Flood Impact Assessment

Hydrology Report

Reeves Creek

Prepared for
Dartanyon Pty Ltd

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1 Introduction

1.1 Background

Cardno has been commissioned to prepare this Flooding Report on behalf of Dartanyon Pty Ltd to support the re-zoning application for the land at 1735 Remembrance Drive, 108-114 and 116-118 Menangle Street, Picton. The application is to rezone the land use from rural landscape to low density residential area, medium density residential area and public recreation area.

The study has been undertaken to determine flood behaviour for the 1%, 2%, 5% and 20% Annual Exceedance Probability (AEP) design floods and the Probable Maximum Flood (PMF). The primary flood characteristics reported for the design events considered include depths, levels and velocities. The study has also defined the Provisional Flood Hazard and Hydraulic Categories within the floodplain.

The primary objective of the study is to define the existing flood behaviour and to demonstrate that the proposed development does not adversely increase the flood risk to any other properties within the floodplain.

1.2 The Site

The proposed land zoning site is located within Wollondilly Shire Council area and is approximately 700m to the north east of Picton Train Station. The site is bounded by an existing residential area to the north, Menangle Street to the east and extends approximately 550m along Reeves Creek to the southeast. The proposed rezoning has a total area of 37.5ha, with around 33ha draining to Reeves Creek and then to the culvert at Menangle Street. The other 4.5ha drains to the drainage easement at Emmett Close to the north of the site. The area being submitted for rezoning (bounded by the red dotted line) is shown in Figure 1-1 below.

The existing site is currently used as pastures; with slopes varying between 10% and 25%. Reeves Creek flows from the southeast to northwest towards Menangle Street. There are four other identified watercourses, which contribute flows to Reeves Creek and then into Stonequarry Creek. The total catchment area draining to the existing culvert at Menangle Street is 145ha with the other 10ha draining to the north separated by the ridge. Figure 1-1 shows the location and the existing landscape of the site.

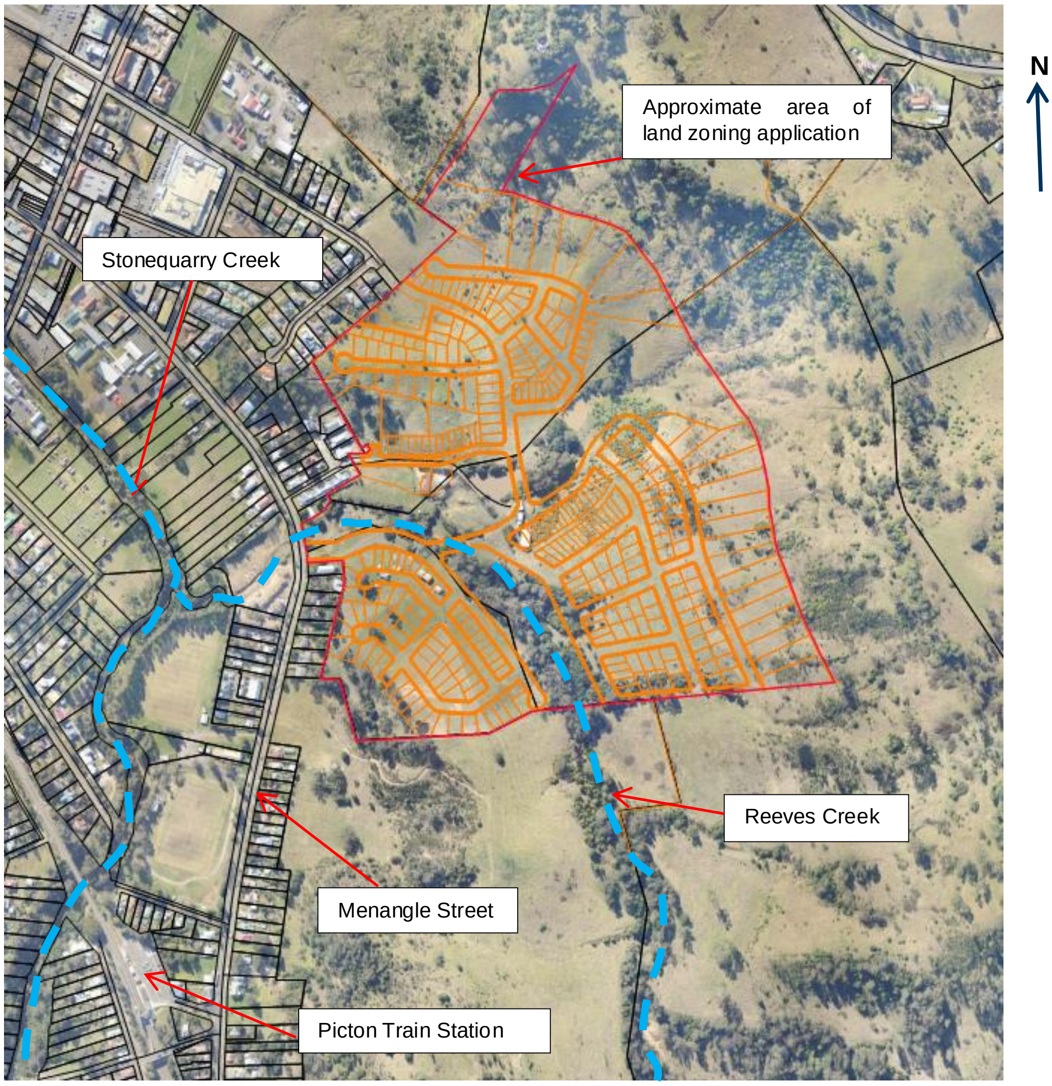


Figure 1-1 Study Area

2 Available Data

The relevant data sources to this current project are discussed below.

2.1 Previous Studies and Reports

The following study has been previously undertaken in this area:

- Cardno (2014a). Stonequarry Commercial Development Hydrology and Hydraulic Assessment
- Cardno (2014b). Stormwater Management Report Reeves Creek
- Worley Parsons (2011). Stonequarry Creek – 2D Flood Modelling and Climate Change (301015-02259 – RP002)
- Worley Parsons (2009). Picton Lands Flood Impact Assessment (301015-01001-00 – 21.12.09).

2.2 GIS Data

LiDAR survey data for the floodplain was made available by Dartanyon Pty Ltd for modelling. Additional Geographic Information System (GIS) data were sourced for the study:

- Airborne Laser Scanning (ALS) data
- Aerial photography – Near Maps

3 Hydrology Model

An XP-RAFTS hydrological model was established by Cardno for the Reeves Creek Development to determine the pre and post-developed site runoff (Cardno 2014b). The hydrological model allows for the definition of runoff behaviour from the catchment, and is a key input to the hydraulic model. Runoff has been estimated for 1%, 2%, 5% and 20% AEP storm events and the PMF for both existing and development scenarios.

The catchment was divided into sub-catchments based on the topographic features, the likely flow paths, as well as the input requirements of the hydraulic model. The sub-catchment layout is presented in **Figure 3-1.** A number of parameters are required in the development of the RAFTS model. The important parameters include initial and continuing rainfall loss rate and Manning's Roughness (Cardno 2014a).

The resulting flows were used as the permissible site discharge (PSD) requirements for stormwater detention basin sizing to ensure pre-development flows are maintained in the post-developed condition (Cardno 2014b).

A detailed Hydrology Report (Cardno 2014b) has been attached along with this document.

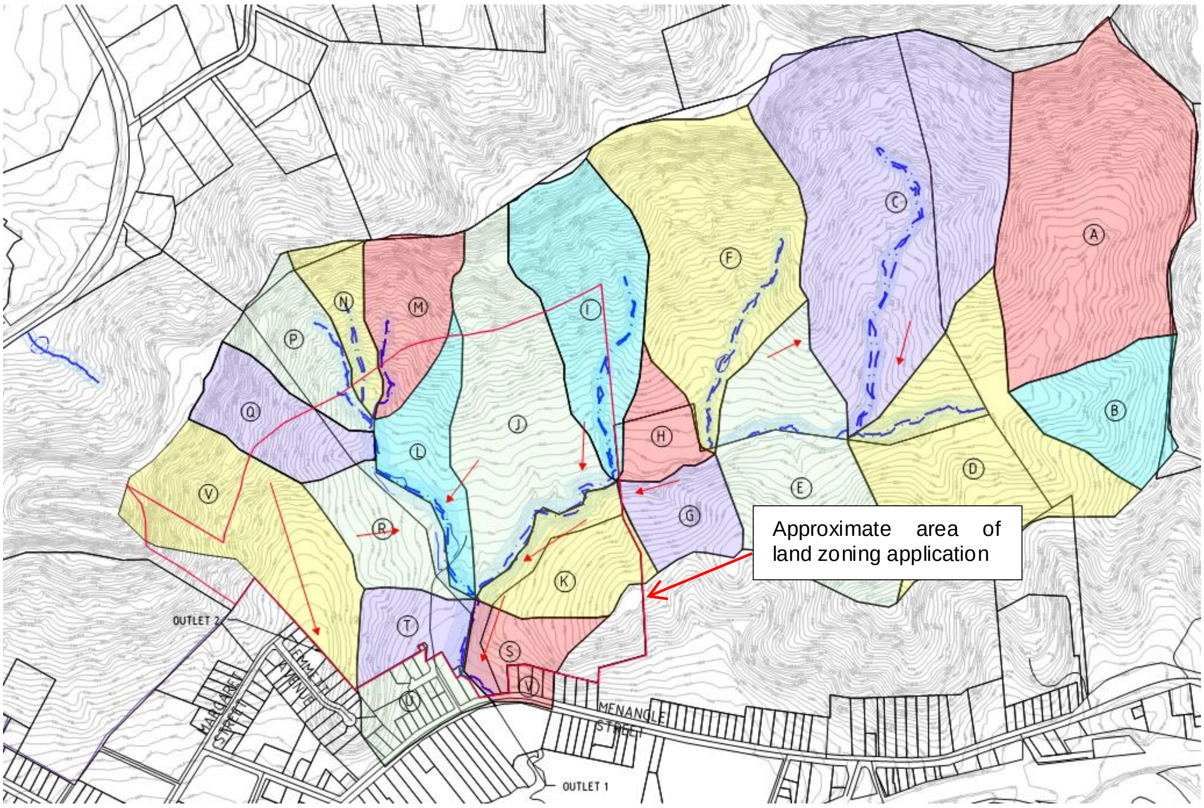


Figure 3-1 Existing Sub-Catchments

4 Hydraulic Model

A hydraulic model of the study area was developed in SOBEK, a linked 1D/ 2D modelling system to estimate flood behaviour. The 1D component was utilised to define the culverts in the study area, while the 2D component was utilised to define the creek and overbank flows.

Cardno has previously developed a hydraulic model for Stonequarry Creek (Cardno 2014a) and we proposed to utilise this model as a base case and further refine the model with additional LiDAR survey information and to extend the model past the Reeves Creek development.

The model includes the tributaries from the upstream area feeding into Reeves Creek and passes through the Menangle Street culvert and discharges into Stonequarry Creek as shown in the Figure 4-1. The model was run using derived inflows from a hydrological RAFTS model developed (Cardno 2014b) as line boundary and lateral inflow inputs.

Two flood modelling scenarios were adopted:

- Pre-development – This is representative of the existing site conditions and the building layouts. Areas within the site were updated from Cardno (2014) to include additional LiDAR survey data.
- Post-development - The pre-development model was subsequently updated to represent the proposed development. This included the design terrain information as well as the stormwater layout. The buildings layout was also adjusted to represent the proposed buildings, as per the concept design information.

4.1 Data

In order to develop the SOBEK model and to verify the model behaviour the following data was utilised.

- Hydrograph inflows at the model boundary for the 20%, 5%, 2% and 1% Annual Exceedence Probability (AEP) and the PMF events.
- Downstream boundary conditions in the form of a fixed Tail Water Levels and Height – Discharge table.
- Roughness used to define the floodplain.
- Topography information.
- Details of the implementation of structures within the hydraulic model.
- Hydrologic model (RAFTS).

4.2 2D Terrain

SOBEK elevation and roughness grids were developed using a resolution of 4m. The terrain data was defined utilising LiDAR survey data which has a nominal vertical accuracy of +/- 0.15m on hard surfaces. The 2D model extends from downstream of the Menangle Street culvert and extends approximately 1.5 kilometres upstream along Reeves Creek. The extent of the model and the pre-developed topography is shown in **Figure 4-1.**

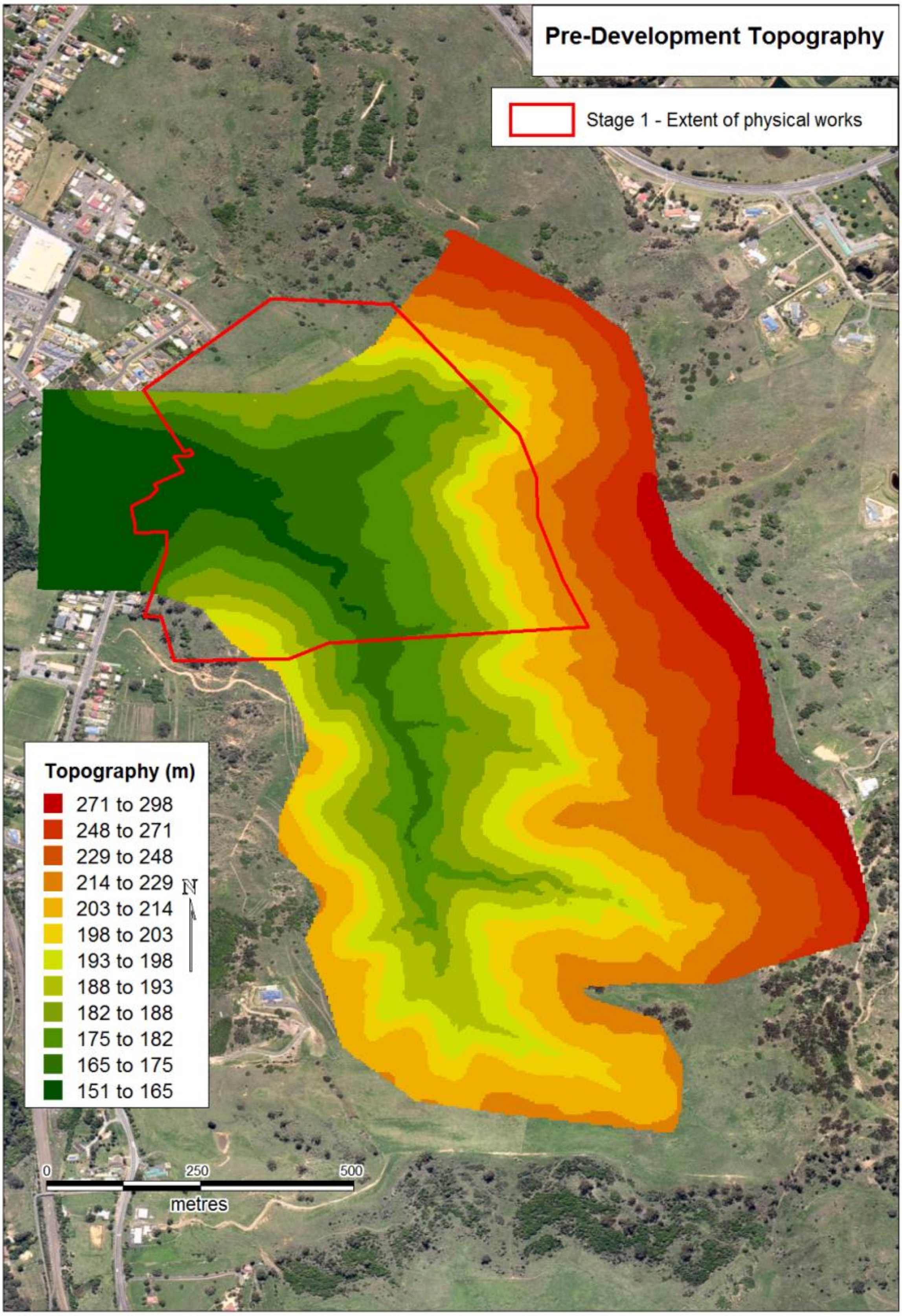


Figure 4-1 Pre-developed topography and Model Extent

The topographic grid was created using a 4m x 4m grid cell resolution which was sufficient to delineate the channel cross sections within the 2D domain as the Riparian Corridor is approximately 30 to 50m in width (7 to 12 grid cells).

The buildings were blocked out using the aerial photography. Care was taken to ensure that all flow paths between buildings were maintained to ensure the floodplain connectivity was not altered. This was completed using detailed up to date aerial photography (Nearmap), Google Street View and field photos.

The post-developed topography is shown in **Figure 4-2.**

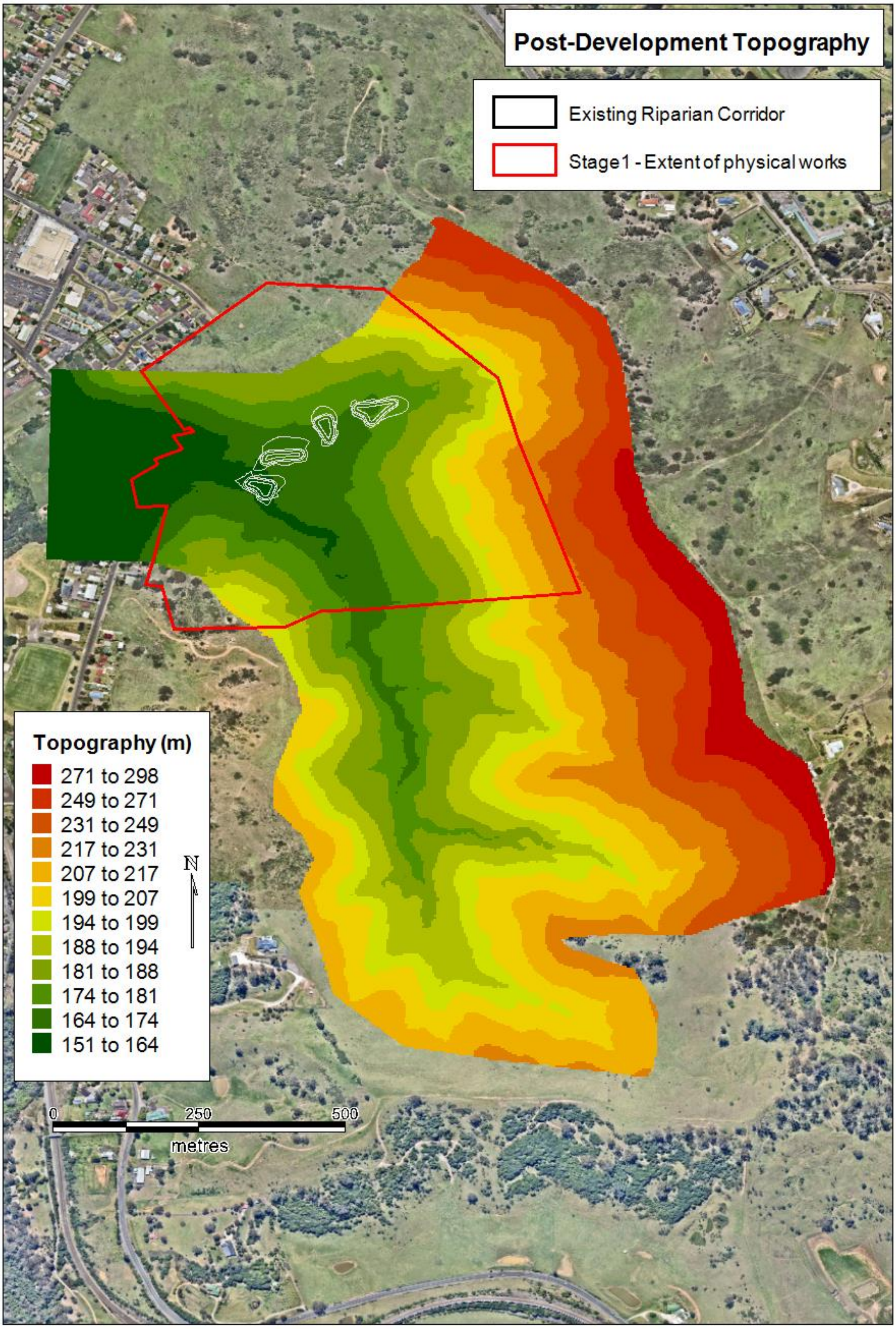


Figure 4-2 Post-developed topography and Model Extent

4.3 1D Model

The 1D components of the model were based on the available survey and observations obtained from Wollondilly Shire Council. Council provided the dimensions for the existing culverts on Menangle Street and the inverts levels were estimated based on information from the LiDAR survey data. The model layout for the proposed development is shown in **Figure 4-2.** The details of proposed pipes and the basins in the post-development scenario can be referred to the Hydrology Report (Cardno 2014b).

4.4 Model Roughness

The hydraulic roughness for the 1D cross sections and 2D model grid was determined using aerial photography and the Stonequarry Creek Commercial Development (Cardno 2014). The roughness maps for both pre and post-developments are shown in **Figures 4-3** and **Figures 4-4**.

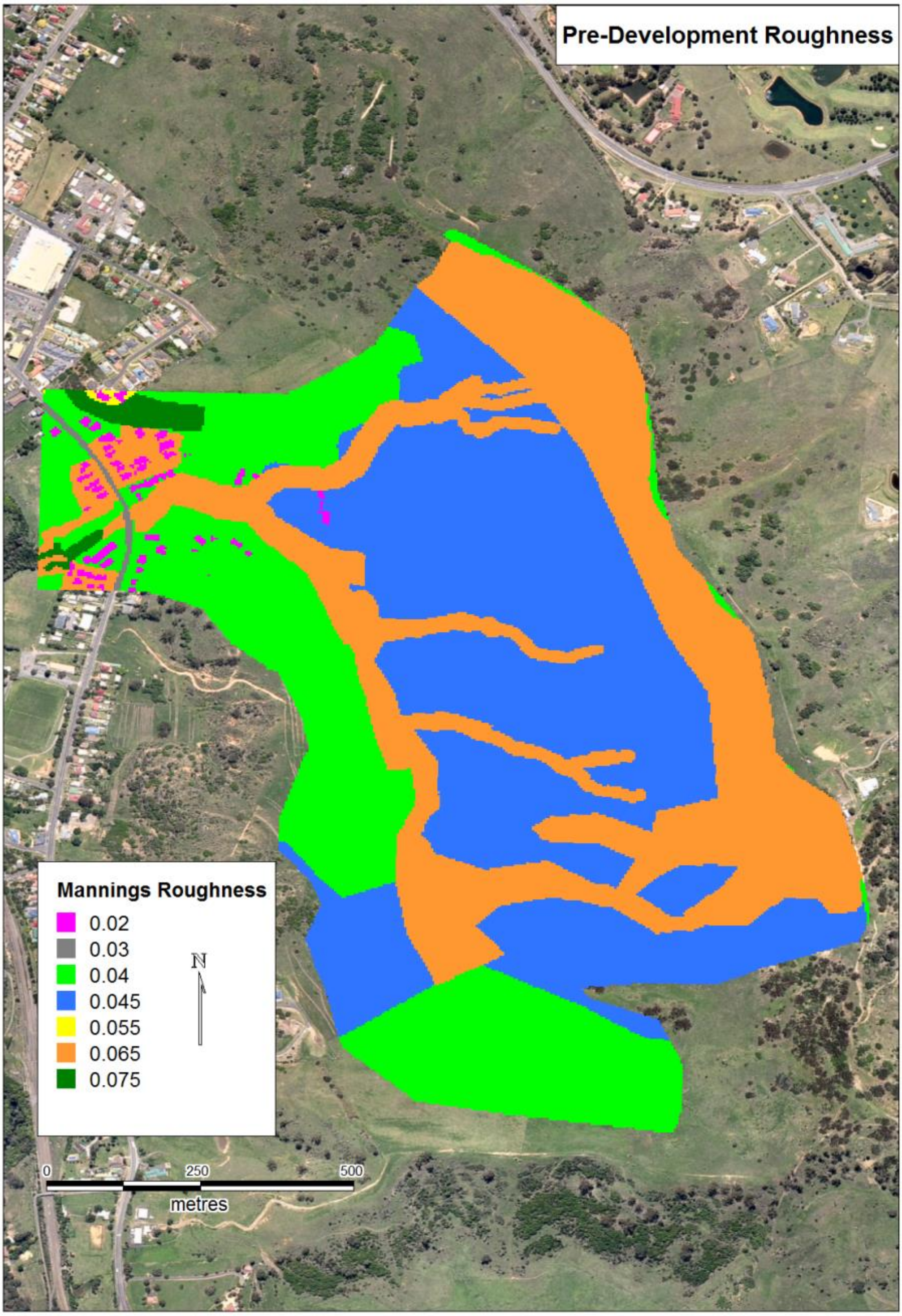


Figure 4-3 Pre-developed Roughness

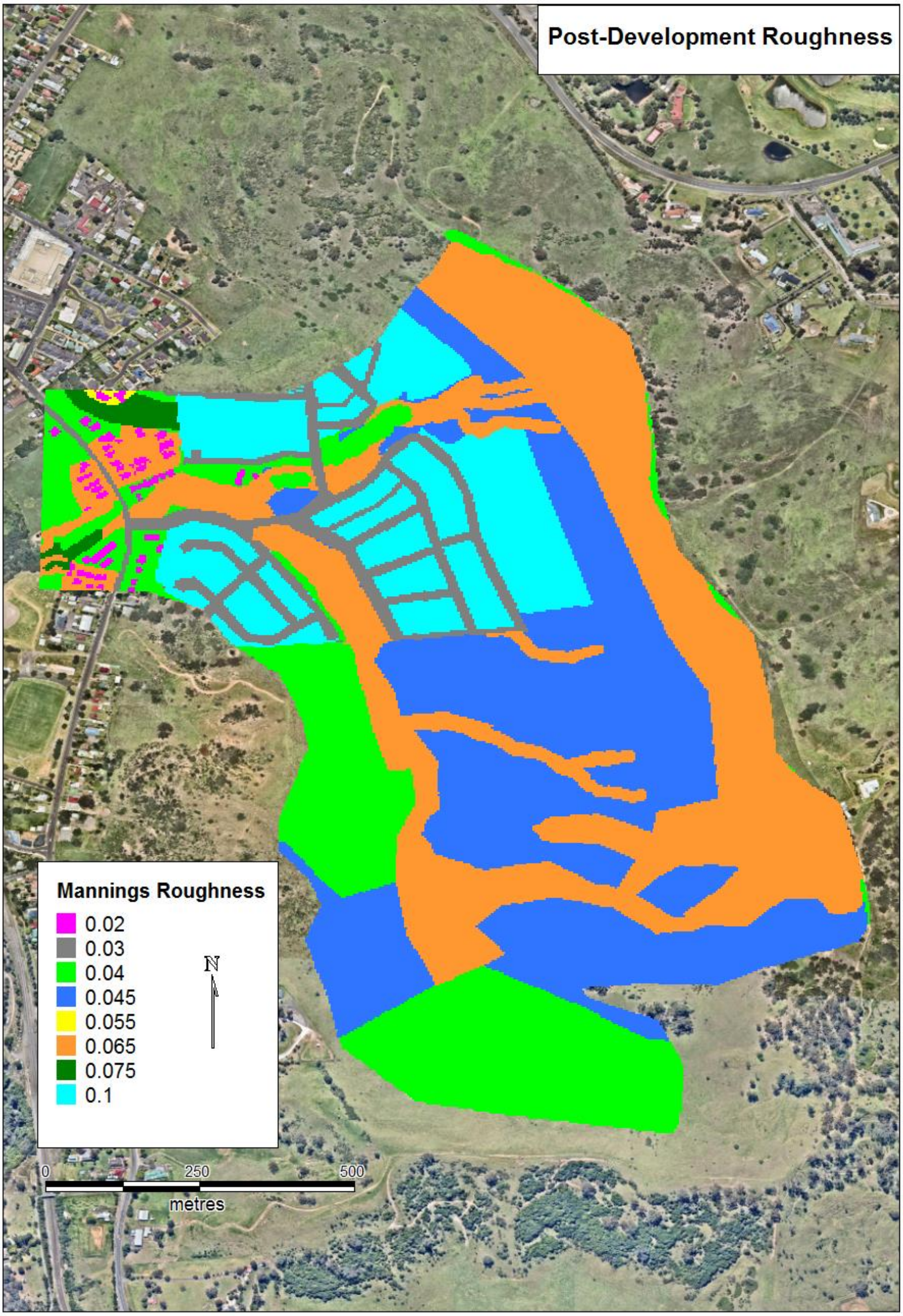


Figure 4-4 Post-developed Roughness

The roughness parameters adopted are shown in **Table 4.1**

Table 4-1 Manning's Roughness Values

Description	Roughness (Manning's 'n')
Industry	0.065
Roads	0.03
Building	0.02
Urban	0.055
Riparian Corridor	0.065
Open Space	0.045
Floodplain with Dense Trees	0.075
Bush	0.065
Grass	0.04
Basins	0.04
Residential	0.10

4.5 Model Boundaries

The upstream boundary conditions for the SOBEK model were derived from the XP-RAFTS model (Cardno 2014b). These were applied as lateral inflows along the Riparian Corridor. The model extends to downstream of Menangle Street. A constant water level boundary (Fixed Tail Water Level, FTWL) was adopted for the model, utilising the peak water level from the Stonequarry Creek Commercial Development (Cardno 2014a). The downstream boundary levels adopted for the study are shown in **Table 4.2.**

Table 4-2 Downstream Boundary Levels

AEP	Tail Water Level (m)
PMF	164.8
1%	157.2
2%	156.9
5%	156.5
20%	156.5

The adopted levels are from the Stonequarry Creek model results (Cardno, 2014a). However, since the 20% AEP was not assessed in the Stonequarry Creek Model, the TWL of 156.5m from the 5% AEP was conservatively adopted for the 20% AEP TWL for the Reeves Creek Model.

The use of a fixed tail water level (TWL) boundary aimed at providing a conservative assessment of the likely flooding within Reeves Creek. Stonequarry Creek has a large floodplain and can be flooded for extended periods of time. If flooding occurs in Reeves Creek during this time then the downstream tail water conditions are likely to be impacted by the flooding within Stonequarry Creek. This scenario is likely to result in the largest impact within the Reeves Creek system. To demonstrate the difference between the fixed TWL and a variable flow boundary a sensitivity assessment has been undertaken. The details of this assessment are discussed in Section 6.2.

5 Flood Model Results

The pre-development and post-development scenarios were modelled for the 1%, 2%, 5% and 20% AEP events, together with the PMF event. Model runs were carried out for the 15mins, 30mins, 45mins, 60mins, 90mins, 2 hour, 3 hour, 6 hour, 12 hour and 18 hour durations. The peak water level, depth, velocity and hazard of the different durations of each AEP were then extracted to assess the flooding in the study area.

5.1 Pre-development Flood Results

The pre-development flood model results for depth, velocity, hazard and hydraulic categories (Appendix A) are provided in a series of figures. The associated figure numbers are outlined in **Table 5-1.**

Table 5-1 Pre-Development Figure Numbers

Flood Event	Peak Depth	Peak Velocity	Provisional Hazard	Hydraulic categorisation
20% AEP	Figure 5.1	Figure 5.6	Figure 5.11	Figure 5.16
5% AEP	Figure 5.2	Figure 5.7	Figure 5.12	Figure 5.17
2% AEP	Figure 5.3	Figure 5.8	Figure 5.13	Figure 5.18
1% AEP	Figure 5.4	Figure 5.9	Figure 5.14	Figure 5.19
PMF	Figure 5.5	Figure 5.10	Figure 5.15	Figure 5.20

The flooding within the study area is predominantly constrained within the Riparian Corridor in all the flood events. The modelling results show that the Menangle Street culvert downstream of the study area is running full and has a capacity less than a 20% AEP. This has the effect of causing the Reeves Creek floodplain to fill as temporary storage for flood events and overtop Menangle Street. The overtopping of the road occurs in all the flood events.

It should be noted that the pre and post development models have been run using a fixed tail water level as the Stonequarry Creek can have sustained periods of elevated flooding. This is a conservative approach to the assessment and this measure was taken as this scenario is the most likely to impact the Reeves Creek flooding. The flooding that occurs within the Stonequarry Creek floodplain is generated by a large upstream catchment (the details of this modelling are summarised in Appendix B) and is not impacted by the addition of the development on Reeves Creek (see Section 6.1 for discussion).

The pre development results show that for Reeves Creek the flows are largely constrained to the deep valley areas up to the 1% AEP event. This is expected due to the steep valley nature of the catchment. In the PMF event the floodwaters extent out of the main floodplain however, much of the flooding at the downstream end of the model is associated with the PMF event within Stonequarry Creek.

For the 20% AEP scenario the downstream tail water level has been set at the 5% AEP flood event for Stonequarry Creek as this was the smallest available event simulated within the Stonequarry Creek investigation. This causes the flooding downstream of Menangle street to appear high for the 20% AEP event but as this was outside the study area (area of interest) this was considered to be appropriate.

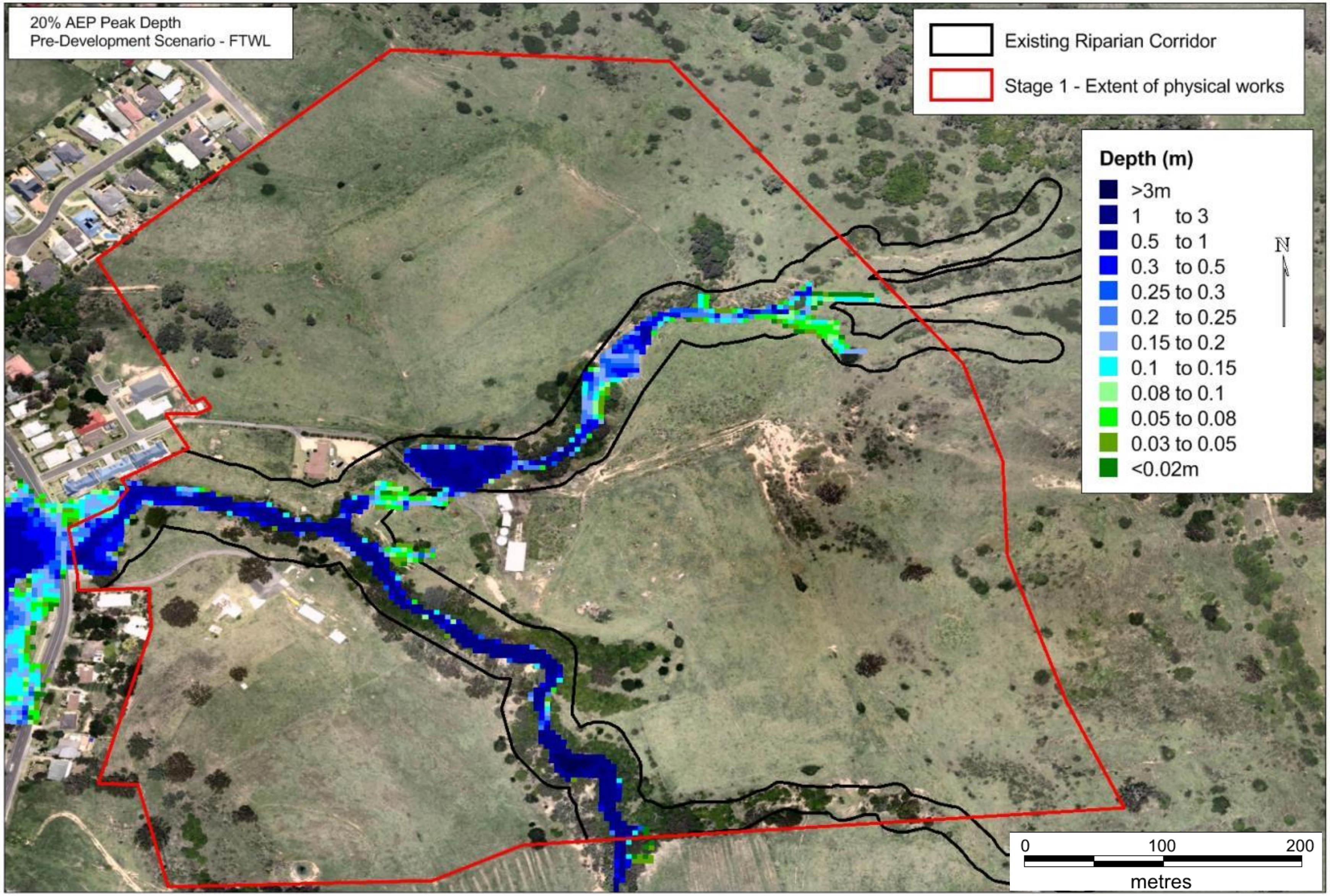


Figure 5-1 20% AEP peak flood depths (Pre-Development - FTWL)

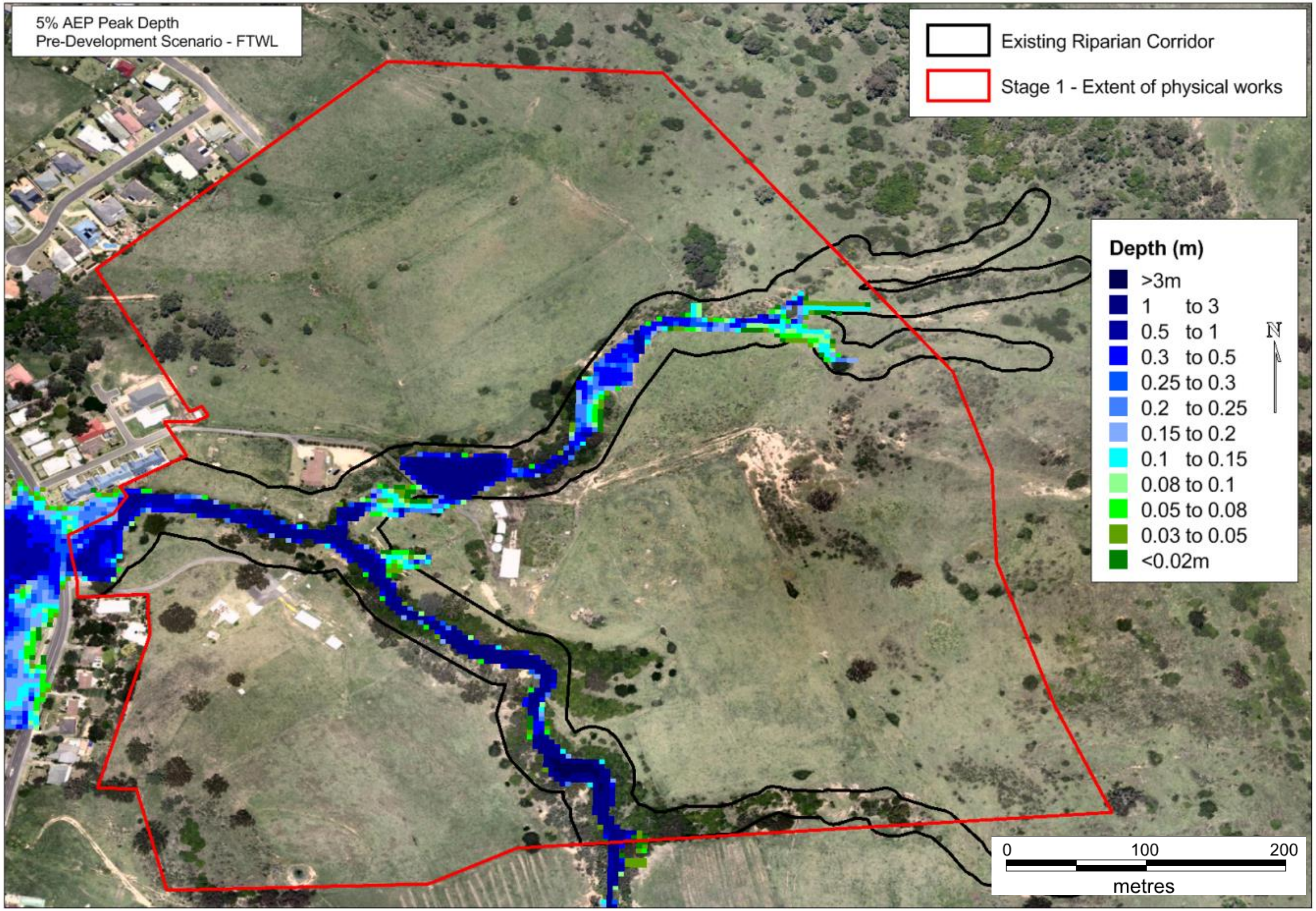


Figure 5-2 5% AEP peak flood depths (Pre-Development - FTWL)

