Memorandum

25 November 2019

To	Memorandum dated 5 September 2019		
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From	GHD Pty Ltd	Tel	(07) 3316 3000
Subject	Cover Page to Memorandum dated 5 September 2019	Job no.	41 32235

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Memorandum

5 September 2019

To	file		
Copy to			
From	[redacted]	Tel	[redacted]
Subject	Dam stability analyses (Revision 1)	Job no.	4132235

1 Scope and purpose

The purpose of this file note is to provide an update on the review of the stability of the dam monoliths under flood loading for the existing dam arrangement and for a potential interim lowering of the primary spillway crest. This includes details of the key inputs and assumptions adopted to date.

The interim lowering as currently designed and reported in GHD (2019)[1], includes a lowering of the primary spillway crest by 10 m to 57.6 mAHD. For comparison purposes, the stability for lowering of the crest by 3 m and 5 m has also been presented.

This file note on dam stability supersedes the previous version issued on 21 December 2018.

2 Input data

2.1 Headwater levels

The headwater levels for the existing dam arrangement were taken from the hydrological results presented in HARC (2019)[2] and are as summarised in Table 2.1. The estimated headwater levels are also presented for arrangements with the primary spillway crest lowered by 3 m, 5 m and 10 m. These were based on developing a rating curve for the proposed arrangement and reading of the level for the peak discharge for the existing arrangement. This assumes there is no change in the flood routing effects with the change in spillway arrangement. Given the limited routing effect, this is considered to be a reasonable assumption for this assessment.

2.2 Tailwater levels

The downstream tailwater levels were adopted from the original design tailwater rating curve included in BDA (2004)[3] as shown in Figure 2.1. It is understood that this rating curve was derived from a Sunwater MIKE11 model which was provided to the Burnett Dam Alliance (BDA).

[1] GHD PTY LTD, 2019, *Paradise Dam Spillway Improvement – Interim Lowering – Detailed Design Report (DRAFT)*, GHD, Brisbane.

[2] HYDROLOGY AND RISK CONSULTING (HARC), 2019, *Paradise Dam Failure Impact Assessment – Hydrology, Dambreak Modelling and Life Loss Assessment Report (Version 1)*, HARC,

[3] BURNETT DAM ALLIANCE (BDA), 2004, *Burnett River Dam – Detail Design Report*, BDA.

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Table 2.1 Headwater levels

AEP (1 in Y)	Peak discharge (m³/s)	Peak reservoir level (mAHD)			
		Existing Dam	3 m lowering	5 m lowering	10 m lowering
FSL	0	67.6	64.6	62.6	57.6
50	11,385	74.5	72.5	70.6	64.3
100	13,820	75.4	73.4	71.6	65.1
200	16,545	76.3	74.5	72.7	65.9
500	20,370	77.5	75.8	74.1	67.2
1,000	23,390	78.3	76.8	75.1	68.1
2,000	27,110	79.2	77.9	76.2	69.1
5,000	35,360	80.7	79.6	78.4	71.2
10,000	48,125	82.8	81.9	80.9	74.2
20,000	72,130	85.7	85.0	84.3	79.0
33,000 (PMPF)	102,070	88.6	88.0	87.4	83.7

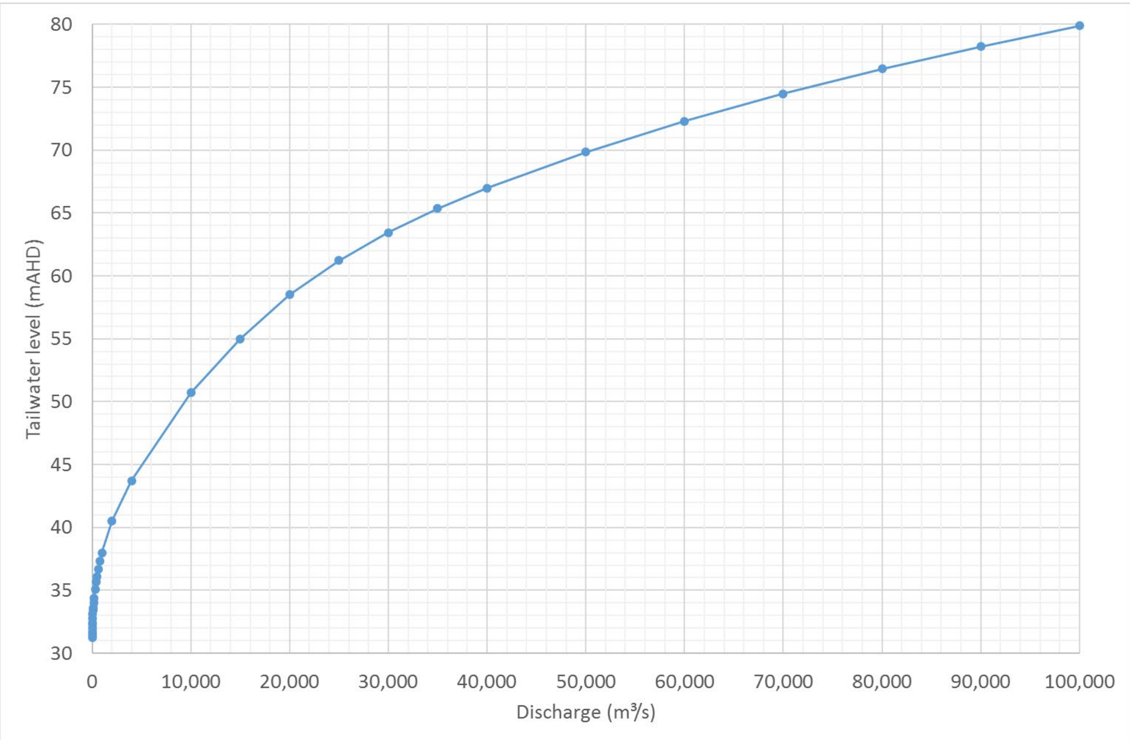

Figure 2.1 Paradise Dam tailwater rating curve from BDA (2004)

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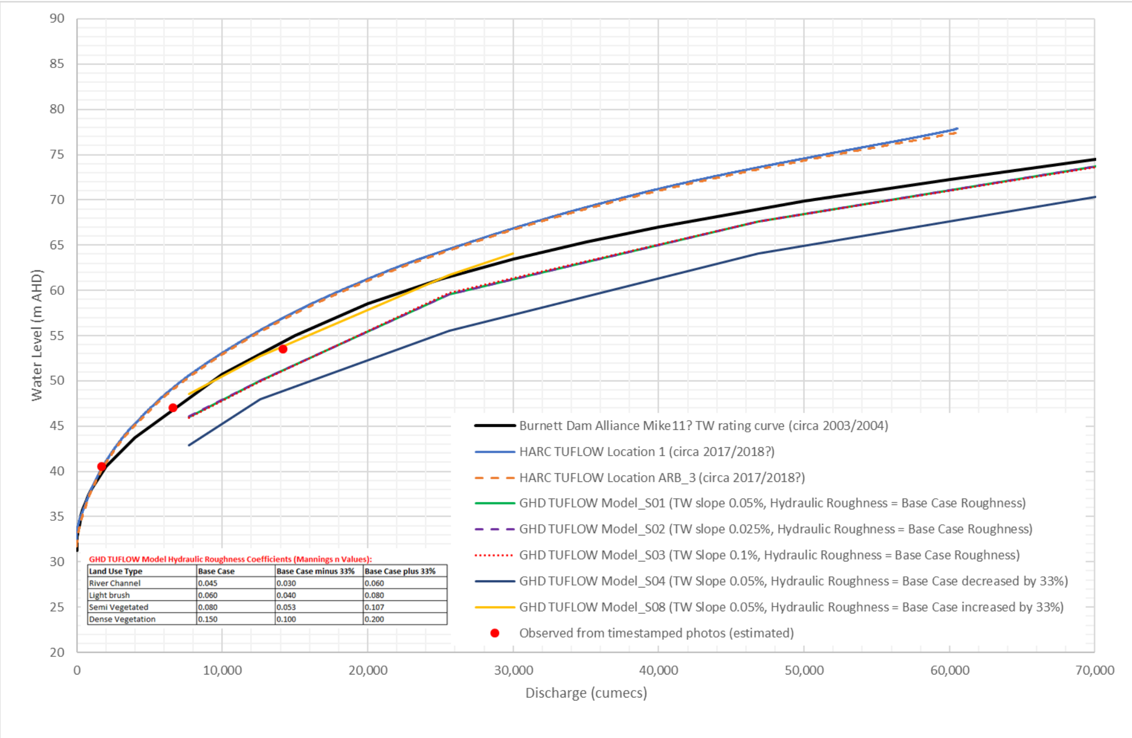
In reviewing the dam stability, it was identified that the stability is heavily dependent on the stabilising effect of the tailwater. As a result, a review of the tailwater rating curve was undertaken, as follows, with the results plotted in Figure 2.2:

- Sunwater initially undertook a review using the TUFLOW model developed by HARC for the downstream hydraulic modelling. The focus of this review was calibration of the levels at downstream gauge locations based on historical events. This model predicted significantly higher tailwater levels at the dam.
- GHD then developed a smaller scale TUFLOW model considering a range of surface roughnesses which predicted tailwater levels lower than the BDA curve.
- The final step was estimation of tailwater levels from date- and time-stamped photographs during flood events since 2010. Three points were identified which plotted roughly over the BDA rating curve, although it is noted that these events cover discharges up to only about 15,000 m³/s. There will be inherent inaccuracies in the estimated levels. To limit the potential error, the points selected were against the abutments where the effects of suppression of the tailwater due to the overtopping would be limited.

