Toddbrook Reservoir Independent Review Report

By Professor David Balmforth

10 February 2020

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Foreword

We have an excellent reservoir safety record in England. There have been no dam breaches resulting in the loss of life since reservoir safety legislation was first introduced in 1930. As the Minister responsible for reservoir safety policy, I want to ensure we maintain a high standard for all our reservoirs. We must learn from experience to ensure that our legislation and the implementation remains effective and keeps pace with the threats from climate change. The incident at Toddbrook Reservoir is a sharp reminder that all our reservoirs need to be looked after and managed well.

Everyone will remember the footage of 1 August 2019, after heavy rainfall caused serious damage to the spillway at Toddbrook reservoir in Whaley Bridge. The immediate concern was the safety of the residents and visitors to the town so as a precaution some 1500 people were temporarily evacuated from their homes and businesses in Whaley Bridge, whilst an immediate drawdown of the water level was instigated, together with urgent measures to shore up and stabilise the spillway. I want to thank all of those involved for the rapid and professional response to that emerging situation.

Incidents like Toddbrook are very rare, but it is important that lessons are learned and shared widely to help the dedicated professionals who manage and maintain our reservoirs in securing the ongoing safety of these structures. Toddbrook reservoir was compliant with existing legislation - nonetheless, it suffered unforeseen and potentially critical damage. To ensure we are able to take account of what lessons can be taken from this event, the Government asked Professor David Balmforth to lead this independent review to explore how the damage occurred and make recommendations for our whole reservoir community to adopt. I welcome his findings and recommendations which will help to further embed reservoir safety.

I want to thank Professor Balmforth and the Review Panel for their work in conducting this review, and also the Environment Agency, the Canal & River Trust, and others for their cooperation with the Review Panel. The Government accepts all the recommendations made and will take these forward with the Environment Agency and other stakeholders.

I expect that all those involved in the operation, management and inspection of all reservoirs to take the time to read and understand this report. You do not need to wait for Government to take action – many of the recommendations can, and should, be implemented by you directly.

George Eustice

George Eustice

Secretary of State for Environment, Food and Rural Affairs

Contents

Executive summary

Following heavy rain that fell between the 27 July and the 1 August 2019, the spillway at Toddbrook Reservoir failed. The rain that led to the failure was well forecast and fell in two separate events, the first occurring over the 27 to the 29 July closely followed by a more severe event from the 30 July to 1 August 2019. These were rare events, the second of which had an estimated annual probability of about 1%. The resulting flood, however, was very much smaller than the probable maximum flood which the spillway should have been able to accommodate. It had dealt with significant floods in the past without apparent damage.

On the morning of the 1 August 2019, a single slab of the spillway chute collapsed into a large void that had formed underneath, and a brown slurry could be seen discharging from under slabs (which had also failed and lifted) further down the spillway chute. During the day the void enlarged, and more slabs collapsed, risking the integrity of the dam. A full-scale emergency was declared, and, as a precaution, 1500 people were evacuated from the town of Whaley Bridge immediately downstream.

Toddbrook reservoir has a capacity greater than 25000m^3 so it is subject to the requirements of the Reservoirs Act 1975 (as subsequently amended). It is also designated "high risk" because of the impact a potential breach might have on the communities downstream, and is therefore subject to regular inspections as set out in the Act. The UK has a good reservoir safety record. Since the Reservoirs (Safety Provisions) Act of 1930 there have been no deaths caused by the failure of a reservoir.

Given that the reservoir was compliant with the legislation and had had a recent inspection, the incident was a surprise. The Secretary of State for the Environment therefore commissioned an Independent Review, to consider effectiveness of reservoir safety legislation and regulations in relation to Toddbrook, and for the reservoir stock as a whole. This is to be undertaken in two Parts: Part A, to focus particularly on any lessons arising from the incident at Toddbrook Reservoir, and Part B, which will depend on the outcome of Part A, to look more widely at the implementation and suitability of the reservoir safety arrangements across the sector at that time. The Review aims to learn lessons from the incident and make recommendations about how the management of reservoir safety in the future might be improved. The emergency response to the incident and the subsequent repair or remedial work to the reservoir are explicitly excluded from the Review. This is the report for Part A.

The Secretary of State appointed me, Professor David Balmforth, Past President of the Institution of Civil Engineers, to lead the review. I have been assisted by Dr Peter Mason, an All Reservoirs Panel Engineer, and Dr Paul Tedd, a specialist in earth embankment dams, thus making up a Review Panel of three. This Panel has reviewed evidence from the incident, visited the reservoir and inspected the spillway in detail, and reviewed a large amount of documentation from the Canal & River Trust (CRT) and the Environment

Agency (EA). They have also interviewed the Supervising and Inspecting Engineers, CRT Engineering staff, the engineers from CRT and Mott MacDonald who responded to the emergency, and staff from the EA who deal with regulations and enforcement.

Overall, I have determined that the most likely cause of the failure of the auxiliary spillway at Toddbrook Reservoir on the 1 August 2019 was its poor design, exacerbated by intermittent maintenance over the years which would have caused the spillway to deteriorate.

The Panel has compiled a detailed account of the events leading up to the collapse of the spillway, and the events as they unfolded on the day. Based on the findings of the Review, it is my opinion that the spillway design was inadequate and not fit for the purpose of conveying the probable maximum flood (for which it should have been designed). There is evidence to show that it had deteriorated over its life.

The lack of an effective cut-off between the spillway crest and the impermeable core of the dam would have allowed water to pass into the embankment fill under the spillway chute. While some of this will have drained downwards through the permeable fill, it is likely that some will have flowed beneath the slabs of the spillway chute causing erosion of its foundation. In addition, seepage of water through a construction joint in the crest had been observed from time to time flowing down the face of the spillway, and this is likely to have seeped into the longitudinal joints which were not fitted with water bars and had only received intermittent maintenance. Any erosion caused by seepage flows could have led to some settlement and cracking of the spillway slabs. Satellite data, received towards the end of the Review, suggests that settlement of the slabs in the area of the initial failure may have accelerated in the months leading up to the failure. However, there is no evidence to suggest that this process could account for the large void that was observed to have formed beneath the chute slabs, and into which failed slabs subsequently collapsed. Some 800 tonnes (400 m^3) of material is estimated to have been eroded from the embankment. This would have required large volumes of water with high energy to convey that volume of material off-site and into the river downstream. That could only have occurred during the actual event, and there is video and photographic evidence to support this.

Various mechanisms might explain how the large volume of water could have found its way beneath the slabs. From the evidence available, the most likely explanation is that this occurred as a result of a process known as crack injection. Crack injection can occur when high velocity flow impacts against a solid object in its path such that the kinetic energy in the flow is converted into pressure. Quite small obstructions in the flow, such as the edge of a joint or crack, can cause this effect. The pressure then forces water through the crack or joint and into the material beneath. The phenomenon appears not to be particularly well known in the UK but has been extensively researched in the USA and attributed to a number of spillway failures similar to that at Toddbrook. Large volumes of water can be injected into the foundation of a spillway over a period of hours.

The embankment fill, on which the spillway is founded, would have been very susceptible to erosion as a result of the injected water. As this erosion took place, the fill would have been fluidized into a slurry which then flowed beneath the slabs down the underside of the chute. Lower slabs were seen to be pushed up and fractured, allowing the slurry to escape through the longitudinal joints that had opened up. Other than the observed discolouration of the water, this effect may not have been evident at the height of the flow down the spillway, but when the flood receded, this was clearly revealed.

It has not been possible to say whether it was the poor design or the intermittent maintenance that was the primary cause of failure on the day. With consistent good quality maintenance over the years leading up to the event, the spillway might not have failed during this event. However, it would have been unlikely to survive the probable maximum flood which is many times greater than the flood in which it failed.

As the events of the 27 July to the 1 August were well forecast, the Panel has considered whether CRT could have done more to avert the incident. From modelling undertaken subsequently, the volume of flow down the spillway would have been reduced had the draw-off valves been opened and the inflow diverted into the by-wash channel earlier. However, the flow down the auxiliary spillway would still have been substantial and it may still have failed.

In his report of his 2018 inspection, submitted to CRT in April 2019, the Inspecting Engineer correctly identified the risk posed by the spillway, and recommended measures that would likely have addressed the deficiencies in the longer term. The incident occurred before work had started, however. He required CRT to complete an investigation of the spillway over the following 18 months and required full maintenance of the spillway to be undertaken. However, in wording his requirements, he did not convey any sense of urgency or require any precautionary measures, such as a draw down of the reservoir. Where significant and credible risks are identified that threaten the safety of a reservoir then the seriousness of that should be unequivocal in the wording and timeliness of measures in the interests of safety (MIOS) and/or statutory maintenance.

The Supervising Engineer has stated that he relied entirely on the views of the Inspecting Engineer. He had reported to the owners on a regular basis regarding the condition of the reservoir and its maintenance. On more than one occasion he had reported on maintenance to the spillway that had not been completed.

CRT have remained compliant with the legislation for the entire time they have had responsibility for the reservoir. The EA have not had to issue any enforcement notices. However, both CRT and the EA have stated that compliance is not the same as safety. This can mean that a reservoir and its Owner can be compliant with the legislation without the reservoir necessarily being safe. This does not appear to be entirely satisfactory and will be investigated further in Part B of this Review.

Introduction

Reason for the Review

On the 1 August 2019 serious damage occurred to the auxiliary spillway at Toddbrook Reservoir as a result of heavy rain over the preceding days, raising concerns of a potential catastrophic failure of the dam. Some 1500 people were evacuated from their homes and businesses in Whaley Bridge. An immediate drawdown of the water level was instigated, and urgent measures taken to shore up and stabilise the dam. Once the immediate danger passed, those evacuated then returned. Since 2012, Toddbrook Reservoir has been owned and maintained by the Canal & River Trust, who remain responsible for ongoing safety, including repairs.

We have an excellent reservoir safety record in this country. Toddbrook Reservoir was compliant with existing legislation and had been recently inspected. Nonetheless, it suffered unforeseen and potentially critical damage that could have led to a catastrophe. The Secretary of State for the Environment believed there was a strong case for an independent review to look into the facts of the case, and also examine whether there are any weaknesses in our legislative or regulatory frameworks that should be addressed. The Secretary of State appointed Professor David Balmforth to lead that Review, assisted by Dr Peter Mason and Dr Paul Tedd.

Terms of Reference

Part A

Objectives

This independent investigation will:

1. Investigate the possible causes for the damage, identify any issues in the operation, inspections or maintenance of Toddbrook reservoir (including the dam and spillway) in the period leading up to the incident on 1 August 2019.
2. Assess the dam's capacity pre 1 August to survive extreme flood events without collapse
3. Assess the roles of the Owner, inspectors and regulator in the management of Toddbrook reservoir.
4. Consider lessons learned from the incident on 1 August 2019 in regards to: the design, maintenance and inspection of the Toddbrook reservoir; and the application and adequacy of current regulations.
5. Make recommendations/proposals on:

- what, if anything, could have been done to predict/prevent damage to the spillway
- any immediately identifiable changes needed to the implementation of existing reservoir safety guidance or legislation, including maintenance, operation and inspection
- any immediately identifiable proposals for changes to current regulations

Reporting

An interim report for Part A will be provided to the Secretary of State within three months.

Out of scope

The review will not include:

- incident response
- ongoing integrity and/or future repairs to Toddbrook Reservoir

Part B

Depending on the findings of Part A of the review, the review may then look at current reservoir safety guidance and legislation. The review would consider:

a) Proposals for any amendments to current legislation, including regulations

b) New/Improved guidance to operators (Owners) and inspectors (supervising and inspecting) that will improve the operation, maintenance and inspection of similar structures going forward

c) Any changes to the requirements for the competence needs, assessment of, appointment or functions of panel engineers

Full terms of reference for this part to be informed by part A and agreed at a later date.

The Review panel

David Balmforth is a Past President of the Institution of Civil Engineers (ICE) and an Executive Technical Director with the international engineering consultancy Stantec. He has a BSc(Hons) in Civil Engineering from the University of Bristol and a PhD in Civil Engineering Hydraulics from the University of Sheffield. He specialises in flood risk management and urban pollution control. Formerly an academic, his recent work ranges from the delivery of £multi-million engineering programmes, to flood advisory work for municipalities in the UK and overseas. He is an advisor to governments on flood risk management and has recently worked to alleviate flooding and water pollution in London, Glasgow, Auckland and Singapore.

From 2013 to 2019 he was chair of the ICE's Reservoirs Committee which qualifies Reservoir Engineers on behalf of the member states of the UK. He was a Specialist Advisor to the EFRA Select Committee during their Inquiry into the Future of Flood Prevention in 2016, a member of Scientific Advisory Group to the National Flood Resilience Review, also in 2016, and a member of the Government Review (Pitt Review) of the 2007 Summer Floods.

David is a former Editor in Chief of the Journal of Flood Risk Management and a Visiting Professor at Imperial College, London. He has published over 50 journal papers, conference papers and design guides.

Peter Mason is a Director of Damsolve Ltd. He has a BSc in Civil Engineering from Woolwich Polytechnic and an MPhil and PhD in Applied Hydraulics related to spillways, from City University, London. He has specific expertise in dams, hydraulic structures, hydropower and all associated works. He is currently chairman of the Board of Management for a major hydropower project in Pakistan and a member of dam safety advisory panels in Canada, Laos, Uganda, Zambia and Albania, the last two being World Bank funded. Over the course of a 50 year career he has been responsible for feasibility studies, contract documentation and designs for major hydraulic works and all types of dams, in approximately 45 countries including the UK, with advice, inspection visits and asset evaluations as a Named Expert in Africa, Asia, Australia and North and South America. This has included advice to Contractors, Owners, Funding Agencies and Prospective Purchasers with lectures and technology transfer to Clients' Engineers.

A Fellow of the Institution of Civil Engineers he has also been a member of various National and International Committees and Panels related to dams. He is a past Chairman of the British Dam Society and of the Institution of Civil Engineers Reservoir Safety Advisory Group. He has been an All-Reservoirs Panel Engineer under the UK Reservoirs Act since 1994 and was a Supervising Engineer under the Act before that. He chairs a Reservoir Safety Advisory Panel for a major UK Water Company and is the Author of over 80 papers and/or articles on named specialities.

Paul Tedd has a BSc(Hons) in Engineering from the University of Leicester, a PhD in Rock Mechanics from Kings College, and a DSc from the University of London. Reservoir safety studies have formed a major part of the last 32 years of his work and has involved research and field monitoring of mainly old puddle clay core dams to assess their condition. He is the joint author of four Engineering Guides on reservoir safety including "An engineering guide to the safety of embankment dams in the United Kingdom" and "Lessons from incidents at dams and reservoirs - an engineering guide" and more than 30 papers relating to dam safety. From 1994 to 2003, he was Honorary Technical Secretary of the British Dam Society and editor of the society's conference proceedings. He was manager of the BRE National Dams Database since its inception in 1987 and was involved in development of the Environment Agency post incident reporting dams database. He was awarded the British Dam Society Bateman Award in 2013. From 2017, he has taught on the Supervising Engineers Course.

Approach to the review

The first task for the Panel was to build a comprehensive evidence base to arrive at their findings and recommendations. The Panel has reviewed over 500 documents provided mostly by the Environment Agency (EA) and the Canal & River Trust (CRT) and the Panel is grateful to them for their cooperation. They have, at all times, been open and helpful. The Panel has also reviewed correspondence from some members of the public who have particular local knowledge, and the results of a physical investigation of the spillway that has been undertaken by CRT since the event. They visited the reservoir in October 2019 and undertook a detailed forensic examination of the spillway and dam. They interviewed staff from CRT who have been responsible for managing the reservoir, including CRT's Supervising Engineer for the reservoir, the Inspecting Engineers from Mott MacDonald who undertook the 2010 and 2018 inspections, the engineers from Mott MacDonald who responded to the emergency, and staff from the EA who are responsible for regulations and enforcement.

As well as investigating the cause of failure, the Panel has also looked into the events leading up to the failure, and the management of the reservoir, tracing its history back to its construction in 1840. They reviewed failures of other similar spillways both in the UK and overseas. In particular, they learnt from investigations and research undertaken by the United States Bureau of Reclamation (USBR)[1]

The Panel has consistently aimed to be open and objective in their Review, basing their findings and recommendations on the evidence that they have collected. Their findings and recommendations are set out in the main body of the report in a clear and concise manner that can readily be understood. Those who wish to see a more detailed explanation of the mechanism of failure of the spillway or a summary of the wider safety aspects of Toddbrook Reservoir can refer to the appendices.

How we keep our reservoirs safe

The need for legislation to monitor and control the safety of UK reservoirs was first highlighted in 1852 following the failure of Bilberry dam and then again in 1864 with the collapse of Dale Dyke Dam near Sheffield and the deaths of 244 people. However, no legislation was enacted until the failure of two more dams in 1925. That resulted in the

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[1] USBR Report DSO-07-07 "Uplift and Crack Flow Resulting from High Velocity Discharges over Open Offset Joints", Dec 2007

Reservoirs (Safety Provisions) Act 1930. This was applied to all reservoirs in Britain (but not Northern Ireland) with a capacity of 5 million gallons (22700m^3) or more. It required new reservoirs to be designed by a suitably qualified engineer who would then also supervise their construction. It also provided for 10yearly inspections of all new and existing reservoirs at or above the threshold capacity.

It was a success, to the extent that since the introduction of the 1930 Act there have been no deaths in the UK due to the failure of a dam or reservoir. However, by the 1970s it was decided, partly because of recent major catastrophes abroad, that reservoir safety regulations should be reviewed. A new Act, the Reservoirs Act 1975[2] was enacted, and this introduced the new role of the Supervising Engineer. The Supervising Engineer monitors and "supervises" the reservoir regularly in-between the 10yearly inspections and reports annually to the Owner on its condition and safety (with a copy of the report sent to the EA). The 1975 Act also modified the minimum size that a reservoir needed to be to come under the Act, raising it slightly to 25000m^3.

The Floods and Water Management Act 2010 made further changes to the threshold capacity at which reservoirs should be monitored and controlled, allowing it to be reduced to 10000m^3. By this time too, devolution of powers to Scotland, Wales and Northern Ireland produced a divergence of reservoir safety requirements. The Reservoir Act 1975 still provided the mainstay of control in England and Wales but changes elsewhere were to come with the Reservoirs (Scotland) Act 2011 and the Reservoirs Act (Northern Ireland) 2015. In Northern Ireland, the key reservoir safety provisions of the Act are still to be commenced. In Wales, the new 10000m^3 threshold has been implemented. It has yet (as of 2019) to be implemented in England, Scotland or Northern Ireland.

Another change to come with the Floods and Water Management Act 2010 was the designation of reservoir by the risk it posed. In fact, such risk is extremely difficult to quantify reliably and consistently, and so although the term "risk" is used in legislation in England, Scotland and Wales, actual designation has been by the potential for downstream damage or "consequence" of failure, e.g. how many lives are potentially at risk. The most recent legislation, that of Northern Ireland, correctly uses the word consequence for designation purposes rather than risk.

To more fully understand how the Act in England works in practice the following sections summarise factors and roles in relation to the Act and its practical implementation.

[2] Note that the Reservoirs Act 1975 and the previous 1930s act do not apply to Northern Ireland

The owner

The owner of a reservoir is usually the person or organisation responsible for operating the reservoir and ensuring its safety. However, this is not always the case. In some cases, another person or organisation will operate the reservoir. An example of this is where the ownership of a reservoir may be split between a number of owners, each owning a part. In such cases one of these owners may undertake the responsibility for operating the reservoir and ensuring its safety. The various Reservoirs Acts therefore refer to the person or organisation that has these responsibilities as the “undertaker”. However, for the sake of simplicity, and to ease the understanding of this report, the term “owner” is used throughout in place of “undertaker”. The owners of all reservoirs are responsible for their safety whether or not they are covered by any specific reservoir legislation.

The Regulator or Enforcement Authority

In England the Regulator or Enforcement Authority responsible for monitoring compliance with the Act is the Environment Agency (EA). They maintain a Public Register of all large raised reservoirs in England covered by the Act and copies of the various reports and certificates issued under the Act to monitor their safety. If a report and/or certificate is due but not received, they will compel Owners to deliver their obligations. If safety works are not completed, as required by a panel engineer, the EA can complete the safety works on behalf of the Owner (and charge the Owner for this work). In emergencies, the EA have additional emergency powers to make a reservoir safe if the Owner does not or cannot carry out their obligations. This should ensure that Reservoirs are being properly supervised and inspected as and when required.

Qualified Civil Engineers/Panel Engineers

There are four Panels under the 1975 Act whose members are those engineers qualified to act as reservoir engineers under the Act:

- the All-Reservoirs Panel
- the Non-impounding Reservoirs Panel
- the Service Reservoirs Panel
- the Supervising Engineer’s Panel

The first three are licenced to design, inspect and supervise the construction of the types of reservoir to which their respective Panel name refers. Members of all four Panels can act as Supervising Engineers. In England they are appointed to their respective Panels by the Secretary of State, acting through Defra, on the recommendations of the Reservoirs Committee of the Institution of Civil Engineers (ICE) acting on behalf of the President of the Institution who is named in the Reservoirs Act for that role. Recommendation to a panel is by examination of their competence and experience, and appointments are for a 5year period, after which the Panel member is re-assessed for their suitability in the role.

Depending on the roles they are fulfilling, a Panel member may be described as the "Construction Engineer" in the case of reservoir construction, the "Inspecting Engineer" in the case of inspection activities and/or more generally as a "Qualified Civil Engineer" (QCE) for the purposes of the Act.

Inspecting Engineers[3] carry out periodic inspections of reservoirs under the Act. The maximum time allowed between inspections is 10 years, but the Inspecting Engineer may specify a shorter period if s/he thinks that is more appropriate. The Reservoirs Act requires that reports should be provided as soon as practicable following the inspection. If a report is not made within 6 months, the IE must notify the EA (the appropriate authority) and provide a written statement of the reasons. Copies of the inspection report and the associated Inspection Certificate are sent to the Owner and the Regulator. The Certificate will set out whether or not s/he requires any measures to be taken "in the interests of safety" (MIOS). Where such works are specified, a period for completion will also be specified and this is enforceable by law. Their completion also requires certification and the whole process is monitored by the Regulator who will intervene if deadlines are missed or certificates not forthcoming.

