APPENDIX G – Flood Study

North Boambee Valley (west)

Flood Study

Final

October 2014

de Groot & Benson Pty Ltd

North Boambee Valley (west)

Flood Study

Final

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Revision Comments.

D2 Addition of 5%, 0.2% design events and March 2009 historic event;
Some addition to chapters 3 & 5;
Major redrafting of chapters 6 & 7.

Final Apr 2013 - Recommendation of flood mitigation works – Large basin 1 and no basin 2.

Final June 2013 – Appendix A addedon Section 94 costing options.

Final Oct 2014 - Fig 6.1 deleted, minor clarification of basin options.

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1 INTRODUCTION

This flood study has been prepared as part of a Local Environment Study of the North Boambee Valley, west of the proposed Pacific Highway bypass corridor.

This study used computer flood modelling to examine:

- The flood behaviour in the valley;
- Its impact on development potential; and;
- How development can proceed without adverse impacts, both within and downstream of the study area.

Development within the floodplain is found to be viable in combination with recommended flood mitigation works.

2 DATA SET

Two previous studies are of relevance:

- Boambee Creek and Newports Creek Flood Study, (ref 1);
- North Boambee Valley Flood Study, (ref 2).

The WMA study utilises two dimensional floodplain modelling using the computer program TUFLOW. This extends from the ocean up Boambee Creek to end a little downstream of the study area. Hydraulic modelling of the study area was not undertaken. However, hydrologic modelling of the study area, as part of the greater catchment, was undertaken using the computer program WBNM.

While the WMA study did not model the flood behaviour through the study area, it can provide a useful downstream boundary condition. The study's reporting of peak flow rates is also of relevance in setting up hydrologic input.

The Bewsher study of 1991 utilised RORB to model hydrology and HEC-II to model creek hydraulics. The RORB model did not include a rainfall gradient across the catchment, which is now the accepted practice post the 1996 and 2009 events. At the outlet of the study area, the Bewsher RORB model predicted a peak 1% AEP flow rate of 213 m^3/s (for the 6 hour ARR87 design event). The more recent WMA modelling, which included a rainfall gradient, predicts 240 m^3/s for the same event. The latter is considered more accurate.

The Bewsher hydraulic modelling did however cover the study area. The steady state one dimensional HEC-II modelling relied of photogrammetric survey for the creek cross section. The accuracy of a few of these sections was compared with Council's current, and more reliable, aerial laser survey (ALS). The accuracy of the Bewsher cross sections was generally found to be poor.

The ground level survey of most relevance for flood modelling through the study area is Council's aerial laser survey (ALS). This data was made available and utilised. No additional ground survey was undertaken.

3 HYDROLOGY

Hydrologic modelling utilises computer software to "model" the process of rainfall and runoff. The output of the modelling are runoff hydrographs at discrete points through the catchment. These are then used as input into the hydraulic modelling.

The computer program WBNM was selected for the hydrologic modelling. The WBNM model used in the WMA study was reviewed, but its division of sub-catchments was considered too coarse for use. A new WBNM model was prepared involving 56 sub-catchments, as shown on Figure 3.1. Key aspects of the model are:

- The model was extended downstream past the study area boundary to a common point with the WMA model. The entire catchment area is 1,208 ha.;
- A rainfall gradient was introduced that used the same elevation relationship adopted in the WMA study, as reported in Table 7 of the WMA report;
- The percentage imperviousness for existing conditions varies from nil for some sub-catchments up to 15% for sub-catchments C1 & C2.
- The model's prediction of peak 1% AEP flow at the outlet was compared with the WMA study. The pervious loss parameter was adjusted to 3.5 mm/hr so that reasonable agreement with the WMA flow of 240 m^3/s was obtained.
- Other model parameters were set to the default as recommended by WBNM, specifically:
  - Catchment lag = 1.6;
  - Impervious lag = 0.1;
  - Initial loss = 0 mm (a wet catchment);
  - Pervious continuing loss (3.5mm, up from 2.5mm used in the WMA model).

It was considered important for the model, at its outlet, to match the WMA model. The WMA modelling has been calibrated, verified against historic events and has been adopted by Council. The modelling undertaken for this assessment is not calibrated or verified to any historic data. In the absence of such, making it consistent with the WMA study was considered appropriate.

The hydrologic modelling of the developed conditions case was undertaken by increasing the percentage imperviousness of the relevant sub-catchments, based on the predicted land yield and zoning type. Future land was modelled at:

- Residential = 50% impervious;
- Industrial/commercial = 90% impervious;
- Rural residential = 12% impervious.

This increased the imperviousness of some sub-catchments markedly and some not at all. Overall the entire catchment's imperviousness increased from 1.8% to 7.6%. Of interest is that the WBNM model predicts no real increase in peak 1% AEP flows at the outlet. A study of the hydrograph does however show a greater total volume of discharge, with greater discharge early on, during the rising limb of the hydrograph. An examination of the catchment shows that the proposed development areas are generally all in the lower half of the catchment. Subsequently, the model predicts the increase in imperviousness leads to more runoff early during the event, but not coinciding with the slower peak descending from the upper catchment.

The WBNM model was then used to prepare both existing and developed condition hydrographs for input into a TUFLOW two dimensional hydraulic computer model of the floodplain. The following ARR87 design events were modelled:

- 5% AEP (20-yr ARI) events (2, 6 & 9hr);
- 1% AEP (100-yr ARI) events (1 – 12 hr);
- 0.2% AEP (500-yr ARI) events (2, 6 & 9hr).

In addition to these design events, the 31 March 2009 event was also modelled. The recorded rainfall from the Manly Hydraulic Laboratory's Newports Creek gauge was applied to the catchment with the same rainfall gradient described above.

4 HYDRAULICS

Hydraulic modelling of the flood behaviour through the study area was undertaken using the computer program TUFLOW. TUFLOW is a un-steady state two dimensional hydraulic model that reliably accounts for the two dimensional flow distribution of flood waters across a floodplain as well as the attenuation effect of floodplain storage.

Key aspects of the TUFLOW model are:

- The extent of the model is shown in Figure 4.1. The only creek named on the CMA 1:25,000 Topographic Map is Newports Creek. For the purpose of reference within this report, its tributaries have been labelled. Note, Trib A is the main creek line in terms of catchment and flows.

- A digital elevation model (DEM) of the existing topography was generated from Council's aerial laser survey (ALS) data with modifications along the creek channels. The ALS data is most reliable across cleared ground where individual point accuracy is generally within plus or minus 0.2m. However, it is less reliable through thick vegetation, which in this case, is often along the creek lines. In areas of thick vegetation the ALS data tended to miss the narrow creek inverts. Using civil design software (12D and Autocad Civil 3D) long sections were plotted along all major creek lines in the model. Creek channels were then carved into the DEM by stringing the low points together, creating a channel that continuously fell towards the outlet. The width of these channels was varied based on close inspection of aerial photography and field observations.

- A 4 by 4 metre modelling grid was adopted. This size was a compromise between run time of the model and accuracy. While a 4 metre grid is a little course to accurately model some of the smaller creeks in low flow conditions, it is ample for the broad flat floodplain in larger events where reliable prediction is most needed.

- Manning's n value of hydraulic roughness was assigned the regions based on experience and the aerial photography. The values adopted were:
  - Pavement = 0.02;
  - Open pasture = 0.03;
  - Sparse vegetation = 0.05;

  - Medium vegetation = 0.08;
  - Buildings = 3.0;

  Most of the creek channels were modelled at 0.05 or 0.08.

- Seven major culvert crossings plus the bridge of North Boambee Road were included. Their size was measured in the field while their level was estimated from the ALS data.

- The downstream boundary was set to overlap with the WMA TUFLOW model prepared for the Boambee and Newports Creek Flood Study (Ref 1). It was necessary to significantly overlap the WMA model in order to locate a boundary with a constant flood level along its length. The location chosen was opposite the Isles Industrial Estate, as shown in Figure 4.1.

  It is noted that the topography of the WMA model upstream of this location did not include the final floodway excavation associated with the Isles Industrial Estate. These floodways were added to the new model.

  The downstream boundary water level was approximately calibrated to match that reported in the Boambee and Newports Creek Flood Study. This was achieved by setting the boundary to normal depth, being a function of the cross section, its hydraulic roughness, the hydraulic slope and the flow rate. The Manning's n hydraulic roughness was adjusted so that at a peak 1% AEP discharge of approximately 240 m^3/s, a flood level of approximately 7.2 mAHD was generated.

  It is noted that the flood level predictions throughout the study area are quite insensitive to the assumed downstream boundary flood level.

Calibration of the WMNM hydrologic and TUFLOW hydraulic models was not attempted, other than to make them consistent with the WMA modelling. Significant effort is required for such, and given the limited flood level information and rainfall records through the study area, would be of questionable value. Never the less, the modelling undertaken here is considered to be substantially more reliable and accurate than Council's current flood study, the Bewsher study of 1991 (Ref 2).

In comparison with the Bewsher Study, the following is noted:

- Bewsher underestimated flood flows and thus flood levels due to not modelling rainfall catchments across the catchments (Section 1)
- Bewsher hydraulic modelling was based on photogrammetry survey of the study area. This study used more recent ALS data. In several areas, there was significant difference between the two, which would result in different calculated flood levels.

Not withstanding the above comments, flood levels in this study are generally within +/- 0.5m of the Bewsher Study levels.

The prepared models are sufficient to reasonable predict flood levels, to assess the impact of potential development and works to mitigate such impacts.

5 EXISTING FLOOD BEHAVIOUR

Design rainfall temporal patterns from Australian Rainfall and Runoff (ref 3) were adopted. Events ranging in duration from 1 hr to 12 hours were modelled. The 2 hour design event generally caused the highest flood levels high in the catchment and the 6 to 9 hr events lower in the catchment, although overall there was little difference from 2 to 9 hour.

The results for existing conditions are summarised in Table 5.1 with reference to figure 5.1. The peak flood surface for the 9 hour design events and the March 2009 event are shown on figures 5.2 to 5.10. The 9 hour design event was chosen as this proves to be the critical event when assessing the impacts of the development and effectiveness of mitigation works.

In considering the flood behaviour of existing conditions, the following is of note:

- The TUFLOW model accurately accounts for dynamic storage. When the floodwaters are routed down the floodplain, Tuflow predicts a peak 1% AEP discharge of 237 m^3/s at the outlet. This is less that the 244 m^3/s predicted by the WBNM hydrologic model. This simply reflects that, in this case, the floodplain has greater storage than the generic relationships used in WBNM.

- Much of the flat floodplain between Newports Creek and Trib A as well as that between Tribs C, E & G, is inundated. Although, even in the 0.2% AEP event, this is generally only to a shallow depth. Typically 0.40m or less. Most of the flood conveyance is undertaken by the creeks and their immediate overbanks.

- A lengthy section of North Boambee Road is inundated in less than the 5% AEP event and would be impassable for significant time. Note, the figures show that part of Highlander Drive is also inundated. The modelled topography in this area does not reflect the filling undertaken in constructing the recent Highlander development and is falsely predicting inundation.

- There is little hydraulic gradient upstream, across and downstream of North Boambee Road. The flood waters here are being held up by a constriction in Newports Creek just downstream of the confluence of the main Tribs A and C (South of Bishop Druitt College). Flooding could be reduced in this area by relatively minor works, being the short westward extension of the Isles Industrial Estate floodway through to Trib A. There is a 2.5m difference in flood levels across the creek bank, which is acting as a levee. However, such works are not recommended without further investigation as they would drain significant floodplain storage and increase flood flows downstream. Flooding around North Boambee Road currently causes relatively little damage compared with that downstream of Isles Industrial Estate.

- Two sections of Englands Road are also overtopped at Tribs K & N. However, this overtopping is fairly shallow and would be for short duration.

- The March 2009 event had peaks slightly less than the design 0.2% AEP events. It's flood hydrographs are plotted with the 0.2% design events in Table 5.1. While its peak is lower, its hydrograph above 200 m^3/s is fatter.

Table 5.1 - Tuflow Flood Modelling Results

Existing Conditions

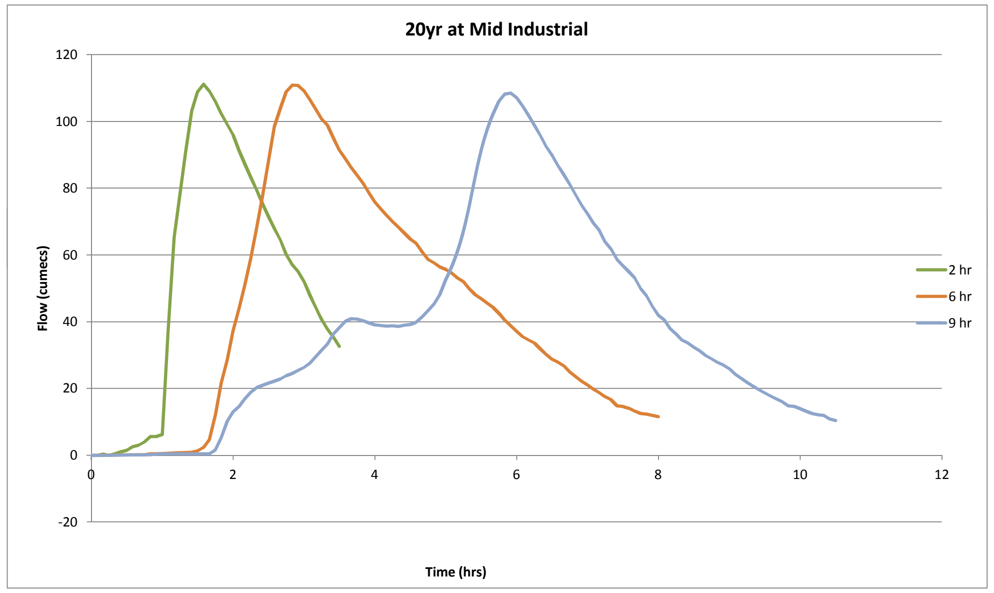
Peak Flows and Levels

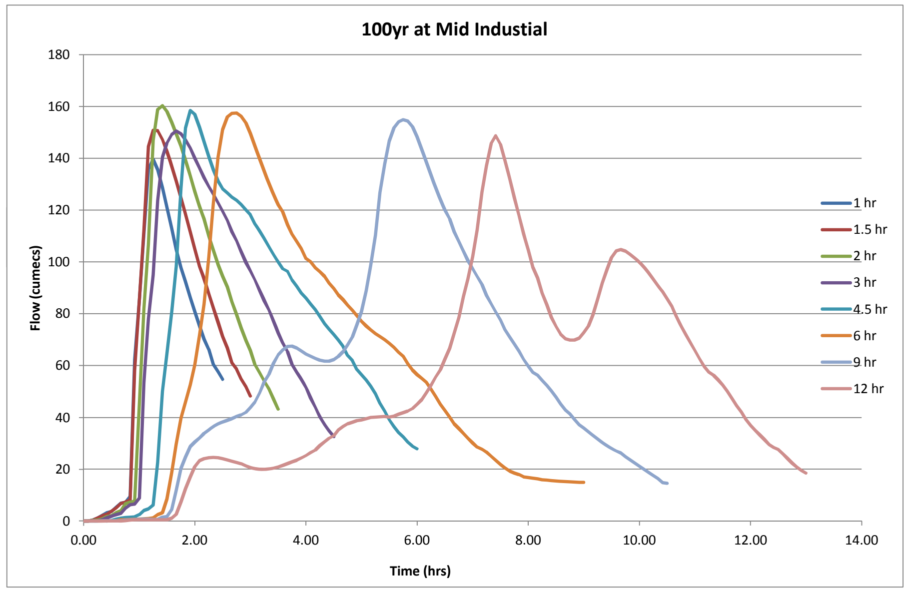
(Refer to Figure 5.1 for the location of the recorded peak values)