Based on this review, the data derived from the time-stamped photographs correlated best with the original BDA tailwater rating curve presented in Figure 2.1, noting however that these data points were for the more frequent flood events. Therefore, the BDA tailwater rating curve was adopted for these analyses.



Land Use Type	Base Case	Base Case minus 33%	Base Case plus 33%
River Channel	0.045	0.030	0.060
Light brush	0.060	0.040	0.080
Semi Vegetated	0.080	0.053	0.107
Dense Vegetation	0.150	0.100	0.200

Figure 2.2 Review of tailwater rating curve

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2.3 RCC lift joint strength

A shear strength of 39.3° has been adopted for the RCC lift joints as outlined in the revised draft GHD file note on the shear strength assessment dated 5th September 2019. This is based on analysis of the direct shear strength tests previously undertaken by SunWater in 2014-15, together with the first batch of direct shear tests on samples obtained as part of the 2019 geotechnical investigations. Further testing is currently underway to confirm this assessment.

3 Loads

3.1 General

The following loads have been considered in this assessment:

- Self-weight of the dam
- Hydrostatic load of the reservoir on the upstream face
- Uplift on the base
- Hydrostatic load from the tailwater acting horizontally and vertically on the downstream face
- Crest pressures due to high overtopping heads greater than the design head
- Silt load on the upstream face of the dam
- Post-tensioning (vertical and inclined) where required for stabilisation

3.2 Self-weight

The self-weight of the monolith is based on the area of the section multiplied by an RCC unit weight. BDA (2005) lists the density of the RCC based on the construction records and this lists a density as low as 2,484 kg/m³. On this basis, a typical density of 2,400 kg/m³ was adopted, which yields a unit weight of 23.5 kN/m³. The analysis has been based on a section at Monolith H with a base level of 32.4 mAHD which is the approximate level of one of the first RCC lift joints. Only this lift joint has been assessed at this time. Under some loading conditions, the upper lift joints may become more critical. A brief assessment of this has been included and is reported below.

3.3 Upstream reservoir load

The upstream reservoir applies a hydrostatic pressure to the upstream face resulting in a triangular load for reservoir levels up to and including the crest level, and trapezoidal for reservoir levels above crest level. No reduction in the upstream hydrostatic pressure, as outlined in USACE (1987)[4], has been included at this time. This is an area that could be reviewed in subsequent assessments, but it is noted that the effect of this is limited.

3.4 Uplift

As outlined in BDA (2004), the original design assumption in relation to uplift within the RCC was that there would be a 50% reduction in uplift at the upstream face and a linear reduction to the tailwater

[4] UNITED STATES ARMY CORPS OF ENGINEERS (USACE), 19877, *Hydraulic Design Criteria*, USACE, Washington, DC.

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pressure at the downstream toe. With a 50% reduction, the pressure at the upstream face is therefore calculated as the tailwater plus 50% of the difference between headwater and tailwater.

As outlined in GHD (2018)[5], consideration has been given to the long term integrity of the anchorage of the upstream face panels and associated membrane. If the membrane is not effective, there is potential for an increase in uplift within the RCC. If failure of the membrane is widespread, it should not be assumed that there is an uplift reduction at the upstream, face when determining the uplift load. Given this, a case with no uplift reduction (ie 100% uplift) at the upstream face has been considered.

In all cases, where the base stress at the upstream face exceeds the tensile capacity (which in this case is assumed to be zero), a crack will form and full uplift will apply in the cracked portion. Where the crack extends past the line of the drains, it is assumed the drains will be ineffective. This requires an iterative solution.

The analysis has also included the effect that the downstream apron/stilling basin has on increasing the uplift. Where there is a downstream apron, the uplift reduces to tailwater at the downstream end of the apron rather than the downstream toe of the dam. The uplift pressure is only applied under the dam and the apron is not considered to form part of the structure for the stability assessment.

An assessment of the actual effectiveness of the upstream membrane and drainage in reducing the uplift within the dam and at the dam/foundation interface was undertaken using the available historical piezometer data. Piezometers were installed in Monoliths C to L and N during construction, with a further three installed in Monolith D (PD-06 to PD-08) in 2017. The details of the piezometers installed during construction are shown on Sunwater Drg No 226879 and 226880, and reproduced in Figure 3.1. Towards the upstream face, this typically includes 1-2 piezometers in the body of the dam, one at the dam/foundation interface and one in the foundation at selected sections. Piezometers PD-06 to PD-08 were installed after one of the upstream face drain outlet pipes was inadvertently grouted up as part of the construction of the monolith strengthening blocks.

The piezometer data was interpreted to give a value for the percent uplift at the upstream face. Piezometer data for selected monoliths are presented in Figure 3.2 to Figure 3.5.

The recorded data shows that the percent uplift at the upstream face is typically less than 50% (ie greater than 50% reduction) although there are some piezometers registering pressures up to about 60%. This includes the piezometers at the dam/foundation interface. It is noted that there is no piezometer data available at reservoir levels greater than about 69.7 mAHD (ie 1.8 m above FSL) and therefore it is not possible to predict that the same reduction in uplift will be achieved at higher reservoir levels.

For this assessment of the existing dam stability, 50% uplift reduction has been taken as the best estimate. It is noted that in the risk assessment, a range of values for the uplift reduction have been considered and therefore, there is no single design value in the upgrade design given to the risk based design approach adopted. Consideration has been given in this assessment to uplift values as high as 60%.

[5] GHD PTY LTD, 2018, *Paradise Dam Facility Strategy & Options Analysis – Preliminary Business Case - Supporting Technical and Environmental Review*, GHD, Brisbane.