In between inspections the reservoir will be monitored by a Supervising Engineer[3]. The Supervising Engineer is required by the Act to produce an annual statement to the Owner, copied to the EA, summarising the condition of the reservoir and the compliance or otherwise of the Owner with his duties under the Act. S/he is expected to make at least one visit each year to review the condition of the reservoir. S/he will analyse any data on reservoir behaviour, such as instrumentation data, being collected by others, and review it on an on-going basis to ensure satisfactory performance. The Supervising Engineer has the power to request an inspection by an Inspecting Engineers at any time if s/he considers the reservoir to be potentially unsafe. S/he may also refer issues directly to the regulatory authority (the EA).

Reservoir types and categories

For the purposes of the Act there are three types of reservoir. Impounding Reservoirs are those where a dam has been built across a natural stream or river and where rainfall will cause the reservoir to fill. Non-impounding reservoirs are remote from natural water

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[3] Inspecting engineers are employed solely for the purpose of undertaking an inspection and producing a report and certificate under Section 10 of the Act. They must be independent of the Owner that is not on their payroll. Supervising engineers are employed continuously and operate under Section12 of the Act to supervise the reservoir. They may be (and often are) permanent employees of the Owner.

courses and fill by pumping. They do not depend on rainfall to fill. Service Reservoirs store treated water for distribution to a community, and are filled by water from a Water Treatment Works.

Reservoirs are normally categorised by the consequences of failure, as described in the ICE guide, “Floods and Reservoir Safety”[4]

The categories are as follows:

- A – where a community downstream could be affected
- B – where isolated individuals (not in a community) could be affected
- C – where loss of life is unlikely but there could be significant infrastructure damage
- D – where downstream consequences are likely to minimal

Category designations in this form are used by Panel Engineers to decide on the most appropriate flood standard for which the dam spillway should be designed. These categorisations do not form part of the Reservoirs Act. The guide also allows more sophisticated analyses to be undertaken where risk is quantified more precisely and compared with likely loss of life, or consequential damage, using an ALARP (as low as reasonably practicable) chart.

Note that the categorisation of reservoirs described above is different from the designation of a reservoir as ‘high-risk’. ‘High-risk’ in the context of the Act refers to whether the EA takes the view that, in the event of an uncontrolled release of water from a large raised reservoir, human life could be endangered. It is designated by the EA using all relevant information/documents they hold and advice from an independent panel engineer.

Toddbrook Reservoir is categorised as Category A and designated High-Risk.

Emergency response

The response to any emergency that affects communities is covered by the Civil Contingencies Act 2004. Responsibility for emergency planning lies with the Local Resilience Forum as set out in that Act. Flooding as a result of uncontrolled discharge from a reservoir is an important part of emergency planning. To assist with this, the Floods and Water Management Act 2010 provided for reservoir Owners to prepare Flood Plans and allows the Secretary of State to direct a reservoir Owner to have such a plan (commonly known as the “On-site plan”). Though this provision has not been implemented (in 2019),

[4] Floods and Reservoir Safety, 4th Edition, Institution of Civil Engineers, London, 2015

the EA has encouraged all reservoir Owners to prepare such plans. The emergency response to the incident on 1 August 2019 is explicitly excluded from the scope of this Review.

Toddbrook reservoir

Toddbrook reservoir is an impounding reservoir, situated approximately 0.5km to the south west of the town of Whaley Bridge in the High Peak area of Derbyshire, 10km north of Buxton, just off the A5004. The reservoir was designed to increase the water supply to the Peak Forest Canal and was formed by the construction of an embankment dam built across the Toddbrook between 1837 and 1840. The reservoir is orientated in a south-west to north-east direction and has a total length of approximately 1km with the embankment being at the north-eastern end (fig 1). It is located immediately upstream of the outskirts of Whaley Bridge.

Figure 1. Location of the dam and its Principal Features

The dam is an earth embankment of the Pennine type. It has a central “puddle clay” core and shoulders of more granular earth-fill (fig 2). The dam is some 24m high by about 310m long with a crest width of approximately 5m. The upstream and downstream slopes of the embankment are both at 1 vertically to 2 horizontally (approx. 26.5°) over the first 15m of dam height below crest level. Below that both slopes flatten to 1 vertically to 3 horizontally (approx. 18.4°). The crest elevation is +187.30 mOD. The reservoir was originally constructed with a single spillway (primary overflow) sited at the left-hand end of the embankment, when looking downstream. The spillway was constructed to prevent water spilling over the embankment dam. Water spilling over the unprotected embankment in this way would be likely to rapidly erode the embankment causing the dam to breach.

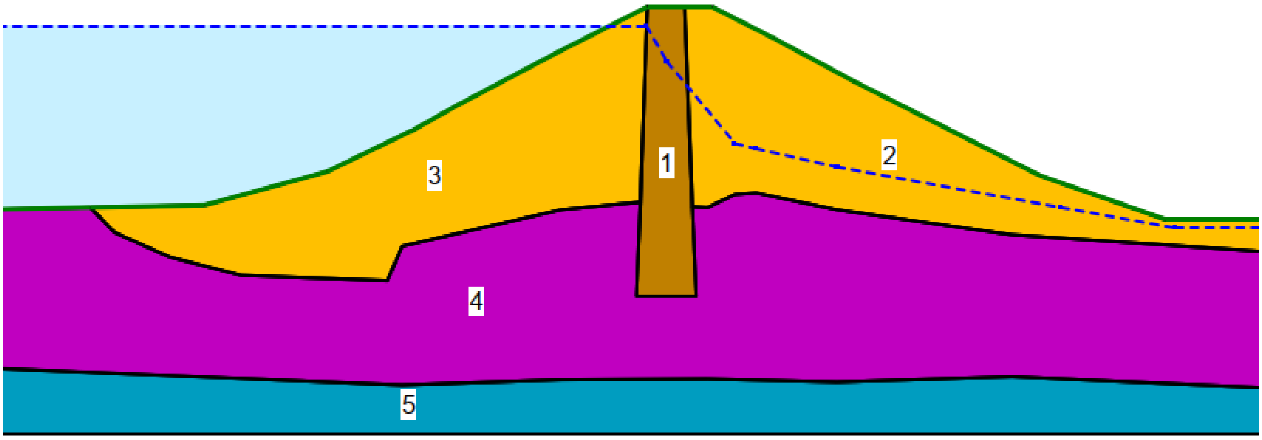
The normal top water level (TWL) of the reservoir is +185.67 mOD. This coincides with the overflow level of the primary spillway weir. Records indicate that the reservoir remains at or above that level (i.e. spilling) for several months each year during the winter.

Toddbrook reservoir has a capacity of 1288000m^3. As the reservoir has a capacity greater than 25000m^3 it is subject to the requirements of the Reservoirs Act 1975 (as subsequently amended). It is also designated “high risk” because of the impact a potential breach might have on the communities downstream, and is therefore subject to regular inspections as set out in the Act. Because of the impact from a potential breach, it is also classified as a Category A reservoir.

History of construction

The present dam site is the third attempt to construct a dam in this location, the previous two, some 150m and 250m upstream, having been abandoned in 1835 and 1837 respectively, due to poor ground conditions and mining activities. Mining within and around the site is recorded as taking place from 1500 to the early 1900s. At the time, Toddbrook was the highest dam to be constructed in the UK. The number of large raised reservoirs that were in existence or were being constructed in England at the time was 414, whereas currently there are 2082.

Toddbrook has a central puddle clay core as its watertight element, supported by shoulders of more granular material (fig 2). According to the earliest concept of puddle clay core dams, the embankment shoulders merely served to support the watertight core and only needed to be stable and reasonably solid. Typically, the core would be placed in layers of not more than 150mm thick and “worked” by men in fisherman's boots to create a watertight seal. At Toddbrook, the core is approximately 3.3m wide at the crest, increasing to 4.8m wide at original ground level, with a shallow clay filled cut-off trench into the underlying glacial till. This type of construction was used for most dams constructed in Britain until the late 1950s. Material for the dam was extracted from the reservoir basin. Settlements associated with primary consolidation would have occurred during and shortly after construction. This would have typically been accompanied by horizontal outward movements from the centre (spreading).



Source: MML

- Material 1 – Puddle Clay Core
- Material 2 – Downstream Shoulder Fill
- Material 3 – Upstream Shoulder Fill
- Material 4 – Glacial Till
- Material 5 – Bedrock

Figure 2. Cross Section through Dam at Toddbrook Reservoir (not to scale)

The dam is founded on a 12m thick glacial till consisting of sandy clay overlying thin layers of laminated clay and gravels. The bedrock consists of a faulted sequence of mudstones, sandstones and shales of the Millstone Grit Series and Lower Coal Measures. The reservoir is situated on the edge of a now disused coal mining area with still active mine drainage tunnels running under the northern end of the dam. In addition to abandoned mine workings, mining records indicate the site of a pumping shaft, possibly 30m from the upstream toe of the dam. The presence of mine shafts within the dam has led to extensive investigations over the years.

A compensation channel was constructed along the north side of the reservoir into which the original (primary) overspill weir discharged, before running down the toe of the dam to join the River Goyt. This was provided to protect the interests of the water mills downstream.

Toddbrook was constructed at a time when the knowledge of extreme rainfall events and the effect of floods passing over dams was limited. Many reservoirs had inadequate spillways allowing the dams to be overtopped and some were washed away. As a result of these incidents, and improved knowledge and guidance, many spillways were enlarged and improved with the addition of auxiliary spillways to take excessive flood water.

On the 12 December 1964, an extreme rainfall event damaged the original (primary) spillway. This led to the Inspecting Engineer requiring the overflow capacity to be increased considerably. Between 1969 and 1970, an auxiliary spillway 76m long and

0.26m above the original spillway crest, was constructed on the face of the embankment (fig 3).

Figure 3. Construction of the Auxiliary Spillway on the Earth Embankment Dam in 1969

Over the years there have been various incidents that have led to remedial works and considerable investment in improvements at the reservoir. These are summarised in table 1 below:

Table 1: Summary of incidents, investigations and remedial works at Toddbrook Reservoir

1840	The reservoir was completed and was equipped with a spillway at NW end of dam discharging into by-wash/compensation channel. The length of the weir crest was 41.2 m.
1880	Complaints about leakage into mines
1895	Old pit shaft investigated as possible cause of leakage reported 1880, but found to be practically dry and tipped full of puddle clay
1930	Leakage observed at toe of downstream slope
1931	Shaft dug within depression on upstream face found in 1930 to depth of 8.5m where decayed vegetation was found at level believed to be the original ground level. No culvert, shaft or water was found, and area made good with clay.
1964	Extreme flood, 1m above TWL for two days. Damage to lower part of primary spillway, erosion of right bank adjacent to toe.
1966	Spillway repaired with reinforced concrete.
1969/70	Auxiliary spillway 76m long, built over southern section of embankment. Sill 260mm above original spillway. (note that recent survey shows this 185mm above the crest of the primary spillway)
1973	Significant discharge down the auxiliary spillway in July
1975/81	Depressions found on upstream face of dam. Reservoir drawn down. Exploratory shafts sunk at various locations on the upstream and downstream face of the dam. A shaft was sunk through the dam and discovered a hitherto unknown culvert and old shaft. During the placing of a clay blanket, a 1.2m diameter masonry culvert was found beneath the dam, possibly for stream diversion during construction. Tracer tests showed this to have formed a leakage path through the dam. Possible route for leakage on the line of the old stream bed. Subsidence observed at bottom of auxiliary spillway.
1981	A pre-cast wave wall was added to the embankment crest.
1982	Retaining walls of auxiliary spillway raised.

1983/84	A line of cement grout was injected under pressure to ensure the clay core was sealed. The line of grouting was 60m along the crest and centred on the point where the original streambed passes under the embankment. It extended some 45m under the left side of the auxiliary spillway crest.
1985	17 No. 75 mm diameter pressure relief holes were drilled through the toe of the auxiliary spillway slab. The holes were backfilled with the lean mix no fines concrete.
1988	A large scour hole developed in the channel at the base of the auxiliary spillway towards the downstream end, undercutting the reinforced concrete toe of the spillway structure. The void was backfilled with stone
1998	Flood and operation of auxiliary spillway. Level 0.5m above TWL and 0.24m above crest of auxiliary spillway. Left hand wall overtopped for 6 hours.
1999	The channel at the base of the auxiliary spillway was rebuilt in reinforced concrete and the take off to the upper feeder was sealed.
2007	Significant discharge down the auxiliary spillway in December
2008	Lower left side wall of the auxiliary spillway was raised with “bus shelter” coping added to the top. Eccentric plug valves were fitted below the original guard and control valves on the draw-of pipes.
2012	Both draw-offs pipes re-lined with a 6.5mm thick internal resin liner capable of withstanding a 20m internal head.
2019	Extreme flood event, auxiliary spillway badly damaged, potential for breach

Sequence of events leading up to the spillway failure

This sequence of events has been compiled from records kept by CRT, engineers and others responding to the event, the Met Office, the EA, the emergency services and members of the public who raised the alarm.

About 160mm of rain fell on the catchment between Saturday 27 July and Wednesday 31 July 2019. The rain was well forecast and fell in two separate events, the first occurring over the 27 to the 29 July closely followed by a more severe event from the 30 July to 1 August 2019.

The sailing club that uses the reservoir reported a sharp rise in water level of between 100mm and 300mm on the 28 July. The first peak of the storm hydrograph was recorded on that day.

A Yellow Weather Warning was issued by the Met Office at 11:14am on the 30 July and updated at 9:37am on the 31 July.

At about 11am on the 31 July, a member the Operations Team at CRT visited Toddbrook Reservoir and closed the valve to the canal feed.

At around 4pm on the 31 July members of the public posted photos and video clips on the internet (fig 4).

Figure 4. The Spill at its height on the 31 July 2019

Around 4pm on the 31 July, the peak of the hydrograph of the second event was recorded. Around 9pm the reservoir reached its maximum water level during the event.

On the 1 August at 8am, a park keeper reported to a member of the Operations Team at CRT that he had not observed any problems when starting his shift.

At 8:30am, a member of the public reported having seen muddy water spurting up from the joints in the spillway. After the event, panels at the foot of the chute were seen to be pushed upwards and it was apparent that a slurry of water and embankment fill had been discharged in some quantity.

Around 9:00am the first panel is thought to have collapsed (fig 5). This was reported at 9:45am by a second member of the public as a section of concrete about “15ft x 20ft” being broken away and that the “earth bank is starting to wash away”. A void could be seen to have formed beneath the slab. Note that at this stage, discharge down the spillway had significantly reduced.

Figure 5. Collapse of the First Slab into the Void (note the slabs that have been pushed up towards the foot of the spillway chute).

At 10:00am CRT Operations staff arrived on site and opened the draw-off and scour valves. A loud crack was heard. Four panels were observed to have fallen into the void (fig 6). A photograph of the spillway was sent by the CRT Surveillance Operative to senior CRT staff.

At 10:46am on 1 August a further warning was issued by the Met Office

At about 11:30am the partially opened inlet sluice to the by-wash channel was opened further. Boards across the channel further downstream, which control the split between the compensation flow by-passing the reservoir and water turned into the reservoir, were removed later.

At 12:00noon emergency services started arriving on site

At 1:50pm the EA issued a warning for severe flooding – danger to life for the River Goyt downstream of the reservoir, which was removed on 8 August.

At about 2:00pm the next row up of panels started to collapse. Quantities of eroded material were still being discharged from the void at this stage.

Figure 6. Four Sections of Panels Collapsed into the Void

By 2:30pm the next column of panels had collapsed. The top row of panels started to collapse, and the left-hand chute wall was undermined (fig 7).

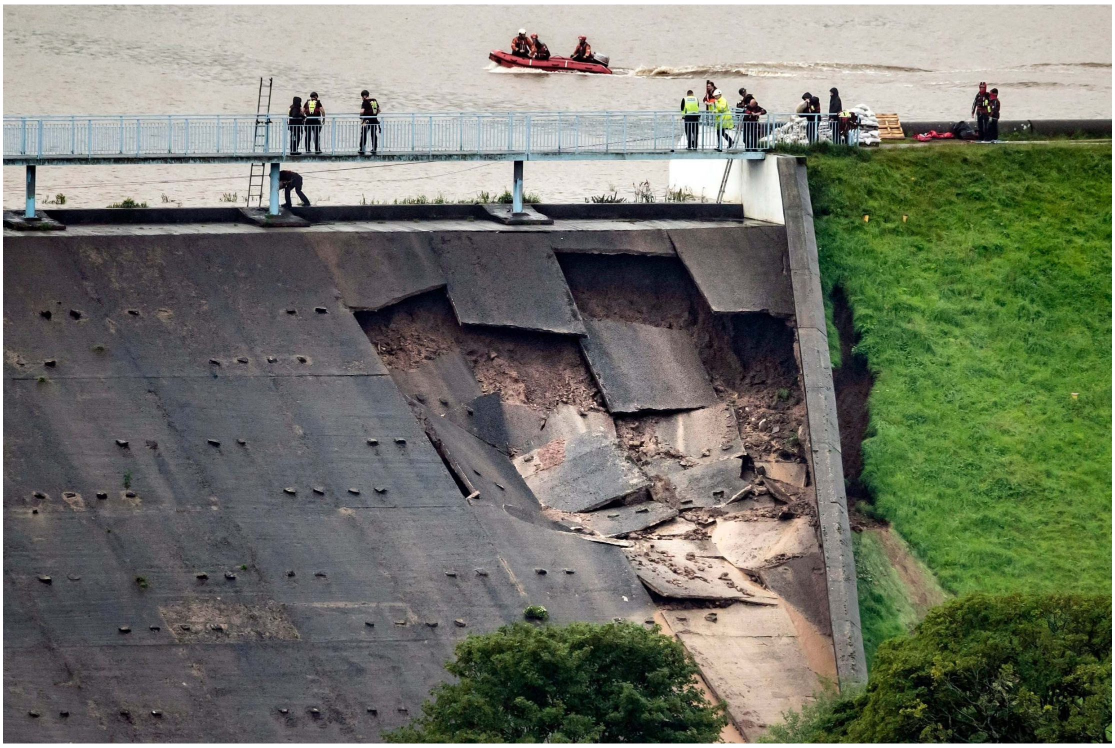


Figure 7. The Final Stage of Collapse of Panels. Note the undermining of the left-hand chute wall.

The failure mechanism

The Panel has arrived at their explanation of the failure by considering the history of the development of the reservoir, operation and maintenance of the auxiliary spillway, and the events leading up to the failure. They also took into account observations made during the event, the inspection of the reservoir during their site visit, and the observations of the engineers who responded to the emergency. The CRT and the EA have provided the panel with access to many documents relating to the reservoir, all of which have been reviewed. In the sections that follow, the convention is for left and right to relate to the direction of flow down the spillway that is as observed from the crest of the spillway looking downstream.

Observations from the 31 July and 1 August

The photographic and video evidence from the 31 July shows a substantial discharge down the spillway, with high velocity flow, and aeration caused by the impact of the flow with the “plums” embedded into the surface of the chute. There is a large roller wave along the left-hand flank wall and the flow lifts away from the spillway at the lower left-hand side to jump over the primary spillway channel into the grassed area beyond. A distinct brown staining of the flow to the left-hand side of the spillway can be observed at this time, which is consistent with the transportation of silt and fines (fig. 4).

Early on the morning of the 1 August, as the discharge down the spillway had started to recede, brown silty water was observed to be spurting from joints in the lower part of the chute on the left-hand side. This may have been occurring for some time but remained unobserved because it was masked earlier by the high flow down the spillway chute. A single slab higher up on the left-hand side of the chute then fractured and fell into a void that had been eroded below the slab (fig.5). A member of the public reported that a large section of slab had been broken away and that the “earth bank is starting to wash away”. Also evident at this time were some lower slabs that had been lifted and fractured and that silt was being discharged in quantities from open joints around these slabs (fig. 5). There is video evidence that during the day large quantities of silt and other material were being eroded and transported down the spillway and into the river beyond. The void below the slab progressively enlarged and as it did so, more slabs collapsed into it (figs. 6 and 7). It is estimated that overall a total of around 400 m^3 of embankment fill, that is about 800 tonnes, was eroded and transported into the river downstream.

Assessment of the spillway

Members of the Panel undertook a close inspection of the remains of the spillway during their visit to the reservoir, and reviewed drawings of the spillway and previous reports of inspections and remedial works.

The spillway appears to have been poorly designed. The key deficiencies in design are:

- the concrete slabs of the spillway chute are too thin
- the slabs do not have sufficient reinforcement, either structurally or for surface crack control
- the dowel bars[5] in the transverse joints are inadequate
- there are no dowel bars or water bars[5] in the longitudinal joints
- there is no underdrainage to the spillway.
- there is no cut-off between the concrete spillway crest and the puddle clay core of the dam

In addition, maintenance has been intermittent over the life of the spillway, and this is likely to have contributed to its deterioration. In particular, the Panel noted

- lack of sealant in some of the joints between the spillway chute slabs
- lack of repair to cracks in the chute slabs and the spillway crest

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[5] Dowel bars are steel bars that join one slab to the adjacent one. They help to prevent movement of one slab from the next. Water bars are flexible membranes between slabs that are designed to prevent water penetrating beneath the slabs. Both are installed at the time of construction. At Toddbrook, dowel bars and water bars are only fitted to the transverse joints in the spillway chute.

- vegetation growing in cracks and joints
- saplings and small bushes growing in cracks and joints. This vegetation must have been deep rooted in order to have survived the flood event

Possible causes of failure

The Panel then considered the possible causes of the large void under the slabs which led to the spillway failure. In doing so, they took particular account of the following:

- the Auxiliary Spillway was poorly designed, insufficiently robust and probably never fit for the purpose of conveying the probable maximum flood. The design was well below the standard of design that would be expected of more modern spillways.

- the spillway lacked a cut-off into the clay core of the embankment and also underdrainage. The slabs were very thin (150mm) and virtually unreinforced. Some dowel bars and water bars were provided in transverse joints between slabs, but not in the longitudinal joints between the slabs.

