

Maroon Dam: A multi-analytical approach for a multi-purpose dam

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Consideration of flood mitigation benefits, water supply, irrigation and recreational usage played an instrumental role in developing the proposed upgrade for Maroon Dam to meet dam safety and flood capacity requirements. Maroon Dam is a 47.4 m high zoned earthfill dam completed in 1974. The dam is a multi-purpose reservoir which is now owned and operated by Seqwater and plays an important role in the local community. Key drivers for the proposed upgrade design included embankment stability, foundation concerns, piping, spillway capacity and erosion of the embankment toe.

Six options were reduced to three through a high level screening exercise. A more detailed assessment of the remaining options was undertaken using a Multi Criteria Analysis and a detailed risk assessment. Consideration of the competing uses of the reservoir was critical in the development and assessment of the preferred option. This paper will present the details of the analytical methods used as input for the Multi Criteria Analysis and the detailed risk assessment for the final proposed design option that will meet the requirements of dam safety and flood capacity without impacting on water supply, irrigation and recreational usage.

Keywords: Maroon Dam, Multi Criteria Analysis, Risk Analysis

Introduction

Consideration of flood mitigation benefits, water supply, irrigation and recreational usage played an instrumental role in developing the proposed upgrade for Maroon Dam (Figure 1) to meet dam safety and flood capacity requirements. The dam is a multi-purpose reservoir, which was constructed near Boonah in Queensland, and is now owned and operated by Seqwater. It plays an important role in the local community. Key drivers for the proposed upgrade design included embankment stability, foundation concerns, piping, spillway capacity and erosion of the embankment toe.



Figure 1. Maroon Dam downstream berm and embankment

Six upgrade options were reduced to three through a high level screening exercise. A more detailed assessment of the remaining options was undertaken using a multi-criteria analysis. A detailed risk assessment was used to evaluate the dam's current risk rating and to assess possible upgrade options against the ANCOLD tolerable risk criteria (ANCOLD 2003).

The results of the multi-criteria analysis and detailed risk assessment were provided as input to the Seqwater Portfolio Risk Assessment (PRA) project, which considered flood capacity and safety risks across the Seqwater portfolio of referable dams. This paper will present the details of the analytical methods used as input for the Multi Criteria Analysis, including the detailed risk assessment for the final proposed design option.

Dam description

Maroon Dam was constructed between 1969 and 1974 and has a maximum height of 47 m and a crest length of approximately 457 m. As shown in *Figure 1*, the dam embankment comprises an earth and rockfill section with a central earth core, outer gravel drains and rockfill, with upstream and downstream weighting berms. The dam crest level is at EL 219.78 m AHD and the full supply level is at EL 207.14 m. (*Hereafter, all levels are given as m AHD*)

The spillway comprises an unlined channel excavated through rock on the right bank. A concrete control structure with crest at EL 217.51 m is located part-way along the excavated channel.

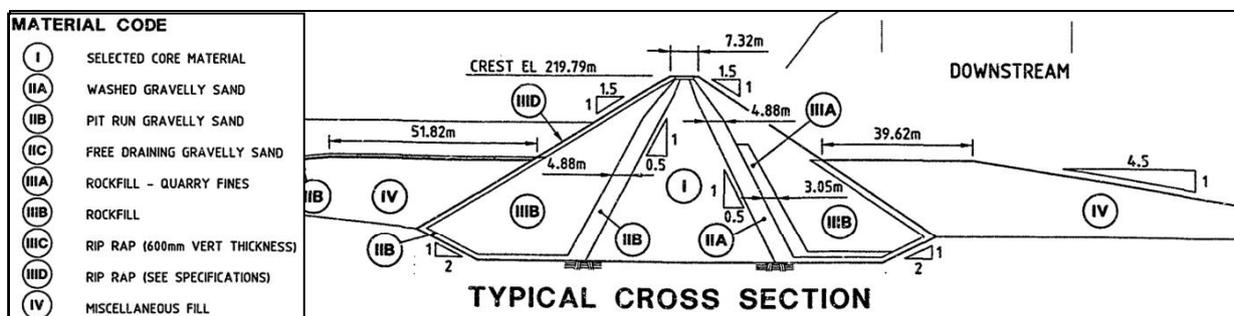


Figure 1. Maroon Dam embankment zoning

The outlet works consists of a low level single inlet tower with a single 3 metre nominal cast insitu reinforced concrete culvert passing through the dam embankment to a valve house located within the embankment and thence via a single 4.4 m nominal cast insitu reinforced 4.4 m x 4.3 m concrete culvert to a downstream outlet structure. The valve house is located within the embankment immediately upstream of the dam centreline. It incorporates two 1219 mm diameter mild steel outlet pipes each controlled by a 1219 mm diameter butterfly valve and a 1067 mm diameter cone dispersion valve.

Based on the maximum population at risk (PAR) of 352, the ANCOLD hazard rating is High A for which the DERM Acceptable Flood Capacity (AFC) guidelines require the spillway to be capable of passing the Probable Maximum Precipitation Design Flood (PMP-DF) as the fallback design flood capacity.

Operational performance

Owing to stability concerns with foundation clay seams and high embankment pore pressures developed during the construction, the reservoir water level has been controlled and was progressively raised after completion in 1974 from EL 201.05 m to the current operational full supply level of EL 207.14 m in 2003.

