

RISK BASED APPROACH IN A SPILLWAY UPGRADE

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ABSTRACT

In line with worldwide best practice, major dam upgrades are currently being undertaken in New South Wales (NSW) to ensure the continued safe operation of these assets in line with the Dam Safety Management Guidelines set out by Australian National Committee on Large Dams (ANCOLD) and NSW Dams Safety Committee requirements. Burrendong Dam, which is one of the major dams owned by State Water, is located in the greater Murray Darling River catchment on the upper reaches of the Macquarie River. The dam is currently being upgraded to increase the flood handling capacity of the spillway to cater for extreme flood events.

The conventional spillway design was found to be expensive therefore State Water embarked on a risk based approach to reduce the risk below As Low as Reasonably Practicable (ALARP). Saddle Dam A, which is located approximately 1.5km east of the existing spillway, was considered to be a viable option for an auxiliary spillway. The main issue was, however, the presence of a shear zone containing brecciated rock and clay gouge, which consisted of broadly graded silty sandy gravels and sandy clays. Hence, the potential for piping and foundation erosions were identified as the major risks for the project. The conventional design approach of a traditional concrete apron slab with ground anchors was found to be very expensive. Therefore, the erosion risks for not providing an apron slab to protect the shear zone were thoroughly investigated. The geotechnical conditions were analysed in order to satisfy the nominal criteria for energy dissipation of spillway discharge flows into the natural environment with an acceptable degree of erosion risk. The outcome of this analysis led to a significant reduction in cost as well as risk.

This paper focuses on the analysis and design of the geotechnical aspects adopted in the risk based approach, which led to significant cost savings to the spillway upgrade project. This paper is intended to provide valuable information for geotechnical professionals handling similar types of projects – especially where there is a potential to reduce cost by accepting a risk based approach.

1 BACKGROUND

Burrendong Dam is in the greater Murray Darling River catchment on the upper reaches of the Macquarie River, downstream of the confluence with the Cudgegong River in central-west New South Wales. The dam is located approximately 32 kilometres upstream of the township of Wellington. Construction of the dam was completed in 1967, and its primary purposes are for river regulation, hydro-electric power generation and flood mitigation. The dam is owned and operated by State Water Corporation.

The main embankment of Burrendong Dam is a zoned-earthfill embankment constructed across the Macquarie River. The intake and outlet works are located near the right abutment of the main embankment. Two earthfill saddle dams and a concrete primary spillway are located along a ridge separating the Cudgegong River from the Macquarie River reach downstream of the main embankment. Spillway flows are controlled by seven radial gates, and are discharged into Macquarie River approximately 2.5km to 3km downstream of the main embankment. Some of the features of Burrendong Dam are shown on Figure 1.

The State Water Portfolio Risk Analysis has shown that Burrendong Dam does not meet current deterministic standards for flood capacity. The PRA outcomes led to the conclusion that the spillway capacity of the dam was inadequate, and had to be upgraded to improve the safety of the dam against larger flood events.

