigate problems, plan improvement works and deal with incidents and emergencies. Under current guidance Undertakers should furnish inspecting engineers a 'data pack' ahead of inspections and we envisage that the History Log would largely serve this purpose. We envisage that the RSMP will provide guidance on the registration and management of reservoir documentation without being too prescriptive on how the Undertaker manages information.

OPERATION AND MAINTENANCE PLAN

Reservoir structures vary greatly in terms of their operational and maintenance requirements. Maintaining design conditions and preserving operability are key factors in keeping reservoirs safe. We envisage that the Operation and Maintenance (O&M) plan should include:

- Details of the equipment and operating processes relating to reservoir safety functions. This would include details of permanently-installed equipment that is likely to be used in managing an emergency.
- Restrictions on reservoir operation (e.g. water level, inflow control etc) imposed during construction or in service by a panel engineer.
- Key maintenance activities and a log of actions taken. These activities could be set out in the RSC.
- Documentation of surveys and planned operations to verify condition. The RSC might require that certain activities are carried out by trained individuals of a certain level of competency, for example in the servicing of spillway gates.

SURVEILLANCE PLAN

Visual inspection of reservoir structures is a vital component of any reservoir safety management system. The current legislation does not ensure that surveillance activities are regular or sufficient. We propose that, at least for the higher hazard class(es), the regulatory controls for reservoir surveillance should be improved. ICOLD Bulletin 154 (2014) states that a lack of financial, management or engineering resources cannot be a justification for inaction in carrying out effective surveillance. Improvements can be brought through training Reservoir Technicians to carry out the surveillance role and for these individuals to be accountable for ensuring that visits are both sufficient and regular. Many of the larger reservoir owners already provide training for site staff which could be assessed against new guidelines to ensure that the burden on the industry is minimised. For many smaller reservoir owners, the need to nominate two or more individuals to complete periodic Reservoir Technician training and certification could represent a significant change. The industry would need to decide how reservoir surveillance training is developed, delivered and maintained. In parallel with this it is proposed that there should be a Reservoir Safety Manager course which would help to raise awareness of reservoir safety management and legal obligations. There are international precedents for requiring formal reservoir safety training of reservoir owners/operators, for example in Norway and Sweden.

The Surveillance Plan would set out the details and certification status of the Reservoir Technicians and provide for a register to record any key findings or events. Guidance on the scope and frequency of surveillance visits to the reservoir could be provided in the RSC.

MONITORING PLAN

Together with surveillance, monitoring is a critical activity for most reservoirs. The current legislation does provide for scoping the minimum requirements for taking instrument readings so we do not envisage that significant changes are needed. There are problems in determining compliance where instruments fail or are damaged or where the minimum frequency of readings is not achieved due to, for example, lack of access during bad weather, staff sickness etc. New guidance will be required in this respect.

The monitoring plan could include:

- A description of how the monitoring requirements link with failure modes and historical performance
- A description of the instruments required for monitoring performance together with the frequency and manner of recording information
- Any special supplementary short-term provisions required by the Supervising Engineer
- A description of where and how the data are stored.

MANAGEMENT OF SURVEILLANCE AND MONITORING INFORMATION

A decision will be required on whether information/data gained through surveillance and monitoring is held or referenced within the respective surveillance and monitoring plans described above or held in a separate part of the RSMP. Periodic reviews/summary reports of the data produced by the Undertaker and reviewed by the Supervising Engineer could then either remain in this part of the RSMP or be transferred into the History Log. With the advent of big data we recognise that a distinction must be made between provisions for the storage of large datasets and general reservoir records. The RSMP should set out the minimum requirements but should not be prescriptive in how or where data is stored and analysed.

EMERGENCY ACTION PLAN

There should be a requirement to maintain a Flood Plan or Emergency Action Plan (EAP). Currently, the Defra guidance requires that all relevant information be contained within the plan and updated/re-certified as required. Many elements of the plan are common with the requirements of other elements of the RSMP. For example, the reservoir owner's contact details would be in the registered information and details of how to open gates and valves would be in the O&M plan. We therefore propose that the EAP need only contain the information which is relevant to dealing with an emergency, for example the use of temporary pumps and details of how the plan should be tested. We recognise that there is value in being able to print off a document that contains all relevant information for dealing with an emergency. We believe guidance could be provided to achieve this without the need to duplicate information within the RSMP.

PERFORMANCE REVIEWS

It is important that the findings of surveillance visits, monitoring data, surveys, investigations etc are critically reviewed by the Undertaker on a regular basis to assess whether the reservoir structures are performing in an acceptable manner and whether any 'trigger level' criteria set out in the RSC or by panel engineers have been exceeded. The research considered whether performance reviews carried out by the Undertaker should be included as part of the RSMP. We consider that although this is critically important to effective reservoir safety management it is not necessarily a matter that needs to be formally captured within the scope of a RSMP. On a broader scale, larger reservoir owners typically carry out portfolio reviews. The role and responsibility of owners in carrying out performance reviews should however be covered in new guidance.

MONITORING COMPLIANCE

The Review recommended that the Supervising Engineer should review the RSMP annually and approve and certify it as being compliant with the requirements. The Review also places responsibility for the safety management of a reservoir first and foremost with the Undertaker. It could be considered an unfair burden on Supervising Engineers to certify that a RSMP is fully correct and complete. The Supervising Engineer only gains a 'snapshot' of reservoir performance typically once or twice each year through site visits, and the plan is prepared and maintained by the Undertaker. Therefore, we recommend that it should not be a requirement that the Supervising Engineer certifies the RSMP.

New legislation will likely require an Undertaker to prepare a RSMP. The contents of the plan should be steered through guidance according to hazard class. The legislation could provide powers for the Supervising Engineer, an Inspecting Engineer and possibly the enforcement authority to serve improvement notices where safety-critical changes are needed to the RSMP to comply with the guidance or to reflect changes at the site. Examples might include:

- The need to update contact details in the Registered Information
- Updating of the History Log with information on an incident
- Corrections to the information relating to a replacement telemetry system in the O&M plan
- Need for a Reservoir Technician to renew certification and record it in the Surveillance Plan
- Need to replace a water level gauge and update the Monitoring Plan
- Need to change the access provisions in the EAP for the installation of mobile pumps.

SUMMARY

This paper sets out the key findings of recent research to inform the scope of a RSMP. The research was informed by a review of international guidance on reservoir safety management and through discussion with several national and international dam operators on current best practice. Under any new reservoir safety legislation, the preparation and maintenance of the RSMP will be critical in driving good reservoir safety practices and promoting dam safety. The further development of proposals for new legislation and guidance will likely be steered through further research and industry consultations.

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Geomembranes in new pumped storage schemes

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SYNOPSIS Construction of pumped storage schemes is increasing to balance electricity networks and to maximise the energy coming from wind and solar sources. The reservoirs of such schemes must be lined with durable watertight facings to prevent water loss, ensure structural safety, and minimise maintenance, involving outage and heavy revenue losses. Geomembranes have been used in pumped storage schemes since the 1990s to restore watertightness of dams forming the reservoirs, and since the middle of the 2010s geomembrane were considered for new pumped storage reservoirs, substituting concrete or bituminous concrete facings. The advantages of geomembranes are numerous, the most important ones being the capability to resist settlement; differential displacement; joint openings; their maintenance-free durability and their repairability, also underwater. Several pumped storage scheme reservoirs have or will have a watertight flexible geomembrane facing, with different site-specific anchorage systems to maintain stability under varying hydraulic and wind loads. The paper presents design concepts, advantages, available systems, and related installation aspects of exposed flexible geomembranes through case histories recently completed or ongoing: Kokhav in Israel and Pinnapuram in India, with geomembranes anchored in trenches, and Abdelmoumen in Morocco, where a lacquered geomembrane was adopted to enhance durability in an environment with high UV radiation.

INTRODUCTION

The accelerating transition to clean energy produced an extraordinary worldwide increase in construction of new wind and solar plants. Wind and solar output, however, is not constant, and needs a storage system. Pumped storage schemes (PSSs) are at present the most dependable and mature storage technology to compensate for the intermittency of wind and solar energy. Consequently, pumped storage schemes are also dramatically increasing, either using and transforming existing powerplants and reservoirs, or creating entirely new schemes.

To provide the needed output to the grid almost instantaneously, a PSS must be kept safe and efficient, and an important role for safety and efficiency is played by the watertightness of its reservoirs: loss of water may initially entail only loss of profitability, but may also, in the long term, jeopardise the safety of the scheme if leakage continues. Since new reservoirs are mostly formed by earth or rock embankments and by excavation in semi-permeable soils, they require a facing system to ensure their watertightness. Due to the deformable nature of embankments and foundations, the facing system must be capable of resisting settlement, accommodating differential displacements where the embankments intercept concrete structures (intakes, spillways), and repeated loading and unloading cycles that can increase the potential for settlement and displacement, and aggressive environments. An additional issue is maintenance: facings that experience local or widespread failures due to the above stresses need frequent maintenance, and maintenance almost always implies outage. Due to the nature of these schemes frequent outage is unacceptable from a financial standpoint.

Geomembranes have been used for many decades to provide or restore imperviousness to dams and reservoirs, and the challenges posed by such structures have been successfully faced and solved. In particular for pumped storage, since the 1990s the application of geomembranes was related to restoring the watertightness of ageing dams forming one of the reservoirs. Only in the mid 2010s were geomembranes applied in new PSSs, and in 2016 what can be considered their first application in a PSS started operating: the 18 Water Saving Basins of the Third Set of Locks of the Panama Canal Expansion, lined with an exposed Carpi geomembrane liner to minimise water losses (Vale et al., 2018), with an average of 5 to 6 filling/emptying cycles per day, have to date undergone more than 15,000 filling/emptying cycles, equivalent to more than 40 years of operation of a PSS with 1 cycle per day.

There are at present 20 pumped storage schemes on which Carpi geomembrane systems have been installed, either as a waterproofing liner in new schemes, or as rehabilitation measures for leaking areas or joints, in the dry or underwater. This paper focuses on new PSSs, and in particular on the new reservoirs forming such schemes.

DESIGN CONCEPTS

The design of geomembrane systems for the reservoirs of new PSSs is based on previous experience in embankment dams and hydropower reservoirs, and on the additional issues to be considered for these plants: repeated loading/unloading, with the associated higher impact on slopes stability, cyclic daily exposure of the liner to UV and wind uplift, higher potential for fatigue phenomena, and the reversibility of the water current, which can amplify the formation of wrinkles and folds. Design parameters are the type of embankment and subgrade, and the loads acting on the geomembrane (varying water levels, wind). The type and thickness of the geomembrane, the drainage system, and the type and pattern of the anchorage system, are selected in function of such parameters.

A geomembrane system is by itself the most sustainable and resilient waterproofing solution, especially when compared to concrete and bituminous concrete liners: the small volume and weight of the components and of the equipment needed for installation make transport and site organisation easier, quicker and less cumbersome, resulting in a lower carbon footprint, and minimised environmental impact.

Geomembrane selection

All types of geomembranes, having permeability much lower than the permeability of traditional liners, are in principle very effective in providing watertightness. However, depending on the tensile properties and dimensional stability their behaviour in the field is very different. In the reservoirs of a PSS a geomembrane liner must be capable of resisting the action of an irregular subgrade under a varying water head, which can cause punctures and bursts; the stresses imparted by settlements in the subgrade; and by the differential displacements at boundaries (joints embankments/concrete structures).

Figure 1 compares the tension-elongation curves of a 3.0mm high density polyethylene (HDPE) geomembrane (in black) and of a Sibelon® CNT composite liner, formed by a 3.0 mm Sibelon® PVC geomembrane heat-bonded at fabrication to a 500g/m^2 non-woven geotextile (in red). For the HDPE, the curve is limited to the range of admissible strains in the field, i.e. below the yield point (at about 12% elongation), after which the behaviour of the geomembrane becomes plastic, the geomembrane thins down locally and elongates like gum, presenting a plastic elongation under essentially constant tension up to the elongation at break. For the PVC composite geomembrane, the curve is limited to the range below the breaking of the backing geotextile (at about 70% elongation), above which however the geomembrane keeps its functional integrity up to its elongation at break. The presence of a yield point may be crucial in hydraulic applications. According to ICOLD (ICOLD, 2010), "Mainly for HDPE, for a stress higher than the yield point, significant partially irreversible deformations (creep) occur after the stress has ceased." Therefore, HDPE geomembranes should be used only where the geomembrane elongation is well below the yield elongation, and with a substantial factor of safety. International literature (Seeger and Müller, 1996; Peggs et al, 2005) indicates that to be on the safe side the allowable elongation of HDPE geomembranes should not exceed 3% to 5%, and that for elongations greater than 3% the "creep" phenomenon is important and cannot be neglected. In the case of some textured HDPE geomembranes, even lower percentages should possibly be considered. Due to relatively poor dimensional stability HDPE geomembranes are also prone to the formation of high wrinkles and folds due to temperature variations. On the contrary, PVC composite geomembranes have no yield. They are characterized by a monotonically increasing tension-elongation diagram that has two peaks: the first peak corresponds to the breaking of the backing geotextile (Figure 1) and the second peak corresponds to the breaking of the geomembrane. Beyond the first peak, the material presents the characteristic behaviour of the geomembrane until failure. The presence of a backing geotextile further reduces the already low sensitivity to temperature variations and to the formation of folds.

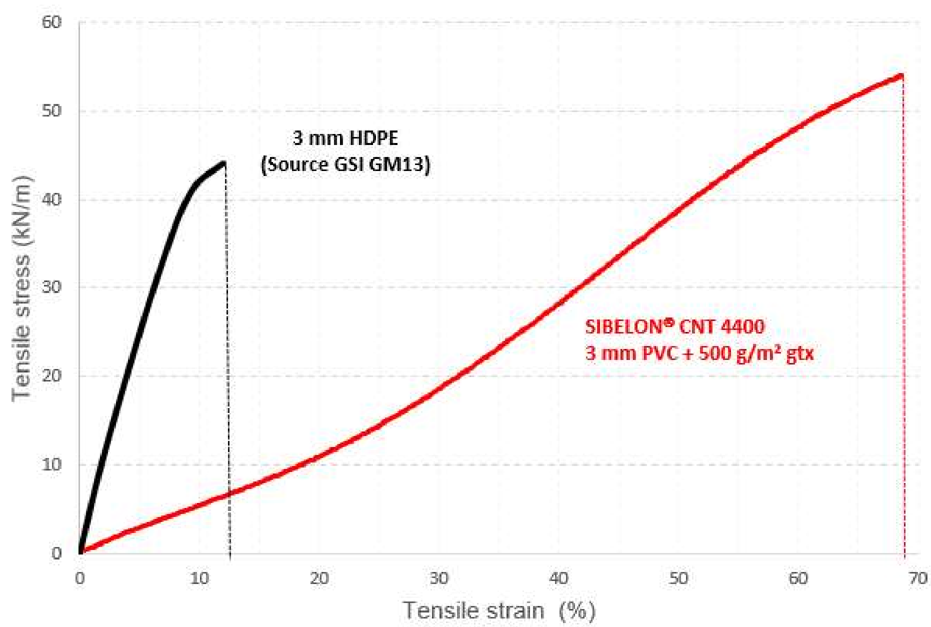

Figure 1. Tension-elongation curve of a 3.0 mm thick HDPE geomembrane (black) and of a PVC composite geomembrane (red)

Managing Risks for Dams and Reservoirs

The capability to withstand differential settlements at the junctions with concrete appurtenances can be ascertained following the Co-Energy concept developed by Giroud (Giroud, 2005). The Co-Energy is the area between the geomembrane tension–strain curve and the tension axis. A geomembrane can withstand a differential settlement when the required geomembrane Co-Energy, related to the stress-strain condition in the field, is lower than the allowable geomembrane Co-Energy. According to Giroud, geomembranes with greater Co-Energy can tolerate larger differential settlements.

The graph (Figure 2) comparing the Co-Energy of an HDPE geomembrane up to the yield point and of the composite geomembrane already considered up to the breaking of the geotextile, clearly shows that the maximum allowable Co-Energy associated with the PVC composite geomembrane (the grey area) is significantly greater than the Co-Energy associated with the HDPE geomembrane (the red area). As a result, the factor of safety with respect to a potential differential settlement is significantly higher for the PVC composite geomembrane than for an HDPE geomembrane of the same thickness. A different geomembrane thickness would not produce a different result.

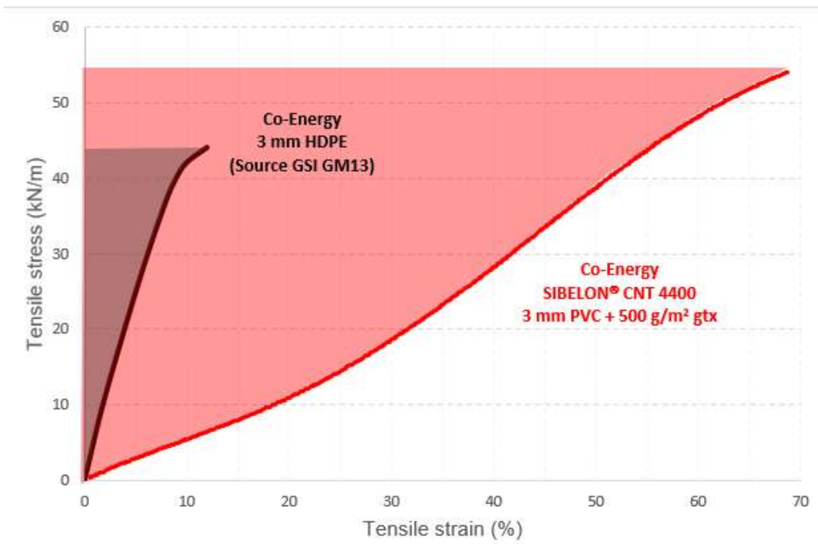

Figure 2. Co-Energy: comparison between the area comprised within the tension-strain curve and the tension axis of a 3.0 mm HDPE geomembrane and of a composite geomembrane (3.0 mm thick PVC geomembrane heat-bonded at fabrication to a 500 g/m^2 non-woven geotextile).

The thickness of the PVC geomembrane and the weight of the associated geotextile, which enhances the puncture resistance and the thermal stability, are selected based on the layers that will be in contact with the liner (subgrade, cover layer if any), on environmental aggression (basically UVs and temperature variations), and on required life span.

Face anchorage system

The geomembrane is typically left exposed, and a face anchorage system keeps it stable against varying water levels and wind uplift. Anchorage at points is rarely adopted; unless loads are not very demanding and the anchors are very closely spaced, the stresses on the geomembrane at each anchor will be unacceptable (ICOLD, 2010). Face anchorage is made by lines, to maintain the geomembrane as tense and adherent as possible to the support layer, to avoid folds that can form during daily variations in water levels, as these folds are areas of stress concentration, potentially leading to more rapid ageing and fatigue phenomena in the geomembrane. The anchor lines can be longitudinal (at crest, berms, bottom) and/or vertical (at slopes, bottom), with spacing calculated with well-known methods (Giroud et al., 1995) depending on the effects of wind suction/variations of the water body. Face anchorage by ballast is possible, but it reduces the volume of impounded water and is generally restricted to areas with heavy traffic (access ramps, intake areas).

Face anchorage is designed in accordance with the type of embankment. In embankments constructed with soil (earthfill), characterised by mild slopes (e.g., 3H:1V), with a bedding layer for the geomembrane liner made of compacted cohesive material, face anchorage typically consists of geomembrane anchor bands embedded in trenches excavated in the bedding layer, to which the geomembrane liner is secured by heat-seams. A drainage layer must be provided under the geomembrane liner, which can be a granular material or a synthetic geodrain; the drainage layer is connected at the slope toe to the drainage network of the reservoir bottom. In embankments constructed with small rockfill (gravel, stones), characterised by relatively steep slopes (e.g., 2H:1V), with a bedding layer for the geomembrane liner made of compacted and selected gravel with sufficient drainage capacity to also act as drainage layer, face anchorage is made by heat-seaming the geomembrane liner to geomembrane anchor bands embedded in trenches, or embedded in the embankment as it is being raised. In embankments constructed with large rockfill (stones, cobbles, boulders), characterised by steep slopes (typically 1.6H:1V), the bedding/drainage layer for the geomembrane liner is made of porous concrete either in the form of a relatively thin layer (200mm-300 mm) or in the form of extruded kerbs (constructed with an extruding machine). Face anchorage is made by heat-seaming the geomembrane liner to geomembrane anchor bands embedded in trenches in the porous concrete, or to geomembrane anchor strips secured to the extruded kerbs; this method can increase void space in the reservoir which in turn has both short and long-term advantages to the overall project.

Perimeter seals

The geomembrane liner is sealed at all peripheries by a watertight perimeter seal. When made on concrete, the seal consists of stainless-steel flat profiles that achieve watertightness by compressing the liner onto the concrete with the aid of regularising resin, rubber gaskets and stainless-steel splice plates. Seals of the embedded type (embedment in top trenches, in slots) are adopted at deformable areas.

Designing for optimised advantages

The main advantages of geomembrane systems have already been outlined. Further optimisation is possible if the design is made in cooperation with the designers of the reservoirs. The face anchorage system and the drainage layers can be adapted to the design and the materials used for the embankments, possibly reducing the amount of granular materials/increasing the steepness of the slopes. Perimeter seals can be designed with an abundance of geomembrane liner, to resist large displacements, so avoiding stressing the geomembrane.

The geomembrane system can be conceived so that it can be constructed in sequence, organising the various tasks to meet faster construction schedules, achieving waterproofing at a much faster rate than would be achieved by using a concrete or asphalt concrete facing. Faster completion will allow earlier power generation.

Pumped storage schemes are structures where minimising outage is critical and selecting a liner that does not need routine maintenance can make a substantial difference for operation costs during the service life of the scheme. The geomembrane system can be designed for enhanced service life and minimum maintenance, and at its end, it can be recycled.

CASE HISTORIES

Earthfill embankments: Kokhav Hayarden reservoirs, Israel 2020/2021

At Kokhav Hayarden, which with an installed capacity of 344MW will be the largest pumped storage project in Israel, the upper reservoir was partly formed by a compacted earthfill embankment, and partly excavated in river deposits of clay and a clay-gravel mixture. The inclination of the slopes is 3.5H:1V and the water fluctuation is around 22m. The lower reservoir is formed by a continuous compacted earthfill embankment, composed of river deposits of clay and a mixture of silty clay and gravel. The inclination of the slopes is 3H:1V, the water fluctuation is around 21 m.

Both reservoirs were originally designed with an HDPE geomembrane facing, placed over a drainage layer of compacted granular material, anchored at the crest by a longitudinal trench and at the bottom by the ballasting action of the water that would always be standing in both reservoirs. This design was deemed susceptible of improvement considering that in a PSS the geomembrane remains exposed for longer times to the potential adverse effects of wind uplift and wrinkle formation due to temperature variations. After a detailed review of the project, to provide higher performance, also in respect to possible large settlements of the subgrade, alternatives of different waterproofing liners were evaluated and a safer and more durable design configuration was approved for construction. The HDPE geomembrane was substituted for a Sibelon® CNT 3100 composite geomembrane (a 2.0mm PVC geomembrane heat-bonded to a 500g/m^2 non-woven polypropylene geotextile), which is more flexible and deformable, and less prone to formation of wrinkles and folds. The face anchorage system was made more robust by vertical anchorage trenches excavated in the embankments, where anchor bands of SIBELON® CNT 2300 composite geomembrane (1.5mm thick PVC geomembrane, heat-bonded to a 350g/m^2 non-woven polypropylene geotextile) were embedded and ballasted with compacted granular material, thus providing stable anchor lines for the waterproofing liner. Based on a maximum design wind velocity of about 150km/h (upper reservoir), an 8m trench spacing for the upper 1/3 of the slope and 16m spacing for the lower 2/3 of the slope were adopted for both reservoirs. The trenches in the upper 1/3 of the slope are larger due to the stronger effect of wind uplift towards the crest.

A synthetic drainage layer was installed over the compacted support on the slopes: it consists of a composite material providing drainage (its geonet component) and filtering (its geotextile component) functions. At the bottom, the drainage layer consists of selected gravel. The drained water is discharged through a network of perforated pipes.

At submersible concrete peripheries, the waterproofing geocomposite is watertight, sealed by a stainless-steel mechanical seal, 80x8mm. To further increase safety in respect of differential displacements, the seal is designed to limit stresses in the waterproofing liner by providing an extra length of geomembrane accommodated in a "settlement slot" that acts as a hinge between the embankment and the concrete structure.

Figure 3. Preparation of subgrade and anchor trenches at Kokhav Hayarden

Figure 4. Components of the waterproofing system

Installation of the geomembrane system in the lower reservoir started on 21 July 2021 and was completed on 29 September 2022. In the upper reservoir it started on 20 July 2020, and was completed on 17 December 2022. In total, 433,000m^2 of geomembrane was installed.

Figure 5. Kokhav Hayarden upper reservoirs' liner being completed

Figure 6. Kokhav Hayarden lower reservoir impounding

Impounding tests are ongoing. Data available at present for the lower reservoir show piezometer pore pressure curves with negative values and lower than the alert values, and for leakage values of 0 l/s in three compartments, and between 0.083 and 0.0183 l/s in the other four compartments. Only at the three compartments adjacent to the intake had the pressure increased, and leakage values were higher, the maximum being of 0.616 l/s. An underwater inspection ascertained that the increase in pressure was due to a leak coming from a local defect (puncturing) of the geomembrane in coontact of a vertical joint of the intake. The defect, found and temporarily repaired underwater, will be permanently repaired when the impounding tests are completed.

Small rockfill embankments: Abdelmoumen reservoirs, Morocco 2021/2023

Abdelmoumen, a PSS on the River Issen in Morocco, will have a 350MW installed capacity, exploiting a 500m water head. Both reservoirs are formed by embankments made of compacted granular material, with a slope inclination 2H:1V and a water level fluctuation of about 20 m. In addition to the usual challenges of pumped storage schemes, Abdelmoumen had two specific aspects: the geographic location required a waterproofing geomembrane capable of resisting particularly intense UV radiation, and the poor subgrade material available for the rather steep embankments could result in significant settlement and/or slope stability issues under the effect of repeated hydraulic loading. Therefore, the slopes had to be properly compacted, and the geomembrane had to be placed in a way that would not affect the slope stability and vice versa – the waterproofing system should not be affected by the settlement of the subgrade.

The selected geomembrane, Sibelon® CNT 4400 L, consists of a 3.0mm PVC geomembrane heat-bonded to a 500g/m^2 non-woven polypropylene geotextile, having a surface lacquered treatment (L) designed to increase the service life of the liner. Accelerated and specific weathering tests have shown that this treatment enhances the life of the material under intense UV radiation with respect to a non-treated liner of the same thickness, while keeping the remarkable flexibility and excellent tensile response of such PVC liners. To assess the effect of cyclic loading on the geocomposite and on a heat-seam, a real-scale testing campaign was carried out in the laboratory, simulating the conditions at Abdelmoumen. The simulation parameters included the Abdelmoumen subgrade material, the Sibelon® CNT 4400 L geomembrane liner, water pressure corresponding to the expected daily fluctuations, and number of fill/empty cycles compatible with the expected usage of the plant: with water pressure fluctuating between 0 and 2 bar during each cycle, we performed cyclic loading at the rate of 144 cycles per day for 127 days. This corresponds to 25 years in operation assuming two fill/empty cycles per day, or 50 years in operation assuming one fill/empty cycle per day. The tested sample and seam were then evaluated and compared with a virgin sample of the same material and seam by an independent laboratory in Germany, for thickness reduction, tension/elongation, and seam resistance to shear and peel. The test results demonstrated that the selected geomembrane liner can withstand cyclic loading with no quantifiable evidence of fatigue, showing no sign of damage or loss of watertightness, and variation of the mechanical properties less than 3% when compared to the ex-work samples.

The anchorage system at Abdelmoumen was based on the concept of avoiding constructing trenches, and of embedding instead rectangular geomembrane anchor strips into the embankments during construction. A specific procedure was developed to embed the geomembrane anchor strips while achieving a good compaction of the subgrade material. The result is a stable slope of compacted material, and effective robust confinement of the anchor strips. A continuous band of geomembrane was heat-seamed unto the strips to create a continuous vertical anchorage line, which distributes the stresses at placement and during operation. The design wind velocity was about 90km/h for both reservoirs, with a safety factor of at least 2.0. The resulting spacing of the vertical anchorage lines on the slopes is 8.0m, measured along the crest. Considering the possibility of a complete emptying of the reservoirs, a specific anchorage system is provided in the bottom, consisting of longitudinal trenches backfilled with concrete at 16m spacing.

Compaction of the embankments and installation of the geomembrane anchor strips started in April 2021 at the lower reservoir and in May 2022 at the upper reservoir. The geomembrane systems were completed in August 2022 at the lower reservoir and in January 2023 at the upper reservoir. Additional anchorage lines consisting of ballasting prefabricated concrete blocks were placed at some top locations of the upper reservoir where wind gusts stronger than expected were experienced (Figure 9). In total, about 195,500m^2 of exposed geomembrane liner installed. Both reservoirs are now impounding (Figure 10).

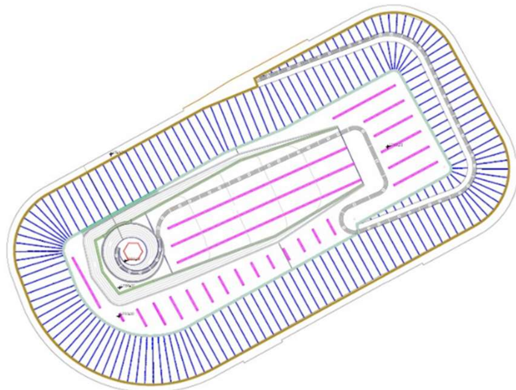

Figure 7. Abdelmoumen: layout of anchor trenches at upper reservoir

Figure 8. Abdelmoumen: anchor strips and anchor bands

Figure 9. Some additional ballast at Abdelmoumen upper reservoir

Figure 10. Upper and lower Abdelmoumen reservoirs impounding

Rockfill embankments: Pinnapuram, India, ongoing

Pinnapuram reservoirs, under construction in India, are part of the Pinnapuram Integrated Renewable Energy pumped storage project, which includes 1000 MW solar, 550 MW wind and 1680 MW of standalone pumped storage capacities. The scheme is a first-of-its-kind pumped storage project developed by Greenko, an independent power producer in India, whose second PSS, Gandhi Sagar, is starting now. The upper reservoir features a 6.5km long rockfill dam, forming a continuous embankment with a nearly rectangular shape in plan, and maximum height of about 40m. The lower reservoir is formed by three separate rockfill dams connecting existing natural slopes, with a total crest length of about 3.3km and maximum height of about 46m.

For Pinnapuram, avoiding/minimising water losses through the reservoirs is fundamental to the project: both reservoirs are far from existing natural river systems and have no/negligible catchment area, so water will be lifted once from the Gorakallu Reservoir irrigation system to fill them and to be used cyclically for energy production. Evaporation losses, if any, will be recouped periodically. A bituminous concrete facing was originally planned to grant watertightness to the dams forming both reservoirs. After further technical and economic assessment, the bituminous concrete was eventually replaced by an exposed geomembrane facing. The geomembrane liner selected for all dams is Sibelon® CNT 4400, the same material used at Abdelmoumen, but without surface treatment.

Managing Risks for Dams and Reservoirs

The layering at the dams, proceeding from the transition layer of rock fill material 3A-US/2B towards the upstream, consists of a concrete layer, needed for the Optical Fibre Cable system required by the owner to allow leak location, followed by a thin layer of porous concrete that constitutes the bedding layer for the geomembrane. The face anchorage system is made by geomembrane anchor bands embedded in trenches created by discontinuing the concrete and porous concrete layers, and then ballasted with concrete and porous concrete (Figures 11 and 12). The regular spacing between the trenches, calculated as usual based on the wind force, the resulting uplift, the water level cycles, is 16m. The porous concrete bedding layer has sufficient drainage capability to act also as full-face drainage layer under the waterproofing liner.

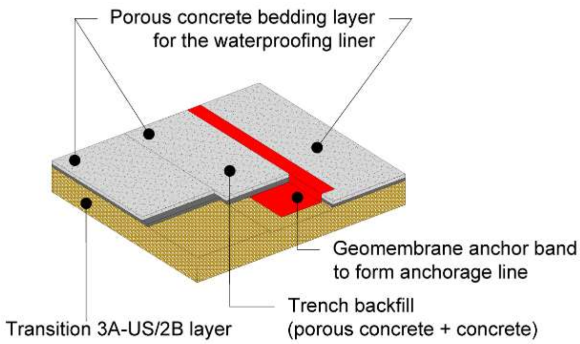

Figure 11. Scheme of anchor trenches at Pinnapuram lower dams

Figure 12. Geomembrane anchor bands embedded in trenches

The geomembrane system was installed in a sequential manner, following the deployment of civil works: earth works for the construction of the embankments, placement of the concrete and of the porous concrete bedding layer for the geomembrane system, concurrent with creation of the anchor trenches, installation and ballast of the geomembrane anchor bands in the trenches, deployment of the waterproofing liner and subsequent fastening to the anchor bands by heat-seaming (Figure 13). At the upper reservoir, to expedite works at the more than 6km long dam, the porous concrete bedding/drainage layer was substituted with a synthetic drainage layer (Figure 14), like the one adopted at Kokhav Hayarden.

Figure 13. Installation of geomembrane on porous concrete at lower dams

Figure 14. Installation of geomembrane on synthetic drain at upper dam

At present, the geomembrane is being installed at the dam forming the upper reservoir (Figure 15) while installation has been completed at the lower reservoir; waterproofing works are ongoing at the tailrace channel, where an exposed geomembrane replaced the bituminous concrete liner originally planned.

Figure 15. Pinnapuram pumped storage scheme: upper reservoir

CONCLUSIONS

Typically, the waterproof facing works represent less than 2% of the cost of a pumped storage scheme, but their efficiency is essential to make the investment profitable. Geomembrane liners propvide safety and efficiency of pumped storage schemes by preventing water loss through the two reservoirs, and by resisting settlement and differential movement. Reducing construction time and costs, and allowing earlier generation, are the assets of this sustainable technology.

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The British Dam Society

St Blazey Flood Storage Reservoir: A Case Study on the Importance of a Holistic Approach to Reservoir Risk Assessment

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SYNOPSIS The St Blazey Flood Storage Reservoir was situated to the north of the town of St Blazey, Cornwall and was impounded by Highway Dam, which crossed the Treffry Canal. The right side of the reservoir was also contained by a sandy railway embankment supported by a masonry wall. Following a Section 10 inspection and failure of a section of the masonry wall retaining the railway embankment, an Inspecting Engineer recommended that the risk posed by the presence of the reservoir be assessed.

AtkinsRéalis undertook a qualitative risk assessment considering the existing arrangement and options to upgrade or discontinue the reservoir, and also performed a high-level strategic review to enable the owner (the Environment Agency) to assess the best solution for the local population. As a result of the assessment, the project team determined that the reservoir presented an unacceptable risk to life and should be discontinued through the removal of Highway Dam.

This paper discusses the methodology used to determine discontinuance as the preferred solution, focusing on how a holistic view on risk versus benefit was adopted, supported by flood modelling to quantitively assess the benefits provided by the reservoir. Furthermore, the paper discusses how consideration of the societal benefit created by the presence of a reservoir is critical in assessing the tolerability of the risk to life, rather than limiting consideration to the likelihood and consequences of failure alone.

BACKGROUND

The St Blazey Flood Storage Reservoir (FSR) was located to the north of the town of St Blazey, Cornwall. The reservoir formed, with Treesmill FSR, part of the Par flood relief scheme, which was constructed in 1976. It was owned and operated by the Environment Agency (EA).

The reservoir was formed by the Highway Dam, located across the line of the Treffry Canal (Figure 1). The reservoir was also retained by a single-track railway embankment on its west flank, carrying the Atlantic Line from Par to Newquay. This embankment was reportedly made of “pure sand” and was not designed to retain the reservoir. The Par River (northwest) side of the railway embankment is supported by a masonry wall.

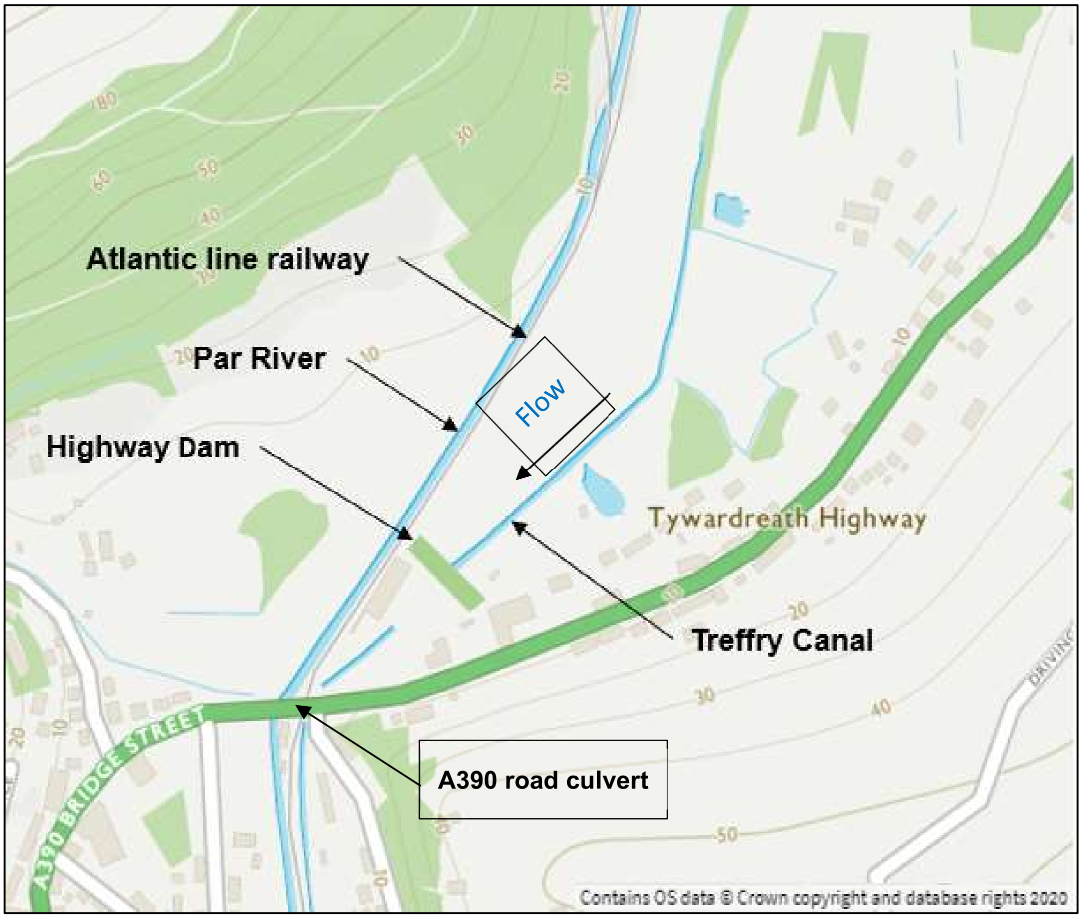

Figure 1. Site plan of St Blazey FSR

The Highway Dam retained water 1.9m above the flood plain level, providing a reservoir capacity of approximately 155,000m^3. The reservoir primarily provided protection in the lower return period events, with the spillway crest (at ~7.9mAOD) starting to operate for flood events between the 50% and 10% Annual Exceedance Probability (AEP) and the dam crest (at ~8.5mAOD) overflowing for floods greater than the 10% AEP event.

The lowest railway embankment level adjacent to the reservoir was 8.7mAOD resulting in the railway embankment overflowing for all flood events greater than the 5% AEP event. The extent of the railway embankment overflowing increased with larger (more infrequent) flood events, as the peak flood level in the reservoir increased.

CONTEXT

An inspection of the FSR, under Section 10 of the Reservoirs Act 1975, was requested by the undertaker due to concerns about the construction materials used in the railway embankment following some repairs to a redundant section of embankment upstream at Ponts Vale. The inspection was undertaken in August 2019. Following heavy rain in October 2019, a section of the masonry wall retaining the railway embankment, around 95m upstream of the dam, failed. The Section 10 inspection report stated that this was reported as a reservoir safety incident to the Enforcement Authority as the railway embankment retains the reservoir during impounding events.

Following the inspection and the reservoir safety incident, the Inspecting Engineer made a recommendation as to Measures to be Taken in the Interests of Safety (MIOS) to review whether the risk posed by the presence of the reservoir was tolerable as defined by the *Guide to Risk Assessment for Reservoir Safety Management* (RARS) (EA, 2013).

To address this recommendation, a Tier 1 reservoir risk assessment was carried out, in accordance with the guidance provided in RARS. Three potential options, established through an options study, were considered: retaining the existing arrangement; improving the reservoir by building a line of sheet piles between the reservoir and the railway embankment; and discontinuing the reservoir.

In parallel with the risk assessment, the project team carried out hydraulic modelling of the options to support the Environment Agency's Strategic Outline Case (SOC), required to obtain funding for future project stages. This hydraulic modelling, which considered other proposed flood risk improvement works in the catchment, provided a quantitative assessment of the operational flood risk benefits of the reservoir.

This paper discusses the importance of considering any changes to flood risk management in the catchment when assessing the benefits provided by a flood storage reservoir and of considering those benefits when assessing the tolerability of the societal risk posed by the reservoir.

RISK ASSESSMENT

Existing arrangement

A Tier 1 risk assessment, as outlined in RARS (EA, 2013), was carried out to evaluate the societal risk associated with the uncontrolled release of the reservoir contents, caused by failure of the railway embankment.

The likelihood of failure of the railway embankment, due to crest overflowing and downstream face instability was assessed as **Extreme** because the embankment had no spillway and the masonry wall had been reported to be in "poor condition" during a 2019 structural survey. The potential magnitude of the consequences, considering the human, economic, environmental and cultural receptors within the inundated area, was designated as **Level 3** because the number of residential properties affected would be more than 30 and less than 300 (assessed considering EA reservoir flood mapping).

RARS provides a methodology for qualitative assessment of the level of risk, by plotting the likelihood of failure of the railway embankment with the magnitude of potential consequences on a simple risk matrix (Figure 2). This indicates that the initial level of societal risk associated with a reservoir breach due to failure of the railway embankment was **Unacceptable**.

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

Figure 2. Simple Tier 1 risk matrix (adapted from RARS (EA, 2013))

An Unacceptable rating means that "the risks are generally believed by individuals and society to be not worth taking, regardless of the benefits" (RARS). Therefore, the reservoir could only be used as a flood storage reservoir if its condition was improved.

Alternative arrangements

As discussed above, the societal risk associated with retaining the reservoir in its existing arrangement and condition was Unacceptable. Two viable alternative arrangements were established to address the risks associated with use of the railway embankment to retain the reservoir:

1. Full discontinuance of the reservoir through removal of the Highway Dam so that it no longer caused water to impound upstream.
2. Retaining and upgrading the reservoir by installing a sheet pile wall along the western side of the reservoir so that the railway embankment no longer formed part of the reservoir impounding structure. This piling could not withstand overflowing.

The societal risks associated with the two alternative arrangements are discussed in more detail below.

The discontinuance option would remove the risk of an uncontrolled release of water from the reservoir, as there would no longer be a reservoir following the removal of the Highway Dam.

Retaining the reservoir would intrinsically retain the risk of an uncontrolled release of water, which could endanger life. However, upgrading the reservoir by installing a sheet pile wall along the western side would reduce this risk from Unacceptable to Tolerable. The level of risk was assessed on the following basis:

- The installation of a sheet pile wall between the reservoir and the railway embankment would make the sheet pile wall part of the reservoir retaining structure, rather than the railway embankment. The new sheet pile wall, designed to retain water, would have a Very Low likelihood of failure.

- The Highway Dam was a clay core embankment with a low hydraulic gradient and with a foundation cut-off. The dam was generally maintained to a good standard, the surveillance was adequate and there were no signs of adverse behaviour. In flood events where the dam overflowed; overflow velocities were assessed to be low and not likely to cause erosion of the downstream slope. The Highway Dam was therefore judged to have a Low likelihood of failure.
- The area inundated in the event of a breach would not change (the flood extents for breach of the railway embankment and Highway Dam are very similar), so the consequence designation would remain at Level 3, as above.
- Referring to Figure 2, a Low or Very Low likelihood of dam failure, combined with a consequence designation of Level 3, results in a societal risk associated with failure of the reservoir that is **Tolerable**.

The outcomes of the qualitative risk assessment of the two alternative arrangements are summarised in Table 1 below.

Table 1. Summary of risks associated with alternative arrangements

Risks	**Discontinue the reservoir**	**Retain and upgrade the reservoir**
Societal risk associated with breach of the railway embankment/ western side of the reservoir	No risk associated with the reservoir, as reservoir removed	Tolerable
Societal risk associated with breach of the Highway Dam	None as dam removed	Tolerable

Risk assessment findings

The Tier 1 risk assessment highlighted that the societal risk associated with retaining the reservoir, in its existing arrangement and condition, was unacceptable. An assessment of alternative arrangements concluded that installation of a sheet pile wall between the reservoir and the railway embankment would reduce the societal risk associated with breach of the reservoir to a Tolerable level and that discontinuance of the reservoir would remove the societal risk posed by the reservoir.

ASSESSMENT OF SOCIETAL BENEFIT

The assessment of the benefits provided by the reservoir was carried out as part of a business case produced to demonstrate that any works recommended would represent a good use of public money. One element of the business case is the Economic Case, for which the cost/ benefit ratio of any options under consideration are presented. For flood risk projects, such as the St Blazey FSR, the benefits are assessed by creating a hydraulic model of the potential options and simulating a range of storm events to understand and compare the expected flood extents and depths at receptors within the catchment for each scenario.

Catchment context

The St Blazey FSR was integrated within a complex hydraulic system which includes the Treffry Canal passing through the reservoir site, the Tywardreath and Treesmill Streams passing through the smaller adjacent Treesmill Reservoir, and the Par River passing to the west of the St Blazey FSR and then through the town of St Blazey.

Managing Risks for Dams and Reservoirs

Simultaneous to the St Blazey FSR project, the detailed design of flood risk management works in the town of St Blazey, downstream of the reservoir, was being carried out. The works formed part of the St Austell Bay Resilient Regeneration (StARR) scheme, which was developed to address flood risk in the area, as it was one of the major influencing factors preventing regeneration to the deprived communities of Par and St Blazey. The key components of the works included defence improvements and culvert replacement along the Par River, floodplain reconnection and surface water management. The EA led on the delivery of main river interventions, whilst Cornwall Council led on the surface water and ordinary watercourse interventions with support from key delivery partners.

Recognising the complex hydraulic connectivity in the catchment, and therefore the potential for the StARR scheme works to impact on the outcomes of the hydraulic modelling for the St Blazey FSR options, the project team decided to consider the StARR scheme works in all modelled scenarios.

Modelled scenarios

Hydraulic modelling was undertaken for flood events from 50% AEP (annual exceedance probability) to 0.5% AEP. Three scenarios were considered in the hydraulic model:

- The existing arrangement, before the implementation of any works. Although this option could not be taken forward (due to the unacceptable societal risk associated with the existing arrangement), it formed the baseline against which the other options were compared.
- Retaining and upgrading the reservoir, through the addition of a sheet pile wall between the reservoir and the railway embankment. This was represented as a glass wall in the model (on the basis that the top of sheet pile level would be set to prevent overflowing in all design events).
- Discontinuance of the reservoir. This was represented by modifying the ground profile within the dam footprint so that the ground levels aligned with those upstream and downstream.

Results of the hydraulic modelling

The total number of properties modelled to experience internal flooding during each event is presented in Table 2, allowing the flood impacts associated with the two feasible arrangements to be compared. The flood impacts of the existing arrangement are not included as it was not a feasible option.

Table 2. Number of properties modelled to experience internal flooding

Event	50% AEP		10% AEP		5% AEP		2% AEP		1% AEP		0.5% AEP	
Property type	Res	Non-res	Res	Non-res	Res	Non-res	Res	Non-res	Res	Non-res	Res	Non-res
Discontinue	2	10	36	37	73	59	126	88	186	124	379	203
Retain and upgrade	3	7	43	46	88	67	135	91	197	126	387	207
Difference	+1	-3[1]	+7	+9	+15	+8	+9	+3	+11	+2	+8	+4

[1] Garage buildings located immediately downstream of the Highway Dam, for whom individual mitigation measures were implemented.

The results show that the number of properties modelled to experience internal flooding was greater with the reservoir retained and upgraded, compared with the discontinuance option. A key reason for this is that with the Highway Dam removed (reservoir discontinued), the shape of the downstream hydrograph is altered, with more water passing downstream outside the peak of the flood, reducing peak water levels. This option therefore better utilises the increased capacity in the Treffry Canal downstream of the reservoir, provided by the StARR scheme. This results in reduced bank overtopping downstream or reduced maximum flood levels in the area upstream of the A390 road, depending on the flood event.

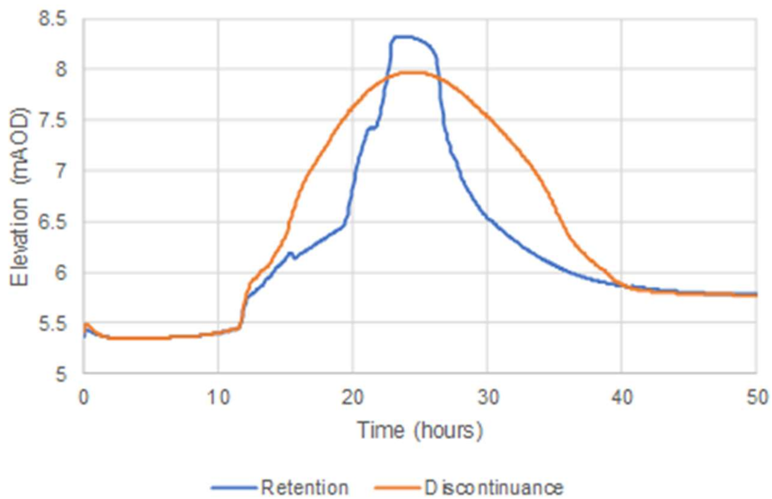

Figure 3. 10% AEP event modelled stage hydrographs just upstream of the A390 road culvert

DISCUSSION

The societal risk associated with breach of the retained and upgraded St Blazey FSR was assessed considering the likelihood and consequences of failure of the Highway Dam and found to be Tolerable. A Tolerable level of risk is defined as "individuals and society are willing to live with the risks so as to secure certain benefits". Therefore, considering only the outcome of the Tier 1 risk assessment, the preferred option would have been to retain the reservoir, on the basis that it was (assumed to be) providing flood risk benefits to the downstream communities in Par and St Blazey.

However, the hydraulic modelling demonstrated that with the StARR scheme works in place, the reservoir provided less flood risk benefits than if it was discontinued. As the presence of any raised reservoir upstream of a populated area creates a risk of loss of life associated with the potential failure of the dam and release of the impounded water, the presence of a reservoir cannot be justified (i.e. the risk cannot be considered tolerable), if the reservoir does not provide any benefits. Therefore, the outcome of the reservoir risk assessment, when considering not only the societal risks posed by the reservoir but also the benefits provided by each arrangement, was that the reservoir should be discontinued.

It is important to note that when sensitivity testing was carried out to model the options without the StARR scheme in place, a greater number of properties were shown to experience internal flooding in the discontinuance option than for the retain and upgrade option. In this scenario, the assumption that the reservoir was providing flood risk benefit and thus the original finding of the risk assessment, that the reservoir should be retained and upgraded, would have been valid. The sensitivity test, therefore, highlighted the importance of taking a holistic approach to catchment flood risk management, considering the impacts that schemes can have on the efficacy of other measures implemented in the same catchment. The test also highlighted the need for aligned project delivery; if the StARR scheme works were not implemented before discontinuance of the St Blazey FSR, the populations of Par and St Blazey would have experienced increased flood risk in the short term.

By taking a holistic approach to the assessment of the societal risks and benefits associated with the St Blazey FSR, the project team was able to bring about increased flood risk benefits over and above the StARR scheme works, whilst removing the public safety risks and the Undertaker's legal obligations associated with the statutory reservoir. The scheme also enabled the culverted section of the Treffry Canal passing through the Highway Dam to be returned to an open channel and environmental enhancements in line with the Water Framework Directive requirements to be carried out within the dam footprint.

Figure 4. Images of the completed St Blazey FSR discontinuance

CONCLUSIONS

Reservoir risk assessments on existing reservoirs are often carried out only considering the likelihood and consequences of failure, based on the (reasonable) assumption that if the reservoir was built it must be providing some societal benefit. Therefore, the risk assessments seek to determine whether the risk associated with the presence of a reservoir is tolerable, or if works need to be done to ensure that the societal risks are as low as reasonably practicable but do not tend to question whether the presence of the reservoir is justified.

The case study presented in this paper has demonstrated the importance of taking a more holistic approach to reservoir risk assessment, ensuring that it does not simply become an exercise of following a methodology to achieve a risk rating. This is of particular importance for flood storage reservoirs, for which consideration of the benefits provided by the reservoir as part of the assessment of the tolerability of risk to life may prove vital in achieving the optimal outcome, particularly if the other flood risk management measures implemented in the catchment have changed since the reservoir was constructed or last assessed.

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The British Dam Society

Buckshole Reservoir: Use of Physical Modelling to Optimise a Risk-based Solution

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SYNOPSIS A risk-based assessment has been undertaken to determine proportionate reservoir safety improvement works at the 11m-high embankment dam retaining the Category A Buckshole Reservoir in Hastings, East Sussex. The study addressed a recommendation made in the interests of safety following a statutory Section 10 inspection in June 2016 that related to the service spillway channel not being of sufficient capacity to accommodate the design flood.

In addition to providing a brief summary of the risk-based approach, this paper will focus on a laboratory-based hydraulic physical model study which was commissioned to inform the detailed design of the proposed new 4m-wide, 90m-long spillway channel and stilling basin. The model study helped to overcome the hydraulic challenges posed by the complex plan alignment of the new channel which broadly followed the right mitre of the embankment. The various components of the new channel, including flow deflectors positioned at various locations along the length of the channel and a bespoke stilling basin at the downstream end, were optimised during the model study.

In the case of Buckshole Reservoir, although the risk-based approach justified adopting a solution that would not strictly meet the standards-based approach for a Category A reservoir, the physical model study was instrumental in identifying modifications to the design to ensure that the spillway channel would safely contain extreme flood flows almost equivalent to the routed Safety Check Flood outflow.

The new spillway channel, completed in November 2022, has significantly reduced the risk of out of channel flow and any resulting damage / breach of this Category A reservoir.

INTRODUCTION

Buckshole Reservoir is located on the northern side of Alexandra Park in the heart of Hastings, East Sussex. The reservoir was originally built in 1852 to serve as a water supply for Hastings. However, it is now only used for fishing and provides a public amenity as part of an important Grade II listed public park in the heart of Hastings. The current owner and Reservoirs Act Undertaker is Hastings Borough Council.

The dam comprises an 11m-high earth embankment with a concrete siphon spillway structure located on the right bank which discharges into a concrete spillway channel running down the right mitre of the dam. Given its location, just upstream of densely populated residential and commercial areas close to and within Hastings town centre, the reservoir is classified as a Category A reservoir in accordance with the latest reservoir safety guidance (ICE, 2015).

A report on the reservoir was issued in 2017 following a statutory inspection under Section 10 of the Reservoirs Act 1975. The report included the following mandatory recommendations:

a) *Obtain dambreak maps and [the downstream flood risk] consequence assessment from the Environment Agency when they have been updated to 2016 "reservoir flood map specification", to quantify the incremental consequences if the dam failed in a major flood;*

b) *The output from the above should then be considered by a Panel AR Engineer [All Reservoirs Panel Engineer], and if appropriate an ALARP study undertaken of measures to increase spillway chute capacity, followed by implementation of measures which are proportionate in cost relative to the reduction in risk achieved.*

This paper will focus on two follow-on studies that were completed to help the Undertaker progress the above recommendations:

- **Risk-based assessment**. After completing a dambreak assessment, the Undertaker commissioned a study to investigate the feasibility of options to address recommendation b) above. The study followed a risk-based approach where the required scope of works was ultimately dictated mainly by the cost versus risk reduction principle as applied in the UK (otherwise known as the ALARP approach).
- **Physical model study**. Armed with the outline scope of works from the risk-based study, a physical model study was commissioned to optimise the various components of the scheme, including the proposed new spillway channel along the right mitre and a bespoke stilling basin just beyond the central downstream toe of the embankment.

SPILLWAY STRUCTURE

The original spillway weir structure was modified in 1985. The modified structure comprises five flow paths of which one is a standard ogee weir and the other four are air regulated siphons. The five spillway openings discharge into a siphon chamber from which water is directed to the spillway channel which follows the right mitre of the dam. The first section of the spillway channel, which was constructed as part of the 1985 works, comprises a rectangular reinforced concrete channel which turns through 90 degrees and then reduces in section to connect to the original 19th century trapezoidal overflow channel some 15m further down the mitre of the dam. The 1985 section of channel has a length of about 35m after which it ties into the much smaller original overflow channel. The original overflow channel has a length of some 45m before discharging into the downstream single channel through Alexandra Park.

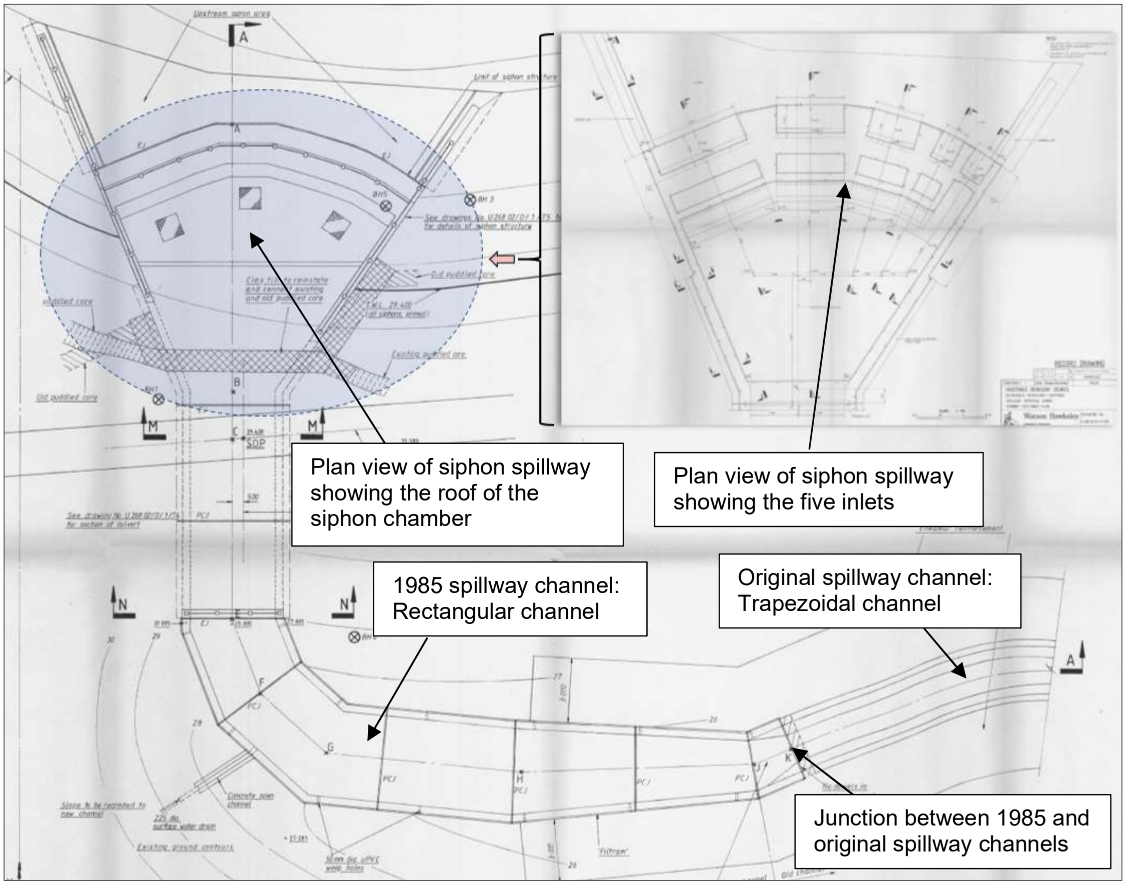

Figure 1. Plan layout of the Buckshole spillway assessed during the risk-based study

The capacity of the siphon spillway was originally determined during a physical model test that was conducted at Newcastle University. The associated rating curve, included in the 2017 S10 report, showed that the spillway should discharge approximately 54m^3/s with the water level in the reservoir at dam crest level, i.e. just before overflowing of the dam occurs. The 2017 Section 10 report stated the following:

"It is concluded that the spillway weir does meet the standard recommended by the ICE for a Category A dam of passing a design flood of 1 in 10,000 chance per year with no damage, and a PMF flood without failing."

A longitudinal section through the spillway arrangement constructed in 1985 is shown in Figure 2 below.

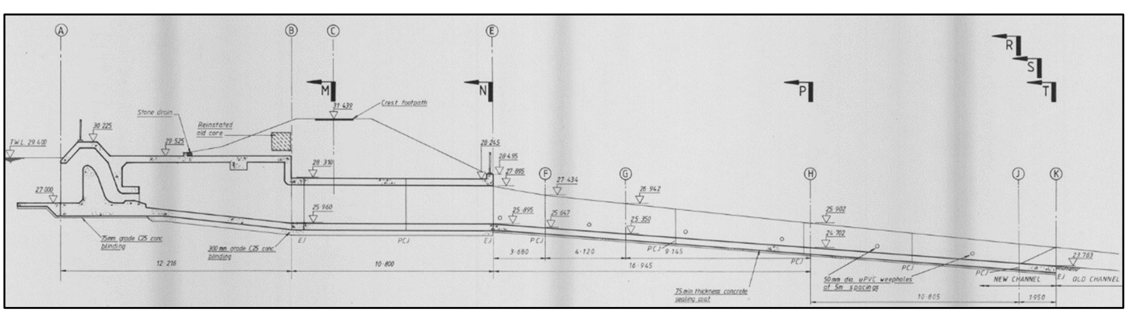

Figure 2. Longitudinal section through 1985 siphon spillway and downstream channel

Managing Risks for Dams and Reservoirs

Although the upper reinforced concrete channel that was constructed as part of the 1985 works contributed to an improved spillway arrangement (Figures 3 & 5 below), a portion of the original masonry channel was retained, significantly limiting the capacity of the system (Figures 4 & 6 below).

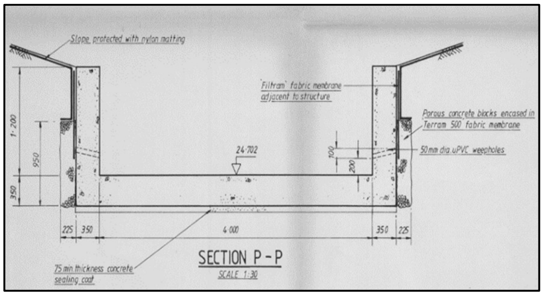

Figure 3. Typical 1985 upper spillway channel section

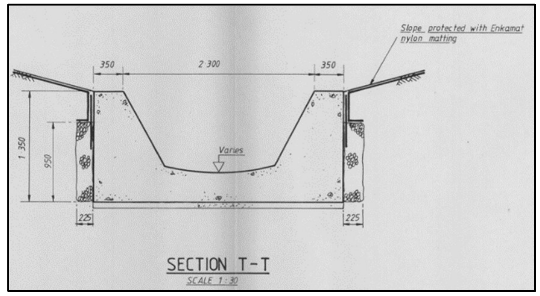

Figure 4. Section of transition to original 19th century masonry channel

Figure 5. View of 1985 spillway channel section (from right abutment)

Figure 6. View of transition to original masonry channel

The 2017 Section 10 report therefore made the following statement regarding the spillway channel capacity:

"In larger flows water will come out of the channel and start to erode the downstream face of the dam. The magnitude of flow and annual chance of failure cannot be estimated reliably without a detailed model study, but it is suggested that erosion sufficient to breach the dam and release the reservoir is quite likely at the 1 in 1,000 chance per year flood.

"It is considered that the spillway channel does not meet the standards for a Flood Category A dam."

This formed the basis for the risk-based assessment which is described in more detail in the following section.

RISK-BASED ASSESSMENT

Background

The UK dam industry is increasingly adopting risk-based reservoir safety management practices. This has mainly been driven by two sets of guidance issued by the Environment Agency: the *Interim Guide to Quantitative Risk Assessment for UK Reservoirs* (Brown and Gsoden, 2004) and the *Guide to Risk Assessment for Reservoir Safety Management* (EA, 2013).

In addition, the current *Floods & Reservoir Safety 4th edition* (FRS4) (ICE, 2015) guidance allows for both a 'standards-based' approach and a 'risk-based' approach when assessing the safety of existing reservoirs.

The standards-based approach follows a set methodology which determines the physical requirements for a dam and associated spillways / overflows to ensure extreme flood events can be passed safely, aiming to ensure the integrity of the dam and spillway / overflow structures. The required standard depends on the flood category of the dam, reflecting the anticipated loss of life and extent of damage, or downstream consequences in the event that the dam was to fail. This failure scenario is referred to as the 'wet-day' failure scenario.

The FRS4 guidance states: *'Where expenditure on remedial works will be significant to meet the standards-based approach to dealing with floods … a risk-based approach could be adopted to assessing the value (cost versus reduction in risk) of undertaking remedial works'*. This approach was adopted for the assessment for Buckshole Reservoir.

ALARP Assessment

The industry-accepted risk-based approach aims to reduce the risk of dam failure 'as low as reasonably practicable' to protect people and property downstream and is referred to as an 'ALARP' approach. It follows a rigorous and logical methodology, identifying options for improvement works where the cost of these works is proportionate to the reduction in risk achieved. According to the Health and Safety Executive guidance (HSE, 2001), the risk has been reduced to an acceptable level where the 'cost to save a life' (see equation below) is less than the 'value of preventing a fatality' (VPF).

$$\text{Cost to save a life (CSL)} = \frac{\text{Cost of risk reduction works}}{\text{Reduction in “likelihood of failure} \times \text{likely loss of life”}}$$

There is no reservoir-specific guidance on selecting the VPF and so the value that is assigned should be selected under the guidance of an All Reservoirs Panel Engineer, whilst also considering the following:

- Direct costs (measurable), such as the earning potential of the victims, injury and long-term health impairment of other victims not included in the 'Likely Loss of Life' (LLOL) value, and emergency services costs.
- Indirect (business losses).
- Intangibles (psychological impact on people, environmental damage) – it could be argued that a value should be assigned to the Intrinsic Value of a Human Life (irrespective of age, health, education, etc.).

For the Buckshole assessment, it was agreed with the All Reservoirs Panel Engineer that the VPF should be at least five times more than that used for roads and railway schemes. The reasoning for this was that whilst the public can be expected to understand and accept the risks associated with travel, their exposure to the risk of dambreak inundation could be considered to be involuntary. The Department for Transport's assessed VPF for road and rail for 2010 was £1.7 million. Therefore, for this study a VPF value of £8M was adopted.

ALARP proportionality is governed by the following:

$$\text{Proportion Factor (PF)} = \frac{\text{Cost to Save a Life (CSL)}}{\text{Value to Prevent a Fatality (VPF)}}$$

Managing Risks for Dams and Reservoirs

Works are justified in accordance with the ALARP principle when the PF < 1. When the PF > 1, then the cost is considered disproportionate to the level of risk reduction, and there is no requirement to carry out improvement works.

Options for Spillway Channel Improvements

The options for upgrading the spillway channel capacity at Buckshole Reservoir were developed through a long-list / short-list process. Eight long-listed options were screened at a high level taking into consideration factors such as technical viability, practicality of implementation, anticipated cost of implementation and anticipated ecological, landscape and heritage impacts. Four options were deemed to be feasible and likely to provide sufficient reduction in risk to be carried forward to the short list of options.

Having established the current annual probability of dam failure as approximately 2.6×10^{-3}, or 1 in 400, the reduction in probability of failure for each of the short-listed options was calculated by means of an event tree analysis, the details of which fall outside the scope of this paper. A summary description and the reduced probability of failure achieved by each of the short-listed options are summarised in Table 1 below. A plan layout of each option is shown in Figures 7 – 10.

Table 1. Summary of short-listed options

Option Ref	Description	Annual probability of failure after works
2	Large capacity concrete channel with covers to contain flows. This option would meet the standards-based approach, i.e., full requirements for a Category A reservoir.	~ 2.5×10^{-6} (1 in 400,000)
3	Large capacity concrete channel following footprint of existing channel. This option would result in a significant risk reduction but would fall short of meeting the standards-based approach.	~ 8.6×10^{-6} (1 in 116,000)
4	Large capacity concrete channel cutting into existing right abutment downstream of 90° bend, resulting in a straight alignment. This option would result in a significant risk reduction but would fall short of meeting the standards-based approach.	~ 5.1×10^{-6} (1 in 194,000)
7	Replace masonry channel with similar-sized rectangular concrete channel and add erosion protection to the adjacent downstream face of the dam. This option would provide the least amount of risk reduction.	~ 6.2×10^{-5} (1 in 16,000)

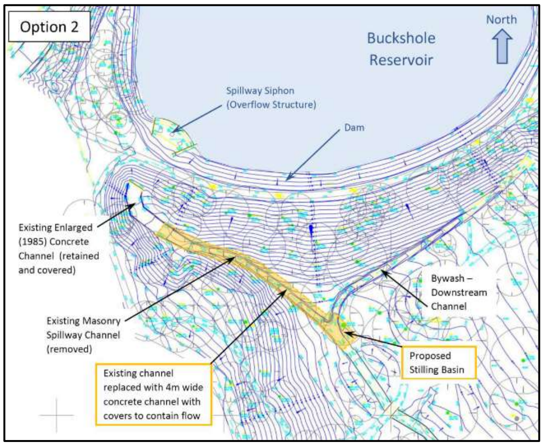

Figure 7. Short-listed option 2: proposed new covered channel

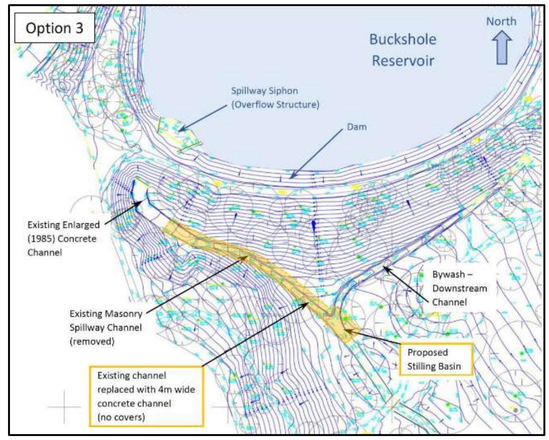

Figure 8. Short-listed option 3: proposed new spillway channel (no covers)

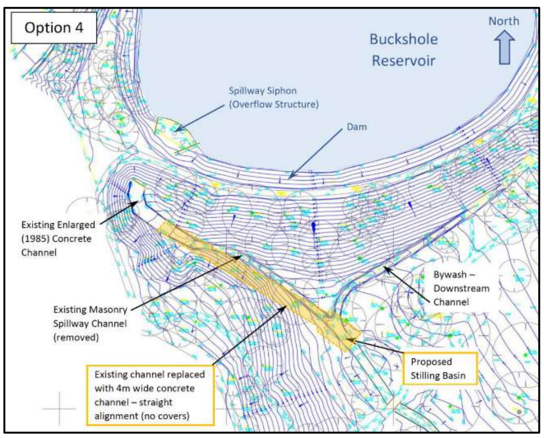

Figure 9. Short-listed option 4: proposed new spillway channel (straight alignment)

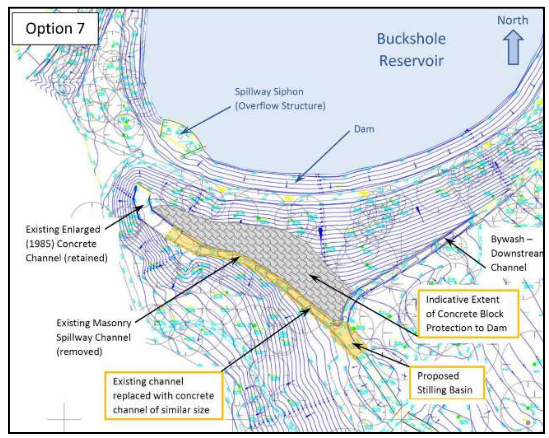

Figure 10. Short-listed option 7: proposed new spillway channel and downstream face reinforcement (concrete blocks)

The risk reductions achieved by each of the short-listed options are shown in Figure 11 below. This shows that for all short-listed options the improvement works reduce the risk into the 'broadly acceptable zone', with the highest cost option, Option 2 providing the greatest risk reduction and Option 7 the least reduction in risk.

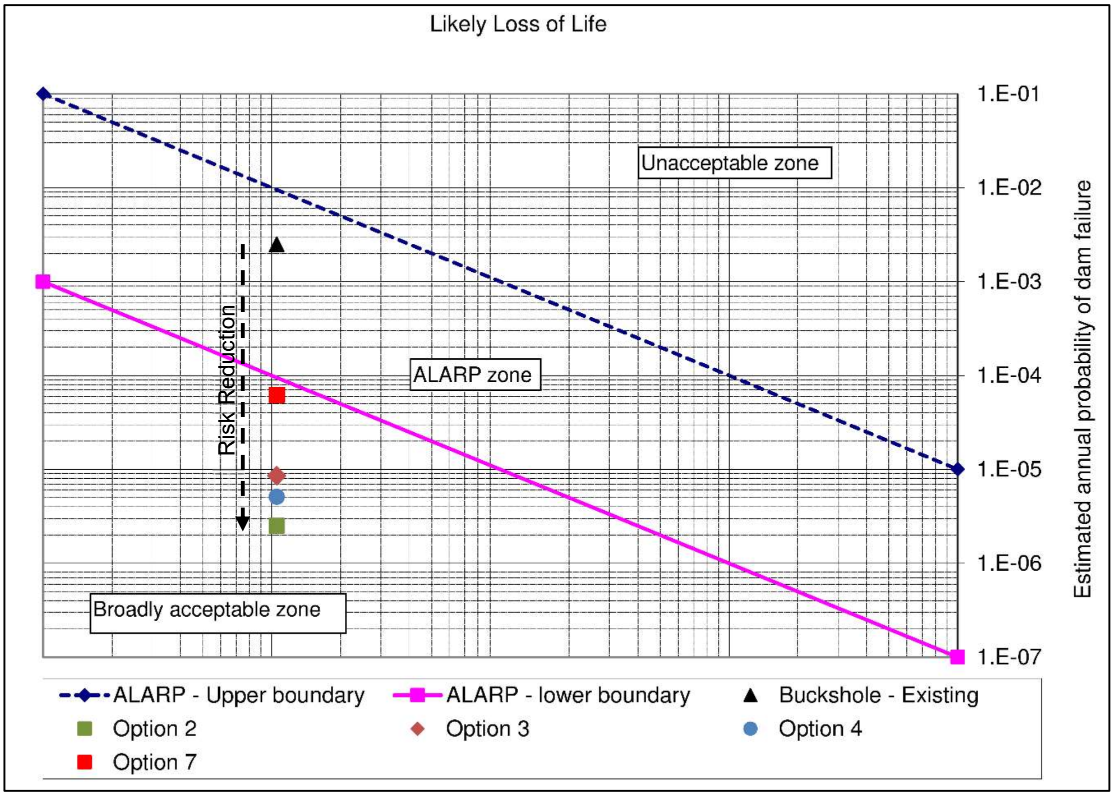

Figure 11. Risk reduction achieved by the short-listed options

ALARP Results

High-level costs associated with each short-listed option were compared with the reduction in risk achieved by the associated works, using the ALARP principle of proportionality. In the case of Buckshole Reservoir, all of the short-listed options were shown to be proportionate, as demonstrated in Table 2 below, compared to the estimated existing probability of dam failure before works of approximately 2.6×10^{-3}, or 1 in 400.

Table 2. Summary of costs and risk reduction benefits for the short-listed options

Option Ref	Description	High-level cost (£k)	Annual probability of failure	Proportionality Factor (PF)
2	Large capacity concrete channel with covers to contain flows.	900	~ 2.5×10^{-6}	< 1
3	Large capacity concrete channel following footprint of existing channel.	650	~ 8.6×10^{-6}	< 1
4	Large capacity concrete channel cutting into right abutment with straight alignment.	750	~ 5.1×10^{-6}	< 1
7	Smaller concrete channel with erosion protection to the adjacent downstream face of the dam.	500	~ 6.2×10^{-5}	< 1

Recommended Option

All the options achieved ALARP proportionality and therefore satisfied the risk-based approach. The choice of which works to implement therefore became one of engineering judgement. Other factors considered included landscape and ecology impacts, public safety, and future operational and maintenance requirements. In addition, the Undertaker's appetite for residual risk and the associated likelihood of future upgrades also played an important role in the decision-making process.

It was ultimately agreed to implement Option 3 (large capacity concrete channel following footprint of existing channel) as this option:

- Provided a significant reduction in the probability of dam failure.
- Minimised adverse landscape and ecology impacts.
- Would reduce public safety risks associated with the existing spillway.
- Offered the lowest ongoing operation and maintenance costs.

Having decided on the preferred works option, the next step was to develop a physical hydraulic model to optimise the detailed design, with the aim of optimising the risk reduction benefits and reducing the cost of the scheme. The study was commissioned mainly in view of the complexities of the hydraulic operation associated with the proposed channel, but also recognising the opportunity for targeted improvements which would further reduce the risk of damage to the dam, bringing it closer to satisfying the standards-based approach for the Category A Buckshole Reservoir.

PHYSICAL MODEL STUDY

CRM Rainwater Drainage Consultancy Ltd was commissioned to undertake the physical model study in two stages. The first stage was to model the existing condition, to confirm the overall hydraulic performance and in particular to identify the events that could potentially lead to failure. This would provide a sense-check of the results obtained during the preceding risk-based assessment. The second stage was to develop and optimise the shape for the proposed new structure through the highly constrained landscape of the park and to test the effectiveness of smaller-scale improvements in further containing flows during the extreme flood events.

Reservoir outflow was limited by the 1985 concrete channel downstream of the siphon spillway to a maximum flow of $44m^3/s$. At events well below this, flow was observed to already be out of bank on all parts of the original 19th century masonry spillway channel further downstream. This flow behaviour would be expected to erode the downstream face of the dam and most likely fail the masonry channel.

The use of a physical model to inform the design of the new spillway channel allowed multiple shape options to be readily tested, with overhangs and superelevation added in some areas to improve flow performance. To make the final site construction as simple and economical as possible, whilst minimising the landscape impacts, the new structure was designed in a series of straight-line portions, each with a uniform rectangular section, which had to have relatively sharp bends in certain locations to avoid valuable and protected trees in the park.

These constraints led to difficult hydraulic conditions in the channel as flow was supercritical throughout, and thus would not readily change direction. To avoid higher walls at the bend sections, wall overhangs or flow deflectors ("bus shelters") were used in many places to contain flow within the channel. This allowed the very complex spillway shape to be constructed with the minimum of visual impact in a sensitive area of the park.

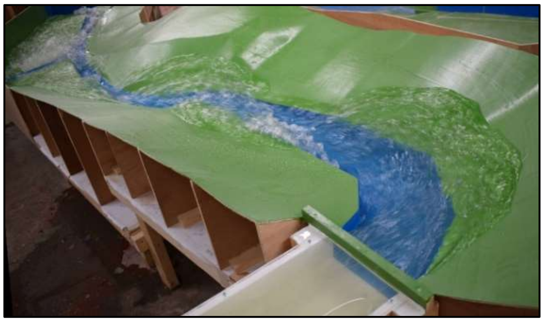

Figure 12. Existing spillway at flow of 44m^3/s (as modelled)

Figure 13. New spillway layout at 44m^3/s (as modelled)

At the downstream end of the channel, and coinciding with the toe of the embankment, a stilling basin was proposed to limit high velocity flows entering the public areas of the park downstream of the reservoir. Again, because of its location, there were significant constraints on this structure. To minimise construction complexity, the depth of the structure below ground level needed to be minimised. However, the height of the structure also needed to be minimised to avoid a significant visual impact. In addition, the difference in level between the stilling basin invert and the existing invert of the downstream channel had to be reduced to minimise the re-acceleration of flows at the downstream end weir of the basin. Further, as the basin discharged into a well-used area of the heritage Grade 2 listed Alexandra Park, any negative aesthetic impact also had to be avoided, in line with conditions imposed in the planning permission. Working within these constraints resulted in a design that would only provide effective stilling up to the 1 in 150-year event. However, it was still possible to modify the design using the physical model to maintain stable attenuated outflows with the basin surcharged during more extreme flood events.

The ideal design for a stilling basin would have incoming flows entering uniformly across the cross section and in line with the longitudinal axis of the basin. However, constrained by important features within the park and planning requirements, this arrangement was not possible. Consequently, the stilling basin design had to accommodate an inflow largely concentrated on the outer wall of the incoming chute, entering the stilling basin at an angle. To improve approach conditions, the final reach of spillway channel was super-elevated, helping to keep the flow evenly spread across the width of the channel. The angle of super-elevation was optimised for lower flow cases where effective stilling would be more critical. The super-elevated design helped to eliminate the worst effects of the approach conditions maximising the stilling of flows. For example, the modelled 1 in 150-year event inflow velocity of 10.8m/s was reduced to 1.8m/s exiting the basin. In the case of the 1 in 1,000-year event the inflow velocity of 11.2m/s was reduced to 4.0m/s.

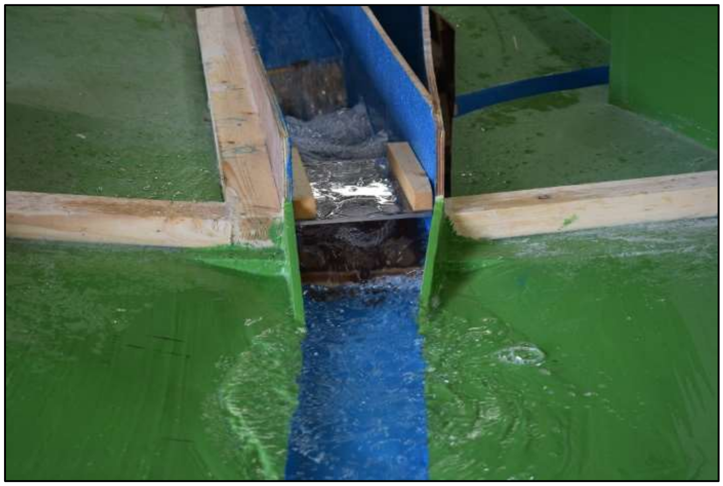

Figure 14. Stilling basin in 1 in 150-year flood event (as modelled, looking upstream)

During more extreme flood events, with the stilling basin surcharged, the velocity reductions were more limited, but still amounted to a notable 25% velocity reduction at the maximum flow. To aid stilling in lower events, baffle blocks ("dragon's teeth") were added. These proved to very effective, although once the basin was surcharged, these features resulted in pluming of flows during more extreme events. To control this effect, the section of stilling basin upstream of the blocks and end weir was covered with a roof section, containing flows and limiting the overall height of the structure. An additional benefit of this feature was that it minimised the visual impact of the stilling basin and allowed the addition of landscape planting as a screen to further reduce the landscape impact of the new larger channel.

Figure 15. Finished spillway channel and stilling basin

CONCLUSIONS

The application of a risk-based approach allows reservoir undertakers to identify and implement works that reduce the risk posed by their reservoirs to an acceptable level or 'as low as reasonably practicable', often resulting in lower capital expenditure than would be the case if simply the standards-based approach was followed. Although the approach may result in a greater level of residual risk compared to that afforded by applying prescribed standards, the risk-based approach justifies this through the application of an industry developed and accepted, rigorous, and defensible qualitative and quantitative methodology.

Whilst simplified 1D hydraulic calculations that can support a risk-based approach are often sufficient to meet the overall objective, in certain cases where the hydraulics are relatively complex, there may be opportunities to further optimise a risk-based solution through physical modelling. As well as providing an opportunity to improve the performance of a design such that the risk benefits are enhanced, this approach can also result in a reduced scheme cost. Further, a physical model can also be used to verify the validity of a risk-based solution, especially in cases where many assumptions are made during the initial risk-based assessment.

The Buckshole Reservoir case study as presented in this paper is considered to be a good example of the value that can be achieved in using a risk-based approach, optimised using physical modelling. The adopted reservoir safety works at Buckshole achieve a standard marginally lower than the standards-based approach; however, given the uncertainties surrounding the hydraulic performance, it was considered that the cost and effort to commission a physical model study were justified, both to validate the risk-based findings and to optimise the proposed risk-based solution. The physical model not only identified 'easy win' modifications to further reduce the risk by more effective flow containment, and reducing scheme cost, but also helped to identify features that would enhance the appearance of the channel, ultimately helping to limit the visual impact within an important landscape setting of a public park.

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Risk Assessments for Reservoir Safety – The Value of a Risk-Based Approach

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SYNOPSIS The Institution of Civil Engineers 'Floods and Reservoir Safety', 4th edition (ICE, 2015) states: *'The risk-based approach using appropriate tools and methods seeks to provide an approach that allows an owner and their advisors to better understand and evaluate reservoir safety risks in a structured way. This then allows for risk-based decisions to be made to reduce risks to people, the environment and the economy but still maintain an important reference to accepted best practice.'*

There is an increasing use in the industry of a risk-based approach to assess reservoir safety. This paper considers four case studies with Undertakers each faced with different threats to their reservoirs, looking at why and how the approach has been applied, aiming for pragmatism in each case whilst maintaining best practice.

An initial screening assessment allows an early view on whether or not the outcome of a risk-based approach is likely to be different to the outcome of a standards-based approach, and therefore whether or not the risk assessment would be of value. Close involvement with the reservoir owner in each case helps to ensure a pragmatic approach to identifying and assessing specific threats, associated probabilities of failure and realistic viable options for improvement works. This involvement has also been found to be critical to ensuring 'buy-in' from the reservoir owner in terms of the assessment outcomes and next steps once options for improvement works have been identified.

INTRODUCTION

There have been and continue to be significant changes that impact how reservoirs are assessed in terms of safety, not least with an increasing awareness of climate change and the prevailing changes in legislation. At the same time the UK has a stock of aging dams and an increasing number of large raised reservoirs as the 10,000m^3 threshold is introduced. A great number of these reservoirs are on private estates as ornamental lakes. Many others, built to serve as water supplies or for industrial use have long since outlived their original purpose and are being sold on or handed over to private owners or local authorities as amenity and fishing lakes. The true cost of owning and maintaining these reservoirs often only becomes apparent following an inspection under Section 10 of the Reservoirs Act 1975. Many owners of these reservoirs are finding that what was once a welcome amenity and an asset becomes a costly liability when the inspection identifies shortfalls in spillway capacity or freeboard, or serious structural or stability issues with the dam.

The application of the relevant standards can be unnecessarily demanding and costly for reservoir owners, in particular where a well constructed and well maintained dam poses little risk to those downstream. When considering the true risk posed by reservoirs the industry has had the benefit of a risk-based approach for many years with well developed and accepted guidance in place. The application of this approach was reinforced with the publication of the fourth edition of 'Floods and Reservoir Safety' (FRS4) (ICE, 2015). Whilst some owners are able to pass on the costs of reservoir maintenance and improvement works to their customers, this is not the case for most private owners or indeed for local authorities where budgets are becoming ever more stretched. The appropriate application of a risk-based approach can offer reservoir owners a more cost-effective and affordable solution for ensuring the right level of reservoir safety to protect people and property downstream whilst still reflecting best practice.

This paper summarises four recent examples of reservoirs which have been found to fall short of the relevant reservoir safety standards and where the risk-based approach has been applied. In each case an appropriate level of pre-screening has been carried out to help the owner decide whether or not a risk-based approach is worth considering. The examples illustrate different outcomes to help understand the extent to which a risk-based approach can be of value. Reflecting on these examples this paper seeks to further raise the awareness of reservoir engineers and those responsible for overseeing and enforcing reservoir safety, and even reservoir owners themselves, of when and how this approach can be applied. In doing so we should hope to maintain and improve the attitudes of the many responsible reservoir owners as they endeavour to fulfil their obligations in respect of reservoir safety.

SUMMARY OF THE RISK-BASED APPROACH

The risk-based approach aims to reduce risk 'as low as reasonably practicable' (ALARP) and is referred to here as an ALARP approach. The approach generally accepted by the industry is based on guidance published by the Environment Agency in the 'Guide to risk assessment for reservoir safety management' (RARS) (EA, 2013). This guidance sets out a rigorous and logical methodology with the aim of identifying options for improvement works that would reduce the risk of failure of a dam to an acceptable level at a cost that is proportionate to the reduction in risk achieved. According to Health and Safety Executive guidance (HSE, 2001), and with reference to RARS, the risk has been reduced to an acceptable level where the 'cost to save a life' (CSL) is less than the 'value of preventing a fatality' (VPF).

The Department for Transport's assessed VPF for road and rail for 2010 was £1.7 million. However, for dams, where the risk to those in the potential inundation area is involuntary, in that the public are not generally aware of the risk posed by reservoirs, it is generally accepted within the industry that the assessed VPF for dams should be approximately five times more than that for roads and railways. Thus, for dams, where the CSL is less than 5 x £1.7M = £8.5M it is considered proportionate to carry out the necessary improvement works.

CASE STUDY 1: EAST MIDLANDS INTERNATIONAL AIRPORT EAST AREA BALANCING POND

The East Area Balancing Pond is a non-impounding reservoir providing temporary storage for water from the airport runways and aprons. As well as the gravity drainage inflows from the airport the reservoir can also receive diffuse overland flows from a direct catchment of some

1.37km^2. A Section 10 Inspection in 2020 concluded that the balancing pond is a Category A reservoir. The report also identified that the balancing pond overflow, a 3m long lowered section of the embankment, provided insufficient capacity to safely convey excess inflows from the airport and overland flood flows from the direct catchment. Accordingly, the report made mandatory recommendations in the interests of safety for an updated flood study and implementation of any necessary improvement works.

The subsequent flood study confirmed a significant shortfall in overflow capacity and concluded that either improvement works should be implemented to satisfy the standard defined in FRS4 for a Category A dam, or to carry out a risk-based assessment to determine whether or not the costs of capital works to increase spillway capacity would be proportionate to the reduction in risk achieved. The cost to carry out improvement works to the required standard was estimated at this stage to be in the order of £300,000.

To help inform a decision on which approach to take it was agreed with the Undertaker to carry out an initial screening assessment

Screening Assessment

For the screening assessment high level information and assumptions were used, as follows:

- Existing probability of failure of 1 in 10,000, i.e. the Design Flood which the flood study showed would overflow the dam crest by approximately 150mm.
- Probability of failure must be reduced to at least 1 in 400,000, notionally the probability of the Safety Check Flood (SCF), the Probable Maximum Flood (PMF)event.
- Perform sensitivity analyses, using a 'back calculation' to determine the limiting cost of 'proportionate' improvement, as follows:
  - Assume a Likely Loss of Life of 1 and vary the downstream economic damage resulting from reservoir failure;
  - Assume the economic damage at £1 million and vary the Likely Loss of Life (LLoL).

Varying the downstream economic damage between £100k up to the maximum assessed value of £5M indicated a range of maximum capital costs for improvement works between £25k and £40k, i.e. that the ALARP calculation would not be sensitive to changes in downstream economic damages. On the other hand, varying the LLoL value was found to be a significant factor which would influence the ALARP calculation. However, in this case, even considering a high LLoL value of five the calculation indicated that the maximum value of improvement works that could be considered to be proportionate would be £127k. A greater cost than this would be disproportionate and the justification for carrying out the works would be low. This value was significantly less than the high level estimate of £300k for improvement works to satisfy standards.

The conclusion from this screening assessment therefore was that a risk-based approach would be appropriate to consider options for improvement works involving a reduced scope rather than the full scope of works required to satisfy the standards-based approach. A full scale ALARP assessment was therefore recommended to confirm the appropriate scope of works, if any, to satisfy the risk-based approach.

Managing Risks for Dams and Reservoirs

Risk Based Assessment

With a reasonable level of detail already available to inform the study a Tier 2 risk assessment was adopted, as set out in Section 8.2.2 of RARS, to determine the current probability of failure of the reservoir due to overflowing of the embankment crest. The methodology used the outputs of the flood study to develop rating curves and applied the assessed flow durations and velocities to CIRIA 116 performance curves (CIRIA, 1987) for plain grass to determine the critical velocity that would be likely to lead to dam failure. The critical velocity was then used to estimate the corresponding depth of flow over the embankment crest, i.e., the depth of flow over the embankment crest that can reasonably be assumed to cause onset of significant erosion leading to the failure of the dam. The flood routing results from the flood study were used to develop a graph plotting total reservoir outflow against annual probability, from which the estimated annual probability of the total outflow at the point of failure can then be read. This value was taken to represent a reasonable estimate of the current annual probability of failure of the reservoir due to overflowing of the crest. For the East Area Balancing Pond the annual probability of failure due to overflowing of the crest was determined to be 8.3×10^{-6}, or 1 in 120,000.

Downstream consequences

An assessment of downstream consequences was made with reference to the available Environment Agency reservoir flood mapping. This assessment indicated the following incremental damages:

Table 1. Wet-day failure of East Area Balancing Pond: estimated damages

Consideration	Incremental impact of reservoir failure
Maximum population at risk	269
Time averaged population at risk	110
Likely Loss of Life (LLoL)	0.11
Cost of third party damages	£8M

Table 2. Wet-day failure of East Area Balancing Pond: Pre-scheme risk to life

Consideration	Probability	Comment	Tolerability
Probability of failure of the dam	8.3×10^{-6} (1 in 120,000)	-	-
Individual risk of death per year	2.9×10^{-8} (1 in 34M)	This is less likely than 1 in 1M prescribed by the HSE (2001) as the boundary between the broadly acceptable and tolerable regions.	The individual risk of death per year lies in the broadly acceptable zone.
Societal life loss per year	9.2×10^{-7} (1 in 1M)	Lives per year: product of probability of dam failure and likely loss of life. (see F-N chart, Figure 1)	Broadly acceptable

An F-N chart relates the probability of dam failure (F) to likely loss of life (N) resulting from that failure, as described in RARS. Such curves may be used to express societal risk criteria and to describe the safety levels of particular facilities, in this case reservoirs. An F-N chart was produced for East Area Balancing Pond to show the current societal risk (Figure 1).

The F-N chart shows that East Area Balancing Pond plots in the 'broadly acceptable zone' in relation to the probability of failure during floods and the resulting consequences.

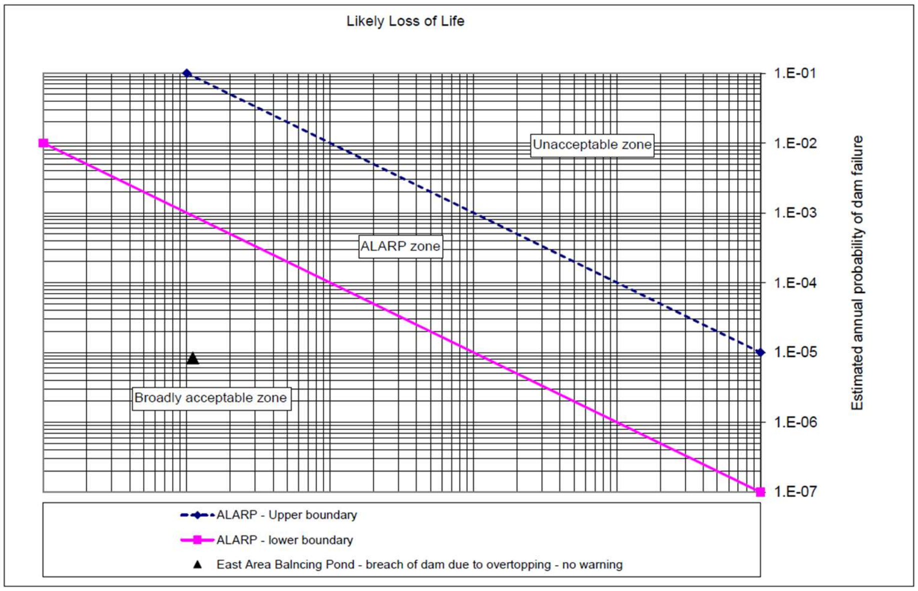

Figure 1. F-N chart: wet-day scenario for East Area Balancing Pond

ALARP Assessment

The HSE (2001) states that when a risk falls within the 'broadly acceptable' region, then further works to further reduce the risk would not usually be required unless reasonably practicable measures are available. RARS argues that this statement by the HSE implies that the ALARP principle still applies to risks that fall within the 'broadly acceptable' region. Therefore, although the risk imposed by East Area Balancing Pond in its current form is within the 'broadly acceptable' region, to strictly satisfy current reservoir safety guidance there is a further requirement to demonstrate that the expenditure related to reservoir safety improvement works would be disproportionate to the reduction in risk achieved.

An initial approach can be followed where an ALARP 'back calculation' is used to determine the cost of works that would be proportionate to the reduction in risk that would match the standards-based approach for a Category A reservoir, in accordance with the FRS4 guidance. This cost can then be compared against a realistic estimate of the actual cost of works that would be required to achieve the standards-based approach for a Category A reservoir. If the actual costs are anticipated to be significantly more than the cost to achieve proportionality, then sufficient proof exists to conclude that any further works to the dam would be disproportionate. The following steps were followed:

- Assume that the probability of failure would need to be reduced to 1 in 400,000; i.e. notionally the probability of the Safety Check Flood for a Category A reservoir.
- Select an appropriate proportionality factor (PF) and discounting factor (DF) using RARS guidance (Appendix B).

- Use the ALARP calculation to determine the maximum cost of proportionate works.
- Compare this cost with a realistic estimate of the actual works required to achieve the reduction in risk required for a Category A reservoir.

The results are summarised in the Table 3.

Table 3. Estimated limit of cost of improvement works proportionate to reduction in risk achieved

Consideration	Value	Comment
Estimated existing probability of failure	1 in 120,000	Reservoir critical outflow: overflows embankment and results in dam breach.
Probability of failure for Category A reservoir following works	1 in 400,000	Assumed return period of the Safety Check Flood for a Category A reservoir.
Estimated economic damage downstream due to reservoir failure	£8M	-
Proportionality factor (PF)	5	Ref RARS
Limit of capital cost of works to ensure proportionality	£1,600	Any expenditure exceeding this amount would be disproportionate in respect of the reduction in risk achieved.

The assessment included a sensitivity analysis, reflecting the uncertainty around both the incremental loss of life and downstream economic damages. This check varied the Average Societal Loss of Life (ASLL) value and the downstream economic damages value to determine the corresponding maximum capital cost of works that could be considered proportionate to the reduction in risk achieved. In both cases a wide range of values had little impact on the ALARP calculation indicating little sensitivity to changes in the ASLL and downstream economic damages. Even a worst case with values significantly higher than those assessed indicated that the maximum cost of works that could be justified would be less than £10k. Indeed, with this cost threshold it is apparent that any works offering even a small reduction in risk could not be justified.

The overall outcome of this assessment was to demonstrate that improvement works could not be justified in this instance. The probability of dam failure in relation to the potential downstream impacts was shown to be already as low as reasonably practicable.

CASE STUDY 2: TAYLOR PARK BIG DAM

Big Dam reservoir is a Category A impounding reservoir located a short distance upstream of a densely populated residential area of St Helens in the north-west of England. A large school is located directly within the reservoir inundation flood area, as is the town centre further downstream. Big Dam reservoir is owned and operated by the local borough council as an amenity feature within Taylor Park. The reservoir is a historic feature and the ageing dam exhibits notable settlement in places. The Section 10 inspection carried out in 2022 determined that there was inadequate wave freeboard across the majority of the dam length and that a short section of the dam had settled to a level where it might be subject to overflowing during extreme flood events. The inspection report also noted poor protection to the downstream face, with significant overshading from trees preventing grass growth.

Standards vs Risk-Based Approach

In discussion with the Council it was agreed that a first step was to properly understand the scale of the issue with a detailed flood assessment and modelling of the performance of the spillway and dam, with a view that this would help to inform a decision on the approach to be taken for determining the necessary improvement works. Accordingly, the flood assessment was carried out which demonstrated that the stillwater flood level would be marginally at the lowest crest level during the Safety Check Flood, with excessive wave overtopping expected during the Design Flood. This outcome suggested that a low wave wall would be sufficient to address these shortcomings and satisfy the standards-based approach for a Category A dam.

Screening Assessment

As with the East Midlands example, a similar screening approach was taken to help decide whether a full risk-based assessment would be of value. In this case the consequences were assessed as being significantly higher. The wet day impacts immediately downstream of the reservoir, unaffected by a concurrent fluvial flood, include a large secondary school, a Fire and Rescue Service station and at least 100 residential properties. Additionally, large incremental damages to both people and property could be expected over a significant area of the valley downstream which includes St Helens town centre and many more residential areas.

For the screening a reduction in the probability of dam failure was assumed to be from 1 in 10,000 (current) to 1 in 400,000 (target for standards). In this instance, a simple sensitivity check confirmed that the threshold cost of capital works was sensitive to both a change in downstream economic damages and a change in LLoL, as demonstrated in the initial screening results summarised in Tables 4 and 5. The results also indicated that the threshold cost was high, with a cost of capital works in the order of £300k shown to be proportionate to the reduction in risk achieved.

Table 4. Screening sensitivity varying economic damages [assumed LLoL = 10]

Downstream Economic Damages	Maximum capital cost of works proportionate to reduction in risk achieved
£10M	£278k
£25M	£322k
£50M	£394k
£75M	£468k
£100M	£541k

Table 5. Screening sensitivity varying LLoL [assumed economic damage = £50M]

Likely Loss of Life (LoLL)	Maximum capital cost of works proportionate to reduction in risk achieved
1	£171k
5	£271k
10	£395k
15	£519k
20	£644k

The results in Table 4 show that any works costing up to between £171k and £644k, the limiting threshold within this range depending on the combination of the LLoL and economic damages adopted, would be proportionate to the reduction in risk achieved. The construction of a low wavewall to prevent wave overtopping was estimated to cost in the order of £100k, substantially below the threshold cost range. The Council was therefore advised that a full risk-based assessment was not necessary as it would not change the outcome. The estimated cost of £100k for a wave wall to meet the Category A dam standards-based approach would be proportionate to the risk reduction achieved and therefore the works should be implemented.

CASE STUDY 3: FURNACE POND

Furnace Pond is a historic reservoir, believed to have been built in the 17th century to provide a reliable source of water for local iron workings. Records suggest that cannons were produced at an adjacent foundry. There has been no significant iron working in the area for nearly 300 years and over that time Furnace Pond, which has remained in private ownership, has been used as a source of irrigation water and as a local amenity, mainly for fishing.

A Section 10 Inspection in 2023 and a review of the downstream consequences confirmed that Furnace Pond is a Category C 'High Risk' reservoir. Downstream impacts in the event of failure would be limited to a number of public footpaths, minor roads and possible shallow flooding of two residential properties. A subsequent up-to-date flood assessment revealed that the spillway capacity and freeboard were significantly below Category C standards when considering the standards-based approach. Further, a survey of the 100m long crest confirmed the presence of a low area exhibiting strong evidence of historic and probably regular overflowing, with the flood assessment suggesting a potential for spilling over the crest during the 1 in 10year flood event.

In discussion with the owner it was agreed that consideration should be given to taking a risk-based approach, noting the relatively low consequences of failure of the dam compared to the likely considerable costs associated with carrying out improvement works to satisfy a standards-based approach. Additionally, the site has many large and mature trees both on and adjacent to the dam, and the abutment areas at both ends of the dam were outside the owner's property boundary.

A high-level ALARP screening confirmed that, in relation to the relatively low downstream damages associated with either the dry-day or wet-day failure scenarios, but apparent high probability of failure, low cost improvement works would be shown to be proportionate. The likely maximum cost of interventions that could be shown to be proportionate in relation to the reduction in risk achieved was estimated as £50k.

Accordingly, a full risk-based assessment was carried out to determine low-cost options that would reduce the risk of dam failure as low as reasonably practicable. In this case options were considered to address both the wet-day and dry-day failure scenarios. A Tier 2 assessment suggested that the current annual probability of failure due to overflowing of the dam, the wet-day scenario, was as high as 2×10^{-2}, or 1 in 50 years. In the case of the dry-day scenario, with failure associated with internal erosion, the probability of failure was shown to be 1.4×10^{-2}, or 1 in 70 years. These remarkably high probabilities in relation to this historic structure are taken as reflecting ongoing ageing and deterioration of the dam, evidenced on site by apparently significant settlement along part of the dam, and notable erosion of the downstream face likely to be as a result of overflowing of the crest.

The assessment process identified combinations of simple options that would reduce the risk of dam failure as low as reasonably practicable, i.e. from RARS: CSL < VPF. These included, for the wet-day scenario, a modified grille to be installed across the service overflow to reduce the potential for blockage, along with minor raising and regularising the crest to reduce the probability of overflowing or wave overtopping and to reduce the potential for concentrations of flows over the crest. For the dry-day scenario, options included improved vegetation management, including the production and implementation of a formal vegetation management plan, and an increased level of surveillance. Combinations of these options were shown to cost below £50k and would therefore be considered proportionate.

The outcome of this assessment, demonstrating a reduction in the probability of dam failure to as low as reasonably practicable, is illustrated in the F-N chart in Figure 2.

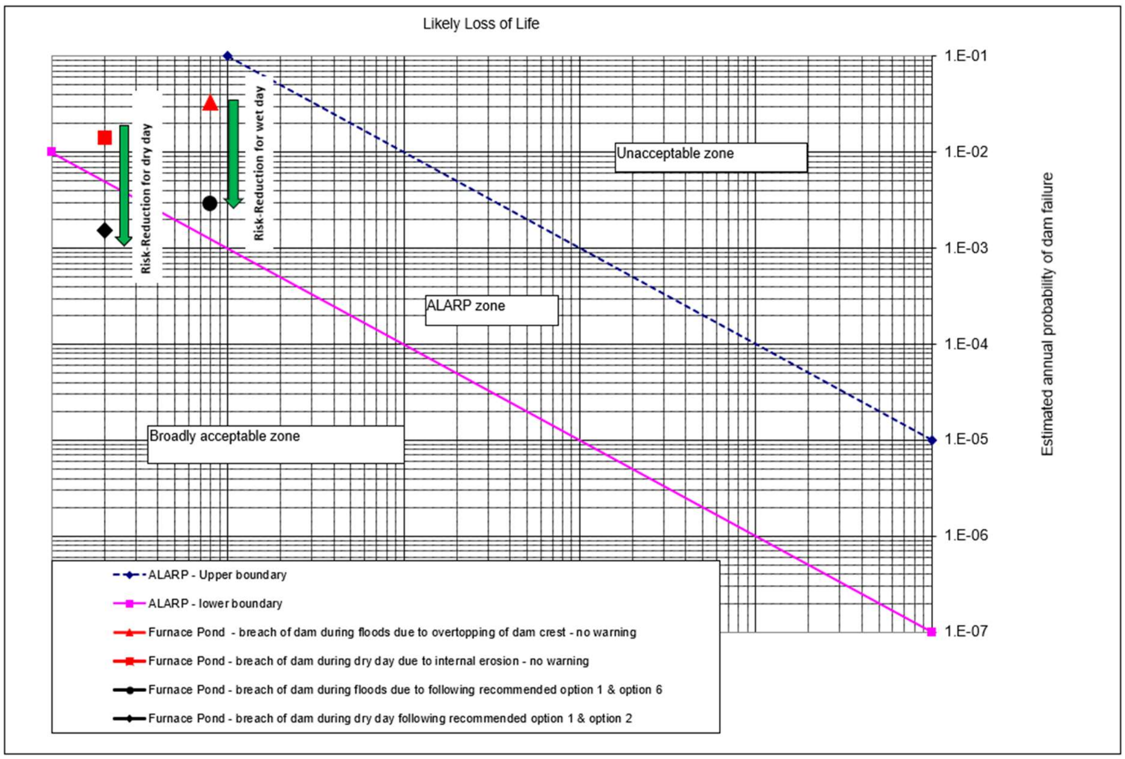

Figure 2. F-N chart: Furnace Pond risk reduction measures

CASE STUDY 4: BUCKSHOLE RESERVOIR

Buckshole Reservoir is a Category A reservoir located a short distance upstream of a densely populated residential area of Hastings in East Sussex. The town centre is also located within the reservoir breach flood inundation area. This 19th century Victorian era reservoir originally formed part of the water supply system for the town and had been operated by the water supply undertaker until the 1970s. At that time the reservoir was taken out of operational service and was passed across to the local borough council as a local amenity and fishing lake.

The Section 10 inspection carried out in 2016 determined that the spillway channel, which follows a sinuous route along the right-hand mitre of the dam, and which also formed part of the original dam works, provided inadequate capacity for extreme flood events and was in a poor condition and in need of either being improved or replaced.

Consequences

To ensure a robust process Stillwater Associates, in collaboration with CC Hydrodynamics, carried out a dambreak assessment of consequences. This assessment revealed significantly lower damages compared to the Environment Agency data available at the time, primarily due to higher damages associated with the wet-day base case yielding a lower incremental impact. For this assessment a range of damages outcomes under different flood events was considered yielding a series of risk curves which were used to determine most likely maximum damages. The relevant flood event was determined as the 1 in 2,000 year event, resulting in an estimated incremental population at risk (PAR) of 828, with a likely loss of life (LLoL) of 1.05 and £11M value of property damage.

Risk-Based Assessment

An initial assessment of the works required to improve or replace the channel concluded that any viable scheme would attract a high construction cost. As a result of the significant damages, and the loss of life and property impacts that could result from failure of the dam, there appeared to be a marginal case for adopting a risk-based approach for determining the necessary improvement works. However, the Council, like many councils, being short of funds was keen to explore options that might reduce the financial burden presented by the measures to be taken in the interests of safety. An options study included the option for discontinuance, but this was ruled out on the basis of cost, environmental impacts and the loss of a well-used public amenity. The logical next step was to carry out a risk-based assessment of viable options with varying levels of risk reduction.

For this assessment, an initial screening identified that failure of the masonry spillway channel presented the critical failure mode. A detailed event tree was prepared to understand the sequence of events that would be expected to lead to dam failure, summarised and illustrated in Figure 3. This process concluded that the failure scenario was a collapse of the spillway channel sidewall leading to erosion of the downstream face of the dam which in turn would destabilise the slope, eventually leading to a slip failure through the crest, initiating a breach.

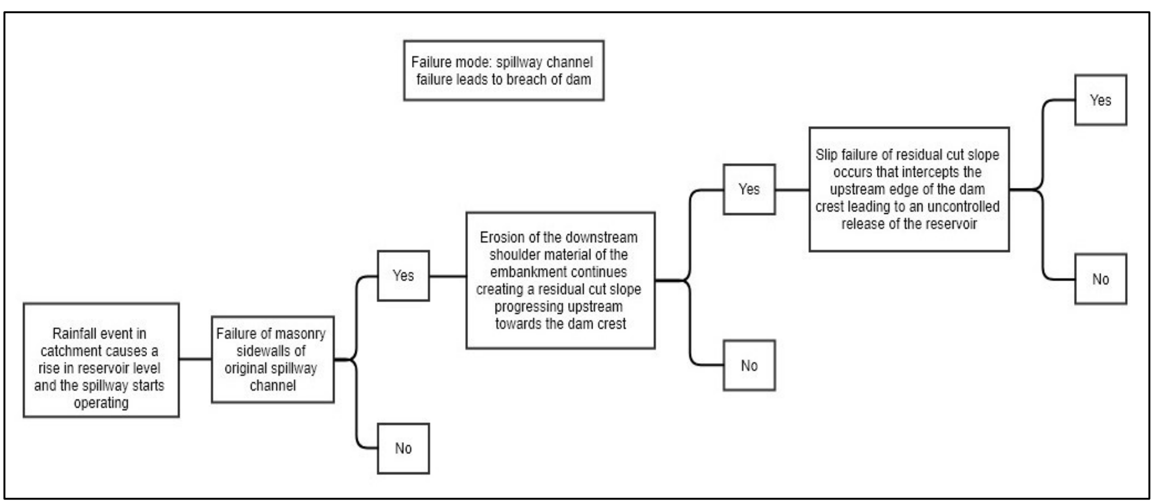

Figure 3. Summary event tree

A fragility curve was developed through a collaborative process involving discussions with the QCE and a study of literature of similar failures that have occurred in the past, such as the Ulley Reservoir incident (Mason, 2010; Hinks et al, 2008). The fragility curve indicated a 10% probability of failure of the masonry channel side wall during flow velocities of around 7m/s, a 50% probability with flow of around 9m/s and a 90% probability with flow of around 11m/s.

Careful consideration was given to the erodibility of the embankment fill materials, drawing on valuable soils information which had been obtained and documented as part of improvement works in the 1970s. Soils were characterised in accordance with an approach developed by Hanson et al (2001).

For the range of flood events considered, the stability of the residual slope was calculated for a critical dam failure slip circle that intercepts the upstream edge of the crest, this taken as initiating a breach. Each factor of safety was then converted to an annual probability of failure in accordance with Figure 8.4 in RARS.

The overall annual probability of failure was determined by summing the products of probabilities associated with each flood event, the corresponding channel sidewall failure and slope failure, for a range of flood events up to the PMF Safety Check Flood. This gave a value of 2.6×10^{-3}, or 1 in 400 chance of dam failure resulting from the loss of the channel sidewall.

The risk-based assessment concluded the following outcomes in terms of risk to life:

Table 6. Buckshole Reservoir: Pre-scheme risk to life

Consideration	Value	Comment	Tolerability
Overall probability of failure of the dam	2.6×10^{-3} (1 in 400)	-	-
Individual risk of death per year	2.8×10^{-4} (1 in 3,600)	Annual probability: product of the probability of failure and probability of loss of life given the dam fails. This is more likely than 1 in 10,000 prescribed by the HSE (2001).	Unacceptable Indicates that spillway channel must be improved to reduce risk of dam failure to an acceptable level.
Societal life loss per year	2.7×10^{-3} (1 in 370)	Lives per year: product of probability of dam failure and likely loss of life. (see F-N chart, Figure 4 below)	ALARP Indicates that spillway channel must be improved to reduce risk of dam failure to an acceptable level and that a risk-based approach can be used.

The risk-based assessment concluded that the level of risk to society was in the ALARP zone and should therefore be reduced as low as reasonably practicable. A series of options was considered for achieving this in discussion with the Council, with a short list reduced to four alternative approaches to replacing the existing spillway channel. These options were assessed in detail against a number of considerations, including ecology, heritage, landscape, operational constraints and safety, as well as cost. The preferred option was then further refined through physical modelling to optimise the design to maximise the benefits and minimise the scheme cost.

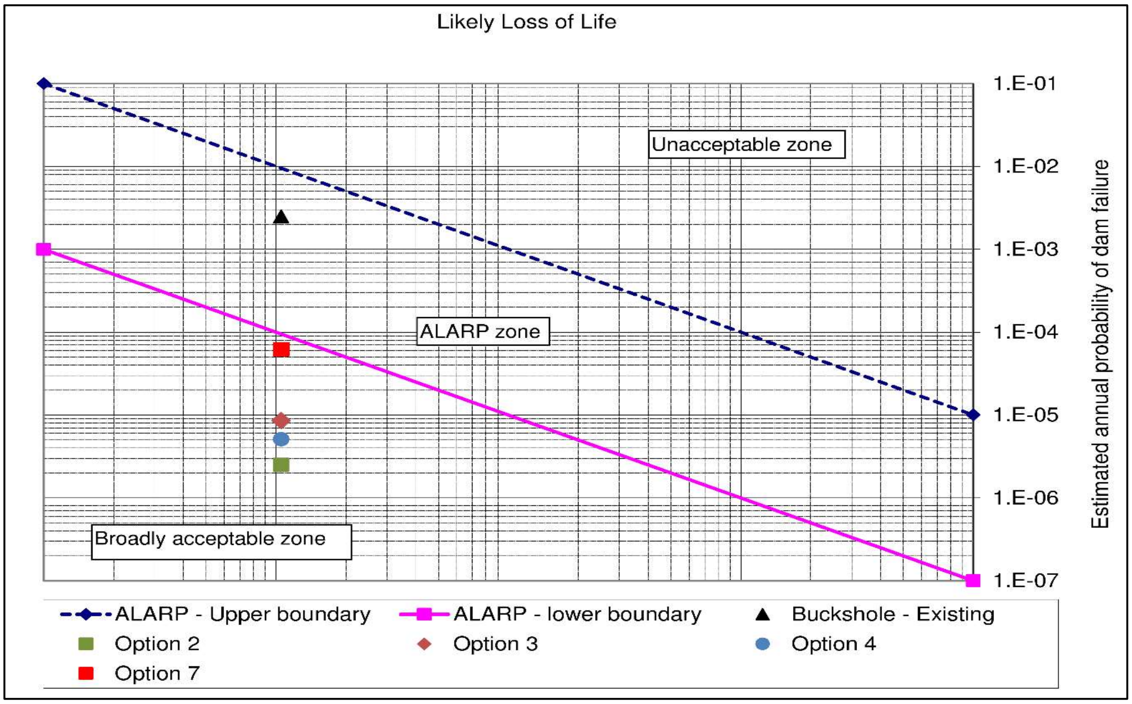

Figure 4. F-N chart: Buckshole Reservoir risk reduction options

Figure 5. Original spillway channel

Figure 6. New spillway channel

This example demonstrates the application of a robust risk-based approach. The high-level screening suggested at best a marginal case to support taking this approach, rather than simply adopting a standards-based approach. In discussion with the Undertaker it was agreed that the risk-based approach should be adopted in an attempt to minimise the cost burden to the taxpayer. The risk-based assessment confirmed the need for the scale of works required and that this outcome, further optimised through physical modelling, was the most cost-effective

CONCLUSIONS

The risk-based approach is increasingly being used in the industry to guide reservoir owners and their advisers in decision making to reduce risks to people, the environment and the economy. A standards-based approach is prescriptive: achieving the standard may unnecessarily burden the owner of a well constructed and well maintained dam that poses little risk to those downstream. As demonstrated with the four case studies presented in this paper a risk-based approach allows wider analysis which may give an optimum solution even meeting future standards in some cases and reducing cases where an owner has to upgrade every time a standard changes.

Pre-screening provides a valuable tool to help Undertakers faced with the potential need for and cost of improvements to decide whether or not a risk-based approach will be of value, or whether the risk is already sufficiently great that a standards-based approach should be followed.

The risk-based approach can justify to the Undertaker that best value is being achieved, which may be particularly relevant to public bodies needing to demonstrate the most appropriate use of available budgets. Further, this approach may prove to be increasingly valuable to the industry and to owners as the stock of ageing and smaller reservoirs increases.

It is to be noted that even when the risk posed by a reservoir has been assessed as acceptable, a residual risk still remains, as is the case for most structures. Given this, it is important that Undertakers understand that the risk can change, either as a result of a change in condition of the dam or due to external factors such as new housing developments downstream.

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The British Dam Society

Ground Investigation through London's raised reservoirs with a summary of ground investigation risks and recommendations, citing techniques used at two sites.

H E FISHER, AtkinsRéalis

SYNOPSIS Thames Water operates and maintains thirty raw water reservoirs across London and the Thames Valley, supplying water services to nine million customers. Most of these reservoirs are retained by a perimeter embankment with a puddle clay core that extends down into the underlying London Clay Formation bedrock. These reservoirs were built with selected material placed downstream of the core to act as a filter, but with no drainage system to monitor; therefore non-intrusive geophysical surveys are regularly carried out by Thames Water to identify areas of excess leakage or seepage, with remediation works being carried out afterwards.

Ground investigation through puddle clay cores is notoriously challenging with a number of key risks; this paper uses two projects as case studies to provide a summary of the ground conditions and associated risks which may be expected at the various reservoir sites around the capital. This paper also summarises the various techniques used for investigating the dams to mitigate these risks and support the construction and remediation of the structures, with a particular focus on the requirements of British Standards and best practice, and the practicality of using these techniques in the field.

INTRODUCTION

Thames Water operates and maintains 30 raw water reservoirs across London and the Thames Valley, supplying water services to 9 million customers. The majority of these reservoirs are raised above the surrounding land and comprise soil embankment dams with a watertight puddle clay core. Puddle clay core embankments were the preferred method for constructing dams in the UK for well over a hundred years before being replaced by the rolled clay core methodology (Reeves & Cripps, 2006). A survey of embankment dams cited in Charles (1989) suggested that, of the 2000 embankment dams in the UK, 65% of them had puddle clay cores. As these reservoirs are now up to or over 100 years old, and due to drawdown during World War Two, defects are occurring within the cores. These defects are being identified by both physical evidence (i.e. ponding at the toe of the embankment) but more recently through geophysical methods which allow for the early identification of seepage before external evidence occurs.

Ground investigation (GI) through puddle clay cores is notoriously challenging with a number of key risks; this paper provides a literature review setting out what to consider when investigating puddle clay cores (with a particular focus on the requirements of best practice and British Standards, and the practicality of using these techniques in the field) and uses two projects as case studies to provide a summary of the ground conditions and associated risks which may be expected at the various reservoir sites around the capital.

The reservoirs selected for discussion in this paper are Island Barn Reservoir and King George V Reservoir; both of which are located on similar natural ground comprising bedrock of London Clay Formation, overlain by River Terrace Deposits and Alluvium. Locally derived soils were typically used for constructing the dams and as such, the two reservoirs' embankments are constructed using similar materials, including London Clay Formation or Alluvium derived puddle clay, and embankment shoulder material largely comprising River Terrace Deposits. From a review of historical case studies, it is expected that the majority of Thames Water's embankment dams are composed of similar material.

PUDDLE CLAY CORES AND THEIR DEFECTS

Puddle clay can be described as 'natural clay of high plasticity reworked and compacted into place to remove all natural fabric or structure [such as sand layers, fissures etc.] and so is a homogeneous material of low hydraulic conductivity' (Reeves & Cripps, 2006). The purpose of the puddle clay core is to create an impervious barrier through the dam and, more often than not, beneath it, therefore it is keyed into the underlying impermeable bedrock. Typically, an embankment with a puddle clay core is less than 15m in height (although some were built as high as 34m), has an upstream slope of 3h:1v, a downstream slope of 2h or 2.5h:1v, and a narrow central core of puddle clay which is keyed into the underlying bedrock strata through the cut-off trench (BRE, 1999). The core itself was constructed in typically 150 to 200mm thick layers and ranges in width (BRE, 1999).

Typically, the clay that was used for the core depended on the materials available close to the dam, with local borrow pits within the reservoir footprint itself often used. In some cases, the as-dug material was used and, in other cases, materials were mixed, again typically with other local materials. In London, the cores are typically formed of reworked London Clay Formation.

Freshly laid puddle clay has the consistency of very soft clay (colloquially likened to toothpaste) and an undrained shear strength of around 8 to 10kPa. It is noted though that the consistency and undrained shear strength will generally increase with time due to settlement and a reduction in water content. Long term undrained shear strength in excess of 20kPa is typical (Reeves & Cripps, 2006). The water content of puddle clays derived from London Clay Formation is generally between 40 and 50%; water contents less or more than this may be a sign of defects within the core. Defects within the puddle clay core may occur due to a variety of factors, including:

- Construction methodology: Potential contamination of the core from poor construction practices, such as the use of timber shoring, may create voiding enabling seepage pathways.
- Construction methodology: The installation of pipes or culverts may lead to a 'cold joint' between the core and the structure which may create a seepage pathway.
- Construction methodology: Differential settlement following construction may cause fracturing (or 'cracking') of the core and develop associated seepage pathways.

- Drawdown of the reservoir: Desiccation of the core during a period of prolonged drawdown (as ensued occurred World War Two) may ensue, leading to fracturing (or 'cracking') of the core with development of associated seepage pathways.
- Vegetation: Tree roots which penetrate the core may cause desiccation of the clay leading to fracturing (or 'cracking') of shrink-swell prone clay cores and creating associated seepage pathways.

Once defects enabling seepage pathways occur in the core, the bulk permeability increases and effectiveness of the core decreases exponentially. Furthermore, the physical movement of water through the seepage pathways, or the chemical weathering associated with it, may cause the fractures or voids to increase in size, join up and potentially cause a failure of the dam. This is the worst case, however Charles (1989) notes that the rate of seepage through a core is generally small and therefore chemical weathering is limited.

GROUND INVESTIGATION THROUGH PUDDLE CLAY CORES – INDUSTRY REQUIREMENTS AND BEST PRACTICE

In order to adequately and efficiently inform the design of the defect remediation, and where possible, investigate the cause and extent of the defects, ground investigation will be required to be undertaken. The nature and extent of the defect may be investigated through the use of non-intrusive methods, i.e. geophysics, but to obtain geotechnical parameters for use in design, intrusive investigation through the use of boreholes and cone penetration testing is often the best course.

It is imperative that the integrity of the dam is safeguarded during any intrusive GI in order to reduce the risk of puncturing the reservoir core and enabling dam failure. Therefore, various organisations (most notably the BRE in the United Kingdom) have provided industry best practice and guidance associated with this activity. Recommendations that are relevant for undertaking GI though puddle clay cores are summarised below. It is noted however that although the information provided below is 'best practice' there may be situations where the recommendations below may not be applicable and alternative methods may be required. BS 5930:2015 'Code of Practice for Ground Investigations', as well as the 'International Levee Handbook' (CIRIA, 2013) provide useful summaries of intrusive and non-intrusive techniques.

Experience

BRE (1996) notes that it is essential that GIs are carried out under the supervision of a geotechnical specialist (i.e. engineering geologist or geotechnical engineer) acting for the client, who is experienced in investigations through dams. This specialist can then ensure the clients objectives are met, including ensuring the safety of the dam is not impaired by the investigation; and confirming that the standard of work is as expected and that the required technical information is gained from the investigation. The Federal Energy Regulatory Commission Division of Dam Safety and Inspections (FERC) (2016) builds upon this and provides recommendations for the minimum qualifications required for the client's specialist, noting that the specialist should be qualified by a combination of education, training, and experience. FERC (2016) also recommends that borehole/drill rig operators must have a minimum of five years of experience in undertaking boreholes and be able to demonstrate clearly on their CV that they have embankment dam experience.

Intrusive Works

Vertical boreholes are a common method of investigation through puddle clay cores; the verticality of the borehole must be monitored throughout advancement to reduce the risk of deflection and the possibility of puncturing the core, which can have serious consequences with regards to dam stability (BRE, 1996).

Rotary drilling using air flush should be used with extreme caution and is not advised in or near to a narrow puddle clay core as high air pressure may be inadvertently generated which may fracture the core (BRE, 1996); it is also the author's opinion that the logistics associated with flush disposal may also make this method unachievable on an embankment dam.

FERC (2016) suggests that cable percussion (or tool) boring is the preferred method of boreholes through embankment dams as it does not need to use any lubrication in the form of air mist or water and therefore has a low potential of causing fracturing. In smaller height embankments, windowless sampling methods may also be suitable. Keeping the general stability of the dam should be the primary concern of the works; with this in mind, the ability of the crest to support heavy plant should be reviewed beforehand (BRE, 1996), especially when undertaking works through the core itself given its low strength properties. This may also include plant or methods of investigation with high vibration. Cable percussion and windowless sample rigs have a light structure and are low vibration techniques so fulfil this requirement well.

Earth pressure can squeeze puddle clay into an uncased borehole, therefore it is required that all boreholes are cased; furthermore, this provides additional stability reducing the risk of hole enlargement and possibly dam collapse. Note though, even where a borehole is cased below a critical depth, usually about 20m, puddle clay can squeeze into the base of the cased borehole. To control this squeeze, the borehole may be supported by filling with water; note though that if the support pressure (i.e. the water in the borehole) is too high, then hydraulic fracture may be induced – it is therefore recommended not to exceed the height of the reservoir water. Note also that when a borehole contains water, the recovered material can be highly disturbed, so care needs to be taken in interpretation and testing (BRE, 1996).

Cone Penetrometer Tests (CPTs) are another commonly adopted technique due to the quick nature and low impact to the dam integrity; casing and maintaining a water head is not required with this method and it also has low vibration. It is also able to provide a near-continuous vertical profile of the soil (but is unlikely to identify specific defects) and can be used to derive a number of geotechnical parameters.

The main risk with CPTs is that they are typically truck or lorry mounted, but may be tracked, and exert pressure on to the ground through stabilisers in order to push the cone rods through the underlying stratum and undertake the test. This pressure on the dam may have significant consequences on the local stability of the dam (BRE, 1996). The pressure of the CPT plant is associated with the capacity of the equipment and its ability to penetrate through various stratum and associated stiffnesses. The equipment capacity needs to be sufficient for the ground conditions being investigated to ensure the required information is obtained. That being said, if there are restrictions on the pressure of the plant, then the required capacity of the CPT equipment may not be reached and the required information not gained.

Reinstatement

Following the completion of the GI, properly designed and carefully executed reinstatement of the ground is important to avoid changes in strength or voiding within the core and to maintain the global integrity of the dam (BRE, 1996). Reeves and Cripps (2006) recommend backfill in puddle clay cores to be approximately 20% solids (bentonite grout) but generally the backfill should mimic or be more permeable than the surrounding ground. Note that stiffer backfill in the form of bentonite pellets may be more suitable for the London Clay Formation bedrock portion of the borehole. Backfilling should be undertaken using a tremie pipe from the base of the borehole to avoid the formation of voids and the subsequent creation of preferential seepage paths (FERC, 2016).

CASE STUDIES

Introduction

Thames Water routinely undertakes geophysical surveys on their reservoir embankments, including Island Barn Reservoir and King George V (KGV) Reservoir. The results of such surveys at both reservoirs highlighted areas within the dams where seepage was likely to be occurring and thus required remediation. It is noted that neither of these reservoirs showed external signs of leakage, such as ponding, hydrophilic vegetation or slope movement, so the geophysical survey identified the seepage at the sites before surface expression occurred. Typically, the seepage was believed to be occurring at the interface between the puddle clay core and the underlying London Clay Formation – this is a common occurrence and has been cited in a number of historical case studies for London-based reservoirs.

Recommendations from the geophysical survey contractor was for the identified areas of seepage to be remediated as per timescales provided within Thames Water's internal risk assessments. It was decided by Thames Water and the appointed Qualified Civil Engineer (QCE) that undertaking remediation directly, as opposed to taking the time to investigate and confirm the seepage pathways, was the preferred way forward. Furthermore, it was deemed unlikely by the Designer that clear evidence of the seepage would be observed during an investigative GI, particularly where a balanced head is being maintained which may reduce the quality of cores and samples from boreholes.

It was therefore requested by Thames Water that a review of possible remediation options be undertaken; and it was decided by the Designer and agreed by Thames Water and the QCE that, for these reservoirs, remediation would comprise the installation of sheet pile cut-off walls through the puddle clay core of the embankments and into the underlying London Clay Formation bedrock. It was agreed that this method would meet the dam safety, effectiveness, buildability, cost, maintenance, and environmental requirements of the schemes (Rettura *et al*., 2018). In order to suitably design the remediation options and the associated enabling works, GIs were undertaken at each of the sites. The aims of the GIs were to confirm the dimensions of the core, identify the top of the London Clay Formation, confirm the construction of the embankments (by comparing the results against as-built drawings), and to provide geotechnical parameters for use in sheet pile design.

For both projects, Thames Water was the Client and Principal Designer who engaged AtkinsRéalis (previously Atkins Ltd.) as Designer for both schemes; as part of this role AtkinsRéalis designed and supervised the GI's, meeting the Client's and reservoir safety requirements. Following this, AtkinsRéalis also undertook remedial design work and supervised the installation of the sheet pile cut-off walls.

To manage the health and safety of the projects, Thames Water employed the same QCE to cover both projects; the QCE directly inputted in to and reviewed the scope of the GI, to ensure reservoir safety was protected, and provided support during the investigations.

The Principal Contractors were Costain Group (Costain) at Island Barn Reservoir and MWH Treatment (MHWT) at KGV. The Principal Contractor's role was to manage the safety of the site activities, including having an action plan in place in case any stability issues arose.

Industry guidance notes the importance of utilising a GI Contractor who has a good level of experience of undertaking works in dams and, although not explicitly stated, it is inferred that having prior experience in GI through puddle clay cores is a necessity. For Island Barn Reservoir, tenders were sent out to Thames Water approved suppliers requesting this level of experience, but the project team were unable to find a contractor that met this criterion in full. Instead, all parties of the project agreed that selecting a specialist GI contractor that demonstrated suitable years' experience on Thames Water sites as a whole was acceptable, providing they were closely monitored by the Designer's on-site supervisor. The supervisor, with the support of the QCE, had a clear understanding of the issues associated with GI through puddle clay cores and the controls needed to be in palace to mitigate risks associated with the limited experience of the available GI contractors. The same GI contractor was employed at KGV due to the experience they gained at Island Barn.

It is noted that the following case studies focus on the exploration of the puddle clay core only. Further GI's were undertaken at both sites to support ancillary works and assessment of slope stability, but these aspects are not the subject of this paper.

Island Barn Reservoir

Island Barn Reservoir is a 0.5km^2 reservoir located in East Molesey, Surrey. The height of the embankment dam is between 6 and 8m and has a crest width of typically 4.6m, with a 2.5h:1v slope on the landward side (downstream) and a 3h:1v reducing to 4h:1v slope on the reservoir side (upstream).

Despite opening in 1911, good as-built drawings showing the dimensions of the core with widths and heights were available. The drawings showed the puddle clay core to be 1.5m wide at the top of the embankment, widening to approximately 2.7m at the base of the embankment. The puddle clay core was indicated to extend thr ough the natural superficial deposits (Kempton Park Gravel Member (local river terrace deposits), and Alluvium) and keyed by 0.9m into the underlying "sound London Clay". Below the original ground level, the core was shown to be approximately 1.8m wide thinning to 0.9m at the base.

In 2016, the Designer designed a GI in order to corroborate the as-built drawings and to obtain relevant geotechnical information – most importantly the stiffness of the underlying London Clay Formation, as the Giken Silent-Piler was to be employed to install the piles. At the time of the works, this type of hydraulic push piler was only suitable for installing piles in ground with an undrained stiffness of approximately 100 kPa. The puddle clay was expected to have a low strength which would have been sufficient for the piler, but the London Clay Formation could have had a strength in excess of 100 kPa.

Boreholes were undertaken at three locations around the reservoir which were identified as leaking from the geophysical survey. The investigation included seven 150mm diameter, fully cased cable percussion boreholes (Figure 1) which were undertaken through the core to depths of up to 24.5m below ground level (bgl).

In advance of undertaking the boreholes at the top of the embankment, hand-dug inspection pits were undertaken to the top of the puddle clay core to identify the depth of the core and confirm the absence of services; two additional inspection pits were also dug on either side of the initial pit to determine the edges, and confirm the width, of the core. This would inform the positioning of both the boreholes and the cut-off wall through the centre of the core – this was imperative to confirm the thickness of the core and to avoid pushing through it.

During the drilling of the boreholes through the core, a balanced head of water was always maintained just below the reservoir level. This is more cautious that the approach given in BRE (1996) and was undertaken in order to: to reduce the risk of squeezing of the puddle clay at the base of the casing (typically being required for holes deeper than 20m); and to reflect that the borehole may intercept the leak and thus balance water pressures would prevent a sudden surge). This was deemed to be a safer method of working, as agreed with the QCE.

As casing was installed for boreholes through and adjacent to the core, the inclination of each section was monitored using a spirit level as it was pushed into the ground, to reduce the risk of the borehole tilting and puncturing the core below ground. Standard penetration tests (SPTs) were also undertaken at 1m intervals to 10m bgl, then every 1.5m. Further *in situ* and laboratory geotechnical testing was also undertaken.

The embankment and reservoir water level were monitored throughout each day by the Designer's site representative, in order to identify any anomalies associated with dam instability or leakage. None was observed during the works.

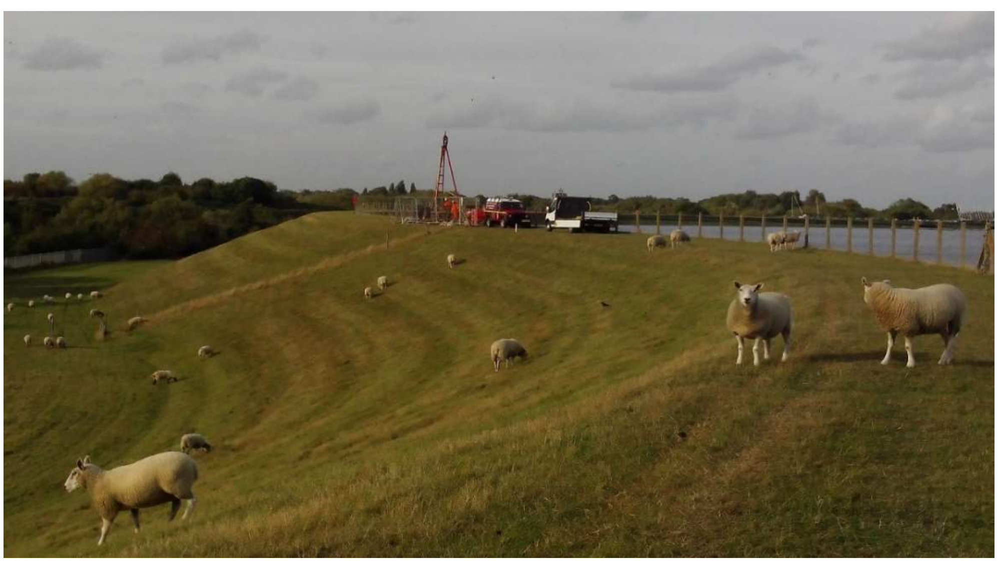

Figure 1. Cable percussion borehole through Island Barn embankment dam.

The GI corroborated the information provided in as-built drawings. The crest of the embankment was between 4.6m and 4.9m wide; the top of the puddle clay core was between 0.8 and 1.1m deep and between 1.2m and 1.5m wide. The puddle clay was found to be generally 'soft' with an undrained shear strength of typically 20kPa. The London Clay

Formation was found to be ‘firm to very stiff’ with stiffness and undrained shear strength increasing with depth (typically 57+6z kPa (where z is top of the stratum)). This is in line with industry expectations and meant a cost-effective sheet pile solution could be undertaken. Whilst the shear strength of the puddle clay was as expected, the water content was slightly lower than anticipated (at 19 to 44% as opposed to published values of 40 to 50%). Given the results of the lab testing and *in situ* testing, it was considered that the need to maintain a balanced water head in the holes did not affect the *in situ* nature of the materials.

To conclude, at Island Barn Reservoir, the work was completed with no issues reported with regards to quality of geotechnical results and dam safety. The parameters for the puddle clay obtained during the GI generally matched those provided in published literature, as did that of the London Clay Formation, allowing for the subsequent successful installation of three sheet pile cut-off walls using a Giken Silent-Piler. The success of the investigation was attributed to a number of factors including: preparation of strong scoping and works information documents which took into consideration best practice for safely investigating puddle cores in reservoir dams; the over-sight and advice provided by the QCE; the use of competent drillers and engineering geologists who took care to understand the associated risks; and supervision of the GI by the Designer.

King George V Reservoir

KGV is located in Enfield and is part of the Lee Valley Reservoir Chain. KGV is the largest reservoir in London with an embankment dam of over 6.5km long. Due to its size, the reservoir has been split into two cells, the ‘northern cell’, and the ‘southern cell’ which is separated by a windbreak embankment running east to west across the centre of the reservoir.

The height of the embankment is around 9.4m from the toe to the crest, and the width of the crest varies across the site from 3.5 to 5.0m. The gradient of the downstream slope of the embankment dam is approximately 2.5h:1v and the upstream slope is 3h:1v at the wave wall and 4h:1v towards the toe.

Historical drawings provided by Thames Water presented the Puddle Clay core as being 1.5m wide at the top of the embankment, widening to approximately 2.7m at the base of the embankment. The core was indicated to extend through the natural superficial deposits and is keyed by 0.9m (300mm) into the underlying ‘sound London Clay’. Below the original ground level, the core is shown to be approximately 1.8m wide thinning to 0.9m at the base.

The geophysical survey procured by Thames Water showed that the dam was leaking through its foundation at a localised section of the northern cell. By comparison to historical drawings, it was determined that the leak coincided with the original path of the River Lea which was diverted northwards during the construction of the reservoir in 1912 (Figure 2).

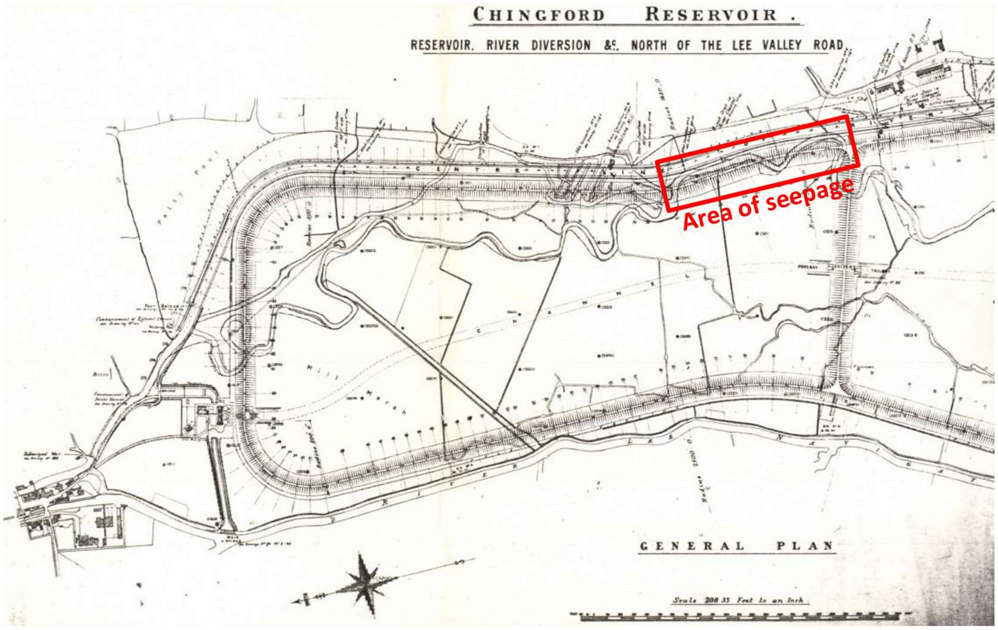

Figure 2. Historical plan of KGV.

A number of historical GIs had been undertaken on the site prior to the Designer's involvement (in 2021), which corroborated the as-built drawings. Following the recommendations in the geophysical survey report to remediate the seepage, the Designer designed a limited GI to confirm relevant geotechnical information to support the use of the Giken Silent-Piler to install a sheet pile cut-off. Two 150mm diameter fully cased cable percussion boreholes (to depths of 20m) were scheduled.

Following the success of Island Barn Reservoir GI, whilst a different Principal Contractor was involved, the same GI contractor was appointed to undertake the works at KGV (Figure 3).

Figure 3. Cable percussion borehole through KGV embankment dam (provided by MWHT).

Managing Risks for Dams and Reservoirs

During the drilling of the first borehole through the core, the driller did not maintain a balanced head of water and this was not identified by the Designer's representative on-site. Significant water ingress was encountered at approximately 15m bgl, a depth roughly consistent with the base of the puddle clay core (Figure 4); this water was under pressure (suggesting it likely came from the reservoir itself as opposed to natural groundwater) and rose up the borehole at a significant rate. The representative on site informed the named Investigation Supervisor who instructed for the hole to be plugged immediately to stop further ingress (also informing the Principal Contractor, the Client and the QCE.)

The base of the hole was plugged using bentonite pellets. The remainder of the hole was backfilled using a bentonite/cement mix slightly thicker than the puddle clay consistency (in order to displace the water in the borehole). For the duration of the remaining works, the borehole was monitored for any evidence that water ingress had continued – no further ingress or other issues with this hole were recorded and therefore no further action was called for by the QCE.

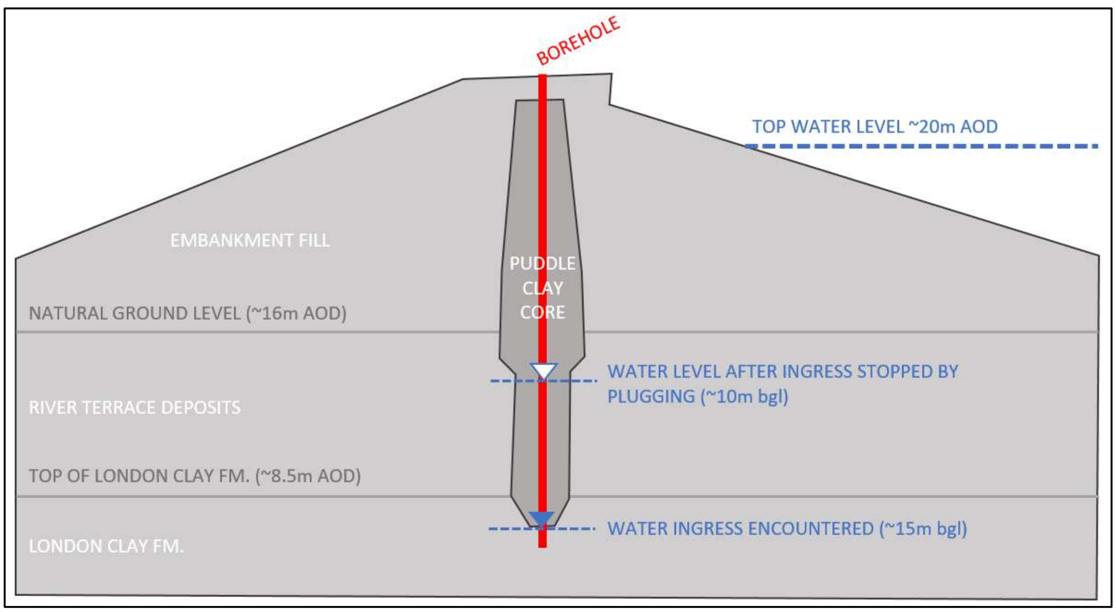

Figure 4. Schematic cross-section of the KGV embankment dam and water ingress incident.

A subsequent investigation into the cause of the incident found that, whilst the same GI contractor was engaged, a different drilling team (from that which undertook the works at Island Barn) was employed due to availability. It had been assumed that the new team had been suitably briefed regarding the specification for the works and the need to maintain a balanced head for dam safety but it had not. Furthermore, the methodology for undertaking the borehole was changed by the GI contractor without notifying the Designer or the QCE (and this was also missed by the Designer's site representative).

Following the incident, the GI contractor was not confident in their own ability to safely continue with the second borehole, even if the original methodology (successful at Island Barn) was followed. In order to limit delays to the programme (relating to demobilisation and procurement of an alternative GI contractor), an open discussion between all parties was arranged. It was agreed that changing the remaining borehole to a static piezocone CPT was suitable (given the available historical GI data) and was suitably low risk for the GI contractor.

The CPT was undertaken using a 3.5 tonne track mounted CPT rig, progressing from the base of a hand-dug inspection pit to 14.82m bgl (approximately 1m into the London Clay Formation). During penetration, the CPT rig was also able to monitor the inclination of the probe to manage the risk of pushing out of the core.

The full depth of 20m was not reached due to the restrictions on pressure on the embankment dam; this was a known and accepted risk (again there was sufficient historical and published information to produce a reasonable design).

The results of the GI generally confirmed the ground conditions expected as per the as-built drawings and historical GI – the crest of the embankment was between 3.3 and 5.1m wide; the top of the puddle clay core was between 0.8m and 0.9m deep and was around 1.65m wide. The puddle clay was found to be generally 'soft' with an undrained shear strength of typically 45kPa which is slightly higher than would be expected based on published values. The water content of the puddle clay was also higher than expected (52% to 75% as opposed to published values of 40% to 50%.

With regards to the London Clay Formation, in the limited GI undertaken, it was found to be 'stiff' with a maximum undrained shear strength of 214kPa. To combat this stiffness, an allowance for lubrication during the installation of the sheet piles was included (although during the installation of the piles, lubrication was not required).

Lastly, with regards to the old course of the River Lea, no evidence of this feature was gained due to the limited depth of the GI. Undertaking another borehole (rather that CPT) may have yielded the required evidence of the presence of the relict river channel, but this information was not essential to the development of the design for a sheet pile cut-off.

In summary, for the KGV Reservoir project, best practice was again scoped but was not fully undertaken, therefore water ingress within the borehole occurred. This could have caused significant embankment stability issues had the borehole not been cased and, whilst instigating the issue, the drillers were sufficiently competent to facilitate backfilling. Furthermore, the need for maintaining a constant balanced water head within the hole was proven as not just being important for stopping puddle clay from 'squeezing' up the base of the hole, but would have reduced the risk of pressurised water ingress.

Through collaboration between all parties, suitable alternative methods were adopted providing a reasonably good set of geotechnical results for sheet pile design; the drawback was the lack of results for the London Clay Formation, resulting in the reliance on published values and slightly conservative design.

The most important lesson to be learnt from this investigation is that, even if the GI contractor (as a company) is experienced in undertaking this specialised type of GI, it cannot be assumed that drilling crew will have had that experience. It is therefore important to ensure all site personnel are fully briefed on the requirements and sensitivity of the GI (with the site supervisor from the client/designer side being constantly alert to changes in approach which may affect safety).

CONCLUSION

Best practice guidance with regards to undertaking ground investigation through puddle clay cores while maintaining dam safety is generally good, and can be summarised as follows:

Managing Risks for Dams and Reservoirs

- It is essential that GIs are carried out under the supervision of a geotechnical specialist and that those undertaking the works are suitably experienced.
- Vertical, low-vibration percussive boreholes are the preferred method, however these need to be fully cased and, where required, a balanced water head below the reservoir water level maintained.
- CPTs are also recommended and provide a wide range of geotechnical parameters, but the ability of the dam to support heavy plant should be reviewed beforehand and the consequence of not obtaining suitable bedrock information risk-assessed.

Experience from the projects discussed in this paper show that best practice must be recognised, communicated and followed in order for geotechnical information to be gained whilst maintaining reservoir safety. The experience of the drillers and engineers must be taken into account together with that of the client/designer representative. The transfer of knowledge between these teams, both on the contractor and the design side, is imperative for the successful and safe completion of ground investigations through puddle clay cores.

There are situations where on-site geotechnical information must be sacrificed in order to ensure safety. In these cases, as-built information and published literature may be used but under the direction of a suitably qualified engineering geologist or geotechnical engineer.

Lastly, as evidenced from the case studies presented in this paper, when investigating leaks it is necessary to maintain a head of water in line with the reservoir level rather than starting to maintain a balanced head once the boreholes have exceeded approximately 20m bgl (and thus only being applicable for embankments over 20m in height) as per BRE (1996). This is because the recommendations in BRE (1996) largely refer to squeezing of the puddle clay, but do not account for any seepage or leakage pathways which may cause rapid water ingress into the borehole, as observed at KGV. Maintaining a balanced head earlier on in the advancement of the hole would reduce the risk of the significant water ingress and its associated risks.

ACKNOWLEGEMENTS
The projects were overseen by the Dr Andy Hughes who was appointed by Thames Water as the QCE for the works at both reservoir sites; as well as Jon Green, Thames Water's Head of Reservoir Safety and Asset Condition. The support and direction of Dr Hughes and Mr Green in support of Thames Water during the works has been invaluable. Special acknowledgement to Mike Hughes and João Ribeiro Correia is also given by the Author for their support during the KGV project and the development of this paper.

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The British Dam Society

Different approaches to assessing and improving stability of dam structures

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S R GOLDS, Mott MacDonald Bentley (MMB)
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SYNOPSIS Mott MacDonald Bentley (MMB) was commissioned by Dŵr Cymru Welsh Water (DCWW) to undertake dam stability works which have flexed through various approaches and different analytical tools to reduce risks and to extend the lives of existing assets. This paper covers the following projects.

Llandegfedd: a stability analysis of a combined overflow and draw-off tower, access bridge and piers under seismic conditions. The tower is a 35m tall concentric twin cylindrical reinforced concrete shell. The bridge is a 90m long reinforced concrete structure with 4No. unreinforced concrete piers with history of alkali-silica reaction.

Rosebush is a concrete arch-gravity dam. MMB undertook stability analysis, employing a 3D Finite Element model under static, thermo-mechanical and seismic loading. The seismic response was computed using fully dynamic analysis with UK-specific accelerograms generated by a tool developed in-house.

Upper Carno: refurbishment and strengthening of this double-leaf masonry structure needed to ensure both static and seismic stability. An innovative technique was employed adding fibre-reinforced concrete to the inner leaf of the masonry wall, coupled with dowels ensuring composite behaviour.

Llyn Egnant: stability analysis of a concrete gravity dam considering the effects of ice and seismic loading concluding that the above ground dam section did not meet modern design standards with further works being required to stabilise the dam.

Pond-y-Gwaith: a peat dam faced by dry stone walls upstream and downstream. Ground investigation was undertaken despite difficult access and sensitive environmental constraints. Analysis using Slope/W; rigid block analysis for overturning and sliding; and finite element structural analysis.

LLANDEGFEDD

MMB was commissioned to undertake analysis of the dam and draw-off tower at Llandegfedd to address a measure in the interest of safety (MITIOS) following an inspection under Section 10 of the Reservoirs Act 1975. Situated near Pontypool, South Wales, Llandegfedd reservoir has a draw off tower which is a concentric twin cylindrical reinforced concrete shell with an outer diameter of circa 9.75m, height of 27.5m to top water level (TWL) and a total height of 35.0m (**Figure 1**).

The seismic assessment was undertaken based on UK guidance for dams and reservoirs: the BRE publication *An engineering guide to seismic risk to dams in the United Kingdom* (Charles et al 1991) and *An Application Note to an engineering guide to seismic risk to dams in the United Kingdom* (ICE, 1998). Following the Swansea earthquake of 17 February 2018, the Qualified Civil Engineer (QCE) for the scheme requested that peak ground accelerations encountered were modelled as part of an ongoing study for information.

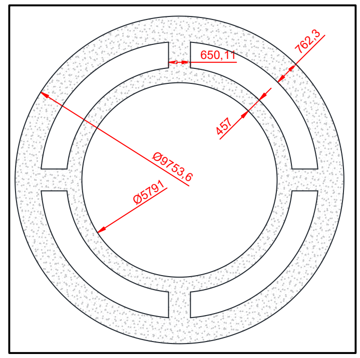
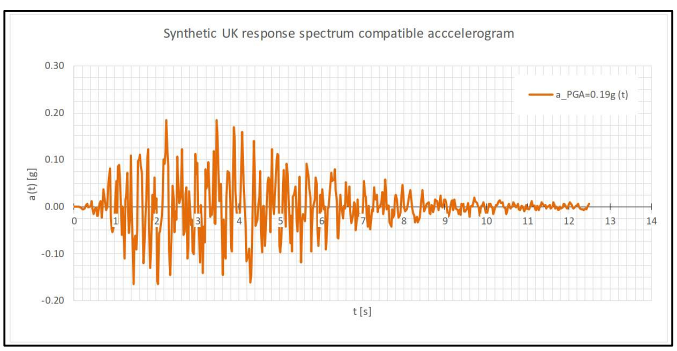


Figure 1. Llandegfedd water draw-off tower and access footbridge

The seismic response of the structure was undertaken adopting an innovative approach consisting of using accelerograms compatible with the response spectrum proposed by the BRE guide for the safety evaluation earthquake (SEE), with peak ground acceleration (PGA) of 0.19g, generated using a tool developed in-house by Mott MacDonald. This allowed for a more accurate determination of the seismic response when compared with the use of the envelope-based approach of response spectra analysis.



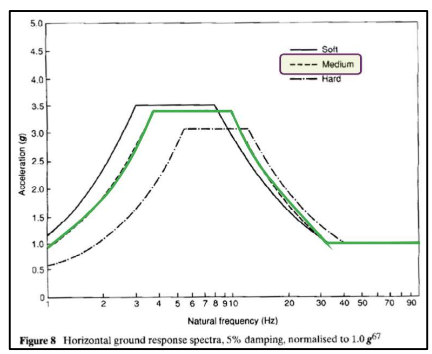
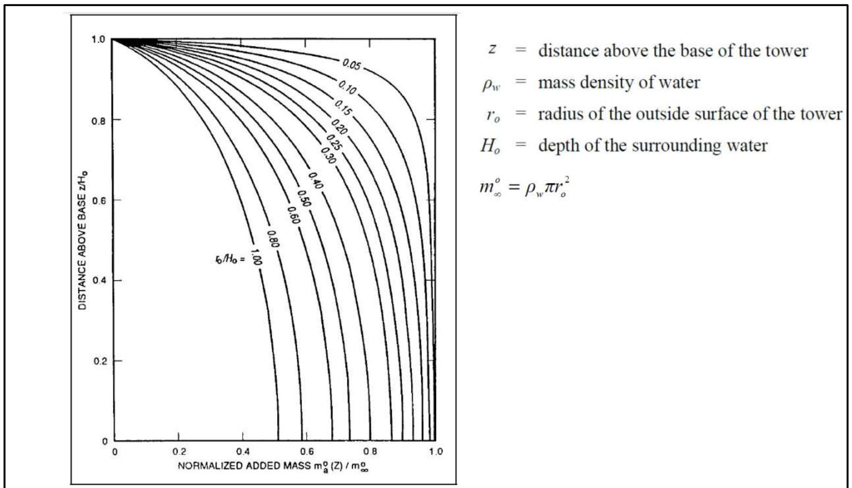


Figure 2. Synthetic accelerogram and BRE Response spectrum

The hydrodynamic effects induced by the mass of water surrounding both the draw-off tower and access footbridge piers in the case of a seismic event were modelled by adding extra mass along the height of the tower. These were derived in accordance with the expressions developed by Goyal and Chopra (1989).



Source: "Structural Design and Evaluation of Outlet Works (EM 1110-2-2400) 2003" [6]

Figure 3. Added masses to model hydrodynamic effects (Goyal et al, 1989)

The seismic response of the draw off tower was computed employing a 3D finite element (FE) model based on a fully implicit dynamic formulation, loaded with the accelerograms previously derived. Both Midas Civil (Midas, 2018) and Project Vifem (Teixeira, 2018) software were used to allow for cross-platform validation of results.

Typical outputs from the analysis can be seen in **Figure 4** below.

