Cardno
Shaping the Future

SUCE Stage 1
Flood Study

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PLANS AND DOCUMENTS
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1 INTRODUCTION

This report assesses the flooding impact of developing Stage 1 of the proposed Ripley Valley Secondary Urban Centre East (SUCE) development.

The flood strategy outlines the requirements to develop the SUCE whilst not compromising the flood immunity provided for existing buildings located in the area.

The strategy has been developed using the flood model created for the assessment of the entire SUCE development. This model has been further refined to better model the local impacts of development.

Since the submission of the Stage 1 report (November 2010) modifications to the Barrams Road crossing design have been made, so as to address the concerns of the neighbouring resident. In order to reduce the height of the bridge, and the subsequent aesthetic impacts, it has been agreed that an increase in flood level resulting from the bridge would be considered acceptable, so long as sufficient freeboard was still provided for the existing residences. As such, it has also been shown that the compensatory excavation that was previously proposed, which was also of concern to neighbouring residents, would no longer be necessary.

The purpose of this report is also to address the issues raised in the Information Response Request provided by the Urban Land Development Association (ULDA).

2 BACKGROUND

A part of the SUCE has been declared by the Urban Land Development Authority (ULDA) as an Early Release Precinct (ERP). This incorporates Stage 1. The Stage 1 site covers an estimated 12.3 Ha, includes 160 lots, and has an equivalent population (EP) of 400 EP.

In order to proceed with these works it is necessary to upgrade the Barrams Road bridge crossing, so as to increase its traffic capacity and improve its flood immunity. Earthworks associated with the Stage 1 area will also be required. As existing buildings are located directly upstream of the Barrams Road bridge crossing, they are potentially impacted upon by these works. The location of the existing buildings is shown in Appendix A. Detailed flood modelling using TUFLOW has been carried out to model the impact of the Stage 1 works.

3 HYDROLOGY

3.1 WBNM Model

The Watershed Bounded Network Model (WBNM) is a hydrologic catchment model that can be used to model the runoff behaviour of rural and urban catchments. It is also able to model the behaviour of detention basins, and so is able to model the existing Bundamba Lagoon. The resultant hydrographs that WBNM produces can be used as inflows to the TUFLOW hydraulic model. TUFLOW will then be able to calculate the appropriate routing of flows.

3.2 Catchment Areas

The catchment areas used in the previous WBNM model were divided up in some areas to provide better representation of the inflows around the proposed development. The original catchments were delineated using CatchmentSIM. CatchmentSIM is a modelling toolkit developed by CRC for Catchment Hydrology which determines catchment areas based on the topography of a catchment.

The revised catchments are shown in Figure 1. The catchment areas are given below in Table 1. The total catchment area is 8,365 hectares. The total catchment area to the Centenary Highway Extension (CHE) crossing of Bundamba Creek is 5,354 hectares. It was assumed that all catchments were 100 percent pervious.

Table 1 Catchment Areas

Catchment Name	Area (ha)	Catchment Name	Area (ha)
A1	256.80	F3	80.80
A2	369.48	G1	151.05
A3	563.98	G2	165.74
B1	46.15	G3	28.72
B2	194.96	G4	48.32
B3	38.77	H1	387.16
B4	83.85	H2	253.27
C1	1071.55	H3	222.94
C2	227.39	I	78.13
C3	91.41	J	92.33
D1	106.83	K	134.60
D2	90.00	L	57.22
D3	18.54	M1	173.34
D4	65.70	M2	115.37
E1	134.95	N1	226.74
E2	80.58	N2	55.44
E3	21.02	O	1141.92
F1	106.64	P1	720.57
F2	99.84	P2	561.47

3.3 Rainfall Intensities

The following rainfall intensity values were used, as recommended by Bureau of Meteorology (BoM) for Ripley Valley (27.7˚S, 152.8˚E):

- 2 year 1 hour rainfall intensity = 45.23 mm/h
- 2 year 12 hour rainfall intensity = 6.97 mm/h
- 2 year 72 hour rainfall intensity = 1.86 mm/h
- 50 year 1 hour rainfall intensity = 88.87 mm/h
- 50 year 12 hour rainfall intensity = 14.35 mm/h
- 50 year 72 hour rainfall intensity = 4.01 mm/h
- Geographical Factor (F2) = 4.39
- Geographical Factor (F50) = 17.12
- Skew (G) = 0.20

3.4 Areal Reduction Factor

For catchments greater than 400 hectares Australian Rainfall and Runoff (AR&R) recommends the use of Areal Reduction Factors (ARF) to account for the fact that it is unlikely that the rainfall intensity experienced at a point will be equal over a large catchment area. The ARF for the catchment area at the CHE was calculated using Rainfall IFD (developed by DERM using the methods outlined by CRC Forge and AR&R). The resultant 24 hour event ARF of 0.95 was conservatively adopted for all events.

3.5 Rainfall Losses

The same initial and continuing rainfall loss values used in the SKM 2000 Ipswich Rivers Flood Study (IRFS) for pervious areas were adopted for the flood study, i.e:

- 100 year ARI - 5mm initial loss and a 2.5mm/h continuing loss;
- 50 year ARI - 10mm initial loss and a 2.5mm/h continuing loss;
- 10 year ARI - 15mm initial loss and a 2.5mm/h continuing loss; and
- 5 year ARI - 15mm initial loss and a 2.5mm/h continuing loss.

The initial loss rates adopted for the study are considered to be reasonable given the likelihood of antecedent rainfall saturating the catchment prior to a design event occurring.

3.6 Bundamba Lagoon

The attenuation of flow afforded by Bundamba Lagoon was included in the WBNM model, as it was in the original flood study (Cardno, 2010). The lagoon is located within catchment O of the model (refer Figure 1). The stage-volume relationship was determined from the survey provided by Council and is shown below in Table 2. Based on information supplied by Council, the outlet was assumed to be a 45m wide broad crested weir at 73.47m AHD. For the analysis, it was assumed that the lagoon is full due to antecedent rainfall, i.e. with a standing water level at 73.47m AHD.

Table 2 Bundamba Lagoon Stage-Volume Relationship

Level (m AHD)	Volume (ML)
73.0	0
73.5	115
75.0	600
75.5	810

The analysis showed that peak flows from catchment O were reduced by up 64.6m^3/s in the 100 year ARI event (for the 1 hour storm). This is due to the attenuation provided by the lagoon attenuating flows even when it is full prior to the storm event occurring.

A number of smaller farm dams are located throughout the catchment. However, to be conservative it was assumed that they provided no attenuation of flow.

3.7 WBNM Model Calibration

The Brown Consulting 2006 review reported a wide range of existing case 100 year ARI peak flows (from 298 m^3/s to 516 m^3/s) at the proposed CHE.

In the SKM 2000 IRFS the 100 year peak flow was 364 m^3/s at cross-section BUND 20672.5 for the 4.5 hour event. In 2006 the SKM analysis of the CHE used a 100 year ARI flow of 516m^3/s. This inflow was calculated by setting up a one node RAFTS model which did not include the impact of the Bundamba Lagoon. The parameters used included; a catchment area of 5520 hectares, a Manning's n of 0.1, a vectored slope of 8% and initial and continuing losses of 5mm and 2.5mm/hr respectively. No value for the Bx storage factor used was provided. Replication of this RAFTS model suggested that a Bx factor of around 0.80 would have been used, which is at the lower end of the range of Bx values typically used. A more typical (higher) Bx factor would have resulted in lower flows.

The Brown Consulting 2006 review recommended adopting a 100 year ARI peak flow of 369 m^3/s at the CHE. ICC has to date assumed that the 100 year ARI flows in Bundamba Creek were equivalent to the 50 year ARI flows used in the SKM 2000 IRFS, i.e. 298m^3/s at the CHE.

A WBNM lag parameter (C value) of 1.45 was adopted for all sub-catchments. This is the same value used in the previous WBNM model. A study carried out by WBNM (January 2007) over several catchments concluded that the average lag parameter for catchments in Queensland was 1.47. The resultant 100 year ARI peak flow at the CHE is 371m^3/s for the 3 hour event and 347m^3/s for the 4.5 hour event. These correlate well to the SKM (2000) and Brown Consulting peak flows, and are therefore considered reasonable to use. The resultant peak flows in TUFLOW at the CHE structure (BUND20590) were 386m^3/s in the 3 hour event and 365m^3/s in the 4.5 hour event.

3.8 Stage 1 Developed Case Flows

Previous analysis of the developed cases has assumed that existing case flows were used, as detention basins will be provided throughout the site. The purpose of these detention basins will be to attenuate peak flows to match those predicted for the existing case. For Stage 1 of the SUCE works a detention basin will not be constructed. Due to the small area being developed it is not expected that this will impact on the peak flows in Bundamba Creek with detention basins being built as part of future stages of the development. However, to ensure that this is the case the WBNM model was modified with Stage 1 in place. The percentage impervious used for Stage 1 was 70 percent. The following modifications were therefore made:

- Catchment D2 is now 6.1% impervious;
- Catchment E3 is now 18.6% impervious; and
- Catchment E2 is now 2.1% impervious.

4 EXISTING CASE TUFLOW MODEL SETUP

4.1 TUFLOW

TUFLOW is a linked one-dimensional (1D) and two-dimensional (2D) hydraulic flood model. It simulates the complex hydrodynamics of floods and tides using the full 1D St Venant equations and the full 2D free-surface shallow water equations.

Version 2009 was used for the analysis.

The previous model was conducted using XPSWMM-TUFLOW. However, as new survey data was being used, it was considered appropriate to transfer it to TUFLOW, which is now ICC's preferred 2D modelling package. TUFLOW will allow for easier representation of developed case scenarios and comparison of results.

The set up of the TUFLOW model is shown in Figure 2.

4.2 Survey Data

The digital terrain model (DTM) used in the original XP-SWMM model was provided by Council and extends over the area from BUND10000 (refer Appendix A) to Patrick Street. The projection is in GDA94 and the datum is Australian Height Datum (AHD).

LIDAR survey was recently completed for the area from upstream of BUND10000 to approximately BUND26000. The projection is in GDA94 and the datum is Australian Height Datum (AHD). This survey was used for the revised TUFLOW model. For the areas that were not included in this new survey the original survey provided by Council was adopted.

Detailed ground survey was also completed around the area of the Barrams Road and Ripley Road intersection. This was converted into a DTM and merged with the LIDAR survey.

A comparison of the original Council survey and the LIDAR survey was completed. In most areas the levels were reasonably consistent, except for some overbank areas, and the areas around the CHE, which was not constructed at the time of the original survey. The new DTM is shown in Figure 3.

The original survey does not appear to have properly "thinned" out the vegetation that occurs along some of the channel banks. This resulted in higher ground levels, and so altered the flooding behaviour between the main channel and the flood plain.

The original survey used in the downstream section of the model still includes the high overbank areas. A sensitivity analysis was conducted to see their impact on peak water levels if they were manually removed from the survey. The results showed that the peak water levels in the downstream section of the model were reduced by around 150mm on average and up to 300mm in some areas. However, the impact did not extend upstream to the area being developed. This is mainly because the reduced channel widths adjacent to the Swanbank Power Station control the flows in this area. Therefore the original, unmodified survey was mostly maintained for the downstream section of the model. However, an area where a powerline pylon had falsely created high levels was removed to improve the model stability in this area.

4.3 Grid Size and Time Step

A 10 metre grid size (which uses levels from the DTM at 5 metre intervals), with a 5 second time step was adopted for the analysis. A 5 metre grid size, with a 2.5 second time step was trialled, along with a nested 5 metre grid to represent the main channel only. The difference in results between the 10 metre and 5 metre grid size was on average only 7mm. As the difference in results was negligible and the coarser 10 metre grid resulted in significantly reduced run times and file sizes, it was considered acceptable for use. The nested finer grid did not reduce the run times sufficiently to warrant its use.

To ensure that the invert of the channel was picked up in the 2D grid, a gully line was digitised along the centre of Bundamba Creek. The negligible difference in results between the 10 and 5 metre grid showed that the 10 metre grid provided sufficient definition of the main channel. The main channel is on average around 40 metres in width.

The continuity error was less than 1 percent for the 10 metre grid, which is within the acceptable range.

The original XP-SWMM model used 1D cross-sections for the main channel, and a 2D grid for the flood plain areas. Given that further analysis on the impact of filling and roughness values was required, a fully 2D model was considered more appropriate as it will allow changes to be more easily compared and quantified.

4.4 Inflow Points

The flow hydrographs from WBNM were input into the TUFLOW model as SA inflows. SA inflows work by defining a flow location polygon in the 2D grid and linking it to a flow hydrograph. The lowest points in the 2D grid covered by the polygon will be wetted first, and the flow will then be distributed to the 2D grid accordingly. The location of the inflow points is shown in Figure 2. For some of the larger catchments the inflows were divided over a few locations.

4.5 Roughness Values

The Manning's n roughness values used in the SKM 2000 IRFS and the XPSWMM-TUFLOW model were 0.10 in the main channel and 0.08 in the floodplain areas. These values were slightly modified for the existing case TUFLOW model, to provide better definition of the existing vegetation types. The floodplain areas are currently characterised by long grass vegetation, with a few areas of denser vegetation. Using a Manning's n roughness of 0.08 for all of these areas was too conservative and would not allow the relative impact of revegetation to be correctly established. The waterway area of the main channel, where minimal vegetation is present, was also better defined. The main waterway area is approximately 10 to 20 metres in width.

The Manning's n roughness values used in this model were:

- Grassed floodplain area – 0.05
- Waterway area of main channel – 0.05
- Vegetated channel banks – 0.10
- Medium floodplain vegetation – 0.10
- Dense floodplain vegetation – 0.15

- Roads – 0.03

The resultant roughness values used in the existing case are shown in Figure 2. For the areas that are not shaded, a default value of 0.05 was used.

A sensitivity analysis was conducted to determine the impact of assuming a Manning's n roughness value of 0.08 for the grassed floodplain area and 0.10 for the waterway area. On average the peak water level increased by around 160mm, with larger increases observed in the upstream section of the model.

4.6 Existing Case Structures

The bridges were all modelled in 2D, using the revised DTM. The deck levels and depths of the bridges were taken from the SKM 2000 IRFS. Appropriate loss factors for piers, decks and railings were applied to the model. In general the form loss coefficients (FLC) used were 0.1 for the bridge piers, 0.5 for the bridge deck and 0.5 for the handrails. To model the impact of the CHE on peak water levels and flows it was removed from the existing case model. The DTM was also modified to remove the road embankments.

4.6.1 Patrick Street Bridge

The Patrick Street bridge is an old timber crossing with three spans that is no longer in use. It is located at BUND27310. The main crossing of this waterway is now a causeway located directly downstream of the bridge, which was picked up in the DTM and so did not need to be modelled as a structure. The low flow culverts underneath the causeway were not included as they will have minimal conveyance capacity during major flows. The causeway was not included in the SKM 2000 IRFS. The deck level of the bridge is 27.16m AHD. The deck depth was 0.56 metres. A blockage of 5 percent was assumed underneath the deck, and 100 percent for the deck.

Photo 1 Patrick Street Bridge

The bridge and causeway structure was modelled in HEC RAS to verify losses. In the 100 year ARI event the peak flow is 497.6m^3/s. The tailwater level was set at 28.47m AHD. TUFLOW predicted a loss over the structure of 560mm. The HEC RAS model, set up using cross-sections extracted from the DTM predicted a loss of 510mm. Therefore TUFLOW is considered to provide a good representation of the losses at this structure.

The results of the analysis indicate that the bridge and causeway will not be trafficable even in minor flood events.

4.6.2 Swanbank Road Bridge

The Swanbank Road bridge is a three span concrete crossing. It is located at BUND25560. The deck level was assumed to be 30.0m AHD from the SKM 2000 IRFS. The deck depth was 0.20 metres. A blockage of 10 percent was assumed underneath the deck, and 100 percent for the deck.

The bridge structure was modelled in HEC RAS to verify losses. In the 100 year ARI event the peak flow is 470.7m^3/s. The tailwater level was set at 41.79m AHD. TUFLOW predicted a loss over of 350mm, just upstream of the structure. The HEC RAS model, set up using cross-sections extracted from the DTM predicted a loss of 80mm. The SKM 2000 Mike 11 model suggested a loss of 10mm across the structure. Therefore TUFLOW provides a slightly conservative assessment of the structure losses. Given that TUFLOW provides a better representation of the flow patterns in this area due to the 2D flow regime the predicted losses are considered acceptable.

The results of the analysis indicate that the bridge will not be trafficable even in minor flood events.

Photo 2 Swanbank Road Bridge

4.6.3 Ripley Road Bridge

The Ripley Road bridge is a three span concrete crossing. It is located at BUND18610. Based on available information, a road level of 47.08m AHD was adopted. The deck depth was 1.03 metres. A blockage of 5 percent was assumed underneath the deck, 100 percent for the deck and 25 percent for the handrails. As part of the development of the area this bridge and the surrounding roads will need to be upgraded so as to provide sufficient flood immunity.

The bridge structure was modelled in HEC RAS to verify losses. In the 100 year ARI event the peak flow in the main channel at this point is 272.3m^3/s (the total flow including the overbank areas is around 363m^3/s) . The tailwater level was set at 48.43m AHD. TUFLOW predicted a loss over of 330mm, just upstream of the structure. The HEC RAS model, set up using cross-sections extracted from the DTM predicted a loss of 740mm. The SKM 2000 Mike 11 model suggested a loss of 130mm across the structure. Therefore TUFLOW provides a result between these two values. Given that TUFLOW provides a better representation of the flow patterns in this area, particularly the lateral flows down Ripley Road the resultant losses are considered acceptable.

The results of the analysis indicate that the bridge will not be trafficable even in minor flood events.

Photo 3 Ripley Road Bridge

4.6.4 Barrams Road

The existing concrete slab causeway across the Bundamba Creek crossing of Barrams Road at BUND19760 was included in the TUFLOW model as a 2D structure. Details were taken from ground survey completed in the area. This crossing will be upgraded to provide sufficient flood immunity.

4.6.5 CHE Crossings

The existing CHE crossings (of Steel Mill Creek (STEE3130) and Bundamba Creek (BUND20590)) were included in the existing case TUFLOW model. Details for the crossings were taken from design drawings prepared by SKM (refer Appendix A). The CHE approaches were picked up in the DTM.

Bundamba Creek

The CHE crossing of Bundamba Creek is a 5 span concrete bridge with a total width of 137.5 metres. The minimum road level is 49.99m AHD. The deck depth was 1.5 metres. A blockage of 10 percent was assumed underneath the deck, 100 percent for the deck and 50 percent for the handrails.

The resultant water levels and affluxes match well with the results of the SKM 2006 analysis. The water level just upstream of the crossing in the main channel increases from 42.90 to 43.64m AHD, an afflux of 740mm. This compares well with the SKM 2006 analysis which reported an afflux of around 700mm just upstream of the bridge. The water levels used in the SKM 2006 analysis for the point just upstream of the bridge were 42.69m AHD in the existing case and 43.39m AHD with the CHE in place.

Steel Mill Creek

The CHE crossing of Steel Mill Creek is a two span concrete bridge with a total width of 54 metres. The minimum road level is 54.23m AHD. The deck depth was 1.5 metres. A blockage of 5 percent was assumed underneath the deck, 100 percent for the deck and 50 percent for the handrails.