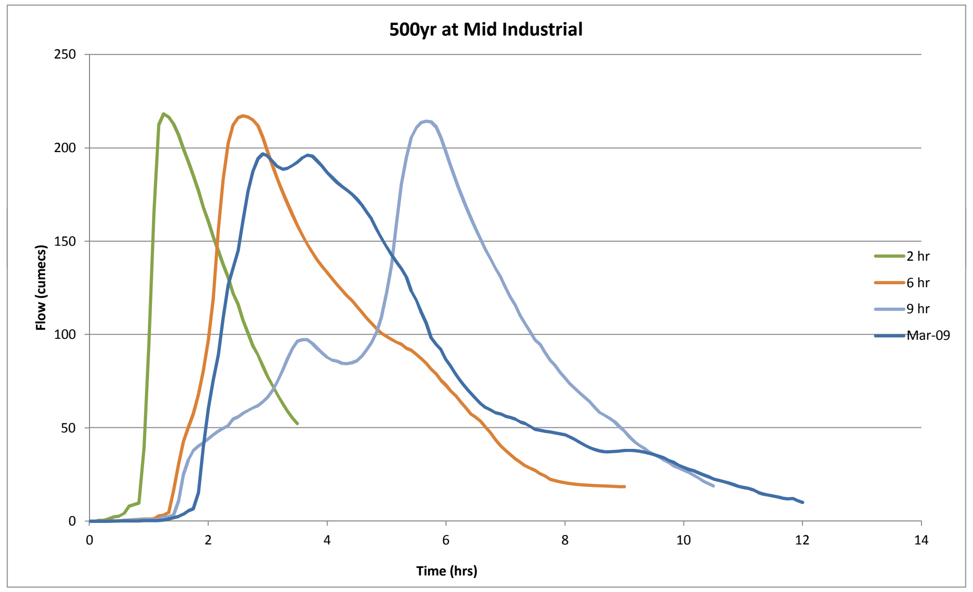
Event	Mid Isles (cumecs)	Model Outlet (cumecs)	Over North Boambee Road # 1 (cumecs)	North Boambee Rd Bridge (cumecs)	Over North Boambee Road # 2 (cumecs)	Basin 2 (cumecs)	Englands Rd at Trib A (cumecs)	u/s Future Highway (cumecs)	Proposed Residential North (cumecs)	Proposed Residential Mid (cumecs)	Proposed Residential South (cumecs)	Proposed Industrial North (cumecs)	Proposed Industrial South (cumecs)	d/s of Basin 1 North (cumecs)	d/s of Basin 1 South (cumecs)	Opposite BDC (cumecs)
20-Year ARI																
2 hr	132.9	133.5	0.0	**38.5**	8.9	**29.9**	59.0	114.4	**14.6**	**36.0**	**11.2**	45.7	**68.6**	31.8	**74.9**	134.4
6 hr	153.5	**156.1**	0.0	37.5	17.4	23.4	**60.5**	**123.1**	11.3	30.6	8.3	**45.8**	66.5	**32.1**	71.1	153.8
9 hr	**156.4**	153.7	0.1	36.6	**18.9**	20.9	60.2	121.2	10.0	28.4	7.0	44.2	65.1	31.5	69.5	**156.5**
100-Year ARI																
1 hr	147.5	144.9	0.0	47.5	11.1	36.6	72.9	137.6	17.3	41.9	16.7	67.0	75.2	41.9	95.8	
1.5 hr	181.3	182.3	0.3	50.8	22.1	38.6	78.2	161.1	18.7	42.9	18.6	75.7	77.3	45.5	99.6	
2 hr	205.3	197.2	0.5	**52.4**	30.9	**40.1**	82.8	174.2	**19.7**	**44.3**	**20.3**	**82.9**	**78.8**	**51.7**	96.3	207.9
3 hr	212.7	207.0	0.6	49.7	36.9	33.8	80.4	169.7	16.3	40.2	14.8	75.4	75.6	45.0	96.1	
4.5 hr	209.1	215.6	0.6	51.0	38.0	35.6	83.2	174.2	17.0	41.3	16.6	81.1	78.0	48.4	**103.0**	
6 hr	232.2	**238.7**	**1.0**	46.1	41.6	31.5	84.7	**180.5**	15.4	38.4	13.8	81.1	77.1	51.2	92.7	233.2
9 hr	**237.7**	228.2	**1.0**	32.0	**45.6**	28.7	**85.3**	176.9	13.9	36.2	12.1	79.1	76.0	50.7	91.3	**237.5**
12 hr	208.4	199.9	0.6	36.5	39.7	30.7	81.5	166.8	14.8	37.1	12.4	74.4	74.9	48.8	90.3	210.4
500-Year ARI																
2 hr	292.2	276.5	2.1	**56.7**	55.6	**53.4**	110.8	242.0	**25.9**	**54.2**	**34.8**	**129.6**	**89.1**	**75.0**	**123.7**	294.7
6 hr	335.9	321.9	3.7	48.9	69.2	42.7	117.7	**249.9**	20.5	46.7	25.2	129.0	88.4	74.3	121.0	335.8
9 hr	**340.1**	**353.0**	**3.9**	34.0	**72.1**	39.2	**117.9**	245.1	18.9	44.3	22.0	127.0	87.4	74.0	120.5	**339.0**
31 March 2009	317.5	333.1	3.0	48.4	72.7	38.0	110.5	225.1	18.1	43.1	20.1	112.6	84.1	67.5	111.5	314.4

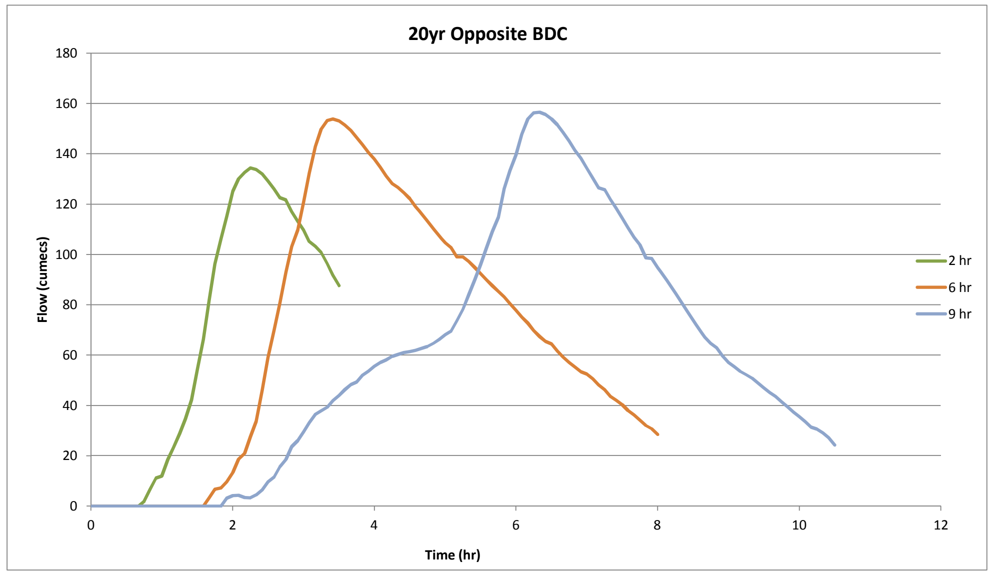
Event	Model dswl (mAHD)	Isles Floodway (mAHD)	Proposed Industrial Mid North (mAHD)	Basin 1 (mAHD)	Englands Rd at Trib A (mAHD)	u/s of Englands Road Culverts No. 4 (mAHD)	u/s of Englands Road Culverts No. 3 (mAHD)	u/s of Englands Road Culverts No. 2 (mAHD)	u/s of Englands Road Culverts No. 1 (mAHD)	Proposed Residential Res 1 (mAHD)	Proposed Residential Res 2 (mAHD)	Proposed Residential Res 3 (mAHD)	Proposed Residential Res 4 (mAHD)	Proposed Industrial Mid South (mAHD)	Basin 2 (mAHD)	d/s of Basin 2 (mAHD)	d/s of Basin 1 South (mAHD)	d/s of Basin 1 North (mAHD)	u/s of NB Rd (mAHD)	Mid Industrial (cumecs)
20-Year ARI																				
2 hr	6.20	**7.61**	**13.46**	**20.67**	**26.16**	**28.58**	**25.30**	**25.27**	15.46	**10.25**	**13.64**	**14.32**	**19.73**	**14.69**	**20.83**	**20.19**	**19.12**	**18.18**	9.43	**111.1**
6 hr	6.43	**7.61**	**13.46**	**20.67**	**26.16**	28.55	**25.30**	25.23	**15.51**	**10.25**	13.58	14.25	19.71	14.67	20.71	20.10	19.10	**18.18**	9.43	110.9
9 hr	**6.46**	**7.61**	13.44	**20.67**	**26.16**	28.54	**25.30**	25.23	**15.51**	**10.25**	13.55	14.22	19.70	14.66	20.67	20.06	19.09	**18.18**	**9.44**	108.4
100-Year ARI																				
1 hr	6.57	7.61	13.74	20.73	26.20	28.65	25.31	25.34	15.56	10.27	13.70	14.36	19.75	14.75	20.94	20.26	19.20	18.27		139.5
1.5 hr	6.94	7.62	13.83	20.73	26.22	28.66	25.33	**25.35**	15.61	**10.28**	13.71	14.38	**19.76**	14.77	20.96	20.28	19.21	18.29		150.8
2 hr	6.91	7.63	**13.90**	**20.74**	26.24	**28.67**	**25.34**	**25.35**	15.63	**10.28**	**13.72**	**14.39**	**19.76**	**14.78**	**20.99**	**20.29**	**19.23**	**18.30**	9.71	**160.3**
3 hr	**7.20**	7.64	13.82	**20.74**	26.23	28.63	25.30	25.29	15.62	10.27	13.68	14.35	19.75	14.76	20.90	20.24	19.20	18.28		150.6
4.5 hr	7.18	7.63	13.88	**20.74**	26.23	28.65	25.33	25.30	15.63	**10.28**	13.69	14.36	**19.76**	**14.78**	20.92	20.25	**19.23**	**18.30**		158.4
6 hr	7.15	**7.65**	13.88	**20.74**	**26.28**	28.63	25.30	25.26	**15.64**	10.27	13.66	14.34	19.74	14.77	20.87	20.21	19.21	**18.30**	9.71	157.5
9 hr	7.19	**7.65**	13.86	**20.74**	26.25	28.61	25.30	25.23	15.63	10.27	13.64	14.33	19.73	14.76	20.81	20.18	19.20	**18.30**	**9.72**	154.9
12 hr	6.93	7.64	13.81	**20.74**	26.23	28.62	25.30	25.23	15.62	10.27	13.65	14.33	19.73	14.75	20.84	20.20	19.20	18.29	9.63	148.6
500-Year ARI																				
2 hr	7.47	7.69	**14.22**	20.80	26.37	**28.76**	**25.40**	**25.42**	**15.75**	**10.31**	**13.78**	**14.44**	**19.79**	**14.85**	**21.14**	**20.40**	**19.32**	**18.41**	9.86	**218.2**
6 hr	7.68	7.88	14.21	**20.81**	**26.39**	28.71	25.37	25.36	**15.75**	10.30	13.73	14.40	19.76	**14.85**	21.03	20.31	19.31	18.40	9.96	217.1
9 hr	**7.70**	**7.91**	14.20	**20.81**	**26.39**	28.69	25.35	25.33	15.73	10.33	13.71	14.39	19.75	14.84	20.97	20.28	19.31	18.40	**9.97**	214.3
31 March 2009	7.62	7.81	14.12	20.79	26.36	28.68	25.34	25.32	15.71	10.32	13.70	14.38	19.75	14.82	20.96	20.27	19.28	18.38	9.92	196.7

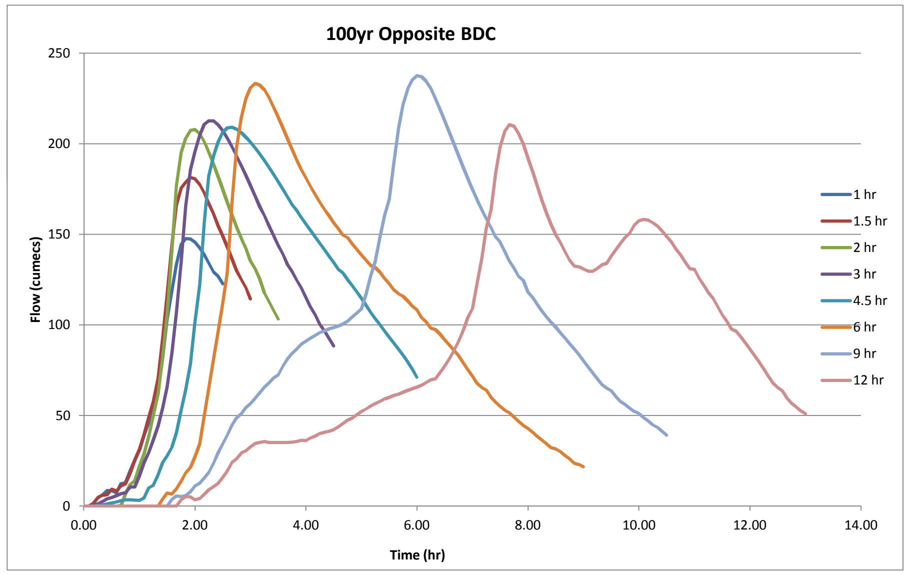
Hydrographs

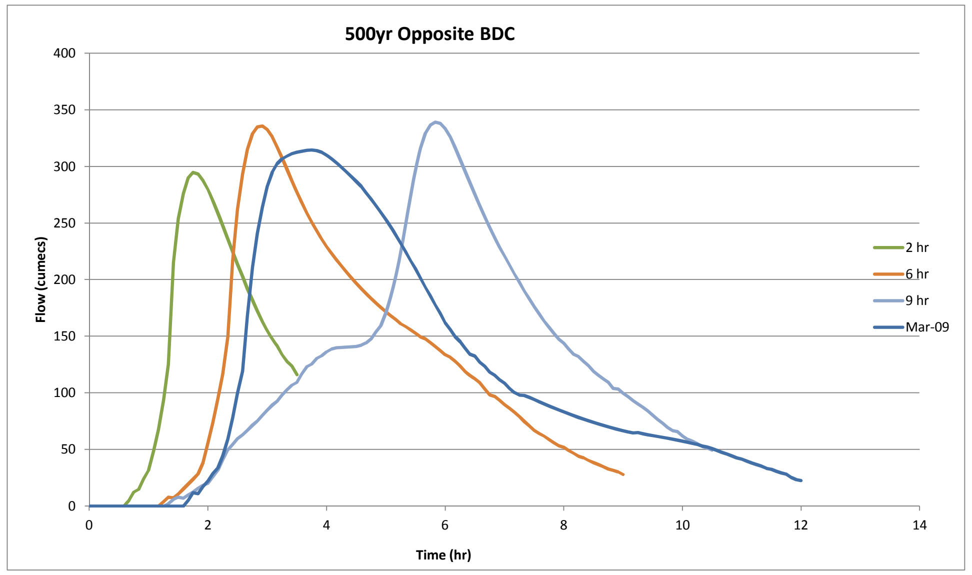












6 POTENTIAL DEVELOPMENT

6.1 Development Strategy

The land identified as potentially developable is shown in Figures 2 to 4 of the Planning Proposal document. This includes significant portions of the floodplain that is currently subject to some inundation in the 1% and 0.2% AEP event. Development in these floodplains will require significant filling to reduce this risk of inundation.

Such filling in the floodplain will adversely impact on flood behaviour in two ways. First, it will locally reduce the capacity of the floodplain to convey the floodwaters. Second, it will reduce floodplain storage. The first impact will increase flood levels local to the filling and upstream. This can be offset by increasing the conveyance capacity of the areas outside the filling by channel works and floodways. Such works, while mitigating the local effect of filling, may reduce the floodplain storage further. The loss of storage reduces the ability of the floodplain to absorb and attenuate the flood wave. This loss results in an increase in peak flows, and hence flood levels, downstream of the development. In this instance, any increase in peak flows and flood levels downstream is likely to have a substantial adverse impact on existing flood liable developments in and around the highway and Cook Drive.

For the purpose of this assessment a flood standard of 0.2% AEP (500-year ARI) was adopted. This is higher than Council's current standard of 1% AEP. This higher standard was adopted to account for possible higher rainfall intensities than currently predicted, and to account for possible impacts of climate change. Note, the 0.2% AEP event generates approximately 43% greater peak flows than the 1% AEP events. The conservative adoption of 0.2% AEP gives greater confidence to the assessment findings.

At less than a metre in average depth, the earthworks needed to raise the proposed developable land above the existing 0.2% AEP flood level is considered economically viable. An assessment was made of what compensatory works would be required to allow such filling and maximise the developable land. The works assessed were:

- Filling of the floodplains to above the 0.2% AEP flood level to provide developable land.
- Compensatory floodways constructed adjacent the filled land to locally offset the loss of floodway conveyance cause by the filling.
- Detention basins constructed upstream of the filled and developed land to offset the loss of floodplain storage. Detention basins are not the only means of providing compensatory flood storage, but they are the most efficient and will yield the greatest developable land yield.

The location and extent of these works are shown on Figures 7.1 & 7.2.

6.2 Filling and Floodways

The flood modelling results for existing conditions reveals relatively shallow inundation over much of the floodplain. Subsequently, provided flood levels are not increased, only relatively shallow filling is required to yield substantial areas of developable land above the 0.2% AEP flood level. A high yield scenario of filling was modelled. The extent of filling was limited by the creek buffer zones and the estimated area required for floodways.

The floodways are required to locally compensate for the filling. They involve excavation of a wide channel in between the filled land and the creek lines. The floor of the floodways are typically 1 to 2 metres higher than the low flow water level in the creek. As such they are generally dry and only carry flow in times of significant creek flow, perhaps only once or twice a year on average.

In this instance the floodways can take advantage of currently cleared land adjacent the creek channels. This allows part of the floodways to be constructed within the environmental creek buffer zones where they do not reduce the developable land yield. Figure 7.1 & 7.2 shows how the modelled floodways partly lie within the creek buffer zones.

It is anticipated that the floodways will be subsequently vegetated. Their hydraulic roughness was modelled as a Manning's n of 0.08, being equivalent to moderately dense vegetation.

It is also anticipated that the excavation of the floodways will provide an economical source of material for the filling.

6.3 Detention Basins

Three sites for possible detention basins were identified – Basin 1, 2 & 3 as shown on Figure A-1 in Appendix A. Of these, Basin 1, which is upstream of the proposed industrial land, has by far the greatest flood storage potential. It could be sized to provide flood mitigation far in excess of that required to compensate for the proposed development within the study area. Basin 2 lies upstream of the proposed residential land and could be sized to either offset the impacts of the residential development alone, or offset all proposed development in the study area. Basin 3 is on tributary N, upstream of Englands Rd and can be sized to compensate for either the industrial development or all of the proposed development in the study area.

All basins are in rural land. At present there are no significant buildings or structures within the basin sites and the temporary increased inundation by flood water would have little adverse impact. Basin 2 is essentially sterilised from residential development by the buffer zones surrounding the quarry. Basin 1 is likewise within the 750 m quarry buffer zone. While industrial development could be possible within this zone, the site, due to its topography and minimal development upstream, was considered the most suitable site for a large detention basin.

During the investigations it was observed that, due to its strategic location, basin 1 could be used to provide significant flood mitigation benefits downstream of the Study Area – including offsetting impacts of the future Pacific Highway upgrade and lowering flood levels in the Isles Industrial area and around the Hospital. Whilst the examination of these is outside the scope of this study, it was decided to model a larger than necessary basin to see what benefits could be achieved.

Three basin scenarios were modelled:

- **Scenario A – Small Basins**. Basins 1A and 2A are conventional culvert basins sized to comfortably compensate for the development in events of 1% and 0.2% AEP. Basin 1 included excavation upstream of the basin wall as a means to both provide fill material for the basin wall and additional detention storage.

- **Scenario B – Modified Basins.** Basin 2A is the same as in Scenario A. Basin 1 however is modified. It's wall is shifted downstream and merged with a proposed road. It's main outlet to Trib A is a constricted open channel rather than a culvert. It was also modelled with far greater excavation upstream of its wall to both increase its flood storage and as a source of fill for the industrial lands. This modified basin better suits potential staging of works.

- **Scenario C – Large Basins.** Basins 1B and 2B are conventional culvert basins as per Scenario A, but are substantially larger. They are sized to not only compensate for the development but to provide significant flood relief to areas downstream of the study area.

6.4 Flood Behaviour of Development Scenarios

The flood behaviour for the three development scenarios was modelled and the following noted:

- Modelling with filling and floodways but no detention basins found a slight, but significant increase in peak discharge and flood level downstream of the developments.