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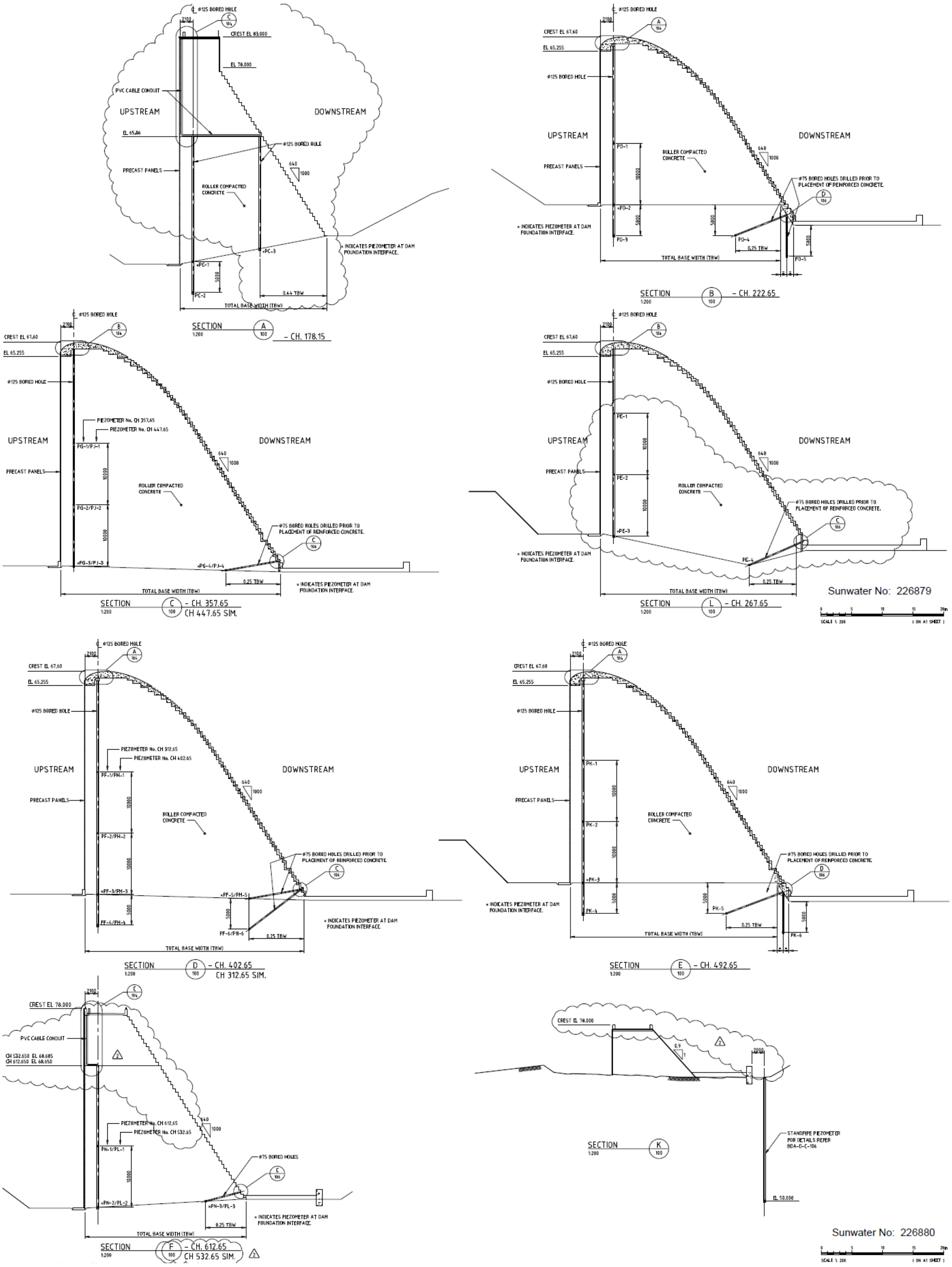

Figure 3.1 Layout of dam piezometers

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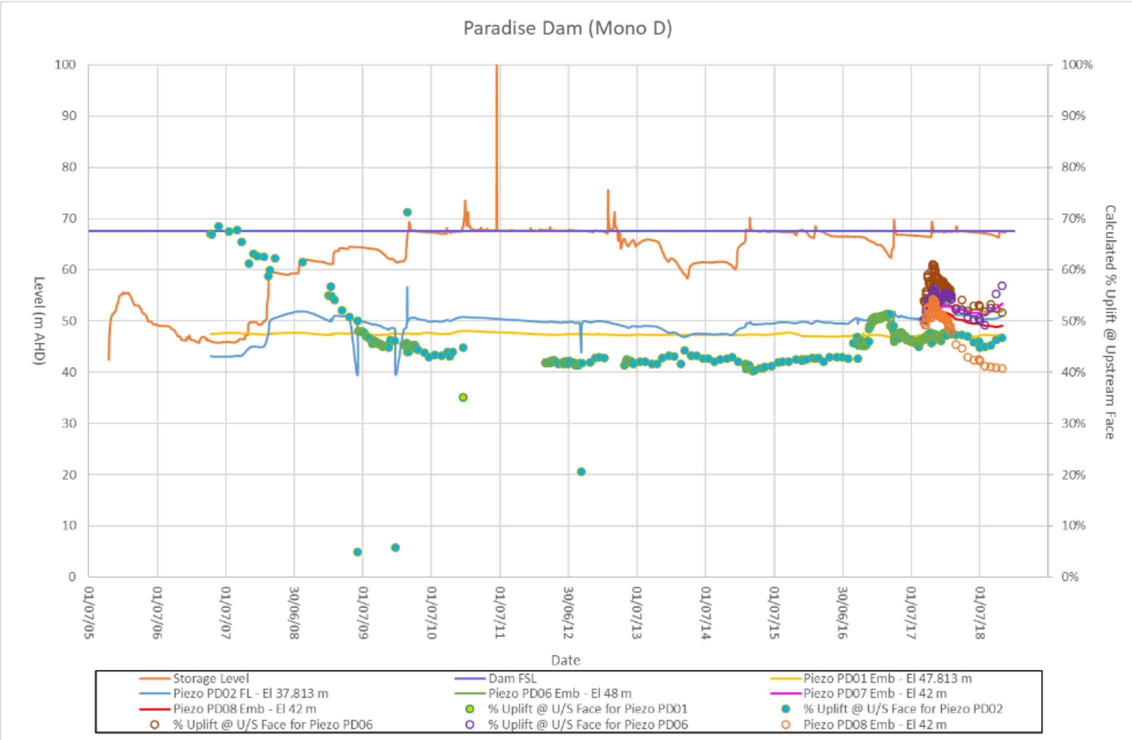