- maintenance over the years had been intermittent with extensive plant growth in cracks and joints for prolonged periods, suggesting open passageways to the embankment beneath. Generally, the slab concrete remained sound but there was honeycombing and/or deterioration at some joints, some missing chute plums, some cracking and evidence of significant prior plant roots through joints and in some cases through slabs.

- the left side wall of the spillway cuts obliquely across the line of chute flow for reasons that are not clear. This would lead to flows “bunching” along the left side wall. The side wall was raised at one point to better contain likely flood flows.

- in 1983 some 45m of the embankment core under the left side of the spillway was grouted as a precaution against seepage and internal erosion. Some grouting also took place under the crest slab at that time.

- the crest dips downwards towards the left and the crest slabs feature open joints and cracks. The core beneath the crest slabs shows signs of local settlement and alteration, possibly though freeze-thaw action and desiccation. Investigations indicated some alteration and hardening of this upper zone of the core.

- in all recent years, the crest slabs and their foundations have been underwater and saturated for prolonged periods. This would have been slightly greater on the left side of the auxiliary weir. Given the porous nature of the headworks, seepages would likely have occurred at such times though joints and cracks and possibly even through the upper zone of the core. There is some evidence of long-term seepages from the crest causing possible erosion and some settlement to chute slabs.

The factors which either immediately caused or contributed to the collapse are discussed and summarised below. They are explained in more detail in appendix B.

The effect of long-term crest seepages

The core and immediate downstream fill were intact when later exposed by the collapsed slabs. However, there is some evidence to suggest long-term seepages from the crest causing erosion and some settlement to chute slabs further downstream. Otherwise the embankment fill material is generally well graded and considered generally not prone to suffusion (migration of fines into coarser material). Some records mention seepages observed downstream as being occasionally “ocherous”, however this relates to iron staining which is considered more likely to come from groundwater.

The downstream shoulder (fill) material is permeable as shown by measured phreatic water levels within the embankment. Any seepages would therefore have tended to flow down through the fill by gravity. Nevertheless, evidence from satellite data appears to indicate that there may have been some prior settlement of slabs in the vicinity of the first upper slab collapse and at another location nearer the chute centre, during the months leading up to the collapse (note that as the data was obtained late in the Review, the Panel have not had the opportunity to validate it). This would suggest some progression of flows and associated erosion further down the chute and under the chute slabs. Ground penetrating radar imagery indicates the third row of panels to be quite thin and feature more localised voids beneath them than for example, the fourth row immediately below them. Localised settlement could have contributed to opening and extending the cracks known to pre-exist in the area of initial upper slab failure and would help explain why failure of the chute occurred in that location. There is also evidence of occasional seepage through the crest of the spillway on to the surface of the chute at a number of locations but particularly on the left-hand side. This may have found its way through longitudinal joints between the chute slabs which are not fitted with water bars, and could have caused further deterioration.

There is no evidence that all this would have caused the large void eventually revealed under the chute slabs, nor would it have been likely to supply the large volumes of water needed to displace and saturate 800tonne (approx. 400m^3) of earth fill during the flood event. Had a large void formed over a period of time prior to the event, the spillway slabs would have had to remain in place unsupported. The evidence from their collapse during the event, and their poor design, indicates that this would have been highly unlikely.

The likelihood of hydraulic fracture within the upper core

Many of the points in the preceding case apply here. While the water level in the reservoir was 200mm to 300mm higher than in previous years, that is a relatively small rise over and above reservoir levels that have been reached consistently in previous years. It would not explain why the initial slab failure occurred where it did. While there can be speculation about such a mechanism there is no evidence for it.

Effects from the oblique left side wall

The initial slab collapse was some distance away from the side wall. The under-mining of the side wall occurred quite late in the flood event (see figs. 6 and 7) and according to witnesses, resulted from an inwards collapse of the ever-widening scour hole, in effect extending the existing scour outwards and under the wall. The oblique angle of the wall would not have affected seepage regimes in the embankment underneath the wall.

All this suggests that while the oblique wall was undesirable with regard to spillway chute flows, it would have had little or no effect on the scour processes which occurred during the flood event.

The possibility of an earth slip or slide being a factor

While aggregate bags were being dropped onto upper zones of the embankment, the downstream areas were being monitored by laser. This revealed no movements to the lower zones. This indicated that there was no underlying slip circle, or other rotational failure, taking place. Photographic evidence shows the scoured material to have essentially liquified rather than remained solid.

The likelihood of stagnation zones developing on the chute during the flood event and injecting water through cracks and joints into the fill beneath the chute

Several lines of vegetation, implying open joints and cracks, can be seen in earlier photographs in the zones immediately above the upper slab which initially failed. Any vegetation, and also any chute plums in those locations, would have developed stagnation zones, injecting pressures and flows into the cracks. When a high velocity flow hits any obstruction, the kinetic energy in the flow is converted into pressure. On a spillway, this pressure can inject flow through joints and cracks into the material below. Large volumes of flow can be injected through quite narrow joints/cracks and the effect can be caused by relatively small obstructions to the flow such as the edge of a crack or joint. A more detailed explanation is given in Appendix B. The localised settlement of slabs referred to earlier could have contributed to enhanced slab cracking and hence aided the process of crack injection.

Calculations using the USBR Report on uplift and crack flow resulting from high velocity discharges over open offset joints[6] indicate possible injection volumes of between 240,000

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[6] USBR Report DSO-07-07 “Uplift and Crack Flow Resulting from High Velocity Discharges over Open Offset Joints”, Dec 2007

and 1.8 million litres of water during a 12-hour flood event, depending on assumed crack length and features (see appendix B).

This is seen as the most likely explanation for the introduction of the large quantities of water needed during the flood event for the extensive scour which occurred due to liquifying the local embankment fill. It also targets precisely the area which subsequently failed. It should be noted that the erosion of soil foundations under lined chute slabs due to stagnation pressures under flow, injecting water into cracks and joints, is a well-established mode of chute lining collapse and failure and has been well researched and quantified in the US. A US research study into the phenomenon refers to it having happened on “numerous” occasions. However, knowledge of this mechanism is perhaps less well known in the UK.

Most likely sequence of events

During the overspill event (which lasted almost 24 hours with high flows likely for some 12 hours) stagnation pressures developed in at least one upstream crack or joint just above the upper slab which initially collapsed. The injection caused by the stagnation pressures during the flood event injected considerable amounts of water under the chute slabs into the area at and above the upper slab which collapsed initially.

The rate of water injection exceeded the capability of the local embankment fill to drain naturally, resulting in its liquefaction into a “slurry” of water, clay, silt and sand. This liquified fill then raised the chute slabs sufficiently, with flood water still flowing over them, such that the slurry flowed beneath them, eventually permanently lifting and displacing some lower ones. This occurred early in the event and with clay, silt and fine sand exiting from the sides of the raised lower slabs throughout 31 July 2019 and into the spillway flow. This was the day preceding the initial upper slab collapse.

An increasing large void developed under the upper slabs as the material was removed. As the flow reduced, the under-pressure ceased and the water drained from the void, the 3rd slab along from the left side and also down, lost support and collapsed downwards into the scour hole. Saturated fill and water then slowly escaped, residual flows continued and the sides of the scour hole “relaxed” and widened leading to the collapse of further slabs and also under-mining the left side wall of the spillway. Note that some pre-saturation of the fill may have been present in upper zones due to seepages under and through the crest slab but for the reasons already stated these are not seen as the prime driving mechanism behind the major scour which occurred.

Possibility of a much earlier upper slab collapse

An earlier upper slab collapse (during the main part of the flood event on 31 July/1 August) would have been evident to those watching, and recorded in video and photos, not only as a “hole” but in terms of vast quantities of muddy water emanating from the upper areas of the chute. There is none recorded. Had the embankment material been opened earlier to direct chute flow it would have resulted in far more extensive and rapid erosion than occurred. It is concluded that the first upper slab collapse occurred at some time between 08.30 and 09.45 on the morning of 1 August 2019.

Cause of Failure

The sequence of events described in the preceding sections resulted from a spillway that was poorly designed and not fit for purpose, coupled with intermittent maintenance over the years. It was the ability of flow to inject through the chute and erode the underlying fill that led to the failure. It has not been possible to say whether it was the poor design or the intermittent maintenance that was the primary cause of this failure. It is unlikely that the spillway would have survived the probable maximum flood even with good maintenance. However, had the spillway been kept clear of vegetation, the joints cleaned out and re-sealed, cracks repaired, and pressure relief holes kept functioning, then it is possible that the spillway may have survived the event of the 31 July and the 1 August 2019, which was very much less than the probable maximum flood.

Given that the puddle clay core was almost exposed, the dam may have breached had the reservoir level not been reduced. If the event had been more intense, or extended for a longer period, catastrophic failure of the dam may have occurred. This could have happened without warning and at night such that widespread evacuation would have been impossible.

Findings and recommendations

Having established the most likely mechanism of collapse of the spillway, the Panel then set about understanding the factors that led to the failure and whether it could have been avoided. They reviewed the events leading up to the incident on the 1 August and what actions could have been taken in anticipation of the event. They asked CRT to undertake additional modelling of the rain event and reservoir response, through their consultants Mott MacDonald, to assist with this.

The Panel reviewed in depth the reports of periodic and supervisory inspections over the history of the dam and interviewed the current Supervising and Inspecting Engineers. This helped them to understand if the inspection process was compliant with the legislation and to determine how that process might be improved. They also looked into the operation and maintenance regime at the dam, and interviewed staff from CRT's Operations and Engineering teams and CRT's Director of Asset Improvement. CRT provided a large number of relevant documents from their archive to assist with this. This helped the Panel to understand how the reservoir was operated on a day to day basis and how well it was maintained.

The EA, as the regulator and enforcement authority, have an important part to play in the overall process that ensures reservoir safety. The Panel wanted to understand their role in the management of reservoir safety. The EA provided copies of relevant documents from their records to assist the Review. The EA staff who direct and manage their regulatory and enforcement functions together with staff responsible for the execution of those duties on a day to day basis were also interviewed.

The Panel is grateful for all those who contributed to the many lessons learnt during this investigatory part of the Review. Findings and recommendations are set out in the sections that follow.

The events leading up to 1 August 2019

About 160mm of rain fell on the catchment between Saturday 27 July and Wednesday 31 July 2019. The rain was well forecast and fell in two separate events, the first occurring over the 27 to the 29 July closely followed by a more severe event from the 30 July to 1 August 2019. These were rare events, with the second having an annual probability of occurrence of around 1%. They resulted in a substantial discharge down the auxiliary spillway for a sustained period. Though serious, these events were considerably less than the probable maximum flood which the spillway should have been able to accommodate. A number of previous events are reported (1973, 1998 and 2007) when a significant flow was discharged down the spillway, apparently without incident (fig 8). Up to the 1 August 2019 the reservoir had remained full for many months.

Figure 8. Spillway in Operation, July 1973

CRT confirmed that no action had been taken as a result of the Yellow Weather Warning issued on the 30 July as the auxiliary spillway at the reservoir had been designed to safely convey the probable maximum flood.

The draw down facility at Toddbrook Reservoir consists of two pipes through the reservoir embankment controlled by valves at their outlet. Together they can deliver a reduction in level of about 0.65m per day when fully open, and with no inflow to the reservoir. Had the valves been opened as soon as the severe weather was forecast, the amount of draw down would have been insufficient in itself to prevent spill over the auxiliary spillway. The reservoir draw-down facilities appear to have been operational as both valves were fully opened on the morning of the 1 August, though this was towards the end of the event.

In addition to drawing down the reservoir level, there are facilities to divert the inflow away from the inlet to the reservoir and into the by-wash channel, and also to block off the by-wash flow from returning to the reservoir via a side weir. These were not operated until the afternoon of the 1 August.

CRT, assisted by Mott MacDonald, have used their latest computer model of the reservoir and upstream catchment to investigate what might have been achieved if this flow diversion had been brought into play earlier. The results show that with the draw-off valves open and the inlet flow diverted into the by-wash channel, the peak flow over the auxiliary spillway would have been reduced from about $9.5m^3/s$ to $8.3m^3/s$, the spill volume reduced from about 423000 to $164000m^3$ and the duration of spill reduced from about 100hours to 40hours. It is not possible to say for certain, but it is likely that it would still have been possible for the auxiliary spillway to have failed in some way even if these measures had been taken. Moreover, the additional flow passing along the by-wash, when added to the flow from the primary spillway, may have created its own problems, both in the vicinity of the reservoir and downstream.

Supervision and inspection

Inspection of the Reservoir

The reservoir has been inspected by a Qualified Civil Engineer at regular intervals since the Reservoirs (Safety Provisions) Act 1930. In more recent years these have been every 10 years, as provided for in the Reservoirs Act 1975, apart from 2010 when an interval of 5 years had been specified by the previous Inspecting Engineer. The latest inspection was carried out in November 2018. It involved a thorough walk-over inspection of the reservoir, including the dam. From previous inspections attention was drawn towards the issues of mine-workings. Also, there was concern that on the draw-offs there were valves only on the downstream end of the pipes (a common practice when the dam was built but not considered to be good practice today). The draw-off pipes had been recently lined to reduce the risk of failure.

The initial impression was that overall the embankment appeared to be in good condition and corresponded to the line, levels and gradients shown on the drawings. The Inspecting Engineer had concerns over the blocked pressure relief holes on the spillway chute, but at that stage was not unduly concerned over the condition of the spillway other than the need to seal slab cracks and repair sealant where necessary. A close inspection of the auxiliary spillway was not undertaken over the full area due to difficulty in accessing the central steep section without ropes for safe working. It is not unusual for a reservoir inspection to be completed without a close inspection of a spillway by direct access.

The cause of spillway failure was likely to have been influenced by small detail at joints and cracks in the slab. It is important these are closely inspected. It is also important that Inspecting Engineers and Supervising Engineers are fully aware of the potential mechanisms of failure of spillway chutes so that they better understand what to look for.

1. **I recommend that the EA commissions new guidance on the failure mechanisms of spillways and how to undertake spillway inspections.** This should include guidance on spillway design based on international good practice and lessons learned from incidents in the UK.

2. **I recommend that Inspecting Engineers and Supervising Engineers inspect spillways closely and by direct access during their visits, with a minimum of one year between Supervising Engineers' spillway inspections.**

3. **I also recommend that the Owner[7] should make the necessary safety preparations in advance to enable such close inspections to take place as a matter of routine.**

After the inspection, the Inspecting Engineer reviewed the drawings of the spillway construction that were contained in a package of information provided by the Supervising Engineer prior to the inspection. It became clear that the auxiliary spillway was not as substantially constructed as might have been expected. Even then, the drawings were not comprehensive. Water bars and dowels were shown on the transverse joints, but no details were given of the longitudinal joints. It was assumed that they would have been constructed in the same manner as the transverse joints (note that following the incident it was discovered that this was not the case). There was apparently no underdrainage provided as part of the spillway construction, or a cut-off under the crest slab. This altered the view of the Inspecting Engineer, who now believed that the spillway might be at risk of hydrodynamic damage. He therefore required an investigation of the spillway together with the necessary follow on action as a Measure in the Interests of Safety (MIOS). This was communicated to CRT when he submitted his draft report at the beginning of April 2019. The Inspecting Engineer required the investigation to be completed by October 2020.

Looking back through previous inspection reports it was apparent that Inspecting Engineers tended to rely on what previous Inspecting Engineers had written. They then worked forward from that. Original records have not always been sought out and checked and it is remarkable that only at the 2018 inspection do the record drawings of the spillway construction appear to have been scrutinised. We understand this is not unique to this reservoir, and believe that, at least in some cases, this may be because records are not always easy to access. However, in 2018, the Supervising Engineer for Toddbrook prepared a comprehensive package of information, including the record drawings, for the Inspecting Engineer prior to his inspection. Going forward, this should be standard practice for all inspections at all reservoirs. When this is not forthcoming the Inspecting Engineer should require it.

The Inspecting Engineer who conducted the 2010 inspection of the reservoir had not seen the record drawings of the spillway construction. When he eventually saw the drawings (when he checked the 2018 inspection report) he was concerned. Had the drawings been reviewed at the time of the 2010 inspection, the deficiencies in the spillway design might have been identified then and remedial action taken. This may have prevented the failure of the spillway in 2019.

4. **I recommend that Owners[7], under the guidance of the Supervising Engineer, make available an appropriate and well-structured package of information on the reservoir to the Inspecting Engineer well in advance of a planned inspection.** The Inspecting Engineer should not overly rely on information in previous inspection reports nor should they have to search out relevant information from archives**.**

The report of the 2018 inspection is thorough and detailed. The Inspecting Engineer identified a number of risks to the safety of the reservoir including that of “hydrodynamic damage to the secondary overflow channel” that was both “significant” and “credible”. Later in his report, the Inspecting Engineer specified a measure in the interests of safety (MIOS) relating to the spillway. It required the Owner to “(a) carry out a review of the secondary overflow channel to demonstrate that it is not at risk of hydrodynamic damage during significant overflow event caused by high velocity flow in the channel or water pressure beneath the base slabs and (b) carry out any necessary improvement works”. 18 months was given to complete part (a) from the date of the report (in effect 2 years from the date of the inspection), with the period for part (b) being determined by the Qualified Civil Engineer who would oversee part (a).

The requirement for MIOS relating to the spillway was written in a style often found in inspection reports. However, it did not convey any real sense of urgency, given the seriousness of the risks identified. The 18month period was what the Inspecting Engineer thought would be reasonable to complete the investigation, and he has subsequently provided a summary of the necessary work to support this. Where significant and credible

[7] In the legal sense “owner” means “undertaker”

risks are identified that threaten the safety of a reservoir then the seriousness of that should be unequivocal in the wording and timeliness of the MIOS.

5. **I recommend that requirements for Measures in the Interests of Safety (MIOS) should be worded so that they unequivocally convey the seriousness of the risk and the urgency of implementation.**

   The Panel have considered whether the Inspecting Engineer should have required a full or partial draw down of the reservoir as a precaution whilst the condition of the spillway was investigated. It is clear from his report that the Inspecting Engineer had some doubt over whether the spillway was able to fully meet the requirement of conveying the probable maximum flood. From the recent modelling work undertaken by CRT, it can be concluded that drawing down the reservoir level by around 5m would likely have avoided the damage to the spillway that led to this incident. The reservoir had been drawn down in the past for a number of years whilst defects had been investigated and remediated. However, Inspecting Engineers may not want to see precautionary measures such as reservoir draw down specified whenever they require a MIOS, unless there is an apparent and imminent threat to safety. A rare future rainfall event might not be considered “imminent”. However, when justified, precautionary measures should be deployed during the delivery of MIOS.

Justification of precautionary measures should be based on the risk (probability and consequence) of an incident compromising the safety of the dam during the period specified for completion of the MIOS. The Inspecting Engineer should consider what measures might be effective at managing that risk and the consequences of their implementation.

6. **I recommend that the EA commissions new guidance to assist Inspecting Engineers in specifying suitable precautionary measures to be in place during the period for completion of Measures in the Interests of Safety (MIOS).**

In addition to the MIOS, the Inspecting Engineer also specified measures to be undertaken under Section 10(3) (b) of the Act (Maintenance), which are often referred to as statutory maintenance. These covered the clearing of vegetation and the repairing and re-sealing of joints and cracks to the auxiliary spillway, to “*prevent spill water passing under the base slab and pressurising the underside*”. He asked that further maintenance be reviewed on a 2 yearly basis and undertaken if required. It would be reasonable to conclude from the wording in his report that the re-sealing of joints and cracks etc. should be done as soon as practicable, although this is not explicitly stated. However, the IE informed the Panel that he had asked the SE to seal all cracks and joints on the auxiliary spillway, clean out all pressure relief holes and drill additional holes at the time of his inspection. It would appear that this had not been completed some 8 months later when the incident occurred.

A draft report was issued to CRT on 4 April 2019. Comments were returned promptly (though the Panel understands that this may not be the case with all Owners) and after receiving these the Inspecting Engineer submitted the final version on 30 April 2019. The Inspecting Engineer reviewed other relevant material after the inspection prior to drafting his report. Given the significance and credibility of risks to the reservoir, our view is that more could have been done to communicate the urgency of the MIOS and statutory maintenance to the Owner at an earlier stage.

One way of achieving this might have been through the Supervising Engineer who accompanied the Inspecting Engineer during his inspection. Verbal feedback was given to the Supervising Engineer at the end of the inspection. The Panel were told by the SE that this did not raise any particular concerns over the safety of the spillway. It was only later when the Inspecting Engineer reviewed the construction drawings for the spillway that he realised some of the implications of the poor design (though the drawings did not give a complete picture). He then apparently changed his view of the risk the spillway posed. The risk is clearly set out in the Inspecting Engineer's report. However, when the 18month period to complete the investigation is added to the 6th month period to finalise the report and then coupled with the wording used to describe the MIOS, it is easy to understand why the requirements might not have been considered to be that urgent. To avoid doubt over the findings and requirements of the Inspecting Engineer there should be an early meeting between the various parties as soon as practicable after the inspection.

7. **I recommend that a formal meeting between the Inspecting Engineer, the Supervising Engineer and the Owner[7] should take place immediately the Inspecting Engineer has determined the findings of the inspection and formulated any requirements for Measures in the Interests of Safety (MIOS and statutory maintenance.** This meeting should take place within one month of the date of the inspection**.**
8. **I also recommend that inspection reports should be submitted without delay, following this meeting.** Owners[7] should respond to drafts promptly.

Supervision of the Reservoir

The Reservoirs Act 1975 requires the Supervising Engineer to advise the Owner on the condition of the reservoir "at all times". Supervising Engineers are expected to visit the reservoir at least once a year. Toddbrook Reservoir had been inspected by a Supervising Engineer twice every year for most of the time over which records exist. Since CRT took over responsibility for the reservoir, there have also been bi-monthly inspections by CRT asset inspectors.

The earlier SE reports were somewhat brief, but this had improved in later years since a standard format for reporting appears to have been introduced. The process, therefore, appears to be compliant with the legislation. However, the annual statements from supervising engineers say very little and are unlikely to be particularly useful to Owners in helping them understand safety risk or prioritise investment.

