



Canal & River Trust

Report on the Nature and Root Cause of the Toddbrook Reservoir Auxiliary Spillway Failure on 1st August 2019



Dr Andy Hughes
February 2020

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Executive Summary

This report seeks to investigate the root cause of the recent incident involving damage to the auxiliary spillway at Toddbrook Reservoir, owned and operated by the Canal & River Trust and situated just outside the town of Whaley Bridge, Derbyshire.

The report describes the type of dam and describes the modification of the dam, by the introduction of an auxiliary spillway 'over the top' of the dam in 1970.

The spillway was designed 'in-house' by British Waterways staff of the time and signed off as acceptable by a Panel 1 Engineer as constituted under the Reservoirs (Safety Provisions) Act 1930.

It appears from a recent survey that the spillway was constructed some 100 mm lower than designed. The spillway appeared to operate satisfactorily until the incident on the 31st July/ 1st August 2019.

Intrusive investigations into the construction of the spillway undertaken, and witnessed by myself, showed that the spillway had been built as required by the drawings to a relatively high standard, in difficult conditions on the steep downstream slope of an embankment dam.

Review of those same drawings identified serious and fundamental flaws in the design, which included: no cut-off into the core beneath the weir slab, no water bars on the vertical joints, very thin slabs with minimal reinforcement, inadequate pressure relief, and poor wall floor slab connections.

With the underside of the slab of the auxiliary spillway, now proven to be below top water level (by approximately 300 mm) as defined by the main spillway weir, there were many days in the spillway's history that water could pass under the slab and over the core of the dam, exacerbated by the normal settlement that would be expected of a Pennine type dam year on year. This settlement, coupled with a rigid concrete slab, would lead to a gap being formed beneath the slab through which water could pass. This water, depending on the duration of the flow and the velocity/ volumes involved, could lead to removal of fill from beneath the slab, as witnessed by people over the years as spurts of discoloured water coming up through the joints on the spillway. This could have been happening for years, the flood event on the 31st July/1st August being the one which caused movement of the slabs and loss of large amounts of material.

The report reviews the various engineering assessments carried out by the independent Inspecting Engineers (appointed by the Secretary of State to the All Reservoir Panel and contracted, for the past two decades at least, from Mott MacDonald), who undertook statutory inspections in accordance with the Reservoirs Act 1975; and relevant statements from the Supervising Engineers, responsible for the Reservoir, over the past 50 years. I conclude, admittedly with the benefit of hindsight, that there were indications, that an experienced reservoir engineer should have identified, of the flaws in the design and the potential for a problem to exist and develop. The Canal & River Trust could reasonably expect to rely upon this expertise to identify and alert them to any risks and the warning signs could have been recognised by those reservoir engineers earlier; indeed the latest Inspecting Engineer should have required more urgent action to investigate the spillway than he specified in his formal report, submitted in April 2019. If that had happened, then the failure/ incident could well have been avoided.

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10th February 2020

1. Introduction

The Canal & River Trust (C&RT) was formed in 2012, taking over the guardianship of British Waterways' canals, rivers, reservoirs and docks in England and Wales.

The C&RT looks after more than 2,000 miles of canal, 2,980 bridges, 1,580 locks and 335 aqueducts. Also included in the portfolio of assets is 72 reservoirs under the Reservoirs Act 1975.

Toddbrook is one of those reservoirs and is situated approximately 0.5 km to the south west of Whaley Bridge in the Derbyshire at National Grid Reference SK 006 809.

During w/c 28th July 2019 the catchment feeding Toddbrook Reservoir experienced two storms in quick succession which caused the reservoir level to rise and, as a result, on the 31st July, the auxiliary spillway came into operation as well as the main spillway which had been discharging water all week.



Toddbrook Reservoir



Flow on spillway channel

The auxiliary spillway then broke up on the left-hand side (looking downstream) with severe damage to the slabs and removal of large amounts of fill from beneath the slabs in that area.



Damage at left hand end



Emergency works (note the void outside the wall)

As a result, a full-scale emergency was declared, on 1st August 2019, more than 1,500 residents in Whaley Bridge were evacuated for six days returning home on 7th August, and emergency action was taken to stabilise the situation and draw down the water level in the reservoir, as part of an effective multi-agency response.

2. Author and Scope of report

I was commissioned by the C&RT to undertake the following task:

- Investigate the nature and root cause of the Toddbrook Reservoir auxiliary spillway failure on 1st August 2019 including a review of the history of the installation and development of the spillway from 1st December 1964 to 1st August 2019.

This report seeks to review the history of the dam in relation to the construction and subsequent performance of the auxiliary spillway as well as the surveillance, monitoring and inspections that have been carried out between its construction and its failure. It seeks to understand why the failure occurred and to develop an understanding of the nature and root cause of the failure.

I obtained a 1st Class Honours degree specialising in Dam Engineering at the University of Newcastle upon Tyne in 1975 and then obtained a PhD in Dam Engineering at the same university in 1978. I gained a DMS with distinction from the University of Lancaster in 1981.

I am now an independent engineer working as an All Reservoir Panel Engineer (ARPE) and have been a Panel Engineer for more than 30 years.

I was responsible for reservoir safety in United Utilities (formerly North West Water) before going to Cyprus to build a new dam. I became a partner of RKL, a small civil engineering consultancy specialising in dam engineering, on my return from Cyprus and then became a director at Arup before managing the reservoir team at Kellogg, Brown and Root and then Atkins before becoming independent running my own business.

I have been Chairman and Vice Chairman of the British Dam Society on two occasions. I have been advisor to Defra on Reservoir Safety and advisor under the SAGE to Government. I regularly teach on a number of specialist training courses in dam engineering and I am a visiting Professor at Bristol University and teach regularly at University College London.

I have published extensively and am author and co-author of a number of textbooks and guidance documents and I have written more than 80 technical papers.

My CV is provided in Appendix E.

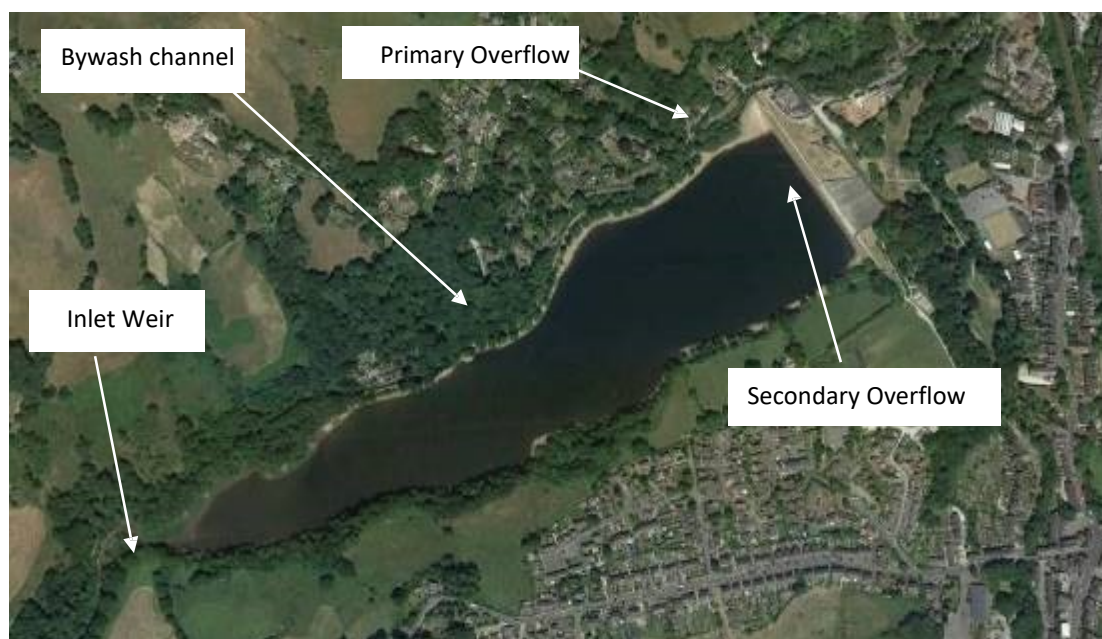
3. History

The dam at Toddbrook retaining the reservoir is a 'conventional' earthfill embankment with puddle clay core completed in 1840. The dam has a maximum height of some 23.8 metres and a crest length of 201 metres.

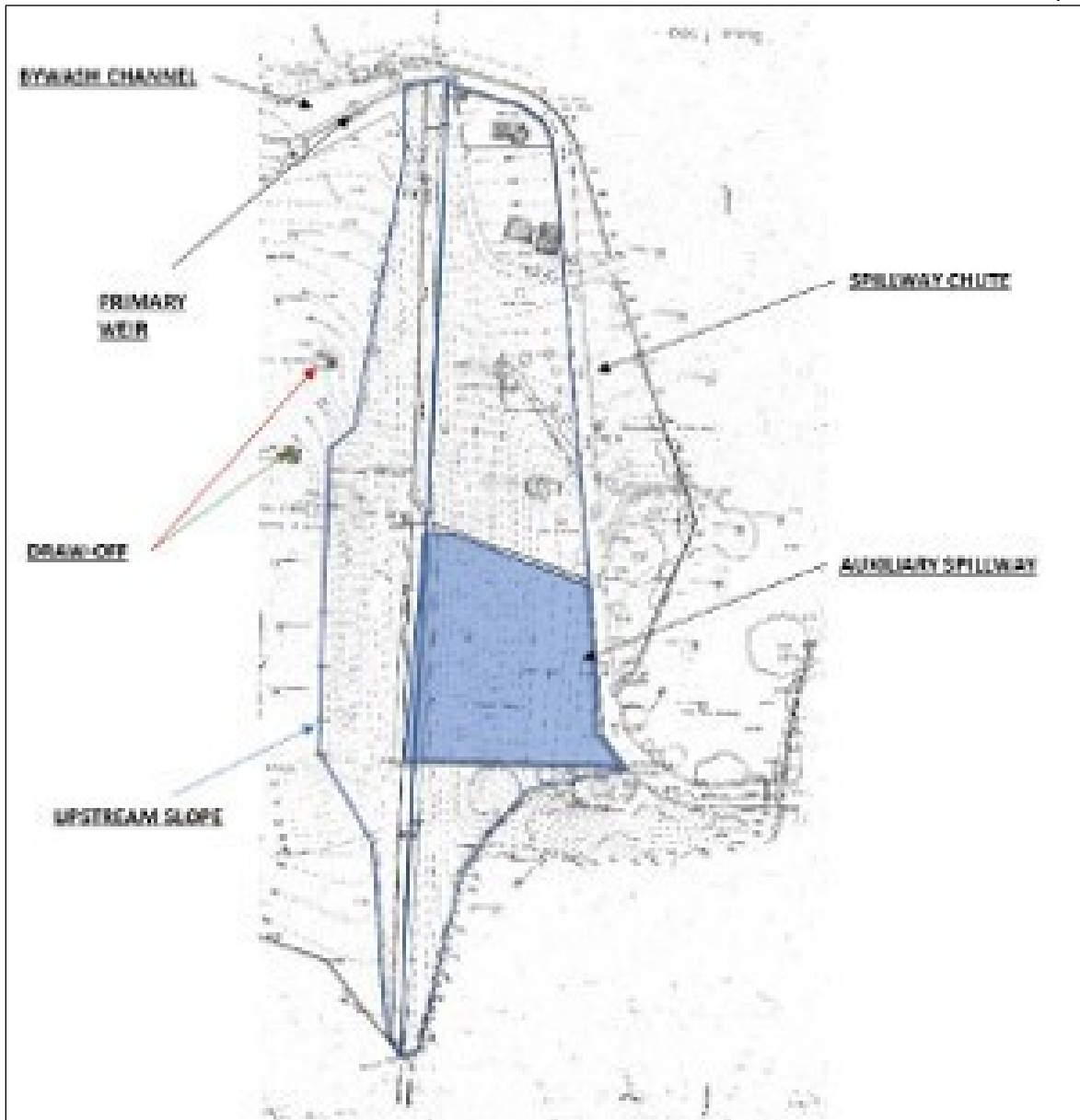
The downstream slopes of the dam are 1 in 2 (v:h) for the first 15 metres below the crest, flattening to 1 in 3 (v:h) thereafter.

The reservoir was built to supply water to the Peak Forest Canal and has continued to do so under the management of the C&RT (formerly British Waterways).

The reservoir stores some 1,288,000 cubic metres of water in a reservoir with a surface area of 158,000 square metres at its top water level of 185.69 metres AOD.



General Arrangement



Plan of Embankment

The dam and reservoir have, over the years, experienced a few problems. The most notable associated with the incident in question was that on the 12th December 1964 when there was a 'significant flood event'. It is said that records indicate that the flood resulted in a peak flood level of 3 ft 4 inches (1.00 metre) over the main spillway for 24 hours. Apparently, it was two days before the flood abated. The main spillway channel, as we know it today, suffered significant damage.

As a result of this event, the Inspecting Engineer (IE) at the time, recommended a second spillway, an "auxiliary spillway", be built to provide additional spillway capacity.

The main spillway has a length of 41.2 metres at a level of 185.69 metres AOD. The spillway is located at the north western end of the dam (left-hand end – looking downstream). The wide spillway crest discharges to a very shallow tumble bay.



Inlet weir to bywash channel



Bywash channel

The bywash channel, which runs along the left-hand side (northern rim) of the reservoir, is controlled by a structure and penstock above the head of the reservoir, then meets the spillway channel.



**Main spillway weir and bywash channel
(weir blocks removed during the incident)**

The combined spillway chute and bywash channel then passes behind what was the reservoir keeper's house and the sailing club before passing across the toe of the embankment from left to right.

The auxiliary spillway was built in 1970, over the central part of the embankment with a chute down the downstream slope to meet the main spillway. The auxiliary spillway has a length of 76.2 metres set 0.26 metres above the main spillway level at a level of 185.95 metres AOD (since the incident it has been established that the weir is some 100 mm lower than this).

4. Auxiliary Spillway – Details of Design and Construction

The spillway was designed 'in house' by British Waterways staff and overseen by a Panel 1 Engineer under the Reservoirs (Safety Provisions) Act 1930⁽¹⁾ (the equivalent of an All Reservoirs Panel Engineer under the Reservoirs Act 1975)⁽²⁾. Later it was overseen by another Panel 1 engineer after the first passed away during the process of design and then construction.

The design was carried out in 1968/69 and construction was undertaken in 1969/70 by John Mowlem and Company Ltd of Leeds.

The design incorporated a concrete slab through the crest of the embankment and a steep concrete chute down the downstream slope to join the overflow channel which runs across the toe from left to right.

(Note: the convention for dam engineering is to stand on the crest and look downstream with the features described as 'left' and 'right').

The chute (or spillway) on the slope does not have parallel sides; the left-hand wall is at an angle such that the channel narrows towards the base. The right-hand wall is at right angles to the weir and runs straight down the face.

The Contractor saw the placing of concrete in a 1:2 slope as a "formidable undertaking".⁽³⁾

The paper on the construction describes the need "to remove 6 ft 6 inches (1.98 metres) depth of soil from the top of the existing embankment, the construction of concrete retaining walls at each side, and a new 250 ft (76.2 metres) long weir over the width of the spillway. The downstream face was to be concreted for a length of 240 feet (73.2 metres) at a maximum slope of 1 in 1.9, and a minimum slope of 1 in 10 terminating with the construction of a new mass concrete wall to an existing outlet channel".

The spillway consisted of a 6-inch (150 mm) thick concrete slabs with 18-inch (450 mm) deep by 18-inch (450 mm) wide 'ribs' running horizontally at the lower end of each bay at approximately 25 ft (7.62 metre) intervals.

Each bay was to be separated from the bay below by an expansion joint ½ inch (12 mm) wide with a dumb-bell water bar cast into the two adjacent slabs and a 'Pliastic' type sealant poured into the top ¾ inch (18 mm) of the joint.

Granite chippings were poured onto the top surface of the wet concrete and tamped in. Stone 'plums' 9-inch x 9-inch x 9-inch (225 mm x 225 mm x 225 mm) were grouted into the wet concrete in a pattern which consisted of 4 No. per 25 ft x 12 ft 6-inch (7.62 x 3.8 metres) bay. The total area of inclined slab was 4,700 square yards (3,930 square metres).



Plums



Granite chips

The concrete used had a minimum cement content of 550 lbs/cubic yard, a water cement ratio of 0.5 by weight, a maximum aggregate size of $\frac{3}{4}$ inch (18 mm) and a minimum 28-day strength of 3300 lbs/sq inch (22.75 N/mm²). Concrete with a 1-inch (25 mm) slump was used.

One layer of mesh was used in the spillway slab and mesh used in the ribs was bent to shape. The mesh was found to be very small, about 3 mm diameter.

Building paper was used below the slab and $\frac{1}{2}$ inch (12 mm) Derbyshire basalt was spread at the rate of 250 square yards/ton.

A 'Smith 26' crane with an 80 ft (24.4 metres) long jib was used to place concrete between the lowered part of the spillway and within 30 ft of the top of the spillway area. Pouring was commenced at the top of each bay, levelled off by hand and then a vibrating ramp was used - two passes were found necessary; before the granite chippings were spread and tamped in by hand. Apparently only 50% of the chippings placed adhered satisfactorily due to the concrete being well compacted.

Concrete in the top bays was placed using 2 No. Thwaites 2-ton hydraulic dumpers running along the top of the embankment, the concrete being shovelled down into the bay by hand – a distance of 27 ft (8.22 metres).

