FINAL REPORT:

Maffra Drainage and Integrated Water Management Strategy

September 2020

WELLINGTON
SHIRE COUNCIL

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Executive summary

Wellington Shire Council is planning for future residential expansion proposed to the north of the existing Maffra township. Alluvium Consulting Australia (Alluvium), along with flood modelling partners Water Modelling Solutions (WMS), were engaged to:

- develop a drainage strategy to accommodate future urban growth,
- undertake a flash flooding assessment,
- incorporate a considered assessment of Integrated Water Management (IWM) opportunities; and
- incorporate passive open space and improved amenity elements in drainage and treatment areas.

This report documents the existing conditions, site values and constraints as they pertain to stormwater management. The report documents the modelling undertaken to develop concept assets that will ensure the adequate management of the quantity and quality or stormwater under a developed scenario.

The report summaries the flood modelling assessment undertaken by WMS, with the full report attached to this report. The modelling indicates that the proposed assets will help alleviate flooding extents throughout Maffra.

The assessment focusses on opportunities beyond upgrading existing stormwater pipes. It focusses on identifying assets which can help alleviate flooding while creating high-quality community assets that provide habitat, amenity, cooling and recreation opportunities. The concept opportunities presented in the report include several wetland/retarding basin assets, increasing the Maffra retarding basin storage, and channel naturalisation opportunities. Stormwater harvesting opportunities are also investigated.

Next steps and recommendations for progressing the drainage assessment within Maffra include:

- Functional design of proposed flood mitigation and stormwater quality assets.
- Functional design of waterways including hydraulic modelling to ensure shear stress thresholds are not exceeded.
- Recommendation of the purchase of land for drainage purposes. This will need to include land for the assets and waterway alignments as currently the waterway passes through private land. The asset locations and arrangements as proposed within this report are somewhat flexible (i.e. can shift slightly should parcel purchase dictate this) but have largely been located in the most appropriate locations (for example of outfall purposes). Functional designs of the assets should follow the purchase of land so the space constraints are known prior to development of the assets.
- The staging of development will need to be confirmed to identify and further develop the assets required with the associated development. Given the Lot 1 and Lot 2 properties are likely to be developed first, the Powerscourt WL/RB will need to be prioritised to enable the development of those sites.
- Proceed with the design for the Maffra Recreation Reserve wetland, incorporating stormwater harvesting functionality and infrastructure.

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Abbreviations

Alluvium	Alluvium Consulting Australia Pty Ltd
AEP	Annual Exceedance Probability
AHD	Australian Height Datum
ARI	Average Recurrence Interval
BPEMG	Best Practice Environmental Management Guidelines (for Urban Stormwater)
DELWP	Department of Environment, Land, Water and Planning
EDD	Extended Detention Depth
IWM	Integrated Water Management
LSIO	Land Subject to Inundation Overlay
MUSIC	Model for Urban Stormwater Improvement Conceptualisation
NWL	Normal Water Level
RB	Retarding Basin
SEPP	State Environmental Protection Policy
TED	Top of Extended Detention
WGCMA	West Gippsland Catchment Management Authority
WMS	Water Modelling Solutions
WL	Wetland

1 Introduction

Wellington Shire Council is planning for future residential expansion proposed to the north of the existing Maffra township. Alluvium Consulting Australia (Alluvium), along with flood modelling partners Water Modelling Solutions (WMS), have been engaged to:

- develop a drainage strategy to accommodate future urban growth,
- undertake a flash flooding assessment,
- incorporate a considered assessment of Integrated Water Management (IWM) opportunities; and
- incorporate passive open space and improved amenity elements in drainage and treatment areas.

The drainage assessment will inform the development of the ultimate masterplan for the site as well as a Developer Contributions Plan (DCP).

This report summarises existing conditions and issues as they pertain to stormwater management in the project area, as well as issues and constraints that may impact upon the implementation of future water management strategies in a post-development scenario. The report covers the analysis undertaken to develop stormwater management treatment options, IWM opportunities, and existing and developed conditions flood modelling.

1.1 Location

The Maffra township is located approximately 20km north-west of Sale in Gippsland. The town centres on the Macalister River, which flows south into the Thomson River. The proposed development area is to the north of the existing township and covers an area of approximately 202 ha. The area is expected to be zoned for residential development. Other notable features include:

- Irrigation Channel
- The Maffra Retarding Basin (RB)
- Fosters Hill to the north of the town
- A Catchment Management Authority (CMA) designated waterway beginning at the outfall of the RB
- A concrete channel running east from Alfred Street which transition into an earthen waterway
- Maffra Wetlands Reserve
- Maffra Golf Course
- Cameron Sports Complex.

The existing features are described in more detailed in Section 2. A site context map is provided in Figure 1.

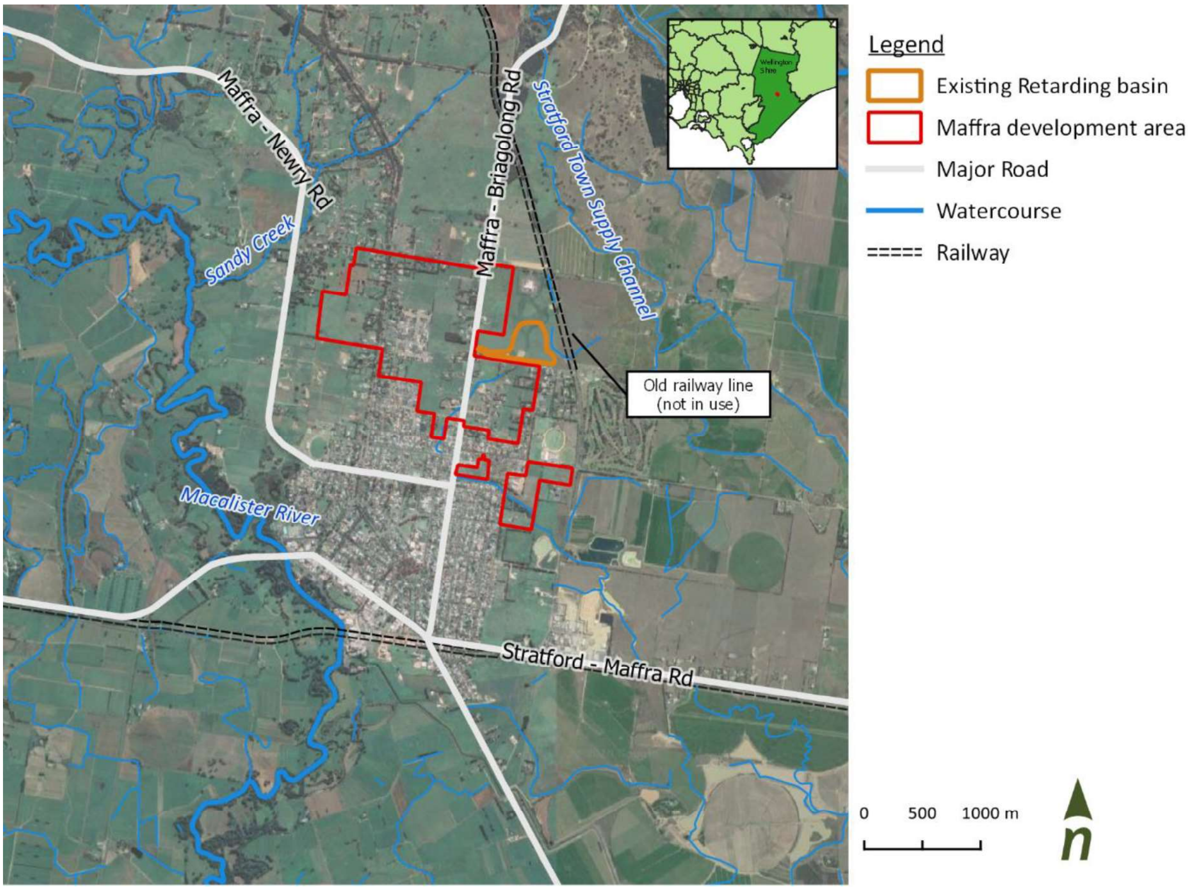

Figure 1. *Maffra township context map*

1.2 Project background

Maffra has a population of approximately 4,000 with this set to grow with future residential development. A large portion of the town, particularly in the north of the township where development is due to occur, has historically suffered from flash flooding via an ephemeral stream that rises to the north of the town at Fosters Hill.

A number of drainage assessments have been undertaken for Maffra to resolve specific issues. In response, a retarding basin was constructed north of the township in 1998/99, and this asset has been reviewed several times to enhance storage and outfall arrangements. It has been determined that the capacity of the existing retarding basin can cater for a 5% AEP event, which is lower than the typical level of service for a retarding basin of 1% AEP. The existing basin is a focus of this assessment. A summary of the key drainage reviews and recommendations is provided in Section 1.4 below.

The proposed development areas are provided in Figure 2. It includes a large area to the north of the existing township and west of the existing RB, as well as some small pockets near the Davis Street drain in the east of town. At the time of this assessment there was no masterplan for this development, but Council has indicated that this will be residential zoning.

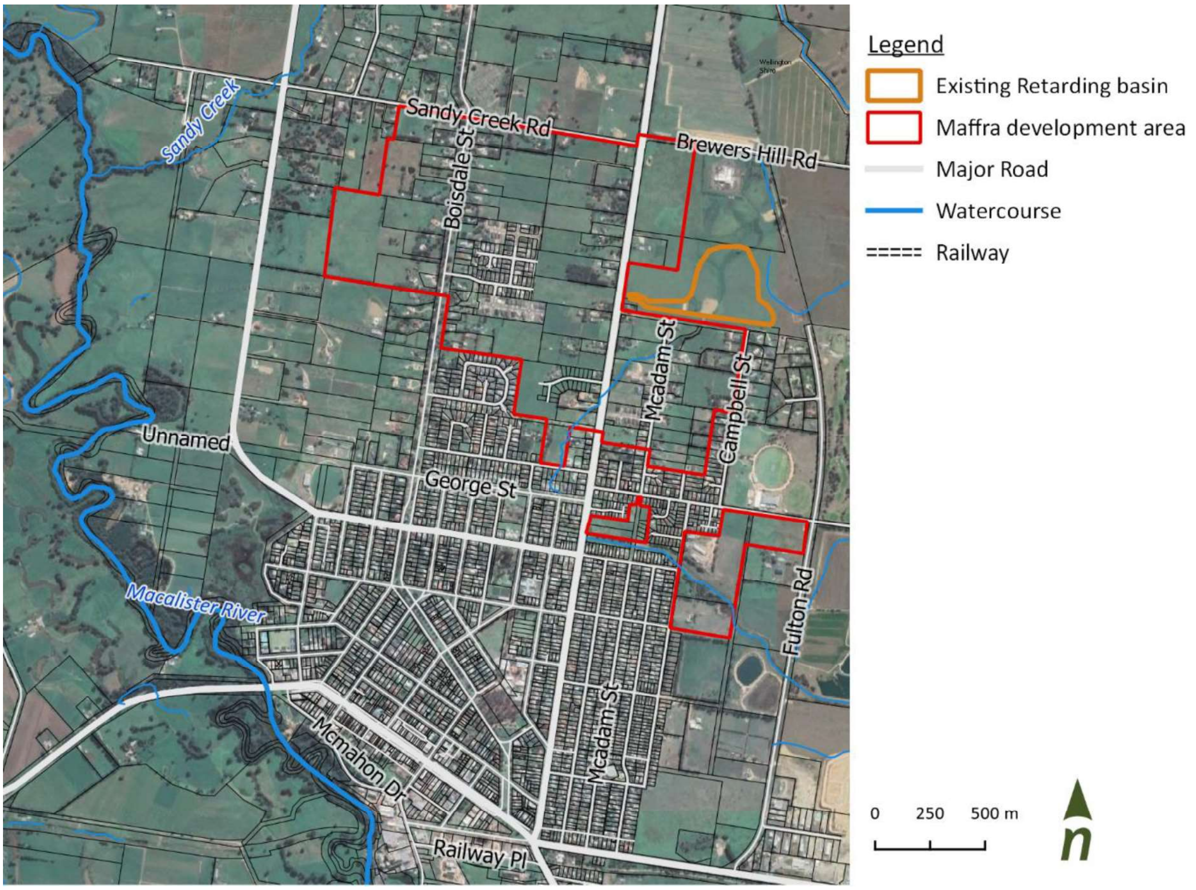

Figure 2. *Proposed growth areas in Maffra*

1.3 Project objectives

The objective of this project is to develop a drainage strategy for Maffra, specifically the area north of Princes Street and Blyth Street. The strategy aims to achieve the following objectives:

- Investigate and map flash flooding extents to help plan for required infrastructure
- Support a strategic planning assessment of the township to enable future residential growth
- Identify stormwater treatment areas to meet industry best practice guidelines, trunk drainage, and overland flow path requirements
- Provide preliminary cost estimates of stormwater management infrastructure
- Investigate and propose solutions which meet Wellington Shire Council's IWM objectives
- Incorporate open space elements which provide for a high level of amenity and guide treatment and outfall designs.

1.4 Background information

For this drainage strategy, the following sources of information have been drawn on:

- Review of Drainage Outfalls, Maffra (Fisher Stewart, 1998)
- Review of Drainage Proposals and Technical Assessment of Hydraulic Characteristics (Fisher Stewart, 1999)

- North-East Drainage System Maffra: Drainage and Retardation Basin Report (Fisher Stewart, November 1999)
- George Street Proposed Replacement Drainage designs (Fisher Stewart, November 1999)
- Retarding basin designs (Fisher Stewart, February 2000)
- Catchment Analysis (Cardno, 2009)
- Maffra Drainage Review (Cardno, 2010)
- Retarding Basin Performance Review and Optimisation – Maffra (Water Technology, July 2014)
- Existing drainage network and culvert information (Wellington Shire Council)
- Proposed growth areas (Wellington Shire Council)
- Aerial imagery (nearmap)
- Elevation data:
    - 0.5 m contour (provided by Wellington Shire Council)
    - LiDAR (provided by WGCMA)
- 1% AEP flood extent GIS layer

Some of the key documents have been summarised below.

Catchment Analysis, Hydrologic Report (Cardno, 2009)
The northwest corner of Maffra has a history of flooding with an undersized retarding basin and insufficiently sized drainage infrastructure to convey flows through the town. The purpose of this assessment was to identify the necessary infrastructure to properly mitigate all storms up to and including the 100 year ARI event.

The site was characterised as follows:

- The catchment to retarding basin is largely rural and approximately 259 ha.
- The catchment downstream of the RB is approximately 56 ha (downstream of Powerscourt Street).
- The catchment downstream of the RB drains to the south west to Powerscourt Street.
- After Powerscourt Street the channel flows south via a shallow open channel and then down Alfred Street. Halfway down Alfred Street an open channel picks up the flow and conveys it east, across Powerscourt Street and out of town.
- At Powerscourt Street a 900mm diameter pipe on the east side of the street, conveying a portion of the flow from the RB south down the street (approximately $0.8m^3/s$).

Hydrologic modelling undertaken established the following:

- The existing RB (approximately $65{,}000m^3$) has reasonable management of the 10 and 20 year storms, but has little effect in mitigating the 100-year event.
- Increasing the RB storage to approximately $133{,}800m^3$ (more than double the existing) and removing one of the 600mm pipe outlets (assuming other outlet arrangements stay the same) reduces the peak 10 year and 20 year storm flow rates downstream of Powerscourt Street, as well as greatly reducing the 100-year peak flow ($8m^3/s$ to $3.5m^3/s$).
- Increasing the basin size to roughly $211{,}900m^3$ and removing one of the 600mm pipe outlets (assuming other outlet arrangements stay the same) again further reduces the peak flow rates for the 10 and 20 year storms downstream of Powerscourt Street, and further reduces the 100-year peak flow to $3.1m^3/s$.
- The peak flows downstream of Powerscourt Street are shown in the table below (from Cardno's report).

Storm	Existing Basin Flow (m^3/s)	Medium Basin Flow (m^3/s)	Large Basin Flow (m^3/s)
10 Year	1.7	1.4	1.1
20 Year	3.4	2.3	1.9
100 Year	8	3.6	3.1

Figure 3. *Peak flows downstream of Powerscourt Street for different RB scenarios (Cardno, 2009)*

Conclusions:

- The basin does little to manage the 100 year event.
- Downstream of the RB there are two peaks due to the timing of the peak discharge from the RB and the peak from the downstream catchment. In both scenarios the peak flow is generated primarily by the downstream catchment, and the peak flow from the basin has a relatively small impact. Therefore the reduction in peak flow from the basin is not realised downstream.
- The conveyance downstream of the RB is insufficient to manage the runoff from the catchment downstream of the RB and needs to be upsized.
- The assessment recommended to upgrade the RB to the medium size basin (double the storage), and to remove one of the outlet pipes.
- Conveyance between Powerscourt Street and Merry Street should be increased depending on the level of protection sought.

Maffra Drainage Review (Cardno, October 2010)
Cardno was engaged by Wellington Shire Council to undertake a review of previous drainage studies and provide a desktop review of proposed drainage solutions for flooding issues in the north-western area of Maffra. Findings and recommendations included:

- The town has a history of flood issues with a drainage system that is, in some areas, inadequate to cater for large rural flows and developmental pressures.
- The current retarding basin has a storage of 65,000m^3 and is effective in the 10 to 20-year ARI events, but only provides relatively a small amount of relief in the 100-year ARI event.
- The overland flow path from the RB catchment, and the catchment immediately downstream of the RB, through the northern part of Maffra is under capacity and has a history of flooding.
- Cardno recommended the replacement of the George St drain from the existing 450mm to 1350 mm diameter pipe from Merry Street to the outfall, at a preliminary cost of $1.3million.
- Recommendations also included upgrading the retarding basin volume and discharge rate to the 'medium' option in Cardno's previous 2009 report (an additional 69,000m^3). This was at an estimated cost of $695,000. These works would also require the formalisation of the overland flow path between Powerscourt Street and Merry Street.

Retarding Basin Performance Review and Optimisation – Maffra (Water Technology, July 2014)
Wellington Shire Council engaged Water Technology to review historic investigations of the Maffra retarding basin, to establish whether the basin performance could be optimised to alleviate downstream capacity constraints. The review included reviewing the stage storage relationship adopted by previous investigations, confirming the outlet details and discharge, undertaking hydrologic modelling and comparing peak flow estimates established in this study against previous studies. The investigation found that the basin weir is engaged in events greater than the 20 year ARI. The report suggests the existing basin volume is less than required to mitigate larger duration ARI events.

A comparison of stage storage relationships found differences in Cardno and Water Technology's basin storage, based on different data sources. Water Technology found the basin volume to be approximately 105,000m^3 based on LiDAR. The basin inflows/outflows estimated by Cardno were also found to be less than Water Technology's, and the peak elevation in the RB were higher than identified in this study.

Water Technology undertook some scenario analysis that looked at alternative basin storage arrangements, as well as alternative outlet arrangements. This was done to understand whether different arrangements could result in better basin performance across a range of durations. The options investigated were:

- Option 1: Fitting a glory hole arrangement to one of the outlets (no increase in storage)
- Option 2: A 20% increase in basin volume and the modified outlet works from option 1
- Option 3: Increasing the basin volume until the 1% AEP does not spill over the weir
- Option 4: Using the optimised basin volume from option 3 and including a glory hole arrangement from option 1.

Conclusions from this analysis included:

- To effectively mitigate events up to the 1% AEP, the storage would need to be tripled
- Even with the basin fully optimised, the local catchment downstream of the basin yields more runoff than the capacity of the 900mm diameter pipe, meaning some flooding impacts will still be potentially experienced downstream.
- The modest modifications (increasing the storage by 20% and modifying the outlet) did have some benefits at the outlet. If it was not realistic to achieve full optimisation of the basin (option 3), the modest works program would still provide benefits to downstream stakeholders.
- Future works should focus on the conveyance downstream of the basin.

1.5 Stakeholders

There are numerous stakeholders to this site. The stakeholders include:

- Wellington Shire Council;
- West Gippsland Catchment Management Authority (WGCMA);
- Department of Environment, Land, Water and Planning (DELWP);
- The development industry;
- Local residents and landowners.

2 Existing conditions

2.1 Current land use

The future Maffra development site is currently zoned 'Farming', reflecting agricultural uses. There is an existing development along Boisdale Street, which includes a sediment pond / storage basin at the outlet of the development. Figure 4 shows the existing conditions, highlighting some key features.

The Maffra area generally outfalls into the Macalister River, and onto the Thompson River.

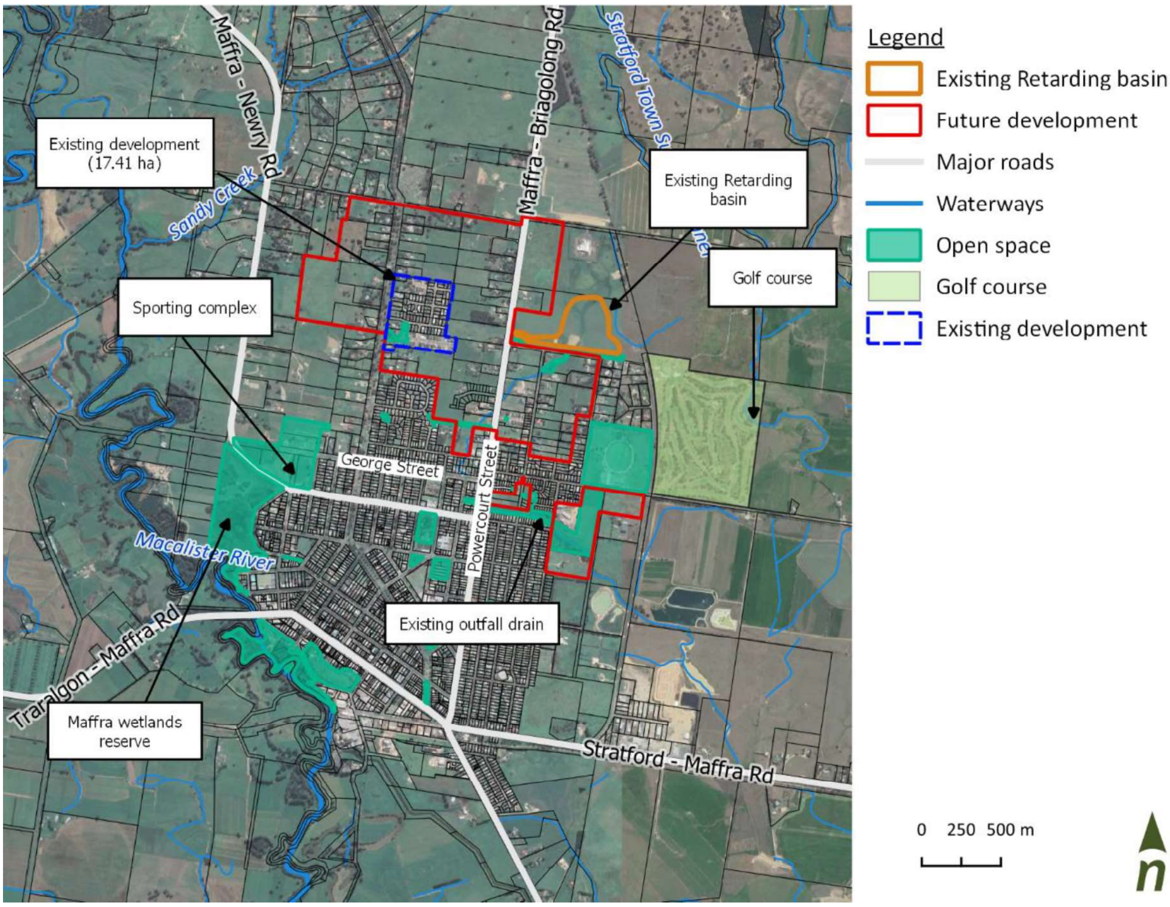

Figure 4. *Maffra proposed growth areas – existing conditions*

2.2 Site visit and opportunities

A site visit was conducted on 6th March 2020 to gain a better understanding of the local terrain, site constraints and opportunities. The site visit was attended by Alluvium staff Dan O'Halloran and Jenny Butcher, Kylee Smith of WMS and Sam Pye of Wellington Shire Council.

Several sites were identified as areas of interest by Council as possible locations for stormwater treatment assets (including the Maffra Recreational Reserve). Other sites visited included the existing RB, the CMA waterway which starts from the RB and flows south-west towards Powerscourt Street and the concrete channel taking flows east from Powerscourt Street out of town. Figure 5 shows the locations visited with descriptions following the map.

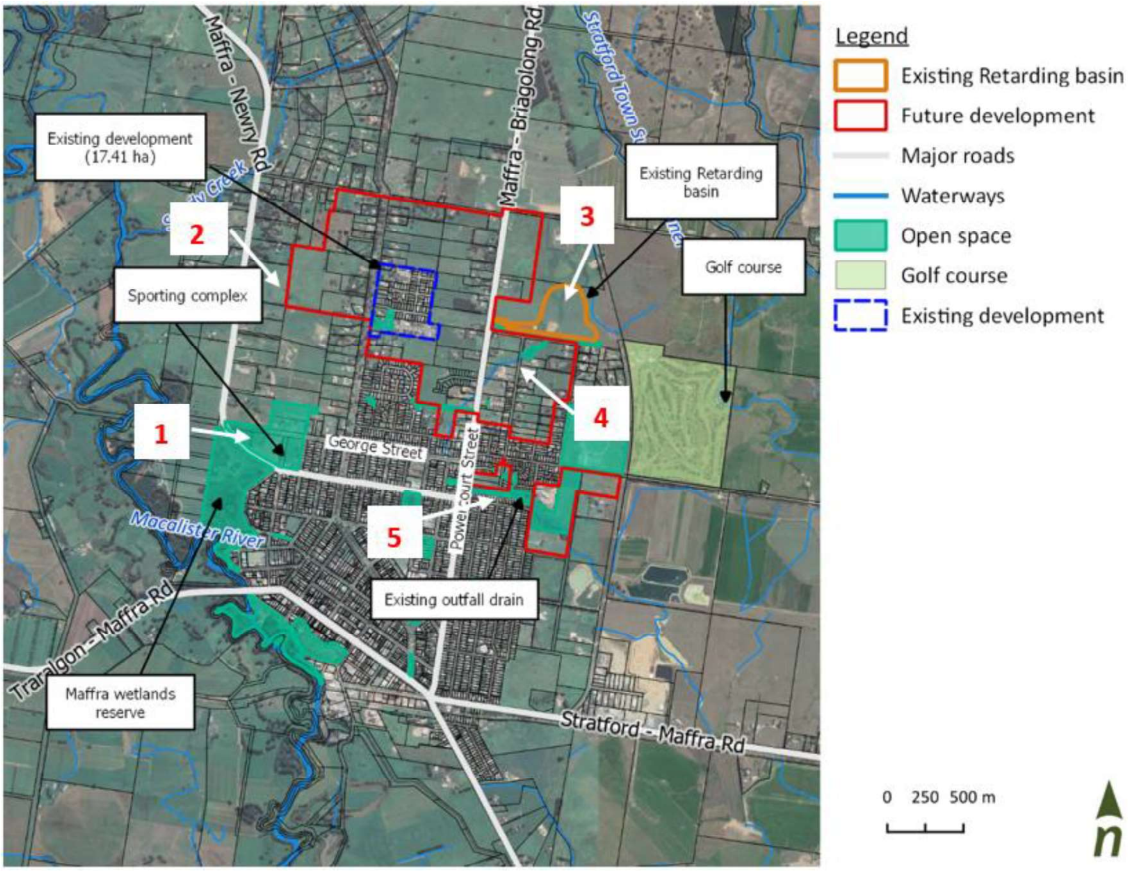

Figure 5. *Locations visited*

Location #1: Maffra showgrounds / Maffra recreation reserve

The first site visited was the existing waterbody that is adjacent to the Maffra showgrounds (Figure 6). This waterbody currently receives water from the catchment to the north-east, in particular via the George Street drain. The 450mm diameter George Street drain currently outfalls at an invert of 22.44 m AHD (Fisher Stewart proposed replacement drainage, longitudinal section, 1999). The waterbody outfalls south under Maffra-Newry Road and into the Maffra Wetlands Reserve. There is currently no treatment to the stormwater that discharges into this system.

There is an opportunity to formalise this waterbody into a functioning wetland, with reuse opportunities through irrigating the adjacent ovals (water is currently pumped from the waterbody to irrigate the ovals). This would benefit the downstream Maffra Wetlands as nutrient loads would be reduced.


Figure 6. Waterbody near Maffra showground and recreation reserve (left), and showing the existing pumping shed (right)

Location #2: Open paddock, north-west of Maffra township next to Maffra-Newry Rd
This site is located in a flat paddock that receives runoff from a hill that crests just to the west of Boisdale Street (Figure 7). Currently water drains through the site and sheet flows west towards the Macalister River through farmland. Given this area will receive runoff from future developable areas, there is a good opportunity to provide treatment and detention at this large, flat open space. A key consideration will be the mature trees that are present on site. These should be retained and protected.


Figure 7. *Looking east towards the hill (left) and south-east on Maffra-Newry Rd (right)*

Location #3: Maffra retarding basin
The Maffra retarding basin is located in the north-east of town and was constructed in the early 2000s following drainage advice from Fisher Stewart. The retarding basin was intended to mitigate flooding issues experienced on the north-east of the town. The catchment feeding into the RB is currently largely rural.

There is an opportunity at this site to increase the storage within the RB to provide a higher level of flood protection (as discussed in section 1.4). There is also an opportunity to provide stormwater treatment within this space. Given a portion of the catchment is proposed to be developed in the future, the storage and treatment requirements within this site will need to be addressed.

Figure 8. *Looking east to the RB outlet (embankment shown on the left of the photo)*

Location #4: Waterway downstream of Maffra retarding basin
There is a CMA-designated waterway which runs from the outlet of the RB south-west towards Powerscourt Street. It then crosses Powerscourt street and continues south to Merry Street and finishes and George Street, where flow continues down Alfred Street and within the George Street drain. The waterway is quite undefined and very shallow in some reaches (Figure 9), as well as being situated very close to private property. As established in previous studies, the conveyance through this system needs to be increased to help mitigate flooding.

Figure 9. *Looking north from Merry Street indicating a very shallow waterway (left) and looking south from Merry Street to the drainage easement between Merry Street and George Street (right)*

The drainage easement from Merry Street to George Street is very narrow (~9.5m) and includes going through a private landholder parcel. A small pipe (300mm diameter) runs underneath this easement.

Location #5: Concrete channel (Davis Street drain), east of Powerscourt Street
From George Street stormwater flows south along Alfred Street, then heads east through a narrow concrete channel and larger grassed floodway arrangement (Figure 10 and Figure 11). The waterway crosses Powerscourt Street again through a series of shallow culverts (Figure 12). East of Powerscourt Street there is again a narrow concrete channel and wide floodway, which presents a good opportunity for naturalising the waterway (Figure 13 and Figure 14). The culverts on the downstream side of Landy Street appear to be

accumulating with sediment and vegetation (Figure 15). East of Landy Street the drain is a narrow earthen drain.

Figure 10. *Looking north up Alfred Street towards the Merry Street drainage easement (left), and the entry into the concrete channel/grassed floodway (right)*

Figure 11. *Looking east from Alfred Street at the concrete channel/grassed floodway arrangement*

Figure 12. *Powerscourt Street culverts for flow heading east out of town (left), and looking back east to Alfred St (right)*

Figure 13. *Looking east at Powerscourt Street showing a narrow concrete channel and wide grassed floodway*

Figure 14. *Looking west at Landy Street (left) and the culvert entry and Landy Street (right)*

Figure 15. *The Landy Street culverts indicating sedimentation and vegetation of the downstream side reducing conveyance (left), and looking east on Landy Street (right)*

2.3 Topography

Figure 16 shows the topography across the Maffra growth area and the region more broadly. Elevation ranges from 38 m AHD along the eastern boundary of the site at Fulton Road, to 57.0 m AHD at the northern boundary of the site at Sandy Creek Road. The site generally falls in a southern direction. The site has grades varying from 0.5%-7.0%.

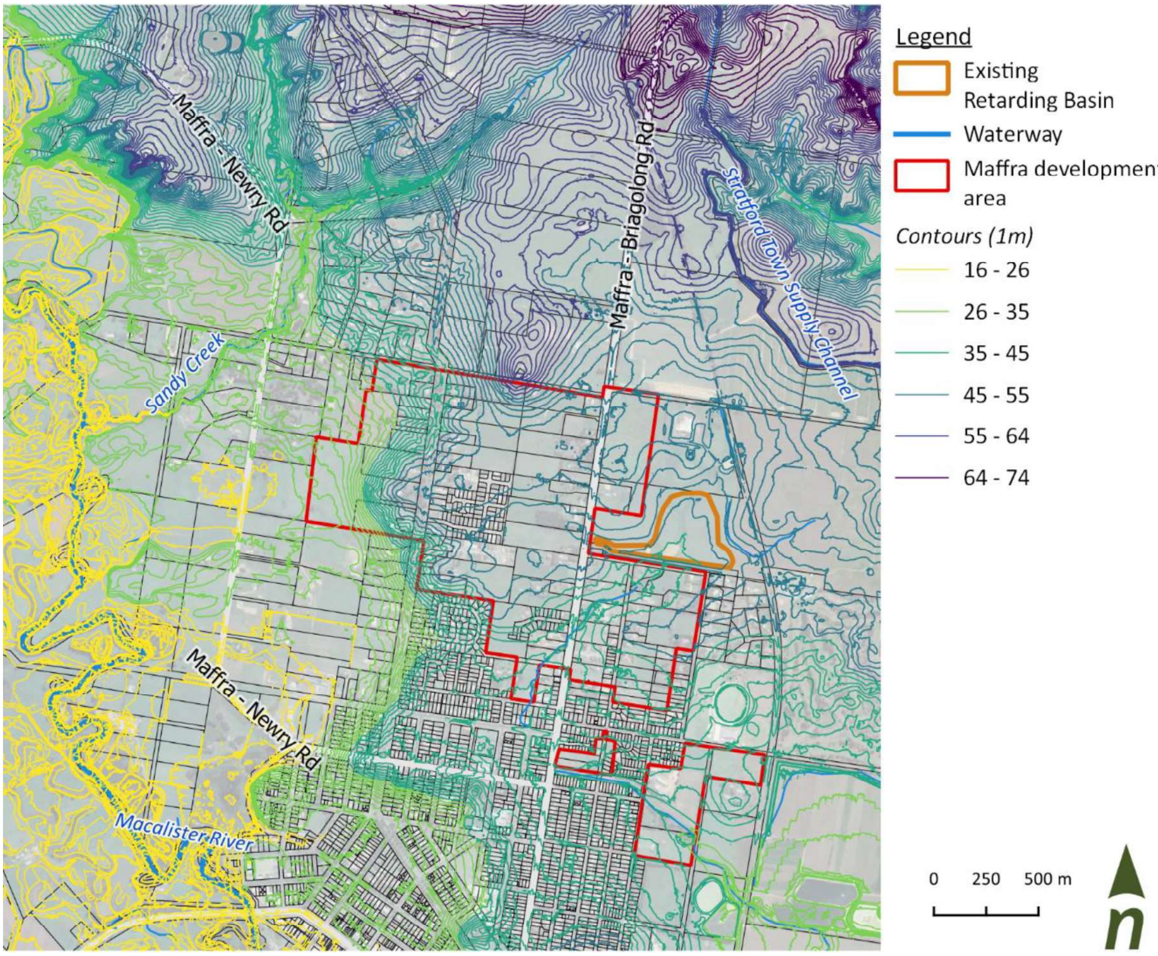

Figure 16. *Topography of Maffra and surrounds*

2.4 Existing services and infrastructure

Figure 17 shows the existing stormwater pipe network through town. Major outfall locations include the George Street drain into the existing waterbody next to the Maffra showgrounds and the Davis Street drain which runs east from Powerscourt Street. Future development will need to connect with the existing stormwater network. The George Street drain already has capacity issues as determined in the Maffra Drainage Review (Cardno, October 2010).

Other key infrastructure includes the Maffra Retarding Basin (detailed below), and the existing retarding basin built as part of the Boisdale development. Other existing services (sewer, water, gas etc.) are not shown on this map and will need to be considered in any future options development.