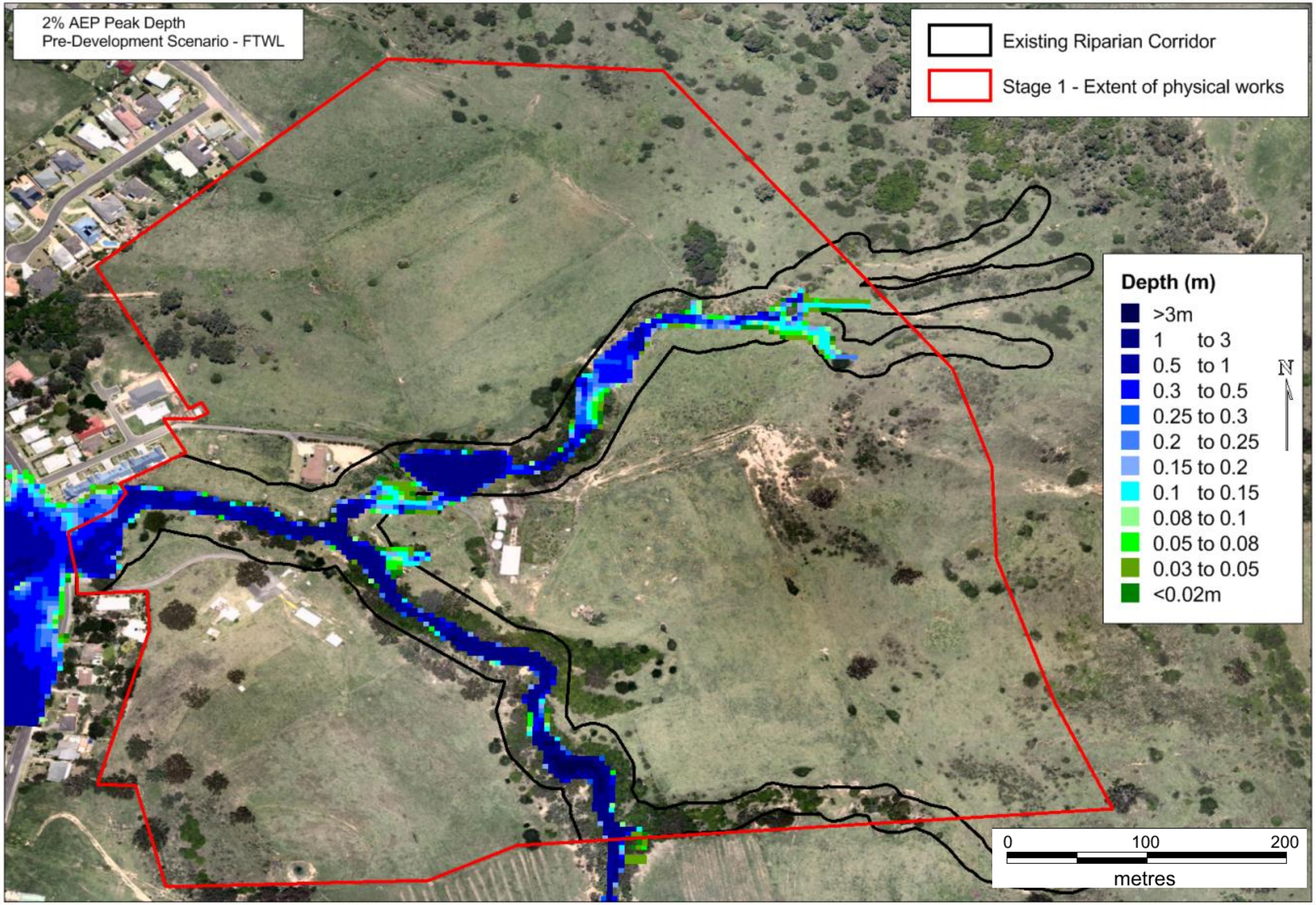


Figure 5-3 2% AEP peak flood depths (Pre-Development - FTWL)

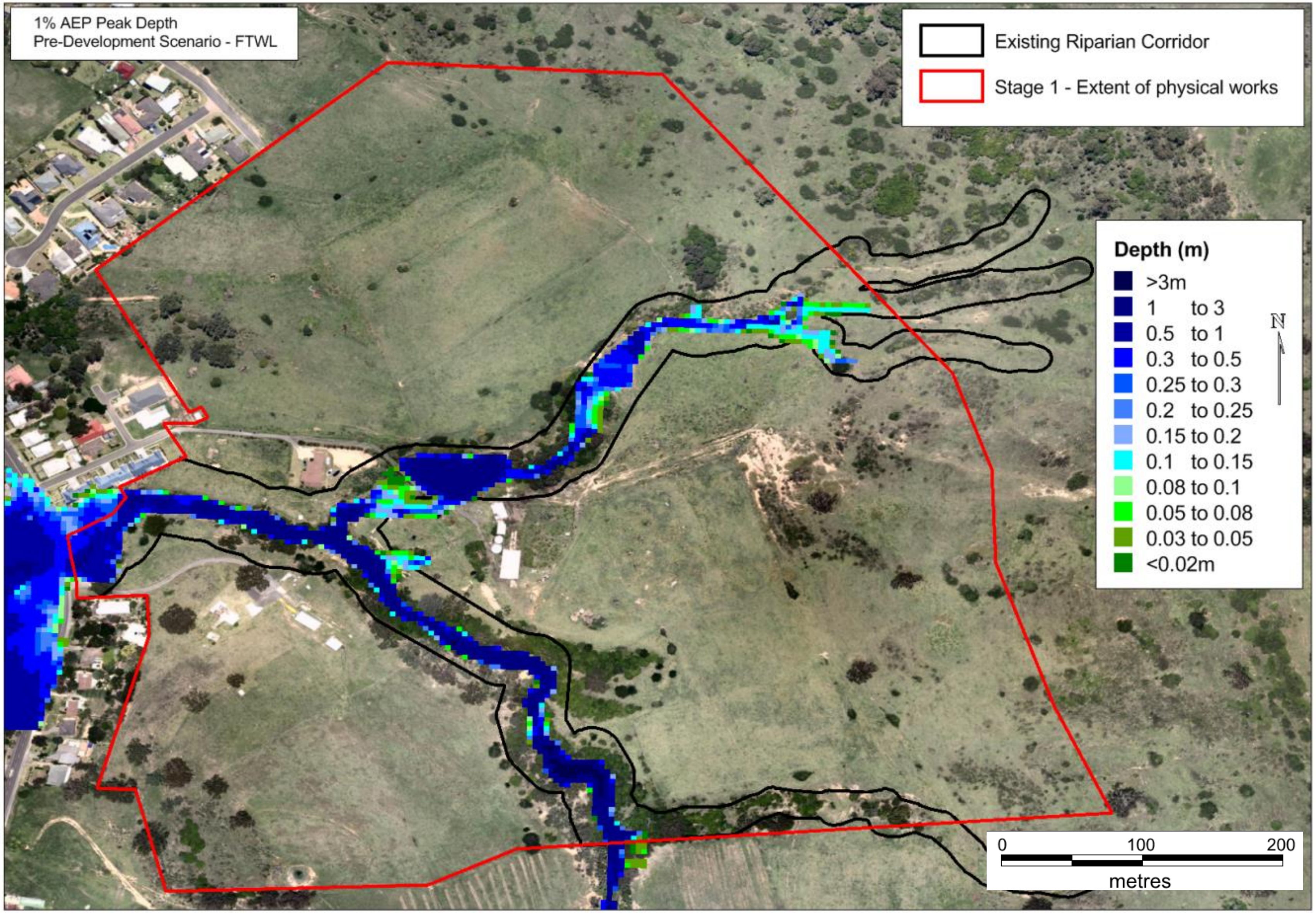


Figure 5-4 1% AEP peak flood depths (Pre-Development - FTWL)

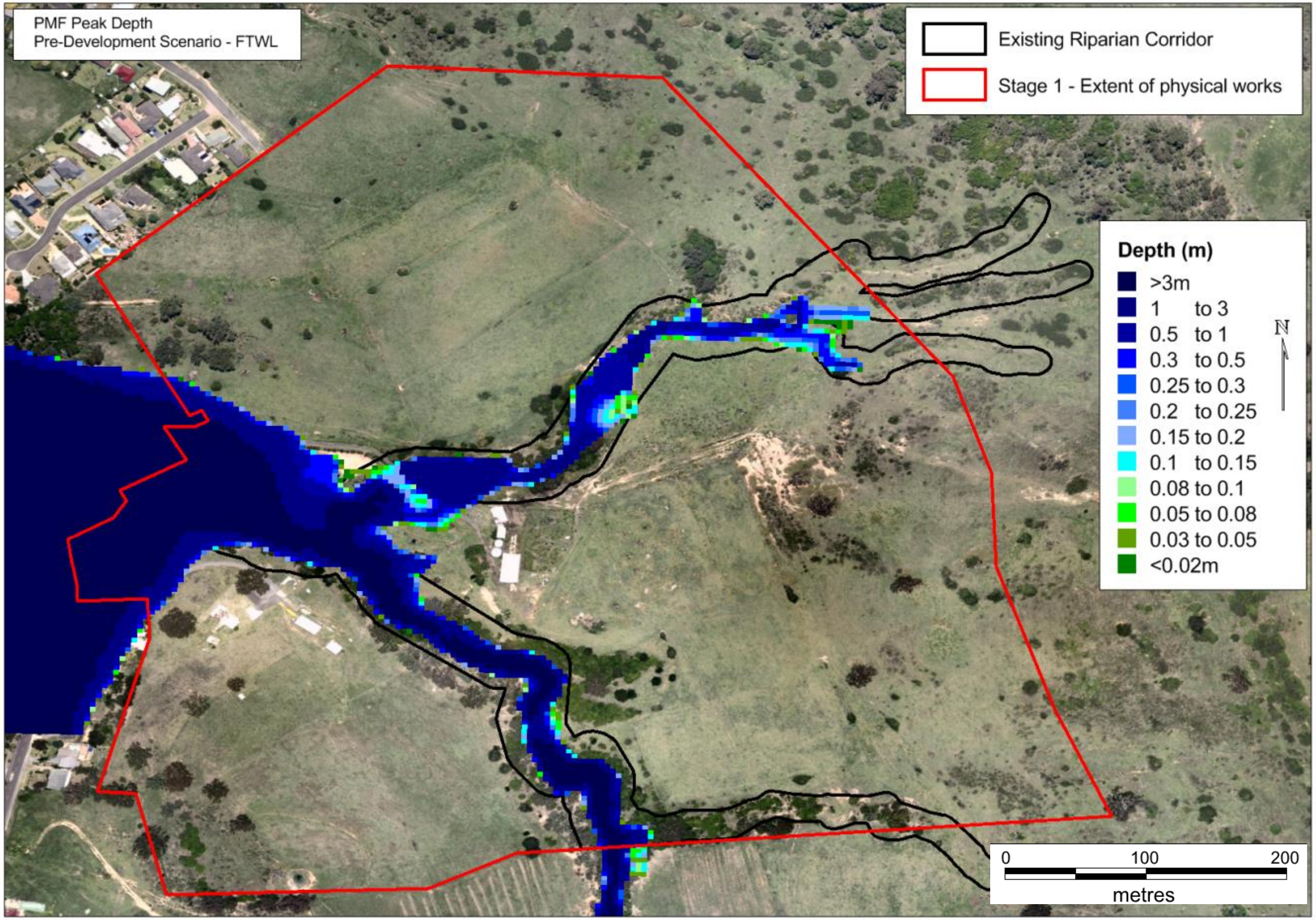


Figure 5-5 PMF peak flood depths (Pre-Development - FTWL)

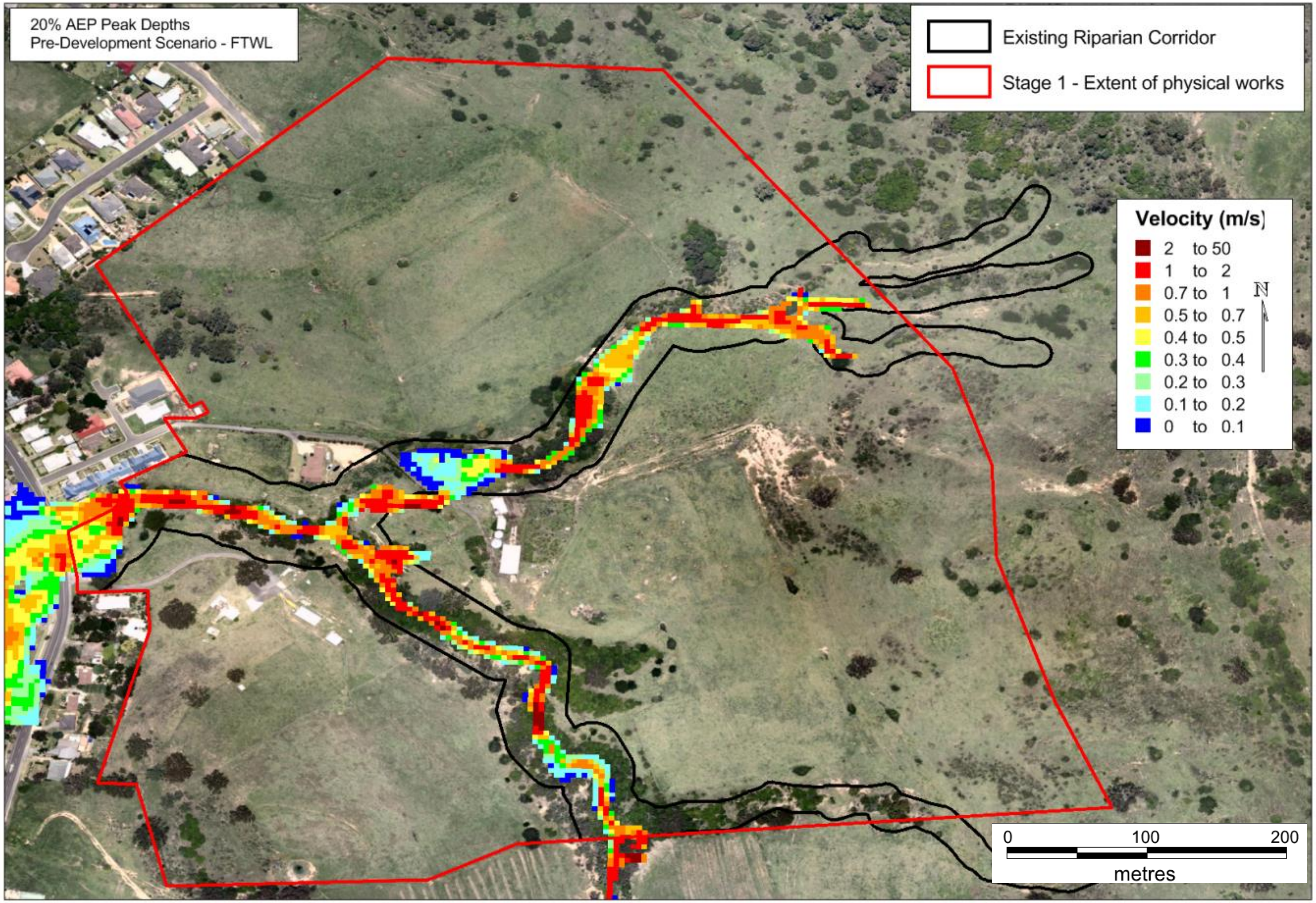


Figure 5-6 20% AEP peak Velocity (Pre-Development - FTWL)

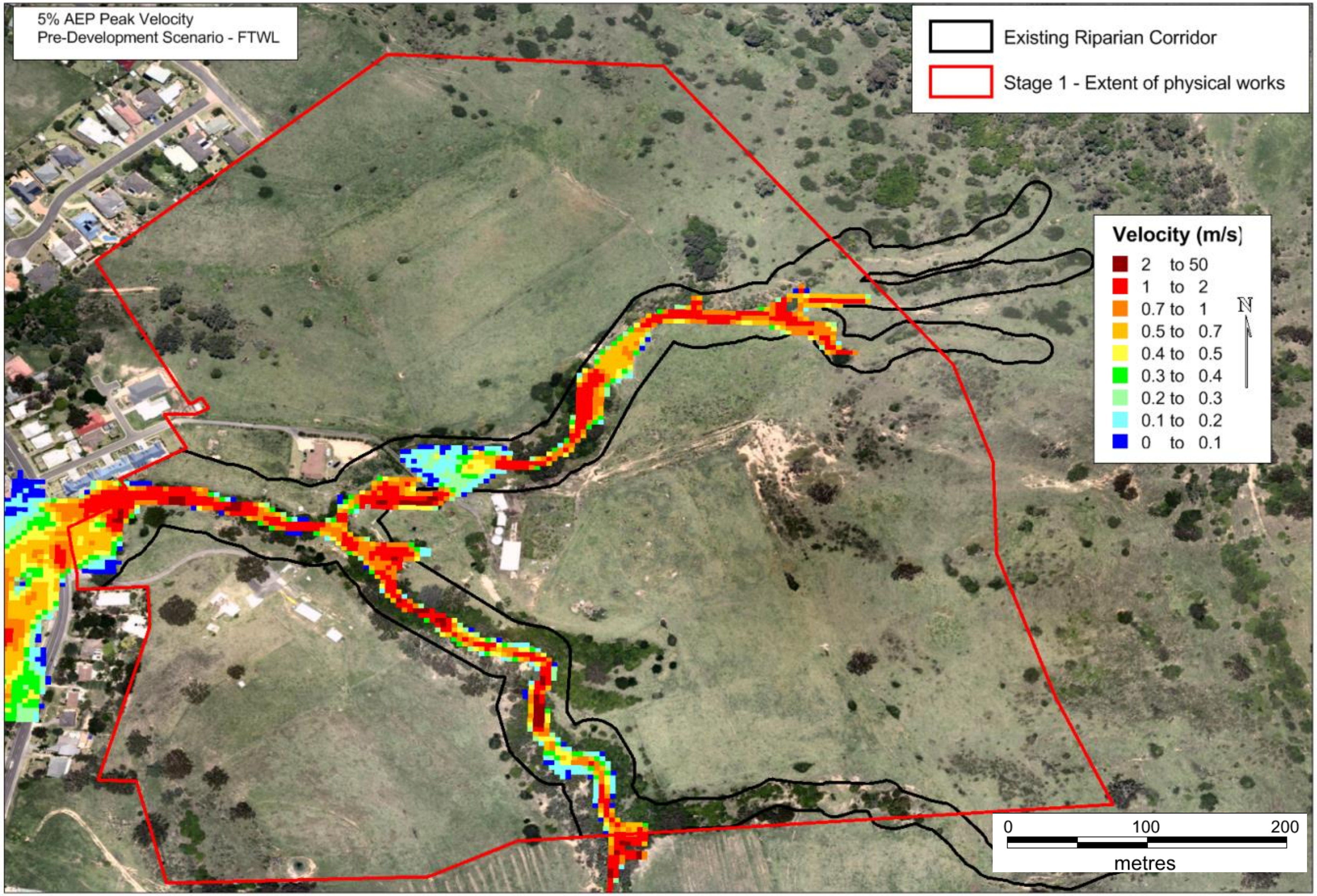


Figure 5-7 5% AEP peak Velocity (Pre-Development - FTWL)

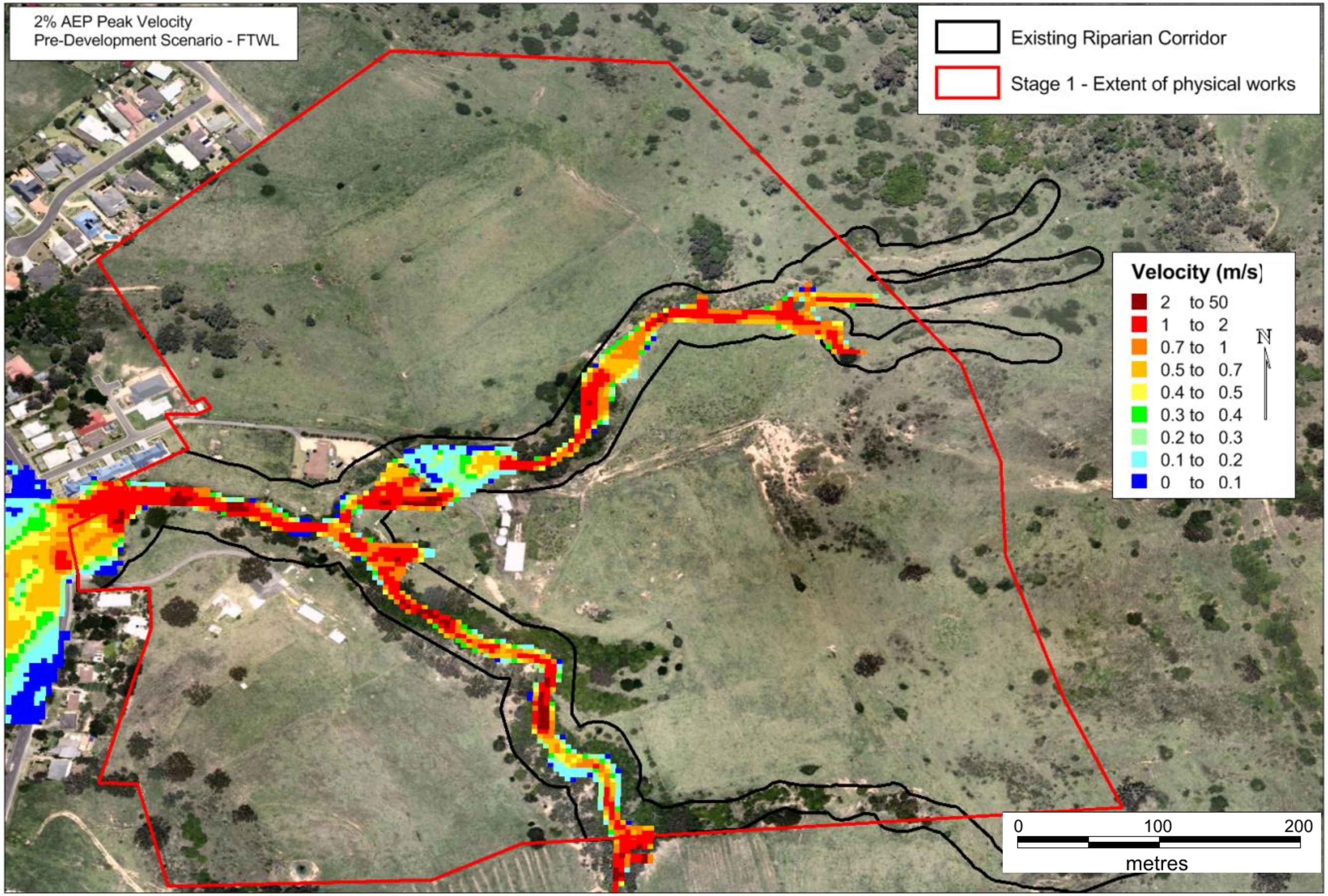


Figure 5-8 2% AEP peak Velocity (Pre-Development - FTWL)

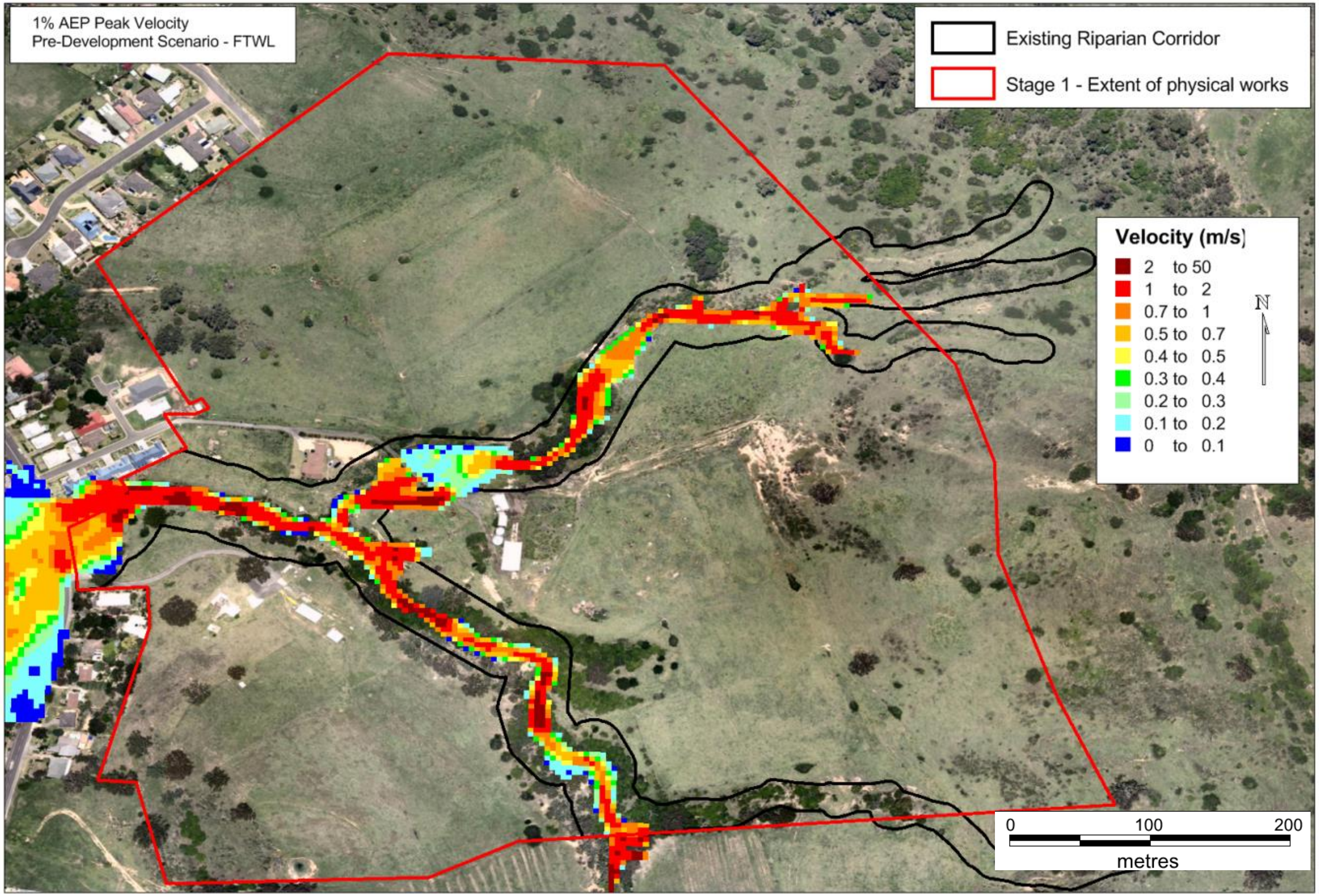


Figure 5-9 1% AEP peak Velocity (Pre-Development - FTWL)

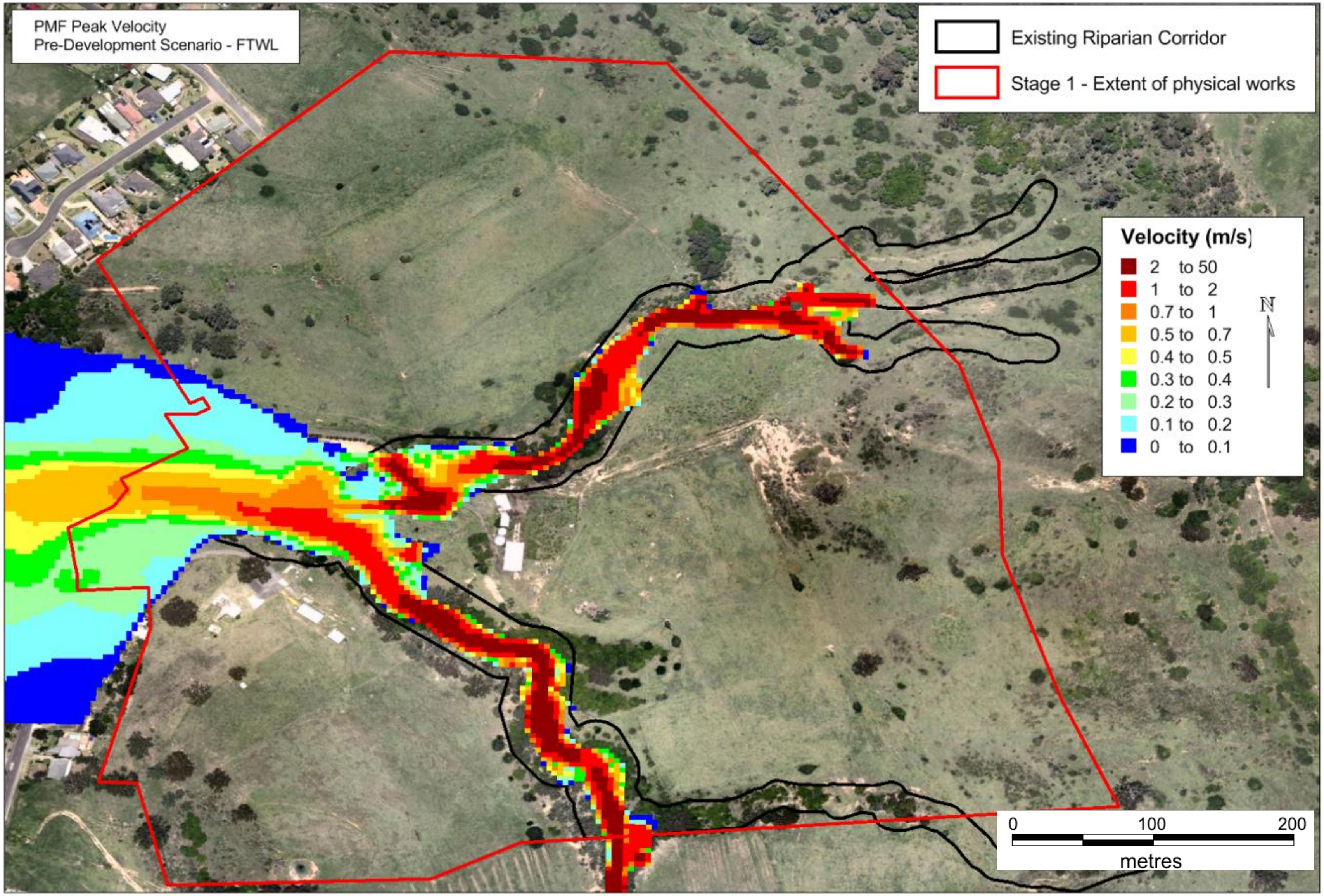


Figure 5-10 PMF peak Velocity (Pre-Development - FTWL)

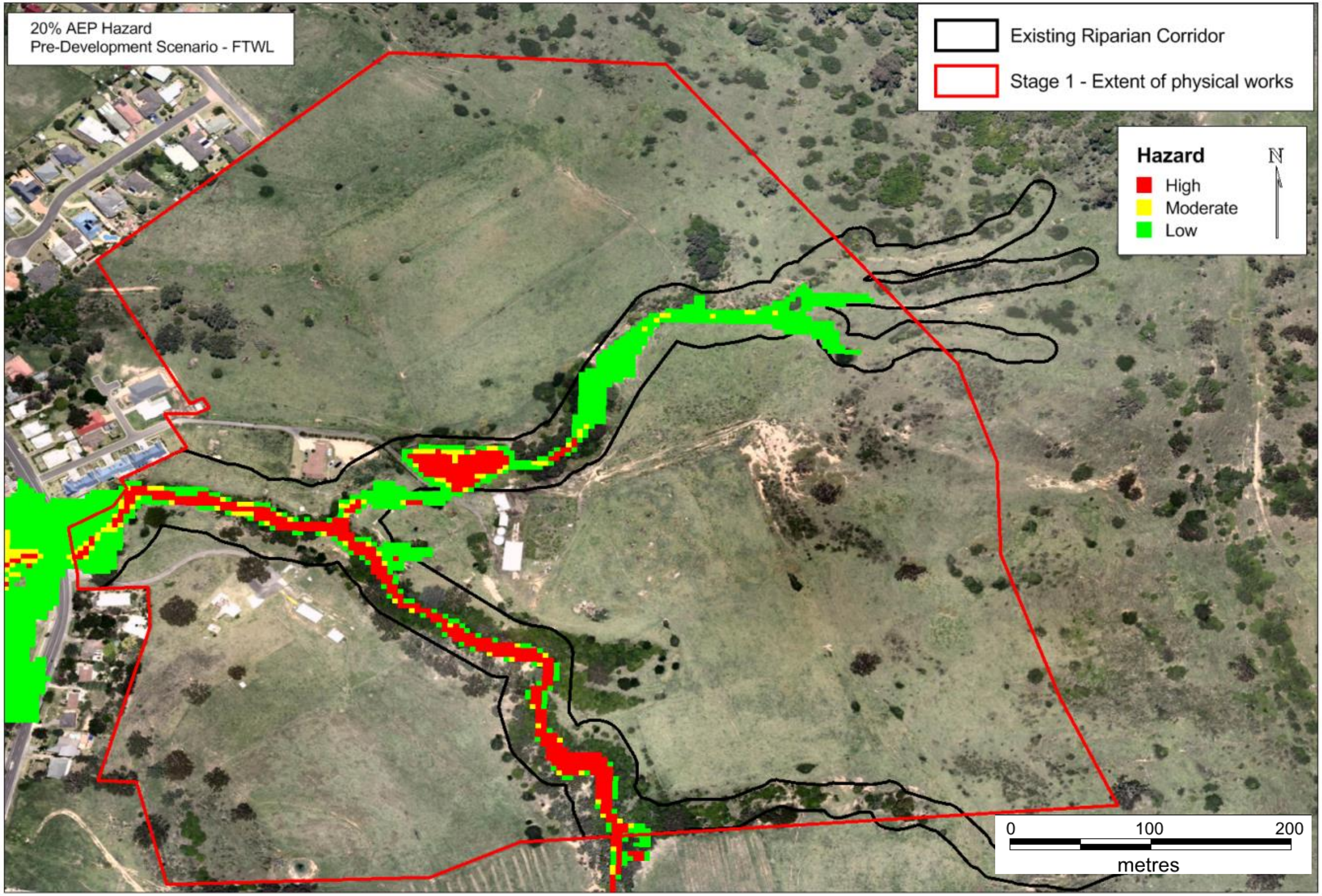


Figure 5-11 20% AEP Hazard (Pre-Development - FTWL)

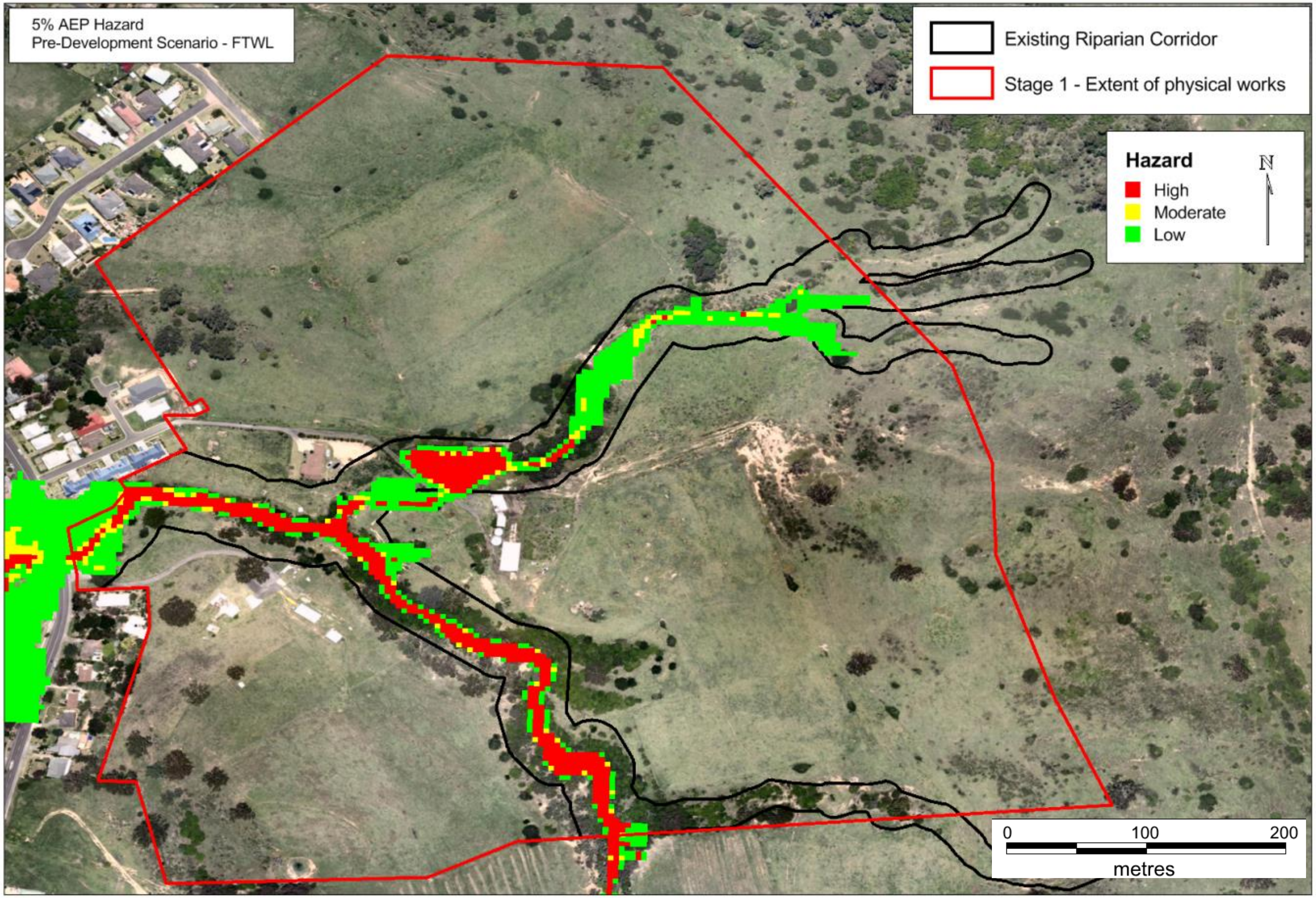


Figure 5-12 5% AEP Hazard (Pre-Development - FTWL)