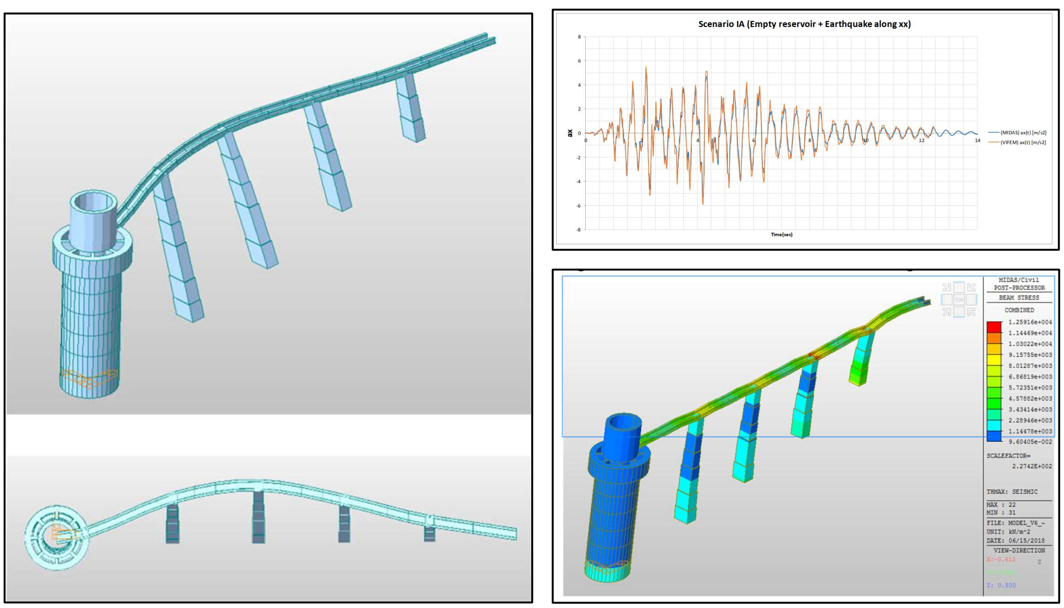


Figure 4. Seismic analysis outputs (MMB)

Historic concern associated with the asset had led to the installation of a steel bulkhead in the tunnel, in the event the valve tower was sufficiently damaged during a seismic event. The output of the assessment, along with the associated study, helped to prove that the valve tower structural performance was adequate, and no capital works were required, thus resulting in the MITIOS sign-off for the associated recommendation being received prior to the statutory date.

ROSEBUSH DAM

MMB was commissioned to undertake a stability analysis of Rosebush reservoir to address a MITIOS following an inspection under Section 10 of the Reservoirs Act 1975.

Located in Pembrokeshire, Rosebush dam was first constructed in 1931 and was subsequently raised in 1941. No calculations were known to exist to prove the suitable stability of the dam, and there were concerns that the dam did not act as a gravity structure alone but also relied on arch action for its stability.

To understand the behaviour of the structure and the significance of the spillway bridge deck to resist failure, 3D modelling of the dam by Finite Element (FE) analysis was undertaken using an efficient combination of Euler Bernoulli beam elements and shell elements (in lieu of a more cumbersome 3D solid finite elements approach), implemented in Midas Civil and Project Vifem.

Figure 5. Rosebush Dam (MMB)

Both the static and seismic response of the dam were analysed. The seismic action was modelled by generating synthetic accelerograms compatible with the horizontal ground response spectra for the UK as defined in the BRE document and ICE guide. The accelerograms were scaled to the appropriate PGA corresponding to both the SEE and the operating basis earthquake (OBE).

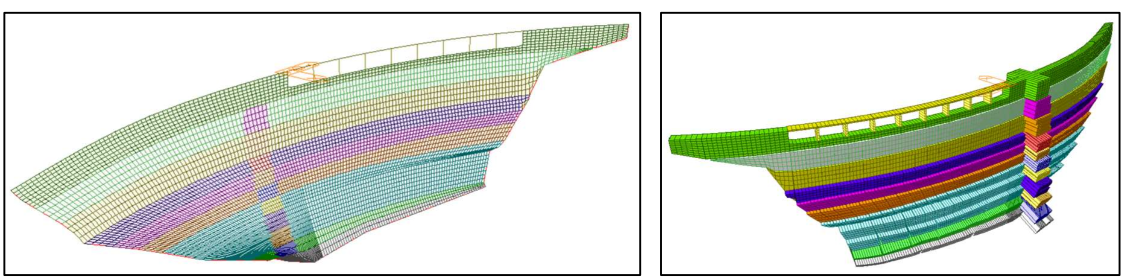
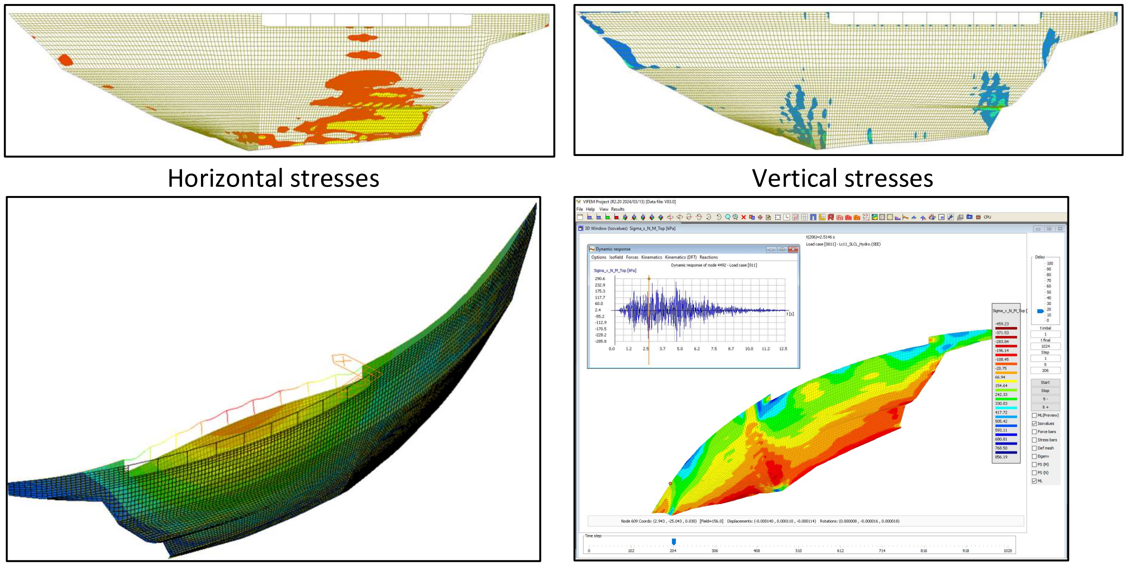


Figure 6. Rosebush Dam 3D FE model: wireframe (left) and rendered (right) (MMB)

The typical outputs of the results of the seismic analysis can be seen below in **Figure 7**



Horizontal stresses   Vertical stresses

Deflected shape   Seismic analysis

Figure 7. Rosebush dam 3D. Typical outputs of the FE model (MMB)

The adoption of a global 3D FE model based on finite elements was essential to accurately evaluate both the level of safety of the asset under normal operation, extreme flood, extreme seismic conditions and the thermomechanical impact of seasonal variations of temperature, which proved to be a governing factor. The 3D FE model was key in determining that the bridge over the spillway was not essential in contributing to the arch effect or the structural response of the dam.

The output of the assessment, along with the associated study, helped to prove that the dam was suitably stable and the structural performance adequate to prove no capital works were required. The study resulted in the MITIOS sign-off for the associated recommendation received prior to the statutory date.

UPPER CARNO DAM SHAFT

MMB was commissioned as part of a wider remediation scheme at Upper Carno reservoir, amongst which was the conversion of a semi-wet well valve shaft to a fully dry tower. The structural lining of the tower shaft was required to withstand both static loading and recommended seismic loading corresponding to an SEE with a maximum PGA of 0.22g. For wider scheme details see parallel paper by Swetman et al (2024).

The adopted solution, following an optioneering stage, consisted of an in-situ concrete lining, doweled to the existing masonry wall to achieve a composite behaviour between the new lining and the double leaf existing brickwork. The local stiffening (and strengthening) of the shaft to avoid distortion to its cross section was assured by steelwork frames placed at different levels installed top-down to act as both permanent and temporary works (Figure 8).

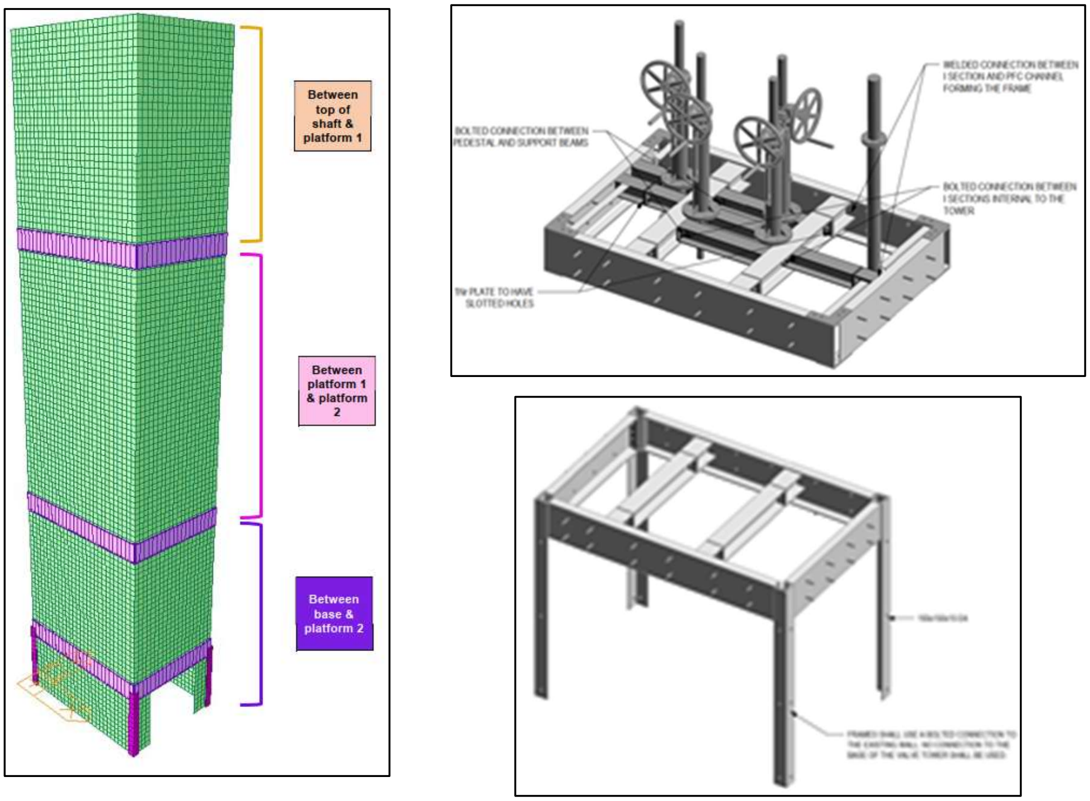


Figure 8. Strengthening zones (concrete - green , steelwork frames - purple) and details (MMB)

To accurately model the behaviour of the structure, a 3D model of the shaft, based on the FE method, was prepared using an efficient combination of Euler Bernoulli beam elements to model metallic members and shell elements to model brickwork and concrete, implemented in Midas Civil and Project Vifem.

To correctly capture the interaction effects between the shaft and the surrounding earth fill dam when conducting the seismic modelling, an ancillary FE model was prepared, which explicitly included the geometry of the dam embankment and its mechanical properties (Figure 9).

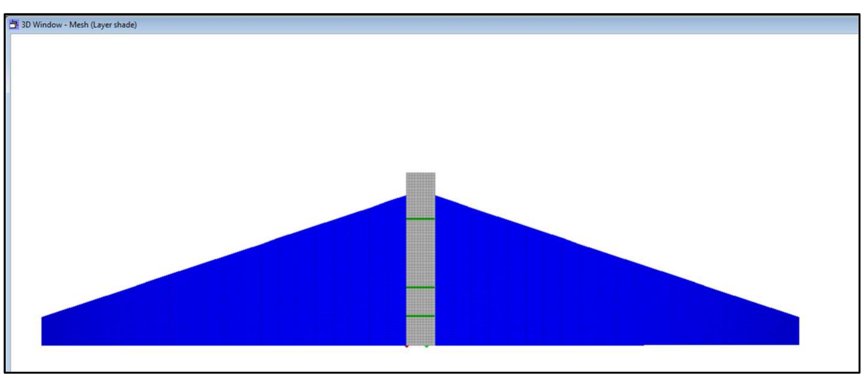


Figure 9. Ancillary FE model for dynamic analysis calibration (MMB)

The distributed spring stiffness and added mass to the shaft walls of the main 3D FE model ensured a good match with the response of the ancillary model in both magnitude, frequency and damping characteristics.

The typical stress outputs from the 3D FE model can be seen in (**Figure 10**) below.

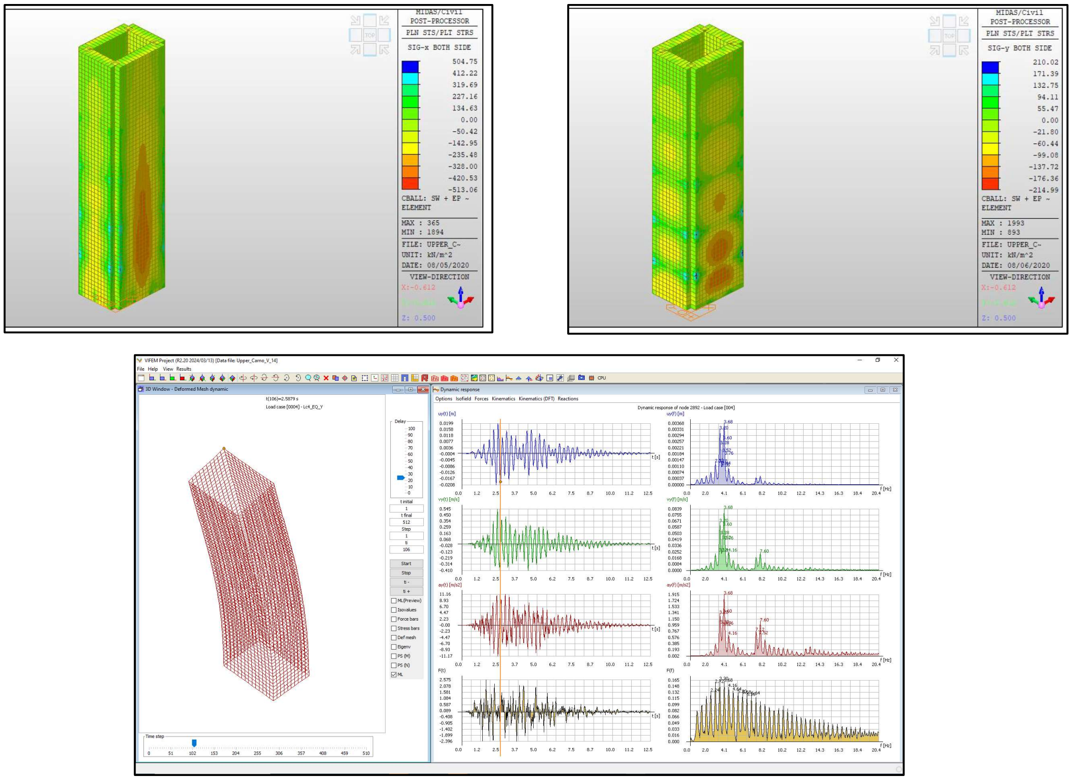


Figure 10. Shaft static and seismic analysis outputs (MMB)

The proposed solution achieved the required capacity to withstand both static and seismic actions, and acted as both temporary and permanent works, without compromising the buildability. This approach allowed the assessment of structural performance to minimise capital works and maximise buildability. The work contributed to a MITIOS sign-off for the associated recommendation received prior to the statutory date.

LLYN EGNANT

MMB was commissioned to undertake works to improve the stability of the dam at Llyn Egnant to address a MITIOS following an inspection under Section 10 of the Reservoirs Act 1975. Situated near Aberystwyth, the reservoir was constructed in 1965 by raising a natural lake. The dam is a concrete gravity dam approximately 75m in length and 12m in total height, of which only 5.6m is above natural ground. The reservoir lies at an elevation of approximately 400mAOD and supplies a treatment works downstream.

The dam itself can be considered in three parts: the central part comprises the overflow weir and spillway, flanked by two non-overflowing walls which tie into the valley sides. The dam is divided into bays approximately 7.6m in width, as shown in Figure 11.

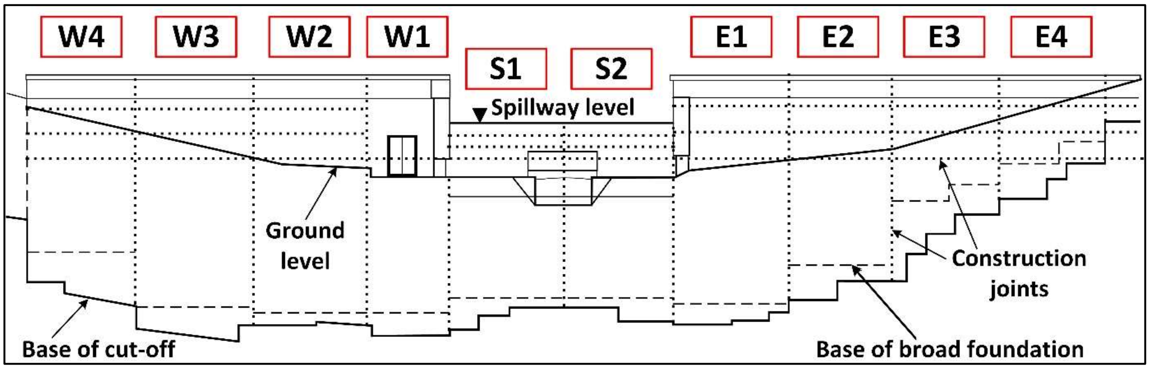


Figure 11. Downstream elevation of the dam (MMB)

For the stability assessment, two key cross sections through the overflow weir (bays S1 and S2) and the left abutment wall (bay E1) were analysed. The worst observed condition of the joints was in bay E1 which also has the greatest exposed dam height above the downstream ground level. The sections were analysed at three locations, all of which were above ground horizontal construction joints (

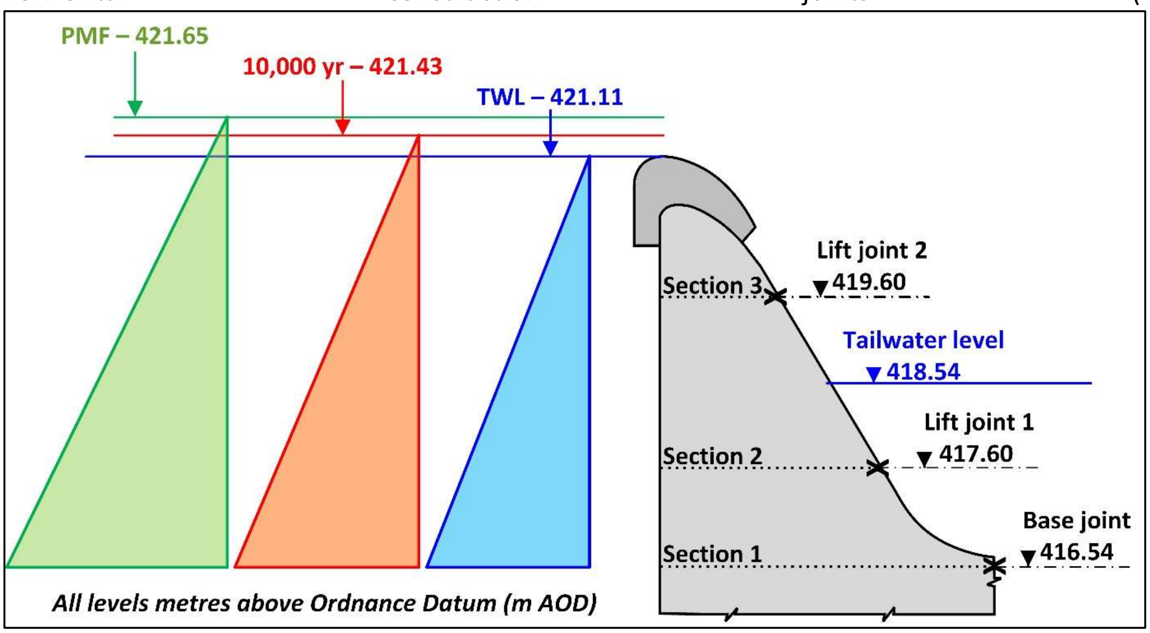


Figure 12).

When assessed against UK and international guidance, the analysis showed that the overflow weir did not achieve the adequate factors of safety (FoS) for sliding under the usual and the unusual scenarios. The FoS for the abutment walls was sufficient in all scenarios other than those including unusual and extreme ice loading.

There was an increased risk of instability both in the spillway weir and the abutment in more unusual and extreme events, where the reservoir is at risk of freezing. Results indicated that the FoS approached unity.

Whilst the risk of failure of the dam due to instability in the short term was low, mitigating measures were required to address the stability concerns under unusual and extreme ice loading.

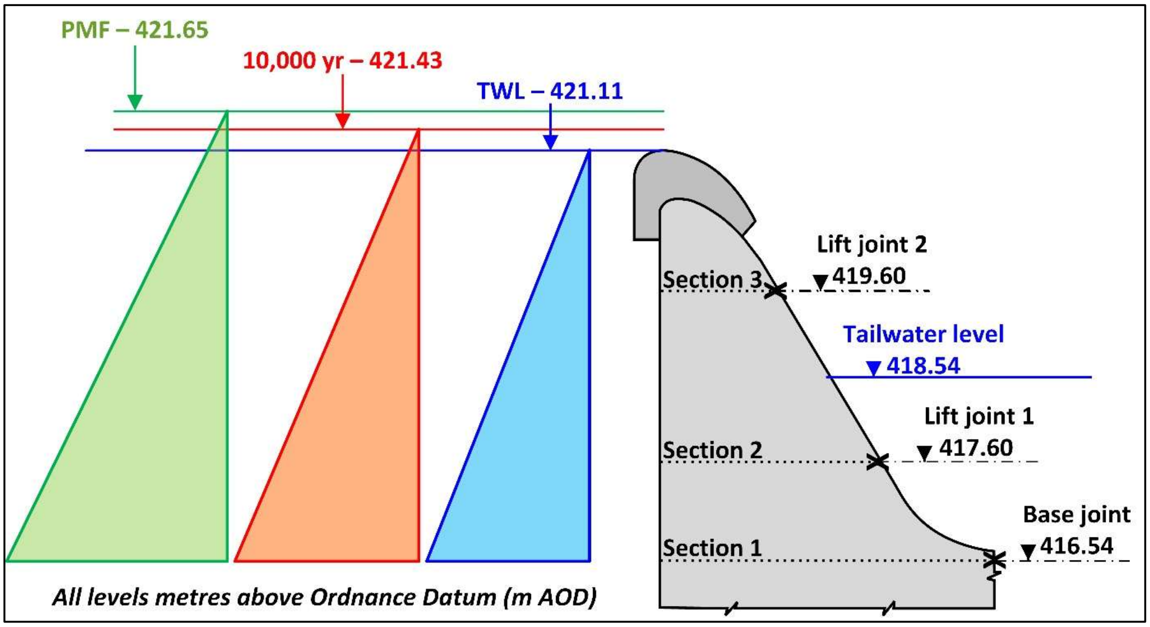


Figure 12. Hydrostatic loading on the spillway weir used in the stability analysis (MMB)

Following the stability analysis and an optioneering exercise, it was recommended to bring the stability of the dam in line with required guidance. This was achieved by installing post-tensioned anchors through the dam into the underlying bedrock.

Figure 13. Anchors being installed on the abutment walls (MMB)

Figure 14. Anchors being installed on the abutment walls (MMB)

POND Y GWAITH

MMB was commissioned to undertake an investigation and analysis of the dam at Pond y Gwaith, in Ceredigion. Constructed around 1900, the dam is 4m in height with a 38m long crest and a centrally placed spillway slab set into the dam with any overflow then passing over gabion boxes onto a concrete slab and then into the downstream channel. Although little was known about the construction of the dam prior to investigation, the results showed a peat dam faced by dry stone retaining walls upstream and downstream at slopes of 2:1 (vertical : horizontal).

Figure 15. Temporary access

Figure 16. Ground investigation showing access arrangement, ramps, plant and water level management pumping system

The geotechnical global stability assessment was undertaken in GeoStudio Slope/W 2021.3 software (GeoStudio, 2024). The Morgenstern-Price analysis type was used and slip circles shallower than 0.1m were excluded. Parameters were applied to all materials based on the results of investigations, treating the masonry walls as rock fill.

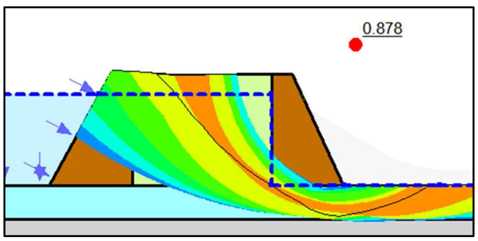


Figure 17. Left of the spillway (ignoring the passive resistance using top water level)

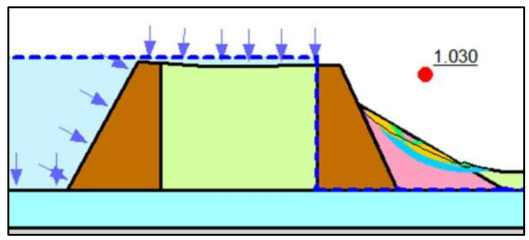


Figure 18. Left of the spillway (including the passive resistance with extreme water level)

Separately, local instability due to tensile stress was checked in the masonry retaining walls to demonstrate that the wall is sufficiently thick to carry the load. The lack of mortar in the dry-stone walls would suggest zero tensile capacity. However, due to the interlocking of the stones, limited tensile strength can be generated in the wall. The basis of this analysis is 'thrust line theory' typically used in the assessment of masonry arches. The assessment showed that the wall is sufficiently thick to accommodate the line of thrust of the load and thus transmit the load into the ground.

Two additional conceptual structural models were created for the analysis of Pond y Gwaith dam: a rigid block conceptual model and a 2D finite-element-based model. Stability sections were assessed for the left-hand side of the dam and the spillway as these represent the most critical sections.

Rigid blocks were used to model the downstream retaining wall of the embankment, similar to the design assessment of a mass gravity retaining wall. The assessment of the downstream wall accounts for the largest destabilising forces with assumed static loading consisting of that from the peat core, the hydrostatic load from the reservoir and a 5kN/m² surcharge to represent possible live loading.

The structural 2D finite element model was created using MIDAS Civil 2022 (Midas, 2022). For dynamic seismic modelling a synthetic accelerogram corresponding to a PGA of 0.125g (1.23 m/s²) was adopted. The response spectrum provided by the BRE guide was used to develop this synthetic accelerogram. An example is presented in Figure 19.

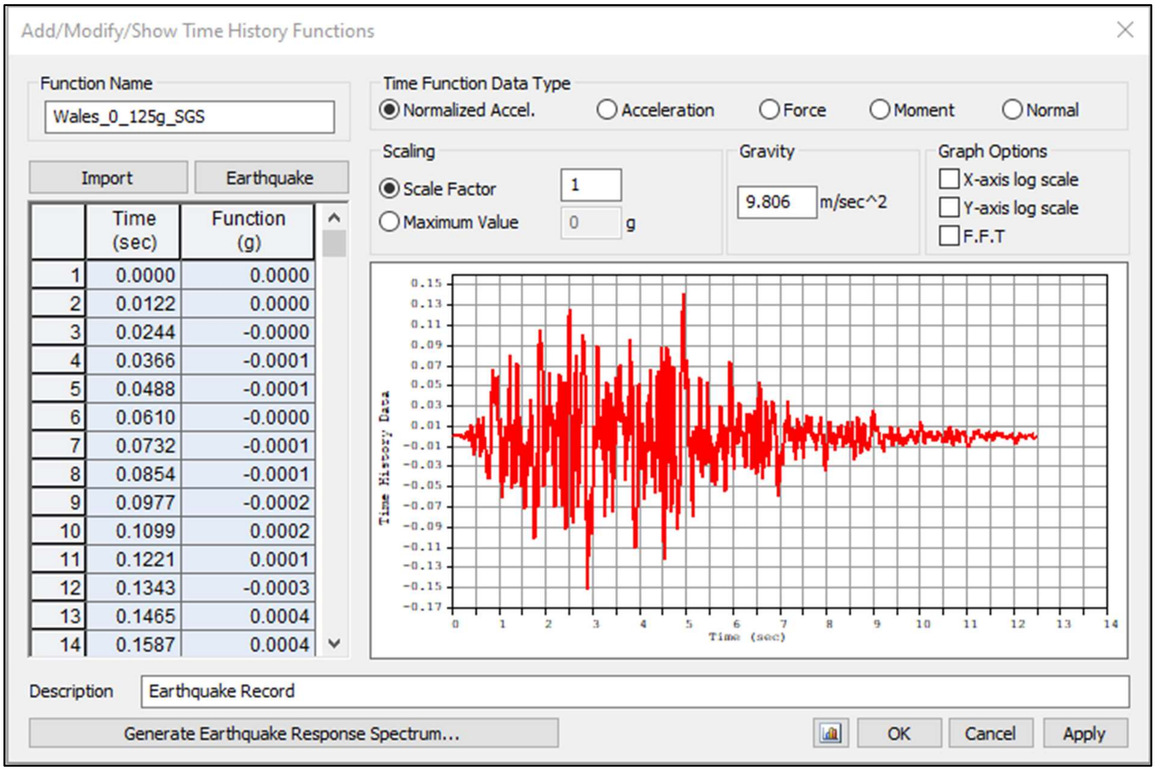


Figure 19. Seismic time history function (MMB)

It is noted that the peat is highly deformable and thus it dissipates the energy released by the earthquake and as such the dam was found to be sufficiently resilient to dynamic seismic loading.

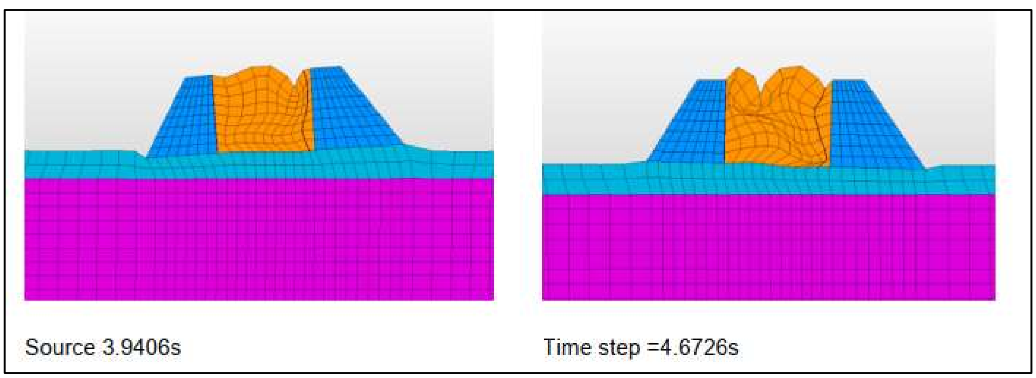


Figure 20. Dam displacement under seismic dynamic loading – displacement is exaggerated (MMB)

The FoS against overturning of rigid blocks were all in excess of 1.5 for static analysis and in excess of 1.1 for seismic analysis. The geotechnical analysis for slip circles and the rigid block structural analysis for sliding returned similar FoS close to unity for critical scenarios when ignoring the passive supporting fill on the downstream face but these were satisfactory when the supporting fill is included.

CONCLUSION

The project team as well as society more broadly have benefitted from a wide range of stability analysis techniques and skills. Making the most of advances in digital technology the project team has connected geographically hybrid teams working on remote dam sites across Wales.

Managing Risks for Dams and Reservoirs

The team has developed in-house tools used in tandem with industry standard software to cross-check results and increase understanding and certainty. In doing so the team has reduced and more accurately assessed the risk of aging dam structures while ensuring a proportionate response leading to significant cost and carbon savings.

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Novel geophysical ground imaging technology for the automated long-term monitoring of reservoir dams

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SYNOPSIS This paper covers the use of a novel geophysical investigation technique, PRIME, undertaken at two Canal and River Trust reservoirs, Slaithwaite and March Haigh. Due to concerns over seepage the Trust commissioned the surveys to try and establish the cause of the potential seepage pathways. This paper will give an overview of the 4D imagery, its methodology, and the results of which have been interpreted with the use of each of the reservoirs' known geological settings, available ground data and construction information. As with all geophysical techniques, it does have its limitations, however these surveys have provided an insight into the suitability of this technique for identifying seepages within embankment dams through long term monitoring and how it can be further developed for use across the Trust's assets.

INTRODUCTION

The technique, timelapse electrical resistivity tomography (or imaging), ERT, is a spatially sensitive geophysical method used to non-invasively image subsurface resistivity to depths of tens of metres. Electrical resistivity is a useful geophysical property for dam monitoring due to its sensitivity to compositional variations and changes in moisture content. The technology is used to generate time-lapse resistivity images, sensitive to changing subsurface conditions that are otherwise obscured. The addition of moisture to geological materials (generally) decreases the electrical resistance of the material, while a reduction in moisture content results in void space being unoccupied and therefore increases the resistivity of the soil or rock.

Managing Risks for Dams and Reservoirs

The two case studies; March Haigh and Slaithwaite Reservoirs, are presented herein. ERT monitoring has taken place on the downstream faces of the dams; 4D enabled images have been produced of the internal moisture dynamics to gain an insight into the potential seepage pathways within the embankment dams, which could ultimately, if further deterioration occurs, could cause internal erosion to potentially progress.

METHODOLOGY

In order to capture changes in electrical resistivity with respect to time we installed PRIME (Proactive Infrastructure Monitoring and Evaluation) resistivity instruments on two earth reservoir dams (see following sections). PRIME is designed to be an automatous resistivity instrument which is left (semi) permanently deployed in field conditions; the instrument can then be interfaced via telemetry and left to automatically collect resistivity measurements at specific times of day. The PRIME instrument uses arrays of electrodes connected via multicore cables, usually routed through shallow trenches/pits. This instrumentation was originally developed as a low cost and low power system to complement already existing monitoring technologies on geotechnical earthwork assets. To generate resistivity images, raw electrical resistance measurements are processed via a 4D smoothness constrained least-squares inversion algorithm described by Loke et al (2022).

Electrode arrays were custom designed to span the width of the earth dams. The electrode spacing affects the resolution of the resistivity images; closer spaced electrodes provide better near surface resolution at the cost of sensitivity at depth. PRIME has a maximum limit to the number of electrodes that can be addressed at one time (256 electrodes for the instruments used in this study), additionally the more cabling and electrodes required the higher the financial cost of deployment. Therefore the electrode spacings were optimised to provide sensitivity to the expected valley depths of the corresponding dams given associated budget and physical constraints (256 and 168 electrodes for case studies 1 and 2 respectively). The electrodes were placed on the downstream side of both dams and routed in either shallow trenches (case study 1) or pits (case study 2). Cabling was then routed into an enclosure (Figure 1 & 2) to connect to the respective PRIME systems. In both cases, the resistivity instrumentation was powered by a solar panel and battery. The electrodes comprised stainless steel spikes, with a length of 300 mm and a diameter of 8 mm. The spike electrodes were installed in small holes packed with graphite granules, which ensured an improved electrical contact with the soil.

Figure 1. Photo of the PRIME enclosure at Slaithwaite Reservoir (case study 1).

Figure 2. Photo of the PRIME enclosure at March Haigh Reservoir (case study 2).

CASE STUDY 1 SLAITHWAITE RESERVOIR

Slaithwaite Reservoir is situated near Marsden, West Yorkshire and was constructed between 1795 and 1799 by the Huddersfield Canal Company. It impounds the waters of Merry Dale Clough which is a tributary of the River Colne and has a volume of 277,400m³.

Published geology (BGS, 2003) indicates that the dam is underlain by alternating sandstones, mudstones, shales, and coal seams of the Carboniferous Millstone Grit Series. The published geological maps indicate no superficial deposits present at the site. However, it is highly likely that prior to construction, there were residual soils in the valley formed from the weathering of the Millstone Grit Series. Complete weathering of the mudstones within the series would form cohesive deposits with the sandstones forming materials of a higher granular nature. Geological mapping data combined with available ground data indicates that the rocks in the Slaithwaite area are mostly mudstone with occasional sandstone. Due to the date of which the embankment was built, it is highly probable that material was sourced locally from within the valley. This would suggest that the embankment is made of the completely weathered solid geology and associated residual soils. Ground investigations were completed in 2020, 1989 and 1974, which covered the dam crest and downstream shoulder and were targeted for spillway upgrade works and core location.

From the available ground data, the embankment fill is described as a dominantly sandy silty clay which is founded directly onto weathered rock. Due to the age of the asset, there are no reliable construction drawings and the presence of a "Pennine type" puddle clay core was assumed. A review of the historical geotechnical testing, specifically plasticity index and particle size distribution, indicated the presence of an engineered core. There are no known records or evidence to support a cut-off trench.

An indicative longitudinal section through the dam axis is presented in Figure 3 and shows a conceptual model through the embankment and foundation using historic boreholes and rock mapping data. The section identifies sandstone units at lower elevations in the right abutment which are interbedded with mudstones. The left abutment appears to consist of a single sandstone unit at approximate crest level elevation with the remainder of the abutment formed from mudstone. The geology of the left abutment is confirmed via rock exposures within the spillway chute. The differences between the geological sequences in the left and right abutment suggest the presence of a fault within the valley bottom. In terms of rock mass permeability, mudstones are typically known to have low porosity and low permeabilities with sandstone tending to have relatively higher porosity and permeability.

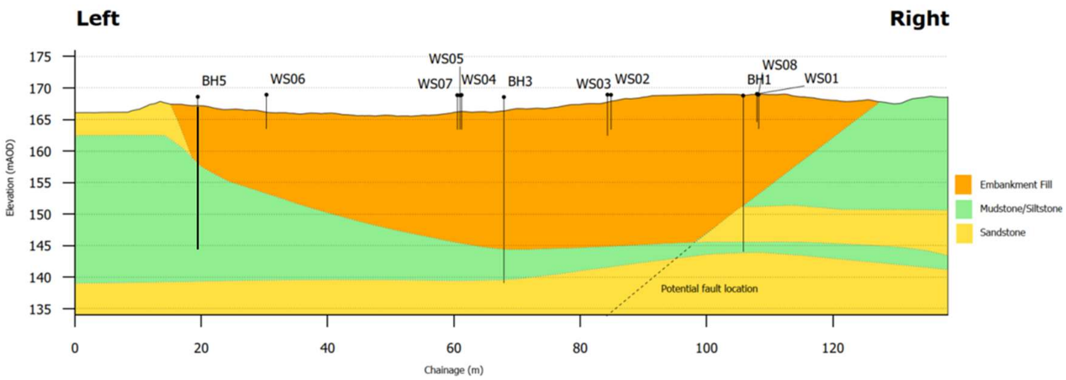

Figure 3. Slaithwaite indicative geological long section.

The outlet arrangement is typical of that found in most early canal reservoirs with masonry outlet tunnels located within the upstream and downstream embankments connected by a cast iron pipe that passes through an engineered core.

The embankment has a history of leakage with references going back to 1797 of leakage associated with the original outlet tunnel. In 1803 settlement, leakage and crushing of the outlet pipe resulted in the canal company abandoning the original outlet position and a new one was constructed which is still in use to this day. The outlet tunnel today is quite damp, and a concentrated leak appears at the upstream end of the outlet tunnel when the reservoir is within 1.2m of top water level (TWL). A telemetry-linked V-notch gauge has been installed at the back of the outlet tunnel to allow seepage flows to be continuously monitored. A PRIME survey was commissioned to contribute to an improved understanding of the leakage sources and pathways within the dam.

Installation of the PRIME system took place during July 2022. ERT lines were installed in shallow hand dug trenches to hide and protect the cables and electrodes. Electrodes within the outlet tunnel were installed as 100 x 100mm² stainless steel plates secured by masonry anchors to the soffit, with a bentonite grout between the plate and wall to ensure a good electrical contact between the electrode and the surrounding ground.

A baseline resistivity survey was conducted at a lowered reservoir level of 164mAOD. Following which, the reservoir was refilled to TWL at 167mAOD. The water level was held there for several weeks while data was continuously collected. Rainfall and leakage rate data were also collected during this period. The baseline survey (Figure 4, top image) indicated that the embankment structure displays significant heterogeneity in terms of its resistivity distribution. It is possible that this heterogeneity is a combination of both embankment material characteristics and moisture related variability. In terms of ERT interpretation, low resistivity could represent a higher content of clay or saturated material with high resistivity possibly representing a more granular material. The baseline ERT survey indicated the crest region suggests there is a transition from higher resistivities in the near surface to lower resistivities at depth – potentially indicating an increase in moisture content or clay content.

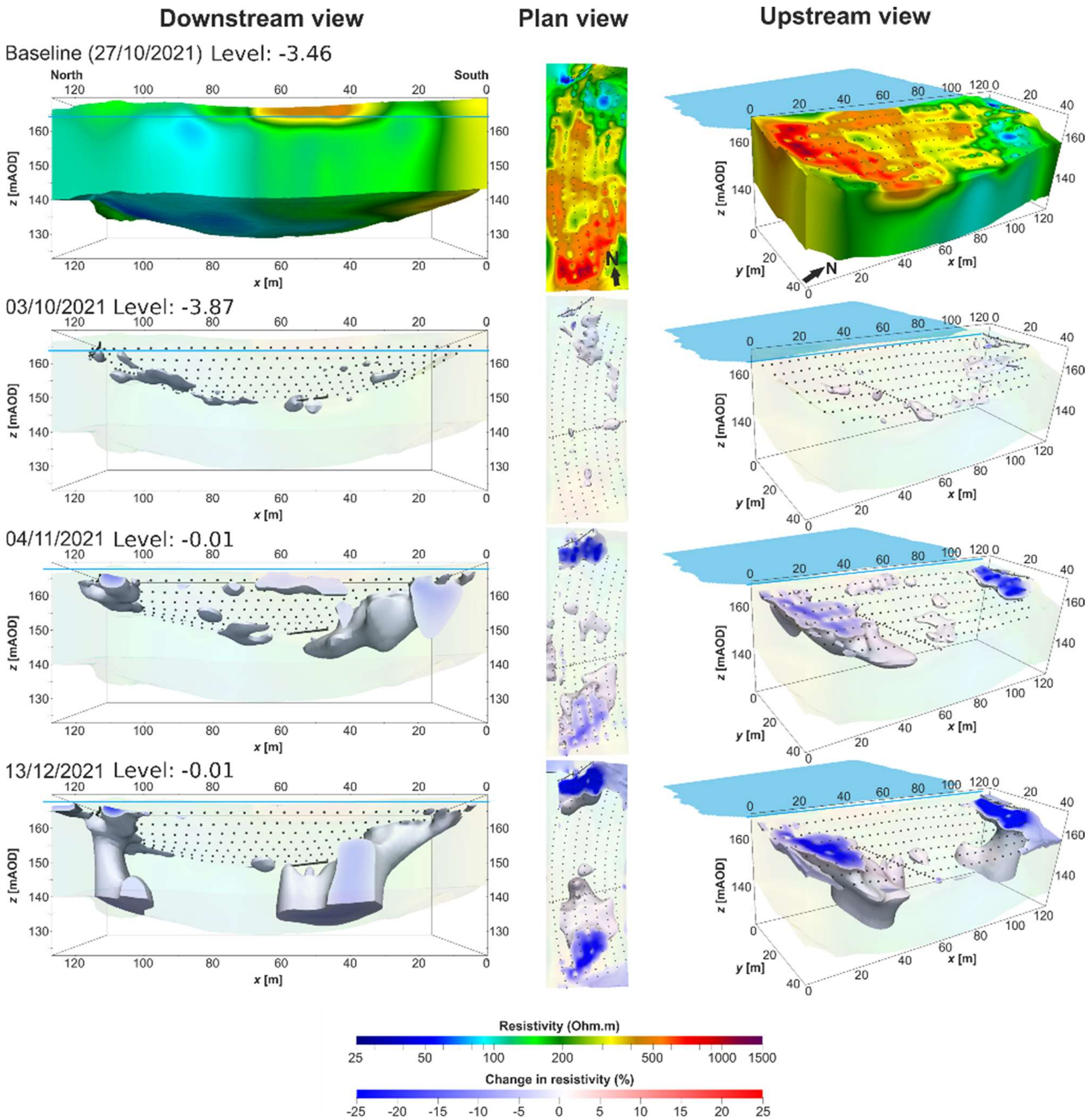

Figure 4. Baseline resistivity images (27/10/21) and a series of 'change' images representing the percentage change in resistivity ranging from 04/11/21 to 13/12/21. Iso-resistivity change set to a minimum of 2.5%. Reservoir level represented by blue line/plane.

During the raising of the reservoir level, PRIME reported significant changes in resistivity across the length of the outlet tunnel, in the dam crest and in the vicinity of the abutments. Figure 4 presents images from the time-lapse data at various stages of reservoir rise and fall. The most substantial changes in resistivity are concurrent with the rapid rise in level of the reservoir and a period of heavy rainfall in early November 2021. Figure 5 presents this monitoring period and changes in resistivity in graphical form. Reductions in resistivity during this time are initially concentrated: (1) in a thin layer in the crest region (represented as red); (2) at deeper levels within the right side of the dam (represented as green); and (3) within the left side of the dam (represented as blue).

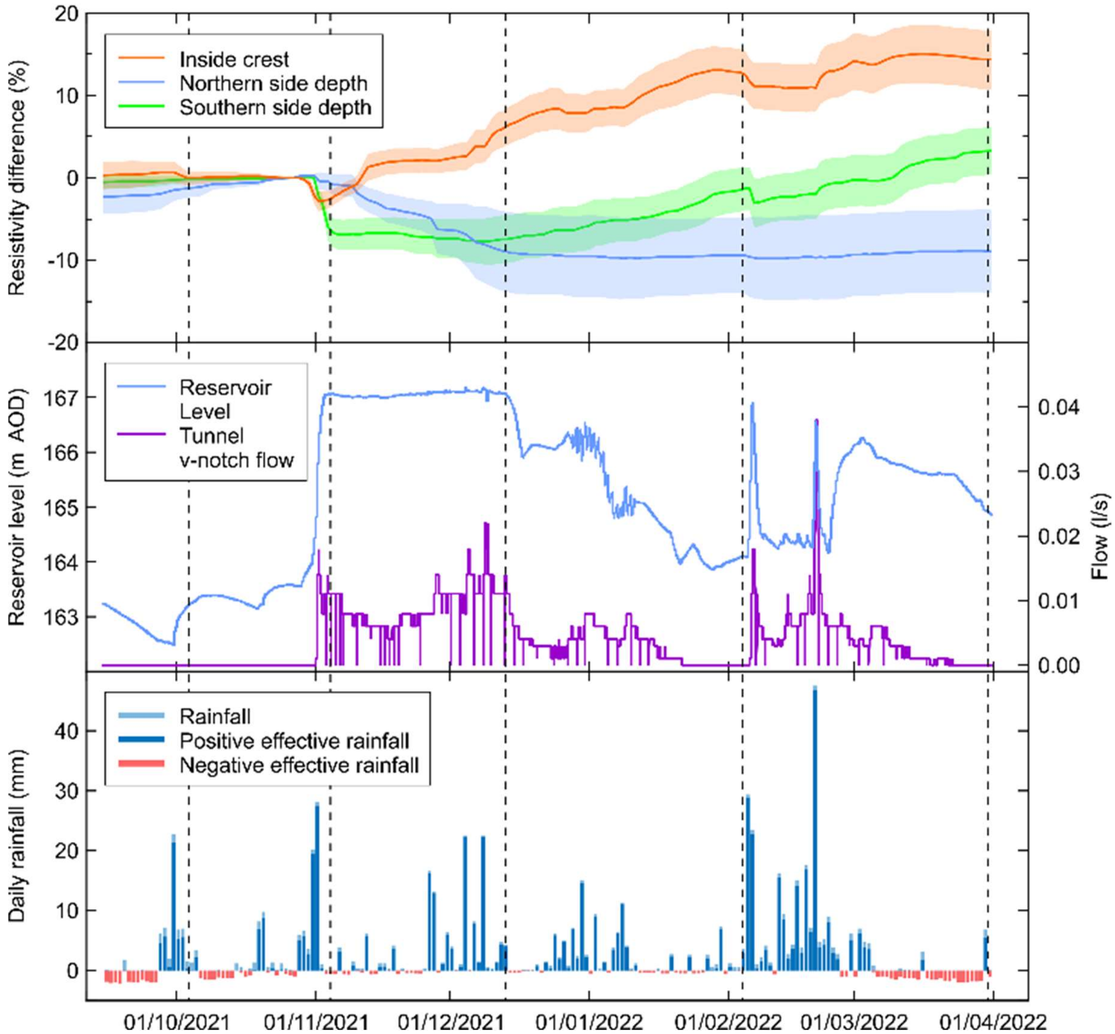

Figure 5. Selected regions of the dam for the period ranging from 15/09/21 to 31/03/22. Resistivity change, rainfall, effective rainfall, seepage flow and reservoir level.

CASE STUDY 2 MARCH HAIGH RESERVOIR

March Haigh reservoir is situated near Marsden, West Yorkshire and was constructed in the 1830s to supply the Huddersfield Canal. It impounds the Haigh Clough stream at the upper reaches of the River Colne catchment and has a volume of 275,550m³. The final constructed height was 20m, but it is thought construction was staged over several years as demand for the canal increased. Evidence of a raising can be seen in a sketch from the Early Dam Builders in Britain (Binnie, 1987) that suggests that the core is to the upstream of the current embankment crest. However, geotechnical investigations and associated lab testing does not support this.

Published geology (BGS, 2012) indicates that the dam is underlain by Upper Kinderscout Grits of the Millstone Grit Series. Observations made of the site-specific geology indicate that the left side of the valley was more shaley with the right side dominated with thickly bedded sandstone units.

Published geology does not indicate superficial materials are present in the area, therefore they are not considered to be of substantial thickness. It is likely that the dam was constructed from the residual soils formed from the complete weathering of the Millstone Grit Series.

Ground investigation at March Haigh was completed in 1999. A review of this data provided an indication of an engineered core with particle size distribution curves showing a higher proportion of fines along the dam axis when compared to the shoulder material. An indicative longitudinal section through the dam axis is presented in Figure 6 and shows a conceptual model through the embankment and foundation using historic boreholes and rock mapping data.

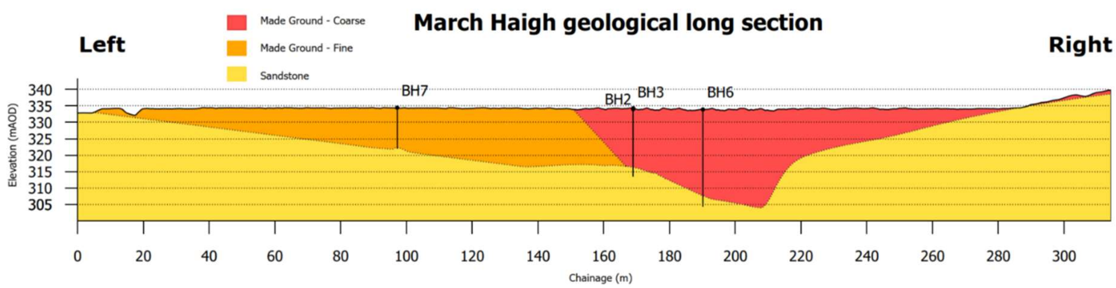

Figure 6. March Haigh indicative geological long section.

As with Slaithwaite, the dam has a typical canal-style reservoir outlet arrangement. The dam has undergone substantial settlement over the years. Following the first statutory inspection settlement was observed in the order of 0.5m over the outlet structure and a subsequent crest “topping up” exercise was completed. Disrupted pitching on the upstream face also indicates a long history of ongoing settlement and raising. Leakage in the outlet tunnel was first noted in the 1978 S10 inspection report. A programme of TAM grouting was undertaken in 1999 to remediate the issue. Leakage reduced following the grouting works but has since returned. To investigate seepages further the Trust commissioned a PRIME survey. The scale of the PRIME instrumentation is smaller than that of Slaithwaite.

Raw electrical measurements were processed in the same manner as for Case Study 1. Figure 7 presents the ERT baseline survey. There are two distinct regions of electrical resistivity in the dam and indicate the embankment-foundation contact is asymmetrical of the left-hand side and right-hand side of the embankment. The left side of the dam is more electrically conductive than the right, both being characterised by resistivities of either less than 100 Ωm or 500 to 2,000 Ωm, respectively. The lower resistivity of the left side of the dam indicates that it is compositionally different to that of the right side. This means it is likely to have a higher clay content in comparison to the right side of the dam. The apparent boundary between the regions of the dam is sharp and represents the construction methodology of the embankment where material is believed to have been sourced from each side of the valley. Weathered shales from the left are likely to contain a higher proportion of silts and clays with the right side of the valley dominated by more sandy material. This boundary also corresponds to the alignment of the outlet culvert indicating that the dam was constructed in two halves.

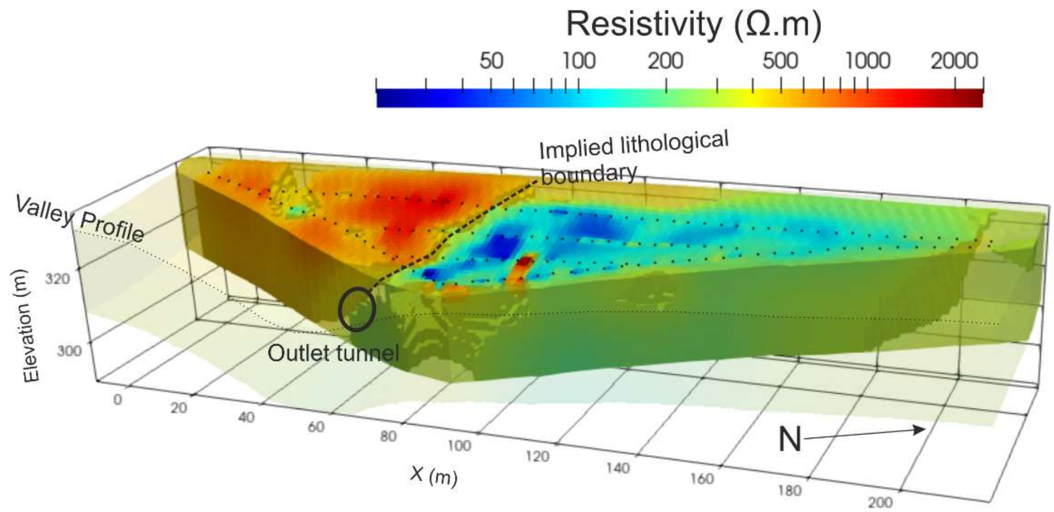

Figure 7. Baseline resistivity image of the downstream side of the March Haigh dam. The boundary between the two dominant resistivities regions of the dam (and by extension lithologies) has been indicated.

We show negative resistivity anomalies that occur in comparison to when the reservoir was recorded at 12.0m below TWL (14th of June through to 7th of July 2023). During the drawdown of the reservoir level the resistivity of the area surrounding the outlet tunnel increased, indicating this area responds rapidly to changes in reservoir level. Changes in resistivities rapidly became negative after a period of rainfall (18th to 21st of June). This was observed across the surface of the dam face, likely because of near surface moisture contents increasing due to infiltration of rainfall. Figure 8 presents the change in resistivity, noticeably decreasing in resistivity surrounding the outlet tunnel, indicating that this part of the dam has a relatively high hydraulic conductivity. The negative resistivity anomaly surrounding the outlet tunnel does increase in size and magnitude as the reservoir level recovers (7th of July through to 31st of July). However, this period also corresponds to days with elevated levels of recorded rainfall. It is therefore difficult to fully decouple the contribution of rainfall and reservoir level increase to the negative resistivity contrast. On the other hand, the rapid response of this part of the dam to reservoir drawdown and rainfall, and differing resistivities, does indicate this part of the dam has a relatively higher hydraulic conductivity. Ongoing observations of leakage made in the outlet tunnel support this hypothesis. Figure 9 shows the average resistivity (and changes) in the outlet tunnel area (in green) for the duration of the study, alongside effective rainfall and reservoir level records.

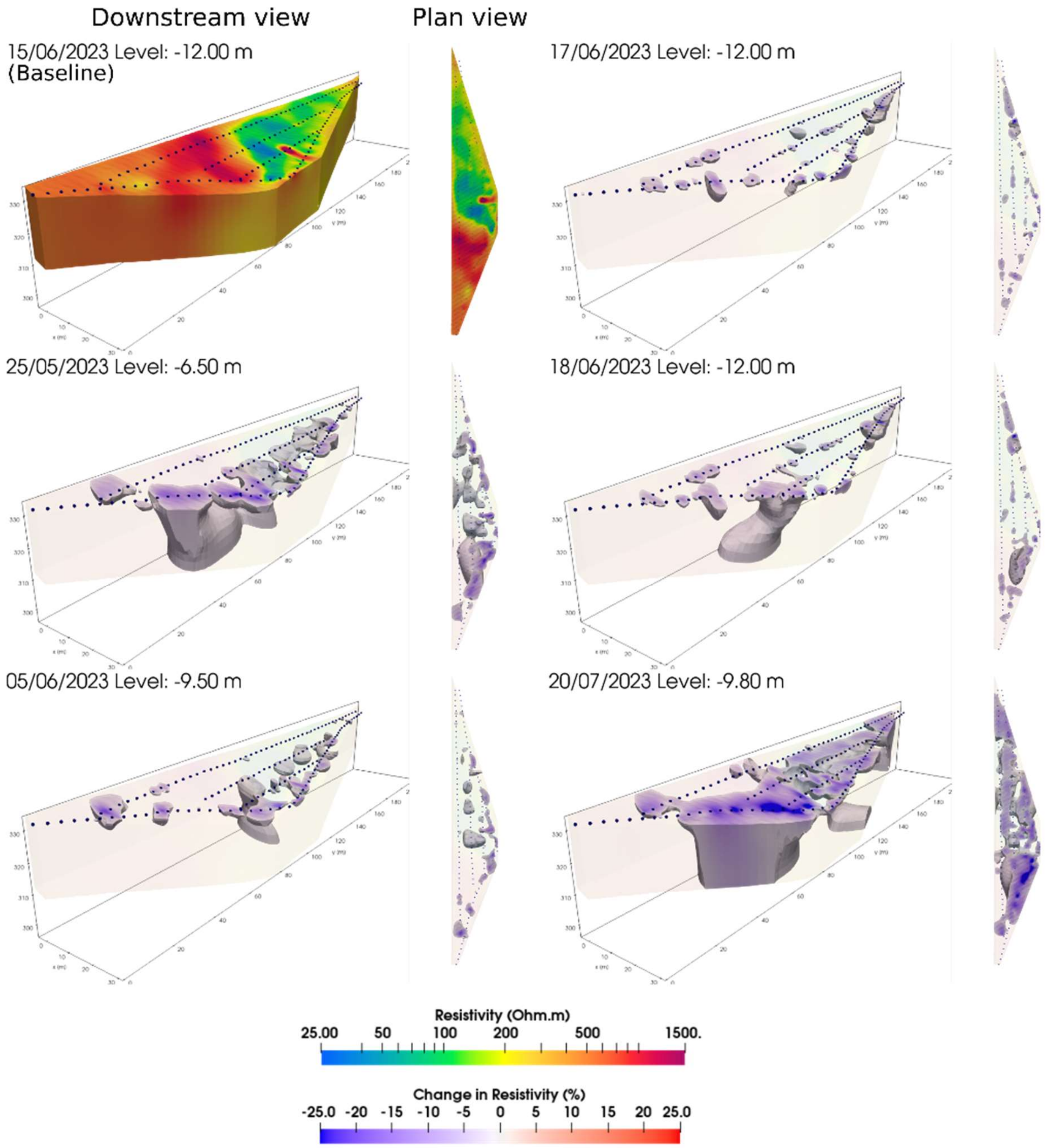

Figure 8. Baseline resistivity images (15/06/23) and a series of change images (% resistivity change) ranging from 25/05/23 to 20/07/23, focussed on the period reservoir level change at March Haigh. Iso-resistivity level in the change images set at -2.5%.

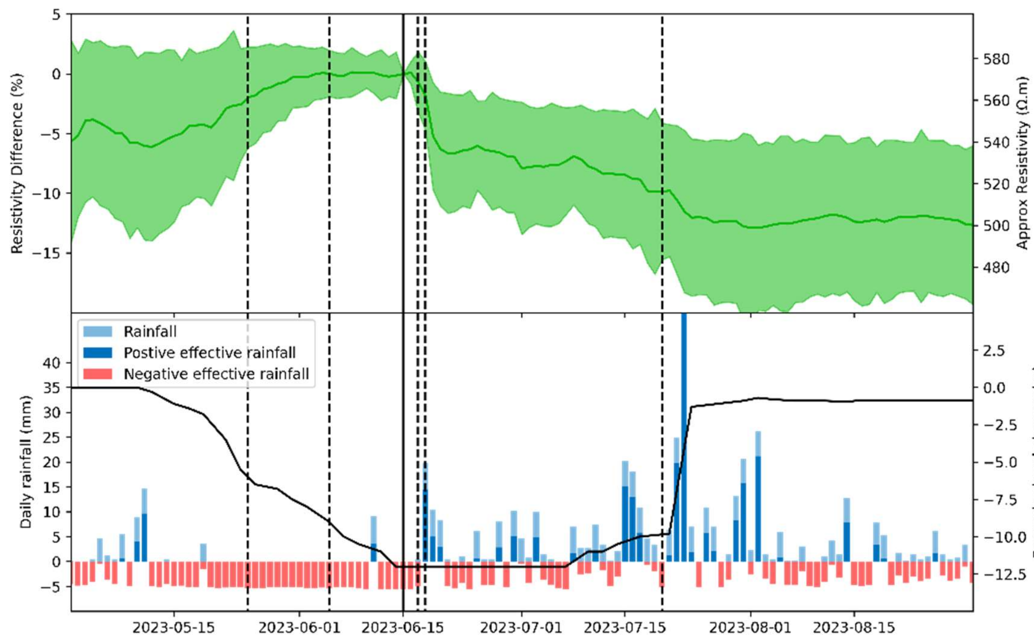

Figure 9. Rainfall, effective rainfall, reservoir level, and resistivity changes (shaded area indicates ± standard deviation) in material surrounding the outlet tunnel, for the period ranging from 01/05/23 to 31/08/23.

DISCUSSION

As with any investigation technique, geophysical survey methods are known to have limitations in their application. The extent of the technique is subject to the array of nodes placed on site. In areas where the site is constrained, this may not always extend outside of the query area to provide control points. In addition, the quality of the resolution decreases with depth, and therefore useful information may not be retrieved for dams in excess of 20m. Downhole sensors could be installed to mitigate these effects of sensors used at the surface.

This report has shown the necessity of having initial geotechnical information available for the site to enable the interpretation of the geophysical surveys. A comprehensive geotechnical desk study including all records ranging from historic drawings to seepage monitoring data is recommended, and ground investigation undertaken if not already available. The ground model should be agreed with technical experts and this information made available to the geophysical contractor prior to commencing. This will enable surveys to be tailored to the potential ground conditions, to target areas of interest and provide maximum value in the data obtained.

March Haigh and Slaithwaite reservoir are both constructed on rock foundations and therefore highlight a distinct boundary change at the embankment-foundation contact. Where embankment dams are founded on soil, the embankment material to foundation material interface may not be as obvious within a geophysical survey due to similar material characteristics.

ERT is unable to differentiate between diverse sources of the moisture change. Although results from the survey indicate there is a relationship between the changes in resistivity and reservoir levels, the changes in resistivity may also be a result of the infiltration of rainfall or groundwater sources from the foundation and abutments.

The ability to vary the reservoir level during the survey is advantageous. This enables the analysis of the relationship between the changes in resistivity within the embankment and the hydraulic head formed by the reservoir level. In these case studies a maximum of 4.5m at Slaithwaite and 12m at March Haigh was able to be achieved. Greater changes in hydraulic head, over longer periods of time, may provide better results and higher changes in resistivities.

There is potential that areas of the embankment remained saturated throughout the monitoring period. Completely saturated material will not show changes in resistivity, therefore not provide data. This may be interpreted that no seepage is occurring, which could lead to an inaccurate representation of the potential seepage pathways through the embankment or abutments.

The PRIME survey from Slaithwaite has indicated some regions of interest within the embankment and abutments which will be further investigated by a targeted intrusive investigation. Results from March Haigh indicate a localised area of interest around the outlet tunnel which coincides with previous remedial works.

CONCLUSION

The 4-dimensional aspect of PRIME has proven useful in identifying potential pathways of seepage and further understanding the embankment construction at both March Haigh and Slaithwaite reservoir. With the ability to change the reservoir level over time, ERT can establish a number of geotechnical aspects of the embankment and its foundations, including its composition, the embankment-foundation boundary, and potential areas of higher porosity or permeability. As discussed above, there are limitations within the current surveys which have taken place using this technique. These warrant further research and consideration when using PRIME on other embankment dams. However, this long term non-intrusive survey could be used as an early identification of changes in embankment composition which could lead to seepage. Further guidance on geophysical surveys specific to dams is needed to enable a consistent approach across the industry.

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Upper Carno: A case study of multidisciplinary remedial works to an embankment dam

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SYNOPSIS Upper Carno is a 14m high embankment dam in south Wales. Items of remedial work had been identified by Dŵr Cymru Welsh Water (DCWW). Investigations undertaken by Mott MacDonald Bentley (MMB) to inform the remedial works highlighted risks, and a subsequent Section 10 inspection resulted in eight measures in the interest of safety concerning the spillway condition and capacity, embankment stability, and drawdown condition and capacity. The resultant suite of remedial works required careful management of interfaces between the various workstreams throughout design and construction, to reduce the risk of failure to acceptable levels and to improve the working life of an aging asset.

This paper outlines the arc of the project and highlights the importance of developing the permanent works, temporary works and dam safety construction risk management together, given their entwined relationship throughout the whole of the project.

BACKGROUND

Upper Carno is the upper in a cascade of two impounding reservoirs situated in south Wales. The 0.34Mm³ reservoir is retained by a single dam, owned and operated by DCWW. The dam was constructed around 1875 to supply industrial customers in the town of Ebbw Vale, approximately 2.5km downstream, via a treatment works situated immediately downstream of the dam. The treatment works has long been demolished, and the reservoir now supplies raw water to Carno WTW via the downstream watercourse and the lower reservoir. The reservoir has a surface area of 0.063km^2, an operational top water level (TWL) of 444.54mAOD and a total catchment area of 5.1km^2.

The dam at Upper Carno is a 14m high, 270m long Pennine-type embankment (Figure 1) with a central puddle clay core. The reservoir is fed via direct and indirect catchments and has the facility to divert some indirect catchment flows around the reservoir in a bywash channel which discharges to the spillway. An overflow weir is situated at the left abutment, at the top of the 190m long spillway. The original draw-off arrangement was via a valve tower in the upstream shoulder and a brick-lined tunnel containing a supply pipe with an offtake to Carno WTW, which ran in a straight line underneath the embankment from the valve tower to the dam toe, and then bent to follow the line of the toe towards the spillway and discharged to the downstream end of the spillway.

The reservoir has a history of remedial and improvement works, most notably, the spillway capacity was upgraded and a section of the toe of the dam stabilised in 1986.

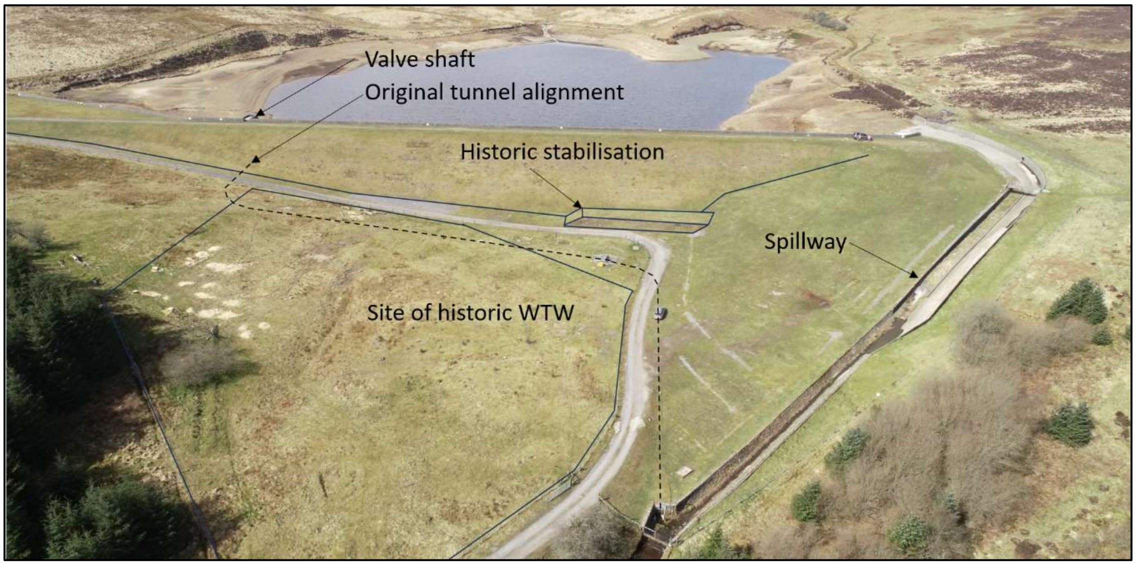

Figure 1. Upper Carno before works (MMB)

INVESTIGATIONS & DESIGN DEVELOPMENT

A Dam Safety Asset Survey undertaken by DCWW had highlighted that much of the pipework and valves at Upper Carno were in poor condition or inoperable. There was a risk that failure of any part of the system could result in an uncontrolled release of water from the reservoir and damage the dam. MMB was appointed to refurbish the existing draw-off system by repairing or replacing valves and pipework as required to reduce the risk of failure to an acceptable level.

A separate scheme was concurrently released to MMB to undertake minor repairs to the masonry spillway and brick-lined tunnel, to refurbish a system of French drains installed as part of the 1986 works, and to carry out a formal risk analysis of failure of the embankment by performing ground investigation and stability analyses of the upstream and downstream slopes.

Embankment stability

A 2016 Section 10 report noted no excessive settlement of the embankment, and the embankment was seen to be in good condition. However, the steep 1V:2H slope of the downstream shoulder was a cause for concern, and there were indications of poor drainage near to the toe of the highest part of the dam.

It is likely that the stability of the downstream shoulder of the dam would have been reviewed as part of the 1986 stabilisation works, however there is no available record of such assessment, and as such, a stability analysis was recommended. Whilst the wider scheme was being designed and constructed, the reservoir was subject to a precautionary drawdown to a minimum of 3m below TWL, informed by temporary works assessments, as a proactive measure to maintain stability factors of safety within tolerable limits and to help minimise the risk of significant operation of the spillway.

Ground investigation

No previous intrusive ground investigations are known to have been undertaken at Upper Carno, thus a comprehensive suite of ground investigation and laboratory testing was undertaken in two phases to inform the stability analysis.

The first phase of ground investigation was carried out during winter 2018 by Geotechnical Engineering Ltd and comprised 7no. dynamic sampling with rotary core follow-on, 7no. hand-dug pits, 1no. observation trench across the dam crest, and 4no. machine excavated trial pits. Eleven permanent piezometers were also installed.

Peat was found under the toe of the dam. Following review of the initial phase of ground investigation, an additional 4no. hand dug pits and 6no. machine excavated pits and testing were undertaken in August 2019 to further understand the extent and nature of the peat.

Slope stability analysis

The stability of the upstream and downstream slopes was assessed using the Spencer method in Geostudio SLOPE/W for the normal and flood conditions (reservoir level at top water level and probable maximum flood level, respectively), and pseudo-static conditions. A combined SLOPE/W and SEEP/W model was used for a rapid drawdown analysis of the upstream shoulder. The load conditions modelled and the target factor of safety (FoS) acceptance criteria were in line with UK and international guidance.

The results indicated that the dam at Upper Carno did not meet the required FoS in the downstream shoulder under normal, flood and pseudo-static conditions. The upstream slope was found to meet acceptance criteria in all cases.

A number of options for improving the downstream shoulder stability FoS were considered. Options that were considered feasible and found through further stability analysis to meet the same acceptance criteria were: slackening of the downstream shoulder; construction of a berm; and permanently reducing the reservoir TWL. The preferred option was agreed as slackening the slope from 1V:2H to 1V:3H over the full height and length of the downstream face. This option allowed the installation of a filter between the existing and new works over the full height of the structure without extensive excavation to the existing embankment face, thus preventing failure by internal erosion. A typical cross-section of the slope stabilisation is shown in Figure 2.

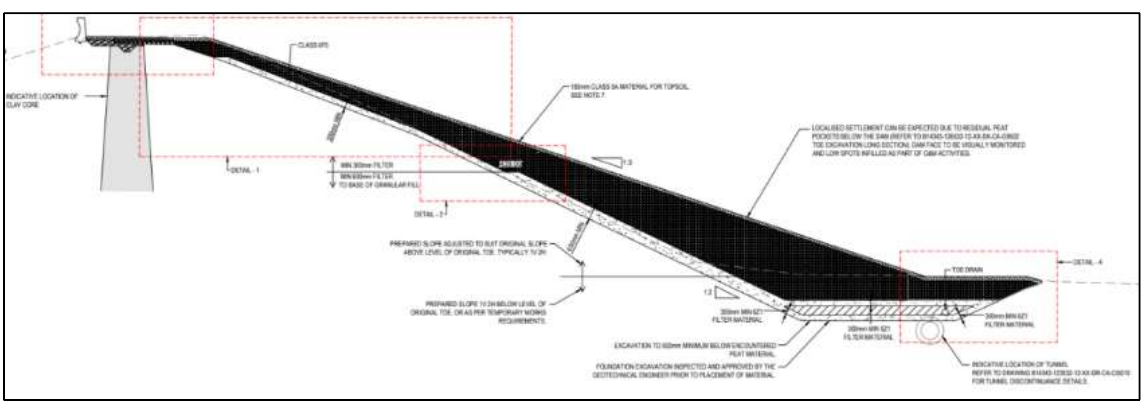

Figure 2. Slope stabilisation typical cross-section (MMB)

Drawoff system

One of the biggest challenges with working at historic assets is often an absence of record drawings. For the Victorian era Upper Carno, the only drawing showing the valve tower and upstream draw-off arrangement was dated 1952, with the upstream pipework labelled as “assumed alignment”.

The original draw-off arrangement consisted of a brick-lined valve tower in the upstream shoulder, separated into a ‘wet’ well and ‘dry’ well by a cast iron dividing wall, referred to as the “feather” on the historic drawing. The drawing shows the wet well being fed from the reservoir by an upper inlet (TWL -2.2m) and a lower inlet at the base of the tower. The lower inlet appeared to be direct buried, capped at its upstream end, with a tee upstand and strainer at low level in the reservoir. Upper, middle and lower offtakes at the “feather” fed a 15” stack in the dry side, which connected to the supply main running through the tunnel. There was a single gate valve at each valve tower offtake and no control on the upstream reservoir inlets. A short scour pipe discharged directly from the wet well into the upstream end of the tunnel.

There are no records of the reservoir being completely drawn down or the wet well being emptied below the middle inlet. The water level in the wet well was always reported to match reservoir level, including when the reservoir level was below the upper inlet level (TWL -2.2m).

A series of surveys and investigations were undertaken to understand the arrangement of the reservoir inlets and the and condition of the pipework in the tunnel and tower.

An underwater survey of the wet well carried out by Edwards Diving Services (EDS) found that the lower reservoir inlet did not discharge into the wet well, as indicated in the 1952 drawing, but passed straight through the wet well, connecting directly to the supply main. It was therefore concluded that for reservoir levels above the middle offtake that the wet well was filled by reservoir head driving flows through the lower inlet, upwards through the stack and out through the middle inlet into the wet well, and thus the reservoir could only be drawn down using the scour to empty the wet well as low as the middle inlet, in a ‘loop-the-loop’ arrangement (Figure 3). The upper reservoir inlet was confirmed as indicated on the 1952 drawing during partial drawdown, but several underwater searches were unable to locate the lower inlet.

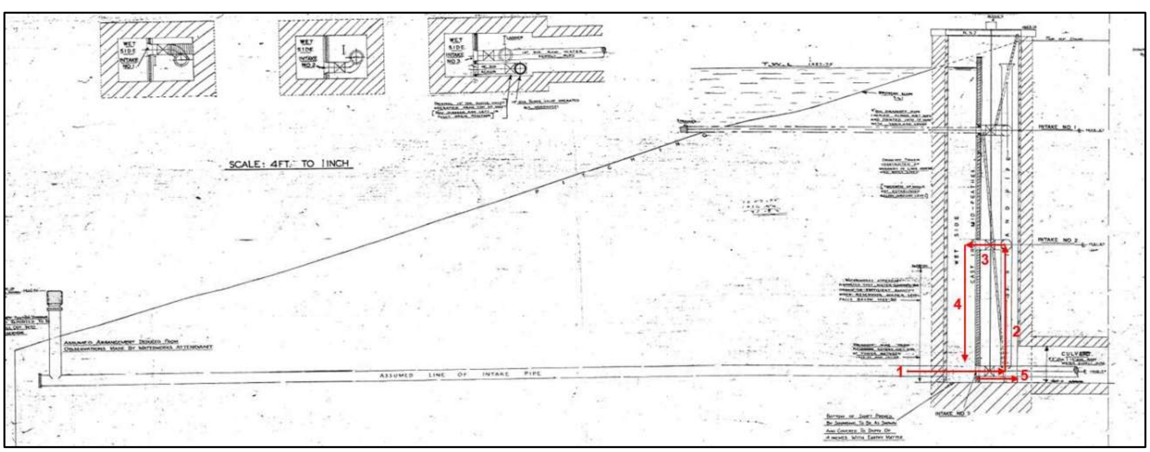

Figure 3. Assumed ‘loop-the-loop’ draw-off arrangement. Extract from 1952 drawing (DCWW)

Visual inspection and non-destructive testing of the supply pipework in the tunnel by MMB during a tunnel inspection in October 2018 found the pipework to be extensively corroded with considerable pitting. The flange joints were noted to be perished and the bolts corroded.

Valve tower

The valve tower was a congested space, with only half of the 2m x 3m plan area being the accessible dry side, which also included a pipe stack and valves restricting access considerably (Figure 4). It was therefore proposed to remove the dividing wall and convert the tower to a fully dry tower and to provide new access metalwork ladders and landings to ensure safe operation for future use.

Coring at the top of the valve tower found that the structure was formed of a double skin of masonry with an infilled cavity. Conversion to a fully dry tower therefore needed to provide watertightness. It was also important to consider composite behaviour between a new structural liner and the existing structure to maximise working room. Although not thought to have been designed to provide structural support, the 2" thick cast iron "feather" was embedded in the valve tower brickwork and it is possible that it acted as a structural component of the tower and its removal could have affected the integrity of the structure.

Tunnel

The original scope of works relating to the tunnel was to undertake minor repair works. However, significant water ingress and visual ovality of the tunnel directly below the embankment shoulder caused the Inspecting Engineer to express concern over the long-term integrity of the tunnel. Options to reduce the risk of failure of the 1.5m diameter tunnel were: total discontinuance and construction of a new tunnel; or structural reinforcement of the tunnel, which was the favoured option. A 1m diameter structural steel pipe with the annulus grouted was proposed to allow the brick tunnel to maintain its shape. This option would maximise the pipework flow diameter should future alterations be required.

Proposed solution

The suite of investigations to inform the design for the remedial works to the draw-off system revealed additional risks that needed to be addressed, thus the scope of works for the proposed solution was significantly greater than that at the commencement of the project.

The proposed solution for the pipework and valves, valve tower and tunnel was as follows:

- Conversion of the valve tower to a fully dry tower by removal of the cast iron dividing wall and installation of a reinforced concrete liner. Replacement of all access metalwork and pipework, including a duty/guard valve arrangement on each reservoir inlet to provide double isolation in the valve tower. The lower inlet upstream of the valve tower would be left in situ.
- Installation of a 25m long structural steel pipe in the existing tunnel under the dam shoulder from the valve tower to the tunnel bend and infilling the annulus between the pipe and original tunnel structure a with non-shrink cementitious grout. Removal of pipework and infilling of the original tunnel beyond 25m downstream of the valve tower to prevent failure as this would be below a section of the slope improvement works.

- To enable the lining of the tunnel and to route all new pipework outside of the footprint of the slackened shoulder, construction of a new access shaft over the tunnel bend and an additional shaft outside the footprint of the proposed stabilising works. Connection of new shafts by a new length of tunnel containing new draw-off pipework.
- Installation of a combined buried scour/supply main from the new shaft to the watercourse, via a submerged discharge valve (SDV) and SDV chamber to dissipate energy from scour flows through a 600mm main operating under full reservoir head. The SDV chamber discharges into the spillway stilling basin.

The drawdown capacity of the selected option was assessed to confirm its suitability in line with *Guide to drawdown capacity for reservoir safety and emergency planning* (EA, 2017), with allowances for futureproofing the system included.

Spillway

Physical investigation

The original spillway at Upper Carno was constructed on the left abutment and formed of a 3.3m wide, rectangular masonry channel, transitioning to a parabolic channel roughly halfway along its 190m length (Figure 10). Spillway improvement works were undertaken in 1986 which included the replacement of the original tumble bay with a 21.5m long crump weir and concrete tumble bay area, narrowing to meet the original masonry channel approximately 30m downstream of the overflow. From this point, the spillway was modified to comprise a composite channel formed from the original 3.3m wide masonry channel invert and right-hand wall and a 2.7m wide reinforced concrete 'L section', with a 200mm high step dividing the left and right sides of the channel and at the top of the masonry section, such that low spillway flows were directed down the concrete (left) side of the spillway, and the masonry channel would come into operation only when flows exceeded 200mm depth. The composite spillway extended as far at the start of the original masonry parabolic section, which was not upgraded as part of the 1986 works.

A visual spillway condition assessment was undertaken by MMB in May 2018. The quality of the masonry was seen to be good, with sections of the invert which required the pointing to be reinstated. There were sections of the masonry invert that had been replaced by mass concrete. The reason for these repairs was unknown but presumably due to historic events of high flows plucking out masonry blocks. The concrete elements were seen to be in generally good repair.

A geophysical survey was undertaken in the spillway chute in June 2019 by Terradat, which indicated that the masonry invert was potentially laid directly on erodible made ground and showed signs of voiding. Cores and trial holes were excavated in the spillway in June 2019 to confirm the GPR results and to understand the presence, nature and detailing of the clay core and its interface with the spillway. The findings of the site investigation showed that the masonry blocks sat directly on erodible soils and no concrete bedding was provided under the masonry spillway. The presence of concrete backing to the masonry wall was limited. With this arrangement, the residual risk of failure of the masonry section of the spillway was considered to be greater than that of the concrete structure.

The clay core was located outside the right-hand wall and at the base of the right-hand wall within the spillway. No features were observed suggesting a formalised connection between the clay core and spillway base. The investigations suggested that the spillway slab was founded partly on the clay core and partly on rock.

Modelling

A flood study was undertaken in October 2019 to inform a capacity assessment of the existing spillway, which was calculated using the "standard step method" for the Design Flood and Safety Check Flood for a Category A dam as defined by Floods and Reservoir Safety 4th Edition (ICE, 2015). The spillway is required to pass the PMF flood of $49m^3/s$.

The results for the Safety Check Flood case indicated localised overtopping of the left (concrete) wall in some areas. The right-hand masonry wall would be overtopped over its full length, and there would be out-of-channel flow on both sides in the parabolic section of the chute.

The assessment of the spillway chute suggested that, for the Design Flood condition, the chute had sufficient capacity for its length where it passes the embankment, with out-of-channel flow only in the parabolic section. This section of the spillway was located downstream of the dam and, adopting a risk-based approach, out-of-channel flows at this location may have been considered acceptable to the QCE and client.

The results from this assessment also highlighted that the masonry elements would be unlikely to withstand velocities arising from the Design Flood and Safety Check Flood.

High level options to overcome the shortfalls in condition and capacity of the spillway were considered, with the favoured options being those within the existing spillway footprint. The selected option was to replace the majority of the spillway with a modern reinforced concrete spillway, typically 6m wide, with wall heights between 1.60m and 2.75m. All masonry elements would be replaced with concrete, and the existing concrete structure left in situ and lined, utilising the historic concrete base as anti-flotation kentledge, which would reduce the concrete usage and associated embodied carbon.

The upstream portion of the spillway and overflow weir was not included in the scope of the works. Due to the partly inconclusive physical investigation in this area, the upstream spillway slab tie-in detail was designed with some flexibility to accommodate variations in locations/direction and extent of the existing core found on site during construction.

The original structure included no means of energy dissipation at the downstream end. A Type I stilling basin with downstream weir was included in the design to prevent erosion of the downstream watercourse during flood events. The length and position of the stilling basin was adjusted during design such that all permanent works are situated within the DCWW land boundary, and so that a suitable outfall for the new drawdown system could be accommodated.

Computational fluid dynamic modelling was undertaken to assess the hydraulic behaviour of the proposed spillway chute for PMF, 10,000yr and 1,000yr flood events to inform required wall heights, and to assess the hydraulic behaviour of the stilling basin for the 1,000yr flood event and its ability to dissipate energy.

CONSTRUCTION

Upper Carno is situated at approximately 440m above sea level, which means that it is subject to high precipitation and harsh weather conditions. This caused significant challenges during construction, particularly as the majority of the works required reservoir drawdown and proven isolation of the inlet pipework to be undertaken safely.

Drawdown

Temporary siphons discharging to the spillway and a pumping arrangement were installed to empty the reservoir. For further details on the temporary siphon system see parallel paper by Carruthers and McAree (2024).

Works were required to the draw-off system, within the reservoir basin and spillway, and high-risk earthworks to the embankment. As such, the construction programme was phased to reduce health and safety and dam safety risks. The temporary siphons were maintained throughout the first phase of works to the draw-off system and the slope stabilisation. With the new draw-off system installed that could provide emergency drawdown, the temporary siphons were removed and works to the spillway were undertaken.

The precautionary drawdown to 3m below TWL maintained by the existing scour pipe was sufficient to isolate the upper reservoir inlet by installing a blank plate.

The plan to isolate the lower reservoir inlet had been to remove the strainer on the upstand and affix a blank plate using divers, and then CCTV survey the inlet pipe to determine whether the upstream end of the pipe was capped as indicated on the 1952 drawing. If the CCTV survey was unable to prove isolation, a line-stop and cap-main could be installed to guarantee isolation. This would involve the installation of a temporary cofferdam in the reservoir and localised desilting. However, as the location of the lower inlet upstand had not been identified with the scour outlet reducing reservoir levels to around that of the middle inlet level, the only option was to drain the reservoir further using temporary equipment. Drawdown was further complicated by several storms and attempting to maintain the reservoir empty during winter, along with the volume of silt in the depths of the reservoir.

Upstream works

The upstand and strainer on the lower inlet eventually emerged when the reservoir level had been drawn down by almost 9m. As the water level continued to reduce, a brick arch became visible near to the upstand, which turned out to be the entrance to an upstream open culvert running to the upstream side of the valve tower that had not been indicated on the 1952 drawing or been picked up by the underwater survey of the valve tower due to silt causing poor visibility.

The scour arrangement draining the wet well was not the assumed ‘loop-the-loop’, but more simply the wet well being fed directly by the open culvert. The original supply main, which had been seen to pass directly through the wet well, was not direct buried, but ran though the upstream culvert.

The design was updated following confirmation of this arrangement. The historic pipework in the upstream culvert was removed and a new concrete plug with through pipework installed at the downstream end, adjacent to the valve tower, to maintain the open culvert whilst allowing conversion of the valve tower to fully dry (Figure 5). Once the plug was completed, safe isolation was installed to undertake the works to the valve tower and downstream tunnel.

Valve tower

The cover slab was removed from the valve tower and a temporary scaffold and lifting system was erected to enable the removal of all historic pipework and the dividing wall. Removal of the "feather" was an onerous task due to its embedment into the brick structure and the heavy weight of 3ft deep, 2" thick cast iron panels.

The valve tower was structurally enhanced by lining internally with a combination of conventionally reinforced and fibre-reinforced concrete, and peripheral steel frames at the levels of the new platforms provided additional bracing. The permanent works, temporary works and construction sequence were developed in tandem to negate the need for structural temporary works. This involved removing the "feather" and installing the structural steelwork in short lifts from the top-down. See parallel paper by Teixeira et al. (2024) for more details.

Figure 4. 'Dry' well tower access (looking up) prior to works (MMB)

Figure 5. Lined fully 'dry' valve tower (looking down) (MMB)

Tunnel and scour

To enable the works to the length of tunnel that was to be retained under the embankment, and to route the new draw-off pipework outside the new dam profile, a 7m diameter shaft was sunk 11m vertically through the embankment shoulder to intercept the existing tunnel. This was utilised to drive the 1m diameter pipework sections into the 1.5m diameter tunnel under the downstream shoulder.

There was concern regarding water ingress into this length of existing tunnel. Internally sealing the tunnel by grouting between the steel liner and tunnel brickwork diverted ingress to the tunnel exterior, where left without intervention, it could lead to internal erosion failure of the dam. To reduce this risk, a sand filter was retrofitted around the exterior of the tunnel by driving a 4.3m diameter steel tunnelling heading from the shaft into the dam foundation material (Figure 6).

A 2.4m diameter tunnel was driven between the 7m diameter shaft and a new 4.5m diameter shaft to install new scour pipework to outside the dam profile. The scour at this point was situated 8m below ground, and was therefore installed by microtunnelling for 80m length from the 4.5m diameter shaft to avoid excavating at such depths. The downstream 60m of scour was installed via conventional open cut to a submerged discharge valve and chamber adjacent to the spillway. A connection to an existing main to Carno WTW was included on the scour. For further details on the permanent drawdown system, see parallel paper by Cornelius & McAree (2024).

The remaining 160m of tunnel beyond the lined section was to be discontinued. As it had begun to deform, and the embankment stabilisation works would add more weight to the parts of the tunnel under the dam, this section of tunnel was required to be infilled. This was undertaken by adding drainage through the tunnel and infilling with an expanding polymer void filler (Figure 7). The downstream 80m of the existing tunnel not under the dam was dug down to, the tunnel soffit removed and the tunnel infilled with drainage and backfilled.

Figure 6. View from new tunnel through 7m shaft to existing tunnel during construction (MMB)

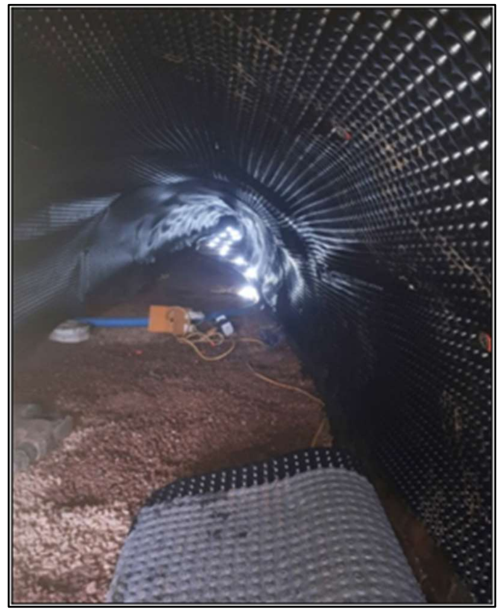

Figure 7. Drainage layer installed in tunnel prior to infilling (MMB)

Earthworks

The slope slackening works consisted of removal of the topsoil from the existing dam and excavation at the downstream toe of the dam to remove peat, up to 3m below the toe. 25,000 tonnes of 6F5 was imported to slacken the slope, a large proportion of which was excavated, crushed and graded for use at Upper Carno from a nearby new service reservoir installation project by MMB for DCWW, which had a significant reduction on the embodied carbon of the scheme and reduced the impact on the local road network.

The slackening detail included a fine and coarse granular filter placed at the interface between the prepared existing embankment shoulder and the imported granular material. The fine filter arrangement protects the dam against future internal erosion and potential failure, whilst the coarse filter acts as a drainage medium and to prevent the fine material entering the 6F5. A new toe drain within the coarse filter drain was installed (Figure 8) to help reduce the piezometric level in the downstream shoulder.

The earthworks were undertaken under reservoir drawdown and working under an Earthworks Temporary Works Risk Management Plan for the excavation of up to 3m depth of peat at the toe of the dam to minimise the risk of slope instability during excavation.

Excavation and backfilling was undertaken in a series of short bays along the length of the dam. The length of each bay was determined based on an anticipated ground model and zoned based on a red-amber-green risk system. For each bay, the toe was excavated to the required depth to remove peat, the toe drain installed and the toe infilled. This sequence was repeated along the length of the dam. Following completion of the high-risk toe excavation works, topsoil was stripped from the upstream shoulder in 500mm vertical lifts and the filter materials were placed by benching into the 1V:2H slope. Imported 6F5 was placed on top and trimmed.

Modification to the crest/core interface was also required to raise the level of the watertight element of the dam such that potential seepage pathways through the dam are reduced for flood scenarios. This consisted of a layer of cohesive material placed between the wave wall foundation and the clay core, and keyed into the clay core

.

Figure 8. Installation of toe drain and filters (MMB)

Figure 9. Works complete (DCWW)

Spillway

The MMB design and contractor team worked closely together during the detailed design phase and it was decided to replace the existing concrete elements instead of lining due to the constructability implications and risk of damage to the existing concrete walls when excavating for shear keys.

As the gradients of the spillway were not overly steep, the construction methods were considered, and it was deemed appropriate to utilise a semi-precast system (Figure 11) with similar considerations as presented in the paper by Robson and Bull (2012). FLI Carlow precast panels with an in-situ base were used through the straight chute section, and the upstream end and stilling basin were constructed fully in-situ.

Shear keys and provision for cross drainage were included. A robust back-of-wall drainage system was also included to reduce the loads due to floatation and therefore optimise the new concrete wall and invert thicknesses. The wider, flatter upstream part of the spillway, where floatation is a bigger risk, sits directly on bedrock, which was utilised to reduce the spillway base thickness required to resist uplift by including a system of dowels connecting the new base to the underlying rock. A concrete cut-off and key-in to the clay core on the right-hand side was included at the upstream end of the spillway to tie the new structure into the existing structure left in situ.

Figure 10. Previous composite spillway (MMB)

Figure 11. New reinforced concrete spillway (MMB)

CONCLUSION

The completed Upper Carno Asset Rehabilitation Scheme (Figure 9) included installation of a new full draw-off system, construction of a new spillway, and slope stabilisation works. The construction sequence and drawdown methods used reduced risk to the dam and managed the interfaces between the different elements of the work. The project was delivered over a two-year programme on site and MITIOS sign-off for the associated recommendations was received prior to the statutory date.

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Looking into reservoir geophysics – emerging technologies

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SYNOPSIS As the UK reservoir stock continues to age, further deterioration of these assets should be expected. In particular, leakage and erosion through embankment dams present a significant risk to public safety. Traditional risk management relies on surveillance, and where leaks have been detected this is often supplemented by monitoring of leakage rates and turbidity. However, these techniques are limited to leaks which emerge through the downstream face of the embankment dam. Understanding and monitoring the conditions within the embankment structure are therefore often limited; however, new innovative geophysical techniques are now available that enable medium to long-term monitoring of subsurface flows.

Here we present results from long-term geophysical monitoring of Oakenholt embankment dam, where subsidence and settlement had been identified as possible symptoms of potential internal erosion. An Electrical Resistivity Tomography (ERT) survey mapped the internal structure and surrounding geology of the dam and found a localised zone of potential moisture ingress. Subsequently, a geophysical monitoring system was installed along the embankment for continuous monitoring of the electrical potential field, which changes in direct response to subsurface ground water flow. Water seeping through the dam was identified in relation to the corresponding reservoir level at which it is initiated. The data provided evidence that the seepage is not currently developing further. This study demonstrates that geophysical monitoring is an effective tool for engineers and reservoir Undertakers.

INTRODUCTION

As the UK reservoir stock continues to age, further deterioration of these assets should be expected. Common deterioration mechanisms that have the potential to present a risk to public safety include leakage and associated erosion through or beneath embankment dams. The Environment Agency, which is responsible for collating and reviewing information on reservoir incidents in England, regularly reports leakage and associated erosion through embankment dams to be one of the most common causes of reservoir safety incidents.

Typically, the management of risk posed by these mechanisms is primarily led by routine visual surveillance, which is limited to observation of only those features that are visible at the ground surface. Where a leakage issue is known, surveillance is often supplemented by monitoring the rate and turbidity of the leak, but this approach is generally limited to only those leaks that happen to emerge through the downstream face of the embankment dam. Understanding and monitoring the conditions within the embankment structure are therefore often limited – as even the most experienced asset manager or dam engineer cannot see into the ground.

Whilst geophysical surveys are not a new concept, the range of techniques and approaches that are available, economical, and effective for reservoir-specific applications has continually developed over recent years. Recent innovations include techniques that embed geophysical equipment into dams, for longer term monitoring of changes to conditions over time. The ability to record temporal / time related changes deep into the structure of an embankment dam has potential to provide compelling evidence of emerging issues.

TerraDat has already presented papers on the research phase of its proprietary SP (Self-Potential)-based water flow mapping and monitoring system (SPiVolt). Examples have also been published showing the system monitoring water flow through an embankment dam (Hamlyn et al., 2021 and 2022). Here, we show how SP monitoring has been used to satisfy the recommendations of a Section 10 report for a dam in North Wales while providing higher spatiotemporal measurements than traditional surveillance techniques.

OAKENHOLT EMBANKMENT DAM

An earth-fill embankment with a puddle clay core was constructed around 1876 across Lead Brook to form Oakenholt Reservoir. The dam is approximately 65m long and 15m high, with a grassed downstream face and an upstream face protected by stone pitching.

On the left-hand side (LHS) of the dam, there has been an overall settlement of the downstream crest of about 10mm since 2002. Superimposed onto this is a localised area of increased settlement (up to 27mm), which has also begun to accelerate (Figure 1). The increased settlement correlates with a zone of subsidence that can be observed in the crest wall; there is also a zone of damaged masonry in the upstream wave wall and downstream of the subsided wall is a small void. These on-site observations were made during an integrated geophysical investigation in 2021, which included an Electrical Resistivity Tomography (ERT) survey. The ERT survey provided a cross-section along the length of the embankment, defining it in terms of geo-electrical units (Figure 2). Most notably, a deep, low resistivity zone was interpreted to be clay-rich material within the core of the embankment. The clay core extends from the valley floor to ~ 29mAOD; overlying this is a 4m thick layer of material which is significantly more resistive, i.e. relatively clay deficient; this material 'pinches out' at ~ 85m chainage, coincident with the zone of settlement of the embankment crest wall. This localised decrease in resistivity is thought likely to be caused by increased moisture within the more granular material beneath the crest. The ERT data was acquired when the reservoir was close to top water level (TWL).

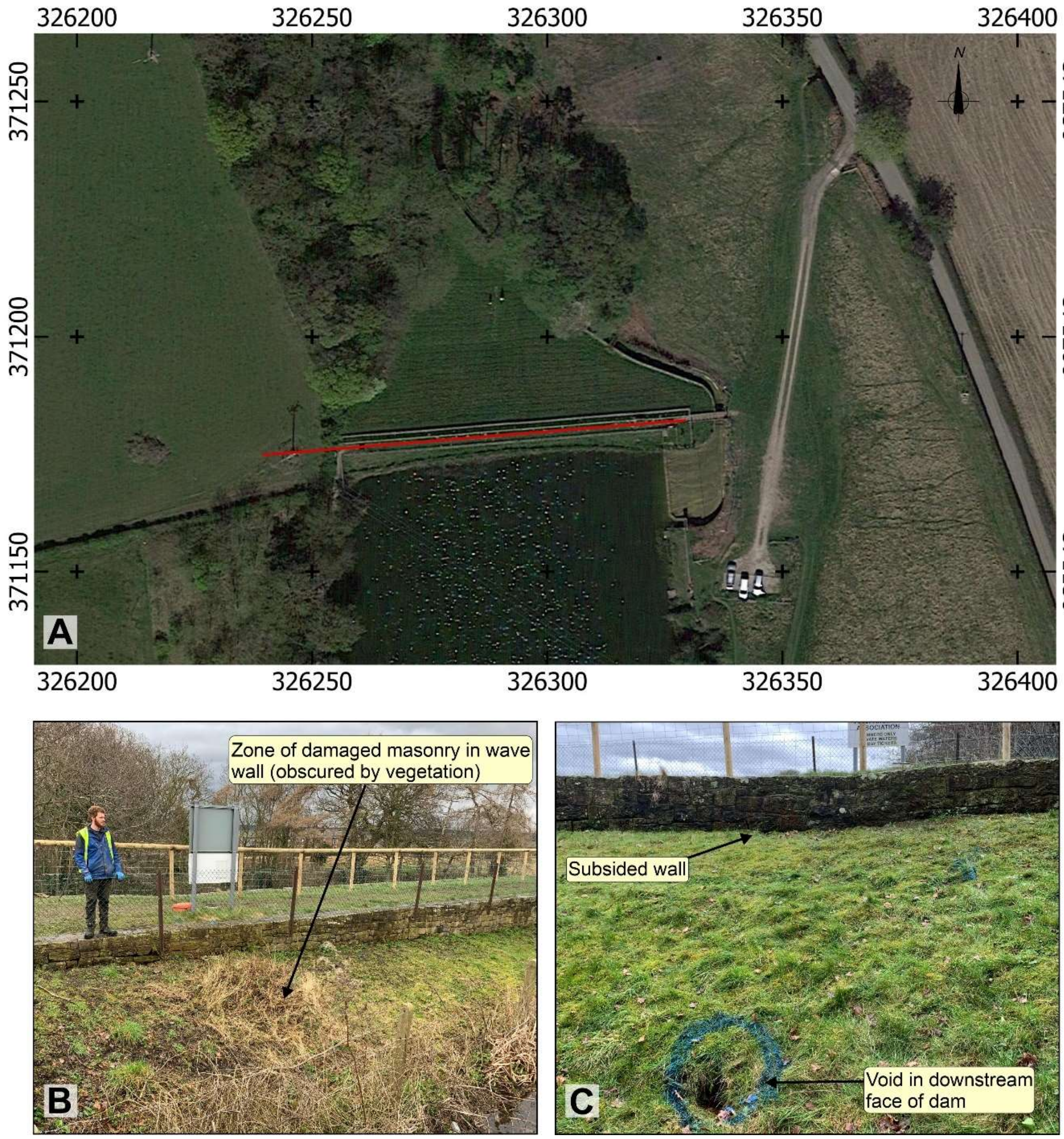

Figure 1. A) Google maps image of Oakenholt dam and location of subsection of ERT profile shown in Figure 2 (red line). **B)** Damaged masonry in upstream wave wall. **C)** Evidence of internal erosion in downstream face.

Based on the void features that have been identified and the localised nature of the subsidence evident at the ground surface and from crest monitoring, the subsidence has been interpreted as most likely caused by loss of fines within the dam due to internal erosion under seepage/leakage. In 2015, Arup completed a S10 Statutory Inspection Report. The inspection identified that the risk of internal erosion (leakage carrying fines) through the body of the dam is credible and significant. Regular surveillance for signs of leakage is noted as the means to mitigate and manage the risk. Annual levelling of the dam is recommended, to keep a record of the surface expression of the subsidence but it should be acknowledged that this will not provide any further information about the ongoing internal processes. Further, there are no obvious flows emerging from the surface of the downstream slope of the embankment, meaning any potential leaks cannot be quantifiably recorded to identify potential change over time. An additional high spatiotemporal resolution surveillance technique with the ability to provide information on any subsurface water movement and its relationship to reservoir level and rainfall is, therefore, highly desirable.

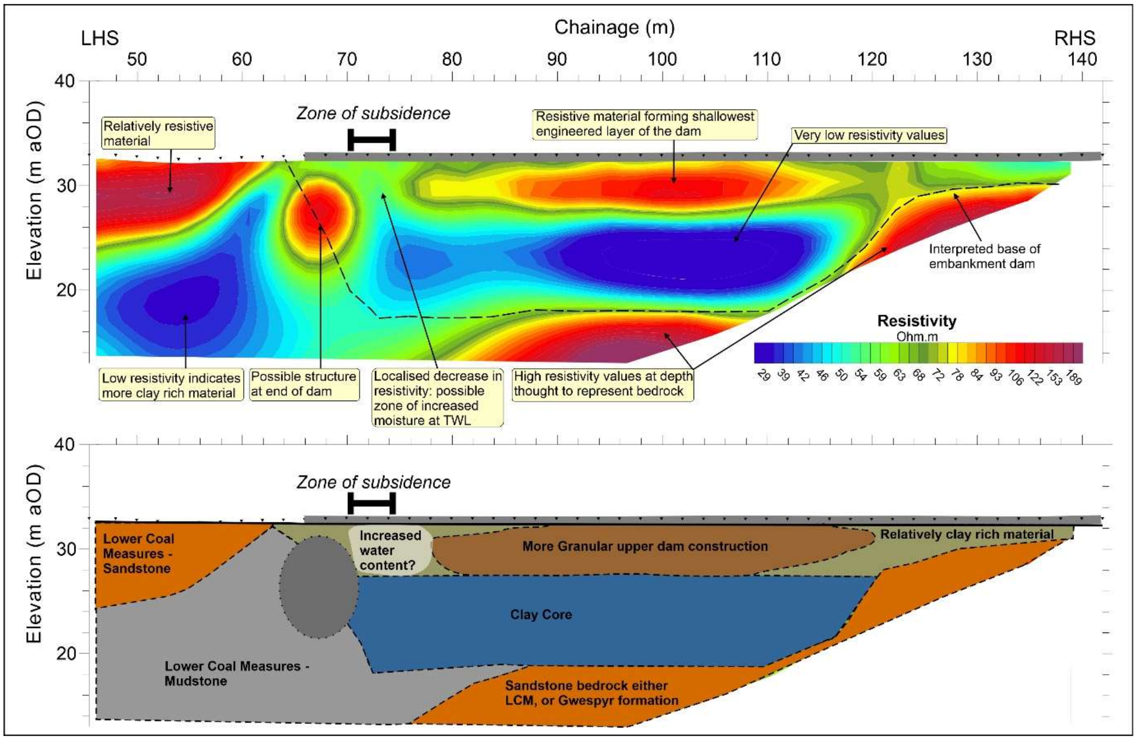

Figure 2. Upper: A subsection of an ERT profile showing the electrical character of the subsurface. Low resistivity (blue) indicates the material is more clay-rich or has an increased moisture content, and increased resistivity (red) indicates drier, clay-deficient material, or competent bedrock. The black bar indicates the zone of settlement observed at the surface. The grey bar shows the location of the wave crest wall. **Lower:** Conceptual model of the dam construction and surrounding geology based on ERT data.

The interpretation of the initial geophysical survey established a detailed model of the geological setting of the dam and its internal structure; this also included a hypothesis of moisture ingress into a shallow zone of more granular material beneath the subsidence in the crest wall. This hypothesis required further verification, and a monitoring array was installed across the embankment to provide information on subsurface water flow and its relationship to the reservoir level and any other hydrogeological factors.

GEOPHYSICAL MONITORING

Geophysical monitoring systems present a step change in geophysical technology from traditional techniques whereby one-time surveys determined the geophysical characteristics of the embankments, including their composition and groundwater conditions. The system used in this instance adopts an array of Self-Potential (SP) monitoring electrodes embedded into the embankment dam for the medium to longer term. This enables long-term temporal / time-based monitoring of the change in geophysical characteristics over time which therefore presents the potential to identify changes in leakage/seepage flows.

The monitoring array continually measured the passively occurring SP field over the area of interest. It is possible to interpret this data to indicate subsurface flow, as discussed in numerous academic papers, such as Boleve et al. (2009} and references therein. Such systems have been created in response to the perceived need for a cost-effective monitoring solution as embankment dams age and inevitably deteriorate.

In brief, SP surveys measure the naturally occurring subsurface electric potential (in millivolts) between two non-polarising electrodes (a reference electrode and a measurement electrode). The reference electrode is situated in an 'electrically quiet' environment, and the measurement electrode is situated within the area of interest. The voltage measurement across the two electrodes is indicative of the naturally occurring electrical potential at the measurement electrode. The system used at this site utilised an array of semi-permanent measurement electrodes positioned across the area of interest to automatically acquire readings at a specified frequency.

Filters and algorithms can be applied to remove the effects of time-varying interference to extract the part of the signal directly attributable to water flow.

INSTALLATION

TerraDat installed a SPiVolt SP monitoring system comprising 32No, 2m spaced, non-polarising electrodes into the embankment as a single monitoring profile across the dam crest (Figure 3). It was configured to record data every ~20 minutes and upload the readings to a server every hour. Rainfall, ground temperature, and air temperature are also recorded to provide additional data for interpretation. The monitoring array was installed by hand behind the downstream crest wall at a depth of ~150mm (Figure 4).

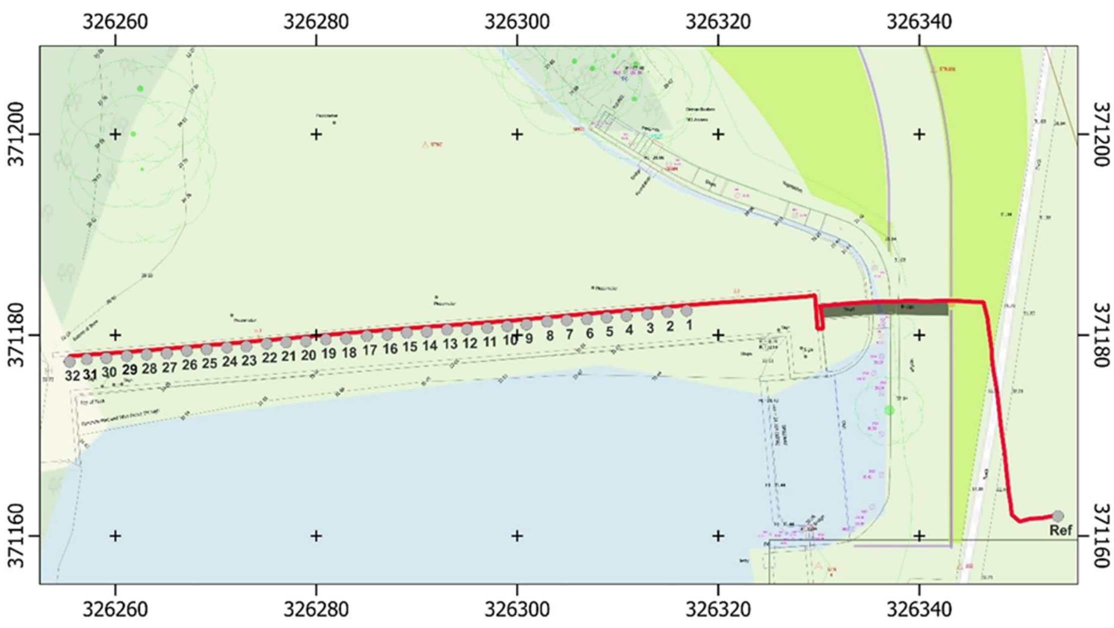

Figure 3. SP monitoring array location plan. Grey circles represent the position of non-polarising electrodes. The red line is the location of the cabling.

Figure 4. SP monitoring array installation at Oakenholt Reservoir. **A)** During installation. **B)** Site conditions following installation.

RESULTS AND INTERPRETATION

The ground exhibits a constant SP field caused by the geology and geochemistry of the subsurface with small temporal fluctuations caused by temperature, atmospheric effects, and subsurface water flow. The contribution from the constant geological effects can be negated by establishing a baseline SP field, which is then removed. The temporal effects not of interest (temperature, atmospheric, and tidal) are removed by appropriate filtering to reveal the time-varying subsurface water flow. Data from the first few days of acquisition are discarded as the electrodes reach an electrochemical equilibrium with the ground.

Subsurface seepage below the dam crest manifests as localised zones of increasingly negative SP response, which typically correlates with high reservoir levels. When the reservoir level is high, shallow flow pathways within the dam crest may be initiated and the increased pressure head will force water through any defects in the core. The SP response across the whole dam structure may also decrease following periods of rainfall as water infiltrates the shallow ground surface and migrates past the electrodes. This effect is reversed when the dam dries out. Positive SP values occur when static water ponds or saturates an area.

The reservoir level was low when the monitoring array was installed; therefore, initial measurements were acquired as the water level increased to TWL (Figure 5). At the beginning of the time series, the monitoring array records negative SP at the LHS of the dam as the reservoir level increases to 30.75mAOD. As the reservoir level rises to 31.2mAOD, evidence of subsurface water flow extends across the dam's left-hand side (LHS) to electrode 23. During this initial observation period, there were two occasions when the reservoir level was drawn down for a couple of days for maintenance works; however, the monitoring array did not record any significant changes or trends in the SP values. This lack of reactivity is likely to reflect the permeability / hydraulic conductivity of the embankment materials, with the embankment holding onto the water within its saturated zone causing a lag between the reduction in reservoir level and significant changes in measurements when conducted over very short time periods.

As the reservoir level remains high, the SP field at this location becomes positive. Positive SP values are observed if the water stops flowing and begins ponding or rising within the structure. It is, therefore, most likely that the ground on the LHS of the dam is becoming saturated over time, with limited discernible flow out of the downstream slope of the embankment. This correlates with on-site observations from the Supervising Engineer, who confirmed that no significant anomalous water flows out of the structure had occurred.

As the water level remains stable for extended periods, the observed SP field shows little variation (within 1 or 2 mV), implying that no other significant hydrogeological factors influence the SP field. Therefore, it is concluded that changes in SP and associated groundwater flow are governed by fluctuations in the reservoir level. Furthermore, as the SP readings through the LHS of the embankment dam remain stable over time, this presents strong evidence that the interpreted ongoing seepage/leakage is not developing. Whilst the monitoring array remains in place, the situation can continue to be monitored against potential development, which provides the Undertaker and Supervising Engineer a level of assurance which in this instance could not be provided through traditional site surveillance and physical leakage monitoring techniques.

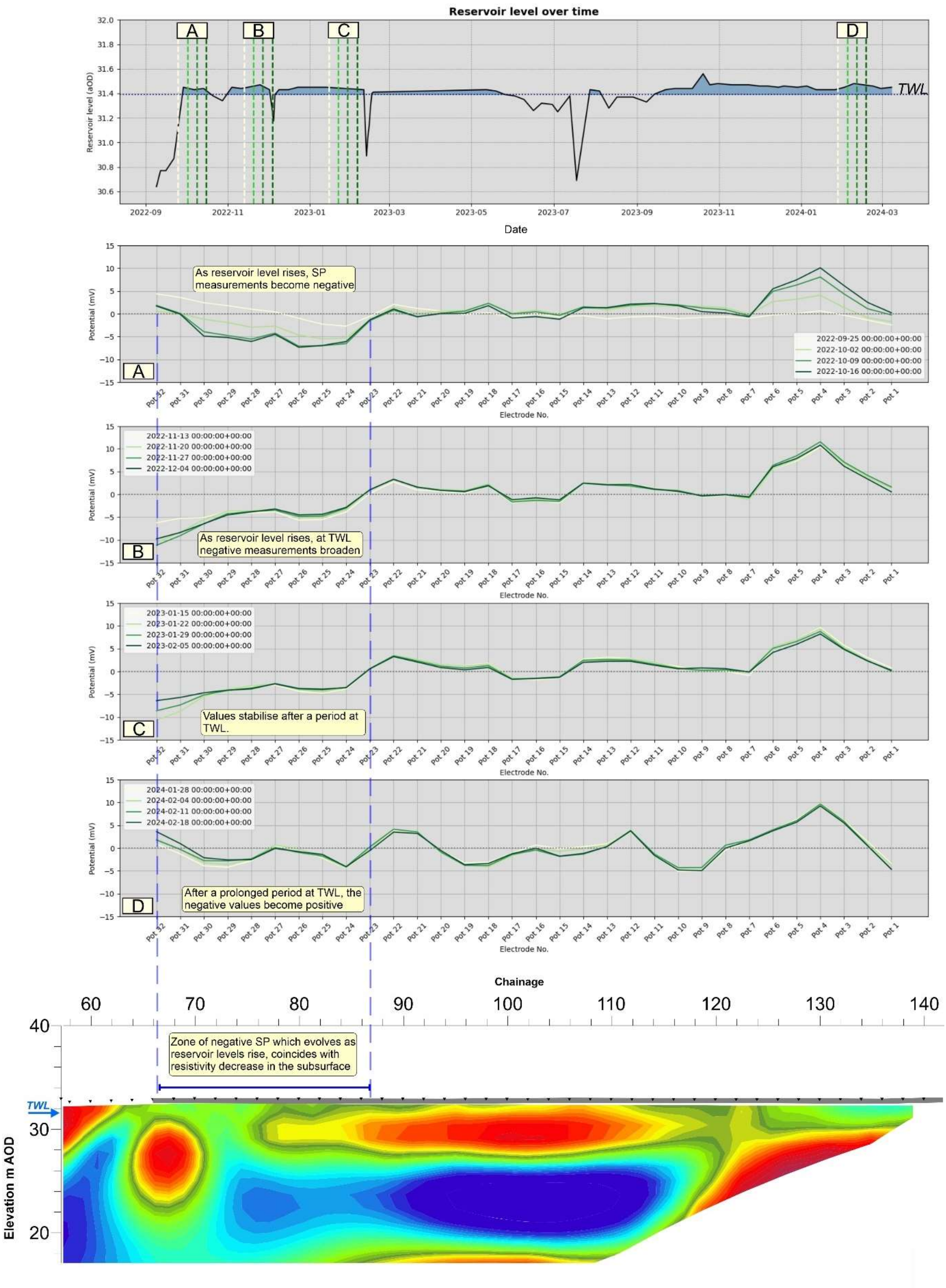

Figure 5. Top) Reservoir levels as recorded by the Client between September 2022 and March 2024. Dashed coloured lines show the reservoir level for the corresponding week of SP data below. **A to D)** Weekly average SP field across the array for the corresponding periods. **Bottom)** Location of the SP data relative to the ERT section collected before installation.

CONCLUSIONS

The on-site observations and initial geophysical survey suggested an area of water seepage/leakage through the LHS of the dam. Geophysical monitoring has proven that water is seeping through the dam and has identified the reservoir level at which it initiates (30.75mAOD). The data also shows that after prolonged periods at TWL the embankment material becomes saturated. This occurs within the upper part of the embankment indicated by the ERT survey as comprising relatively clay-deficient material (Figure 6). SP monitoring has provided a methodology for observing leakage when there is little or no surface expression of a leak.

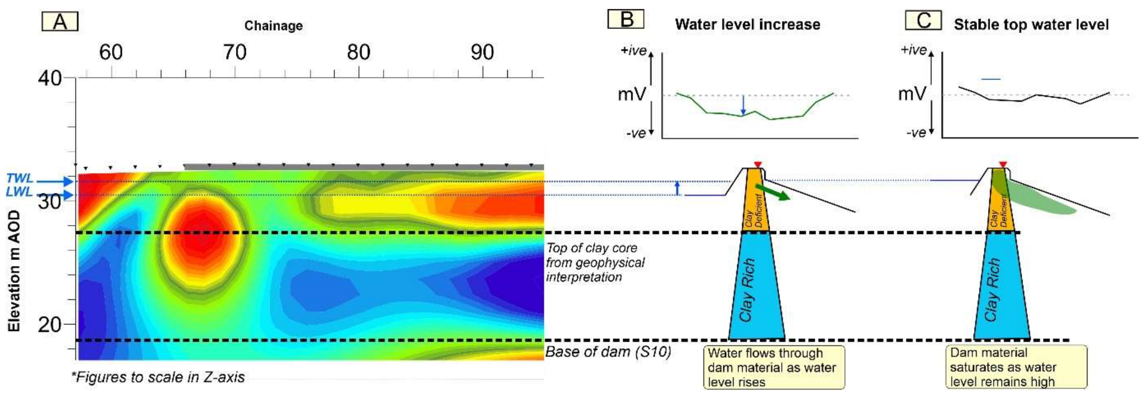

Figure 6. Summary conceptual model of water flow into Oakenholt dam. **A)** ERT profile over area of leakage, with top water (TWL) and low water level (LWL, the level when the SP monitoring array was installed) shown**. B)** Schematic of water moving through material under the SP monitoring array as water level is raised (green arrow). **C)** Material becoming saturated after prolonged period.

Geophysical monitoring systems provide methods of measuring and monitoring the SP field, which is interpreted to describe areas of water flow. In this instance, it provides a valuable surveillance technique making nearly 29,000 measurements over a 14-month surveying period. This study demonstrates that SP monitoring is an effective tool which engineers and reservoir Undertakers can deploy to provide objective long-term monitoring of seepage/ leakage through embankment dams, in common situations where the symptoms of such mechanisms cannot be viewed through routine surveillance or traditional flow measurement techniques. It is widely applicable to embankment reservoirs as it is minimally invasive, cost-effective, and expandable.

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The British Dam Society

Lessons for dam safety in the UK from the landslide-generated waves incident in the Apporo dam reservoir, Japan

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SYNOPSIS We report and analyse the damage caused by landslide-generated waves in the Apporo reservoir (Japan) and take lessons for dam safety in the UK. The incident occurred in September 2018 following an M6.6 earthquake and typhoon Jebi. Apporo dam is a trapezoidal Cemented Sand and Gravel dam with a height of 47.2m. The simultaneous occurrence of the earthquake and the typhoon triggered thousands of landslides. Through field surveys, we identified several landslides on the banks of the reservoir at a close distance to the dam, causing a runup height of 5.3m at the shore. Visible damage, confirmed by site engineers, indicated that the waves damaged the reservoir bank revetments. Here, we model the landslide using Plaxis 3D, replicate the landslide-generated waves applying empirical equations, and discuss the lessons for dam safety in the UK. Using GIS data on elevation, rainfall, and seismicity, we identified the UK regions most susceptible to landslides. Region 3, the highest risk area, contains 252 large reservoirs, indicating the need to include landslide-generated wave risks in assessments of potential failure modes. We discuss prediction capabilities that can be applied for hazard and risk assessment of UK reservoirs regarding landslide-generated waves and propose a four-step methodology for such assessments.

INTRODUCTION AND BACKGROUND

Landslide-generated waves were reported at the Apporo dam reservoir (Hokkaido, Japan) on 5th September 2018 (UTC) following a magnitude (M) 6.6 earthquake and the passage of the Super-Typhoon Jebi (Figure 1). Due to the almost concurrent occurrences of the earthquake and the typhoon, thousands of landslides were generated in the region leading to significant damage to properties and infrastructure and killing 36 people (Yamagishi and Yamazaki 2018; Zhang et al. 2019). According to various reports, nearly 6,000 landslides were generated in the region (e.g. Aimaiti et al. 2019; Zhang et al. 2019), some of which occurred in the Apporo dam reservoir and caused damage (Figure 1) (Heidarzadeh et al. 2023). The field surveys conducted by the authors confirmed the damage from landslide-generated waves; however, the damage was limited and did not threaten the dam's safety.

Landslide-generated waves in reservoirs are considered as major threats for dam safety worldwide. Heller and Ruffini (2023) identified 33 past landslide-generated waves, due to both subaerial and partially submerged landslides, which resulted in a cumulative death toll in excess of 58,000 due to the waves combined with associated phenomena such as volcanic explosions and landslides. Some of them occurred in reservoirs including the catastrophic Vajont reservoir event in 1963 in Italy where a 240 million m^3 of soil mass on the left valley flank became unstable. The generated wave overtopped the 262m tall arch dam by approximately 70m and destroyed the village of Longarone killing about 2,000 people (Müller 1964). A number of landslide-generated waves were repeatedly observed in the Three Gorges Reservoir in China in 2003 (Qianjiangping, Yin et al. 2015), 2008 (Gongjiafang, Huang et al. 2012) and 2015 (Hongyanzi, Xiao et al. 2018). Although generated far away from the dam, these waves resulted in severe damage in the proximity of the slide impact by reaching runup heights of up to 39m and killing people in both 2003 and 2015 events.

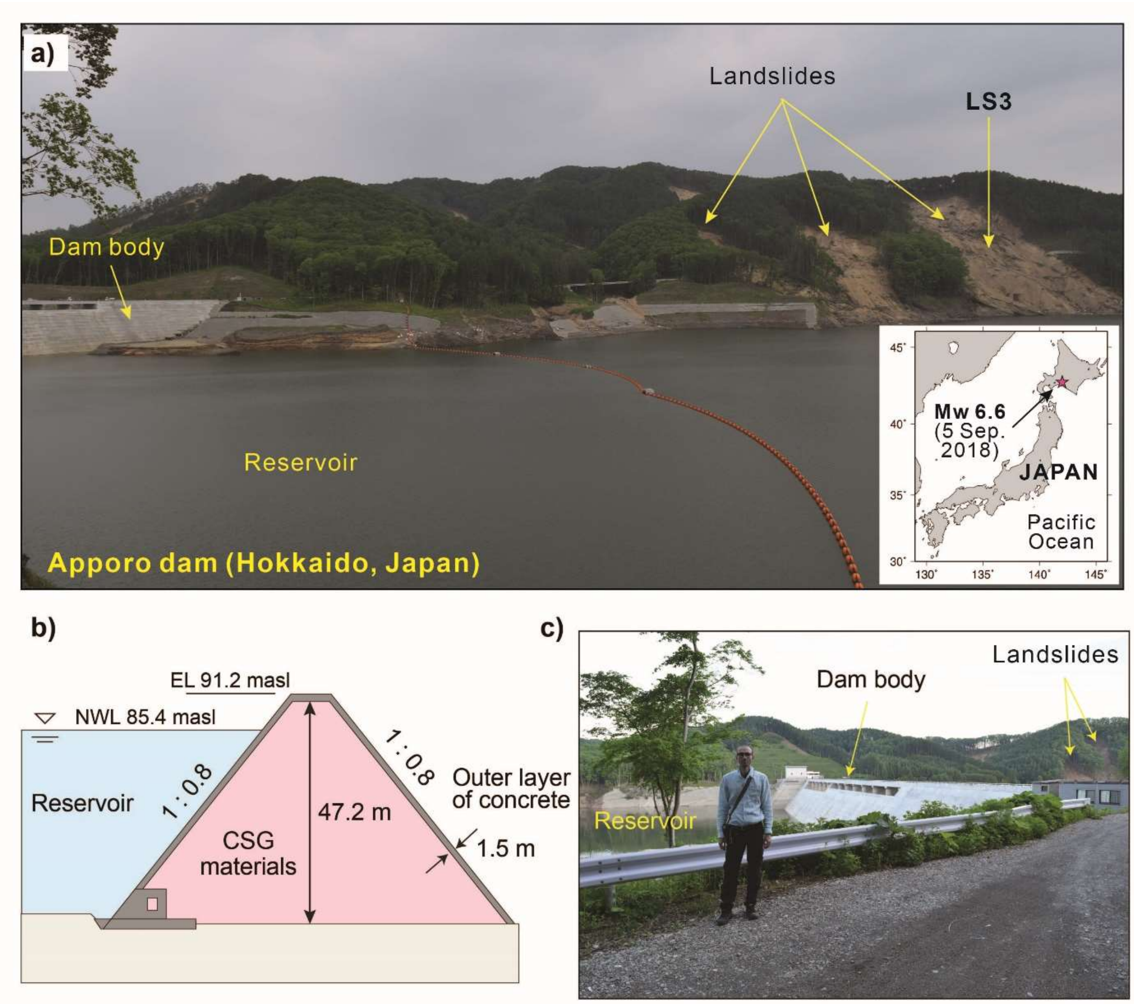


Figure 1. a): Location of the dam and the landslides due to the 5th September 2018 M6.6 earthquake. **b):** A sketch of the dam body cross section. **c):** A photo showing the dam, the reservoir and a few co-seismic landslides. NWL and masl are abbreviations for "Normal Water Level", and "metres above sea level", respectively.

The Clyde reservoir in New Zealand is a rare example where a creeping mass could be stopped (MacFarlane and Jenks 1996). Huang et al. (2023) suggested to remove parts of the WangJiaShan landslide in the Baihetan reservoir in China to reduce the wave risk. Other measures to minimise damage are evacuation of the population, reservoir drawdown, controlled slide blasting and, when designing the dam, provision of adequate freeboard or adding a wave protection wall on the dam crest (Evers et al. 2019). An extreme measure would be partial removal of the dam.

Empirical equations help in the preliminary determination of the risks associated with a threatening landslide. Such empirical equations can be derived from laboratory experiments under systematic variation of the governing parameters under idealised conditions. The generic empirical equations express the wave parameters in functions of these governing parameters (Heller and Ruffini 2023). Figure 2 shows the relevant slide and wave parameters during wave generation, propagation and runup. The governing parameters are the bulk slide volume $\cancel{V}_s$, slide density ρ_s, slide thickness s, slide width b_s, slide impact velocity V_s, slope angle α, and still water depth h. These parameters can be expressed dimensionless as the slide Froude number F = $V_s/(gh)^{1/2}$, the relative slide thickness $S = s/h$ and the relative slide mass $M = \cancel{V}_s\rho_s/(\rho_w b_s h^2)$ where g is the gravitational acceleration, and ρ_w the water density. In the approach from Evers et al. (2019), applied herein, these parameters are merged into the impulse product parameter P = $\mathrm{F}S^{1/2}M^{1/4}\{\cos[(6/7)\alpha]\}^{1/2}$ (Heller and Hager 2010). Important wave parameters are the maximum wave amplitude a_M and height H_M as well as their evolutions $a(r, \gamma)$ and $H(r, \gamma)$ with the radial distance r and wave propagation angle γ. The wave runup at a dam or shore is characterised with the runup height R and the potential overtopping volume $\cancel{V}$ depending on h in front of the dam or shore, the runup angle β, the freeboard f and the dam crest width b_K (Figure 2).

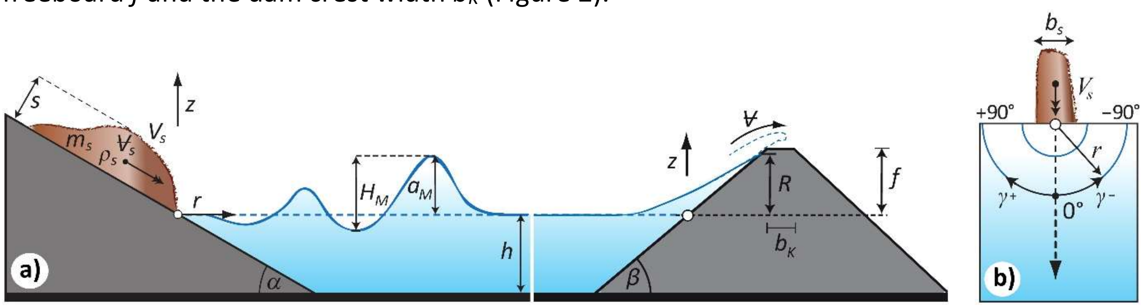


Figure 2. Definition sketches showing **a):** A side view of the slide, landslide-generated waves propagation and runup at an embankment dam, and **b):** A plan view of the slide and wave propagation in an idealised reservoir.

In the UK, the safety of dams is controlled by well-established laws, which are the Reservoirs Act 1975 in England and Wales (Acford 2015), and the Reservoirs (Scotland) Act 2011 in Scotland (Macdonald 2011). Under this legislation, panels of specialist engineers carry out regular Supervising Engineer inspections at least annually, and additional independent Inspecting Engineer inspections are carried out at least every 10 years. Guidance to these engineers suggests consideration of the reservoir rim stability, but it is not common or practical for the Engineer to walk the entire reservoir rim or to undertake geotechnical investigations. Other than comments on changes of land use from recent maps, or obvious slips visible from the dam, there is currently no standard analysis that can be carried out to assess the susceptibility of the reservoir to a landslide induced wave.

The purpose of this paper is to report the findings of the field surveys conducted by the authors following the landslide-generated waves incident in the Apporo dam reservoir, to supplement the surveys with modelling efforts and to take lessons for dam safety in the UK.

Managing Risks for Dams and Reservoirs

We also present prediction capabilities applicable for hazard and risk assessment of UK reservoirs concerning landslide-generated waves, and propose a four-step methodology.

CHARACTERISTICS OF THE DAM AND RESERVOIR

Japan has a large portfolio of dams and is considered one of the most active countries in terms of dam construction worldwide. There are over 2,200 large dams (height more than 15 m) in Japan, the majority of which are constructed for irrigation purposes (Itsukushima 2022; Sasaki and Kondo 2018). The Apporo dam, with a height of 47.2m and a crest length of 516m (Figure 1b,c), is constructed using the cemented sand and gravel (CSG) technology which is a relatively new technology developed in Japan for dam construction. According to the Japan Commission on Large Dams (2018), there are several benefits for constructing dams using the CSG technology, including: smaller carbon emissions, higher stabilities for the dam, and lower maintenance costs. A cross section of the dam is shown in Figure 1b where a 1.5m concrete layer is seen as the top protective layer of the dam body. The capacity of the reservoir is approximately 47 million m3 of water.

DATA AND METHODS

The methodology employed in this research is a combination of field surveys, modelling of maximum wave amplitudes based on empirical equations, and numerical modelling of slope failures. Field surveys were conducted in the period from 29 May to 4 June 2019 to collect data on the landside sizes, locations, and the wave runup heights. The landslide and the damage from the waves were surveyed, photographed, and their information were recorded with the aid of a TruPulse 200 laser rangefinder.

For modelling maximum wave amplitudes of subaerial landslide-generated waves, a wide range of empirical equations is available (Heidarzadeh et al. 2023; Heller and Ruffini 2023). The manual developed by Evers et al. (2019) has been commissioned by the Swiss Federal Office of Energy, responsible for dam safety in Switzerland. In contrast to other approaches, this manual consists of a collection of empirical equations centred around the impulse product parameter P to holistically predict the effects of landslide-generated waves in lakes and reservoirs including wave runups, overland flows, dam overtopping volumes, and flow depths as well as forces on dams. This manual is used here with the aim to predict the observed runup height R at the shore and also a value at the dam. Evers et al. (2019), and the previous version in Heller et al. (2009), have been applied for preliminary hazard assessments in a number of locations including in Austria (Gabl et al. 2015), the Himalaya (Sattar et al. 2021), Switzerland (Fuchs and Boes 2010) and Turkey (Ersoy et al. 2019). As the waves in the Apporo lake propagate freely on semi-circles, the 3D approach in Evers et al. (2019) is most suitable.

The Finite Element Method (FEM) numerical package Plaxis 3D has been used in this research to solve the full hydro-mechanical coupling between soil deformations, consolidation and groundwater flow simultaneously using the Biot's theory (Biot 1956). The theory assumes the soil consolidation is driven by the evolution of excess pore water pressure within the solid element. The soil deformation (e.g., displacement and strain fields) is solved by FEM, while the fluid flow analysis uses the Finite Difference Method (FDM) to solve the pore water pressure field. This approach is critically important in the context of slope stability analysis because the slope deformation is affected by the changes of pore water pressure, and thus the changes in effective stress. The Hardening Soil model with small-strain stiffness (HS-Small model) was used to characterise the behaviour of the topsoil (~up to 3m below the ground surface), while the classical Mohr-Coulomb model was employed for the deeper ground. The HS-Small model will show typical hysteretic behaviour under the earthquake cyclic shear loading. The ground movement earthquake signals were first processed by applying the baseline correction and then applied at the base of the model. The bottom of the model has a fixed compliant base boundary condition, while other boundaries have a normally fixed free-field condition.

The data regarding earthquake mainshock and one-month aftershocks were provided by the United States Geological Survey (USGS) earthquake catalogue: (https://earthquake.usgs.gov/earthquakes/search/).

Typhoon data were downloaded from the ZOOM EARTH weather data: (https://zoom.earth/storms/jebi-2018/#map=satellite-hd).

Data regarding reservoir water level and volume before and after the earthquake were supplied by the Hokkaido Prefecture Authorities.

CONCURRENT OCCURRENCE OF EARTHQUAKE AND TYPHOON

Figure 3 shows the earthquake mainshock (M6.6), and its numerous aftershocks within one month after the mainshock (Figure 3a) along with the track and timing of the Super Typhoon Jebi (Figure 3b). Note that here only aftershocks with magnitude above M4 are shown. The area was hit by 47 earthquakes with M≥4 within one month following the mainshock (Figure 3a). The timing of the typhoon shows that Jebi arrived in the earthquake epicentral area and the dam location around 7 pm on 4th September 2018, approximately 23 hours before the mainshock M6.6 (Figure 3a). Therefore, the area was wet and possibly the soil was saturated at the time of the mainshock and aftershocks. It is challenging to separate the contributions of the earthquake and the typhoon to the occurrence of over 6,000 landslides in the region. However, it is possible to state that the simultaneous incidence of these two extreme natural hazards exacerbated the individual destructive impacts of each.

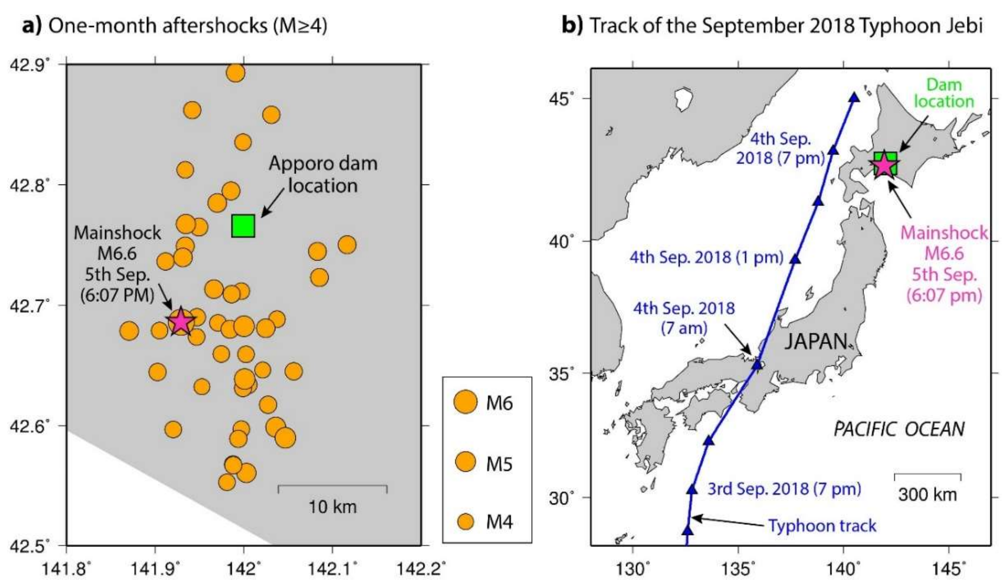


Figure 3. a): The mainshock (M6.6) and one-month aftershocks in the region. **b):** The track of the Super Typhoon Jebi and its timing.

FIELD SURVEY RESULTS

First, we start by looking at the reservoir water level before and after the earthquake as shown in Figure 4. It can be seen that the reservoir water level rose by 30cm within approximately one hour after the earthquake. This is mostly attributed to the intrusion of landslide materials into the reservoir (Figure 4b). Based on our fieldwork in the area and conversations with site engineers, we have witnessed several landslides in the reservoir, some of which were very close to the dam body (Figures 1 and 4). Site engineers guided us to the location of damage to revetments at reservoir banks where we recorded damage details and measured wave runup (Figure 5). The damage shown in Figure 5b was non-existent before the earthquake as confirmed through conversations held with site engineers. Several landslides were easily visible around the damage location (Figure 5b). The runup (R in Figure 2) was measured as $R = 5.3$m (Figure 5a) considering the reservoir water level at the time of the earthquake.

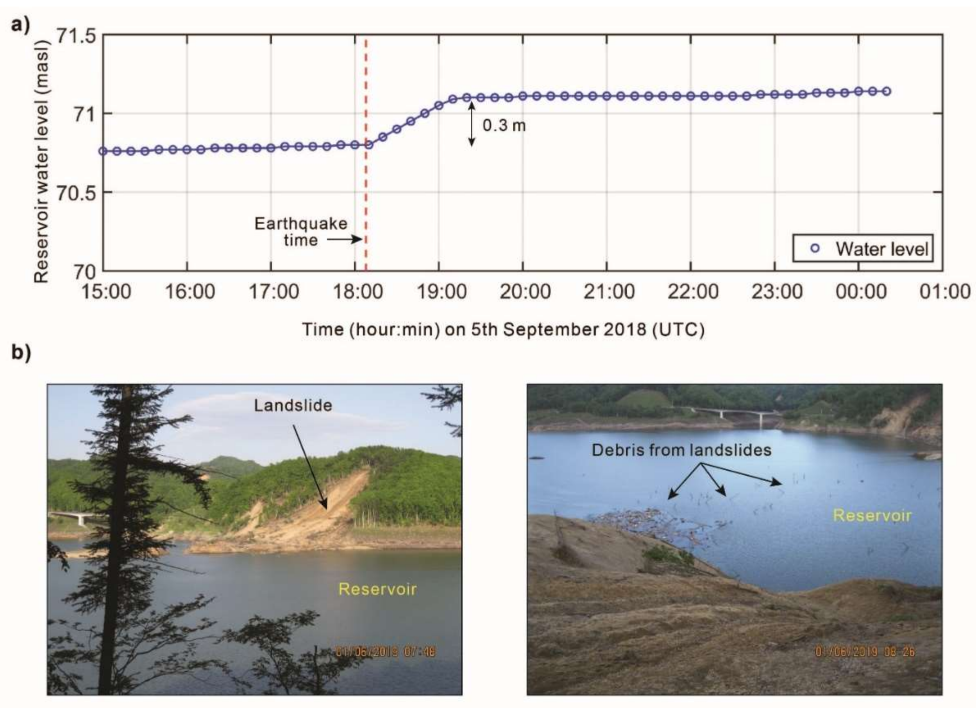


Figure 4. a): Reservoir water level before and after the earthquake. **b):** Photos showing the intrusion of landslide materials into the reservoir.

Figure 5. a): Surveyed wave runup point in the banks of the reservoir due to the landslide-generated waves. **b):** A photo showing damage due to the landslide-generated waves.

MODELLING SLOPE STABILITY

Figure 6 presents the slope displacements at the failure state after 200 s of ground earthquake shaking. This analysis was conducted assuming that the groundwater level was at the ground surface after the long-term rainfall. The numerical result indicates that major failure occurred within the topsoil at the middle to upper section of the slope, above the reservoir water level. The shallow nature of the slope failures (Figure 6) is consistent with field observations reported by Heidarzadeh et al. (2023).

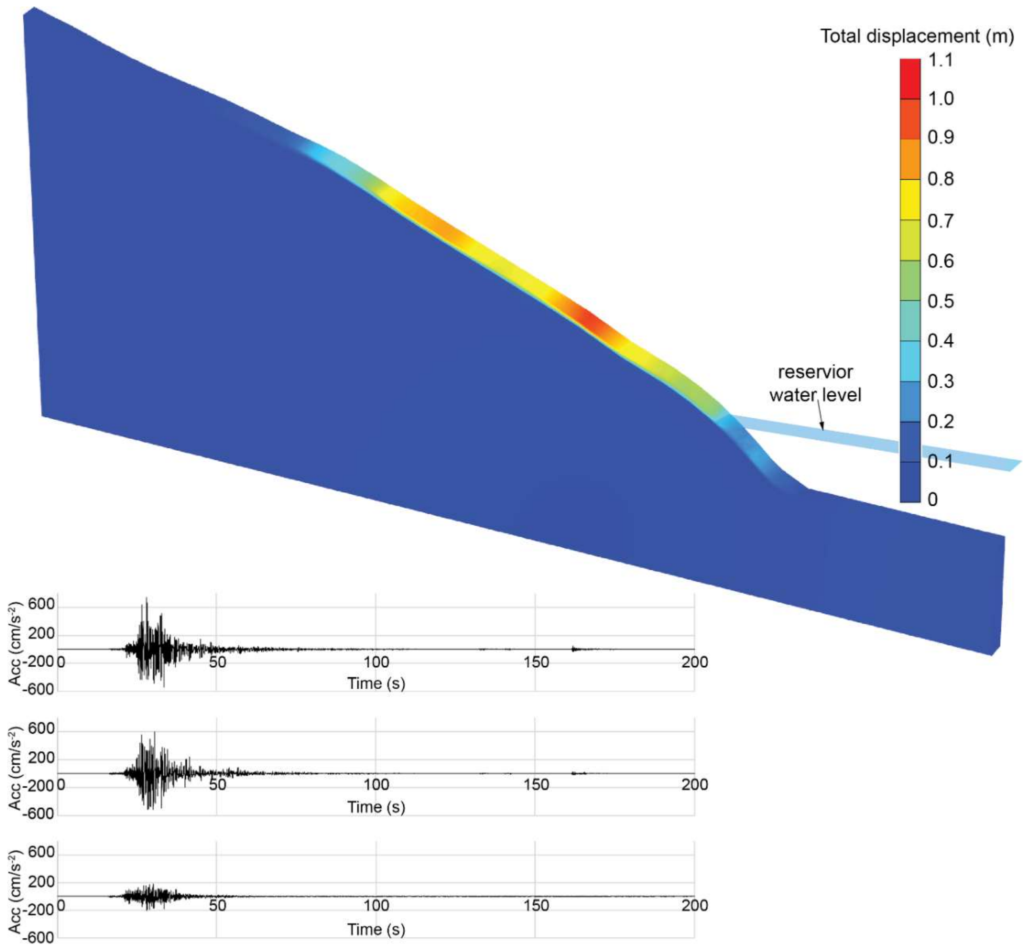


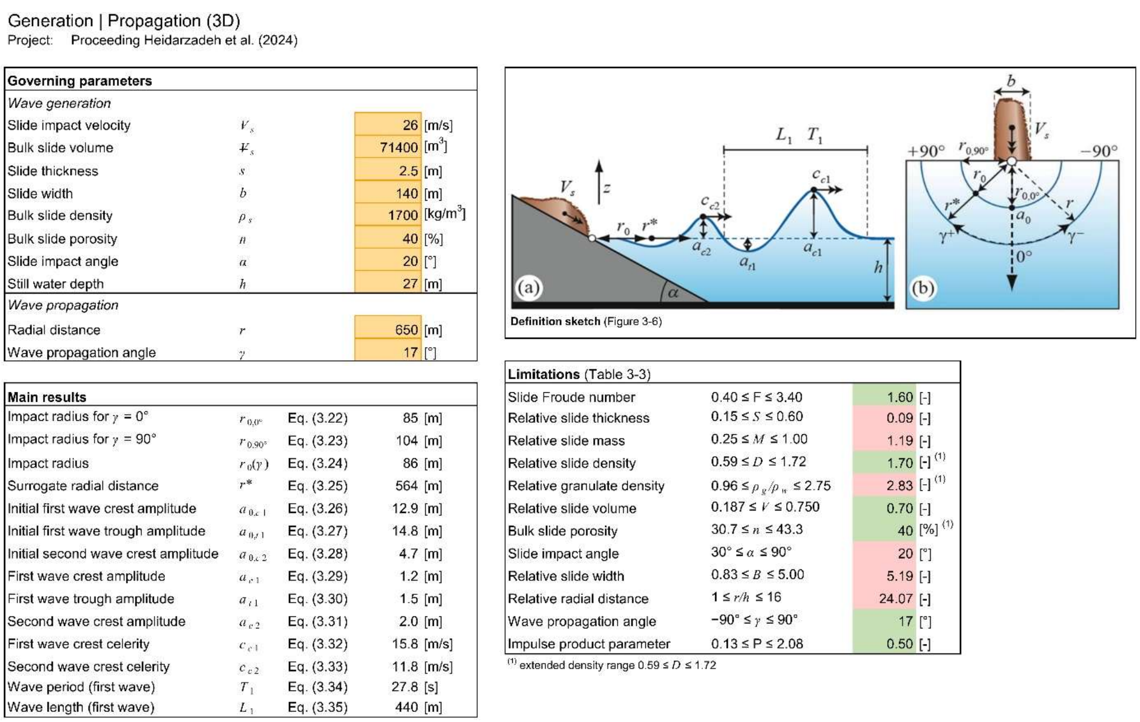
Figure 6. Simulated slope stability analysis of the slopes facing the Apporo dam reservoir using Plaxis 3D. The slope elevation profile was obtained from Google Earth. The ground water level is assumed to be at the ground surface after the long-term rainfall. The earthquake signals were recorded at the Mukawa station (42.7609N, 142.1344E), 11.4 km away from the dam. "Acc" is an abbreviation for "Acceleration".

PREDICTING THE WAVE RUNUP USING EMPIRICAL EQUATIONS

Table 1 contains the parameters for landslide 3 from Heidarzadeh et al. (2023). A slide porosity n = 40% has been assumed and the wave propagation angle g (Figure 2) is estimated as 17°. The spreadsheet (step 1) of Evers et al. (2019) is shown in Figure 7 which predicts a maximum wave amplitude of 12.9m (a0,c1 in Figure 7) and a maximum height of 27.7m in the impact zone (a0,c1 + a0,t1). The waves decay over the distance r = 650m to an amplitude of 1.2m (ac1) and a wave height of 2.7m (ac1 + at1) offshore the runup location. The corresponding runup height (not shown in Figure 7) is 6.5m, which is 23% larger than 5.3m observed in the field.

Table 1. Parameters for landslide 3 from Heidarzadeh et al. (2023) for wave generation, propagation and runup at the shore and at the dam for the Apporo dam incident. Here, V_s = [2gDz(1 – tan$d$cot$a$)]$^{1/2}$ from Evers et al. (2019) and the slide porosity is assumed as n = 40%.

Parameter	Value	Parameter	Value
Slide impact angle α (°)	20	Shore: Radial distance r (m)	650
Vertical drop height Δz (m)	85	Shore: Wave propagation angle γ (°)	17
Dynamic bed friction angle δ (°)	12	Shore: Still water depth h (m)	27
Slide impact velocity V_s (m/s)	26	Shore: Runup angle β (°)	10
Slide width b_s (m)	140	Shore: Observed runup height R (m)	5.3
Maximum slide thickness s (m)	2.5	Dam: Radial distance r (m)	680
Bulk slide volume $V\!\!\!/_s$ (m^3)	71400	Dam: Wave propagation angle γ (°)	60
Bulk slide density ρ_s (kg/m^3)	1700	Dam: Still water depth h (m)	27
Slide porosity n (%)	40	Dam: Runup angle β (°)	51
Still water depth h (m)	27	Dam: Freeboard f (m)	20.4


Generation | Propagation (3D)
Project: Proceeding Heidarzadeh et al. (2024)

Governing parameters			
Wave generation			
Slide impact velocity	V_s	26	[m/s]
Bulk slide volume	$V\!\!\!/_s$	71400	[m^3]
Slide thickness	s	2.5	[m]
Slide width	b	140	[m]
Bulk slide density	ρ_s	1700	[kg/m^3]
Bulk slide porosity	n	40	[%]
Slide impact angle	α	20	[°]
Still water depth	h	27	[m]
Wave propagation			
Radial distance	r	650	[m]
Wave propagation angle	γ	17	[°]

Main results				
Impact radius for γ = 0°	$r_{0,0°}$	Eq. (3.22)	85	[m]
Impact radius for γ = 90°	$r_{0,90°}$	Eq. (3.23)	104	[m]
Impact radius	$r_0(\gamma)$	Eq. (3.24)	86	[m]
Surrogate radial distance	r^*	Eq. (3.25)	564	[m]
Initial first wave crest amplitude	$a_{0,c1}$	Eq. (3.26)	12.9	[m]
Initial first wave trough amplitude	$a_{0,t1}$	Eq. (3.27)	14.8	[m]
Initial second wave crest amplitude	$a_{0,c2}$	Eq. (3.28)	4.7	[m]
First wave crest amplitude	a_{c1}	Eq. (3.29)	1.2	[m]
First wave trough amplitude	a_{t1}	Eq. (3.30)	1.5	[m]
Second wave crest amplitude	a_{c2}	Eq. (3.31)	2.0	[m]
First wave crest celerity	c_{c1}	Eq. (3.32)	15.8	[m/s]
Second wave crest celerity	c_{c2}	Eq. (3.33)	11.8	[m/s]
Wave period (first wave)	T_1	Eq. (3.34)	27.8	[s]
Wave length (first wave)	L_1	Eq. (3.35)	440	[m]

Limitations (Table 3-3)			
Slide Froude number	0.40 ≤ F ≤ 3.40	1.60	[-]
Relative slide thickness	0.15 ≤ S ≤ 0.60	0.09	[-]
Relative slide mass	0.25 ≤ M ≤ 1.00	1.19	[-]
Relative slide density	0.59 ≤ D ≤ 1.72	1.70	[-] [(1)]
Relative granulate density	0.96 ≤ ρ_g/ρ_w ≤ 2.75	2.83	[-] [(1)]
Relative slide volume	0.187 ≤ V ≤ 0.750	0.70	[-]
Bulk slide porosity	30.7 ≤ n ≤ 43.3	40	[%] [(1)]
Slide impact angle	30° ≤ α ≤ 90°	20	[°]
Relative slide width	0.83 ≤ B ≤ 5.00	5.19	[-]
Relative radial distance	1 ≤ r/h ≤ 16	24.07	[-]
Wave propagation angle	−90° ≤ γ ≤ 90°	17	[°]
Impulse product parameter	0.13 ≤ P ≤ 2.08	0.50	[-]

[(1)] extended density range 0.59 ≤ D ≤ 1.72

Figure 7. Spreadsheet "Generation | Propagation (3D)" of Evers et al. (2019) with the input parameters from Table 1 (orange), satisfied (green) and not satisfied (pink) limitations and main results. For details about this empirical approach, see Evers et al. (2019) and the accompanied spreadsheet at: https://zenodo.org/record/3492000#.XmAQwW52uas.

Nevertheless, this 23% discrepancy in step 1 can be considered acceptable given that the empirical equations in Evers et al. (2019) are based on idealised conditions including mesh-packed granular slides in the 3D approach, and constant slide impact and wave runup slopes. Note also that the wave amplitude at the base of the shore slope (approximately in the centre of the reservoir) is used to calculate the runup height with a 2D runup equation, i.e. whilst the amplitude decay due to lateral energy spread is taken into account up to this base, it is neglected on the slope itself. Further, six of the 12 parameter limitations of the empirical equations for wave generation and propagation are not satisfied (Figure 7) whilst all four for the runup are satisfied (not shown in Figure 7). There are uncertainties regarding the slide parameters and water depth too. The effects of wave parameters due to deviations of these idealisations are not covered in step 1 of the spreadsheet developed by Evers et al. (2019). However, they are described and quantified in step 2 (Evers et al. 2019), which can be important.

At the dam centre, the spreadsheet of Evers et al. (2019) predicts R = 2.6m for the parameters given in Table 1. Note that there are no corresponding observations from the field for the dam centre. This runup is significantly smaller than the freeboard of 20.4m (at the time of the earthquake) such that there was no immediate danger for the downstream population. Given that the method of Evers et al. (2019) provides only preliminary estimates, it is strongly recommended to conduct a comprehensive, prototype-specific numerical or scaled laboratory study if the predicted R at the dam is close to the dam's freeboard.

HAZARDS IMPLICATIONS FOR THE UK

In the UK, the highest risk of landslide-induced waves on reservoirs would appear to be in steep V-shaped valleys, with higher-than-average rainfall and seismic risk. The mountainous regions of Wales and Scotland meet these criteria, and indeed Wales has had several examples of major landslide tragedies (on coal waste tips) in the past. Whilst major earthquakes are rare in the UK, the highest seismicity levels are shown in standard guidance (Figure 5 of Charles et al. 1991) as mid/northwest England, Wales, and north-west Scotland; areas where rainfall is also high.

Figure 8 shows an initial estimation of the areas of the UK most susceptible to landslide risk to reservoirs, using GIS layers for elevation, rainfall, and seismicity. Comparing the highest risk zone (Region 3, in Figure 8) with the coordinates of all reservoirs on the public registers (as of May 2024), shows that some 38 large-raised reservoirs in England, 111 reservoirs in Wales and 103 controlled reservoirs in Scotland fall within this highest risk zone. This initial analysis of landslide risk to UK reservoirs indicates that less than 10% of large reservoirs on the registers may need to have this risk included when undertaking a quantitative risk assessment of potential failure modes. Panel engineers undertaking inspections of dams in these higher risk areas should be extra vigilant of reservoir rim stability indicators when undertaking their visits.

CONCLUSIONS

In this article, we highlighted the hazards and risks from landslide-generated waves in dam reservoirs by reporting, analysing and modelling an incident that occurred in Apporo dam reservoir in Japan in September 2018. Main findings are:

- By field surveys, we measured a runup height of 5.3m at a location in the reservoir banks.
- The empirical model of Evers et al. (2019) was applied to replicate the runup which successfully predicted the observed runup with an acceptable discrepancy of 23%.
- Numerical modelling using Plaxis 3D revealed that the slopes were fully saturated before the earthquake, and earthquake shaking triggered the landslide.

Managing Risks for Dams and Reservoirs

- We found that fewer than 10% of large UK reservoirs may need to include landslide risk in their failure mode assessments. Panel engineers should be especially vigilant regarding reservoir rim stability during inspections in higher risk areas (Regions 2 and 3, in Figure 8).
- To conduct preliminary quantitative assessments of the potential for landslide-generated waves in UK dam reservoirs, we recommend the following four steps:

i) **Step 1:** Investigate whether the reservoir is located within the 'Regions 2 and 3' of our Figure 8 or not.

ii) **Step 2:** If the answer to Step 1 is positive, conduct numerical modelling of landslides and assess Factor of Safety (FoS) of the slopes and estimate displacements (Figure 6).

iii) **Step 3:** If there is a potential for failure (e.g., FoS < 1.2), apply the approach of Evers et al. (2019) to estimate the amplitudes of the landslide-generated waves (Figure 7).

iv) **Step 4:** For cases where the estimated wave amplitudes are close to the dam freeboard, consider appropriate remedy/resilience measures.

