



# Mine Subsidence Impact on Picton Weir

## Structural Investigation - Finite Element Analysis

Tahmoor Coal Pty Ltd – SIMEC

21 August 2024

Project 311023-50344

**Advisian**  
Worley Group

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### **Appendix A Picton Weir Data**

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# 1 Executive summary

A series of 3D finite element analyses (FEA) of Picton Weir (aka Bargo Dam or Bargo Weir), which is a mass concrete gravity arch weir initially constructed in 1899, with the crest raised once in 1910 and again in 1947, subject to mining-induced far-field regional movement were performed. The weir is founded in Hawkesbury Sandstone. Typical Class I/II Hawkesbury Sandstone rockmass properties were assigned to the rock terrain. The concrete strength and stiffness were based on published information of concrete thin-arch dams constructed in the 19<sup>th</sup> and early 20<sup>th</sup> century. The long-term concrete properties were conservatively assessed and adopted in the analysis.

A review of the risk analysis report performed in 2002 revealed the consequence of dam failure was classified as **Very Low Category**. The then NSW Dam Safety Committee has no requirements for acceptable flood capacity or earthquake stability for this consequence category. The ANCOLD guidelines recommended this weir should pass 1 in 100 to 1 in 1000 AEP floods. However, the risk analysis found that the weir could fail due to downstream rock scour undercutting the foundation when a spillage of 2m above the spillway crest occurs. A downstream impact slab was recommended but it was uncertain if it was ever constructed.

According to the Dam Safety NSW website (<https://www.damsafety.nsw.gov.au/publications/list-of-declared-dams-in-nsw>) Picton Weir is not a declared dam list as of January 2024. It may be that the weir height does not exceed 15m, and the potential loss of life is less than one person – also see Worley Consulting (2024) dam break analysis PAR/PLL assessment. It is probable that the weir was a “prescribed” dam under the old Dams Safety Act (1978). We do not know when and under what circumstances it was removed from the declared dams list. Note that since 1 November 2019, the DSC was abolished and replaced by Dam Safety NSW.

In regard to earthquake requirement, the ANCOLD 2019 Earthquake Guidelines only provide guidance for dams in the LOW to EXTREME categories. There is no guidance for dams that are of VERY LOW category. For LOW category, the earthquake to consider would be 1:500 AEP for Operating Basis Earthquake (OBE) and 1:1000 AEP for Safety Evaluation Earthquake (SEE). Based on AS1170.4, the Australian standard on earthquake, a preliminary estimation of the peak ground acceleration is 0.09g for OBE and 0.12g for SEE. Clarification on earthquake requirement for a non-declared dam should be sought from Dam Safety NSW.

In this study, three baseline cases were considered: (1) the storage is at the spillway crest level referred to as the full supply level (FSL); (2) the storage is empty (Dry); and (3) flood discharge with the upstream water level at the weir crest which is about 0.9m above the spillway crest, referred to as the dam crest flood (DCF). The weir has a significant amount of silt and its pressure acting on the weir and upstream riverbed was included in the analysis. The top of silt was conservatively assumed to be 5m below the weir crest.

Valley closure and upsidence at the weir location was assessed by MSEC. The movements were predicted to be about 20 mm. However, MSEC suggested that other modes of valley movement to be considered in the analysis. The following valley movement types were considered in the study: (1) valley closure only; (2) valley opening only; (3) upsidence only; (4) lateral shear only; and (5) valley closure and upsidence. The upsidence profile was idealised as just a vertical step change at the middle of the weir as suggested by MSEC. The appropriate prescribed displacement corresponds to the different valley movement types was applied to the model side and base boundaries. The movement was gradually increased until it reached 40mm - twice the amount of the predicted value. It should be

noted that the original design of the weir would not have considered any of the above valley movements.

The concrete damage was assessed based on how much strength capacity was mobilised when the weir was subjected to the increasing valley movements. A tensile strength of 2 MPa and a compressive strength of 20 MPa were used in a Drucker-Prager Yield Function, which used the computed principal stresses in the weir, to calculate the strength mobilisation factor. When this factor is equal to or greater than 1.0, it implies the concrete material has failed either in tension, compression or in shear. By tracking the development of the strength mobilisation and assessing the damage extent, the critical valley movement was estimated.

The analysis results showed that the weir is most vulnerable to valley opening, followed by upsidence, valley closure with upsidence, lateral shear and finally valley closure. Their corresponding critical movements are:

- Valley opening only: ~ 8 mm
- Upsidence only: ~ 8 mm
- Valley closure and upsidence: ~ 10 mm
- Lateral shear only: ~ 15 mm
- Valley closure only: ~ 15 mm

We provisionally suggested that for the mine subsidence management plan the trigger level should be half of the above movements. The valley opening only and upsidence only could lead to collapse and partial collapse of the weir. It is noted that these two types of movement on their own are unlikely to occur based on past survey data at similar sites (MSEC to confirm). The most probable movements are valley closure accompanied with upsidence, and valley closure with little or no upsidence. We do not know if lateral shear in a valley has been observed before. These last three types of movement could lead to cracking of the weir resulted in a significant leakage. Localised or complete weir collapse is unlikely to occur. Note that dewatering would not prevent the weir from cracking except there will be less or no seepage/leakage of water through the cracks when dewatered. A reduced water level upstream would also mean a lesser hydrostatic pressure acting on the weir and therefore the weir stability would increase.

The current sliding FOS of the weir was estimated using the conventional stability calculation to be above 2.00. The FEA model could provide an insight into the extent of concrete damage as a result of valley movement. However, the model has a limitation that it cannot provide a reliable the crack location or to quantify the crack length. Any pre-existing cracks in the weir structure can affect the modelling results. These defects were not included in the model as they are difficult to quantify without a detailed survey of the upstream and downstream faces of the weir. Therefore, it was difficult to estimate for certain as to when the weir or a portion of the weir become unstable as a function of valley movement. Furthermore, the applied valley movement profiles in the model were idealised, and they cannot be verified by limited movement survey data.

To manage the risk of the weir structure from mining damage, a detailed survey and monitoring plan should be implemented in order to measure 3D displacement across the valley, across the weir, along the weir crest, downstream riverbed for sign of upsidence (note that it may not be possible due to the presence of water), and the downstream river banks close to the weir (to capture upsidence and to provide additional information on the other valley movement modes). Periodic visual inspection of the weir should be done so to monitor any propagation of existing cracks/defects and to capture any new cracks happening during mining.

When considering mitigation actions before, during and post-mining, the following should take into consideration:

- The weir has been classified as a **Very Low** consequence category if the weir suffers a dam break event. It was assessed by SKM in 2000 that under the worst case scenario the estimated loss of life is 0.1 person during a Sunny Day failure and the same for the probable maximum precipitation design flood (PMPDF) with dam failure. A recent dam break study of the weir was carried out by Worley Consulting (2024). For the Sunny Day failure the potential loss of life (PLL) was estimated to be 0.09. The PLL at the two houses along the fire trail downstream of the weir was estimated to be 0.63 during the probable maximum flood (PMF) without weir failure, and 0.64 during the PMF with weir failure, giving an incremental PLL of 0.01.
- The current and future function of the weir. Currently it is not used for water supply or recreation. Water leakage of the damaged weir could be acceptable as long as it does not lead to collapse of the structure. It should be noted that the outlet valve has been left open allowing water releasing downstream all the time.
- The risk analysis done in 2002 showed the spillway is inadequate in passing the required floods unless an impact slab or apron has been installed downstream of the weir to prevent scour of the rock. If the impact slab has not been installed, then the weir could fail (collapse) during extreme flood events or when flood level is 2m or more above the spillway crest. More cracks in the weir as a result of mining could further reduce the spillway capacity from passing floods. In other words, the damage caused by mining is unlikely to cause an immediate weir failure, but the damaged weir may fail when the next significant flood occurs.
- The environmental impact and heritage significance of the weir. Can the weir be demolished completely or partially? Assuming the current outlet valve may not be adequate to drain water during floods, putting an opening through the weir or demolish it could address the above issue of spillway discharge capacity.
- Last but not least, the cost and timing to implement any mitigations/remediations. Impact on mining operation.

Below are a few suggestions to be considered:

Before mining:

- Strengthen concrete from cracking – not practical as we do not know the actual valley movement modes.
- Lower storage/dewatering – if easy to do using the existing outlet valve.
- Partial or complete demolition of the weir (if permitted to proceed) – Cost and timing.
- Cutting a slot in the rock abutment on one side of the weir to isolate the weir from valley movement – need to determine the extent of the slot (how deep and how long). Access, cost and timing need to be considered.

During mining:

- Repair cracks – not practical
- Strengthen concrete from cracking – not practical
- Lower storage/dewatering – if easy to do using the existing outlet valve

- Cutting a slot in the rock abutment on one side of the weir to isolate the weir from valley movement – need to determine the extent of the slot (how deep and how long). How long does it take to implement the work?
- Pause mining activities if movement of valley is deemed excessive

#### Post-mining:

- Re-assess weir if it can still safely pass anticipated floods – this will determine the repair/remedial options
- Repair cracks
- Strengthen weir with post-tensioned anchors
- Stabilise weir by placing mass concrete downstream of weir
- Decommission – partial or complete demolition of weir

Performing mitigations during and post-mining is considered to be not practical. Mitigations be best done before mining, and any residual issues such as cracks and leakage can be repaired post-mining. The following pre-mining mitigation options should be investigated further.

- Partial or complete demolition of the weir as it currently does not provide any service to the community. Its removal will alleviate the risk of a weir failure.
- Install post-tensioned anchors to stabilise the weir in case it suffers a significant damage that may make the weir or part of the weir become unstable. Damage such as cracks and leakages can be repaired post-mining.
- Install a downstream concrete apron to provide more passive resistance to stabilise the weir. It can also minimise the risk of erosion of the downstream rock abutment/foundation that may undermine the weir during large flood events.

## 2 Introduction

Advisian (Worley Group) has been requested by Tahmoor Coal Pty Ltd (SIMEC Group) to investigate mine subsidence impact on the Picton Weir. The weir is located on Bargo River, NSW, and therefore it was also known as Bargo Weir. The weir could be affected by far-field ground movement caused by LW S1A to S7A as part of the Tahmoor South Project (see Figure 2-1 and Figure 2-2). The ground movement associated with the mined longwalls can potentially affect the structural integrity of the weir.

Picton Weir is a heritage-listed structure which was designed and installed by the Public Works Department in 1899. The original crest was at RL 912' impounded 24.5 million gallons (111.38 ML) of water. In 1910, the level was raised 8 ft (2.438 m), increasing the storage to 38.5 million gallons (175.02 ML). It was raised a further 7 ft (2.134 m) in 1947, bringing the capacity to 67 million gallons (304.59 ML)

The general arrangement of the weir is shown in Figure 2-3. The weir data is summarised in Appendix A.

Based on the surveillance report, the weir is about 13 m high, and it is located in a sandstone gorge (Hawkesbury Sandstone). The weir is a mass concrete gravity arch structure with a radius of 120 ft (36.58 m), where the abutment is keyed into the sandstone cliffs. As shown in Figure 2-2, the crest is 4 ft (1.22 m) wide and the base is 13.62 ft (4.15 m) wide. The maximum thickness of the weir is 13.62' (4.15 m) at its base. There is a shear key at the base of the weir.

The main objectives of the study include a structural assessment of the weir at its current condition through nonlinear finite element analysis, analysis of the effect of the weir structure when there is a far-field movement occurring, examination of the concepts that could alleviate the impact on the weir structure and lastly, provide survey monitoring strategy and trigger levels into the Mine Management Plan and the Triggered Action Response Plan (TARP) which is being prepared by MSEC.

Details of the methodology, inputs, assumptions, results, discussion and conclusions are described in this report.

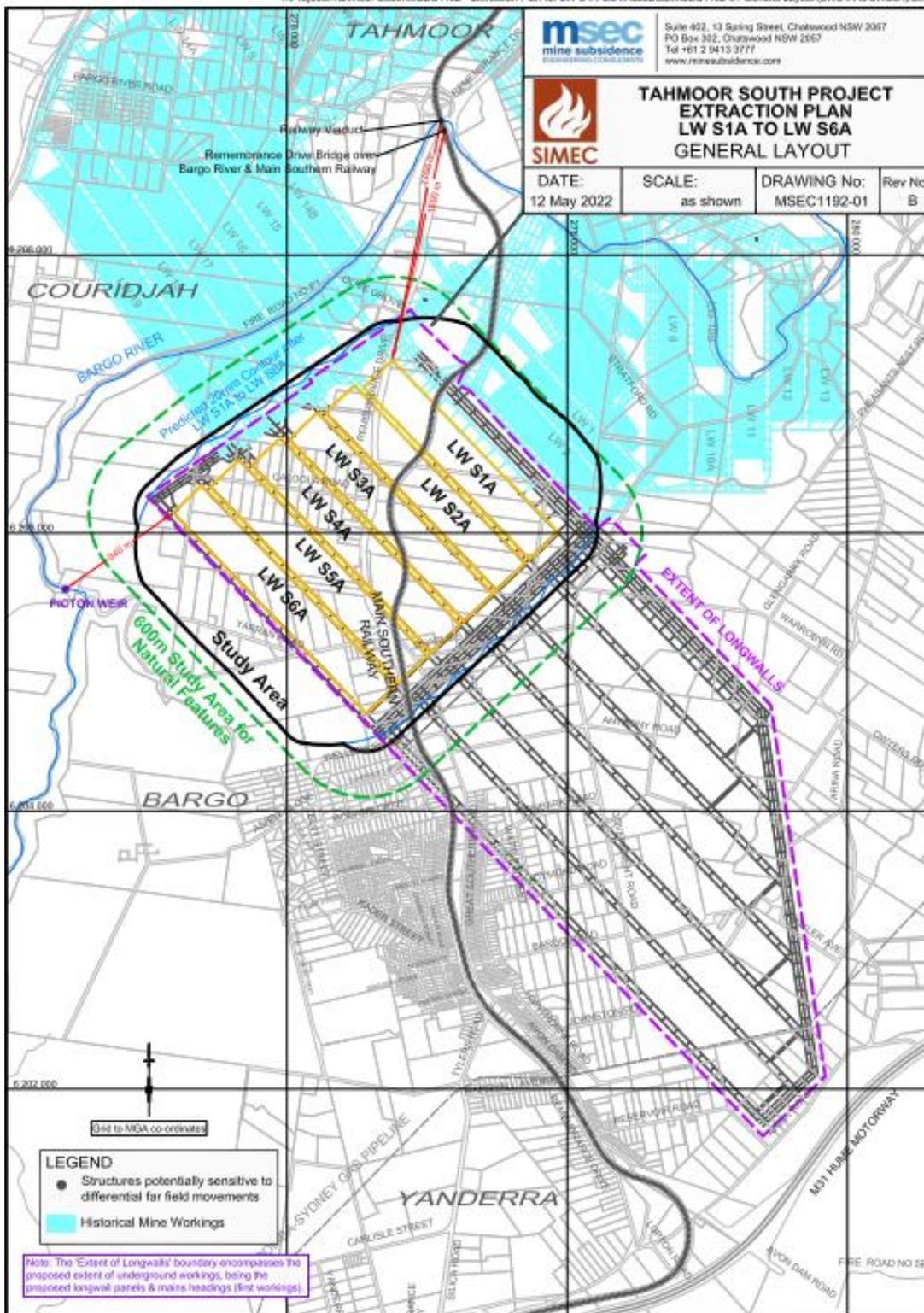
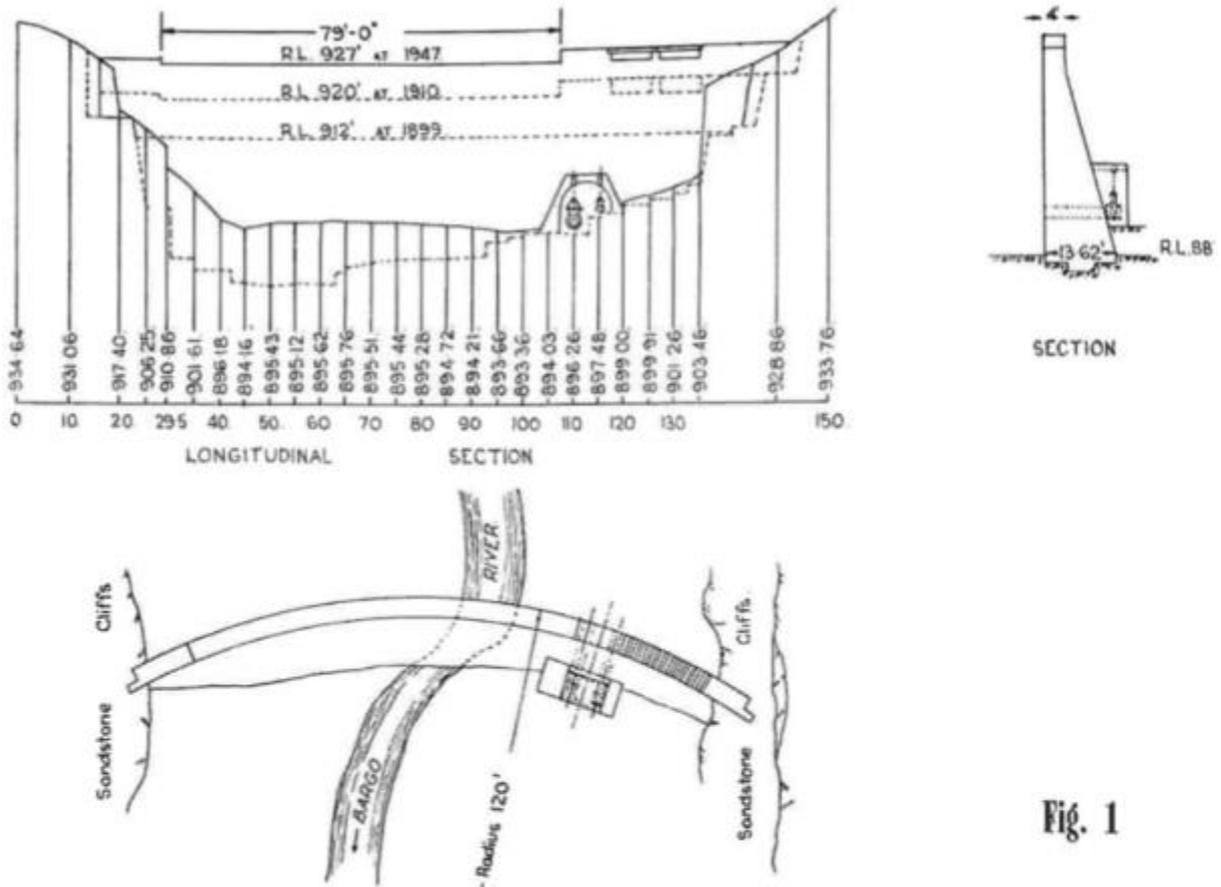


Figure 2-1: Proposed longwall layout (LW S1A to S6A) in relation to Picton Weir (Source: MSEC)





**Fig. 1**



Figure 2-3: Picton Weir – general arrangement (top) and photo of the weir (below) (Source: Sydney Water Board

Journal Vol. 4 No.1, 1953)

## 3 Finite Element Analysis

### 3.1 Model Description

The geometry and dimensions of the weir model were generated based on the supplied general arrangement drawings.

The terrain model of the weir foundation and abutments was based on the contour data provided by the surveyor, Michael Nicholson. It was simplified as the details upstream and downstream of the concrete weir would not affect the structural behaviour. Several views of the geometry and finite element model are shown in Figure 3-1 to Figure 3-6. Note that the available drawings of the weir indicate the elevation in feet, whereas the Lidar data provided by the surveyor (see Figure 3-7) is in AHD (m). As such, the RL in unit mAHD,  $RL(m) = RL(ft) \times 0.3048 + 0.47m$ . It was assumed the spillway crest RL of 927' as shown in the drawing (see Figure 3-8) has not significantly changed since 1947.

The gravity arch weir is represented by hexahedral finite elements with a uniform mesh. There are at least 14 elements across the weir section so that the bending action of the structure can be captured accurately. The shear key on the heel of the weir of each block is included in the model as it will provide sliding stability.

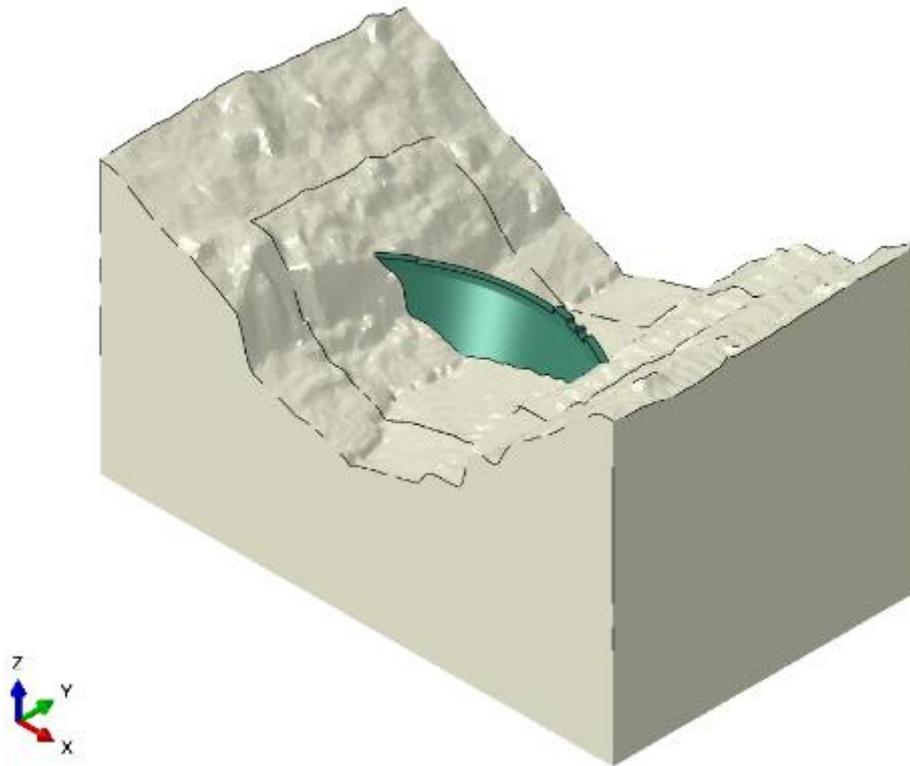
The rock foundation is represented by a mixture of hexahedral and tetrahedral elements. In order to avoid the weir behaviour being affected by the model boundaries, the foundation boundaries are extended to a distance of at least 1.5 times the maximum height of the weir.

The orientation of the model is such that:

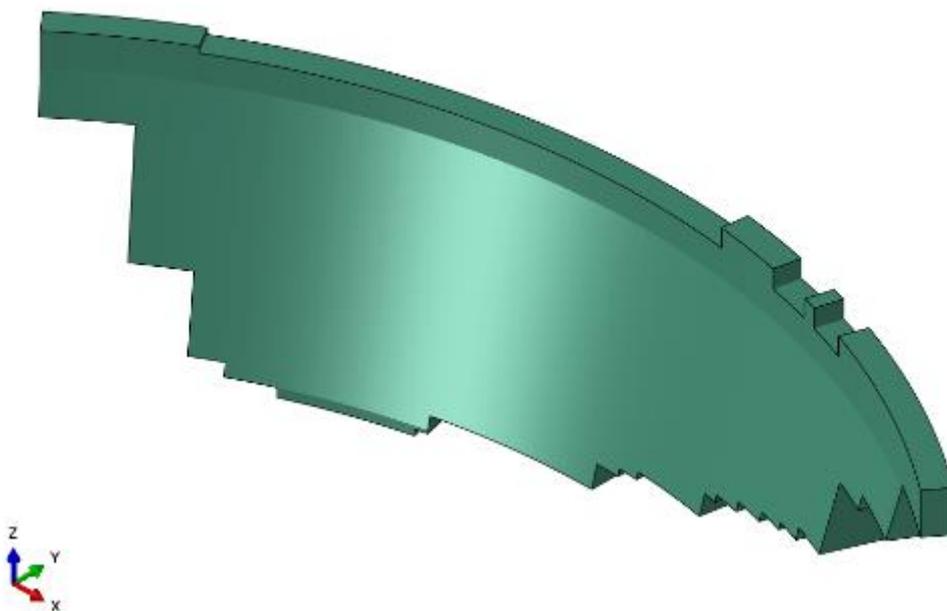
- Global x-direction: Cross-valley direction with positive x pointing towards the left abutment
- Global y-direction: Upstream-downstream direction with positive y pointing towards upstream
- Global z-direction: Vertical direction with positive z point upwards

A cylindrical coordinate system is used for extracting forces and stresses developed in the arch sections.

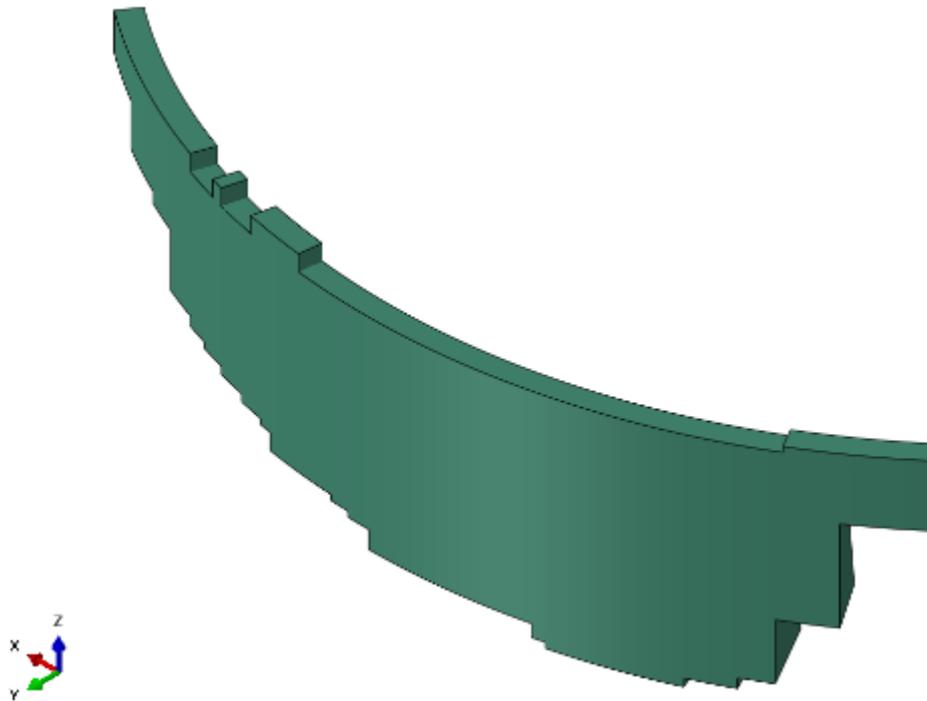
The modelling and stress analysis were performed using the finite element software, Abaqus, which is licensed to Advisian. Consistent SI units were used in the software: that is, length (m), mass (kg), time (s), force (N), temperature (°C), pressure and stress (Pa).



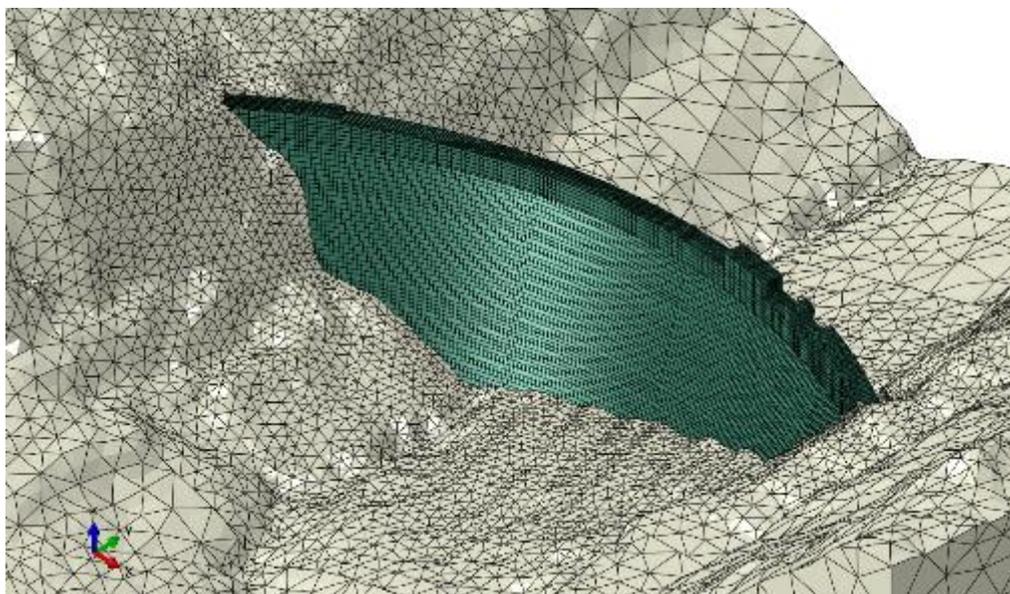
*Figure 3-1: An overview of the terrain and weir model*



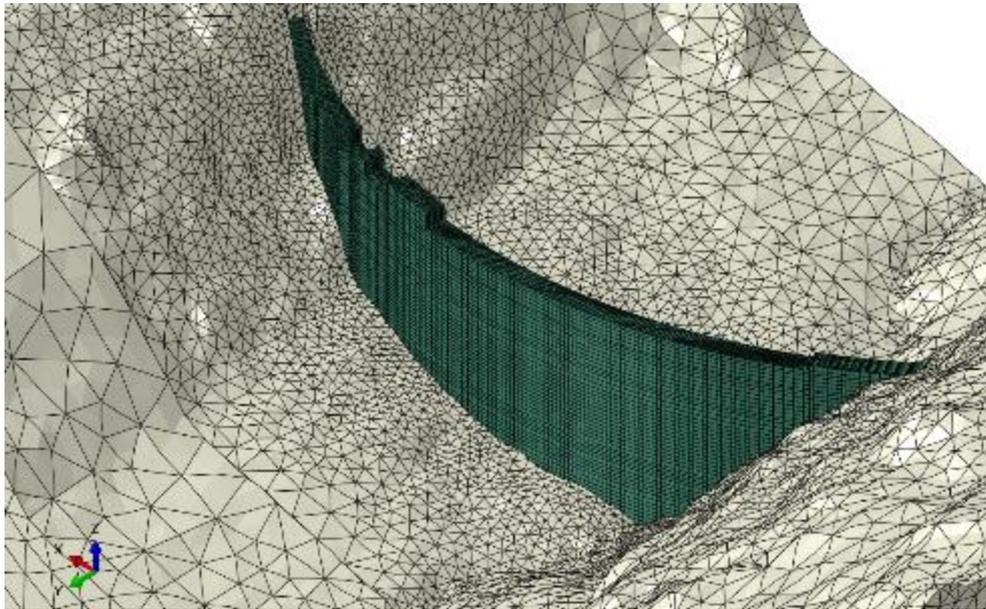
*Figure 3-2: An iso-view of the weir look from downstream*



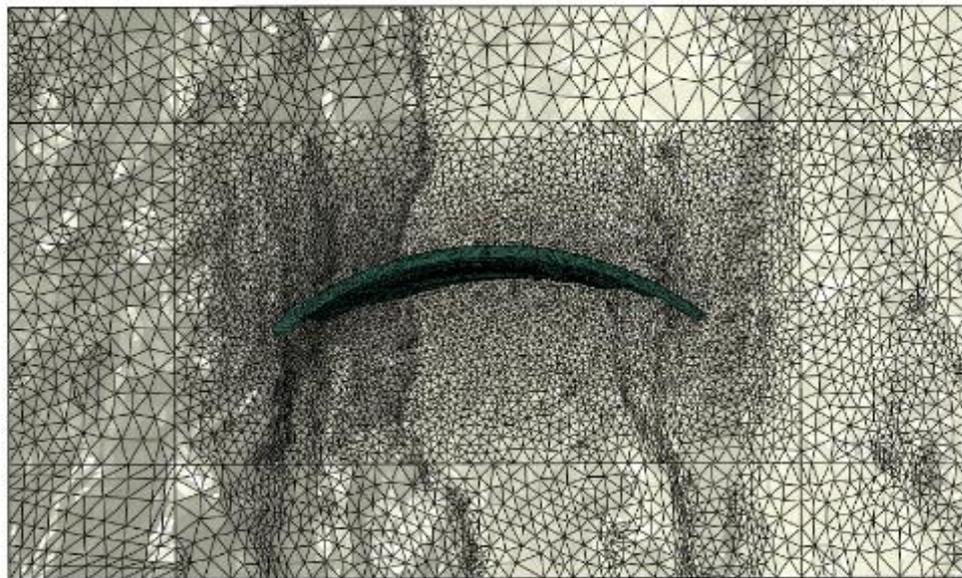
*Figure 3-3: An iso-view of the weir look from upstream*



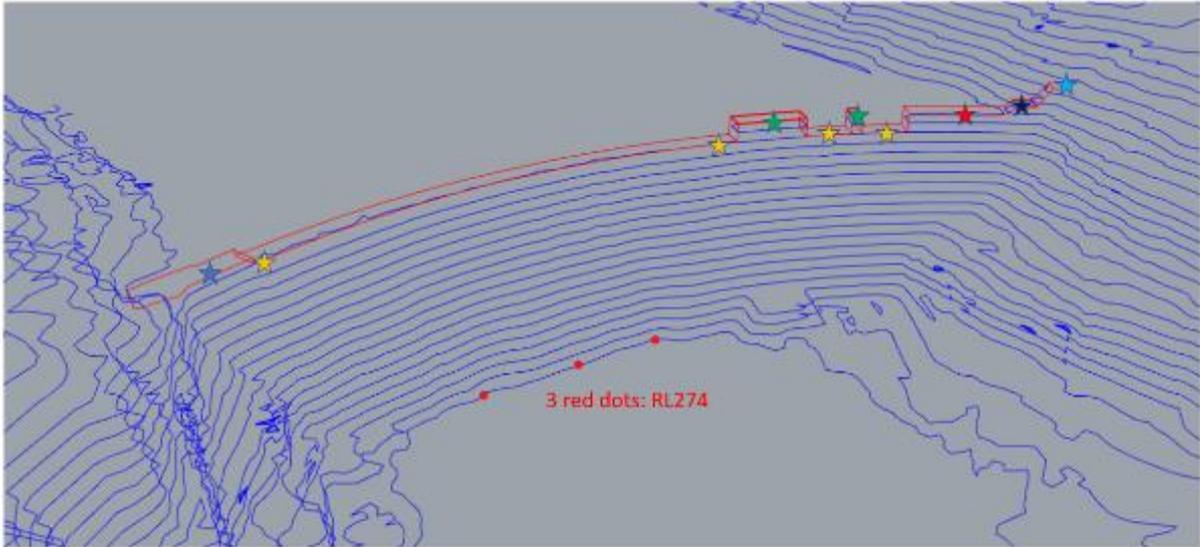
*Figure 3-4: An iso-view of the finite element mesh of the weir and the surrounding terrain from downstream*



*Figure 3-5: An iso-view of the finite element mesh of the weir and the surrounding terrain from upstream*



*Figure 3-6: Plan view of the finite element mesh*



- 283.02 ★
- 283.28 ★
- 283.92 ★
- 283.975 ★
- 284.36 ★
- 285.26 ★

Figure 3-7: Contours with elevation data provided by Michael Nicholson

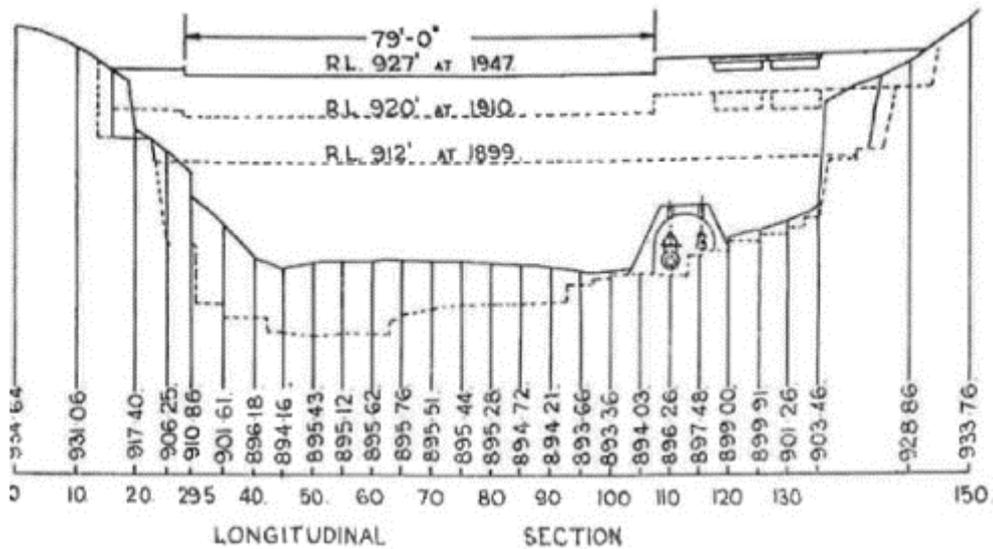


Figure 3-8: Elevation view from the weir drawing which shows the crest level at each stage

### 3.2 Geostatic Stress

An initial geostatic stress was set up in the rock terrain in the model. The stresses were allowed to be redistributed in accordance with the geometry of the valley in the model. In the absence of site-specific stress data, the following initial stress field according to Pells (2004) was applied in the model:

$$\sigma_1 = \sigma_{NS} = 1.5 + 1.2\sigma_v \text{ to } 2.0\sigma_v \quad \text{MPa}$$

$$\sigma_2 = \sigma_{WE} = 0.5\sigma_1 \text{ to } 0.7\sigma_1 \quad \text{MPa}$$

$$\sigma_3 = \sigma_v = 0.024H \quad \text{MPa}$$

$\sigma_{NS}$  is the horizontal stress in the North-South direction and  $\sigma_{WE}$  is the horizontal stress in the West-East direction, and  $H$  is the depth.

### 3.3 Material Properties

The material properties used in the static analyses are summarized in Table 3-1. The foundation stiffness and strength parameters were obtained based on the PSM reports and PSM's advice on the rockmass properties. The weir is founded in Class I/II Hawkesbury Sandstone. Typical material properties corresponding to the sandstone classification are used in the model. No rock defects or joints are explicitly included in the finite element model.

However, regarding the weir concrete properties, it should be noted that there are no material test data of the weir available for this study. The concrete strength is based on those typically constructed in that era — the structure is a Darley-Wade thin-arch dam design.

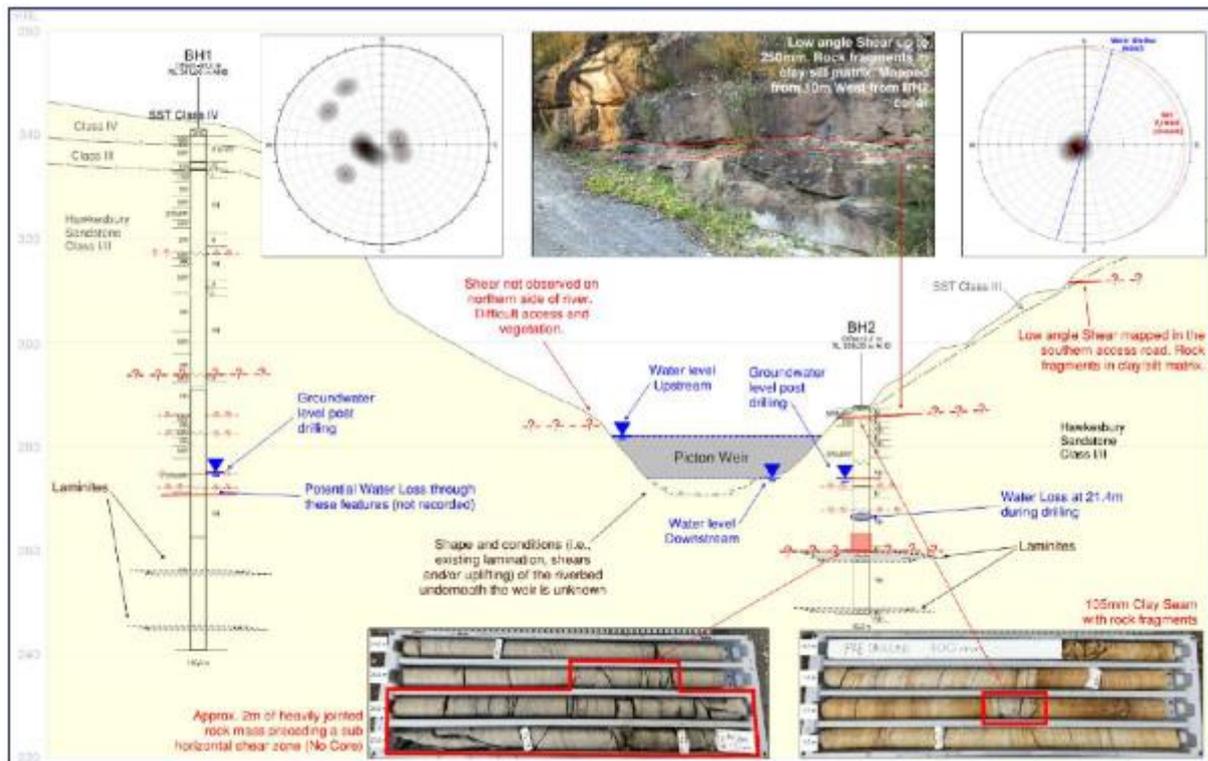


Figure 3-9: PSM Conceptual Geological Model – Cross Section through Picton Weir (Source: PSM)

Table 3-1: Material Properties - Static Analysis

Materials	Parameters (Static Analysis)		
	Young's modulus, E	Poisson's ratio, $\nu$	Bulk density, $\rho$
<b>Units</b>	GPa	-	Kg/m <sup>3</sup>
<b>Concrete</b>	25	0.2	2446
<b>Rockmass</b>	2.5	0.25	2650

### 3.4 Boundary Conditions

The following fixities were assigned to the rock foundation block:

- US and DS faces: vertical roller support, restrained in the y-direction
- Left and right faces: vertical roller support, restrained in the x-direction
- Foundation base: restrained in all three directions

Note that these boundaries were switched to displacement-controlled during valley movement in the valley movement analysis step.

### 3.5 Load Cases

The following loads were considered in the analysis:

DL:	Dead load (gravity) of the weir wall structure
FSL:	Hydrostatic load with storage at full supply level (U/S RL 283.02 mAHD)
DCF:	Dam crest flood event (U/S RL 283.92 and D/S RL 276.5)
Silt:	Top of silt at RL278.02, assumed a bulk unit weight of 17 kN/m <sup>3</sup>

It should be noted that the FSL in the present study is on the spillway crest.

Three baseline cases were considered:

1. FSL: DL + Silt + hydrostatic
2. Dry: DL + Silt
3. DCF: DL + Silt + hydrostatic for DCF

For each baseline case, the flowing analysis steps were performed:

1. Redistribute geostatic stress in the terrain (without weir)
2. Construct weir in three stages (gravity on concrete)
3. Apply hydrostatic pressure (FSL, DCF)
4. Apply silt pressure (FSL, Dry, DCF)
5. Apply valley movement gradually to 40mm

#### 3.5.1 Siltation Pressure

According to the 2004 surveillance report, the presence of silt was observed when the storage level was very low during dry periods. It was estimated that the silt was approximately 5m below the spillway crest. In the analysis, the buoyant unit weight of 7 kN/m<sup>3</sup> was applied to the upstream face of the dam and the affected terrain when the storage is at FSL and during dam crest flood, DCF events. However, for the dry conditions, the bulk unit weight of 17 kN/m<sup>3</sup> was applied. The horizontal silt pressure acting on the wall was assumed to be hydrostatic (i.e.  $K = 1.0$ ), which was a conservative assumption.

#### 3.5.2 Hydrostatic Load

Hydrostatic pressure corresponding to the FSL (RL 283.02 mAHD) was applied to the wetted surfaces of the upstream terrain and the upstream face of the weir. On the other hand, the tailwater pressure was applied to the wetted surfaces of the downstream terrain and the downstream face of the weir.

### 3.5.3 Dam Crest Flood Cases

For the dam crest flood cases considered in the analysis, the hydrodynamic effect of the flow over the spillway was not considered. A hydrostatic pressure corresponding to the dam crest flood level was assigned to the model and this assumption was conservative in terms of arch actions on the abutments. The storage level corresponding to the dam crest flood level is:

Upstream RL 283.92 mAHD	0.9 m above spillway crest
Downstream RL 276.5 mAHD	6.52 m below spillway crest

### 3.5.4 Mining Impact

To assess the mining impact, five mining-induced ground movement cases and combinations were considered as shown in Table 3-2. Each type of ground movement is illustrated in Figure 3-10 to Figure 3-14.

Table 3-2: Ground movement cases to assess the mining impact.

	Valley closure	Valley opening	Upsidence	Lateral shear
Valley closure	Y		Y	
Valley opening		Y		
Upsidence			Y	
Lateral shear				Y



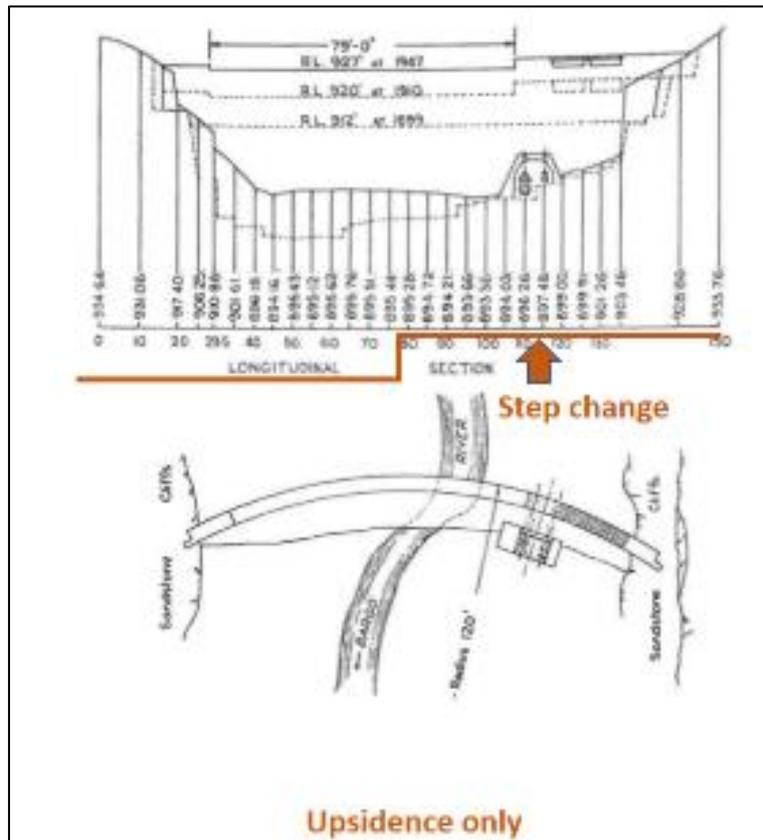


Figure 3-12: Upsidence movement

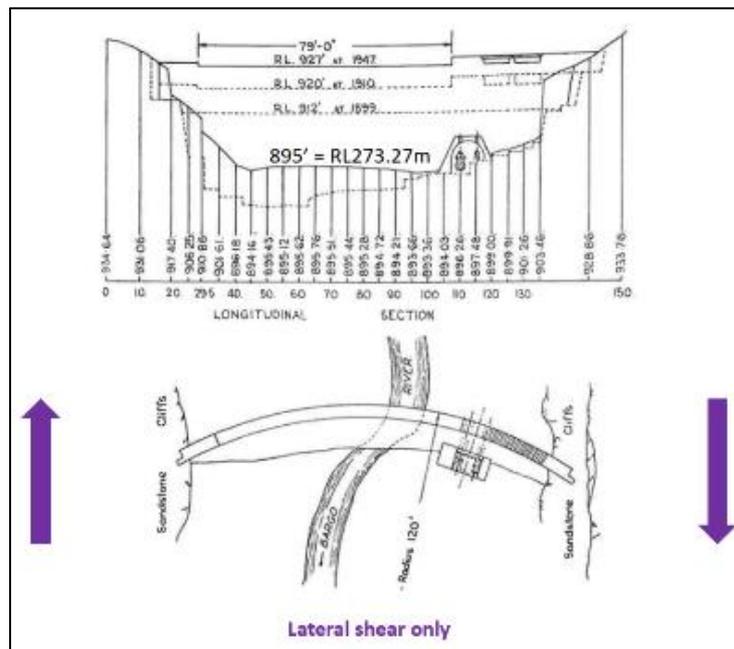


Figure 3-13: Lateral shear movement

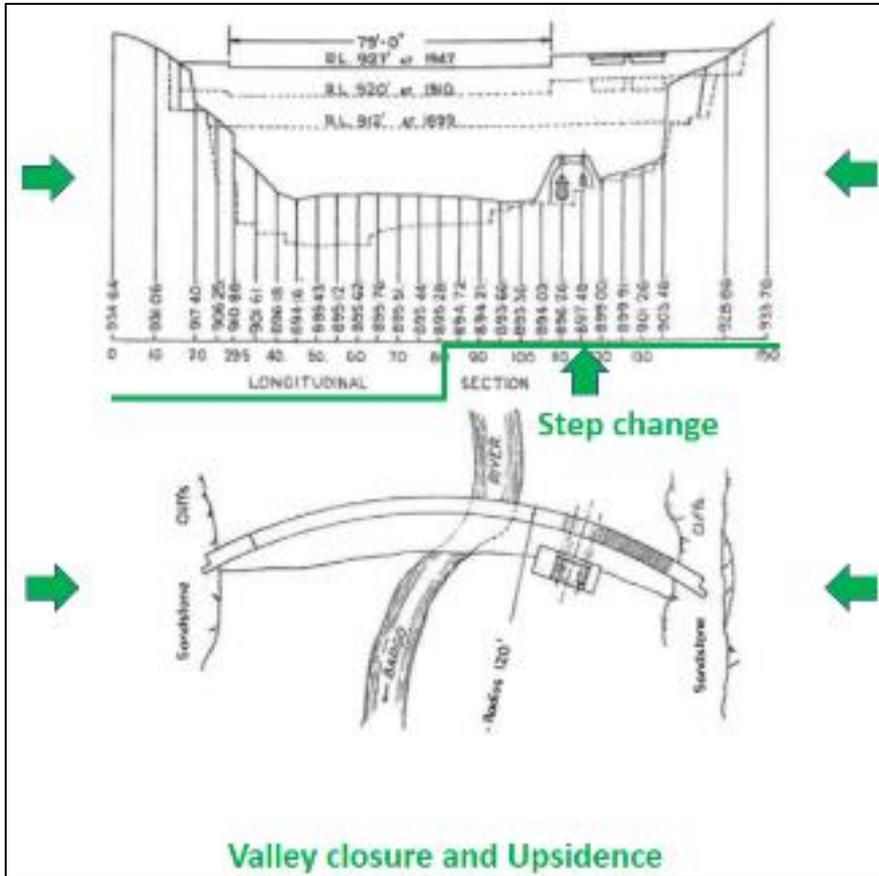


Figure 3-14: Valley closure and upsidence movement

## 4 Assessment Criteria

### 4.1 Weir Strength Capacity for Assessment

For the typical concrete mix in the 19th and early 20th century, the compressive strength is likely to vary from 15 to 20 MPa (Creager, 1917) accounting for strength gain with time (USBR, 2005). It has been reported that (HECEC, 1999) the allowable compressive stress is 5 MPa. According to the ANCOLD 1998 guideline for concrete gravity dam for earthquake design, the factor of safety for compression is 4. A similar safety factor of 4 is also recommended by USACE (1994). Therefore, the compressive strength of 20 MPa is a reasonable value. Note that a higher safety factor of 5 for Darley-Wade thin arch dams in NSW was reported (Chanson and James, 1998).

