A Unique and Holistic Approach to the Erodibility Assessment of Dam Foundations

Richard Herweynen and Colleen Stratford

Principal Dams Consultant, Hydro Tasmania Consulting Pty Ltd
Dams Consultant, SMEC Australia Pty Ltd

Assessing the potential for erosion of foundation rock downstream of a spillway is a problem faced on many dams, whether new or existing. The problem is made particularly difficult not only due to the uncertainty in determining the erosion potential of the rock, but also due to the variable hydrologic characteristics of flood events.

The selected spillway option for Wyaralong Dam comprises a centrally located primary spillway with a secondary spillway located on the left abutment. A stilling basin energy dissipater is provided at the toe of the primary spillway. Downstream of the secondary spillway, an apron channel will direct flows back to the stilling basin. However, for flood events larger than the 1 in 2000 AEP event, the capacity of the secondary spillway apron is exceeded and flows spill out across the left abutment of the dam towards the river channel. Erosion of this left abutment was viewed to be a potential dam safety issue, and as such, careful consideration was required during the design stage to determine the acceptability of this spillway arrangement.

In order to provide structure to a problem which often relies solely on engineering judgment, a decision process was developed, taking into consideration some of the more definable aspects of the problem. These aspects included the geological characteristics, the initial hydraulic characteristics, the flood duration, the nature of erosion should it occur and the stability of the dam. This paper describes the decision process and methodology used at Wyaralong Dam to determine the acceptability of erosion. This paper will present the process in a way that it can be used by others in future dam projects, both new and upgrades.

Keywords: *Dam, Erosion, Spillway, Foundation, Secondary Spillway*

1. Introduction

A dam safety review of a concrete gravity dam using a risk based assessment framework may lead to the question – can the abutment handle some degree of overtopping? If this can be demonstrated then potentially no flood upgrade is required.

With the constant challenge for Value Engineering, the design of a new concrete gravity dam may lead to a concept of a secondary spillway arrangement, where potentially a lower design standard is applied as compared to the primary spillway. No matter what, it must be demonstrated that the dam will remain safe for all of the design flood conditions.

The design team for the Wyaralong Dam project, consisting of Hydro Tasmania Consulting, SMEC and Paul Rizzo Associates, were recently challenged with this same issue. Based on options studies undertaken for the roller compacted concrete (RCC) dam, there were real advantages in a general arrangement that incorporated a secondary spillway. However, the cost of this option depended on the level at which the secondary spillway initiated and the level of erosion protection required at the base of the secondary spillway. In order for this solution to be adopted the design team needed to demonstrate, to both the Expert Review Panel and the Dam Safety Regulator, that the dam would remain safe for floods up to the PMF. Although there were some tools available for this assessment, they had a high degree of uncertainty and were not holistic in their approach (ie. they did not answer the question on whether the dam would remain safe). As a result of this challenge, the Wyaralong Dam Alliance Design Team developed a detailed methodology for the erodibility assessment of dam foundations, which was both unique and holistic in its approach.

This paper will present this approach to the erodibility assessment of dam foundations, providing the dam engineer with sufficient detail to apply this methodology to other dam projects. It will then demonstrate how this methodology was applied to the Wyaralong Dam project, including some details of the final designed developed by the team.

2. Proposed Methodology for Erosion Assessment of Dam Foundations

The proposed methodology is a series of decision points. At each decision point a question is asked and answered, which then leads to the next decision point. The process leads to an end point where a clear statement can be made on the safety of the dam, ie. whether the proposed arrangement is acceptable for the design floods. Figure 1 shows this series of decision points in graphical form, which is the framework of the proposed methodology.

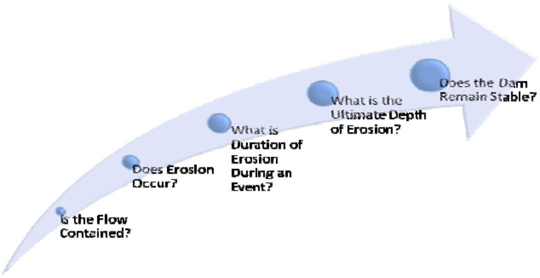

Figure 1 – Decision Points for Assessment of Acceptability of Left Abutment Erosion

Each of the steps is described in detail below:

2.1 Step 1 - Flow Containment

The first stage in assessing the acceptability of the spillway arrangement is to determine the likelihood of flows spilling out across the unprotected abutment. To answer this question, the hydrological and hydraulic characteristics of the spillway are required, including the outflow flood frequency curve and the flow behavior downstream of the spillway. Physical or computerised hydraulic modeling is used to provide a better understanding of when flows commence exceeding the capacity of the apron, and the locations where the flow will spill over the abutment rock.

