

Design of Rehabilitation Works at Ulley Reservoir

D. M. CROOK, Arup, Leeds, UK

J CLAYDON, All Reservoir Panel Engineer

P. KELHAM, Arup, Leeds, UK

R. A. KING, Arup, Leeds, UK

D PHILLIPS, Rotherham Metropolitan Borough Council (RMBC), UK

SYNOPSIS. Ulley Reservoir, near Rotherham in South Yorkshire, suffered serious damage during a storm in June 2007 and required emergency repairs. As a result of the incident, a Section 10 Inspection was recommended. The subsequent report identified a number of significant matters in the interests of safety.

The development and implementation of three of the measures are described in this paper, namely the rehabilitation of the upper three metres of the core, a new spillway and a new drawdown system. In addition the reservoir safety contingency arrangements for constructing the works with the reservoir part full are described.

INTRODUCTION

Ulley reservoir is located close to the town of Rotherham in South Yorkshire. The reservoir is formed by a 16m high and 205m long earthfill embankment dam with a central puddle clay core. The dam was built between 1871 and 1873 and was originally used for water supply but is now used for public recreation. On 25 June 2007 an incident occurred at Ulley reservoir, resulting in evacuation of the public due to fears about catastrophic failure of the dam. The failure of one of the original 1870s spillway structures, during high velocity spillway flows, resulted in scour of the downstream face of the embankment dam. Emergency works were carried out successfully and an inspection was undertaken by an All Reservoirs Panel Engineer under Section 10 of the Reservoirs Act. The subsequent report recommended nine matters in the interests of safety. This paper describes how three of the most challenging measures were addressed. These were the rehabilitation of the upper part of the embankment core; provision of a spillway capable of accommodating a probable maximum flood (PMF); and provision of a drawdown system capable of achieving 1m

MANAGING DAMS: CHALLENGES IN A TIME OF CHANGE

per day under Q_{10} conditions; i.e. ninety percent of the time. The Section 10 report contained other recommendations in the interests of safety as listed below but the resolution of these is outside the scope of this paper. The incident was written up by Hinks et al and Crook et al.

SECTION 10 INSPECTION REPORT

The Section 10 Inspection was carried out by Jim Claydon and the subsequent inspection report included the following recommendations in the interests of safety:-

1. The PMF be recalculated taking into account the catchment area occupied by impermeable surfaces.
2. A site investigation to be carried out to determine the condition of the core and remedial works undertaken as necessary.
3. A spillway or spillways be constructed to pass the PMF.
4. The hole in the crown of the former diversion tunnel to be repaired.
5. The scour pipe to be replaced with a larger diameter pipe discharging to the channel downstream of the dam.
6. The redundant pipework to be removed.
7. The Morthen spillway to be filled in with stone to support the toe of the embankment.
8. The wave protection to the upstream face to be repaired to a level determined by the modified overflow.
9. Mining specialists be engaged to provide a statement on the coal that has been mined under the dam, the coal remaining and the likelihood of further settlement.

POST INCIDENT DECISIONS AND ACTIONS

Rotherham Metropolitan Borough Council (RMBC) had two options after the completion of the Section 10 report. These options were to discontinue the reservoir or to rehabilitate it. After much debate within the council, the decision was taken in September 2007 to rehabilitate the reservoir, the driving criteria being the importance of the reservoir to Ulley Country Park, which includes a sailing club and wet wildlife habitats. Discontinuance was seen by RMBC to be a loss of amenity for the residents of Rotherham.

Arup was instructed to continue investigation works for the embankment whilst the rehabilitation design and construction supervision was put out to competitive tender. Arup was successful with the tender and were appointed as designers and project managers for the construction supervision in June 2008.

