

Remedial works at Sutton Bingham Reservoir

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SYNOPSIS. Sutton Bingham Dam, near Yeovil in Somerset, was completed in 1955 and is possibly the last embankment dam in the UK to have been constructed with a puddle clay core. The dam behaved satisfactorily until October 2006 when the annual settlement readings along the crest showed a sudden increase of up to 80 mm over a length of about 45 metres. There was further settlement of 73 mm during the period from October 2006 to January 2008. The settlement together with the observed distortion of the concrete slabs was indicative of an incipient slope failure in the upstream shoulder of the dam.

As soon as the problem was drawn to the attention of the Panel Engineer in October 2006 he asked for the water level to be held down 2 m below Top Water Level to mitigate the consequences of any future slip. Meanwhile intensive monitoring and site investigations were put in hand. Considerable difficulty was encountered maintaining the target water level when significant inflows occurred during the early months of 2007 and this highlighted the need to increase the drawdown capacity of the reservoir.

The reasons for the slope failure are discussed in this paper and include the effects of the annual operational reservoir drawdown during the summer months. The major component of the remedial works was the flattening of the upstream slope of the dam to 1(V):5(H) using rock fill to give a satisfactory factor of safety.

The timing of the construction of the remedial works required detailed consideration to optimise construction activities without jeopardizing abstraction from the reservoir and operation of the water treatment works.

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EMERGENCY DRAW-DOWN FACILITIES

The original bottom outlet from the reservoir takes the form of a 450 mm diameter pipe and valve discharging into the bottom of the bell mouth spillway 11.8 m below Top Water Level. With the 57.5 ha reservoir full this has a capacity of about 1.74 m³/sec which is sufficient to lower water level by 262 mm/day in the absence of inflow to the reservoir. The average inflow is estimated to be 0.1m³/s. However the reservoir has a catchment area of 30km² and typically overflows for several months each winter. If only periods when the reservoir is overflowing are considered, the Q50 and Q10 inflows are estimated to be 0.52 m³/sec and 1.46 m³/sec respectively. At the higher of these inflows there was only sufficient capacity to lower water level by 42 mm/day.

The Inspecting Engineer asked for the bottom outlet capacity to be increased by at least 1.7 m³/s but the Owner elected to provide a total of 4.33 m³/s new capacity, bringing the bottom outlet capacity to 6.07 m³/s. This was done by installing two 600 mm diameter pipes through the wall of the bellmouth spillway 5 m below TWL. Each pipe was fitted with a control valve and a guard valve. Operating in conjunction with the existing scour pipe, the final arrangement allows the water level to be lowered by 838 mm/day from top water level against an inflow of 0.5m³/s and to draw the reservoir down to 75% level within approx. 3 days. The reasons for choosing a higher capacity than that originally requested were that, due to the cost of the temporary works, the additional cost of two pipes was relatively modest and also to provide some additional control over water levels during construction of the rock fill stabilisation work.

Twin 700 mm diameter holes were first drilled through the 3 m thick concrete wall of the bell mouth into a temporary steel limpet type cofferdam which had been installed on the outside of the spillway wall. The 600 mm diameter pipes were then inserted and grouted in. The installation of the valves was completed later with the reservoir level drawn down.



Figure 1- Limpet type cofferdam

The 600 mm diameter valves each weighed 0.77 tonnes so there was a need for a steel platform bolted to the spillway to support the four valves. This structure was provided with impressed current cathodic protection.



Figure 2 - New bottom outlet valves

The valves are operated by electric actuators installed on a new steel bridge leading from the original valve house.

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The north-west side of the spillway shaft has settled since construction and, in 2006, it was 38 mm lower than the south-east side. It is therefore unfortunate that the weight of the valves, platform and access bridge had to be carried on the north-west side of the shaft. There was no obvious alternative. However analyses suggested that there will be negligible acceleration of the tilt of the shaft. This will need to be kept under observation in the future.



