CHAFFEY DAM — AUGMENTATION FOR IMPROVING WATER SUPPLY RELIABILITY AND FLOOD SECURITY UPGRADE

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ABSTRACT

Chaffey Dam, a 54 m high earth and rockfill dam completed in 1979, is situated on Peel River 43 km southeast of Tamworth in NSW of Australia. It provides urban water supplies to Tamworth and irrigators down the river. The dam is prescribed under the NSW Dams Safety Act in the Extreme Consequence category. It is required to safely pass the Probable Maximum Flood (PMF). Currently the dam does not meet this flood security requirement although it is safe for day-to-day operations.

State Water Corporation implemented upgrade works to progressively improve the flood capacity of Chaffey Dam, including the installation of a 1.8 m high parapet wall along the crest of the dam in 2005, and the construction of a 35 m wide auxiliary spillway in the left abutment in 2011. While planning for further upgrade works to enable the dam to safely pass the PMF ultimately, State Water considered the need to improve the reliability of water supplies to Tamworth and Peel Valley irrigators as water demand grows. Black & Veatch was commissioned by State Water to design for flood security upgrade and augmenting the storage capacity of Chaffey Dam from 62 GL to 100 GL. This paper describes the background studies and the design for raising the dam by 6.8 m to increase its flood capacity to the full PMF, and for augmenting its storage to 100 GL by raising the sill level of the Morning Glory Spillway and modifying the auxiliary spillway.

INTRODUCTION

Chaffey Dam, a 54 m high, 430 m long earth and rockfill dam, stores 62 GL of water for urban supply, mainly at Tamworth, and for irrigation down the Peel River.

As an Extreme Consequence dam prescribed under the NSW Dams Safety Act (1978), Chaffey Dam needs to be able to safely pass the Probable Maximum Flood (PMF) with the reservoir full. Currently the dam cannot meet this flood security requirement. As the owner of the dam, State Water Corporation (SWC) has implemented upgrade works to progressively reduce the flood risk posed by the dam. The original flood capacity of Chaffey Dam was equivalent to a flood event with an Annual Exceedance Probability (AEP) of approximately 1 in 50,000. In 2005, a 1.8 m high precast concrete parapet wall was constructed along the crest of the dam. This interim flood risk mitigation measure moderately increased the flood capacity of the dam to a 1 in 100,000 AEP event.

Meanwhile, the local community expressed concern over the future reliability of water supplies to Tamworth and Peel Valley irrigators as Tamworth's water supply demands

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grow. Tamworth Regional Council (TRC) has, therefore, reviewed its long-term strategy for bulk water supplies for Tamworth for improving the security of the Tamworth water supply (HWA 2005). SWC then carried out further study to investigate long-term options for upgrading the flood capacity of Chaffey Dam to full PMF, and to improve the reliability of the water supply system through augmenting the dam (GHD 2007). This further study recommended the construction of a 35 m wide auxiliary spillway in the left abutment of the dam, and raising the dam in stages to improve flood security and augment the storage. Construction of the proposed 35 m wide auxiliary spillway was completed in 2011, increasing the flood capacity of the dam to a 1 in 500,000 AEP flood event. Figure 1 shows the current upstream view of Chaffey Dam after completion of the auxiliary spillway in the left abutment.