Mowlem provided full-time supervision by an Agent/Engineer and General Foreman and a British Waterways 'Supervising Engineer' (not within the meaning of the current Act) visited twice weekly, presumably to check the quality of construction.

The asymmetrical shape of the spillway is unusual, and it is not known why the left-hand wall was not kept parallel to the right-hand wall. It may have been some attempt to save money, but it is likely that the spillway was not model tested which might have identified hydraulic and energy dissipation problems particularly at the junction of the spillway channel and the auxiliary spillway channel. The design adopted would suggest a lack of understanding of basic hydraulics by those involved in the British Waterways design at the time.

Clearly water tends to run downhill in straight lines and so if there is a wall running obliquely to the slope the water is bound to hit the wall and accumulate against it with associated forces, turbulence and cross waves etc. In addition, 'plums' on the face are likely to disrupt flows, cause turbulence and negative pressures.

4.1 Toddbrook Reservoir Safety Works – Final Feasibility Report, Jacobs, May 2008

In May 2008, Jacobs was appointed to review Reservoir Safety Works at Toddbrook. Jacobs state that they were commissioned by British Waterways to “*review the existing hydrological assessment and prepare hydrographs for the given return periods, and then to undertake a spillway capacity evaluation*”. As a result, the left-hand wall was raised (it appears on two occasions) over the lower part of its length. The raising suggested was by up to a metre, with the removal of the plums and gabion mattresses outside the spillway. Some wall raising was done but it appears that the provision of the gabions and the plum removal was not done.

Their study did identify:

- that the crump weir, just downstream of the weir, causes the 'main' overflow to drown out at outflows exceeding 5 cumecs;
- the peak water level for the probable maximum flood (PMF) to be 187.13 metres AOD.

The exercise carried out was to decide whether to raise the left-hand side wall and evaluate the degree of raising.

The report⁽⁴⁾ describes the 'inlet', the 'chute' and the 'outlet' of the spillway. They state that the slab is 'constructed' in 7.6 metres (25 ft) square panels jointed by PVC water-stops. (*This is not correct – the drawings show water bars only on horizontal joints and this was proved in trial holes excavated under my direction*).

They identified that the alignment to the left-hand wall could cause the generation of cross waves which, along with concentration of water against the wall and rebound effect from the embedded plum stones, results in a water level and spray to a high depth relative to the wall height; that can almost certainly cause overtopping of the wall.

The results predict a flow of 164 cumecs in the PMF event with a depth at the weir of 1.3 metres. They predicted high water levels against the left-hand wall towards the bottom of the chute. They attributed a high degree of air entrainment in the flow as a result of the effects of a high velocity, and plum stones causing turbulence.

They state, “*the problem with the auxiliary spillway lies with the oblique orientation of the left wall such that in an extreme flood the left wall would be severely overtopped*”

which would result in erosion of the downstream shoulder of the embankment". They, at no time, discuss the ability and adequacy of the overflow from a hydraulic performance point of view.

One of the options to meet the freeboard requirements was to permanently lower the reservoir full supply level but this was not pursued.

They identified three options for the auxiliary spillway.

They followed "Option One" which involved raising the wall in concrete which would be dowelled into the existing wall. They state that rockfill gabion mattresses would be put over the embankment surface in the vicinity of the wall to then protect the embankment from erosion from spray or if the wall is overtopped. They also say that the wall section would incorporate a shape deflector to minimise overflow. They state the plum stones should be removed.

In conclusion, they recommended lowering the primary spillway sill level by 23 centimetres to a level of 185.60 m AOD and raising of the left wall by up to 1 metre and provision of gabion mattresses on the embankment surface.

They also made a recommendation that the spillway be regularly inspected "after considerable flows" to check for seepages or possible slope erosion but they do not say why they felt seepages should be checked for after considerable flows.

5. Legislative Framework

The UK reservoir safety system is built around legislation that has developed with time, usually in a reactive way after failures, and one based on the skill and expertise of individual engineers – some of which are called 'Panel Engineers' – engineers appointed to specific Panels (viz lists) by the Secretary of State, who can be contracted by reservoir undertakers to perform the roles as set out in the legislation.

Legislation has been introduced and has been modified over the years as one way of trying to improve reservoir safety in the UK.

The legislative framework has been developed via:

| | | |
|---|---|--|
| The Waterworks Clauses Act 1863 | - | after the failure of Bilberry in 1862. |
| The Reservoirs (Safety Provisions) Act 1930 | - | after the failures at Dolgarrog and Skelmorlie in 1925. |
| The Reservoirs Act 1975 | - | after the failure at Malpasset, Vajjont, Warmwithens and incidents at Lluest Wen and Balderhead. |
| The Water Act 2003 | - | when it was concluded that the enforcement system was variable in quality. |
| The Flood and Water Management Act 2010 | - | after the Pitt report. |

The “panel system” currently consists of four Panels:

| | | |
|--------------------------------|-----|--|
| All Reservoir Panel | AR | |
| Non-impounding Reservoir Panel | NIR | Engineers can undertake the role of IE, Construction Engineer, Qualified Civil Engineer, Referee roles as well as the role of a Supervising Engineer (SE). |
| Service Reservoirs Panel | SR | |
| Supervising Engineer Panel | SE | SE role is to identify changes which could indicate the start of a failure mode. |

Various engineers and supporting staff, reservoir keepers, technicians etc, are involved in the process of trying to keep dams safe for the owner of the reservoir; however, the owner has the ultimate and total responsibility for the safety of the structure.

Statutory Inspections

These ‘periodic inspections’ (at least once every ten years) are carried out by an ARPE, who inspects the dam, looks at its performance, looks at how it is being monitored and maintained, witnesses the operation of valves etc. The IE can ask for investigations to be carried out or for actions to be taken such as an immediate lowering of water level. That engineer writes a report outlining the condition of the dam and can make recommendations in the interests of safety and maintenance which are ‘enforceable’. These recommendations must be carried out within a specified timescale by the undertaker. That engineer can also make recommendations for maintenance elements, monitoring and surveillance issues and record keeping and set the time to the next inspection. The next inspection will be carried out within ten years, or less if they are worried about the condition/ operation of the dam. They can instruct immediate drawdown if any aspect of the reservoir condition is a possible cause of immediate concern.

The contents of the report are stipulated in Schedule 5 of SI 2013 No. 1677⁽⁵⁾. Recommendations under Section 10(3)b of the Reservoirs Act 1975 as to maintenance of the reservoir relate to maintenance works, which if not undertaken could lead to deterioration of the reservoir to such an extent as to impair its safety.

The Reservoirs Act 1975 states that where an IE includes in his/her report any recommendation as to measures in the interests of safety (MITIOS) then “*the undertakers shall, within the period specified in the report, carry the recommendation into effect*”.

In my opinion, the date set by the IE is an indication of how serious the issue is. In effect if he/she makes a recommendation as to a MITIOS the IE is in effect saying, “if you don’t do this work within this period then the risk of possible failure could become unacceptably high”.

Supervising Engineer’s Examinations

The SE is responsible “*at all times*”⁽²⁾ in advising the owner on the condition/behaviour of the dam in any aspect which might affect its safety.

He/she is a Panel Engineer – a member of the Supervising Engineer Panel – a trained individual who should advise the owner of any change to the operation, performance or condition of the dam which might affect safety. He/she is likely to visit a Category A dam

(a dam where a failure would cause loss of life in a community (>10 people)) normally twice a year, as suggested by the IE, although the Reservoirs Act 1975 stipulates once a year minimum. In an annual statement to the owner, which is copied to the Regulator (the Environment Agency in England) he/she can give advice to the owner on maintenance and surveillance and as a result of changes to the Reservoirs Act 1975 by the Flood and Water Management Act 2010, he/she can now direct things to be done to assist him in his duties. However, the main power he/she has lies in the ability to call for a Section 10 Inspection earlier than planned if he/she is worried about something which is developing, or indeed when actions advised are being ignored or insufficient progress is being made. These engineers can be full-time employees or can be visiting consultants. In the case of Toddbrook this has always been a British Waterways/C&RT employee.

Owner's duties and responsibilities

An owner/undertaker of a dam has a duty to maintain its dam and its appurtenant structures to ensure it remains in a safe and operational state. This will include things such as grass cutting, removal of weeds, saplings and trees, operation of valves and penstocks, repairs to wave walls, to upstream protection systems (pitching), repairs to spillways and bywashes etc.

Good practice with respect to dam safety includes having trained staff visit the sites several times a week and after 'extreme' events. Currently best practice would dictate visits every 48 hours by trained staff for a dam with high consequence of failure predicted. Many companies have trained staff visit their sites twice a week.

The Author believes from his knowledge that most water companies' budget of the order of £15k per reservoir/per annum for maintenance works but this is a very general figure and larger dams and those with problems associated with poor design, vandalism, particular forms of construction etc. can need greater levels of expenditure per year.

6. Summary of Inspecting Engineers' Reports for Toddbrook Reservoir 1965-2019

6.1 1965

The Panel 1 engineer in 1965 reported⁽⁶⁾ on the flood of the 12th December 1964 which occurred at 5 pm when the spilling water was a few inches above spilling over the crest (of the original main spillway). He described that by 8 am on the 13th December the head over the weir was 3 ft 4 inches (1 metre) and it remained at that level for approximately 24 hours taking another two days to fall back to spillway crest level.

The outflow was estimated as 700 cusecs (19.80 cumecs) and damage was caused to the lower part of the spillway. He did question what would happen if a catastrophic flood (2 x Normal Maximum Flood - about 6000 cusecs) was experienced at the site – but felt that any damage in the area of the spillway would not directly threaten the safety of the dam.

6.2 1966

The Panel 1 engineer reported⁽⁷⁾ in a letter he wrote on the 9th December 1966, that the owner had undertaken to prepare a scheme and invite tenders for a new emergency spillway near the centre of the dam. A 250 ft long spillway with a crest level of 612.35 ft (186.644 metres) AOD over the top of the dam was proposed.

In his report of 28th September 1970, he reported that the damage noted from the report from the last inspection had been repaired satisfactorily.

He notes *“The new emergency spillway has been constructed in accordance with the drawings proposed by the Owner and approved by me”*.

6.3 1972

The Panel 1 engineer inspected and reported⁽⁸⁾ in October 1972. He inspected the dam on three occasions to write his report, all of which were at a time when the water level was well below top water level.

A new bridge had been constructed over the ‘new’ spillway in 1970. He stated that *“The overflow weir and emergency spillway are adequate to deal with the severe flood likely to be experienced”*.

6.4 Rofe, Kennard and Lapworth, 1985

The Section 2 Report⁽⁹⁾ of 1985 mentions *“ice at the northern end of the spillway”* (left-hand side) – which he puts down to wave ‘slop’ *“at the area adjacent to the wall where it is slightly lower than in the middle of the spillway”*. The IE goes on to say that there did not appear to be any evidence of leakage through or around the spillway at this point. The reservoir was ‘nearly full’ at the time.

[With current knowledge of the spillway construction, after the recent incident, I believe it is far more likely that the ice was actually caused by water passing under the slab and then exiting on the downstream face through joint.]

In his report the IE does predict “*flooding taking place at the confluence with the side spilling and river channels due to the rather inept arrangement here*”. He identifies a problem but considers that it “*would not cause any damage to the embankment*” and as a result makes no recommendations.

He also strangely makes no report on the condition of the spillway or its joints and so it is not possible to comment on the condition of the spillway at that time.

6.5 1996

In 1996⁽¹⁰⁾ the Panel 1 engineer examined the secondary spillway and stated, “*the emergency spillway showed no sign of seepage or movement at the crest*”. However, he went on to say that “*at the foot of the emergency spillway, there were signs of movement in the central section between the vertical wall and the triangular chamfered section*”. “*Only one relief discharge pipe was working. I consider that the relief pipes should be cleared to relieve any uplift pressures and the cracks infilled and sealed*”.

The same report discusses the results of settlement points on the crest and base of the spillway chute. The results indicate some settlement of 5 mm in 10 years [*I think the dates quoted are incorrect*] and some ‘uplift’ in the 80’s and the return to level in later years.

6.6 Mott MacDonald, 2006

The IE undertook the inspection in February 2005 and reported⁽¹¹⁾ in January 2006. He reported that he carried out a “*full walkover survey of the auxiliary overflow*”. He states that the “*structure had an appearance of neglect and lack of routine maintenance*”. He goes on to say:

“the overall appearance and the condition of the channel was poor and from a simple asset management point of view it is recommended that action be taken to refurbish the works. Measures should include the removal of silt and debris from the channel floor, removal of vegetation from slab joints, resealing of joints as and where necessary, reinstatement of drainage holes through side walls and isolated patch repairs to concrete”.

He then goes on to say:

- *“weep flow was observed beneath the sill on the left-hand side.*
- *there is evidence of water pressure beneath the weir slab.*
- *trees are growing through the joints... the joints need clearing out and patch repairs made to the sealant.*
- *there is overhanging vegetation on the right-hand side of the spillway channel.*
- *sand and vegetation have collected at the downstream right-hand side which led to a blockage of the drain hole beneath the end sill”.*

He goes on to describe that the SE of the time reported that “*there is evidence that the hydraulic performance of the auxiliary overflow channel is not entirely satisfactory*”.

He quotes the flood of March 1998 when the reservoir level rose to about 0.5 metres above top water level, equivalent to 0.24 metres above the sill of the auxiliary weir. “*Under these conditions there was a considerable flow down the auxiliary channel, and it was observed that there was water and spray passing over the lower part of the left-hand wall for a period of about six hours*”. He stated that the channel tapers as it passes over the

downstream face and that this situation is affected by the alignment of the left wall that is skew to the direction of flow.

He felt that this showed that the wall alignment would adversely affect the hydraulics of the channel and may result in cross wave generation that would cause/exacerbate the wall overtopping. Under a more serious storm than that which occurred in 1998 he felt that this could cause erosion of the downstream face and undermining of the channel. He considered this situation to be unacceptable and recommended that the *“potential problem be investigated further with a view to removing the risk”*. He indicated that possible solutions might involve a localised realignment of the lower section of the wall in order to prevent overtopping, or protection to the downstream face on the outside of the wall so as to prevent erosion should overtopping occur. He considered the matter ought to be addressed in the interests of safety – i.e. an issue which he considered could progress to a point where failure of the dam could occur. Within that recommendation he added the sentence *“cleaning out, reinstatement and localised repairs are also required to other parts of the auxiliary overflow”*.

This recommendation led to a review of the spillway by Jacobs in 2008 (Section 4.1 above).

6.7 Mott MacDonald, 2010

The IE report of March 2010⁽¹²⁾ notes that in 2008 the lower left-hand wall of the auxiliary spillway was raised to fully contain the PMF within the channel. He certified the works as complete.

The water level at the time of the inspection was 12 metres below top water level and the IE did a *“full walkover survey of the auxiliary spillway”* covering both the inside of the channel and the embankment areas behind the side/retaining walls.

The IE stated that in 2009 the structure did not have the appearance of neglect and a lack of routine maintenance witnessed in 2006. Apparently, the joints had *“recently been cleared out and the sealant replaced”*. Also, there were signs of patch repairs on the face although they apparently predated the *“most recent remedial works”*. The concrete was seen to be ‘uniform’ with no signs of cracking.

They state that the alignment of the new sealant along the downstream end sill *“clearly indicates that the slabs appear to have accommodated the movement without undue distress, any future movement should be monitored but I do not consider the current situation to be a cause of undue concern”*.

There appears to be no consideration of why the movement has taken place and no recommendation or means of monitoring.

Later in the report it was noted that *“small puddles of standing water were against the spillway end sill [...] the source of the water was not immediately evident and likely to indicate some leakage of groundwater into the spillway [...] such leakage will help to relieve any water pressure under the slabs”*.

In receiving the crest levelling results of the auxiliary spillway weir, the IE points out the level pins were lost in 2008 when the remedial works were undertaken – new pins S01 to S09 were installed in 2009. Looking at settlement of the crest of the dam and the auxiliary

spillway crest he noted the most significant changes in level were either side of the auxiliary spillway.

The IE comments that displaced pitching either side of the auxiliary spillway abutments be repaired. He notes the only significant risk outstanding to the dam is that of the draw-off pipes where there is no upstream control.

6.8 Mott MacDonald, 2019

The IE reported in April 2019 after an inspection on the 14th November 2018 when the water level was 4 metres below top water level.

The IE uses much of the information from earlier reports as one might expect from any IE including detailing of the 1964 flood, the construction of a new auxiliary spillway in 1970 and the raising of the lower left-hand wall of the auxiliary spillway in 2008.

A description of the auxiliary spillway and a plan is given in Section 9.6 of his report. The report suggests a 'full walkover' of the secondary overflow channel was carried out by the IE.

The IE found the relief wells in the lower end of the secondary overflow channel base slab to be blocked off or choked with vegetation.

He recommended that "*the relief wells needed clearing out and reinstating with some form of hood to prevent overflow water from entering in the reverse direction - additional relief points are also recommended*". He does not state why additional relief points are needed, how many are needed, where they are required, or indeed any evidence of substandard performance of the spillway which led to his recommendation. However, he clearly understood the need for pressure relief.

Again, he noted that the drainage to the end of the right-hand end of the secondary overflow channel was blocked.

In the commentary on the auxiliary spillway it was noted that the inspection was made from inside the lower end of the channel looking up and from the crest weir channel at the top end looking down.

In the report the IE states that the weir structure was 'level and straight'. He noted "*All of the visible elements of the reinforced concrete look in good condition being both straight and true with no signs of significant deterioration or undue movement*". He noted "*the exception being the slabs which have cracked at the lower end of the spillway channel, near the change in gradient and particularly on the crest overflow weir slab*".