- Under all detention basin scenarios and events modelled, the flood levels downstream of the study area (opposite Bishop Druitt College) and further downstream are reduced over existing conditions.

- A study of the flow results shows that a small basin 2 provides minimal benefit. Greater value would be gained by optimising the floodways downstream and relying solely on basin 1 for flood storage.

- As is expected, Scenario C, with large detention basins, provides the greatest reduction in flood level and peak flows. The peak 0.2% AEP flow downstream of the study area opposite Bishop Druitt College was reduced by 25% from 340m^3/s to 255m^3/s.

6.5 Planning, Construction and Staging

The modelling found that on average 0.8 - 0.9m of filling is required across the industrial floodplain and 0.6m across the residential floodplain. The cost of such earthworks is anticipated to be viable at approx 10-15% of the developed land's value. This will partly depend on the source of fill and how far it has to be transported. The most economical source will be the excavation of the adjacent floodways. Under all three scenarios the floodway excavation provides approximately one half of fill required for the industrial land and only one tenth for the residential land. It will therefore be in the developer's interest to expand the floodways as a source of additional fill.

The filling of the floodplain and the excavation of the adjacent floodways can be staged. That is, the floodways need only be constructed opposite the particular area of the floodplain being filled and developed at that time.

While filling and floodways can be easily staged to accommodate the rate of development, the construction of detention basins is more difficult. While not impossible, basins can be difficult to economically stage and, as their location is somewhat fixed, will generally be in land under different control than that being developed. To ensure no adverse impacts downstream, the detention basins will need to be constructed early in the development process. It is the ability to plan, fund and construct the basin(s) that must be carefully considered.

6.6 Alternative Scenarios without Detention Basin 1.

Development scenarios through both floodplains is possible without the need for a detention basin. To offset the impact of filling, the excavated floodways would need to be greatly increased and carefully designed so that they not only compensate for the loss of conveyance, but also provide the required flood storage. Such floodways and storages would required far greater land and would seriously reduce the developable land yield.

From a flood mitigation stand point, the provision of detention basin(s) is the preferred development scenario as it is the most effective means of providing controlled flood storage while still allowing maximum development land.

6.7 Other Considerations.

The proposed development through the industrial land is important to the overall development strategy of the valley. Not only as a source of industrial land, but also as a means to provide better flood access to the proposed residential lands and as an alternative quarry haul route. If the development through the industrial lands were not to proceed, North Boambee Road would require significant raising and culvert works in order to provide an acceptable level of flood access to the existing and proposed residential areas. The issue of quarry haul trucks travelling through residential roads would remain and become greater as residential development proceeds.

The incremental cost of increasing Basin 1 from a small to a large basin is not great while the flood mitigation benefit downstream is likely to be large. As such, there may be scope to include at least part of the basin's construction within Council general flood mitigation program. Indeed, this could be a means of kick starting the initial staging of the basin, with section 94 contributions funding the remaining.

The preliminary alignment of the future highway bypass is shown on in the figures. The modelling of the highway bypass is beyond the scope of this study, however, it is noted that it has the potential to adversely impact on flood behaviour. The highway will cut across the floodplain on an embankment which will remove approximately 52,000 m^3 of floodplain storage in the 0.2% AEP event. To ensure that the bypass does not adversely impact of flood behaviour, it will be necessary for its design to include:

- Bridge and or culvert crossings of the creek lines with adequate conveyance capacity; and;
- Compensatory flood storage. Such storage could be won by excavation at appropriate locations in the flood plain or the construction of detention basins.

The basins discussed above could be sized to account for the future bypass and perhaps the RMS should be included in the planning for basin 1.

7 CONCLUSIONS & RECOMMENDATIONS

This study has considered the impact on flood behaviour of potential development in the North Boambee Valley. The valley is characterised by hilly terrain with two gentle sloping floodplains. Flood modelling has found that a fair portion of these floodplains becomes inundated in the 0.2% AEP (500-year ARI) flood event. However, for much of the floodplain, this inundation is quite shallow, at typically 0.5 metres or less.

Once all other environmental constraints are accounted for, the floodplains have the potential to provide substantial developable area. Area that is quite flat and hence suitable for large industrial lots, which is in short supply in the region. The southern floodplain between Newports Creek and Trib A is considered suitable for industrial zoning while the northern floodplain offers the potential for residential land. The southern industrial zone can also offer an alternative haul route for the quarry and improved flood access to the proposed residential land.

The study examined several development scenarios that maximise the developable land in the floodplain while providing compensatory works to mitigate adverse flood impacts. The works include the filling of the floodplain, excavation of floodways and compensatory flood storage in the form of detention basins. The works were sized for the present 0.2% AEP (500-yr ARI) design event. This higher than normal flood standard was adopted to allow for the likely effects of climate change and the possible current 'under' estimation of local rainfall intensities. This adoption of a conservative flood standard gives greater confidence in the results.

The results of the modelling found that:

- Flood mitigation works, in the form of filling, floodways and detention basins can yield substantial developable land in the floodplain with acceptably low flood risk and can mitigate any adverse impacts elsewhere;

- A large detention basin at site 1 (Basin 1B) is recommended for consideration. It can compensate for the filling of the proposed industrial land and the residential land and can provide significant flood mitigation benefits to flood affected properties downstream of the study area. In addition, it could compensate for the future Pacific Highway works. In terms of relative benefits of the basin to the three downstream beneficiaries, an estimate has been made based on the impounded storage benefiting each area as shown below in Table 7.1

Table 7.1 – Basin 1B Benefits

Beneficial Area	Impounded Storage Volume (m^3)	Percentage of Total Storage attributable to beneficial area
Proposed NBVw development	75,000 m^3	22%
Offsetting Pacific Highway Upgrade Embankment	50,000 m^3	15%
Downstream Flood Mitigation Benefits	215,000 m^3	63%
TOTAL	340,000 m^3	100%

- Should a large Basin 1B proceed, basins at sites 2 and 3 are unlikely to provide economical benefit and are not recommended.

- However, should large Basin 1B not proceed, then a smaller basin at site 1 is not recommended. Basin sites 2 or 3 offer more favourable locations to build a smaller basin sized solely to compensate for the proposed development. Either site can support a basin sufficiently large to compensate for all proposed development in the whole study area. The construction of one basin at either site is likely to be the most economical solution. The alternative is to construct smaller basins at both sites which has the advantage of disturbing the benefit more evenly throughout the two main valleys of the study area.

The recommended development and flood mitigation works are summarised in Table 7.2 to 7.4 and shown on Figures 7.1 to 7.3. Their impact on peak flood levels is shown on Figures 7.4 to 7.6 and summarised in Table 7.3.

With regard to the funding of the detention basin the following is noted:

- To ensure no adverse impact of flooding downstream, the construction of the detention basin should proceed early in the development of the valleys. Certainly before substantial filling of the floodplains has occurred. As such, it is unlikely that the basin can be constructed from received section 94 contributions. Forward funding with associated borrowing and interest costs will most likely be required.

- As the large basin 1B provides substantially greater benefit than that required to compensate for the development, only a small portion of its cost can be recouped through section 94 contributions. Other sources of funding would need to be found.

The cost implications of various basin sizes and their possible implementation through Section 94 funding is discussed in Appendix A – Section 94 costing considerations.

Should funding constraints preclude large basin 1B then either basin 2 or 3, or smaller versions of both, should proceed, refer to Appendix A. A single basin at either site can provide the necessary flood compensation at a construction cost in the order of $1.1 to 1.2 million. Constructing smaller basins as both sites is likely to have higher construction costs in the order of $2 million. Further detailed investigation is required at both sites to determine the most economic solution, but, to the current level of investigation, a single basin at site 3 offers the greatest economy by a small margin.

Table 7.2 – Recommended Filling & Floodways

Location	Industrial Area on Newports Creek & Trib A	Residential Area on Trib C
Filling area & developable land yield (Ha)	22.1	16.7
Average fill depth (m)	0.70	0.64
Fill volume (cu.m)	150,000	107,000
Floodway area (Ha)	9.6	3.2
Average floodway depth (m)	0.87	0.60
Excavation volume (cu.m)	84,000	19,000

Table 7.3 – Recommended Large Detention Basin 1B

Top of wall level	24.0 mAHD
Crest width	6m (12m along road)
Ave wall height	4.8 m
Max wall height	7.0 m
Ave side slopes	1:3
Wall length	390 m
Approx wall volume	52,000 m^3
Excavation upstream of wall	33,000 m^3
Outlets (RCBC)	3 by 3.6m x 2.4m RCBC
Peak 0.2% AEP Flow In	195 m^3/s
Peak 0.2% AEP Flow Out	130 m^3/s
0.2% AEP Top water level	23.6 mAHD
Approx 0.2% AEP storage (1)	343,000 m^3
Approx construction cost(2)	$4,750,000

Notes: (1) Detention storage calculated as the additional flood volume over the existing flood surface.
(2) Construction costs exclude land and compensation costs.

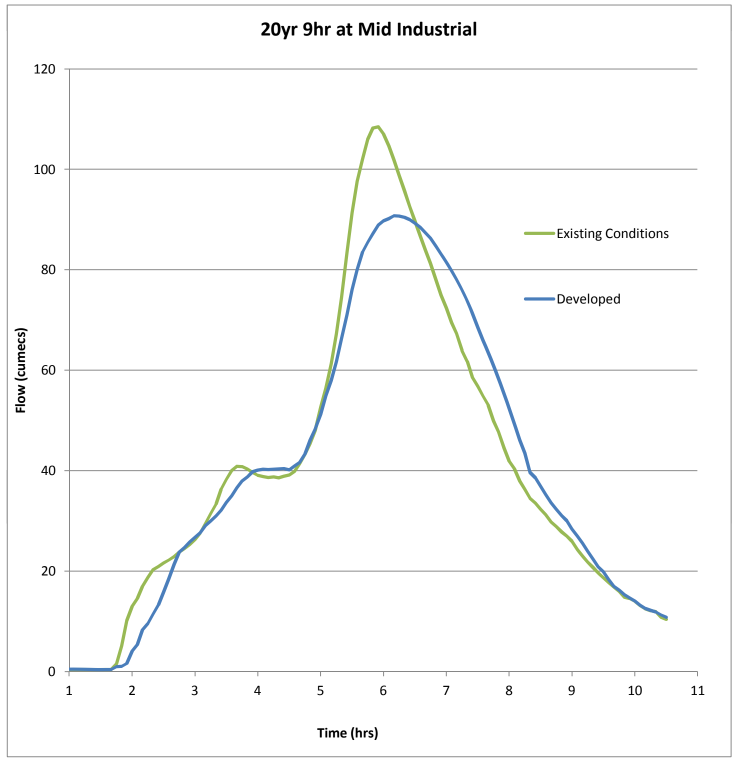
Table 7.4 - Tuflow Results for Recommended Development

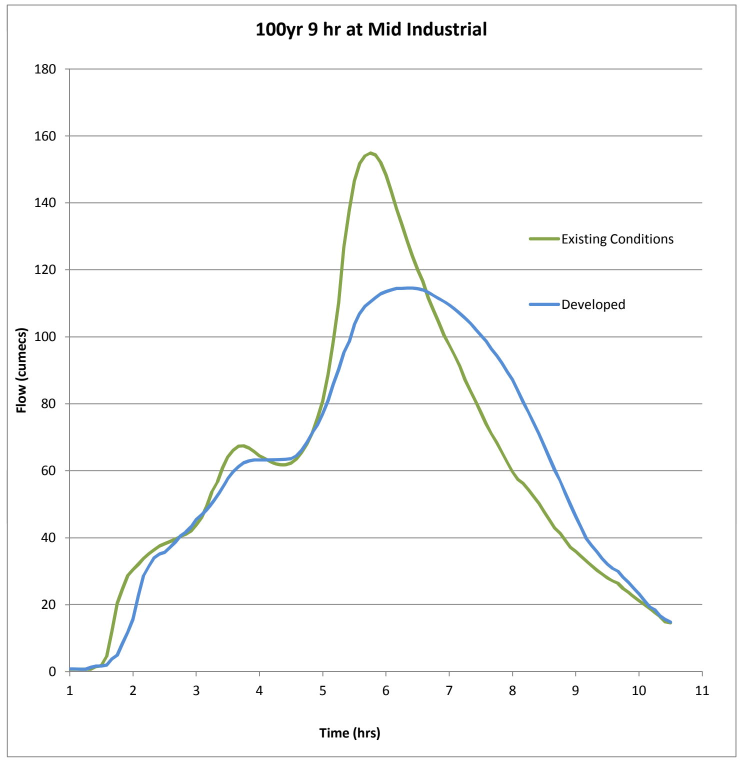
Peak Flows and Levels (with difference over existing conditions in brackets)

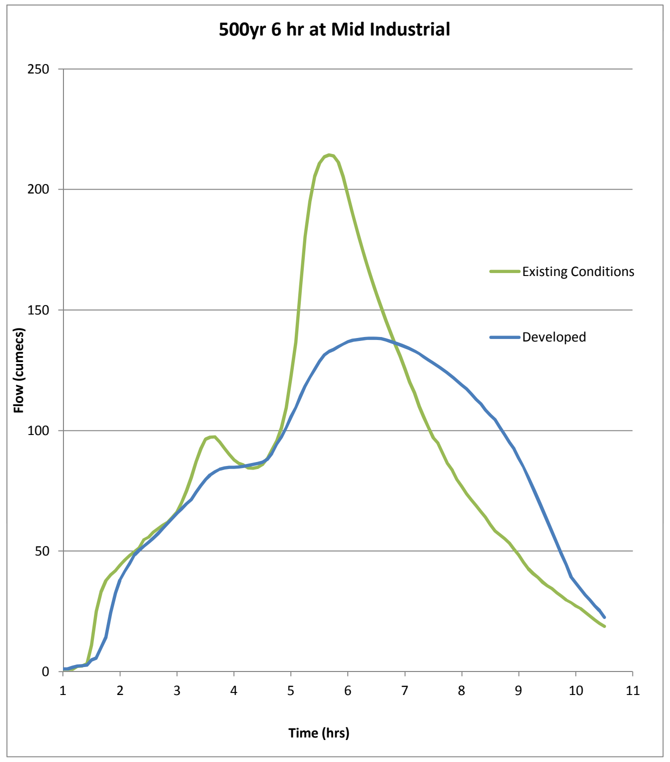
Event	Mid Isles (cumecs)	Model Outlet (cumecs)	Over North Boambee Road # 1 (cumecs)	North Boambee Rd Bridge (cumecs)	Over North Boambee Road # 2 (cumecs)	Basin 2 (cumecs)	Englands Rd at Trib A (cumecs)	u/s Future Highway (cumecs)	Proposed Residential North (cumecs)	Proposed Residential Mid (cumecs)	Proposed Residential South (cumecs)	Proposed Industrial North (cumecs)	Proposed Industrial South (cumecs)	d/s of Basin 1 North (cumecs)	d/s of Basin 1 South (cumecs)	Opposite BDC (cumecs)	Model dswl (mAHD)	Isles Floodway (mAHD)
20-Year ARI																		
9 hr	139.5	136.4	0.0	36.0	17.4	21.0	60.2	102.4	10.6	34.6	0.0	44.8	46.0	37.1	48.5	139.0	6.3	7.1
	(-16.9)	(-17.3)	(-0.1)	(-0.6)	(-1.6)	(0.0)	(-0.0)	(-18.8)	(0.6)	(6.3)	(-7.0)	(0.6)	(-19.1)	(5.6)	(-21.1)	(-17.6)	(-0.16)	(-0.54)
100-Year ARI																		
9 hr	196.9	192.4	0.5	29.8	49.4	28.7	85.4	131.7	14.5	47.6	0.0	53.9	60.7	43.7	64.2	196.0	6.9	7.1
	(-40.8)	(-35.8)	(-0.6)	(-2.2)	(3.8)	(0.0)	(0.0)	(-45.2)	(0.6)	(11.4)	(-12.1)	(-25.2)	(-15.4)	(-7.0)	(-27.1)	(-41.5)	(-0.32)	(-0.55)
500-Year ARI																		
9 hr	264.2	256.9	1.3	32.9	82.6	39.2	117.9	162.1	19.6	65.3	0.0	63.2	75.1	50.9	79.0	262.3	7.4	7.5
	(-75.9)	(-96.1)	(-2.5)	(-1.2)	(10.5)	(0.0)	(0.0)	(-83.0)	(0.7)	(20.9)	(-22.0)	(-63.8)	(-12.4)	(-23.0)	(-41.5)	(-76.7)	(-0.34)	(-0.37)
31 March 2009																		
	259.5	271.1	1.2	51.6	70.3	38.1	110.5	169.2	18.8	62.4	0.0	67.2	79.2	53.0	83.0	256.5	7.3	7.5
	(-58.0)	(-62.0)	(-1.8)	(3.2)	(-2.4)	(0.0)	(0.0)	(-55.9)	(0.7)	(19.2)	(-20.1)	(-45.4)	(-4.9)	(-14.5)	(-28.5)	(-57.9)	(-0.28)	(-0.29)