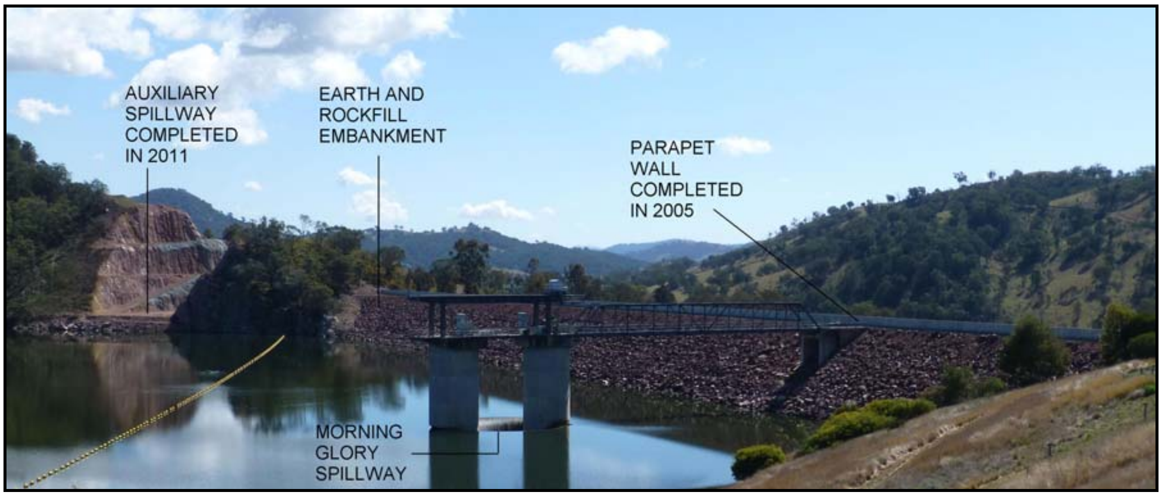

Figure 1. Upstream view of Chaffey Dam.

In 2011, Black & Veatch was commissioned by SWC to carry out the detailed design for the raising of Chaffey Dam to increase its flood capacity to the full PMF, and to augment its capacity from 62 GL to 100 GL. The major components of the upgrade works include:

1. raising of the embankment dam by 6.8 m;
2. raising the sill level of the Morning Glory Spillway by 6.5 m to raise the Full Supply Level (FSL) of the dam and hence augment the storage to 100 GL;
3. raising the work platform of the Morning Glory Spillway by 6.8 m to match with the raised embankment crest level, including raising and extending the access bridge to the Morning Glory Spillway;
4. removing the existing 4 m high fuse plug embankment in the auxiliary spillway and replacing it with two sections of 8.2 to 8.9 m high fuse plug embankments; and
5. realigning bridges and sections of roads along the reservoir rim to levels above the raised FSL.

The cross-section of the raised embankment dam in Figure 2 shows the first three components of the proposed upgrade works.

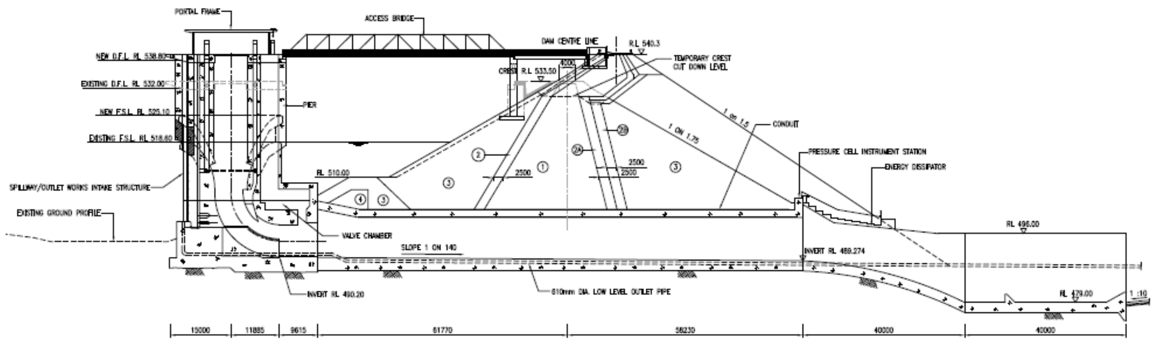

Figure 2. Cross-section showing raising of the embankment, the Morning Glory Spillway and the access bridge.

This paper describes the background studies and the design of the key components of the dam raising works by Black & Veatch.

RELIABILITY OF WATER SUPPLY AND LONG-TERM UPGRADE OPTIONS

Demand Projection

Extrapolation of the census data gave a projected population for Tamworth of 46,000 by the year 2033 (GHD 2007). Past statistics indicated that the demand per person and per dwelling was influenced by annual rainfall. The demand per person decreased as rainfall increased and varied between 225 m^3 to 280 m^3per annum. Research indicated that climate change would likely cause increase in temperature in the Tamworth region. The effects of climate change on rainfall were unsure. It was, however, anticipated that increase in temperature would result in an increased demand for irrigation water to maintain lucerne production and productivity in the dairy industry.

Regional Water Supply Augmentation Options

The study on reliability of water supply considered a number of regional water supply augmentation options, including

- augmenting Chaffey Dam from 62 GL to 80 GL, 100 GL and 120 GL;
- stormwater harvesting;
- construction of new storages;
- regional water transfer from nearby sources, such as increasing supply from the 6.3 GL Dungowan Reservoir via the 54 km Dungowan pipeline; and
- increasing extraction from Peel River.