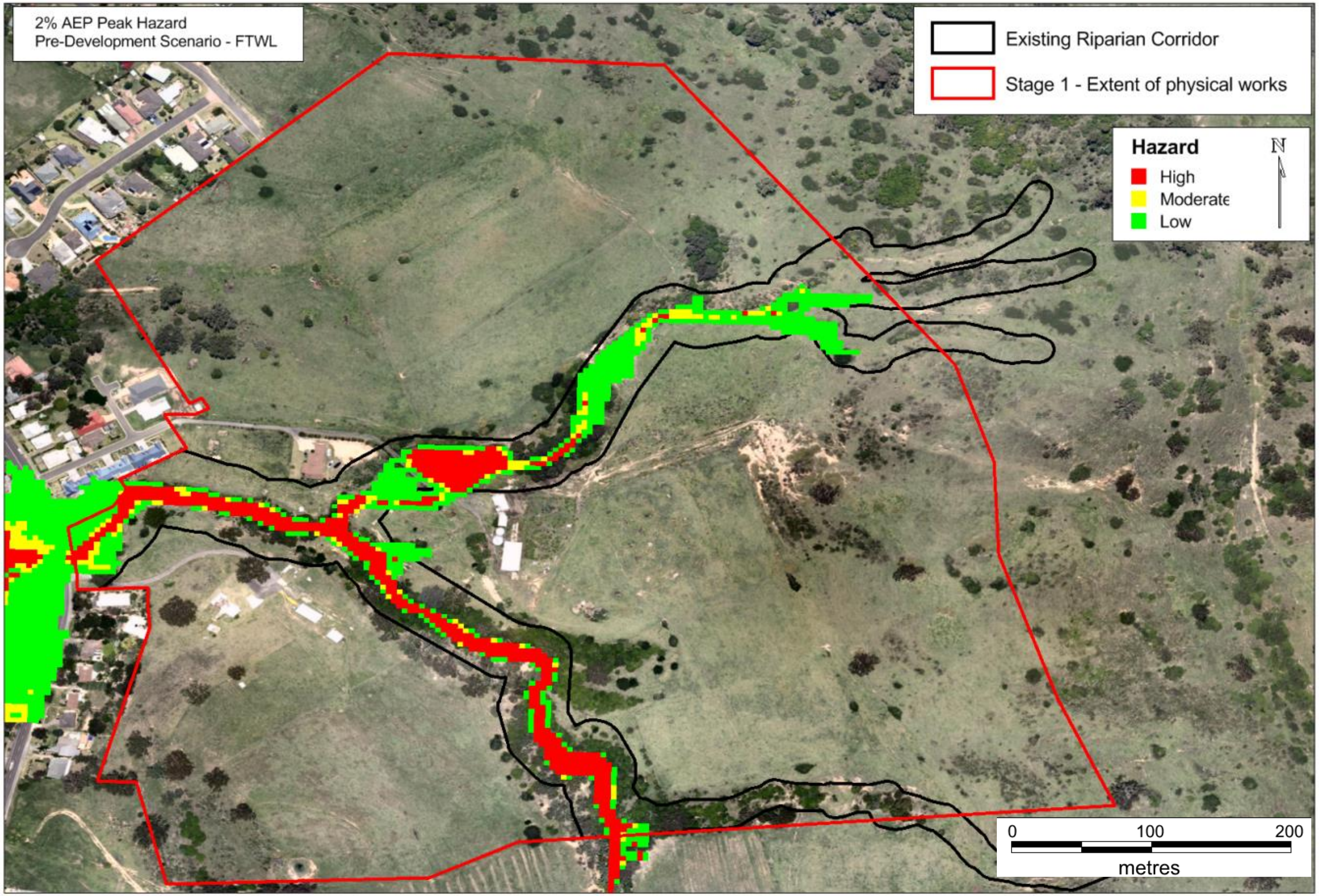


Figure 5-13 2% AEP Hazard (Pre-Development - FTWL)

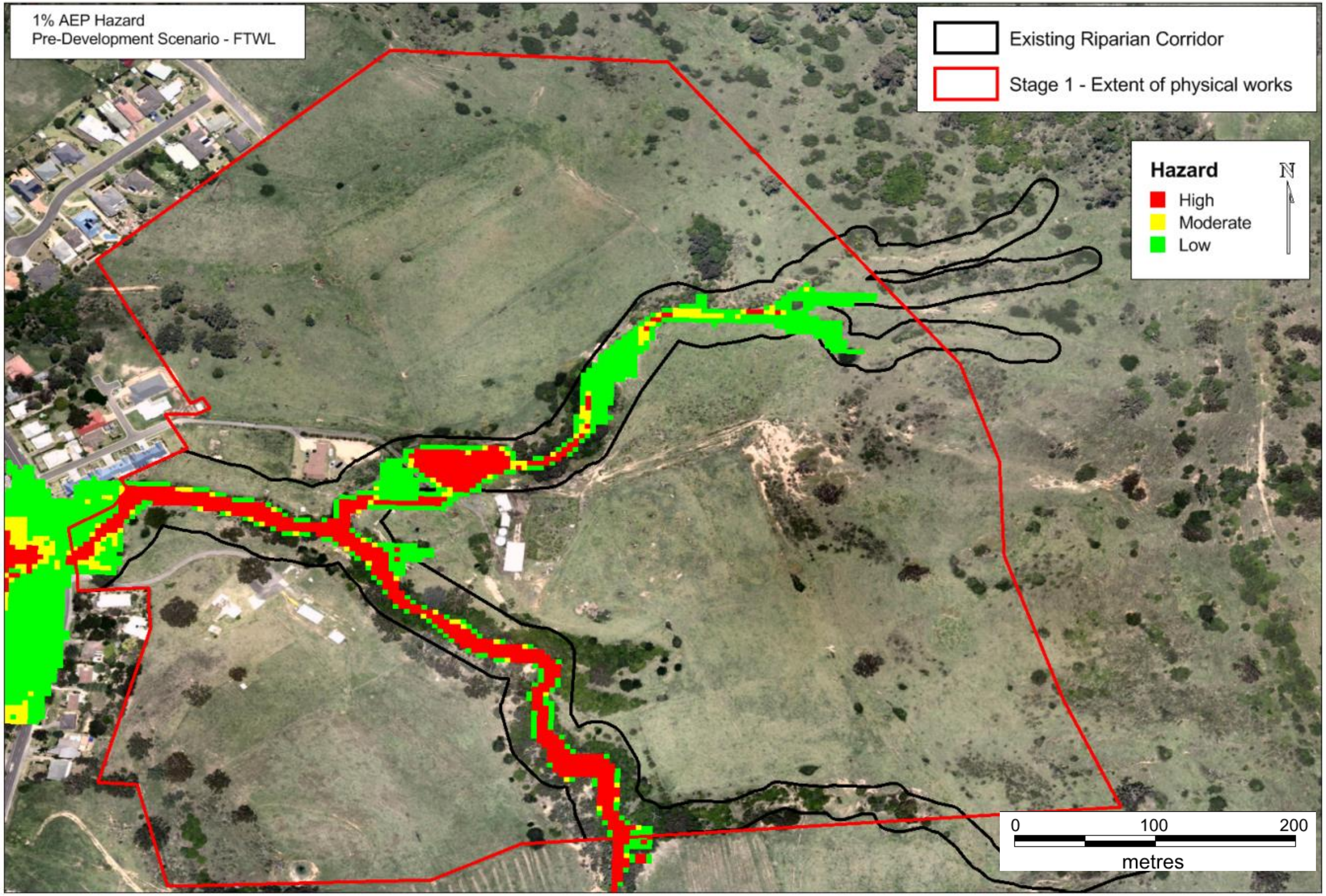


Figure 5-14 1% AEP Hazard (Pre-Development - FTWL)

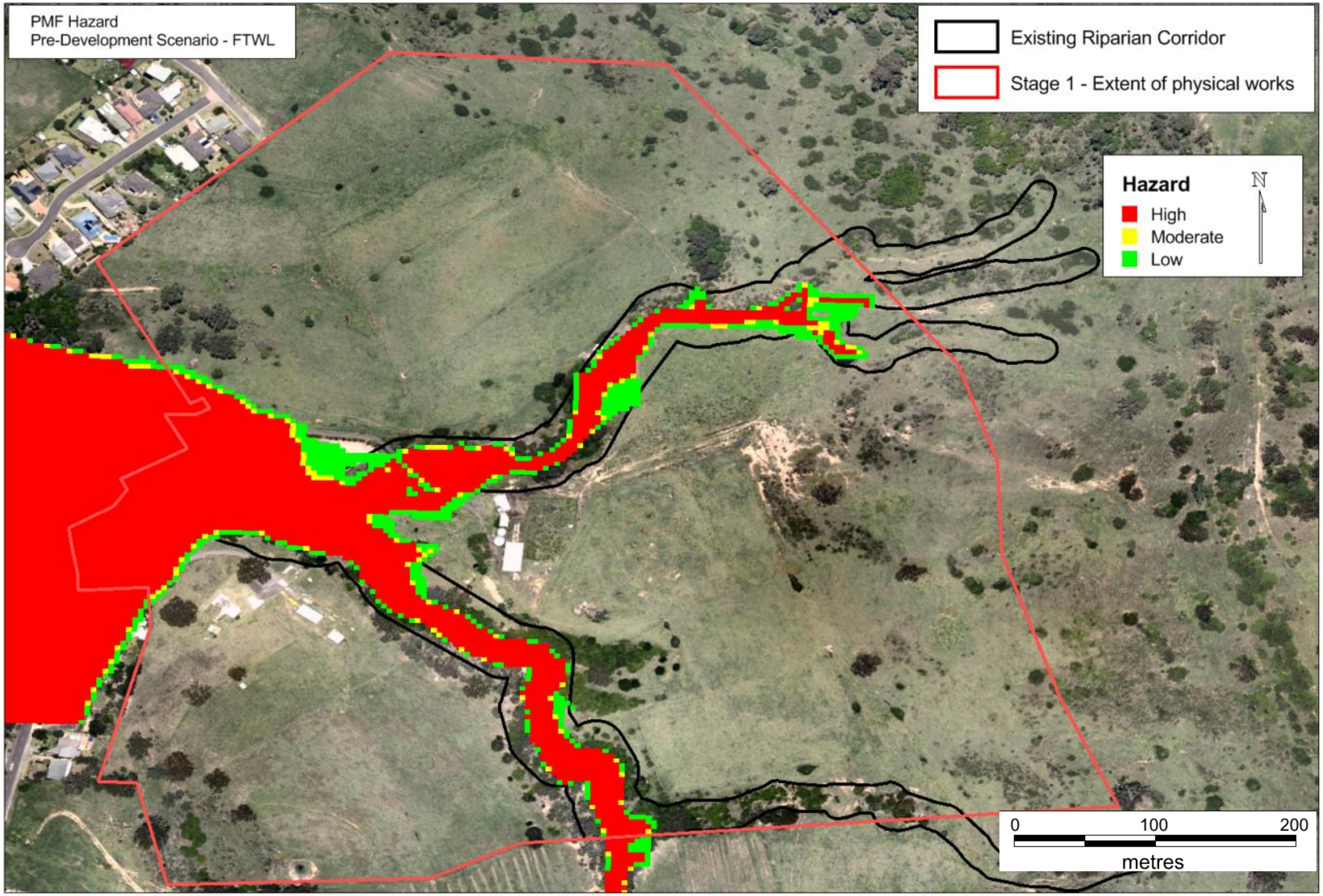


Figure 5-15 PMF Hazard (Pre-Development - FTWL)

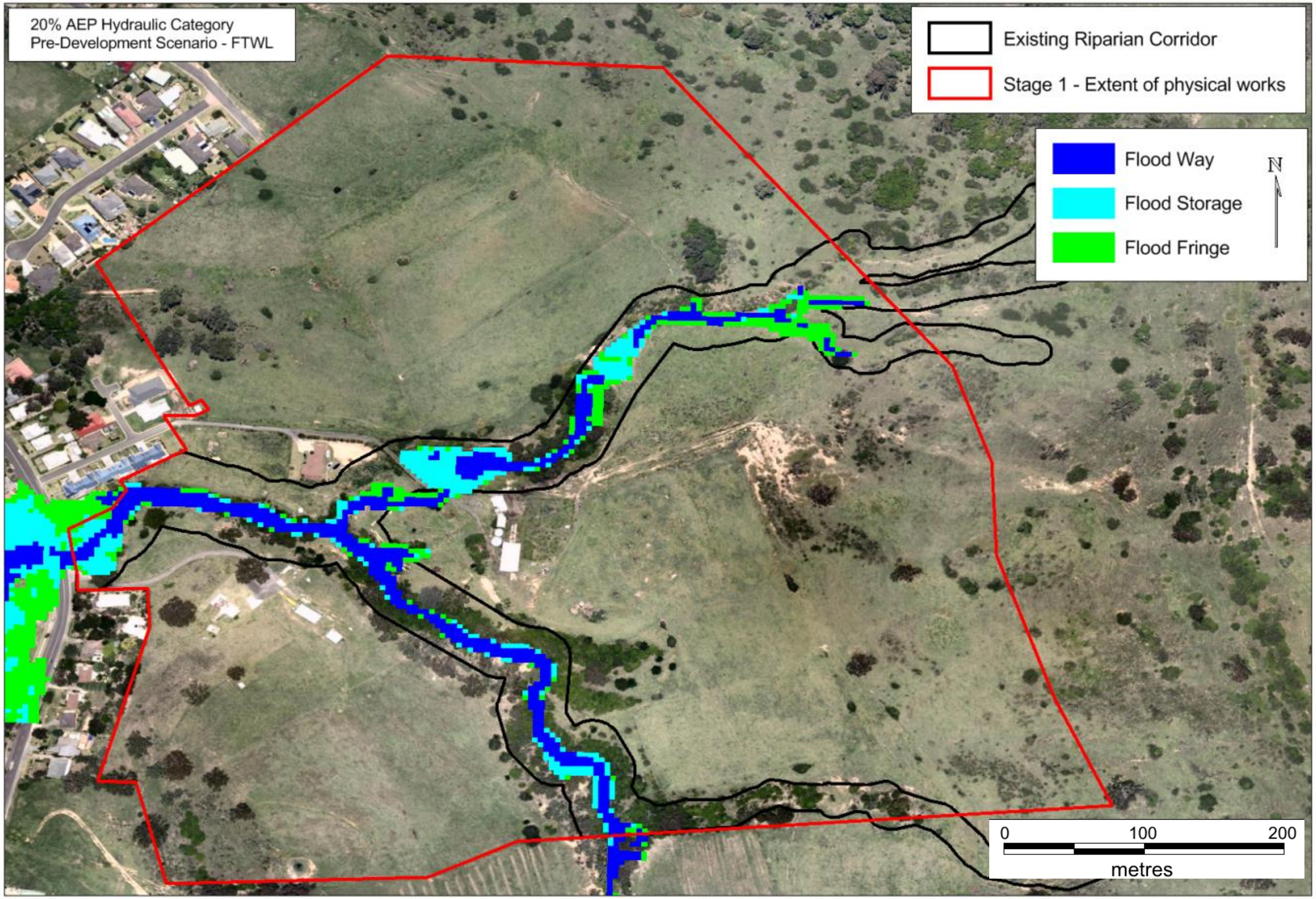


Figure 5-16 20% AEP Hydraulic Category (Pre-Development - FTWL)

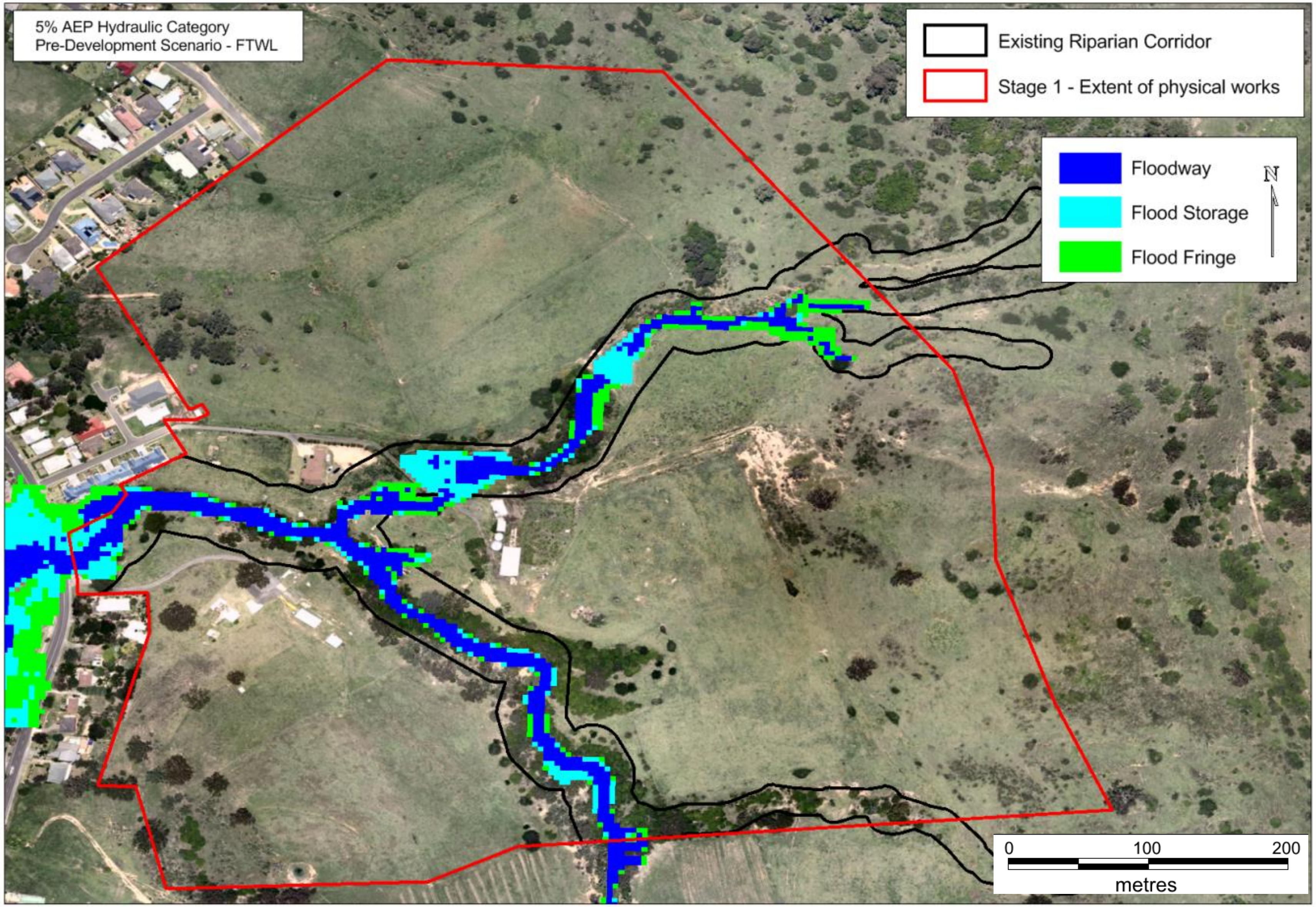


Figure 5-17 5% AEP Hydraulic Category (Pre-Development - FTWL)

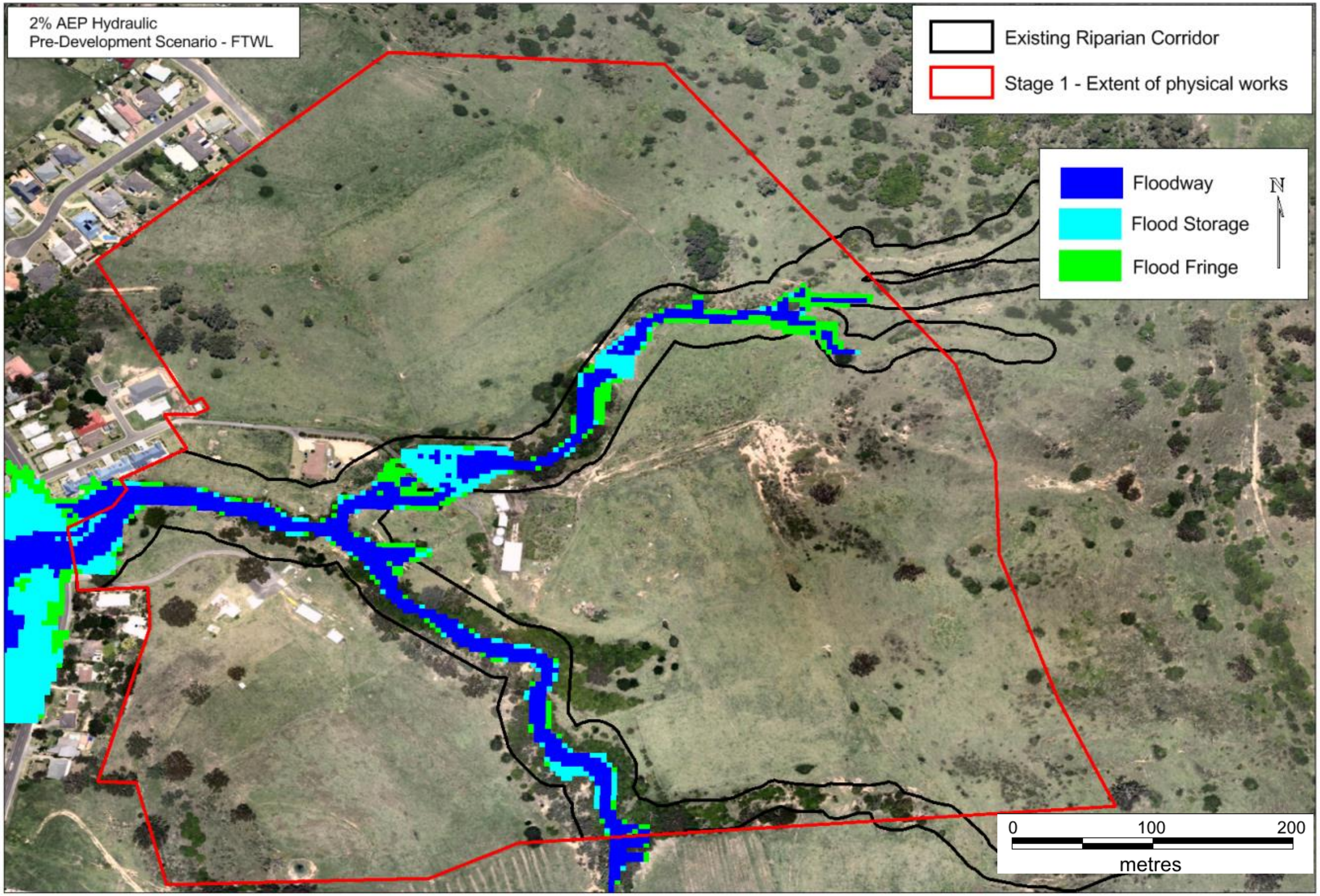


Figure 5-18 2% AEP Hydraulic Category (Pre-Development - FTWL)

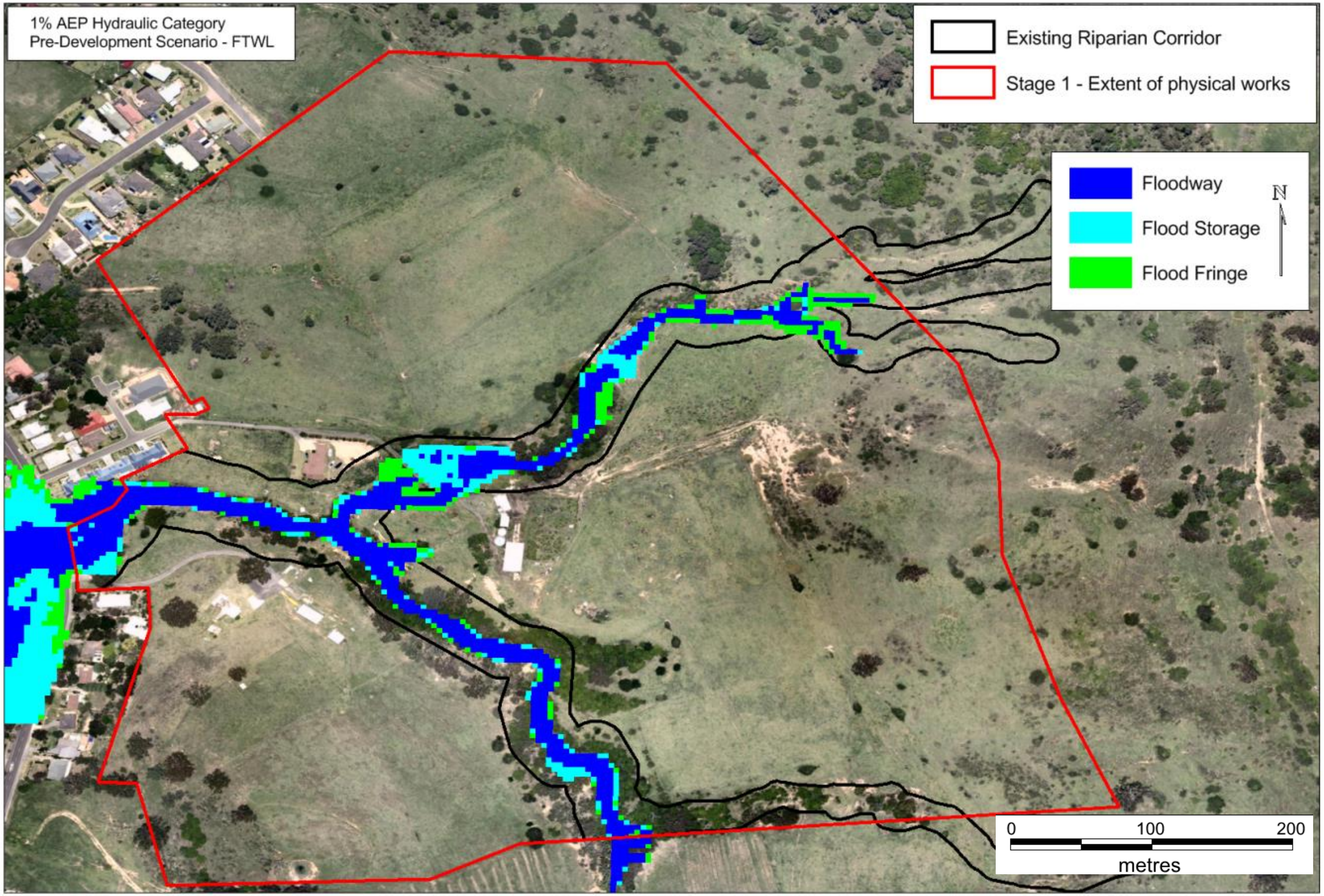


Figure 5-19 1% AEP Hydraulic Category (Pre-Development - FTWL)

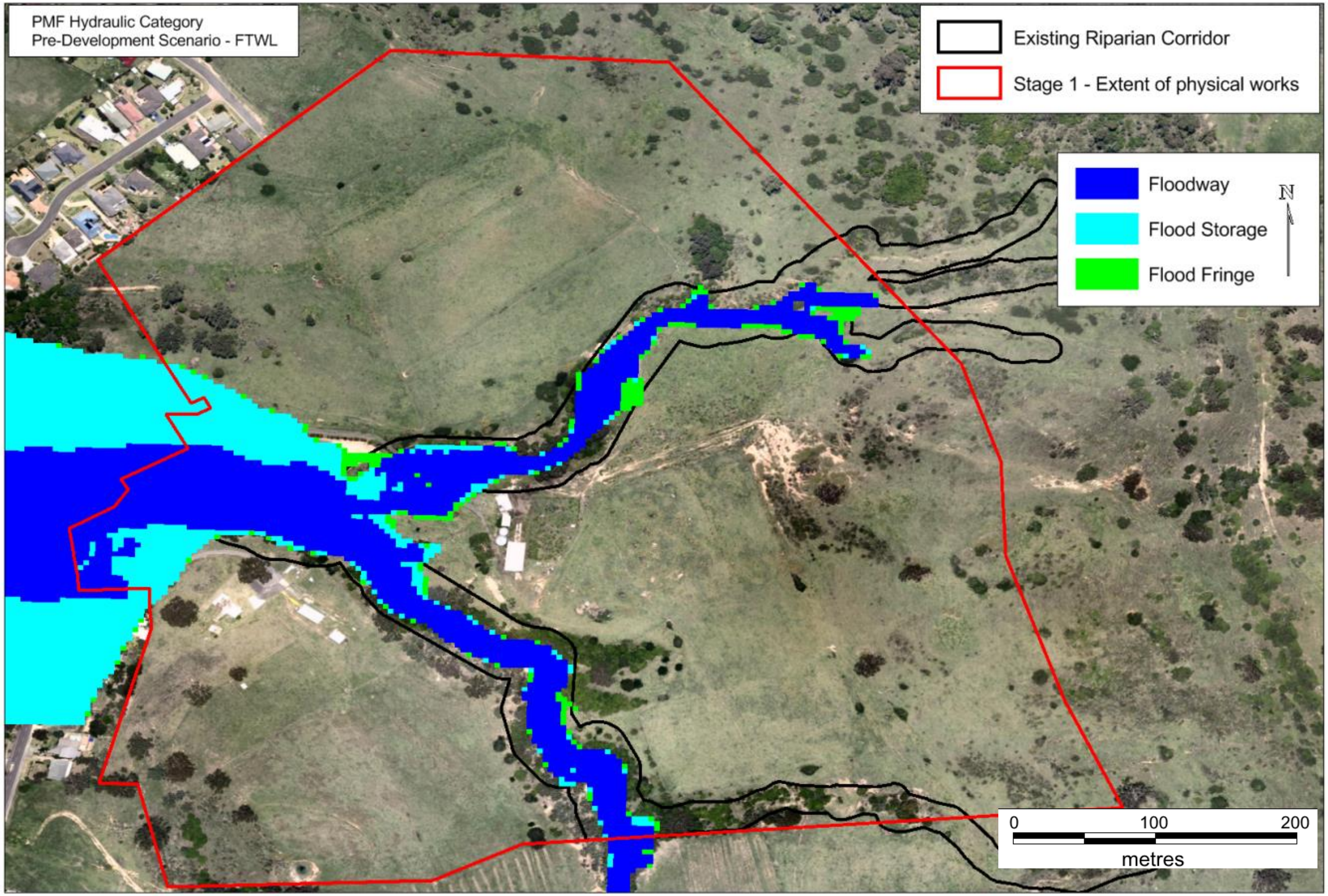


Figure 5-20 PMF Hydraulic Category (Pre-Development - FTWL)

5.2 Post-development Flood Results

The post-development flood model results for depth, velocity, hazard and hydraulic categories are provided in a series of figures. The associated figure numbers are outlined in **Table 5-2**.

Table 5-2 Post-Development Figure Numbers

Flood Event	Peak Depth	Peak Velocity	Provisional Hazard	Hydraulic categorisation
5 year ARI	Figure 5.21	Figure 5.26	Figure 5.31	Figure 5.36
20 year ARI	Figure 5.22	Figure 5.27	Figure 5.32	Figure 5.37
50 year ARI	Figure 5.23	Figure 5.28	Figure 5.33	Figure 5.38
100 year ARI	Figure 5.29	Figure 5.29	Figure 5.34	Figure 5.39
PMF	Figure 5.30	Figure 5.30	Figure 5.35	Figure 5.40

The Post-development results are similar to that of the pre-development results. Within the figures the proposed basin alignment is evident in the depth plots. This is the main change to the flood behaviour within the model which aims at retarding the additional runoff from the catchments back to the existing levels. The details of the assessment of the change in flood depths is presented and discussed in Section 6.1.

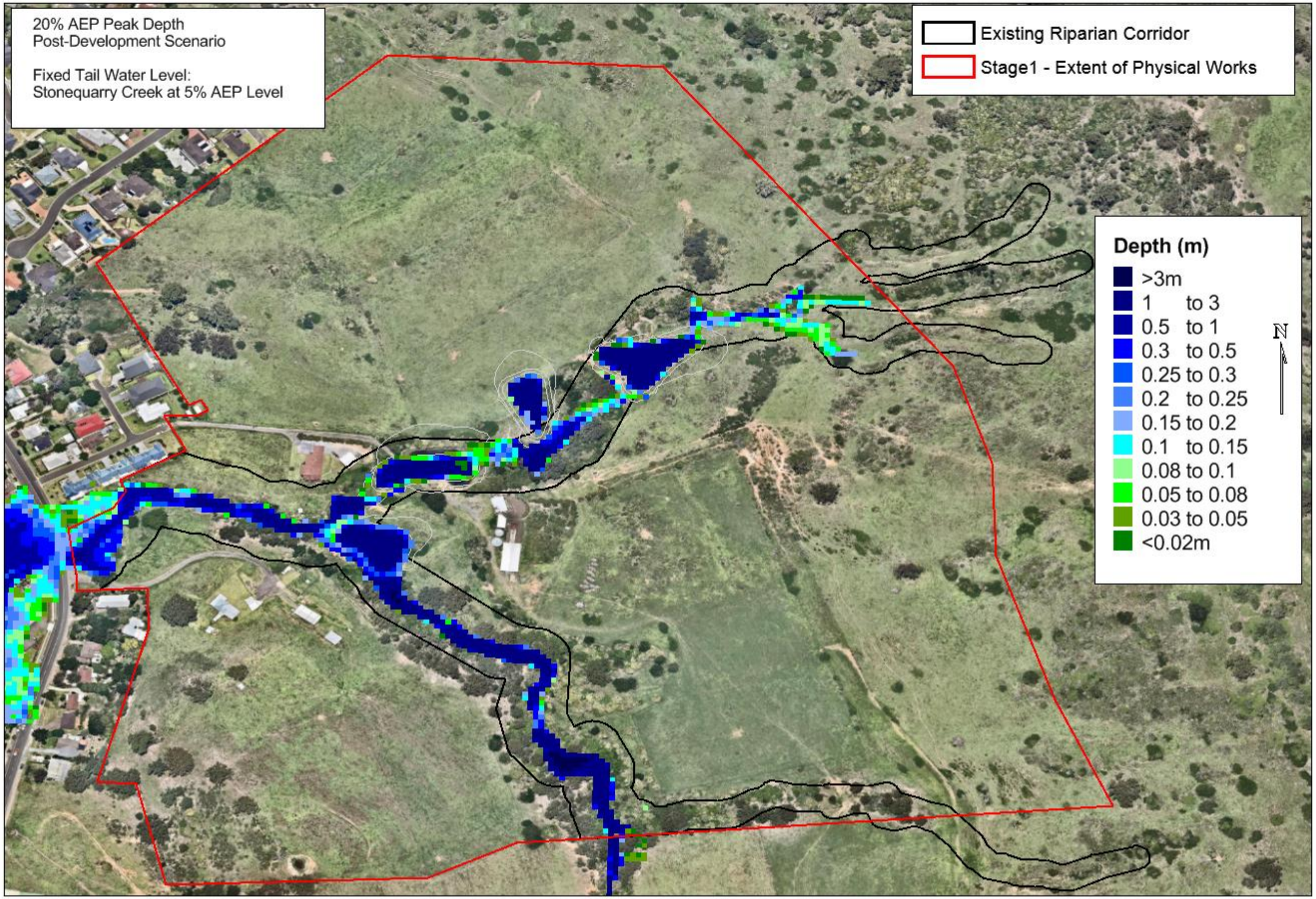


Figure 5-21 20% AEP peak flood depths (Post-Development - FTWL)

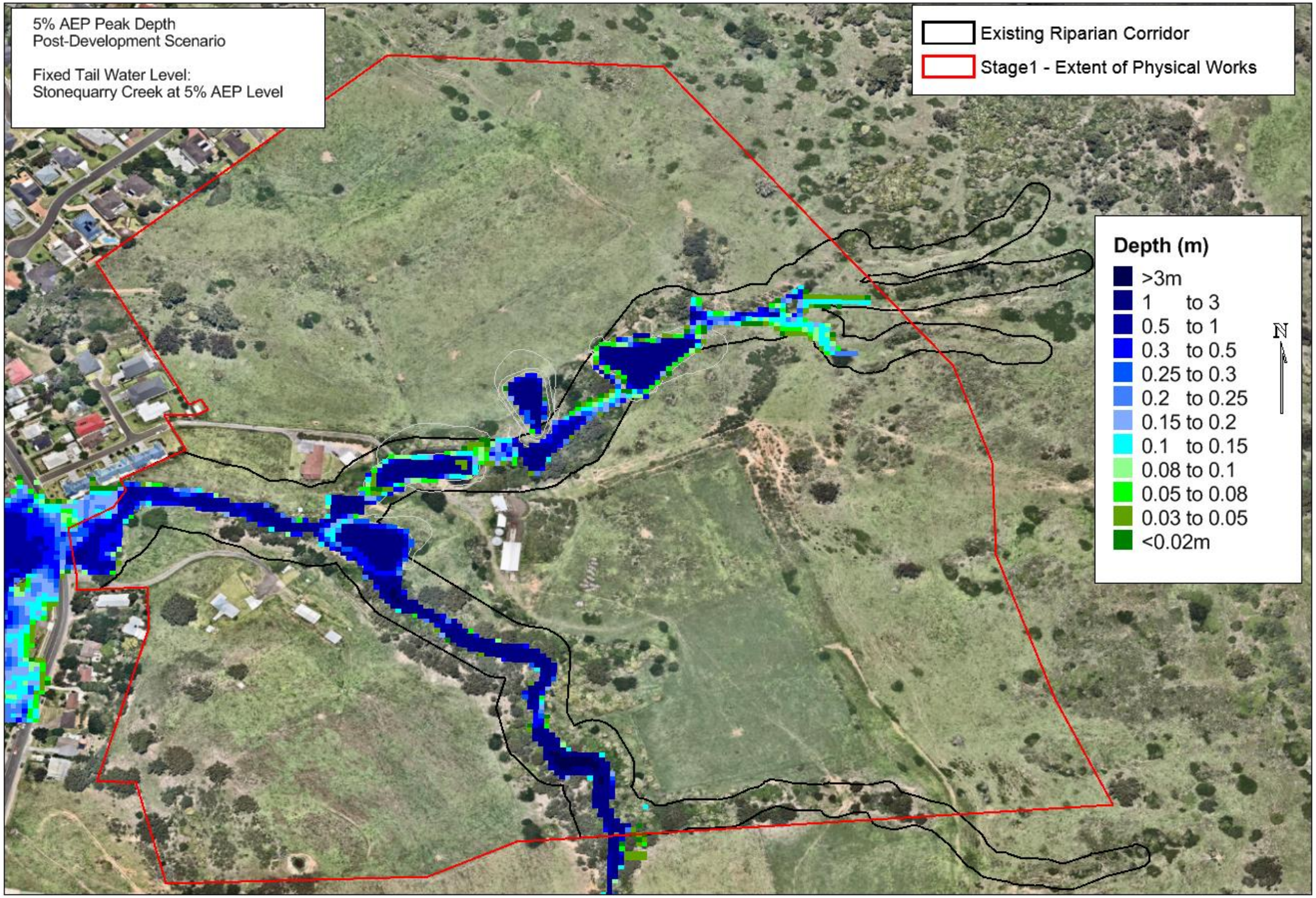


Figure 5-22 5% AEP peak flood depths (Post-Development - FTWL)