He recommended that "*these cracks need cleaning out and fully grouted and sealing to prevent water getting under the slab and pressuring the underside*".

He noted 'most' joints sealants were seen to be intact but supporting vegetation growth and recommended "*all joints should be cleaned off, carefully inspected and repaired as necessary and any missing joints should be replaced*". I believe he meant jointing material and sealant.

He noted that on the crest, the crest footbridge column bases had not been spaced to

match the channel base slabs and span the slab joints. He considered it likely that the slab joint water bars have been damaged, and the waterproofing integrity lost in this area. Maintaining the sealant in good condition in this area was therefore considered to be important by the IE.

The IE noted that the weep holes in the end wall and pressure relief holes in the base slab appeared to be blocked. He recognised that given the thin, lightly reinforced base slabs and lack of any kind of underdrainage this did not give much protection against uplift forces and may explain some of the cracking. He noted *“it is therefore important that the recommendations in [his] Section 10.3.4 are carried out and all joints and cracks are well sealed particularly those through the crest”*.

Unlike any other IEs reported in this document, he recognised a major failure mode by identifying that the record drawings do not show any form of cut-off into the clay core as the channel passes through the crest of the embankment.

There was evidence from satellite photographs, taken when the reservoir was full but without spill over the auxiliary spillway obtained by the IE, that water was passing under the weir and re-appearing further down the channel. He felt that this may be due to the aforementioned cracking in the slabs, but it may also be due to water passing under the slab due to the lack of a cut-off. As the underside of the slab is below top water level (by 300 mm) water could be permanently tracking on the underside. (It has now been shown that the underside of the slab may be only 10mm or so above the main weir crest level). This, combined with any water passing through the cracked base slabs, will be pressurising the underside of the channel as it passes down the face of the embankment. He therefore recommended *“in conjunction with the crack repairs, investigations should be undertaken to prove that water is not passing under the weir when either at or just above TWL and some form of cut-off provided”*.

The IE noted on the secondary overflow that the following features were added:

- *transverse stone drain under the base slab behind the spillway end wall.*
- *outlet slot in spillway end wall.*
- *well point on right-hand side (and later one fitted on left-hand side).*
- *pressure relief drains in the floor slab approaching the end wall (added later).*

During the visit by the IE, the SE apparently cleared some of the weep holes in the secondary overflow end wall, but the pressure relief pipe and outlet slot were blocked with vegetation and debris. The well point pipes were capped off. The SE did not describe the inadequate design features associated with the spillway to the IE.

The IE noted that the instrumentation records were not continuous and that they had a five-year gap (2001-2007) when managed by British Waterways. Readings taken in 2013 and 2015 were not plotted. The latest survey was taken 23rd May 2018.

The IE noted that despite the embankment height of 23.8 metres and age, the embankment generally showed a relatively small amount of settlement over the period. The exceptions being pins No SP15 and SP16 which are located at the sides of the secondary overflow channel and pins Nos SP4 and SP12.

The IE thought that the movement in pins No SP15 and SP16 could be explained by their location, next to the secondary overflow, and were therefore likely to be sat on the construction backfill. He doesn't explain the movement of SP4 and SP12.

The IE felt that the supervision that is provided by the undertakers "*is of a high standard*" and includes regular surveillance and inspection visits which are carried out by "*suitably experienced staff of the Canal & River Trust*".

The SE visits the site at least twice a year and the local operations team on a twice weekly basis. There are procedures in place whereby the local operations staff are required to inform the Central Reservoir Safety Unit and the SE if there is any indication of unusual or unexplained conditions. However, he does say that "*it is clear that some maintenance is not up to best practice*". Clearly, maintenance is an essential part of dam safety management for any structure and must be provided by every owner at appropriate intervals – unfortunately the SE did not pursue this issue at times when he felt it was insufficient.

The IE made the following recommendations as to MITIOS:

- (i) a) *carry out a review of the secondary overflow channel to demonstrate it is not at risk of hydro-dynamic damage during significant overflow events caused by high velocity flow in the channel or water pressure beneath the base slabs and*
b) *carry out any necessary improvement works – **this recommendation to be carried out within 18 months from the date of this report and b) any 'follow-on' work completed within the timescale set by the QCE.***
- (ii) a) *carry out an investigation in to the effectiveness of any 'cut-off' arrangements around the secondary overflow channel as it passes through the embankment crest, in eliminating the risk of water tracking under the slab and pressuring the underside and b) carry out any necessary improvement works to eliminate this risk – **this recommendation to be carried out within 18 months from the date of this report and b) any 'follow-on' work completed within the timescale set by the QCE.***

The IE also recommended a system of toe drainage be added to the base of the embankment and any outfalls fitted with V-notches so that any flow increases could be picked up during the weekly site checks.

Unfortunately, he failed to repeat the recommendations to put more relief wells into the slab, (although the IE may have reinforced this need in future after the review which he had recommended).

The IE recommended that "*all existing joint sealants in the secondary overflow channel 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 two-yearly basis and undertaken if required***". Unfortunately he didn't specify when the works should be done. However, I believe that he meant for the works to be done as soon as reasonably possible and reviewed thereafter in two years' time, but this may not be clear to the owner.

He also recommended “*all cracking in the concrete in the channel base of the secondary overflow to be injected and sealed to full depth to prevent spill water passing through to the underside of the base slab and pressuring the underside. **Further maintenance to be reviewed on a two-yearly basis and undertaken if required***”. Again, he didn’t specify when the works should be done.

I can see from the way in which this was worded that an owner may not understand the importance of the issue.

The IE stated that the SE should take special note of the condition of the joint sealants and crack repairs on the secondary spillway, ensure that the pressure relief pipes on the secondary overflow base slab are in full working order and are not blocked, and the correct functioning of any crest cut-off under the secondary overflow channel sill when the reservoir is at TWL or just above (but below the level of the secondary overflow sill) by observation of leaking joint sealants and discharging pressure relief pipes.

He also recommended that new pins be installed on the secondary spillway channel and that they should be monitored at a frequency of not less than once every 12 months (but initially six months for a period of two years) and on crest pin SP04 monitored at a frequency of not less than six months (for the next two years).

6.9 Commentary on IE’s Reports 1985-2019

In the 1985 report on the comment on ‘wave slop’, and now with the knowledge of the spillway construction and the recent incident, I believe it is far more likely that the ice was actually caused by water passing under the slab and then exiting on the downstream face through joints.

In the report of 1996, again, with the benefit of hindsight, from the evidence presented, it could be said that consideration of a failure mode at the auxiliary spillway due to uplift should have been considered at this time.

The failure mode recently experienced seems not to have been considered.

In the report of 2006, it appears the IE did not identify the issues associated with the basic design and construction of the slab or the possibility of a failure of the structure.

It also appears that the Quantitative Risk Assessment (QRA) carried out as part of the inspection completely missed the failure mode which has been experienced recently.

In the Mott MacDonald IE’s report of 2010 it appears that again a potential failure mode had been missed.

In Mott MacDonald’s report of 2019 the IE noted that the drainage to the end of the right-hand side of the secondary overflow channel was blocked, and also identified and had a good understanding of the potential problem; but as we will see he did not give this issue, in my opinion, sufficient importance.

The IE observed the pressure relief valves to be blocked. In my opinion it is an important maintenance task to keep pressure relief valves open – and one which I believe the IEs and SEs should comment upon in their reports and statements and get

operational/maintenance staff to carry out.

The IE identified that instrumentation readings were not plotted and analysed. This is disappointing as it is important that instrumentation is read and reviewed in a timely manner to ensure the adequate performance of any dam.

Clearly where there is instrumentation at a dam it should be read at intervals as directed, analysed as soon as possible after the data is received, and any actions necessary taken; however, there is no instrumentation associated with the auxiliary spillway other than level pins.

The IE makes recommendations as to MITIOS, but I believe the timescale associated with the second recommendation does not give enough importance to the issues - which was probably influenced by what all previous IEs had considered to be satisfactory performance over the preceding 48 years. However, it is clear that the failure mechanism experienced could have been happening for many years. If the seriousness of this failure had been recognised, he could have ordered an immediate investigation coupled with an immediate drawdown and water level restriction.

If the recommendation to install more relief holes had been stated by the IE with a clear and short timescale for the work to be done then voids may have been discovered.

If the possibility of the presence of water paths and voids under the slab had been recognised, a number of studies could have been actioned (e.g. GPR surveys, Willowstick surveys, intrusive investigations).

7. Summary of Supervising Engineers' Statements

The C&RT (and British Waterways pre-2012) has employed SEs with responsibility for Toddbrook. The current SE has been responsible for the reservoir since 2015 (and was also the SE for a short period in 2005/6). He has responsibility for 25 reservoirs and has many year of experience as an SE. Prior to this, the previous SE was in post for the period 2009-2015, giving a degree of continuity in the role.

I have reviewed a number of the statements made by SEs going back to 2000 and set out some extracts below.

February 2000 (Water Level at -0.2 m)

Overflows and spillways - Structural condition - Emergency overflow
Joints and cracks in the top apron and upper sloping slabs have been filled. There was an increasing flow down the left-hand edge (to a trickle at the bottom of the slope) which seemed to start somewhere between the first and second joints down from the top apron (need a dry day to check properly).

February (-0.4 m) and May 2001 (+0.05 m)

The following matters are highlighted for attention: outstanding Inspecting Engineer's recommendations:

Fill and seal cracks in emergency spillway them and clear the pressure relief pipes in the toe wall (the reservoir attendant reported that during a recent flood when water overflowed and emergency spillweir, water was seen spurting from beneath the spillway slab through small holes in the side wall footings, indicating that water can get under the slabs and cause uplift pressures to build up beneath it).

December 2004 (+0.01m)

Overflows and spillways - Structural condition - Emergency overflow

- *water was trickling through one pressure relief hole in the end sill, and trickling, on the spillway surface, immediately upstream of the end sill from about halfway along it; water was seeping out of cracks and joints in the spillway concrete.*

January 2006 (TWL)

Overflows and spillways - Structural condition

- *Auxiliary spillway: Ochreous water was running through the weep holes in the spillway channel below the emergency spillway and most noticeably from the weephole 3m to the LHS of the spillway (clear) – feeder; water was seeping through cracks and joints at the downstream end of the auxiliary spillway and through the invert of the spillway.*

September 2007 (-2.75 m)

Overflows and spillways - Structural condition

- *Seepage from lower joint of auxiliary weir starts at joint between piers 5 & 6 (numbered from L) of footbridge.*

March 2010 (-0.02 m)

Overflows and spillways - Structural condition

- *Auxiliary spillweir – seepages and small clear ‘spring’ observed at toe level. Ochreous staining from lower end of one of the centre longitudinal joints. Isolated short lengths of sealant have been lifted at the toe area. Low flows/seepage from weep holes of channel side of toe upstand.*

August 2010 (-10.7m)

- *seepages and small clear ‘spring’ observed at toe level.*
- *Ochreous staining from lower end of one of the centre longitudinal joints.*

March 2011 (TWL)

- *Seepage from joints toward right side – at first joint (3 – 4m from right wall) at third bay down.*

February 2014 (+0.1m)

- *Damp patches adjacent to left side wall of auxiliary spillway at fourth concrete wall bay from top.*

October 2014

- *grass vegetation was establishing within the sealed joints*

October 2015

- *slight leakage was noted under the weir crest adjacent to the right-hand side of the 5th pier from the right-hand end and adjacent to the left-hand side of the 9th pier*
- *leakage noted under the weir crest end had frozen on the face*
- *a spring at the base of the auxiliary spillway from a joint, similar seepage elsewhere*

January 2016 (+0.145 m)

- *leakage from under the weir crest had frozen on the face*
- *slight leakage under the weir crest adjacent to the right-hand side of the 5th pier from the right-hand end*
- *slight leakage under the weir crest between the 4th and 5th piers from the right-hand end*
- *slight leakage under the weir crest adjacent to the left-hand side of the 9th pier from the right-hand end*
- *leakage under the weir crest between the 9th and 10th piers from the right-hand end*
- *ochreous seepage from joints near the middle of the apron at the base*

December 2016 (+0.085 m)

- *leakage under the weir crest as noted above*
- *slight ochreous seepage from a number of joints and cracks near to the base (as noted 25/06/15, 15/10/15, 10/06/16)*
- *a trickle of water at the base*
- *the weepholes at the base of the auxiliary spillway were mostly dry, though two, 3 m below the little weir, had ochreous trickles; water was running from a several weephole around 4 m upstream from the little weir; the weepholes in the channel above the auxiliary weir were dry*

July 2017 (-2.63 m)

- *just upstream from the little weir in the bypass channel, ochreous and oily seepage from a joint toward the base*
- *3 m to the left of the little weir in the bypass channel, ochreous and oily seepage from a crack*
- *a trickle of water in the channel at the base starting 2 slabs upstream from the little weir in the bypass channel*
- *in the raised section at the base, where a spring is sometimes seen, the area was damp*
- *the weep holes in the vicinity of the standard pipe in the auxiliary weir were running, six in total*
- *several of the weep holes above the little weir were running*
- *ochreous seepage from two of the weep holes below the little weir*

January 2018 (+0.01 m)

- *slight ochreous seepage from a number of joints and cracks near to the base*
- *just the left of the little weir in the bypass channel, ochreous and oily seepage from cracks and joints*
- *leakage under the weir crest as noted before*
- *saplings were growing on the spillway*

June 2018

- *most of the saplings (in the auxiliary spillway joints and cracks and reseal) have been cut down; the sealant remains to be repaired. The pressure relief holes need to be drilled out and filled with gravel*
- *saplings have been cleared from the apron: a few small ones are starting to regrow.*

April 2019 (+0.07 m)*Downstream Apron*

- *water was flowing from the joint at the base of the spillway 3 m above the little weir (as usual when the reservoir is full)*
- *the seepage under the weir crest was running down the left-hand side wall and was disappearing into a crack in the flatter section towards the base*
- *seepage from a several of the joints and cracks at the base of the spillway*
- *water trickling from the slot at the base of the weir*

8. Investigations

8.1 Investigations

A series of cored holes and trial pits were dug under my direction and witnessed during a site visit on the 13th September 2019. Subsequently I asked for some additional holes to be cored in the crest weir where there had been 'large' takes of grout injected into the crest slab upstream of the weir once the water level had been lowered.

In some ways unfortunately, but necessary at the instruction of the IE, a grouting exercise, presumably to fill voids under the crest slab, was undertaken before my investigations.

I also asked for additional holes to be excavated to prove the wall foundation on the left-hand side and the slab/wall connection.

A detailed examination of the whole of the spillway slab and walls was undertaken by me on a rope and harness system with the assistance of a rope access team – the same team undertaking the core drilling and trial hole excavation exercise.

8.2 Findings

In general, the findings of the survey found the construction to be as detailed on the drawings. I looked at the following drawings:

| Drg No. | Date | Title |
|----------------------|------------------|---|
| | | Toddbrook Reservoir – Details of New Overflow Weir and Spillway Details |
| 1060B | Jan 1968 | Toddbrook Reservoir, Proposed Spillway – Details of Spillway |
| BO577900/WIP/TOD/002 | 2008 (Jacobs) | Remedial Measures of Auxiliary Spillway Plan, Sections and Details. |

The intrusive sampling via cored holes and trial holes directed by myself in association with the last IE showed the concrete to be of what appeared to be good quality and well compacted. The thickness of concrete appeared, surprisingly (considering the steepness of the slope and the construction equipment available at the time), to be uniform up and down the slope, and across a number of panels.



Excavation on the slope



Excavation on the slope



Water bar at joint



Excavation showing dowels and water bar

The joint detail, the water bar, the transverse ribs, reinforcing steel and dowel bars were all found to be as per the drawings. The reinforcing steel was found to be very small – of the order of 3mm in diameter.



Small diameter reinforcing bar

Relief wells were noted in some of the lower slabs. These seem to have been drilled sometime after construction of the spillway. However, these relief wells were seen to be obstructed either by root systems of saplings/coarse vegetation or by stones/ gravel and clearly had not been cleaned for many months/years.



Blocked relief wells



Blocked relief wells



Blocked relief wells



Variation of slope across spillway

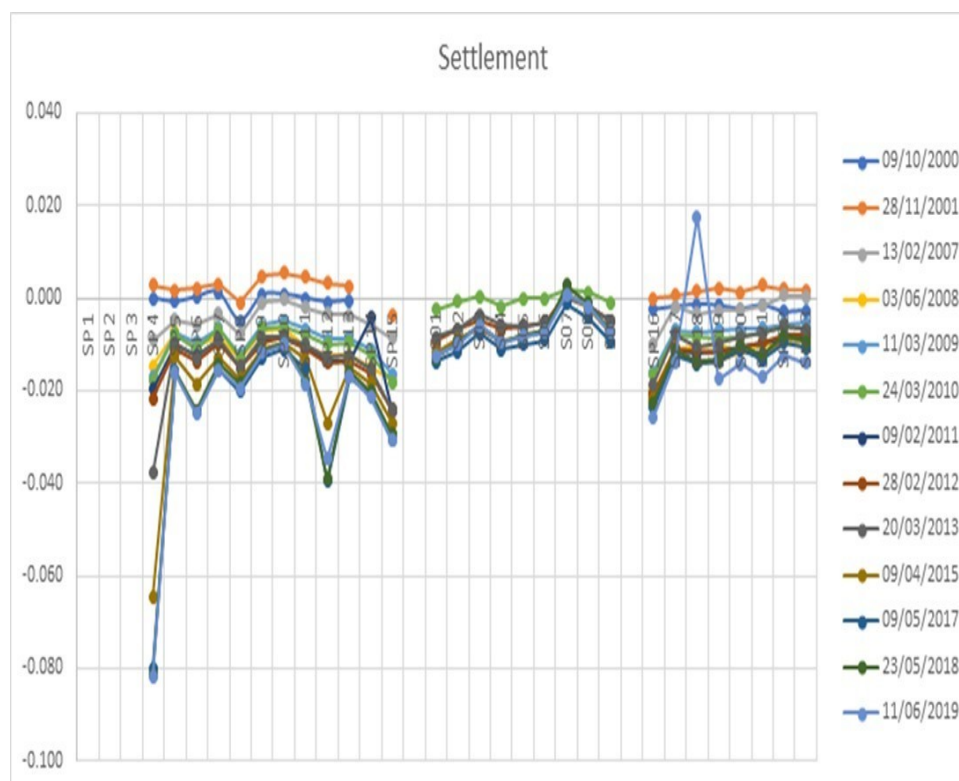
The line and level of the slope, outside the area of damage was seen to vary across the slope and particularly in the lower part with the most uniform slope adjacent to the right-hand wall. The variation of slope in the lower part was probably as a result of relaxation of the construction tolerances to accommodate the merging of the spillway channel with the main overflow channel but some may have been due to movement of the slabs over time.