2.2 Step 2 - Likelihood of Erosion Occurring

The next question posed on the decision framework is the likelihood of erosion occurring if flows exceed the apron capacity. While there are many methods for estimating the likelihood and extent of erosion, there are extensive variables in the mechanisms of erosion which lead to uncertainty in this aspect, such as the properties of the rock, the orientation and spacing of rock defects, and the hydraulic characteristics.

One method commonly used to determine erosion is the Erodibility Index Method, described in Annandale (2006). This method assesses the likelihood of erosion by comparing the hydraulic characteristics of the flow, in terms of stream power, against the erosion potential of unprotected flow path, using the following relationship:.

$$P = K^{0.75}$$

Where P = erosive capacity of water (stream power)

K = erodibility index

The erodibility index is determined by the following relationship:

$$K = M_s K_b K_d J_s$$

Where M_s = Mass Strength Number,

K_b = Block Size Number,

K_d =Discontinuity Bond Shear Strength Number, $K_d = J_r / J_a$

J_s = Relative Ground Structure Number

The erodibilty index is then compared against the erosive capacity of water, or 'stream power', which is determined by the following relationship:

$$P = s_f v y \gamma$$

Where P = stream power (kW/m^2)

v = velocity (m/s) – from model

y = depth (m) – from model

γ = density – 9.81kN/m^3

s_f = energy slope – determine using Manning's equation

2.3 – Step 3 - Likely Duration of Erosion

The rate of erosion is an important consideration in determining the extent of erosion for any given flood event. While the Erodibility Index provides a means of determining the overall extent of erosion, it does not relate erosion to time. A simplified process can be adopted for investigating the likely duration of erosion by using the flood design hydropgraphs for various events. The scour threshold flow determined in Step 2 is compared against the design outflow hydrographs to provide an indication of the likely duration over which the flow exceeds the scour threshold (the duration over which erosion potentially occurs).

Although the process does not provide a rate for erosion, it gives an indication of whether the damaging flows are likely to occur for just a few hours, or many days. Based on the knowledge of the geological conditions and characteristics of the hydraulics, this allows engineering judgment to be used to assess the level of risk associated with the various design flood events.

2.4 – Step 4 - Ultimate Depth of Erosion

The ultimate depth of erosion can be assessed by asking the following questions:

- Does the quality of the foundation rock improve with depth to a point where the stream power is not sufficient to erode the rock (ie. erodibility index increases)?
- Does the rock erode to a point where a scour hole forms and ponding water forms in the hole providing natural dissipation of the energy, leading to a point where the scour hole stabilises (ie. stream power decreases)?

Like many erosion assessment theories, the Erodibility Index can be applied over foundation depth increments and an assessment of the rock quality can be undertaken at each increment to determine increases in the erodiblity index.

The analysis of decreases in stream power in a scour hole (or headcutting analysis) assumes that a scour hole develops immediately downstream of the concrete apron, as shown in Figure 2.

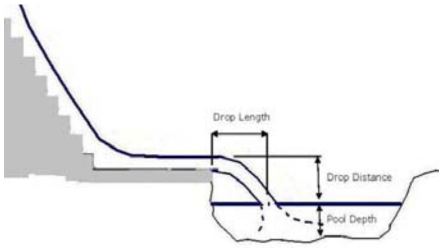

Figure 2– Headcutting Erosion

The trajectory of the flow can be calculated using the following relationship:

$$z = x\tan\theta - \frac{x^2}{K_2\left[4\left(D_i + h_v\right)\left(\cos\theta^2\right)\right]}$$

Where z = vertical distance

x = horizontal distance

θ = issuance angle

K_2 = coefficient allowing for the effects of air resistance (a value of 1.0 was selected)