Figure 3.2 Monolith D piezometer data

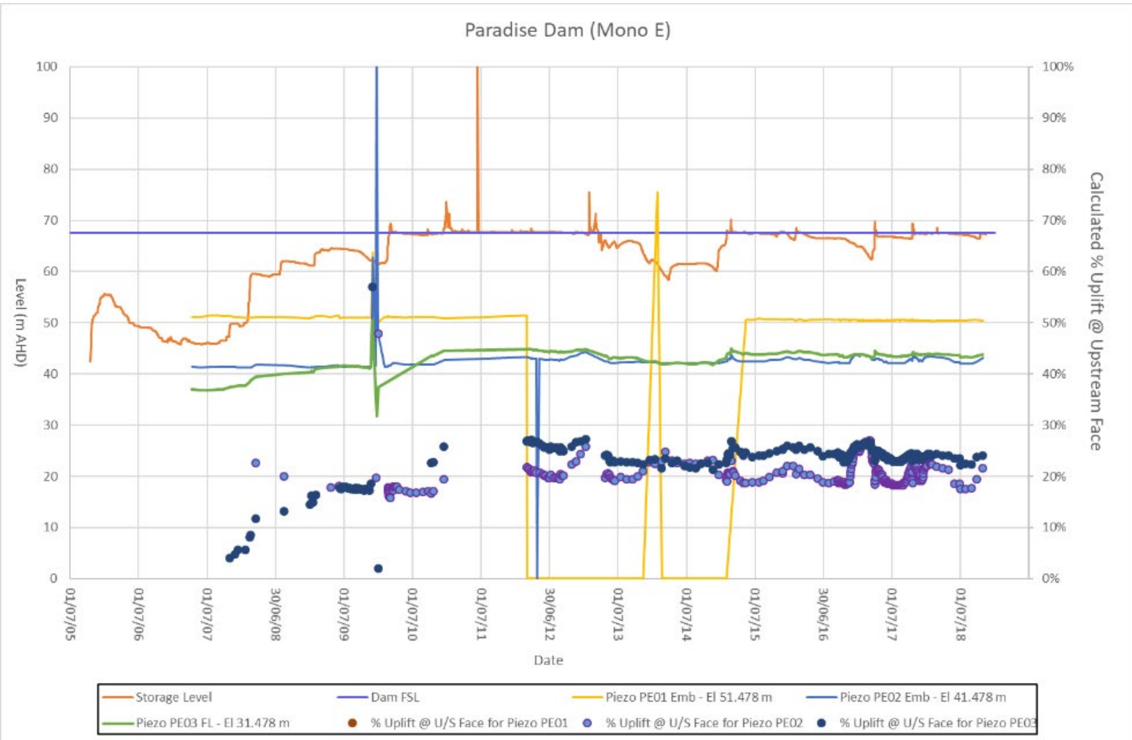

Figure 3.3 Monolith E piezometer data

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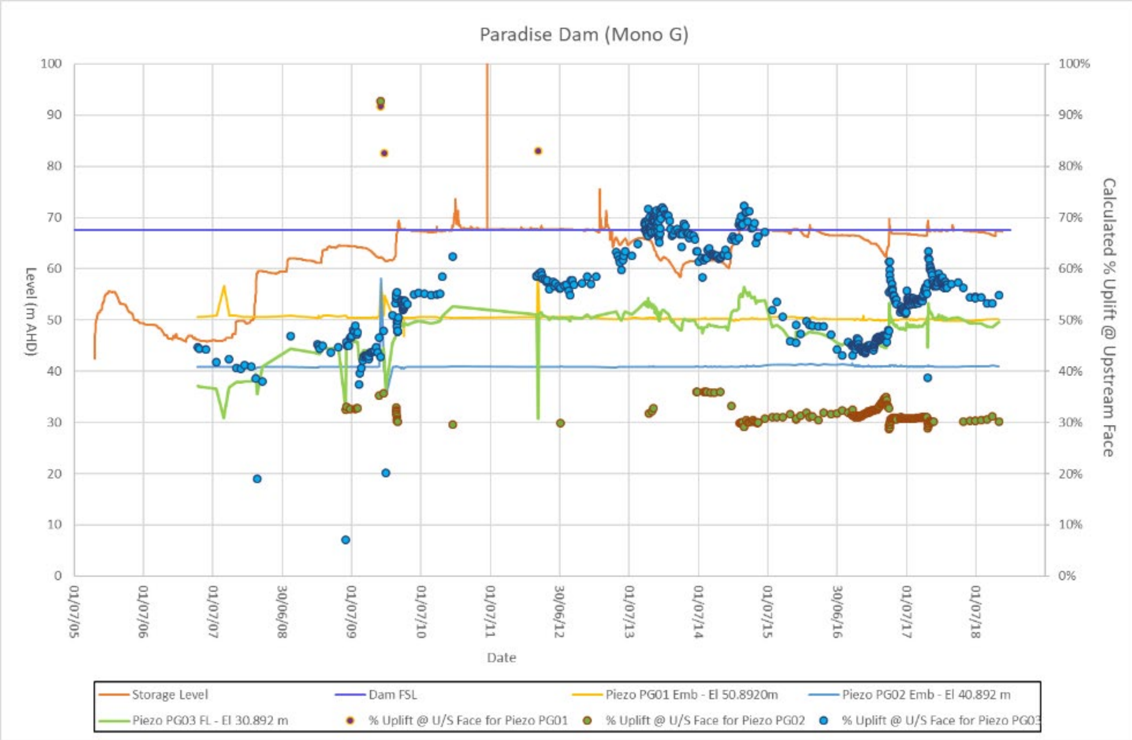

Figure 3.4 Monolith G piezometer data

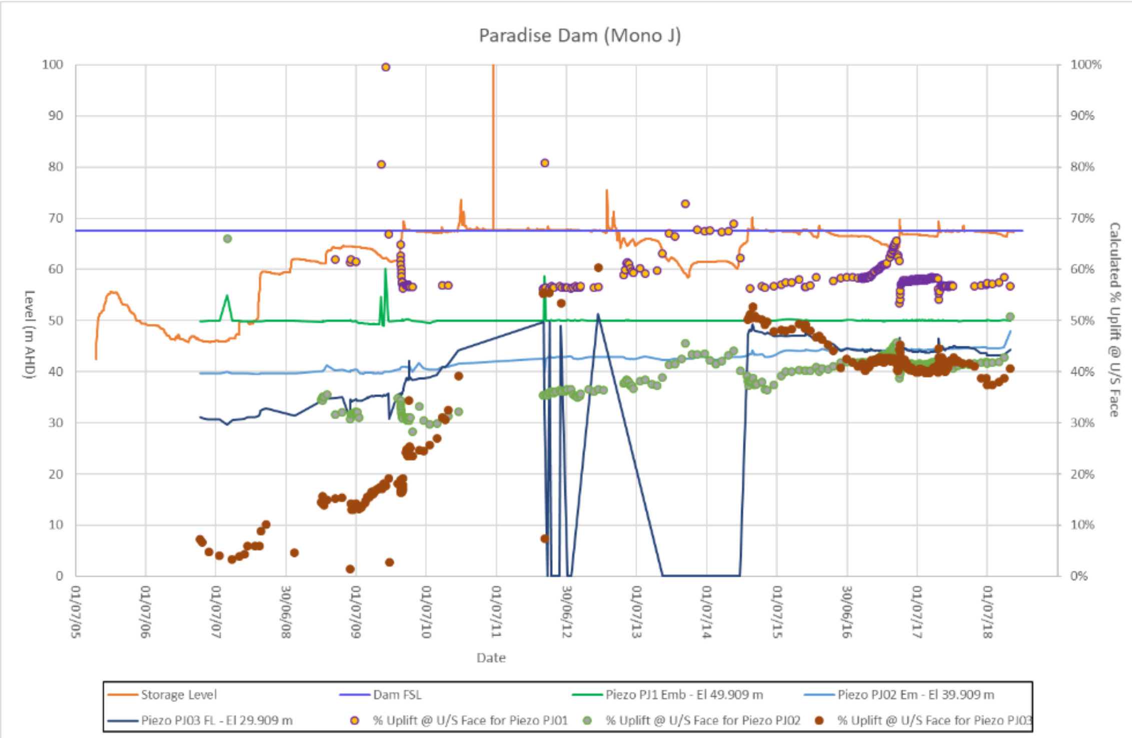

Figure 3.5 Monolith J piezometer data

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3.5 Downstream tailwater load

Where the dam section is not being overtopped, the downstream tailwater exerts a hydrostatic pressure on the downstream face of the dam monoliths. Where the downstream face is sloping, the tailwater also applies a vertical load to the downstream face.

However, as noted in ANCOLD (2013)[6], the force on the downstream face of an overflow section may fluctuate due to the turbulence in the stilling basin. ANCOLD includes a comment that "USACE (1995) notes that 'the influence of tailwater regression can reduce the effective tailwater depth to as little as 60% of the full tailwater depth'".

The actual tailwater depths against the downstream face of the dam were extracted from the CFD modelling presented in GHD (2018)[7], noting that this behaviour differs for different flood events. Data from the physical hydraulic model (PHM) study undertaken for the original design in 2004, taken from BDA (2004), was also reviewed and compared against the CFD results. These results are presented in Figure 3.6 and Figure 3.7.

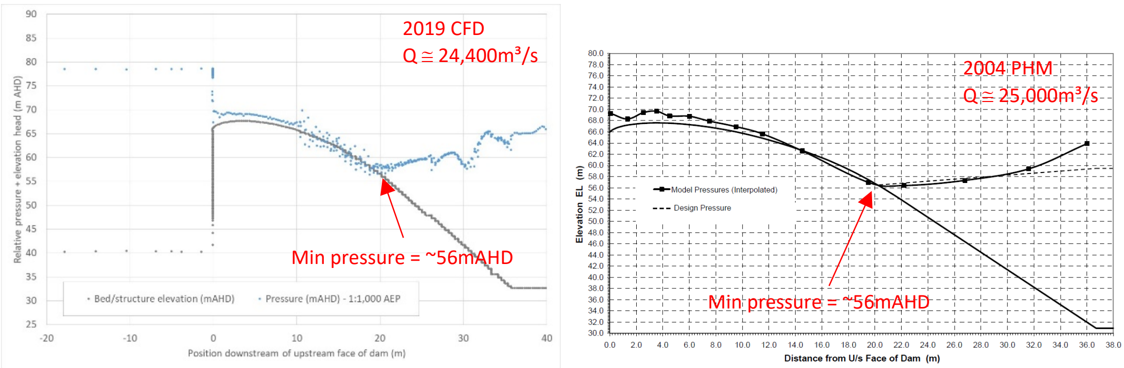

Figure 3.6 Comparison of CFD and PHM downstream face pressures – 1:1,000 AEP

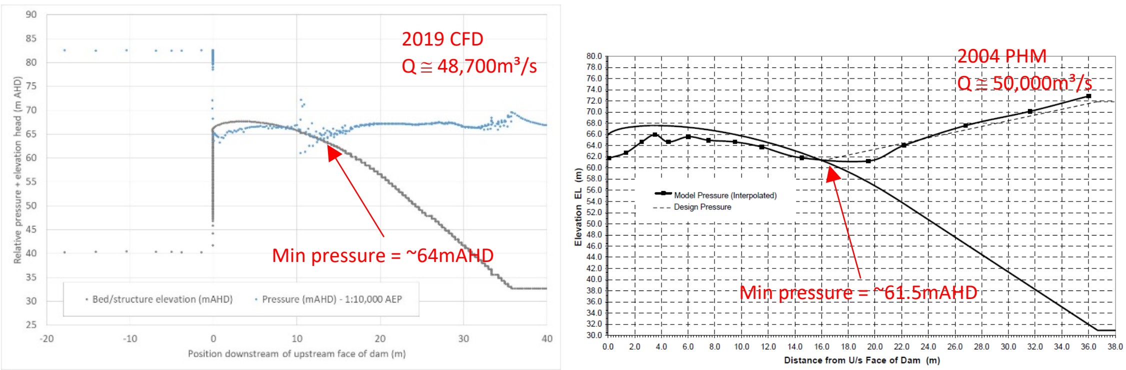

Figure 3.7 Comparison of CFD and PHM downstream face pressures – 1:10,000 AEP

For the 1:1,000 AEP event, the tailwater level is approximately 61 mAHD. Based on a downstream foundation level of approximately 31 mAHD, the pressure against the downstream face from the CFD

[6] AUSTRALIAN NATIONAL COMMITTEE ON LARGE DAMS (ANCOLD), 2013, *Guidelines for Design Criteria for Concrete Gravity Dams*, ANCOLD.