EMBANKMENT CORE ASSESSMENT AND REHABILITATION

The core was investigated as part of the post incident assessment. The details of the preceding desk study and ground investigation were written up in a paper by King *et al* (2009). This work allowed a ground model of the embankment to be developed, potential seepage paths to be identified and properties of the fill materials to be assessed for their susceptibility to internal erosion. Prior to conducting the ground investigation a review of historical records indicated the presence of a puddle clay core with a filter zone on each side. Useful historical drawings were obtained, including cross-sections of the dam from the time of construction. Unfortunately, the drawings were not consistent and site works were crucial to allow a ground model of the embankment to be developed.

In 1969 works were carried out to the upper three metres of the core to provide sufficient freeboard between normal operational top water level and the top of the clay core. This work comprised excavating a 0.75m (2.5 feet) wide trench some 1.8m to 3m (6 to 10 feet) deep along the crest and placing a weak plastic concrete above the then top water level. The records show the concrete should have contained 4% of cement and an admixture which is believed to have acted as a retarder. The ground investigation and subsequent construction works found that in many places the plastic concrete was still plastic though elsewhere it had hydrated and formed a weak brittle concrete. Trench sheeting made of corrugated iron was still present and voids were found under the concrete. One of these locations was upstream of the 2007 scour hole where seepages had been observed during the incident. There was enough doubt about its integrity to point towards its removal and replacement regardless of the integrity of the rest of the core.

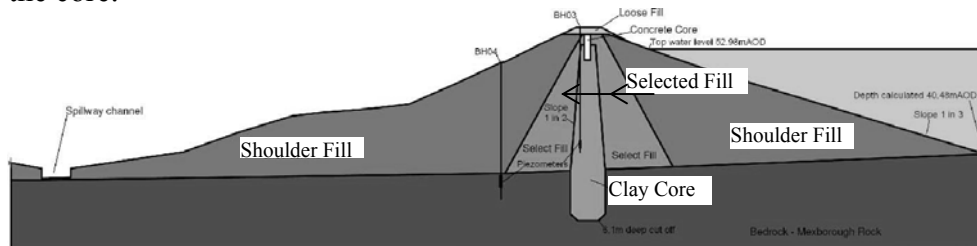


Figure 1. Section through embankment

Following the ground investigation a geotechnical interpretive report was produced that gave information about the existing state of the embankment and puddle clay core. A ground profile through the embankment and underlying strata was created by combining the historical information with the ground investigation data. This ground profile is shown above in Figure 1. The interpretation work included an assessment of the integrity of

MANAGING DAMS: CHALLENGES IN A TIME OF CHANGE

the puddle clay core but it was not possible to give a definitive answer as to the quality of the core and its efficacy as a water proof barrier.

An assessment of the risk of hydraulic fracture was made, based on the geometry of the relatively slender clay core, which was obtained largely from historical records of the dam's construction. The dispersibility of the puddle clay core was then assessed to determine whether or not clay would disperse into water if the core fractured and a seepage path developed. The assessment methods and results of these tests, which were inconsistent and did not correlate with information from other reservoirs, are summarised in Table 1 and discussed in the paper by King et al (2009).

Table 1. Results of dispersibility analysis in comparison with Brown & Bridle

| Dam | Liquid Limit | Plastic Limit | %<75µm | Double Hydrometer | Crumb Test | Pinhole Test |
|-------|--------------|---------------|-----------|-------------------------------|------------------------------------|---|
| A | 31 | 12 | 62, 69 | 47%, 42% | Both Dispersive (1 x G3, 1 x G4) | Not tested |
| B | 27 to 54 | 8 to 28 | 39 to 99 | 9 tests 5> 50% 1x 3050% | 10 tests 1 x G4, 2 x G2, 7 x G1 | 9 tests, 1x D1, 3 x ND3, 4 x ND2, 1xND1 |
| C | 35 | 19 | 76% | 29%, 30% | Both G3 | ND2 |
| Ulley | 26 to 48 | 5 to 25 | 32 to 56% | 73 to 109% | 5 x G1, 1 x G3 | 2 x ND2, 4 x ND3 |

Note: The term G refers to dispersive where the maximum is G4 and the minimum is G1. The term ND refers to non-dispersive.