D_i = thickness of jet

h_v = velocity head - $v^2/2g$

g = acceleration due to gravity

The flow trajectory can be assessed for various flood events and for various scour hole depths. For each scenario, the angle of the flow at the point of impact in the scour hole can be calculated using the following relationship:

$$\zeta = ar\tan\left[\tan\theta - \frac{x}{2K_2\left[\left(D_i + h_v\right)\left(\cos\theta^2\right)\right]}\right]^{-1}$$

Assuming that water collects in the base of the scour hole, decay in the velocity will occur relative to the depth of water in the scour hole. The velocity decay with plunge pool depth can be determined using the following relationship.

$$\ln\left(\frac{v}{v_j}\right) = 0.638\ln\left[\left(\frac{\rho_j}{\rho_w}\right)\left(\frac{v^2}{gL}\right)\right] - 1.848$$

Where: v = Velocity at base of plunge pool

v_j = Velocity at point of impact

ρ_j = aerated density of water

ρ_w = density of water

L = Length from point of impact to base of plunge pool along flow direction

The stream power at the base of the plunge pool can then be determined using the following equation:

$$P_j = \frac{\gamma v_j^3 \sin\theta}{2g}$$

These two methods (assessing increases in erodiblity index, and decreases in stream power through the development of a scour hole) can be used in the scour threshold assessment to provide an indication of the point where the erosion may stabilise.

In addition, it is also necessary to estimate the extent to which a backroller will form in the base of the scour hole. This backroller could potentially lead to undermining of the apron and dam. The following relationship can be used to estimate the percentage of flow developing into a backroller:

$$\frac{q_1}{q_3} = \frac{1+\cos\theta}{1-\cos\theta}$$

Where q_1 = flow downstream

q_3 = backroller flow

2.5 – Step 5 - Stability of the Dam

The final question which needs to be asked is whether the dam remains stable following erosion of the toe of the dam. To answer this question, the ultimate depth of erosion determined in Step 4 of the process is used in the stability analysis. Sliding and overturning stability is assessed using traditional stability theory to determine factors of safety through failure planes at the base of the dam through the scour hole. This process can be undertaken for various depths of scour holes.

3. Applying this Methodology to Wyaralong Dam

3.1 Initial Concept Design

General

As part of the bid phase for the Wyaralong Dam, the two final proponents went through a two month Value Engineering Process driving the following two outcomes:

- Certainty of outcome, and
- Lowest out turn cost.

As a result of this process, the design team developed the preliminary design of an RCC dam with a 0.7H:1.0V downstream slope, and the following spillway arrangement:

- A 120m long centrally located primary spillway with a crest level of RL63.6m and a smooth downstream face
- A 160m long secondary spillway on the left abutment with a crest level of RL66m and a stepped downstream face
- A 25m wide stilling basin with a sloping invert at the base of the primary spillway
- A 10m wide concrete apron at the base of the secondary spillway with an containment wall which directs flow down the toe of the dam back to the primary stilling basin.

Hydraulic Modelling

A physical hydraulic model study was undertaken for this preliminary design, providing a greater understanding on how the proposed arrangement will perform under various flood conditions. The model was later modified to extend the primary spillway by 15m, and reduce the length of the secondary spillway by 10m. A photo of the physical model is shown in Figure 3.

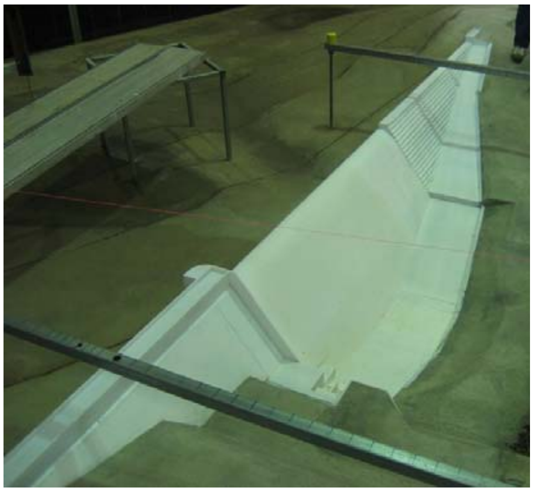

Figure 3 – Wyaralong dam physical hydraulic model

The physical model was used to investigate the following aspects of the left abutment:

- The point at which the secondary spillway apron capacity is exceeded
- The flow velocity and depth of water along the left abutment for various flow events.
- The flow direction and profile over the left abutment.