[7] GHD PTY LTD, 2018, *Paradise Dam – Hydraulic Modelling Review and CFD Model Development*, GHD, Brisbane.

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and PHM is equal to 83% of the full tailwater pressure. For the 1:10,000 AEP event with a tailwater level of approximately 70 mAHD, the downstream pressure of 61.5 mAHD equates to 78% of the full tailwater pressure.

Table 3.1 presents a summary of the effective tailwater pressures derived from the CFD modelling. On the basis of this review, an effective tailwater pressure equal to 80% of the tailwater depth was adopted for the stability analyses for the existing dam.

Table 3.1 Tailwater levels and pressure against the primary spillway downstream face

Discharge (m^3/s)	Tailwater level (mAHD)	Pressure against D/S face (mAHD)	Effective tailwater from pressure (%)
17,490	56.9	52	81%
20,820	59.5	53	77%
24,440	61.0	56	83%
48,740	69.7	64	85%

3.6 Crest pressures

At reservoir heads greater than the design head for the ogee crest, the flow over the spillway crest induces a negative pressure, resulting in an upward acting force on the crest. According to BDA (2004), the design head for the ogee crest is 12.5 m, which is equivalent to a reservoir level of 80.1 mAHD which is marginally greater in magnitude than the 1:2,000 AEP flood. For higher heads than the design head, a negative (upward) pressure would be expected, and conversely, a positive (downward) pressure would be expected for lower heads.

Crest pressures were taken from the CFD modelling for Option 2 which has the same crest geometry as the existing dam. Downstream changes such as the stilling basin will not affect the crest pressures. These pressures are plotted in Figure 3.8. This data follows the expected trend with the 1:1,000 AEP event resulting in a small positive (downward) pressure and the 1:10,000 AEP event and PMPDF resulting in negative (upward) pressures.

This uplift was included as a simplified uniformly distributed pressure over a length of the crest based on the CFD modelling results to apply an appropriately conservative crest uplift at high discharges. For the 1:10,000 AEP event, an uplift of 25 kPa was applied over a 10 m length and for the PMPDF, an uplift of 50 kPa was applied over the same 10 m length. For all more frequent events, no crest pressure was included.

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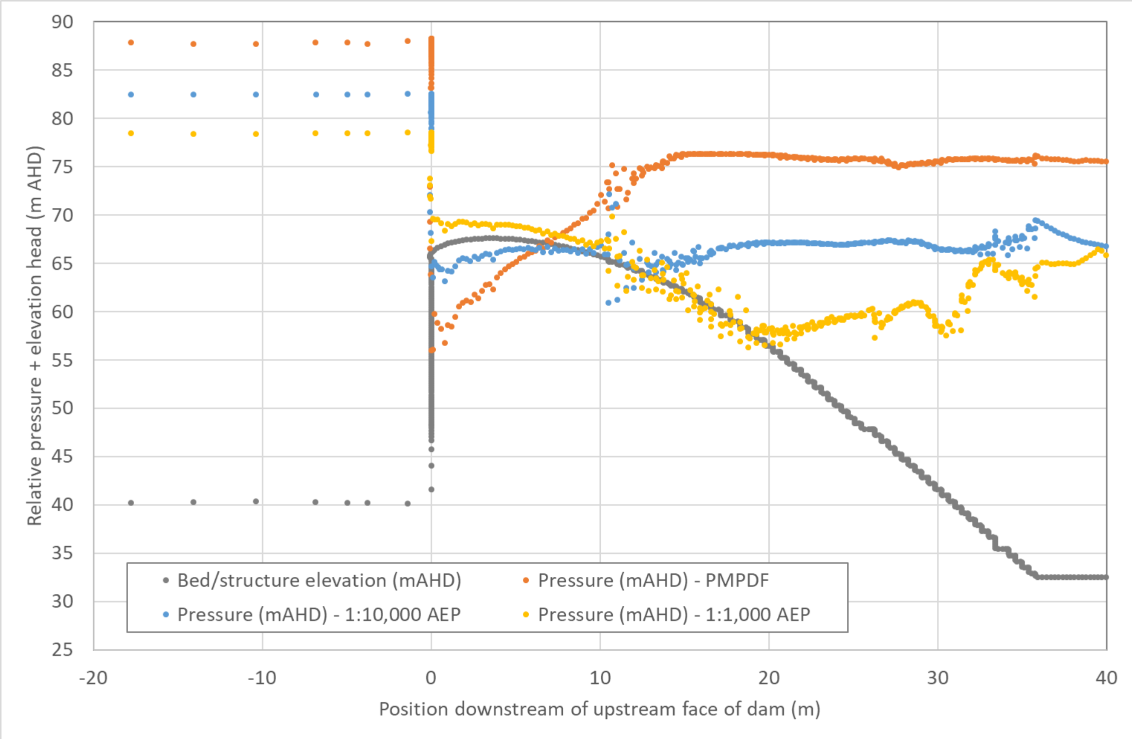

Figure 3.8 CFD crest pressures

For the 10 m interim lowering, the results of CFD modelling were used to derive the hydraulic loading on the crest. Given the broad flat crest for this arrangement, the pressure is downward acting over the range of floods considered and is generally uniform. The pressure head on the crest in terms of elevation was extracted and this was then converted into a load. A 20% reduction was applied to the load given the simplified nature of the assessment. The pressure head values from the CFD were as follows:

- 1:100 – 62 mAHD – 690 kN/m
- 1:2,000 – 65 mAHD – 1,160 kN/m
- 1:15,000 – 69 mAHD – 1,790 kN/m
- PMPF – 77 mAHD – 3,040 kN/m

Loading for other flood events was interpolated. For the 3 m and 5 m lowering cases, no CFD modelling has been undertaken. It was considered that this would vary between the crest pressures adopted for the 10 m lowering and those for the existing dam. In the absence of other information, the following was adopted as a first pass estimate of the loading for these two cases:

- 5 m lowering – 50% of the downward acting force listed above for the 10 m lowering
- 3 m lowering – 50% of the uplift forces adopted for the existing dam

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3.7 Upstream silt load

An allowance was included for silt build-up against the upstream face of the dam. In the absence of any other information, the same basis as used in the AECOM stability analyses included in SunWater (2016)[8] was used, which was silt to a level of 38 mAHD with a unit weight of 18 kN/m³. These assumptions have not been verified as part of this assessment.

4 Load cases and acceptance criteria

This assessment has considered only flood loading at this time. The acceptance criteria for "residual strength – well defined" have been adopted. As noted in the revised draft GHD file note on the shear strength assessment, it is questionable whether the adopted strength truly represents a "well-defined" strength. In relation to the use of "well-defined" acceptance criteria, the following guidance is taken from ANCOLD:

- ANCOLD (2013)
  - "…means a sufficient number of tests have been done on concrete core from the dam and lift surfaces to give the strength parameters with reasonable certainty"
  - "…should be exceeded by about 80% of the test data"
- ANCOLD (2018)[9]
  - "…to use the "well defined strength" factors of safety there needs to be a very low likelihood that the strengths will be significantly lower than used for the analyses."
  - "…may be acceptable to use the "well defined" factors of safety if a sufficiently conservative approach is taken to selecting the shear strength."

It is considered that the approach in selecting the design shear strength is sufficiently conservative to use the "well-defined" criteria, especially when considering the statistical approach described in the revised draft GHD file note on the shear strength assessment.