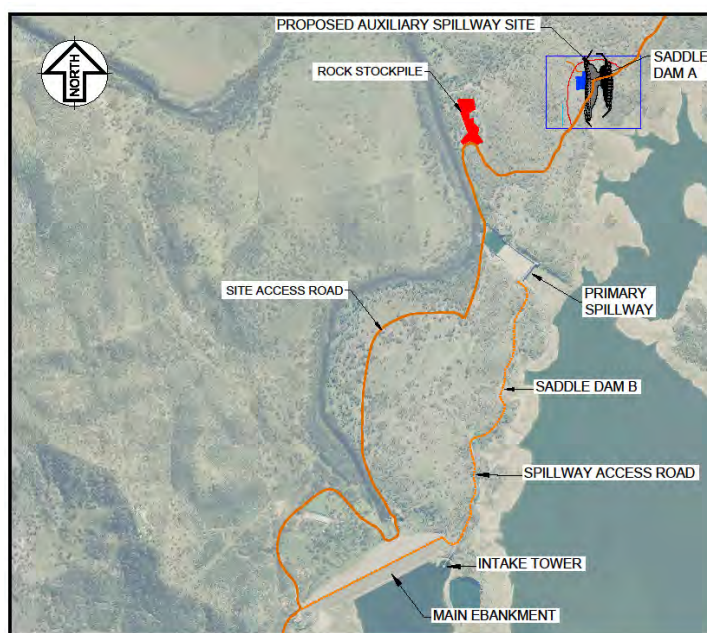


Figure 1: Layout of the dam

Following extensive investigations, a two stage upgrade was adopted as the strategy to reduce the risk of the dam from being overtopped during extreme flood events. Stage 1 consisted of raising the dam by 1.8m, and Stage 2 was to construct 120m wide auxiliary spillway to increase the outflow capacity of the dam.

The saddle dams were found to be the attractive options for the new auxiliary spillway. The existing Saddle Dam B is located between the main embankment and the existing gated spillway, and Saddle Dam A was located approximately 1.5km east of the existing gated spillway. Saddle Dam A was adopted as the preferred location for the auxiliary spillway due to its position away from the main spillway and favourable hydraulic behaviour. Due to financial constraints, the challenge for the project was to manage the geotechnical and hydraulic conditions in developing a risk based spillway design to optimise the overall cost, which would make the project viable to State Water.

The option for upgrading the existing gated spillway was estimated to be in the order of \$30M to \$40M. On the other hand, the option of an auxiliary spillway at Saddle A was estimated to be in the range of \$10M for a conventional type concrete apron spillway to \$5M for a risk based partially erosive spillway (if triggers), which were acceptable to State Water.

Construction of the auxiliary spillway has been completed in August 2014 at a cost of \$5M. The hydraulic analysis and concept designs were carried out in house by State Water. Geotechnical investigations and detailed designs were undertaken by NSW Public Works (Public Works Report, 2013) and URS Pty Ltd (URS Detailed Design Report, 2013), respectively. The construction works were undertaken by Bardavcol Pty Ltd.

This paper mainly focusses on the geotechnical challenges encountered at the Saddle Dam A site, and the assessments that lead to the optimum cost effective design and construction of the spillway. Aspects of hydraulic characteristics and their impact on erosion are also discussed.

2 GEOLOGY AND GEOTECHNICAL ASPECTS

The investigations at the Saddle Dam A included geological mapping of a trench and a series of vertical cored boreholes. The trench was 1.5 m deep and extended 134 m across the gully parallel to the axis of the existing saddle dam (shown in Figure 2). Rock cores were drilled in the left and right shoulders, at the centre of the saddle dam and in the centre gully floor downstream of the dam.