Some of the findings from the physical model were as follows:

- The secondary spillway commenced flowing at around 1000 m^3/s (around 1 in 100 AEP).
- The concrete apron at the toe of the secondary spillway was found to contain the flow up to around 2000m^3/s (approx. 1 in 2000 AEP) with only a small amount of water flowing over the apron wall.
- At 2500 m^3/s (approx 1 in 6000 AEP), some of the flow is directed down the apron, while the remainder of the flow travels over the left abutment towards the stilling basin. The wall height required to contain the flow down the apron would need to be around 2.4m high (from the apron invert) at the change in direction of the apron. The velocity down the left abutment is less than 5m/s.
- The flow behaviour for a total discharge of 6400m^3/s is very similar to that observed for the 5000m^3/s event, with all of the flow directed beyond the downstream edge of the secondary spillway apron.

Measurements of the flow profile were taken during the detailed design modelling. A sketch of the flow profiles for various flow events are shown on Figure 4.

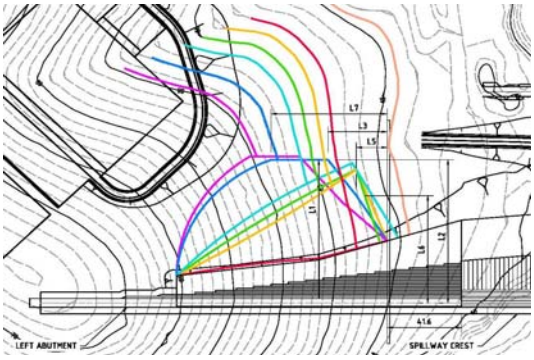

Figure 4 – "Wetted" Area for various flows over the left abutment

Geotechnical and Geological Assessments

In addition to the physical hydraulic modelling, extensive geotechnical investigations were carried out on the proposed dam site.

The left abutment is characterised by open grassland with a few stands of trees downstream of the structure. The area comprises a 1 to 2m thick layer of soil generally comprising dense, medium grained clayey sand, increasing in thickness to around 10m near the river bed. The soil layer is underlain by early Jurassic Gatton Sandstone. The Gatton Sandstone is composed of sandstone with subordinate conglomerate and minor shale and siltstone interbeds.

The 16 boreholes and costean on the left abutment provide useful information on the characteristics of the sandstone foundation, including weathering, bedding planes, joints and other defects which will influence the erodibility of the rock.

Weathered areas of the sandstone formation have a brown discolouration compared to the grey fresh rock. The thickness of the weathering varies along the dam foundation with deeper weathering generally occurring at the abutments. Clay and weak or decomposed rock seams and joints are more prevalent in the more highly weathered material near the rock surface. The foundation level of the dam will generally be located beneath the distinctly weathered zone.

The major defect sets are orientated in unfavourable dip and dip directions with regard to erosion of the left abutment. There is one major near-vertical defect set which strikes almost parallel with the direction of flow. The other two major defect sets (one sub-vertical and one dipping at 15 degrees) are both striking almost perpendicular to the direction of flow, and dipping away from the flow. The bedding plane is orientated similarly to these two major joints sets with a strike perpendicular to the flow and dipping away from the flow at 7 degrees. A summary of the major defects is provided in Table 1 (and presented graphically in Figure 5).