This assessment has considered only flood loading at this time. The load cases and acceptance criteria were based on ANCOLD (2013), as follows:

- Flood events up to the 1:50 AEP event (ie more frequent than 1:50 AEP) are classified as "usual" events with a minimum required sliding factor of safety of 1.5 and with the resultant to be inside the middle third (ie no tension on the base), then;
- Flood events greater than the 1:100 AEP event and up to the 1:2,000 AEP event are classified as "unusual" events with a minimum required sliding factor of safety of 1.3 and with the resultant to be inside the middle half (ie tension allowed); then
- Flood events greater than 1:2,000 AEP (ie less frequent than 1:2,000 AEP) are classified as "extreme" events with a minimum required sliding factor of safety of 1.1 and with the resultant to be inside the base (ie tension allowed)

[8] SUNWATER LIMITED, 2016, *Paradise Dam – Dam Safety Review (Revised Report 2016)*, SunWater, Brisbane.

[9] AUSTRALIAN NATIONAL COMMITTEE ON LARGE DAMS (ANCOLD), 2018, *Practice Note – Geotechnical Investigations of Dams, Their Foundations, and Appurtenant Structures (DRAFT D)*, ANCOLD.

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At this stage, the stability has only been assessed for the following normal and flood loading conditions:

- FSL (usual)
- 1:50 (usual)
- 1:100 (unusual)
- 1:2,000 (unusual)
- 1:10,000 (extreme)
- PMPDF (extreme)

5 Results

5.1 Back-calculation using the January 2013 flood

In the first instance, an analysis was run to assess the stability of the primary spillway at the peak of the January 2013 flood of record. This assessment was based on Monolith H which is a full-height monolith within the primary spillway. It is noted that the foundation level for Monolith D is higher with the lowest RCC lift joint at approximately 38 mAHD. The factor of safety is similar for Monolith D as reported below for Monolith H with the reduced self-weight offset by the reduced uplift and reservoir load.

According the dam safety review, the peak reservoir level in the January 2013 flood was 76.25 mAHD with a peak discharge of approximately 16,500 m^3/s. It is noted that this level and discharge does not exactly match the data in Table 2.1. The tailwater for this event was estimated to be 56.1 mAHD based on the BDA tailwater rating curve. The estimated factor of safety with a shear strength of 39.3°, 50% uplift and 80% effective tailwater was found to be 1.11 with the resultant inside the middle third, ie no tension.

It is typical for back-calculation exercises to be undertaken using best-estimate parameters. Other than the tailwater load assumptions, the shear strength and uplift reduction are also critical with a high degree of uncertainty.

As noted in Section 3.4, there are some monoliths where the uplift is up to about 60% and it is also noted that there is no data for reservoir levels above about 69.7 mAHD (ie 1.8 m above FSL). If the uplift were 60% as indicated by some of the piezometers, the sliding factor of safety would be 1.06. If there was a non-linear uplift trend at higher flood levels with uplift of up to say 70% during the event, the factor of safety would reduce to 1.00.

In terms of the effect of shear strength on stability, adopting the median shear strength of 42° together with 50% uplift yielded a factor of safety of 1.22.

This highlights the sensitivity of the stability to these key input assumptions. It is noted that there is a considerable degree of uncertainty in these parameters. For example, there is no piezometer data for reservoir levels above 69.75 mAHD to confirm that the same uplift reduction applies at elevated storage levels.

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During the 2013 flood event, it is considered likely that the factor of safety for the dam in sliding was of the order of 1.0-1.2. With an AEP of approximately 1:200, this event would be classified as an unusual event with a minimum required factor of safety of 1.3. The stability may have been marginal if the key inputs and assumptions adopted in the stability analyses prove to be consistent with actual conditions.

5.2 Existing dam stability assessment

This section presents the stability results for the existing dam over the range of flood events considered. The stability of the primary spillway (Monolith H), left abutment (Monolith C) and secondary spillway (Monolith N) are presented in Figure 5.1 to Figure 5.3.

Figure 5.4 presents a comparison of the stability of Monolith H in terms of sliding factor of safety based on the 2018 shear strength assumption of 37° compared with the 2019 shear strength of 39.3°. For comparison, this figure also presents the sliding factor of safety based on the BDA design shear strength of 250 kPa and 35°. This figure is based on uplift at the upstream face of 50%.

The stability of the monolith varies depending on the base level assumed for the section. This was assessed and the results are presented in Figure 5.5. This figure is based on uplift at the upstream face of 50%. It is noted that the stability analyses for the higher base levels, ie 40 mAHD and above, do not include a spillway apron downstream whereas the case with the base at 32.4 mAHD includes a 20 m long apron.