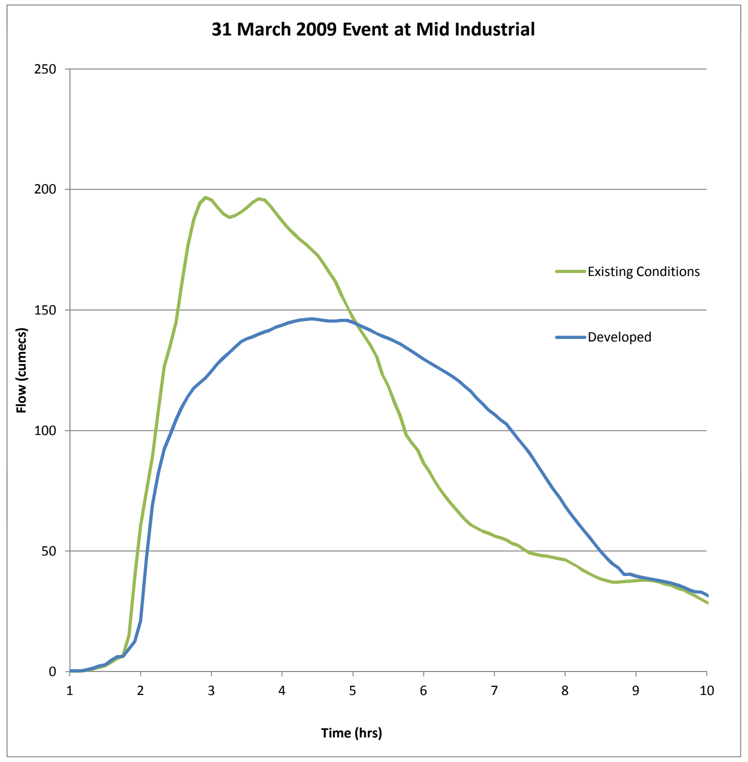
Event	Proposed Industrial Mid North (mAHD)	Basin 1 (mAHD)	Englands Rd at Trib A (mAHD)	u/s of Englands Road Culverts No. 4 (mAHD)	u/s of Englands Road Culverts No. 3 (mAHD)	u/s of Englands Road Culverts No. 2 (mAHD)	u/s of Englands Road Culverts No. 1 (mAHD)	Res 1 (mAHD)	Proposed Residential Res 2 (mAHD)	Proposed Residential Res 3 (mAHD)	Proposed Residential Res 4 (mAHD)	Proposed Industrial Mid South (mAHD)	Basin 2 (mAHD)	d/s of Basin 2 (mAHD)	d/s of Basin 1 South (mAHD)	d/s of Basin 1 North (mAHD)	u/s of NB Rd (mAHD)	Mid Industrial (cumecs)
20-Year ARI																		
9 hr	13.2	21.3	26.2	28.5	25.3	25.2	15.5	10.2	13.4	14.2	19.7	14.3	20.7	20.1	18.6	17.8	9.4	90.7
	(-0.20)	(0.61)	(0.00)	(-0.00)	(0.00)	(0.00)	(0.00)	(-0.07)	(-0.14)	(0.00)	(0.01)	(-0.34)	(0.00)	(0.02)	(-0.44)	(-0.40)	(-0.06)	(-17.7)
100-Year ARI																		
9 hr	13.4	22.3	26.3	28.6	25.3	25.2	15.6	10.2	13.6	14.3	19.7	14.5	20.8	20.2	18.7	17.8	9.6	114.6
	(-0.51)	(1.54)	(0.00)	(0.00)	(0.00)	(0.00)	(0.00)	(-0.10)	(-0.04)	(0.01)	(0.02)	(-0.30)	(0.00)	(0.01)	(-0.46)	(-0.45)	(-0.11)	(-40.3)
500-Year ARI																		
9 hr	13.5	23.6	26.4	28.7	25.3	25.3	15.7	10.2	13.8	14.4	19.8	14.6	21.0	20.3	18.8	17.9	9.8	138.3
	(-0.73)	(2.77)	(0.00)	(0.00)	(-0.01)	(0.01)	(0.00)	(-0.16)	(0.07)	(0.01)	(0.02)	(-0.24)	(0.00)	(0.01)	(-0.47)	(-0.49)	(-0.17)	(-76.0)
31 March 2009																		
	13.5	24.0	26.4	28.7	25.3	25.3	15.7	10.2	13.8	14.4	19.8	14.6	21.0	20.3	18.9	17.9	9.8	146.3
	(-0.61)	(3.20)	(0.00)	(0.00)	(0.00)	(0.01)	(0.00)	(-0.15)	(0.05)	(0.01)	(0.02)	(-0.19)	(0.00)	(0.01)	(-0.42)	(-0.44)	(-0.14)	(-50.3)

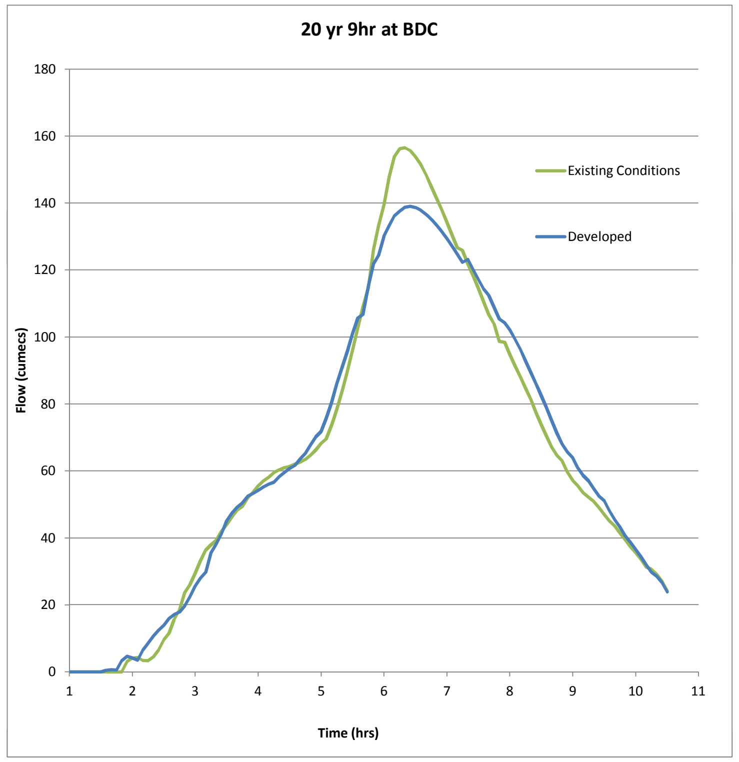
Hydrographs, Existing & Developed Conditions

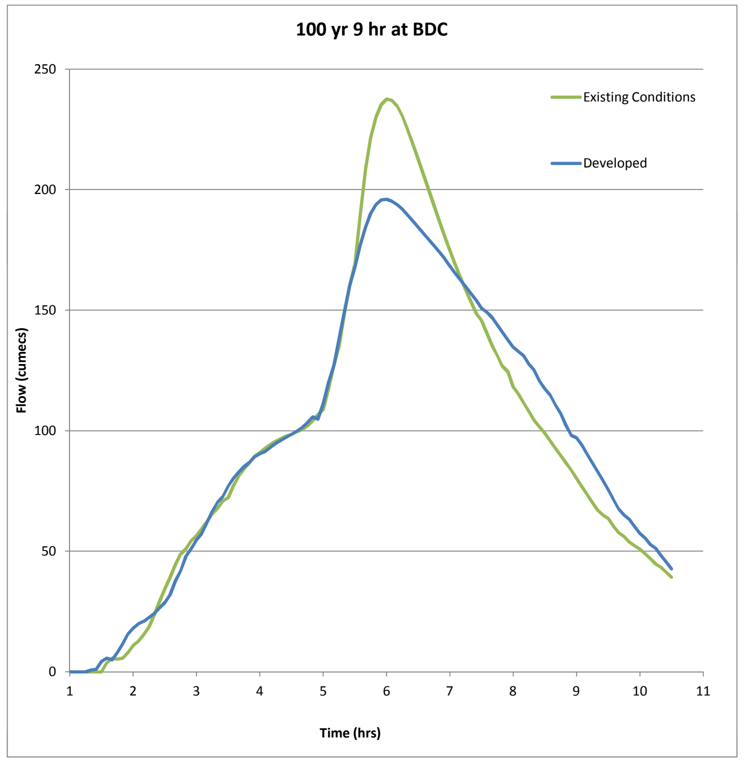


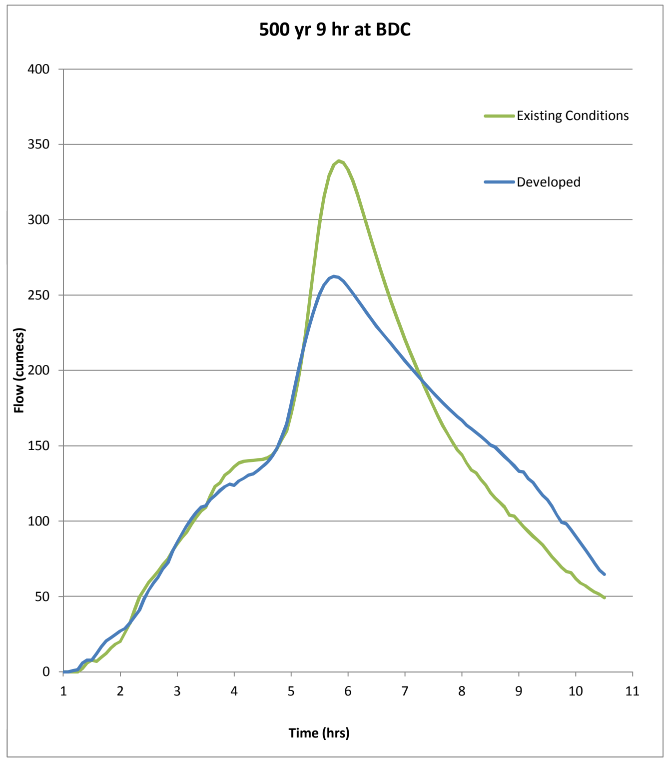


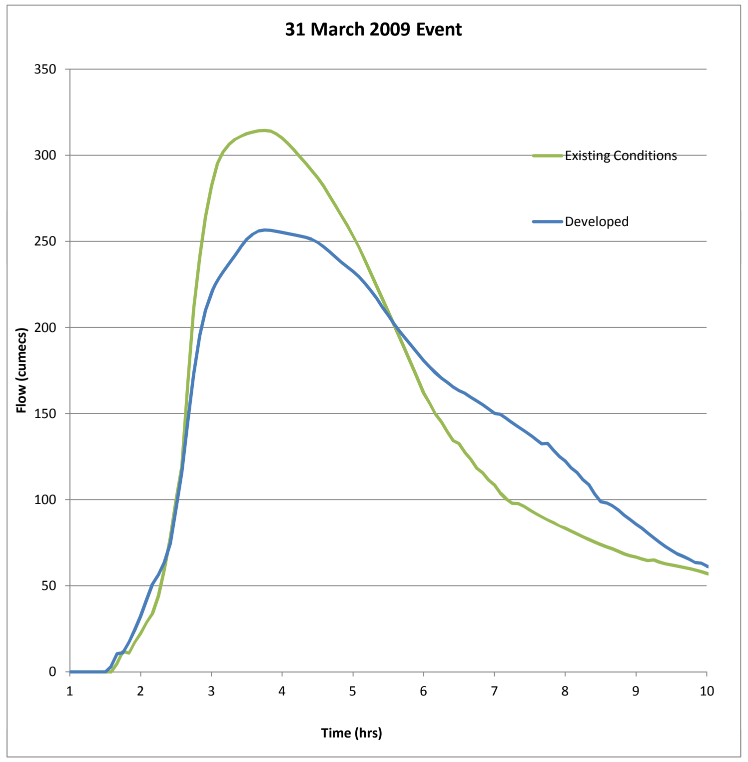












8 REFERENCES

1. Boambee Creek and Newports Creek Flood Study, (WMA Water, Jan 2011).

2. North Boambee Valley Flood Study, (Ref Bewsher Consulting, 1991).

3. Australian Rainfall and Runoff, (Institute of Engineers, Australia, 1987)

Appendix A – Section 94 Costing Considerations

Construction Cost Estimates

The construction costs of the basin was estimated based on key quantities. Rates for which were disseminated from the tendered rates for the recently constructed Bennett's Road Basin, as shown on the Table A-2 – Construction Rates. The estimated construction cost for the recommended large detention basin (Basin 1B) is $4.75 million. This excludes land acquisition costs and design fees and contingencies.

It is noted that the impacts of the basin options may require additional filling in the floodplain. These costs are also noted in the Estimates.

Section 94 Costing Implications

Building the large detention basin is a significant capital cost funding project. As noted in Section 7, only 22% of this cost is applicable to the North Boambee West Study area. As such, before this basin can be constructed, funding from sources other than section 96 would need to be confirmed.

The following process is envisaged:

1. A detailed flood study is undertaken to optimise the design.
2. At the same time detailed negotiations are entered into with the NSW Government for possible flood mitigation funding and the RMS for funding for the possible storage offsets for the Pacific Highway Bypass.
3. Scenario 1: If agreement can be reached between all parties and funding secured, then Basin 1B would be proceeded with.
4. Scenario 2: If agreement is not reached then smaller more efficient basin(s) sized just to compensate for the development within the Study area would be optimised. Some preliminary examination of possible options has been undertaken, with two sites identified, Basin 2 and basin 3 as shown on Figure A-1.

It is noted that the detention basin(s) are required early in the development of the valley, particularly for the proposed Industrial Area. As such the basin(s) would need to be forward funded by Council with the cost recouped through section 96 contributions.

Costing for Scenario 1:

Based on the costing in Table A-2, the proportion of costs of the Detention basin to the three beneficiaries is:

Table A-1 – Scenario 1 – Cost Apportionment

Beneficial Area	Percentage of Total Storage attributable to beneficial area	Cost Apportionment $
Proposed NBVw development	22%	$1,045,000
Offsetting Pacific Highway Upgrade Embankment	15%	$715,000
Downstream Flood Mitigation Benefits	63%	$2,990,000
TOTAL	100%	$4,750,000

Costing for Scenario 2:

Three possible basin options sized to solely compensate for the proposed development within the North Boambee West Study Area were identified. Their construction only costs were estimated at:

- A single basin at site 2 $1.2 million
- A single basin at site 3 $1.1 million
- Smaller basins at both sites 2 & 3 $2.0 million

Section 94 Cost Allowance

As can be seen the construction cost applicable to the North Boambee West Study Area under the two scenarios is between $1.05 and $2.0 million.

For the purpose of Section 94 calculations, these costs excludes land purchases, design, survey, project management and contingencies. As the work will need to be forward funded, principal and interest components should also be allowed for in the Section 94 calculations.

Table A-2 - Construction Rates

North Boambee Valley - Detention Basin Costing

Construction Rates

Construction rates are based on those tendered by Peter Ryan Earthmoving for the construction of the Bennet's Road detention basin.
The schedule was reduced and totalled as follows:

Reduction of Bennet Road basin costs

Items	Description	Amount	Assumed proportional to	Unit	Quantity	Rate
1 to 3	Preliminaries, enviros	$103,000	Fixed	Item	1	$103,000
4	Foundations	$441,000	Wall length	lm	230	$1,917.39
5.1		$2,000		Not relevant		
5.1.1 to 5.1.9	RCBC culvert inc headwalls	$334,000	Culvert length	lm	53	$6,302
5.2 to 5.3.6	Culvert head and tail works	$141,000	Culvert	Each	1	$141,000
6 to 6.2.6	Embankment	$1,362,000	Wall volume	cu.m	32805	$41.52
6.3 to 6.6.2	Spillway, approx 55 cumecs & 8m wall height.	$327,000	Flow, wall height and topography	Item	1	$327,000
7	Misc inc demolition, service relocation and road building.	$106,000		Not relevant		
	TOTAL	**$2,816,000**				

From this reduction the following rates were established.

Item	Description		Unit	Rate	
1	Prelims		Item	$100,000	
2	Foundations		lm	$1,900	
3	Culverts - RCBC	3.3 x 1.2m	lm	$6,300	
		3.6 x 2.4m	lm	$8,500	By judgement. Somewhat proportional to width but not height.
		1.8 x 1.5m	lm	$4,000	
		2.1 x 1.8m	lm	$4,500	
4	Culvert head and tail works		each	$100,000	
5	Embankment		cu.m	$41.50	
6	Spillway		Item		By judgement, based on peak 1% flow and wall height.

Table A-3 - Estimated Basin Construction Costs

Estimated Basin Construction Costs

Basin 1B

Large detention basin as recommended in the final flood study located upstream of the proposed industrial area.
Storage volume of 340,000 cu.m.

Item	Description	Quantity	Unit	Rate	Amount	
1	Prelims	1	Item	$100,000	$100,000	
2	Foundations	390	lm	$1,900	$741,000	
3	Culverts - 3 by 3.6 x 2.4m RCBC	100	lm	$8,500	$850,000	
4	Culvert head and tail works	3	each	$100,000	$300,000	
5	Embankment	52000	cu.m	$41.50	$2,158,000	
6	Spillway, 120 cumecs & 7m high	1	Item	$600,000	$600,000	$4,749,000

Basin 1A

As per basin 1B, but sized only to compensate for the proposed valley development.
Storage volume of approx 80,000 cu.m.

Item	Description	Quantity	Unit	Rate	Amount	
1	Prelims	1	Item	$100,000	$100,000	
2	Foundations	325	lm	$1,900	$617,500	
3	Culverts 5 by 3.6 x 2.4m RCBC	112	lm	$8,500	$952,000	
4	Culvert head and tail works	3	each	$100,000	$300,000	
5	Embankment	19000	cu.m	$41.50	$788,500	
6	Spillway, 120 cumecs & 5m high.	1	Item	$430,000	$430,000	$3,188,000
7	Additional filling through industrial area	33150	cu.m	$20	$663,000	
						$3,851,000

Basin 2

Located on Trib C upstream of the proposed residential area.
Storage volume of approx 75,000 cu.m.