9. **I recommend that the EA commissions guidance for Supervising Engineers on the production of their visit reports and annual statements so that progress on delivering MIOS and statutory maintenance and reasons for any delay are clearly set out, and the current condition of the reservoir is clearly communicated to the Owner[7]**

When the Panel met with the Supervising Engineer, they noted that up to 2018 he was mostly concerned over the stability of the dam. He believed that the dam was stable as long as the pore water pressure in the downstream fill was not too high. But he had concerns over the poor maintenance and reliability of the piezometers and the failure to monitor levels since the 2005 inspection. Also, he was concerned over the monitoring of flows in the mine drainage adits and the seepage flows from the reservoir (see appendix C). He identified maintenance of the spillway as a particular problem, for example the stumps of trees left in pressure relief holes. Despite numerous requests for maintenance, little had happened over a period of time (the section on Maintenance and Operation provides more details). He was concerned that this had not been addressed.

The Inspecting Engineer shared his findings with the Supervising Engineer on the day of the inspection and there were apparently no surprises when the draft report arrived in April 2019. New areas to be addressed as a result of the inspection were the spillway and the capacity of the by-wash channel. The Supervising Engineer explained that there was nothing so significant in the recommendations as to push this up through the hierarchy of CRT. The Supervising Engineer wasn’t concerned particularly about the findings or requirements but knew that it would take some organizing to get everything done by the deadlines.

On the 2 July 2019 the Supervising Engineer sent an e-mail to the CRT Asset Manager (to whom he reports) relating to the measures in the interests of safety (MIOS) and other works and assessments at Toddbrook. This was then posted to the CRT Regional Engineer to go to the monthly approvals board. The Supervising Engineer did not question the measures specified by the Inspecting Engineer. He did not see this as his role, other than the practicalities of addressing the measures such as timescale. In this case the time required to deliver the MIOS was a challenge in his view. The issue was not that the actions could not be done in the timescale, but the procedures to get them done.

In a meeting with the CRT Principal Asset Engineer and CRT Regional Engineer the Panel was told that as a result of the inspection, the reservoir moved down from a condition grade C to D[8], but the measures in the interests of safety (MIOS) required by the Inspecting Engineer were not considered to be that urgent, as an 18 month period for completion had been specified. They told the Panel that CRT could have acted immediately if the Supervising Engineer had said that it was urgent. Based on the investigation conducted after the event, an investigation could have been completed in a few months. If something significant had been identified it would have been acted on.

The Panel has carefully considered the role of the Supervising Engineer. He clearly plays an important part in managing the safety of the reservoir. He has been in this role for a significant time and typically supervises 25 CRT reservoirs. We would have expected him to have had the necessary authority and support to ensure that any of his recommendations or commentary on lack of progress would be taken seriously. Yet from the Supervising Engineer's reports and statements it appears that this has not always been the case, with actions not progressed, sometimes for several years. This also appears to be the case with previous Supervising Engineers for the reservoir.

If a Supervising Engineer is concerned over lack of maintenance, then s/he can inform the EA or call for a Section 10 inspection. However, the Supervising Engineer clearly did not believe that his role was to challenge either the recommendations of the Inspecting Engineer or the response he habitually received from CRT over his reports. According to the Organogram supplied by CRT, the Supervising Engineer reports into the Engineering Team at a relatively junior level, and within the same team that would implement any recommendations he might make. It is the EA's view that many SEs are subject to commercial pressures and may have difficulty in influencing their employer.

There is no requirement under the Reservoirs Act for a Supervising Engineer to be independent as there is with an Inspecting Engineer. There are advantages in Supervising Engineers being employed by the Owner since it is the Owner whom they advise on a regular basis. However, supervision of reservoirs can only be effective if Supervising Engineers are properly supported and have the means of reporting directly to the person nominated for managing safety at a senior level in the Owner's organisation. Lessons can be learnt from other industries which manage safety in a more strategic way, where safety personnel are taken out of operational units and given a direct line of access to Director level.

10. **I recommend that Owners[7] manage the safety of their reservoirs in a way which gives Supervising Engineers reporting lines directly to the individual responsible for corporate safety at Director Level (or equivalent).** In the case of

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[8] Condition Grade C is denoted "Fair" by CRT and described as "Minor defects may develop into structurally significant defects in long term (generally > 10years) "and Grade D is denoted "Poor" and described as "Structurally significant defects leading to loss of stability in the medium term (generally 5 to 10years)". The Grades vary from A to E.

single owners[7], clubs or small companies, the Supervising Engineer should report directly to the owner or equivalent.

Legislation and regulations

The Environment Agency is the regulating and enforcement authority for reservoirs in England. They have had this role since 2003. Similar organisations exist for the other member states of the UK. The day to day aspects of ensuring compliance is handled by the Reservoirs Team. This consists of 13 staff, equivalent to 9 full time posts. They are arranged into two teams, one dealing with compliance and monitoring, and the other with enforcement. There are 2082 large raised reservoirs which come under the requirements of the Reservoirs Act (1975 as amended 2010), 214 of which the EA is the Owner. Management of these 214 reservoirs is kept separate from the regulatory and enforcement function.

Inspection reports and other reports are not reviewed in detail. The EA focuses on required MIOS, and statutory maintenance measures (since 2015). The requirements regarding MIOS and the associated timescales for completion are logged and monitored. It is not the role of the EA to determine their appropriateness – that lies with the Inspecting Engineer (and Construction Engineer where relevant). Since 2015 similar action is taken with respect to maintenance items which have now become enforceable. The EA would not normally expect an Owner to challenge the recommendations of an Inspecting Engineer (although the legislation does allow for this). This would be the role of the Supervising Engineer if there were concerns.

All MIOS (and now also statutory maintenance items) are followed up. When these have not been completed in the required time, they will be initially followed up informally with the Owner. If no action results, then the requirements are enforced. The EA has used its reserve powers twice in the last year to complete outstanding work.

Overall there is 97% compliance, so reservoirs are believed to be safe. The EA believe that the process is very rigorous. The EA consider the 2018 Inspection Report for Toddbrook to be good when compared with the overall standard of reports. The EA also considers CRT to have been one of the more responsible Owners based on their record of compliance.

The EA were asked about the link between compliance and safety. They stated that compliance did not mean the same thing as safety. Similar comments were made during interviews with CRT staff. There appears to be a fundamental issue here about the way reservoir safety is ensured. The monitoring and enforcement of reservoir safety as undertaken by the EA is based on compliance. It is likely that Owners take a similar approach (especially given that non-compliance can lead to prosecution). Yet, as the incident at Toddbrook so aptly demonstrates, a compliant reservoir might not necessarily be safe.

There is clearly a need to close the gap between compliance and safety. Yet relying on a rule-based compliance model might undermine the whole basis on which safety is judged, that is the expert opinion of competent and experienced engineers. This suggests that there is a need for a more fundamental review of how the Act is implemented, and how useful the supporting regulations and guidance are on this particular point.

11. **I recommend that there is a systematic review of how the current Reservoirs Act, and the associated Regulations and Guidance, are implemented.** This should consider the roles and responsibilities of qualified engineers, whether compliance with the Act is sufficient to ensure safety, and how safety is formally assured.

The panel has also considered whether there should be some form of scrutiny or checking of individual inspections but can find no argument to support this. In their view any additional checks would likely result in a division of responsibility (actual or perceived) which would lead to a worsening of the process. The responsibility for conducting an effective inspection should remain with the Inspecting Engineer. However, the legislation does allow the Secretary of State to make regulations to provide for the assessment of the quality of reports and written statements prepared by Inspecting and Supervising Engineers. So far this has not been implemented.

12. **I recommend that the Secretary of State requires periodic review of the safety management process, including sampling of the inspection and enforcement process to ensure these remain fit for purpose.** This should include engagement with other sectors charged with managing the safety of critical infrastructure to better understand the processes that are used there.

The EA also suggested that on completion of an inspection and the submission of the report, an Inspecting Engineer shall issue a certificate of "safe to operate" in the same way that a Construction Engineer certifies that the reservoir "may safely be used for the storage of water up to a level of, subject to the following conditions" on completion of major works. This suggestion is worthy of further consideration.

13. **I recommend that the potential of an Inspecting Engineer issuing a Certificate of Safe to Operate be explored. This should include a review of practice in other safety critical infrastructure sectors. It should also consider liability implications and whether some form of qualifying statement may be needed to accompany the certificate.**

Recognising that risk is not consistent across all reservoirs, or throughout their lifespans, the Panel has also considered whether a single maximum period between inspections is appropriate. At present, a Supervising Engineer can request an inspection at any time and an Inspecting Engineer may request an inspection to follow after a period of less than 10 years (a period of 5 years has been used on one occasion previously at Toddbrook). The reservoir stock in the UK is ageing and it would be appropriate to consider if the statutory maximum 10 year period between inspections is still appropriate in every case.

14. **I recommend that the statutory maximum period between inspections is reviewed to determine if it is still appropriate in every case in the light of the ageing reservoir stock.**

Operation and maintenance

The Panel has reviewed the inspection regime at the dam, interviewed operations staff, reviewed CRT documented maintenance procedures and received copies of proformas used in the twice weekly inspections. Operations staff are properly briefed as to what to look for, and how to report concerns so that action can be taken. However, once concerns are reported there appears to be no process for them to follow if work is not done other than to continue to report it. In part this might be due to the operations staff belonging to the operations team whereas the engineering team are responsible for commissioning the subsequent work. These teams belong to different reporting structures in the CRT management hierarchy.

There is a long history of intermittent maintenance of the reservoir. In 2006, the Inspecting Engineer noted that the auxiliary spillway had an appearance of neglect and lack of routine maintenance. He recommended joint sealing and cleaning, removal of vegetation and patch repairs. In the 2010 inspection the Inspecting Engineer noted that the spillway appeared to be in good condition (fig 9).

However, from 2015 to 2018 the Supervising Engineer stated, in four separate reports, that vegetation was again apparent on the spillway. There is also photographic evidence to support this (fig 10). The Panel has been provided with documentary evidence on vegetation clearance on the spillway in recent years. However, vegetation clearance is only part of the necessary maintenance. It also needs to include the cleaning out and resealing of joints, and the repair of any cracks.

Figure 9. Evidence of Good Maintenance with Joints cleaned out and Re-sealed, September 2008

As discussed earlier, the current Supervising Engineer was clearly concerned over his recommendations not being acted on in a timely manner.

15. **I recommend that Owners[7] ensure that their organisational structure provides a clear path of responsibility for routine and required maintenance.** For single owners[7], clubs and small companies the owner or equivalent should be directly responsible for maintenance

Figure 10. Evidence of Poor Maintenance with Vegetation and Saplings Growing in Joints, January 2018

There have been deficiencies in recording potential seepage flows and pore pressures in the dam due to inadequate maintenance of instruments. This is unlikely to have been material to the failure of the spillway though it could affect other important aspects of reservoir safety, e.g. embankment stability and leakage. More information on these aspects is included in Appendix C.

It is vital that all reservoirs are adequately maintained. This is particularly true where a spillway is situated directly on or adjacent to an embankment dam. There are many aspects, both detailed and substantial, that can compromise safety. It is possible that this is not sufficiently well recognised by Owners, and possibly some Supervising and Inspecting Engineers.

In his April 2019 report of his 2018 inspection, The Inspecting Engineer made three requirements for statutory maintenance to the spillway chute, covering the removal of vegetation and the sealing and repair of joints and cracks.

According to the Supervising Engineer's reports for 2019, little further action, other than the clearing of vegetation from the spillway, had been taken on the Inspecting Engineers recommendations by the end of May 2019, and it is understood that this was still the case at the time of the event. The outstanding maintenance items relating to the cleaning out and sealing of joints and open cracks should have been undertaken as soon as the Inspecting Engineer's views were known. Indeed, it should not have required an inspection to have put this in place. Given that the lack of maintenance to joints and cracks is likely to have been material to the failure of the spillway, early action on this after the inspection could possibly have avoided the failure.

Some maintenance matters are now a statutory requirement for the Owner, and the EA have produced a reporting form. However, the EA does not have the same legal powers to deal with statutory maintenance as they have with MIOS. There is no power to enforce by means of a notice nor any means to carry out works and recharge.

16. **I recommend that the EA makes Owners[7] more aware of the vulnerability of spillways to poor maintenance and repair. They should remind Owners[7] of their responsibility for the safety of their reservoir(s) and of the need for regular maintenance and repair of spillways at appropriate intervals, without waiting for Supervising Engineers or Inspecting Engineers to draw this to their attention.**

17. **I recommend that Owners[7] complete any outstanding maintenance of spillways urgently and that they respond promptly to the recommendations of Supervising Engineers. Inspecting Engineers should make full use of the provision for statutory maintenance when setting out their requirements following an inspection.**

18. **I also recommend that the Secretary of State gives powers to the EA to enforce by means of a notice outstanding statutory maintenance and the powers to carry out such works and recharge.** This may require a change in legislation.

During his 2018 inspection of the reservoir, the Inspecting Engineer observed that a number of regular measurements that might indicate potential defects to the dam were not being undertaken. Part of the reason for this was inadequate maintenance of monitoring equipment. Monitoring should generally be linked to the key failure modes of a structure.

Nowadays such monitoring can be quite sophisticated, for example using embedded strain meters to monitor stress changes and embedded fibre optic cable to measure the temperature changes associated with leakage. There is also an increasing trend to have such information relayed on a continuous, real-time basis, to a central monitoring station so as to minimise the response time for possible remedial action. Since the event, CRT has installed sensors at the inlet to the reservoir, along the by-wash channel and within the reservoir itself to monitor water levels.

In the case of spillways, over and above immediate visual inspection, non-intrusive methods such as ground penetrating radar can be used to check for potential voids under stabs. The initiation of such measures does not have to wait for a 10yearly inspection. The Panel would also like to draw Reservoir Owners' attention to the potential of Lidar data and satellite data to aid the real time monitoring of spillways on earth embankments. Early detection of movement could allow early interventions that might prevent future deterioration or failure.

19. **I recommend that Owners[7] regularly maintain all existing long-term monitoring equipment on reservoirs, so as to keep the equipment serviceable, and that they take and record measurements at appropriate intervals.** All measurement data should be retained in a usable format in the Prescribed Form of Record for ongoing use by Supervising Engineers and for use by Inspecting Engineers at their inspections**.**

Immediate actions for other reservoirs

Given that the spillway failure was due to poor design exacerbated by deterioration and intermittent maintenance, it is possible that there may be similar spillways in a similar condition on other dams. These may be limited to spillway designs of a similar age built over embankment dams.

20. **I recommend that the EA should urgently seek to identify any reservoirs with potentially similar spillway characteristics to those at Toddbrook.**

21. **I recommend that where these are identified, Owners[7] urgently arrange for detailed inspections to be undertaken, with any necessary precautionary measures put in place to ensure they remain safe whilst any remedial work is undertaken (**note that at the time of writing this action had already been progressed).

22. **I recommend that the EA urgently produces a technical note on the Toddbrook Reservoir incident to inform all reservoir engineers and owners of the lessons learnt from the incident.**

Summary of findings and recommendations

Summary of findings

In addressing the Terms of Reference for this Review, I have sought to answer four key questions:

- was there anything prior to 1 August that could have undermined the integrity of the spillway, and/or identified any actual or emerging issues with the dam?
- what was it about the events of 31 July and 1 August that led to the damage at this particular time, and were the immediate actions taken by CRT appropriate?
- were the actions taken by CRT, and the EA as the regulator, as a result of the November 2018 inspection, reasonable and proportionate?
- are there any changes to maintenance and inspection guidance or legislation that need to be considered in the light of what happened at Toddbrook?

My overall finding is:

The cause of failure at Toddbrook Reservoir on the 1 August 2019 was the poor design of the spillway, exacerbated by intermittent maintenance over the years which would have caused the spillway to deteriorate. It has not been possible to say whether it was the poor design or the intermittent maintenance that was the primary cause of failure on the day. With consistent good quality maintenance over the years leading up to the event the spillway may not have failed. However, it would have been unlikely to survive the probable maximum flood which is many times greater than the flood in which it failed.

My remaining findings are summarised below.

Was there anything prior to 1 August that could have undermined the integrity of the spillway, and/or identified any actual or emerging issues with the dam?

The original design of the auxiliary spillway was flawed for the following reasons:

- the concrete slabs of the spillway chute were too thin
- the slabs did not have sufficient reinforcement
- the dowel bars in the transverse joints were inadequate

- there were no dowel-bars or water-bars in the longitudinal joints
- there was no underdrain to the spillway.
- there was no cut-off between the spillway crest and the puddle clay core of the dam

All the above factors contributed in some way to the failure. The spillway was not fit for the purpose of conveying the probable maximum flood.

The lack of a cut-off allowed water to leak under the spillway crest when water levels in the reservoir were high and this is likely to have caused deterioration of the spillway over the years. Water has also been observed to seep through the crest and onto the surface of the spillway chute on the left-hand side from time to time. This may have penetrated the longitudinal joints, which were not fitted with water-bars and intermittently maintained, contributing to the deterioration.

Monitoring of levels of the spillway crest show some settlement over the life of the spillway. Notes in the reports of supervising and inspecting engineers indicate some settlement of the lower slabs of the spillway with some cracks forming. Satellite data received during the Review appears to show a sudden settlement of the spillway crest in July 2018, and settlement of the spillway chute in the area of initial failure starting in January 2019 and continuing progressively until the events of the 31 July and 1 August. Satellite data is not commonly used to monitor the condition of reservoirs in the UK. It appears to have great potential in identifying changes in reservoir structures that might otherwise go un-detected, and should be explored further to assess its value for general implementation.

What was it about the events of 31 July and 1 August that led to the damage at this particular time, and were the immediate actions taken by CRT appropriate?

The rain that led to the failure was well forecast and fell in two separate events, the first occurring over the 27 to the 29 July closely followed by a more severe event from the 30 July to 1 August 2019. The latter was a substantial and rare event, with an annual probability of occurrence of about 1%. The resulting flood was very much less, however, than the probable maximum flood which the spillway should have been capable of withstanding. The spillway had discharged significant flood flow in the past, particularly in 1998 and 2007, without any apparent consequences.

The event of the 31 July/1 August subjected the spillway to high velocity flow for a sustained period. As a result of poor design of the spillway, exacerbated by intermittent maintenance and further deterioration in the months leading up to the event, some of that high velocity flow was injected through the cracks and joints in the spillway chute into the fill below. This was in large quantities and caused the fill to liquify into a slurry which then flowed down beneath the slabs, lifting them at the lower end of the chute and discharging the slurry into the river downstream. This process eroded a large void beneath the slabs. During the morning of the 1 August the flow down the spillway started to recede. The first upper slab failure is believed to have occurred between 8.20 and 9.30 that morning. As the water drained from the void, chute slabs began to fall into the void. There was then a slump in the fill which extended the void, causing further slabs to fail and the left-hand wall of the chute to be undermined. At this point there was a risk that the puddle clay core of the dam might have become exposed. Had this been breached, the dam could have collapsed. This led to the placing of bags of aggregate in the void to stabilise the dam.

The failure of the spillway only became apparent on the morning of the 1 August when the first upper slab collapsed. CRT responded quickly to the emergency and opened the draw-off valves. There was nothing they could do at that stage to prevent the slump of the fill, further collapse of chute slabs or the undermining of the wall. Later in the day they increased the diversion of flow into the by-wash channel to reduce the amount entering the reservoir. The Panel have considered if earlier action by CRT might have avoided the failure. The discharge over the auxiliary spillway could have been reduced if the draw-off valves had been opened and the upstream flow diverted into the by-wash channel when the weather event was forecast. However, there would still have been a substantial flow down the auxiliary spillway, and it may still have failed.

Were the actions taken by CRT, and EA as the regulator, as a result of the November inspection, reasonable and proportionate?

The Inspecting Engineer accurately identified the key safety issues at the dam at the time of his inspection in 2018, as far as he was able from his inspection. He also reviewed the spillway drawings (which do not give a complete picture of the poor design). He was clearly concerned over the spillway, identifying the risk it posed to the safety of the reservoir as being both “significant” and “credible”. Because the Inspecting Engineer was concerned about the spillway, he required the Owner of the reservoir, CRT, to investigate its condition further and gave 18 months for this work to be completed. He also required the maintenance issues at the spillway to be addressed: “All existing joint sealants in the secondary overflow channel to be thoroughly cleaned, inspected and replaced if required to prevent spill water passing under the base slab and pressurising the underside. Further maintenance to be reviewed on a 2 yearly basis and undertaken if required.” The work to address this had not been started at the time of the incident. However, he did not require any precautionary measures, for example, the drawing down of the reservoir level, to be implemented whilst this work progressed. Nor did he explicitly state that the work was urgent.

CRT have indicated that they believed the reservoir to be in sound condition, but this appears to be inconsistent with their downgrading of the condition of the reservoir from C to D as a result of the 2018 inspection (Grade D is denoted “*Poor*” and described as “Structurally significant defects leading to loss of stability in the medium term, generally 5 to 10 years”). CRT have also stated that they planned to complete the MIOS within the periods specified by the Inspecting Engineer. Unfortunately, the event occurred before these works could be completed.

Are there any changes to maintenance and inspection guidance or legislation that need to be considered in the light of what happened at Toddbrook?

The intermittent maintenance of the spillway at Toddbrook reservoir is particularly concerning. This occurred despite repeated requests from Supervising Engineers. It seems that, apart from routine operational work (e.g. grass cutting and vegetation clearance), little work was done at the reservoir unless expressly required under statute by an Inspecting Engineer. Had full maintenance and repair to the spillway been completed prior to the event, as required by the IE, it is possible that the failure might have been avoided.

Prior to the 2018 inspection there had been four inspections of the reservoir since the construction of the auxiliary spillway. The various Inspecting Engineers had commented on the risks posed to the reservoir, but none prior to 2018 had specifically identified the risk of structural failure of the spillway. Apparently, none of these Inspecting Engineers had reviewed the drawings of the spillway design. In preparation for the 2018 inspection, the Supervising Engineer compiled comprehensive information on the reservoir and provided it to the Inspecting Engineer prior to the inspection (note that this was a different Supervising Engineer from those associated with earlier inspections). This included copies of the drawings of the spillway. These were not seen by the Inspecting Engineer at the time of the 2010 inspection. It is remarkable that, over the 50 years that the auxiliary spillway has existed, it is only the last Inspecting Engineer that questioned the design of the spillway.
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With the benefit of hindsight, a precautionary draw down of the reservoir might have been advisable, but this should be judged against the information the Inspecting Engineer had to hand at the time. Precautionary measures are left to the judgement of the individual Inspecting Engineer. Inspecting Engineers are likely to be reluctant to require precautionary measures if the threat to safety of the reservoir is not imminent. However, the experience at Toddbrook reveals that there may be a flaw in this approach. It would be wise to consider further if it would be prudent to provide more guidance to Inspecting Engineers on this important matter.