Reliability of Water Supply

Assessment of the reliability of water supply was based on standard of service for NSW town water supplies defined by the Department of Water and Energy (DWE), and consultation with the irrigators (GHD 2007). The four criteria adopted were:

- water restrictions should be imposed no more than 5% of the time
- restrictions should be imposed no more frequently than every 10 years on average
- the carry over reserve should be able to supply restricted demand, which is equivalent to 20% less than normal demand, during the worst drought on record.
- a minimum reliability level for general security entitlements for irrigators in the Peel Valley was defined as a 70% probability of announcing an 80% water allocation on the 1st of July each year.

The Integrated Quantity and Quality Model (IQQM), a hydrological modeling tool developed by the Department of Natural Resources (DNR) for evaluation of water resource management policies, was used to forecast yield and to assess the ability of the water supply system to meet the needs of water users under possible future Tamworth high security water demand levels and management rules. Modeling was undertaken for Tamworth demand levels of 10, 12, 14, 16.8 and 20.5 GL per annum to represent a range of urban growth scenarios. The key findings of the supply reliability study were:

- comparing the costs of various options indicated augmentation of Chaffey Dam was the most cost effective option to augment water supplies within the Peel Valley;
- augmentation of Dungowan Reservoir was not feasible due to unfavourable geotechnical conditions at the left abutment of the dam; and
- allowing for contingencies such as climate change or the decommissioning of the 54 km Dungowan pipeline, which had limited flow capacity and was in poor condition requiring high level of maintenance, would require at least a 100 GL augmentation in the storage capacity of Chaffey Dam. The augmentation would accommodate high projected growth in Tamworth water demands.

Selected Long-Term Options

The long-term options study (GHD 2007) investigated more than 50 upgrade options, and recommended a final set of 8 upgrade options, including options for flood security upgrade only and for both flood security upgrade and storage augmentation. In order to reduce the flood risk posed by Chaffey Dam as quickly as possible, SWC implemented the first stage of the upgrade works by constructing the 35 m wide auxiliary spillway in the left abutment of the dam to increase the flood capacity of the dam to a 1 in 500,000 AEP flood event. Immediately after completion of the auxiliary spillway in 2011, SWC secured funding for implementing the final stage of the flood security upgrade works that would also augment the dam to 100 GL.

DESIGN OF EMBANKMENT DAM RAISING

Confirming the Height of Dam Raising

Black & Veatch carried out a series of reservoir flood routings which indicated that the storage would rise to a maximum level of RL541.50 m during the PMF. Analysis of wave run-up and wind set-up suggested that a minimum freeboard of 0.6 m above the maximum reservoir level would be adequate. Therefore, to achieve a raised crest level at RL542.10 m, the embankment dam would need to be raised by 6.8 m.

Dam Raising Options

Sixteen dam raising options were studied which could be grouped into four categories:

1. pure vertical raising – achieved by the construction of a reinforced earth block on top of the existing embankment crest.
2. combined vertical raising and downstream buttressing by rockfill – achieved by placing rockfill on the downstream slope to partly raise the height of the embankment, and then constructing a vertical reinforced earth block on the crest of the raised embankment. Options with different height of vertical raising by up to 5 m have been investigated.
3. pure downstream buttressing by rockfill – achieved by placing rockfill on the downstream slope to raise the height of the embankment. Options with different slope gradients varying from 1 in 1.50 to 1 in 1.75 have been investigated.
4. Upstream and downstream buttressing by rockfill – achieved by placing rockfill on both the upstream and the downstream slopes to raise the height of the embankment. Options with different downstream and upstream slope gradients varying from 1 in 1.50 to 1 in 1.75 have been investigated.

Stability and Deformation Analysis

Preliminary slope stability analyses using Morgenstern and Price Method (1965) were carried out on the 16 dam raising options considering the following loading scenarios:

- steady state seepage with storage at raised FSL at RL525.1 m
- steady state seepage with storage at PMF level at RL541.5 m
- end-of-construction condition before increasing storage to the raised FSL
- rapid drawdown condition, including the following two scenarios
  - draw down from PMF level to raised FSL
  - drawdown from the 1 in 10,000 flood level to the raised FSL, assuming breach of the fuse plug
- Operating Basis Earthquake (OBE) with reservoir at raised FSL
- Maximum Design Earthquake (MDE) with reservoir at raised FSL

Results of the above slope stability analyses indicated that all the dam raising options, with the exception of pure vertical raising, would have adequate slope stability. Among the various options, those involving downstream rockfill buttressing were identified as relatively robust and causing less environmental problems during construction.