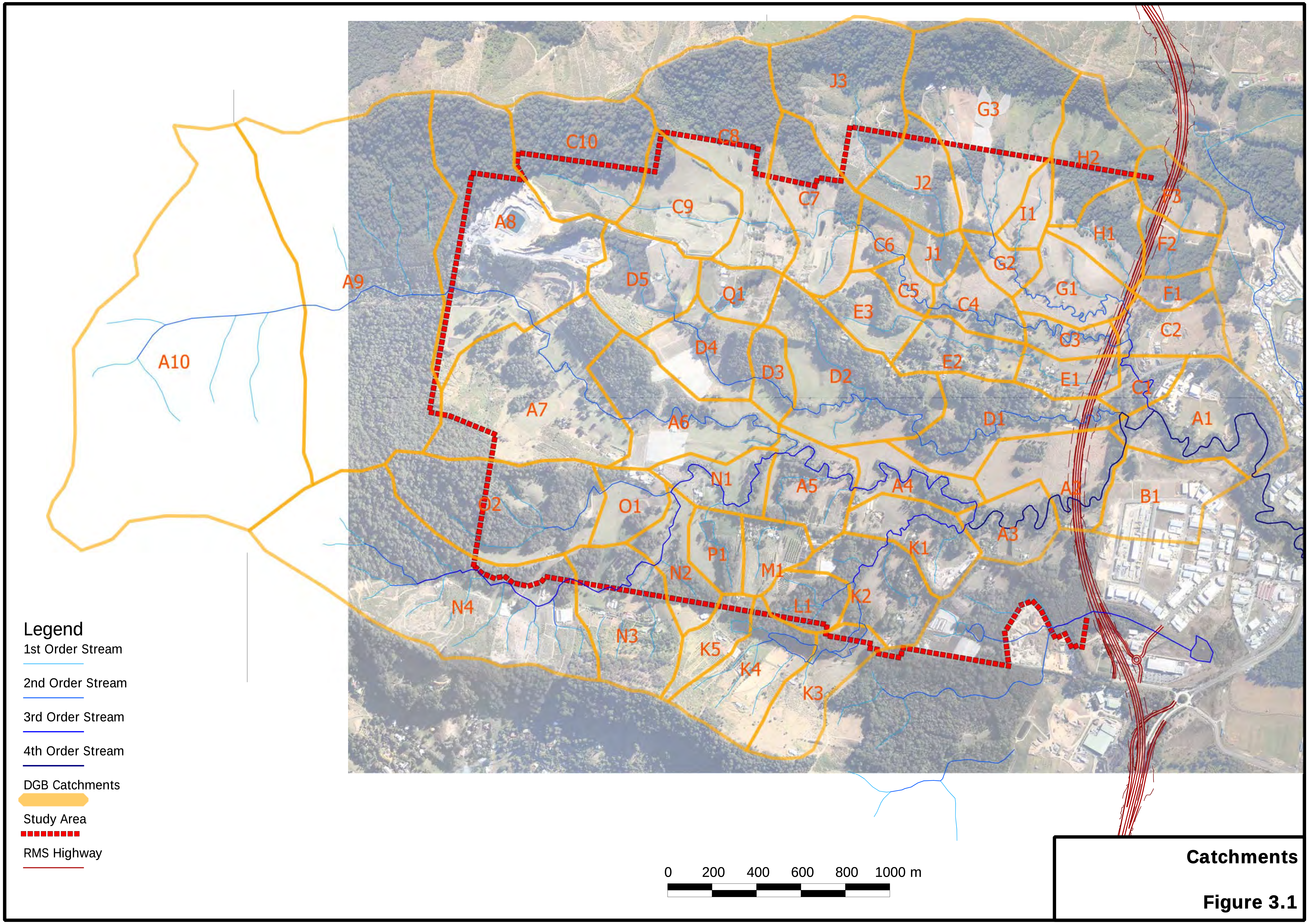
Item	Description	Quantity	Unit	Rate	Amount	
1	Prelims	1	Item	$100,000	$100,000	
2	Foundations	100	lm	$1,900	$190,000	
3	Culverts 1 by 1.8 x 1.5m RCBC	40	lm	$4,000	$160,000	
4	Culvert head and tail works	1	each	$100,000	$100,000	
5	Embankment	9400	cu.m	$41.50	$390,100	
6	Spillway, 31 cumecs & 7.5m high.	1	Item	$250,000	$250,000	$1,190,100
7	Additional filling through industrial area	38500	cu.m	$20	$770,000	
8	Reduced filling through residential (150m	-25050	cu.m	$20	-$501,000	
						$1,459,100

Basin 3

Located on Trib N upstream of the Englands Rd.
Storage volume of approx 75,000 cu.m.

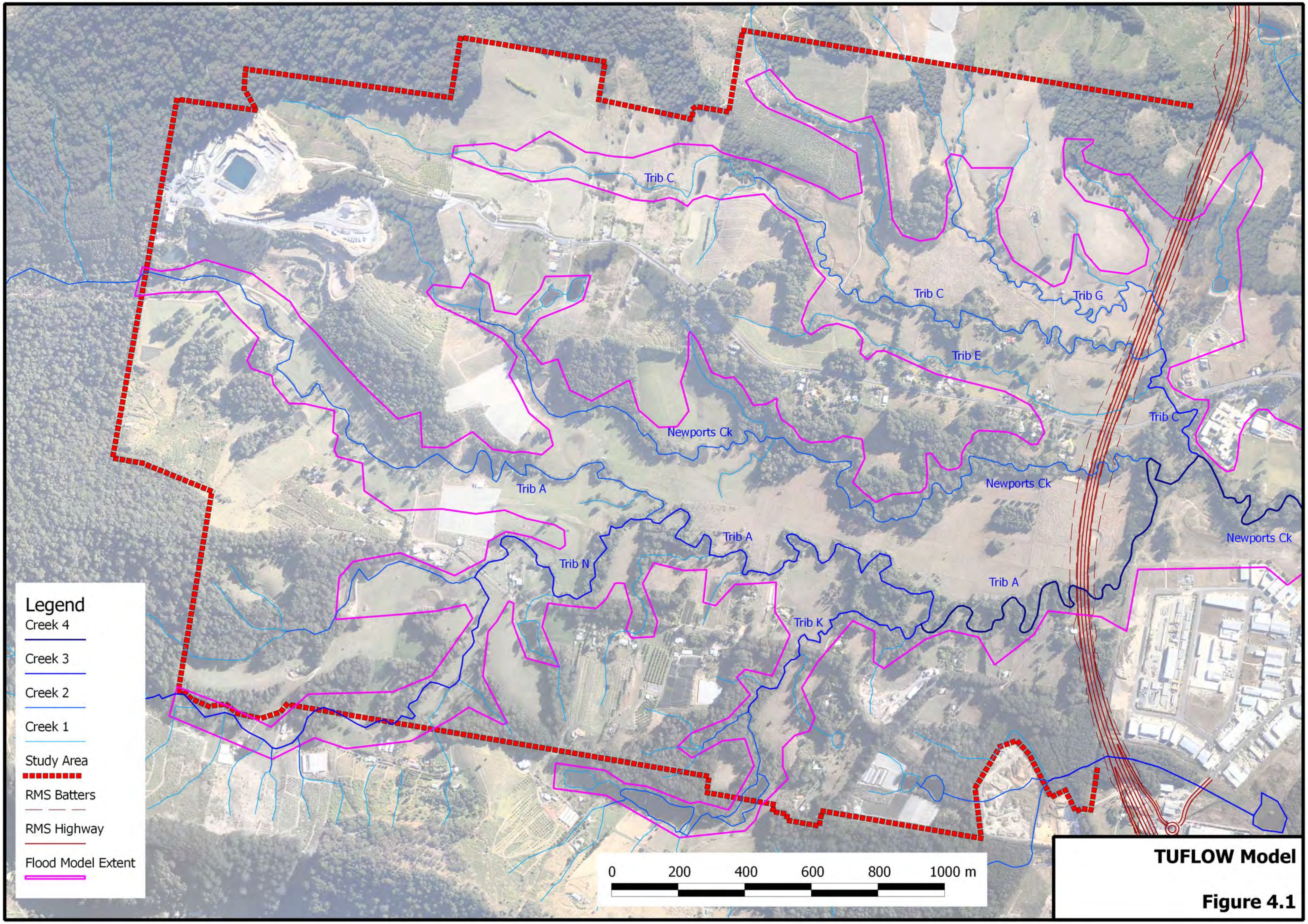
Item	Description	Quantity	Unit	Rate	Amount	
1	Prelims	1	Item	$100,000	$100,000	
2	Foundations	104	lm	$1,900	$197,600	
3	Culverts 1 by 2.1 x 1.8m RCBC	34	lm	$4,500	$153,000	
4	Culvert head and tail works	1	each	$100,000	$100,000	
5	Embankment	6800	cu.m	$41.50	$282,200	
6	Spillway, 40 cumecs & 6.5m high.	1	Item	$280,000	$280,000	$1,112,800
7	Additional filling through industrial area	33150	cu.m	$20	$663,000	
8	Reduced filling through residential (Nil)	0	cu.m	$20	$0	
						$1,775,800