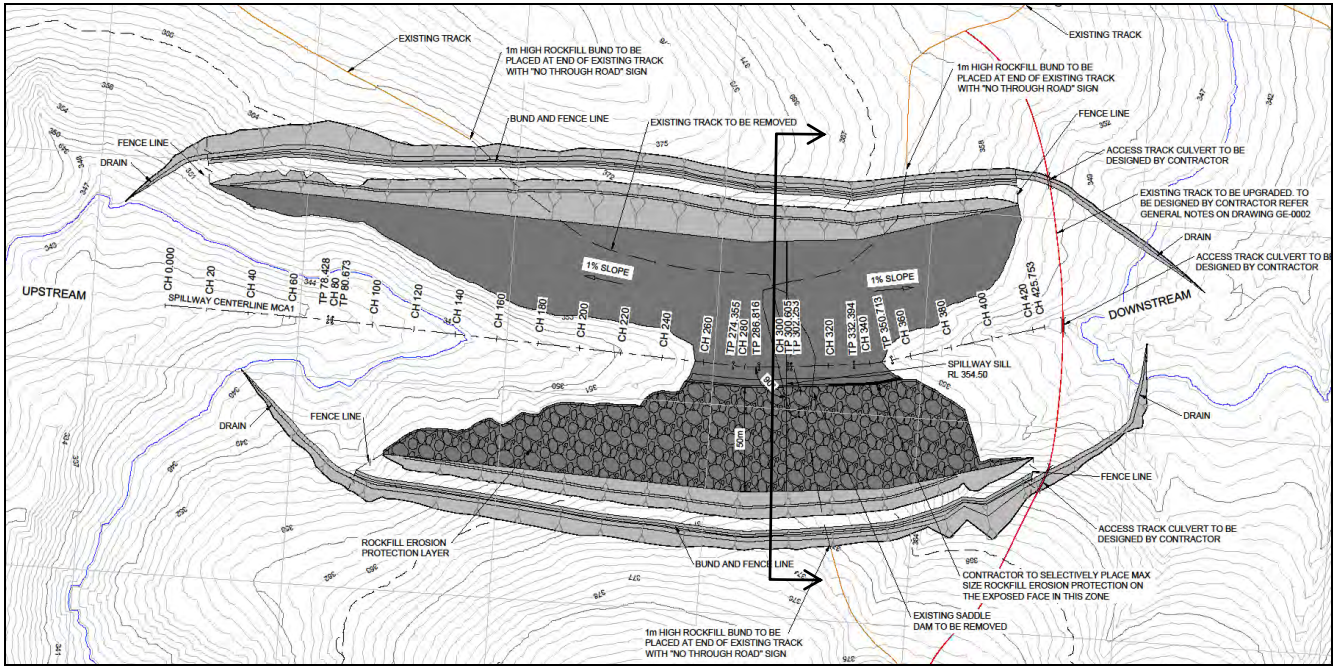


Figure 2A: Plan

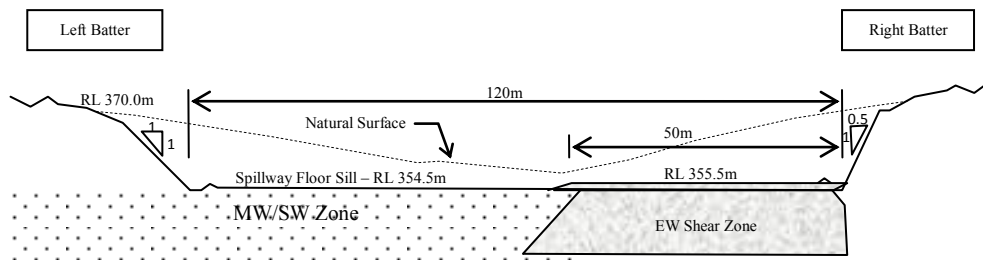


Figure 2B: Cross-section

Figure 2: Spillway layout at Saddle Dam A

The regional geology of the area consists of a Hill End Synclinal Zone, which forms part of the Lachlan Fold Belt. The underlying bedrock geology is indicated to be of inter-bedded meta-sandstones and meta-siltstone rocks of the Devonian age Crudite Group. Several major fault zones are indicated to pass through the saddle dam site. The defect orientations were logged and plotted across the trench and were split into three groups that correlate with the areas in the spillway floor, and in the spillway walls with the representative stereonetts plotted for each section and the whole trench. The orientation of the bedding and joints was identified to vary across the site. Two major joint sets were identified with two other minor joint sets also identified. The joints were generally found to be iron stained, occasionally in filled with calcite, rough and discontinuous with very close to close spacing increasing with better weathering grade and depth. The open surface defects were generally in-filled in the slightly weathered rock. Bedding was moderately spaced.

Shears were dominant features in the saddle, and were observed to be generally sub-parallel to the bedding but other orientations were also observed. Calcite stringers and veins up to 100mm thick and sandy clays (extremely weathered meta-sandstones) were associated with the shear zones. The potential for dykes was also noted in the area. The meta-siltstones were apparently more weathering resistant than the meta-sandstones. The soil to rock interface was noted to be irregular with highly or moderately weathered rock underlying the soil. No ground water was observed. Point load tests were not possible due to the fractured nature of the rock. In general, decreasing weathering grade from soils and extremely weathered rock to slightly weathered rock was observed across the site, and fresh rock was not encountered.