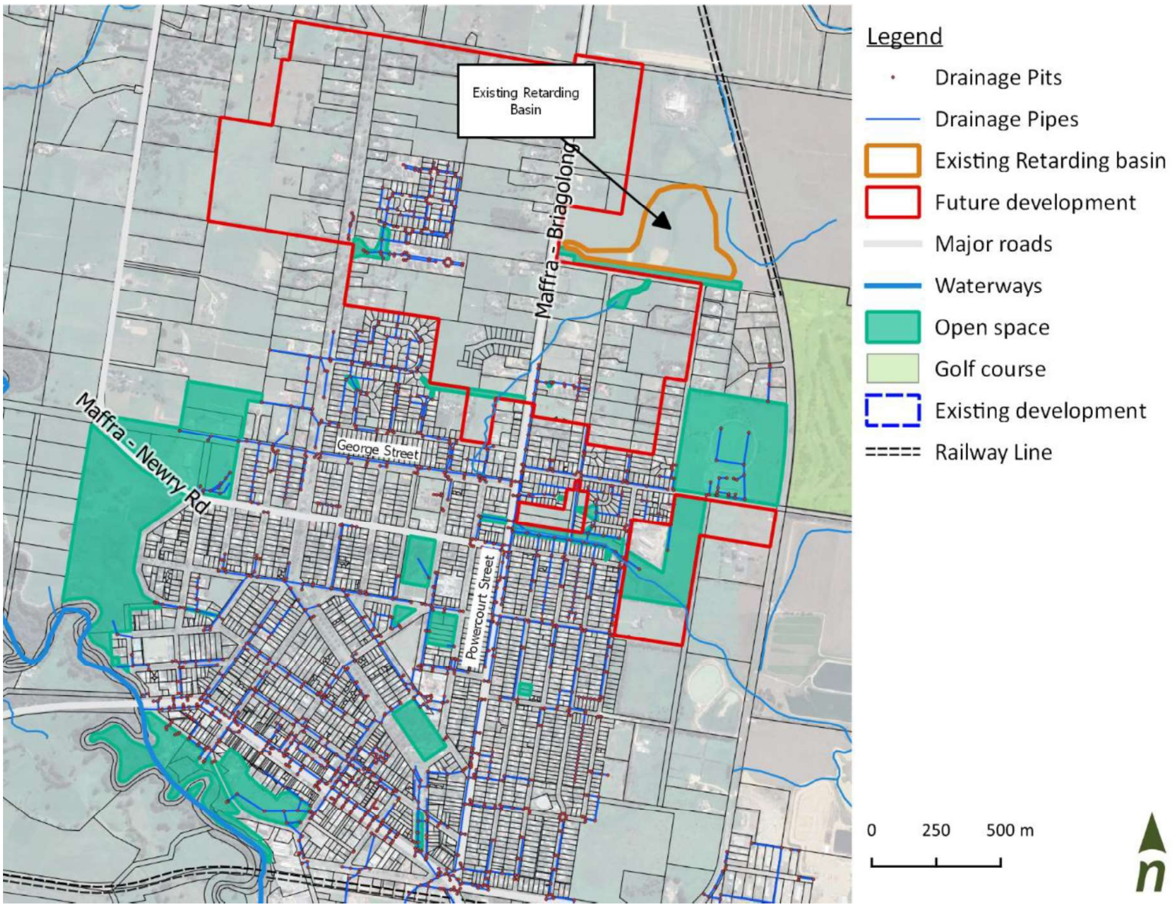

Figure 17. *Existing services and infrastructure through the Maffra development area and surrounds*

Maffra retarding basin

The existing Maffra Retarding Basin (RB) is located east of Maffra-Briagolong Rd and south of Brewers Hill Rd. The upstream catchment is predominately rural, with the existing Maffra residential housing located immediately downstream of the retarding basin. The retarding basin has a peak storage capacity of 112,000 m^3 (at 45.6 m AHD) and a storage depth of 2.45 m (according to LiDAR). The retarding basin is designed to control stormwater runoff for events up to and including the 20 year ARI event. The basin does little to manage the 100 year ARI event. The retarding basin is controlled by two 18m long 600mm outlet pipes (upstream invert 43.15 m AHD at a grade of 2.5%), with an overflow weir activated during events greater than the 20 year ARI. The overflow weir has a width of 5m, it is built of rock gabions and has an energy dissipating stair step along the downstream side of the embankment wall. The dimensions have been obtained from Fisher Stewart's retarding basin design drawings, and Maffra Stormwater Drainage Memo confirming key design dimensions following construction (3rd March 2003). We note that the weir width is different to that adopted by both Cardno and Water Technology in their assessments (10m and 24m respectively).

Downstream of the retarding basin, flows are conveyed in a south west direction under Powerscourt Street and heads south to Merry Street and George Street. A 900mm pipe at Powerscourt Street conveys a portion of the runoff from the retarding basin down Powerscourt Street. When flows the Powerscourt Street drains capacity, runoff continues to the west of Powerscourt Street and discharges through a shallow open channel to Merry Street, before eventually outfalling back across Powerscourt Street further south.

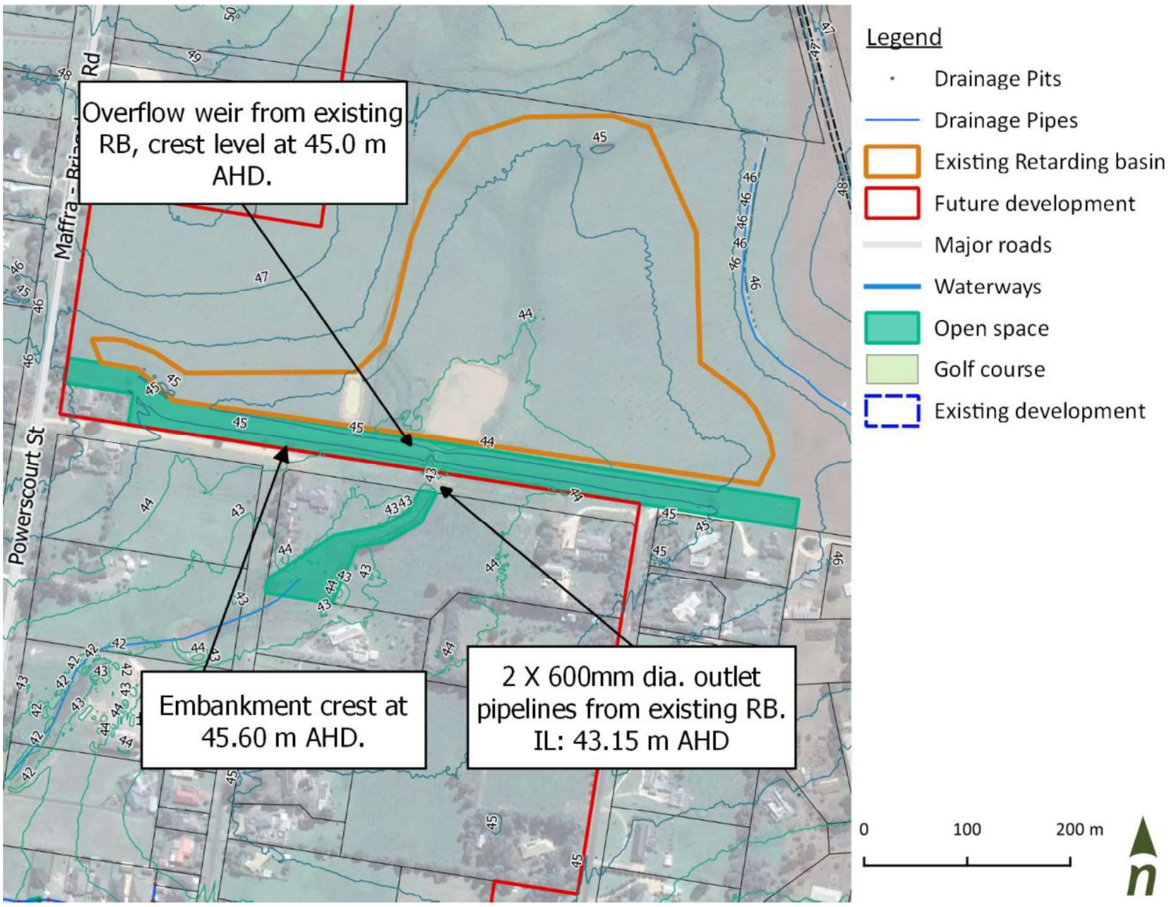

Figure 18. *Existing conditions – Retarding Basin*

The Cardno and Water Technology reports analysed the existing capacity of the retarding basin, and the storage and outfall requirements needed for the RB to control stormwater back to the existing 1% AEP flows. These assessments are summarised in section 1.4. The retarding basin storage was revised as part of this assessment to establish existing conditions peak inflows and outflows.

2.5 Catchments

The site is located within the Thomson River catchment, which flows in a south-easterly direction (Figure 19). The catchment is generally rural with some urbanised areas.

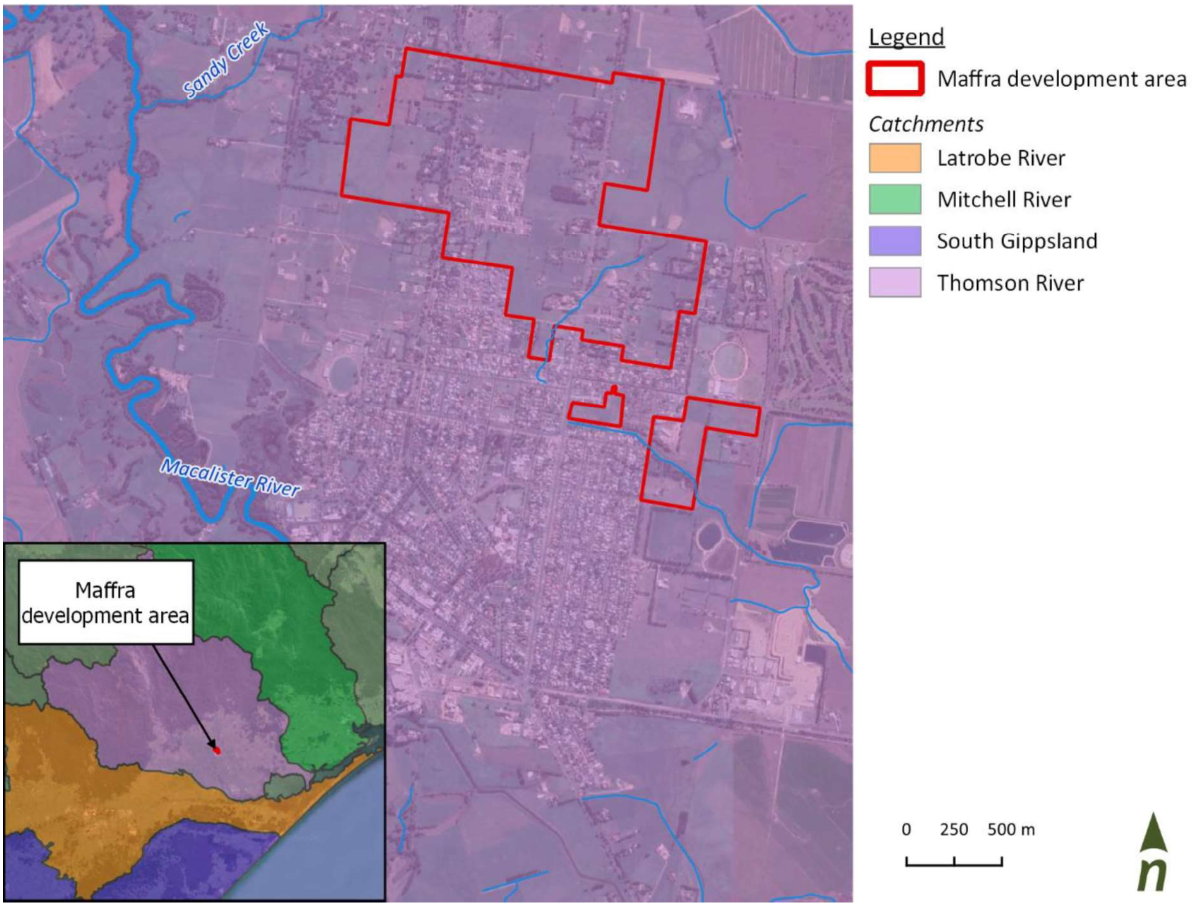

Figure 19. *Catchment context*

The Maffra development area can be described as generally draining in a south-westerly and south-easterly direction. The western catchment eventually outfall into the Macalister River. The western catchment has been divided into a north-western and south-western catchment. The eastern catchment flows through the Davis Street drain out of the township.

The sub-catchments of the Maffra development site are shown in Figure 20. These sub-catchments are discussed further in Section 4.

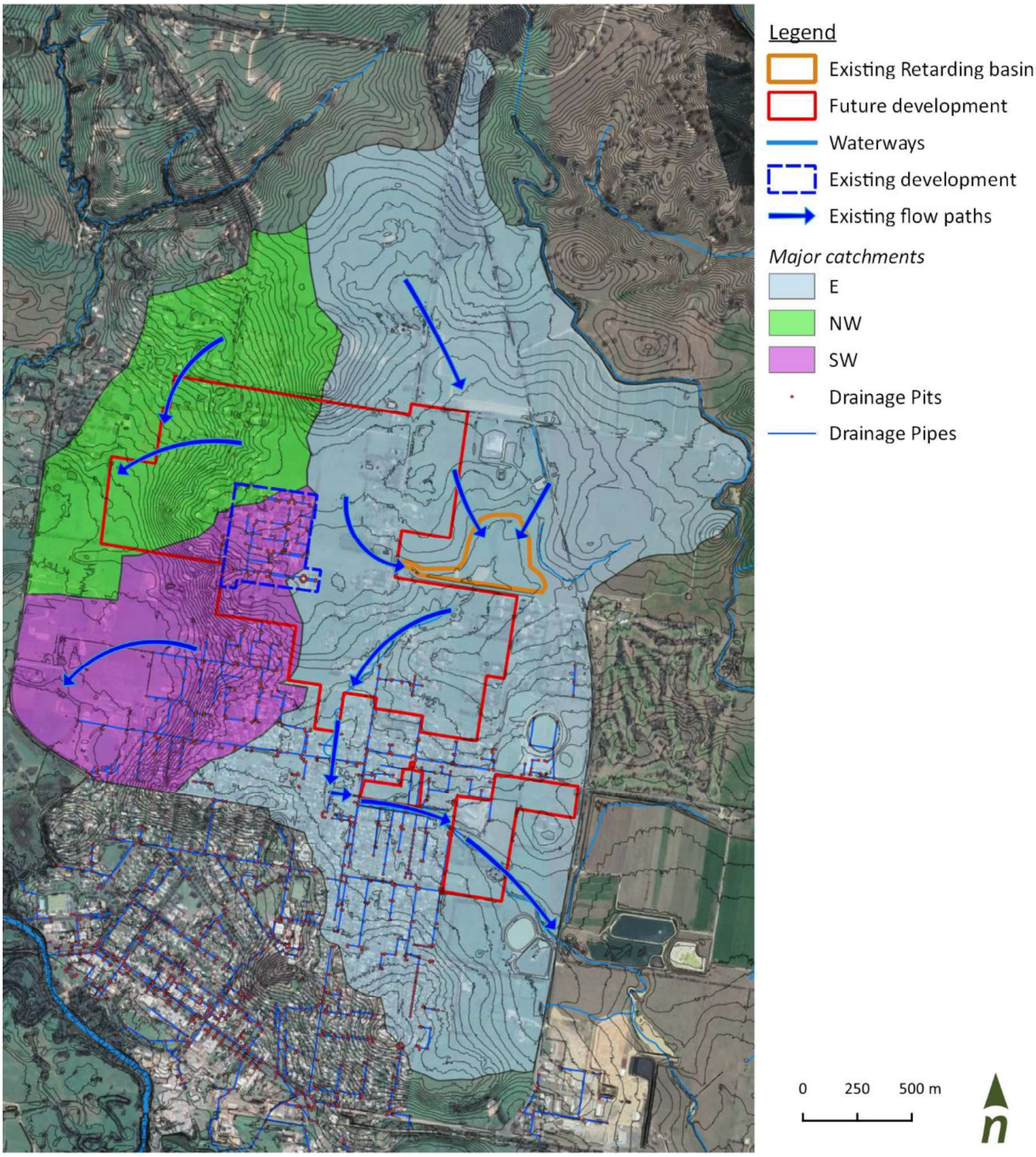

Figure 20. *Sub-catchments of the Maffra development area*

2.6 Broader flooding context

The best available information for Macalister River comes from the West Gippsland Floodplain Management Strategy 2018-2027. Figure 21 shows the 1% AEP flood extent along Macalister River.

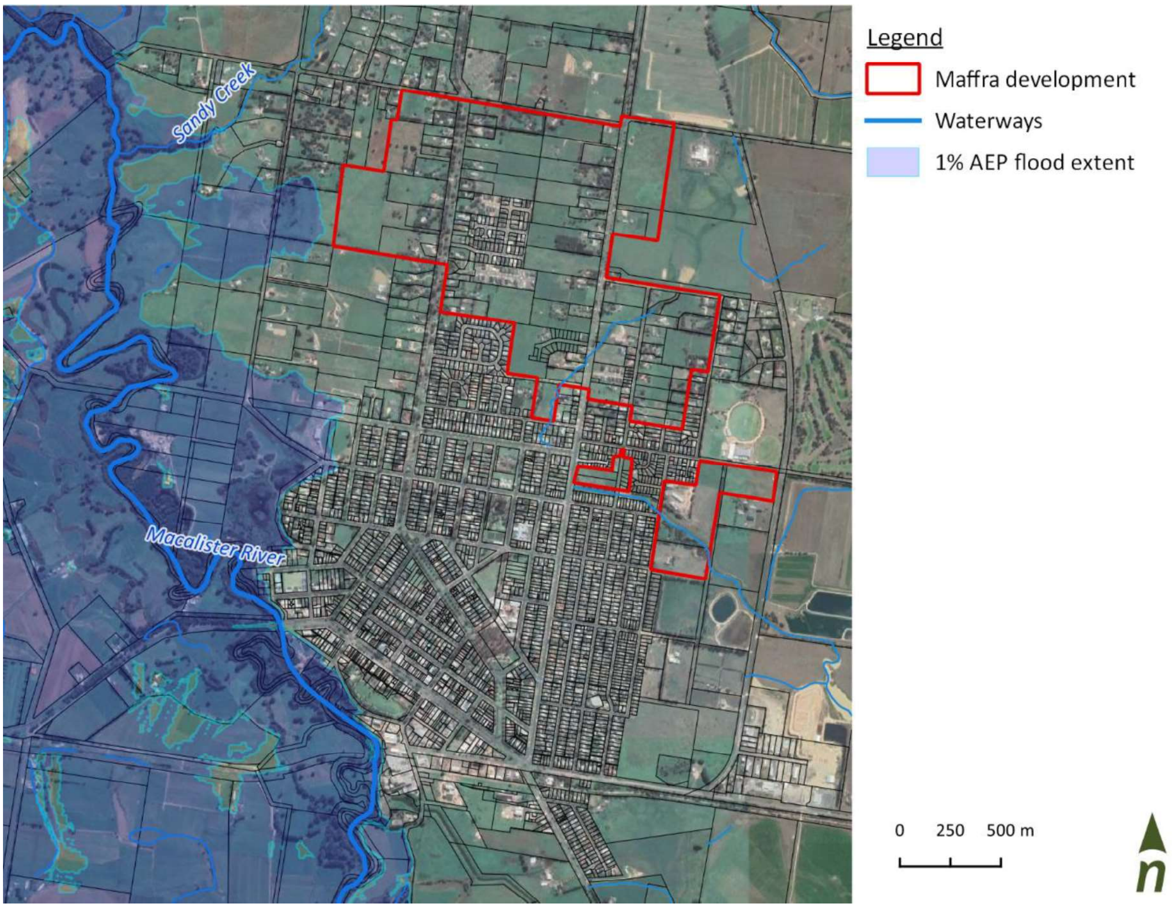

Figure 21. *1% AEP flood extent adjacent to the Maffra site*

2.7 Existing conditions flood modelling

Water Modelling Solutions (WMS) undertook the flood modelling as part of this project. The flood modelling component of the project involves investigation and mapping of existing conditions for the 20% and 1% Annual Exceedance Probability (AEP) events as well as support for investigation of mitigation options for the township flooding under 20%, 1%, Probable Maximum Flood (PMF) an Climate Change events. The outcome of the project will be the development of sufficient flood information such that Council can undertake effective floodplain management and the information can be used by a variety of stakeholders for land use planning, flood management planning, treatment and mitigation.

The full flood modelling report is included in Appendix E, with all inputs, assumptions and results documented. This section summarises some key findings for the existing conditions.

Hydraulic modelling has been undertaken for the Maffra Township utilising rainfall-excess hydrology supplied by Alluvium (RORB modelling). The modelling utilised the industry standard software, TUFLOW with a 1-dimensional drainage network connected to a 2-dimensional terrain. A range of events were modelled for both the existing and developed scenarios including sensitivity scenarios for PMF and Climate Change for 2100 RCP4.5 and RCP8.5 for the developed scenario. Three indicative temporal patterns – front, mid and rear loaded, were chosen to represent the ensemble modelling as recommended in ARR2019.

Figure 22 to Figure 24 provide the 1% AEP existing conditions mapping. For the 20% AEP, water surface levels and velocity mapping see the full flood modelling report.

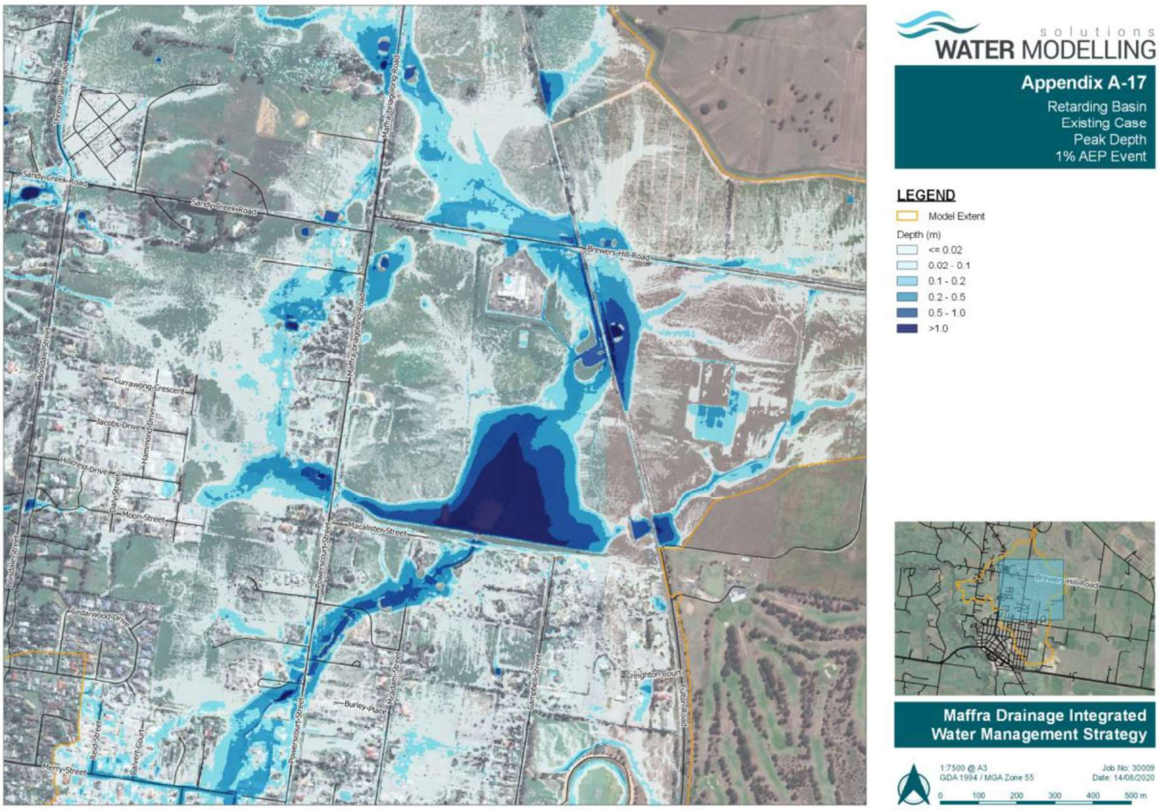

Figure 22. *Existing conditions flood modelling – Maffra RB - 1% AEP*

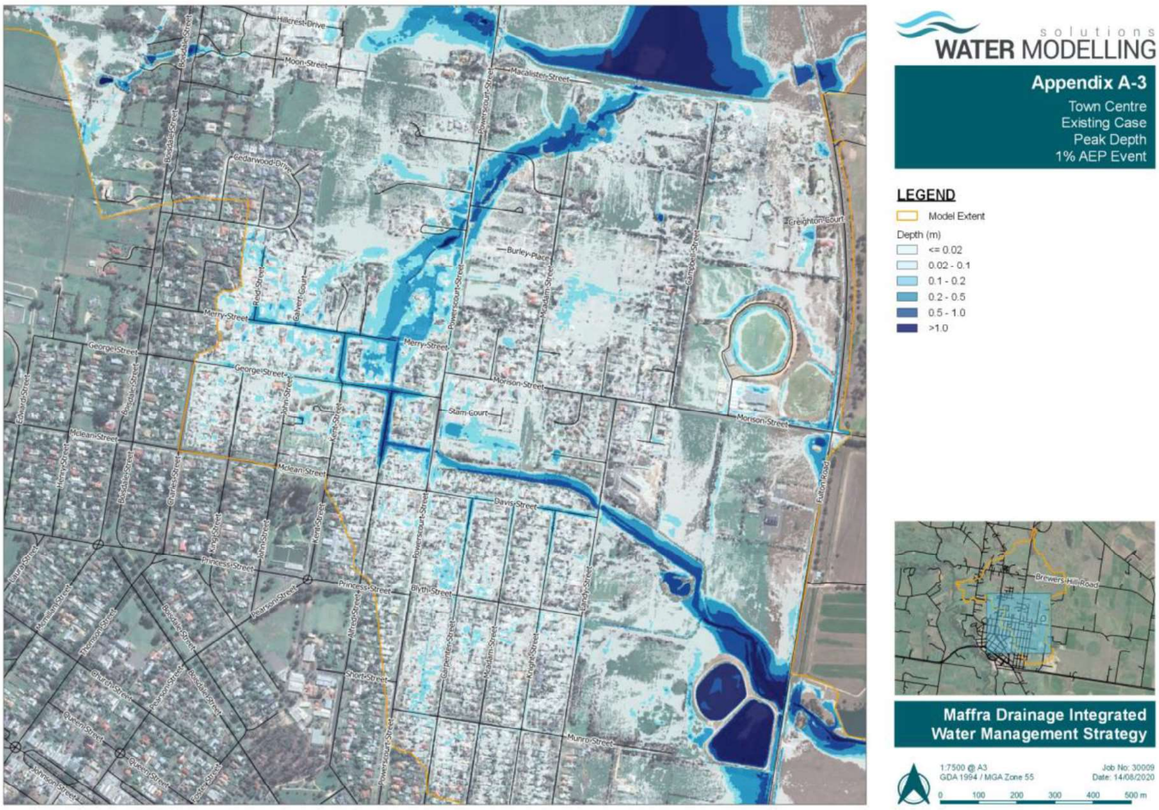

Figure 23. *Existing conditions flood modelling – Town Centre - 1% AEP*

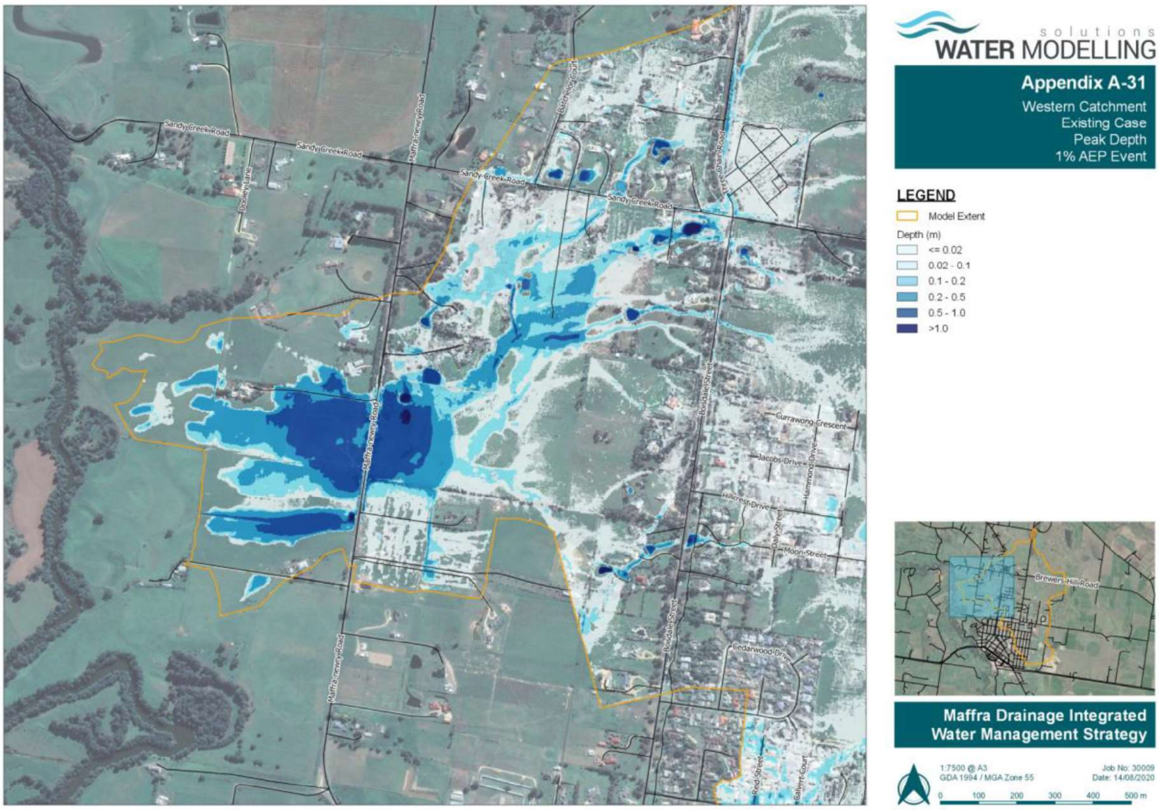

Figure 24. *Existing conditions flood modelling – Western catchment- 1% AEP*

Summary of results:

- Under existing conditions for the 1% AEP event, the majority of flooding is occurring from the north east along the ephemeral stream from the location of the basin. In some locations flood depths along the ephemeral stream exceed one metre.
- There is significant pooling of water along Alfred Street prior to the flows turning east and following the Davis Street Drain downstream to the outlet of the model at Fulton Road.
- Throughout the remainder of the township, flooding is relatively shallow overland flows due to local catchment rainfall with depths typically less than 100mm.
- Flood behaviour under existing conditions for the 20% AEP event is similar with a lesser degree of severity.

2.8 Site Values

Throughout the Maffra site, the Ecological Vegetation Class (EVC) of the site is classed as plains grassy woodland and plains grassy forest (Figure 25). To the west of the site, near the Macalister River exists floodplain riparian woodland, with billabong wetland aggregates along the Macalister River. There are obviously areas within this that would be heavily modified.

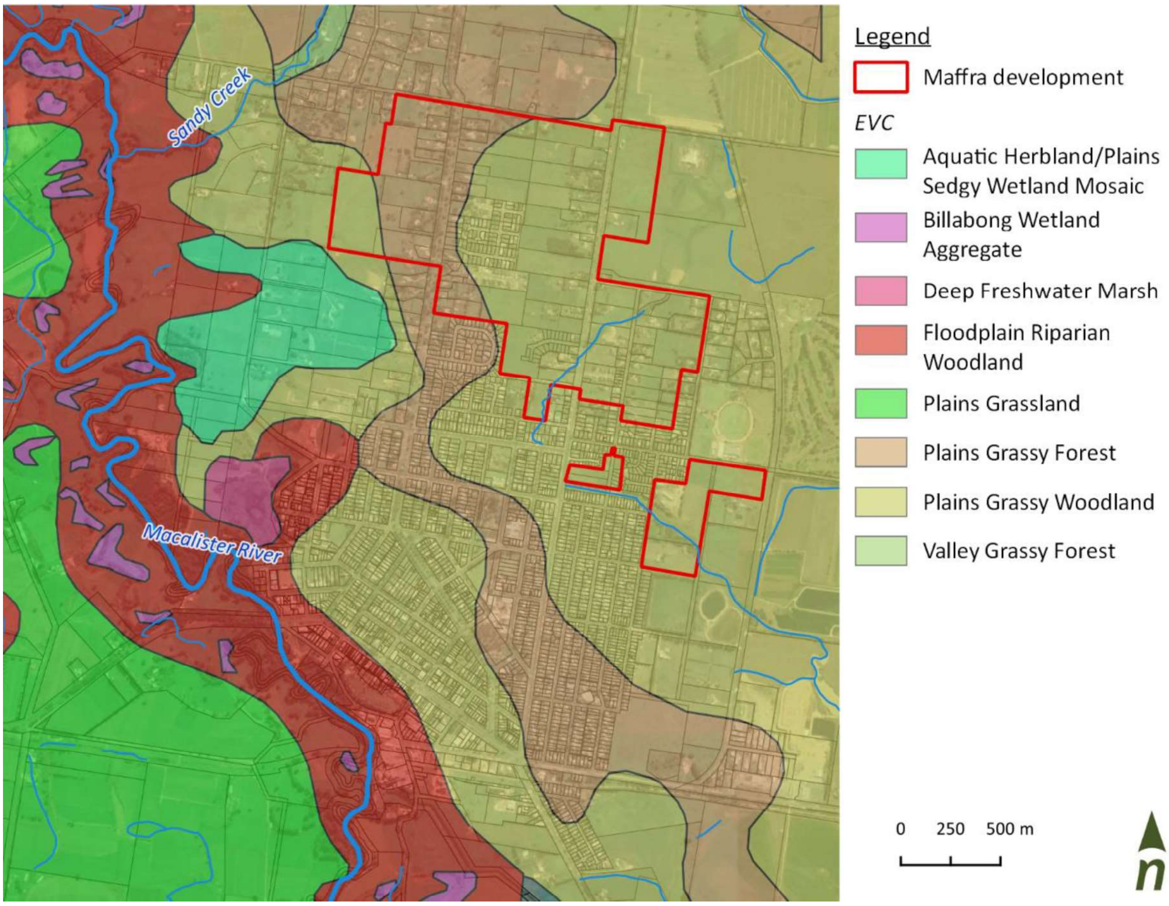

Figure 25. *EVCs within the Maffra area and surrounds*

3 Post development objectives and conditions

The following sets out the aim, objectives and approach of the drainage assessment for the post-development conditions.

3.1 Aim

For any drainage assessment the aim is to define the flood mitigation and stormwater quality management requirements for the post development conditions (the future land use of the site). In doing so, the work will define the stormwater quantity and stormwater quality assets required to control the impact of development on downstream receiving environments, and comment upon the optimal layout of those assets to support complimentary water cycle objectives.

The design and layout of the proposed treatment assets are provided at a conceptual level.

3.2 Objectives and approach

There are four main objectives of any surface water management plan:

1. **Stormwater quantity management**

Fully developed 1% AEP stormwater runoff rates are to be retarded back to the equivalent 1% AEP pre-development peak flow rates before discharging downstream. This is typically achieved through the implementation of retention (or detention) systems within the catchment.

This assessment focuses on this aspect of drainage assessment requirements.

2. **Stormwater conveyance**

Stormwater conveyance is typically designed to a major and minor flow regime where:

- Minor flows i.e. up to and including the 20% AEP storm event (approximately the 1 in 5-year ARI event), are conveyed via the sub-surface stormwater network.
- Major flows i.e. between the 20% AEP and 1% AEP event are conveyed on the surface via roadways and waterways.

The entire pipe and road network has not been assessed as part of this this assessment, however proposed waterway enhancements/naturalisations have been assessed to convey the 1% AEP. In addition to this, the flood modelling establishes the flood extent, depth and safety risk along roads.

3. **Stormwater quality treatment**

Stormwater treatment concepts are required to meet the State Environmental Policy (SEPP) best practice environmental management (BPEM) pollution reduction targets before being discharged into drainage networks and into receiving waters. These targets are defined as:

- 70% removal of the total Gross Pollutant load
- 80% removal of total Suspended Solids (TSS)
- 45% removal of total Nitrogen (TN)
- 45% removal of total Phosphorus (TP)

This assessment focuses on this aspect of drainage assessment requirements.

4. Integrated water management

Drainage assessments should seek to incorporate IWM opportunities, in line with the Shire of Wellington's IWM plan (2019/20) . The vision and outcomes of that plan are described in section 8 below. This plan includes an assessment of IWM opportunities associated with the proposed treatment and flood mitigation assets including stormwater harvesting and channel naturalisation opportunities.

3.3 Future land use

To determine the stormwater quality requirements of the precinct, the post-development conditions of the site are modelled. While it is understood that the layout and proposed land use concepts are subject to change over time, the assumption all future development will be residential has been adopted to be the basis of the modelling.

The layout of the precinct and specifically the density of proposed development and the proportion of open space will impact the volume of stormwater runoff and therefore the treatment and flood mitigation systems required. In the first instance, adopting the assumption of a residential land use is a reasonable approach in lieu of any development masterplans.

4 Catchment analysis

With an understanding of existing site conditions, drainage infrastructure, existing flood issues, and the proposed development area, an analysis was undertaken to define treatment and detention opportunities, and their corresponding catchments and land uses. The Maffra site catchments were determined using the LiDAR data provided by WGCMA. It is important to map these catchments to understand the pollutant loads generated off them (discussed in section 6). This was previously shown in section 2.5.

As stated previously, in lieu of a development masterplan the development has been assumed to be general residential. For the purposes of surface water modelling, each land use type assumes a fraction impervious. The fraction impervious assumes the proportion of land that is likely to be impervious or paved. This impacts the volume of stormwater runoff generated in a specific rainfall event for a specified land size.

The adopted fraction impervious values have been summarised in Table 1. The land uses include those outside of the development area.