FIGURES



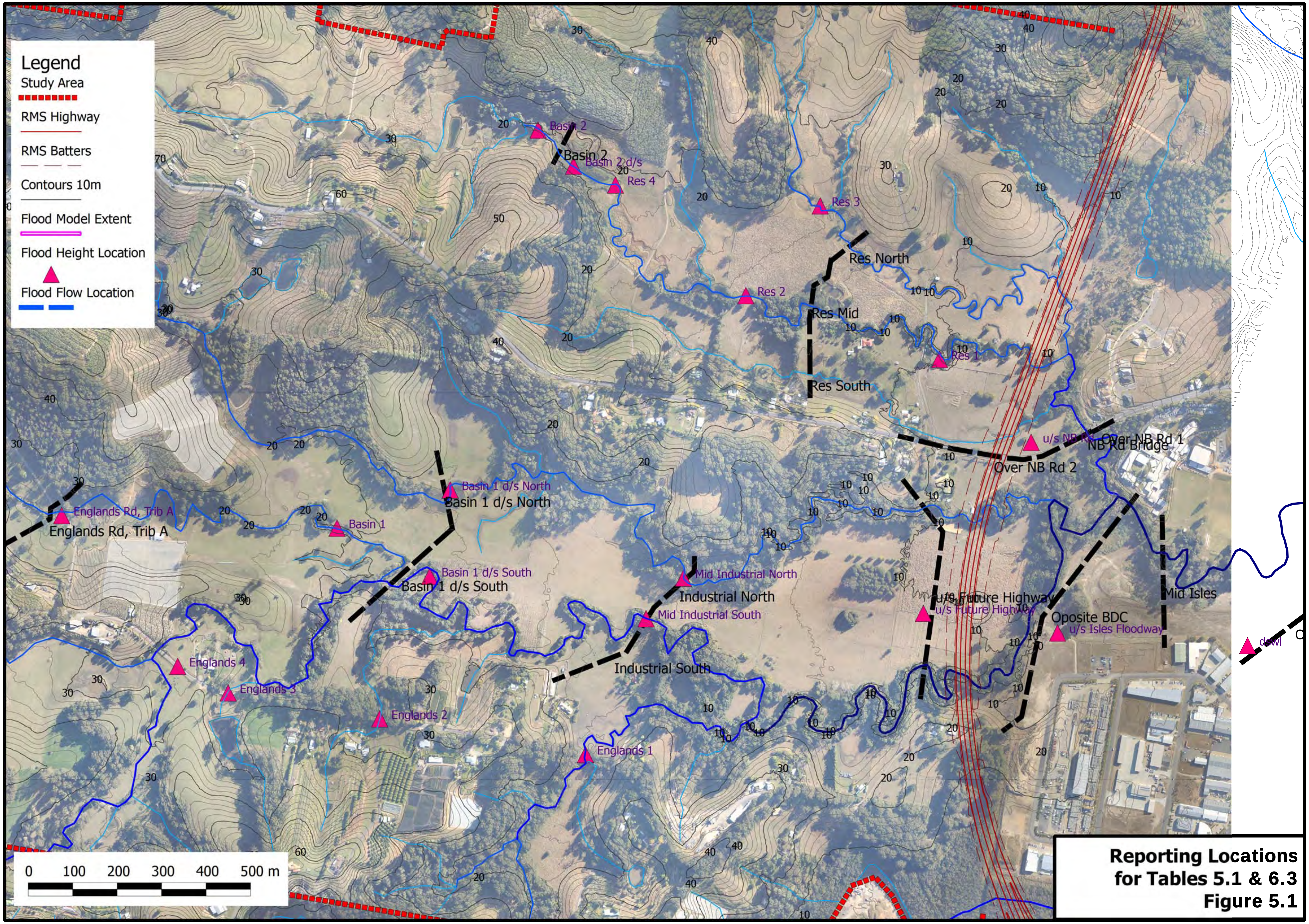
Catchments

Figure 3.1



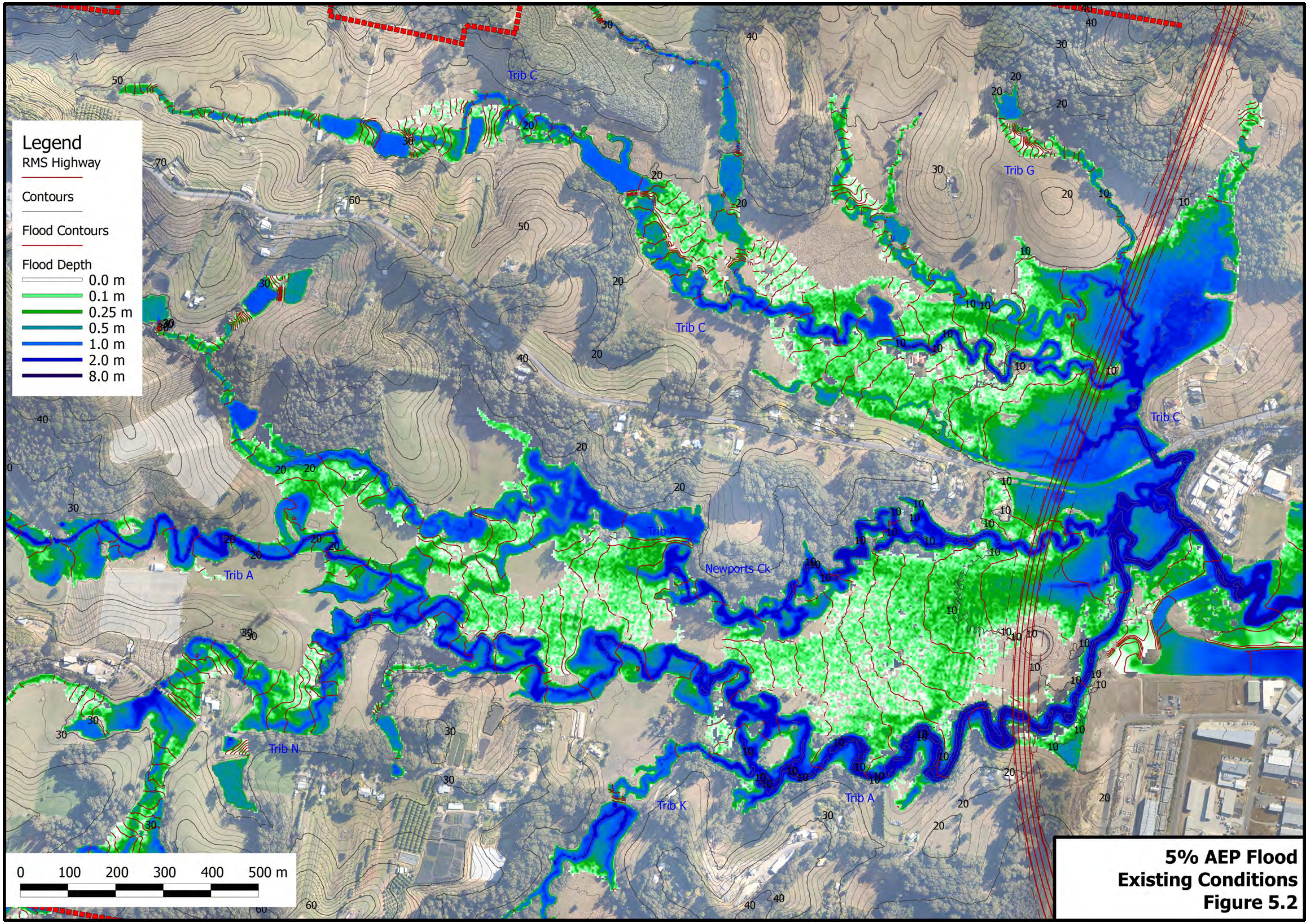
TUFLOW Model

Figure 4.1

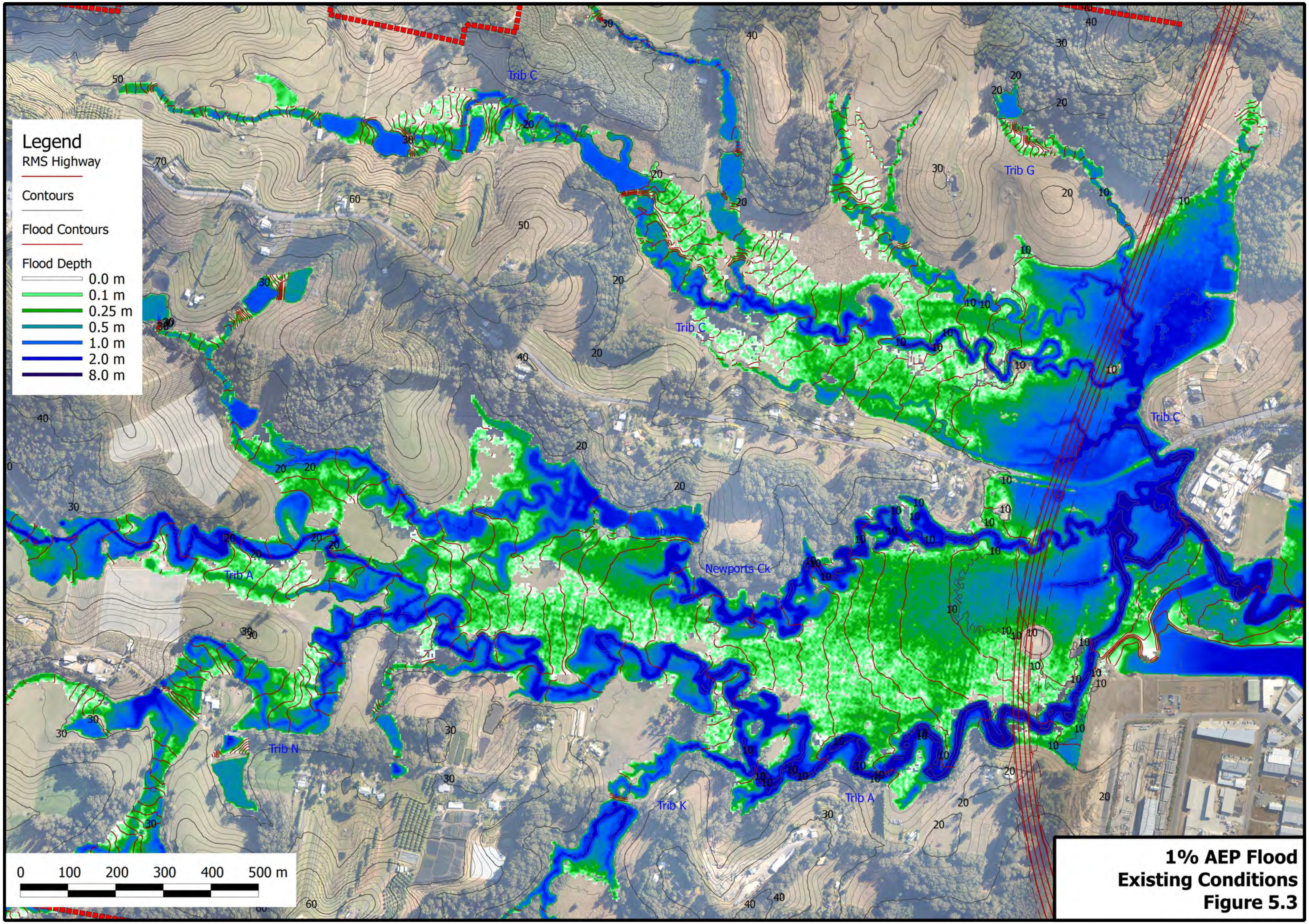


Reporting Locations for Tables 5.1 & 6.3

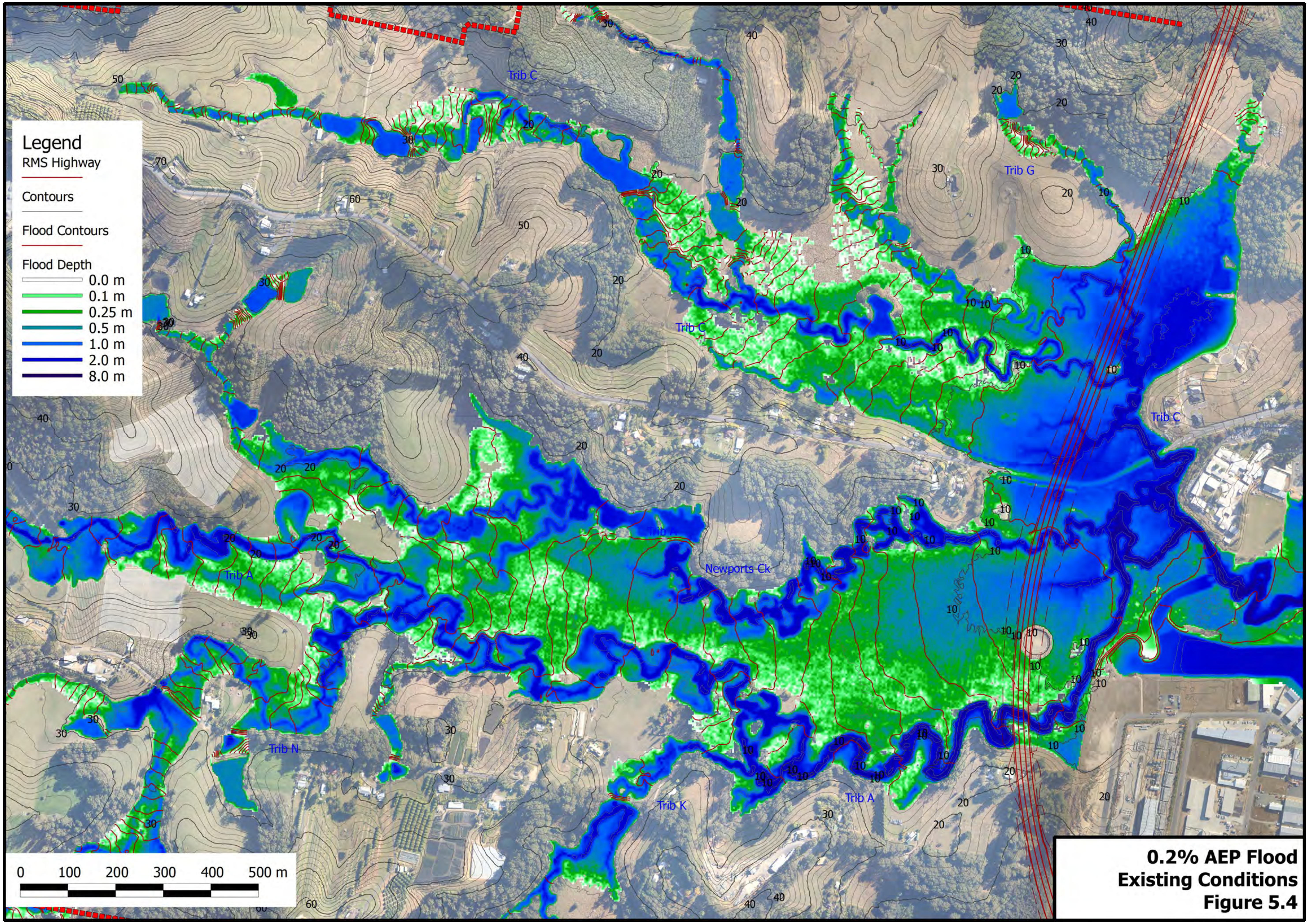
Figure 5.1



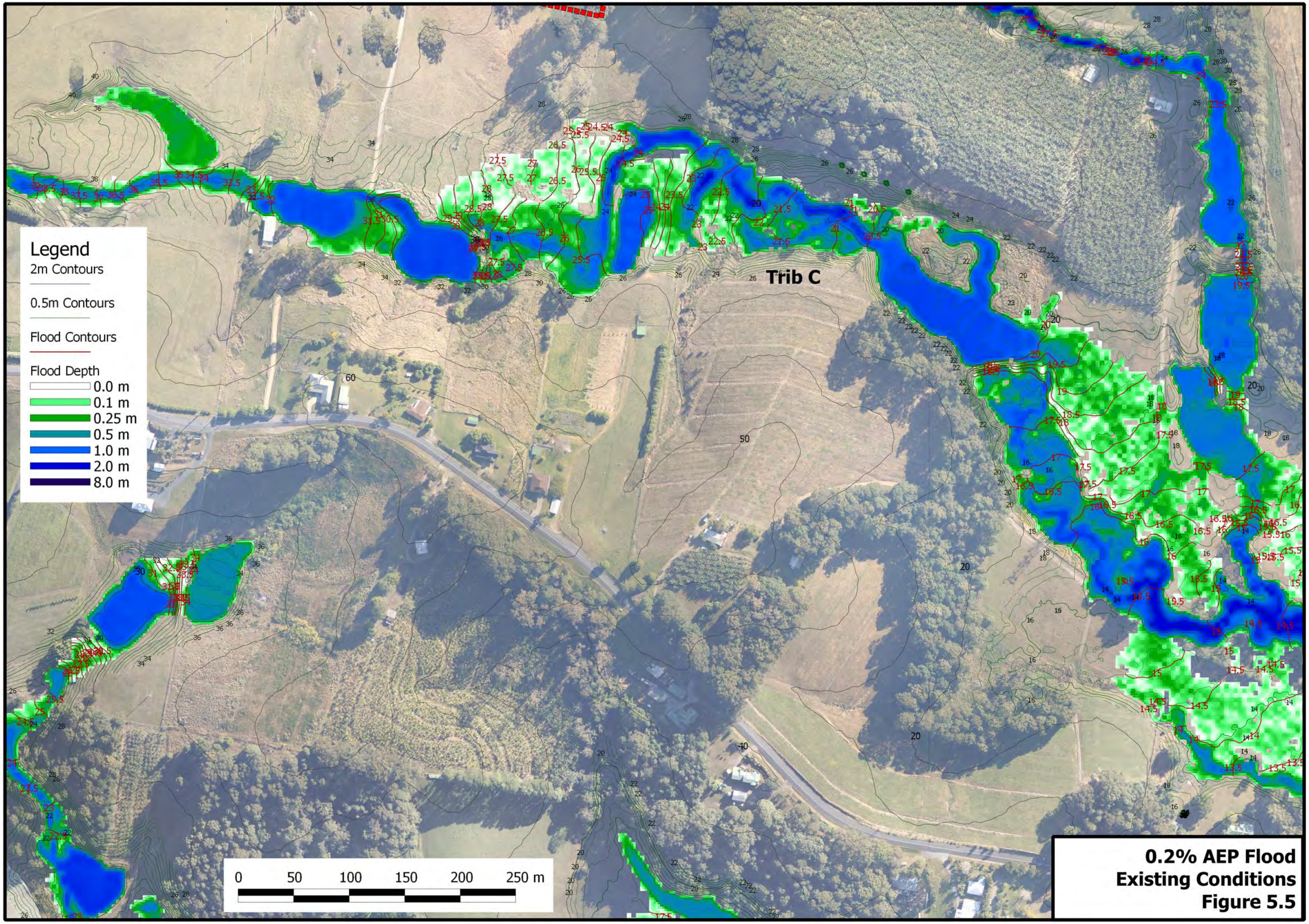
5% AEP Flood
Existing Conditions
Figure 5.2



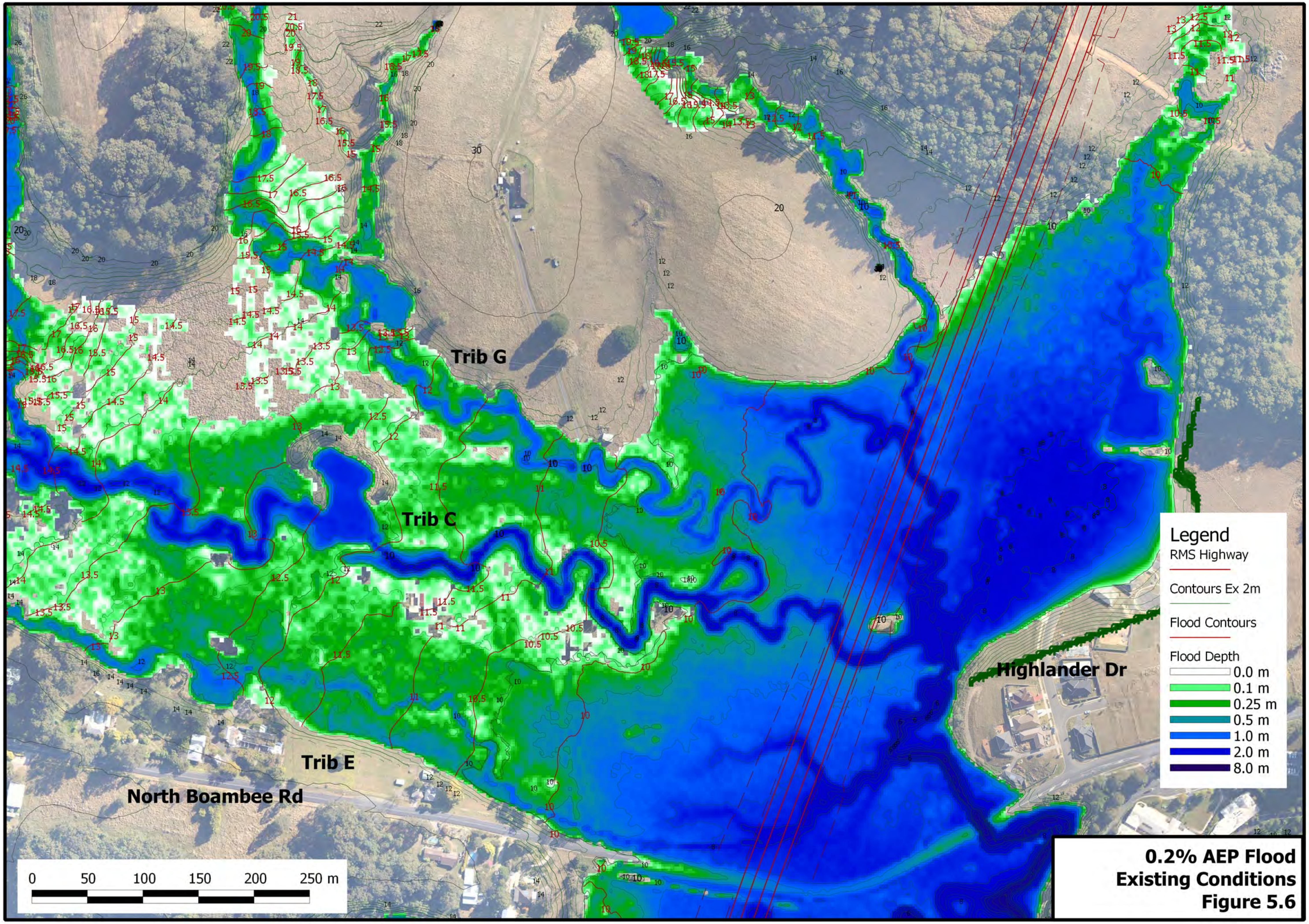
1% AEP Flood
Existing Conditions
Figure 5.3



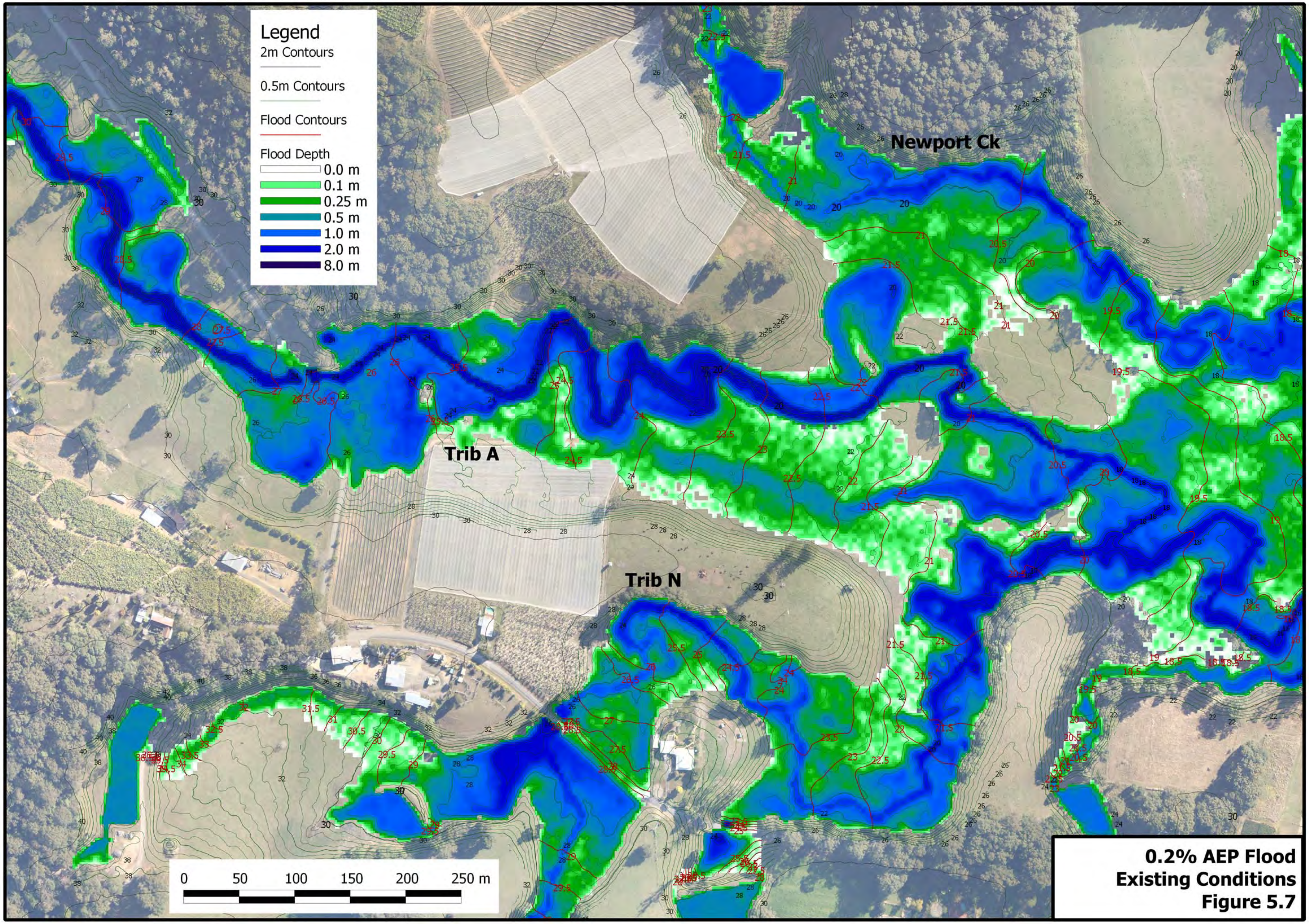
0.2% AEP Flood Existing Conditions Figure 5.4



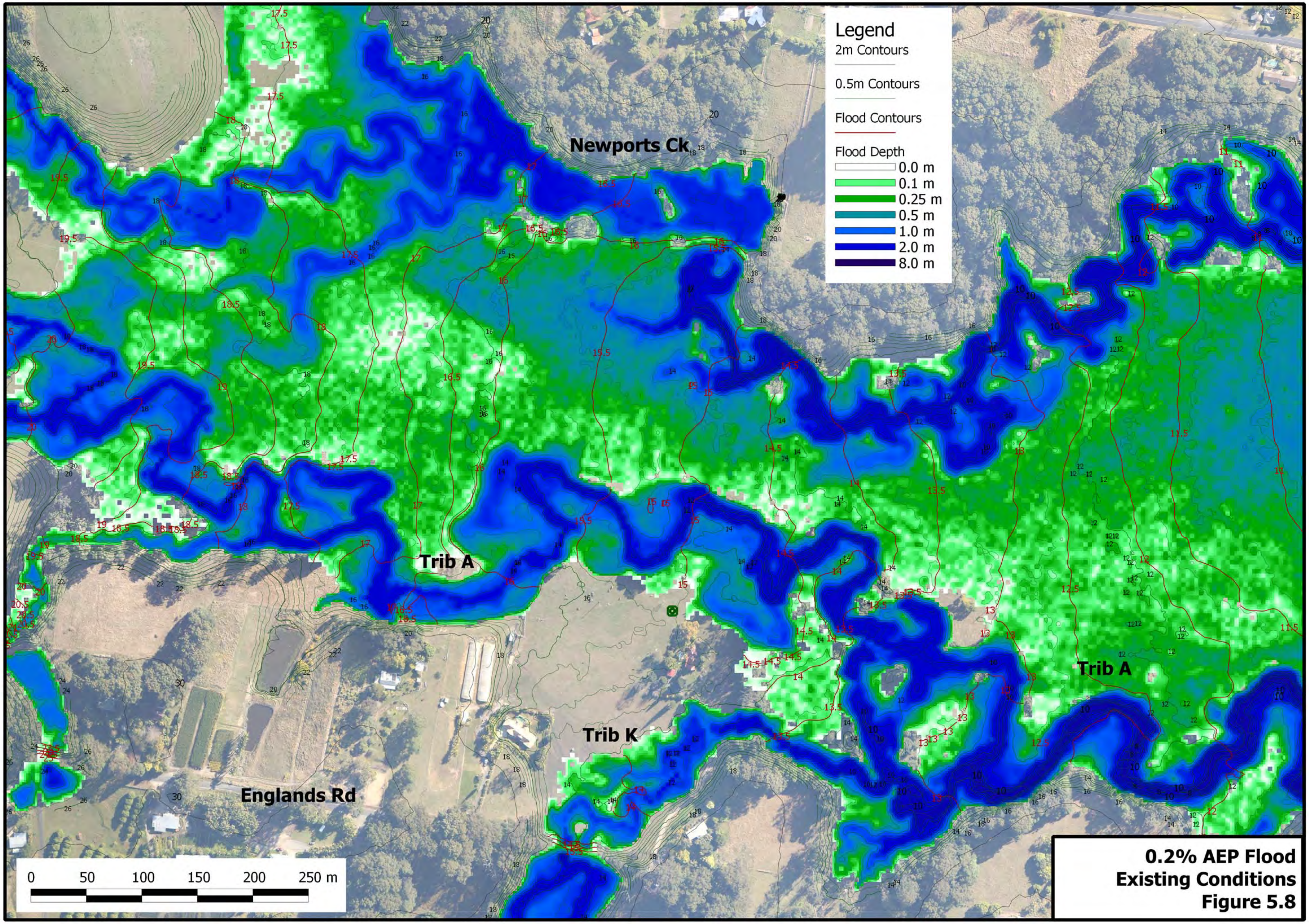
0.2% AEP Flood
Existing Conditions
Figure 5.5