A key finding of this Review is that the elements in the process of inspection, supervision, and delivering remedial action are at times disconnected. There is a need for better communication at all levels, but particularly between the Inspecting Engineer, the Supervising Engineer, and the Owner following the inspection of a reservoir. The Owner should be left in no doubt as to what MIOS and maintenance measures are required, their urgency, and whether any precautionary measures are needed whilst they are completed.

During this Review, the Panel have considered if compliance with the legislation is sufficient to ensure the safety of reservoirs. CRT staff and the EA have stated that compliance is not the same as safety. That is, a reservoir and its Owner can be compliant with the legislation without the reservoir necessarily being safe. There is also some evidence to show that there are different views about what constitutes “safe”. Whilst it might not prove possible to completely align compliance with safety, there is certainly a case for exploring this further in part B of the Review.

Summary of recommendations

During the Review I have identified the potential for improvement in many areas of reservoir safety management. The recommendations apply to all those individuals and organisations involved in reservoir safety management. Some of the recommendations are for direct implementation whilst others recommend further work to better understand what changes might be needed. As much as possible of that further work should be completed as Part B of this Review.

Inspection of the Reservoir

I recommend that:

1. **The EA commissions new guidance on the failure mechanisms of spillways and how to undertake spillway inspections.** This should include guidance on spillway design based on international good practice and lessons learned from incidents in the UK.

2. **Inspecting Engineers and Supervising Engineers inspect spillways closely and by direct access during their visits, with a minimum of one year between Supervising Engineers’ spillway inspections.**

3. **The Owner[7] should make the necessary safety preparations in advance to enable such close inspections to take place as a matter of routine.**

4. **Owners[7], under the guidance of the Supervising Engineer, make available an appropriate and well-structured package of information on the reservoir to the Inspecting Engineer well in advance of a planned inspection.** The Inspecting Engineer should not overly rely on information in previous inspection reports nor should they have to search out relevant information from archives.

5. **Requirements for Measures in the Interests of Safety (MIOS) should be worded so that they unequivocally convey the seriousness of the risk and the urgency of implementation.**

6. **The EA commissions new guidance to assist Inspecting Engineers in specifying suitable precautionary measures to be in place during the period for completion of Measures in the Interests of Safety (MIOS).**

7. **A formal meeting between the Inspecting Engineer, the Supervising Engineer and the Owner[7] should take place immediately the Inspecting Engineer has determined the findings of the inspection and formulated any requirements for Measures in the Interests of Safety (MIOS) and statutory maintenance.** This meeting should take place within one month of the date of the inspection**.**

8. **Inspection reports should be submitted without delay, following this meeting.** Owners[7] should respond to drafts promptly.

Supervision of the Reservoir

I recommend that:

9. **The EA commissions guidance for Supervising Engineers on the production of their visit reports and annual statements so that progress on delivering MIOS and statutory maintenance and reasons for any delay are clearly set out, and the current condition of the reservoir is clearly communicated to the Owner[7]**

10. **Owners[7] manage the safety of their reservoirs in a way which gives Supervising Engineers reporting lines directly to the individual responsible for corporate safety at Director Level (or equivalent).** In the case of single owners[7], clubs or small companies, the Supervising Engineer should report directly to the owner or equivalent.

Legislation and regulations

I recommend that:

11. **There is a systematic review of how the current Reservoirs Act, and the associated Regulations and Guidance, are implemented.** This should consider the roles and responsibilities of qualified engineers, whether compliance with the Act is sufficient to ensure safety, and how safety is formally assured.

12. **The Secretary of State requires periodic review of the safety management process, including sampling of the inspection and enforcement process to ensure these remain fit for purpose.** This should include engagement with other sectors charged with managing the safety of critical infrastructure to better understand the processes that are used there

13. **The potential of an Inspecting Engineer issuing a Certificate of Safe to Operate be explored. This should include a review of practice in other safety critical infrastructure sectors. It should also consider liability implications and whether some form of qualifying statement may be needed to accompany the certificate.**

14. **The statutory maximum period between inspections is reviewed to determine if it is still appropriate in every case in the light of the ageing reservoir stock.**

Operation and maintenance

I recommend that:

15. **Owners[7] ensure that their organisational structure provides a clear path of responsibility for routine and required maintenance.** For single owners[7], clubs and small companies the owner or equivalent should be directly responsible for maintenance

16. **The EA makes Owners[7] more aware of the vulnerability of spillways to poor maintenance and repair. They should remind Owners[7] of their responsibility for the safety of their reservoir(s) and of the need for regular maintenance and repair of spillways at appropriate intervals, without waiting for Supervising Engineers or Inspecting Engineers to draw this to their attention.**

17. **Owners[7] complete any outstanding maintenance of spillways urgently and that they respond promptly to the recommendations of Supervising Engineers. Inspecting Engineers should make full use of the provision for statutory maintenance when setting out their requirements following an inspection.**

18. **The Secretary of State gives powers to the EA to enforce by means of a notice outstanding statutory maintenance and the powers to carry out such works and recharge.** This may require a change in legislation.

19. **Owners[7] regularly maintain all existing long-term monitoring equipment on reservoirs, so as to keep the equipment serviceable, and that they take and record measurements at appropriate intervals.** All measurement data should be retained in a usable format in the Prescribed Form of Record for ongoing use by Supervising Engineers and for use by Inspecting Engineers at their inspections**.**

Immediate actions for other Reservoirs

I recommend that:

20. **The EA should urgently seek to identify any reservoirs with potentially similar spillway characteristics to those at Toddbrook.**

21. **Where these are identified, Owners[7] urgently arrange for detailed inspections to be undertaken, with any necessary precautionary measures put in place to ensure they remain safe whilst any remedial work is undertaken (**note that at the time of writing this action had already been progressed).

22. The EA urgently produces a technical note on the Toddbrook Reservoir incident to inform all reservoir engineers and owners of the lessons learnt from the incident.

Appendices

Appendix A. Glossary of terms, abbreviations and acronyms

Adit: A horizontal passage either leading into a mine for access, extraction or drainage

ALARP: As low as reasonably practicable

Alkaline Matrix: Describes the alkaline environment in concrete which is important in preventing any corrosion of steel reinforcement in the concrete

Basin: Area of land in which the reservoir is formed

BRE: Building Research Establishment

BWB/BW: British Waterways Board/British Waterways, refer to the two previous owners of the Reservoir prior to CRT taking ownership

Breach: A break in the wall or embankment of a dam

By-wash: A channel that allows water to by-pass (the reservoir)

Catchment: The area of land draining into a reservoir

Category (of Reservoir): Refers to the consequence of failure of a reservoir on the communities downstream

Chute: The part of a spillway which carries water away from the headworks and transmits it downstream

Civil Contingencies Act: The 2004 Act defines responsibilities for dealing with emergencies

Compensation Channel: A by-wash channel designed to maintain a minimum downstream flow in a river after abstraction for some use

Construction Engineer: A Qualified Engineer responsible for supervising construction works under the provisions of the Reservoirs Act

Core: The central part of an earth embankment dam that provides water tightness

Crack Injection: The process where water is injected through joints and cracks in a spillway under pressure, as a result of stagnation pressures

Crest: The top of a spillway or embankment.

CRT: Canal & River Trust. CRT is the owner of Toddbrook Reservoir

Cut-off: A feature in an embankment that cuts off the path for water to flow within the embankment. It is often used to provide a water-tight seal at the base of the core of the dam or between the crest of a spillway and the core

Design Flood: The flood that a spillway is required to convey by design. For Category A reservoirs, the design flood is the 1 in 10000year flood. See also Safety Check Flood.

Draw Down: The controlled reduction of water level in a reservoir

Draw Down Index: The ratio of the draw down per day as a percentage of the reservoir height

Draw Off: The draining of water from a reservoir

Dowel Bars: Steel bars that join one slab to the adjacent one, or other part of the structure. They help to prevent movement of one slab from the next

EA: Environment Agency. The EA is the Reservoir Regulator for England

Enforcement Authority: See Regulator

Factor of Safety: The ratio of the load at which failure would occur with the actual applied load

Filter: Material that prevents the passage of particles being transmitted in suspension by a fluid

Fines: The smaller particles within a body of earth or graded material

Flip Bucket: See Ski Jump

Flood Plan: A plan drawn up by the undertaker of a reservoir to specify the measures to be taken in the event of an emergency. Also known as an on-site flood plan

Floods and Water Management Act: The 2010 Act was the result of the 2007 floods and the Pitt Review. Provisions of the Act amended some of the provisions of the Reservoirs Act 1975

Fluidization: The change in a substance from the solid state to the liquid state so that it flows

Frequency: The average rate of occurrence of a particular event at a particular location. Usually expressed as an annual probability. It is the inverse of return period

Grout: A cement-based slurry injected into the ground to fill voids and stabilise it, and/or to provide a barrier to water (see cut-off)

Head: The pressure exerted by a body of water (or other liquid). Conventionally measured by the vertical distance to the water surface

Heave: The uplift force on a body or foundation, or movement, caused by the swelling of the ground

Honeycombing: Where concrete is stony and contains voids due to the absence of cement paste and other fine material

Hydraulic Fracture: The fracturing of a soil, typically a clay, due to local water pressures exceeding the internal stresses present within the clay (for example due to self-weight) at that location

Hydraulic Gradient: The variation of head of water from one point to another (e.g. the fall of pore water pressure within permeable material).

Hydrodynamic: Describing forces generated by or resulting from a flowing fluid (often water)

ICOLD: International Commission on Large Dams

Impounding: Reservoirs that receive their inflow from a river or watercourse

Inclusion: Material that is trapped inside another material during its formation

Injection: See crack injection

Inspecting Engineer: A Qualified Engineer responsible for inspecting reservoirs under the provisions of the Reservoirs Act

Kinetic Energy: The energy in a flow due to its velocity

Lateral: Across the direction of flow. See also transverse

Left Hand: Conventionally spillways are viewed from the crest looking downstream (in the direction of flow)

Longitudinal: Along the direction of flow

MIOS: Measures in the Interests of Safety. They are specified by an Inspecting Engineer and are enforceable under the provisions of the Reservoirs Act

OD: Ordnance Datum. The national datum to which survey levels conventionally relate.

Organogram: A diagram depicting the way a corporate group of people are organised

Overflow: A structure built to allow a body of water to overflow

Owner: The person who owns a dam. In the context of this report the term “owner” is used to denote the undertaker

Permeable: Ability for water to flow within the substance or body

Permeability: The degree to which a substance or body is permeable

Petrographic: A tool to study the mineralogical and chemical composition of materials

Phreatic: The hydraulic gradient of groundwater (in the context of reservoirs this refers to water within the permeable parts of an embankment dam)

Piezometer: An instrument for measuring the pore water pressure within a soil

Piping: The development of a water passage (or pipe) within a soil through which water then flows

Plug Valve: A valve that shuts off the flow by movement of a cone shaped plug into a socket

Plums: Large pieces of rock embedded in concrete. At Toddbrook these took the form of rock upstands on the surface of the auxiliary spillway chute

PMF: Probable Maximum Flood. Spillways on Category A reservoirs under the provisions of the Reservoirs Act must be able to pass the PMF with no more than minor damage. See Safety Check Flood.

Pore Pressure: The pressure of groundwater within a soil. Often referred to as pore water pressure

Probability: The likelihood of occurrence. Can be expressed as a frequency or return period

Puddle Clay: A water-proof material formed by working clay with water

Qualified Engineer: An engineer qualified under the provisions of the Reservoirs Act

Regulator: The Government body responsible for ensuring that the provisions of the Reservoirs Act are complied with. The Regulator enforces the provisions of the Act and is therefore often referred to as the Enforcement Authority

Reservoirs Act: The 1975 Act forms the basis for regulation of reservoirs in England. It was amended by the Floods and Water Management Act 2010

Reservoir Panels: Registers of Engineers qualified under the provisions of the Reservoirs Act. These are compiled and maintained by the Regulator

Return Period: The average period over which an amount is equalled or exceeded. Usually applied to rainfall and expressed as 1 in xx years. It is the inverse of frequency

Risk: The combination of probability and consequence. Note that the term is often misused, either as a substitute for probability or a substitute for consequence

Right Hand: See Left Hand

Roller Bucket: A form of energy dissipator that rolls over the flow at the lower end of the spillway into a lateral collection channel

Safety Check Flood: The flood which the spillway is designed to pass without the dam failing. For Category A reservoirs, this is the Probable Maximum Flood, which in the UK, has a magnitude of approximately twice that of the 1 in 10000year flood

Sill: The slab forming the crest of a weir or spillway

Scour: Erosion occurring due to the flow of a fluid over an erodible material

Shear: The sliding of one surface of a material over another

Shoulder: The fill that supports the watertight core of an earth embankment dam

Ski Jump: A form of energy dissipator formed by un upward curve at the lower end of the spillway which projects the flow over into a plunge pool. Sometimes referred to as a Flip Bucket

Slip: Instability failure of soil due to insufficient shear strength

Slurry: A dilute mix of solid particles and water

Stagnation Pressure: The pressure created when a flowing fluid is stopped by collision with a solid surface

Standpipe: An open-ended vertical pipe

Statutory Maintenance: Maintenance specified by a Qualified Engineer that is enforceable under the provisions of the Reservoirs Act

Suffusion: A type of internal erosion where fines are transported by seepage flow from one location to another

Supervising Engineer: A Qualified Engineer appointed to supervise a reservoir under the provisions of the Reservoirs Act

Take: The amount of grout used for a particular purpose during a grouting operation

Till: Usually referred to as Glacial Till. A stiff clay formed from material eroded by a glacier

Transverse: Perpendicular to the direction of flow. See also lateral

Toe: Where the slope of the embankment surface meets the original ground

TWL: Top Water Level. For a reservoir this is usually taken as the level of the primary spillway weir crest. The level in the reservoir will exceed TWL when the spillway(s) operate

Under Drain: A drain formed under a spillway, embankment or other structure to drain away any seepage water

Undertaker: The organisation or individual legally responsible for the operation of a dam.

Uncarbonated: Concrete carbonation is the reaction of calcium hydroxide in concrete with carbon dioxide in the environment. It is accelerated by damp conditions. If concrete is uncarbonated this process is not present and means the concrete is of good condition

USBR: United States Bureau of Reclamation

V-notch: A vee shaped notch cut into a plate and set in the flow. It enables the rate of flow to be deduced from the water level

Water-Bar: A flexible material cast into adjacent slabs of concrete to prevent water penetrating through the joint between the slabs. Sometimes referred to as a Water Stop

Wave Wall: A wall built along the crest of a dam to prevent waves overtopping the dam

Weir: The top edge of an overflow or spillway over which the water flows. See also sill

Yellow weather warning: Severe weather is possible over the next few days and could affect people and infrastructure in the area concerned. Can be issued in conjunction with Flood warnings

Appendix B. Analysis of the failure mechanism of the spillway

This appendix gives further details of the analysis of the failure mechanism of the spillway, to supplement the description in the main body of the report. It is written for those with some knowledge of reservoir engineering.

Background

Records indicate that the reservoir remained at or above top water level (TWL) level for several months each year during the winter (fig B1). The top water level (TWL) of the reservoir is the level of the primary weir crest, which is +185.67 mOD. The crest of the 76.2m long auxiliary weir was supposedly set 260mm above that level, but it has recently been established that it is now only about 180mm above TWL. In fact, the difference is 164mm on the left side and 184mm on the right side, implying a slight, 20mm dip towards the left. Survey pins sited on the downstream ends of the crest slabs indicate a level drop of approximately 75mm towards the left side of the 76.2m long weir crest. The weir is likely sited over the central clay core of the embankment whereas the slabs featuring the survey pins are sited over the downstream fill.

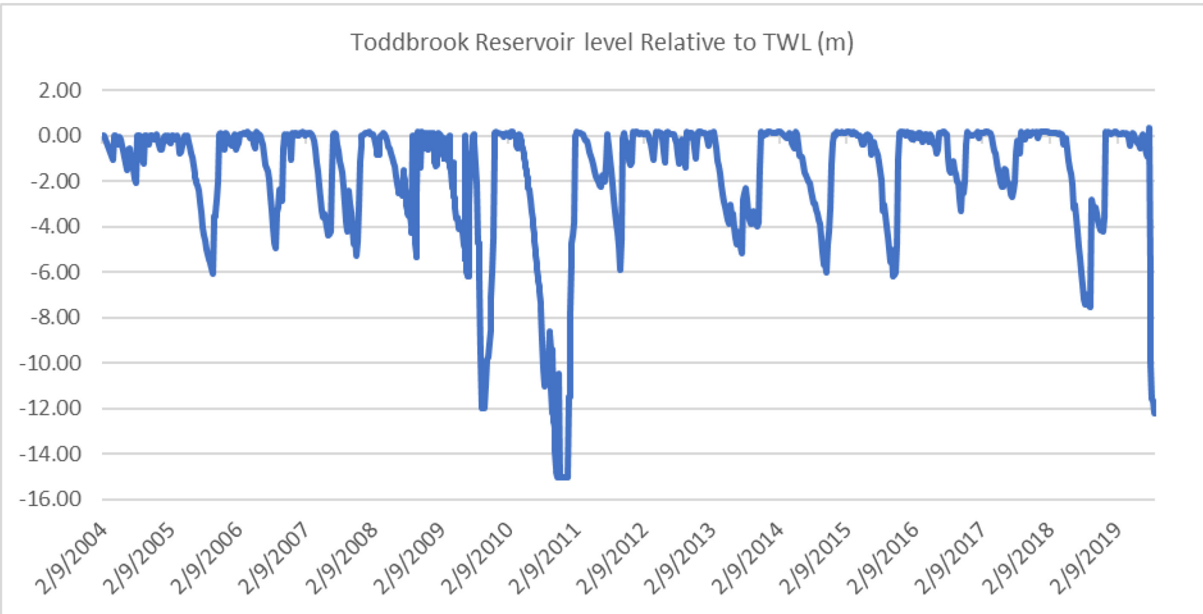


Figure. B1 Reservoir Levels in relation to Top Water Level from 2004 to 2019

The auxiliary weir discharges to a spillway chute comprising concrete slabs, sited on the downstream face of the dam. There are spills recorded on the auxiliary chute in 1973, 1983, 1998, 2007 and of course more recently on 31 July 2019. It was this last event which led to the partial failure of the chute. The events leading to the partial chute failure were two separate events, the first occurring over 27 to 29 July followed closely by a more severe event from the 30 July to 1 August 2019. Sometime after midnight on Tuesday 30 July (i.e. in the early hours of 31 July) reservoir levels rose and flows commenced over the auxiliary spillway. The flows continued throughout 31 July attracting sightseers and even media attention. During that time video footage shows bursts of silt laden / muddy water within the outflows leaving the downstream end of the chute.

On the morning of 1 August some residual flow continued over the spillway chute. At 08.30 a member of the public reported seeing silty water coming from lower joints in the spillway (assumed to be from the sides of lifted panels rather than through joints as such). At 09.45 a hole is reported in one of the upper parts of the chute (fig B2).

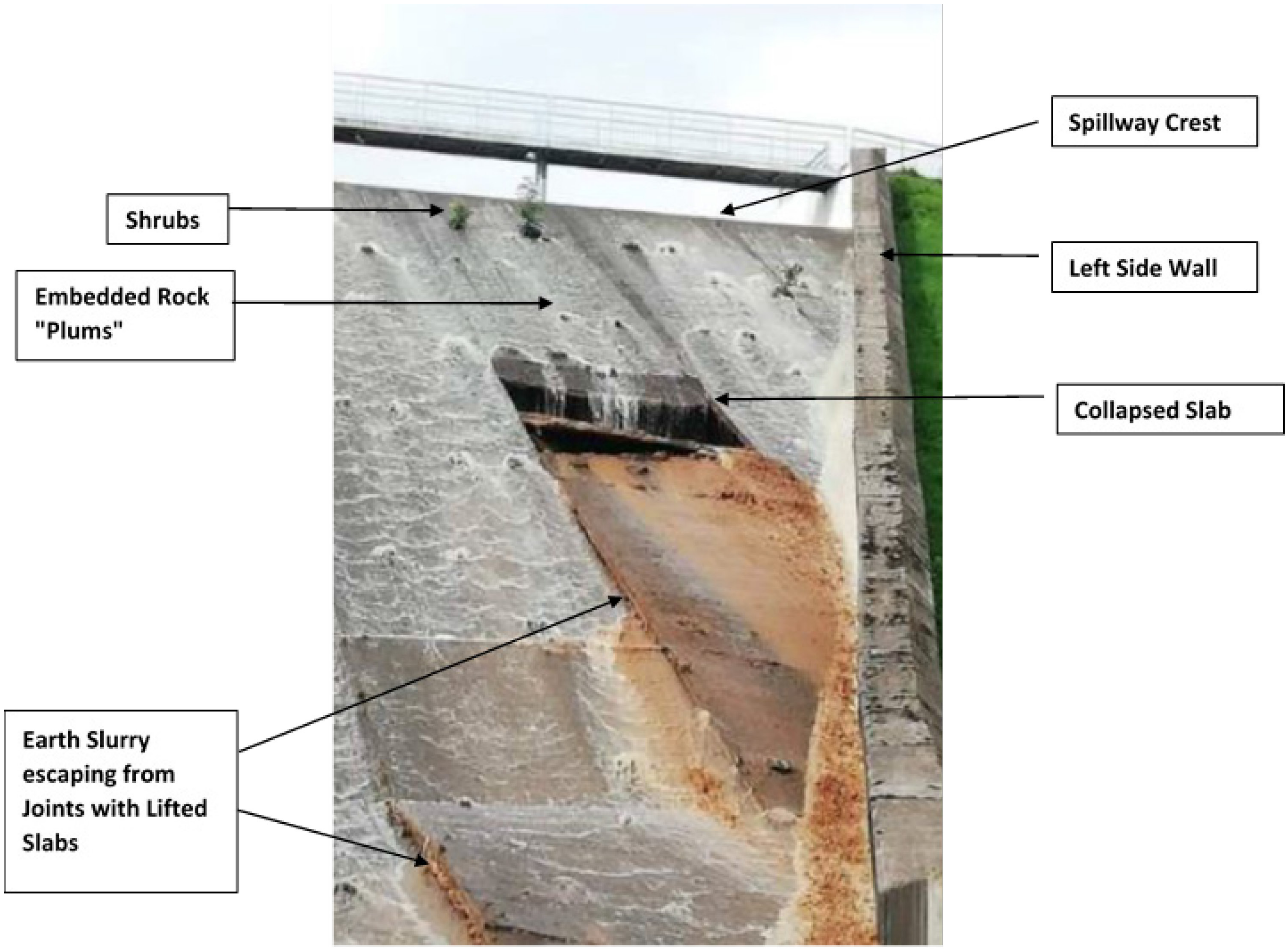


Figure. B2 The initial slab collapse

Basic structural design and construction

Design. The design of the spillway was carried out in the late 1960s in-house, by the British Waterways Board (BWB) but, it is understood, with guidance and comment by an engineer from of Sir William Halcrow & Partners (Halcrow). Indeed, following the construction of the works, the same engineer inspected Toddbrook on 28th October 1970 under the Reservoirs Act (1930). Section 6(b) of that report states that: -

> "*The new emergency spillway has been constructed in accordance with the drawings prepared by the Owner and approved by me.*
> *The overflow weir and emergency weir are adequate to deal with the severest flood likely to be experienced*".