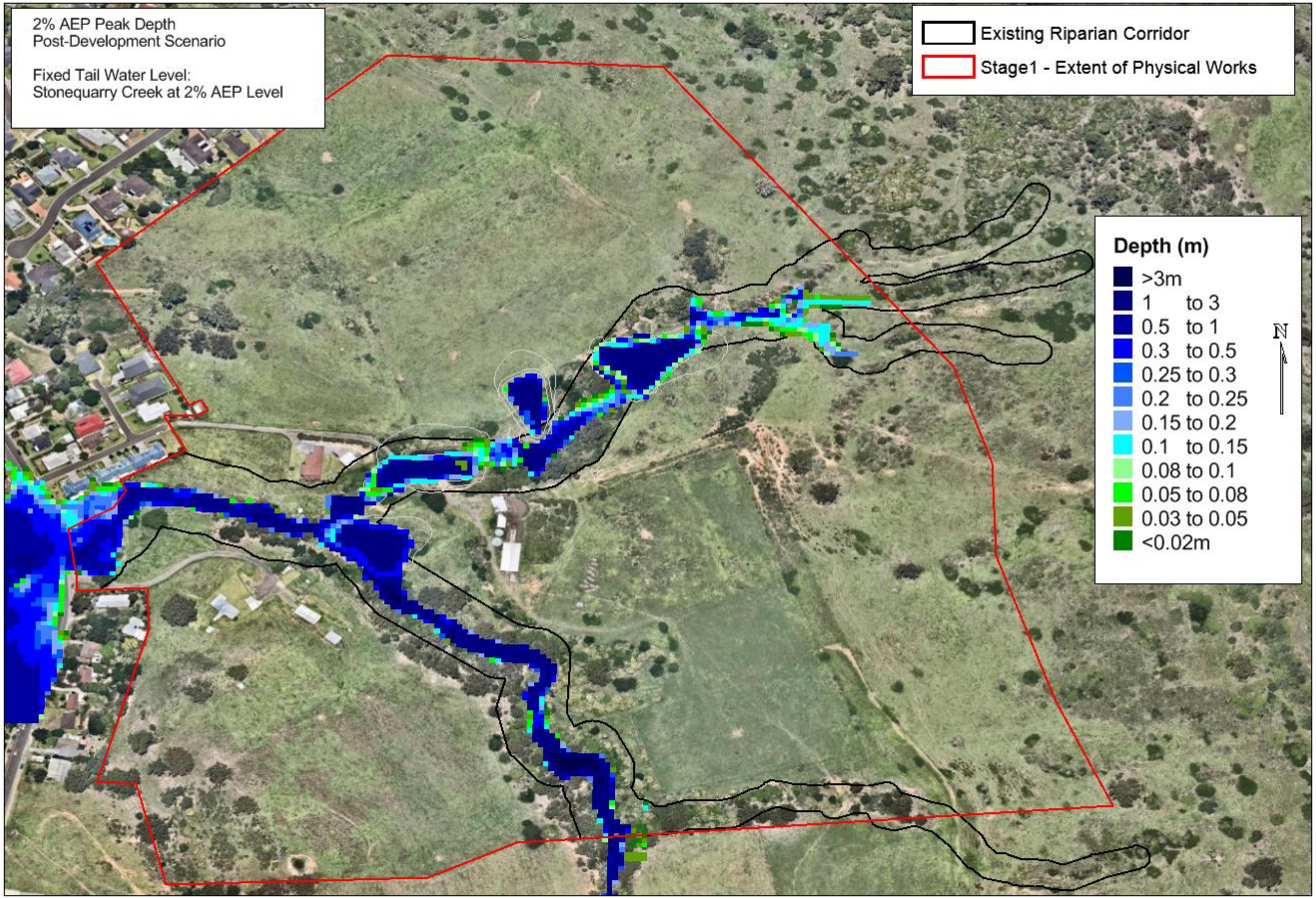


Figure 5-23 2% AEP peak flood depths (Post-Development - FTWL)

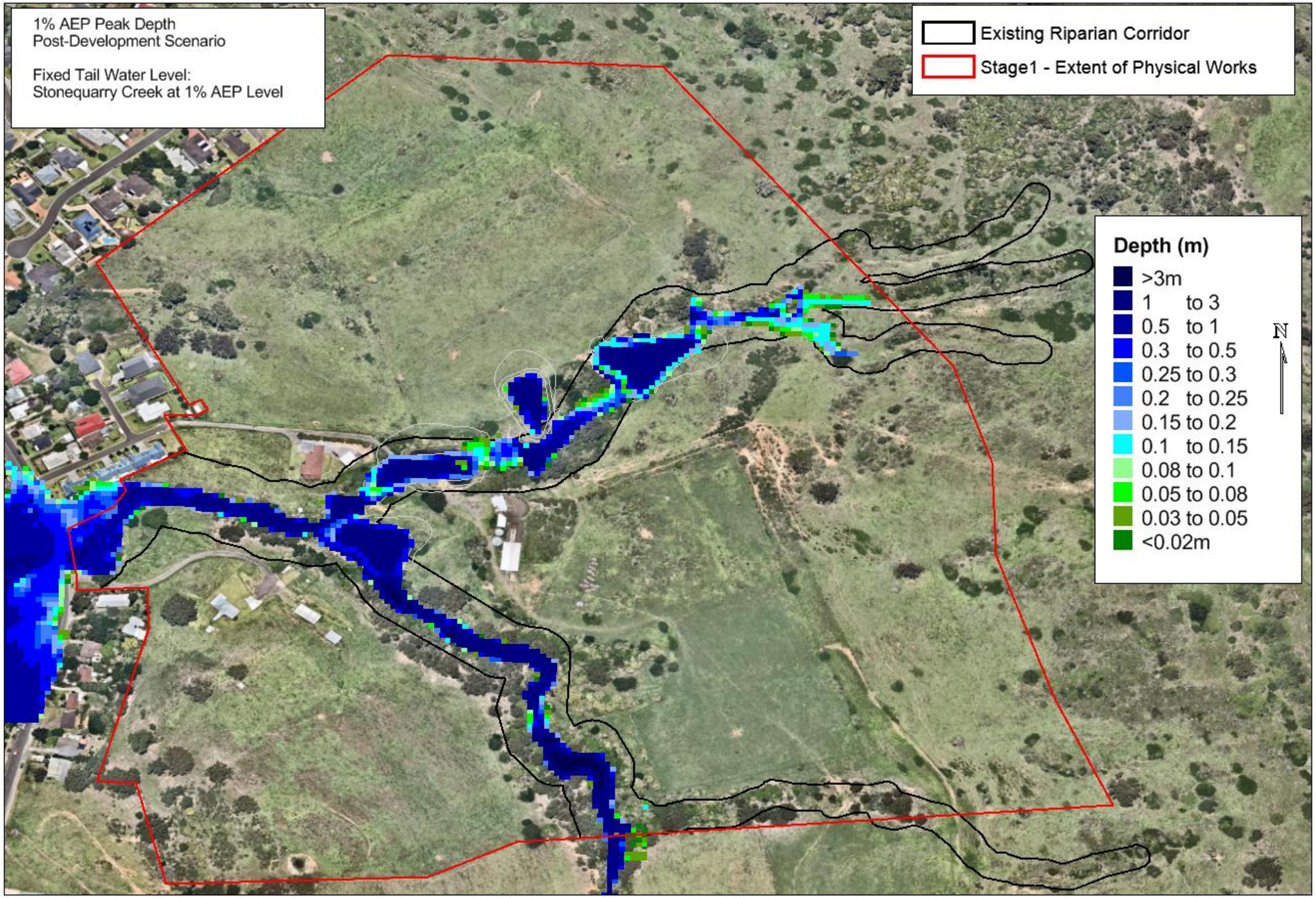


Figure 5-24 1% AEP peak flood depths (Post-Development - FTWL)

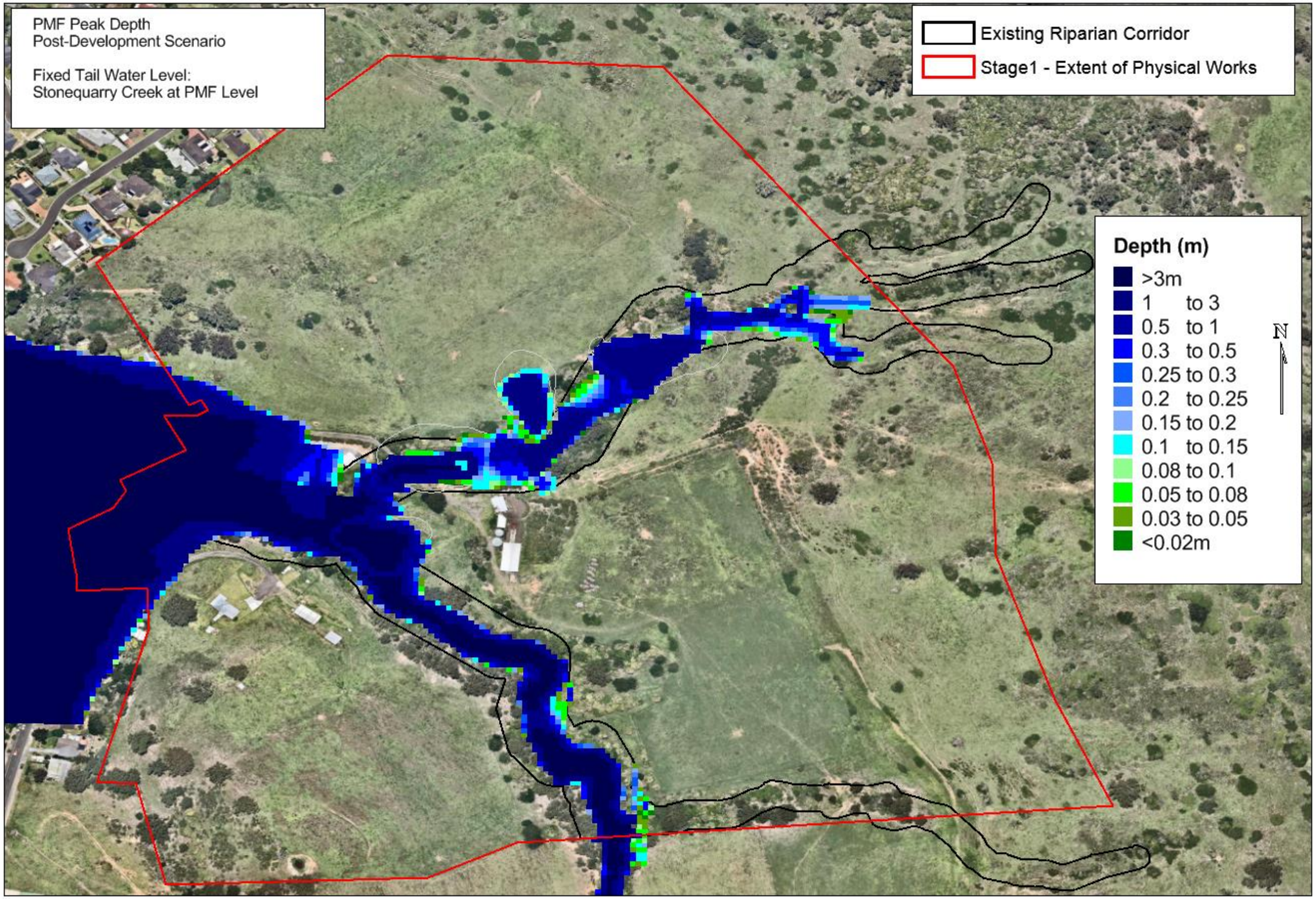


Figure 5-25 PMF peak flood depths (Post-Development - FTWL)

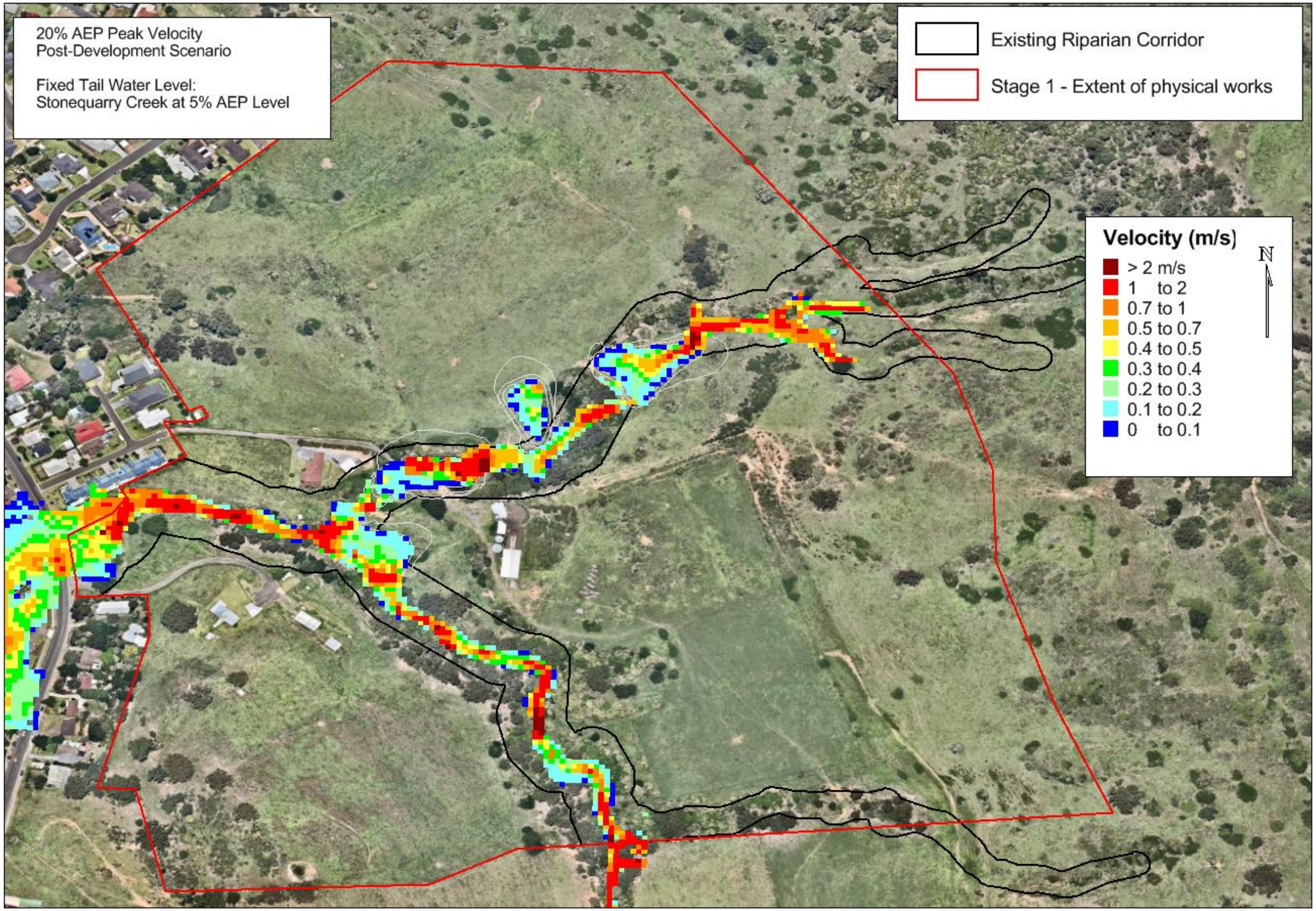


Figure 5-26 20% AEP peak Velocity (Post-Development - FTWL)

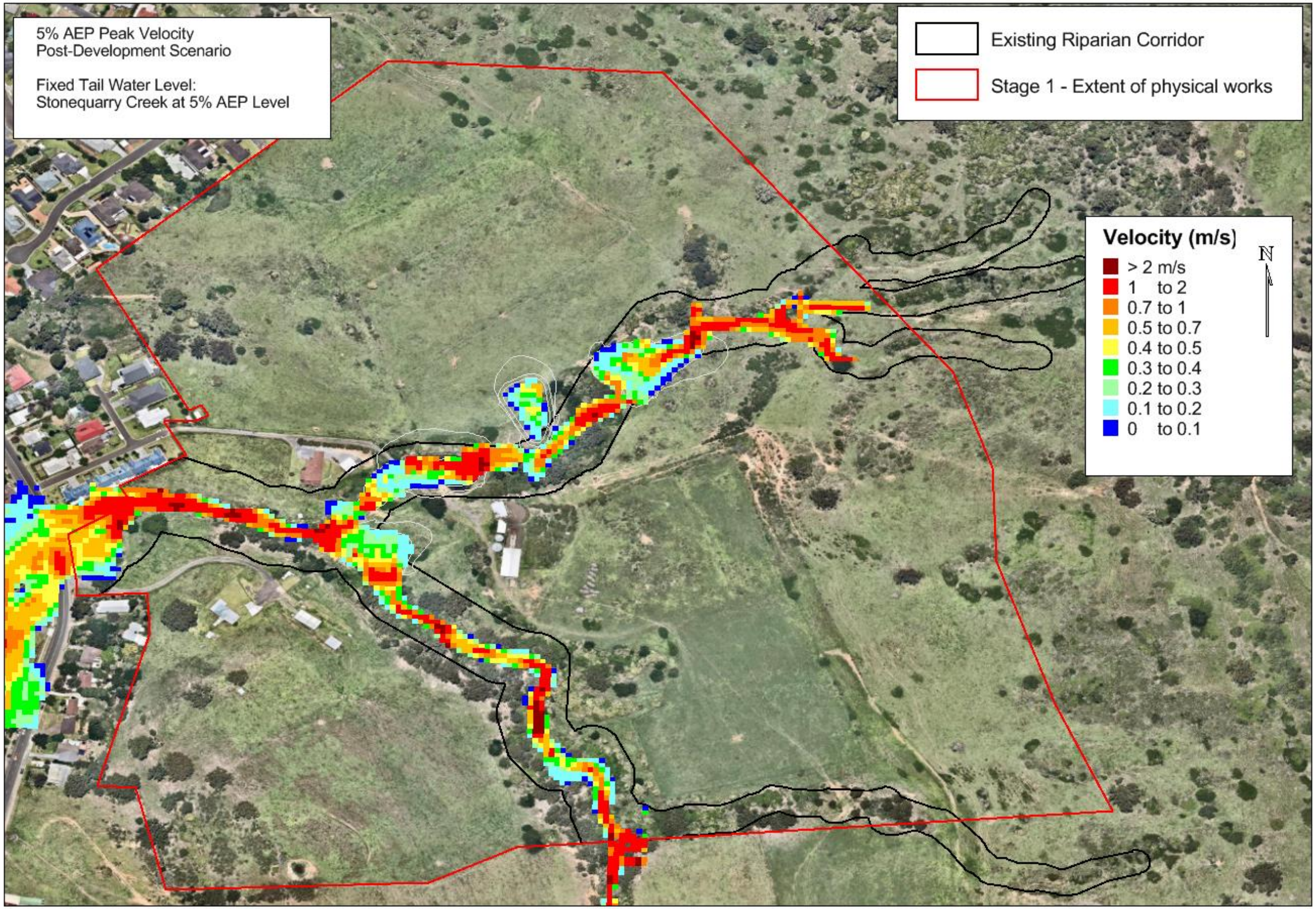


Figure 5-27 5% AEP peak Velocity (Post-Development - FTWL)

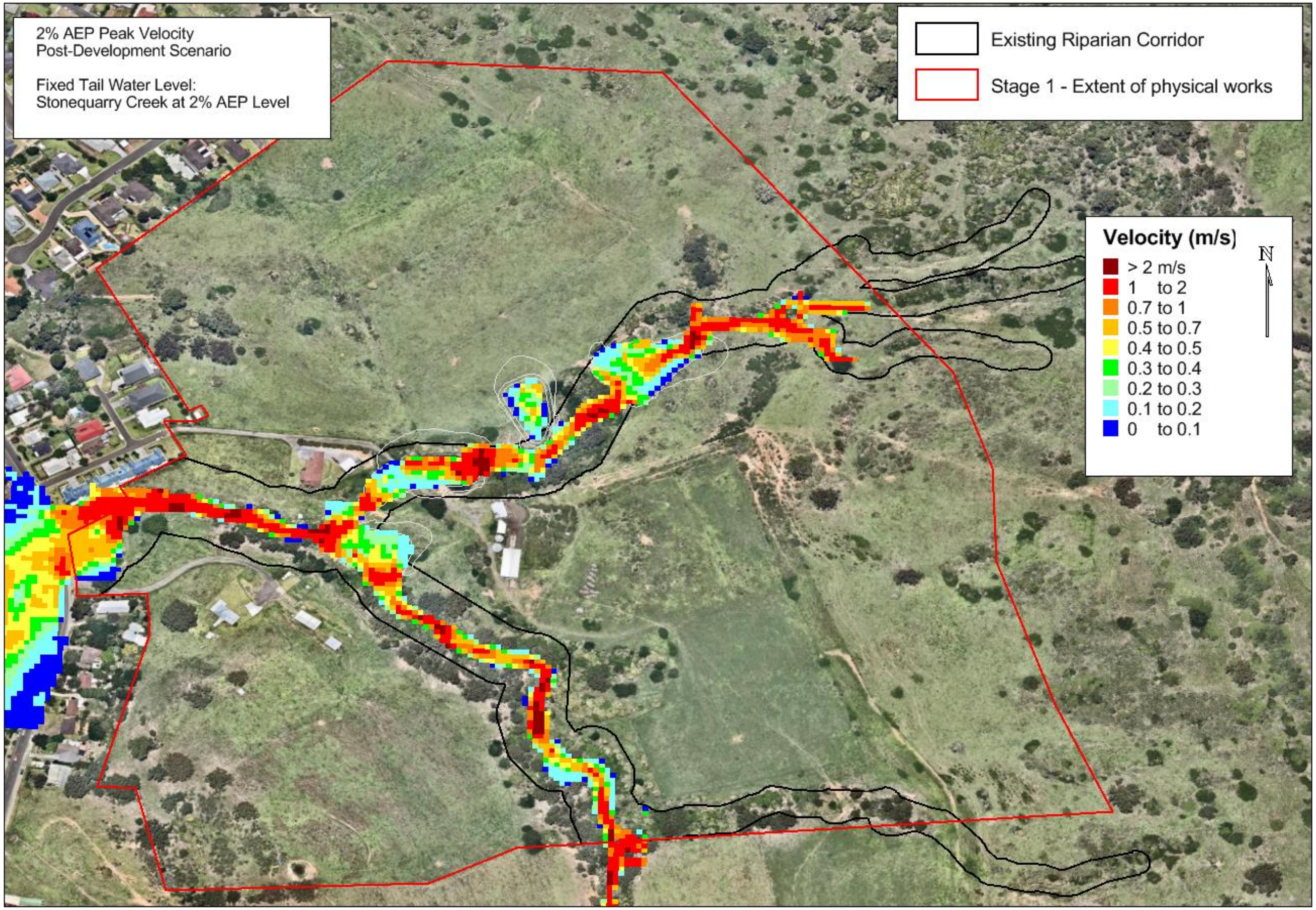


Figure 5-28 2% AEP peak Velocity (Post-Development - FTWL)

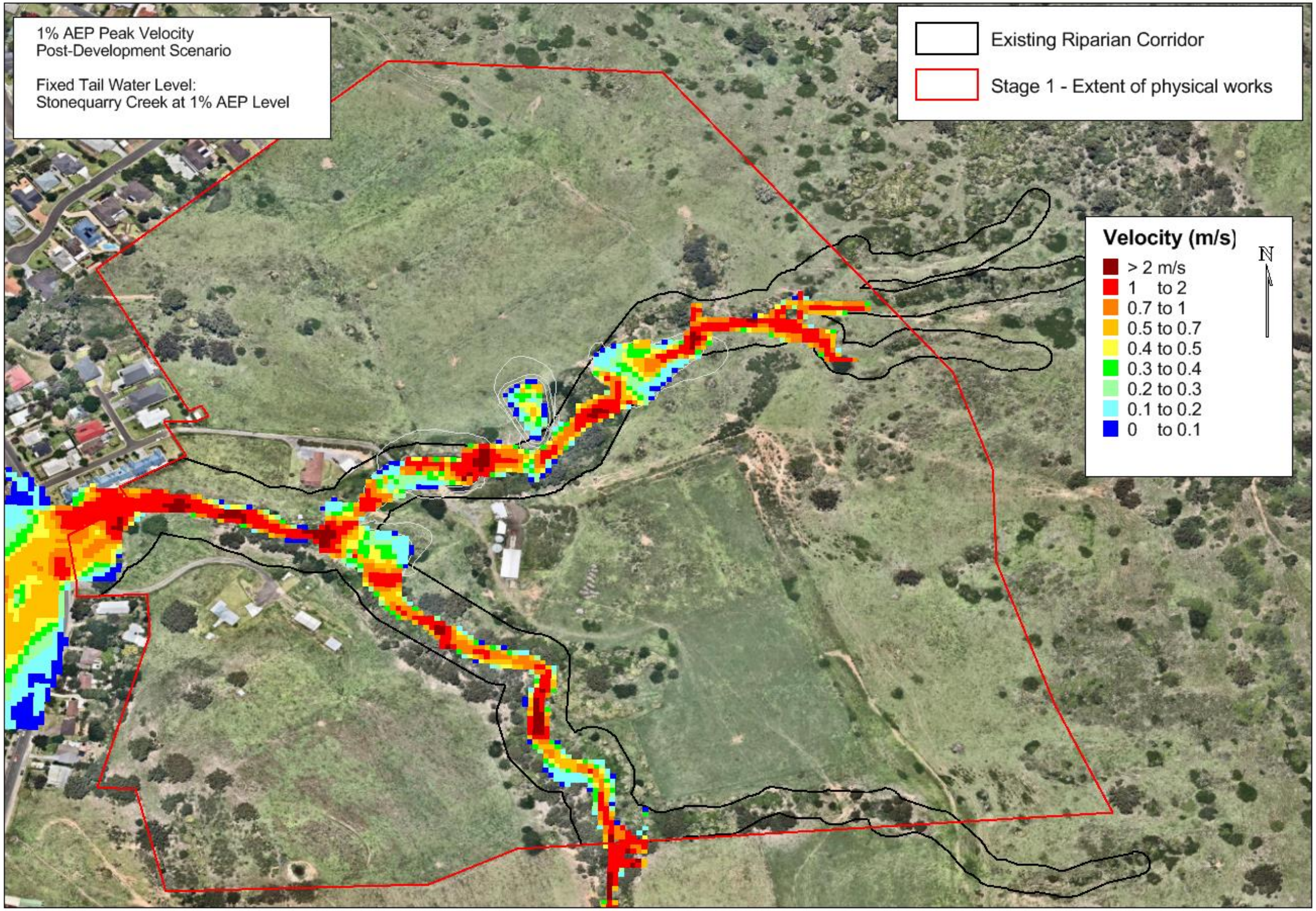


Figure 5-29 1% AEP peak Velocity (Post-Development - FTWL)

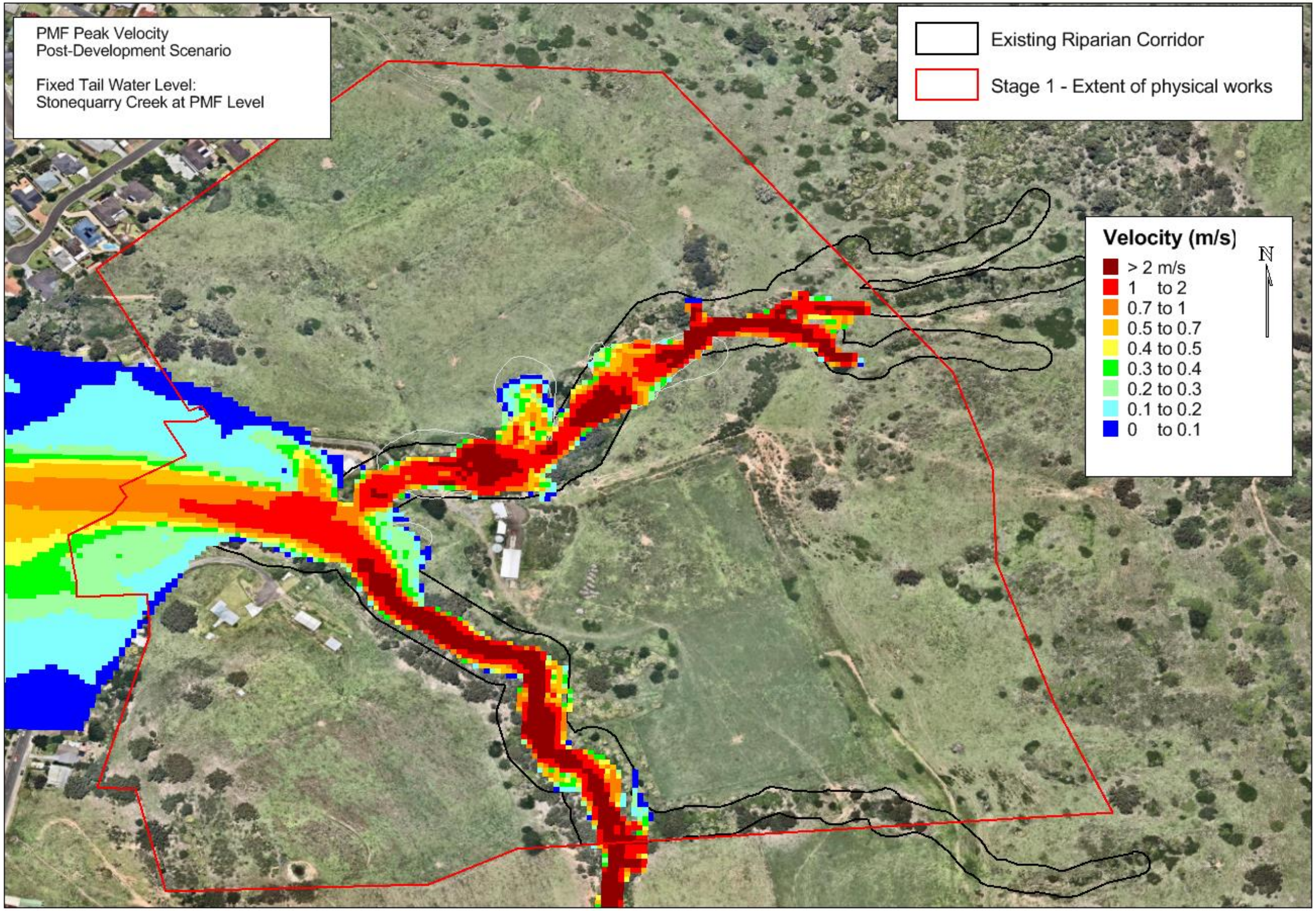


Figure 5-30 PMF peak Velocity (Post-Development - FTWL)

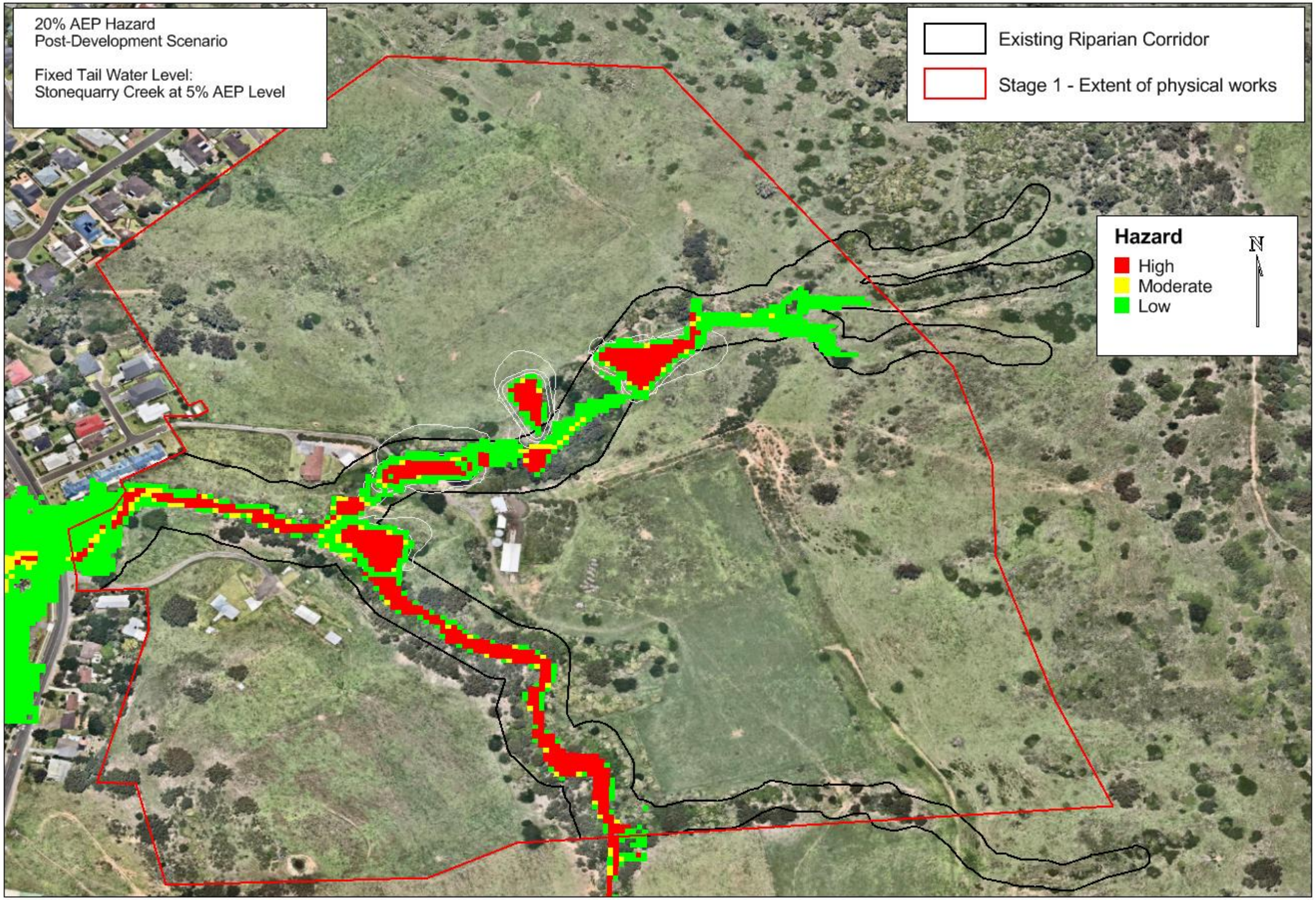


Figure 5-31 20% AEP Hazard (Post-Development - FTWL)

Figure 5-32 5% AEP Hazard (Post-Development - FTWL)

Figure 5-33 2% AEP Hazard (Post-Development - FTWL)

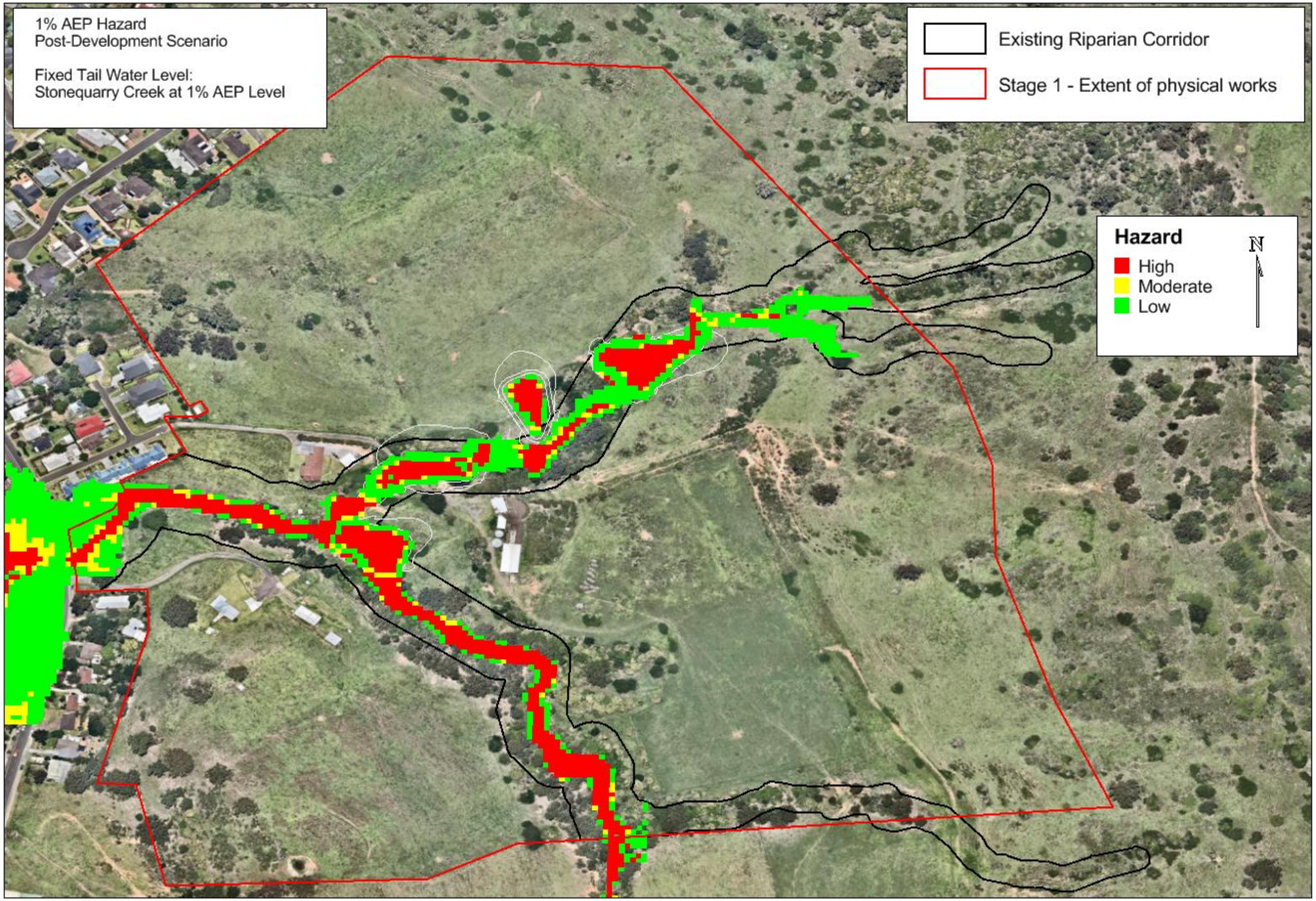


Figure 5-34 1% AEP Hazard (Post-Development - FTWL)

Figure 5-35 PMF Hazard (Post-Development - FTWL)

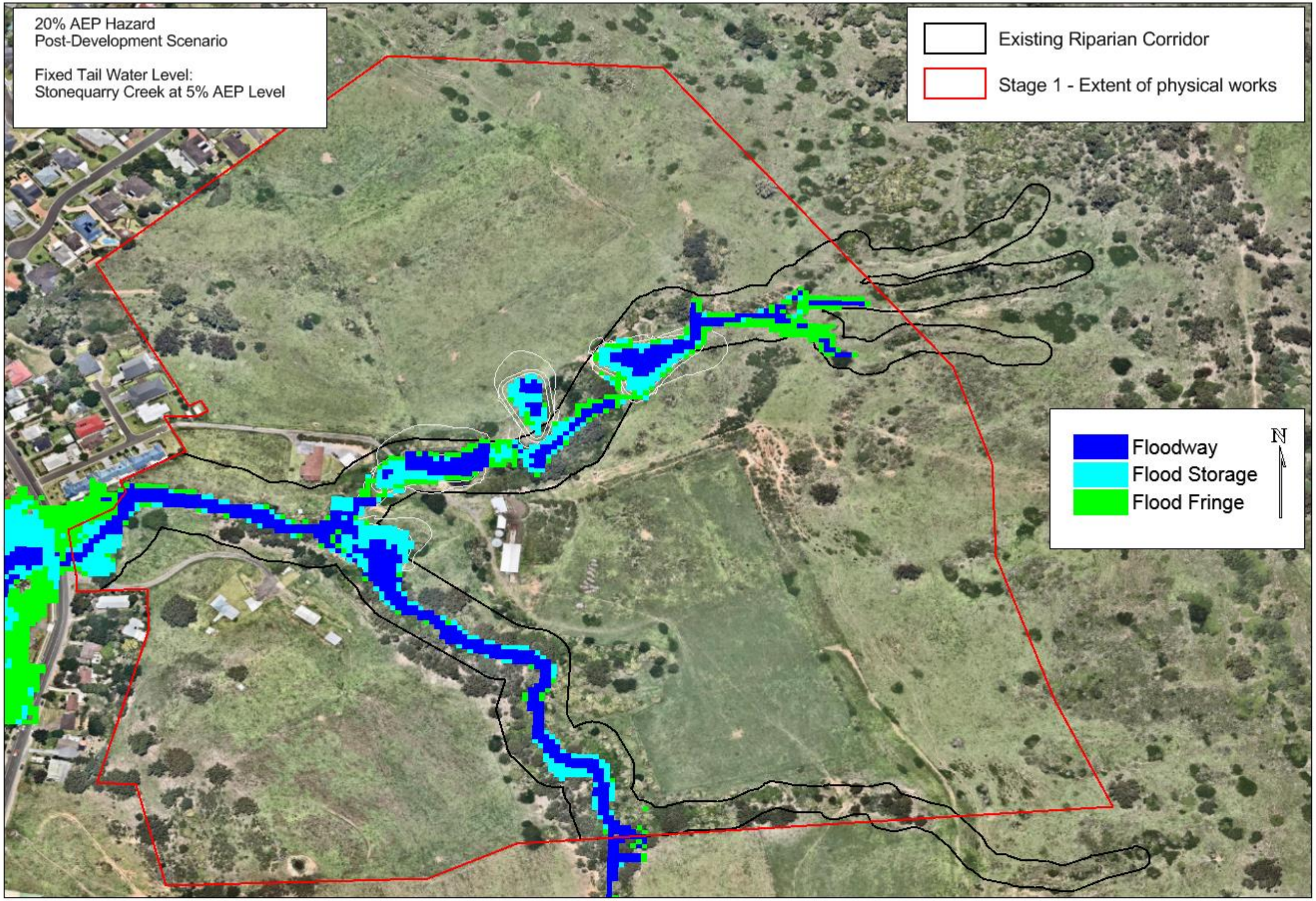


Figure 5-36 20% AEP Hydraulic Categories (Post-Development - FTWL)