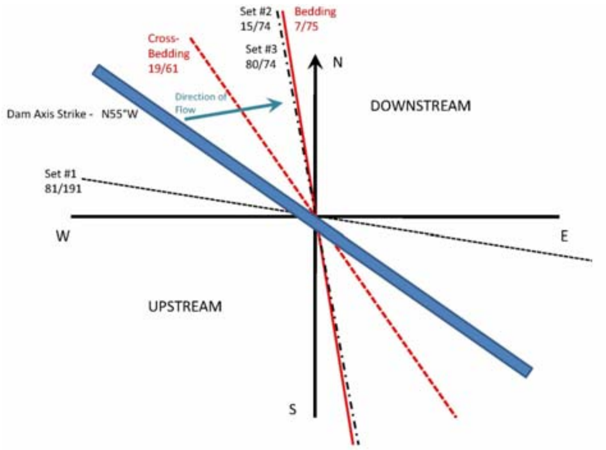

Figure 5 – Major Defect Dip and Dip Direction versus Dam Axis

Table 1 – Summary of Defects

Defect Number	Dip/ Dip direction
Joint Set 1	81/191
Joint Set 2	15/74
Joint Set 3	80/74
Bedding	7/75
Cross-bedding	19/61

Joint spacing at the site is variable. The sub-vertical defect with a strike perpendicular to the flow (Set #3 on the attached plot) occurs as clusters of 3 to 4 joints spaced at 0.5 m to 1m apart, and these "clusters" of joints occur every 7 m to 12m. The joints are planar, irregular and typically have iron oxide staining.

Decomposed seams and zones show similar orientation to the identified joint sets and bedding and are inferred to have formed through weathering and decomposition of joints. They typically contain extremely weathered rock and residual soil consisting of silty sand. They range in thickness from less than 10mm to 300mm.

Infilled seams and zones generally have a more wavy shape than the joints and decomposed seams. The infilled sand clay typically has medium plasticity. A 2.5m wide infilled zone consisting of stiff to very stiff sand clay occurs at the base of the left costean.

3.2 Applying the Methodology to Wyaralong

Each of the decision steps presented in Section 2 of this paper were undertaken for Wyaralong Dam.

Is the Flow Contained?

The physical model study was used to determine the point at which flow spilled out over the left abutment. It was acknowledged that the small scale of the model prevented the energy losses from being accurately replicated. Additional hydraulic calculations using stepped spillway theory were undertaken to predict the likely energy losses on the downstream steps, however a conservative approach was adopted by excluding these energy losses from the assessment.

The hydraulic assessment indicated that the secondary spillway would initiate at the 1 in 100 AEP flood event, and flow from the spillway would be directed along the apron slab at the base of the secondary spillway towards the main stilling basin. However, the physical model indicated that flow from the apron slab would spill out over the left abutment for the 1 in 2000 AEP event (approx 2000 m^3/s).

Will Erosion Occur?

The Scour Threshold method was used to assess the likelihood of erosion occurring once flows spill out across the left abutment, taking into consideration the stream power and the erosion potential of the material.

The stream power along the left abutment was assessed using the following methods:

- Method 1 (used to assess initiation of erosion) - Stream power was calculated using velocity and depth measurements recorded along the left abutment of the physical model for various flow events.
- Method 2 (used to assess initiation of erosion) - A HECRAS model was developed of the left abutment for a unit width of flow.

Annandale's Erodibility Index method was used to determine the erosion potential of the material based on the geotechnical assessments undertaken during the bid and detailed design phases of the project. It became apparent that there was a large variance in the subjective parameters adopted for the analysis. As such, an average, lower bound and upper bound index was assessed for the rock, as shown in Table 2.

Table 2 –Erodibility Index

Item	Soil	Rock		
		Average	Lower Bound	Upper Bound
Erodibility Index	**<1**	**488**	**39**	**3364**

The analysis indicated that if the lower bound parameters were adopted, together with the stream power estimate using the physical model study, erosion may start to occur at a flow of around 3500m^3/s (1 in 50,000 AEP). However, given the conservativeness of some of the parameters adopted in the analysis, it is probably more likely that erosion of the rock would not occur until flows around 5,000m^3/s (1 in 400,000 AEP).

Table 3 – Scour Threshold

Total Flow – m^3/s	Distance from U/S Face (m)	Potential for Erosion					
		Lower Bound (EI = 39)		Best Estimate (EI = 448)		Upper Bound (EI = 3364)	
		PM	H	PM	H	PM	H
3000	28	No	No	No	No	No	No
	70	No	No	No	No	No	No
3500	28	No	No	No	No	No	No
	70	Yes	No	No	No	No	No
	110		No		No		No
5000	28	Yes	No	No	No	No	No
	70	Yes	No	No	No	No	No
	85		No		No		No
6400	28	Yes	No	No	No	No	No
	70	Yes	No	No	No	No	No
	80		No		No		No

PM- Physical Model

H – HECRAS

What is the Duration of Erosion?