Taking the combination of hydraulic fracture risk, possibility of dispersive clays and potential seepage paths through the clay core and concrete raising interface, it could be assumed that seepage through the core was possible. It was therefore likely that embankment failure would be governed by the potential for internal erosion of the embankment materials. It could also be assumed that if the material on the downstream side of the core did have either the potential to resist internal erosion, or the ability to filter eroded clay particles to prevent further loss of clay core, then the risk of failure by internal erosion would be much reduced. Hence an analysis of internal erosion was undertaken on the clay core and then on the select fill and shoulder fill successively downstream.

The analysis of filter properties was undertaken using a variety of methods from published literature. The scenarios considered were: -

- if the puddle clay core were to be considered dispersive and particles of clay were transported through to the adjacent select fill, or

- If silt particles from the core were transported through, then the select fill would need to be an adequate filter, trapping clay or silt particles and preventing further erosion.

The filter potential of the select fill gave positive results as shown in Table 2. However, dispersibility tests on the core material were inconclusive and there is the possibility that a seepage pathway could occur through the puddle clay core which did not transport clay particles into the select fill. In this event it was the potential for the select fill to resist internal erosion which governed the safety of the embankment.

Table 2. Results of select fill filter properties

| Method | No erosion criteria | Ulley results | Criteria met |
|--------------------|---|---------------------------------|--------------|
| Johnston et al | (D15 of filter/ D85 of core) < 5 | 0.0075/0.023 = 0.33 | Yes |
| Foster & Fell | DF15 ≤ 9DB85 | 0.0075 < 0.207mm | Yes |
| Sherard & Dunnigan | DF15 ≤ 7D85 to 12D85 | 0.0075 < 0.161 to 8.4 | Yes |
| Bridle 2008 | Permeability of Filter < 4x10 ³ cm/s | 1.9 to 3.5x10 ⁷ cm/s | Yes |

The results of the particle size ratio analysis differed between methods and even within the same method depending on which portion of the particle size curve was considered. This was because the select fill particle size curve was found to be gap graded with in some cases up to 25% gravel content. The potential for internal erosion of the select fill could be assessed by comparing the permeability of the material with the hydraulic gradient expected to calculate the discharge velocity of porewater. The calculated seepage velocities at Ulley were found to be much lower than would be required to cause erosion within the select fill.

As the analyses of the potential for the select fill to resist internal erosion gave varied results, it was necessary to consider the material downstream of the select fill. If the select fill did erode, it is then the shoulder fill which would filter the select fill. The filter properties of the shoulder fill material were assessed by comparing the coarse particle size of the select fill with the fine particle size percentage of the shoulder fill. For most samples analysed, the shoulder fill material was found to be an adequate filter

Should seepage occur through the core and select fill material into the shoulder fill material, without fines being eroded, the ability of the shoulder fill material to resist internal erosion would govern the risk of embankment failure by internal erosion. The methods from the published literature which

MANAGING DAMS: CHALLENGES IN A TIME OF CHANGE

analyse the shape of the grain size curve were compared. Some of the results showed the potential for instability and some did not. The difference was due to varying amounts of coarse gravel within the samples and the position of some of the analyses on the grain size curve. The conclusion was the shoulder fill material is at risk of internal erosion due to the poorly sorted and gap graded nature of this material.

The assessment of filter properties and the potential for internal erosion demonstrated how difficult it can be to obtain a definitive classification of actual embankment materials when using grain size curves alone. This is due to the variable nature of materials within an old embankment dam.

As previously stated, remedial works to the upper part of the core would definitely be required. Whether or not works were required at a greater depth depended on the risk of internal erosion. Should these be considered significant enough to potentially cause failure of the dam, then remedial works at depth would be required. However, if they were small, then the disturbance caused by remedial works could not be justified, as further problems might be initiated, such as creation of voids during excavation or piling.

An event tree was developed based on the analyses carried out and probabilities were assigned to each branch of the tree, either based on test results or on engineering judgement. Even in the worst case scenario, the probability of failure was calculated to be around 1% as detailed by King et al (2009).