The reservoir capacity below EL 207.14 m is used for water supply and irrigation while the reservoir capacity between the full supply level EL 207.14 m and the spillway level EL 217.51 m is maintained for flood retention. The reservoir level has been successfully controlled using the outlet works, with the peak reservoir level during floods being EL 210.0 m, which occurred on 11 January 2011. The operational performance was taken into consideration in the probabilistic hydrological analysis used for the risk analysis.

Investigations and modelling

Previous studies and site investigations had identified low factors of safety for the embankment and abutments as a result of weak layers in the foundation. These low factors of safety combined with the risk of piping through the abutments and upper part of the embankment need to be addressed to meet current design standards. In addition, inadequate spillway capacity and the risk of erosion in the unlined spillway chute during a major flood event also needed to be considered further in terms of operational impacts. The operation of the unlined spillway adjacent to the right abutment would result in a significant level of scour at the base of the embankment and deposition downstream. This erosion and consequent deposition could significantly impact on the embankment stability and the post flood performance of the outlet works to draw down the reservoir to the operational Full Supply Level.

During construction, the dam foundation was found to have a number of low-strength clay seams that were continuous, parallel and perpendicular to the axis of the dam, which impact the stability of the dam embankment under flood loading.

Spillway and embankment upgrade works are required to achieve the Acceptable Flood Capacity (DEWS 2013) required by the Dam Safety Regulator. Options to meet the required flood capacity include increasing the downstream weighting berm, control of the reservoir water level by modification of the existing spillway or provision of a new spillway on the left abutment. Water supply and recreational use identified that the Full Supply Level was to be maintained and therefore lowering the storage level was not a viable option. A number of analytical methods were used to validate the design components, including:

- a revised geotechnical model of the foundation for a rigorous assessment of the clay seam strength within the foundation
- a review of the existing instrumentation data to evaluate the foundation and embankment pore pressure response to reservoir level
- stability modelling of the embankment and foundation
- hydrological analysis to derive flood frequency data, including the PMF;
- a hydraulic model of the spillway to evaluate the effect on the embankment stability resulting from erosion of the embankment toe area
- a finite analysis of the outlet conduit to evaluate the effect of the additional loading resulting from raising the berm level.

Geotechnical model of the foundation

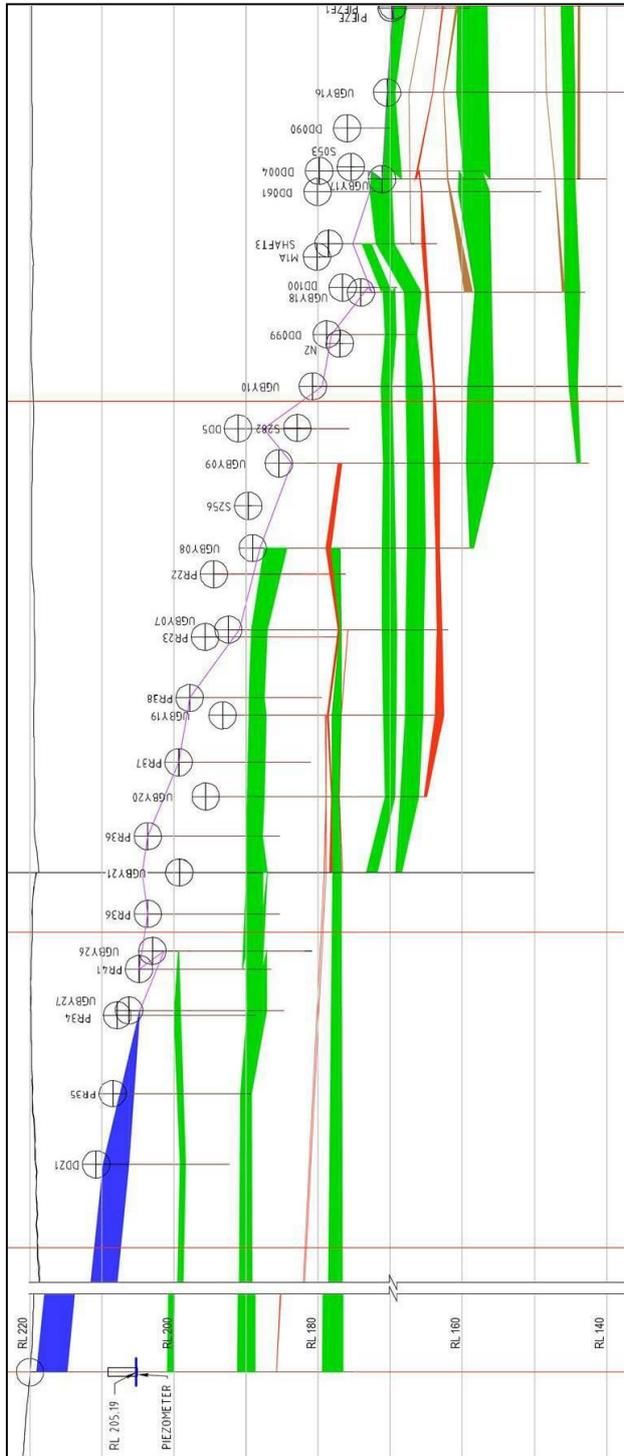
Maroon Dam is situated in the Walloon Coal Measures of Mid-Jurassic Age consisting of shale, siltstone, sandstone and coal seams of terrestrial origin. A detailed three-dimensional geological model of the foundation area was developed using the available design investigation borehole data, grout hole drill data and more recent investigation boreholes. The geological model included approximately 200 data logs which greatly improved the accuracy, however, provided a major challenge in data entry. This model was used to take cross sections along the dam axis and identify potential failure planes used for the stability analyses. The section along the dam centreline is shown below in **Error! Reference source not found.** The green zones shown on this figure represent dolerite sills, the red zones show the clay layers and the blue zones show the surface rhyolite.