The design flood hydrographs were reviewed, based on erosion initiating at around the 3,500m^3/s. As can be seen in Figure 6, the parts of the flood hydrograph where erosion may occur is highlighted. For the 1 in 50,000 AEP event (3,500m^3/s), it is estimated that the erosive part of the hydrograph would only last around six to eight hours. For the PMF event (6,400m^3/s), the flood duration where erosive stream power may occur is around 15 hours. While it is acknowledged that this duration is lengthy enough for erosion to occur, it is judged that the event is relatively short and particularly for the smaller events, may not allow enough time for the full depth of erosion to be realised.

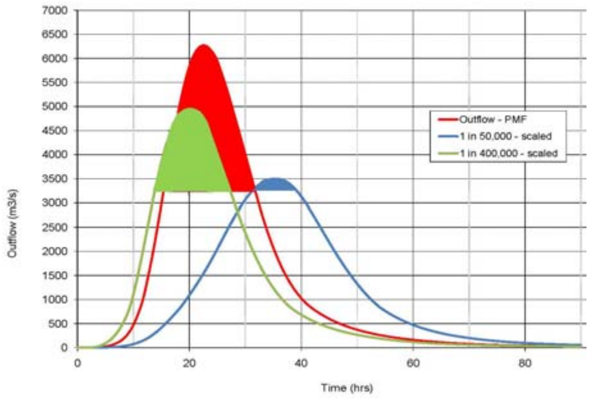

Figure 6 – Flood Duration

What is the Ultimate Depth of Erosion?

The ultimate depth of erosion was determined using methods previously described. The critical factor in the analysis of the scour hole is determining the likelihood that these pool depths will be realised. A simple HECRAS model was developed to assess the likely plunge pool depths assuming that a channel erodes beyond the toe of the concrete apron. Assuming a 10m wide channel along the toe of the apron with an invert slope of 5H:1V, and assuming the water is contained within the channel, the average flow depth would be around 5m and 2m for the 1 in 50,000 (3500m^3/s) and PMF events (approx. 6400m^3/s) respectively. Consequently, the pool depths assessed are judged to be conceivable. Based on this assessment, it is considered likely that the scour hole will stabilise at a depth of around 3-4m for the 1 in 50,000 AEP event, and 6 to 7m for the PMF event.

At these depths, the angle of impact for the trajectory is still relatively flat (around 15 degrees from horizontal. At such a flat impact angle, the majority of flow will be directed in the downstream direction, with only a small proportion (less than 2%) of flow directed back towards the apron. Consequently, based on the plunge pool depths investigated, it is considered very unlikely that any significant backroller effect causing undermining of the apron slab will develop.

Does the Dam remain Stable?

An assessment was undertaken of the dam stability assuming that a scour hole of 7m deep develops immediately beyond the concrete apron.

The stability analysis of the dam with a scour hole at the end of the downstream apron was undertaken for the 1 in 50,000 AEP event (approximately 3500m^3/s) for the section in the approximate vicinity of the tailwater level. At this point, the height of the dam is around 15m. The 1 in 50,000 AEP event was selected for the stability analysis instead of the PMF event as the tailwater is significantly higher for the PMF event.

Three key events were analysed as described in Table 4. For each of each load case, the analysis was first undertaken assuming that uplift is reduced by 67% at the line of the drains. However, it is noted that the sliding plane (located 7m below the base of the dam) may be below the expected base of the drains and as such, an uplift condition was also assessed assuming an undrained situation (ie. triangular uplift from reservoir to tailwater level).

Table 4 – Load Cases

Case	Event	U/S Water Level	D/S Water Level	Uplift
1a	During the 1 in 50,000 AEP	68.2	50.96	Reservoir Head to Tailwater Level with 67% reduction
1b				Reservoir Head to Tailwater Level (undrained)
2a	As the flood subsides – primary spillway operating but no flow over secondary	66	44.58	Reservoir Head to Tailwater Level with 67% reduction
2b				Reservoir Head to Tailwater Level (undrained)
3a	Post flood – reservoir at full supply level - no spillway operation	63.6	35.5	Reservoir Head to Tailwater Level with 67% reduction
3b				Reservoir Head to Tailwater Level (undrained)

The results of the analysis are presented in Table 5.