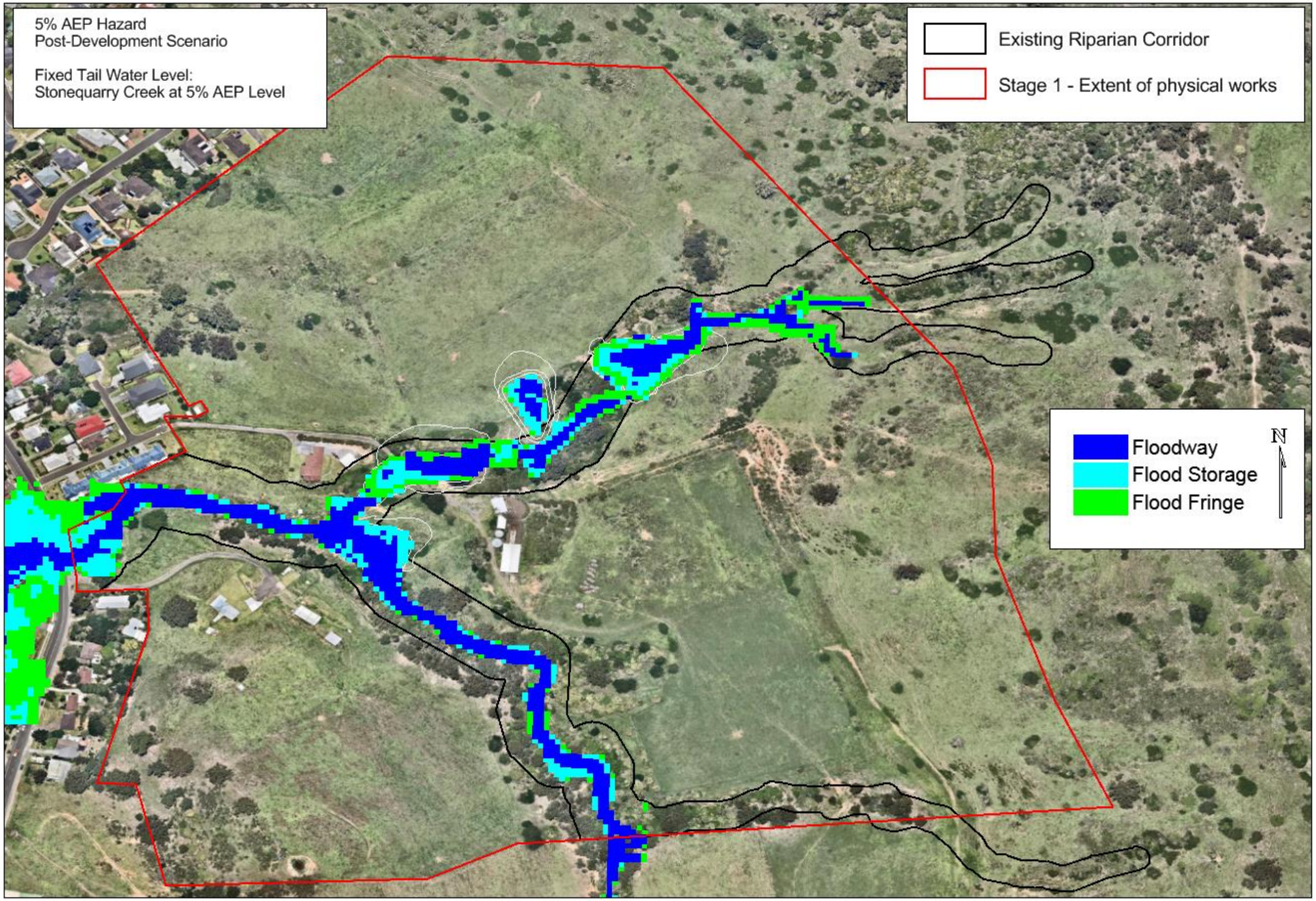


Figure 5-37 5% AEP Hydraulic Categories (Post-Development - FTWL)

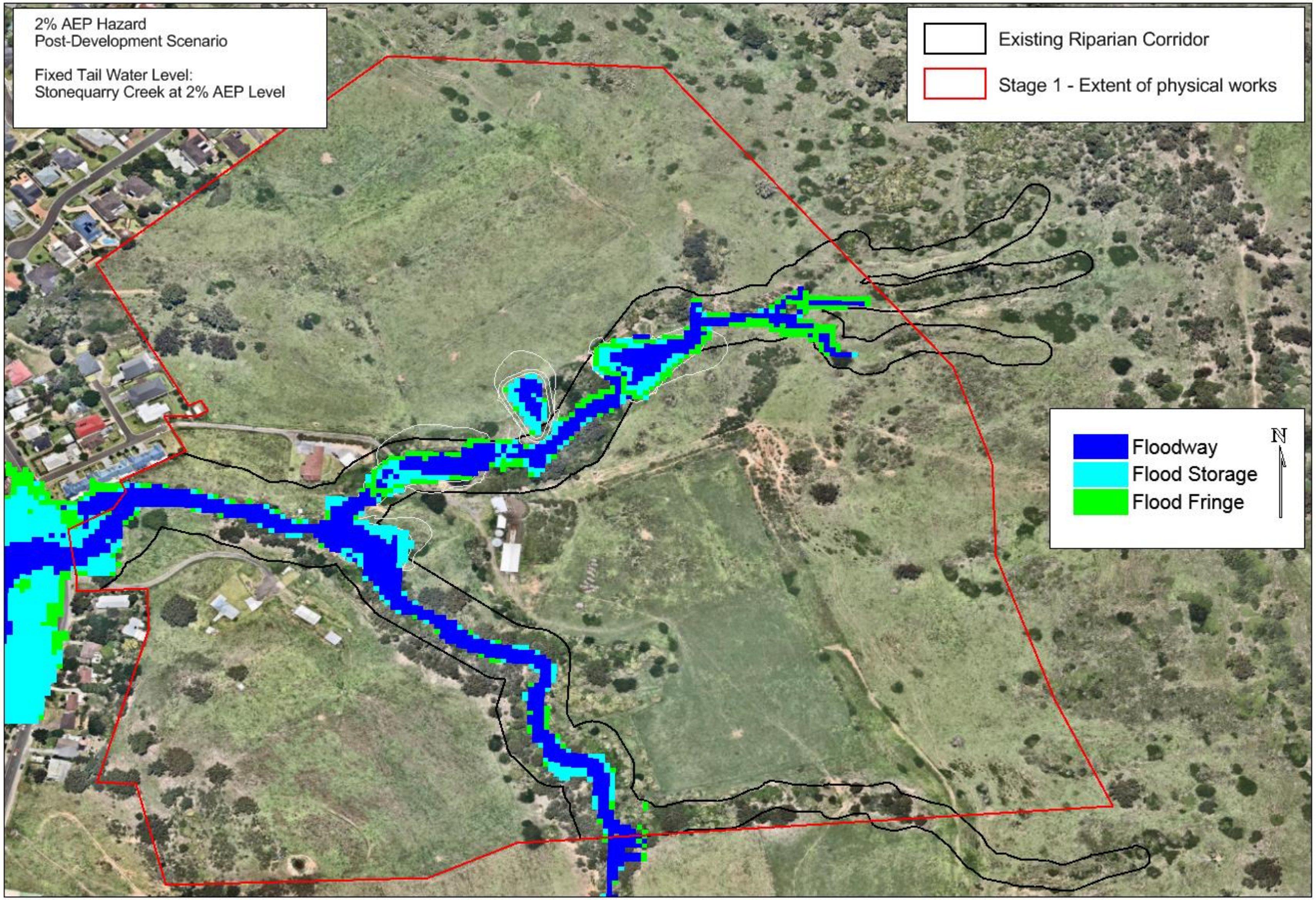


Figure 5-38 2% AEP Hydraulic Categories (Post-Development - FTWL)

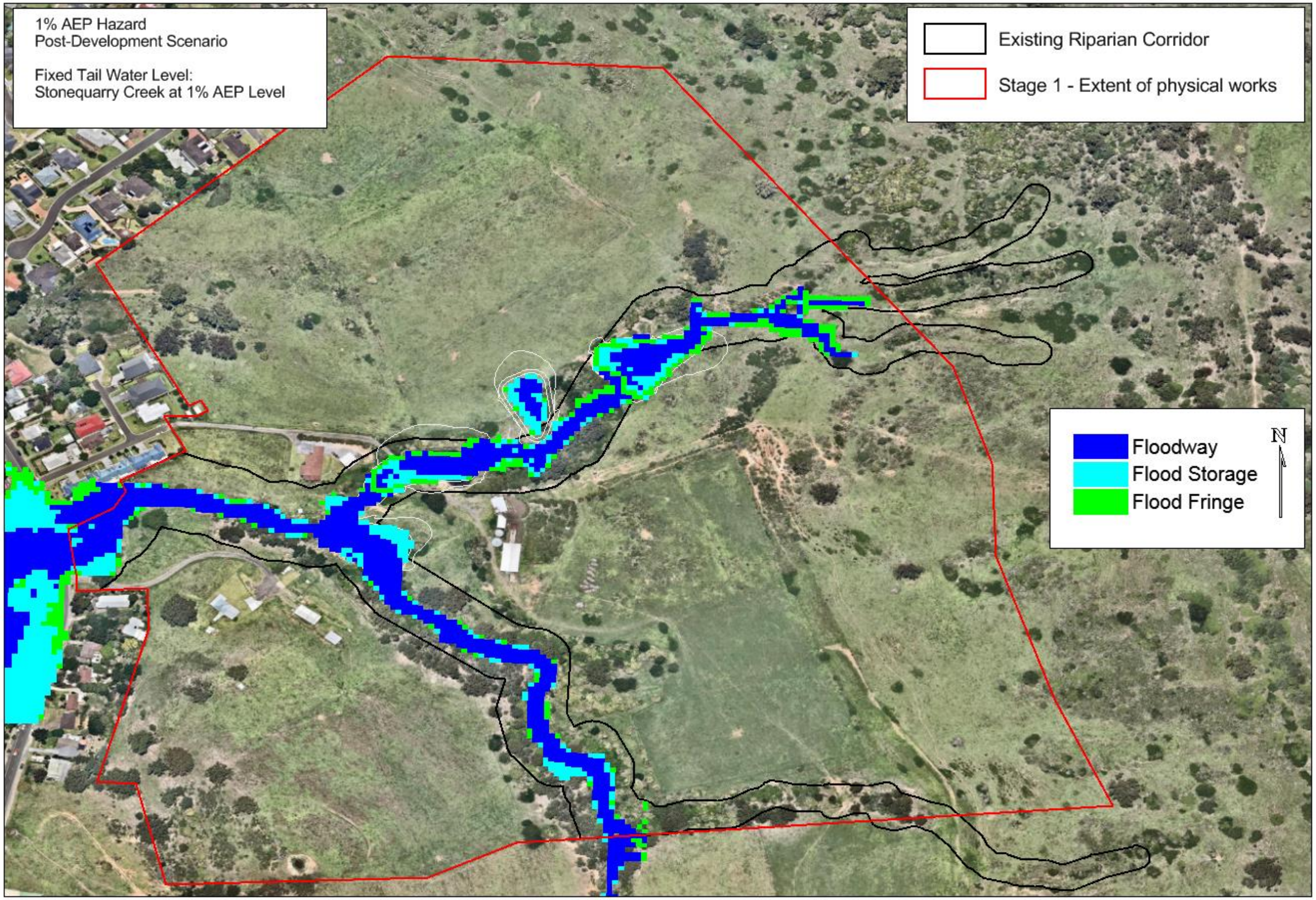


Figure 5-39 1% AEP Hydraulic Categories (Post-Development - FTWL)

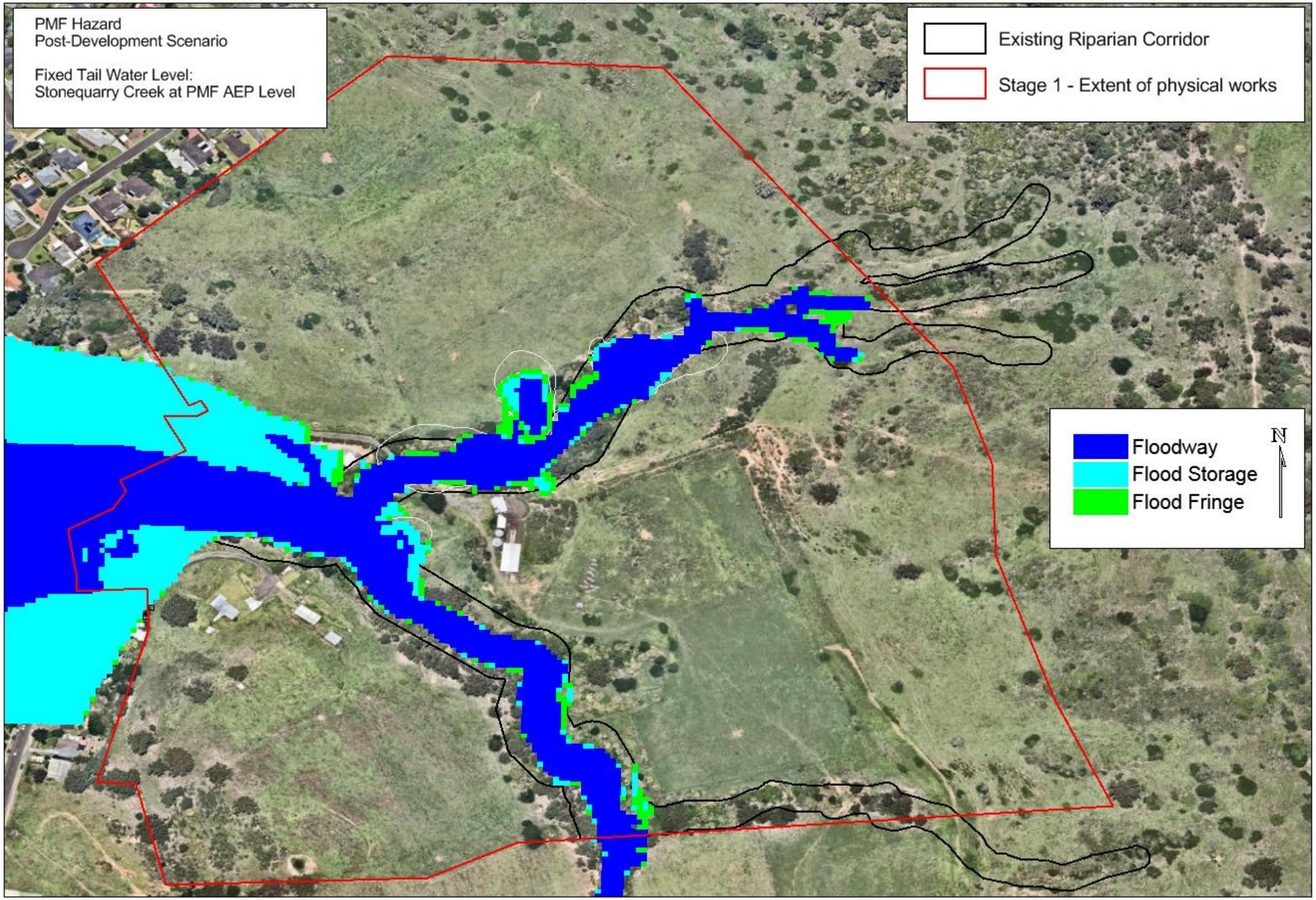


Figure 5-40 PMF Hydraulic Categories (Post-Development - FTWL)

6 Flood Impact and Risk Assessment

The developed SOBEK model was used to assess the flood impacts and flood risk of the proposed development. The proposed development is at the concept stage and as such no detailed design or layout has yet to be established. As such the purpose of this preliminary assessment is to demonstrate the likely impacts that the proposed development will have on the peak flood conditions both within the proposed rezoned area and downstream of this location.

6.1 Flood Impact and Risk

The post-development flood behaviour resulted in only minor changes to the flood behaviour. Flood level impact figures (i.e. difference in peak water levels) are shown in a series of figures. The associated figure numbers are outlined in **Table 6-1.**

Table 6-1 Pre-Development Figure Numbers

AEP	WL Difference
20% AEP	Figure 6.1
5% AEP	Figure 6.2
2% AEP	Figure 6.3
1% AEP	Figure 6.4
PMF	Figure 6.5

The results show that there are no adverse impacts due to the proposed development within the flood plain and that the majority of the changes in flood behaviour were contained within the proposed development. The main increases to flood levels are associated with the basins which have been used to retard and store the floodwaters.

For the 5% and 2% AEP flood events the introduction of the proposed basins reduce the peak depths of flooding both within the study area and for some of the downstream area adjacent to the Menangle Street culverts. This would serve to reduce the flood impact in frequent events in this location by between 2 to 5 cm.

In the 1% AEP event there is almost no change in flood levels downstream of the proposed development. This demonstrates that the proposed basins have been adequately sized to retard the peak flow rates back to the existing conditions. Within the study area there are some localised increases to flood depths due to change in runoff rates, however these are within the main flow paths only and are all less than 5 cm. Levels in and around the basins have changed as flows have been stored in new locations and removed from existing storage points. This is expected due to the required basin arrangement.

For the PMF event it is also evident that the downstream conditions are unchanged due to the proposed development. Within the development area it can be seen that the basins are now overtopped. Roads traversing the major flow paths of Reeves Creek are likely to be impacted during the PMF event. Some changes in peak depths as a result for the proposed basin arrangement are noted upstream in Reeves Creek these locations are within the existing riparian zone.

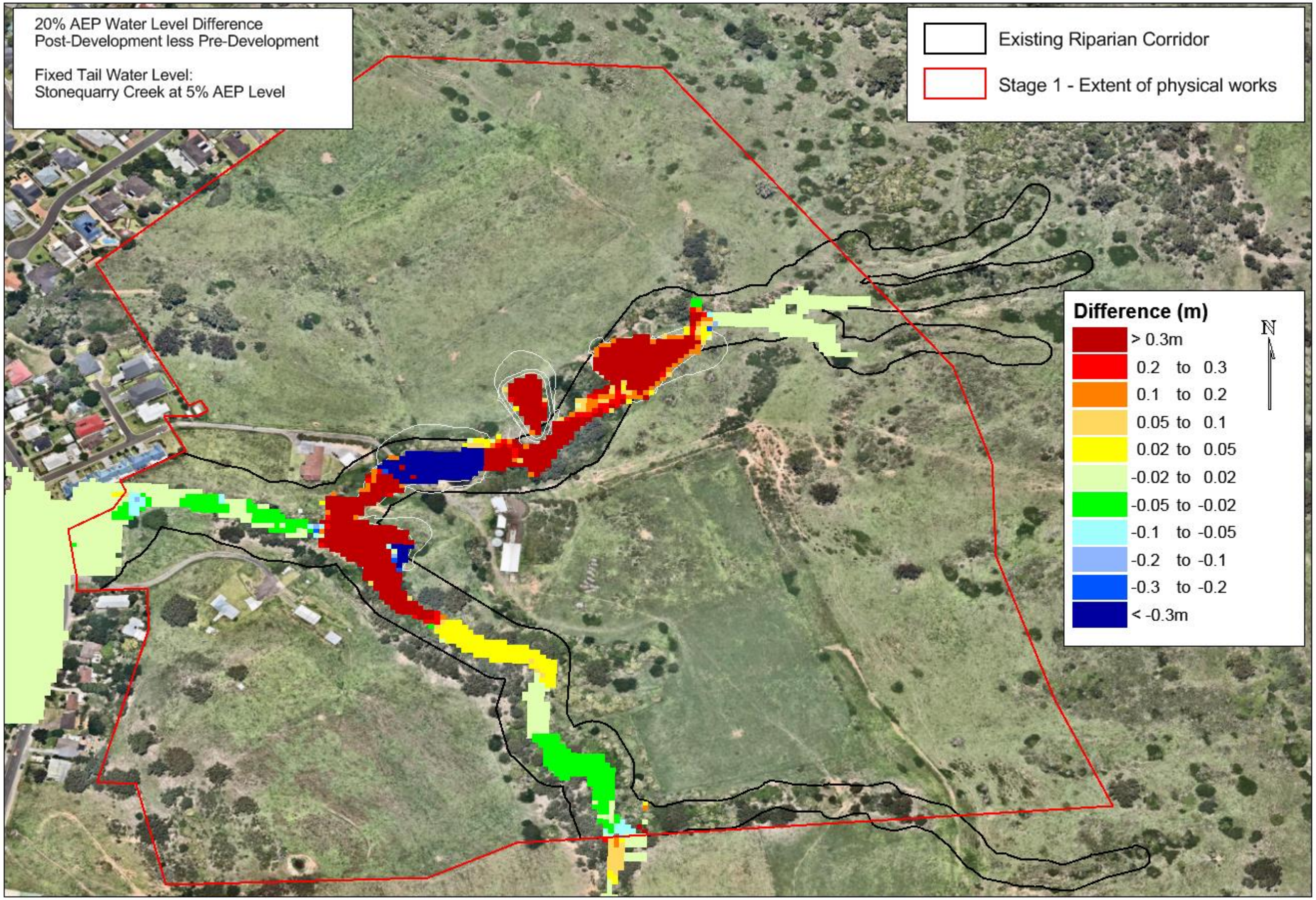


Figure 6-1 20% AEP Water Level Difference (Post-Development less Pre-Development - FTWL)

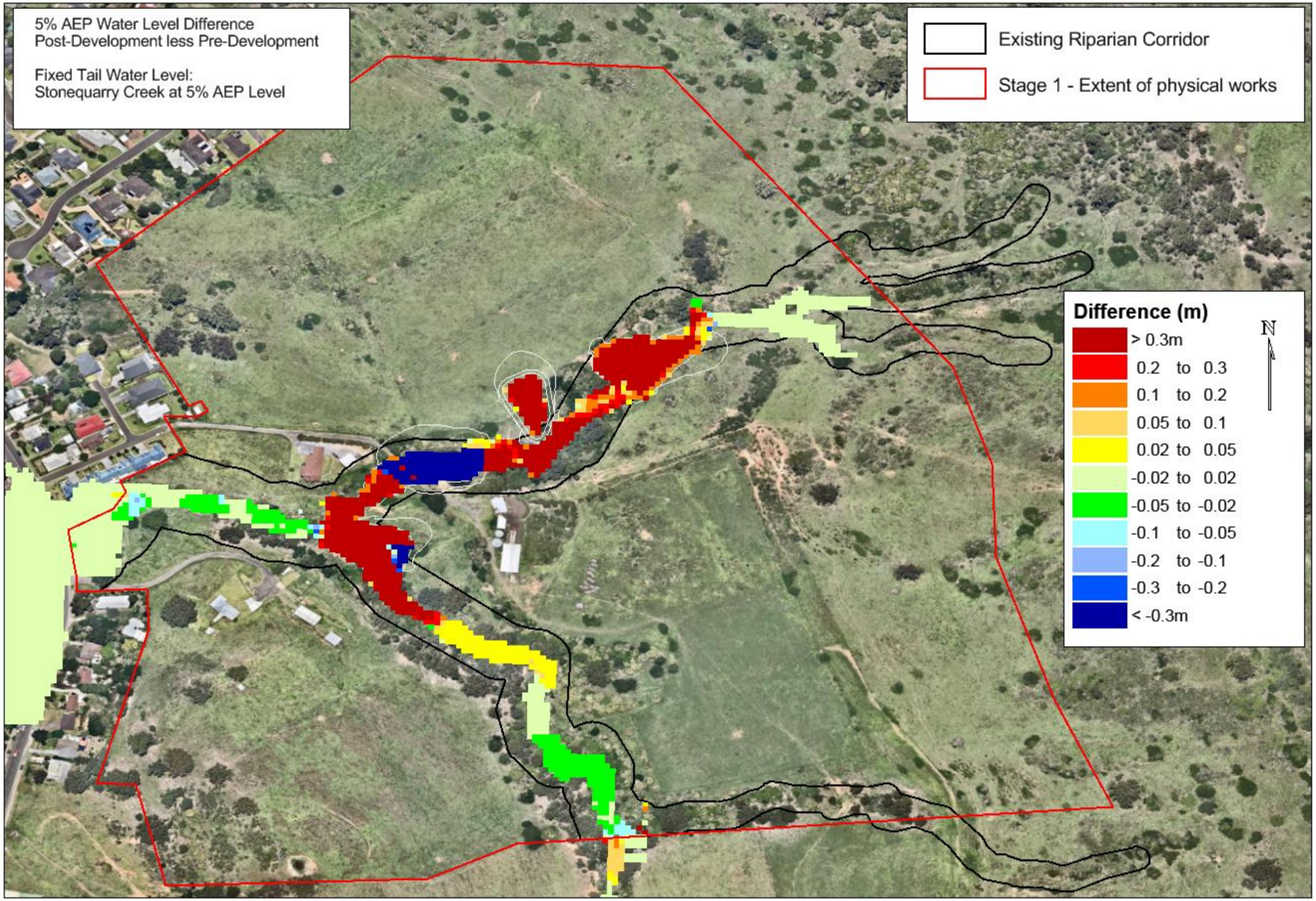


Figure 6-2 5% AEP Water Level Difference (Post-Development less Pre-Development - FTWL)

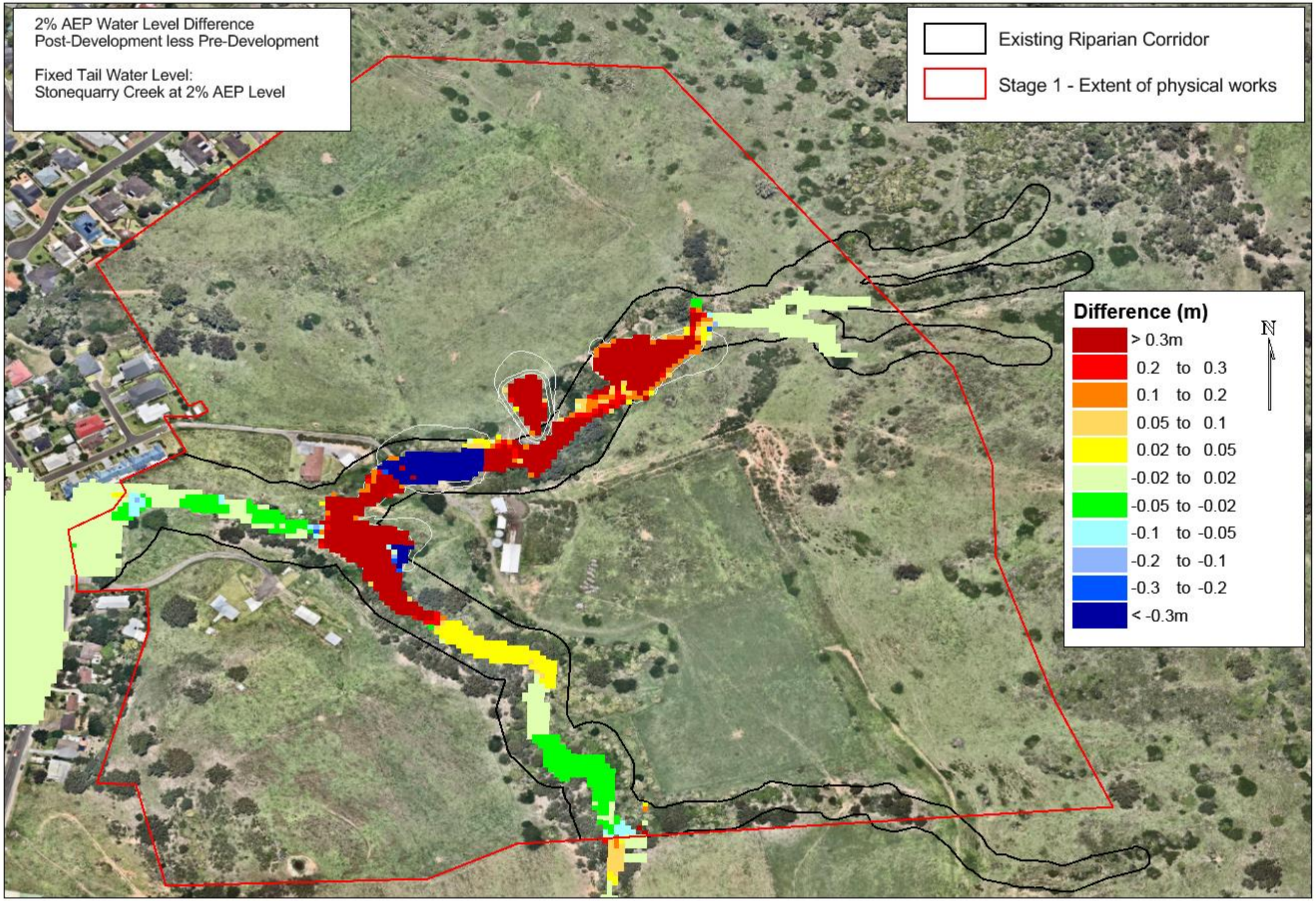


Figure 6-3 2% AEP Water Level Difference (Post-Development less Pre-Development - FTWL)

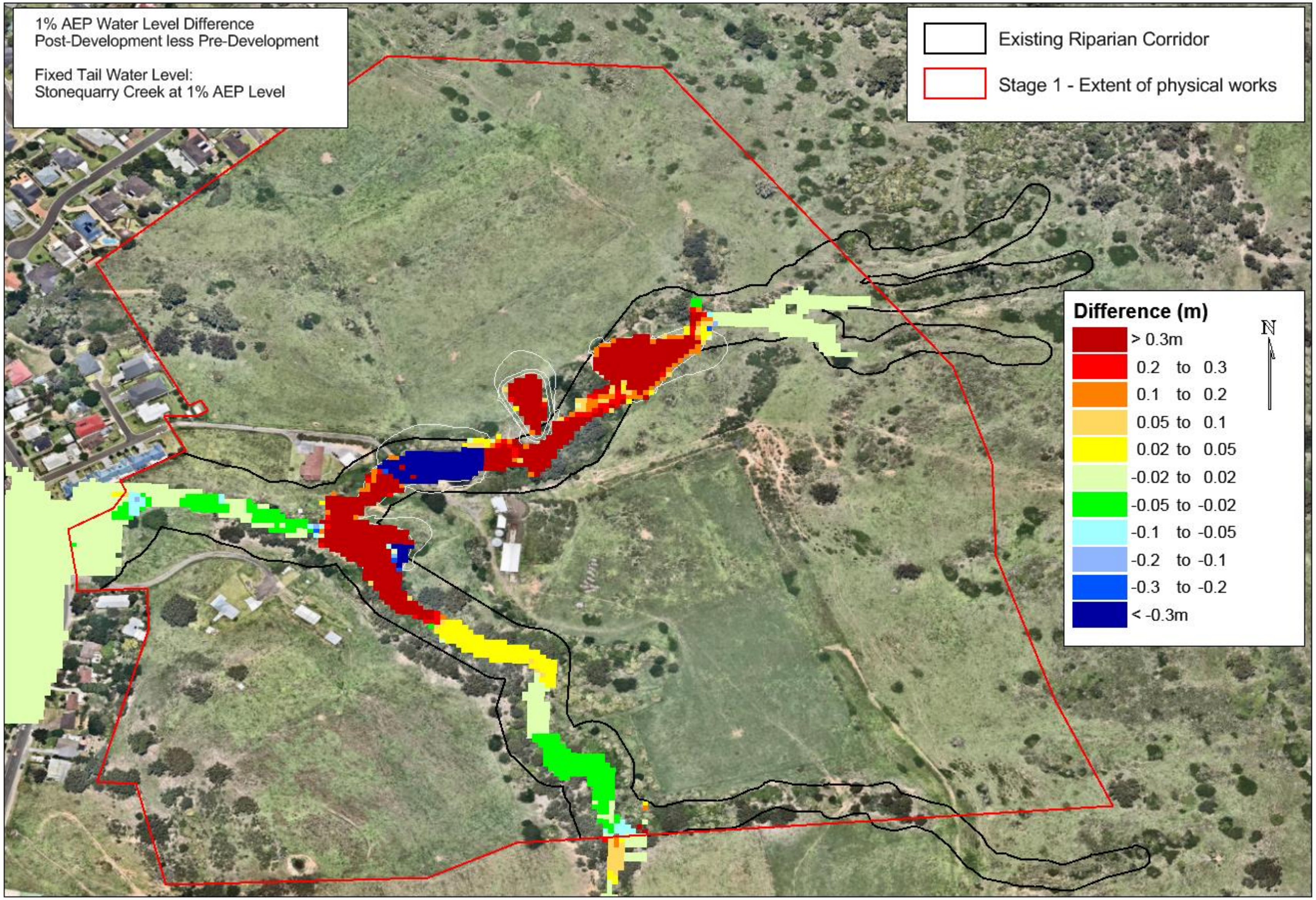


Figure 6-4 1% AEP Water Level Difference (Post-Development less Pre-Development - FTWL)

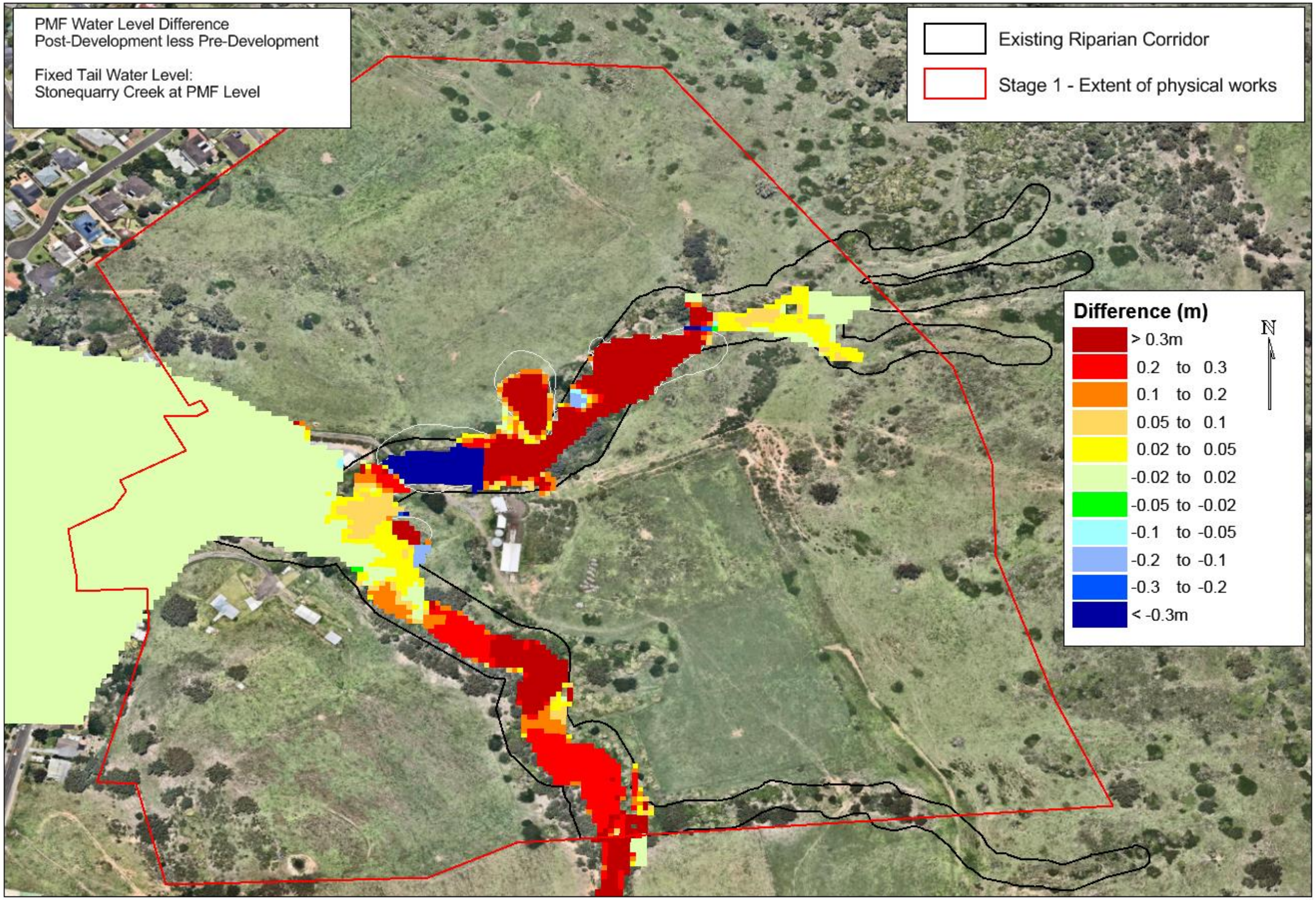


Figure 6-5 PMF Water Level Difference (Post-Development less Pre-Development – FTWL)

6.2 Sensitivity Analysis

A sensitivity analysis was undertaken utilising the 1% AEP flood event to test the validity of adopting a constant water level boundary (**Section 4.5**). The sensitivity analysis involved adopting a stage-discharge boundary (Variable Tail Water Level, VTWL) instead of a constant boundary to assess the potential impacts downstream.

The discharge boundary was set downstream of the Menangle Street culvert and the stage-discharge relationship which was sufficient to define the flows exiting the model near this location. The relationship was developed using a cross section taken at this location and defined using the Manning's Equation. **Figure 6-6** shows the plot of the Q-H relationship adopted.