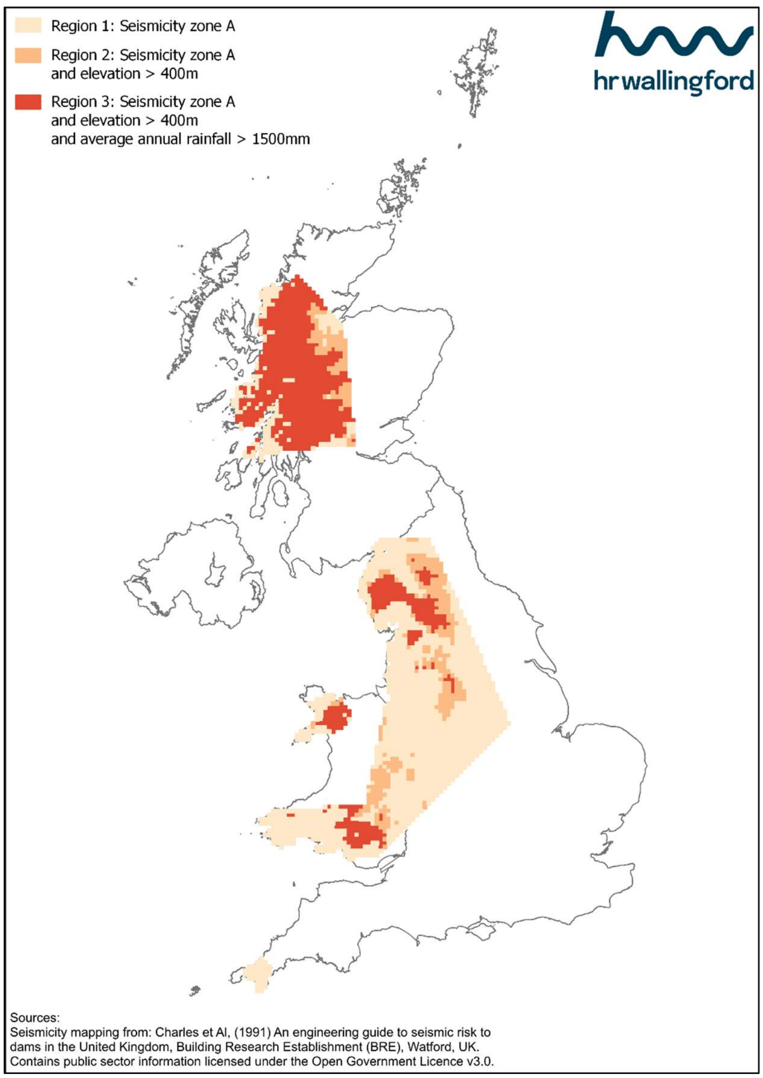


Figure 8. Areas of elevated risk of landslide into reservoirs in the UK based on available GIS data on elevation, rainfall, and seismicity.

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Megget Reservoir: Investigation into potential internal erosion in an asphaltic concrete core rockfill dam

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SYNOPSIS Megget Reservoir is an impounding reservoir in the Megget valley of the Scottish Borders, which is used to supply water to Edinburgh and the Lothians. The dam retaining the reservoir is a 56m high asphaltic core rockfill dam constructed in 1982, believed to be the only one of its type in the UK.

In 2021, an inspection under Section 47 of the Reservoirs (Scotland) Act 2011 observed dark fine silty deposits within the drainage gallery, which had historically gone unreported. There were concerns that, due to the unknown nature and origin of the deposits, the erosion of material could increase risks to dam safety. AtkinsRéalis carried out an investigation into the source of the material, including a series of advanced soil characterisation tests, alongside a separate comprehensive study into the monitoring data at the reservoir. The outputs were used to determine the likely nature and origin of the material, as well as qualitatively assessing the risks to dam safety due to the erosion and recommending a future monitoring regime to mitigate the associated risks.

INTRODUCTION

Megget Reservoir is an impounding reservoir in the Megget valley of the Scottish Borders. It is owned and operated by Scottish Water (the "Reservoir Manager") and is used to supply water to Edinburgh and the Lothians. The reservoir is largely rectangular in shape, with a capacity of around 64Mm^3 and a surface area of 2.6km^2 at top water level. The dam retaining the reservoir is a 56m high asphaltic concrete core rockfill dam constructed in 1982, believed to be the only one of its type in the UK.

In 2021, during the visit for inspection under Section 47 of the Reservoirs (Scotland) Act 2011, the Inspecting Engineer observed dark fine silty deposits within the drainage gallery. Anecdotal evidence at the time suggested that the steady accumulation of these deposits had been managed for some time (potentially since construction). However, there was no mention of the deposits in previous inspection reports and no records were available of: the rate of accumulation, the frequency of clearance or nature and origin of the deposits.

Given the lack of record and understanding of the nature and origin of the deposits, there was no way of confirming if internal erosion was affecting dam safety. A listed (safety) measure was therefore recommended: "Samples of the deposit within the drainage collection channel should be taken and analysed to establish its nature and likely origin."

Four hypotheses were tested as part of the investigation:

- The deposits originate from the bituminous core of the dam.
- The deposits originate from the concrete adjacent to the bituminous core.
- The deposits originate from the transition material downstream of the bituminous core.
- The deposits originate from the drainage material downstream of the bituminous core and situated between the transition material and the concrete gallery.

It was also noted during the inspection that a significant amount of data was being recorded at the reservoir. However, the Reservoir Manager had not been given advice on how to effectively analyse and interpret this data. Therefore, in parallel with the investigation, a recommendation was made to undertake a desk study to review existing monitoring data, published technical papers and available rainfall data.

This paper describes the investigation into the source of the material, alongside the separate comprehensive study into the monitoring data at the reservoir. The findings of these were used as a means to test the hypotheses and subsequently make recommendations for future monitoring and interpretation.

BACKGROUND

Dam

Megget Dam is 56m high and spans 570m at its crest. The construction of the reservoir was completed in 1982 by Lothian Regional Council (LRC).

The dam has two shoulders of gravel fill and a central vertical asphaltic core. The core is located under the crest of the dam and slightly offset to the upstream side. Adjacent to the core are transition zones extending 1.5m upstream and downstream of the core. Beyond the transition zones the embankment consists of a gravel fill material. The crest and downstream berm are protected with grass and concrete blocks, with the upstream berm and embankment slope being covered in riprap above a filter layer to separate it from the main gravel fill (Figure **1**).

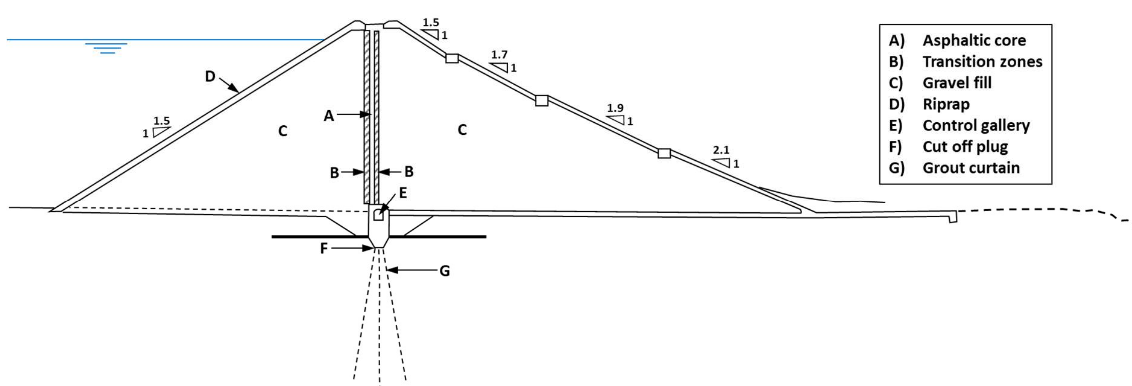

Figure 1. Cross section through Megget dam

A concrete inspection gallery runs along the length of the dam at the base of the asphaltic core. Drainage channels run along the length of the gallery, with drainage pipes from the above drainage material (immediately below the transition zone) discharging into the drainage channels (Figure 2 and **Figure 3**).

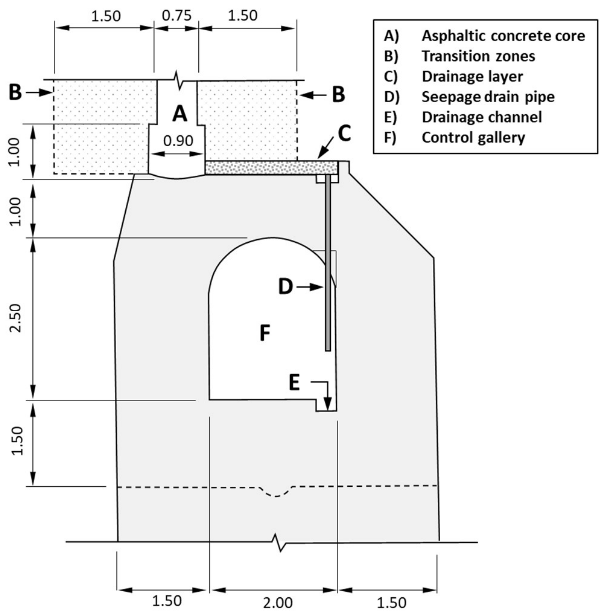

Figure 2. Cross-section through gallery

Figure 3. Photograph of drainage gallery

Construction Details

It is reported that the gravel used as fill for the shoulders was sourced from borrow pits within the reservoir basin (LRC, 1995). The alluvial gravel was described as graded, including coarse silt, sands, gravels, and cobbles, with all particles above 125mm being excluded.

The transition zones on the either side of the core use the same material as that used in the shoulder (with an upper size limit of 100mm and a restriction of 5% - 10% fines passing 2mm), allowing for any seepage through the core to be directed to the control gallery channels, via the drainage layer and seepage drain pipes (Figure 2, notes C and D).

Given the age of the structure, it is likely that the drainage material is an open graded aggregate. The origin or nature of the material is not stated in the records; however it is possible that the material may have been imported and of similar nature to that used in the asphaltic core.

An asphaltic concrete core is said to have been selected as, whilst clay deposits were available close to the site, there was not enough to complete the core, and the use of asphaltic material was the cost-effective alternative. The core is 55m tall at the highest point and 555m long at the top of the embankment dam. The thickness of the core varies with height; for the first 1m above the control gallery it is 900mm thick, then 700mm and finally reducing to 600m thick for the top 23 metres. A mastic material was used as a contact layer between the core and the control gallery.

The design of the asphaltic concrete and the mastic mixes were carried out by a specialist subcontractor. All coarse aggregates used in the asphaltic concrete were crushed basalts, with natural sand and fine crushed aggregate also being used. The final asphaltic concrete and mastic mixes are given in **Table 1**.

Instrumentation

A comprehensive suite of instrumentation was installed during construction to monitor key parameters including seepage, pore pressure, movement and water levels in both the dam and the reservoir, as detailed in **Table 2**.

Table 1. Mix design of the asphaltic concrete and the mastic mix

Asphaltic concrete	%	Mastic	%
20 mm aggregate	14.0	Crushed fines	32.0
14 mm aggregate	16.8	Natural sand	33.0
6.3 mm aggregate	11.1	Limestone filler	20.0
Crushed fines	20.5	Bitumen	15.0
Natural sand	20.5		
Limestone filler	10.3		
Bitumen	6.8		

Table 2. Instrumentation at Megget dam

Monitoring parameter	Instrument(s)	Description
Seepage	V-notch weirs	Collect water from a drainage layer at the base of the transition zone. Water discharges into a drainage collection channel within the drainage gallery. Flow rates are measured over three V-notch weirs with Gauge 1: North shoulder seepage; Gauge 2: North shoulder + Central area; Gauge 3: South shoulder seepage. Additional flow chambers equipped with V-notch weirs near the stilling pool capture seepage from the valve house and downstream culvert drains.
	Additional measures	Seepage is collected in a drainage blanket and measured at the downstream toe.
Pore Pressure	Hydraulic piezometers	Installed in the embankment foundation, both upstream and downstream shoulders and along the culvert in sections both upstream and downstream of the core.
	Standpipe piezometers	Eleven standpipe piezometers are positioned in the bedrock beneath the downstream shoulder.
Movement	Survey stations (SU)	Eleven (SU) for horizontal and vertical movements on the downstream shoulder
	Settlement stations (SE)	Five (SE) for vertical movement

Monitoring parameter	Instrument(s)	Description
	Inclinometers (I)	Ten (I) installed during construction, not monitored currently but available
	Monitoring stations	Thirty-eight monitoring stations in the grouted rip-rap on the upstream face
Water Level	Sensors	Water levels in the reservoir are continuously monitored with sensors in the draw-off tower
	Metric gauge board	Attached to the south-west side of the outlet tower

The instrumentation in the dam was being maintained, readings taken at regular intervals and the results stored on the Reservoir Manager's database. Although a substantial quantity of data was being collected, the Reservoir Manager had not been given directions to allow for effective analysis and interpretation of the data to realise its full potential.

Sampling and Laboratory Testing

Two phases of sampling and laboratory testing have been undertaken. The first phase provided some insight into the nature and likely origin of the material, however there was insufficient data to determine which hypothesis was most likely. Therefore, a second phase was recommended.

First Phase - Sampling

A site visit was made on 7th December 2022, where the following samples were collected from the drainage channel within the gallery, along with water samples from the reservoir:

- 1no. soil sample and 2no. water samples from "north" point
- 1no. soil sample and 1no. water sample from "north-mid" [central] point
- 1no. soil sample and 2no. water samples from "south" point

The north and south samples were collected from the drainage channel immediately upstream of V-notch Gauge 1 and 3. The north-mid [central] samples were collected from the drainage channel immediately upstream of Gauge . Figure 4 shows that the materials were visually more variable than originally observed.

Figure 4. Photographs of dark sediments along the drainage channel

First Phase - Testing

The samples were sent to a third-party testing laboratory with the following tests being specified:

- Chemical test suites (metals and inorganic) on all gallery water samples
- Chemical test suites (metals, inorganic, and organic) on the north soil sample
- Fourier Transform Infrared Spectroscopy ("FTIR") on the north-mid soil sample
- Petrographic analyses[1] on the north-mid and south soil samples

Chemical and FTIR testing were specified to determine if bituminous or organic materials were present in the samples. Petrographic tests were specified to determine the minerology of the samples to allow for comparison to materials used in the construction of the dam.

First Phase - Findings

The chemical test suite results were inconclusive as to the presence of bituminous material in the samples, with the majority of the organic tests indicative of bituminous material being below the limit of detection.

The results of the petrographic analyses are shown in the table below, expressed as percentage by volume of the sample.

Table 3. Petrographic south and north-mid sample constituents

Constituent	**Percentage by volume (of sample)**	
	South sample	**North-mid sample**
Quartz	43	-
Soil	22	-
Substantially altered rock	10	2
Quartzite	5	-
Opaque debris	2	58
Calcite	-	7
Precipitated calcite	-	33
Other*	18	<1

*Other constituents were typically <5% of the sample and are not fully listed for brevity.

Of note is the substantial percentage of "opaque debris" found in the north-mid sample. This was described as *"[resembling possible manganese oxide] and precipitated calcite with lesser amounts of calcite grains, substantially altered rock (too fine to be fully distinguished) and traces of chert"*. This material appeared opaque and black on the petrographic thin section images, meaning it could not be described. Approximately 29% by weight of this sample passed the 63µm sieve, with the <63µm fraction appearing to be very similar in appearance to the opaque debris.

[1] Testing was carried out in accordance with BS EN 932-3: 1997. Tests for general properties of aggregates (BSI, 1997) and in-house test methods.

The FTIR results indicated that no discernible polymeric/hydrocarbon material was present and bitumen was not found in the sample. The FTIR spectra indicated the presence of limestone and quartz in the sample. However, FTIR testing only detects certain compositions including organics. Materials including metals (such as possible manganese oxide) would not show up on FTIR spectra (Figure 5).

In order to resolve some of inconclusive findings of this phase, the inspecting engineer recommended further tests (based on the recommendations of the testing laboratory), which included Scanning Electron Microscope ("SEM") tests to confirm the nature of the "opaque debris".

Second Phase - Testing

In consultation with the testing laboratory, an SEM test with energy dispersive x-ray microanalysis ("EDX") was undertaken on the north-mid sample collected during the first phase. EDX testing is used to determine mineralogical composition of samples. This testing aimed to determine the nature of the "opaque debris" observed previously, which formed a substantial volume of the material at 58% of the sample.

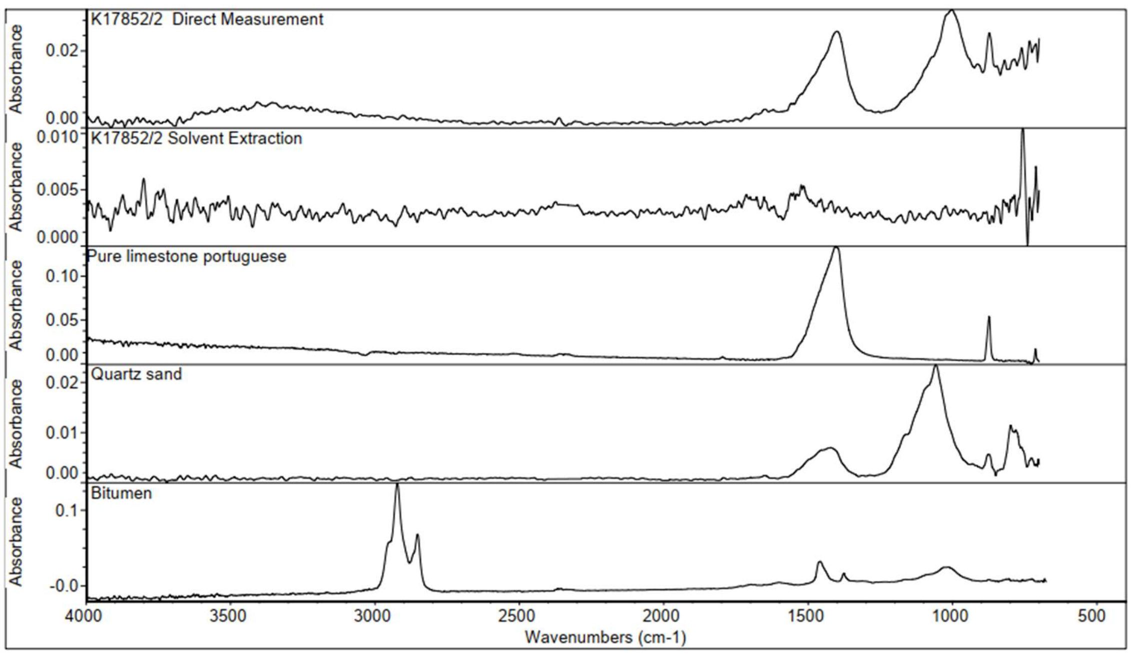

Figure 5. Fourier Transform Infrared Spectroscopy spectra for "north-mid" sample

Second Phase - Findings

The key finding from the second phase of investigation was that the opaque debris (and hence a large proportion of the sample) is clay-grade and predominantly manganese oxide, which would be opaque in the petrographic analysis and black to dark brown in the sample, both of which were found to be the case.

Table 4. SEM and EDX test results

Grain size	Approximate proportion (%)	Description of material
<5µm	80	Irregular clay-grade particles that are typically composed of clay minerals and high proportions of manganese oxide. The average composition of these particles exhibits over 50% manganese (54.2% MnO). These particles would be opaque in thin section and would be black to dark brown in hand specimen.
5-20µm	5	Spherical grains that are dominantly composed of clay minerals and rarely iron oxide. The composition [is typical of] clay minerals within the spherical particles.
20-50µm	15	Angular grains that are dominantly composed of lithic material. The grains are frequently composed of quartz and feldspar.

MONITORING DATA ANALYSIS

Having a better understanding of the nature of the material provided a good basis for testing the hypothesis but to further test the hypotheses into the likely origin of the deposits, advantage was taken of the growing understanding of the performance of the dam, from the preparation of a Monitoring Report – made in response to a separate recommendation made by the Inspecting Engineer (as a listed (safety) measure). The development of the Monitoring Report included a desk study to review existing monitoring data from the dam, published technical papers and available rainfall records.

Available data

There was a significant amount of data that had been recorded from the instrumentation that was utilised in the studies. However, the data was collected in such a way that interpretation of the data (and therefore the behaviour of the dam) was not possible without additional manual processing. Prior to use therefore, all data was assessed to allow anomalous data to be corrected or removed and presented in a format that allowed for visual and statistical assessment.

Seepage monitoring

Collection values for head over the V-notch weirs are typically collected weekly (going back to April 2008). These values were converted to flows and divided by the linear length of dam that drained into each of the respective V-notches to allow for effective comparison between them. The flows per linear metre of dam were plotted against reservoir level and rainfall readings from the nearest available rain gauge (**Figure 6**).

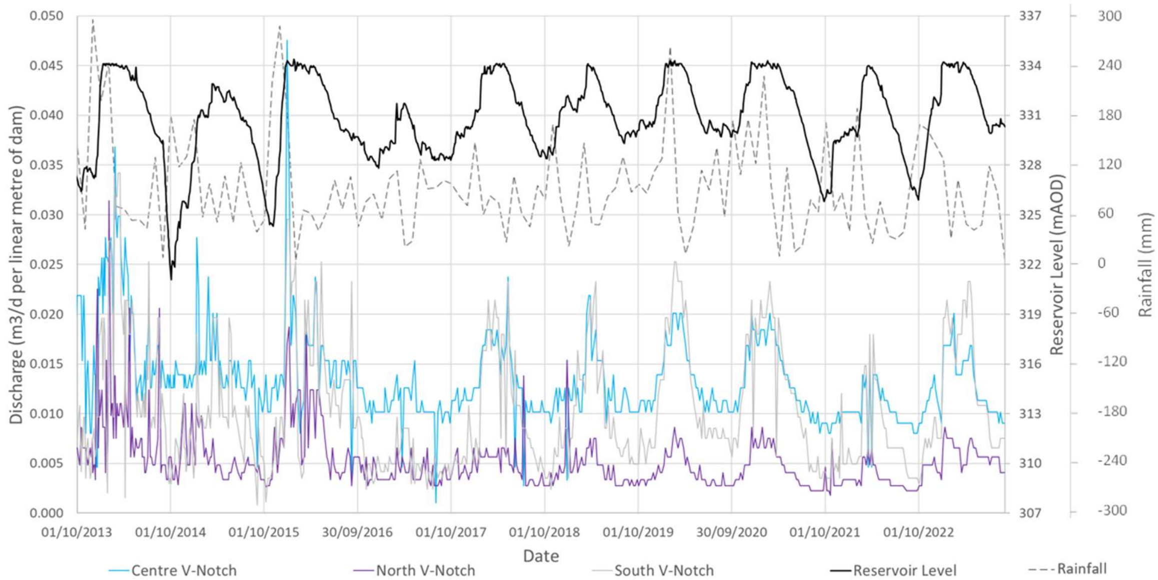

Figure 6. V-notch flow rates per linear metre of dam against reservoir level and rainfall

Pearson's Correlation Coefficient ("PCC") was also calculated to compare the relationship of flow rate per linear metre over each V-notch weir with both reservoir level and rainfall. The coefficient was calculated for all results recorded (between 2008 and 2023), results recorded between 2018 and 2023 and between 2022 and 2023 (Table 5). The aim of this calculation was to determine whether the correlation coefficient to reservoir level and rainfall was stronger in more recent years, which may indicate worsening conditions.

The PCC can vary between -1 to +1 with a score near 1 showing a strong positive correlation, a score of 0 showing no correlation; and a score near -1 showing a strong inverse correlation. However, it should be noted that the calculations for PCC have a large limitation, as it only takes into consideration the dates where readings were available for both variables being compared. Dates where only one of the variables were measured could not be used in the calculation of PCC and had to be selected out of the data set.

Table 5. Pearson's Correlation Coefficient for v-notch flows against reservoir level and rainfall

V-notch	PCC (flow vs. reservoir level)			PCC (flow vs. rainfall)		
	2008-2023	**2018-2023**	**2022-2023**	**2008-2023**	**2018-2023**	**2022-2023**
Centre	0.41	0.78	0.83	0.08	0.43	0.99
North	0.39	0.71	0.80	-0.22	0.07	0.75
South	0.67	0.84	0.88	0.30	0.06	0.47
Chamber A	0.18	0.16	0.00	-0.36	0.14	0.84
Tailbay (North)	0.20	0.19	0.26	-0.29	0.43	0.97
Tailbay (South)	0.14	0.16	0.07	-0.15	-0.22	0.42

An opinion was reached that the V-notch flows in the gallery showed a strong response against reservoir level, with a secondary response to rainfall. The maximum recorded flows appear to be relatively consistent over the range of data, with no obvious signs of increasing flows.

The PCC for unit flows against reservoir level showed a strong correlation across all V-notches in the gallery over the five years prior to the study, however the correlations were much weaker when considering all the data. This might suggest seepages measured at the V-notches are becoming more responsive to changes in reservoir level over time (maybe as a result of increasing leakage). However, it was recognised that there was a lack of available data to calculate PCC over the last year as there were very few dates where data was collected for both variables. This explains why values such as 0.99 were obtained for the centre V-notch flow rate against rainfall between 2022 and 2023, as there were only three dates with data for both variables during this period. This means the value obtained is much less reliable and this should be used with caution when interpreting behaviour.

It should be noted that the maximum combined flow of all three V-notches in the gallery was recorded at $10m^3$/day in 2023 at top water level, whereas the certificate of efficient execution of works (LRC, 1995) mentions that seepage through the core was $290m^3$/day in June 1983 and decreased to $140m^3$/day in April 1987. The seepage flows through the core are therefore substantially less than they were.

Piezometers

The available hydraulic piezometer readings on the upstream and downstream of the core were reviewed, with any erroneous readings identified, corrected or removed. The revised data was analysed (with mean, 5% and 95% confidence intervals calculated) and values plotted against reservoir level in order to visualise the effectiveness of the grout curtain.

The plots indicate that the piezometers upstream of the core (within the foundation) had a strong response to changes in reservoir level, while downstream piezometers showed a much weaker response (at lower head levels). Both responses were very much expected and showed that the grout curtain and core are still effective (Figure 7). PCC was also calculated between piezometric level and reservoir level, which showed a much stronger correlation for piezometers upstream of the core as compared to the downstream piezometers.

Charts were also plotted for standpipe piezometer levels against reservoir level over time, however there was no clear visual response, likely due to the larger volume of flow required to register a change in standpipe piezometer levels.

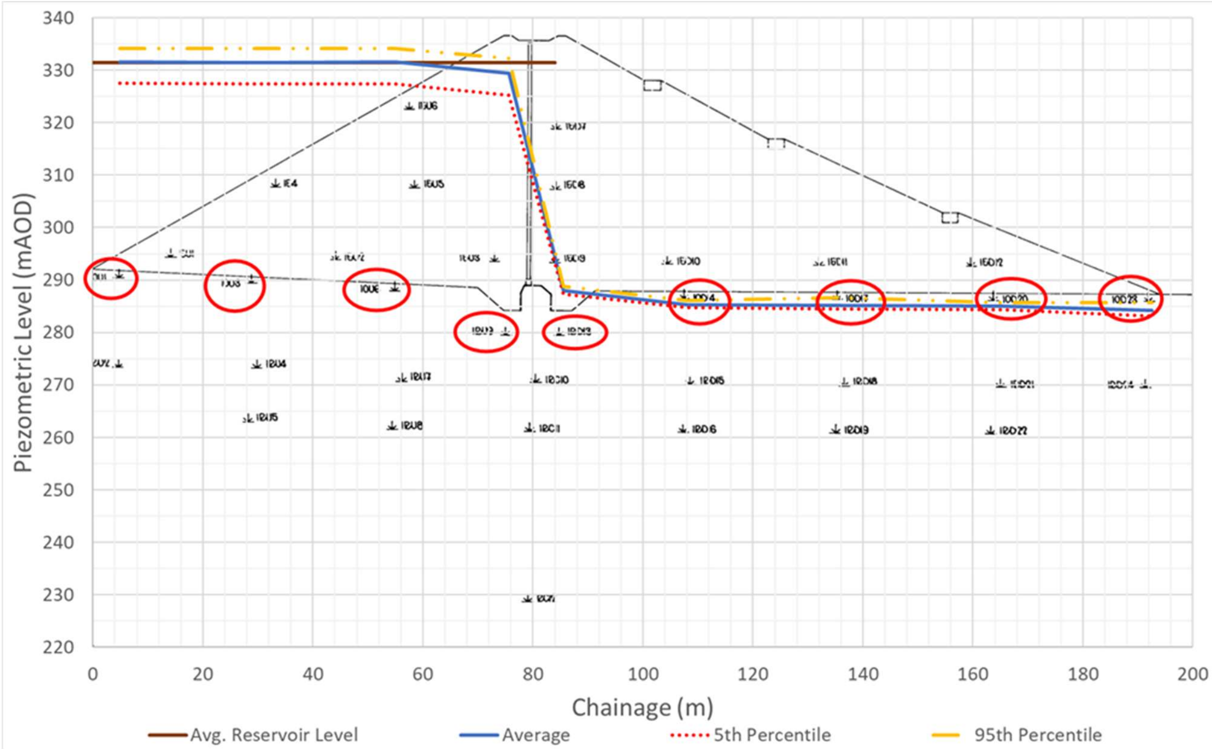

Figure 7. Mean, 5% and 95% confidence interval piezometric levels against average reservoir level through foundation piezometers

Movement monitoring

Plots were created for vertical movement of settlement stations at each elevation, to show the cumulative vertical movement of each point at different chainages from initial readings taken in 2010. Similarly, the cumulative horizontal movement of each station was plotted for movement in x-direction and y-direction as well as the vector sum. The plots showed no clear trend in either the cumulative vertical movement or horizontal movement of the points.

A previous report into the movement monitoring data (BRE, 2010) identified a gradual downward trend between 1982 and 2010. This, along with evidence that other dams with an asphaltic core behaved in a similar way (with the core settling vertically downwards and displacing horizontally downstream after construction) (Feng et al, 2020) led to a conclusion that the lack of an obvious ongoing trend is likely to be due to the current surveying techniques not having the levels of precision required to provide reliable results.

DISCUSSION

The petrographic analysis in combination with SEM and EDX determined the presence of quartz, calcite and precipitated calcite in the south and north-mid soil samples, with a high concentration of clay-grade particles consisting of manganese oxide in the north-mid sample. A review against construction records indicates that the reservoir basin was used as the borrow pit for the embankment shoulder material, and that the results of the analyses are consistent with the material used for construction of the embankment shoulders and transition zones.

The analysis also showed similar materials that would be expected based on concrete eroding adjacent to the core. However, if the eroded deposits were principally from the concrete (or from the core itself) the quantity of the material is such that it would be expected to lead to an increase in the seepage rates over time. The analysis of the monitoring data shows that the seepage rates against reservoir level are not worsening over time.

Organic compounds were found in chemical tests of the north soil sample, potentially indicating the presence of bitumen. However, the chemical and FTIR test results showed that the material was not bituminous or organic in nature. Given the above, it is therefore highly unlikely that the deposits are formed from eroding concrete adjacent to the core or from the core itself.

It is likely that the drainage material has come from a similar source to the aggregate used in the core or the reservoir basin, however the makeup of the material is not known as there are no construction records relating to this material. Given the age of the structure, it is likely that the drainage material is an open graded aggregate and therefore has no filtering effect on fines passing through the transition zone.

Considering the findings of the investigation and monitoring study, the following is a summary of the conclusions relating to the nature and likely origin of the channel deposits:

- It is highly unlikely that the material within the drainage channel originates from the bituminous core.
- It is highly unlikely that the material within the drainage channel originates from the concrete adjacent to the core.
- It is highly likely that the material within the drainage channel originates in whole or substantially from the transition zone.
- It is possible that the material within the drainage channel originates in part from the drainage material downstream of the bituminous core and situated between the transition material and the concrete gallery.

The completion of the investigation and monitoring study satisfied the aforementioned recommended safety measures, culminating in the issue of the relevant Interim Inspection Compliance Certificates (“IICC”) under the Act.

CONCLUDING REMARKS

The conclusions suggest that the material originates either wholly or substantially from the transition zone (and possibly in part from the drainage material). This suggests that the grading of the drainage material is such that it does not act as a filter and therefore, the current seepage is causing detachment of fines and their subsequent transportation from the transition zone into the drainage channels (the particle size distribution of samples taken from the drainage channel shows up to 95% of particles passing the 63µm sieve).

The scale of the dam structure is such that, if the hydraulic (seepage) conditions remain unchanged, the internal erosion and transportation of fines (and subsequent accumulation of deposits in the channel) should either remain steady or very gradually reduce with time.

Routine monitoring of flow rates across the v-notches in the drainage gallery will highlight any changes in seepage. Increased seepage rates may result in an increased rate of erosion of fine particles and possibly cause detachment and transportation of coarser particles. Given that the drainage material does not act as a filter, such internal erosion may over time lead to a small but meaningful local reduction in the volume of the transition zone (thereby reducing support of the bituminous core).

It is therefore important that, when making observations of any accumulation of deposits in the control gallery channel, attention is given to both the rate of accumulation and any noticeable change in the particle size distribution. There would be benefit in periodically collecting samples to confirm particle size distribution (sedimentation by pipette).

In addition, the need for clear interpretation of any collected monitoring data is vital to understand the behaviour of the dam. This includes analysing the data for any potential erroneous readings that may have arisen through human or instrument error. Interpretation using statistical analysis can be effective, however it is important to ensure that the various records are collected and recorded on the same days to allow for these statistical analyses to work effectively and that there are sufficient data points to allow for meaningful analysis.

ACKNOWLEDGEMENTS

We would like to express our sincere gratitude to Scottish Water for their invaluable support and collaboration, both during the investigation and in the preparation of this paper.

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Understanding the flood risk benefit of small reservoirs and recommendations for maintenance

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SYNOPSIS Coombe Valley Dam is a 4,500m^3 flood storage reservoir located in Teignmouth, Devon, constructed in the 1980s as compensation for development and subsequently transferred to Teignbridge District Council (TDC). As it is outside of the Reservoirs Act 1975 (HMG, 1075) (the Act), it has not had the stringent maintenance regime required for registered reservoirs that would complement its design function. However, responsibility remained under the Health & Safety at Work Act 1974 (HSWA) (HMG, 1974)and under Rylands v Fletcher 1868 (see in Howarth, 2002). Dam information had no assurance of accuracy, and the flood protection and standard were unknown.

This paper presents the hydrological study and hydraulic modelling employed to understand the dam's standard of protection and assess flood risk benefit provided by simulating a hypothetical dam removal scenario. Details of the model validation are presented to demonstrate how evidence from a recent storm was used to give confidence to the study with otherwise limited data. Assessment of the model outputs is discussed to estimate the number of properties benefiting from the reservoir.

Recommendations were made to allow TDC to operate the reservoir within the spirit of the Act. The paper provides management guidance to similar asset owners with limited experience as reservoir Undertakers (Owners and operators as defined under the Act).

INTRODUCTION

Reservoirs above 25,000m^3 capacity are currently required to be registered under the Act. However, there are significant numbers of flood storage and surface water compensation reservoirs throughout the UK that, whilst falling below the capacity required for the Act, may provide a level of flood protection which warrant assessments of their risk and maintenance within the spirit of the Act. Moreover, an estimated 1,503 additional reservoirs are likely to fall under the Act if the statutory volume is decreased from 25,000 m^3 to 10,000 m^3 (Penman and Golds, 2022). Schedule 4 of the Flood and Water and Water Management Act 2010 (HMG, 2010) makes amendments to the Act. Similar legislation changes in Wales in 2016 approximately doubled the number of statutory reservoirs.

Managing Risks for Dams and Reservoirs

Undertakers may be unaware of legislative requirements of the Act, or if they understand their reservoir to be non-statutory, may not recognise liabilities under common law, or additional duties under HSWA. Many smaller reservoirs are important assets providing a high level of flood protection which have associated risk of failure due to reduced statutory requirements. These assets would therefore benefit from being maintained in the spirit of the Act, regardless of whether they become statutory in the future.

THE SITE AND CONSTRAINTS

Coombe Valley Dam is a 4,500 m^3 capacity flood storage reservoir situated 1.5 km northwest of Teignmouth in the County of Devon along the Bitton Brook. An aerial view of the flood storage reservoir is provided in Figure 1. The embankment slopes are 1 in 3 on both the upstream and downstream faces with a 1m wide crest. A metal walkway with locked security gate provides access from the crest to an outlet tower (the 'spillway').

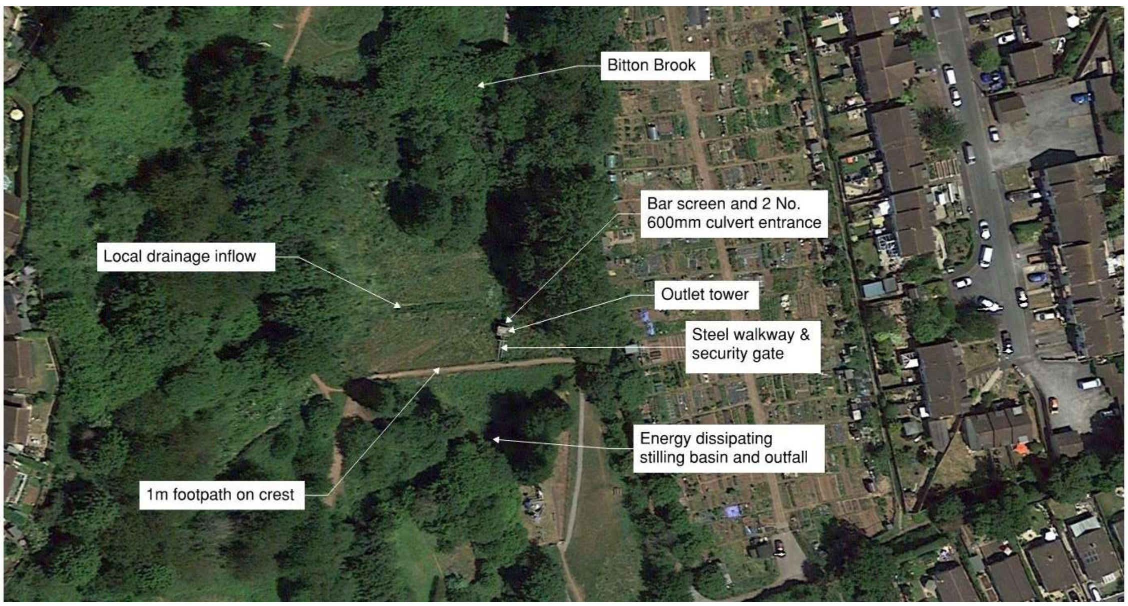

Figure 1. Coombe Valley Dam plan overview

The reservoir was constructed in conjunction with a local housing development in the 1980s with ownership later transferred to TDC. Whilst not a statutory reservoir under the Act, the Undertaker recognised their liabilities and duty under HSWA and therefore commissioned Binnies to undertake a flood study and optioneering report.

The overflow is a vertical 975mm diameter bellmouth culvert within 2.2m stacked concrete precast manhole rings. The top of the bellmouth spillway and top water level (TWL) is at 45.53m AOD according to historic drawings. The footpath crest elevation of the embankment of 46.40m AOD provides 870mm of freeboard. Dual 600mm diameter culverts at invert 40.90m AOD join the 975mm outlet culvert extending through the embankment. Upstream control is provided by two penstocks maintained at 50% open since construction. A near vertical bar screen is provided over the entrance to the culvert. An energy dissipating stilling basin is located on the downstream side which contains a concrete baffle. Historic construction drawings show the core as silty material with embankment fill detailed as gravelly material.

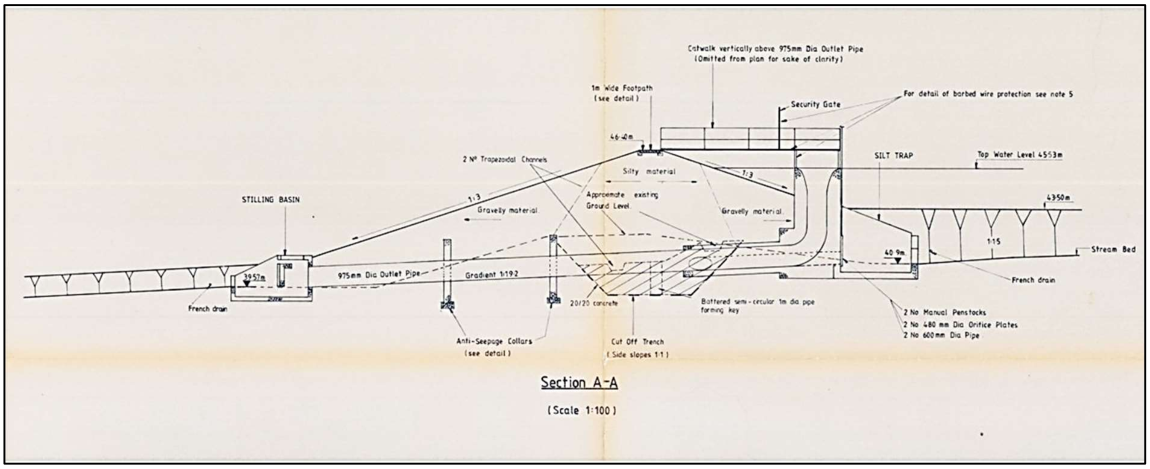

Figure 2. Dam cross section from historic drawing

Current Condition

A site visit was conducted on 4th October 2023 to assess the condition of the reservoir. Figure 3 shows the overgrown state of the upstream face and outlet structure. The vegetation encroaching on the screen above the precast manhole rings housing the 975mm overflow structure presents a blockage risk. The downstream face was similarly overgrown. Figure 4 shows the current condition of the bar screen over the dual 600mm diameter culvert entrance in connection with the overflow. TDC noted that this screen was prone to debris build-up and that cleaning it was a persistent maintenance issue due to the heavily wooded upstream catchment. TDC has a maintenance contract which includes yearly vegetation and mechanical and electrical services. The contract includes reactive grill maintenance but is not sufficient to meet the needs of the dam. TDC has considered installation of a tree catcher within the catchment to improve the issue. Vegetation was also growing in the security fence at the outlet to the downstream energy dissipating chamber, preventing surveillance of the condition within the culvert outlet.

Figure 3. Photo showing overgrown condition of upstream face and overflow structure

Figure 4. Photo of bar screen over inlet works prone to blockage.

METHODOLOGY

Overview

Greater detail regarding the dam's flood protection function needed to be established to be able to evaluate management options. Hydrological analysis and hydraulic modelling were undertaken to understand the current level of flood protection provided by the dam (the 'baseline'), and the potential impact on flood risk from its removal ('dam removal').

The modelling study approach was as follows:

1. **Hydrological analysis** to generate flows in the Bitton Brook for the catchment to the dam (the 'upstream' catchment) and the downstream catchment.
2. Flows were then routed using **hydraulic models**
   i. For the baseline scenario only, the upstream catchment flows were firstly routed through a one-dimensional (1d) model of the dam to a) understand the dam's current standard of flood protection, and b) to create attenuated flows after passing through, and potentially over, the dam control structures.
   ii. For both the baseline and dam removal scenarios, flows downstream of the dam were simulated in a two-dimensional (2d) model of the downstream catchment to understand flooding.

Catchment and Hydrological Analysis

The following summarises the hydrological approach used to estimate model inflows for the sub-catchments upstream ('catchment 1') and downstream ('catchment 2') of the dam:

1. Delineation of the **catchment boundaries** ('watersheds') for catchments 1 and 2.
2. Retrieval and review of **hydrological catchment descriptors**.
3. Determination of the **critical storm** duration for the whole catchment.
4. Calculation of **peak flow** rates for different flood events ('return periods').
5. Generation of a **hydrograph shape** to create model inflows.

The former 2016 modelling study was reviewed to confirm suitability. The 2016 study performed catchment delineation via the Flood Estimation Handbook (FEH) web-service which is based on coarser resolution data and may be inaccurate for small catchments. The Bitton Brook catchment delineation was therefore revised by performing GIS analysis on LiDAR Digital Terrain Model (DTM) data together with appropriate visual inspection. Figure 5 shows the revised Bitton Brook catchment extent in orange.

The sub-catchment to the dam ('catchment 1 previous') was calculated using the same approach (shown in Figure 5 by the yellow outline). However, following a site visit it was apparent that this calculated area did not include the areas serviced by the local surface water network draining into the upstream storage area. This was confirmed by service plans available from South West Water (SWW) as draining a portion of the area to the west of the dam that was understood in the 2016 study to be within the downstream sub-catchment.

Catchment 1 was adjusted using manual inspection of SWW's plans to include the yellow shaded areas shown in Figure 5. The updated catchment 1 is shown by the red line where

Table 1 compares catchment areas. This analysis shows that the catchment area of the dam is 0.28km^2 larger (13%) than previously estimated by a 2016 study.

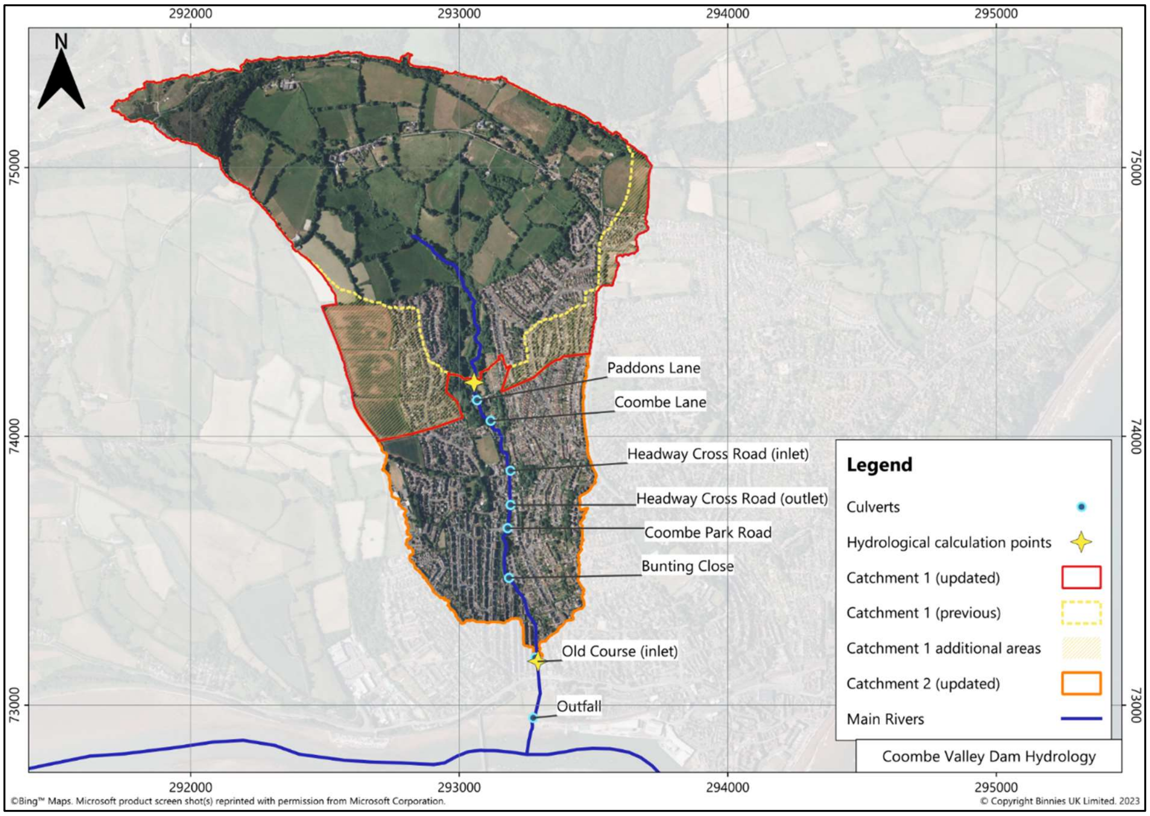

Figure 5. Revised catchment to the dam and the Bitton Brook outfall

Table 1. Catchment areas [km^2]

Catchment	Former Study	FEH	GIS analysis	Updated
Bitton Brook	n/a	1.93	2.15	**2.15**
Catchment 1	1.39	1.39	1.29	**1.57**
Catchment 2*	n/a	0.54	0.86	**0.58**

*Area for catchment 2 is the additional area such that the Bitton Brook is the sum of catchment 1 and catchment 2.

The Revitalised Flood Hydrograph 2 (ReFH2) rainfall run-off method was used to estimate peak flows. This method generates peak flows for a given flood event by routing rainfall depths for a given storm duration (Depth Duration Frequency [DDF]) through an empirical model controlled by hydrological catchment descriptors.

The storm duration was iteratively adjusted within this model to obtain the maximum peak flow rate for the whole Bitton Brook catchment. The critical duration for both catchments was found to be five hours. The ReFH2 model was also used to generate design storm hydrograph shapes for the 5-hour critical storm to which peak flows were fitted.

Reservoir Model

A 1d hydraulic model of the dam and reservoir was constructed in Flood Modeller software. This approach allowed the capacity of the reservoir and hydraulic controls (i.e. the outlet, spillway, and dam crest) to be simulated for a range of flood magnitudes. The 1d model was also used to generate outflow hydrographs from the dam to apply to the flood routing model.

Managing Risks for Dams and Reservoirs

Figure 6a shows a schematic of the 1d reservoir model 'nodes'. The model consists of an inflow connected to a 1d reservoir unit. The flow routes out of the 1d reservoir are controlled by two 1d-spill units representing the spillway and dam crest, while two orifice units represent the outlet penstocks. A small section of dummy channel with a normal depth boundary was inserted to provide a downstream boundary condition.

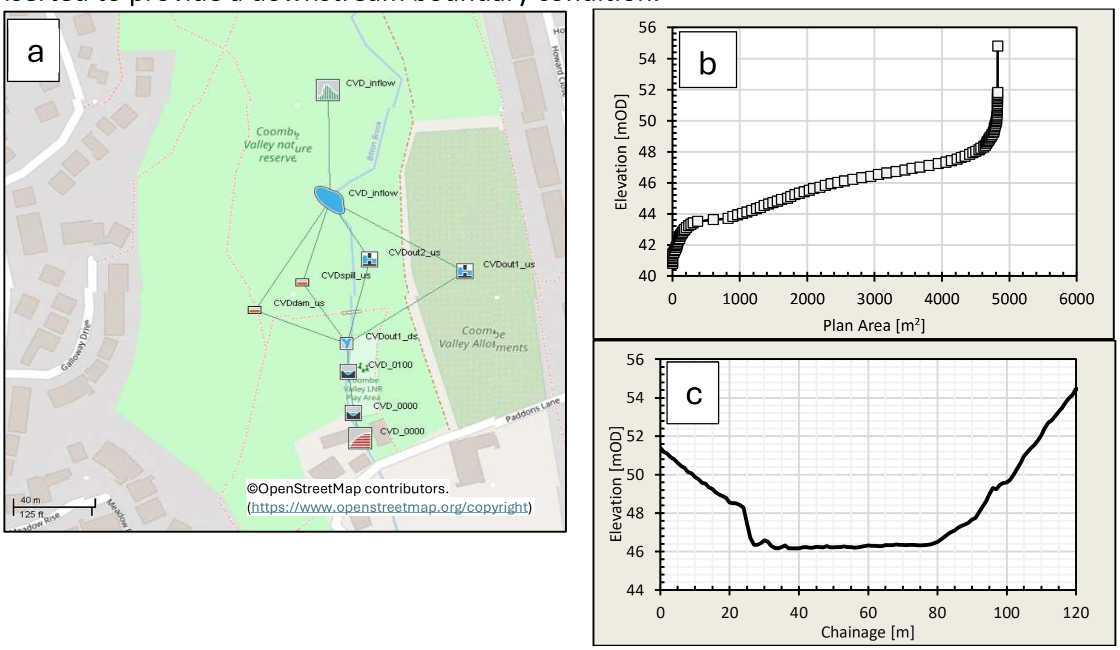

Figure 6. 1d Flood Modeller reservoir model schematisation

The capacity of the reservoir impounded by the dam was calculated using Flood Modeller's in-built 1d-reservoir tool which creates elevation-area relationships for a given input topography. The LiDAR DTM data was analysed to generate this relationship, with results shown in Figure **6**b.

The 1d spill representing the circular concrete spillway (Figure 3) was set to 45.53m AOD at a length of 3.06m as given in the 1984 design drawings. No detailed topographic survey of the dam crest was available. As such, the crest elevations were generated from GIS analysis of the LiDAR DTM, as seen in Figure 6c.

The two penstocks which control flows out of the reservoir were represented as two circular orifice units. The invert levels and bore areas of these units were defined using the 1984 design drawings. The penstocks are operated at 50% closed, as such the bore area was reduced to reflect this.

Only the existing dam conditions were simulated for the reservoir model. This is because it was assumed that, should the dam be removed, the flows passing the former location of the dam would be the same as those arriving upstream and therefore there would be no need to route these inflows through a model without the dam in-place.

Flood Routing Model

A 2d hydraulic model of the catchment downstream of the dam was constructed and simulated using TUFLOW hydraulic modelling software. TUFLOW is an industry standard tool for simulating flood flows for studies of this type.

The 2d model simulates both the channel and floodplain in 2d. Culverts were embedded in the 2d domain as 1d channel features, however the open channel sections of the Bitton Brook were not modelled as 1d elements (as is often customary) given the absence of suitable survey data. Figure 7 shows the extent of the 2d model, extending from the downstream face of the dam in the north, and ending at the frontage with the River Teign estuary in the south.

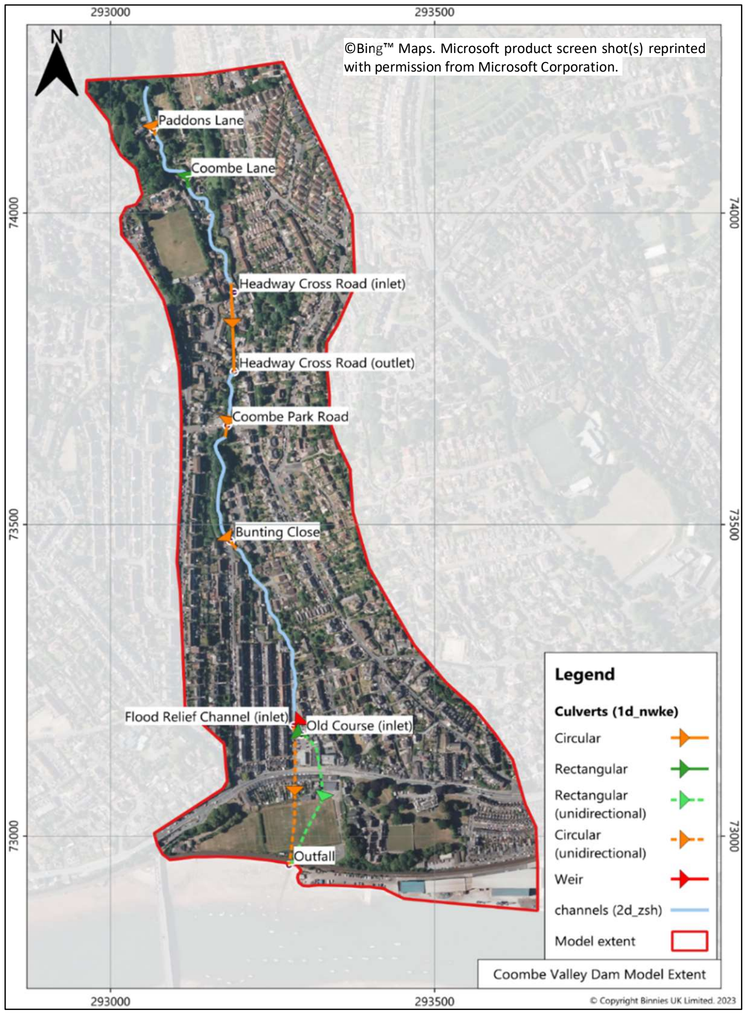

Figure 7. 2d model extent downstream of the dam

The model topography was based on the 1m LiDAR DTM. However, the narrowness of the Bitton Brook combined with high vegetation coverage meant that its representation within the raw LiDAR DTM was limited. As such, to ensure a reasonable representation of in-channel flows the Bitton Brook open channel sections were enforced within the DTM using additional model features (2d_zsh layers in TUFLOW as shown by the blue lines in Figure 7).

Flood extents in the catchment are strongly influenced by the various culverts and bridges that lie within the Bitton Brook. In addition, the LiDAR DTM did not have any culverts or bridges filtered from the raw elevation model data and therefore the elevation model represents ground level above these structures. The example in Figure 8 at Bunting Close shows how the recording of ground surface levels above the culvert creates an artificial dam across the watercourse. Therefore, the representation of these structures along the Bitton Brook was an important component of the model.

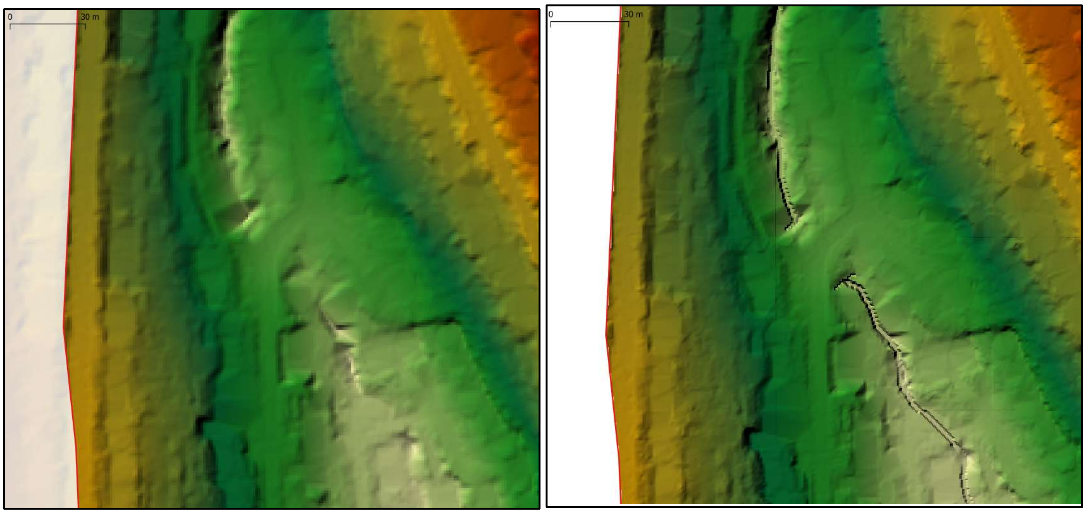

Figure 8. Bitton Brook channel enforcement in the DTM

Eight 1d culverts were embedded into the model to represent the in-channel features shown by the arrows in Figure 7. The dimensions and invert levels of the culverts were supplemented with site measurements provided by TDC or estimated where necessary. Survey data of these culverts and the open channel sections would provide more accurate information, however the absence of this data does not limit the conclusions of this study. One weir has been included to represent the side spill from the Bitton Brook into the flood relief culvert. The flood relief culvert and the old course (dotted green line in Figure 7) were both assumed to have tide flaps at their outfalls into the River Teign estuary.

The outlets of the flood relief culvert and old culvert course were connected to a 1d boundary condition which simulates tidal variations in the River Teign estuary. A Mean High Water Spring (MHWS) tide level and profile was applied such that the peak of the tide occurred at the peak of the flood giving conservative conditions. The MHWS level was taken from the EA Coastal Flood Boundary dataset while a representative profile was extracted from Admiralty Total Tide for the estuary side of Teignmouth.

The 2d model was simulated for the 2, 5, 10, 50, 100, 200, and 1000-year floods for the baseline and dam removed scenarios. The impact of climate change was also tested using the 1 in 100-year flood by uplifting peak flows by 46% in accordance with EA guidance.

RESULTS

Reservoir attenuation

Prior to a site visit in October 2023, the catchment experienced intense rainfall on 17 September 2023, resulting in the reservoir filling and almost reaching the spillway. Wrack marks and debris from the event were evident during the visit and allowed the peak water level in the reservoir to be estimated (at approximately 0.08m below the level of the overflow, at 45.45m AOD). The model was validated against this event to see if a similar water level was achieved.

The recorded rainfall on 17 September 2023 from the nearest rain gauge (4km away at Ashcombe) was retrieved and processed through the ReFH2 model to generate an estimated inflow hydrograph. This gauge recorded 66mm of rainfall over five hours which, according to the FEH DDF model, equates to a 1 in 140-year event. The 17 September 2023 estimated inflows were run in the reservoir model and compared to the estimated maximum water level recorded from the site visit. Figure 9 compares the model water level (block line) in the reservoir to the estimated flood level of 45.45m AOD (dotted blue line). The modelled water level is within 0.2m of the estimated flood level giving reasonable confidence in the modelling approach.

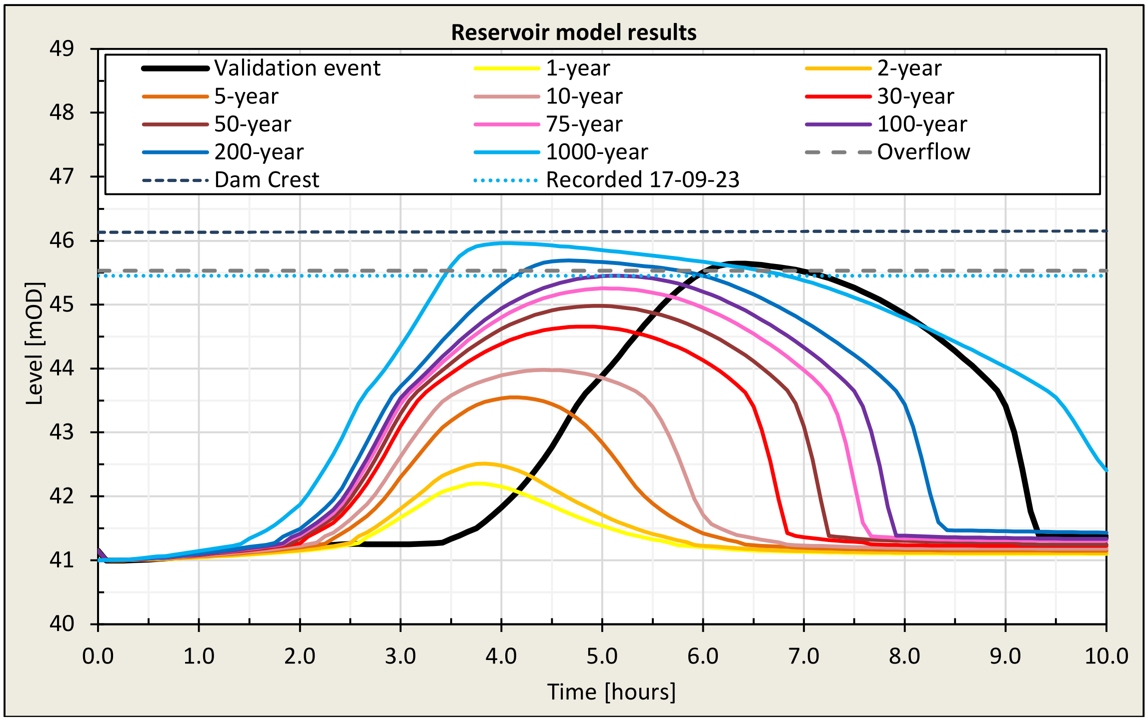

Figure 9. Validation event water level profile upstream of the dam

Current Flood Protection

The results from the 1d reservoir model are also plotted in Figure 9 below showing how water levels vary with time for the 5-hour storm design events. The dam spillway (45.53m AOD) and crest (46.2m AOD) are shown by the dashed and dotted lines, respectively, indicating when the levels would exceed each threshold.

The maximum water levels upstream of the dam show that the spillway would not become activated until the 1 in 200 year ('Q200') flood, and that the crest would not be exceeded even during the 1 in 1000 year flood. The standard of protection of the dam is therefore at least a 1 in 1000 year.

Dam Removal Scenario

Table 2 summarises the impacts of removing the dam in terms the estimated properties added to the flood outline for the range of modelled floods demonstrating the flood risk benefit provided by the dam. Figure 10 shows an example of the impact of flood extent for the 1 in 10-year flood. It is evident that the dam provides protection to a significant number of properties (between 11 and 18 depending on the flood) when compared to the absolute number of properties at risk in the baseline.

Table 2. Dam removal summary in terms of estimated flooded properties

Flood event [year]	Baseline	Dam Removed	Change
2	0	0	0 (none flooded)
5	0	0	0 (none flooded)
10	3	21	18
50	10	27	17
100	12	29	17
200	29	40	11
1000	60	71	11
100+46% climate change*	43	54	11

*The upper allowance for peak river flow for the 2050s has been used from the South Devon Management Catchment.

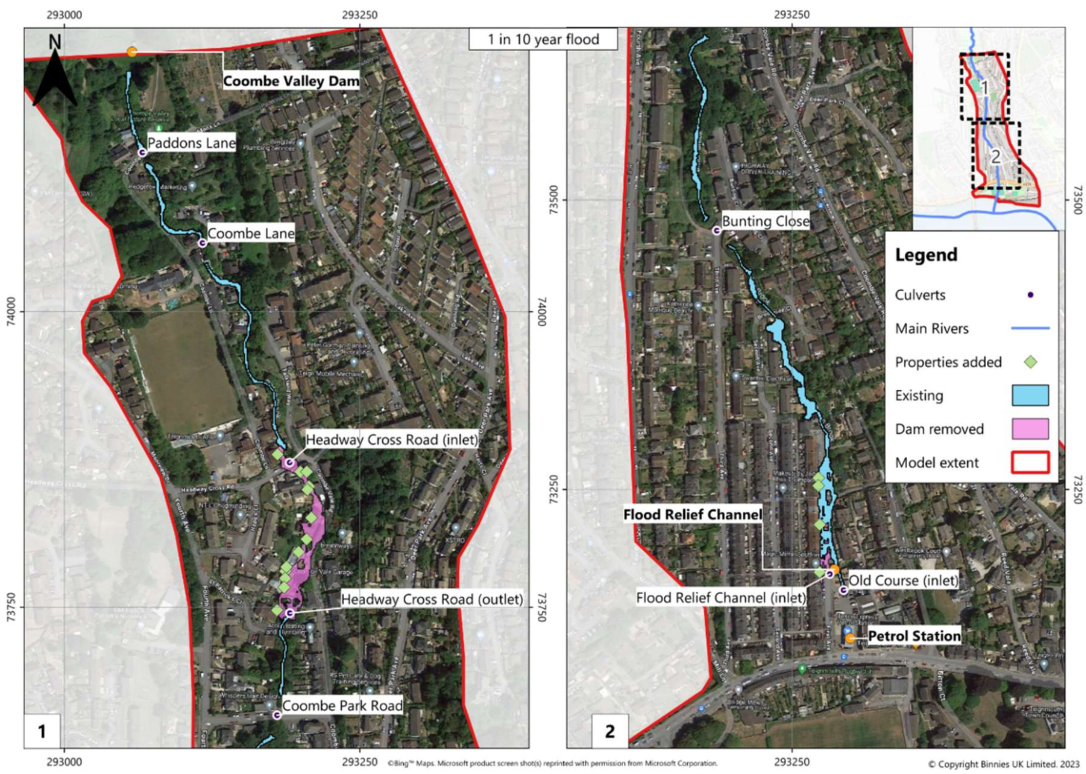

Figure 10. Example change in flood outline by removing the dam – 1 in 10 year flood

ANALYSIS OF IMPLICATIONS ON MANAGEMENT

With confirmation of the level of protection offered by the dam, an options appraisal was carried out. Scenarios considered included (1) do nothing, (2) remove dam, (3) retain dam at reduced capacity, (4) retain dam at current capacity and bring up to standard, and (5) retain the dam at increased capacity. Given the high standard of protection provided by the dam and the increase in properties impacted by the dam removal scenario, the option to retain the dam at its current capacity and treat it within the spirit of the Act, was identified as the preferred option. This option balanced capital costs with flood protection benefits whilst ensuring the Undertaker's duties under relevant legislation were met.

Recommendations for maintenance and surveillance

To bring the reservoir up to standard in the spirit of the Act, a series of recommendations were made. To better inform the Undertaker how deterioration of the dam could lead to risk of failure, several potential hazards at the dam were provided based on EA guidance (EA, 2016). A Risk Assessment for Reservoir Safety (RARS) following EA guidance (EA, 2013) has not been carried out at this stage. Risks to the embankment included vegetation overgrowth, potential animal activity, mitre runoff, seepage, settlement, crest fissuring, and internal erosion along the culvert. Other external threats included blockage of screens and overtopping. A list of recommendation made to bring the reservoir up to standard in the spirit of the Act were provided and are listed in **Table 3**.

Table 3. Recommendations to bring the reservoir up to standard of the Act

Feature	Recommendations
Embankments and Crest	• Clear small vegetation and establish good grass cover • Check for animal burrows. Engineer/ecologist to advise on removal • Check for signs of cracking, movement, or creep
Outlet culvert and Overflow	• Carry out CCTV survey to establish condition of assets • Reseal joints as needed • Clear any debris blocking screens or culvert entrance/outlets
Instrumentation	• Install telemetry system similar to 'Meteor' used at EA reservoirs • Install gauge board for water level monitoring telemetry calibration
Emergency Planning	• Develop a plan similar to an 'On-Site Plan'. Include emergency response contacts, drawdown rate analysis, and valve information

Additional recommendations made to improve maintenance and surveillance in the future are listed in Table 4. These were established based on common industry practice as well as EA and CIRIA guidance (EA, 2011a, 2011b; CIRIA, 1996, 2003, 2015, 2019, 2020)

Table 4. Recommendations for future surveillance and maintenance

Recommendations for future surveillance	Recommendations for future maintenance
• Monthly visits by an appointed 'reservoir surveillance engineer' to visually inspect dam and remove debris from screen • Inspect dam following flood events for blockages, seepage, or settlement • Carry out annual inspection and report similar to Section 12 • Maintain a document similar to Prescribed Form of Record to document maintenance and record water levels • Five yearly asset survey similar to T98	• Maintain grass below 150mm • Regular operation of penstocks and valves • Clear any debris around overflow screen or upstream debris screen • Check condition of safety equipment • Monitor silt buildup in upstream channel • Consider installation of tree catcher upstream in catchment

CONCLUSION

Modelling and optioneering at Coombe Valley Dam supported the Undertaker in understanding their roles and responsibilities under relevant legislation and in the spirit of the Act. Recommendations made helped establish suitable inspection and maintenance regimes. This helped secure revenue expenditure to ensure compliance and flood risk benefits are maintained in the future. TDC is considering similar studies for other non-statutory reservoirs.

As has been demonstrated by the recent experience following the enactment of >10,000m^3 capacity reservoirs in Wales in 2016, there are numerous dam structures which have a capacity below 25,000m^3 but still, due to location and/or height, could cause damage to property or life is they failed. This is not just limited to >10,000m^3, but structures below that capacity, particularly where they have been constructed for flood alleviation purposes such as Coombe Valley Dam.

It is important that these structures are recognised as dams, with the same issues, liabilities and maintenance requirements as registered dams under the Act, whilst providing additional benefits such as flood alleviation. This paper aims provide an example of how similar structures can be analysed to subsequently inform the Undertaker of benefits provided, limitations, and best ways to operate and maintain the structure going forward to minimise risks, using existing guidance, methods and standards that are available within the industry.

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The British Dam Society

Lessons Learnt from the First Inspections of Reservoirs (with capacities of 10,000m^3 - 25,000m^3) in Wales

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SYNOPSIS This paper outlines the process and challenges Dŵr Cymru Welsh Water (DCWW) has faced working with our regulator Natural Resources Wales (NRW) to carry out the first inspections of 52 reservoirs with capacities of between 10,000m^3 and 25,000m^3. These reservoirs included a small number of impounding and non-impounding assets but the majority are service reservoirs (SRVs) and raw water tanks (RWTs).

BACKGROUND

Following the change of registration threshold for reservoirs in Wales on 1st April 2016, we initially identified 39 reservoirs that fell between the old threshold of 25,000m^3 and the new, lower threshold of 10,000m^3. Those 39 were made up of 34 service reservoirs, four impounding reservoirs and one non-impounding reservoir. Prior to this threshold change, we had a total of six service reservoirs that came under the Reservoirs Act, 1975 (hereafter, 'the Act').

During initial discussions with NRW that took place in 2020, we set out our own ambition to have all assets with capacities between 10,000m^3 and 25,000m^3 formally inspected by 31st March 2025. During these discussions it was agreed that we should prioritise the first inspection of the 34 service reservoirs.

We subsequently received the regulatory position statement *'First Inspection of Reservoirs Prior to Designation under the Reservoirs Act 1975.'* (NRW, 2020) that set out:

> *"This Position Statement sets out how NRW will regulate the inspection of reservoirs under the Reservoirs Act 1975 during the period of risk designation 2020-2025. It allows undertakers of reservoirs to carry out reservoir inspections prior to a confirmed risk designation - before an inspection is legally required - and have that inspection carried forward and accepted as valid when the reservoir designated. It also clarifies the use of section 8 to secure a Final Certificate for a large raised reservoir and the timing of inspection under section 10."* (NRW, 2020).

This gave us a regulatory framework within which we needed to carry out the first inspections.

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PROGRESS TO DATE

We have made good progress in our first inspection programme, with 28 of the original 34 service reservoirs having fully undergone first inspection under Section 8 of the Act and a further two service reservoirs having undergone partial inspection (where more than one compartment requires inspection, these are usually completed sequentially). Only four of the original 34 service reservoirs identified have not yet been inspected.

However, we have since identified a further 13 assets that meet the criteria for first inspection, making a total of 52 assets requiring first inspection. Of these additional 13 sites, seven have been fully inspected.

To ensure that dam safety is maintained, Welsh Water made the decision to appoint a Supervising Engineer to each site immediately after the change in threshold was announced in 2016, despite this not being a legal requirement whilst these reservoirs are awaiting designation. For small impounding assets that are <10,000m^3 and do not fall under the ambit of the Act in Wales or those designated as not high risk, we assign internal trainee Supervising Engineers to undertake examinations which are reviewed and approved by an internal Supervising Engineer.

CHALLENGES ENCOUNTERED

Although we have made good progress on carrying out first inspections, we have experienced several challenges that were unforeseen, underestimated or not fully understood. These are presented in no particular order:

1. Alignment with existing work.
2. Operational challenges in emptying service reservoirs.
3. Limited supply chain / availability of experts in service reservoirs.
4. The number of Measures in the Interest of Safety (MITIOS) arising from first inspections.
5. The availability and willingness of Panel Engineers to carry out first inspections on service reservoirs.
6. Inconsistency in reports and differing approaches to the issuing of certificates from Inspecting Engineers.

1. Alignment with Existing Work

The first inspection programme was an additional programme of work that sat alongside our existing regime of statutory inspections under Section 10 of the Act, as well as a significant capital investment programme. As is well documented, there is a very limited pool of All Reservoirs Panel Engineers (ARPEs) so this required careful thought and planning on how we would use these. By the end of Asset Management Plan period 7 (AMP7 - that is the31st March 2025) we will have carried out 43 inspections under Section 10 of the Act, in addition to delivering a £147m capital investment programme. Attempting to carry out 52 first inspections under Section 8 of the Act on top of this was ambitious.

2. Operational Challenges with Service Reservoirs

Service reservoirs which supply treated drinking water are subject to the Act in Wales if they are designed or capable of storing >10,000m^3 of water above the natural level of the land surrounding the reservoir. Service reservoirs are also subject to statutory regulation by the Drinking Water Inspectorate (DWI) to maintain hygiene standards.

Emptying of a service reservoir is also needed to enable cleaning and disinfection to take place to meet bacterial standards for drinking water. If not managed appropriately, these actions may interrupt water supply. A service reservoir needs to be emptied to enable a full and proper safety inspection. There may be multiple cells within a reservoir which can be drawn down independently of each other at different times, but the Inspecting Engineer must be satisfied about all cells to complete an inspection.

Service reservoirs are often inter-dependent with each other to allow continual network supply. The timing of drawdown must be achieved in a way which maintains continuity of supply. Removing a service reservoir from operation for inspection and cleaning activities reduces the resilience of the distribution system and continuity of drinking water supply. Additionally, if there are works ongoing at the Water Treatment Works (WTW) which supplies the service reservoir or water network system this can also add a significant constraint by further reducing the resilience or the ability of the system to recover.

In our experience, when tanks have been emptied for inspection, they have often been kept empty whilst water quality defects are repaired. This has had a knock-on effect of when the next cell / nearby service reservoir can be scheduled for drawdown and inspection . In some cases, service reservoirs have remained empty for up to two years for these reasons. This also raised the question of the maximum permitted period of time between the inspection of multiple cells at the same service reservoir.

3. Limited Supply Chain and Expertise

As an undertaker that only had six service reservoirs under the Act up until 1st April 2020, we had a small framework of contractors approved to work on service reservoirs. Many of these contractors were local companies, not geared up to work on a national programme. In addition to this, there was limited understanding of the requirements of the Reservoirs Act by some of our own colleagues in Production and Distribution. Whilst the business has worked hard over the past 8-10 years to raise the profile of reservoir safety through the promotion of sizeable capital projects on our portfolio of impounding reservoirs, there was little to no mention of service reservoirs and other assets (raw water tanks etc.). Although this has definitely improved throughout the delivery of our first inspection programme, there is further work to be done to educate colleagues about the need for external supervision when carrying out work on these assets.

4. MITIOS Arising from First Inspections

To date we have had 85 MITIOS from first inspections carried out under Section 8 of the Act and with limited resources this has placed significant pressure on a small team. The number of MITIOS on service reservoirs has exceeded the number of MITIOS on our portfolio of impounding reservoirs for significant periods of the last three years.

5. Availability and Willingness of Panel Engineers

With the well discussed small pool of ARPEs and members of the SR Panel (Peters et al, 2018) finding panel engineers willing to undertake inspections has at time been a challenge. We have encountered several occurrences with engineers where they have no availability to work on or inspect a service reservoir but when asked about working on an impounding reservoir (sometimes a few days later) they have availability. It is not clear whether this has been a coincidence on a number of occasions, due to a potentially greater financial reward for services on impounding reservoirs or having less of an interest in working on these structures.

6. Inconsistency and Differing Approaches to Issuing Certificates

This is the issue that has had the biggest impact on the delivery of the programme, and one that we did not anticipate. The NRW position statement states: "*All large raised reservoirs must be supplied with a Final Certificate, along with a Certificate of Efficient Execution of Works. For reservoirs already constructed but only registered since 2016, these certificates are provided by a Construction Engineer under section 8 of the Reservoirs Act 1975. For new reservoirs constructed since 2016, or those which are to be altered, section 6 applies"* (NRW, 2020*).* When we received the first reports under Section 8, it was immediately clear that each ARPE had a different approach to this issue. Some included the Final Certificate as an annex to the Section 8 report, whereas some were not willing to issue a Final Certificate until all MITIOS had been certified complete.

This became particularly problematic when we made the decision to appoint a singular Qualified Civil Engineer (QCE) to sign off all MITIOS arising from the first inspection programme. This decision was taken with the best of intentions, to ensure that there was one standard required for overflow assessments, drawdown assessments, condition surveys etc. This would give us one point of contact who would be familiar with our programme of works and the challenges facing us. Whilst this worked well in terms of securing 10(6) certificates, it presented new difficulties when it came to issuing a Final Certificates.

DESIGNATION AND INSPECTION OF OTHER SMALL RESERVOIRS

In addition to traditional service reservoirs that form a large part of this programme of first inspections, there are a number of other assets such as raw water tanks and settlement lagoons that are also included. Some of these were not part of our original programme because it was not immediately obvious that some of these assets - usually located at Water Treatment Works (WTW) or pumping stations - had the potential to hold large volumes above natural ground level and therefore met the criteria of the Act.

An example of this is the settlement lagoon at Bolton Hill WTW near Haverfordwest, where we were aware of the traditional service reservoir and the two on-site raw water tanks but had never considered the lagoon as having the potential to fall under the Act. Whilst on site carrying out the first inspection of one of the raw water tanks, the Inspecting Engineer identified the potential of the lagoon to require inspection. The lagoons consist of three parallel compartments, each around 90m long and 21m wide.

The available drawings show them as built by infilling a valley, with maximum height above the base of the valley of around 7m although the lagoon depth was only 2m. There is some uncertainty over whether what is shown as "original ground" at the downstream toe was an earlier infilling for the inlet main to the WTW. The lagoons allow the WTW to normally operate as a "dry site" with wash and supernatant water recycled back into the raw water supply. They also provide a means of improving water quality, by passing site drainage from west to east, with the lagoons desilted by excavation every few years.

As the lagoons had not previously been identified as falling under the Act, it is fair to say that they had not been maintained to the same level as the rest of our portfolio. Amongst other things, the vegetation had been allowed to become extensive, the chamber covers were not visible, the washouts were no longer operational, and there was very little understanding of the flows between the three lagoons or the drawdown capacity.

FIRST INSPECTIONS OF ASSETS NO LONGER IN OPERATION

In the process of identifying assets that met the new, reduced threshold of 10,000m^3, a number of assets were recognised as non-operational. Having carried out a series of checks to ensure these assets did not form part of any drought plans or total loss contingency plans, it was decided that abandonment under the Act would be pursued. However, inspecting these assets under Section 8 before completing any required Measures in the Interest of Safety prior to going down the abandonment route did not make practical sense. This was raised during discussions with NRW and a different approach for these assets was agreed: these assets could be inspected under Section 14 of the Act and formally abandoned once all recommendations are completed and the Inspecting Engineer is satisfied that the measures have been efficiently executed.

Llwyn Du

One of the assets that met the above criteria was Llwyn Du service reservoir, located in Abergavenny in Monmouthshire. The service reservoir was taken permanently out of service in 2012 due to leakage and water quality issues. The first inspection of the asset was completed in January 2022, with an excerpt from the Inspection Report stating:

This is a report under Section 14 of the Reservoirs Act 1975 (1975 Act) as amended by the Flood and Water Management Act 2010 (FWMA), and includes the following items in Welsh Statutory Instrument 2016 No.80 (W.37)

- *items specified under Schedule 5, and*
- *a certificate, as prescribed under Schedule 4.*

This inspection was commissioned by the Undertaker, Dŵr Cymru Welsh Water (DCWW), as

- *Although the reservoir was taken out of service and emptied in 2012 due to leakage and water quality issues it was never formally abandoned under reservoir safety legislation.*
- *It was also overlooked in the transfer for regulation from the Environment Agency to NRW in 2016, so this inspection is the first recorded under the Reservoirs Act.*

There has been some debate over which sections of the Act apply, but it is understood that NRW and DCWW have agreed the following pragmatic approach:

a) This report and Inspection certificate under Section 14 to formally abandon the reservoir.

b) A certificate of efficient execution (CEE) of works as if under Section 8, with an annex describing the reservoir, following the principles of Section 7(6) (Appendix A to this report).

c) A Final certificate as if under Section 8 once any MIOS have been completed.

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The reservoir is abandoned under Section 14 of the Reservoirs Act, rather than discontinued under Section 13, so the works needed are to secure that the reservoir is "incapable of filling accidentally", rather than "incapable of holding". This report is therefore structured as a Section 14 report, with the process shown in Flow Chart A.9 of the Guide to the Reservoir Act (ICE, 2014).

The capacity at Llwyn Du was assumed to be <25,000m^3 prior to inspection. However, a routine pre-inspection asset investigation was undertaken in order to inform the Inspecting Engineer of the asset's history. The search brought to the attention of the department a discrepancy between the registered capacity and actual escapable volume of the service reservoir that has been operationally isolated for over a decade. The Welsh Office register from 1984 lists the reservoir, as does the BRE dams database, but neither list a capacity so it was unclear if it was over 25,000m^3. When an archive investigation was conducted it was uncovered that the operational capacity and registered capacity was significantly lower than the capacity to the overflow and therefore the full escapable volume would have been sufficient to exceed the threshold under the Reservoir Act 1975 definition (escapable volume calculated at 31,819m^3). As a result, it was decided that an immediate Section 14 inspection should be undertaken.

This presented a number of challenges, not least balancing reservoir safety whilst delivering sensible solutions with options that satisfied the expectations of the ARPE and could be financially justified whilst still reducing risk to as low as reasonably practicable.

The argument can be made that the Abandonment of a Service Reservoir is a misleading notion, as conventional Abandonment would entail isolation on its inlet and outlet from its distribution system, while the structure remains intact. This would remove the potential for intentionally or accidentally filling the reservoir, but there still remains the potential to fill naturally via ingress. Nevertheless, Abandonment was pursued - Discontinuance options were considered too environmentally damaging, costly, and difficult to justify against the small likelihood of a catastrophic failure.

The solutions for Abandonment still present significant engineering challenges. Primarily, the remediation and replacement of a 65m washout and overflow pipe at depths of 7m across terrain that has a history of slippage. The settlement of the surrounding land is known to be the cause of the collapse of the original pipework.

There are few available alternatives and this work must be completed by November 2025, so we are expecting plenty of challenges ahead.

WORK ARISING FROM FIRST INSPECTIONS

We have noted a number of themes that have emerged from the 85 MITIOS that have arisen from first inspections to date, especially on service reservoirs. To give a flavour of these, twenty sites have required overflow capacity assessments, a dozen sites have required drawdown capacity assessments, ten sites have had MITIOS relating to drainage surveys, and half a dozen sites have required topographical surveys.

As with all inspections, there is also a degree of subjectivity from individual Inspecting Engineers. We have seen this more starkly on our first inspection programme, with significant differences on what constitutes a MITIOS on an SRV. Across a small handful of sites, we have received MITIOS for vegetation management plans, vegetation clearance and even bramble clearance which, whilst all undoubtedly are best practice, can be argued are not critical to the integrity of the reservoir.

Throughout the programme of first inspections, our level of maturity and understanding has grown and this has given us the confidence to challenge the contents of Inspection Reports. In our experience, it is essential that this is done via constructive conversations with the Inspecting Engineer that can only take place once mutual trust has been established. Building good relationships with the small pool of Inspecting Engineers used on this programme has been key to its success.

Whilst most MITIOS timescales would be perfectly fine in isolation, it is important to consider each measure with a Wales-wide view and a consideration of what other assets are out of service, what other measures are deliverable during the same period, other operational challenges, and the availability of our small supply chain. Giving this context has helped us successfully challenge timescales – there are examples of us doubling and even trebling the amount of time given to deliver Measures in the Interest of Safety.

ON SITE EMERGENCY FLOOD PLANS

As none of the 52 assets that form our first inspection programme had ever previously been inspected under the Act, none of them had On Site Emergency Flood Plans (OSEFPs). Whilst this is a not a legal requirement in Wales in the same way it is in England, it is best practice that we adhere to as a responsible undertaker and is also commented upon by the Supervising Engineers in their Annual Statements.

Writing 52 OSEFPs alongside delivering the programme of first inspections and delivering the MITIOS work presented another significant challenge and placed further pressure on the limited resource we have within our team. To manage this, we successfully negotiated with NRW that we would have a satisfactory (as judged by the Supervising Engineer) OSEFP in place for each site within 12 months of receiving a final risk designation. This allowed us to spread the workload over the five years of the AMP and gave us manageable timescales to work within. Alongside this, we have also had success in moving the requirement for an OSEFP out of the MITIOS category and into Directions in Respect of Records Under Section 11 of the Act.

CASE STUDIES

A number of case studies have been included below to highlight some of the challenges we have encountered in the delivery of our first inspection programme.

Radyr Service Reservoir

Radyr Service Reservoir is located in Radyr, approximately 5.5km northwest of Cardiff. Built around 1970, it is a 4.7m deep service reservoir with an escapable volume of around 19,000m^3. The reservoir is retained by a reinforced concrete perimeter wall, with in situ concrete floor and roof. The first inspection under Section 8 (of compartment no.2) took place in June 2020, with the final compartment expected to be emptied and made available for inspection sometime later in 2020. However, compartment no.2 failed its flood test. Ingress was identified along the northwest joint between the roof and wall. In order to remediate this ingress, a 30m trench was excavated to expose the joint (Figure 1). This allowed the existing material to be removed, and a new bandage applied (Figure 2).

Upon the commencement of the internal inspection, it was already a known concern that compartment no.2 had failed the flood test procedure and that investigative trial holes were being dug. The overriding water quality concerns allowed for an extended investigation phase. Following the guidance from the Inspecting Engineer on likely MITIOS following the internal inspection of compartment no.2, we were able to mobilise the survey team to develop a 3D model of the compartment to retrospectively create construction drawings – we had been unable to locate the original as-built drawings so this needed rectifying. We also used this opportunity to fully map the drainage on site. Water quality concerns dictated that compartment no.2 remained empty for almost 18 months. In the prolonged period between inspections, we were able to progress one potential MITIOS to the point of completion, and have 50% of the internal schematics completed. The subsequent schematics were completed between the completion of the inspection phase and the MITIOS delivery date. Compartment no.1 was inspected in December 2021.

Figure 1. Excavation of the roof / wall joint at Radyr SRV. (Courtesy of DCWW).

Figure 2. Repair of the roof / wall joint at Radyr SRV. (Courtesy of DCWW).

Sluvad No.2 Service Reservoir

Sluvad No.2 Service Reservoir is one of three service reservoirs on site at Sluvad Water Treatment Works, near Pontypool. The three reservoirs were constructed in stages between 1961 and 1992, and although there is no exact known date of construction of Sluvad No.2, it is believed to be around 1964. The reservoir comprises two equally sized compartments of approximately 6800m^3 with a reinforced concrete roof, columns and floor slabs and mass concrete gravity walls.

Prior to the first inspection in January 2022, compartment A failed a flood test along the dividing wall. Water quality concerns determined that the best course of action following the failed flood test and the area of the failure that the adjoining compartment should be isolated from the network and drained to eliminate potential water quality parameter failure.

Investigations determined the membrane installed in the mid 1990s was beyond repair and a membrane reinstallation was required. The excavation of the grass cover and membrane layers (Figure 3) permitted the Inspecting Engineer to undertake a thorough visual inspection of the roof during the subsequent inspection in February 2024, and to see firsthand the ingress repairs prior to the new membrane being installed (Figure 4). With over two years between the inspection of the two compartments, it was agreed that compartment A would be reinspected at the time of the inspection of compartment B in February 2024. W hilst having such an extended period of time between inspections was not ideal, the Inspecting Engineer was kept up to date throughout and the repairs carried out between inspections meant the inspection report contained no MITIOS.

Figure 3. The exposed roof slab at Sluvad No.2. (Courtesy of DCWW).

Figure 4. The new roof membrane being installed at Sluvad No.2. (Courtesy of DCWW).

Tongwynlais No.2 SRV

Tongwynlais No.2 Service Reservoir is one of two reservoirs located on a hill approximately 7.5km northwest of Cardiff, constructed sometime between 1990 and 1993. Tongwynlais No.2 has a capacity of 21,000m^3 and is approximately 6m high. The structure appears to comprise a reinforced concrete base slab and roof with mass concrete outer walls.

Immediately following the High-Risk designation of Tongwynlais No.1 (the inspection of which was completed in February 2021) it was decided the next stage would be to undertake the inspection of Tongwynlais No.2. Compartment 2B was taken out of service and inspected in July 2021. The inspection coincided with temperatures in South Wales reaching 30°C and an unusually prolonged period of dry weather resulting in the declaration of a drought by the Welsh Government. This brought difficulties mobilising sufficient tankers to facilitate the flood test as the tanker fleet was mobilised supplementing the network, and when the flood test was completed, the tank was shown to be suffering with significant ingress. Standard Welsh Water flood test procedure dictates that the roof is visibly saturated, and that a minimum flood depth of 25mm should be achieved over the top of the roof. The depth of water is confirmed by strategically dug trial holes that consider historic ingress repairs and the fall of the roof. Due to the temperatures and limited tankers, it was recognised that sustaining a 25mm flood was not achievable, and it was agreed that the upstands, hatches and the roof joints would be targeted. The targeted flooding showed that a number of upstands were not watertight and significant ingress was recorded (Figure 5). The secondary access hatch required sealing at the joint of the upstand and the roof. The existing bandage was removed, a layer of Natcem 35 was applied between the upstand and the roof and following a curing time a MasterSeal bandage applied along the joint (Figure 6).

Following the repairs to the tank the compartment was to be brought back in to service in November 2021. To date the inspection is still in progress, and due to the period of time that has elapsed since the inspection of compartment 2B, this will be reinspected at the time compartment 2A is made available for inspection. An ongoing capital programme is improving resilience across the network to facilitate the recommencement of the inspection.

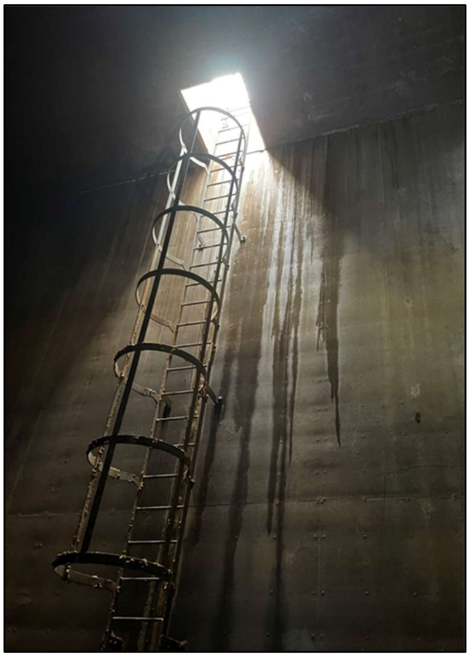

Figure 5. Roof ingress at Tongwynlais No.2. (Courtesy of DCWW).

Figure 6. Repairs around a roof hatch at Tongwynlais No.2. (Courtesy of DCWW).

Stumpy Service Reservoir

Stumpy Service Reservoir is a reinforced concrete reservoir with two compartments situated in the town of Barry in the Vale of Glamorgan. It was constructed in 1955 and has a capacity of 15,552m^3. In total, there are four compartments located within the boundary of the site, two of which are not connected and are regarded as redundant tanks.

At the commencement of our programme of first inspections, there was a known inability to remove Stumpy SRV from service whilst maintaining supply to the 15,000 properties directly fed by the reservoir. Bypassing the SRV increased the peak flow in the inlet main, as well as increasing the peak head loss and peak velocity which presented an unacceptable risk of discolouration in an area that had already experienced supply outages and water quality concerns. To facilitate the emptying of the service reservoir for cleaning and inspection, the inlet main required conditioning to deal with this higher flow, and new pressure relief valves were installed on the main.

The reservoir was subsequently made available for inspection in November 2022. However, the challenges did not end there. We have a MITIOS that is proving difficult to conclude to the satisfaction of the QCE. Initial investigations have proven that the overflow (Figure 7) has insufficient capacity. In addition, it has not been possible to conclusively demonstrate that the overflow discharges to the assumed discharge point (Figure 8). Attempts to prove the discharge location have been inconclusive due to the distance 500m distance through third-party land and the lack of inspection chambers along the assumed route. Conventional next steps would be to empty the reservoir and undertake physical investigations from the point of overflow. However, this is considered as having high operational risk for the continuity of supply because the refilling of the reservoir during the winter of 2022 required supplementary tankers to maintain customer supply. As we approach the statutory due date of the MITIOS - which coincides with the high demand summer months - we find ourselves at an impasse. The alternative option of filling the tank to the point of overflow is considered a potential threat to water quality for the 15,000 properties and industrial customers.

Figure 7. The overflow at Stumpy SRV. (Courtesy of DCWW).

Figure 8. The assumed discharge point at Stumpy SRV. (Courtesy of DCWW).

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The British Dam Society

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

A P COURTNADGE, Jacobs

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

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

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

INTRODUCTION

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

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

Managing Risks for Dams and Reservoirs

DESCRIPTION OF THE RESERVOIR

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

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

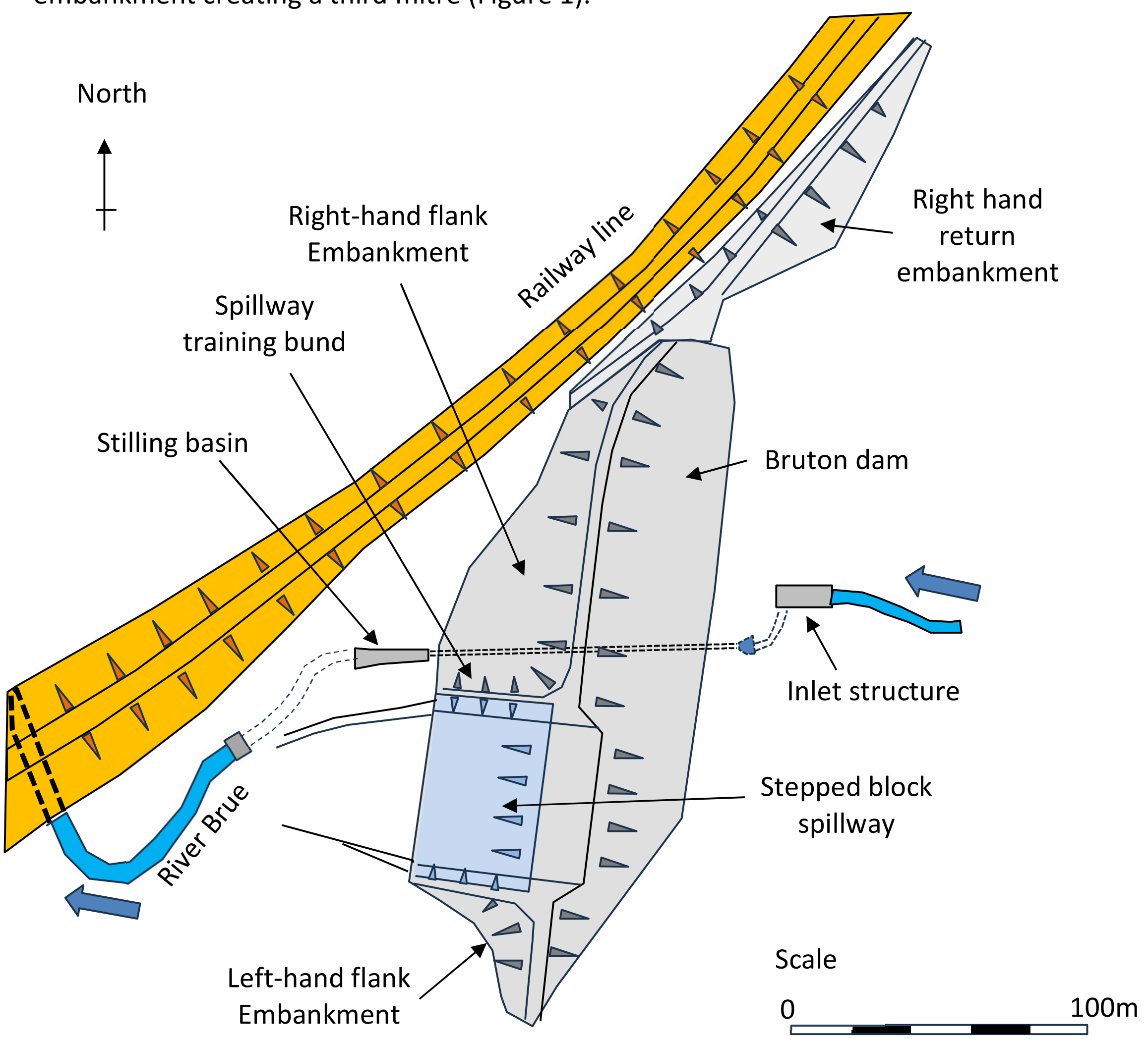

Figure 1. Schematic plan of Bruton Dam

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

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

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

FLOOD ESTIMATES

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

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

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

Table 1. Flood estimates

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

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

ASSESSMENT OF SPILLWAY CAPACITY

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

Weir capacity and freeboard

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

Capacity of spillway chute (wedge blocks)

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

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

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

Velocities on main embankment face

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

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

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

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

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

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

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

Table 2. Peak velocities on grassed downstream face

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

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

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

Notes.

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

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

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

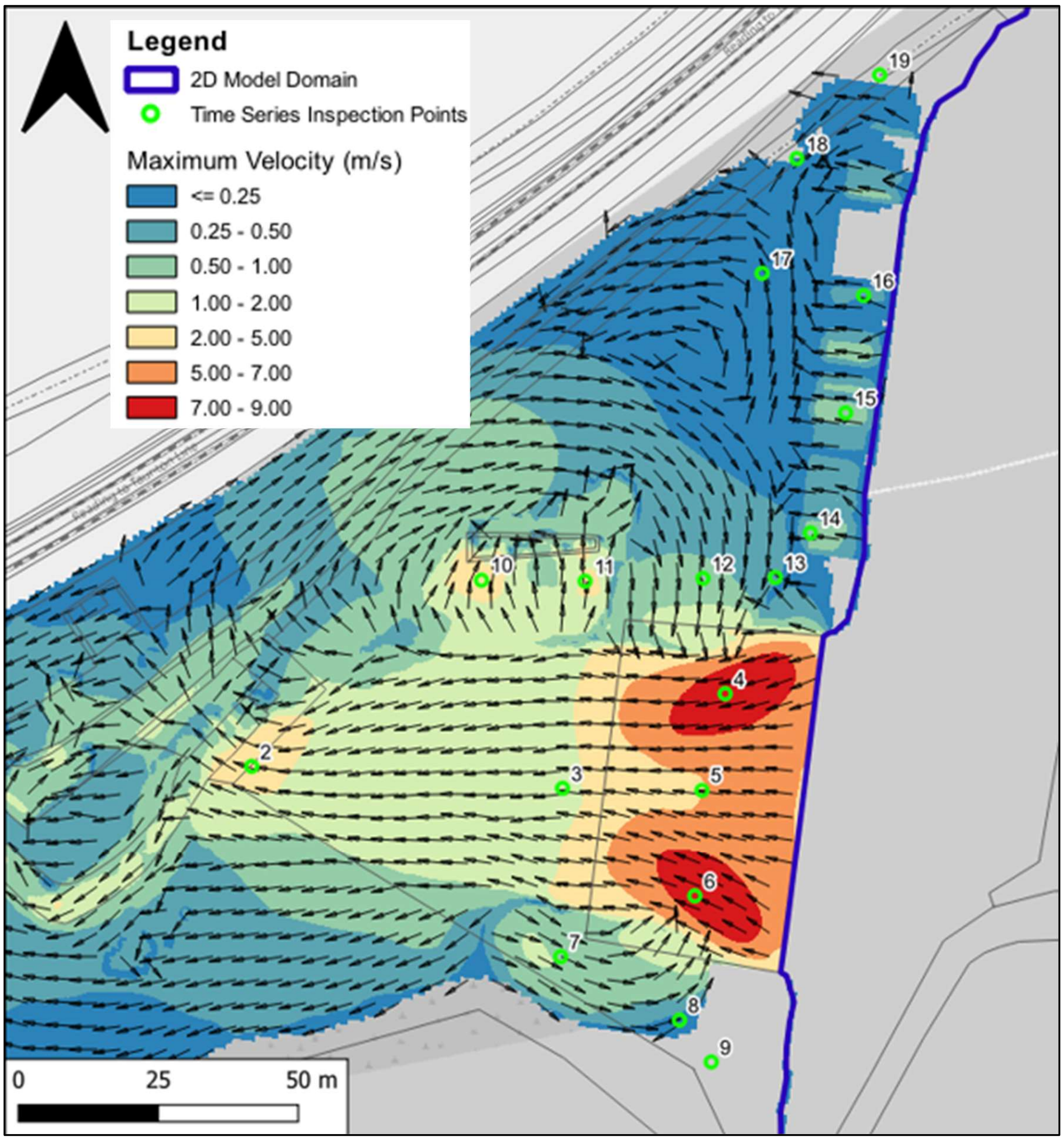

Figure 2. 10,000-year event maximum velocity grid with velocity point inspection locations

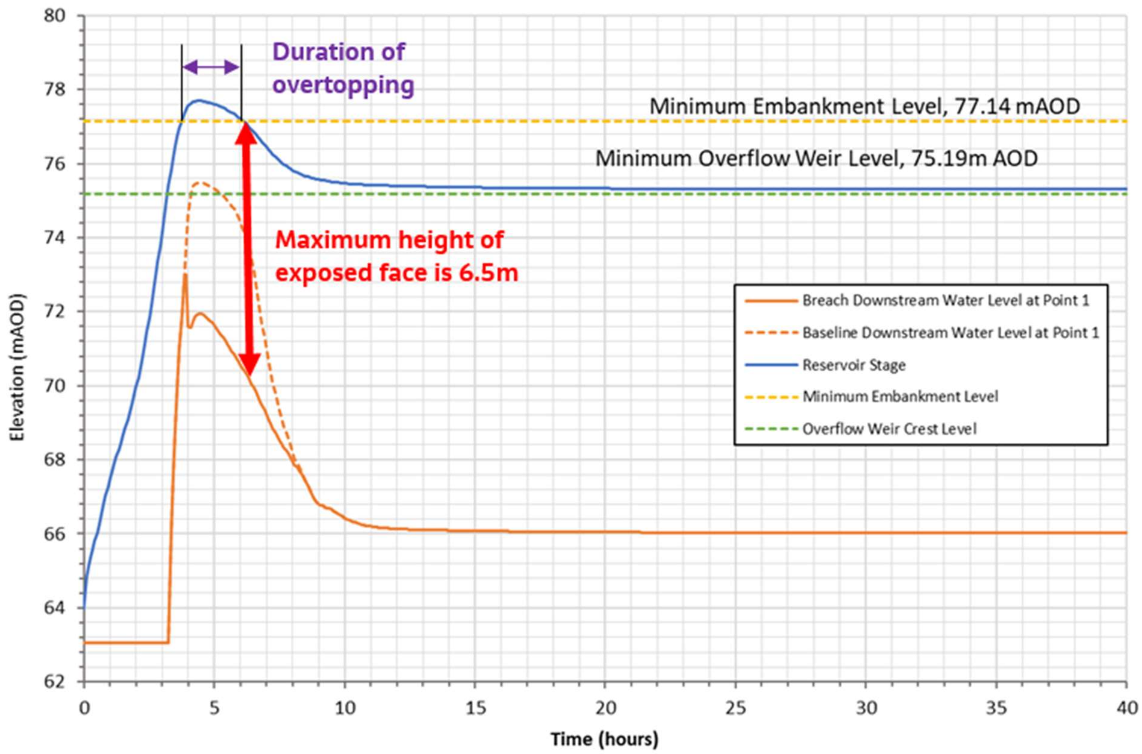

Figure 3. PMF Maximum Water Level with 21m wide railway breach

Summary of spillway adequacy

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

ALARP STUDY

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

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

Potential failure modes and current probability in failure

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

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

Consequence of failure

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

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

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

    - RFM would not have allowed for tailwater in estimating the peak breach discharge, so for all breach scenarios the peak breach flow is likely to be overestimated.

    - The RFM specification (EA, 2020,Section E.4.4) assumes a high erodibility dam but Bruton dam is built from an intermediate plasticity clay so is likely to be medium erodibility (see Table 10 of ICOLD, 2013).

    - The RFM modelling will not have considered the beneficial effect of the railway embankment and dam failing consecutively in terms of smoothing out the peak of the breach flows and allowing time for warning and evacuation. Indeed, the RFM does not allow for any warning or evacuation at all.

    - The fatality rates assumed by the RFM specification are based on a straight line best fit to observed deaths in flash floods and fluvial flooding in UK, with the data points shown on Figure 9.1 of the guide to risk assessment for reservoir safety (EA, 2013). It is noted that the USA use much lower fatality rates particularly where the Depth x Velocity is less than $1m^2/s$ (USBR, 2015), suggesting that the RFM fatality rates may be conservative.

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

Tolerability of current risk

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

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

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

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

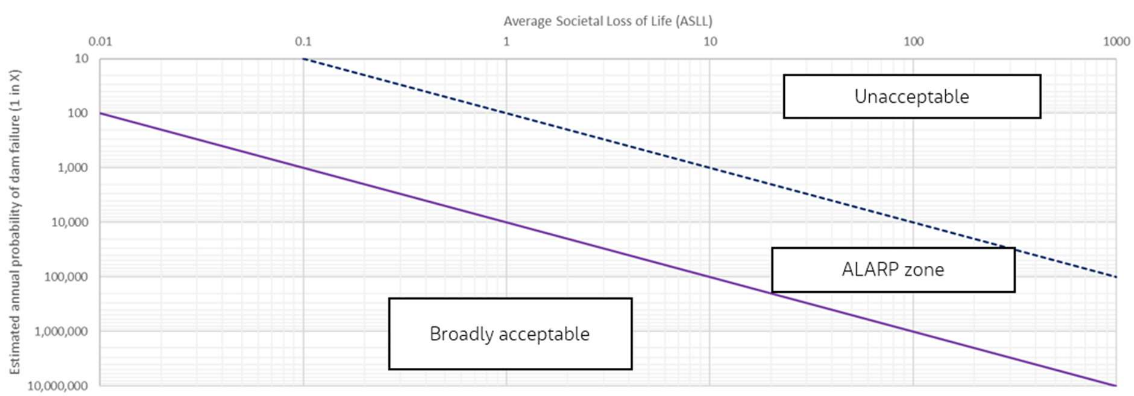

Figure 5. Consequence diagram for ALARP assessment (HSE, 2000 and Figure 9.2 of EA,2013)

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

Assessment of whether upgrade works would be proportionate

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

1. Multiply the reduction in annual probability of failure generated by a particular scheme by the Average Societal Loss of Life (ASLL), which is a statistical measure for the number of lives that could be lost in a catastrophic breach.
2. Discount this over a 100-year appraisal period to give a present value of likely savings in lives, using a factor of 57 (See Table 6 of Brown et al, 2014).
3. Multiply the reduction in annual probability of failure by the potential cost of third-party damages if the dam were to breach.
4. Again, discount this over a 100-year appraisal period to give a present value of the risk savings for third party damage, using a discount factor of 57 as above.
5. The CPF is then calculated as the capital cost of the scheme minus the present value of the risk savings for third party damage, divided by the present value of likely savings in lives.

The CPF is then compared with the value of preventing a statistical fatality (VPF), with the current published VPF by the Department of Transport for road and rail schemes being around £2M.

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

Candidate options

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

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

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

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

Notes,

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

Table 4. Option combinations evaluated for ALARP

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

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

DISCUSISON

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

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

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

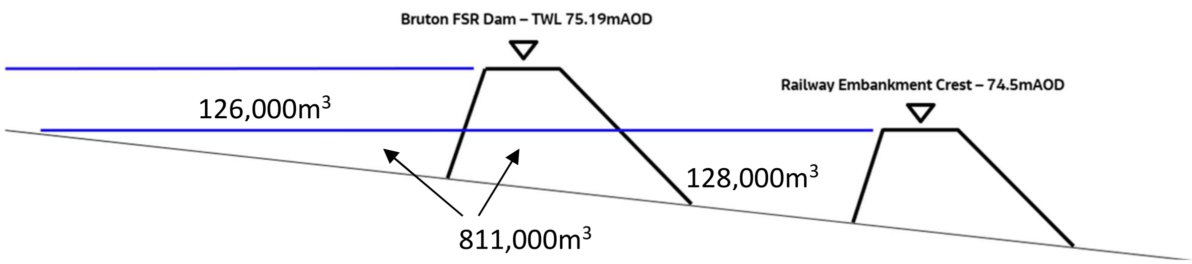

Figure 6. Illustration of escapable volume with and without the dam present

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

Table 5. Definition of incremental damages

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

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

CONCLUSIONS

The following conclusions are made:

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

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The British Dam Society

A Pragmatic Approach for Mitigating Siltation Clearing in Confined Spaces and Culverts in Flood Storage Reservoirs

S YEOH, Jacobs
S GARATTINI, Jacobs

SYNOPSIS Siltation can pose a significant challenge in flood storage reservoirs, particularly within confined spaces such as culverts. As part of the Lincolnshire and Northamptonshire (L&N) Reservoirs Remedial Works programme, Jacobs and the Environment Agency implemented a novel solution to help manage the risk associated with clearing sediment build-up in the control structures and culverts at Rase North and Rase South flood storage reservoirs.

The flood control structures here, comprising typical culverts crossing the main embankment, suffer from significant build-up of siltation, likely due to changes caused by development and climate change. The Environment Agency (as reservoir Undertaker) is facing ever-increasing challenges and costs due to increased frequency of silt clearance, exacerbated by confined space working conditions. Conventional silt traps and sediment excluders are impractical due to their substantial footprint, and are cost prohibitive.

To address these issues, 'in-channel chambers' were designed and installed in the river channel upstream of the control structures to help catch incoming sediment. The design approach, part of a trial initiative, aims to improve maintenance practices and mitigate health and safety risks by minimising the need for confined space entry during silt clearance. The unique construction of these chambers within the river channel and bed helps mitigate adverse impacts on the environment, morphology and hydraulics of the river channel.

This paper presents a practical solution applicable to similar reservoirs in environmentally sensitive areas facing siltation problems, that require regular maintenance and lack space to implement other conventional solutions to intercept and contain sediment inflow.

INTRODUCTION

Background Information

The Lincolnshire and Northamptonshire (L&N) Reservoirs Remedial Works Programme is an initiative aimed at rectifying identified defects at a number of statutory flood storage reservoirs across Lincolnshire and Northamptonshire. The programme encompasses remedial works at two sites along the River Rase, namely Rase North and Rase South reservoirs, situated near the town of Market Rasen in Lincolnshire.

Rase North is an offline reservoir located approximately 600m east of Market Rasen along the River Rase. The reservoir features an in-channel throttle pipe which is designed to divert water within the reservoir when the culvert capacity is exceeded. The dam structure comprises engineered fill embankments with a 4.0m wide crest, featuring a 1 in 5 slope on the downstream face and 1 in 3 slope on the upstream face. The flow control structure of Rase North discharges into the River Rase through a twin culvert located underneath the dam.

Rase South is an online reservoir situated along a tributary of the River Rase approximately 200m to the north-west of the Market Rasen golf club. Its dam is constructed with engineered fill embankments featuring a 4.0m wide crest and 1 in 4 slope on both sides. The flow control structure of Rase South comprises a twin culvert system beneath the dam.

Both reservoirs are designated high risks reservoirs under the Reservoirs Act 1975. The two flood storages combined provide flood protection to approximately 200 properties in the town of Market Rasen, safeguarding them from potential fluvial flooding resulting from a 10% Annual Exceedance Probability (AEP) event.

Figure 1. Location of Rase North and Rase South FSRs in relation to Market Rasen

The Problem

The River Rase presents a challenging scenario due to its sandy catchment with superficial geological composition, consisting of Blown Sand and Alluvium overlying Kimmeridge Clay deposits. This geological setup, compounded by changes resulting from development activities, such as altered agricultural practices and intensified farming, and exacerbated by climate change and population growth have collectively contributed to a substantial accumulation of silt within the river, leading to adverse effects on the local ecology and impacting the operation of the flood storage reservoirs, compromising flood control capabilities and increased maintenance costs.

At Rase North, the throttle pipe regularly experiences silt build-up, hindering its functionality. Access to this pipe is restricted during flood events, accessible only through the dam or spillway. Even under normal conditions, access remains challenging due to the existing bank geometry (steep and narrow), as shown in Figure 2. This maintenance issue was highlighted in the most recent Section 10 report for the flood storage reservoir, prompting recommendations for remedial works to enable safe access for silt clearance from this critical pipe structure.

Similarly, the control structure at Rase South has recurring siltation issues, aggravated by natural erosion of the river banks along the golf club and further upstream. Over the years, efforts to desilt the areas have been undertaken three times (in 2015, 2018, and 2021), removing between 50 and 80 tonnes of silt each time. However, the process of clearing the control structure remains laborious and difficult due to the need for confined space access, posing significant health and safety risks for workers.

In an attempt to increase local flow velocity (to promote silt clearance in the control structure), gabions were installed immediately upstream to restrict cross-sectional areas of flow. However, this intervention did not produce the desired effect and the features were later removed. Despite several other initiatives, such as the Lincolnshire Chalk Stream Project (2021) focusing on erosion control measures in the catchment areas, the Undertaker faces escalating challenges and costs associated with frequent sediment clearance exacerbated by difficulties to access the culvert structure and working in confined space conditions.

Conventional silt traps and sediment excluders are deemed not viable here due to their large footprint, requiring a substantial upstream area of land, making them cost prohibitive. Conventional silt traps would not only demand additional land acquisition but if built within the river channel would require watercourse realignment, thus adversely impacting the local geomorphology and ecology in the river channel.

Figure 2. Access to Rase North throttle pipe

Figure 3. Sediment build up at Rase South control structure

Figure 4. Rase South control structure being desilted (working in confined space)

Figure 5. Undercutting of both banks along River Rase

THE IMPLEMENTED SOLUTION

Design Principles

'In-channel chambers' were designed and constructed upstream of both the throttle pipe at Rase North and the culvert control structure at Rase South to help trap sediment. The placement of these chambers was carefully chosen to reside in open areas, specifically in relatively straight sections of the channel, to minimise adverse effects on river flow and morphology. The chambers are also located in close proximity to existing access routes to facilitate efficient operations for clearing trapped sediment from the chambers.

The underlying principle behind these in-channel chambers was to establish a compartment below the river channel's bed level where sediment could be captured and easily extracted periodically. Key design considerations encompassed:

- Minimising adverse effects on river flow and morphology
- Ensuring safe access and operation for maintenance equipment
- Avoiding confined space restrictions or risks for workers
- Eliminating the need for specialist equipment to remove sediment
- Excluding considerations of sediment inflow types or distributions

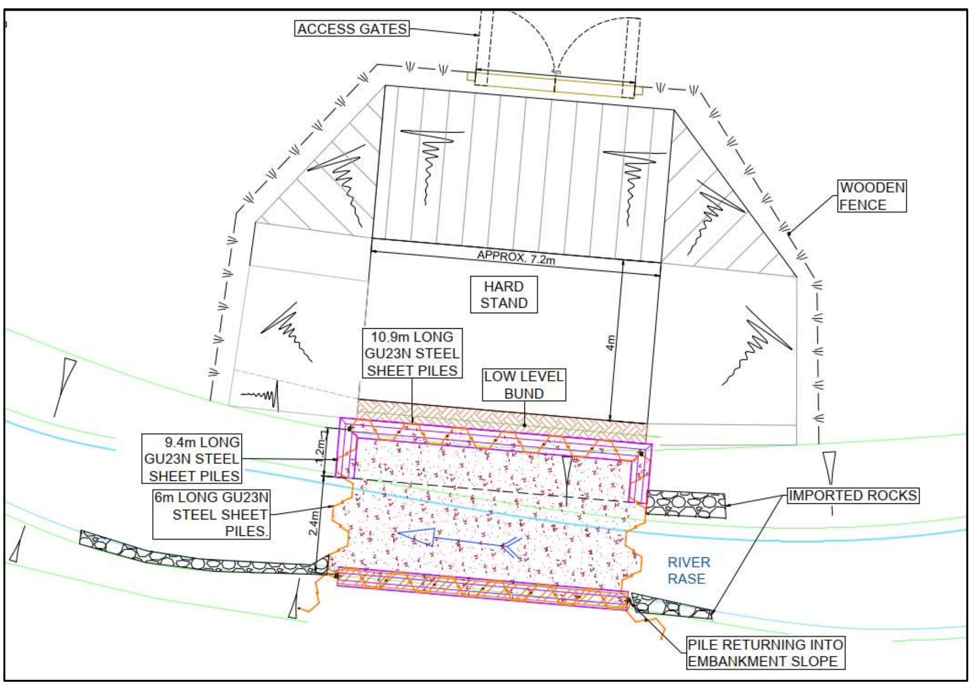

Figure 6. Rase South in-channel chamber plan arrangement.

The chambers are rectangular in shape (approximately 5m x 8m), tailored to fit the river channel's geometry, and constructed using sheet piles of varying lengths with a steel capping beam. Each chamber has the capacity to trap up to 40m^3 of silt. The installation of in-channel chambers was integrated within the local bank reprofiling and toe improvements to mitigate bank erosion. Sheet piles across the river channel are aligned and flush with the existing channel bed level to minimise changes to flow level, thus reducing impacts on ecology and flood storage operation.

In creating these 'in channel' chambers, the chamber's bed was positioned 1300mm below existing river bed level with a 300mm layer of boulders at the base of the chamber to delineate the maximum clearing depth, preventing inadvertent excessive excavation that could compromise the chamber's structural integrity (Figure 7).

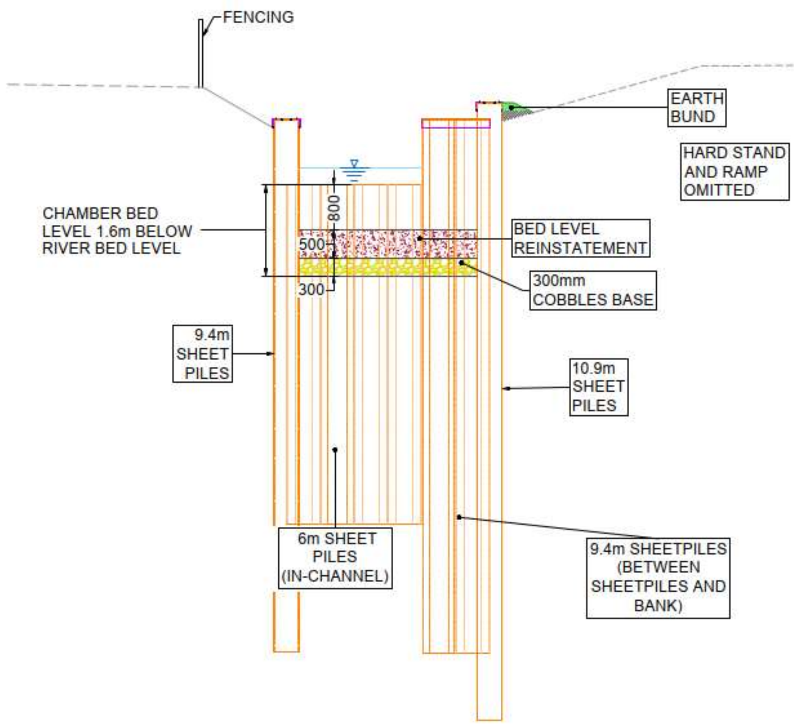

Figure 7. Typical section of in-channel chamber

To minimise material washout at the upstream end of the chamber, rip-rap rocks were positioned along the bank adjacent to the sheet piles. Additionally, return sheet piles at a flared angle were installed into the bank to help minimise turbulence and eddies at the interface between the chamber and the river banks.

A hard-standing area was incorporated into the design to facilitate plant access and safe operation. This design enables efficient silt clearing with plant operations right up to the chamber, minimising excavation reach and eliminating the need for long reach or specialist equipment. The design includes an extensive deep sheet pile structure to ensure the chamber's structural integrity under maximum and critical loading scenarios during emptying. A small bund between the hard-standing area and the sheet piles serves as edge demarcation, managing the risk of falling into the chamber and ensuring workers' safety.

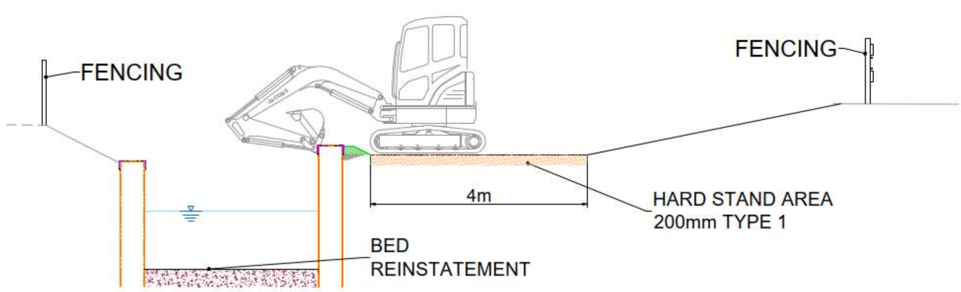

Figure 8. Rase South in-channel chamber typical proposed section.

Environmental considerations

The River Rase is a chalk stream, a rare and valuable habitat often likened to England's rainforests. In addition to adhering to standard good environmental practices, the impact of the in-channel chambers on both local ecology and geomorphology underwent thorough scrutiny during the design phase.

To prevent the creation of a step in the riverbed level, which could impede the passage of coarse fish and eels, the top of the sheet piles within the river channel was aligned and set flush with the existing bed level. This design also ensures that downstream water levels remain unchanged, crucial for preserving fish habitats, especially during the summer months.

Collaborating with Environment Agency (EA) fisheries, biodiversity and geomorphology team, the project incorporated a Natural Flood Management (NFM) solution at Rase South to trial additional mitigation measures against erosion of the river banks and channel (Figure 9). The implemented solution involved locally reprofiling the watercourse to create a low-flow channel and installing a 600mm mattress of compacted brushwood across the riverbed contained by faggots. Positioned upstream of the in-channel chamber, this solution acts as a natural barrier, encouraging the river out of the bank and onto the floodplain, thereby depositing fines in the process. This NFM solution was implemented as a trial with the potential for broader application in the catchment area, and its effectiveness is being assessed.

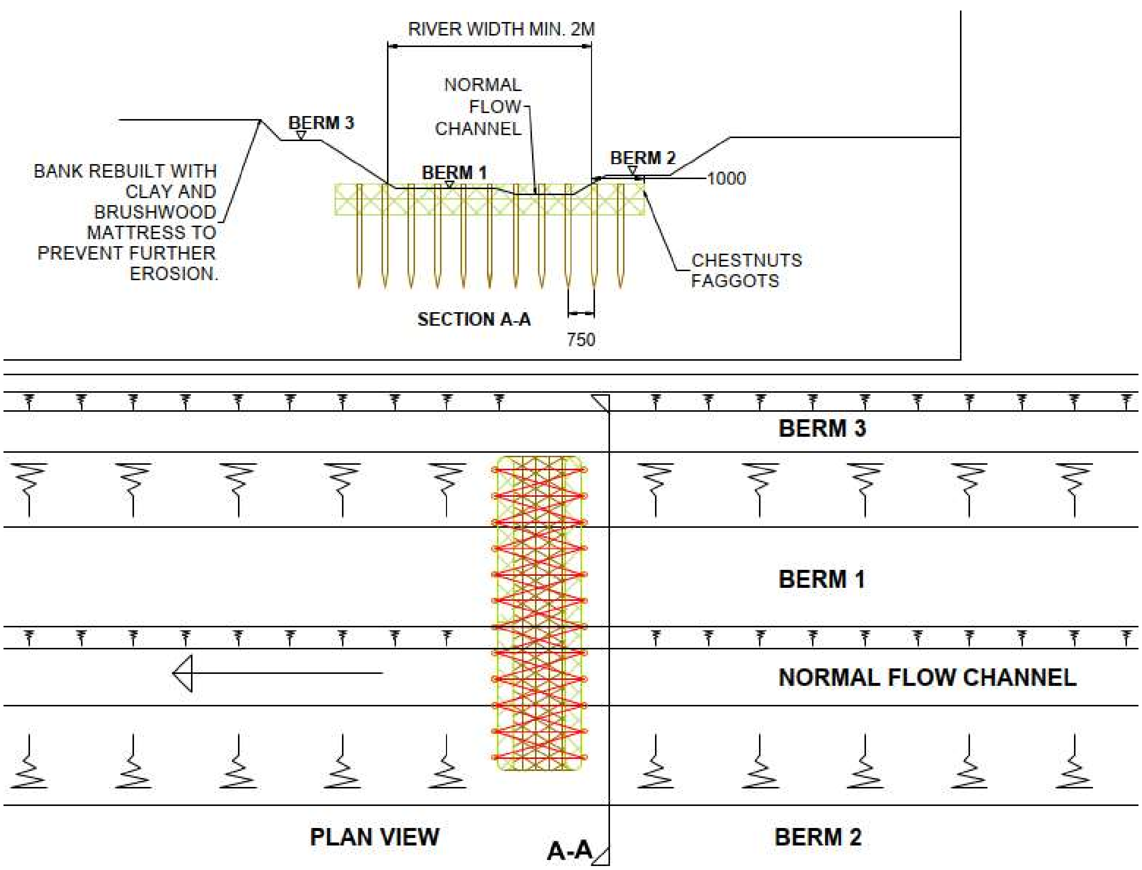

Figure 9. Brushwood mattress as Natural Flood Management mitigation

The innovative design of the in-channel chambers contained within the watercourse contrasts with conventional silt traps and sediment excluders, which often necessitate watercourse realignment or significant alterations in river geometry. Construction of such large structures typically trigger a Water Environment Regulation (WER) assessment, formerly known as the Water Framework Directive (WFD) assessment and may require remedial works. By opting for in-channel chambers, which are relatively simpler and smaller in size, the project ensured minimal changes to the watercourse, thus excluding the need for WER assessments. This approach mitigates localised alterations to watercourse geometry/ geomorphology while effectively managing sedimentation issues. Figures 10a and 10b illustrate the completed works at Rase South.

Figure 10a. Rase South in-channel chamber looking upstream

Figure 10b. Rase South in-channel chamber looking downstream (NFM upstream of the chamber)

OPERATION AND EFFECTIVENESS

Since their construction in 2023, by Jackson Civil Engineering, the in-channel chambers have played a pivotal role in streamlining the maintenance process within both Rase reservoirs. Their operation has allowed for regular clearing, a task now conducted on a three monthly basis. Frequency of clearance is planned by the Undertaker taking into consideration operational needs, ensuring that small amounts of sediment are periodically removed. Approximately 20 tonnes of sediment per site were removed every three months and spread thinly over a large adjacent area to avoid damaging the grass or other vegetation. This approach of periodic clearing not only enhances the efficiency of the maintenance process but also aids the disposal of the material. By spreading the removed sediment in small quantities across the site, the need for removing larger quantities and arranging waste permits/ off-site disposal is significantly reduced.

Managing Risks for Dams and Reservoirs

Feedback from the Undertaker confirms that the in-channel chambers have met their design objectives effectively. The process of silt removal is being executed as envisaged, without the requirement for specialist equipment. A small excavator suffices for clearing the chambers, and the disposal is efficiently managed using a dumper truck. This streamlined approach not only ensures effective sediment removal but also contributes to operational cost savings and resource optimisation. Moreover, the successful reduction of sediment buildup within the confined space culvert has directly addressed the concerns raised by the Inspecting Engineer. The implementation of these chambers has provided an effective and manageable solution to the persistent challenge of siltation at these flood storage reservoirs.

More detailed data collection efforts are ongoing to validate the positive feedback received regarding the impact of these chambers on the maintenance regime at Rase reservoirs. The focus is on gathering detailed information regarding the quantities and types of sediment removed. This data-driven approach aims to optimise the maintenance operation of the chambers further and enhance their long-term effectiveness to help assess their applicability at other similar locations within the catchment. One of the notable advantages of employing in-channel chambers for siltation management is their replicability. This flexibility allows for the installation of additional chambers within the same watercourse, catering to specific site requirements or adjusting clearing frequencies. The inherent design of these chambers ensures minimal environmental impact and facilitates straightforward operation for the removal of trapped sediment.

While it may be premature to quantify the precise cost savings associated with this approach, some of the benefits (when compared to the previous maintenance regime) are already apparent (Table 1). These include mitigating the need for confined space work, saving personnel time, streamlining regulatory compliance, and providing a more efficient means of silt removal to prevent sediment buildup, thereby contributing to the overall resilience and sustainability of the flood storage reservoirs.

Table 1. Estimated comparison for previous and current operations at Rase reservoirs

Previous	Current
Estimated cost for clearing silt (including survey) in the confined space flow control structure: £120,000 (approximately every three years/ site)	Ongoing estimated costs associated with periodic clearing (three monthly): £8000 per year/ site
Estimated cost for construction for one typical conventional silt trap, excluding land purchase is £200, 000 (actualised price) [3]	Construction (sheet pile works only) for one 'in channel chamber' is £120,000 (actualised price)
Typical duration to remove silt (include planning and inspection) from confined space culvert structures: 5 days /site	Duration to clear silt from in channel chamber: 0.5 day/ site (currently carried out once every three months)
Number of operatives involved in undertaking inspection and silt removal from confined space culvert structures: 4 persons	Number of operatives involved in removing silt from in-channel chamber: 2 persons
Estimated person hours for planning, inspection and removing silt from confined space culvert structures: 120 hours/ operation/ site	Estimated person hours for planning and removing silt from in channel chambers: 10 hours/ operation/ site

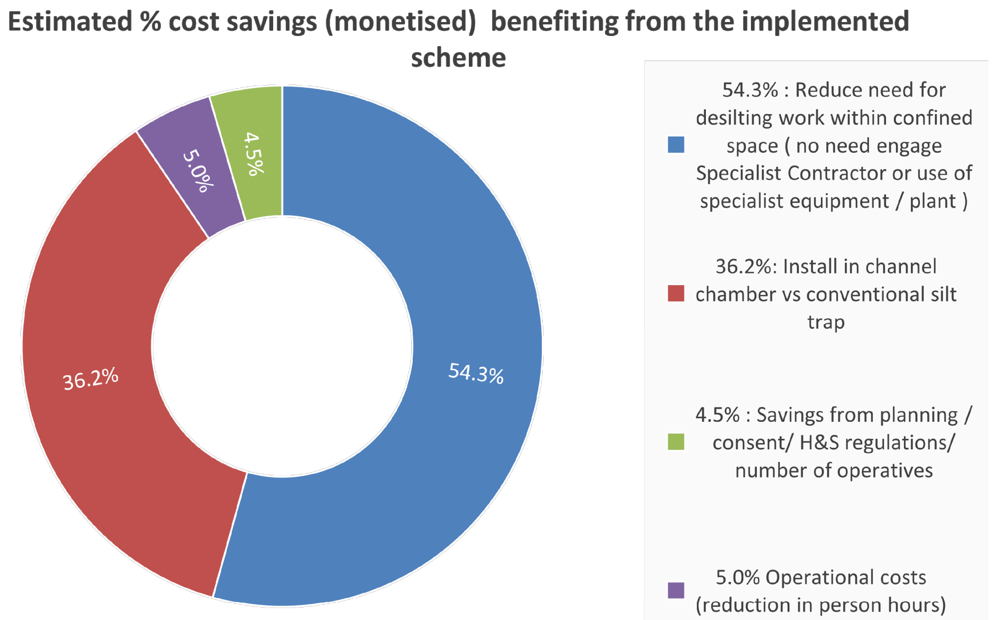

Figure 11. Estimated breakdown of cost savings

CONCLUSION

This paper showcases a pragmatic solution that has been implemented at Rase North and Rase South reservoirs to address the challenges associated with clearing siltation within confined spaces at flow control structures, a common feature in flood storage reservoirs and a prevalent issue in flood storage reservoir management.

The solution comprises the strategic installation of in-channel chambers to facilitate silt removal and the application of Natural Flood Management (NFM) techniques to minimise silt reaching the critical control structures. This initiative not only ensures operational efficiency but also enhances safety by minimising the risks associated with confined space work. This combined approach has significantly reduced the frequency and costs associated with clearing silt in confined spaces. The solution was successfully implemented in April 2023, and its effectiveness and performance are being evaluated, with positive feedback from the Undertaker highlighting tangible cost savings and operational ease.

It is important to note that these solutions at Rase reservoirs are not presented as the definitive answer to siltation challenges in all flood storage reservoirs; however this paper highlights the practical applicability, impact and viability of such interventions. In-channel chambers have shown to be effective and useful especially in environmentally sensitive areas where regular siltation clearing maintenance is imperative but space constraints limit traditional mitigation options.

The solutions discussed here offer a potential blueprint for addressing similar challenges and effective means of preventing excessive sediment buildup in control structures enabling safe silt removal, thereby mitigating the risks associated with confined space operations and help improve the resilience of critical structures against siltation-related issues at flood storage reservoirs. This approach highlights proactive management strategies and innovative solutions to address siltation challenges effectively, championing sustainable sediment management practices.

ACKNOWLEDGEMENT
The authors are grateful to Paul Arnold and the Environment Agency for providing useful information and permission to publish details of the project.

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Design and Construction of an Open Stone Asphalt Spillway for Wychall Flood Storage Reservoir

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SYNOPSIS Wychall Flood Storage Reservoir is a Category A flood storage reservoir on the River Rea in the Kings Norton area of Birmingham. A section 10 inspection in 2020 identified shortcomings in the existing spillway provision and recommended measures in the interests of safety. A subsequent flood study identified that the spillway needed to be lengthened and reinforced to withstand overflowing velocities of up to 7.3m/s. The reservoir is located within a local nature reserve and great importance was placed on maintaining biodiversity and minimising the visual impact of any alterations. Opportunities to reduce the carbon footprint of the project was also a priority. Possible options for reinforcing the spillway were a cast in-situ cellular reinforced concrete system, precast concrete blocks, or open stone asphalt (OSA). Following a review of options, OSA was selected as the preferred solution for reinforcement of the spillway. The paper will describe the design, construction and future maintenance of the spillway. It will also discuss the practicalities and benefits of using OSA instead of more conventional reinforcement systems.

The client was the Environment Agency, the designer was Mott MacDonald, the contractor was Jackson Civil Engineering and Hesselberg Hydro (UK) Ltd were a sub-contractor who installed the OSA.

INTRODUCTION

Wychall Reservoir is a flood storage reservoir on the River Rea in the Kings Norton area of Birmingham. The present dam structure was completed in 1991 and the Final Certificate issued in 1995. The dam is essentially a homogeneous clay embankment. It is built on the site of a dam built between 1804 and 1815 by the Worcester and Birmingham Canal Company. Immediately downstream of the dam, on the left flank, was Wychall Mill (now a ruin) which was fed via a feeder stream on the left side of the reservoir from a sluice gate on the River Rea.

The River Rea runs in a channel along the south side of the reservoir and passes beneath the extreme right flank of the dam in a culvert. This channel carries the dry weather flow in the River Rea. In a flood event the River Rea spills into the reservoir via an inlet weir located about 400m upstream of the dam. There is an elevated pathway along the north side of the Rea between the river and the reservoir. This separates the flow in the River Rea from the reservoir in all but extreme flood events when it is inundated and the River Rea channel forms part of the reservoir. As such the reservoir is strictly an on-line structure, although it functions as an off-line structure on low return period flood events.

The reservoir has a main spillway comprising an octagonal drop structure set within the upstream shoulder, connected to a culvert which passes beneath the embankment, and a 73m long, grass, emergency spillway on the embankment crest. The emergency spillway has a central 3m deep sheet piled cut-off which is topped with a reinforced concrete capping beam set within the dam crest. The main and emergency spillways have crest levels of 143.0mAOD and 143.5mAOD respectively.

There is a berm, carrying a tarmac access road, which runs along the downstream shoulder from the right abutment to the left side of the emergency spillway. The toe of the sheet pile cut-off is just below the levels of the access road.

The reservoir was inspected in April 2020. This identified two issues with spillway provision as follows:

- The emergency spillway had a steep, largely unprotected, downstream face with many trees (Figure 1) in the area between the crest and the access road. It was understood that this has been accepted previously because the line of sheet piles through the centre of the dam meant that the downstream face could be regarded as sacrificial. However, this argument was not considered to be tenable as the sheet piles were too short to be able to support the upstream shoulder on their own.
- Flood modelling suggested that the spillway capacity was inadequate and the main, unprotected, section of the embankment would overtop in the PMF.

Figure 1. Original spillway arrangement

For these reasons the following MIOS were recommended:

- Undertake a flood study to confirm whether the dam can safely pass floods up to and including the PMF,
- Undertake a dam breach analysis to determine the impact of a dam breach caused by overtopping in credible failure scenarios,
- Determine appropriate measures (at outline design level), if required, to enable the dam to safely pass floods up to and including the PMF,
- Modify the dam to safely pass floods up to and including the PMF.

FLOOD STUDIES AND BREACH MODELLING

A flood study was undertaken in April 2021. The peak inflow in the PMF was estimated to be $288m^3/s$ with there being minimal attenuation in the reservoir. The study confirmed that spillway provision was inadequate and that there would be significant overtopping of non-spillway sections of embankment.

A separate dam breach analysis was undertaken to check the appropriate categorisation for the reservoir. This was undertaken because there was a possibility that, in a PMF, the downstream area would be flooded to the extent that the breach outflow would be inconsequential. The peak, wet day dam breach outflow was estimated to be $78m^3/s$. It was found a dam breach in the PMF did still cause a significant increase in the population at risk, so it was accepted that the dam needed to be modified to safely pass the PMF.

It was thereafter determined that to pass the PMF, the length of the spillway needed to be extended by 25m and that the spillway needed to be capable of withstanding velocities of up to 7.3m/s. In addition, the left flank needed to be raised to above PMF level and the right flank made capable of limited overtopping. The layout of the dam in shown in Figure 2.

OPTIONS FOR MODIFYING THE SPILLWAY ARRANGEMENT

In the outline design phase two high level options were proposed to modify the spillway to enable the safe pass of floods up to and including the Probably Maximum Flood (PMF). The options considered were:

1. Extending the spillway by 25m, raising the left flank crest maintaining the 4m width, reprofiling the spillway and right flank and providing erosion protection along the spillway face.
2. Extending the spillway by 25m and installing a new 10m sheet pile wall upstream of the existing sheet pile wall with the assumption that the downstream slope would be sacrificial. Raising the left flank of the crest and maintaining the 4m width.

Option 1 was taken forward to detailed design stage as it offered the advantages of being less intrusive, lower complexity and would ensure the embankment integrity was retained during flood events.

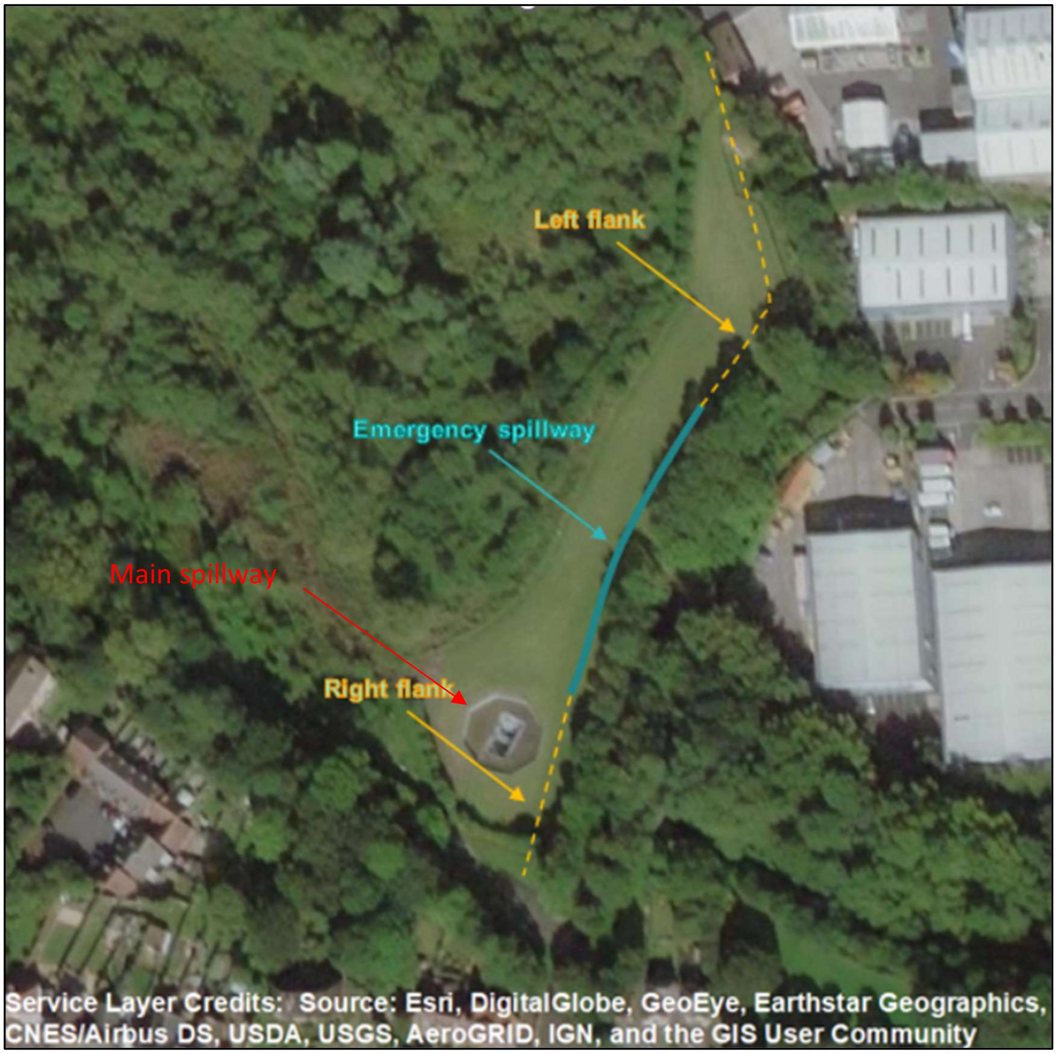

Figure 2. Plan of Wychall Reservoir with indicative spillway layout

ENVIRONMENTAL CONSIDERATIONS

Carbon

To meet their net zero carbon ambitions the client was keen to identify lower carbon revetment options than traditional concrete block systems. They suggested OSA as a potential alternative, as it had been used on a similar project. Further carbon savings were made by using fibre reinforcement in the concrete crest beam.

Visual aesthetics

Another priority for the client was the visual aesthetics of the completed works to appear non-engineered, particularly as the reservoir is in a publicly accessible Local Nature Reserve (LNR). This led to a desire for the chosen revetment to be dressed with soil, and vegetation established. It was accepted this would be sacrificial in overflowing events. Previous examples of OSA being used as spillway protection have taken advantage of covering it with soil, and this gives the extra advantage of reducing the effect of UV degradation.

SELECTION OF SPILLWAY PROTECTION SYSTEM

The spillway had to be designed to withstand a velocity of 7.3m/s. This ruled out the use of geotextile type grass reinforcement. As such, the usual options to consider were a cast in-situ cellular reinforced concrete system or some form of concrete block system. A concrete block system was not considered to be appropriate as the spillway has an irregular shape which would not lend itself to using panels of reinforced blocks. At this point the conventional thinking was challenged by the Client and the designer was requested to investigate the use of Open Stone Asphalt (OSA). This was, in part, driven by recent success that the client had had with OSA on another scheme. Open Stone Asphalt is a homogeneous, permeable mixture of coarse aggregate and asphaltic mastic which comprises bitumen, sand and filler (Bieberstein, 2004).

Open Stone Asphalt (OSA) and the cellular reinforced concrete system were therefore identified as the preferred options that could be used to provide erosion protection. See Table 1 for a comparison of the two materials; this was prepared during the design stage.

DESIGN AND DETAILING OF THE SPILLWAY EROSION PROTECTION

Spillway cross section

The adopted arrangement was a 150mm thick layer of OSA overlying a 100mm drainage layer placed on a non-woven geotextile, as shown in Figure 3. The downstream face was cut back to an angle of 1v:2.5h which was as far as it could be taken whilst preserving space for an access track on the crest. The OSA was extended across the crest to tie into the existing capping beam on the sheet pile cut-off. The OSA was taken down to the level of the tarmac access road. Although this was only about half the total height of the embankment it was acceptable as the downstream toe area would be inundated in an extreme flood.

A toe drain was provided and the OSA was locally thickened to 400mm at the tie-in to the crest beam.

At either end of the spillway there are transitions to higher levels to contain the spillway flow. These were readily formed with OSA.

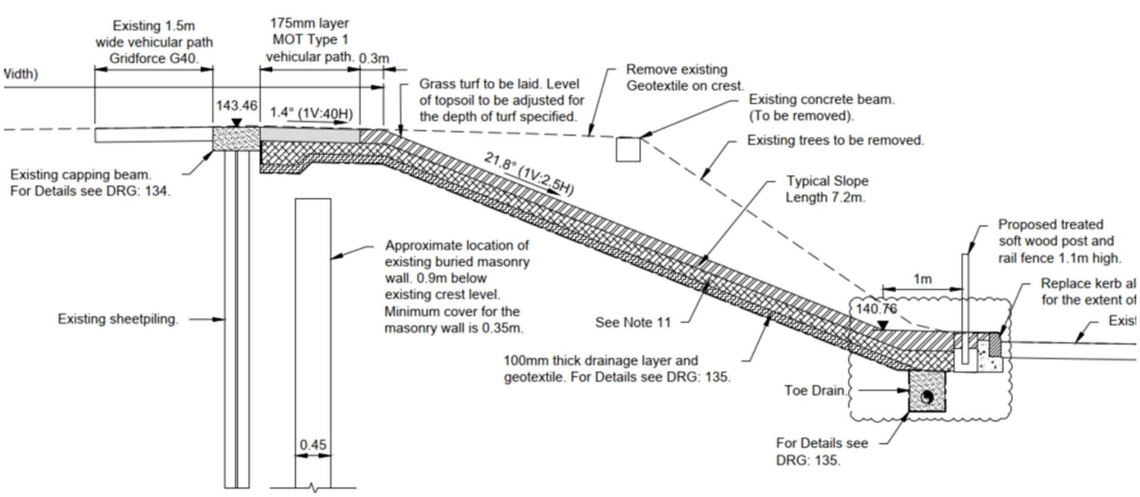

Figure 3. Spillway cross-section

Table 1. Comparison of cellular reinforced concrete system and OSA (specific to Wychall Reservoir where appropriate)

Ref	Cellular reinforced concrete system	Open Stone Asphalt
Velocity limit	Up to 8 m/s	Up to 8.6 m/s (Bakker, 2008; Hesselberg Hydro, undated)
Slope	Can be laid on slopes up to 1:1	Stable on slopes up to 1:2 without anchors
Design life	100-years	50 years
Carbon footprint	The Client's carbon tool calculates cellular reinforced concrete system erosion protection at a rate of 0.5 T CO_2/m^2.	0.015 T CO_2/m^2 (based on 150mm layer of OSA with a layer of geotextile). The supplier is currently trialling warm asphalt which would further reduce the carbon footprint.
Aesthetics	Relies on the grass spreading in the pockets	OSA can be covered with topsoil and either seeded or turfed.
Lead time	6 weeks	4 weeks if prior notice is given.
Installation time	Approximately 5-7 weeks	2 weeks installation for a site like Wychall.
Construction	Toe beams and expansion and contraction joints are required Requires sand layer and geotextile layer.	OSA is placed in a continuous layer, with no construction joints reducing impact from movement. Geotextile separation layer will be required, and a drainage layer typically recommended. Edging details are likely to be required
Health and safety	Formers and mesh to be installed before concrete is poured. Hot works is required to remove the top of the void former.	OSA is a hot installation (typically 130°C - 170°C) so gloves, eye protection and overalls are required. Reduced manual handling although edging is likely be required
Other		Possible risk that the topsoil layer is washed away during large flood events. This will not impact the functionality of the erosion protection.

Material selection/development

OSA comprises 20/32mm Aggregate bound together with a bituminous mastic comprising bitumen, filler and sand. The design of the OSA concentrates on the following:

- The 20/32mm aggregate must have a good natural affinity to bitumen. Limestone is usually used but some gritstones and basalt have proved to be suitable.

- The bituminous mastic must have the correct viscosity. It must be low enough to fully coat the coarse aggregate but high enough to prevent segregation of the material during transport and placing.
- A volumetric check is carried out to ensure the amount of mastic is sufficient to coat the coarse aggregate with a 0.9mm - 1.3mm layer.

Standard OSA can be mixed in virtually all batch-mixing plants and can be transported for a maximum time of approximately 1½ hours from plant to site.

A ‘Warm Mix Additive’ is now common in most plants which enables asphalt mixtures to be used at lower temperatures. This means they can be mixed at lower temperatures to reduce energy consumption, or they can be transported further and still remain usable.

When a plant is producing OSA for the first time it is essential for a contractor with experience in OSA to supervise the process. Minor tweaks to the constituent percentages may be required to produce the optimum mixture.

Design life/maintenance

The design life of OSA is considered to be in the order of 50 years. When OSA was first installed as coastal revetments in 1988 and as a dam revetment in 1991, a design life of 25 years was accepted. These examples have performed well to date and many examples in Europe are older, so the design life has now been increased to 50 years. This has been accepted by the client. In future, a longer period may be considered if the revetments continue to perform well.

There does not appear to have been a difference in performance if the OSA is, or is not, exposed to UV light. It is noted the top film of mastic over the aggregate will oxidise if exposed; however, the mastic between the aggregate (which holds the OSA together) is a larger volume and most tends to be in shade within the layer where UV light does not penetrate.

In the event of settlement tension cracks could develop if settlement/movement is rapid (unlikely). If cracks do appear they can be cleaned and filled with hot-poured bituminous mastic.

Joints with structures are initially sealed with hot-poured mastic. In the event of these joints opening up at all due to differential settlement they can be cleaned and re-filled with hot-poured bituminous mastic.

In the event of surface damage (e.g. impacts from water-borne debris) the area to be repaired can be cleaned, edges prepared and primed, and OSA can be used to re-fill. In the event of small holes (up to about $1m^2$) appearing these may be filled with a mixture of coarse aggregate and bituminous mastic, or resin-bonded aggregate.

CONSTRUCTION

Site works were undertaken between 19th April and 14th November 2022. The placement of the OSA was undertaken during a two-week period, from 6th to 15th Sept 2022.

OSA was placed over an area of $1250m^2$, with 270m of edge details where the OSA is thicker.

Site constraints
The site was constrained, with access via a busy residential main road. Works on the spillway were either conducted from the crest, with a maximum 20kPa surcharge limit, or from the existing 6m wide asphalt track. These factors limited the plant and equipment the contractor could use and delivery timings and frequencies.

Preparation works
Preparation works involved trimming back the embankment slope to the desired profile, placing the geotextile and drainage layer and constructing the toe drain. All preparation works were completed before the OSA was laid.

OSA installation
The process for the installation of the OSA was as follows.

- OSA delivered into a steel delivery skip placed along the toe of the dam.
- Material transferred from delivery skip into a 6T site dumper, taken to the crest of the dam and discharged into a 10T capacity skip on the crest.
- A 13m long-reach excavator placed the material on the slope and profiled it using a travelling shutter to control layer thickness.
- A smaller excavator at the toe completed areas beyond the reach of the crest excavator.
- When the OSA was completed the hot-poured mastic seal was applied to the OSA edges that abutted concrete or steel structures.

Figures 4 and 5 show the procedure in operation.

OSA quality assurance
Various checks were recorded as the works continued, as follows

- Formation/drainage layer and edge details checked for line/level
- Edge of previous day's OSA cleaned & primed
- OSA delivery checks included a visual check (no segregation, well coated aggregate) and a check that temperature was correct (130°C - 170°C)

Finishing works
Once the OSA had been placed the spillway was turfed to enhance its aesthetic appearance. To mitigate for the loss in trees and contribute towards biodiversity net gain targets, the client suggested a wildflower turf, instead of a standard grass mix. This was considered acceptable from a technical perspective as the vegetation and soil above the OSA does not contribute to the erosion protection.

A UK low growing native turf consisting of 20% grass and 80% wildflowers was installed. To provide strength and stability the turf incorporates a fine degradable net in its root zone. Additionally, swathes of bulbs were planted to further increase biodiversity. There remains the option to adjust the diversity of the wildflower mix or revert to a traditional grass mix in the future, if required.

Figure 6 shows the final appearance of the completed spillway.

Figure 4. Placing OSA (dumper being loaded at downstream toe)

Figure 5. Placing OSA (long reach excavator taking OSA from skip on crest)

Figure 6. Completed spillway with turf installed

PRACTICALITIES OF OSA

Practical considerations

- In-situ material which is quick to place and easily follows irregular shapes and contours of dam spillways without awkward joints.
- Easy to place around manholes and other concrete/steel structures on the spillway.
- Thermoplastic properties give good resistance to impact loads whilst also allowing finished revetment to follow settlements expected with new earthworks.
- OSA is stable on slopes up to 1 in 2 without the use of anchors. Where stability of the revetment is a concern due to uplift pressures/high flows then support can be provided at the crest. Geotextile beneath the OSA layer can be extended at the crest and buried beneath concrete sill or in a trench.
- At the toe and sides of the spillway the edges of the OSA are usually thickened to resist any tendency for the layer to 'flap' under high flows. This also gives the edges greater security against scour.
- Day joints are formed by cleaning and priming the existing OSA edge so that the new hot-placed OSA fuses the two materials together, forming a 'monolithic' plate without joints.

- Being a bound material, if damage does occur to the revetment, e.g. vandalism, damage is limited. With concrete blocks, often the removal of one block can lead to rapid progressive failure.
- Vegetation growing through the asphalt will not damage it as the flexible material can withstand deformations over time (avoid trees/large shrubs).
- In the event of internal erosion occurring in the dam voids may develop. Voids beneath asphalt will result in the flexible material following the voids and therefore they can be picked-up during routine dam inspections. Concrete has the ability to span voids for a period of time and so may go un-noticed until catastrophic failure occurs.

Environmental advantages

- Lower carbon content when compared to a concrete-block system capable of withstanding similar loading.
- OSA is compatible with the environment – it is used in drinking water reservoirs, SSSIs, etc. Asphalt is manufactured with bitumen refined from petroleum which is inert and will not harm the environment. Tests investigating the leachability of PAHs, heavy metals and other chemicals from bitumen show that concentrations in the test water was well within the surface water limits for EU countries and were also more than an order of magnitude lower than the current EU limits for potable water.
- OSA can be produced at practically any asphalt mixing plant, so the material procurement will benefit the local economy.
- At the end of its design life, OSA can be re-used as 'Recycled Asphalt Planings'. The OSA is crushed, and the resulting aggregate/bitumen can be used in new road asphalt mixtures. Both OSA that has, or has not, been exposed to UV light can be recycled; testing is conducted on the bitumen element to determine the quantities of new bitumen required for the recycled asphalt.

Limitations

- An asphalt plant within 1½ hours travel time of the site is required.
- Access for road delivery lorries to within approximately 3km of the works location is required.
- Access for an excavator with sufficient reach is required at either the crest or the toe of the revetment area to enable installation.
- Working area must be above water. OSA can be placed underwater but only as a prefabricated mattresses.
- OSA cannot be placed in heavy rain or very strong winds. The OSA may cool too quickly (minimum temperature 110°C) and in heavy rain steam restricts visibility of the excavator operator. Light rain is acceptable and OSA can be held in delivery wagons/sheeted over in the event of showers.
- There is no minimum ambient temperature requirement but if ice is present on the formation soils OSA should not be placed.

MAINTENANCE AND PERFORMANCE

Access improvements

A key requirement for the client is ensuring their reservoirs are safe and cost effective to maintain. Their operations teams were actively involved in the project and suggested operational safety improvements, including an access berm and slackening the spillway's crest transitions to 1V:6H from 1V:3H. Retrofitting these to an existing asset was simplified by using OSA, as it can easily be installed at transitions.

Maintenance regime and equipment

The change in spillway revetment and resulting change in vegetation requirements has allowed a change in maintenance regime and equipment. Previously, grass cutting was conducted six times per year, using ride on equipment operating on the slope. However, the frequency can now be reduced, maximising biodiversity benefits and resulting in a lower operational carbon footprint. The equipment the client intends to use is a tractor mounted flail arm. This is so the wildflowers can be cut without equipment being driven over the surface, as there are concerns this would disturb the soil layer. To allow safe tractor access, the client specified a minimum crest width of 4m. To allow for occasions when the tractor and flail are not available, the slopes (1V:2.5H) and accesses have been designed to also allow the safe use of a remote-controlled mower.

A one-year wildflower maintenance contract was formed. In its first full summer (2023) the soil above the OSA appeared to hold sufficient moisture for the wildflowers to successfully flower. They were cut and arisings raked off in autumn. Due to the spring 2024 growth, it was decided no early cut was required and an autumn cut is likely all that is required.

Establishment/since construction

A plastic grid and MOT type-1 track were reinstated on the spillway crest, to prevent rutting during emergency and operational access. The track was seeded with an amenity grass seed mix; this did not establish well over the first winter, leaving the crest exposed. It is thought to have contributed to shallow longitudinal tension cracks (up to 45mm deep) opening along the downstream shoulder during the first spring/summer season. Also, shrinkage cracks opened along the downstream toe kerb. All cracks were filled with general purpose topsoil and are being monitored. None have reopened and vegetation on the crest track has now established.

The reservoir has impounded water once since construction completed, during Storm Babet on 20th October 2023. However, water levels were well short of the emergency spillway level, so the OSA and sacrificial topsoil have not been overflowed. Surveillance of the reservoir during impoundment, using a reservoir specific checklist, identified no performance concerns.