Thus, in general, it can be stated that the construction was as designed but flaws exist, and the quality and quantity of maintenance has been variable.

9. Performance

9.1 Performance from instrumentation

Level pins had been installed on the weir and levelled over the years. The results of the levelling exercise over a period of 19 years shows a settlement profile which confirms the left-hand end of the weir and slab to be some 12 – 13 mm lower than the right-hand end. The points in the middle of the plot below (S01 to S09) are on the auxiliary spillway and show the left-hand end to have settled more than the right-hand end.



Plot of level surveys

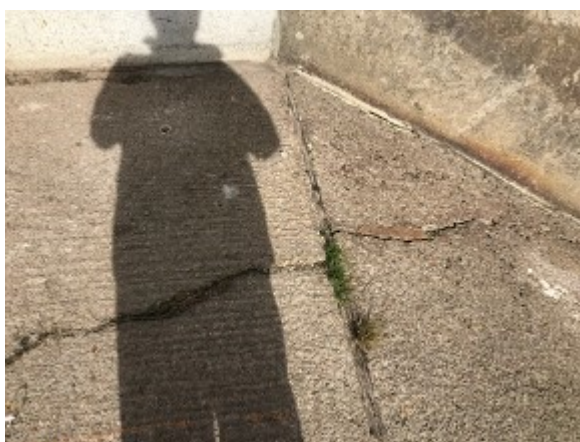
I had assumed that the concrete slab carrying the weir was constructed to a level of 185.95 metres AOD (or perhaps 100mm lower) and over a period of some 49 years there has been uneven settlement of the crest – the most being at the left-hand end. This would mean the underside of the slab would be at a level of 185.800 metres AOD, or only 150 mm above top water level. Recent investigations have found that the slab was built to a lower level, about 100 mm lower, in which case the underside of the slab could be some 300 mm below water level measured at the main weir. Inspection of the area showed there to be cracking throughout the area of the auxiliary spillway slab but with more severe cracking at the left-hand end and adjacent to the left-hand wall of the spillway. These cracks are shown developing with time over successive IEs' reports - where photographs were provided (unfortunately few reports had photographs).



Crack to spillway



Crack to spillway



Crack to spillway



Crack to spillway

There are a number of other, more easily explained, areas of cracking across the structure. Some are due to shrinkage, some due to imposed loads from the bridge and some may be due to the day to day movement of the embankment in operation under different load conditions.

Generally, the settlement profile of the crest of an old embankment dam would be expected to be in the range of 1-2 mm a year. Thus, since 1970 the settlement of the embankment below the slab would be expected to be in the range of 49-98mm.

Hence, it would be expected that a 'gap' would open up between the underside of the slab unless the slab was built to follow the settlement of the structure or be designed with the settlement in mind; which clearly it was not.

Inspection of a cored hole on the upstream side of the slab near to the left-hand wall of the auxiliary spillway during my visit on the 13th September identified a gap amounting to some 50 mm between the embankment fill and the underside of the slab – enough to be able to put my hand into the gap.

As part of the 'stabilisation works' following the incident the contractor was instructed, by the ARPE, to grout the underside of the slab via three rows of grout holes. It is not known what pressures were used but grout was introduced fairly uniformly over the whole of the crest area. Subsequent additional holes were drilled to prove and try to quantify the size of the 'gaps'/voids beneath the slab.

Analysis of the data is given below.

Records of the grout takes were provided to me which showed nine holes were drilled into each of the 19 slabs across the weir of the auxiliary spillway.

The slabs varied in size, but the grout takes were fairly uniform across the slabs.

The "remedial works" directed did not seek it appears to provide a cut-off for some reason, when this could have been achieved quite easily.

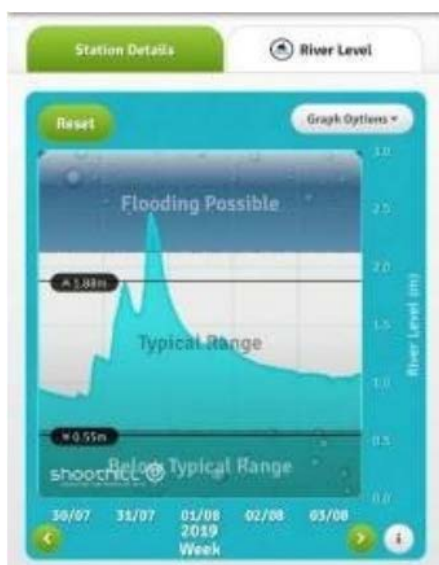
| Bay | Size (m x m) | Area (m ²) | Grout take litres | Gap under slab (mm) (assuming uniform) |
|-----------------|--------------|------------------------|-------------------|--|
| 1 (failure end) | 3.5 x 3.2 | 11.2 | 20 | 2 |
| 2 | 3.2 x 3.25 | 10.4 | 22 | 2.1 |
| 3 | 3.8 x 3.22 | 12.24 | 25 | 2.04 |
| 4 | 1.9 x 3.24 | 6.15 | 13.5 | 2.19 |
| 5 | 1.5 x 3.24 | 4.86 | 11 | 2.26 |
| 6 | 3.9 x 3.12 | 12.17 | 24 | 1.97 |
| 7 | 3.5 x 3.24 | 11.34 | 29 | 2.55 |
| 8 | 4.2 x 3.25 | 13.65 | 27 | 1.97 |
| 9 | 3.1 x 3.2 | 9.92 | 30 | 3.02 |
| 10 | 3.7 x 3.25 | 12.02 | 27 | 2.25 |
| 11 | 3.7 x 3.2 | 11.84 | 30.5 | 2.57 |
| 12 | 3.9 x 3.16 | 12.32 | 33 | 2.67 |
| 13 | 3.8 x 3.17 | 12.04 | 30 | 2.49 |
| 14 | 4.1 x 3.16 | 12.96 | 33 | 2.54 |
| 15 | 3.5 x 3.17 | 11.09 | 27 | 2.43 |
| 16 | 3.7 x 3.16 | 11.69 | 30 | 2.54 |
| 17 | 3.7 x 3.17 | 11.72 | 27 | 2.30 |
| 18 | 3.4 x 3.2 | 10.88 | 27 | 2.48 |
| 19 | 4.0 x 3.16 | 12.64 | 51 | 4.03 |

This suggests a smaller gap than I had anticipated at the left-hand end, but it should be remembered that the slab at this end had dropped some 13 mm in the 18 years and thus could have opened up a significantly bigger gap over 49 years and the settlement may have closed it. However, as part of the investigation, I was able to put my hand into a borehole drilled through the weir slab onto the core and put my fingers into a gap estimated to be 40 – 50 mm in depth. Thus, the grouting does not seem to have been totally effective and the exercise left evidence of the voids under the weir slab.

10. Incident on 1st August 2019

10.1 Hydrological event

There is limited hydrological and hydraulic information available but based on data from level records and a gauging station just downstream it appears that the site was subject to two hydrological events in quick succession - a 1 in 43 year event followed by a 1 in 92 year event two days later - resulting in a significant combined event.



Flood event – 31st July 2019

10.2 Sequence of events including photographic evidence

Photographic evidence and evidence from the public suggests that the unravelling of the spillway started at the base of the triangular section near the left-hand wall. This might be expected since if there is no pressure relief the pressures would be largest in this area.

Members of the public who walked across the bridge over the spillway at about 8.30 am on the 1st August 2019 noticed a vertical plume of water near the base of the spillway jetting well above the level of the slab. Apparently by the time they walked back a short time later the flow and plume were larger and was discoloured indicating removal of material under some considerable pressure. So clearly at this time there was a mechanism taking place where material was being removed by water flowing under the slab.

The evidence on site and from other sources showed that a slab at the base moved first. This may well have also contributed to water overshooting the spillway channel by deflecting the flow upwards to the left.

The mechanism described is that water gets in at the top of the chute under the weir and then flows under the slabs – some gets to the bottom where it generates enough pressure as an uplift force to lift a slab.

Photographic evidence showed that later an upper slab moved downward. Apparently cracking sounds were heard as presumably a slab broke in half. This movement may have been caused by uplift pressures being developed under the slab pushing it up into

the flow - or it could have been suction forces pulling the slab up, or it could have, and perhaps more likely, caused the slab to move down into a void and it cracked, having insufficient space.

Whichever way the slab moved, a void was then formed and within a few hours more slabs broke up and fill was washed out or more likely sucked out from below the slabs. The flow subsided after a few hours and at that point it is clear a number of slabs were spanning over a large void which existed under the slabs and extended back up towards the crest and core. Subsequently these unsupported slabs, with little reinforcement in them, collapsed into the void created, or cracked and collapsed.

Unfortunately, photographs are not available covering all of the areas all of the time but at some point, water scoured beneath the wall, where there was little, or no foundation and water started flowing overland down the downstream slope. Flow continued causing an erosion channel which appears to have been covered by, and bridged by, the turf until sufficient support to the turf was lost, forming a deep gully on the downstream face.

More information to describe the interpreted sequence of events is presented in Appendix C.

11. Root Cause Analysis

I have assembled information from the IE Reports, the SE Statements, the design drawings, records of performance of the dam and the structure and intrusive investigations into the structure since the incident and I have come to a conclusion as to the root cause of failure.

11.1 Review of design

Fundamentally the original design does not follow recognised best practice and is seriously flawed; it has features within it or that have not been provided which have led to the failure which occurred on 1st August 2019. It is likely that flows took place under the slab almost from day one, when the water level in the reservoir was high enough.

In my opinion the original design was flawed and so the situation was an incident/accident waiting to happen – it was just a question of how, when and to what degree, unless some intervention was taken in the meantime.

In summary, the flaws in the design were:

- there was no cut-off into the core/crest of the dam
- there were only water bars in the horizontal joints
- there were no water bars in the vertical joints
- there was very minimal reinforcement in the slabs
- the wall/floor slab joint was poor
- there were very few relief holes through the slab
- the wall foundations were very shallow in places
- there is no underdrainage
- the joints between slabs were poor

- there was no anchorage

The fundamental issue of concern with the design is that no cut-off was provided into the core beneath the slab. This has resulted in a situation where continued settlement of the embankment beneath the slab has led to a gap and a passage of water beneath the underside of the slab and the crest of the dam/core which has increased with time (if one accepts most embankment dams settle 1 or 2 mm per year), with water passing over the crest and core and beneath the slab. This process could erode the fill/core increasing the size of the gap. This could focus in a particular area once it had occurred – and the evidence would suggest this is at the extreme left-hand end of the spillway where the cracking is most severe. There is cracking along at least half of the crest of the spillway where the crest concrete slab has probably been left unsupported.

Other elements of the design considered to be poor practice included:

- water bars have been provided on horizontal joints but not on vertical joints, which of course allows water to flow beneath the slabs when over the crest, but also allows water to exit from below the crest, when joints are open.
- ‘plums’ which disturb the flow.
- very minimal reinforcement steel in the slabs.
- small number of connecting dowels between slabs and only on horizontal joints.
- no pressure relief to the slabs (a small number of holes were provided to some of the lower slabs at a later date).
- the wall floor slab joint was poor with little ‘embedment’ and cut-off.
- a convergent wall on the left-hand side leading to increased pressures and turbulence along the base of the wall.

The top of the weir to the auxiliary spillway was designed to be only 260 mm above the weir of the main overflow with the underside below the level of the main spillway weir level – actually it seems to have been constructed only some 164 mm above main weir level and so the base could be as much as 300 mm below main weir level. The water level records are not continuous but do provide a fairly good record for the period 1970 to 2013.

The records show that there have been 31 water levels recorded weekly when the auxiliary spillway was operating since 1970, and 424 water levels recorded weekly when the water was above the base of the slab – days when water could have been passing beneath the slab and forming erosion channels, and in some cases removing fill. Given that water levels are normally read on a weekly basis, typically a Monday, the spillway would have been operating for between 31 and 217 days with the water level at or above the base of the spillway slab for between 424 and 3,000 days since its construction.

All dams settle with time – nominal movement being 1 to 2 mm settlement per year – and thus if the dam settles beneath the weir over the years a gap will form; indeed this gap was found during the investigation, meaning that water would pass under the crest slab and then flow under the slab on the downstream face. This flow would produce an uplift under the slabs unless drained via drains or pressure relief holes and could cause removal of material over a protracted period of time.

Unfortunately, if the lowering works to the left-hand wall, recommended by Jacobs in 2008 (Section 4.1), had been carried out, they would have solved the 'problem' with no cut-off in the short-term. However, at no time, it would appear, did Jacobs investigate the construction elements of the spillway – namely that water stops were not provided in both directions, that the slabs just butted into a slot in the bottom of the wall, and the foundation stepped down the slope leaving very little cut-off into the face of the embankment in a number of places. In my opinion, they did not check the suitability of the wall for raising and the gabion reinforcement and the shaped deflectors were not provided, and the plums were not removed.

11.2 Failure mechanism

In my opinion, it is likely that erosion of fill from beneath the slabs took place over many years and there is evidence of water spouting at the base being observed over the years – as set out in 12.6 - and thus it is likely that the spouts and flow will have carried fill from beneath the slabs.

I believe evidence from site suggests that slabs were lifted by forces generated as uplift forces beneath the slab, assisted by suction forces which may have been generated in turbulent water and 'skimming' flows over the slabs. Movement of the slab and removal of material from beneath the slabs was then inevitable.

Once slabs had lifted, foundation material could be easily sucked and washed from beneath the slabs leaving them unsupported.

Photographic evidence was obtained by those on site, by members of the public, from posting on the internet and from other useful sources such as footage from the drone camera flown by the police. This has enabled the sequence of slab failure to be developed.

The first slabs to move were at the bottom and then slabs moved higher up and were left bridging over large voids. These slabs eventually collapsed into the hole/ void beneath even when there was no flow over the weir.

The relatively thin slab just butts into the wall via a small rebate fitted with a water bar. On the left-hand side the foundation of the wall steps down as the wall moves down the slope with the result that at intervals there is little to no foundation to the wall at the slab.

It appears, at one of these points, as the adjacent slab was lost and material was removed from the area, that erosion was able to take place under the wall and indeed under the adjacent grass and topsoil. Thus, we can see that some water was able to escape the channel on the left-hand side. This eroded some fill which was taken away by the flow and was discharged down the slope, but the grass/turf topsoil again bridged over the erosion feature which then subsequently collapsed.

Any spillway that is built into a dam, and certainly one that is fitted as an addition, cuts through and is effectively breaching the core and then provides a potential path for water to 'leak' through to the downstream side.

Any spillway should have a cut-off beneath the slab to ensure if the core settled that

there would be a 'connection' to block and prevent waterflow.

11.3 Evidence of water passing under the crest

It is clear from my investigations that a gap had opened under the slab and this gap was quite large. This gap would then allow water to be passed through to the downstream shoulder. Frequent inundation of the underside of that slab would allow any water passages to increase in size with time with more and more material to be removed, often unnoticed. Eyewitness reports by a local resident who often walked the crest of the dam, from a communication with the C&RT: *"I am confident that there are significant water erosion channels under some apron panels because on occasions I have seen pulses of water mixed with air, hiss and bubble and burst out of the spillway panel gaps at the LH corner where the first panels were lifted. It happens when the reservoir is high enough for strong waves to be hitting the underside of the apron with enough pumping power to provide these observable pulses on the spillway."*

This phenomenon was also described by the Panel Engineer on site on 1 August, once the flow over the spillway had stopped, and could well have been caused by wave action, whilst the level was still high, causing air and water to be forced through the voids under the crest slab.

These eye-witness reports give further evidence and strength to the argument that this was a progressive failure mode involving the passage of water under the slabs on the left-hand side for many years. In other words, voids of size unknown had been forming for many years and then the event of the 1st August caused the breakup of the slabs and allowed removal of a significant amount of material at that time. It appears that previously neither the Independent Engineers, nor the Supervising Engineers, chose to investigate the issue even though there was evidence and reports of material being removed.



**Flow from the top
of the spillway**



**Accumulated water at the base of the
spillway**

The photographs above were taken in April 2019 by the SE at a time when the reservoir water level was at +0.07m and water was just topping the primary weir, so neither spillway was spilling but a significant amount of water was definitely passing beneath the auxiliary spillway weir slab. It was noted that there is very little vegetation in the joints/cracks. The water accumulating at the bottom is considered to have passed beneath the weir (there is no overflow), come out through a joint, and collected at the

base.