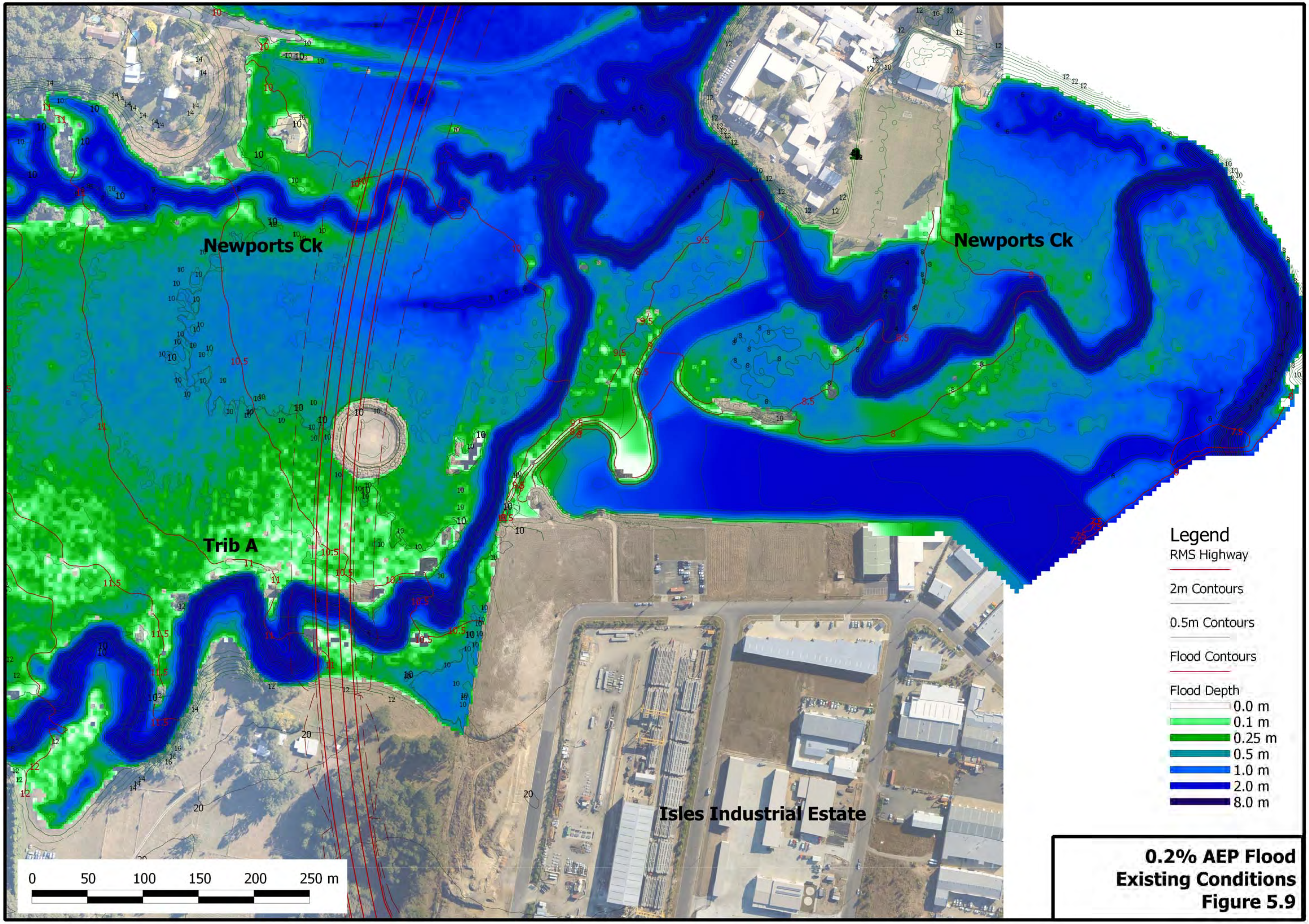
0.2% AEP Flood
Existing Conditions
Figure 5.6



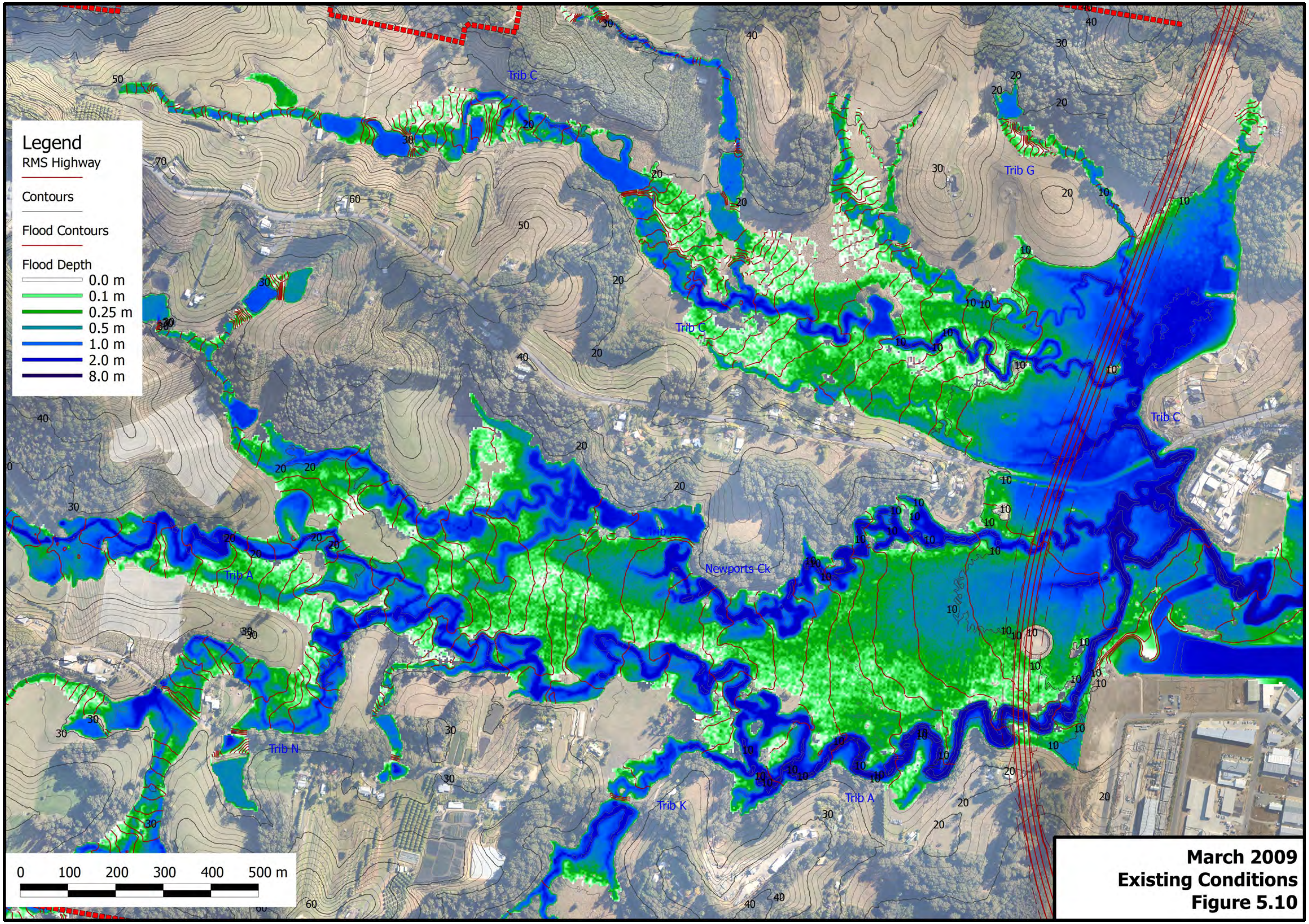
0.2% AEP Flood
Existing Conditions
Figure 5.7



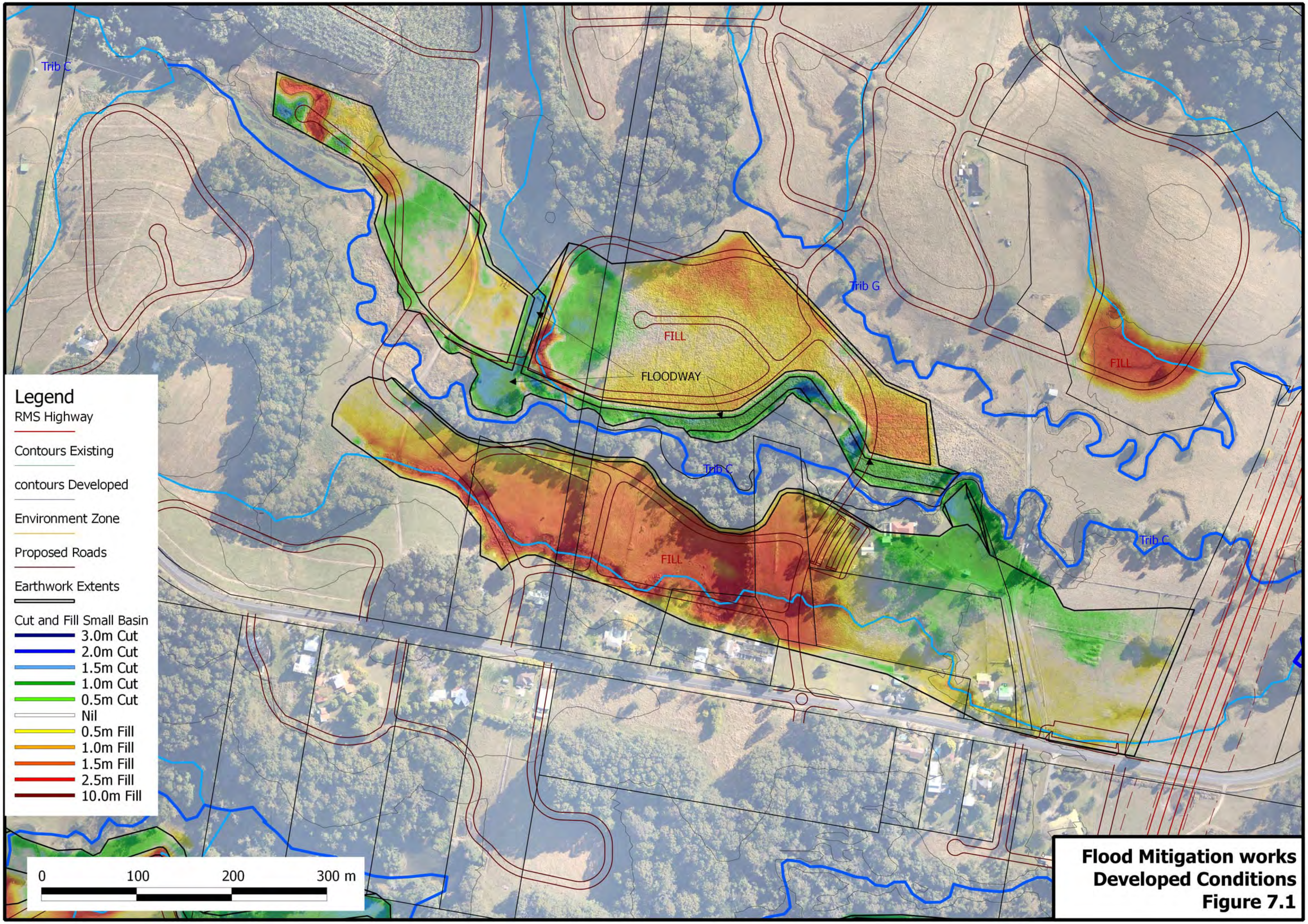
0.2% AEP Flood
Existing Conditions
Figure 5.8



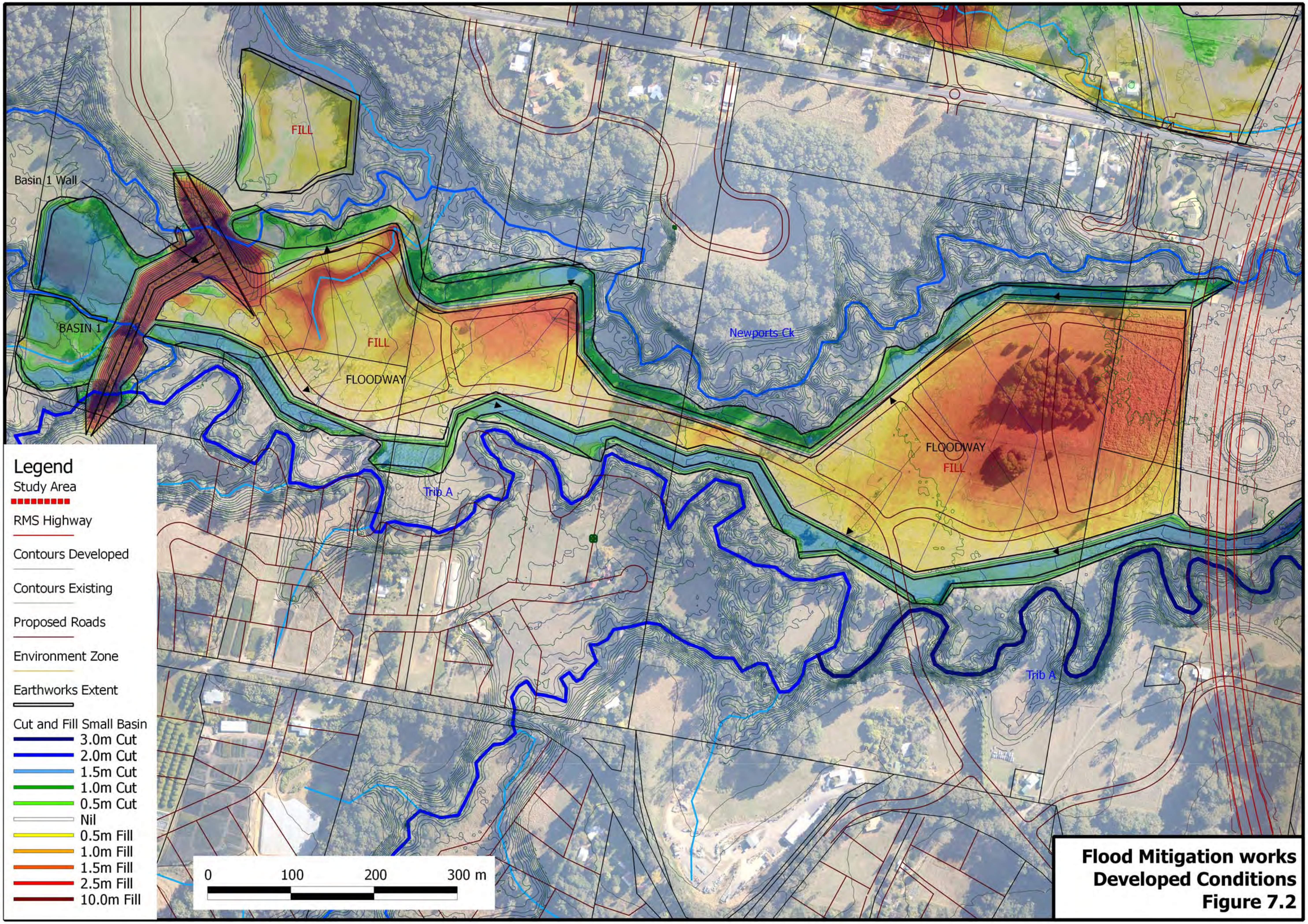
0.2% AEP Flood
Existing Conditions
Figure 5.9