The tensile strength of plain concrete is typically 10% of the compressive strength. A 2 MPa tensile strength is used for the present assessment.

Assuming the concrete has not suffered any alkali-aggregate reaction, the stiffness typically varies from 20 to 35 GPa. A value of 25 GPa was used for the concrete stiffness.

Table 4-1: Material strength for assessment

Materials	Compressive strength	Tensile strength – static
	(MPa)	(MPa)
Concrete	15 to 20 use 20	2

### 4.2 Assessment Methodology

The performance of the weir was assessed by examining the computed strength mobilisation when mining-induced ground movement was applied.

The structural integrity of the entire weir was assessed using the computed principal stresses. A Drucker-Prager yield function based on the material's compressive and tensile strengths was used to calculate how much strength capacity is mobilised. If the mobilisation reaches 100% and occurs over an extensive region, then there is sufficient damage to cause leakage and in the worst case, structural collapse. At each point in the elements, the following condition was checked if the left-hand side exceeds the right-hand side. If so, the material has failed.

$$\sqrt{\frac{1}{6} [(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]} = A + B (\sigma_1 + \sigma_2 + \sigma_3)$$

The LHS term is the equivalent Mises stress where  $\sigma_1$ ,  $\sigma_2$  and  $\sigma_3$  are the principal stresses and the RHS term represents the material capacity where the parameters A and B are given by:

$$A = \frac{2}{\sqrt{3}} \left( \frac{\sigma_c \sigma_t}{\sigma_c + \sigma_t} \right); \quad B = \frac{1}{\sqrt{3}} \left( \frac{\sigma_t - \sigma_c}{\sigma_c + \sigma_t} \right)$$

where  $\sigma_c$  = compressive strength and  $\sigma_t$  = tensile strength of the weir material.

By tracking the development of the Drucker-Prager strength capacity mobilisation, the amount of damage caused by various mining-induced ground movements can be quantified, and from the affected regions, how the weir may fail.

In addition to the strength capacity mobilisation parameter, the principal compressive and tensile stresses in the concrete were also assessed to confirm the damage expectation.

## 5 Results and Assessment

The development of strength mobilisation contours plots for the three baseline cases and the five valley movements are provided in Appendix B. Each plot shows a view of the weir looking from downstream (top plot) and from upstream (bottom plot). The contour scale is such that strength mobilisation equal to or greater than 1.0 shows the concrete material has failed (i.e. the grey zone).

Figure 5-1 shows the amount of strength mobilisation in terms of the percentage volume of the weir as a function of valley movements. It can be observed that there is not much difference between the baseline cases. In fact, the dry weir appears to be marginally more critical. This implies dewatering may not prevent structural damage as the stress state in the weir is mainly driven by the valley movement. However, dewatering will reduce the consequence from potential structural failure of the weir.

The other observation is that the weir is more vulnerable to (worst movement type first):

1. Valley opening
2. Upsidence
3. Valley closure and upsidence
4. Lateral shear
5. Valley closure

Figure 5-2 to Figure 5-6 show the development of strength mobilisation at a couple of selected locations for the 5 types of valley movement.

By examining the contour plots (Appendix B) of the extent and the graphs of strength mobilisation development, the critical valley movement at which the concrete has failed for the five movement types are summarised in Table 5-1.

Table 5-2 provides a summary of the potential failure mode of the weir when the critical valley movement is reached and some suggested mitigations during and post mining to consider. The locations of the failures will provide a guide to the locations where crack monitoring should occur depending on the likely valley movement type.



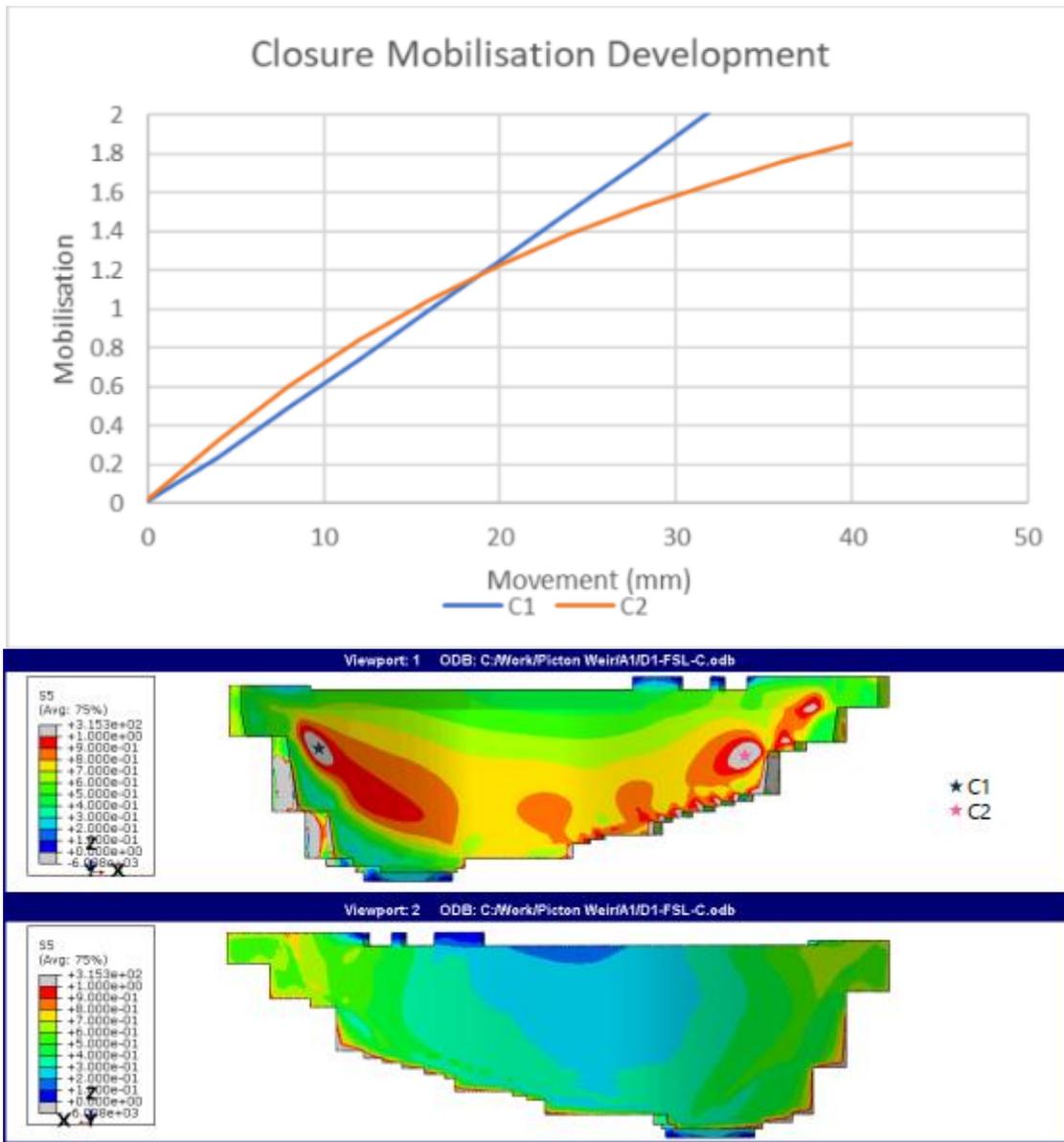


Figure 5-2: Valley closure - strength mobilisation development (top) at selected locations (below)

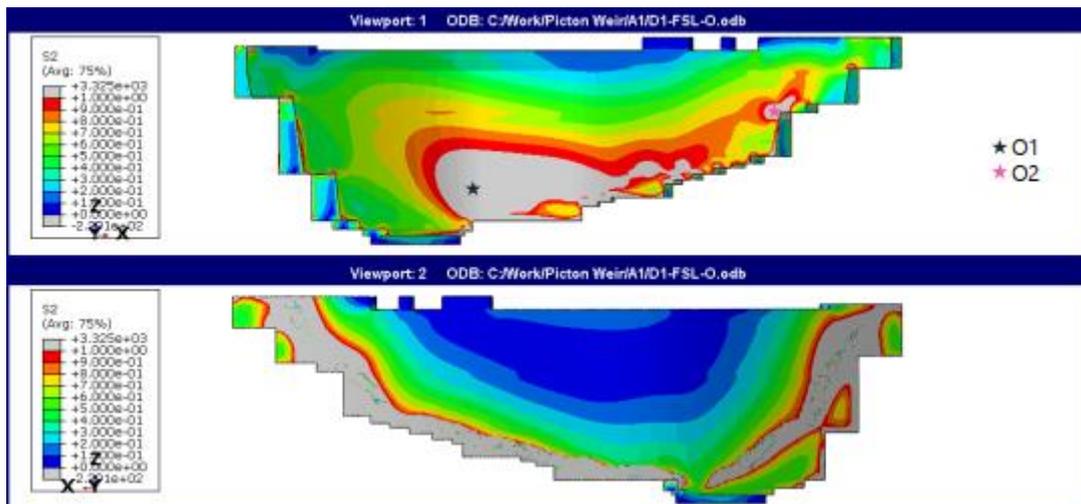
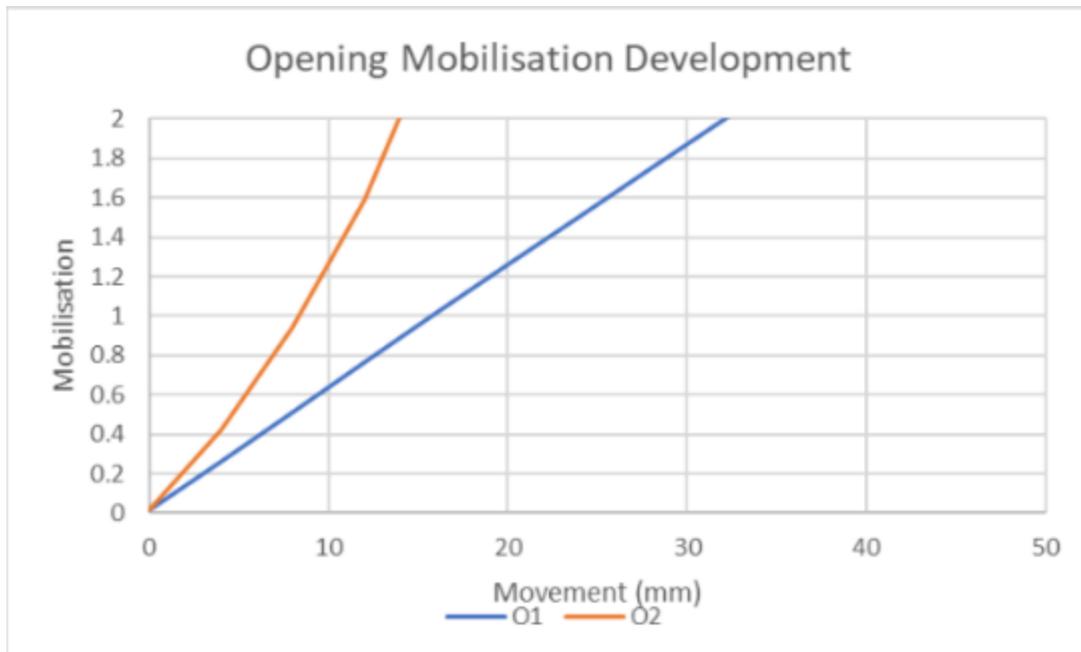


Figure 5-3: Valley opening - strength mobilisation development (top) at selected locations (below)

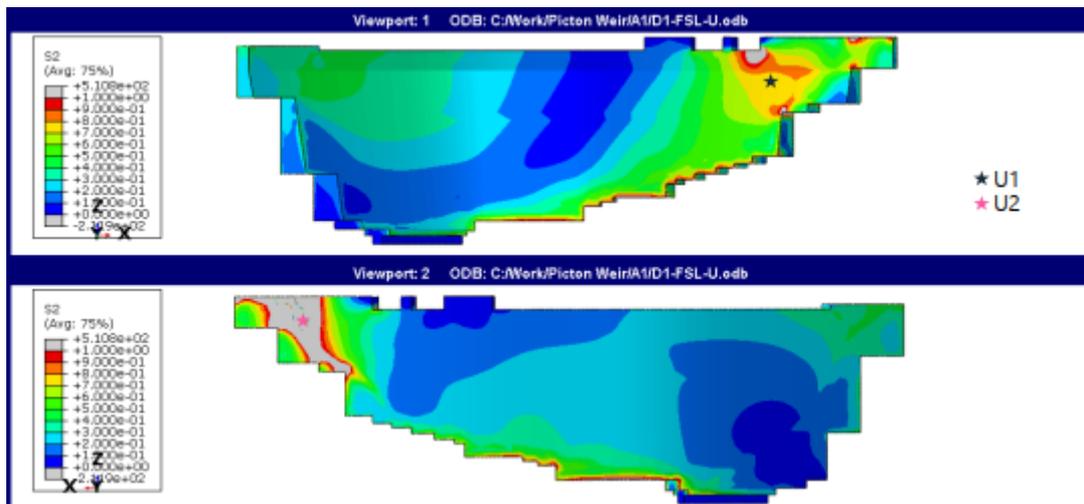
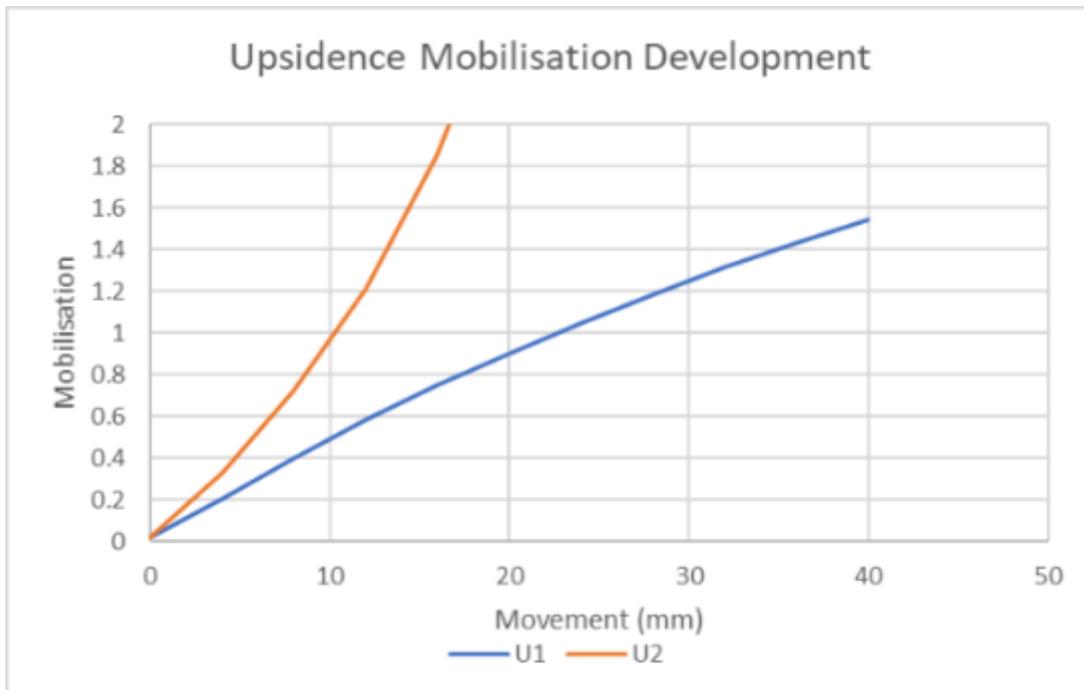


Figure 5-4: Upsidence - strength mobilisation development (top) at selected locations (below)

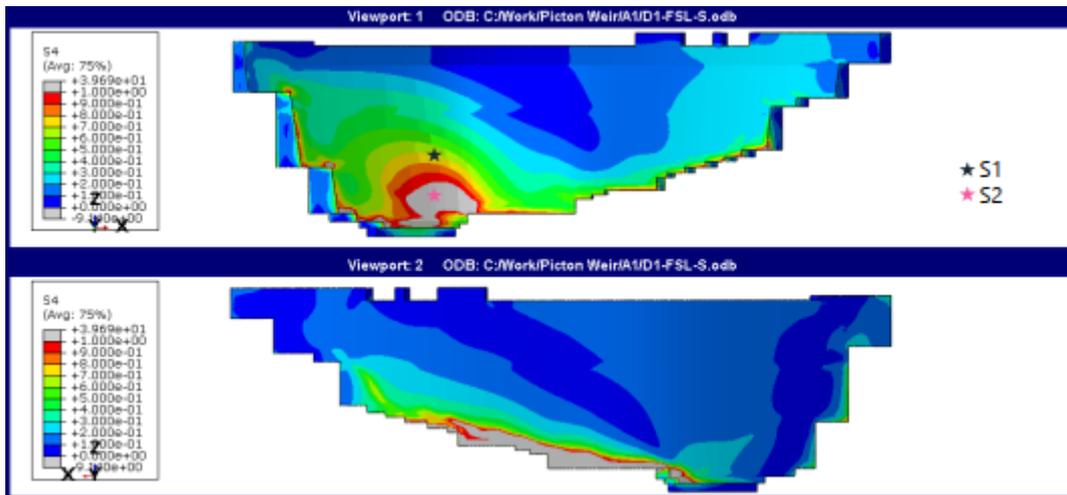


Figure 5-5: Lateral shear - strength mobilisation development (top) at selected locations (below)

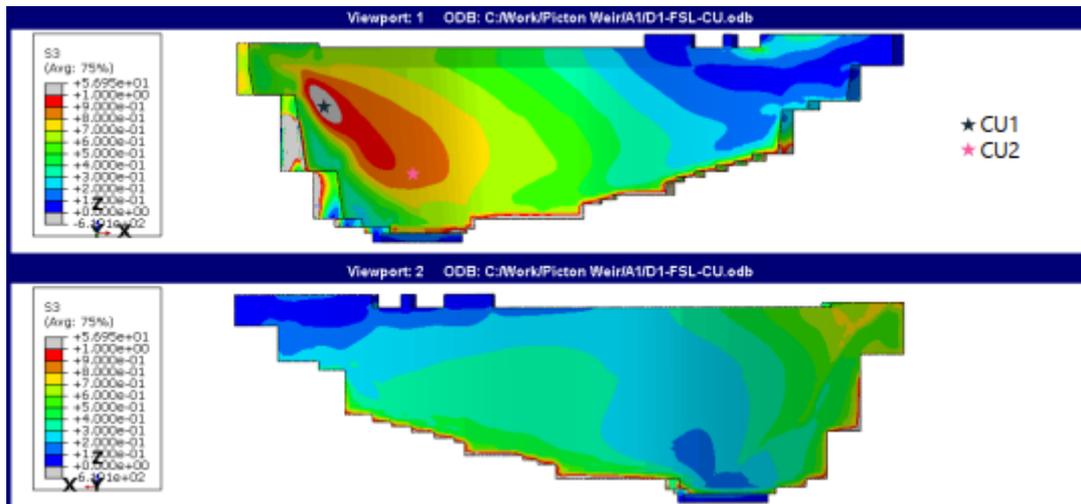
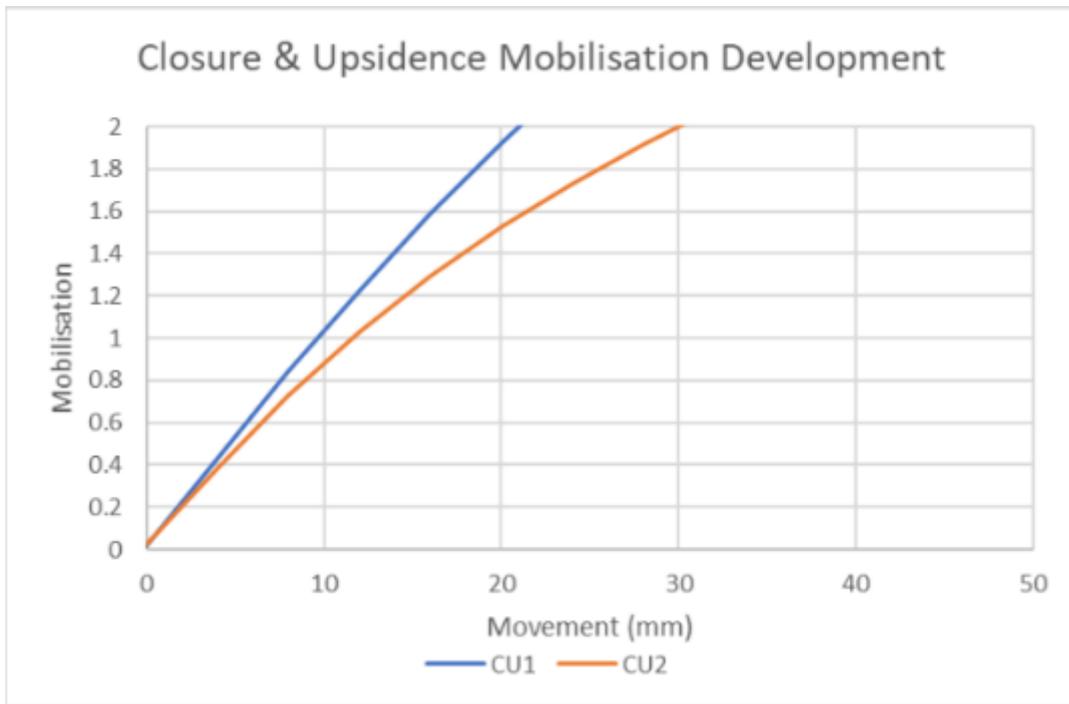


Figure 5-6: Valley closure and upsidence - strength mobilisation development (top) at selected locations (below)

Table 5-1: Critical valley movement based on development of strength mobilisation

Movement types	Critical movement	Suggested trigger movement	Potential Failure mode
Valley opening only	~ 8 mm	4 mm	Vertical cracks across weir
Upsidence only	~ 8 mm	4 mm	Shear induced vertical cracks along upsidence region
Valley closure & upsidence	~ 10 mm	5 mm	Horizontal cracks downstream of weir & shear induced vertical cracks in upsidence region
Lateral shear only	~ 15 mm	7.5 mm	Shear induced vertical cracks across weir
Valley closure only	~ 15 mm	7.5 mm	Horizontal cracks downstream of weir

Table 5-2: Summary of failure mode and suggested mitigation during and post mining

Valley movement type	Likelihood of movement	Weir damaged at	Potential failure mode	Suggested trigger level	Actions	Post-mining actions
Valley Opening Only	Unlikely	8 mm	Cracks in D/S face at base; cracks along U/S rock interface. Leakage through crack zone. Through cracks at base in middle of weir. Wall in the middle becomes unstable & collapse.	4 mm	Review survey data. Review weir condition. Lower storage. Stop mining. Cut slot - isolate weir from further movement	Repair cracks. Strengthen weir with post-tension anchors and/or concrete placement D/S of weir. Decommission weir: remove entire weir or remove partial (centre) weir.
Upsidence Only	Unlikely	8 mm	Cracks in left U/S face from top to base; cracks in left D/S face near top. Leakage through the damaged left side. Localised collapse of the left weir section.	4 mm	Review survey data. Review weir condition. Lower storage. Stop mining.	Repair cracks. Strengthen weir with post-tension anchors and/or concrete placement D/S of weir. Decommission weir: remove entire weir or remove partial (centre) weir.
Valley Closure & Upsidence	Likely	10 mm	Cracks in right D/S face along rock interface. Leakage through cracks.	5 mm	Review survey data. Review weir condition. Lower storage. Stop mining. Cut slot - isolate weir from further movement	Repair cracks. Strengthen weir with post-tension anchors and/or concrete placement D/S of weir. Decommission weir: remove entire weir or remove partial (centre) weir.
Lateral Shear Only	Possible	15 mm	Cracks in right D/S face at base; cracks at U/S along dam/rock interface in the middle. Leakage through cracked base.	7.5 mm	Review survey data. Review weir condition. Lower storage. Stop mining. Cut slot - isolate weir from further movement	Repair cracks. Strengthen weir with post-tension anchors and/or concrete placement D/S of weir. Decommission weir: remove entire weir or remove partial (centre) weir.
Valley Closure Only	Likely	15 mm	Cracks in D/S face on both sides of weir at and above rock interface. Leakage through localised crack zones.	7.5 mm	Review survey data. Review weir condition. Lower storage. Stop mining. Cut slot - isolate weir from further movement	Repair cracks. Strengthen weir with post-tension anchors and/or concrete placement D/S of weir. Decommission weir: remove entire weir or remove partial (centre) weir.

## 5.1 Stability Assessment

The current sliding stability of the weir was estimated using conventional static equilibrium calculation where the sliding factor of safety (FOS) across a sliding surface is equal to the ratio of available sliding resistance to the net de-stabilising sliding force. A 2D section at deepest section of the weir was considered assuming the crest was 13m above the riverbed. Although some parts of the weir base was keyed into the foundation, it was conservative to assume the key was absent and that there was no passive resistance provided by the rock foundation. The section considered is shown in Figure 5-7 where the sliding plane is across A-A. The forces acting on the weir section are:

$W$  = self-weight of the concrete section

$H_{us}$  = hydrostatic load due to the upstream water level

$S$  = silt load (the buoyant unit weight of  $7 \text{ kN/m}^3$  was used)

$H_{ds}$  = hydrostatic load due to the downstream tailwater

$U$  = uplift pressure

The net destabilising force,  $F = H_{us} + S - H_{ds}$

The sliding resistance along A-A,  $R = L_{in} c' + N' \tan \varphi'$

Where  $L_{in}$  = intact length along A-A = base width – crack length

$N' = W - U$

$c'$  = cohesion along the intact length

and  $\varphi'$  = friction angle

The sliding FOS =  $R/F$ .

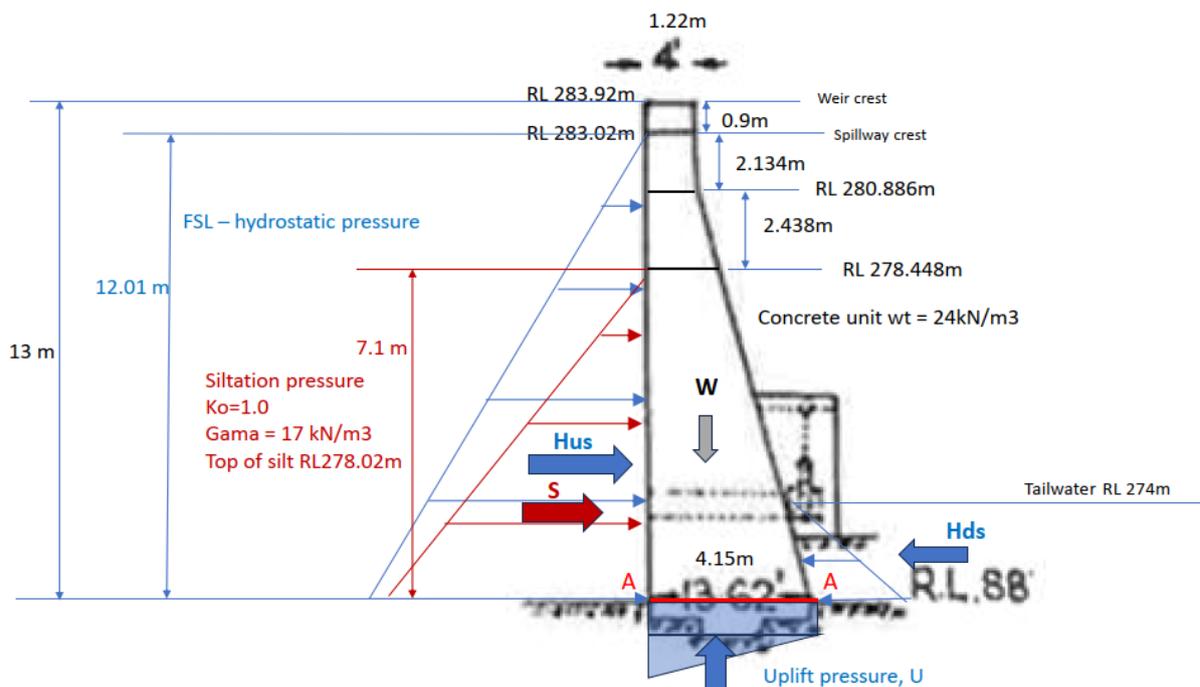


Figure 5-7: Section considered for sliding stability assessment

For flood cases, the upstream hydrostatic load and the uplift pressure will be increased in accordance with the upstream water level. For simplicity, the tailwater level remains the same and the weight of water flow across the crest was ignored. They are conservative assumptions.

For earthquake assessment, a pseudo-static method was used where the horizontal acceleration,  $a$ , was applied to the mass of the weir,  $M_{weir}$ , and to the added water mass,  $M_{water}$ , above the silt level at FSL. In this case, the net destabilising force is:  $F = H_{us} + S - H_{ds} + M_{weir} a + M_{water} a$ . The added water mass was calculated based on the Westergaard method. The acceleration corresponding to the

different earthquake annual exceedance probability is described in Section 6. The following accelerations were used in the stability assessment:

$$\text{OBE AEP} = 1 \text{ in } 500: a = 0.09 \text{ g}$$

$$\text{SEE AEP} = 1 \text{ in } 1000: a = 0.12 \text{ g}$$

Since there are no material test results for the shear strength parameters of the concrete and concrete/rock interface, the following values based on ANCOLD (2013) and EPRI (2018):

Dam/rock interface:

Lower bound:  $\varphi' = 65^\circ$  and  $c' = 345 \text{ kPa}$

Concrete lift joint:

Unbonded best fit:  $\varphi' = 49^\circ$  and  $c' = 500 \text{ kPa}$

The FOS as a function of crack length at the base for concrete/rock interface and concrete lift joint are shown in Figure 5-8 and Figure 5-9 respectively. It can be seen the weir is stable as long as the crack does not extend beyond 70% and 65% of the base width for concrete/rock interface and concrete lift joint respectively. When the storage is at FSL, the FOS is quite high (above 2.0) when the crack is below 20% of the base width. This is in a reasonable agreement with the FOS > 2.0 reported by DLWC (2002), although no details of the calculation and assumptions are provided in that report.

The above calculation method can be used to assess how much tension is required for the post-tensioning anchor mitigation.

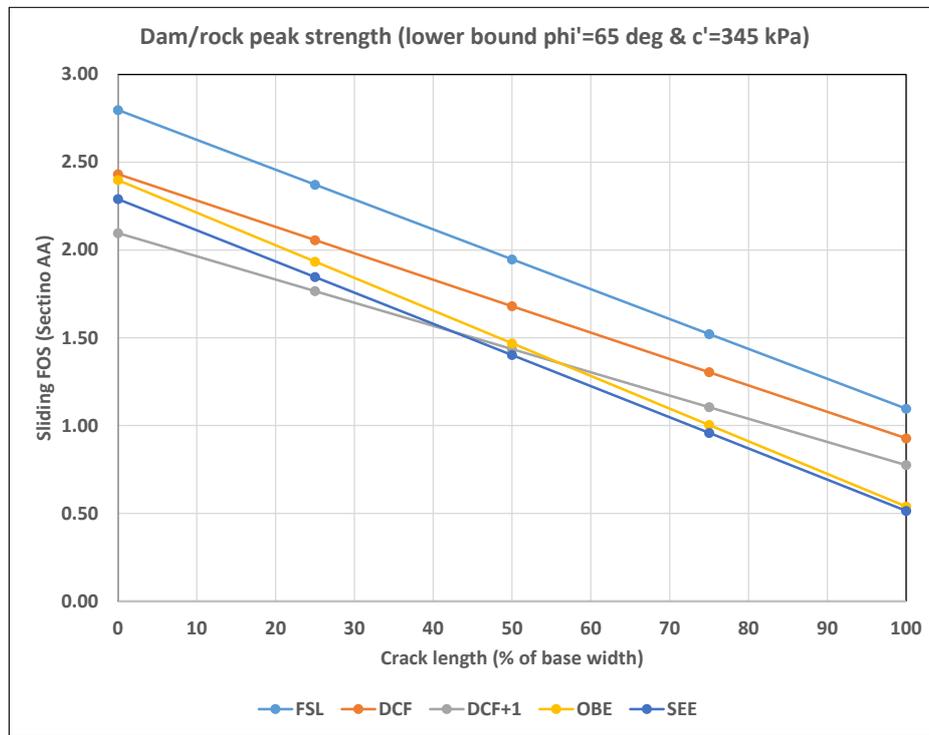


Figure 5-8: Sliding FOS concrete/rock interface as a function of crack length

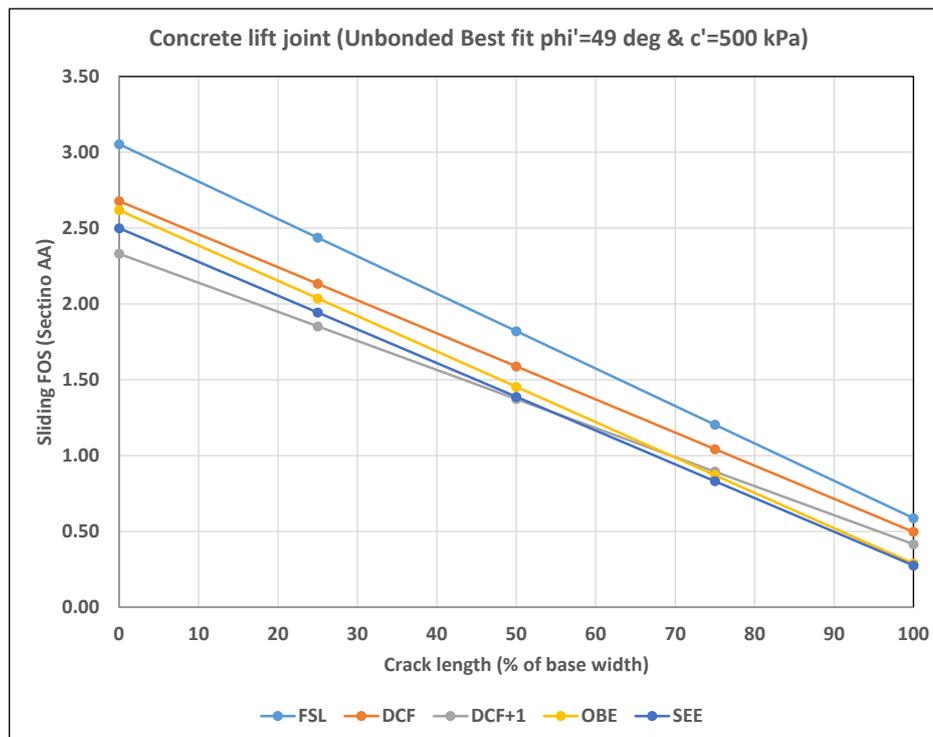


Figure 5-9: Sliding FOS concrete lift joint as a function of crack length

## 6 Discussions & Conclusions

A review of the risk analysis report performed in 2002 by DLWC revealed the consequence of dam failure was classified as **Very Low Category**. The then NSW Dam Safety Committee (DSC) has no requirements for acceptable flood capacity or earthquake stability for this consequence category. The ANCOLD guidelines recommended this weir should pass 1 in 100 to 1 in 1000 AEP floods. However, the risk analysis found that the weir could fail due to downstream rock scour undercutting the foundation when a spillage of 2m above the spillway crest occurs. A downstream impact slab was recommended but it was not certain if it was ever constructed.

According to the Dam Safety NSW website (<https://www.damsafety.nsw.gov.au/publications/list-of-declared-dams-in-nsw>) Picton Weir is not a declared dam list as of January 2024. It may be that the weir height does not exceed 15m, and the potential loss of life is less than one person – also see Worley Consulting (2024) dam break analysis PAR/PLL assessment. It is probable that the weir was a “prescribed” dam under the old Dams Safety Act (1978). We do not know when and under what circumstances it was removed from the declared dams list. Note that since 1 November 2019, the DSC was abolished and replaced by Dam Safety NSW.

In regard to earthquake requirement, the ANCOLD 2019 Earthquake Guidelines only provide guidance for dams in the LOW to EXTREME categories. There is no guidance for dams that are of VERY LOW category. For LOW category, the earthquake to consider would be 1:500 AEP for Operating Basis Earthquake (OBE) and 1:1000 AEP for Safety Evaluation Earthquake (SEE). Based on AS1170.4, the Australian standard on earthquake, a preliminary estimation of the peak ground acceleration is 0.09g for OBE and 0.12g for SEE. Clarification on earthquake requirement for a non-declared dam should be sought from Dam Safety NSW.

Based on recorded earthquakes close to the weir from the Geoscience Australia database, events of magnitude 3.0 and above are summarised in Table 6-1. Personnel from NSW Department of Commerce (now NSW Public Works) and State Water inspected the weir a few days after the 4.2 magnitude earthquake in Bowral 11 Dec 2003. It was reported that there was no apparent effect on the weir. Stronger earthquakes occurred before as shown in the table.

Table 6-1: Recorded earthquakes with magnitude 3 and above near Picton Weir

Description of earthquake	Date(YYYY-MM-DD)/Time	Magnitude	Distance from Picton Weir (km)	Comments
Bowral NSW	1961-05-21T21:40:03	5.80	32	
Picton NSW	1973-03-09T19:09:14	5.50	24	
Appin NSW. Felt	1999-03-17T01:58:10	4.80	20	
Bowral NSW. Nearest station RIV. Felt in Canberra and Sydney.	2003-12-11T10:19:20	4.20	26	NSW Dept of Commerce & State Water inspect the dam 17 Dec 2003 after the earthquake. There was no apparent effect on the dam from this earthquake.
Lake Avon NSW. Nearest station RIV. Felt by many people in the Sydney and Wollongong areas.	2002-02-14T13:27:20	3.90	34	
Near Appin, NSW. (Reported felt).	2017-01-03T14:13:08	3.84	17	
Appin, NSW	2023-09-08T14:25:03	3.54	21	
Appin, NSW	2013-10-07T20:15:41	3.51	22	
West of Picton NSW.	2009-08-28T22:20:06	3.40	20	
Appin aftershock	1981-11-19T12:18:52	3.30	31	
Damage was reported from 'The Oaks' and Picton and the shaking was widely felt.	1996-12-10T12:54:26	3.30	12	
Near Picton N.S.W. 9kms NW Thirlmere	1996-12-10T12:58:35	3.20	15	
NW of Mittagong NSW. Nearest station RIV.	2007-08-18T03:57:39	3.20	20	
Appin, NSW.	2013-10-24T12:49:39	3.10	24	
Appin, NSW.	2023-06-18T04:05:47	3.08	20	
W of Camden NSW. Nearest station RIV.	2006-06-18T15:49:34	3.00	25	
CATARACT DAM	1990-03-30T11:33:00	3.00	26	

The present analysis results showed that the weir is most vulnerable to valley opening, followed by upsidence, valley closure with upsidence, lateral shear and finally valley closure. Their corresponding critical movements are:

- Valley opening only: ~ 8 mm
- Upsidence only: ~ 8 mm
- Valley closure and upsidence: ~10 mm
- Lateral shear only: ~ 15 mm
- Valley closure only: ~ 15 mm

We provisionally suggested that for the mine subsidence management plan the trigger level should be half of the above movements. The valley opening only and upsidence only could lead to collapse and partial collapse of the weir. It is noted that these two types of movement on their own are unlikely to occur based on past survey data at similar sites (MSEC to confirm). The most probable movements are valley closure accompanied with upsidence, and valley closure with little or no upsidence. We do not know if lateral shear in a valley has been observed before. These last three types of movement could lead to cracking of the weir resulted in a significant leakage. Localised or complete weir collapse is unlikely to occur. Note that dewatering would not prevent the weir from cracking except there will be less or no seepage/leakage of water through the cracks when dewatered. A reduced water level upstream would also mean a lesser hydrostatic pressure acting on the weir and therefore the weir stability would increase.

The current sliding FOS of the weir was estimated using the conventional stability calculation to be above 2.00. The FEA model could provide an insight into the extent of concrete damage as a result of valley movement. However, the model has a limitation that it cannot provide a reliable the crack location or to quantify the crack length. Any pre-existing cracks in the weir structure can affect the modelling results. These defects were not included in the model as they are difficult to quantify without a detailed survey of the upstream and downstream faces of the weir. Therefore, it was difficult to estimate for certain as to when the weir or a portion of the weir become unstable as a function of valley movement. Furthermore, the applied valley movement profiles in the model were idealised, and they cannot be verified by limited movement survey data.

## 6.1 Monitoring & Survey

To manage the risk of the weir structure from mining damage, a detailed survey and monitoring plan should be implemented in order to measure 3D displacement across the valley, across the weir, along the weir crest, downstream riverbed for sign of upsidence (note that it may not be possible due to the presence of water), and the downstream river bank close to the weir (to capture upsidence and to provide additional information on the other valley movement modes). Periodic visual inspection of the weir should be done so to monitor any propagation of existing cracks/defects and to capture any new cracks happening during mining.

The current condition of the weir based on the 2023 photos showing existing cracks is provided in Appendix C.

## 6.2 Mitigations

When considering mitigation actions before, during and post-mining, the following should take into consideration:

- The weir has been classified as a Very Low consequence category if the weir suffers a dam break event. It was assessed by SKM in 2000 that under the worst case scenario the estimated loss of life is 0.1 person during a Sunny Day failure and the same for the probable maximum precipitation design flood (PMPDF) with dam failure. A recent dam break study of the weir was carried out by Worley Consulting (2024). For the Sunny Day failure the potential loss of life (PLL) was estimated to be 0.09. The PLL at the two houses along the fire trail downstream of the weir was estimated to be 0.63 during the probable maximum flood (PMF) without weir failure, and 0.64 during the PMF with weir failure, giving an incremental PLL of 0.01.
- The current and future function of the weir. Currently it is not used for water supply or recreation. Water leakage of the damaged weir could be acceptable as long as it does not lead to collapse of the structure.
- The risk analysis done in 2002 showed the spillway is inadequate in passing the required floods unless an impact slab or apron has been installed downstream of the weir to prevent scour of the rock. If the impact slab has not been installed, then the weir could fail (collapse) during extreme flood events or when flood level is 2m or more above the spillway crest. More cracks in the weir as a result of mining could further reduce the spillway capacity from passing floods. In other words, the damage caused by mining is unlikely to cause an immediate weir failure, but the damaged weir may fail when the next significant flood occurs.
- The environmental impact and heritage significance of the weir. Can the weir be demolished completely or partially? Assuming the current outlet valve may not be adequate to drain water during floods, putting an opening through the weir or demolish it could address the above issue of spillway discharge capacity. The release or deposition of the silt in the storage will need to be addressed.
- Last but not least, the cost and timing to implement any mitigations/remediations. Impact on mining operation.

Below are a few suggested actions to be considered:

### **Before mining:**

- Strengthen concrete from cracking – not practical as we do not know the actual valley movement modes.
- Lower storage/dewatering – if easy to do using the existing outlet valve.
- Partial or complete demolition of the weir (if permitted to proceed) – Cost and timing.
- Cutting a slot in the rock abutment on one side of the weir to isolate the weir from valley movement (Figure 6-1) – need to determine the extent of the slot (how deep and how long). Access, cost and timing need to be considered. Note that this slotting option to relieve stress has been performed at Marhnyes Hole at Georges River.

### **During mining:**

- Repair cracks – not practical
- Strengthen concrete from cracking – not practical
- Lower storage/dewatering – if easy to do using the existing outlet valve
- Cutting a slot in the rock abutment on one side of the weir to isolate the weir from valley movement – need to determine the extent of the slot (how deep and how long). How long does it take to implement the work?
- Pause mining activities if movement of valley is deemed excessive

**Post-mining:**

- Re-assess weir if it can still safely pass anticipated floods – this will determine the repair/remedial actions
- Repair cracks or leakage using spraycrete
- Strengthen weir with post-tensioned anchors (Figure 6-2)
- Stabilise weir by placing mass concrete or concrete buttress downstream of weir (Figure 6-3)
- Decommission – partial or complete demolition of weir

Note there are many Australian dams that have post-tensioned anchors retrofitted to mitigate against floods and/or earthquakes. For example, Catagunya Dam, Lake Manchester Dam, Moogerah Dam Fairbairn Dam and Tinaroo Falls Dam. Spraycrete or shotcrete have been successfully applied to the upstream face of the concrete dam to prevent leakage, for example at Lake Parramatta Dam and Lithgow No.2 Dam. Placement of concrete block to provide added stability have been carried out at Hume Dam and Moogerah Dam. Concrete buttress can also provide the same function and this is to be implemented at Mt Bold Dam.

Performing mitigations during and post-mining is considered to be not practical. Mitigations be best done before mining, and any residual issues such as cracks and leakage can be repaired post-mining. The following pre-mining mitigation options should be investigated further.

- Partial or complete demolition of the weir as it currently does not provide any service to the community. Its removal will alleviate the risk of a weir failure.
- Install post-tensioned anchors to stabilise the weir in case it suffers a significant damage that may make the weir or part of the weir become unstable. Damage such as cracks and leakages can be repaired post-mining.
- Install a downstream concrete apron to provide more passive resistance to stabilise the weir. It can also minimise the risk of erosion of the downstream rock abutment/foundation that may undermine the weir during large flood events.