In addition, good design practice would be to seal the core against the side walls – sometimes against a key and recently against a smooth wall. The concrete slab on the slope would again be sealed against infiltration into the fill but be flexible with water bars on all joints. There would be keys to prevent slippage and then there would be under drainage and relief wells to collect and then release any pressured flows under the slabs.



Shallow foundation to the left-hand wall



Cracking at extreme left end



Increased cracking at left end



Movement between spillway and wall



Movement between spillway and wall and cracking



Longitudinal crack



Longitudinal crack



Longitudinal crack



Cracking to crest

12. Failure Modes

12.1 Significance of cracking

Cracking is evident on the structure and may be considered as a potentially significant contributory factor in the failure, and it is necessary to review this.

Most spillways built from concrete crack to some extent perhaps due to shrinkage or movement, but cracks occur for other reasons – in this case probably due to loss of support to a lightly reinforced slab. Whatever the cause it is important to investigate any cracking, within a reasonable period, to understand the reason for its occurrence and to deal with the cracks if necessary.

Many other spillways built during the Victorian era for example, are built from spillways with joints, some very wide, some very tight and these are often subject to significant periods of high velocity and long periods of flow - but do not fail. Hence the presence of joints is not of itself a concern but must be reviewed by the engineers carrying out inspections/examination. Recent incidents include the one at Butterley Reservoir (which part failed due to negative pressures being formed at changes in slope) and at Naden, again due to loss of adhesion below the blocks and then the application of negative pressures to the tops of some of the slabs due to changes in elevation. However, neither of these would seem to have any direct relevance to Toddbrook because although there were cracks, open joints and other features due to changes in geometry the failure mechanism was completely different. It is possible to say that, even with cracks, spillways don't necessarily fail but there are also other reasons for failure including flow beneath the slabs, negative pressure, and uplift pressures.

Joints are installed in slabs for a number of reasons including:

- if concrete slabs are too big they crack between joints.
- joints allow articulation/movement.
- joints in some instances allow pressure relief.

12.2 Cracks and joints on the Toddbrook auxiliary spillway

There is cracking evident on the auxiliary spillway at Toddbrook albeit, in my experience, not very different from the norm on other concrete spillways that I have seen other than the concentrated cracking at the left-hand side of the crest. The cracking is much more severe on the left-hand side, particularly on the crest, and this cracking on the left side is likely to be caused by settlement. This is unavoidable and, in this position, in my opinion doesn't reflect any lack of maintenance. However, as each dam is unique, when cracking is observed then it should be investigated within a reasonable period and dealt with in an appropriate way depending on what the investigation shows.

There are longitudinal cracks right across the crest but when one considers the layout of features on the crest that crack is downstream of the core and where loading from the bridge occurs – again, it could be caused by loss of support. Any water ingress here would contribute to erosion but it is more likely to be caused by water passing under the slab given the lack of cut-off.

The horizontal joints between the concrete slabs have water bars, whilst the vertical joints did not have water bars, a serious design flaw, but they appear to have been sealed – a strange design/concept.

Over the years the joints have deteriorated and there is evidence that they have been repaired, and the auxiliary spillway has operated on a number of occasions in the past without failure. The spillway has operated with vegetation in joints and also with resealed joints, without failure.

At times vegetation has become established in the joints. This does not mean that joints have been open or indeed that the vegetation has gone through into the sub-base to the slab. Often vegetation can grow along a joint where silt/soil has accumulated, perhaps after the sealing strip has been lost, creating a shallow root system.

Clearly vegetation should not be left to accumulate on or close to any structure, particularly if it has a 'woody' root system. A maintenance regime should be set up by an owner/undertaker to remove vegetation before it does any damage.

12.3 Consideration of other modes of failure

Other potential modes of failure would include internal erosion, instability and the forming of large volumes of water through the joints/cracks into the area beneath the slabs which then fail via uplift pressures (stagnation zones).

In reviewing other modes of failure, it must be based on evidence and a recognition that significant amounts of fill - perhaps as much as 400 cubic metres - was removed at some time prior to or during the failure.

When the incident was taking place, bystanders witnessed spurts of dirty water. A video at the time also showed dirty water being discharged on the left-hand side which would indicate some movement of fill. This, coupled with the fact that water and fill was pushed under the left-hand side wall, as detailed elsewhere in this report, leads to the conclusion that some fill had been lost before the event and hence that the slabs were bridging over voids which had been created earlier.

The downstream fill is a well graded sand and gravelly material which will compact well and such that it would not settle markedly over the life of the dam; and this is demonstrated by the settlement profile obtained. However, it is the sort of fill which, if water is added to it, would flow and create channels. For the construction of the spillway the fill would have been removed down to formation level and the formation is likely to have been rolled/compacted, even by foot traffic before the slab was poured. A layer of building paper was also put down and it could be that water flowed on top of that in part. Hence, the face is likely to be quite dense. This would have discouraged water to pass down into the body of the dam if water got under the slab but water will always find a weak point to start erosion and form a water path. Certainly, there is evidence that water flows under the slab and exits through joints/ cracks.

12.4 Government Independent Review

The Review by Professor Balmforth has come to the conclusion that “*the most likely cause of the failure of the auxiliary spillway was its poor design*” – which I can agree with, but goes on to say that this was “*exacerbated by intermittent maintenance [...] which would have caused the spillway to deteriorate*”. In my experience, most owners carry out maintenance at intervals and the Review does not actually quantify the deterioration that has been cited. The spillway at the time of the incident was actually in quite a good condition with only some minor concrete repairs and perhaps some joint sealant missing and some vegetation growing in some of the joints.

The Review accepts that water has passed under the slab for a long time – I believe since construction - and that has caused erosion of fill which has led to additional settlement and cracking. Satellite data recently received – set out in 12.7 - shows settlement data indicating accelerated movement just before the incident. This supports the failure mode I have proposed, for a substantial quantity of the fill to have been removed before the event. The event on 31st July / 1st August 2019 was merely the ‘final straw’ after decades of erosion beneath the slabs – with a substantial quantity of material removed at the time of the event.

The Review proposes the process of water injection through cracks as the primary failure mode and whilst I accept some injection may have taken place, I submit there is no quantifiable evidence to support this, whereas there is evidence to support the long and protracted period of erosion beneath the slab. There seems to be no explanation as to why the process of crack injection only happened on the left-hand side. Further, it is difficult to see that the ‘slurry’ (which is suggested was formed beneath the slab) was able to be formed without an exit point.

Intermittent maintenance has been suggested as a possible primary cause of failure but the Review also suggests that the spillway was unlikely to have survived the PMF. I would say that intermittent maintenance is provided by most owners and the fundamental design flaws in the spillway were the primary cause of failure.

In short, I agree that that the auxiliary spillway was poorly designed. I would agree that maintenance was intermittent as it has been at most reservoirs in the country in recent decades. No longer do companies have the staff who do things on a daily basis, they wait until sufficient work is required to give an order for a few days’ work. I cannot agree that there was extensive plant growth and thus that this suggests “*open pathways to the embankment beneath*” – vegetation can grow along the joints which have ‘filled’ with silt – it doesn’t mean it has gone through to the embankment and there was no evidence of and it is considered highly unlikely that vegetation can breach a hard plastic water bar.

I acknowledge that there are American papers that suggest that significant amounts of water can be injected into narrow cracks through spillways and of course this issue of stagnation pressures and the ‘injection’ of large volumes of water through cracks and joints on spillways *must* be considered and allowed for in any new design and/or remedial works. I accept some water may have entered the joints during the event, but I consider high velocity flow down the slab is likely to continue down the slope rather than turn through 90 degrees. The limited number of plum stones and coarse vegetation in the damaged area would not, I believe, disturb the flow sufficiently to cause the

damage during the event – in my opinion, it had already been damaged over many years and there is evidence to support this.

So, whilst I consider this process may have played a small part, I think there is much more evidence for a progressive failure mode caused by water passing under the weir. The alternative theory doesn't seem to be able to explain why the failure didn't occur over the full width of the spillway or why the damage was confined to the left-hand side.

There is evidence over the nearly fifty years since the spillway was introduced that flow under the slab was taking place and I believe this to be the most significant contributor to the failure mode.

12.5 Failure Modes - internal erosion / hydraulic fracture

After the fill was removed by water getting under the slabs, the core was left supported by only a small amount of shoulder fill. There apparently were no signs of hydraulic fracture or leakage through the core.

There were no signs of instability in the face and so I believe the modes of failure of internal erosion and instability were not experienced at Toddbrook.

Two key issues need to be explained in the understanding of the failure mode.

The first is a significant amount of material was lost and secondly why the failure didn't happen on the right-hand side.

The joints would have to be open and allow the large amounts of fill to be released before the slabs collapsed. In fact, a small number of joints were open at the base.

12.6 Evidence of progressive failure

There are many aspects of evidence for the progressive failure that I believe to be the primary mode of failure of the spillway, which include:

- Eye-witness reports at various times over the last 50 years describing water spurting up at the base of the spillway – that water being dirty and clearly carrying material away from the spillway – but only on the left-hand side.
- Eye-witness reports describing water coming out of the joints in the upper parts of the spillway as well as dirty water, and the sound of air being expelled from beneath the slab.
- Mott MacDonald engineers on-site during the incident describe water and air pulsing through the upper part of the damaged area after the overflow had stopped – this is likely to be the effect of wave action pushing water and air into voids which connect the upstream face through to the eroded void.
- Historic descriptions of 'ice at the northern end of the spillway' – put down to wave slop. This is not apparent elsewhere on the spillway and is considered to be more likely to be water passing through the slabs in the upper part after passing beneath the weir.

- Increased and progressive settlement at the left-hand side of the spillway – compared with the right-hand side.
- Photographic evidence across the decades of water coming out of various joints and ponding at the base when the spillway is not overflowing.
- Comments from several a SEs over the years of leaks through the left-hand side of the spillway
- Comments from several SEs about wet patches outside the left-hand wall.
- Knowledge that the left-hand wall has no foundation
- Cracking predominantly on the left-hand side of the spillway. This cracking can be attributed to a combination of effects including:
 - Voiding beneath the slabs
 - Settlement of a flexible and settling embankment beneath a rigid concrete structure
 - Settlement of the fill averaging 2mm a year
 - The left-hand end is on the highest part of the embankment and therefore likely to suffer the greatest amount of settlement
 - The left-hand end is directly above the original stream channel at the site and the diversion culvert which may have been built
 - The area includes the area that accepted a significant amount of grout at low level during earlier grouting exercises.
 - The fill is of a type that when wetted will exhibit a significant degree of collapse consolidation.

Review of the SE's records provide evidence of flow under the crest slabs and the potential for water pressure under the slabs on the chute. This is particularly so on the left-hand side where strong flows have been observed together with water spurts at the left-hand wall footings. Furthermore, in 2014, the SE observed damp patches adjacent to the left side wall of the auxiliary spillway at the fourth concrete wall bay from the top at a time when the water level was at +0.1m. At this level water would have been at or around the top of the crest slab and with the absence of a cut off, water would have been passing over the core onto the downslope side.

The shallow or non-existent foundation to the left-hand wall, coupled with the oblique orientation, could have led to a long term 'short-cut' for water that would be able to pass under the wall to emerge on the slope outside the wall.

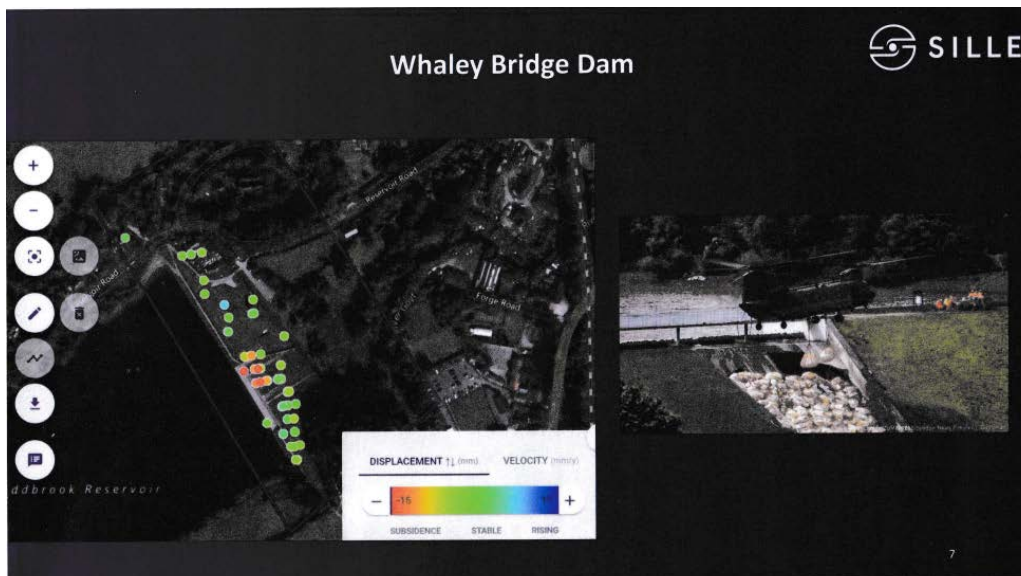
The photograph below, taken in February 2014, with the water level at +0.1m, shows the damp patches adjacent to the left side wall of the auxiliary spillway at the fourth joint down the spillway which is evidence of water getting under the slab and also passing under the wall where there is no foundation.



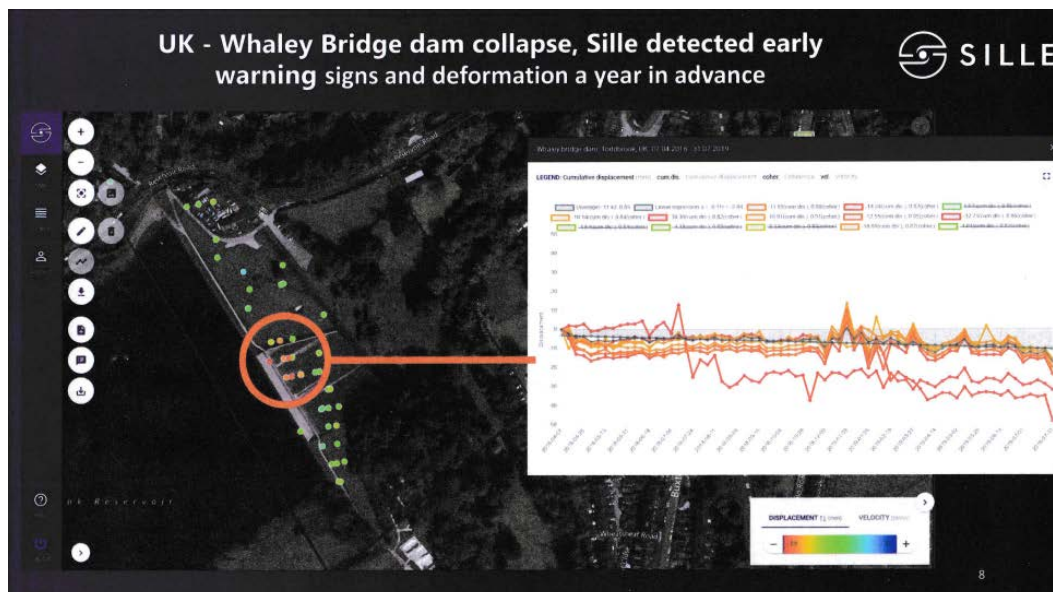
The damp patches pictured were located in the area of the hole that formed under the wall and the scour channel that formed during the 1st August event when water escaped from the side of the spillway.

12.7 Evidence from satellite surveillance

In addition to the above, other evidence supporting a progressive failure has come to light from satellites of the Copernicus Project which is said to be able to detect movements of 1mm. The satellites called Sentinel 1A and IB provided information on a number of points on the auxiliary spillway over the last 18 months. The data shows, in the graph below, that there appears to have been a 30mm drop immediately downstream of the left-hand end of the spillway in mid/late July 2018 and also at a slab lower down on the left-hand side in January 2019. This would again suggest loss of support under the spillway long before the incident indicating that some fill had already been removed from beneath the slabs.



Survey points



12.8 Supervising Engineer Records

The SE records shown in section 7 provide evidence of long-term seepage under the slab and of water passing beneath the chute slabs with associated uplift pressures and evidence of ochreous seepage leaking from the spillway. This, and the presence of wet patches adjacent to the wall, also indicate soil conditions at the surface of the downstream shoulder that encourage water flow over the surface and under the slabs.

During their regular SE examinations, the SEs frequently record leakage under the spillway crest and water seeping from joints, cracks and pressure relief holes in the lower section of the auxiliary spillway, indicating the passage of water beneath the crest and chute slabs and the potential for uplift pressures over a prolonged period.

13. Conclusions

So, having established – as described in section 11 – that the basic spillway design was fundamentally flawed, the question which is, and will be, posed is: “could and should the incident have been predicted and avoided?” Should the root cause have been spotted and acted upon sooner?

First, I note the following:

- C&RT (and British Waterways prior to 2012) has followed the requirements of the law of having inspections by independent engineers at appropriate times.
- C&RT has employed a SE.
- C&RT has employed an IE at the appropriate times.
- C&RT has supplied some maintenance at times.

So, could things have been improved? My comments are of course made with the benefit of hindsight.