Table 1. Adopted fraction impervious values for each proposed land use type

PSP proposed Land use	Adopted zone description	Adopted zone code	Fraction imperviousness
Medium Density Residential	General Residential Zone –Standard densities (Allotment size 300-600 m^2)	GRZ	0.60
Low Density Residential	Allotment size >1001 m^2	LDRZ	0.10 - 0.20
Road Zone	Major roads and freeways	RDZ1	0.70
Rural Zone	Agricultural / Farm land	RUZ	0.05
Local Park / Open Space	Public Park and Recreation Zone	PPRZ	0.05 - 0.10

4.1 Sub-catchments

Based on the existing topography and the Maffra township, the eastern catchment of the Maffra development site was divided into 24 sub-catchments (Figure 26), and 20 sub-catchments for the western catchment (Figure 27) . The area of each sub-catchment and the fraction imperviousness are summarised in Table 2 & Table 3. This catchment information was used for the treatment modelling inputs, in order to determine the target pollutant reduction load required for the Maffra development. The existing Maffra township and the upstream rural area are not included in the treatment requirements. The sub-catchment information was also used as inputs for the hydrologic modelling (Section 5), which informed the flood modelling.

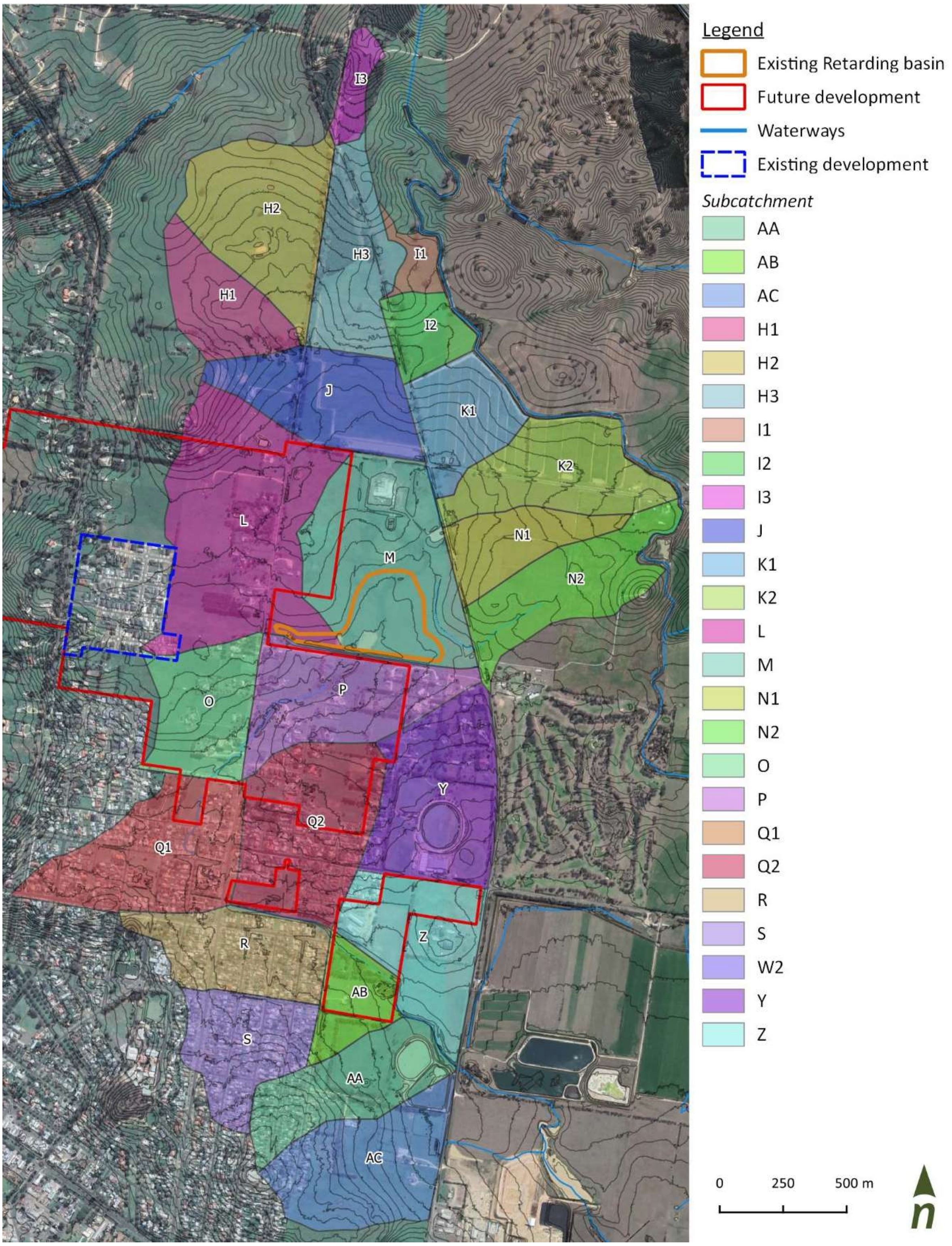

Figure 26. *Sub-catchment layout of the eastern Maffra catchment*

Table 2. Developed conditions effective imperviousness area by sub-catchment (EAST)

Sub-Catchment	Proposed land use	Area (ha)	Fraction impervious	Effective impervious area (ha) (Area x Fraction impervious)
AA	General Residential	5.23	0.60	3.14
	Roads	2.95	0.70	2.07
	Rural Zone	13.91	0.05	0.70
	Subtotal:	**22.09**	**0.27**	**5.90**
AB	General Residential	5.95	0.60	3.57
	Roads	1.32	0.70	0.92
	Rural Zone	2.94	0.05	0.15
	Open Space	0.78	0.05	0.04
	Subtotal:	**10.99**	**0.43**	**4.68**
AC	General Residential	2.49	0.10	1.50
	Roads	1.50	0.70	1.05
	Rural Zone	24.24	0.05	1.21
	Subtotal:	**28.24**	**0.13**	**3.76**
H1	Open Space	16.33	0.05	0.82
	Subtotal:	**16.33**	**0.05**	**0.82**
H2	Roads	28.78	0.05	1.44
	Rural Zone	1.63	0.70	1.14
	Subtotal:	**30.40**	**0.08**	**2.58**
H3	Roads	1.62	0.70	0.98
	Rural Zone	19.70	0.05	1.14
	Subtotal:	**21.32**	**0.10**	**2.12**
I1	Rural Zone	21.32	0.05	0.21
	Subtotal:	**21.32**	**0.05**	**0.21**
I2	Rural Zone	8.97	0.05	0.45
	Subtotal:	**8.97**	**0.05**	**0.45**
I3	Roads	1.45	0.70	1.02
	Rural Zone	3.78	0.19	0.19
	Subtotal:	**5.23**	**0.23**	**1.20**
J	Roads	1.39	0.70	0.97
	Rural Zone	23.28	0.05	1.16
	Subtotal:	**24.67**	**0.09**	**2.14**
K1	Roads	0.75	0.70	0.53
	Rural Zone	15.43	0.05	0.73
	Subtotal:	**16.19**	**0.08**	**1.30**
K2	Roads	2.07	0.7	1.45
	Rural Zone	22.22	0.05	1.11
	Subtotal:	24.29	0.11	2.56
L	General Residential	38.61	0.60	23.16
	Roads	4.51	0.70	3.16
	Rural Zone	10.41	0.05	0.52
	Subtotal:	**53.53**	**0.50**	**26.84**

Sub-Catchment	Proposed land use	Area (ha)	Fraction impervious	Effective impervious area (ha) (Area x Fraction impervious)
M	General Residential	6.37	0.60	3.82
	Roads	0.59	0.70	0.41
	Rural Zone	35.51	0.05	1.78
	Subtotal:	**42.46**	**0.14**	**6.01**
N1	Roads	1.15	0.70	0.80
	Rural Zone	12.78	0.05	0.64
	Subtotal:	**13.93**	**0.10**	**1.44**
N2	Roads	1.25	0.70	0.88
	Rural Zone	21.99	0.05	1.10
	Subtotal:	**13.93**	**0.10**	**1.97**
O	General Residential	18.96	0.60	11.38
	Roads	1.20	0.70	0.84
	Subtotal:	**20.16**	**0.61**	**12.22**
P	General Residential	18.93	0.60	11.36
	Low Density Res	3.45	0.20	0.69
	Roads	3.55	0.70	2.48
	Rural Zone	1.72	0.05	0.09
	Subtotal:	**27.65**	**0.53**	**14.62**
Q1	General Residential	21.78	0.60	13.07
	Roads	9.78	0.70	6.85
	Subtotal:	**31.57**	**0.63**	**19.92**
Q2	General Residential	28.25	0.60	16.95
	Roads	4.01	0.70	2.80
	Open Space	0.99	0.05	0.05
	Subtotal:	**33.26**	**0.60**	**19.81**
R	General Residential	14.57	0.60	8.74
	Roads	5.73	0.70	4.01
	Open Space	0.70	0.05	0.03
	Subtotal:	**20.99**	**0.61**	**12.78**
S	General Residential	14.07	0.60	8.44
	Roads	5.60	0.70	3.92
	Subtotal:	**19.67**	**0.63**	**12.36**
Y	General Residential	1.32	0.60	0.79
	Low Density Res	10.91	0.20	2.18
	Roads	3.76	0.70	2.63
	Open Space	15.66	0.10	1.57
	Subtotal:	**31.64**	**0.23**	**7.17**
Z	General Residential	12.09	0.60	7.25
	Roads	2.13	0.70	1.49
	Rural Zone	11.51	0.05	0.58
	Subtotal:	**25.73**	**0.36**	**9.32**
	Total:	**556.75**		

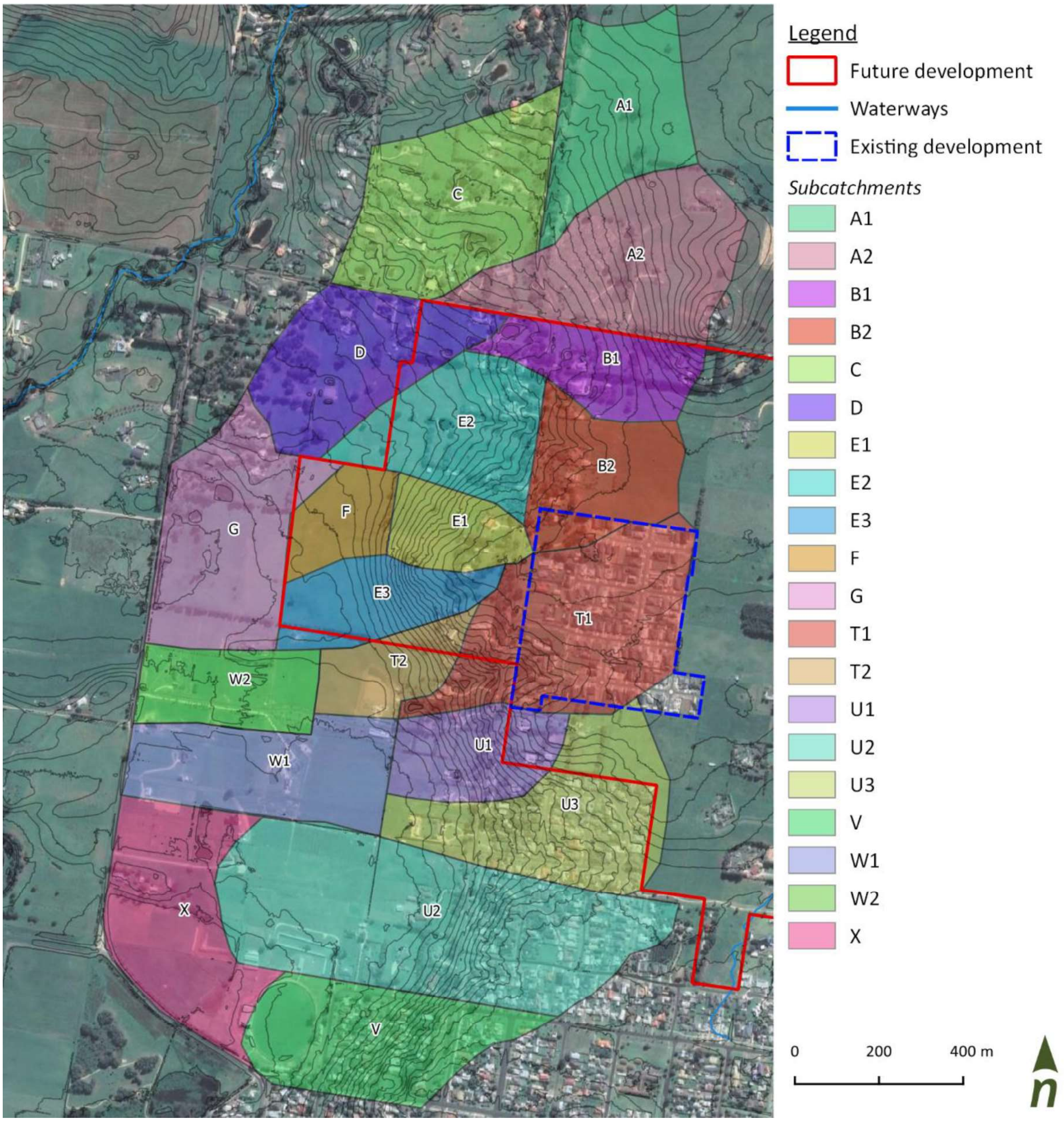

Figure 27. *Sub-catchment layout of the western Maffra catchment*

Table 3. Developed conditions effective imperviousness area by sub-catchment (WEST)

Sub-Catchment	Proposed land use	Area (ha)	Fraction impervious	Effective impervious area (ha) (Area x Fraction impervious)
A1	Roads	1.75	0.70	1.22
	Rural Zone	11.00	0.05	0.55
	Subtotal:	**12.75**	**0.14**	**1.77**
A2	Roads	2.25	0.70	1.57
	Rural Zone	15.32	0.05	0.77
	Subtotal:	**17.57**	**0.13**	**2.34**
B1	General Residential	7.33	0.60	4.40
	Roads	0.81	0.70	0.57
	Subtotal:	**8.14**	**0.61**	**4.96**
B2	General Residential	9.24	0.60	5.54
	Roads	1.18	0.70	0.82
	Subtotal:	**10.41**	**0.61**	**6.36**
C	Low Density Res	14.73	0.10	1.47
	Roads	2.37	0.70	1.66
	Rural Zone	0.14	0.05	0.01
	Subtotal:	**17.24**	**0.18**	**3.14**
D	General Residential	2.59	0.60	1.56
	Roads	0.03	0.70	0.02
	Rural Zone	11.03	0.05	0.55
	Subtotal:	**13.65**	**0.16**	**2.13**
E1	General Residential	5.20	0.60	3.12
	Roads	0.43	0.70	0.30
	Subtotal:	**5.64**	**0.61**	**3.43**
E2	General Residential	9.45	0.60	5.67
	Roads	1.06	0.70	0.74
	Rural Zone	1.49	0.05	0.07
	Subtotal:	**12.00**	**0.54**	**6.49**
E3	General Residential	7.23	0.60	4.34
	Rural Zone	0.80	0.05	0.04
	Subtotal:	**8.03**	**0.55**	**4.38**
F	General Residential	5.04	0.60	3.03
	Subtotal:	**5.04**	**0.60**	**3.03**
G	General Residential	0.76	0.60	0.45
	Roads	0.56	0.70	0.39
	Rural Zone	15.15	0.05	0.76
	Subtotal:	**16.48**	**0.10**	**1.61**
T1	General Residential	16.43	0.60	9.86
	Roads	2.05	0.70	1.44
	Rural Zone	1.70	0.05	0.08
	Subtotal:	**20.18**	**0.56**	**11.38**
T2	General Residential	1.21	0.60	0.73

Sub-Catchment	Proposed land use	Area (ha)	Fraction impervious	Effective impervious area (ha) (Area x Fraction impervious)
	Rural Zone	4.57	0.05	0.23
	Subtotal:	**5.79**	**0.17**	**0.96**
U1	General Residential	2.50	0.60	1.50
	Roads	1.41	0.70	0.99
	Rural Zone	4.16	0.05	0.21
	Subtotal:	**8.08**	**0.33**	**2.70**
U2	General Residential	11.57	0.60	6.94
	Roads	1.94	0.70	1.36
	Rural Zone	18.85	0.05	0.94
	Subtotal:	**32.36**	**0.29**	**9.24**
U3	General Residential	13.56	0.60	8.14
	Roads	0.89	0.70	0.62
	Rural Zone	2.35	0.05	0.12
	Subtotal:	**16.79**	**0.53**	**8.87**
V	General Residential	5.02	0.60	3.01
	Roads	5.67	0.70	3.97
	Open Space	3.82	0.10	0.38
	Subtotal:	**14.50**	**0.51**	**7.36**
W1	Roads	0.15	0.70	0.11
	Rural Zone	13.45	0.05	0.67
	Subtotal:	**13.60**	**0.06**	**0.78**
W2	Roads	0.18	0.70	0.13
	Rural Zone	7.75	0.05	0.39
	Subtotal:	**7.93**	**0.06**	**0.52**
X	Roads	0.39	0.70	0.27
	Rural Zone	15.10	0.05	0.75
	Subtotal:	**15.49**	**0.07**	**1.03**
	Total:	**253.58**		

5 Stormwater quantity – hydrologic analysis

The hydrologic analysis of the Maffra development site was undertaken to determine the pre and post-development peak runoff flow rates (m^3/s) for various flood events throughout the catchment. The hydrologic analysis is used to determine the storage capacities of proposed retarding basins required to retard the fully developed peak stormwater runoff rates back to pre-developed conditions, and to determine the flows entering the stormwater quality treatment wetlands proposed. The hydrology results are also used as inputs for the flood modelling.

5.1 Hydrologic modelling

The hydrologic analysis was undertaken using RORB (v6.31), which is a runoff-routing software designed to simulate attenuation and time of concentrations to produce flood estimates at specified catchment locations.

A RORB model was created for the Maffra site to determine:

- Existing peak flows
- The impact of development on peak flows
- The reduction in peak flows that is possible using retarding basin storage etc.
- The impact of climate change on peak flows

The RORB models were built by delineating the major catchments into sub-areas based on topography and potential road alignments. The catchments, reach lengths and nodes used to build the RORB models are detailed in sections 5.3 to 5.5. These sections detail the peaks flows and storage requirements for the east catchment, north-west catchment and south-west catchment. The fraction impervious values adopted for the developed conditions models were provided previously in Table 2 and Table 3. The same fraction impervious values were adopted for the stormwater treatment modelling (in MUSIC).

5.2 Input parameters

Model inputs were obtained from the ARR2019 data hub and the Bureau of Meteorology's Intensity Frequency Duration (IFD) data. Full details on inputs and assumptions used for the hydrologic modelling can be found in Appendix A.

Stage storage for the Maffra RB existing conditions was established using LiDAR (provided by WGCMA). Earthworks models were created in 12d using this LiDAR to create an existing conditions surface, and proposed designs were built into this model. This allowed an accurate establishment of design conditions stage-storage relationships.

Climate change and Probably Maximum Flood scenarios
Climate change scenarios have been adopted within the hydrologic models built. The purpose of adopting climate change scenarios is not to design assets to these increased peaks, but to perform a sensitivity check on how increased peak flows will move through the systems designed. For example, how an increased peak 1% AEP will sit within the provided freeboard in a proposed retarding basin. Probable Maximum Precipitation (PMP) is defined by the Manual for Estimation of Probable Maximum Precipitation (World Meteorological Organisation, 2009) as: "...the theoretical maximum precipitation for a given duration under modern meteorological conditions." This can be used to calculate the Probable Maximum Flood (PMF) for a catchment.

The climate change and PMF scenarios have been used as inputs to the flood modelling. The approach adopted for establishing these scenarios has been:

- the use of Bureau of Meteorology (BoM) IFD curves derived for the site.
- that the IFD curves are adjusted to reflect increased intensity arising from climate change.

- ARR 2019 recommends the adoption of a 5% increase in rainfall intensity per degree of global warming (Book 1, Chapter 6) for events up to the 1% AEP.
- RCP 4.5 and RCP 8.5 were adopted for climate change. The catchment is located within the Southern Slopes cluster, which estimates the temperature increase in the RCP 4.5 scenario of 0.5 to 3 degrees during the year 2100 (midpoint of 1.75 degrees selected), and a temperature increase in the RCP 8.5 scenario of 3.6 degrees in the year 2100.
- This approach results in a 9% increase in rainfall intensity for the RCP 4.5 scenario for events us to the 1% AEP, and an increase of 19% in rainfall intensity for the RCP 8.5 scenario.
- The IFD data is used for events up to 1 in 2000 AEP (as available through BoM). For design events larger than this (very rare) a Probable Maximum Precipitation (PMP) has to be calculated first in accordance with ARR2019 using the BoM's Generalised Short Duration Method (GSDM) to be able to calculate a Probable Maximum Flood (PMF). The GSDM is appropriate for catchments up to 1000km^2 and for rainfall durations up to 6 hours. The very rare AEP design rainfalls can be interpolated between the rare and PMP if desired. A PMP has an equivalent AEP of approximately 1 in 10,000,000.
- The increase in rainfall intensity is not applied to events greater than the 1% AEP. As stated in ARR2019 "This approach has an appropriate degree of conservatism as PMP estimates are updated by the Bureau of Meteorology from time to time. This will ensure that any future climate change signal is captured and thus the PMP should not be further adjusted to take into account potential climate change implications."

Further details on the climate change and PMF parameters are provided in Appendix A.

5.3 Storage design – East catchment

The aim of the RORB modelling is to establish critical peak flows and the storage requirements within the Maffra development site. The east catchment option considers upgrading the storage availability within the existing Retarding Basin, and providing storage downstream of the RB, within the future development. This is to control ultimate developed conditions critical flow rates back to pre-developed conditions before ultimately discharging into the existing downstream drainage network.

The RORB model setup for the east catchment is provided in Figure 28.

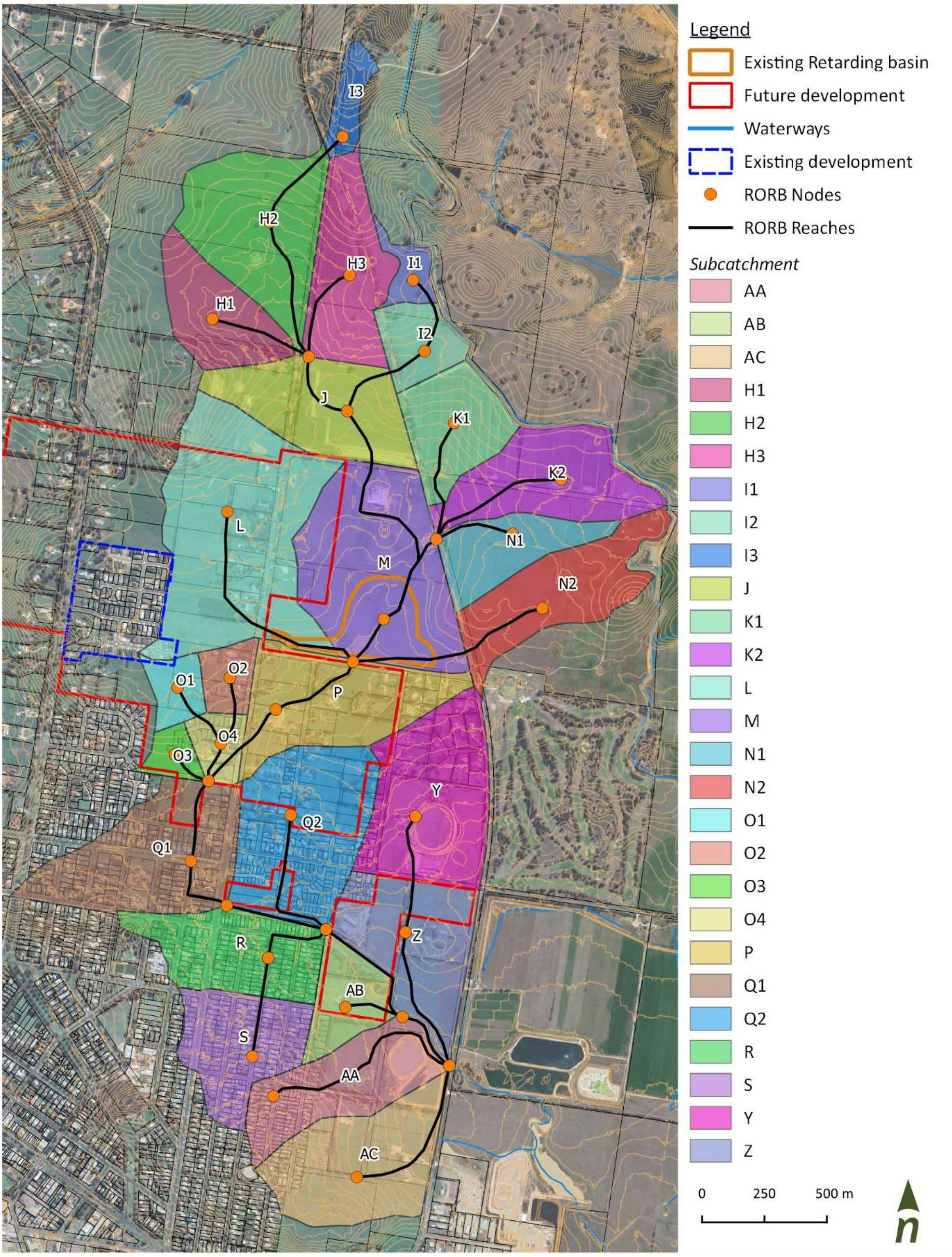

Figure 28. RORB model for the eastern catchment of the Maffra development

As can be seen in the mapping, a portion of the contributing catchment entering the Maffra RB will be developed in the future (catchment L), but majority of the catchment will remain as agricultural. As for the proposed RB location near Powerscourt Street, the entire contributing catchment will be becoming future residential land (catchments O1, O2, O3, O4), and therefore this will have a significant impact on peak flows.

The RORB model was computed for the pre and post developed conditions under the 1% AEP flood event. The results are shown in Table 4. For the Maffra RB the results show the peak flows for the existing and developed conditions flowing into and out of the RB. No change in storage or outlet properties are included in these results. The Powerscourt results show the change in peak flow associated with the development (i.e. no RB yet).

Table 4. 1% AEP event RORB modelling results for the east Maffra catchment

	Maffra RB	Powerscourt catchment
Catchment area (ha)	293	20
Existing storage (from LiDAR) (m^3)	112,000	-
Pre-developed critical flow rate (m^3/s)	17.56 (inflow) 6.21 (outflow)	2.58
Developed critical flow rate (m^3/s) (no mitigation measures)	20.55 (inflow) 6.49 (outflow)	3.55

Following the establishment of existing and post-development peak flows without mitigation, the retarding basins have then been modelled and sized to control the 1% AEP flow. The total required area for each asset has been calculated assuming a 1(V):5(H) batter to existing surface, and an allowance of (preferably) 600mm of freeboard on top of the peak 1% AEP flood depth. The systems are designed so they are not in fill.

For the Maffra RB, the storage was increased assuming the following:

- The current RB footprint was used, with an increase in storage occurring within this. The storage was increased as much as practically and safely possible within this space to reduce outflows and downstream flooding issues as far as practically possible.
- The base of the RB at the pipe outlet would be maintained, with deepening occurring north from this (i.e. not requiring a change in the outlet invert). A grading north at 1 in 500 was assumed. This was modelled in 12d, an earthworks modelling program to ensure an accurate reflection of stage storage relationships was input to the RORB modelling.
- The outlet was altered to only have one of the 600mm pipes operating, to peak limit flows in the more regular events. The weir crest (45.0 m AHD) and width (5m), as well embankment elevation (45.6 m AHD) were kept the same.

Given the existing downstream flooding issues associated with the eastern catchment of the Maffra area, the retarding basin designs have considered controlling flows to reduce the intensity of flows experienced along Merry Street & Alfred Street (i.e. maximised storage where possible to alleviate flooding).

Table 5 shows the required capacities of the retarding basin based on the RORB modelling conducted.

Table 5. East Catchment retarding basin requirements

Parameter	Maffra Retarding Basin	Powerscourt Wetland/ Retarding Basin
Peak RB outflow (m^3/s) (1% AEP)	3.05	1.52
Peak RB storage (m^3)	190,000	5,870
Peak RB flood depth (m)	45.35 m AHD	41.02 m AHD
Freeboard above peak flood depth	250 mm	600 mm
Outlet pipe size (mm)	1 X 600 mm dia.	2 X 600 mm dia.
Surface Area	9.74 ha	1.03 ha

An overview of the RB locations and footprints are provided in Figure 29. This map also shows the wetland that is required from a treatment perspective adjacent to the Maffra RB (discussed in Section 6 of this report). A wetland was not positioned within the RB itself as it would not be able to outfall in this location.

A more detailed map of the asset design is provided in the Concept Designs section of this report – Section 7.

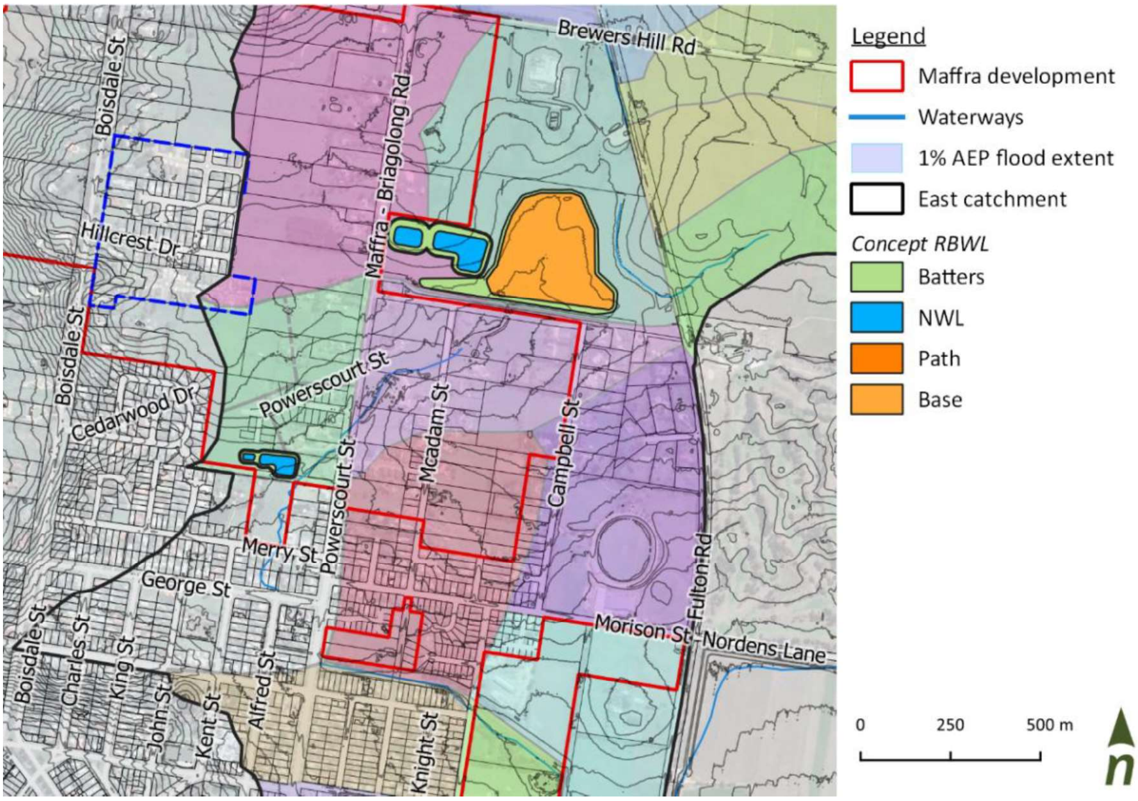

Figure 29. Retarding Basin / wetland locality plan for the eastern catchment

The climate change and PMF modelling results are provided below (assuming same mitigation measures as above; no additional measures).

Table 6. Climate change and PMF modelling for the Maffra RB

Parameter	RCP 4.5 (9% increase in rainfall intensity)	RCP 8.5 (19% increase in rainfall intensity)	PMF
RB inflow (m^3/s) (1% AEP)	23.12	26.07	101.65
Peak RB outflow (m^3/s) (1% AEP)	4.19	5.43	67.88
Peak RB storage (m^3)	205,000	217,000	307,000
Peak RB flood depth (m)	45.48 m AHD	45.60 m AHD	45.77 m AHD
Freeboard above peak flood depth	0.12 m	0.00 m	overtopping

Table 7. Climate change and PMF modelling for the Powerscourt RB

Parameter	RCP 4.5	RCP 8.5	PMF
RB inflow (m^3/s) (1% AEP)	3.96	4.42	17.14
Peak RB outflow (m^3/s) (1% AEP)	1.87	2.31	15.08
Peak RB storage (m^3)	6,240	6,590	10,400
Peak RB flood depth (m)	41.08 m AHD	41.13 m AHD	41.64 m AHD
Freeboard above peak flood depth	0.52 m	0.47 m	overtopping

5.4 Storage design – North West catchment

The aim of the RORB modelling is to establish critical peak flows and the storage requirements within the Maffra Development Site. The north west catchment considers and end of catchment Wetland/ Retarding Basin to control ultimate developed conditions critical flow rates back to pre-developed conditions before ultimately discharging into the downstream waterway (Macalister River).

Figure 30 provides an overview of the RORB model for this catchment. As can be seen from the map, approximately half of the catchment will be converted to residential land under the development scenario. This will therefore have a significant impact on peak flows.

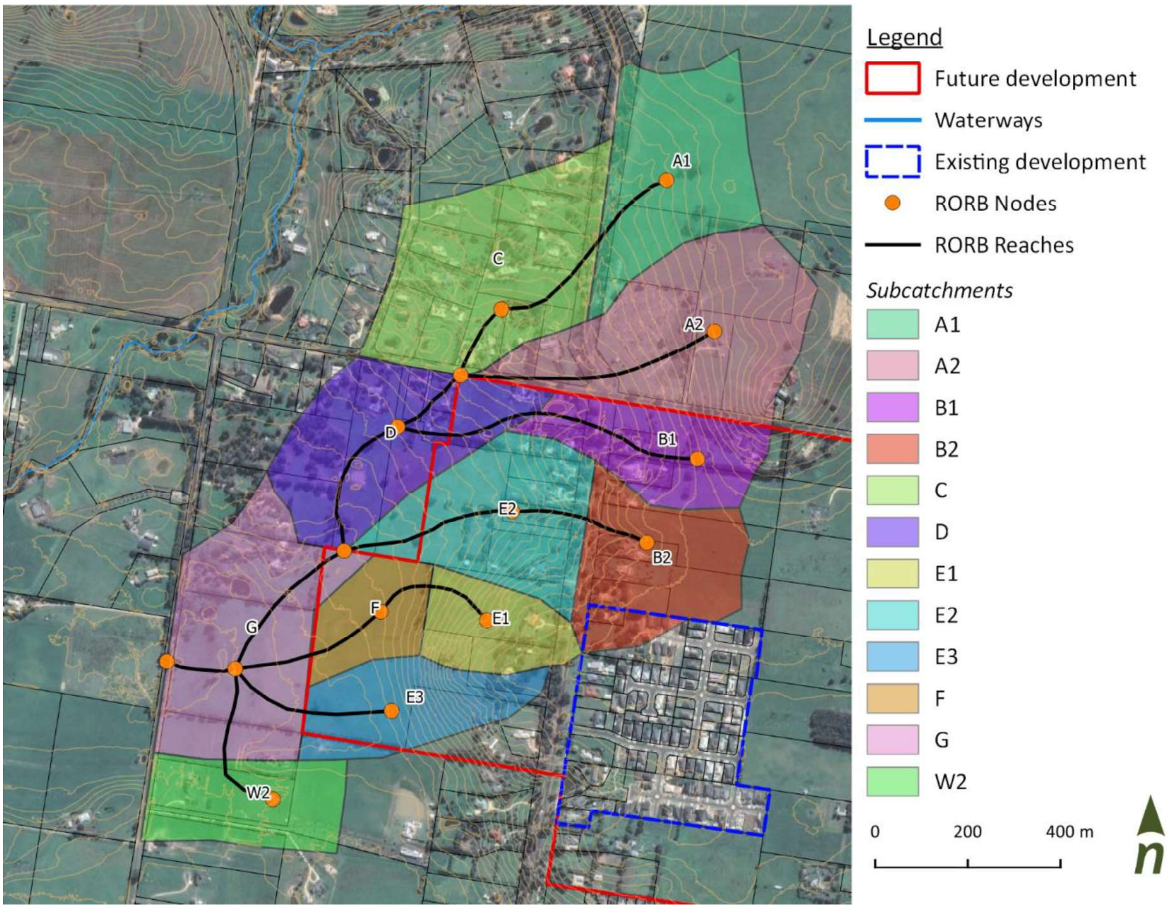

Figure 30. RORB model for the north western catchment of the Maffra development

The RORB model was computed for the pre and post developed conditions under the 1% AEP flood event. The results are shown in Table 8.

Table 8. 1% AEP event RORB modelling results for the north west Maffra catchment

	Catchment outlet
Catchment area (ha)	135
Pre-developed critical flow rate (m^3/s)	10.22
Developed critical flow rate (m^3/s)	13.15

The retarding basin for this catchment has been modelled and sized to control the 1% AEP flow. The total required area for the asset has been calculated assuming a 1(V):5(H) batter to existing surface, and an allowance of 600mm of freeboard on top of the peak 1% AEP flood depth. The system is designed to not be in fill.

Table 9 shows the required capacity of the retarding basin based on the RORB modelling conducted.