The geological interpretation showed the following:

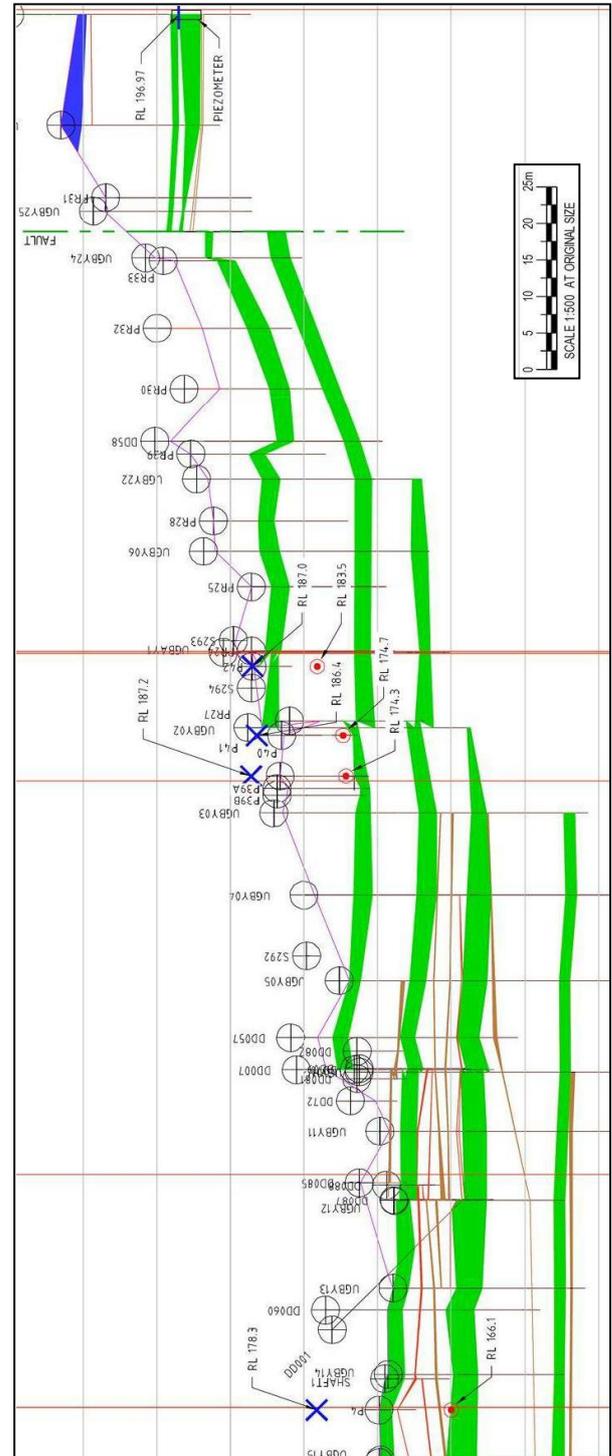
- possible failure surfaces pass through variable material zones,
- shear strengths will vary along failure surfaces,
- previous landslides are present at EL 178 m on the abutments.

Three sections with associated potential failure planes were identified in the geological model and used for stability analyses, as follows:

- Section UA in valley bottom: Potential failure plane EL 165 m (See Figure 7.)
- Section UB on left abutment: Potential failure plane EL 180 m and 185 m
- Section UF on right abutment: Potential failure plane EL 180 m.



Left Bank



Right bank

Figure 3. Maroon Dam geological section on centreline

The site is highly complex with evidence of faulting, igneous intrusions, stress relief and tectonic shearing. The material along the potential failure planes comprises clay within the rock formation, landslide clay and shear material. Statistical analysis of the residual strength data obtained from historical testing of these materials, together with ring shear testing, was used to develop the minimum, expected and maximum strengths for the failure planes. A further analysis was completed using the minimum shear strength with no roughness. This data was used in the slope stability models to derive the factors of safety for the design and for input to the Portfolio Risk Assessment completed for the Seqwater dams, including Maroon Dam.

Instrumentation data

The instrumentation installed at Maroon Dam comprises 55 hydraulic and 10 vibrating wire piezometers, 6 inclinometers, 3 electric settlement installations, 7 V-Notch weirs, and 47 surface movement stations. During recent investigations, Seqwater installed a further 4 inclinometers to improve the understanding of embankment performance.