However, by modern standards the design was insubstantial in many areas including;

- Inadequate chute slab thickness,
- Inadequate embedded reinforcement,
- Poor jointing and sealing details between slabs, especially laterally,
- A lack of under-drainage,
- No cut-off between the spillway crest and the underlying core of the embankment.

The comparison between the spillway at Toddbrook and Carr Mill dams is both interesting and relevant as both are owned by CRT and are only 40 miles apart. However, while Toddbrook was designed by the BWB with advice from Halcrow, the much more substantial design at Carr Mill was done only 5 years later by Halcrow, on behalf of the BWB. With the same Owner and Engineer involved in both dams only 5 years apart it is unclear why the BWB did not query the need for the more substantial design at Carr Mills, or if they accepted the design at Carr Mills, why they did not then question whether the works at Toddbrook were still adequate.

Interestingly, practice overseas seemed to have also changed about this time. The United States Bureau of Reclamation (USBR) publish extensive guidelines for the design of dams and associated works. This includes broad recommendations for the design and detailing of spillway chutes on soil foundations. Their guidance for such chutes changed noticeably between the 1960 edition of their book, "*Design of Small Dams*" and the later 1973 edition. For example, the 1960 edition indicates only one layer of (upper) steel reinforcement in spillway chute slabs, whereas the 1973 edition shows both top and bottom steel. However, whereas both editions recommend reinforcement in the tops of slabs as an "anti-crack" measure, what little steel wire reinforcement was included in the slabs at Toddbrook was placed in the bottom of the slabs.

It is likely that any structural design for the auxiliary spillway would have been in accordance with code of practice, CP 114 which was the dominant design code in the UK at the time for concrete structures. This was essentially a building design code and so would have carried with it an implicit assumed design life of 60 years. However, the auxiliary spillway at Toddbrook is deficient by even that standard. For example, the concrete cover to the embedded reinforcement in the lower face of the slabs at Toddbrook was recorded on drawings to be ½” (12.5mm) whereas Clause 307 of CP114 requires that where such reinforcement in slabs is in contact with earth, the cover should be 1½” (37.5mm). Indeed, it is not clear what function the embedded wire mesh in the Toddbrook chute slabs was expected to fulfil. The ⅛” (3mm) diameter bars at 150mm centres would not be considered as designed reinforcement by standards in 1969/70. In particular there was no “anti-crack” steel provided in the upper surface of the slabs. Modern requirements would be for designed reinforcement in both upper and lower surfaces of such slabs.

The chute concrete at the Toddbrook Auxiliary spillway is shown on record drawings to have been a 1:2:4 mix. This was a standard “design” mix at the time with 1:2:4 referring to the volumetric proportions of cement, sand and gravel. The nominal 28 days strength of such a mix would be assumed to be 3000 psi (approx. 20.5 MPa). This would be considered low by current standards for spillways, but in fact the Toddbrook concrete would have continued to gain strength well beyond 28 days, possibly reaching 30 MPa or more, eventually. Typical chute details are shown on figure B3 which is taken from a record drawing of the auxiliary chute. The drawing did not give any details for the longitudinal chute joints. It would also seem that a joint shown located on the downstream side of the weir upstand was relocated upstream of the weir on site. Such a joint location would allow water ponded upstream of the crest upstand to penetrate the slab if no water bar was incorporated in it.

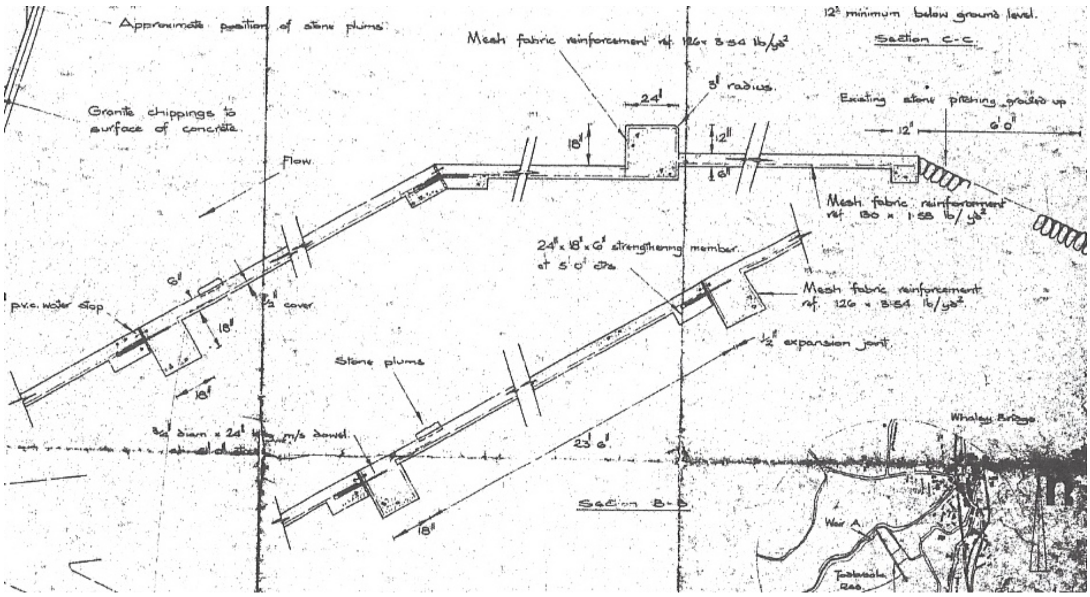


Figure. B3 Typical details of the auxiliary chute taken from a record drawing

Construction. Details of spillway construction are given in a paper provided by an engineer from Mowlem, CRT229 "*Report on the construction of a new spillway for British Waterways Board*" 28/11/69. The paper notes that the crest of the embankment was locally taken down by 6' 6" (1.98m) to form the foundations for the upper spillway crest. It further notes that topsoil over the area of the auxiliary chute was removed by hand as it was impossible to use machines on the steep 1(v) on 2 (h) slope. After the topsoil had been removed, the spillway strips were excavated by hand and side shutters (formwork) were fixed in position. Split shutters were used where water bars passed into adjacent slabs. Building paper (rather than a concrete screed that would be used nowadays) was laid on the formation (ground) prior to placing the mesh and concrete. Concrete was poured in from the top of each 25 ft long (7.62m) by 12'6" (3.81m) wide bay and levelled by hand. Two passes of a vibrator achieved a satisfactory finish. It is not known if any ground (formation) compaction occurred prior to the slab concrete being placed but it is thought to be unlikely. No mention of such compaction is made in the detailed 1969 "Report on Construction" by Mowlem, the Contractor. In fact, it highlights the difficulties of working on such a steep slope and how early plans to use plant and equipment had to be abandoned in favour of working by hand.

Lateral joints were sealed with dumbbell water bars, and incorporated ½" (12.5mm) jointing material sealed on the upper surface with a plastic type sealant, approx. 20mm deep, poured into the joint. The paper notes that 3 bays were achieved each day although placing concrete, and working, on a 1(v) to 2 (h) slope was problematic. Rain produced further difficulties. Nevertheless, the contract was finished in just 9 months.

Ageing and maintenance

Ageing. All works have a design life. BS EN 1990, Eurocode "*Basis of structural design*", (Eurocode 0), gives indicative design working lives for design purposes for various types of structures, as follows:

Category 1 – Temporary structures, not including structures or parts of structures that can be dismantled with a view to being re-used – 10 years
Category 2 – Replaceable structural parts, e.g. gantry girders, bearings – 10 to 25 years
Category 3 – Agricultural and similar buildings – 15 to 30 years
Category 4 – Building structures and other common structures – 50 years
Category 5 – Monumental building structures, bridges etc – 100 years

Prior to the introduction of the Eurocode, a design life of 60 years was required for buildings, though this period was never stated explicitly in any of the structural design codes. Although dams are classed as monumental structures, as noted in the preceding section, it is likely that any design for the auxiliary spillway at Toddbrook would have been in accordance with CP 114. The auxiliary spillway at Toddbrook is now 50 years old. If indeed the works were designed based on CP 114, it could arguably be said that they were, anyway, now approaching the end of any "design" life which could have been expected.

Maintenance. A typical maintenance regime for any spillway would comprise regular inspections, ensuring that the works were free of weeds and associated roots and that any joint sealant was intact and maintained as necessary. It would also be monitored for general line and level, concrete cracking and any other forms of distress such as surface degradation. The implications of cracking or other forms of distress would be assessed and remedial action taken as and when required.

Photographic evidence suggests that for the first 20 years of the chute's life, up until the early 1990s, the works were maintained reasonably well as there is no evidence of historic weed growth on the crest or chute at that time. Cracks started to appear and develop during the 1990s. By 1999 weed growth was evident on the crest and by December 2006 photographs show extensive weeds on the chute. By the following year some of these weeds had developed into fully grown shrubs (fig B4). The locations were generally focused on those joints not sealed with water bars, such as the longitudinal joints on the chute. The fact that the vegetation appears quite significant and lush suggests that the roots were able to reach through the chute concrete into the embankment soil beneath and also to find adequate moisture.

Figure. B4 Well established vegetation on the crest slabs and on the chute in 2007

Interestingly one photograph dated September 2007 shows lines of extensive weed growth laterally across the chute immediately above the slab which initially failed (fig. B4). One of these vegetation lines may be located in a joint between slab panels but at least one other does not line up with the chute joints and so was certainly in a significant crack. The fact that a lateral joint was significantly affected would also suggest that the embedded water-bar in that joint was not working as intended. Although the vegetation generally followed joint lines there were also isolated instances where vegetation had taken hold though apparently intact slabs, both on the crest and on the chute. Extensive joint surface sealing was carried out in 2008, but in the absence of embedded water-bars such surface sealing does not guarantee water tightness. It is largely done to protect any jointing material, such as the ½” Flexcell used at Toddbrook, from degradation. It is also unclear whether cracks through slab panels were sealed in the same way as joints. From the chute inspection carried out by the Review Panel other chute cracks show no sign of having been sealed.

By February 2013 weeds started to re-establish on the chute and by 2015 photographs show a sapling well established through a drainage hole. Vegetation continued to develop through 2016, 2017 and 2018. In one image dated 25 June 2016 vegetation lines along joints and cracks immediately above the bay which initially failed are again strongly in evidence. These seem to comprise the same joints and cracks already referred to as appearing in a September 2007 photograph, plus additional ones. At one point in 2019 there is some evidence of vegetation removal in upper areas of the chute, possibly by the use of weed killer, however elsewhere on the chute, established saplings remained in evidence. Despite the possible removal of weeds in early 2019, the photograph on 1 August 2019 showing the initial slab failure, also shows two well established shrubs in the chute bays immediately above the area which failed and apparently coinciding with some embedded impact rocks (plums) on the chute. These were clearly sufficiently established to have withstood all the significant chute flows of the previous 24 hours. This again suggests well established stem and root growth through the chute and into the embankment soil beneath.

The regime of maintenance at CRT in recent years for works such as Toddbrook seems to have been for maintenance requests to be raised which are then delegated to the delivery arm of CRT. They comprise an in-house team which can work directly in some defined areas while other routine aspects, such as grass cutting, have sometimes been subcontracted. It is also understood that, as Toddbrook is not a drinking water supply reservoir, weed control on the chute is generally by sprayed weed-killer. More substantial weeds may require manual removal. There is evidence of one established sapling growing through a chute drainage hole, but which was then “removed” by being severed at its base, still leaving the drainage hole blocked. It is unclear whether those carrying out maintenance receive any engineering direction during the work, in terms of how their work may affect chute performance. The evidence suggests that this is not generally the case.

Chute inspection

During their visit to Toddbrook, two members of the Panel inspected the chute by rope access and mapped it in terms of condition and features. Where the slabs remained intact, most of the joints were also intact, although there were some noticeable joint spalls in places. Most of the rock “plums” were also still embedded in the chute concrete. However, a number were missing. In some places the sockets left by these had been filled with concrete, but many remained open. Those which were open featured exposed aggregate indicating that turbulent flow had removed the surface fines. In some cases, rusting steel mesh could be seen at the bases of the holes. In one hole the thin layer of concrete remaining at the base of the socket had been partially lost and was exposing embankment fill beneath. The areas around the protruding rock “plums” also tended to feature exposed aggregate, indicating that the flow turbulence caused by the rocks had removed the cement paste and concrete fines from the chute surface in their immediate vicinity.

Where concrete cores had been taken, the concrete generally appeared to be hard and durable internally, although strength testing and petrographic analyses would have been useful to clarify this further. The coring also revealed the slabs to be of variable thickness, but generally not less than the intended “design” thickness of 150mm. Where slabs had been raised by the incident, visible edges indicated the slabs often to be of variable thickness. Edge concrete also seemed to reflect a high degree of honeycombing and voids. This suggested a lack of durability in these areas and that the edges of the slabs and hence the concrete at joints, had not been compacted initially to the same standard as perhaps other parts of the slabs.

In a number of areas, the concrete surface had degraded, although these areas were not extensive. The chute slabs were, in the main, also un-cracked although where cracking had occurred it was noticeable and generally lateral across panels. These areas were mapped and recorded during the inspection together with conditions such as surface degradation, remains of significant plant roots still embedded in the chute and any missing rock plums. The number and size of embedded plant root remaining in the chute was surprising. Where encountered they had generally been cut near-flush with the concrete surface but still remained embedded, presumably with root systems into the embankment soil beneath.

During the inspection, a number of extracted cores were tested using phenolphthalein as an indicator of possible alkalinity loss. The results from cores which had been extracted and exposed for several weeks were inconclusive. However, tests on “fresh” cores which had been extracted on the same day as subsequent testing, universally showed deep purple staining of the matrix throughout. This indicated that the concrete remained un-carbonated and retained an alkaline matrix, even after 50 years. The general impression gained from the weed growth, areas of spalling, missing rock plums, areas of surface degradation and unsealed cracks, was that the chute was due for some remedial attention, especially given its somewhat insubstantial nature.

Crest seepages and other factors

The Auxiliary Spillway, and in particular the crest, was prone to leakage and water ingress. By 2008 it was also apparent from the waterline at higher reservoir levels that the left (north-west) side of the spillway sat at a lower level than the right side. Survey records from levelling pins indicate that the difference is approximately 75mm and has been relatively stable at that value, certainly since 2012. However, recent survey checks along the auxiliary weir crest, which is sited over the embankment core, indicate a level difference of only 20mm. The auxiliary weir crest was originally supposed to have been set 260mm above the elevation of the primary weir crest. In fact, the difference is now 164mm on the left side and 184mm on the right side, implying a drop, or settlement, since 1970, of 96mm on the left side and 76mm on the right side assuming that the auxiliary spillway was constructed as designed. The foundations of the auxiliary weir crest are approximately 450mm below its crest which means that whenever the reservoir reached top water level, the whole auxiliary spillway upstream crest slab and its foundations would have been underwater.

The auxiliary spillway crest slab also features a number of open cracks and other permeable features;

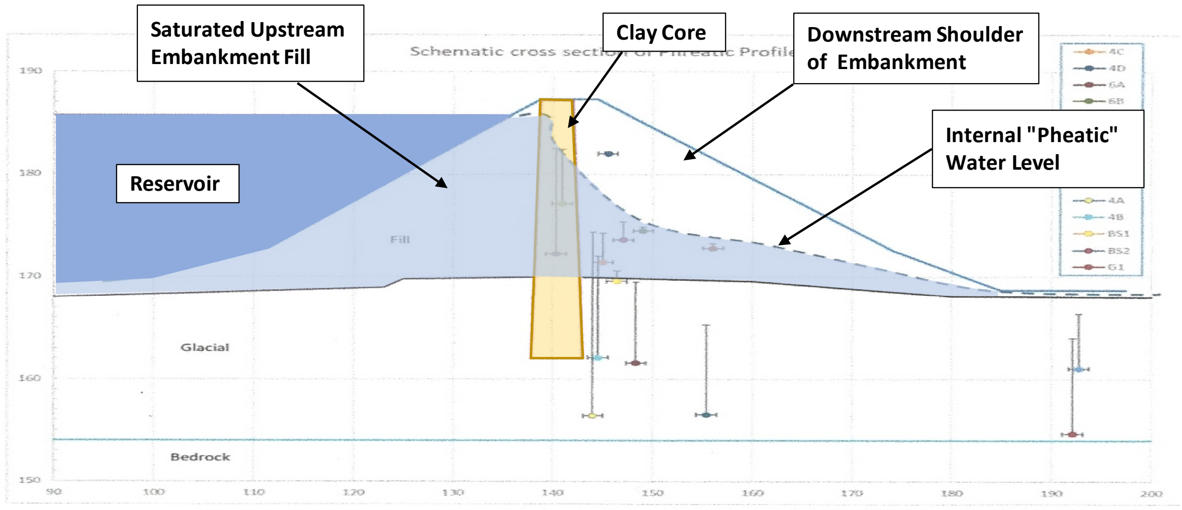
- there would appear to be a joint between the upstream crest slab and the weir crest. This is not shown on drawings but is clear from the amount of vegetation visible in that location on past photographs and from talking to CRT staff. It is not known whether it contains a water-bar but it may have been sealed from time to time with a surface sealant.
- a significant crack has developed along the whole length of the downstream crest slab. Photographs indicate that it was particularly developed by 2006 but there are indications of its presence earlier than that (fig B5). It “zigzags” around the foundations for the walkway bridge and seems to reflect differential settlements across the slab. It may have been surface sealed in 2008 but would likely have continued to move and open after that. During overspills this crack would allow flows into the fill, immediately downstream of the core.
- specific cracks have also occurred in the downstream crest slab. One such crack can be seen on a Dec 2006 photograph (fig B5) but was sealed in 2008. It was to the left-hand side of the crest.
- the longitudinal joints of the auxiliary chute do not feature water bars. Based on observations of historic vegetation growth, it is also likely that they are not present in the continuation of these joints over the crest slab.

All these features would allow water to ingress at times of chute flow, and some indeed even at times when the water was high but not actually at auxiliary weir crest level. There is evidence of this occurring at a number of places on the crest, including at both left and right ends and the centre, but more prominently on the left (fig B5).

Figure. B5 Historic crest cracking and isolated seepages on the left side on the chute

It is also apparent from the drawings and from subsequent investigations, that there is no cut-off wall below the auxiliary weir crest slab, sealing it into the embankment core beneath. The potential permeability of the foundation zone immediately below the 300mm crest slab was investigated by the Review Panel using techniques specifically developed for embankment dams, by the United States Bureau of Reclamation (USBR) and others. This predicted that a zone approximately 600 mm deep below the crest slab could be subject to permeability and cracking through periodic drying shrinkage and also freeze-thawing. Indeed, it is understood that recent site investigations under the crest slab have indicated such a zone where the material is "harder" and more brittle than for the main embankment core material beneath. It is then entirely likely that at times of high-water level, seepage flows would have permeated under the crest slab on a regular basis. There are indications of such localised seepages on the left side of the spillway as long ago as January 1991, although such events also seem sporadic rather than continuous.

It is clear then that there were routes for water to pass though and/or under the auxiliary weir crest slab. However, it is not necessarily the case that these would have continued under the downstream chute slabs, nor that such crest seepages would have led to any significant migration of material. The downstream shoulder material is essentially permeable and well graded, albeit with zones of variability. It would not therefore be prone to internal fines migration through suffusion (the ability of fine material to pass into and through voids in a coarse matrix, such as gravel). It is also likely that gravity would take any seepages coming beneath or through the auxiliary spillway crest downwards through the fill material rather than them continuing under the chute slabs. Figure B6 shows the phreatic water surface within the embankment recreated in the recent Mott MacDonald study based on measured piezometer levels. It illustrates the very low phreatic (pore water pressure) levels in the downstream shoulder, indicating a free draining material.



Source: MML

Figure. B6 Measured phreatic water levels within the embankment

With regard to material loss though any such seepages, although all recent years show sustained reservoir levels at or above TWL for many months each winter, there was no evidence of significant fine material losses at downstream seepage outlets. Seepages in lower parts of the chute have been described as either clear or "ocherous", with the later describing staining by iron oxides, probably from groundwater, rather than material erosion. In addition, when the incident occurred and the downstream fill material was locally lost, the core and that part of the immediate downstream shoulder, remained intact, albeit featuring occasional and localised "bursts" of water and fill material. The occasional bursts of water and fill material have been the subject of some debate by the Panel. One explanation is that these bursts represented the relief of locked-in pressures within the core, once much of the downstream restraint had been lost. Another explanation is that the bursts represented local piping which periodically re-sealed as material upstream was drawn into the seepage paths. Certainly, once the upstream reservoir dropped below the level of the auxiliary spillway slab, these bursts of water and soil effectively stopped, which tend to favour the second explanation.