The spillway foundation was to be at a depth of approximately 14m cut into predominantly meta-siltstone that is slightly weathered (SW) on the right and moderately weathered (MW) on the left. There is an approximately 1-2m thick zone of lesser quality rock and soil near the surface. The spillway floor was proposed to be at RL 354.5m, approximately two thirds of which is inferred to comprise moderately weathered rock and one third extremely weathered (EW) sheared meta-sandstone.

The data indicated that the bedding was adversely oriented with dips out of the slope allowing potentially unstable planar failures for the left batter. Further reviews of these defect orientations established that the left batters should be 2H:1V and 1H:1V in EW to HW and in SW to MW materials, respectively. The right batters were designed at 2H:1V and 0.5H: 1V in EW to HW and in SW to MW materials, respectively. The channel bench widths were set at 1m to prevent vehicle access to comply with Work Health and Safety requirements. The highly weathered material was expected to persist up to 3m in this area. This slope has been revegetated to further improve the stability and provide erosion protection.

In light of the above findings, allowance was made in the design for the use of shotcrete for as a risk based treatment of the batters. Areas, which were prone to erosion and degradation, could lead to undermining of the slope and minor rock falls. The areas identified during construction such as highly fractured areas, shear zones, and dykes, were considered for local treatment with shotcrete. Allowance for rock bolts was also made to improve safety in areas, where random adversely oriented defects were expected to potential slides or toppling hazards, which could compromise the integrity of the spillway or the safety of workers. The use of these strengthening measures was considered in the risk based estimate, as well as in the contract as provisional sums. The need for these strengthening measures was assessed on site at the time of construction.

3 SPILLWAY DESIGN CONSIDERATIONS

Extensive hydraulic modelling was carried out by State Water by using MIKE21 to establish the geometry of the auxiliary spillway required to pass various flood events, including 1:10,000 AEP, 1:20,000 AEP and Probable Maximum Flood (PMF). The velocities along the spillway channel obtained from the above modelling are presented in Figure 3.