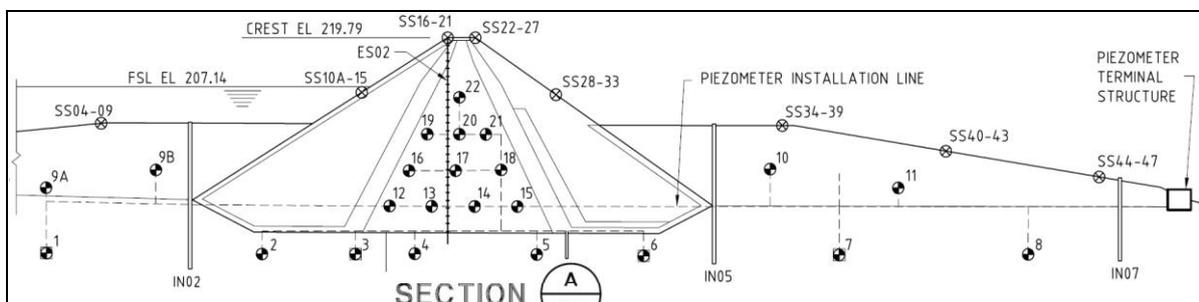


Figure 2. Maroon Dam instrumentation at river bed section

The data from the piezometers at each location of the embankment was used to predict the increase in foundation and embankment pressures for reservoir levels up to the highest design level.

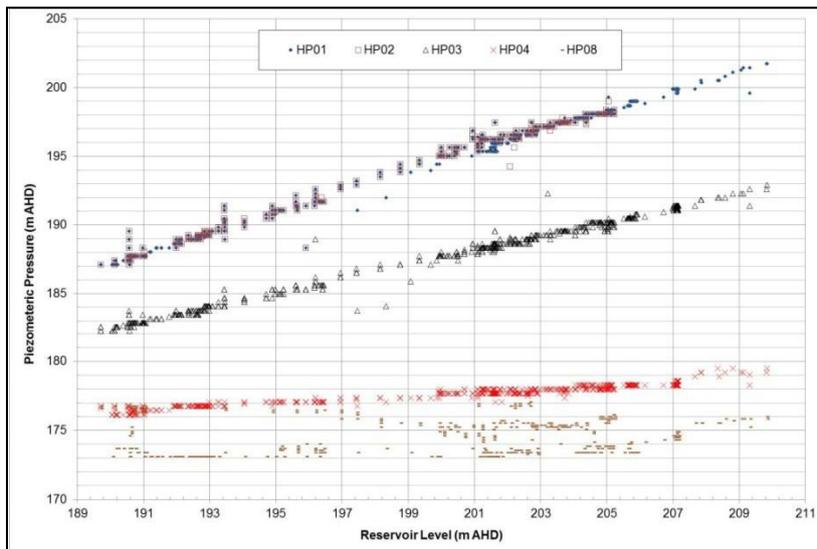


Figure 3. Maroon Dam response of foundation piezometers at river bed section

Analysis of the inclinometer data showed that the embankment was performing as expected with the exception of inclinometer IN07 located on the right abutment. This instrument showed a significant downstream movement from a depth of about 11 m to the surface in 1975, after which the movement was negligible or non-existent below about 2 m. It was confirmed using test pitting in the vicinity of the inclinometer that the movement in the upper 2 m was occurring within the soils in the upper profile.

The settlement data from ongoing monitoring indicated that the dam is performing satisfactorily with the exception of two points. These are located on the downstream berm along the alignment of the outlet conduit. Significantly higher settlements have been observed for these survey points compared with the remainder. This is believed to be associated with reduced compaction of the fill adjacent to the right side of the outlet conduit during construction.

The seepage records (Figure 6) generally showed linear trends of increasing seepage with elevated reservoir levels, with the exception of VNB located on the right abutment. VNB drains the pressure relief holes between about EL 180 m and EL 195 m. As can be seen from this figure, there is a change in the trend line for reservoir levels above EL 200 m. This is likely to be associated with highly fractured dolerite in this area (**Error! Reference source not found.**).

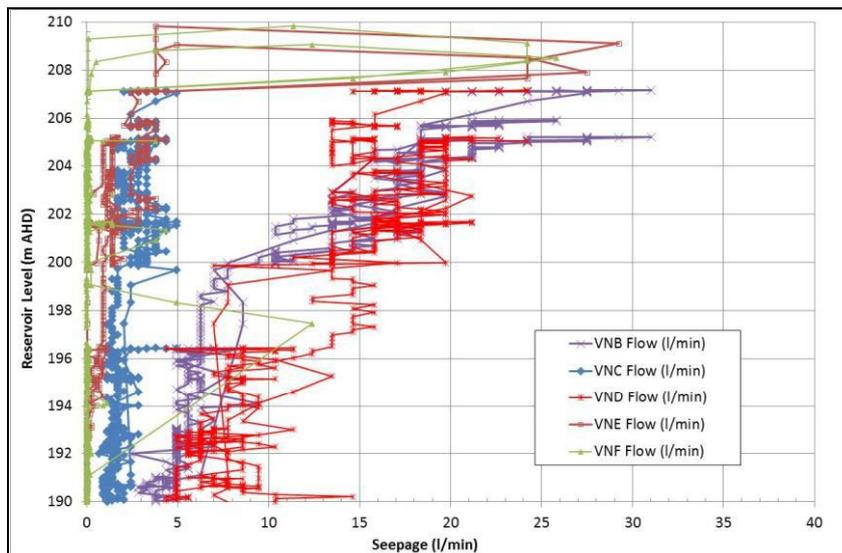


Figure 4 Maroon Dam V Notch Weir Seepage records

Stability modelling

Understanding the shear strength of the weak layers in the foundation was crucial for the slope stability analysis of the embankment. There was considerable uncertainty about the material strength parameters of these weak layers. During the original design and construction, the weak clay layers were tested and found to have a residual strength of 8° . Following expert review, this was increased to 13° to account for shearing through the rock mass in the foundation (Knut 1975).