Since the costs of embankment raising would be highly dependent on the design slope gradient of the downstream rockfill buttress, and that the results of static and seismic slope stability analyses suggested adequate factor of safety could possibly be achieved by steepening the gradient of the rockfill buttress to 1 in 1.5, further detailed slope stability analyses were then carried out to refine the design gradient of the rockfill buttress. As the MDE load scenario was found to control the embankment design, more sophisticated deformation analyses using finite element techniques were carried out to model the effects of construction and the effects of the MDE using the time-history approach. Results of the deformation analysis indicated that both static and earthquake-induced stresses and deformations in the raised embankment would be within acceptable limits.

Figure 3 shows the results of pseudo-static slope stability analysis of the selected dam raising option for the MDE loading, whereas Figure 4 shows the results of deformation analysis of the embankment for the MDE loading using time-history analysis.

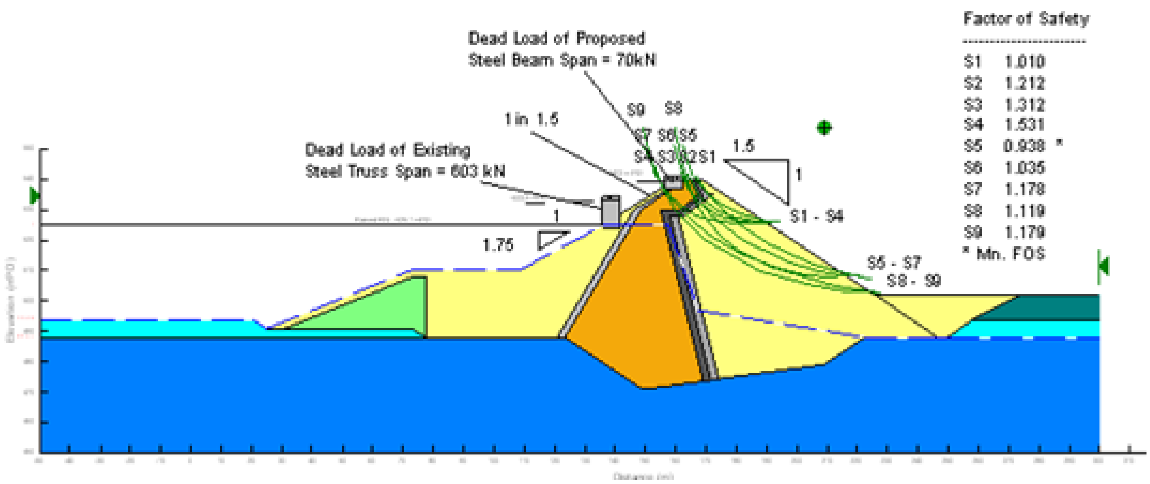

Figure 3. Results of pseudo-static slope stability analysis for MDE (1 in 10,000 AEP) loading.

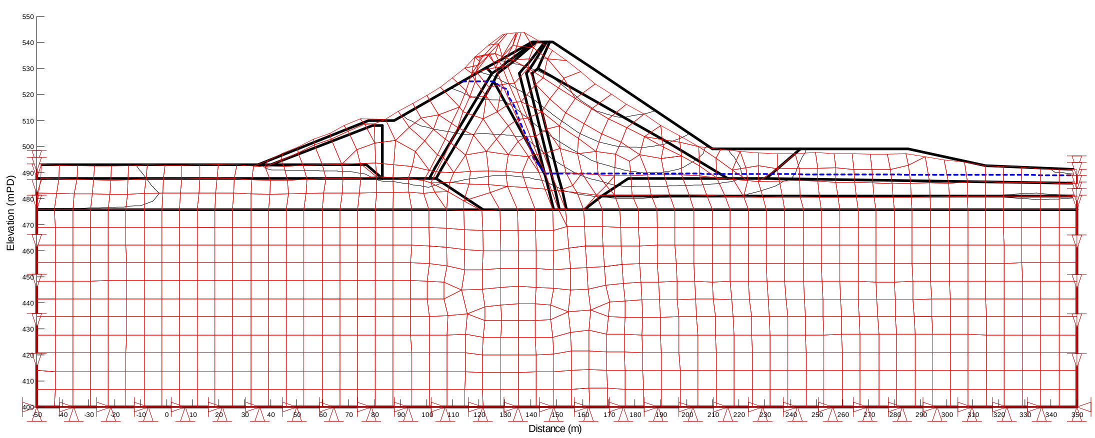

Figure 4. Maximum relative horizontal displacements using horizontal and vertical acceleration time histories from the Chi-Chi Earthquake matched to the MDE horizontal target spectrum of Chaffey Dam site.