To complete the assessment of the integrity of the embankment at Ulley Reservoir a brief review of the other types of internal erosion, as defined by the ICOLD European Working Group on Internal Erosion, was undertaken. These are: suffosion; backward erosion (piping) and contact erosion. These types of internal erosion are considered to be unlikely at Ulley, but concentrated erosion into the foundation is a possibility at Ulley, due to the fractured bedrock on which the dam is founded. However, locations are impossible to predict and no action is required at this time.

Following the review of the potential for internal erosion, it was felt that the risk of damaging the puddle clay core at depth as a result of doing any work outweighed the risk of a defect either being present or occurring at depth within the puddle clay. It was decided to limit the depth of remedial work to just below the base of the concrete core extension.

Several options were considered and many ruled out with the following reasoning:

- Dig out concrete and replace with clay – this would have required a significant source of clay and a deep excavation.
- Grouting around the concrete/clay interface – it would be difficult to assure quality control and low overburden stress could result in escape of the grout and failure to consolidate the soil/concrete interface.
- Sheet pile cut-off upstream of concrete – steel piles were rejected for a variety of reasons including the difficulty in ensuring they intersected the puddle clay, and driving problems associated with the presence of construction debris.
- Install horizontal drains and monitor – the risk of further seepage is considered too high without intervention.

It was therefore considered that a bentonite cement slurry wall would be most appropriate to achieve a seal with the puddle clay and provide the desired permeability, which should be the same as the original core. This option could accommodate any small ongoing settlement of the embankment and an hydraulic excavator could remove the concrete core raising and install the slurry trench in the same process. A key into the puddle clay core would be required, but this solution would not alter the state of the puddle clay at depth.

In order to facilitate the construction of the bentonite cement slurry wall it was decided to lower the crest level by 0.5m to 54.8m AOD to provide a wider crest width as well as sufficient freeboard during a PMF event based on a normal top water level of 51.75m AOD. Over the years changes to the top water level had resulted in a much reduced crest width.

SPILLWAY

Prior to the incident there were three spillways. The locations and details for the original spillways were described by Crook et al (2009).

During the June 2007 incident the Ulley spillway was destroyed, the 1943 spillway did operate successfully, but the height of the Morthen spillweir prevented flows passing down this overflow.

As a result of the Section 10 Report, a new flood study was undertaken, which identified a PMF inflow in to the reservoir of 134.8 m³/s. To accommodate this significant increase in the design flow it was necessary to carry out a fundamental assessment of the viable overflow options for this reservoir. The possible solutions needed to maintain the top water level that

MANAGING DAMS: CHALLENGES IN A TIME OF CHANGE

had existed prior to the incident, which was 51.75m AOD, and in order to preserve the appearance there was a desire to achieve the necessary wave freeboard without having to provide a wavewall. Following flood routing studies an optimum spillweir length of 20m was identified, which would safely discharge the peak PMF outflow of 125 m³/s and allow the crest of the embankment to be reduced by 0.5m

The basic options for the spillway locations were to use one of the mitres or to locate on the embankment as shown in Figure 2, all of which could be engineered to meet the objectives.