Table 9. North west catchment retarding basin requirements

Parameter	**Retarding Basin**
Peak RB outflow (m^3/s)	9.52
Peak RB storage (m^3)	20,700
Peak RB flood depth (m AHD)	24.33
Freeboard above peak flood depth	600 mm
Outlet pipe size (mm)	3 X 1350 mm dia. (or equivalent)*
Surface Area (ha)	2.53

*the outlet structure can be managed by a box culvert equivalent size, with 2 cells and a link slab

An overview of the North-West WL/RB location and footprint is provided in Figure 31. A more detailed map of the asset design is provided in the Concept Designs section of this report – Section 7.

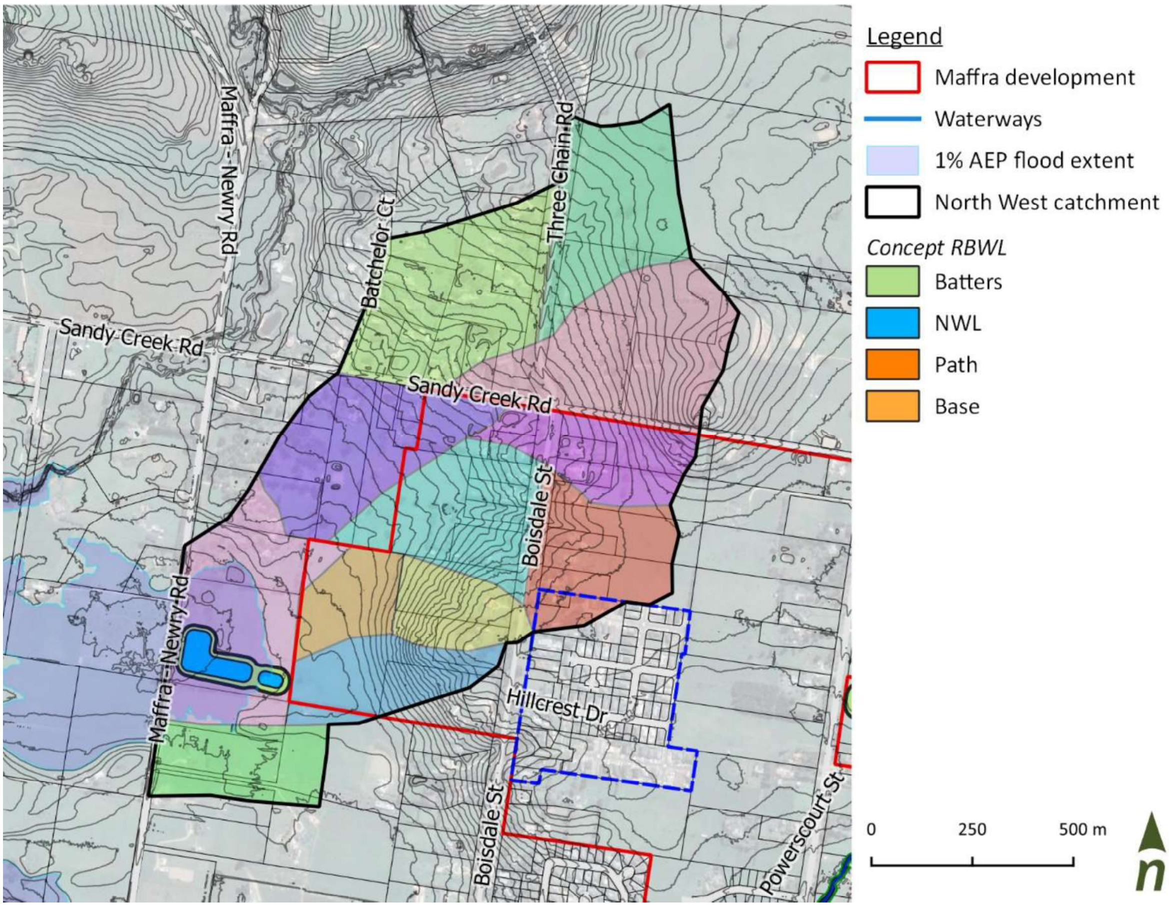

Figure 31. Retarding Basin / wetland locality plan for the north western catchment

The climate change and PMF modelling results are provided below (assuming same mitigation measures as above; no additional measures).

Table 10. Climate change and PMF modelling for the North west RB

Parameter	RCP 4.5 (9% increase in rainfall intensity)	RCP 8.5 (19% increase in rainfall intensity)	PMF
RB inflow (m^3/s) (1% AEP)	14.77	16.60	62.44
Peak RB outflow (m^3/s) (1% AEP)	10.64	12.14	62.01
Peak RB storage (m^3)	22,600	24,700	37,700
Peak RB flood depth (m)	24.43m AHD	24.54 m AHD	25.24 m AHD
Freeboard above peak flood depth	0.67 m	0.56 m	overtopping

5.5 Storage design – South West catchment

The south west catchment considers an end of catchment Wetland/ Retarding Basin to control ultimate developed conditions critical flow rates back to pre-developed conditions before ultimately discharging into the downstream Maffra Wetlands Reserve.

Figure 32 provides an overview of the RORB model for this catchment. As can be seen in the map, only a very small portion of the contributing catchment is proposed to be developed. This therefore indicates there will be a small change in peak flows.

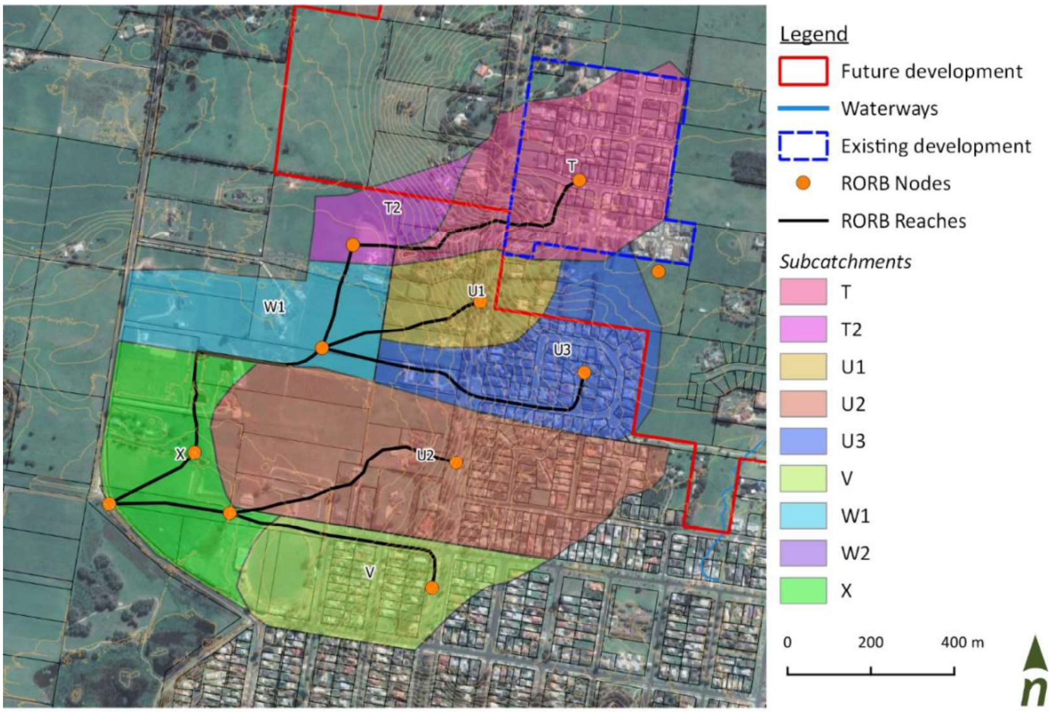

Figure 32. RORB model for the south western catchment of the Maffra development

The RORB model was computed for the pre and post developed conditions under the 1% AEP flood event. The peak flow results at the end of the catchment are shown in Table 11. As can be seen from the results, there is a small change in peak flow from existing to developed conditions. The climate change scenarios and PMF are also provided.

Table 11. 1% AEP event RORB modelling results for the south west catchment

	South west catchment
Catchment area (ha)	127
Pre-developed critical flow rate (m^3/s)	11.58
Developed critical flow rate (m^3/s)	14.88
Developed critical flow rate (m^3/s) RCP 4.5	16.69
Developed critical flow rate (m^3/s) RCP 8.5	18.71
Developed critical flow rate (m^3/s) PMF	68.42

The changes in peak flow are attributed to a small portion of the catchment being developed. This will be managed through a combination of buried oversized pipes or underground system and stormwater tanks within the development itself. As a result, only stormwater quality treatment works are required within the south west catchment.

6 Stormwater quality treatment

A key principle for the development of the Maffra development is that all stormwater is to be treated to BPEMG (Best Practice Environmental Management Guidelines) before being discharged from the study area. As such, the Maffra development site will require numerous treatment techniques in order to achieve the targeted reduction in pollutant load concentrations. The following BPEMG targets have been adopted:

- 70% removal of the total Gross Pollutant load
- 80% removal of total Suspended Solids (TSS)
- 45% removal of total Nitrogen (TN)
- 45% removal of total Phosphorus (TP).

A MUSIC (Model for Urban Stormwater Improvement Conceptualisation) model was developed to estimate the pollutant loads generated from the developed conditions Maffra scenario. This allowed us to understand the target pollutant load reduction, and therefore test the sizing and treatment capacity of various opportunities required to meet the pollutant reduction targets. Reduction requirements were determined for each catchment, and treatment system sizes were calculated. This modelling and asset sizing does not seek to treat existing residential areas or agricultural areas, only future residential areas. However, where there is opportunity to treat existing residential areas, this has been adopted.

6.1 Modelling inputs

The key modelling inputs for the MUSIC model are rainfall and evapotranspiration. Generally, for MUSIC a ten year rainfall period is selected for a site which is a good representation of the average rainfall. The period adopted should consider a completeness of record, and representation of wet and dry periods. Council did not have a template rainfall dataset, so some analysis was done to ensure an appropriate dataset was used.

A historic rainfall dataset (1968- 2020) was obtained from the Bureau of Meteorology (BoM) for the Stratford rainfall gauge (085078). The average annual rainfall over this entire period was established and used to select a ten-year period from the historic dataset which produced a similar annual average rainfall. The average annual rainfall from BoM is 654.6 mm. The period from 1982 -1991 was adopted which has an annual average rainfall of 667.5mm.

The monthly average evaporation for Sale was also obtained from BoM and adopted for this modelling.

When modelling wetlands in MUSIC, an Extended Detention Depth of 0.35m is adopted and a detention time of 72 hours is aimed for. This allows sufficient contact time with the vegetation, and therefore treatment of the stormwater.

The inlet pond areas for each wetland were sized using the Fair and Geyer equation, where sediment basins are required to meet a 95% sediment capture efficiency of coarse particles ≥ 125 µm diameter for the peak 4EY (4 Exceedances per Year) event. The sediment basins were assumed to have an average depth of 0.8m, and the volume was used in the MUSIC modelling. The details of these calculations are provided in Appendix B.

6.2 East catchment

The catchment nodes used in the east catchment model have been calculated based on the areas, land uses and associated fraction impervious values used in the RORB modelling (provided in Table 2). The MUSIC model layout is shown in Figure 33. These assets have been sized to treat the loads being generated off the future developable area to best practice. This includes loads being generated from new development downstream near the Davis Street Drain. The wetland next to the Maffra RB and the Powerscourt wetland are therefore offsetting development downstream. This is a good way of achieving treatment targets, but also consolidating treatment assets.

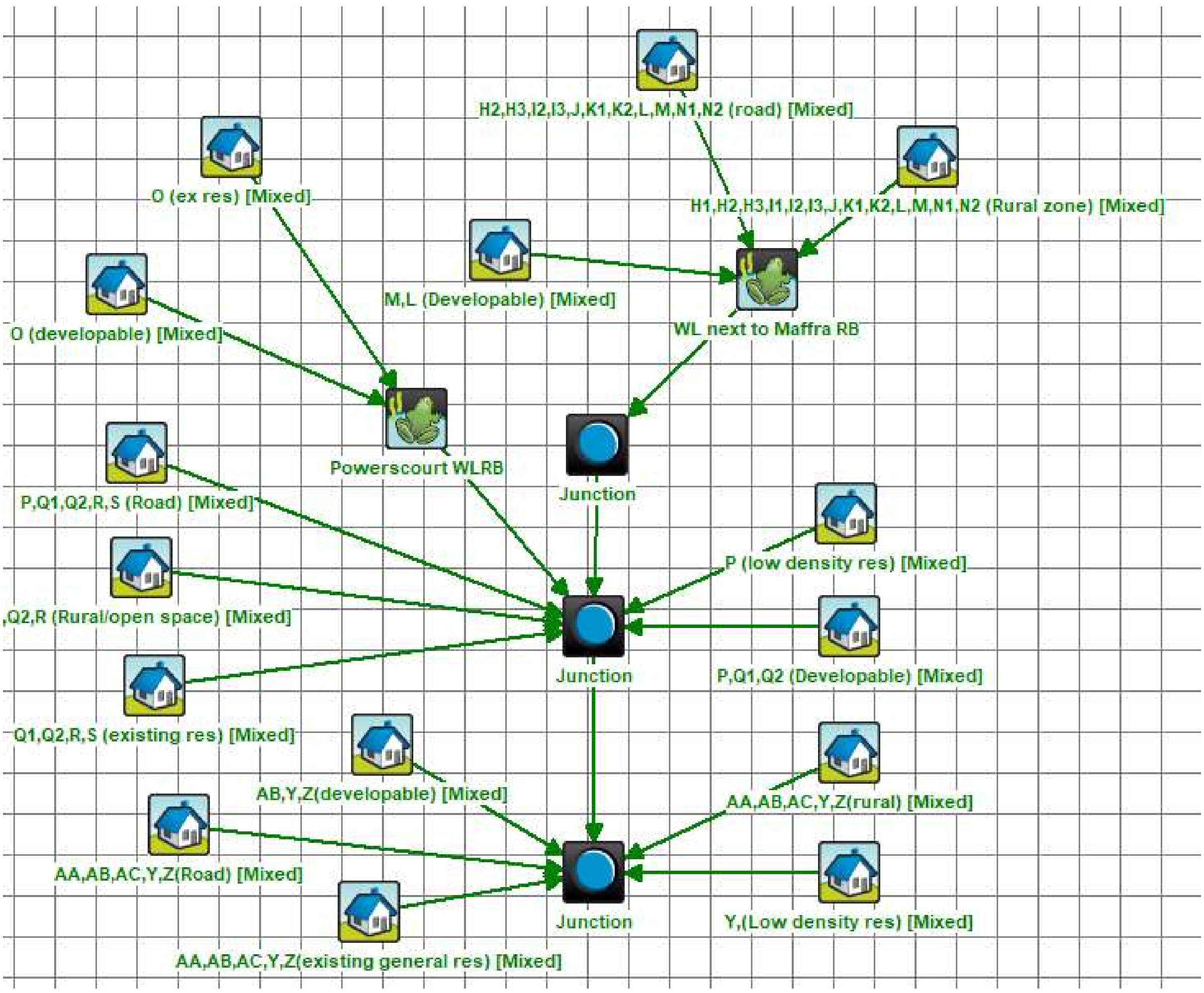

Figure 33. *MUSIC model for the eastern catchment of the Maffra site*

Asset Performance

The MUSIC modelling determined the sizing required for the two wetland assets located at each of the catchment low points. The wetlands have been designed to inform the retarding basin stage-storage relationship presented in Section 5.3. The details of the Maffra east treatment systems are shown in Table 12.

Table 12. Treatment asset parameters for Maffra east wetlands

	WL (next to Maffra RB)	Powerscourt RBWL
NWL area, m^2	10,000	4,000
Inlet pond area, m^2	3,500	800
Inlet pond volume m^3	2,800	640
Average depth wetland, m	0.40	0.40
Extended detention, m	0.35	0.35
Extended detention time, hr	71.6	71.1

The results of the MUSIC modelling analysis demonstrate that BPEMG targets are met with the performance of those assets, as shown in Table 13.

Table 13. Overall MUSIC modelling results – Maffra east treatment system (wetland)

	Source load	Developable load	Residual load	% Reduction	Kg/yr removed
Total Suspended Solids (kg/yr)	202,000	77,000	135,000	87.0%	67,000
Total Phosphorus (kg/yr)	448	158	320	81.0%	128
Total Nitrogen (kg/yr)	3,520	1,157	2,820	60.5%	700
Gross Pollutants (kg/yr)	36,200	15,030	23,600	83.8%	12,600

6.3 North west catchment

The catchment nodes used in the north-west catchment model have been calculated based on the areas, land uses and associated fraction impervious values used in the RORB modelling (provided in Table 3).The MUSIC model layout is shown in Figure 34. This wetland has been sized to treat all contributing catchments to best practice (i.e. not only the developable areas). This has been done because the space is available, and the existing residential areas have previously not been treated to best practice. This will result in better quality water entering the Macalister River.

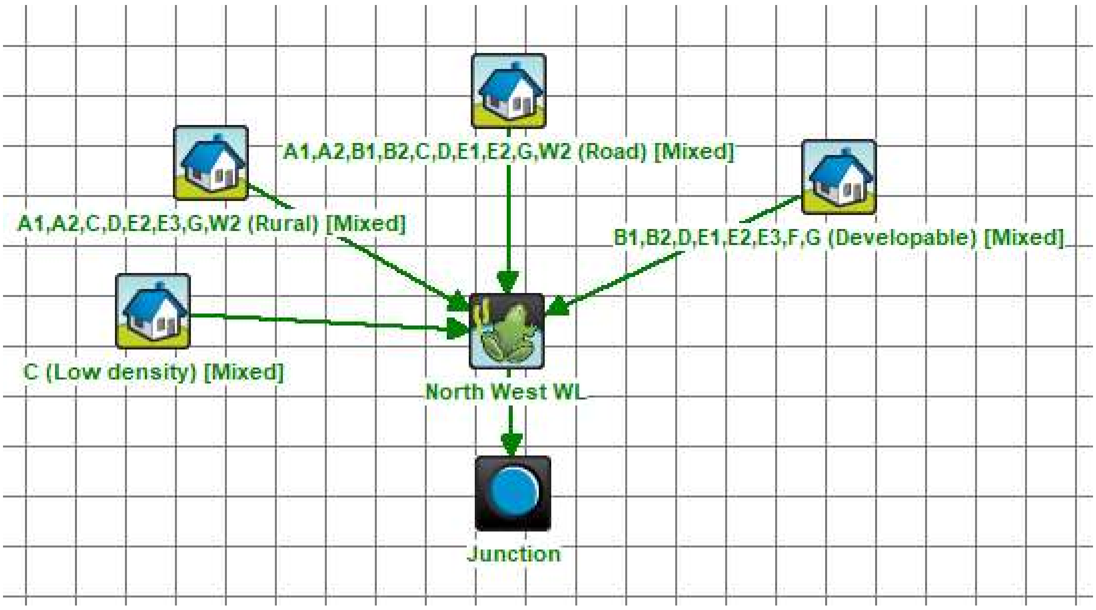

Figure 34. *MUSIC model for the north western catchment of the Maffra site*

Asset Performance

The MUSIC modelling determined the sizing required for the wetland to meet best practice. The wetland has been designed to inform the retarding basin stage-storage relationship in Section 5.4. The details of the Maffra north west treatment system are shown in Table 14.

Table 14. Treatment asset parameters for Maffra north west wetland

	North West RBWL
NWL area, m^2	11,500
Inlet pond area, m^2	1,800
Inlet pond volume m^3	1,440
Average depth wetland, m	0.40
Extended detention, m	0.35
Extended detention time, hr	72.2

The results of the MUSIC modelling demonstrate that BPEMG targets for the entire catchment are met with the wetland, as shown in Table 15.

Table 15. Overall MUSIC modelling results – Maffra north west treatment system (wetland)

	Source load	Residual load	% Reduction	Kg/yr removed
Total Suspended Solids (kg/yr)	44,100	4,950	89 %	39,150
Total Phosphorus (kg/yr)	90.9	24.3	73.3 %	67
Total Nitrogen (kg/yr)	664	362	45.5 %	302
Gross Pollutants (kg/yr)	8,560	0	100 %	8,560

6.4 South west catchment

The catchment nodes used in the east catchment model have been calculated based on the areas, land uses and associated fraction impervious values used in the RORB modelling (provided in Table 3). The MUSIC model layout is shown in Figure 35.

Only a very small portion of the contributing catchment is proposed to be developed, and therefore pollutant load generation associated with the development will be relatively small when compared to the entire catchment (refer to Figure 32 for an overview of the catchment). This wetland has been sized to fit within the space available (i.e. the existing waterbody site) and can therefore treat stormwater off the entire contributing catchment. This is a positive outcome for the Maffra Wetlands which currently receive untreated stormwater from this catchment.

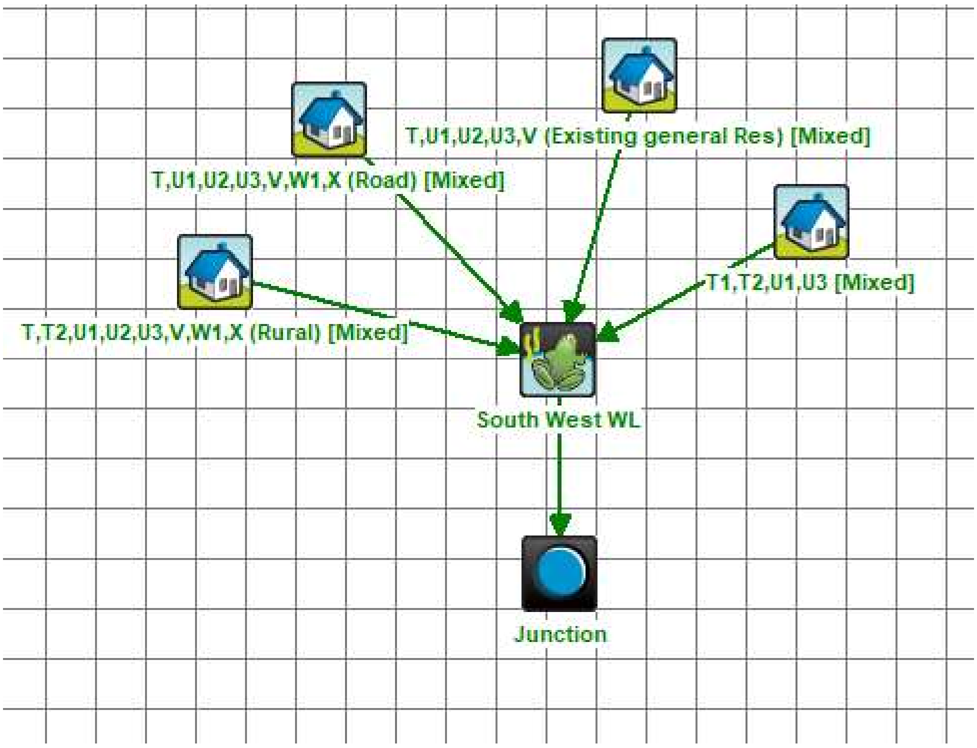

Figure 35. *MUSIC model for the south western catchment of the Maffra site*

Asset Performance

As previously mentioned, the space available to fit a wetland really drove the sizing of the wetland, and therefore the associated treatment results. The details of the Maffra south west treatment system are shown in Table 16.

Table 16. Treatment asset parameters for Maffra south west wetland

	South West RBWL
NWL area, m^2	14,000
Inlet pond area, m^2	1,700
Inlet pond volume m^3	1,360
Average depth wetland, m	0.40
Extended detention, m	0.35
Extended detention time, hr	72.1

The results of the MUSIC modelling analysis demonstrate that BPEMG targets (for the entire catchment) are met with the performance of the wetland, as shown in Table 17.

Table 17. Overall MUSIC modelling results – Maffra south west treatment system (wetland)

	Source load	Residual load	% Reduction	Kg/yr removed
Total Suspended Solids (kg/yr)	49,200	6,870	86 %	42,330
Total Phosphorus (kg/yr)	110	32.5	70.3 %	78
Total Nitrogen (kg/yr)	862	473	45.1 %	389
Gross Pollutants (kg/yr)	8,990	0	100 %	8,990

7 Concept designs

The concept designs for the options investigated are presented within this section. Each option includes:

- The macrophyte treatment area (NWL) as established in MUSIC
- The storage requirements as established in the hydrologic modelling
- A Normal Water Level (NWL) identified by looking at the topography of the site, as well as the inclusion of 0.35m EDD and any freeboard requirements
- An approximate overall footprint based on the selected NWL and battering up to existing surface at a 1 in 5 grade
- Indicative inlet pipe, transfer pipe (sediment basin to wetland), and outlet pipe locations
- A 2.5m path allowance around the site (alignments to be defined in later design stages).

Other factors that influenced the configuration of the asset included:

- The ability to outfall
- The requirement to meet a length to width ratio of at least 4:1 [MZ4 in the constructed wetlands manual], and therefore the associated maximum width, and how this fit in with the surrounding terrain
- Meeting velocity requirements
- Minimising excavation requirements where possible
- A desire to not have the assets in fill (i.e. no reduction in flood storage).

The concept options are shown in Figure 36, Figure 37, Figure 38 and Figure 39 below. The configuration of these assets can be refined in later design stages, but these concept designs provide a conservative indication of land take and key infrastructure requirements. It should be noted that it is not an issue if the assets sit within the Macalister River 1% AEP flood inundation. The assets should, however, sit outside of the 10% AEP inundation extent so not to be regularly inundated, which could drown out vegetation.

Sections through these assets have also been provided based on the earthworks modelling. Note these do not include the internal bathymetry of the systems (i.e. only show down to the NWL). These are provided in Appendix C.

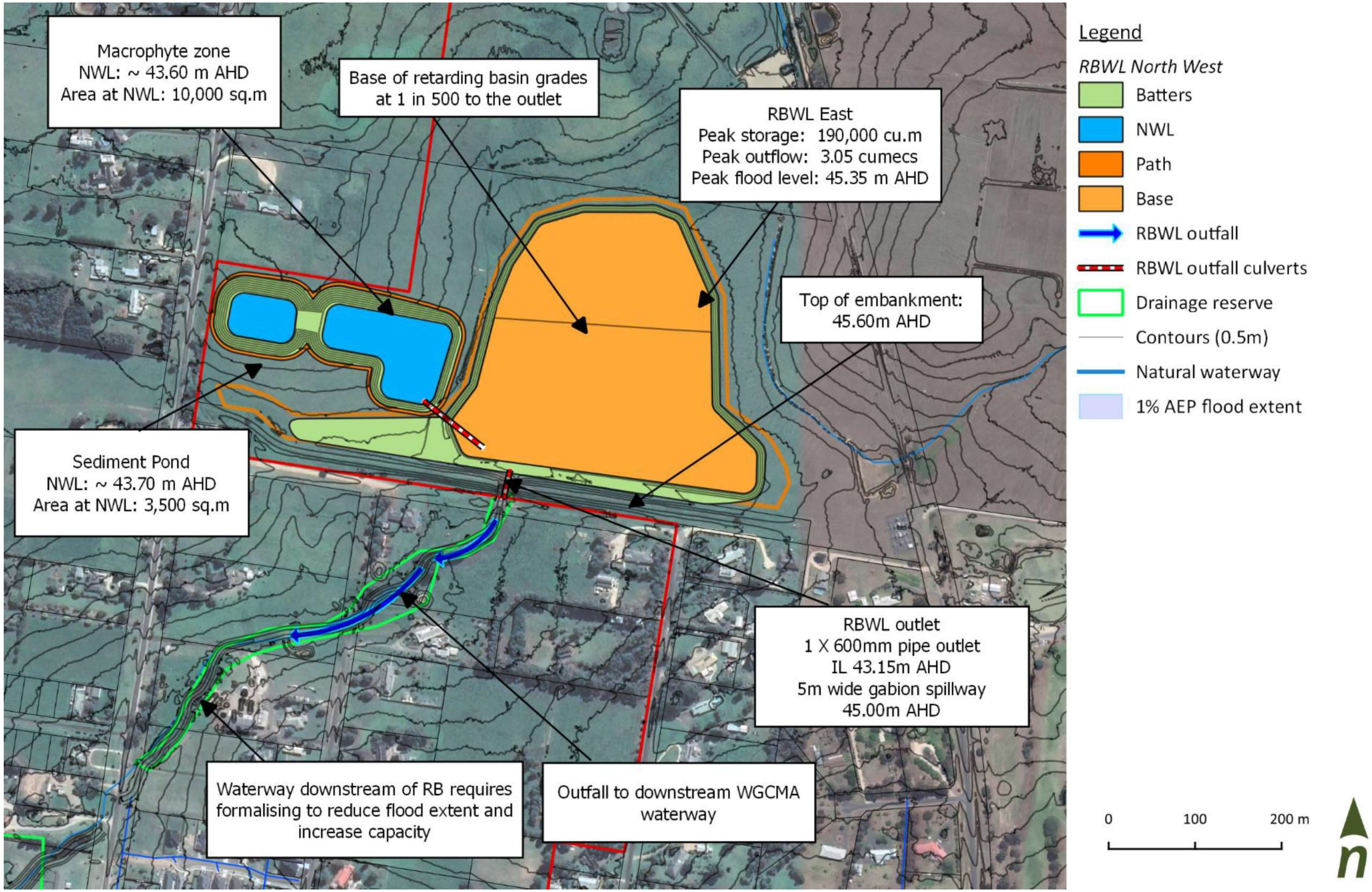

Figure 36. *Maffra Retarding Basin / Wetland*

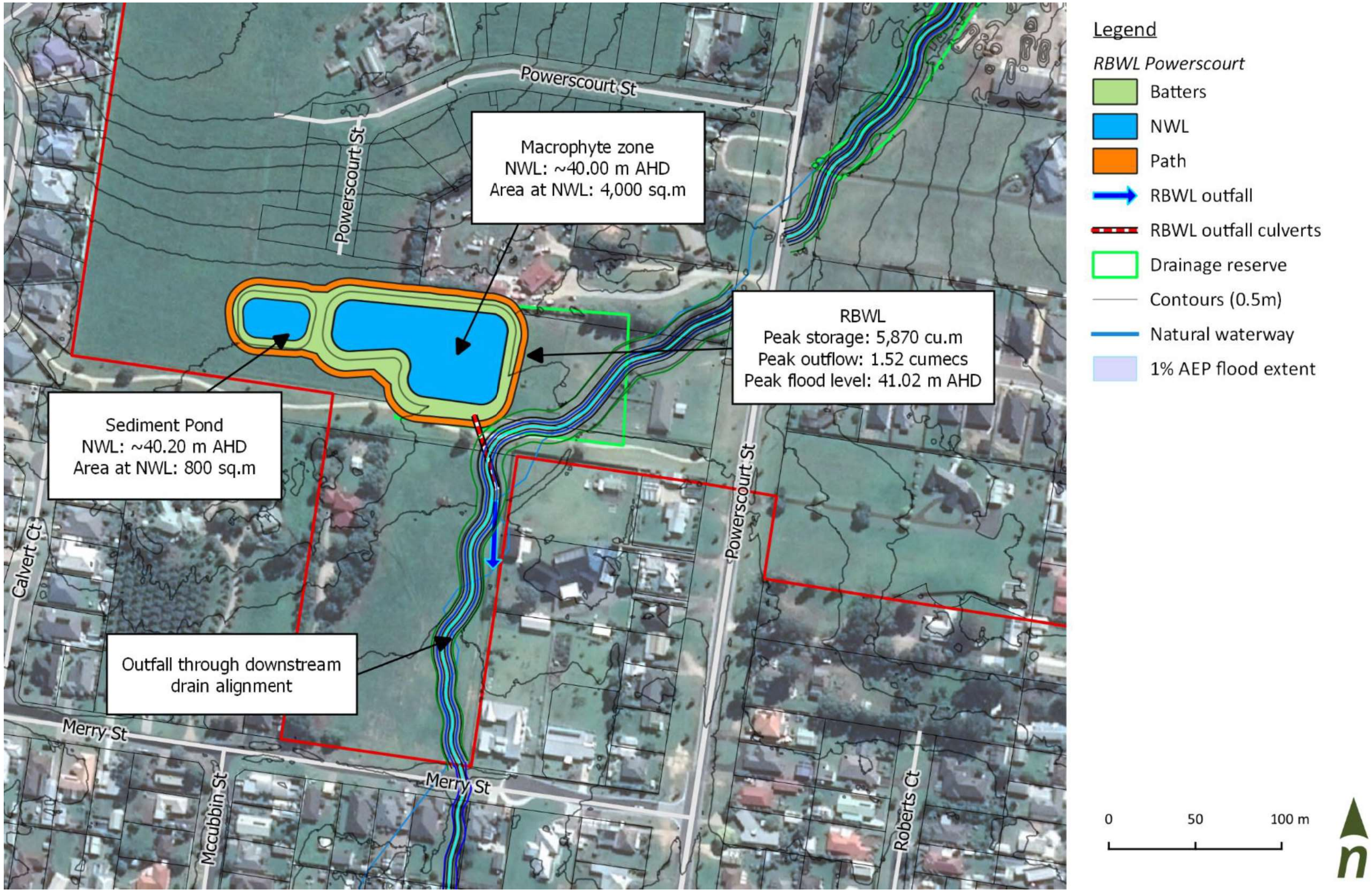

Figure 37. *Powerscourt Street Wetland / Retarding Basin*

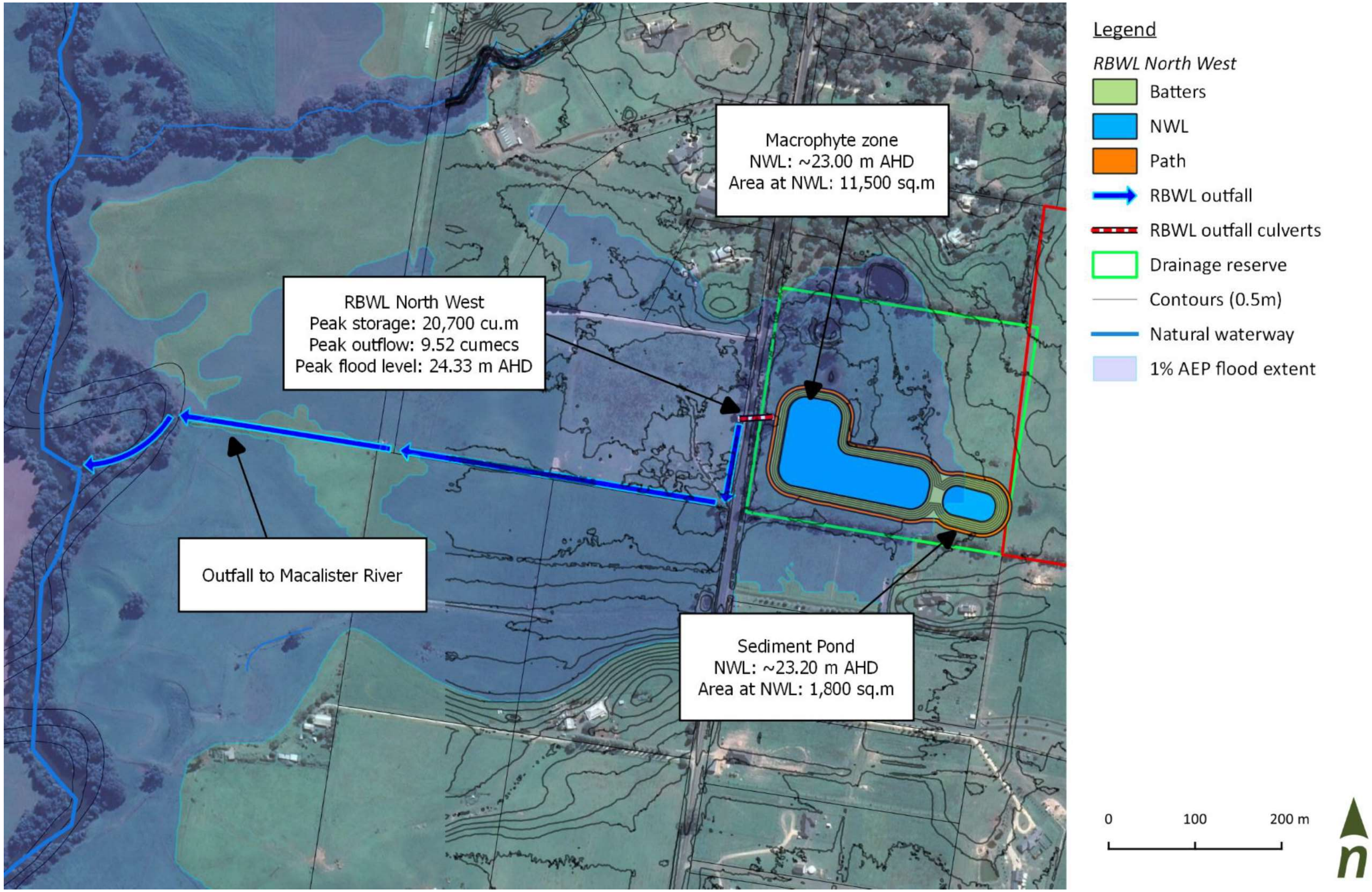

Figure 38. *North west Wetland / Retarding Basin*

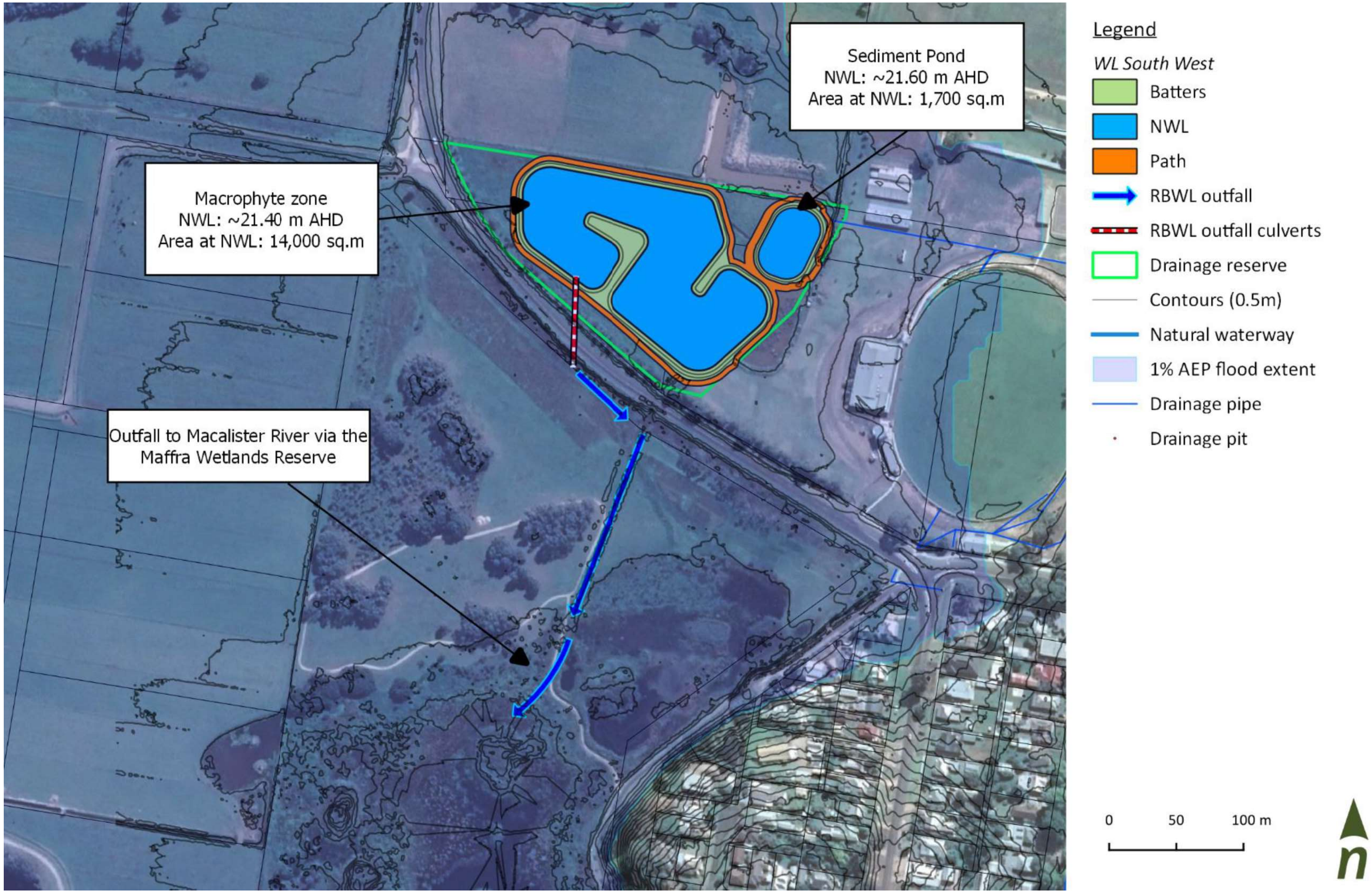

Figure 39. *South West Wetland*

8 Integrated water management

Integrated water management (IWM) considers the range of opportunities that urban water management presents to make use of available water on a fit for purpose basis while creating a greener, healthier, more aesthetically pleasing natural and urban landscape. The Shire of Wellington IWM Plan was prepared in 2019/20 setting out the vision of *Working together to sustainably manage water for current and future generations.* Some of the desired outcomes of that strategy that have informed the identification of IWM opportunities within Maffra include:

- Identifying fit for purpose water supplies (including stormwater harvesting)
- Healthy and valued waterways, wetlands and lakes
- Healthy and valued agricultural, rural and urban landscapes and
- Community values reflected in place-based planning.