Inspection, surveillance and repairs

A maintenance plan was agreed with the client to cover queries regarding future OSA inspection and maintenance. It included that removing the soil layer to expose the OSA was not routinely required, unless features such as slips and depressions were identified during regular visual inspections or surveillance. Minor OSA damage could be repaired by competent operatives; however, the manufacturer should be consulted for anything else. The plan also covered surveillance activities during impounding events, such as monitoring the toe drain outfall.

CONCLUSIONS

A new spillway arrangement capable of withstanding a velocity of 7.3m/s was required to safely pass the PMF. Following an evaluation of different types of spillway reinforcement, Open Stone Aggregate (OSA) was selected. The OSA provided an extremely practical means of reinforcing the spillway. It was placed in a relatively short time period with no complications. To provide an aesthetic/environmental finish the OSA was overlain with sacrificial turf.

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The British Dam Society

Draycote Reservoir – Drawdown Enhancement

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SYNOPSIS Permanent siphons are increasingly being fitted to increase the discharge capacity at reservoirs to ensure that the precautionary drawdown provision to mitigate the risk posed by the reservoir satisfies recent guidance. Routine 'wet' testing of reservoir drawdown systems is fundamental to providing confidence that they can be relied upon in emergency situations.

This paper summarises the optioneering, design and construction of the three, 1200mm diameter vacuum-primed siphon system installed at Draycote Reservoir in 2023 to enhance the existing drawdown capacity and testing functionality. The paper will discuss the arrangement and functionality of the drawdown enhancement works, including for routine 'wet' testing; the risk of pollution, including of invasive, non-native species, and flooding during testing and emergency operation; and constraints imposed by the water resources and amenity functions of the reservoir and site.

INTRODUCTION

Severn Trent Water, the Client, has a proactive approach to reservoir safety, with an integral element of this being Portfolio Risk Assessment (PRA). Since 2010, the Client has undertaken three PRAs across their full stock of statutory reservoirs. These PRAs have enhanced the Client's knowledge and understanding of their structures, as each dam is in effect a prototype. Another strand of this proactive approach by the Client is Pre-S10 Inspections, which are commissioned two years ahead of the statutory inspection to provide an early indication of the studies and works likely to be required. In common with all the Client's statutory reservoirs, this proactive approach was applied to Draycote Reservoir.

Draycote Reservoir is a lowland reservoir built in the 1960s and is impounded by six embankment dams (Figure 1). It provides a bulk, raw water supply to an adjoining water treatment works (WTW), principally for distribution to Rugby and its surrounding area. The reservoir is fed by pumped flows from its downstream watercourse, the River Leam, and by pipeline from Stanford Reservoir and Brownsover Pond. Whilst classified as an impounding reservoir, the direct catchment is small relative to the reservoir's size and provides minimal contribution to water storage. Legally binding environmental restrictions on releases from the reservoir via the existing Valve Tower to the Draycote Brook, a minor tributary of the River

Leam, amount to a mere 2Ml/day, reflecting the reservoir's location at the top end of the catchment.

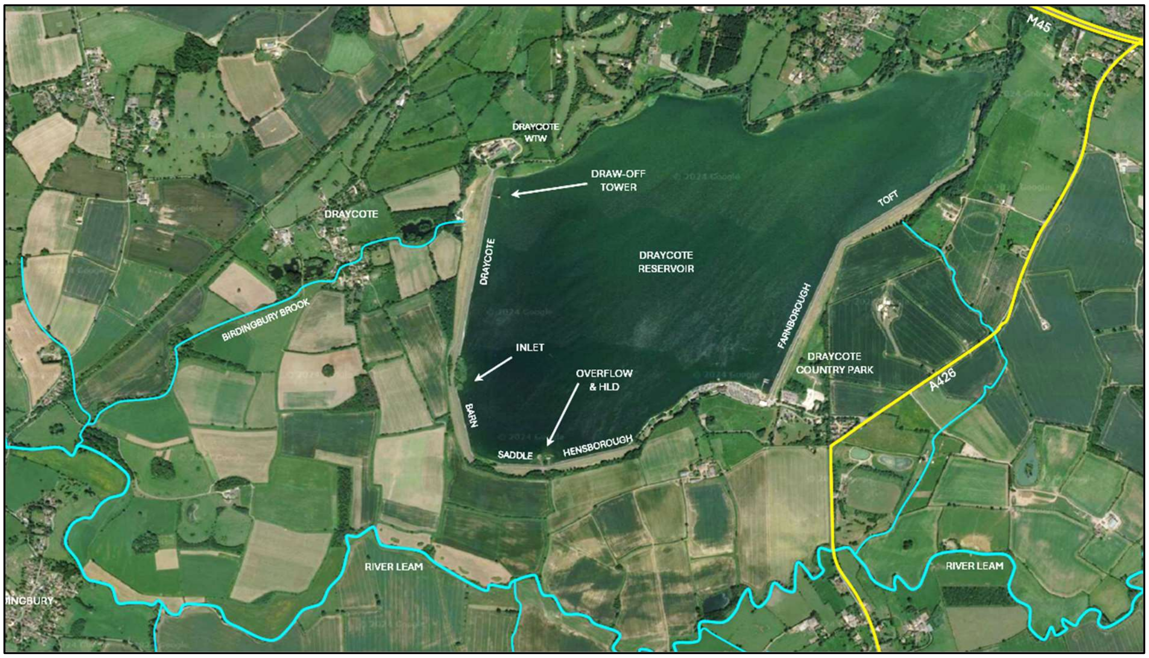

Figure 1. Site Layout of Draycote Reservoir

OPTIONEERING

Overview

Following receipt of the latest S10 Inspection Report in October 2019, the Client promoted a project to address the following measures to be taken in the interest of safety (MIOS):

- Undertake a study to identify options to improve the installed drawdown capacity to "meet latest UK industry guidance", including "a review of the vulnerability of the embankments to internal erosion and any risk mitigation provided by the embankment zoning".
- Upgrade the installed drawdown facilities in line with the agreed preferred solution, subject to a minimum installed drawdown rate of 0.7m per day over the upper 5m of the reservoir depth, equivalent to the top approximately 50% volume.
- Infill the Toft Culvert, including measures to secure the existing pressurised pipe.

For conciseness, only the elements of MIOS 1 and 2 relating to drawdown are discussed further in this paper. It should be noted, however, that MIOS required following a S10 do not necessarily reflect the safety of the reservoir or the lack thereof, as design standards and opinions change with time. Draycote Reservoir is a structure of its era, with the originally installed drawdown capacity considered inadequate against today's standards (EA, 2017).

Stantec, the Optioneering Consultant, was appointed to undertake a study to investigate options to enhance the installed reservoir drawdown capacity. Each option was assessed against its cost, buildability, impact on reservoir safety, operational requirements, and other key project constraints.

Existing Facilities and Drawdown Requirements

The existing drawdown capacity was provided by an 18” diameter scour from the Valve Tower discharging to Draycote Brook (approximate capacity of $1.85m^3/s$), and the High-Level Draw-off system (HLD), comprising a 1600mm diameter culvert from the reservoir to the HLD Discharge Chamber at the end of the spillway channel and then a 42” diameter culvert – the HLD Scour – to an outlet structure at the River Leam (approximate capacity of $4.15m^3/s$). Combined, the existing scour and HLD system provided an average drawdown rate over the top 5m of 0.13m/day.

The ‘basic recommended standard’ for drawdown capacity in accordance the drawdown guidance (EA, 2017) was confirmed by the Optioneering Consultant to be 0.99m/day (equivalent to 5%H/day). The existing drawdown capacity was therefore in significant deficit, requiring an additional capacity of approximately $21m^3/s$ to fully satisfy the ‘basic recommended standard’.

The “minimum 0.7m/day” drawdown rate in the MIOS was originally set on the basis that granular drainage zones in the embankments from previous stability enhancements, including the construction of substantial berms on the upstream and downstream sides of all embankments, may provide filtering properties, and thus some protection against the threat of internal erosion, which had been identified as the principal threat at the reservoir from a previous quantitative risk assessment. Assessment of the drainage zones by the Optioneering Consultant concluded, however, that the drainage zones were too coarse to meet filter guidance, and hence, would not provide suitable mitigation against internal erosion.

The drawdown enhancement proposals and the past performance of the reservoir were reviewed by the Client’s Independent Panel of All Reservoirs Panel Engineers, which concurred with the views of the Inspecting Engineer / Qualified Civil Engineer (QCE) for the works, that a revised minimum average drawdown rate over the top 5m depth of 0.8m/day should be applied. This drawdown capacity would be supplemented by temporary imported pumps to achieve the ‘basic recommended standard’.

Key Project Constraints

As a large, impounding reservoir, Draycote Reservoir presented various constraints:

- Lack of hydraulic capacity within the Draycote Brook and River Leam to receive the emergency drawdown flows. This presents a potential risk of property flooding and damage. To avoid downstream flooding, operational discharge to the Draycote Brook is currently limited to $0.18m^3/s$ and is avoided to the River Leam via the HLD system.
- Requirement to undertake annual ‘wet’ testing of the installed drawdown facilities.
- Water resources: Draycote Reservoir serves an adjacent WTW. The reservoir can only be filled through the winter months by river abstraction, limiting the potential to drawdown the reservoir to facilitate the construction works.
- Water quality: invasive, non-native species (INNS), including zebra mussel and demon shrimp are present within the reservoir.
- The reservoir is the Client’s most popular visitor site, having over 500,000 visitors per year, and hosts a visitor centre, sailing club, and fishery.
- The Client is investigating options to raise the TWL to provide additional water storage.

Drawdown Options

The Optioneering Consultant reviewed the following options:

- Option 1 – provide a washout-tee on the existing draw-off main.
  - **Rejected** due to insufficient increase in drawdown capacity and unacceptable increase in flood risk along the Draycote Brook during testing and emergency operation.
- Option 2 – increase capacity of the existing HLD system.
  - **Rejected** due to insufficient increase in drawdown capacity and unacceptable increase in flood risk along River Leam during testing and emergency operation.
- Option 3 – construct additional HLD system(s).
  - Several arrangements were considered. Sufficient additional drawdown capacity could have been provided, for example, by two, 2m square culverts. **Rejected** due to higher comparative costs; relatively more intrusive works, including into the dam core; and significantly higher initial discharge flows (up to approximately 50m^3/s), resulting in a significant and unacceptable comparative increase in flood and environmental risks, and severely limiting the options for 'wet' testing the system due to the discharge flows.
- Option 4 – construct new siphons.
  - **Preferred and selected**. Discussed within paper.

Preferred Drawdown Solution

A preliminary drawdown capacity assessment by the Optioneering Consultant confirmed that the installation of three or four 1200mm diameter siphons would satisfy the required drawdown rate (Table 1). These arrangements were therefore taken forward for further assessment.

Table 1. Siphons – Preliminary Average Drawdown Rate (m/day over top 5m)

Option	Existing TWL	Future TWL (+0.6m)
3 No. 1200mm siphons	0.82	0.89
4 No. 1200mm siphons	1.04	1.14

Siphon Location

The optioneering study considered each of the six embankments for siting the siphons:

- Draycote Main – **rejected** due to restricted capacity of downstream watercourse (<2m^3/s), and the higher risk of installation through the largest embankment.
- Barn and Saddle – **rejected** due to constricted landownership downstream of the embankment; the proximity of the reservoir intake structure; and the local topography / bathymetry being unsuitable for siphon hydraulics and drawdown.
- Toft and Farnborough – **rejected** due to constricted landownership downstream of the embankment, and because emergency flows would be conveyed by small tributaries to the River Leam, risking flooding of the A426 road (main site access) and other properties.
- Hensborough – **preferred and selected** due to proximity and access to the River Leam; preliminary flood modelling indicated no additional sensitive receptors would be

impacted during emergency operation; its proximity to the existing HLD system; the suitable local topography / bathymetry for siphon hydraulics and drawdown; and the existing available access for construction traffic, plant and laydown.

Drawdown Testing

Routine testing of drawdown facilities is a fundamental part of reservoir safety to ensure that there is full confidence that the system can be operated and relied upon in an emergency event. Where practicable, this is best simulated by full 'wet' test conditions. This is particularly true for large siphon systems as they (i) are more complex than typical gravity outlet systems, typically requiring the use of mechanical and electrical equipment, and (ii) have a complex operation sequence to allow priming and operate / terminate their discharge.

As the River Leam is located approximately 0.5km downstream via third-party land with no connecting watercourse, there is a need to provide a temporary flow storage structure upstream of, and / or a flow conveyance structure to, the river to enable routine 'wet' testing of the drawdown enhancement works without causing flooding and environmental issues. All other major reservoir siphon schemes allow for full "wet" testing to be undertaken.

A temporary flow storage structure – a Detention Pond – was selected as the preferred option to capture the discharged testing flows instead of discharging them to the River Leam.

DESIGN DEVELOPMENT

Outline through Detailed Design

Mott MacDonald, the Designer, was appointed to undertake the outline and detailed designs. The outline design focused on developing the concept design from the Optioneering Consultant, with three key areas identified for more detailed consideration:

- the required drawdown depth to facilitate construction;
- the method of 'wet' testing the siphons, including the form of the Detention Pond; and
- the conveyance of emergency discharge flows to the River Leam.

The concept design set the siphon crest levels such that a 6m-deep excavation through the dam crest was required, necessitating a reservoir drawdown far beyond the reservoir's typical annual cycle. The Client also stated a preference to avoid heightening the embankment crests. One way in which the temporary drawdown depth was reduced was by investigating various configurations of the siphon crest valves and resulting embankment crest levels. The excavation and temporary drawdown depth was decreased by approximately 1.6m by orientating the valves horizontally, rather than vertically. Whilst this widened the Crest Chamber by pushing the siphons further apart, staggering the valves minimised this impact whilst allowing access for operation and maintenance activities (Figure 2).

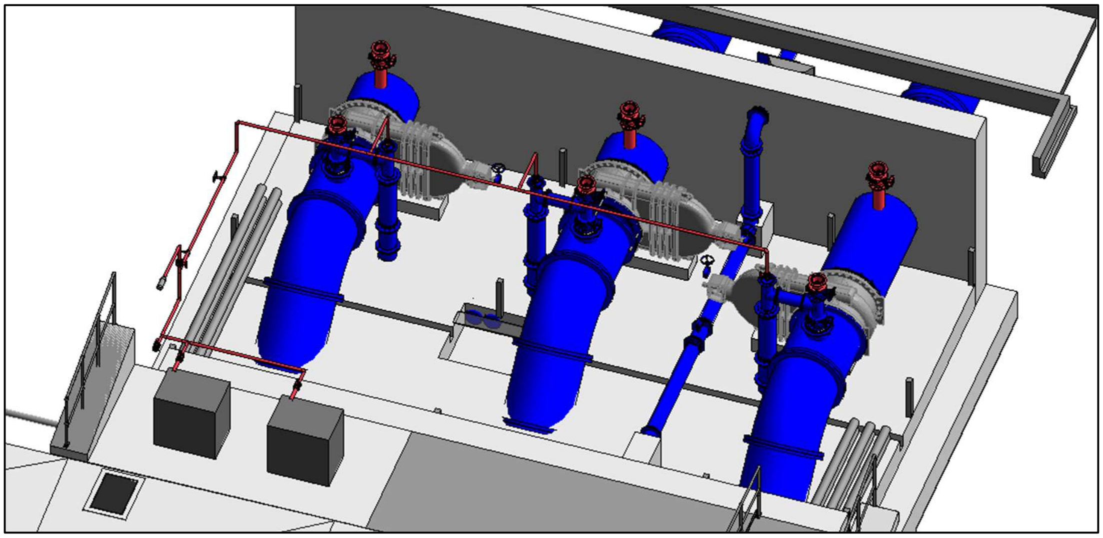

Figure 2. Crest Chamber valves and pipework (walls omitted for clarity)

A sheet pile cut-off was proposed within the embankment clay core to divorce the Crest Chamber and downstream construction works from the reservoir. This did not reduce the depth of the drawdown but did dramatically reduce the duration that the drawdown would be required for, minimising the impact to supply. Once the siphons had been installed through the sheet piles, the reservoir could return to TWL and follow its natural cycle, with the works to be sequenced to align the required drawdown with the lowest level during the natural cycle.

Due to the challenges around discharging directly to the River Leam and the resulting current inability to test the HLD system, the Client requested that the HLD Discharge Chamber at the end of the spillway channel be connected to the planned Detention Pond, via a new, valved conduit, to allow testing of the HLD system and the subsequent return of the testing flows back to the reservoir. The diameter of this HLD Testing pipe was set to maximise the discharge through the HLD system by minimising the throttling of flows through the existing HLD Scour.

The concept design proposed that the siphon downstream legs be laid within concrete culverts to provide double containment; however, double containment was deemed unnecessary by the Designer if the operational methodology was set to leave the siphons empty when not in use and the crest valves closed to avoid passing water from the reservoir. The rationale was that any failure would be immediately noticed during testing or emergency operation and the siphon discharge could then be terminated and the siphon drained. Thus, the residual threat within the siphon downstream legs would be from small-scale leakage only. The siphon bedding fill was therefore wrapped in sealed geomembrane, with an associated drainage outlet provided to allow any leakage to be readily identified.

The Optioneering Consultant proposed that the Outlet Chamber, located at the downstream toe of the embankment, be a vertical stilling basin with submerged discharge valves. However, early in the outline design other options were considered to 'design out' both the approximately 5m deep excavation and the expensive submerged discharge valves. The option selected by the Designer was to install an impact-style stilling basin, designed in accordance with US Bureau of Reclamation design guidance for "Type VI" outlet structures (USBR, 1987). This allowed gate valves to be used instead of submerged discharge valves, as the required energy dissipation would be provided by the outlet structure, and significantly reduced the excavation depth required, as the outlet structure is installed close to existing ground level on its downstream side.

Whilst the concept design did not address the risk of water freezing within the siphon upstream legs, it was identified during the outline design as part of a Hazards and Operability (HazOp) review with the Client. As the siphon priming method necessitates permanent compressors, the HazOp considered two options to utilise the compressed air to mitigate the risk: (1) agitation of the water surface within the siphons to disrupt ice formation; and (2) dewatering the pipes by pressurising the pipes to drive the water out. The Client, however, deemed the risk to be sufficiently low that such measures where not taken forward to construction. The pipes are buried to a set depth, however, to facilitate integration with the rip-rap protecting the embankment upstream face and to minimise the public safety risk of becoming trapped between the siphons where accessible.

The Client raised concerns over the potential for fouling of the siphon pipes from the growth of zebra mussels. Based on industry experience, however, this risk appeared to be low because the water within the siphon upstream legs will be relatively static, decreasing the likelihood of dissolved oxygen and food movement into the pipes. There remained a concern, however, that there would still be diffusion of oxygen and food into the initial leg of each siphon, which could facilitate zebra mussel growth. As a precaution, therefore, the first approximately 6m length of each siphon upstream leg was lined with a vinyl-ester resin to decrease the roughness of the pipe barrel to minimise the potential for zebra mussels to attach to the pipe and grow.

Priming

To enable operation of the siphons, they must first be filled with water, i.e. be fully primed. Three methods were considered:

- Suction priming – Suction pump connected to the crown of the siphon to draw water into pipe from the reservoir.
- Water priming – Pipe infilled via water pump or other piped conduit (pressure or gravity) connected to the crown of the siphon. (When the reservoir water level is above the crown of the siphon, the siphon may be considered 'self-priming' if it fully infills with water without intervention.)
- Vacuum priming – Compressed air is driven through a venturi air ejector at the crown of the siphon which creates a negative differential pressure (i.e., suction) across the venturi and thus the siphon. This draws out any air within the siphon which is then replaced by water drawn from the reservoir.

A vacuum priming arrangement was selected as the preferred method. Vacuum priming is increasingly being installed on siphon schemes as it minimises the scale of plant to be brought to site during testing and emergency events (e.g., high-capacity suction pumps, which are not always readily available). Vacuum priming of siphons has not previously been provided to a system as large as the three 1200mm diameter siphons provided at Draycote Reservoir. A venturi air ejector on each siphon is driven individually by a common compressor unit to prime each siphon sequentially. A target time of two hours to have all drawdown facilities fully operational, once on site and instructed to do so, was set by the QCE.

Managing Risks for Dams and Reservoirs

Due to the level of the crown of the siphons within the Crest Chamber and the typical natural reservoir cycle, the siphons are unlikely to ever be self-priming, despite this being theoretically possible at TWL. Whilst the siphons could be left in a primed state once initially primed this, as stated above, was avoided to negate the requirement for double containment of the siphon downstream legs. To minimise reliance on imported plant (e.g. high-capacity suction pumps or compressor units), the Client's preference was for the priming arrangement to be fixed. A fixed system also ensures that the Client's Operations staff will be familiar with the operation methodology for the system in the event of an emergency.

A cross-connection from the HLD testing pipe to the downstream leg of each of the siphons enables the siphon downstream legs to be infilled up to the reservoir level at the time of operation via the HLD system. This reduces the volume of air to be removed from the siphon via the vacuum priming system and thus the time to prime the siphons.

Each siphon reaching prime is demonstrated to the operator by (i) the change in discharge via the venturi exhaust from a 'spray' / 'mist' to a flow of water to the common sump drain, and (ii) the head within the siphon, shown by the comparative readings on the pressure meter located immediately upstream of the outlet gate valve and the reservoir water level element and observed via the control panel within the Crest Chamber.

On either side of the siphon crest valves, a vent is provided to enable each leg of the siphons to be balanced to atmospheric pressure when not in operation. This prevents the build-up of gases from the breakdown of organics in the water and allows the water level in the upstream legs to balance with the reservoir to avoid the pipes floating, negating the requirement for significant quantities of ballast. These vents, along with the outlet and crest gate valves, allow multiple options for terminating the siphon discharge in case of valve failure. The options, in order of preference being: close outlet valve; close crest valve; then open all vent valves to break the siphon prime – if both the crest and outlet valves cannot be closed, breaking prime will only fully terminate the siphon flows when the reservoir is below approximately TWL-1.5m.

Detention Pond

A Detention Pond is proposed as the preferred method of allowing full simulated 'wet' testing to be freely undertaken by the Client. The Detention Pond captures the testing flows and allows the discharge to be returned to the reservoir via a return pumping station and rising main, thus avoiding: (i) the loss of water to be used for public supply; (ii) any increase in flood risk to or along the receiving watercourse; and (iii) environmental licensing / discharge consent restrictions due to water quality, (e.g., discharge of untreated water contaminated with INNS).

Flood modelling by the Designer confirmed that there are no new sensitive receptors (e.g. private property or public infrastructure) impacted for emergency discharge flows coincident with peak flows along the River Leam over a range of flood events, but that the impact to some existing sensitive receptors already affected by river flooding may be exacerbated.

Design Summary

The solution developed during the outline and detailed design can be summarised as follows:

- Install a triple 1200mm diameter vacuum-primed siphon system over and through Hensborough Embankment, discharging to a Detention Pond, with all valves and instrumentation operated and monitored via a control panel in the Crest Chamber which also links back to the Client's existing systems in the local WTW.

- Upgrade the existing HLD system to enable it to be 'wet' tested and enhance its capacity, with the new HLD Testing pipe facilitating a cross connection to each of the siphons, optimising the time required to prime the siphons.
- Construct a Detention Pond, with associated return pumping station and rising main, to enable full 'wet' testing of the siphons and HLD system whilst avoiding the release of raw reservoir water, overland or as otherwise conveyed, to the River Leam.
- The enhanced drawdown system will empty the upper 5m reservoir depth in approximately five days, with the discharge varying between approximately $30m^3/s$ and $13m^3/s$. The resultant average drawdown rate satisfies the required minimum of 0.8m/day.
- The works caused negligible impact to the Client's water resource requirements for public water supply during construction, with the works able to be completed whilst the reservoir followed its natural cycle, i.e., no significant artificial reservoir drawdown was required to lower the reservoir below its natural levels.
- Operation of the existing HLD system control valves was previously via a 'wax' unit powered by a portable generator; therefore, this project provided an excellent opportunity to provide electrical actuation to these valves to increase the reliability of their operation. The electrical actuation is powered via the permanent connection to mains electricity supply to be provided to the Crest Chamber.

CONSTRUCTION AND OPERATION

Following the design, the construction phase was award to JN Bentley, the Contractor, on a build-only contract. A general arrangement plan for the scheme is shown on Figure 3.

Badger Sett Move and Site Set-up

An ecological study completed in 2020 identified a large and active badger sett at the right-hand abutment of Hensborough Embankment, immediately adjacent to the spillway channel and HLD Discharge Chamber – indicative area shown in Figure 3. The location of the badger sett clashed with the working area for the HLD Testing pipe and precluded access down the right-hand mitre of the embankment, restricting construction access opportunities.

To undertake the construction works, the badger sett had to be moved, which presented a significant programme risk. The alternative was to re-design that aspect of the works and leave the badgers in place. Whilst practical options were identified, the risks to the wider construction scheme and to the embankment itself were such that it was decided to re-locate the badger sett. This was completed in late 2021 following licencing from Natural England.

One of the key attractions for the more than 500,000 annual visitors to Draycote Reservoir is the approximately 8km complete circular walk around the reservoir. One of the original, key project drivers was to maintain this circular route. The Designer proposed for this to be maintained via an augmented footpath via third-party land during construction, which would also facilitate additional space for construction traffic and laydown; however, the land was not secured, so the circular route was severed for the duration of construction.

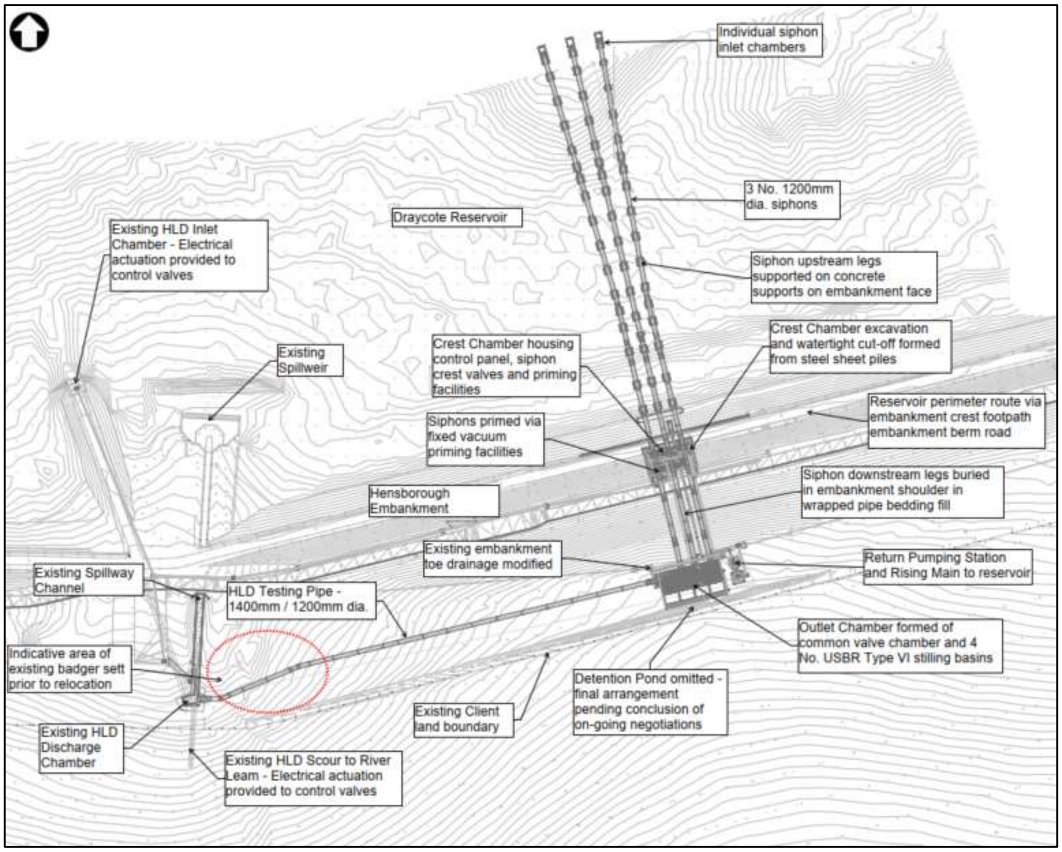

Figure 3. General Arrangement of the Drawdown Enhancements

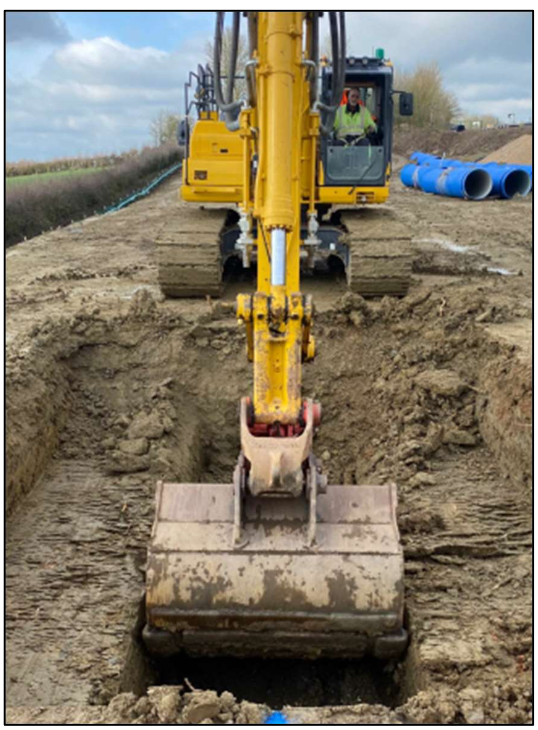

Figure 4. Installation of HLD Testing pipe

Figure 5. Installation of sheet piles using silent press (downstream)

Siphon Construction

Construction of the siphons commenced in early 2023 with the installation of the steel sheet pile cut-off into the embankment clay core to allow the upstream and downstream works to progress independently. Sheet pile wing walls were also installed in both upstream and downstream directions to facilitate construction of the Crest Chamber and minimise the excavation extent required. The sheet pile cut-off within the clay core extended to a depth of approximately 13.5m and the piles, in conjunction with a temporary stiff frame of props and walers, allowed for an excavation to 5m below crest level – see Figures 4-12 for construction photos.

Figure 6. Installation of sheet piles using silent press (upstream)

Figure 7. Installation of siphon upstream legs

Diving operations to install the upstream siphon pipework and individual inlet cages began in earnest in 2023. It soon became apparent, however, that there was significantly more silt than anticipated from the previous bathymetric information. The design allowed for a depth of silt along the line of the siphons based on the previous information, but a detailed dive survey undertaken immediately prior to pipe laying confirmed that there was an additional depth of silt of up to~700mm and a discrepancy with the local bathymetry.

Combined, this meant that the siphon upstream leg would be greater than 2m above the embankment face at points. By this time, however, the pipework had already been procured; therefore, there was minimal scope to amend the alignment of the pipework. The Designer worked within the limits of the procured pipework to re-profile the upstream siphon legs to follow the embankment face as closely as possible and re-designed the upstream pipe supports to minimise their maximum height and ensure their stability. These two changes successfully ensured that the design remained valid, construction was able to continue without delay, and that there was minimal resultant impact to the pipework procurement.

Figure 8. Backfilling siphon pipes downstream of Crest Chamber

Figure 9. Blinding of Crest Chamber

Figure 10. Crest valves showing horizontal and staggered orientation

Figure 12. Construction of Outlet Chamber with outlet valves

Commissioning

At the time of commissioning, the Detention Pond had not been constructed. Whilst no water could be discharged from the siphons, each was fully primed as part of the final commissioning exercise. The accepted commissioning methodology set by the QCE consisted of priming each siphon without use of the cross connection from the HLD – the worst-case condition – in less than two hours and holding the siphons at prime for a minimum time of 30 minutes.

Priming of all three siphons was successfully demonstrated, with each primed from empty in approximately 35 minutes – the typical time to prime each siphon using the cross connection from the HLD is estimated to be approximately 15 minutes. The siphons were shown, via the installed instrumentation, to hold their prime for far longer than the 30-minute target set by the QCE.

The project fully achieved its objective to satisfy all MIOS by enhancing the reservoir drawdown capacity to provide an average drawdown rate over the top 5m depth of at least 0.8m/day. The Section 10(6) certification was issued by the QCE ahead of the MIOS deadline.

Flood Plans

The Client has Flood Plans in place for each of their statutory reservoirs and conducts a test of their emergency (on-site) plans at a selected site each year. The most recent exercise at Draycote Reservoir was in 2015 and accrued several "lessons learnt".

The previous emergency drawdown at Draycote Reservoir was principally by temporary pumps established along each embankment. The 2015 exercise provided an appreciation of the logistics, establishment, and servicing (e.g., fuel, personnel, etc.) for the pumping installations required during an emergency event. The production of the inundation mapping, which showed impacts extending into several counties towards the west of the reservoir and beyond the M5, informed and captivated the attention of Local Resilience Forum responders.

The Client's Flood Plans, and the exercise undertaken at Draycote Reservoir in particular, provide confidence that the Client can enact the emergency (on-site) plan, including the operation of the significant capacity of temporary pumps when and where required.

REFERENCES

EA (2017). *Guide to drawdown capacity for reservoir safety and emergency planning, Report SC13001.* Flood and Coastal Erosion Risk Management Research and Development Programme, Environment Agency, Bristol, UK.

USBR (1987). *Design of Small Dams.* Bureau of Reclamation, US Department of the Interior, Washington DC, USA

The British Dam Society

Multiple Types of Spillway Installation/Refurbishment in Wales (Ten years of experience)

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SYNOPSIS Over the last decade Natural Resources Wales (NRW) has undertaken design and construction of a number of new spillways (and drawdown facilities) as well as refurbishment of numerous existing structures. This has included works at several new and existing flood storage reservoirs, but also a large number of historic reservoirs brought within the Reservoirs Act 1975 by the changes in registration capacity implemented within Wales from 2016.

With a portfolio of newly registered reservoirs, a full programme of investigation works, studies and evaluations was undertaken to determine the risks associated with the different dam structures and subsequent mitigation works required. The range of spillways has included conventional concrete spillways, Armorloc, Armorflex, Dycel, Grasscrete, Reno mattress/gabion, overtopping crest design and a labyrinth weir.

This paper will discuss the design and construction of these different spillway types and their relative merits for the specific locations; design factors affecting the choice of spillways; and issues and difficulties encountered (and overcome) during construction. It also considers the lessons learnt during the process, subsequent operational performance and a commentary on the appropriateness of selecting and implementing various spillway types for a range of sites.

INTRODUCTION & HISTORY

On the 1st April 2016 in Wales, the Minister for Natural Resources approved amendments to the Reservoirs Act 1975 (HMG, 1975) and its regulations, enacted from the recommendations made by Sir Michael Pitt following extensive flooding in 2007, updating Schedule 4 of the Flood and Water Management Act 2010 (HMG, 2010). This brought the inclusion of reservoirs >10,000m^3 capacity into the Act, from the previous capacity of >25,000m^3. The steps taken by Welsh Government to amend the regulations are a reaffirmation that reservoirs hold a public safety risk which justifies its own primary legislation.

From its inception in 2013, with one eye on the impending amendments, NRW had identified 74 potential reservoirs in its ownership or management. Following the assessment of these bodies of water, 45 sites were confirmed as reservoir with a capacity greater than 10,000m³, and in fact 19 sites had a capacity over 25,000m³ and had to be registered immediately.

As part of this assessment and planning work for the 2016 capacity changes, reservoir inspections at the sites highlighted that many of the historic reservoir structures were in a dilapidated state. Many were historic mining reservoirs and had been devoid of any maintenance since their abandonment in the early 20thC (Shaw et al, 2021). Their existing spillway structures were either badly eroded with insufficient capacity, or entirely failed, with water flowing through unprotected breaches. Immediate intervention, under QCE guidance, was therefore often necessary to safeguard the reservoir and prevent further damage to the spillway structure.

The most effective and efficient method of temporarily protecting the existing spillways from further erosion damage was often utilising a combination of heavy duty plastic sheeting and sandbags or concrete filled bags. The spillway channel (or breached locations) would be cleared of any obstructions or sharp objects, with the plastic sheeting then laid within the channel and sides. Rows of sandbags or concrete filled bags would then be placed on the sides of the spillway channel, to weigh down and secure the plastic sheets as well providing further erosion protection to the sides of the spillways (Figures 1 and 2). Often sandbags were also employed on the crest of these dams to afford the required freeboard.

These temporary arrangements would be frequently checked by NRW's Reservoir Keepers, as well as the Supervising Engineers during their 6-monthly visits, with any damage or deterioration immediately reported to NRW's in-house Operations Teams, whereby prompt remedial works were undertaken. Many of these temporary arrangements were in place for several years whilst the permanent MIOS works were being planned and designed. They generally proved very effective in safeguarding the reservoir and prolonging the life of the existing spillways until new, robust and permanent spillways were provided.

Figure 1. Temporary repairs at Tynymynydd.

Figure 2. Temporary repairs at Pandora.

Following a risk-based approach NRW has implemented a program of work over the past 11 years to undertake essential safety works generally under Measures in the Interests of safety (MIOS) to address these issues.

TYPES OF RESERVOIRS

The NRW reservoir stock is varied in terms of purpose (Morris et al, 2018) and includes:

- Flood storage – Legacy Environment Agency Wales Reservoirs
- Conservation, habitat creation & water level management – Legacy Countryside Council for Wales

- General amenity – Legacy Forestry Commission Wales*
- Historical and heritage structures – Legacy Forestry Commission Wales*
- Water supply – Legacy Forestry Commission Wales*

* Reservoirs on the FCW estate are mostly structures left over from the mining industry (mainly lead) which operated between 1830 - 1905 with the exception of some sites which reprocessed tailings up to 1960. It is also worth noting that these sites, with little intervention over the last century, have become important habitat and are generally designated.

MIOS DEADLINES

The primary driver on NRW reservoirs requiring work over the last ten years has been MIOS requirements. The large number of reservoirs registered at one time (2014/15/16), increasing the portfolio from 11 to 41, led to the requirement for numerous Section 10 inspections (or Section 8 inspections if constructed after 1930 with no final certificate issued) within 12 months of the Final Risk Designation.

Predominately, MIOS from this initial round of inspections included the completion of topographic and bathymetric surveys, vegetation clearance, flood studies and inundation mapping. These studies subsequently established the correct category of the reservoirs and established the spillway capacity requirements, which were generally inadequate for the reservoirs not previously registered – indicating their original designs do not meet modern standards.

Table 1 provides a summary of NRW reservoirs. Each site has differences in terms of existing features present (such as spillways and outlet structures), the condition of embankments and the environmental/ location factors specific to the sites. It should be noted that these all are impounding with the exception of Pen y Gwaith, which is spring fed.

Table 1. Key details and spillway types of Natural Resources Wales' statutory reservoirs

Dam	Cat[a]	Purpose	Type	Date	H[b] (m)	Capacity (m^3)	Existing Spillway	New/Refurb Spillway
Afon Wydden	A	FSR	HD	1995	5	29,000	Reno Mat.	Refurb
Bwlch Nant yr Arian	A	Rec.	HD	1995	3	28,530	Armco Pipe	RC inc. Drawoff
Cowbridge	A	FSR	HD	2006	4.4	989,000	Sleepers/ Armorloc	Refurb
Cyfty	B	Mg/WS	MY	19C	6	13,600	Masonry	Concrete + rip rap
Goddionduon	C	WS	HD	1900	1.5	60,000	Masonry	Labyrinth inc. Drawoff
Llaeron	NA	Mg	Pen.	19C	20	450,000	Breached	N/A
Llyn Lleywelyn	B	Folly	MY	1850	4	14,200	Concrete	RC Multistage
Llyn yr Wyth Eidion	NA	Habitat	HD	1994	1.2	36,000	Reno Mat.	Refurb / Fishpass

Dam	Cat[a]	Purpose	Type	Date	H[b] (m)	Capacity (m^3)	Existing Spillway	New/Refurb Spillway
Llyn Fuches Las	B	Mg/FSR	HD	19C	3-4	11,110	Masonry	TBC
Llyn y Parc	A	Mg	CG	19C	3-4	49,445	Breach	Concrete
Lower Hendre Ddu	NA	Mg	RF	19C	5	39,000	Masonry Culvert	Gabion Basket
New Pool	A	Rec	HD	19C	14	44,500	Breached	TBC
Pandora	B	Mg	MY	19C	3	10,000	Breached	RC
Pen y Gwaith	B	Mg/WS	MY	19C	3	12,500	Rock	RC
Pont y Cerbyd	A	FSR	HD	1990	1.7	30,500	Armorloc	Armorflex
Pontarddulais[c]	A	FSR	HD	2014	9.3	170,000	N/A	Grasscrete
Prince Llewelyn	NA	Mg	MY	19C	6	4,500	Masonry	Masonry
Pysgodlyn	B	WS/Rec	CG	1870	1.7	17,630	Concrete	Refurb / Armorloc
Ratcoed	NA	Mg	HD	19C	8	90,000	Breached	N/A
Rhiw Bach Quarry	NA	Mg	RF	1930	3	26,000	Breached	N/A
Tynymynydd	B	Mg/WS	HD	19C	1.5	46,000	Breached	Concrete / Dycel
Llyn Tegid	A	FSR	HD	17C/ 20C	3-4	21.8Mm^3	Concrete Weir	Overflow / Grass

[a]Category (ICE, 2015); [b]Height (of dam); [c]Still under Construction Engineer.

Purpose: FSR – flood storage Reservoir, Rec. – Recreational, Mg – Mining, WS – Water Supply.

Type: CG – concrete/masonry gravity, CB – concrete buttress, ME – modern embankment, PE– Pennine embankment, HD –homogeneous dam, RF – rockfill dam, SR – service reservoir, MY – Masonry with peat core.

FUNDING

Funding was a major issue for the projects. NRW funding is limited and has to be prioritised accordingly. The number of reservoir sites and the significant MIOS requirements coming from the S10 inspections led to more than 150 MIOS deadlines within a three-year period. Some of these related to studies, but increasingly these then led to substantial works, such as the requirement for new or upsized spillways, drawdown facilities or freeboard generation.

The schemes that have been delivered have cost £25m+, ranging from £150k (simple, formal discontinuance) up to £7m for large overtopping sites (Llyn Tegid) (Figure 8), although those with new spillways and associated works such as new berms, have typically cost £0.5m-£1.5m. Timelapse videos of some of the construction can be found here:

- https://naturalresources.wales/about-us/what-we-do/our-projects/reservoir-safety-projects/gwydir-reservoirs/?lang=en

- https://naturalresources.wales/about-us/what-we-do/our-projects/reservoir-safety-projects/llyn-tegid-gwynedd/?lang=en

DESIGN

In terms of the design works undertaken at the sites there have been three primary focuses:

1. clarification of the dam category via inundation modelling,
2. the provision of a spillway capable of meeting the design flood conditions, as set out in *Floods and Reservoirs 4th Edition* (ICE, 2015), relative to the dam category,
3. ensuring suitable drawdown arrangements are in place to meet the Guide to drawdown capacity for reservoir safety and emergency planning, (EA, 2017).

These have formed the crux of safety works but NRW has also undertaken other significant improvement works at some of the sites, including improved public and operational access, H&S improvements, leakage reduction, improved drainage, dam raising, stability berms, gravity shoulders and wave protection.

Hydraulics

The reservoirs within the NRW portfolio range from Category A to Category D (ICE, 2015). Those which have had work completed within the last 10 years are typically Category A (primarily flood storage reservoirs) or Category B. Typically smaller historic mining dams have a Category B designation due to the limited numbers of properties present downstream and the smaller capacities of the reservoirs. These typically required new or formalised spillways, such as at Pen y Gwaith and Pandora (both Category B). The spillway selection was driven by a number of factors:

- Calculation of the flows and associated velocities, for which CIRIA Report 116 (CIRIA, 1987) was used (Figure 3), and freeboard requirements, for which Floods and Reservoir Safety 4th edition (ICE, 2015) was used.
- Sensitivity analysis was also used to determine whether it was appropriate to reduce dam raising (conventional weir vs labyrinth weir).
- Space available for the weir(s)
- Ground conditions and other mitigating factors such as environmental aspects.

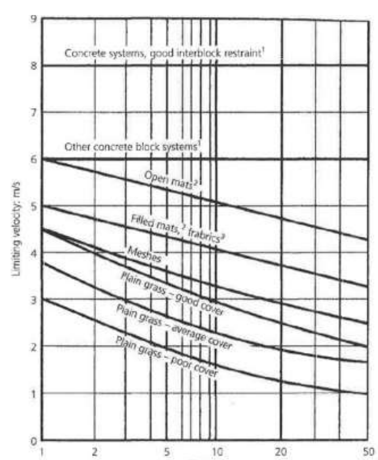

Figure 3. Recommended limited values for erosion resistance. Chart from CIRIA (1987).

Managing Risks for Dams and Reservoirs

The spillway types selected can be broken down into the following basic types:

- Small reinforced concrete broad crested weirs – used at Pen y Gwaith and Pandora.
- Overtopping – used flood storage reservoirs such at Pont y Cerbyd, where the majority of the embankment acts as the spillway.
- Labyrinth weirs (reinforced concrete) – Goddionduon water supply reservoir (Figure 4).
- Multistage – Llyn Llewelyn (Figure 5), and locations where auxiliary weirs have been designed to cater for lower flows, make best use of space, or for fisheries purposes.

Figure 4. Llyn Goddionduon RC Labyrinth weir incorporating drawoff.

Figure 5. Llyn Llewelyn RC multistage weir, incorporating drawoff.

Examples: *Pen y Gwaith (Figure 6) and Pandora (Figure 7) reservoirs, both Category B. Design requirement for the spillways of 1 in 1000yr, safety check flood of 1 in 10,000yr. Similar events with 2.5hr duration, 122mm rainfall for Pen y Gwaith and 3.5hr, 116mm for Pandora. Peak flows vary with Pen y Gwaith 1.3m³/s for the safety check flood but Pandora 5.6m³/s. Both have spillways designed as broad crested weirs as using* $Q=Cd \sqrt{g}\, b\, H^{1.5}$*, with the coefficient of discharge,* $Cd = 0.544$.

Figures 6 & 7. Pen y Gwaith & Pandora RC broad crested weirs.

Figure 8. Llyn Tegid Overtopping embankment using reinforced grass.

Geotechnical Factors

Wherever possible, with the exception of flood storage, spillways have been located on the mitres of the dam, and therefore usually within competent rock. This has a number of distinct advantages, as the rock can be used to form part of the spillway, and less material is required for the spillway structure, reducing the amount of concrete required, associated transport, costs and knock on reduced carbon benefits.

Example: *Cyfty (Category B) – the spillway was located on the right-hand mitre in rock, with a concrete beam forming the spillweir and additional rip rap in channel downstream*.

Space

The available space for any new spillway or drawoff works has been a key factor in terms of the design. With the flood storage reservoirs, it is typically the whole length of the dam that has acts as the spillway for design events. This reduces the depth and velocity over the spillway and hence allows a lower specification selection of erosion resistance. These can provide a better aesthetic, looking more like a natural bank due to the grass cover. Where space is at a premium, or the embankment itself is short in length, the driving factor may be to generate sufficient freeboard. In this instance labyrinth weirs can be considered, generating a lower water level compared to a broad crested dam, due to the additional weir length generated. These can also be seen as quite attractive structures, more interesting than a standard weir.

Example: *Llyn Goddionduon (Category C) where the dam was very short in length due to the fact it was a raised natural lake and achieving a suitable freeboard to meet the 1 in 1000 yr design event was the primary driver. The labyrinth weir at the site (Figure 4) allows this to be achieved, with a total weir length of 7m, but an overall spillway width of just 4.88m.*

Heritage and Aesthetics

Wherever possible, the selection of spillways has tried to take account of the surrounding environment and associated aesthetics. In addition, many sites are historic and have heritage value.

Example: *At Prince Llewelyn masonry was used (Figure 12), in keeping with the historic structure.*

Environmental SSSI impacts, approvals and licences

All required licencing and approvals was undertaken, with enhancements made wherever possible. As most sites were designated, it was imperative not to cause any unnecessary disturbance during the works.

Example: *Prince Llewelyn – non-native fish were rescued and relocated in a pond on Anglesey.*

SMNR/Wellbeing Wales

We had to comply with The Well-being of Future Generations Act (Wales) 2015 (HMG, 2015) on all schemes. Therefore, consideration is given to all users, for example by incorporating improved access.

Drawdown – incorporation of facilities

One interesting aspect on the historical mining sites was that they typically lacked any form of usable outlet. It was clear that historically they had had outlets to leats or downstream streams but that these had ceased to operate long ago, although in some cases indications were still visible (e.g. timber posts sticking up out of the water). Therefore new drawdown facilities had to be provided. Factors that affected the design of the drawdowns were:

- Location – the point at which the drawoff would be most effective, ideally the lowest point in the reservoir.

- Whether a new spillway was also required and its location – could the drawoff be incorporated to save space and construction costs.
- Access for operational staff to allow them to operate the penstock.
- Upstream control was preferred.

At several sites which incorporated drawdown facilities into the spillway structure, sustainably sourced oak footbridges, supplied by a local company based in Llanrwst, North Wales, were installed. These provide safe access to operate the drawdown facility, as well as enhancing the aesthetics of the structures and ensuring continuation of the dam crest footpaths.

Spillway Materials

The following materials have been used on spillways across the NRW portfolio:

- Reinforced concrete – insitu construction
- Grasscrete – insitu construction (Figure 9)
- Armorloc – precast units brought to site and assembled.
- Armorflex / Dycell – precast units brought to site and assembled (Figures 10 and 13)
- Reno mattress/gabion – assembled on site (Figure 11).
- Overtopping crest design – using some form of reinforced grass.

SPILLWAYS INSTALLED

Figure 9. Pontarddulais, Grasscrete.

Figure 10. Pont y Cerbyd, Armorflex.

Figure 11. Afon Wydden, Reno Mattress

Figure 12. Prince Llewelyn, Masonry.

Dam	Cat	New Weir Construction	Drawoff incorporated	Cut offs	Design	Details	Construction Issues
Bwlch Nant y Arian	A	RC Concrete and rip rap downstream	A low level 400w x 600h penstock, incorporated into main spillway design via low level channel.	Base slab cast onto concrete blinding/rock, small cutoff/toe upstream (300mm). Sloped RC concrete side wall (300mm wide).	Peak inflow 5.5m3/s (winter PMF), peak outflow 1.98m3/s. Velocity 2.1m/s to 5.4m/s along the spillway channel. Q=Cd √g b H1.5, Cd= 1.7.	2.5m long weir, 1.87m high,14.3m wide channel. 10.5m long rock mattresses at the end of the spillway channel providing protection from hydraulic jump. RC 300mm thick, sloped side walls.	Poor access, retarder used in concrete, coffer dam installed, good rock. Existing spillway/outlet (Armco pipe) remained flowing during the works before it was removed and infilled accordingly.
Goddionduon	C	RC Concrete and rip rap downstream	A low level 450w x 350h penstock, incorporated into main spillway design.	500mm deep 300mm wide RC cut off in base slab. Sloped RC concrete side wall on sides to spillway.	Peak inflow 4.89m3/s (1 in 10000), peak outflow 3.72m3/s.	Total length of Labyrinth weir 7m long, overall spillway width 4.88m, channel 8m. Rip rap protection extending 8m downstream and 500mm thick.	Steep and narrow access within forest, original contractor going into administration – 6 month delay.
Llyn Llewelyn	B	Concrete - multistage weir	A low level 400w x 400h penstock, incorporated into main spillway design.	Sloped cut off wall on sides to spillway. 300mm RC cut off in base concrete into mass concrete plug under spillway (2m).	14.49m3/s (10,000yr safety check), velocity 1.8m/s, Q=Cd L H1.5, Cd= 1.7. 1D Flood Modeller Pro up to 1 in 1000.	7.8m long weirs (high level 4m and low level 3.8m) and 11.7m channel (with rip rap on both upstream and Downstream).	Extreme weather – heavy rainfall events (difficultly drawing down the reservoir and controlling flows) followed by high temperatures (concrete pours/surface).
Llyn Tegid	A	Overflow Weir/ Reinforced Grass	Separate - Existing river gates	N/A	Embankment overtops in design events discharge 0.26m3/s / m up to 0.6m3/s /m. Velocity 2.2-3.4m/s (10,000yr), 3-3.9m/s (PMF).	Rip rap on upstream face, asphalt footpath crest, downstream slope (1500m) protection with 3D geotextile membrane (C350 Vmax). Protection extend over berms, or otherwise ~ 2m beyond existing embankment toe line.	Removal of trees from existing embankment. Reuse of existing riprap. Keeping rabbits under control while the fresh grass established.
Pandora	B	Concrete - broad crested (sensitivity analysis for labyrinth weir to check dam raising).	A low level 300w x 300h penstock, incorporated into main spillway design.	Sloping outside side walls to spillway. Mass concrete base onto rock.	5.6m3/s (10,000yr), velocity 1.12m/s, Q=Cd √g b H1.5, Cd= 0.544 (value for streamlined broad crested)	Simplistic 3.5m long RC weir, 2.3m high, 12.1m channel, positioned next to road for access purposes. Masonry chute 10m long downstream of spillway.	Greater depth of silt than anticipated, piled coffer dam for lower section. Concrete infill to the rockhead below the new spillway. Spillway central location.
Pont y Cerbyd	A	Armorflex	Separate 1.5m x 1.2m culvert through embankment (left hand side of new spillway).	N/A Crest beam with Armorflex.	125.8m3/s inflow (PMF), velocity 7.5m/s, Cd = 1.5.	51m long RC crest beam (0.7mx0.4m), 20 wide spillway channel, 1.8m high redi-rock wing walls. Armorflex laid panels downstream, extended beyond toe of embankment. Downstream slope 1V:5.5H.	Winter working – reservoir impoundments and spillway operating during construction, with only 50% of spillway crest available whilst other 50% was being worked on.
Pontarddulais	A	Grasscrete, stilling basin downstream.	1.52m x 2,25m culvert under spillway (discharging into stilling basin).	Foundation down to formation level clay.	111.78m3/s (Summer PMF) spillway discharge (assuming culvert blocked), velocity 7.88m/s.	50m long grasscrete , ~60m wide (crest far side of stilling basin) with 1.86m high wing walls at highest point on crest. 1V:4.5H downstream slope and stilling basin.	Some issues around placement of clay and installation of grasscrete.
Tynymynydd	A	Concrete weir and Dycel sides, rip rap downstream.	N/A	2 RC concrete walls with Dycel in between, with compacted clay underneath.	17.79m3/s inflow (Summer PMF), velovity 7.5m/s, Cd = 1.7.	RC crest beam in spillway channel (length 10m, width 3.8m), flanked by Dycel access ramps on both sides leading to rip-rap lined downstream channel	Dam with peat core. 2020 Covid Pandemic – contactor demob during first lockdown.

Figure 13. Tynymynydd, Concrete/Dycel.

Figure 14. Bwlch, reinforced concrete spillway including drawoff.

CONSTRUCTION ISSUES

Access

One of the most significant problems on NRW sites has been access. Many locations are remote and within Sites of Special Scientific Interest (SSSIs) or other designated areas; it has therefore taken time to obtain permissions/licences to improve access.

Example: *Rhiw Bach Quarry discontinuance - no access available to the site other than through bogs, a forestry coup with no access or historically important heritage site. This reservoir was discontinued with helicopters used to bring materials to site and the new outlet channel constructed by creating a notch in the embankment and leaving it to naturalise.*

Ground Conditions

Ground investigation works can only ever give an indication of anticipated ground conditions. It is not uncommon to find different, challenging, foundation conditions that have to be allowed for on site.

Example: *At Pandora reservoir (Figure 15) there was greater depth of silt than anticipated. This resulted in concrete infill to rockhead below the new spillway. In addition, it had been anticipated that the rock would be weathered and fractured, therefore permeable, in the location of the berm. More intact rock was encountered, with no such fracturing, suggesting low permeability. A slope stability indicated this could give rise to excess water pressures within the clay, leading to failures at the toe of the slope. Therefore a zone of higher permeability material (crushed slate) was installed, connecting to the toe drain.*

Concrete setting times

With some of the other remote sites extended concrete batching to placement times were also an issue.

Example: *Bwlch Nant yr Arian (Figure 14) - Concrete for the spillway had to be offloaded from delivery wagons and then transported in smaller vehicles down a hillside for subsequent installation. A retardant was used to ensure the concrete did not go off before it was placed.*

Inundation

The time at which a new spillway is at highest risk of failure is during construction and initial operation, therefore careful consideration was always given to emergency planning and temporary protection.

Example: *At Cyfty reservoir (Figure 16) even though 80% of the inflow was diverted and the reservoir was drawn down, significant rainfall events led to inundation of the works.*

Figure 15. Pandora rock foundation.

Figure 16. Cyfty spillway inundation.

Aesthetics

Wherever possible the selection of spillways has been designed to take account of the surrounding environment and associated aesthetics but sometimes there are issues with delivering the desired result.

Example: *Pen y Gwaith - Exposed aggregate finish was hampered due to hot weather at the time of pour, adversely impacting the effectiveness of the surface retarder product. Scabbling and some hydro-demolition was needed achieve the desired finish.*

Fish spawning

The fish spawning season can significantly impact construction programmes and restrict working periods.

Example*: Llyn yr Wyth Eidion – Spawning meant that works associated with a fishpass could not be completed in one season, with the contractor having to pull off site and come back to undertake spillway/embankment gabion basked repair works 6 months later.*

Temporary Works

Reservoir drawdowns, flow diversion, siphons and coffer dams were required to assist in the construction of the various spillways. Access routes, cost and constructability were all considered during the selection design.

OPERATIONAL PERFORMANCE

As spillways continue to be improved or replaced NRW has the opportunity to evaluate the success of the installations. To date the structures have performed well, but some operational consequences have been identified which will be taken into account for subsequent projects:

1. Where open mat concrete systems have been installed and spillways have operated before grass has established, material has been lost with subsequent reinstatements required, or even more significant action such concrete infilling.

Managing Risks for Dams and Reservoirs

2. In recent years, the frequency of spillways operating has increased, due to more frequent and higher intensity storms, resulting in more operational time/cost.
3. On flood storage reservoirs where embankments have been lowered to provide an uncontrolled washland incorporating a barrier bank, reinforced turf has been used. Although this met the specification in terms of flow/velocity and inundation time, little consideration was given to land use. Livestock was introduced to some embankments, leading to damage over a short space of time and exacerbated during a flood event. Consideration to different products is needed for a spillway is subject to other uses.
4. Operations and maintenance – NRW has instigated a regular (monthly) Reservoir Keepers' forum to spread experience across the portfolio of reservoirs. This has been crucial in discussing issues such as seepage, monitoring equipment being installed (V notches, crest pins, CCTV, telemetry), and also maintenance equipment such various grass cutting machinery. It has fed back into design in terms of preferences of different of slopes, access methods (steps vs pathways), valve/penstocks, handle arrangements and operational effectiveness of spillway types (during floods). As an example, ropes are being incorporated into some spillway designs to facilitate S12 inspections.

LESSONS LEARNT

The programme of reservoir works undertaken since 2014 is a continuous process, with priorities set by risk level. This has allowed lessons learnt to be applied across design, construction and operations and simplified procurement routes. The first capital scheme from new registrations was Cyfty reservoir, which took six years, finishing in 2021. Therefore it has now been operational for three years. Some lessons learnt across the portfolio include:

- Procurement – during the 11-year period of works, NRW consultancy and contractor frameworks changed from a mini competition to direct award with two designers and four contractors. The benefits of this have been a continuity in design and construction teams, moving from one reservoir project to the next, with lessons from one applied to another.
- Early Contractor and Designer Engagement with QCE – A decision was made to allow the designer and contractor who undertake the scheme to attend S10 inspections. This allowed them to understand likely MIOS requirements, and afforded the opportunity for them to discuss solutions and any potential issues (access, plant, locations).
- Annual Lessons Learnt Workshops – A two-day annual lessons learnt workshop was held with designers, contractors, site supervisors, as well as Project Managers, Project Executives and commercial teams. Within this workshops, overviews of schemes would be presented, lessons shared or issues highlighted and discussed, with common themes identified, potential improvement listed and actions with timescales allocated to implement them across the portfolio.
- Enabling Works – A decision was made early on in some projects, that to assist the main construction works, it was beneficial to undertake enabling works to improve access. Due to work within SSSIs and other designations, approvals could take a significant amount of time. As such, separating the access works from the main contract whilst design of the main works was being undertaken saved significant time relative to the MIOS deadlines.

- Legacy Dams – Due to the extensive number of legacy dams encountered, unusual conditions have been found throughout (peat cores, old outlets, unusual foundations) and had to be resolved on site. Frequently old maps from 1880 indicated a sluice present, but none were visible or found by divers. Upon draining the reservoirs, old buried cast iron outlet pipes of unusual size or wooden culverts were found. These had to be removed/grouted or sealed in some way. These typically linked to seepage locations previously observed.
- Designations – The majority of sites received a provisional High Risk designation from the Enforcement Authority; in some instances this was challenged and studies undertaken to provide greater detail with respect to reservoir details and downstream implications should they fail. By undertaking various studies and detailed inundation modelling it was possible to provide sufficient information to a QCE to advise that a Not High Risk designation was applicable. This evidence was presented to the Enforcement Authority and subsequently, following review, several sites were amended to Not High Risk.
- Aesthetics – Many structures had significant aesthetic and historical legacy. New designs had to try to preserve as much as possible whilst not affecting stability. For example, masonry faces were retained with new filter drains/ berms installed downstream.

CONCLUSIONS

There are many different types of spillway that have been designed and installed on NRW reservoir sites over the last ten years. Their selection, design and construction have been influenced by flow requirements, relevant industry guidance (ICE, 2015; CIRIA, 19787), their location, available space, suitable foundations, aesthetics and the specific duration and scale of design storms. Each spillway has distinct benefits, but also potential issues. The final selection is a balance of all of these, whilst meeting the key driver to safely transfer flow from the reservoir to downstream without jeopardising the integrity of the dam.

ACKNOWLEDGEMENTS

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The British Dam Society

Challenges in inspecting and assessing performance legacy bellmouth drop shaft and siphon spillways

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SYNOPSIS The majority of impounding reservoirs have overflows comprising a spillway discharging into an open channel that leads to a receiving watercourse downstream. During the 20th Century, alternatives were developed including bellmouth drop shafts and siphons. These can introduce efficiencies but can be difficult to analyse. This was recognised at design stage and model testing was typically undertaken to develop the head/ discharge curve. Over time, many of the model test reports have been lost, including the caveats about the limits of the studies. The duty to independently assess all aspects of a dam has been emphasised by the Safety Review Report that was issued following the Toddbrook incident. This can present difficulties in not just confirming the head/ discharge curve but also the physical inspection of the structures. This paper looks at examples, problems encountered and ways forward.

INTRODUCTION

Originally, dams were built using open spillweirs and channels to remove flood water from the reservoir to the downstream receiving watercourse. During the 20th Century, alternatives were developed to the open channel spillways including bellmouth drop shafts and siphons. Hydraulically, those types of spillways are more efficient in passing higher flow volumes in limited space. Often, these are the only overflow provision, and the safety of the reservoir depends on their efficient operation. Any failure of either a bellmouth drop shaft or siphons can lead to overtopping and an uncontrolled release of water.

The report into the incident at Toddbrook included a recommendation that spillways should, where possible, be physically inspected to look for and to try to quantify defects. This can be a challenge for siphons and bellmouth drop shafts. The very nature of them makes physical inspection almost impossible using conventional means. This paper's aim is to highlight some of the challenges encountered in practice when inspecting and maintaining these types of spillways.

The challenges associated with these types of spillways are exacerbated by the increasing unpredictability of climate patterns. Alterations in atmospheric conditions have led to an escalation in the predicted volumes of rainfall for various return periods. Consequently, overflow facilities, once deemed adequate, are progressively approaching their capacity limits.

Simultaneously, the inherent design of bellmouth drop shaft spillways and siphons does not readily accommodate modifications aimed at enhancing performance under conditions of increased flow. Performance rating curves, typically derived from historical reports, are intrinsically linked to the specific design features of the respective spillways. However, it is important to note that caveats from the original design performance assessments are frequently overlooked. This omission can lead to further inaccuracies in comprehending the performance of these spillways under the revised hydrological conditions.

BELLMOUTH DROP SHAFTS TYPICAL CONSTRUCTION AND VARIATIONS

The bellmouth drop shaft typically incorporates a circular opening weir at the top water level (TWL). The vertical cross-sectional shape of the weir may resemble a bellmouth, which lends the structure its name, although this is not a universal characteristic. Indeed, there exists at least one instance where the cross-section remains constant from the apex to the base of the structure.

The lower extremity of the drop shaft is designed with a radiused curve that guides the flow towards an outlet tunnel (refer to Figure 1). Notably, there is a documented example featuring an abrupt ninety-degree bend at the base.

The construction material predominantly used for these structures is reinforced concrete, chosen for its ability to withstand the forces and velocities encountered. However, it is not uncommon to observe masonry facing, which, while aesthetically pleasing, can be subject to the effects of jet velocities and negative pressures on the structural surface.

The drop shaft may exist as a standalone structure or be integrated into the valve tower. Accessibility varies; some structures may feature a footbridge, while others may be completely inaccessible. There is a particular case where, despite theoretical accessibility, the platform configuration precludes a convenient line of sight to the drop shaft.

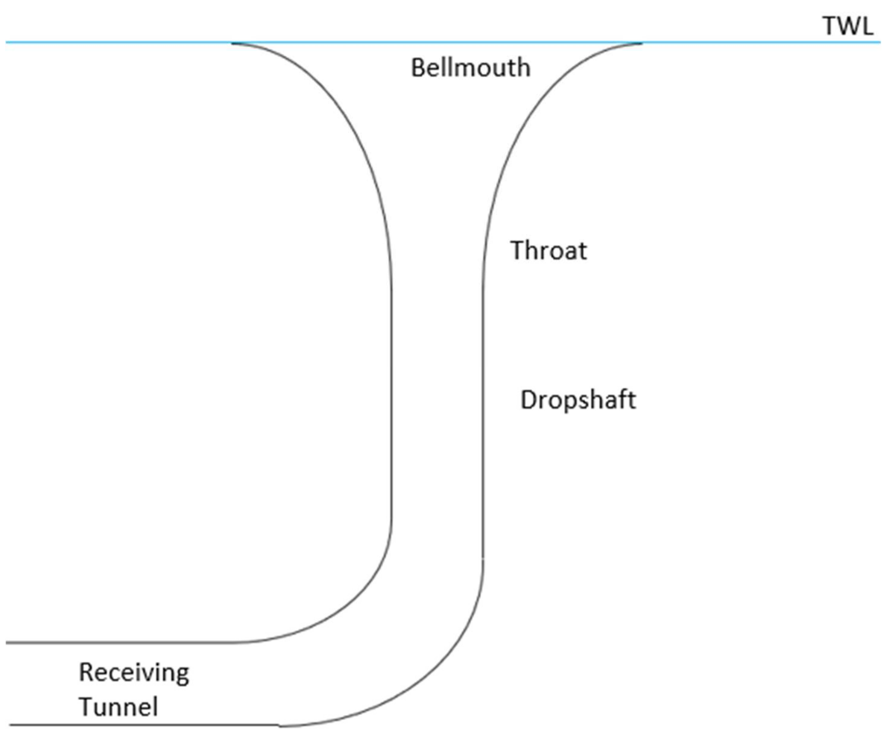

Figure 1. Typical bellmouth spillway section

BELLMOUTH DROP-SHAFT INSPECTION CHALLENGES

The prevailing best practice in reservoir inspections necessitates the inspecting engineer to conduct a physical examination of all components of the spillway structure. However, the unique geometry of bellmouth dropshaft spillways, which typically feature a vertical shaft extending several tens of metres, often precludes the possibility of direct visual inspection.

In instances where a footbridge is present, it becomes feasible to observe the structure from the top. This vantage point offers a clear view of the spillweir, although the visibility of details diminishes with increasing depth. At the throat of the structure, the lighting conditions may be suboptimal, making it challenging to discern specific features. Illustrative examples of the potential views are provided in **Figure** through Figure 5.

Figure 2. Bellmouth view from footbridge

Figure 3. Leakages seen from footbridge

The views in Figure 2 and Figure 3 are obtained from the perspective available from the footbridge. While inspecting the spillway from that vantage point can induce a sense of vertigo, it provides a reasonably clear view of the upper components of the structure. In Figure 2, the throat of the drop shaft is not visible due to insufficient lighting. Figure 3, however, reveals certain defects. Notably, there is a small jet of water emanating from the left, and a significantly larger jet of water in the upper middle.

Assuming the reservoir water level is below the top water level, it should be feasible to traverse the tunnel to inspect the lower portion of the shaft. This, however, can present a challenge. The invert may be slippery, yet navigable. At the base of the drop shaft, typically, the curve steepens from the slight decline of the tunnel to the vertical shaft. The distance that can be traversed is contingent on the traction between the boot and the concrete and the curve degree, which is likely to be suboptimal, and the inspecting engineer's nerve. This endeavour must be balanced with the imperative to maintain safe working practices and prevent slips and falls.

The image depicted in Figure 4 is representative of typical observations from the base of the drop shaft. The contrast created by the light penetrating from the top, particularly on sunny days, complicates the task of observing the sides of the structure. Upon enhancing the image, as shown in Figure 5, potential bands of calcite become discernible, suggesting the possibility of cracking. Conversely, on rainy days, the act of looking upwards can pose its own set of challenges with droplets falling directly on the inspecting engineer, obscuring leakages and increasing the slips and falls risk.

Figure 4. View from the base of the drop shaft

Figure 5. Post-processed image

SIPHON SPILLWAYS TYPICAL CONSTRUCTION

Siphons represent another category of enclosed spillway structures. When utilised as spillways rather than emergency drawdown facilities, they are typically designed to be self-priming, implying that the siphons do not require pumps to evacuate air from the structure. These structures can be found on both embankment and concrete dams.

The structure can be broadly characterised as a weir, topped with a hood. The profile of the siphon is often sinuous, a design feature intended to optimise the efficiency of the streamlines. The profile may incorporate steps to segregate the flow on the discharge side and secure the flow against the hood. This action seals the structure outlet and inhibits air intake from the downstream end of the structure (See Figure 6).

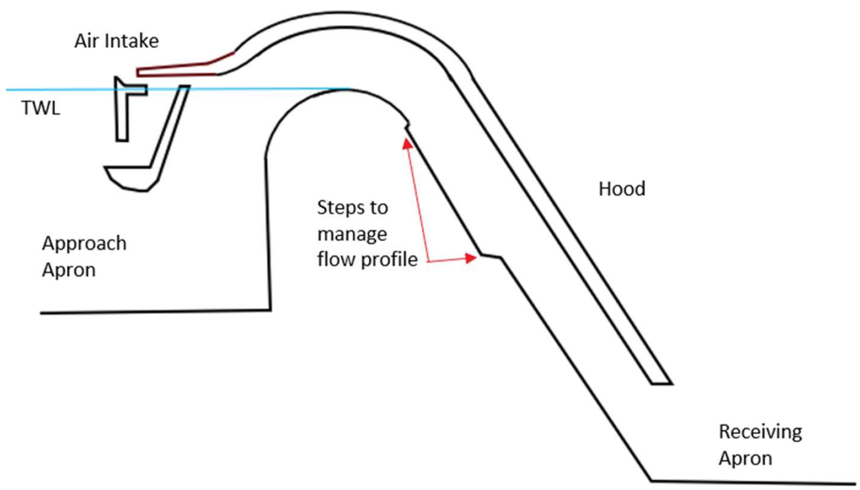

Figure 6. Typical siphon arrangement

The siphons usually have steep chutes, so that the velocity of the flow that develops on the siphon chute promotes air entrainment. In conjunction with the formation of a seal at the inlet, the air is expelled from the siphon, thereby priming it. Additionally, air regulation may be employed to manage the initiation of the priming process.

SIPHON SPILLWAYS INSPECTION CHALLENGES

Siphons can present an even more complex challenge for visual inspection. The inlet leg is characteristically elongated and steep. Inspecting the upstream side may necessitate access by boat, a strategy that is only viable if the water level is below the entry point. The presence of deep water could render the use of ladders impractical. Even in dry conditions, the placement of a loose ladder could pose safety risks.

Should access to the upstream side be possible, the view is likely to be limited. While it is feasible to identify defects, it is equally plausible to overlook them. In the case of structures composed of reinforced concrete, deterioration is inevitable, manifesting as spalling, exposed rebar, and rust spots.

Air-regulated siphons may feature an accessible air intake. On the downstream side, access may be available in proximity to the siphon, or it may only be feasible to observe from the toe of the dam. Gaining access inside the siphon is likely a specialist task, given the confined space and vertical heights. Consequently, the outcome is an impression of the condition, accompanied by numerous caveats.

Figure 2. Siphon inlet inspection from a boat

Figure 3. Siphon inspection with reservoir drawn down

As depicted in **Figure 2**, this perspective of the siphon inlets can be captured from a boat, however the health and safety during a boat inspection is highly dependent on the tranquillity of the day. The presence of wind can induce waves forming, which upon reaching the dam, may reflect and amplify in magnitude, thus potentially rendering the inspection process from a boat rather damp for the inspecting engineers. Occasionally, the reservoir water level is down, and it is possible to perform an inspection of the siphon inlets in the dry as shown in **Figure 3**.

Internally, the structures tend to be in reasonable condition. However, structural defects can be spotted in siphons constructed some time ago. For example, as shown in **Figure 4**, rebar corrosion can be observed on the inside of this siphon structure, causing the cover concrete to spall. At this site, the damage is 5m above the observation point, which can be challenging to spot in reduced light conditions or from a rocking boat.

Figure 4. Spalling on siphon roof, view from siphon inlet.

Figure 5. Rebar corrosion in air intake

For optimal hydraulic performance, siphons require relatively smooth surfaces without steps to obtain the design flows. The damage shown in **Figure 4**would be disruptive and reduce the efficiency of the siphon. In addition, as siphons, when primed, operate with relatively high flows and velocity, and depend on vacuum seals being created under such structures. Irregularities and step changes like this can cause negative pressures and cavitation to occur locally, expanding the damaged area during operation.

In **Figure 5**, corroding rebar can be seen in an air intake. The opening is only 300mm high so access to repair is difficult. There are hidden faces and if corrosion is present on one face, it is likely to be present elsewhere.

If repairs to the siphons have been carried out, it is difficult from a distance to determine if the original profile has been preserved or if there is unevenness or even steps. There could be a difference in the roughness where a repair mortar might be smooth and the concrete might be rough. Roughness can arise due to dissolving of the cement matrix particularly in areas of soft water.

Figure 6. Siphon internal structure, view from downstream end of siphon.

As illustrated in Figure 6, the roof is situated approximately 10 metres above the observation point at the downstream end of the siphon. The drawings indicate the presence of steps designed to separate the flow from the invert and pin it against the siphon hood. However, the visibility and condition assessment of these steps pose a challenge. The precarious nature of a loose ladder on a curving invert renders it unsafe, thereby limiting observations to a distant perspective.

FLOW CAPACITY ASSESSMENT

Engineers have long recognised that traditional hand calculation methods are not suitable for evaluating the capacity of bellmouth dropshafts and siphon spillways. To address this, physical model testing has been implemented to formulate the discharge curves. The resulting discharge curves of these tests are frequently encapsuled in the inspection reports. However, over the course of time, the original model test reports can become disassociated from the primary reservoir file, particularly during the transition from paper to digital formats. Consequently, the only accessible data is often confined to the information contained within the inspection report, which is often limited to the rating curve only. Details about the modelling study become lost.

The physical modelling reports comes with caveats. All model testing is constrained by laboratory space so the biggest scale that can be used might be limited. Trying to model everything might mean a scale that is too small, and the micro water behaviour starts to dominate. Alternatively, parts of the spillway can be omitted, to allow for bigger scale. However, that might mean unforeseen limits on the capacity.

Efforts to retrieve model test reports from alternative archives can be undertaken, albeit with varying degrees of success. It is noteworthy to mention that models predating the 1970s were constructed using imperial units, and on occasion, conversion errors can be present, thereby becoming an evident fact once quoted in the inspection reports.

Capacity Assessment Challenges for Bellmouth Dropshaft Spillways

On bellmouth dropshafts, physical models often replicate the weir, the dropshaft and the start of the receiving tunnel. This is because their efficiency is considered to primarily depend on the capacity of the throat exceeding the discharge at the weir. Once the weir discharge exceeds that of the throat, then any increase in discharge depends on sharply raising the water level. With such model set up, immediately, there are assumptions about the hydraulic capacity downstream of the cut-off point of the model which could become dominant, if the gradient is slack and if the tunnel is long.

At the weir and the top of the drop shaft, the models tend to include any bridge piers, but not necessarily the topography or nearby appurtenant works. This raises the question of whether the flow is from the full perimeter of the bellmouth or whether physical restrictions such as towers or earthworks could adversely affect the flow paths. Only the original report can answer such questions.

Capacity Assessment Challenges for Siphon Spillways

In the case of siphons, the models generally used Perspex that is fairly smooth and a low friction material, so that the air regulation and evacuation can be observed. Therefore, the scale model would struggle to represent the actual friction of the siphon walls, thus introducing uncertainty to the siphon discharge efficiency.

Features are introduced to the siphon geometry to manage and maximise the siphon performance over the range of possible flows, such as steps on the downstream face and air intakes. However, due to the scaling factors of the model air entrainment cannot be modelled accurately, as surface tension and air pressure does not change with scale.

Limitations in the laboratory space might also mean that the test only takes place on a single siphon. In the model test reports, there are warnings given that the results might not be replicable if the siphons are deployed in an array. The warning can become lost and an optimistic rating curve becomes established fact. Post modelling, the designers might have specified slightly different threshold levels to ensure different priming of siphons in an array of siphons to avoid surges as all the siphons prime simultaneously. This indicates a divergence of the finished facility away from the original model data.

DISCUSSION

Alternative Inspection Methods

With the recent development of technologies, alternative inspection options have emerged. In instances where secure access from above is available, point cloud devices can be deployed into a bellmouth dropshaft. These devices are capable of capturing detailed dimensions and identifying cracks or other discontinuities. Although the data necessitates post-processing, it provides a comprehensive record of the current condition. If concerning features are detected, it may be feasible for roped access specialists to furnish more detailed information.

Detailed point-cloud surveys with remote operated vehicles and drones of siphons have also been successfully conducted. However, unless the Inspecting Engineer possesses training in specialist access techniques (roped access in confined spaces) to be able to access and inspect the siphon barrels in person, the information that they will be using will inevitably be second-hand. Despite this, point cloud surveys offers a more comprehensive overview compared to merely observing from the structure's ends.

Challenging Capacity Assumptions

As discussed above, it is important to be able to question interpretations and challenge previously held opinions. Does the structure being inspected match the structure that was modelled? What are the differences, and could they make a material effect on the capacity of the structure?

As mentioned previously, climate chaos is forcing a rethink on the magnitude of storms for given return periods. Generally, there is an increase to reflect the greater moisture bearing capacity of a warmer atmosphere. As a given, the flood study needs to be reassessed.

The original designs of siphon and dropshaft spillways tended to be highly optimised for given flow figures, with some marginal allowance. There was a flow figure in mind and an allowance for unknowns. A position can be reached where there is uncertainty about the ability of the system to pass the safety check or the design flows. In part, there is insufficient information to be able to use the original design model tests to provide a definitive answer.

In such instances, it is prudent to re-analyse. It is also prudent to obtain an accurate survey. This can pick up features such as adjacent structures or earthworks that can change the approach stream lines and can use actual levels and dimensions. For siphons, it is feasible to verify if different thresholds were provided or if the facility was constructed to a single level. Unevenness can also be taken into account.

The way forward is to use Computational Fluid Dynamics (CFD). This is effectively a 1:1 scale model. There are limits on the computer space but it is possible to obtain a model 'as-existing' and find the actual flow capacities.

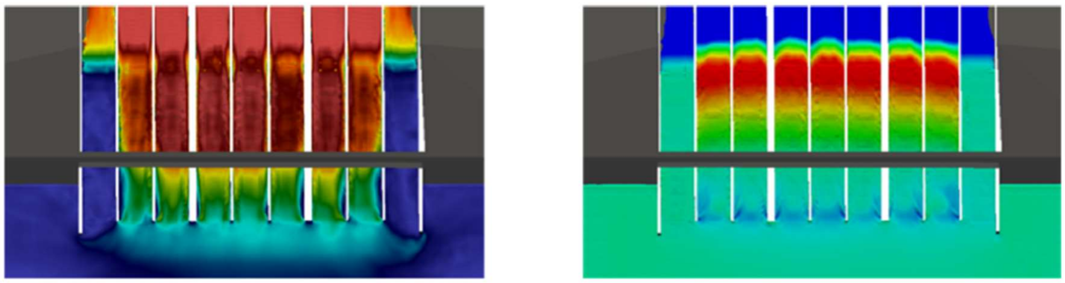

Figure 7. Siphon CFD outputs

The meanings of the colours in **Figure 7** are not important for this paper. They show different discharges for two storm events. The output does show that the siphons, that all had the same threshold level, do not have the same discharges in an array. There are effects at the ends and to the two side units.

This does mean that a discharge curve for the reservoir can be developed. In turn, the level rise for each storm event can be determined.

A feature that was not always included in the modelling was partial blockage. The level rises under PMF conditions can be high. Depending on wind speed and direction, there is a potential for debris to make its way to the outlet. Not all outlets have robust debris barriers. There is a potential for debris to wash into and cause blockage. It is beneficial to include a sensitivity analysis into the effects of blockage. This might feed into a plan to intercept debris. It is certainly important for the development of the flood plan.

As noted previously, these facilities were designed efficiently. A more accurate assessment helps to identify the adequacy of these facilities. Inadequate would be defined by the loss of freeboard because the level rise to achieve the discharge is too much. Solutions are beyond the scope of this paper.

CONCLUSION

In conclusion, there are challenges to follow the recommendations given in the Safety Review. There are techniques that can obtain better information that can improve the quality of the recommendations. It is important to understand the actual discharge curve for the reservoir and that original model tests might not provide sufficient data to do so. In the absence of good information, assessment depends on accurate surveys and re-analysis using CFD. Sensitivity testing can be included to check the effects of blockages.

The British Dam Society

Re-establishing and Improving Scour Capacity at Daer Reservoir

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SYNOPSIS Daer Reservoir was formed in the 1950s by the construction of a 43m high, 790m long earthfill embankment dam with a concrete corewall.

During the winter of 2021/22 monitoring showed that the water levels in the vertical drains on the downstream side of the corewall were significantly higher than previously recorded and there was increased wetness from the downstream face. A review of the monitoring data found that drainage flows increased significantly once the reservoir was within 2m of the full supply level. Attempts to lower the reservoir using the scour (bottom outlet) pipe found it to be restricted by debris from the valve house that had collapsed in 2005. Following works in 2022 to clear rubble from within the scour pipe, it was found that the 24" needle valve which controls the scour discharge was in poor condition and uneconomical to refurbish.

While the scour was being cleared, a notch was cut in the spillweir to aid control of the reservoir level. During this time, a 24" diameter washout off the supply main was used to control the reservoir level. Due to this frequent operation, the washout valve became damaged. Repair of the valve would have required shutdown of flow to the works.

This paper will briefly outline the investigations and cause of the wet areas and the difficulties and measures taken to control reservoir levels, and the works undertaken to re-establish and improve drawdown capacity. These include replacing (upsizing) the scour needle valve, installing permanent penstock gates within the notch formed in the spillweir, and replacement of the 24" diameter washout off the supply main without interrupting flow to the works.

INTRODUCTION

Introduction

Daer Reservoir is impounded by a 43m high, 790m long earthfill embankment dam with a concrete corewall. It was formed in the 1950s to provide drinking water and currently supplies a population of around 200,000. Construction drawings show that the corewall was formed using 6ft high tongue and groove precast concrete panels as permanent formwork with an in-situ concrete infill. The wall was formed in 24ft wide panels with a central 4" diameter plug of poured bitumen. The corewall is described in more detail in a paper by McHugh et al (2023).

The outlet (Figure 1) comprises a 42" supply main with three draw-offs connecting to a 42" diameter stack pipe in a dry valve tower. The supply main passes through a culvert below the dam before turning and rising over the spillway channel on its way to the adjacent Water Treatment Works (WTW).

There are two washouts off the supply main; a 12” washout at the head of the tunnel controlled by a 24” butterfly and a 12” fixed cone discharge valve, and a 24” washout at the left-hand end of the spillway bridge controlled by a gate valve. There is also a disused 900mm diameter bypass on the right-hand side of the spillway bridge with a 600mm diameter branch at the toe of the dam. This bypass had not been operated in some time and there were few records of this available.

The scour comprises a 36” pipe which is encased in the concrete floor of the culvert with a branch off to a compensation turbine and a branch off to a spill turbine, both located within the turbine building at the downstream end. The compensation flow at the reservoir is discharged through the compensation turbine and, when the reservoir is (or close to) spilling, the spill turbine operates to generate energy from what would have otherwise been wasted water. The scour discharges through a 24” diameter needle valve, the body of which is encased within the concrete foundation of the turbine building.

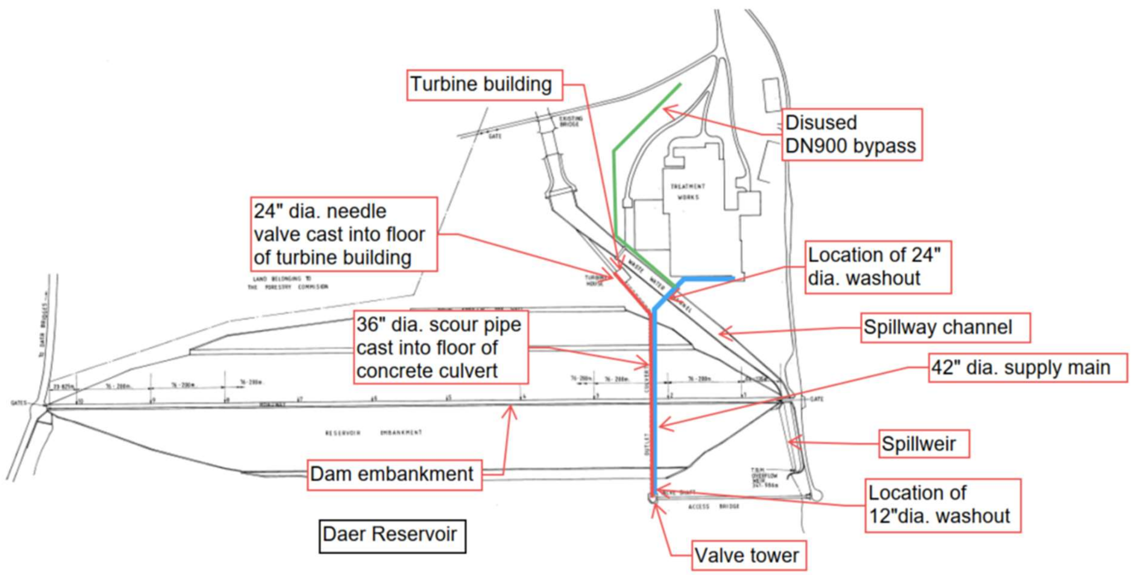

Figure 1. Outlet Pipework at Daer Reservoir

The reservoir has a side channel overflow (Figure 2) which is formed of precast concrete crest blocks on an in-situ base, discharging into a concrete channel down the left-hand mitre of the dam.

Figure 2. Overflow Weir

In the winter of 2021/2022, wet areas were identified on the downstream face of the dam. Given the previous slip at the site, described in a paper by Morrin et al (2016), there was a concern that further slips could occur. The Supervising Engineer monitored the wet areas and observed that, while they did not appear to be getting larger, there was an audible 'popping' sound at them which could indicate flowing water beneath the ground. As a precautionary measure, the water level in the reservoir was lowered until the cause of the wet areas could be investigated. When lowering the water level, the scour ran at full flow for around 24 hours before a sudden and substantial reduction in flow occurred. Subsequent investigations into the reduced scour flow, which included inserting an endoscope from the downstream end of the needle valve, found the upstream end to be restricted by concrete blocks. These were presumed to be from the valve house that had suddenly and catastrophically collapsed in 2005 (Figure 3). Consequently, in order to maintain control of the water level in the reservoir during this time, the Qualified Civil Engineer (QCE) instructed that a notch (approximately 4m wide x 1.5m high) be cut into the overflow weir. Given the catchment area of 47km^2, this was seen as the only viable way of attempting to keep water levels below the level at which there was a noted change in performance, whilst the scour was cleared. Forming a notch also had the benefit of offering the ability to enhance drawdown in the future by the inclusion of penstock gates.

During the time that the scour was non-operational, 24" diameter washout off the supply main was used to control the water level in the reservoir. This more frequent operation resulted in damage to the valve which eventually became inoperable, fortunately after the scour had been cleared.

The attempts to improve core wall drainage and the rehabilitation of the scour pipe was discussed in a paper by McHugh et al (2023) and an update on these items will be provided in this paper, along with a discussion on further drawdown reinstatement works and the installation of remote monitoring devices to provide real time information on the behaviour of the dam.

Figure 3. Valve Tower House Collapse

UPDATE ON REMEDIATION OF WET AREAS

In the winter of 2021/22, Mott MacDonald was instructed by Scottish Water to assist in identifying the cause of the wet areas, and the reduction in scour capacity, and devising a solution. It was concluded that the increased wetness and drain levels that had been observed were largely due to the ever-diminishing capacity of the corewall drains due to infilling by fines, resulting in increased flow of water from the corewall to the downstream face via horizontal pathways to the downstream face. Concentrated flow from a few of the joints were observed, by CCTV survey, at high water levels. It was decided to attempt to restore drainage capacity in the first instance, rather than attempt to stem the leakage from the joints, for example by reaming out and regrouting the bitumen joints, as overall leakage rates were low.

The drain clearance works commenced in May 2022 using a vacuum evacuator with a 3” hose to suck out infill from the drains. For the first few drains, the removed material was sampled, and particle size distribution testing was undertaken. This showed that the infill to the drains was likely to have come from the embankment (most likely the downstream). While using a CCTV unit to undertake the works, the precast concrete half-pipe drains were found to have large gaps in the joints. While probably not part of the original design, this assists drainage from the embankment into the drain but also allows migration of material from the embankment into the drain. Fibreglass patches were installed into the drains which were found to be in very poor condition and were at risk of collapse. The patches were only over a short vertical distance so would not have had a material impact on infiltration rates.

During the clearing works, some larger obstructions were encountered which could not be cleared using the 3” diameter hose of the vacuum evacuator. Some of the drains had the concrete cap from the drain lodged in them at a shallow depth (around 2m) and others had bricks / blocks at greater depths (up to 30m). For the drain covers lodged at shallow depths, the embankment was locally excavated to remove the obstruction by hand. For deeper obstructions, a drilling rig was setup on the crest to break up the obstruction which could then be removed by vacuum excavator.

At the time of writing, all (approx. 100) drains have been worked on and a further five drains remain to be cleared using the drilling rig method. The impact of clearing deeper obstructions in the drains on the water level in drain 23, which is located within the wet area, is shown in Figure 4.

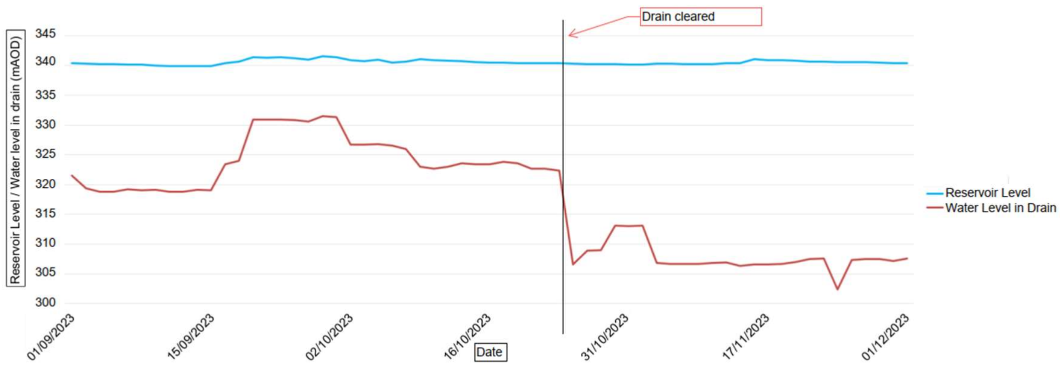

Figure 4. Impact of drain clearing on water level within drain

To avoid the risk of larger obstructions being dropped, or falling, into the drains, new drain covers were installed on all the drains.

After clearing the drains, a clear reduction in the water level in the drains was observed which indicates the works have been successful in restoring drainage capacity. Due to the inoperable scours at the site, the water level in the reservoir has been held in a range of between TWL-1.25m and TWL-1.5m since the drain clearing works were started. The impact of the drain clearing works when the reservoir is at TWL is therefore not yet fully known and will be monitored when the scours are rehabilitated and the reservoir level allowed to return to TWL. A longitudinal section is plotted in Figure 5 showing: the depth to the base of the drain before any clearing works, the depth to the base of the drain after clearing works, and the as-built depth measured from record drawings. This shows the extent of the infilling and the beneficial impact of the drain clearing. The plot was produced ahead of the final stage of drilling works to tackle drains 20, 21, 35, 50, and 70 which is ongoing at the time of writing.

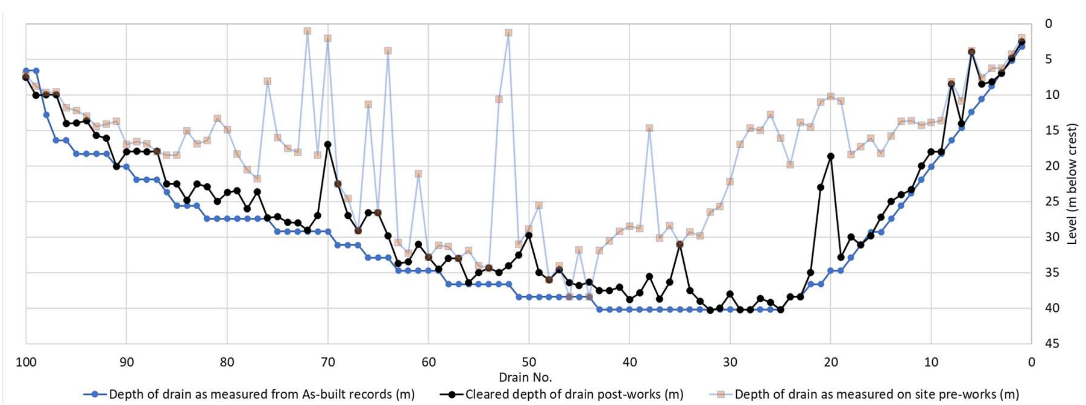

Figure 5. Longitudinal section through dam showing original depth vs cleared depth with as-built depth of drains

SCOUR REHABILITATION

The reduced scour capacity was found to be due to rubble which had fallen into the pipe when the valve tower house suddenly and catastrophically collapsed in 2005 (Figure 3). As discussed in the paper by McHugh et al (2023), the initiating event of the collapse was unidentified; however, poor bed jointing between the pre-cast corbels which resulted in long term over stressing and cracking, coupled with ongoing deterioration due to water ingress and freezing, was attributed as the primary cause of failure.

The rubble was cleared from within the scour pipe in 2023 and, while the pipe was empty, the opportunity was taken to partially dismantle the needle valve and replace seals and re-grease. During the strip down, cracks were noted on the valve internals as well as extensive cavitation damage to the valve body (Figure 6 and 7). The crack is thought to have been caused by rubble impact and the cavitation damage is thought to have been caused by mis-operation with the valve never fully closed, instead remaining at 1% open, outwith the operational range for the valve. There was a risk that this damage could have caused the valve to seize open, which would have prevented the turbines from operating and resulted in issues with providing compensation flows, or seize closed, which would render the scour unusable for drawdown capacity. As the valve is built into the concrete foundation of the turbine building, with restricted access, and is encased in concrete it is unable to be removed and in-situ repairs were not possible (the needle valve in operation is shown in Figure 8). A decision was therefore made to remove and replace the needle valve.

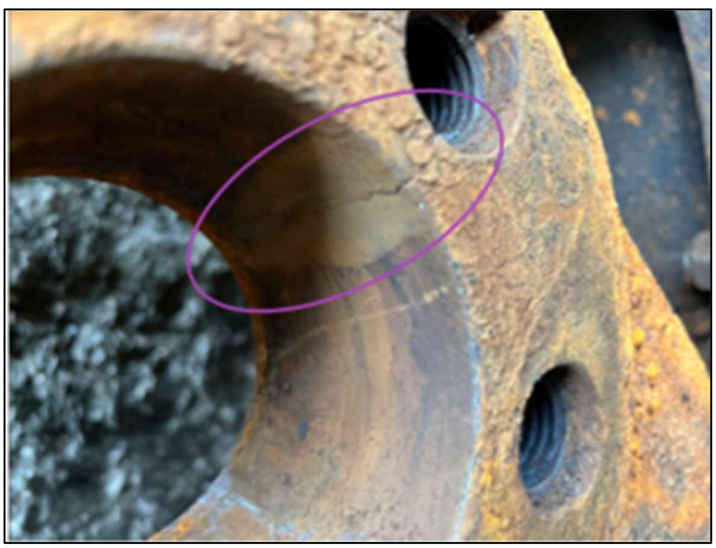

Figure 6. Cracking to valve internals

Figure 7. Cavitation to valve body

Figure 8. Needle Valve in operation in foundation of turbine building

DRAWDOWN ENHANCEMENTS

Drawdown capacity

Prior to undertaking any works, the existing drawdown capacity at Daer Reservoir comprised:

- 36” diameter scour pipe, tapering to 24” dia. needle valve: 4.8m^3/s at TWL
- 12” washout off the supply main: 1.1m^3/s at TWL
- 24” dia. washout off the supply main: 3.4m^3/s at TWL

This provides a total capacity of 9.3m^3/s which, over a reservoir surface area of 2km^2, results in a drawdown rate of 402mm/day. According to Table 6.2 of the Drawdown Guide (EA, 2021), the recommended minimum rate for a Category A earthfill embankment dam greater than 20m height is 1m/day. This is for a reference dam of earthfill embankment with clay core; however, the dam at Daer Reservoir has a concrete corewall which would likely be less susceptible to internal erosion than the reference dam, so a lower drawdown rate was judged as being acceptable. However, due to the size of the reservoir and catchment and known leakage issues, it was considered would be prudent to maximise the drawdown capacity at the site. The solutions to maximise drawdown capacity at the site are discussed in the following sections.

Scour Pipe Upsizing

As the 24” diameter needle valve at the downstream end of the 36” diameter scour line was to be replaced, the possibility of upsizing the valve was investigated. The headloss across the needle valve is relatively high so upsizing the valve has a substantial increase in the capacity of the pipe. The civil works required in replacing the valve are largely the same whether remaining with 24” or upsizing to 36” so the only major increase in cost is for the valve itself. From supplier estimates, the budget price for a 36” valve is around double the price of a 24” valve but this increase in price is relatively low when compared to the overall project budget.

The existing and proposed arrangement of the pipework at the turbine house is shown in Figure 9. The existing 36” scour passes through the back wall of a sump under the floor of the building before tapering to a 900mm diameter spool pipe with two offtakes for the turbines. The existing spool pipe has an integral taper (to 24”) welded as part of the second offtake and connects to the original scour line downstream which discharges through the needle valve cast into the foundation of the turbine building.

In order to upsize the needle valve to 900mm diameter, all in-line pipework on the scour line within the turbine house is to be replaced. From discussions with the turbine installer, the spool pipe was installed ahead of the turbines, and it is not clear if it is able to be removed through the doors of the building with the turbines still in place. The proposed pipework is therefore in two sections to for ease of installation / removal. Even with this increased flexibility, it will be difficult to fully fabricate the pipe offsite to match the angle, length, and orientation of the offtakes. It is, therefore, proposed to offer the pipe up and tack weld the offtakes in-situ before taking the pipe away for final fabrication.

Compensation flow at the site is normally provided through the compensation turbine (fed by the scour line) but when the turbine trips or is offline for maintenance, compensation is currently maintained through the needle valve opening a small amount. To reduce the risk of further cavitation damage to the new fixed cone valve, a DN300 tee off the scour branch for compensation flows is included. This is controlled by a DN300 gate valve (guard) and a DN200 fixed cone valve with hood (duty) with the duty valve automatically opening to a set percentage (to be calibrated onsite) when the turbine trips before tapering back depending on the flows read from a downstream flow meter.

The existing needle valve is to be replaced with a fixed cone valve with hood which directs the discharge flows to within the downstream channel. The new valve is to be installed within the channel downstream of the building to make future maintenance easier. To facilitate this, a new concrete plinth was installed within the channel to provide a support for the valve and act as a working area for the valve removal works.

At the time of writing, the replacement works for the needle valve are due to commence in June 2024.

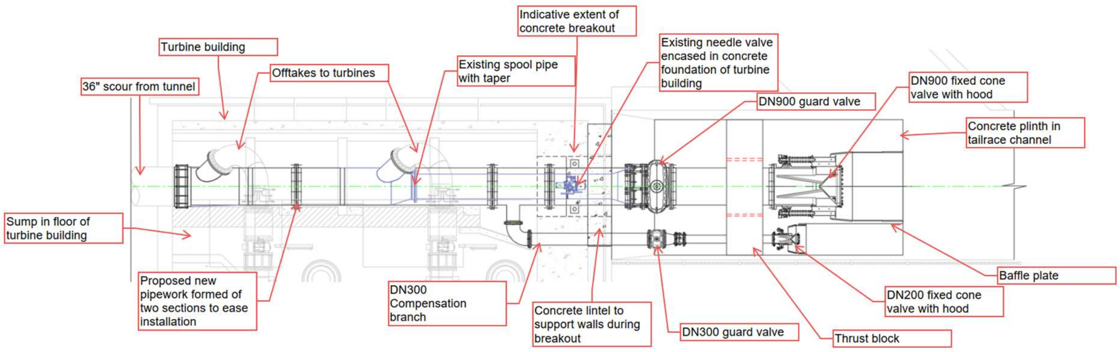

Figure 9. Scour upsizing works

Weir Notch Reinstatement

When the reservoir level is plotted against recorded leakage, there is a step change in the relationship at around TWL-1.5m to TWL-2.0m. Figure 10 shows the leakage measured at the two measuring chambers at the toe (rainfall effects have been removed by only considering data points for which the total rainfall for the previous three days was less than 1mm). This shows a step change in behaviour at around TWL-2.0m. This change is shown at different levels at other monitoring points, hence the range provided. The top 2m of drawdown is therefore key in reducing the leakage through the dam and the installation of penstock gates would be beneficial in reducing the water level in the reservoir to this 2m threshold, or as close to it as possible.

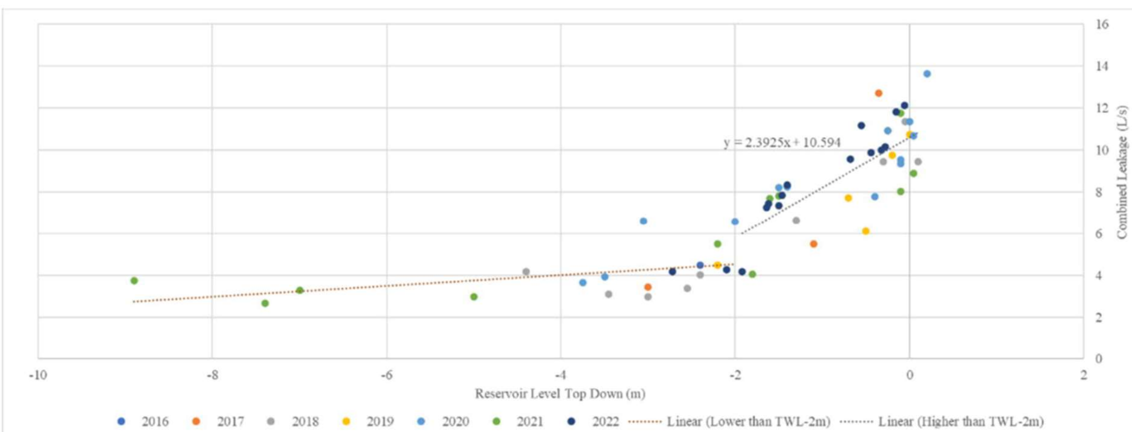

Figure 10. Leakage vs Reservoir Level

In order to maximise the size of the opening and utilise an existing concrete baffle wall to support an access gantry, a new concrete structure was formed upstream of the notch formed in the weir. The penstock gates are to be fixed to the downstream side of the wall with stoplogs (to allow maintenance on the penstocks) installed at the upstream side. There is a removable section of floor at the gantry and a lifting beam to allow the stoplogs to be lowered into position. The gantry is also covered by open mesh fencing to allow operations staff to safely access the gantry during adverse weather, with a section left uncovered to allow the stoplogs to be lifted onto the gantry from the adjacent roadway. Under normal conditions, the penstocks will be operated from a control panel within a new kiosk adjacent to the spillway channel. The supply to this kiosk is backed up by a site generator in the event of a mains power failure and if the onsite generator also fails, the penstocks can be operated by hand from the gantry. The penstock design is shown in Figure 11. At the time of writing, the penstock installation works are due to commence in June 2024.

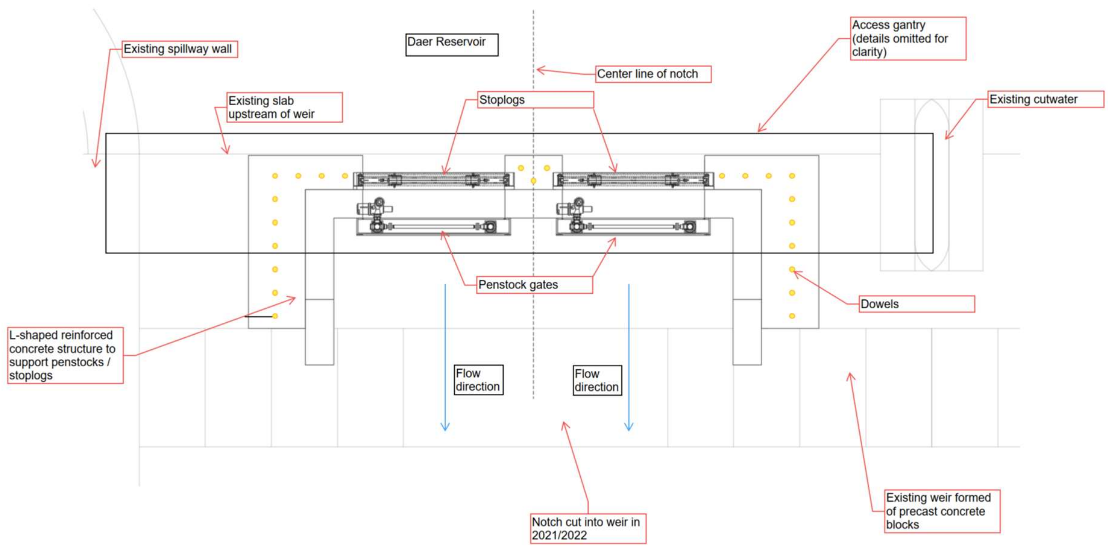

Figure 11. Penstock Design

Washouts off Supply Main

The 24" washout at the spillway channel is controlled by a gate valve which has seized closed and is inoperable but repairable. There is no guard on the 24" washout, so in order to repair the 24" gate valve (and replace the 12" butterfly), the supply main would have to be emptied which would have an impact to the works.

There are two options for the works while emptying the supply main; shutdown the works for the duration of the repair / replace or provide flows by other means (including hot tapping off the supply main). Initially, a shutdown of the works was planned but when the works were detailed it became clear that a number of shutdowns would be required (up to 6 No.) and there were concerns over whether there would be silt / sediment issues when starting up the works again, resulting in water quality issues. This option was therefore discounted in favour of providing flow by alternative means (temporary siphons). Due to the required length of the siphons and the need for contingency in case of breaking down of the siphon priming pumps, they were deemed to be prohibitively expensive.

Around this time, the disused bypass was identified, and investigations were undertaken to ascertain the route of this pipe and its functionality. The pipe was found to be in good condition and the valves on the pipe were found to be operational; therefore it was feasible to use this pipe. A decision was made to modify the bypass to provide additional drawdown at the site which would make up for the loss of the two existing washouts by upsizing the washout at the toe from 600mm diameter to 900mm diameter. At the time of writing, the design of the bypass extension is being developed but is likely to comprise around 50m of DN900 pipework discharging into the spillway channel via an impact style discharge basin.

Summary of Drawdown Enhancement

The proposed alteration works would provide (at TWL):

- 36" diameter scour pipe, discharging through a 900mm diameter needle valve: 5.9m^3/s

- 4m wide x 1.2m deep notch in weir: 9.7m^3/s
- 900mm diameter bypass extension: 5.2m^3/s

The proposed works will increase the drawdown capacity at TWL to 20.7m^3/s (890mm/day). This is an increase in drawdown capacity of 11.4m^3/s (123% increase). The most likely failure scenario is likely to be leakage through the corewall joints overwhelming drainage capacity, saturating the downstream face leading to slope instability and loss of support to the concrete corewall. A critical level in mitigating against this failure mode is TWL-2.0m as this is the level at which leakage through the dam substantially decreases. The installation of the penstock gates provides a high initial discharge, helping to mitigate against this failure mode, with the upsized scours available below this level. Automated remote monitoring would be expected to detect increased drainage rates and levels enabling drawdown.

DRAIN CLEARANCE UPDATE AND REMOTE MONITORING

Remote Monitoring Install

Currently, the monitoring at the site comprises twice weekly (increasing to daily during higher reservoir levels) recording of the water level at 14 of the core drains, recording leakage flow rate in the headwalls at the berm and the left-hand mitre, and recording flow rate in the two chambers at the toe of the dam which collect all leakage and rainfall at the dam. This recording regime places an onerous requirement on operations staff, as well as potential health and safety issues gathering readings in poor weather conditions, so it was decided to install remote monitoring devices at the site to ease this pressure. The remote monitoring devices also send the readings to a web-based platform to allow real time data to be taken at a frequency as desired by Scottish Water, currently 15-minute intervals.

Pressure transducers were installed at 20 of the core drains, selected as those which show a clear link to changes in reservoir level. For measurement of leakage flows, V-notch boxes and ultrasonic sensors were installed at three of the headwalls and at the measuring chambers at the toe. Pressure transducers were also installed at piezometers 17 and 19 which are in the line of the 2013 slip and close to the wet areas previously identified. A raingauge will also be installed with the aim of being able to remove the effects of rainfall on the leakage monitoring model; currently radar rainfall records are used. At the time of writing, the readings from the remote sensors are currently being calibrated against manual reads to ensure continuity. Having the remote sensors in place during the refill will provide real time information on the behaviour of the dam. The output from the sensors is included in Figure 12 and shows the response in the drain to the changes in reservoir level when the reservoir reaches around TWL-1.7m.

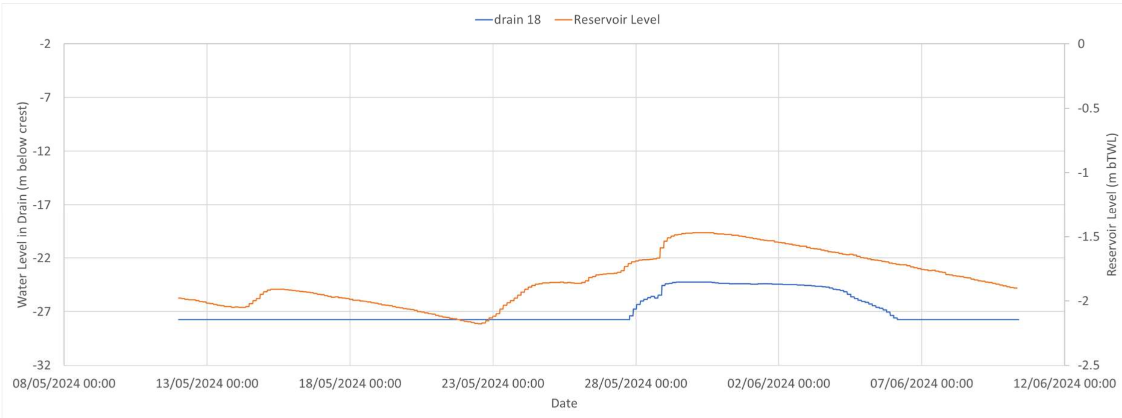

Figure 12. Example Monitoring Plot

DISCUSSION AND FUTURE WORKS

To date, the clearing of the core drains appears to have been successful in reducing the water level in the drains, but the true test will be when the reservoir is returned to a normal operating state (at TWL) over the winter of 2024/2025. Allowing the reservoir to return to the normal operating levels is contingent on the completion of the works to enhance the drawdown capacity at the site.

Having remote monitoring installations in place for the refill and return to normal operation will provide early indications of the dam's behaviour to allow the reservoir to be drawn down again to TWL-1.5m. If leakage rates at TWL are overwhelming the drainage capacity, then works to reduce leakage rates through the corewall may be required. Such works might include investigating those vertical joints that have been seen to be leaking, reaming out the bitumen plug and stemming the leakage by grouting, for example with an acrylic grout. If required, such works will be challenging and likely costly to safeguard water supply quality. Periodic drain clearance will be required unless a means of preventing ingress of fines can be found.

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The British Dam Society

Improving the emergency drawdown reliability at Llyn Brenig reservoir – Part II

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S SHAKESPEARE, Dŵr Cymru Welsh Water

SYNOPSIS This paper builds on the paper published and presented by Tudor and Morgan (2018) at the 20th BDS Biennial Conference in Swansea for the design of improvements of emergency drawdown reliability at Llyn Brenig.

Dŵr Cymru Welsh Water (DCWW) appointed Mott MacDonald Bentley (MMB) to install and commission the upgraded scour facilities, including the extensive temporary works required to enable construction to take place, and replacement of the "Goliath" crane mounted to the top of the valve tower located circa 300m from the reservoir's shoreline.

Management of water levels and isolations were required to enable gate replacement whilst maintaining a desirable volume of stored water. Draining the reservoir was not feasible due to the operational requirement to maintain flows to the River Dee for abstraction purposes. Issues arising during construction and performance of the enhanced system following commissioning and handover are also covered in the paper.

INTRODUCTION

Llyn Brenig is located in the county of Conwy around 15km south of Denbigh, north Wales. The reservoir feeds compensation flows to the River Dee and is a critical asset to the Dee Valley Consultative Committee in unison with Llyn Celyn and Llyn Tegid. The reservoir has a stated volume of 61,550,000m^3 and is impounded by a 50m high rockfill dam with a 1200m long crest length, constructed in the 1970s.

Following a statutory inspection carried out July 2015, two recommendations were made under section 10(3) of the 1975 Reservoirs Act. The first of these recommendations was thus: *"Remedial work and refurbishment of the hydro-mechanical and electrical components of the scour outlet works shall be implemented in order to improve reliability, security and operability of the system".*

Tudor and Morgan (2018) discuss how the design had been undertaken and identified the remedial action to be taken through to delivery on site. The remedial works taken forward to construction were:

- Replacement of the "Goliath" crane mounted to the top of the combined draw-off and overflow tower

- Installation of a new secondary isolation gate
- Replacement of the scour bulkhead gate
- Replacement of the primary scour gate
- Replacement of all gate control systems including new control panel and caballing

Third party users
The reservoir is a popular visitor attraction and required careful planning to minimise the effect on patrons. Osprey nesting, sailing activities, fishing competitions and even the RAC Rally had to be accommodated during the construction period.

GOLIATH CRANE REPLACEMENT
The existing crane on top of the draw-off tower was noted to be difficult to operate, unreliable and beyond reasonable repair. The decision was taken to replace the crane with an installation which could be controlled via a remote operating station, making the operation easier and safer by removing the need for elements of work at height (WAH).

The crane operates on two rails fixed to a steel structure and is supported by six concrete plinths. Following a structural assessment, the support structure was proven to be salvageable, however it required replacement of four of the six reinforced concrete supports. Temporary works were required to prop the 26t crane and support steelwork and jack the existing gantry crane and support steelwork up circa 3mm to allow the safe removal and installation of the new plinths (Figure 1). Once all the plinths had been replaced the existing crane was removed and the new one put in place, utilising the floating barge and associated lifting activities, including a 100t crane (Figure 2). The working platforms had to be anchored to the reservoir bed and a suitable weather window identified to carry out the removal and installation activities (Figure 3). Operative training was carried out following construction completion (Figure 4) and commissioning completed, should the need arise to install isolations for reservoir safety purposes.

Figure 1. Hydraulic jack supporting crane during plinth replacement (MMB)

Figure 2. Temporary quay constructed to allow installation of pontoons and lifting apparatus (MMB)

Figure 3. Pontoon in position and crane replaced (MMB)

Figure 4. Completed crane installation (ARUP)

ISOLATIONS

Llyn Brenig forms part of the critical Dee Valley Regulation system along with Llyn Celyn and Llyn Tegid. Flows are regulated in the River Dee for the benefit of water supply and industrial purposes. Initially it was requested that the reservoir level be lowered to remove the hazard associated with water pressures of up to 5 bar at the base of the reservoir tower, however this was not possible due to the aforementioned supply requirements . An alternative method had to be identified to carry out the works safely during construction and for those residing downstream. The decision was taken to isolate the tower from the reservoir body by utilising the provisions in the original design; namely to install the scour bulkhead gate (Figure 5). Given the age of the existing steel gate and condition of the assets, at a little under 50 years old, it was decided to replace the scour bulkhead gate in order to provide a guaranteed factor of safety (FOS) of 3 against failure. Finite element modelling was completed in order to provide confidence in the design and testing was carried out to prove strength and durability prior to installation. The new gate weighs 5.3t and was manufactured in Spain by Orbinox. With the existing scour bulkhead in place, a baseline for leakage flows passing the bulkhead was established as 6l/s/m. On replacing the gate with the newly fabricated gate, leakage flows were again measured and compared to that of the original gate. Leakage was established as 6l/s initially, however over a period of 10 days, the flows reduced to 3.1l/s as the seals bedded into the structure (Figure 6). In addition to replacing the scour bulkhead, work was programmed such that two points of proven isolation with continuous bleed and monitoring were maintained throughout the project.

Figure 5. New Scour Bulkhead being lowered into position (MMB)

Figure 6. Scour Bulkhead – leakage monitoring station. Note window for visual check

Table 1. Phasing of construction works showing isolations

	Scour Bulkhead Gate	**Primary Gate**	**Secondary Gate**
Existing Installation	Closed	Closed	
Stage 1	Closed	Closed	Installation
Stage 2	Installation	Closed	Closed
Stage 3	Closed	Installation	Closed
Stage 4 – Current operation	Removed	Closed	Closed

Prior to progressing onto the next stage, it was necessary to commission and test each of the newly installed gates. This meant two control panels and electrical systems were being utilised, one to operate the new and one to operate the existing installations simultaneously. Each stage was reviewed by the QCE prior to commencing works on the next stage.

ACCESSING THE VALVE TOWER

The valve tower is located approximately 300m from the embankment of the dam with no permanent bridge in place. Under normal circumstances access to the top of the valve tower is via the draw-off tunnel, and an internal staircase ascending 50m. It was not possible to traverse the new gates in one piece along the overflow tunnel and then manoeuvre them into position due to size limitations. It was decided to float the new gates across the reservoir to the valve tower using barges and then lower them into position at the base of the tower utilising the on-site crane. This solution brought with it a few complexities, including:

- The shoreline of Llyn Brenig is generally very shallow; this causes an issue with draught of vessels.
- It was necessary to construct a temporary quay to enable barges and boats to be craned onto the reservoir body and assembled.
- Water levels had to be managed within a tight band to allow for flood contingency whilst working in the live overflow and physical quay operation.
- A freeboard of 2m from the overflow was maintained throughout the works to prevent any overflows during construction.
- A 200t crane was situated on the quay to load the barges with materials and plant which were then ferried across to the valve tower. The newly replaced gantry crane was then utilised for lifting and lowering the new gates into position.

SECONDARY (DUTY) GATE

The new secondary gate was fabricated in Spain by Orbinox (part of the AVK group) and shipped to site in one piece; this weighed in excess of eight tonnes. Flow baffles in the overflow shaft had to be removed to allow access for the new gate to be lowered into position. These baffles were re-furbished and replaced upon completion of the works.

In anticipation of undertaking works to the primary gate, the project team was faced with the challenge of developing a methodology for removing the concrete surrounding the existing primary scour gate whilst working behind single isolation (Figure 8). This was required to allow replacement of the existing gate and frame. Following extensive review and development of temporary works, where the objective was to focus on the removal or reduction of risk for operators working under single isolation, the team worked closely with a specialist demolition subcontractor to develop a method to remove the concrete with engineering precision using remote/robotic control whilst being able to monitor the structure. This methodology was utilised for the secondary gate as a precursor to the primary gate works.

A rebate was cut into the existing structure to enable the invert of the channel to remain as existing. Remote operated plant was utilised for hydro-demolition to take place safely with constant monitoring for water ingress though the structure. No ingress was identified, showing the quality of the original installation. Upon lowering into position, studs on the frame of the gate were welded to the existing reinforcement prior to grouting the remaining void, securing the gate into position (Figures 9 and 10). During this process leakage flows around the existing primary gate and new bulkhead gate were constantly monitored for changes. All works were carried out under DCWW's Gold command system to monitor progress and resolve any identified issues. Following installation, the gate was commissioned and tested against full reservoir head.

Figure 8. Installation of primary scour gate

Figure 9. Completed reinforcement assembly prior to concrete profiling installation (MMB)

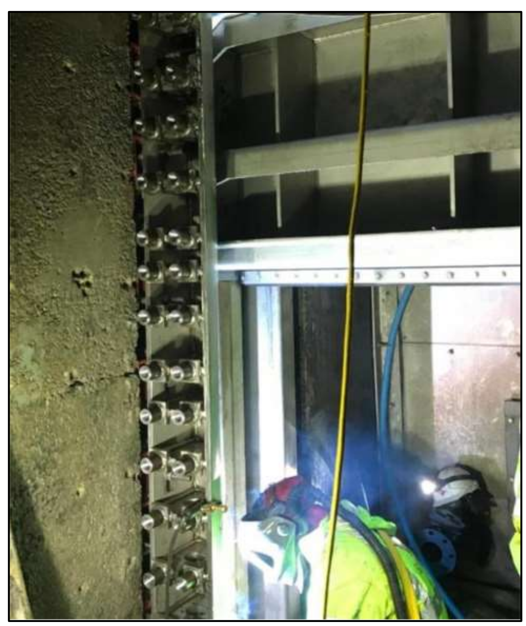

Figure 10. Welding of primary gate to existing tower structure (MMB)

PRIMARY (GUARD) GATE

Following successful testing of the new secondary gate, works progressed to the replacement of the primary gate.

Over the course of developing the temporary works for the scheme, the team developed an aluminium screen (Figure 6) which help to control a number of risks for the primary gate installation. The primary purpose of the screen was to monitor the existing leakage rate passing the bulkhead gate; this was completed by installing a throttled valve on each side of the aluminium screen and a float switch connection to a visual and audible alarm. 24hr CCTV monitoring was also in place to assess the condition of the work area prior to entry, and throughout the working day by a confined-space-trained 'top-man'. The second risk was to protect the bulkhead gate from any damage from debris, which was achieved through a robust but manoeuvrable aluminium screen. The third risk/complication was managing the existing leakage water during concreting/grouting works, something overcome by installing two temporary valves within the aluminium screen to direct/pipe incoming waters around the working area.

The primary gate was also manufactured in Spain by Orbinox and transported to site in one piece, weighing 8.2 tonnes. The leakage around the bulkhead scour gate was now established and settled, and constant monitoring established the leakage rate at 6l/s, which was deemed acceptable for works to commence. In a similar construction methodology to the secondary gate, following gate removal from the supporting frame, remote operated hydro-demolition techniques were used to remove the concrete surrounding the frame. Constant monitoring was in place to identify any seepage through the structure; similarly to the primary gate, none was noted during the works. Following on from hydro-demolition the new gate was lowered into position and again welded shear connections were attached to the existing reinforcement prior to final grouting.

CHANNEL PROFILING

Upon installation of the two new gates, flow profiling was installed using reinforced concrete to assist in preventing cavitation from occurring as flow passes at high velocity (Figure 11). The reinforcement for the flow profiling was anchored into the existing structure utilising around 1000 steel dowels, each of which required a 30mm hole to be drilled 300mm deep. Remote operated plant was utilised to carry out the drilling activities.

Once the reinforcement and formwork were in place, concrete was transported from the quay to the tower in a concrete skip and then lowered to the point of use by the on-site gantry crane. The concrete specified was a high strength concrete specified to help minimise the impact of abrasion during scour operation. Although not a large concrete pour in terms of volume, the development of the mix with the supply chain, contractor, designer and QCE was key to being able to balance the functional requirements along with the practical constructability. The concrete works had to be planned and executed well, which took into account weather and transport times to enable the correct workability at the time of placement.

As part of the works to upgrade the scour system, a new motor control centre (MCC) with associated power and control cabling was installed between the valves and the operations room on site. The new MCC enables remote operation of all the gates and draw off valves, which allows for safe operation without the need to manually stand next to valves during operation.

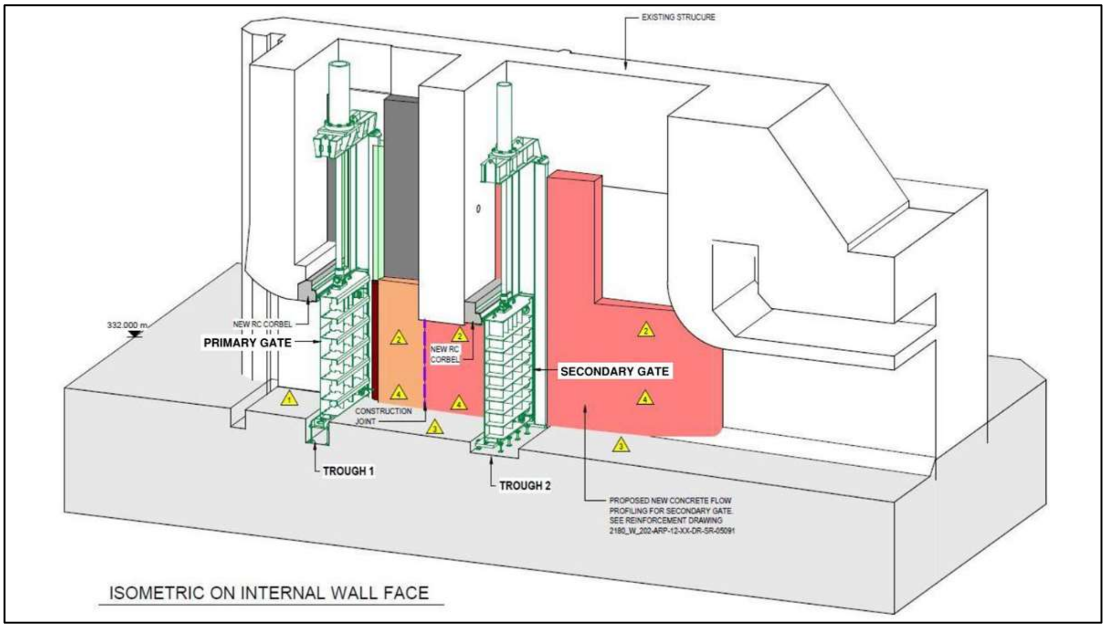

Figure 11. Chanel profiling (ARUP)

CONCLUSIONS

All valves were successfully installed, commissioned and the interests of safety recommendation certified ahead of the regulatory date. Testing of the new gates and valves is undertaken against full reservoir head on a six-month rolling programme.

The improved scour system offers greater control and safer operation both for frequent tasks and infrequent or emergency situations. The redundancy provided ensures minimum risks to the downstream catchment in the event of an emergency discharge. The high specification of all new equipment along with the facilitation of safer maintenance and testing procedures will ensure the systems reliability for many years to come.

Works to the main draw-off system, power supplies and control system also improve the everyday operations of the dam and reduce the frequency of confined spaces access.

ACKNOWLEDGEMENTS

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The British Dam Society

Managing risks associated with the infilling of the adit at Tunstall Reservoir

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SYNOPSIS Significant leakage was observed on the left flank of Tunstall Dam upon first filling of the reservoir. A concrete-filled cut-off wall was extended into the abutment in 1879, which significantly reduced the flow but did not resolve the problem. In a further attempt to manage leakage, the adit and shaft that had been used to form the cut-off wall was repurposed and extended with additional drifts cut into the hillside to capture and transfer leakage flow.

Several measures in the interests of Safety (MIOS) arose out of a Section 10 inspection in 2021, one of which recommended *"fill the tunnel (i.e. adit) whilst providing some drainage"* in response to a concern about its structural integrity and the potential risk to the embankment dam and spillway in the event of a collapse. The remedy comprised filling the adit passing beneath the dam and appurtenant works with expanding geopolymer introduced via a series of injection holes drilled from the surface. Due to the depth, there was a significant risk that the drillholes would miss their target and that drilling might damage the existing 18-inch diameter cast iron pipe conveying leakage flows beyond the dam structure.

This paper describes how risks were managed and mitigated, the key aspects of the investigations and design process, and the works that took place to satisfy this MIOS measure.

OVERVIEW

Description of the site

Tunstall Reservoir is located 5km to the north of Wolsingham in Durham. It previously fed a treatment works below the dam, which has since been decommissioned. The reservoir is now used for river flow compensation and amenity purposes. It is impounded behind an earthfill embankment with a central puddle clay core. The dam is about 25m high and 300m long.

Dam construction began in 1873. Significant leakage was observed through the left abutment during first filling. A 1.8m wide, 27m deep concrete cut-off wall was extended some 82m into the abutment in 1879 and it significantly reduced, but did not eliminate, the leakage. In addition, a wedge of open jointed rock between the end of the existing puddle clay-filled trench and the concrete-filled extension was carefully removed and replaced with brickwork, with cement grouting upstream to reduce the leakage passing beneath the brickwork.

The formation of the brickwork wedge necessitated the driving of tunnels and shafts. The main drive from the dam toe was known as the Drift By-pass Tunnel, which henceforth will be referred to as *the adit*. The failure of the cut-off wall extensions to stem the leakage prompted further tunnelling to intercept flow and carry it away downstream via an 18-inch pipe laid along the adit.

Description of the adit

Some 120m of the 200m long adit lies below the footprint of the dam and spillway, with the rest lying within the hillside. Water passing through the left flank is intercepted by the tunnels and flows into the adit, where a low brick wall transfers the water into an 18-inch cast-iron pipe. Figure 1 illustrates the layout of the adit and associated features.

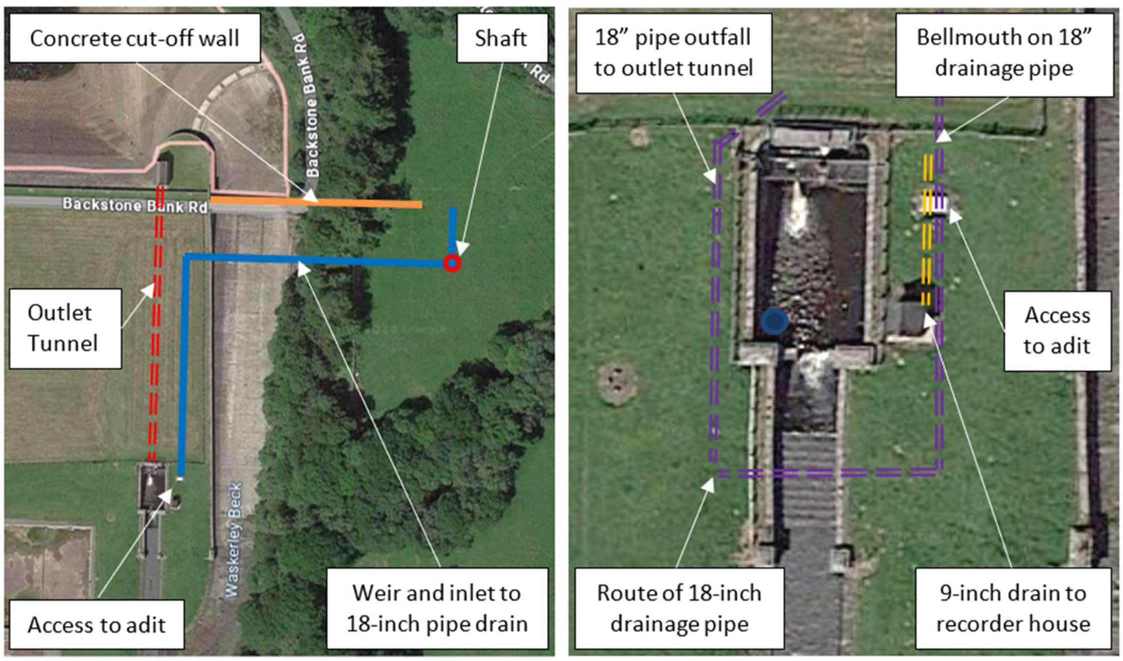

Figure 1. Extent of the adit and key features (Google Maps).

The initial 9m long brick arch section at the adit entrance is unlined and unsupported as far as the 90° bend, apart from occasional steel beams in the soffit (Figure 3). The section between the bend and the inlet to the 18-inch diameter drainage pipe is also brick lined, but thereafter the tunnel is unlined. The adit dimensions vary but are generally about 1.7m high by 1.4m wide. The maximum cover to ground level below the dam is 16m but increases up to 40m as the adit extends into the hillside. A forced air ventilation system (Figure 3) was installed in 2001 but the Undertaker has restricted entry to essential works since 2016.

The 18-inch diameter adit pipe conveyed leakage and groundwater to the treatment works but was also provided with an outfall to the outlet tunnel. A weir near the tunnel portal enables that flow to be measured. A bellmouth overflow on the pipe some 12m upstream of the downstream end of the adit allowed excess flow to discharge onto the adit floor and drain away via a 9-inch collector drain to the recorder house where the flows are measured over a V-notch weir before discharging into the outlet basin downstream of the outlet tunnel portal.

Figure 2. View of pipe inlet and weir. Note standing water on the invert of the adit.

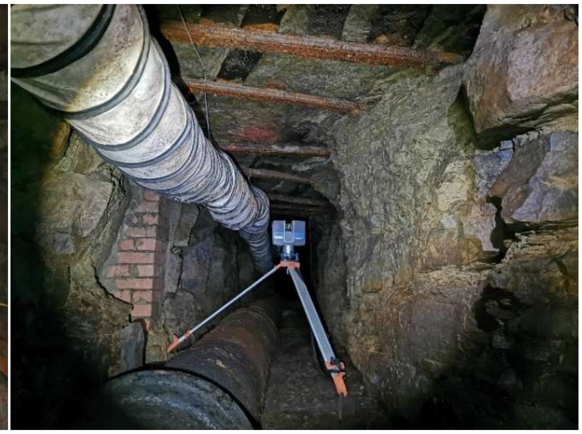

Figure 3. View in the unlined section of the adit. Notice the beams across the soffit.

Initial Surveys

A previous recommendation under Section 11 of the Act had been made to measure and record drainage/seepage flows against rainfall and reservoir level. Several surveys were undertaken in 2020/21 to inform the upcoming Section 10 inspection including:

- A photographic record of the adit condition (including Figures 2 and 3).
- A 3D laser scan and topographical survey to confirm location, extent, and dimensions.
- A magnetometric resistivity survey (by *Willowstick*) to identify leakage flow paths in the left abutment, which suggested that flow was also passing through the cut-off wall.

RECOMMENDATIONS IN THE INTEREST OF SAFETY

Dr Andrew Hughes carried out the Section 10 inspection in February 2021 and the above-mentioned surveys and monitoring data helped inform the Inspecting Engineer regarding the long-term condition of the adit, the potential for a future collapse and implications of a total blockage on the safety of the dam. Amongst other matters, the Inspecting Engineer recommended the following MIOS to be implemented by 23 February 2024:

iii: Works are carried out to stem the leakage over the core (cut-off wall) as identified by the Willowstick survey.

iv: Once the majority of flow into the shaft and tunnel is stemmed that the tunnel (i.e. adit) and shaft be filled with, say, foamed concrete whilst still providing some drainage.

Northumbrian Water Ltd (NWL), the asset owner, appointed Esh-Stantec initially to deliver a *Concept & Definition* (C&D) contract to assess options, identify a preferred option, and subsequently to deliver the approved solution under the *Design & Construction* (D&C) phase of the project. Work commenced in July 2023 with Esh-Stantec as both Principal Designer and Principal Contractor and with Ian Carter acting as Qualified Civil Engineer (QCE).

SCOPE OF WORKS

MIOS 3 – Stemming the leakage over the core

A ground investigation (GI) was undertaken in late 2022 to investigate the cut-off wall condition and identified flow paths. Details of that study lie beyond the scope of this paper.

The findings from the GI, together with a review of reservoir levels / flows in both the 18-inch pipe and 9-inch collector drain in the adit, were presented to the QCE. No evidence of flow through the cut-off wall was found. The physical evidence confirmed the concrete-filled cut-off wall extension was in good condition, and that leakage generally passes around this wall, rather than through it. Clay was found directly above the concrete beneath the hillside and this material was used to seal the uppermost part of the access tunnel once concreting had been completed.

Figure 4 shows the leakage entering the adit predominantly through the shaft near its eastern end. Flows in the adit are closely related to reservoir water level. They decrease significantly when the water level falls 6m below the top water level (TWL). Figure 5 shows the correlation between reservoir levels and recorded flows in both the 18-inch drainage pipe and the 9-inch perforated collector drain, collecting flows from the bellmouth overflow.

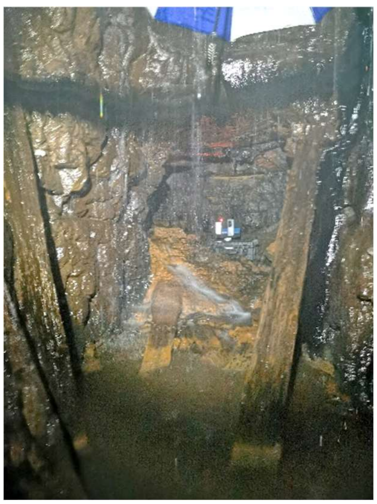

Figure 4. View of flow cascading down the shaft.

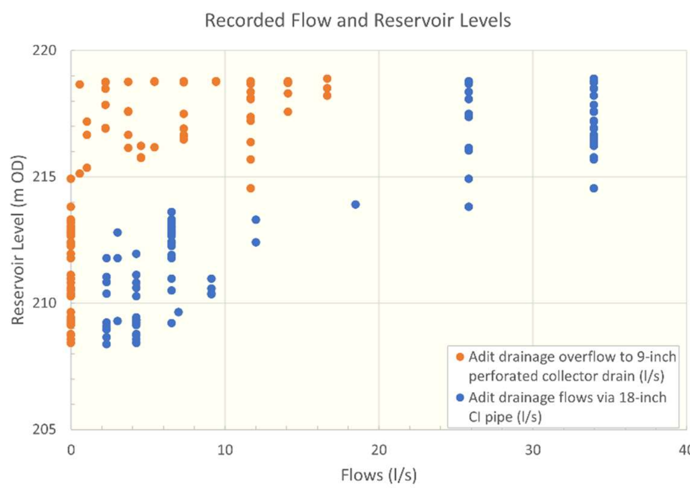

Figure 5. Relationship between reservoir level and flows recorded in both pipes systems.

Regular readings of flows both in the 18-inch and 9-inch pipes began in October 2019. Prior to the works, the maximum recorded leakage flow was 51 l/s with the reservoir water level standing at TWL for a prolonged spell. Leakage flows have historically dropped to 2 l/s with the water level 10m below TWL. It was therefore decided that flow into the adit should be stemmed during the construction phase by lowering the reservoir level and maintaining it around 6m below TWL. In view of the nature of the cut-off wall and its very good condition, no additional works were required to improve its watertightness.

MIOS 4 – Adit infill works

Concept Stage

Various options were considered during the *Concept* phase to address the Inspecting Engineer's requirements. These considered the extent of the adit to be filled, the different materials that could be used to achieve this purpose, and how they might be implemented.

Filling the entire extent of the adit was considered but was not the preferred solution due to the significant constraints associated with that option. These constraints included difficult access, required confined space activities, land ownership issues, the presence of a SSSI, and an ever-increasing depth of cover above the adit, amongst other things. It was decided that adit filling beneath the dam and spillway would be sufficient to eliminate the risk to the reservoir.

Three possible infill materials were considered:

a) Cement grout was discounted due to potential pollution concerns, given that the material might mix with groundwater and leak out through the bedrock, causing an incident.

b) Foamed concrete was discounted because of the adit length and in consideration of concerns linked to the remote location and timely delivery of concrete.

c) Expanding geopolymer foam was selected as the preferred material. It had been used successfully elsewhere and was easier to place, more economic and had a lower carbon footprint than the other options.

Definition and Detailed Design Stage

The agreed concept solution was developed during the *Definition* phase and detailed during the *Design & Build* stage of the project. Additional surveys of the adit and both drainage pipes were carried out at the start of the construction phase, and the information fed back into the final scope of works, which is summarised below:

- Part filling of the adit, i.e. the 120m section of adit below the dam and spillway, since a collapse of the adit in the abutment would not compromise the dam safety.
- Relocation of the 18-inch overflow bellmouth to facilitate future maintenance.
- The brick lined section immediately upstream of the entry point would remain unfilled, so as to retain future access to the 9-inch perforated pipe.
- The infill to be expanding geopolymer foam with a minimum compressive strength of 100kPa. *Geobear Ltd* to be appointed as the specialist supplier and subcontractor.
- Foam to be injected into the adit via injection points drilled from the ground surface and spaced at 10m intervals. The injection holes would be lined with plastic casing, which would be left in place.

Risks associated with undertaking of the works to satisfy the solution agreed with the QCE can be broadly divided into two categories, *Dam Safety risks* and *Health & Safety risks.* The next sections of this paper set out these risks, mitigations put in place, and remaining residual risks.

RECOGNITION OF RISKS

For the development of the concept design and supporting risk register the project team referred to the information provided by previous surveys. Undertaking additional surveys in the adit was only deemed possible during the design and construct phase, once the water level was drawn down 6m below TWL, and an inspection had been completed by *MRS Training & Rescue* using a team trained and equipped to enter similar unknown and uncontrolled environments.

Managing Risks for Dams and Reservoirs

The dam safety risks associated with the concept design were therefore based on several assumptions, which were verified as the works progressed. Table 1 summarises the dam safety risks, the assumptions made and the identified mitigation.

Table 1. Dam Safety Risks, Assumptions & Mitigation

Ref	Risk	Assumption	Mitigation	Residual Risk
1	High water levels in the reservoir.	• Water levels in the adit pipe are directly related to levels in the reservoir.	• Established flow/water level relationship. • Management plan for water level in place. • Regular coordination meetings.	• Storms during the works. • Inability to control water level.
2	Drilling of injection holes miss the adit.	• Previous survey coordinates are sufficiently accurate.	• Setting out data verified on site. • Injection coordinates provided to adit centre based on survey. • Slope climbing rig to be used for drilling. • Drilling to start from downstream end where cover is less. • Drilling programme extended to allow for “misses”.	• Multiple attempts to find adit with the drilling of the injection holes. • Programme extended. • Higher project cost.
3	Geopolymer enters and fills drainage pipe.	• 18-inch drainage adit pipe in good condition with no major cracks or holes.	• Formwork at downstream end to prevent expanding geopolymer from blocking the pipes. • Overflow bellmouth capped and replicated in an alternative location. • Surveys undertaken to identify potential defects in the pipe. • Flow monitored in the pipe to check for change in flow regime during geopolymer filling.	• Surveys miss identifying potential geopolymer entry points into the drainage pipes.
4	Drilling damages pipe inside adit.	• CI pipe can tolerate small impacts. • Adit would remain stable during the drilling.	• Pre-drilling entry to visually inspect the pipe. • Post-drilling surveys using drones and high- definition cameras. • CCTV surveys of the pipework attempted.	• Damage missed by pre-injection survey.

Ref	Risk	Assumption	Mitigation	Residual Risk
5	Waxcap fungi constrain surface works.	• Waxcap mitigation strategy in place before site works.	• Ecological mitigation to mitigate any impact on waxcap habitat. • Track mats provided to minimise topsoil damage by the drilling rig.	• Drilling Programme extends into Waxcap season.
6	Spacing between injection points insufficient.	• Anticipated performance and behaviour of chosen geopolymer	• Liaison with specialist subcontractor to confirm required spacing. • Polymer injection and expansion rate verified at start of site works. • Videos and photos taken to verify completeness of the injection works.	• Additional drilling required during the project.
7	Potential pollution.	• Polymer Geopolymer material is low viscosity (does not flow freely through open jointed rock).	• Fill material is non-hazardous and expands quickly, minimising loss through jointed bedrock • Watching brief for signs of geopolymer in drainage flows.	• Pollution to watercourse.
8	Drone survey unfeasible beyond the 90° bend	• Signal unlikely to travel beyond bend	• Antenna inserted through injection point to provide signal beyond the bend.	• Drone fails during survey.
9	Selected fill material unsuitable	N/A	• Minimum shear strength specified for expanding geopolymer. • Samples taken on site for Q&A testing.	• Safety of dam embankment compromised.
10	Collapse of the drillholes	N/A	• Holes cased	N/A

RECOGNITION OF HEALTH & SAFETY RISKS

Designers under the Construction (Design and Management) Regulations 2015 are required to apply the general principles of prevention in preparing their design to minimise risks to health, safety and well-being during the construction, operation and demolition phases of a project or asset. For this reason, the proposed solution aimed to minimise time working within the adit given the risks associated with working in this type of confined space. Expanding geopolymer was chosen as the preferred material as it helps mitigate the risks. Table 2 summarises the risks identified in the Hazard Identification Checklist (HIC) and Significant Risk Log (SRL) developed during the project.

Table 2. Significant Risks to health, safety and wellbeing, Mitigation and Residual Risks

Ref	Hazard	Activity affected by risk	Mitigation	Residual Risks
1	Live Reservoir	• Person-entry inside the adit. • Geopolymer injection	• Water management plan in place to reduce leakage flows into the adit.	• Storms during the works. • Management of the water level.
2a	Confined space / low oxygen	• Inspections in the adit. • Installing formwork at upstream and downstream ends of the section to be infilled.	• Geopolymer injection from surface to avoid man entry. • Pea gravel stop end provided at upstream end installed from ground level to avoid formwork upstream. • Ventilation pipe present but uncertain efficiency. • A single >10m entry went beyond the lined section of the adit. • Specialist sub-contractor appointed to carry out person-entry survey. • No person-entry post hole drilling permitted.	• Effectiveness of filling operation was only visible by remote monitoring means.
2b	Confined space / unknown structural condition			
3	Steep slope	• Drilling of injection points	• Slope climbing rig used for drilling, using appropriate anchors. • Temporary platform created on the spillway to create level surface. • Water Management Plan for dry access to spillway	• Working on a steep slope

MANAGEMENT AND MITIGATION OF THE HEALTH AND SAFETY RISKS

The site works started towards the end of August 2023. Establishing and actioning the water management regime was the priority action for the project team. As noted in Table 1, a reservoir level of TWL-6m was required for the duration of the works, and it was perceived that this might eliminate inspection of the adit during the Design and Build stage to a single person-entry exercise to establish the present-day condition of the adit and its pipework.

Furthermore, it was essential that the spillway did not operate while the injection holes in the spillway were drilled, and that water did not spill down them into the adit while the injection holes were open. Also, minimisation of “excess water” in the adit was desirable to ensure the that the geopolymer resin reaction was uninhibited and successful.

The water level management plan was agreed between contractor and client, who remained responsible for the management of the reservoir throughout the works. Roles and responsibilities were set out, as well as contingency plans to be implemented if control of the water level was lost, or under threat. The plan was tested on several occasions in late 2023 due to 'named' storms, at which time mitigating action was required.

Following the single entry, a Design Safety Review (DSR) took place, and the project team was challenged to amend the design to eliminate the need for any further confined space entry and work. The concept design had allowed for the installation of a plywood shutter close to the inlet of the 18-inch diameter pipe, to prevent the geopolymer from entering and blocking it. It also made provision for pipe protection at the injection points to minimise the risk of pipe damage. This proposal was also re-assessed and discounted given the visual observation that the pipe was in a good condition with walls about one inch thick.

A practical and acceptable alternative was found: the injection points were re-positioned, and the two points closest to the internal adit weir were re-purposed. One point was used to convey stone to create a pea gravel barrier, while the adjacent point became an observation point. Pea gravel was introduced via a tremie pipe to form a mound within the adit to stop the advance of the expanding geopolymer upstream (Figure 6). A basic CCTV camera was introduced into the inspection point to monitor the progress of the geopolymer foam.

It was critical that the drainage pipe remained free-draining and unobstructed. Given the limited quality offered by basic cameras, the project team monitored the foam advance using high resolution cameras with improved lighting. The improved imagery confirmed that there was a gap between the pea gravel mound and the soffit which would have allowed the foam to overtop the barrier. Fortunately, the high-resolution cameras allowed the advancing front of polymer foam to be closely monitored, and infilling to be stopped before it reached the window in the barrier, as illustrated in Figure 7.

Figure 6. View of the pea gravel mound.

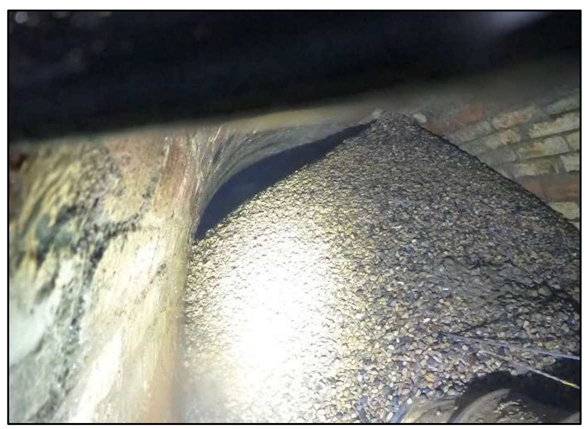

Figure 7. Post injection view of the mound with foam just visible beyond the gap.

Entry to the most upstream portion of the adit was eliminated by the creation of the pea gravel stop end, however strict adherence to the no entry policy did mean that the opportunity to check for hitherto unforeseen defects was lost, as well as the opportunity to mitigate the associated financial impact.

A few entries were required at the downstream end to relocate the bellmouth overflow, but these activities were deemed to be lower risk, due to the proximity to the entry point and the brick-lining in that section. The activities were nevertheless supported by forced air ventilation and confined space rescue teams.

MANAGEMENT AND MITIGATION OF DAM SAFETY RISKS DURING THE SITE WORKS

The drilling of the twelve injection and verification holes took place after the initial person-entry survey had taken place. Initially, the preferred position for the holes was thought to be close to the adit sidewall and away from the drainage pipe. However, given the risk that the drill holes would deviate off-line, either due to drill set-up, ground conditions or survey error (arising from transfer of control below ground), the location was changed to the centreline to maximise the chances of hitting the target.

Drilling commenced from the downstream end of the adit, where the cover was smaller, and the first six holes were drilled successfully. However, as the holes became deeper, the risk of missing the adit increased, and the injection hole at the 90° bend had to be attempted three times before it could be successfully completed. The inspection point at the far end broke through close to the side wall and the pea gravel injection hole was off centre, allowing a window in the mound, as can be seen in Figures 6 and 7.

The injection of the geopolymer only began once the drilling was completed and the QCE was satisfied that there was an effective strategy in place to prevent the polymer foam from extending beyond the pea gravel mound and entering the upstream end of the 18-inch pipe. The risk associated with that outcome was considered high due to the likely difficulty of removing hardened geopolymer from the pipe and the possible build-up of water pressure elsewhere. The pre-injection condition survey of the pipe was intended to mitigate this risk and identify any potential points of ingress. However, as it was carried out by confined spaced specialists, the survey missed some of the details that would have been captured by well-trained, professional reservoir engineers. The need for a better survey specification and more effective communication with survey specialists was a lesson learnt by the design team.

Given the reticence of the Contractor, Designer and Undertaker to authorise a further confined space entry into the adit, the QCE sought evidence that drilling operations had not damaged the pipe. Drone surveys were attempted but met with mixed success due to the limited space and obstructions therein, not to mention signal communication in the underground environment. However, insofar as could be determined visually, the survey confirmed that the downstream section of the pipe was in good condition with no obvious defects. In addition, high-resolution cameras were inserted at each injection point to inspect the pipe at spot locations. Figures 8 and 9 show examples of pre- and post-drilling surveys.

Adit infilling progressed at about 10m/day. Given its nature, once injection of the expanding geopolymer starts, then it needs to continue until the next injection point is reached. A downhole camera was used to confirm the position of the geopolymer foam front and to stop the injection before blocking the next injection point. Quality assurance was made more challenging by the steam generated by the geopolymer expansion process and the poor light. The expertise of the subcontractor in this matter was key to a successful outcome. Figure 10 shows a view of the geopolymer advancing through the adit.

Figure 8. Adit view before start of drilling.

Figure 9. Minor roof fall after drilling of hole.

Figure 10. Expanding geopolymer foam in the adit

A watching brief was put in place for potential signs of resin ingress into the 18-inch drainage pipe. The injection process progressed steadily from the downstream end, but traces of resin were observed in the drainage pipe at the 90° bend and the operation was stopped while an investigation was carried out.

Upon review, the most likely explanation was that the geopolymer foam entered via a gap between the chamber at the bend and its cover slab. The foam mixed with water therein and was carried along the drainage pipe before it had the opportunity to expand and cure. If the injection point had been slightly further away from the chamber, or had the adit been drier at the time of injection, then this might have been avoided.

It seems likely that the failed attempts to drill the injection hole at this location probably contributed to the damage to that chamber. Either way, a post-drilling inspection by a trained reservoir professional would almost certainly have spotted the defect and raised concerns. While remote inspection technology has advanced in leaps and bounds in recent years, there are still occasions where it has not outpaced the "Mark 1 Eyeball".

Geopolymer injection was suspended temporarily in response to the incident. Fortunately, the material properties of foam are such that removal by high-pressure jetting is possible and far less problematic than the alternatives considered at Concept stage (i.e. grout or foamed concrete). Insofar as was possible, the hardened foam was cleared from the drainage pipe. Some foam remains beyond the bend, but it does not appear to be restricting flow.

The injection sequence was modified following the stoppage because injection terminated before reaching the target injection point. Injection recommenced once an alternative approach had been agreed with the QCE. Injection resumed from the last injection point to maximise the chance of a successful outcome at the pea gravel mound, with the gap between the two geopolymer foam fronts being filled in the last phase. There was no opportunity to obtain visual confirmation that the foam filled the entirety of that void, but the increase in backpressure at the injection lance suggests that there is no residual void at that location.

CONCLUSION

The location of the adit at Tunstall reservoir and the nature of the required remedial works presented several challenges to the project team (including Client, Designer, Contractor, and Sub-contractors) beyond the normal management of risks to dam safety.

The priority of the project team was to complete the required works without compromising health and safety and to minimise confined space entry into the adit. The project objectives were achieved by risk management throughout the various stages of the project via a combination of:

a) early identification of risks and appropriate mitigation measures,

b) effective collaboration between the Client’s operations and capital delivery teams,

c) use of technology to allow remote inspection of the adit and the filling process, and,

d) selection of appropriate fill material and installation techniques.

Without doubt, health and safety risks were managed effectively and no near misses or incidents were registered on site during the adit infill works. However, rigid adherence to the no-entry policy did result in missed opportunities to detect and avoid problems, which ultimately introduced significant additional cost to the project.

The British Dam Society

Valve Tower GRP Lining - Llyn y Fan Fach Refurbishment

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SYNOPSIS A Section 10 inspection on Llyn y Fan Fach dam led to MITIOS, one of which was to arrest the structural cracking and eliminate leakage into the draw-off shaft. This paper covers the investigation that was done into the causes of cracking, including finite element modelling; the different options that were looked in to, such as decommissioning, demolition and rebuild, less intrusive repairs and structural lining, and why the chosen solution was lining the tower with a structural FRP liner and its design. The paper then looks at the construction, including procurement, delivery, installation and how it functions in place.

INTRODUCTION

Llyn y Fan Fach is a reservoir at the head of the Afon Sawdde, located within the Bannau Brycheiniog National Park (Figure 1). Constructed between 1914 and 1919 by conscientious objectors it raised the original lake by approximately 3m. The spillway for the reservoir is via a 6.1m ogee weir in the centre of the main dam. This discharges into a masonry and concrete stilling basin and spillway channel before flowing freely into the Afon Sawdde. The reservoir was previously used to supply water to a treatment works downstream, but is now used to provide compensation flows to a fish farm that was created at the same location as the treatment works.

Figure 1. Llyn y Fan Fach Reservoir

Managing Risks for Dams and Reservoirs

Alongside routine maintenance, a Section 10 Inspection under the Reservoirs Act 1975 was undertaken on 15th October 2019. The inspection report stipulated the following Measures in the Interests of Safety (MITIOS):

- To arrest the structural cracking and eliminate leakage into the draw-off shaft. This shall be in the form of a new structural liner to the valve shaft and sealing of the existing structure.
- The spillway apron and vehicle crossing shall be reconstructed to safely convey the safety check flood.

The MITIOS also acknowledged that the dam could also be discontinued.

Key participants

- Client: Dŵr Cymru Welsh Water (DCWW)
- Main contractor: Morgan Sindall Infrastructure (MS)
- Consultant designer: Arup
- Mechanical contractor: Whitland Engineering Ltd
- GRP liner: iLine Technologies Ltd
- Precast concrete: FLI Precast Solutions Ltd
- Formwork: Cordek Ltd (using Filcor 90)
- Shuttering support: PERI UK
- Concrete repair: Beton Bauen Ltd
- Underwater survey/repairs: Edwards Diving Services
- Over pumping: Pump Supplies

FEASIBILITY DESIGN

As the Project progressed through feasibility several options were considered, one of which was the discontinuation of the reservoir and restoring the area back to its original state.

It became apparent that there were several constraints associated with discontinuance. Although the reservoir is not in public supply, there is an agreement in place to provide water to private stakeholders downstream. Separately DCWW was notified that CADW had placed an interim Grade II listing on the dam structure and other curtilage structures. Interim designation was issued in February 2021, with full Grade II listed status designated in February 2022.

Following the feasibility assessments, DCWW's collaborative Risk and Value Process was followed which showed the preferred option to be the refurbishment of the dam, which included works to the draw-off shaft and spillway.

TOWER REFURBISHMENT

Investigation into cause of valve tower cracking

An initial investigation was undertaken to determine the most likely driver of the cracking so that a suitable and effective solution could be implemented that would stem any future leakage and deterioration of the structure.