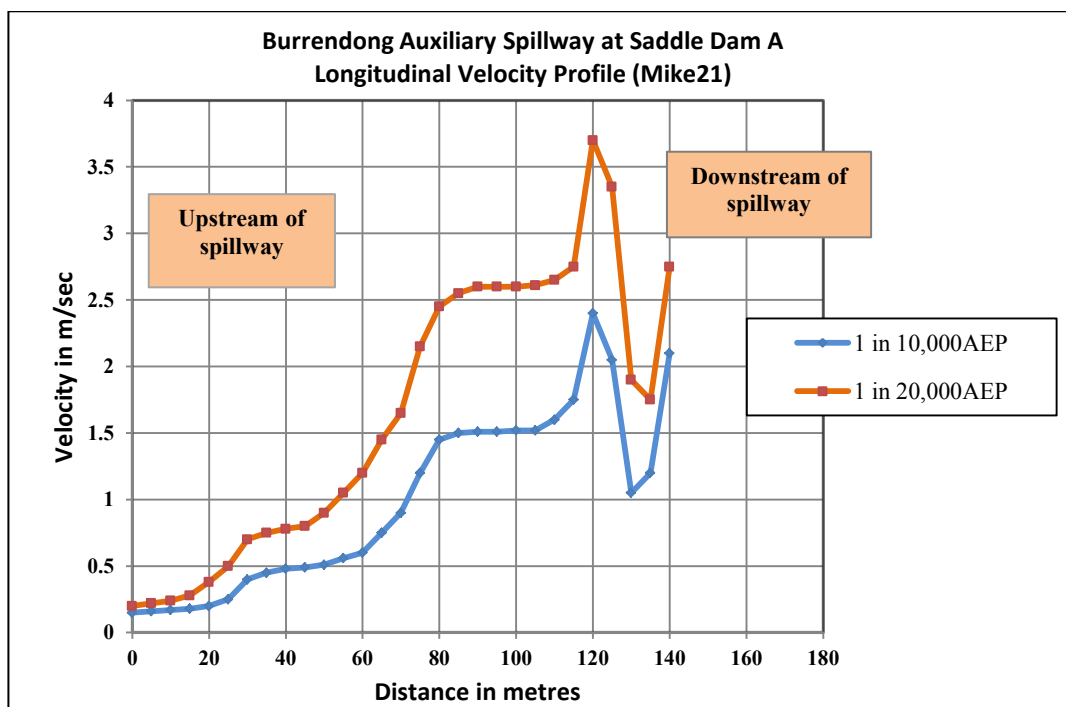


Figure 3: Longitudinal Velocity Profiles for flood events

It can be seen from the above figure that the velocities are low through the auxiliary spillway, and then accelerating through the downstream gully. The spillway entrance sill level was set at RL 354.5m with a base width of 120m. These velocities indicated that there would be significant erosion, if no protection works are incorporated in the design.

Therefore, erosion protection was considered as one of the critical elements in the design of the spillway, which could influence the overall viability of the project in terms of its cost. A conventional solution including concrete apron with anchors was found to be very expensive, whereas a risk based solution was financially very attractive, provided that the erosion risks during flood was acceptable to State Water. Therefore, the assessment of erosion and its potential resistance to uncontrolled flows were investigated to optimise the spillway design.

The assessment of the potential for erosion was carried out by using the method proposed by Annandale (Annandale, G.W, 1995) for erosion of rock masses. The erosion indices are presented in Table 1.

The floor of the auxiliary spillway was founded predominately within moderately weathered meta-sandstone and meta-siltstone. The rock mass was adopted as fractured, and is characterised by a relatively low Rock Quality Designation (RQD) (typically < 50%). A 20-30m wide shear zone comprised of extremely weathered meta-sandstone is inferred to strike parallel to the channel on the right hand side of the channel floor (see Figure 2). The investigations indicated that the shear zone and EW material extends to depths greater than 10m depth below FSL (RL 344m AHD).

Table 1: Erosion indices (EI)

Rock Mass Unit	EI
EW sheared meta-sandstone	0.15-0.55
MW meta-sandstone rock	11-440
SW meta-sandstone rock	20-380

The primary erosion mechanisms were considered to be:

- Surface erosion of exposed rock mass in the unlined sections, including left and right batter faces and channel floor;
- Headcut erosion initiating in the natural gully downstream of the spillway channel and progressively eroding back through the channel floor (shown in Figure 4). Head cut occurs when an erosion gully forms at a point of weakness and flow concentration then flow into the gully and forms a back roller which progressively erodes the face of the gully such that it extends towards the source of flow. The shear zone was considered to be the most vulnerable area to this form of erosion, but it could also trigger elsewhere as well.

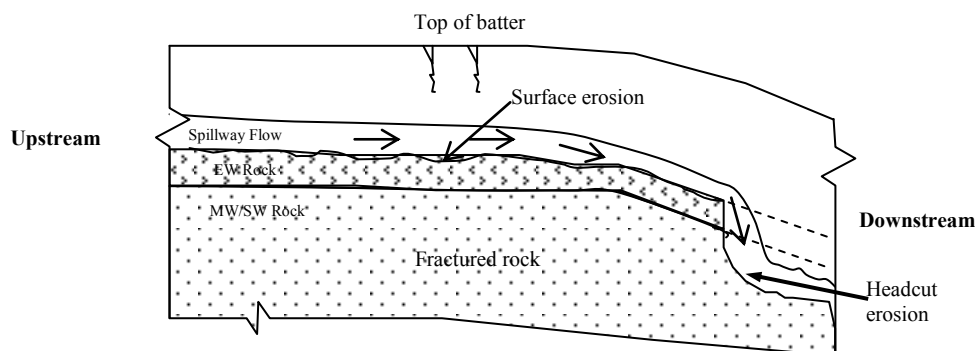


Figure 4: Erosion mechanism at spillway floor

The Stream Power values for various spillway discharges through the auxiliary spillway were based on HEC-RAS modelling, and are provided in Table 2. At the downstream edge of the spillway channel floor, the model predicted locally higher velocities of 2.4m/s and 3.7m/s, respectively for the 1 in 10,000 and 1 in 20,000 AEP flood events (see Figure 3). The Stream Power will also be locally higher in the area of the higher velocities. These locally higher velocities at the downstream end of the spillway floor were considered critical for the surface erosion assessment.