The resultant water levels and affluxes match well with the results of the SKM analysis. The water level just upstream of the crossing increases from 47.41 to 47.68m AHD, an afflux of 270mm. This compares well with the SKM 2006 analysis which reported an afflux of around 160mm just upstream of the bridge (Bridge 2).

4.6.6 Existing Culverts

The existing culverts under Ripley Road and the CHE were included in the TUFLOW model as 1D culverts. Entrance and exist losses of 0.5 and 1.0 respectively were assumed. The details of the culverts were taken from ground survey completed in the area or from design drawings.

Ripley Road

Three sets of culverts are located across Ripley Road. The first is near the intersection of Ripley and Barrams Road and includes:

- 9/1200 by 750mm RCBCs
- 10.9 metres length
- Upstream invert level = 44.70m AHD
- Downstream invert level = 44.62m AHD

The second is just north of the intersection of Watsons Road and Ripley Road and includes:

- 3/1200 by 600mm RCBCs
- 11 metres length
- Upstream invert level = 45.68m AHD
- Downstream invert level = 45.65m AHD

The third is just south of the intersection of Watsons Road and Ripley Road and includes:

- 3/375mm RCPs
- 13 metres length
- Upstream invert level = 45.83m AHD
- Downstream invert level = 45.62m AHD

CHE

To drain the eastern floodplain of the CHE the following culvert was added:

- 1/1200mm RCPs
- 100 metres length
- Upstream invert level = 40.69m AHD
- Downstream invert level = 40.16m AHD

As this crossing provided minimal conveyance of flows in the overbank area it previously had not been included in the TUFLOW analysis. However, as minor flow events are also being investigated it was added to the model to determine its impact.

The SKM 2006 analysis of the CHE reported an afflux of 1310mm just upstream of the culvert. The TUFLOW analysis showed an afflux of 1260mm just upstream of this culvert. This afflux extends far upstream, as shown in the following image, with affluxes of around 1000mm experienced 500 metres upstream, around the location of Stage 1 works. At Barrams Road bridge, approximately 800 metres upstream the afflux is around 20mm in the main channel. The results of TUFLOW demonstrate that the reduction in peak flows afforded by the CHE was only minimal.

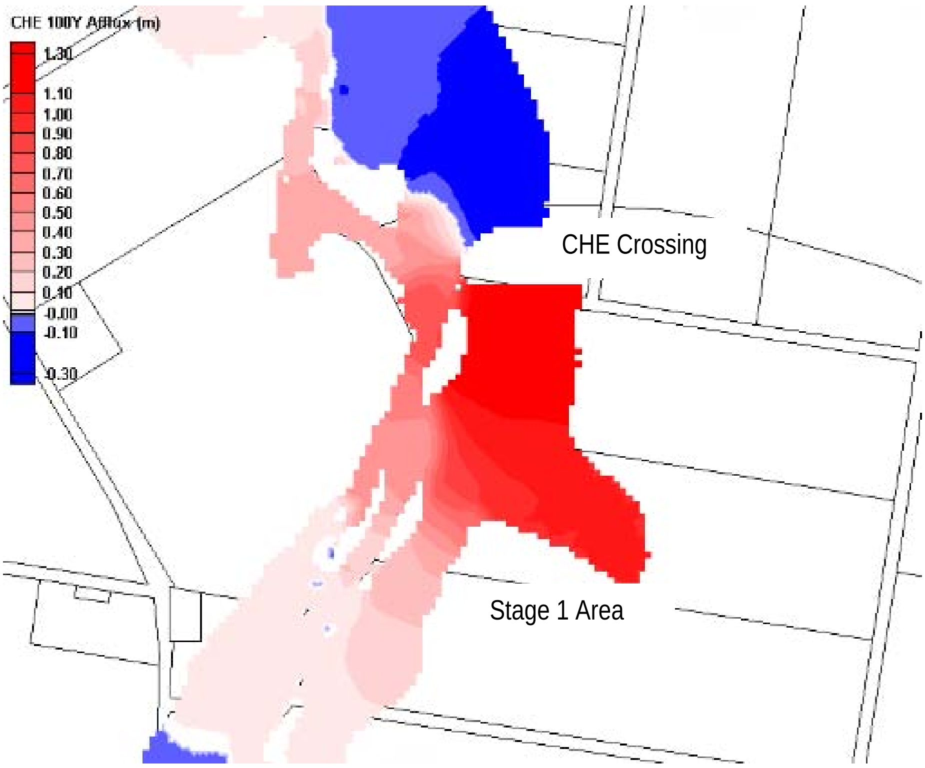


Image 1 Water Level Afflux due to CHE

Details for the western tributary culvert were not included. Flows in this area were input just downstream of the bridge. As a high bank exists between the main channel of Bundamba Creek and this western tributary it does not contribute to floodplain storage.

4.7 Downstream Tailwater Condition

The downstream tailwater condition was assumed to be a HQ rating curve automatically calculated by TUFLOW, based on the downstream cross-section, and assuming a bed slope of 0.5 percent. The location of the downstream boundary is shown in Figure 2.

TUFLOW is able to calculate the water level that would be the result of the specific flow at that point using the DTM and the assumed roughness values at that point.

In this case it should be noted that the assumed tailwater made negligible impact on results. This is because the Patrick Street bridge located just upstream of the boundary and the flow constriction provided by the high overbank areas in the downstream section of the model act as the main flow controls in this area, as shown by the sensitivity analysis of removing the bunds.

4.8 Model Runs

The existing case model was run for a range of storm durations ranging from 1 hour to 12 hours. The 1, 2, 3 and 4.5 hour events resulted in the peak water levels for various sections of Bundamba Creek. There was minimal difference in the results between the 1 and 2 hour events, and the 1 hour event was the peak event for only a small proportion of the model. Therefore the 2 hour event was used to determine the peak water levels for the upper reaches of the model. There was also little difference (around 50mm) between the 3 and 4.5 hour event peak water levels. However as the 3 hour event covers the area being developed both model it was kept as a model run. The 2, 3 and 4.5 hour events were therefore used to determine the peak water levels throughout the model. Previous XP-SWMM and SKM IRFS modelling only used the 4.5 hour event. The 100, 50, 10 and 5 year ARI events were run for all scenarios. Previous analysis have only run the 100 and 10 year ARI events.

4.9 Comparison Points

Numerous comparison points for peak water levels and velocities were set up in the TUFLOW model. The location of these points is shown in Figure 2. Numerous comparison lines (prefixed with Q_) were also set up to compare the peak flow results for various scenarios. These flow lines output the peak flow in the 2D domain. It should be noted that for some of the new structures the peak flows from the 1D culverts will be excluded from these results.

4.10 Existing Case Model Comparison

4.10.1 Comparison to SKM IRFS

The results of the existing case TUFLOW model, without the CHE in place, were compared to the results of the SKM IRFS Mike 11 model for the 100 year ARI event. The results are shown below in Table 3. The results from the TUFLOW model using only the 4.5 hour run (as used in the SKM IRFS Mike 11 model) and the maximum water level of all TUFLOW runs are shown in the table. It should be noted that the locations of the SKM cross-sections are slightly different to the cross-sections used in TUFLOW (refer Figure 2). To aid in the comparison of the models additional comparison points were set up at the original SKM Mike 11 cross-section locations.

Table 3 100 Year ARI Peak Water Level Comparison to IRFS

SKM Mike 11 Cross-section Location	SKM IRFS Existing Case	TUFLOW No CHE Existing Case (4.5 hour event)	Difference (mm)	TUFLOW No CHE Existing Case (all events)	Difference (mm)
BUND10000	81.57	82.20	629	82.56	993
BUND10615	77.50	77.28	-221	77.52	24
BUND11600	74.69	74.35	-340	74.71	24
BUND12705	69.91	69.85	-59	70.31	401
BUND13165	68.07	68.01	-57	68.27	200
BUND13940	62.39	62.82	426	63.46	1069
BUND14495	59.06	60.86	1800	61.05	1985
BUND15055	57.27	59.31	2036	59.41	2140
BUND15700	57.02	56.98	-40	57.09	74
BUND16395	55.06	55.51	448	55.70	636
BUND16900	53.39	53.82	429	53.95	558
BUND17530	51.80	52.56	755	52.64	836
BUND17885	50.50	51.14	638	51.23	731
BUND18730	48.36	49.00	635	49.04	679
BUND19280	46.15	46.47	319	46.58	427
BUND19800	44.32	45.19	865	45.26	936
BUND20440	42.88	42.88	-5	42.89	12
BUND21370	40.80	41.67	872	41.71	906
BUND22120	39.32	39.36	41	39.38	64
BUND22785	38.75	38.47	-283	38.48	-268
BUND23515	38.12	37.82	-297	37.84	-276
BUND24130	36.90	35.07	-1826	35.07	-1826
BUND24760	33.73	33.52	-212	33.52	-212
BUND25075	32.74	33.19	446	33.19	446
BUND25580	31.73	31.87	137	31.87	137
BUND26540	29.58	30.47	888	30.47	888
BUND26780	29.04	30.34	1304	30.34	1304
BUND27380	28.41	28.52	109	28.52	109

As shown in Table 3 there are significant differences between the two models, similar to the differences observed with the previous XPSWMM modelling. The difference in some areas is up to 2140mm increase in peak water level. The Browns Consulting 2006 review stated there was a degree of accuracy in the SKM 2000 Mike 11 model of ± 2.7 metres. At the location of the CHE (BUND 20440) there was only 12mm difference in peak water level. At the downstream section of the model, near where the SKM IRFS had been calibrated with recorded flood levels, the difference in peak water levels was only around 109mm.

The sections where there are the greatest differences between the two models are areas where the Mike 11 model overestimated the conveyance by incorrectly including the floodplain area. This was done as they did not define in the cross-sections where the top of the channel bank was. For example at BUND15055 a large floodplain of approximately 400 metres width is located on the eastern overbank. However the water level in the main channel has to reach around 59m AHD to overtop the banks. TUFLOW correctly contains the flow within the main channel. Only when the peak water level overtops the banks does the floodplain get included in the flow conveyance.

The most likely reason that the peak flow levels in the area from BUND22785 to BUND24760 have been reduced in the TUFLOW model is the changes to the UPBRANCH1 connection with Bundamba Creek, and the new constriction of the channel near the Swanbank Powerstation.

The TUFLOW model uses more recent and detailed survey of the area and better represents the effect of the floodplain, of levees along the main channel and the interactions between the various flow paths. It is therefore considered a more accurate model of Bundamba Creek.

The area of Ripley Valley being developed extends from around BUND17000 to BUND23500. In this area the TUFLOW model generally reports higher peak water levels.

4.11 Comparison to May 2009 Event

There is one river height and rainfall gauge located within the study area, the Ripley Alert gauge at 27.72˚S and 152.81˚E (around BUND16478). The SKM IRFS did not use this gauge to calibrate the Mike 11 model as information was not available for the chosen calibration events. Instead they used river height gauges at the downstream end of Bundamba Creek. These gauges are located outside of the TUFLOW model extent and therefore could not be used for calibration of the model.

The accuracy of the Ripley Alert gauge is questionable, with ICC being unsure of the correct datum level. It has been known to fail regularly, in particular during the recent rainfall event in January 2011. ICC is in the process of applying to have more gauges installed in the catchment.

The May 2009 flood event was run in TUFLOW and compared to the reported levels at the Ripley Alert gauge. The reported peak water level was 54.30m AHD. The peak water level at BUND16478 was 54.75m AHD. It is not known whether the Bundamba Lagoon overflowed during this event. A scenario was run assuming that it did not overflow. The resultant peak water level at BUND16478 was 54.26m AHD.

Recorded water marks were also taken during the May 2009 flood event at various points around the catchment. However, at this time they have not yet been properly surveyed. Once this data becomes available further verification and calibration of the model will be possible.

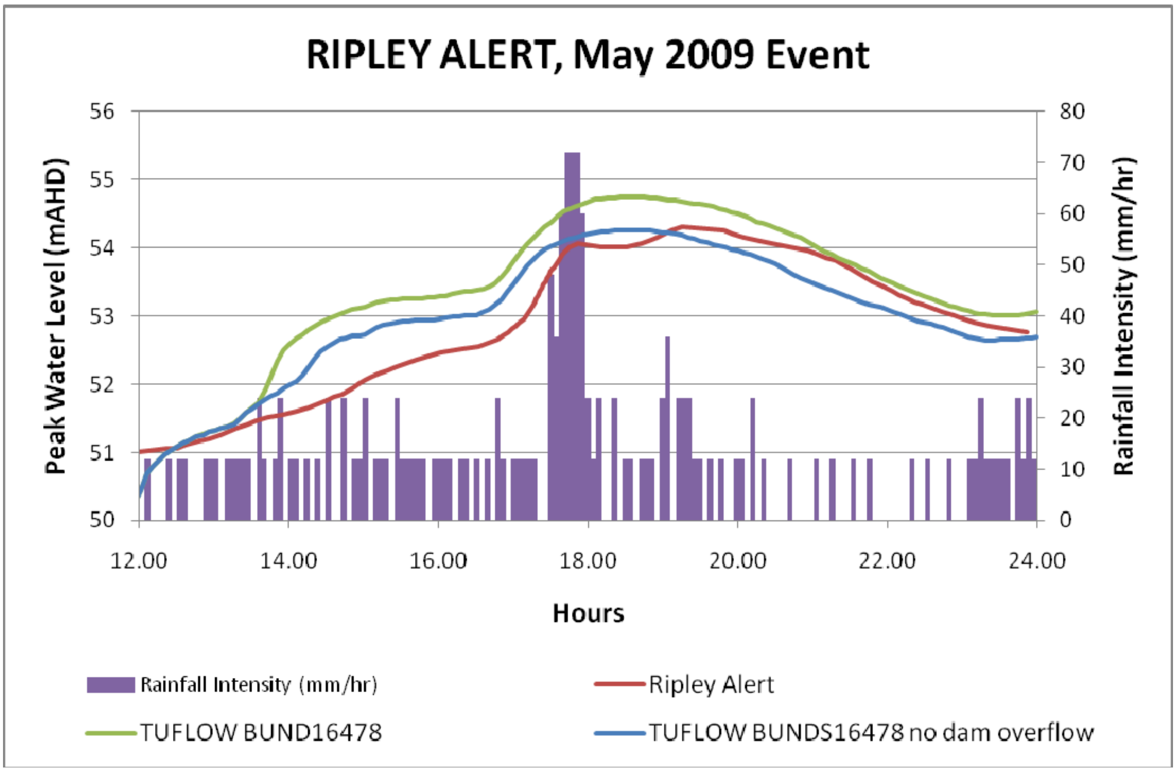


Image 2 May 2009 Flood Event

As the graph shows, the TUFLOW model predicts slightly higher peak water levels, and a greater volume of flow than the Ripley Alert gauge reported at this location. From the results it appears that the lagoon started to overflow at some point during the storm event. The initial overestimation of flow could be due to attenuation provided by farm dams located throughout the catchment. It should be noted that the WBNM inflows assumed that Bundamba Lagoon was full prior to the storm. It is not known whether this was the case prior to the May 2009 flood event. It is therefore considered that the TUFLOW model provides a conservative estimate of peak water levels.

The results of TUFLOW and an IFD frequency analysis indicate that the May 2009 event was roughly equivalent to a 10 year ARI event. During this event 213mm of rainfall was recorded at the gauge over a period of 24 hours. However this longer duration event is not the critical storm duration for the catchment.

5 DEVELOPED CASE TUFLOW MODEL SETUP

5.1 Proposed Structures

Additional bridge crossings of Bundamba Creek are necessary to provide adequate access to the SUCE areas being developed. Apart from the CHE the existing crossings of Bundamba Creek do not have sufficient flood immunity, and are overtopped during the 5 year ARI event. The proposed bridge crossings will also provide some attenuation of flows and compensate for the filling proposed within the floodplain.

For Stage 1 works an upgrade of the Barrams Road crossing is necessary. Details of the proposed structure are shown in Figure 5.

In addition to the new crossing at Barrams Road, the ultimate works will include the upgrade of the Ripley Road bridge to improve its flood immunity as well as increase its width, and allow for four lanes of traffic. Similarly, Ripley Road to the north of the crossing, as well as Watsons Road will need to be upgraded. A new four lane road to the west of Ripley Road, north of Watsons Road, is also proposed. Ripley Road and the new road will require 50 year flood immunity. Watsons Road will be upgraded to provide it with 10 year flood immunity. Additional culvert crossings beneath these new roads will be required so that existing flow patterns are replicated. The details for these works have not been included in the analysis for Stage 1 works.

5.1.1 Barrams Road Bridge

The proposed Barrams Road crossing includes a four-span bridge of 100 metres total width across Bundamba Creek. The assumed width of the bridge is 30 metres. The bridge is located at BUND19760. The proposed bridge was modelled in 2D, and the culverts were modelled in 1D.

The minimum road level is 46.50m AHD, which provides it with 100 year ARI flood immunity. The deck depth is approximately 1.5 metres. A blockage of 10 percent was assumed underneath the deck, 100 percent for the deck and 50 percent for the handrails.

It should be noted that this deck level is lower than used in the previous analysis, so as to comply with the visual aesthetic requirements of the neighbouring resident. Previous analysis had the bridge soffit level above the 100 year ARI flood level so that the deck did not impede the flow conveyance.

5.1.2 Barrams Road Culverts

For the western tributary it is proposed to have the following culverts:

- 3/4200 by 2700mm RCBCs
- 40 metres length (at an angle)
- Upstream invert level = 42.90m AHD
- Downstream invert level = 42.50m AHD

The area just upstream of these culverts will have to be excavated by around 500mm so that the slope of the culverts can be reduced. The headwall will also need to rounded to reduce the entrance losses at this point. An entrance loss of 0.2 and exit loss of 1.0 were used.

A channel of approximately 20 metres width and 1 metre deep will be provided to link to the culverts of Ripley Road. The channel will be vegetated, except for a rock-lined low flow channel. The alignment of the channel was designed to minimise disturbance to existing vegetation.

For the eastern floodplain the following culverts are required:

- 5/3300 by 2100mm RCBCs
- 30 metres length
- Upstream invert level = 43.70m AHD
- Downstream invert level = 43.50m AHD

These culverts will be located approximately 30 metres to the west of the existing low point in the floodplain. This is to allow for future development of this area for sportsgrounds. To ensure that flows can be directed towards these culverts some minimal excavation upstream will be required. Downstream of these culverts a rock-lined channel is proposed to convey flows to the main channel of Bundamba Creek. The channel will be approximately 25 metres wide and half a metre deep. The purpose of this channel is to divert low flows away from future sportsgrounds areas that will be located downstream of the bridge. During major flows the channel will overtop and flow will also be conveyed across the eastern floodplain. The headwall will also need to be rounded to reduce the entrance losses at this point. An entrance loss of 0.2 and exit loss of 1.0 were used.

5.2 Barrams Road HEC RAS Analysis

The main bridge crossing and the culverts were modelled in HEC RAS to verify the losses over the structures. Cross-sections for use in HEC RAS were extracted from the DTM.