There are records of the cracks that go back to 1965. These were repaired in 1980, and a report in 2010 noted that three dominant cracks were suffering leakage. An inspection report in 2019 speculated that freeze-thaw action and temperature variations led to the cracking of the structure. A 3D scan of the valve tower interior was undertaken in 2021, which was used to inspect the location and extents of the cracking (Figure 2)

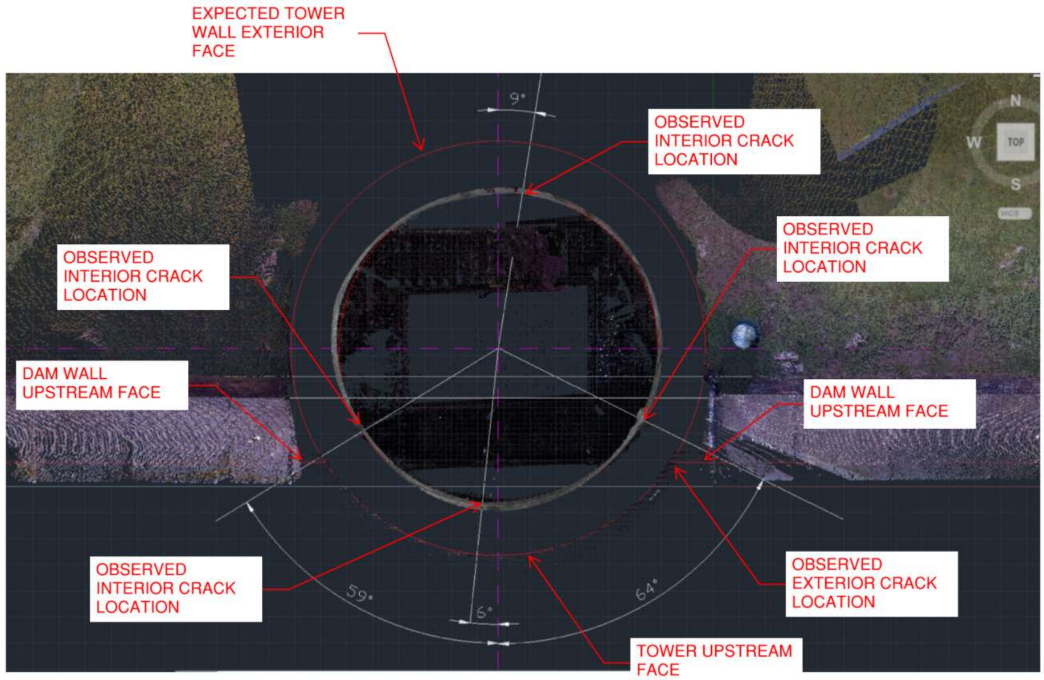

Figure 2. Observed locations of leaking cracks in valve tower wall

Assessing the likely causes of cracking in the assumed unreinforced gravity structure led the team to hypothesise that cracking has originated due to volumetric changes in the adjacent dam walls exerting pressure on the valve tower.

A finite element model of the structure was built to provide supporting evidence for this hypothesis. Volumetric change was considered in the longitudinal axis of the dam wall only, and the investigation acknowledged there were limitations to the analysis. These limitations were acceptable as the analysis was only used to demonstrate locations in which tension stresses were being developed. The structure was first analysed with 1D elements, based on the results the zones in Figure 3 should be presenting cracking:

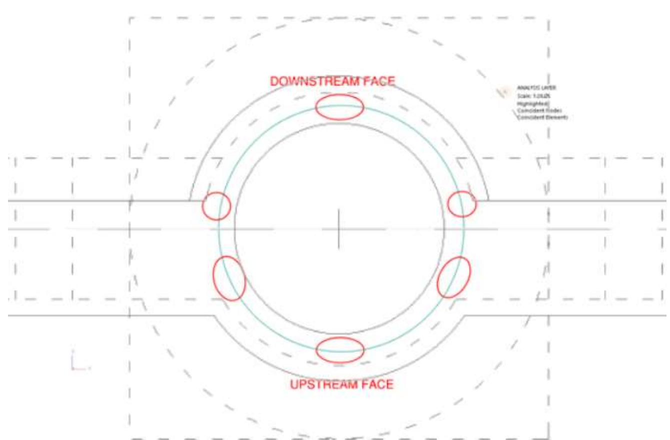

Figure 3. Crack locations predicted by 1D analysis

For further investigation, a 2D analysis was undertaken in GSA and similar results were found but show more detail on the location and development of the stresses (Figure 4).

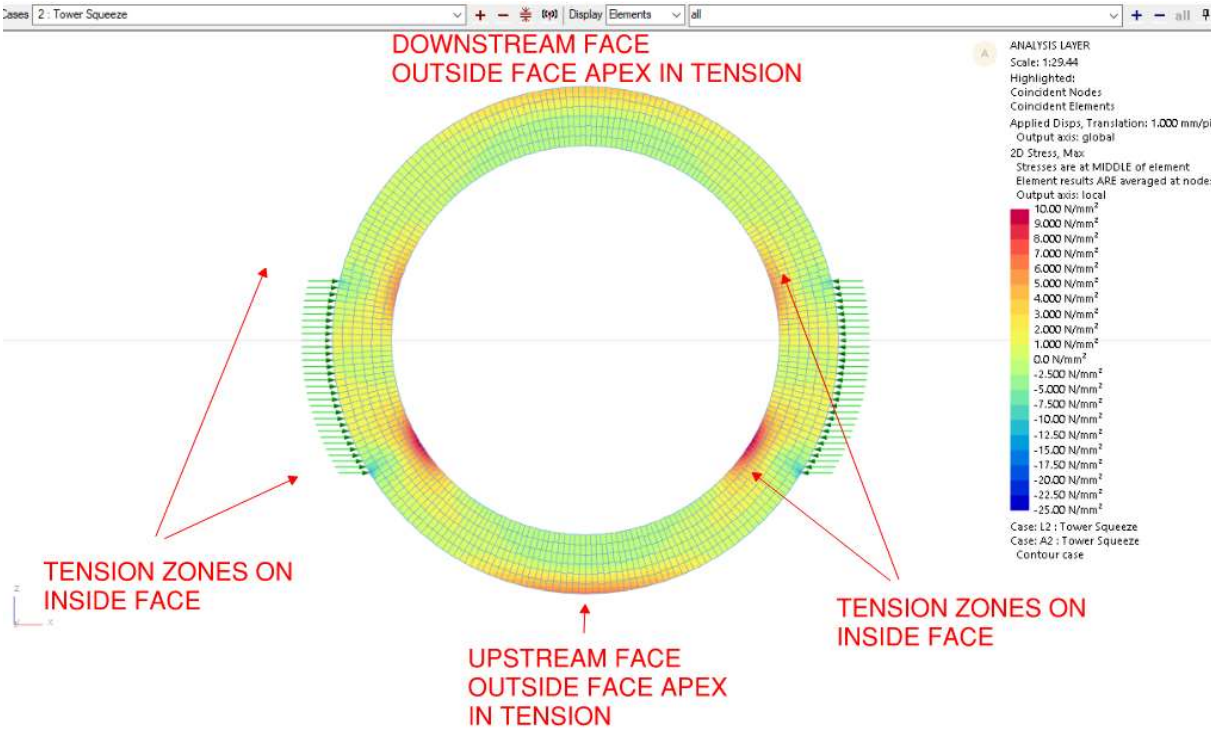

Figure 4. 2D Stress results for dam wall expansion

Comparison with the results of the model and the point cloud data concluded that there was significant evidence that volumetric change of the dam walls that connect to the valve tower was inducing tension stresses in the shaft walls causing cracking. Tension areas predicted by models of this action correlated well with the observed real-world cracking.

It is also possible the cracking was contributed to via differential lateral movement of the dam wall and valve tower under the cyclic operational loading as a secondary driver.

The acceleration of leaking in the most recent years may be due to exacerbating factors associated with the cracks fully penetrating the section, such as the water inflow washing out fines, freeze-thaw action occurring in and around the newly exposed faces of the cracks, and debris ratcheting the open cracks preventing them from closing.

A solution was then developed to mitigate the leaking and protect the structure against further deterioration.

Proposed Solution

GRP Liner

A full height liner made from a designed GRP material. A void was left between the GRP liner and the existing tower to allow for installation tolerances and this void was filled with a cementitious grout. The liner was a full annulus piece with no penetrations that would allow water ingress. Fixings were built into the liner to accept connections from the internal access staging that is required in the permanent case. The GRP liner was designed to withstand the full lateral pressures. GRP has some inherent flexibility that allows the dam wall to continue to expand and contract, with the liner designed for the expected movements.

External Concrete Facing

A reinforced concrete structure on the external face of the valve tower. A post-fixed anchor system was used to tie it back to the existing dam wall on either side of the valve tower. The concrete was not bonded to the existing concrete but separated by way of a membrane. A movement joint was provided at the apex of the arch so that stresses due to thermal movement of the dam wall are not transferred into the new structure. This is important as preventing the thermal movement from happening is not feasible, so instead mechanisms for this to occur without causing damage were provided.

The following are the possible failure mechanisms which were addressed by the design:

- Lateral Hydrostatic Pressures – to be resisted by the GRP liner, replacing the structural requirement of the existing concrete to resist hydrostatic pressures from the reservoir. For robustness the external concrete casing was also designed to resist hydrostatic pressures independently of the GRP structure.
- Lateral Embankment Pressures - existing downstream arch continues to perform structurally.
- Thermal Actions – existing cracks continue to act as movement joints eliminating stress build-up due to thermal actions in the existing structure. The GRP liner designed to deform with the tower elastically. The external concrete casing structure has a movement joint to alleviate these stresses.
- Environmental Attack - cracks raked out and sealed with a flexible sealant to allow the existing concrete to move under thermal strains and keep the cracks free from obstructions that could lock the joint and cause ratcheting actions to widen the crack. The new concrete casing insulates the cracked areas from further environmental deterioration.
- Upstream Arch Stability - instability caused by actions external to the valve tower, e.g. hydrostatic or wind loads, are transferred to the GRP liner. The structural cases that the existing cracked concrete used to perform have been replaced by the new structures and the existing concrete no longer has any structural purpose. To prevent the cracked concrete from instability it is restrained on either side by the new GRP liner and the new external concrete casing.
- Seismic Loads - the external concrete casing structure and GRP liner was designed to resist loads due to seismic actions. The seismic loads were analysed in a pseudo-static manner. The loading on the two structures was due to a ground acceleration identified as PGA = 0.15g acting on the loose cracked concrete mass, self-weights, and retained reservoir water. The reservoir water level considered was top water level.
- Accidental Construction Loads - the contractor managed stability during construction.

A robust approach to the design of this external structure was adopted where both the GRP liner and the external structure were designed ignoring favourable effects from one another, i.e. both structures were designed to resist hydrostatic, hydrodynamic and seismic acceleration loads (Figure 5). In the case of the GRP liner, these loads were defined and communicated to the GRP supplier for design, which was in turn checked by Arup.

In the case of the external casing structure, it was as a horizontal cantilever fixed back to the dam wall with post-fixed fasteners. It was designed to resist the cracked concrete from displacing during a seismic event in one direction and to resist hydrostatic and hydrodynamic forces in the other. The structure was modelled as a 2D shell in FEA software with the anchors as pin supports.

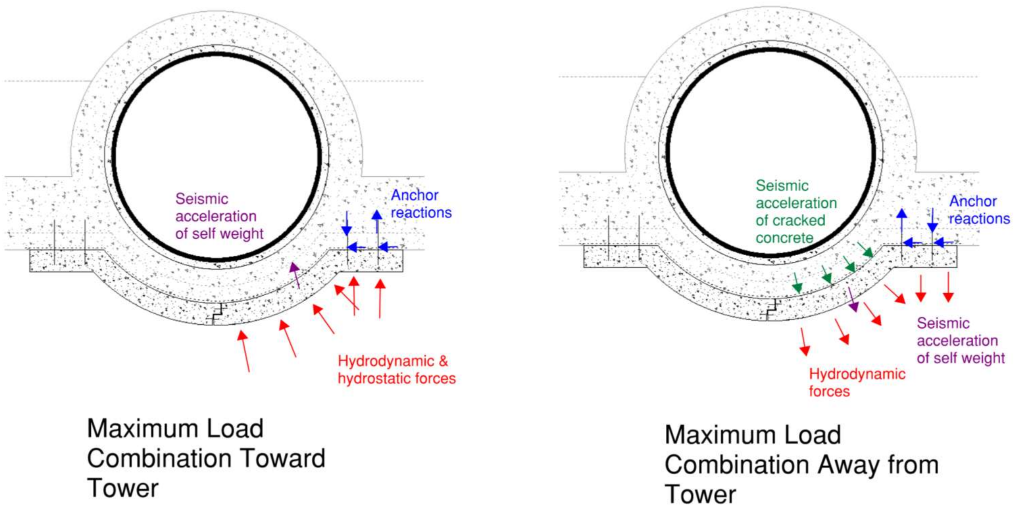

Figure 5. External structure load cases

OTHER WORKS

Spillway Replacement

The proposal was to fully replace the lower spillway with a precast structure capable of passing the 1,000yr (design) and 10,000yr (safety) flood events for the Category B dam. The wall heights were designed to contain the flows including full USBR freeboard. In order to maintain cultural heritage of the existing spillway, the spillway was lined with stonework.

Consenting – LBC, BNG, SSSI/SAC, Macrophyte

The dam is located within the National Park, a Site of Special Scientific Interest (SSSI), Special Area of Conservation (SAC) and is a Listed Building, therefore consenting was a significant constraint on the works.

Proactive interaction and timely submission of the Listed Building Consent (LBC) application minimised programme impacts. As part of this consent a biodiversity net gain (BNG) enhancement scheme was undertaken to help improve the local environment and offset the impact of the works, which saw the planting of over 200 new trees and the installation of a dipper box by the main watercourse.

Separately a SSSI assent was submitted to NRW to support the works and ensure acceptable management was undertaken. In addition to common protection and standard methods, a rare macrophyte was identified in the reservoir that was monitored and resurveyed post works to confirm the impact.

DELIVERY

It was clear that constructing this scheme was going to be challenging due to the site constraints, including remoteness, narrow access roads, SSSI areas, National Park status, listed structures, bridge weight limits and popularity of the location with members of the public. From the main site office to the working area there was a 240m altitude difference over the 2.4km access road.

Due to the site location, opportunities for off-site fabrications were pursued to minimise the challenge of getting fresh concrete to the working area. Due to the distance from the concrete batcher, narrow access roads and having to offload concrete for transportation to the dam, it would take nearly two hours to get concrete to the workface.

A significant constraint was the low clearance and low capacity bridges on approach to the site. This restricted the size of plant that could be transported to site and, in turn, the size of components that could be handled. Therefore, a large number of precast concrete units were required to make up the spillway structure with a considered and minimised use of in situ stitches. Another example where this constraint influenced delivery was that the tower required GRP sections with a 2.7m internal diameter being transported over a 2.4m wide bridge. The rings were therefore formed in half sections and assembled in-situ (Figure 6).

Figure 6. Installation of GRP Lining

The use of prefabricated products gave the additional advantage of reducing site risks from concrete washout, excess materials, reduced potential quality issues, minimised working at height and reduced the amount of plant to be transported to the working area. This also helped to reduce the overall programme compared to if conventional methods been used, and had a positive effect on minimising the construction impact and carbon footprint on the project.

The site team utilised Filcour90 from Cordek expand polystyrene shuttering (Figure 7) to make the external repairs to the drawdown tower and dam face. This innovative shuttering solution used the output from 3D surveys to fabricate the intricate curvature of the existing tower and corbel detail and then mould the shuttering off site, allowing delivery to site in manageable sections. Given the location of the dam, with the poor weather conditions at times which the site team faced, traditional timber shuttering would not have been able to form the shape of the tower.

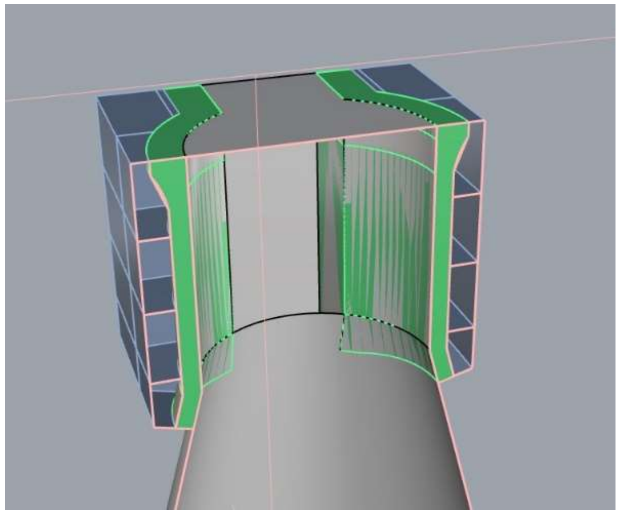

Figure 7. 3D external formwork

Demolition and reconstruction of the spillway was the most critical aspect of the project due to the inherent risks on the dam structure from scouring if it were to spill during construction (Figure 8). Water levels in the reservoir were lowered to provide a minimum storage capacity for a 1 in 150 year storm event as was stipulated by the Welsh Water Dam Safety Team and the QCE. To ensure that water levels were managed in the reservoir throughout the scheme, a temporary storm pumping system was established using 12" electric pumps. The pumps were located on floating pontoons on the reservoir to keep them off the reservoir bed to avoid silt issues. Due to the criticality of the works, a Dam Safety Construction Management Plan was developed to control any increased risk to the safety of the dam during construction. Duty - standby pumps and generators were installed with auto-changeover facility and telemetry systems that automatically called personnel in the event of an issue.

Figure 8. Spillway construction and lining

Under normal operating procedures the water from the reservoir was drawn off using a submerged siphon pipe which passed through the draw off tower, tunnel and down the mountain. The first valve on the siphon pipe, located in the tower, had to be replaced requiring our specialist diving contractor to install a temporary blanking plate within the reservoir to stop flows in the pipeline., An additional temporary over-pumping set up was installed to maintain the required flows down the mountain to supply the local fish farm. Isolating flows enabled the removal of the existing valve and replacement with two new units and pipes and subsequent re-commissioning of the siphon pipe. Remote operated vehicles were used to minimise manned diving and also to prove isolations before anyone entered the water. All diving works were closely managed with the site management and Client representatives by operating under "gold command" which provides detailed planning and scrutiny of all activities to always ensure the safety of personnel.

CONCLUSION

The use of the GRP liner has provided a robust, corrosion resistant solution to arrest the structural cracking of the tower, providing a flexible solution that will withstand both the varying seasonal conditions at the site and the extreme load-cases that it could be subjected to (Figure 9).

The remoteness of the site and access provisions at Llyn y Fan Fach were challenging, so the prefabricated GRP liner solution provided significant constructability advantages over traditional materials such as in-situ concrete. A similar approach could be considered for other applications in future schemes.

Figure 9. GRP Lining Installed

The scheme was completed in two phases from July 2022 until late December 2022 and from May 2023 to September 2023 and delivered successfully on time. Completion of the scheme has now provided DCWW with an asset that can be operated for many years to come and has enabled the statutory obligations under the Reservoirs Act 1975.to be closed off.

Managing Risks for Dams and Reservoirs

Throughout the scheme there were no reported accidents or injuries and a great safe working and positive intervention reporting culture was clear to see. 'How Are We Doing' feedback was obtained through the project from DCWW and received excellent comments.

The scheme has also now been signed off as achieving Perfect Delivery by Welsh Water which is a testament to the hard work and professionalism shown throughout all stages of the project.

The British Dam Society

Holistic photographic surveys and AI defect identification of the shaft and tunnels at Dinorwig Power Station

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A PRITCHETT, Engie
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SYNOPSIS In 2023, the high pressure shaft and tunnels at the Dinorwig hydro pumped storage scheme were fully drawn down for the first time since operations started on site. This presented owner/operator Engie with an opportunity to collect data about the condition of the concrete in the 10m diameter, 476m deep shaft, and several kilometres of large diameter tunnels feeding the power station.

Engie engaged CC Informatics to undertake the surveys. The project required the development of an imaging platform which could be attached to either an automated shaft inspection robot, or to a trolley within the tunnels. The project collected approximately 38,000 high resolution photographs, totalling almost 1 TB of data. These were subject to interrogation by CCI's patent pending AI, AssetScan, to look for cracks, surface loss, and previous patch repairs. The photographic and AI data was then presented to Engie and their engineers in large 2D drawing formats and databases.

The data was used to: compare and validate information from historical underwater remotely operated vehicle (ROV) data; create a baseline database of information to allow potential future change detection; and verify concrete core strength data. In the future the technology may be used to identify defects under internal pressure, identify feature dimensions other than area (width and length of defect), and assess permeability of the concrete liner.

DINORWIG POWER STATION

Dinorwig Power Station is a closed loop pumped storage power station located in Llanberis, North Wales. The station was constructed in the 1970s and was first commissioned in 1982. The station operates by balancing water volumes between reservoirs Marchlyn Mawr (upper reservoir) and Llyn Peris (lower reservoir). A schematic of the power station is shown in Figure 1. Since commissioning, the high pressure waterway system has not been fully drained down.

While underwater ROV inspections have been carried out periodically (MMT Services, 2021), clear close-up views of the concrete manifold and penstock liners had not been viewed since construction. Replacement of two of the main inlet valves (MIVs) required the system to be drained down for the first time in over 40 years (Stantec, 2021), creating a window in which to investigate the condition of the concrete liner with various techniques, with a view to verifying the remaining design life.

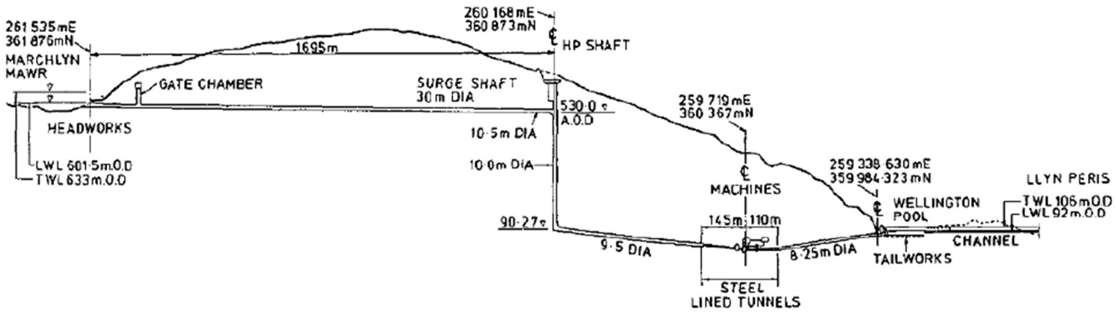

Figure 1. Dinorwig waterways layout (CEGB, 1980)

2023 System Drawdown

The high pressure hydraulic system was drawn down over approximately 12 days in May 2023. Following the drain down a series of condition assessment works were carried out including a photographic survey, concrete core extraction, concrete testing and analysis and in-situ stress testing. The works were carried out to provide parameters for geo-mechanical modelling, to identify any new or existing defects for repair or monitoring, and to provide a detailed condition assessment to outline the remaining design life for the structure.

Photographic Survey Requirements

The photographic survey was commissioned with the aim of creating both a photographic and vectored database of the high pressure shaft, manifold and penstocks to visually assess the current condition of the concrete liner and to create a baseline of data in which future surveys can be compared against to monitor for change.

PHOTOGRAPHIC SURVEYS

CCI was engaged by Engie to undertake the surveys of the insides of the shafts and tunnels at Dinorwig. It was agreed that AssetScan, a computer vision AI developed by CCI, would also be used to undertake an analysis of the position of cracks, spalling concrete, and past patch repairs for all surveyed surfaces.

Shaft Survey

This data collection survey focussed on the vertical shaft and was undertaken in May 2023. This made use of a specialist vehicle developed for the capture of photographic imagery in shafts and tunnels. In summary, the vehicle consists of a control platform and a camera platform incorporating a high gain radio system. This is shown in Figure 2.

The control platform used is a robotic control platform which uses large diameter, high power drone thrusters in a vector configuration to enable both rotational and translation control within the cross section of the shaft. Additionally the vehicle has a number of instruments that can be used in GPS denied environments: four laser scanners which can be used to determine the position within the cross section of the shaft, as well as magnetometers and gyros which were used to monitor approximate bearing and keep rotational speeds to a minimum, and keep the vehicle pointed in a single known direction.

The camera platform allows the mounting of seven SLR cameras – six in a circumferential orientation and one mounted axially. The cameras have been set to trigger using a common timer, which in combination with the target winch speed resulted in high resolution images being captured every 0.3m of depth. The camera head also incorporated high power LED lighting sufficient to illuminate surfaces some distance from the cameras.

The vehicle was powered by two lead acid batteries which were enclosed in sealed and reinforced housings. This was undertaken to minimise the risk of electrical fires caused by high electrical loads from the control platform.

The vehicle was lowered on a winched steel cable. Despite using anti-twist cable, the use of both a low resistance bearing at the mounting location and the powered control platform was necessary to keep the vehicle stable on descent. Each descent took approximately one hour.

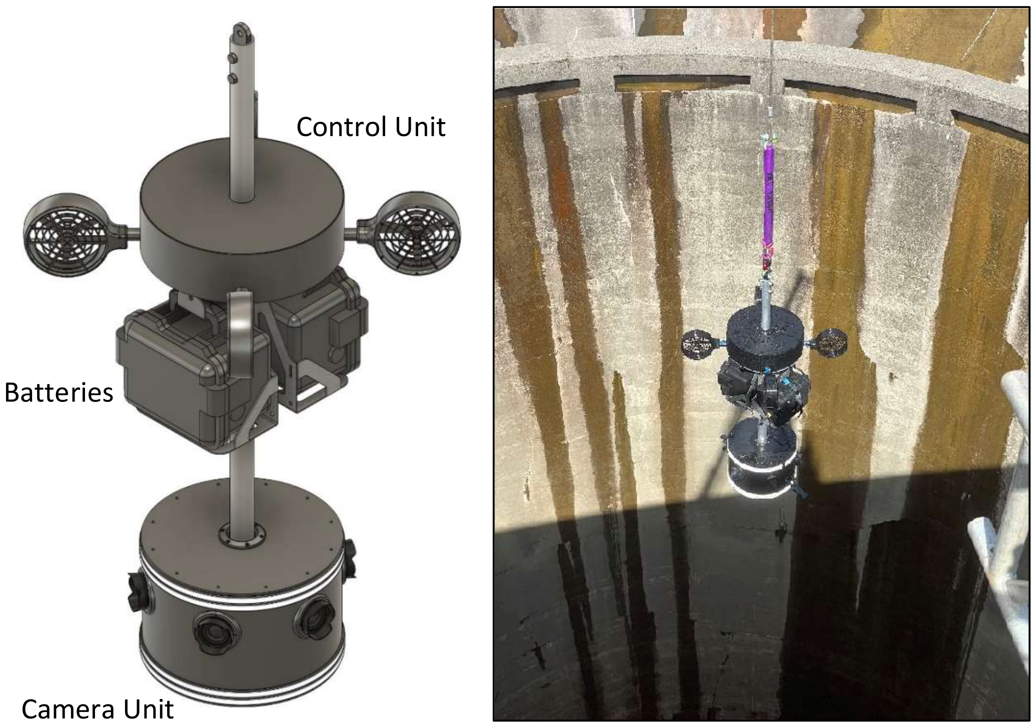

Figure 2. Shaft Inspection Vehicle – design render (left) and implementation (right)

Tunnel Survey

The second data collection mission focussed on the tunnels between the shaft and the main inlet valves, inclusive of the 10m diameter tunnel and the 3m diameter unit penstocks. This required that the same camera platform was deployed using an alternative wheeled vehicle, shown in Figure 3 and Figure 4**.**

Initially it was planned to make entry into the tunnels via a 600mm diameter manhole. As such, a vehicle was designed using metallic truss and light weight scaffold poles. This included bespoke manufactured wheel fittings and a roped mast deployment to ensure that the vehicle could be constructed internally within the tunnel without needing a larger access portal.

The vehicle mast was used to deploy the camera platform inclusive of lighting. The triggering of the cameras was undertaken using a magnetic sensor on the wheels of the vehicle, such that the images were captured at 0.3m intervals.

The vehicle was light enough to mobilise manually, and to preserve weight and construction complexity no motorised platform was deployed. Steering was undertaken with the use of bespoke manufactured fittings.

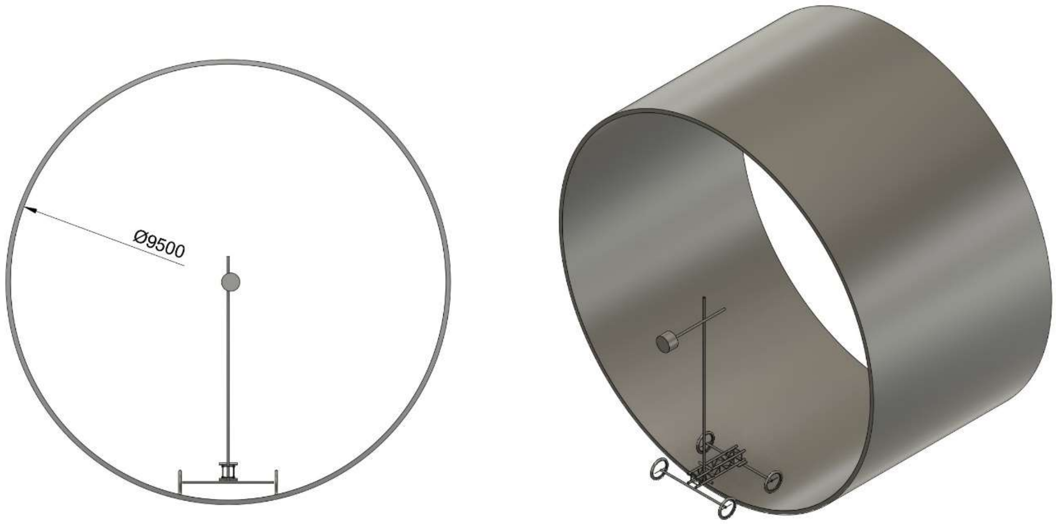

Figure 3. Tunnel Inspection Vehicle design for c.10m diameter configuration

INTERPRETATION

The data collection missions captured approximately 38,000 images, totalling almost 1TB of data. In its captured format, this data would be difficult to use.

The photographic data was post-processed using two sequential methods. The first was to orthorectify each image to 'flatten' the surface captured, thereby allowing stitching of an orthomosaic. The second was to process the images using the AssetScan AI to automatically identify defects of interest.

Figure 4. Tunnel Inspection Vehicle, implementation in c.4m diameter configuration

Image Remapping

The remapping of the images was undertaken using a set of calibration images captured of a flat grid. This, in combination with the known radius of the shaft/tunnel, was used to generate a warping profile which was used to stretch each image. The warping was undertaken using a subroutine available in the OpenCV image processing library (Bradski, 2000). An example of this is shown in Figure 5.

Once each image was flattened, a process was used to determine the radial position of each image in comparison with the neighbouring cameras. This allowed the construction of dense image ‘rings’ of the entire circumference of the shaft and tunnel at each position as an orthomosaic. The edges of each image were feathered to ensure a smooth transition between each image. The native resolution of the images was approximately 1mm/pixel. Following this, a central band of the image, representing 300mm, was extracted and stacked against the preceding and following ‘rings’ to form a large orthomosaic of the entire shaft and tunnel. An example of this is shown in Figure 6.

This allowed the production of 13 Tagged Image Format (tif) files, totalling more than 25GB of processed photography data. The results were projected onto 11 large A0 printing templates for use by Engie and their engineers.

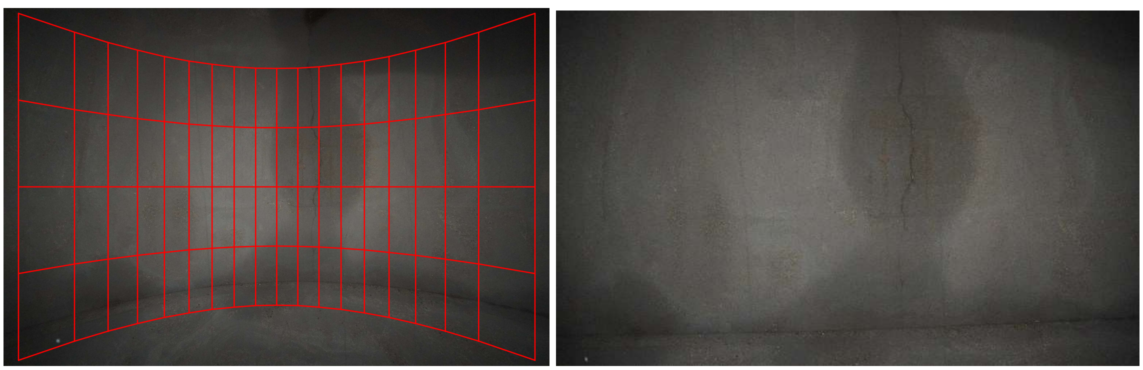

Figure 5. Image Remapping – (left) orthorectification warping mask (right) approximate orthorectified image

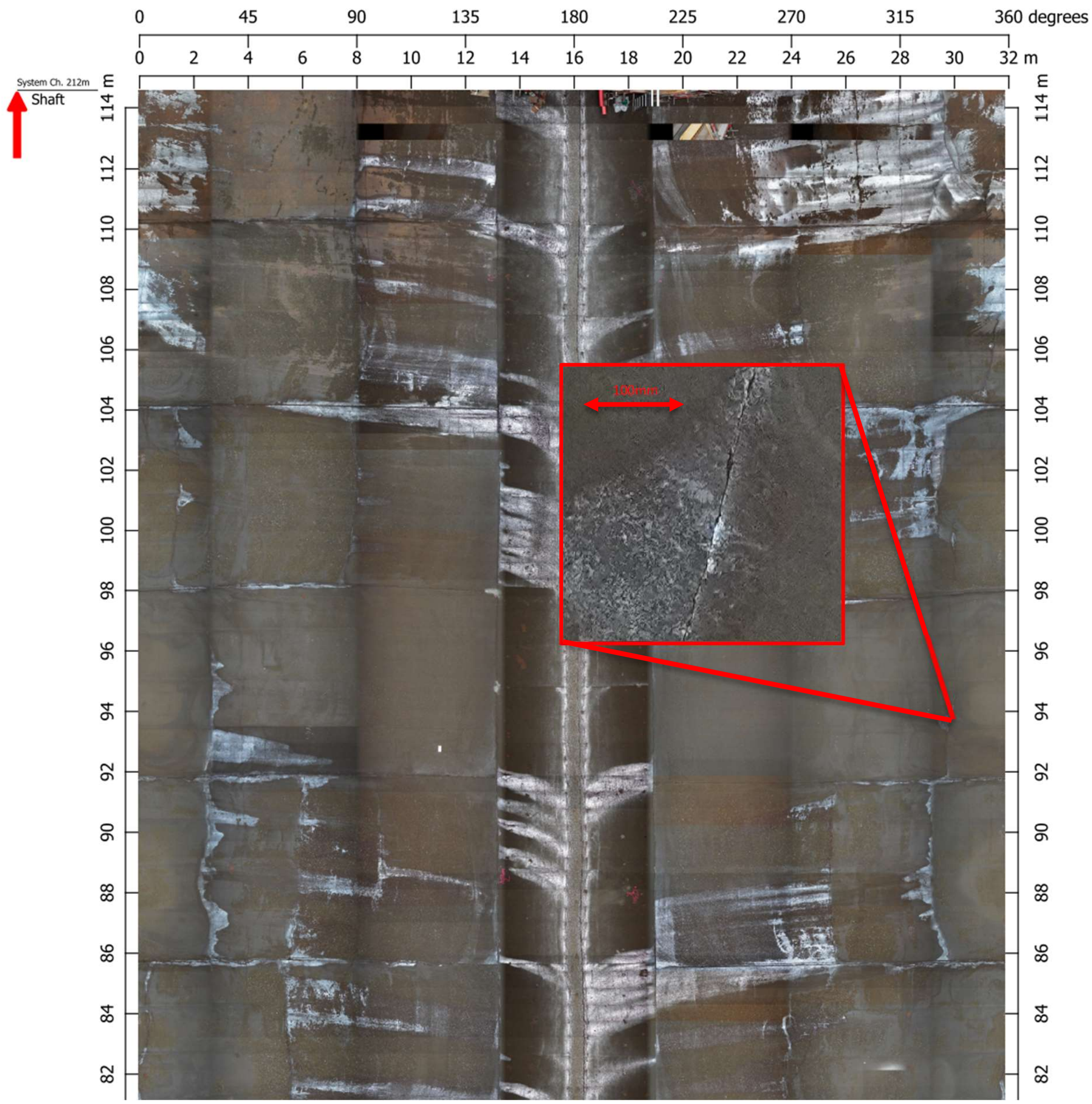

Figure 6. Composited orthomosaic – example outputs for c.10m diameter tunnel

AssetScan AI

The images were also subject to interpretation by AI. The scale of the surveyed surfaces was so large as to make it difficult to manually review the captured dataset. The primary purpose of the AI was to draw the attention of engineers to positions of defects. Three types of feature were specifically identified in the images: cracks, surface loss, and previous patch repairs. Further to this, two other types of feature were identified: formwork joints and formwork joints that were visibly compromised and potentially cracked.

Following previous work undertaken with AssetScan (Coombs, 2022), an existing off-the-shelf concrete AssetScan model was applied to the image datasets. It became clear that the high wetness of the concrete surfaces in the shaft would preclude use of this particular model since defects would appear quite differently. As such, a 'wet' concrete AI model was used for the shaft, and a 'dry' concrete AI model was used in the tunnels.

Following a detailed review at individual pixel resolution, the AI data was simplified into vector geometry and summarised on both drawings and tables. This could then be used by engineers to determine the location of surface indications that warranted attention throughout the entire length of the surveyed structure. This was mapped against chainage and orientation within the tunnel portal. An example of the simplified data is shown in Figure 7.

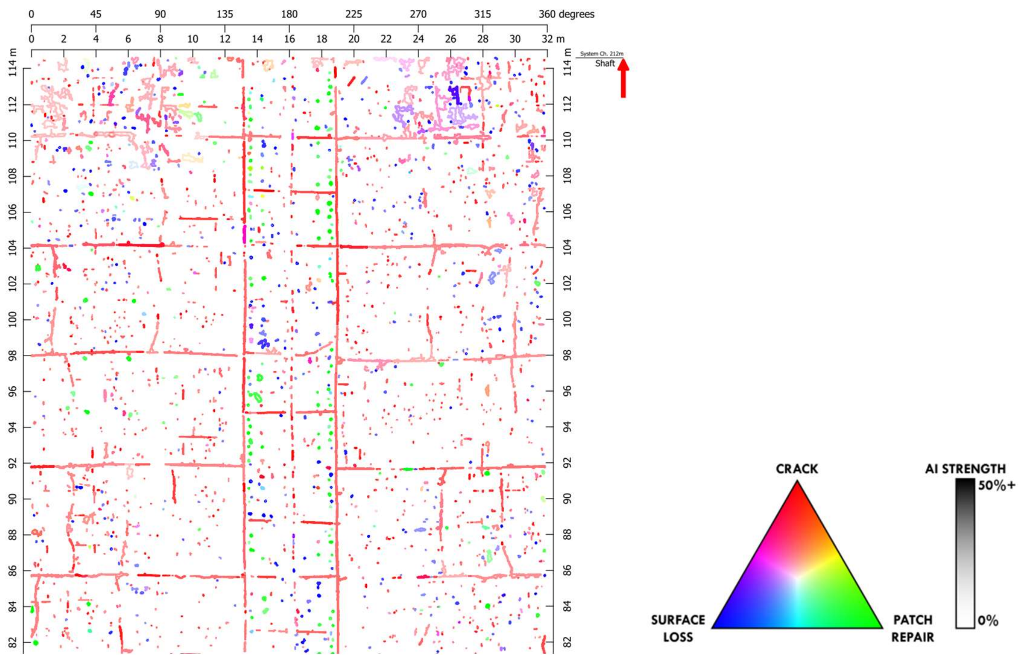

Figure 7. Composited AI data – tunnel example

EVALUATION OF DATA

In application at Dinorwig

There are examples of hydromechanical elements of the power station that have been surveyed in 3D to 1mm accuracy, such as within the spiral casings of the turbines, which provides engineers the facility of a virtual reality 'walk through' the structure and manually spot defects or measure components. For a structure as large as the high-pressure waterway system, however, AI analysis reduced the chance of missing any potential defects which may not have been picked up in manual visual checking.

As expected, the quality of the imagery captured as part of this project was much better than previous efforts with ROVs. The ROV surveys produced lesser quality imagery due to the presence of suspended solids and poor transmission of light. The ROV surveys were also difficult to interpret in terms of quality measurements for any identified defects, as well determining their size and position.

The outputs of this survey were useful in that they allowed:

- A snapshot in time of the high-pressure waterway system. The outputs of this survey may continue to be useful into the future if the survey is carried out again in following drawdowns, to allow detection of possible deterioration to allow extrapolation of deterioration which could inform future maintenance or timelines for refurbishment. As such, the survey not only provides information presently but is an investment for future monitoring and surveillance.

Managing Risks for Dams and Reservoirs

- Quantification of groundwater seepage, using photographic imagery, which give a general overview of concrete permeability as an input into "Factor of Safety" verification (Stantec, 2023).
- Preliminary video footage of the high-pressure shaft liner, which confirmed that there were no immediate concerns with regards to potentially loose concrete. This assisted with assessment of risk regarding personnel entering the manifold (First Hydro, 2022).

In general application

Holistic photographic surveys of tunnels and shafts are useful in that they capture not only the defects of interest, but also the relative size, position, and context of said defects. This means that engineers can quickly identify areas of interest on drawings and maps, and then rapidly find them during site reconnaissance. Additionally, it allows the development of surveillance targets on repeat inspections. The images generated by this project are approximately 1mm/pixel. This means that hairline cracks may not be visible in the images. Despite this, such a high resolution capture of a structure could act as a baseline for comparative assessment for larger defects.

The orthomosaics generated by a holistic photographic survey are difficult to manually review due to both their size and extent. Some of the high resolution TIFs generated for the shaft were in excess of 10GB, for example. On standard office equipment these can be challenging to view. Further to this, the image extent and resolution makes manual inspection of all defects an excessively time consuming activity. By extension, such an exercise would also be prohibitively costly for a large structure such as Dinorwig.

AssetScan demonstrates that this problem can be overcome with the use of computer vision. An AI model with sufficient complexity and trained on a suitably large dataset is able to interpret these datasets and return outputs which are easier to review manually. The AI results could then be used to draw the attention of an engineer to potential defects. In combination with high resolution orthomosaics, this would then allow engineers to assess the current condition of large structures.

Additionally, since the holistic photographic data capture method used is repeatable, and since the AI is able to locate most defects, this process could be repeated as part of an automated change detection assessment. For example, should a second survey be captured, then this data could be parsed by the AssetScan AI. The results from both surveys could then be compared directly. It is a trivial task to locate where the AI geometry has significantly changed between surveys, where each photographic survey has been taken relative to some known position. This could therefore be used to automatically point engineers to defects which are changing, or to defects which have developed since the previous survey.

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The British Dam Society

Adapting earthworks design for adverse weather conditions

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SYNOPSIS Undertaking earthworks in winter and wet weather is generally avoided due to construction difficulties and potential quality implications. However, with changing climate and programme related challenges, it may not always be possible to avoid this.

Barrowford reservoir has had a long history of seepage and stability issues and due to the constrained nature of the site, the preferred solution was agreed to reduce the top water level and to regrade the slope within the existing site boundary to improve the factors of safety. The north embankment showed signs of accelerated settlement when compared with the other embankments and signs of internal erosion having been noted in the history of the site. A filter blanket was designed for the north embankment to prevent migration of the fine material.

Delays in construction meant that winter working was required in order to maintain regulatory compliance. This paper summarises how the works was investigated and designed to improve slope stability and reduce risk of internal erosion at Barrowford Reservoir and how the design was revised part-way through construction in consultation with the Construction Engineer, Undertaker and Contractor to allow winter working to be undertaken and quality was maintained by adopting a method specification with performance testing of the earthworks.

BACKGROUND

Site Overview

Barrowford is a non-impounding storage reservoir that was formed in 1886 by the construction of a perimeter earth embankment some 1,000m long which retains a volume of 453,840m³ at top water level (TWL). The maximum height of the embankment is 8.8m. The upstream slope is 1 in 2.5 (V to H) and is lined with stone pitching. The crest is some 2m wide and grass covered with gravel footpath. The downstream slope is generally 1 in 2 (V to H) and is grass covered. Barrowford is owned and operated by the Canal and River Trust (the Trust).

The historic data indicates that the reservoir was constructed around a natural depression, however some cut and fill is evident along the line of the embankment, with cut to the north and fill to the south, east and west. The fill from the embankments was likely sourced from material excavated from this cutting, the basin and possibly also from the nearby canal.

Managing Risks for Dams and Reservoirs

The reservoir is founded on Glacial Till of variable composition including distinct bands of granular deposits. The Glacial Till overlies bedrock of the Mill Stone Grit.

Historic Context

Inspection records of the embankment extend back to 1931, however the first leak was recorded in 1981. Various leaks, superficial slips, sink holes, depressions, etc were recorded from 1981 onwards with 1983, 1984, 1991, 1994, 1997, 1999 being particular cases. It is unclear why this change in behaviour occurred. A review of historic climate data indicated a general trend of increasing temperatures but nothing distinctive is apparent in the early 1980s.

A possible change in the operation of the reservoir may have occurred during this time which may have precipitated this behaviour – such as greater fluctuation in reservoir levels or extended drawdowns, but records were not available to confirm this.

A number of investigations were undertaken at Barrowford over time including ground investigations, ground temperature measurement for leakage by GTC (Kappelmeyer GmbH), Willowstick resistivity survey for evidence of leakage and a British Geological Survey (BGS) geophysical survey.

A series of interventions were undertaken to stop or manage the leakage at the reservoir, including installation of trench sheets in the upper portion of the embankment to parts of the west and south embankments and most recently installation of counterfort drains to the north-east embankment in 2008. Approximately 25% of the dam is known to have had works done to the upper part of the embankment to address seepage related issues.

The Trust would regularly attend to site to remediate topsoil slips on the slope over the winter period where high rainfall would precipitate movement. This resulted in additional burden on the maintenance teams each year.

The Trust, as operator of Barrowford, commissioned Mott MacDonald to develop solutions to address leakage and slope instability of the embankments to ensure the safe continued operation of the reservoir following a Measure in the Interests of Safety under the Reservoirs Act 1975.

ANALYSIS AND DESIGN OF PROPOSED SOLUTION

Review of Monitoring Data

Piezometers and drainage

Long term piezometric data was available for seven cross sections spread along all but the west embankment. Some spot records were available along three cross sections of the west embankment from a ground investigation in 1991. All cross sections consisted of three piezometers each – one in the crest, one in the downstream shoulder and one in the toe. Toe drains were only present on the north-east embankment.

Along all but the south-east embankment, a strong change in piezometer readings was noted when the reservoir was at or above top water level for prolonged periods. This was supported by the drainage monitoring data and it was known that seepage ceases whenever reservoir level is dropped.

The piezometers closest to the reservoir (irrespective of tip level) were seen to have the most direct relationship with reservoir levels. Piezometer readings suggested that the core was of variable quality, but this did not produce significantly high pore pressure in the downstream fill nor foundation.

The review of the data indicated that foundation seepage occurs at discrete localised coarse deposits on the north and north-east embankment. It was considered that foundation seepage is of a modest extent given the limited piezometric response and absence of significant issues observed in the area of foundation seepage.

Settlement

Long term monitoring pins were present generally at 20m intervals along the crest of the dam. The settlement and strain experienced by the embankments were noted to be in line with expected behaviours. The rates experienced by the north-east, south-east and west embankment, were on average, recorded to be in line with this range.

On the north embankment, extensive settlement and high rates of strain were noted to have occurred in the embankment which could not be attributed to reservoir draw down nor compressible founding soils. It was noted that the zone of excessive movement along the north embankment corresponded directly to the area where seepage and sinkholes had been observed. It was concluded that the excessive crest settlement was most likely caused by the erosion of the fill when seepage flows overtop the (low) core and pass out through discrete preferential paths in the downstream shoulder.

Review of Historic Investigation

Extensive investigation has been undertaken at Barrowford reservoir which is summarised in Table 1.

Table 1. Summary of Ground Investigation at Barrowford Reservoir

Year	Description
1991	Soil Mechanics (9 boreholes and 6 trial pits to west embankment)
2007	White Young Green (9 boreholes to north-east embankment)
2007	GTC Kappelmeyer (Geophysical survey south-west corner (south Embankment)
2015	Hyder (6 window samples to north embankment and 6 to south embankment)
2017	Arcadis (6 trial pits to north embankment)
2017	GTC Kappelmeyer (Geophysical survey north-east and south embankment)
2018	Arcadis (30 trial pits along entire crest)
2019	Arcadis (4 window samples to south embankment)
2018	GTC Kappelmeyer (Geophysical survey along entire embankment)

Year	Description
2019	Willowstick (Geophysical survey along entire embankment)
2019	BGS (Geophysical survey to North-East and South embankment)
2020	Arcadis 4 window samples and 7 trial pits spread across all embankments, partly to ground truth the BGS study

Ground Investigation

The majority of the historic ground investigation had been conducted from the crest of the embankment. A review of all ground investigation concluded that the embankment fill was highly variable with a likely central clayey zone rather than a well-defined clay core. This clay was easily identified in some areas, and less so in others. In some cases, over a 10m length of crest, there would be evidence of clay central to this, with no evidence of clay from the ground investigation 5m either side. There was no evidence of a cutoff into the foundation.

Kappelmeyer

A Kappelmeyer survey was undertaken in March 2017 along a 130m length of the embankment that was of greatest concern. This included the whole of the north embankment and some 40m of the north-east embankment. The investigation recorded seepage flow through the embankment at two discrete lengths totalling 30m of the total 130m under investigation. The results showed high level seepage down to a depth of 2.5m below crest level.

A second Kappelmeyer survey was undertaken along the whole embankment in January 2018. The investigation recorded small seepages down to a depth of 2m below crest level. These investigations correlated with the piezometer data and drainage flow measurements recorded along the embankment.

Willowstick

A Willowstick survey was undertaken in 2019 and identified a number of seepage paths.

In the north embankment, the survey did not record seepage through the embankment but did record seepage through the foundation at two preferential locations, A and B (Figure 1).

- Inferred Seepage Path A: A series of three piezometers were installed at the location of the inferred seepage path into the foundation. All three piezometers recorded a distinct response to variation in reservoir level. The monitoring data related well to the findings of the Willowstick survey.
- Inferred Seepage Path B: Limited piezometric data was available in this location and due to a lack of long-term evidence the relationship could not be confirmed. However, the borehole log recorded the presence of blowing sand at elevation some 3m about the inferred seepage path. The presence of the blowing sands is indicative of high confined water pressure which would tend to support the presence of a seepage path in this area.

- A piezometer located in between the two preferential locations recorded no response with variation in reservoir level - which correlates with the findings of the Willowstick survey.

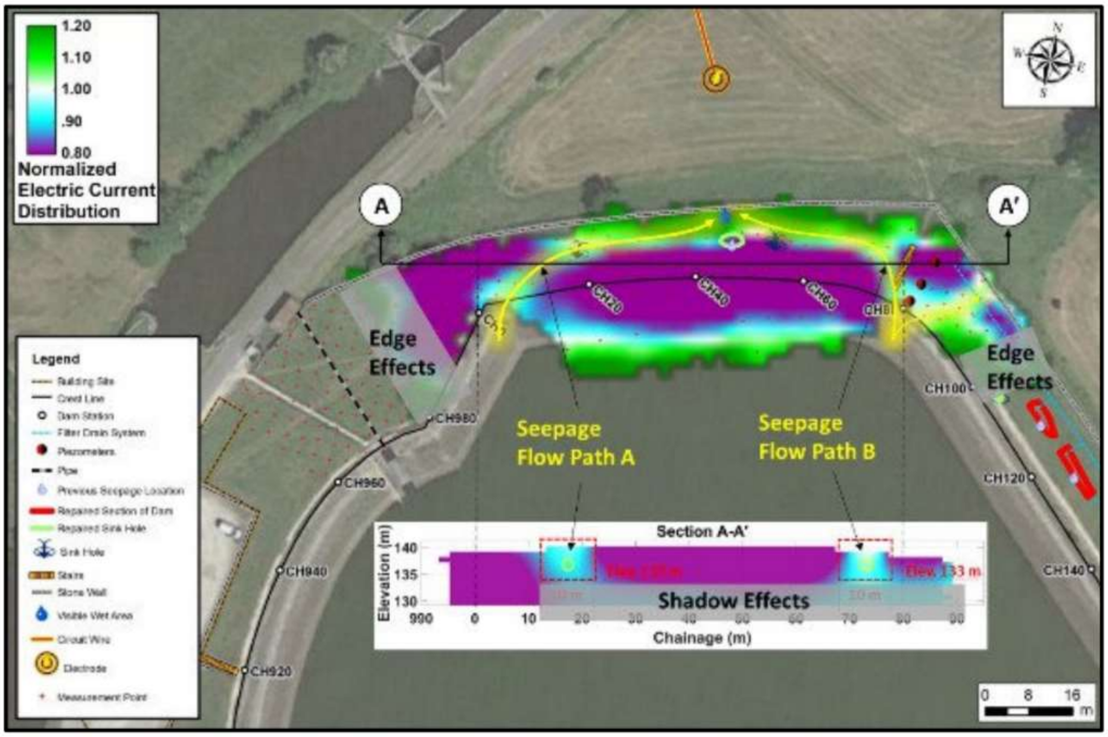

Figure 1. Seepage Paths in the North embankment

In the north-east embankment, the survey identified seepage at three preferential locations, C,D and E (Figure 2).

- Seepage Path C: An inferred seepage path was recorded in the foundation. No piezometers were installed here, although drains are installed along the downstream toe, however the invert of the drainage is above the inferred seepage path. The intensity of the reading at C is akin to that recorded at seepage paths A and B.

- Seepage Path D: An inferred seepage path D is recorded in the foundation and a piezometer was installed in the foundation in this location. A relationship between piezometric level and reservoir level exists here but it is modest – piezometric levels varying by 0.7m for 6m of reservoir head, a ratio of 1 in 8.6 variation in reservoir level. Willowstick records a weaker signal here than at inferred seepage Path C, indicating greater seepage there than at D.

- Seepage Path E: Seepage throughout the embankment is indicated over a 40m length a in the upper embankment. This does not correlate with drainage, piezometer nor Kapplemeyer data. A nominal seepage path was also inferred in the foundation at a discrete point in this area. No piezometer data was available here. The indicated seepage here was very low and was not considered a significant concern.

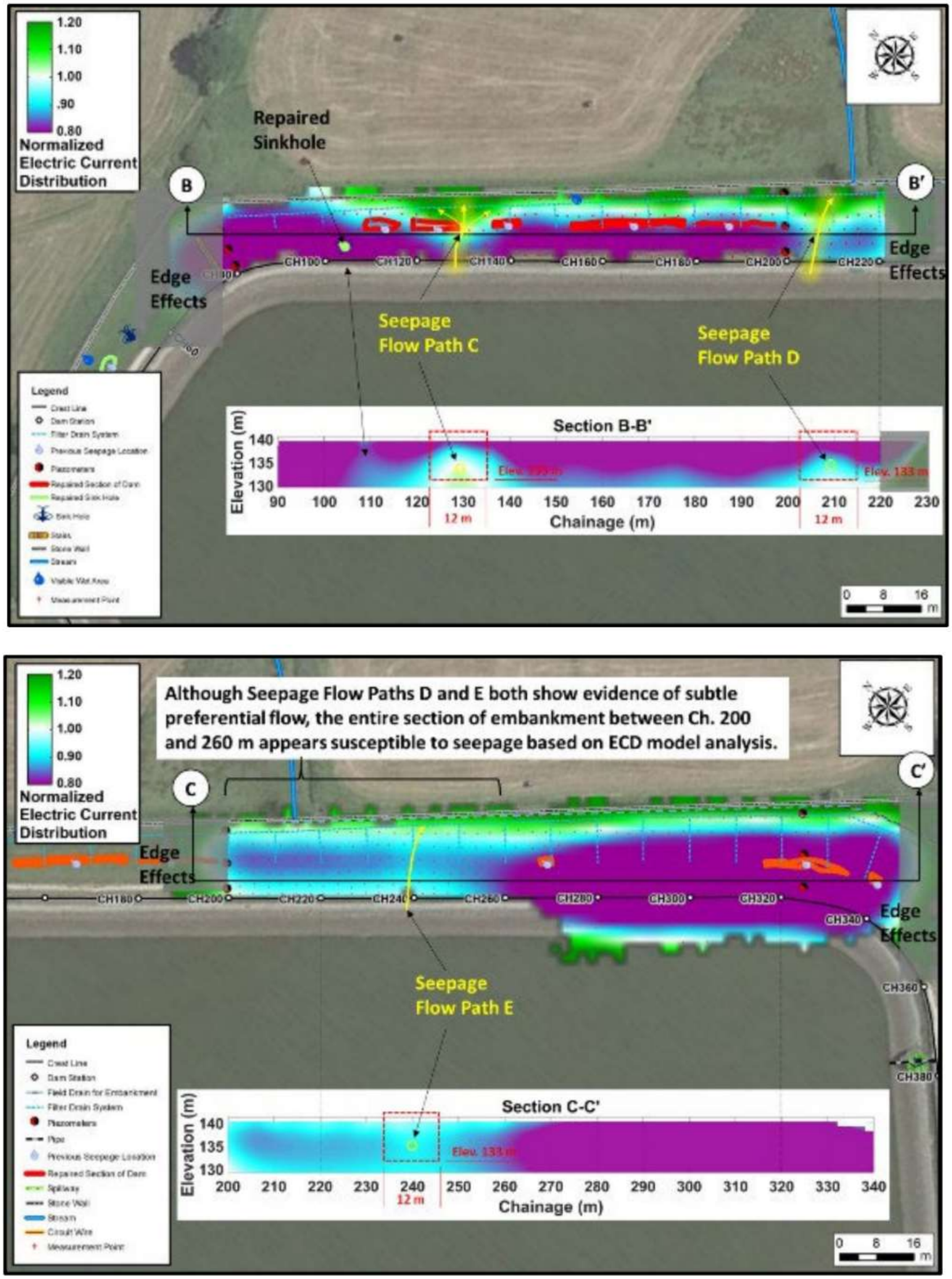

Figure 2. Seepage Paths in the North-East Embankment

In the south embankment, the survey identified seepage at one preferential location (Figure 3). High level seepage was indicated in the embankment upstream of the core for approximately 130m in length of the embankment. This corresponds to the piezometer data (as the core readings record a connection but shoulder fill does not) and generally with the Kapplemeyer survey.

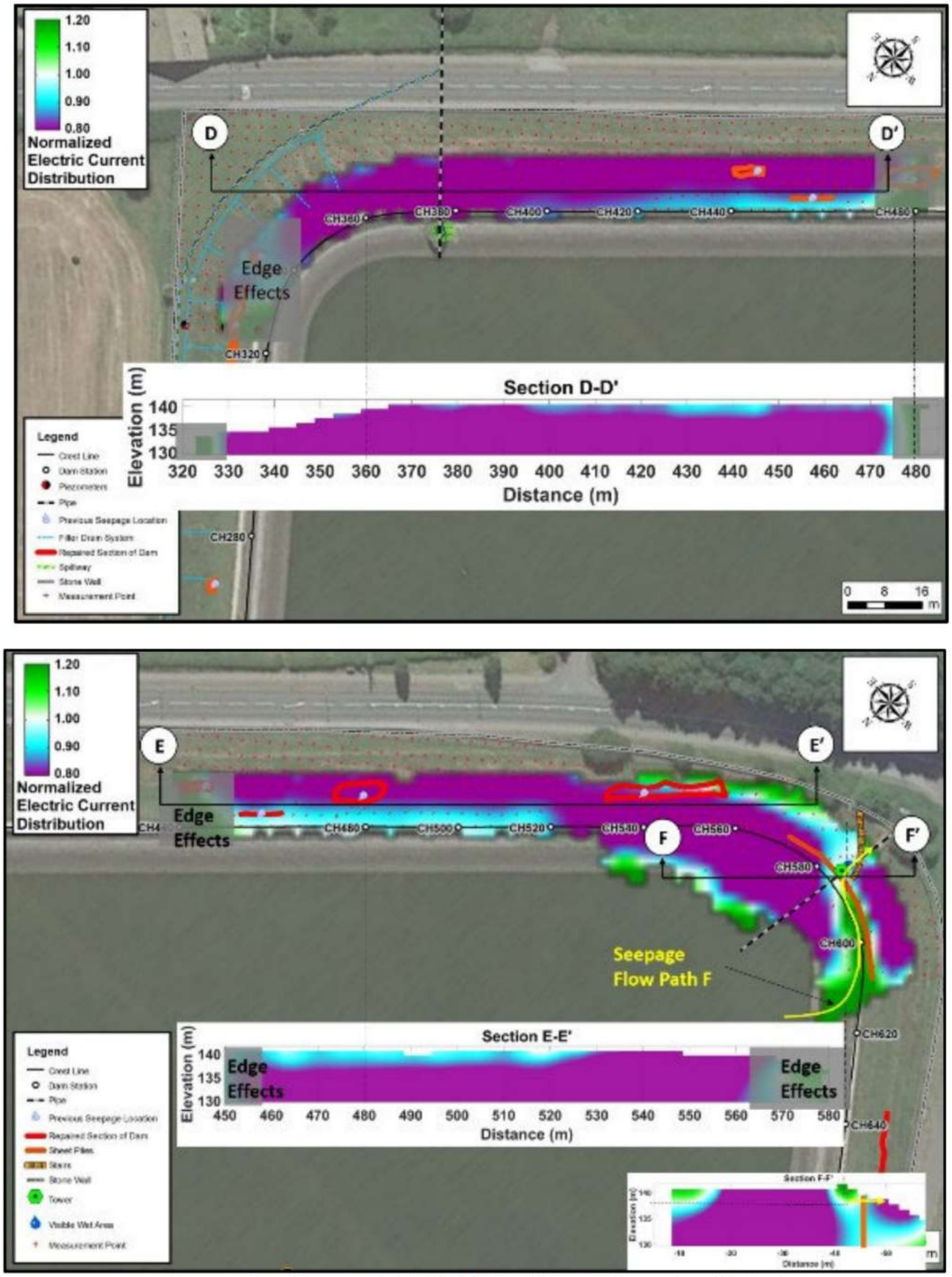

Figure 3. Seepage Paths in the South Embankment

In the west embankment, the survey identified a possible seepage-prone area for a 40m length (Figure 4). A line of piezometers installed at this location record no such issue in the downstream shoulder fill nor the foundation. A high-level relationship was noted in the core piezometers and was also recorded by the Kappelmeyer survey.

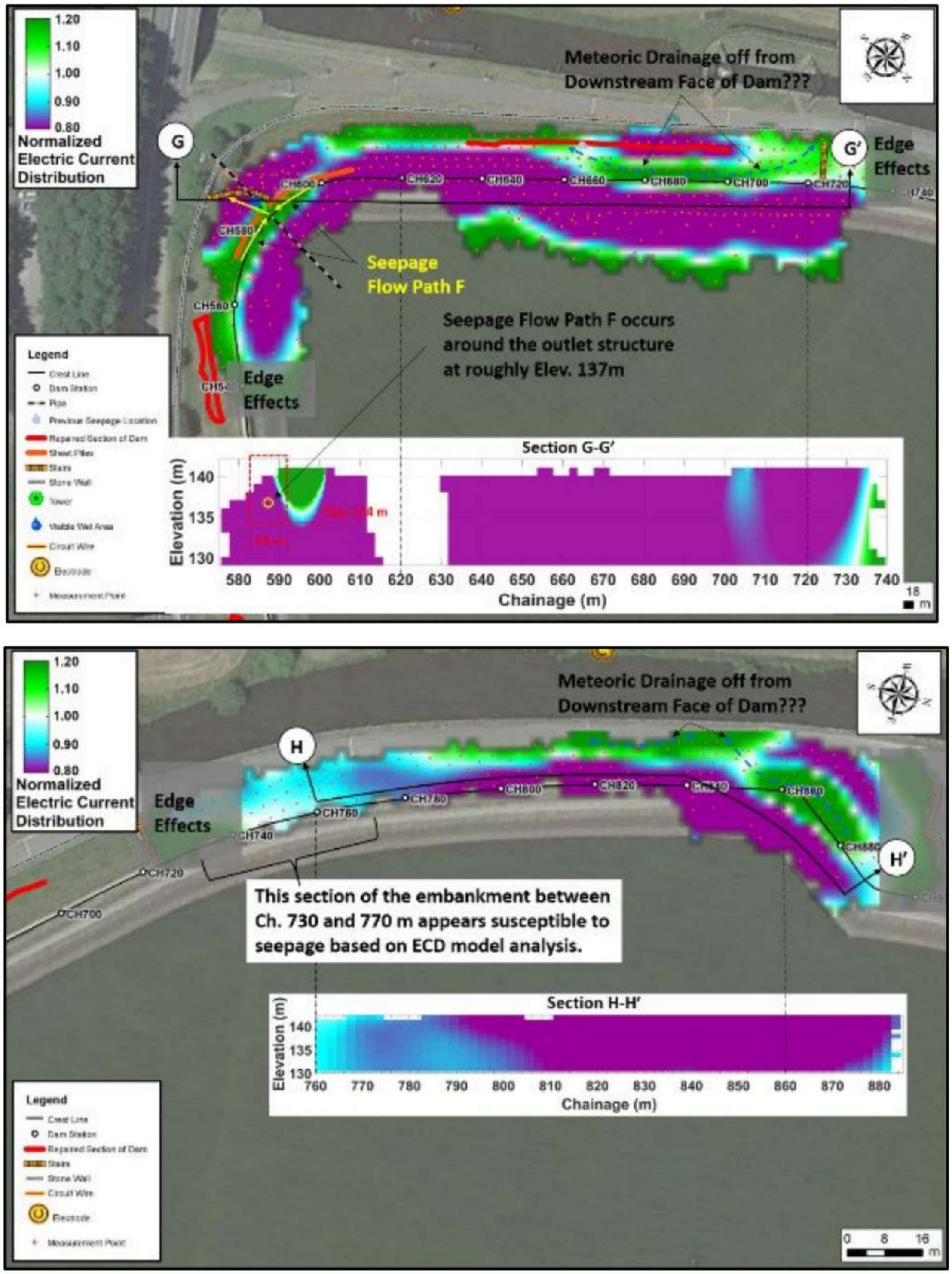

Figure 4. Seepage Paths in the West embankment

Geophysical survey and Ground Truthing

The BGS Geophysical survey results were made available shortly after Mott MacDonald begun work to optioneer potential solutions. The survey had used a combination of techniques to build up a picture of the embankment. The techniques used included electrical resistivity tomography and multi-channel analysis of surface waves. The results of this investigation concluded that there was a high potential for a raised upper core, doglegged above the original core towards the downstream side of the crest. Historic records did not indicate any known raising of the core so it was thought that if this was the case it was undertaken towards the end of the original construction period.

In order to ground-truth the results, ground investigation including trial pitting and window sampling was undertaken at the locations where the BGS survey had targeted.

The survey also revealed a localised area of shallow granular material on the downstream face of the two cross sections produced. During the works, these were revealed to be wall drains, likely to have been put in during the original construction to help manage pore pressures. These are detailed further in the paper by Brown et al (2024).

DESIGN

Slope Stability Analysis

Slope stability analysis was undertaken considering a range of scenarios including the static case, seismic analysis, rapid drawdown and the temporary construction case. Four different sections, one along each embankment were analysed. The slope stability analysis showed a marginal factor of safety in the static case in the existing embankment, as expected based on site observations.

Suffusion and Erosion Investigation

During the review of the existing data, it was clear that the north embankment had suffered high level seepage, sinkholes and undergone excessive settlement which could not be explained by either reservoir drawdown or compressible founding soils. Therefore, an investigation as to whether erosion was the root cause of these issues was undertaken.

The assessment found that the fill and foundation at the north embankment were extremely unlikely to be susceptible to internal erosion from the mechanisms of suffusion, backward erosion and contact erosion. The extent of seepage through discrete granular layers in the foundation was therefore determined to not be the root cause of the excessive movement of the north embankment.

Erosion of fine soil through concentrated leakage in the embankment fill was noted to be a possibility with regard to either cracking in desiccated soils at the crest and /or zones of permeable fill, the latter being the more plausible.

It was noted that the zone of excessive movement along the north embankment corresponded to the presence of all noted seepages and recorded sink holes as observed in the downstream face. The excessive crest settlement here was noted to possibly be caused by the erosion of the fill when seepage flows overtop the (low) core fill and passing out through discrete preferential paths in the downstream shoulder.

Dispersive soils were not recorded in the embankment fill. It was noted that there was potential for dispersive soils to be present in the foundation but such were likely to be of limited extent based on available ground investigation data and given the less onerous conditions (lower height and lower hydraulic gradient) compared to the rest of the embankment were unlikely to contribute to the recorded issues in the embankment shoulder.

Preferred Solution

A range of options was considered and concept designs developed in conjunction with a contractor. It was noted that the core is likely to be of such a varied nature, quality, condition and extent that accurate determination of its details was not practicable. The design and effective construction of any core raising option would be hampered by the variability in nature and position of the lower core.

Following a review of all the options and undertaking a buildability and cost build-up exercise, the proposed solution which was agreed was to permanently reduce the top water level by 1.8m and regrade the downstream shoulder to a slope of 1:2.5 so that the operation of the reservoir level avoids, as much as practicable, the reservoir being impounded at or above the upper core. This was supported by slope stability analysis to confirm adequate factors of safety were being achieved.

The adoption of this approach was supported with evidence from the BGS geophysics, long term monitoring data and temperature monitoring results which showed a potential linear defect in the core at this elevation.

Separate Consideration of the North Embankment

Due to the accelerated settlement on the north embankment and evidence of washout of the embankment, it was agreed that a slightly different approach would be taken at the north embankment.

To address this, the crest was to be left 500mm higher than the rest of the embankment to allow for the accelerated settlement. A filter blanket was designed to filter potential seepage from the embankment and prevent migration of the fine material. The final profile of the north embankment was designed to be shallower than the rest of the embankment with a profile of 1:3, with the filter to be covered with a compacted cohesive material and ultimately topsoiled and seeded.

New settlement pins were proposed along the whole of the crest of the embankment at 10m intervals, however the frequency was reduced to 5m for the north embankment, and settlement pins were also installed at the toe, to help identify movements of the slope.

Due to the presence of the preferential flow paths identified in the foundation during the Willowstick survey, as well as the presence of blowing sands, boreholes were proposed to target the permeable layer in the areas identified so that the groundwater could be alleviated and monitored long term.

CONSTRUCTION

Sourcing of filter material

Throughout 2023, suitable material suppliers were searched for to meet the specified requirements of both filter materials. Both fine and coarse filter material required a narrow grading with no fines smaller than 0.75mm and 2mm respectively. Due to these requirements, sourcing an acceptable material was challenging, with the Contractor deciding on sourcing a bespoke blended material for each in order to achieve the requirements of the specification. In July 2023, a crushed microdiorite basalt from the Minffordd Quarry was selected for the coarse filter and a blend of crushed tuff and quartz from the Cefn Graianog Quarry was selected for the fine filter.

Compaction trials

Compaction trials for both coarse and fine filter were initially undertaken in early June 2023 (Figure 5). The trials consisted of the construction of two small embankments approximately 20m in length by 5m width. The number of proposed layers being defined as the number required to determine a suitable compaction method to achieve the specified relative density of between 70% and 80%. A number of combinations of compactive plant, vibration frequency, layer thickness and passes were undertaken in order to define the optimum compaction methodology.

Figure 5. Fine Filter Compaction Trial

Upon undertaking the compaction trial of the fine filter, it failed to achieve the minimum relative density requirements of 70% reliably. Upon analysis of the trial results, it was determined that the minimum density testing results used to derive relative density were unusually high with results of 1.67Mg/m^3. Table 1 of BS8002 indicated, with a unit weight of 15 to 17kN/m^3 appropriate for uncompacted fine filter, that a realistic range of minimum density should be between 1.316 to 1.574Mg/m^3 with an average of 1.37Mg/m^3. Upon retesting the material for minimum density, the value reduced and as a result both material trials were re-attempted and met the relative density requirements of the specification.

Particle Size Distribution (PSD) testing undertaken throughout the compaction trial to validate the material source showed evidence that up to 5% material finer than 2mm diameter was present in the coarse filter material, contrary to the requirements of the specification which stated no material finer than 2mm was acceptable. Thorough processing of the material, including washing, eventually achieved the specification requirements by reducing the material below 2mm diameter to as low as was practically achievable (3%). The risk of increasing <2mm material during the wetter winter months was raised, as processing materials in a wet environment can result in higher entrained silts and clays in the processed material. To mitigate this risk, Mott MacDonald completed a site visit to the Cefn Graianog Quarry in order to ensure the material was washed and processed to a sufficient standard in all weathers.

Move to winter earthworks

Mobilisation of the Contractor began in October 2021. Works to regrade the west embankment were generally undertaken in summer 2022 with provision for coir matting where this was undertaken after August to help stabilise the topsoil over the winter period.

Works to regrade the north-east and south embankment were generally undertaken in summer 2023. Due to programme and procurement related delays, works to the north embankment, including the filter blanket were delayed until September 2023 and with inclement weather and the statutory deadline fast approaching, and the need for the reservoir to be back in service for the 2024 boating season, the design was revisited with all parties.

Adaptation of design

Due to the above noted delays, it was now approaching winter and as such there was concern that the relative density requirements would be difficult to achieve in wet weather. To combat this, the compaction specification was altered to a method specification, with the provision that in-situ relative compaction should be regularly monitored to reduce the risk of over compaction. Both filters were to be compacted in 300mm layer thicknesses and subject to four complete passes of a vibratory smooth drum roller with a static weight of 8T. Filter located on the slope incline was to be compacted in 225mm layer thicknesses and subject to four complete passes of a vibratory plate compactor with a frequency of 60Hz. Accounting for the difficulties of placing and compacting a cohesive material in the 'wet' season the shoulder fill atop the filter was changed from a cohesive to a granular material (SHW Type 803/1). The material surrounding the filter was altered to an associated hybrid specification, part end product and part method-related.

Compacting on top of peat

Winter working also meant the formation level to the filter blanket, which comprised 2m of peat overlying 2m of Alluvium, had become saturated with water due to a combination of surface runoff from heavy rains and artesian groundwater issuing from a nearby existing open well. Attempts were made by the Contractor to start work in the area, however the ground proved very soft due to the saturated conditions and they were unable to achieve the required compaction. It was necessary to improve the underlying peat via the provision of coarse granular material (200mm SHW 6G). This material was "pushed" into the formation with an excavator bucket, to 'tighten up' the peat, providing a suitable surface on which to place and compact the filter material. The 6G was designed to be placed in such a way that there was no continuous coarse granular layer at the base of the filter, which would have provided an unintended pathway for water flow.

Managing artesian groundwater

To combat the artesian groundwater issuing from a nearby well, the water was re-directed into the recently laid drainage run which ran along the toe of the proposed filtered embankment. Upon first placement of the first layer of filter material (which was to be fine filter material) in early November, the ground had frozen, due to its north facing location, whereas the filter material stockpile remained unfrozen and free moving. This significantly aided compaction of the first layer of filter material placed on the improved peat formation, building out of the wet material and ensuring a firm embankment base.

Two pressure relief wells were installed in the north embankment to alleviate artesian groundwater in the areas identified by the Willowstick Survey. Challenges were encountered during installation with managing the artesian groundwater and allowing suitable installation of the filter material around the perforated pipe. This was overcome by socketing the casing into a suitably impermeable layer of ground to provide conditions to assist with installation.

Testing

Once the first layer was complete, Nuclear Density Meter (NDM) testing and Sand Replacement Density (SRD) testing were undertaken on the fine filter and NDM testing completed on the coarse filter (as this material was unsuitable for SRD) proving a relative compaction of 89% on average was achieved in the fine filter and 92% in the coarse filter. The 803/1 shoulder fill material, requiring 95% relative compaction for compliance achieved an average of 99% with 100% of tests passing. PSD testing was also undertaken to ensure the filter materials had not significantly degraded (i.e. increase in finer sized particles due to particle breakdown) during compaction with acceptable post compaction results of no more than 4% below 2mm diameter achieved for the coarse filter, which was deemed acceptable. The coarse filter, fine filter and Type 1 embankment fill were subsequently placed without incident (Figure 6), with in-situ density testing and PSD testing carried out on each layer to ensure compliance to the specification.

Figure 6. Compaction of coarse filter

CONCLUSIONS

Whilst winter earthworks can prove challenging, this paper demonstrates that by adapting the design of earthworks, it may be possible to continue working through adverse weather conditions whilst maintaining safe working conditions and satisfying the requirements of the specification. The challenges faced at Barrowford included frozen ground, adverse weather, compaction on top of peat and working with elevated groundwater levels.

ACKNOWLEDGEMENTS

The authors are grateful to the Canal and River Trust for their permission to reproduce the data. The views expressed in the paper are those of the authors and not necessarily those of the Canal and River Trust and the Construction Engineer.

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The British Dam Society

Hydrological Risk Management for Proposed Mentarang Induk Hydroelectric Project in Indonesia

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SYNOPSIS Stantec has been engaged by PT Kayan Hydropower Nusantara, Indonesia to review the catchment hydrology and hydropower operation for the proposed Mentarang Induk Hydroelectric Project (MIHEP) in North Kalimantan, Indonesia. The project includes a 230m high concrete faced rockfill dam, gated spillways structure, 1375MW surface powerhouse and a reservoir (226km^2). This project is planned to displace fossil fuels sourced electricity in Indonesia.

Stantec re-established a rainfall-runoff model for the Mentarang catchment to generate long-term flows. The performance of the model was significantly improved due to a longer period of observed flow record supporting the updated model calibration. This provided a better understanding of the flows at Mentarang dam site. Stantec also conducted a climate change assessment using three widely recommended Global Circulation Models (GCMs) from the Coupled Model Intercomparison Project Phase 6 (CMIP6). The assessment suggests that under a mean ensemble of the three selected climate models, there would be 10% to 15% increase in future flows compared with the baseline period of 1990-2014. Reservoir operation was established incorporating the reservoir control rules and latest flows generated. The projected increase in future flows indicates improved power output for MIHEP. However, these findings should be considered with the caveat that GCMs have high uncertainty in projecting future precipitation and river flows.

PROJECT BACKGROUND

The proposed Mentarang Induk Hydroelectric Project (MIHEP) is one of the largest Hydropower projects in Southeast Asia planned on the Mentarang River in North Kalimantan, Indonesia[1]. The project includes a 235m high and 815m long concrete faced rockfill dam, a surface powerhouse with five Francis turbines (5 × 275 MW), a gated spillway structure with six large radial gates and it will create a large reservoir with surface area of226 km^2.

MIHEP will provide affordable, reliable, and renewable energy to the industries in Indonesia's Green Industrial Park (KIPI) at Tanah Kuning, North Kalimantan. KIPI is Indonesia's largest

[1] Indonesia breaks ground on $2.6bn Mentarang Induk hydropower project (nsenergybusiness.com)

green industrial park and a National Strategic Project (PSN) serving as a catalyst for Indonesia's Renewable Energy-Based Industry Development (REBID) initiative[2]. PT Kayan Hydropower Nusantara (PTKHN), which is developing MIHEP, is a joint venture company between PT Adaro Energy Indonesia Tbk (Adaro), Sarawak Energy Berhad (SEB) and PT Kayan Patria Pratama (KPP). Stantec has been engaged by PTKHN to review the hydrology, hydropower energy yield assessment and to support the project owner during the expected due diligence process to be conducted by the Lender's Engineer.

INPUT DATA

Observed meteorological data

Meteorological data including nine rainfall and three pan evaporation stations in the region were provided by PTKHN. Most of the stations sit within the Baram Catchment in Sarawak as described in (SMEC, 2014). Baram is a neighbouring catchment with long term meteorological and hydrological records. The details of these stations are provided in Table 1 and Figure 1.

The majority of the nine rain gauges have missing data in their record: Bario (1 year), Ba Kelalan (2 years), Lio Matu (3 years), Long Bawan (10 years), Marudi (1 year), Nunukan (1 year). Baram, Lg Pilah and Mentarang have no missing data in their records. Two rain gauges (Mentarang and Long Bawan) lie within the Mentarang Catchment. Nunukan lies on the eastern coastline of North Kalimantan. Rainfall depth and distribution are similar across the Baram and Mentarang catchments with the lower elevations of each catchment generally receiving more rain than the higher elevations.

The three pan evaporation stations record daily evaporation totals in millimetres. Miri and Belaga have records from 1988 to 2022 while Batang Ai Dam has a record from 1991 to present. Miri is located on the coast whilst Belaga is situated inland. Batang Ai Dam is located by an inland lake, 250km southwest of Belaga.

Satellite precipitation

There are several Satellite Precipitation Products (SPPs) available which provide precipitation coverage over Southeast Asia that is more temporally and spatially complete than rain gauge networks. Several studies have evaluated the performance of SPPs across Southeast Asia. Liu et al (2020) investigated three SPPs (GSMaP, IMERG and CHIRPS) against rainfall gauges over Bali Island, Indonesia from 2015 to 2017. The results demonstrated that IMERG achieved the highest performance on the daily time step whereas CHIRPS outperformed on the monthly time step. Wiwoho (2021) compared three different SPPs including CHIRPS, GPM and PERSIANN. CHIRPS had the best daily performance compared to these other products in Brantas, Indonesia. Liu et al (2022) argues that daily CHIRPS has high spatial resolution and is suitable for catchment scale studies when compared to rain gauge observations.

Therefore, CHIRPS (Funk et al, 2015) is chosen to infill station rainfall data. The nine rain gauges have different periods of record with missing data points. CHIRPS mitigates these challenges, as a source of rainfall estimates to address such gaps temporally and spatially across a catchment of interest. This is necessary for the hydrological modelling to generate long-term flows.

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[2] Mentarang Induk Hydroelectric Project (MIHEP) (ptkhn.com)

Table 1. Meteorological Stations with time duration and long-term average

Station Type	Station Name	Station Owner	Elevation (m aSL)	Time Duration			LTA (mm/yr)	
				Start	End	Total (years)	Total	From 2018
Precipitation	BaKelalan	DID	945	2001-01-01	2022-12-31	22.0	2331	2551
Precipitation	Baram	SEB	40	2013-05-13	2023-05-09	10.0	3731	4310
Precipitation	Bario	DID	1,046	1988-01-01	2022-12-31	35.0	2217	2265
Precipitation	Lg Pilah	DID	40	1998-01-01	2022-12-31	25.0	4627	4746
Precipitation	Lio Matu	DID	204	1988-01-01	2022-12-31	35.0	3559	3548
Precipitation	Long Bawan	BMKG	1,125	1988-01-01	2017-08-15	29.6	2464	-
Precipitation	Marudi	DID	17	2001-01-01	2022-12-31	22.0	2826	3160
Precipitation	Mentarang	PTKHN	23	2018-02-10	2023-03-02	5.1	4307	4307
Precipitation	Nunukan	BMKG	35	1998-01-01	2017-08-31	19.7	2439	-
Pan Evaporation	Miri	DID	18	1998-01-01	2022-12-31	25.0	1775	1810
Pan Evaporation	Belaga	DID	56	1998-01-01	2022-12-31	25.0	1564	1294
Pan Evaporation	Batang Ai Dam	SEB	112	1991-01-01	Present	32.4	1675	1628

Mentarang rating curve

A detailed statistical analysis was carried out on 19 Mentarang River flow gaugings to improve the river rating curve during the tender design (Entura, 2020). A HEC RAS model was also developed for the Mentarang river channel to extend this rating curve above gauged flows. The resulting rating gives similar mean flows to the PTKHN rating for the period of record.

However, for the section of the rating between 1,048m^3/s (maximum gauged flow) and 3,015m^3/s (maximum recorded flow) the HEC RAS rating is considered more accurate than the PTKHN rating because it is based on hydraulic modelling.

River water levels and discharge data

River water level records for five years at the Mentarang Hydrometric Station located at the Mentarang Dam site were available. The station's logger records 15-minute stage data. The river stage data were converted to flow using the developed rating curve and are plotted in Figure 2.

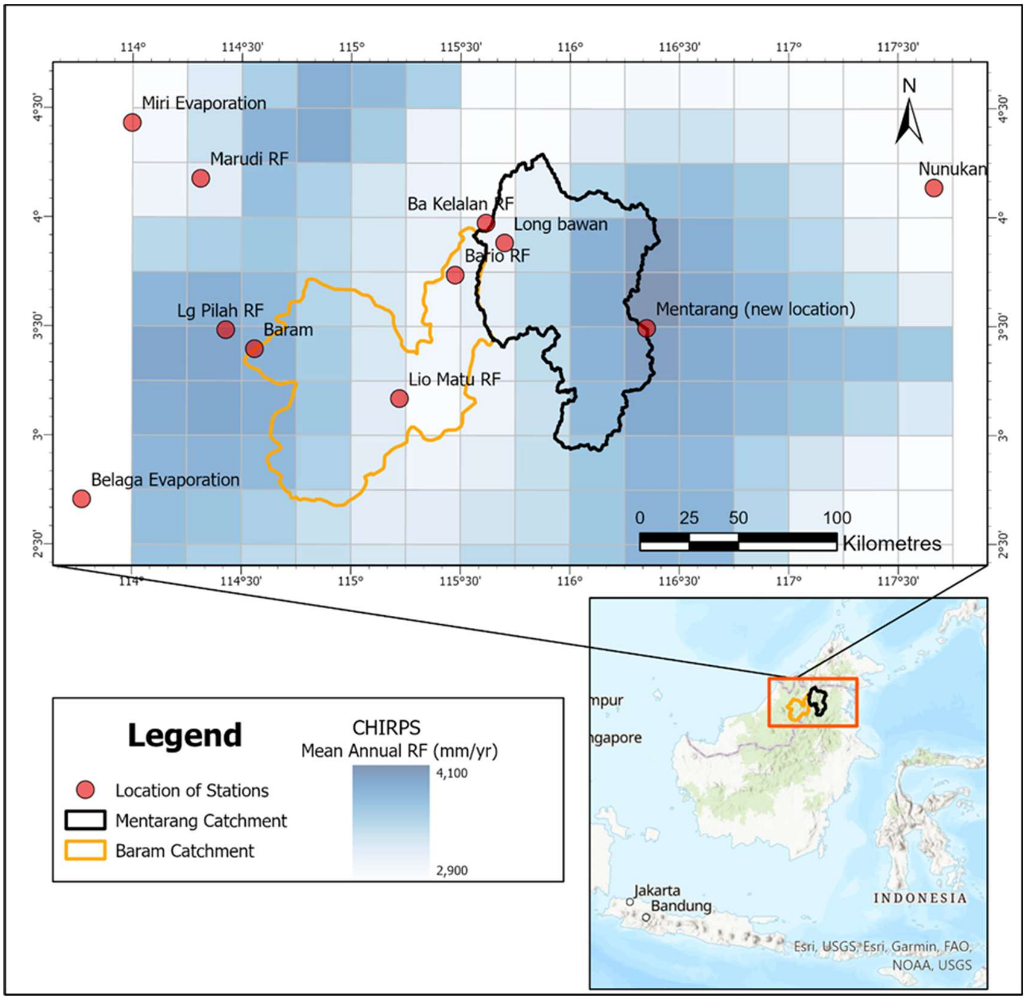

Figure 1. Map showing the meteorological and hydrometric stations in relation to the Mentarang Catchment and spatial distribution of annual rainfall from CHIRPS satellite precipitation product

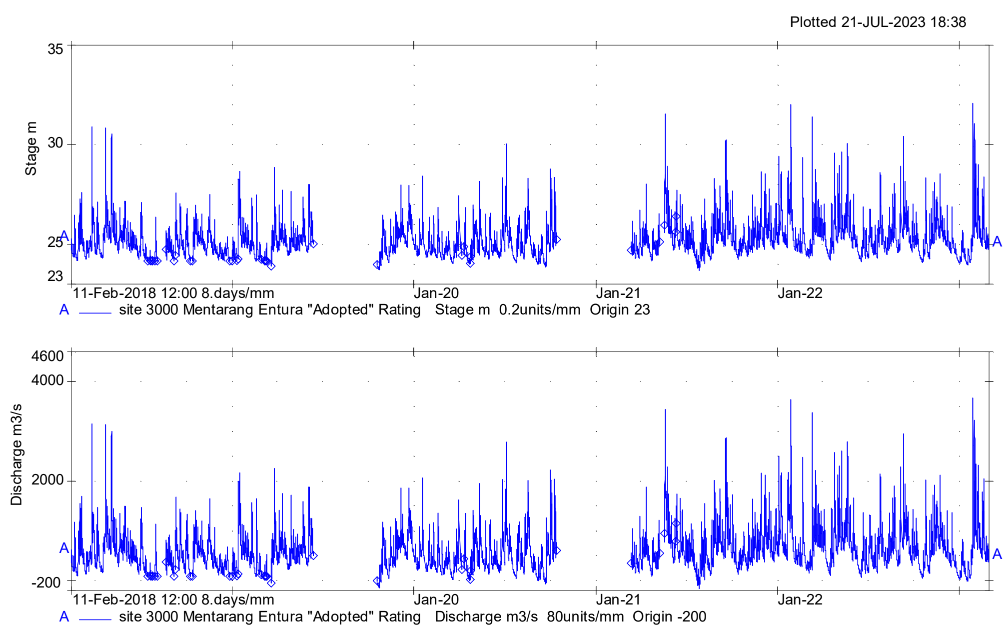

Figure 2. Stage and flow hydrographs at Mentarang Gauging Station

LiDAR survey for the reservoir area

A Light Detection and Ranging (LiDAR) survey for the Mentarang Reservoir area was conducted during the feasibility study (Norconsult, 2019) to improve understanding on the reservoir storage capacity. Reservoir surface areas and storage volumes at various elevations were calculated from the LiDAR data to plot an elevation area storage curve (Figure 3). These curves are then used in the reservoir operation model.

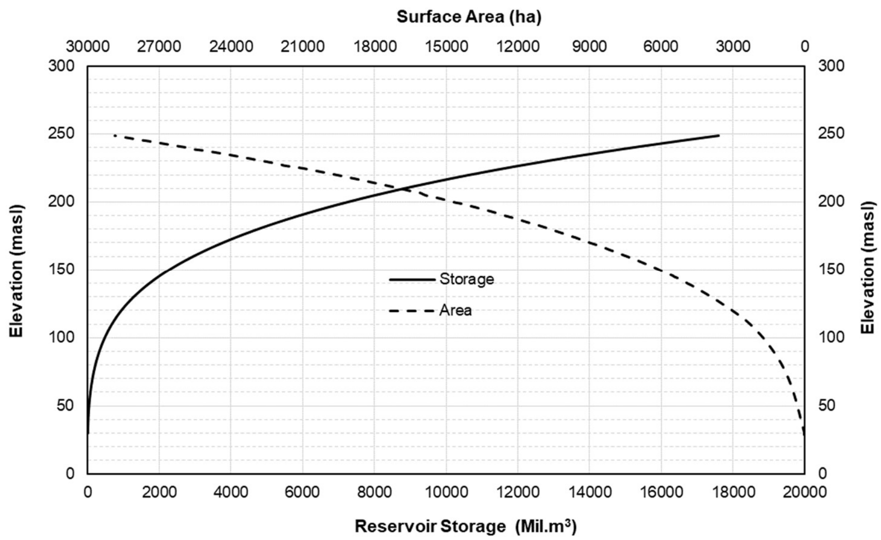

Figure 3. Elevation Area Storage Curve for Mentarang Reservoir

Tailwater rating, spillway discharge rating and waterway head losses

Tailwater rating curve, spillway discharge rating and waterway head loss were reviewed and updated while finalising design during tender design stage. Tailwater rating was developed taking into consideration both spillway and powerhouse discharges. Head losses in waterways have also been calculated for the designed penstocks. All five conduits have slightly different lengths and therefore resulted in slightly different head loss for each conduit. All these data were used here as finalised in the tender design.

Power plant data

The proposed MIHEP is designed to have five Francis turbines (5 x 275 MW) with a total installed capacity of 1,375 MW. The main features of the power plant are shown in Table 2.

Table 2. Power plant design features as per tender design (Entura, 2022)

Feature	Description
Type	Surface
Number of units	5
Unit type	Francis
Rated net head	195.1 m
Minimum net head	175.0 m
Rated output per unit	275 MW
Max. turbine output	307 MW
Unite rated discharge	151.6 m^3/s
Unit maximum discharge	166.8 m^3/s
Minimum tailwater level (flood protection)	24.6 masl
Minimum tailwater level (machine setting)	23.8 masl

The turbine performance/efficiency curve was also provided in the tender design which was adopted in the reservoir operation modelling.

HYDROLOGICAL YIELD

Rainfall Runoff Modelling

GR4J is a lumped parameter hydrological model (Perrin et al, 2003), and was used to develop a rainfall-runoff model for the MIHEP catchment. The model characterises catchment rainfall-runoff processes using four parameters, converting input time series of rainfall and potential evapotranspiration (PET) to specific discharge (that is, river flow per unit area of catchment). GR4J was also used in the previous hydrological study of the Mentarang catchment (Entura, 2020), where it was calibrated with only two years of observed flows.

The model was re-calibrated using the now five years of observed flows at the Mentarang Dam Site and then long-term flows were generated using the rainfall and PET data. Analysis of overall mean flows shows that the model matches observed mean flows well, with simulated mean flows of 593m^3/s versus mean observed flows of 595m^3/s over the five years of observed flows as shown in Figure 4. The flow duration curves for the period of 2018-2022 were compared as shown in Figure 5.

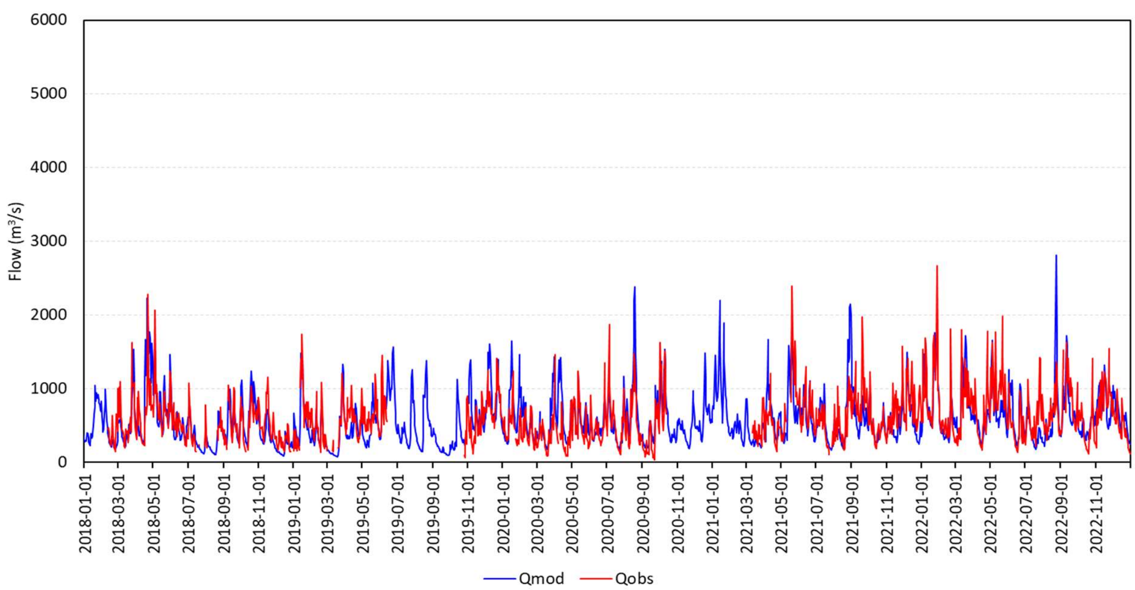

Figure 4. Observed vs Modelled flow during calibration

There is a significant difference between the average modelled flow since 2018, i.e 572m^3/s, and the long-term average modelled flow. The long-term average modelled flow is 518m^3/s for the period of 1993-2022; the 1993-2022 period excludes the earlier 12 years period (1981-1992) for which the model is dependent on less reliable and less complete rain gauge data. The reasons for this are also related to the trends in meteorological inputs, particularly the decline in PET. Therefore, the latest 30-year period is adopted for this hydrological analysis. A comparison of flow duration plots for observed, modelled and long term simulated flows is shown in Figure 5.

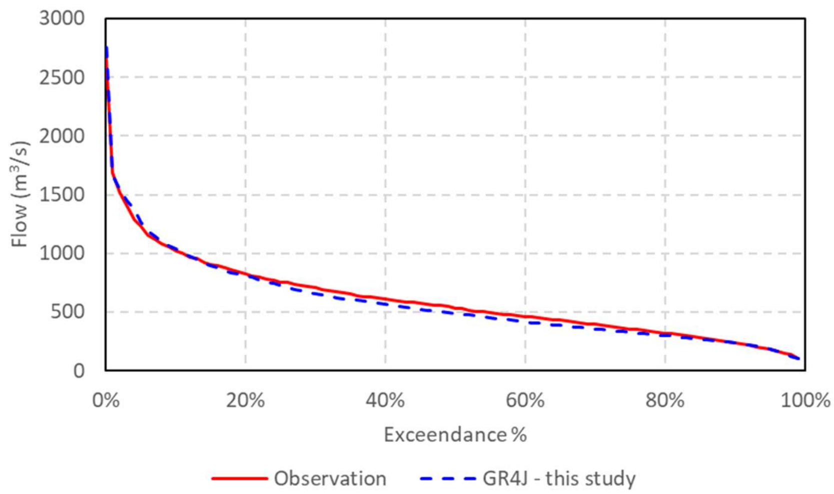

Figure 5. Flow duration curve for observed vs GR4J calibrated flows

Uncertainty

Confidence intervals have been calculated based on a log transform of the model results. Analysis of the residuals shows that log-based confidence intervals give a better representation of the uncertainty at all flow magnitudes. The confidence intervals, computed in log-transformed space, are presented in Figure 6. The uncertainty in flow is proportionate to the magnitude in flow, because of the log-transformation. This uncertainty can be minimized in the future by expanding hydrometric monitoring in the MIHEP catchment.

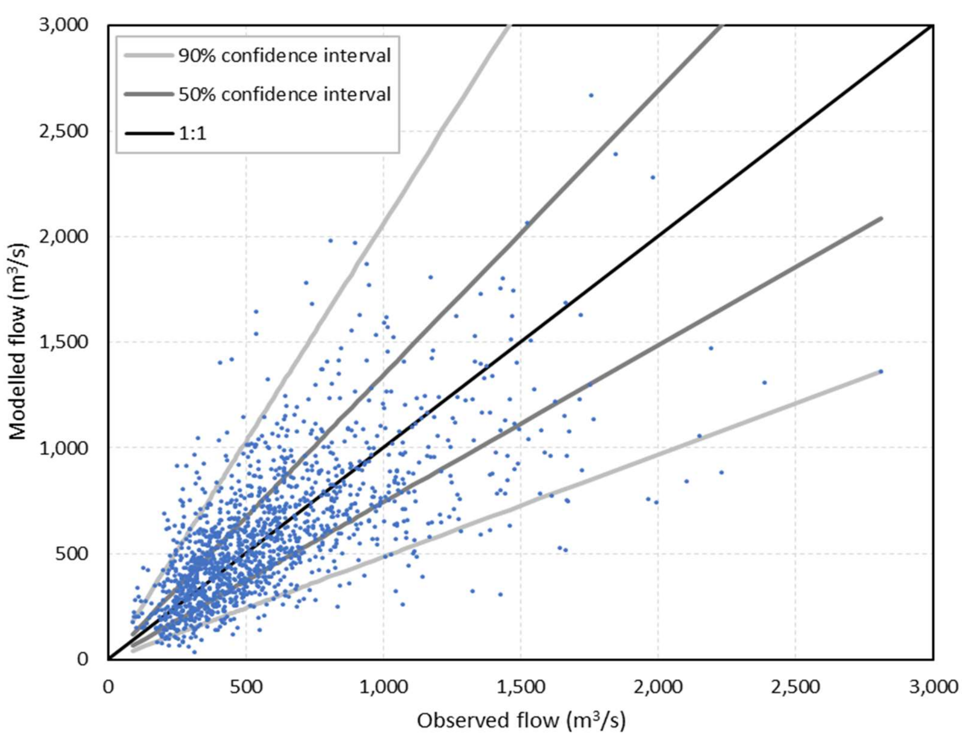

Figure 6. Modelled to observed daily mean flows, with confidence intervals

Managing Risks for Dams and Reservoirs

Comparison with previous work

Model uncertainty between this study and previous methods is presented in Figure 7. Previous studies attempted to estimate flow at Mentarang based on flows at Baram pro-weighted by catchment area, as well as using GR4J. The comparative confidence intervals shows that the current study has reduced the uncertainty of modelled flows, with the confidence interval closer to the match line between observed and modelled flows. However, as noted above, there is opportunity to further improve rainfall-runoff model with extended periods of observed record in the future.

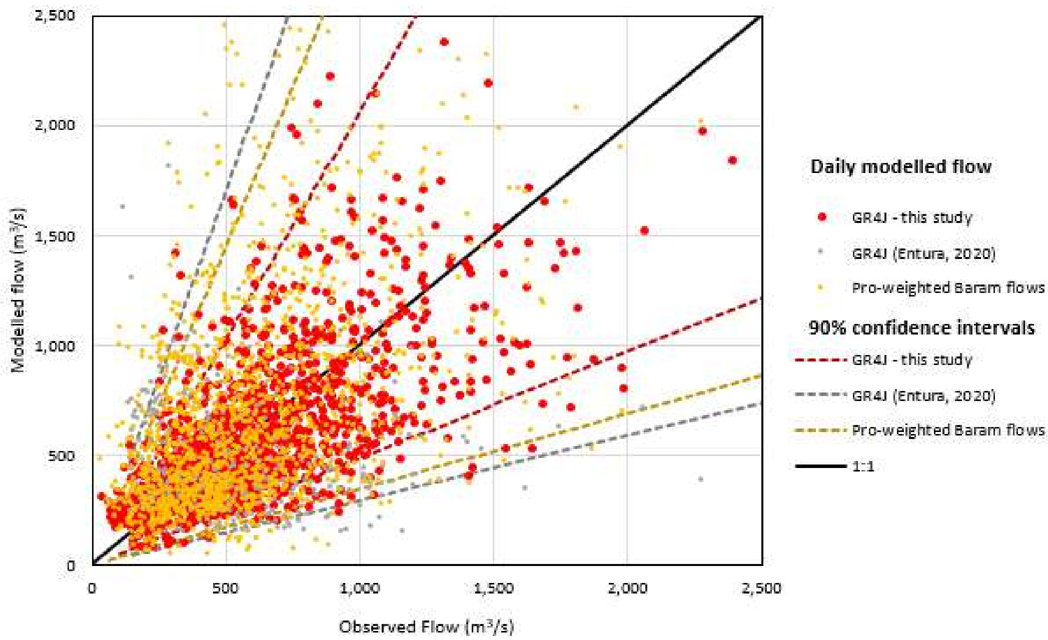

Figure 7. 90% confidence intervals of modelled daily mean flow compared with previous models

Climate change impact on future river flows

Hydrological assessment requires not only a good understanding of historical flows, but also consideration of likely changes in climate and how this will influence future rainfall and evaporation. The forecast changes to rainfall and evaporation relevant to Kalimantan have been applied to the calibrated GR4J model to forecast the likelihood of increases or decreases in flows at Mentarang.

Iqbal and Shahid (2021) investigated the performance of 35 GCMs of CMIP6 and compared against the Aphrodite SSP for mainland Southeast Asia. The results found that *mri-esm2-0*, *ec-earth3* and *ec-earth3-veg* were the most suitable subset of GCMs for rainfall projections in this region with a bias of less than 25%. A number of studies have conducted similar approaches and found that *ec-earth3-veg* worked best in Indonesia and other Southeast Asian countries (Pimonsree and Kamworapan, 2023; Sa'adi and Rohmat, 2022; Hamed and Nashwan, 2023). Bo, et al (2021) argues that *cams-csm1-0* has difficulty modelling seasonal rainfall which is related to El Niño Southern Oscillation (ENSO) events whereas Li and Chen, (2022) claim that *cams-csm1-0* is among the five best performing models (*cams-csm1-0*, *giss-e2-1-g*, *mri-esm2-0*, *access-esm1-5*, and *cesm2-waccm*) for producing a reliable future summer projections in East Asia. A summary of literature review and model performance is provided in Table 3.

Table 3. GCMs performance over South East Asia

Model Name	Performed well	Performed poorly
cams-csm1-0	(Li & Chen, 2022)	(Bo, et al., 2021)
canesm5	(Hamed & Nashwan, 2023)	
cnrm-esm2-1		
ec-earth3-veg	(Sa'adi & Rohmat, 2022) (Pimonsree & Kamworapan, 2023) (Iqbal & Shahid, 2021) (Desmet & Ngo-Duc, 2021) (Hamed & Nashwan, 2023)	
fgoals-g3		(Kurniadi & Weller, 2022)
gfcll-esm4		
Ipsl-cm6a-lr		(Kurniadi & Weller, 2022)
miroc-es2l		
miroc6		
mri-esm2-0	(Iqbal & Shahid, 2021) (Li & Chen, 2022)	
ukesm1-0-II		

Based on a literature review as presented in Table 3, five journal articles agree that *ec-earth3-veg* projects rainfall well, with *cams-csm1-0*, and *mri-esm2-0* performing reasonably well. Therefore, these three GCMs were included in a Multi-Model Ensemble for the MIHEP watershed to explore climate change impacts in the future.

Results of climate change predictions

The results of the future forecasts give a mixed picture with respect to changes in flow. The analysis has been conducted for two future 25-year epochs, 2026–2050 (2030s) and 2051–2075 (2060s), compared to a historical 25-year baseline from 1990 to 2014. The mean modelled flow within this baseline period was 495m^3/s. The predicted percentage changes in flows are presented in Figures 8a and 8b for both forecast epochs. The 90% confidence intervals presented for individual climate models and SSPs (5% to 95%) represent interannual variability, for example due to ENSO.

On average, the ensemble of climate models and SSPs predict an increase in flows through the 21st century. However, it is important to note that the forecasts of individual climate models diverge from each other significantly. Furthermore, these projections focus on annual average flows and do not capture possible seasonal changes in climate variability, such as changes in frequency of El Niño events or the frequency and intensity of extreme rainfall events and dry periods.

Managing Risks for Dams and Reservoirs

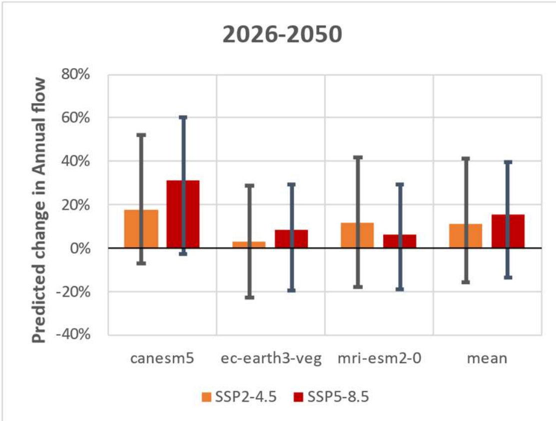

Figure 8a. Predicted change in flow during 2030's, compared to 1990–2014 baseline (Error bars show 90% intervals of interannual variability)

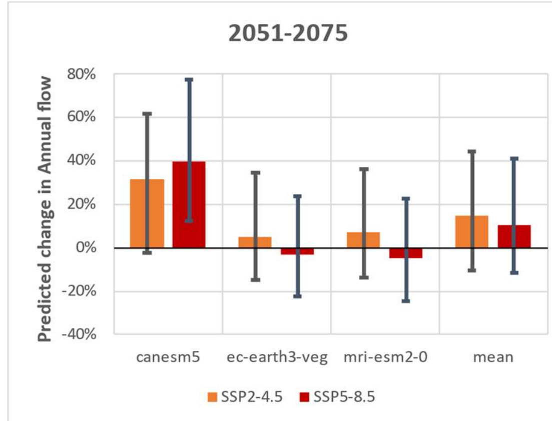

Figure 8b. Predicted change in flow during 2060's, compared to 1990–2014 baseline (Error bars show 90% intervals of interannual variability)

RESERVOIR OPERATION MODELLING

A reservoir operation model was developed in the HEC ResSim tool for the Mentarang Reservoir. HEC ResSim comprises a graphical user interface (GUI) and a computational programme to simulate reservoir operations. It is developed and made available by the U.S. Army Corps of Engineers Hydrologic Engineering Center (HEC). Version 3.3 was used for this study. Reservoir operation control rules were updated as described below and then the model was simulated with updated long-term flows generated during this study to assess firm power and mean annual energy available from MIHEP.

Reservoir Control Rules

During periods of low water levels (<210masl), there is a risk of not releasing ecological flows from the MIHEP powerplant to the downstream river. The spillway crest level is set at 210masl and if the reservoir water level drops below 210masl (minimum operating level), the ecological flow release cannot be discharged over the spillway. Additionally, the spillway is designed in such a way that the minimum discharge should be 750m^3/s on one chute to ensure that the jet from the flip bucket will impact in the plunge pool to minimize erosion. Releasing water via the spillway during low flow periods is also not a sensible decision.

Therefore, the plant operating rules are set to allow a 1m buffer above the minimum operating level (MOL) to pass only ecological flows through the two penstocks and generate minimum power equivalent to the ecological flow release. The following reservoir control rules were adopted in the reservoir operation model as described below.

- Stop all units below 210masl (MOL).
- Generate minimum power (400 MW) between 210masl and 211masl to ensure ecological flow release (225m^3/s).
- Generate firm power between 211masl and 230masl.
- Generate full power and release flood water through spillway between 230.0masl and 237.8masl.

Firm Power

An analysis of power reliability was conducted to understand the firm power available at various reliability levels. Reservoir simulations were performed for the 30-year period January 1993 to June 2023, to assess the changes in reliability of target firm power values, as shown in Figure 9 below. Results from this analysis will help PTKHN to negotiate a power purchase agreement with their potential customers.

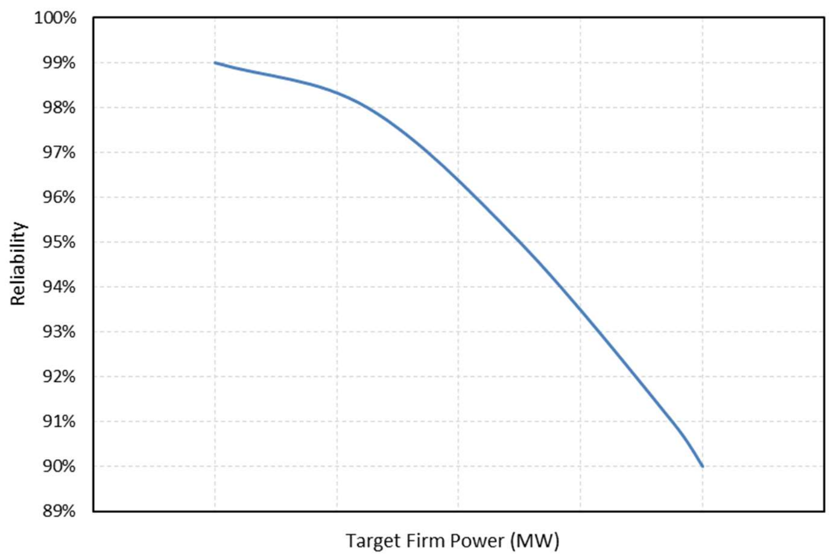

Figure 9. Target firm power vs hydrological reliability
Note: Scale on X-axis redacted due to commercial sensitivity

Figure 10 shows the reservoir water levels for 30-year (historical) operation for 95% target firm power output. It is noted that there would be three events when the reservoir level hits the minimum operating level. Figure 11 shows the MIHEP flow duration plot for 95% reliable firm power dispatch.

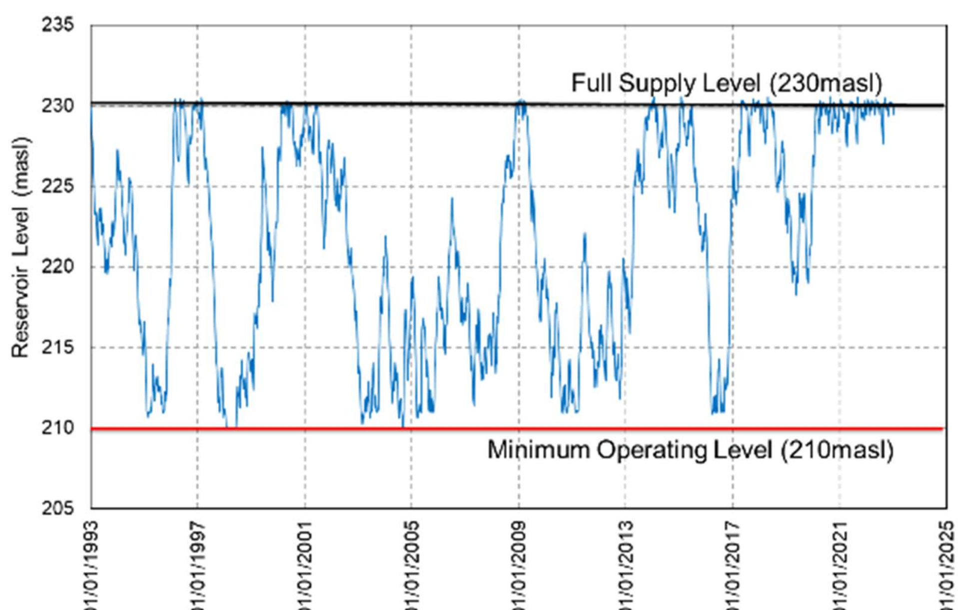

Figure 10. Reservoir water levels for 30 years historical operation

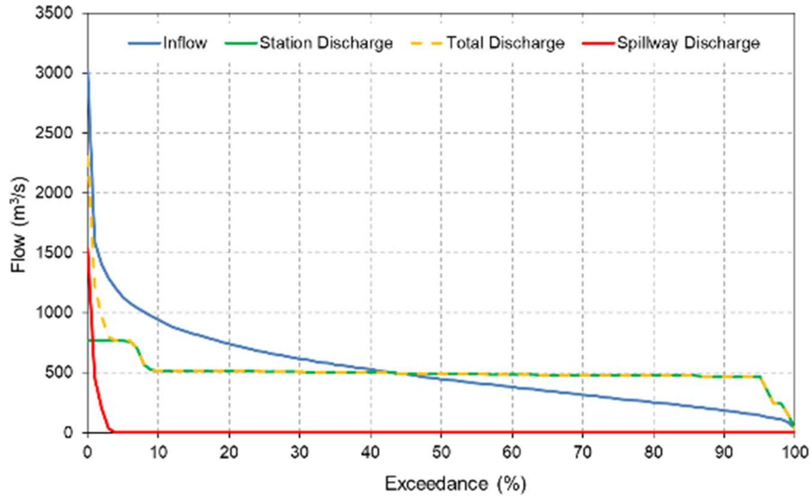

Figure 11. Inflow and outflow duration curves for 30 years historical operation with firm power of 850MW

Reservoir spills were calculated for the 99%, 95% and 90% target firm power operation,

Climate Change Impact on Hydropower

The climate change assessment suggests that overall, there would be an increase in rainfall over the MIHEP catchment in the future, which would result in higher river flow into the MIHEP reservoirs.

In line with the differences in rainfall predictions, the *canesm5* climate model predicts an increase in river flow through time. The other two climate models (*ec-earth3-veg* and *mri-esm2-0*) predict a smaller increase, compared to *canesm5*, in flows in the 2026–2050 window compared to the baseline period of 1990-2014. This assessment suggests that under the mean ensemble of three climate models, there would be 10% to 15% increase in future flows compared with baseline period of 1990-2014.

Therefore, it is projected that the MIHEP would generate more power and annual energy in the future than estimated from the historical flows. However, these findings should be considered with the caveat that climate change assessment and GCMs have high uncertainty in projecting future precipitation and river flows in various regions.

Based on 30 years of historical flows, the plant factor for the designed plant is 63% and existing plant capacity would be adequate for the projected increased flows under the mean ensemble.

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The British Dam Society

PMP - Maximum Precipitation, Probably

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SYNOPSIS Climate change poses significant challenges to the accurate estimation of probable maximum precipitation (PMP), a crucial parameter used in the design and assessment of flood control infrastructure. This paper investigates the potential implications of climate change on current predictions of PMP and its derived parameter, probable maximum flood (PMF). Case studies from Scotland, Wales and England highlight real-world examples of the challenges posed by climate change and the importance of incorporating climate change considerations in PMP and PMF estimations.

INTRODUCTION

Climate change is recognised as one of the most pressing global challenges of our time. Its impacts are widespread (Figure 1), affecting various aspects of the Earth's systems, including the hydrological cycle and precipitation patterns. In the reservoir industry one of the critical concerns related to climate change is its potential implications on the estimation of probable maximum precipitation (PMP). Understanding the potential changes in extreme precipitation events is crucial for effective flood management, infrastructure design, and the protection of vulnerable communities.

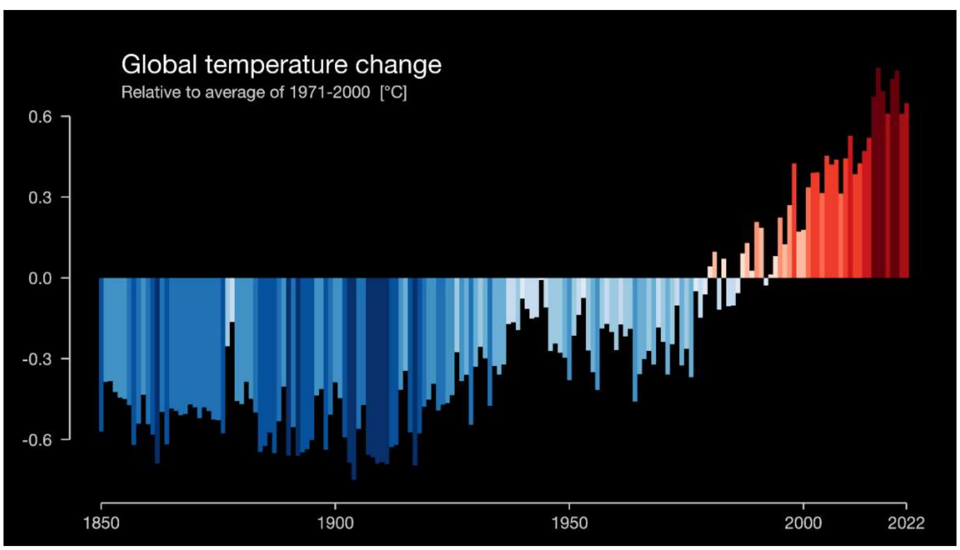

Figure 1. Global temperature change due to climate change

(Graphics and lead scientist: Ed Hawkins, National Centre for Atmospheric Science, University of Reading., National Centre for Atmospheric Science, UoR.Data: Berkeley Earth, NOAA, UK Met Office, MeteoSwiss, DWD, SMHI, UoR & ZAMG)

PROBABLE MAXIMUM PRECIPITATION (PMP)

PMP is defined as the ‘‘theoretical maximum precipitation for a given duration under modern meteorological conditions” (WMO, 2009, p1). Hydrologists use a PMP magnitude to calculate the Probable Maximum Flood (PMF) in the case where the consequence of a dam overtopping is deemed unacceptable.

Probable Maximum Precipitation (PMP) refers to the theoretically maximum amount of precipitation that could occur over a given area within a specific duration. It represents an extreme weather event that is unlikely to occur but is used as a design criterion for high hazard reservoir systems. PMP estimation helps engineers and planners assess the maximum potential flood that a structure needs to be designed to withstand, ensuring the safety and resilience of infrastructure.

The most common methods used to derive PMP are the storm maximisation (hydrometeorological) approach (WMO, 1973 and 2009) and the statistical approach – Hershfield method (1965). The storm maximisation and transposition method requires more site-specific data. Where site-specific data are limited, a statistical method is applied. This method requires annual maximum rainfall series in the region for required storm durations for which the PMP to be estimated. Factors that influence calculations of PMP values are:

- rainfall of intended storm durations,
- temperature,
- relative humidity,
- altitude,
- wind direction,
- dew point temperature, etc.

The prediction of PMP has evolved over time, driven by advancements in meteorology, hydrology, and statistical analysis. Early approaches relied on empirical methods that utilised historical rainfall data and simple statistical extrapolation techniques. However, these methods had limitations in terms of their spatial and temporal representation of extreme precipitation events.

With advancements in computing power and access to more extensive datasets, modern techniques for predicting PMP have emerged. These techniques incorporate more sophisticated statistical models, numerical weather prediction models, and storm transposition methods. They aim to simulate extreme precipitation events by considering the physical processes and atmospheric conditions that contribute to their occurrence.

To calculate the Probable Maximum Precipitation (PMP), one typically follows established guidelines and methods. The specific approach may vary depending on the region and the available data. However, a general overview of the process is shown in Figure 2.

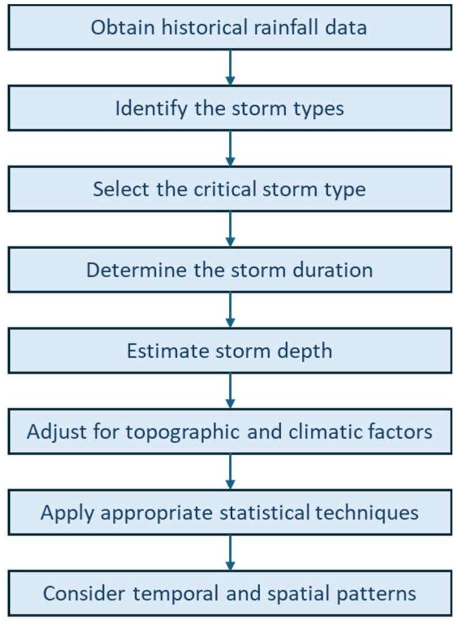

Figure 2. Typical process of PMP determination

PMP is not a probabilistic estimate. It represents a theoretical maximum precipitation value. However, PMP estimation does involve the consideration of probabilities associated with extreme weather events. PMP is probably the maximum precipitation. In applying PMP/PMF, the terminology, nature of the estimation process, and confidence limits need to be understood.

PMP is primarily used as a design criterion for hydraulic structures in flood-prone areas. It provides a basis for determining the capacity and resilience of infrastructure, such as reservoir and spillway systems, to withstand extreme precipitation events.

By considering PMP in the design process, engineers ensure that these structures can safely accommodate the maximum potential flood and prevent catastrophic failures. The accurate estimation of PMP is crucial for protecting lives and property, enhancing the resilience of infrastructure, and enabling effective flood risk management. As climate change continues to alter precipitation patterns and most of the factors used to estimate PMP, understanding the potential implications on PMP becomes increasingly important for ensuring the safety and sustainability of our communities.

PROBABLE MAXIMUM FLOOD (PMF)

In the United Kingdom, the estimation of probable maximum flood (PMF) is an integral part of flood management and the design of reservoir and spillway systems. The Floods and Reservoir Safety (ICE, 2015) Table 2.1 sets guidelines for scale of floods that must be accommodated by spillways depending on the threat posed by the structure. It outlines the recommended standard for determining the maximum flood that a hydraulic structure should be designed to withstand.

PMP serves as a fundamental input for estimating PMF. The relationship between PMP and PMF is established based on hydrological principles and historical flood data. PMP represents the upper limit of potential precipitation, while PMF reflects the maximum flood that could result from that extreme precipitation at any given location.

Managing Risks for Dams and Reservoirs

To evaluate PMF, engineers combine PMP with additional factors such as catchment characteristics, rainfall-runoff processes, and hydraulic routing. These factors help determine how the extreme precipitation would translate into a flood event, considering the local hydrological conditions and the response of the watershed.

The calculation of PMP and PMF involves an analysis of several factors that influence the magnitude and behaviour of extreme precipitation events and resulting floods. The following factors are considered:

- Storm Characteristics: This includes the intensity, duration, and spatial distribution of rainfall associated with the extreme event. Historical storm data and statistical methods are used to estimate the maximum possible storm characteristics.
- Watershed Characteristics: The physical characteristics of the catchment, such as size, shape, topography, land cover, soil type, and infiltration capacity, play a significant role in determining the response of the watershed to extreme precipitation. Hydrological models are employed to simulate the rainfall-runoff processes within the catchment.
- Climatic Conditions: Local climate patterns, including atmospheric moisture availability, prevailing weather systems and snowmelt, are important considerations. Climate data, such as historical rainfall records, are analysed to understand the likelihood and magnitude of extreme precipitation events.
- Hydraulic Routing: Once the flood hydrograph is derived from the combination of PMP and watershed response, hydraulic routing techniques are employed to simulate how the flood hydrograph propagates through the river system. This step allows engineers to determine the flood peak and associated flood levels at various locations downstream.

The estimation of PMF involves uncertainties associated with each factor considered in the calculation. Confidence limits could be assigned to these factors to quantify the range of uncertainty. These limits represent the confidence interval within which the true value of the factor is expected to lie. The confidence limits for individual factors could be determined through statistical analysis, historical data analysis, and expert judgment. By considering the range of possible values for each factor and their associated probabilities, a comprehensive assessment of the uncertainties could be obtained.

The aggregate confidence limit on PMF is a composite measure that accounts for the combined uncertainties from all the factors involved in its calculation. It represents the overall range within which the true PMF is expected to lie, considering the uncertainties in storm characteristics, watershed response, climatic conditions, and hydraulic routing. Micovic et al (2015) assessed the variation in these factors for a dam in British Columbia and found that PMP could be more than 40% higher than the single-value PMP estimate. They recommended presenting PMP as a range within confidence limits as opposed to the single value which implies a, perhaps false, degree of certainty.

The PMP/PMF method differs from probabilistic methods of flood prediction in its approach to extreme events. PMP/PMF represents a deterministic approach that focuses on estimating the maximum potential precipitation and the corresponding flood event. It provides a conservative design criterion to ensure the safety of hydraulic structures.

In contrast, probabilistic methods of flood prediction consider a range of probabilities associated with different return periods or exceedance probabilities. These methods analyse historical data and statistical distributions to estimate the likelihood of various flood magnitudes occurring within a specific time frame.

For example, a 1 in 10,000-year flood event corresponds to a low probability event, like throwing five sixes in succession with a fair die. Probabilistic methods provide a quantitative assessment of the probabilities associated with different flood magnitudes and return periods.

One would expect probabilistic precipitation predictions would asymptotically approach the PMP at the extremes.

PMF is used as a design standard instead of a more extreme probabilistic flood event for several reasons:

- Safety and Risk Management: PMF provides a conservative estimate of the maximum flood that a hydraulic structure needs to withstand. It incorporates safety margins and ensures that the structure is designed to accommodate extreme events with a high level of confidence. This approach helps mitigate the risks associated with catastrophic failures.
- Infrastructure Resilience: Designing hydraulic structures based on PMF ensures their resilience to a wide range of extreme flood events. By considering the upper limit of potential precipitation, engineers can create structures that can handle a significant range of flood magnitudes, providing a level of protection for both the infrastructure itself and the communities downstream.
- Regulatory Compliance: Many countries have regulatory requirements that mandate the use of PMF as a design criterion for flood control infrastructure. Compliance with these regulations ensures that the structures meet the specified safety standards and contribute to overall flood risk reduction efforts.
- Data Limitations: Probabilistic methods rely heavily on historical data for accurate estimation of probabilities and return periods. However, historical data may be limited in duration or quality, especially for rare or extreme events. PMF estimation, on the other hand, provides a more conservative approach that is not solely reliant on historical records, making it suitable for cases where data limitations exist.

CLIMATE CHANGE

Climate change is projected to bring significant changes to the climate of the United Kingdom. The Intergovernmental Panel on Climate Change (IPCC) and other scientific studies provide insights into the potential climate scenarios. While specific projections may vary, some key changes anticipated in the UK include:

- Increased Temperature: Rising global temperatures are expected to lead to warmer conditions in the UK (Figure). This can result in changes in precipitation patterns, evaporation rates, and the overall water cycle dynamics.
- Altered Precipitation Patterns: Climate models indicate that the UK may experience changes in precipitation patterns, including alterations in the frequency, intensity, and distribution of rainfall events. This can lead to more intense rainfall during certain periods and regions, potentially increasing the risk of extreme precipitation events.

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- Sea Level Rise: The ongoing warming of the planet is causing the melting of polar ice and thermal expansion of seawater, resulting in rising sea levels. This can lead to increased coastal flooding and enhanced vulnerability of low-lying areas, particularly during storm events.

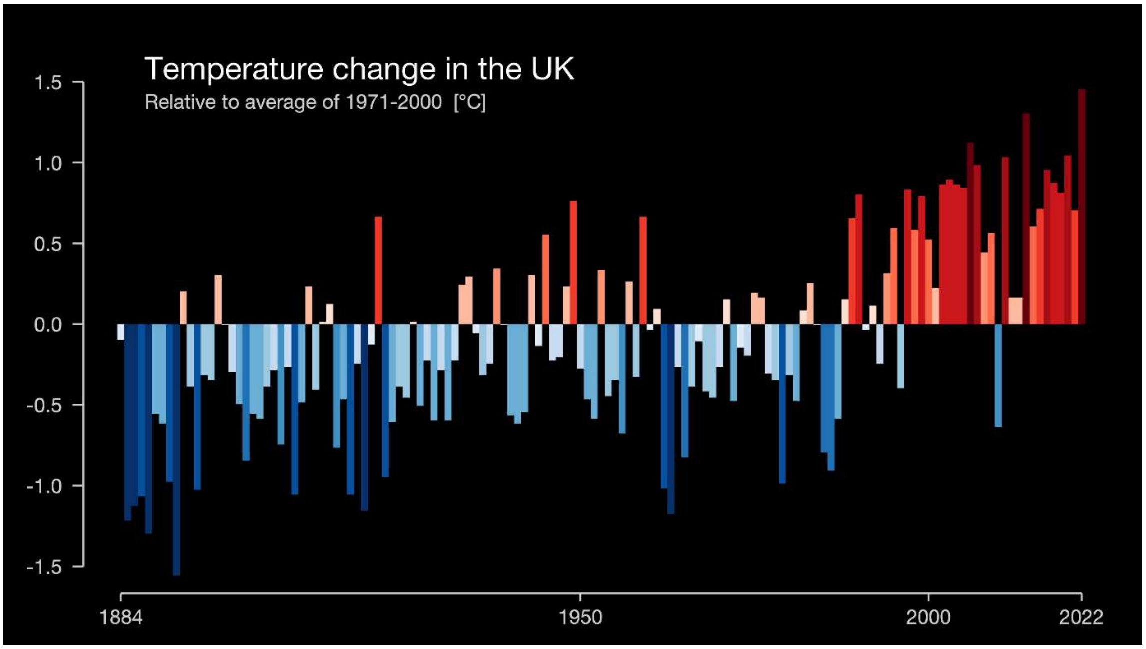

Figure 3. Temperature change in the UK due to climate change

(Graphics and lead scientist: Ed Hawkins, National Centre for Atmospheric Science, University of Reading., National Centre for Atmospheric Science, UoR.Data: Berkeley Earth, NOAA, UK Met Office, MeteoSwiss, DWD, SMHI, UoR & ZAMG)

The estimation of PMP and PMF can be affected by climate change in several ways. Some of the factors considered in their calculation that could be influenced by climate change include:

- Precipitation Intensity: Changes in precipitation patterns may result in altered rainfall intensities. Higher intensity rainfall events can impact the estimation of PMP and subsequently affect the estimation of PMF.
- Rainfall Distribution: Climate change can lead to changes in the spatial and temporal distribution of rainfall. This can impact the design and operation of hydraulic structures as the timing and duration of extreme events may shift.
- Seasonality: Climate change may also influence the seasonality of rainfall, potentially affecting the frequency and magnitude of extreme precipitation events during specific times of the year. This can have implications for estimating PMP and PMF.
- Temperature Effects: Rising temperatures associated with climate change can impact the hydrological cycle, including evaporation rates, soil moisture, and snowmelt dynamics. These temperature-related factors can influence the estimation of PMP and PMF.

It is important to note that the exact nature and magnitude of these climate change impacts on PMP and PMF are subject to uncertainties and depend on regional climate characteristics and specific climate change scenarios.

In the UK, the impact of climate change on reservoirs has been considered in previous studies such as those by:

- Babtie (2002), which found a typical +5% sensitivity in total surcharge level to worst case UKCIP98 projected rainfall and windspeed changes to the 2050s.
- Atkins (2013) referred to an earlier study by Collier (2009) that showed increases in 1-hour rainfall accumulations of 7% for each degree of temperature rise up to 25°C but also found decreases of 8-hour rainfall accumulations with temperature. The Atkins study concluded that "currently research is not robust enough to include as guidance values".

Our understanding is that the ongoing Environment Agency research project (FRS19222) to assess existing methods for estimating PMP and PMF, and to develop new UK methods and guidelines does not include climate change within its remit.

Researchers around the world are also considering the potential implications of climate change on PMP and PMF estimation. There are studies applying climate models to derive updated PMP and PMF estimates for specific reservoir catchments. For example:

- United States: Gangrade et al (2018) tested future climate conditions for the Alabama-Coosa-Tallapoosa river basin and found significant increases in PMF in the near-future (+18%) and far-future (+69%).
- Australia: Visser et al (2022) found evidence of increasing dew point temperatures over the past 60 years with further increases predicted over the coming decades and concluded this is incompatible with the assumption of a fixed PMP. PMP estimates across Australia are predicted to increase by 13%-33% on average by 2100.
- Canada: Clavet-Gaumont et al (2017) considered five Canadian river basins, applied regional climate model simulation results to PMP and snowpack and found increases of up to 20% to future spring PMF. Similarly, in a study of PMP and PMF within Quebec, Rouhani (2016) found increases of up to 25% to the PMF, although reductions of up to 25% were also found for other catchments.
- Malaysia: Sammen et al (2022) estimated increases of 49% (2031-2045) and 123% (2060-2075) to the PMF inflow to a Malaysian reservoir, based on projected rainfall from a regional climate model.
- Chile: Lagos-Zuniga and Vargas (2014) found an increase of as much as 175% to PMF inflows for an Andean reservoir basin in Chile by 2045-2065.
- Japan: Kobayashi et al (2022) described their application of future climate change meteorological model outputs to estimate PMP and PMF for reservoir catchments in Japan.
- Thailand: Jothityangkoon et al (2013) tested climate change scenarios for a large reservoir catchment and found an increase to the PMF of up to 7.5%.

These examples highlight the global recognition of the importance of assessing the impacts of climate change on extreme precipitation events and their implications for flood management and infrastructure design. However, there is currently a lack of strong guidance on how this should be applied for reservoir safety assessments.

Managing Risks for Dams and Reservoirs

Assessing the specific changes in confidence intervals for PMF predictions due to climate change is a complex task that requires comprehensive climate modelling and hydrological analysis. While specific comparisons may vary depending on regional characteristics and climate change scenarios, some general observations can be made.

Climate change can introduce additional uncertainties in estimating PMF due to the uncertainties associated with projecting future climate conditions. The changes in precipitation patterns, intensities, and seasonality add complexity to the estimation process, potentially widening the confidence intervals. However, advancements in climate modelling and downscaling techniques can help improve the accuracy of climate projections and reduce uncertainties. Incorporating climate change scenarios in PMP and PMF estimation can provide a more comprehensive understanding of potential future flood risks and contribute to more robust design and management strategies.

RESERVOIR RELATED FLOOD PREDICTIONS

The consideration of climate change allowance in the estimation of PMP and PMF can have significant implications for flood management and the design of reservoir and spillway systems. Some potential implications include:

- Increased Design Capacity: Incorporating climate change projections in PMP and PMF estimation may require an increase in the design capacity of hydraulic structures. Higher precipitation intensities and altered rainfall patterns may necessitate the construction of larger reservoirs or the modification of existing ones to accommodate the anticipated increase in flood magnitudes.
- Adaptation Measures: Climate change allowance may require the implementation of adaptation measures to enhance the resilience of hydraulic structures. This could include the construction of additional spillways, higher wave walls, the installation of flood control gates, or the implementation of improved monitoring and early warning systems to mitigate the potential impacts of more frequent and intense flood events.
- Risk Assessment and Management: Climate change allowance in PMP and PMF estimation can inform more comprehensive risk assessments and management strategies. It enables decision-makers to evaluate the potential consequences of extreme floods under future climate scenarios and prioritise investments in flood control infrastructure and emergency response systems accordingly.

Climate change poses challenges to the use of past data for probabilistic flood event prediction. Historical data, which forms the basis of probabilistic methods, may not adequately capture the changing climate conditions and the associated shifts in flood patterns.

Climate change introduces non-stationarity, implying that past flood records may no longer provide a reliable representation of future flood probabilities. As the climate changes, the underlying assumptions about the probability distributions and return periods of flood events may become outdated.

To address this challenge, climate-informed approaches are being developed to incorporate projected climate change scenarios into probabilistic flood event prediction. These approaches integrate historical data with climate models and statistical techniques to account for the changing hydrological conditions and provide more robust estimates of future flood probabilities.

CASE STUDIES

To investigate potential climate change impacts on existing reservoirs, specific case studies and examples from Scotland, Wales and England have been developed to highlight the regional implications of climate change on PMP and PMF estimation and flood management practices as described below.

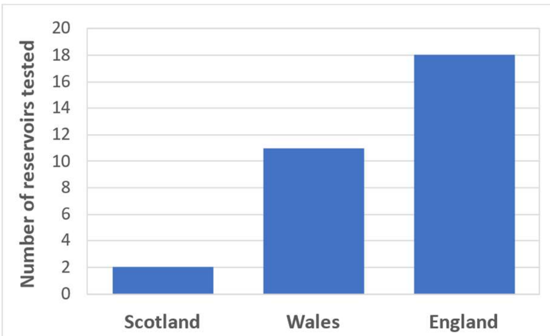

Figure 4. Reservoir locations within case studies

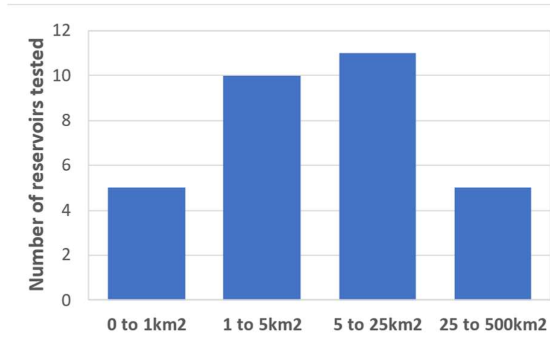

Figure 5. Catchment sizes within case studies

The data used within the case studies is summarised in **Figure 4** to **Figure 7**. These figures show the geographic locations of the reservoirs (**Figure 4**), the catchment sizes (**Figure 5**) and climate change factors applied to rainfall (**Figure 6**6) and runoff (**Figure 7**7).

The approach used for these case studies was to:

- Take a selection of reservoirs for which flood studies had previously and recently been undertaken by Binnies, which could easily be rerun for climate change scenarios. A total of 31 reservoirs was included.
- Include a range of locations, catchment sizes, reservoir sizes and reservoir types.
- Repeat the previous flood routing calculations with climate change allowances applied within the reservoir inflows.
- Test applying climate change allowances in two separate ways. Firstly, applying rainfall allowances to increase PMP and from this re-calculate PMF. Secondly, applying runoff allowances to directly scale the present-day PMF hydrograph.
- Apply glass walls to the dam crest to prevent stillwater overflowing. This is to give a fair indication of how much dam raising would be needed to prevent overflowing.
- Test the PMF – taking the present-day worst case of summer or winter PMFs only.

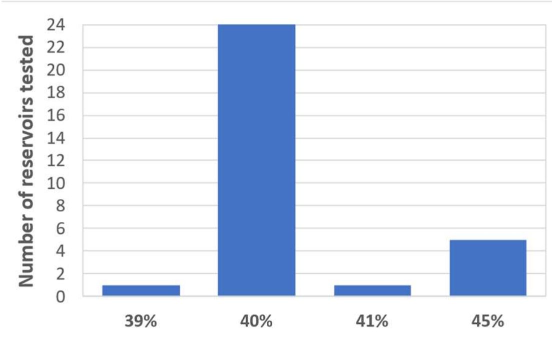

Figure 6. Climate change increase factors applied to rainfall within case studies

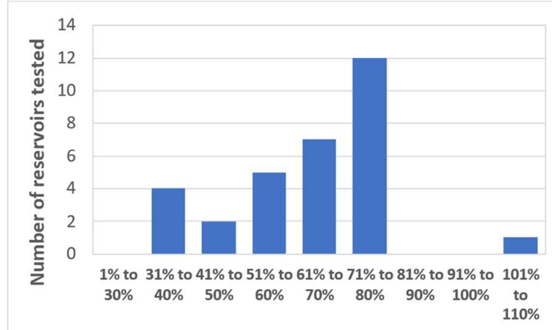

Figure 7. Climate change increase factors applied to runoff within case studies

Climate change was implemented using the allowance factors recommended within current Environment Agency (EA), Natural Resources Wales (NRW) and Scottish Environmental Protection Agency (SEPA) guidance for fluvial flood risk assessment and modelling. This guidance is not intended, or usually used, for reservoir flood studies. We readily acknowledge that the climate change factors used were developed to represent different flood generating mechanisms, but we are using them here in the absence of alternative PMP/PMF specific values.

The climate change guidance documents give different values for different emissions scenarios and timeframes. For this paper, we have used the largest change factors, represented the highest emission scenario and longest timeframe, so as to give an upper estimate for possible climate change impacts based on these allowances.

For Scotland (SEPA, 2023) rainfall and runoff change factors are given for ten river basin regions, for a single emissions case and one time frame (2100). We used:

- Peak rainfall intensity allowances for the year 2100. These are intended for catchments smaller than $30km^2$ but were used for each reservoir for comparison to the other case studies. Rainfall factors range from +35% to +48% across Scotland.
- Peak river flow allowances for year 2100. These allowances are intended for catchments greater than $50km^2$ but were used for each reservoir for comparison to the other case studies. Flow factors range from +34% to +59% across Scotland.

For Wales (NRW, 2021):

- Peak rainfall intensity allowances are provided as Central and Upper estimates for the 2020s, 2050s and 2080s. The same values apply across the whole of Wales. We used the 2080s Upper estimate (+40%).
- Peak river flow allowances are provided as Lower End, Central and Upper End estimates for the 2020s, 2050s and 2080s with three regions defined. We used the 2080s Upper End estimates (ranging from +45% to +75%).

For England (EA, 2022):

- Rainfall and flow datasets can be selected from an interactive map, which gives detailed subdivisions of river catchments across England.

- Peak rainfall intensity allowances are provided as Central and Upper End allowances for the 2050s and 2070s. Different values are given for the 3.3% (1 in 30) and 1% (1 in 100) annual exceedance rainfall events. We used the 2070s Upper End 1% exceedance factors (as the largest value available).
- Peak river flow allowances are provided as Central, Higher and Upper estimates for the 2020s, 2050s and 2080s. We used the 2080s Upper estimates.

Results of the case study flood routings are shown in **Figure 8** and **Figure** 9. In **Figure 8**, the stillwater flood rise with the present day PMP/PMF estimates is compared to the two climate change cases with rainfall and flow allowances applied. In **Figure** 9, the applied climate change peak flow allowances are plotted against the percentage increase in stillwater flood rise. There is little to be drawn from an equivalent plot of rainfall intensity allowances given that very similar change factors were applied to all the reservoirs.

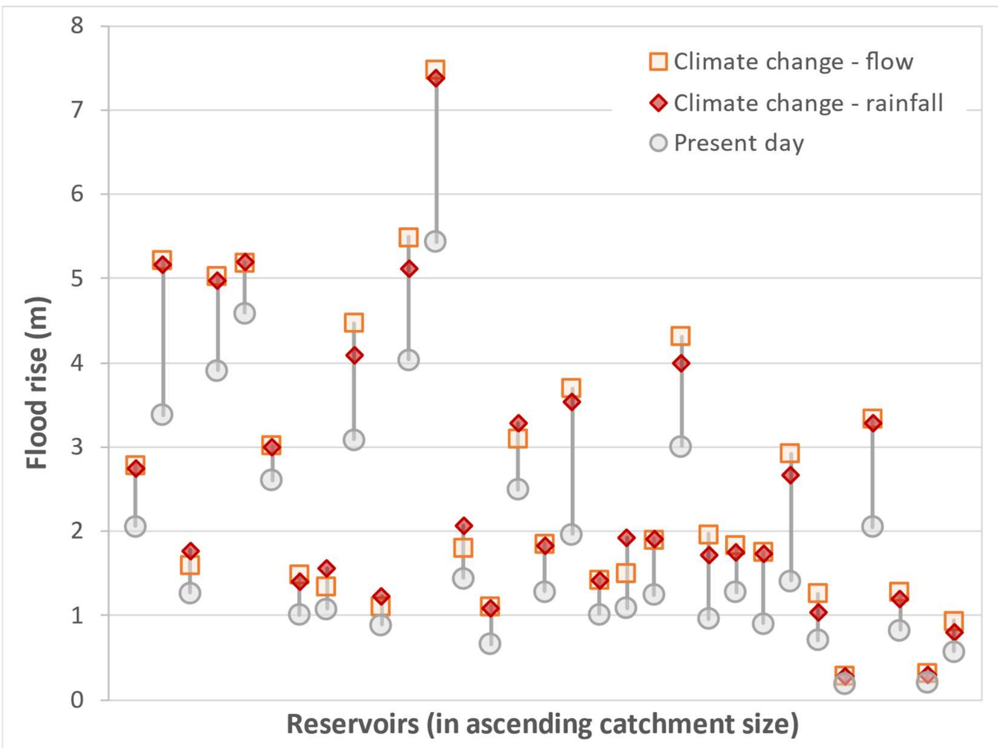

Figure 8. Case study results – impact on stillwater flood rise

The main findings from these case studies are that:

- The impact on stillwater flood rise from applying the peak rainfall intensity allowance or the peak flow allowance is generally similar. On average, the flow allowance gives slightly larger increases, but this is not the case for all locations.
- To quantify the predicted changes:
    - Rainfall intensity allowance gives a minimum increase of 0.09m, maximum increase of 1.94m and average increase of 0.74m.
    - Flow allowance gives a minimum increase of 0.10m, maximum increase of 2.04m and average increase of 0.78m.

Managing Risks for Dams and Reservoirs

- The changes are significant:
    - At 6 of the reservoirs, the increased stillwater flood rise is enough for the dam to overflow, when it does not in present day conditions.
    - At 12 of the reservoirs, the available wave freeboard would be significantly reduced.
- At the other 13 reservoirs tested, the present day PMF peak stillwater level was already above the minimum dam crest level.
- There is not a consistent relationship between the increase in peak flow to the increase in flood rise (**Error! Reference source not found.**). This depends partly on the overflow arrangements at each reservoir:
    - Where there is an undrowned spill weir, the increase in flood rise will be less than the peak flow factor.
    - Where there is a constraint on the outflow, such as a culvert structure or bridge over the spillway entrance, the increase in flood rise can be higher than the peak flow factor.

We again note that we used the highest climate change allowances from the guidance. These are upper end estimates for the end of the century. In the shorter term, the recommended factors are smaller. However, these could still lead to significant reductions in the wave freeboard available at some of these reservoirs.

These case studies demonstrate that applying standard climate change allowances, which are widely used in fluvial flood risk assessment, to reservoirs for the PMP/PMF, results in significant increases to predicted stillwater flood rise. If climate change allowances were required within reservoir flood studies, it would inevitably result in many spillways no longer being able to fully discharge the PMF without dam overflowing or significant wave overtopping.

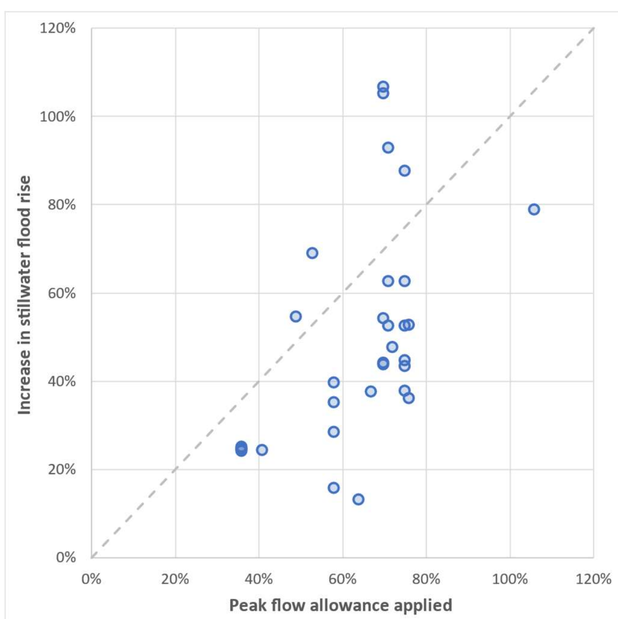

Figure 9. Case study results – peak flow allowance compared to stillwater flood rise increase

CONCLUSIONS

The factors considered in PMP and PMF calculations, including precipitation intensity, rainfall distribution, seasonality, and temperature effects, will be influenced by climate change.

Incorporating climate change allowances in PMP and PMF estimation is crucial to ensure the resilience of hydraulic structures in the face of future climate conditions. Our case studies for UK reservoirs using current flood risk climate change guidance indicate a typical increase in PMF stillwater flood rise of around 0.75m by the end of the century with the upper end emissions scenarios.

Researchers in many countries around the world, including the United States, Australia, and Malaysia, are actively considering the impacts of climate change on extreme precipitation events and assessing the impact on PMF predictions.

More research is required to understand confidence intervals for current PMF predictions even before uncertainties around climate change are introduced. While climate change introduces uncertainties in estimating PMF, advancements in climate modelling techniques and downscaling methods offer opportunities to enhance the accuracy of climate projections and reduce uncertainties.

The implications of climate change allowance for PMP and PMF include the potential need for increased design capacity, adaptation measures, and comprehensive risk assessment and management strategies.

Climate change also challenges the use of past data for probabilistic flood event prediction, emphasising the importance of climate-informed approaches that integrate historical data with climate projections.

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The British Dam Society

Numerical simulation and assessment of a clay embankment dam experiencing climate-induced deformation

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SYNOPSIS Thames Water's Reservoir Safety Group noted movement in the south-eastern slope of the Stoke Newington (East) Reservoir embankment, deemed to be excessive for the relative size of the embankment and showing a marginal increase in settlement rate over time. By modelling the climatic conditions as a boundary condition within a Finite Element and Finite Difference Analysis model and simulating the periods for which measurement is available, the mechanism of deformation within the embankment could be identified. PLAXIS was found to be useful for setting up boundary conditions to simulate the fluid-mechanical conditions within the embankment, but the software does not accurately translate this behaviour into representative stress and strains. FLAC was used as an alternative to model these conditions using the saturation profile from PLAXIS as a starting point. The deformation predicted by FLAC shows a good correlation with the monitoring results available, allowing the asset owner to forecast the strains to be developed in the embankment in the future and to set up inclinometer trigger levels for monitoring the asset. With this information in hand, together with the knowledge that climatic boundary conditions are due to worsen with global warming, a solution was later developed to mitigate the risk of embankment instability.

INTRODUCTION

The influence of climate on clay earthfill embankments has been well documented since Walbancke and Vaughn (1976), in their seminal study, illustrated how climate has a more significant effect on pore pressure changes within the downstream slope than the reservoir itself. Furthermore, seasonal ratcheting in active clays has been documented, particularly in railway cuttings in stiff London Clay, e.g. Skempton (1977), and many studies have investigated strain softening behaviour in this material e.g. Potts et al (1997).

This behaviour and resulting deformation can only be adequately captured using Finite Element or Finite Difference Analyses (FEA or FDA). The use of FEA/FDA to analyse deformations in earth embankments is often undertaken using complex constitutive models. These complex constitutive models are typically outside the realm of routine analysis due to time and budgetary constraints of asset owners. In these circumstances, a simple constitutive model undertaken using ubiquitous FEA/FDA software such as PLAXIS or FLAC is a useful tool for dam engineers. Once the mode of deformation is understood, an effective construction solution can be developed.

BACKGROUND

Stoke Newington (East) Reservoir

Stoke Newington (East) Reservoir was constructed in 1833 as a non-impounding reservoir that draws water from the adjacent New River, which was constructed in the 17th century. The reservoir comprises an approximately 7.5m high homogenous clay embankment. The site, shown in Figure 1, is in north London on former agricultural land comprising London Clay.

Figure 1. Stoke East Newington Reservoir with area of highest settlement circled © Google (2024).

The asset owner noted movement in the south-eastern embankment of the reservoir and as per Figure 2, seasonal behaviour has been noted, with shrinkage and swelling behaviour observed in summer and winter respectively. These movements, recorded through surveys of surface-level monitoring pins, installed in 1999, have been deemed to be excessive for the relative size of the embankment and have shown a marginal increase in settlement rate over time. Settlements within the area of interest highlighted in Figure 1 averaged between 4mm/year and 15mm/year between 2004, when a new temporary benchmark was set up, and 2021 at the time of the investigation.

After a critical review of the monitoring results, it was suspected that the frequency readings, and ultimately phreatic surface output, of the Heavy-Duty Vibrating Wire Piezometers (HDVWP) were being affected by a mechanism separate from, or perhaps in addition to, seepage from the reservoir. As such, Thames Water's Reservoir Safety Group, on the advice of the Panel Engineer, requested that further analysis be undertaken to ascertain the mechanism causing movements within the embankment and to assess global stability.

Figure 2. North-eastern portion of the embankment with shrinkage of clay in summer creating cracks in the crest path, looking north-east.

METHODOLOGY

A phased approach to the problem was undertaken, starting with limit equilibrium and one-dimensional consolidation approaches followed by a more complex FEA/FDA analysis of deformation. The same philosophy was applied to the ground investigation (GI), initially obtaining GI related to total strength and thereafter obtaining a greater density of data providing deformation, permeability and effective strength parameters required to formulate a constitutive model.

Model geometry

The south-eastern embankment, with a maximum height of 7.5m, was divided into five distinct materials for the analysis, as shown in Figure 3. High-strength reworked London Clay (HSMG) recorded between 2.5m and 3.5m depth is typically described as firm to stiff and friable. Desiccation cracks, decreasing in prevalence with depth, were noted in this material during the ground investigation. Although the overconsolidated London Clay was remoulded during construction, the construction method and subsequent desiccation have resulted in high effective stresses (from negative pore water pressures) causing overconsolidation.

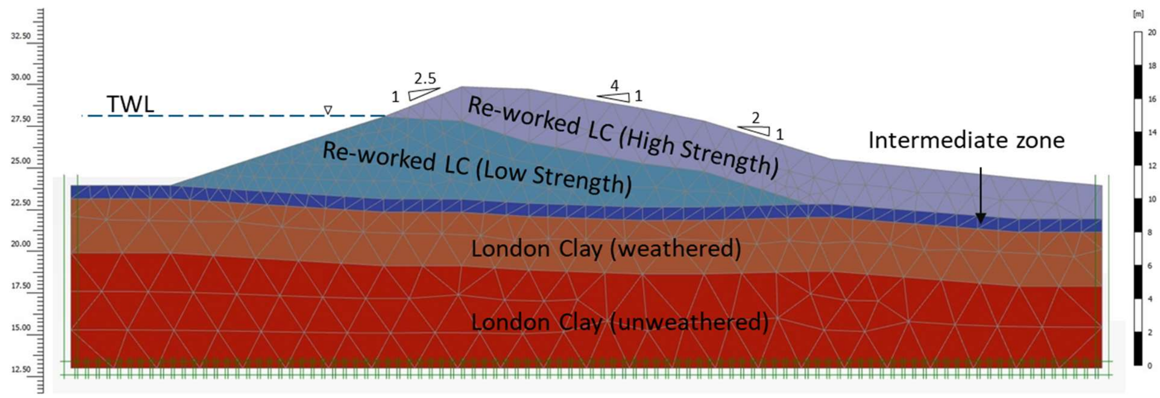

Figure 3. Cross section at CH 273 analysed in PLAXIS and FLAC.

Below this material, soft, low to medium strength reworked London Clay (LSMG) was recorded to the base of the embankment at depths between 6.0m and 7.5m. The softened material corresponds with the material beneath the steady-state phreatic surface and overlying capillary zone. The high swelling pressures recorded in laboratory swelling tests indicate that the soft material could swell beneath the phreatic surface under the prevailing overburden pressures.

An intermediate layer of London Clay between 0.1m and 1.0m thick (based on CPTu records) is located directly beneath the embankment. This material has a similar composition to in-situ London Clay, although it is suspected that it has become less stiff by reworking or swelling due to past site activities such as agriculture (tilling) and climate, respectively.

Weathered London Clay, typically comprising overconsolidated silty clay, forms the foundation material to a depth of between 10m and 11m where unweathered material of similar composition was encountered.

Model Conditions

The ability of FEA and FDA to simulate the behaviour and deformation of geotechnical structures is helpful in such an assessment, where moisture changes within the embankment, influenced by seepage from the reservoir as well as from climatic controls, have significant influence on the behaviour of plastic clays. The first step in such an analysis is to develop a constitutive model which can represent the behaviour of the soil in a mathematical framework.

The FEA software PLAXIS 2D version 2023.2 was used for the analysis of this project, whilst the FDA program FLAC version 8.1 was used together with PLAXIS to check and improve upon deformation results. Two-dimensional analysis is considered to be appropriate in this context, as plane strain conditions can be assumed when analysing linear structures such as embankments.

Constitutive model

Because the stress-strain behaviour of soils is highly non-linear and stress-dependent, a single stiffness modulus is not sufficient to accurately reflect the deformation changes that take place prior to shear failure, the normalised stress-independent stiffness formulation presented as part of the Hardening Soil constitutive model, first presented by Schanz, Vermeer, & Bonnier (1999), was adopted for the methodology used to estimate the stiffness properties of the materials used in analysis in both PLAXIS and FLAC.

The input parameters for this model for a reference stress of 100kPa were obtained from a combination of laboratory data, empirical correlations, and CPT records. A summary of the key parameters defining the constitutive model is provided in Table 1.