Table 2: Estimated Stream Power Values

Flood Event (AEP)	Surface Erosion		Headcut Erosion
	Velocity (m/sec)	Stream Power (kW/m ²)	Stream Power (kW/m ²)
1 in 10,000	2.4	0.2	12-22
1 in 20,000	3.7	0.6	22-34
PMF	9.0	6.2	>500

4 STREAM POWER ASSESSMENT AND EROSION INDICES

The Stream Power and Erosion Index (EI) values are presented in Figure 5. EI values have been calculated as outlined in Annandale (Annandale, G.W, 1995). It can be seen that in the case of the flood frequencies 1 in 10,000 AEP to 1 in 20,000 flood events, surface erosion of the EW shear zone material is predicted to start occurring, however surface erosion of the MW and SW rock is unlikely to occur including within the area of the locally higher velocity at the downstream end of the spillway channel floor. Head cut erosion is predicted to occur relatively rapidly within the EW material as the EI for this material plots well above the threshold line, as indicated in Figure 5.

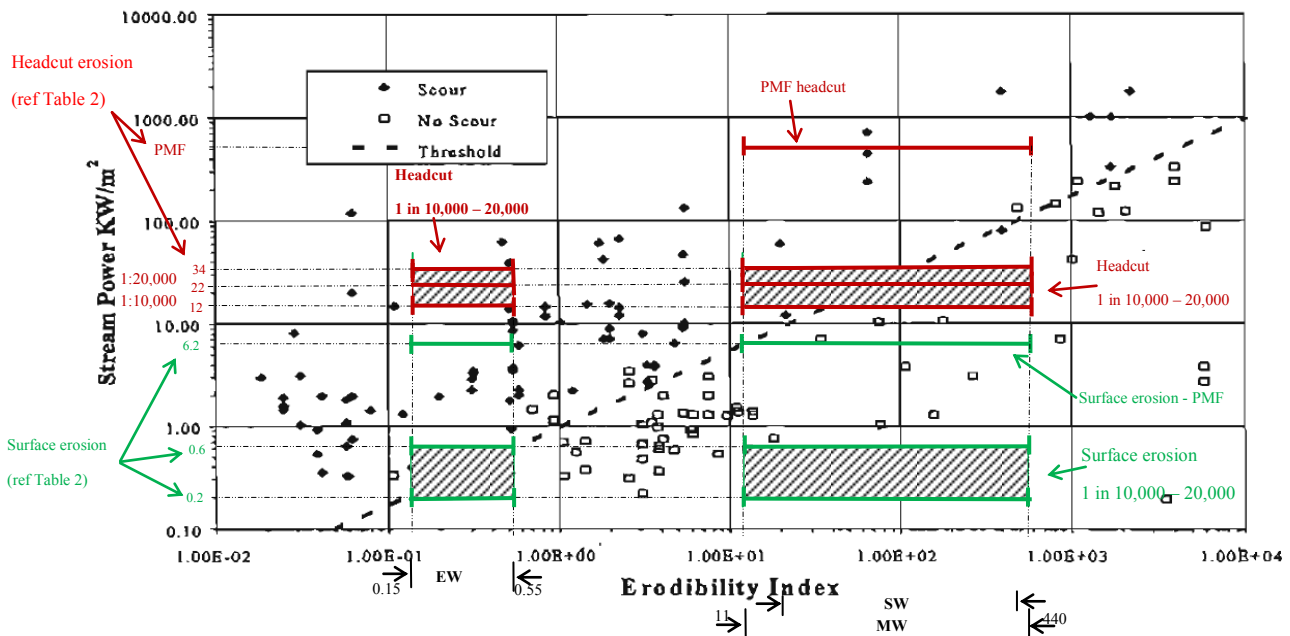


Figure 5: Prediction of Erosion using Annandale (1995)

Head cut erosion is also predicted to occur in the MW and SW rock mass but at a much slower rate based on the EI values plotting on either side of the threshold erosion line. The rate of head cut advancement is discussed further below.