Figure 3 - New bridge providing access to valve spindles and actuators

UPSTREAM SLOPE STABILITY

Sutton Bingham Dam has a maximum height of 15.2 m and impounds a reservoir with a capacity of 2.6 Mm³. The reservoir area is underlain by Middle and Upper Jurassic formations consisting mainly of clays and limestones overlain, in the valley bottom, by a thin cover of recent alluvium. Water in the reservoir submerges successively the Oxford clay, Cornbrash limestone, Forest Marble (clays, mudstones and shelly limestone) and, at the shallow end, Fullers' Earth while the dam itself is founded on Forest Marble with its ends in the overlying Cornbrash limestone and Oxford clay.



Figure 4 - Wave wall and concrete slabbing prior to the start of the works

The Cornbrash limestone is very permeable and a cut-off trench was excavated to a depth of more than 70 feet (21.3 metres) beneath the surface on the left bank. The grout curtain extends beneath the dam and for some distance at either end.

The upstream face of the dam, with which this paper is primarily concerned, slopes at 1(V): 3(H) at the top slackening to 1(V): 4 (H) and ultimately 1(V): 6(H) further down. A paper in 1957 by R.C.S.Walters and R.J.C.Walton stated that the shoulders of the dam were composed of Cornbrash limestone and Forest Marble Clay. The site investigations in 2006/2007 however discovered very little of the Cornbrash limestone fill suggesting that, at least in the area exhibiting movement, the upstream shoulder was composed mostly of the Forest Marble Clay.

Discussion on the above paper in March 1958 drew the comment by A.C.Penman that piezometers in the upstream shoulder of the dam were particularly slow to respond to changes of water level in the reservoir. In the light of the 2006 movements this remark now seems particularly significant.

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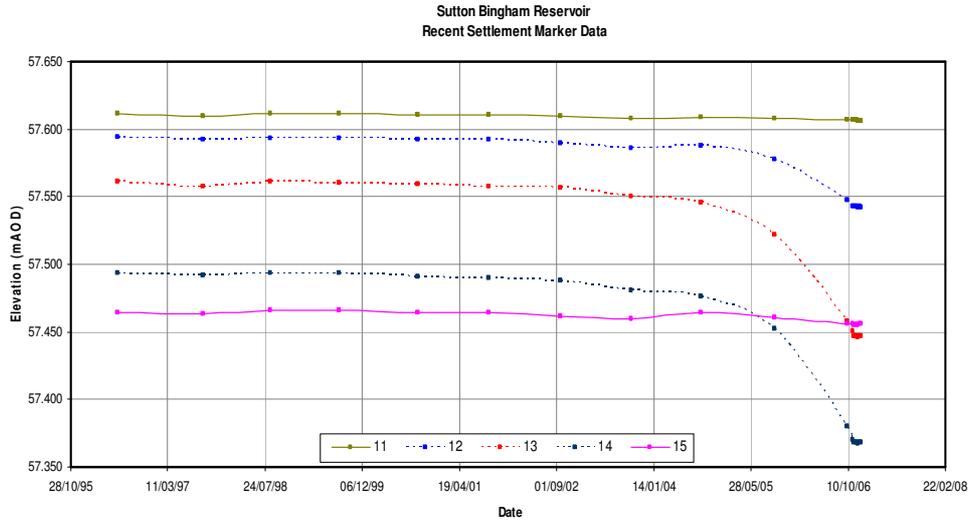


Figure 5 – Settlement at selected monitoring points

In 1974 Binnie and Partners had been commissioned by the Wessex Water Authority ‘To investigate and report on the feasibility and cost of enlarging Sutton Bingham Reservoir as a means of augmenting water resources within the Parrett catchment’. As part of this investigation there were five boreholes and three trial pits on the dam. Working from these data a cohesion of 10 kPa and ϕ of 25 degrees were assumed for the Forest Marble in calculations performed at the end of 2006. With rapid drawdown assumed to 4.1 m below TWL the Factor of Safety falls to 1.01.