Table 1. Summary of input parameters for the constitutive model

Material type	Bottom Depth*	p_{ref} (kPa)	E_{50}^{ref} (kPa)	E_{oed}^{ref} (kPa)	E_{ur}^{ref} (kPa)	Power (m)	v_u	k_0^{nc}
High Strength Made Ground (HSMG)	2.5	100	31.0 x 10^{-3}	30.8 x 10^{-3}	93.0 x 10^{-3}	0.80	0.495	0.65
Low Strength Made Ground (LSMG)	6.9	100	38.0 x 10^{-3}	31.0 x 10^{-3}	114.0 x 10^{-3}	0.90	0.495	0.65
Intermediate layer (reworked London Clay)	7.7	100	39.0 x 10^{-3}	32.0 x 10^{-3}	117.0 x 10^{-3}	0.90	0.495	0.64
Upper (weathered) London Clay	10.9	100	41.0 x 10^{-3}	34.0 x 10^{-3}	123.0 x 10^{-3}	0.80	0.495	0.66
Lower (unweathered) London Clay	17.0 **	100	46.0 x 10^{-3}	38.0 x 10^{-3}	138.0 x 10^{-3}	0.80	0.495	0.66
Source	Explora-tory holes		Correlation with CPTu results (Lunne, et al., 1997) checked against laboratory tests					

* (mbgl) from crest road ** not proven

Analysis

Boundary restraints are no different from a typical deformation analysis with Y_{max} set as a free boundary, Y_{min} fully fixed and the X_{max} and X_{min} boundaries normally fixed; these are shown graphically in Figure 3. To correctly model movements associated with seasonal swelling and shrinkage, a discharge boundary condition is the most appropriate approach to drive seasonal transient pore water pressure cycles, and therefore stress cycles, to produce the most realistic displacements associated with seasonal wetting and drying. The built-in *Climate* function in PLAXIS, called *Precipitation*, is a useful discharge boundary tool, as runoff is automatically activated on the relevant boundary once the soil becomes fully saturated.

The boundary conditions are modelled separately for winter and summer, with winter conditions assumed to occur over five months and summer over seven months in line with findings by Posthill (2018). The model was run for ten annual cycles, simulating the period between 2010 and 2020. Winter conditions are modelled by a uniform discharge function representing the mean rainfall over the relevant season with parameters constrained to a maximum 0.1m head on the boundary and a minimum -1.0m head associated with the maximum height of water on the slope before runoff and 10kPa suction, respectively.

The measured rainfall was obtained from the closest weather station to the site i.e., Hampstead weather station (http://nw3weather.co.uk/wxdataday.php).

The summer boundary condition, i.e. net evapotranspiration, is represented by a negative discharge which has been calculated as a daily incremental function of the total soil moisture deficit (90mm at the end of summer) of a published study in similar conditions (Smethurst et al, 2012). The constraints on this function correspond with a 40kPa suction at the end of summer (min head of -4.0m) and runoff as with winter, i.e. maximum head of 0.1m. These boundary conditions are informed by previous studies on active clays in London (Posthill, 2018). A relaxation phase is created after each evaporation to view the stresses and deformations developed each year before the next cycles are modelled.

A fully coupled fluid-mechanical analysis is run for each phase; each winter season is run for 150 days, whilst each summer season is run for 215 or 216 days, where applicable. After the flow net and suctions are set up in the embankment, the pore pressures defining each phase are progressively taken from the previous phase.

RESULTS

The initial results of the transient pore water changes within PLAXIS showed positive results; the saturation profile shown in Figure 4 indicates a good interplay between boundary conditions and internal fluid-mechanical conditions within the model.

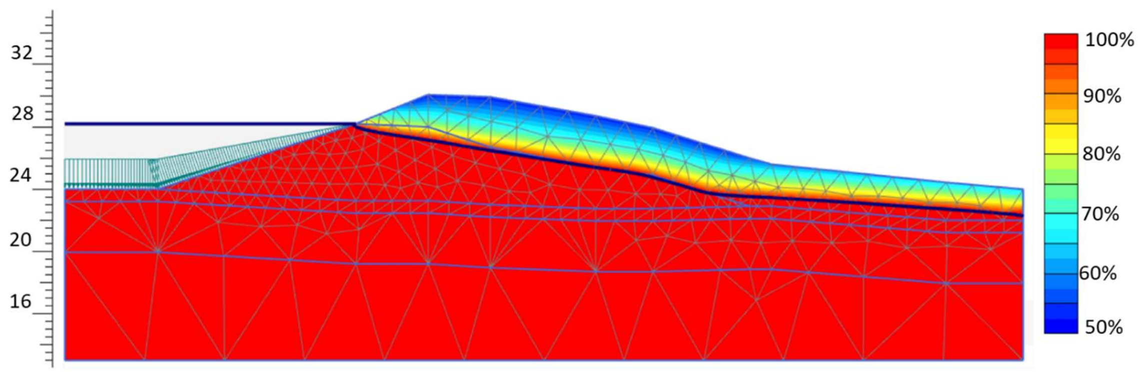

Figure 4. Typical saturation profile at the end of summer.

However, the way in which these (pore water) changes influence stress and deformation cycles in PLAXIS was immediately picked up as a potential shortcoming for this mechanism of cumulative deformation. A decision was made to undertake a check of the results in FLAC. To ensure parity between the two software codes, the saturation profile from PLAXIS was replicated in FLAC. The results of the deviatoric strain and displacements are provided in Figures 5 to 8.

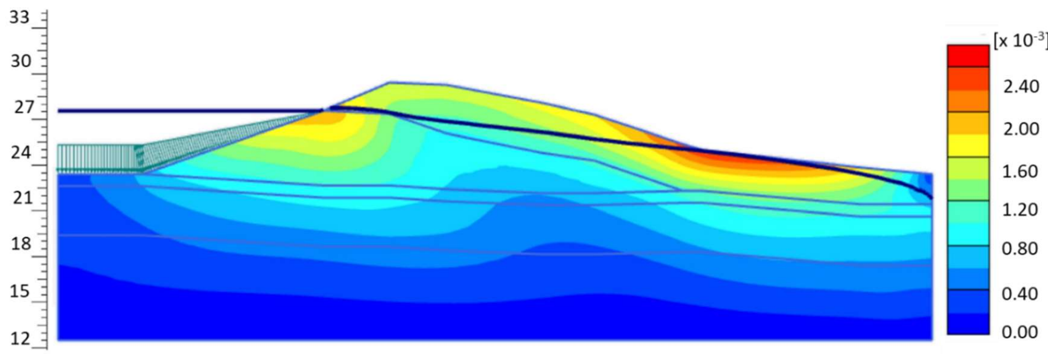

Figure 5. PLAXIS output showing total displacements (metres) after 10 years.

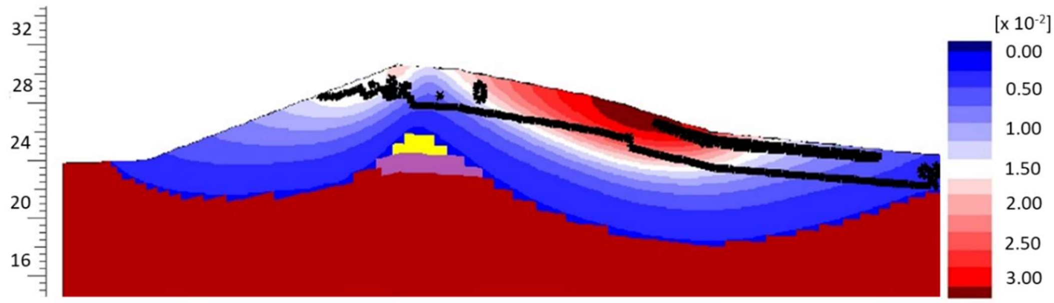

Figure 6. FLAC output showing total displacements (metres) after 10 years with material boundaries in background as per Figure 3 and phreatic surface shown in black.

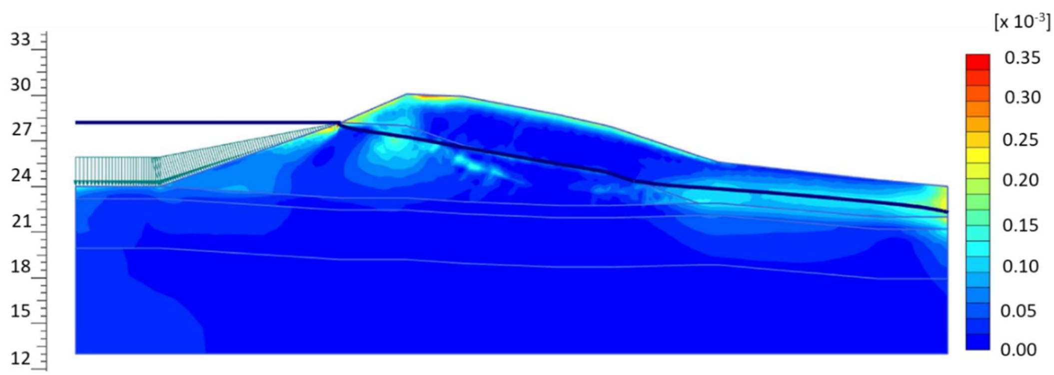

Figure 7. PLAXIS output showing shear strain after 10 years (end of Summer).

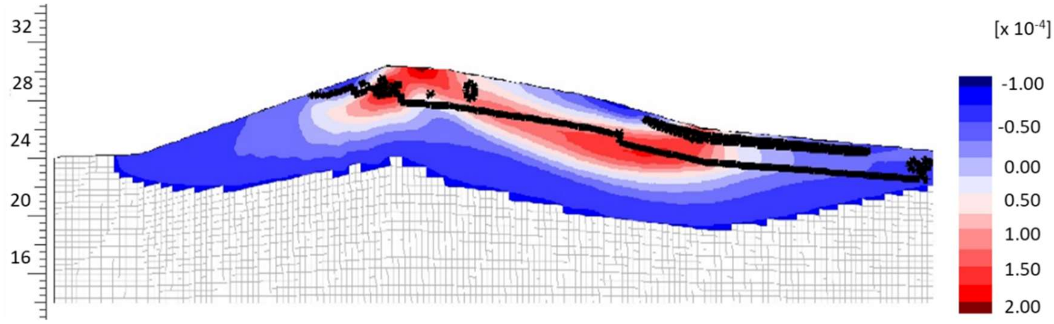

Figure 8. FLAC output showing shear strain after 10 years and phreatic surface shown in black.

DISCUSSION

Both the deviatoric strain and deformation results in FLAC immediately appear more realistic for a ratcheting profile when compared with the PLAXIS results. PLAXIS results showed no progressive stress or strain propagation over time with the formulation seemingly limiting the progression of these even where strain reset functions are disabled. Settings in the FLAC formulation tend to allow the progression of stress and strain over time without hindrance and allow the constitutive model to be influenced by stress levels in a more realistic manner for this mode of deformation.

The two most significant influences in this contrast in results are speculated upon. Firstly, within PLAXIS, the stiffness regains its maximum value for a stress level when the direction of loading changes. FLAC's formulation appears better at accounting for both stress and direction changes, allowing more nuanced stiffness degradation behaviour to be captured. Secondly, the finite difference method by default recalculates the stress-strain and strength conditions per node in a large strain environment whilst PLAXIS is formulated to be more efficient with small strain.

Any further discussion on the back-end formulation differences between the two software codes will be left for a different forum, as the purpose of this paper is to present a working solution for dam engineers and guidance on the appropriate software to use. It should also be noted that the software support team from PLAXIS was consulted about the limitations described and provided helpful support, but no working solution was found at the time. Future versions of PLAXIS, or the current version with a user defined model and manual alteration at each phase, may well solve the limitations noted here but this too is beyond the scope of this paper.

A comparison of Figures 5 to 8 with inclinometer results shown in Figure 9 shows the formulation procedure in PLAXIS is limited in practically representing the strain and deformation elements of this embankment's behaviour. The base of the shear zones downslope of the crest at approximately 3m depth noted in Figure 8 align well with the zone of general downward movement recorded in Figure 9. As the FLAC displacements and strains better match the inclinometer displacement the FLAC model was selected for interrogation to provide further insight for the project.

Displacements

Total displacements at the end of the end of the 10-year cycle are shown in Figure 6. FLAC predicts a displacement towards both faces of the embankment. Although observations on site tend to agree with the findings of the FLAC analysis, i.e. hummocky ground indicative of shallow movements mid-slope on the downslope face and a wave wall which has rotated at the top of the upstream face, it is considered likely that only regions above the phreatic surface on the upstream slope are to be significantly affected by seasonal ratcheting.

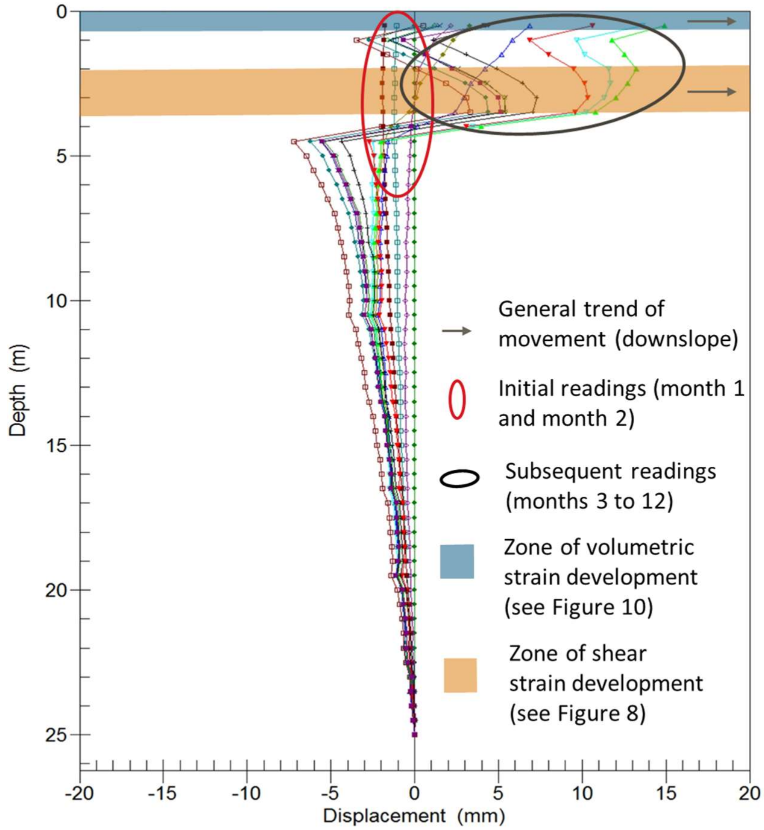

Figure 9. Cumulative inclinometer results between initial installation on 2nd July 2020 and 23rd June 2021 for the embankment section under investigation.

Volumetric strains

Volumetric strains resulting from 10 cycles (10 years) are presented in Figure 10. FLAC predicts volumetric strains concentrated just above the phreatic surface under the downstream face. The volumetric strains calculated in the FLAC analysis are the results of effective stress changes due to changes in pore pressures. The pore pressure response is not fully recoverable at the end of each season but is rather 'stored' within the embankment with pore pressures close to the surface boundary recovering quicker. A general downslope movement is initiated at shallow depths, resulting from seasonal movement along the free boundary and aligns with the inclinometer movement noted in Figure 9. This behaviour illustrated by the FLAC analysis is considered to be in close agreement with previous studies, e.g. Potts et al. (1997).

Figure 10. Volumetric strains resulting from 10 cycles.

Potential shear plane development

The magnitude of strains shown in Figure 8 is important as previous studies have shown that at strains of approximately 20%, residual strengths in London Clay typically begin dictating shear resistance (Skempton, 1977; Potts et al, 1997). Reworked London Clay typically requires higher shear strain to engage residual strength. However, as the mechanism is progressive, these strains will likely accumulate and propagate through the slope until a slip surface dictated by residual strengths is developed.

Based on the results of nineteen cycles, representing ten years of seasonal fluctuations, it is predicted that the amount of shear strain developed within the slope amounts to 8% in the downstream slope, developing from the toe then upslope. Although strain is also generated in the upstream slope, here the model differs from reality as the reservoir level and capillary zone restricts pore water fluctuations significantly. This was not considered an issue for the purposes of the study, but further controls on the spatial distribution of materials experiencing two-phase flows or pore pressure changes can be introduced in scenarios where upslope deformation is being assessed.

The strain values could be used in conjunction with a limit equilibrium safety map to assess the implication of residual conditions on the stability of various slip surfaces, in some cases with factors of safety below unity, typically at shallower depths. The progressive nature of strain development in active clays together with these results informed the All Reservoirs Panel Engineer (ARPE) that a measure in the interest of safety (MITIOS) was required to reduce the risk of embankment instability and works are being progressed.

Whilst these works are being progressed, the results have also been used by the ARPE to establish trigger levels for inclinometer readings which correspond with various increments of strain. These provide the Supervising Engineer with a practical means of monitoring the asset.

Settlement prediction

The end-of-cycle settlements for each annual simulation are provided in Table 2. It is apparent that the FLAC estimations follow a progressively increasing magnitude which aligns with the observations made by the asset owner.

Table 2. End-of-cycle yearly crest settlement estimations predicted by FLAC

Year	1	2	3	4	5	6	7	8	9	10
	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019
Settlement (mm)	1.5	2.6	3.9	5.1	6.4	7.8	9.3	10.8	12.4	14.1

The best-fit trend lines for the set of estimations from FLAC were then calculated based on 200 years' worth of winter and summer cycles and are presented in Figure 11 below.

Actual settlement records obtained from surface monitoring pins (CH300 and CH200 in Figure 11) were also provided by the asset owner.

The settlement records between 2010 and 2020 are plotted on the FLAC predicted settlement line using 2010 as the benchmark for the first reading. This time range corresponds to a period of 177 – 187 years after the completion of the embankment (in 1833).

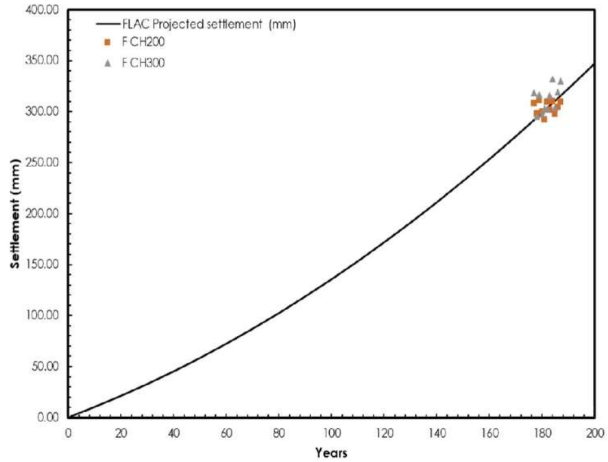

Figure 11. Retroactive projected settlement of FE analysis vs normalized predictions from settlement monitoring at CH200 and CH300 (normalised to 2010).

It is evident that the trend and magnitude from actual records fits quite closely to the FLAC predictions from 200 simulations. The FLAC curve implies that since 1833 approximately 330 to 340 mm of settlement took place that was induced by seasonal boundary effects. It is also clear that the amount of settlement is increasing over time.

The FLAC 200-year simulation in Figure 11 is a close approximation when compared with settlement data over the last 10 years i.e. 35mm simulated compared with 36mm and 150mm for both nearby settlement monitoring points.

We do not have measurement data from the last 183 years available and the current measurement shows clear spatial variability. This is not unexpected considering the age of the asset with different loading histories across the length, the period in which it was constructed and the variability of vegetation and surface cover across the embankment.

In light of this uncertainty, the level of correlation between simulated and measured results is considered acceptable for verification of the accuracy of deformation simulation.

Other possible mechanisms

The study has confirmed that consolidation resulting from fluctuating reservoir water levels as provided by TWUL is unlikely to contribute significantly to the current deformation of the embankment. Consolidation occurring after embankment construction would be considered to be of a similar order of magnitude to those currently experienced i.e. in the order of 300mm, depending on the initial compaction achieved and associated void ratios. However, displacements resulting from consolidation decrease with time and are expected to be largely complete 187 years post-construction, based on CPTu interpretation of consolidation parameters. Current displacements resulting from consolidation are likely to be negligible in comparison with total settlements currently recorded and those associated with seasonal deformation.

CONCLUSION

The results of this study have shed light on the mechanism causing the significant settlement recorded at Stoke Newington (East) Reservoir over the last 20 years and experienced by the embankment since construction in 1833. PLAXIS has been useful in creating a link between climatic boundary conditions and pore water response but is limited in modelling the cumulative stress and strain response to these changes. It is suggested for future works in embankments made up of active clays and potentially experiencing climate-induced deformation, that FLAC is used as the simulation tool to understand behaviour. This would ensure continuity from start to finish in the modelling process.

The FLAC analysis has shown the development of shear strains at locations in line with observed inclinometer records. The level of correlation between modelled and recorded crest settlement is considered acceptable for verification that the FLAC model accurately represents the observed deformation behaviour.

The FDA (FLAC) software, through its inbuilt computational process, has captured the progressive nature of deformation and strain propagation within the embankment. This has provided insight into the reasons for significant settlements which in places have been observed to increase over time. This occurrence is likely to result in residual strength conditions in areas where significant shear strain has developed.

The results have also been used to establish trigger levels to provide the Supervising Engineer a practical means of monitoring the asset.

Through an understanding of the amount of deformation and strains which have and are likely to continue developing within the embankment, a general timeframe for the generation of residual conditions along shear planes can be understood. It should also be noted that a study by Posthill (2018) has found that the phenomenon of seasonal ratcheting is being expedited by climate change.

This information on the mechanism, potential timing and further changes to the climate-induced deformation allows the asset owner to make informed decisions on possible remedial measures.

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The British Dam Society

The use of vibrating wire piezometers to measure matrix suction in dams

R MONROY, Klohn Crippen Berger

SYNOPSIS A knowledge of pore water pressures in embankment dams and in mining dams is essential to monitor performance. In many instances, this knowledge forms part of a critical risk control to prevent a high consequence event, such as global instability and release of containment. Yet the field measurement of pore water pressures can be difficult. This is particularly the case when unsaturated conditions prevail for long periods. Vibrating wire piezometers are used in many instances to monitor negative pore water pressures in dams, both in the foundation and in the fill; yet these instruments, which can measure small sub-atmospheric pressures, have not been designed to operate in an environment of sub-atmospheric pressures indefinitely. This paper touches on two topics that are of interest to the dam engineer: (i) the effect of degree of saturation and matrix suction on liquefaction potential, and (ii) the measurement of matrix suction in the field using vibrating wire piezometers.

INTRODUCTION

A knowledge of the amount of pore fluid (soil-water wetness) or the pressure within the pore fluid (soil-water potential) in a soil-fluid system is needed to predict the performance of a geotechnical structure. This is particularly the case for mining dams located in seismic areas, where a sufficiently low volume of pore fluid, or a sufficiently low pressure within the pore fluid, is used to manage the risk of liquefaction during an earthquake. Although this statement also applies to embankment dams founded on potentially liquefiable deposits, it is most relevant in the case of mining dams that often rely on the strength of the deposited tailings for stability. In many instances, the tailings, which consist of fine sand and silt grains derived from the grinding of ore, will have been deposited as a slurry. Densities may be low, and both the soil-water wetness and the soil-water potential within the tailing mass will govern their potential for liquefaction.

FIELD MEASUREMENT OF SOIL-WATER WETNESS AND SOIL-WATER POTENTIAL

The amount of pore fluid in a soil can be expressed as a gravimetric water content (ratio of water mass to dry soil mass), volumetric water content (ratio of water volume to total soil volume), or degree of saturation (percentage of void space that contains water). Gravimetric water content can be measured from soil samples by drying the material. This, together with a knowledge of bulk density and specific gravity of the soil grains, allows determination of the in situ volumetric water content and degree of saturation.

An estimate of volumetric water content can be obtained the field by measuring the electromagnetic properties of the soil-fluid system, using techniques such as time domain reflectometry (TDR) or nuclear magnetic resonance (NMR). However, conversion of volumetric water content to degree of saturation requires a knowledge of gravimetric water content and specific gravity of the soil grains, which can only be obtained from soil samples. This means that although it is possible to determine the degree of saturation of material in a dam or foundation at a particular instance, it is not possible to monitor its variation with time. In addition, the electromagnetic properties of the soil-fluid system are influenced by factors other than water content, which adds to the difficulty in interpreting the data.

The soil-water potential of soil in the field can be measured with instruments that make direct contact with the pore fluid. Positive pore fluid pressures (pressures above atmospheric pressure) can be measured directly with one of several types of piezometers available, including hydraulic piezometers, vibrating wire piezometers, and electrical resistance piezometers. Negative pore fluid pressures (pressures below atmospheric pressure) can be measured directly with a tensiometer. It is also possible to measure negative pressures indirectly using, for example, electrical conductor sensors or thermal conductor sensors. Indirect methods measure a property related to the negative fluid pressure and require calibration of the sensor.

A NOTE ON NEGATIVE SOIL-WATER POTENTIAL

Soil-water potential, or the potential energy per unit mass in the soil, includes several components, of which gravitational potential and pressure potential are the most relevant for engineering practice. Potential can be expressed in three equivalent ways: energy per unit mass, energy per unit weight (hydraulic head), and energy per unit volume. It is customary to report pressure potential in terms of hydraulic head (units of length) or energy per unit volume (units of pressure).

Gravitational potential is given by the elevation of a point relative to an arbitrary reference level. Pressure potential is measured in relation to atmospheric pressure. Soil-water at a hydrostatic pressure greater than atmospheric pressure is defined as having a positive pressure potential; when the soil-water pressure is below atmospheric pressure, the pressure potential is taken as negative and is referred to as matrix suction (reported as a positive quantity).

Matrix suction results from both capillary and adsorptive forces between the soil water and the soil matrix. This quantity captures the total effect resulting from the affinity of water to the matrix of the soil, including its pores and particle surfaces, which bind water in the soil and lower its potential energy below that of bulk water. Formally, matrix suction is defined as the negative gauge pressure, relative to the external gas pressure on soil water, to which a solution identical in composition with the soil solution must be subjected to be in equilibrium through a porous membrane wall with the water in the soil. In practice, matrix suction (s) is calculated by taking the difference between the pore-air pressure (u_a) and the pore-water pressure (u_w); i.e. $s = u_a - u_w$.

EFFECT OF DEGREE OF SATURATION AND MATRIX SUCTION ON LIQUEFACTION OF SOILS

There is now an extensive body of literature that considers the liquefaction resistance of unsaturated coarse-grained soils, such as sands and silty sands. This work, conducted in the laboratory, has focused primarily on assessing the effect of changes in degree of saturation on resistance to cyclic stress-induced liquefaction. In addition to degree of saturation, some recent studies have also reported matrix suction prior to and during a cyclic test.

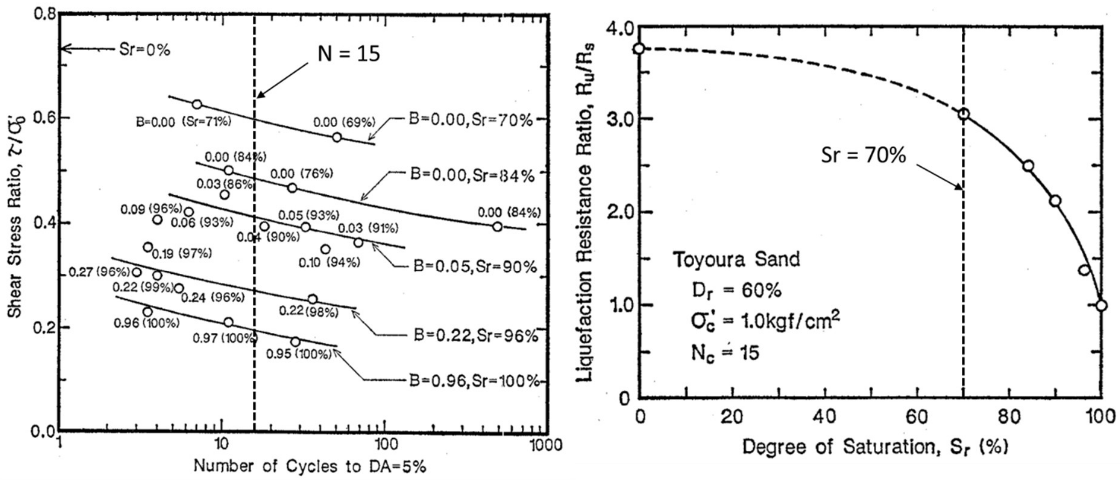
Cyclic-induced liquefaction, which can result from ground shaking during an earthquake, is caused by the densification of loose material during cyclic stress changes and principal stress rotation, which can result in an increase in pore water pressures. For dense soils, cyclic stress changes will result in a reduction in stiffness and potentially in deformations during loading (cyclic mobility). For loose soils, cyclic loading can result in an undrained strength reduction and brittle failure (liquefaction). Cyclic-induced liquefaction is one of two types of liquefaction phenomena, the other being static liquefaction. The latter results from a large undrained strength reduction due to an increase in pore water pressure during monotonic stress change (loading or unloading). Static liquefaction is associated with brittle failure.

Cyclic liquefaction in the laboratory is normally determined by measuring the number of uniform cycles required to reach a particular failure criterion, such as (i) 5% double amplitude (DA) strain, or (ii) excess pore water pressure equalizing the initial effective confining stress. During a test, different levels of uniform cyclic stress are applied to the sample, and the data is presented in the form of cyclic stress ratio (CSR) against number of cycles to reach failure (N). The CSR is the cyclic shear stress normalized by the initial normal stress.

Figure 1(a) shows a plot of CSR (labelled *Shear Stress Ratio* τ/σ_o') against N (labelled *Number of Cycles to DA = 5%*) obtained by testing Toyoura sand in a hollow cylindrical torsional shear (Yoshimi et al 1989). Toyoura sand is a research material widely used in Japan with the following characteristics: d_{50} = 0.175 mm, d_{10} = 0.129 mm, coefficient of uniformity (C_u) = 1.52, and fines content (FC) = 0%. Liquefaction during a cyclic test was defined as the number of cycles required to yield a double amplitude shear strain of 5%. The figure includes B-values[1] measured during initial consolidation together with degree of saturation prior to the application of the cyclic load (labeled as *B* and S_r, respectively, in the figure). As the initial degree of saturation decreases from 100% to 70%, the cyclic resistance of Toyoura sand increases markedly.

Figure 1(b) shows the variation in the shear stress ratio required to cause a double amplitude shear strain of 5% after 15 uniform cycles (corresponding to an earthquake of magnitude 7.5). The figure also shows the static shear strength of dry sand, defined as the shear stress at a shear strain of 2.5 %. The greatest increase in cyclic resistance in Toyoura sand takes place as the degree of saturation reduces from 100% to 70%. The ordinate in Figure 1(b), labelled *Liquefaction Resistance Ratio,* R_u/R_s, corresponds to the cyclic resistance at a particular degree of saturation (R_u) normalized by the cyclic resistance of saturated material (R_s).

[1] Ratio of the increase in pore water pressure to the increase in cell pressure.



(a) (b)

Figure 1. Results from hollow cylindrical torsional shear tests on Toyoura sand (modified from Yoshimi et al 1989)

Figure 2(a) shows the results from cyclic loading tests carried out on a silty sand (50% FC) using a triaxial cell capable of monitoring matrix suction during a test (Banerjee et al 2022), with suction being controlled and measured using the axis translation technique. The target relative density of the soil at the start of the test was 50%. Samples were first saturated and thereafter dried to the desired initial matrix suction, which ranged from 0 kPa to 30 kPa (corresponding to degrees of saturation of between 100% to 70%). During the undrained tests, pore-air pressures and pore-water pressures were measured independently to record changes in matrix suction. Tests were stopped after the double amplitude axial strain had reached 5%, or after the number of cycles had exceeded 300. The figure plots the variation in CSR (labelled *Cyclic Resistance Ratio, CRR*) with N (labelled *Number of cycles*) for different initial values of suction. An increase in matrix suction results in an enhanced cyclic resistance of the silty sand.

Figure 2(b) and Figure 2(c) show the variation in cyclic resistance at 20 uniform cycles with changes in matrix suction and degree of saturation, respectively. The ordinate in both figures is given in terms of a liquefaction resistance ratio (LRR), corresponding to the cyclic resistance of the unsaturated material normalized by the cyclic resistance of the saturated material. For the silty sand tested, a large increase in cyclic resistance occurs as degrees of saturation reduce from 100% to 90%, corresponds to an increase in matrix suction from 0 kPa to around 2 kPa. The data indicates that a reduction in degree of saturation below 75%, associated with an increase in matrix suction above 10%, is accompanied by a marked increase in cyclic resistance.

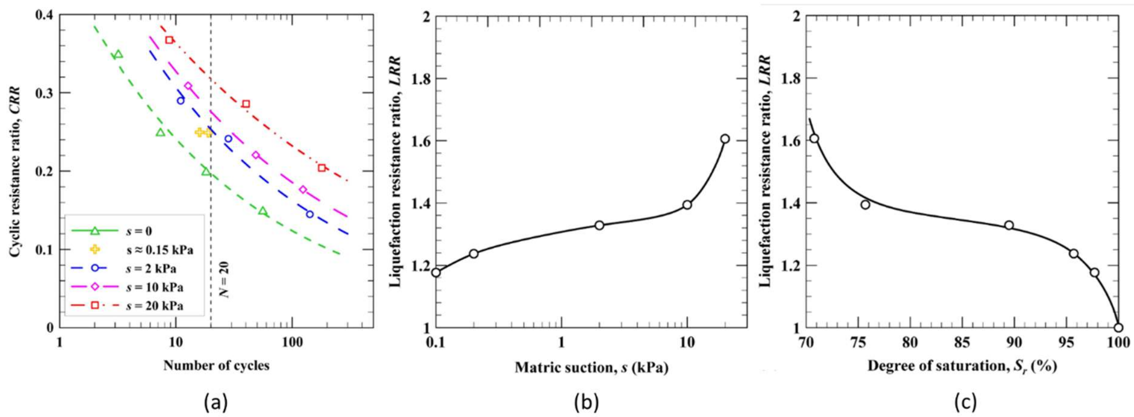


Figure 2. Results from triaxial tests on silty sand (modified from Banerjee et al 2022)

AN EXAMPLE OF THE NEED TO MONITOR MATRIX SUCTION IN THE FIELD

Both Figure 1 and Figure 2 presented in the previous section show that cyclic resistance to liquefaction is very sensitive to changes in degree or saturation and matrix suction. A reduction in the initial degree of saturation in Toyoura sand from 100% to 90% translated to a doubling of the cyclic resistance when N was 15 (Figure 1). A similar reduction in degree of saturation in the silty sand tested with the suction controlled triaxial cell, associated with an increase in matrix suction from 0 kPa to 2 kPa, was accompanied by an approximately 40% increase in cyclic resistance when N was 20 (Figure 2). Although a reduction in degree of saturation (and an increase in suction) results in an enhanced response of a soil during cyclic loading, the test results presented in Figure 1 and Figure 2 also show that unsaturated material with a high degree of saturation can experience cyclic mobility, and possibly cyclic liquefaction if the initial state is loose enough; i.e., it cannot be assumed that if the degree of saturation falls below 100% the risks of cyclic mobility and liquefaction disappear. This presents a challenge to the engineer, as explained below.

Figure 3 shows a confining dam part of a tailings facility now under active care. There has been no deposition of tailings in the facility for the past 40 years and work is progressing towards final closure. The dam was constructed in the upstream direction using the coarse fraction of the tailings to create an outer shell and then tailings were deposited in the impoundment hydraulically as a slurry.

Interpretation of cone penetration test (CPT) soundings indicates that most of the tailings in the dam are dry or have low degrees of saturation: dynamic and equilibrium pore pressures are negligible. This material is labelled as ‘Dry tailings’ in Figure 3. The CPT data also indicates the presence of layers of fine tailings near the base of the dam where dynamic pore pressures are high. The material in these layers is interpreted to have a high degree of saturation and the layers are labelled as ‘Wet tailings’ in Figure 3. In addition to being wet, the normalised tip resistance corrected to an equivalent clean sand value ($Q_{tn.cs}$), proposed by Robertson and Wride (1998), obtained in this wet material is below 70. $Q_{tn.cs} \leq 70$ is the criterion given by Robertson (2010, 2016, 2022) to determine, at a screening level, if a soil is susceptible to undrained brittle response and to liquefy. Therefore, the layers of wet tailings depicted in Figure 3 are assumed to have the potential to liquefy during an earthquake.

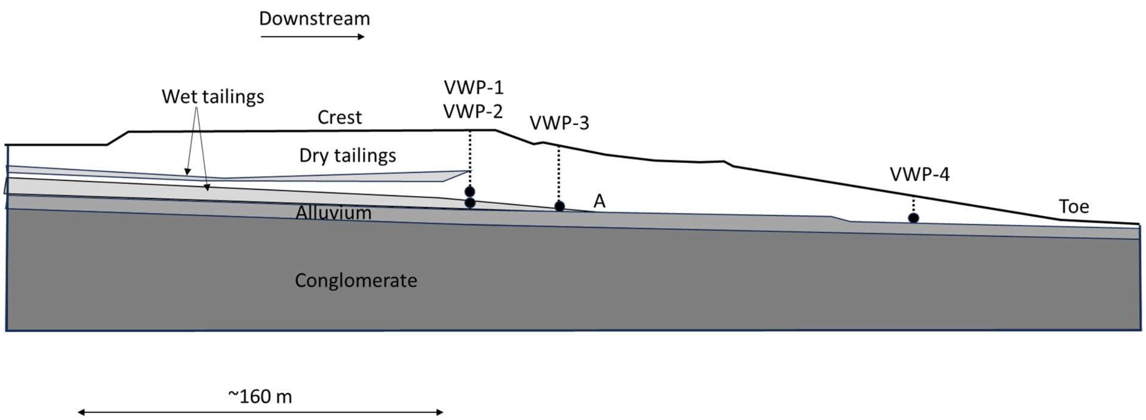


Figure 3. Example of a mining dam with layers of potentially liquefiable tailings, labelled as "wet tailings".

The basal layer of wet tailings with the potential to liquefy has been interpreted to extend from the impoundment to Point A (Figure 3). Beyond Point A, only dry tailings are thought to be present in the dam. Although the dry tailings beyond Point A are in a loose state ($Q_{tn.cs}$ value are still below 70), given the low degrees of saturation, the material is thought not to be susceptible to experience liquefaction during an earthquake.

A two-dimensional limit equilibrium stability analysis assuming liquefaction of the basal layer of wet tailings that extends to point A gives a high factor of safety (FoS) for global instability of around 2.6. This corresponds to a post-earthquake loading condition and indicates that the dam would be stable during an earthquake even if the wet material underwent an undrained brittle response, with the strength of the tailings in the basal layer reducing to the undrained residual strength. The high FoS is due to the stabilizing effect of the unsaturated material near the toe of the dam, beyond Point A, which is assumed to retain its strength during an earthquake. The material near the toe of the dam, however, is in a loose state. This means that an increase in degree of saturation at the base of the dam beyond Point A could potentially result in material in this zone becoming susceptible to an undrained brittle response and to liquefy during an earthquake. If the basal layer of wet tailings is extended from Point A to the toe of the dam, the post-earthquake FoS reduces to 0.9. An increase in saturation could occur, for example, due to a rising water table or from the prolonged storage of water in the impoundment.

Currently, conditions at the base of the dam, within the tailings mass, are monitored with four non-vented, non-flushable vibrating wire piezometers (VWP). The locations of piezometers are shown on Figure 3. Instruments have been labelled as VWP-1, VWP-2, VWP-3, and VWP-4 in the figure. VWP-1 and VWP-2 were installed within the same borehole at different depths in July 2017; whereas VWP-3 and VWP-4 were installed in separate locations in August 2019. The piezometers are fitted with low air-entry (LAE) filters and were installed by the fully grouted method.

The response of the four piezometers since installation until November 2021 is shown in Figure 4. The plots indicate the location of the sensor and the total head recorded over time. VWP-1, VWP-2 and VWP-3 have been reporting negative pore water pressures since installation, with maximum suctions of 10 kPa (VWP-1 and VWP-3) and 50 kPa (VWP-2) measured up until November 2021. Readings in VWP-4 have fluctuated within the range of ±2 kPa during the reporting period.

A fully saturated, non-vented VWP that is making direct contact with the pore fluid in the surrounding soil will record barometric pressure fluctuations. This appears to have been the case in the four piezometers after installation, where the initial response shows fluctuations in piezometric readings. The period during which fluctuations are observed, labelled as 'Fluctuation' in Figure 4, ranged from 5 to 14 months, with the longest period corresponding to piezometer VWP-4 (where readings fluctuated between positive and negative values). After this initial period, the variation in piezometric readings with time in all four plots traces a smooth curve, suggesting that the piezometers may have desaturated.

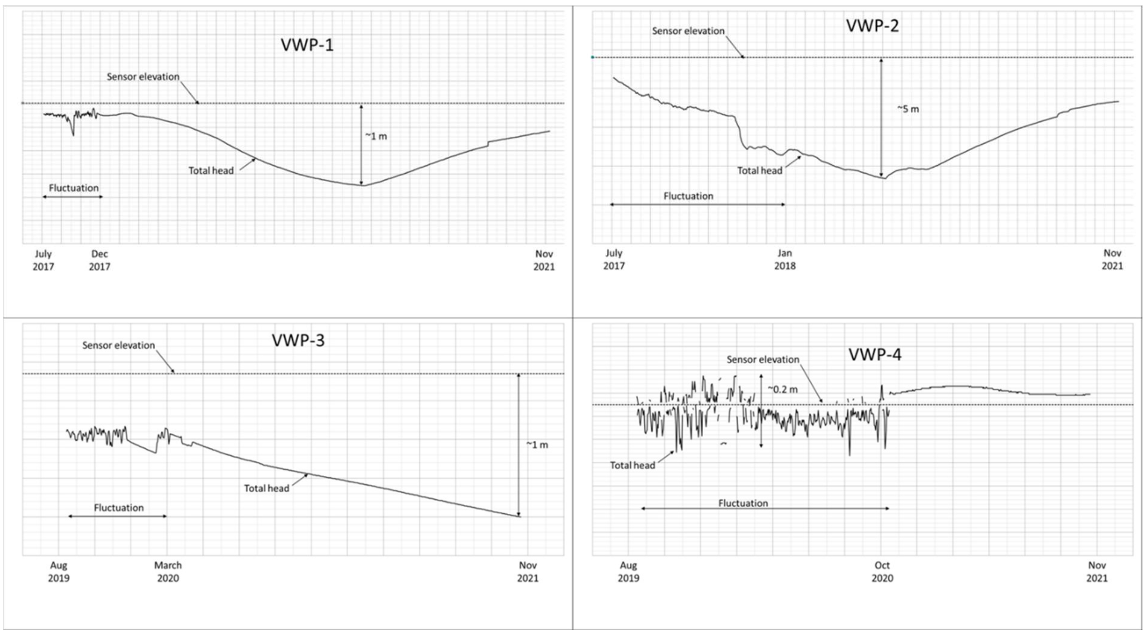


Figure 4. Readings recorded in VWP installed by the fully grouted method

Piezometers VWP-1 to VWP-4 are currently used to monitor conditions at the base of the dam shown in Figure 3. The expectation is that the instruments will respond to changes in pore water pressure, and hence alert of an increase in saturation and an associated risk of liquefaction during an earthquake. These piezometers are part of a critical control for the dam.

Considering the possibility that piezometers VWP-1 to VWP-4 may have desaturated, two questions arise:

- How reliable are long-term measurements of matrix suction derived from non-flushable VWPs installed with the fully grouted method?
- Should non-flushable VWPs installed with the fully grouted method be used in situations where matrix suctions in the surrounding soil can prevail for long periods?

THE USE OF VIBRATING WIRE PIEZOMETERS INSTALLED WITH THE FULLY GROUTED METHOD TO MEASURE MATRIX SUCTION

The generic term for an instrument that measures matrix suction directly is a tensiometer. This consists of a porous filter and a means of measuring stress, which are separated by fluid retained in a reservoir. Tensiometers work in a similar manner to piezometer: they allow water to flow (in the case of a tensiometer, out of the device) until the internal energy of the water filling the tensiometer's reservoir reaches a state of equilibrium with the internal energy of the soil-water. This, however, does not mean that the tensile stresses in the tensiometer and in the soil-water are similar, since both capillary and adsorbed components of potential are present in the latter case.

Any piezometer fitted with a diaphragm, such as a VWP, has the potential to measure matrix suction (i.e. it can be used as a tensiometer); however, the successful measurement of matrix suction requires (i) that the water in the piezometer reservoir is in contact with the water in the soil, and (ii) that the piezometer remains saturated (Ridley 2015). If the first condition is not met, the water in the piezometer reservoir will not be able to reach equilibrium with the soil-water; if the second condition is not met, the accuracy of any suction measurement will be uncertain.

There are four factors that restrict the measuring range of a tensiometer, including VWPs (Ridley 2015): (i) the procedure used to remove air from the tensiometer, (ii) the volume of water in the tensiometer reservoir, (iii) the material used to manufacture the body of the tensiometer, and (iv) the pore size of the porous filter (given by the air-entry value). The first three factors are associated with the formation of vapour cavities as the water in the tensiometer reservoir is subjected to a hydraulic tension. Cavitation (the formation of vapour cavities) can occur within the liquid or at the boundary between the liquid and the tensiometer reservoir wall. The fourth factor has to do with the ingress of air into the tensiometer reservoir when the difference between the tensile stress in the water within the tensiometer reservoir and the atmospheric air pressure outside the tensiometer reservoir reaches the air-entry value of the porous filter. This causes air to be drawn through the filter under the influence of the difference in pressure.

Over the past three decades there has been an increase in the use of VWPs installed by the fully grouted method in geotechnical projects. Contreras et al (2008) discuss the subject and build on work originally carried out by Vaughan (1969). The authors present results from finite element analyses that indicate how errors in the measurement of pore water pressure are only significant when the permeability of the cement-bentonite grout is three orders of magnitude greater than the permeability of the surrounding soil. If the permeability of the grout is lower than the permeability of the surrounding soil, measurement errors will be minimal. The authors also present several examples of the successful use of the fully grouted method for piezometer installation in geotechnical practice. Additional examples are given in Dunnicliff (2008).

Besides simplifying the installation method, the use of cement-bentonite grout as backfill for piezometer installation offers the additional advantage of remaining saturated when in contact with soils that have high matrix suction (something unlikely to happen when a sand pack is used as backfill around a VWP). The use of grout is, therefore, preferable when soil suctions are likely to be encountered in the field. Given that the pore size of the porous filter in a VWP will restrict the measuring range of the instrument, it would seem appropriate to use a VWP fitted with a high air-entry porous filter, together with a fully grouted installation, when soil suctions need to be measured in the field.

Simone and Sorensen (2018) carried out a study that looked at the performance of VWPs fitted with both high air-entry (HAE) and low air-entry (LAE) filters placed in fully grouted boreholes. Seventeen non-flushable VWPs were installed in very low permeability, stiff, overconsolidated clay. VWPs from two different manufacturers were used and the HAE filters were saturated using five different methods. In addition, three different cement-bentonite mixes were employed. An additional test was carried out in the laboratory by sealing a VWP with a HAE filter in a block of cement-bentonite grout and placing the instrument in a 3m high pipe filled with water.

The authors report that within weeks of installation the piezometers with HAE filters started to give erroneous readings. After eight months, only one of the nine piezometers fitted with a HAE filter gave credible readings. The main reason for the poor performance was attributed to unsatisfactory filter saturation, with the cement-bentonite grout being the main problem. This conclusion was based on the observation that similar VWPs with HAE filters were able to measure successfully positive pore water pressures when placed in direct contact with the clay.

Simone and Sorensen (2018) concluded that non-flushable VWPs fitted with HAE filters have a high risk of malfunctioning when placed in fully grouted boreholes. When employing this method of installation, they recommended the use of LAE filters.

The above recommendation is captured in the current ISO standard on measurement of pore water pressures using piezometers (ISO 2020). Annex E, which is normative, considers the installation of piezometers with the fully grouted method. It states that "*high air entry porous filters shall not be used with the fully grouted method unless there is a means of removing air from the piezometer*". Furthermore, Annex F, which is also normative, includes the following two statements:

- *"To successfully measure soil suctions all parts of the piezometer system (e.g. the backfill material, the porous filter and the fluid reservoir) shall remain saturated at all times and the water in the piezometer shall be in continuous contact with the water in the soil at all times.*
- *If air forms inside the piezometer it shall be removed and saturation of the device shall be restored. NOTE: Air can be removed by flushing water into a flushable piezometer or by removing the piezometer and resaturating it."*

The above implies that that non-flushable VWPs installed by the fully grouted method, even if fitted with HAE filters, should not be used to measure matrix suction in the field for long periods, given that (i) there is uncertainty in the performance of a HAE filter embedded in cement-bentonite grout, (ii) saturation of the piezometer system cannot be ensured, and (iii) it is not possible to resaturate the instrument once in place. The measurement of suctions in the field requires the use of a piezometer that can be retrieved and resaturated if needed, or the use of a flushable piezometer. An example of the successful use of a flushable piezometer to measure suctions is given, for example, in Ridley et al (2003).f

SUMMARY

This paper has briefly touched on a couple of topics that are of interest to the mining dams engineer and, to a lesser extent, to the embankment dams engineer. The first subject has to do with the effect of degree of saturation and matrix suction on the potential for a soil to liquefy during an earthquake. Lower degrees of saturation and higher suctions translate into enhanced resistance during cyclic loading; however, experimental data suggests that unsaturated materials still have the potential to experience cyclic mobility. The second topic has to do with the measurement of matrix suctions in the field. An example is given of a situation where this forms part of a critical control for a dam. Measurements are currently done with non-flushable VWPs installed by the fully grouted method. Although this method of installation offers advantages, non-flushable VWPs installed by the fully grouted method, even if fitted with HAE filters, appear not to be suitable for the task of measuring matrix suctions. The measurement of suctions in the field requires the use of a piezometer that can be retrieved and resaturated if needed, or the use of flushable piezometers.

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The British Dam Society

Leakage Remediation Works at the Hampton Distributing Reservoir

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J R CORREIA, former AtkinsRéalis
P MARSDEN, Keller
C SMITH, Thames Water

SYNOPSIS Hampton Distributing Reservoir is a non-impounding reservoir built in 1900s and located in Hampton, southwest London. The reservoir, formed by a typical puddle clay core embankment, has a total perimeter of 800m and a storage capacity of 32,000m^3.

An increase in embankment settlement was detected, starting from 2011, based on annual crest levelling surveys, which was then followed up with a non-intrusive geophysical survey in 2020. This identified a distinct leakage path at the foundation level of the reservoir embankment. In order to mitigate the risk of seepage-induced instability such as internal erosion, leakage remedial measures were proposed to arrest the leakage.

Limited working space and difficult access were some of the main constraints for the remedial works. Following an optioneering/feasibility study, permeation grouting using Tube-a-Manchette (TaM) was identified as the most practical remedial solution. Grouting works were carried out on both sides of the clay core to target flow paths and create a low permeability zone reducing the leakage/seepage through the dam.

This paper presents the key aspects of the project, from the initial investigative works to construction, covering also the optioneering and design of the grouting works. Challenges and lessons learnt from the project are also highlighted.

INTRODUCTION

Hampton Distributing Reservoir (locally known as ‘Red House Reservoir’) is located in Hampton, southwest London. It is a small non-impounding reservoir built in 1900s, owned and operated by Thames Water Utilities Limited (TWUL). Water supplied by the Staines Reservoirs Aqueduct is temporarily stored in the Hampton Reservoir, and then gravitates to the Grand Junction Reservoir at the Hampton Water Treatment Works.

The reservoir is formed by a typical puddle clay core embankment with a maximum height of 3m. It is approximately triangular in plan with a length of 250m, base width of 150m, and a total perimeter of 800m. It has a storage capacity of 32,000m^3.

The typical cross section of the embankment is shown in **Figure 1**. The main characteristics of the embankment section are the following:

- Maximum height of 3m with a 1.8m wide crest, 1v:2h downstream slope and 1v:3h upstream slope, the latter protected by concrete slabs from crest to toe.

- Embankment shoulders are formed by clayey sandy Gravel on a stripped surface of original ground level over Kempton Park Gravel Formation.
- A 0.9m wide puddle clay core that passes in a trench through the Kempton Park Gravel Formation and is keyed into the underlying London Clay Formation with a 1.2m deep embedment which results in a total height of 8.5m for the puddle clay core/trench (not fully shown in the cross section below).

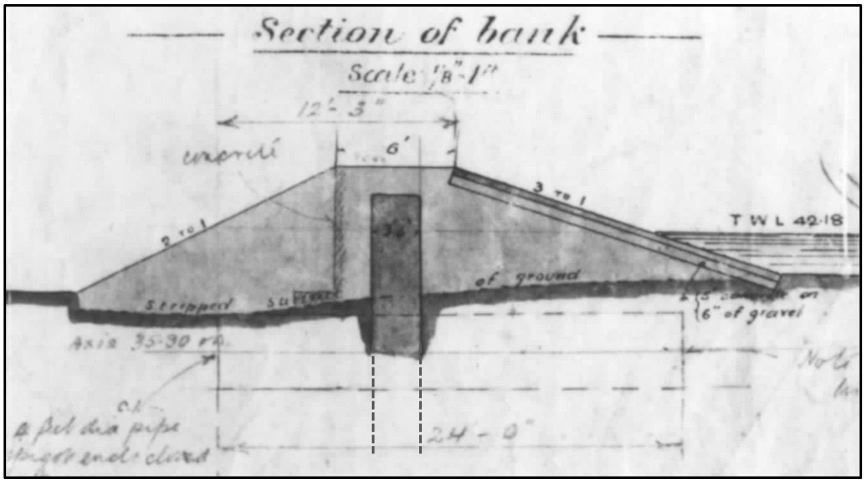

Figure 1. Extract of record drawing showing a typical section of the embankment, with a clay core extending to the London Clay Formation at depth.

THE PROBLEM

Embankment crest levels were monitored annually at nine points on the reservoir rim. In 2011 an inspection was carried out under Section 10 of the Reservoirs Act. As a part of the reservoir inspection, crest surveys data were reviewed with the results showing an average settlement rate of 3mm/year at the southwest side of the dam over the period between 1988 to 2011, which was considered as a normal amount of settlement. There was no significant settlement at other monitoring points. The inspection report recommended that annual monitoring of the embankment to be continued.

Between 2011 and 2019, average settlement continued at just under 3mm/year, except at one monitoring point at the south side of the dam, which recorded an increase in average settlement of 6.4mm/year, with two years where settlement exceeded 10mm. The QCE (Qualified Civil Engineer under the Reservoirs Act) was consulted, and the crest surveying frequency increased.

Due to a continuing trend of settlement, in 2020 the reservoir Supervising Engineer (under the Reservoirs Act) requested a geophysical seepage survey in order to investigate potential leakage problem in that section of the embankment. The survey identified a zone of leakage extending some 20m on the south side of the dam (Figure 2) at the same location where the larger settlement was recorded.

Seepage survey results showed the leakage at a depth of approximately 9m below the crest level which corresponds to the bottom of the puddle clay trench. It was suspected that the leakage passed through the clay core at the interface with the London Clay formation. The concentrated leakage paths could lead to internal erosion of embankment materials. If the internal erosion was allowed to develop further, the integrity of the dam could be compromised, which could eventually lead to its failure.

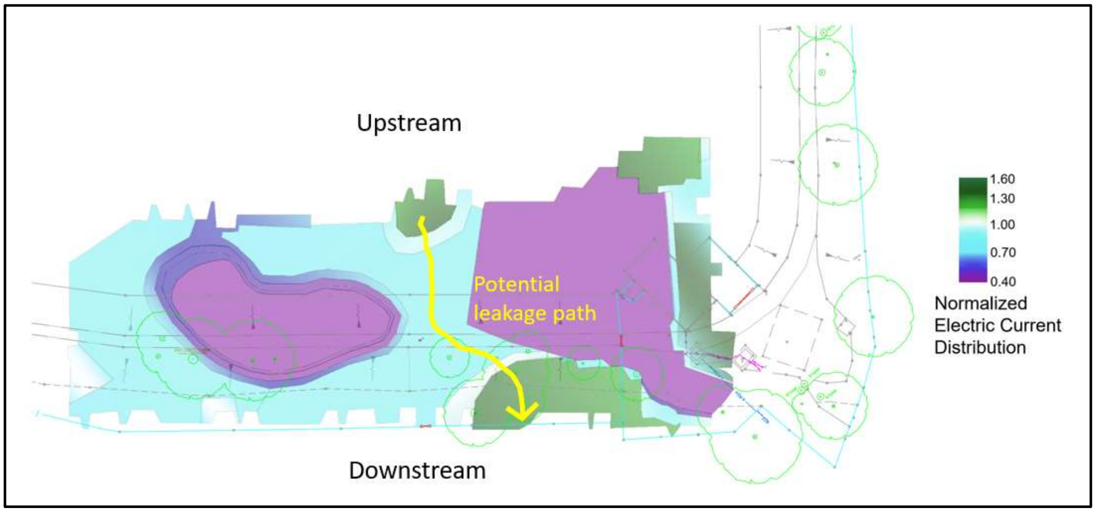

Figure 2. Geophysical seepage survey showing the leakage zone

TWUL (the Client) commissioned MWH Treatment (MWHT, the Main Contractor) to undertake leakage remediation works and subsequently MWHT commissioned AtkinsRéalis as designer to support the implementation of the project during design and construction. The assignment started with a ground investigation to better understand the embankment characteristics, an options appraisal to identify an appropriate solution for the remedial works, and was followed by the design and construction support. MWHT commissioned Keller as the geotechnical contractor who provided technical advice for the grouting works and carried out the construction.

OPTIONEERING

The optioneering study was carried out to identify the most appropriate leakage remedial solution in terms of the effectiveness, buildability, sustainability and cost. Remedial solutions using either a piled cut-off wall or grouting were considered.

Difficult site access and limited working space were the main challenges in the project. The width of the embankment is only 1.8m. The embankment slope and downstream toe are populated by some large trees and vegetation which limits the headroom on crest, as shown in Figure 3 below. The reservoir area is a Ramsar site and a 'Site of Special Scientific Interest (SSSI)'.

A sheet piled cut-off wall is a proven method to provide a low permeability continuous barrier along an embankment, which was recently used in other reservoirs in the London region such as Island Barn, William Girling and King George V reservoirs. This solution for the Hampton Distributing reservoir would require installation of 10m long sheet piles from the embankment crest through the puddle clay core into the underlying London Clay formation.

However, due to the very narrow crest, piling works would have to be assisted by a mobile crane set up either at the toe of the embankment or on a floating pontoon on the reservoir. Either option would have required significant enabling works. Considering the site constraints and ecological sensitivity of the site, the pile cut-off wall solution was not considered feasible.

An alternative remedial solution using permeation grouting was proposed. The technique involves injecting low pressure cement grout into the ground using the Tube-a-Manchette (TaM) method. The grouting works would not require heavy plant hence avoiding the need for significant enabling works. Drilling works could be conducted on the narrow crest by a small drilling rig to create boreholes for TaM pipe installation.

GROUND INVESTIGATION

In September 2022, a new ground investigation (GI) was carried out by Structural Soils Ltd in order to better understand the ground conditions and provide geotechnical parameters for the design of remediation works. In addition, the new GI also provided confirmation of width, depth and position of the puddle clay core in the works area.

The GI works started with hand-dug slit trenches on the embankment crest to expose the clay core and to confirm its alignment. Dynamic probing was conducted at three locations, followed by low-vibration percussive boreholes through the centre of the clay core down to the London Clay formation. A small Windowless Sampling rig compactible for drilling works on the narrow crest was used (Figure 3). These exploratory holes were spread through the 30m chainage, to confirm the depth and condition of the clay core and the London Clay where the core keyed in. The boreholes were fully cased which protected the thin clay core from hydraulic fracturing and hole collapse. Verticality was checked throughout the drilling works in order to reduce the risk of penetrating the sides of the clay core.

Figure 3. Small portable drilling rig on narrow crest

During the GI, two boreholes were terminated at a shallower depth after water strikes were observed at 6m to 8m below crest level within the suspected leakage zone. The soil samples at these levels showed that the puddle clay core was very soft with high moisture content. The levels where water strikes were encountered were slightly higher than the leakage zone determined in geophysical seepage survey (9m below crest level), which suggested that the problems in the clay core could be more widespread than originally anticipated.

A percussive borehole was carried out at the toe of the embankment to provide samples and data for the natural strata. The level of the interface between the Kempton Park Gravel and London Clay formation was also determined. In situ permeability testing was conducted to determine the permeability of the soil (Kempton Park Gravel) underlying the embankment.

The particle size distribution and permeability of the foundation materials were used to inform the grouting design.

THE GROUTING SOLUTION

Kempton Park Gravel (KPG) formation beneath the embankment consisted of a clean sand-gravel mixture with a permeability generally ranging between 10^{-4} and 10^{-5}m/s. The geophysical survey and dam settlement monitoring indicated that pronounced water flow paths had developed in discrete locations. It was, therefore, predicted that zones of higher permeability would be present where the finer grained elements of the soil had been eroded.

To target the erosion paths, a grid of grout injection points was established using the Tube-a-Manchette (TaM) system. Each TaM pipe consisted of a tube with injection ports at regular centres over the intended grout injection zone. The injection sleeves consisted of perforations, covered with a rubber sleeve to form simple non-return valve. The TaM pipes were sealed into the ground with a low strength sleeve grout. Each injection sleeve could be isolated with the use of a double inflatable packer to allow the precisely controlled grout injection in the target soil at the required pressure. Each injection sleeve could be used multiple times to allow a phased approach to the grout injection.

A cross section of the proposed target zone for permeation grouting is presented in Figure 4. Two rows of TaM pipes were installed upstream of the dam core and two more rows were installed downstream of the core. The inner row grout holes were vertical. However, due to the limited crest width, the grout holes on the outermost row were inclined ('raked') with an angle of 10° which provided a broader grouted zone at the base where leakage was predicted to be most pronounced. This approach provided sufficient space for personnel to safely work on the crest.

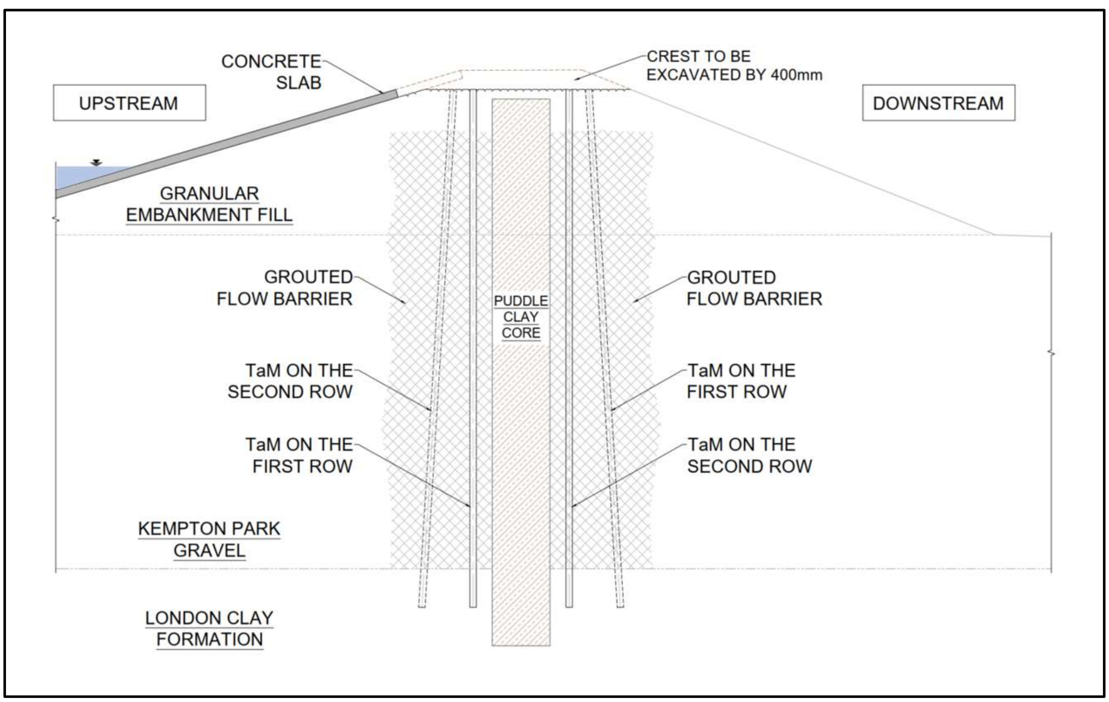

Figure 4. Grout injection zone within the dam

The grout holes were provided with a minimum 1m toe-in to the London Clay in order to achieve a good contact. The findings from the GI works indicated that the leakage zone may extend higher than the most pronounced paths determined by the geophysical survey. Therefore, the targeted zone of grouting was 8m deep extending from the dam shoulder into the London Clay formation, below the base of the Kempton Park Gravel.

A plan view of the grout hole arrangement on the embankment is presented in Figure 5. Two rows of TaM pipes were installed on an equilateral triangular grid on each side of the puddle clay core. The holes were spaced at 1m centres, in line with the CIRIA C774 (CIRIA, 2018) recommendation for medium to fine sand permeability ranges between 10^{-4} and 10^{-5}m/s.

The grout injection sequence was agreed with the QCE. Alternate primary and secondary grouting sequence was adopted. Injection data including grout injection volumes and flow rates were reviewed after each grouting cycle. The data were then used to identify zones of high-volume grout take and to determine the need of grout injections in the next phase.

The grouting works were carried out in five phases in the following sequence:

Phase 1: Trial grouting

Phase 2: Injection of Primary TaMs of the first row at downstream and upstream

Phase 3: Injection of Secondary TaMs of the first row at downstream and upstream

Phase 4: Injection of Primary TaMs of the second row at downstream and upstream

Phase 5: Injection of Secondary TaMs of the second row at downstream and upstream

The primary/secondary TaMs and the first/second rows are defined in Figure 5.

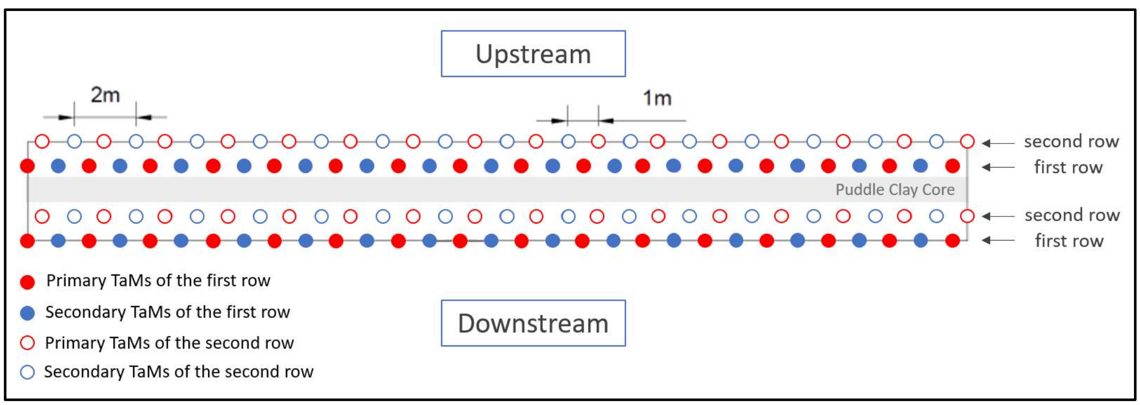

Figure 5. Grout borehole arrangement

After the five phases of grouting, additional reinjections were commenced on the selected sleeves where both high injection volumes and high flow rates were observed. The data was again reviewed and if necessary, the grouting was extended or repeated until satisfactorily low grout volumes and low flow rates were observed.

Cement based grouts were used to provide the longevity required. A cement bentonite grout mix was used as sleeve grout to seal the TaM pipes in place. It was also used in the initial grout injections to provide a low-cost solution to grout the most pronounced leakage paths.

The geotechnical contractor provided quotes for the grout mixes in Table 1. Microfine or Ultrafine cement grout were also considered due to their enhanced penetrability compared to cement bentonite grout. Several grout mixes were tested during trial injections and microfine cement was selected for the grout injection, to permeate as much of the soil as practical.

Table 1. Proposed grout mix

Type	Cement	Additive	Mix design	Particle size
Cement bentonite grout	Ordinary Portland Cement (CEM II)	Bentonite	1:10:20 Bentonite-Cement-Water sleeve grout mix	D95 < 50 – 75 µm
Microfine cement grout	Microfine cement (MasterRoc MP650 SR)	Superplasticiser: MasterRheobuild 1000	1:1 water cement ratio with 1.5% additive	D95 < 16 µm
Ultrafine cement grout	Ultrafine cement (MasterRoc MP800 SR)	Superplasticiser: MasterRheobuild 1000	1:1 water cement ratio with 1.5% additive	D95 < 12 µm

CONSTRUCTION

The construction phase commenced in late August 2023. Firstly, a temporary wider working platform was constructed by lowering the crest to allow for sufficient working space and to ease the drilling of the holes further away from the centre of the crest. In addition, a flat compound area of approximately 10m by 10m was used for material storage and equipment such as grout pump module and grout mixer, as shown in Figure 6.

Figure 6. Site compound area for grouting equipment

Because of the requirements of maintaining freeboard and allowing for sufficient cover to the puddle clay core, the maximum depth of excavation to create a wider working platform was limited to 400mm. Due to limited working space and difficult access, the geotechnical contractor used a small drilling rig (Klemm 702) with a width of only 750mm when it is tracked into position, which helped to overcome the accessibility constraints (Figure 7).

Figure 7. Drilling works on the crest

Before the construction, precautionary measures were put in place to minimise noise and vibration due to the ecological sensitivity of the site. Sound barrier blankets were installed around the works area and routine noise monitoring was carried out to ensure noise levels were within acceptable limits. The small earthwork equipment and drilling rigs also helped to minimise vibrations.

Since the works area was in close proximity to the reservoir, a containment system was installed on the crest to contain drill and grout arisings during the construction. Contamination risk to groundwater and reservoir water was managed through a careful control of the maximum grout volume per sleeve and injection pressure in order to limit the grout spread. In addition, routine sampling and testing for pH value and turbidity were carried out throughout the construction period.

The reservoir was in operation during construction. Access for the operational staff was maintained during the works. Given the limited working area, careful planning was carried out to ensure that site activities did not obstruct access to the outlet screen, the remaining part of the crest, the overflow weir or any operational valves.

In order to confirm the assumptions such as grout mix and grout pressures, trial grouting was carried out. Injection data such as grout volume and grout flow rates were extracted from the pump module, which allowed monitoring and confirmation of the effectiveness of grouting.

Cement bentonite grout was tested in the trial grouting initially as it is a more economic option. However, the volume of grout take at each sleeve was much lower than the targeted volume. Therefore, a microfine cement grout mix was also tested, which generally allowed a higher grout injection volume, indicating more effective permeation of the soil in the leakage zone. It was concluded that microfine cement grout would ensure better results hence it was used in the grouting works.

Grouting was carried out on the embankment, starting firstly with the Phase 2 (i.e. Injection of Primary TaMs of the first row as shown on Figure 5). In each phase, the downstream row of grout holes was grouted first, followed by the upstream row. The aim was to allow grout injected on the upstream row to flow into any gaps between the zones of grout injection on the downstream row (CIRIA, 2018).

To maximise the efficiency of the works, all the TaM pipes were installed prior to injection of the microfine cement grout. This allowed the drilling rig to operate in a systematic sequence in the constrained workspace.

The grout was injected from the bottom of the TaM pipe, progressing upwards with each sleeve in turn. The grout volume, average flow rate and flow rate at termination were recorded for subsequent review. The grout injection parameters were also recorded and graphed against time using the computer-controlled grout injection pumps. This allowed careful monitoring of grout takes, pressures and flow rates against the depth/zone being injected.

The target injection pressure was limited to soil overburden pressure during injection. Grouting was carried out at this target pressure at each port, until the termination criteria, either flow rate of less than 2 litres per minute or total grout take of 100 litres was reached.

Following the completion of daily grouting work cycle, grout data saved in the pump module was extracted and subsequently fed into a 3D model. Graphical output from the 3D model was generated to present the injection parameters at the as-built locations of each grout port. This allowed daily recording and monitoring of grouting parameters as the work progressed. It also facilitated the effective use of the observational approach, in which regular reviews of the grouting data was used to determine the extent of the subsequent grout injections. Figure 8 shows the graphical output of the 3D model which presents grout volume at each TaM sleeve.

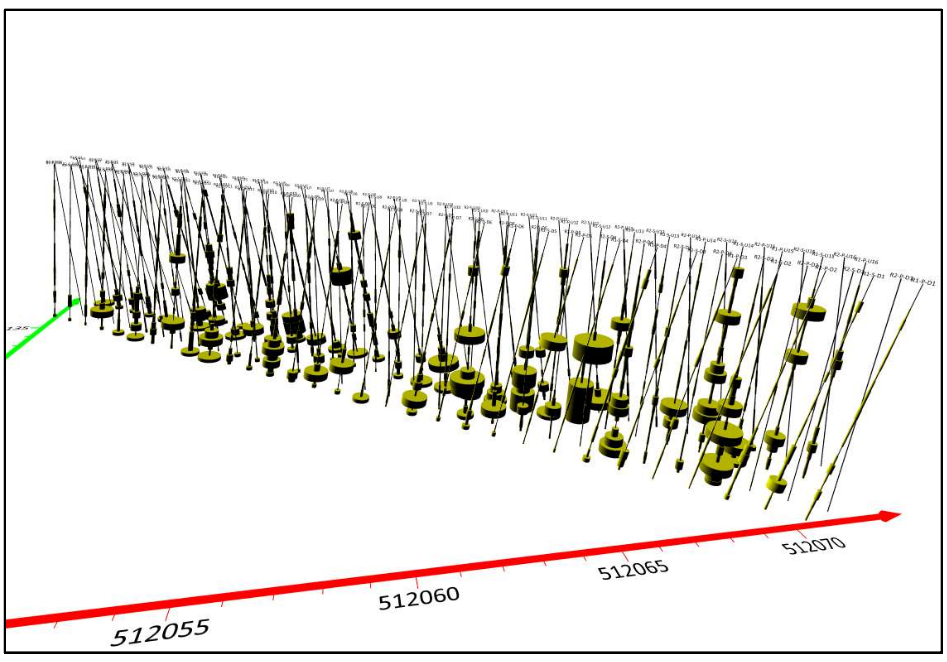

Figure 8. Graphical output from 3D model showing volume of grout take at each port

A swift decision was required to meet the construction programme as the next phase of grouting was determined based on the available data from previous injections. An efficient communication chain was established between the QCE, contractors and the designer's site representative. In order to facilitate communication, grout data and findings were shared to the wider project team after each grouting cycle, usually on a daily basis. The findings were also discussed during frequent meetings (twice a week) and emails which allowed collaborative decision making between the QCE and the geotechnical contractor.

During the Phase 2 grouting in the primary grout holes, high volumes of grout take and flow rates were observed at the interface between the puddle clay core and London Clay where leakage was found. Fissures at the top of Weathered London Clay could have contributed to the high injection volumes at those levels.

After the Phase 2 grouting and in discussion with the QCE, it was decided that more grouting was required due to high volumes of grout take. Therefore, the remaining three phases of grouting were carried out sequentially. Grout data at each sleeve was monitored throughout each grouting phase.

Generally, high injection volume was observed in the leakage zone in all four phases of grouting. However, there was an obvious trend of decreasing injection volume in each sleeve as the grouting works advanced. After the completion of all four phases, there was a small number of sleeves where high injection volume was recorded. Additional reinjection was conducted in the selected sleeves where both high grout volume and high flow rates were observed. The volume of grout take in the regrouted sleeves was small (<10L per sleeve). It was then considered that no further grouting would be practical or required.

The construction lasted for approximately three months starting from mid-August 2023. In total 112 no. grout holes were constructed along the 30m long leakage zone. The total grout injection volume using the TaM system was approximately 27m^3. The average volume of grout take per metre (length along the chainage) was 0.9m^3.

A post construction geophysical seepage survey was carried out in November 2023 as a 'compare' investigation to identify effectiveness of the remediation works. The results showed that leakage path through the dam has been successfully stemmed by the grouting works.

CONTRACTUAL ARRANGEMENT

The grouting works consisted of five phases in which the first two phases were fixed scope of works. This was the minimum grouting works that the contractor was requested to carry out. The remaining three phases of grouting would depend on grout injection data from the prior phase. On this basis, a lump sum cost was defined for the first two phases of works in the contract. Grouting works for the remaining phases were re-measurable based on actual injection volumes and number of grouted holes.

Early input from the geotechnical contractor was essential in the tender design stage as it helped minimise risks and aid constructability. Although the grouting design was carried out by the designer, it happened in a collaborative manner with the technical advice from the geotechnical contractor being incorporated in the construction package.

CONCLUSION

A potential leakage problem at the Hampton Distributing Reservoir was identified by a review of settlement monitoring data. The investigation was followed up with a geophysical seepage survey which identified a distinct leakage path through the embankment dam. During the investigation phase, a combination of geophysical seepage survey and ground investigation was helpful to confirm the extent/location of leakage. The leakage path could have developed further overtime and led to dam failure due to internal erosion. In 2023, permeation grouting was carried out within the 30m long leakage zone which successfully arrested leakage through the dam.

When challenging constraints such as difficult access, limited working space and ecological sensitivity are encountered on a site, cut-off wall solutions may not be practicable due to their disruptive nature and significant enabling works required. In these situations, grouting is a proven method which works well at small reservoir sites, especially where heavy machinery and large lay-down area are not allowed. Grouting could also provide a cost-effective solution and reduce the carbon footprint of the project, as it does not require significant enabling works.

Identification of the key seepage paths allowed an effective grouting solution to be planned. Analysis of the grout injection data through daily 3-dimensional modelling, allowed the observational method to be used to identify and target the key seepage paths. The rapid assimilation and visualisation of the grouting data allowed all parties to work as one team, with quick decision making that focused the grouting in the zones where it was most required. This focused approach contributed to an effective use of grouting, minimising the costs and allowing the works to be completed within programme.

ACKNOWLEDGEMENT
The project was overseen by Dr Andy Hughes from Dams and Reservoirs Ltd. who was appointed by Thames Water as the Qualified Civil Engineer for the works at the Hampton Distributing Reservoir. The support and direction by Dr Hughes to the successful delivery of the project is greatly appreciated.

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The British Dam Society

Case studies from permanently installed siphon works

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SYNOPSIS Adequate draw down of reservoirs by gravity means only may not always be feasible. Siphons may be seen to be a suitable option and efficient means of drawing off the upper portion of a reservoir volume. This paper looks to cover case studies of schemes completed in recent years.

Based upon multiple examples of physical projects undertaken, this paper will look into the constraints, planning and decision-making involved leading up to and executing improvement works, along with the temporary works, permanent works and commissioning of permanently installed siphons. The intention of this paper is that the learning taken from these works may be of use to others in the industry.

All works were undertaken on statutory reservoirs and as such had been planned and undertaken with the supervision of an All Reservoirs Panel Engineer.

Mott MacDonald Bentley (MMB) planned and undertook works at the following:

- Warland Reservoir
- Warley Moor Reservoir
- Lower Barden reservoir
- West Hallington reservoir

WARLAND RESERVOIR

Introduction

Warland Reservoir is situated on the western slope of Blake Moor, above Littleborough, Lancashire. The reservoir was originally constructed around 1857 by the Rochdale Canal Company to maintain water levels in the Rochdale canal, and was considered to be one of the largest dams in England at the time. It has a 1500m long, 20m high embankment formed of homogenous earth fill. The reservoir is now owned and operated by United Utilities.

One of the project drivers was to improve the drawdown capacity to achieve 1m per day.

Managing Risks for Dams and Reservoirs

There were numerous challenges and constraints on the project including:

- Location: The reservoir is 375m above sea level, regularly freezes over in the winter with temperatures as low as -20°C recorded.
- Drawdown: For work to be carried out in the basin, the reservoir would have to be drawn down by at least 8m. In such a location, with very little means of controlling water level, this was a key challenge.
- SSSI/blanket bog: The reservoir was surrounded by a Site of Special Scientific Interest (SSSI), designated for the blanket bog.
- Site access: The access was a 3km narrow track to the site, in poor condition.
- Silt: The reservoir basin was known to contain large amounts of silt.

Figure 1. Warland Reservoir drawn down by 8m, and upstream pipe work installed

Design Development

Early optioneering of the drawdown requirements identified that a siphon through the dam crest was the only viable option to achieve the required capacity.

Several combinations of siphon pipe sizes and materials were considered. The preferred option was for 3No. DN600 ductile iron pipes through the dam crest. These would be located midway between the valve-tower and the eastern abutment on the part of the dam that gave the shortest linear distance of siphon whilst still achieving the required upstream submergence depth. The revised position approximately halved the amount of temporary access road required to construct the access ramp into the reservoir basin.

Three pipes were preferred over one to significantly reduce the scale of temporary works for lifting. The arrangement also enabled each pipe to be individually tested in a controlled manner, reducing the risk of downstream erosion due to high flows. In the event of a failure on one of the three siphon lines, the system offers redundancy, still allowing for a significant drawdown to be undertaken.

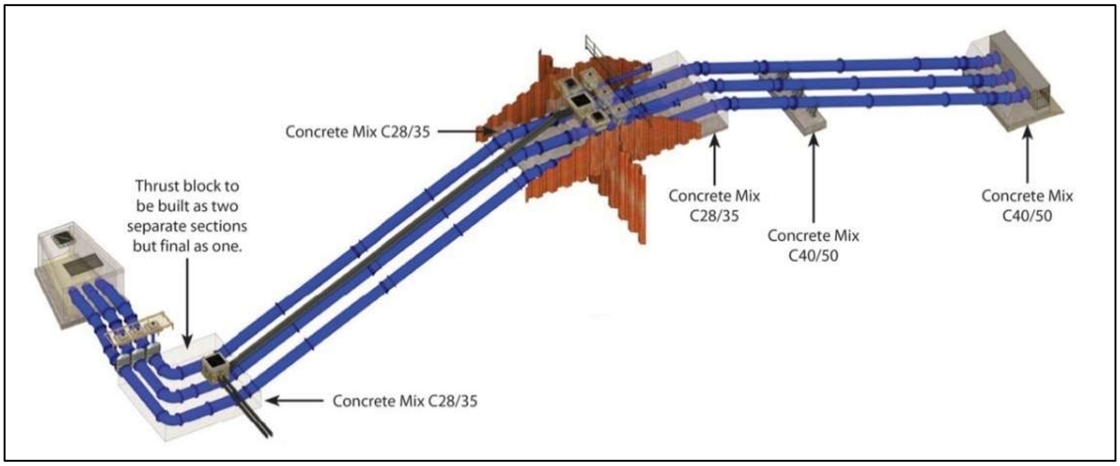

Figure 2. 3D model of the siphon system and temporary works on the dam crest

A transition chamber at the downstream end of the siphon was designed to combine the flows from the individual siphon pipes into one single DN1500 concrete pipe. This runs under the access track near the toe of the dam, discharging into a chamber at the toe at the start of the clough.

With the siphon pipes located above top water level, the risk of leakage through the embankment would be eliminated. However, this meant the maximum level the siphons could draw down to was around -5m below top water level. Although the reservoir catchment was not forested, there remained a risk of blockage of the siphons from floating debris. An inlet screen was designed which would also prevent unauthorised access to the pipes if the reservoir was ever drawn down below the inlet level in the future.

Construction

Construction of the project commenced in 2017 and began with significant works required to upgrade the existing 3km access track to the reservoir. This included temporary propping of a bywash channel, a new temporary crossing of the channel, widening and strengthening works to allow construction plant to access the site.

The reservoir was drawn down by 8m to facilitate the construction of the upstream section of the siphons. The inlet works were undertaken first, and once installed, the reservoir was allowed to partially re-fill. To pressure test the pipes a blanking plate was left on the end of each pipe, which was removed by divers following a successful test.

There were significant concerns about embankment stability, given that a large proportion of the embankment was constructed from peat, an amount of which had been added as part of the stability works in 1923. To improve the ground conditions, the area at the toe of the embankment, where the transition chamber was constructed, had to be artificially raised with granular fill to allow sheet piles to be installed to form a stable excavation.

When the reservoir was drawn down for installation of the upstream section of the siphons, there was a significant concern about a deterioration of water quality discharged downstream. The team developed a silt monitoring plan with the Environment Agency, including trigger points with agreed actions. Temporary pipework modifications were made in the catchment to blend the discharge waters with those of a neighbouring reservoir to reduce turbidity.

Managing Risks for Dams and Reservoirs

WARLEY MOOR RESERVOIR

Introduction

Warley Moor Reservoir is located in West Yorkshire and is owned and operated by Yorkshire Water Services (YWS). The reservoir is impounded by two dams with a crest level of 407.3mAOD. The overflow system is formed of concrete culverts with a trapezoidal grass reinforced channel situated above to take excess flows. A stilling basin connects the two culverts as well as taking flows from the scour pipe.

Figure 3. Aerial view of Warley Moor Reservoir

One of the objectives of this project was to improve the drawdown capacity to achieve 350mm per day.

The key challenges on the project were very similar to that of Warland Reservoir, including extreme weather conditions, management of silt during drawdown and issues with slope stability due to the presence of peat.

Design Development

The existing embankment at Warley Moor had shown signs of local shallow slips along the downstream face. Slope stability modelling was carried out at the start of the scheme to ensure that the installation of the siphon pipework did not significantly impact the stability of the embankment. It was decided to bury the upstream pipework instead of installing it above ground due to the existing slope stability issues at the site.

The pipe material was also reviewed with an aim to minimise the additional load on the embankment. Structural calculations determined that SDR17 pipework was sufficient for the combination of negative pressures and soil loading. This plastic pipework provides greater flexibility compared to rigid or semi-rigid pipe materials such as steel or ductile iron. The flexibility is beneficial for accommodating differential settlement that could occur on the embankment.

The plastic pipework is also significantly lighter than steel pipework. This further reduces the bearing pressure on the embankment, minimising the risk of embankment slips as well as reducing the weight of plant required to lift the pipework on the slope. The PE pipework can also be welded prior to installation, therefore reducing the amount of time there is an open excavation on the embankment.

Due to plastic pipework not being used for siphons previously by YWS, it was only used for pipework downstream of the crest. The pipework was also dual contained with a drainage outlet to allow leakage to be identified (Figure 4).

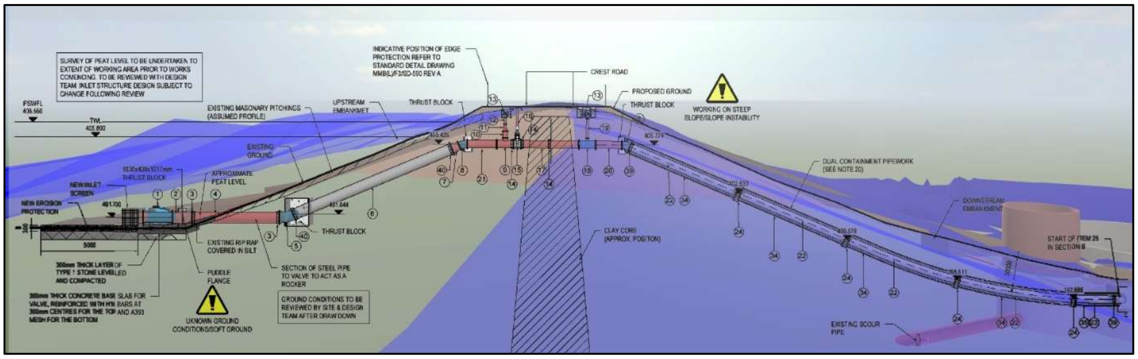

Figure 4. Cross section through siphon

Construction

The dual containment solution utilised 600mm diameter twin wall pipework as the outer pipe with the 350mm HDPE siphon pipework threaded through. The construction methodology involved excavating the trench and installing the dual containment pipework first. Spacers were designed to be welded onto the siphon pipework using offcuts from plastic pipework on site, reducing waste and costs (Figures 5 and 6). The purpose of the spacers was to keep the siphon in the centre of the dual containment pipe, prevent the pipework from moving excessively when the siphon was in operation and to prevent the pipework from catching on the inside of the twin wall. The innovative design led to minimal programme increases and the spacers were effective at allowing the siphon pipework to smoothly slide down the twin wall dual containment pipework.

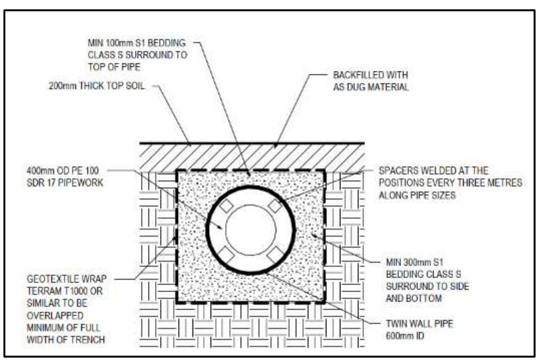

Figure 5. Cross section through downstream dual contained pipework

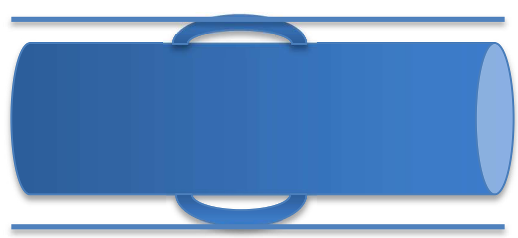

Figure 6. Illustration of “spacers” on the pipework

BARDEN LOWER RESERVOIR

Introduction

Lower Barden Reservoir is the lower of two reservoirs located near Bolton Abbey and within the Yorkshire Dales National Park. The reservoir is an impounding reservoir with a crest length of approx. 640m and a maximum height of 28m with a capacity of 2.23 Mm³. The spillway chute consists of a series of curved steps with vertical upstands forming pools. Lower Barden Reservoir is owned and operated by Yorkshire Water Services (YWS).

Managing Risks for Dams and Reservoirs

One of the project objectives was to improve the drawdown capacity so that the reservoir water level may be lowered by 925mm per day. The main challenge of this project was that the spillway was being refurbished in parallel with the new siphon construction.

Design Development

The siphon was designed to be self-priming at top water level, requiring manual priming at levels lower than its pipe crest level (Figure 7). The siphon is capable of drawing down the reservoir by approximately 5.0m from top water level.

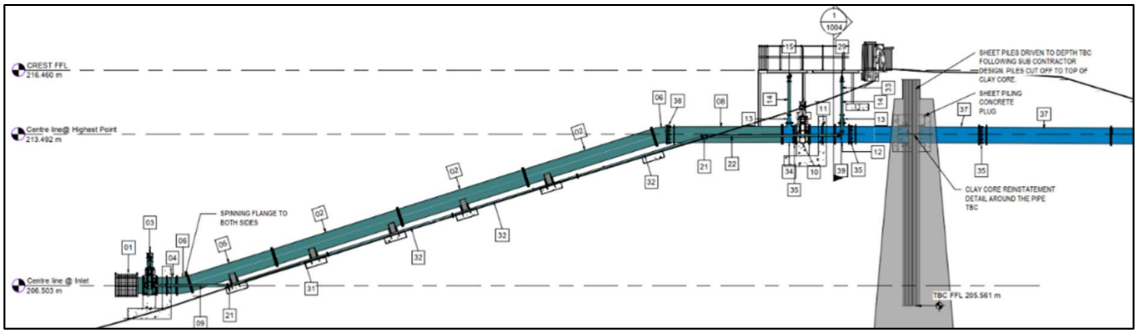

Figure 7. Barden Lower Siphon long section

A solution was developed where the new siphon would discharge through the floor of the newly developed spillway, along with some minor amendments to the hydraulic design of the siphon (Figure 8).

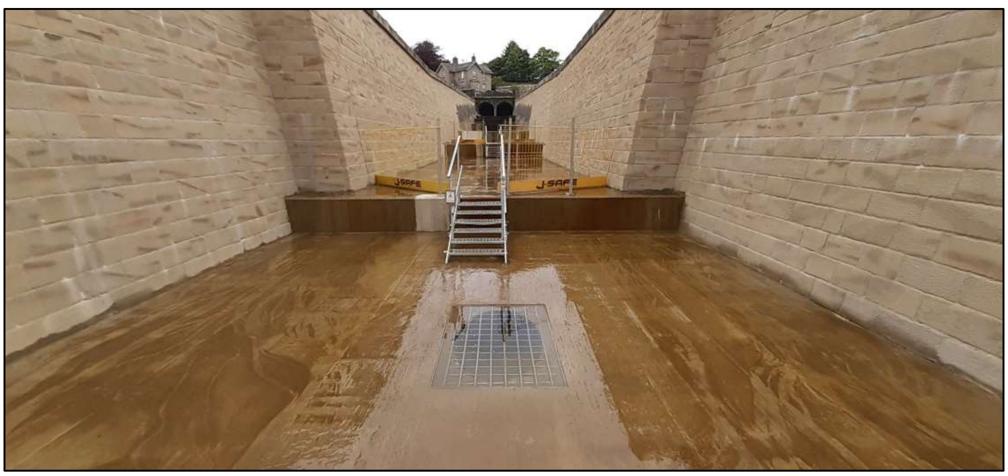

Figure 8. Siphon outlet in spillway invert

Construction phase

Enabling works began in May 2022. Due to the restricted access, crane pads could only be placed on one side of the spillway and the programme of works needed to be carefully planned to ensure that the two schemes worked in tandem, without blocking off access to each other during the spillway and siphon construction.

The siphon works required a significant draw down of the reservoir level to be able to work on the inlet structure safely (Figure 9). As this was quite a long duration for the works, it would impact the programme significantly and equally impact the spillway structure. It was decided to work from both ends of the pipe where practicable and to use a make-up piece at a bend on the downstream side of the embankment as a connection. This would aid in any minor tolerance issues in the pipework and ensure that the pipe closed correctly.

To aid the construction of the pipework, the Leica iCON system was used for setting out. This involved directly using the 3D modelling from the design of the pipe into a handheld tablet device and enabled accurate setting out, without the need to update drawings or continually check that the drawings were the most relevant. This also assisted in delivering the as built positions of the pipework.

Figure 9. Upstream leg of siphon

WEST HALLINGTON RESERVOIR

Introduction

West Hallington Reservoir is a Northumbrian Water Group (NWG) asset situated 1km to the northeast of the village of Colwell in the Tynedale district of Northumberland. The reservoir was constructed between 1884 and 1890 for Newcastle & Gateshead Water Company for the purpose of municipal water supply. The reservoir was built adjacent to the earlier East Hallington Reservoir and operates as a non-impounding structure. It has a maximum depth of 12.1m, a capacity of 3.3Mm³ and a surface area of 50 hectares at the Full Supply Level (FSL) of 155.1m AOD.

The objective of this scheme was to increase the draw down capacity such that the reservoir level can be drawn down by 4.8m in 8 days.

Design Development

Mott MacDonald Bentley (MMB) developed a design solution which utilises two 700mm diameter siphons on the west embankment to operate at a combined flow rate of 3.2m³/s. The new siphon pipeline, alongside a temporary pump arrangement and the existing scour pipes, will meet the required drawdown capacity.

The siphon location was moved during design development to the west embankment due to concerns about stability and leakage through the south embankment and because it was easier to access this area of the site.

The siphon flows will be conveyed across a stretch of adjacent field which NWG has purchased. The flows will then either percolate to ground or gravitate to the Coal Burn and eventually to the River North Tyne.

Managing Risks for Dams and Reservoirs

A flood risk assessment (FRA) was carried out to support the planning permission and, as due diligence, to ensure that properties would not be impacted downstream. The assessment looked at the extent of flooding during operation of the siphon, along with fluvial, surface water, groundwater and finally reservoir flood risk due to an uncontrolled release. The FRA concluded that the maximum modelled flood extent from a drawdown of Hallington Reservoir is in agreement with the flow path identified by the Environment Agency in the long-term flood risk map and no communities are shown to be at risk due to the proposed drawdown. As periodic testing of the reservoir drawdown will occur during dry periods, it is not considered that a drawdown of West Hallington Reservoir at this location would increase flood risk elsewhere.

The mechanical equipment associated with the siphon is housed within a high security kiosk. Due to the remote site location, the cost of a permanent power supply for the kiosk building services was prohibitive. A solar powered solution has been installed utilising roof mounted PV cells, DC/AC converter and battery storage. This sustainable solution provides sufficient energy for lighting and heating within the kiosk with negligible running costs.

SYNCHRO 4D was utilised during the latter stages of the design process. The software enables the programme to be linked with the model to run a construction simulation (Figure 10).

Figure 10. SYNCHRO 4D simulation

The simulation highlighted several pinch points when materials must be delivered or when water levels must be lowered or raised and helped drive efficiencies. SYNCHRO 4D was also useful for explaining the construction process to the client, to site operatives and to visitors.

Construction

A borehole at the toe of the embankment on the dry side indicated the presence of water-bearing sands and gravels and so interlocking sheet piles were specified that punched through the sands and gravels into the underlying clays and so cut off groundwater flows into the excavation.

A cofferdam was installed to provide a dry area to construct the intake bay. It was erected in the wet by operatives wearing buoyancy suits, and then pumped out and an excellent seal was achieved. The cofferdam also allowed plant access around the pipework whilst the stone pitching was reinstated.

Figure 11. Siphon intake bay constructed within the cofferdam.

The intake bay was located along the Small Burn which originally ran across the middle of the reservoir and was diverted into the adjacent Coal Burn in the 1880s via a DN375 pre-cast concrete pipe. The ground around the intake bay was found to be poor and was excavated down to competent sub-grade and reinstated with 340 tonnes of gabion stone and 60 tonnes of Type 1 capping material. As this area is normally under a 5m depth of water it was inaccessible during the ground investigation stage.

As the DN700 pipes are fabricated from coated mild steel, a Type 1 pressure test (water loss method) was required. All bolts had been tightened and the torque readings recorded but achieving a pass proved difficult due to sunny weather. A temperature rise on the above - ground pipework led to a pressure loss of more than 0.2 bar in the water filled pipe voiding the test on a number of occasions. Eventually cloud cover enabled a valid test.

Figure 12. DN700 siphon pipework during installation

ACKNOWLEDGEMENTS

The authors wish to thank Yorkshire Water, Northumbrian Water and United Utilities for permission to publish this paper.

The British Dam Society

Leakage Remediation at a Small Heritage Reservoir

P D DOWN, Mott MacDonald

SYNOPSIS Abbeydale Industrial Hamlet, on the outskirts of Sheffield, is a former steel-working site along the River Sheaf and has become a museum open to the public. The site, including the reservoir and dam, is designated a Scheduled Monument and the forge works are Grade I listed. Several other buildings within the site are Grade II* listed. It has a history thought to go back to 1685, with the present-day site reported to date from the early 18th century. To provide power for the machinery, a small reservoir was constructed and filled with water abstracted from the River Sheaf. The reservoir was enlarged as the site developed although remains below 25,000m^3 capacity, and thus is not registered under the Reservoirs Act 1975 (as amended).

There has been a history of leakage from the reservoir. In November 2022, the most recent leakages and damage to structures were investigated with the aim of developing suitable remedial measures. At the end of May 2023, significant leakage from the reservoir into one of the Listed buildings occurred resulting in emergency action being taken. This paper details the issues encountered, works previously performed, recent investigations and the development of remedial works to provide a longer-term solution.

INTRODUCTION

In its heyday, the Abbeydale Works was one of the largest water-powered mill sites on the River Sheaf. It produced agricultural tools, such as scythes, grass hooks and hay knives. Production continued until 1933. Two years later, the site was donated to the City of Sheffield (now Sheffield City Council). The Council restored it to working order for the Conservation of Sheffield Antiquities. It was then developed as a museum by the City of Sheffield Museums Department, opening to the public in 1970. It has been in use as a working museum since.

Abbeydale Industrial Hamlet, including its reservoir, is currently operated by Sheffield Museums Trust, a registered independent charity that operates six of the city's leading museums and heritage sites. The Council still has an involvement with the site when necessary. Some of the 'Hand Forge' buildings are in continued use by blacksmiths.

The site comprises a long reservoir, aligned south-west to north-east, with the mill complex to the north-east end. Water is abstracted from the River Sheaf, a short distance south of the reservoir, and transferred via a goit channel. The reservoir is normally kept full, with water spilling at the overflow and conveyed along a masonry spillway channel back into the river. Penstock gates at the north-east end of the reservoir can be opened to allow water to be discharged and power waterwheels, to the 'Grinding Hull' and 'Tilt Forge' buildings respectively, before being returned to the river (Figure 1).

Managing Risks for Dams and Reservoirs

HISTORY OF THE RESERVOIR

As detailed in a Sheffield City Museums publication, during the early years of the site, a goit channelled water from the River Sheaf to the waterwheel (or wheels) behind two buildings which existed at the time. Around 1777, a dam and reservoir, approximately half the current size, was provided. After construction of the 'Tilt Forge' building in 1785, the reservoir was increased to a surface area of 18,800m^2 to provide greater capacity for two 5.5m diameter waterwheels. Further development of the site included the construction of the Workmen's Cottages by 1793 and the current 'Grinding Hull' building in 1817. The reservoir's surface area was then reduced to 15,000m^2 when removed silt was placed at the southern end and partly formed Beauchief Gardens.

The surface area of the current reservoir has further reduced due to siltation at the south end where flows from the River Sheaf enter.

DESCRIPTION OF THE RESERVOIR

The current reservoir has a surface area of approximately 10,000m^2 and a storage capacity of less than 25,000m^3 at a Top Water Level (TWL) of 98.1m AOD. The storage basin is puddle-clay lined. Due to the age of the reservoir, there are very limited records relating to its construction.

The reservoir is impounded along its eastern and northern sides by earth-fill embankments, up to 5.2m high, with a crest level between 98.4m AOD to 98.7m AOD. The form of construction is currently unconfirmed but, given its age, the embankment is anticipated to be formed from general fill materials with no water-retaining core. Puddle-clay is understood to have been used in more recent remedial works. To the upstream face, there is a vertical masonry wall, approximately 1.0m high for much of its length, around the reservoir perimeter.

In normal operation, the water level is maintained by a 3.5m wide, broad-crested, overflow weir in the upstream side of a masonry spillway penstock structure constructed within the north-eastern section of the East Embankment. The structure includes mid-level and low-level drawdown penstock gates. Within this structure, a masonry arch-shaped culvert, with an invert level of approximately 93.5m AOD, discharges into a 1.75m wide masonry spillway channel that runs along the south boundary of the 'Hand Forge' buildings and conveys flows back into the River Sheaf.

Several buildings and structures have been constructed within the downstream shoulder of the embankment dam. These include the 'Grinding Hull' and 'Tilt Forge' buildings. The 'Overflow and Spillway Penstock Structure', detailed above, has been built through the entire width and depth of the East Embankment. A masonry structure incorporating penstock gates, with invert levels approximately 1.3m below TWL, for the 'Grinding Hull' and 'Tilt Forge' waterwheels, has also been built through the entire width and depth of the East Embankment. The structure extends below the reservoir basin to form pits to accommodate the waterwheels.

HISTORIC LEAKAGES AND REMEDIAL WORKS

There has been a history of leakages and remedial works associated with the reservoir at Abbeydale Industrial Hamlet. The known records date from approximately 30 years ago although it is possible there were issues that pre-date this.

Late 1990s
Void repair works were reported to have been performed along the north end of the reservoir between the 'Grinding Hull' building and Abbeydale Road South (A621). The flagstones to the surface were lifted and puddle clay placed in layers before the surface was reinstated. Detailed information relating to these works has not been identified.

2001-02
There are very limited details regarding the remedial works performed or the reason they were required. From a record photograph, it appeared works were performed to the penstocks for the 'Grinding Hull' and 'Tilt Forge' waterwheels. To facilitate this, a significant draw-down of the reservoir level was implemented. Although unconfirmed, this may have been achieved by operating the mid- and low-level outlet penstocks to discharge water to the spillway. To the left (north) of the 'Grinding Hull' penstock, an engraved stone, marked "Reconstructed March 2002" and set in the surface, records the works.

2014
The remedial works performed in 2014 were due to the following issues experienced:

- A breach of the East Embankment, adjacent to the 'Overflow and Spillway Penstock Structure', and erosion of the embankment surface due to flowing water.
- Voids in the upstream vertical masonry wall along the East Embankment.
- A void in the reservoir clay lining immediately upstream of the penstock gate for the 'Grinding Hull' waterwheel.
- Leakage of reservoir water into the 'Tilt Forge' building.
- Leakage of reservoir water into the 'Grinding Hull' building.

The following remedial works were recorded (Figure 1):

- Removal of a mature tree within the East Embankment adjacent to the 'Tilt Forge' building.
- Installation of a 26m long, 6m deep cement-bentonite grout curtain within the East Embankment, from the 'Tilt Forge' building to the 'Overflow and Spillway Penstock Structure'.
- Repair of the breach to the East Embankment left (north) of the 'Overflow and Spillway Penstock Structure'.
- Packing and repointing of the perimeter wall masonry along the East Embankment.
- Replacement of the penstocks to the 'Grinding Hull' and 'Tilt Forge' waterwheels. The upstream aprons were also improved.
- Puddle clay repairs – exact location(s) and extents unconfirmed.

There were no records available to the author to indicate the investigation works performed to inform the design of the remedial works.

A construction drawing indicated the proposed replacement of the clay lining to the northern end of the reservoir basin. However, there is no record of this being performed. It appears the works were limited to localised repairs in the vicinity of the remedial works along the East Embankment and adjacent to the penstocks.

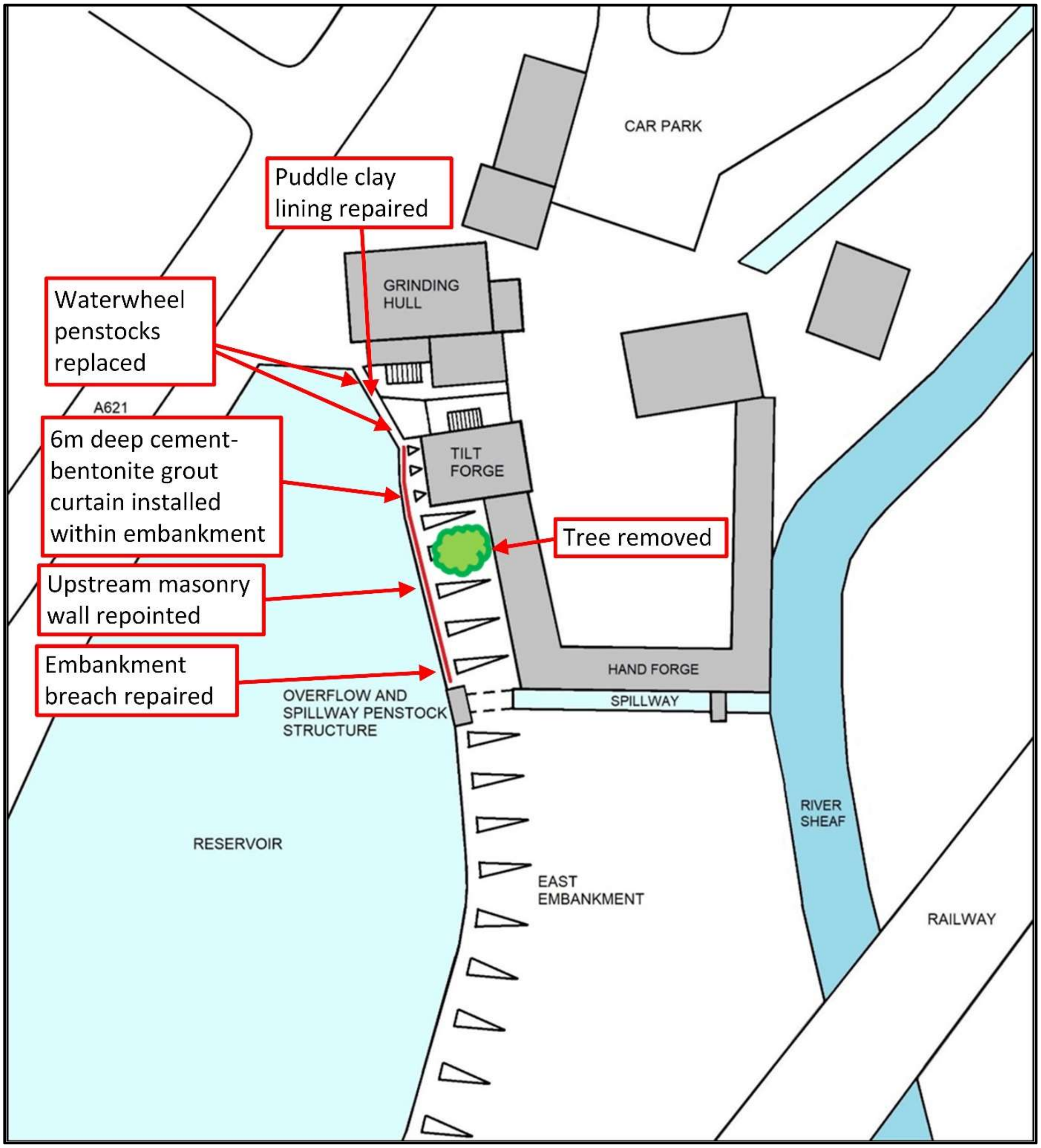

Figure 1. Abbeydale Industrial Hamlet, remedial works performed in 2014

CURRENT DAMAGE AND LEAKAGES

In November 2022, a request was received from the Client to visit the site, assess the situation and provide advice for any remedial works required. During the subsequent visits, the following issues were identified (Figure 2):

- Damage to the downstream end of the masonry spillway channel.
- A void under the 'Hand Forge' building adjacent to the spillway.

- A void and flowing water within the East Embankment crest adjacent to the 'Overflow and Spillway Penstock Structure'.
- Poor structural condition of the masonry spillway culvert. Ingress of water was observed through the culvert wall.
- Historic movement of the upstream vertical masonry wall along the East Embankment.
- Minor ingress of water into the 'Tilt Forge' building.
- Reports of ingress into the 'Grinding Hull' building with damp areas observed.

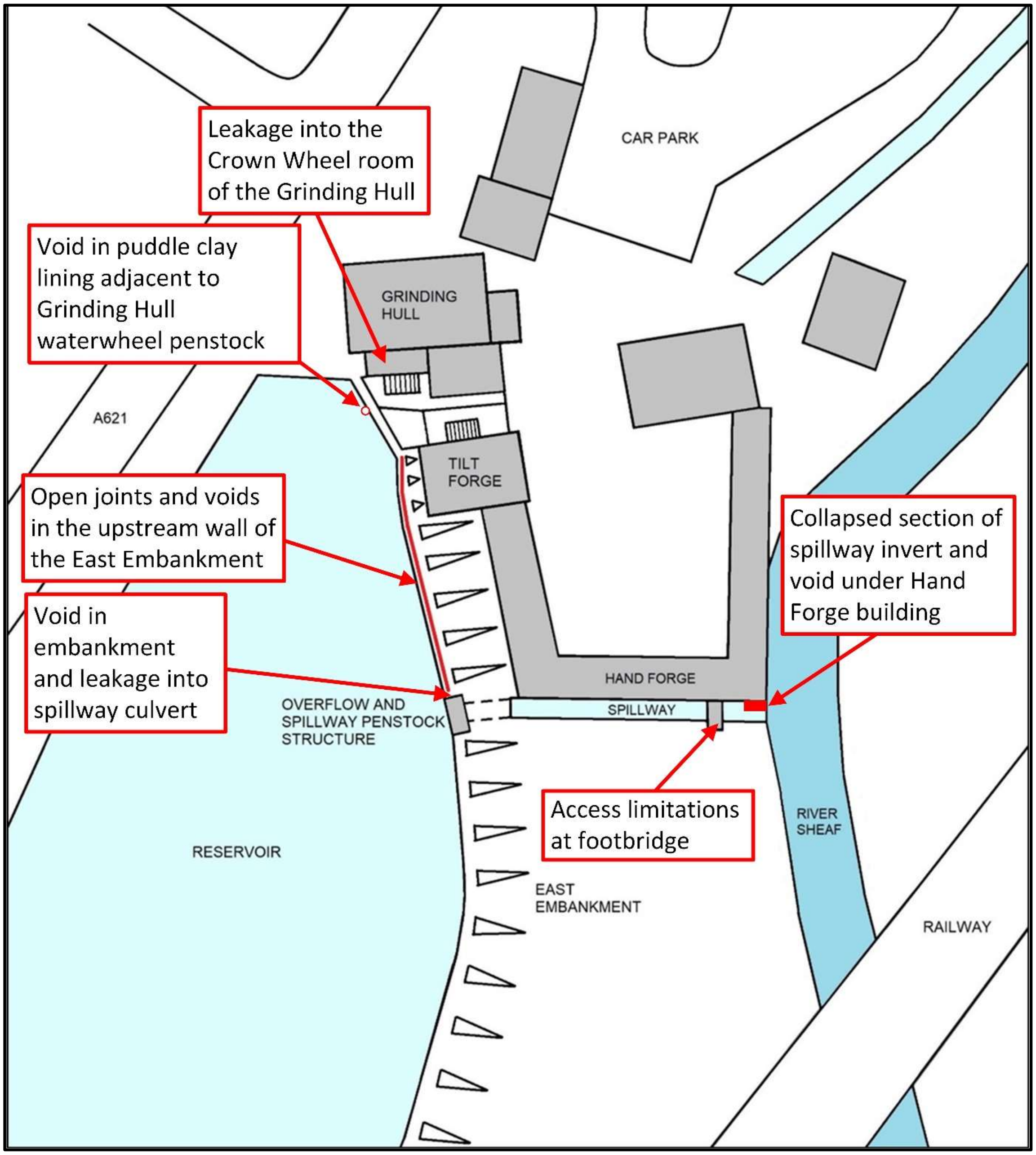

Figure 2. Abbeydale Industrial Hamlet, damage and leakages in 2022-23

The damage to the downstream end of the spillway comprised a collapsed section of the masonry invert approximately 1.2m long, 1.0m wide and 0.3m deep (Figure 3). There was damage to the end of the spillway, with missing stonework, where it discharged into the River Sheaf. It appeared the action of flows along the River Sheaf had damaged the downstream end of the spillway and washed out material from beneath, creating a void into which the invert collapsed. Within the river channel, immediately adjacent to the downstream end of the spillway, a scour hole, approximately 300mm deep below the water level, was present. A void extended from the spillway under the south-east external corner of the adjacent 'Hand Forge' building. This did not appear to have affected the structural integrity of the building and there were no obvious signs of distress.

Figure 3. Damage to the downstream end of the spillway channel

The void within the East Embankment crest was at the same location as experienced in 2014 and there was evidence of previous remedial works (Figure 4). During examinations between November 2022 and July 2023, it appeared to increase in size. At the latter visit, the void measured 1.6m long by 0.8m wide and its base was 0.9m below the adjacent top of wall level. Water was entering the void directly through the upstream wall of the East Embankment. It also entered the void from beneath the adjacent section of embankment crest indicating leaks from the reservoir at other locations along the upstream wall. The water flowed towards the 'Overflow and Spillway Penstock Structure' where it then disappeared into the body of the embankment. This flowpath differed to that experienced in 2014.

Figure 4. Void within the East Embankment adjacent to the overflow and spillway structure

The spillway culvert was arch-shaped with a flat, stone-lined bed and of aged condition. It was approximately 8m long, 1.4m wide and 1.1m high. There was no mortar visible in the joints and some displaced masonry was evident, especially to the soffit and to the right (south) wall (see Figure 5). Towards the upstream end, ingress of water was observed approximately half-way up the left (north) wall, coinciding with the location of the void in the East Embankment above.

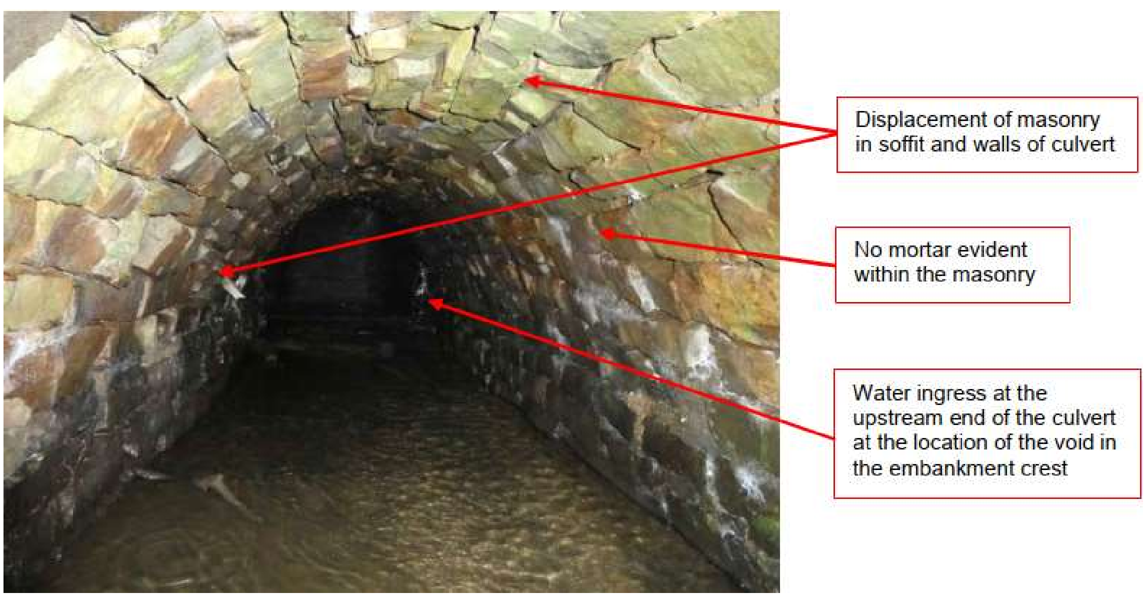

Figure 5. General condition of spillway culvert

Managing Risks for Dams and Reservoirs

The East Embankment's upstream wall showed evidence of having been raised in the past as the upper courses were of a different construction style to those beneath. This may be associated with the reservoir enlargement around 1785. The top of wall level appeared to have reduced as the embankment had settled over time. There was deformation of the masonry to the East Embankment wall at each end, due to differential settlement, at the connections with the 'Overflow and Spillway Penstock Structure', to the southern extent, and the 'Tilt Forge Penstock Structure', to the north. There was a difference in top level of around 150mm as compared with the adjacent penstock structures. The deformation at the wall ends had resulted in opening of the masonry joint. Adjacent to the 'Tilt Forge' building, there was additional settlement of the top of wall level. This coincided with the location where leakage had occurred prior to the remedial works in 2014. Closer examination of the East Embankment Wall confirmed the presence of several voids along its base and continuing through to the embankment behind. In addition, the wall did not appear to extend below the ground level at the reservoir's edge and no foundations were apparent.

Minor seepage of water into the 'Tilt Forge' building was observed through the walls in the north-west corner, adjacent to the reservoir. There was also some water ingress through the floor in the south-west corner of the building. From discussions with museum staff, the water ingress had been occurring for a long time and had not been completely solved by the 2014 remedial works. In July 2023, when the reservoir water level was 750mm below Top Water Level, no water ingress through the walls of the 'Tilt Forge' building was observed.

During the initial visits, the museum staff reported water ingress within the 'Grinding Hull' building. The walls of the 'Crown Wheel' room, adjacent to the waterwheel and closest to the reservoir, were damp at the time but no notable water ingress was observed there or elsewhere in the building. There was evidence of historical movement of the south (external) dividing wall between the 'Crown Wheel' room and the waterwheel pit. The masonry to the end of the wall had been displaced towards the opening for the waterwheel axle. In addition, there was displacement and bulging of the wall face into the 'Crown Wheel' room.

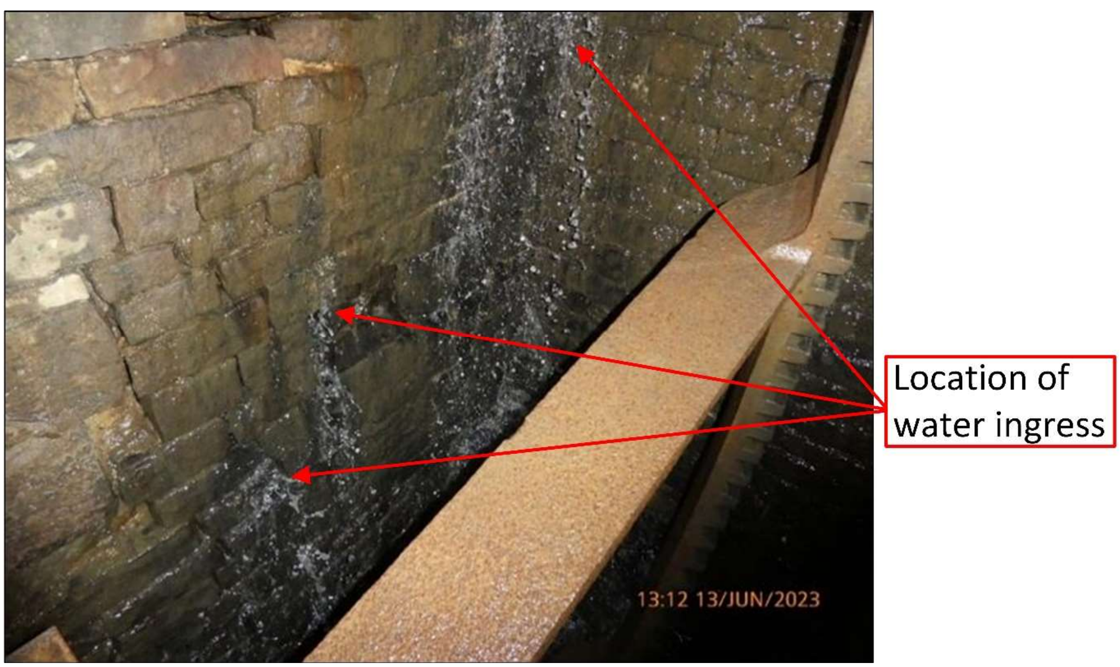

Figure 6. Significant water ingress through the walls of the 'Grinding Hull' building

Down

At the end of May 2023, there were reports of notable water ingress into the 'Grinding Hull' building. A subsequent visit confirmed significant ingress of water through the masonry walls of the building, especially in the 'Crown Wheel' room (Figure 6). The source of the leakage was undetermined at that time. It was obvious that the situation had substantially changed and a leak from the reservoir was suspected. Emergency remedial action was required and the Client was requested to arrange draw-down of the reservoir as quickly as possible, monitor the situation and provide regular updates.

EMERGENCY REMEDIAL ACTION

Water ingress had filled the cavities within the 'Grinding Hull' building walls, pressurising within them, and escaping through open joints in the masonry. There was concern this would cause further displacement and damage to the walls with the potential for their failure. This would be catastrophic for the Listed building and risked an uncontrolled escape of reservoir water. Due to the potential heritage and environmental impacts, a site meeting to discuss and agree emergency remedial action was urgently arranged and attended by representatives of Historic England, South Yorkshire Archaeological Service, the Environment Agency and other key stakeholders.

The following remedial actions were discussed:

- Provide emergency support to the walls of the 'Grinding Hull' building.
- Draw-down the reservoir.
- Perform chemical injection grouting works to seal the leakage path.

Figure 7. Scaffolding support to the walls of the 'Grinding Hull' building

Managing Risks for Dams and Reservoirs

Due to the urgent need to safeguard the heritage assets, as well as address the risk of a reservoir breach, agreement was gained during the site meeting to install emergency propping to the structure. Scaffolding was installed relatively quickly with associated notification to Historic England (Figure 7). As much of this was in and around the 'Crown Wheel' room, the waterwheel was rendered inoperable. In addition, due to concerns about vibrations affecting the structures, instructions were given to the museum staff to not operate the 'Tilt Forge' waterwheel. The 'Grinding Hull' building was closed to staff, except for regular monitoring of the leakage situation and remedial works, and to the general public.

Another course of action was to implement an emergency draw-down of the reservoir's water level. The key aims were to significantly reduce, or stop, the water ingress into the 'Grinding Hull' building and locate the source of the leakage. As the reservoir level could vary during normal operation, a draw-down was not considered to have heritage impacts. However, there were environmental requirements in relation to investigating and rescuing protected species and fish and to manage the risk of silt discharge into the River Sheaf.

Whilst environmental surveys were ongoing, a partial draw-down of the reservoir was implemented to help reduce the water ingress and pressure on the walls of the 'Grinding Hull' building. This had limited impact, indicating that the source of the leakage was deeper within the reservoir. As a result, there was renewed urgency for an increased draw-down. A test for white-clawed crayfish produced a negative result. Therefore, a further draw-down of the reservoir was able to be performed once fish rescues had been completed.

Due to their age and condition, there were concerns that the mid- and low-level drawdown penstocks would not close again once opened, thus, risking complete draining of the reservoir and significant silt discharges into the River Sheaf. Therefore, these were not used for the reservoir draw-down. As the waterwheels were out of operation, it was not possible to use their penstocks to draw-down the reservoir level either. As a result, reservoir waters were pumped out.

Figure 8. Void in the reservoir basin's puddle clay lining adjacent to the 'Grinding Hull' penstock

Once there had been a sufficient draw-down, a void was observed in the edge of the reservoir basin's puddle clay lining, adjacent to the 'Grinding Hull' waterwheel penstock, with water discharging into it (Figure 8). This appeared to be the source of the water leakage into the 'Grinding Hull' building and was at a similar location to that prior to the remedial works in 2014.

Due to the low level of the leakage location, there was concern that a draw down significantly below this level would expose the puddle clay lining creating a risk of it drying and cracking. Conversely, rainfall events could result in inflows greater than the discharge capacity of the pump, thus, resulting in an increase in reservoir levels. As a result, in August 2023, chemical injection grouting of the area between the reservoir and the 'Grinding Hull' building was performed to minimise the leakage risk should the reservoir level rise (Figure 9). During these works, the Contractor reported significant voiding immediately adjacent to the 'Crown Wheel' room and at another location nearby. The total volume of the voids was estimated to be 11.5m³. To grout the deeper voids, the Contractor requested to work from within, and drill through the masonry of, the 'Crown Wheel' room. However, this was not possible due to the presence of the scaffolding supports and the heritage impact. Therefore, grouting was restricted to those areas that could be accessed from outside of the building. On completion, the emergency chemical injection grouting was seen to be successful in controlling water ingress into the 'Grinding Hull' building.

The emergency remedial actions are to remain in place until permanent remedial actions can be completed.

Figure 9. Chemical injection grouting adjacent to the 'Grinding Hull' building (courtesy of Sheffield City Council)

POTENTIAL FAILURE MECHANISMS AND PATHWAYS

From the initial surveys and walkover investigations, the following potential failure mechanisms and pathways were identified:

- Removal of masonry from the end of the spillway channel and erosion of material from beneath the spillway and 'Hand Forge' building due to the action of flows along the River Sheaf. The risk of damage is highest during flood events when there is localised turbulence at the end of the spillway and along the Hand Forge building.
- Insufficient containment of water by the reservoir basin's puddle clay lining. Along the East Embankment, the clay lining extends to the base of the vertical masonry perimeter wall, approximately 700mm below Top Water Level.
- Voids within the vertical masonry perimeter wall and the possible lack of a water-retaining core within the East Embankment. The grouting works performed in 2014 appear to have had limited success in preventing leakage through the embankment.
- The possible presence of permeable and/or poorly compacted fill around the structures formed within the embankment in late 1700s / early 1800s.

The above potential failure mechanisms and pathways are to be reviewed and amended, as necessary, as more information is collated about the reservoir and its associated structures.

FUTURE INVESTIGATION WORKS

Further investigation is required to assess the on-site conditions more fully, provide information to aid the determination of potential failure mechanisms and pathways and to assist the development of future remedial measures. The following investigation works have been proposed:

- Topographical and bathymetric surveys
- Culvert survey
- Ground investigation
- Survey of the drystone wall that forms the right (south) wall of the spillway channel.
- Heritage surveys
- Environmental surveys

As the function of the Abbeydale Industrial Hamlet museum has been detrimentally impacted by the reservoir draw-down and the temporary support scaffolding within the 'Grinding Hull' building, there is a desire to complete permanent remedial measures as quickly as possible. The types and extents of the investigation works have been developed with this aim in mind with a focus on providing the essential information required within a suitable timescale. In addition, development of the investigation works has considered the site constraints, especially as regards access limitations, a requirement to minimise heritage and environmental impacts and a need to obtain the necessary consents.

At the time of writing, the investigation works detailed above were due to be commenced.

FUTURE REMEDIAL MEASURES

As detailed earlier, there have been repeated leakages at the reservoir. Previous remedial works appear to have been targeted at specific issues. Whilst the works in 2014 were more extensive in nature, as compared with those previously, they provided short-term benefits. The leakages adjacent to the 'Overflow and Spillway Penstock Structure' and the 'Grinding Hull' waterwheel penstock have re-established since and the re-pointing to the East Embankment's upstream wall has now been largely eroded.

A need for remedial measures that successfully address the risk of leakage and damage in the medium- to long-term has been recognised. There is a requirement for these to be implemented in ways that respect the heritage status of the site and in keeping with the original appearance. The Client also requested details of proposed works so that the necessary funding could be raised. As a result, the following outline proposals were developed:

- 'Grinding Hull' penstock structure and building: Grouting works, using bentonite-cement grout, to infill any remaining voids within the ground between the penstock structure and the 'Grinding Hull' building.
- 'Grinding Hull' and 'Tilt Forge' penstock structure: Provision of a concrete cut-off beam between the concrete aprons, located upstream of the penstocks, with reinstatement of the puddle-clay lining above. The aim is to reduce the risk of a flow-path between the edge of the puddle clay lining and the face of the existing structure.
- East Embankment: Replacement of the upstream vertical masonry wall, for 24m approximately, between the 'Tilt Forge' penstock structure and the 'Overflow and Spillway Penstock Structure'. The base level of the new wall is to be 600mm deeper than existing and formed upon a concrete slab foundation that will extend under the reservoir's puddle clay lining and form a cut-off key. The masonry from the existing wall is to be used to face its replacement.
- East Embankment: Reconstruction of the upper section of the existing embankment along the line of the replacement upstream wall. Puddle clay, or another suitable water-retaining approach, is to be provided to the back of the replacement wall and key into its concrete slab foundation. The aim is to continue the water retaining element of the reservoir above Top Water Level, to just below the final embankment crest level. The crest level will be reinstated to match the levels of the adjacent structures and, thus, address settlement that has occurred since its original construction.
- East Embankment: Grouting of the void that extends down the side of the 'Overflow and Spillway Penstock Structure'. Fast-setting chemical grout is proposed to seal the lower part of the void adjacent to the spillway culvert and address the risk of grout loss into the spillway and River Sheaf. It is anticipated this work will be performed in advance of, or in parallel with, the embankment reconstruction.
- Spillway Channel: Removal of the downstream end of the existing spillway channel and provision of a concrete foundation slab and wall backing upon which the masonry will be reinstated. At the downstream end, the concrete will extend below the riverbed to provide a scour protection key. The void under the 'Hand Forge' building will be infilled.

Managing Risks for Dams and Reservoirs

The aim of the remedial measures is to restore the watertightness of the reservoir, thus, stabilising the current situation and addressing the risk of further damage to the Scheduled Monument and Listed structures. Remediation of the damage already incurred to the 'Grinding Hull' building is not included, as this was present prior to the latest leakage, and will require addressing separately. However, completion of the remedial measures will address the risk of water ingress into the walls and permit the removal of the current temporary scaffolding supports.

Detailed design of the remedial measures will be progressed after the results of the investigation works are confirmed. Modification of some, or all, of the measures detailed above may be implemented once an improved understanding of the structures is gained.

CONSTRAINTS AND CONSENTS

Access limitations form a key constraint at the site. There is no direct vehicle access into the reservoir or onto the embankment. The primary pedestrian access involves walking through a room in the 'Hand Forge' building (currently used by a blacksmith), crossing a timber footbridge over the spillway and then walking up steps onto the top of the 'Overflow and Spillway Penstock Structure'. An alternative access onto the embankment is possible from the north via a narrow walkway over the top of the 'Grinding Hull' and 'Tilt Forge' penstock structure. The embankment crest is approximately 1.5m wide. As a result, the outline proposals for the remedial measures aim to avoid the use of large equipment or the need to lift plant, equipment and materials over the Listed buildings. Access across the reservoir basin was implemented for the remedial works in 2014. Whilst this remains an option to be considered, there is an associated risk of damage to the basin's puddle clay lining.

Consent is required from Historic England for any works that will impact the Scheduled Monument. In addition, Listed Building Consent is required from the Local Planning Authority (LPA) for works that will impact a Listed building. Consents are necessary for the investigation works as well as the permanent remedial measures. As a result, there is a need for an understanding of the history, significance and construction of the site and its components so that the impact of any works can be assessed. Consultations with the key stakeholders will be required to agree the proposed work approach and permitted materials. A sufficient level of detail needs to be included within the consent applications to describe the works proposed and how impacts to the structures are to be minimised and mitigated.

Surveys to determine the presence, or otherwise, of protected species at the site are currently ongoing. Several established trees are present along the line of the spillway and there is an aim to minimise impact to these. The presence of Japanese Knotweed has already been confirmed at specific locations within the site. Further consultation with key stakeholders, including Natural England and the Local Planning Authority, will be undertaken to agree the mitigations required and applications submitted accordingly. It has been recommended that the services of a terrestrial and freshwater ecological Clerk of Works (ECoW) services be provided during construction of the works.

For the works to the downstream end of the spillway channel, environmental permits are required from the Environment Agency for works affecting a main river and its floodplain.

SUMMARY

The situation at Abbeydale Industrial Hamlet highlights some of the issues associated with working on a non-registered reservoir, for which records are often more limited. Whilst, there has been an aim to keep the reservoir and associated structures in good, operational condition, due to their importance within the working museum, previous works to address leakage and damage have had limited, short-term success.

As detailed in this paper, works to resolve the current situation are ongoing. Surveys and investigations performed to date have provided an initial understanding of the reservoir's construction and the issues associated with it. Based on this knowledge, it has been possible to implement emergency remedial works and develop outline details for permanent remedial measures. However, an improved understanding is required to facilitate the preparation of detailed designs for construction purposes.

The issues of working on a heritage structure have also been indicated. The access facilities are not in accordance with those expected of modern structures and, therefore, this poses particular challenges for inspection and maintenance works. For Abbeydale Industrial Hamlet, there is also a need to be respectful of the history and forms of construction used and to minimise and mitigate any impacts of new works. Due to the Scheduled Monument and Listed building designations, specific agreements and consents will be required and these will have an impact on the design, construction and programme of the works.

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The British Dam Society

The 2020 national seismic hazard maps for the United Kingdom

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SYNOPSIS The 2020 seismic hazard maps for the United Kingdom (UK) update the previous national maps published in 2007 and are intended for use with the National Annex for the revised edition of Eurocode 8. The 2020 national seismic hazard model uses an up-to-date earthquake catalogue for the British Isles, for which the completeness periods have been reassessed. It also uses a modified version of the 2007 source model and incorporates some advances in ground motion modelling since 2007, including host-to-target adjustments for the ground motion models selected in the logic tree. For the first time, the national maps for the UK are provided for not only peak ground acceleration but also spectral acceleration at 0.2s and 1.0s for 5% damping on rock and the return periods of 95, 475, 1100, and 2475 years. The maps confirm that seismic hazard is generally low in the UK and is slightly higher in North Wales, the England-Wales border region, and western Scotland. We disseminate the updated seismic hazard maps via a dedicated webpage, downloadable data, models and outputs, interactive mapping tools, linkages with professional bodies and industry, as well as public seminars, webcasts, and attendance in scientific conferences.

INTRODUCTION

We have developed a new national seismic hazard model (NSHM) and accompanying national hazard maps (Mosca et al., 2020, 2022) for the United Kingdom (UK), an intraplate region with low levels of seismicity. The 2020 seismic hazard maps update the previous maps published by Musson and Sargeant (2007; hereafter referred to as MS07). The key changes between the 2007 and 2020 NSHMs are the following:

- The earthquake catalogue has been extended from June 2007 to 31 August 2018. Data from the earthquake catalogue of Manchuel et al. (2018) for France and the International Seismological Centre Bulletin database (ISC, 2021) have been used to improve data completeness in the English Channel, Northern France, and the North Sea.
- All magnitudes have been converted to Mw using the relation of Grünthal et al. (2009). This is an update of Grünthal and Wahlström (2003), which was used by MS07.
- The catalogue analysis, including the assessment of completeness and declustering, uses transparent and reproducible approaches.

- The seismic source characterisation (SSC) model, including the zone geometry, the maximum magnitude, and the computation of the earthquake recurrence parameters, has been modified with respect to MS07.
- A new ground motion characterisation (GMC) model that accounts for advances in ground motion modelling since 2007 has been used. This includes host-to-target adjustments (HTTAs) for the selected ground motion prediction equations (GMPEs) in the GMC model.
- The national seismic hazard maps have been computed for a larger area, which also includes the Shetland Islands, than in MS07.
- The national maps describe the hazard in terms of spectral acceleration at 0.2s ($SA_{0.2\ s}$) and 1.0s ($SA_{1.0\ s}$) to meet the requirements of Eurocode 8 and the drafting of a National Annex for the revised edition of Eurocode 8: Design of structures for earthquake resistance.

Engineers from the British Standards National (BSI) committee B/525/8 for Structures in Seismic Regions (the committee responsible for the UK input to Eurocode 8) guided the design requirements for the seismic hazard maps. This ensures that the 2020 maps are used to guide the application of the revision of the Eurocode 8 in the UK calibrating the design seismic requirements to the seismicity levels of the country. Although the UK is a low seismicity region and the design seismic action is not required for standard residential and commercial buildings, design seismic action is recommended for buildings with high economic, social, and environmental consequences (e.g. chemical power plants and dams) where the exceedance of the regional hazard at a specific site is above a certain threshold (Booth et al. 2008; BS NA EN 1998-1 2008).

SEISMO-TECTONIC CONTEXT

The UK lies in the northwest part of the Eurasian plate at the northeast margin of the North Atlantic Ocean, approximately 1,500km northeast of the Mid-Atlantic Ridge and around 2,000km north of the plate boundary between Africa and Eurasia. As a result of this geographic position, the UK is characterised by low levels of earthquake activity (Figure 1; e.g. Musson, 2012a). Evidence for this comes from observations of earthquake activity dating back several hundred years, which suggests that although there are many accounts of earthquakes felt by people, damaging earthquakes are rare. The observed seismic activity in the British Isles provides evidence of ongoing local crustal deformation. However, the nature of the crustal strain field and its relation to the observed distribution of earthquake activity is still not clearly understood due to very low strain rates in the region. Tectonic stresses generated at the Mid-Atlantic Ridge due to forces acting perpendicular to the spreading ridge, as well as strains resulting from the collision of Africa with Europe, are expected to result in a uniform stress field with approximately NW–SE-oriented compression and NE–SW-oriented extension (e.g. Gölke and Coblentz, 1996; Heidbach et al., 2016). This stress field will result in the tectonic loading of existing fault structures.

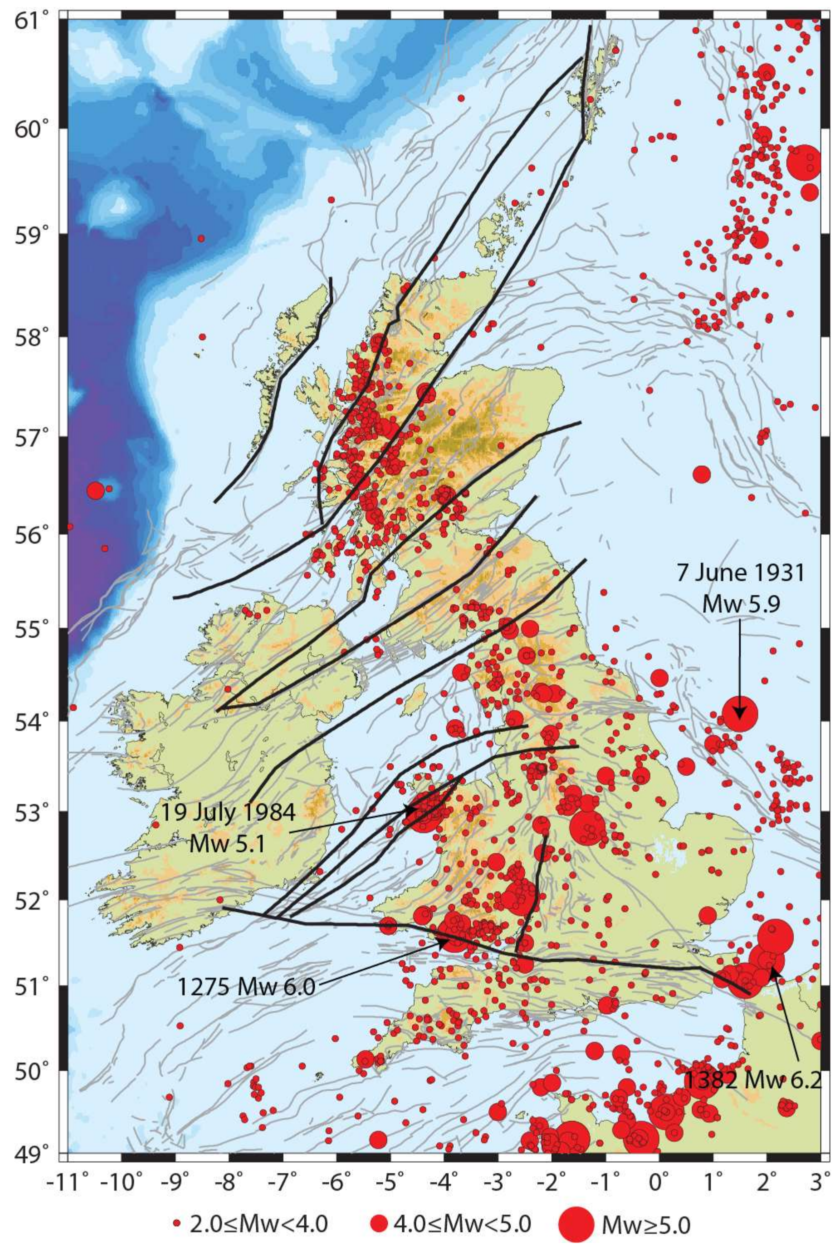

Figure 1. Seismotectonic map for the British Isles. Faults (thin grey lines) and major tectonic structures (bold dark grey lines) are from the British Geological Survey DigMapGB series. Red circles show earthquakes and are scaled by magnitude.

Seismicity in the British Isles is concentrated in a north-south band along the length of Britain, mainly along the western flank. This band gets wider moving south. The northeast of Britain, the northwest Atlantic margin and Ireland all show an absence of notable seismicity (Figure 1). The geographical distribution of instrumentally recorded earthquakes from 1970 to the present generally follows the distribution of historical seismicity over the last 300 years but with a generally smaller magnitude. There are a few exceptions to the correlation between instrumental and historical seismicity, such as the historical earthquakes in the Dover Straits, SW Wales and around Inverness in NE Scotland, where there has been relatively little instrumentally recorded seismicity. This highlights the fact that instrumentally recorded seismicity is not a reliable indicator of earthquake activity either in the past or in the future.

In common with many regions of diffuse intraplate seismicity, it is difficult to unequivocally associate earthquakes in the entire study area with specific faults for the following reasons. Firstly, no earthquake recorded either historically or instrumentally has produced a surface rupture. Secondly, uncertainties in the epicentral location and depth of the earthquakes are typically several kilometres.

The largest earthquakes in the study area are the ~6.0 Mw 1275 and ~6.2 Mw 1382 events in South Wales and the Dover Strait (Figure 1), respectively, but their magnitude and location estimates are associated with large uncertainties. The largest instrumentally recorded earthquake in the UK catalogue occurred on 7 June 1931 (5.9 Mw) in the Dogger Bank area of the North Sea; whereas, the largest (4.9 Mw) onshore earthquake in the UK since 1970 occurred on 19 July 1984 near Yr Eifl on the Lleyn (or Llŷn) Peninsula in northwest Wales.

NATIONAL SEISMIC HAZARD MODEL

Figure 2 shows an overview of the logic trees used for both the SSC and GMC components of the 2020 NHSM.

The SSC model consists of a single seismic source model with 22 source zones, each of which is an area where seismicity has an equal probability of occurring anywhere within it. It draws heavily on previous regional source models, including MS07 and the 2013 European Seismic Hazard Model (ESHM13) of Woessner et al. (2015), with some additional modifications to account for recent developments in the understanding of tectonics in the UK. We used the maximum magnitude (Mmax) distribution proposed for the British Isles by Meletti et al. (2009) for the ESHM13 model. It consists of four values (6.5, 6.7, 6.9, and 7.1 Mw with weights of 0.5, 0.2, 0.2, and 0.1), which were applied to all zones. The distribution for the hypocentral depths is between 5km and 20km, with a modal depth of 15km, as proposed by MS07. Strike-slip faulting, with north-south or east-west fault planes, has the highest weight, in agreement with calculated fault plane solutions for instrumentally recorded earthquakes in the last 30 years (Baptie 2010). The expected frequency-magnitude distribution (FMD) for each seismic source zone is quantified using the Gutenberg-Richter frequency-magnitude law (Gutenberg and Richter, 1954). The results of the FMD are expressed by a 5×5 matrix of possible values for the recurrence parameters (i.e. the activity rate *a* and the *b*-value), determining 25 triplets of *a* and *b* and their weight to account for the uncertainty in these parameters.

The GMC model consists of five GMPEs that were considered to be applicable for the UK. Specifically, these are Atkinson and Boore (2006), Rietbrock et al. (2013), Bindi et al. (2014), Boore et al. (2014), and Cauzzi et al. (2015). Since the strong motion recordings for the UK consist only of weak motion recordings and contain few recordings at near source-to-site distances, the selection of the suite of the GMPEs for the GMC model, together with the assignment of their weights, combines: (1) the results from the comparison of the ground motion predictions computed from candidate GMPEs with the recorded ground motions in the UK using various statistical approach; (2) the outcome from a workshop involving key experts on ground motion modelling. We corrected the ground motion predictions from the five GMPEs for the HTTAs using the approach of Al Atik et al. (2014) to account for differences in site conditions between the host regions, for which the GMPEs were derived, and the target region (i.e. the UK). This process accounts for both the effects of elastic amplification due to shear wave velocity structure and near-surface attenuation at a site, which is described by the parameter κ_0.

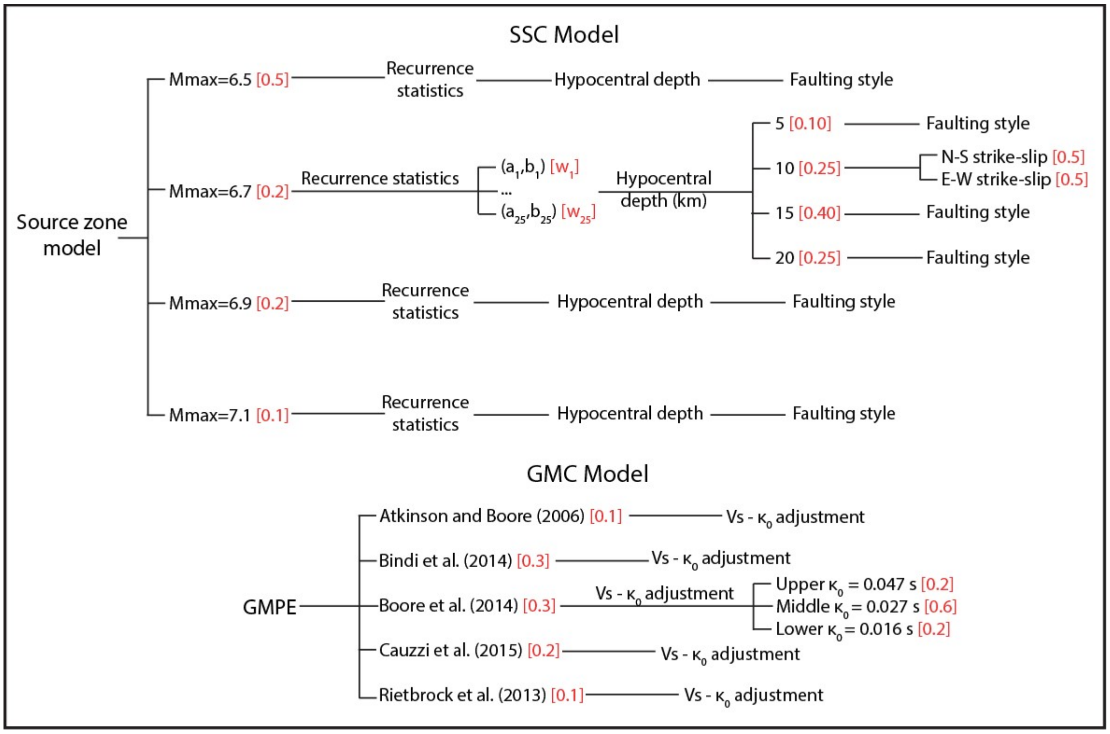

Figure 2. SSC and GMC logic tree for the NSHM for the UK .

NATIONAL SEISMIC HAZARD MAPS

We calculated the hazard using Monte Carlo-based probabilistic seismic hazard analysis (PSHA) to generate artificial catalogues by random sampling of the probability distributions in the SSC model (Musson, 2000). Musson (2012b) and Mosca (2019) show that the Monte Carlo-based approach is compatible with the Cornell-McGuire type approach for PSHA and provides the same output given the same initial model.

The minimum magnitude (Mmin) in a hazard calculation is defined as the threshold for potentially damaging earthquakes (e.g. Bommer and Crowley, 2017). Here, we used Mmin of 4.0 Mw to include the probability that the impulsive nature of small earthquakes and their high-frequency content could be potentially causing damage.

The hazard calculations were carried out for the region between 49°N - 61°N and 8.5°W - 2°E for a grid of 4141 points spaced 0.125° in latitude and 0.25° in longitude. We computed the hazard for peak ground acceleration (PGA), $SA_{0.2\ s}$, and $SA_{1.0\ s}$ with 5% damping for Vs30 (time-averaged shear wave velocity for the top 30 m) of 800 m/s and the return periods of 95, 475, 1100, and 2475 years. Figures 3 and 4 show the national hazard maps for return periods of 475 years (10% annual frequency of exceedance in 50 years) and 2475 years (2% annual frequency of exceedance in 50 years), respectively. For 475 years, PGA is less than 0.04g for most of the UK, except for North Wales and the England-Wales border region where the hazard reaches around 0.09g and 0.05g, respectively (left panel of Figure 3). A similar spatial variation is observed at 0.2s but the effects are more pronounced (central panel of Figure 3). At 1.0s, accelerations are smaller than 0.02g (right panel of Figure 3) but show less variation across the UK. For a return period of 2475 years, the Channel Islands, North Wales, the England-Wales border region through to North Central England, the Lake District and north-west Scotland are the areas of highest hazard for PGA and $SA_{0.2\ s}$ (Figure 4). The highest hazard values (0.25g for PGA and 0.47g for $SA_{0.2\ s}$) are observed around Snowdonia, in North Wales.

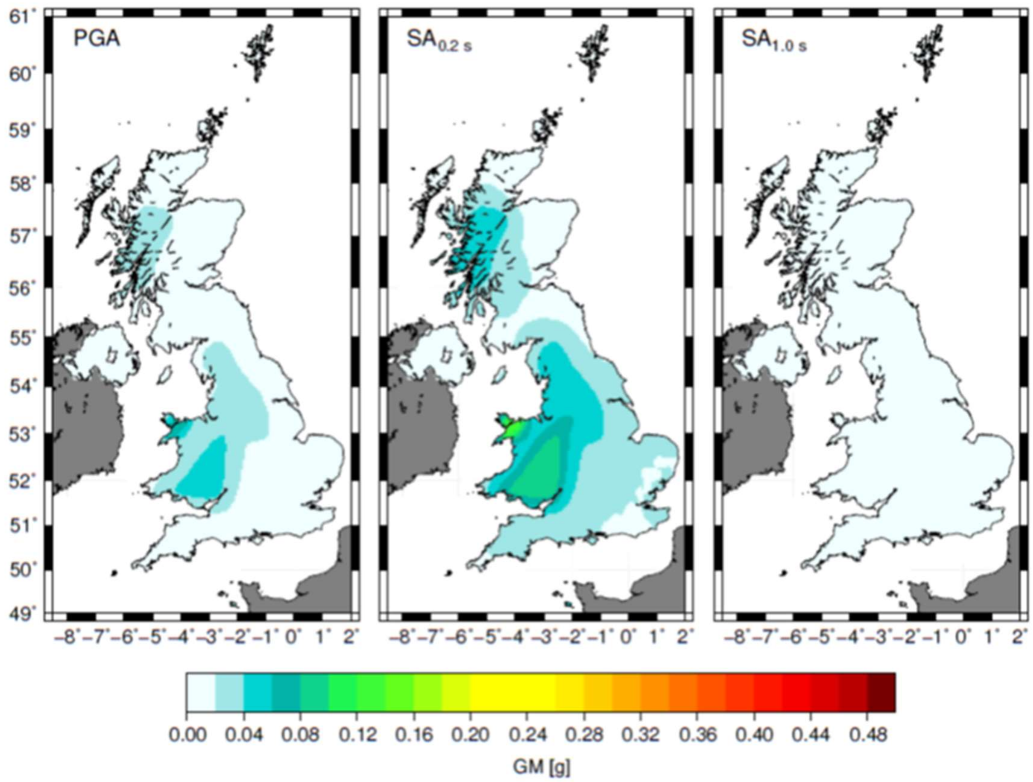

Figure 3. Hazard map for PGA, $SA_{0.2s}$, and $SA_{1.0s}$ at the 475-year return period.

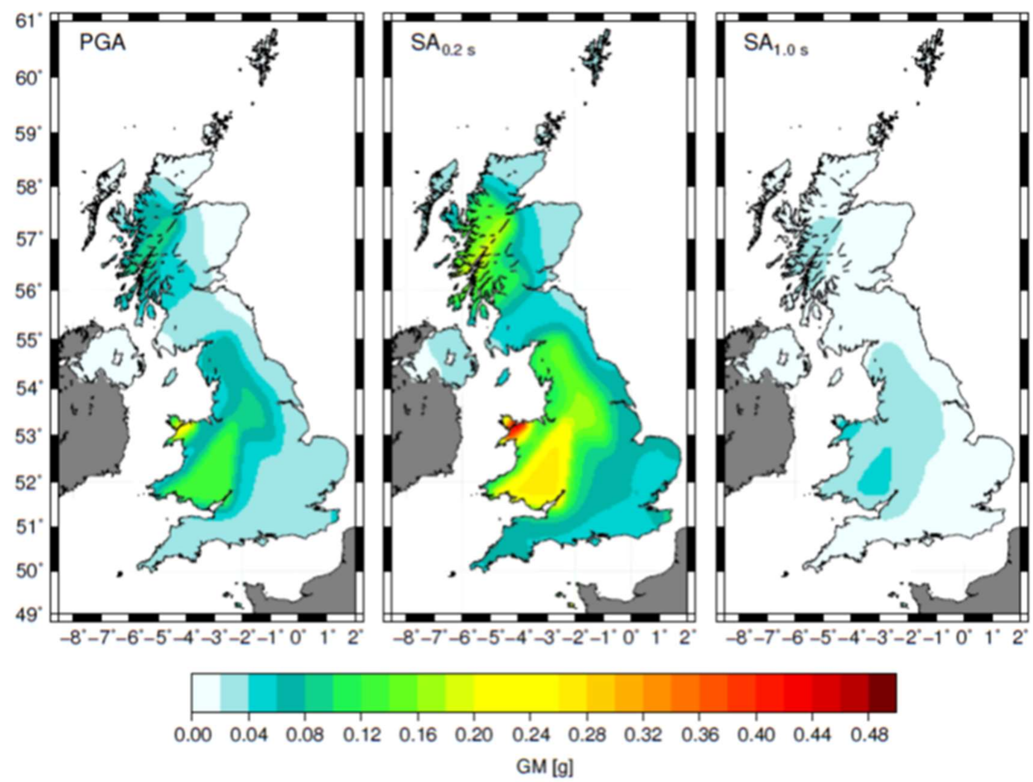

Figure 4. Hazard map for PGA, $SA_{0.2s}$, and $SA_{1.0s}$ at the 2475-year return period.

DISSEMINATION OF THE RESULTS

To increase the visibility of the 2020 NSHM for the UK and make it accessible and available to a wide range of users, we used various channels and tools.

The products of the NSHM are accessible to the public through a dedicated webpage (http://www.earthquakes.bgs.ac.uk/hazard/UKhazard.html) and an interactive mapping tool (https://www.bgs.ac.uk/map-viewers/geoindex-onshore/). The former allows users to download all elements of the NSHM model and the output files in text format. The latter allows users to view the hazard maps interactively, navigate to a specific area of interest, query the maps, and download the hazard values at a specific location or area of interest. It is the first time that the seismic hazard maps for the UK are interactively accessible to the public. Furthermore, accessible data to the public ensure the transparency of the hazard model.

To promote the work with end-users (e.g. the engineers' community in the UK), we communicated the results of this project to professional bodies, such as BSI committee B/525/8 and the Institution of Civil Engineers (ICE), and presented them in a public talk of the Society for Earthquake and Civil Engineering Dynamics (SECED). We also disseminated the 2020 national hazard maps on the BGS website (https://www.bgs.ac.uk/news/developing-new-seismic-hazard-maps-for-the-uk/) and the ICE website (https://www.ice.org.uk/news-and-insight/the-civil-engineer/november-2020/updated-seismic-hazard-maps-for-the-uk) and published them in a peer-reviewed journal (Mosca et al., 2022). Finally, we presented the outcomes of this project at a number of scientific conferences, e.g. the SECED conference in September 2019, the annual meeting of the Seismological Society of America in April 2021, the 3rd European Conference on Earthquake Engineering and Seismology in September 2022.

CONCLUSIONS

We have developed the 2020 seismic hazard model for the UK and accompanying hazard maps for PGA and spectral acceleration at different return periods using a Monte Carlo approach for PSHA and objective and reproducible data-driven analyses.

National hazard maps are only a first-order approximation of seismic hazard for engineering structures and help to identify regions of high seismic hazard to inform the need for site-specific risk assessments. The decisions to construct the seismic hazard model are not driven by the specific site of interest as it happens for site-specific PSHA but are taken uniformly across the region (e.g. Musson and Sargeant 2007; Gerstenberger et al. 2020). A site-specific assessment might be required if the hazard exceeds some given threshold at the site after the appropriate site conditions for the site are taken into account. Also, NSHMs usually do not consider the hazard for long (≥ 10,000 years) return periods that are important for highly critical structures, such as dams and LNG power plants. To compute the hazard for such long return periods, the effects of distant large earthquakes and the occurrence of earthquakes at very long recurrence intervals should be accounted for. The former requires computing the hazard at longer spectral periods, and the latter requires a detailed geological investigation in the area within 300km of the site to understand when these faults were last active (e.g., IAEA, 2022).

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The British Dam Society

Developing an Understanding of the Reservoir Safety Risks of Non-Statutory Reservoirs

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SYNOPSIS In 2022, Severn Trent Water (STW) appointed Arup to carry out a project to appraise the reservoir safety risks posed by 71 reservoir sites with capacities identified in the range 10,000 to 25,000m^3 above natural ground level. Following the Flood and Water Management Act 2010, which amended the Reservoirs Act 1975 (the Act), it was anticipated that these reservoirs would likely be brought into the Act when the threshold is amended to 10,000m^3; this would increase the number of statutory reservoirs within STW's portfolio. By investigating and studying each reservoir, the project helped STW to understand the potential increase in financial risk which could occur because of additional regulation. This considered both operational requirements and capital works, to ensure the potential statutory reservoir safety risks posed by the reservoirs are minimised and managed in good time.