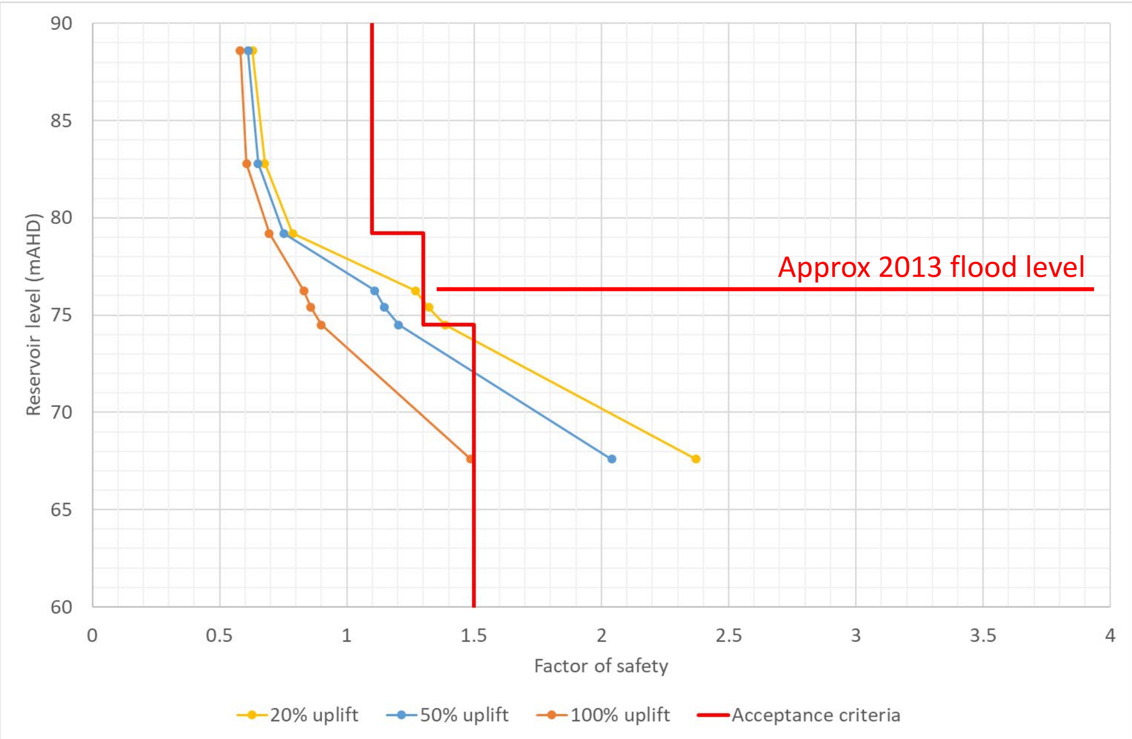

Figure 5.1 Primary spillway stability – Monolith H

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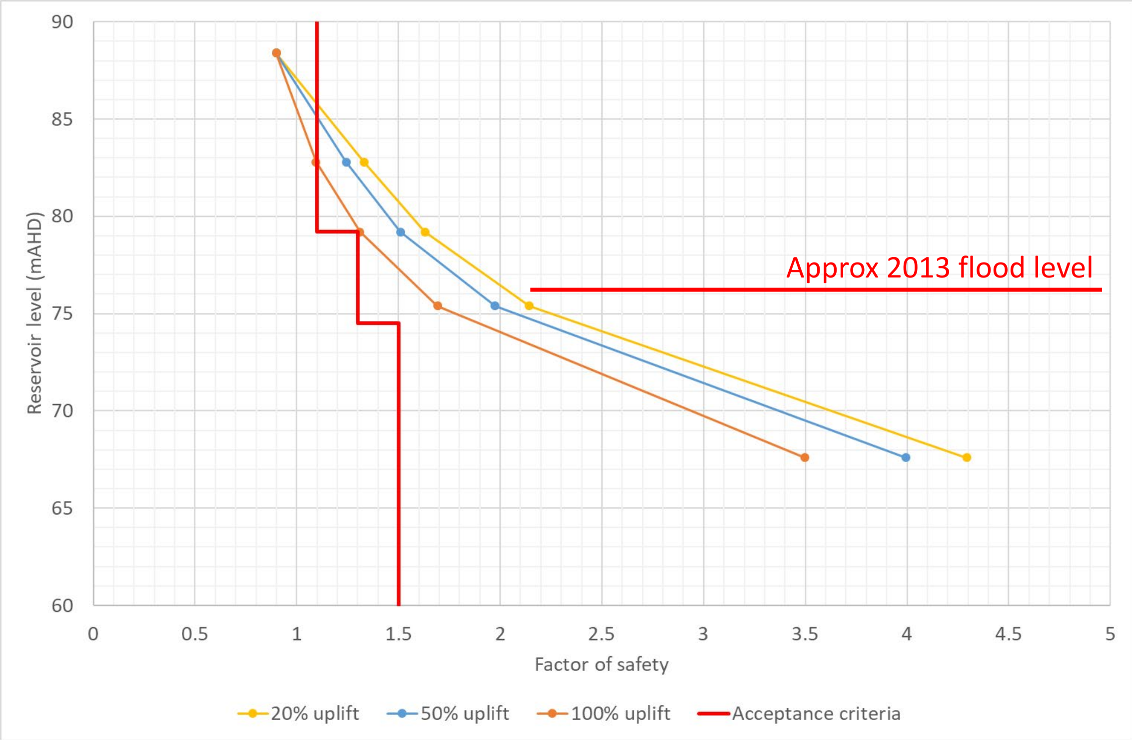

Figure 5.2 Left abutment stability – Monolith C

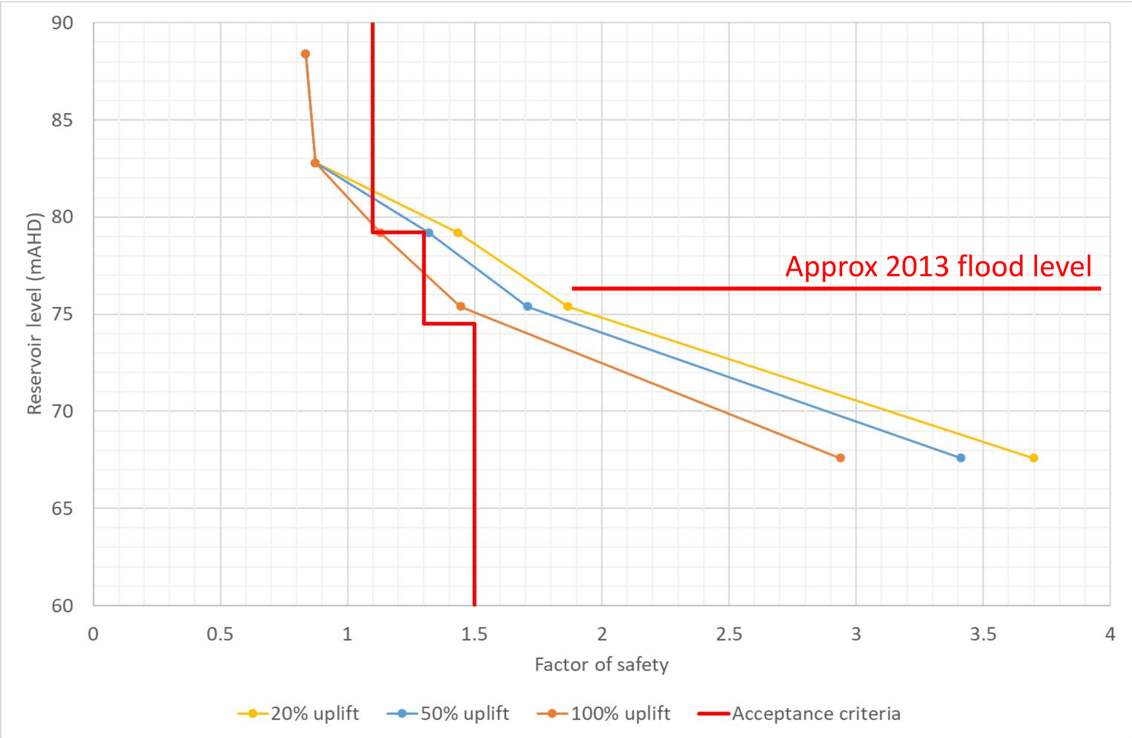

Figure 5.3 Secondary spillway stability – Monolith N

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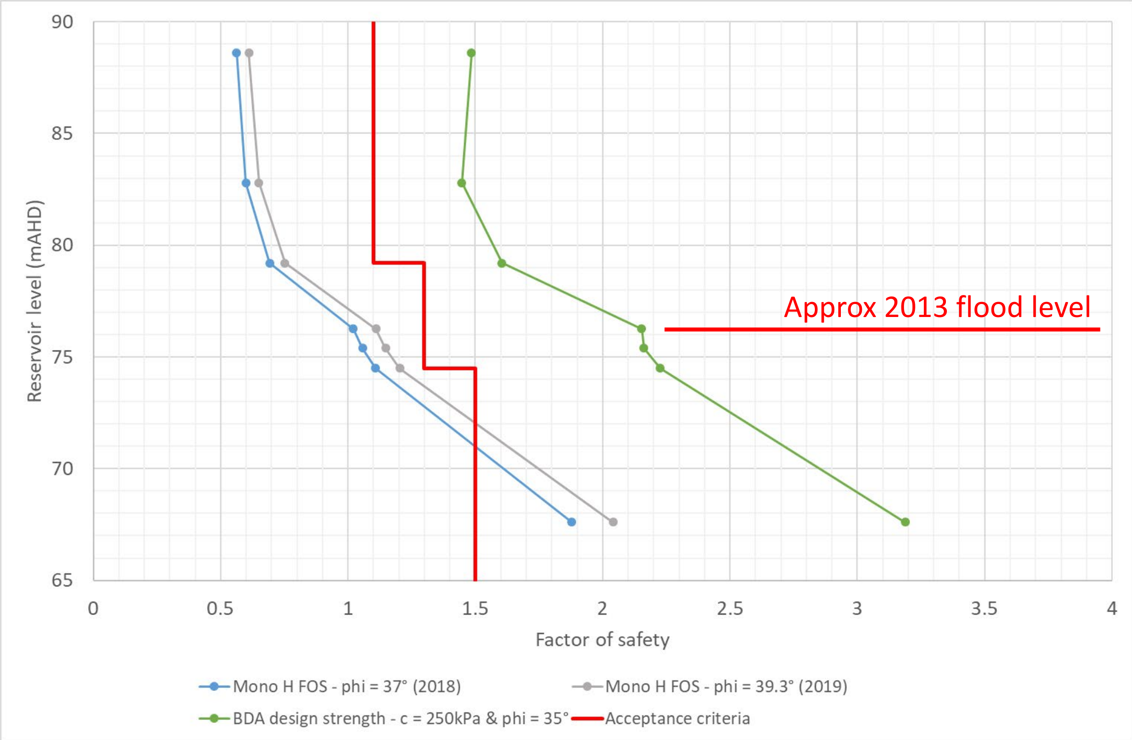

Figure 5.4 Comparison of stability for Monolith H with 2018 and 2019 shear strength assumptions