The subsequent installation of inclinometers showed the depth of the slip surface to be close to that predicted by these early analyses. However the analyses do not explain why the concrete slabbing on the upstream face of the dam had apparently slipped about 100 mm down the face overturning the toe beam. It would appear that this was an independent phenomenon although probably also related to the low shear strength of the Forest Marble.

Early in the investigation cracks were noted in the fill immediately downstream of the wave wall. When these were explored with trial pits softening of the sides of the cracks was observed prompting speculation that the cracks had opened during the dry summer of 2006 and that water had entered them during the autumn. This may have played a part, but the principal mechanism of failure was probably the annual ‘rapid’ drawdown which occurs during the summer months. During the periods May to September 2005 and 2006 levels were drawn down by about 4 m with the upstream piezometers showing very little response.

Recent settlement data is shown on Figure 5. An unanswered question is why significant settlement was only observed in 2005 (when it caused little concern) and in 2006. The reason that there was negligible settlement in previous years is still unexplained although the appearance of cracks immediately downstream of the wave wall in 2006 (and possibly also in 2005) might possibly offer an explanation.

OPTIONS FOR REMEDIAL WORKS

The following four options were considered for the remedial works:

- Option 1 Remove the concrete slabs and slacken the upstream face of the dam to 1(V): 5(H) by placing rock fill.
- Option 2 Construct a granular toe berm with its crest about 3.4m below TWL and then place rockfill at a gradient of 1(V): 5(H) above it.
- Option 3 Construct a sheet pile wall along the upstream face of the dam and use it to retain rock fill above it at a slope of 1(V): 5(H).
- Option 4 Dig out the slipped Forest Marble in the upstream shoulder of the dam and replace it with granular material.

Option 1 was eventually chosen for the following reasons:

- Conventional and robust solution.
- Least sensitive solution to variations in ground model and geotechnical parameters.
- Construction works generally of one form – “earthworks”.
- Construction works are relatively unaffected by changes in reservoir level and/or adverse weather conditions.

The main disadvantages with the option were that it would require the deployment of waterborne plant and require the greatest volume of rock fill – with associated logistical and environmental issues.

DESIGN OF UPSTREAM SLOPE

Factors of safety for the existing and proposed slopes were calculated as follows:

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Design Case	Required	Existing	Remedial Option
Operational Drawdown	1.3	1.0	1.42
Operational Drawdown plus Seismic	1.05		1.13
Reservoir Full	1.5	1.29	1.67
Reservoir Full + Seismic	1.2		1.40
Emergency Drawdown	1.1		1.32

The 10,000 year Peak Ground Acceleration was 0.08 g. The analyses considered 2/3 of this figure as pseudostatic methods were used.

The above factors of safety were considered satisfactory.

The design required the breaking up and removal of the existing 125mm thick concrete slabs. There was then a fine filter (Filter 2) overlain by a coarse filter (Filter 1) on the uncovered surface. Filter 1 and 2 layers both have a minimum thickness of 150mm. The rip rap wave protection layer is 360mm (2-rocks layer) thick. The wave condition used in the design of the rip rap sizes was hind-cast using Jonswap spectrum for a fetch length of 1,000m and 1 in 200 years return period +50% wind speed.

The rip rap was designed using Van der Meer (1998) for Initial (minimal) Damage Level, which corresponds to no damage level (0 to 5%) in Hudson's formula. The design rock size, W50 is 15kg, based on a 1 in 5 slope and a permeability factor of 0.1 for the impermeable embankment material.

Achieving the correct grading for the rip-rap proved extremely demanding. First the quarry sent material that was much too small and when this was rejected they supplied material that was far too large. Many loads had to be sent back to the quarry and even when the stones were in place some had to be dug out and replaced.



Figure 6 - Recently placed rip-rap Underwater Works

Fill at the upstream toe of the dam and part way up the slope had to be placed under water. This required the establishment of a temporary wharf on the left bank of the reservoir and the importation of barges and a pontoon.