Previous modelling of this area in Mike 11 had resulted in lower peak water levels due to incorrect inclusion of the floodplains in the main channel conveyance.

The 100 year ARI peak flow is 274.8m^3/s in the main channel, 63.8m^3/s in the western tributary and 48.5m^3/s in the eastern floodplain. The tailwater level was set at 45.2m AHD. The resultant HEC RAS afflux was 130mm in the main channel, 230mm in the western tributary and 210mm in the eastern floodplain.

The TUFLOW affluxes at this point were 250mm in the main channel, 200mm in the western tributary and 520mm in eastern floodplain. The results match well with the results of HEC RAS, and are slightly conservative. The larger affluxes shown in the eastern floodplain are thought to be due to the channel excavation works which are not picked up in the HEC RAS analysis.

5.3 Culvert Blockage Sensitivity Analysis

A TUFLOW case was run that assumed 25 percent blockage of the Barrams Road culverts. Given the large size of these culverts it is unlikely that this degree of blockage will occur. The results of the analysis showed that an afflux of around 110mm occurs upstream of the western tributary culverts and an afflux of around 200mm occurs upstream of the eastern floodplain culverts. The afflux dissipates to around 0mm 100 metres upstream of the western culverts and around 350 metres upstream of the eastern culverts.

5.4 Stage 1 Filling

The area of the Stage 1 works will be filled above the 100 year ARI flood level. To allow for battering, a 10 metre buffer around the boundary of these works was also assumed to be filled.

The playground area proposed on the northern side of Barrams Road will be provided with a minimum of 5 year ARI immunity. To ensure that it is not frequently inundated it was assumed to be raised to 44.3 m AHD. The results of the TUFLOW modelling showed that with the Stage 1 works in place the playground would have around 20 year ARI flood immunity. TUFLOW analysis showed that the 10 year ARI flood levels in this area were around 43.8m AHD. To ensure that it has sufficient immunity with future works completed the higher level of 44.3m AHD was used. It is expected that it will be possible to lower this level following further analysis.

6 TUFLOW MODEL RESULTS

6.1 TUFLOW Model Scenarios

The 100, 50, 10 and 5 year ARI events were modelled, using the 2, 3 and 4.5 hour storm durations for each event.

The following scenarios were run in TUFLOW:

Case 1. No CHE – existing case but without the CHE crossings in place.

Case 2. Existing Case - the existing case scenario with the CHE crossings in place.

Case 3. BarrBr - Barrams Road bridge and culverts added to determine its impact.

Case 4. BarrBr_Stg1 - filled Stage 1 area and playground, developed case inflows used. Barrams Road bridge also upgraded.

6.2 TUFLOW Results

6.2.1 Peak Water Levels

The peak water levels at several reporting points around the Stage 1 development are shown below in Tables 4 and 5.

Table 4 100 and 50 Year ARI Peak Water Levels

Location	Ground /Floor Level	100 Year ARI Peak Water Level (m AHD)				50 Year ARI Peak Water Level (m AHD)			
		No CHE	Ex Case	Barr Br	Barr Br & Stg1	No CHE	Ex Case	Barr Br	Barr Br & Stg1
BLDG1	46.22 (floor)	45.35	45.37	45.63	45.64	45.13	45.15	45.23	45.23
BLDG2	45.95 (floor)	46.09	46.09	46.09	46.08	45.86	45.85	45.84	45.83
BLDG3	46.59	47.01	47.01	47.01	47.01	46.77	46.76	46.77	46.77
BLDG4	47.60	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
BLDG5	48.21	48.33	48.33	48.33	48.33	48.27	48.27	48.27	48.27
BLDG6	50.54	51.04	51.04	51.04	51.04	50.90	50.89	50.90	50.90
BLDG7	55.16	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
us_BarrL u/s western culverts		45.35	45.36	45.56	45.56	45.13	45.14	45.25	45.18
RB1		45.46	45.46	45.72	45.73	45.34	45.34	45.34	45.34
RB2		45.31	45.31	45.70	45.70	45.02	45.02	45.17	45.17
RB3		45.10	45.12	45.64	45.64	44.85	44.85	45.13	45.13
RB4		44.53	44.67	44.66	44.76	44.12	44.14	44.29	44.39
RB5		44.92	44.95	45.05	45.06	44.77	44.78	44.81	44.81
RB6		44.26	44.47	44.46	44.47	43.81	43.91	44.02	44.05
RB7		43.63	44.17	44.15	44.16	43.24	43.75	43.78	43.79
RB8		43.09	44.09	44.07	44.07	42.69	43.73	43.74	43.74
RB9		43.03	44.08	44.06	44.06	42.67	43.72	43.74	43.74

Location	Ground /Floor Level	100 Year ARI Peak Water Level (m AHD)				50 Year ARI Peak Water Level (m AHD)			
		No CHE	Ex Case	Barr Br	Barr Br & Stg1	No CHE	Ex Case	Barr Br	Barr Br & Stg1
RB10		42.90	44.07	44.05	44.05	42.56	43.72	43.73	43.73
RB11		42.84	44.07	44.05	44.05	42.53	43.71	43.73	43.73
BUND18847		47.15	47.16	47.18	47.17	46.96	46.96	46.97	46.97
BUND18986		47.12	47.12	47.14	47.14	46.93	46.92	46.93	46.93
BUND19131		46.97	46.98	47.00	47.00	46.75	46.75	46.76	46.76
BUND19395		45.92	45.94	46.06	46.05	45.62	45.62	45.67	45.67
BUND19710		45.35	45.38	45.63	45.64	45.14	45.15	45.24	45.24
BUND19760 Barrams Rd		45.34	45.37	45.62	45.63	45.13	45.14	45.21	45.21
BUND19805		45.25	45.27	45.19	45.19	45.03	45.05	44.96	44.96
BUND19940		45.17	45.20	45.11	45.11	44.96	44.98	44.89	44.89
BUND20161		44.19	44.38	44.35	44.36	44.11	44.18	44.14	44.14
BUND20433		43.43	43.96	43.93	43.93	43.36	43.64	43.63	43.64
BUND20527		42.91	43.67	43.64	43.64	42.85	43.34	43.34	43.34
BUND20570 CHE		42.90	43.64	43.62	43.62	42.83	43.33	43.33	43.33
BUND20618		42.89	43.31	43.29	43.29	42.82	43.10	43.09	43.10
BUND20717		42.65	43.02	43.01	43.01	42.57	42.85	42.84	42.85
BUND20864		42.55	42.88	42.87	42.87	42.46	42.69	42.69	42.69

Table 5 10 and 5 Year ARI Peak Water Levels

Location	Ground /Floor Level	10 Year ARI Peak Water Level (m AHD)				5 Year ARI Peak Water Level (m AHD)			
		No CHE	Ex Case	Barr Br	Barr Br & Stg1	No CHE	Ex Case	Barr Br	Barr Br & Stg1
BLDG1	46.22 (floor)	44.96	44.96	N/A	N/A	N/A	N/A	N/A	N/A
BLDG2	45.95 (floor)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
BLDG3	46.59	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
BLDG4	47.60	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
BLDG5	48.21	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
BLDG6	50.54	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
BLDG7	55.16	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
us_BarrL u/s western culverts		44.46	44.46	44.69	44.69	44.38	44.38	44.26	44.27
RB1		45.28	45.28	45.28	45.28	45.25	45.25	45.25	45.25
RB2		44.85	44.85	44.89	44.89	44.79	44.79	44.82	44.82
RB3		44.69	44.69	44.80	44.80	44.63	44.63	44.72	44.72
RB4		43.91	43.91	43.81	43.81	43.87	43.87	N/A	N/A
RB5		N/A	N/A	44.29	44.29	N/A	N/A	43.98	43.97
RB6		43.51	43.51	43.34	43.34	43.46	43.46	N/A	43.01
RB7		43.02	43.02	42.89	42.89	42.98	42.98	N/A	N/A
RB8		42.20	42.88	42.47	42.50	42.15	42.76	42.03	42.06

Location	Ground /Floor Level	10 Year ARI Peak Water Level (m AHD)				5 Year ARI Peak Water Level (m AHD)			
		No CHE	Ex Case	Barr Br	Barr Br & Stg1	No CHE	Ex Case	Barr Br	Barr Br & Stg1
RB9		N/A	42.88	42.47	42.50	N/A	42.76	N/A	N/A
RB10		41.98	42.88	42.47	42.50	41.88	42.76	42.03	42.06
RB11		41.97	42.88	42.47	42.50	41.86	42.76	42.03	42.06
BUND18847		46.29	46.29	46.31	46.31	45.88	45.88	45.90	45.89
BUND18986		46.22	46.22	46.24	46.24	45.76	45.76	45.78	45.78
BUND19131		46.04	46.04	46.06	46.06	45.58	45.57	45.61	45.60
BUND19395		44.85	44.86	44.95	44.95	44.48	44.49	44.60	44.60
BUND19710		44.40	44.42	44.56	44.56	44.04	44.05	44.22	44.22
BUND19760 Barrams Rd		44.39	44.41	44.52	44.52	44.01	44.02	44.19	44.18
BUND19805		44.27	44.30	44.33	44.33	43.88	43.90	44.01	44.00
BUND19940		44.21	44.24	44.27	44.27	43.81	43.83	43.95	43.93
BUND20161		43.61	43.66	43.69	43.69	43.29	43.33	43.38	43.38
BUND20433		42.94	43.07	43.06	43.06	42.72	42.80	42.83	42.83
BUND20527		42.58	42.76	42.73	42.73	42.43	42.54	42.56	42.55
BUND20570 CHE		42.57	42.76	42.73	42.73	42.42	42.54	42.55	42.55
BUND20618		42.56	42.64	42.62	42.62	42.41	42.45	42.46	42.46
BUND20717		42.34	42.41	42.39	42.39	42.20	42.24	42.25	42.24
BUND20864		42.25	42.32	42.31	42.31	42.10	42.13	42.15	42.14

Impact of CHE

As shown in these tables the impact of the CHE on peak water levels was quite significant, particularly in the eastern floodplain overbank area (points RB1 to RB11), where the afflux was as much as 1230mm in the 100 year ARI event. Around the Stage 1 works the increase was around 1000mm. At Barrams Road the afflux was around 20mm. At building 1 the increase was around 10mm.

Impact of Barrams Road Bridge

The impact of Barrams Road bridge in the 100 year ARI event is an increase of 250mm just upstream of the bridge in the main channel, an increase of around 520mm in the eastern floodplain just upstream of the bridge, an increase of 200mm just upstream of the western culverts and an increase of 260mm at building 1. No impacts are observed at any of the other buildings. The afflux in the main channel dissipates to around 20mm around 600 metres upstream of the bridge. Water levels downstream of the bridge are reduced by up to 90mm. At the CHE the peak water levels are reduced by 20mm.

For smaller ARI events the impact of the bridge is not as great.

For the 10 year ARI event the ground level at building 1 is no longer inundated due to the impact of the channel that will be excavated upstream of the Barrams Rd culverts.

Downstream of the bridge in the eastern floodplain the peak water levels are reduced for the 10 and 5 year ARI events. This is due to the impact of the excavated channel. This reduction in peak water levels in this area will increase the usability of this area for future sportsgrounds.

Impact of Stage 1 Works

The impact of Stage 1 works, which included using developed case flows has shown to be negligible in regards to peak water levels. In the area just adjacent to the fill the peak water level increases by around 100mm. However this does not extend far and does not impact the main channel of Bundamba Creek.

6.2.2 Peak Flows

The peak flows at several reporting points around the Stage 1 development are shown below in Tables 6 and 7.

Table 6 100 and 50 Year ARI Peak Flows

Flow Comparison Line	Location	100 Year ARI Peak Flow (m^3/s)				50 Year ARI Peak Flow (m^3/s)			
		No CHE	Ex Case	Barr Br	Barr Br & Stg1	No CHE	Ex Case	Barr Br	Barr Br & Stg1
Q_BUND18420		363.6	363.3	363.7	363.5	292.7	290.9	291.9	291.7
Q_BUND18630	d/s of Ripley Rd Bridge	321.1	321.0	321.0	321.0	271.4	269.6	271.0	271.0
Q_BUND18910		340.6	340.9	340.5	340.7	281.9	280.0	281.5	281.4
Q_BUND19260		370.5	370.4	370.0	370.0	298.6	296.5	297.8	297.7
Q_BUND19460		387.7	387.9	386.2	386.2	309.7	307.7	309.0	309.2
Q_BUND19740		387.6	387.1	341.9	342.5	311.4	309.2	281.2	288.2
Q_BUND19790	Barrams Rd Bridge	387.4	386.8	255.6	255.4	311.2	308.9	241.2	241.2
Q_BUND20030	d/s of Barrams Rd Bridge	388.2	387.1	379.5	379.9	312.2	310.0	307.7	307.8
Q_BUND20360		386.2	383.5	377.7	377.8	309.6	305.3	305.1	305.2
Q_BUND20590		388.8	386.1	379.7	379.5	311.1	306.8	306.3	306.4
Q_BUND20615	u/s of CHE	388.6	382.3	375.9	375.8	311.1	303.1	302.8	302.9
Q_BUND20680		387.7	385.2	378.7	378.7	310.3	305.8	305.5	305.7
Q_BUND20880		391.2	388.5	382.2	382.4	313.2	308.9	308.0	308.5
Q_BUND20990		390.6	387.8	381.1	381.1	312.2	307.6	307.1	307.3
Q_BUND21440		391.0	388.4	381.6	381.7	312.4	307.9	306.8	307.2
Q_BUND21570		390.3	387.9	381.5	381.3	312.9	308.9	307.2	307.6
Q_BUND21700		390.6	388.2	381.0	381.5	312.1	307.5	306.8	307.3
Q_BUND21990		388.3	385.4	379.0	378.9	309.2	304.8	303.9	304.4
Q_BUND22300		462.3	456.7	447.3	447.3	367.9	361.3	359.5	360.4
Q_BUND22740		459.9	453.8	445.2	445.9	367.7	361.3	359.1	360.0
Q_BUND23030		466.4	459.7	451.3	451.8	373.5	368.7	365.8	366.7
Q_BUND23290		467.3	460.5	453.8	454.3	376.1	370.9	367.9	368.9
Q_BarrRd_L	Barrams Rd – left tributary	64.4	63.8	52.8	52.5	45.6	45.0	36.2	35.9

Flow Comparison Line	Location	100 Year ARI Peak Flow (m^3/s)				50 Year ARI Peak Flow (m^3/s)			
		No CHE	Ex Case	Barr Br	Barr Br & Stg1	No CHE	Ex Case	Barr Br	Barr Br & Stg1
Q_BarrRd_R	Barrams Rd – right floodplain	47.8	48.5	61.1	61.3	21.9	21.9	21.6	21.6
Q_RipleyBr	Ripley Rd bridge	272.3	272.3	272.4	272.3	254.1	253.2	253.9	254.0

Table 7 10 and 5 Year ARI Peak Flows

Flow Comparison Line	Location	10 Year ARI Peak Flow (m^3/s)				5 Year ARI Peak Flow (m^3/s)			
		No CHE	Ex Case	Barr Br	Barr Br & Stg1	No CHE	Ex Case	Barr Br	Barr Br & Stg1
Q_BUND18420		176.9	176.9	176.9	176.9	147.2	147.5	147.5	147.2
Q_BUND18630	d/s of Ripley Rd Bridge	175.1	174.9	175.0	175.0	147.0	146.9	147.2	146.9
Q_BUND18910		175.7	175.9	176.0	176.0	146.9	146.7	146.9	146.8
Q_BUND19260		180.6	180.8	181.2	181.2	150.9	150.7	150.9	150.6
Q_BUND19460		186.5	186.6	187.5	187.5	156.7	156.5	156.5	156.5
Q_BUND19740		193.8	193.9	191.7	191.7	162.5	162.2	158.3	160.0
Q_BUND19790	Barrams Rd Bridge	193.6	193.8	176.1	176.1	162.3	162.3	146.8	146.7
Q_BUND20030	d/s of Barrams Rd Bridge	193.1	193.3	197.4	197.4	161.9	162.1	159.9	161.3
Q_BUND20360		194.0	193.6	191.2	191.2	162.6	159.9	160.9	160.1
Q_BUND20590		195.6	195.3	189.5	189.5	163.8	160.3	161.7	161.4
Q_BUND20615	u/s of CHE	195.5	192.4	187.4	187.4	163.5	157.3	159.9	159.5
Q_BUND20680		195.1	194.9	189.5	189.5	162.6	159.9	161.2	160.7
Q_BUND20880		197.1	196.6	191.5	191.5	163.7	160.9	162.3	161.8
Q_BUND20990		196.7	196.1	191.3	191.3	163.2	160.2	161.7	161.5
Q_BUND21440		196.5	195.6	190.6	190.6	161.9	157.7	160.2	159.8
Q_BUND21570		196.9	195.8	190.8	190.8	161.6	156.8	159.0	158.9
Q_BUND21700		196.2	195.1	189.9	189.9	161.1	156.3	158.5	158.0
Q_BUND21990		194.5	194.1	189.1	189.1	160.4	155.4	157.4	157.3
Q_BUND22300		229.9	226.4	223.2	223.2	185.6	178.2	182.0	182.0
Q_BUND22740		228.4	224.5	221.3	221.3	183.7	176.6	180.3	180.0
Q_BUND23030		230.3	226.3	225.5	225.5	185.1	178.4	181.8	182.0
Q_BUND23290		230.3	226.7	225.8	225.8	185.2	178.6	182.4	182.6
Q_BarrRd_L	Barrams Rd – left tributary	24.2	24.2	17.5	17.5	18.8	18.8	13.5	13.5
Q_BarrRd_R	Barrams Rd – right floodplain	12.2	12.2	11.8	11.8	9.4	9.4	9.3	9.3
Q_RipleyBr	Ripley Rd bridge	174.9	174.9	174.8	174.8	145.8	145.7	146.0	145.8

Impact of CHE

As shown in Tables 6 and 7 the 100 year ARI peak flows are reduced by around 6m^3/s downstream of the bridge. Although the peak flows downstream of the bridge had been decreased as the majority of flow is now directed to the main channel of Bundamba Creek the peak water levels had increased slightly. Peak water levels downstream of the eastern approach were reduced.

Impact of Barrams Rd Bridge

As shown in Tables 6 and 7 the 100 year ARI peak flows are reduced by around 6m^3/s downstream of the bridge. This resulted in a reduction in peak water levels downstream of the bridge.

Impact of Stage 1 Works

As shown in Tables 6 and 7 the impact of Stage 1 works was negligible in regards to peak flows. Given that Barrams Road bridge provides some attenuation of peak flows the impact of the works can be said to be beneficial to downstream properties.

6.2.3 Peak Velocities

The peak velocities at several reporting points around the development are shown below in Tables 8 and 9.