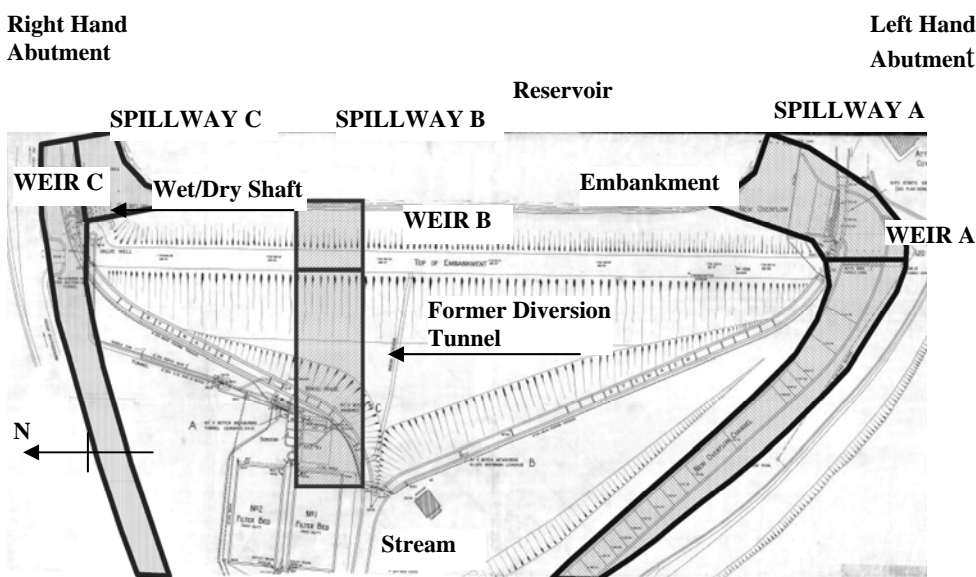


Figure 2. Proposed spillway options

The left hand mitre option, Option A, was rejected both on cost and technical grounds because it required significant excavation of the approach channel, spillweir and chute spillway to lower the ground level by at least 1.2m. It would involve demolition of the existing concrete spillway and construction of the longest chute from the abutment to the downstream watercourse.

The assumption that parts of the 1943 spillway could not be used because of its condition was subsequently proved to be correct as significant leakage was observed during construction whilst discharging pumped outflows. Another major disadvantage was that it could not be used to provide an emergency gravity discharge capability during construction.

The right hand mitre, Option C, was rejected on both cost and technical grounds because it is located in fairly steeply sloping sidelong ground and

the gap between the end of the embankment and the site boundary wall is very limited. Also, the shaft for the drawdown system is located near the right hand end of the embankment. Since this shaft needed to be retained in some form, as described in the emergency drawdown system below, there is limited space for a 20m spillweir and chute spillway. There was also concern that foundations conditions for the chute spillway may be less than ideal because of cuttings and excavations for historic water treatment structures and the drawoff tunnel and access shafts

The remaining option was to position the spillway on the embankment which is Spillway B in Figure 2. This was considered to be the best option in both technical and cost terms; however it presented the problem of assessing the future behaviour of a structure founded on fill.

Historical records of settlement of the embankment are limited. The embankment has existed since 1875 and has been consolidating ever since. There has also been deep coal mining under the reservoir and the embankment, which has now stopped. In 1943 and 1969, the crest was raised as part of other works. However, there is sufficient detail to be confident that there is now only residual consolidation of the embankment taking place.

The make up of the embankment shows a reasonably granular material, which means that the potential for continuing consolidation of the shoulders is limited, but there is the potential for the core to consolidate by greater amounts. However at the spillway location, because the loading on the embankment has been significantly reduced by the lowering of the crest level to 54.8m AOD, the potential for settlement has been significantly reduced. The consolidation potential is at its maximum at the core position and progressively reduces towards the toes in each direction.

The embankment is founded on rock, though it is rock that is shattered. The conclusion the design team reached is that although there is a potential for consolidation it will not be significant. To take account of the consolidation, the spillway structure is articulated which will allow a small amount of rotation at each joint. Careful attention was given to the detailing of the joints to provide for rotational movement and to retain hydrodynamic forces within the spillway structure. Where the spillway crosses the core, there is a downstand beam which will maintain an intimate contact with the clay. The stilling basin and allied structures are founded in rock and can be taken to be relatively stable.

MANAGING DAMS: CHALLENGES IN A TIME OF CHANGE

The hydraulic profile was initially designed by manual calculation. The channel tapered in plan as much as possible without introducing cross waves. The vertical profile was adjusted to minimise excavation and wall height. A triangular weir was adopted as this has been shown on other recent schemes to have a good coefficient of discharge at all heads. The stilling basin was designed as USBR Type 4 with blocks. Once the arrangement was sufficiently advanced, CRM Rainwater Drainage Consultancy was asked to prepare a physical model of the design. The purpose of the model was to validate the design of the weir and chute and to check the stilling basin performance for different tailwater conditions. The site topography meant the model had to extend away from the embankment, across a main road and along the stream bed downstream of Pleasey Road.