As part of the upgrade works, the potential failure zones within the foundation were re-evaluated using the geological model and three zones of weakness were identified. Figure 7. presents the geology at the maximum section showing these zones, as follows.

- landslide clay
- clay zones within the claystone, siltstone and mudstone rock
- shear zones.

To assess the material strength for each of these zones, the historical direct shear and triaxial test data was collated and reviewed and additional ring shear tests were completed using samples obtained during the 2013 geotechnical investigation program. Shear strength functions were developed for the average, upper and lower bounds of each zone. The weighting for each zone was estimated for the potential failure planes, as shown for Section UA on Figure 7. . Using the weighting and the average, upper and lower bound strength functions, the strength versus normal stress distribution was estimated for each identified failure surface. Macro scale joint waviness was also considered for the analyses together with a range of flood levels for the slope stability analyses.

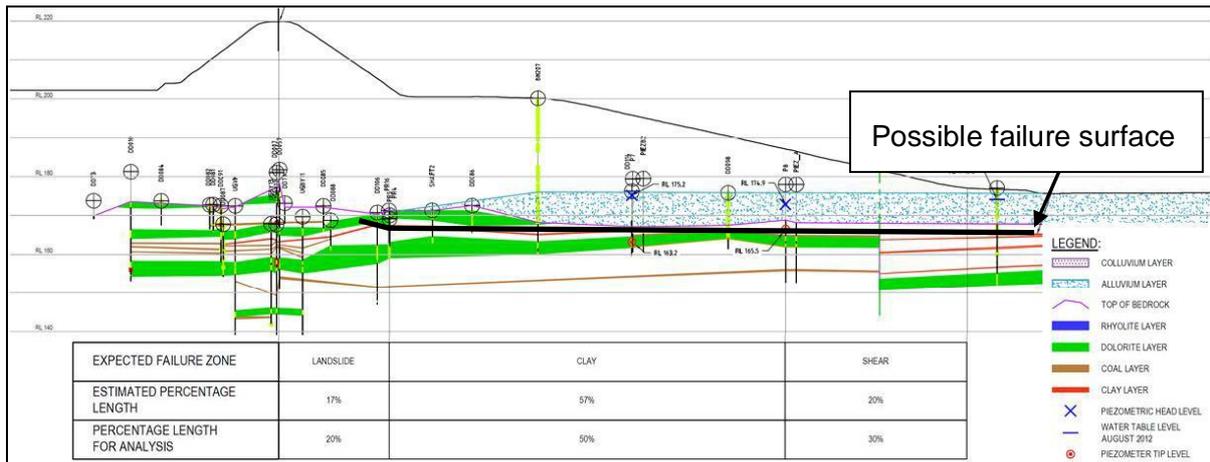


Figure 7. Geology of slope stability section UA, maximum section

The stability analysis results were found to be satisfactory at reservoir levels below full supply level for all strength conditions. However, the factor of safety using the minimum clay seam (weak layer) strength is expected to fall below 1.0 for reservoir levels exceeding EL 215 m.

Hydrological analysis

During the Portfolio Risk Assessment, a reservoir level frequency curve was developed by URS and GHD, as shown on Figure 8. (GHD 2013). As part of the detailed study for the upgrade options, a probabilistic reservoir level frequency curve was developed for two cases. The first was the existing spillway and the second was a proposed scenario for lowering the spillway level from EL 217.5 m to EL 209.75 m to address the upgrade options. The resulting flood frequency curves together with the previous data developed for the PRA are shown on Figure 8. , together with the representative reservoir levels used for the risk analysis.