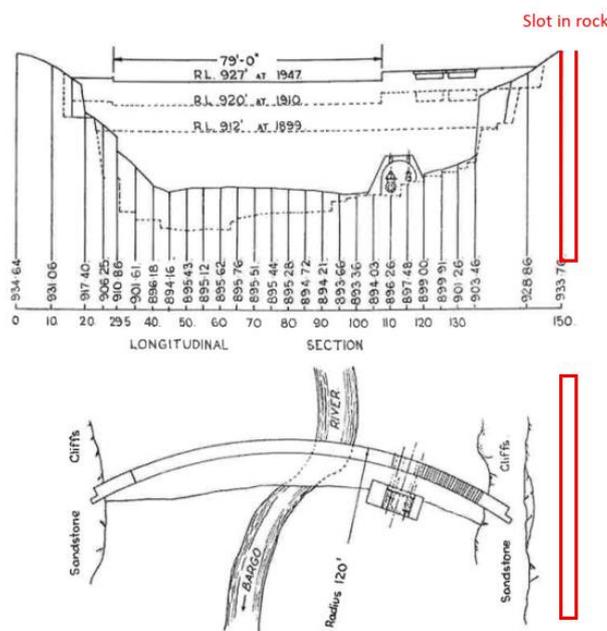


Figure 6-1: Cut slot adjacent to the weir to isolate it from valley movement (concept only)

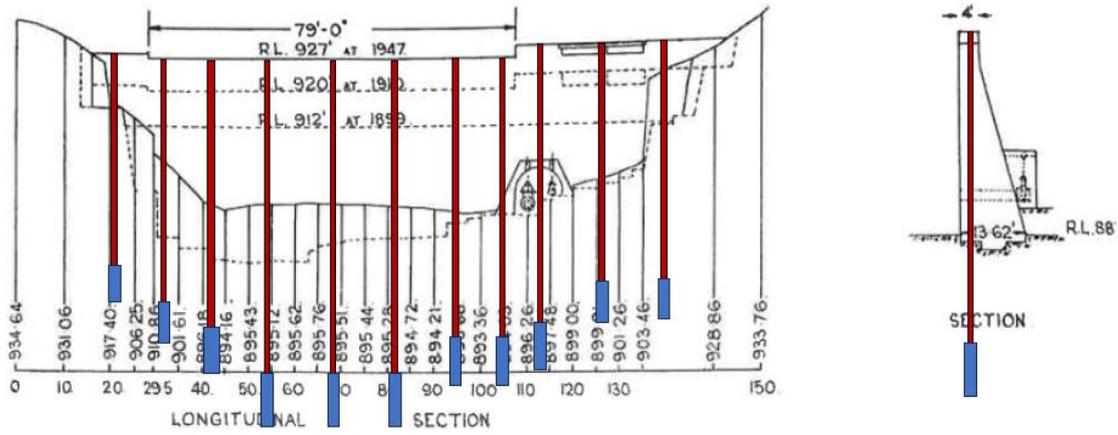


Figure 6-2: Post-tensioned anchors concept

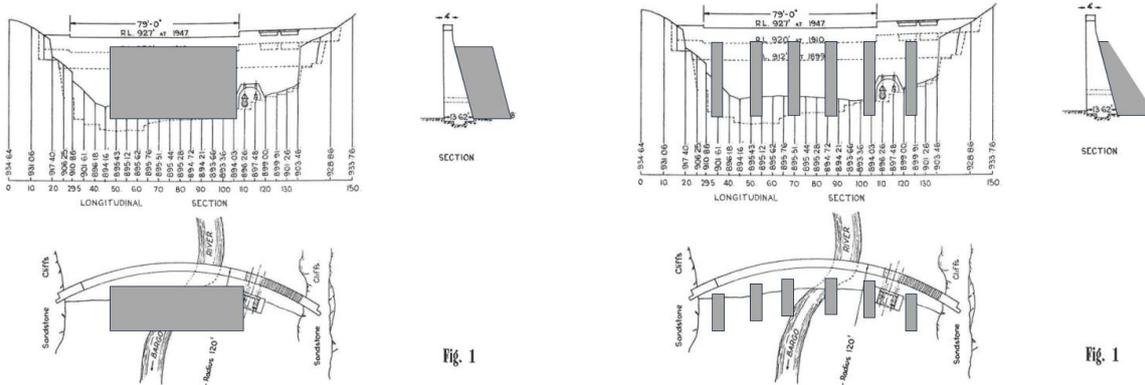


Figure 6-3: Mass concrete blocks (left), concrete buttress (right) stabilization concepts

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### 7.1 Supplied information

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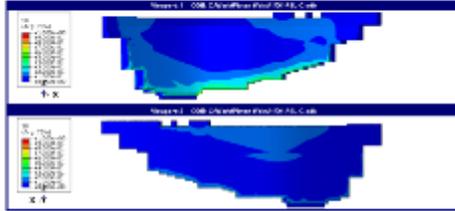
## Appendix A Picton Weir Data

Table A-1 Picton Weir Data (Source: Bargo Dam 2004 Surveillance Report and Bargo Dam – Risk Analysis 2002)

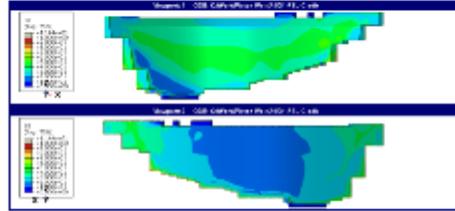
Picton Weir Data Sheet	
<b>Name of Dam</b>	Picton Weir (previously known as Bargo Weir/Dam)
<b>Owner</b>	Wollondilly Shire Council
<b>Year of Completion</b>	1898 and later raised in 1910 and again in 1947
<b>River</b>	Bargo
<b>Purpose</b>	Previously provided water supply to Picton. Now not used for water supply or recreation
<b>Type</b>	Concrete gravity arch-dam
<b>Height above Lowest Foundation</b>	13 m
<b>Length of Dam</b>	42 m long at the crest
<b>Arch Radius</b>	120 ft (36.576m)
<b>Estimated Storage Capacity</b>	150 ML
<b>Catchment Area</b>	66 km <sup>2</sup>
<b>Spillway Crest Level</b>	RL 283.89 m
<b>Weir Crest Level</b>	RL 283.92 m
<b>Maximum Thickness of Concrete at Base of Dam Arch</b>	4.14 m
<b>Crest Width</b>	1.22 m

# Appendix B Analysis Results

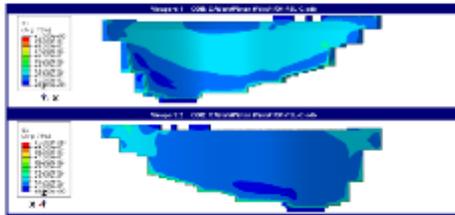
## Results – FSL Valley Closure



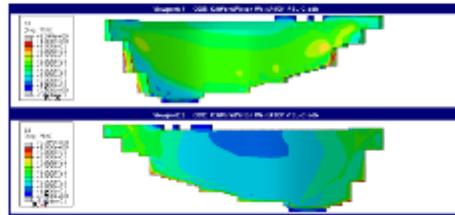
No closure



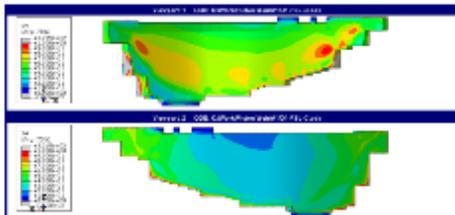
8 mm closure



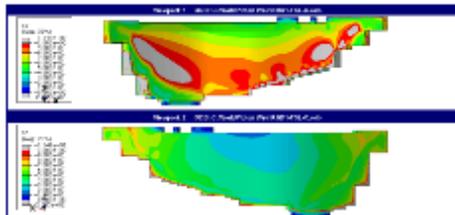
4 mm closure



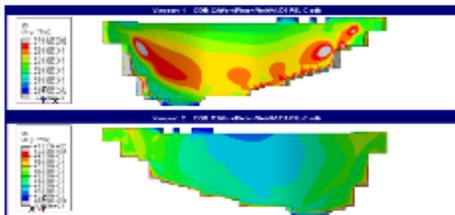
12 mm closure



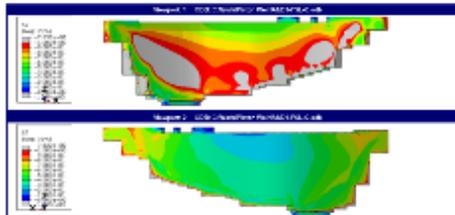
16 mm closure



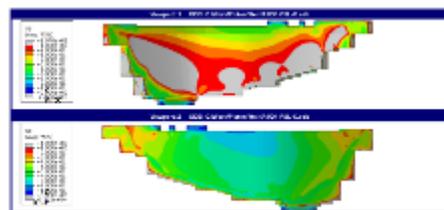
24 mm closure



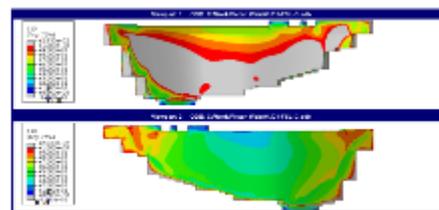
20 mm closure



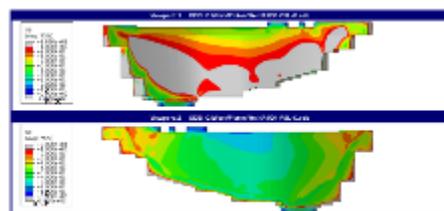
28 mm closure



32 mm closure

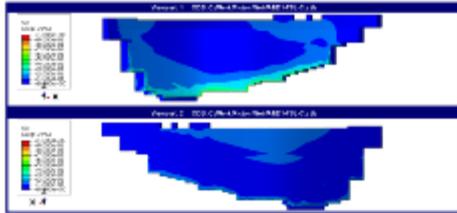


40 mm closure

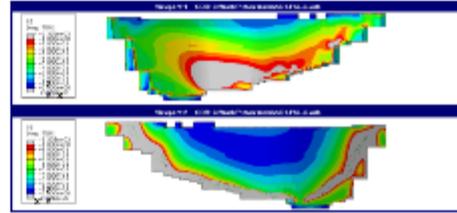


36 mm closure

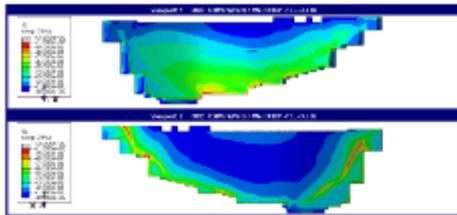
Results – FSL Valley Opening



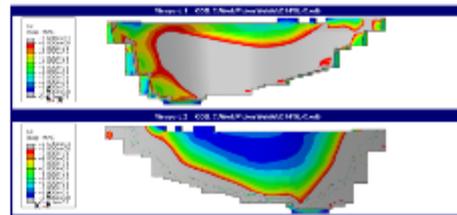
No opening



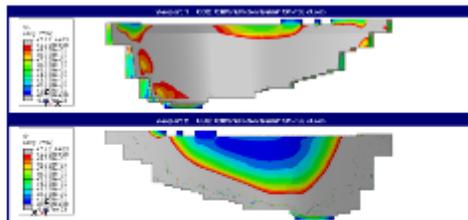
8 mm opening



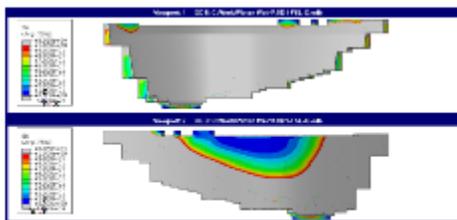
4 mm opening



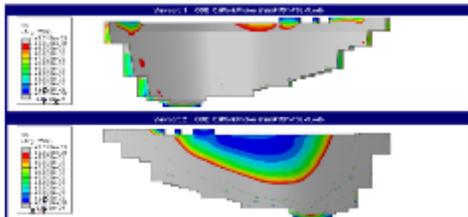
12 mm opening



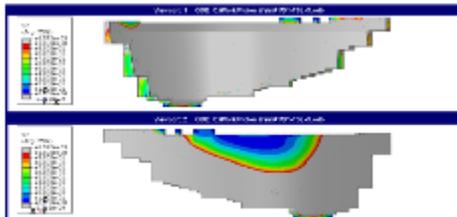
16 mm opening



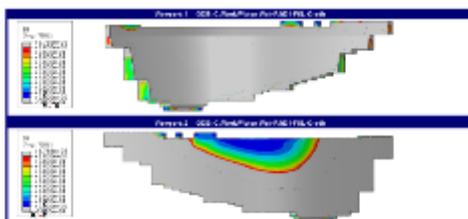
24 mm opening



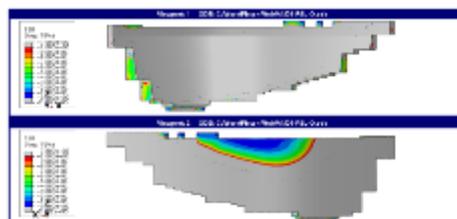
20 mm opening



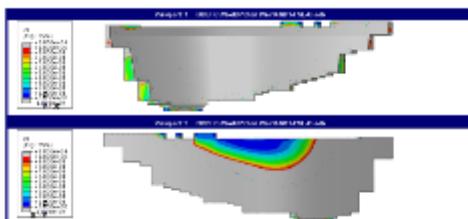
28 mm opening



32 mm opening

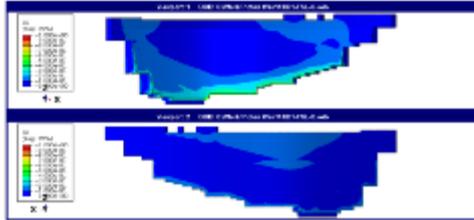


40 mm opening

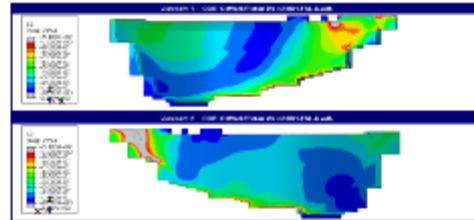


36 mm opening

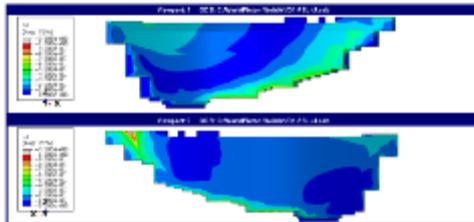
## Results – FSL Upsidence



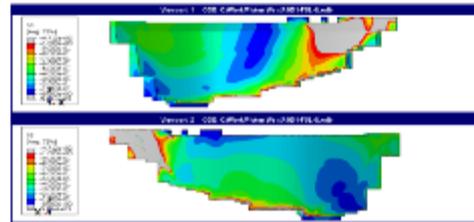
No upsidence



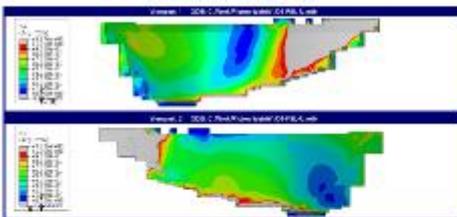
8 mm upsidence



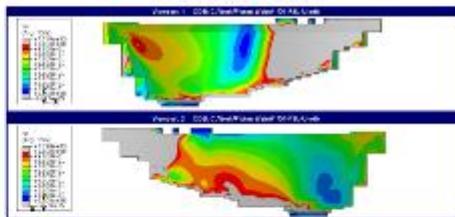
4 mm upsidence



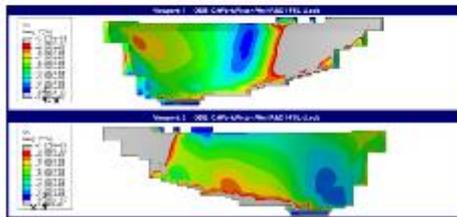
12 mm upsidence



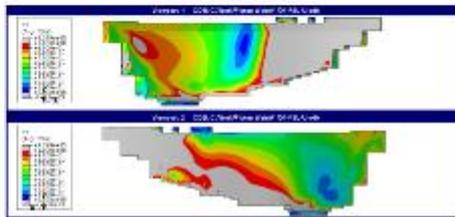
16 mm upsidence



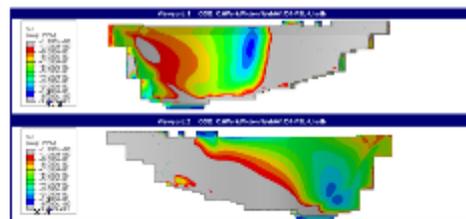
24 mm upsidence



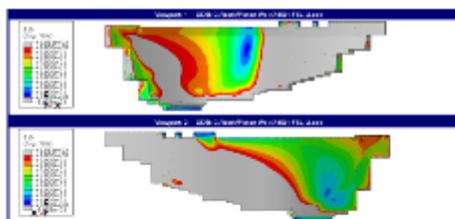
20 mm upsidence



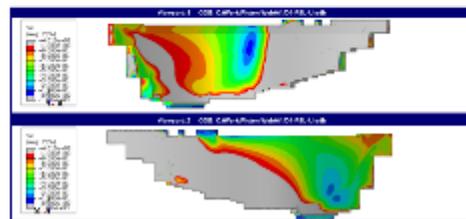
28 mm upsidence



32 mm upsidence

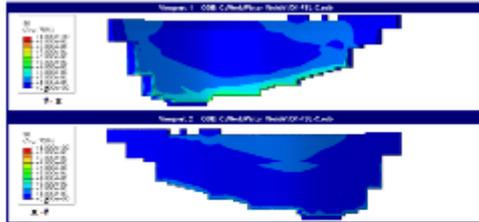


40 mm upsidence

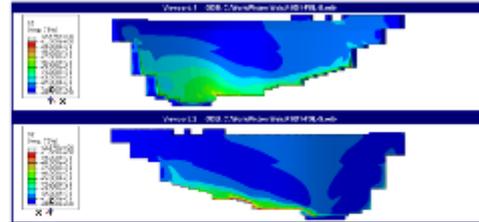


36 mm upsidence

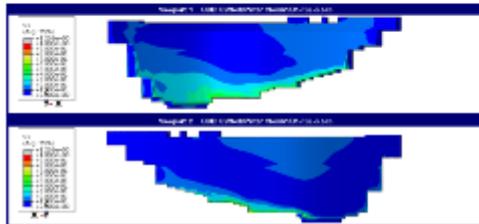
## Results – FSL Lateral Shear



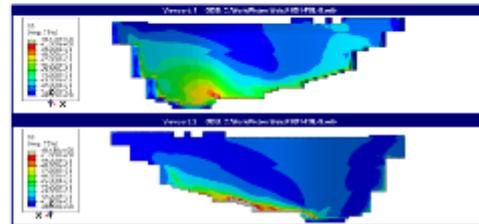
No lateral shear



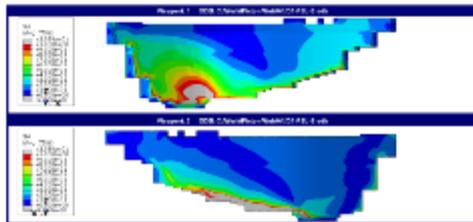
8 mm lateral shear



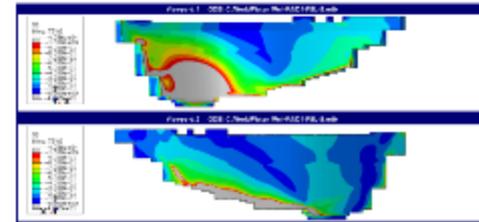
4 mm lateral shear



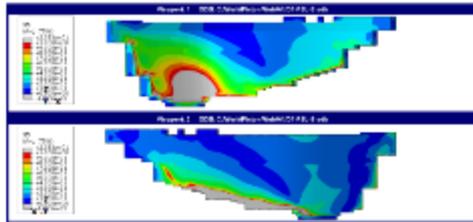
12 mm lateral shear



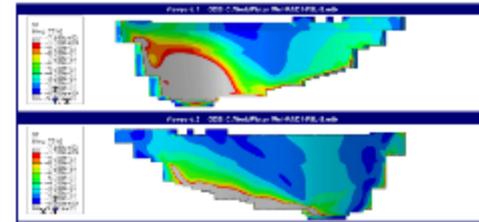
16 mm lateral shear



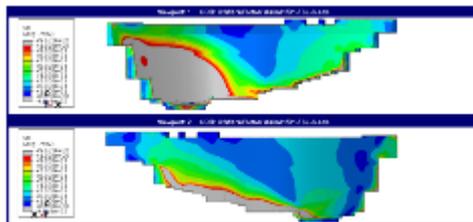
24 mm lateral shear



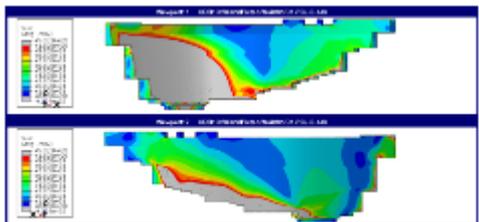
20 mm lateral shear



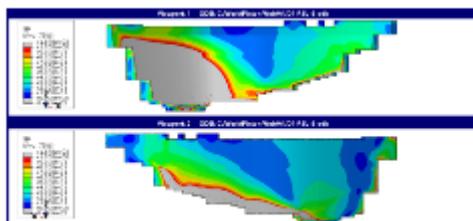
28 mm lateral shear



32 mm lateral shear

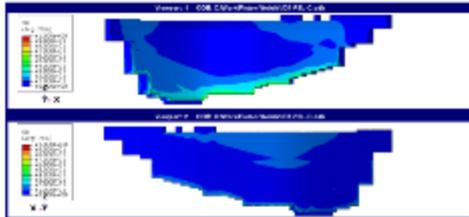


40 mm lateral shear

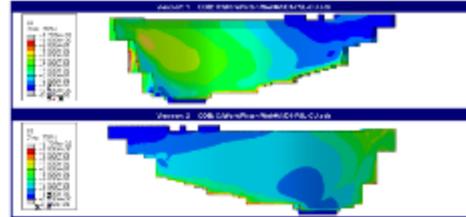


36 mm lateral shear

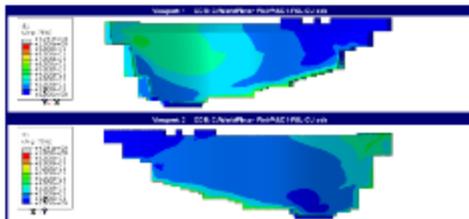
## Results – FSL Valley Closure & Upsidence



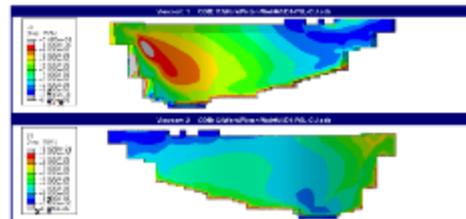
No closure & upsidence



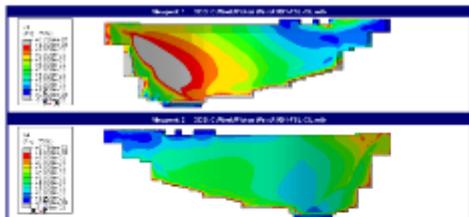
8 mm cl & up



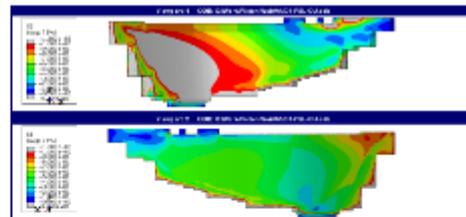
4 mm cl & up



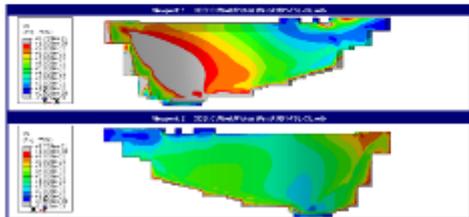
12 mm cl & up



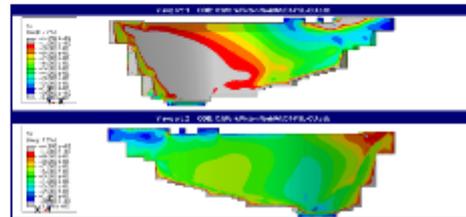
16 mm cl & up



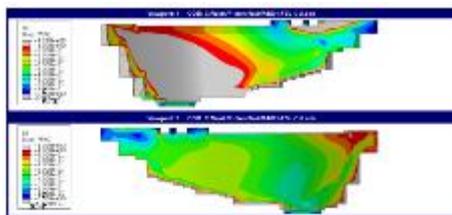
24 mm cl & up



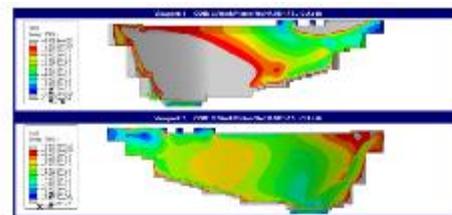
20 mm cl & up



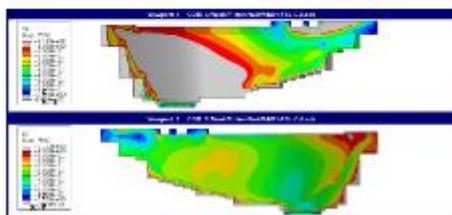
28 mm cl & up



32 m cl & up

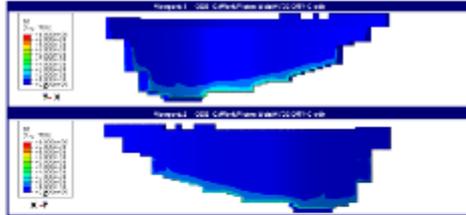


40 mm cl & up

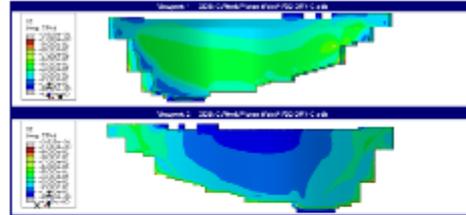


36 mm cl & up

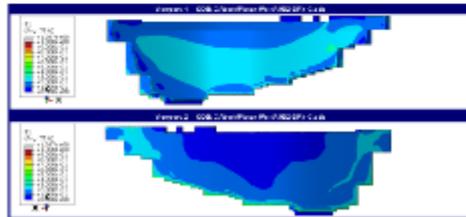
Results – Dry Valley Closure



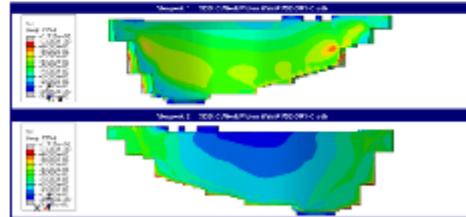
No closure



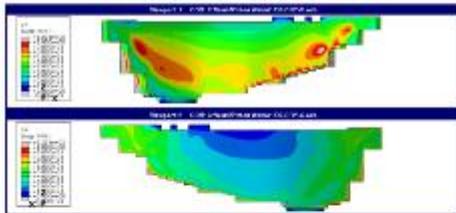
8 mm closure



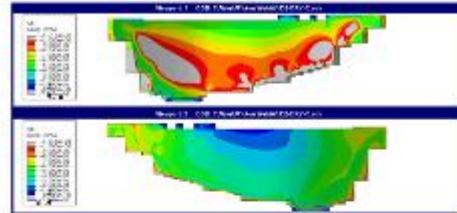
4 mm closure



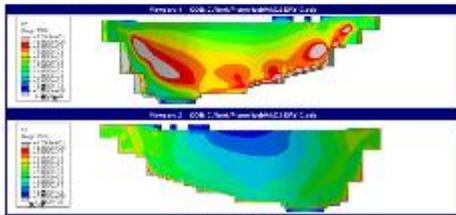
12 mm closure



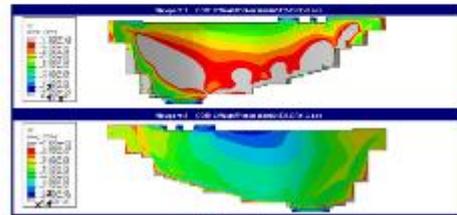
16 mm closure



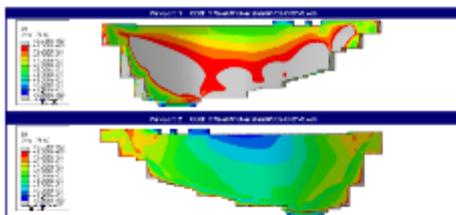
24 mm closure



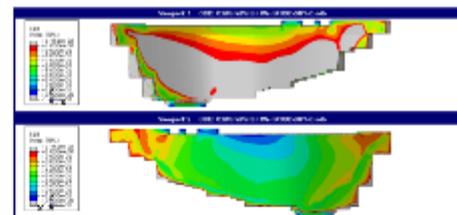
20 mm closure



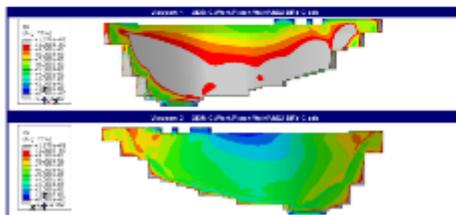
28 mm closure



32 mm closure

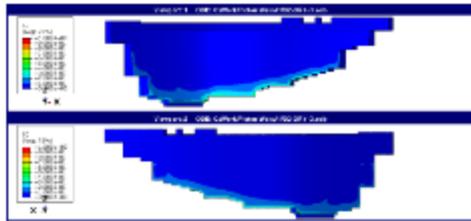


40 mm closure

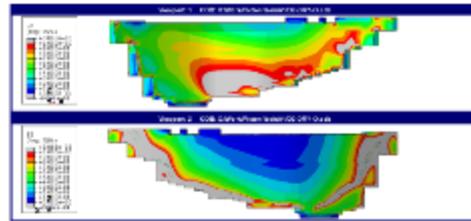


36 mm closure

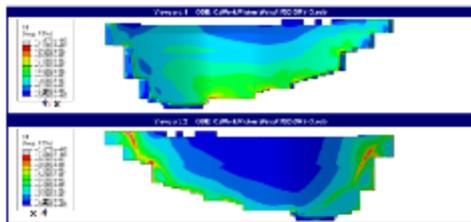
Results – Dry Valley Opening



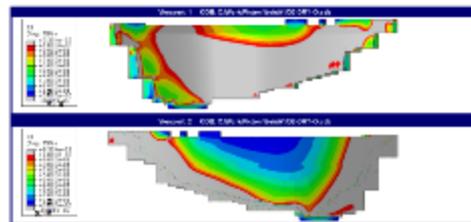
No opening



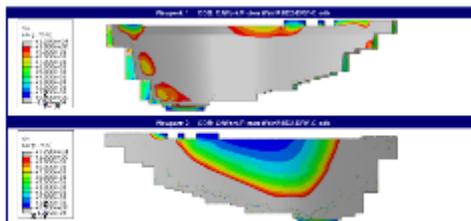
8 mm opening



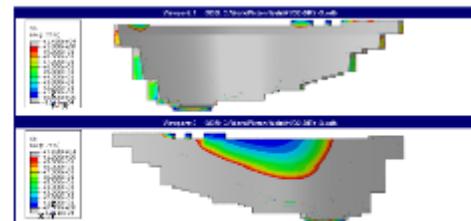
4 mm opening



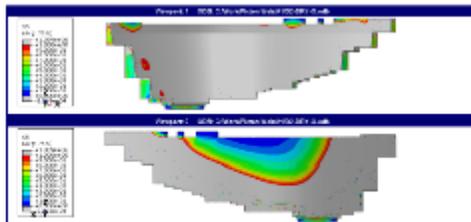
12 mm opening



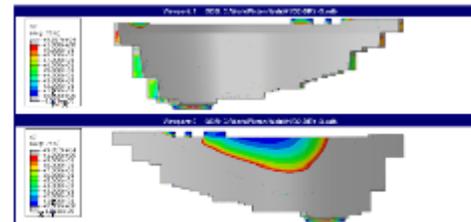
16 mm opening



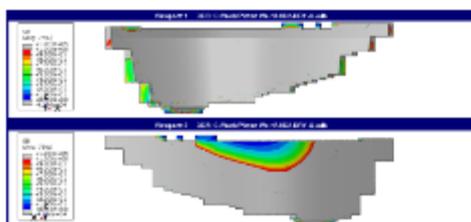
24 mm opening



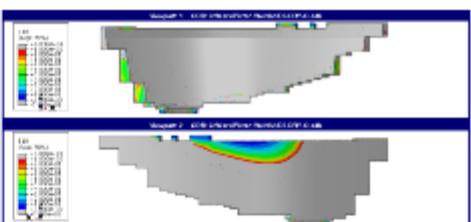
20 mm opening



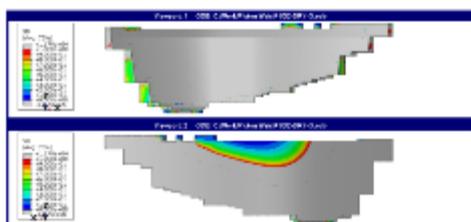
28 mm opening



32 mm opening

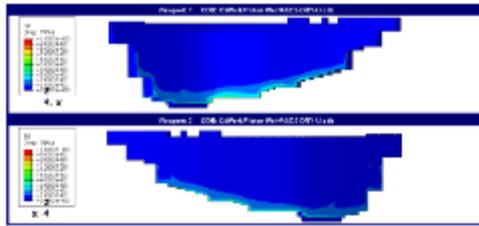


40 mm opening

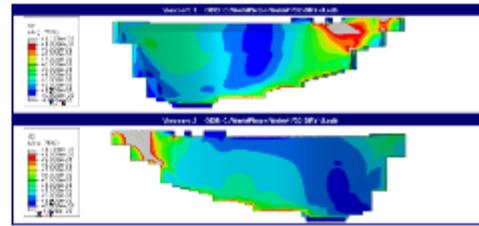


36 mm opening

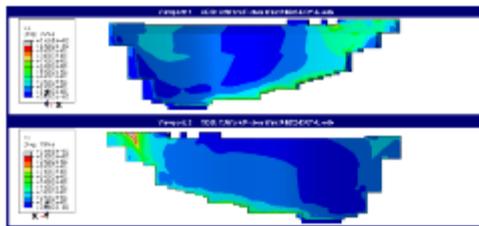
## Results – Dry Upsidence



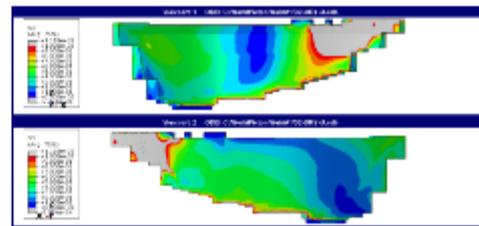
No upsidence



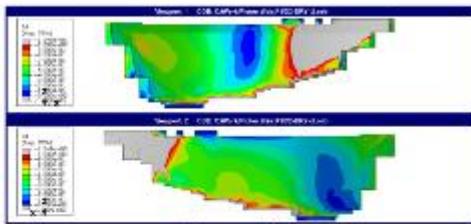
8 mm upsidence



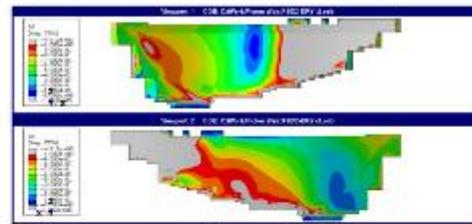
4 mm upsidence



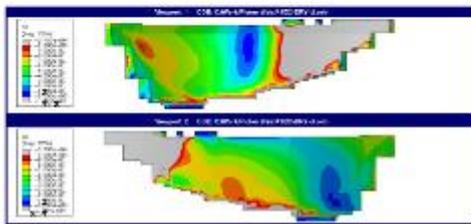
12 mm upsidence



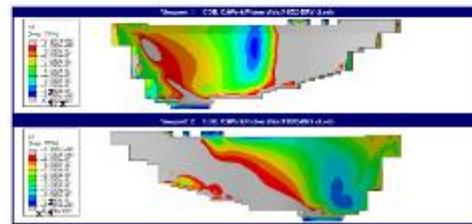
16 mm upsidence



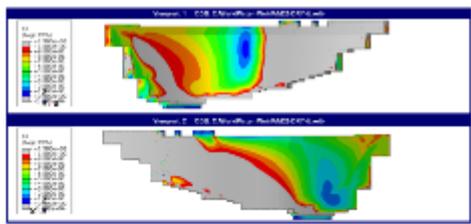
24 mm upsidence



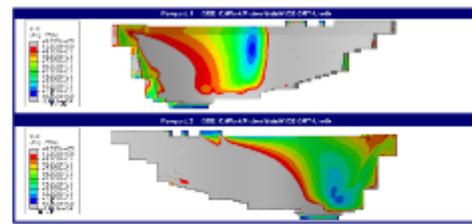
20 mm upsidence



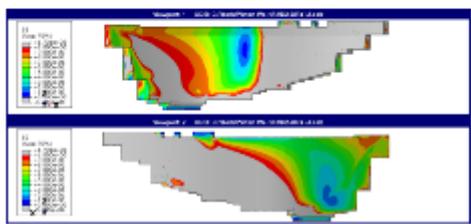
28 mm upsidence



32 mm upsidence

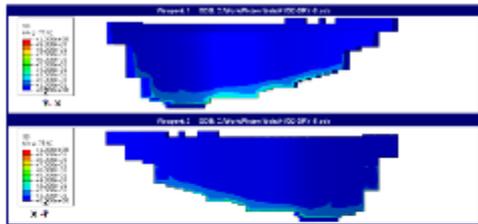


40 mm upsidence

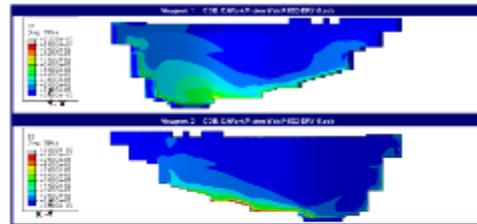


36 mm upsidence

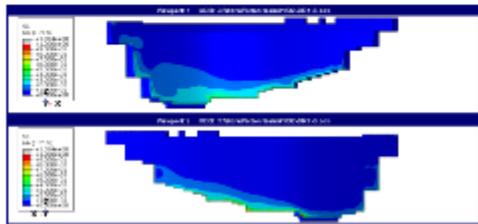
Results – Dry Lateral Shear



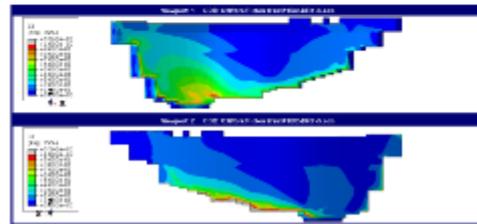
No lateral shear



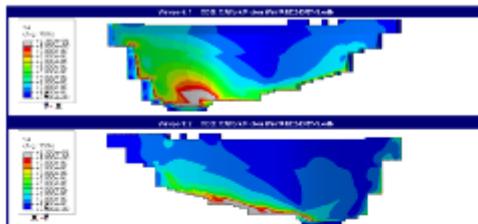
8 mm lateral shear



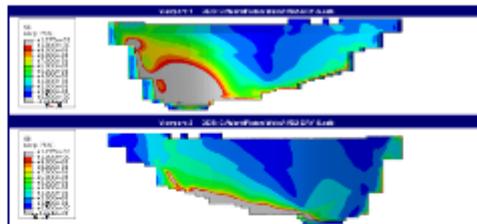
4 mm lateral shear



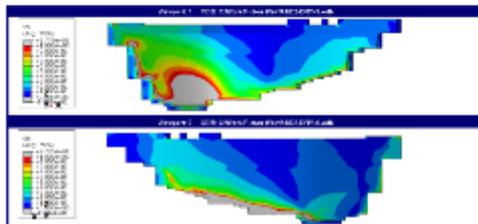
12 mm lateral shear



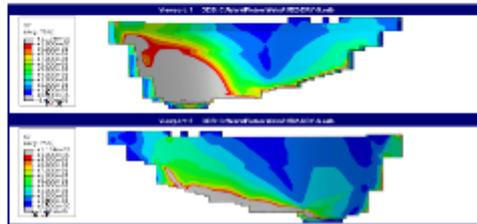
16 mm lateral shear



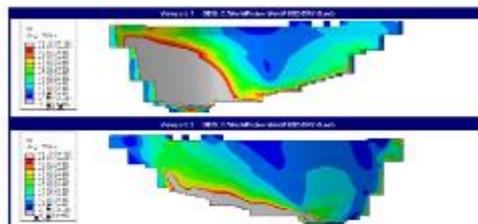
24 mm lateral shear



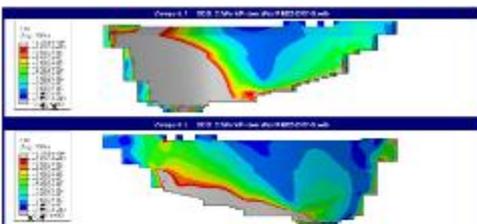
20 mm lateral shear



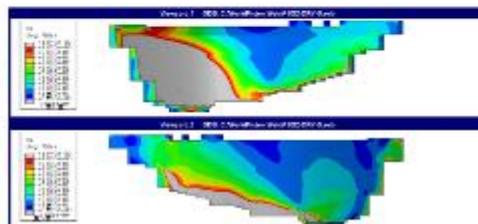
28 mm lateral shear



32 mm lateral shear

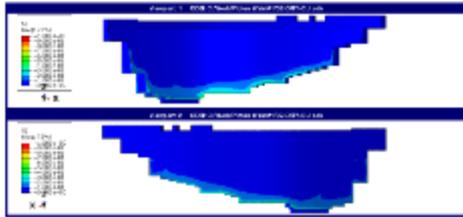


40 mm lateral shear

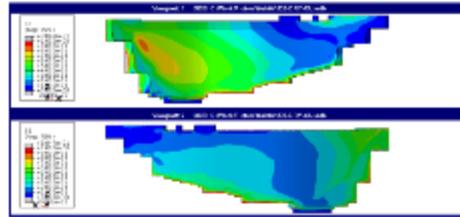


36 mm lateral shear

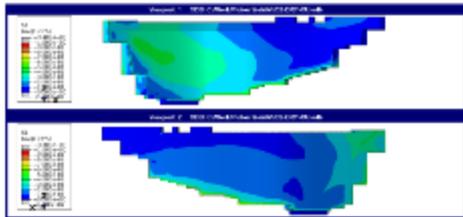
## Results – Dry Valley Closure & Upsidence