The modelling was very useful and provided calibration for the spillweir and capacity of the Pleasey Road culvert which controlled the tailwater in the stilling pool. Due to the depth of flow over the road it was decided to run the model assuming that the road would be washed away. This demonstrated that the stilling basin no longer had sufficient tailwater to contain the hydraulic jump. Modifications were then made to the model and subsequently the design to increase the length of the stilling basin and size of the baffle and chute blocks.

There are stream training works downstream of the stilling basin to return the discharge into the original channel, which occurs a short distance upstream of the road crossing.

The design life for the spillway structure has been taken to be 120 years. Between the weir and the stilling basin are three bays consisting of floor slab and walls, with joints design to articulate. An underdrainage system has been provided to measure leakage through the joints.

EMERGENCY DRAWDOWN SYSTEM

Prior to the 2007 incident the drawdown system was more or less that which had been provided as part of the original construction. The drawdown system was located on the right hand abutment and comprised an upstream intake tunnel driven through the foundation rock in a north westerly direction; a wet/dry shaft upstream of the crest; and a downstream discharge tunnel containing a 15" pipe leading to the former water treatment works as shown in Figure 3. The drawdown system discharged into the former Morthen spillway.

Following the Ulley 2007 incident the Section 10 Inspection identified that the side walls to the Morthen spillway were considered to be unsafe and recommended that the spillway needed to be filled in to protect the integrity

of the embankment slope that was supported by the spillway. Infilling of the spillway also meant rendering the discharge point for the drawdown system beyond use. This was not possible until upstream control of the drawdown system had been achieved. Hydra-Jest attended site and were able to get one of the wet shaft valves shut, which allowed the 15" pipe in the downstream tunnel to be drained.