One argument in support of some downstream migration of water through the fill was provided late in the review by satellite data monitoring historic ground levels at Toddbrook, This appears to indicate abrupt drops to the downstream chute in two locations, one in the area of initial upper slab failure and another at the centre of the chute. Dates of July 2018 and February 2019 are given for these events, but it is not clear which date refers to which location. This can be considered in association with the results of a Ground Penetrating Radar (GPR) survey which was carried out over the remainder of the chute, after the failure. The GPR survey indicated the third row of panels down to be generally quite thin and also to feature more localised voids than, for example, the fourth row immediately beneath them, which showed no voids. Intermittent exposure to water, either from seepages or through the unsealed longitudinal joints, may well have led to localised erosion and/or to pockets of differential settlement. The GPR survey certainly indicates such pockets, but not “paths” as such. Any differential settlement which occurred in the area of the initial upper slab failure could have further opened the pre-existing cracks in that location.

In summary, the auxiliary chute crest was almost certainly permeable and the upper left part of the crest perhaps especially so. At times of high-water level there could have been regular seepages through or under the slab. The Panel considers that these would generally have dissipated downwards by gravity into the downstream shoulder without excessive removal of fines (see Fig. B6). However, it is also acknowledged that some downstream migration of seepage could have occurred leading to either localised erosion or differential settlement of the spillway chute in some locations. Moreover, the conditions for such crest seepages would have been present for prolonged periods in all recent years when the reservoir level was at TWL or higher, for many months. The Panel concludes, however, that while such local fill saturation caused by the permeability of the auxiliary crest and its foundations may have exacerbated the events which occurred in 2019, they would not of themselves have caused the failure.

Hydro-fracture of the embankment core was also considered by the Review Panel. However, this would have been a more deep-seated event. Also, whilst the reservoir was briefly 200 to 300mm higher than in previous years, this was a relatively small rise over and above reservoir levels that have been reached consistently in previous years. It would also not explain why the initial slab failure occurred where it did. In short, while there can always be speculation about such a mechanism there is no evidence for it.

One other mechanism discussed and reviewed by the Panel was whether the void at the upper part of the slope and apparent “heave” at the base could have represented a deep-seated slip or slide of the embankment. However, it was noted that when aggregate bags were later being dropped on to upper zones of the embankment, the downstream areas were being monitored by precise laser survey equipment. This revealed no movements whatever to the lower zones thus indicating that there was no underlying slip or other rotational failure, taking place. Furthermore, photographic evidence shows the scoured material to have essentially liquified rather than remained solid.

Basic hydraulic design

The basic hydraulic design of the upper crest and downstream chute at Toddbrook is fairly conventional. From the crest the water cascades down the chute following the steepest path. This is the slope defined by the main embankment on which it is founded. The design unit discharge of the chute is low at approximately 1 $m^3/s/m$. Even if there needs to be some flow re-distribution, given the uncertainties over the capacity of the outlet channel from the main weir, the auxiliary chute unit flows would be still be unlikely to exceed 2 $m^3/s/m$, which is also low by spillway standards. One anomalous design feature is the way in which the left side wall of the chute cuts obliquely across the line of flow. It is not clear whether this was a mistake or simply intended to avoid some underground workings. Either way it is an unfortunate hydraulic feature which tends to collect and focus flows along its base. Such flows will have enhanced depths and turbulence. This has resulted in the wall being increased in height since the spillway was first commissioned, although it is unclear whether this was hydraulically assessed nor whether it would be capable of containing the PMF or even the 1 in 10000year event.

Although the left hand wall tends to increase the local flow depths and turbulence along its length (fig B7), it would not appear to have had any particular influence on the nature of the slab collapse which occurred some distance away from it and away from the turbulence that the wall creates. The view of the Panel is that the scour initially originated at the third row of panels in from the left and that it spread laterally, eventually undermining the wall foundations, causing them to collapse inwards rather than being eroded outwards. This also reflects the thoughts of the engineering staff on site at the time who witnessed events as they unfolded.

Figure. B7 Plan of the auxiliary chute and photo of flows impacting against the oblique left side wall

Another unusual feature of the chute is that it contains embedded rocks (plums). Normally such chutes are either planar and smooth, to minimise hydraulic friction loss and hence flow area, or alternatively they contain deliberate features such as baffles or steps to dissipate energy. Baffled chutes tend to contain quite substantial features (baffles) to interrupt the flow and dissipate energy. They also require the associated chute slabs to also be more substantial than those at Toddbrook, given the turbulent impact loading transmitted to them by the baffles. It is not quite clear what was intended at the Toddbrook chute although it has been suggested that the rocks were placed as much as ornamental features rather than as any serious attempt to dissipate energy. They will, however, have attracted local turbulence, impact forces and “stagnation” pressures and these aspects will be discussed in a later sub-section. The chute terminates in something akin to a roller bucket. At very high flows this could cause flows to break free and “flip out” in the manner of a “ski jump” or “flip bucket” type spillway chute. Such a discharge would skip over the downstream channel projecting water from the main chute into the children’s play area further downstream. Indeed, this happened during the recent floods but focused at the left side of the chute. How the remainder of the chute would behave under any larger, “design” flood events would need the be confirmed by physical or computational modelling.

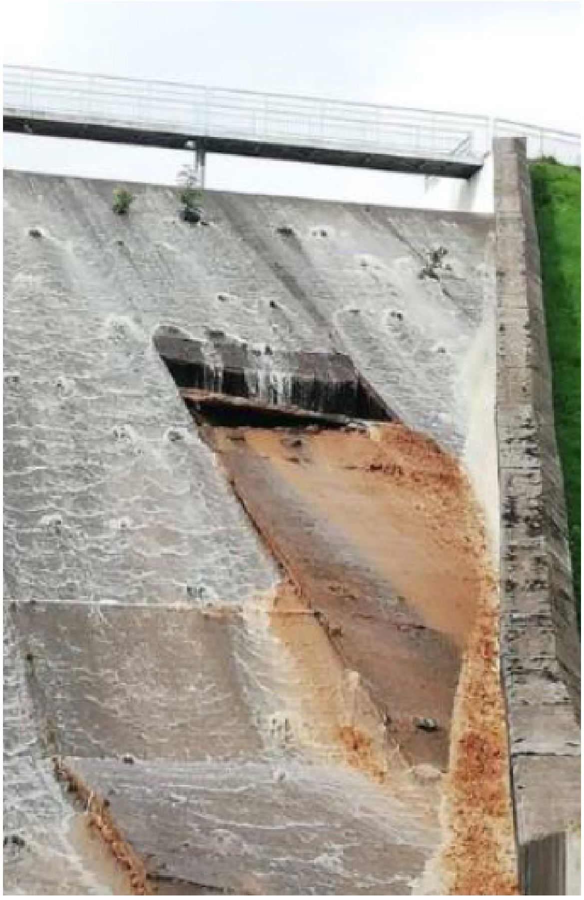
The Panel considers it likely that the person designing the chute was not a specialist in engineering hydraulics and that it was unlikely to have been hydraulically model tested, otherwise the hydraulic deficiencies of the initial design would have become apparent. They consider it highly unlikely that the oblique alignment of the left side wall would have focused or concentrated any seepage flows coming though the crest works. As events on the 1 August 2019 demonstrated, the wall foundations are relatively shallow. Any crest seepages would therefore have flowed downwards through the embankment fill and even with some lateral spread, would have passed well below the wall foundations.

Location and nature of failure

To understand and analyse the mode and sequence of the chute failure, it was vital to establish the location of the initial slab failure. This was not straightforward as initial views from downstream foreshortened the perspective and suggested that the initial failure was located higher on the chute than it was. However, it proved possible to clarify matters using the patterns of embedded rocks on the chute in relation to the main joints and based on historic photographs. The result is illustrated in Fig. B8.

It can be seen from figure B8 that the initial slab failure occurred at the third row of slabs down from the crest and the third row of slabs in from the left side of the chute. The slab was not lifted but rather collapsed downwards into a void which must have formed prior to the slab collapse. Interestingly the upper and lower slab joints remain intact, albeit that the upper slab is deflecting downwards. This implies that the reinforcing wire had either snapped or had failed due to corrosion. The upper section of slab can be seen to be “hanging” on the connecting joint dowel bars.

Earth slurry in the form of what appears to be silt, clay and sand can be seen exiting through open joints due to raised slabs, further down the chute. It has clearly “flowed” from the scour hole above implying a fluidization of the fill in the area of the scour hole. This in turn implies that considerable volumes of water had reached the scoured area during the course of the earlier flows down the chute. The earlier descriptions of events and of silty flows leaving the chute during the flood, see above, indicates that this process had been going on for most, if not all of the preceding day, during the main flood event.



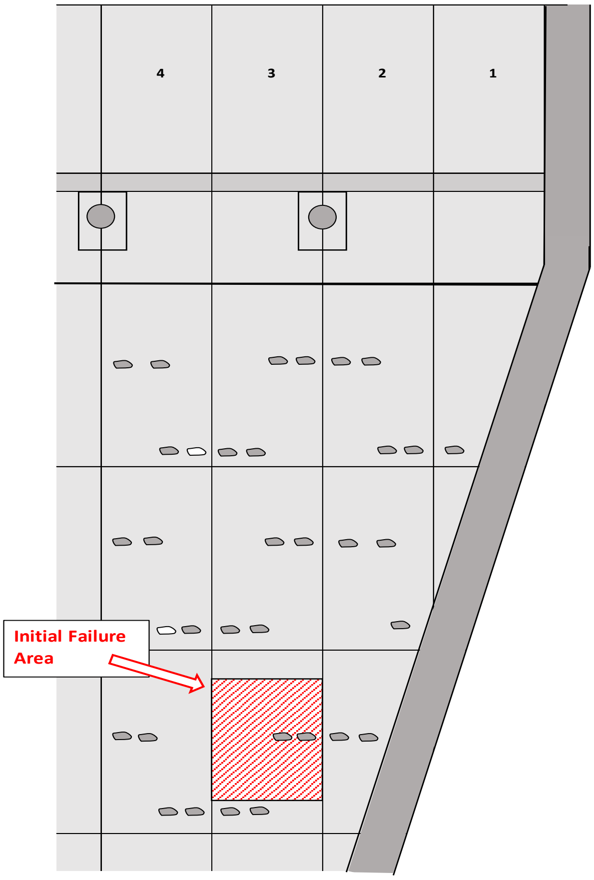


Figure. B8 Location of initial slab failure

Hydro-dynamic effects

There are two particular hydro-dynamic effects to which the Toddbrook chute would have been especially susceptible. The first involves the dynamic impact of flows onto the protruding rocks set into the chute. The second involves the potential high-pressure injection of water beneath the chute where pressure-generating anomalies exist in conjunction with features able to transmit such pressures and flows down through joints and cracks through the chute slabs. The initial chute slab failure occurred between 17m and 22m down the chute as measured along the slope. Embedded rocks on the chute slab would have been present at approximately 19m down the slope. The size, shape and protruding height of these blocks varied over the chute, but 300mm wide and 60mm high was typical. This particular location would have been approximately 8.5m vertically below crest level with spillway flows, allowing for some friction losses, at approximately 12 to 13 m/s. Allowing for "form" losses on their projected area, impact loads on the twin rocks would have been in the order of 0.2 tonnes but likely fluctuating between approx. 0.13 and 0.27 tonnes. The upward deflections of the jets would also have caused equal-and-opposite reactions, with equivalent downward loads on the slabs at such locations.

Another key hydrodynamic factor to consider when a spillway is operating, is the potential for injecting pressurised flow into joints, cracks and holes in the chute at locations where surface anomalies cause locally high-pressure build-up to occur. Such pressures are known as “stagnation” pressures and occur where interruptions to the flow convert velocity head into a localised pressure head. Such features can typically be misaligned joints, local steps caused by concrete spalling, and any other protrusions which locally interrupt smooth flow down the chute. In the case of Toddbrook the embedded rocks on the chute and established vegetation in chute joints and cracks with roots down though the slabs could fall into the latter category, see commentary in the previous section.

This phenomenon has caused many chute invert slab failures. In December 2007 the United States Bureau of Reclamation (USBR) issued a Report on the matter entitled; “Uplift and Crack Flow Resulting from High Velocity Discharges Over Open Offset Joints”. The Report contains illustrations which are reminiscent of the failure at Toddbrook (fig B9). It also states that; “Uplift on chute slabs due to the transmission of pressures through open cracks and/or joints has long been an area of concern at the Bureau of Reclamation and damage has occurred on numerous occasions due to this phenomenon”. The “uplift” in this case means the pressure generated under the slab through jet injection, causing it to lift. It goes on to say that associated, “erosion of foundation materials resulting from flows into cracks or joints (are) a significant problem on soil foundations.”

Figure 3.—Undermining of the chute slab has occurred due to flow entering at joints and cracks and transporting foundation material through unfiltered drains.

Figure 4.—Structural collapse of the slab system occurs when enough undermining has occurred to cause loss of support. This damage is typical of slabs placed on soil foundations (note: this slab is not reinforced so failure is more evident).

Figure. B9 Illustrations of two chute slab failures taken from the referenced USBR report

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A further comment in the Report, perhaps especially relevant at Toddbrook where there is no underdrainage, is that; “This problem is generally more of a concern for structures where the chute and the underdrain systems may be in poor condition due to ageing or improper design and is especially critical for chutes that are founded on soil, since joint/crack flow can lead to erosion and undermining of the chute foundations and structural collapse of the chute slab”.

In order to examine the effect that pressure generating anomalies can have, the USBR Report gives examples of pressures generated in cracks and joints due to offsets into the flow and based on an exhaustive set of modelling. For example a chute flow velocity of 12 to 13 m/s (approximately 41 ft/s) passing over a vented gap 3mm (⅛”) wide and with an offset protruding just 3mm (⅛”) into the flow would generate a pressure in any associated crack or joint equal to a water pressure head of just over 3m or a potential uplift of 3 t/m^2. Any significantly protruding feature would cause full “stagnation” pressures in the crack or joint of almost 7m giving a potential uplift force of almost 7 t/m^2. By comparison the weight of the 150mm chute slab at Toddbrook is just 0.36 t/m^2. Clearly the chute slabs at Toddbrook would not be able to resist such an uplift force beneath them, however the soil foundations would have been somewhat permeable and the issue at Toddbrook would have been more one of flow injection into the foundations due to the locally high pressures. The rates at which flow could have been injected are also covered in the USBR Report and were assessed by both physical and mathematical modelling. Examples from the USBR Report are given in figure B10. Small crack widths and flow impediments are both assumed to be 3mm(⅛”).

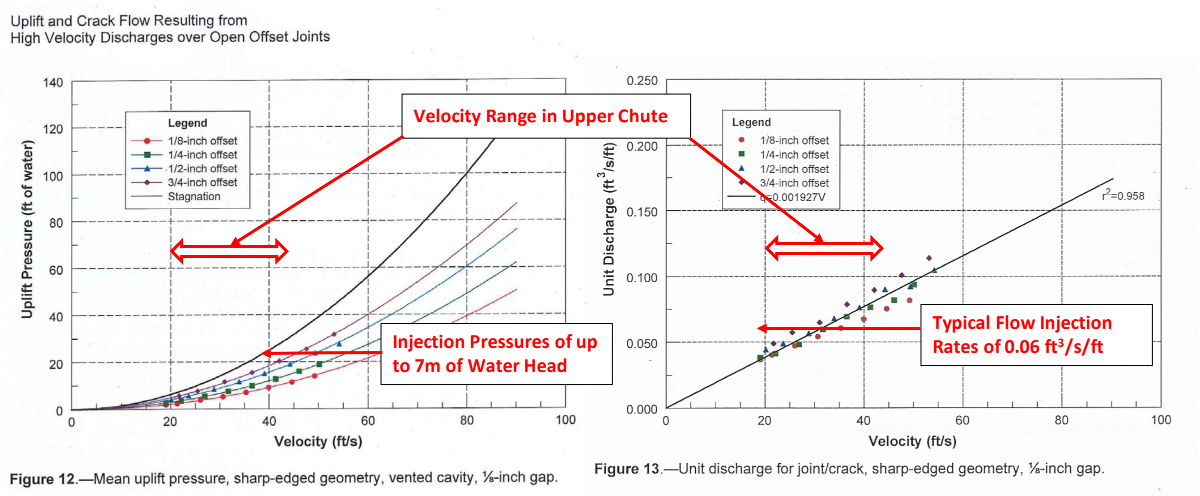

Figure. B10 Illustrating the pressures and injection flow rates possible due to a 3mm chute impediment.

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The flow velocity range applicable over the upper parts of the Toddbrook auxiliary chute and immediately above the initial failure slab can be taken as being in the range 6 to 12 m/s (20 to 40 ft/s). From figure B10 it can be seen that this would generate a unit flow into the crack of between 0.040 and 0.075 ft^3/s/ft. Taking a conservative value of 0.060 ft^3/s/ft, that converts in metric units to 5.576 l/s/m (litres/second/metre) of crack. The Toddbrook chute panels are 3.82m wide, so that assuming two cracks, each 3.82m long across one panel, the flow rate becomes 42.6 l/sec, or 2,556 l/min or 153,576 l/hr. Over the course of a 12 hr (say) flood event this means that approximately 1.84 million litres of water would be injected into the slab cracks or joints. Even a nominal 1 m long crack giving just 13% of the figure would imply the introduction of nearly 240,000 litres of water.

It should be stressed that these values are not definitive as the slabs in question are no longer either intact or available for inspection. However, it is clear from historic photographs showing lines of established vegetation, that at least three such cracks and/or joints existed across the panels immediately above the panel which initially failed (fig. B11). The mechanism demonstrates a means by which water would have potentially been available during spillway flows, to target exactly the foundation area concerned and in sufficient quantities to accomplish much of the substantial erosion and soil liquefaction which occurred on the day. It would also explain why large amounts of silt/earth colouring was apparent in the spillway flow but not in prior drain outlets and other seepages when the spillway was inactive.

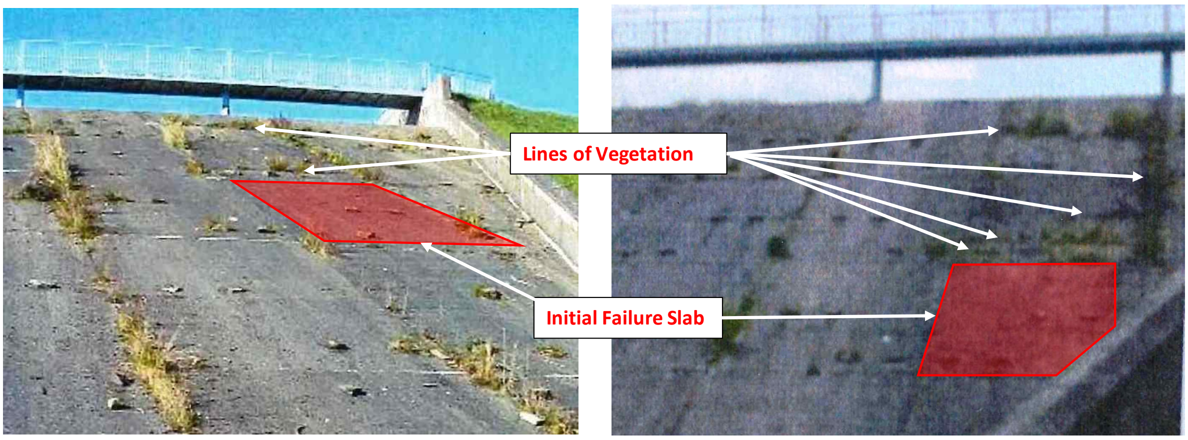

Figure. B11 Lines of vegetation in 2007 and 2016, implying open cracks/joints immediately above the slab which failed initially

Conclusions

Following the preceding discussions, it is concluded that the most likely sequence of events leading to the partial collapse of the auxiliary spillway is as follows:

- during the overspill event (which lasted almost 24 hours with high flows likely for some 12 hours) stagnation pressures developed in at least one upstream crack just above the slab which initially failed.

- the rate of water injection exceeded the capability of the local embankment fill to drain naturally, resulting in its liquefaction into a “slurry” of water, clay, silt and sand.
- this liquified fill then raised the chute slabs sufficiently, with flood water still flowing over them, such that the slurry flowed beneath them, eventually permanently lifting and displacing some lower ones.
- this occurred early in the event and with silt and mud exiting from the sides of the raised lower slabs throughout 31 July 2019, which was the day preceding the initial slab collapse.
- an increasing large void developed under the upper slabs as the material was removed.
- as the flow reduced, the under-pressure ceased, and the water drained from the upper void. The 3rd slab along from the left end and also down, lost support and failed downwards into the void.
- saturated fill and water then slowly escaped, residual flows continued and the sides of the scour hole “relaxed” and widened leading to the collapse of further slabs and the under-mining of the left side wall of the spillway.

With regard to the mechanism involved it can be noted that:

- several lines of vegetation, implying open joints and cracks, can be seen in earlier photographs in the zones immediately above the slab which initially failed.
- any vegetation, and also any chutes rocks in those locations, would have developed stagnation zones injecting pressures and flows into the cracks.
- calculations using USBR Report DSO-07-07 “*Uplift and Crack Flow Resulting from High Velocity Discharges over Open Offset Joints*”, dated Dec 2007 indicate likely injection volumes of between 200,000 and 1.8 million litres of water during a 12-hour flood event, depending on assumed crack length and features.
- the erosion of soil foundations under chute slabs due to stagnation pressures under flow, injecting water into cracks and joints, is a well-established mode of chute failure and has been well researched and quantified in the US. A US research study into the phenomenon refers to it having happened on numerous occasions.
- as further background it can be noted that the Auxiliary Spillway did not meet current design standards and was deficient in many aspects, such as having very thin (150mm) slabs with virtually no reinforcement. Maintenance over the years had been intermittent with extensive plant growth in cracks and joints for prolonged periods, suggesting open passageways to the embankment beneath. Occasional repairs had been done but only to some aspects of the chute. The thin slabs and poorly detailed joints would have rendered the chute especially vulnerable to deterioration due to such factors. Generally, the slab concrete remained sound but

there is honeycombing and/or deterioration at some joints, some missing chute rocks, some cracking and evidence of significant prior plant roots through joints and in some cases through slabs.