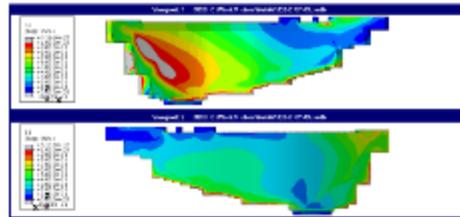
No closure & upsidence



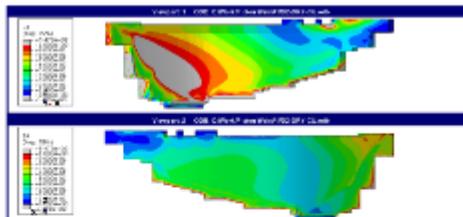
8 mm cl & up



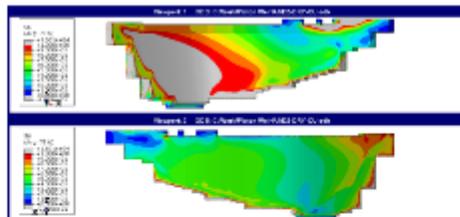
4 mm cl & up



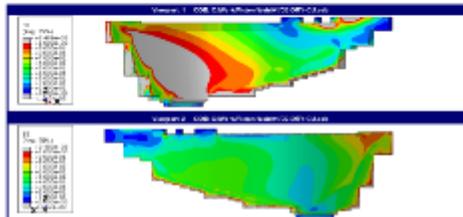
12 mm cl & up



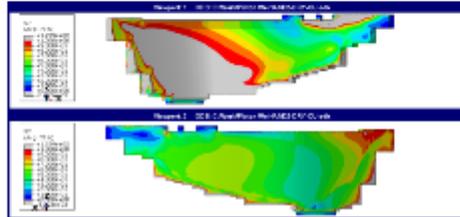
16 mm cl & up



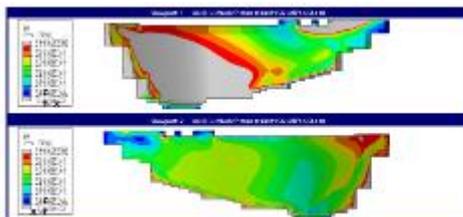
24 mm cl & up



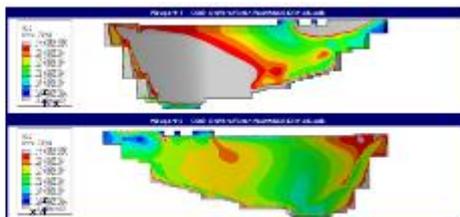
32 mm cl & up



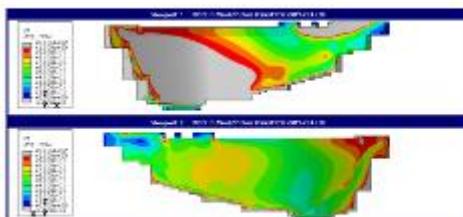
24 mm cl & up



32 mm cl & up

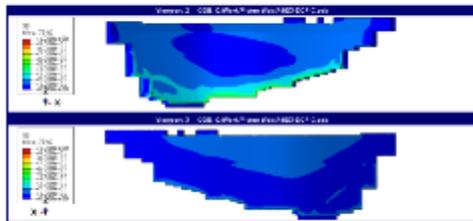


40 mm cl & up

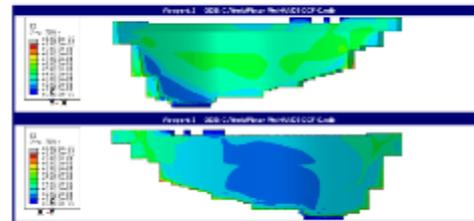


36 mm cl & up

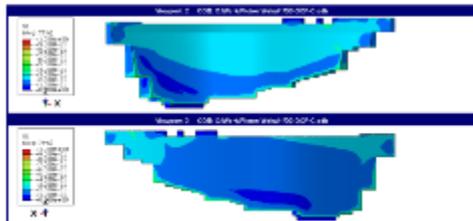
Results – DCF Valley Closure



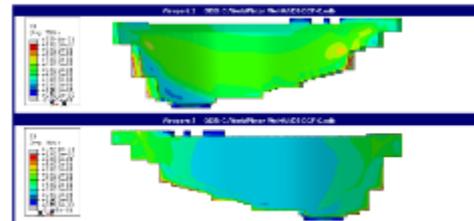
No closure



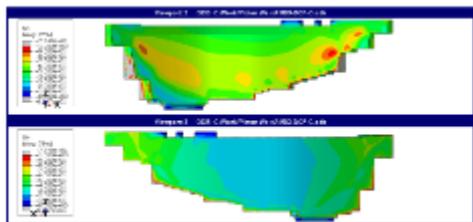
8 mm closure



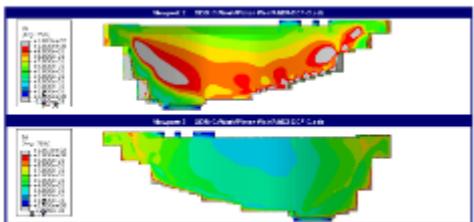
4 mm closure



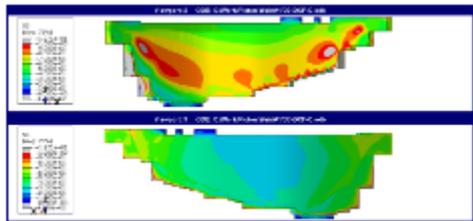
12 mm closure



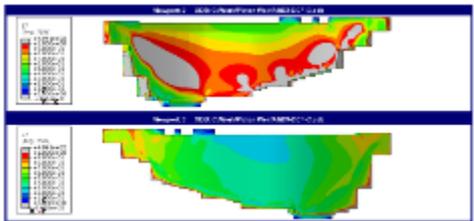
16 mm closure



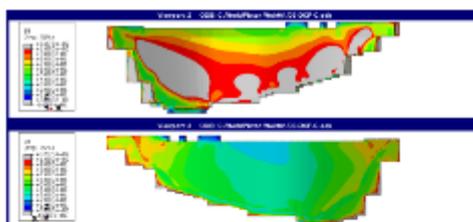
24 mm closure



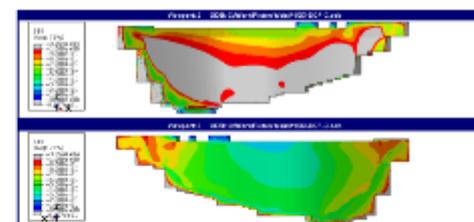
20 mm closure



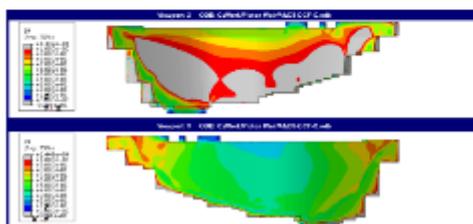
28 mm closure



32 mm closure

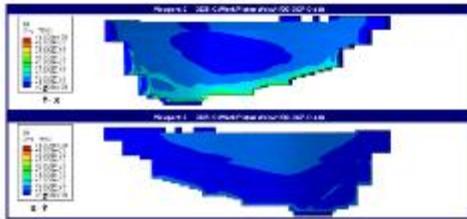


40 mm closure

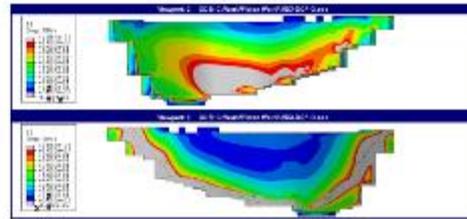


36 mm closure

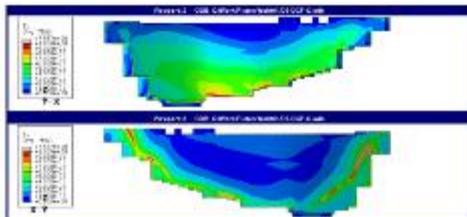
## Results – DCF Valley Opening



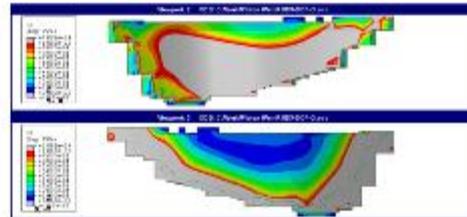
No opening



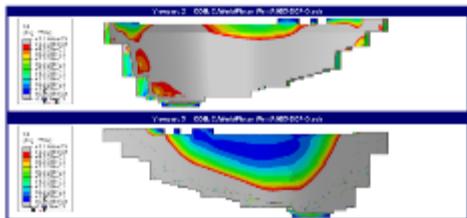
8 mm opening



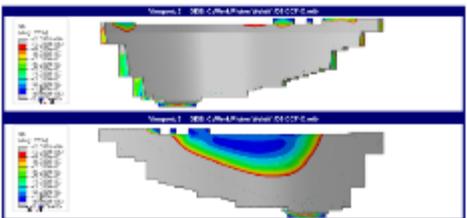
4 mm opening



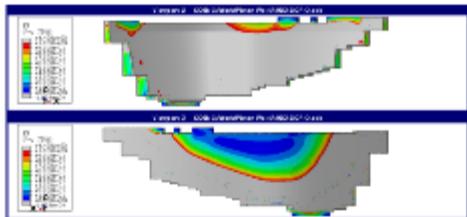
12 mm opening



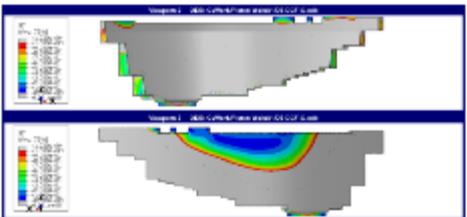
16 mm opening



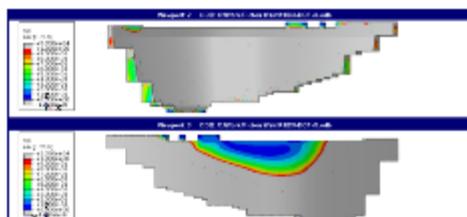
24 mm opening



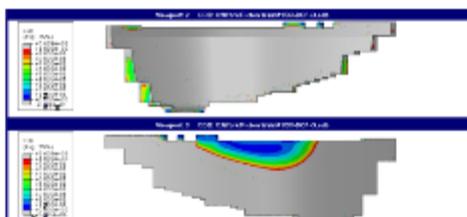
20 mm opening



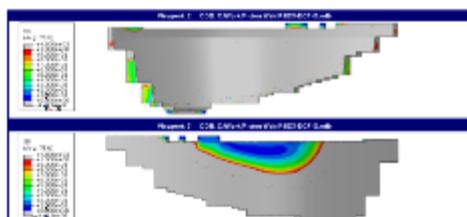
28 mm opening



32 mm opening

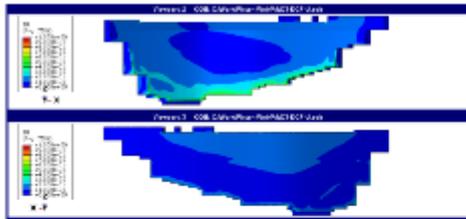


40 mm opening

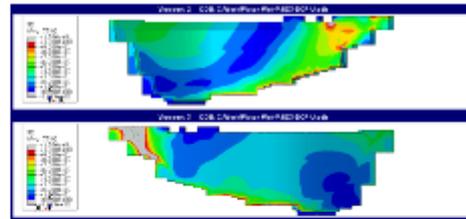


36 mm opening

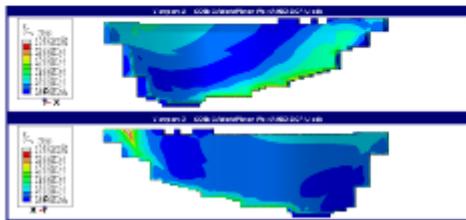
## Results – DCF Upsidence



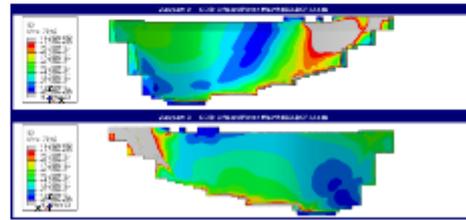
No upsidence



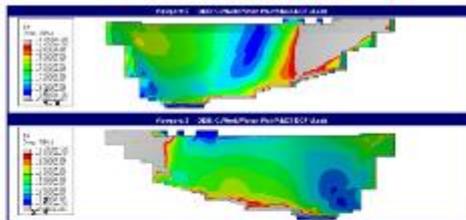
8 mm upsidence



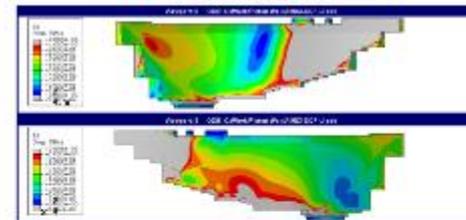
4 mm upsidence



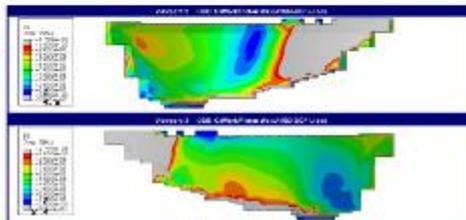
12 mm upsidence



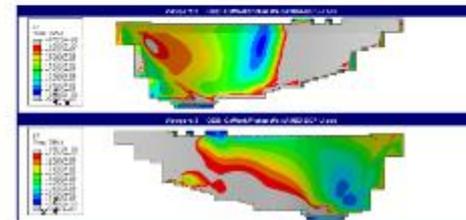
16 mm upsidence



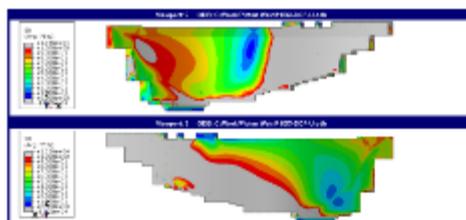
24 mm upsidence



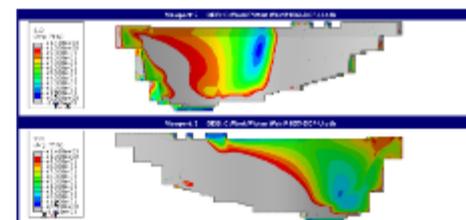
20 mm upsidence



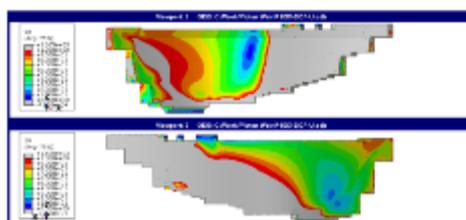
28 mm upsidence



32 mm upsidence

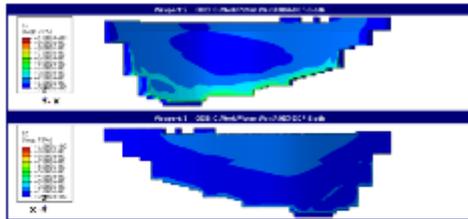


40 mm upsidence

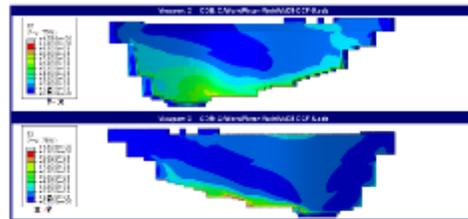


36 mm upsidence

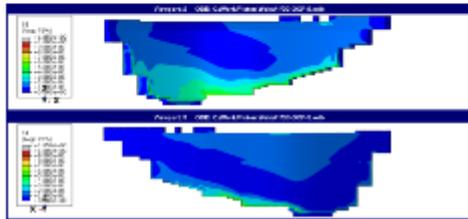
Results – DCF Lateral Shear



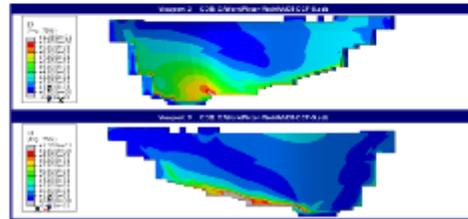
No lateral shear



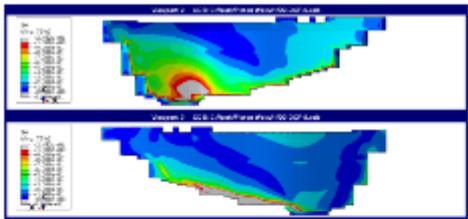
8 mm lateral shear



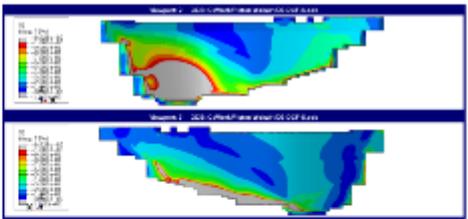
4 mm lateral shear



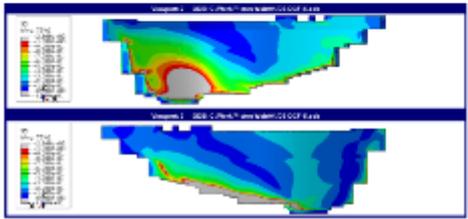
12 mm lateral shear



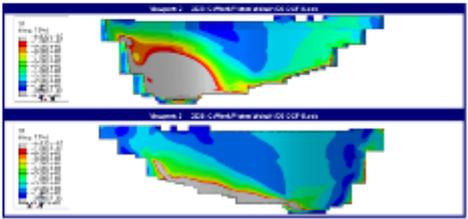
16 mm lateral shear



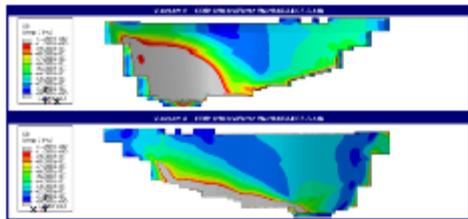
24 mm lateral shear



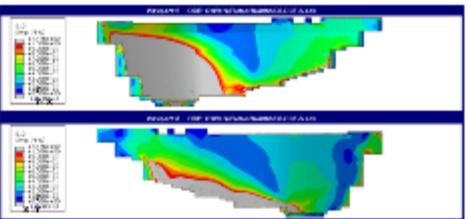
20 mm lateral shear



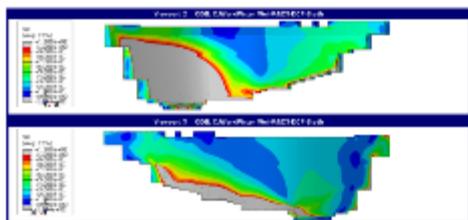
28 mm lateral shear



32 mm lateral shear

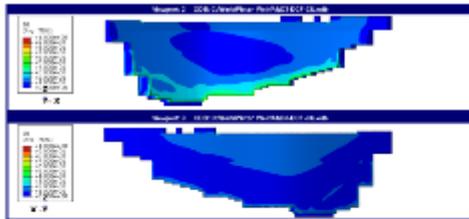


40 mm lateral shear

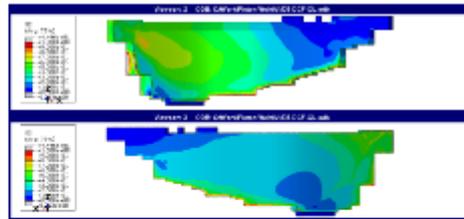


36 mm lateral shear

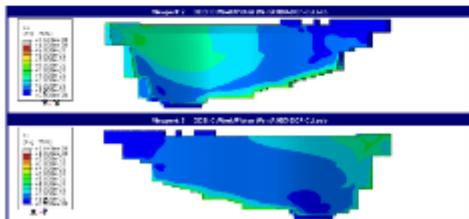
## Results – DCF Valley Closure & Upsidence



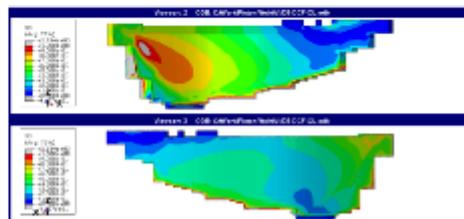
No closure & upsidence



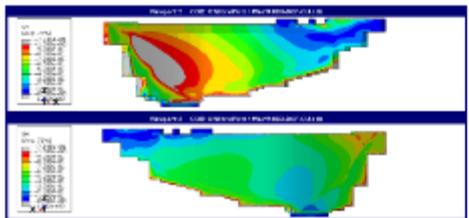
8 mm cl & up



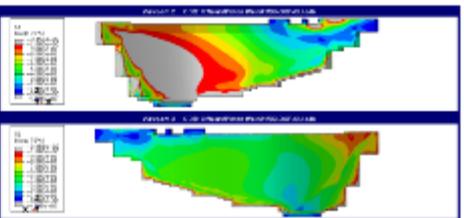
4 mm cl & up



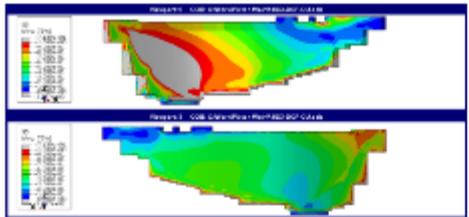
12 mm cl & up



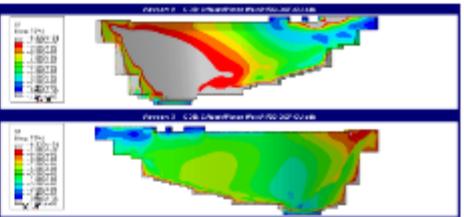
16 mm cl & up



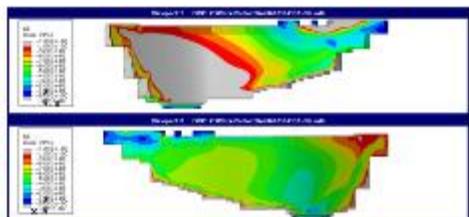
24 mm cl & up



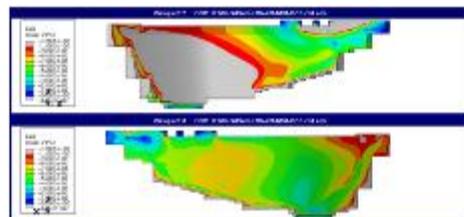
20 mm cl & up



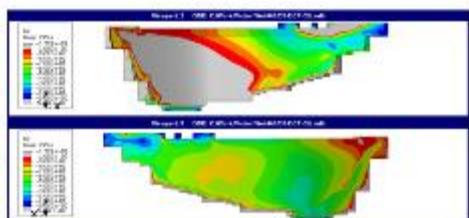
28 mm cl & up



32 mm cl & up



40 mm cl & up



36 mm cl & up

# Appendix D Current Weir Conditions

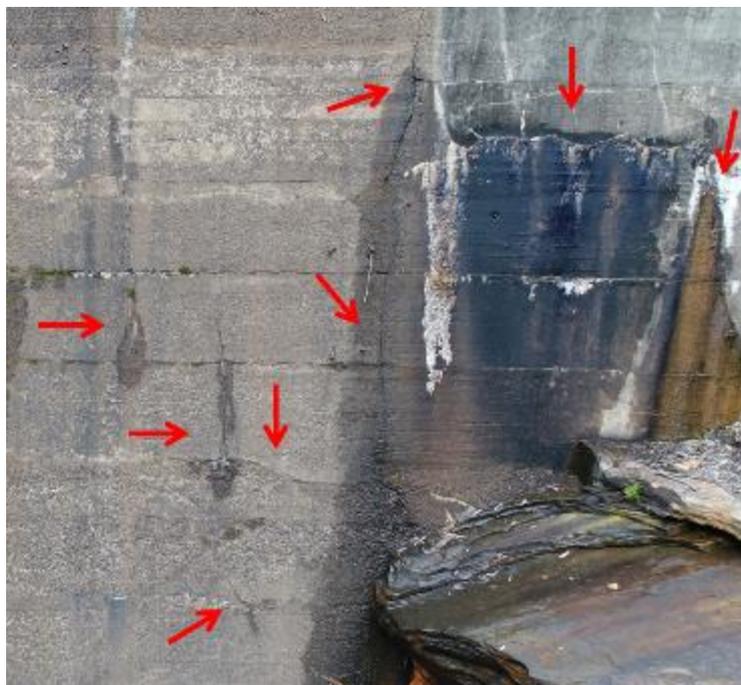


Figure D-1 Photographic record of cracks on downstream face from left abutment (Source: 2023 survey from Michael Nicholson)



*Figure D-2 Photographic record of cracks on downstream face from left abutment, including the outlet works structure (Source: 2023 survey from Michael Nicholson)*



Figure D-3 Photographic record of cracks on downstream face at the middle region (Source: 2023 survey from Michael Nicholson)



Figure D-4 Photographic record of cracks on downstream face from right abutment (Source: 2023 survey from Michael Nicholson)



*Figure D-5. Photographic record of minor cracks on the dam crest at the right-hand side key-way joint (Source: NSW Department of Commerce 2004)*

*IMPORTANT: Do not delete this section break*

**SIMEC MINING**

# **Picton Weir Dam Failure Analysis**

**PAR / PLL Assessment**

311023-50344  
Rev 0



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ABN 17 096 090 158

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PROJECT 311023-50344: Picton Weir Failure Analysis – PAR / PLL Assessment							
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## 1. Introduction

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Picton Weir (previously known as Bargo Dam) is located along the Bargo River between the towns of Bargo and Buxton within the Wollondilly Shire Local Government Area (LGA). The weir is situated about 200 metres downstream of the confluence of Hornes Creek and the Bargo River. It is a concrete gravity-arch dam which was constructed in 1898 and subsequently raised in 1910 and again in 1947. A recent photograph of Picton Weir is shown in **Figure 1-1**.

The dam is also located near an area of mine subsidence (DLWC, 2002). Accordingly, there is potential for the dam to be damaged by the effects of far field regional ground movement. In recognition of this risk, SIMEC Mining has engaged Worley to undertake a dam break analysis to assess the flood impacts of the potential failure of Picton Weir.

Worley Consulting (formerly Advisian) has previously developed flood models for the eastern portion of the Wollondilly LGA, including the stretch of the Bargo River in the vicinity of Picton Weir. The development of these models and the associated flood results are documented in 'Wollondilly Shire Flood Study – Broad Scale Assessment (in draft)' (Advisian, 2023).

The flood models developed for the Flood Study were modified and used for the purposes of this dam break analysis. This report documents the methodology and findings of the dam break analysis, including the estimation of the potential Population at Risk (PAR) and Potential Loss of Life (PLL). The scope of work did not include assessment of the dam consequence category.



**Figure 1-1** Photograph of Picton Weir looking upstream along the Bargo River

## 2. Background

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### 2.1 Site Description

Picton Weir is situated along the Bargo River approximately 30 kilometres to the south-west of Sydney in the Wollondilly Shire of New South Wales. It is located just downstream of the confluence of Hornes Creek and the Bargo River.

The river flows in a northerly direction towards Tahmoor before turning east towards the confluence with the Nepean River, approximately 14 kilometres downstream of Picton Weir. It flows through primarily undeveloped bushland characterised by incised and steep valley walls and dense vegetation on either side of the river channel.

Notable features along the Bargo River include a fire trail (Fire Road No P1) which starts at Bargo River Road and follows the river closely until it reaches Picton Weir, after which it continues southward through bushland before joining Wilson Drive to the south of Buxton. There are also two houses located near the start of this fire trail and about 4.8 kilometres downstream of Picton Weir (refer **Figure 2-1**).

There are several other roads downstream of Picton Weir which could be inundated by flooding along the Bargo River. These include Bargo River Road, Stratford Road, Rockford Road / Arina Road, Charlies Point Road and Sugarloaf Creek Road.

The Old Hume Highway (Remembrance Driveway) and the railway also cross the Bargo River to the south of Tahmoor.

### 2.2 Previous Investigations

#### 2.2.1 Bargo Dam Risk Analysis (2002)

A risk analysis for Bargo Dam was completed as part of a series of risk assessments for dams owned by the Department of Land & Water Conservation. This report presented findings on the probability of various failure modes as well as the consequences of dam failure in terms of loss of life and an estimate of the potential economic damages. The findings presented in this report are based on a dam break analysis completed by Sinclair Knight Merz in 2000. The dam break analysis report was not provided to Worley for the purposes of this investigation.

The key findings are summarised in **Table 2-1** and **Table 2-2** below.

**Table 2-1 Failure Probability Distribution (Dam Risk Analysis 2002)**

Failure Mode	Probability of Failure (per annum)
Flooding	$6.50 \times 10^{-3}$
Sunny day conditions (excl. piping)	$1.22 \times 10^{-6}$
Earthquake	$2.36 \times 10^{-5}$
Piping	$1.44 \times 10^{-6}$

**Table 2-2 Total Losses (Dam Risk Analysis 2002)**

Case	Loss of Life			Economic Damages (\$M)
	Lowest Estimate	Best Estimate	Worst Case Estimate	
Sunny Day failure	0	0.01	0.1	0.048
PMF failure	0	0.01	0.1	0.25

It was noted that the losses for the PMF failure presented in **Table 2-2** above represent total losses. The report states that incremental losses were not assessed as the PMF "under 'no failure' conditions would only produce a flood which varies a little from a flood under 'failure' conditions" (DLWC, 2002).

### 2.2.2 Bargo Dam Surveillance Report (2004)

A surveillance inspection of Bargo Dam was carried out by a State Water representative on 31<sup>st</sup> March 2004. Key information obtained from this surveillance report are listed below.

- The dam was in a satisfactory condition at the time of the inspection.
- The dam was constructed in 1898 and later raised in 1910 and again in 1947.
- The dam is a 13-metre-high concrete gravity-arch structure.
- The estimated storage capacity is 150 ML (150,000 m<sup>3</sup>).
- The total catchment area is estimated to be 66 km<sup>2</sup>.
- Previous site investigations have noted a significant silt deposit built-up behind the dam to a level approximately 5 metres below the dam crest.
- Tahmoor Colliery is undertaking underground mining operations approximately 1.5 kilometres from the dam.

### 2.2.3 Picton Weir Survey (2022)

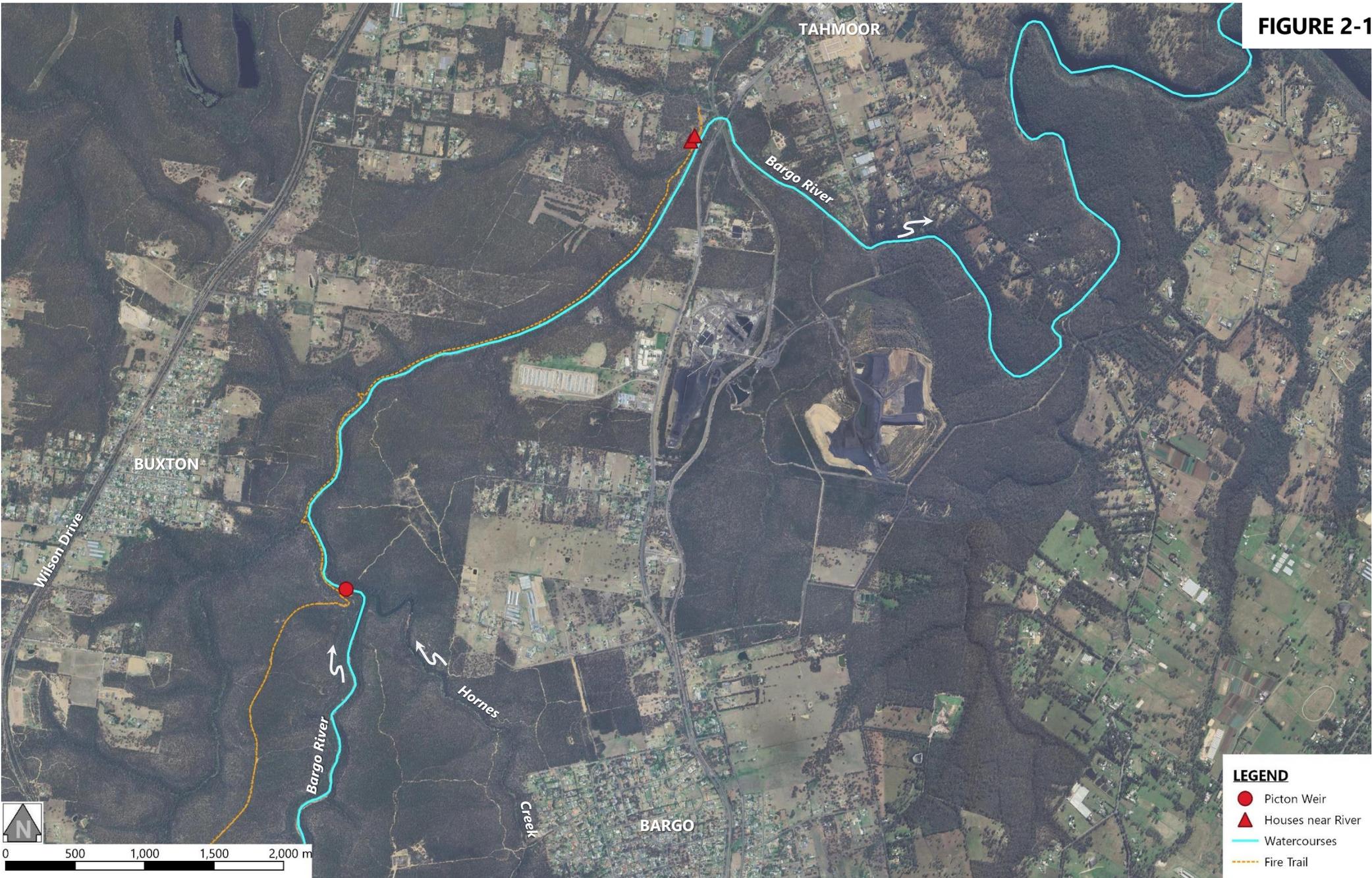
A survey of Picton Weir was carried out by Michael Nicholson Consulting Pty Ltd in October 2022. The survey captured the weir crest at an elevation of 283.0 mAHD.

#### **2.2.4 Wollondilly Shire Flood Study – Broad Scale Assessment (2023)**

Wollondilly Shire Council engaged Worley Consulting (*formerly Advisian*) to undertake a multi-stage project aimed at better understanding and documenting flood risk within the Shire. The first stage of the project involved a broad scale assessment covering all areas of Wollondilly Shire except the National Parks and State Conservation Areas. The second stage of the project involves further detailed flood assessments for smaller study areas with a view to the associated flood study reports forming the basis for future floodplain risk management studies and plans.

As part of the broad scale assessment, Worley Consulting developed WBNM hydrologic and TUFLOW hydraulic models to assess flooding across the Wollondilly Shire LGA. The models include the catchments of Bargo River and Hornes Creek in the vicinity of Picton Weir. These models were adopted as the basis for establishing flood conditions downstream of Picton Weir for this dam break assessment.

FIGURE 2-1



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SITE LOCATION

## 3. Hydrologic and Hydraulic Modelling

---

### 3.1 Flood Modelling Approach

Worley Consulting analysed failure of Picton Weir for both Sunny Day conditions as well as during the Probable Maximum Flood (PMF).

The PMF is the largest flood that could conceivably occur and is usually estimated from the Probable Maximum Precipitation (PMP) in combination with the worst flood-producing catchment conditions (i.e., heavy antecedent moisture). The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given catchment area at a particular location. In Australia, the PMP for catchment areas with a time of concentration up to 6 hours can be estimated using the guidance found in *'The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method'* (Bureau of Meteorology, 2003).

The following approach was adopted by Worley Consulting for the dam break analysis:

1. Adoption of relevant parts of the WBNM hydrologic model developed for the Wollondilly Shire Flood Study (2023) to derive PMF inflow hydrographs.
2. Development of a 1D HEC-RAS model to derive the dam breach hydrographs for the Sunny Day failure and PMF failure scenarios.
3. Adoption and refinement of the TUFLOW hydraulic model developed for the Wollondilly Shire Flood Study (draft, 2023) to determine downstream flood behaviour (i.e., flood extents, depths, velocities and hazards) under both failure and non-failure scenarios.

### 3.2 Hydrologic Modelling (WBNM)

The WBNM hydrologic model developed for the Wollondilly Shire Flood Study (2023) was adopted to obtain PMF inflow hydrographs for the dam break assessment. The details of the WBNM model and the assessment of the PMF are provided in Section 4.3 and Section 5.3 of the 2023 Flood Study, respectively.

Key parameters and assumptions for the WBNM model adopted for the dam break analysis are listed below.

- The subcatchment delineation is unchanged from the 2023 Flood Study. The subcatchments draining to and downstream of the Picton Weir were extracted and adopted to derive PMF inflow hydrographs. The adopted subcatchments are shown in **Figure 3-1**.
- The 2023 Flood Study simulated durations of 1.5 hours, 2 hours and 3 hours for the PMF. The 3-hour PMF was determined to be the critical duration event for the catchment draining to the Picton Weir.
- A Probable Maximum Precipitation rainfall depth of 483 mm was adopted for the 3-hour duration storm in accordance with the guidance found in *'The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method'* (Bureau of Meteorology, 2003).
- An initial loss rate of 1 mm and a continuing loss rate of 0 mm/hr were adopted in accordance with guidance outlined in Book 8 Chapter 6 of Australian Rainfall and Runoff 2019 (ARR 2019).

### 3.3 Dam Breach Hydrographs (HEC-RAS)

#### 3.3.1 HEC-RAS Model Development

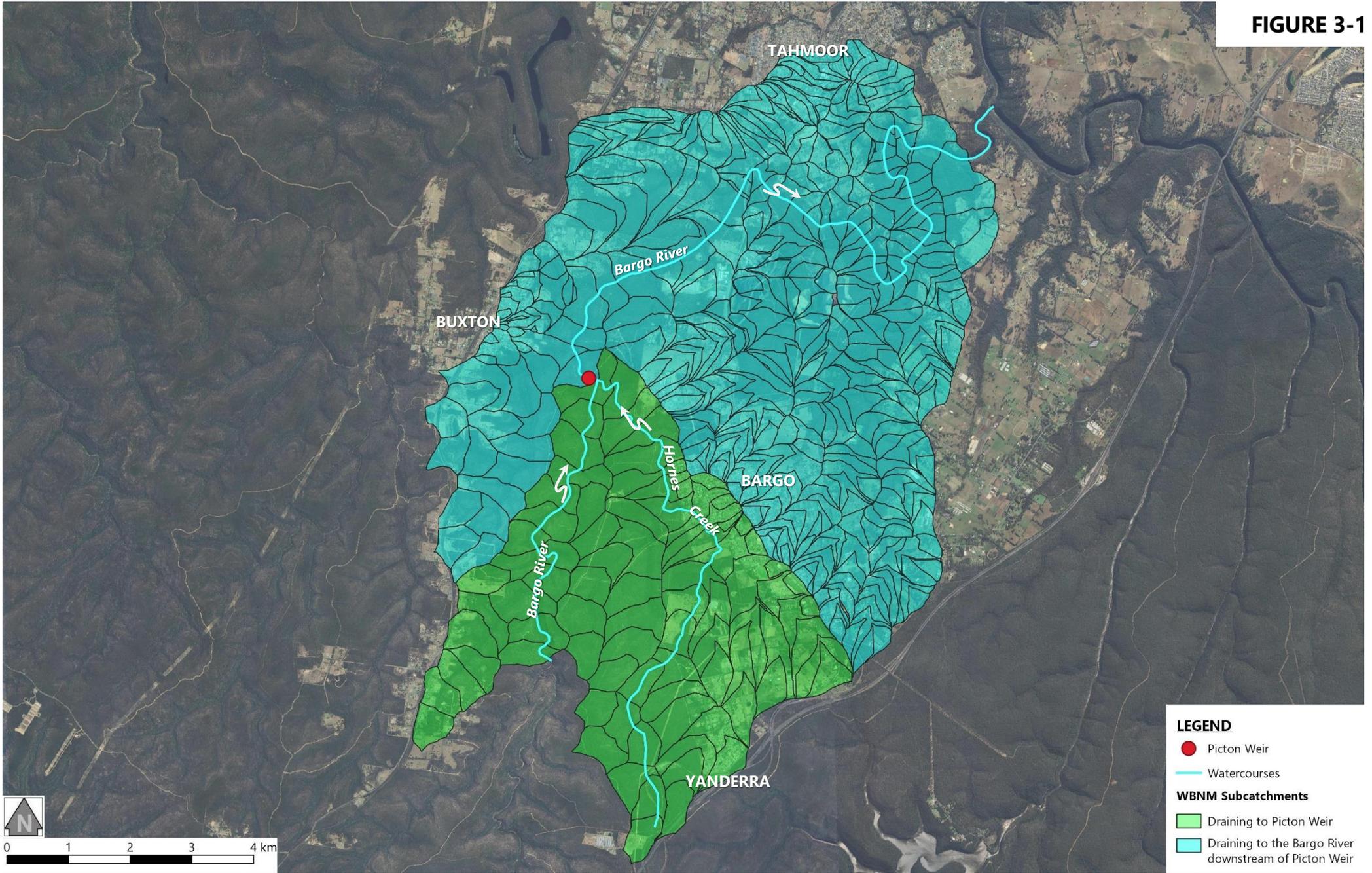
A one-dimensional hydraulic model was developed using the HEC-RAS software to generate dam breach hydrographs for the Sunny Day failure and PMF failure scenarios.

An approximately 1.5-kilometre stretch of the Bargo River downstream of the dam was represented using 13 primary cross-sections with intermediate interpolated cross-sections. The topography of these cross-sections were based on the NSW Spatial Services 1-metre LiDAR DEM for Wollongong, which was captured in 2019.

An inline structure was used to represent the dam wall. The structure was assumed to be 2 metres thick based on a visual estimate of the provided site photographs.

The extent and layout of the model is presented in **Figure 3-2**.

FIGURE 3-1

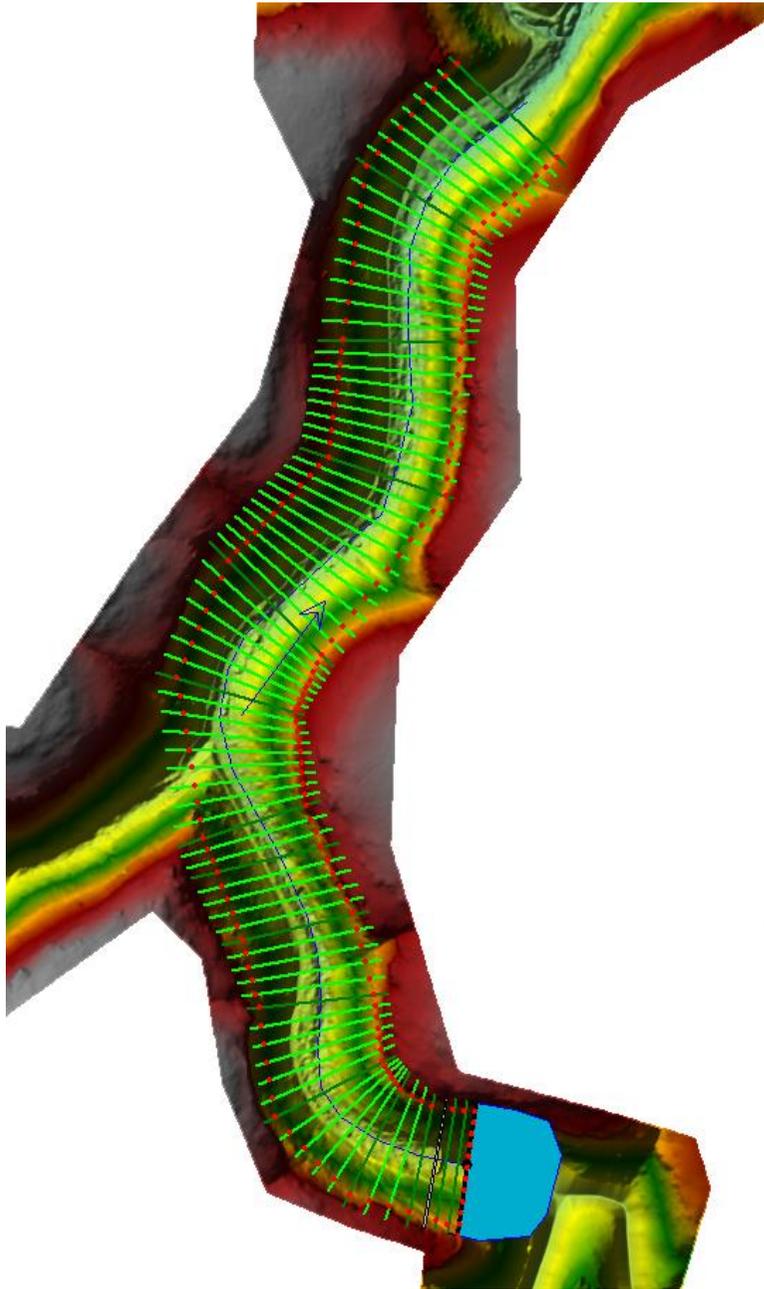


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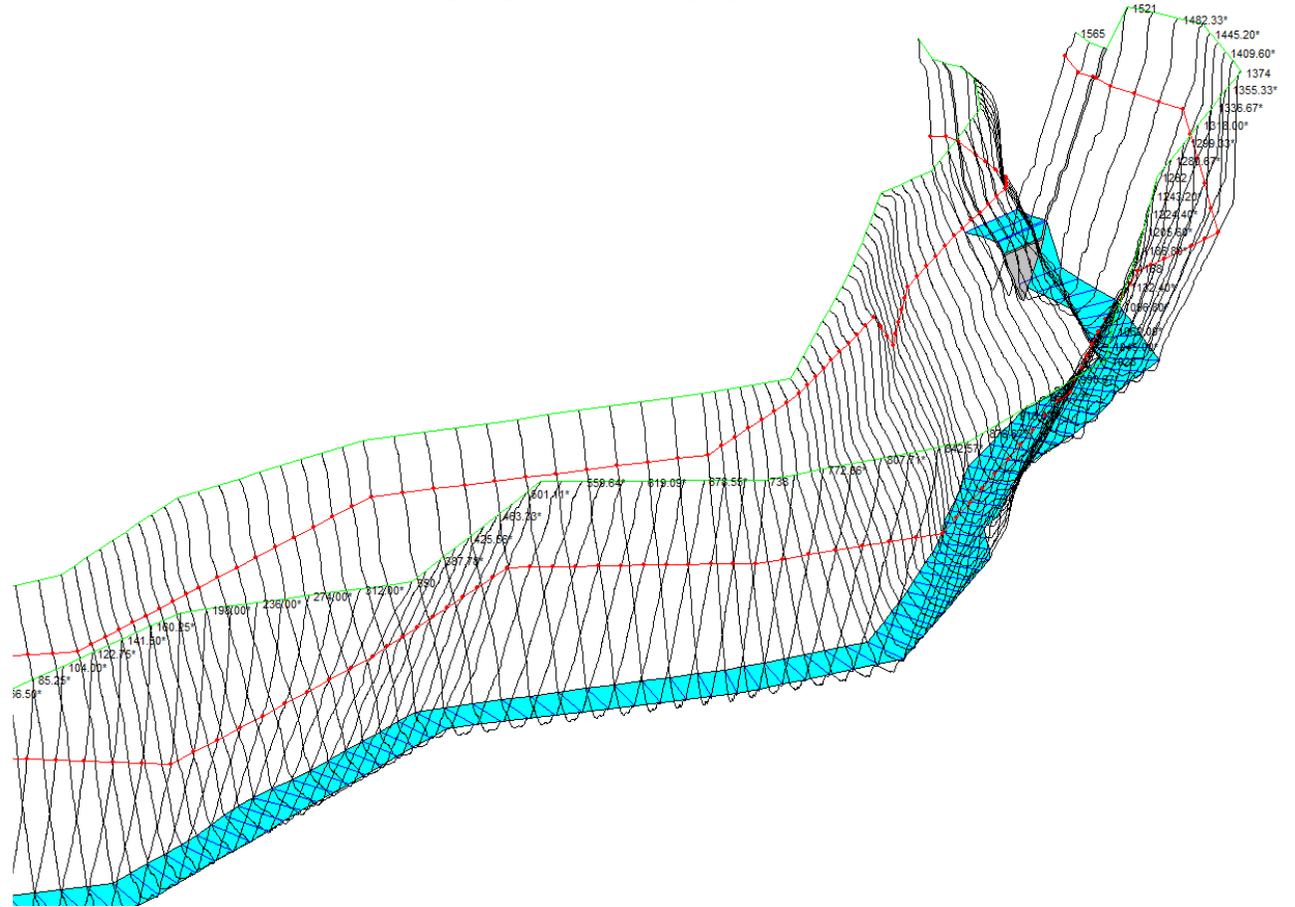


**ADOPTED WBNM SUBCATCHMENTS**

FIGURE 3-2



Picton\_Weir\_02\_PMFDB Plan: PictonWeir\_PMF\_DB 18/04/2024



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HEC-RAS MODEL SCHEMATIC

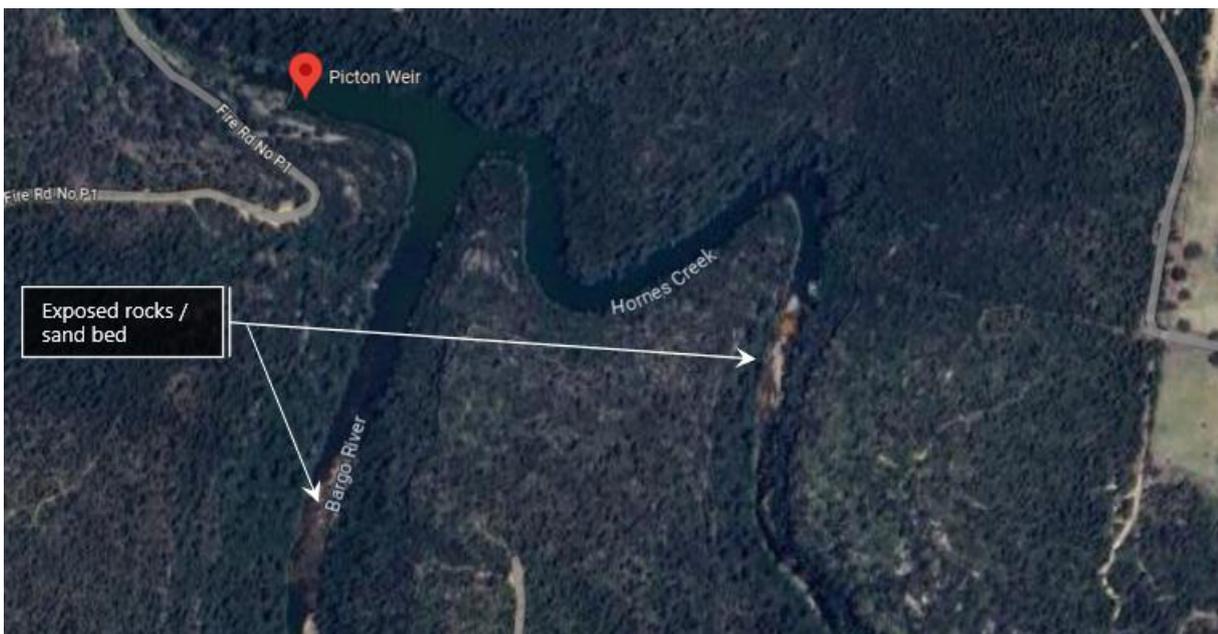
### 3.3.2 Picton Weir Stage-Storage Relationship

There is no available information on the stage-storage relationship for the water stored behind the Picton Weir, except that the storage capacity of the dam is estimated to be about 150 ML (State Water, 2004).

Worley Consulting adopted the following methodology to approximate the stage-storage relationship for the dam.

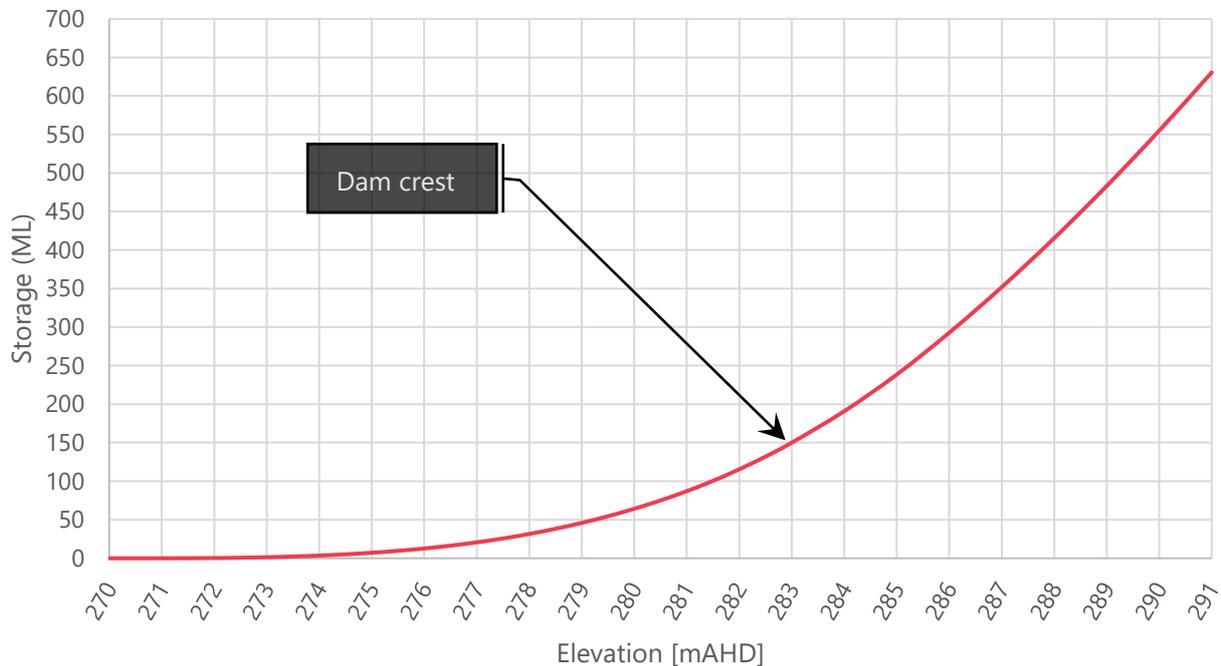
- a) The elevation of the riverbed at the location of the dam was assumed to be 270 mAHD, which is based on the dam crest level of 283 mAHD (from the 2022 survey) and the design dam wall height of 13 metres (State Water, 2004).
- b) The bathymetry of the Bargo River and Hornes Creek was approximated by assuming a uniform riverbed slope of about 1% upstream from the dam. This grade is similar to the water surface elevation captured by the 2019 1-metre LiDAR DEM.
- c) A composite DEM was then developed by stamping the approximated bed levels along Bargo River and Hornes Creek onto the 2019 1-metre LiDAR DEM.
- d) A volumetric analysis was then completed using the composite DEM to develop a stage-storage relationship. This volumetric analysis estimated the storage capacity of Picton Weir to be about 250 ML.
- e) The overestimation of capacity, compared to the previously documented 150 ML, is attributed to the approximation of the bathymetry of Bargo River and Hornes Creek according to a uniform riverbed slope of 1%, which appears to assume an unrealistically low bed level along several stretches of the watercourses which have exposed rocks and sand beds. Examples of these areas are shown in **Figure 3-3**.
- f) Accordingly, the raw stage-storage relationship calculated in step (d) was factored-down to achieve a total storage capacity of 150 ML.

It is noted that the full storage capacity of Picton Weir was assumed as water for the purposes of the dam break analysis (i.e., it was assumed that there was no silt stored behind the weir). This approach was adopted as there was no available information on the volume of silt currently built up behind the weir. This approach is considered to be conservative as it would result in a larger volume of water being released in the dam break failure model simulations.