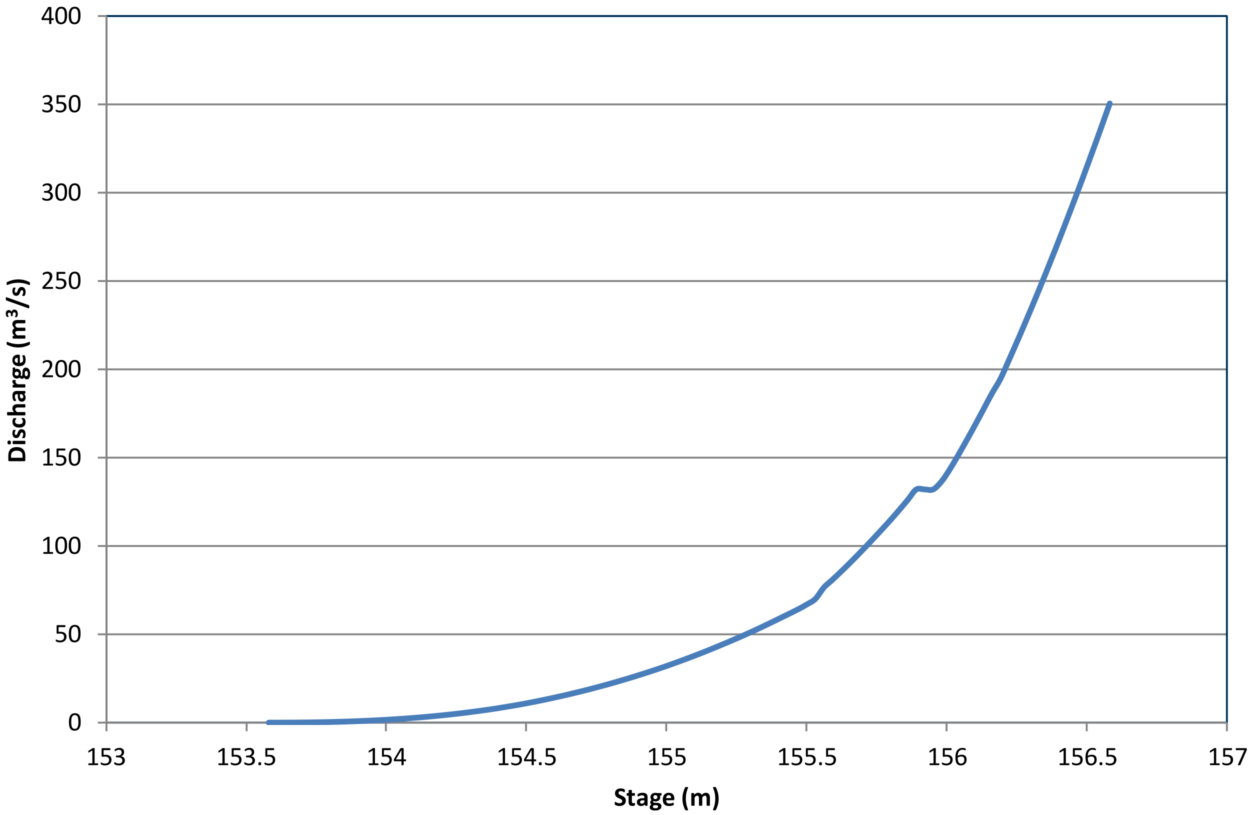


Figure 6-6 Stage Discharge Relationship

The purpose of this assessment was to determine the impacts of the development downstream when the Stonequarry Creek levels were not impacting Reeves Creek. This was done by considering only the local catchment flows from Reeves Creek with no existing flood for Stonequarry Creek. **Figure 6-7** shows the differences in water levels in a 1% AEP. The results show that there is no impact downstream as a result of the proposed development. On this basis, it was decided that no further sensitivity analysis was required for the other flood events.

As the purpose of this study is to demonstrate that the proposed pre to post development does not increase the flood depths off the site it was determined that this shows that for the 1% AEP event there is no change in the flood behaviour downstream of the site and that the retarding basins are functioning adequately up to the 1% AEP event.

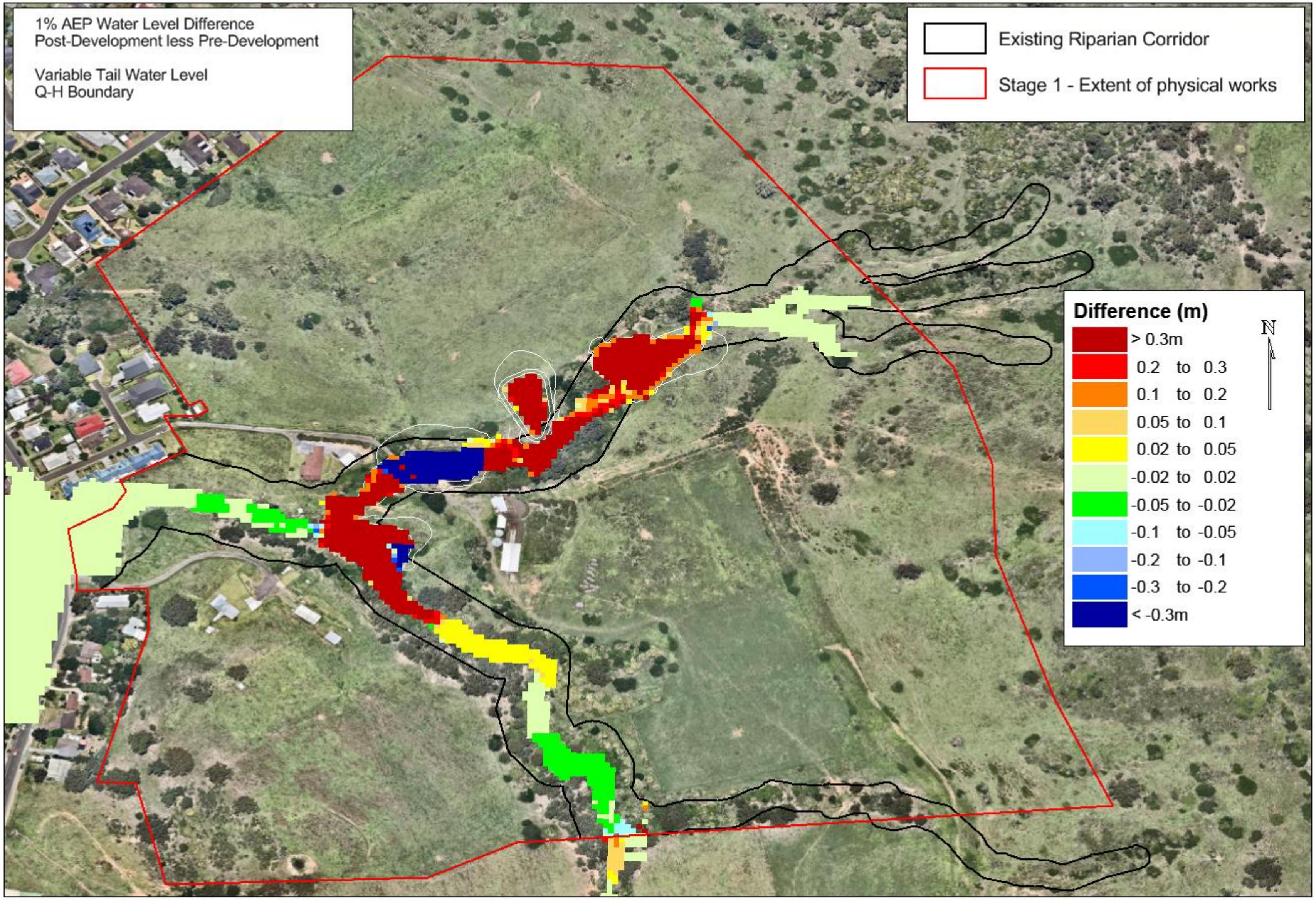


Figure 6-7 1% AEP Water Level Difference (Post-Development less Pre-Development - VTWL)

6.3 Summary and Recommendations

From the preliminary assessment it is evident that proposed concept design has no impact on the peak flood levels observed downstream of the site. Within the study area the main changes to the flood behaviour are the introduction of the series of basins to retard the additional surface water runoff due to the proposed development. This assessment demonstrates that these basins are adequately sized and that the changes to the flood conditions are minor within the study area. Again it is important to note that there is no downstream change to the flood conditions as a result of the proposed development.

7 Emergency Planning and Evacuation

The proposed development is within a low flood risk area. The development is located on the small, stand-alone tributary of Reeves Creek which has flooding largely constrained to the main riparian corridors.

This discussion aims to provide advice for the emergency planning and evacuation requirements that should be developed in detail as part of the detailed design and development process. The detailed Flood Emergency Management Plan (FEMP) is required to be developed in conjunction with the submission of the development approval which will occur following the detailed design. Currently the application is for the rezoning of the area.

7.1 Development Risks

The area to be rezoned and developed is located on the tributaries of Reeves Creek, which experiences a low magnitude and frequency of severe flooding. The proposed rezoning area has been determined to be a mix of floodway and flood fringe in the main riparian corridors. However, the majority of the developable site is not impacted by floodwaters.

The risks associated with the rezoned land include:

- Isolation risks for large events (although the isolation period is not expected to be significant as Reeves Creek is a small catchment). The isolation risk is mitigated by the fact that the development will be connected by roads both to the north and south that will not be inundated. Care should be taken in the final design so that no areas are likely to be isolated.
- Low flood warning times – low warning times for floods can lead to cars being impacted on low lying roads and crossings or some property damage.
- The PMF event for the proposed rezoned area will impact the riparian corridors and any low lying roads. It is unlikely that any properties will be impacted during the PMF event.

7.2 Emergency Planning

Emergency Planning should be developed with consultation with the Wollondilly Council, NSW SES and relevant stakeholders to ensure that a unified and consistent approach is maintained through this area of Picton. The future residents should be made aware of the flooding extents and risks associated with the riparian corridor and flood detention basins.

7.3 Evacuation

For the majority of the proposed area for rezoning no evacuation is required. Access should be possible both north and south from the proposed development via the proposed roads. These access routes are depicted **Figure 7.1**.

Generally evacuation routes are a short distance but there may be very small or no warning for large flood events. Local drainage was also not modelled as part of the model and local conditions such as heavy rain, storms, winds etc that typically occur during time of flood will make evacuation difficult. Multiple safe access routes to areas above the PMF are recommended.

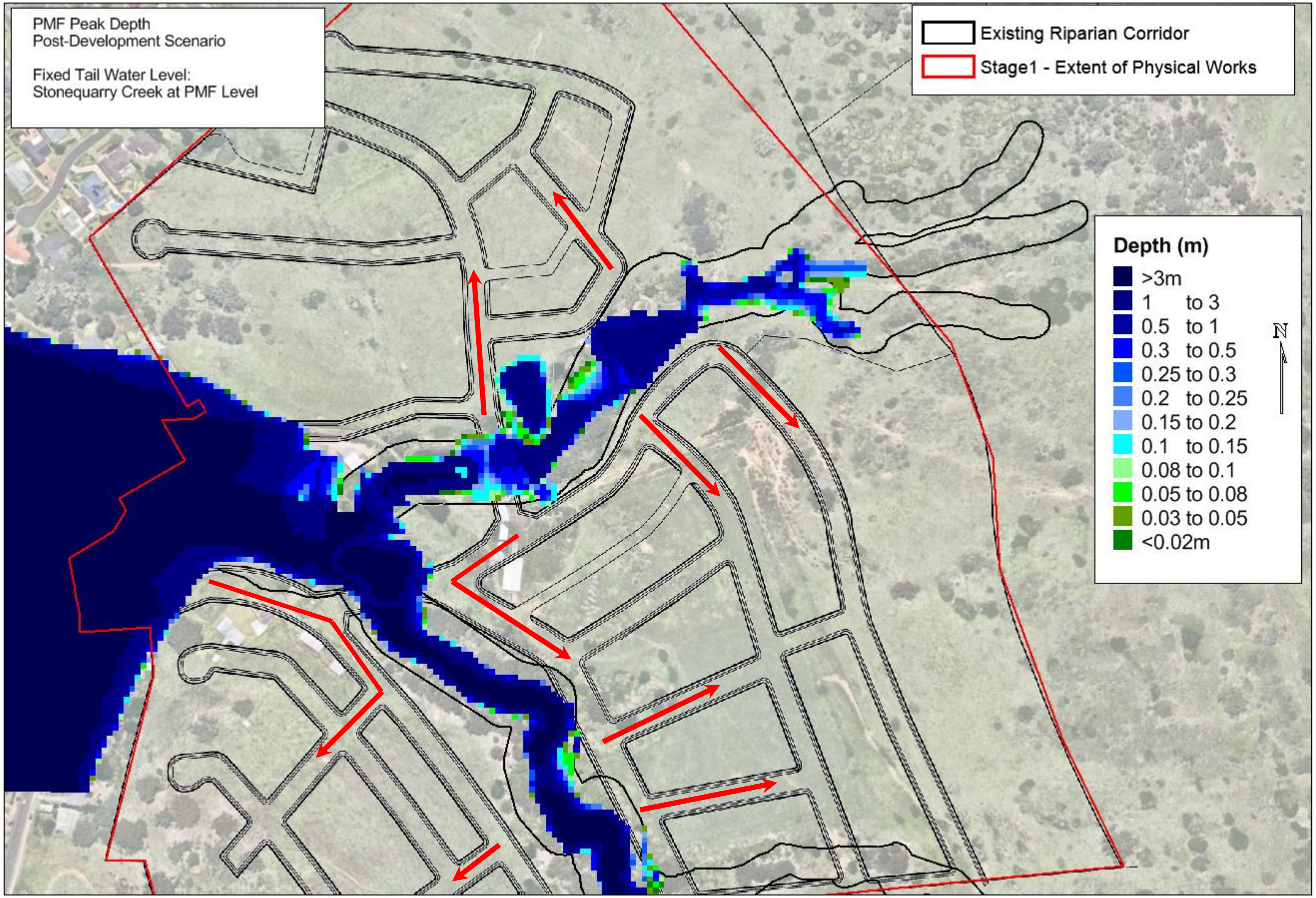


Figure 7-1 Proposed evacuation routes for the development

8 Conclusion

Cardno has completed a flood assessment for Reeves Creek Catchment which includes the land at 1735 Remembrance Drive, 108-114 and 116-118 Menangle Street, Picton. Both the existing conditions and the proposed development have been considered. The study demonstrates the flood behaviour for both existing and proposed design for the 1%, 2%, 5% and 20% Annual Exceedance Probability (AEP) design floods and the Probable Maximum Flood (PMF).

Under current conditions the flooding within the study area is mostly constrained within the Riparian Corridor in all the flood events. The modelling results show that the Menangle Street culvert downstream of the study area has less than a 20% AEP and causing overtopping of Menangle Street in all the flood events. From a floodplain management perspective Council could examine upgrading these culverts to reduce the frequency of overtopping of Menangle Road to improve the roads flood protection. If this culvert is to be upgraded it is recommended that additional flood modelling is undertaken. It should be clearly noted that the proposed development does not change the flood behaviour at Menangle Road as demonstrated by this assessment and as such, any future upgrades can occur independently of the proposed development.

From the preliminary assessment it was evident that proposed concept design has no impact on the peak flood levels observed downstream of the site. Within the study area the main changes to the flood behaviour are the introduction of the series of basins to retard the additional surface water runoff due to the proposed development. This assessment demonstrates that these basins are adequately sized and that the changes to the flood conditions are minor within the study area. Again it is important to note that there is no downstream change to the flood conditions as a result of the proposed development.

9 References

Cardno (2014a), Stonequarry Commercial Development - Hydrology and Hydraulic Assessment, Sydney, Australia.

Cardno (2014b), Stormwater Management Report - Reeves Creek, Sydney, Australia.

Worley Parsons (2011). Stonequarry Creek – 2D Flood Modelling and Climate Change (301015-02259 – RP002), Australia.

Worley Parsons (2009), Picton Lands Flood Impact Assessment (301015-01001-00 – 21.12.09), Australia.

APPENDIX A
FLOOD HAZARD AND HYDRAULIC CATEGORIES

Cardno
Shaping the Future

Flood Hazard

Flood hazard can be defined as the risk to life and limb and damage caused by a flood. Provisional flood hazard is determined through a relationship developed between the depth and velocity of floodwaters as detailed in the Floodplain Development Manual (NSW Government, 2005). High provisional hazard indicates possible danger to personal safety; evacuation by trucks difficult; able-bodied adults would have difficulty in wading to safety; potential for significant structural damage to buildings. Provisional flood hazard has also been prepared for both the pre-development and post-development scenarios.

Hydraulic Category

Hydraulic categorisation of the floodplain is used in the development of a Floodplain Risk Management Plan to assist in defining primary flow paths and flood storage areas. The Floodplain Development Manual (NSW Government, 2005) defines flood prone land as one of the following three hydraulic categories:

Floodway

Areas that convey a significant portion of the flow. These are areas that, even if partially blocked, would cause a significant increase in flood levels or a significant redistribution of flood flows, which may adversely affect other areas.

Flood Storage

Areas that are important in the temporary storage of the floodwater during the passage of the flood. If these areas are substantially removed by levees or fill, there would be resulting elevated water levels and/or elevated discharges. Flood Storage areas, if completely blocked would cause peak flood levels to increase by 0.1m and/or would cause the peak discharge to increase by more than 10%.

Flood Fringe

Remaining area of flood prone land, after Floodway and Flood Storage areas have been defined. Blockage or filling of this area will not have any significant effect on the flood pattern or flood levels.

APPENDIX B

EXTRACT FROM THE STONEQUARRY CREEK SOBEK MODEL DEVELOPMENT

1.1 Reference Documents

The following documents have been reviewed as part of this assessment:

- Wollondilly Development Control Plan 2011 (DCP)
- Wollondilly Local Environment Plan 2011 (LEP)
- Stonequarry Creek – 2D Flood Modelling and Climate Change (301015-02259 – RP002) – Worley Parsons, 2011
- Picton Lands Flood Impact Assessment (301015-01001-00 – 21.12.09), Worley Parsons, 2009.
- Plans showing bridge over Stonequarry Creek at Picton, Argyle Street – Department of Main Roads, NSW – Plan 0002 496 BC 0294 – 1979.
- Managing Urban Stormwater: Environmental targets, Department of Environment and Climate Change, 2007.

2 Hydraulic Model Development

This section outlines the model development for the Stonequarry Creek area. An existing RMA2 model that was created by Worley Parsons and is documented in the report *Stonequarry Creek – 2D Flood Modelling and Climate Change* (Worley Parsons, 2011). As part of this current assessment, this model has been updated to SOBEK which is a more widely adopted modelling package. The SOBEK model has been validated and compared to the existing RMA2 model to ensure that the results are consistent.

2.1 Existing RMA2 Model

The existing RMA2 model representing the Stonequarry Creek floodplain was established by Worley Parsons for the Wollondilly Shire Council. The details of this model are described in the *Stonequarry Creek – 2D Flood Modelling and Climate Change* report (Worley Parsons, 2011). Figure 2-1 is extracted from this report and outlines the original model area (as defined by the "1989" HEC2 modelling) and the current model extent as defined by Worley Parsons.

The model includes the tributaries from the upstream area feeding Stonequarry Creek and terminates downstream of the Railway Bridge. The model was run using derived inflows from a hydrological RAFTS model developed by Worley Parsons as line boundary inputs. Local drainage rainfall and runoff was not included in the model as this is a very small component of the overall catchment flows. The local drainage network has not been developed as part of this assessment.

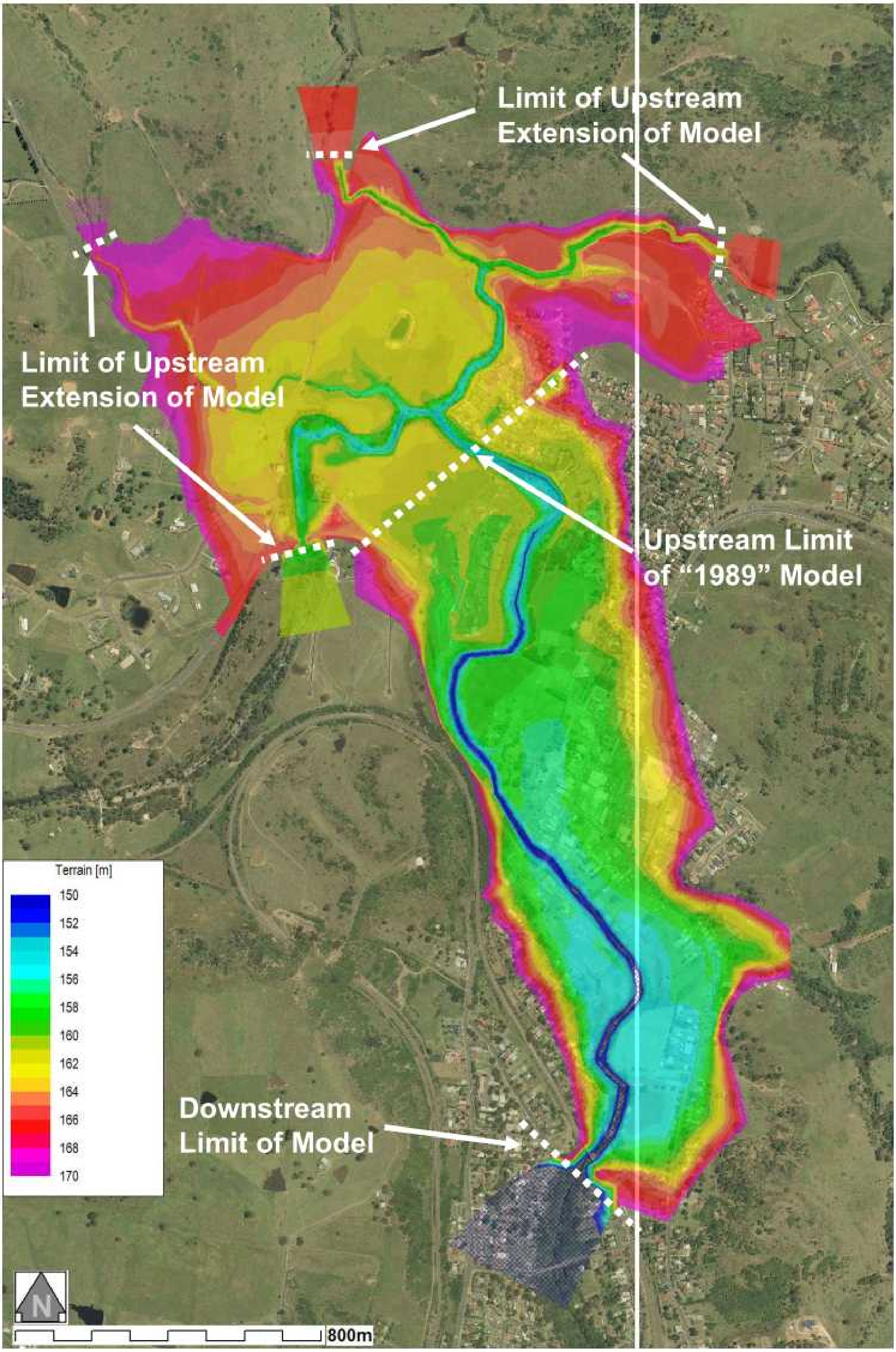


Figure 2-1 Current model for Stonequarry Creek (Worley Parsons, 2011)

2.3 RMA2 Data

In order to develop the SOBEK model and to verify the model behaviour Worley Parsons has supplied the following information:

- Hydrograph inflows at the “1989 model boundary” for the 5%, 2% and 1% Annual Exceedence Probability (AEP) and the PMF events.
- Downstream boundary conditions in the form of a Height – Discharge table.
- Images of the roughness used to define the floodplain.
- No topography was supplied.
- Details of the implementation of structures within the hydraulic model.
- Hydrologic model (RAFTS).

2.3.1 Upstream Boundary Conditions

The upstream boundary conditions extracted from the RMA2 model have been supplied by Worley Parsons in the form of a time – discharge boundary at the "1989" model boundary (see Figure 2-1). The hydrographs supplied for each event are summarised in Figure 2-3. The peak flow rates at the upstream boundary of the model are stated on the figure. These hydrographs have been used in the developed SOBEK model.

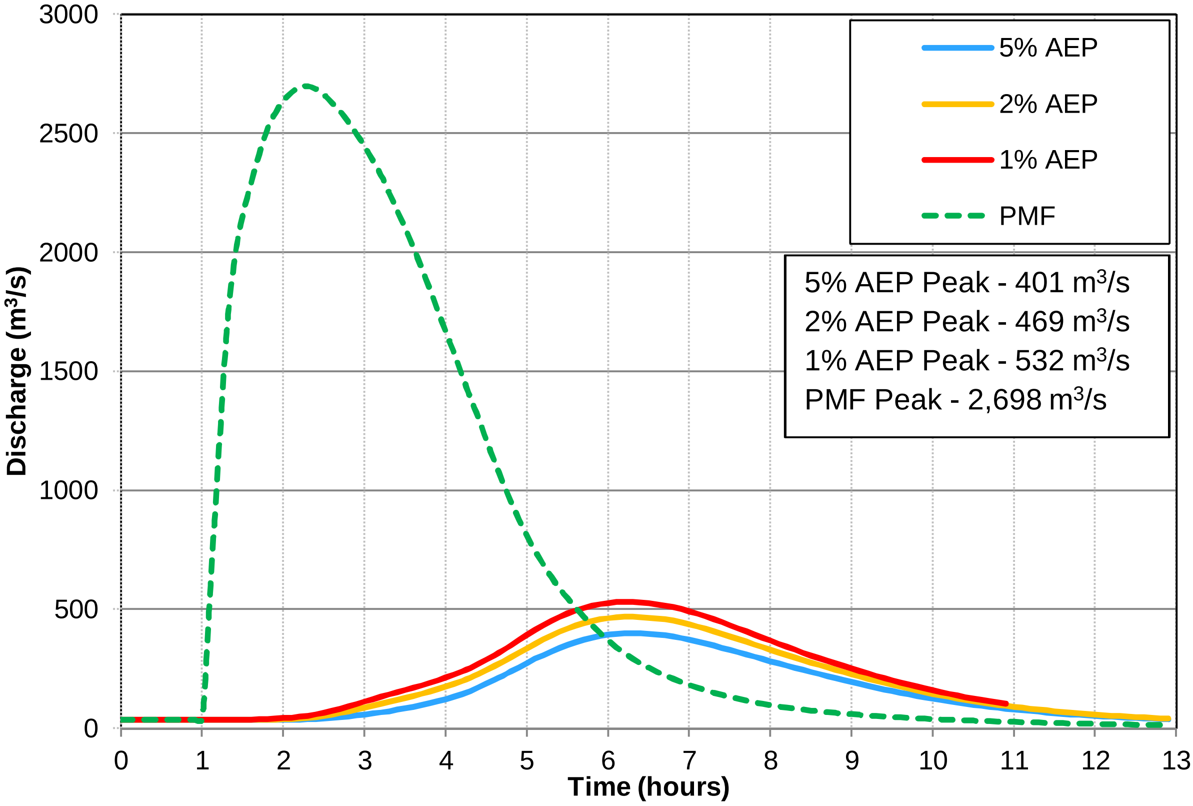


Figure 2-3 Inflow hydrographs supplied from Worley Parsons

2.3.2 Downstream Boundary Conditions

The downstream boundary condition is controlled by a stage-discharge relationship in the RMA2 model. This relationship is shown in Figure 2-4. The relationship extends up to a peak discharge rate of 6,000 m^3/s, which is suitable up to the PMF event peak flow rate. The discharge boundary was set downstream of the Railway Bridge where the flows are constricted. Worley Parsons did not specify the exact location of the flow boundary, however the stage-discharge relationship is sufficient to define the flows exiting the model near this location.

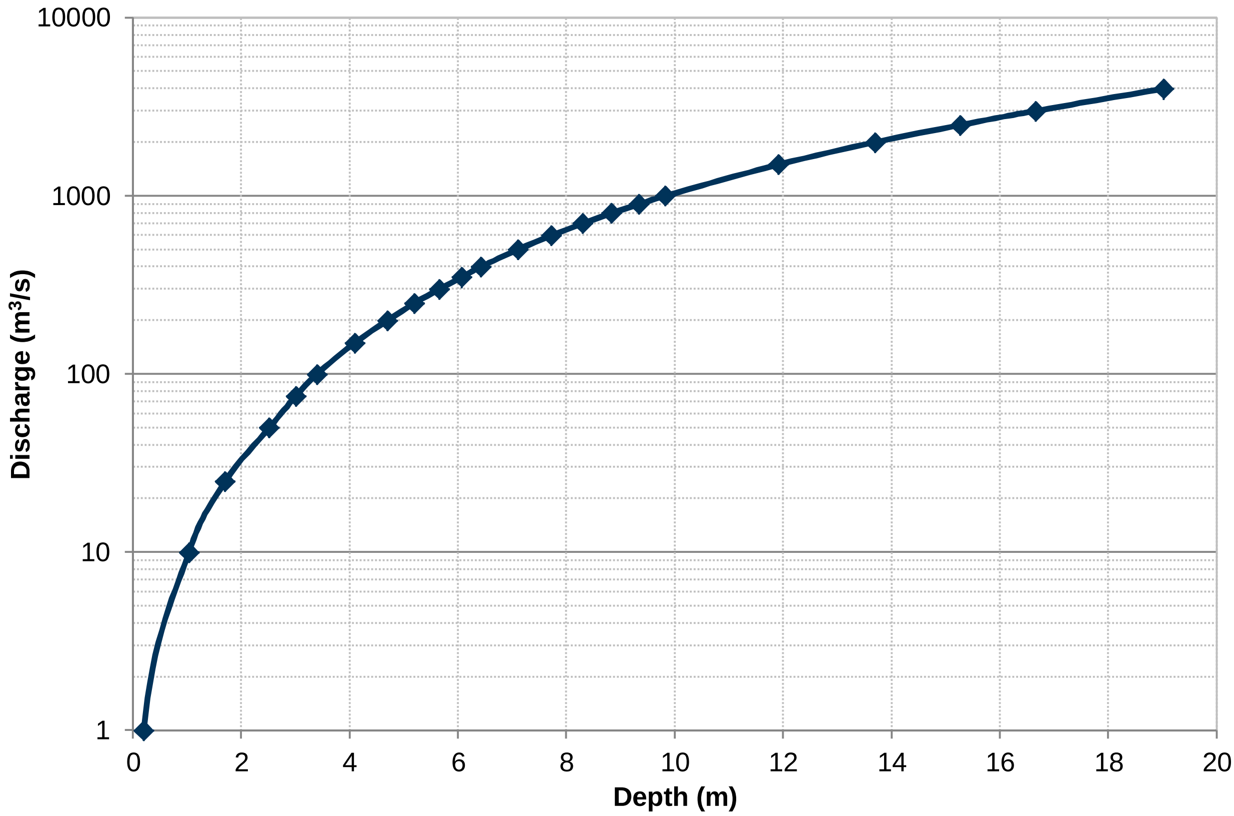


Figure 2-4 Stage – Discharge relationship supplied by Worley Parsons

2.3.3 Roughness

The roughness parameters were supplied to Cardno via three PDF maps of the floodplain. The roughness parameters included in these images are shown in Table 2-1.

Table 2-1 Roughness parameters from the RMA2 model

Description	Roughness (Manning's 'n')
Moderately Vegetated Channel	0.040
Densely Vegetated Channel	0.060
Grassed Floodplain	0.040
Floodplain with Sparse Trees	0.060
Floodplain with Dense Trees	0.075
Roads	0.03
Industrial	0.065
Residential	0.055
Buildings	Blocked

2.3.4 Structures

The main structures included within the RMA2 model area include the Argyle Street Bridge and the Railway Bridge. From discussions with Worley Parsons the RMA2 model incorporated these structures by opening the flow channel, representing the bridge abutments in the topography and setting the roughness at 0.055. The bridge decking was not included in the modelling.

2.4 Model Development

Cardno have developed a version of the Stonequarry Creek model which extends from the “1989” model boundary past the Railway Bridge. This model covers the area as shown in the topographic grid shown in Figure 2-5. The model was not required to extent as far north as the RMA2 model due to the location of the proposed development.

The model was upgraded to SOBEK to facilitate a more detailed assessment of the proposed development. SOBEK can explicitly represent structures as 1D elements that automatically link to the 2D terrain information. Details of the benefits of using SOBEK are discussed in Section **Error! Reference source not found.**.

2.4.1 Topography

The topography was developed based on LiDAR information supplied by Wollondilly Shire Council. The LiDAR was obtained in 2012 by AAM. The data is stated to have a nominal vertical accuracy of +/- 0.15m on hard surfaces.

The grid was created using a 4m x 4m grid cell resolution which was sufficient to delineate the channel cross sections within the 2D domain as the typical main channel is approximately 30 to 60m in width (7 to 14 grid cells). The main channel was modified to reflect the cross sections as per the HEC2 modelling. The LiDAR would not have been able to pick up the bottom of the channel due to the standing water level in the creek and the dense vegetation, and this was manually adjusted in the topography.

The Argyle Street bridge was blocked at the bridge deck height as a 1D structure was setup as per the plans obtained from Wollondilly Shire Council (see Appendix A). The structure implementation is detailed in Section 2.4.3.

The buildings were blocked out using the roof height information as provided by Wollondilly Shire Council. This data set involved points sampled from the rooftops of the buildings. Care was taken to ensure that all flow paths between buildings were maintained to ensure the floodplain connectivity was not altered. This was completed using detailed up to date aerial photography (Nearmap), Google Street View and field photos.

The developed topography is shown in Figure 2-5.

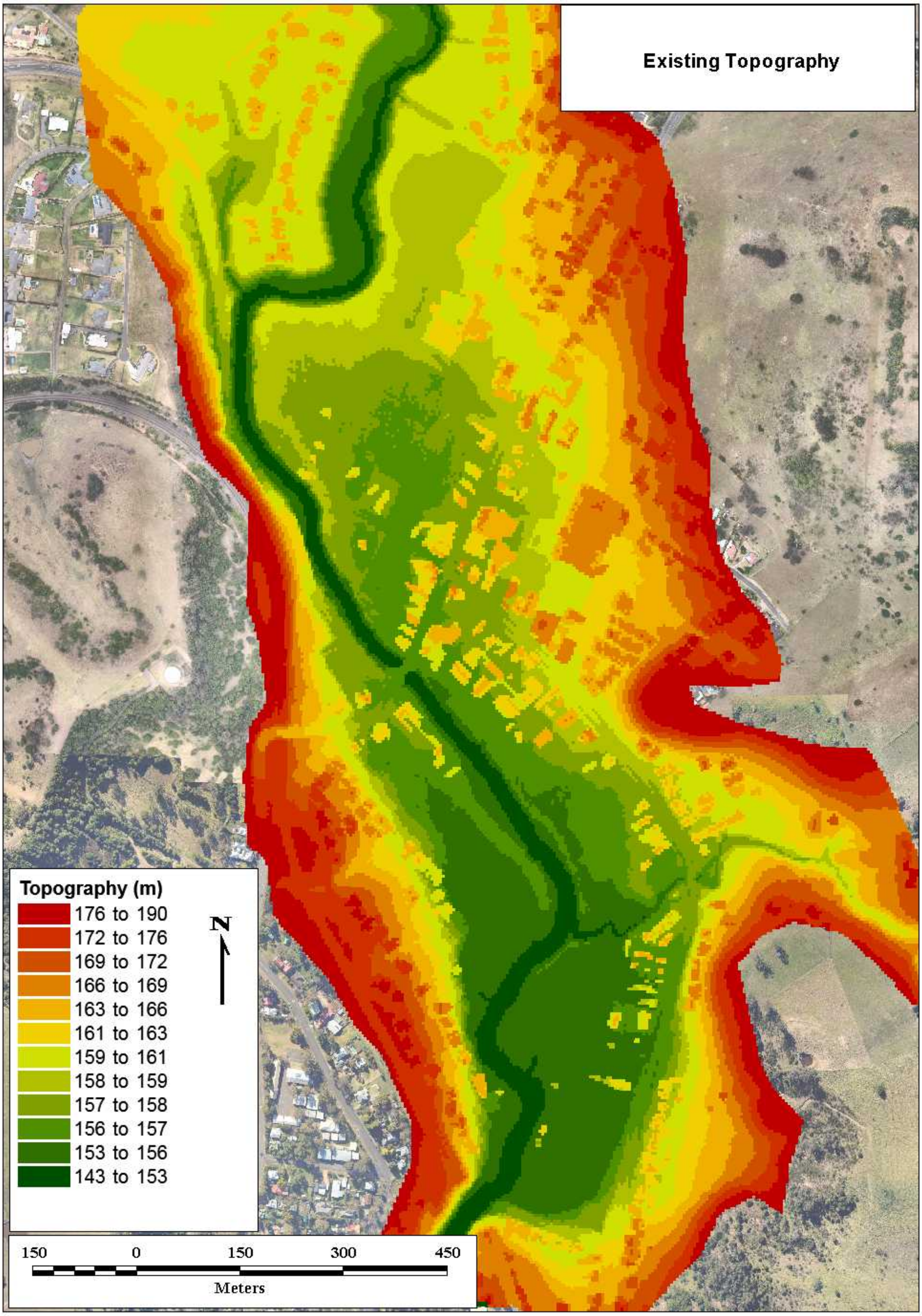


Figure 2-5 Proposed topography and model extent for the revised SOBEK model

2.4.2 Roughness

The roughness was represented across the floodplain using Manning's 'n' which is consistent with the Worley Parsons modelling. Some minor adjustments were made to the roughness values to best reflect the catchment roughness in SOBEK.

The roughness parameters used in each of the models is shown in Table 2-2. The only changes to the roughness values were:

- Roads - the roughness lowered from 0.03 to 0.018 as this is more representative of a typical roughness value for roads, and
- Residential buildings - Worley Parsons blocked the structures completely and did not need a roughness whereas Cardno required a roughness as the buildings were only partially blocked.

The roughness grid is shown in Figure 2-6. The developed roughness matches the Worley Parsons roughness layers closely.

Table 2-2 Roughness parameters for the Cardno SOBEK model

Description	Roughness (Manning's 'n')	
	Worley Parsons	Cardno
Moderately Vegetated Channel	0.040	0.040
Densely Vegetated Channel	0.060	0.060
Grassed Floodplain	0.040	0.040
Floodplain with Sparse Trees	0.060	0.060
Floodplain with Dense Trees	0.075	0.075
Roads	0.03	0.018
Industrial	0.065	0.065
Residential Area	0.055	0.055
Buildings	Blocked	0.2

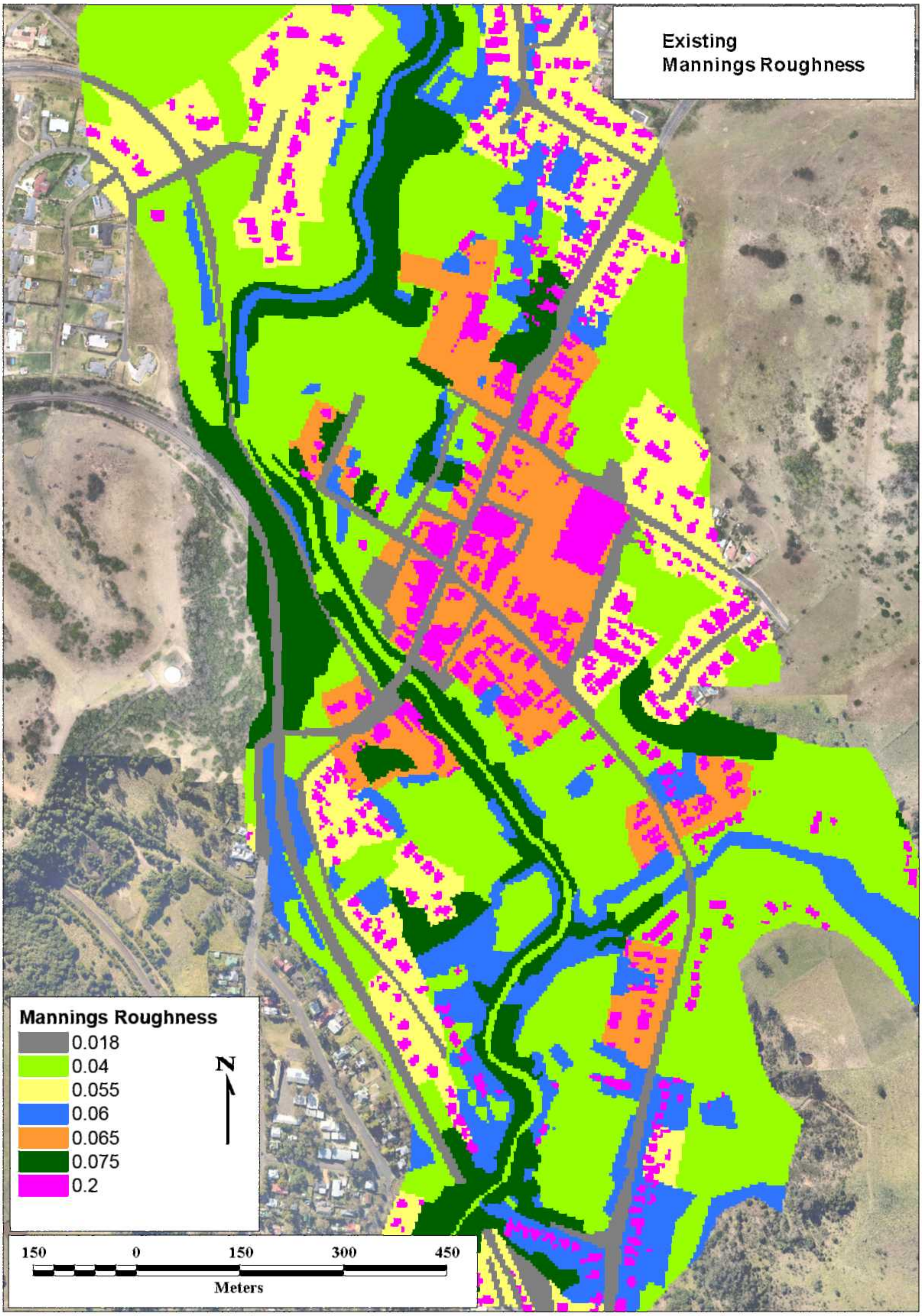


Figure 2-6 Manning's roughness for the Stonequarry Creek model in SOBEK

2.4.3 Structures

A significant change from the RMA2 model was the specific inclusion of the Argyle Street Bridge. SOBEK has the capacity to incorporate the bridge as a 1D element that will interact with the 2D grid. This allows the barrier of the bridge to be adequately represented as the bridge structure itself is a significant blockage to flow until floodwaters overtop the structure. Implicitly modelling the bridge also allows for the entrance and exit losses of the structure to be included.