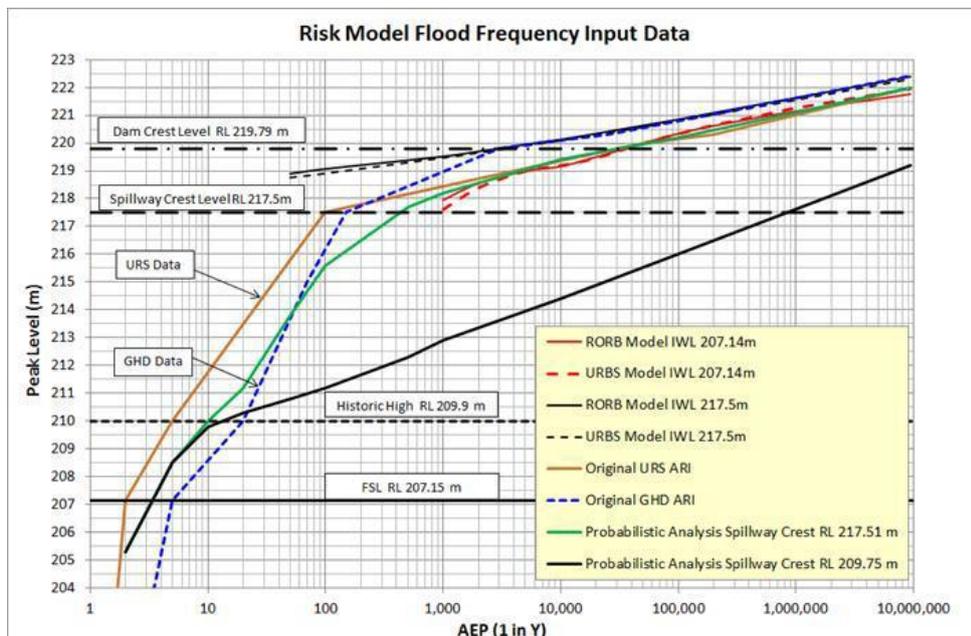
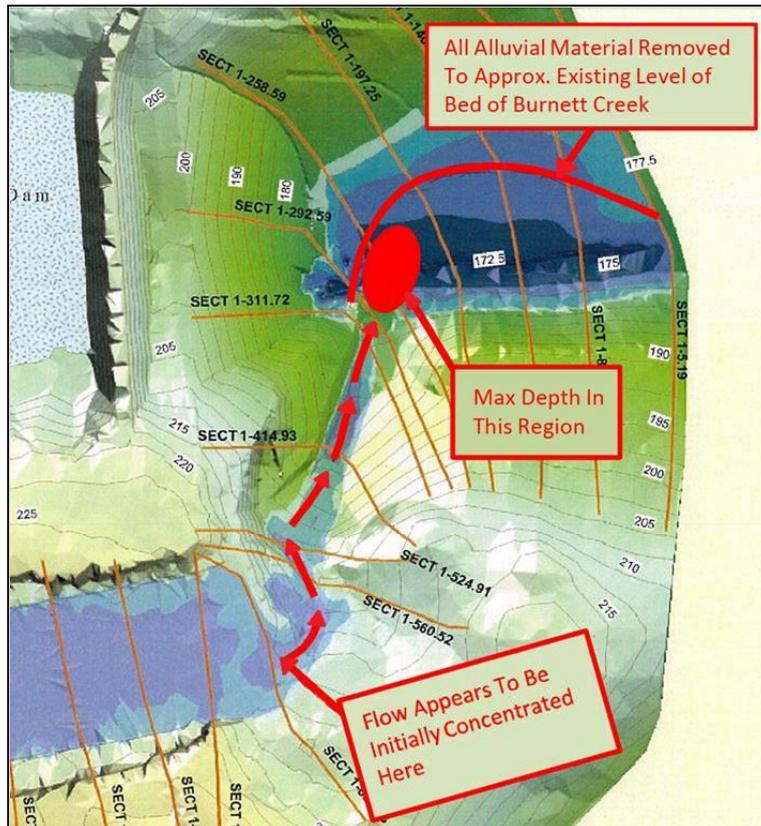


Figure 8. Maroon Dam flood frequency analysis

Spillway operation

The existing unlined spillway drops down a steep unlined channel from EL 217.5 m at crest level to EL 178 m at the river channel (Burnett Creek). It is expected that when the spillway

operates, there will be significant erosion in the return channel and deposition of scoured material further downstream. Erosion assessments indicate a scour hole will form at the outlet of the spillway channel, in the area of the embankment toe. This erosion could lead to a significant reduction in the embankment stability owing to loss of surcharge from the weighting berm on the weak clay zones. A Mike 21 hydraulic model of the spillway chute was used to evaluate the potential erosion and slope stability analyses carried out using the geometry of the eroded cross section. The dimensions of the potential scour hole were estimated using the methodologies outlined in HEC-14 (FHWA 2006) to provide guidance on the maximum dimensions of scour holes in cohesionless soils. In undertaking this analysis, the following assumptions were used:



- The downstream material consists of predominantly sand.
- There is no vertical drop in the discharge from the spillway channel into the Burnett Creek system.
- During a discharge event, clearwater conditions exist.

The schematic of the erosion model is shown in Figure 9, where flow depths are indicated in blue shading. The results indicated that for flows greater than 200 m³/s (a 1 in 2000 AEP flood event) with durations of more than 60 minutes, there is potential for the maximum depth of the scour hole to approach and/or exceed the 9 m depth of alluvial sediments in the river bed area.

Figure 9. Spillway erosion modelling results

Outlet conduit

The design drawings show that the downstream section of the outlet conduit has no shear reinforcement. This, combined with the reduced section thickness at this location, was a cause for concern with the additional loading proposed for a stability berm. The Modulus of Deformation for the foundation siltstone bedrock was estimated using RocLab and found to be approximately 500 MPa. Strand7 was used to analyse the key cross section for the outlet conduit, showing that the conduit floor is overloaded under the maximum section of the proposed stability berm. This resulted in the need limit the height of fill to be used for the new stability berm to about 23.5 m.

Six upgrade options

A high-level screening exercise was carried out to consider all feasible upgrade options, which were then discussed further in an options review workshop with attendees from Seqwater and GHD. The proposed options comprised combinations of embankment raising

through a parapet wall, foundation grouting and pressure relief systems, stabilisation through larger weighting berms, spillway modifications on either the left or right abutments with various invert levels and widths. *Table 1.* presents the options.