**Figure 3-3 Exposed Rocks and Sand Beds along the Bargo River and Hornes Creek**

The final adopted stage-storage relationship for the Picton Weir is shown in **Figure 3-4**. The relationship accounts for storage levels above the dam crest of 283 mAHD, such that can occur during the PMF.



**Figure 3-4 Adopted Picton Weir Stage-Storage Relationship**

### 3.3.3 Dam Breach Parameters

Modelling of a dam breach involves estimating key breach parameters such as the geometry of the breach and time of formation. There is a variety of published methods for earthen embankment dams where the breach parameters are estimated through empirical equations developed by analysing previous dam failure cases.

However, for concrete dams breach parameters are typically estimated based on the dam construction type. The HEC-RAS dam break guide (USACE, 2014) has compiled possible ranges for dam breach parameters for concrete dams from several US federal agencies. The range of recommended parameters is summarised in **Table 3-1** below. The adopted breach parameters and other key assumptions are documented in **Table 3-2**.

**Table 3-1 Recommended Breach Parameters for Concrete Dams (source: USACE, 2014)**

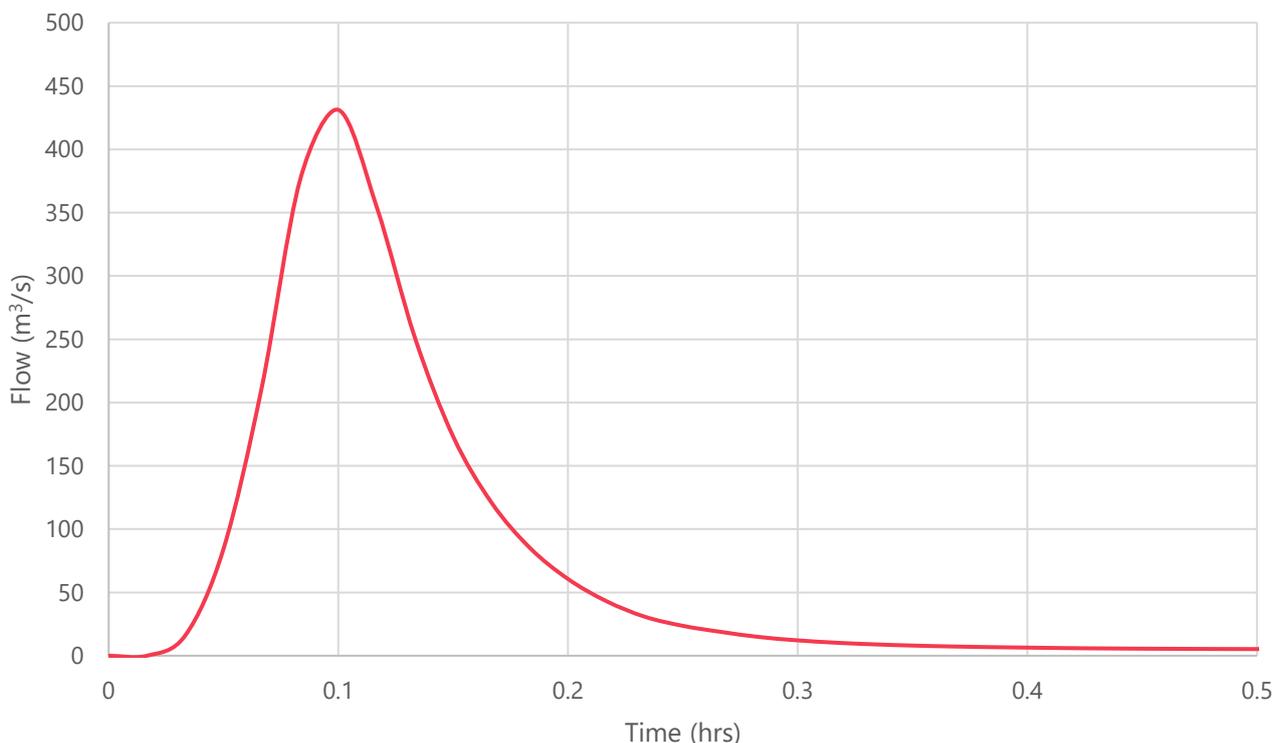
Dam Type	Average Breach Width	Breach Side Slope	Failure Time (hrs)	Source
Concrete Gravity	Multiple monoliths	Vertical	0.1 to 0.5	USACE1980
	Usually $\leq 0.5L$	Vertical	0.1 to 0.3	FERC
	Usually $\leq 0.5L$	Vertical	0.1 to 0.2	NWS
	Multiple monoliths	Vertical	0.1 to 0.5	USACE 2007
Concrete Arch	Entire dam	Valley wall slope	$\leq 0.1$	USACE1980
	Entire dam	0 to valley walls	$\leq 0.1$	FERC
	0.8L to L	0 to valley walls	$\leq 0.1$	NWS
	0.8L to L	0 to valley walls	$\leq 0.1$	USACE 2007

**Table 3-2 Adopted Dam Breach Parameters for Picton Weir**

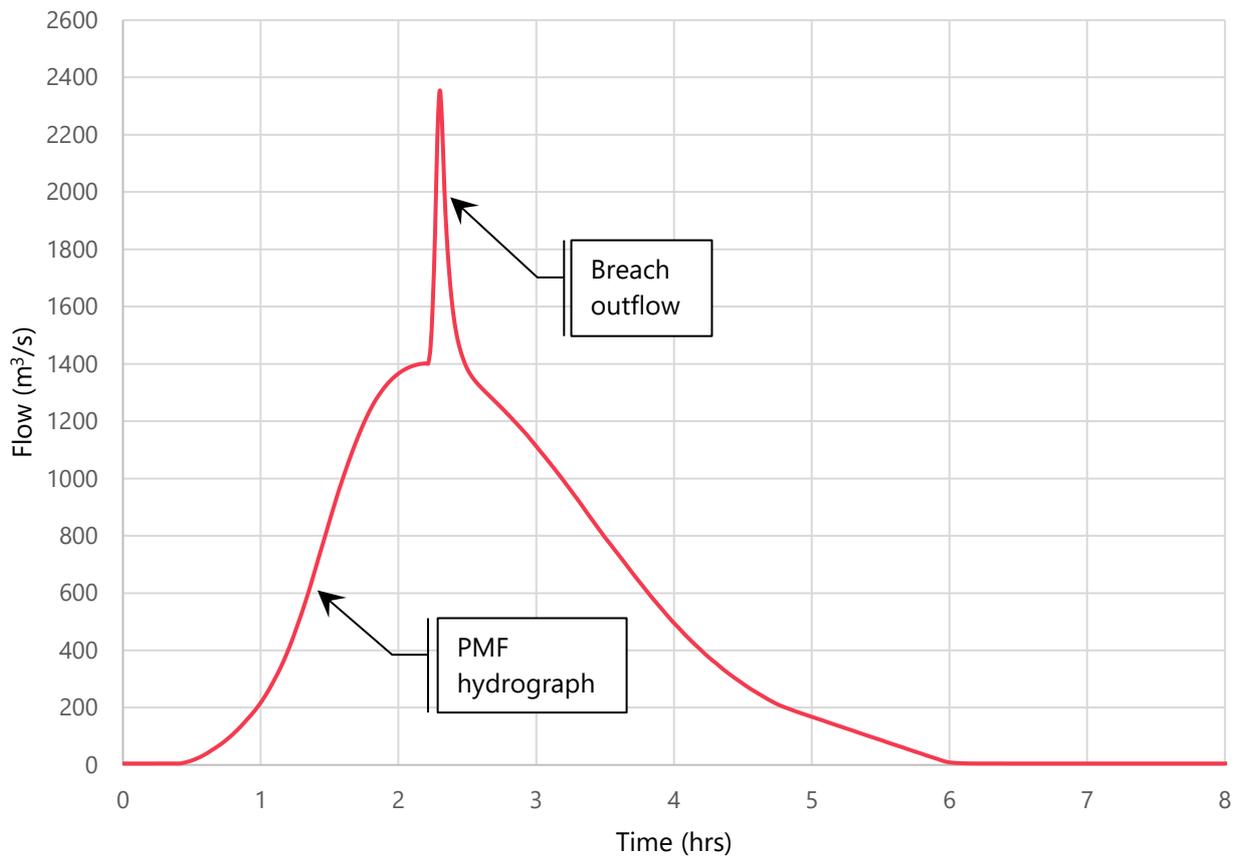
Parameter	PMF Failure	Sunny Day Failure	Justification
Breach geometry	Failure of entire structure	Failure of entire structure	Conservative estimation based on recommendations in (USACE, 2014)
Time of formation	0.1 hours	0.1 hours	Conservative estimation based on recommendations in (USACE, 2014)
Water level at breach initiation (mAHD)	290.4	283.0	PMF failure: Conservatively assumes that dam failure occurs once the peak flood level is reached behind the dam wall. Sunny Day failure: assume that the dam is full of water to the spillway crest level at the moment of failure.
Storage of dam at initiation WL (ML)	580	150	Storage behind dam wall estimated using stage-storage relationship shown in <b>Figure 3-4</b> .
Tailwater level downstream of dam (mAHD)	274.0	274.0	Water surface captured by 2019 1m LiDAR, assumed to be existing baseflow in the river during sunny day conditions and prior to the onset of the PMF.

### 3.3.4 Dam Breach Hydrographs

The dam breach parameters adopted in **Table 3-2** were input to the HEC-RAS model and used to produce the breach hydrographs presented in **Figure 3-5** and **Figure 3-6**. These were then applied to the TUFLOW hydraulic modelling to simulate the downstream failure wave.



**Figure 3-5 Sunny Day Failure Breach Hydrograph**



**Figure 3-6 PMF Failure Breach Hydrograph**

## 3.4 Hydraulic Modelling (TUFLOW)

### 3.4.1 TUFLOW Hydraulic Model Modifications

Modifications made to the TUFLOW model developed as part of the Wollondilly Shire Flood Study (draft, 2023) for the purpose of the dam break assessment are described in the following.

- Modification of the TUFLOW model domain and grid size
  - The Wollondilly Shire Flood Study model covered the entire eastern portion of the LGA and adopted a grid size of 6 metres x 6 metres.
  - For the purposes of the dam break study, the model was truncated to include only the stretch of the Bargo River between Picton Weir at the upstream end and its confluence with the Nepean River near its downstream end (refer **Figure 3-7**).
  - Given the smaller model domain, a grid size of 2 metres x 2 metres was adopted to simulate flood behaviour at a finer resolution.
- Modification of inflow hydrograph boundaries
  - The Wollondilly Shire Flood Study model applied local runoff hydrographs at each of the 3,394 subcatchments adopted for that study.
  - For the purposes of the dam break assessment, total runoff hydrographs were input into the truncated model to represent Bargo River flows at the location of Picton Weir as well as for several tributaries draining into the Bargo River downstream of the dam.

- The location of the input of total runoff hydrographs is shown in **Figure 3-7**.
- Local runoff hydrographs were input into the truncated model for subcatchments draining directly into the Bargo River downstream of Picton Weir.
- Refinement of hydraulic roughness delineation
  - The spatial variation in hydraulic roughness adopted for the Wollondilly Shire Flood Study model was adopted as a basis for the truncated TUFLOW model.
  - The delineation of hydraulic roughness zones was refined to better represent features of the floodplain on a more local scale.
  - The hydraulic roughness delineation adopted for the dam break study is shown in **Figure 3-8**. The depth-varying Manning's 'n' coefficients for each material type were unchanged from the Wollondilly Shire Flood Study.
- Downstream boundary condition
  - A normal-depth downstream boundary condition was applied to allow flows to drain out of the model. A downstream slope of 1% was adopted.
  - The location of the downstream boundary is shown in **Figure 3-7**.

### 3.4.2 TUFLOW Model Simulations

The modified TUFLOW model was used to simulate the following events:

1. Sunny Day failure.
2. PMF 3-hour event for the non-failure scenario (PMF non-failure);
3. PMF 3-hour event for the failure scenario (PMF failure);

FIGURE 3-7

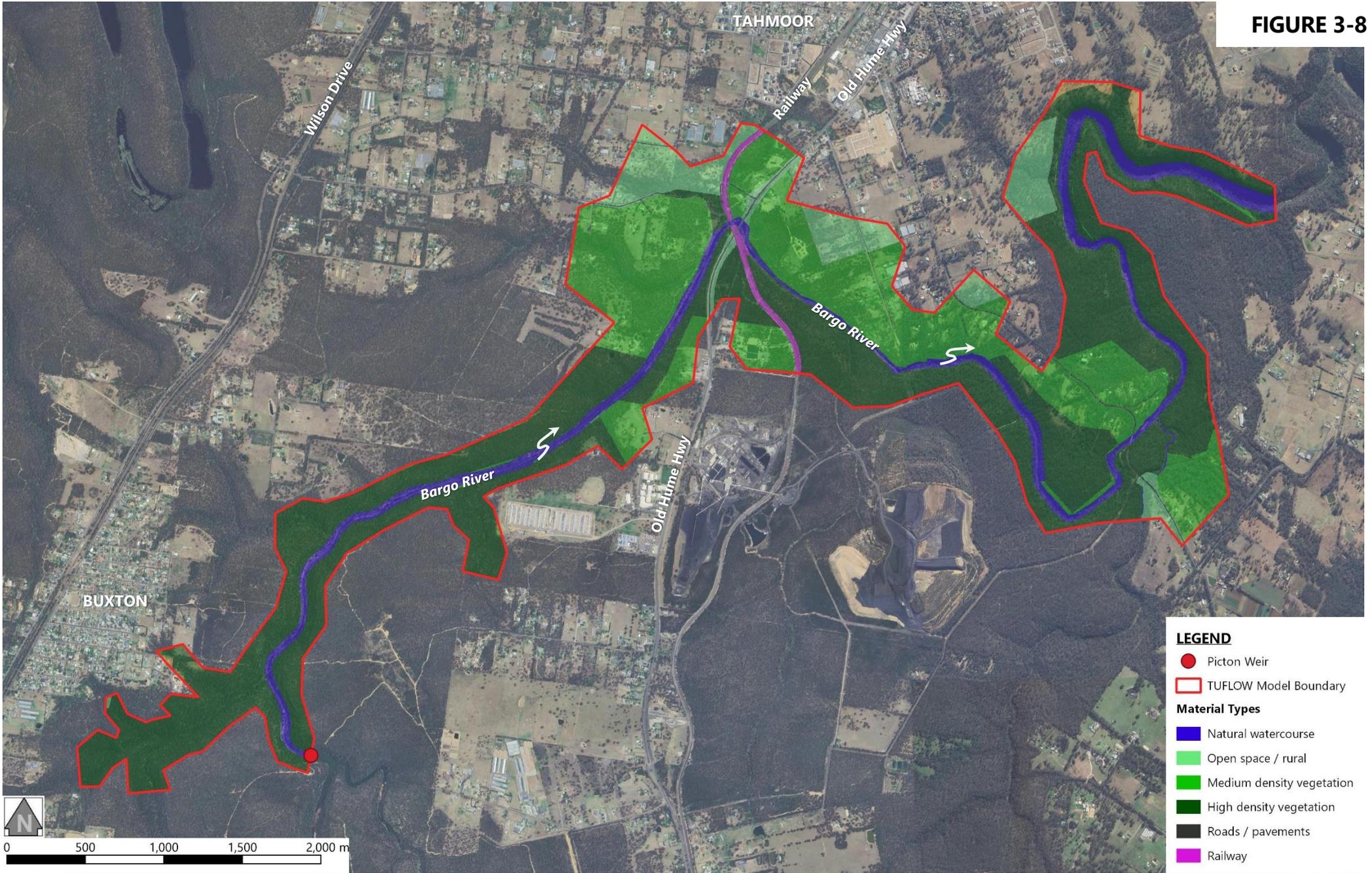


Prepared by:



### TUFLOW MODEL DOMAIN AND BOUNDARY CONDITIONS

FIGURE 3-8



Prepared by:



TUFLOW MODEL MATERIAL TYPES

## 4. Assessment of Flood Impacts

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### 4.1 Inundation Maps

The following inundation mapping has been developed from the TUFLOW model results and is presented in **Appendix A**.

- Figure A-1: Sunny Day failure peak flood depths [Full Model Extent]
- Figure A-2: Sunny Day failure peak flood levels [Full Model Extent]
- Figure A-3: Sunny Day failure peak flow velocities [Full Model Extent]
- Figure A-4: Sunny Day failure flood hazard categories [Full Model Extent]
- Figure A-5: PMF non-failure peak flood depths [Full Model Extent]
- Figure A-6: PMF non-failure peak flood levels [Full Model Extent]
- Figure A-7: PMF non-failure peak flow velocities [Full Model Extent]
- Figure A-8: PMF non-failure flood hazard categories [Full Model Extent]
- Figure A-9: PMF failure peak flood depths [Full Model Extent]
- Figure A-10: PMF failure peak flood levels [Full Model Extent]
- Figure A-11: PMF failure peak flow velocities [Full Model Extent]
- Figure A-12: PMF failure flood hazard categories [Full Model Extent]
- Figure A-13: Sunny Day failure peak flood depths [Houses on Fire Rd No P1]
- Figure A-14: Sunny Day failure peak flood levels [Houses on Fire Rd No P1]
- Figure A-15: Sunny Day failure peak flow velocities [Houses on Fire Rd No P1]
- Figure A-16: Sunny Day failure flood hazard categories [Houses on Fire Rd No P1]
- Figure A-17: Sunny Day failure Queensland DNRME PAR Criteria [Houses on Fire Rd No P1]
- Figure A-18: Sunny Day failure Graham 1999 Flood Severity [Houses on Fire Rd No P1]
- Figure A-19: PMF non-failure peak flood depths [Houses on Fire Rd No P1]
- Figure A-20: PMF non-failure peak flood levels [Houses on Fire Rd No P1]
- Figure A-21: PMF non-failure peak flow velocities [Houses on Fire Rd No P1]
- Figure A-22: PMF non-failure flood hazard categories [Houses on Fire Rd No P1]
- Figure A-23: PMF non-failure Queensland DNRME PAR Criteria [Houses on Fire Rd No P1]
- Figure A-24: PMF non-failure Graham 1999 Flood Severity [Houses on Fire Rd No P1]
- Figure A-25: PMF failure peak flood depths [Houses on Fire Rd No P1]
- Figure A-26: PMF failure peak flood levels [Houses on Fire Rd No P1]
- Figure A-27: PMF failure peak flow velocities [Houses on Fire Rd No P1]
- Figure A-28: PMF failure flood hazard categories [Houses on Fire Rd No P1]
- Figure A-29: PMF failure Queensland DNRME PAR Criteria [Houses on Fire Rd No P1]
- Figure A-30: PMF failure Graham 1999 Flood Severity [Houses on Fire Rd No P1]

## 4.2 Description of Flood Behaviour

### 4.2.1 Sunny Day Failure Scenario

The failure flood wave is expected to remain largely within the confines of the Bargo River channel during a sunny day failure of Picton Weir. The flood wave is predicted to inundate certain sections of the fire trail (Fire Rd No P1) up to a distance of about 1.6 kilometres downstream of the dam.

The flood wave is not expected to inundate any houses nor any other roads / trails, or the railway. Peak flood levels for the sunny day failure scenario are several metres below the railway bridge deck as well as the bridge deck of the Old Hume Highway crossing of the Bargo River.

### 4.2.2 PMF Non-Failure Scenario

During the PMF non-failure scenario, floodwaters are predicted to overtop the banks of the Bargo River leading to the inundation of overbank areas. However, the flood extents remain relatively narrow due to the incised nature of the Bargo River floodplain downstream of Picton Weir. Notable features which are inundated during the PMF non-failure scenario include:

- Large portions of the fire trail (Fire Rd No P1);
- The two houses located near the start of the fire trail;
- A section of Bargo River Road near the intersection with the fire trail;
- A section of the railway to the north of the Bargo River, which is a combination of Bargo River flows and local overland flows arriving from the direction of Tahmoor to the north;
- Sections of several local roads downstream of the railway including Stratford Road, Rockford Road / Arina Road, Charlies Point Road and Sugarloaf Creek Road.

It is noted that PMF levels are several metres lower than the Old Hume Highway bridge crossing of the Bargo River. Flood levels peak near the railway bridge deck level, but the bridge is not overtopped by mainstream flood flows along the Bargo River. The flood mapping in **Appendix A** reflects the flow pattern beneath these bridges (i.e., the flood-free bridge decks are not shown).

### 4.2.3 PMF Failure Scenario

Flood level and flow velocity difference mapping was prepared from the modelling results to quantify the impacts of a dam failure during the PMF. Difference maps are created by comparing peak flood level and flow velocity estimates at each grid cell in the flood model from the results of simulations undertaken for both failure and non-failure scenarios. This effectively creates a contour map of predicted changes in peak levels and velocities (i.e., increases and decreases) and allows visual assessment of the impact of a dam failure on existing peak levels / velocities.

Flood level and flow velocity difference mapping was developed and is presented in **Figure 4-1** and **Figure 4-2**, respectively. As shown in the legend, increases in peak levels / velocities are represented as different shades of red. The white shading indicates increases in peak levels and velocities that are less than 0.3 metres and 0.1 m/s, respectively.

The failure of Picton Weir at the peak of the PMF is expected to increase flood levels by about 2.5 metres in areas immediately downstream of Picton Weir. Flood level increases of 0.3 metres and greater are expected for a distance of about 5.2 kilometres downstream of the dam. The magnitude of the flood level increases continues to decrease further downstream and becomes negligible once the flood wave reaches Rockford Road / Arina Road, Charlies Point Road and Sugarloaf Creek Road.

Peak flow velocities are predicted to increase by up to 3 m/s immediately downstream of the dam. The magnitude of increases in flow velocities reduces as the flood wave propagates downstream. Increases in flow velocity become negligible (i.e., < 0.1 m/s) about 6 kilometres downstream of Picton Weir.

Peak flood levels are predicted to increase by 0.3 to 0.4 metres at the location of the two houses located at the start of the fire trail (Fire Rd No P1). Flow velocities at the location of the houses typically increase by 0.2 to 0.4 m/s.

Despite the predicted increase in peak flood levels, there are no notable increases to the PMF extent. This is due to the incised nature of the Bargo River floodplain downstream of Picton Weir. The model results indicate that the PMF extent typically does not increase laterally by more than 15 metres.

It is noted that flood levels during the failure scenario are still predicted to be several metres lower than the Old Hume Highway bridge crossing of the Bargo River. The PMF failure flood wave peaks near the railway bridge deck level, but the bridge is not expected to be overtopped by flows along the Bargo River as a result of a dam break. The flood mapping in **Appendix A** reflects the flow pattern beneath these bridges (i.e., the flood-free bridge decks are not shown).

FIGURE 4-1

**LEGEND**

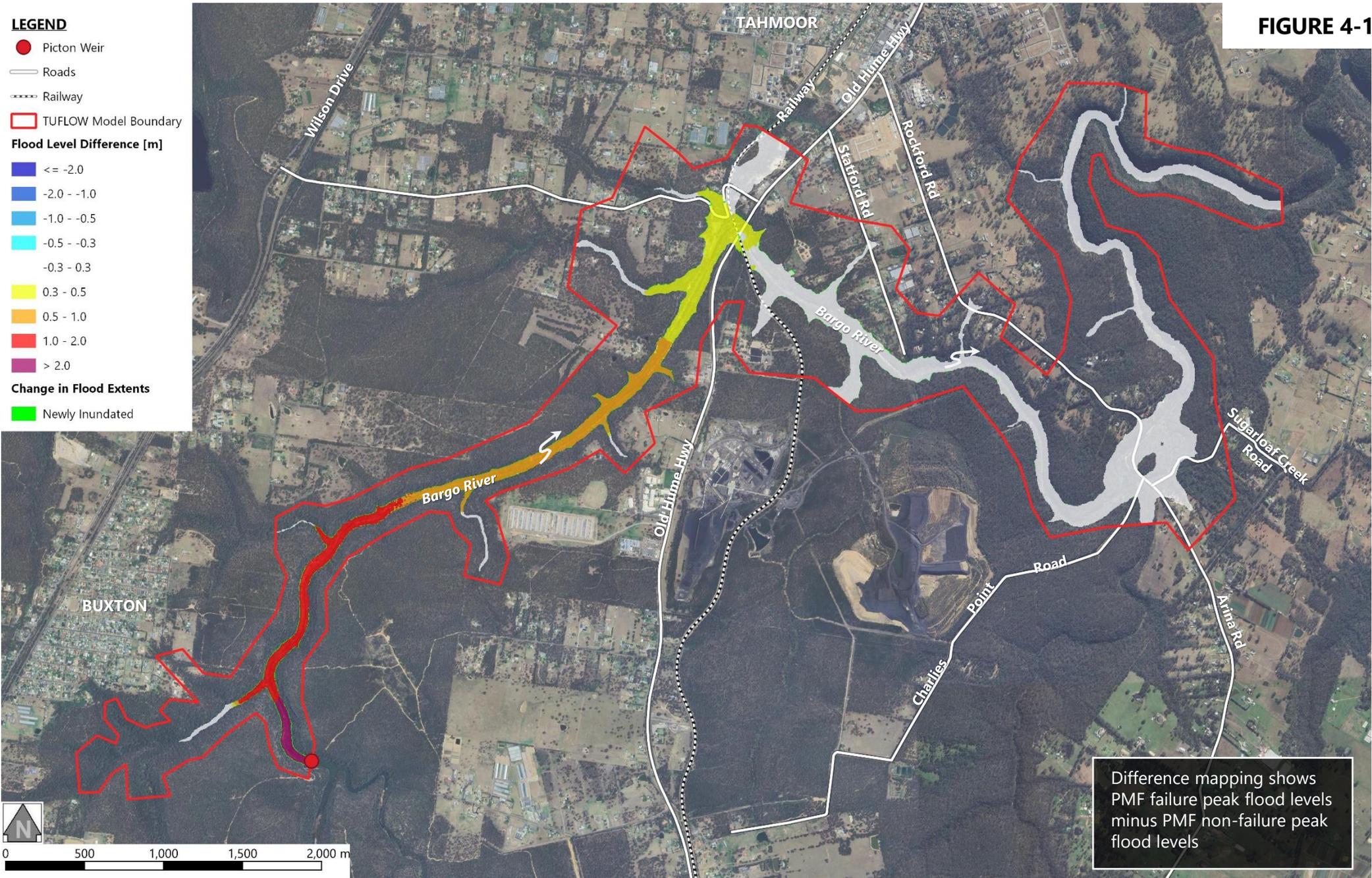
- Picton Weir
- Roads
- ⋯ Railway
- ▭ TUFLOW Model Boundary

**Flood Level Difference [m]**

- ≤ -2.0
- 2.0 - -1.0
- 1.0 - -0.5
- 0.5 - -0.3
- 0.3 - 0.3
- 0.3 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- > 2.0

**Change in Flood Extents**

- Newly Inundated



Prepared by:



**INCREASE IN PEAK FLOOD LEVELS  
DUE TO DAM FAILURE  
DURING THE PMF**

FIGURE 4-2

**LEGEND**

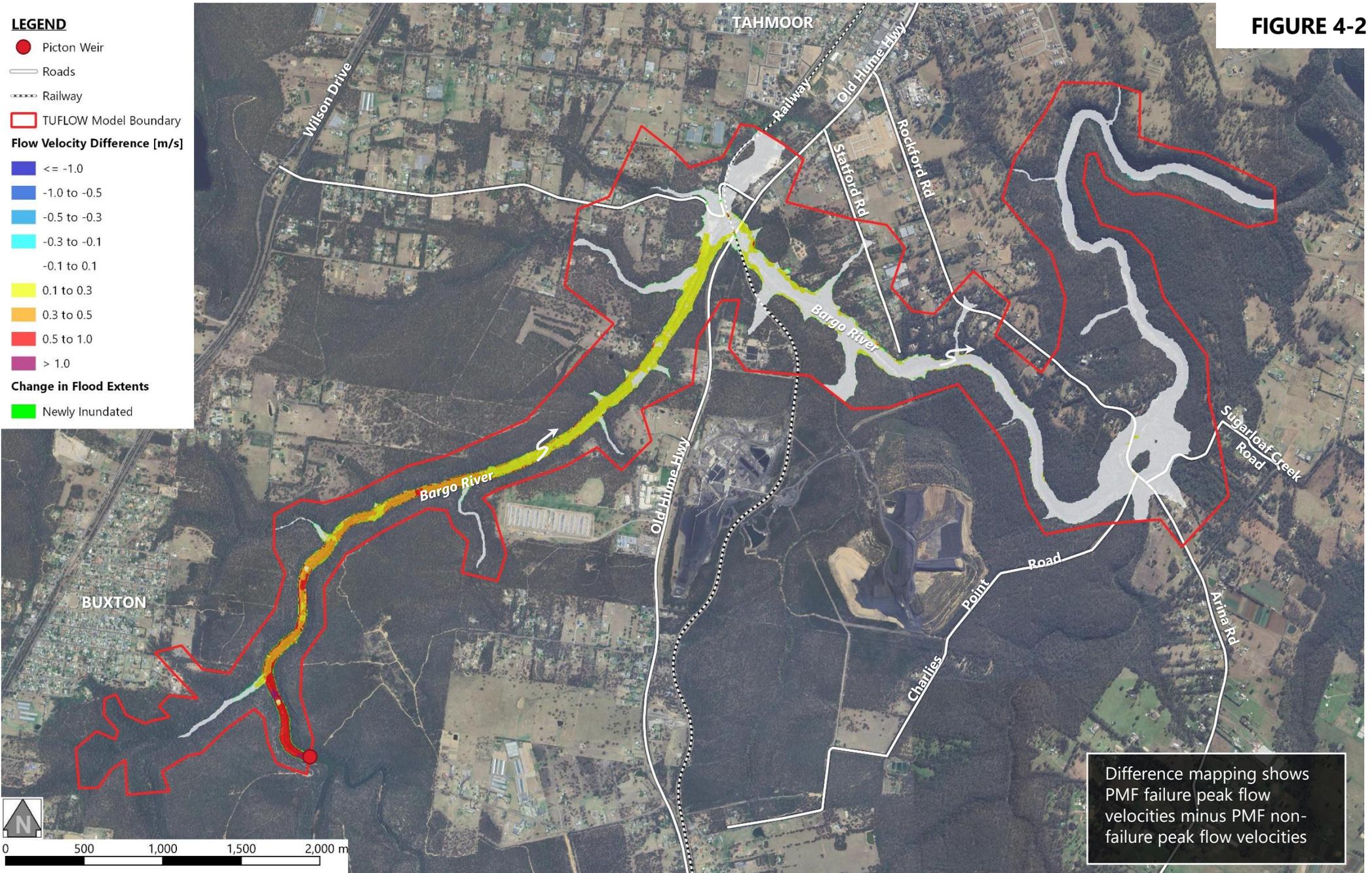
- Picton Weir
- Roads
- ⋯ Railway
- ▭ TUFLOW Model Boundary

**Flow Velocity Difference [m/s]**

- ≤ -1.0
- 1.0 to -0.5
- 0.5 to -0.3
- 0.3 to -0.1
- 0.1 to 0.1
- 0.1 to 0.3
- 0.3 to 0.5
- 0.5 to 1.0
- > 1.0

**Change in Flood Extents**

- Newly Inundated



Difference mapping shows PMF failure peak flow velocities minus PMF non-failure peak flow velocities

### 4.3 Population at Risk & Potential Loss of Life

The impact of a dam break is typically assessed by estimating the Population at Risk (PAR) and Potential Loss of Life (PLL).

For a dam failure during a flood scenario, the dam break impact is estimated as the difference in the PAR under a flood scenario without dam failure compared to the PAR during the same flood with dam failure. This is referred to as the incremental increase in the PAR due to flooding.

Similar to the PAR assessment, an incremental PLL is calculated by comparing the PLL under a flood scenario without dam failure to the PLL during the same flood with dam failure.

The total PAR and PLL is also estimated in order to quantify the impact of a dam failure for the Sunny Day Failure scenario.

#### 4.3.1 Methodology for PAR / PLL Estimation

##### Permanent Population

Worley Consulting adopted the comprehensive assessment method from the Queensland Department of Natural Resources, Mines and Energy (2018) guidelines as the criteria to estimate the permanent population at risk (i.e., people within buildings). The assessment criteria are shown in **Table 4-1**, while the mapping of floodplain areas which exceed the assessment criteria is shown in Figure A-17, Figure A-23 and Figure A-29 (refer **Appendix A**).

**Table 4-1 Queensland DNRME (2018) Population at Risk Criteria**

Depth (D) (m)	Depth (D) x velocity (V) (m <sup>2</sup> /s)	Population at Risk (Yes/No)
$0.3 \leq D < 0.5$	$DV < 0.4$	No
$0.3 \leq D < 0.5$	$DV \geq 0.4$	Yes
$D \geq 0.5$	DV not considered	Yes

The PLL is calculated by applying a fatality rate to the PAR. The NSW Government Gazette No. 113 (March 2022) lists the guidelines found in 'A Procedure for Estimating Loss of Life Caused by Dam Failure' (Graham, 1999) as an acceptable method for calculation of the number of fatalities. , specifically Table 7.

Table 7 from Graham (1999) is reproduced in **Table 4-2**, which outlines a range of fatality rates based on flood severity, warning time and understanding of flood severity.

The NSW Government Gazette requires the flood severity categories to be defined as follows:

- **High:** Velocity-depth product ( $V \times D$ )  $\geq 15$  m<sup>2</sup>/s AND maximum rate of rise  $\geq 3$  metres per 5-minute period.
- **Medium:**  $V \times D \geq 4.6$  m<sup>2</sup>/s AND depth  $\geq 3.0$  m, in areas not classified as 'High' flood severity.
- **Low:** Inundated areas which do not meet the criteria for 'Medium' flood severity.

There are no areas of 'High' flood severity in the vicinity of buildings in the downstream floodplain for any modelled scenario as the maximum observed rate of rise does not exceed the required threshold of 3 metres

in 5 minutes. The flood severity for the different modelled scenarios is presented in Figure A-18, Figure A-24 and Figure A-30 (refer **Appendix A**).

**Table 4-2 Graham 1999 Recommended Fatality Rates**

Flood Severity	Warning Time (minutes)	Flood Severity Understanding	Fatality Rate	
			Suggested	Suggested Range
High	No Warning	Not Applicable	0.75	0.30 to 1.00
	15 to 60	Vague	Use the values shown above and apply to the number of people who remain in the dam failure floodplain after warnings are issued. No guidance is provided on how many people will remain in the floodplain.	
	More than 60	Precise		
Medium	No Warning	Not Applicable	0.15	0.03 to 0.35
	15 to 60	Vague	0.04	0.01 to 0.08
		Precise	0.02	0.005 to 0.04
	More than 60	Vague	0.03	0.005 to 0.06
		Precise	0.01	0.002 to 0.02
Low	No Warning	Not Applicable	0.01	0.0 to 0.02
	15 to 60	Vague	0.007	0.0 to 0.015
		Precise	0.002	0.0 to 0.004
	More than 60	Vague	0.0003	0.0 to 0.0006
		Precise	0.0002	0.0 to 0.0004

### Transient Population

The methodology outlined in 'Flooded cars: estimating the consequences to itinerants exposed to dam break floods on roads' (Campbell et al, 2013) was adopted to estimate the risk that a dam break poses to itinerant road / trail users. More specifically, Campbell's Method estimates the risk to itinerants under three possible situations, as follows.

**Situation A:** Road users being present in the flood inundation zone at the time that the dam break flood wave passes through.

**Situation B:** Road users driving / entering the flood inundation zone as the flood wave is passing through (i.e., during the flood).

**Situation C:** Road users driving / entering into the inundation zone after the flood wave has passed through and having an accident due to damage that has occurred to the road.

This method offers guidance on estimating an appropriate PAR based on factors such as Annual Average Daily Traffic (AADT), time of day in which the dam break could occur, typical travel speed and length of inundated road.

The PLL is then estimated by applying factors which relate to the probability of road users taking action to avoid the dam break hazard, having an accident once in the flood inundation zone and the vulnerability of people once an accident has occurred.

The recommended equation to estimate the PLL is:

$$PLL = PAR \times P_{NET} \times P_{A:NE} \times V_{D:A}$$

where:

$PAR$  = estimated Population At Risk

$P_{NET}$  = probability of road user taking no action to avoid the flood (between 0 – 1)

$P_{A:NE}$  = probability of an accident once inside the flood inundation zone (between 0 – 1)

$V_{D:A}$  = probability of a fatality once an accident has occurred (between 0 – 1)

Campbell's Method requires the user to make several judgements on these probabilities and associated road user behaviour in order to calculate the PAR / PLL. These parameters and assumptions, and other key points are as follows.

- The number of road users present in the flood inundation zone as the flood event occurs (Situation A) is determined by taking into account the expected road users per hour, speed of travel and length of the inundated road.
- Situation B and Situation C are defined as mutually exclusive. Situation B should be analysed for high traffic volume scenarios, while Situation C is only applicable for very low traffic volume scenarios.
- Road users are more likely to take action to avoid the dam break hazard in conditions of good visibility (i.e., during the day).
- For Situation A there is naturally no chance that a road user would be able to take any action to avoid the hazard and hence the probability of taking no action will be 1.
- Road users are more likely to enter the dam break hazard if the flood conditions look shallow.
- A factor of 0.1 should be applied to the sunny day estimate of the probability of drivers taking no action to avoid the hazard in scenarios for infrequent floods (i.e., rarer than the 1% AEP flood). This is because Campbell acknowledges that it is likely for less cars to be on the road during a heavy rainfall event and drivers would be more cautious when there is significant flooding within the catchment in general.
- The probability of an accident occurring once within the flood inundation zone ( $P_{A:NE}$ ) is mainly governed by the expected velocity-depth product in the inundation zone. The adopted threshold between a higher probability of an accident and a lower probability of an accident is 0.45 m<sup>2</sup>/s.

A copy of Campbell's Method is included as **Appendix B**.

### 4.3.2 Sunny Day Failure Total PAR / PLL Estimation

As outlined above, the Sunny Day Failure flood wave is not expected to inundate any houses, roads or the railway. Accordingly, there will be no persons at risk in these areas.

The flood model results show that only certain sections of the fire trail (Fire Rd No P1) up to 1.6 kilometres downstream of Picton Weir will be inundated during the Sunny Day failure scenario. The total length of the fire trail expected to be inundated by the dam failure flood wave is about 530 metres.

Campbell's Method was applied to estimate the PAR / PLL along the fire trail during the Sunny Day failure scenario. The following key assumptions were adopted.

- The users of the trail are predominantly bushwalkers. No vehicles are expected to use the trail on a regular basis.

- The trail is not used at night. The trail is only used within a 12-hour window during daylight hours.
- There is no Annual Average Daily Traffic (or similar) data available for this trail. In lieu of available data, it is assumed that there could potentially be up to five (5) groups of bushwalkers walking in groups of up to four (4) persons on any given day. This is considered a conservative estimate for weekdays and reasonable estimate for weekend days.
- The average walking speed is 4 km/h.
- It is assumed that only Scenario A from Campbell's method will apply. It is assumed that Scenario B and Scenario C will not be applicable for bushwalkers as they would not enter the flood inundation zone given that they will have sufficient time to see and avoid any inundated areas when travelling at walking pace.

The estimation of PAR / PLL for the Sunny Day failure scenario is documented in **Table 4-3** below. Note that the same result would come from an assumed bushwalker traffic on the trail of 10 groups of 2 persons.

**Table 4-3 Parameters for Estimation of Sunny Day failure PAR / PLL**

ID *	Parameter	Value	Comment
[2]	Proportion of day with users	0.5	Assume 12-hour window for bushwalkers to be on the trail.
[3]	Temporal spatial probability of exposure to hazard	0.055	$\frac{530 \text{ metres}}{4 \text{ km/h} \times 1000} \times \frac{5}{12} \text{ groups per hour}$
[4]	Probability of taking no action to avoid hazard	1	No chance of avoiding hazard as bushwalkers would already be in inundation zone for Scenario A.
[5]	Probability of an accident once in water	1	Conservatively assume that an accident will occur once in the inundation zone.
[6]	Exposure	0.028	[2] x [3] x [4] x [5]
[7]	PARv	4	Assumed number of bushwalkers in each group.
[8]	<b>Weighted PARv</b>	<b>0.11</b>	<b>[6] x [7]</b>
[9]	Fatality Rate	0.8	Adopted from fatality rate curves in USBR (2015), specifying a maximum fatality rate of 0.8 for areas of inundation where $DV \leq 10 \text{ m}^2/\text{s}$ . Inundated areas along fire trail typically have $DV \leq 10 \text{ m}^2/\text{s}$ .  A fatality rate of 0.8 aligns with the recommended range of 0.7 to 0.999 in (Campbell, 2015).
[10]	<b>Weighted PLL</b>	<b>0.09</b>	<b>[8] x [9]</b>

\*Parameter ID per Table 5 of (Campbell, 2015)

The potential for bridge collapse was not considered during the assessment. As discussed above, the Sunny Day failure wave will be several metres below the bridge decks at the railway and Old Hume Highway bridges over the Bargo River. Accordingly, significant damage to the bridges is not expected.

### 4.3.3 PMF Failure Incremental PAR / PLL Estimation

The extent of inundation of downstream features during the PMF non-failure and failure scenarios is summarised in **Table 4-4**. The table effectively shows the impact of the dam failure on the length or area of inundation.

**Table 4-4 Summary of Inundated Areas Downstream of Picton Weir**

Feature	Area / Length of Inundation		Comment
	PMF Non-Failure	PMF Failure	
House 1 on Fire Rd No P1	Entire house	Entire house	Failure of the dam results in an increase in peak flood levels of typically by 0.3 to 0.4 metres at the houses.
House 2 on Fire Rd No P1	125 m <sup>2</sup>	170 m <sup>2</sup>	
Fire Trail (Fire Rd No P1)	4,770 metres	4,840 metres	Failure of the dam results in an increase in peak flood levels of between 0.4 and 2.5 metres along the length of the fire trail, from the start of the trail at Bargo River Road to the sections of the trail immediately downstream of Picton Weir.
Bargo River Rd	220 metres	235 metres	Dam break flows typically increase the peak flood level by about 0.4 metres along this road.
Railway	650 metres	650 metres	Inundation is dominated by local overland runoff and not from flows along the Bargo River. The dam break does not impact on flood behaviour along the railway.
Stratford Rd	40 metres	40 metres	
Rockford Rd / Arina Rd	560 metres	560 metres	Increases in flood levels and flow velocities due to the dam break are negligible in these areas further downstream.
Charlies Point Rd	115 metres	115 metres	
Sugarloaf Creek Rd	300 metres	300 metres	
Old Hume Highway	0 metres	0 metres	The road is not inundated by flows along the Bargo River.

It is assumed that there will be no cars driving on the roads during the extreme rainfall that leads to the Probable Maximum Flood, particularly given that the roads are not major thoroughfares. Likewise, it is assumed that bushwalkers will not be active along the fire trail during an extreme rainfall event.

Furthermore, the incremental increase in the inundation lengths of roads under the dam failure scenario is only minor; 15 metres (7%) along Bargo River Road and 70 metres (1.5%) along the fire trail.

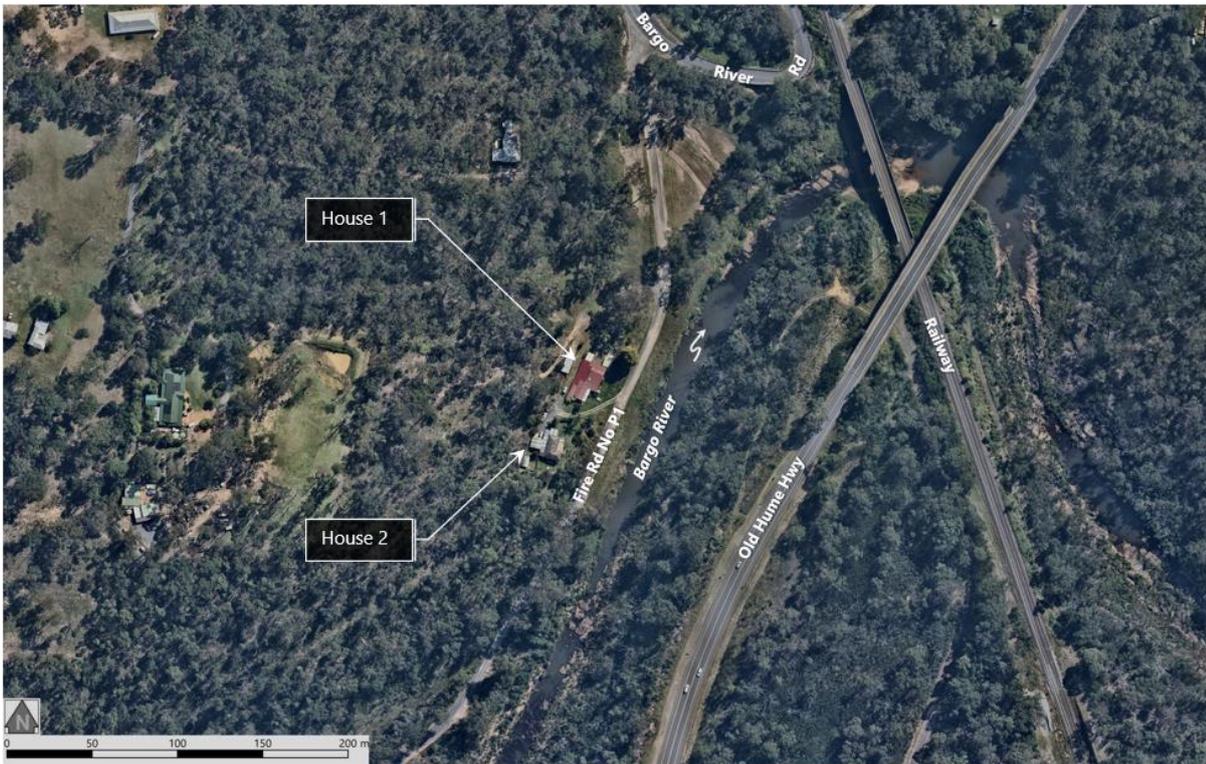
Therefore, it is concluded that the incremental transient PAR will be effectively zero (0). Similarly, the incremental transient PLL will be zero (0).

The potential for bridge collapse was not considered during the assessment. The PMF flood level with or without dam failure will reach a similar level at the bridges; several metres below the deck of the Old Hume Highway bridge, and near the deck level for the railway bridge (although not overtopping). Accordingly, significant damage to the Old Hume Highway bridge is not expected. If some damage occurs to the railway bridge during the PMF, it would be similar for both the dam failure and non-failure scenarios, thereby not resulting in any significant incremental increase in risk.

The permanent PAR and PLL at the two houses along the fire trail was estimated using the methodology in Table 7 of '*A Procedure for Estimating Loss of Life Caused by Dam Failure*' (Graham, 1999). A summary of the PAR and PLL assessment is provided in **Table 4-5**. The location of the houses is shown in **Figure 4-3**.