The RMA2 model did not include the Argyle Street Bridge but included the abutments and increased the surface roughness to capture some of the losses that are likely to occur through this location.

The Argyle Street bridge was modelled using structural plans obtained from Wollondilly Shire Council (see Appendix A). The head loss across the structure was checked (detailed in the model validation section) and this matched the HEC2 losses.

2.4.4 Boundary Conditions

The boundary conditions were setup based on the information that was made available from Worley Parsons. For the upstream boundary this was an inflow hydrograph for the 5%, 2%, 1% AEP and PMF events. Only the critical duration was supplied to Cardno. The inflow hydrographs are shown in Section 2.3.1 and Figure 2-3.

The downstream boundary condition was a stage-discharge boundary. Again this was supplied by Worley Parsons and the details of this relationship are summarised in Section 2.3.2 and in Figure 2-4. The channel at this location is a well-defined cross section and checks were made using the Manning’s Equation against the supplied stage discharge boundary. The boundary supplied was consistent with the cross checks.

2.5 Model Validation

The SOBEK model has comprehensively assessed against the RMA2 model to determine if the model was adequately matching and representing the Stonequarry Creek floodplain. The detailed assessment and discussion is shown in Appendix C.

Ultimately without full access to the RMA2 model it is not possible to determine the exact reasons for the differences between the model results. Some of the potential reasons are discussed and outlined. Overall, the SOBEK model is producing conservative results compared to the RMA2 model and is fit for purpose for the assessment of the proposed development.

The primary use of the developed SOBEK model is to assess the impacts of the floodplain from the proposed development and as such the model is fit for purpose.

2.6 Establishing Existing Conditions

The peak depths derived from the SOBEK model are shown in Figure 2-7 to Figure 2-10. These peak flood depths form the basis for the existing conditions for the development assessment.

The floodplain at Picton is constrained at the southern end of the catchment through the old railway bridge and the constrained floodplain downstream of this location. This has the effect of causing the Picton floodplain to fill as temporary storage for flood events. This is particularly evident in the 1% AEP and PMF events where the peak depth increase from 2 metres deep during the 1% AEP event up to over 8 metres deep in the PMF event (at Argyle Street near the Argyle Street Bridge).

The peak velocities have been shown for the 1% AEP and PMF events in Figure 2-11 and Figure 2-12. The peak velocities through the floodplain are low at less than 1 m/s for event up to the 1% AEP flood event, however for the PMF event the velocities increased to between 2 to 3 m/s in most areas across the floodplain.

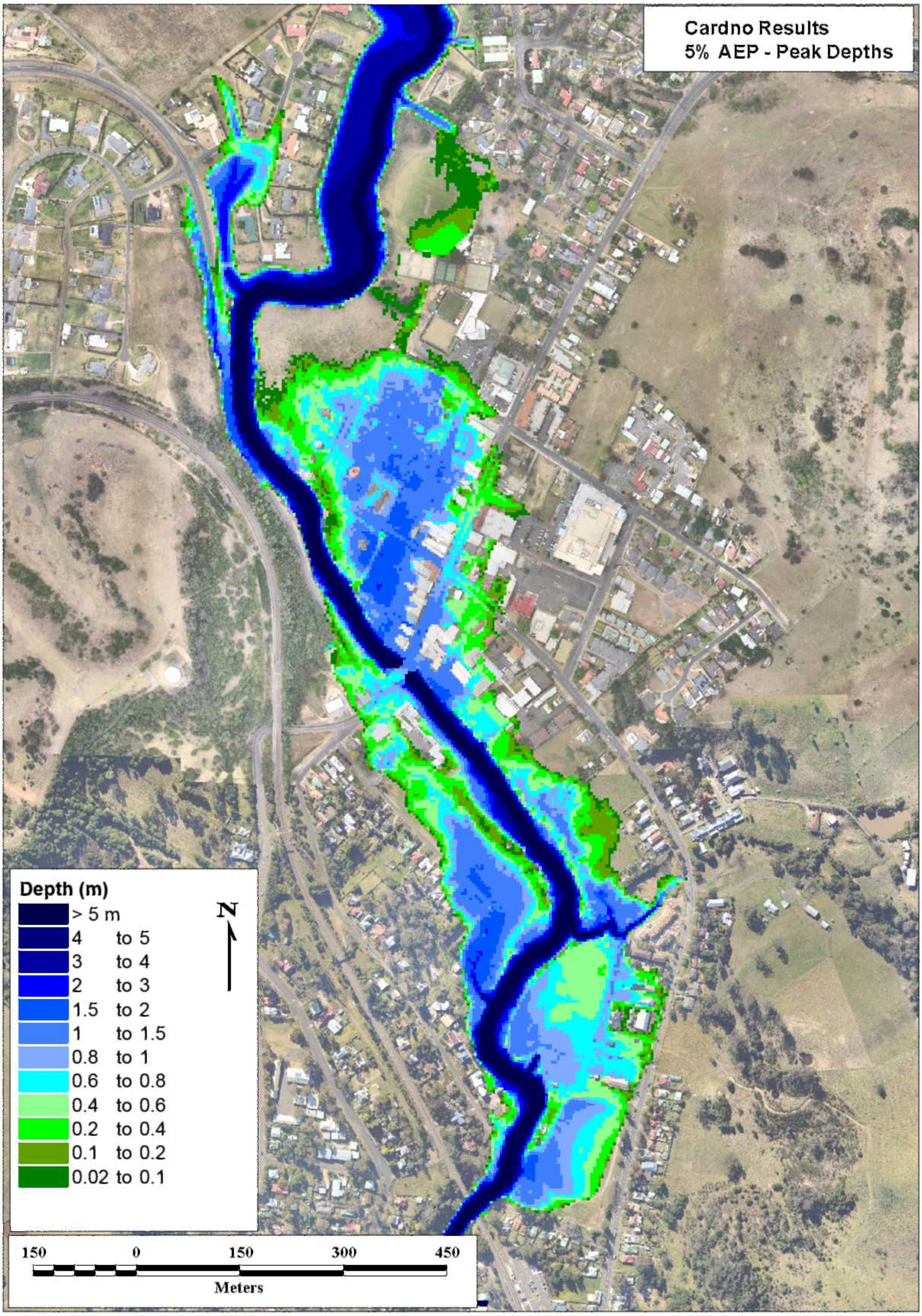


Figure 2-7 5% AEP peak flood depths from the SOBEK model (existing conditions)

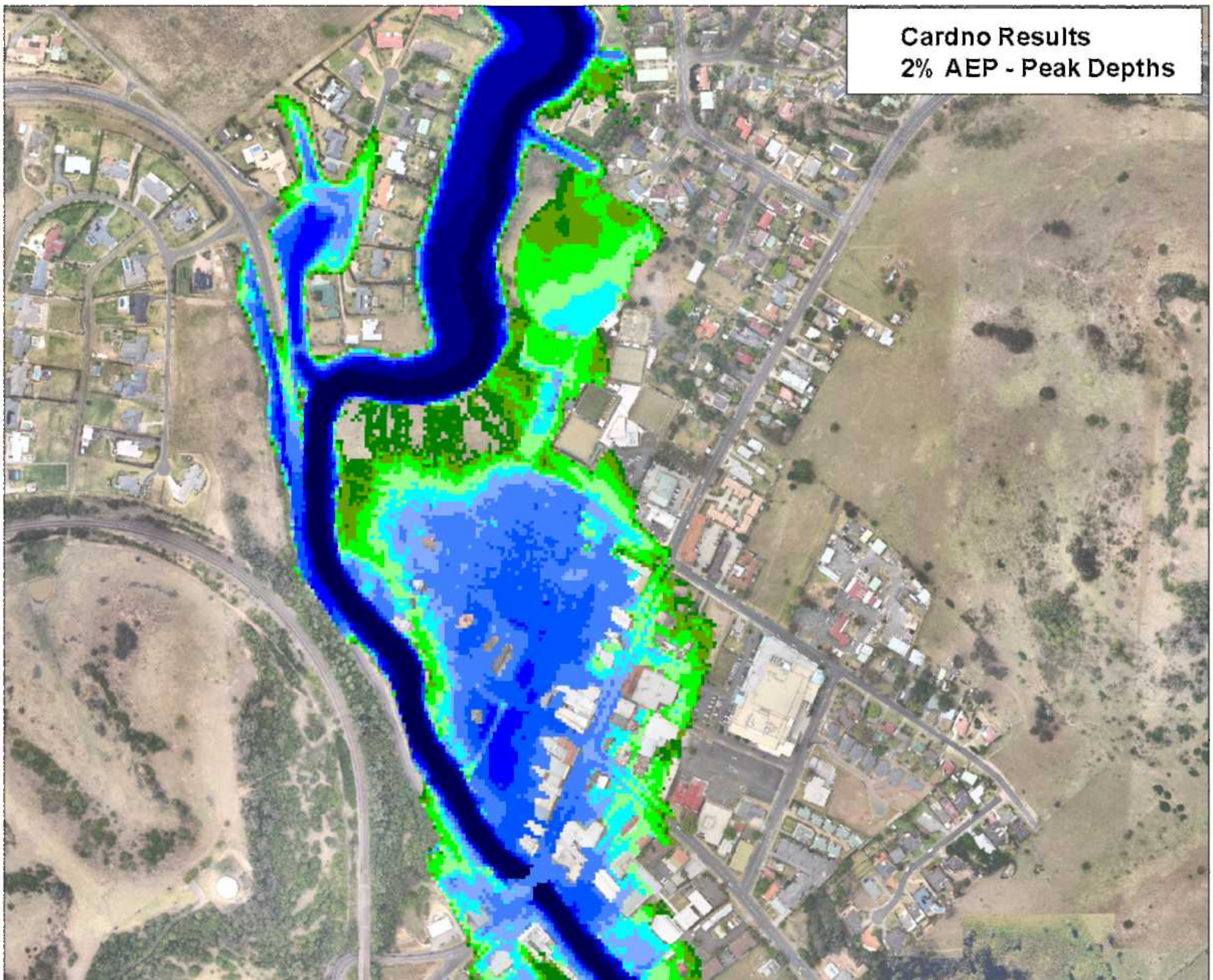


Figure 2-8 2% AEP peak flood depths from the SOBEK model (existing conditions)

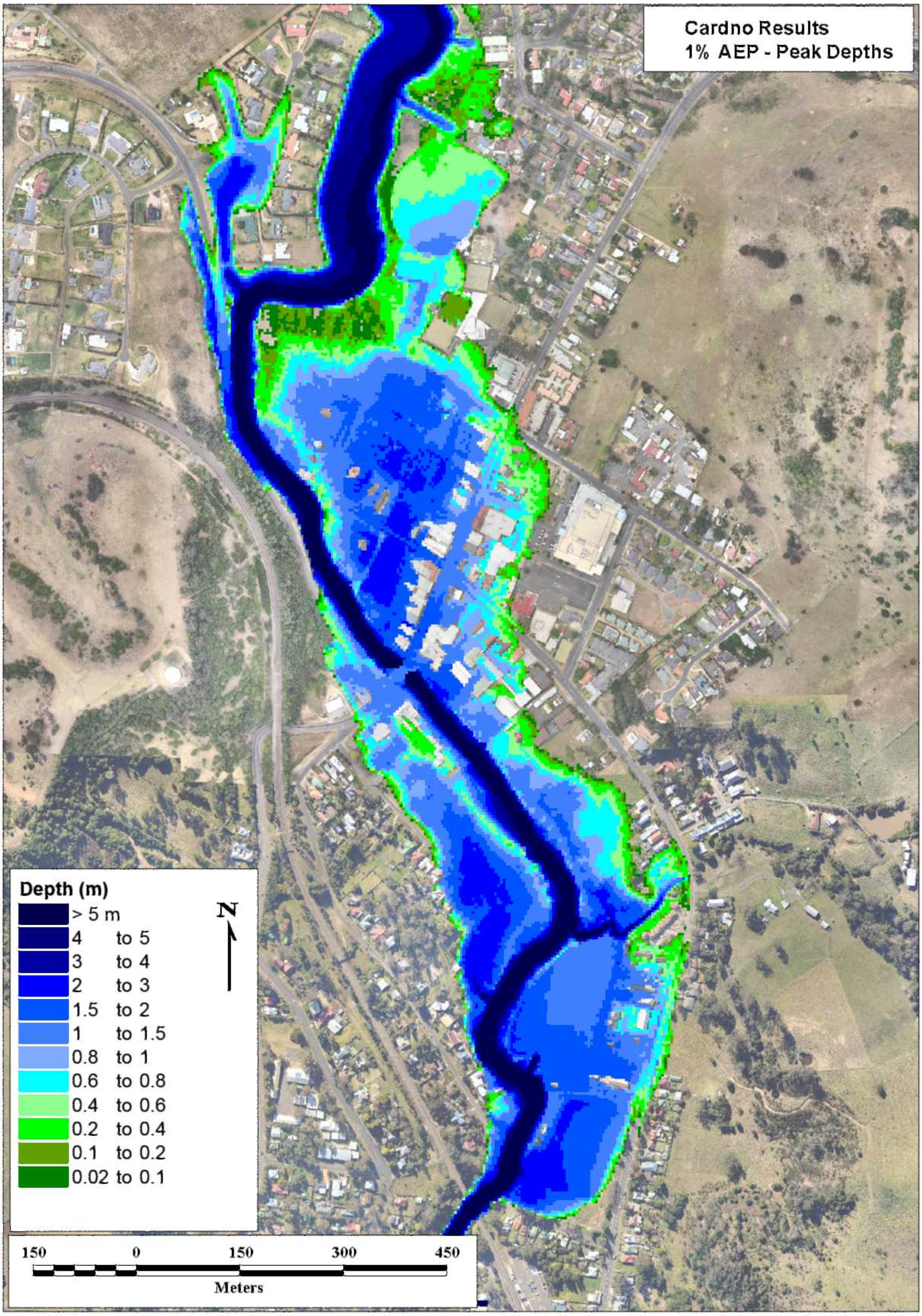


Figure 2-9 1% AEP peak flood depths from the SOBEK model (existing conditions)

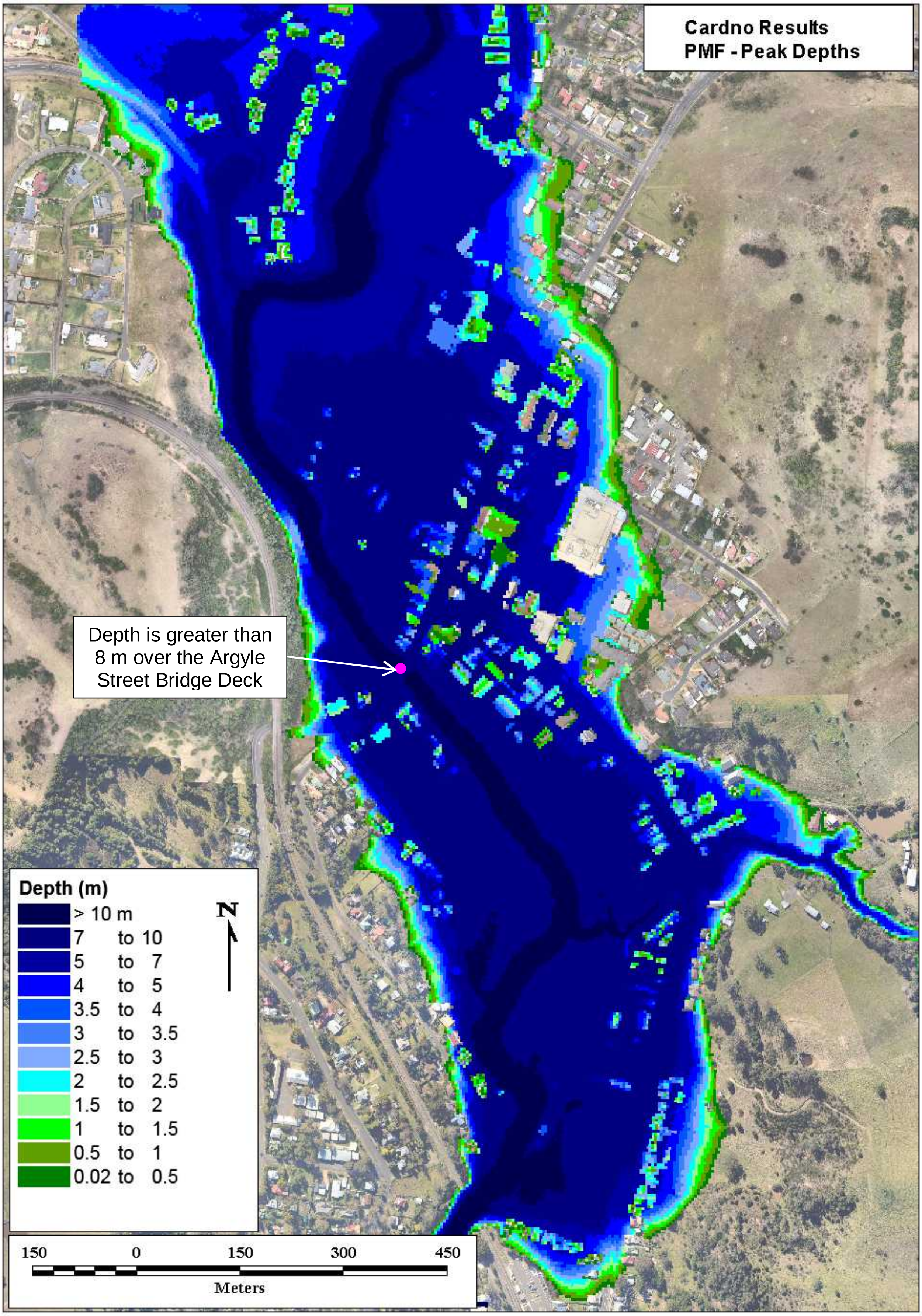


Figure 2-10 PMF peak flood depths from the SOBEK model (existing conditions)

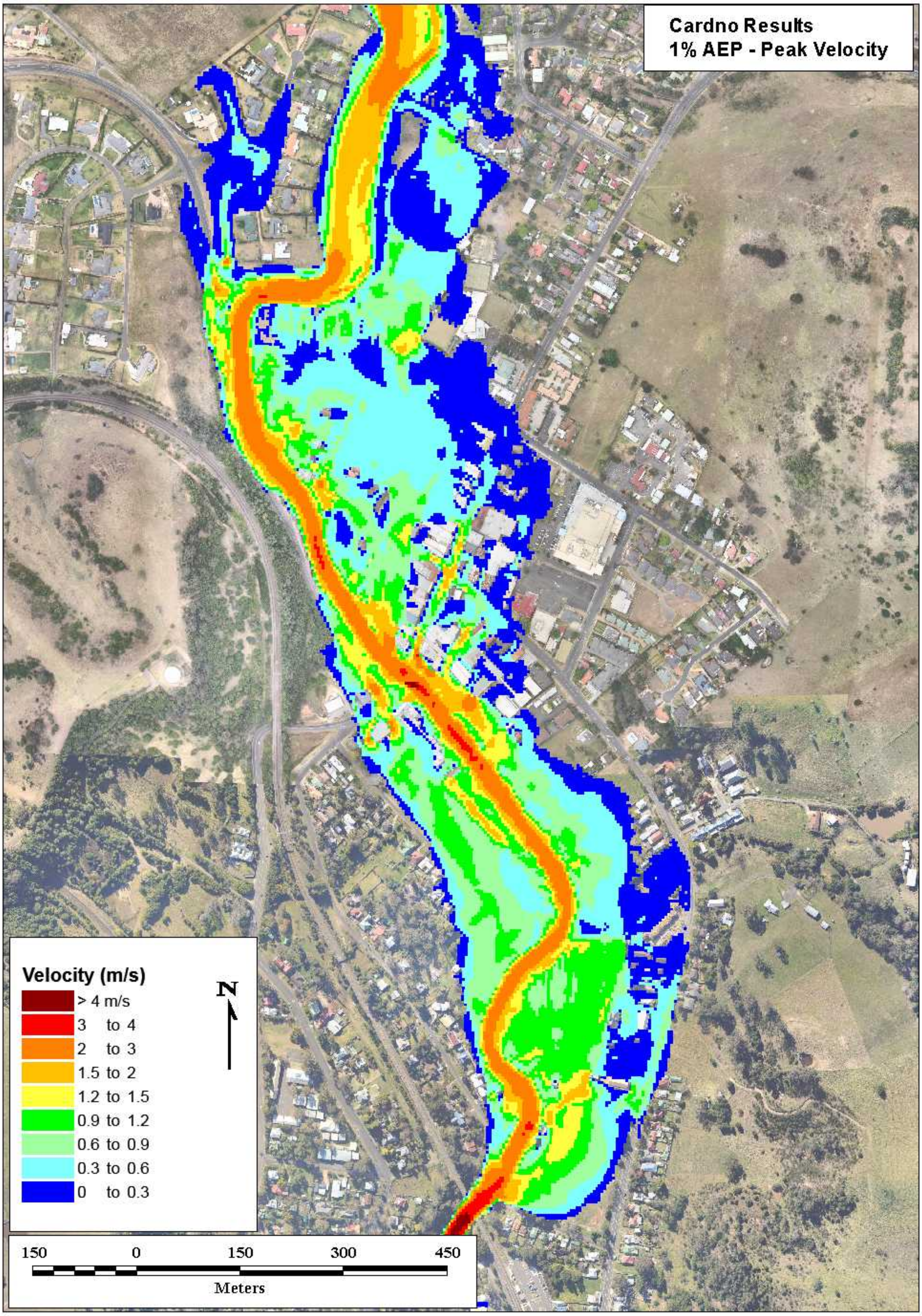


Figure 2-11 Peak velocity for the 1% AEP event from the SOBEK model (existing conditions)

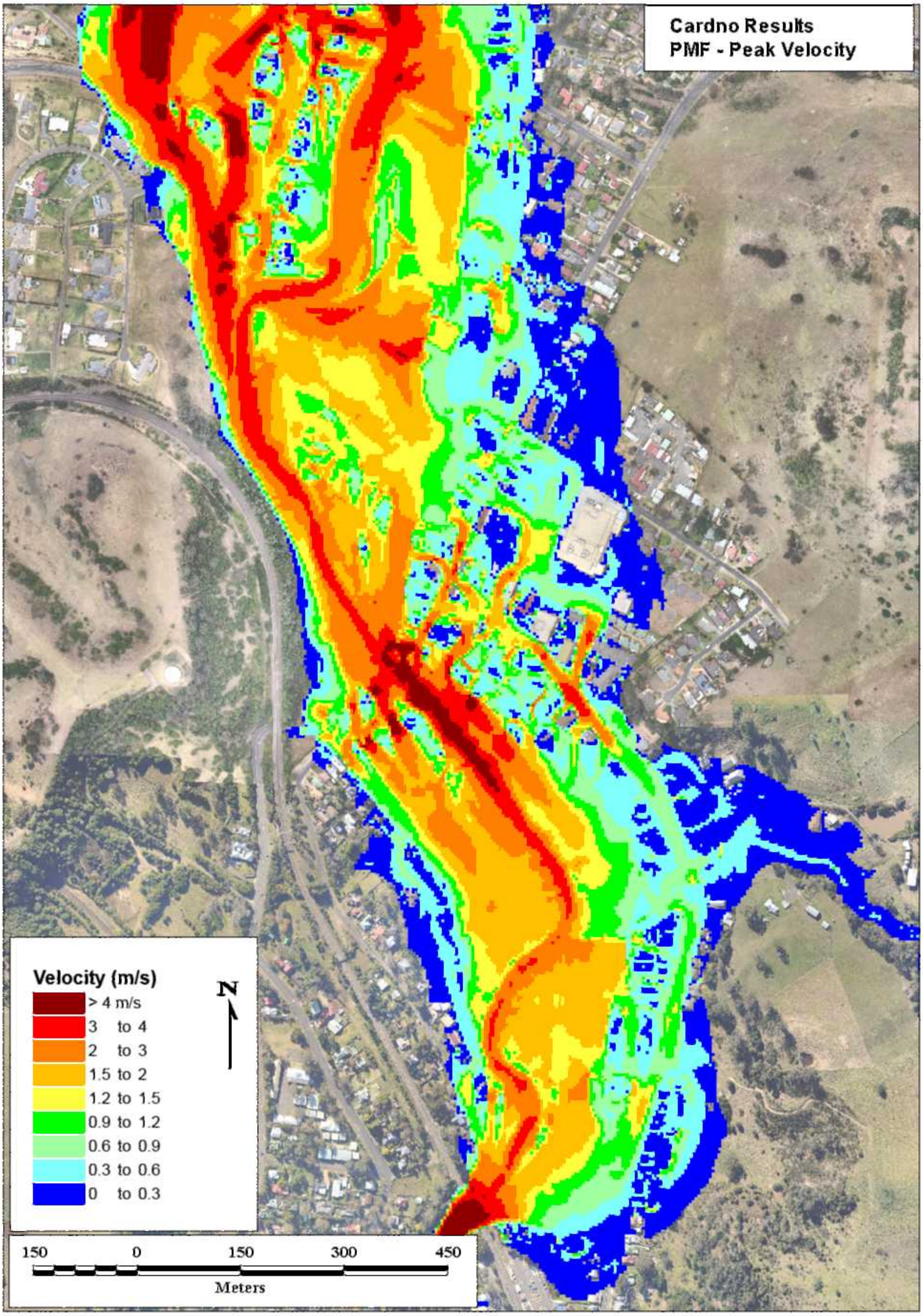
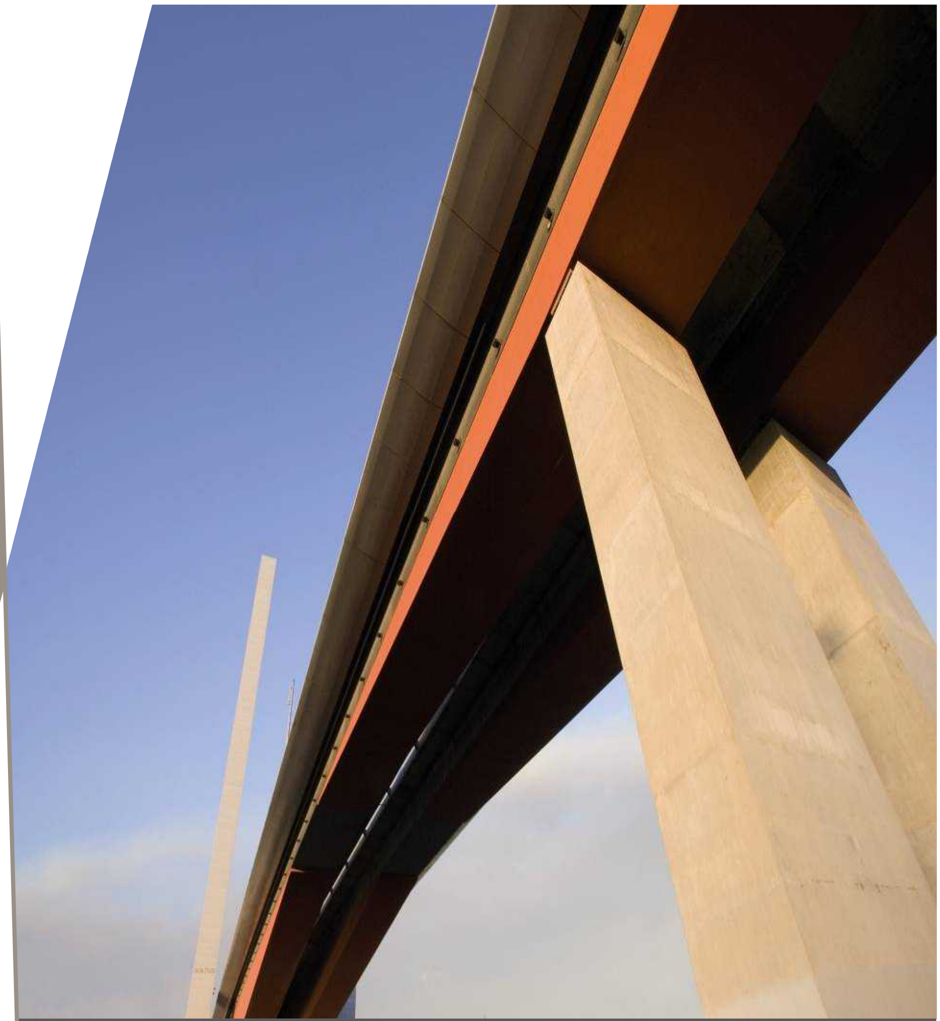


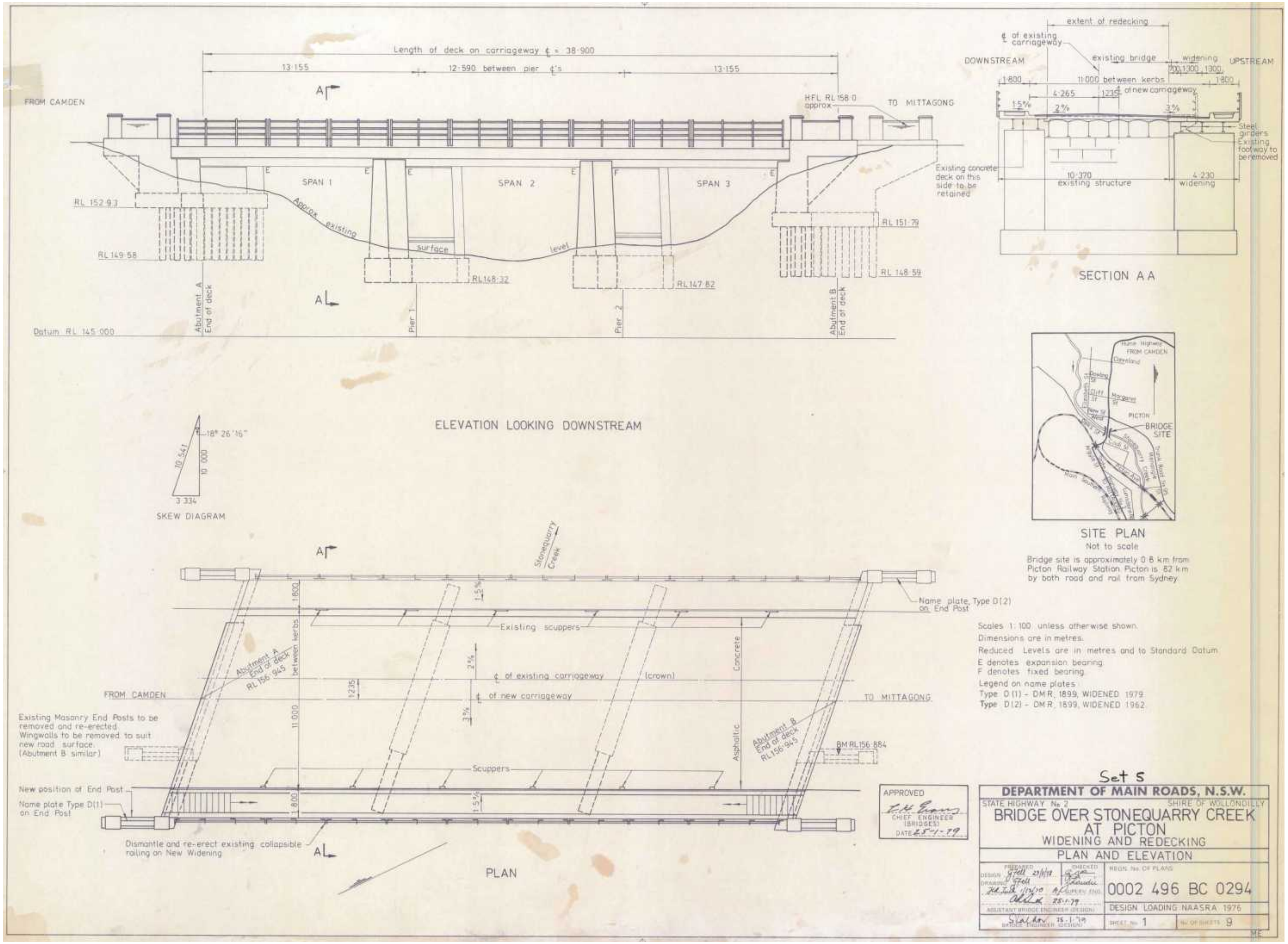
Figure 2-12 Peak velocity for the PMF event from the SOBEK model (existing conditions)