Table 8 100 and 50 Year ARI Peak Velocities

Comparison Point	Location	100 Year ARI Peak Velocity (m/s)				50 Year ARI Peak Velocity (m/s)			
		No CHE	Ex Case	Barr Br	Barr Br & Stg1	No CHE	Ex Case	Barr Br	Barr Br & Stg1
BUND18847		2.44	2.44	2.43	2.44	2.43	2.43	2.42	2.37
BUND18986		0.86	0.86	0.86	0.86	0.85	0.85	0.85	0.85
BUND19131		1.57	1.57	1.55	1.55	1.55	1.55	1.54	1.54
BUND19395		0.34	0.35	0.51	0.51	0.56	0.58	0.53	0.53
BUND19710	u/s of Barrams Rd	1.31	1.31	1.22	1.22	1.26	1.25	1.24	1.24
BUND19940		1.00	1.00	0.91	0.91	1.16	1.16	0.89	0.89
BUND20161		2.88	2.89	2.81	2.81	2.80	2.72	2.70	2.71
BUND20433		1.58	1.34	1.38	1.38	1.57	1.49	1.48	1.49
BUND20570	CHE	1.75	1.15	1.31	1.31	1.69	1.13	1.31	1.30
BUND20717		1.80	2.01	2.00	2.00	1.83	1.91	1.92	1.91
BUND20864		1.34	1.58	1.58	1.58	1.31	1.50	1.49	1.50
us_BarrL	upstream of western culverts	1.28	1.28	0.53	0.52	1.27	1.27	0.98	0.98
RB1		0.68	0.68	0.47	0.47	0.61	0.61	0.60	0.60
RB2		0.44	0.44	0.31	0.30	0.34	0.34	0.29	0.29
RB3		1.09	1.08	0.68	0.68	0.86	0.86	0.48	0.48
RB4		1.03	0.92	1.14	1.05	0.85	0.83	1.08	0.97
RB5	channel d/s of eastern floodplain culverts	1.48	1.50	0.59	0.58	1.01	1.08	0.73	0.73

Comparison Point	Location	100 Year ARI Peak Velocity (m/s)				50 Year ARI Peak Velocity (m/s)			
		No CHE	Ex Case	Barr Br	Barr Br & Stg1	No CHE	Ex Case	Barr Br	Barr Br & Stg1
RB6		1.11	1.03	1.06	1.22	0.75	0.66	0.80	0.94
RB7		1.67	1.19	1.21	1.37	1.02	0.58	0.80	0.88
RB8		0.01	0.01	0.01	0.01	0.02	0.00	0.01	0.00
RB9		0.70	0.42	0.41	0.40	0.44	0.22	0.24	0.25
RB10		0.58	0.33	0.32	0.32	0.34	0.21	0.22	0.22
RB11		0.65	0.34	0.33	0.33	0.40	0.23	0.24	0.24

Table 9 10 and 5 Year ARI Peak Velocities

Comparison Point	Location	10 Year ARI Peak Velocity (m/s)				5 Year ARI Peak Velocity (m/s)			
		No CHE	Ex Case	Barr Br	Barr Br & Stg1	No CHE	Ex Case	Barr Br	Barr Br & Stg1
BUND18847		2.33	2.32	2.30	2.30	2.39	2.39	2.38	2.38
BUND18986		0.83	0.83	0.82	0.82	0.85	0.85	0.85	0.85
BUND19131		1.46	1.46	1.41	1.41	1.57	1.58	1.55	1.55
BUND19395		0.63	0.62	0.51	0.51	0.53	0.55	0.53	0.52
BUND19710	u/s of Barrams Rd	1.10	1.09	1.07	1.07	1.28	1.29	1.22	1.22
BUND19940		1.34	1.34	0.83	0.83	1.50	1.50	0.90	0.90
BUND20161		2.67	2.61	2.62	2.62	2.85	2.85	2.77	2.77
BUND20433		1.50	1.31	1.44	1.44	1.61	1.57	1.51	1.49
BUND20570	CHE	1.30	1.11	1.19	1.19	1.13	1.73	1.30	1.30
BUND20717		1.75	1.77	1.94	1.94	1.96	1.88	1.96	1.96
BUND20864		1.49	1.42	1.37	1.37	1.57	1.37	1.55	1.55
us_BarrL	upstream of western culverts	1.21	1.21	0.77	0.77	1.21	1.21	1.10	1.10
RB1		0.53	0.53	0.53	0.53	0.52	0.52	0.51	0.52
RB2		0.29	0.29	0.25	0.25	0.26	0.26	0.24	0.24
RB3		0.69	0.69	0.51	0.51	0.66	0.66	0.47	0.47
RB4		0.66	0.66	0.44	0.43	0.56	0.56	N/A	N/A
RB5	channel d/s of eastern floodplain culverts	N/A	N/A	0.21	0.18	N/A	N/A	0.70	0.70
RB6		0.47	0.47	0.28	0.31	0.41	0.41	N/A	0.04
RB7		0.55	0.55	0.26	0.28	0.46	0.46	N/A	N/A
RB8		0.02	0.00	0.00	0.00	0.02	0.00	0.00	0.00
RB9		N/A	0.05	0.01	0.02	N/A	0.04	N/A	N/A
RB10		0.08	0.04	0.01	0.01	0.07	0.02	0.01	0.01
RB11		0.17	0.04	0.02	0.02	0.30	0.02	0.01	0.01

Impact of CHE

As shown in Tables 8 and 9 the 100 year ARI peak velocities are reduced upstream of the CHE by around 0.2m/s and increased downstream of the bridge by around 0.2m/s. Upstream of the eastern approach the peak velocities are reduced by around 0.3m/s.

Impact of Barrams Rd Bridge

As shown in Tables 8 and 9 the 100 year ARI peak velocities are reduced by around 0.1m/s just upstream of the bridge and reduced by around 0.1m/s downstream of the bridge.

The peak velocity in the channel upstream of the western culverts is reduced, with a maximum of 1.1m/s in the 5 year ARI event. For the channel downstream of the eastern culverts (RB5) the peak velocity is around 0.7m/s. These velocities are not considered high enough to cause scour issues. However, the inlets and outlets of the culverts will be rock-lined to prevent erosion. As high velocities are experienced on the top right bank of the eastern channel this area will also need to be rock-lined.

Impact of Stage 1 Works

The Stage 1 works have negligible impact on peak velocities.

7 RESPONSE TO SUPPLEMENTARY INFORMATION REQUEST QUERIES

The requests for information received from ULDA in relation to flooding are addressed below.

> *Provide detailed information on the development, calibration and verification for the flood model used.*

Details on the development, calibration and verification of the flood model are provided in this report (refer Sections 3 and 4). The model has been calibrated to match a 100 year ARI peak flow of 369m^3/s at the CHE The resultant peak flow in the TUFLOW model at this point was 386m^3/s, therefore the model is considered to be conservative. Calibration to the Ripley Alert gauge showed that the predicted water levels are slightly higher than what was reported at this point. However, given that it is unknown whether the Bundamba Lagoon was overflowing a conservative over-estimate of peak water levels is considered reasonable. Further survey regarding recorded flood levels will be required. When this information is available further verification of the flood model will be possible.

As it is, the TUFLOW model predicts generally much higher flood levels around the Ripley Valley development area than previously calculated in the SKM 2000 IRFS. This is mainly because the conveyance of the floodplains was erroneously included in the Mike 11 model.

The extent of the model was determined by the survey data provided. The model extends approximately 7 kilometres upstream and 4 kilometres downstream of all development in the area. This is considered to be sufficient to fully model the extent of any impacts from the development in the area.

> *The calibration of the flood model should include use of flood levels recorded/obtained from the 2008 and 2009 floods from landholders and/or ICC.*

Further survey regarding recorded flood levels is will be required. When this information is available further verification of the flood model will be possible.

> *Any use of flood gauges should include an assessment of the accuracy of the flood gauge and justification of any significant deviation from the recorded shape of the hydrographs. Also, details of initial losses used and their justification are required.*

The May 2009 event was rerun assuming that Bundamba Lagoon does not overtop. It was found that the peak water levels were slightly less than what was recorded at the gauge. As the analysis assumes that the lagoon does overtop it is considered to provide a conservative estimate of peak water levels and flows.

The initial and continuing losses adopted for the May 2009 event were 50mm and 2.5mm/hr. The high initial loss had minimal impact on the peak flows and water levels. For design events it is not usual to use high initial losses as it should be assumed that the catchment has been wetted by precedent rainfall (as was the case before the peak of the May 2009 storm event occurred).

The shape of the hydrograph matches fairly well with what was predicted by TUFLOW, particularly for the timing of the main rise and fall. It appears that the Bundamba Lagoon may have started to overtop at some stage during the event which dragged out the peak of the flow. Prior to the peak water level occurring TUFLOW appears to overestimate the water levels. This could be because the intensity of the rainfall was not consistent over the catchment and/or due to attenuation provided by existing farm dams.

The accuracy of the Ripley Alert gauge is questionable, with ICC being unsure of the correct datum level. It has been known to fail regularly, in particular during the recent rainfall event in January 2011. ICC is in the process of applying to have more gauges installed in the catchment.

> *At the resubmission stage provide a full copy of the TUFLOW model with results (all cases) to enable a full assessment of model assumptions.*

A full copy of the TUFLOW model will be provided with this report.

> *The report identifies that Ripley Road would only have an immunity of 50 year ARI. The issue of flood evacuation and risk for the housed population needs addressing.*

The upgrade of this road is beyond the scope of Stage 1 works. However, the 50 year flood immunity for Ripley Road was the advised flood immunity required by Council. The existing road has minimal flood immunity. Barrams Road, due to the bridge upgrade, will have greater than 100 year ARI immunity which will provide a secondary access/egress point.

> *The report indicates that a bridge of 100m length was assumed with a bridge deck above the 100 year ARI flood levels. No further details of the bridge are provided.*

The total bridge span is 100 metres. Due to the requirements of the neighbouring resident the bridge deck has been lowered to a minimum of 46.5m AHD (the previous analysis assumed the soffit level was above the 100 year ARI level). Further details of the bridge structure are provided in this report. Although ICC only required 50 year ARI immunity for the bridge it will not be overtopped during a 100 year ARI event.

The form loss co-efficient assumed for the bridge piers was 0.1. A blockage factor of 10 percent was also used.

> *The report indicates that floodplain culverts with a total width of 29.1m were assumed. The infrastructure report provides the following comment "either side of the bridge, banks of box culverts approx 2.5m high and totalling up to 25m in width will be required (see Flooding Report)". Details on how these culverts fit into the road embankment are required.*

The total width of floodplain culverts is 29.1 metres. Details on how these culverts fit into the road embankment are shown on the civil design drawings.

> *The report indicates that the "headwall will also need to be rounded to reduce the entrance losses". What losses were assumed and how are these justified?*

The entry and exit losses assumed were 0.2 and 1.0 respectively, these are the standard loss values for rounded entrances. HEC RAS modelling showed that the resultant affluxes reported by TUFLOW were slightly conservative.

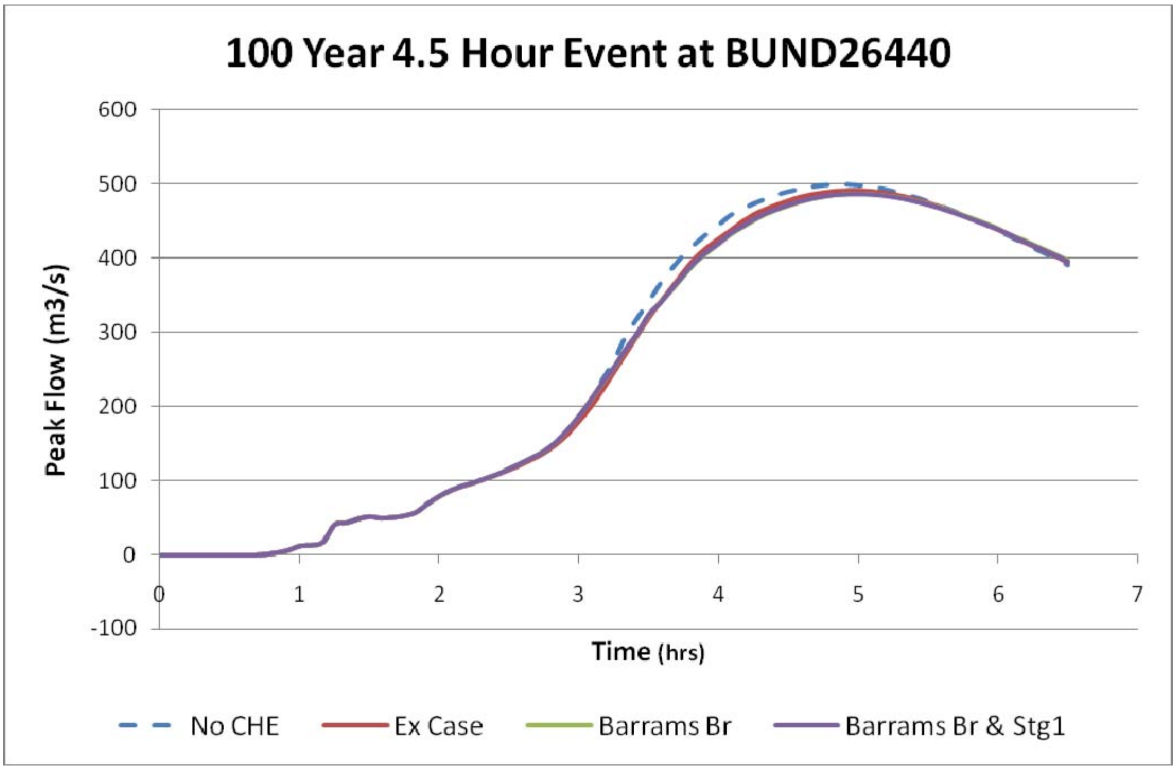
The report indicates flow would pass through floodplain culverts under Barrams Road. What is the impact of culvert blockage on the flooding behaviour, especially at the three houses upstream.

A TUFLOW case was run that assumed 25 percent blockage of the Barrams Road culverts. Given the large size of these culverts it is unlikely that this degree of blockage will occur. The results of the analysis showed that an afflux of around 110mm occurs upstream of the western tributary culverts and an afflux of around 200mm occurs upstream of the eastern floodplain culverts. The afflux dissipates to around 0mm 100 metres upstream of the western culverts and around 350 metres upstream of the eastern culverts. Therefore only building 1 would be impacted upon by blockage of the culverts. However, sufficient flood immunity would still be provided.

The study does not provide any assessment of the impacts to the shape of the flood hydrograph. It is not clear whether rates of floodwater rise or recession are affected (or are volumes increased).

Previous reporting had looked at the impact of development on the timing of the peak flows along the length of Bundamba Creek. This assessment showed that filling in the catchment was offset by the attenuation provided by the northern Bundamba Creek crossing (not included in this study). Revegetation works along the banks of Bundamba Creek also provided some attenuation of peak flows, and increased the response time at the downstream end of Bundamba Creek.

A flow hydrograph at a downstream location (BUND26440) showing the four cases run in this study is shown below. As this graph demonstrates the construction of the CHE provided some minimal attenuation of the peak flow at the downstream end of the model. The construction of Barrams Road bridge and Stage 1 works (including developed case flows) had negligible further impact.



The flood study does not appear to include an assessment of changes to discharge characteristic from the site (as assessed and defined by the SWMP).

Previous analysis of the developed cases has assumed that existing case flows were used, as detention basins will be provided throughout the site. The purpose of these detention basins will be to attenuate peak flows back to the existing case. For Stage 1 of the SUCE works the detention basin will not be constructed. Due to the small area being developed it is not expected that this will impact on the peak flows in Bundamba Creek. However, to ensure that this is the case the WBNM model was modified with Stage 1 in place. The percentage impervious used for Stage 1 was 70 percent. The following modifications were therefore made:

- Catchment D2 is now 6.1% impervious;
- Catchment E3 is now 18.6% impervious; and
- Catchment E2 is now 2.1% impervious.

The assumption that the park will be above the 5 year ARI flood level is not supported in the report but stated.

The 5 year ARI case had previously not been run, and the park level had been conservatively estimated using 10 year ARI levels. To ensure that it is not frequently inundated the playground was set at 44.3 m AHD. TUFLOW analysis has since shown that the 5 year ARI flood levels in this area are around 43.7m AHD, and the playground has around 20 year ARI immunity. Therefore the playground level could be lowered if necessary.

The contributing catchment area and delineation has not been included in the report.

As this had been shown in previous reports it was not provided for the Stage 1 summary report. It is included with this report as Figure 1.

Delineation of subcatchments has not been included and assessment of typical subcatchment parameters has been omitted from the methodology. For example, area, impervious proportion, slope, catchment roughness

As this had been addressed in previous reports it was not provided for the Stage 1 summary report. Details of the subcatchment parameters are shown in this report.

Details of the adopted hydrologic modelling software and assessment methodology have not been included.

As this had been addressed in previous reports it was not provided for the Stage 1 summary report. Details of the software used is provided in this report.

Hydrologic parameters that are typically input to a hydrologic model have not been included. For example, rainfall depths, rainfall losses (initial and continuing) and temporal patterns. If standard procedures in AR&R were adopted this should be stated and the calculated values listed.

As this had been addressed in previous reports it was not provided for the Stage 1 summary report. Details of the hydrological parameters used are provided in this report.

No information is available regarding the assessment of critical storm duration for Bundamba Creek either at the location of the development or at its outlet

As this had been addressed in previous reports it was not provided for the Stage 1 summary report. Details of the critical storm durations are provided in this report. The 3 hour event results in the peak water levels around the Stage 1 development.

It is not clear if consideration has also been given to the ultimate developed catchment given extensive future urbanisation planned within the Ripley Valley UDA Boundary which will significantly increase the proportion of impervious area.

The existing case flows are to be used in the ultimate case as detention basins are to be adopted throughout the development. Inclusion of the impact of the detention basins using the developed case flows will be addressed when design of the detention basins has been finalised. For this analysis Stage 1 was assumed to be developed. The resultant impact on peak flows was shown to be minimal.

Sensitivity testing for potential future flows under higher rainfall conditions has not been included in the report.

Previous reporting has looked at the potential impact of climate change. The results of the analysis are summarised below.

The development of Ripley Valley provides a unique opportunity to plan and accommodate for the impacts of any future climate change. Most climate scientists agree that average temperatures will increase in the future. The projected range of temperature increases is 0.4 to 2.0℃ in 2030 (relative to 1990) and 1.0 to 6.0℃ in 2070 (Hughes, 2003). There is much uncertainty as to what impact this will have on rainfall. However, it is generally agreed that although the average annual rainfall is likely to decrease, the occurrence of extreme rainfall events is likely to increase. This is because higher temperatures will increase the amount of water vapour available in the air, which increases globally by 7 percent per degree of temperature change (Jakob, 2009). The consequence of this is that there will potentially be more rainfall during major storm events.

The federal government is currently funding an update to Australian Rainfall and Runoff, which will address how flood modellers should account for future climate change. As the results of this review are not yet available, a simple sensitivity analysis was considered the best approach. Two scenarios were therefore run:

- *10 percent increase in rainfall*
- *20 percent increase in rainfall*

The results showed that if the 100 year ARI rainfall is increased by 10 percent then peak water levels will on average increase by 140mm. Similarly, a 20 percent increase in rainfall will raise peak water levels by 270mm. The impact of the increased rainfall varies, being more pronounced where the floodplain width is reduced.

The existing case flood levels determined for Bundamba Creek are considered to be a conservative estimate It was assumed that Bundamba Lagoon would be full prior to the storm event. However, as climate change will probably result in a drier climate (a decrease in average annual rainfall) this is less likely to occur in the future. Therefore it will have more capacity to provide attenuation of peak flows.