Table 5 – Sliding Stability

Load Case	Event	Factor of Safety against Sliding	
		ϕ = 35 degrees	ϕ = 40 degrees
1a	During the 1 in 50,000	1.30	1.56

1b	AEP	1.03	1.24
2a	As the flood subsides – primary spillway operating but no flow over secondary	1.66	1.99
2b		1.45	1.74
3a	Post flood – reservoir at full supply level - no spillway operation	2.18	2.61
3b		1.97	2.37

As can be seen from these results, the dam maintains a factor of safety greater than 1.0 for all load cases analysed, with the lowest factor of safety occurring during the flood event assuming a triangular uplift profile from reservoir to tailwater level and a friction angle of 35 degrees.

It is concluded that even if a scour hole develops at the toe of the dam, the dam will remain stable.

3.3 Conclusion Arrived at for Wyaralong Dam

A summary table of the decision framework and the results for Wyaralong dam are presented in Figure 7.

The systematic process provided confidence that the difficult and somewhat uncertain task of assessing the erosion potential was based on sound engineering leading to a high level of confidence in the proposed solution. Furthermore, the analysis process allowed refinements to be made in the design to ensure a higher level of robustness.

Some of the key refinements which were made during the process were:

- Minor modifications were made to the crest level and length of the primary and secondary spillways to ensure that the capacity of the primary spillway was maximized, and that the secondary spillway does not initiate until relatively infrequent events.
- The geometry of the apron at the base of the secondary spillway was modified slightly during the detailed design phase. A deflector wall was included at the top of the apron to assist in directing lower flow events along the spillway apron. The choke in midway along the apron channel was also removed by altering the width of the apron slightly.
- The soil above the apron wall will be the first area to erode. Rip rap protection has been included at the top of the apron wall to provide initial protection to this area.

Flood Event AEP	Is the Flood Contained?	Does Erosion Occur?	What is the Duration?	What is the Ultimate Depth?	Does the Dam Remain Stable?
1 in 100 (≈1000m³/s)	N/A	No	N/A	N/A	N/A
1 in 2000 (≈2000m³/s)	Yes	No	N/A	N/A	N/A
1 in 50,000 (≈3500m³/s)	No	Unlikely	6-8 hours?	4m below invert	Yes
1 in 400,000 (≈5000 m³/s)	No	Maybe	12 hours?	6m below invert	Yes
PMF (≈6400m³/s)	No	Maybe	15 hours?	7m below invert	Yes

Figure 7 – Decision Framework for Wyaralong

- While the analysis indicated that the likelihood of a backroller developing is low, the potential for undermining of the apron slab was not ruled out, and as such, ground anchors are included throughout the apron slab to assist in 'stitching' the foundation rock together.
- The depth of the foundation drains in the vicinity of the apron slab was reviewed to ensure that the failure planes beneath the dam are drained.

Final Spillway Design for Wyaralong Dam

The final design for the Wyaralong Dam spillway comprises:

- A 135m long primary spillway with a crests elevation of RL63.6m and a smooth downstream face
- A 150m long secondary spillway with a crest level of RL66.3m and a stepped downstream face, which is designed to initiate at the 1 in 100 AEP flood.
- A 25m wide stilling basin at the toe of the primary spillway
- A reinforced anchored concrete apron at the toe of the secondary spillway which varies in width from 10m at the left end, to 25m where it joins the stilling basin.
- A 2m high apron wall at the downstream edge of the apron which directs water along the apron slab.

A photo of the left abutment area during construction is provided in Figure 8.

Figure 8 – Wyaralong during Construction

5 Summary

In summary, a unique and holistic approach has been developed for the erodibility assessment of dam foundations. The framework of the methodology is around a series of key questions, leading to a conclusion on whether the dam is safe or not. The details of the methodology have been provided and it has been applied to the case study of Wyaralong Dam. This should provide the dam industry with a tool to apply to both new and existing dams.

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