Embankment Zoning Arrangement

In order to achieve good continuity between the existing embankment and the new construction, the top part of the existing embankment is to be cut down by 3 to 4 m to approximately RL530.0 m to expose the clay core and clean filter materials before the various zones (core, filters, rockfill, etc.) will be raised to their required height. Cutting down the embankment crest to RL530.0 m during construction will significantly increase the risk of overtopping. To minimize the risk of overtopping during construction, it is proposed to advance the construction of the downstream rockfill buttress to at least RL535.3 m (i.e. the existing parapet wall crest level) before commencing excavation in the embankment crest. With a proper construction emergency plan in place and the use of a geo-membrane to protect the upstream face of the raised rockfill buttress, the flood risk of the existing dam will be controlled within tolerable limit during excavation in the embankment crest. Figure 5 shows the proposed zoning details at the connection between the old and new construction at the existing embankment crest which take into account the advancement of the downstream rockfill buttress up to RL535.3m before the core and the filter zones are raised.

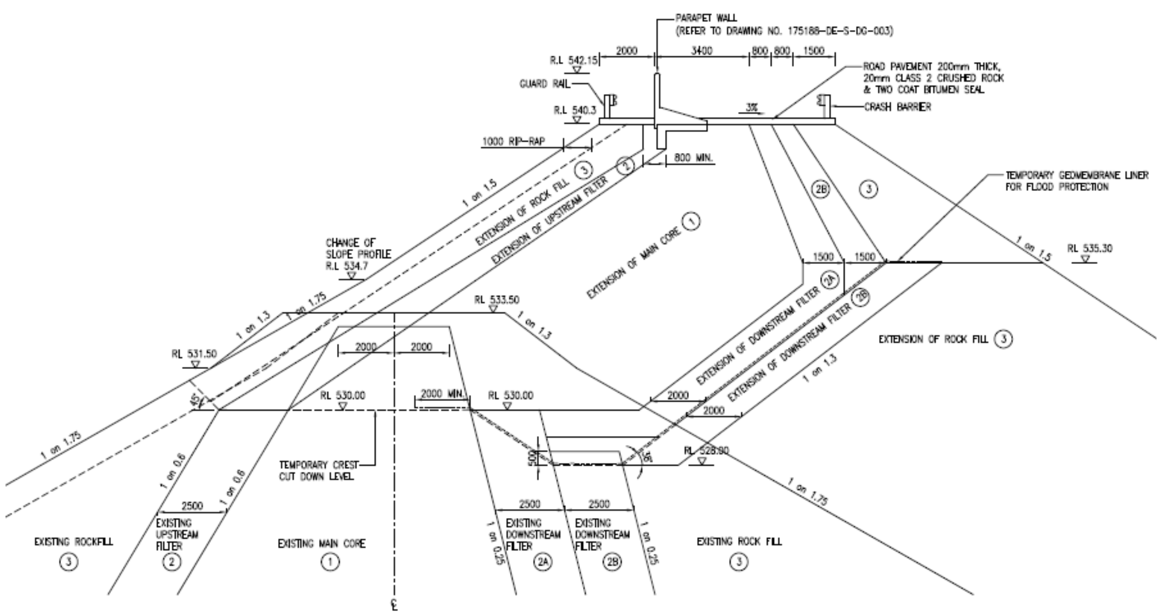

Figure 5. Proposed zoning details between old and new construction near the existing embankment crest.

RAISING OF MORNING GLORY SPILLWAY

Design Profile of the Raised Morning Glory Spillway

Augmenting the storage capacity to 100 GL would require raising the sill level of the Morning Glory Spillway by 6.5 m from RL518.6 m to RL525.1 m. As part of the processes of routing floods through the reservoir in order to assess the peak flood level in the reservoir, the stage-discharge ratings of the Morning Glory Spillway, before and after it is raised, and the auxiliary spillway have been reviewed. This also involved a detailed review of the past hydraulics studies, including the physical model testing for the Morning Glory Spillway carried out in the 70s and the late 80s. Black & Veatch identified some incompatibility between the stage-discharge rating function that had been adopted in recent years for the evaluation of proposals for providing an auxiliary spillway, for raising of the dam and for installation of a fuse plug in the auxiliary spillway. To avoid the incompatibility, Black & Veatch recommended modifying the design of the raised Morning Glory Spillway profile so that its stage-discharge rating curve would be a close match to the rating function used in the design of the upgrade works which were already completed. The modified design profile of the raised Morning Glory Spillway would

- avoid the need for aeration above the throat;
- retain the proposal for adapting the existing air inlets so that they emerge in an overhang directly below the throat; and
- achieve a close match to the transposed existing throat rating.