The paper explains the methodology that was applied to carry out the assessment, together with the key themes discovered, including common reservoir safety risks and recommended mitigation actions, as well as an exploration of the challenges and opportunities of the process. In conclusion, the recommendations made in relation to reservoir safety risks of the non-statutory reservoirs, how STW used the outputs to feed into their asset management planning process and the next steps that STW is taking to manage the risks identified are all described.

NEED FOR THE PROJECT

Whilst STW has over 60 statutory reservoirs of all types, there are some 700 smaller reservoirs, tanks and other water retaining structures in the business's asset portfolio. Discussions within the reservoir safety community indicated that it was likely that the Act's applicability would be extended in England by reducing the retained volume minimum criterion from 25,000m^3 to become consistent with Wales, at 10,000m^3. Since a large proportion of STW's smaller assets lay within this volume range, it was considered prudent to anticipate such a change in the legislation and carry out some further desk study work to understand the potential magnitude of future investment requirements at these assets, building on an earlier study by Mott MacDonald some ten years ago.

SCOPE AND METHODOLOGY

The main objective of the project was to help STW to understand the reservoir safety risks posed by a number of their reservoir sites with capacities identified in the range 10,000m^3 to 25,000m^3. For each site, Arup was commissioned to carry out a desk study, supported by a site visit for the impounding reservoirs, and advise, from an All Reservoir Panel Engineer's viewpoint, on any potential issues which may become safety matters should each reservoir become subject to the amended volume criterion of the Act.

Supported by the previous Mott MacDonald study, STW carried out initial screening of their sites to provide a list of sites where the reservoir assets could fall within that range. The list comprised the following:

Table 1. Candidate, Non-Statutory, STW Reservoirs

Reservoir Type	Number in ARUP Study
Impounding	
Flood Storage	4
Other Impounding	1
Sludge Lagoon	7
Non-Impounding / Other	
Service	54
Tank	4
River Weir	1

Available data was provided by STW for each of the reservoir sites. This typically included a reservoir data sheet, schematics and information about the operation of the assets, internal inspection reports (for service reservoirs and tanks) and occasionally design and construction drawings and reports and monitoring data.

The project was overseen by two All Reservoir Panel Engineers (ARPE). Site visits were undertaken to the 12 open reservoir sites by these engineers, of which seven were found to contain more than one reservoir.

A spreadsheet report template was developed collaboratively and agreed with STW, as the main deliverable of the project for each site. The report template included a summary; list of data used; information about the reservoir; reservoir condition; findings and recommendations; and site visit notes and photos. Each spreadsheet report was approved by an Arup ARPE before issue to STW.

For each site, based on estimates of the total capacity of the reservoir and the capacity of the reservoir above natural surrounding ground level, the report indicated the likelihood of it being classed as a Large Raised Reservoir under a potentially amended Act. The accuracy of the estimates was limited by the information available; this typically included top water levels and tank dimensions from operational and inspection reports, which was sometimes augmented by as-built records. Google Earth Pro and LIDAR data were used to supplement the estimates.

Key information about the intrinsic and current condition of the reservoir assets was reported, and the resulting key risks to reservoir safety were determined. The Risk Assessment for Reservoir Safety guidance, RARS (EA, 2013) was used to help identify key threats and risks. Methods to mitigate each of the risks were reported, and recommendations to minimise or better understand these risks were made. The likely cost of implementing recommendations was estimated, based on high / medium / low cost categories. Operational issues that could affect the implementation of each recommendation were noted in the reports.

Each recommendation was assigned a risk rating and, for the reservoirs that would possibly or definitely fall under a potentially amended Act, an indicative timescale for carrying out the recommendation was proposed. The indicative timescales were those which Arup ARPEs would typically suggest in an inspection report, itself completed after designation of the reservoir by the Enforcement Authority. Whilst actions on those reservoirs that were considered not likely to fall under an amended Act were not given timescales, a risk rating was still assigned, as ideally such recommendations would still be implemented to reduce identified risk.

A high-level screening was undertaken to help understand the likely 'high risk' / 'not high risk' classification for each reservoir, which would need to be confirmed at the time of designation. Ordnance Survey contour maps were used to understand the likely direction of flow in the event of a breach, taking a conservative approach to possible flow paths where topography was uncertain. Maps and satellite imagery were inspected to identify possible receptors along those flow paths. Where a potential impact of a breach on sensitive receptors, such as residential properties, community facilities and roads, was identified the reservoir was conservatively assumed to be 'high risk'.

Each draft report was submitted to STW to allow the relevant area teams to review and comment prior to a final report being issued.

OUTPUTS AND KEY THEMES

Volume Classification of Non-Statutory Reservoirs

The study found that, of 71 English reservoirs examined, 45 reservoirs are likely to fall under a potentially amended Act in England, i.e. if a large raised reservoir (LRR) is defined as having an escapable volume in excess of 10,000m^3. As shown in Table 2 below, there were an additional 15 reservoirs in the study where this was defined as 'possible'; in many cases this uncertainty was due to not having level data of sufficient accuracy. The study found two sites where open reservoirs were estimated to have volumes such that they may fall under the current Act. These two sites were: an impounding reservoir which had previously been modified to allow it to be discontinued; and a sludge lagoon of sufficient surface area to suggest that, if contents are proved to be flowable, could have sufficient volume to fall under the Act. Subsequent to the study, STW has carried out more detailed checks of the volumes of these reservoirs and proceeded with registration as appropriate.

Risk Classification of Non-Statutory Reservoirs

The high risk reservoir screening exercise determined that, of the 58 reservoirs that were assessed as likely to be classified as high risk, 15 were assessed as 'possibly' falling under a potentially amended Act, owing to the same uncertainty of their storage capacity above natural ground level described above. Table 2 presents the spread of these findings across different asset types. This screening was necessarily conservative and based on readily available basic data; however, it provided a high level estimate to help STW understand its potential liabilities with regards to reservoir regulation.

Managing Risks for Dams and Reservoirs

Table 2. STW sites reviewed considering the proposed changes to the Act, and outcomes of that review

Reservoir Type	Assets reviewed	Expected to be LRR and 'High Risk'	Possible LRR and 'High Risk'	Total 'High Risk'
Service reservoir	54	35	13	48
Tank	4	1	1	2
Flood storage reservoir	4	4	0	4
Impounding reservoir	1	1	0	1
River weir	1	0	1	1
Sludge lagoon	7	2	0	2
Total	**71**	**43**	**15**	**58**

Common Reservoir Safety Risks and Recommended Mitigation Actions

The report for each reservoir summarised key reservoir safety risks, potential mitigations and recommendations. Many of these risks related to service reservoirs or tanks and reflected common failure modes associated with buried tanks. In many cases, a lack of data meant that it was not possible to fully understand the extent of the risk: for example, not knowing the capacity of a service reservoir overflow pipe. Whilst STW maintains detailed records of their statutory reservoirs, it was found that less information was available for the non-statutory sites and, where this was available, in the case of service reservoirs the emphasis was generally more towards water quality issues. Common themes are summarised in Table 3.

Table 3. Reservoir Safety Risks and Mitigation Recommendations

	Reservoir Safety Risk	Mitigation / Recommendation	% of sites affected
1	Lack of data about reservoir design, construction, or current condition.	Collate available records and/or carry out surveys to understand reservoir design details and inspections to understand current condition.	93%; physical surveys at 75% of the sites
2	Overfilling due to unknown or insufficient overflow capacity leading to pressurisation of roof, structural damage and erosion of fill. Available information is not sufficient to confirm if the existing overflow is adequate.	Collect information about overflow arrangement and confirm or assess adequacy of overflow capacity.	92%
3	Deterioration of reservoir structure - floor and/or wall plus joints leading to leakage and erosion of supporting fill.	Regular internal inspection; "drop" tests; monitor for seepage; surveillance visits.	89%
4	Pressurised pipe failure leads to loss of supporting fill – extent of risk depends on type of inflow and position of the inlet / outlet valves.	Confirm route / condition of pipework / valves; monitor for leakage; surveillance visits.	89%

	Reservoir Safety Risk	Mitigation / Recommendation	% of sites affected
5	Inability to lower the reservoir level in an emergency due to insufficient drawdown capacity. Available information is not sufficient to determine drawdown provision.	Collect information about draw-off/scour arrangements and confirm or assess adequacy of drawdown capacity.	90%
6	Deterioration of underdrainage leading to washout of fill under perimeter walls, or blockage of drainage system. Available information is not sufficient to confirm drainage layout and condition.	Confirmation of washout / underdrain route; internal inspection (CCTV) of underdrains; monitor for seepage; surveillance visits.	80%
7	Excessive pressure variations due to rapid filling or emptying (in the case of a burst on the outlet of a tank) and insufficient vent capacity.	Confirm likely rapid drawdown extents and review ventilation provision.	90%

Magnitude of Cost

In order to prepare future investment plans, STW needed to understand the likely magnitude of cost for additional operational activities and capital work that could result from the candidate reservoirs being brought under the Act in the future. For some candidate reservoirs, there were some direct recommendations for remedial works; however, due to the available data, recommendations for the open reservoirs, service reservoirs and tanks were generally for additional studies or further information gathering.

It is likely that the recommended studies and surveys will comprise only the first stage of project work, although this is no reflection of the safety of the current structures. Remedial works at a proportion of the sites, for example overflow or drawdown capacity improvements, may be required as subsequent work stages to achieve full compliance with the Act. Due to uncertainty regarding the potential nature and extent of follow-on work, the cost of this was not estimated for each site.

Reservoirs that fall under the Act have a necessarily higher level of ongoing management expenditure. As well as general surveillance and maintenance activities, these reservoir-specific activities include:

- Periodic appointment of an ARPE to carry out initial and Section 10 inspections;
- Breach assessment to confirm High Risk/Not High Risk and inform the On Site Plan (if this is not carried out by the Enforcement Authority);
- Supervising Engineer appointment and supervisory duties;
- Preparation and maintenance of a Prescribed Form of Record and On Site Plan; and
- Possible Risk Assessment for Reservoirs Safety Assessment as STW's RARS programme matures.

Managing Risks for Dams and Reservoirs

Operational Issues

The identified operational issues were generally related to the ability to reduce each reservoir's water level to enable internal inspection and/or maintenance work. For many of the service reservoirs and tanks, there was information on the existence of a system bypass, but it was generally not possible to tell from that information whether network water supplies could be maintained whilst a cell was taken out of service. For each reservoir, such arrangements would need to be confirmed, and potentially improved with capital expenditure, to enable regular internal inspection.

Risk rating and timescales

Nominal timescales were assigned to each recommendation and were intended to be applied from the date at which the reservoirs are first inspected following a decision that the reservoirs fall under the Act, if that is confirmed to be the case. A small number of recommendations were assigned higher risk ratings to assist prioritisation of investigations and improvements in the period prior to the potential change in the Act, to reduce reservoir safety risk. For candidate reservoirs where the Act has not previously applied, the absence of an enforceable process of inspection, monitoring and maintenance means that sites may be less well understood and maintained, meaning that some sites have a number of existing issues.

Summary of Study Recommendations

The key recommendation from the study was for further data collection at the majority of sites, to improve understanding of key risks. The detailed next steps were:

1) Analysis of outputs from the study to understand portfolio-wide risks and likely cost of mitigation;
2) Analysis of outputs from the study to highlight any critical specific risks at reservoirs;
3) Topographical surveys to inform capacity assessment;
4) Collection of any additional asset information and data;
5) Internal structural surveys and pipework surveys to inform overflow and drawdown capacity assessments;
6) Overflow capacity assessments;
7) Drawdown capacity assessments;
8) Studies to confirm operational issues, e.g. ability to bypass reservoirs; and
9) More detailed breach assessments to understand if reservoirs would be designated as high risk and to inform emergency planning.

Use of Project Outputs in the Five-Yearly Asset Management Plan (AMP) Process

The project outputs were used in Severn Trent Water's Periodic Review (PR24) submission, to identify, scope, substantiate and price capital work on the current non-statutory asset base, including:

- Work that will be required to facilitate statutory Inspections, such as provision or improvement of ability to isolate cells or reservoirs;

- Supplementary asset information, likely to be required to assist a statutory inspection; this included CCTV surveys of drainage and scour infrastructure, topographical, bathymetric and measured structural surveys, further seeking and collation of asset information etc.; and
- Work items that were highly likely to be included as recommendations from an ARPE's Inspection, such as overflow / drawdown capacity calculation or improvement, instrumentation / monitoring improvements, increased reservoir surveillance.

At the time of writing, the PR24 submission is with OFWAT for review. The draft submission contained the following work elements, based on a balanced and risk-based view, which set out the need to tackle the highest risk assets, by:

- Undertaking a prioritised programme of statutory inspections on 45 of the reservoirs that are expected to fall under the amended Act as potentially being 'High Risk'. This includes the employment of specialist staff to carry out the inspections, together with smaller investments required to monitor these sites;
- Providing overflow upgrades at 13 service reservoirs and 5 lagoons to meet the likely enhanced asset standard required under the Act;
- Enhancing two service reservoirs to support the structural changes required to ensure that STW can discharge its duties in line with the amended Act; and
- Additional Environment Agency charges for the regulation of statutory reservoirs (e.g. registration, annual subsistence).

CHALLENGES AND OPPORTUNITIES

Output Format

A report was prepared for each site as the main deliverable of the project. STW requested that this be completed in a spreadsheet format so that it could potentially be easily integrated into a Prescribed Form of Record template, should the sites become registered under the Reservoirs Act. Formats for the open reservoirs and service reservoir/tanks reports were prepared on that basis, and included guidance notes and references to typical failure modes referring to table 7.2 of RARS (EA, 2013). Following the completion of the initial batch of reservoirs, the format was reviewed and amended to take on feedback from STW and the Arup project team. Due to the varied nature and formats of the data available about each site, it was not possible to fully automate the collation of data into the report format.

Background Data

Owing to the nature of the sites studied, positioned in the non-statutory range, the available recorded information varied in type, extent and quality from site to site. Whilst the sites are closely managed from a water hygiene standpoint (in terms of 10-yearly surveys, reservoir cleaning programme and bacteriological performance), the structural data holdings, including as-built records, structural surveys and inspections, are less well-developed. The additional historical complications of depot and office moves and closures, evolving boundaries of areas and responsibilities and data degradation also contributed to the challenge of locating and acquiring definitive records. Whilst digital business continuity plans generally maintained sufficient information for that discrete purpose on each asset (such as generalised construction, levels and volumes), fuller engineering details and drawings are not routinely stored in this format. This necessitated extensive hard copy archive work at physical locations across the business, which benefitted the project through the acquisition of drawings and details for the majority of cases.

Data Uncertainty and Drawing Conclusions

The availability and quality of data impacted on the preparation of the reports for each reservoir. Several of the service reservoir and tank reports were prepared on very limited data which meant that the findings and recommendations were more generic and had to reflect typical concerns for that type of reservoir.

There were instances where there was conflicting data about a reservoir; for example, the capacity of a service reservoir may be stated differently on a key information sheet and an internal inspection record. Drawings were used to confirm information where available; otherwise, engineering judgement was used, and any differences were noted in each report.

Project Management & Execution

As a collaborative team, it was considered important that the following issues were resolved, ideally at a very early stage or even before the project started:

- A clear, resourced, project programme, with sites logically batched in terms of assessment and report delivery;
- Realistic programme time assumptions on initial background data sourcing and exchange;
- Ability to be light-footed within the programme to absorb time risks and maintain effective delivery;
- A small, dedicated team for consistency of reporting;
- Supporting resources for data seeking, arranging / hosting site visits and reviewing draft reports;
- A secure means of organising and sharing often quite large sets of digital data, and exchanging and collaborating consistently on many reports for drafting and review;
- The fullest possible data set for each candidate site, to enable the consultant to most effectively review, assess and report in one iteration; and
- A template output (in the case of this project, an Excel file with content loosely modelled on the Prescribed Form of Record) with scope to flexibly accommodate differing sites, inputs and outputs.

CONCLUSION

The study undertaken provided an initial assessment of STW's potential statutory reservoir holdings, and a priced evidence base to support STW's submission to OFWAT for the future safe introduction and management of these reservoirs. The study highlighted that, for a water company, the majority of reservoirs requiring regulation once an amended Act is implemented are service reservoirs and tanks. It also concluded that the collection of further asset information is required to be able to more fully understand the potential reservoir safety risks. The potential for sludge lagoons to be included in an amended Act means that additional training could be required for the operational waste teams that manage these assets, assuming they are not as familiar with the requirements of the Act as raw water operational teams.

From each of the parties' standpoints, the study provided benefits in terms of:

- The client acquired clearer definition of additional tasks to be carried out, including further data seeking and substantiation, prior to the potential statutory change; and
- The knowledge of the consultant's engineers was improved on the wide variety, condition and age of service and other reservoirs typically operated by a water company.

REFERENCES

EA (2013). *Guide to risk assessment for reservoir safety management - Report SC090001* (RARS). Environment Agency, Bristol, UK

The British Dam Society

Overflow and outlet screens

J BENN, JBA Consulting
J HOWARD, JBA Consulting
C DALE, JBA Consulting

SYNOPSIS Overflow/outlet screens are often fitted to reservoirs to prevent human exposure to hazards, to catch large debris, or to prevent fish and mammals being washed downstream.

Whatever its primary purpose, a screen will collect debris and block temporarily. This blockage can lead to an increase in reservoir water level and could alter the stage-discharge relationship of the overflow or outlet. Furthermore, blocked screens will reduce the available freeboard and overflow capacity. Their impact must therefore be considered in reservoir flood studies and the design of outlet structures. This is particularly important for flood storage reservoirs (FSRs) that operate infrequently and rely on maintaining the design stage-discharge relationship to achieve the required flood attenuation.

Case studies are presented concerning two FSRs fitted with self-activating flow-control devices on their outlets that failed to operate as anticipated on first filling. In both cases the unexpected operation was attributed to downstream screens fitted to mitigate perceived hazards. A third case study concerns the impact of a 'fish' screen placed in the overflow spillway of an amenity lake.

This paper summarises research on the impact of screen size on fish and mammal passage, and on debris movement, in particular the relationship between debris volume and bar spacing. It looks at some alternative screen design and management measures to reduce the impact on reservoir water level and overflow capacity.

INTRODUCTION

Screens can fulfil several purposes on a reservoir, the main ones being:

- Trapping of debris which would otherwise present an unacceptable risk of blockage to an overflow, gate or outlet ('debris screens') (Figure 1),
- Reduction of exposure to hazards and hence harm from accidental or intentional entry ('security screens') (Figure 2),
- Prevention of fish / bird / mammal 'wash-out' into an overflow or outlet ('fish or mammal screens') (Figures 3 and 4).

Figure 1. Multi-stage inlet screen to a FSR outlet

Figure 2. Single stage inlet security screen to a reservoir overflow culvert

Figure 3. Fish screen on a reservoir overflow spillway inlet

Figure 4. Mammal screen on inlet channel to an amenity lake

Screens normally consist of vertical or inclined metal bars and may consist of a single or multiple stages.

The screen bars form a permanent blockage to flow resulting in increased headlosses and a ‘backwater’ effect. In *extremis* they can reduce the pass-forward flow.

Regardless of its purpose, a screen within a reservoir or water carrying conduit will collect debris. Therefore even if its primary purpose is not to trap debris, the impacts of debris causing temporary blockage to the screen must be considered in design. This involves a three-stage process:

(1) Estimate the amount of blockage (either directly as a proportion of screen area, or by first estimating debris load and converting this to an equivalent blocked area),

(2) Estimate the new upstream water level for the required flows, either by:

   a. assuming the blockage is impermeable and acts as a temporary weir (generally most representative for inlet screens), or

   b. assuming the blockage is permeable and the remaining clear area of the screen acts as an orifice (generally most suitable for outlet screens).

(3) If the temporary blockage is substantial (i.e. where the remaining clear area of the screen is less than the opening area of the downstream structure), a check should also be made for potential choking of the flow.

Information on the hydraulic analysis of blockage is available in Benn *et.*(2019), Pavlov (2022) and ICOLD Bulletin 176 (2021). The latter two documents cover the mechanisms of blockage of spillways by large floating debris while the former is more focussed on culvert screens. The available methods to predict potential debris volume fall into one of three categories:

i. Empirical methods based on existing data on flood debris transport;

ii. Evaluation of the upstream catchment for debris sources; and

iii. Evaluation of debris transport in past flood events.

All provide estimates with considerable amounts of uncertainty.

While there has been international research on debris generation and how it accumulates on screens at reservoirs (e.g. USBR, 2016), there has been limited UK research on this topic. The international research focusses mainly on large woody debris under conditions that are likely to apply only to reservoirs with the largest catchments in the UK.

UK-specific research concerns culvert screens in rivers rather than in overflows and conduits at reservoirs. Debris loads are typically dominated by smaller floating debris such as leaves and twigs (typically 60% - 90% of total load by volume). While small debris load generally increases with flow and wind speed, it has a weak correlation with flood return period. A more significant factor is the period of time that has elapsed since the last elevated water levels in the reservoir or contributing watercourses. This suggests that it is the slow accumulation of debris on channel and reservoir margins which is more important than flood magnitude in determining debris sources.

In the context of screens on reservoir outlets and smaller overflows such as seen on flood storage and amenity reservoirs, operational experience tells us that it only requires small debris such as leaves, twigs and litter to result in a temporarily blockage. This is especially so where the clear bar spacing is less than 260mm. Wallerstein and Arthur (2012) showed that a reduction in clear bar spacing from 260 mm to 160mm on a culvert inlet screen led to a threefold increase in trapped debris – nearly all being small floating debris.

All screens require a monitoring and maintenance regime. Without one, their effectiveness, especially for preventing significant blockage, is significantly reduced. However, even the best surveillance and maintenance regimes will not be able to keep a screen completely free of accumulated debris, especially during a flood event.

DEVIL'S BRIDGE POND, SHEFFIELD

This is a privately-owned impounding reservoir constructed in 2010 on Blackburn Brook upstream of the Thorncliffe Business Park in Chapeltown, Sheffield (Figure 5).

It consists of a 7m high, 112m long earth embankment dam along the reservoir's southern edge. Its purpose is to store water during high flow events and hence to provide flood protection to a business park located immediately downstream. It was designed to provide a 150-year return period standard of protection.

Managing Risks for Dams and Reservoirs

The reservoir has a storage capacity of 32,600m^3 when full to the point of spilling (which is a level of 100.6m AOD). Twin Type 1098C Hydro-Brake™ vortex flow control units located in a reinforced concrete chamber (Figure 6) are designed to limit the 'pass forward' flow out of the reservoir via two low-level 1.5m diameter concrete pipe outlets to 5.7m^3/s when the reservoir is full to the point of spilling. The flow of 5.7m^3/s is estimated to be the maximum capacity of the Newton Bank Road culvert which carries the Blackburn Brook beneath the Business Park downstream of the reservoir.

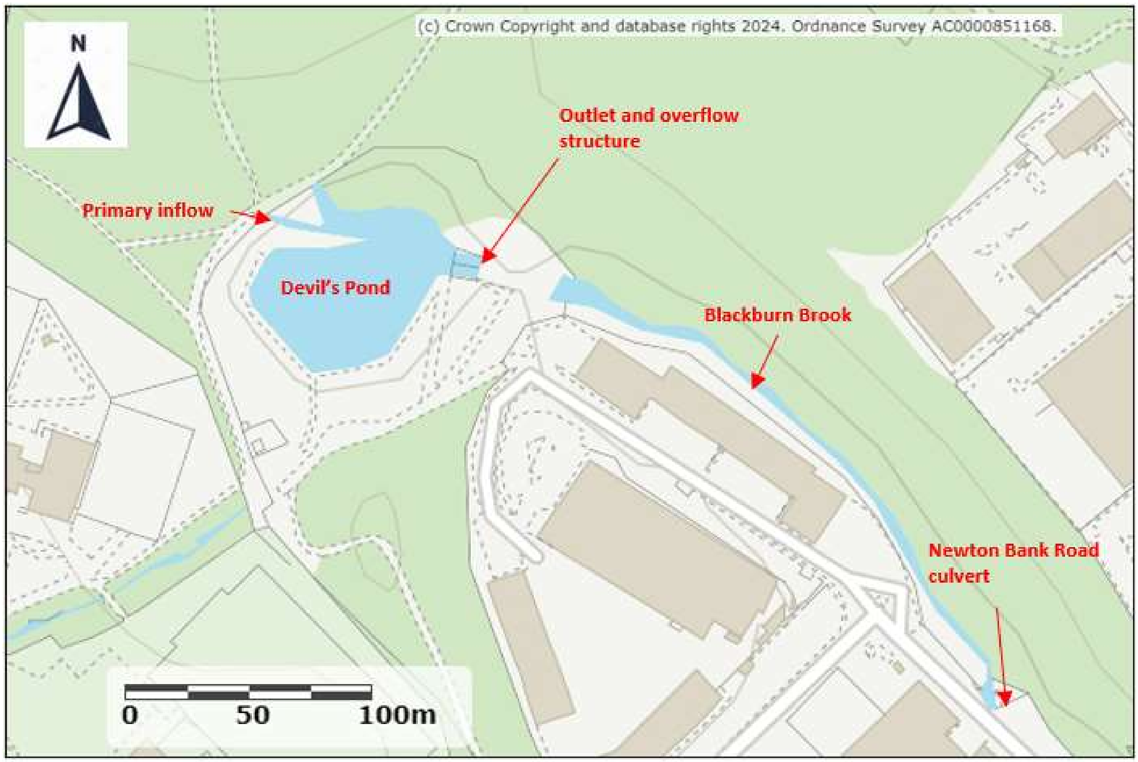

Figure 5. Devil's Bridge Pond

Twin 3.2m high, 200mm clear spacing vertical bar debris screens cover the entry to the twin Hydro-brakes™ (Figure 7). The net effective area (i.e. the area between the bars below the overflow level) is 15 times larger than the inlet area of the Hydro-brakes™. The control structure is also designed to allow easy removal of large debris by means of a concrete invert slab and access ramp.

The original design risk assessment concluded that the hazard to people presented by the twin pipe culverts was low and that it could be mitigated by the security fence around the reservoir perimeter and the site security measures which included CCTV. However a post-construction site-wide public safety audit recommended the addition of 200mm x 180mm 'mesh' security screens to the outlet which were subsequently installed against the Supervising Engineer's advice (Figure 8).

Figure 6. Devil's Bridge Pond – aerial view of the Hydro-brake™ chamber and dam. Note the access ramp leading down to the screen.

Figure 7. Devil's Bridge Pond – debris screens on inlet of the Hydro-brake™ chamber. Note the concrete invert to aid cleaning of the screen.

Figure 8. Devil's Bridge Pond – outlet culverts with security screens open

Figure 9. Devil's Bridge Pond – outlet culverts with security screens closed

Following heavy rain during the 7th and 8th November 2019 the reservoir filled for the first time and the overflow spillway started to operate, discharging the excess flow that could not be passed through the Hydro-Brakes™ and the low-level outlet. The outlet security screens were closed during the event, and because of the high water levels in Blackburn Brook they could not be safely opened (Figure 9).

Following the event the screens, culverts and overflow spillway were inspected. While the inlet debris screens had minimal debris accumulation, the outlet security screens had accumulated small debris (mainly grass) representing a 25% loss of area between the screen bars.

Using the observed rainfall and a hydrological model, the peak inflow to Devil's Bridge Pond was estimated to be 3.5 m^3/s. This had an estimated return period of between 5 and 30 years. This was substantially less than the design 150-year inflow of around 11 m^3/s for the reservoir to fill completely. The modelling shows that if the Hydro-Brakes™ had operated in accordance with the manufacturers rating curve the reservoir level should have peaked at 98.77 mAOD – well below spill level of 100.6 mAOD.

Further analysis using hydraulic modelling showed that the tailwater effect from the partially blocked outlet screens had prevented the Hydro-brakes™ priming fully and they therefore acted as simple fixed orifices. This reduced the average pass forward flow through them by 62% (Figure 10).

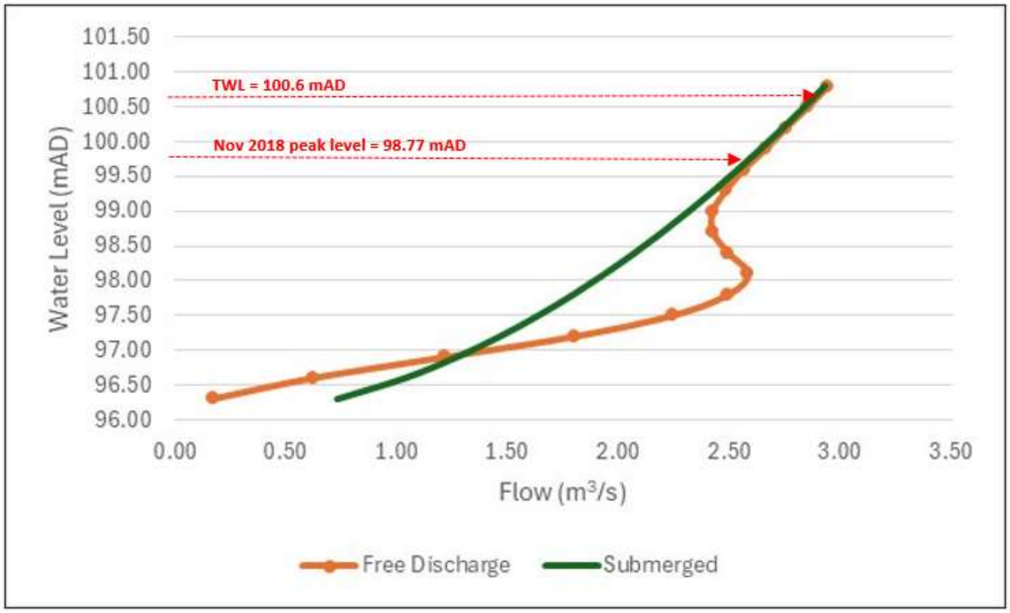

Figure 10. Comparison of Devil's Bridge Pond Hydro-brake™ rating curves under free and submerged outlet conditions (for a single 1098C Hydro-brake™ unit)

Had the design flood event occurred in October 2019 with the security screen closed, the reservoir would not have provided the standard of protection expected, with potentially severe consequences both for public safety and property damage as the flow capacity of the Newton Bank Road Culvert would have been exceeded and the business park would have flooded. The presence of the security screens, even though they are normally open, presents a hazard to the safe operation of the reservoir. This hazard must be balanced against the consequences of unauthorised access to the outlet culverts.

Since the November 2019 event, the outlet screens are now normally left open (or they are opened as soon as the reservoir starts to fill). A further high flow event in October 2023 - which is estimated to have been greater than that seen in 2019 (4.0 m^3/s) - saw the reservoir fill and reach a level of 99.9 mAOD. The reservoir did not spill. The outlet screens were open before and during the event.

POCFAS RESERVOIR, YORKSHIRE

The Pocklington Flood Alleviation Scheme (POCFAS) protects the town of Pocklington in the East Riding of Yorkshire. The main component of the scheme is an 87,000m^3 capacity on-line flood storage reservoir. The reservoir outlet consists of a 1800mm x 1800mm box culvert with an upstream inlet comprising a 1000mm diameter orifice controlled by a self-activating 'Hydroslide®' scissor gate (Figure 11). There is a debris screen just upstream of the scissor gate with 140 mm spacing between the bars. The net effective area of the debris screen is 14 times the orifice area. A security screen was installed at the downstream end of the culvert to prevent unauthorised access. The security screen has a spacing of 100 mm between the bars (Figure 12). The net effective area of the security screen is 0.9 times the culvert area.

The upstream debris screen is designed to be easily cleared using rakes from the access steps and platforms provided. In contrast, the security screen includes no provision for cleaning, although in an emergency the whole gate can be dropped to the horizontal using an emergency release. Once the emergency release is operated there is no easy way to lift the screen back into position.

Figure 11. POCFAS – scissor gate at the 1000mm diameter inlet to the FSR control structure (photo taken from inside the debris screen)

Figure 12. POCFAS – outlet security screen in November 2023

The FSR was completed in 2019 and had its first substantial filling in November 2023. During this event it was noted there was temporary blockage of the inlet and outlet screens from small floating debris of 20% and 40% of the effective area respectively. The resulting headlosses resulted in the impounded water level being 500mm higher than would have occurred if the screens had been completely clear of debris. This represents approximately 12% of the live storage volume. Following the 2023 event the removal of the outlet security screen is being considered based on a risk assessment using the CIRIA C786 manual.

PRIVATE RESERVOIR, NORTHUMBERLAND

This fishing lake is in Northumberland. It is impounded by a 3.5m high homogenous embankment dam. Its catchment area is approximately 6.8km^2 mainly comprising woodland and open moor of moderate gradient. It lies immediately downstream of a much larger reservoir.

Its overflow consists of a 5m wide inlet weir leading into a stepped masonry channel running down the right hand mitre.

Due to concern about the loss of fish and ducklings from the lake, an inclined 15mm clear spacing bar screen was placed by the owner on the inlet extending to the full height of the overflow wing walls (Figure 13).

Figure 13. Fishing Lake in Northumberland - Fish screen on the inlet to the spillway

Figure 14. Fishing Lake in Northumberland – flows during Storm Babet, October 2023

Visual monitoring of the screen over a period of four years has shown that the fish screen is easily blocked with small floating debris and requires regular cleaning with a rake. While raking is possible when the lake is not spilling it becomes more problematic when water levels are higher due to the lack of a safe access platform. It was concluded for flood study purposes that the effective starting reservoir water level should be the top of the fish screen. This showed there was minimal freeboard and even a modest flood rise would result in flow over the dam crest. In October 2023, following heavy rain, the screen did indeed block and acted as a weir. The dam crest was overtopped for most of its length (Figure 14).

Following consultation with a fish and bird ecologist and the Supervising Engineer, a replacement screen has been designed which is half the height of the current one and with a wider 25mm spacing between the bars. This is a compromise between fish and duckling protection and flood risk. To discourage fish movement towards the screen an apron of gravel has been placed in front of the screen.

MAMMAL PASSAGE

In some situations it is important to allow for mammal passage through a screen. Benn *et. al.* (2019) suggests that a 150 mm high gap is left at the base of screens for fish and eel passage (Figure 17).

For aquatic birds, eels and mammals, consideration should be given to the provision of ramps to allow weirs and steep drops to be negotiated (see examples in Figures 15 and 16). They can also replace the need for a 'fish' screen in some circumstances. These are best as simple wooden 'plank' structures and do not need to be able to withstand floods.

More recent work by the Environment Agency in England (Environment Agency, 2024) has looked at providing for beaver and other mammal passage through screens. The advice from this research is to:

i. Provide a rectangular (letterbox-shaped) opening of any orientation to allow the beaver to flatten out sideways and squeeze through.

ii. Provide an opening size of 200mm by at least 250mm for the comfortable passage of adult beavers.

iii. Avoid square openings unless oversized.

iv. Avoid sharp edges that could cause injury. Exposed bar ends or edges should be rounded.

v. For a screen with 150mm centre-to-centre bar spacing, opening size should be 140mm by at least 300mm - this is the minimum requirement. The screen must have a gap between the toe of the bars and the stream bed, which should be at least 150mm high, larger if this can be achieved without compromising the security function of the screen (e.g. part of the opening is permanently below water).

vi. Any horizontal bar at the toe of the screen must have one or more breaks in (see Figure 18).

Figure 15. 'Duck' ramp installed on an overflow weir as an alternative to a screen

Figure 16. Eel pass installed on a reservoir spillway

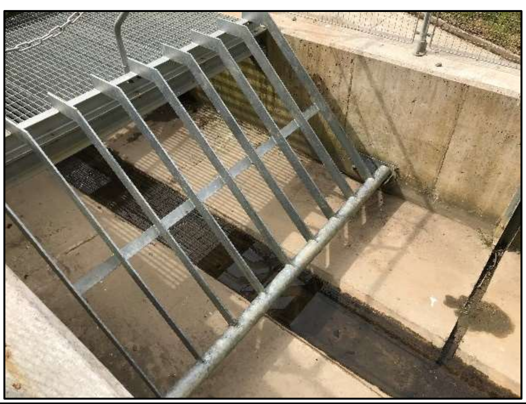

Figure 17. Example detail to allow for mammal, fish and eel passage through a screen

Figure 18. Example detail to allow for beaver through a screen

Beavers of course can cause serious damage to dams and also block overflows through their activities (Brown, 2012).

CONCLUSIONS

Screens have a role to play in reservoir safety, environmental management, and public safety. Drawing on the case studies above and the authors' experience, the following screen 'rules' for reservoirs are suggested to complement the guidance given in CIRIA C786 (Benn *et.al*., 2019), Pavlov (2022) and ICOLD Bulletin 176 (2021):

1. Use of security screens to prevent human entry can lead to enhanced risk of blockage from debris. Other management measures should be used wherever possible.
2. Inlet debris screens should have clear spacing between the bars appropriate to the debris size that could lead to a significant blockage.
3. For any overflow inlet screen with bars of less than 260mm clear spacing an allowance should be made to the design top water level to allow for temporary blockage of the screen. This should typically extend to the equivalent of two-thirds the height of the screen but for screens of less than 0.5 m height this should be the full height of the screen.
4. Screens with clear bar spacing of less than 260mm are very prone to temporary blockage even from small debris such as leaves. If the screen is on the inlet to an overflow, then explicit consideration should be made for this temporary blockage on the stage-discharge relationship.
5. The net effective screen area of an inlet screen on an overflow should be at least seven times the design flood flow area. The screen opening area required will ultimately depend on the consequences of screen blockage and could be higher.
6. Consideration should be given to 'tree pole' primary screens upstream of inlet screens to trap larger floating debris.
7. Inlet screens on overflow / outlet culverts should have a by-pass.
8. Outlet screens are not advised on outlets with vortex control devices. If a screen is used, a check should be made on the impact on hydraulic performance from temporary screen blockage. Any outlet screen should have a means of opening it in anticipation of high flows or if it starts to block with debris.
9. If a security outlet screen is required to an overflow or outlet works, then an inlet screen should also be provided.
10. The clear gap spacing between bars on outlet screens should be no smaller than the bar spacing on inlet screens on the same structure.
11. Design of security screens should consider how the screens will be cleared and maintained under both normal and flood conditions.
12. Fish screens designed to prevent fish/animal 'wash-out' from a reservoir should be no higher than 200mm above normal water level and should be at least 600 mm lower than overflow side walls.
13. Consider 'duck ramps' as an alternative to a screen to prevent bird/mammal injury.

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The British Dam Society

River Roding Flood Storage Reservoir – CFD modelling and optimisation of a double baffle outlet to manage risk of tailwater

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The River Roding flood storage reservoir design has recently been completed and construction commenced in Spring 2024. The new 1.4Mm^3 flood detention reservoir will be retained by a 7m high and 570m long homogeneous earth embankment with a passive, 'double baffle' flow control structure. This structure will comprise six reinforced concrete bays each with a crump weir and double baffle arrangement. This will be the third double baffle flow control structure to be constructed in the UK, following Banbury and Chapelton reservoirs.

The double baffle structure is an alternative to more conventional vortex flow control devices, all of which are sensitive to downstream tailwater conditions; in this case due to a downstream road embankment. Double baffle structures are better suited for higher pass-forward flows and are less vulnerable to debris blockages than vortex devices.

Computational Fluid Dynamics (CFD) was first used to validate the modelling approach, based on the Banbury physical scale modelling to reduce the risk of the control structure not performing as designed. Following this an iterative approach between CFD and fluvial analysis was used to scale the design to achieve the optimum depth discharge characteristics. Due to the importance of tailwater depth, CFD simulations were run comparing how various upstream and downstream water depths affected the performance of the control structure.

A novel aspect is provision of a low flow bay with incorporation of a fish pass. Future adaptation has been incorporated into the design though the use of various bay widths and incorporating an additional spare bay.

INTRODUCTION

The River Roding in Essex has a long history of flooding. The river responds rapidly to rainfall events, particularly in the middle and lower reaches where there is less floodplain available for storage and a greater number of properties at risk of flooding. This was demonstrated in 2000, when fluvial and pluvial flooding caused damage to over 400 properties in Woodford, northeast London. Some protection from regular flooding is afforded by a manmade network of flood defences. However, once climate change is taken into account, a significant proportion of these areas remain within the Environment Agency (EA) designated Flood Zones 2 and 3. Therefore, the EA has deemed it necessary to carry out works to alleviate future flood impacts by creating a new Flood Storage Reservoir (FSR) to provide protection into the future.

Detailed design of the River Roding flood storage reservoir has recently been completed and construction of the scheme commenced in Spring 2024 with BAM as the Principal Contractor. The reservoir will have a storage volume of 1.4Mm^3 retained by a 7m high, 570m long homogeneous earth embankment. Flows are managed by means of a passive, reinforced concrete flow control structure in the form of a crump weir and double baffle arrangement. The structure is divided into six bays. This will be the third double baffle outlet to be utilised in the UK, following Banbury (Akers et al, 2004 & 2012) and Chapelton dams (Gowens etc al, 2010).

OBJECTIVES AND OPTIONS FOR FLOW CONTROL

As with any flood storage reservoir, a control structure is required to control flows through the dam. To optimise storage capacity, the ideal flow control structure would allow all flow to pass downstream until the capacity of the downstream channel (the 'pass-forward flow') is reached and would then discharge exactly the pass-forward flow for all stages above this. In reality, it is difficult to achieve such accurate control even with a fully automated gated system, but there are various forms of flow control which accomplish these objectives to a greater or lesser extent. Active flow controls systems, using moving gates, generally provide the most efficient form of control but were discounted at the options appraisal stage due to the client's preference for a passive system. This preference is due to the increased operational and maintenance requirements associated with moving gates and because with any form of active flow control they may be blamed (rightly or wrongly) for any residual flooding due to (perceived) maloperation. Other advantages and disadvantages of active versus passive flow control are discussed in Brown et al (2022).

Alternative passive options that were considered include vortex devices (i.e. Hydrobrake™) and gates controlled with a float mechanism (e.g. Hydro-Slide™) but due to the high pass-forward flow required (62m^3/s) these were not practical at this site.

The concept of a double baffle structure is explained in Akers (2004) along with references relating to the hydraulic design. Essentially the structure includes a crump weir with two concrete baffle walls downstream. As the reservoir fills and water levels rise, the hydraulic control switches from weir flow to orifice flow beneath the upstream baffle, and then to weir and orifice flow controlled by both the upstream and downstream baffles in turn (Figure 1). By optimising the geometry of the structure, the characteristics of the ideal rating curve can be achieved.

The double baffle control structure at Banbury has been in operation since 2012 and has performed well, although at 38m^3/s the pass forward flow is significantly less than that required for the River Roding scheme, and the tailwater conditions are different. There was a risk that these factors may severely affect the hydraulic performance of a similar structure on the River Roding and this risk needed to be assessed and managed.

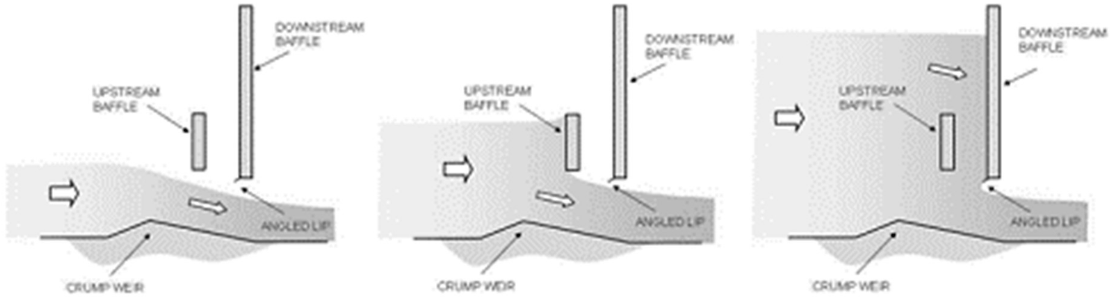

Figure 1. Three flow modes for double baffle flow control structure (Ackers et al, 2004)

DESIGN OF FLOW CONTROL STRUCTURE

Justification of options of physical model vs CFD

At the commencement of the detailed design process there were significant uncertainties regarding how the double baffle design would function when impacted by the elevated downstream water levels resulting from the nearby road bridge. Additionally, development of the detailing was envisaged to achieve acceptable outcomes in terms of low flow performance. To overcome these risks a high degree of flexibility and longevity (compared to physical models) was required from 3D modelling in order to support the overall design effort. This resulted in the selection of CFD modelling as the most appropriate and efficient tool to support the process, as it could be validated against the earlier designs of similar schemes and then optimised as needed, in parallel to design development and other modelling, including the key fluvial and hydrological inputs.

Previous data from Banbury: "theory from previous studies"

Earlier physical modelling data from the Banbury Flood Alleviation Storage scheme was utilised as the basis for modelling of the new structure. This provided geometry and performance data from physical modelling of a comparable scheme (albeit with different flow control characteristics). This enabled development for Roding to achieve the target pass-forward flow and depth-discharge performance.

CFD process

A multistage CFD modelling was employed to firstly validate the CFD approach against the Banbury physical model data (Figure 2), and then to utilise the validated modelling approach in conjunction with a 3D representation of the river and bridge to investigate and develop the performance of the larger Roding control structure.

The general CFD modelling processes were as illustrated in Figure 3. By utilising a digital model for the testing, significant changes to the geometry and scenarios were readily achievable throughout the process, providing numerous benefits to the design process including rapid integration with fluvial modelling and flexibility to trial different aspects without time consuming and costly modifications to a physical model.

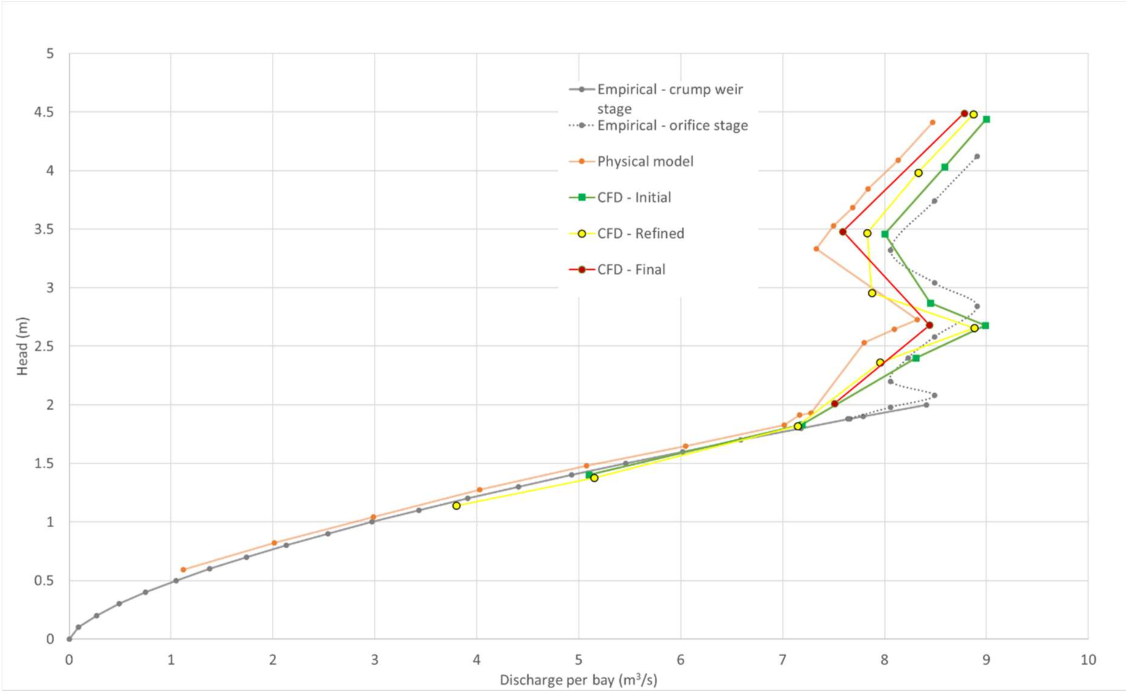

Figure 2. CFD depth-discharge validation against Banbury design data

Figure 3. Typical CFD modelling workflow

The flexibility of the modelling approach provided opportunities to implement the predicted depth-discharge performance from the CFD in the fluvial model (Flood Modeller), test performance and then evaluate performance of alternatives before undertaking further development without the need for retaining a large physical model within a laboratory during periods of 3D model downtime.

The phases of work were grouped as follows:

- Validation study – matching the geometry and flows from Banbury
- Initial design testing – analysis of a wider design with similar longitudinal section and a low flow channel to target suitable performance for the Roding scheme
- Development testing – modelling of alternative design concepts to obtain the required depth-discharge performance, mitigate backwater influence and to achieve acceptable low flow channel characteristics (see Figure 4 for an example output from the development tests)
- Additional review – review of modelling outputs to inform geomorphological and fish passage performance
- Final analysis – additional testing of refinement to the low flow channel geometry to promote fish passage under low flow conditions

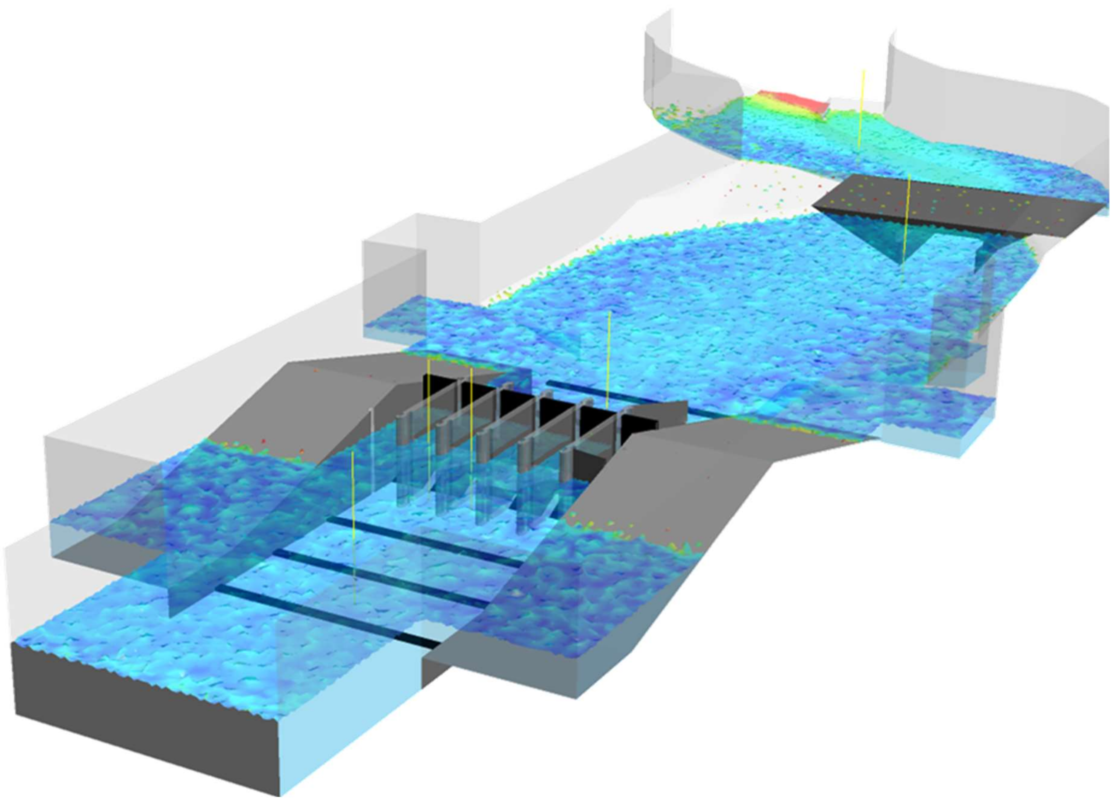

Figure 4. 3D Render of design development CFD model

Following the 3D modelling activities, a robust design was defined with site specific adaptations (raising of the crump weirs and baffles in the high flow bays, and lowering baffles and adding a short, notched crump weir in the low flow bay) to balance the opposing objectives of effective flow control for high flows and low flow performance. Modifications to the Banbury double baffle arrangement, aside from scaling the width of the structure, were found to be essential due to the significant backwater influence at the structure location – without these the structure was shown to produce an unsatisfactory depth-discharge relationship and would not have achieved the key objectives of the scheme (under baseline testing the hydraulic performance was found to be too linear, without the necessary inflections in the depth-discharge curve – see comparison in Figure 5). Through design amendments the primary risks of utilising a passive control structure, such as potential for blockage, were managed without compromise to provide effective impoundment performance for storage under high flows.

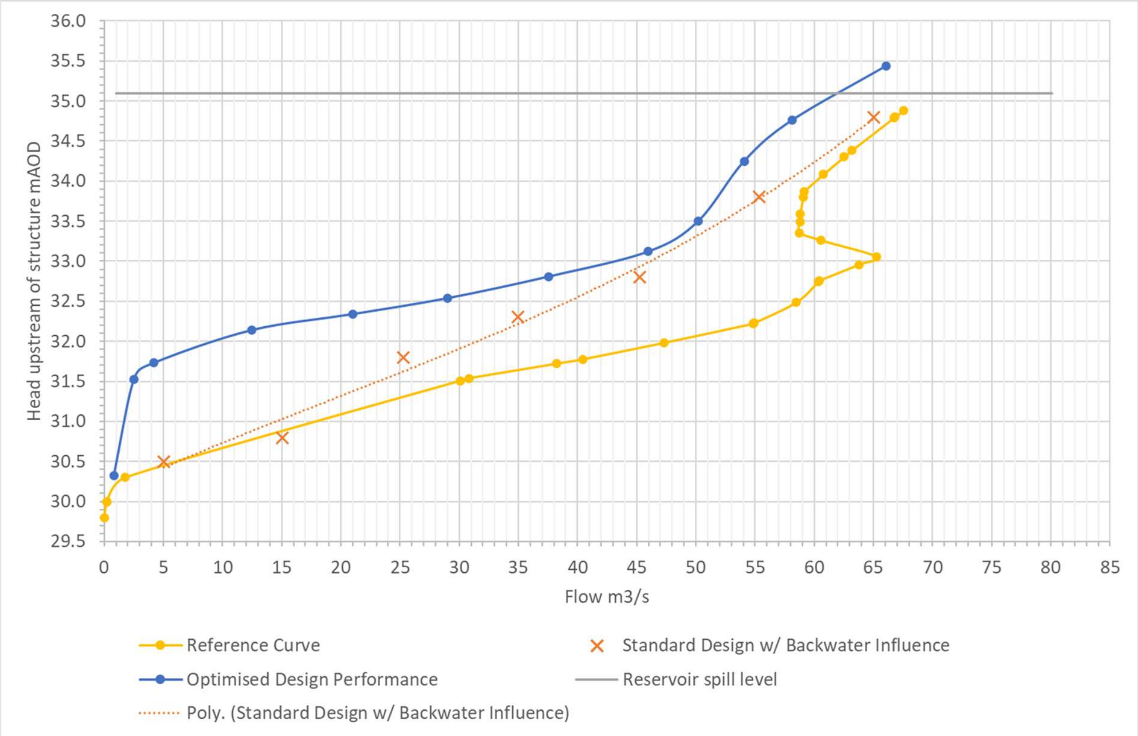

Figure 5. Comparison of depth-discharge performance predictions

Following design development using CFD, the specific depth-discharge curve was defined and then fed back to the fluvial model for retesting, thereby enabling confirmation of suitability and addressing the risks of the scheme not delivering the required flood risk management performance.

Future adaption and climate change

Future changes in climate and development introduce potential risks with utilising a passive flow control structure. However, enhanced operational flexibility was provided, as illustrated in Figure 5, by having two different size bay widths; the inclusion of an additional spare bay; and provision to close off any bay or combination of bays with stop logs. Through selecting which bays are isolated the pass forward flow and utilisation of storage can be managed during operation, providing a high level of flexibility despite the passive nature of the control structure.

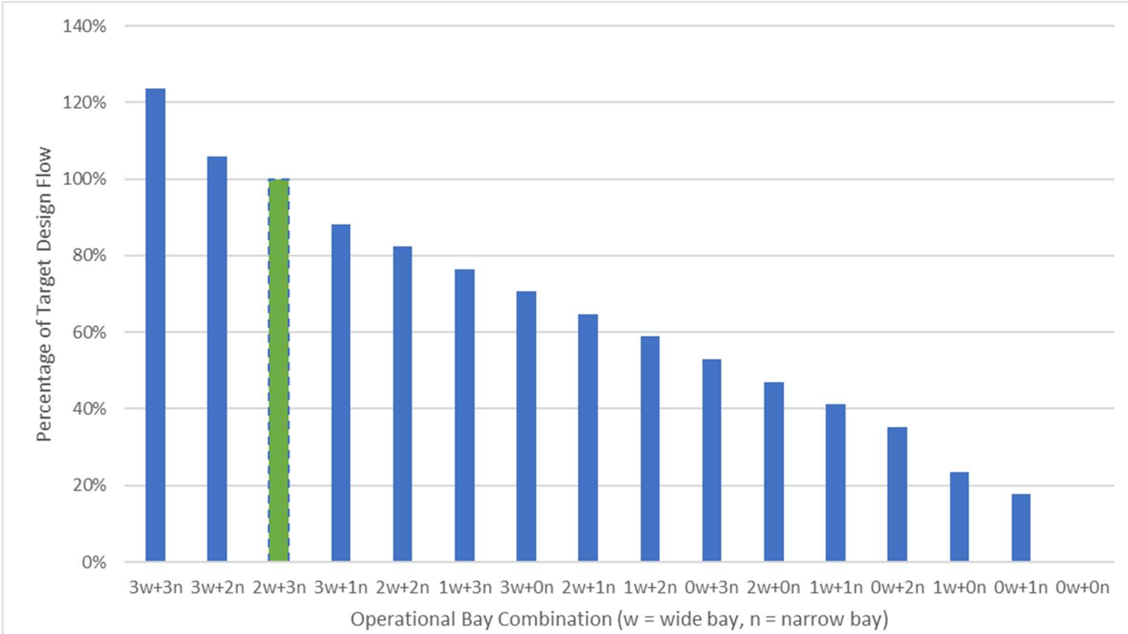

Figure 6. Illustration of operational flexibility through isolation of wide and narrow Bays

Managing floods during construction

The control structure will be completed prior to the embankment and spillway and it is therefore important that the control structure is able to pass flood flows during the construction period. This has been achieved by having two stages of construction. The majority of the structure will be built in the initial stage, including the whole of the low flow bay but excluding the weirs and baffles within the other five bays. The remaining weirs and baffles will only be built once the dam and spillway is safe to impound and the Preliminary Certificate has been issued under the Reservoirs Act.

This approach minimises the risk of flood damage during construction and avoids the need for working in water during the second stage of construction. The two-stage construction is facilitated using reinforcement couplers.

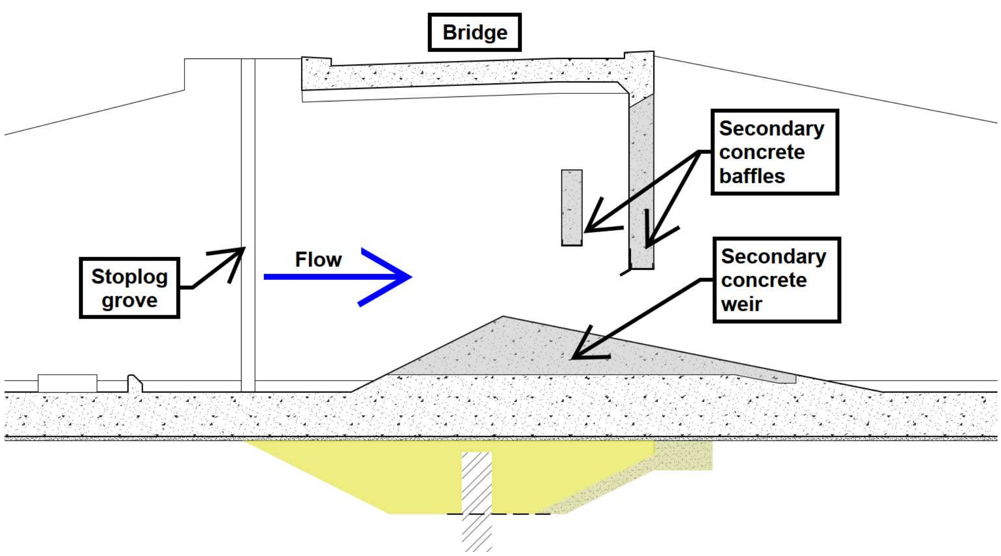

Figure 7. Staged construction of bays

Reducing environmental risk

Following development of the hydraulic design for passive flow control through the Roding double baffle structure, the CFD model was developed and tested to help inform environmental risks associated with constructing a control structure on the watercourse. During detailed design features were added to improve fish passage characteristics under low flow conditions. The key design amendments considered were to add notches in the cross walls and the crump weir of the low flow bay, and to incorporate stones cast into the base slab to promote near-bed low velocity regions. These features and an example of the corresponding model results are shown in Figure 8.

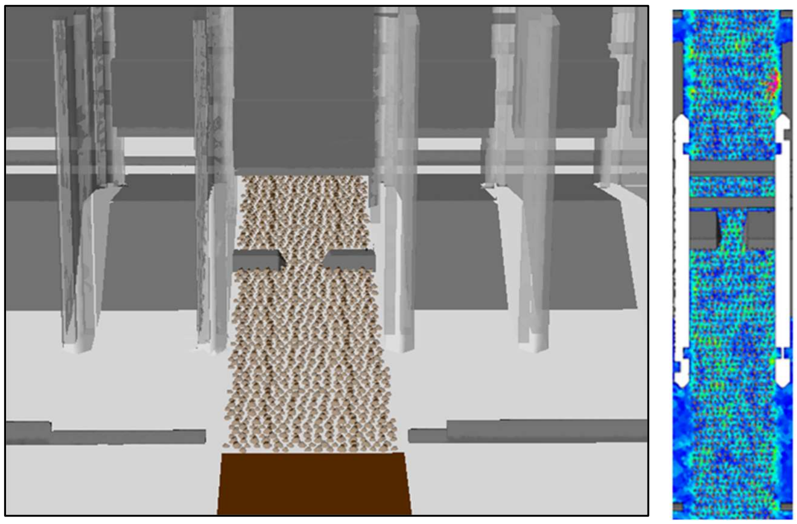

Figure 8. Model representation of low flow channel with cast-in stones (left)
CFD prediction of near-bed velocity under low flow (right)

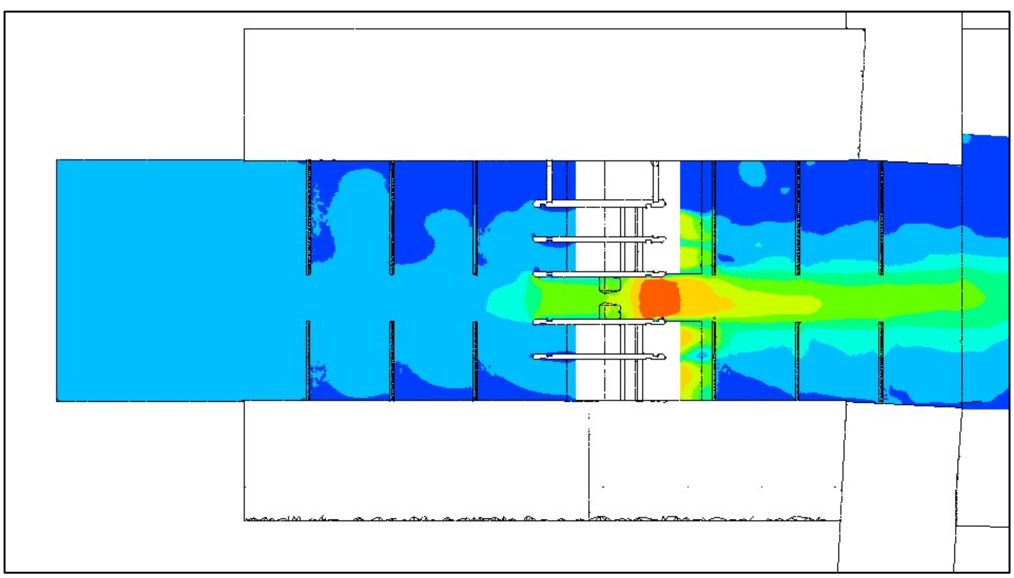

Figure 9. CFD predictions of near bed velocity

Extended analysis was also undertaken using the CFD model to investigate flow conditions over an expanded number of low to medium flows to inform sediment transport and geomorphological effects. This included assessment of near bed velocity (as illustrated in

Figure 9) and quantification of the shear stresses across the base slab. The CFD model results were also utilised to investigate the conditions beyond the new engineered structure to inform design of the transitions as the flow returns to the natural watercourse downstream.

The outputs from these supplementary analyses have been utilised through the detailed design process to provide detail on the impacts of the design refinements and reduce risk to ecology, natural river processes, and through ensuring that the fundamental flow control functionality is not affected.

SUMMARY OF KEY DIMENSIONS AND FEATURES OF CONTROL STRUCTURE

The key features and dimension of the control structure are summarised in Table 1.

Table 1. Summary of key features and dimensions

Feature	Units	Value / Description
Pass-forward flow	m^3/s	62
No. bays		Three @ 2.77m width; three @ 1.94m width (including one spare bay closed off)
Top water level	m AOD	35.1
Base slab level	m AOD	29.6
Low flow weir level	m AOD	29.8 (low flow notch 29.6)
Standard weir level	m AOD	30.9
Bed	-	Roughened concrete with embedded stones in low flow bay
Trash management	-	None. Any debris too large to pass through the flow control structure is likely to become lodged against the upstream piers where it can be later be removed when conditions allow by a Hiab or crane from the bridge deck or by accessing the upstream apron with suitable plant via the ramp on the west bank of the river.
Instrumentation		Water level sensors and CCTV

CONCLUSIONS

Flows through the River Roding flood storage reservoir will be controlled by a passive 'double baffle' flow control structure which is designed to optimise the reservoir operation by minimising premature impounding and capping peak flows more efficiently compared to a simple flume or orifice. Although the design concept has been proven at two similar structures in the UK there was a risk that high tailwater at this site and the need for a significantly higher pass-forward flow could prevent a double baffle structure from working effectively at this site.

Computational Fluid Dynamics (CFD) was used to assess this risk and optimise the hydraulic design of the structure. The CFD model was initially calibrated using the results of a physical hydraulic model which had previously been tested for the Banbury scheme. CFD allowed various design iterations to be tested and later enabled the design of measures to improve fish passage during low flows.

Operational flexibility, including climate change, was provided by having two different size bay widths, the inclusion of an additional spare bay, and provision to close off any bay or combination of bays with stop logs.

Managing Risks for Dams and Reservoirs

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Risk assessment of existing flood detention (storage) reservoirs

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SYNOPSIS The Environment Agency is carrying out a portfolio risk assessment of their portfolio of around 200 large flood detention reservoirs (FDRs), to inform their reservoir safety management and operation.

The 'Guide to risk assessment for reservoir safety management' (RARS) was published in 2013 and provides a methodology for risk assessing existing reservoirs in the United Kingdom. It was intended primarily for reservoirs which are normally full, where indicators of poor condition can be observed. It was therefore necessary to extend RARS to cover FDRs, and this paper describes the key elements of the extension to the RARS Tier 1 methodology. It is anticipated similar extensions could be applicable to FDRs owned and operated by other agencies. The next step is to extend RARS for Tier 2 and 3.

Washland flood detention reservoirs are in effect partially bunded reservoirs, so some aspects of the approaches adopted here will also be applicable to non-impounding reservoirs.

INTRODUCTION

Modern management of reservoir safety is moving towards a risk-based approach, recognising that risk can never be zero (unless the asset is removed), but that risk can be reduced to as low as reasonably practicable, where the benefits of the asset in reducing damage from operational floods outweigh the consequences and risks of the dam failure and release of the reservoir.

In the UK, the first guide to provide a means of quantifying the risks to the public from reservoir failure was published in 2014 – the Interim Guide to Quantitative Risk Assessment for UK reservoirs (Brown and Gosden, 2004), with this being updated and extended in the Guide to Risk Assessment for Reservoir Safety Management (RARS) in 2013 (EA, 2013).

RARS is intended primarily for reservoirs which are normally full, where indicators of poor condition can be observed. However, flood detention reservoirs (FDRs) are normally empty and only fill during floods to reduce the effects of flooding downstream, hence it was necessary to extend RARS to cover FDRs.

Managing Risks for Dams and Reservoirs

The term “flood detention reservoir” (FDR) is used in this paper, rather than flood storage reservoirs (FSRs), as this more closely follows international practice and emphasises that the purpose is to attenuate, rather than store floods.

This paper describes the key:

a) challenges in applying RARS to FDRs

b) elements of the extension to the RARS Tier 1 methodology to accommodate FDRs

c) comments on the likely extension needed for Tier 2

It is anticipated similar extensions could be applicable to FDRs owned and operated by other agencies.

PROJECT OBJECTIVES

In 2013, the Environment Agency commissioned HR Wallingford to convene a consortium of experts to produce RARS. The aim of the guide was to provide a tool for reservoir safety management and, although not a statutory requirement, it is viewed as best practice for reservoir owners/operators. It takes a three-tiered approach to assessing risk moving from qualitative (Tier 1) to quantitative (Tier 2 & 3).

The overall objective of this project is to provide an improved baseline understanding of risk associated with all the Environment Agency FDRs. Specifically, this involved the completion of Tier 1 assessments for over 200 reservoirs, following RARS guidance and best practice to demonstrate a pro-active and exemplary approach to reservoir safety management.

To help manage and deliver this process, the Environment Agency commissioned HR Wallingford to develop a web application that provides a digital version of the Tier 1 assessment process as documented within the published guidance. The RARS Tier 1 App was designed to allow multiple users at different organisations to undertake the risk assessment work in a structured and auditable manner.

The assessments were carried by teams of experienced reservoir engineers at Jacobs and Binnies, who assessed the reservoirs on the east and west sides of England respectively. The project team included staff who had written the original RARS guide, which facilitated the reviewing and refining the risk assessment process.

RARS METHODOLOGY FOR RISK ASSESSMENT

This was written in 2013, building on the Interim Guide (Brown and Gosden, 2004), with various erratum (Wallis and Brown, 2014 and 2017) incorporated in the 2017 edition, which is on the website at https://www.gov.uk/flood-and-coastal-erosion-risk-management-research-reports/risk-assessment-for-reservoirs. In addition, there were some extensions published in Peters et al (2016), developing key themes.

The process and key stages within the risk assessment methodology follow the process as shown in Figure 1.2 of the RARS guide.

ENVIRONMENT AGENCY PORTFOLIO OF FLOOD DETENTION RESERVOIRS (FDRS)

The Environment Agency is the undertaker for 217 FDRs, of which around 79 are washland reservoirs, concentrated in the low-lying regions of Yorkshire, East Anglia and the Somerset Levels as shown on Figure 1 of Courtnadge and Brown (2022). Washlands are similar to bunded and non-impounding reservoirs in that there are a number of perimeter banks, and the likely consequences of failure are likely to vary with position around the perimeter.

This project was applicable to both impounding flood detention reservoirs constructed across valleys to impound floodwater, and to washland reservoirs. Courtnadge and Brown (2022) describes some of the key challenges in assessment the safety of washland reservoirs and describes the approach that has been adopted in this project.

EXTENSION OF RARS TO FDRS

General

In practice this occurred in several stages as queries arose both in developing/testing the App and later when applying the App where the standard RARS methodology shown in the software was not always intuitive or directly applicable in certain circumstances to FDRs. There were also refinements after practitioner feedback from the two consultants upon completion of a pilot of ten initial assessments each. It was therefore necessary to both update the App, and to produce supplementary guidance to align the large project team, which included:

a) Guidance produced by HR Wallingford for use of the App

b) FAQs (many of which were clarifying standard RARS terminology in the App, for specific use on FDRs)

c) Supplementary in-house prompt list for completing an assessment, produced and used in-house by each of the companies carrying out the risk assessments

The extensions to RARS are summarised in Table 1 and discussed under each step of the risk assessment in the following text.

The RARS App

This is a cloud-based system. Users undertaking a risk assessment are able to log in and select, from a pre-populated list of EA reservoirs, which reservoir they wish to assess. If the selected reservoir has not previously been assessed, an empty Tier 1 Assessment form is opened. This mirrors all the steps that are defined in the RARS Guide for a Tier 1 assessment. Upon completion, the assessment is 'Submitted' whereupon all the data are posted to the secure cloud server.

Once complete, the App allows the assessment for a reservoir to be reviewed at any time. Upon selecting a reservoir, if already assessed, the latest data are retrieved from the database and loaded into the Tier 1 Assessment form where they can be reviewed, updated and resubmitted. This ensures that the App becomes a valuable resource for reviewing and later updating the latest Tier 1 risk assessment for each reservoir in the EA FDR portfolio.

Table 1. Areas where extensions have been added to RARS to accommodate FDR

Step	Aspect	Need for extension	Extension to RARS Tier 1
Prep.	Define critical dam location	Washland reservoirs have multiple perimeter banks; often not self-evident which is highest risk bank	See Courtnadge & Brown (2022). Default was to assess highest consequence (i.e. location assumed for Reservoir Flood Mapping (RFM))
1b	Potential consequences	National Reservoir flood mapping (RFM) now has two scenarios, dry day and incremental wet day.	Add comparison and decision step (use maximum consequence scenario)
2a	Intrinsic condition	Many FDRs lack information on internal zoning	Extend Table 4.17 of RARS
2a	Current condition score	As not normally full, normally no indicators of performance available while retaining water	Extend Table 4.18 of RARS
2b	Spillway chute	Likelihood of failure due to scour of grass reinforced spillway	Not covered by RARS. Method developed
2b	Slope stability	Phreatic surface in most reservoirs governed by steady seepage from full reservoirs, FDRs subject to periodic overflow but otherwise dry	Extend Table 4.6 of RARS

The Tier 1 Assessment form follows the published guidance very closely. The first part is to enter some key properties of the reservoir, for example, type, capacity, dam crest, width and height, upstream and downstream slopes, PMF value, spillway capacity and so on. Next, in Step 1: Risk Identification, the user identifies the credible failure modes and reviews the potential consequences (pre-loaded from assessment of the RFM mapping by EA). In Step 2: Risk Analysis, for each credible failure mode the form allows the likelihood of failure to be assessed using previously entered data wherever possible. Lastly the risk is calculated using the likelihood and consequence matrix from the RARS Guide.

Finally, in Step 3: Risk Evaluation, the reservoir engineers give judgements on options to reduce the risk, their recommendations and other considerations, before uploading the assessment. Throughout the assessment, there are boxes for entry of supporting information and metadata (e.g. free text reference or weblink to data) that might be useful when reviewing the results and moving on to Tier 2 level assessment.

The RARS App has brought several advantages over a more traditional (e.g. spreadsheet based) approach:

- There is consistency across all assessments, including those entered by different engineers and organisations
- The app has enabled the assessments to be undertaken more efficiently
- There is a documented sign-off process and means for storing additional supporting information
- Being an online form, updates to the RARS App are instantaneous across all 217 assessments, with no need to update individual computers or any risk of people having old versions of software

- The results are stored securely in the cloud; many users can input and review data concurrently and it is possible to make global updates to the data such as those described in this paper.
- The data can be updated and resubmitted and an audit trail produced tracking progress of the understanding of the risk at each site over time. It will be possible to determine changes in risk across the portfolio of reservoirs over time

A dashboard viewer has been created to show the headline summaries for all reservoirs with a map, graphs and tables being available to look for trends and outliers

Review and validation of output

The output of each consultant was revised and validated in-house, with further reviews of the completed assessments by HR Wallingford and the EA. The data and principals in Section 15.2 (Basis of a tiered set of tools) of RARS were used in this review.

PREPARATION

As with any risk assessment of an existing reservoir, a key stage is collating the available data needed for the assessments. This had to be provided by the reservoir manager and was similar to the information needed for a periodic inspection under Section 10 of the Reservoirs Act. This was recommendation 4 from the Balmforth report Part A (2020). The first step of the App was to populate key data on physical attributes of the reservoir, and the App was extended in use to provide space to comment on the provenance of the data.

Dam location to be used in risk assessment

A significant challenge for washland reservoirs was identifying which dam section was to be assessed, with options shown in Table 2. At Tier 1 level, it was assumed that if a reservoir is retained by multiple dams then the assessment would be for the highest consequence dam (e.g. for a washland this would normally be the barrier bank, or where no barrier bank the highest part of the riverside bank).

Table 2. Considerations at washland reservoirs to define location of bank subject to risk assessment

Location	Factors which may make highest risk
River bank	Likely to be lower than barrier bank so overflows first. Sometimes varying construction, and some may have originally been transportation embankments e.g. old railways
Transverse banks (across flood plain)	May be housing, or other receptors, present remote from reservoir
Barrier bank	Housing present below crest of barrier bank, which would be inundated if barrier bank failed during a flood

Key dimensions of dam on which risk assessment carried out

Some of the features present at washland reservoirs, and how they were assessed are shown in Table 3.

Table 3. Approach adopted in defining features of dam to be analysed

Aspect	Adopted in the Tier 1 PRA
Number of spillways	Include option for two spillways in the app, so that a check can be made on the spillway and main river bank (in terms of operating as an overflow).
Absence of spillway	Assume river bank acts as a spillway
Catchment area	Direct catchment of the reservoir, which for washlands reservoirs is the reservoir area and any direct catchment on the adjacent valley side, rather than the indirect catchment for the adjacent main river (Courtnadge and Brown, 2022).
Spillway crest length	As FDRs often have earth spillways with no well-defined "weir crest" and depths of overflow are modest and similar to irregularities in crest level, the effective length of the spillway was reduced to provide a more realistic estimate of the length likely to overflow (e.g. for riverbanks 10% of the length)

STEP 1 RISK IDENTIFICATION

Step 1a Failure mode identification

Threats, failure modes and breach types (for Tier 1) of RARS was amended such that the failure modes shown in Table 4 were analysed for FDRs.

Table 4. Failure modes considered in Tier 1 assessments

Threat	Failure mode		Comment
Internal	FM1	Internal erosion in embankment	
	FM2	Internal erosion in foundation	
	FM3	Internal erosion along interface between structure and embankment	
External	FM4	Flood – crest overflow	
	FM5	Floods – overflow of sides of chute	Not often considered credible at FDR
	FM6	Slope Instability of downstream slope	
	FM7	Floods - scour of downstream slope	Not covered by RARS. Method developed

Step 1b Potential Consequences of dam failure and release of reservoir

This was pre-populated from the national Reservoir Flood Mapping (RFM) data held by the Environment Agency. However, this was also expanded by the EA Geomatics team to include the other measures of consequences not included within RFM data, namely community health assets, economic activity, environment and cultural heritage.

For washlands the dry day normally has higher consequences and is used to assess risk. This is because in the wet day scenario if the washland is at capacity or spilling, it is likely that the adjacent watercourse is at the same raised level, and fluvial flooding is likely have occurred downstream due to runoff from the adjacent main river catchment and warning/ provisions made.

STEP 2 RISK ANALYSIS

Step 2A likelihood of failure due to internal threats

As FDRs are normally dry, indicators of poor condition may often not be observed. It was therefore necessary to extend RARS Tables 4.17 and 4.18 to cover FDRs, as shown by the red text in Tables 5 and 6.

Table 5. Extensions to RARS Table 4.17: Supplementary guidance on assigning intrinsic condition score for embankment dams (Tier 1)

Intrinsic condition score	Extent to which feature means dam is vulnerable to failure, that is, criticality in failure modes analysis		
	Embankment		Foundation
	Features present at site	Fallback for section of flood detention reservoir being assessed where no information (Note 2)	
5 – Body of dam/foundation vulnerable to failure	Embankment shoulder does not act as a filter to core Hydraulic gradient across core > 5	Historic transportation, or flood defence, embankment forms part of section of embankment being assessed	a) Erodible or compressible foundation b) No foundation treatment such as slush grout/dental concrete on open jointed hard rock foundation
4	Erodible core material (silt or dispersive)	a) Embankments built by developer before 2000 b) embankments built with colliery spoil	
3	a) Downstream slope steeper than 2H:1V b) Abutment slopes > 1V:1H or steps > 0.1H c) No filtered drainage in downstream shoulder		No foundation cut-off
2	Core material low plasticity clay	Modern dam built since 2000 (i.e. likely to have been designed after Environment agency founded in 1996)	
1 – Design/ construction inherently resistant to failure	Filtered core		On in situ rock, which is low permeability/been adequately treated to reduce risk of internal erosion

Notes:

1. Selection of score is judgement by user. Either take highest score (worst case) across both columns as giving condition (not average or minimum), or where several vulnerable features combine to give higher score. Where unsure (for example, no drawings) then do not score zero, but score most likely condition (for example based on typical construction practice at time the dam was built or upgraded).
2. Amended following the same approach as set out for Tier 2 in Table 8.17 of RARS

Table 6. Extensions to RARS Table 4.18 Supplementary guidance on assigning current condition score for embankment dams (Tier 1). 'Current condition scoring' system for probability of failure due to internal threats

Current condition score	Extent to which feature is symptomatic of performance and thus likelihood of failure		
	Surveillance and Monitoring	**Reservoir operation/ability to lower reservoir**	**Extended guidance for FDR**
3	• Surveillance <2 per week in dams which are vulnerable to rapid failure (Note 2) • No surveillance (dam not vulnerable to rapid failure) • For flood detention reservoirs the surveillance during impounding events is applicable, which is normally daily, so this is not normally the governing consideration		Normal for washlands, which are more difficult to check every metre length
2	• No instruments at dam, or readings not evaluated within one week of reading • Poor ability to inspect (that is, large leak would not be detected	• Never been filled - for example flood detention reservoir • No fixed bottom outlet/means of lowering reservoir in an emergency • Annual refill is rapid (>10% of dam height/week) • Rate of lowering with fixed bottom outlet < Hinks formula	Normal for impounding reservoirs

Notes:

1. No change to features for seepage quantity or deformation, or Current condition scores 1, 4 and 5.
2. Selection of score is judgement by user. Take highest score (worst case) across all columns as giving condition (not average or minimum). Where unsure (for example if no settlement or seepage monitoring) then do not score zero but score most likely condition.
3. Dams which include one or more of the following are vulnerable to rapid failure – (i) non-cohesive core, (ii) sandy foundation, (iii) outlet pipe in cut and cover trench with no sand collar filter

Step 2B Likelihood of failure due to external threats

FM7 Erosion of surface protection to spillway

It was recognised at the onset of the project that a key failure mode, erosion of a grass spillway, was not included in RARS, so the matrix shown in Figure 1 was developed to provide a Tier 1 assessment of the likelihood of failure.

Figure 1 Likelihood of failure for grass spillways

velocity in safety check flood) / CIRIA 116 allowable velocity (% overstress)	Quality of information on design and construction (note 2)			Source/ comment
	Poor e.g. no construction records	Good i.e. drawings include anchor/ overlap details	Excellent e.g. trial excavations to confirm depth of topsoil	
>175%	Extreme	Extreme	Very High	
150%	Extreme	Very High	High	
125%	Very High	High	Moderate	
100%	High	Moderate	Low	This is allowable velocity, so would not expect failure if properly designed i.e. these values selected on basis of RARS table 15.3. This also reflects that the values of v calculated by the app will be peak values, not the full 10 hour duration
75%	Moderate	Low	Very low	
50%	Low	Very low	Very low	

Notes

1. For reinforced grass systems the above assumes average quality grass (for plain grass systems the grass quality should be compared to the design/ S10 requirements). Where grass is poor (based on most recent S12 or S10, whichever is most recent) then increase level of risk by one class
2. As well as the quality of grass cover, the effectiveness of grass spillways also depends on the depth of reinforcement below the surface, the detailing of laps and anchors trenches, and the presence of granular layers beneath concrete systems. We suggest the input data includes assessment of quality of the grass reinforcement

FM6 Slope stability

The methodology given in RARS for assessment of the likelihood of slope instability of an embankment dam was developed for a normal reservoir which is full most of the time, so the downstream slope is dry with a phreatic surface governed by seepage through the dam core/ foundation.

This is inappropriate for flood detention reservoirs, where the downstream slope is normally also the downstream side of the spillway, and thus subject to periodic overflow when the spillway is operating, when the slope is likely to saturate and thus be subject to a different pore pressure regime from the above. Tables 4.6 and 4.8 of RARS were therefore amended for use in checking the stability of a spillway slope under overflow as shown in Table 7. Table 8 gives an updated example illustrative of output for RARS Box 4.4. The App includes a switch to select whether the slope stability is being assessed for the spillway slope, subject to overflow, or a non-overflow section of the perimeter bund.

Table 7 Extension to Table 4.6 of RARS Indicative modern slope design

Soil type	Downstream face (no overflow)		Downstream face of spillway (subject to overflow) (Note 2)
	Modern design slope (Note 1)	Source	
Sand, gravel	2.5H:1V	Section 9.2.3 of CIRIA Report 161 (Kennard 1996a)	3.6H:1V
Low plasticity clays	3.0H:1V		4.3 H:1V
High plasticity clays	4H:1V	Figure 10 of Vaughan et al. (1979). For more detailed assessment where slope angle is related to geological origin of the construction material reference can be made to Table 4 of Parsons and Perry (1985).	5.7H:1V

Notes

1. Downstream slope on good foundation. Where pre-existing shear surfaces are present at the ground surface (for example, due to periglacial action), then much flatter slopes would be required. For example, the redesign of Carsington dam adopted flatter slopes (Johnston et al. 1999) and overall slopes of around 10H:1V have been required on some dams to ensure foundation stability.
2. Equal to 70% of the slope in column 2. This value of 70% has been derived using spencer stability charts (Spencer, 1967), and assuming that RARS Table 4.6 refers to slope with c' of zero and ru of 0.25.
3. Table 4.8R Likelihood of release of reservoir given slope instability. Add note 1. Reduce output likelihood score by one increment where spillway to flood detention reservoir

Table 8. Extension of RARS Box 4.4R. Example for illustrative purposes of instability of embankment slope.

Parameter	Units	Embankment face with no overflow		Downstream face of grass spillway (subject to overflow)	
		Value/ Score	Remarks	Value/ Score	Remarks
Slope angle SA	H;V	2.5		4.0	Typical on grass spillways
Crest width	m	11	36 feet (11m)	5	
Dam height	m	12		8	
C/H		0.9		0.63	
Modern design standard slope angle (Table 4.6) SM	H:V	3.0	Using Table 4.6 – Assume low plasticity clay. Slope = 3.0H:1V based on Kennard (1996)	4.3	Assume low plasticity clay
Difference to modern slope design (SM-SA)/SM	%	17%		7%	steeper
Likelihood of slope failure (Table 4.7)		High	Slope is up to 25% steeper	Moderate	
Likelihood of release of reservoir given slope instability (table 4.8)		Reduce by two increments	as example at base of Table 4.8	Reduce by one increment	as new Note to table 4.8
Overall likelihood of reservoir failure		Low		Low	

Steps 2D and 2E Consequence analyses and level of risk
These are prepopulated in the App, but with provision for the user to make manual corrections.

Steps 2F Review outputs
The App includes the suggested checks in RARS with the user to populate whether they consider the assessment's output complete, credible and are confident in the output, together with any comments on data gaps etc.

Step 3 Risk evaluation
The App includes the suggested checks in RARS under Steps 3d and 3e (earlier steps not applicable to Tier 1, or for 3b not in project scope) with the user to populate whether they are satisfied, together with any comments.

DISCUSSION

Challenges/ lessons learnt
The main challenges were achieving consistency between assessors, and in achieving a common understanding of headings/ terminology in the App. The exact terminology used in the RARS Guide was reproduced in the App but this was not always easily understood by the users in relation to FDRs, particularly where terms varied for different reservoir types. This was resolved by production of supplementary "guides" and in-App prompts. The project intends that these assessments would then be used by Environment Agency asset managers, so these guides and training will be critical if the asset managers are to understand, and use, the risk assessments in managing their assets.

Another challenge was the project programme, as the App was updated several times, both to clarify headings and/or to add the extensions needed for FDRs, and update it with the consequence data from the latest national RFM outputs as these became available.

Validation of Tier 1
The outputs were reviewed against each other, and against the indicative range of likelihood of failure of UK dams given in Figures 15.3 and 15.4 of RARS (using the implied ranges of quantitative values in Table 15.3 of RARS).

The main anomaly discovered was that by including economic activity and environmental designations at Tier 1, this often resulted in the highest (i.e. a class 4) consequence even when there is no population at risk. Thus it is implied, for example, that a single SSSI is equivalent to multiple fatalities. As these receptors cannot easily be monetised at Tier 2, RARS plots them separately from the property damage and life loss when assessing risk, as shown in Figure 9.3 of RARS, and it may be worthwhile doing the same at Tier 1.

Implications for Tier 2 and 3
The extensions listed in Table 2 will also be necessary for Tier 2, as FDRs have fundamental differences from reservoirs which are normally full of water. In addition, it will be necessary to consider how to treat dry and wet day failure scenarios. This is not straightforward as the "dry day" for FDRs is when the reservoir is full in an operational flood, but not spilling, so it may be appropriate to derive two separate probabilities for internal threats, relating to dry and wet day failures.

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Another challenge will be developing methodology to ensure consistency in identifying failure modes at flood detention reservoirs, as these vary from normal dams. Although the principals in sections 16 ('Guidance on failure mode identification') and 7 ('Tier 2 – Step 1 Risk Identification') of RARS can be used, it is likely that a framework will need to be developed, trialled and then reviewed against actual performance. Useful data to validate the output would involve collecting data on:

- Annual failure rates of flood embankments (fluvial and coastal) as these have many similarities to FDRs
- Incidents and failure of control systems on active flow control systems.

It is also noted that internationally good practice in carrying out risk assessment has developed significantly since 2013 and some aspects of these may be of value in extending the Tier 2 analysis to FDRs.

CONCLUSIONS

The Environment Agency has carried out a Tier 1 risk assessment on their portfolio of 217 flood detention reservoirs (FDRs), which allowed screening of reservoirs where risk is tolerable, and those where more detailed study is necessary. This has necessitated various extensions to the Tier 1 methodology in RARS and this paper describes these extensions and refinements relevant to FDRs. Similar extensions could be applicable to FDRs owned and operated by other agencies. The updated methods have been encoded within a web-based application that has been used by multiple staff at multiple consultant organisations to undertake the consistent risk assessments, and this has produced a live database of Tier 1 assessments for all EA's FDRs.-

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The British Dam Society

Design of Dam Safety Measures for three Dams in Zambia

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SYNOPSIS Arup was commissioned by the United Nations Office for Project Services (UNOPS) in 2022 to design measures for improvements to the safety of three dams in Zambia. The three dams were all homogeneous earthfill dams, but with differing geometries and spillway forms. All three dams share the common problems of inadequate flood capacity, lack of freeboard, scour damage and irregular dam construction with excessive seepage in places, but they differ in the hazard they pose to the downstream population.

The paper will describe the assessment of the downstream hazard posed by each dam, the identification of the proposed solutions and optimisation of the resulting designs.

INTRODUCTION

UNOPS was tasked to address the unresolved issues from the Zambia Water Resources Development Project funded by the World Bank. In 2022 Arup was appointed by UNOPS to design solutions for the dam safety issues on three of those dams: A, C and K. Arup visited the three sites in July 2022.

The dams were originally designed in 2014 and constructed in 2016 by local organisations and contractors. Others had carried out inspections of the dams and identified a number of deficiencies in common including the need to verify the construction quality of the dam and key design parameters.

Dam A

Dam A is an 11.5m high embankment dam situated in the Eastern Province of Zambia. The slopes are approximately 1:2.5 to 1:3 with a 4m to 5m wide crest. The overall storage capacity has been previously estimated at 710,000m^3, with surface area of the waterbody of 13.5ha and catchment of 516km^2. The reservoir is suffering from extensive siltation and is expected to have a limited useful life. The reservoir was designed for livestock watering, domestic water, recreation and fishing. However, it is understood that the reservoir is currently only used to provide water for livestock watering, with no supply offtake facilities included for other uses.

On the west side of the site there is an L-shaped concrete service spillway 110m long with a sloping concrete wall and masonry crest; the structure appears to be founded on rock. Low areas of dam crest were observed next to the weir abutment walls.

The recommendations in the interest of dam safety include mitigation measures to address the lack of sufficient overflow capacity and freeboard, leakage and erosion of the service spillway, leakage in the low level outlet, and extensive scour in the downstream channel. Seepage was noted as upwelling a few metres downstream of the dam toe in a single location.

Figure 1. Dam A Site arrangement (Google Earth, Arup elaboration)

Dam C

Dam C is an 8m high earth embankment dam in Laupula Province, Zambia. The overall storage capacity has been previously estimated at 3,000,000m^3, with surface area of the waterbody of 90.7ha and a catchment of 374km^2. The reservoir is to provide water for irrigation and livestock watering.

The dam alignment has a 90-degree bend to accommodate the service spillway approach channel and weir. The design is understood to have been changed during construction to locate the spillway weir on a rock outcrop. There is a secondary embankment to the right of the primary spillway weir with low areas in the crest adjacent to the primary spillway. For the main east-west dam the existing upstream and downstream slopes are typically at a gradient of 1:3. The embankments north and west of the spillway are steeper at around 1:2; visibly steeper than the main dam geometry and with a narrower crest.

The service spillway is a weir L-shaped in plan, 32m long, with stepped chute. There is a partially completed emergency spillway to the left-hand side of the embankment, however the crest level of this emergency spillway area is only marginally below the dam crest and concerns were raised about the lack of capacity and freeboard.

The site had been inspected by UNOPS in November 2021 following a 5m long crack being identified in the crest of the steep sided embankment to the left of the spillway approach, which had apparently been repaired prior to the arrival of UNOPS engineers. It also noted undercutting of stone pitching, abutments and training walls at the spillway and excessive leakage at a valve chamber. The dam was otherwise found to be in fair condition though some seepage was noted downstream of the dam toe.

Figure 2. Dam C Site arrangement (© UNOPS)

Dam K

Dam K is a 9m high earthfill embankment dam in the Copperbelt Province of Zambia. The overall dam storage capacity has previously been estimated to be 2,030,000m^3, with the surface area of the reservoir being approximately 63ha and a catchment of 287km^2. The main embankment length (running from west – east) up to the west side of the service spillway is approximately 210m. A marshy area was identified downstream of the dam and no toe drain could be identified.

There is a service spillway on the east side of the dam (left bank) with a three-stage unlined drop structure with three straight concrete weirs. The overall discharge structure length is approximately 105m, with weir lengths of 30m. There is scour undermining around the drop weirs and the training banks along with potential seepage. The spillway discharges into a vegetated channel and flows in the south-westerly direction towards a culvert that passes underneath a major highway. There is an incomplete emergency spillway on the west side of the site. The incomplete spillway is covered by vegetation and the current ground level has not been lowered, resulting in minimal freeboard between this channel and the dam crest.

The reservoir is to provide water for domestic supply, aquaculture and livestock watering though no water supply draw-offs are currently in use.

Figure 3. Dam K Site arrangement (Google Earth)

Managing Risks for Dams and Reservoirs

DAM CLASSIFICATION

There is no specific dam classification system legislated for use in Zambia, so we considered a range of references. Zambian Dam Guidance (MAFF, 1998) makes reference to a system (Kabell, 1986) which classifies dams into four categories based on dam size (height, volume of reservoir) and hazard potential (loss of life, economic).

Dam A is located in a very remote area with no settlements observable downstream, so the risk to life and economic risk is therefore low to very low. It was debated whether to classify Dam A as class 2 or 3, depending on whether the risk to life could ever truly be "impossible". Ultimately a classification of 2 was chosen as economic losses were considered moderate and a smaller flow estimate had been obtained, so a higher classification combined with lower flow estimate was used.

Dam C would classify as a Medium size dam based on dam height and potentially large based on the large storage capacity, though this is expected to be slightly less than $3Mm^3$. The hazard potential would classify as Moderate to High as loss of life would be likely (possible to probable) and economic loss appreciable. The dam is therefore the highest Class 1.

Dam K would classify as a Medium size dam as loss of life would be unlikely (improbable) due to only sparse residential properties in the valley, therefore class 2 was selected.

An independent comparison has been made with ICOLD Bulletin 157 (ICOLD, 2016) which classifies small dams as follows:

> Dams 5m to 15m high and with a $H^2.V^{0.5} < 200$ (H height in metres, V storage in Mm^3).

It then classes the dam based on $H^2V^{0.5}$ and the downstream consequences. Dams A and K classify as small dams and with fewer than 10 lives expected to be at risk. Dam C has a $H^2V^{0.5}$ greater than 200 with potential loss of life greater than 10 and was therefore considered a higher risk large dam.

Table 1 Design and Safety Check Flood selection

Dam	Size	Hazard Potential	Kabell Classification	ICOLD 157	Design Flood	Safety Check Flood	
				$H^2V^{0.5}$	PHC		
A	Medium	Very Low to Low	2	121	2	1 in 500y	2000y
C	Medium to Large	High	1	293	3	1 in 2000y	1 in 10,000y
K	Medium	Low	2	118	2	1 in 500y	2000y

The definitions of design and safety check floods in Bulletin 82 (ICOLD, 1992), as also adopted in UK practice, were adopted. Kabell recommends design floods of between 100y for class 4 and 2,000y return periods for class 1 with dry freeboard allowance. For safety check floods it recommends 250y to 10,000y return period floods with no dry freeboard.

For comparison, ICOLD Bulletin 157 recommends safety check floods of 50y (PHC1) to 1,000y (PHC3) for rural areas and 1,000y (PHC1) to 10,000y (PHC2) for more populated areas. French guidance on small dams (CFGB, 2002) also recommends a minimum design flood of 5000y return period for $H^2V^{0.5}$ in the range of 100 to 700 and an absolute minimum of 1000y where there is a risk to life. The magnitude of safety check floods from Kabell is therefore comparable to the approach in many countries, however we additionally included freeboard in the safety check case.

A hydrological analysis was performed, initially using a regional flood method intended for Zimbabwe (Mitchell, 1998) as the closest available method. As rainfall data became available, a rainfall runoff analysis was performed. Rainfall data for a 30-40 year period was obtained from a wide range of rain gauges from different sources with at least three gauges in reasonable proximity to each site. No river flow gauge data could be obtained. Attempts were also made to use satellite rainfall data. However, this was found to not accurately identify peaks in rainfall.

Hydrological analysis had also been performed by a previous consultant using a South African form of modified rational method, which proved a useful comparison. The three estimates are shown in Table 2 below. The Arup and previous estimates generally showed good agreement and these estimates were typically lower than the Mitchell method. This was expected as most of Zambia experiences less significant rainfall than Zimbabwe where that regional flood equation was developed. The exception to this was at Dam C where the Arup estimates were greater. This is because more local rainfall data had been obtained and the site was found to be in a higher rainfall area proximate to a large inland lake. For Dam K Arup produced lower flow estimates from similar rainfall data, as the land cover was found to differ significantly from the earlier estimates.

Table 2 Flood estimates

	Peak flow (m^3/s)		
Method	**Dam C**	**Dam A**	**Dam K**
Safety Check	**10,000y**	**2,000y**	**2,000y**
Mitchell Method 1998	2072	1768	1265
Rational Method 2018	820	638	1175
Arup Type II	2083	686	877

DAM BREACH ASSESSMENT

Following the initial costing of the preferred option for each dam, the dam hazard categorisation was questioned by the funding agency. It was felt that the potential consequences of failure of Dams C and K were lower and that a smaller safety check flood should therefore have been selected. We therefore carried out a dam breach assessment with incremental consequence estimation to establish the hazard categorisation of dams C and K in a quantitative way. Dam A was not assessed as the consequence of failure could clearly be seen to be minimal due to the remoteness of the site.

The following flow scenarios (after ICOLD, 2005) under which failure of the dams can occur were considered in the analysis:

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- Sunny Day Failure, where the failure occurs under normal flow conditions (not associated with a flood event) and at the normal operating head water levels (water level at spill level).
- Rainy Day (Flood) Failure, where the failure of the dam is associated with the occurrence of a flood of a given return period and at head water levels above normal operating levels. For this scenario the potential range of safety check events was considered for each dam.
- Baseline Non-breach Scenario, where the flood inflow hydrograph is modelled passing through the river valley without routing through the reservoir. This scenario was run to enable performing incremental hazard assessment for the flood failure.

As the dam classification and therefore the return period of the safety check event was under examination, a range of possible safety check events was run for both dams.

The following dam breach models were applied for both sunny day and rainy-day scenarios: Xu and Zhang (Xu, 2009), USBR SEED (USBR, 1995) and Molinaro (ICOLD, 2005) to generate the breach hydrographs. For the sunny day failure for both Dam K and Dam C, the Xu and Zhang hydrograph was selected, as it was most applicable for low height dams. Xu and Zhang produced a credible result lying below USBR SEED, which is known to be conservative, but above Molinaro. Froehlich time to peak for Xu and Zhang is recommended by the UK's Environment Agency (Environment Agency, 2013) and produces a faster time to peak, so it was adopted in combination with Xu and Zhang peak flow. For consistency and using the same reasoning as for the Sunny day scenario, the Xu and Zhang hydrograph was selected for the Rainy Day scenario. The breach hydrographs were added to the peak of the inflow hydrographs for each return period.

The dam break analysis was carried out using a 2-dimensional model developed in the latest TUFLOW HPC (version 2020-10-AF). Topographic information was limited to freely available data. Following a review of potential topographic data sources, ALOS Global Digital Surface Model "ALOS World 3D" – 30m (AW3D30) was adopted. TUFLOW HPC's Quadtree Mesh capability was used to allow for variable cells sizes to be adopted within the model. The topography was manually modified to enforce the primary waterway channels for the full extent of the model and to manually smooth heavily vegetated areas (where vegetation captured in the AW3D30 data artificially interfered with the conveyance of flood flows). Manning's n roughness values were prepared initially using Open Street Map (OSM) data.

For the consequences assessment, the study focused on inundated buildings and potential fatality rates from the building occupancy. This was due to lack publicly available information about average road usage, as well property values. Flood extents and velocity rasters from the modelling were intersected with the building overlays form OSM. The buildings layer was reviewed against the aerial imagery within the maximum flood outlines and where additional buildings were visible, they were manually added to the overlay. Where building use can be suggested from the aerial photography, the building use was also recorded. As many buildings in the original layer did not have use indicated, it was assumed that buildings with unknown purpose with footprint of less than 100m^2 are residential, and any buildings above that size are used for commercial/agricultural purposes.

The occupancy rate for the residential buildings was 5.1 people per household, as obtained from the 2015 Living Conditions Monitoring Survey (Zambia Central Statistics Office, 2016). As no information was found on occupancy rates for agricultural, commercial or industrial buildings, occupancy rate of 1 person per building, present at all times, was assumed. Due to lack of information of typical times spent outside of residential homes, the occupancy rate for the residential properties was not time averaged, i.e. it was assumed that 5.1 residents are always present. As the occupancy rate for the agricultural/commercial/industrial buildings was relatively low and irrespective of the building size, it was considered that there is little probability of double counting of the population. A 5m buffer was created around each building to compensate for the grid size around buildings. The maximum velocity and depth that were within the building envelope was used to calculate the exposure risk and fatality rate for each building (**Error! Reference source not found.**).

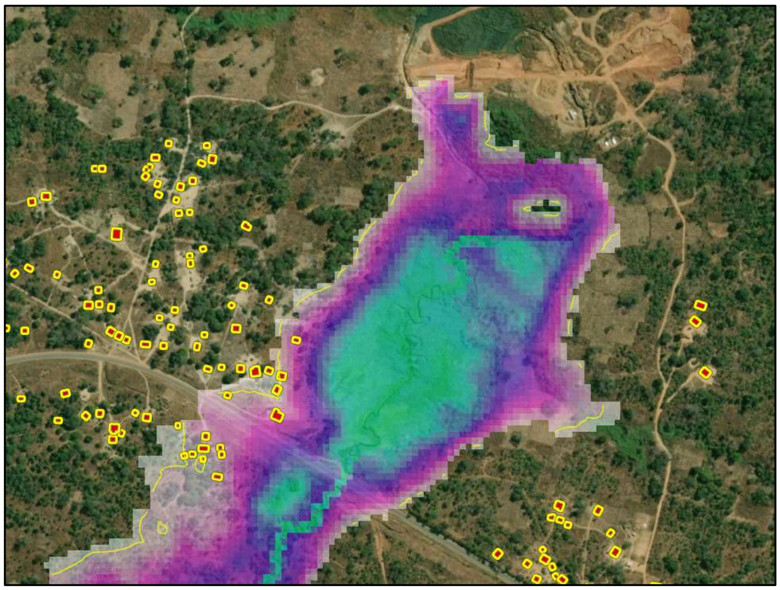

Figure 4. Dam K Rainy Day Flood Extents and Affected Properties with Buffer Zone

For the hazard assessment the Loss of Life is the critical measure at which the hazard rating of a dam is determined. Both the RARS methodology (Environment Agency, 2013) and Defra guidance (Defra, 2005) were used to produce a possible range of results. For the Sunny Day scenario the likely loss of life at Dams C and K was found to be little or none. For the Rainy Day Scenario, the Baseline No Breach Population at risk, loss of life and injured people were subtracted from the "Breach" measurements to calculate the incremental consequences considering with and without flood warning systems being introduced.

The consequences were plotted on an F-n Plot for both dams to assess the Hazard Category. Dam C showed high consequences and justified the selection of Class 1 by the Kabell system, as originally it was qualitatively assessed. Dam K was on the borderline between ALARP and unacceptable safety risk for a safety check event of 2000-year return period, and unacceptable for a return period of 500-year event. Combined with the damage on the major road downstream, it was decided to keep Dam K as Class 2.

EXISTING FLOOD CAPACITY

At Dam A the safety check flood estimate represented only a slight increase compared to the apparent design flood flow of 580m^3/s. Under free discharge conditions this flow was found to be a credible estimate of the spillway discharge capacity. However, an assessment of the tumble bay and downstream channel suggested submergence of the weir, reducing its capacity. This was supported by video evidence from the client, showing a past flood event where only shallow overtopping of the weir occurred but the downstream channel could be seen to be full.

Dam C was found to only be able to pass around $125m^3/s$ before overspilling of the low sections of dam crest adjacent to the primary spillway. This occurs at around the ground level of the emergency spillway, where negligible flow occurs.

At Dam K the existing spillway is expected to pass around $304m^3/s$ with a reservoir water level of 1195.75mEL (a head of 3.2m over the primary weir); this represents zero freeboard to the dam crest adjacent to the spillway weir with $281m^3/s$ on the primary spillway and the remaining $23m^3/s$ in the emergency spillway. However, the banks of the spillway were assessed as only being able to safely pass $152m^3/s$, increasing to $174m^3/s$ if localised low points were filled.

OPTIONEERING

Multiple options were considered by Arup to address the safety measures for all three of the dams. The option of decommissioning or lowering of the full supply level was ruled out as unacceptable for all of the dams in a separate high-level optioneering exercise before Arup's commission. There were common problems to be addressed for all three dams, namely inadequate overflow capacity, excessive seepage, areas of over-steep embankment slopes and irregular crest elevation. The geometry of the dams, size of the reservoirs, required discharge capacity and the downstream consequences of failure, however, differed significantly.

For Dam A various options were considered, including (1) raising the dam with no new spillway works; (2) constructing a new spillway east of existing or (3) constructing a new spillway on the right abutment. The right bank was quickly discounted as it would be partially constructed on the dam, partially on the abutment and would require extensive works to returning the flow to the downstream river past the dam toe. It was therefore determined that there was no benefit to pursuing option (3) in preference to option (2).

A new spillway would need to be of similar length to the existing weir and so the same degree of dam raising. Different lengths were considered but even significantly longer spillways required significant dam raising and the downstream channel becomes a limiting factor.

Therefore raising the dam with use of the existing spillway was found to result in the need to raise the dam by approximately 3m and to reinforce the existing spillway and abutments with new structures to permit the raising of the dam crest and filling in the existing low areas of crest. It also required the widening of the downstream channel to prevent submergence of the weir. This option was selected as the most economic.

The options considered for Dams C and K included upgrading the existing service spillways and completing the emergency spillways or building new service spillways. For both existing spillways it was determined that large scale works would be required to allow them to safely pass significant flows, even in combination with an entirely new spillway elsewhere. There was also an advantage in being able to utilise the existing spillways to pass flows during construction of the new spillways. New spillways would therefore be designed for both dams with the original spillways decommissioned on completion. Two preferred options were identified, the first to construct a long channel spillway, which for Dam K would be located on the right bank on the line of the incomplete emergency spillway. For Dam C this would be similarly located at the left bank emergency spillway. The second option was to construct a spillway over the dam crest itself. The first option could be constructed on dry higher ground with the reservoir fully impounded, the disadvantage being that the long chute length would result in a high construction cost. The second option would be cheaper to construct but we considered it necessary to dewater the reservoir to safely construct it, as the existing flood capacity was so low we would want a cofferdam equal in height to the existing dam. Initially the client preferred the first option as it was not desirable to lose the reservoir storage. However, after some design development of the first option a cost estimate was produced. As a result the design was changed to the on-the-dam option which was estimated to be 60% cheaper.

For all three dams discrepancies were found between the original dam design drawings and what had been constructed. Some topographical surveys and ground investigations had been carried out, but some inconsistencies remained. As a result, we specified new topographical surveys and ground investigations to be carried out and worked closely with the local companies to ensure accuracy of the results. All three dams had similar geology with residual soils of varying proportions of dense silt, sand and stiff Clay with some areas of looser transported materials. At Dam C ground conditions below the dam were largely Medium to Very dense residual clayey Silt with some areas of transported sands and gravels and fill of sandy Clay. Dam K had similar residual soils but with areas of stiff sandy Clay and others of sandy Silt and clayey sand, fill material is sandy Gravel. At Dam A it was predominantly Medium dense clayey Sand with soft weathered sedimentary bedrock at a depth of around 4m, the fill material sandy Clay. In all cases the existing fill was found to have a permeability of between 10^{-7} and 10^{-8}m/s. In addition to testing of the dam fill the original borrow pits were identified and tested for obtaining additional fill.

The three dams were all found to be homogeneous despite drawings suggesting impermeable cores. Otherwise the dams were found to be well compacted. We carried out slope stability analysis and found that the dams achieved acceptable factors of safety in all load cases with the exception of areas where the slopes had been over-steepened. We also carried out seepage analysis based on both laboratory testing of the existing dams and back analysis of the seepage observed on site to calibrate the seepage rates used. The analysis was then repeated for the raised dam and flood cases. The planned raising of the dams allowed us to regularise the slopes at 1:3 (v:h) and incorporate a downstream filter layer and toe drain. The seepage would therefore remain, however the material had been found to be non-dispersive and with the filter layers the risk of migration of fines and internal erosion was mitigated. Additional works included providing a rock armour protection to the faces of the dam to provide protection from waves and cattle, which had been observed at all sites.

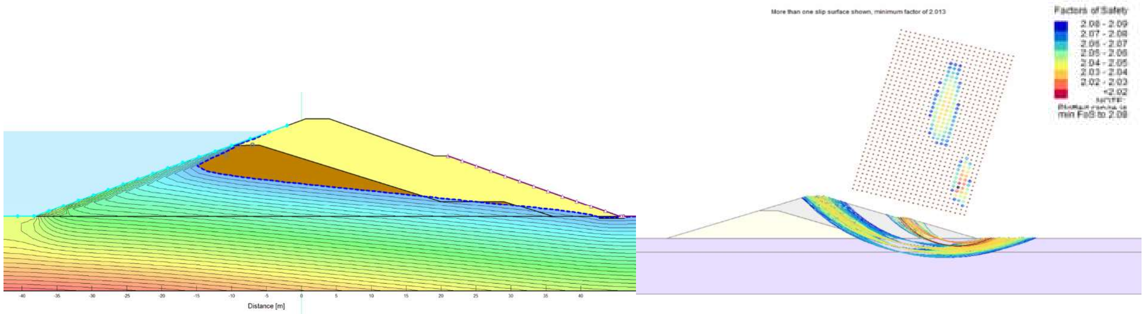

Figure 5. Dam C Seepage and stability analysis

DESIGN OF DAM A

The dam was raised to pass the design flow with free discharge over the existing weir; this then set the required height of dam raising. Raising the dam would also allow the overly steep section of dam to be corrected, the inclusion of a filter layer, rock toe and drainage to control the seepage and prevent internal erosion. Raising the dam meant the need to also raise the abutments walls of the weir which retained the end of the dam. The existing construction was unknown so we designed a new wall to line within the existing weir, doweled to the existing, to support and raise the wall.

To improve weir capacity we widened the downstream channel. This was analysed treating the downstream channel as a side channel weir to capture the water profile downstream of the L-shaped weir. Widening the channel also allowed us to re-line the channel bank protection to prevent scour. Given the importance of the downstream water level and preventing submergence of the weir and the complexity of the downstream geometry we carried out CFD modelling which allowed us to reduce channel excavation and identify the requirements for bank protection.

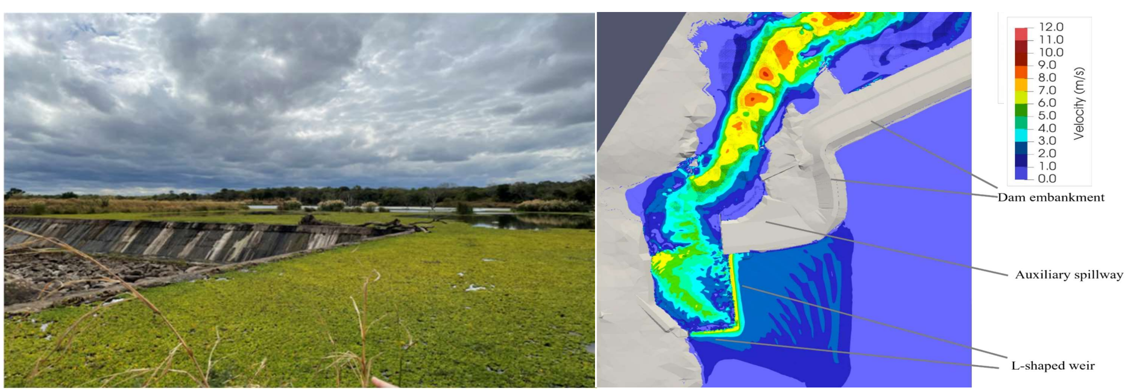

Figure 6. Dam A weir and CFD results

DESIGN OF DAM C AND DAM K

Spillway Weir type

A new over-the-dam spillway in combination with dam raising was selected as the preferred option for both Dams C and K. The cost of supplying concrete was a significant cost element, and reducing the size of the concrete spillway structure was a design priority. As the designs on the two dams were similar it was decided that the spillway weir design will be developed and modelled for Dam C and then scaled for Dam K.

Both ogee and labyrinth weir design options were developed for a hydraulic head of 5m at dam C in the safety check flood. Traditionally, ogee designs have been the more commonly adopted solution in scenarios with high hydraulic head due to their proven efficiency in handling such conditions. The primary concern with labyrinth weirs in high head cases is the potential for weir interference significantly reducing the efficiency. However, research by Crookston (Crookston, 2010), that extended the theoretical labyrinth weir efficiency coefficient curves up to ratio of H:P=2 was used to optimise the labyrinth weir design.

The analysis demonstrated that even with the severe reduction in the discharge coefficient, the labyrinth design demonstrated better efficiency in terms of overall spillway width when compared to the ogee design (Martin, 2024). The chosen labyrinth configuration provided a spillway width of 65m for Dam C, and a 22% potential reduction in concrete volume compared to the ogee. The labyrinth design was taken forward for CFD modelling to prove the concept and for further development into a detailed design.

Design and CFD Modelling

For Dam K the labyrinth was modified by reducing the number of cycles, but not changing the height, arm length and angle of the labyrinth. The head over the weir for the Dam K design flood was similar to the Dam C safety check; in this way the rating curve obtained from the CFD modelling of Dam C could be scaled. The labyrinth cycles were reduced to four (from seven in Dam C), and the overall width of the spillway was estimated to be 37.5m. The labyrinth weir design in combination with the updated hydrological assessment, allowed for the dam raising to be limited to approximately 1.5m above the existing dam crest.

This allowed us to undertake a single CFD model for Dam C and then use these results to verify the performance at Dam K. We found that the upstream reservoir water level was higher, at 1192.85mEL, than we had predicted in the calculations (1192.3mEL). We determined that this was likely due to the large head, around the maximum from the research studied, resulting in slightly greater interference over the labyrinth than predicted, but also due to the approach conditions to the weir. As a result, the wingwalls were flared to reduce the entry losses; the wall height had been set as a parapet height of 1.2m following the dam face profile, partly to provide edge protection but also to allow some overtopping to ease the approach to the weir but keeping the flow above the dam to reduce scour risk. This amendment was successful in reducing the peak safety check reservoir still water level to 1192.56mEL, however with reservoir attenuation this reduced to 1192.23mEL, within the original target.

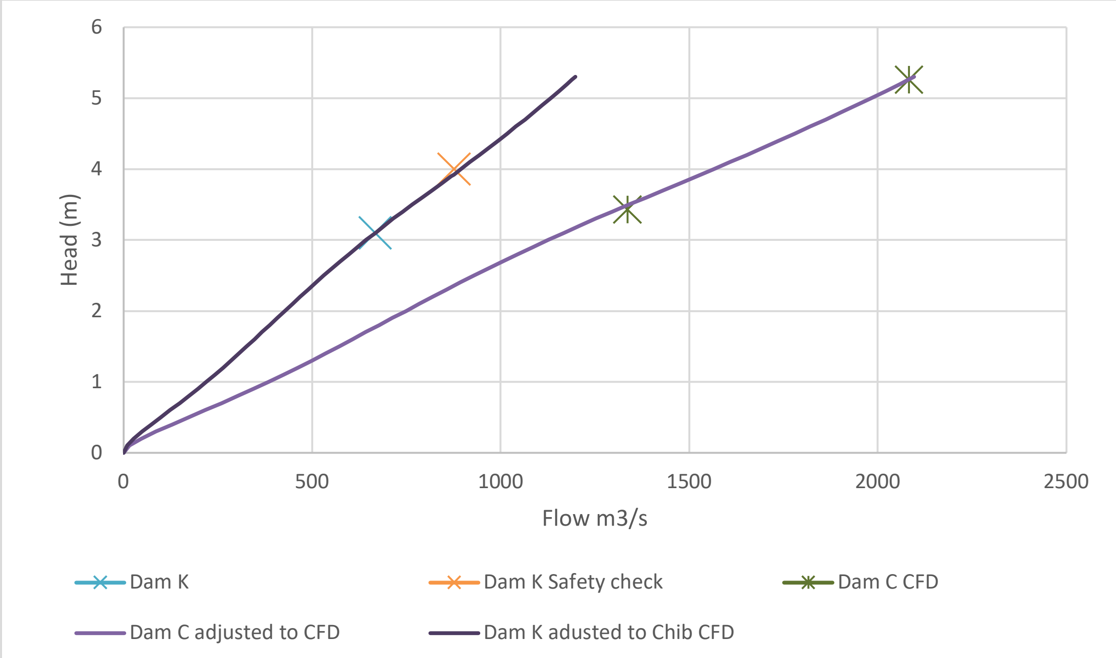

Figure 7. Rating curves with CFD results

CONCLUSIONS

Three different dams were all found to have largely the same dam safety problems which required resolution. A common approach was taken to the dam raising and seepage control measures due to the similarity of the dams and materials. After investigation differing approaches were taken to resolving the flood capacity shortfall, with Dam A making use of the existing structure with enlargement of the downstream channel and raising of the dam and abutments. For dams C and K entirely new spillways over the dam were designed. This was in part due to the hazard potential of the dams but also due to the site conditions, and the desire to reduce capital cost on the project. Overall, this paper highlights the importance of considering cost-effective design alternatives in dam safety projects, especially in remote locations, where access to materials, labour and data can be limited.

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The British Dam Society

Control of reservoir water levels during construction when existing scour facilities are not available

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SYNOPSIS Whilst working on existing scour and draw-off systems it is not unusual to have to install temporary works in order to maintain specified drawdown levels, supply to water treatment works and compensation flows to the downstream watercourse. This paper investigates the many ways of achieving the required controls and reviews the potential costs and pitfalls associated with each of the identified options from schemes that Mott MacDonald Bentley (MMB) has undertaken.

Based upon multiple examples of projects undertaken, the paper presents temporary pumping, siphons and associated priming and control, learnings realised and solutions implemented to resolve commissioning issues. All works were undertaken on statutory impounding reservoirs and as such have been planned and undertaken with the supervision of an All Reservoirs Panel Engineer.

MMB planned and undertook the following works:

- Level control – Pant Yr Eos Reservoir
- Level control - Upper Carno Reservoir
- Compensation and augmented river flows - Usk Reservoir
- Compensation flows – Gouthwaite Reservoir
- Level control – Cwmtillery Reservoir
- Additional drawdown capacity – Castell Nos Reservoir

This paper summarises the methodology behind each of the installations, reviews the scale of costs for purchase, hire and maintenance, strengths and weaknesses for each of the installations and lessons learned for future projects.

INTRODUCTION

Many of the projects MMB has completed in recent years have required the need for water level control outside of the usual operation of draw-off valves and natural draw downs. Water quality, existing valve condition, operating restrictions and other works on-going can all affect the ability to use existing installations. The preference from an affordability point of view is to maximise the use of existing assets where appropriate and minimise use of temporary pumping and power generation as this is expensive in terms of both cost and carbon.

Managing Risks for Dams and Reservoirs

Key questions to ask when planning works are amongst others:

- Can the inflows be diverted?
- What duration are the planned works expected to take?
- What magnitude of flows are required?
- Are there any supply or compensation flow requirements?
- How critical is the supply or compensation?
- Are there any restrictions on discharge to the receiving watercourse?
- What are the options, which are most cost beneficial, or more reliable?

PANT YR EOS RESERVOIR

Pant yr Eos is a 597,000m^3 volume reservoir, with an embankment dam of 27m in height and crest length of 280m with a direct catchment of just over 1km^2 located near Newport, south Wales. In order to undertake upgrade works to the drawoff system in 2018, reservoir level control was required. At Pant yr Eos, single gate valves within the reservoir body control flow into a wet well shaft, from which flows are piped through the tunnel under the downstream shoulder, to the downstream watercourse. In order to complete works on the original drawoff facilities, it was necessary to lower the reservoir to an acceptable level. A by-wash channel was available and suitable for use during the construction works, and as such, it was possible to divert the majority of the incoming flows. The remaining flows amounted to circa 30 l/s which was required to be pumped due to lift (>20m) and water quality. Pumps were deployed using floating pontoons (Figure 1) to ensure water quality was maintained throughout the works and flows discharged to the downstream watercourse via the existing bywash channels. A single duty pump was utilised to keep costs lower, with the water level within the reservoir allowed to rise and fall between set levels which enabled works to progress unhindered. For more project details, see parallel paper by Cornelius and McAree (2024).

Figure 1. Pumping arrangement adjacent to Pant yr Eos dam. Right-hand bywash visible in the background

UPPER CARNO RESERVOIR

The dam at Upper Carno is a 14m high, 270m long Pennine-type embankment with a central puddle clay core. The reservoir has a volume of 0.34Mm³, a surface area of 0.063km^2, an operational Top Water Level (TWL) of 444.54mAOD and a total direct and indirect catchment area of 5.1km^2. Works were undertaken to many aspects of Upper Carno; for further details please see parallel paper by Swetman et al (2024).

A number of approaches were applied at Upper Carno and modified as the project progressed through distinct stages. Initially, the existing by-wash channel was cleared of vegetation and its use reinstated. This enabled diversion of the majority of flows received from one of the two stream inlets, directing flows to the head of the existing spillway (Figure 2).

The second inlet was controlled using duty / standby pumps, again pumping direct to the head of the spillway. A siphon system was reviewed for this element, however this was not hydraulically feasible. The pumping of the inlet was sized to control up to Q_{10} flows, with any flow above this retained in the reservoir basin and dealt with via siphons as the need arose.

Figure 2. Upper Carno – Arial image of Upper Carno drawn down with pumps and siphons shown to left hand side (Google Earth – modified)

Informed by ongoing studies by MMB, and while undertaking a Section 10 inspection under the Reservoirs Act 1975, the Inspecting Engineer instigated a need to control water levels within the reservoir to a minimum of 3m below Top Water Level (TWL) until all works had been completed.

To achieve the water level control with limited or no availability for use of the scour system over a prolonged two-year period, duty / assist siphons were also installed using Bauer pipes with ductile iron fittings. Siphons were feasible for durations of the scheme as their hydraulic operation could maintain the flow required and were operated when required. Given the duration of installation, the system was cost beneficial to purchase the materials and install siphons when compared with other means, including pumping. The materials were transferred to the Client upon completion of the works for their future use.

The siphon was pump-primed and designed to operate from 6m below TWL, with flow rates of up to 350l/s (175l/s each pipe) discharging to the spillway (Figure 4). For the majority of the construction period only one siphon was utilised and trimmed daily to maintain a balance with inlet flows. During storm conditions the second pipe was put into service and left running until the storm had abated or levels had reduced to normal. Initial commissioning showed some air ingress; however, this was solved by first using sealant on each of the Bauer joints and later, wrapping joints with plastic wrap.

The siphon was in place for approximately two years and required little maintenance, following commissioning, for the duration of the project. A small centrifugal pump was purchased to allow priming of the siphon (Figure 3). The use of a siphon here saved the need to pump up to 350l/s continuously for two years, providing a significant saving in terms of both cost and carbon.

Figure 3. Upper Carno siphon priming point

Figure 4. Upper Carno siphon discharge point.
Note flows from By-wash channel

USK RESERVOIR

Usk Reservoir is formed by an earth embankment dam, which completed constructed in 1955 with an approximate capacity of 12,268,000m^3. The dam is 480m in length, with a maximum height of 31m, and supplies raw water to Bryngwyn Water Treatment Works as well as providing compensation water to the River Usk, which is classified as a Special Area of Conservation (SAC) and a Site of Special Scientific Interest (SSSI). MMB was appointed to primarily replace the aged pipework within the dam tunnel due to its condition, while secondly maximising the drawdown capability through this tunnel.

To enable the replacement of the tunnel pipework, flows were required to be maintained by other means to the downstream watercourse for river regulation and downstream abstraction. A temporary solution was required to be designed to maintain compensation flows for the planned 18-month construction period. In addition, the possibility of needing to augment flows to the River Usk meant capacity of up to 50 MLD had to be catered for at all times of the year.

With no bywash available, the solution adopted at Usk was to use a twin pump-primed siphon system installed from the reservoir basin to the spillway's stilling basin (Figure 5). Each siphon consisted of 280m long, 400mm OD HDPE butt-fused welded pipework, with ductile iron fittings. The siphon was designed to operate down to 4m below top water level, which was typical for Usk Reservoir during the summer, as the water level fluctuates. The system used a pump to prime the siphon, and the pump could also be used in the event that the siphon loses prime to discharge the minimum compensation requirement. As the compensation was mandatory and to environmentally sensitive areas, a back-up system was installed in the event of a prolonged drawdown, with the added capacity of floating electric submersible pumps to add additional flows. The duration of installation was such that the siphon system was cost beneficial to purchase the materials and install siphons and back-up pump pipelines when compared with other means, including pumping. The materials were transferred to the Client upon completion of the works for their future use. The back-up pumps were hired from a pump supplier given their limited use.

As flows to the receiving watercourse were critical a duty / standby installation was installed so that should something untoward happen to the operating siphon a second pipe would be immediately available. Flow monitoring was installed with an automated alarm system activated should flows stop for any reason. A 24 hour call out was instigated with a view to returning flows as soon as possible, or within 2hrs after a notification. Upon installation, both pipes were pressure tested as for any permanent pipework. A battery-operated flow meter was installed to give a daily record of flows discharged. For the majority of time, flows were routinely discharged via the siphon methodology (Figure 6). The siphon operated successfully down to TWL-4m as designed, although auxiliary pumping was utilised to maintain flows to the River Usk following a prolonged dry spell in September 2021.

The siphon upstream leg was installed by Edwards Diving Services (EDS), with the crest and downstream sections, testing, commissioning and operation by MMB. The siphon was found to be reliable, and throughout the construction phase, saved approximately 260,000 litres of diesel, and 700 tonnes of CO_2, when compared to over-pumping.

Figure 5. Usk Reservoir siphon and over pumping intakes

Figure 6. Usk Reservoir siphon outfall

CWMTILLERY RESERVOIR

Cwmtillery reservoir is formed by a 15m high, 150m long earth embankment dam. The reservoir has a 148,000m^3 storage volume, 41,000m^2 surface area and a 2.83km^2 catchment area. The reservoir is situated north of Newport, South Wales. The reservoir's primary use is to supply the water treatment works (WTW) adjacent to the dam. In 2022, investigation works were undertaken into the spillway by MMB where the reservoir water level was required to be controlled. The existing drawoff arrangement has a combined scour and supply main and as such it was not possible to feed both raw water supply to the WTW and drawdown the reservoir using the scour at the same time.

With no bywash available, the decision was made for twin, pump-primed siphons to be installed to maintain water levels around 2m below top water level as investigations were undertaken in the spillway. The priming pipework from Upper Carno was re-used and installed within the spillway (Figure 7), however the number of bends within the spillway meant it was easier to deploy flexible wire armoured flanged pipework to complete the installation rather than utilising the previously deployed Bauer pipework for the siphons length.

Figure 7. Cwmtillery Reservoir – Siphon priming installation

The duration of was long enough to balance the cost with pump installation along with the cost saving of re-utilising some pipework from Upper Carno siphons. However, the installation was also short enough duration such that it was cost beneficial to hire the wire armoured flanged pipework from a pump supplier. The siphon upstream leg was installed by Edwards Diving Services (EDS), with the crest and downstream sections, testing, commissioning and operation by MMB.

The siphon operated well and needed little intervention other than trimming of flows following installation. Flow rates of up to 350l/s were routinely discharged with no pumping required, again saving on fuel costs and associated carbon.

GOUTHWAITE RESERVOIR

Gouthwaite impounds the River Nidd in North Yorkshire, by a composite dam. The dam is of masonry faced cyclopean concrete to the left-hand side (170m long, 15m high) and an earth embankment (165m, 12m high) to the right-hand side of the valley. The reservoir has a volume of 7.11Mm^3, a surface area of 134,000m^2 and a catchment area of 115.5km^2. The primary function of the reservoir is to provide riparian flow and flood protection to the downstream watercourse.

During the installation of permanent siphon pipework in 2023, it was necessary to close the existing scour valves due to water quality risks associated with silt bed movements. A floating pump arrangement was installed to provide compensation flows of up to 720l/s for the relatively short duration of the project. A duty/standby pumping arrangement was installed with power provided from duty/standby generators situated on the dam crest. The need to discharge substantial flows, potentially at water levels greater than 5m depth, coupled with the short downstream discharge pipe meant a siphon was less suitable and pumps more reliable for the statutory discharge required.

Figure 8. Gouthwaite Reservoir – Pumping pipework arrangement

CASTELL NOS RESERVOIR

Castell Nos reservoir is formed by a 100m wide, 12m high earth embankment dam. The reservoir has a volume of 91,000m³, a surface area of 20,000m² and a catchment area of approx. 8km².

To supplement the existing draw-down facilities (scour and siphon) with a cost beneficial solution, two additional HPPE siphons were installed over the spillway crest. To augment the flow required, twin siphons were installed, with the pipework of 400mm OD HDPE butt-fuse welded pipework, with ductile iron fittings. The siphon crest and downstream sections, testing, commissioning were by MMB. The upstream legs were installed in the reservoir basin utilising divers (Edwards Diving Services), who were also used along the spillway edge where it was secured into position with pipe straps. Issues with silt within the reservoir basin prevented commissioning initially. The silt was removed and the non-return valves placed such that silt would not affect their operation. Priming for this siphon is from an adjacent supply main which can be utilised with careful valving. This siphon now forms part of the permanent works and is routinely tested with all other reservoir safety critical valves.

Figure 9. Castell Nos – Twin siphons installed

CONCLUSIONS

Where water levels need to be controlled, it is always best to use existing assets to their full potential ahead of employing costly temporary measures. Where durations are short, pumping flows up to 300l/s may typically be suitable. For medium-term projects it may be decided to install a system utilising hired in pipework. For longer-term installations it may be beneficial to purchase the pipework, which may help guard the against cost over-runs and provide a usable asset that can be reused multiple times. Siphons typically only generally operate to approximately 6m below TWL and have a limited hydraulic flow range when compared with pumps. The commissioning and operation of siphons may require more management; however if used in an appropriate manner they can provide a reliable and cost-beneficial solution. If the discharge flow is of a critical nature it may be prudent to have a duty-stand by arrangement with pumps available.

The use of siphons should be encouraged where appropriate as they are typically cheaper to install and operate than over pumping, with the added benefit of removing fuel from site which can pose an environmental risk. Additionally, carbon savings can soon add up with a saving of around 2.6kg CO_2 for every 1 litre of diesel saved.

An understanding of the underwater topography and of the presence of silt and debris will assist in planning works. Good quality bathymetric, diver and ROV (remotely operated vehicle) surveys can prevent issues during installation and commissioning.

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The British Dam Society

Springwell Service Reservoir, managing and effectively mitigating ground risks in design and construction.

M EDMONDSON, Mott MacDonald Bentley
J TAYLOR, Mott MacDonald Bentley

SYNOPSIS Northumbrian Water Group appointed Mott MacDonald Bentley to design and construct a new 43ML service reservoir in Springwell, Gateshead to increase network capacity and resilience. The structure is 100m by 75m by 9m deep adopting a semi-precast concrete solution. It is founded largely on competent incompressible sandstone however, the southernmost third will encroach over significantly more compressible weathered rock. This presents a risk of early and long-term differential settlement that could impact reservoir structural integrity and potential safety if not adequately managed in design and construction.

Concept and Definition design by WSP involved extensive intrusive ground investigation work. Modern 3D digital geotechnical design tools (Leapfrog Works and Settle 3) have allowed designers to fully predict both immediate and future settlements of the structure. Initial assessments, based on first interpretation of borehole data, were beyond tolerable limits for practical and sustainable design of the structure requiring either alternative foundation solutions or ground improvement. To mitigate this risk a simple surcharging solution comprising temporary construction of an 8m high monitored surcharge bund, formed from site won materials, represented the most cost effective and sustainable solution.

Discussed are the geotechnical design processes and outputs through key design and construction phases: development of a representative 3D ground model; iterative 3D settlement analyses in collaboration with structural designers; design, implementation and monitoring of surcharging; back analysis of surcharge monitoring data to establish representative ground stiffness parameters for structural design; and validation of assumptions during construction.

INTRODUCTION

Northumbrian Water ('the client') identified a need for a new 43ML service reservoir (SR) to increase wholesome water supply network resilience in the South Tyneside area. The proposed site in Springwell, near Gateshead, Northumberland comprised a open grassed sloping field with an approximate 1 in 10 fall from north to south. Initial optioneering, outline design and early investigations and surveys were undertaken by WSP ('concept designer'). Following a competitive tendering process Mott MacDonald Bentley ('contractor') was appointed to undertake detailed design and construction of the project.

The contractor elected to design and construct a semi-precast concrete structure to allow construction completion within a very constrained delivery programme. Detailed structural design and supply of the structure was undertaken by FLI Precast Solutions ('sub-contractor'). Detailed design commenced in early 2023 with site construction commencing in May 2023.

The structure required a substantial temporary excavation of the sloping site. On completion the structure is to be fully landscaped to reduce visual impact on the local community. The initial investigation was up to 11m deep at its deepest to the north and was largely within competent Sandstone bedrock. The southern most third of the excavation however was within largely weathered Sandstone generally recovered as a residual Sand and Gravel.

The weathered extent of the formation strata presented distinct geotechnical design challenges due to the distinct relative differences in formation stiffness across the structure footprint. Namely, the greatest risk to the structure was short- and long-term differential settlement. This paper discusses the design approach by the contractor in close collaboration with the sub-contractor alongside the success of solutions implemented to reduce risks to tolerable structural design limits.

PROPOSED SERVICE RESERVOIR STRUCTURE AND GROUND LOADING

Form of Structure

The SR was to be a semi pre-cast DfMA solution comprising a combination of precast wall units, columns and roof beams with wall infills, base slab, and roof screed cast in-situ. Learnings from the contractor's previous experiences of similar structures were taken account of throughout this project (Aujla et al, 2021). The SR is split into two compartments (east and west) and has a total storage capacity of 43ML of wholesome water ready for customer supply. The key parameters of each compartment are listed in Table 1 below.

Table 1. SR Compartment Parameters

Parameter		Parameter	
Compartment Size (Internal)	49.0 x 72.5m	**Top of Roof Level**	141.70 – 140.98mAOD
Height of Wall Panels	8.75m	**Formation Level**	133.22 – 132.50mAOD
Internal Clear Height of SR	8.2m	**Base / Roof Slope**	1 in 100
No. of Compartments	2	**Top Water Level (TWL)**	139.00mAOD

Due to the method of construction adopted for the structure it inherently has a significant sensitivity to settlement and more critically differential settlement.

Structural Loads

Early initial structural design established the following loads would be applied to the ground during key loading stages through initial construction, testing and completion phases of the project:

- Wall loading following initial wall panel placement = 56kPa.
- General loading with SR cells full under leak testing = 100kPa
- Maximum finished loading to SR perimeter on completion of landscaping = 175kPa.

GROUND MODEL

Three phases of ground investigation (GI) were commissioned by the concept designer during outline design. The first phase of GI provided a broad understanding of the site's geology and identified a potential for Mudstone with a higher degree of weathering in the SW corner of the site. Two further phases of GI followed to focus on the weathering profile over the southern third of the SR. The difference in weathering profile is attributed to the site's natural topography and the relative difference in depth of excavation required to achieve the formation level for the SR.

A detailed 3D ground model was developed in detailed design based on all the available GI data across the site. The volume of available input data gave confidence that the ground model would be representative and reliable. Interrogation of the ground model identified that the SR formation strata would approximately comprise competent Sandstone over the northern most two thirds and largely Sandstone weathered to a residual Sand and Gravel to the southern third. Figure 1 illustrates a horizontal section cut at the structure formation level showing the general transition from competent to weathered rock with contours illustrating the general thickening of weathering to the South.

A band of weathered Mudstone (identified as a residual clay on borehole logs) was identified to underly both the weathered Sandstone and a thin band of competent Sandstone in the SW corner. It is considered that the Mudstone encountered that was logged as a residual Clay was significantly influenced by drilling with water flush opposed to natural in-situ weathering processes. This is interpretation is explored further in subsequent sections of this paper.

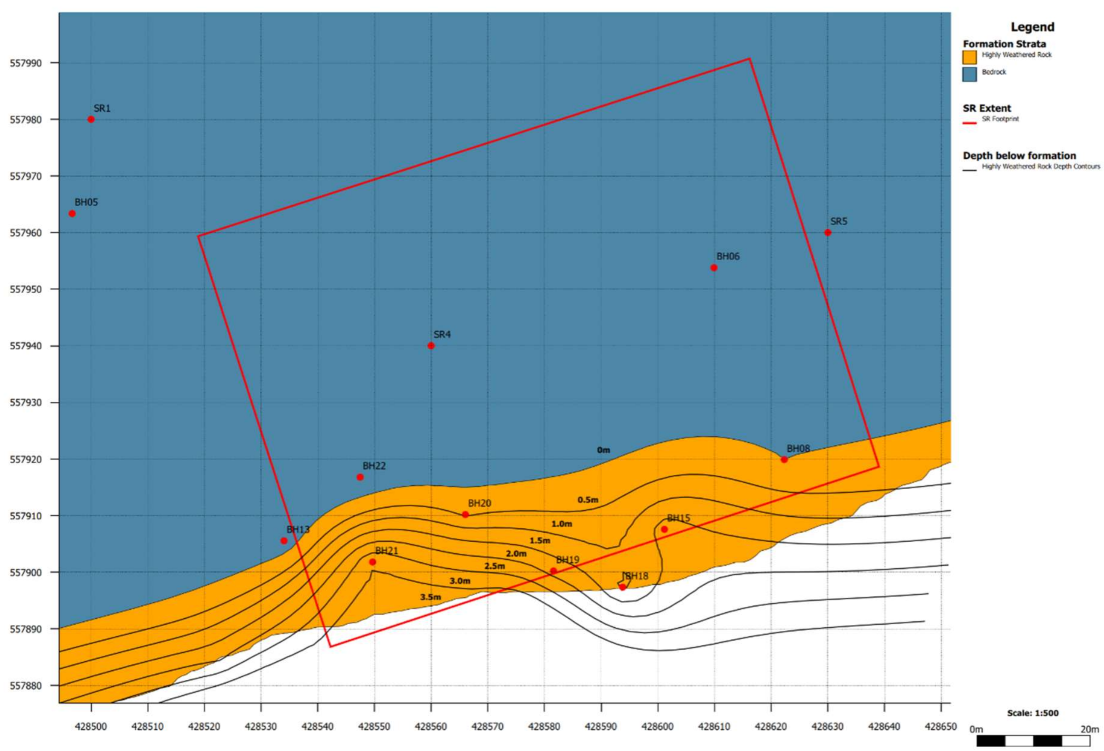

Figure 1: Formation strata; distribution of competent and weathered rock alongside interpreted thickness of weathered Sandstone

Table 2 summarises the initial geotechnical design parameters derived from the available in-situ and laboratory testing available from the various phases GI undertaken.

Table 2: Geotechnical Parameters

Geological Stratum	Geotechnical Parameter	Characteristic Design Value
Weathered Mudstone Firm to very stiff CLAY with low cobble content.	Unit weight, γ (kN/m^3)	18
	Drained Modulus of Elasticity (kPa)	10,000
	Coefficient of Volume Compressibility (m^2/MN)	0.10
Weathered Sandstone Medium dense SAND & GRAVEL.	Unit weight, γ (kN/m^3)	19
	Drained Modulus of Elasticity (kPa)	20,000
Mudstone Very weak to weak MUDSTONE.	Unit weight, γ (kN/m^3)	23
	Intact Rock – Young's Modulus (GPa)	1.7
	Rock Mass – Young's Modulus (kPa)	340,000
Sandstone Weak to medium strong SANDSTONE.	Unit weight, γ (kN/m^3)	23
	Intact Rock - Young's Modulus (GPa)	5
	Rock Mass – Young's Modulus (GPa)	1

INITIAL SETTLEMENT ANALYSES

Hand Calculations

Hand calculations were first undertaken to gain a basic understanding of the potential total and differential settlement of the SR. It was anticipated that settlement of the structure founded directly over competent rock would be minimal; however, settlement over weathered strata could exceed 55mm. Such potential differential settlement was generally considered intolerable for a semi-precast structure that was required to be watertight with a limiting crack width of 0.2mm. Due to the criticality of differential settlement more complex 3D settlement analyses were undertaken utilising Settle3 settlement design software.

3D Settlement Analyses

The 3D ground model developed was transposed into specialist 3D settlement analysis software adopting geotechnical design parameters as summarised in Table 2. This facilitated more complex and critical analyses of potential settlements across the structure based on the variability of the underlying ground conditions. This approach allowed the soil-structure interaction to be iteratively assessed. The approach established a representative worst credible output for which any appropriate mitigation measures that may be required could be considered.

Three key loading stages through to asset in service were established for analyses:

- Initial loading from precast wall units when placed on setting-out strips (temporary foundations).
- First filling of reservoir cells during water testing.
- Construction of landscape fill with reservoir fully loaded and in service.

Initial analyses indicated that settlements over the competent Sandstone could be in the order of 1-5mm whilst over the weathered rock to the southernmost extent of the structure could exceed 50mm (Figure 2). Given the very defined zone of weathered rock to the south, this would result in a very concentrated change of deflection and settlement. The potentially sudden transition may result in an abrupt angular distortion of settlement within the structure that was intolerable for structural design. This therefore required a different foundation solution, or ground improvement was required.

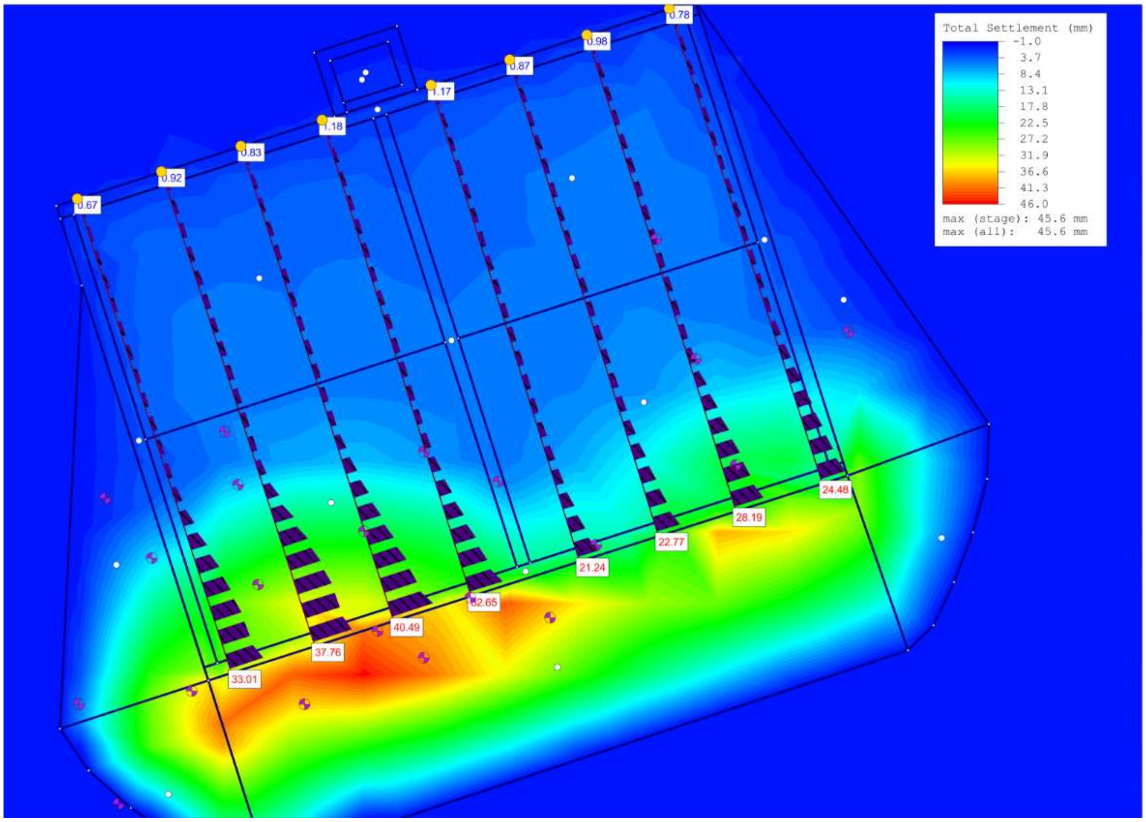

Figure 2: Settle3 preliminary settlement assessment

SETTLEMENT MITIGATION OPTIONS

Options to mitigate potentially excessive settlements included: a) excavate and replace with known compacted fill to competent rock; b) excavate and replace with mass fill concrete; c) piled foundations or d) ground improvement.

Of the options considered an opportunity was identified in the construction programme allowing a simple surcharging solution negating the need for alternative deep soil improvement techniques. A surcharging solution was pursued with the advantage this also returned the lowest embodied carbon option of those under consideration; the solution was implemented utilising freely available site won arisings.

GROUND IMPROVEMENT BY SURCHARGING

Surcharge requirements by analysis

On site there was a significant volume of available arisings to be excavated to achieve formation level of the structure, this meant there was an abundance of excess material available to consider a simple surcharging option. It was established that following initial excavation works there was a three-month period, prior to first delivery of structural elements, to facilitate surcharging of the site.

The established 3D settlement model was used to design the extent and size of surcharge bund required to reduce future ground settlements to a tolerable level for the structure. The design compared variations in the required surcharge bund height versus available surcharge timescales. It was established that application of a surcharge load of 144kPa (equating to an equivalent bund height of 8m) for a period of three months would induce a similar magnitude of settlement to that of the permanent in-service structure over its design life. Figure 3 illustrates predicted settlements resultant from the surcharge bund over the southernmost extent of the structure with settlement in the range of 29-50mm. Surcharging would remove a significant proportion of the likely settlement prior to construction of the SR. The magnitude of potential differential settlement would be reduced to tolerable structural design limits and minimise structural reinforcement requirements.

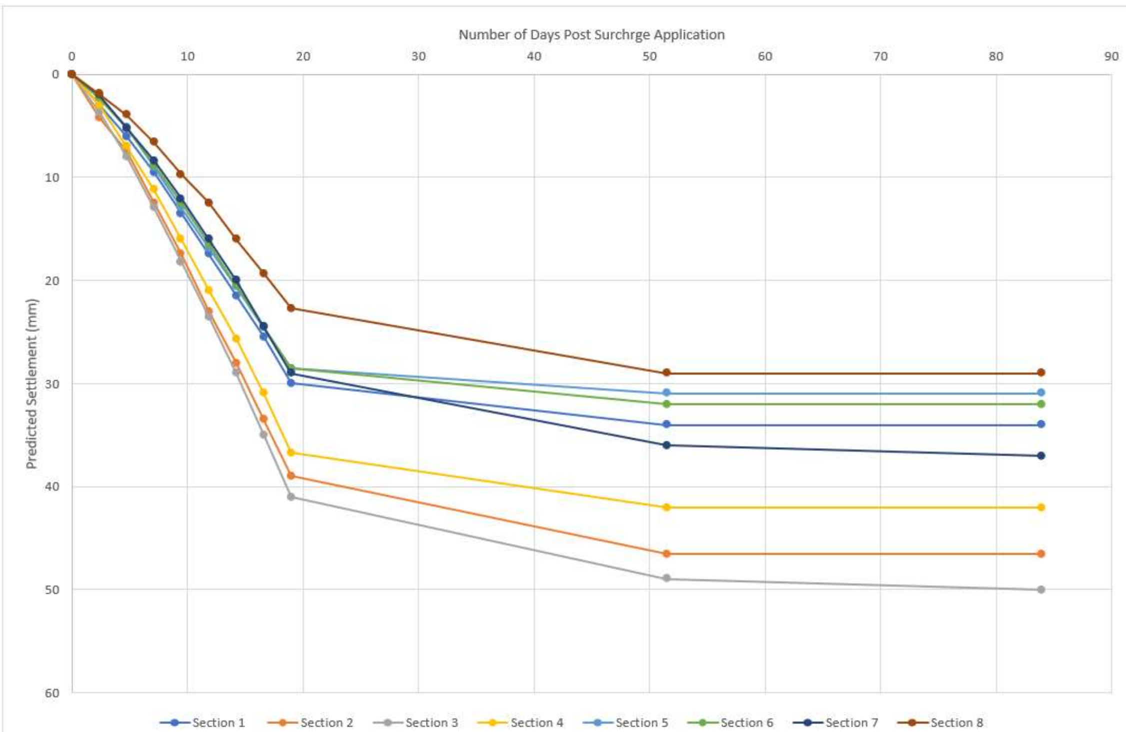

Figure 3: Expected Settlement Induced from Surcharge Bund (Section 1 = West, Section 8 = East)

Surcharge Bund Construction

The surcharge bund was constructed using as dug material comprising a combination of Glacial Till and weathered Sandstone to a height of 8m (Figure 4). The top of the surcharge bund extended 16m into the SR footprint on the western wall and 2m on the eastern wall to apply loading to the full extent of weathered strata. The approximate extent of the surcharge bund is shown in blue in Figure 5.

Figure 4 Constructed surcharge bund

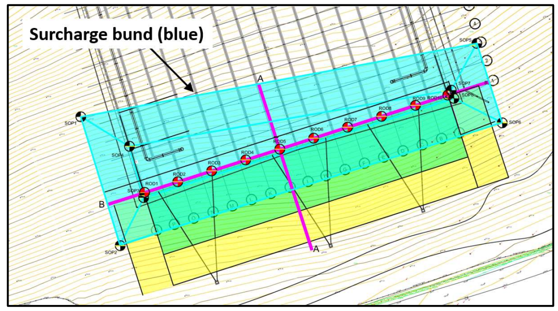

Figure 5: Extent of surcharge bund

Settlement Monitoring of Surcharge Bund

To facilitate settlement monitoring isolated to the underside of the surcharge bund, excluding potential consolidation settlement within the bund itself, eight rod settlements gauges (RSGs) were installed prior to bund construction. The RSGs were aligned with the southern wall of the structure (Figure 5). The RSGs comprised a 300mm² base plate, 1m steel vertical extension rods and a protective plastic surround to isolate monitored movement to the base plate. RSGs were embedded in a fine sand surround to mitigate potential for disturbance during construction. The sand surround measure, however, was of limited success on this occasion and loss of verticality was observed, largely due to the significant plant size adopted during construction; this is discussed further below.

During bund construction and throughout the planned surcharge period RSG elevation readings were taken by site engineers. The cumulative change compared to initial baseline readings were continuously reviewed by design engineers.

Observed Settlement

Figure 6 presents observed settlement for each of the RSG's. Notably, there is an apparent 'rebound' in the readings following initial construction at day 20. This apparent rebound is a result of mathematical adjustment of settlement data to account for loss of RSG verticality that was induced by heavy construction plant constructing the bund.

Figure 6 illustrates a pronounced initial steep increase in settlement during the construction and progressive raising of the surcharge bund. Whilst data from a limited number of RSGs (Section 5 and 6) suggest an initial 'heave' this is attributed to the selected monitoring instrumentation that was rapidly replaced by a precise level monitoring instrument with a +/- 1mm accuracy; it is not considered that the underlying ground 'heaved'. Some of this observed heave, however, could be linked to RSG disturbance whilst placing fill materials.

Beyond the construction period it is observed that no further discernible settlement occurs (Figure 6). This observation confirmed that settlement of the underlying strata was limited to immediate settlement with little evidence of further consolidation settlement. This observed behaviour gave confidence that the reported weathered Mudstone, recorded as residual Clay, was more likely simply a drilling induced phenomenon opposed to an in-situ condition and likely long-term material behaviour. As such, risk of future consolidation settlement of the permanent structure could be discounted.

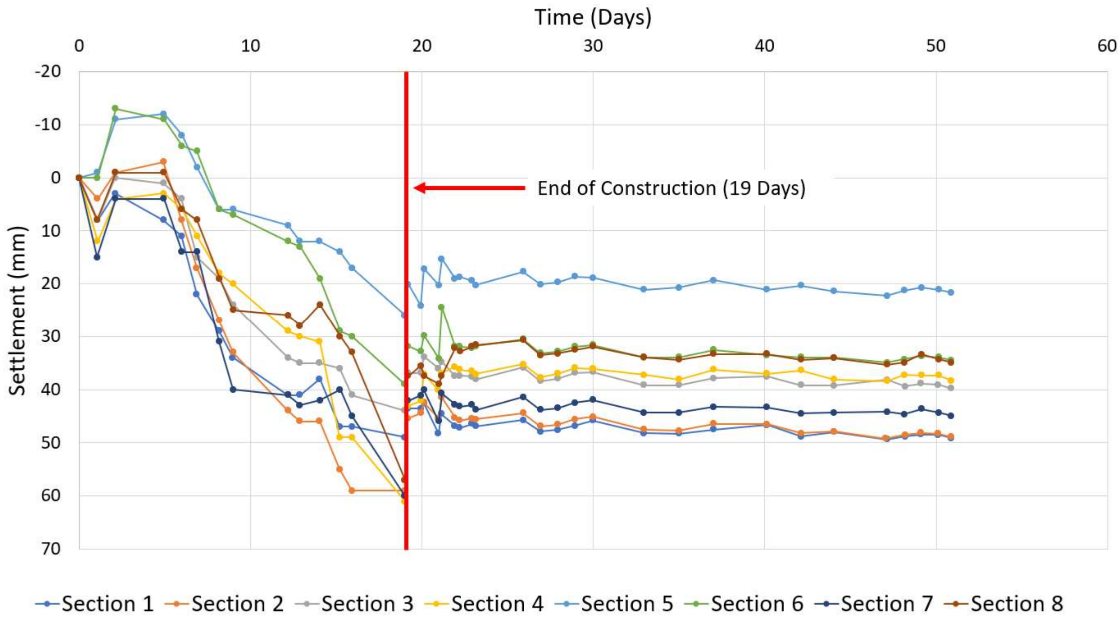

Figure 6: RSG Settlement Monitoring

The settlement of each RSG against that predicted from the initial settlement analyses to determine the size of bund required and surcharge timescales (Figure 3) is summarised in Table 3. The observed settlements for the surcharge bund were in the range of 20mm-47mm; this is comparable with the initially predicted magnitude of settlement. Overall, the agreement between actual and predicted settlements was favourable, with observed variations primarily attributed to inherent differences between interpreted and actual ground conditions. It is however, notable that the predicted total settlement was realised within 51 days, considerably sooner than anticipated. Figure 7 illustrates the predicted and actual settlement profiles for RSG 2, illustrating a very close alignment between predicted and observed.

Table 3: Predicted versus observed settlement

RSG Number	Predicted Settlement from 3 Months of Surcharging (mm)	Actual Settlement at 51 days (<3 months) of Surcharging (mm)	Difference Between Actual and Expected (mm)	Percentage of Predicted Settlement Achieved (%)
RSG 1	34	47	+13	138
RSG 2	46.5	46	-0.5	99
RSG 3	50	38	-12	76
RSG 4	42	38	-4	90
RSG 5	31	20	-11	65
RSG 6	32	33	+1	103
RSG 7	37	43	+6	116
RSG 8	29	34	+5	117

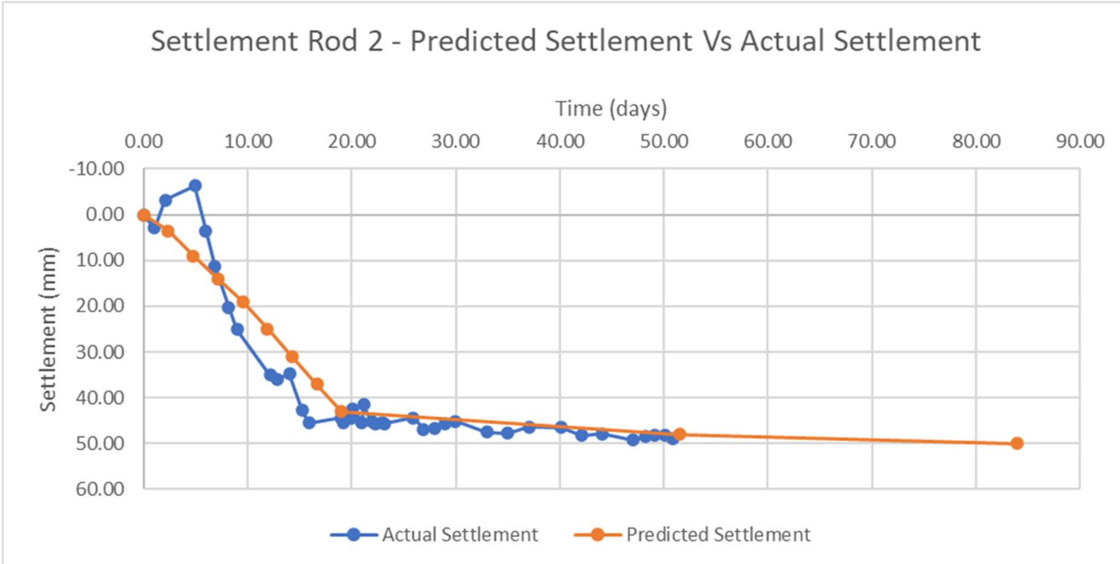

Figure 7: Example actual settlement vs predicted settlement for RSG2

Surcharge Review & Removal

It was observed that actual and predicted settlement magnitudes were comparable and further that settlement had plateaued and was not showing evidence of ongoing consolidation settlement. Furthermore, observed settlements were comparable of greater than that predicted for the permanent structure. It was therefore concluded after a period of 51 days, some 40 days earlier than was predicted, that sufficient settlement had occurred and that the surcharge bund could be removed early; effectively saving a month on the overall construction programme.