The IWM opportunities identified for Maffra are therefore driven by a desire to reduce potable water use, support healthy natural assets and enhance the town's aesthetic. By way of context Figure 40 (sourced from the Shire's IWM Plan), indicates that the greater proportion of Council's water use in Maffra goes to the irrigation of sports and recreational facilities, parks and reserves. As such the 'fit for purpose' use of stormwater to irrigate open spaces, for example, will contribute both to the greening of Maffra but also significantly reduce the Shire's water use in the town.

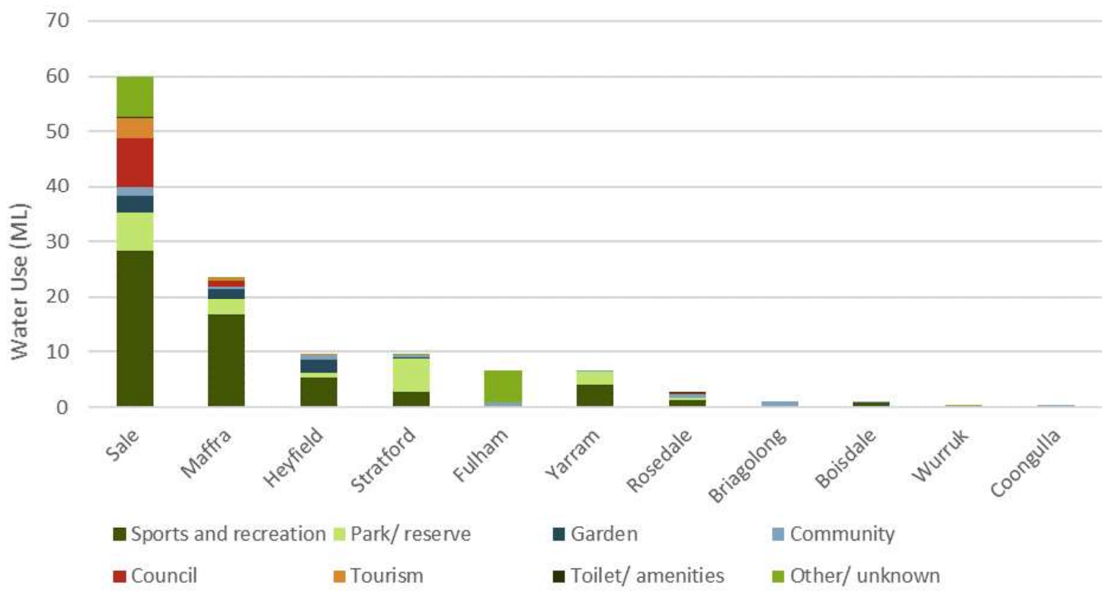

Figure 40. *Wellington Shire Council water use breakdown (2018-19)*

During this project, as part of the site visit and in consultation with the Shire of Wellington, the following IWM opportunities were identified.

8.1 Stormwater harvesting

The Strategic Directions Statement for Gippsland (IWM Forum, 2018) states that "Maffra currently relies on water from the Macalister Irrigation District to water open spaces. Stormwater harvesting presents an opportunity to utilise an alternate water source, improving the town's water security".

Three locations were identified as being logical candidates for stormwater harvesting schemes:

- Maffra Recreation Reserve
- The Maffra Golf Club and
- Cameron Sporting Complex.

Maffra Recreation Reserve

This reserve is located on Edward and McLean St and is currently irrigated with non-potable water from the small waterbody to the north of Maffra – Newry Road, upstream of the Maffra Wetlands Reserve. A stormwater harvesting scheme for the Maffra Recreation Reserve would assume that water is harvested from a formalised wetland in that location to the west of the reserve as per the concept presented under Figure 39 above.

The following stormwater harvesting analysis assumes that water is drawn from that concept wetland. The aim is to identify the storage requirements for a harvesting scheme assuming a target volumetric supply reliability of 80% for the adjacent reserve.

Assumptions:

- Irrigation area = 2.2 Ha
- Irrigation rate – 5 ML/Ha/year
- Average annual demand – 11 ML/year

Sensitivity analysis was undertaken with the results summarised in Figure 41 below. It suggests that to reach our desired reliability of 80%, a storage of approximately 400kL is desirable. The key question then is if the harvesting project was to proceed, whether it would require a standalone storage, or incorporated into the wetland itself? Our preference would be for the later and therefore would recommend that this requirement be included in future wetland design requirements.

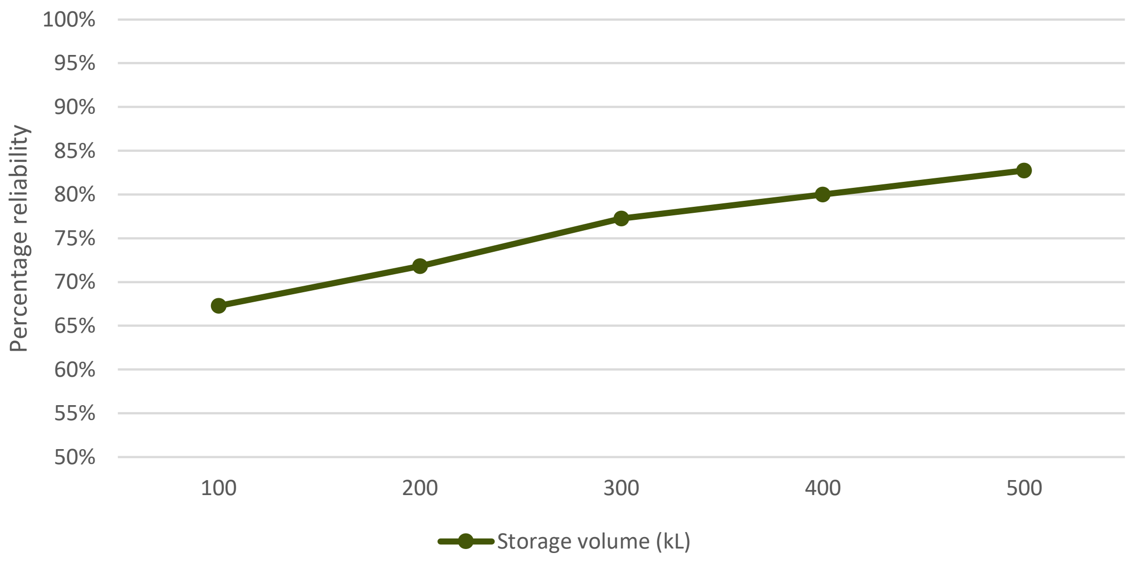

Figure 41. *Maffra Recreational Reserve storage reliability relationship*

The Maffra Golf Club

This opportunity relates to the wetland concept design presented in Figure 36 above in relation to the Maffra Retarding Basin. The opportunity here is to provide treated stormwater to the Maffra Golf Course (to the east) and potentially to the Cameron Sporting Complex to the south east.

A similar analysis was undertaken in relation to these spaces as per the Maffra Recreational Reserve.

Assumptions:

- Irrigation area = 24 Ha (or 50% of the total area)
- Irrigation rate – 5 ML/Ha/year
- Average annual demand – 120 ML/year

Clearly this is a far greater irrigation demand than the previous example, requiring a greater storage volume. In fact the modelling suggests that up to 2 ML of storage is required to meet 62% of demand. In this case, and if a harvesting scheme is to be pursued here, the required storage may be a combination of in-wetland storage, as described above, and use or expansion of the dam on the golf course site (volume unknown).

Cameron Sporting Complex

The Cameron Sporting Complex is a more conventional recreation spade with an area of 5 Ha and assumed average annual demand of 25 ML. The analysis points to a storage requirement of 500kL, however at a distance of approximately 1km from the RB wetland to the complex, there are likely to be transmission costs that won't be present in the Maffra Recreational Reserve example.

Again, the aim would be to incorporate this additional storage into future iterations of the wetland design.

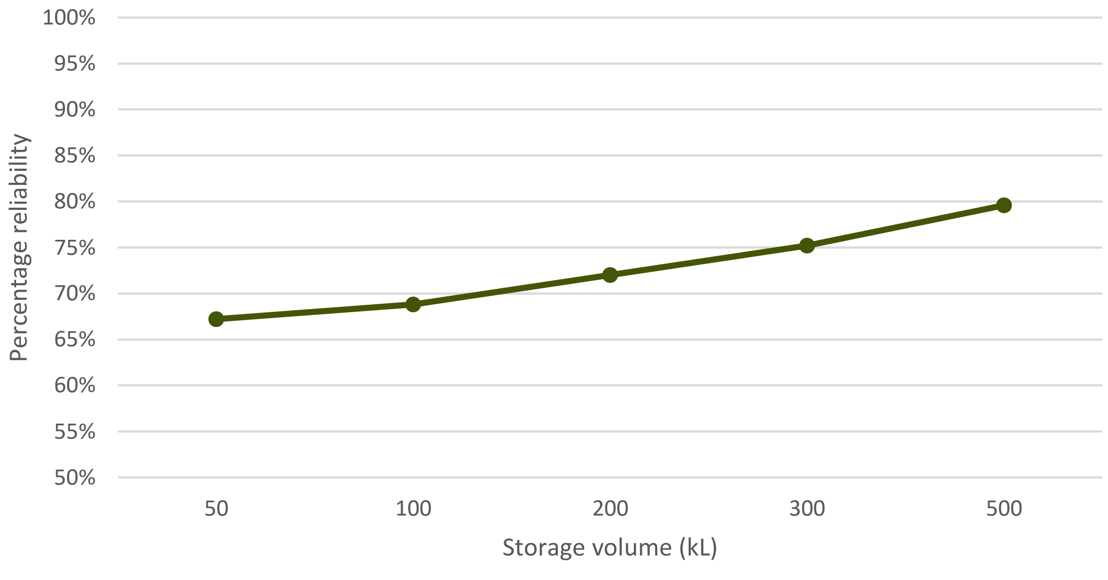

Figure 42. *Cameron sporting complex*

When comparing the stormwater harvesting options available on a purely qualitative basis, the Maffra Recreational Reserve scheme has advantages over the Maffra RB options based on:

- a reasonable volume of storage required to achieve the desired reliability and the potential to incorporate some or all of that volume into the future wetland design to optimise the volume of storage that is required external to the wetland
- the proximity to the end user reducing transfer infrastructure and energy
- existing extraction pump station and irrigation infrastructure, reducing capital costs
- community benefit (broader than delivering a benefit to golf course members only, for example), and
- the extraction of stormwater contributing to the health of the Maffra Wetlands Reserve downstream.

On the first dot point, the question of incorporating storage into the wetland design, the options include:

- A stand-alone, above ground storage of 400kL (approx. 3.5m high and 12m in diameter). Below ground has not been considered due to cost.
- Designing the wetland so that some storage, (if not all) is incorporated into the wetland itself in the form of a harvesting pond off the back of the wetland (i.e. not drawing down the wetland water levels directly). This implies additional controls and valving to control levels as well as an outlet pump station to control outlet flowrates.

The later has not been investigated as part of this stage of design but could be achieved by configuring the wetland (including valving) to enable the wetland to function in two modes: summer (while harvesting) and winter. Summer mode would require a change to key parameters including inlet volume, extended detention depth (EDD) and permanent pool depth, so that additional storage is delivered while stormwater treatment requirements (residence time) is maintained. The outlet flow rate would be determined by a low flow pump station located at the wetland outlet that would pump directly to the irrigation network. In winter, when irrigation water is not required, wetland outflows would flow downstream. The reason we propose two modes is so that in winter we are not relying on a pumped outlet all year round and the energy consumption and cost that that implies.

At this stage it is unclear if the storage requirement of 400kL could be accommodated within the wetland. However, if we assume an increase in EDD of 150mm (from a typical 350mm to 500mm) across the Maffra Recreational Reserve wetland area of 14,000m^2, it corresponds to a volume of 2.1ML, so theoretically it seems worth investigating further. Operationally it may require the site to be visited twice a year to adjust valving etc. to switch between modes.

This idea will need to be investigated further and should be incorporated into the scope of future functional designs for comparison to an external 400kL storage option.

8.2 Channel naturalisation

Channel naturalisation refers to the process of transforming a channel into a more natural state to provides improved environmental, social and economic outcomes. It reflects an approach that brings together best practice waterway engineering, science, ecology, landscaping and community connection to natural environments and assets. Channel naturalisation is best considered where the following drivers are present:

- Deteriorating channel conditions are evident
- There are changing perceptions of urban waterways and stormwater management and the community demands or would benefit from improved waterway conditions
- Urban renewal in the surrounding and upstream catchment
- Enhanced social and environmental outcomes are being sought including the creation of high-quality community spaces.

Benefits

Table 18 summarises some of the benefits associated with channel naturalisation.

Table 18. Benefits of undertaking channel naturalisation

Environmental	Social	Economic
Ecological restoration Habitat creation Improved local biodiversity Urban cooling Improved water quality Ecosystem services	Improved amenity Community engagement (co-design process) Connecting people to waterways Place making Activating space Mental health benefits Providing recreational opportunities Improved safety Improved connectivity	Increased property value Well-designed and constructed waterways can have a longer asset life than concrete channels Reduced health costs

Challenges

Some of the challenges in undertaking channel naturalisation include:

- Capital cost
- Available open space (i.e. naturalised channels requires greater lateral space when compared to concrete channel as the conveyance is not as efficient)
- Ensuring control of local stormwater connections (i.e. ensure outfall is feasible)
- Potential for flooding
- Extended time it takes for vegetation establishment and associated benefits (e.g. property value)
- Multi-agency collaboration and funding mechanisms
- Existing infrastructure constraints (e.g. underground services, existing roads and culverts etc.)

Case study – Blind Creek

Despite these challenges, channel naturalisation can and has transformed urban landscapes providing much needed recreational and relaxation space for their local communities. Some example photos from the Reimagining Blind Creek project that Alluvium recently undertook with Melbourne Water as part of their "Reimagining Your Creek" program are provided in Figure 43 and Figure 44 below to illustrate how the benefits of naturalisation can be realised.

Figure 43. *Blind Creek naturalisation in Boronia, Melbourne. The grassed floodway before naturalisation (left) and concept designs top right and bottom right.*

Figure 44. *Blind Creek naturalisation in Boronia, Melbourne immediately post-construction (prior to vegetation establishment). Stepping stones and picnic areas (left) and a meandering waterway between mature trees (right)*

Naturalisation opportunities
There are two main naturalisation opportunities within Maffra and these are detailed below.

- **Downstream of the Maffra RB**: The waterway immediately downstream of the Maffra RB, extending down to Merry Street and George Street. This opportunity is largely driven by the need to formalise the waterway and increase capacity to alleviate existing (and potential future) flooding issues. This waterway is presently shallow in parts and not of high ecological or social value. This is a CMA-designated waterway. Transforming this waterway into a high-quality waterway could also provide recreational opportunities through Maffra, as well as enhancing ecological value and urban cooling opportunities.

- **The Davis Street Drain**. There is a significant opportunity for naturalising the existing narrow concrete channel that begins at Alfred Street, flows east across Powerscourt Street, past Landy Street and east out of town. This drain currently consists of a shallow and narrow concrete channel within a larger grassed floodway. It therefore presents a good opportunity for creating a natural waterway, which requires a wider space. This opportunity is driven by an opportunity to alleviate flooding by increasing capacity, but also an opportunity to create a high-quality community asset through town.

Design objectives
The design objectives for naturalisation of the channels are as follows:

- Safely convey large flood events within the waterway corridor and reduce or maintain current flood extents as modelled in base case (existing) conditions.
- Provide an appropriate level of erosion protection to public and private assets using native vegetation as the primary channel boundary material, in preference over rock or other hard engineered materials, subject to the design criteria being achievable.
- Have a naturalistic and variable form with an abundant and diverse native vegetation.
- Be a safe environment for the community to interact with and provide an appropriate level of direct and indirect access to the waterway.
- Provide for the establishment of abundant and diverse native vegetation species within the waterway and provide suitable non-vegetative physical habitat.
- Ensure sufficient access and space for all required maintenance activities that is safe for WSC and CMA staff and contractors to access and maintain.

Some high-level concepts and provided on each opportunity below, as well as opportunities and constraints for each option.

Waterway downstream of the Maffra retarding basin concept
There is a need to formalise the waterway immediately downstream of the Maffra RB to alleviate flooding extents and enhance conveyance. This will become increasingly important as the surrounding area develops. The improvement of this waterway has been identified in previous drainage assessments. The waterway is currently a CMA-designated waterway. The waterway becomes very shallow towards Merry Street (Figure 45), where it then passes through private property (and a small underground drain), before connecting with the George Street Drain and Alfred Street (overland flows). This area is known to have significant flooding issues.

Figure 45. *The shallow waterway (looking north from Merry Street)*

A concept-level waterway was designed from the Maffra RB to George Street using the following design assumptions and criteria:

- The existing depression has been used to guide the waterway alignment. The alignment has been shifted where necessary to avoid crossing too many properties, and to allow an access buffer between the waterway and property fences. The waterway has been designed in 12d, an earthworks modelling program, based on the existing surface created from LiDAR. No hydraulic modelling to test shear stress has been conducted at this stage (later design stages).
- The waterway is a compound arrangement. That is, a low flow channel set into a high flow channel. Low flow channels are traditionally designed to take between the 4EY and 1EY flows, and the high flow channel should have the capacity to take the 1% AEP flows. This is in line with the Constructed Waterways Design Manual (CWDM) (Melbourne Water, 2019).
- The waterway should meander to create diversity in planform.
- Flows were adopted from the RORB model assuming post-development conditions, and the flood mitigation (storage) options previously presented in Section 5. These are the peak flows (Table 19). At this stage the larger flow events (i.e. the downstream peak flows) have been adopted to design the entire waterway. This is a conservative approach which can be refined in later design stages.
- The low flow channel should have a minimum base width of 3m, minimum depth of 0.5m and 1:3 batters (in line with the CWDM), as well as benches, which create diversity in form and habitat niches.
- The waterway low flow channel was designed to have a base width of 3m, depth of 0.5m, 1 in 3 batters, 2m wide benches either side of the LFC, manning's n value of 0.05 (vegetated), and longitudinal grade of approximately 1 in 200 (varies). This results in a low flow channel capacity of approximately 1.6m^3/s. This is conservatively slightly more than what might be needed (1EY) given the grade varies (i.e. a flatter grade will reduce capacity). The same goes for the HFC.
- The high flow channel was designed to have a depth of 0.5m (resulting in an overall waterway depth of 1m) and 1 in 5 batters to existing surface. This results in an overall waterway capacity of 9m^3/s.
- An equivalent top width of approximately 15m (varies with batter extent). Note this is the approximate hydraulic width. There is a discussion on waterway corridors following both the naturalisation concepts.
- The waterway should tie into existing culvert infrastructure. At this concept level no culvert upgrades are proposed (this has not been assessed). Waterway pools should occur upstream and downstream of the culverts as this can help with sediment drop-out and therefore avoiding culvert blockages.
- The waterway narrows south of Merry Street to simply a low flow channel cut into the existing shallow floodway. This site is challenging given the limited space available and the fact that the waterway goes through private property. It is recommended that land be purchased for a drainage easement.
- At George Street the configuration of the George Street pipe to take low flows and Alfred Street to take overland flows would remain.

Table 19. Peak flows used for the waterway design downstream of Maffra RB

	1EY (m^3/s)	1 % AEP (m^3/s)
RB out	0.3	3.05
Downstream of Powerscourt Street (and Powerscourt WLRB)	1.0	6.14

A concept plan of the waterway is provided in Figure 46 below. Landscape sketches are provided in Appendix F.

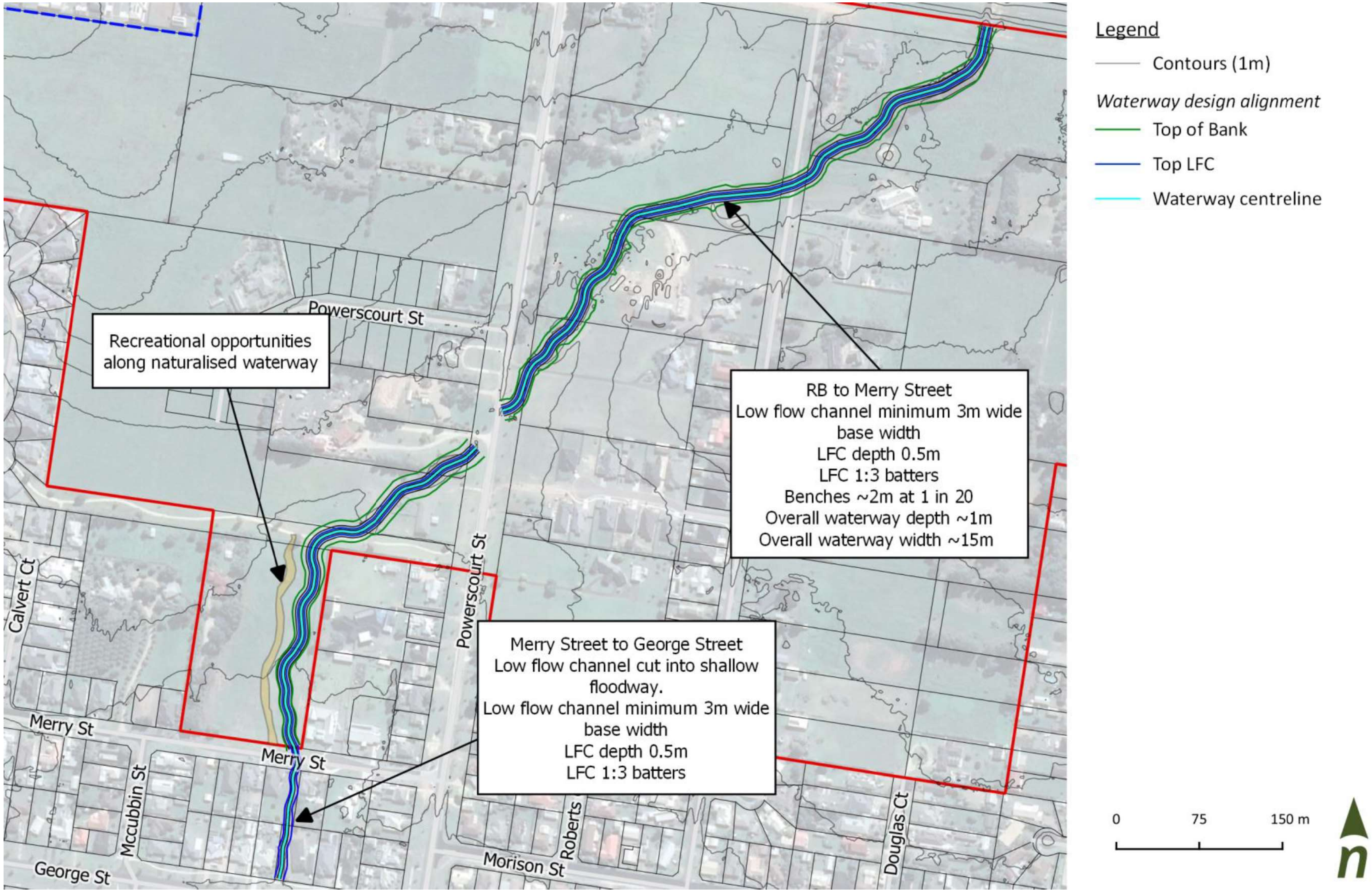

Figure 46. *Waterway formalisation downstream of the Maffra retarding basin*

Davis Street Drain concept
There is a good opportunity to naturalise the concrete channel between Alfred Street and Landy Street, continuing the waterway east until it transitions to existing levels within the waterway. Between Alfred Street and Powerscourt Street there is a narrow concrete channel and grassed floodway between residential properties (approximately 18m wide, see Figure 47). Some mature trees line the boundary. This waterway takes flows from Alfred Street and is essentially a continuation of the waterway that comes out of the Maffra RB.

East of Powerscourt Street the arrangement is again a shallow and narrow concrete channel within a wide grassed floodway, but the space available here is much greater at a width of approximately 34m. East of Landy Street the waterway becomes an earthen channel before transitioning into a more natural waterway upstream of Fulton Road.

The opportunity here is a good one because the site is not space constrained and could therefore fit a wider waterway, but also because it presents some recreational opportunities alongside the waterway. A naturalised waterway through this site could improve flooding conditions and create habitat opportunity.

Figure 47. *Looking east from Alfred Street (left) and looking east from Landy Street at the earthen channel (right)*

A concept-level waterway was designed from Alfred Street to the east of Landy Street using the following design assumptions and criteria:

- The proposed waterway alignment is through the centre of the grassed floodway (the concrete channel hugs the southern boundary of the easement east of Powerscourt Street). The alignment has been centred in this space to allow a buffer between the waterway and property fences, as well as to allow for path networks. The waterway has been designed in 12d, an earthworks modelling program, based on the existing surface created from LiDAR. No hydraulic modelling to test shear stress has been conducted at this stage (later design stages).
- The waterway naturalisation follows a similar arrangement to the previously presented waterway (a compound channel), although this arrangement is simpler in that a low flow channel is proposed to be set into the floodway. Therefore, the floodway will act as the high flow channel as currently occurs. Low flow channels are traditionally designed to take between the 4EY and 1EY flows, and the high flow channel should have the capacity to take the 1% AEP flows. This is in line with the Constructed Waterways Design Manual (CWDM) (Melbourne Water, 2019).
- The waterway should meander to create diversity in planform.
- Flows were adopted from the RORB modelling assuming post-development conditions, and the flood mitigation (storage) options previously presented in Section 5. These are the peak flows (Table 20). At this stage the larger flow events (i.e. the downstream peak flows) have been adopted to design the entire waterway. This is a conservative approach which can be refined in later design stages.

- The low flow channel should have a minimum base width of 3m, minimum depth of 0.5m and 1:3 batters (in line with the CWDM), as well as benches, which create diversity in form and habitat niches.
- The waterway low flow channel was designed to have a base width of 6m, depth of 0.6m, 1 in 3 batters, manning's n value of 0.05 (vegetated), and longitudinal grade of approximately 1 in 200 (varies). This results in a low flow channel capacity of approximately 4.0m^3/s.
- An equivalent top width of the low flow channel of approximately 10m (varies with batter extent).
- The waterway should tie into existing culvert infrastructure. At this concept level no culvert upgrades are proposed (this has not been assessed). Waterway pools should occur upstream and downstream of the culverts as this can help with sediment drop-out and therefore avoiding culvert blockages. Evidence of culvert blockage is apparent at the Landy Street culverts so the works here would improve this.
- The waterway would transition to match in with existing waterway invert levels just upstream of Fulton Road.

Table 20. Peak flows used for the waterway design (Davis Street Drain naturalisation)

	1EY (m^3/s)	1 % AEP (m^3/s)
Start waterway (Alfred Street)	1.7	10.69
Landy Street	4.1	22.94

There is plenty of space for paths networks alongside the naturalised waterway. A formal concrete path could be placed on one side of the waterway to allow faster movement, and an informal, narrower gravel path could be situated on the other side to allow people to slowly move through the site.

Naturalising this site would improve the amenity, provide recreational and cooling opportunities, reduce flooding extent and provide diversity in waterway form and habitat opportunities.

A concept plan of the waterway is provided in Figure 48. Landscape sketches are provided in Appendix F.

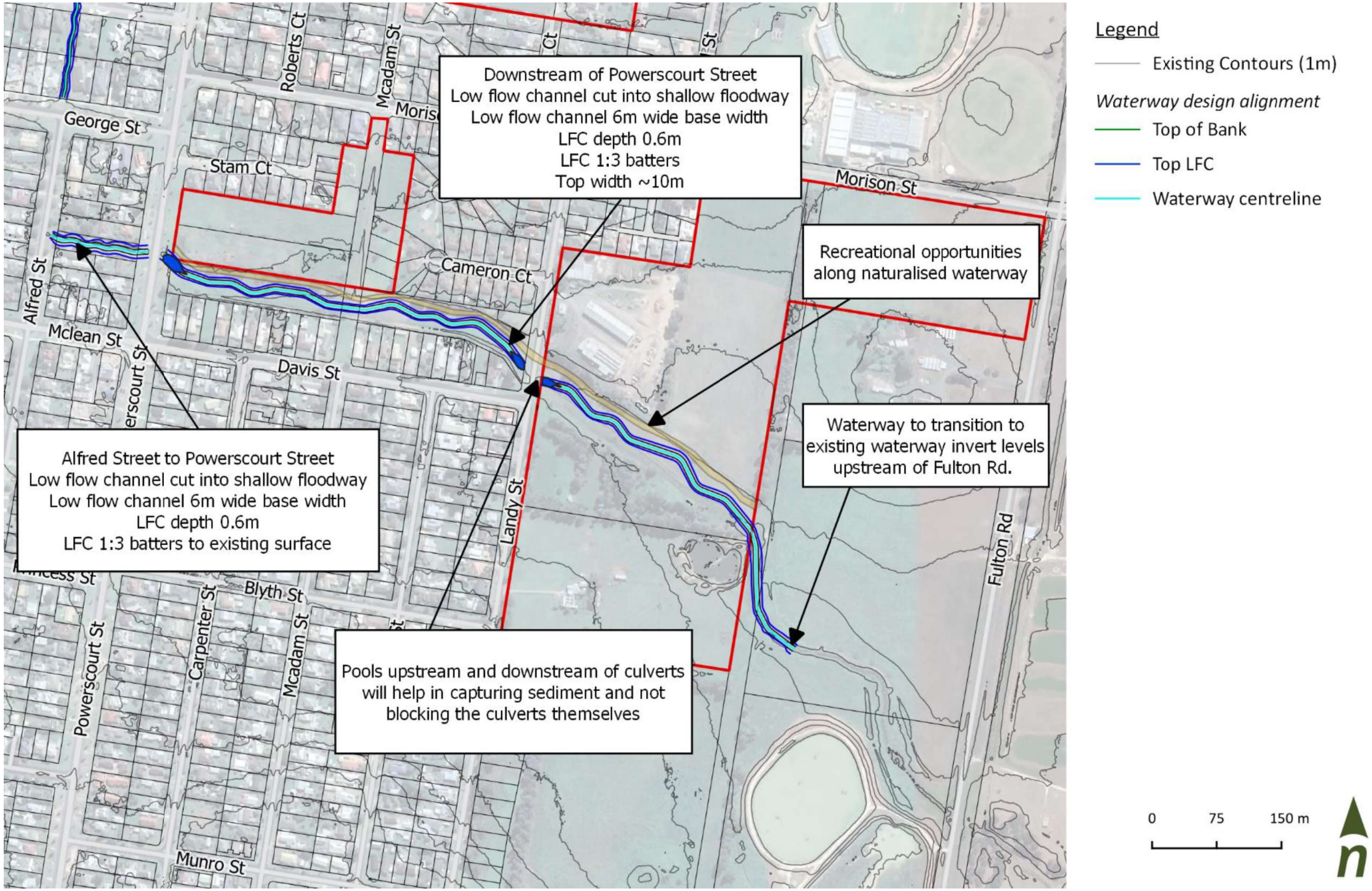

Figure 48. *Davis Street naturalisation concept design*

Waterway corridors

The waterway dimensions provided in the sections above only talk to the hydraulic widths. That is, they are not the overall waterway corridors. A waterway corridor usually includes allowances for access and recreation and vegetated buffers. Melbourne Water's Waterway Corridor Guidelines (revised guidelines under finalisation at the moment) provide the recommended corridor widths for both natural and constructed waterways. The required corridors widths for the constructed waterways are influenced by the hydraulic width. A schematic of the corridor cross section is provided in Figure 49, showing 12.5m of vegetated buffer on each side (including access paths), and a core riparian zone in the centre which includes the hydraulic width.

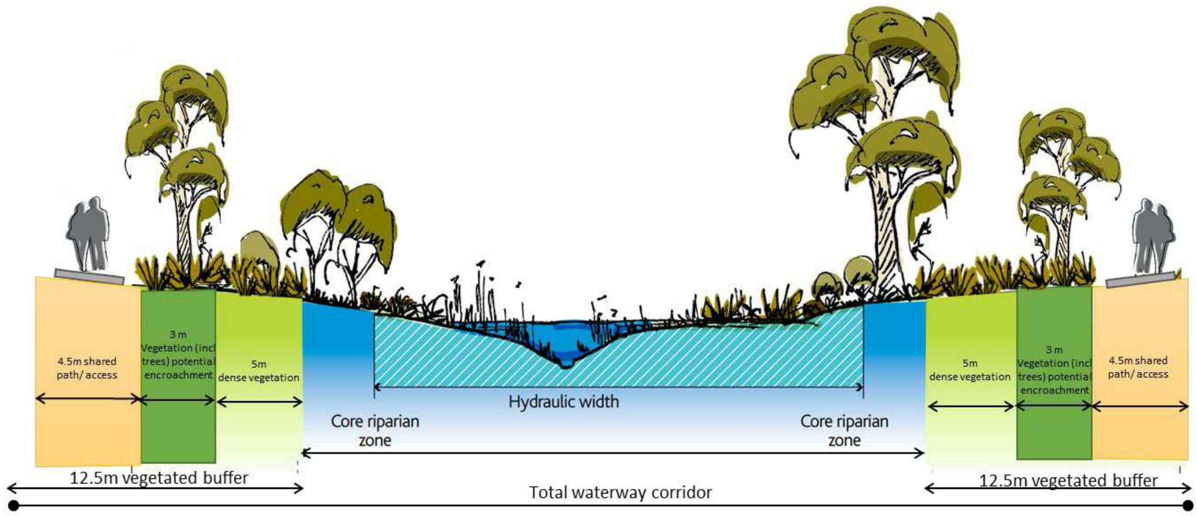

Figure 49. *Schematic cross-section of setback sub-zones for constructed waterways (Melbourne Water, Draft Revised Waterway Corridor Guidelines, 2019)*

The guidelines suggest that for a waterway with a hydraulic width of 15m (which is approximately the hydraulic width of the RB waterway), that a corridor of 45m is required. This includes a 20m core riparian zones and 25m buffer width. Although a 45m wide waterway corridor is recommended here, there is potential to reduce this corridor to 30m and still meet vegetation, recreation and access outcomes. This could be done for the reach from the RB to Powerscourt Street. Under this scenario the vegetated buffer zone would be reduced (noting that the waterway itself is vegetated). There may be a formal, wider access path on one side (allowing maintenance access), and a narrower, informal gravel path on the other side. If open space elements are included alongside the waterways, these can be incorporated into the waterway corridor. They should not, however, intersect with the hydraulic width (i.e. unencumbered open space).