Black & Veatch's proposed modified profile of the raised Morning Glory Spillway is as shown by the red curve in Figure 6(b) which is based on the Waterways Experiment Station standard spillway profile. For comparison, the proposed profile in previous studies is shown in Figure 6(a).

Computational Fluid Dynamic Modelling

In order to replicate, in the raised spillway, the approximate estimated discharge capacity of the present Morning Glory Spillway, it was necessary to have a reliable means of determining the contraction characteristics of the raised throat. Empirical data on the subject had been used to derive the required throat diameter. Computational Fluid Dynamics (CFD) analyses were then used to corroborate the contraction coefficient of the throat and thereby confirm or adjust the throat diameter.

The additional objectives of the CFD analyses were to:

- corroborate the empirical rating for the rim of the raised morning glory spillway when under weir control;
- investigate the effect of weir submergence at the transition from weir control to throat control;
- demonstrate the flow conditions that will occur under both weir and throat control;

- provide contact pressures between the water and the inner profile of the morning glory spillway, to aid in an assessment of the cavitations potential in the new morning glory spillway;
- provide visual representations of the above; and
- provide data on flow velocities and pressures along the inner surface of the raised spillway for assessing the potential for erosion on the spillway surface due cavitations.

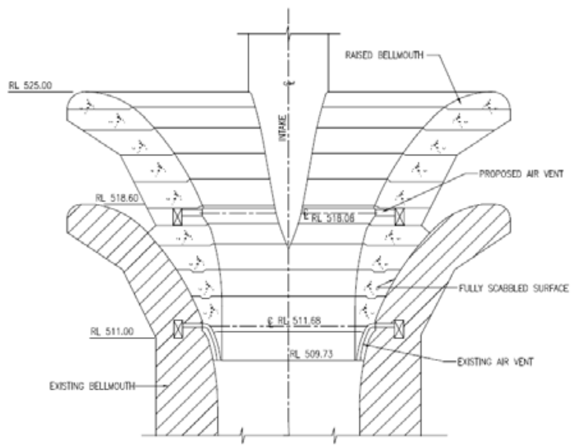

Figure 6(a). Proposed raised profile of Morning Glory Spillway in previous studies.

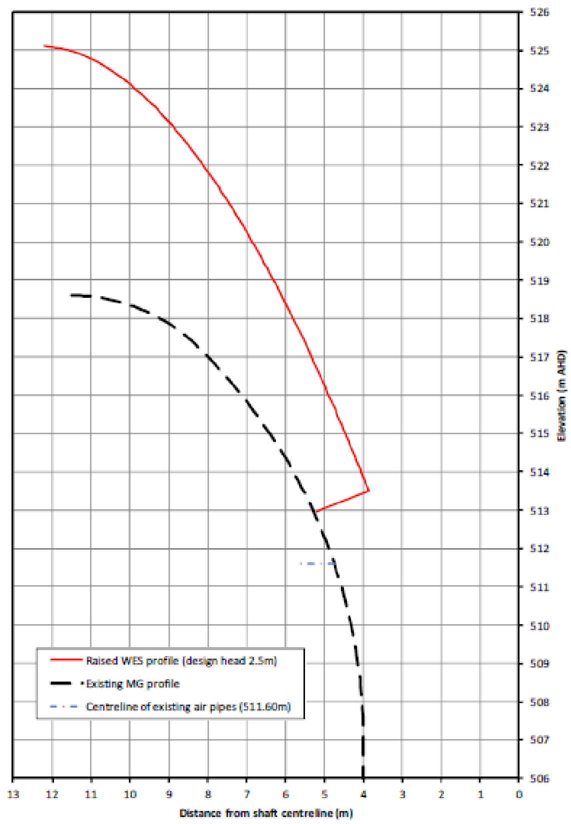

Figure 6(b). Black & Veatch (2013) proposed profile (red curve) for the raised Morning Glory Spillway.