13.1 The sequence of events

The evidence from photographs, eyewitnesses and the investigations suggest that the first panel that moved was near the base of the spillway. The slab moved upward on the right-hand side breaking the reinforcement and water bar relieving some of the pressure but also releasing fill from beneath the slabs. Eyewitness reports from local residents walking their dog noted water jetting up by over half a metre which when they came back the flow was running ‘dirty’. The photographs of the time show the turbulence in the flow which is in the same area as the turbulence caused by the upturned end of the slab and the convergence with the main spillway flow.

The diagrams in Appendix C show how the slabs moved. Within a few hours the first panels in the upper section moved.

It is clear that the overflowing water forced many hundreds of cubic metres of fill from beneath the slabs. Of course, there could have been, and it is likely that, some material had been removed from beneath the slabs during the preceding 40 years. Cracks were heard when the panels broke.

The photographs show, over a number of hours, that some of the upper slabs did not move, but after the overflowing water had reduced, there was collapse of some of the upper slabs into a large void. Thankfully the overflow had significantly reduced at this time.

Flow also took place under the left-hand outer wall. My investigation has shown that, because of the way in which the wall steps down, at points there is little or no foundation below the base of the slab, and thus it is hardly surprising that - as fill was removed from beneath the slab and the wall - water then exited onto the downstream face where it flowed on the surface, also eroding the sandy fill from beneath the turf which bridged an erosion channel until the turf cover failed. There was then an open erosion channel,

but again luckily the overflow had stopped otherwise a new element of the failure process might have progressed. The erosion channel was located in the area where wet patches were observed during the February 2014 SE examination.

13.2 The Role of Inspections / Inspecting Engineers

The dam was inspected by a number of ARPEs who carried out statutory inspections and submitted reports over its life in accordance with the Reservoirs Act 1975. Inspections were carried out as detailed in Section 6.

It could be argued that the C&RT (and British Waterways pre-2012) relied on the expertise of their IEs to identify any potential failure modes within this dam and reservoir. Some explicitly stated they carried out or reviewed risk analysis and failure mode analysis, others not so, but they were all undertaking (at the very least) an observational risk assessment and had access to drawings etc. to support their inspection. Thus, it could be said that the potential problems should have been picked up by others, the experienced engineers carrying out inspections. None of the IEs ordered a drawdown, none did any sort of leakage survey or survey to detect voids under the slab, and none actually questioned the likely hydraulic performance of the structure other than in terms of its capacity.

Certainly, there were drawings which showed no cut-off beneath the weir slab and there was physical evidence of distress. None of the IEs identified that water could have been moving beneath the slabs for many years, creating voids.

With the benefit of hindsight there were a number of indications that the dam and in particular the auxiliary spillway was in distress. None of the IEs, other than the most recent IE, questioned the design/ the inadequacies of the design, and when Jacobs were asked to look at the hydraulic performance of the structure they just really concentrated on the wall height; although they expressed some concerns about the situation they did not look in detail at the form of construction and the foundation etc. The IE did research the form of construction and made recommendations in the interests of safety to investigate – the main one relevant to this incident being:

“(i) a) Carry out a review of the secondary overflow channel to demonstrate it is not at risk of hydro-dynamic damage during significant overflow events caused by high velocity flow in the channel or water pressure beneath the base slabs and b) carry out any necessary improvement works.

This recommendation to be carried out within 18 months from the date of this report and b) any ‘follow-on’ work completed within the timescale set by the QCE

This and the other recommendations are provided in Appendix D.

The IE took five months to write the report and gave 18 months from the date of the report to carry out the recommendations, whilst at the same time he made another recommendation detailed below.

The Reservoirs Act 1975 states that if an IE has not provided a report before the end of a period of 6 months, beginning with the date of the completion of the inspection, then that IE must notify the Regulator and give a written statement of the reasons. Most IE's, I believe, interpret this as meaning that they have up to 6 months to write their

report.

The IE recommended that:

“(vi) The previous flood studies need to be reviewed and updated to the latest 2015 ICE F&RS 4th Edition Guidance and both freeboard and overtopping rates checked at the Design and Reservoir Safety Check floods. In addition, the following matters need to be investigated and confirmed:

- a) The exact flow split between the primary and secondary overflows.*
- b) The actual level of the clay core to be determined by trial hole.*
- c) The crest overtopping rate at the Design Check flood (assuming no wave wall) to check if the wave wall construction or stability is of concern.*
- d) The freeboards of the various bridge crossing on the primary spillway channel.”*

and gave 12 months to investigate.

In my view, again with the benefit of hindsight, he did not give the right level of priority or urgency to the issues associated with the spillway. A draft report was sent and circulated internally within the C&RT on 15th April 2019. At this point the IE had identified a significant flaw in the design and had really identified the failure mode but not stated that explicitly. The SE, an employee of C&RT, is an experienced engineer who had all the resources of C&RT in terms of access to drawings, instrumentation data, reports from reservoir keepers, statements from earlier SEs, and importantly time afforded to ‘in-house’ SEs, and when presented with the IEs recommendations did not seem to grasp the significance of the recommendation associated with the form of the spillway.

The IE did not order an immediate investigation and did not request for the water level to be lowered which he could have done within his report or in an accompanying letter and this would have triggered some action which could, in my opinion, have prevented the incident.

The IE and the SE, although informed of the potential for a significant problem to arise, did not - it would appear - understand the severity of the problem and pursue the issue, investigate the problem, nor report the problem further up the organisation.

Some could argue that C&RT employ Panel Engineers to give them the best advice possible and they therefore should have no reason to question the report or advice or the timescales stated within. The Trust ought to be able to rely on both the IE and its SE to identify any serious concerns, and hence it is entirely reasonable for the Trust to have taken its assurance concerning the dam from the active involvement of these relevant experts. I would have expected the SE to have discussed and agreed the content of the IEs report at the time of the Inspection and certainly at the draft report stage and thus being aware of the potential issues at the site.

It appears the severity of the problem was not recognised and not transmitted by the SE up the organisation to more senior management. Apparently, the issue of concern at the time of the inspection in the mind of the SE was slope stability.

Furthermore, although I believe the importance was not made clear, I can see that an owner given 18 months in which to carry out the investigation, and who did not

appreciate the importance of the problem could assume that the investigation may only take a few months and therefore might not start the investigation for some months. Thus, I believe these elements should have been progressed more quickly which could have avoided the failure/incident. This will be a recommendation I will suggest for possible amendments to the Reservoirs Act 1975 in the future – that some recommendations should state a start date.

All but one of the IEs did not identify the failure mode and the one who did, in my opinion, did not give it sufficient importance.

13.3 The Role of Supervising Engineers/ Examinations

The first question I ask the SE when I carry out an inspection is “have you any particular worries or concerns at this dam?”. I expect the SE to have fully researched the dam and to have thought about potential failure modes.

SEs, even with easy access to the drawings at the time, and who at times witnessed signs of distress, or what should have been interpreted as signs of distress, did not identify the potential problems. This could be described at the time and as a failing on the part of the SEs.

In my opinion, these recommendations should have triggered some action, as the SE should know every detail of his/her dam to be able to perform his/her duties satisfactorily.

Again, in my opinion, the SE had not recognised the poor elements of design and having received the IE report did not give the recommendations sufficient importance.

In reality, although lack of maintenance, which I will discuss in 13.4, could be argued to have contributed to the problem, it is likely that this type of failure/incident would have occurred on this occasion or during later events even if a consistent standard of regular maintenance had been provided. For example, sealing of joints would not allow pressure relief – open joints allow pressure relief but also material to be removed. This issue needs to be considered on a case by case basis. Relief wells should never be blocked but perhaps there weren't enough relief wells in any case.

13.4 Maintenance

Within the reports by the IE, and the statements by the SE, there are recommendations and advice associated with maintenance of the structure and criticisms have been made in the press, and by local residents, about the level of maintenance at the site. As stated above, maintenance could probably not have saved the structure but generally maintenance is a very important part of any reservoir safety programme.

It is clear that at times joints have been resealed and vegetation removed – for example I have seen evidence that vegetation clearance took place on the spillway in January 2018 and again in early 2019 - but it is clear that at times sufficient maintenance is provided and at other times not, and certainly at the time of the incident some vegetation was growing in the joints and the relief wells were blocked at least in part.

When asked, the SE stated that his annual statements contained advice to do elements

of maintenance, but these were often not addressed, and he would find that he would make the same recommendations year in year out – but this advice was ignored or not prioritised. In my opinion, if this situation exists it should be elevated as a problem within the organisation right up to director level if necessary or the SE should call for an inspection under Section 10, as now maintenance recommendations can be enforceable. The Reservoirs Act 1975 states that the SE is responsible at all times, and I believe it is incumbent on an SE to do whatever he/she can to advise the owner, at whatever level is necessary, to try to ensure the safety of the reservoir.

Clearly there have been problems with the frequency of maintenance interventions - as evidenced by a number of photographs and comments from the public, with the quality of maintenance provided, and the links between the technical expertise that exists in the engineering roles and operational maintenance staff carrying out works. This leads to problems of lack of maintenance, which is very visible, and this attracts public criticism.

The links between reservoir engineers undertaking inspections and operational staff undertaking regular surveillance and basic maintenance work should be improved to ensure there is a clear understanding of the need for good and timely surveillance and maintenance.

14. Wider Considerations for Future Reservoir Management and Oversight

Most of the comments I have made are made with the benefit of hindsight where one knows what has happened and you work backwards to find the cause and I believe there is evidence to support the conclusions made. However, any failure mode will give indications of poor performance before the failure exists. Whilst it might be argued that things have been missed, or not given enough emphasis, what has been lacking recently is education in the form of constant reminders via detailed bulletins from incidents and accidents that used to come from the Regulator; although there is an annual reservoir incident report. Certainly, continuous professional development must be an important element needed for appointment to or reapplication to the Panel, and this should focus on educating all about incidents and accidents in detail so that they can learn from them.

When an IE makes a recommendation, he/she must now specify a date by which the recommendation must be achieved. Thus, an 'end date' is specified i.e. "you must achieve by [...]", but although perhaps implicit, a 'start date' is not specified. This means an undertaker/owner can delay starting work for some time and still meet the requirements of the Reservoirs Act 1975, and often when starting 'late', owners miss the 'end' date. Perhaps engineers should specify start dates, or it should be a requirement of the legislation?

Appendix A – References

- (1) *Reservoirs (Safety Provisions) Act 1930*, HMSO, 1931
- (2) *Reservoirs Act 1975*, HMSO, 1975
- (3) *Report on Construction of Inclined Spillways* – prepared for BWB by John Mowlem & Co Ltd – 28th November 1969.
- (4) *Toddbrook Reservoir Safety Works, Feasibility Report Final*, Jacobs, May 2008.
- (5) *Schedule 5 of Statutory Instrument No. 1677* – HMSO, 2013
- (6) *Inspecting Engineer's Report under Section 2 of the Reservoirs (Safety Provisions) Act 1930* dated 1965.
- (7) *Inspecting Engineer's Report under Section 2 of the Reservoirs (Safety Provisions) Act 1930* dated 1966.
- (8) *Inspecting Engineer's Report under Section 2 of the Reservoirs (Safety Provisions) Act 1930* dated 1972.
- (9) *Inspecting Engineer's Report under Section 2 of the Reservoirs (Safety Provisions) Act 1930* dated 26th July 1985.
- (10) *Inspecting Engineer's Report under Section 10 of the Reservoirs Act 1975* dated 14th March 1996.
- (11) *Inspecting Engineer's Report under Section 10 of the Reservoirs Act 1975* dated 12th January 2006.
- (12) *Inspecting Engineer's Report under Section 10 of the Reservoirs Act 1975* dated March 2010.

Appendix B - Drawings

| Drg No. | Date | Title |
|----------------------|---------------|---|
| | | Toddbrook Reservoir – Details of New Overflow Weir and Spillway Details |
| 1060B | Jan 1968 | Toddbrook Reservoir, Proposed Spillway – Details of Spillway |
| BO577900/WIP/TOD/002 | 2008 (Jacobs) | Remedial Measures of Auxiliary Spillway Plan, Sections and Details. |

Appendix C – Movement of Panels - Sequence

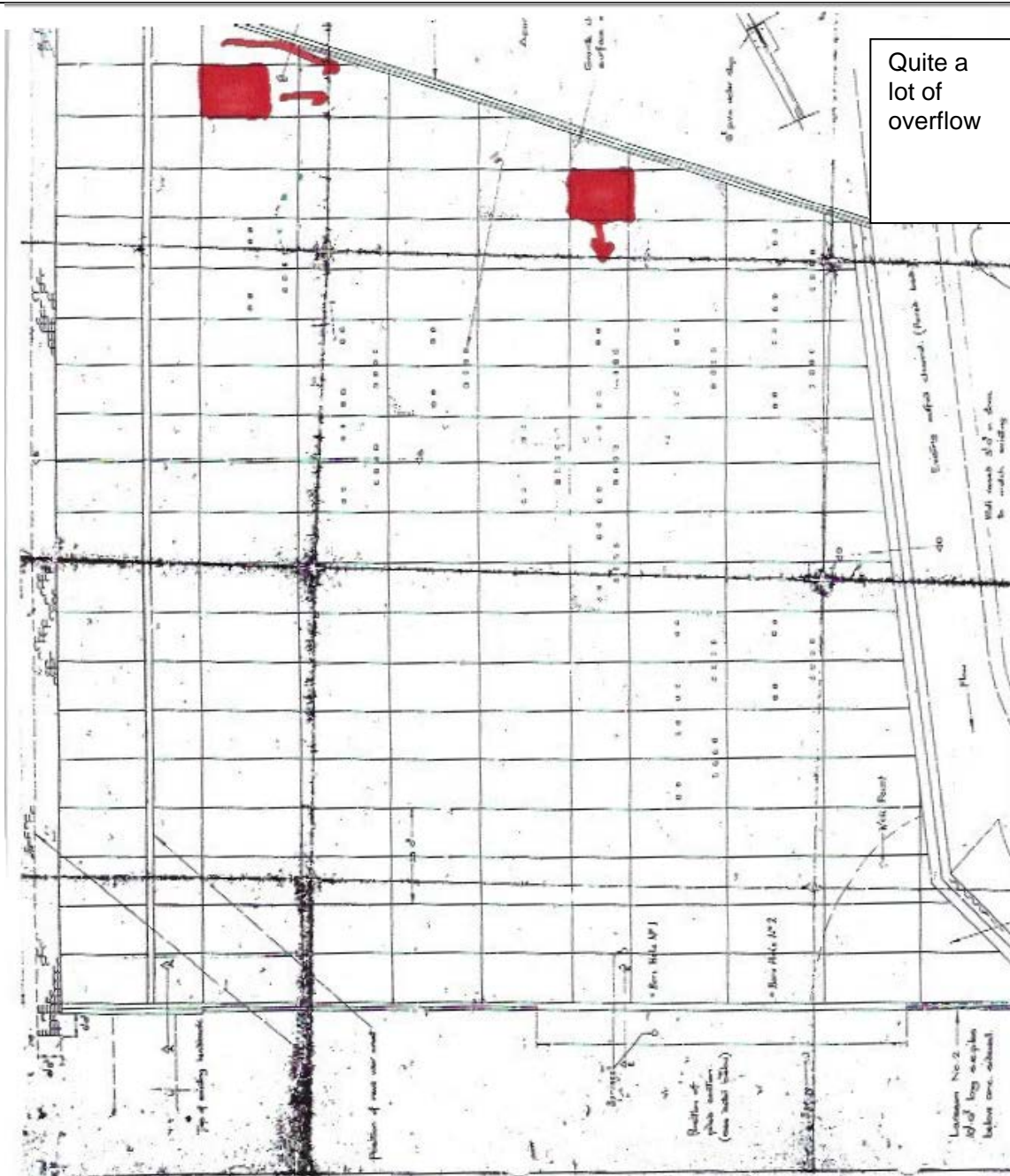
The photographs and drawings below show the sequence of slab movement and failure on 1st August 2019 that I have determined from the photographic record.