- the presence of embedded rocks (plums) on such thin slabs can best be described as unfortunate. Some seem to have attracted plant growth while the rocks themselves interrupt any flow down the chute, locally converting the velocity energy in local “stagnation” pressures, capable of injecting water into any local open cracks or joints.

To summarise, it is the Panel’s opinion that;

- the most likely initiation event and mechanism for the introduction of the volumes of water necessary to liquify 800t of fill is joint and/or crack injection due to the development of “stagnation” pressures on the chute.

- the sporadic nature of maintenance work, allowing long periods of extensive plant and root growth, would have led to deterioration of the chute, which then allowed the joint and/or crack injection described above

- the inadequate design of the chute would have contributed to its eventual collapse

- crest seepages would likely have occurred, as they will have, every year due to the prolonged high reservoir levels during winter months. But the core and immediate downstream fill were intact when later exposed by the collapsed panel. Indeed, the minimal change in surveyed spillway crest levels of only 6 to 10mm in the past 20 years indicates minimal, if any, loss of underlying material. Nor was there prior evidence of any long-term silt/earth losses in downstream seepages. The downstream shoulder (fill) material is quite permeable as evidenced by measured phreatic water levels within the embankment. Seepages would have tended to fall downwards through the fill by gravity rather than progress laterally under the slabs. Where crest seepage may have migrated downstream under the chute the result would appear to have been localised pockets of either erosion or differential settlement.

- while sustained long-term seepages could well have caused some saturation to the upper areas of downstream fill close to the core, and some erosion and/or differential settlement beneath the chute slabs, there is no evidence that this would have caused the large void eventually revealed under the chute slabs. It would not have been able to supply the large volumes of water needed to displace 800t (approx. 400 m^3) of earth fill.

- even with good maintenance the inadequately designed spillway would not be have been capable of accommodating the probable maximum flood

Appendix C. Other safety aspects of Toddbrook reservoir

Introduction

Four principal risks were identified in the 2019 Inspection Report. The potential modes of failure were identified as “credible” and “significant”:

1) Potential hydrodynamic damage to the secondary spillway
2) Internal erosion through the embankment
3) Slope instability / high pore pressure
4) Earthquakes causing collapse of mine workings

The hydrodynamic damage to the auxiliary spillway and the causes of failure are described in the main report and in more detail in Appendix B. The remaining hazards are described in the following sections. It is written for those with some knowledge of reservoir engineering and geotechnics.

Internal erosion through the embankment

Internal erosion involves the removal of solid material, usually in suspension, from within an embankment or its foundation. Many early dam failures and incidents were attributed to internal erosion on first filling.

Erosion through the embankment will only take place if:

- there is a mechanism to create a leakage path
- the path once formed stays open - this is related to hydraulic gradient or head
- the clay (core material) is erodible due to the passage of water and if it is not halted by some form of downstream filter, which could be the properties of the downstream fill

The various mechanisms of leakage may be associated with construction defects or weaknesses, permeable inclusions in the core, differential settlement and stress conditions, permeable foundations, including fissured rock, solution holes, or mine workings. Hydraulic fracture through a clay core is often cited as a possible mechanism to create a leakage path. The susceptibility of a clay core dam to hydraulic fracture is related to stress reduction associated with reduced differential settlement of the core relative to stiffer adjacent fill, foundations, abutments, structures, or conduits. Problems associated with seepage and leakage within embankment dams may be due to critical conditions developing at interfaces between dissimilar materials.

Internal erosion is often hidden and usually localised. It can take place very slowly over many years and may only become apparent with the appearance of turbid leakage, the appearance of a sinkhole or localised deformation of the embankment. On first filling it can occur quite rapidly. In recent years the likelihood of internal erosion developing in embankment dams has been the subject of UK guidance and a Bulletin by the International Commission on Large Dams. At least one Owner of several Pennine type dams in the UK routinely assesses the risk of internal erosion using a set of “Internal Erosion Toolbox” guidelines produced by the United States Bureau of Reclamation and others.

Internal erosion and leakage at Toddbrook

Toddbrook has a long history of leakage and internal erosion associated with complaints of leakage into mine workings, and the formation of substantial depressions (sinkholes) on the upstream slope and leakage at the downstream toe. The complex leakage investigations in the late 1970s and early 1980s involved the construction of exploratory shafts, boreholes and instrumentation led to the dam being drawn down for seven years, twice the length of time it took to construct the dam. The incidents, investigations and remedial works are summarised in Table C1 and Figures C1 and C2.

Table C1. History of leakage and internal erosion

Date	Event
1880	Complaints about leakage into mines
1895	"Old pit shaft" investigated as possible cause of leakage reported 1880 but found to be practically dry and tipped full of puddle.
1930	Leakage observed at toe of downstream slope. A manhole (now known as Shaft No 5, see Figs.C1 and C2) was constructed to make continuous inspections of this leakage. A depression was found on the upstream slope.
1931	A shaft was dug within this depression on upstream face to depth of 8.5m where decayed vegetation found at level believed to be original ground level. No culvert, shaft or water was found, and area made good with clay and pitching re-instated.
1975	In November, when the reservoir was low, a depression was noted in the approximate position on the upstream face as the 1931 depression. A stone plaque “culvert 40 ft deep” was found in the depression. See Fig C.1
1976	Autumn, the depression was more pronounced.

1977	In Autumn, 120 mm of subsidence had occurred since 1975 and the reservoir was emptied. When the reservoir was drawdown, a crater 4 to 5m across, which was partially infilled with silt into which a middle size tree had been sucked.
1978	An exploratory shaft was sunk at the location of the crater and several boreholes were put down between this and shaft No 5 on the downstream side. In one borehole from the crest a 0.6m dia. cast-iron pipe was found. Tracer tests showed that there was a connection between this borehole and the shaft.
1980	Flows from V-notch weir in 1980 probably come from seepage beneath the dam. Flows do not appear to respond to reservoir level or rainfall.

Investigations and remedial works

Despite extensive investigations, uncertainty remained over the location of mine shafts, the cause of the depressions and the route of the leakage paths. Potential leakage paths were discovered during the investigations. These included the cast iron pipes indicated in Figure C2, which are used to draw down the reservoir level, and a 1.2m diameter masonry culvert, possibly used as a stream diversion during construction.

Remedial works in 1981 involved placing a 1.2m compacted clay blanket layer over part of the reservoir floor and the upstream slope where the depression occurred, as shown in Figure C1. During the placing of the clay blanket, the masonry culvert was discovered. Tracer tests showed this to have formed a leakage path through the dam. Figure C1 shows it terminating on the upstream side of the core.

Concerns and uncertainty that leakage and possibly internal erosion could take place through the core and foundation, led to a grout curtain being installed in 1983. Grout takes (a “take” is the amount used) varied significantly. One large grout take, some 1400kg of cement (approximately 5 cubic metres of grout), occurred between 15m and 25m below crest level, and was injected below the auxiliary spillway crest slab at the level of the original stream bed, where the masonry culvert was built, just upstream of the core. Elsewhere grout takes of up to 3400kg of cement (approximately 11 cubic metres of grout) were recorded. A cement/bentonite/water grout mix was used, typically 10/1/30 but varying depending on its use. Apart from reducing leakage through and under the core, the grout would have strengthened the core, increasing its shear strength against potential deep-seated slope instability.

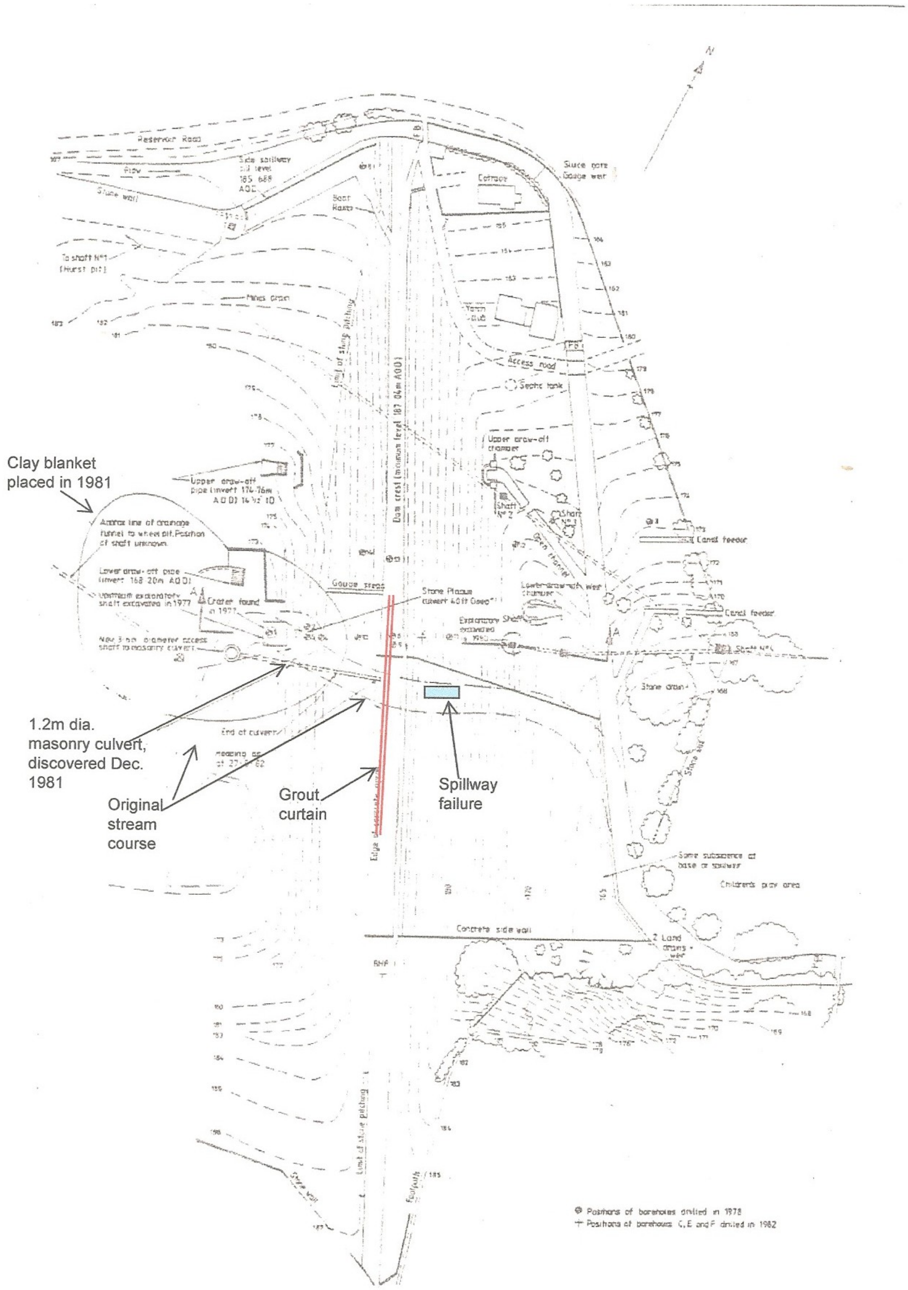

Figure C1. Plan showing location of grout curtain and clay blanket remedial works
(Based on British Waterways Drawing HH /BWB /02/03, 1984)

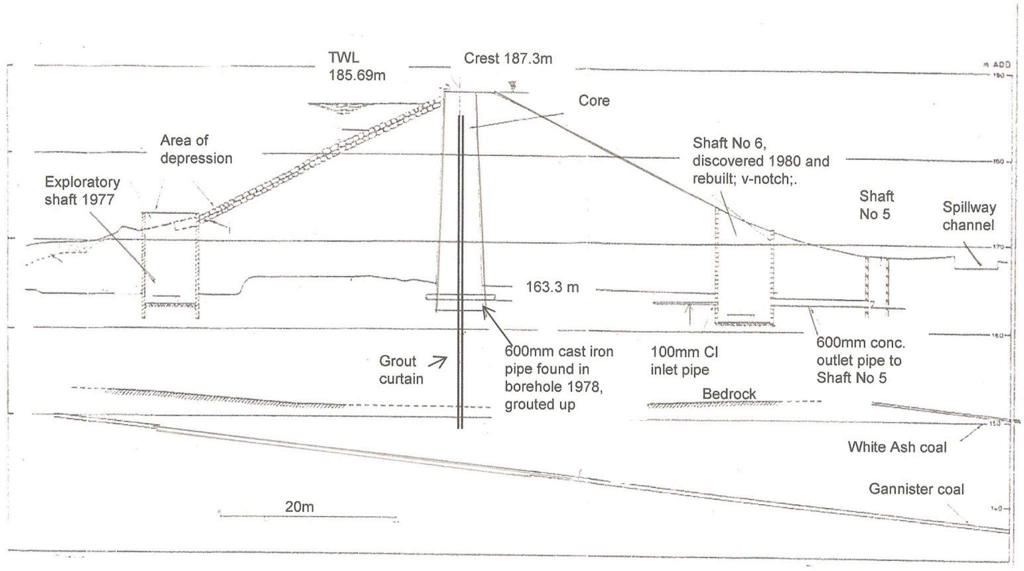

Figure C2. Section showing location of grout curtain various investigation and monitoring shafts (Based on British Waterways Drawing HH /BWB /02/03, 1984)

Current risks posed by internal erosion through the core

Piezometric measurements immediately upstream and downstream of the core can give an indication of the effectiveness of the core. Following on from the 1981-83 remedial works, the piezometers in the upstream fill reacted directly to the fluctuating reservoir level demonstrating that the full reservoir head is acting against the core. The upstream piezometers in the foundation glacial till showed little response to changes in reservoir levels demonstrating its apparent effectiveness as a cut-off. Generally, the piezometers in the downstream fill indicate low pore water pressures which are not responding significantly to changes in reservoir level, which could indicate the core is effective.

During his 2018 inspection, the Inspecting Engineer did not observe any movement in the surrounding ground which may be affecting the stability of the embankment, but noted that the foundations of the dam are traversed by historic tunnel drains and various mine workings (some of which are not fully recorded). He was concerned that the variable construction materials and lack of formal filtering meant a likely high risk of internal erosion. Piezometers previously installed to monitor porewater pressures for use in stability analyses were no longer working. He required the Owner to carry out improvements to the leakage monitoring regime within the embankment to enhance drainage reliability and leakage detection. The intention was to assess the risk of embankment instability due to an increase in pore water pressure and the likelihood of internal erosion caused by leakage into the underlying mine workings or drainage tunnels.

Embankment slope instability and the potential effects of a seismic event

Background

Slope instability occurs when the disturbing forces acting on an underlying plane (surface) within the ground, due to the weight of materials above that surface, are larger than the resisting forces characterised by the shear strength of the soil. This instability can result in mass movement of the soil downwards and outwards. Instability in an embankment may result from a shear failure either in the embankment alone or in the fill and foundation. The failure usually takes the form of shearing along a slip surface which may be circular in a homogeneous material or where weak zones or layers exist. It is likely to be non-circular, as in the case of a dam with a weak puddle clay core and a weak clay foundation. Slips may be shallow or deep seated. Also, the presence of sands that could be prone to liquefaction needs to be considered.

Slope instability at Toddbrook since construction

Since construction, as far as is known, there are no reported incidents of slope instability on the main embankment slopes despite them being relatively steep at 1V:2H, with the exception of the minor sloughing of the clay blanket on the upstream side. Also, a persistent issue of water on the landslip is apparent from the hillside adjacent to the sheet pile retaining wall at the toe of the auxiliary spillway. Springs and running sands were identified in this area.

The lack of evidence of slope instability over such a long period since construction indicates that the shoulder fill appears to have a sufficiently high shear strength and to drain freely to allow excess pore pressures to dissipate rapidly on reservoir drawdown. This was confirmed by piezometer measurements located in the upstream fill in the 1980s. There are relatively few instances of slope instability incidents at reservoirs in service in the UK compared with other types of incidents. Generally, they have been associated with exceptional circumstances, such a saturation of the downstream due to wave spray, rather than gradual age deterioration.

The first time a slope stability analysis was carried out at Toddbrook was in 1985 as reported by the Inspecting Engineer. This was 140 years after construction with no concerns being expressed about slope stability in that time. Stability analyses have also been reported by a Geotechnical Desk Study Report and Seismic Stability Report CRT 2 2007 and 2019 Toddbrook Reservoir Stability Review, 16 September 2019.

The analyses undertaken during the three studies of stability all indicate a Factor of Safety above 1.3 for the static case. In 2018 the Inspecting Engineer noted that the downstream face of the embankment had been shown to have an adequate factor of safety, dependant on the pore water levels in the embankment. He required the Owner to carry out improvements to the borehole monitoring regime within the embankment to reduce the risk of embankment instability due to an increase of the phreatic surface levels not being adequately recorded. He also required improvements to the embankment level monitoring regime to reduce the risk of surface levels not being adequately recorded and instability going unnoticed.

Seismic effects on slope stability

The dam falls within Category III as defined by “An Engineering Guide to the Seismic Risk to Dams in the United Kingdom”, J. A. Charles et al, 1991 BRE Report BR210. Two of the studies analysing the seismic effects used a pseudo-static analysis. Both propose further studies using dynamic analysis. Dynamic analyses require specialist skill and substantial assembly of data. However, knowledge of shear stress variation during earthquake shaking is limited. The embankment is around 160 years old and is likely to have been subjected to some seismic loading in the past, with no obvious ill effects.

Earthquakes causing collapse of mine workings

This section considers the stability of the mine workings and drainage tunnels. Studies by British Waterways mining reports concluded that, with the exception of the White Ash seam, which is the shallowest of the three major seams, there was a low risk of collapse of roadways and workings. Collapse of the White Ash seam, although only 0.3 to 0.6m thick could cause settlement. Monitoring of the old mineworking drainage tunnel is important, particularly after a seismic event. Generally, underground structures, if well built, are resilient to seismic events.

The Inspecting Engineer did not require any MIOS relating to this aspect of the dam as a result of his 2018 inspection but recommended surveillance after such an event and additional monitoring.

Instrumentation and monitoring

The previous sections indicate the importance of monitoring in managing the safety of reservoirs. This section explains the role of instrumentation and monitoring in more detail.

Instrumentation has been installed at Toddbrook reservoir at various times since the 1978 investigations:

- flow measurements on drainage tunnels using V- notch on weirs at the river outfall and in Shaft No 6
- piezometers in the core, shoulders and foundation
- level stations on the embankment dam to determine settlement

Piezometric measurements

Piezometers have been installed during the various ground investigation works; all being standpipe (Casagrande) type. Although observations have been sporadic and incomplete, partly due poor installation and maintenance, they have provided an overview of the ground water levels in the embankment.

Typically, piezometers measurements are used to:

- compute factors of safety for the embankment slopes in effective stress limit equilibrium stability analyses by measuring pore water pressures in the embankment slopes and foundations
- measure the effectiveness of the core or cut-off by placing piezometers either side of the core. A knowledge of the ground conditions is essential for positioning the tips to measure the maximum head and avoid obtaining false readings due to perched water tables.
- provide information relevant to assessing slope stability in the upstream fill during rapid drawdown.

They can also be used for permeability measurements. As far as is known no permeability measurements have been made in the piezometers at Toddbrook. Despite the limitations of the piezometers installed at Toddbrook, the data indicates the core is functioning to prevent high piezometric pressure developing in the downstream fill.

Crest settlement

The routine monitoring of crest settlement on embankment dams can provide a valuable insight into their behaviour and has an important role in assessing safety. It is important to determine whether long-term settlements, measured in this case 180 years after construction, are due to any type of incipient malfunction of the dam which could lead to a failure if remedial action is not taken.

Along the embankment, crest settlements over the past 20 years have been fairly nominal, typically 10 to 20mm. There has been slightly more settlement of pins SP4, SP12, SP15 and SP16. These are noted the in the 2018 Inspection Report. The slightly higher settlements of SP15 and SP16 are, as the Inspection Engineer notes, likely due to them being sited on the backfill behind the spillway side walls. However, the Inspecting Engineer recommended further investigation of SP4 and SP12, neither of which are adjacent to the auxiliary spillway.

The presence of the curved waterline at TWL is common on many typical Pennine dams in steep sided valleys. This is due to settlements being proportional to the depth of fill. Toddbrook has a relatively flat valley and there is no observable indication of large settlements in the deeper parts of the dam.

Spillway crest slab settlement

Current levels along the auxiliary weir crest indicate that it may have settled between 76 and 96 mm in that time, depending on how accurately it was set during construction 50 year ago. Surveying over the past 20 years indicates that auxiliary crest settlements (as measured along the downstream ends of the downstream crest slabs) have typically settled only 6 to 10mm in that time. This is a minimal amount. The downstream slabs also have a 75mm (approx.) drop towards the left side, although along the auxiliary weir crest the fall is only 20mm. Such a small amount of settlement over the past 20 years would not normally give any indication of distress. However, the measurements would not indicate any localised loss of material under the crest slabs. Note, however, that the recent satellite data apparently indicates a greater settlement of the left-hand side of the crest in the months leading up to the event.

Assessment of embankment movement and settlement observations

From the very first inspection in 1933 it was stated that there was no indication of embankment movement or settlement. The embankment has the appearance that little settlement has occurred due to the lack of a curved water line at TWL on the upstream face. This can be largely accounted for by the presence of a relatively flat valley. The wave wall built in 1986 also shows no sign of distortion or cracking. Settlements are generally very small for a 24m high dam with the few notable exceptions.

The observations of settlement although not complete, and sometimes erratic, would not have given any cause for concern that the auxiliary spillway could malfunction. However, the severe zig-zag cracking of the spillway crest slab is most likely due to settlement of the underlying fill. Further information on internal erosion, slope instability and the assessment of monitoring of embankment dams can be found in:

JOHNSTON T A, MILLMORE J P, CHARLES J A and TEDD P, “An engineering guide to the safety of embankment dams in the United Kingdom”, Building Research Establishment report BR363. BRE, Garston, Watford, UK, second edition, 1999.