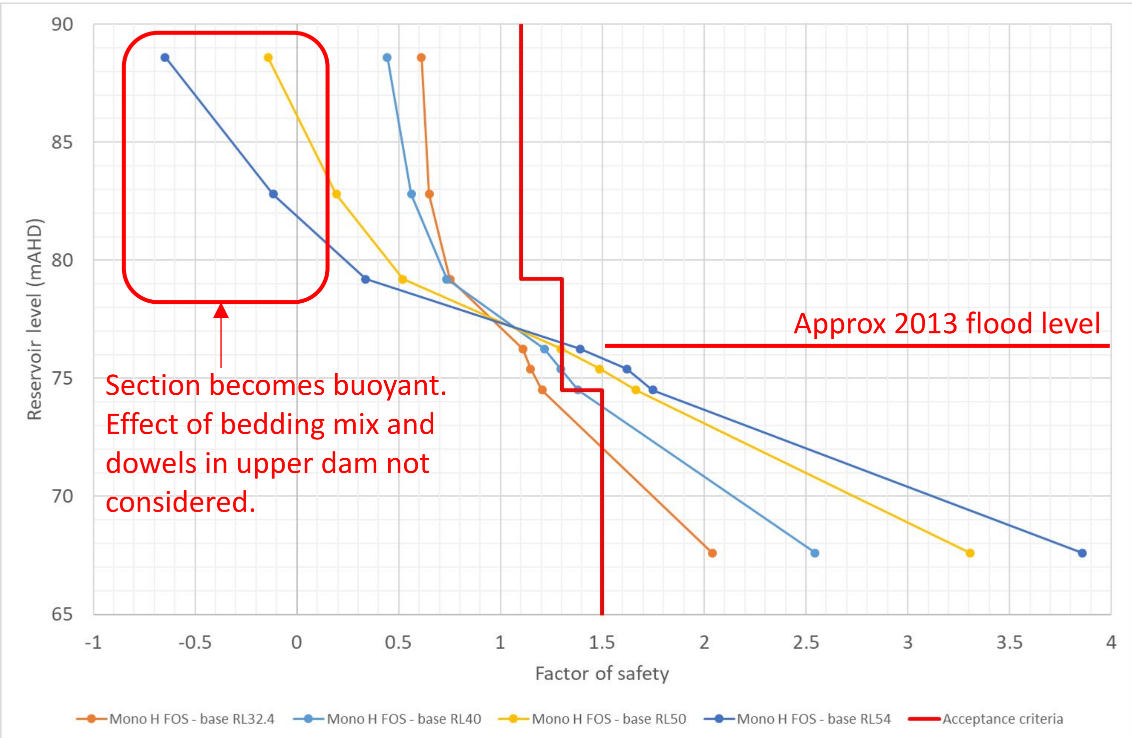

Figure 5.5 Comparison of stability for Monolith H with varying section base levels

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5.3 Stability with potential interim lowering of primary spillway crest

An interim lowering of the primary spillway crest by 10 m to 57.6 mAHD has been proposed as an option to reduce the dam safety risk in an efficient manner. An assessment of the stability of the lowered section has been undertaken. In addition to this, the stability of a section with the crest lowered by 3 m and 5 m has also been undertaken to show the potential incremental improvement that may be gained as the crest is lowered. The results of the stability analyses are presented in Figure 5.6.

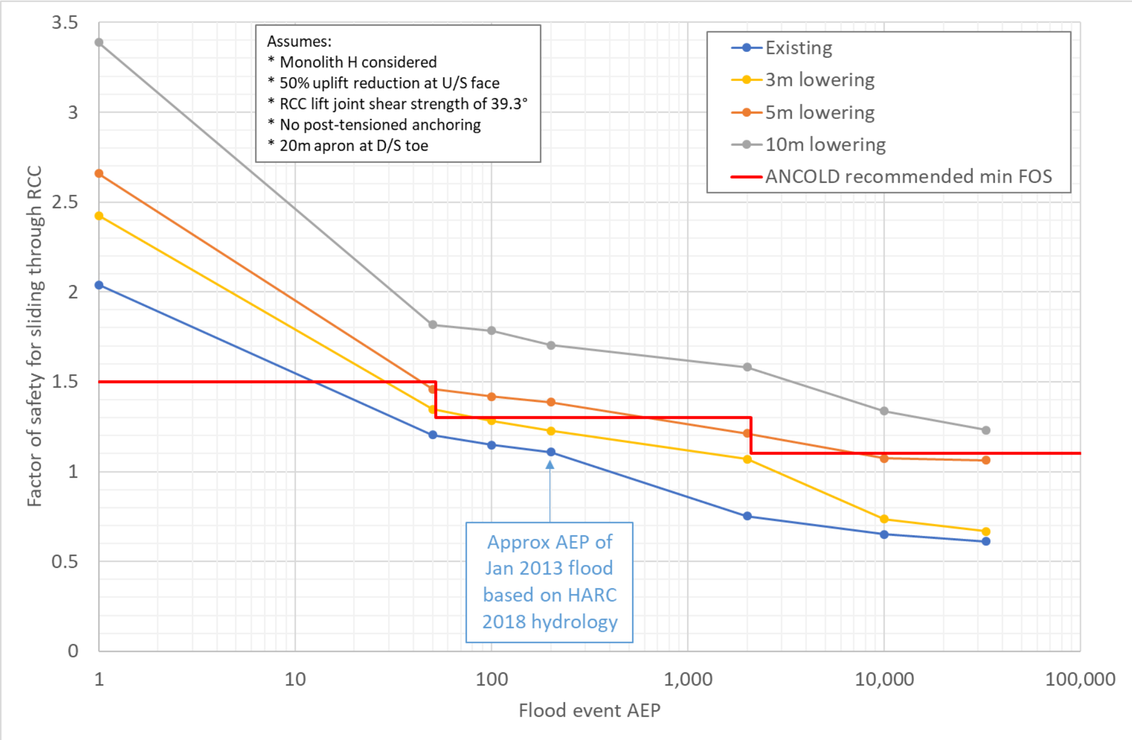

Figure 5.6 Summary of stability for Monolith H interim lowering

5.4 Comparison with 2016 Dam Safety Review results

A comparison was made of the results based on the current assessment with the results presented in the 2016 dam safety review (DSR). The stability analysis spreadsheets used for the DSR were provided by SunWater on 14th November 2018. Key differences in the input assumptions are as follows:

- The shear strength parameters used in the DSR analyses were 50 kPa cohesion and phi of 47° compared with no cohesion and phi of 39.3° used herein.
- The uplift force in the DSR analyses appears to have been calculated as the tailwater pressure multiplied by the base width rather than linearly reducing from upstream to downstream.
- The DSR analyses did not include the effect that the 20 m long stilling basin apron has in increasing the uplift under the dam. It is understood that the assumption was that the drains under the stilling basin apron would be effective in relieving the pressure but it is considered that the drains could even be pressurised due to the turbulence where the drains exit.

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- The basis of the calculation of the tailwater pressures on the downstream face is unclear in the DSR analyses. It appears that a reduced water level was adopted against the downstream face based on the model study. The horizontal load was then been calculated as a triangular pressure distribution based on the pressure from the full tailwater but acting only over the height from the reduced downstream face water level to the lift joint in question. Similarly, the vertical load was calculated from a triangular pressure distribution using the full tailwater pressure acting over the horizontal width calculated from the reduced downstream face water level. The basis of these assumptions is not clear. This results in a significantly greater tailwater load than from the current assessment.
- There are minor differences in the headwater/tailwater combinations given that the hydrology has changed since the analyses for the DSR were undertaken.

The following sliding factors of safety excluding any anchorage are presented in Table 5.1:

- Using GHD 2019 stability model with assumptions as outlined in the previous sections herein with a 20 m long stilling basin apron
- Using GHD 2019 stability model with the 2016 DSR shear strength, headwater/tailwater combination and a 20 m long stilling basin apron
- Taken the factors of safety directly from the spreadsheets used for the 2016 DSR for a lift joint at 33.6 mAHD

Table 5.1 Comparison with results from 2016 Dam Safety Review

Load case	Factor of safety		
	GHD 2019 model		2016 Dam Safety Review model [10]
	2019 parameters	2016 parameters	
FSL	2.09	3.04	3.29 (3.29)
1:100	1.16	1.76	2.27 (2.24)
1:2,000	0.74 (6.7 m crack)	1.17 (6.7 m crack)	2.17 (2.03)
1:10,000	0.63 (9.9 m crack)	1.01 (9.9 m crack)	2.25 (1.68)
PMPDF	0.59 (11.3 m crack)	0.97 (11.3 m crack)	1.85 (0.61)

6 Summary

The following comments are provided as a summary of the results:

- The stability of the primary spillway monoliths is considered to have been marginal at the peak of the 2013 flood event based on the assumptions outlined in this file note.

[10] The factors of safety quoted are for Monolith K Lift Joint #23 at 33.6 mAHD as listed on pdf page 792 of SunWater (2016). The figures in brackets are the minimum factor of safety based on consideration of all lifts, noting that the upper lifts became more critical for higher floods in the DSR analyses.

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- The key reason for the stability of the dam as currently assessed is the significantly lower shear strength on the lift joints than assumed in the design and in the 2016 dam safety review.
- Assumptions in relation to uplift and the effectiveness of the upstream membrane are critical to the stability of the dam.

Regards

[redacted]

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