Key assumptions for the assessment are as follows:

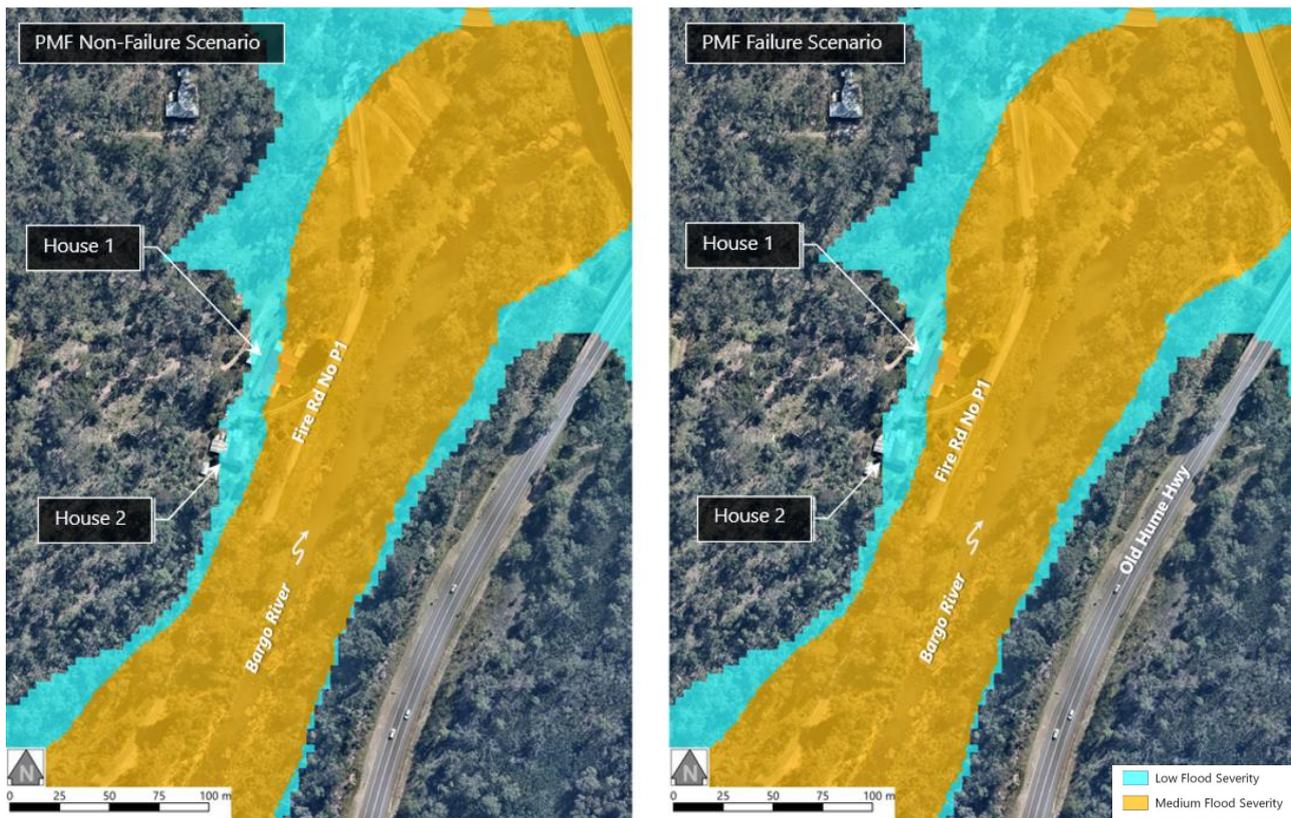
- Four (4) occupants are conservatively assumed to be in each house. The 2021 Census data available from the Australian Bureau of Statistics specified that the average household size in the Bargo area is 2.8.
- It was conservatively assumed that there would be no warning time for the occupants of the houses to evacuate, given that the critical PMF duration is 3 hours.
- For House 1, according to interrogation of LiDAR topographic data, the PMF in both the non-failure and failure scenarios is expected result in a significant depth of flooding above floor level, meaning there would be no measurable difference in PAR under each scenario.
- For House 2, the flooding in both scenarios is expected to reach approximately the floor level of the house, which is perched above sloping land. For lack of any detailed floor level information, the portion of house footprint inundated provides a reasonable reflection of the potential for the house to be damaged and occupants be placed at risk.
- The flood severity mapping for the non-failure and failure scenarios is shown in **Figure 4-4** below. A comparison of the mapping shows that the delineation between areas of 'Low' and 'Medium' flood severity are very similar for both non-failure and failure scenarios. Therefore, it was considered appropriate to adopt the same fatality rate under non-failure and failure scenarios for both houses.
- The mapping shown in **Figure 4-4** indicates that the footprint of House 1 is evenly split into areas of 'Low' and 'Medium' flood severity. The recommended fatality rate for areas of 'Low' and 'Medium' flood severity with no warning time are 0.01 and 0.15, respectively. A fatality rate of 0.15 was conservatively adopted for the assessment of PLL at House 1.



**Figure 4-3 Plan showing Location of the Two Houses along the Fire Trail**

**Table 4-5 Estimation of Permanent PAR / PLL for the PMF Scenarios**

Parameter	House 1		House 2		TOTAL	
	PMF Non-Failure	PMF Failure	PMF Non-Failure	PMF Failure	PMF Non-Failure	PMF Failure
No. of occupants	4	4	4	4	<b>8</b>	<b>8</b>
% of house footprint inundated	100%	100%	64%	87%	-	
PAR	4	4	2.56	3.48	<b>6.56</b>	<b>7.48</b>
Flood Severity (Graham, 1999)	Part 'Low', Part 'Medium'	Part 'Low', Part 'Medium'	'Low'	'Low'	-	
Fatality Rate	0.15	0.15	0.01	0.01	-	
PLL	0.6	0.6	0.026	0.035	<b>0.626</b>	<b>0.635</b>
Incremental PAR					<b>0.92</b>	
Incremental PLL					<b>0.01</b>	



**Figure 4-4 Flood Severity Mapping for the PMF Non-Failure and PMF Failure Scenarios**

#### 4.3.4 Summary of PAR / PLL

The total PAR / PLL for the Sunny Day Failure scenario and the incremental PAR / PLL for the PMF Failure scenario is summarised below.

- Total Sunny Day Failure PAR: 0.11
- Total Sunny Day Failure PLL: 0.09
- Incremental PMF Failure PAR: 0.92
- Incremental PMF Failure PLL: 0.01

The low PAR/PLL for the Sunny Day Failure scenario reflects the fact that no houses are inundated in this scenario and the only risk is to itinerant users of the fire trail.

The relatively small incremental PAR/PLL during the PMF failure scenario is reflective of the fact that the two houses on Fire Rd No P1 are already affected during the PMF (without dam failure) and therefore, the incremental impact of dam failure is expected to be small. The incremental PLL due to failure during the PMF is particularly low given there is no material difference to the risk at House 1, and the flood severity at House 2 is "low" and hence a very low fatality rate is applied to the incremental PAR.

## 5. Conclusion

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The above report documents the methodology and findings from investigations undertaken to assess the impact of failure of Picton Weir for both the Sunny Day failure and PMF failure scenarios.

The WBNM model developed for the '*Wollondilly Shire Flood Study – Broad Scale Assessment*' (Advisian, in draft 2023) was adopted as the basis for defining the hydrology of the catchment draining to Picton Weir, which includes the Bargo River as well as Hornes Creek, as well as areas downstream of the weir. Based on the hydrologic analysis completed in the 2023 study, the 3-hour storm was adopted as the critical duration Probable Maximum Flood event.

The TUFLOW hydraulic model developed for the '*Wollondilly Shire Flood Study – Broad Scale Assessment*' was truncated and refined to define flood behaviour along the Bargo River downstream of Picton Weir.

A one-dimensional HEC-RAS model was also developed to estimate the dam breach hydrographs for both the Sunny Day failure and PMF failure scenarios. The breach hydrographs were then input into the refined TUFLOW hydraulic model to simulate the dam failure flood waves. The flood model results were used to estimate the Population at Risk (PAR) and Potential Loss of Life (PLL) for the scenarios assessed.

The key findings from the dam failure analysis are summarised in the following.

- During the Sunny Day failure scenario, the failure flood wave is largely confined to the Bargo River channel. The failure flood wave is predicted to inundate about 530 metres of the fire trail (Fire Rd No P1) downstream of Picton Weir.
- The potential PAR / PLL for the Sunny Day failure scenario is attributed only to the possibility for bushwalkers along the fire trail to be impacted by the failure flood wave. Campbell's method (2015) was used to estimate the itinerant PAR and PLL along the fire trail to be 0.11 and 0.09, respectively.
- During the PMF non-failure scenario, floodwaters inundate large stretches of the fire trail as well as sections of Bargo River Road, the railway north of the Bargo River, Stratford Road, Rockford / Arina Road, Charlies Point Road and Sugarloaf Creek Road. Two houses along the fire trail near the intersection with Bargo River Road are also predicted to be impacted.
- Failure of the Picton Weir during the PMF results in significant increases to flood levels and flow velocities in areas immediately downstream of the dam. However, these flood impacts decrease as the failure flood wave propagates downstream.
- The failure of Picton Weir during the PMF is not predicted to notably increase flood extents as the downstream floodplain is very incised in nature.
- During such extreme rainfall as expected in the PMF, it was assumed that there would be no itinerants using the local roads and the fire trail that are predicted to be inundated. Furthermore, the incremental increase in the inundation lengths of roads and the trail under the dam failure scenario is only minor. Therefore, the itinerant PAR / PLL during the PMF scenarios is estimated to be zero.
- The total PAR and PLL at the two houses along the fire trail during the PMF non-failure scenario is estimated to be 6.56 and 0.63, respectively.
- The total PAR and PLL at the two houses along the fire trail during the PMF failure scenario is estimated to be 7.48 and 0.64, respectively.
- Therefore, the incremental PAR and PLL for the PMF failure of the dam is estimated to be 0.92 and 0.01, respectively. The incremental PLL due to failure during the PMF is particularly low given there is no

material difference to the risk at House 1, and the flood severity at House 2 is “low” and hence a very low fatality rate is applied to the incremental PAR.

Finalisation of the Wollondilly Shire Flood Study is not expected to result in any changes to flood behaviour modelled along this stretch of the Bargo River. Accordingly, there would be no need to revisit this dam break study upon finalisation of the Flood Study for Wollondilly Shire Council.

## 6. References

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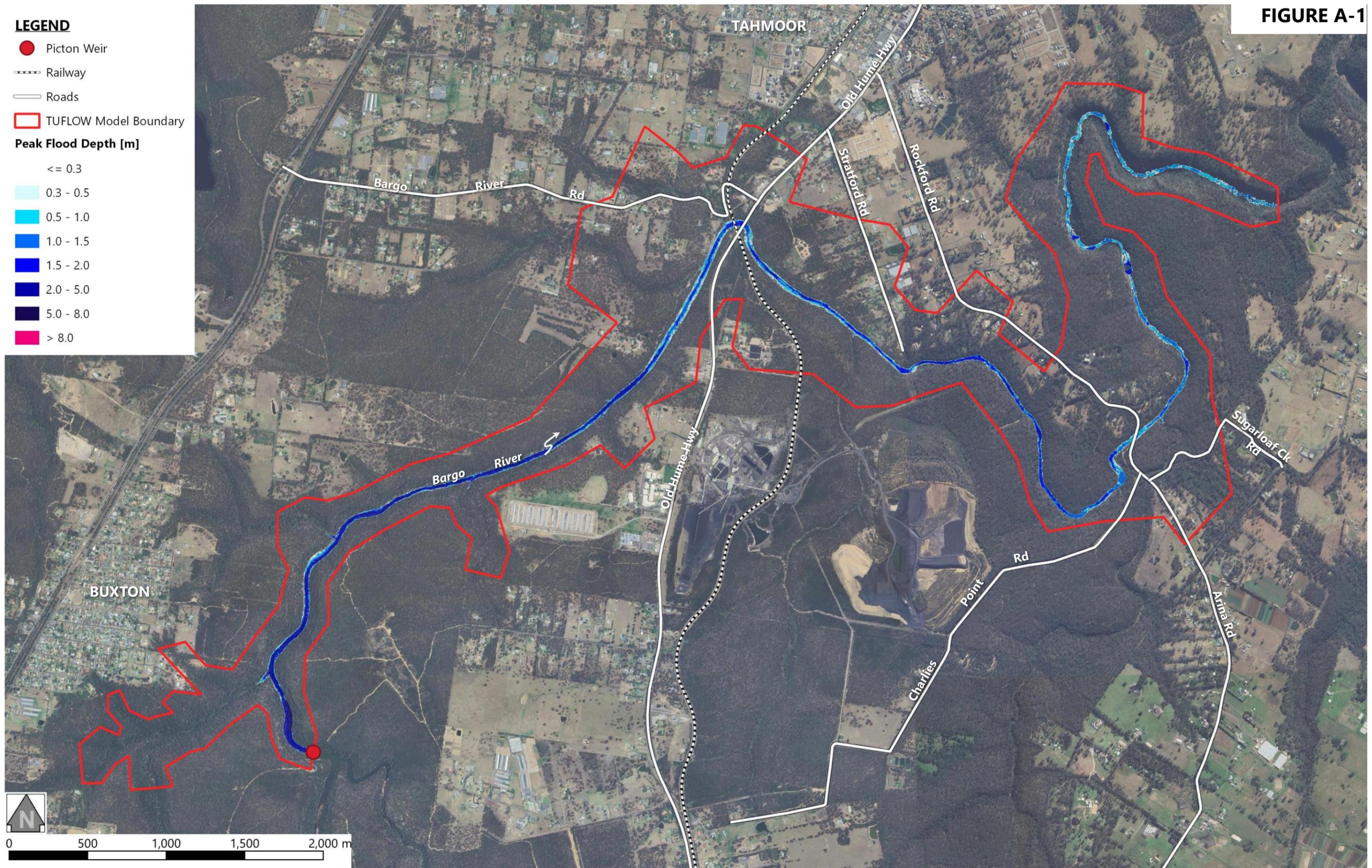
## **Appendix A. Flood Inundation Mapping**

**LEGEND**

- Picton Weir
- ⋯ Railway
- Roads
- ▭ TUFLOW Model Boundary

**Peak Flood Depth [m]**

- ≤ 0.3
- 0.3 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 5.0
- 5.0 - 8.0
- > 8.0



Prepared by:



**SUNNY DAY FAILURE SCENARIO  
PEAK FLOOD DEPTHS  
[FULL MODEL EXTENT]**

**LEGEND**

- Picton Weir
- Railway
- Roads
- TUFLOW Model Boundary

**Peak Flood Levels [mAHD]**

- ≤ 190
- 190 - 200
- 200 - 210
- 210 - 220
- 220 - 230
- 230 - 240
- 240 - 250
- 250 - 260
- 260 - 270
- 270 - 280
- 280 - 290
- > 290

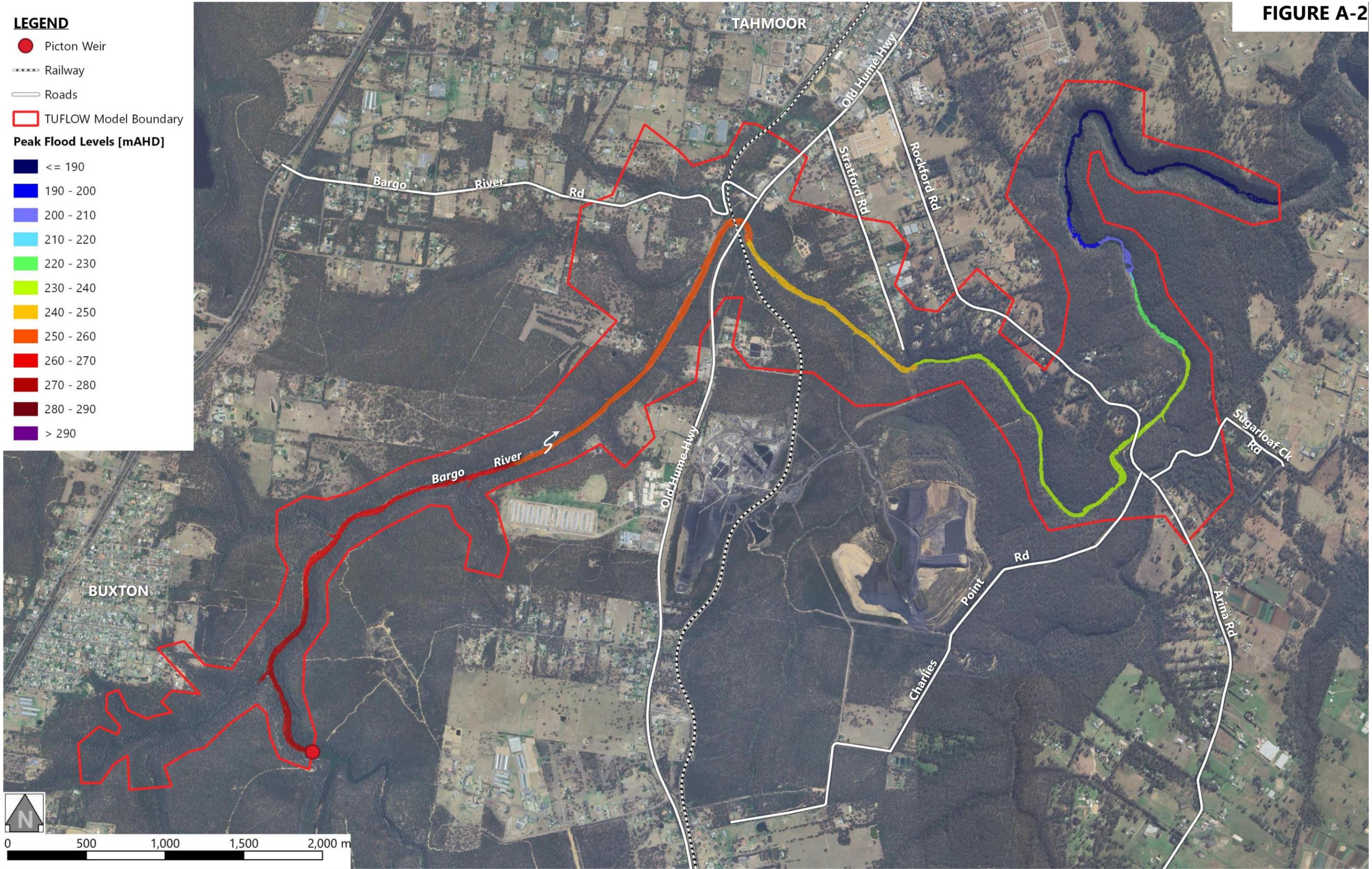


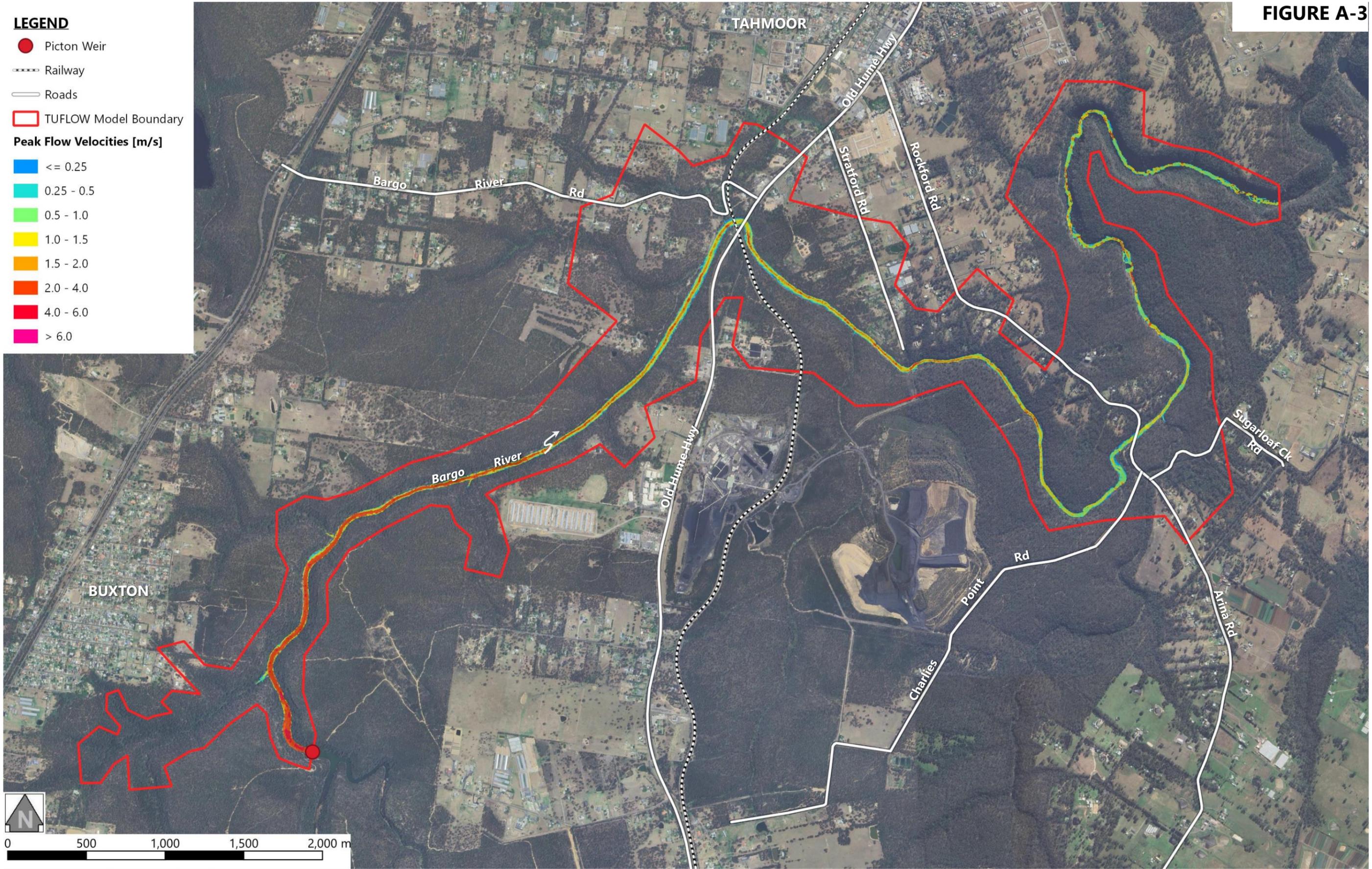
FIGURE A-3

**LEGEND**

- Picton Weir
- Railway
- Roads
- TUFLOW Model Boundary

**Peak Flow Velocities [m/s]**

- Blue:  $\leq 0.25$
- Cyan: 0.25 - 0.5
- Green: 0.5 - 1.0
- Yellow: 1.0 - 1.5
- Orange: 1.5 - 2.0
- Red: 2.0 - 4.0
- Dark Red: 4.0 - 6.0
- Magenta:  $> 6.0$



**LEGEND**

- Picton Weir
- Railway
- Roads
- TUFLOW Model Boundary

**ARR2019 Hazard Category**

- H1 - Generally safe
- H2 - Unsafe for small vehicles
- H3 - Unsafe for all vehicles, children & elderly
- H4 - Unsafe for all vehicles & people
- H5 - Unsafe, buildings vulnerable to structural damage or failure
- H6 - Unsafe, all buildings vulnerable to failure



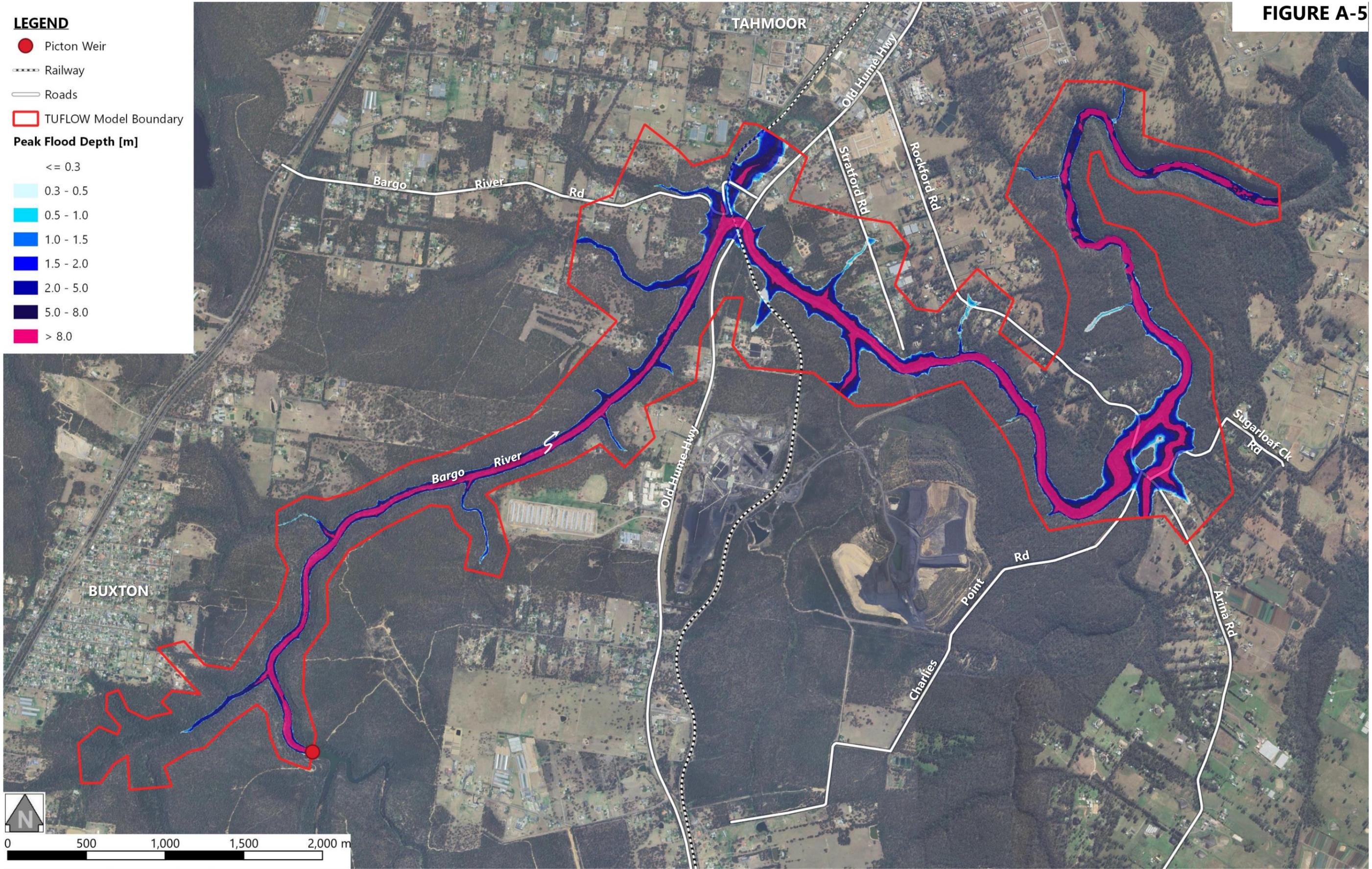
FIGURE A-5

**LEGEND**

- Picton Weir
- Railway
- Roads
- TUFLOW Model Boundary

**Peak Flood Depth [m]**

- ≤ 0.3
- 0.3 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 5.0
- 5.0 - 8.0
- > 8.0

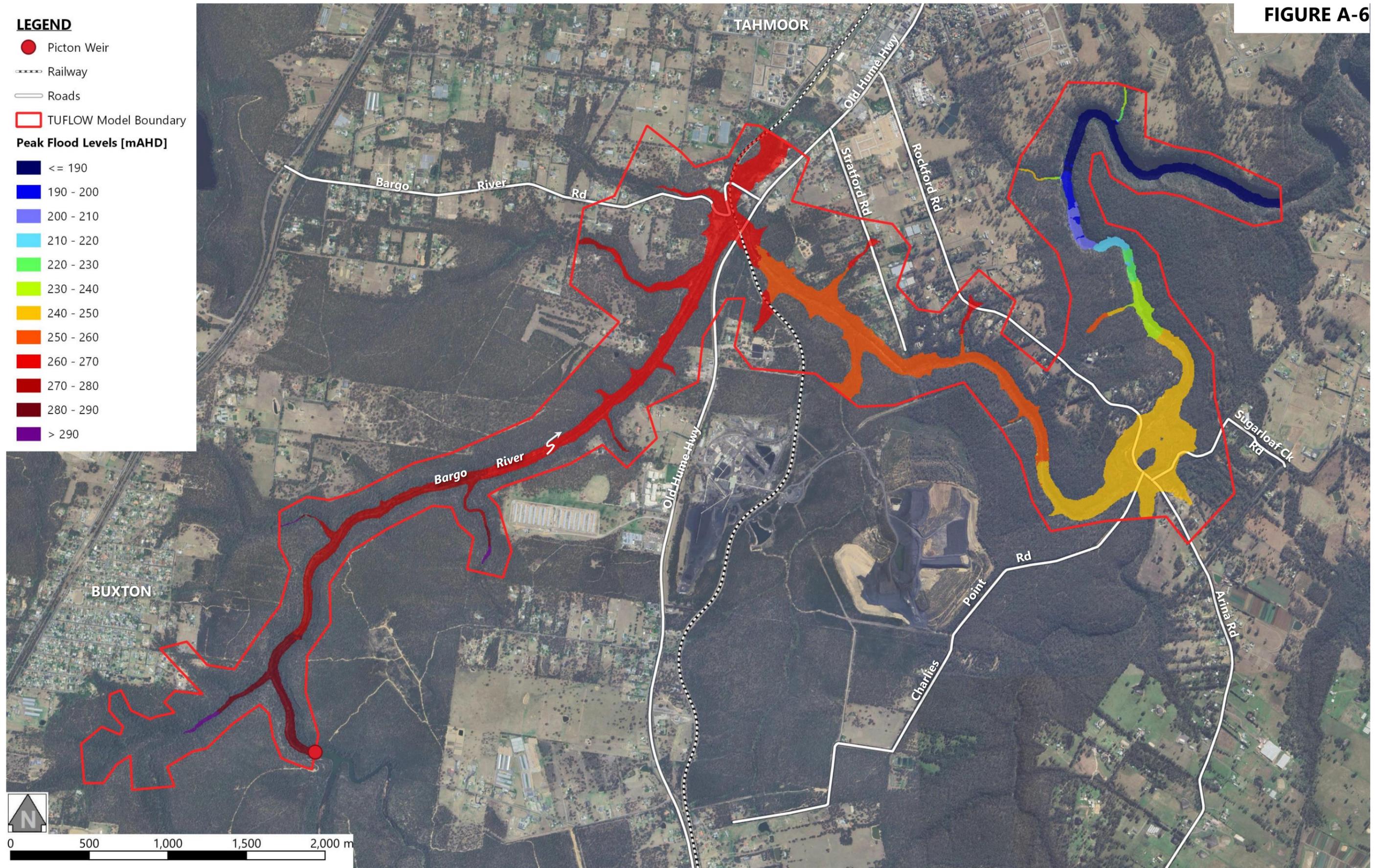


**LEGEND**

- Picton Weir
- Railway
- Roads
- TUFLOW Model Boundary

**Peak Flood Levels [mAHD]**

- ≤ 190
- 190 - 200
- 200 - 210
- 210 - 220
- 220 - 230
- 230 - 240
- 240 - 250
- 250 - 260
- 260 - 270
- 270 - 280
- 280 - 290
- > 290



Prepared by:



**PMF NON-FAILURE SCENARIO  
PEAK FLOOD LEVELS  
[FULL MODEL EXTENT]**

FIGURE A-7

**LEGEND**

- Picton Weir
- ⋯ Railway
- Roads
- ▭ TUFLOW Model Boundary

**Peak Flow Velocities [m/s]**

- Blue:  $\leq 0.25$
- Cyan: 0.25 - 0.5
- Green: 0.5 - 1.0
- Yellow: 1.0 - 1.5
- Orange: 1.5 - 2.0
- Red-Orange: 2.0 - 4.0
- Red: 4.0 - 6.0
- Magenta:  $> 6.0$



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**PMF NON-FAILURE SCENARIO  
PEAK FLOW VELOCITIES  
[FULL MODEL EXTENT]**

**LEGEND**

- Picton Weir
- Railway
- Roads
- TUFLOW Model Boundary

**ARR2019 Hazard Category**

- H1 - Generally safe
- H2 - Unsafe for small vehicles
- H3 - Unsafe for all vehicles, children & elderly
- H4 - Unsafe for all vehicles & people
- H5 - Unsafe, buildings vulnerable to structural damage or failure
- H6 - Unsafe, all buildings vulnerable to failure



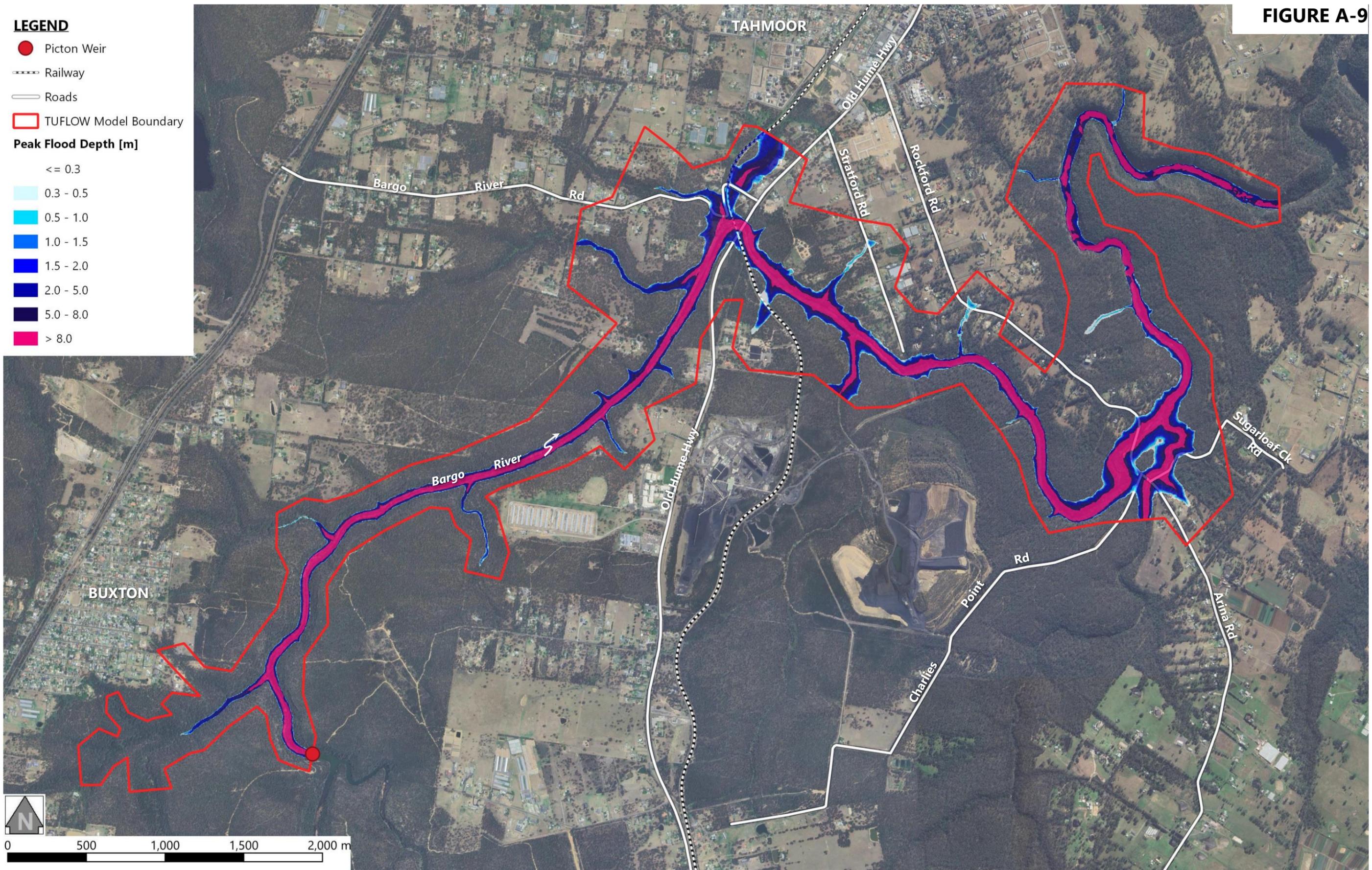
FIGURE A-9

**LEGEND**

- Picton Weir
- Railway
- Roads
- TUFLOW Model Boundary

**Peak Flood Depth [m]**

- ≤ 0.3
- 0.3 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 5.0
- 5.0 - 8.0
- > 8.0



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**PMF FAILURE SCENARIO  
PEAK FLOOD DEPTHS  
[FULL MODEL EXTENT]**

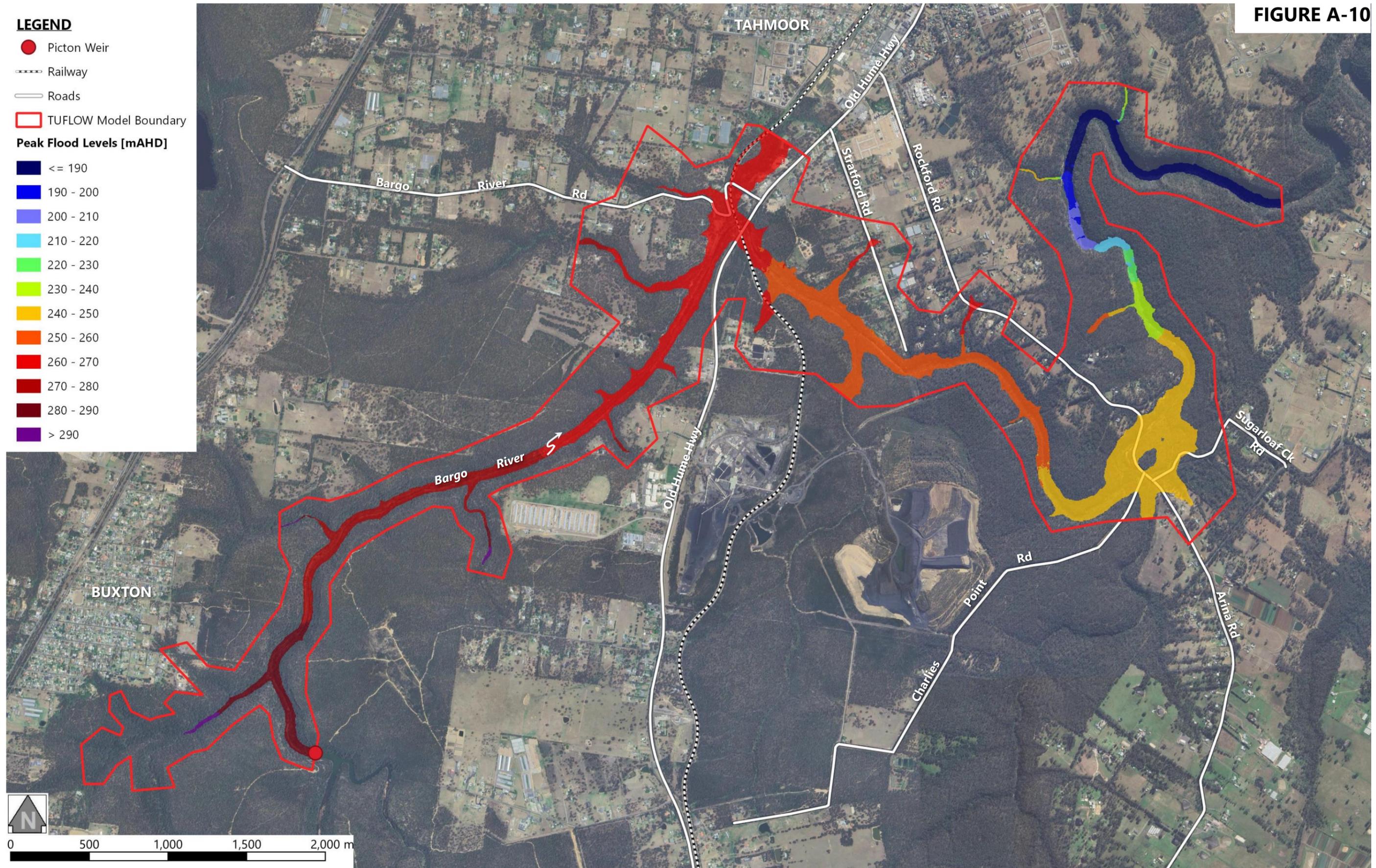
FIGURE A-10

**LEGEND**

- Picton Weir
- Railway
- Roads
- TUFLOW Model Boundary

**Peak Flood Levels [mAHD]**

- ≤ 190
- 190 - 200
- 200 - 210
- 210 - 220
- 220 - 230
- 230 - 240
- 240 - 250
- 250 - 260
- 260 - 270
- 270 - 280
- 280 - 290
- > 290



Prepared by:



**PMF FAILURE SCENARIO  
PEAK FLOOD LEVELS  
[FULL MODEL EXTENT]**

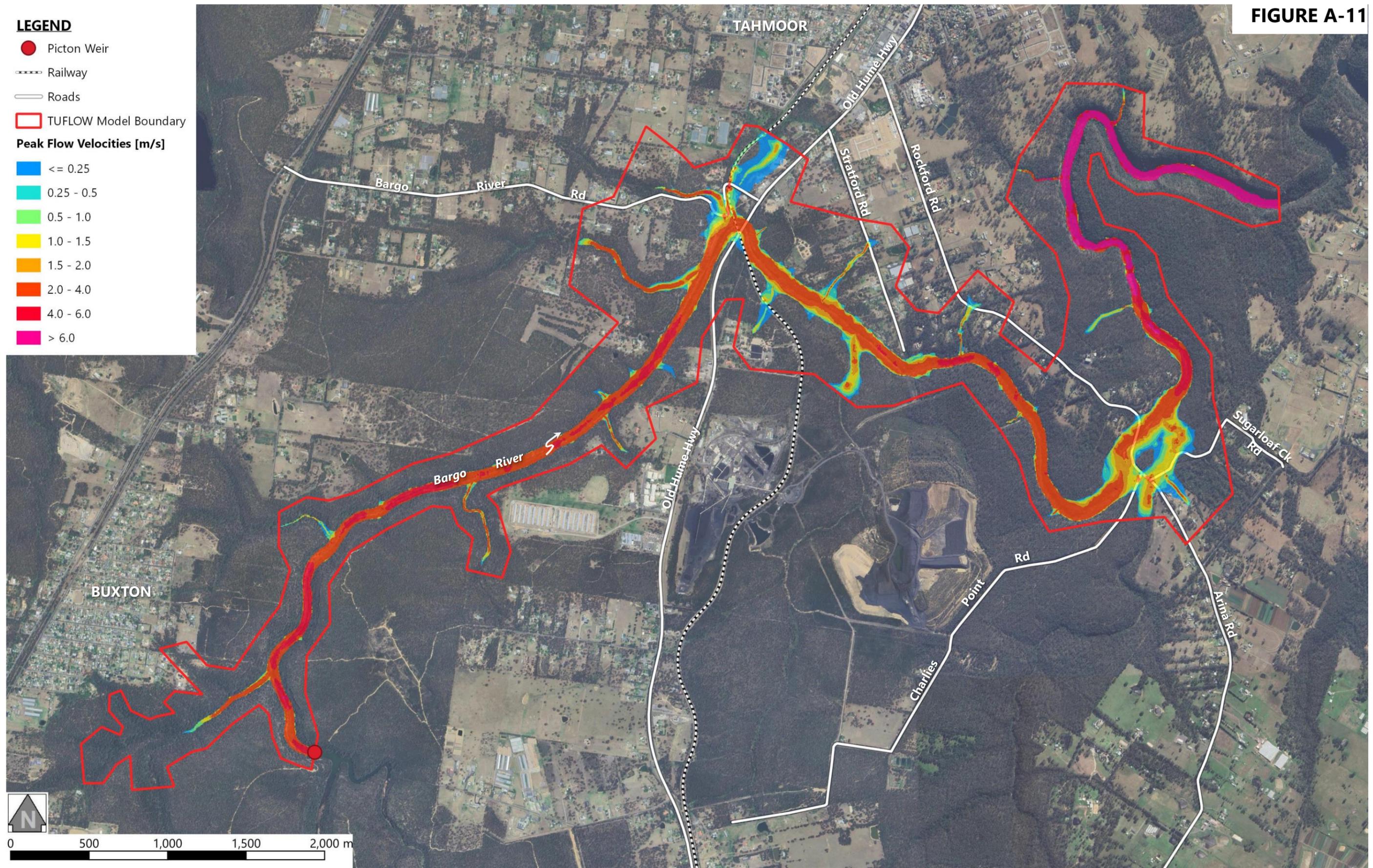
FIGURE A-11

**LEGEND**

- Picton Weir
- Railway
- Roads
- TUFLOW Model Boundary

**Peak Flow Velocities [m/s]**

- Blue:  $\leq 0.25$
- Cyan: 0.25 - 0.5
- Green: 0.5 - 1.0
- Yellow: 1.0 - 1.5
- Orange: 1.5 - 2.0
- Red-Orange: 2.0 - 4.0
- Red: 4.0 - 6.0
- Magenta:  $> 6.0$



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**PMF FAILURE SCENARIO  
PEAK FLOW VELOCITIES  
[FULL MODEL EXTENT]**

FIGURE A-12

**LEGEND**

-  Picton Weir
-  Railway
-  Roads

 TUFLOW Model Boundary

**ARR2019 Hazard Category**

-  H1 - Generally safe
-  H2 - Unsafe for small vehicles
-  H3 - Unsafe for all vehicles, children & elderly
-  H4 - Unsafe for all vehicles & people
-  H5 - Unsafe, buildings vulnerable to structural damage or failure
-  H6 - Unsafe, all buildings vulnerable to failure



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**PMF FAILURE SCENARIO  
ARR 2019 HAZARD CATEGORIES  
[FULL MODEL EXTENT]**

FIGURE A-13

**LEGEND**

-  Railway
-  Roads
- Peak Flood Depth [m]**
-  <= 0.3
-  0.3 - 0.5
-  0.5 - 1.0
-  1.0 - 1.5
-  1.5 - 2.0
-  2.0 - 5.0
-  5.0 - 8.0
-  > 8.0

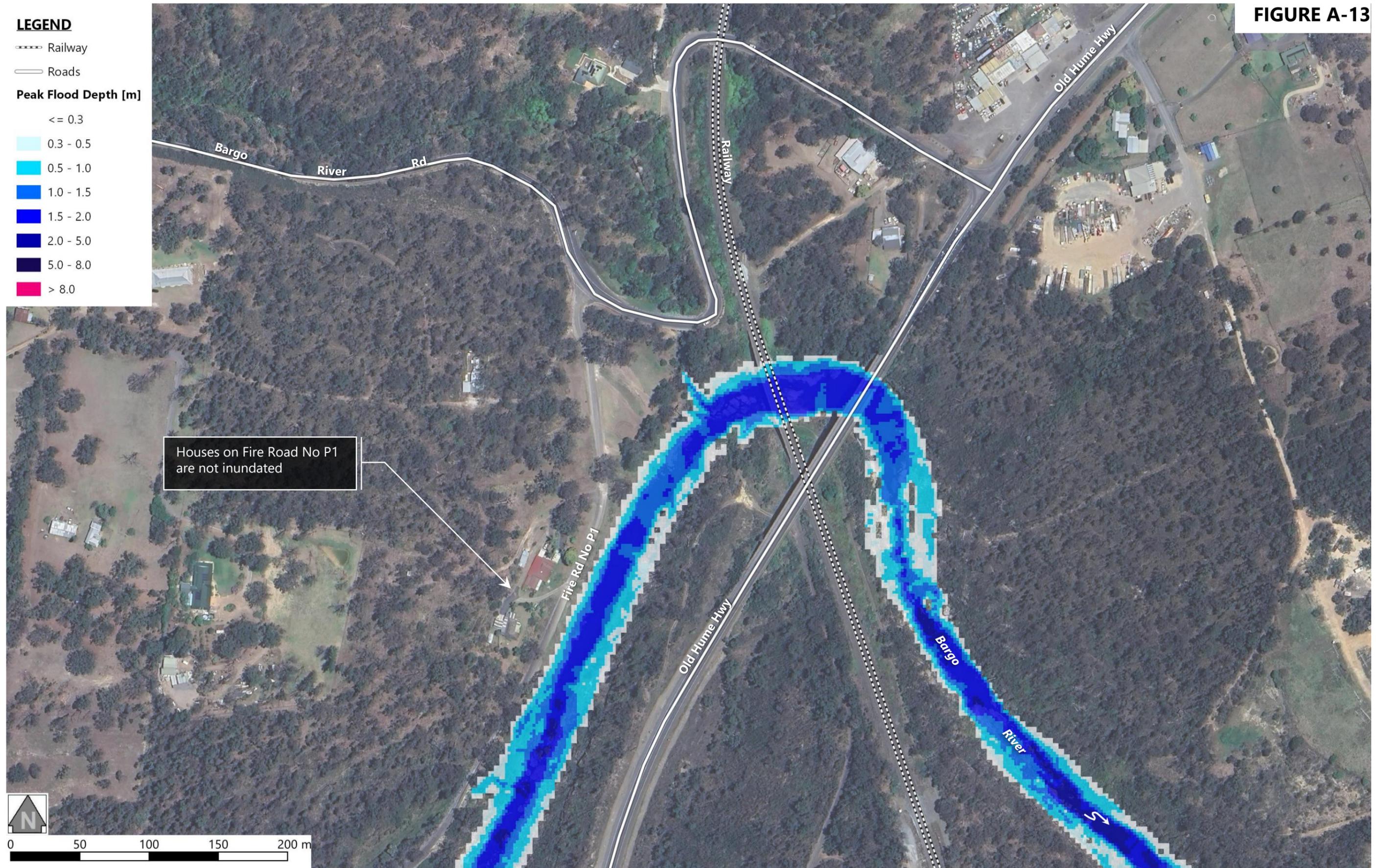


FIGURE A-14

**LEGEND**

--- Railway

— Roads

**Peak Flood Levels [mAHD]**

- <= 245
- 245 - 247
- 247 - 249
- 249 - 251
- 251 - 253
- 253 - 255
- 255 - 257
- 257 - 259
- 259 - 261
- 261 - 263
- 263 - 265
- > 265



Prepared by:

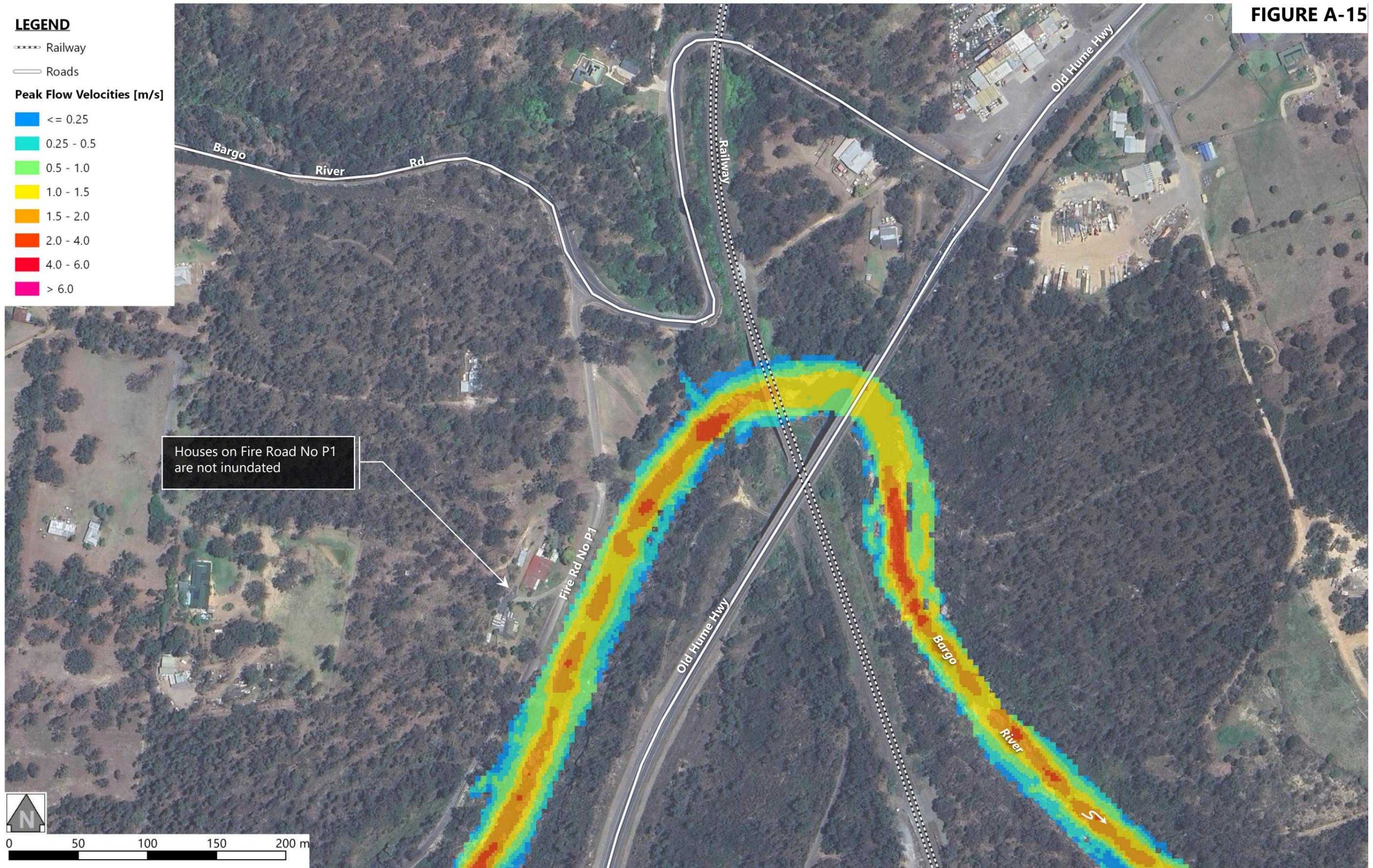


**SUNNY DAY FAILURE SCENARIO  
PEAK FLOOD LEVELS  
[HOUSES ON FIRE ROAD NO P1]**

FIGURE A-15

**LEGEND**

-  Railway
-  Roads
- Peak Flow Velocities [m/s]**
-   $\leq 0.25$
-  0.25 - 0.5
-  0.5 - 1.0
-  1.0 - 1.5
-  1.5 - 2.0
-  2.0 - 4.0
-  4.0 - 6.0
-   $> 6.0$



Prepared by:



**SUNNY DAY FAILURE SCENARIO  
PEAK FLOW VELOCITIES  
[HOUSES ON FIRE ROAD NO P1]**

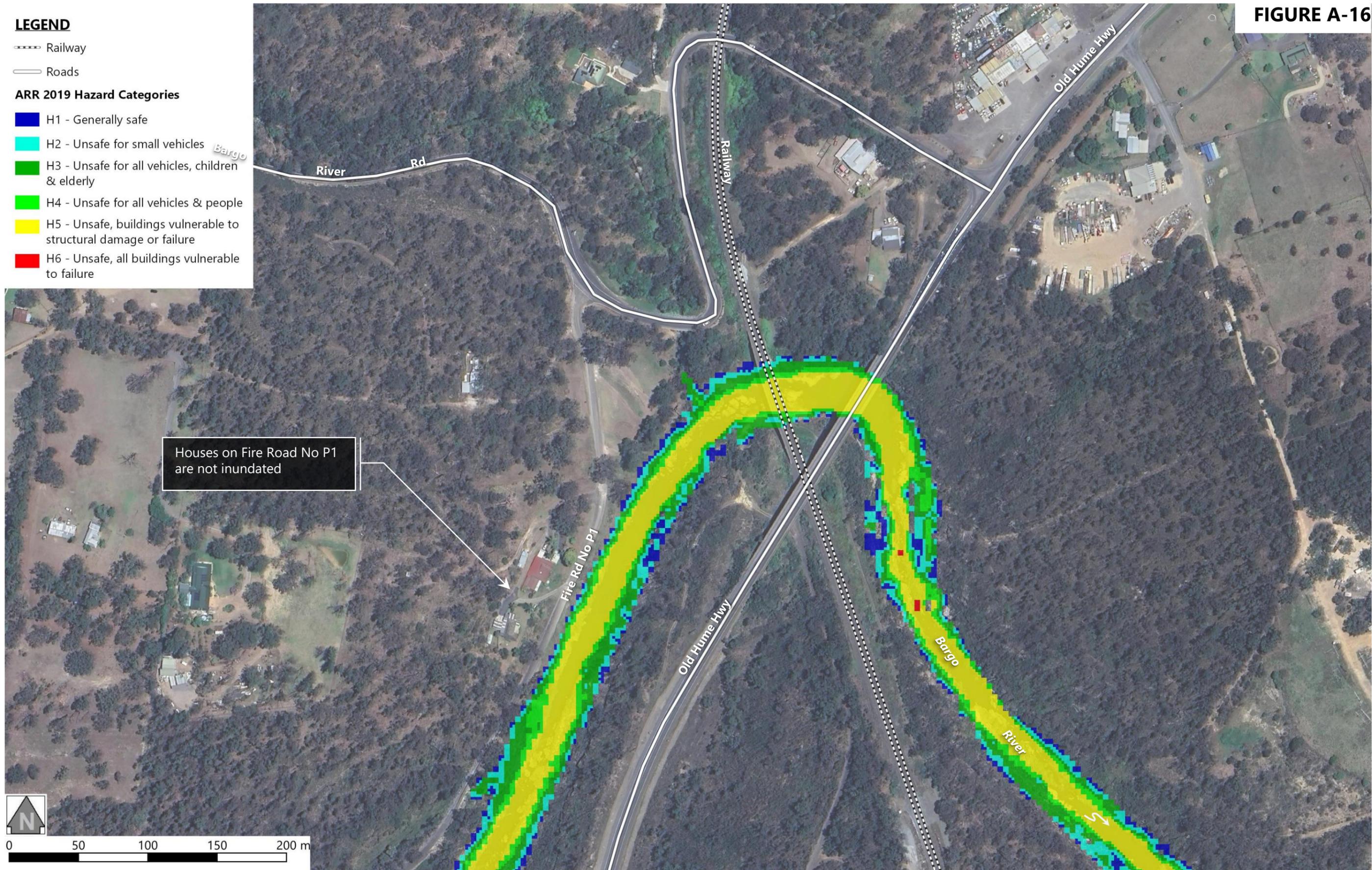
**LEGEND**

--- Railway

— Roads

**ARR 2019 Hazard Categories**

- H1 - Generally safe
- H2 - Unsafe for small vehicles
- H3 - Unsafe for all vehicles, children & elderly
- H4 - Unsafe for all vehicles & people
- H5 - Unsafe, buildings vulnerable to structural damage or failure
- H6 - Unsafe, all buildings vulnerable to failure



Prepared by:



**SUNNY DAY FAILURE SCENARIO  
ARR 2019 HAZARD CATEGORIES  
[HOUSES ON FIRE ROAD NO P1]**

FIGURE A-17

- LEGEND**
- Railway
  - Roads
- PAR Criteria [QLD DNRME]**
- PAR not counted in this area
  - PAR counted in this area

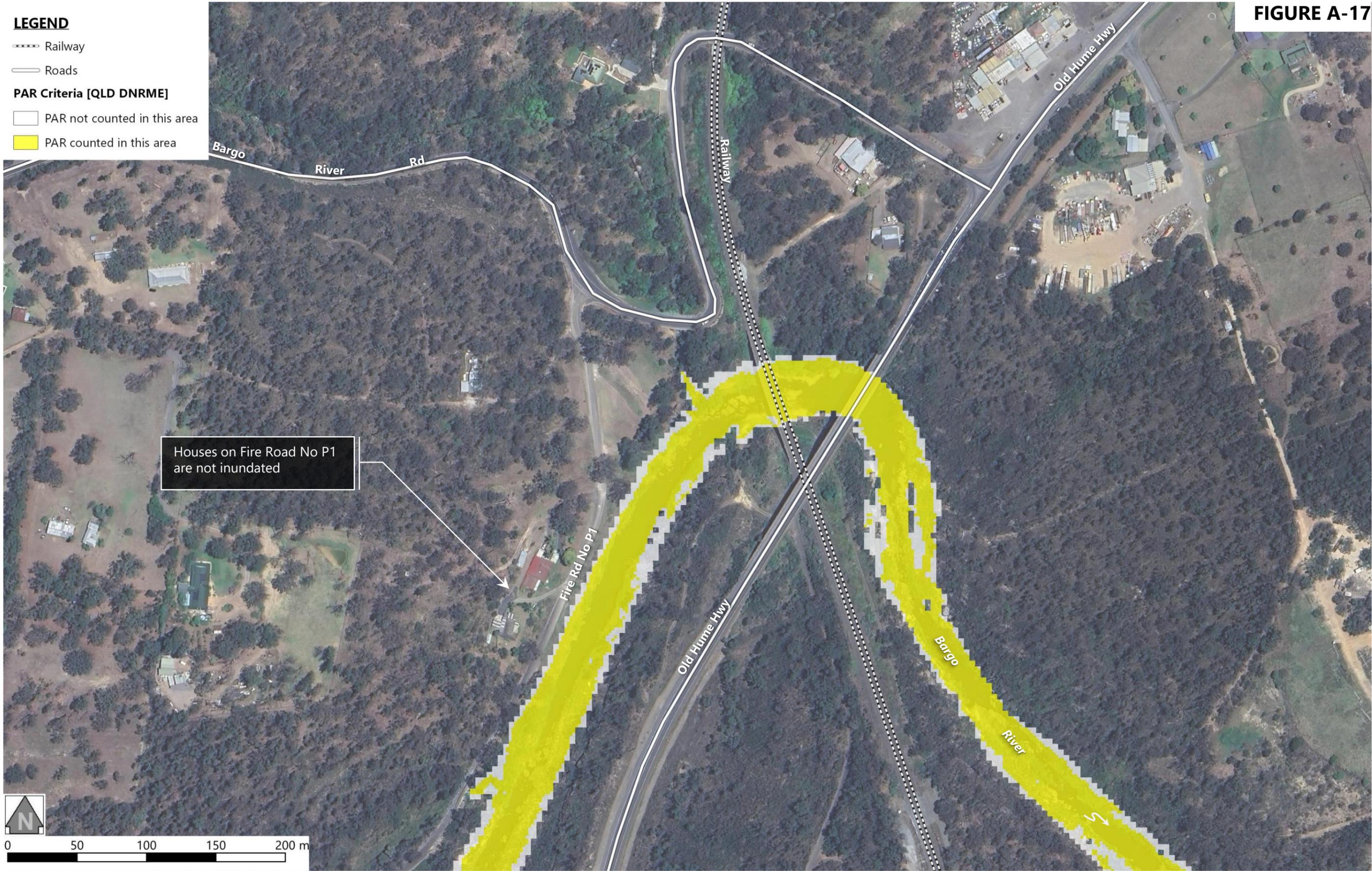


FIGURE A-18

**LEGEND**

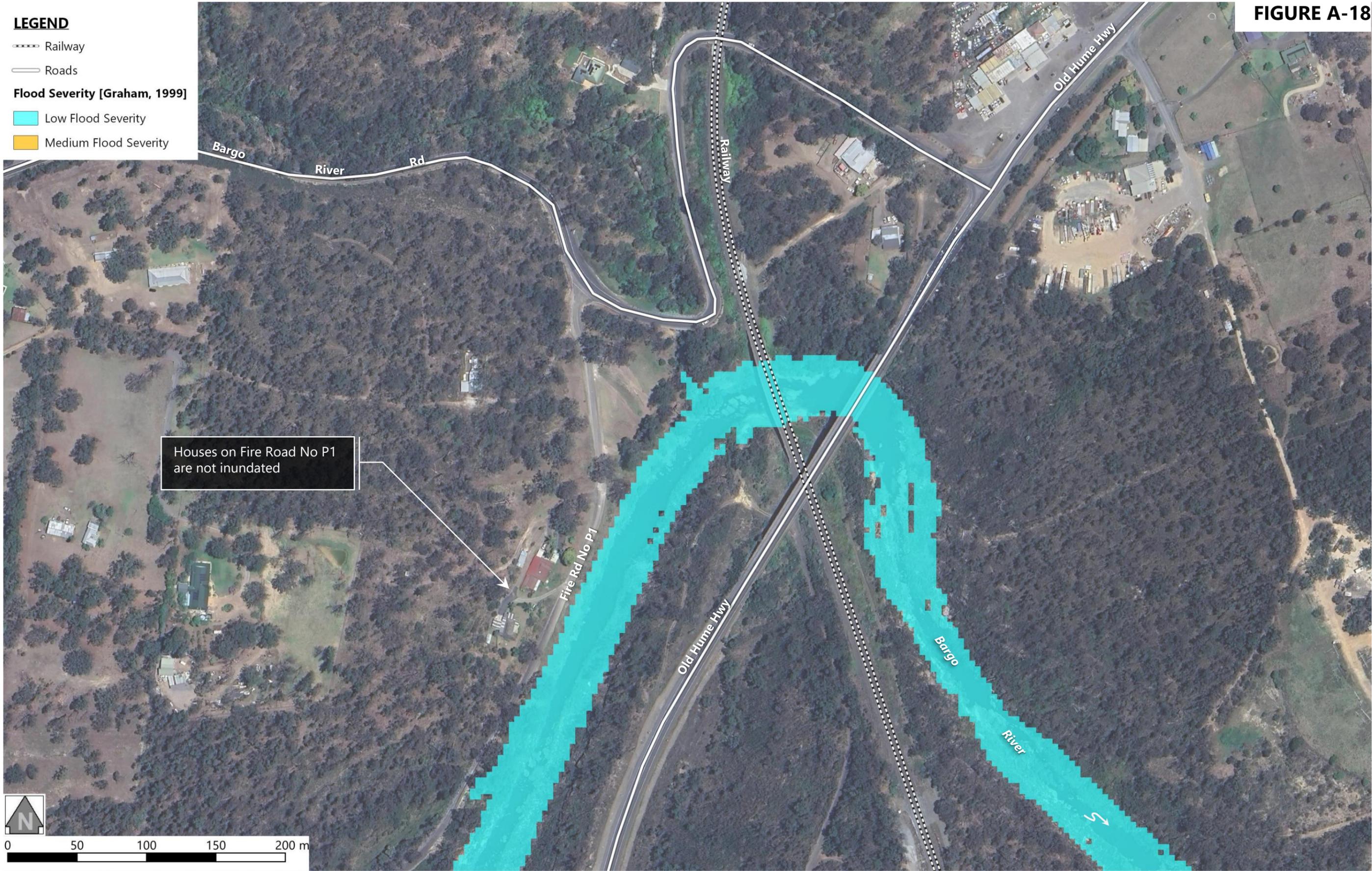
--- Railway

— Roads

**Flood Severity [Graham, 1999]**

Low Flood Severity

Medium Flood Severity



Prepared by:

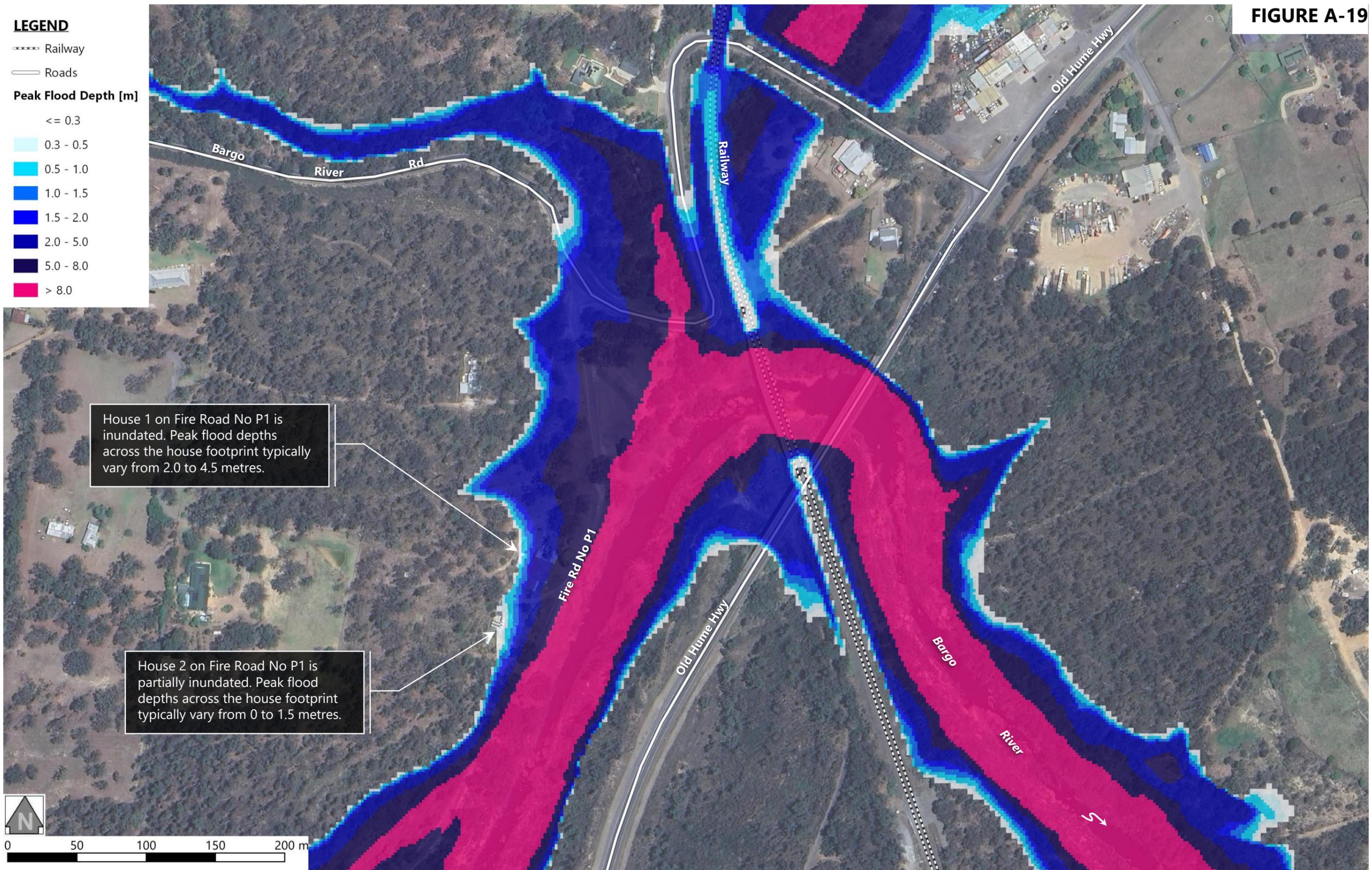


**SUNNY DAY FAILURE SCENARIO  
GRAHAM 1999 FLOOD SEVERITY  
[HOUSES ON FIRE ROAD NO P1]**

FIGURE A-19

**LEGEND**

-  Railway
-  Roads
- Peak Flood Depth [m]**
-  <= 0.3
-  0.3 - 0.5
-  0.5 - 1.0
-  1.0 - 1.5
-  1.5 - 2.0
-  2.0 - 5.0
-  5.0 - 8.0
-  > 8.0



Prepared by:

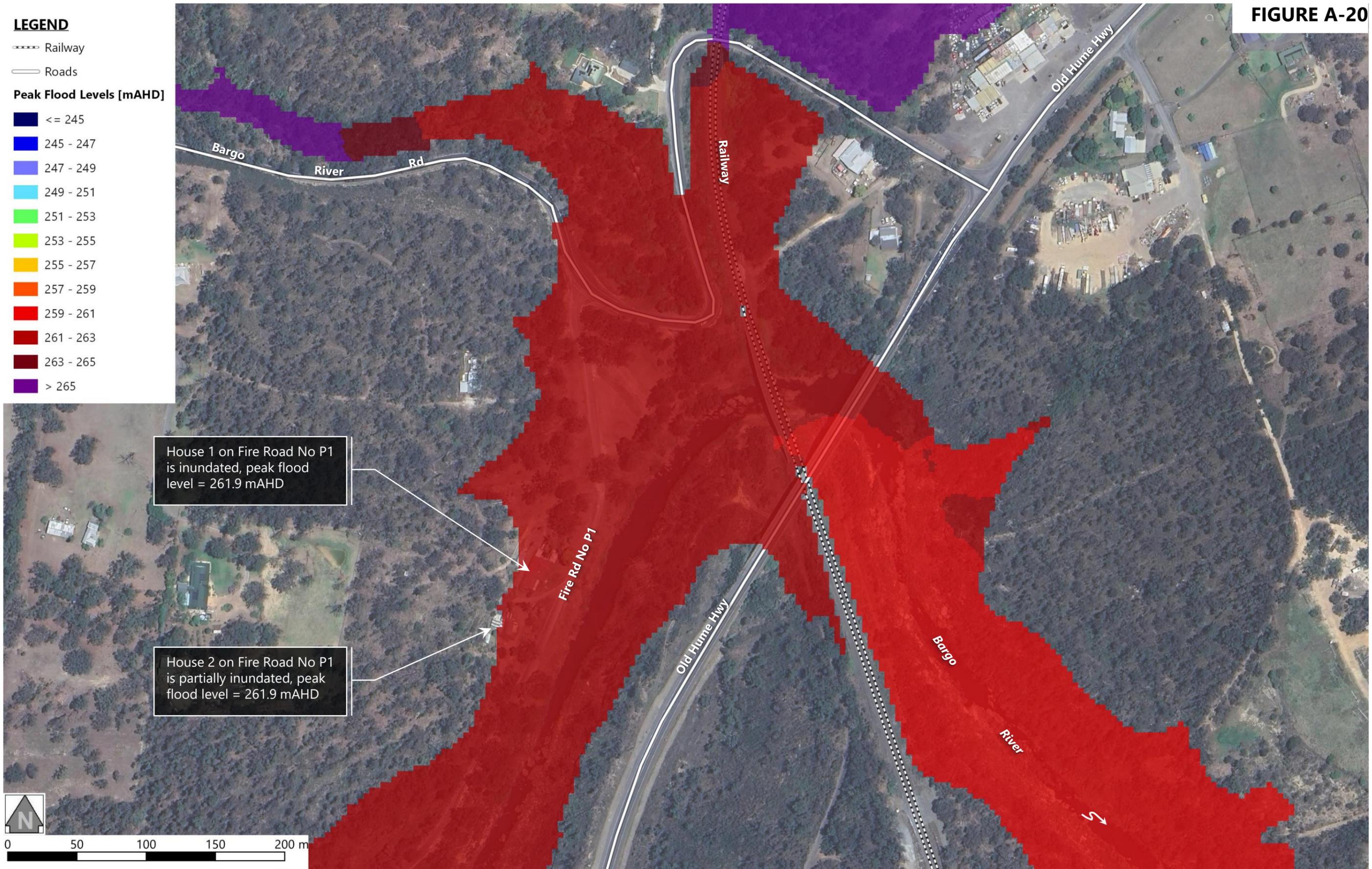


**PMF NON-FAILURE SCENARIO  
PEAK FLOOD DEPTHS  
[HOUSES ON FIRE ROAD NO P1]**

FIGURE A-20

**LEGEND**

-  Railway
-  Roads
- Peak Flood Levels [mAHD]**
-  ≤ 245
-  245 - 247
-  247 - 249
-  249 - 251
-  251 - 253
-  253 - 255
-  255 - 257
-  257 - 259
-  259 - 261
-  261 - 263
-  263 - 265
-  > 265



Prepared by:

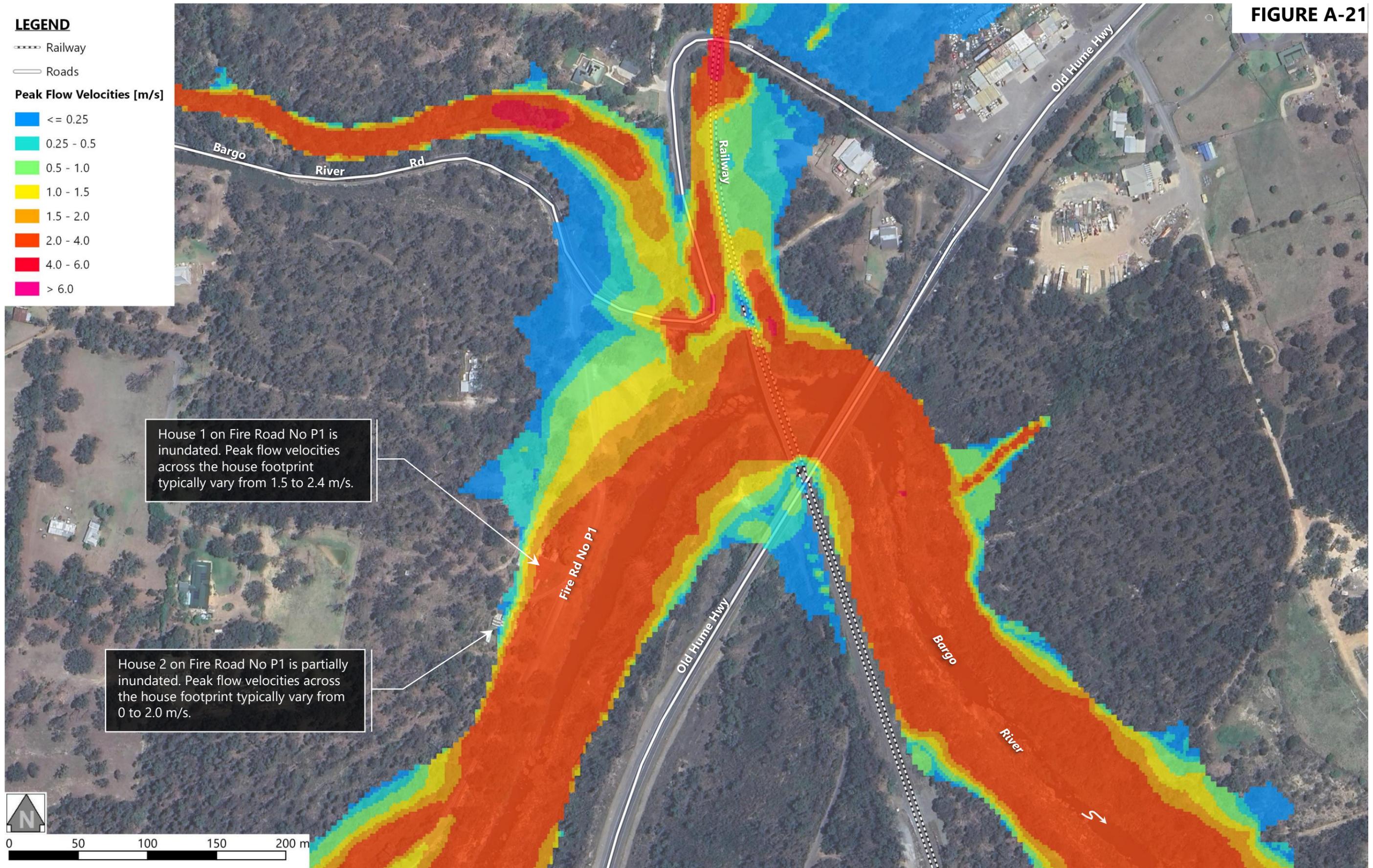


**PMF NON-FAILURE SCENARIO  
PEAK FLOOD LEVELS  
[HOUSES ON FIRE ROAD NO P1]**

FIGURE A-21

**LEGEND**

-  Railway
-  Roads
- Peak Flow Velocities [m/s]**
-  ≤ 0.25
-  0.25 - 0.5
-  0.5 - 1.0
-  1.0 - 1.5
-  1.5 - 2.0
-  2.0 - 4.0
-  4.0 - 6.0
-  > 6.0



House 1 on Fire Road No P1 is inundated. Peak flow velocities across the house footprint typically vary from 1.5 to 2.4 m/s.

House 2 on Fire Road No P1 is partially inundated. Peak flow velocities across the house footprint typically vary from 0 to 2.0 m/s.

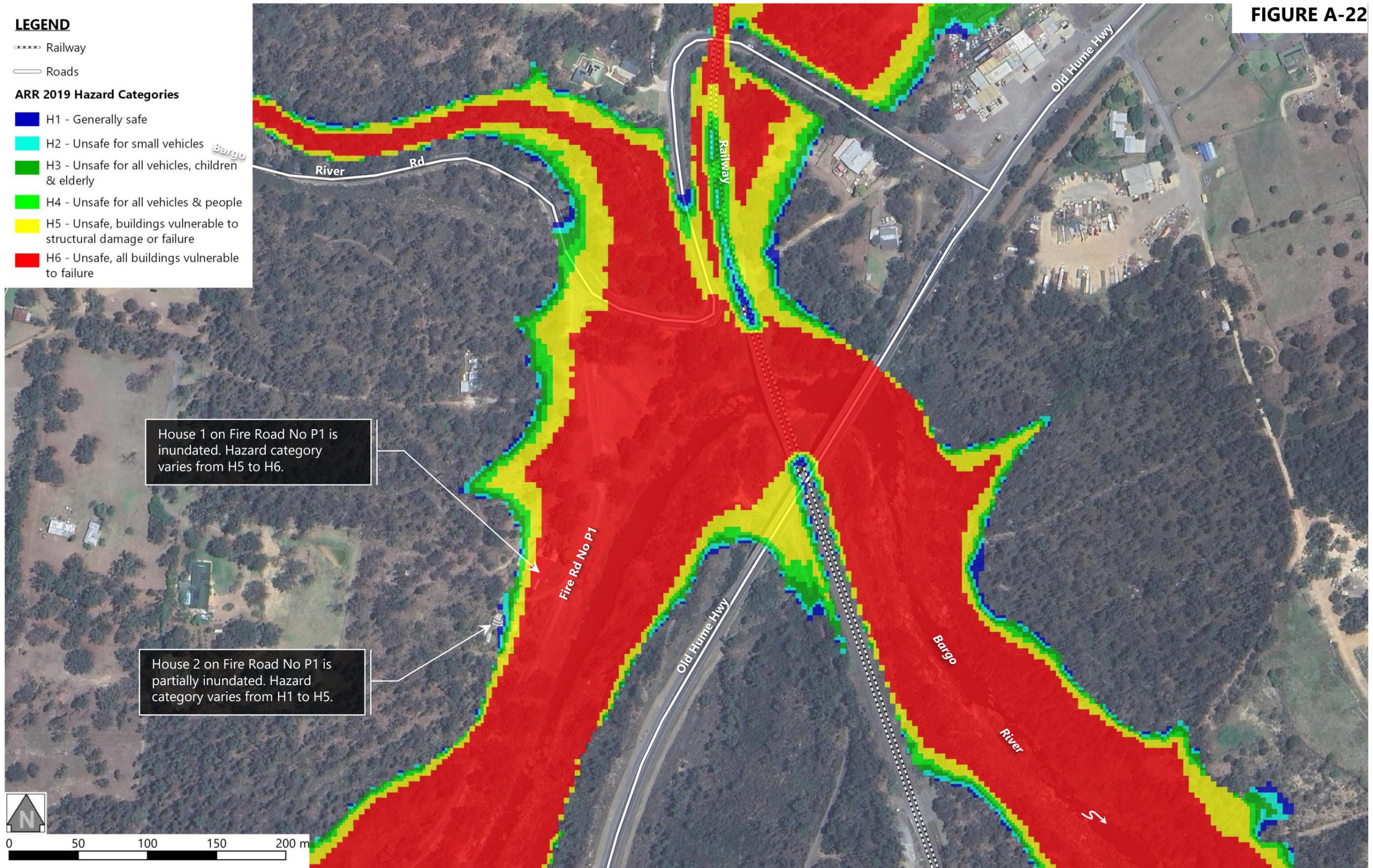
**LEGEND**

--- Railway

— Roads

**ARR 2019 Hazard Categories**

- H1 - Generally safe
- H2 - Unsafe for small vehicles
- H3 - Unsafe for all vehicles, children & elderly
- H4 - Unsafe for all vehicles & people
- H5 - Unsafe, buildings vulnerable to structural damage or failure
- H6 - Unsafe, all buildings vulnerable to failure



Prepared by:



**PMF NON-FAILURE SCENARIO  
ARR 2019 HAZARD CATEGORIES  
[HOUSES ON FIRE ROAD NO P1]**

FIGURE A-23

**LEGEND**

- Railway
- Roads

**PAR Criteria [QLD DNRME]**

- PAR not counted in this area
- PAR counted in this area

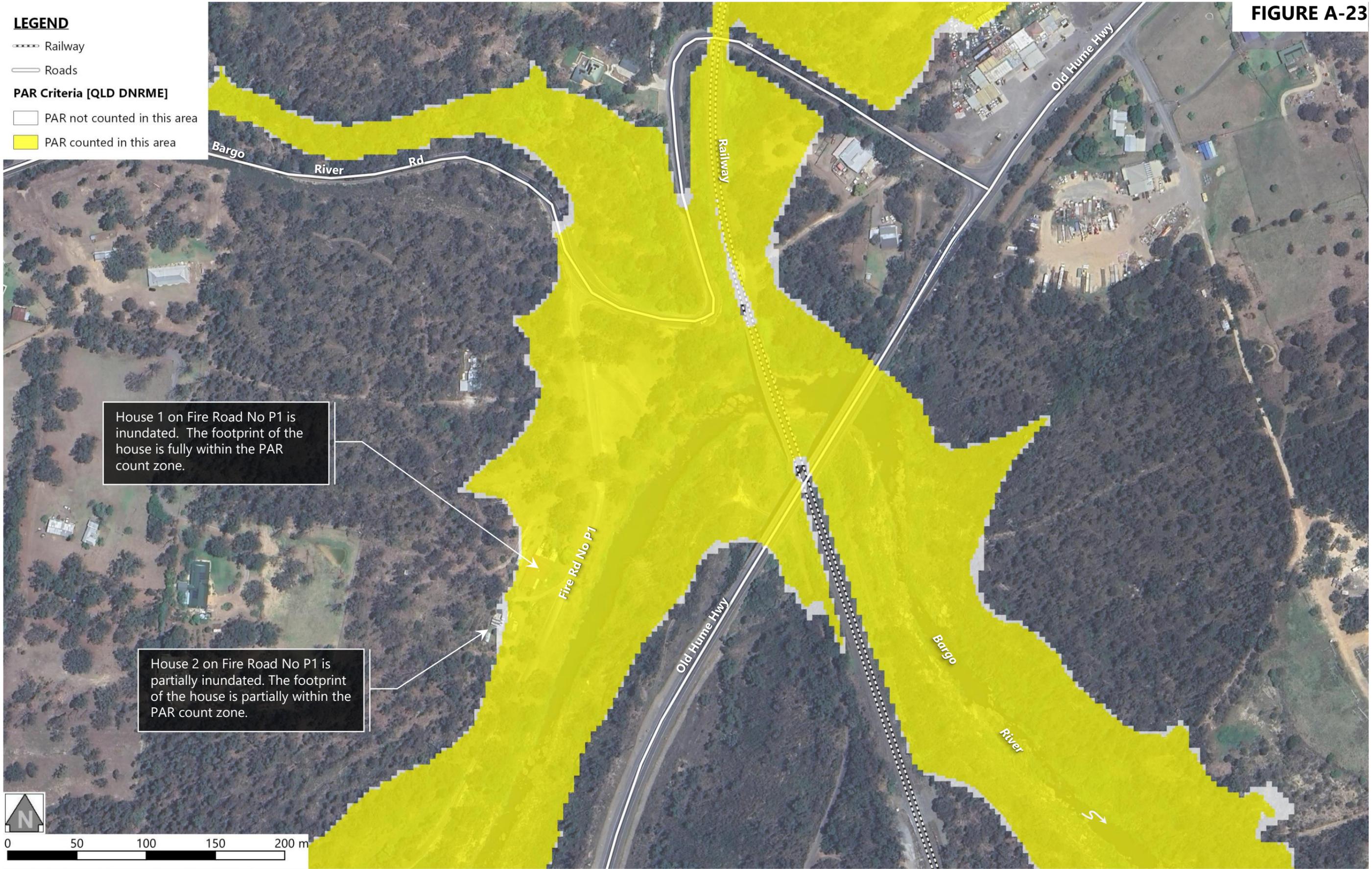


FIGURE A-24

**LEGEND**

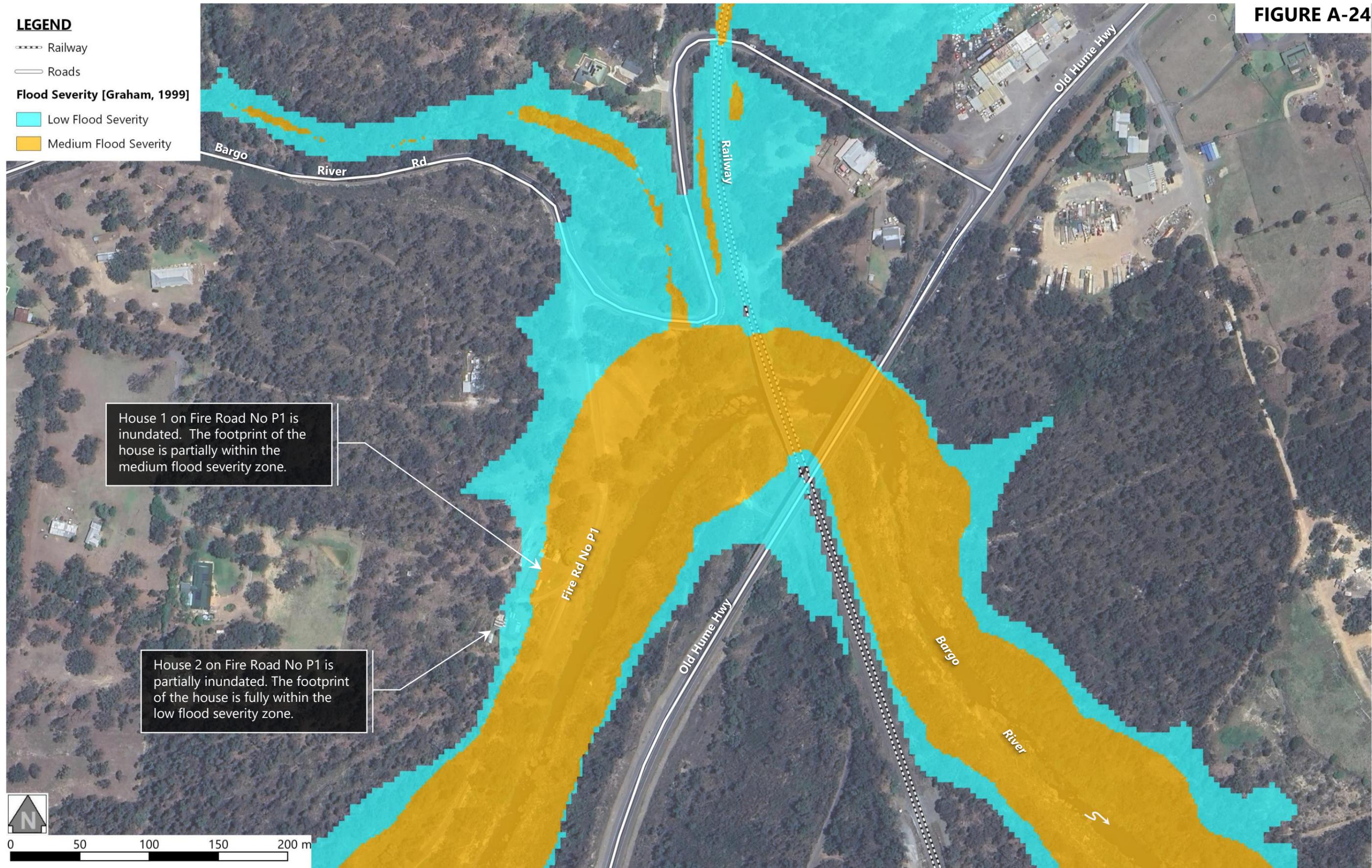
--- Railway

— Roads

**Flood Severity [Graham, 1999]**

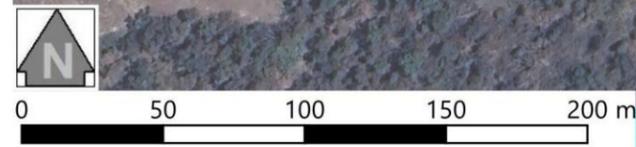
Low Flood Severity

Medium Flood Severity



House 1 on Fire Road No P1 is inundated. The footprint of the house is partially within the medium flood severity zone.

House 2 on Fire Road No P1 is partially inundated. The footprint of the house is fully within the low flood severity zone.