For the PMF event, surface erosion is predicted to occur relatively rapidly within the EW shear zone material, however surface erosion is unlikely to occur for the MW and SW rock. Head cut erosion is predicted to occur relatively rapidly within the EW material and MW/SW rock as the EI for this material plots well above the threshold line for the PMF event.

In order to estimate the rate of advancement of head cut erosion within the unlined earth spillway, NRCS earth spillway erosion model (NRCS, 1997) was utilised. The results of the assessment are shown in Table 3.

Table 3: Estimated Rate of Head Cut Advancement

Flood Event (AEP)	EW Shear Zone		MW Rock	
	Average Rate of Head Cut Advance (m/hr)	Distance of Head Cut Advance (m)	Average Rate of Head Cut Advance (m/hr)	Distance of Head Cut Advance (m)
1:10,000	1.5 – 3.3	36 - 80	0 – 1.1	0 - 24
1:20,000	12 - 16	>400	0 – 2.6	0 - 88
PMF			93 - 145	>>200

For the 1 in 10,000 AEP flood event, the estimated average rate of head cut advancement is 1.5 m/hr to 3.3 m/hr within the EW shear material. The estimated duration of discharge through the auxiliary spillway is 24 hours. The estimated distance of head cut advancement is less than the spillway channel length and so for this event, head cut erosion is unlikely to progress far enough upstream within the EW material or MW rock to cause an uncontrolled release through the saddle feature. For the 1 in 20,000 AEP flood event, the estimated average rate of head cut advance is 12 m/hr to 16 m/hr, and the predicted distance of head cut advancement over the duration of spillway flows (34 hours) is significantly greater than the spillway channel length. Therefore, for the 1 in 20,000 AEP event head cut erosion is likely to occur across the full length of the channel within the EW shear material and lead to an uncontrolled release by deep erosion through the saddle feature. However, head cut erosion within the MW rock is unlikely to head cut across the full length of the channel. For the PMF event, rapid head cut erosion is predicted within the EW shear zone and also the MW rock, and this is likely to lead to an uncontrolled release by deep erosion through the saddle feature.

The following points summarised the findings of the above analysis:

- For the 1 in 10,000 AEP event, the unlined spillway is likely to experience surface erosion and head cut erosion within the EW shear zone material. However, this is unlikely to lead to an uncontrolled release of the reservoir. Surface erosion is not predicted to occur within the MW and SW rock.
- For the 1 in 20,000 AEP event, the unlined spillway is likely to experience surface erosion and head cut erosion within the EW shear zone and MW/SW rock. If left unprotected, then head cut erosion within the EW shear zone is likely to be sufficient to cause an uncontrolled release by deep erosion in the base of the channel floor. However, this is unlikely to lead to an uncontrolled release of the reservoir. Surface erosion is not predicted to occur within the MW and SW rock.
- For the PMF event, significant head cut erosion of the EW shear zone and MW/SW rock is predicted to occur and this is likely to lead to an uncontrolled release of the reservoir by deep erosion of the spillway channel floor.