This flood study has also looked at the impact of revegetating the creek corridors. Manning's n roughness values of 0.20 and 0.10 were used. Using the higher roughness values resulted in peak water levels approximately 250mm higher than with a roughness value of 0.10. The lower roughness values will most likely be adopted, but the higher roughness values will be used to set the minimum development levels. This conservative assumption, coupled with the likely increased storage capacity of Bundamba Lagoon, will ensure that there is sufficient freeboard to account for the future impact of climate change.

Climate change is likely to increase the peak flows in Bundamba Creek, which could increase the frequency of flooding for existing properties at the downstream end of the creek. The construction of the northern Bundamba Bridge crossing and the revegetation of the creek corridors (even with a lower roughness value) will significantly reduce the peak flows at the downstream end of the model. This will help to reduce the impact of climate change for the existing properties.

> *It is not clear if any calibration to historic events or verification of the hydrologic model has been undertaken to validate the hydrologic model outputs.*

Details of the calibration of the model are provided in this report. Further verification will be undertaken once recorded flood levels are surveyed.

> *It is not clear what data has been used to generate the DTM for the hydrologic model.*

As this had been addressed in previous reports it was not provided for the Stage 1 summary report. Details of the DTM used are provided in this report.

> *No information is provided on the setup of the hydraulic model.*

As this had been addressed in previous reports it was not provided for the Stage 1 summary report. Details of the hydraulic model setup are provided in this report. Figure 2 also shows the TUFLOW model setup.

> *No information is provided on the Barrams Road crossing for the existing case.*

As this had been addressed in previous reports it was not provided for the Stage 1 summary report. Details of the existing Barrams Road crossing are provided in this report.

> *Proposed Structures. Details of how the hydraulic structures at Barrams Road have been represented in the model have not been included.*

As this had been addressed in previous reports it was not provided for the Stage 1 summary report. Details of the proposed Barrams Road crossing are provided in this report.

> *Blockage. Debris blocking of structures is a critical factor in their performance. An appreciation of the susceptibility of structures to blockage does not appear to have been undertaken.*

Blockage sensitivity analysis has been undertaken.

> *One potential limitation of modelling structures in a 2D/1D domain is that entrance and exit losses may be under or over estimated.*

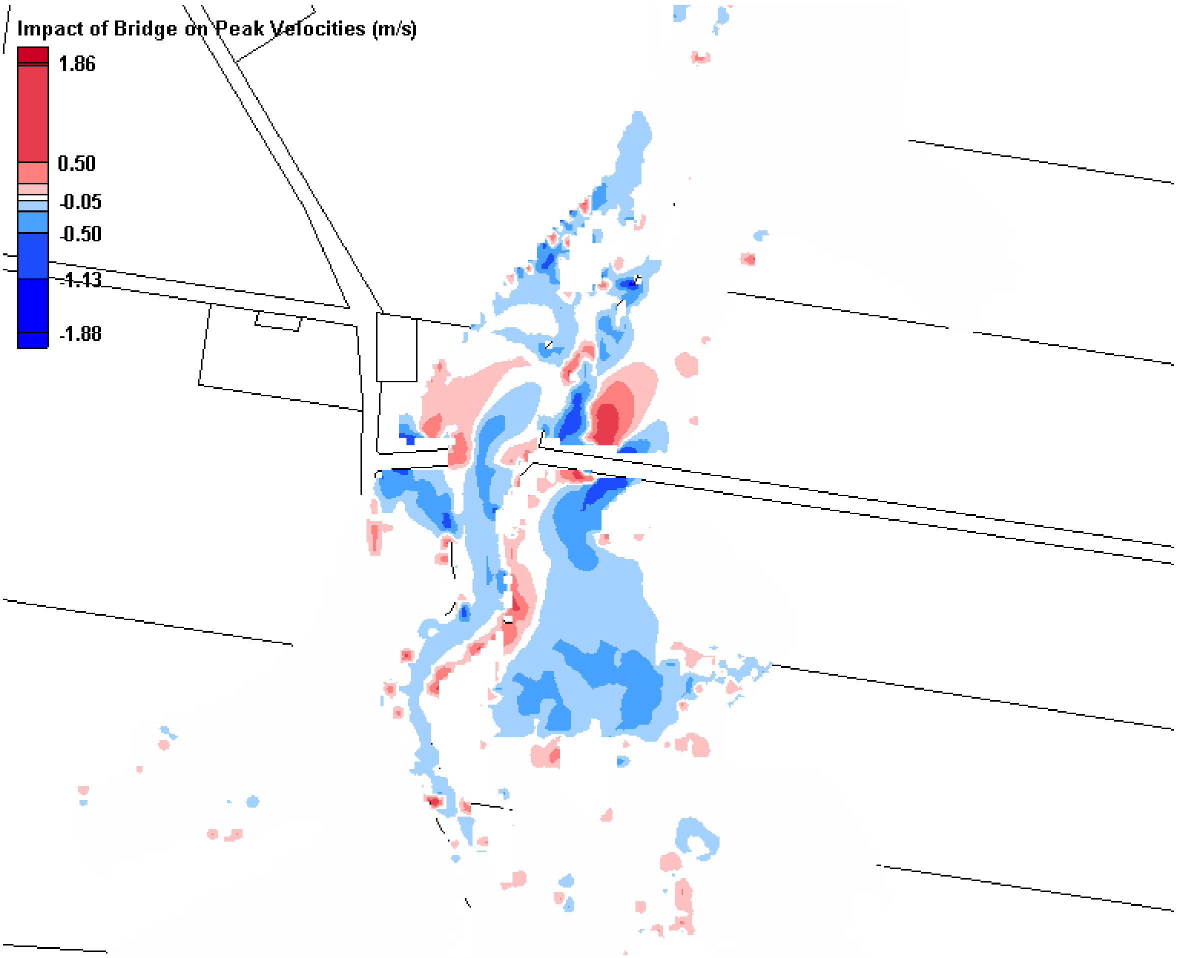
The affluxes at all structure were verified using HEC RAS for all structures. The details are provided in this report. TUFLOW generally slightly overestimates the losses at the structures compared to the results of HEC RAS. As TUFLOW provides more detailed representation of flow patterns, particularly the interactions between the main channel and overbank areas, these higher losses were considered to be a reasonable and conservative assumption.

> *QUDM requires that hydraulic models be calibrated where possible against historical flood events. No information is provided regarding the calibration of the TUFLOW model.*

Details of the calibration to date are provided in this report.

> *Overbank. The subject site is known to comprise dispersive soils which are readily eroded thus increasing the likelihood of scour if left unprotected. Image 6 in the flooding report indicates that overbank velocities downstream of the bridge on the eastern side are increased in the 100 year ARI event of the preferred solution (Case 3).*

Following recent discussions and agreement with surrounding land owners the compensatory excavation proposed in this overbank area is no longer necessary and has been removed from revised proposals. The revised impact on peak velocities is shown below.



The increase in velocities that is observed on the eastern overbank is because the bridge increases the extent of inundation during the 100 year ARI event in this area. Therefore although the peak velocities in this area are fairly low (0.1 to 0.7m/s) the afflux is large as this area previously was not inundated. This area is not inundated during the 50 year ARI event. Given the infrequent nature of flows in this area, and the low velocities observed, no scour protection works are considered necessary. However future revegetation of this overbank area will help stabilise this area.

The peak velocities downstream of the culverts are around 0.8m/s in the western tributary and 0.7m/s in the proposed eastern channel. These velocities are not considered high enough to be of concern. The increase in velocities observed downstream of the eastern culverts occurs where the flows overtop the constructed channel. This area will need to be rock-lined to avoid erosion.

Increased flows. As discussed in the hydrology review above, there is potential for higher flows in Bundamba Creek due to planned urban expansion and future rainfall conditions. This may increase flood levels such that the bridge soffit becomes submerged in the 100 year ARI event. This may result in additional impacts on upstream property owners.

The soffit level of Barrams Road bridge was previously assumed to be above the 100 year ARI level for all future scenarios, and so the previous design still provided sufficient flood immunity at Building 1 even with a 30 percent increase in rainfall intensities. However, following discussions with neighbouring residents the deck and soffit level has been reduced to 45.0m AHD. It is therefore submerged in the 100 year ARI event. As a sensitivity analysis a case was run with an increase in rainfall intensity of 10 percent. The results show that due to the lower deck level the water level increases by around 630mm at Building 1 (compared to the existing case without the bridge). This means the building would have a freeboard of around 230mm. Without climate change the increase due to the bridge and Stage 1 works is around 260mm, which provides 590mm of freeboard to the floor level. The afflux does not extend far, from building 2 to building 7 the afflux due to climate change is only between 150 and 30mm, which is within the expected range.

Given that it is a requirement of the resident of the affected buildings to lower the level of Barrams Road bridge, and that the degree of climate change is speculative and beyond the control of the developer it is considered reasonable to allow this degree of afflux, which would occur in around 50 years time. New buildings within the area will be provided with sufficient freeboard to account for climate change and rehabilitation of the creeks.

Detention basins are proposed throughout the development, and so flows are not expected to increase significantly due to development within the valley. Once details of the detention basins proposed are available further analysis of the impact will be possible.

Barrams Road bridge is still trafficable during a 100 year ARI event, even with a 10 percent increase in rainfall intensity. As Barrams Road bridge was only required to be designed to have 50 year ARI immunity this is considered to be a good outcome.

Mitigation measures. Increased runoff from the development described in item 4.1 may adversely impact on downstream flooding. The report does not explain how increased runoff volume and peak flows will be managed

Detention basins will be provided throughout the development. The increased impervious area from Stage 1 was shown in this report to result in negligible impacts to peak flows and runoff volumes.

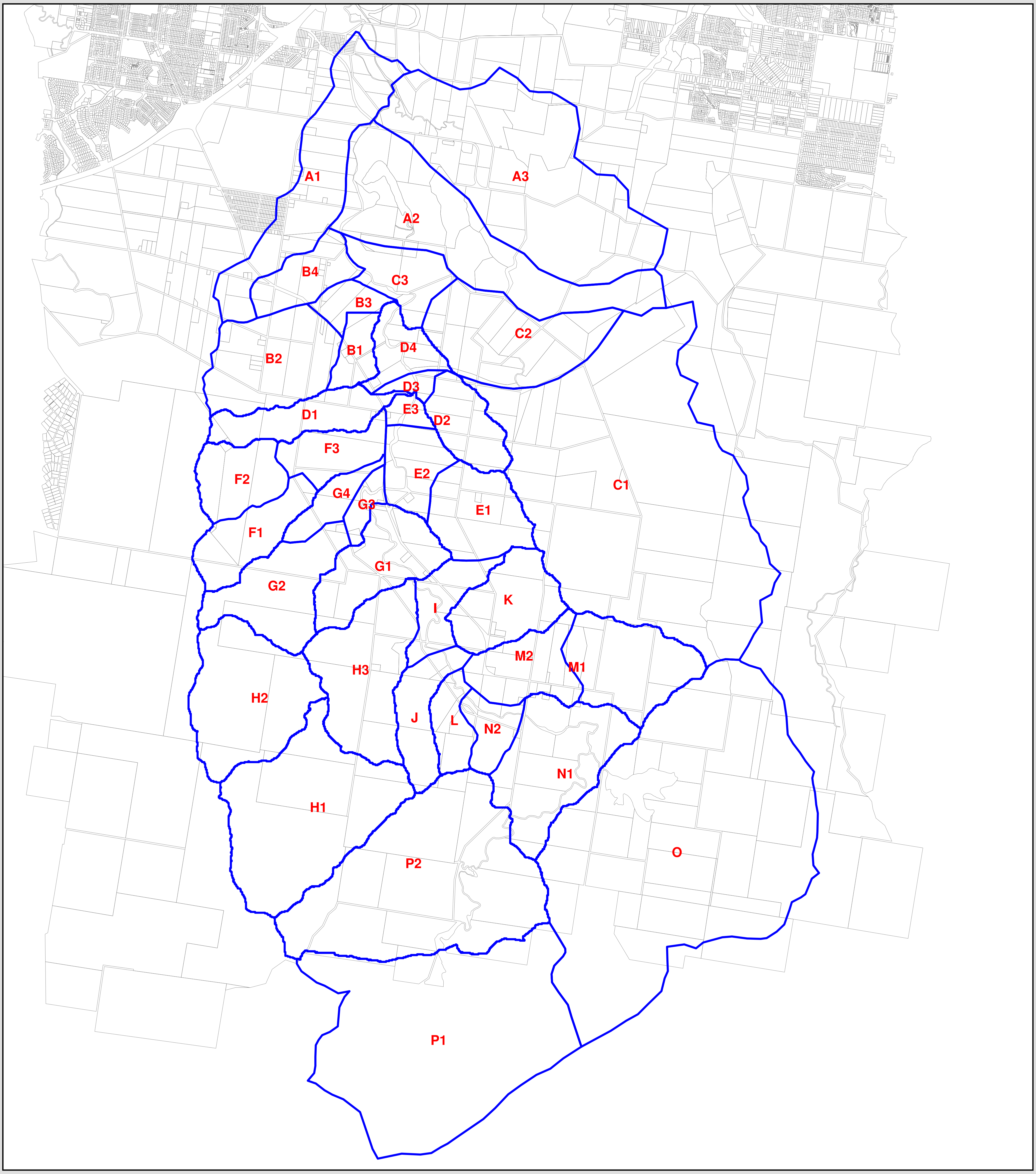
Flood Risk Management. The report does not contain any references to the management of flood risks and hazard.

These details and figure were provided in a previous report, and not in the summary report for Stage 1. Barrams Road bridge will not be inundated during the 100 year ARI event. Any additional requirements in relation to flood hazard can be conditioned.

8 CONCLUSION

TUFLOW modelling has shown that the impact of constructing Barrams Road bridge and Stage 1 works is minimal in terms of peak water levels, flows and velocities. At Building 1 the increase in water level is 260mm, but 590mm of freeboard is still provided to the floor level. At the other buildings the impact is negligible. The construction of the bridge provides some minimal attenuation of peak flows, and so flood levels for properties downstream will be improved.

Figures



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LEGEND

Catchment Boundary

DCDB

N W E S

Figure 1
Catchment Areas

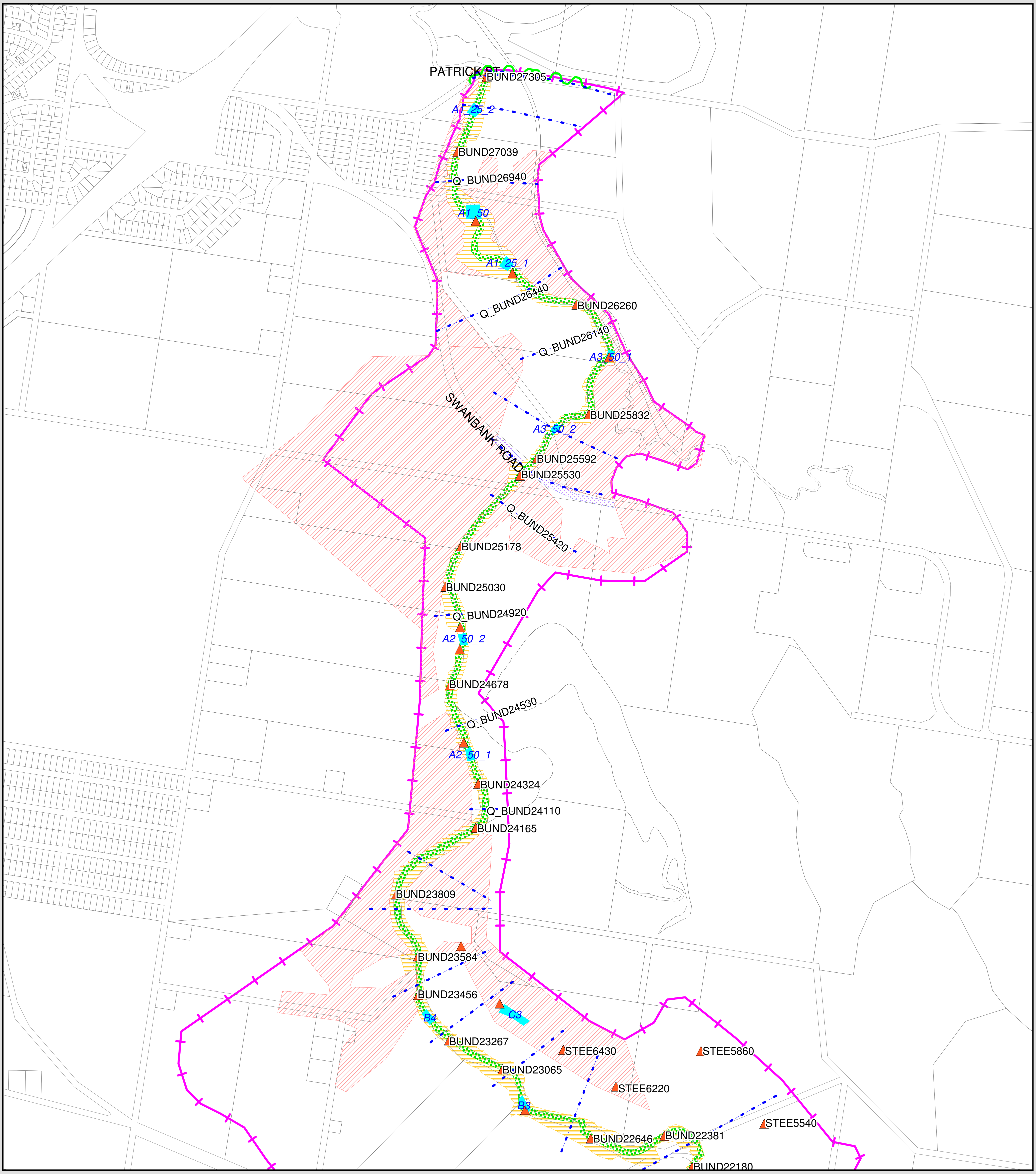
SUCE Stage 1

Flood Study

Scale: 1:50,000

500 0 500 1000 1500

Metres



LEGEND

N W E

TUFLOW Model Extent

Downstream Boundary

TUFLOW Results Output

Peak Water Level and Velocity

Peak Flow

Inflow Point

Existing Roughness Values

- channel banks (n=0.10)
- waterway area (n=0.05)
- medium vegetation (n=0.10)
- dense vegetation (n=0.15)
- roads (n=0.03)
- all other areas (n=0.05)