Hydrology and Hydraulic Assessment



APPENDIX A
ARGYLE STREET BRIDGE PLANS

Cardno
Shaping the Future



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APPENDIX B
WORLEY PARSONS RESULTS FROM THE RMA2 MODEL

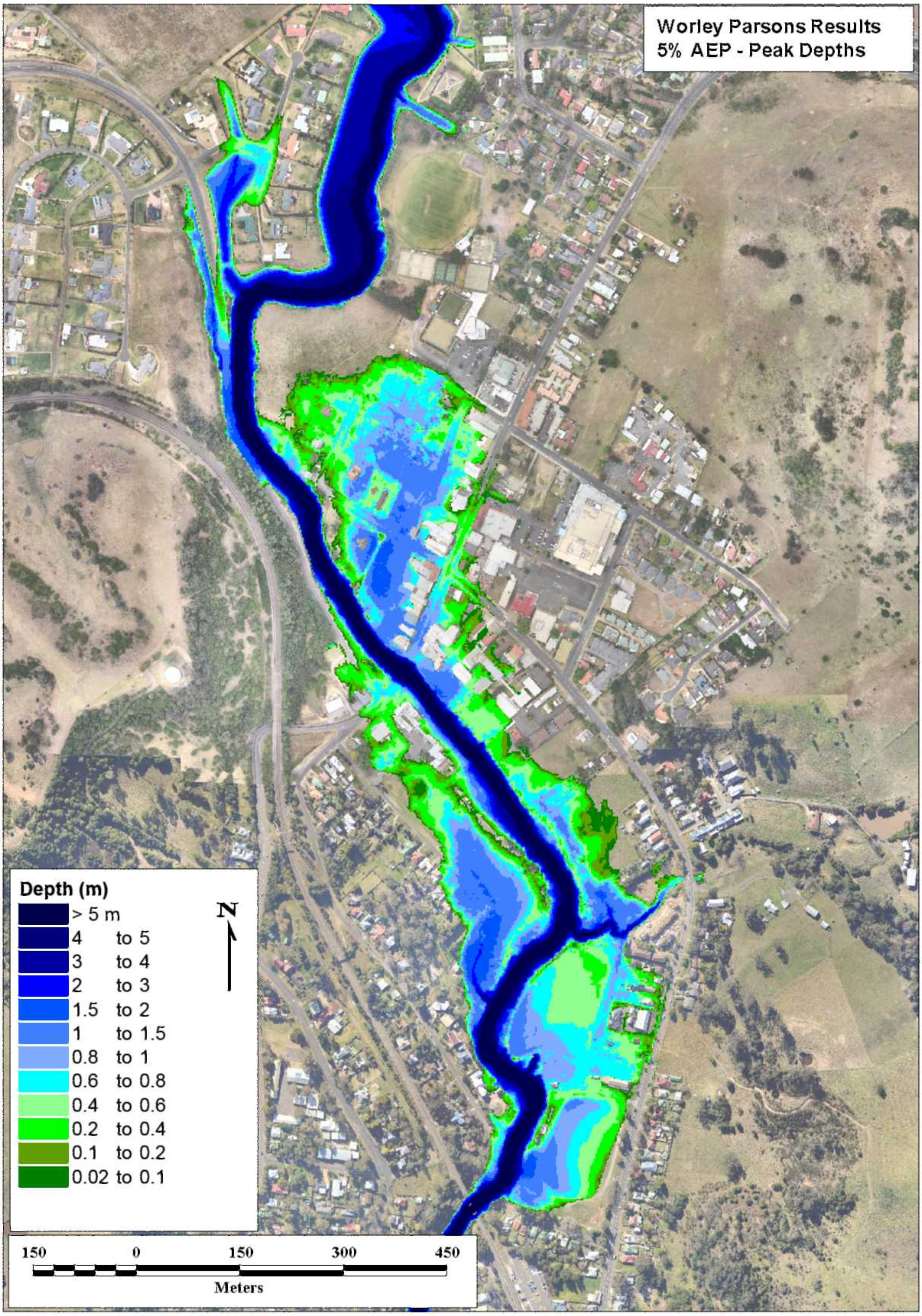

Figure A.1 5% AEP peak flood depths – Worley Parsons RMA2 model

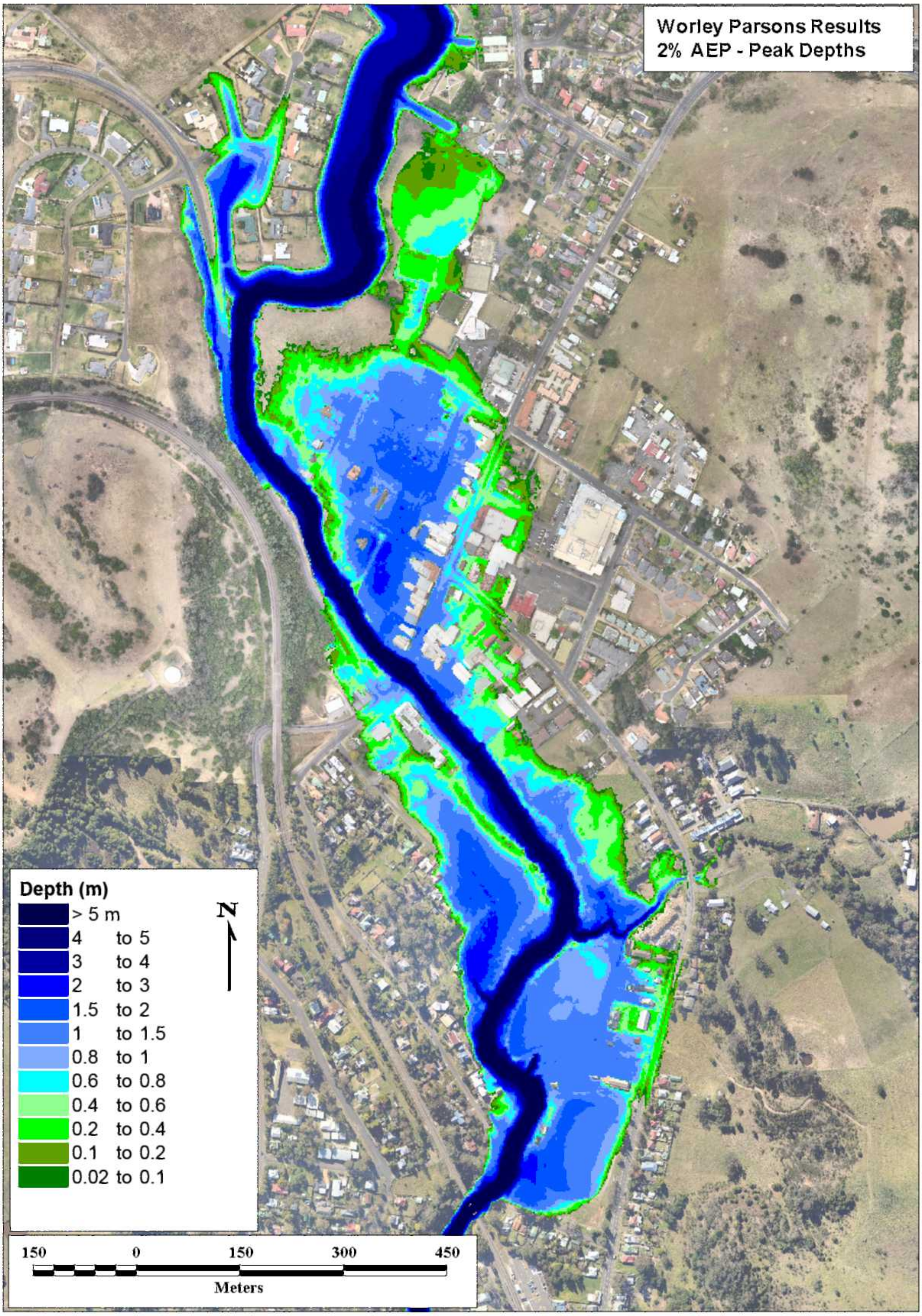

Figure A.2 2% AEP peak flood depths – Worley Parsons RMA2 model

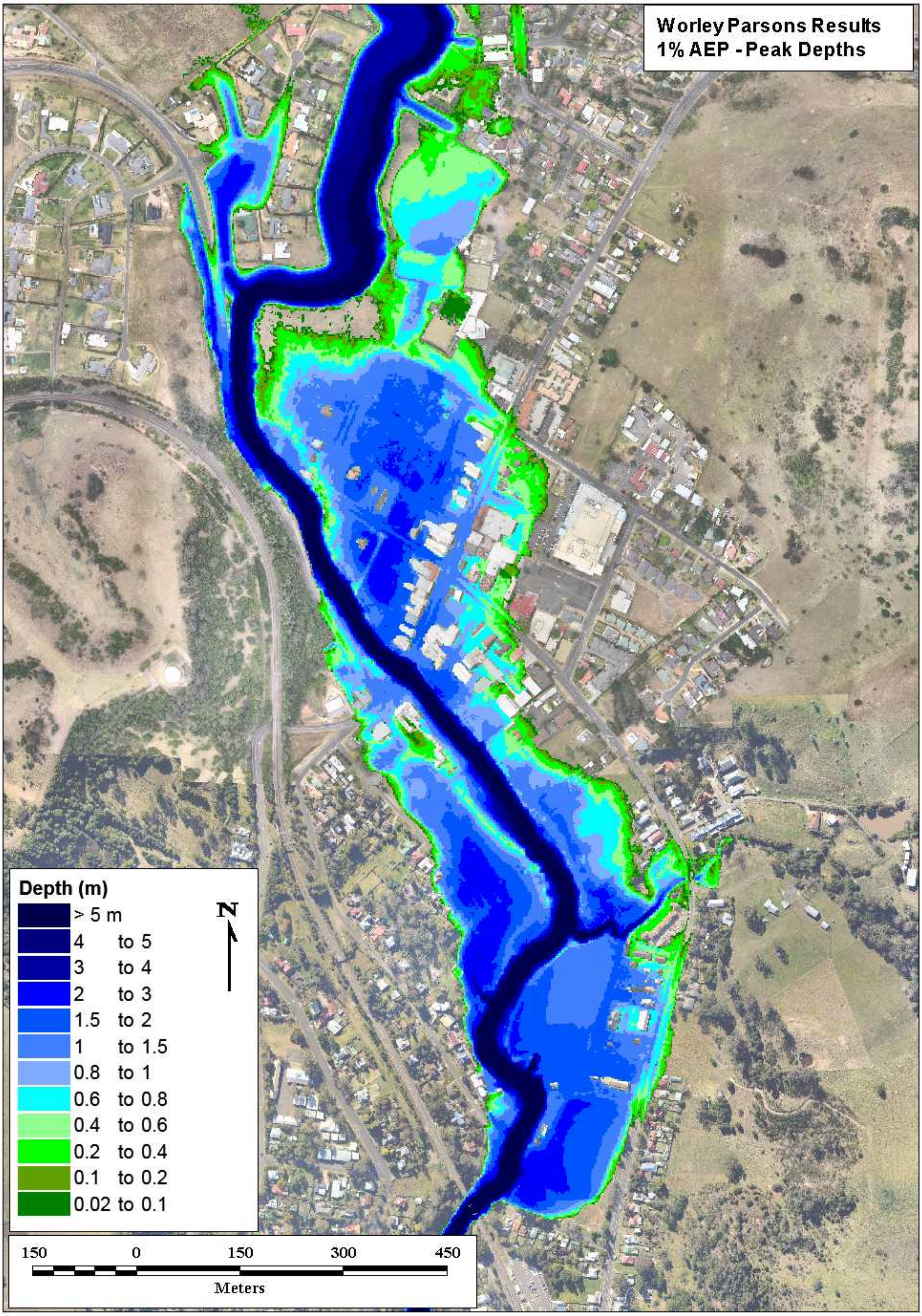

Figure A.3 1% AEP peak flood depths – Worley Parsons RMA2 model

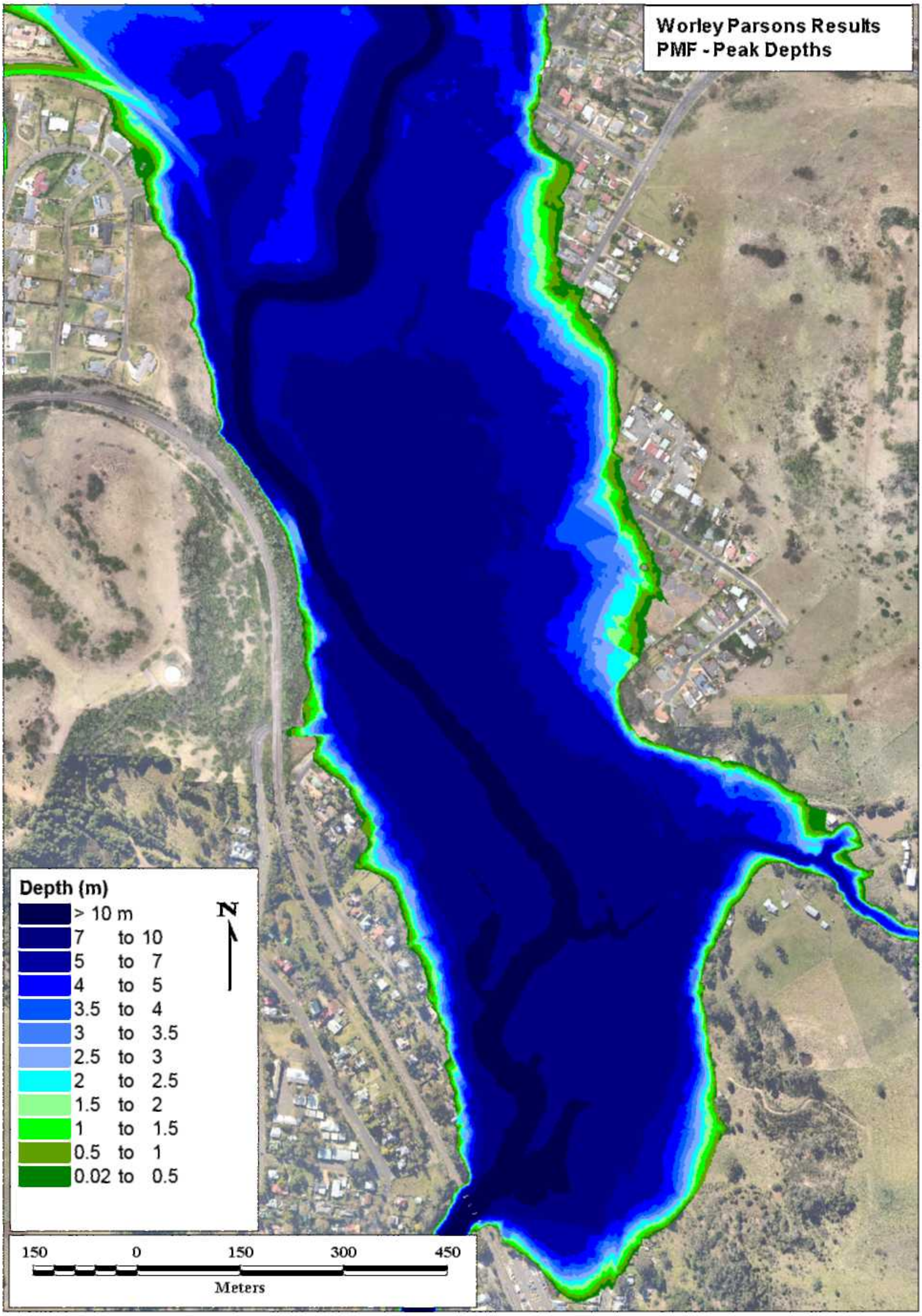

Figure A.4 PMF peak flood depths – Worley Parsons RMA2 model

APPENDIX C
SOBEK MODEL VALIDATION

C Model Validation

In order to validate the derived SOBEK model Worley Parsons supplied ascii result grids for the maximum flood depth, water surface elevation and velocity for the 5%, 2% and 1% AEP and the PMF event. The primary focus was to replicate the maximum flood depths for each of the events or understand why possible differences exist.

The peak depths as supplied by Worley Parsons are shown in Appendix B for the 5%, 2% and 1% AEP and the PMF events. These grids formed the basis for the comparison between the SOBEK and RMA2 model.

The model validation process started with the 1% AEP flood event as this event is the most important from a planning and assessment perspective for the proposed development and the township. The SOBEK model was developed and checked at various stages against the results obtained from the RMA2 model.

As Worley Parsons did not supply a topographic layer that was used in their RMA2 model this was unable to be checked for consistency between the developed topography in SOBEK and the final topography used in the RMA2 model. Both topographic layers were derived from the same LiDAR and differences should be minimal.

It was unclear whether the RMA2 model results were filtered or adjusted following their simulation. Cardno have applied no filtering or adjustment to the SOBEK outputs in comparing them to the RMA2 model results.

In assessing the SOBEK and RMA2 models Cardno has undertaken over 20 model runs to determine the cause of differences and to assess the sensitivity of the SOBEK model to changes in the input parameters. The key model runs are discussed in the validation section for each event.

C.1 1% AEP comparison

The 1% AEP event was simulated and the resulting peak water surface elevations (WSE) were compared between SOBEK and RMA2. The difference plot is shown in Figure C.1.

The majority of the floodplain is within +/- 50 mm between the two sets of model results. This confirms that the inflow hydrograph and downstream boundary supplied from Worley Parsons are consistent with between the two models and that the two models are representing overland flood behaviour consistently.

The areas shown in magenta are areas where the Cardno model had additional overbank flows that were not present in the RMA2 model, this is a key difference between the two modelling systems. SOBEK can explicitly handle wetting and drying of grid cells and hence it automatically allows overbank flows to occur. The RMA2 model system required a more iterative approach to determining the breakout areas and where overbank flows will occur. The two key areas where this occurs are shown on Figure C.1.

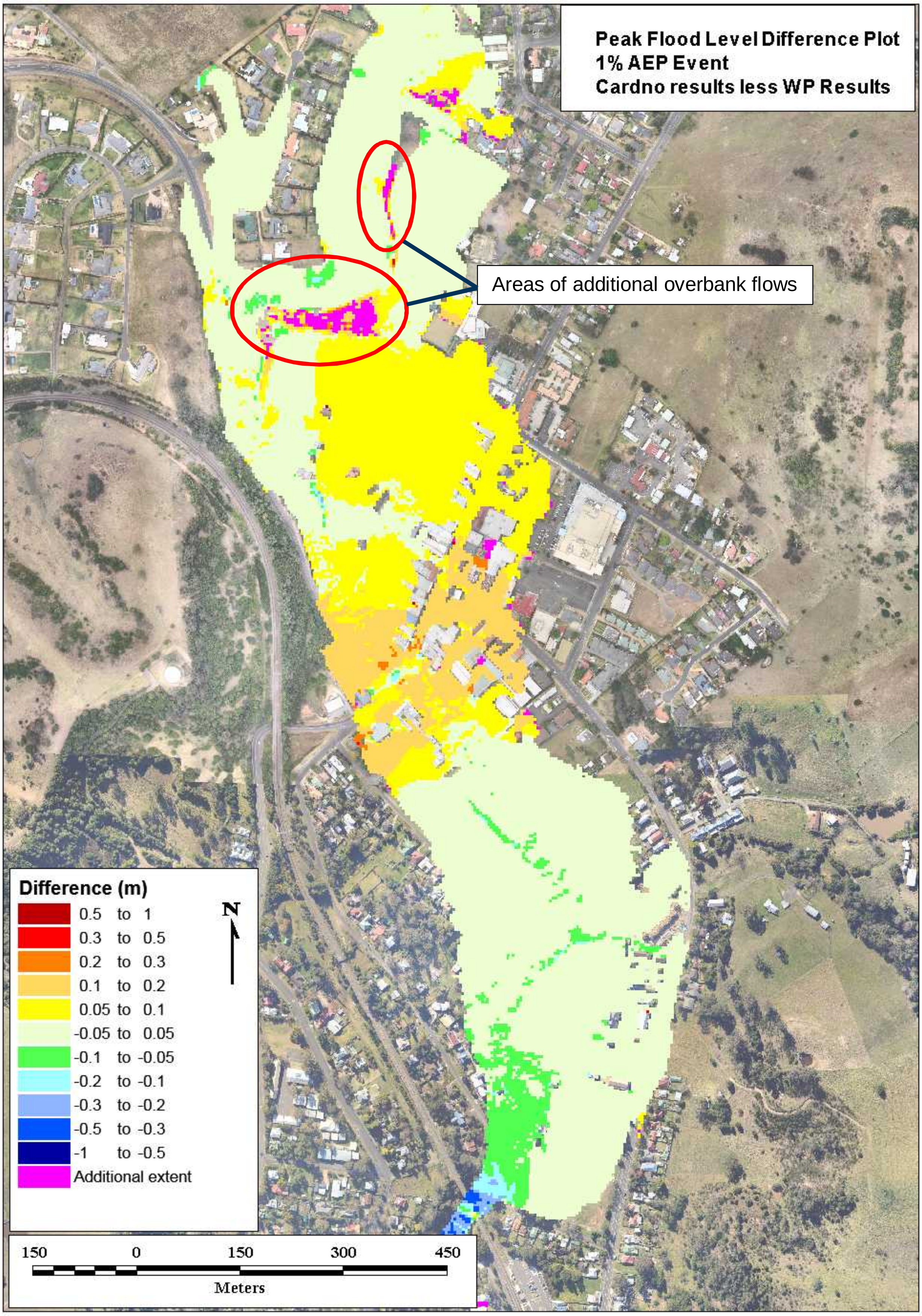

Figure C.1 Difference plot – 1% AEP event SOBEK model less RMA2 model

Overbank flow assessment

Additional examination of these areas indicates that the peak water surface elevations (WSE) between the SOBEK and RMA2 model are typically matching to within +/- 2 to 3 cm. The main differences occur just before the large bend in the river and at this location in the main channel the peak WSE match exactly between the RMA2 and SOBEK. This would imply that either the RMA2 model has a higher topography in this location or that as no breakout was expected in this area then the grid was removed from the RMA2 model. The peak WSE in the RMA2 model are higher in the main channel than the derived topography from the LiDAR in this location.

The spot height checks are shown in Figure C.2. The additional overbank flow increases the peak WSE in the SOBEK model as compared to the RMA2 model.

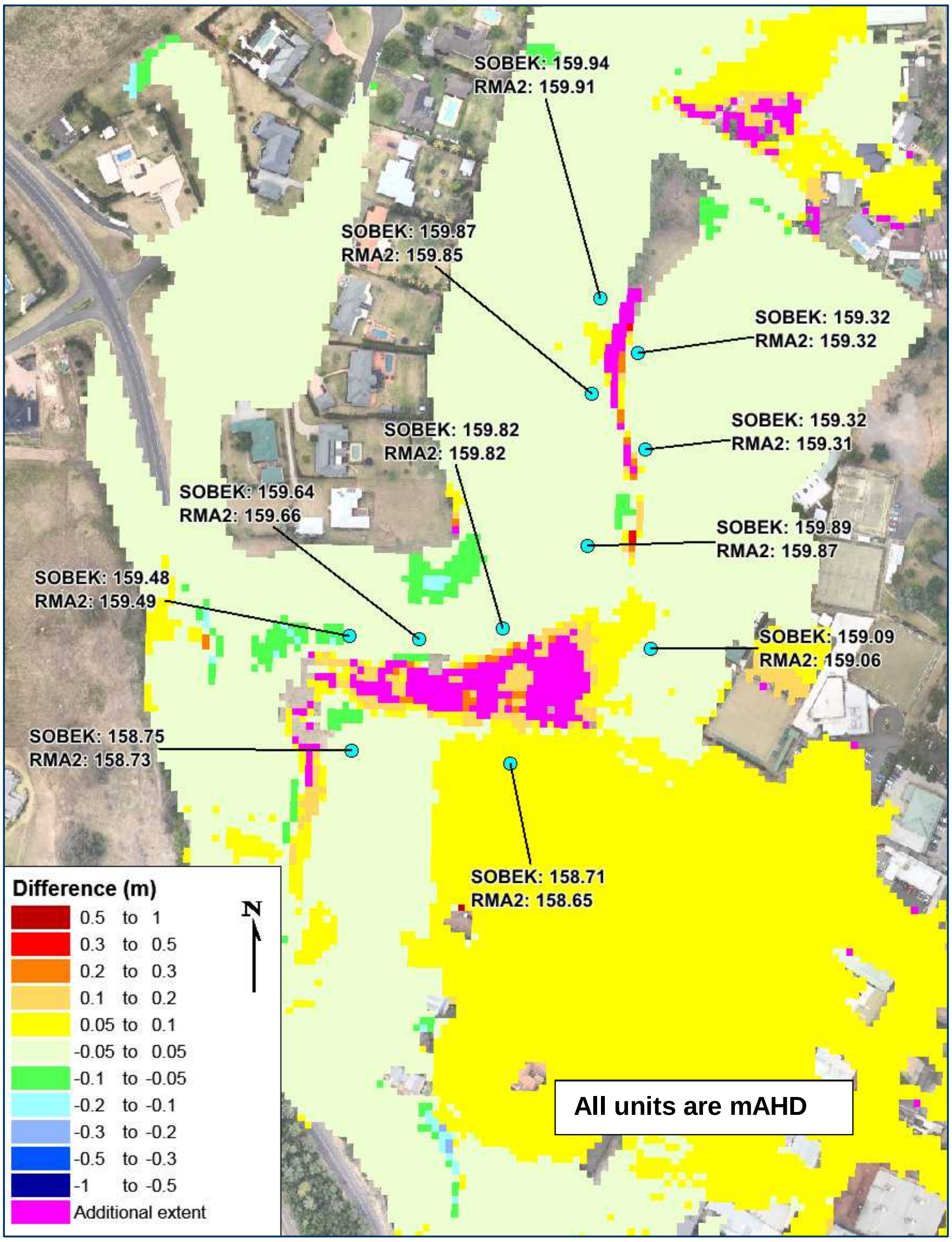

Figure C.2 Spot peak water surface elevation results for the 1% AEP event

Argyle Street bridge assessment

A main difference in the peak WSEs is upstream of the Argyle Street bridge. This is a critical area for the Stonequarry Creek model as this location is a major hydraulic control for the floodplain. Changes to the flood behaviour though this area cause the peak WSEs through the township to change. The inclusion of the Argyle Street bridge has reduced the flood conveyance through this area and cause the peak WSE to increase by approximately 20 cm.

To determine if this structure representation was part of the cause of this increase in peak flood depth a simulation was run where the representation of the Argyle Street bridge was setup similarly to the RMA2 model. The bridge deck was removed, the roughness increased and the bridge abutments were included in the channel bank.

The difference plot is shown in Figure C.3. The key differences between these results and with the Argyle Bridge structure included is the peak WSE immediately upstream of the bridge has been lowered by approximately 50 to 100 mm. This reduction in peak WSE was limited to approximately 150 m upstream of the structure.

The peak WSE through the township was reduced but the peak WSE was still 10 to 20 cm higher in the SOBEK model as compared to the RMA2 model. No substantial changes in the peak WSE were noted due to the removal of the bridge structure aside from the main channel as discussed above.

The figure shows that the additional overbank flows are still present following the change in bridge representation.

The inclusion of the Argyle Street bridge structure impacts the flood regime and as such will be included in the final SOBEK model as this is a truer representation of the impacts of the structure in this location.

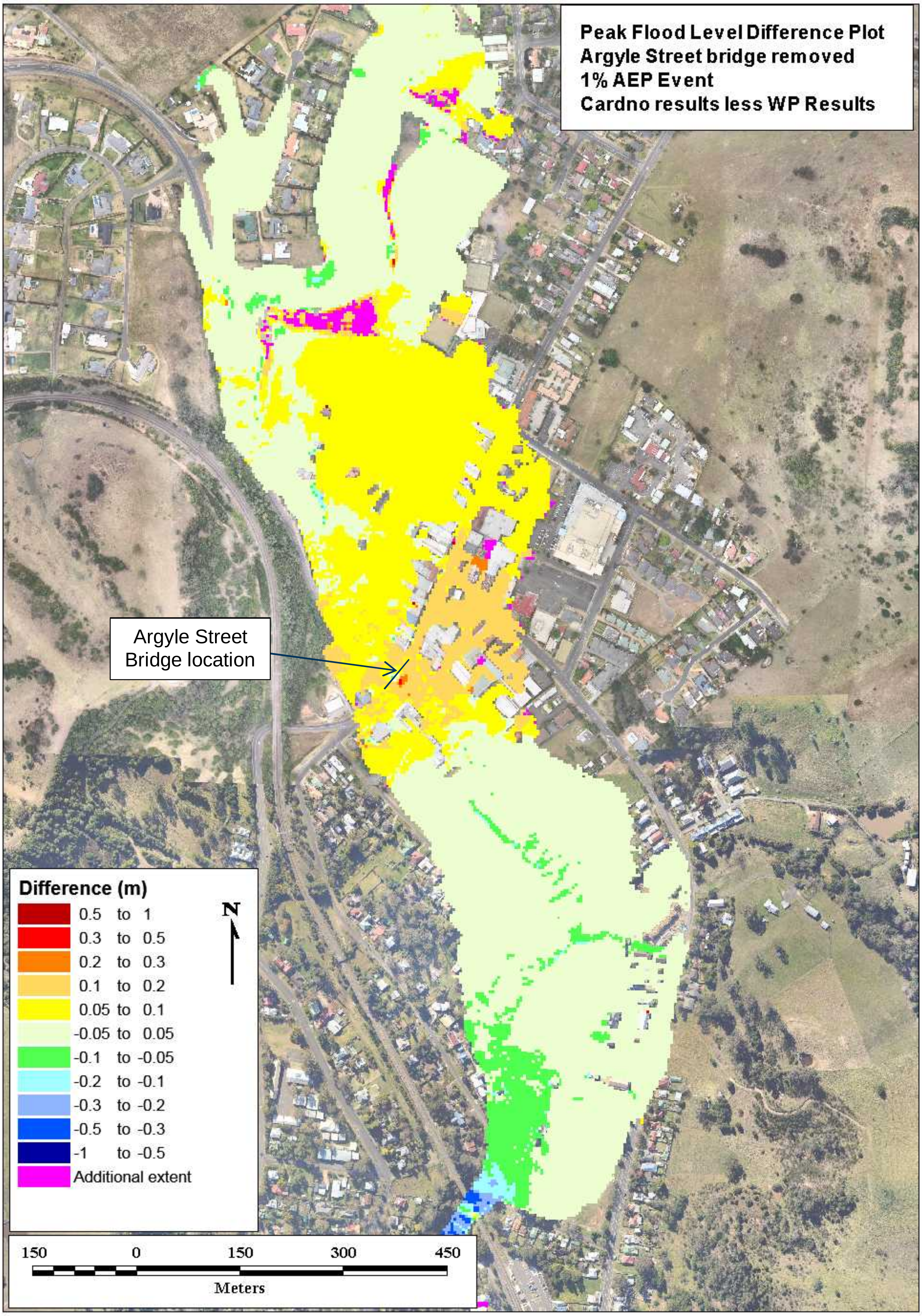

Figure C.3 Difference plot – 1% AEP event SOBEK model with Argyle Street bridge removed less RMA2 model

Summary

Overall the SOBEK model replicates the RMA2 model results throughout the floodplain to within +/- 50 mm. This is expected as the upstream and downstream boundary conditions are the same for both models.

An unknown difference is the topography layers used for the modelling as we did not have the RMA2 model grid to compare. Similarly we had only a PDF representation of the roughness elements.

The main differences between the two models are upstream of the Argyle Street bridge though the main township. Even though the peak WSEs differ in the township area, the peak flood extents are largely identical. There are only small areas where overbank flows differ and this gives credibility to the results obtained from both models.

Overall the peak WSEs are predicted to increase through the use of the SOBEK model. As the extent is largely consistent with the RMA2 model it is anticipated that the SOBEK model is acceptable for the assessment of the impacts of the proposed development. The SOBEK model will be used for assessment of differences between existing and developed conditions so the relative change to floodplain behaviour will be assessable.

C.2 2% AEP Comparison

For the 2% AEP flood event the results are consistent with the 1% AEP assessment in that the majority of the floodplain maintained peak WSEs at +/- 5 cm. The additional overland flow paths to the north of the proposed site are evident.

The increased overbank flows cause the peak WSEs through the township to increase as compared to the RMA2 model results. Interestingly the overland flow path that originates near the Hume Oval matches the peak WSE between SOBEK and RMA2. This implies the breakout through this area is being modelled similarly between the two models. Further downstream though the RMA2 model shows no breakout occurring near the bend in Stonequarry Creek. The peak WSE within the RMA2 model for the 2% AEP event are higher within the main channel than the top of bank topography (derived by Cardno) in this area by up to 60 cm. This suggests that breakout would be likely to occur over this bank area as suggested by the SOBEK model but this was not permitted within the RMA2 model or alternatively the RMA2 model has a higher top of bank level.

These additional over bank flows, coupled with the inclusion of the Argyle Street bridge structure lead to increased levels though the main township of up to 20 cm. Increases upstream of the Argyle Street bridge are up to 25 cm.

Overall the flood extent is maintained aside from the additional overbank areas that are inundated and some minor differences due to the topography through the township.

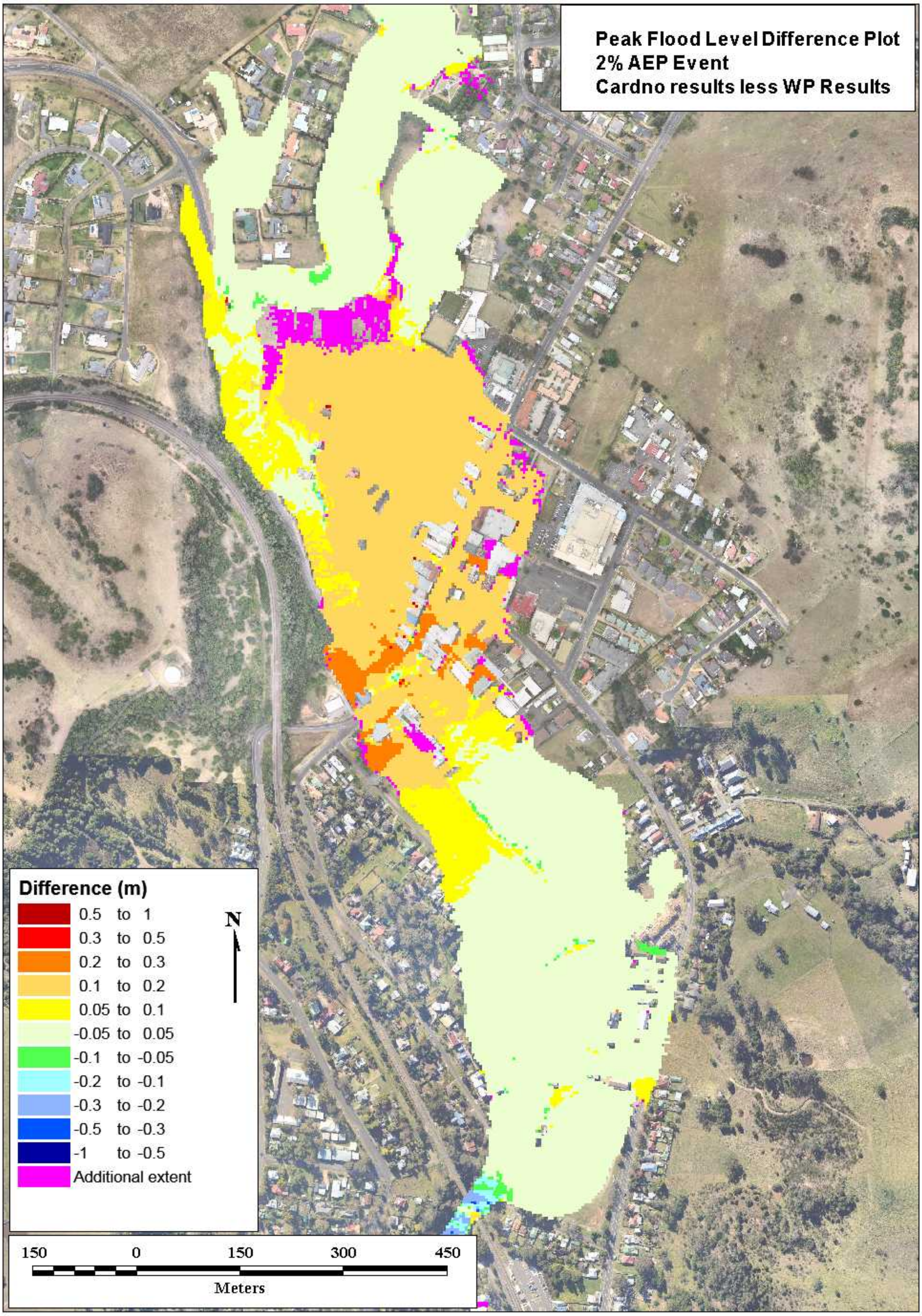

Figure C.4 Difference plot – 2% AEP event SOBEK model less RMA2 model

C.3 5% AEP Comparison

The key model differences of the Argyle Street bridge and overbank flow consideration have the largest impact on the 5% AEP flood event. This event has the most direct interaction with the Argyle Street bridge. The differences between the peak WSEs between the SOBEK and RMA2 model are shown in Figure C.5.

The figure shows large areas of increased flood extent in the SOBEK model results. There are a number of areas within the floodplain where breakouts occur in the SOBEK model that have not been predicted by the RMA2 model. We have undertaken a detailed assessment of the ground levels in the vicinity of the river breakout and are confident that the flood behaviour shown is correct.

Peak depths through the main floodplain near the township see peak WSEs increase by up to 30 cm. Upstream of the Argyle Street bridge the peak WSE increase by up to 36 cm.

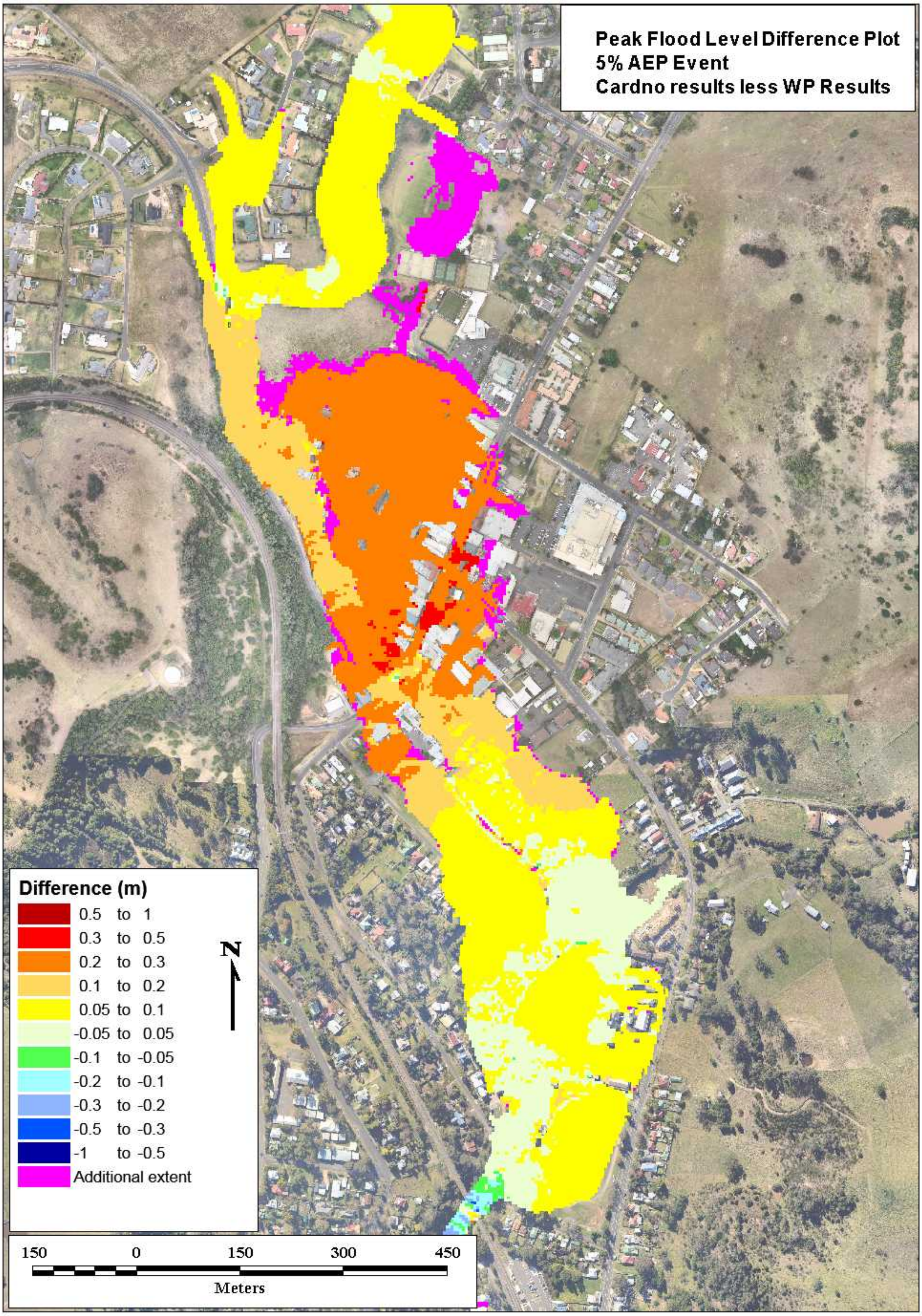

Figure C.5 Difference plot – 5% AEP event SOBEK model less RMA2 model

C.4 PMF Comparison

The PMF event was assessed and exhibited similar characteristics as the other model runs. The peak WSE increased upstream of the Argyle Street bridge and decreased downstream.

At the upstream end of the model there are some increases in WSE due to the interaction of boundary of the model and this area.

Overall the flood extent is maintained and the WSEs change by +/- 30 cm. The depths through the floodplain are several metres deep so this will not impact the emergency response recommendations.

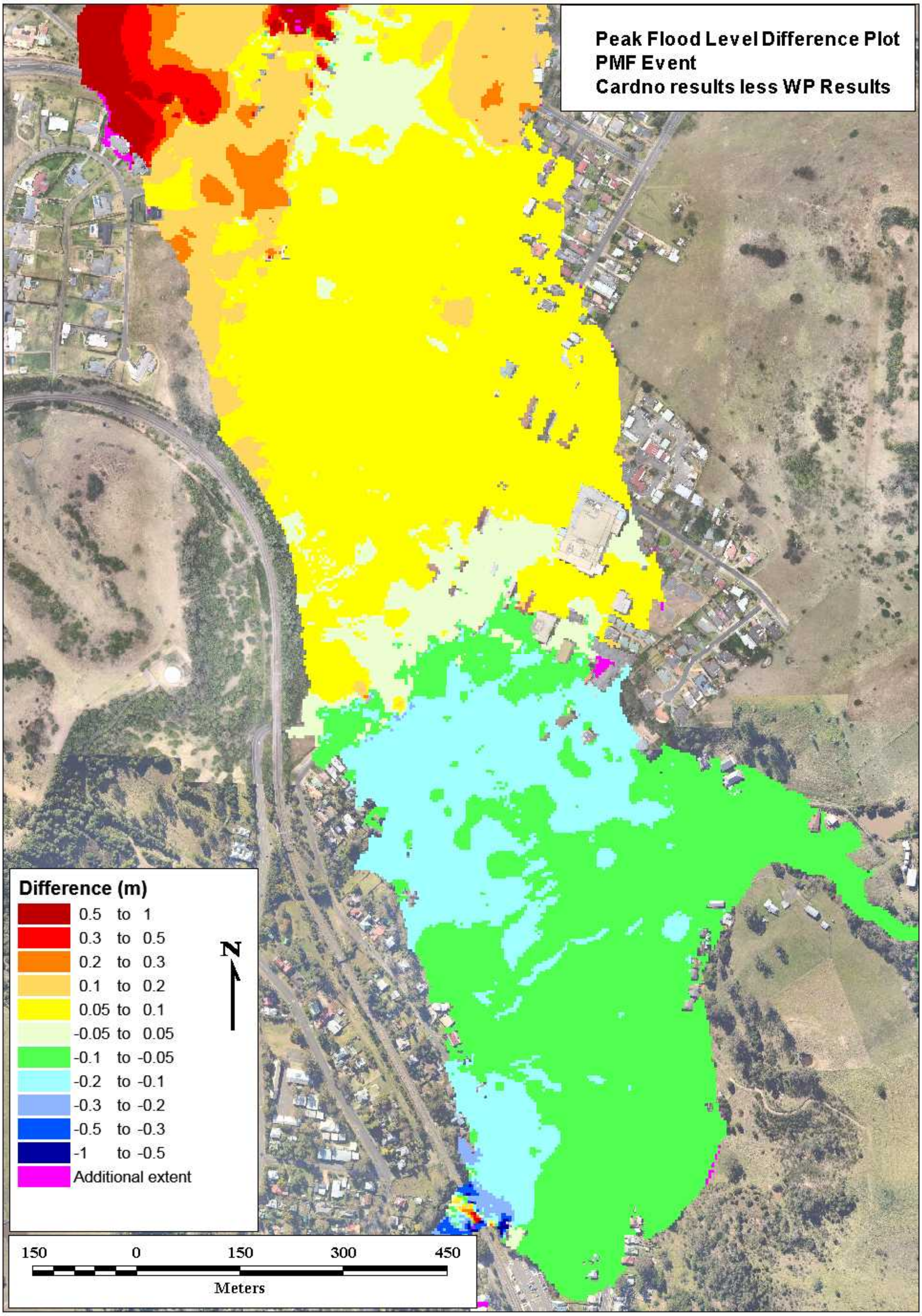

Figure C.6 Difference plot – PMF event SOBEK model less RMA2 model

C.5 Summary and Recommendations

Ultimately without full access to the RMA2 model it is not possible to determine the exact reasons for the differences between the model results. Some of the potential reasons are discussed and outlined. Overall, the SOBEK model is producing conservative results compared to the RMA2 model and is fit for purpose for the assessment of the proposed development.

The primary use of the developed SOBEK model is to assess the impacts of the floodplain from the proposed development and as such the model is fit for purpose.