5 OPTION DEVELOPMENT

Option 1 consisted of placing a layer of rock rip rap protection over the erodible EW shear zone to prevent it being eroded for spillway discharges up to the 1 in 20,000 AEP event. The rip rap layer at the downstream end of the channel would slow the head cut erosion through the erodible EW material. The purpose would be to allow this coarser rockfill to collapse into any head cut erosion features that develop in the gully downstream of the channel to stop or slow the advancement of the head cut. Option 2 consisted of constructing a conventional anchored reinforced concrete slab and downstream cut-off wall in addition to the rip rap protection over the EW shear zone. The concrete slab would be 10m wide, 120m long and 0.3m thick. The slab would also serve as the flow control sill. The downstream cut-off wall would be at least 2m deep into the fractured rock mass. The side batters would also be protected with shotcrete, mesh and rock bolts over a 10m wide zone in line with the concrete slab. Option 2 represents a more conventional approach to providing erosion protection. Option 3 was to use shotcrete with rock anchors as an erosion protection at the floor of the spillway. Option 4 was similar to Option 2 except a deep, structural cut-off wall would be constructed at the downstream end of the slab to prevent head cut erosion through the auxiliary spillway channel for flood events up to the PMF. A deep cut-off wall would be required in this case due to the significant depth of erosion that could develop within the fractured rock mass and EW shear zone under large spillway flows. Investigations indicate the EW shear zone extends at least 10m below channel invert level and probably extends significantly below this. This option was discussed, and it was agreed not to pursue it due to the likely significant construction costs (i.e. \$10M - \$20M), which was considered to be not warranted for addressing the business risks associated with deep erosion of the auxiliary spillway channel.

Options 1 and 2 were selected for further consideration and are discussed below. An assessment of the risks of an uncontrolled release by deep erosion in the base of the spillway channel floor was carried out to select the level of

erosion protection measures for the spillway channel. The key issue from the erosion assessment was whether head cut erosion is likely to initiate at the downstream end of the channel, and then whether there is sufficient time for it to progress over the full length of the spillway channel. This is not considered to represent a dam safety risk as the rate of enlargement of the “breach” through the very broad natural saddle feature would not be sufficient to cause a significant flood wave on top of the already large discharges from the primary and auxiliary spillways. The estimates of the probabilities of the preliminary risk assessment are provided in Table 4.

Table 4: Probability of Uncontrolled Release

Option	Estimated Incremental Cost Over Base Case	Annual Probability of Uncontrolled Release
Base case - No erosion protection measures	Nil	1 in 11,000
Option 1 - Rip rap protection over shear zone	\$0.6M	1 in 40,000
Option 2 - Rip rap protection over shear zone, plus anchored concrete slab	\$2.5M	1 in 180,000

The results in Table 4 indicate that the estimated annual probability of an uncontrolled release is 1 in 11,000 with no erosion protection measures. This probability is dominated by the potential for head cut erosion through the erodible EW shear zone for flood events rarer than 1 in 10,000 AEP. The probability of uncontrolled release can be reduced to 1 in 40,000 if localised rip rap protection measures are provided over the EW shear zone (Option 1). This option provides erosion protection measures designed for the flood events up to 1 in 20,000 AEP event. The estimated construction cost increase for providing the rock rip rap protection works is in the order of \$0.6M. For this option, the residual risk is dominated by head cut erosion within the MW rock mass for events rarer than 1 in 20,000 AEP. Further risk reduction could be achieved by constructing an anchored concrete slab with a cut-off wall at the downstream end in addition to the rip rap protection measures over the EW shear zone (Option 2).

The estimated construction cost increase for Option 2 is in the order of \$2.5M and the probability reduces to 1 in 180,000. This represents more than an order of magnitude of reduction in the business risk compared to that of an uncontrolled release with no erosion protection measures. Option 1 was adopted as the preferred option.

The preferred option for the erosion protection measures that was selected by State Water comprises constructing a layer of rockfill rip rap material over the more erodible shear zone. The design intent of the erosion protection measures are summarised as follows;

- To provide protection over the more erodible Extremely Weathered shear zone materials so that the erosion resistance is more comparable to the moderately weathered rock areas of the spillway floor; and
- To reduce the likelihood of an uncontrolled release caused by excessive erosion within the floor of the spillway channel consistent with the outcomes of the spillway erosion risk assessment described above. The grading characteristics of the rockfill protection layer have been selected by assessing the size of rock particles required to resist erosion caused by large spillway flows passing over a rip rap layer.

6 ACKNOWLEDGEMENTS

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