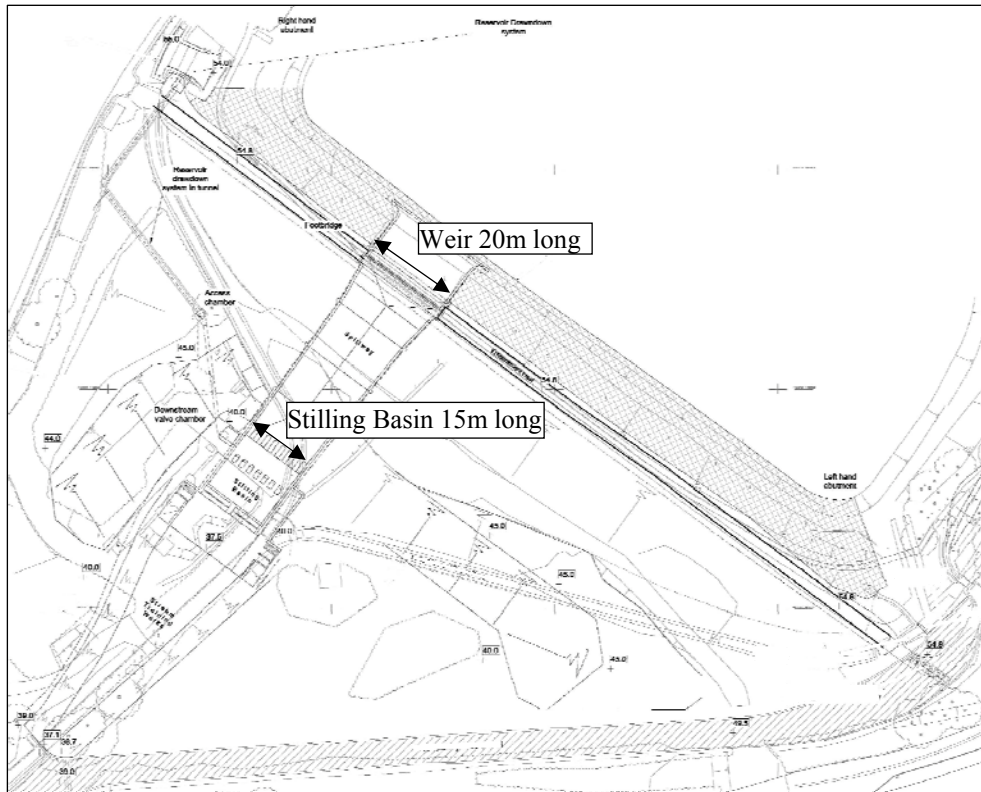


Figure 3. Location plan of spillway and drawdown system

Following the infilling of the Morthen spillway there has been a total reliance on temporary pumps to regulate the reservoir level, but this was considered to be acceptable in view of the intended programme for refurbishment works.

Various options were considered that would achieve 1m per day under Q_{10} conditions i.e. where the inflow is only exceeded for 10% of the time. Analysis determined that a pipeline with a minimum internal diameter of 500mm was necessary to achieve this drawdown requirement.

The next consideration was to set the levels for the drawdown system since the reservoir is only retained for recreation and no longer required for water supply. As such, the water level barely changes during the year except in

MANAGING DAMS: CHALLENGES IN A TIME OF CHANGE

response to weather conditions such as very dry or very wet weather. This led to the question being asked if a drawdown system was necessary. The advice given to RMBC was a system is necessary in the event of an emergency. The next question was the level to which the reservoir might need to be drawn down to. In RMBC's opinion there was no need to be able to completely empty the reservoir. In fact, there are serious wildlife issues if the reservoir is emptied should the valves be left open accidentally or maliciously.

Following discussion with the Panel AR Engineer, a compromise was reached, which would not unduly stress the wildlife but which would ensure safety of the embankment. The solution provided 5.75m drawdown from the normal operational top water level of 51.75m AOD. Initially a drawdown of 4m was considered. The Panel AR Engineer was not completely happy with this and felt a lower level would be more appropriate. The agreed drawdown level of 46m AOD is some 5m above the bed of the reservoir and corresponds to a retained volume of 100,000 m³.

The next issue was where to place the drawdown system and its means of operation. The possible options considered were either on or near the line of the existing drawdown system or next to the proposed spillway. In terms of operation, the options were to be entirely gravity operated or to incorporate some syphonic action.

In setting the position and arrangement of the new drawdown facility a number of constraints needed to be identified. The main issues were: the existing upstream tunnel had last been inspected in 1969 and was in an unknown condition; the valves in the wet shaft needed to be removed because they were life expired; the reservoir could not be emptied to provide access to the upstream end of the tunnel and to remove the hydraulic load from the valves because of environmental considerations. The option of installing a cofferdam was considered but doubts about the granular nature of the ground over and around the tunnel plus uncertainties of its plan position meant that there were too many unknowns. The presence of the 45° bend near the wet/dry shaft ruled out using a liner even if the intake level was lifted to 46m AOD. The final conclusion was it was not possible to reuse the upstream tunnel and it would need to be abandoned.

By contrast, the reuse of the downstream masonry arch tunnel of height 1.3m was viable. It had the added benefit of passing through the embankment's waterproofing. Although there had been historic leakages into this tunnel, over time and a number of remedial grouting schemes, the stage had been reached where there was confidence in the performance of

the tunnel. There was concern that if a new route was created through the waterproofing it might prove difficult to seal and cause problems in the future. This effectively ruled out the option of constructing a new high level discharge route through the waterproofing.

By this stage, the location of the new spillway had been fixed. The obvious end point for any drawdown system was into the stilling basin. Since the stilling basin incorporated a permanent pond, then any discharge from the drawdown system could be underwater and dissipate the energy.