Results of CFD modeling indicated that raising the overflow level of Chaffey dam by 6.5 m would result in modest increases in flow velocities at key locations in the Morning Glory Spillway shaft, including at the bend at the upstream end of the tunnel, but these would not be sufficient to make a material change to the cavitations risks. Figure 7 shows examples of some findings of the CFD modeling.

RECONFIGURATION OF THE FUSE PLUGS AT THE AUXILIARY SPILLWAY

Design of Fuse-Plug Arrangement

In order to lower the flood risk as quickly as possible, SWC constructed an auxiliary spillway in 2011 before funding for augmenting the storage was confirmed. The auxiliary spillway channel sill level is at RL525.85 m which is 5.25 m lower than the sill level proposed in the previous options study (GHD 2007). This gives the auxiliary spillway higher flood discharge capacity and provides as much interim flood protection as possible to the dam. Subsequently SWC decided to maintain the auxiliary spillway sill level at RL525.85 m in the final dam raising works. As a consequence of the additional height of the fuse plug above sill level with augmented storage, breaching of a full width fuse plug would release considerably more water downstream than for the current fuse plug.

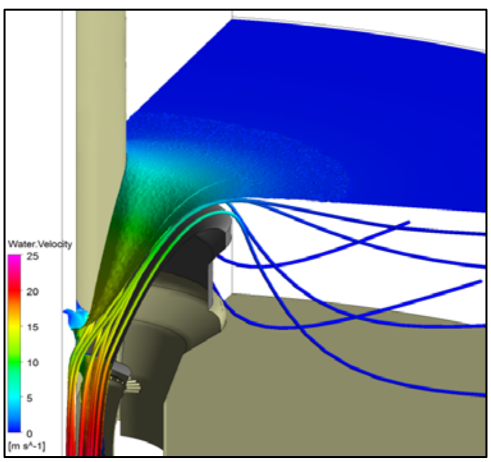

Figure 7(a). Weir control flow at 650 m^3/s.

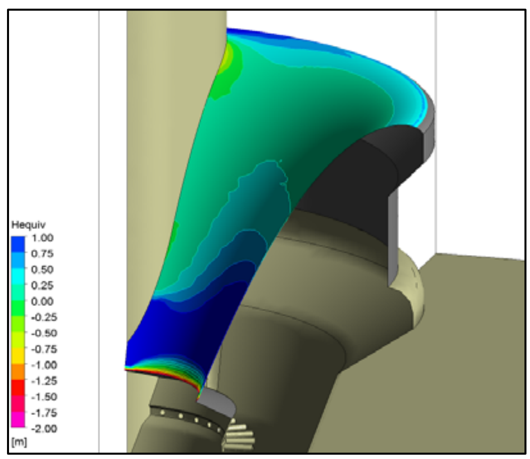

Figure 7(b). Pressure distribution under weir control flow at 650 m^3/s.

SWC intends to stagger the flood release through the auxiliary spillway by dividing the fuse plug across the 35 m auxiliary spillway into two sections, breaching at the 1 in 10,000 and 1 in 20,000 AEP events. The intention is to minimise the flood release when the first section of the fuse plug breaches during the 1 in 10,000 event, while avoiding an excessive increase in the maximum flood level during the PMF.

A range of ten different fuse plug configurations have been tested to investigate the sensitivity of reservoir peak levels and outflows to the relative width of the two fuse plugs; and the flood events they breach. A Mike-11 model of the Peel River and its tributaries downstream of Chaffey Dam was used to route the outflow hydrograph from Chaffey and assess flood depths in the downstream valley.

In conclusion, the design option that meets the design objective of minimizing flood discharge during the first breach of the fuse plug in a 1 in 10,000 event is defined by a 10 m wide fuse plug with trigger level of RL533.78 m and a 23.5 m wide fuse plug with trigger level of RL534.49 m. This design will result in a peak reservoir level reaching RL541.5 m during the PMF.