There was some concern that on removal of the surcharge bund the underlying strata could partially heave due to elastic rebound. As such, RSGs were carefully monitored during removal of the bund, however no particular rebound was observed during deconstruction.

Overall, the surcharge bund surpassed expectations by achieving the desired results in less time than predicted. Furthermore, with the actual settlements being very close to that predicted this gave additional confidence that the developed ground model was a reasonably accurate reflection of true ground conditions.

ACCURATE IDENTIFICATION OF THE POINT OF TRANSITION BETWEEN COMPETANT AND WEATHERED ROCK

After the removal of the surcharge bund it was important to accurately locate the transition between competent and weathered rock such that a number of bespoke SR wall panel units could be placed to span this transition. Trial trenches were located based on the 3D ground model and were excavated under supervision of a Geotechnical Engineer. Figure 8 illustrates the inspection trenches employed to pinpoint the actual transition location. On identifying the point of transition this was recorded by site engineers to allow wall panels to be accurately located during future construction.

Figure 8: Transition from competent to weathered rock inspection trenches

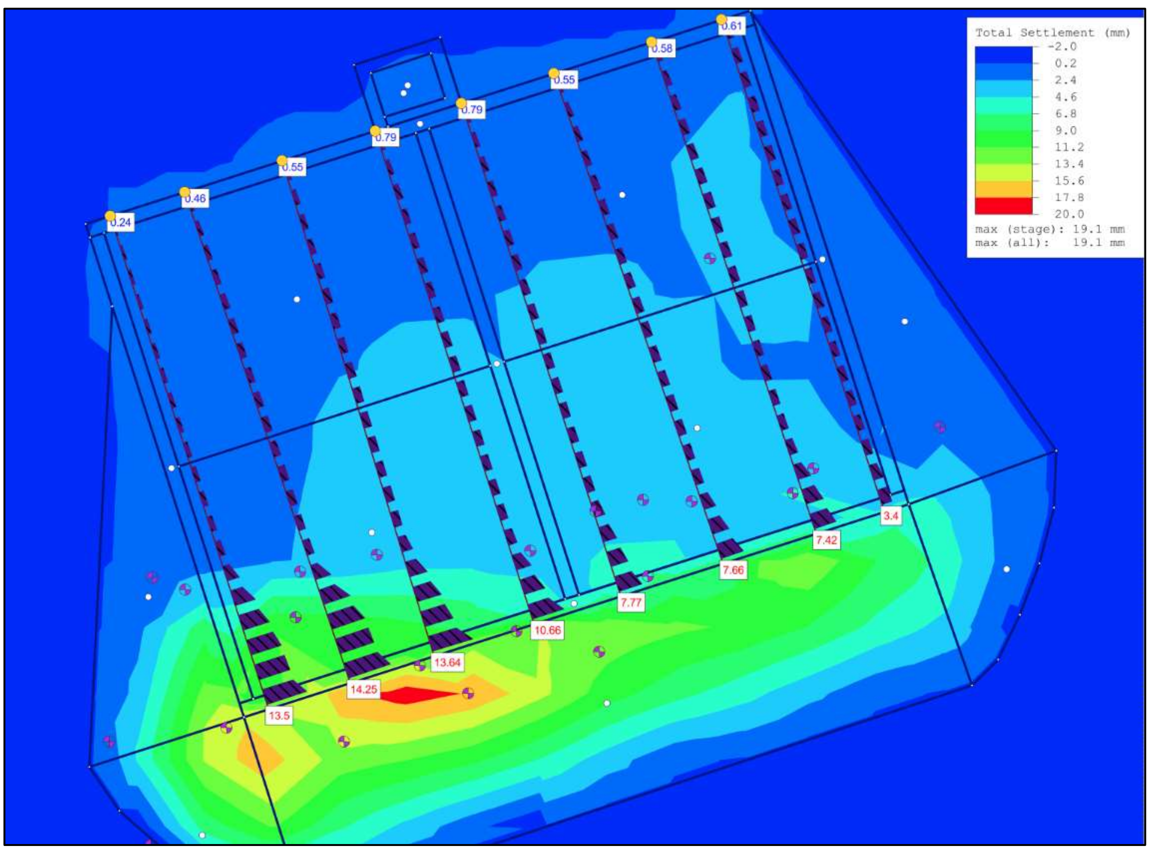

Figure 9: Settle3 Extract Showing Residual Settlement After Surcharging

BACK ANALYSIS OF SURCHARGE MONITORING DATA

The Settle3 settlement model was revised to reflect the accurate position of the transition from weathered to competent rock. Following revision to the ground model the post surcharge model was used to output revised representative modulus of subgrade reaction (ground stiffness) design parameters for the improved consolidated ground. Revised design parameters were adopted in structural finite element design to determine what, if any, long term settlement might be realised by the structure. Figure 9 illustrates the predicted long-term settlement based on worst credible structural loading. The maximum predicted future settlement directly below the structure is in the order of 14mm. Also, the likely maximum angle of distortion (differential settlement) over the weathered rock exceeds 1:1,000 which was deemed acceptable.

CONSTRUCTION SETTLEMENT MONITORING

As previously discussed, three significant loading cases are expected to induce the largest magnitude of settlement: landing wall units; water testing and backfilling. Throughout construction settlements of the structure will be monitored at these key stages. Monitoring positions will be set up on the setting-out strip, the external face of the walls, and the top of the walls at locations around the site. Baseline readings will be taken before any load is applied allowing for the calculation of cumulative settlement.

Construction to date has largely been over the identified competent rock and only marginally encroaching on the identified weathered zone. Fi gure 10 illustrates the extent of progress to date (May 2024). Observed settlements from site monitoring have consistently been below that predicted. This alignment between observed settlement and predicted behaviour instils further confidence in the accuracy of the ground model in Settle 3.

Figure 10: Overview Photo of the Site (taken on 09/05/2024)

PRECAUTIONS DURING WATER TIGHTNESS TESTING

When such structures are subject to water tightness testing (commonly referred to as a drop test) it is normal that one cell is initially filled and tested with water then pumped to other cells to test each cell individually. On this site however, there remains a low residual risk of differential settlements inducing excessive cracking over the transition between weathered and competent rock. It is unusual for such a structure to be constructed over strata with such significantly contrasting relative stiffnesses. To best mitigate the risk of excessive cracking being induced during testing it is proposed to take a different approach to initial filling and the resultant significant first ground loading.

For this structure it is planned that first filling for testing will introduce water into both cells simultaneously to 50% of the capacity of each cell. This will in effect allow even load distribution and significantly reduce the risks of differential settlements between the two cells. At this point the water in the west cell will be transferred to the east cell (lesser expected settlement magnitudes) and the east cell will then be fully tested. The water will then all be transferred to the west cell and this cell fully tested. In doing this it will in essence avoid 'shock' loading either SR cell and significantly reduce the risk of differential settlement between the cells. Close monitoring of settlement will be undertaken throughout this stage of work such that, if required, further measures can be implemented to avoid excessive structural distress.

SUMMARY AND CONCLUSIONS

Prevailing ground conditions beneath the proposed structure represented a significant geotechnical challenge owing to the stark contrast in relative ground stiffness's resulting in a significant risk of excessive structural settlements. To reduce this risk extensive ground truthing, detailed ground modelling and 3D settlement analyses were undertaken. It was ultimately concluded that sufficient ground improvement could be achieved by surcharging the site to induce potential future settlement early in construction.

Construction and monitoring of a significant but simple surcharge bund have removed the risk of initially intolerable predicted structural settlements. This has allowed the SR to be designed and constructed on a shallow reinforced pad foundation instead of a potentially more costly and carbon intense alternative foundation solution.

Borehole data suggested that Mudstone units weathered to a residual Clay may be present that could result in a long-term consolidation settlement risk. Observations from settlement monitoring provided evidence that underlying strata was largely granular in behaviour with no evidence of potential for long term consolidation settlement. It was concluded that the Mudstone was disturbed during drilling with water flush. Interpretation of factual data should not simply be taken on face value; experience, judgement and further proving should be applied such that over-conservatism does not creep into design.

Construction activities and continued monitoring to date has confirmed observed structural settlements less than predicted.

A residual risk of inducing potential differential settlement during first filling of the SR for water tightness testing was identified. To best reduce this risk it is planned to fill the individual SR cells concurrently during first filling to 50% of their individual capacity; this is generally not an industry-followed procedure. This methodology will in effect smooth initial structural differential settlement between the individual cells. It is recommended that this procedure be adopted as industry good practice for future such structures.

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The British Dam Society

Recent underwater geomembranes solutions for dams and canals

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SYNOPSIS Underwater geomembrane technologies to stop or minimise leakage and grant safe and efficient operation started being adopted on dams in 1997, and have since been used to repair the full face of the dams, specific leaking areas, or failing joints. In the 2010s the combined expertise in waterproofing dams underwater and canals in the dry led to the development of a patented geomembrane system for underwater repair of canals in flowing water. Sibelonmat® is a watertight, factory prefabricated mattress formed by two geomembranes which are interconnected to form a void space, deployed underwater on site, to line the entire cross section of the canal or parts of it, and joined underwater to the adjacent mattresses by heavy duty watertight zips. The mattresses are then filled with cementitious grout to permanently ballast the bottom geomembrane that provides watertightness, while the top geomembrane confines the grout. This paper presents the state-of-the-art technologies in still water and in flowing water through two recent underwater projects: Studena, a 55m high buttress dam in Bulgaria, and the Kembs embankment, part of the Grand Canal d'Alsace navigation waterway in France.

INTRODUCTION

Ageing of hydraulic structures is almost always associated with decreased watertightness, which over time may jeopardise the efficiency of the structure, and ultimately its safety. To allow the structure to continue operating safely and efficiently, seepage must be stopped or minimised. Geomembrane systems are a proven method to restore watertightness in ageing dams since the beginning of the 1960s.

Until the early 1990s, to install a geomembrane system the dam had to be dewatered, which is sometimes impossible, or possible at unacceptable financial and/or operational, environmental, and social costs. Research carried out in the years 1995 and 1996 led to the development of a Carpi geomembrane system that could be installed underwater without impacting on the operation of the reservoir, and was followed in 1997 by a real project on a dam in USA, presented at a BDS Conference (Scuero et al, 2000). Many underwater projects have been completed since then, and different systems have been used depending on the extent of the areas to be waterproofed, i.e. the whole upstream face of the dam, or one or more areas where unacceptable leakage had been detected, or local damages (failing joints, cracks, holes). All such systems have in common the fact that the geomembrane has been installed at the upstream face of the dam, in still water conditions.

A new challenge came at the start of the 2010s, when the issue of ageing canals was addressed. Ageing of canals always entails decreased watertightness, hence loss of water, and at times reduced flow, which is magnified by the often-long path from the source to the users. Many studies have been performed on water losses in unlined and lined canals, especially for irrigation canals (Giroud and Plusquellec, 2017; Plusquellec, 2019). Other studies conducted across the globe have shown different values for the average amount of water losses, depending on the regions; overall it can be said that the water loss can exceed 35%. To reduce water loss, unlined canals must be lined, and lined canals where the lining has deteriorated must be repaired. For durable repair, the canal must be dewatered, which can be unfeasible, or feasible only at unacceptable inconveniences and costs. Since the underwater technologies then available were not applicable in most canals, because divers can safely operate only in still or almost-still water (water velocity less than 0.5 m/s), the objective was to develop a geomembrane system that could be installed underwater with the canal in full operation.

The geomembrane system for canals in full operation is the outcome combining the experience gained in underwater projects in dams, the lessons learned in projects executed in dewatered canals, and the knowledge acquired in studies and testing executed in flowing water conditions. The resulting system, Sibelonmat®, has been adopted in three pilot projects in canals. The paper presents the research, the solutions, and the two most recent applications, carried out by Carpi for underwater geomembrane systems in still and in flowing water conditions.

UNDERWATER GEOMEMBRANE SYSTEMS IN DAMS

Advantages and peculiarities of underwater geomembrane systems

When design is adequate and installation is carried out and controlled in a proper way, the quality of a geomembrane system installed on a dam underwater is comparable to that of a system installed in the dry, hence the technical assets are the same: capability of granting long-term safety and efficiency, because geomembranes are practically watertight and maintain watertightness over time, have no defective joints or cracks through which water can infiltrate, and, furthermore, can accommodate settlement, differential displacement, opening of joints, and opening of new cracks, thanks to the tensile properties that allow an elongation largely exceeding that of other traditional remedial measures.

Underwater installations on dams must on the other hand consider the almost always poor visibility, the need of limiting the time of each dive when diving in deep water, and the increased security/safety measures. While any diving depth can be attained, if depth regularly exceeds 50m, saturation diving and therefore a decompression chamber permanently in operation are required. Consequently, underwater works do not proceed as quickly as dry works, and they are more expensive for obvious reasons.

Performing underwater works can be a necessity, or a choice based on the evaluation of the costs, not only financial, of dewatering, and of the benefits deriving from continuing operating the dam, which in hydropower dams means revenues that can balance the higher underwater costs. The extent of the underwater works is another choice to be made: full-face underwater repair minimises the possibility of leakage coming from any unlined upstream portions of the dam, but may be unpractical, especially when large surfaces, great depths, and high diving costs, are at stake, or when leakage comes from a relatively small portion of the dam. The solutions can be to identify the areas leaking most and select the surface to be lined which maximises the benefit of the geomembrane system, or to line the dam in separate stages planned to meet the operational or budgetary needs of the owner. Bulletin 135 (ICOLD, 2010) discusses underwater issues and presents the applications existing at that time. Since 2010, projects of underwater repairs of failing joints and leaking areas have adopted the same or similar systems. An outstanding example is Llyn Teifi spillway in UK, a very demanding project due to the extremely complex geometry, with multiple convex and concave corners in very narrow spaces. The exposed geomembrane waterproofs the concrete spillway and the southern wing wall, extending beyond the two leaking side joints of the spillway, and downstream over the ogee weir, to cover the horizontal joint between the spillway and the new concrete constructed downstream.c The project was completed in 2016; performance data from the owner, dated October 2019, testify to the continuous good performance of the system.

Underwater installation at Studena, Bulgaria 2018

Studena, a 55m high and 259m long buttress dam in Bulgaria, crest elevation 845m, is a recent example of an underwater repair adapted to maximise the efficiency of the geomembrane system while meeting the operational needs of the owner. It is also the first example of an improved tensioning system for underwater installation, as described below.

Studena is a multipurpose dam used for potable and industrial water supply, for irrigation, for regulating the water of the Struma River and its tributaries, for protecting the arable land and settlements downstream of the dam against floods and for power production. The dam is located in the European-Continental climate zone, in a mountainous climate region where snowfalls begin in mid-October. The snow cover persists from 100 to 200 days depending on the altitude, and the snow depth can be from 1m to 3.4m. There are frequent ice formation and freeze/thaw cycles, and this harsh climate required protecting the concrete with a shotcrete layer. Nevertheless, after about 50 years of operation the dam and its appurtenant structures were badly deteriorated, with blistered shotcrete no longer attached to the concrete (Figure 1), cracks on working joints, vertical cracks, and damaged structure of the concrete, visible in the zones where the shotcrete was detaching. An inspection carried out by experts ascertained that water was penetrating the dam through damaged expansion joints that needed repair, and that the clogging of drain holes and piezometers in the gallery made it impossible to obtain true information about seepage at the dam and about the water level rise along the wall-foundation contact.

Although the dam wall was stable and no significant leakage seemed to be occurring, given the importance of the structure and to prevent a critical situation that could later threaten water supply and require more expensive works, the Bulgarian Government decided to implement a complete rehabilitation project to extend the functional life of the dam by at least 50 years, ensuring water supply and safety of the structure, which is in a seismically active region. The project, financed by the World Bank, had as its most relevant part the rehabilitation works to protect the dam concrete. A tender for the dam rehabilitation was issued by the Ministry of Regional Development and Public Works under the World Bank rules. The tender required as waterproof protection liner a 2.5mm thick polyvinylchloride (PVC) geomembrane heat-bonded at fabrication to a 500g/m^2 nonwoven needle punched anti-puncture geotextile, to be placed on a 2,000g/m^2 cushion geotextile protecting the liner against excessively aggressive rough areas. The geomembrane had to be secured to the upstream face with stainless-steel vertical steel shapes and components, clamps and anchors secured to the dam body. The geomembrane system had to be drained, with seepage water discharging into the gallery via two transverse pipes; acceptance criteria were seepage not exceeding 0.9 l/s for the whole dam, or 0.5 l/s for one drainage pipe to the gallery.

The works had to be carried out in such conditions as to guarantee the safety and the proper technical operation of the dam and of its appurtenant structures while providing continuous water supply, which meant that most of the works had to be carried out underwater, and without affecting the quality of the supplied water. During the tender procedure the decision was taken not to extend the waterproofing system down to the entire damaged area (elevation 802m), to avoid working in the sediment layer and creating turbidity. The waterproofing geomembrane system was installed from elevation 843.3m to elevation 814.0m, with underwater works from elevation 838m downwards (Figure 2).

Figure 1. Deteriorated upstream face at Studena dam

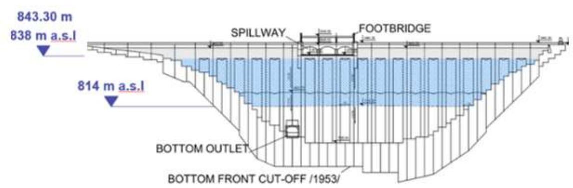

Figure 2. In grey, area lined in the dry, in blue, area lined underwater

The tender was awarded to a consortium of companies. Our company designed and installed the geomembrane system. As required by the specifications, the waterproofing liner, Sibelon® CNT 3750, is a 2.5mm thick plasticised PVC geomembrane heat-bonded to a 500g/m^2 non-woven needle punched polypropylene geotextile. The waterproofing liner has a drainage system behind, consisting of the drainage gap created by the anchorage system between the waterproofing liner and the dam face, and of a bottom drainage collector consisting of a longitudinal 500mm high band of a highly transmissive drainage composite formed by a cuspated drainage geonet thermally bonded on both sides to a non-woven polypropylene geotextile acting as a filtering layer to avoid clogging of the geonet. Two discharge holes drilled from the gallery to the upstream face, equipped with discharge pipes with a valve at the downstream end and with an upstream anti-intrusion stainless-steel plate, and four ventilation pipes at crest, to balance the air pressure beneath the waterproofing geomembrane in case of sudden changes in the atmospheric pressure, complete the drainage system.

The complex geometry of the dam required a complex face anchorage system, comprising (Figure 3) tensioning profiles (1) in the convex corners, point anchors (2) in the triangular recesses in the buttresses, batten strips (3) in all concave corners, and mechanical peripheral seals watertight against water under pressure (4) at the top and bottom peripheries of the sealing system. All fastening components are stainless-steel.

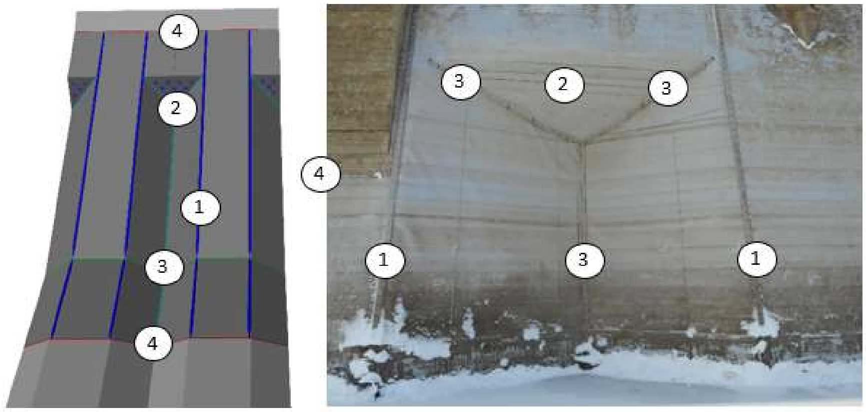

Figure 3. Tensioning profiles (1, in blue), point anchors (2, in black), batten strips (3, in green), and perimeter seals (4, in red)

An efficient tensioning system was essential in view of the heavy snow and ice formation that could exert a dragging effect on the waterproofing liner if it were not perfectly tensioned. The underwater tensioning system used previous projects and presented in ICOLD Bulletin 135 was modified to improve the tensioning effect. The new tensioning profiles (1), intrinsically watertight, are a patented development of the system adopted in previous installations: while previously the tensioning effect was essentially achieved by the installation procedure, at Studena the tensioning effect is achieved by the geometry of the two profiles, as with the tensioning profiles used in dry installation. The point anchors (2) at the triangular recesses consist of long shaft anchors with a thick steel washer to distribute the uplift forces transmitted to the geomembrane liner by wind. Anchors are made watertight by a SIBELON® C 3250 geomembrane washer (the same geomembrane that composes the waterproofing liner, but without geotextile) placed on top of the steel washer and heat-seamed to the underlying waterproofing liner, which was possible because installation of such anchors was carried out in the dry. The batten strips (3) and perimeter seals (4) are made intrinsically watertight: the batten strips using two flat profiles and suitable gaskets to evenly distribute the compression that achieves watertightness, the perimeter seals spreading a resin bedding on the concrete to create a smooth surface and remove possible voids where the seal is placed, and using a rubber gasket under the profiles and splice plates at abutting profiles.

Works started in August 2017 with the civil works related to surface preparation, immediately followed by installation of the geomembrane system in the dry, which was completed by the inset of autumn 2017. The crew remained at site to prefabricate the 4m wide panels that would make underwater installation quicker, and to provide supervision for the underwater works. To minimise the amount, and consequently the costs, of such works, installation had to be carried out in the period of low water levels, which was a major challenge because it coincided with the coldest months, November through to February. In such months, diving often required breaking the ice in the reservoir. The maximum diving depth was 28m, i.e., 8m more than the contractual 20m depth.

The first underwater installation tasks were related to the drainage system: drilling the holes for drainage discharge; watertight fixing of a steel plate over the upstream area of the hole to prevent water flowing into the hole during drilling from the downstream; installing and fixing the discharge pipes; the anti-intrusion plates and a drainage band at the upstream side. The major surface preparation works consisted of the removal of the unbonded shotcrete by diamond saw cutting and high-pressure water jetting, followed by cleaning and by levelling of the roughest concrete with mortar. The rough concrete was covered by a 2000g/m^2 non-woven needle punched polypropylene anti-puncture geotextile fastened with impact anchors (Figure 4). Along the anchorage lines, the concrete surface was levelled with mortar. The same steps were performed underwater, where levelling of the rough concrete was made with resin. The geomembrane sheets/panels were then deployed on the upstream face (Figure 5) and connected and fixed with the Carpi patented face anchorage system described.

Figure 4. The 2000 g/m^2 non-woven geotextile is installed on the concrete after removal of the unbonded shotcrete, in the dry and underwater

Figure 5. Unrolling of a geomembrane sheet underwater, over the 2000 g/m^2 non-woven geotextile

Figures 6 and 7 are related to the tensioning profiles: the first profile is fastened to the concrete with mechanical anchors; the edges of two geomembrane sheets are overlapped at the profile; and the second Omega-shaped profile is positioned over the first one and tightly connected to it, thus forcing the waterproofing liner sheets into a new position that results in a tensioning effect.

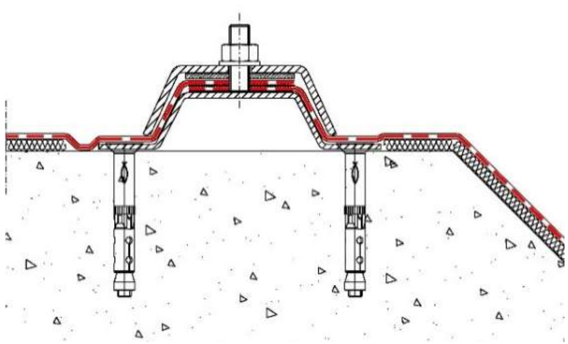

Figure 6. Scheme of the underwater tensioning profiles at Studena dam

Figure 7. Diver connecting the two tensioning profiles underwater

At completion of underwater works, the crew horizontally overlapped the geomembrane installed above water over the one installed underwater by the divers and welded on the overlapping a horizontal geomembrane cover strip, to make a watertight junction between the two geomembranes. Waterproofing works were completed within schedule despite difficult climatic conditions, on 27 December 2018. The total surface lined was 5,498m^2, of which 1,348m^2 was above water and 4,150m^2 under water. Total leakage from the geomembrane system, compliant with tender requirements, is shown in Figure 8.

Figure 8. The works during a low water level period, and total leakage from the two drains

AN UNDERWATER GEOMEMBRANE SYSTEM IN FLOWING WATER CONDITIONS

Development of a new solution

Lining underwater a canal that is in full operation poses a critical challenge: the flowing water. Water in motion requires a robust anchorage system to keep the geomembrane stable, not only against the water flow, but mainly against uplift. Uplift can be caused by wind when the canal is empty, but higher uplift may occur if accidental damage in the geomembrane allows flowing water to infiltrate behind it. Many anchorage lines are needed to resist the uplift, in addition to those needed to join adjacent geomembrane sheets. Diving times become very long, costs increase, and the water speed may even impede diving and require outage.

The obvious answer to the problem was to change the anchorage concept, and instead of anchorage by lines conceive a system where the waterproof liner was incorporated into the anchoring system. Composite mattresses incorporating a watertight layer were already available in the industry, basically consisting of two textile layers, either containing a bentonite mixture in powder or granules, or confining a cement grout injected at site; the wetted bentonite, or the thickness and cement content of the grout, provided the watertightness and at the same time anchorage by ballast. These mattresses, however, have several drawbacks: to the knowledge of the authors, there is no experience of bentonite mattresses installed in flowing water, and they require a dead weight confining the bentonite so that the bentonite expansive reaction can be activated. Grouted mattresses can in fact be installed in a wet environment, but if grouting is not carried out continuously cold joints will form, the inevitable shrinkage of the grout will create cracks, and through cracks watertightness will be lost. Both types of mattresses entail the risk of water pollution by leaching cement components; the watertightness of the joints between adjacent mattresses is questionable; the connection to concrete appurtenances is tricky and not reliable in the long term, and they are prone to cracking if settlement occurs.

The solution was to create a mattress whose watertightness would be granted not by the material inside the mattress, but by a robust watertight geomembrane of the same type that has been successfully performing in canals for decades. The geomembrane has proven to be able to resist the rough subgrade of deteriorated canals, to be sealed watertight underwater to concrete appurtenances, and to resist differential displacements and settlements. The new patented mattress is formed by such a membrane, and by an impermeable system confining the inexpensive grout that is injected at site providing the required ballast without any risk of water pollution. The device that allows watertight joining of adjacent mattresses underwater was developed jointly with one of the leading zipper manufacturers in the world, and is an impermeable heavy-duty zip, integrated at fabrication to the mattress in a flexible way that allows it to adapt to the irregularities of the canal, compensating for possible misalignments between mattresses. The zip is generally pulled underwater using custom-designed equipment and unmanned procedures, while divers are employed when the water speed is low, for underwater control if needed.

The new mattress has been adopted for three pilot projects in canals, in Egypt and Italy for irrigation, and most recently in the Grand Canal d'Alsace navigation waterway in France, the subject of the following case study.

The Kembs embankment, France 2020

Kembs embankment is part of the Grand Canal d'Alsace, a 150m wide and 52km long navigation canal from Kembs to Vogelgrun, in the eastern part of France, whose construction started in 1932. Managed by EDF, Electricité de France, the canal started operating in 1959, and over decades of service deteriorated, with leakage occurring. EDF-CIH, the Centre of Hydraulic Engineering of EDF, deeming that traditional solutions for repairs such as concrete patching would not be satisfactory in the long term, especially at lower levels (the depth of the canal can reach 8m-10 m), explored the technical and economic feasibility of alternative long-term solutions. A pilot project was carried out at Kembs with two systems, one of which was the aforementioned mattress.

The project requirements were to restore watertightness, with required permeability coefficient $k < 1 \times 10^{-9}$m/s, and to provide a new upstream concrete layer at least 120mm thick and capable of withstanding the expected stresses from self-weight, differential deformations, irregularities of the existing layer, hydrostatic and hydrodynamic pressure, flow speed up to 1.5 m/s, boat wash, flow variations due to tripping of a hydroelectric generation group, tidal range, wake, propeller swirls, and impact of anchors or shocks by boats in the event of an accident. The works had to be carried out without stopping the operation of the canal, which is also used for hydropower, or the navigation of barges (on average 60 convoys per day). Further constraints were the presence of a high-voltage electricity line to the right of the work area, and the presence of construction joints of the 4.75m x 5.30m concrete slabs lining the canal. To measure the performance of the new revetment, EDF-CIH required the installation of a system to detect and locate the leaks that could occur in the lined section, and to monitor the leakage rate.

The mattress designed for Kembs has as waterproofing liner a 2.5mm thick Sibelon® C 3250 PVC geomembrane, and as grout confinement layer SIBELON® C 2600 R, a 2.0mm thick scrim-reinforced PVC geomembrane. The monitoring and leak location system comprises a drainage layer, with measurement of flow rate, an inclined piezometer, temperature and pressure sensors, and an optical fibre cable (OFC) system. The innovation for Kembs was to integrate the drainage layer, the OFC system, and the grouting hoses with the mattress at fabrication. The drainage geonet and OFC are attached to the bottom of the waterproofing liner (Figure 9) and the grouting hoses are embedded between the two geomembranes, so that the panels leave the factory incorporating all these elements plus the underwater joining system (the watertight zips). This innovation reduces the diving time, and is consequently a safer installation method, and guarantees there is no loss of cement in the water.

Figure 9. At left and middle, integrating the OFC and drainage layer at the bottom side of a mattress. At right, rolled void mattresses ready for installation at Kembs crest. The drainage geonet is the black material, and the integrated zip can be seen along the edge of the panels

The stretch to be lined was about 50m long, spanning the existing deteriorated concrete slabs of one embankment from crest down to about elevation 236.3m, i.e. on about 28m of slope, covering 80 slabs and spanning 26 vertical and horizontal joints. Five mattresses, each 10m wide and 28.4m long, were prefabricated to waterproof the area, in total 1,445m^2.

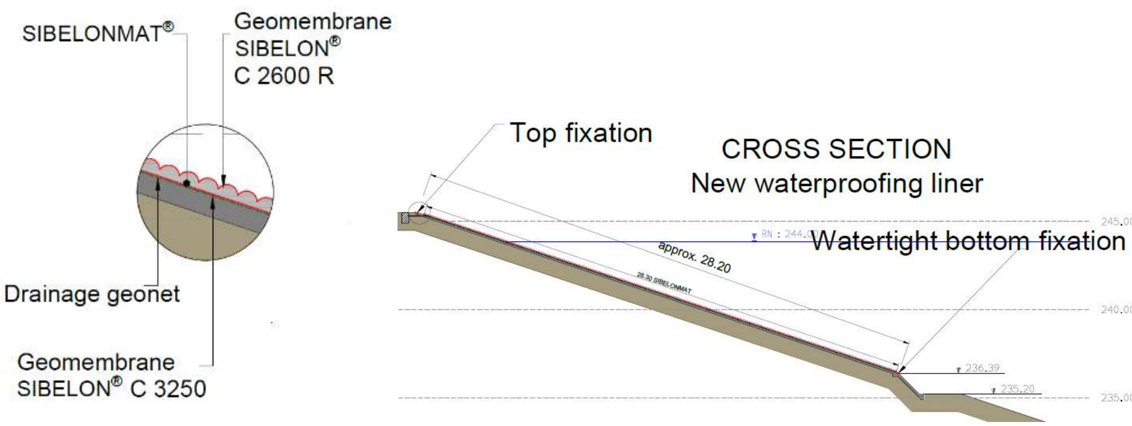

Figure 10. Cross section of the mattress and of the lined slope

The mattress is fixed at the top by a stainless-steel seal, watertight to rain and waves. The side peripheries have a standard watertight stainless-steel perimeter seal, the bottom perimeter seal is an L-shaped stainless-steel profile that also acts as support for the filled mattress. Special details were developed and tested in real scale for the underwater terminations of the zips, in a pressure vessel under water pressure of 40m for 144 hours and 60m for 48 hours.

Installation was carried out in 2020, from 22 October to the end of November. The high voltage line made it necessary to take customised safety measures, and to adapt the procedure for conveying the rolls to limit the height of the handling equipment. Navigation management was carried out first by providing information to the navigation services, limiting the speed of traffic, then by placing buoys delimiting the work area. Despite these measures, the site suffered repeated wash from the passage of boats, which however did not disturb the smooth running of the waterproofing works. Water speed during installation was variable, with a maximum average speed of 1m/s inside the canal, and a little less along the embankment, which allowed the divers to perform their underwater tasks without shelter.

The divers checked the conditions of the slabs before executing the treatment at joints. As often happens, the real conditions of the subgrade were somewhat different from those anticipated. The thickness of the slabs was not uniform, at some points being only 30mm, which required adopting different types of anchors (mechanical, semi-mechanical, resin based). The vertical joints at the bottom did not exist, therefore joint treatment was necessary only for the horizontal joints, seven upstream and six downstream. The solution envisaged for joint treatment had to be modified, because when drilling started a strong suction was experienced, which indicated that the resin could possibly have been sucked into the holes. The divers treated the joints with a suitable underwater resin. Each rolled mattress was set on a customised unrolling device, temporarily anchored at the crest, and unrolled down to the bottom (Figures 11 and 12). The divers controlled the correct unrolling and joining of the panels, and executed the bottom and side perimeter seals, while at the crest the top fixation was completed by the above-water crew. Adjoining mattresses were joined by pulling the zip from the dry, under the control of the divers.

Figure 11. The empty mattress deployed to underwater placement

Figure 12. Navigation ongoing during underwater works

After the panels had been joined, using the integrated grouting hoses the hollow space between the two geomembranes was injected from the crest with cement grout (Figure 13), thus reducing the diving time, increasing the safety of the divers, optimising the cost of the solution and, by preventing loss of cement in the water, providing a solution totally respectful of the environment. Figure 14 shows the completed mattress.

Figure 13. Grouting the mattress

Figure 14. Mattress completed

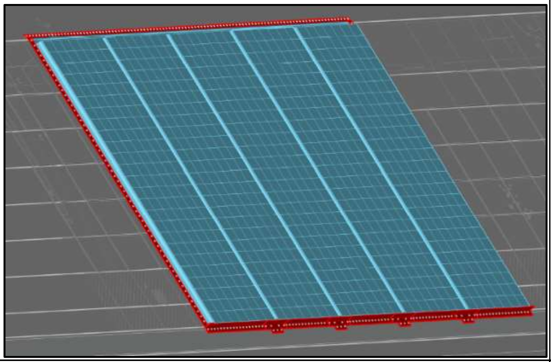	
Figure 15. Scheme of the lined area	**Figure 16**. View of five mattresses

Cross-checking the results of the multiple monitoring means will be possible to obtain a better knowledge of the actual behaviour of the installed solution. EDF measured the monitoring data upon receipt of the experimental plot with the tested solutions, and plans to carry out regular measurements to check their efficiency over time. The first results are encouraging and confirm the interest in developing this kind of technique. An important improvement in watertightness has been observed and is monitored to evaluate with accuracy the performance over time.

In terms of cost, underwater solutions are still more expensive than dry solutions, but each project must be assessed considering also financial, social and environmental dewatering costs. Furthermore, research is continuing and other solutions are already at a good development stage, to reduce costs and make underwater installation more competitive.

CONCLUSIONS

Underwater projects with geomembranes are technically very well performing, are the most sustainable solution, and are becoming more and more interesting also from a financial viewpoint. Pilot projects like Kembs enable improved knowledge and foster development of environmentally friendly solutions.

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The British Dam Society

The role of the Construction Engineer and Panel of Specialists in the modern contracting world

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SYNOPSIS The Reservoirs Act requires that for any new reservoir or alteration to enlarge an existing reservoir, a Construction Engineer is employed to design and supervise the construction or alteration. Procurement procedures have moved on from when the Reservoirs Act was written. Is there a need to rethink the role of the Construction Engineer for the modern contracting arena?

The Coxon report (Coxon, 1986), produced after the failure of Carsington dam in 1984, recommends that a panel of specialists reviews and comments on the design and construction of any major new dam. Expert panels are common on international projects and in some water companies, but there is less recent experience of panels for new dams in UK. We propose possible organisational arrangements for panels of specialists appointed for the new reservoirs that are proposed in England over the next 20 years.

INTRODUCTION

In England and Wales, the role of the Construction Engineer is defined in the Reservoirs Act (HMG, 1975). However, procurement procedures and contracting arrangements are quite different in the 2020s compared with 1975. The Construction Engineer is often from a different organisation to the designer; they do not necessarily directly design or supervise the construction.

The role of the Construction Engineer and the possible organisational arrangements need to be considered as early as possible during the development of a reservoir scheme. In the following section we discuss the alternative arrangements and their advantages and disadvantages based on recent experience on major new reservoir projects.

One of the aims of a panel of specialists is to draw on specialist expertise away from day-to-day project and contract issues. How can they operate to provide the best value possible to a project whilst maintaining independence? The paper will describe possible organisational arrangements, reporting lines and the relationship with the Construction Engineer.

The scale of the future water resources challenges and the possible supply side options was set out in a previous paper (Welbank, 2022). Since then, water companies have refined their Water Resource Management Plans which, subject to approval by Defra, should be published in 2024. A summary of the latest position was issued in March 2024 (EA, 2024). The revised draft water resources management plans contain proposals for seven new reservoirs by 2050.

Managing Risks for Dams and Reservoirs

At least five of the new water supply reservoirs proposed in England may ultimately be delivered by external privately financed entities, following a procurement exercise run by the water companies. How do we ensure that the statutory roles under the Reservoirs Act endure through such a procurement process?

Both the Construction Engineer and the panel of specialists need to fit into, and be an effective part of, the wider team delivering the new reservoir project, which includes:

- The reservoir owner or undertaker (or promoter, as the ultimate owner may change during development of the scheme).
- The designer and the contractor, or potentially the design-build contractor.
- Investors.
- Operation and maintenance teams.

The project team will also need to engage with a wide range of stakeholders, such as investors, financial regulators, quality regulators (Environment Agency and Drinking Water Inspectorate), the public, lobby groups, and potentially third parties who will receive a bulk water supply from the reservoir.

ROLE OF THE CONSTRUCTION ENGINEER

The role of the Construction Engineer is defined by section 6 of the Reservoirs Act (1975):

> *"No large raised reservoir shall be constructed (whether as a new reservoir or by the alteration of an existing structure or area that is not a large raised reservoir) or shall be altered so as to increase or decrease its capacity, unless a qualified civil engineer ("the Construction Engineer") is employed to design and supervise the construction or alteration; and where the use of a reservoir as a reservoir has been abandoned, and the reservoir is to be brought back into use after being altered so as to increase or decrease its capacity, that shall be treated for purposes of this Act as the construction of a new reservoir."*

Ultimately, the Construction Engineer must personally certify to the Enforcement Authority (the Environment Agency in England or Natural Resources Wales) that the works are satisfactory and fit to retain water. The role is non-partisan. In a criminal case, an expert's opinion must be objective and unbiased; it is the duty of an expert witness instructed by either party to act in the cause of justice. In an equivalent way it is the duty of a Construction Engineer to act in the cause of public safety.

The natural meaning of the language in the Act, "*… is employed to design and supervise the construction or alteration…*" is clear. This is the legal requirement set out by the Act – the Construction Engineer is to design and supervise the construction of the works. This may be a one-man exercise for a small dam, but most likely the responsible engineer will direct a team to carry out work to their satisfaction.

However, this clear legal requirement has been corrupted.

Procurement needs and procedures have moved on from those applicable when the Reservoirs Act was written. In the 1970s, the design-bid-build process applied; clients appointed consultants to design infrastructure, projects were tendered and then contractors constructed what was defined on drawings and specifications. Design-build was rare; the ICE

Conditions of Contract were in vogue. The consultant appointed a Resident Engineer to impartially administer the contract and supervise construction with their staff.

Progress, demand for efficiency and less commercial controversy has led to modern procurement of design-build projects and demand for innovation. Application of the Act has evolved so that the Construction Engineer is now often from a different organisation to the designer; they do not necessarily directly design or supervise construction of the works. This is explicitly acknowledged and accepted by the Guide to the Reservoirs Act (ICE, 2014) which is the official guide published by the Institution of Civil Engineers with the help of prominent members of the reservoir community and members of various government agencies, including the Environment Agency and Defra.

The Guide sets out three options for the procurement of the services of a Construction Engineer:

- The Construction Engineer can be an *employee of the consultancy engineering firm* leading the reservoir design.
- The Construction Engineer can be *independent of the design and construction firm*.
- The Construction Engineer could be a *direct employee of the reservoir owner*.

Employee of the consultancy engineering firm

The first approach reflects the original intention of the Act. The Guide notes advantages of this approach such as: the engineer should be better integrated into the design team, have better experience of the staff involved in the design, and communications should be more effective.

This approach allows the Construction Engineer to directly bring to bear all the experience and technical knowledge that qualified them for appointment to the appropriate Panel under the Act. A single mind directing the design should ensure a clean philosophical approach and avoid the potential for design-by-committee compromise. These must be overwhelming advantages to the successful outcome of the project.

This is true under traditional design-bid-build procurement. However, the more recent propensity for design-build contracts brings potential tensions between the parties to the project. Commercial pressures could be brought to bear on a Construction Engineer by members of a design-build consortium team. One might hope that exacting standards of professionalism would provide some protection, but Construction Engineers are only human.

Independent of the design and construction firm

For the second approach, with an independent Construction Engineer, the guide proposes that the arrangement might offer a greater degree of independence and challenge to the design and construction process as well as being contractually independent and free from commercial pressures where the design team is a junior partner in a joint venture.

However, this heightens potential for conflict between the parties.

With this approach the designer would naturally be commercially liable for the design – they expend the effort and receive the design fee; the Construction Engineer is compensated for their time, but their fee is unlikely to be proportionate to the potential liability associated with a major reservoir. The designer is commercially liable, but the Construction Engineer is legally responsible. There are civil and criminal legal issues to consider. In the event of a problem, would commercial liability stay with the designer? Perhaps, but probably only if there were no controversies during the design. Commercial pressures can be ruthless, and the common-sense approach taken for granted by the engineering community might prove naïve.

With this approach, the Construction Engineer must tread lightly to avoid instructions that will incur claims or compensation events to the employer from the designer or design-build contractor. The guide suggests that this approach frees the Construction Engineer from commercial pressures, but actually it introduces a new set of pressures and an incentive to compromise.

This approach is also not compliant with the black and white requirements of the law – with this arrangement can the Construction Engineer truly be said to be "… employed to design and supervise the construction or alteration…"? The designer is explicitly employed to carry out the design. True, the Construction Engineer can veto aspects of the design that are unacceptable to them, but there are potentially massive pressures to collaborate and compromise unless there is an obvious increase in risk that the Construction Engineer determines is unacceptable. The Construction Engineer's instincts, honed through years of varied experience, may not be sufficient to influence a designer set to follow an alternative course. If the designer is inexperienced, the Construction Engineer will expend effort educating and attempting to influence the designer to their way of thinking. The good instincts of the Construction Engineer might be put down as preferential engineering to the detriment of reservoir safety.

If the designer is also a qualified engineer under the Act, perhaps an All Reservoir Panel Engineer, it is foreseeable that a difference of opinion could be unreconcilable even though both parties aims are to produce a design that is safe.

This is an inefficient approach that could increase project risk and is potentially not legal.

An independent Construction Engineer may be appropriate for small projects where risks are low, and the project is of a scale that makes an independent panel of specialists inappropriate. Success relies upon the professionalism of the engineer and how they negotiate any ethical dilemmas that might arise. However, for a major project with a panel of specialists to provide independent design assurance as recommended by the Coxon report (described below), it could be argued that any advantages of having an independent Construction Engineer are outweighed by the disadvantages.

Direct employee of the reservoir owner

There is no requirement under the Act for the Construction Engineer to be independent of the employer and so the third option listed in the Guide is theoretically legal. However, one might question potential conflicts of interest from an employee simultaneously acting as an agent for the government to police public safety. Professional standards could prohibit this arrangement. In practice this is not an option because there is really no incentive for someone at a client to be an ARPE as they cannot inspect their own reservoirs, and it is unlikely that their employer would cover the professional indemnity insurance for them to do external work.

It is apparent that none of the options offered by the Guide are ideal. It is essential that the arrangements are planned as early as possible in the development of the project, including some thought applied to how conflicts might be resolved, and contracts are procured appropriately.

This is an industry issue which would be benefit from a joint review and subsequent update to the Guide.

CONTINUITY FOR THE CONSTRUCTION ENGINEER ROLE

The major reservoirs currently contemplated by UK's water companies will take many years to complete (Table 1). Development periods in the range of 10 to 20 years are not uncommon for major projects with Development Consent Order planning, design, construction and filling. The Construction Engineer has responsibilities under the Act for a minimum of three years after construction before the Final Certificate can be issued. The Guide to the Reservoirs Act suggests that the intention of the Act is for a single engineer to be responsible for the complete development of a single reservoir.

Given the potential time scales involved, this is impractical.

Either only the youngest (and least experienced) Construction Engineers could be appointed, or the engineers will be expected to continue well beyond normal retirement age. A more reasonable approach might be to anticipate the need to change a Construction Engineer, perhaps through ill health, accident, or retirement.

A resilient approach would be to assign a Construction Engineer from an organisation that has engineers qualified to step into the gap should it be necessary and has a pipeline for reservoir engineers in development. In such an organisation one would not expect the Construction Engineer to work in isolation, even though personally responsible for the reservoir; there would be design reviews and conferences. The Construction Engineer's work would become a development opportunity for tomorrow's reservoir engineers.

DESIGN ASSURANCE

Large employers developing major infrastructure are rightly concerned about design assurance. Many demand several levels of assurance. This can be provided in a few ways:

- Designers all follow quality control and assurance processes; most are certified to ISO 9001:2015 – Quality Management Systems. This might involve numerical check and review, as well as internal formal design review meetings with independent senior/experience technical staff.
- The Construction Engineer requirement is a form of design assurance.
- The Panel of Specialists process, described below, is another level of design assurance.
- Since the collapse of the steel box girder bridges in Milford Haven and Melbourne in 1970 there has been a culture of independent design checking for major bridges in the UK (Firth, 2007). This culture has organically grown through the major projects arena including projects such as the Millenium Dome, Heathrow T5, Crossrail, and Thames Tideway Tunnel. Given the magnitude of the potential consequences of a dam failure compared to the more limited impacts from a bridge failure, it seems reasonable to apply the independent check culture to reservoirs. For a low additional cost relative to overall project cost, the client can obtain several added benefits including:
    - risk reduction

- confidence that the design criteria are appropriate, especially if the structure or problem is innovative or unusual
- confidence that the design is in accordance with the agreed criteria
- reassurance that the finished structure, if properly built in accordance with the design, is likely to perform as intended
- another consultant who may share some of the liability if problems arise later.

If the Construction Engineer is independent of the designer, once principles are agreed, they must somehow satisfy themselves that the design is numerically correct. For a small low risk project, it may be that a Construction Engineer is satisfied that the designer's quality system is sufficient given that the designer is notionally liable for the design. However, for a major project the Construction Engineer may take the view that the project may not be certifiable without independent checks of safety critical elements.

PANEL OF SPECIALISTS

Purpose of a panel

The main purpose of a Panel of Specialists (also known as Panel of Experts or Reservoir review panel) is to provide a separate independent review of the design and construction of the reservoir. For a major reservoir, the design is now most commonly carried out by a design-build consortium, supervised by the Construction Engineer. The intention is that the panel can scrutinise, challenge, and advise on the design with a different perspective, away from the immediate time and cost pressures of the project.

The report on the failure of Carsington dam (Coxon, 1986) recommended the appointment of a Board (panel) of Specialists to review and comment on a project as the work proceeds. The remit of a panel is described as:

- It requires reports to be prepared in anticipation of routine meetings which, in their very presentation, lead to key elements being identified and assessed.
- Discussion with the parties involved can bring attention to special matters arising.
- The Board, by standing aside normally from contract issues, can, where necessary, interject alternative views.
- Reports, where necessary critical but certainly impartial, are sent to the owner as well as the engineer.

Coxon emphasised that it is important to recognise that the responsibilities placed on the Construction Engineer are in no way diminished by the appointment of a review Board (panel).

The World Bank requires independent reviews of new dams (World Bank, 2020). Their guidance includes:

- The objective of the independent review is to examine safety and quality of the design in an objective manner to detect any potential safety issues that may have been overlooked by the client and designer.
- Effective panels are small (three or four members).
- The panel should be free to review any aspect.

- The panel members should be made up of individuals who are not afraid to state their opinions yet are able to work collectively in a group setting.

The Balmforth independent review of reservoir safety (Balmforth, 2021) includes some consideration of the approach to safety taken in other sectors. It reviews the approaches adopted in the nuclear industry and the rail industry. In both sectors there is specific legislation giving regulators powers and duties to review safety processes and reduce risk.

The Reservoirs Act does not provide the regulator with similar powers, and it places responsibility for reviewing the safety of the design on the Construction Engineer.

After the problems at Carsington, Severn Trent Water has retained a panel of specialists to review all their major reservoir projects (known as the Review Panel). Some details are provided in a paper for the British Dams Society conference in 2012 (Hope, 2012). The panel comprises two eminent dam engineers, who report directly to the Director of Water Services, thus providing an independent route of corporate governance.

Panels of Specialists have been established for some of the upcoming new reservoir projects including the South East Strategic Reservoir Option (SESRO), Fens and Lincolnshire reservoirs. Havant Thicket reservoir, which is under construction, reformulated its Panel of Specialists in 2023.

Composition of the panel and reporting lines

Normal best practice is to have an odd number of panel members with one person acting as the chair.

All the new reservoirs proposed in England will be earth fill embankment dams. Thus, the principal areas of expertise required on a panel are likely to be dam design, embankment stability, geotechnics, and engineering geology. Other aspects that may be significant on a case-by-case basis could include hydrology and hydraulics, mechanical and electrical equipment, and concrete design.

As the reservoir projects will also need to deliver biodiversity and environmental net gain, there may be a case for including an environmental expert.

Panel meetings will include:

- meetings of the panel on their own.
- design review meetings with the design-build contractor's designer, Construction Engineer, owner, and programme management team.

Given the emphasis in the Balmforth review that the ultimate responsibility for the reservoir rests with the owner, and the precedents internationally, the panel should have a direct reporting line to the owner. There are several ways this could be achieved. The independent chairperson for the panel could have a direct reporting line into the owner's Board. Alternatively, the chair of the panel could attend audit committee meetings as required.

Organisational arrangements

Modern practice for major infrastructure projects is to form integrated teams or alliances of designers based on "best person for the job" regardless of organisational allegiance. Whilst this may work in the delivery of other infrastructure projects, they do not operate within a statutory regime such as the Reservoirs Act 1975.

Managing Risks for Dams and Reservoirs

There may be a perceived advantage in reducing the total number of specialists involved. However, with the legislative background for reservoirs, and past experience including the Coxon report, it is considered that an organisational structure that maintains independence is preferable. A possible generic arrangement is shown in Figure 1.

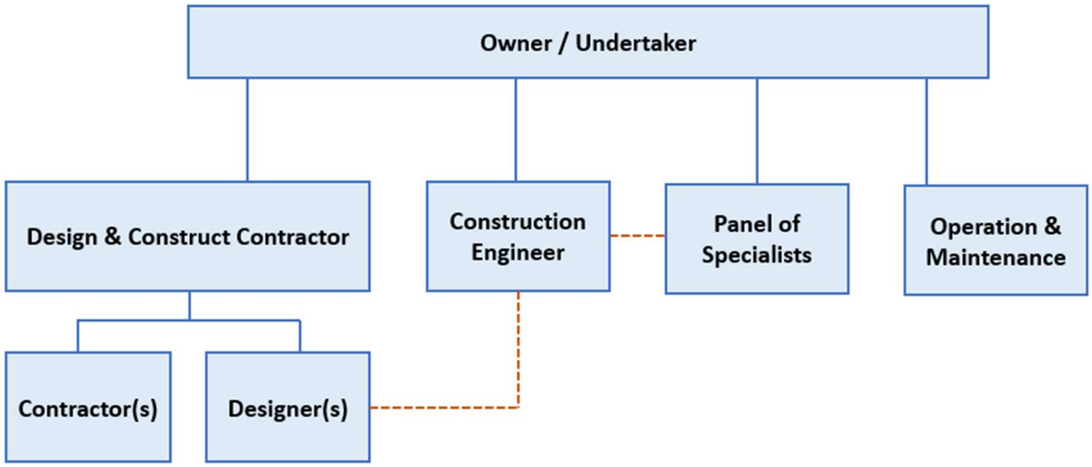

Figure 1. Typical organisation chart for a new reservoir

PROJECT DELIVERY MODELS

Background

Ofwat's policy position is that major new infrastructure should be delivered by competitive delivery models, outside the water companies normal capital investment programme. Two new delivery methods (RAPID, 2023) are proposed:

- **Direct procurement for customers (DPC)**. DPC is a process whereby companies put major infrastructure projects out to competitive tender for delivery by third parties. It is applicable for all discrete projects above a size threshold of £200m. The successful bidders for DPC projects, known as the Competitively Appointed Providers (CAPs), will be responsible for designing, building, financing, maintaining and potentially operating the infrastructure for a defined concession period.
- **The Water Industry (Specified Infrastructure Projects) (English Undertakers) Regulations 2013 (SIPR) model.** This is the model used for Thames Tideway Tunnel. SIPR is appropriate where the size or complexity of the project could threaten the incumbent water company's ability to continue to provide services for its customers. In practice this means SIPR is being considered for projects with a value in excess of £1bn. This model requires the infrastructure to be specified by the Secretary of State or Ofwat if, in their opinion, a project meets various tests (Ofwat, 2024). An Infrastructure Provider (IP) appointed under SIPR may be issued with a project licence, therefore being directly regulated by Ofwat i.e. they become a new undertaker regulated under the Water Industry Act 1991. The IP is responsible for designing, building, financing, maintaining and operating the infrastructure. The IP is the owner of the reservoir in perpetuity.

In both models the initial development of the new reservoir projects, including design, planning permission, stakeholder consultation etc. is undertaken by the incumbent water company before the project transfers to either the CAP or IP. The promoting water companies are responsible for running the procurement exercise required.

The aim of the projects is to generate additional water resources that act conjunctively with existing reservoirs and sources to provide greater resilience during droughts, thus in all cases the operation of the reservoir in water resources terms will remain with the water company as part of its wider system operation role.

In most cases in the past the owner of the reservoir and the user (or operator) have been one and the same organisation. Section 1(4) of the Act implies that the user of the reservoir for the purposes on an undertaking (such as a water supplier) rather than the owner is the undertaker under the Reservoirs Act. Thus, subject to confirmation by lawyers, although the SIPR model would create a new undertaker, it appears that responsibility for the Reservoirs Act would remain with the water company. This also gives rise to additional considerations regarding maintenance of the reservoir.

The key premise of both models is that, in a similar way to the Thames Tideway Tunnel project, the new investors will be able to raise the finance for the projects efficiently. The approach to risk management will be key, indicating an even greater need for early ground investigations, trial embankments and design resolution etc. as early as possible, ideally before contract and financial closure. The delivery approaches planned for the proposed new reservoirs are summarised in Table 1.

Table 1. Summary of new reservoirs and procurement approaches

Reservoir	**Promoters**	**Procurement approach**	**Timeline**
South East Strategic Reservoir Option (SESRO)	Thames Water	SIPR	Operational in 2039; Construction start 2030
Fens	Anglian Water and Cambridge Water	SIPR	Operational by 2036; Construction start 2029 - 2031
Lincolnshire	Anglian Water	SIPR	Operational by 2040; Construction start 2029 - 2031
Cheddar Two	South West Water and Wessex Water	DPC	Operational by 2035; Construction start 2030
Broad Oak	South East Water	DPC	Operational by 2035; Construction start 2028

Other new reservoirs included in Water Resource Management Plans are at an earlier stage of development with delivery methods still to be determined. If they are smaller in size with less complexity it is possible they will be delivered conventionally as part of the water companies' capital investment programmes with full responsibility for the duties under the Act remaining with the water company.

Implications related to the Reservoirs Act

The Reservoirs Act envisages a single entity is responsible for the planning, design, construction, operation and maintenance of a reservoir, although generally the implementation phase of planning, design and construction are contracted out but under the direction and control of the owner. The undertaker is generally the owner or operator of the reservoir and has ultimate responsibility for the safety of the reservoir. The undertaker appoints the Construction Engineer and in due course the Supervising Engineer.

Table 2. Procurement approaches and the Reservoirs Act

Procurement approach	Undertaker	Responsibilities
SIPR	Lead promoter is the undertaker up to the appointment of the IP. The IP becomes the owner of the reservoir on award of the project licence by Ofwat. The water company remains as undertaker under the Act.	• The appointment of the Construction Engineer may remain with the water company, but the Construction Engineer will have to interact with the IP and their designer and contractor. • Reporting lines for the Panel of Specialists will need to adapt to suit the split of responsibilities. • Operation of the reservoir will remain with the water company in order that they can optimise its use in the wider water resources system. • Maintenance of the reservoir is likely to require a detailed allocation of responsibilities between the IP and the water company.
DPC	Lead promoter remains the undertaker under the Reservoirs Act throughout.	• Design and construction would be the responsibility of the CAP according to the contract terms between the promoter/undertaker and the CAP. • Operation of the reservoir will remain with the water company. • The appointment of the Construction Engineer and the Panel of Specialist will remain with the undertaker. • Maintenance of the reservoir is likely to require a detailed allocation of responsibilities between the CAP and the water company. • Ofwat also require the appointment of an Independent Technical Adviser, to obtain assurance around the costs and delivery of a DPC project both during the construction programme and to operate over the life of the DPC project.

The alternative delivery methods outlined above create some departures from the vanilla approach set out in the Guide to the Reservoirs Act (ICE, 2014), as highlighted in Table 2.

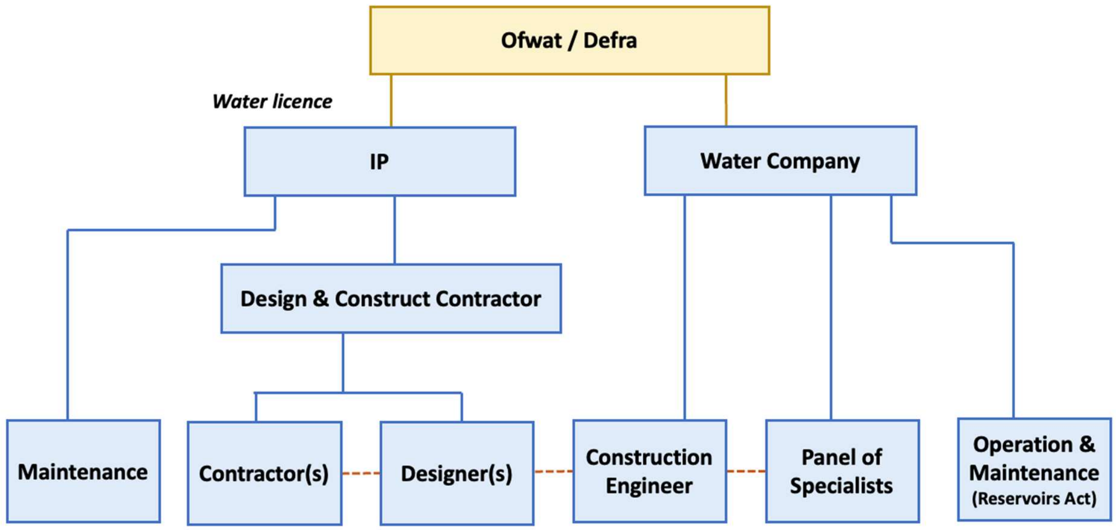

Figure 2. Potential SIPR model

In both cases there will be some migration of roles and responsibilities during the project lifecycle. Figures 2 and 3 provide some initial views of potential organisational arrangements during implementation by which time the CAP or IP will be in place.

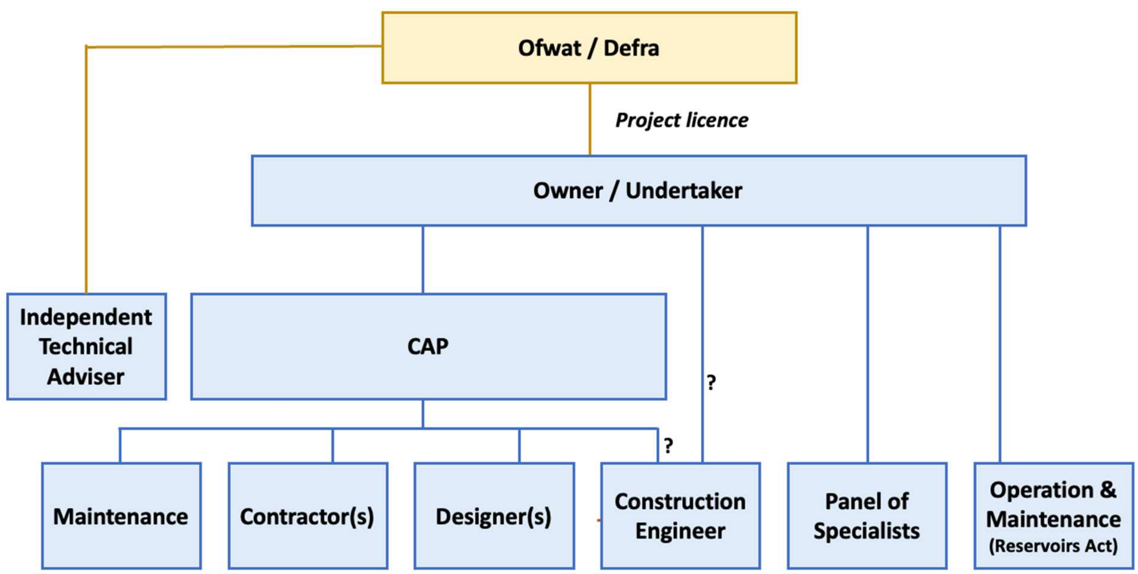

Figure 3. Potential DPC model

CONCLUSIONS

As an industry, we should recognise that procurement and contracting arrangements have moved on since the era when the Reservoirs Act was written and since the last major reservoirs were constructed in the UK. The traditional procurement approach assumed by the Act is unlikely to apply to any of the new reservoirs planned in England over the next 20 years, but the legal requirements do not change.

Managing Risks for Dams and Reservoirs

As a profession, we need to make sure that new infrastructure is safe and as economical as possible.

Regarding the statutory role of the Construction Engineer, organisational arrangements need to be considered as early as possible during the development of a reservoir scheme, with regular reviews as the project progresses over the subsequent 15 to 20 years. None of the options set out in the Guide appear to be ideal, so we would advocate that the industry considers the issues collectively, that the law is reviewed, and the Guide is updated accordingly.

In the initial stages of project development and outline design, the Construction Engineer should be appointed from the design consultancy engaged for the design. Once the project moves into the delivery phase the arrangement for the Construction Engineer's appointment needs to be considered hand in hand with the project procurement plans. The overriding objective is to achieve a completed dam that is safe over its long life, even if this means foregoing some potentially cheaper notions in the short term.

For design-build projects it might be appropriate to novate the Construction Engineer to the successful consortium. Alternatively, a reference design prepared by the Construction Engineer could be made a more rigid contractual requirement, with deviations only permitted with acceptance of the Construction Engineer. This may seem a regression towards design-bid-build, but substantial design work is already required to secure a DCO or planning permission, so this approach avoids duplicating that effort.

A Panel of Specialists serves as an additional safeguard to scrutinise the design and construction away from the day-to-day project and contract issues. To provide best value they need the ability to engage with the designer but also report directly to the owner on the 'big picture.'

At least five of the new water supply reservoirs proposed in England may ultimately be delivered by external privately financed entities following a procurement exercise run by the water companies. It will be critical to carefully define and manage responsibilities for operation and maintenance to ensure the overall requirements of the Act are met. To maintain continuity, it will be necessary for the Construction Engineer role and the Panel of Specialists to adapt to new arrangements as the project moves into its contract and delivery phase.

Overriding all these project and contract specific issues is the need for the industry to resource the multiple roles for panel engineers and reservoir specialists in these projects. Delivering on the recommendations in the review of the future supply of panel engineers (ICE, 2022) will be crucial.

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The British Dam Society

The Effect of Pretreatment of Organic Matter on the Outcomes of Dispersion Tests

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SYNOPSIS Internal erosion in clayey soils is associated to the identification of dispersion as this can be a major contributing factor in piping failure of earth embankment dams. For dams constructed without filters and of poor construction, it is critical to understand the nature of dispersive soils so they can be treated or appropriate remedial measures applied. This paper describes tests carried out using the Double Hydrometer Test, a type of physical dispersion test, on a representative core sample from a Pennine-type dam in Yorkshire. The determined potential for dispersion is compared for the soil tested with pretreatment using hydrogen peroxide to remove organic matter and without pretreatment. As well as highlighting the importance of pretreatment in determining the potential for dispersion, the results demonstrate that the amount of soil used in the hydrometer test should be carefully considered to avoid both hindered settling (using too much soil) at one extreme and poor hydrometer response (using too little soil) at the other.

INTRODUCTION

Internal erosion is defined as the detachment of soil particles within a soil mass due to the flow of subsurface water. This process is associated with seepage and leakage, which may pose a safety issue for small dams, levees and dikes and a more significant threat to the long-term safety of large embankment dams. However, the mechanisms and parameters involved in the progression of internal erosion in non-plastic and plastic soils are distinctive and therefore the methods of assessment for the potential for internal erosion for these two types of soils are different. For example, internal erosion via suffusion in non-plastic soils develops when an internally unstable soil with poor gradation (e.g. gap grading) and underfilled voids leads to highly stressed particle contacts in the coarser fraction and loose erodible fine-grained particles in the soil's finer fraction (Ronnqvist & Viklander, 2014). Similar associations of local packing and seepage flow are behind other forms of internal erosion in non-plastic soils, such as contact erosion and concentrated leak erosion. In contrast, the process of internal erosion in plastic or cohesive soils typically develops when water flowing through a crack removes material from the walls of the crack and transports it into the interstices of the downstream shoulder, foundations or drainage system; this process is closely linked to the identification of dispersive clays (Atkinson et al, 1990).

Dispersion

The presence of dispersive soil can be a major contributing factor to piping failure of earth embankment dams, particularly for dams constructed without filters and of poor construction (Jeyabalamoorthy, 2007). Dispersive clays are soils in which the physico-chemical state of the clay fraction of the soil is such as to cause individual clay particles to deflocculate / disperse and repel each other in the presence of relatively pure water and are therefore highly susceptible to erosion and piping (ICOLD, 1990). Granular soils can dislodge and move in water and may be highly erodible, but the internal erosion process is mechanical. Erosion in embankments constructed with dispersive soils occurs in areas of high crack potential such as around conduits, at the contacts between zones of incompatibilities of stresses, strains and deformations both within the embankments and at foundation and abutments or in areas of desiccation cracks, differential settlement cracks, saturation settlement cracks and / or during hydraulic fracture (ICOLD, 1990).

Several methods for identifying dispersive clays have been proposed, these include the following tests: Physical Tests including Crumb, Pinhole, Double Hydrometer (also known as Dispersion Test) and Chemical Tests including Sodium Adsorption Ratio (SAR) and Percentage of the Exchangeable Sodium Cation (ESP) and a number of different Auxiliary Tests (grading, Atterberg limits, pH, etc.). However, researches on identification of dispersive soils have not yet established a single test that will identify dispersive soils. Coupled with this, it has been observed that identification of dispersive clays by visual description and classification index tests (i.e., gradation, Atterberg limits) is not sufficient to conclude the potential of soils for dispersion (ICOLD, 1990). On the contrary, studies have shown that physical and chemical tests may indicate different dispersivity classes and dispersivity potential cannot be established accurately using a single test method. It has been concluded that the erodibility of materials having the same appearance and index properties can vary even at short distances and that the dispersiveness of susceptible materials may increase with time.

The aim of this paper is to provide some preliminary outcomes of on-going research on the identification of dispersive soils in the UK, with a focus on Double Hydrometer testing on a clay core sample from a Pennine-type dam. We compare the results of tests on clayey soil pretreated for organic matter with that not pretreated. We also compare with results of tests undertaken by a commercial contractor on samples from nearby in the same dam core. From the outcomes we suggest some changes to practice that may improve the outcomes from hydrometer dispersion tests.

Investigation in the UK

The majority of the old Pennine-type dams in the UK were constructed with a puddle clay core and without the benefit of modern-day well-designed filters; selected fill however was placed next to the puddle clay core. This selected fill was more cohesive than the general embankment fill and might function as a filter (Tedd et al, 1987). Frequently the cut-off was a deep puddle clay filled trench excavated in an open jointed rock which might not offer any protection against erosion. Where the puddle clay in an embankment or its foundation is unprotected, the internal stability of the soil in the fill and foundations when subjected to drag forces from seepage and leakage is critical to the long-term performance of the dam. That is, while all clays will erode under severe conditions, in assessing the performance of existing structures it is important to determine the working erosion resistance of puddle clay core dams.

The specifications for the creation of puddle clay are indicated in the following criteria (Moffat, 1990), (it should be noted that this specification was developed in the 1940s but the general criteria prior to this generally remains the same):

- tenacious clay will not disintegrate when a well kneaded ball of 75mm diameter is immersed in water for at least 48 hours;
- sand content of 10% to 25% is considered desirable to control shrinkage;
- sand content of 40% to 50% is accepted if uniformly distributed in the clay matrix; and
- grading and consistency limits.

On this basis, it can be concluded that the cores of the Pennine-type dams may be clay rich or deficient and plastic or non-plastic, and as such, the cores could lie between the classic definitions of unstable due to low-plasticity granular instability or dispersion.

Within the UK, physical dispersion tests are the most common (and frequently only) type of laboratory test undertaken to assess the potential of soils for internal erosion and often the conclusions in the assessment are inconclusive. Physical dispersion tests such as Pinhole, Crumb and Double Hydrometer are scheduled for testing on soils undertaken during ground investigation across various numbers of samples. Often one type of testing dominates the others, while sometimes very limited testing is undertaken on an incorrect type of material (i.e. one that is granular) indicating that the understanding of both physical and chemical properties of dispersive soils is not strong. It should be noted that soil dispersivity tests do not measure the erodibility of soils per se, but measure dispersivity as an index of the likelihood of initiation of erosion.

DOUBLE HYDROMETER TESTING

Sample Location

The research undertaken by the authors includes collection of soil samples from various Pennine-type and Modern embankment dams in the Yorkshire and Northumbrian regions for both physical and chemical dispersion testing. In this paper, the soil sample considered was taken from the core of a reservoir which is dubbed here “Reservoir X”, which is a typical “Pennine” type embankment of the mid-late Victorian era. This reservoir and its neighbouring reservoir were constructed in the 1870s. The embankment has a crest length of over 600m and a maximum height of 20m, with an overflow located at one end. The structure was zoned with a central puddle clay core. Selected clayey material was placed in inner zones on either side of the clay core with thicker layers of more stony material in the outer zones. The dam embankment was made watertight by the puddle clay core, which was carried down into a cut-off trench, with the depth of the cut-off trench varied along its length up to 18m deep. The deeper sections of the trench were partly infilled by concrete.

Superficial Deposits are shown to be absent across the reservoir, however peat is shown across the wider valley area in the western and southern regions. The solid geology comprises Millstone Grit Group bedrock that underlies the reservoir, and which is characterised by grits and sandstones, interbedded with siltstones, mudstones, marine shales, thin coal seams and seat-earth. There are no mapped faults passing beneath the reservoir basin or dam. Areas of landslip are shown on the geological mapping across the wider valley area, the closest area being approximately 150m away from the reservoir.

Managing Risks for Dams and Reservoirs

It is understood that there have been no serious reservoir safety incidents associated with the reservoir, however there is little information as to how the dam embankment performed in its first half-century. There is a variation in embankment settlement in some of the banks despite the relatively small difference in the height of the banks, with a maximum settlement of 117mm recorded over the last 27 years. The surface of the downstream shoulder is very irregular when compared to many nearby dams of similar age and recorded variable drainage flows indicate that leaks could initiate, develop, and enlarge through internal erosion and that flow may be by passing the measuring points. In the late 1990s a small cavity was discovered near the downstream toe of the embankment. This was subsequently investigated and the cavity was tentatively attributed to the possible presence of a timber post that had been incorporated into the face of the embankment which had subsequently rotted away.

Sample Description

A ground investigation (GI) was undertaken on Reservoir X in 2022 as recommended in a Section 10 report for the reservoir, under Section 10(2) of the Reservoirs Act, 1975. A bulk sample of the clay core was taken from 11.5m depth in a borehole (denoted “BHA”) located on the crest of Reservoir X; this sample was tested using the Double Hydrometer method with the results presented in this paper. This sample is described as very soft to firm, greyish black, slightly gravelly, slightly sandy, very silty CLAY.

It should be noted that a Double Hydrometer test was also undertaken by the GI Contractor on another clay core sample at 6.5m depth in borehole (“BHB”) located approx. 60m from BHA, as scheduled by the Consulting Engineer.

Sample Preparation and Testing

The Double Hydrometer test for BHA 11.5m was undertaken with reference to BS1377 (Head, 2011). This test is based on the degree of dispersion of clay particles achieved during a hydrometer test. The test compares the percentage of clay in a sample that has been artificially dispersed to that of another sample which has no artificial dispersing agent added. The dispersion is taken as the ratio of the percentage of clay (particle diameter 2 microns in BS standards and 5 microns in other standards i.e. ASTM) of Sample A to Sample B (see description below). Common criteria for evaluating the results are outlined in USBR 5405 (Umesh et al, 2011) with a value of <30 taken for non-dispersive soils, a value of >50 for highly dispersive soils and anything between 30 and 50 indicating moderately dispersive soils.

Double Hydrometer Testing in BHA 11.5m

The recommended mass of test specimen was approximately 100g, which is the amount recommended for soils with particle diameter up to 2mm with any gravel size particles comprising <10% of the sample. The Wet Sieving method was used for the silty clay soil samples (for sizes less than 2mm) down to a particle size of 63 microns. Sedimentation by hydrometer test was subsequently undertaken on the remaining soil with at least 15% of fines passing the 63 micron sieve. For clay, Head (2006) recommends a mass of soil used for sedimentation (i.e. using hydrometer) of 30g, but he also notes that the mass may depend on the type of soil, stating that too much soil can prolong a test unnecessarily and too little soil can provide unreliable results. Hence, it is recommended that if in doubt, trial tests should be undertaken. Prior to testing the 20g of soil sample (as discussed below in Test 2), hydrometer testing on other core samples from other boreholes in Reservoir X was undertaken on a 30g soil sample, however, hindered settling was still observed using this mass of sample.

In this paper, the first set-up includes 100g for the original sample (Test 1), with the sample being pretreated for organic matter. It should be noted that the BS standard (BSI, 2016) indicates that organic matter present must be removed by chemical treatment (known as the pretreatment stage) prior to the sedimentation test (either via pipette or hydrometer test). The set-up includes the following:

- Sample A: Pretreated main specimen (after drying) and hydrometer test without mechanical stirring and using distilled water only.
- Sample B: Pretreated main specimen (after drying) with standard hydrometer sedimentation test with mechanical stirring and dispersant solution (33g of sodium hexametaphosphate and 7g of sodium carbonate in distilled water for a 1L dispersant solution).

It was also noted in Head (2006) that for inorganic soils, pretreatment is not necessary, however where the effects of pretreatment on the results are uncertain, parallel tests should be carried out (with and without pretreatment on two similar specimens). To check the effect of removing organic matter on the dispersivity of soil, the recommended parallel tests were also prepared for the same sample as above, again using 100g as the initial soil quantity.

- Sample C: as sample A but not pretreated.
- Sample D: as sample B but not pretreated.

A second set-up (Test 2) was undertaken using a 20g mass of main specimen after it was observed that 'hindered settling' had affected Sample B in Test 1. Hindered settling is further discussed in the Results section below.

The pretreatment of soils utilised the addition of 150ml of hydrogen peroxide on the dried mass specimen, allowing the sample to stand overnight, then heating and boiling the pretreated sample the following day until the volume of liquid was reduced to about 50ml. Simultaneous to the pretreatment for organic matter, a further check for the presence of calcareous matter was undertaken by adding HCl to a small portion of the sample to check if acid pretreatment was also required; the sample did not react with HCl. The samples pretreated for organic matter (Samples A and B) were then filtered and dried, with the pretreated dried mass subtracted from the untreated dried mass of the original specimen to derive the percentage loss of organic matter.

Double Hydrometer Testing in BHB 6.5m

The Double Hydrometer Test undertaken for a sample from BHB at 6.5m depth was undertaken by the GI Contractor with reference to BS 1377 (BSI, 2022) which refers to the hydrometer sedimentation test in BS EN ISO 17892-4 (BSI, 2016). In this updated standard, the use of hydrogen peroxide to remove organic matter is given as optional only (Clause 4.5.4) while pretreatment to remove organics prior to sieving, if required, should state the method on the test report together with the amount of material removed. Furthermore, Clause 5.3.2.4 indicates that pretreatment is recommended if organic material and/or carbonate compounds are present – but this statement is less strong than the recommendation by Head (2006) to check for the influence of organics and which makes reference to BS 1377 (BSI, 1990).

It is also noted in Clause 5.3.2.1 that the initial soil specimen, prior to preparation, should be large enough to give 20g to 30g of material smaller than 63 microns and that a suspension concentration of around 25g of sediment smaller than 63 microns per litre of solution is regarded as ideal.

For the GI report, no details of the amount of original mass specimen or mass that went in the hydrometer testing and details of pretreatment were provided. It is possible that 100g of soil specimen was used and / or that no pretreatment of soil was undertaken. This information is observed to be generally absent in all GI factual reports that the researchers have seen to date.

RESULTS AND DISCUSSION

Table 1 shows a summary of Double Hydrometer test results for Test 1 and Test 2.

Table 1. Double Hydrometer Test Results for Soil Sample taken from Reservoir X

Properties	**BHA 11.5m**			
	Pre-treated		**Not Pretreated**	
	A	**B**	**C**	**D**
Test 1 - Original mass of specimen approx. 100g (see Figure 1)				
Dry mass of specimen (g)	79.97	80.91	77.21	86.23
Moisture content (%)	21	21	23	23
Organic Matter lost after PT (g)	2.65	4.13	-	-
Organic Matter lost after PT (%)	3.31	5.1	-	-
Total dry mass ≥63μm (g)	6.94	10.14	37.50	14.13
Total mass for hydrometer test (g)	70.38	66.64	39.71	72.10
Percentage clay (passing 2μm) (%)	22	21	8	42
Dispersion (% clay A/B or C/D)	≈100		19	
Test 2 - Original mass of specimen approx. 20g (see Figure 2)				
Dry mass of specimen (g)	16.67	15.62	-	-
Moisture content (%)	21	23	-	-
Organic Matter lost after PT (g)	0.27	0.26		
Organic Matter lost after PT (%)	1.62	1.66	-	-
Total dry mass ≥63μm (g)	1.62	0.07	-	-
Total mass for hydrometer test (g)	14.78	15.29	-	-
Percentage clay (passing 2μm) (%)	37	53	-	-
Dispersion (% clay A/B)	70		-	

Notes:
Samples A and C – without mechanical shaking and dispersant
Samples B and D – with mechanical shaking and dispersant

Test 1 - Original mass of specimen approx. 100g

Figure 1 shows the PSD curves for BHA 11.5m Samples A to D, where samples A and B correspond to pretreated samples without dispersant and mechanical shaking and with dispersant and mechanical shaking, respectively. It can be seen that the curve flattens within the silt region for Sample B, indicating the occurrence of hindered settling of silt. Hindered settling is the reduction of sediment settling velocity at increasing sediment concentration due to grain interactions (Te Slaa et al, 2012). This was also observed on a localised level (within the medium silt region) in Sample A. The readings in Samples A and B are therefore considered inaccurate, as indicated by the estimated derived dispersion of 100.

The pretreatment percentage loss of organic matter found for Samples A and B are 3.3% and 5.1%, respectively. Based on BS EN ISO 14688 (BSI, 2018), soils with organic content of <6% are considered to be "low" in organics.

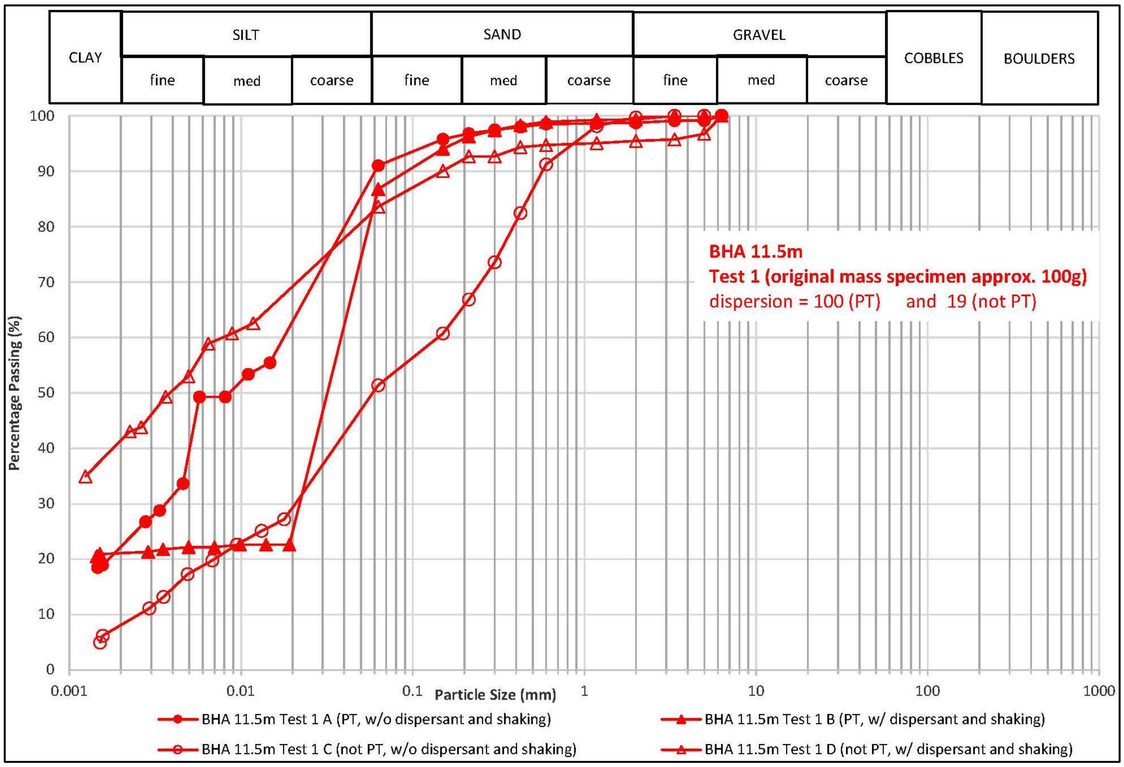

Figure 1. Plot of double hydrometer test results for BHA 11.5m Test 1 (original mass = 100g, mass for hydrometer test = 40 to 72g)

Test 2 - Original mass of specimen approx. 20g

To resolve the issue on hindered settling, various amounts of similar soil specimen were tested. It was observed that the hindered settling on Test 1 Sample B could only be avoided by using an original soil specimen mass of 20g (resulting in the mass of soil sample tested in the hydrometer being 15g, which is less than the baseline value recommended by Head, 2006). Figure 2 shows that the dispersion ratio from Samples A and B is 70 and an organic matter content of <2% was lost during the pretreatment. This test demonstrated that the degree of dispersion of soils determined using the Double Hydrometer Method is sensitive to both the amount of soil being tested and by pretreatment.

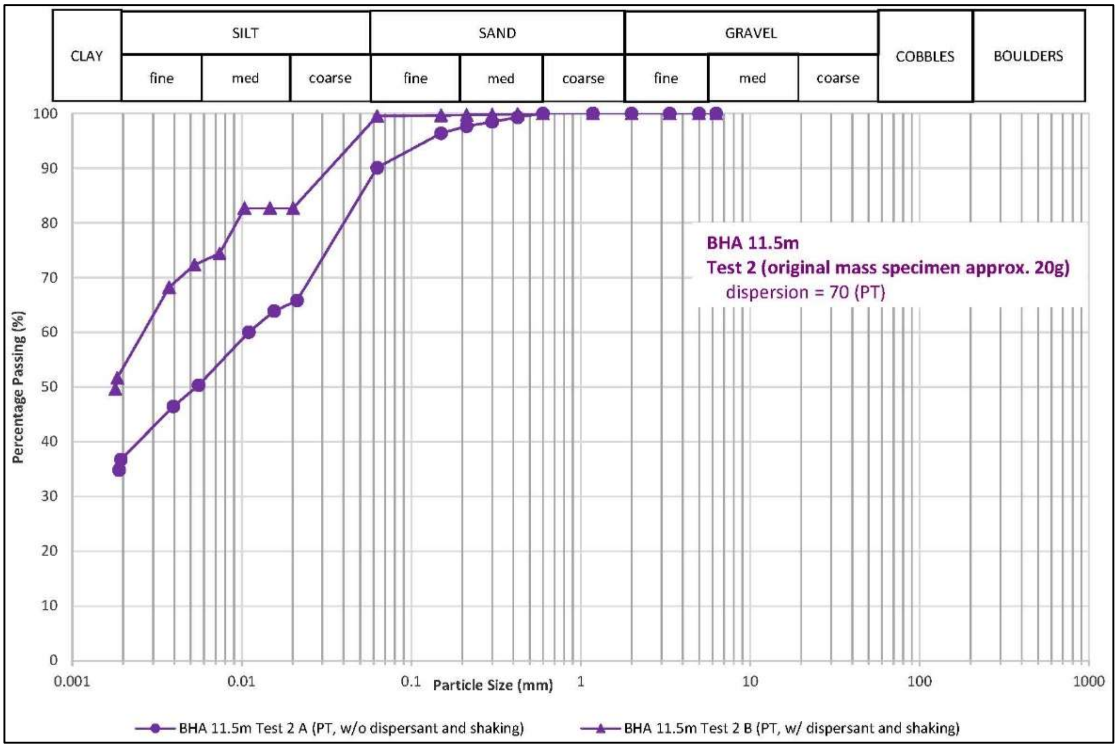

Figure 2. Plot of double hydrometer test result for BHA 11.5m Test 2 (original mass = 20g, mass for hydrometer test = 15g)

Test for BHB 6.5m

Table 2 and Figure 3 show a comparison of the amount of clay measured for the not pretreated samples in BHA 11.5m (Samples C and D) and BHB 6.5m (Data A and B). The amount of clay in Samples C and D are almost the same amount as those in Data A and B. Furthermore, the degree of dispersion measured (19 and 14) are also almost similar for both datasets with both results suggesting non dispersive soils.

Figure 4 shows that the samples with mechanical shaking and dispersant (Sample D and Data B) follow a similar trend. Although the other two samples, Sample C and Data A, do not show a similar trend, the amounts of clay measured in these samples are similar at 8% and 5%, respectively.

Table 2. Comparison of Double Hydrometer Test Results for BHA 11.5m and BHB 6.5m

Properties	**BHA 11.5m (Original mass of specimen approximately 100g, Not Pretreated)**		**BHB 6.5m (unknown mass, unknown if Pretreated or Not Pretreated)**	
	C	**D**	**Data A**	**Data B**
Percentage clay (passing 2 μm) (%)	**8**	**42**	**5**	**37**
Dispersion (% clay A/B or C/D)	**19**		**14**	

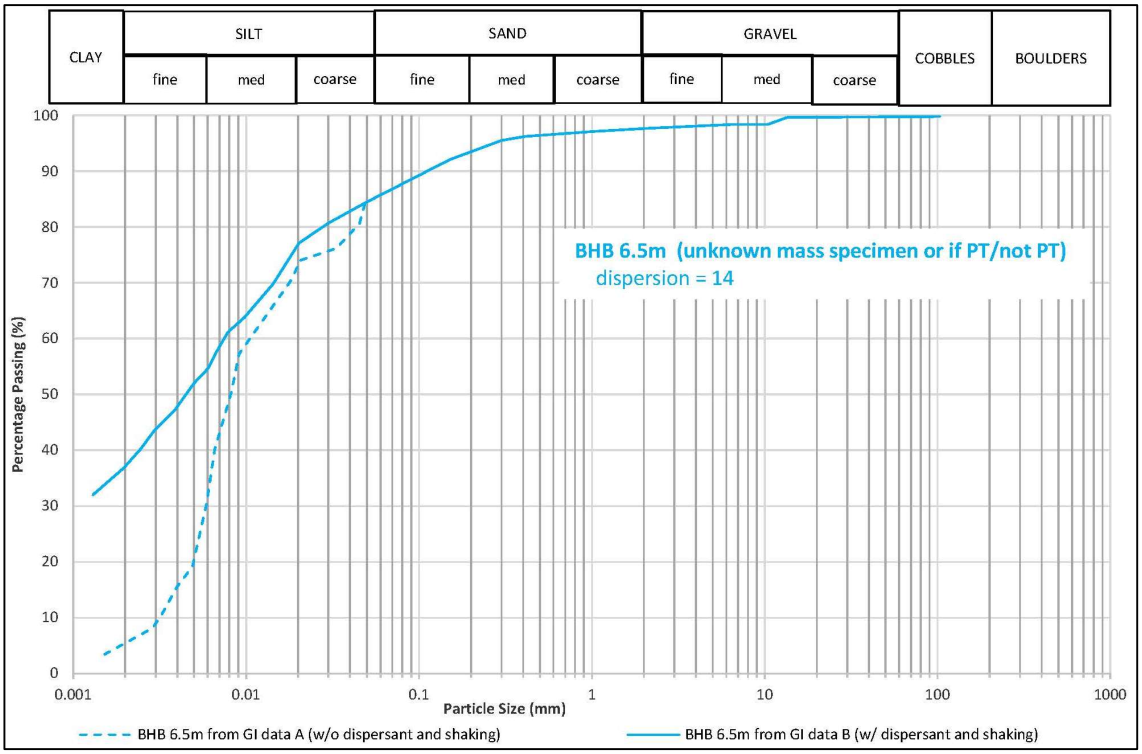

Figure 3. Plot of double hydrometer test results for BHB 6.5m (unknown mass of specimen and unknown if sample was pretreated for organic matter content)

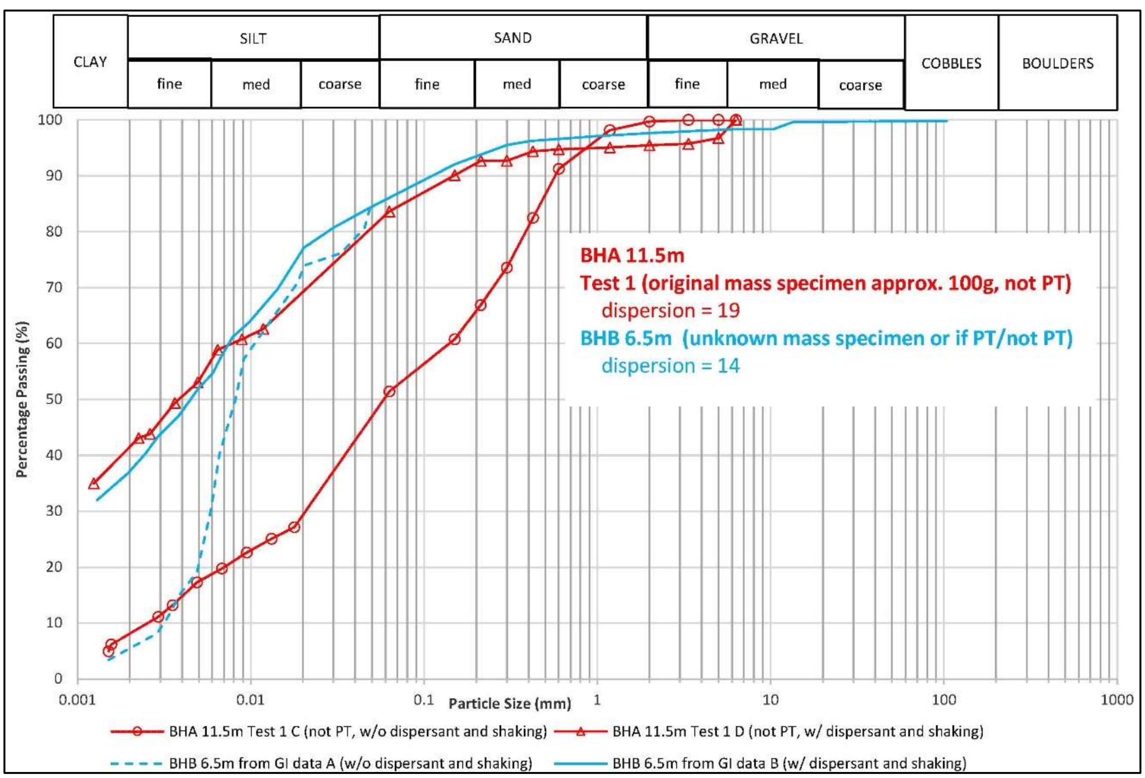

Figure 4. Comparison of plots of double hydrometer test results for BHA 11.5m and BHB 6.5m

DISCUSSION

On the basis of the above, it can be assumed that the Hydrometer Testing on BHB 6.5m undertaken by the others during the Ground Investigation in Reservoir X did not undergo pretreatment of organics, showing a degree of dispersion of 14 that indicates the soil tested is non dispersive (Degree of Dispersion <30). In contrast, where soil tested is pretreated for organics (BHA 11.5m Samples A and B using 20g original mass specimen or 15g soil for hydrometer testing), the degree of dispersion was found to be 70, classifying the soil as highly dispersive (Degree of Dispersion >50).

Organics may act to inhibit dispersion, but their presence can be highly variable within a dam. The treatment of soil to remove organics in the clay prior to hydrometer testing ensures that the underlying nature of the soil is revealed. It should be noted that six organic content tests (test standard not specified) were undertaken by the GI contractor on clay cores taken from other boreholes on the crest of Reservoir X, ranging from 1.4 to 4.8%. The measured organic content in BHA 11.5m for the 100g soil specimen ranged from 3.3% to 5.1% and about 1.7% using the 20g soil specimen. The variation in the percentage loss can be attributed to the heterogeneity of the soil, such that sampling a larger amount of soil will possibly include more organics from the bulk sample, while soil with smaller samples may be highly variable in general.

CONCLUSIONS AND ONGOING RESEARCH

The amount of soil tested for Double Hydrometer Testing should be sufficient enough (not too high and not too low) in order to provide the best dispersion test results. It may be necessary to conduct several hydrometer tests in order to establish what the most appropriate amount of soil is, in order to avoid hindered settling on the one hand, and a generally poor result through lack of soil on the other. The use of pretreatment should be routine, even where soil organics are found to be low. This is because, while the presence of organic material may reduce the dispersivity of a clay, its presence may be highly variable within a dam.

Further investigations are currently being undertaken by the authors on core samples, shoulder fill and natural soil samples from various reservoirs, predominantly in Yorkshire and Northumbrian regions, to better establish the criteria for dispersion; these include physical dispersion tests (Crumb, Pinhole and Double Hydrometer) and chemical dispersion tests comprising X-Ray Diffraction (XRD) and determination of Total Dissolved Solids (TDS). The erosion properties of soils that will be identified using these various methods will be further investigated through a Hole Erosion Test (HET) apparatus currently being constructed in the university where the authors are affiliated. The erosion rate index obtained from HET will give a guide to how quickly a pipe will develop in a dam.

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The British Dam Society

Managing risk at Victoria Service Reservoir

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SYNOPSIS Victoria Reservoir is a reinforced concrete service reservoir located in the heart of Bristol. Constructed in 1914, it is one of the oldest reservoirs of its type in the UK. It was constructed on the site of an earlier open service reservoir. During the second world war, the reservoir was damaged and repaired on account of bombs being dropped on it during the Bristol Blitz. In more recent decades, a series of investigations and repairs have been carried out to assess and maintain its structural condition and operational performance. This paper will describe and discuss the various challenges faced by the operator in managing the safety and operational risks associated with a very old reinforced concrete service reservoir.

INTRODUCTION

The design and construction of service reservoirs in the UK has greatly changed over the last 150 years with an ever-increasing focus on maintaining the quality of the stored potable water. Victoria Reservoir in Bristol is one of the UK's oldest active service reservoirs. This paper looks back over its history and discusses the present-day challenges in continuing its operation and ensuring its compliance with safety and water quality regulations.

HISTORY

The reservoir is believed to date from 1848. Plans from 1877 show that Victoria Reservoir started its life as a rectangular open reservoir formed with a lining of puddle clay and masonry. Figure 1 shows the original reservoir. The reservoir received water from Barrow Treatment Works to the south of Bristol and pumped it to the original Durdham Down Reservoir near Clifton. Victoria Reservoir was converted to a covered twin-cell concrete service reservoir in 1914. The total capacity was and remains approximately 30,000m^3. The design followed the Mouchel-Hennebrique system of ferrocement, an early form of reinforced concrete. This utilised a cement mortar matrix and layers of small diameter wire mesh in combination with more traditional steel reinforcement bars. The construction was monolithic and the walls were constructed encastre with the roof slab. Elements of the original reservoir construction were retained including the overflow/washout shaft and the underdrain system. The original overflow shaft can be seen in Figure 2. The side walls are relatively thin at 150mm, supported internally by counterforts. The side walls were backed with puddle clay but there is no back-of-wall drainage system.

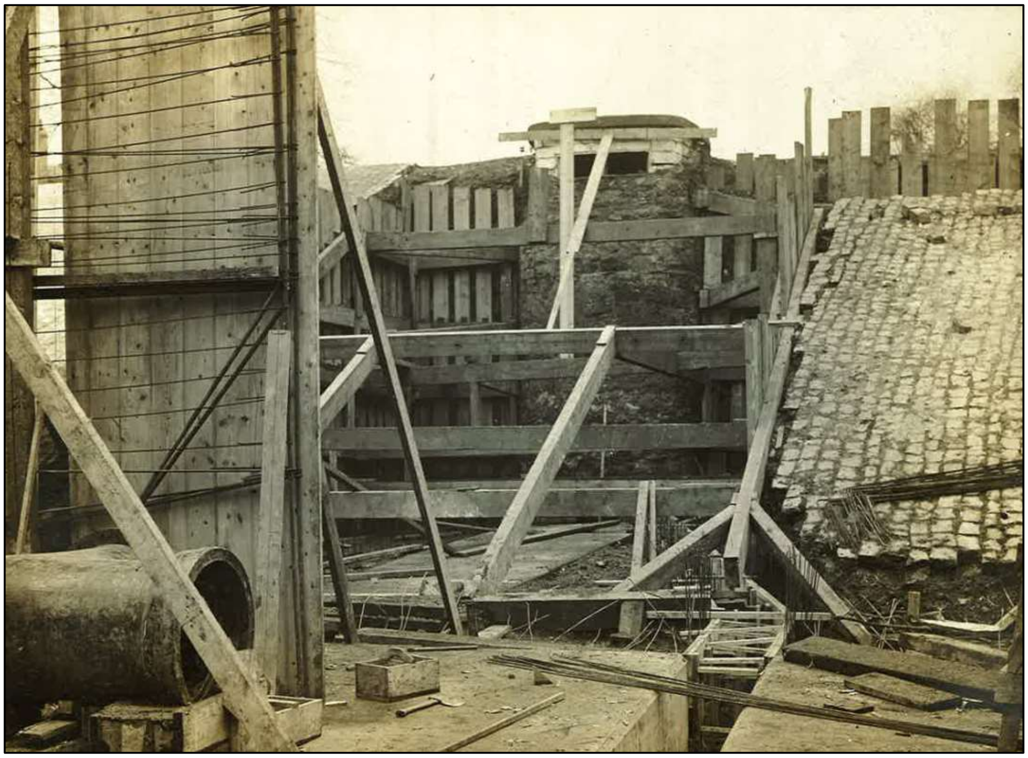

Figure 1. Original open service reservoir with the Engine House and Boiler House in the background.

Figure 2. Reservoir conversion works in progress showing the original overflow shaft and reservoir lining.

The reservoir was damaged during the Second World War. During the Bristol Blitz (1940-41), bombs damaged both compartments although the extent of the damage was not significant and the reservoir was repaired. One area of damage was sustained in the northern compartment and two areas of damage were sustained in the southern compartment. Drawings from 1949 indicate that the adjoining pumping station was reconstructed following the war.

Some remedial and improvement works have been carried out, particularly over the last 50 years. A bitumen liner was applied to the roof slab to reduce infiltration. Roof vents were removed to reduce contamination risk. Concerns raised regarding the structural condition of various concrete elements in the late 1980s led to investigations. Major repairs were carried out in the mid-late 1990s to some of the concrete roof beams and columns. The southern compartment floor slab was thought to be leaking at this time. The internal concrete walls, floor and column surfaces were treated with Flexcrete Cementitious Coating 851 to reduce leakage and to help arrest concrete deterioration.

The reservoir is in a highly urbanised part of Bristol and presents a high hazard to local residents. It is a statutory reservoir regulated under the provisions of the Reservoirs Act 1975.

CONCRETE DETERIORATION

In parallel with a statutory inspection of the reservoir in 2013, an investigation of the roof structure of the northern chamber was carried out. The roof slab soffit displays numerous cracks, most of which have 'healed' through calcite deposition. Cover meter readings to the roof beam reinforcement gave mean values between 17-36mm. Schmidt hammer testing of the original concrete indicated compressive strength values in excess of 40 N/mm^2 but much lower values for some areas that had been repaired. The risks of concrete deterioration through carbonation, chloride-induced corrosion, sulphate or chemical attack or freeze-thaw actions causing degradation in the form of cracking, spalling, delamination and surface softening or erosion were evaluated. Carbonation was considered the primary mechanism of deterioration, but the rate of deterioration will likely have been arrested by the high moisture conditions within the reservoir. Cracks in the roof slab were primarily attributed to thermal movement. Local areas of spalled concrete on roof beams were attributed to a loss of the protective passivation layer on the steel reinforcement through carbonation. This protective film is formed as a result of the high alkalinity in the cement paste but becomes unstable when the pH decreases or the film is destroyed through contact with chlorides. Chlorides can be present in concrete as calcium chloride was a common accelerating admixture during cold weather concreting from the end of the 19th century until the 1970s. The investigation concluded that with an appropriate proactive maintenance regime, the residual operational life of the reservoir roof should exceed 50 years (to c.2060), giving a projected overall service life of approximately 150 years or more.

Figure 3 shows an image of the inlet pipework and concrete from the time of construction and a similar image taken in 2023. It can be noted that the concrete elements generally remain in very good condition after more than 100 years in use.

Figure 3. Southern wall and inlet pipework in 1915 (above) and 2023 (below)

CRACK MONITORING

Crack monitoring within the reservoir is difficult given that access is infrequent, typically every 2-3 years, time-limited as there is pressure to return the reservoir to service quickly, and carried out in low light conditions.

Formal recording of cracking patterns on the internal walls, floors, roof slabs and beams started in 1995 and was undertaken manually by sketching the extents of larger cracks onto hard copies of printed drawings. This process was improved from 2007 with the manually sketched cracking on site later transferred to CAD record drawings when back in the office. A different colour of CAD layer was used for each survey so change could be detected over time. Differences in the personnel, the viewing locations and the lighting during these infrequent inspections meant the results were indicative only, but it allowed the undertaker to track the behaviour of the larger cracks.

In 2019, improvements in technology allowed trialling of 3D laser scanning of the interior. This has several benefits in that it is rapid, covers all areas (floors, walls, roof) in one go and does not rely on good lighting for results. It is also repeatably consistent and produces a large amount of digital data that can be interrogated later. The remaining issue is that manual review of the data and logging of the cracks is still the most reliable way of recording the results. Investigations into the use of an artificial intelligence (AI) engine is being explored at present for the automatic interpretation of the scan data and subsequent change detection when comparing to previous scans.

THERMAL EXPANSION OF THE ROOF

Inspection of the northern compartment in 2013 revealed fresh horizontal cracks through some of the internal buttresses. It was speculated that exceptionally hot weather in Bristol in 2006 may have instigated the cracking through thermal expansion of the roof slab. The roof slab has a surface layer of 100mm of gravel and 75mm of grassed topsoil above the bitumen membrane. Instrumentation of the compartment was recommended in the interests of safety to better understand how the cracking may have occurred and whether the crack widths are increasing over time. In 2017, a number of tilt beams and strain gauges were installed (Figure 4).

Unfortunately, many of the instruments failed to perform well on account of the conditions within the reservoir affecting the electronics, particularly the high humidity levels and chlorine off gas above the water level. Data sets were obtained over a four-year period to 2021 before the instruments had to be abandoned.

As would be expected with roof expansion forces being transmitted into side walls, greater strain values were detected towards the top of the wall buttresses than at cracks lower down the buttresses. Tilt beam readings were also greater near the roof. Actual deflections across the cracks were however quite modest with the greatest values being less than 0.5mm and more generally the readings were less than 0.2mm. The variations in strain did not correlate well with changes in reservoir water level, indicating that thermal gain is the primary driver for the wall cracking. The investigation results, reviewed as part of the 2023 statutory inspection, gave no immediate concern for the safety of the reservoir although some form of roof insulation may be considered by the operator going forward, especially in light of climate change.

Figure 4. Strain gauge installations across two new horizontal cracks extending partly through an internal wall counterfort.

SEEPAGE MONITORING

Seepage monitoring at the reservoir is restricted by the original design provisions. There is a single underfloor drain serving each of the two compartments. These drain to the base of the overflow shaft so combined flow readings are monitored except when one compartment has been emptied. Mean annual underdrainage flows increased nine-fold between 1999 and 2010 but the trend did not continue and has since partially reversed. The increase was most likely attributable to leakage past the washout valves into the base of the overflow shaft where all drainage is directed, including roof drainage. This highlights the challenges associated with monitoring reservoir performance where there are not separate monitoring provisions for each drainage system. The reservoir features no back-of-wall drainage system. Seepage into the surrounding embankments would likely be limited by the puddle clay backing to the walls. The stability of the surrounding embankments is generally managed through regular surveillance for any wet spots at the toe.

There is a system of perimeter drains which do not specifically serve as toe drains but could receive flow in the event of reservoir leakage. These date from the original construction. They are difficult to survey but some information on connectivity has been gained through flow testing.

PRESERVING WATER QUALITY

Whilst creating a covered water retaining structure was a huge step forward for water quality, the new (1914) covered structure used Gatic covers which were neither weather or insect resistant. Air vents were installed, again without insect mesh along both walls. Material access covers of concrete planks were weather-proofed with lead sheet under a soil covering. Water sampling was not even considered.

Over the years the following measures have been added and improved through Technical Guidance Notes by the Drinking Water Inspectorate (DWI) and Public Health England (now UK Health Security Agency), Security and Emergency Measures Direction Policy and good industry practice:

- Covers have changed from Gatic to GRP, to now double skinned tamper-monitored covers in sight of CCTV.
- The pump station changed from steam powered to electric in 1956, requiring installation of a deeper outlet sump and new outlet main.
- Bristol Water opened a laboratory for water samples in 1963.
- The roof was stripped of topsoil, the seal on the material access covers improved and the whole roof covered with Bituthene membrane in 1977; the heaviest item of plant allowed on the 75mm thick roof slab being a wheel barrow.
- A level recorder house was added and removed, to make way for a level control kiosk and sampling kiosks.
- In 1989, the DWI was formed and weekly water sampling from Potable Water Structures was enforced.
- Air vents were removed, covers changed to galvanised steel with ventilation apertures with improved seals and insect mesh.
- In 2002, the Bristol Water laboratory closed and water sample testing was contracted out.
- Overflow weirs were covered with hinged flaps and insect mesh.

CONCLUSIONS

Victoria Reservoir is an example of a very old reinforced concrete reservoir formed on the site of an even older open reservoir which has provided potable service storage for over one hundred years and is likely to do so for at least another 50 years. Like many such reservoirs, it is located within a community so the safety of the reservoir is of paramount importance. The age of the reservoir presents numerous challenges in maintaining the quality of the stored water and in monitoring the structural condition. Modern technologies have been deployed to better understand the nature and magnitude of movement in the side walls and in monitoring any new indications of structural deterioration. The structures are now inspected using a risk-based approach, with both the structural and water quality conditions assessed, with these criteria setting the internal inspection frequency to two, four or six years with allowance made for Section 12 and 10 safety inspections to occur within these drain-down periods. Over its remaining service life, the condition of the structure will demand a more proactive approach to maintenance. The reservoir has survived the impact of time, a bombing and increased regulatory standards. In modern times, climate change appears to have caused some minor deterioration of the structure. Nevertheless, with an appropriate maintenance regime the reservoir appears set to provide many more decades of service to the people of Bristol.

The British Dam Society

Case studies from challenging pipes and valves works

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SYNOPSIS With current UK dam stock ageing and infrastructure meeting or surpassing its intended asset life, critical maintenance and replacement of key pipework and valves becomes necessary. The design and construction of historic assets may not have considered aspects such as ease of operation, maintenance and replacement. This paper provides case studies of recent works completed with particularly challenging environments, from projects in Wales.

Based upon multiple examples of physical projects undertaken, this paper will look into the constraints, planning, decision making involved leading up to and executing improvement works, along with the temporary works, permanent works and commissioning. The intention of this paper is to share the learnings taken from these works, which may be of use to others in the industry.

The client for the schemes presented was Dŵr Cymru Welsh Water (DCWW), and the Principal Designer and Principal Contractor was Mott MacDonald Bentley (MMB) and Edwards Diving Services (EDS) as the diving contractor.

LLYN CELYN RESERVOIR

Llyn Celyn reservoir is situated approximately 7km north of Bala in North Wales. The reservoir is formed by a gravel-fill embankment, 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 (ICE, 2015) and has a capacity of approximately 81,000,000m^3. The reservoir is owned and operated DCWW, but the water level management and releases are the responsibility of Natural Resources Wales (NRW) as part of the Dee Regulation Scheme.

MMB undertook investigations to assess the conditions of the valves on the site, and reviewed the drawdown capacity against the latest guidance (EA, 2017), resulting in the following works:

- Replace existing inoperable 60-year-old 36-inch butterfly valve (V5), with a 900mm gate valve, located approximately 300m into the dam tunnel. The discharge capacity of the 36-inch scour pipeline is circa 6.4m^3/s.
- Replace the existing 2Nr 60-year-old 52-inch fixed cone valves (M1 and M2). The discharge capacity of the 66-inch supply pipeline is circa 22.4m^3/s.
- Installation of two new drawdown facilities built into the primary spillway.

Managing Risks for Dams and Reservoirs

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 specialist diving contractors and Natural Resources Wales (NRW) to ensure the works could be safely completed, whilst minimising the risk to water resource.

Two main pipeline systems are in place at Llyn Celyn, the 36” line associated with the scour and the 66” supply line associated with river regulation and the hydro turbines.

36” Valve Replacement (V5)

The function of V5 is to act as a burst control valve; shutting down the system should the downstream valves or pipework fail. The asset life of the original mechanically operated butterfly valve had lapsed, and the decision was made to replace this with an electrically actuated gate valve. In order to safely replace the valve, temporary isolations upstream of the 2Nr existing gate valves (S1 and S2) were installed to avoid working under single isolation, following HSE guidance (HSE, 2006) regarding the safe isolation of plant. The works were planned alongside EDS who developed a temporary isolation arrangement using inflatable bungs connected to steel plates. These were installed via a floating pontoon (Figure 1) lowered through a 1.5m diameter diving shaft in sections and re-assembled at depths of approximately 30m. The existing gate valves formed the primary isolation; negligible leakage passed the valves. The temporary bungs formed the secondary isolation, whilst the steel plates (Figure 2) formed a tertiary isolation should failure occur of the bung and gate valve downstream. Schematics of this are shown in Figures 3 and 4.

Figure 1. Pontoon

Figure 2. Temporary isolations

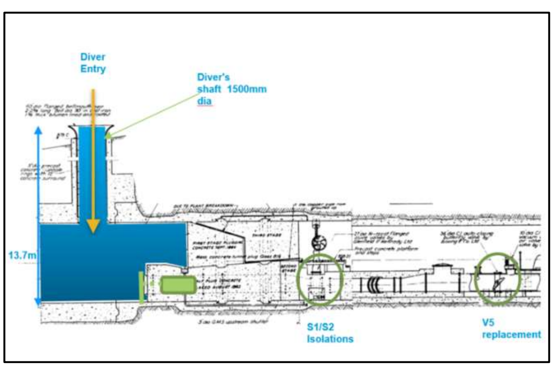

Figure 3. Section view of isolations

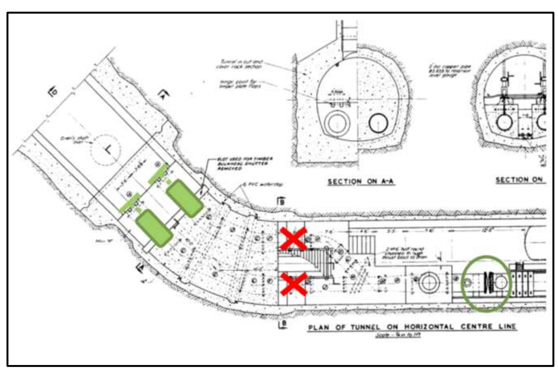

Figure 4. Plan view of isolations

Due to the location of the valve, additional temporary works and lifting arrangements were needed within the tunnel to facilitate the removal of the existing, and installation of the new valve/pipework arrangement. To facilitate this, a bespoke trolley and lifting gantry (Figure 5) system was installed by Mona Engineering, with the new gate valve weighing approximately 2.3 tonnes. The valve and pipework were lowered into the tunnel via an opening in the roof (Figure 6), transferred to the end of the tunnel on the trolley and lifted by the overhead gantry for the final 20m before being lowered into position and pipework connected (Figures 7 & 8). Given the constraints around isolation and diving, the works were carried out under Welsh Water's 'Gold Command' to monitor progress and resolve any identified issues. Upon completion of the works and the successful pressurisation of the system, divers removed the temporary isolations upstream.

Figure 5. Pipework removal

Figure 6. New valve being lowered being into tunnel

Figure 7. New valve installation

Figure 8. New pipework installation

52" discharge regulator valve replacement (M1 & M2)

Located at the downstream end of the 66" discharge system, the function of M1 and M2 is to act as terminal discharge valves, allowing flow regulation to the river. Similar to V5, the asset life these valves had expired and required replacing. The new 52-inch fixed cone valves were longer than the original valves and weighed over four tonnes. Given the size of the new valves and existing dimensional constraints, each valve could not be installed in its horizontal position. Installation via the stilling basin would have required emptying the stilling basin along with substantial over pumping, ranging between 1.5 to 16m^3/s to maintain statutory releases to the river. The project team worked with Mona Engineering to develop a bespoke lifting frame and methodology to lift and lower each valve into position with limited working room (Figure 9). The lift started with the valve in a horizontal position until it was within the building, transitioned to a 45-degree nosedive (Figure 10), before returning to horizontal as it was fixed to the upstream flange.

Figure 9. New valve installation

Figure 10. New valve installation

Drawdown enhancement valves

In order to enhance the drawdown capacity, two sets of two hydraulically actuated gate valves were installed, connected to new pipework through the primary spillway (drop shaft) wall, with trash screens at the intakes. The valves are fully submerged when the reservoir is at top water level and will be remotely operated by a hydraulic power unit (HPU) using a hand pump or petrol engine. The total discharge capacity of this system is circa 13m^3/s.

To enable the works a 9m high, 10m long scaffold was erected up and over the drop shaft spillway to provide access (Figure 11). With the works being within the reservoir basin, and within the existing spillway, the project team carefully considered the safety of the teams, the reservoir water level with NRW, and managed the risk associated with water resource.

All works were able to be undertaken during the period when the reservoir level was managed under NRW's Temporary Control Rules that were put in place to facilitate other works to construct an auxiliary spillway, and other valve replacement works described above.

Two 7m long 1.25m diameter cores were taken through the spillway to facilitate the pipework installation. A 70-tonne (Figure 12) crawler crane was used to lift the valves and lower them between the boat fender and spillway (Figures 13 & 14).

Figure 11. Scaffold installation

Figure 12. New valves and crane

Figure 13. New drawdown facility

Figure 14. New drawdown facility

LLANDEGFEDD RESERVOIR

Llandegfedd Reservoir is situated approximately 4km southeast of Pontypool. The reservoir is formed by an earth embankment dam across the valley of the Sor Brook which is a tributary of the River Usk and is quoted as having a capacity of 24,470,000m^3.

In order to enhance the drawdown capacity, a similar arrangement to Llyn Celyn was adopted, by the installation of three sets of 700mm rising spindle gate valves, installed at 6m below top water level, which discharge into a combined draw off / overflow tower (Figure 16).

The project was programmed around the annual drawdown of the reservoir. To facilitate the installation, taking account of a variable water level, a suspended scaffold (Figure 15) with lifting beams was constructed from the top of the valve tower to gain access to the working area. Barges were utilised to transfer the new valves and fittings to the tower.

The alternative solution to achieve the same output was to install large diameter siphons and run pipework to the downstream watercourse. Significant carbon and cost savings have been achieved through delivering this solution.

Figure 15. Scaffold arrangement

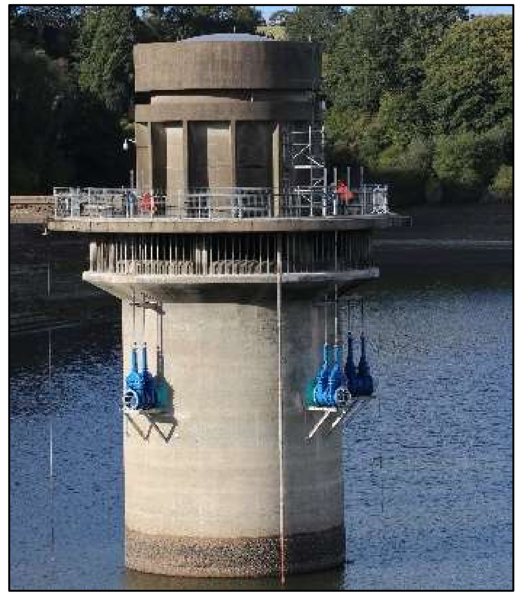

Figure 16. Installed drawdown facilities

USK RESERVOIR

Usk Reservoir is formed by an earth embankment dam, which completed constructed in 1955 with an approximate capacity of 12,268,000m^3. The dam is 480m in length, with a maximum height of 31m, and supplies raw water to Bryngwyn Water Treatment Works. The reservoir also provides compensation water to the River Usk which is classified as a Special Area of Conservation (SAC) and a Site of Special Scientific Interest (SSSI). The project focuses on the replacement of the reservoir draw-off pipework within the dam's outlet tunnel.

The project was to design and construct the replacement of both 18-inch scour mains in the 2.4m diameter horseshoe-shaped tunnel to improve emergency drawdown capacity and to provide a facility for enhanced releases to the River Usk. The works also included for 'enhanced releases' allow a range of discharges from the reservoir, with the aim of providing benefits to the River Usk and its habitats.

Optioneering

The historic pipework and tunnel characteristics caused a variety of constraints on the new system that needed to be considered when finalising the desired pipework arrangement. The project aimed to safely maximise the potential drawdown capacity whilst working within these constraints.

At the upstream end of the tunnel, the historic 18-inch pipes pass through a concrete plug, which could not be replaced without a full drawdown of the reservoir. Emptying the reservoir was not feasible due to Usk Reservoir supplying large volumes of raw water for supply and compensation purposes. As a result, the historic 18-inch pipe section formed a constraint on the design and construction of the permanent works.

A long list of options was developed, with the chosen solution to replace the twin 18-inch pipework with a single larger diameter pipe, offset to one side of the tunnel (Figure 16). This option maximised the outflow from the reservoir and maximised space for access, inspection and maintenance. To merge the two 18-inch pipe sections from the tunnel plug, an asymmetric manifold (Figure 15) was designed to combine the flow, with guard and duty valves upstream of this (Figure 17). Enhanced releases are provided by two flow control valves installed offline to the new drawdown pipework (Figure 18), that could be remotely operated using a telemetry system located in a new control kiosk. The tunnel is circa 190m long and has two 45-degree bends. To facilitate the construction and future maintenance, a screed was applied to the floor of the tunnel. A remote-controlled pipe bogie was utilised to move the pipe sections and valves to their final position.

Design considerations

Another consideration in the pipework design was to limit the flow velocity through the twin 18-inch sections of pipework and valves. If the system was operated for a prolonged duration with excessive velocities, there would be a risk of causing damage to the system through cavitation and excessive wear.

Figure 17. Manifold at bulkhead

Figure 18. Scour pipework and thrust block

Figure 19. Existing valves concreted in place

Figure 20. Discharge valves

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Following the feasibility stage, the pipework system was further optimised to improve hydraulic efficiency whilst maintaining velocities to a suitable level. The manifold was optimised to achieve balanced flows between both legs, to prevent significantly higher velocities within a single leg. The results of the optimisations enabled the diameter of the larger pipework to be reduced from 900mm to 800mm diameter, leading to a reduction in material costs and embodied carbon by approximate 10%.

For details around the siphon temporary works installed as part of this scheme, see parallel paper by Carruthers and McAree (2024).

PANT-YR-EOS RESERVOIR

Pant-yr-Eos Reservoir is situated approximately 2km east of Risca in the City of Newport, Monmouthshire. It is impounded by a 27m high, 280m long embankment dam with clay core, and has a storage capacity of approximately 0.6Mm^3. The reservoir was completed in 1878 for provision of water supplies to Newport.

Improvement works were required to allow safe passage of the safety check flood, remedial works to the masonry spillway, improvements to the emergency scour system and a new filtered drainage blanket on the downstream embankment toe with associated instrumentation.

Drawdown Study and Remedial Works

The existing draw-off system consisted of a wet masonry valve shaft located a short distance upstream of the dam, which is accessed by a steel footbridge. The valve tower includes an open approach channel with parallel masonry walls through the upstream shoulder. The masonry walls are propped by an array of iron props. A masonry culvert passes through the core and under the downstream shoulder.

Within the valve shaft, gate valves at four levels convey water from the approach channel into the wet tower. From the base of the wet shaft, a gate valve conveys water into a 450mm pipeline through the masonry culvert to an outlet headwall at the downstream toe of the embankment, where it continues to the decommissioned water treatment works. This pipeline is capped off downstream of the treatment works. The water level in the reservoir was controlled via a 150mm washout, branching off the pipeline prior to the treatment works. Only the gate valve on the washout was operable, with the bottom draw-off valve and the valve at the base of the wet shaft seized in the open position.

The scour system consisted of a short length of 300mm pipe from the base of the approach channel, through the wet tower, discharging into the masonry culvert passing through the dam, at the base of the concrete plug. The scour valves were inoperable and buried under circa 4m of silt.

A drawdown assessment was completed and proposed various options to improve the drawdown capacity to meet the published UK guidance (EA, 2017). The options considered to increase drawdown capacity were compared by considering the technical, system resilience, construction, cost, programme, environmental, carbon, operational, and maintenance risks and impacts. The chosen solution to increase drawdown capacity converted the historic supply main into the emergency drawdown system with provisions to re-configure for supply if required in the future. This included replacing the four gate valves at the interface between the approach channel and wet shaft (Figures 21 and 22), and the valve at the bottom of the wet shaft. The existing pipework through the culvert was maintained, and a 450mm washout provided at the toe of the embankment with a stilling basin prior to discharging to the watercourse.

The original emergency scour system was then discontinued, enabling major environmental benefits in the prevention of large volume of silt removal. The works were undertaken with a partial drawdown of the reservoir utilising a suspended scaffold to replace the top three valves and underwater works using divers to replace the fourth valve.

The drawdown study was undertaken in conjunction with an assessment of the slope stability of the upstream face under rapid drawdown conditions. The study aimed to provide rates that the reservoir can be drawn down safely during a routine operational drawdown and an emergency drawdown, to help inform operational procedures and emergency planning.

In order to complete the works, various isolations were required at different stages of the scheme in order to safely deliver the works. With the reservoir partially drawn down, the 150mm washout valve isolated, with an additional blank plate installed, the adjacent feed to the treatment works was tapped to prove the downstream isolation was effective. This enabled divers to safely produce a template of the lowest valve's bespoke flange, which was used to fabricate and install a blank plate. This subsequently enabled works within the wet shaft and the embankment toe to progress. Isolations to replace the fourth valve were provided by the new valve at the base of the wet shaft and the new washout valves, with the pipeline being isolated from the decommissioned treatment works.

For details around the control of the water levels during the construction period, see parallel paper by Carruthers and McAree (2024).

Figure 21. Scour valves and spindles

Figure 22. Access within valve tower

UPPER CARNO

The dam at Upper Carno reservoir is a single earth embankment dam approximately 15m high and 280m long, and it impounds the Ebbw River. The reservoir is believed to date from around 1875 and currently impounds 3,400,000m^3 of water. Works were undertaken to many aspects of Upper Carno; for further details please see parallel paper by Swetman et al. (2024).

The drawdown facilities at Upper Carno consisted of a wet tunnel that conveyed water approximately 70m to a valve tower located immediately upstream of the dam crest. The valve tower was a congested space (2m x 4m plan area), which was split in half with a cast iron wall embedded into the valve tower. This wall allowed for a 'wet' upstream side and 'dry' downstream side which housed a pipework stack and all the draw-off valves.

From the valve tower, water was conveyed through a short section of scour pipeline, where it would discharge directly into the downstream tunnel (Figure 23) when operated. The supply system would convey water through pipework located in the tunnel, until it was beyond the footprint of the embankment, where it would be directly buried to the downstream water treatment works. The tunnel would continue to convey the water from the scour pipeline to the spillway located downstream.

The works to refurbish the system included retaining the wet tunnel upstream of the valve tower and install a trash screen at the intake. To enable the drawdown of the reservoir for the works, temporary twin siphons were installed to draw-off the top levels of the reservoir, in conjunction with a pump arrangement to fully drawdown the reservoir. For further details on the temporary siphon system see parallel paper by Carruthers and McAree (2024).

The valve tower was converted into a dry tower by removal of the central wall, and the installation of a plug at the interface between the wet tunnel and shaft. The pipework stack and associated valves were all replaced within the shaft.

The existing tunnel immediately downstream of the valve tower, under the embankment, was 1.5m diameter and had significant water ingress and had begun to deform in shape (Figure 23). Therefore, it was lined with a 1m diameter pipe, with the annulus infilled with structural grout, which formed part of the new draw-off system. In order to enable the works to the tunnel, and to re-route the new draw-off pipework outside of the dam profile, a 7m diameter tunnel was sunk 11m through the embankment to intercept the tunnel to drive the pipework sections and tunnel the new pipework away from the dam (Figures 24 and 25).

Downstream of this shaft, a 2.4m diameter tunnel was driven to install the dam draw-off pipework to outside of the dam profile. From this point, the draw-off pipework was micro-tunnelled at 9m depth for 80m (Figure 26) and conventionally open cut for 60m to a submerged discharge valve and chamber adjacent to the receiving watercourse.

The draw-off works were completed, commissioned and received the MITIOS sign off for the associated recommendations prior to the statutory date.

Figure 23. Original tunnel

Figure 24. Shaft installation

Figure 25. Tunnel installation pre-infilling

Figure 26. Micro-tunnelling scour main

CWMWERNDERI RESERVOIR

Cwmwernderi Reservoir appears to have been constructed by 1901 and is situated 5km northeast of Port Talbot. The embankment impounds the headwaters of Nant Cwmwernderi, and is approximately 75m long, 23m high, and has a stated capacity of 159,000m^3.

The existing drawdown system at Cwmwernderi did not have reliable upstream control, or safe access to the valve tower, due to the condition of the valve tower, and associated access bridge. The original scour system consisted of a penstock that was in the closed position and inoperable. The supply system had a washout circa 1km downstream of the site and was limited to reducing the reservoir level to approximately 7m below top water level due to the lower draw-off valve being in the closed position and inoperable. The spigot socket pipework in the tunnel was installed circa 1911 and had no formal thrust restraint at the bends. The drawdown capacity with the supply pipework did not meet drawdown guidance (EA, 2017).

The scheme to remediate the lack of upstream control, the valve tower and access bridge, and drawdown capacity was planned to be delivered in two phases. The first phase of works consisted of providing a new outlet near the toe (Figure 27) of the embankment to convert the historic supply pipe into a scour pipe, and to provide thrust restraints (Figure 28) to the existing pipework within the unlined rock tunnel. The existing unlined rock tunnel varies in shape and diameter, reducing to around 1.2m high in places.

Figure 27. New scour outlet

Figure 28. Thrust restraints

LLYN BRENIG

Llyn Brenig is located in the county of Conwy around 15km south of Denbigh, north Wales. The reservoir feeds compensation flows to the River Dee and is a critical asset to the Dee Valley Consultative Committee in unison with Llyn Celyn and Llyn Tegid. The reservoir has a stated volume of 61,550,000m^3 and is impounded by a 50m high rockfill dam with a 1200m long crest length, constructed in the 1970s.

The scheme included the scope below. For further details, see paper by Carruthers et al. (2024).

- Replacement of the "goliath crane" mounted to the top of the combined draw-off and overflow tower.
- Installation of a new secondary isolation gate
- Replacement of the scour bulkhead gate
- Replacement of the primary scour gate
- Replacement of all gate control systems including new control panel and caballing

CONCLUSIONS

Careful consideration, consultation and planning is essential for complex pipework and valve systems refurbishments to existing dam infrastructure. Defining a suitable methodology to undertake the works safely and quickly, while working within the constraints of a given scenario is essential. Involvement between asset owners, permanent works and temporary works designers, contractors and specialist subcontractors is seen as essential as early as possible to the planning, programming, pricing and stakeholder management required to successfully execute such complex projects.

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The British Dam Society

A Field Monitoring Data-Driven approach to Dams and Reservoirs: Risk Reduction Through Predictive Maintenance

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SYNOPSIS The challenges associated with reliable assessment of the conditions of geotechnical and structural elements of ageing dams and reservoirs are becoming more complex and critical, due to the combined effects of Climate Change and the need for optimised and sustainable maintenance (and construction) solutions. The paper focuses on how a quantitative understanding of the current behaviour of dams and reservoirs via field monitoring can help overcome such significant challenges. The paper presents a general approach to the monitoring of geotechnical and structural elements; it also discusses the use of specific technologies for the monitoring of some fundamental parameters of interest for dams and reservoirs. The use of field monitoring data for risk reduction and maintenance optimisation purposes revolves around meaningful and trustable field data (and metadata) as well as the robustness and durability of the monitoring system as a whole. The paper discusses the importance of high-quality field instrumentation, high-quality installation and high-quality data analysis, alongside the importance of the role and involvement of a Monitoring Specialist. Finally, the paper discusses the potential of using Digital Twins to help the interpretation of the field monitoring data and provide an assessment of the assets via numerical models (e.g. finite elements models, finite differences models, etc.) which, via Artificial Intelligence tools, can enhance predictions on the basis of field monitoring data.

INTRODUCTION

The use of monitoring data from instrumentation installed on existing dam and reservoir assets for asset assessment purposes is not new. During the course of the last fifty years, reservoirs have been recognised as strategical and complex assets to which is associated a high level of risk due to the potentially disastrous consequences of an incident. However, it should be noted that too often the attention (and the monitoring systems) is concentrated on the body of the dam (whatever its nature) rather than on all the potentially critical assets which constitute a dam and reservoir assets, e.g.:

- dam (arch, gravity, earth, rockfill, etc.),
- natural slopes enclosing the artificial water body,
- transitions between the dam and the surrounding natural features,
- penstocks,
- tunnels,
- M&E

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The development of powerful numerical tools supported by Artificial Intelligence techniques can unlock significant benefits when combined with field monitoring data. In such a data-driven approach the field monitoring data are used to "train" the numerical model and continuously increase the reliability and the accuracy of the predictions. These in turn can provide a powerful tool for the optimisation of maintenance planning and maintenance interventions.

This approach is the so-called "predictive maintenance" and is currently being applied in Europe mainly to bridge structures. However, the aforementioned techniques and concepts are completely asset-agnostic. These can be applied to any asset, provided the numerical models are sound and the field monitoring data are reliable and of high-quality. The need for a data-driven approach had its roots in the following main factors, which cannot be captured by the current assessment approaches:

- ageing assets,
- effects of Climate Change,
- sustainability (through optimised maintenance strategies).

The first two present the challenge of the unknown, whilst the third one can only be faced effectively as the solution of an optimisation problem. As such, in all cases the solution must rely on data acquired from the field which can shed light on the current status and behaviour of an asset and its evolution under changing conditions. As such, it can be inferred that reliable and adequate field monitoring data are (or can be) a key component of the endeavour to overcome the aforementioned challenges.

There is obvious potential in using Instrumentation and Monitoring (I&M) data to improve the understanding of the behaviour of existing assets, especially when seeking optimisation in terms of asset maintenance.

However, in the very same way as any asset modelling technique (analytical, numerical or other) relies upon the reliability and quality of the input parameters, any data-driven approach relies upon the reliability and the quality of the monitoring data and of the associated metadata. The principal aim of this paper is to discuss the main concepts and challenges that should inform the definition ad the deployment of a monitoring system (and an associated data dissemination software) which is able to provide data (and metadata) which are a) reliable and b) of an adequate quality. As will become clear in the following sections, such targets can be achieved only if all the interested parties (asset owner, consultants, field monitoring specialist) recognise the highly technical and complex nature of all the field monitoring activities (definition, deployment, data management and validation, etc.) and are engaged in a cooperative effort.

It should be recognised that a monitoring system fit for asset management and maintenance purposes should not be seen only as a system able to "ring alarm bells" in emergency conditions. The main purpose of such system should actually be to provide:

a) an accurate understanding of the evolution of the parameters of interest for an asset far before any adverse effect produces visible damage and

b) a significant amount of time and quantitative information (i.e. data and metadata)

so that:

i. measures can be taken early on to avoid reaching an emergency condition;

ii. asset maintenance schedule can be optimised;

iii. asset maintenance solutions can be optimised and their sustainability increased.

These concepts are presented graphically in Figure 1, where the exemplar evolution in time of a generic parameter of interest is compared with an exemplar associated curve which presents the increase in risks and associated maintenance/remediation costs as the value of the parameter of interest evolves toward a "critical value". In the figure, the "critical value" is reached at time TC. If the damage becomes visible at time TB, then the difference TC-TB represents the amount of time available for maintenance/remediation if no adequate monitoring system is in place. If TA is the time when an adequate monitoring system becomes operational, the amount of time available for maintenance/remediation is represented by the difference TC-TA. As such, the scope of an adequate monitoring system should be to provide reliable and meaningful data within the amount of time represented by the difference TB-TA. It should be stressed that TB and TA are influenced by the actions (or the lack of action) from time TA, so that an adequate use of the field monitoring data can be an effective way to prolong the operational life of an asset (e.g. a reservoir) while minimising the risks and allow the optimisation of a maintenance schedule and maintenance interventions while optimising costs/resources and maximising sustainability.

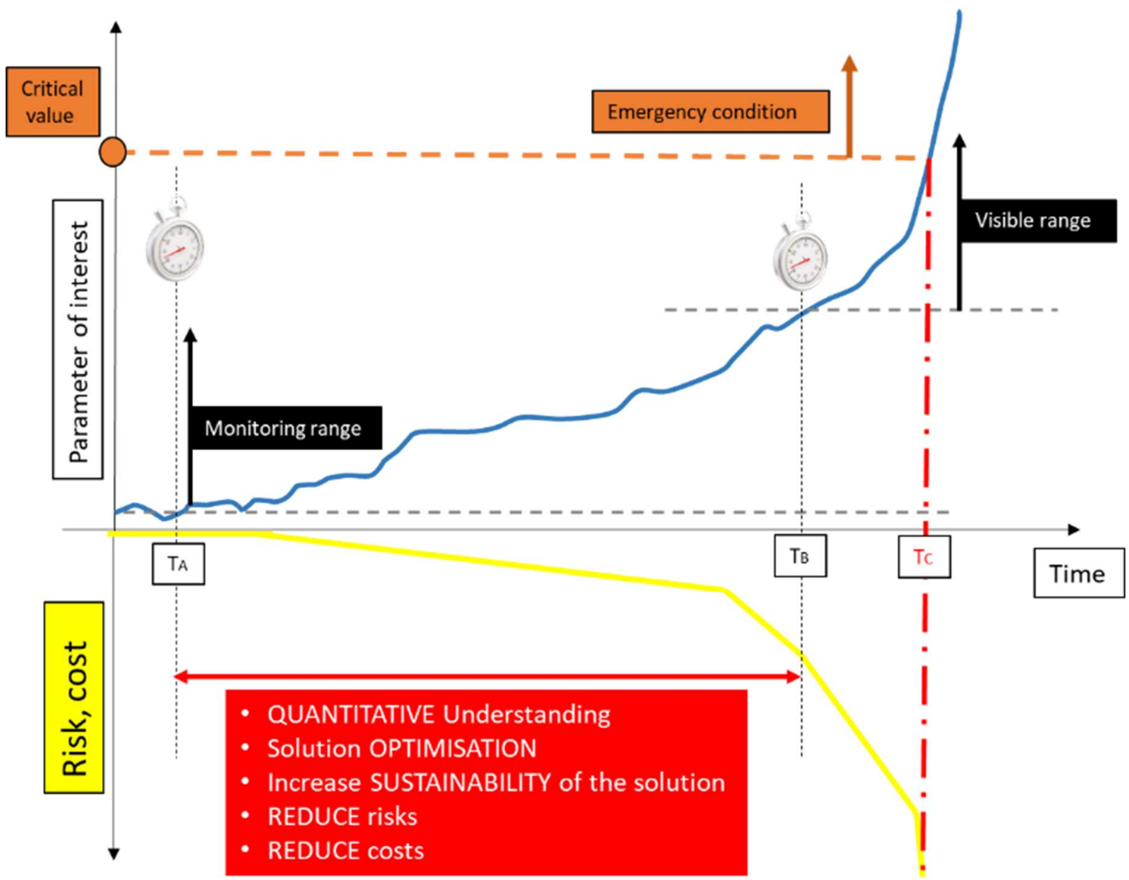

Figure 1: schematic representation of the evolution of a parameter of interest, the associated risks and costs and the benefits of the installation of an adequate field monitoring system.

PARAMETERS OF INTEREST

The parameters of interest of an asset (or part of it) are those quantities which are deemed critical to understand its current status, its behaviour and to predict future evolutions. Such parameters can or cannot be directly measurable, and it should always be assumed that they are asset-specific (and as such, it must be assumed that the development and deployment of a meaningful monitoring system is asset-specific too). Asking for a monitoring solution for "a dam", "a slope", or "a penstock" should not be regarded as a meaningful requirement.

Although similarities and previous experiences can always be beneficial, each existing asset has its own location, its own history (including construction history), its own boundary conditions, its own materials, crack patterns, specific risks, etc. As such, the definition of the parameters of interest for each asset should be the result of a dedicated analysis. This in generally true for all types of assets, and is particularly critical for reservoirs. The idea of monitoring reservoirs can often be confused with the concept of monitoring the dam. However, the reservoir should be always seen as a complex system which involves not only the dam itself but also the natural slopes surrounding the retained water body (which are for instance subject to significant and periodical changes of the hydraulic boundary conditions), as well as the influent watercourses (where applicable), the penstocks, the transition between natural slopes and the dam, the tunnels, the galleries, etc. All of these components have their own parameters of interest, which should be assessed on a case-by-case basis if an adequate field monitoring system is to be developed.

One of the fundamental aspects from an I&M perspective is that the result of the monitoring of a parameter of interest is a "discrete" time-series of (scalar) values which are measured by a real instrument at the location where it is installed. Therefore, the very first step towards a meaningful set of data must be a very clear definition not only of the parameter itself, but also of a number of other requirements associated with the "discrete" and "real" nature of the results provided by the monitoring system. It will be shown in the following sections that such information is a fundamental initial step towards the definition of the constituents of an optimal I&M system, i.e. the right instruments, the right installation methodology, the right communication system, the right software and the right maintenance arrangements. The requirements include (but are not limited to, depending on the specific application):

a) which parameters are to be monitored (e.g. strain, crack width, displacements, displacements, groundwater pressure, water pressure within penstocks, temperature, tilt, surface water velocity for open channels, vibrations, water levels, vertical and horizontal displacement, etc.).

b) where are the above parameters to be measured (e.g. in which location along the slope or the dam, at which depth underground, etc.).

c) what is the expected range of the parameter value (e.g. the expected displacement of the slope, the expected range of water pressure, the expected deformation of the dam, etc.).

d) what are the specific regulatory (or acceptable) limits/alert values for the parameters of interest.

e) what is the required acquisition frequency (e.g. 1 reading per hour, 1 reading per day, 1 reading per month, how many Hz in case of dynamic measures, etc.).

f) what are the requirements around the in-situ precision and accuracy.

g) the duration of the baseline monitoring period.

h) what kind of redundancy is required, if any.

i) the expected duration of the monitoring (e.g. a few months, several years, etc.).

j) the required metadata to facilitate data analysis and interpretation.

In respect of point f) above, the in-situ precision can be defined on the basis of a semi-probabilistic approach framework (Fornelli, 2022). One of the fundamental features of such a framework is the definition of two different sets:

I. the set of parameters of interest (X_T) for a specific project, and

II. the set of measurable parameters (X_M), that is, the set of parameters that can be directly measured with appropriate instrumentation.

In general, the elements of the set X_T are functions of the elements of the set X_M, where the functional relationship depends on the choice of the instruments and the monitoring set up.

A “trigger value” T_{XT} can be defined as a specific value of one of the parameters within the set XT. It is assumed that the monitoring data (elements of X_M) are normally distributed. For each adequate set of measurements, a mean value (μ) and a standard deviation value (σ) can be calculated (Taylor, 1982). It is then possible to define the required in situ precision on the basis of the in situ standard deviation σ_{XM1} of an adequate set of measurements associated with X_{M1}, in the sense that a higher precision corresponds to a lower σ_{XM1}. In particular, the in situ standard deviation is defined as the standard deviation of an adequate set of measures taken at some point in time during the baseline monitoring (see point g) above). It is then required that the probability of a measure X_{M1} to be within the interval $[\mu-\beta\cdot\sigma_{XM1} ;\mu+\beta\cdot\sigma_{XM1}]$ is larger than a given probability value P.

$$Pr(\mu-\beta\cdot\sigma_{XM1} \leq X_{M1} \leq \mu+\beta\cdot\sigma_{XM1}) \geq P$$

In a situation where the mean value of the assumed normal distribution coincides with the trigger value T_{XT}, it makes sense to ask the product ⍰(P) ·⍰XM1 to be not larger than a given fraction of the trigger value T_{XT}, that is:

$$\beta(P) \cdot\sigma_{XM1} \leq \alpha \cdot T_{XT}$$

Where α is a non-dimensional positive real coefficient restrained by:

$$0 \leq \alpha << 1$$

The choice to refer to “trigger values” is deliberate, as this is currently a common approach across the industry; however, the proposed approach is applicable to most probable values or otherwise defined values of the parameters of interest.

The previous inequality can be rearranged as follows:

$$\sigma_{XM1} (P, \alpha, T_{XT}) \leq (\alpha \cdot T_{XT})/(\beta(P)),$$

which provides the maximum value of the in situ standard deviation which verifies the condition:

$$Pr(\mu-\beta\cdot\sigma_{XM1} \leq T_{XT} \leq \mu+\beta\cdot\sigma_{XM1}) \geq P ,$$

for some chosen value of P, a and TXT.

The above is applicable regardless of any specific significance of the value T_{XT}, as it is effectively a way to define a minimum requirement on σ_{XM1}. The values of P (and hence of $\beta(P)$) and α can be chosen for each trigger value and should be selected on the basis of an assessment specific to such trigger value and to the parameter of interest X_T.

Further details are included in Fornelli (2022) which extends the framework to the case of multiple "trigger values", as well as to the more general case where the parameter of interest (to which the triggers are applied) does not coincides with a single chosen measurable parameter, but is instead a function of one or more measurable parameters. In fact, in this case the previously proposed inequality $Pr(\mu\text{-}\beta\cdot\sigma_{XM1} \leq T_{XT} \leq \mu+\beta\cdot\sigma X_{M1}) \geq P$ does not hold, because X_T and X_M do not coincide. In fact, the previous condition should be changed to

$$Pr(\mu\text{-}\beta\cdot\sigma_{XT} \leq T_{XT} \leq \mu+\beta\cdot\sigma_{XT}) \geq P\ .$$

The above represents just one of the many factors that should be taken into account when selecting the instruments for a monitoring project. In fact, all the points from a) to j) above should be taken in due consideration. From a point of view of long-term monitoring, which is often associated with reservoir assets performance monitoring and maintenance, special attention should be given to the robustness and durability of the hardware (instruments, cabling, acquisition and communication systems, etc.) and to the redundancy of the system.

Data interpretation should be one of the main goals of any monitoring exercises; as such, it seems important to stress here the (often forgotten) importance of the metadata (point j) above). In this context, this term indicates all the data which are not directly associated to the monitoring of any parameter of interest, but rather to help establish a causality relationship between the evolution (in time) of the parameters of interest and the "actions" on the asset which are responsible for such evolution. Examples of metadata (just to quote a few) are of course the application of loads (either static or dynamic) on the asset, the change in atmospheric conditions, rainfall events, works on the asset or on nearby assets. A correct monitoring data interpretation crucially relies on adequate qualitative and quantitative metadata. As such, adequate means for metadata recording (instruments, scans, reports, etc.) should be an integral part of any monitoring system.

In the author's experience, the involvement of an I&M specialist from this very first stage, where parameters of interest and associated requirements are defined, can provide a significant contribution to a successful outcome. An understanding of the capabilities, limitations and durability of different instrument types (and hardware in general), of the associated installation procedures and of the specific requirements and constraints of the asset are extremely useful to keep a holistic view of the scheme.

CHOOSING THE RIGHT INSTRUMENTS

As discussed in the previous section, the choice of the optimal instrument depends on a number of considerations around the parameter of interest that it needs to measure (or contribute to measure). There are numerous producers on the I&M market; there are several ways to measure the same parameter and of course there are several instruments with their specific range, precision, durability, etc.

A list of instruments and their capabilities and applicability limits is well beyond the scope of this paper, and, taking into account the wide spectrum of potential parameters of interest on dam and reservoir assets, it would be a very arduous task. What is of interest here is to stress that choosing the right instrument for a specific application, i.e. to reliably and meaningfully monitor a parameter of interest, takes much more than to browse a catalogue or to type a few keywords on an Internet search engine. One of the fundamental factors in choosing an instrument for an asset performance monitoring system is its durability. It is worth stressing again that each instrument is a physical device which reacts to the surrounding environment and is subject to external factors as much as the asset on which is installed. As such, the presence of an aggressive environment (both above-ground and under-ground) is fundamental information (as in the case of a piezometer installed within an aggressive aquifer). Also significant is the exact location of where an instrument will be installed and which kind of actions are likely to be exerted on the instrument, such as in the case of a joint meter that could be subject to torsion as a consequence of the movement of the joint.

The expected range of the parameter of interest can significantly influence the choice of the instruments; for instance, with reference to the monitoring of the underground displacements of a slope (or an embankment), there are several recorded cases where the deformation is highly localised around a “slipping surface”. As such, the instrument (e.g. an inclinometer or a ShapeAccelArray) has to be adequate to withstand significant localised movements without loss of functionality. Another significant example in the same “geotechnical” context is the choice of piezometer sensors: these have to be selected taking into account, amongst other things, the permeability of the soil layer where the sensor will be installed, as well as the required response time and the likelihood of the development of negative pore pressure (suction) around the sensor, which can make standard Vibrating Wire piezometer sensors provide unreliable readings (Nader and Ridley, 2022).

Exemplars in this sense can be drawn from the point of view of the monitoring of the underground displacements of the (artificial) slopes of an earth dam and of the natural slopes enclosing the retained water body. In both cases, the monitoring of the underground displacements and of the evolution of the pore water pressures is of paramount importance to understand and predict the long-term behaviour of the assets, especially in relation to the creep behaviour, the ageing/damage propagation of the materials and the effects of the climate change. It has been stressed in the previous section that “to monitor a slope” or “to monitor a dam” should not be regarded as meaningful requirements; “procure a strain gauge” or “procure a piezometer” are not meaningful requirements either.

The choice between a system in which the data are collected manually and one which is instead provided with an automated data collection system is also a fundamental one. The optimal solution in terms of data acquisition strongly depends on the specific site needs and constraints (e.g. data acquisition frequency), as well as Health and Safety considerations. Within the framework of maintenance (long-term) monitoring schemes, it is usually convenient to choose a robust automated system, due to considerations around the difficulty of access and the remoteness of the assets across the country. However, it should always be taken into account that no automated system can reliably run (especially for long-term applications) without maintenance. This can be associated to the instruments and the cabling or the communication systems in general. Adequate choices in terms of redundancy, type of hardware and robustness of the I&M system as a whole can help in minimising (although not remove entirely) maintenance-related activities.

In summary, choosing instruments for a field monitoring data-driven approach to the maintenance of dam and reservoir assets should be the result of a careful consideration of the requirements (see previous Section, points a) to j)) and their significance for the specific situation at hand. In fact, such choices should be based as much as possible on a rational approach to the more general goal of obtaining reliable and adequate high-quality data, such as the one which has been presented in the previous section with reference to the requirements around the in-situ precision. In this context, the help of a specialist I&M consultant is obviously beneficial.

THE CRITICAL ROLE OF INSTALLATION AND SOME PROCUREMENT CHALLENGES

All field data produced by an asset monitoring system come from instruments installed on the asset. Although this may sound obvious, it is easy enough to forget that such data are basically a measure of how each instrument reacts to the changes it experiences. As such, if the instrument is not properly installed, if it is not "comfortable" in the way it has been "connected" to the to the structure, to the ground or to the asset in general, it cannot be expected to provide data which are an actual representation of the behaviour of the asset (at the location of the instrument). Therefore, the critical role that the installation procedures (and materials) play in achieving reliable and high-quality monitoring data cannot be stressed enough.

There are a number of activities that need to be carefully planned and undertaken (both in controlled environments and on site) to perform a successful installation. The handling of the instruments is obviously important to avoid damaging the hardware, and there is a significant amount of detail to be considered when installing field instrumentation which depends on the instrument of choice as well as on the specific local details of the asset. For all instrumentation, it is obviously essential that the bonding between the instrument and the asset is such that the changes experienced by the asset at the location of the instrument are transferred to the instrument minimising the disturbances due to the installation; for instance in terms of displacement/deformation and temperature effects.

A typical example is the installation of crack-meters or joint-meters on structures, where the details of the connection between the instrument and the structure shall be defined to minimise the differential displacement due to the connection, which may involve drilling and grouting (on concrete and masonry structures) or welding (on metal structures). A similar situation arises when connecting fibre optics to structures, where appropriate solutions in the form of clips or epoxy resin need to be selected and potentially tested for ensuring data reliability as well as limiting the impact on the asset. The criticality of the installation process is even more evident when installing field instruments underground, as may be the case for inclinometers, extensometers, piezometers, etc. In this case the continuity between the asset (ground, groundwater) and the instrument is removed during the installation process due to the drilling operations. Therefore, such continuity needs to be restored as much as possible and taking into account the local conditions of the asset and the nature of the parameter of interest that the instrument is intended to monitor. Furthermore, the installation at depth requires a number of details to be carefully considered and checked, such as the torsion of the inclinometer casing during the lowering operations and the installation of the Vibrating Wire piezometers with the filters facing upwards to allow any residual air to leave the instrument.

These examples represent just an extremely limited selection of the considerations that are required to provide reliable field monitoring data. However, in the author's opinion, they are useful to clarify the fundamental role of the installation process (and of the amount of detail associated with it). The efforts and resources required to identify the parameters of interest, the associated requirements and to define and procure the optimal instruments, can be entirely wasted if the data are made unreliable by an inadequate installation.

As such, it is essential that the installation of each instrument and, more in general, of every component of the I&M system, must carefully defined, planned and carried out by experienced personnel and under the constant supervision of an I&M specialist.

One of the main challenges to the above is the current common procurement model for I&M activities. In most instances, it is based on a Bill of Quantities with instruments and installation rates. The main effect of this kind of procurement model is that the "perceived value" of an I&M system is associated with the procurement of the instruments rather than with obtaining reliable, high quality useful data. Also, it makes it very difficult for the I&M specialist to be engaged at an early stage to provide support and useful insight for the optimisation of the field monitoring scheme. As per previous considerations, the overall risk is that, if the data reliability and quality are not identified as the true benefit of an I&M system, then the possibility to apply a data-driven approach to dam and reservoir asset maintenance is jeopardised. On the basis of the considerations developed in this and previous sections, it should be recognised instead that the I&M would be better procured as a service, and should be focussed on the quality of the data rather than on the cost of the instruments.

DIGITAL TWINS

The idea of using field monitoring data to inform and optimise the construction process dates back at least to Terzaghi's and Peck's works around the use of Observational Method in Geotechnical Engineering. The recent huge development of numerical analysis techniques and software, associated with the increasing capabilities of Artificial Intelligence (AI) algorithms nowadays allows the use of the field monitoring data from real assets as part of complex digital models. The numerical modelling (of the asset) and the field monitoring data (from the instruments installed on the asset) are integrated within a framework which allows the predictions of the former to increase in their accuracy on the basis of the latter. The numerical models are "trained" via AI algorithms on the basis of the evolution of the monitored parameters of interest and metadata. Such an approach is quite new, and the associated nomenclature still somehow undefined. It is easy enough to find "trainable" models (numerical solver + AI) referred to as "Digital Twins". The idea is that the trained numerical model becomes a digital "replica" of the real asset, so that it can provide an accurate understanding the current behaviour of the whole asset (e.g. in terms of the evolution of the displacement field, strain field, pore water pressure field, etc.), as well as provide an accurate prediction of the future behaviour of the asset under given boundary conditions. The general concept is further explained in Figure 2: the monitoring data from a general dam and reservoir asset are used to train the numerical models and provide predictions; these are subsequently interpreted to assess the need for (and, if needed, optimise) adequate maintenance intervention. It should be noted that the possibility of optimising maintenance interventions is connected to the quantitative nature of the outputs of the model, as opposed to the mostly qualitative nature of standard visual inspections. The reliability of the predictions increases in time as more field monitoring data become available (i.e. the "training" increases).

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However, it should be noted that the reliability of the current and future behaviour obtained by such models depends directly on the reliability (quality) of the field data used to "train" the model. In effect, the usefulness, and more widely the adequate functioning, of the digital predictive framework output is based (and strongly depends) on the field monitoring data availability, reliability and quality. As such, it is the author's opinion that a "Digital Twin" should indicate an integrated framework including the (validated) field monitoring data as well as the numerical models "trained" using such data.

It is worth stressing that the use of "Digital Twins" for asset maintenance/management is currently in phase of deployment in Italy.

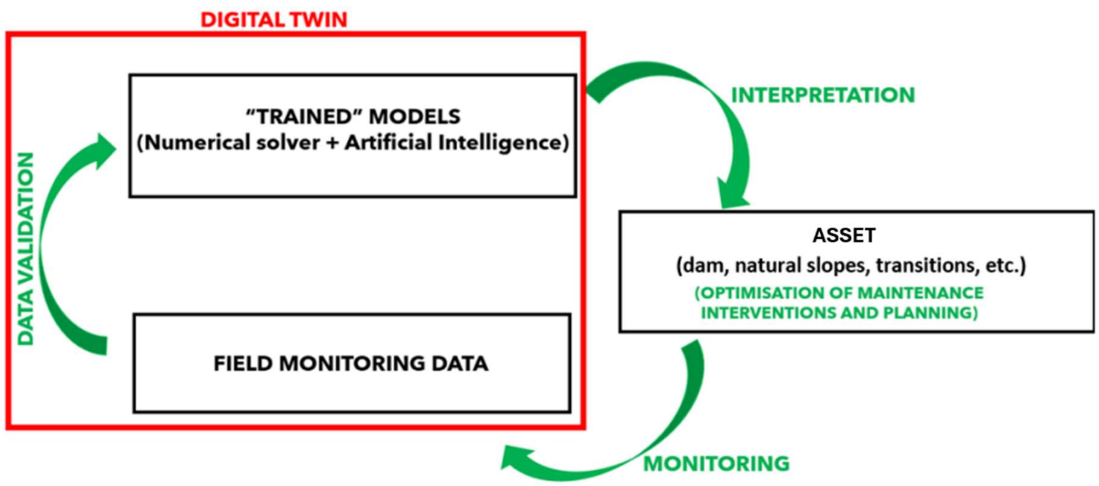

Figure 2: scheme of the constituents of a "Digital Twin" and general concept of data-driven predictive maintenance for dam and reservoir assets.

CONCLUSIONS

The paper has highlighted the fundamental importance of reliable, high-quality field monitoring data in the context of challenges posed by the UK's ageing dam and reservoir assets, together with the uncertainties associated with climate change and the needed optimisation (towards an increase in sustainability) of maintenance programmes and interventions.

The definition of a monitoring scheme able to provide adequate data relies on several steps and a significant amount of theoretical and practical experience, as well as cooperation throughout several different disciplines. Also, the deployment of a robust, reliable monitoring system able to provide high quality data requires a careful choice of instruments and an extreme attention to detail in the installation phase.

The early engagement of an I&M specialist alongside other parties has been recognised to be of paramount importance for a successful deployment. The challenges associated with the current common procurement models for I&M have been discussed, and in particular the need for recognising that the value of such system should be associated to the quality of the data rather than to the cost of the instruments, as well as recognising that, due to its transversal and highly technical nature, the I&M should be procured as a service.

There is potential for significant opportunities associated with the implementation of Digital Twins for dam and reservoir assets, in relation to the optimisation of maintenance planning and interventions; however, these rely strongly on the recognition of the fundamental importance of the reliability and high quality of the monitoring data, and of all the contributing factors discussed within the previous sections. To this end, it is the author's opinion that Instrumentation and Monitoring should be considered an Engineering discipline in its own right by the Construction Industry bodies and at Academic level.

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