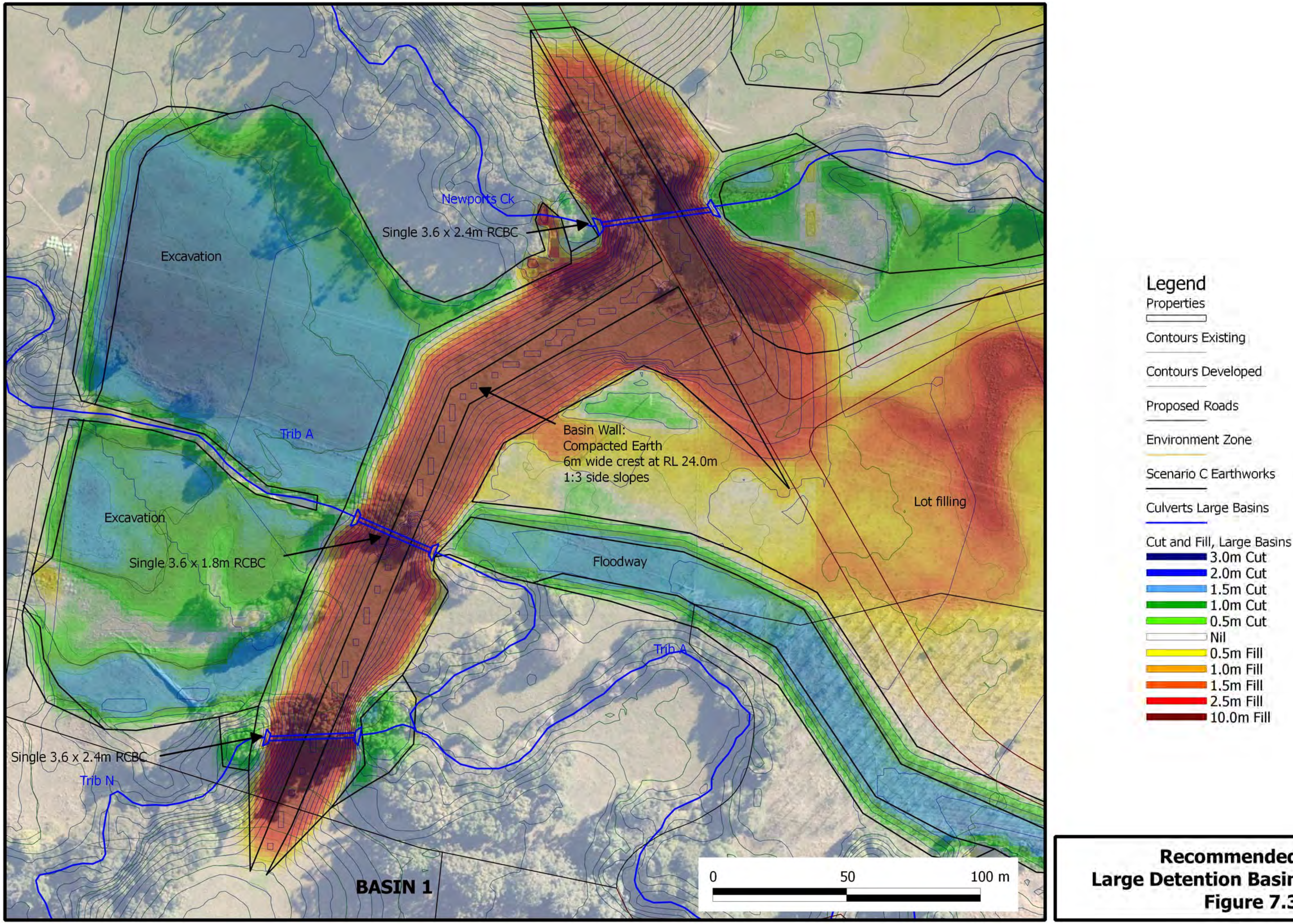
March 2009
Existing Conditions
Figure 5.10



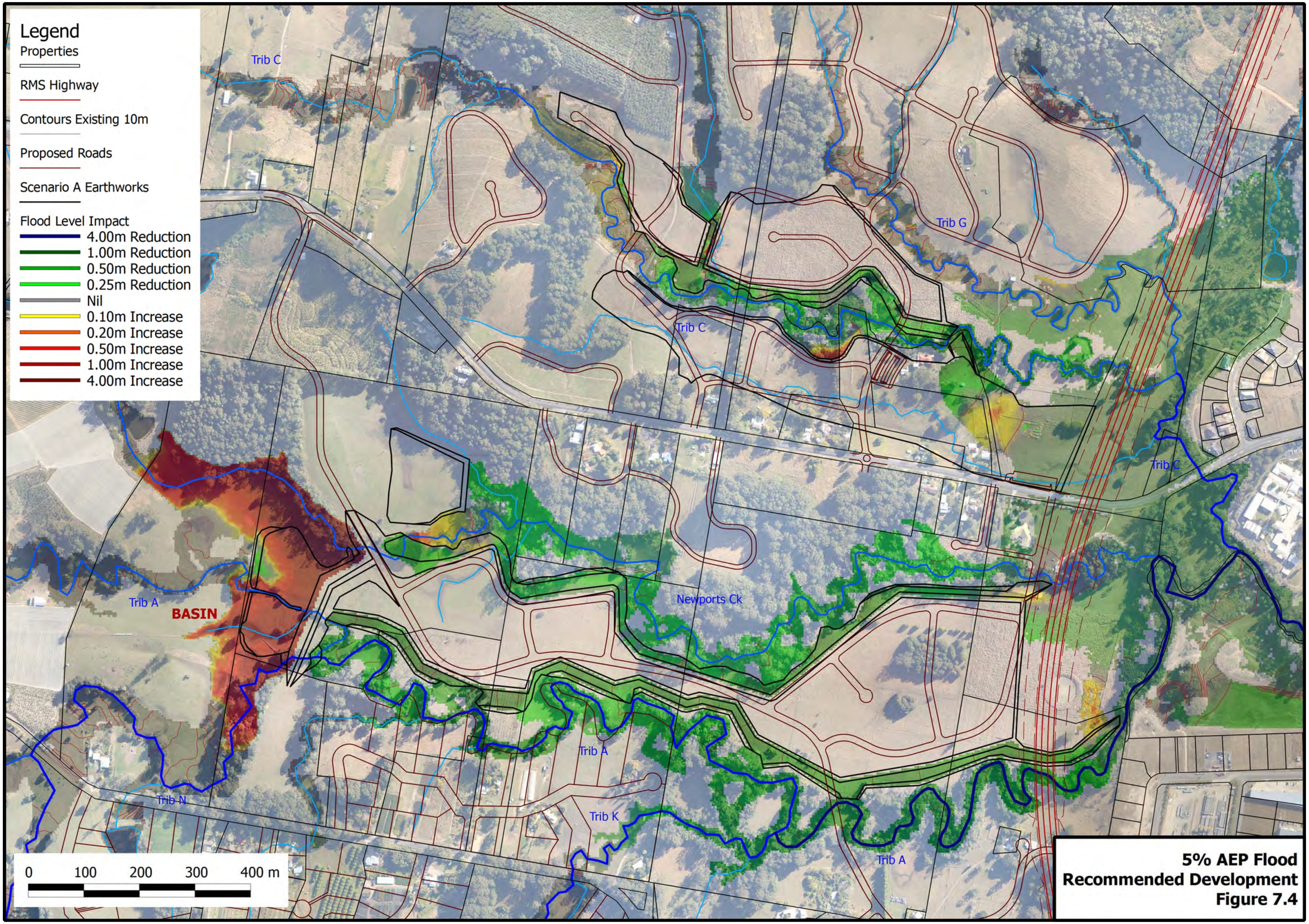
Flood Mitigation works
Developed Conditions
Figure 7.1



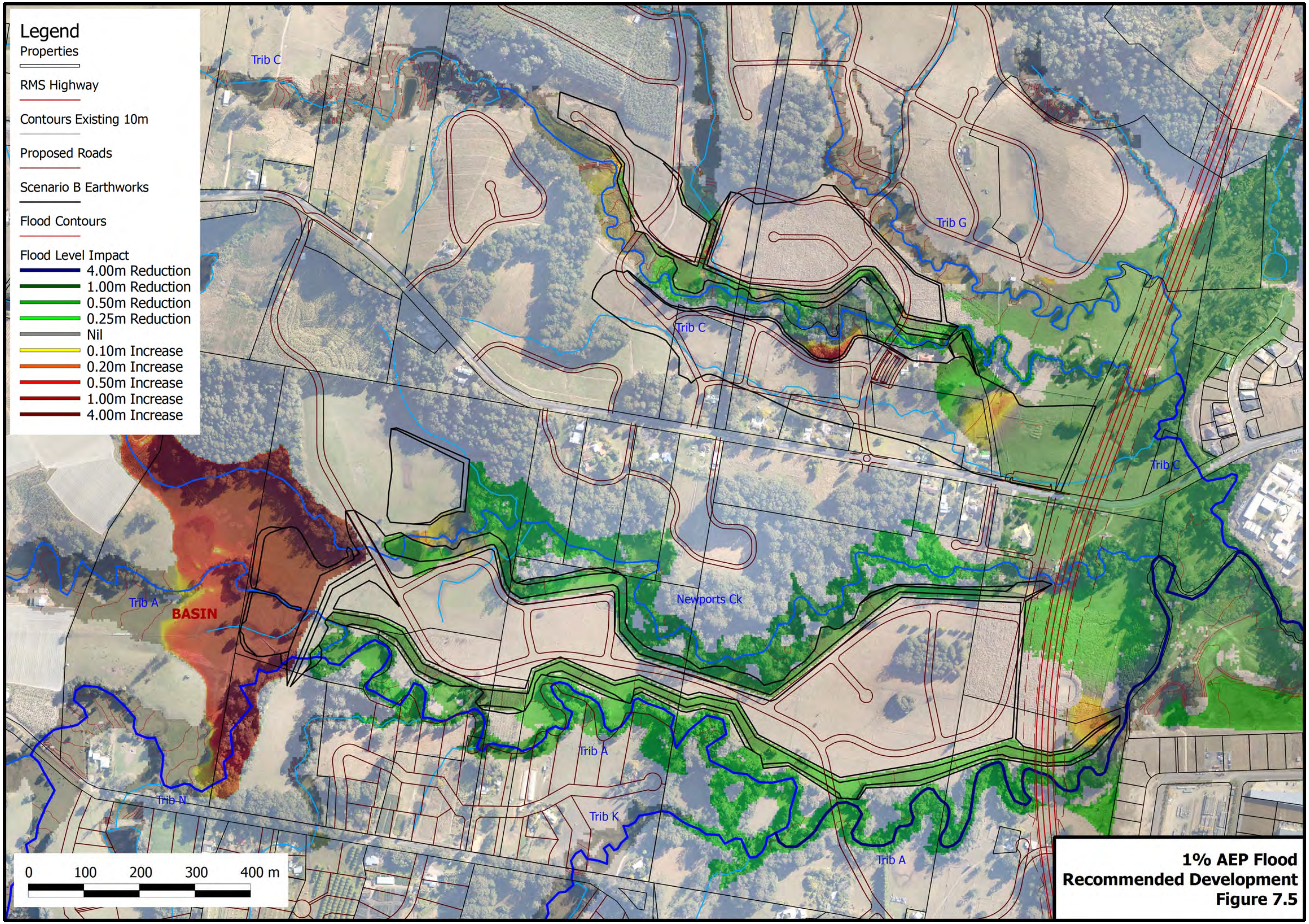
Flood Mitigation works Developed Conditions Figure 7.2



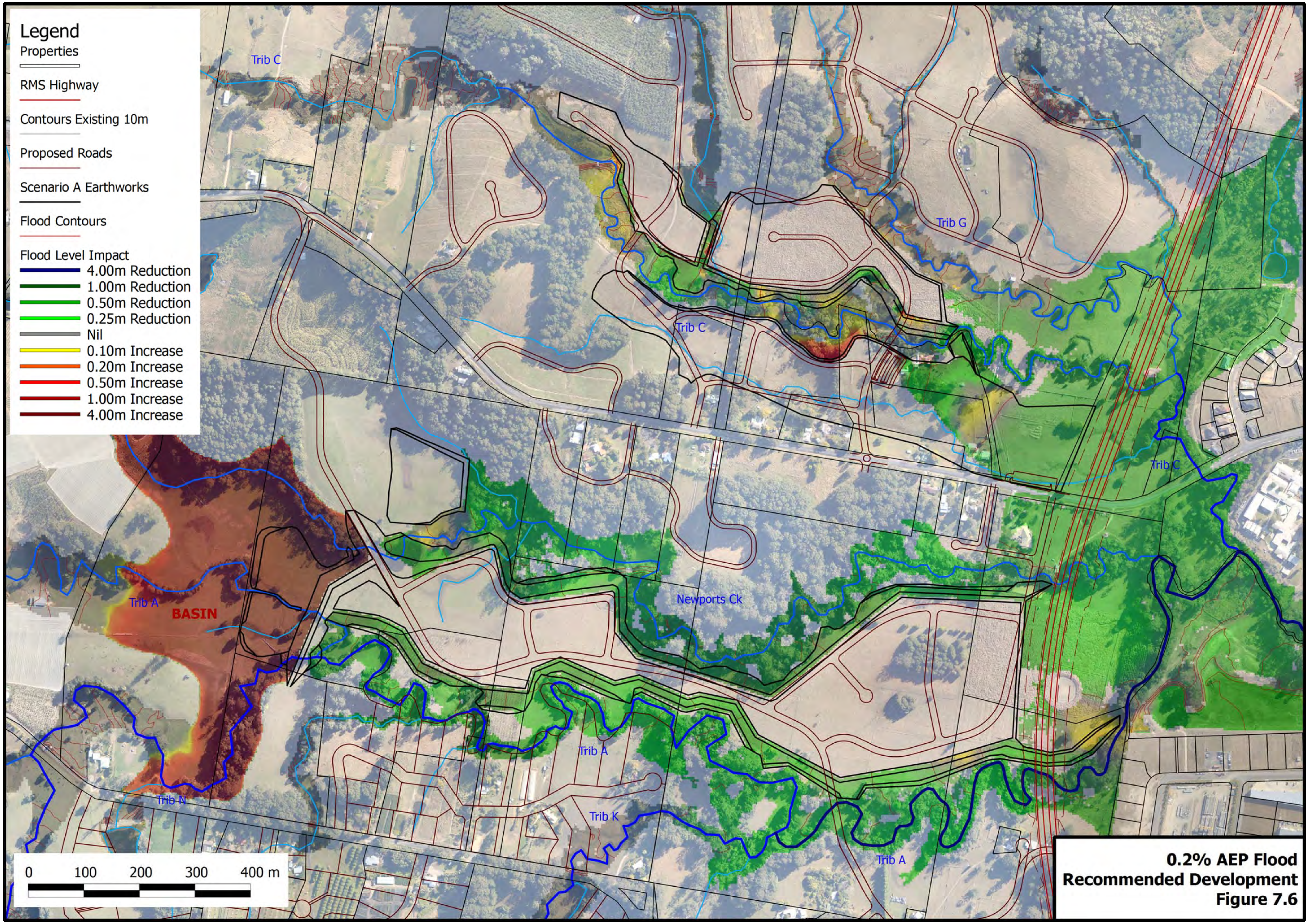
Recommended Large Detention Basin Figure 7.3



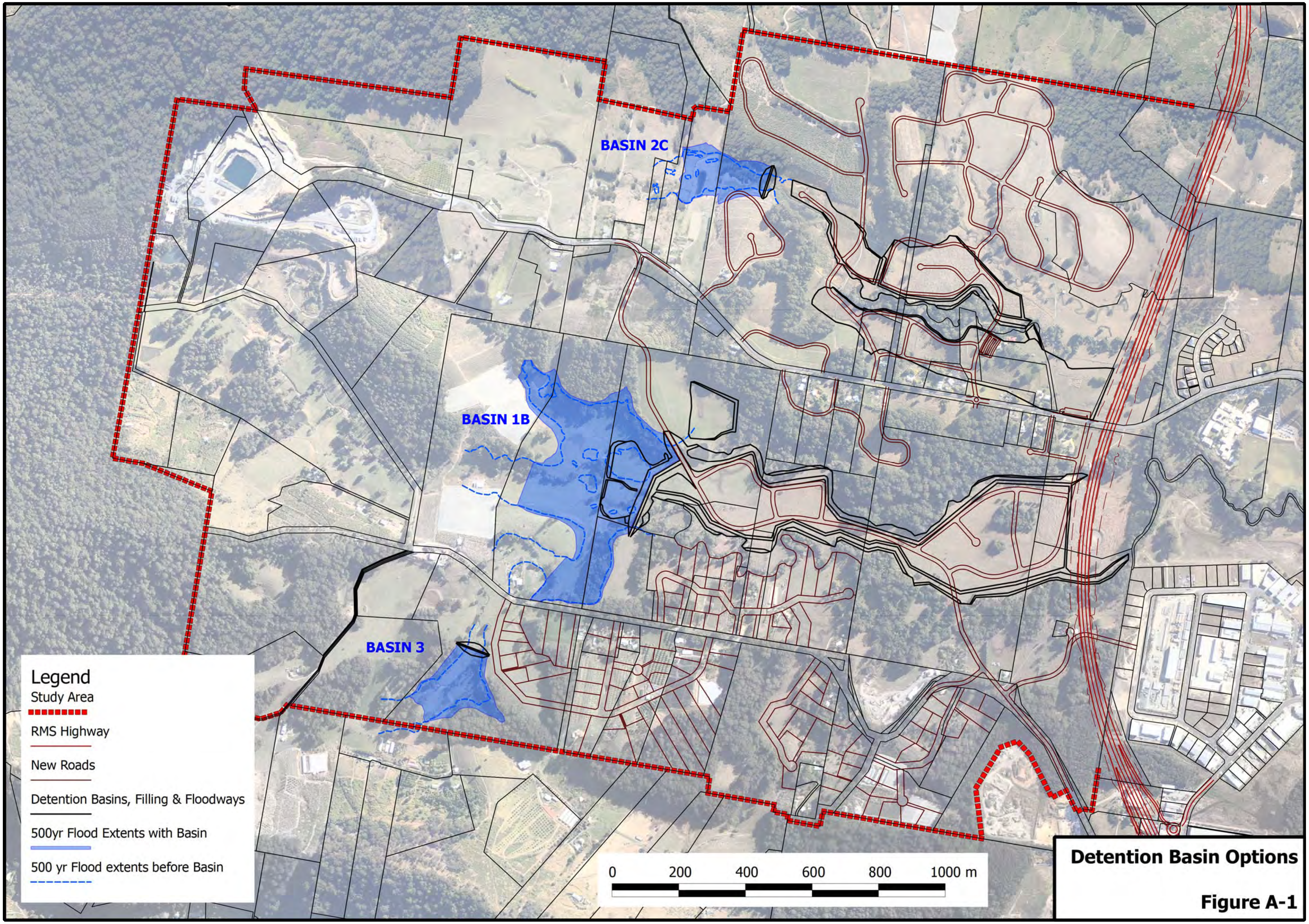
**5% AEP Flood
Recommended Development
Figure 7.4**



1% AEP Flood
Recommended Development
Figure 7.5



0.2% AEP Flood Recommended Development Figure 7.6



Detention Basin Options

Figure A-1