Figure 2.1
TUFLOW Model Setup

SUCE Stage 1

Flood Study

Scale: 1:12,500

125 0 125 250 375

Metres

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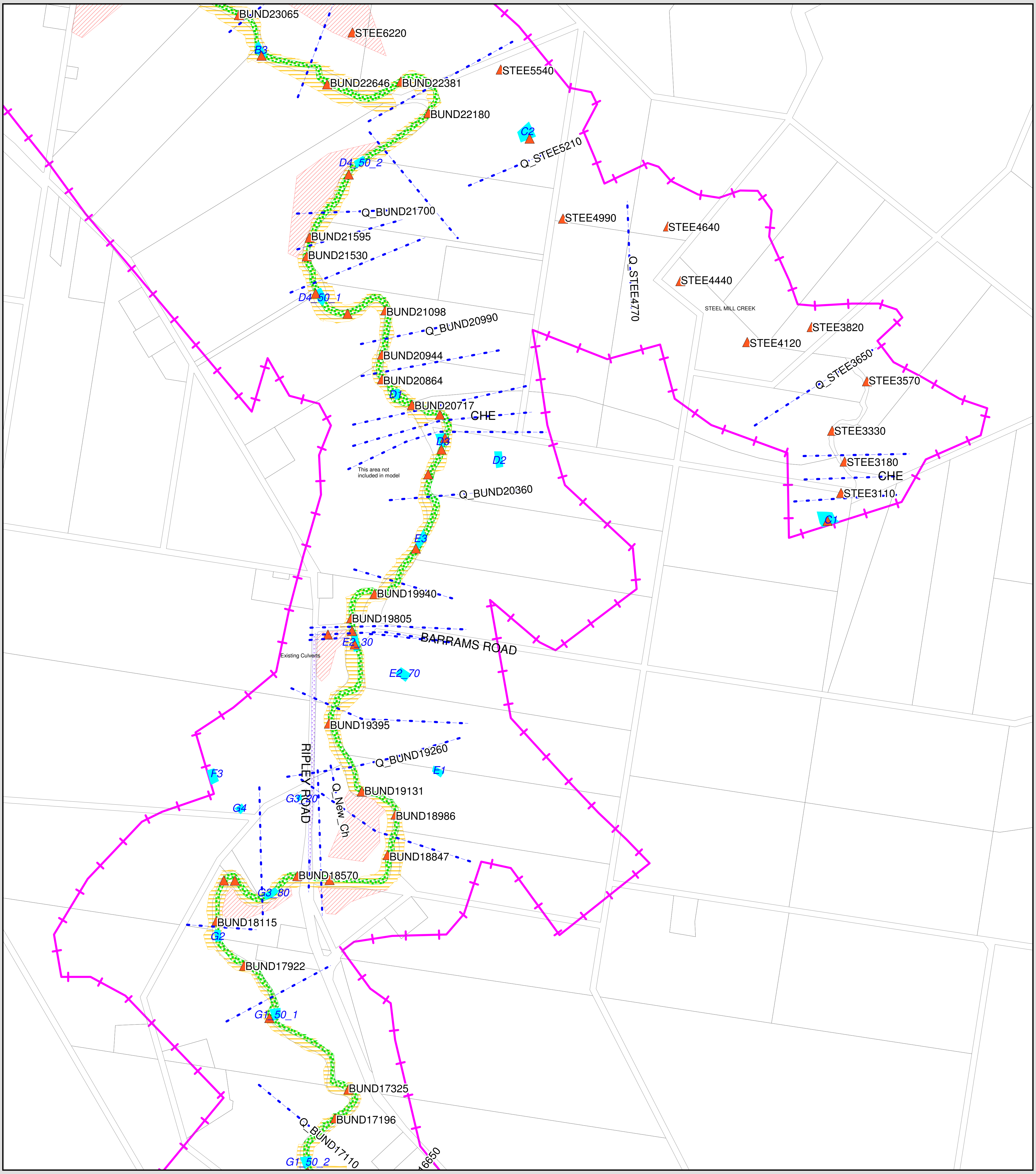

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LEGEND

N W E

TUFLOW Model Extent

Downstream Boundary

TUFLOW Results Output

Peak Water Level and Velocity

Peak Flow

Inflow Point

Existing Roughness Values

- channel banks (n=0.10)
- waterway area (n=0.05)
- medium vegetation (n=0.10)
- dense vegetation (n=0.15)
- roads (n=0.03)
- all other areas (n=0.05)

Figure 2.2
TUFLOW Model Setup

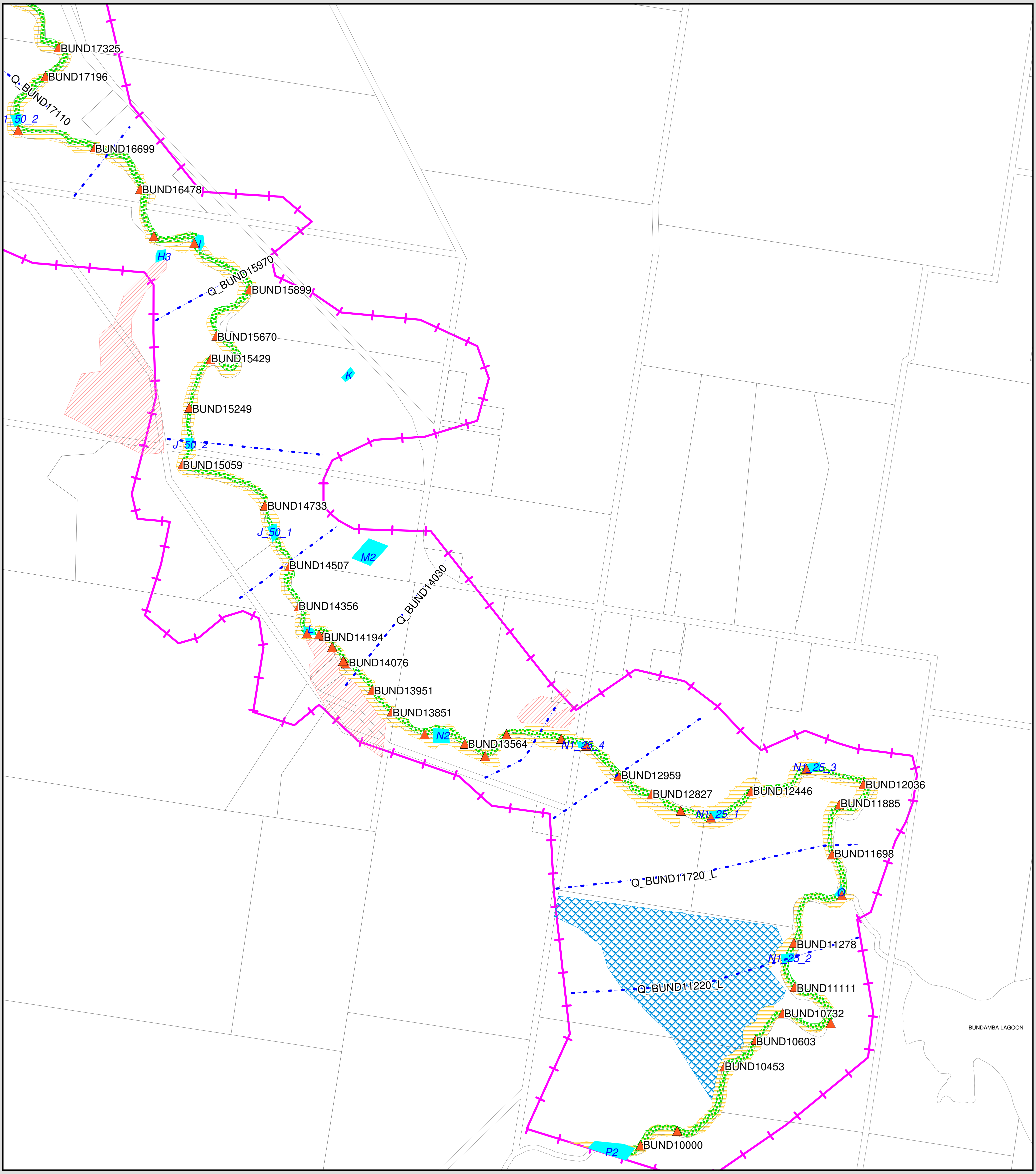
SUCE Stage 1

Flood Study

Scale: 1:12,500

125 0 125 250 375

Metres



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LEGEND

N W E

- TUFLOW Model Extent
- Downstream Boundary

TUFLOW Results Output

- Peak Water Level and Velocity
- Peak Flow
- Inflow Point

Existing Roughness Values

- channel banks (n=0.10)
- waterway area (n=0.05)
- medium vegetation (n=0.10)
- dense vegetation (n=0.15)
- roads (n=0.03)
- all other areas (n=0.05)

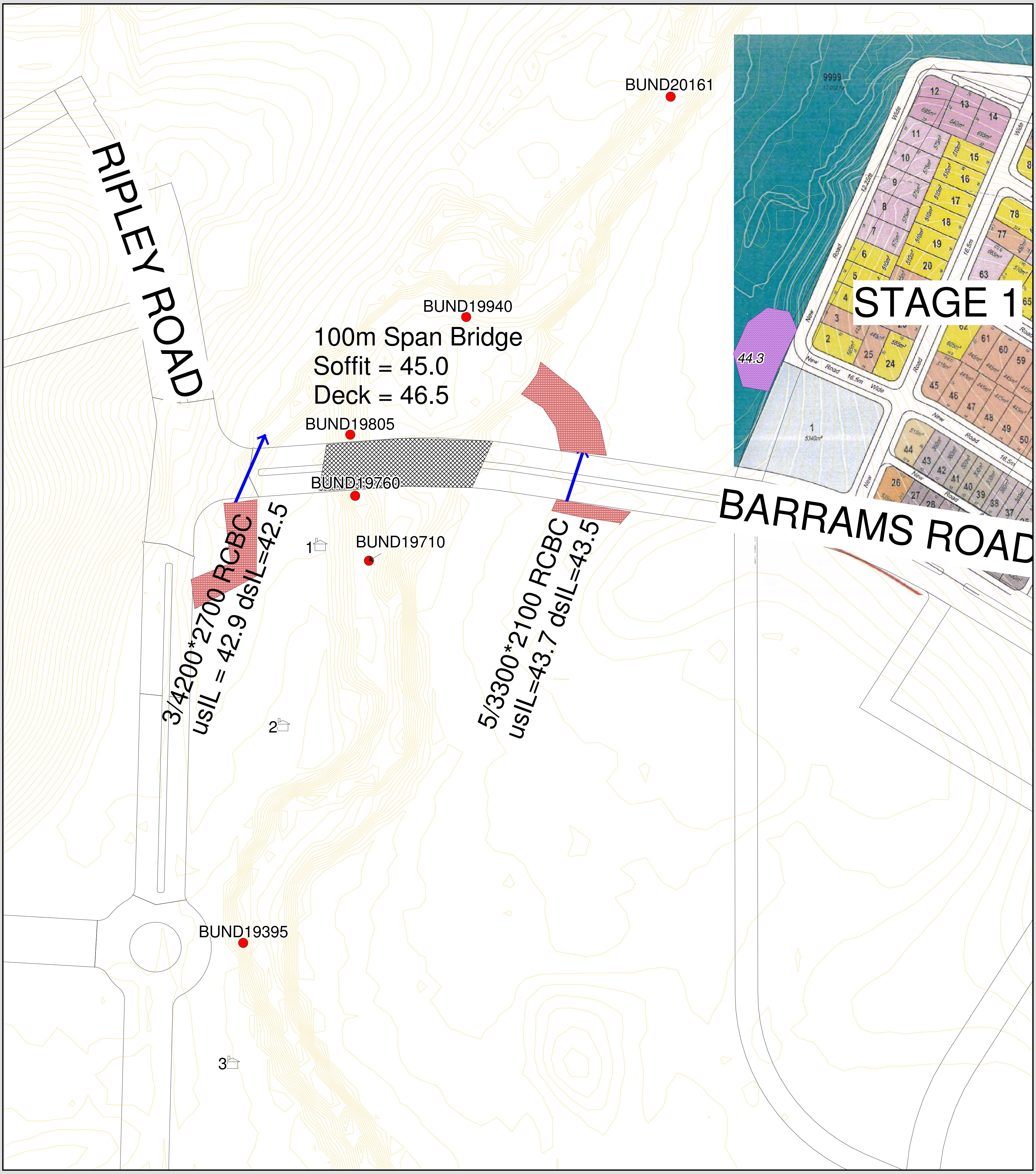
Figure 2.3
TUFLOW Model Setup

SUCE Stage 1

Flood Study

Scale: 1:12,500

125 0 125 250 375
Metres



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LEGEND

- Proposed Bridge
- Proposed Culverts
- Playground
- Drainage Channel
- Roads
- 0.5m Contour
- Bundamba Creek Flood Comparison Point
- Existing Building

N E S W

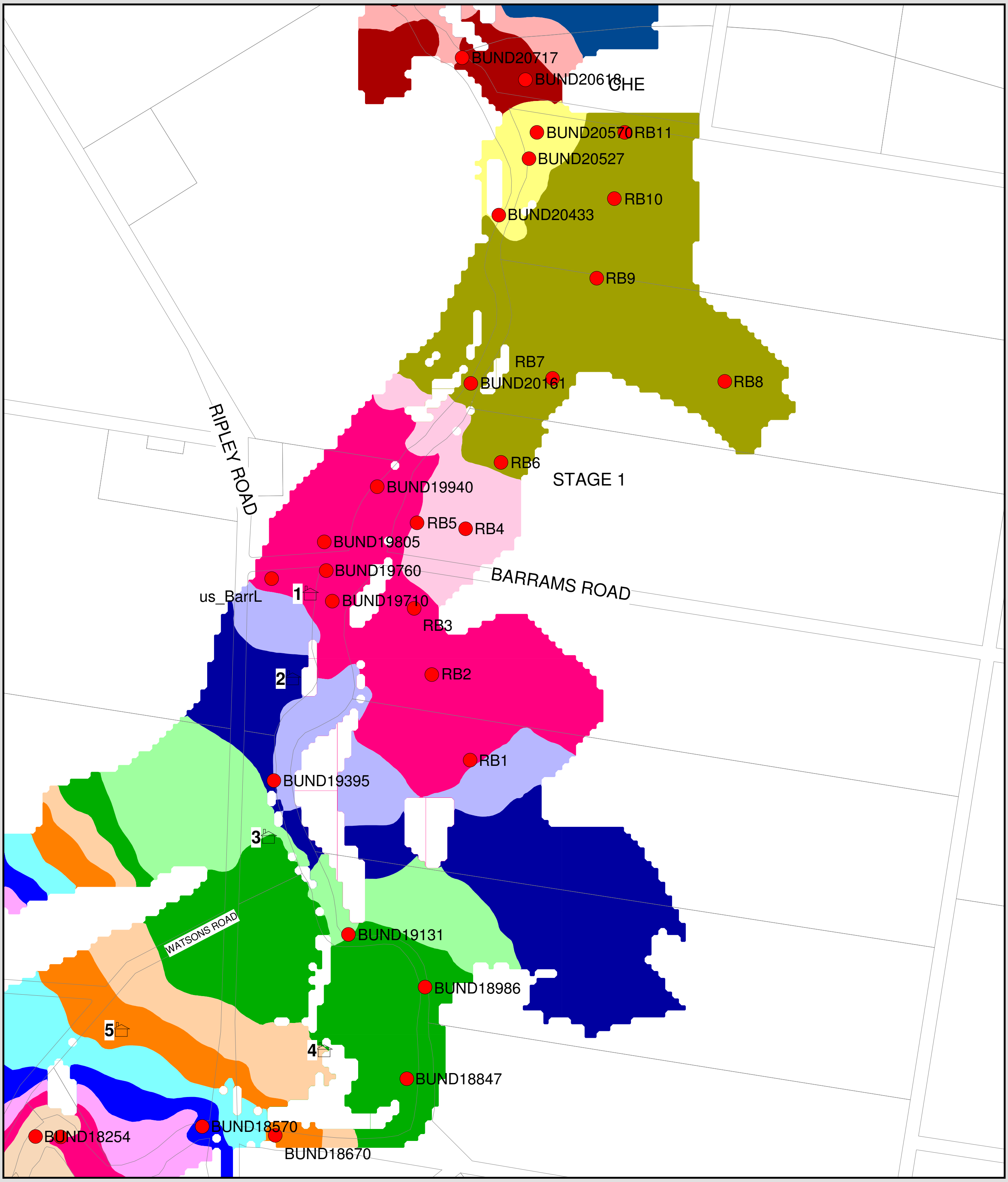
Figure 3
Proposed Stage 1 Development

SUCE Stage 1

Flood Study

Scale: 1:2,500

25 0 25 50 75
Metres



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LEGEND

Peak Water Level (m AHD)

- 40.50 to 41.00
- 41.00 to 41.50
- 41.50 to 42.00
- 42.00 to 42.50
- 42.50 to 43.00
- 43.00 to 43.50
- 43.50 to 44.00
- 44.00 to 44.50
- 44.50 to 45.00
- 45.00 to 45.50
- 45.50 to 46.00
- 46.00 to 46.50
- 46.50 to 47.00
- 47.00 to 47.50
- 47.50 to 48.00
- 48.00 to 48.50
- 48.50 to 49.00
- 49.00 to 49.50
- 49.50 to 50.00
- 50.00 to 50.50

Reporting Point

Building

DCDB

N
W E
S

Figure 4
Case 2 (Ex Case)
100 Year ARI
Peak Water Level

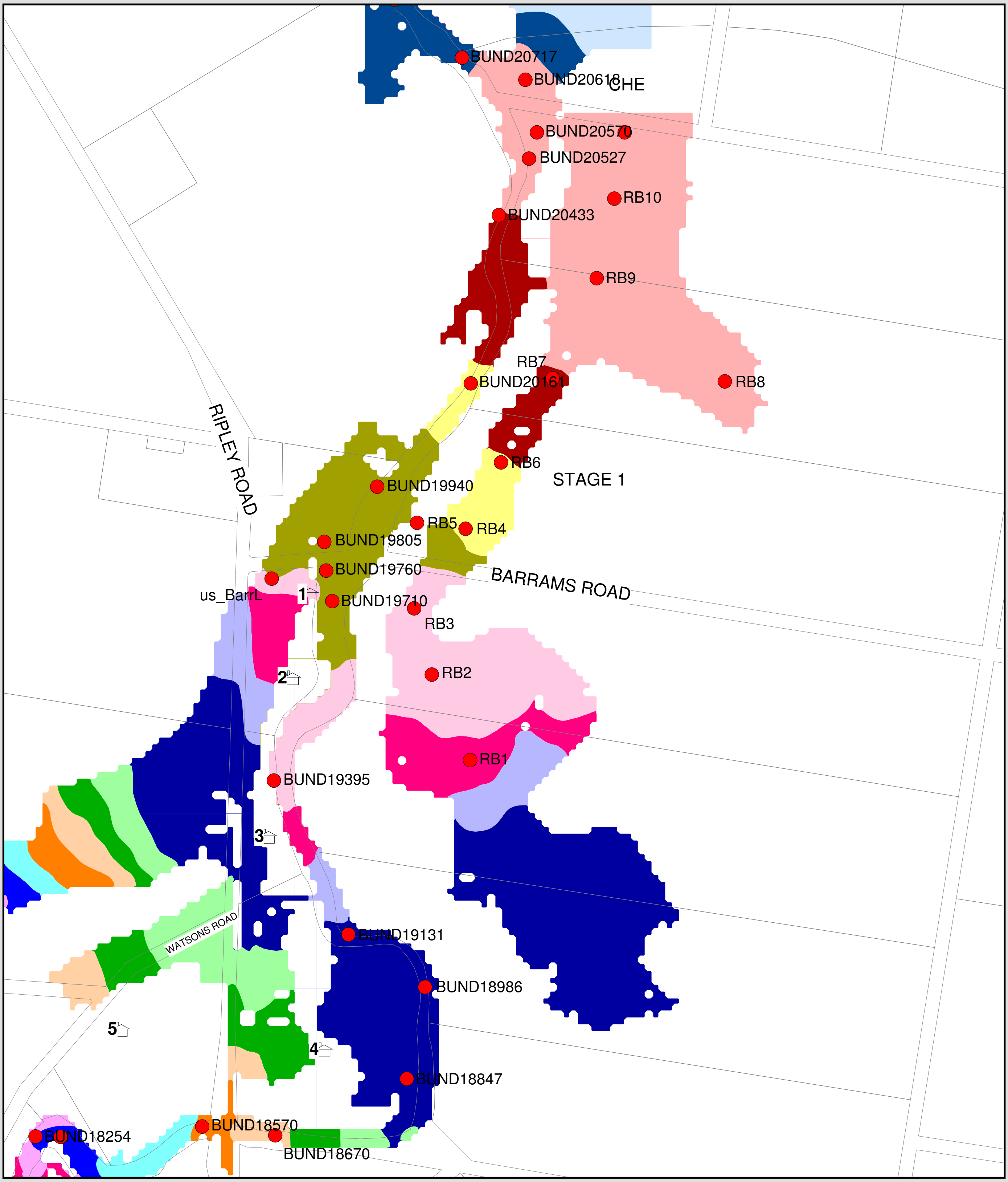
SUCE Stage 1

Flood Study

Scale: 1:8,000

80 0 80 160 240

Metres



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Date:	February 2011
Revision Number:	
Designed by:	
Client Name:	AMEX Pty Ltd