Figure 7 - Plant and barges for underwater work

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PROGRAMME AND CONSTRUCTION MANAGEMENT

The objectives set by the owner were to complete the measures in the interests of safety as quickly as possible whilst not jeopardising the water supply to its customers. The water treatment works fed from the reservoir supplies around 55,000 customers. Due to the extensive placement of rock fill underwater it was expected that the water quality in the reservoir would deteriorate such that it was not treatable (this proved to be the case when work started in August 2007). Additional constraints included the need to avoid winter working on the clay embankment, allowance for the fact that the existing scour may not be able to fully control water levels, and inclusion of sufficient time after completion of the work for refilling of the reservoir before the following summer.

After detailed consideration of all these factors the agreed approach was to commence work in late summer (August 2007) after the first summer peak demands, with a target completion date of January 2008, allowing 2 months for the reservoir to refill before 1 April. The first activity was the installation of the additional scour pipes so that water levels could be controlled but this would have to commence when the water level was 2m below TWL, hence the need for the limpet dam referred to above. As the underwater rock fill level came up the water level in the reservoir was drawn down. During this particularly sensitive period the monitoring frequency for the settlement markers and inclinometers was reduced to every other day.

As there remained the possibility of high water demands during the early part of the construction programme and the risk of loss of supply from the water treatment plant, Wessex Water also implemented a resource contingency plan involving maximisation of the output of other treatment plants, increasing the transmission capacity into the supply area and bringing into service standby sources. Although primarily acceleration of future projects, it should be noted that the cost of these works exceeded the cost of the remedial works to the dam.

The project was constructed by Wessex Engineering and Construction Services (WECS), Wessex Water's in house delivery arm, with the design undertaken by Halcrow. Both parties worked closely together during the design development to ensure optimum constructability, within the project constraints. The only major sub contract was for the construction of the temporary wharf and provision of all the water borne construction plant. All materials including the rock were procured directly and the rock placing above water carried out directly by WECS.

The time constraints required the options appraisal to be carried out in parallel with the ground investigation. In addition an independent peer review by an All Reservoirs Panel Engineer was integrated into both the optioneering and detail design stages. These activities enabled the design concept to be agreed in March 2007 and the project budget set. Detail design and procurement then progressed in parallel, with site establishment and stockpiling of rock commencing in July 2007.

COST

The cost of the remedial work is approx. £ 3.4 million. A simplified breakdown is given below:

Description	Cost £m
Site investigations, monitoring	0.253
Environmental mitigation, archaeological surveys etc	0.036
Slope stabilisation works	
Enabling works and temporary access road	0.164
Rock fill (23,500 t) and rip-rap (6,500 t)	0.402
Plant for under water placing (10,000 t)	1.651
Plant for above water placing (20,000 t)	0.181
Miscellaneous repairs	0.010
Emergency Draw-down Facilities	
Drilling, 2 No. 600 mm diameter pipes, valves, bridge etc.	0.192
Design, project management, preliminaries, supervision etc	0.515
Total	3.404

CONCLUSIONS

The slope movements at Sutton Bingham were not particularly dramatic but might well have become so. Very careful monitoring was carried out during the period of the remedial works to give early warning of any acceleration of the movement. Careful control was also maintained over changes in water level during this period.

The time taken from the first site visit by the Panel Engineer (19 October 2006) to the completion of the £ 3.4 million remedial works (February 2008) was about 16 months. During this period the initial appraisal was made followed by site investigation, detailed design and construction. The compressed timescale was made possible by the use of Wessex Water’s in-house construction arm.

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A very obvious lesson from the incident was the value of the annual level survey even where there had been 50 uneventful years since the original completion of the dam.

Another useful lesson learned from the incident was the value of a bottom outlet with sufficient capacity to control water levels in an emergency. This was provided relatively easily at Sutton Bingham – in many cases it would be more difficult.

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ACKNOWLEDGEMENTS

The authors acknowledge the kind permission of Wessex Water Services Ltd. to publish this paper