Difficulties with providing the waterway corridor will be encountered in the narrow section between Merry Street and George Street and Alfred Street and Powerscourt Street. The Davis Street Drain naturalisation reach indicates that ~40m is available for a waterway corridor within the current easement. Downstream of Landy Street the corridor is likely to need to be 45m. Again, adjacent open space may be included within this. Where land ownership allows for a wider corridor, this should be adopted.

The waterway corridors will need to be confirmed in subsequent design stages when the hydraulic width is confirmed, however it is recommended that a *minimum* of 30m-45m be provided to accommodate the hydraulic width, access and vegetation buffers.

8.3 Smart or 'talking' tanks

The idea of smart rainwater tanks was discussed with the Shire in the context of new development being planned within areas that are currently subjected to flooding. The smart or talking tanks concept seeks to use on lot rainwater storages collectively to meet catchment wide objectives. This may include:

- Flood mitigation
- Improved waterway health
- Reduced demands on potable water supplies.

In principle, a central operator, most likely a water authority, is charged with controlling the levels within the rainwater tanks in response to prevailing weather conditions and the likelihood of rainfall. Should a rainfall event be anticipated, then the tanks can be 'bled' to provide air space within the tank. This reduces the flashiness of flows leaving each property, and collectively, the peak of the given rainfall event.

Most notably this has been applied at 'Aquarevo', a residential development within South East Water's business area in Metropolitan Melbourne. Their data is some of the most advanced on this subject with results suggesting approximately 26% of stormwater runoff is being reduced.

While the concept is being proved, that conditions would be very different from Maffra, given the likelihood of larger lots and relatively smaller roof sizes, unlike Aquarevo where the house takes up much of the block. In this instance the impact may be lessened, however as a theoretical investigation, it may be worth understanding the impact of distributed and smart rainwater tanks on the peak flows of moderate rainfall events (rather than the 1% AEP event) within flood effected areas of Maffra.

Some of the key barriers includes:

- Understanding the impact and benefit of the concept through additional flood modelling.
- Attracting collaboration and potential funding support from Gippsland Water to undertake that work and to partner should that work proceed.
- Engaging Council, developers and prospective residents in a program whereby an external party has control over the level in the rainwater tank (including the likelihood for access to maintain that infrastructure).

9 Staging

The drainage assessment is for the ultimate development scenario. Development will not necessarily occur in a linear upstream-downstream sequence ('out of sequence development'). Developments are frequently constructed out-of-sequence, by different developers and designed by different consultant teams.

Development staging must provide for early delivery of ultimate waterway/drainage infrastructure including stormwater quality treatment. Where this is not possible, development must demonstrate how any interim solution adequately manages and treats stormwater generated from the development and how this will enable delivery of an ultimate drainage solution, all to the satisfaction of the responsible authority.

During the finalisation of this strategy it was highlighted that the Lot 1 and Lot 2 properties west of Powerscourt Street were likely the first to be developed. These are shown in Figure 50, along with some concept RBs location and sizing to manage stormwater from the sites (preliminary assets as developed by WSC).

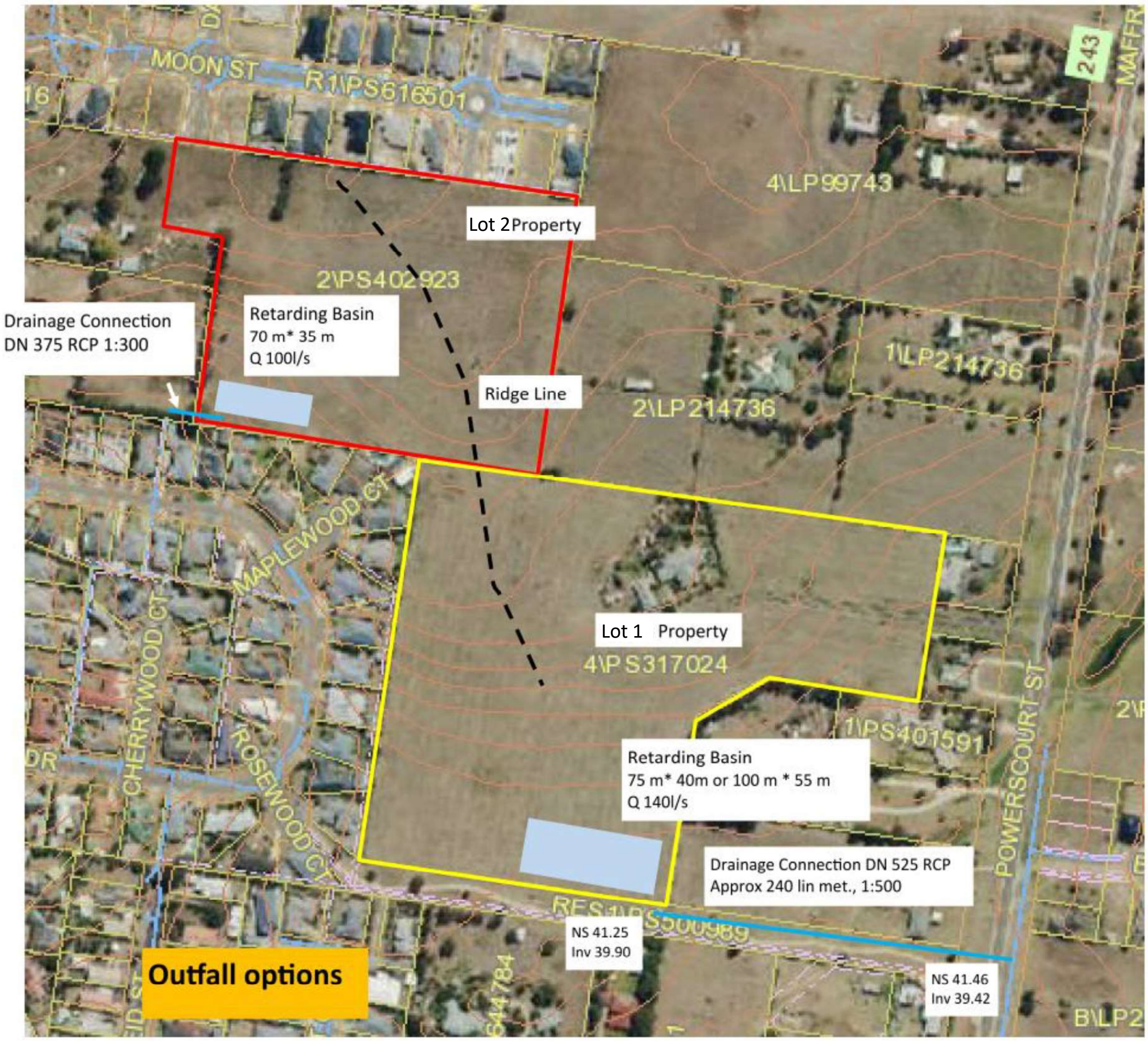

Figure 50. *Properties likely to develop first and high-level proposed assets (source: WSC)*

The RB shown in the Lot 1 property is similar in terms of the location to the Powerscourt WLRB proposed as part of this drainage strategy (Figure 37). The Powerscourt WL/RB proposed is larger than that shown in the above concept as it is receiving water from a catchment greater than just the Lot 1 property (including part of the Lot 2 property).

It is recommended that the RB be sized for the ultimate RB conditions (i.e. entire contributing future residential catchment and not just the Lot 1 property). An interim solution of the RB being sized just for the Lot 1 property contribution could occur but would just need to be upsized in the future. This is likely a question and funding and timing of future development to the north of this property.

The Powerscourt WLRB is also proposed to be sited further east such that outfall into the proposed formalised waterway is more efficient. The purchase of land is likely to be required to enable this drainage strategy to work.

No asset within the Lot 2 Property was initially proposed as part of the broader strategy (i.e. that the developable land forms a very small part of the overall catchment into the wetland adjacent to the showgrounds, and therefore storage requirements are small -see Figure 32). However, an asset to manage the quantity of water coming off the property will be required to ensure downstream existing residential houses are not impacted by the development. Council has recommended that the detention be provided through a combination of buried oversized pipes or underground system and stormwater tanks.

A small retarding basin was considered, but this would be a small asset at the back of the housing development. It would therefore provide little in the way of amenity and recreation opportunities, biodiversity outcomes and passive open space. The asset would also add to maintenance requirements and require dedicated maintenance access for what would be a small asset. Therefore, underground detention systems would be preferable.

Table 21 provides the recommended potential staging of works.

Table 21. Recommended staging steps

Staging	Recommended works
1	Should the Lot 1 and Lot 2 properties be developed first, assets will need to be built here first to manage runoff associated with the development. This will be critical in terms of managing impacts on downstream housing.
2	The Maffra RB works should occur as early as possible as this will have a significant impact on downstream flooding. The wetland treatment asset is less critical (it should be built prior to the upstream development occurring).
3	The waterway downstream of the Maffra RB should be formalised following the Maffra RB works. This will again reduce flooding extents, enabling more land to be developed surrounding the waterway and improving amenity.
4	The north west WL/RB should be built once development within that catchment is due to begin.
5	The south west wetland works are less critical in terms of timing as the works here will really be dealing with treatment of the existing residential catchment and improving the quality of the water entering the Maffra Wetland Reserve.
6	The Davis Street naturalisation again can occur at any time. The works here will improve flooding conditions but also the amenity of the area.

10 Costing

Costing of the various stormwater quality treatment and storage options has been undertaken to provide a preliminary cost estimate (strategy level). This includes the costs associated with the construction of the retarding basins and wetland treatment aspects of the designs, as well as the proposed waterway works. The treatment options were costed using Melbourne Water's Water Sensitive Urban Design (WSUD) Life Cycle Costing data sheet (see Appendix D). The cost estimates are intended to be a starting point only and do not include land costs. They do include all excavation, planting and infrastructure, although this will be subject to site constraints and complexity. Table 22 presents the capital, establishment, and ongoing costs of the treatment assets (the capital costs include planning and design costs).

Table 22. Maffra treatment wetland costings

Option	Assets	Treatment area (NWL) (m2)	Capital Cost per m^2	Capital cost	Maintenance cost per m^2 per year	Establishment cost (first 2 years)	Maintenance Cost ($/year)
Catchment East Wetland (adjacent to Maffra RB)	Wetland	10,000	$75	$750,000	$0.5	$35,000	$5,000
	Sediment basin	3,500	$150	$525,000	$5.0	$122,500	$17,500
	TOTAL	**13,500**		**$1,275,000**		**$157,500**	**$22,500**
Catchment O Wetland (Powerscourt Wetland)	Wetland	4,000	$100	$400,000	$2.0	$56,000	$8,000
	Sediment basin	800	$200	$160,000	$10.0	$56,000	$8,000
	TOTAL	**4,800**		**$560,000**		**$112,000**	**$16,000**
Catchment North West Wetland	Wetland	11,500	$75	$862,500	$0.5	$40,250	$5,750
	Sediment basin	1,800	$150	$270,000	$5.0	$63,000	$9,000
	TOTAL	**13,300**		**$1,132,500**		**$103,250**	**$14,750**
Catchment South West Wetland (next to showgrounds)	Wetland	14,000	$75	$1,050,000	$0.5	$49,000	$7,000
	Sediment basin	1,700	$150	$255,000	$5.0	$59,500	$8,500
	TOTAL	**15,700**		**$1,305,000**		**$108,500**	**$15,500**
Total				**$4,272,500.00**		**$481,250**	**$68,750**

*2-5 times ongoing maintenance cost (we assumed 3.5 x)

Table 23 provides an estimate of the costs associated with the storage (excavation) requirements for the RBs. These are not captured in the treatment estimates, and thus have been provided below. Delivery costs such as design and planning, site establishment and contingency (30%) have been included. This has been done as a percentage of the total of all assets (i.e. not for each individual asset). Again, land costs have not been included.

Table 23. Retarding basin works cost estimate

Retarding basin works	Quantity	Unit	Rate	Amount		
Maffra RB - Excavation for RB storage	99,173	m3	$10.0	$991,730		
Powerscourt RB - Excavation for RB storage	8,081	m3	$10.0	$80,810		
Catchment North West RB - Excavation for RB storage	39,000	m3	$10.0	$390,000		
Total RB excavation works				**$1,462,540**		
Delivery costs				**Maffra RB**	**Powerscourt RB**	**North West RB**
Traffic Management (5%)			5%	$49,587	$4,041	$19,500
Environmental Management (0.5%)			0.5%	$4,959	$404	$1,950
Survey & Design (10%)			10%	$99,173	$8,081	$39,000
Supervision & Project Management (9%)			9%	$89,256	$7,273	$35,100
Site Establishment (2.5%)			2.5%	$24,793	$2,020	$9,750
Contingency (30%)			30%	$297,519	$24,243	$117,000
Subtotal Delivery				**$565,286**	**$46,062**	**$222,300**
Total RB works (excavation + delivery)				**1,557,016**	**$126,872**	**$612,300**
Total estimate RB works costs				**$2,296,187.8**		

Table 24 below provides a cost estimate for the waterway works. This includes excavation (as established in the earthworks modelling), planting and some allowances for rockwork, paths and erosion control. Note these are very high-level costs based on preliminary concepts. The concept designs should be developed further to refine costs and capture all items, in particular landscaping elements that may be desired (e.g. final path network, seating, waterway crossings, stepping stones, viewing platforms etc.).

Table 24. Waterway works cost estimate

	Waterway works	Quantity	Unit	Rate	Amount	
Waterway transformation downstream of RB	Excavation RB to Powerscourt Street	5803	m3	$10.0	$58,030	
	Excavation Powerscourt to Merry Street	4178	m3	$10.0	$41,780	
	Excavation Merry Street to George St	278	m3	$10.0	$2,780	
	Waterway planting RB to Powerscourt	9105	m2	$17.0	$154,785	
	Waterway planting Powerscourt to Merry Street	5550	m2	$17.0	$94,350	
	Waterway planting Merry Street to George St	726	m2	$17.0	$12,342	
	Allowance for rockwork and drainage (e.g. connections)	1	No.	$200,000	$200,000	
	Allowance for jute matting and mulch (erosion control and weed suppression)	1	No.	$250,000	$250,000	
	Allowance for path network	1	No.	$150,000	$150,000	
	Total civil and planting works				**$964,067**	
Davis Street waterway naturalisation	Excavation Alfred Street to Powerscourt	900	m3	$10.0	$9,000	
	Excavation Powerscourt Street to Landy Street	2601	m3	$10.0	$26,010	
	Excavation Landy Street east (to transition back to natural waterway)	3371	m3	$10.0	$33,710	
	Waterway planting Alfred Street to Powerscourt	1140	m2	$17.0	$19,380	
	Waterway planting Powerscourt Street to Landy Street	4570	m2	$17.0	$77,690	
	Waterway planting Landy Street east (to transition back to natural waterway)	4650	m2	$17.0	$79,050	
	Allowance for rockwork and drainage (e.g. connections)	1	No.	$200,000	$200,000	
	Allowance for jute matting and mulch (erosion control and weed suppression)	1	No.	$200,000	$200,000	
	Allowance for path network	1	No.	$150,000	$150,000	
	Total civil and planting works				**$794,840**	
	Total (both waterways)				**$1,758,907**	
	Delivery costs				**RB waterway**	**Davis St waterway**
	Traffic Management (5%)			5%	$48,203	$39,742
	Environmental Management (0.5%)			0.50%	$4,820	$3,974
	Survey & Design (10%)			10%	$96,407	$79,484
	Supervision & Project Management (9%)			9%	$86,766	$71,536
	Site Establishment (2.5%)			2.50%	$24,102	$19,871
	Contingency (30%)			30%	$289,220	$238,452
	Subtotal Delivery				$549,518	$453,059
	Total individual waterway works costs				**$1,513,585**	**$1,247,899**
	Total estimate waterway works costs				**$2,761,484**	

A summary of the asset capital costs is provided in Table 25.

Table 25. Asset cost estimate summary (capital)

	Asset	Estimated cost
	East wetland	$ 1,275,000
	Maffra RB works	$ 1,557,016
Powerscourt WL/RB	Wetland	$ 560,000
	RB works	$ 126,872
North west WL/RB	Wetland	$ 1,132,500
	RB works	$ 612,300
	South west wetland	$ 1,305,000
	Waterway transformation downstream of RB	$ 1,513,585
	Davis Street waterway naturalisation	$ 1,247,899

11 Developed conditions flood modelling

As discussed in Section 2.7, Water Modelling Solutions (WMS) undertook flood modelling as part of this study. WMS undertook developed conditions flood modelling using a design Digital Elevation Model (DEM) provided by Alluvium which incorporated the various treatment and storage options. This DEM did not include storage below the asset NWLs. Developed conditions hydrologic inputs were also provided to WMS for input into the modelling.

The full flood modelling report is included in Appendix E, with all inputs, assumptions and results documented. This section summarises some key findings for the existing conditions.

Key findings include:

- The developed scenario results in a reduced extent of flooding more broadly across the township and due to the development of the upstream Maffra RB and formalisation of the waterways.
- In general, water levels are lower in the proposed constructed channel than along the existing channel due to flow being further retarded upstream by the increased detention basin size.
- There are some locations of afflux in the developed scenario where culverts have not been upgraded and sized as part of this project (e.g. where waterway deepening is proposed). The design of culvert upgrades is required for the next phase of the study and will ensure no adverse impact on the flooding.

Potential culvert upgrades required as part of works:

- Powerscourt Street (as part of Maffra RB waterway transformation)
- Powerscourt Street (as part of Davis Street naturalisation)
- Landy Street (as part of Davis Street naturalisation)

Figure 51 to Figure 53 provide the 1% AEP flood mapping under developed conditions. Detailed water level, depth velocity and afflux maps are provided in the full flood modelling report.

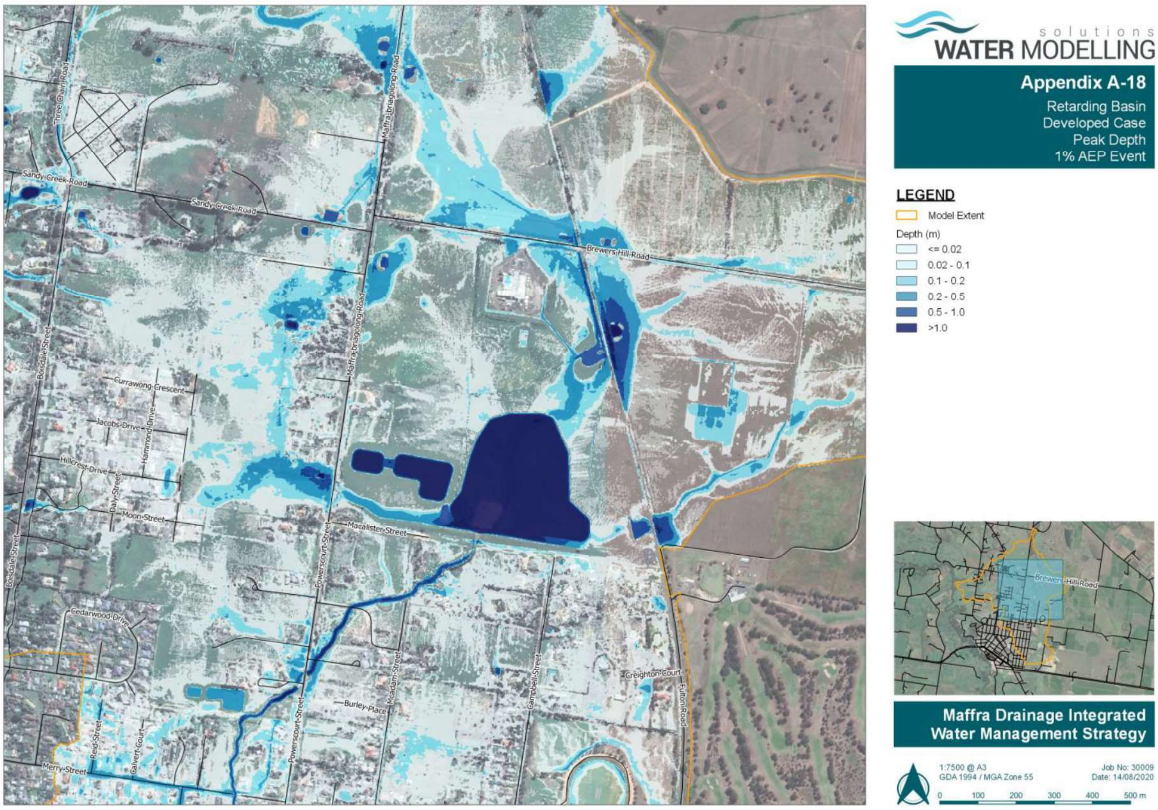

Figure 51. *Developed condition flood modelling – Maffra RB- 1% AEP*

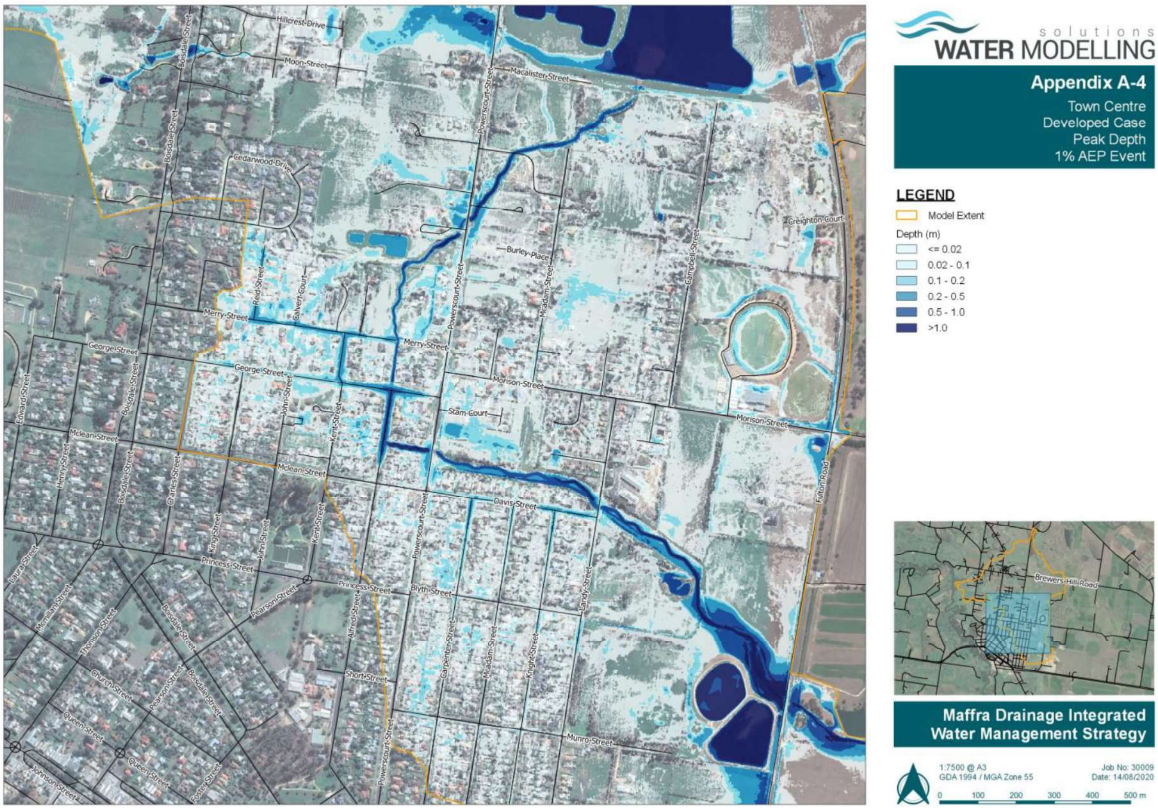

Figure 52. *Developed condition flood modelling – Town centre - 1% AEP*

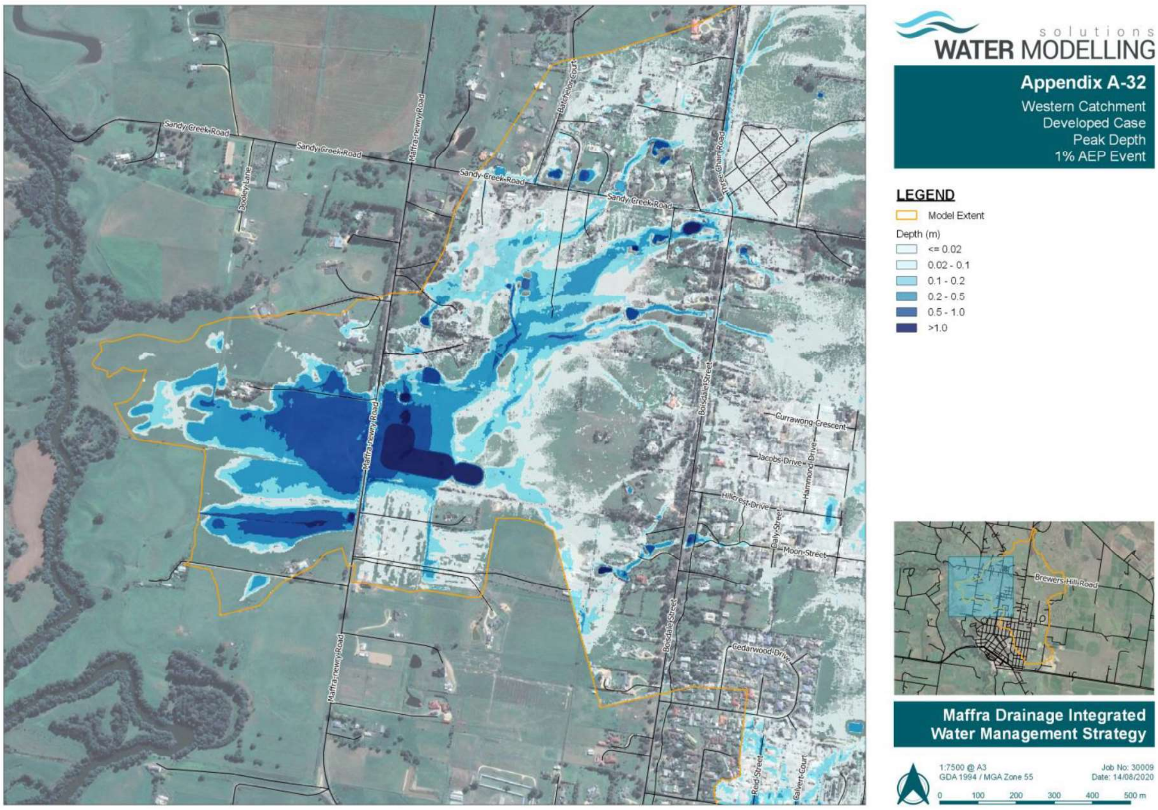

Figure 53. *Developed condition flood modelling – Western catchment- 1% AEP*

12 Summary and recommendations

Wellington Shire Council is planning for future residential expansion proposed for the north of the existing Maffra township. Alluvium and Water Modelling Solutions (WMS) were engaged to:

- develop a drainage strategy to accommodate future urban growth,
- undertake a flash flooding assessment,
- incorporate a considered assessment of Integrated Water Management (IWM) opportunities; and
- incorporate passive open space and improved amenity elements in drainage and treatment areas.

Numerous drainage assessments have been conducted within Maffra over the years, focussing on how to alleviate flooding issues particularly in the north-east of town. This assessment builds on those previous studies, identifying problem flood areas through the flood modelling, identifying drainage requirements that will be driven by future development to manage both stormwater quantity and quality, and developing concept designs for necessary assets to meet those requirements.

The assessment focusses on opportunities beyond upgrading existing stormwater pipes. It focusses on identifying assets which can help alleviate flooding while creating high-quality community assets that provide habitat, amenity, cooling and recreation opportunities.

Several options were identified within this report to meet stormwater quality, quantity and IWM objectives. These are summarised below:

- Enhancing the existing Maffra retarding basin through increasing flood storage (by ~99,000m^3) and reducing the magnitude of frequent flows through blocking one of the outlet pipes. The existing RB is currently undersized and contributing to downstream flooding issues. This will only be exacerbated with future development within the contributing catchment. Increasing the storage capacity will help alleviate downstream flooding issues through decreasing outflow, although downstream assets are also required to deal with local residential catchments as the timing of peak events will be different.
- A stormwater treatment wetland is also proposed adjacent to the Maffra RB. This has not been placed within the RB floor as it would not be able to outfall. It is proposed to therefore sit beside the RB, treating stormwater from the future residential areas and outfalling into the RB.
- A wetland/retarding basin is proposed downstream of the Maffra RB to treat local future residential areas and mitigate flooding. This WL/RB (the Powerscourt WL/RB) will likely need to be developed as early as possible given the surrounding parcels have already been identified as being ready to develop. Outflows from this asset (held back to pre-developed flow rates) are proposed to outfall into the formalised waterway.
- The waterway downstream of the Maffra RB is a CMA-designated waterway. The waterway is very shallow and informal in parts (for example near Merry Street) and is therefore proposed to be formalised to increase conveyance and reduce flooding extents. The proposed works within the Maffra RB will help reduce flows exiting the RB and therefore is part of the solution for managing the flooding issues currently experienced, however formalising this waterway will ensure flows are adequately and safely conveyed through both existing and future residential areas. Formalising this waterway will also allow stormwater outfall from the future residential areas, as well as proposed assets (e.g. the Powerscourt WL/RB). The waterway works are largely driven by flood management objectives, however transforming this waterway provides an opportunity to enhance ecological outcomes, improve amenity and provide recreational opportunities through waterway walking tracks.
- A wetland/retarding basin is proposed to manage stormwater quality and quantity associated with future development within the north-west catchment. This system will need to outfall to the Macalister River via a channel.

- The existing waterbody adjacent to the Showgrounds is proposed to be converted into a constructed wetland. The concept design uses the space available to fit a wetland and initial modelling indicates that the wetland can treat the entire contributing catchment to best practice (i.e. existing development and future development). Converting the waterbody into a treatment wetland will improve the quality of the water discharging into the Maffra Wetlands Reserve. Stormwater harvesting (which is already present at this site) can occur off the back of this wetland, either via a separate harvesting pond (i.e. so to not draw down the water in the wetland) or a tank. Storage is not required within this asset as there is no increase in peak flows due to such a small portion of the overall catchment being future development.
- A waterway naturalisation opportunity exists with the Davis Street Drain through the east of town. This drain currently exists of a narrow and shallow concrete channel in a larger grassed shallow floodway. Creating a 'naturalised' waterway through cutting in a meandering vegetated low flow channel will increase conveyance, improve the amenity of the site, provide cooling, enhance ecological outcomes, and enhance recreational opportunities through the provision of paths alongside the waterway.
- Opportunities for stormwater harvesting and irrigation associated with those wetland assets, with the highest priority harvesting opportunity proposed being from the wetland adjacent to the Maffra Recreational Reserve.

The options are provided in a summary map in Figure 54.

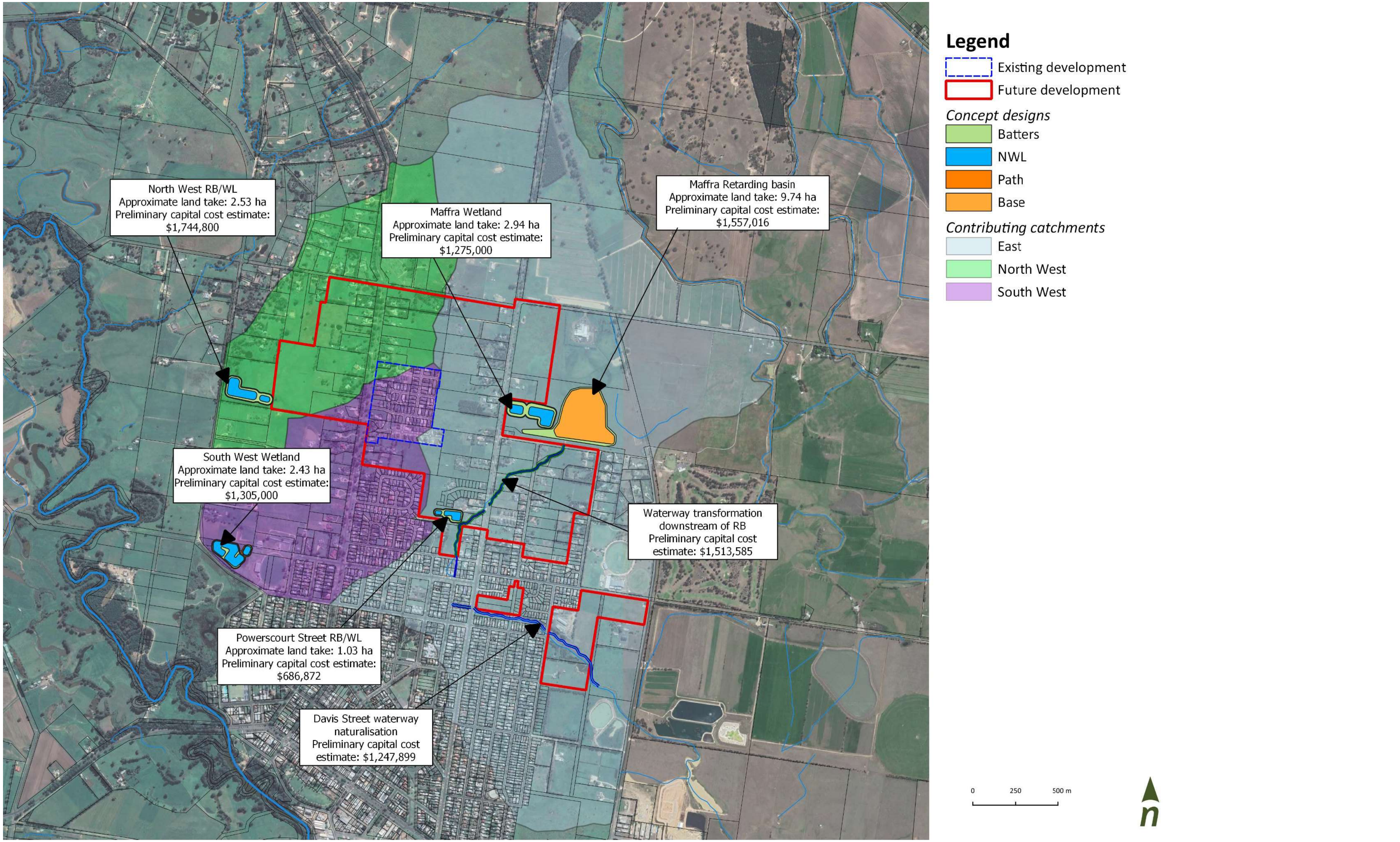

Figure 54. *Summary map of proposed assets*

Next steps and recommendations for progressing the drainage assessment within Maffra include:

- Functional design of proposed flood mitigation and stormwater quality assets
- Functional design of waterways including hydraulic modelling to ensure shear stress thresholds are not exceeded
- Recommendation of the purchase of land for drainage purposes. This will need to include land for the assets and waterway alignments as currently the waterway passes through private land. The asset locations and arrangements as proposed within this report are somewhat flexible (i.e. can shift slightly should parcel purchase dictate this) but have largely been located in the most appropriate locations (for example of outfall purposes). Functional designs of the assets should follow the purchase of land so the space constraints are known prior to development of the assets.
- The staging of development will need to be confirmed to identify and further develop the assets required with the associated development. Given the Lot 1 andLot 2 properties are likely to be developed first, the Powerscourt WL/RB will need to be prioritised to enable the development of those sites.
- Proceed with the design for the Maffra Recreation Reserve wetland, incorporating stormwater harvesting functionality and infrastructure. As part of that design, review and confirm the optimised storage volume and investigate storage options including
  - a stand-alone, above ground tank, and
  - storage incorporated into the wetland. As part of the second option, investigate the potential for the operational parameters of the wetland to change, such that harvesting (including additional storage capability) is accommodated during summer months, while conventional operation of the wetland resumes during non-irrigation periods.

13 References

Cardno (2009). Catchment Analysis

Cardno (2010). Maffra Drainage Review

CSIRO (1999), Urban stormwater best practice environmental management guidelines.

Fisher Stewart (1998). Review of Drainage Outfalls, Maffra.

Fisher Stewart (1999). Review of Drainage Proposals and Technical Assessment of Hydraulic Characteristics.

Fisher Stewart (November 1999). George Street Proposed Replacement Drainage designs.

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Fisher Stewart (February 2000). Retarding basin designs.