Computational Fluid Dynamic Modeling and Assessment of Erosion Potential

The key consideration affect the design of the auxiliary spillway cutting is that it should allow for possible major maintenance or repair due to auxiliary spillway discharge, which will occur very infrequently. In order to assess the potential for erosion of the sides and floor of the unlined auxiliary spillway channel, a series of CFD modeling were carried out to study the flow regime along the auxiliary spillway. Quantifying the precise behavior of rocks in relation to high flows and erosion is difficult because of the varied parameters involved, such as rock strength, degree of weathering, discontinuity configuration, joint characteristics and the like. However, the Erodibility Index (EI) method (Annandale 1995) provides a means to assess erodibility potential. The exposures along the auxiliary spillway channel are recognized as:

- Jasper (ferruginised chert) and chert, sometimes interbedded with siliceous siltstone. These rocks are slightly weathered, extremely high strength, and massive to variably fractured.
- Siltstone with some spilite, highly weathered to residual soil, low strength or weak, and highly fractured with some interbedding of stronger beds of siliceous siltstone and jasper or chert that may be of very high strength. The siltstones are covered with colluvial soil on the valley slope towards the downstream end of the spillway.

Results of CFD modeling indicates that the unit stream power will reach the maximum value of 5.3 kW/m^2 when the first section of the fuse plug breaches during a 1 in 10,000 AEP flood. A slightly lower stream power will occur in the event of the PMF (Figure 8). The assessed unit stream power is compared to critical threshold stream power required to initiate erosion listed in Table 1. The capability of the jasper and chert unit to resist erosion indicates that, in the event of the auxiliary spillway carrying flows from extreme events any erosion that does occur is unlikely to threaten the safety of the dam itself. Depending on the acceptability of the erosion profile towards the downstream end of the spillway and beyond it to the valley floor, there may be a need for remedial works to the downstream end of the channel after an extreme event.

Table 1. Erodibility Index and Critical Threshold Stream Power.

Geological unit	Erodibility index (EI)	Critical threshold stream power required to initiate erosion
Jasper and chert Extremely high strength	1400–4000	200–450 kW/m^2
Siltstone Extremely low to medium strength High to very high strength	0.2–0.75 8.5–61	0.3–5 kW/m^2

CONCLUSIONS

Chaffey Dam is classified as an Extreme Consequence dam which is required to be able to safely pass the PMF. Currently the dam cannot meet this flood security requirement. SWC has been implemented upgrade works to progressively reduce the flood risk posed by the dam. In 2005, a 1.8 m high parapet wall was constructed along the crest of the dam to increase the flood capacity of the dam to a 1 in 100,000 AEP flood event. In 2011, the flood capacity of the dam was further increase to a 1 in 500,000 AEP event by constructing a 35 m wide auxiliary spillway in the left abutment. The auxiliary spillway forms part of the final upgrade scheme for increasing the flood capacity of Chaffey Dam to the full PMF, and augmenting the storage from 62 GL to 100 GL. The other key components of this final upgrade scheme include raising the embankment dam by 6.8 m, raising the sill level of the Morning Glory Spillway by 6.5 m to increase the FSL, modifying the fuse plug embankments in the auxiliary spillway, and realigning sections of roads and bridges. Construction of these upgrade works is ready to commence.

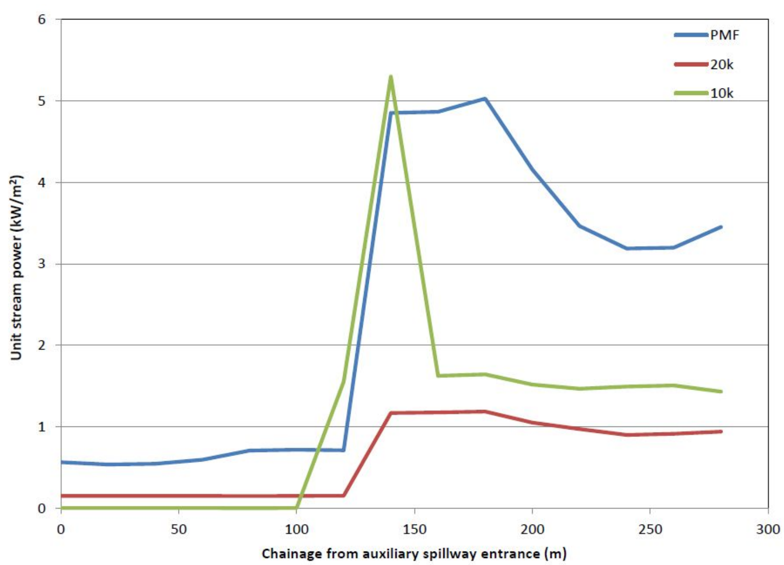

Figure 8. Unit stream power derived from the results of CFD modeling of flow through the auxiliary spillway.

ACKNOWLEDGEMENTS

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