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**PMF NON-FAILURE SCENARIO  
GRAHAM 1999 FLOOD SEVERITY  
[HOUSES ON FIRE ROAD NO P1]**

FIGURE A-25

**LEGEND**

-  Railway
-  Roads
- Peak Flood Depth [m]**
-  <= 0.3
-  0.3 - 0.5
-  0.5 - 1.0
-  1.0 - 1.5
-  1.5 - 2.0
-  2.0 - 5.0
-  5.0 - 8.0
-  > 8.0

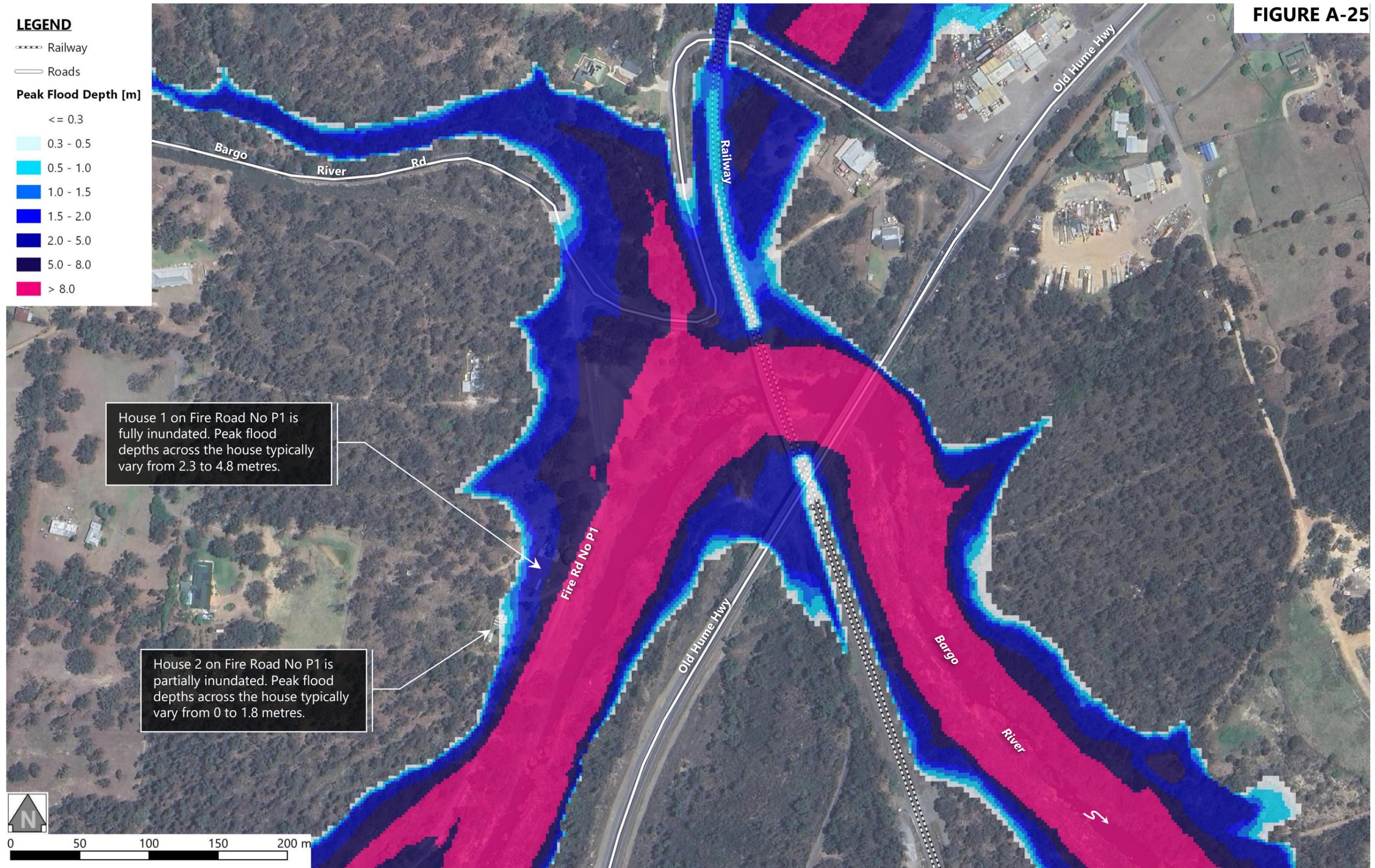
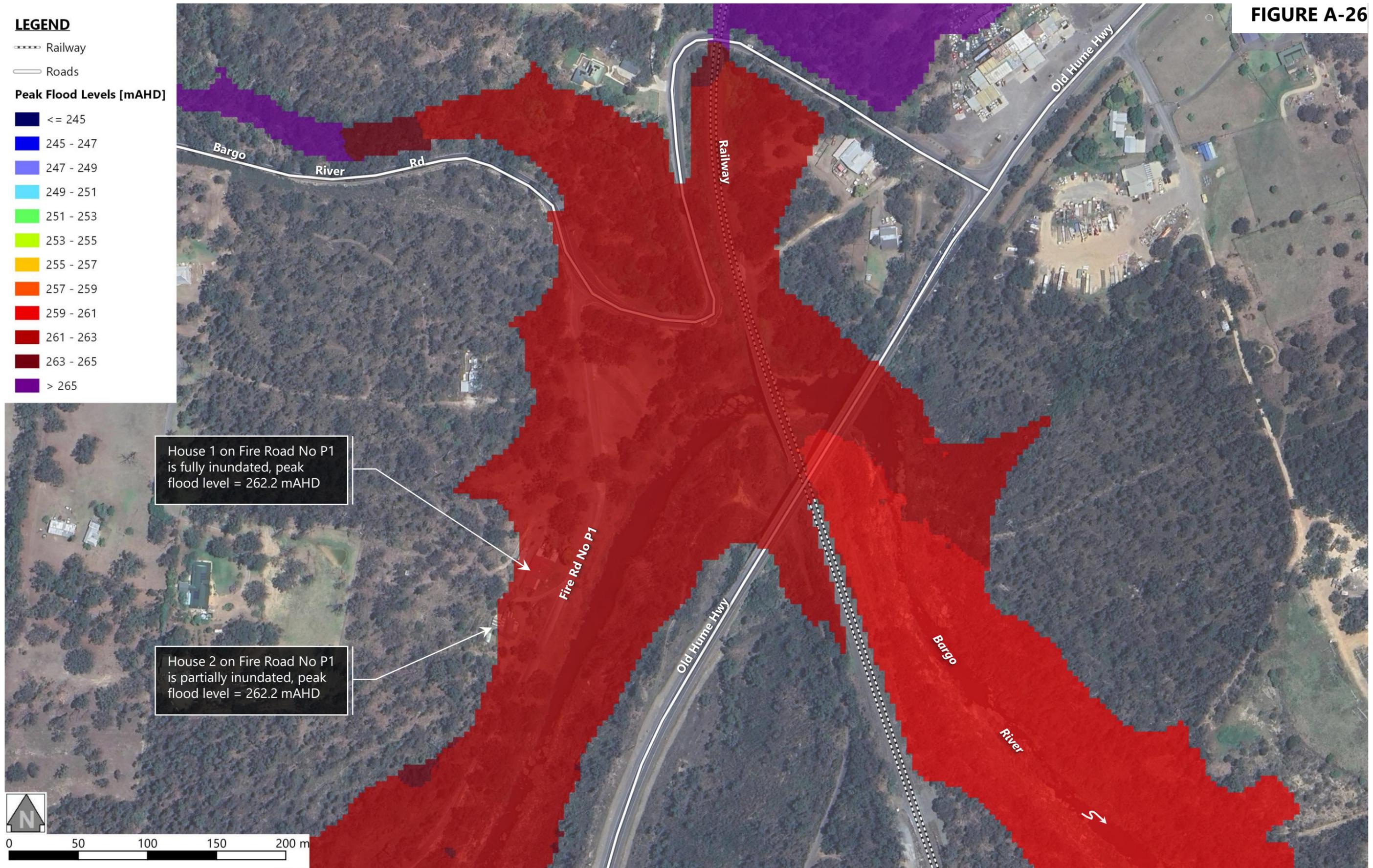


FIGURE A-26

**LEGEND**

-  Railway
-  Roads
- Peak Flood Levels [mAHD]**
-  <= 245
-  245 - 247
-  247 - 249
-  249 - 251
-  251 - 253
-  253 - 255
-  255 - 257
-  257 - 259
-  259 - 261
-  261 - 263
-  263 - 265
-  > 265



Prepared by:

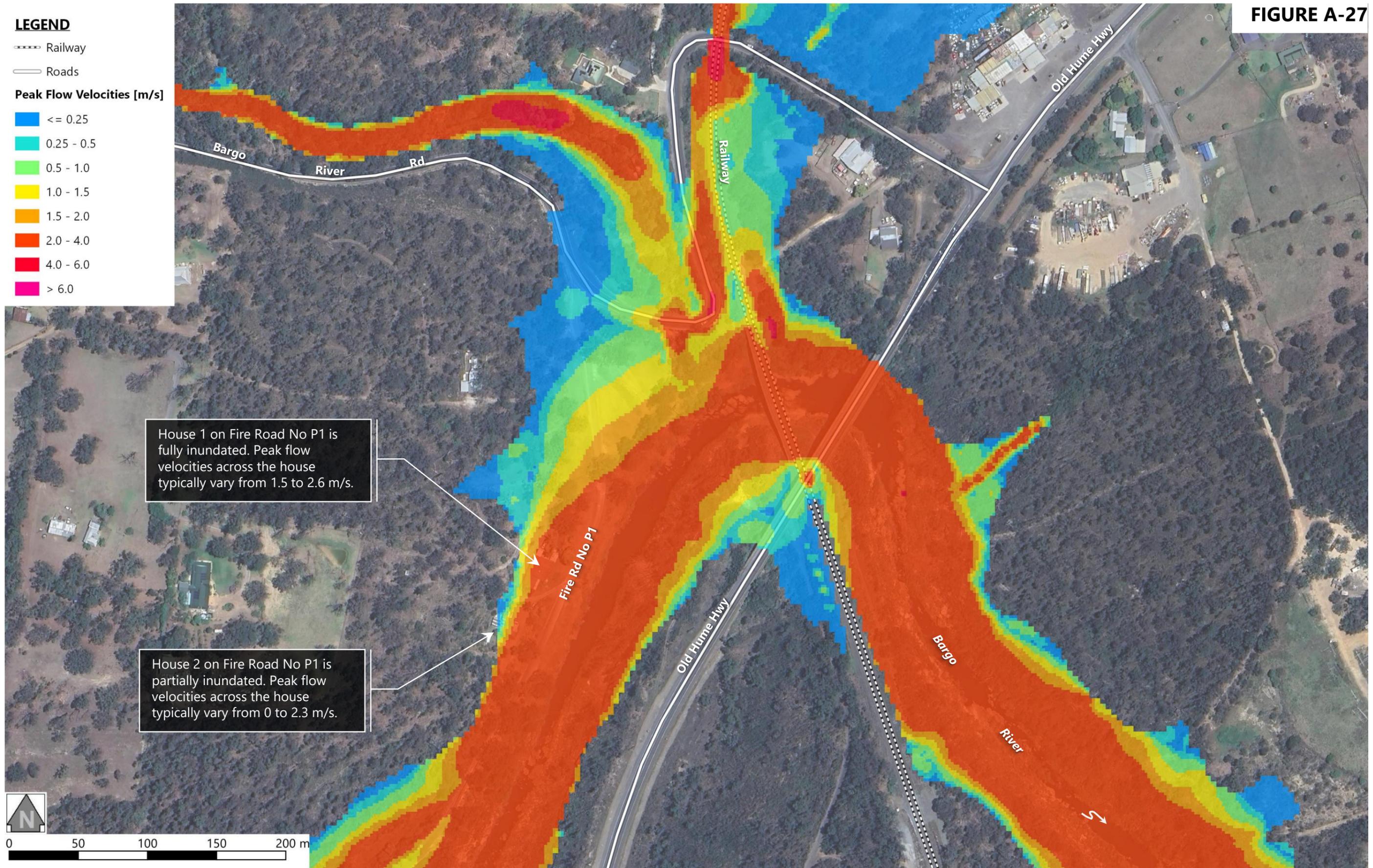


**PMF FAILURE SCENARIO  
PEAK FLOOD LEVELS  
[HOUSES ON FIRE ROAD NO P1]**

FIGURE A-27

**LEGEND**

-  Railway
-  Roads
- Peak Flow Velocities [m/s]**
-   $\leq 0.25$
-  0.25 - 0.5
-  0.5 - 1.0
-  1.0 - 1.5
-  1.5 - 2.0
-  2.0 - 4.0
-  4.0 - 6.0
-   $> 6.0$



Prepared by:



**PMF FAILURE SCENARIO  
PEAK FLOW VELOCITIES  
[HOUSES ON FIRE ROAD NO P1]**

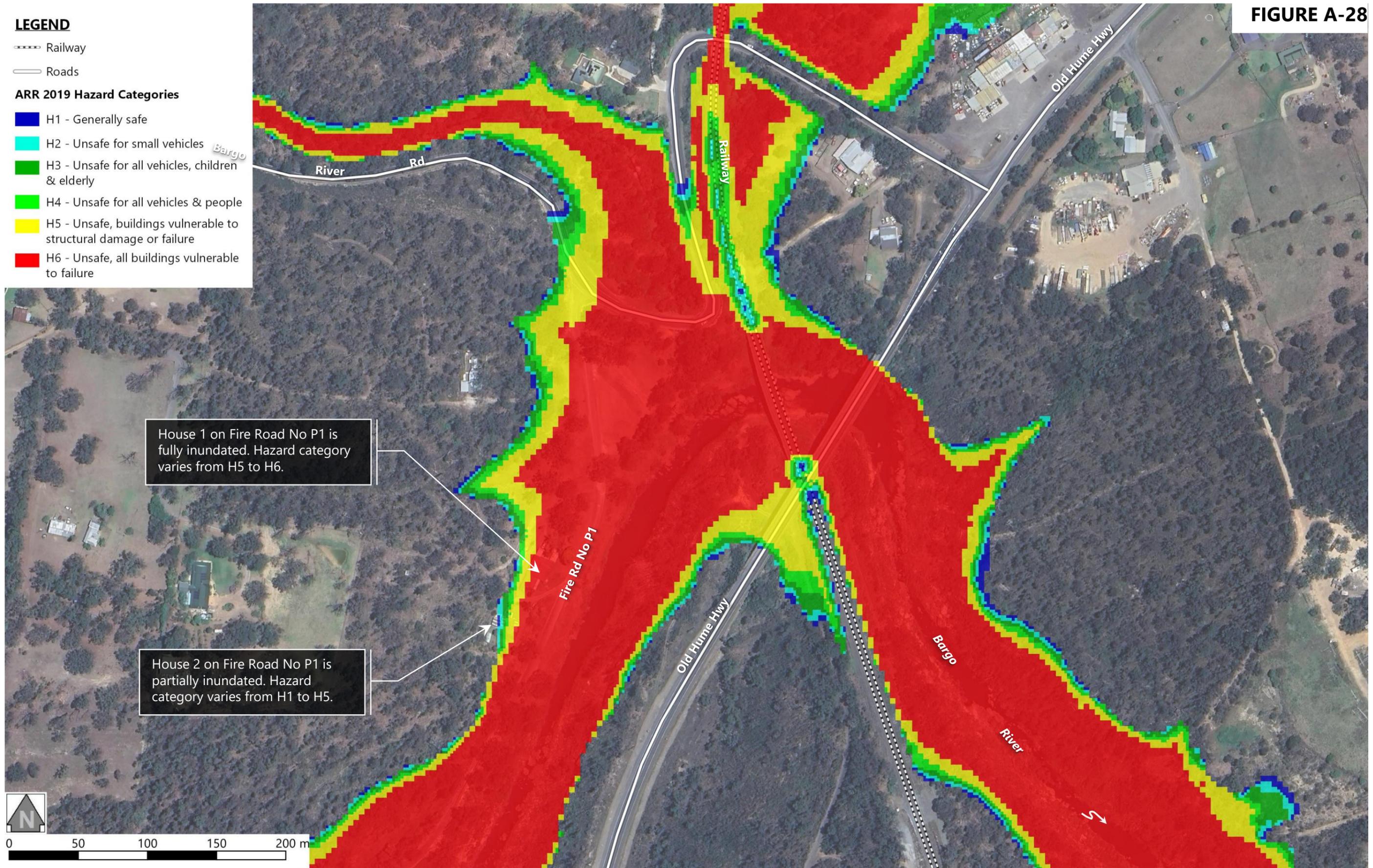
**LEGEND**

--- Railway

— Roads

**ARR 2019 Hazard Categories**

- H1 - Generally safe
- H2 - Unsafe for small vehicles
- H3 - Unsafe for all vehicles, children & elderly
- H4 - Unsafe for all vehicles & people
- H5 - Unsafe, buildings vulnerable to structural damage or failure
- H6 - Unsafe, all buildings vulnerable to failure



Prepared by:

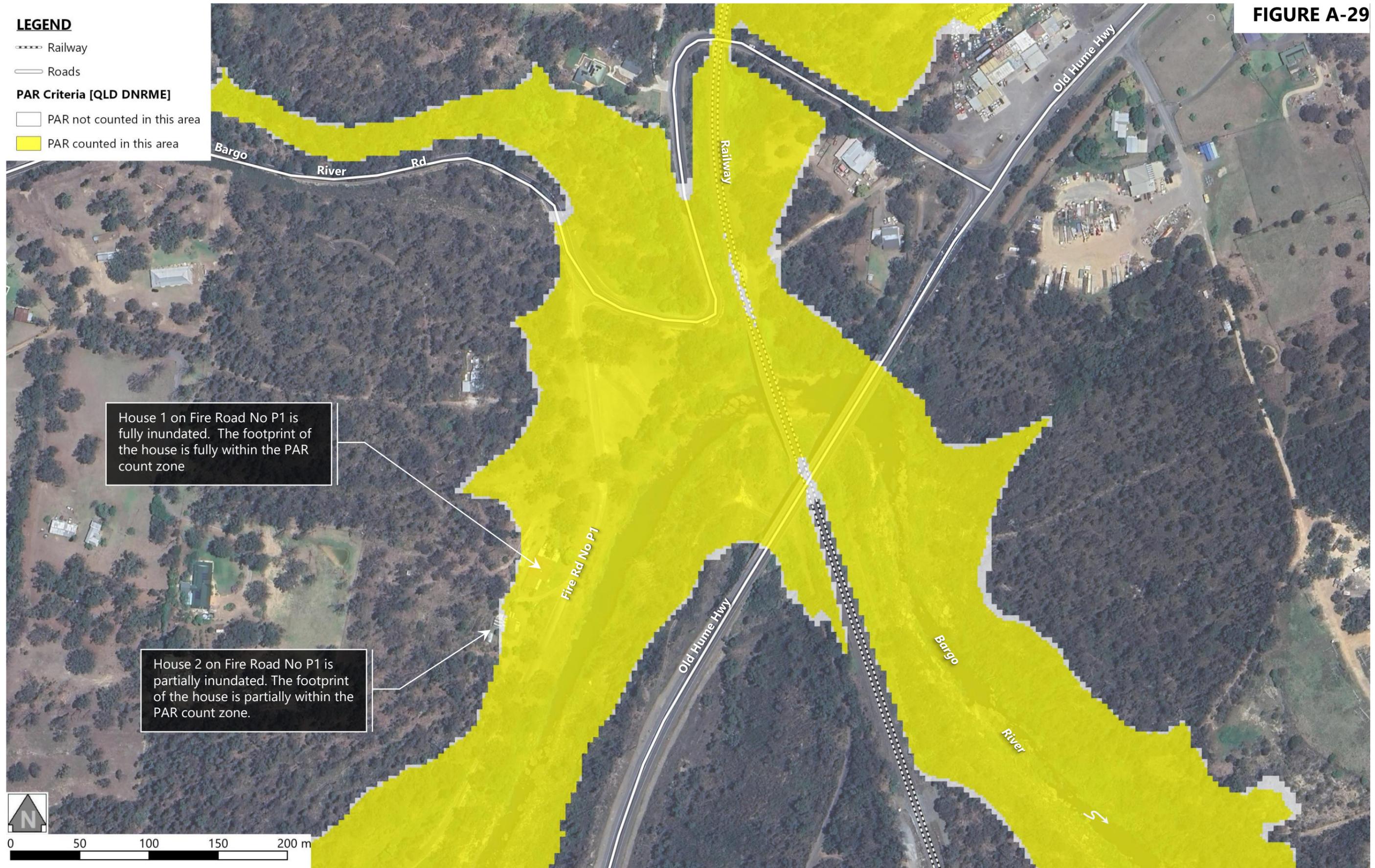


**LEGEND**

- Railway
- Roads

**PAR Criteria [QLD DNRME]**

- PAR not counted in this area
- PAR counted in this area



Prepared by:



FIGURE A-30

**LEGEND**

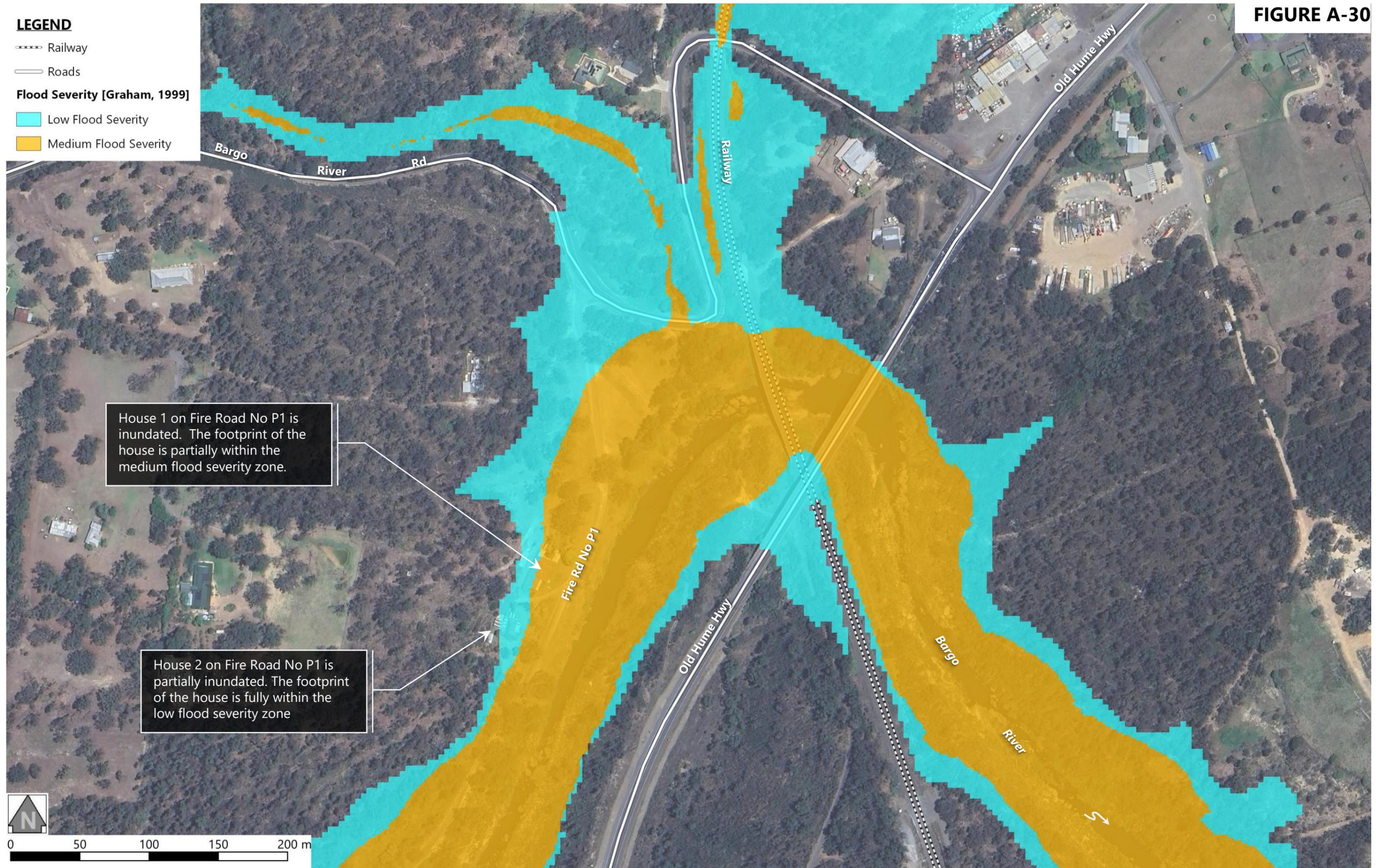
--- Railway

— Roads

**Flood Severity [Graham, 1999]**

Low Flood Severity

Medium Flood Severity



Prepared by:



**PMF FAILURE SCENARIO  
GRAHAM 1999 FLOOD SEVERITY  
[HOUSES ON FIRE ROAD NO P1]**

## **Appendix B. Campbell's Method (2015)**

# Flooded cars: estimating the consequences to itinerants exposed to dambreak floods on roads

Jamie Campbell<sup>1</sup>, Gregg Barker<sup>2</sup>, Paul Southcott<sup>3</sup> and Michael Wallis<sup>4</sup>

<sup>1</sup> Dams and geotechnical engineer, Entura, Tasmania, Australia.

<sup>2</sup> Senior dams and geotechnical engineer, Entura, Tasmania, Australia.

<sup>3</sup> Specialist dams engineer, Entura, Tasmania, Australia.

<sup>4</sup> Specialist hydraulics engineer, Entura, Tasmania, Australia.

*The assessment of consequences of dambreak is used as input to the design parameters of dams, dam safety requirements and dam risk assessments. For many low consequence category dams, the consequences of failure can be dominated by itinerants, in particular vehicles on roads within the dambreak inundation area. Estimating the population at risk (PAR) and potential loss of life (PLL) rigorously is mathematically complex, requires significant user judgment and can be very sensitive to input assumptions. This paper presents a simple, practical tool that has been developed to assist engineers and analysts in assessing the PLL of itinerant road users within a dambreak inundation zone. The tool allows for a logical and defensible analysis based on an event tree approach and provides guidance on appropriate factors to be used in calculating the overall fatality rate of people exposed to the dambreak hazard. This paper details the tool and how to apply it to typical dambreak problems, providing the reader with the information required to estimate the consequences on itinerant road users; the paper also details how the concepts discussed can be applied to other itinerants.*

**Keywords:** Dam failure, dambreak, consequences, consequence category, risk assessment, itinerants, population at risk, potential loss of life.

## Introduction

Dambreak consequence assessments are a key method in dam engineering to assess the relative risk of dams and as input to the selection of engineering and surveillance standards the dam should comply with.

ANCOLD (2012) defines the population at risk (PAR) as all people who would be directly exposed to flood waters assuming they took no action to evacuate. The PAR includes dwellings and other buildings where people assemble, as well as itinerants. The potential loss of life (PLL) is the proportion of the PAR estimated to lose their lives as a result of the dambreak hazard. PLL is estimated by applying an appropriate fatality rate or vulnerability factor to the PAR (Graham 1999).

ANCOLD (2012) has further clarified the inclusion of itinerants into the estimation of dam failure consequences. Additionally, the use of PLL instead of PAR is now possible in the assessment of dam consequence category. Many dams have roads downstream which are at risk of inundation by a dambreak flood and itinerant road users may therefore be an important consideration in assessing the dam consequence category where there is not a permanent population at risk.

The estimation of PAR and PLL for itinerant road users can be mathematically complex, requires significant judgment and can be very sensitive to input assumptions. Nonetheless, it may be critical in the consideration of consequences, particularly when there is little to no permanent PAR, or the PAR is a long distance downstream of the dam and has long warning times. It allows accurate definition of the consequence category of dams, particularly in the Very Low to Significant consequence categories and allows risk assessments of these dams to be assessed on a societal basis. A practical tool has been developed to assist dam engineers and risk analysts in assessing the risks to itinerant road users within a dambreak inundation zone. The tool allows for a logical and defensible

analysis based on an event tree approach to estimate the weighted PAR and PLL due to itinerant road users.

This paper details the following:

- Background concepts and theory.
- Input data and quantification of parameters.
- Example application of the tool.
- Other applications of the tool.
- Conclusions and suggestions for further work.

## Background concepts and theory

AGS (2007) presented an equation to estimate the risk of loss of life due to landslide hazards. The equation is presented below with the parameters defined generically.

$$R_{LoL} = P_H \times P_{S:H} \times P_{T:S} \times V_{D:T} \quad (1)$$

where

$R_{LoL}$  is the annual probability of loss of life of an individual.

$P_H$  is the annual probability of the landslide hazard.

$P_{S:H}$  is the probability of spatial impact of the hazard.

$P_{T:S}$  is the temporal probability of the individual being present at the location of impact.

$V_{D:T}$  is the vulnerability (fatality rate) of the individual given the impact.

The above equation was modified to estimate the life safety risk (LSR) of itinerant road users due to dam failure. The authors have assumed that dambreak modelling and inundation mapping has adequately defined the inundation area and hence  $P_{S:H}$  is equal to 1.0 giving:

$$LSR = P_f \times P_{T:S} \times PAR_V \times P_{NE:T} \times P_{A:NE} \times V_{D:A} \quad (2)$$

where

$P_f$  is the annual probability of dam failure for the particular dambreak scenario,

$P_{T:S}$  is the temporal spatial probability of a vehicle being exposed to the dambreak hazard,

$P_{NE:T}$  is the probability of the driver of the vehicle taking no action to avoid the particular dambreak hazard,

$P_{A:NE}$  is the probability of an accident given that the driver of the vehicle has taken no action to avoid the particular dambreak hazard,

$V_{D:A}$  is the vulnerability (fatality rate) of the vehicle occupants given that an accident has occurred, and

$PAR_V$  is the population at risk within the vehicle.

ANCOLD (2012) proposes that the hazard posed to itinerant road users arises from three possible scenarios:

1. Being present in the flood inundation zone at the time that the dambreak flood wave passes through.
2. Driving into the flood inundation zone before the dambreak flood wave has passed.
3. Driving into the inundation zone after the flood wave has passed through and having an accident due to damage of the road.

## Input data and quantification of parameters

The procedure required to estimate the weighted  $PAR_V$ , PLL and f-N pairs for itinerant road users includes eight steps:

1. Perform hydraulic modelling of the dambreak outflow downstream and gather the data on the road.
2. Determine the temporal spatial probability ( $P_{T:S}$ ).
3. Select the scenarios to be included.
4. Determine the  $PAR_V$ .
5. Determine the probability of taking no action to avoid the dambreak hazard ( $P_{NE:T}$ ).
6. Determine the probability of an accident ( $P_{A:NE}$ ).
7. Determine the vulnerability of the vehicle passengers ( $V_{D:A}$ ).
8. Calculate the weighted  $PAR_V$ , weighted PLL and f-N pairs.

### Step 1 – Perform hydraulic modelling of the dambreak outflow downstream and gather the data on the road

Hydraulic modelling of the dambreak flood is discussed in a number of publications (ANCOLD, 2012) and is not repeated here. The data which are required from the dambreak modelling includes:

- Maximum flood depth over the road (D),
- Maximum flood velocity over the road (V),
- Length of inundated road (L), and
- Duration of inundation of the road ( $T_I$ ).

### Step 2 – Determine the temporal spatial probability ( $P_{T:S}$ )

Temporal spatial probability ( $P_{T:S}$ ) is the probability of a vehicle being exposed to the particular dambreak hazard. Factors which influence  $P_{T:S}$  include: time of day (TD), vehicle speed, dambreak modelling data and traffic volume.

#### ***Time of day***

The time of day (TD) affects the number of itinerants who may be travelling on a road subjected to a dambreak flood. For simplicity, we recommend that each day of the week is assumed to be split into ‘business hours’ (10 hours) and ‘after hours’ (14 hours). The user may choose to assess the time of day in small time intervals to take into account factors such as “peak hour” traffic. This is appropriate where data exists for traffic volumes for smaller time intervals and there will be a difference in factors used to assess conditional probabilities (i.e., “peak hour” traffic causing a traffic jam, resulting in an inability to escape or evacuate the flooded area, increasing the vulnerability of people in the area).

#### ***Vehicle speed***

Vehicle speed (S) is important for estimating the time that a vehicle is in the flood inundation zone. For estimation purposes, the vehicle speed can be taken as the road speed zone or the road approach conditions (i.e., a 60 kph corner). For this paper, vehicle speeds of 50 kph for residential roads and 100 kph for highways and rural roads have been adopted. Values can be interpolated for other speeds if required.

#### ***Dambreak modelling data***

The maximum flood depth (D), maximum flood velocity over the road (V), length of inundated road (L) and duration of inundation ( $T_I$ ) are determined from hydraulic modelling of the dambreak outflow downstream. The data is used to determine the time a vehicle would be in the flood inundation zone and for assessing vehicle stability.

#### ***Time in inundation zone***

The time ( $T_V$ ) a vehicle is in the flood inundation zone is estimated to be:

$$T_V = \frac{L}{S \times 1000} \quad (3)$$

where

L is length of inundated road (m), and

S is vehicle speed (kph).

### Traffic volume

Generally, transport departments or authorities are able to provide annual traffic volume data (annual average daily traffic – AADT) for a particular road that is being investigated in a dambreak event; however, if data is not available then an estimate must be made. The AADT is used to estimate the number of vehicles per hour that use the road during both business hours and after hours.

### Temporal spatial probability ( $P_{T:S}$ ) for scenario A

The temporal spatial probability of a vehicle being present in the flood inundation zone at the time that the dambreak flood wave passes through (i.e., scenario A) is calculated as follows:

$$P_{T:S-A} = N_V \times T_V \quad (4)$$

where

$N_V$  is the number of vehicles per hour during the particular dambreak scenario.

Separate estimates of  $P_{T:S-A}$  may be made for business hours and after hours.

### Step 3 – Select the scenarios to include

Table 1 below provides guidance on which scenarios to include and the  $P_{T:S}$  that should be adopted for scenarios B and C, based on the temporal spatial probability ( $P_{T:S}$ ) estimated for scenario A, the traffic volume (AADT), the duration of inundation of the road in days ( $T_I$ ) and consideration of the likely extent of damage to the road.

**Table 1. Guidance on selection of scenarios to include**

Contributing factors	Scenario A Being present in the flood inundation zone	Scenario B (note 1) Driving into the flooded area during the flood	Scenario C Driving into the flooded area after the flood wave
High traffic volume $P_{T:S-A} \geq 0.5$	High contribution to risk $P_{T:S-A}$ from step 2	High contribution to risk $P_{T:S-B} = 1.0$	Negligible contribution to risk $P_{T:S-C} = 0$
Moderate traffic volume $0.05 \leq P_{T:S-A} < 0.5$	Moderate contribution to risk $P_{T:S-A}$ from step 2	High contribution to risk $P_{T:S-B} = 1.0$	Negligible contribution to risk $P_{T:S-C} = 0$
Low traffic volume $P_{T:S-A} < 0.05$ AADT $\geq 500$ AADT $\times T_I \geq 1.0$	Negligible contribution to risk $P_{T:S-A} = 0$	High contribution to risk $P_{T:S-B} = 1.0$	Negligible contribution to risk $P_{T:S-C} = 0$
Very low traffic volume $P_{T:S-A} < 0.05$ AADT $< 500$ AADT $\times T_I < 1.0$ Serious road damage or washout	Negligible contribution to risk $P_{T:S-A} = 0$	Negligible contribution to risk $P_{T:S-B} = 0$	High contribution to risk $P_{T:S-C} = 1.0$

Note 1: This table simplifies the assessment of scenarios and allows the user to consider each scenario as a separate independent event and hence common cause adjustments are not necessary. If the user determines there is some dependency of Scenario B to Scenario A

(i.e., cars approaching the flood see an accident from a car present in the flood zone, this can be accounted for in estimating  $P_{NE:T}$ )

Note 2: Consider the possibility of vehicles approaching the flooded road from both directions. This is more likely on roads with high travel speeds, poor sight distances and after hours.

#### Step 4 – Determine the $PAR_V$

The  $PAR_V$  is the number of occupants assumed to be in each vehicle that is exposed to the dambreak hazard. The user needs to assess the average trip made to assign a  $PAR_V$ . Typically the number of occupants is between 1 and 2 persons.

#### Step 5 – Determine the probability of taking no action to avoid the dambreak hazard ( $P_{NE:T}$ )

The probability of the driver of the vehicle taking no action to avoid the dambreak hazard ( $P_{NE:T}$ ) is assessed by user judgement. The factors which influence  $P_{NE:T}$  are listed below:

- Sight distance,
- Time of day,
- Vehicle speed (S),
- Weather conditions,
- Safety of the road (including road surface, road grade and alignment, safety barriers, impact features, raised elevation), and
- Driver experience/stupidity.

User judgement should be applied by considering the above influencing factors and using the mapping scheme developed by USBR (2012) which is given in Table 2 below, with guidance as to how the factors affect the conditional probability.

**Table 2. Mapping scheme (USBR 2012)**

Descriptor	$P_{NE:T}$	Guidance (for Sunny Day)
Virtually certain	0.999	
Very likely	0.99	Situation of high speed, short sight distance, night time resulting in the driver not being aware of the dambreak until they are upon it.
Likely	0.9	
Neutral	0.5	
Unlikely	0.1	Good sight distance and appropriate breaking distance (speed) resulting in being able to stop before the flood, but depth of water is low so drivers may try entering regardless (driver experience).
Very unlikely	0.01	
Virtually impossible	0.001	

In dam risk assessment there is often a need to assess dam failure consequences under ‘sunny day’ and ‘flood failure’ scenarios. It is reasonable to assume that drivers of vehicles will be more cautious when there is significant flooding within the dam catchment. The user should therefore exercise judgement in the estimation of  $P_{NE:T}$  for the flood failure scenario, with the result being less than the sunny day failure scenario. For infrequent floods (i.e., >1:100 AEP), the authors suggest applying a factor of 0.1 to sunny day estimates.

#### Step 6 – Determine the probability of an accident ( $P_{A:NE}$ )

The probability of an accident given that the driver of the vehicle has taken no action to avoid the dambreak hazard ( $P_{A:NE}$ ) is assessed by user judgement. The factors which influence  $P_{A:NE}$  are listed below:

- Vehicle speed (S),

- Maximum flood depth (D),
- Maximum flood velocity over the road (V), and
- Road damage (including culverts and bridges).

$P_{A:NE}$  is also influenced by vehicle stability, which in turn, is a function of vehicle type and size, maximum flood depth (D) and maximum flood velocity over the road (V). To simplify the methodology, a large passenger vehicle (length > 4.3 m and kerb weight >1250 kg) has been adopted as the typical vehicle and the methodology is based on the stability criteria proposed in Shand *et al.* (2011).

Table 3 provides suggested values of  $P_{A:NE}$  for different hazard scenarios, vehicle stability criteria and vehicle speeds (S). As  $P_{A:NE}$  is a judgemental probability; it is acceptable to select intermediate values for speed zones between those listed. As expected,  $P_{A:NE}$  values are lower at lower road speeds, as there is more chance of the driver avoiding an accident.

**Table 3. Probability of an accident ( $P_{A:NE}$ )**

Population at Risk Scenario	Vehicle stability criteria	Vehicle Speed (S)	
		100 kph	50 kph
Vehicle within inundation zone – (A)	$D * V \leq 0.45 \text{ m}^2/\text{s}$ with limiting $D < 0.4 \text{ m}$ and $V < 3.0 \text{ m/s}$	0.30	0.10
Vehicle driving into inundation zone during event – (B)		0.30	0.10
Vehicle driving into inundation zone after event - minimal damaged road surface – (C)		N/A	N/A
Vehicle driving into inundation zone after event - minimal washout of culvert or bridge – (C)		N/A	N/A
Vehicle within inundation zone – (B)	$D * V \geq 0.45 \text{ m}^2/\text{s}$ with limiting $D > 0.4 \text{ m}$ and $V > 3.0 \text{ m/s}$	0.90	0.80
Vehicle driving into inundation zone during event – (B)		0.90	0.80
Vehicle driving into inundation zone after event - significant damaged road surface – (C)		0.3	0.1
Vehicle driving into inundation zone after event - significant washout of culvert or bridge – (C)		0.9	0.8

Notes -            D = Flood depth on road (m)            V = Flood velocity over road (m/s)

### Step 7 – Determine the vulnerability of the vehicle passengers ( $V_{D:A}$ )

The vulnerability of the vehicle passengers ( $V_{D:A}$ ) (i.e., probability of loss of life of the passengers given that an accident has occurred) is assessed by user judgement. The factors which influence  $V_{D:A}$  are listed below:

- Vehicle speed (S),
- Safety of the road (including road surface, road grade and alignment, safety barriers, impact features, raised elevation),
- Presence of dangerous water at the accident site,
- Time of day, and
- Estimated response time for first aid and emergency services.

The potential types of accidents have been identified with different vulnerabilities. The types of accidents are listed below and guidance is provided on the estimation of  $V_{D:A}$ :

1. Vehicle accident in deep, fast flowing water:

In this situation, a vehicle is virtually certain to be swept off the road with a high probability of drowning of the passengers. As such, this is an override situation with  $V_{D:A}$  = between 0.7 and 0.999.

Deep, fast flowing water is assessed downstream of the road and needs to be sufficient to submerge the car and continue to wash it downstream, causing a drowning risk. This is referencing the DV on the road.

2. Vehicle accident in shallow, slow-moving water, or no water

$$V_{D:A} = BF \times W \times TD \times RT \times I \quad (5)$$

Parameters are defined and factors suggested in Table 4 below.

**Table 4. Parameters and factors used to estimate vulnerability ( $V_{D:A}$ )**

Description	Factors	
<b>Road speed zone</b>	90 - 110 kph	40 - 80 kph
Base factor, BF (No water) – Note 3	0.1	0.01
<b>Impact</b>	Factor, I	
Impact with safety barrier or run off road	0.2	
Impact with ditch, fence or small tree	0.4	
Impact with large tree, off embankment or into breach	2.0	
<b>Water present</b>	Factor, W	
Vehicle in shallow slow moving water	2.0	
Vehicle not in water but has impact accident	1.0	
<b>Time of day</b>	Business Hours	After Hours
Factor, TD	1.0	2.0
<b>Response time (Note 4)</b>	< 30 min	≥ 30 min
Factor, RT	1.0	2.0

Note 3 - The base factor (BF) has been based on Tasmanian state road data (DIER 2010; DIER 2011) which shows that for all road accidents, the fatality rate is 0.6% and for roads with a speed zone of 90-110 kph the fatality rate is 3%. As a vehicle accident associated with a dambreak hazard is considered to be more dangerous than the average, the base fatality rates have been increased to 10% (90-100 kph) and 1% (speed zone 40-80 kph) respectively.

Note 4 - Time for emergency response (SES or ambulance) to arrive at flood zone.

### Step 8 – Calculate the weighted $PAR_v$ , weighted PLL and f-N pairs

Once all the above steps have been completed, the weighted  $PAR_v$  and weighted PLL can be calculated for use in a dam consequence category assessment and f-N pairs can be calculated for use in a dam risk assessment. Table 5 below shows how the calculations can be undertaken in tabular format. Note that separate calculations may need to be made for 'sunny day failure' and 'flood failure' scenarios.

**Table 5. Calculation of weighted  $PAR_V$ , weighted PLL and f-N pairs**

$P_f$	<u>BH or AH</u> <u>24</u>	Scenario	$P_{T:S}$ Steps 2, 3	$P_{NE:TS}$ Step 5	$P_{A:NE}$ Step 6	Exposure	$PAR_V$ Step 4	Weighted $PAR_V$	$V_{D:A}$ Step 7	Weighted PLL	f (note 5)	N (note 5)
[1]	[2a]	A	[3a]	[4a]	[5a]	[6a] = [2a] x [3a] x [4a] x [5a]	[7a]	[8a] = [6a] x 7[a]	[9a]	[10a] = [6a] x [7a] x [9a]	[11a] = [1] x [6a]	[12a] = [7a] x [9a]
		B	[3b]	[4b]	[5b]	[6b] = [2a] x [3b] x [4b] x [5b]	[7b]	[8b] = [6b] x 7[b]	[9b]	[10b] = [6b] x [7b] x [9b]	[11b] = [1] x [6b]	[12b] = [7b] x [9b]
		C	[3c]	[4c]	[5c]	[6c] = [2a] x [3c] x [4c] x [5c]	[7c]	[8c] = [6c] x 7[c]	[9c]	[10c] = [6c] x [7c] x [9c]	[11c] = [1] x [6c]	[12c] = [7c] x [9c]
	[2b]	A	[3d]	[4d]	[5d]	[6d] = [2b] x [3d] x [4d] x [5d]	[7d]	[8d] = [6d] x 7[d]	[9d]	[10d] = [6d] x [7d] x [9d]	[11d] = [1] x [6d]	[12d] = [7d] x [9d]
		B	[3e]	[4e]	[5e]	[6e] = [2b] x [3e] x [4e] x [5e]	[7e]	[8e] = [6e] x 7[e]	[9e]	[10e] = [6e] x [7e] x [9e]	[11e] = [1] x [6e]	[12e] = [7e] x [9e]
		C	[3f]	[4f]	[5f]	[6f] = [2b] x [3f] x [4f] x [5f]	[7f]	[8f] = [6f] x 7[f]	[9f]	[10f] = [6f] x [7f] x [9f]	[11f] = [1] x [6f]	[12f] = [7f] x [9f]
<b>TOTAL</b>								= $\Sigma$ [8a] to [8f]		= $\Sigma$ [10a] to [10f]		

Note 5: Where  $PAR_V \times V_{D:A} < 1$ , it is suggested that  $f = P_f \times \text{Exposure} \times V_{D:A}$  and  $N = PAR_V$ .

Note 6: For each conditional probability assessed, the user should document the reasoning and determination behind the factors selected to enable future users to repeat the calculations.

## Example application of tool

To demonstrate how the tool is applied, an example is given below. In this scenario it is assumed there is no permanent population and the itinerant population dominates. For this paper, only the sunny day scenario during business hours is calculated.

### Step 1

In this scenario, the following details are relevant or have already been obtained by the dambreak inundation modelling and gathering of data:

- the highway immediately below the dam has an annual average daily traffic of 7,250.
- most traffic is travelling at 100 kph on this stretch of road and the inundation zone is approximately 500 m wide.
- there is good line of sight on each approach to the inundation zone, however traffic at either end of the inundation zone cannot see each other.
- the flooding over the road occurs for 30 minutes.
- the maximum flood depth over the highway is 0.3 m with a velocity of 2.0 m/s.
- assume that 80% of traffic occurs during business hours (10 hours a day) and 20% of traffic occurs after hours (14 hours a day).

### Step 2 – Determine the temporal spatial probability

For scenario A – vehicle in the inundation zone, we work out the number of vehicles on the stretch of road on average during business hours. So:

$$T_V = \frac{500}{100 \times 1000} = 0.005 \text{ hours}$$

$$N_V = \frac{7250 \times 0.8}{10} = 580 \text{ cars/hour}$$

$$P_{T:S-A} = 580 \times 0.005 = 2.9 \text{ cars}$$

Hence 2.9 cars are, on average, in the inundation area during the flood (i.e., 2 cars with a 100% probability of being in the zone and 1 car with a 90% probability of being in the zone). In this example the highway is not greatly affected by “peak hour” traffic and the breakdown of traffic data is not available, hence this simplification is considered adequate for estimation of the itinerants exposed.

### Step 3 – Select the scenarios to be assessed

As  $P_{T:S-A} > 0.5$ , this is considered a high traffic volume road (from Table 1); hence we assess scenario A and scenario B.

For scenario B – vehicle entering the inundation zone, Table 1 gives a  $P_{T:S-B} = 1$ . However in this instance the road users at one end cannot see the road users at the other end, hence the  $P_{T:S-B} = 1$  for cars approaching the inundation zone from each end. This is due to the assumption in the methodology that if the first car enters the flood inundation zone or takes action to stop entering, then subsequent cars will see the first car and take action to stop. Hence it is only necessary to consider the first car that approaches the flood zone.

### Step 4 – Determine the Population at Risk (PAR)

Average occupancy rates for vehicles in Australia range from 1.2 to 1.5 people per car. So for Scenario A where on average we have 2.9 cars in the zone, we will adopt a  $PAR_V$  of 2 for one car, and a  $PAR_V$  of 1 for the remaining 1.9 cars.

For Scenario B, we will adopt a  $PAR_V$  of 1 for a car coming in one direction and a  $PAR_V$  of 2 for the car coming in the other direction.

### **Step 5 – Determine the probability of taking no action to stop prior to the flood wave**

For Scenario A, the vehicles are already in the inundation zone, hence the probability of taking no action to stop is irrelevant to the scenario and  $P_{NE:T-A} = 1.0$

For Scenario B, we need to assess the likelihood of taking no action from each direction (as a car having an accident at one end cannot see the car at the other end).

From each end there is good sight distance and the driver will see the flood as they approach. The driver will be travelling at 100 kph and will need good distance to stop. It is assessed as likely that the driver will have sufficient time to stop, however given the low depth of water over the road; it is possible that the driver will try to enter the flood regardless. Hence a judgement assessment of unlikely - neutral has been made and a  $P_{NE:T-B} = 0.3$  adopted for each end.

### **Step 6 – Determine the probability of an accident once in the inundation area**

For Scenario A, the peak  $DV = 0.6 \text{ m}^2/\text{s}$  and hence there is sufficient flood depth to either cause aquaplaning or to wash the car off the road causing an accident. Given that the car is travelling at 100 kph, a  $P_{A:NE-A} = 0.9$  is adopted.

For Scenario B, the peak  $DV = 0.6 \text{ m}^2/\text{s}$ , however it is likely that the driver has slowed significantly, either in an attempt to stop or in attempt to cross the road, so a 50 kph travelling speed is adopted and a  $P_{A:NE-B} = 0.8$  is adopted.

### **Step 7 – Determine the probability of a fatality once an accident occurs**

For this section of road there are no significant culverts or embankments and the water downstream of the road is a similar depth to that flowing over the road. Hence the result of an accident is the car being in shallow, slow moving water (i.e., not significant enough to submerge the car).

For Scenario A, it has been established that the car is travelling at 100 kph, it is business hours and there is water present. The road section is located within 20 minutes of emergency services and the likely result of the accident is hitting a ditch, fence or small tree in the downstream inundation area. Hence

$$V_{D:A-A} = 0.1(\text{BF}) \times 1.0(\text{TD}) \times 1.0(\text{RT}) \times 0.4(\text{I}) \times 2.0(\text{W}) = 0.08$$

For Scenario B, the parameters are all the same, except it has been established that the car is travelling at 50 kph in the flood zone. Hence

$$V_{D:A-B} = 0.01(\text{BF}) \times 1.0(\text{TD}) \times 1.0(\text{RT}) \times 0.4(\text{I}) \times 2.0(\text{W}) = 0.008$$

### **Step 8 – Calculate the weighted PLL and the probability of PLL**

The calculations of PLL are given as a summary in Table 6.

### **Other applications of tool**

Whilst the tool detailed in this paper is designed for using for itinerants on roads, the concepts are equally applicable for other itinerants. The basic methodology has been used for camping areas, parks, sports grounds and manned power stations or water treatment plants. An example of utilising this method for a sports ground is given below.

**Table 6: Summary table**

Scenario	P <sub>T:S</sub>	P <sub>NE:T</sub>	P <sub>A:NE</sub>	PAR <sub>V</sub>	V <sub>D:A</sub>	Weighted PLL	f	N
Vehicle in the inundation zone – (A)	1	1.0	0.9	2	0.08	0.14	0.072 x P <sub>f</sub> <sup>[7]</sup>	2
	1.9	1.0	0.9	1	0.08	0.14	0.137 x P <sub>f</sub>	1
Vehicle entering the inundation zone – direction 1 – (B)	1	0.3	0.8	2	0.008	0.0038	0.0019 x P <sub>f</sub> <sup>[7]</sup>	2
Vehicle entering the inundation zone – direction 2 (B)	1	0.3	0.8	1	0.008	0.0019	0.0019 x P <sub>f</sub>	1
<b>Total</b>						0.29	0.074 x P <sub>f</sub> <sup>[7]</sup>	2 <sup>[7]</sup>
							0.209	1

Note 7 –This is a simplified estimate where the PLL is calculated as either all the PAR<sub>V</sub> exposed to the flood, or none of the PAR<sub>V</sub> exposed to the flood, both occupants either dying in the car or neither dying. This can be expanded to estimate the probability of a fatality for each case of N; however this gets mathematically complex for large numbers of N.

A sports ground is utilised on Saturdays for a 6 hour period, the sports ground is also utilised 2 hours in the evening twice a week for training. During Saturdays there is on average 80 people at the sports ground, while there is normally 25 people during training periods. During a dambreak the sports ground is inundated by 0.5 m deep water travelling at 3.0 m/s.

The temporal spatial probability is calculated by working out the exposure in any given week (i.e., 4 + 6 hours/168 hours).

In this example, there is no warning until the water levels begin to rise. It is expected that people will attempt to self-evacuate by running to high ground. It is considered likely that people would escape the flood waters and using the Bureau of Reclamation, 2003 table (given for probability of not taking action) a probability of 0.1 is assigned for having an accident (i.e., being in the flood inundation zone). In this instance we have combined the probability of not taking action with the probability of an accident.

Once washed away by the flood waters, it is considered that the probability of a fatality will be similar to the probability of dying by drowning due to deep fast moving water. Hence a fatality rate of 0.7 is adopted.

The calculations of PLL are given as a summary in Table 7.

**Table 7: Summary table**

Scenario	P <sub>T:S</sub>	P <sub>NE:T</sub>	P <sub>A:NE</sub>	PAR	V <sub>D:A</sub>	Weighted PLL	f	N
Training	0.024	0.1	1.0	25	0.7	0.042	0.0024 x P <sub>f</sub> <sup>[7]</sup>	18
Saturday- game	0.036	0.1	1.0	80	0.7	0.20	0.0036 x P <sub>f</sub> <sup>[7]</sup>	56
<b>Total</b>						0.24		

Note 7 – Refer to note on Table 6.

## Conclusions and suggestions for further work

A simple and practical tool has been developed to estimate the weighted population at risk, potential loss of life, and f-N pairs of itinerants associated with roads, with the concepts applicable to a wider range of itinerant cases. The tool allows for a consistent, defensible and appropriate assessment of itinerants for individual dams or a portfolio of dams. This is critical to proper assessment of risks where the main PAR for a dam are itinerants, who have frequently in the past either been ignored or poorly estimated, leading to poor allocation of scarce dam safety funds. Future repeatability of the tool requires the user to sufficiently detail the reasoning behind why factors were assigned.

This tool has been successfully applied to a number of dams already, and where possible has been based on available statistics. However statistics involving road itinerants in natural and dambreak floods is difficult to obtain. Future work would include researching available statistics to provide a database of cases to refine the factors used in this paper. The initial objective of this tool was to provide some standard overall factors for the potential loss of life and f-N results that could be applied to generic road types, however in developing and applying the tool, it was found that there were many variables to consider and each case was different. With a greater database of cases where the tool is applied and verified, it is envisaged in future that the initial objective could be achieved.

The authors would also like feedback from the industry and professional dam engineers and risk analysts as to how applicable the tool and factors are. If the reader has any feedback the authors would welcome it and feedback can be sent to [Jamie.Campbell@entura.com.au](mailto:Jamie.Campbell@entura.com.au).

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