2



3



4

Pictures of flooding and Toddbrook Reservoir in Whaley Bridge
Initial lifting of lower slabs and failure of upper slab before 11am



Spillway at 11:11am

5

Thursday 1st August 2019 at approximately 11:11am



6



7

Photos published Thursday
1st August 2019 13:59



8

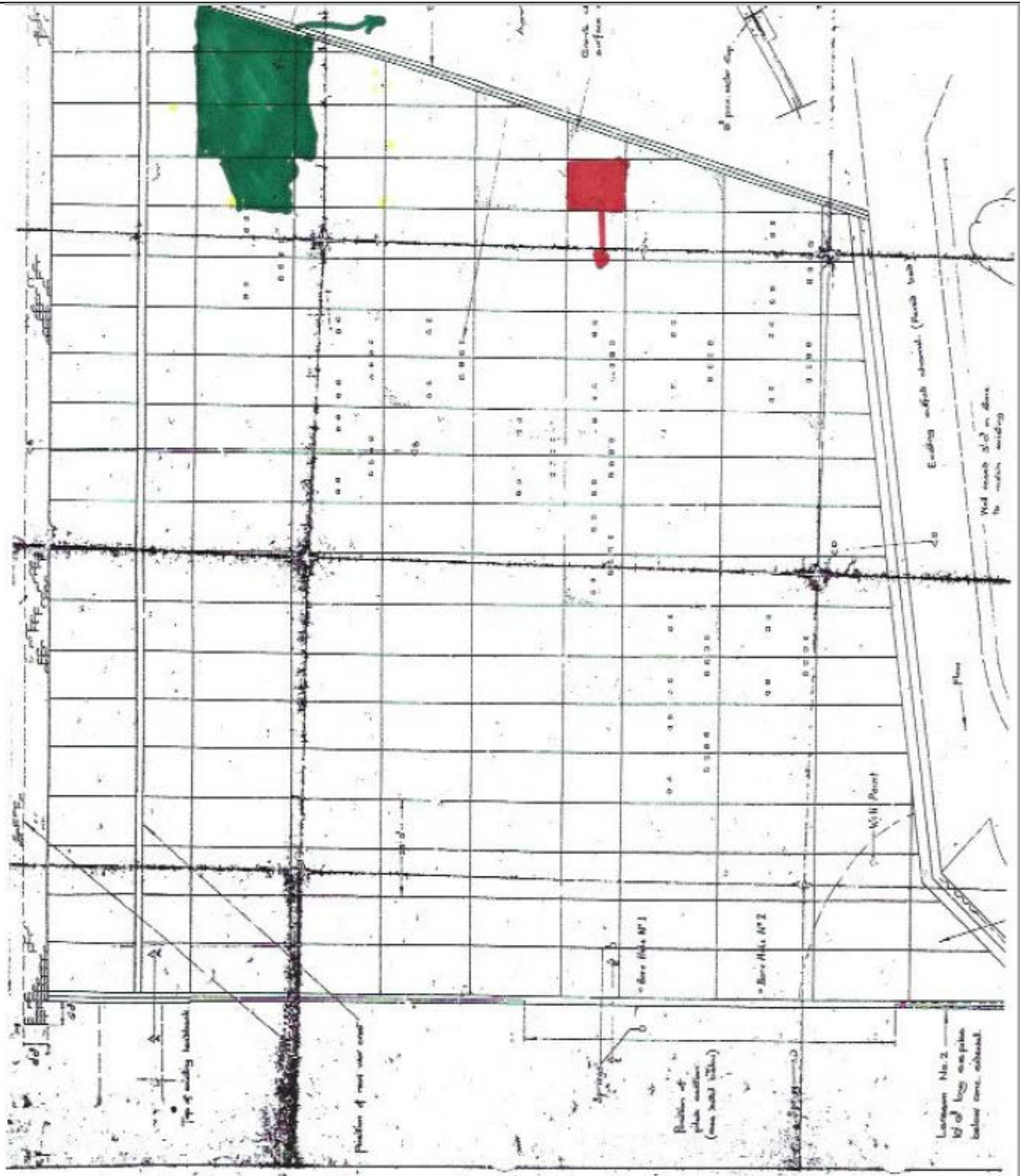
11.38am



9



10

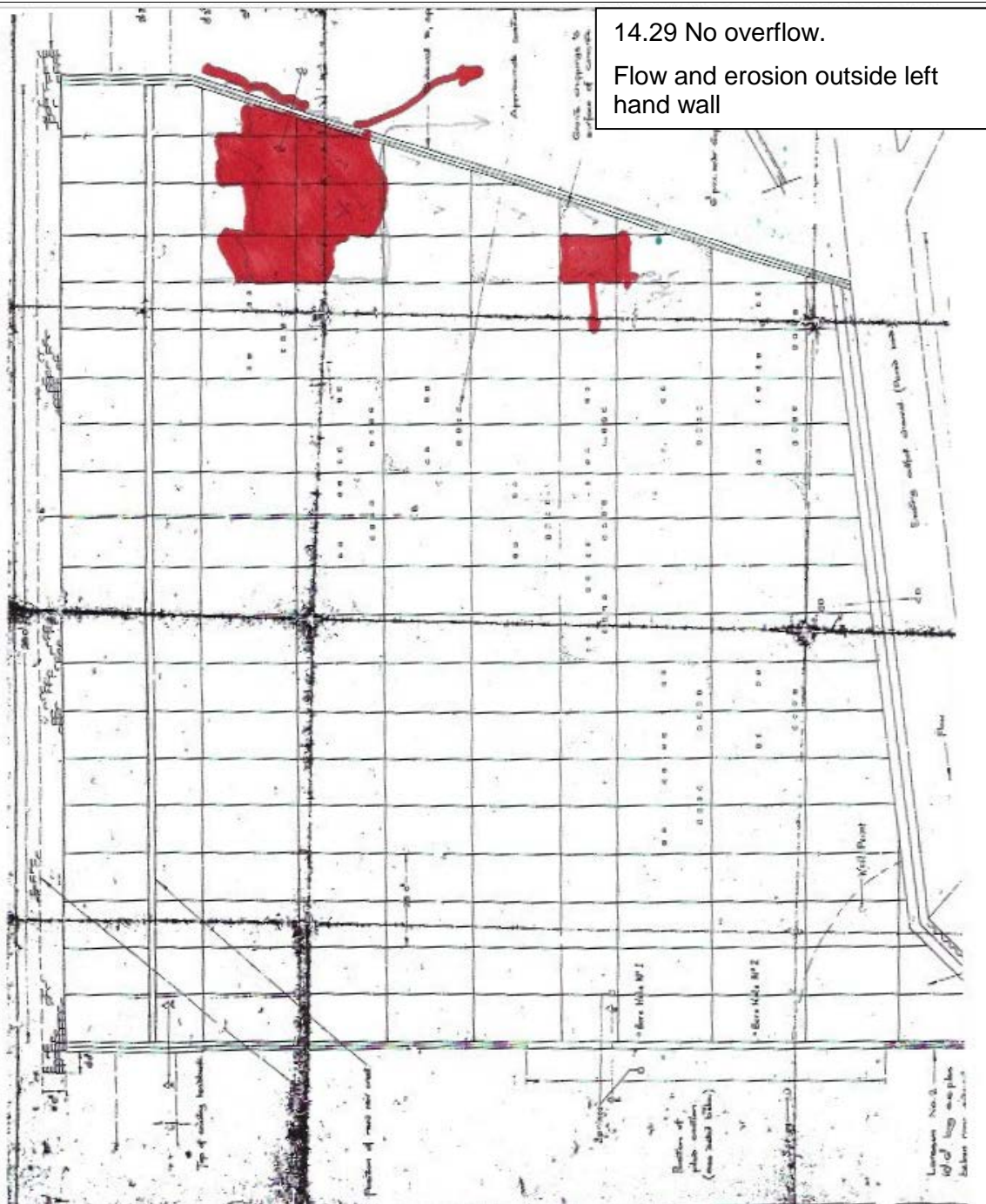


11



BBC drone footage

12



13

14:29



14



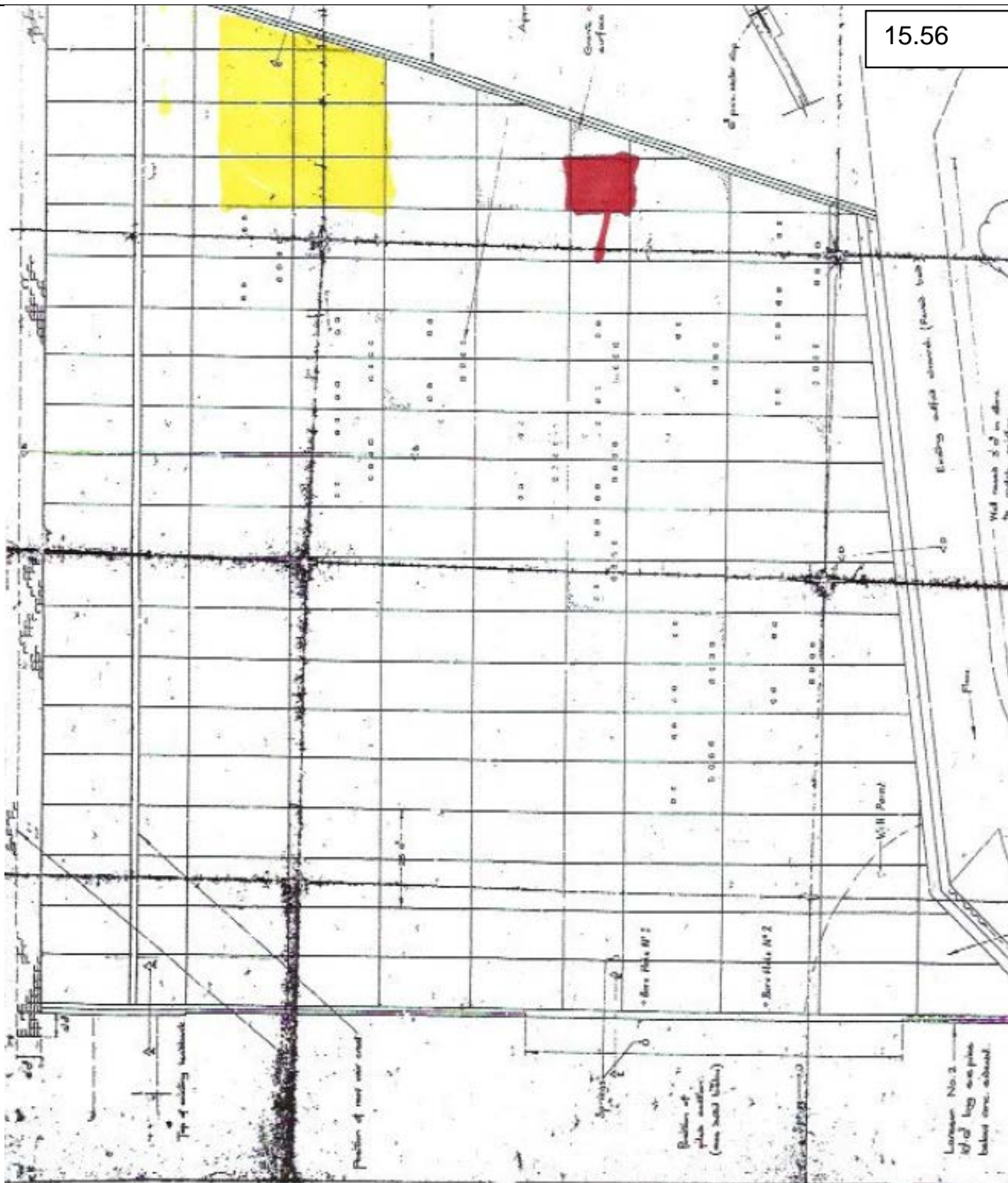
15

Afternoon of 1st August 2019: (valves had taken water down to the level of the auxiliary spillway crest and sandbagging had stopped any overflow across the slightly lower left-hand end of the weir). 14:29 David Brown





16



17



Slabs collapsed into enlarged void



Thursday 1st August 2019 13:59



Appendix D – Recommendations of the Inspecting Engineer, April 2019

“(i) a) Carry out a review of the secondary overflow channel to demonstrate it is not at risk of hydro-dynamic damage during significant overflow events caused by high velocity flow in the channel or water pressure beneath the base slabs and b) carry out any necessary improvement works.

This recommendation to be carried out within 18 months from the date of this report and b) any ‘follow-on’ work completed within the timescale set by the QCE

(ii) a) Carry out an investigation into the effectiveness of any ‘cut-off arrangements around the secondary overflow channel as it passes through the embankment crest, in eliminating the risk of water tracking under the slab and pressuring the underside and b) carry out any necessary improvement works to eliminate this risk.

This recommendation to be carried out within 18 months from the date of this report and b) any ‘follow-on’ work completed within the timescale set by the QCE.

(iii) Carry out improvements to the leakage monitoring regime within the embankment to reduce the risk of embankment instability due to an increase of the phreatic surface levels or internal erosion caused by leakage into the underlying mine workings or drainage tunnels not being adequately recorded. These improvements to include:-

a) The V-notch monitoring gauges at both the river outfall and in shaft No 6 should be fully operational at all times so that any flow increases can be picked up during the weekly site checks.

b) A system of toe drainage should be added to the base of the embankment and any outfalls fitted with V-notches so that any flow increases can be picked up during the weekly site checks.

c) The spring issue located in the right-hand abutment should be uncovered and fitted with a suitable monitoring point so that any flow increases can be picked up during the weekly site checks.

d) The installation of a flow monitoring point to collect the water discharging from secondary overflow end wall slot to collect all (dry-day) seepage flows.

e) All existing drainage tunnels, culverts and pipes should be cleaned out and CCTV surveyed for signs of distress and repaired if necessary. The volume of any material removed should also be recorded.

This recommendation to be carried out within 18 months from the date of this report.

(iv) 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. These improvements to include:-

a) A minimum of two piezometers to be installed on each of the previous sections (adjacent the draw-off pipes) used for the 2008 Stability analysis.

b) Once installed the piezometric levels need plotting against the historical data, so that a continuous record is maintained and fluctuations can be readily identified

c) Predetermined piezometric trigger levels to be agreed, in order that the necessary steps can be taken if the Factors of Safety reduce below acceptable levels.

This recommendation to be carried out within 18 months from the date of this report.

(v) Carry out improvements to the embankment level monitoring regime to reduce the risk of surface levels not being adequately recorded and instability going unnoticed. These improvements to include the installation of the following levelling pins in the secondary overflow channel: -

a) Across the downstream edge of the crest crossing (top of steepsection).

b) Across the base slab at the change in gradient.

c) Across the base slab end wall.

This recommendation to be carried out within 12 months from the date of this report.

Appendix E - CV

NAME Dr Andy Hughes
QUALIFICATIONS BSc (Hons), PhD DMS, CEng

Key Experience

- *Andy Hughes has over 40 years experience in the study, design and project management of water supply and flood alleviation schemes, asset management and risk studies and reservoir safety works. He is an All Reservoirs Panel Engineer, and is a geotechnical engineer and hydraulic engineer and hydrologist and has written a large number of technical papers and research documents. He has managed groups of individuals and professionals, department and firms to the benefit of each organisation.*
 - *Dr Hughes has been responsible for the design and condition of a number of new dams of all forms.*
 - *Remedial works at more than 300 dams have been designed and supervised under the direction of Dr Hughes.*
 - *Work in the UK, Canada, Australia, Hong Kong, Korea, Nigeria, Sudan, Ireland, USA, Israel, Austria, Sweden, Norway, Ireland, South Africa and Laos*
 - *Dam grouting*
 - *Inspections and condition reporting*
 - *Expert witness role*
 - *Geotechnical reviews including seepage analysis and slope stability*
 - *Hydraulic and hydrological analysis*
 - *Repairs including grouting, diaphragm walling, secant and sheet pile walls*
 - *Willowstick surveys*
 - *PRA & Risk Assessment reviews*
 - *Organisational reviews*
 - *Training at all levels*
-

CIRIA

Lead author – CIRIA publication – Grouting in Dams 2018

This project provides for the first time a UK reference standard in the form of a thoroughly researched and comprehensive good practice guidance report on grouting in dams, applicable to both embankment and concrete structure. It also applies and is applicable to levees and other water retaining structures.

Due Diligence

Due diligence studies for a number of owners. Reservoirs include Cahora Bassa Dam and dams owned by Alcan Smelters at Lochaber, Scotland which include concrete, earthfill and rockfill dams. Laggan and Blackwater were also the subject of due diligence studies. For Severn Trent in the UK due diligence studies and valuation studies have been carried out for their ten largest structures including Derwent, Ladybower and Howden.

Expert Panel Work

Chairman of a Panel reviewing organisation and operation and performance of all dams (mainly concrete) in Ontario Power, Canada.

Expert Witness Role

For Electricity Supply Board of Ireland for 2 concrete gravity dams – Inniscarra & Carrigadrohid. Acted as Expert Witness in the court case (Commercial Court) in Dublin. Claim £60M against Defendant (ESB)

Expert Witness Role

Expert witness for Yorkshire Water – Butterley Reservoir planning appeal

Expert Witness Role

Expert witness for City of London – Judicial Review Challenge against the City of London – Royal Courts of Justice.

Flood Alleviation Studies

- Construction Engineer for the Badsey Brook structure at Broadway, Oxfordshire - 2017/2018. An earth embankment dam with control structure and reinforced grass overflow in a very sensitive area.
- Inspecting Engineer for all Environment Agency Anglian Region Flood Alleviation studies – 2016/2019 (framework).
- Qualified Civil Engineer for remedial works at the River Till Flood Alleviation structure following a slip in the embankment.
- Advice as an All Reservoirs Panel Engineer for the Freshney scheme in North Lincolnshire.

Report on Toddbrook Auxiliary Spillway Failure on 1st August 2019

- Panel Engineer for all Flood Alleviation structures in Thames region – 2015/2018.
- Qualified Civil Engineer for remedial works on Staples Road Flood Alleviation structure following a slip after a burst water main.
- Qualified Civil Engineer for repairs to the spillway system of Pont y Cerbyd Flood Alleviation structure, Pembrokeshire.
- Delivery of training course on the design, construction and repairs of flood alleviation structures.

Advisor

To the Welsh Water Board on the maintenance, capital works and operation of 82 dams for the Company including 30 concrete dams. Review of all pipework and valves and individual assessment of safety, spillway capacity etc.

Advisor

Willowstick surveys on a number of concrete dams and earthfill embankments around the world.

Inspection

Inspection and reporting on maintenance and remedial works at more than 70 concrete dams and several hundred earthfill embankments.

Member of the Sage Committee (Scientific Advisory For Emergencies)

Strategic advisor to Government for the Toddbrook event, 2019.

Independent Reviewer Of The Toddbrook Incident

Review and the need for organisational change for the Canal and River Trust – September – December 2019.

Torside Reservoir – United Utilities

QCE and advisor to UU following an incident at Torside Reservoir in 2019.

DIRFT III Project, Northamptonshire

Construction Engineer for the second phase of the Clifton Brook Tributary – Flood Mitigation Reservoir situated at the northern end of the DIRFT III site.