Fisher Stewart (March 2003). Maffra Stormwater Drainage Memo. A memo provided to Council on the retarding basin as-constructed design dimensions.

Geoscience Australia (2019). Australian Rainfall and Runoff: A guide to flood estimation.

Water Technology (July 2014). Retarding Basin Performance Review and Optimisation – Maffra.

Water Modelling Solutions (September 2020). Maffra Drainage and Integrated Water Management Strategy Hydraulics Report.

Appendix A
Hydrologic modelling

Input parameters

Model inputs were obtained from the ARR2019 data hub and the Bureau of Meteorology's IFD data. An initial loss continuing loss model configuration was adopted.

For all models:

- Temporal Patterns - Southern Slopes (Vic/NSW)
- Catchment fraction imperviousness based on values in Table 2 and Table 3
- *K_c=1.25 * dav (for Victorian catchments Pearse et al. 2002)*

The kc values adopted for each model are shown in Table 19 as well as the initial loss (IL) and continuing loss (CL) values. The justification of the kc equation adopted for the models is provided in the calibration section below.

Table 26. RORB models and parameters used

RORB model	Total Area (km^2)	Kc	m	IL (mm)	CL (mm/hr)
East	5.57	3.28	0.8	16	2.7
North West	1.35	1.24	0.8	16	2.7
South West	1.27	1.30	0.8	16	2.7

Method

The RORB models were used to estimate key design flows throughout the catchment and size retarding basin storages. In accordance with best practice modelling procedures, at least 4 subareas exist upstream from the point of interest. The hydrologic modelling considered an ensemble simulation for the 1% Annual Exceedance Probability (AEP) event, for durations 10 minutes to 72 hours. From the ensemble simulation, ten temporal patterns were used to determine peak runoffs for each duration. The median flows (i.e. 6^{th} highest peak flow) for each storm duration was determined, and the peak critical flow with respect to storage was calculated.

Following the release of the updated Australian Rainfall & Runoff (ARR) 2019 guidelines in April 2019, a new approach is to be undertaken when estimating peak runoff from a specified catchment. Key changes that will influence the hydrologic modelling outputs include:

- Updated Intensity Frequency Duration (IFD) data based on updated rainfall data from a number of rainfall stations. This is sourced from the Bureau of Meteorology's (BoM) website.
- Running the model based upon an ensemble of temporal patterns sourced from the AR&R data hub and determining the median peak flow for a given storm event and duration, rather than using a single temporal pattern.
- Using Areal Reduction Factors from a modified version of the Bell's method, which is sourced from the ARR data hub, rather than using Areal Reduction Factors sourced from AR&R 87 (Siriwardena and Weinmann).
- Using an Initial Loss / Continuing Loss model, rather than a Runoff Coefficient model.
    - Where Initial Loss values are generally 10-25 mm (based on ARR Datahub),
    - and Continuing Loss values of 1-3 mm/h (based on ARR Datahub).

Given stormwater management infrastructure was previously designed and assessed following ARR 87 design guidelines, the updated ARR 2019 guidelines includes a more conservative approach to hydrologic modelling, and higher peak runoff volumes are generally estimated when compared to the ARR 87 guidelines.

Rainfall estimation calibration

In line with the Australian Rainfall & Runoff (2019), calibration of the hydrologic model (i.e. RORB model) is required in order to determine the estimation of rainfall intensities for a specific site.

The Australian Rainfall & Runoff 2019 guidelines suggests that the model is calibrated in line with the Regional Flood Frequency Estimation model (RFFE), whilst using Initial Loss (IL) & Continuing Loss (CL) values provided from the ARR datahub.

Following the review of the Cardno (2009) and Water Technology (2014) Maffra retarding basin assessment reports, the peak flow estimations for the RORB model are not directly comparable as the models were estimated in line with the previous Australian Rainfall & Runoff (1987) guidelines, where RORB model calibration was determined in line with the rational method of flow estimation.

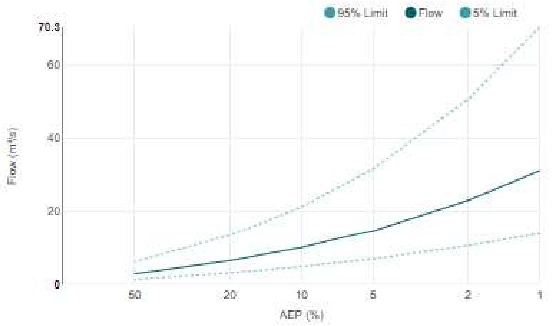
A summary of the Cardno and Water Technology reports Kc values and 1% AEP flows from their modelling are provided in Table 27 below.

Table 27. RORB model parameters & 1% AEP flows

RORB model	Kc	Catchment entering RB	RB inflow	RB outflow	Powerscourt St
Cardno	3.54	256 ha	8.5 m^3/s	7.9 m^3/s	8.0 m^3/s
Water Technology	2.05	289 ha	10.5 m^3/s	7.1 m^3/s	6.5 m^3/s

Given the design models within the Cardno and Water Technology reports have considered the catchment immediately upstream of the existing retarding basin, the kc calibration has been determined for a similar catchment (i.e. 293 ha as established in our catchment mapping). Figure 55 below provides the RFFE model output.

Results | Regional Flood Frequency Estimation Model



AEP (%)	Discharge (m^3/s)	Lower Confidence Limit (5%) (m^3/s)	Upper Confidence Limit (95%) (m^3/s)
50	2.89	1.35	6.18
20	6.49	3.13	13.6
10	10.1	4.85	21.3
5	14.8	6.99	31.7
2	23.1	10.6	50.6
1	31.2	14.0	70.3

Input Data	
Date/Time	2020-06-26 14:11
Catchment Name	Catchment1
Latitude (Outlet)	-37.942
Longitude (Outlet)	146.996
Latitude (Centroid)	-37.931
Longitude (Centroid)	147.002
Catchment Area (km^2)	2.93
Distance to Nearest Gauged Catchment (km)	16.84
50% AEP 6 Hour Rainfall Intensity (mm/h)	5.711271
2% AEP 6 Hour Rainfall Intensity (mm/h)	14.32875
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 v1
Region Source (User/Auto)	Auto
Shape Factor	0.78
Interpolation Method	Natural Neighbour
Bias Correction Value	-0.083

Figure 55. *RFFE rainfall estimation – Maffra site*

Given the RFFE is significantly different to the flows determined in the Cardno and Water Technology reports, an analysis of varying regional Victorian Kc formulas was completed. Noting the average annual rainfall for Maffra according to the BoM website is less than 800mm. The following formulas investigated were:

- $Kc = 0.49 \times A^{0.65}$ (for regions with mean annual rainfall less than 800mm), i.e. Kc = 1.50
- $Kc = 1.25 \times D_{av}$ (for Victorian catchments Pearse et al. 2002), i.e. Kc = 3.28
- $Kc = 2.57 \times A^{0.45}$ (for regions with mean annual rainfall of greater than 800mm), i.e. Kc = 5.57

When running the RFFE model, there appears no data points of relative catchment size to the study area (our catchment is 293ha up to the RB), which does suggest the flow from RFFE is not directly relatable and a flow between the upper and lower confidence limit is more likely (i.e. 14.0 m³/s to 70.3 m³/s for the 1% AEP) (Figure 49).

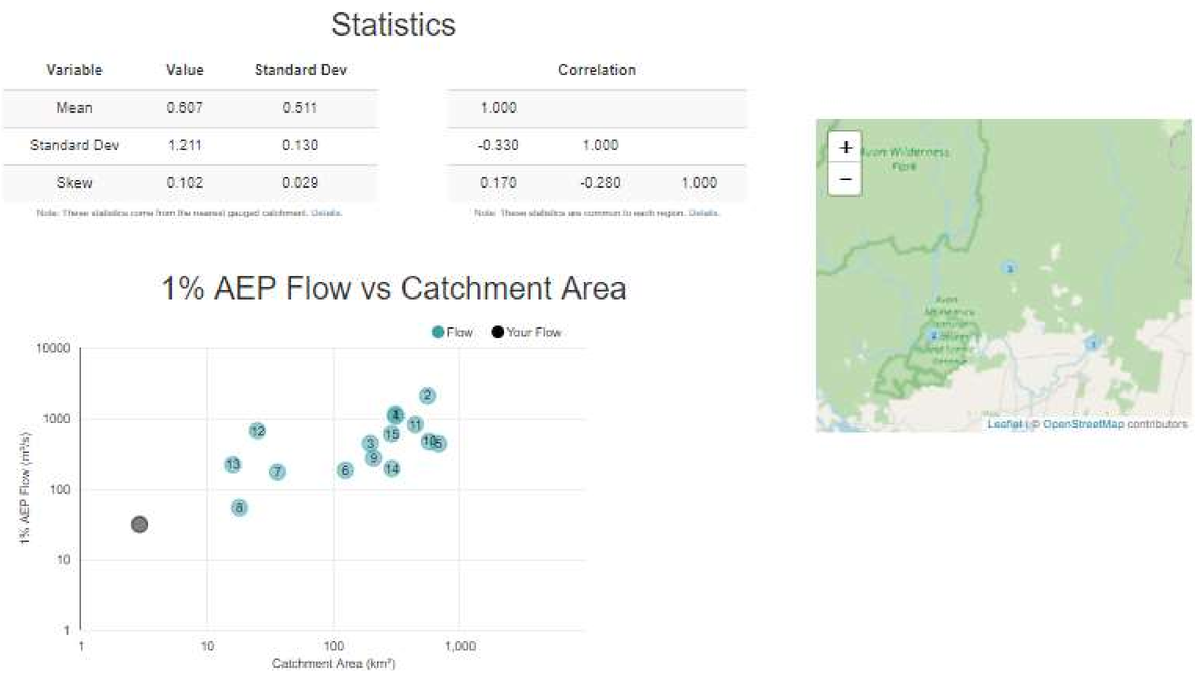


Figure 56. *RFFE rainfall station statistics – Maffra site*

As a result, a check was performed using the rational method and factored this up for a rural catchment, whilst applying an area size factor (Fa) and the ARI factor (Fy) from the VicRoads drainage manual, where:

$$P_Y = P_{10} F_Y F_A$$

Where

P_Y = the discharge factor for "Y" year ARI

P_{10} = the 10 year ARI factor read from Figure 7.2.1 (rural catchments only)

F_Y = an ARI factor read from Table 7.2.8

F_A = an area size factor, which may be read from Figure 7.2.2 or calculated from:

$F_A = 1.0$ (F_A Not Applicable)
If A > 5000 ha

$F_A = \{1.6 - 0.6(A-1000)/4000\}$
If 1001 < A < 5000 ha

$F_A = \{2.1 - (A/2000)\}$
30 1 < A < 1000 ha

$F_A = 2.0$ approx.
If 0 < A < 300 ha

And Fy is taken from the table below: (Table 7.2.8)

Average recurrence Interval (years)	F_Y
1	0.65
2	0.75
5	0.90
10	1.00
20	1.10
50	1.20
100	1.30

Source: After Table 5.4 of ARR 1987 (Ref 13)

Where P_{10} = 0.184

The rational flow when applying the above factors resulted in a peak 1% AEP flow of 19.0 m^3/s (Bransby Williams).

In comparison, when not applying the areal factors, rational results in ~10 m $^3/s$ (similar to Watertech/Cardno).

Following an analysis of the RFFE tool and rational method, the Pearse et al formula for Victorian catchments (i.e. 1.25 * dav), giving a peak RB inflow of 17.56 m^3/s for the 1% AEP RB inflow correlated the most with the rational method flows, which still lies within the confidence limit of the RFFE while remaining relatively similar to the rational calculations when areal reduction factors are considered.

As a result, the Pearse et al. formula for Victorian catchments was chosen for the Kc model calibration, and flows were determined using RORB.

A summary of results are shown below for the Maffra RB inflow (existing conditions) (Table 26).

Table 28. Summary of Kc calibration flows

Kc	RB inflow (m^3/s)
1.50 ($0.49*A^{0.65}$) Rainfall <800mm	31.10
3.28 (1.25*dav) Victorian catchments	17.56
5.57 ($2.57*A^{0.45}$) Rainfall >800mm	11.89
RFFE	14.0 - 31.20 - 70.3
Rational (applying Vicroads Areal factors) (Bransby Williams)	19.00
Rational (applying Vicroads Areal factors) (ARR87)	20.39
Rational (applying NO Vicroads Areal factors)	10.00
Watertech	10.50
Cardno	8.50

PMF modelling

GSDM PMP estimation parameters are shown in Table 30. Table 30 shows the PMP depths and intensities for the site. GSDM temporal patterns for rainfall depth increments are shown Figure 57. Note that areal reduction factors are built-in to the GSDM PMP estimation by the standard depth-duration-area curves (see Figure 58). The depth-duration curves have bene used to determine the PMP rainfall depth and intensity (see Table 30).

Table 29. GSDM parameters for the site (eastern catchment)

	Eastern catchment	North west catchment	North south catchment
Catchment area (km^2)	5.58	1.35	1.27
Terrain type	Smooth	Smooth	Smooth
Elevation adjustment factor	1	1	1
Moisture adjustment factor	0.55	0.55	0.55

The losses were assumed as IL = 0mm and CL = 1mm, for a PMP with equivalent AEP of approximately 1 in 10,000,000.

Table 30. Estimated PMP depths and intensities for different duration events

	Eastern catchment		North West		North south	
Duration (hr)	**PMP depth (mm)**	**PMP intensity (mm/hr)**	**PMP depth (mm)**	**PMP intensity (mm/hr)**	**PMP depth (mm)**	**PMP intensity (mm/hr)**
0.25	130	520.0	140	560.0	140	560.0
0.5	180	360.0	190	380.0	190	380.0
0.75	230	306.7	240	320.0	250	333.3
1	270	270.0	280	280.0	290	290.0
1.5	300	200.0	320	213.3	320	213.3
2	340	170.0	360	180.0	360	180.0
2.5	360	144.0	380	152.0	380	152.0
3	380	126.7	400	133.3	400	133.3
4	410	102.5	440	110.0	440	110.0
5	440	88.0	470	94.0	480	96.0
6	470	78.3	500	83.3	500	83.3

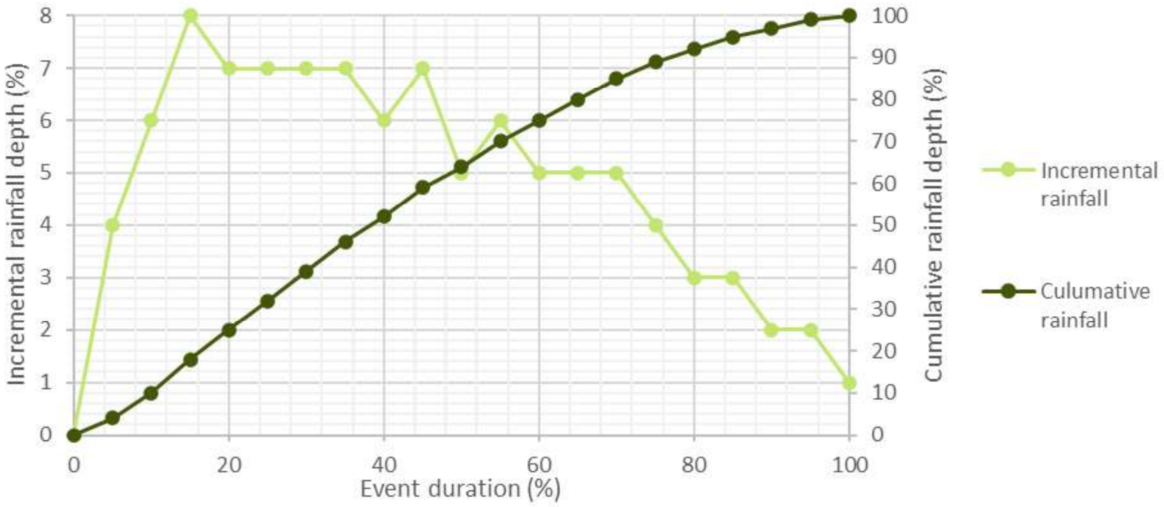


Figure 57. *GSDM rainfall temporal patterns*

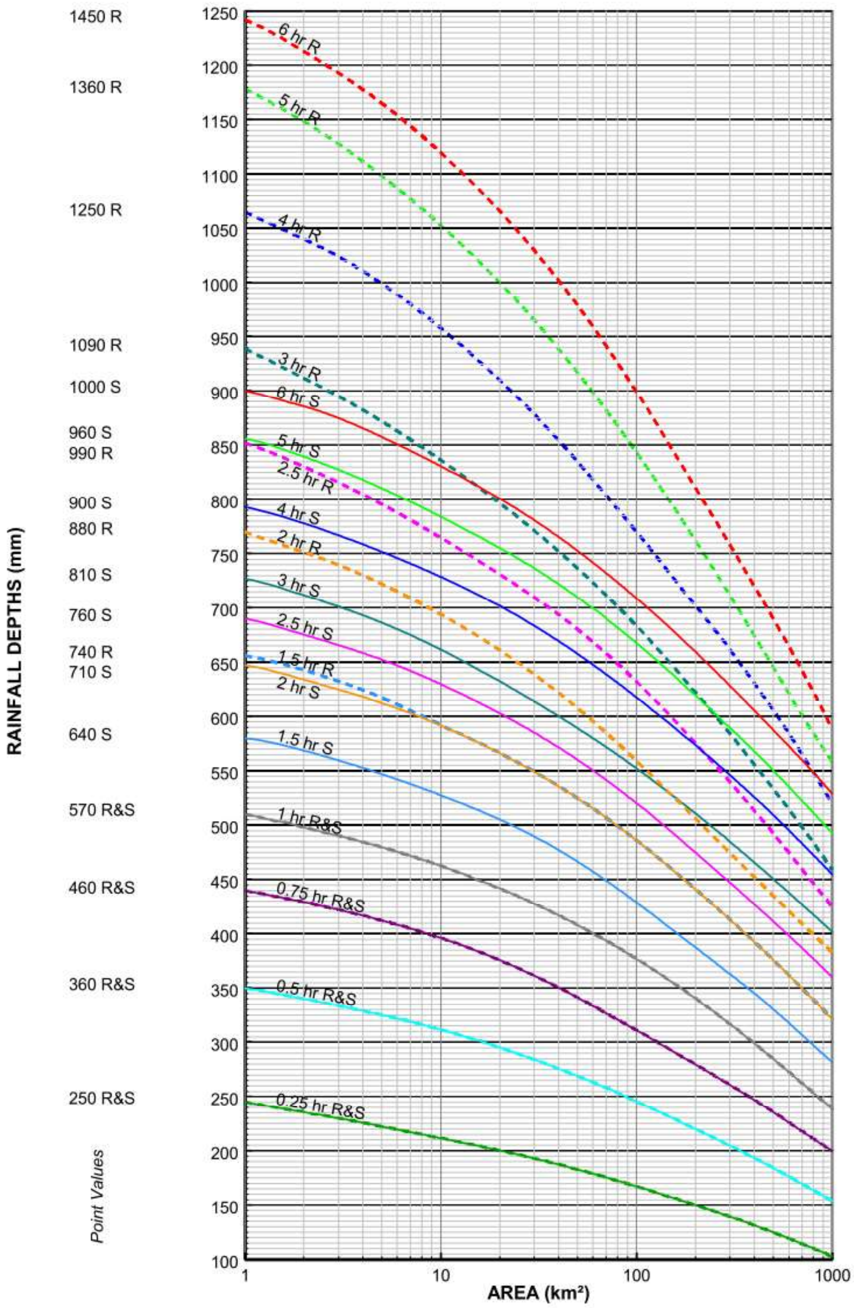


Figure 4: Depth-Duration-Area Curves of Short Duration Rainfall

10

THE ESTIMATION OF PROBABLE MAXIMUM PRECIPITATION IN AUSTRALIA: GENERALISED SHORT-DURATION METHOD
JUNE 2003

Figure 58. *Depth-Duration-Area curves of short duration rainfall for PMP depths*

Model setup

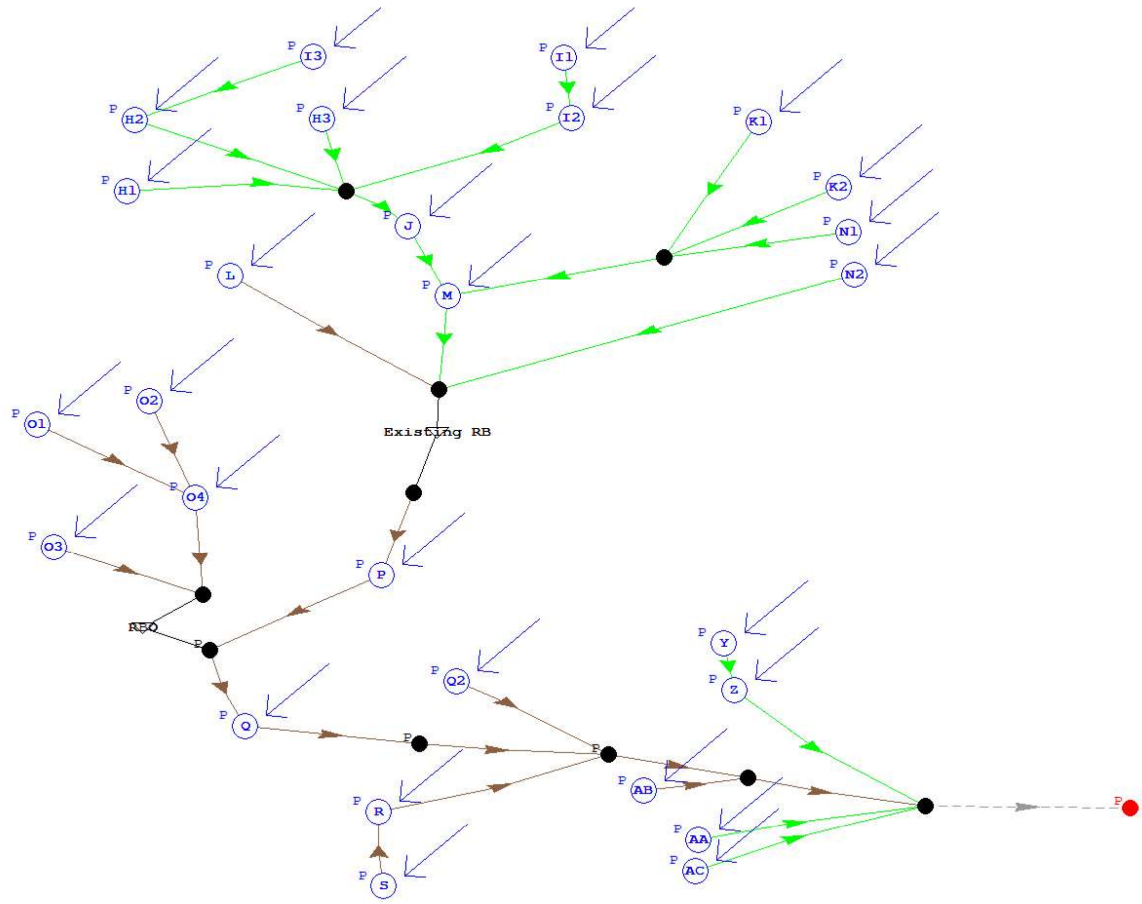


Figure 59. *East catchment RORB model (developed conditions)*

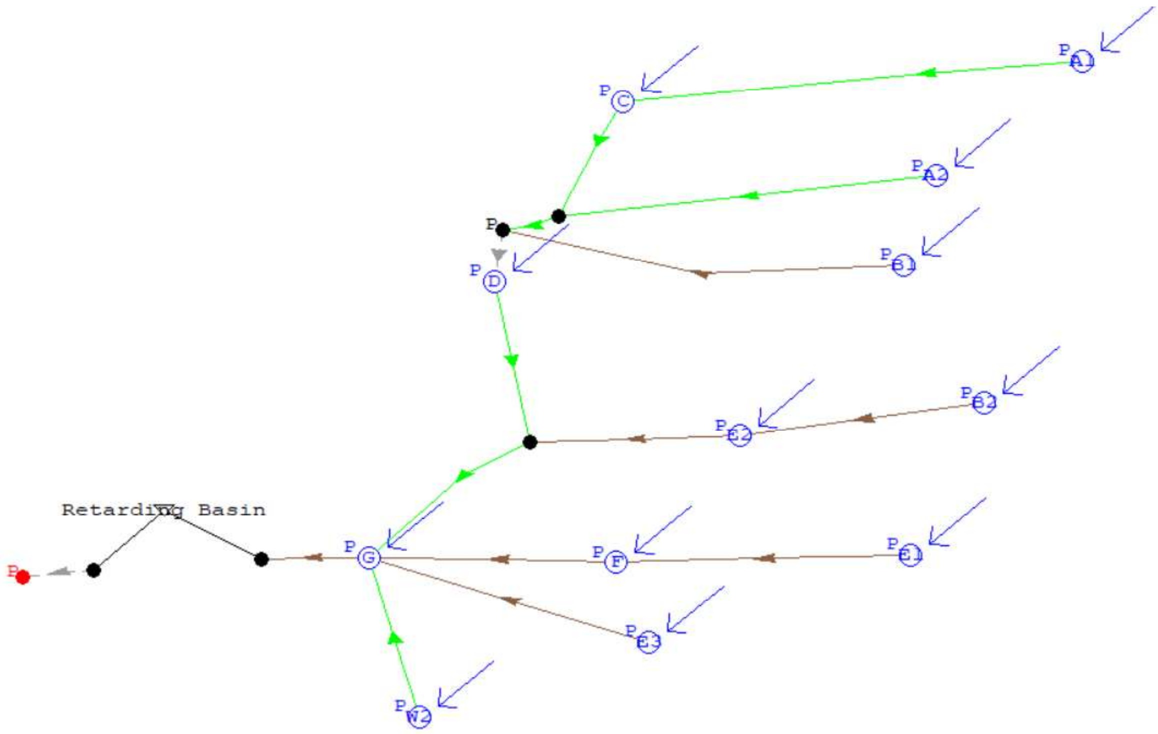


Figure 60. *North west catchment RORB model (developed conditions)*

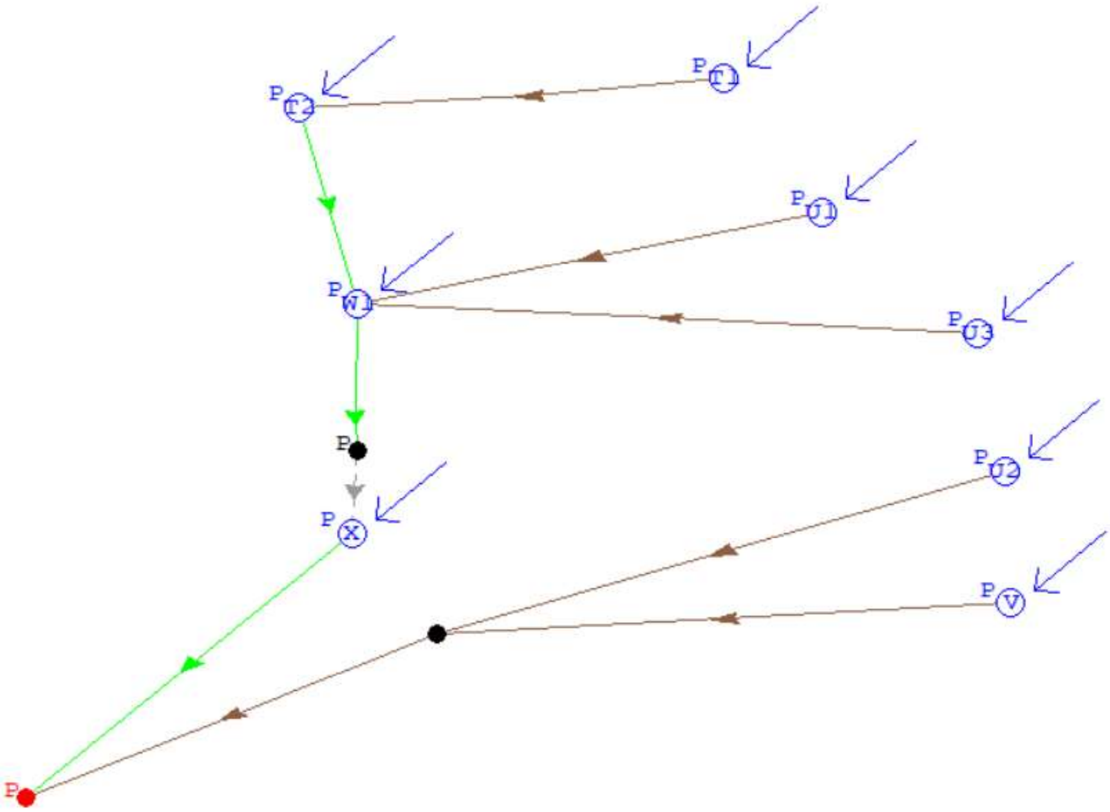


Figure 61. *South west catchment RORB model (developed conditions)*

Appendix B
Treatment modelling

Modelling inputs

The MUSIC (Model for Urban Stormwater Improvement Conceptualisation) model that was developed for each of the scenarios included the following input parameters:

- A historic rainfall dataset (1968- 2020) was obtained from BoM for the Stratford rainfall gauge (085078). The average annual rainfall over this entire period was obtained from the Bureau of Meteorology (BoM) and used to select a ten-year period from the historic dataset which produced a similar annual average rainfall. The average annual rainfall from BoM is 654.6 mm. The period from 1982 -1991 was adopted which has an annual average rainfall of 667.5mm.
- The monthly average evaporation for Sale was also obtained from BoM.
- MUSIC model run at a 6-minute timestep.
- Fraction impervious values and areas for sub catchments consistent with Table 2 and Table 3.
- Wetlands designed to not exceed 72.0 hours detention time, to prevent terrestrial and aquatic vegetation from 'drowning'.

Figure 62 outlines the iterative process of sizing the treatment infrastructure in MUSIC.

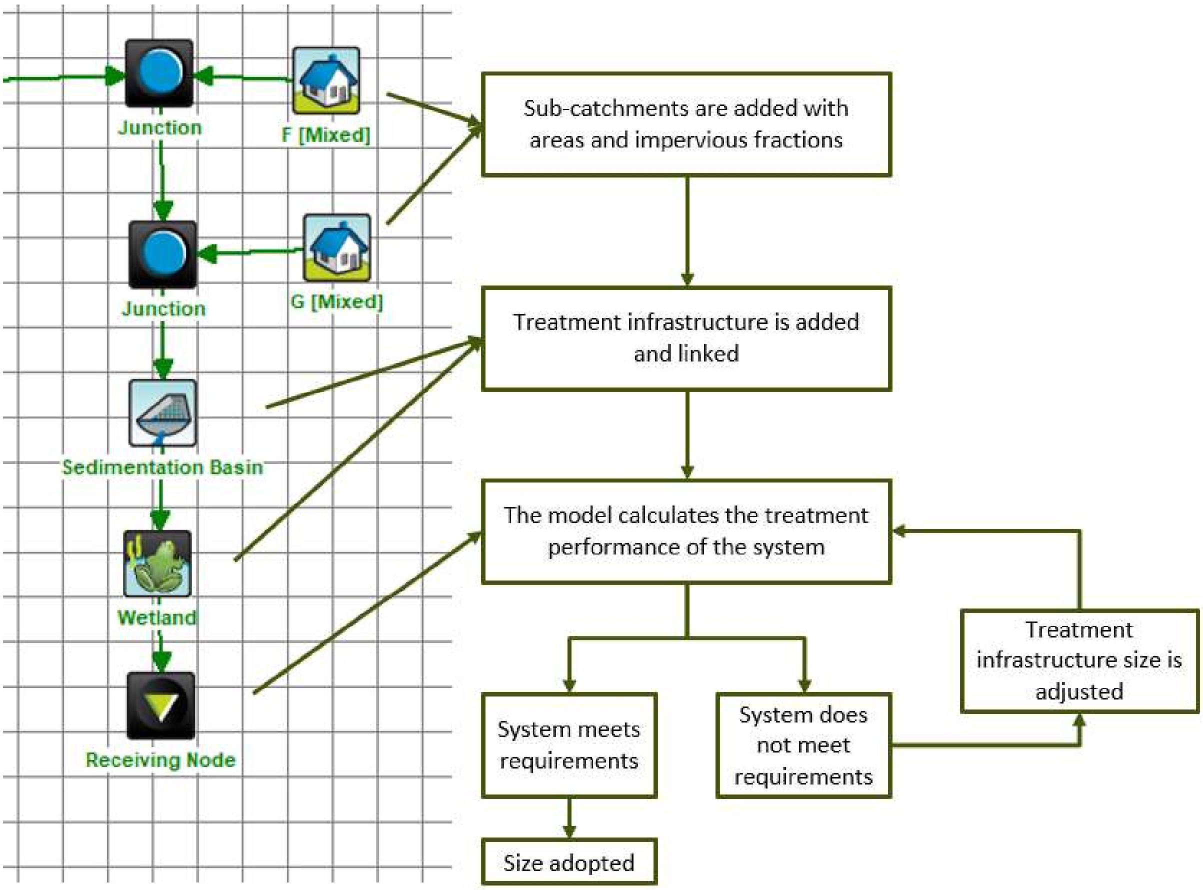


Figure 62. *Simplified MUSIC Method*

Sediment Basin sizing

The sediment basins in the treatment modelling have been sized using the Fair and Geyer equation, where sediment basins are required to meet the following criteria:

- Capture 95% of coarse particles ≥ 125 µm diameter for the peak three-month ARI event

The sediment basin sizing was used for the inlet pond in the wetland node (assuming an average depth of 0.8m).