In the end, three location options were identified. Option A (shown below in Figure 4), was a pipeline in trench along the line of the existing tunnel to the wet/dry shaft at alternative levels of 46.0m AOD and 48.5m AOD for a gravity only option or a combined gravity/ siphon system respectively. The pipework would then be routed through the downstream tunnel to discharge into the stilling basin.

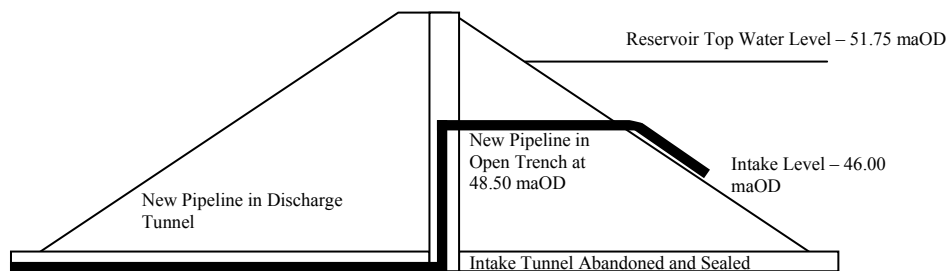


Figure 4. Option A – Combined gravity and syphon system

Option B was a high level option to pass through the right hand abutment and not use any of the tunnels. Option C was located adjacent to the new spillway, but to limit the amount of excavation the gravity flow would be limited to the top 2m. The pipe would be placed under the spillway to give a soffit level of say 49.75m AOD, then the remaining depth to 46m AOD would have to be removed by syphonic action.

Option A was chosen because it already had a route through the waterproofing via the downstream tunnel. All excavation works would be upstream of the waterproofing and these works could be done independently of the spillway works. The route is longer and has higher losses than Option C; however hydraulic analysis showed that under siphon conditions Option A had the lowest negative (vacuum) pressure. Once the new pipeline had been installed, tested and commissioned the outlet shaft and downstream tunnel were filled with foam concrete.

MANAGING DAMS: CHALLENGES IN A TIME OF CHANGE

EARLY CONTRACTOR INVOLVEMENT

In view of the short time allowed to achieve completion of the matters in the interests of safety, RMBC appointed their term contractor to implement the design. This allowed meetings to take place where options could be discussed and this did provide very useful input into the design process. An example of the early input was the problem of installing the drawdown pipe into the discharge tunnel which allowed suitable details to be devised.

The early contractor involvement had a number of other advantages in that it allowed the design and the tendering programmes to overlap and gave the contractor the opportunity to look at the documents and have discussions about the requirements for each element of the works. The co-operation of all parties associated with the scheme has continued into the construction programme. Particular items have included the establishment of a reservoir safety team within the construction team who have developed the protocols for dealing with possible storm events and the call-out contacts for various types of event. There was close liaison about the maintenance of the flood defence at the position of the new spillway where small adjustments of the permanent works allowed a neat solution to be implemented.

CONCLUSION

Initially it was anticipated that solutions to the three main matters in the interests of safety that required addressing could be found very easily. However, as more information was collected, no clear cut solutions presented themselves and significant optioneering was required to consider the pros and cons of individual solutions. Inevitably, there have had to be compromises but the act of going through each option very carefully and rigorously has provided a result that will ensure the safety of the reservoir in the future and is the best value for the owner.

REFERENCES

- Hinks J.L. Mason P.J. and Claydon J, (2008) Ulley Reservoir and high velocity spillway flows, in *Ensuring Reservoir Safety into the Future*, pp 227-237, Thomas Telford, London.
- Crook D. M. Kelham P. Phillips D. and Machen I, (2008) Supervising Engineer's account and reflections on major reservoir safety incident, in *Ensuring Reservoir Safety into the Future*, pp 238-250, Thomas Telford, London.
- King R. A. Gilbert R. Claydon J. Crook D. M. and Phillips D, (2009), *Rehabilitation of Ulley Reservoir: assessment of the integrity of an embankment dam*, LTBD Graz Conference