Yorkshire Water - Chapel House, Cumbria

Design and the overseeing of construction of a diaphragm wall to replace the core of a clay core embankment

Thames Water - King George V / Island Barn

Design and overseeing of a diaphragm wall used to replace the core in a Thames Water dam just above the Olympic site. At Island Barn sheet piling (15 m deep) was installed to cut off seepage.

City of London – Hampstead Heath

Designer and Panel Engineer for the dams on Hampstead Heath. Remedial works include new spillways on 7 dams, the raising of 3 dams and the construction of a new earthfill dam.

RCC Experience

RCC dams are not common in the UK. Dr Hughes has inspected, as part of a safety evaluation, Milton Brook, the only RCC dam in the UK.

Research undertaken at university into the different forms of RCC including rich mix and lean varieties.

RCC dams have been reviewed as part of due diligence studies for a number of schemes around the world.

RCC dams have been designed as part of a dam selection process at a number of sites, including Nurek in Russia.

The safety of RCC dams has been reviewed as part of the work of the ICOLD Dam Safety Committee of which Dr Hughes is a member.

Federal Government of Nigeria Federal Ministry of Water Resources – Feasibility Studies and Design for Irrigation and Water Resources Management Investment in Sokoto-Rima Basin in Northern Nigeria

Dr Hughes inspected Bakolori and Zobe Dams as a Dam Safety Expert. The findings of these inspections included recommendations for remedial works including diaphragm walls as well as other aspects relating to the improvement of the dam safety of both embankment and auxiliary (spillway HEP) equipment at these sites.

Federal Government of Nigeria Federal Ministry of Water Resources – Feasibility Studies and Design for Irrigation and Water Resources Management Investment in Sokoto-Rima Basin in North West Nigeria

Dr Hughes inspected Goronyo dam in north west Nigeria and made recommendations for remedial works including a diaphragm wall cut off after an extensive review of performance. Other recommendations relate to a legislative framework for Nigeria, training for staff and prioritisation of maintenance works.

Shek Pic Reservoir, Hong Kong, January 2019

Review of valve replacement/refurbishment.

Panel Engineer - WDD Asset Protection against Tunnelling in Hong Kong

Dr Hughes acted as the Panel Engineer assessing the effect of tunnelling on all WDD (Water Development Department) assets in Hong Kong as tunnelling for new drainage and road tunnels took place.

Welsh Water – PRA (Probabilistic Risk Assessment)

Dr Hughes directed the probabilistic Risk Assessment of more than 80 dams for Welsh Water. This work allowed Welsh Water to prioritise their capital programme, identify the risks, and to compile their Asset Management Plan.

Thames Water – PRA

Probabilistic risk assessment advising on the probability of failure of all Thames Water reservoirs. Remedial works and further studies have been identified.

Welsh Water Advisor to the Board

Dr Hughes reviews the annual report on reservoir safety and reports to the Board on the progress towards risk reduction and organisational change to improve surveillance and management of their dams.

Scottish Water – Redundant Reservoirs

Dr Hughes directed a project to give strategic direction to Scottish Water to enable the disposal or re-use of some 98 'redundant' reservoirs no longer used for water supply.

Scottish Water – Small Raised Reservoirs

Dr Hughes directed a project to visit and identify works which will be required at some 150 reservoirs in Scotland as the Reservoirs (Scotland) Act, 2011 comes into force.

Welsh Water – Rhymney Reservoir

Dr Hughes provided support and direction during a serious emergency at one of the Company's reservoirs during a period of high rainfall and snow melt when a spillway broke up. Subsequently design and construction of a new spillway and TAM grouting to the core.

CIRIA

Dr Hughes was lead author and Project Director for a guidance document on conduits (pipes, tunnels, culverts) in dams. This is a best practice guide for the profession and was published in September 2015.

Farm Reservoirs

Consortium member of HR Wallingford's Guide to Risk Assessment of farm reservoirs.

Forestry Commission

Report on 17 embankment dams – review of the condition and reporting on each dam on their condition. Subsequently costing of remedial works, surveillance and monitoring, and maintenance issues. The works were then prioritised to give a programmed expenditure profile and maintenance schedule over 5 years.

Willowstick

Dr Hughes is director in charge of a business which provides the facilities of Willowstick (leakage detection) as an agent for Willowstick in the USA. To date surveys have been carried out for South East Water, South West Water, United Utilities, Yorkshire Water and Thames Water as well as Crawley Borough Council and CEB in Sri Lanka, Verbund in Austria, Norconsult in Norway etc.

Xe Pian-Xe Namnoy Hydroelectric Power Project – Laos, 2012

Lead Engineer (Dams) for lenders due diligence study for \$1 billion hydroscheme including 6 dams ranging from 78 metres to 8 metres in height and 14 kilometres of tunnel. Installed power capacity is 420MW. This involves 6 monthly visits to the scheme to give an overview of progress with reports to the lending bankers.

Non Statutory Reservoirs – Environment Agency

Dr Hughes has visited and reported on the condition of 32 reservoirs in the Thames Region of the Environment Agency. This review looked at the condition of the reservoir embankment, its hydraulics and hydrology, what remedial works and maintenance would be required, and provided a prioritised list of works.

ESB, Ireland Technical Expert Witness for the Electricity Supply Board of Ireland who faced floods on the Liffey, Erne, Shannon and Lee. On each river ESB have dams and the work has involved reviewing the operation of the dams and their gates as well as the hydrology. Claims exceed £300M.

Thirlmere Reservoir, United Utilities Dr Hughes has been asked to review the operation of Thirlmere Reservoir after the floods of 2010. UU face damage claims after floods passed through Thirlmere Reservoir.

Lake Bala, Wales – Environment Agency Dr Hughes led a team which reviewed the hydrology of the site, the stability of the embankments and the stability (seismic and static) of the gated control structure. The gate structure has also been assessed for condition and operation. Remedial works have been designed and will be instigated.

Chairman of the Board of Review–Dead Sea, Israel Chairman of a Board of Review comprising himself, Professor Eduardo Alonso of Spain and Professor George Gazetas of Greece which seeks to assist the Dead Sea Preservation Group, an Israeli Government organisation, to oversee the design of a new 17 kilometre long dam in the Dead Sea. A particularly challenging project. The Board are overseeing the works of the Dutch consultant DHV.

Advisor to the British Government/Defra on Reservoir Safety Appointed after an initial 3 year period to be advisor to Defra via a call-off contract. This contract was extended after the initial 3 years and involved the crucial time when the new Floods and Water Management Bill was being put together – as advisor.

Advisor to the Scottish Government As the Scottish Government prepare to have a 'new' Reservoirs Act very similar to the Bill/Act in England and Wales. Dr Hughes gave the Scottish Government advice on legislation and also the impact of the Act in terms of its potential financial burden.

Advisor to the Environment Agency as they took over the role of Enforcement Authority.

Advisor on policy, all aspects of the Reservoirs Act 1975, provider of training courses for all staff including Enforcement Officers, operational staff and head office staff. Advice on aspects of the Act, dates and inspections as a QCE.

Research Contractor to Defra – provider of 'call off' advice to the UK Government on all aspects of reservoir safety. Review of all publications associated with reservoir safety. Review of all publications associated with reservoir safety each year with identification of issues relevant to the UK. Author of a review into the need to rewrite the 'Guide to the Reservoirs Act' – rewrite of the Guide to the Reservoirs Act 1975. On call facility for emergency or advice.

Inspections and Advisor to a large number of UK Water Companies – Inspecting Engineer, Supervising Engineer, Construction Engineer, Referee and Qualified Civil Engineer to a large number of water companies involving:

Supervision - South East Water, UU, Eon, Scottish & Southern Energy and many private owners
Inspection - South West Water, Welsh Water, North West Water (United Utilities), Northumbrian Water, Yorkshire water, Severn Trent Water, Thames Water, Scottish & Southern Energy etc.

QCE - to Thames Water, Welsh Water, South West Water, South East Water, Scottish & Southern Energy, Yorkshire Water.

Dr Hughes is retained by a number of companies, one in particular Thames Water to give advice on any reservoir at any time in their portfolio.

Emergencies

Over the last few years I have assisted companies in a number of emergencies including:

| | | |
|------------------------|----------------------|---|
| Ogston Reservoir | - Severn Trent Water | - exploding pipework and valves |
| Rivington Reservoir | - North West Water | - internal erosion |
| Queen Mother Reservoir | - Thames Water | - failure of inlet tunnel |
| Arlington Reservoir | - National Trust | - overtopping |
| Shon Sheffrey | - Welsh Water | - poor programming of spillway construction |
| Rhymney Bridge | - Welsh Water | - failure of spillway |
| Palmers Dam | - Environment Agency | - failure of penstocks |
| Heapey Reservoir | - private owner | - failure of bypass |

Portfolio Risk Assessment

Portfolio Risk Assessment has been undertaken for North West Water (United Utilities) at the moment to enable risks to be identified and then minimised and programmed capital works.

The output is being used as part of the companies' submission to OFWAT – the regulator. Risk Assessment is

also being carried out for Southern Water and South West Water.

Advisor to RUSAL Ltd

Advisors to RUSAL Limited – a Russian Aluminium Company who are wishing to restart construction of what was to be the second highest dam in the world.

Qualified Civil Engineer working to Costain/Mouchel

Qualified Civil Engineer working with Costain/Mouchel on Yorkshire Water's Capital Programme involving a number of schemes including valve replacement, overflow replacement, and slip lining of pipelines.

Qualified Civil Engineer working to Mott MacDonald/Bentley

Qualified Civil Engineer working with Costain/Mouchel on Yorkshire Water's Capital Programme involving a number of schemes including siphon renewal, grouting to control leakage, valve tower works including valve replacement, spillway works, model evaluation, raising of dams and stability repairs. Works exceed £100M/year.

Bushey Heath No. 5 Reservoir

Repairs to a service reservoir following failure of the Reservoir. Remedial works included joint repairs, the replacement of under slab drainage and improvement to drainage flow measurement.

Repairs to Llyn Morwynion

Design of remedial works to an earthfill dam where erosion of the fill and core to the culvert through the dam was causing settlement of the crest and the real possibility of failure. The work involved the design, construction and supervision of a grouting programme.

Repairs to Llyn Alaw

Design of remedial works to stem leakage through a concrete gravity dam and to evaluate and control uplift pressures. Subsequent remedial works including post tensioned and anchors which were designed and supervised during installation.

Stability Analysis – Fiddlers Ferry Ash Lagoon

Stability Analysis and assessment of the condition of the ash lagoons at Fiddlers Ferry Power Station.

Replacement of Pipework and Valves at Cantref Reservoir

Removal, refurbishment and replacement of Lerner Johnson valves on the scour facilities at Cantref Reservoir South Wales. The works include detailed assessment of the current condition of the valves in terms of operation, metallurgy etc. and then removal and replacement under difficult and confined conditions.

Gurnal Dubbs, Cumbria

Design and construction of a scheme to reduce the level of the reservoir to reduce the leakage through a 6m high earthfill dam.

Venford Dam, Dartmoor

Commissioning of a physical model test to study the overflow at Venford Dam for South West Water. Subsequent design of remedial works involving an auxiliary spillway involving a labyrinth weir and training walls to the treatment works in a highly environmentally sensitive area.

Parkmill Dam, Woodchester

Design of increased spillway capacity works including a new overflow and overflow channel including a stilling basin and energy dissipating facilities. Also included was a new access bridge.

Clunie Dam – Scottish and Southern Energy

Director and Panel Engineer involved with the analysis and investigation of the stability of the mass gravity dam known as Clunie. Design, procurement and supervision of a rock anchoring system for the dam. This scheme won an award for innovation.

Faskally Dam – Scottish and Southern Energy

Director and Panel Engineer for a scheme to raise the cut off at Faskally Dam, Pitlochry. This dam is a popular visitor attraction and the work, which included tunnelling by adit and open excavation onto the existing cutoff had to be planned carefully to ensure public safety and construction efficiency.

Training Courses – South East Water/NRW/Thames/ Welsh Water/United Utilities

Training courses have been officially designed and delivered to a number of companies for all levels of staff from reservoir keepers and operatives right up to Director level.

Training Courses – Thames Water

A training course has been devised for Thames Water and examined and accredited under the City and Guilds Scheme for all of its water operatives engaged with service reservoir and water tower operation maintenance and

inspection. More than 200 people have been trained.

Operational Organisational Reviews – United Utilities and Yorkshire Water

Dr Hughes has been involved in carrying out organisational and operational reviews for both Yorkshire Water and United Utilities. These reviews have looked at organisational improvements in structures and reporting links and also procurement methods and programming methodologies. This has resulted in some organisational changes, clarification of roles and some changes in procurement methods.

Witley Court – English Heritage

Design Engineer and Panel Engineer for remedial works in an old earth embankment which suddenly started to exhibit signs of distress. The works included site investigation, subsequent monitoring and then design and supervision and construction of remedial works which included drainage and a stabilising berm

KGV – Thames Water

QCE under the Reservoirs Act 1975 responsible for the design and construction of enhanced drawoff works. These works include the provision of a siphon drawoff facility and emergency plan.

Technical Advisor on Tunnels associated with Reservoirs, Thames Water

Advisory to Thames Water on all tunnels associated with reservoirs including calculation of all confining pressures and Factors of Safety, inspection, remedial works, pressure restrictions, surge protection etc.

Technical Advisor on Tunnels – Ontario Power Generation (OPG)

Advisor to OPG on reservoir safety issues including tunnel safety and tunnel construction including the 15 metre diameter Niagara Tunnel.

Technical Advisor – Thames Ring Main Tunnel – Honor Oak Extension

Advisor to Thames Water on the shaft sinking, tunnel construction and connections to the Honor Oak Reservoir on the London Ring Main. Constructions of tunnel with concrete linings and shotcrete lining.

Panel Engineer responsible for Reservoir Safety

Panel Engineer to a large number of owners of dams – some 200+ per year including condition surveys of small and large diameter tunnels and remedial works.

Panel Engineer – Llandegfedd Reservoir – Welsh Water

Panel Engineer for the design and contract supervision of tunnel repairs including grouting to Llandegfedd Reservoir.

Refurbishment of Birchen Clough Tunnel

Refurbishment of 1 kilometre of flood diversion tunnel after damage to the lining.

Venford Reservoir – South West Water

Responsible for the design and supervision of new stilling basin works and a new overflow system controlled by a labyrinth weir.

Kennick, Tottiford & Trenchford – South West Water

Design and contract supervision of remedial works at the 3 sites including new spillways, crest raising and wave wall installation.

Team Member and Director – QRA Guide to Quantitative Risk Assessment – Environment Agency

Responsible for the drafting of a new Guide to Quantitative Risk Assessment involving failure mode analysis (FMECA) and consequence analysis.

Director and lead Dam Engineer – Guide to Tolerable Risk at Small Reservoirs – Defra

For a research study to 'tolerable risk' for small reservoirs including an examination of appropriate standards, a study of dams to be excluded etc.

Tummel Bridge Aqueduct – Southern & Scottish Energy

A study into the reasons for cracking and poor drainage including site investigation and stability analysis of the rockfill embankments. The final report made recommendations for a staged approach to remediation and repair.

Dead Sea Remediation – Israeli Government

Dr Hughes is Chair of a three-man international panel to ensure technical excellence associated with proposals to build a 20 kilometre dam in the southern part of the Dead Sea,

Dead Sea Remediation – Israeli Government

Dr Hughes leads a team of risk analysis to examining the project risks of all elements of the work proposed in the Dead Sea including embankment construction, drainage, freeboard allowance and hydrology.

Reservoir Inundation Mapping (RIM) – Environment Agency

Dr Hughes was a member of the Quality Review Team for a project which produced reservoir inundation maps for all dams subject to legislation in England and Wales. The work included direction as to the appropriate degree of mapping, the choice of software, the assumptions for breach mechanisms and the means of representing the information.

KGV Reservoir – Thames Water

Dr Hughes responded to a request from Thames Water upon the discovery of a leak at KGV reservoir in North London. A Willowstick survey was undertaken and subsequently a slurry wall replacement core wall was designed and supervised on site.

Abingdon Reservoir- Thames Water

Dr Hughes is Chairman of a Technical Board assembled to ensure the technical quality of the design of a proposed new, 17 kilometre long, 35 metre high earthfill dam to be sited near Abingdon in Oxfordshire.

Ontario Power – Organisational Panel, Canada

Dr Hughes Chairs a group of international experts who gather each year to examine an element of the dam safety programme of the company and to suggest improvements. The work includes interviewing staff, review policy documentation, examining sites and reporting to the Board of OPG.

Samanalawewa Dam, Ceylon Electricity Board, Sri Lanka

Dr Hughes directed a study and carried out an examination of the 200 metre high rockfill dam called Samanalawewa, Sri Lanka. The study was to identify the source of leakage in the right abutment, to report on the safety of the structure and to recommend remedial works. The project included a detailed Willowstick survey, on-site inspection and the design and costing of remedial works. A further stage of the project is planned to extend the Willowstick area, and carry out a grouting programme.

Controlled Refilling of drawdown reservoirs – Thames Water

Dr Hughes devised, implemented and supervised the controlled filling of clay embankments where restricted water levels had been applied more than 20 years before. The work involved a review of the properties of the core materials, construction of the core, sampling and testing of the core material including the evaluation of core sections and permeability. The reservoirs were refilled under controlled conditions to great success.

Warren Dam – Environment Agency

Dr Hughes was appointed Qualified Civil Engineer for the discontinuance of Warren Dam; a 12 metre high earthfill dam located near Portsmouth.