LEGEND

Peak Water Level (m AHD)

- 40.50 to 41.00
- 41.00 to 41.50
- 41.50 to 42.00
- 42.00 to 42.50
- 42.50 to 43.00
- 43.00 to 43.50
- 43.50 to 44.00
- 44.00 to 44.50
- 44.50 to 45.00
- 45.00 to 45.50
- 45.50 to 46.00
- 46.00 to 46.50
- 46.50 to 47.00
- 47.00 to 47.50
- 47.50 to 48.00
- 48.00 to 48.50
- 48.50 to 49.00
- 49.00 to 49.50
- 49.50 to 50.00
- 50.00 to 50.50

Reporting Point
Building
DCDB

N W E S

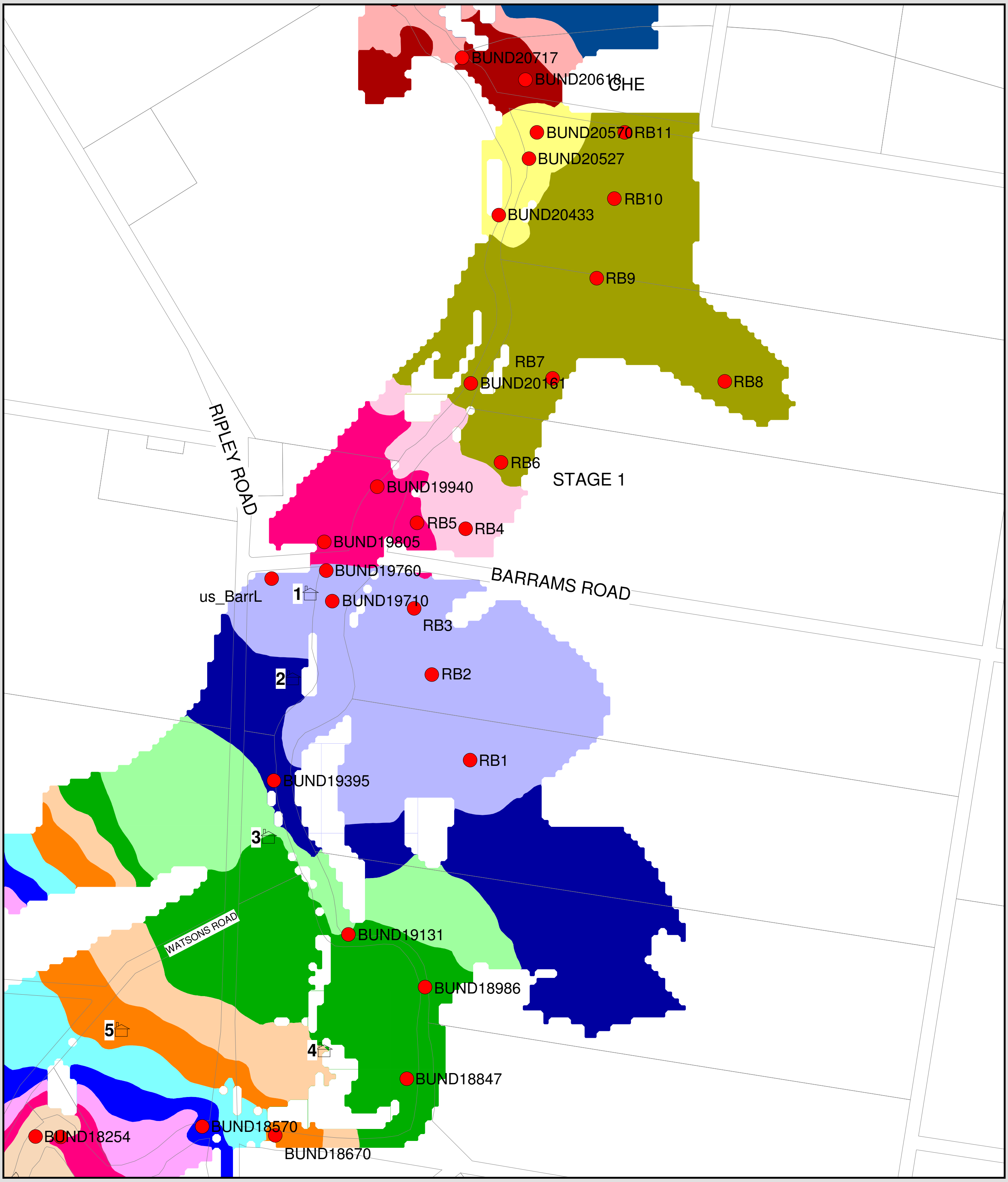
Figure 5
Case 2 (Ex Case)
10 Year ARI
Peak Water Level

SUCE Stage 1

Flood Study

Scale: 1:8,000

80 0 80 160 240
Metres



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Project No:	7901-47-22
Date:	February 2011
Revision Number:	
Designed by:	
Client Name:	AMEX Pty Ltd

LEGEND

Peak Water Level (m AHD)

- 40.50 to 41.00
- 41.00 to 41.50
- 41.50 to 42.00
- 42.00 to 42.50
- 42.50 to 43.00
- 43.00 to 43.50
- 43.50 to 44.00
- 44.00 to 44.50
- 44.50 to 45.00
- 45.00 to 45.50
- 45.50 to 46.00
- 46.00 to 46.50
- 46.50 to 47.00
- 47.00 to 47.50
- 47.50 to 48.00
- 48.00 to 48.50
- 48.50 to 49.00
- 49.00 to 49.50
- 49.50 to 50.00
- 50.00 to 50.50

Reporting Point

Building

DCDB

N E S W

Figure 6
Case 4 (BarrBrStg1)
100 Year ARI
Peak Water Level

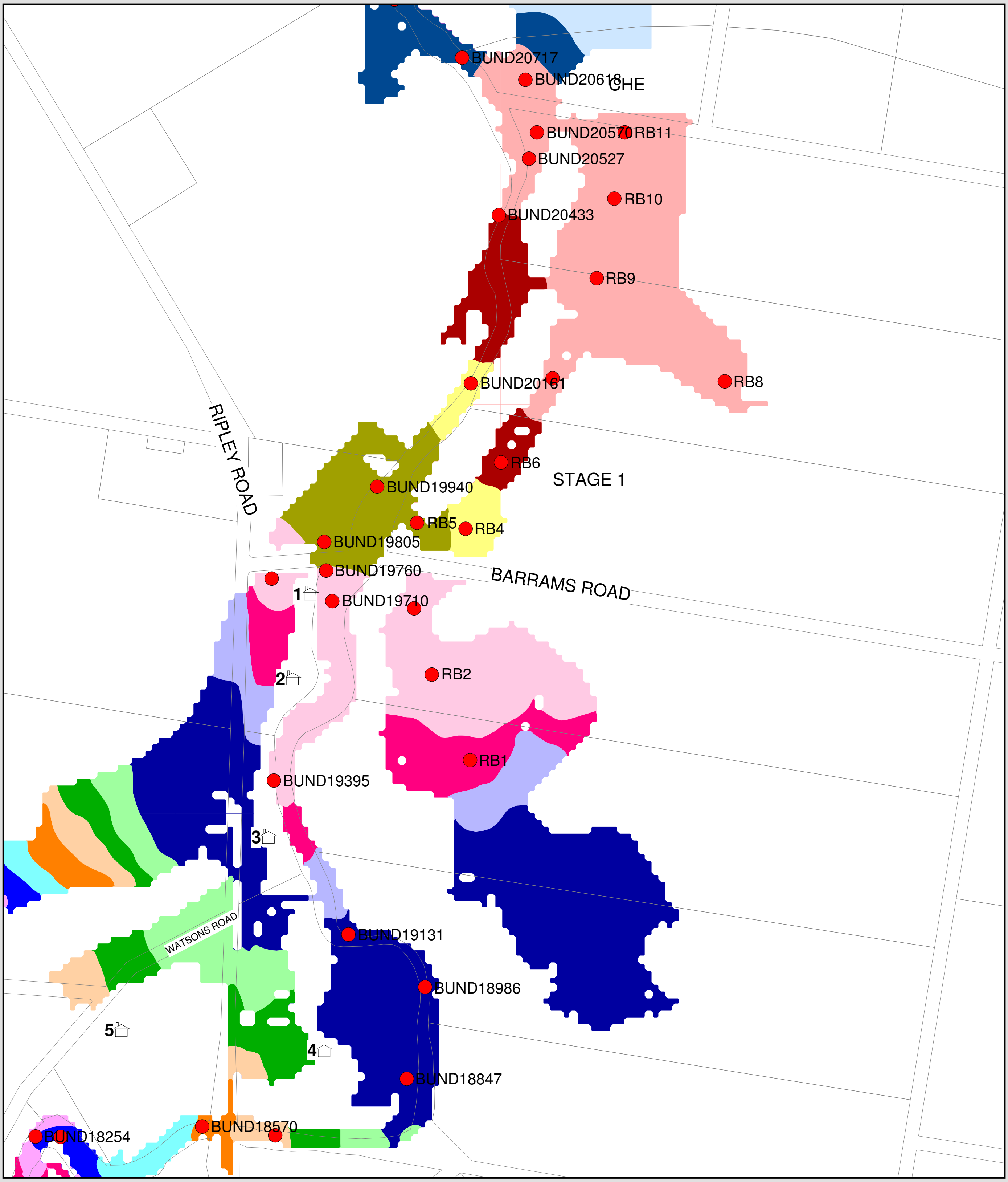
SUCE Stage 1

Flood Study

Scale: 1:8,000

80 0 80 160 240

Metres



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Revision Number:	
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Client Name:	AMEX Pty Ltd

LEGEND

Peak Water Level
(m AHD)

40.50 to 41.00
41.00 to 41.50
41.50 to 42.00
42.00 to 42.50
42.50 to 43.00
43.00 to 43.50
43.50 to 44.00
44.00 to 44.50
44.50 to 45.00
45.00 to 45.50
45.50 to 46.00
46.00 to 46.50
46.50 to 47.00
47.00 to 47.50
47.50 to 48.00
48.00 to 48.50
48.50 to 49.00
49.00 to 49.50
49.50 to 50.00
50.00 to 50.50

Reporting Point
Building
DCDB

N W E S

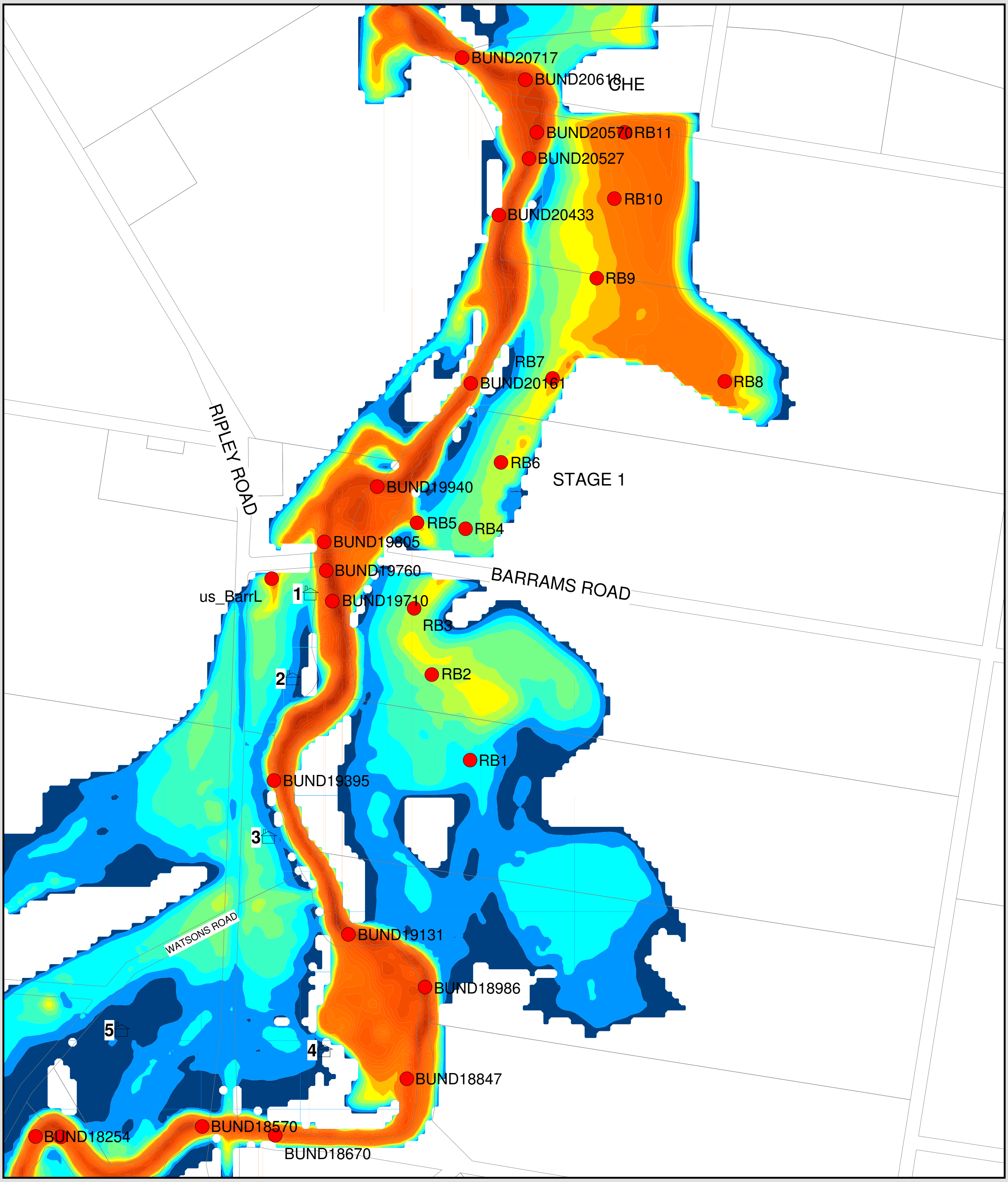
Figure 7
Case 4 (BarrBrStg1)
10 Year ARI
Peak Water Level

SUCE Stage 1

Flood Study

Scale: 1:8,000

80 0 80 160 240
Metres



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Project No:	7901-47-22
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Revision Number:	
Designed by:	
Client Name:	AMEX Pty Ltd

LEGEND

Peak Depth (m)

- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 1.25
- 1.25 to 1.50
- 1.50 to 1.75
- 1.75 to 2.00
- 2.00 to 2.25
- 2.25 to 2.50
- 2.50 to 2.75
- 2.75 to 3.00
- 3.00 to 3.50
- 3.50 to 4.00
- 4.00 to 4.50
- 4.50 to 5.00
- > 5.00

Reporting Point

Building

DCDB

N W E S

Figure 8
Case 4 (BarrBrStg1)
100 Year ARI
Peak Depth

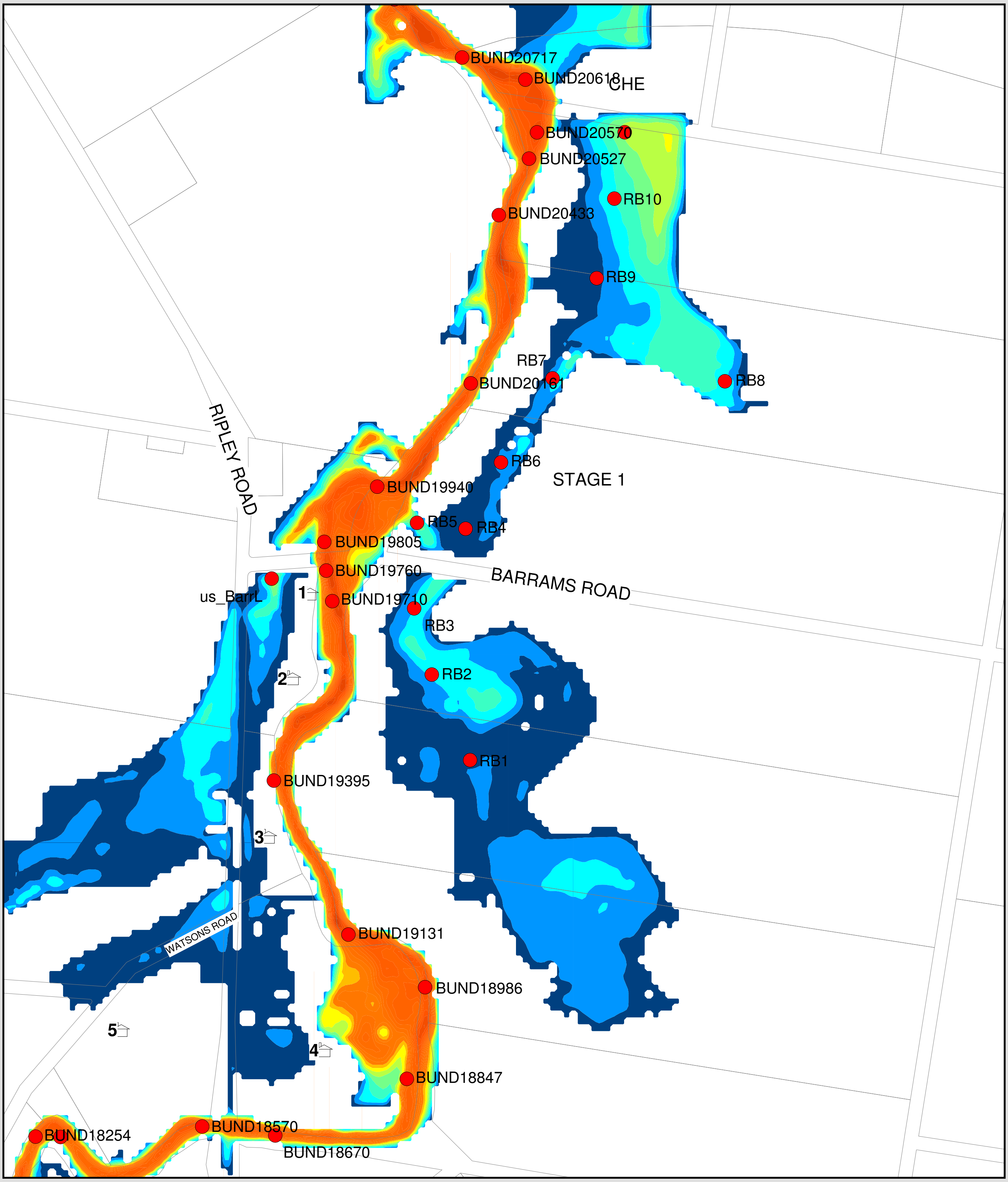
SUCE Stage 1

Flood Study

Scale: 1:8,000

80 0 80 160 240

Metres



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Date:	February 2011
Revision Number:	
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LEGEND