Table 1. Maroon Dam upgrade options considered to satisfy safety criteria

ID	Option	Design Water Level (m EL)	Description
A1	Embankment and RHS Spillway	222	Parapet wall to EL 222 m
			Foundation drains and curtain grouting from EL 210 to 222 m
			Ground improvements for embankment; or 25 m wide Weighting berm FOS =1 at Water EL 221.7 m. Borrow material from right abutment of spillway or downstream channel area of spillway to help reduce erosion down the channel when the spillway operates.
			Spillway options to Limit Flood Level to EL 222 m include: - width 30 m with entrance invert at EL 210.5 m - width 50 m with entrance invert at EL 213.0 m Place material in remainder of spillway channel to prevent flow.
A2	Embankment and RHS Spillway	219.5	Foundation drains and curtain grouting from EL 210 to 219.5 m
			Use excavated material for 25 m wide weighting berm giving FOS >1 with water EL 219.5 m
			Spillway to limit flood level to EL 219.5 m - width 50 m with entrance invert at EL 210.5 m or - width 30 m with entrance invert at EL 207.7 m Place material in remainder of spillway channel to prevent flow.
A3 1	Embankment and RHS Spillway	217.0	Foundation drains and curtain grouting from EL 210 to 217.5 m
			Use excavated material for weighting berm FOS >1.0 at Water EL 217 m
			Spillway to limit flood level to EL 217 m - Width 50 m with entrance invert at EL 207.8 m
A3	Embankment and RHS Spillway	215.0	Foundation drains and curtain grouting from EL 210 to 215 m
			Dispose of excavated material for weighting berm FOS >1
			Spillway to limit flood level to EL 215 m - width 55 m with entrance invert at EL 206 m - width 70 m with entrance invert at EL 207.5 m
B1	Embankment and LHS Spillway	222.0	Parapet wall to EL 222 m
			Foundation drains and curtain grouting from EL 210 to 222.0 m
			Use excavated material for 25 m weighting berm giving FOS >1 with Water EL 222.0 m

ID	Option	Design Water Level (m EL)	Description
			Spillway to limit flood level to EL 222.0 m - width 30 m with entrance invert at EL 210.5 m. - width 50 m with entrance invert at EL 213 m Place material in existing spillway channel to prevent flow.
B2 1	Embankment and LHS Spillway	217.0	Foundation drains and curtain grouting from EL 210 to 217.5 m
			Use excavated material for weighting berm giving FOS >1.0 at water EL 217 m
			Spillway to limit flood level to EL 217 m - Width 50 m with entrance invert at EL 207.8 m
B2	Embankment and LHS Spillway	215.0	Foundation drains and curtain grouting from EL 210 to 215.0 m

The selection of the screening criteria and options also considered the following:

- Operation of the reservoir with a reduced FSL of EL 203.5 m to provide additional flood buffering. This was ruled out as a feasible option due to potential impacts on water entitlements for downstream users. Furthermore, it would be difficult for Seqwater to increase usage charges to cover the reduced supply from the dam.
- Wider spillway channel excavations will result in a significant amount of excess material that will need to be disposed of.
- There is a significant amount of uncertainty associated with the left abutment spillway options due to limited geotechnical information.

Options were then assessed with the following criteria using a Multi Criteria Analysis (MCA) to identify the three most suitable for detailed risk assessment:

- business impact,
- constructability,
- cost,
- environmental and social,
- compliance,
- operation and maintenance,
- risk,
- staging, and
- technical merit.

The three options selected for further consideration using the MCA, were as follows:

- Option A1 (Weighted Score Total 645)
Parapet wall, embankment stability improvements and minor spillway protection works for reservoir level of EL 222.4 m.
- Option A2 (Weighted Score Total 490)
30 m wide channel on the right abutment to limit reservoir level to EL 219.5 m.

- Option B1 (Weighted Score Total 495)
30 m wide channel on the left abutment to limit reservoir level to EL 219.5 m.

Three upgrade options - detailed risk assessment

The MCA determined the weighted score for each option based on further detailed assessment of screening criteria considering merits and demerits against a pre-determined scale. The results of the MCA indicated that Option A1 was preferred to Option B1 and then Option A2.

The Seqwater Portfolio Risk Assessment (PRA) was used as a basis to calculate the current societal risk. The partitions used for the risk assessment were derived from the URS/Seqwater, GHD and probabilistic reservoir level data, as shown on Figure 10. The probabilistic reservoir level data was derived from methods using historical seasonal trends. This clearly indicates that the current societal risk for the dam is above the ANCOLD Tolerable Limit and upgrade works are required to lower the risk.