Peak Depth (m)

- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 1.25
- 1.25 to 1.50
- 1.50 to 1.75
- 1.75 to 2.00
- 2.00 to 2.25
- 2.25 to 2.50
- 2.50 to 2.75
- 2.75 to 3.00
- 3.00 to 3.50
- 3.50 to 4.00
- 4.00 to 4.50
- 4.50 to 5.00
- > 5.00

Reporting Point

Building

DCDB

N W E S

Figure 9
Case 4 (BarrBrStg1)
10 Year ARI
Peak Depth

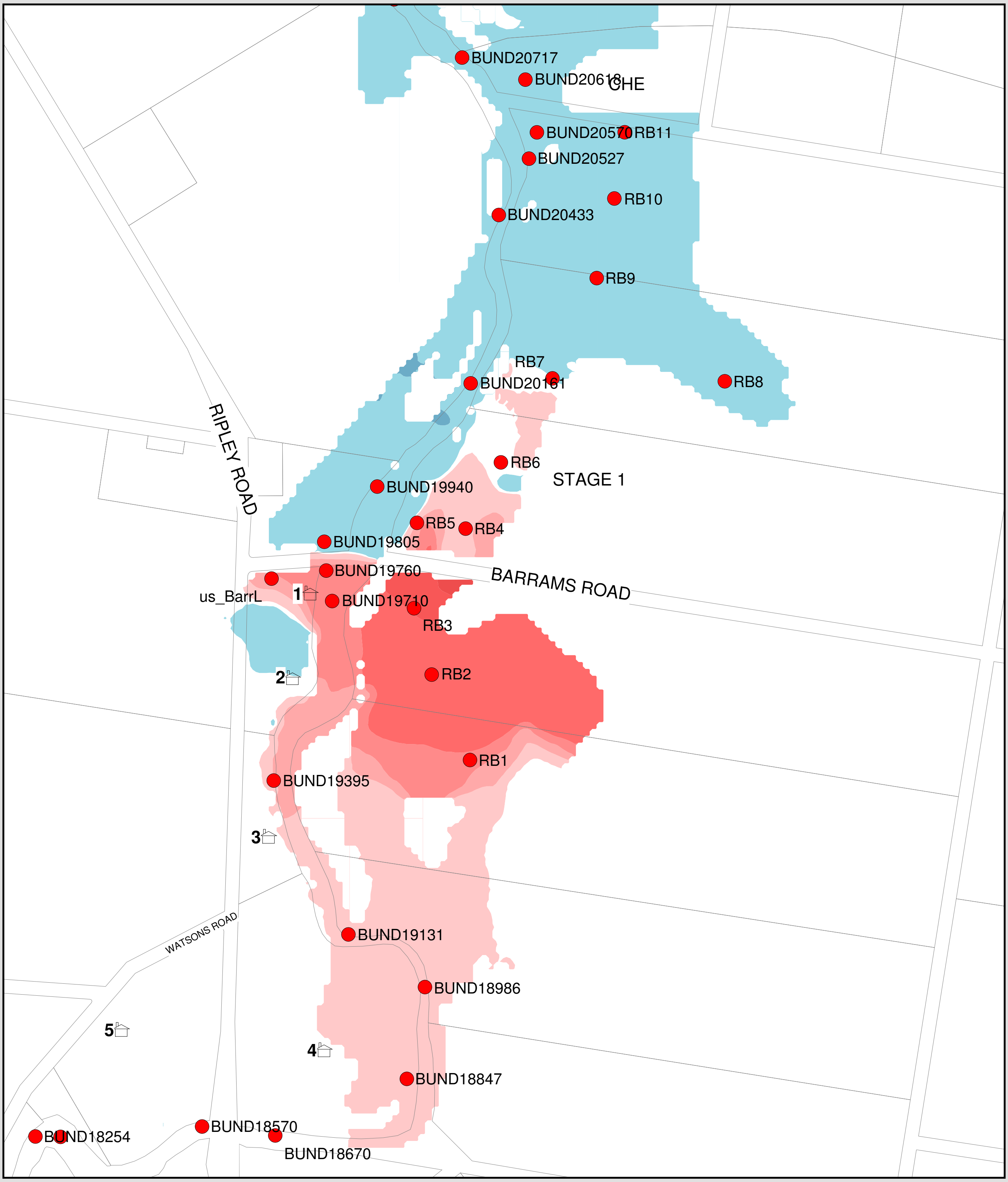
SUCE Stage 1

Flood Study

Scale: 1:8,000

80 0 80 160 240

Metres



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LEGEND

Peak Afflux (m)

- < -0.20
- -0.20 to -0.10
- -0.10 to -0.01
- -0.01 to 0.01
- 0.01 to 0.10
- 0.10 to 0.20
- 0.20 to 0.30
- 0.30 to 0.50
- 0.50 to 0.70
- >0.70

- Reporting Point
- Building
- DCDB

N W E S

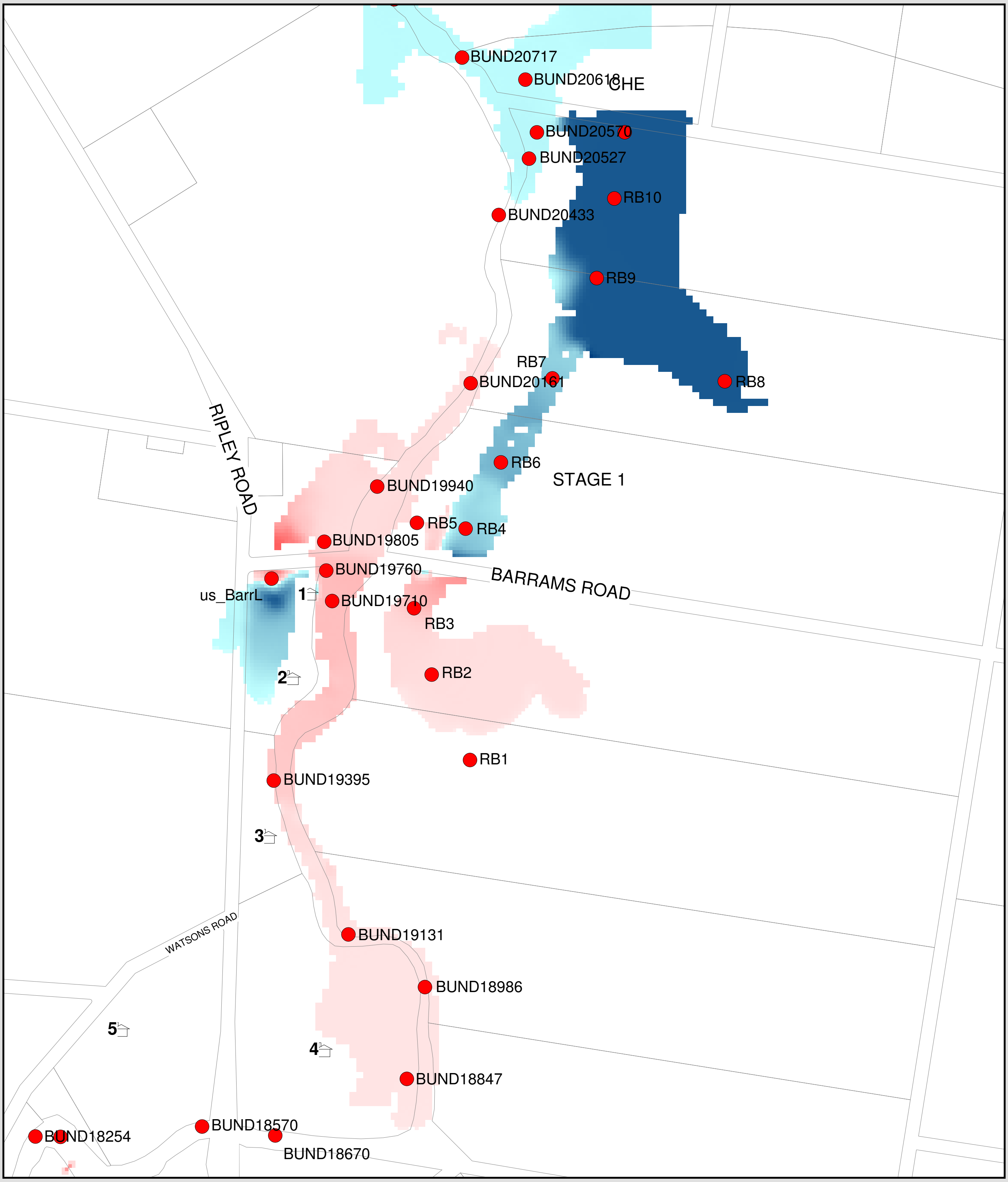
Figure 10 Case 4 (BarrBrStg1) 100 Year ARI Peak Afflux

SUCE Stage 1

Flood Study

Scale: 1:8,000

80 0 80 160 240
Metres



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LEGEND

Peak Afflux (m)

- < -0.20
- -0.20 to -0.10
- -0.10 to -0.01
- -0.01 to 0.01
- 0.01 to 0.10
- 0.10 to 0.20
- 0.20 to 0.30
- 0.30 to 0.50
- 0.50 to 0.70
- >0.70

- Reporting Point
- Building
- DCDB

N W E S

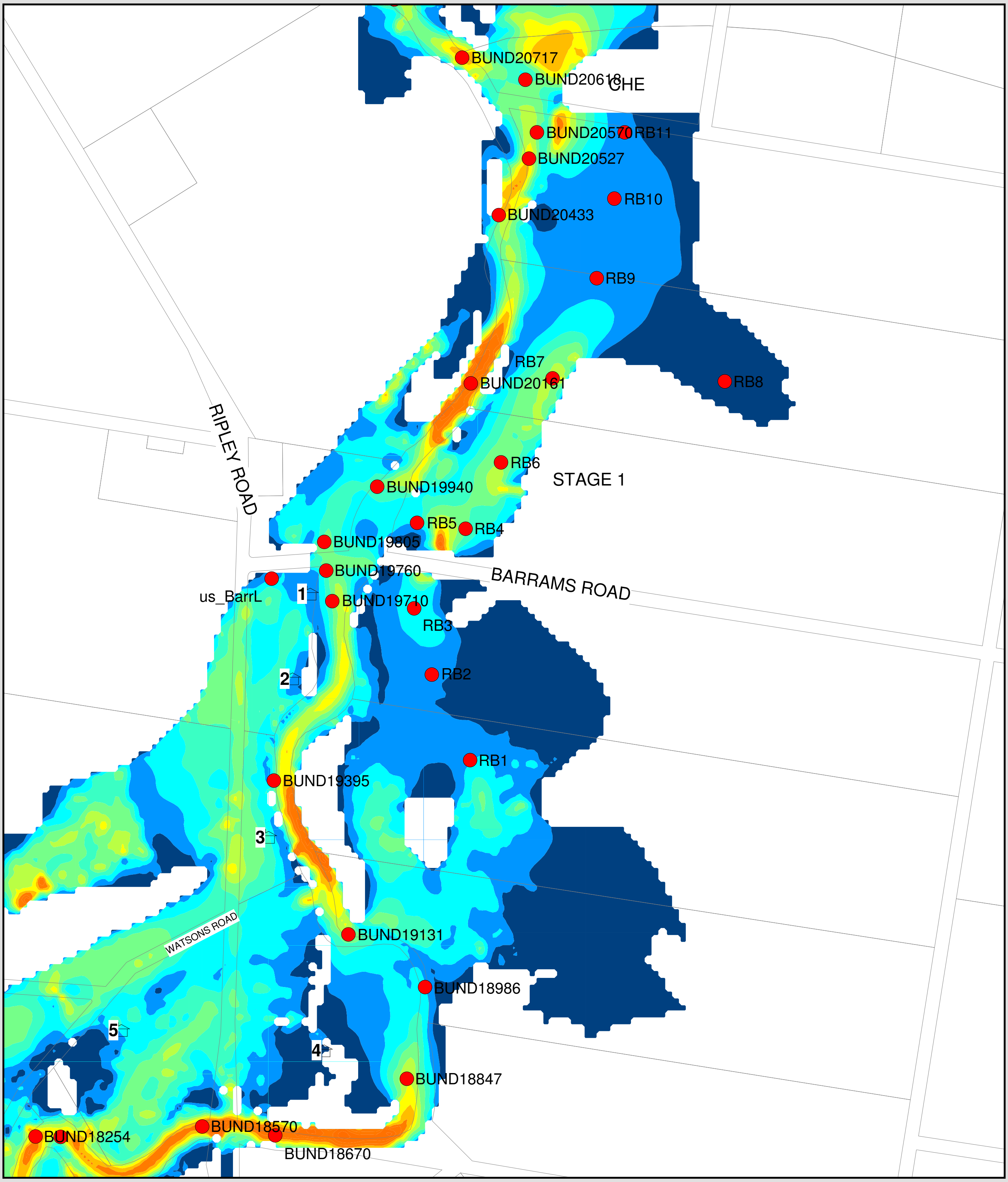
Figure 11
Case 4 (BarrBrStg1)
10 Year ARI
Peak Afflux

SUCE Stage 1

Flood Study

Scale: 1:8,000

80 0 80 160 240
Metres



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LEGEND

Peak Velocity (m/s)

- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 1.25
- 1.25 to 1.50
- 1.50 to 1.75
- 1.75 to 2.00
- 2.00 to 2.25
- 2.25 to 2.50
- 2.50 to 2.75
- 2.75 to 3.00
- 3.00 to 3.50
- 3.50 to 4.00
- 4.00 to 4.50
- 4.50 to 5.00
- > 5.00

Reporting Point

Building

DCDB

N W E S

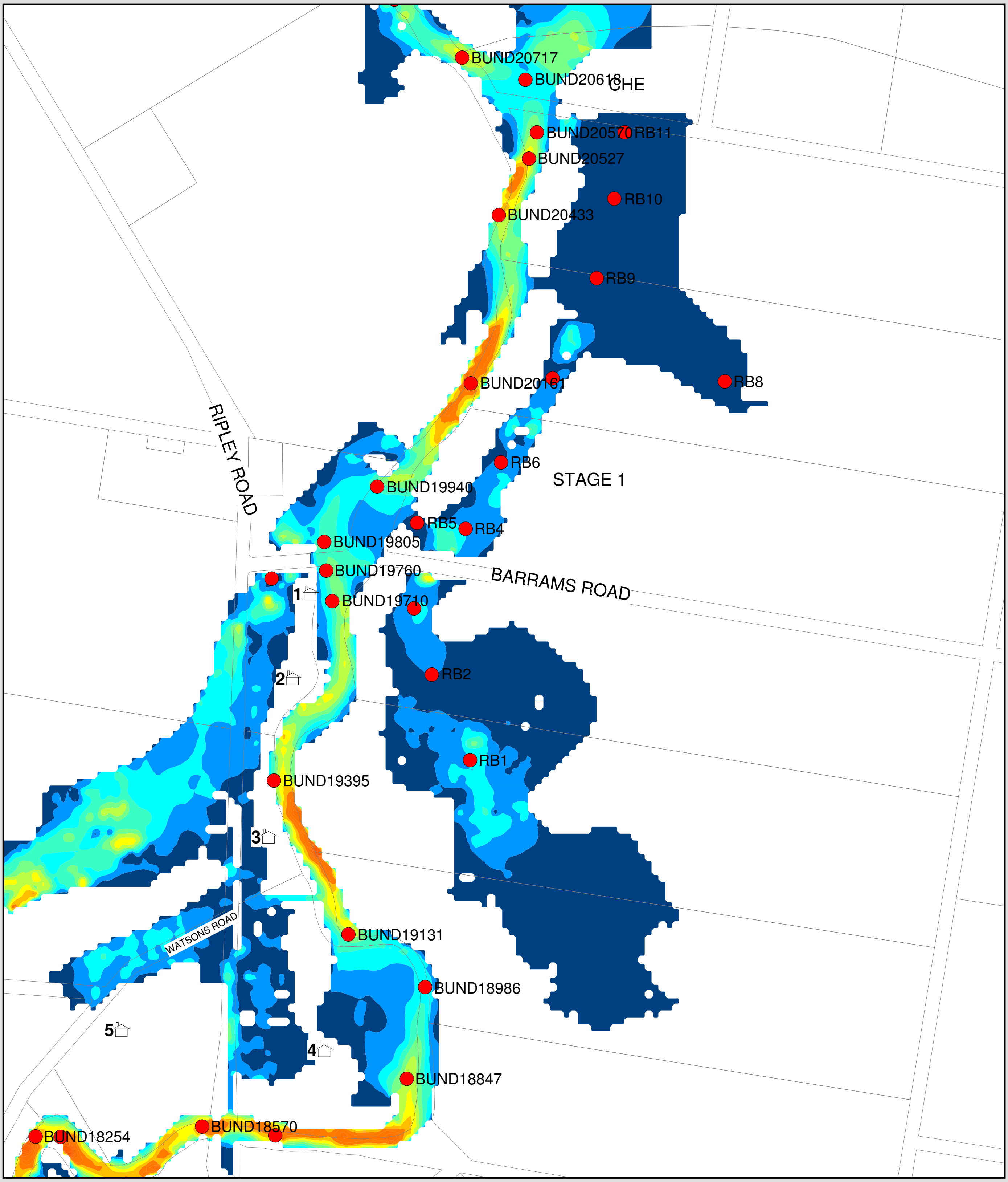
Figure 12
Case 4 (BarrBrStg1)
100 Year ARI
Peak Velocity

SUCE Stage 1

Flood Study

Scale: 1:8,000

80 0 80 160 240
Metres



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LEGEND

Peak Velocity (m/s)

- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 1.25
- 1.25 to 1.50
- 1.50 to 1.75
- 1.75 to 2.00
- 2.00 to 2.25
- 2.25 to 2.50
- 2.50 to 2.75
- 2.75 to 3.00
- 3.00 to 3.50
- 3.50 to 4.00
- 4.00 to 4.50
- 4.50 to 5.00
- > 5.00

- Reporting Point
- Building
- DCDB

N W E S

Figure 13
Case 4 (BarrBrStg1)
10 Year ARI
Peak Velocity

SUCE Stage 1

Flood Study

Scale: 1:8,000

80 0 80 160 240
Metres