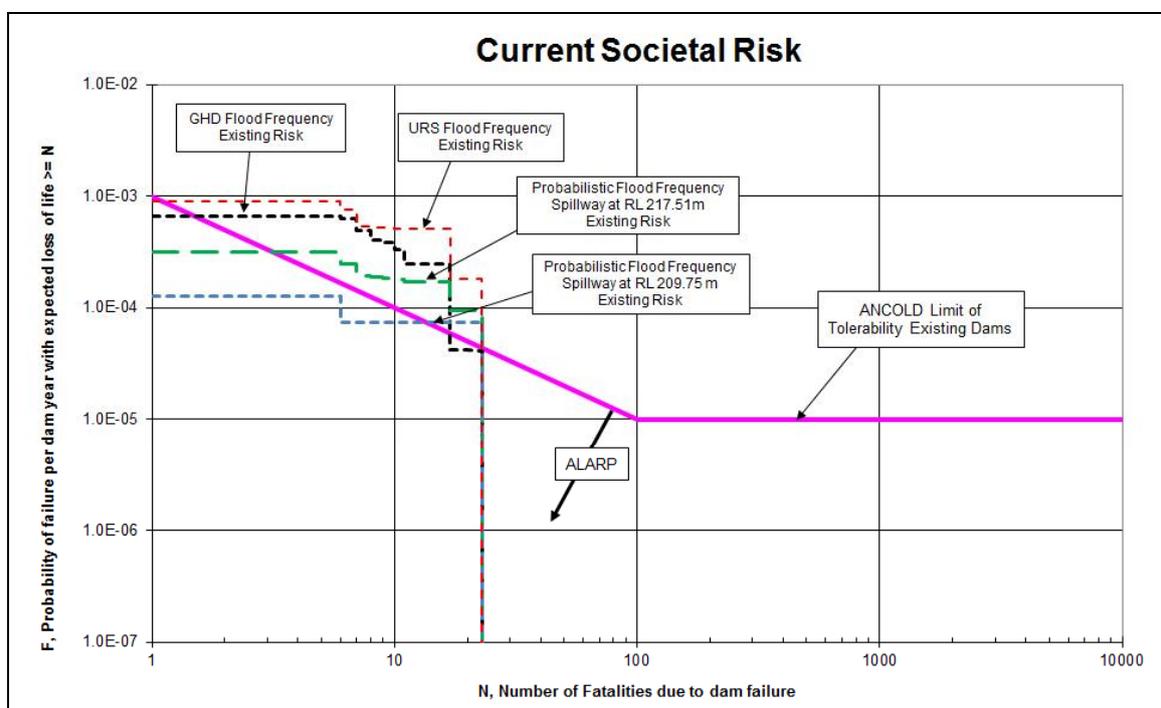


Figure 10. FN curve of current societal risk

The risk analysis produced a range of results that showed some variation between the URS/Seqwater, GHD and probabilistic risk distributions for the reservoir level. It was found, however, that the majority of the risk (70% to 98%) occurs with the partition interval of EL 218.65 m. This level was used to re-evaluate the percentage compliance for flood capacity using the Fallback approach in the Queensland Guidelines. It was found that the spillway is only capable of passing 15.7% of the AFC flood with the starting reservoir level at the spillway crest level of EL 217.5 m, or 32.7% with the starting reservoir level at EL 215 m, which was the level appropriate to the Probabilistic Analysis start level.

The risk analysis was also used to evaluate the upgrade staging and acceptance in accordance with the AFC Guidelines. The analyses showed that the requirement for passing 25% of the AFC flood by 2015 could be achieved with all of the flood distributions with the exception of the URS/Seqwater flood distribution. The existing dam was found to be 60% compliant using the GHD and probabilistic flood frequency data.

The cost estimates and risk analysis data were used to calculate the Cost to Save a Statistical Life (CSSL) for each of the options, as shown in Table 2.

The CSSL values clearly show that the remedial works for the abutment grouting and drainage works together with the berm stabilisation in 2015 is cost effective. The construction of the Crest Wave wall for Option A1.0 or the Spillway for Options A2 and B1 are not cost effective as the CSSL value is well in excess of the \$100M marginal benefit value given in the ANCOLD Guidelines on Risk Assessment (ANCOLD 2003).

Table 2. Cost to Save a Statistical Life summary table for probabilistic flood frequency data

Description	CSSL Cost (\$M)		
	A1.0	A2	B1
2015 Abutment	46	46	46
2025 Berm	195	195	195
2015 Berm and Abutment	87	87	87
2035 Crest Wall	21,464		
2035 Spillway		16,536	13,510

Final proposed design option

The final proposed design was Option A1.0, which will meet the requirements of dam safety and flood capacity guidelines without impacting on water supply and recreational usage. Construction is scheduled as follows:

Stage 1 by 2015

- Abutment Piping Upgrade comprising grouting and pressure relief systems to EL 219.5 m
- Stability Upgrade with a 30 m wide berm using material borrowed from the spillway chute.

Stage 2 by 2035

- Abutment Piping Upgrade comprising grouting and pressure relief systems to EL 222.4 m
- Berm to 47 m wide with toe erosion protection using concrete piles
- Overtopping Upgrade Crest wave wall to EL 222.8 m.

The staging of the works will allow the greatest reduction in risk for the embankment in the interim before the final upgrade is required. The AFC Guidelines specify that Stage 2 will be required for compliance to be achieved by October 2035. Further detailed design work will be required to determine whether Stage 2 will proceed with the crest wave wall or the provision of a right bank concrete lined spillway (Option A2).

Conclusion

The use of a multi-criteria option selection process has allowed the adoption of a solution which considers the multiple uses of the reservoir and surrounding areas.

Additionally the risk assessment process allowed a staged upgrade approach to be adopted meaning deferral of capital expenditure. This approach will allow re-evaluation of the overtopping risk in the future to confirm whether the Stage 2 works are required and which is the preferred option.

Acknowledgements

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