Table 31. Sediment basin sizing for Maffra retarding basin WL

	Parameter	**Proposed design**
Conditions	Contributing Catchment (ha)	284.7
	Area of Basin (m^2)	3,500
Capture Efficiency	Settling Velocity of Target Sediment (mm/s) [Particle size 125 µm]	11
	Hydraulic Efficiency (λ)	0.11
	Permanent Pool Depth, dp (m)	0.50
	Extended detention depth, de	0.35
	Number of CTSR's, n	1.12
	Depth below permanent pool that is sufficient to retain sediment, d* (m)	0.50
	Design Discharge (m^3/s) [Q3-month]	1.19
	Capture Efficiency	98.7%
	Check (>95%)	OK
Sediment Storage	Sediment Loading rate, Lo (m^3/ha/yr)	2.0
	Desired clean-out frequency, Fr	5
	Storage volume required, St	2,808
	Available sediment storage volume	2,927
	Check (Available storage > required storage)	OK
Sediment dewatering	Depth for dewatering area (m)	0.5
	Area required for dewatering (m^2)	5,616

Table 32. Sediment basin sizing for Powerscourt retarding basin WL

	Parameter	Proposed design
Conditions	Contributing Catchment (ha)	20.17
	Area of Basin (m^2)	800
Capture Efficiency	Settling Velocity of Target Sediment (mm/s) [Particle size 125 µm]	11
	Hydraulic Efficiency (λ)	0.11
	Permanent Pool Depth, dp (m)	0.50
	Extended detention depth, de	0.35
	Number of CTSR's, n	1.12
	Depth below permanent pool that is sufficient to retain sediment, d* (m)	0.50
	Design Discharge (m^3/s) [Q3-month]	0.28
	Capture Efficiency	98.6%
	Check (>95%)	OK
Sediment Storage	Sediment Loading rate, Lo (m^3/ha/yr)	2.0
	Desired clean-out frequency, Fr	5
	Storage volume required, St	196
	Available sediment storage volume	547
	Check (Available storage > required storage)	OK
Sediment dewatering	Depth for dewatering area (m)	0.5
	Area required for dewatering (m^2)	392

Table 33. Sediment basin sizing for the north west retarding basin / wetland

	Parameter	Proposed design
Conditions	Contributing Catchment (ha)	134.7
	Area of Basin (m^2)	1,800
Capture Efficiency	Settling Velocity of Target Sediment (mm/s) [Particle size 125 µm]	11
	Hydraulic Efficiency (λ)	0.16
	Permanent Pool Depth, dp (m)	0.50
	Extended detention depth, de	0.35
	Number of CTSR's, n	1.12
	Depth below permanent pool that is sufficient to retain sediment, d* (m)	0.50
	Design Discharge (m^3/s) [Q3-month]	0.79
	Capture Efficiency	97.1%
	Check (>95%)	OK
Sediment Storage	Sediment Loading rate, Lo (m^3/ha/yr)	2.0
	Desired clean-out frequency, Fr	5
	Storage volume required, St	1,316
	Available sediment storage volume	1,372
	Check (Available storage > required storage)	OK
Sediment dewatering	Depth for dewatering area (m)	0.50
	Area required for dewatering (m^2)	2,632

Table 34. Sediment basin sizing for the south west retarding basin / wetland

	Parameter	Proposed design
Conditions	Contributing Catchment (ha)	126.9
	Area of Basin (m^2)	1,700
Capture Efficiency	Settling Velocity of Target Sediment (mm/s) [Particle size 125 µm]	11
	Hydraulic Efficiency (λ)	0.16
	Permanent Pool Depth, dp (m)	0.50
	Extended detention depth, de	0.35
	Number of CTSR's, n	1.12
	Depth below permanent pool that is sufficient to retain sediment, d* (m)	0.50
	Design Discharge (m^3/s) [Q3-month]	0.90
	Capture Efficiency	97.8%
	Check (>95%)	OK
Sediment Storage	Sediment Loading rate, Lo (m^3/ha/yr)	2.0
	Desired clean-out frequency, Fr	5
	Storage volume required, St	1,231
	Available sediment storage volume	1,296
	Check (Available storage > required storage)	OK
Sediment dewatering	Depth for dewatering area (m)	0.5
	Area required for dewatering (m^2)	2,462

Appendix C
Wetland sections

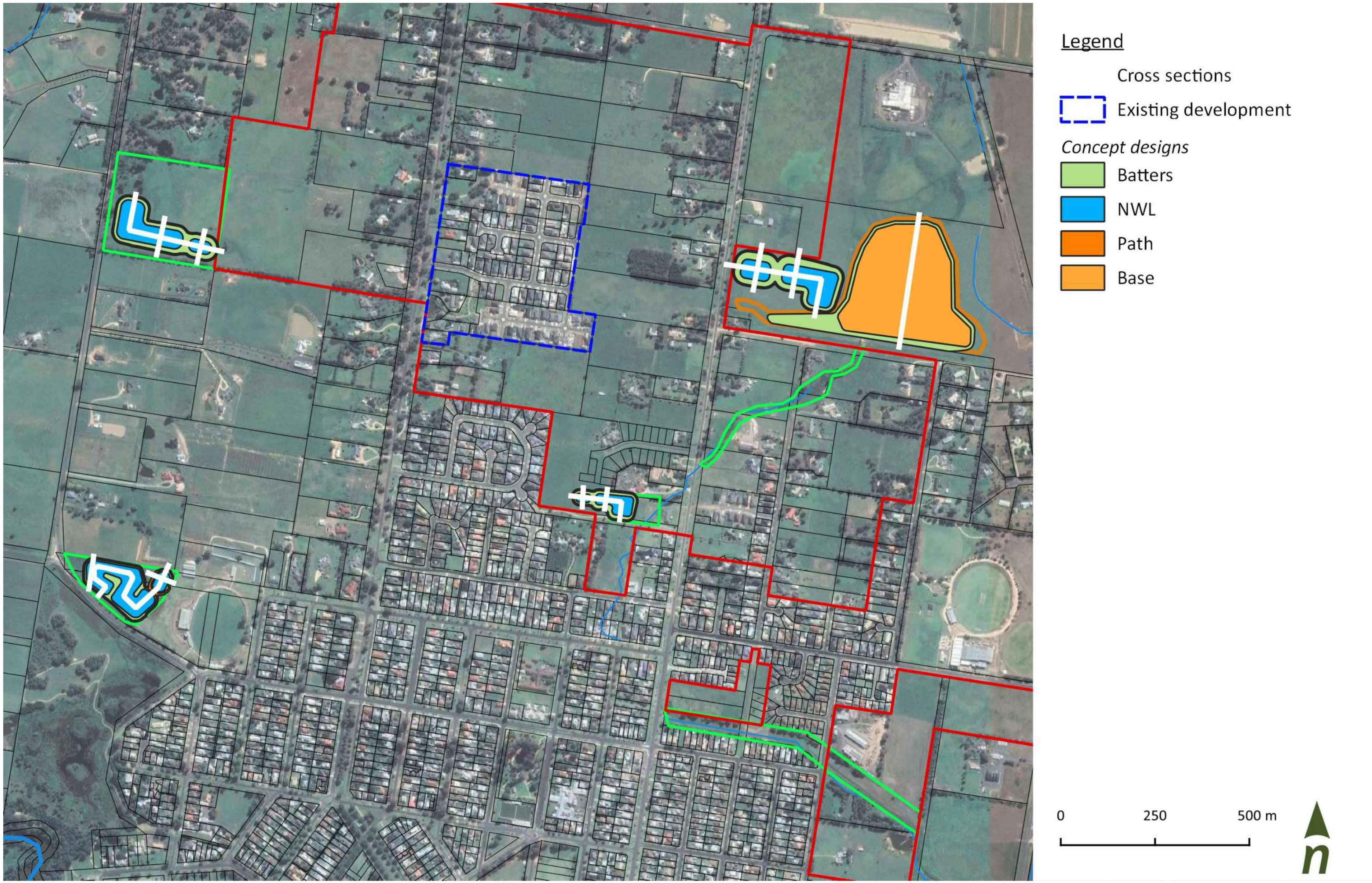


Figure 63. *Cross section plan view*

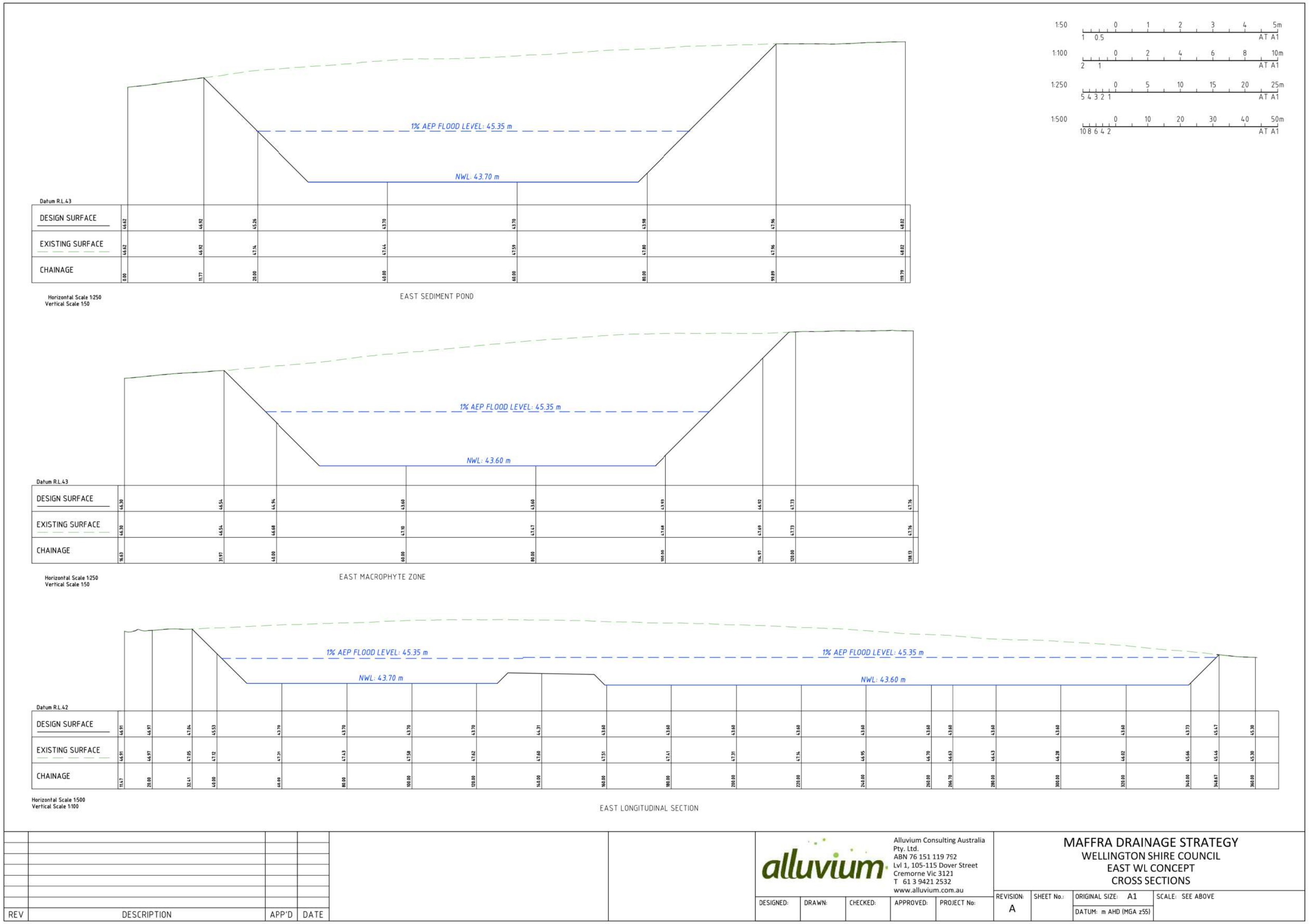


Figure 64. *Maffra Wetland cross sections*

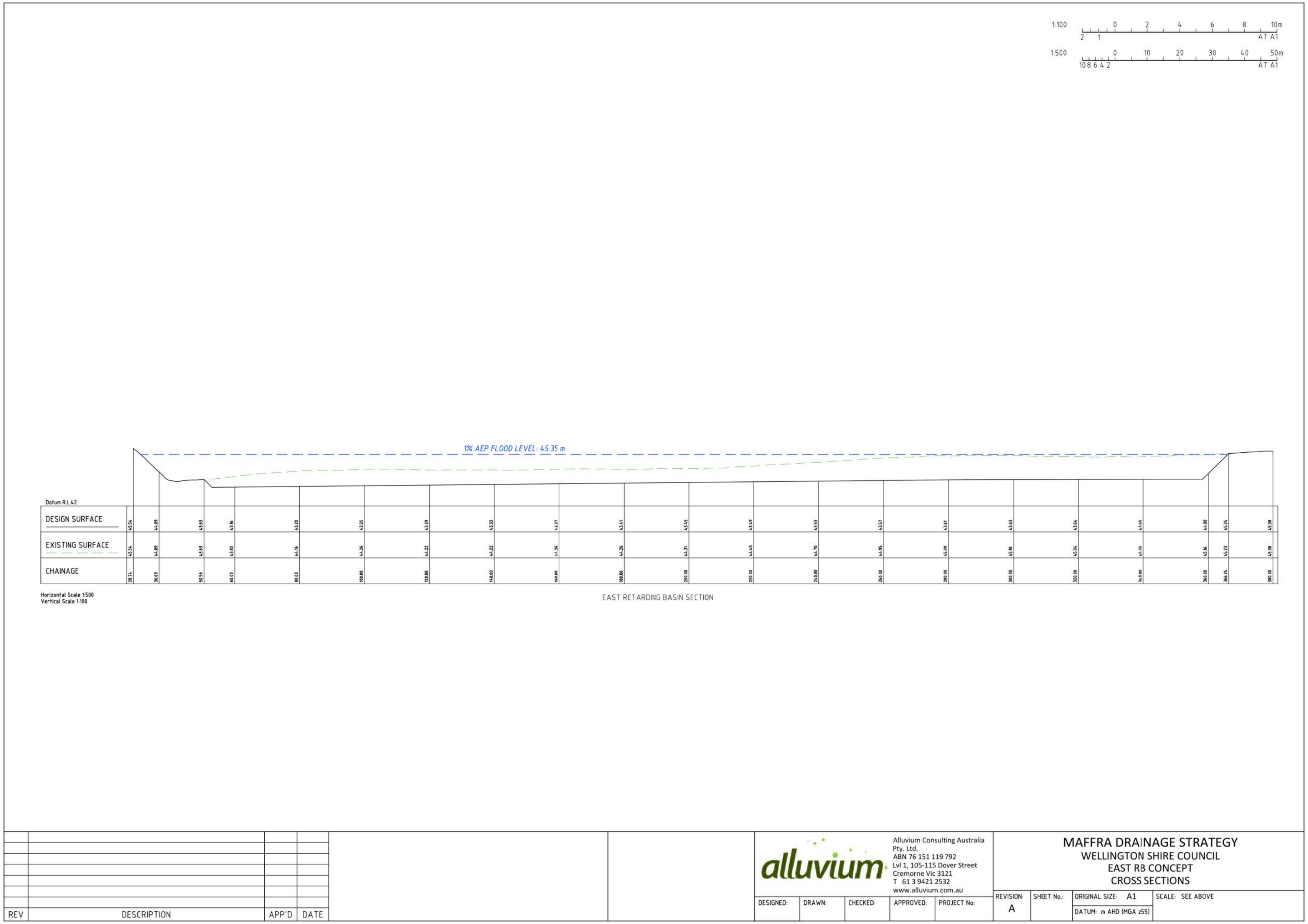


Figure 65. *Maffra Retarding Basin cross sections*

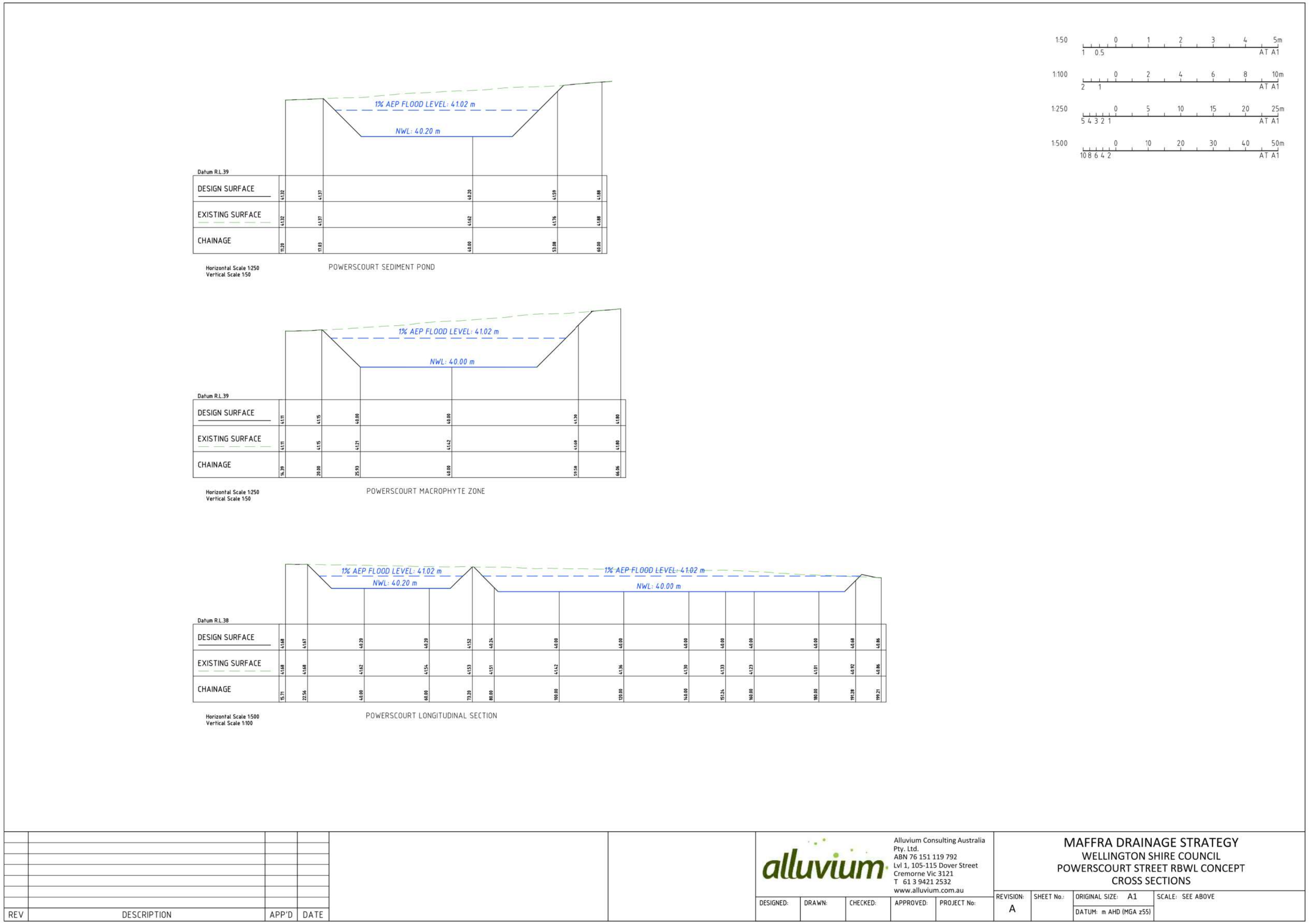


Figure 66. *Powerscourt Retarding Basin cross sections*

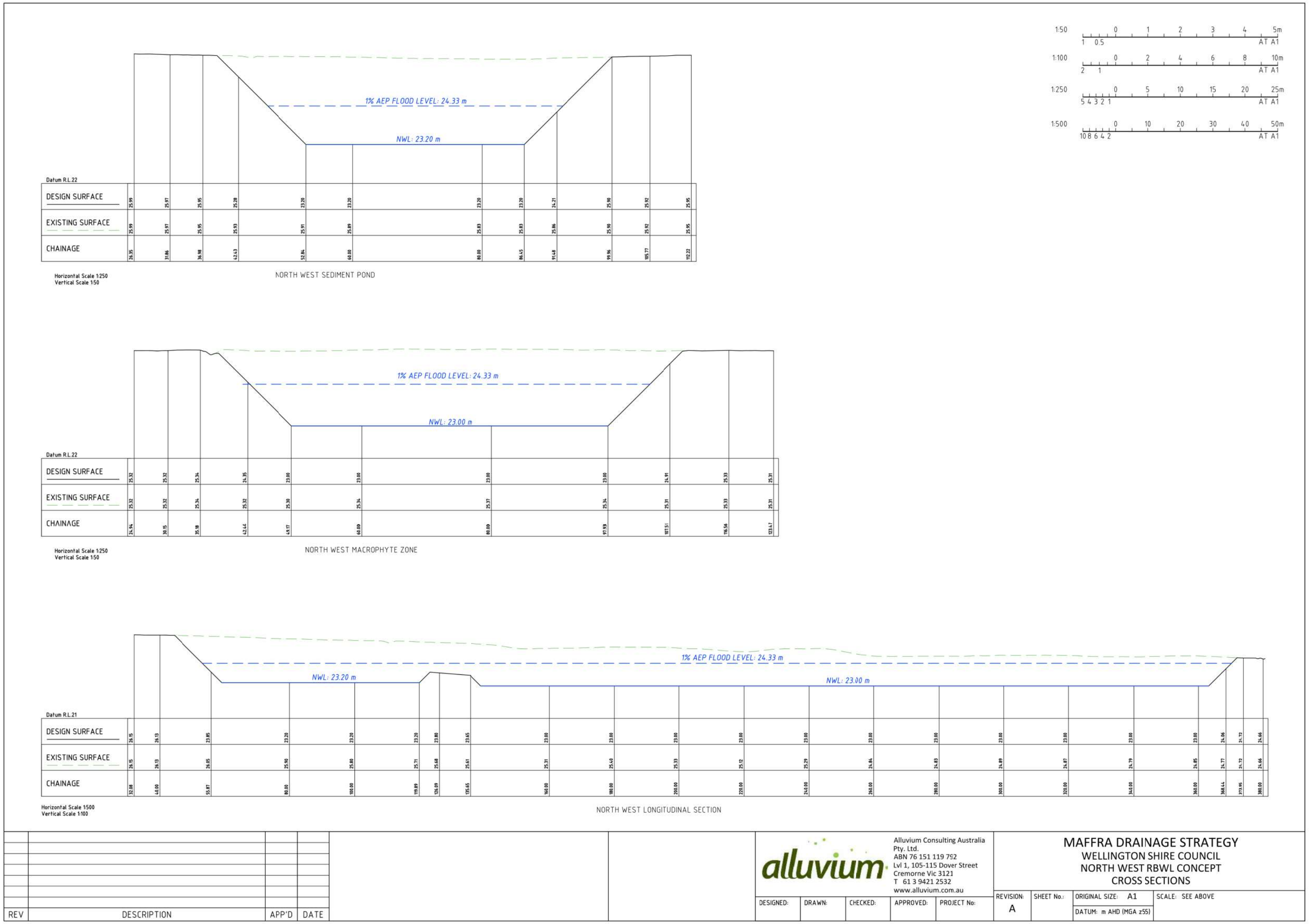


Figure 67. *North west Wetland / Retarding Basin cross sections*

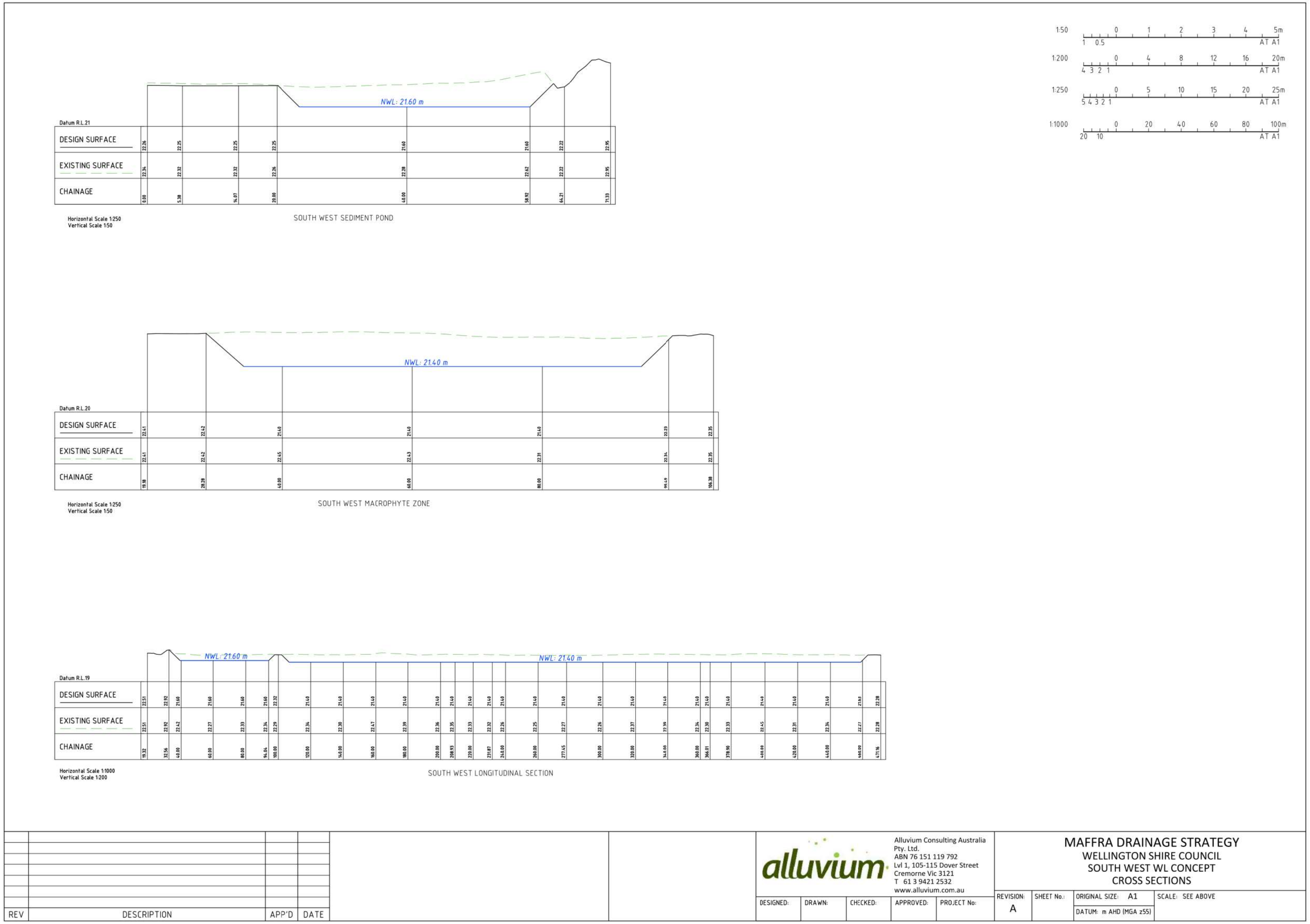


Figure 68. *South West Wetland*

Appendix D
WSUD life cycle costing

Water sensitive urban design
Life cycle costing data

Melbourne Water has recently developed a life cycle costing data table to assist councils in estimating costs associated with stormwater treatment asset planning during the design, construction, establishment, maintenance and renewal phases. The data will inform council budgets and ensure allowances for stormwater treatment assets are based on whole of life cycle costs.

The life cycle cost information is grouped according to asset type, size, service level (maintenance frequency) and, where possible, contracted rates versus in-house works. Other factors including traffic management and access issues are also considered.

A summary of the life cycle costs for asset construction, maintenance (establishment and ongoing) and renewal is provided overleaf.

BENEFITS OF WATER SENSITIVE URBAN DESIGN

Water Sensitive Urban Design aims to integrate the urban water cycle into urban design. The social and environmental benefits of stormwater treatment systems are widely recognised and include:

- improved urban waterways
- greener open spaces and enhanced urban landscapes
- reduced localised flooding
- improved amenity in our local communities
- alternative water supply option.

HOW COUNCILS CAN USE THE DATA

The life cycle costing data can be used by councils to refine stormwater treatment asset management planning. In particular, the life cycle costs will enable councils to better plan for maintenance of stormwater treatment assets and refine budgets for life cycle costs of individual stormwater treatment assets. This includes informing and assisting councils to better forecast budgets for the management of stormwater treatment assets.

The incorporation of realistic maintenance costs into council budgets will help ensure that stormwater treatment assets are adequately maintained; and therefore help reduce the financial burden to councils associated with rectifying assets that are failing due to inadequate maintenance.

It is expected that the maintenance cost estimates provided will assist councils to get better value for money when negotiating maintenance contracts.

For more information on inspection and maintenance schedules and sample maintenance contract documentation please refer to the Melbourne Water WSUD Maintenance Guidelines on our website **melbournewater.com.au**

For access to the full Life Cycle Costing Report, please contact the Melbourne Water Stormwater Team at livingrivers@melbournewater.com.au

healthy waterways

State Government Victoria | Melbourne Water

ASSET	ASSET PARAMETERS	CONSTRUCTION[1]	MAINTENANCE		RENEWAL
			ESTABLISHMENT (FIRST TWO YEARS)	ONGOING	
WETLANDS[2]	< 500 m^2 500 to 10,000 m^2 > 10,000 m^2	\$150/m^2 \$100/m^2 \$75/m^2		\$10/m^2/yr \$2/m^2/yr \$0.5/m^2/yr	No data
SEDIMENT BASINS[2]	< 250 m^2 250 to 1000 m^2 > 1000 m^2	\$250/m^2 \$200/m^2 \$150/m^2		\$20/m^2/yr \$10/m^2/yr \$5/m^2/yr	Remove and dispose of: Dry waste = \$250/m^3 Liquid waste = \$1,300/m^3
ON-STREET RAINGARDENS[3]	< 50 m^2 50 to 250 m^2 > 250 m^2	\$2000/m^2 \$1000/m^2 \$500/m^2		\$30/m^2/yr \$15/m^2/yr \$10/m^2/yr	Minor reset = \$50 to \$100/m^2
BIORETENTION BASINS[3]	< 100 m^2 100 to 500 m^2 > 500 m^2	\$1000/m^2 \$350/m^2 \$250/m^2		\$5/m^2/yr	No data
TREE PITS[3]	< 10 m^2 total 10 to 50 m^2 total > 50 m^2 total	\$8000/m^2 \$5000/m^2 \$1000/m^2	Two to five times ongoing maintenance cost	No access issues = \$150/asset/yr Traffic issues or specialist equipment required = \$500/asset/yr	No data
GRASS SWALES AND BUFFER STRIPS[4]	Seeded – no subsoil drain Seeded – subsoil drain Turfed – no subsoil drain Turfed – subsoil drain Native grasses established	\$15/m^2 \$25/m^2 \$20/m^2 \$35/m^2 \$60/m^2		\$3/m^2/yr	No data
VEGETATED SWALES AND BIORETENTION SWALES[4]		150/m^2		\$5/m^2/yr	No data
IN-GROUND GPTS	< 300 L/s 300 to 2000 L/s > 2000 L/s	\$50,000/asset \$150,000/asset \$250,000/asset	N/A	Inspection = \$100/visit Cleanout = \$1000/visit	No data

1 Includes planning and design
2 Area at normal water level
3 Area of filter media at bottom of extended detention
4 Total vegetated area

Disclaimer: The cost estimates provided should be considered as a starting point only and represent the best cost estimates available based on current information (Oct 2013). The cost estimates will be reviewed and refined over time as better data becomes available. It should be noted that data are generally based on 'standard residential' developments and the cost of equipment hire is not included in the estimates.

Appendix E
Flood modelling report

MAFFRA DRAINAGE AND INTEGRATED WATER MANAGEMENT STRATEGY HYDRAULICS REPORT

AUGUST 2020

PREPARED FOR

Alluvium for Wellington Shire Council

Project Details	
Report Title	Maffra Drainage and Integrated Water Management Strategy Hydraulics Report
Prepared for	Alluvium for Wellington Shire Council
Document Name	30009-R01-MaffraDrainageIWM-B

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D	

Image Source: Smith, K (2020), Inception Meeting Site Visit Photograph, 6/3/2020

EXECUTIVE SUMMARY

Wellington Shire Council have engaged Water Modelling Solutions in conjunction with Alluvium to undertake a Drainage and Integrated Water Management Strategy (D&IWMS) for the township of Maffra in Eastern Victoria. The flood modelling component of the project involves investigation and mapping of existing conditions for the 20% and 1% Annual Exceedance Probability (AEP) events as well as support for investigation of mitigation options for the township flooding under 20%, 1%, Probable Maximum Flood (PMF) and Climate Change events. The outcome of the Maffra D&IWMS will be the development of sufficient flood information such that Council can undertake effective floodplain management and the information can be used by a variety of stakeholders for land use planning, flood management planning, treatment and mitigation. This Hydraulic Report is an addendum to the main Maffra Drainage and Integrated Water Management Strategy and details the hydraulic modelling portion of the study.

Historically, Maffra township has experienced periodic flash flooding via an ephemeral stream, over a long period of time. Whilst some engineered mitigation solutions have been previously built, specifically a levee and retarding basin, these solutions have been undersized and are not sufficient measures to reduce the flooding throughout the township.

In addition, any prior studies that have previously been undertaken for the township have been completed under the now superseded ARR1987 guidelines.

The flash flooding within the township arises from intense rainfall events within the catchment to the north of the Township, where the existing George Street drain is old and under-capacity. In addition, there is a significant detention basin located at the northern end of the ephemeral stream. The Macalister River runs across the south west corner of the main township and the town is bisected by Powerscourt Street.

Hydraulic modelling has been undertaken for the Maffra Township utilising rainfall-excess hydrology supplied by Alluvium. The modelling utilised the industry standard software, TUFLOW with a 1-dimensional drainage network connected to a 2-dimensional terrain.

A range of events were modelled for both the existing and developed scenarios including sensitivity scenarios for PMF and Climate Change for 2100 RCP4.5 and RCP8.5 for the developed scenario. Three indicative temporal patterns – front, mid and rear loaded, were chosen to represent the ensemble modelling as recommended in ARR2019.

Flood flow behaviour under existing conditions shows that flow travels from the north east at the location of the detention basin along the ephemeral stream, splitting at Merry Street with some flow travelling west along Merry Street and the remainder travelling south along Alfred Street. Significant ponding of flow occurs at the Alfred Street / Merry Street junction and along Alfred Street between Mclean Street and George Street. The township is also experiencing shallow sheet overland flow broadly across residential areas due to local catchment flash flooding, to an approximate depth of 100mm.

Flood flow behaviour under proposed conditions shows that flood levels within the proposed constructed channel is typically lower due to upsizing the detention basin. However, there are some areas of afflux due to the proposed design.

The flow behaviour is similar in the 20% AEP event with lower flood depths and a lesser flood extent observed.

The flow behaviour and afflux is discussed in detail in Section 4. Typically, the afflux observed under design conditions is due to culverts acting as a hydraulic control where they are not proposed to be ungraded in line with the upgrades to the surrounding channel. It is recommended that culvert upgrades be considered at the next stage of design.

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LIST OF ABBREVIATIONS

AEP	Annual Exceedance Probability
D&IWMS	Drainage and Integrated Water Management Strategy
PMF	Probably Maximum Flood
PMP	Probable Maximum Precipitation
WGCMA	West Gippsland Catchment Management Authority
WSC	Wellington Shire Council
WMS	Water Modelling Solutions

1 INTRODUCTION

1.1 BACKGROUND

Historically, Maffra township has experienced periodic flash flooding via an ephemeral stream, over a long period of time. Whilst some engineered mitigation solutions have been previously built, specifically a levee and retarding basin, these solutions have been undersized and are not sufficient measures to reduce the flooding throughout the township.

In addition, any prior studies that have previously been undertaken for the township have been completed under the now superseded ARR1987 guidelines.

Given the above, in conjunction with planned new residential growth, the Wellington Shire Council commissioned a Drainage and Integrated Water Management Strategy that utilises the latest ARR2019 guidelines.

1.2 PROJECT SCOPE

The Integrated Water Management Plan incorporates the following key items:

- Investigation and mapping of existing conditions for the 20% and 1% AEP events;
- Support for a strategic planning assessment for new residential growth areas;
- Investigation and proposal of mitigation options for the township flooding;
- Investigation of Integrated Water Management solutions; and
- Incorporation of passive open space elements to provide for a high level of amenity.

The outcome of the D&IWMS is the development of sufficient flood information that Council can undertake effective floodplain management and the information can be used by a variety of stakeholders for land use planning, flood management planning, treatment and mitigation.

This report is an addendum to the main Maffra Drainage and Integrated Water Management Strategy, and details the hydraulic modelling portion of the study.

1.3 STUDY AREA

According to the Australian Bureau of Statistics 2016 Census data, the population of Maffra is 4,316 people (Commonwealth of Australia, 2019) and according to WSC, there is future residential growth proposed, especially to the north of the existing township.

The Macalister River runs across the south west corner of the main township and the town is bisected by Powerscourt Street. There is an ephemeral stream that rises to the north of the town at Fosters Hill.

Flash flooding within the township arises from intense rainfall events within the catchment to the north of the Township, where the existing George Street drain is old and under-capacity. The study area is illustrated in Figure 1-1 and the flow path through the ephemeral stream is illustrated in Figure 1-2. In addition, there is a significant detention basin located at the northern end of the ephemeral stream, this is also illustrated in Figure 1-2.

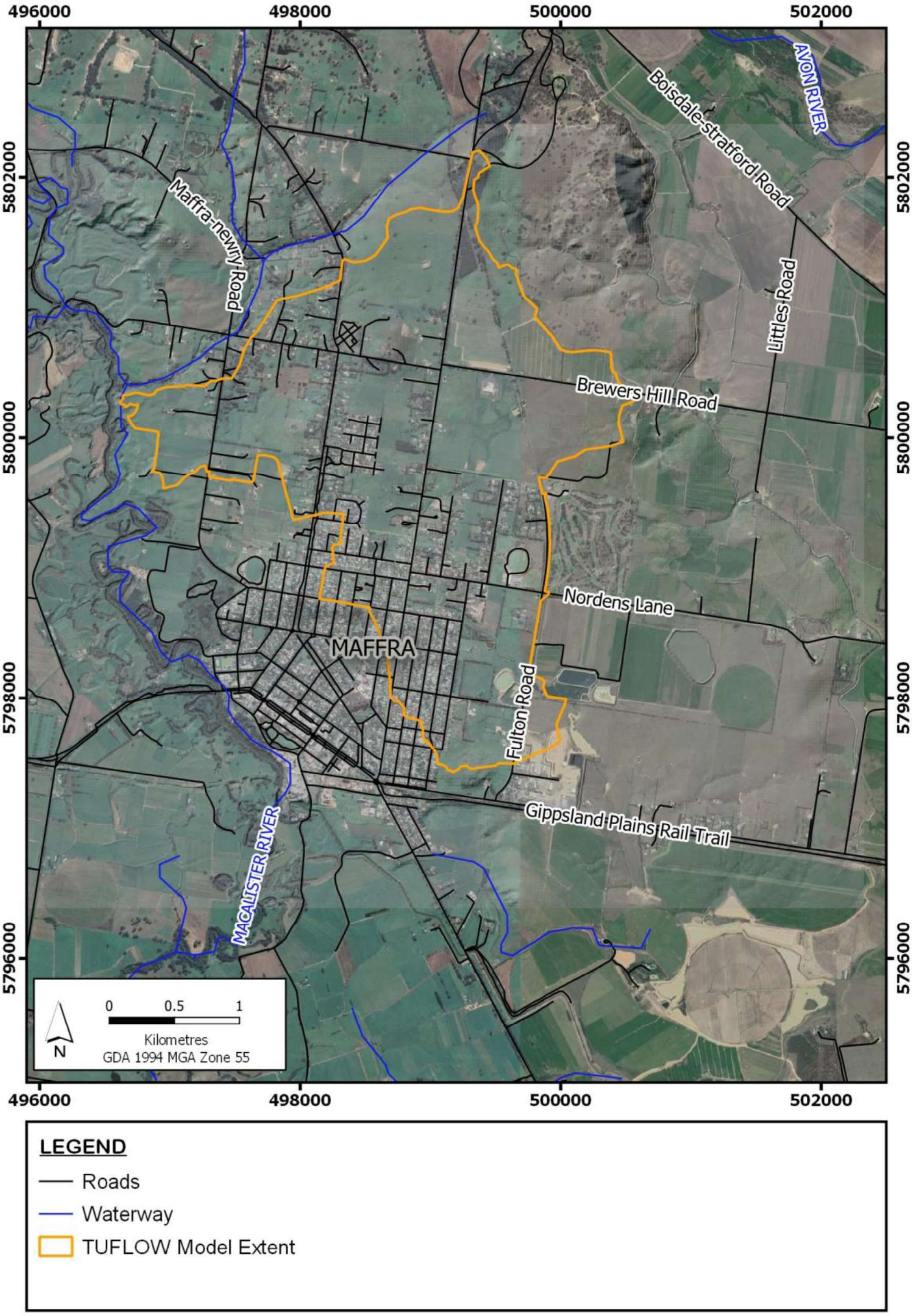


Figure 1-1 Study Area

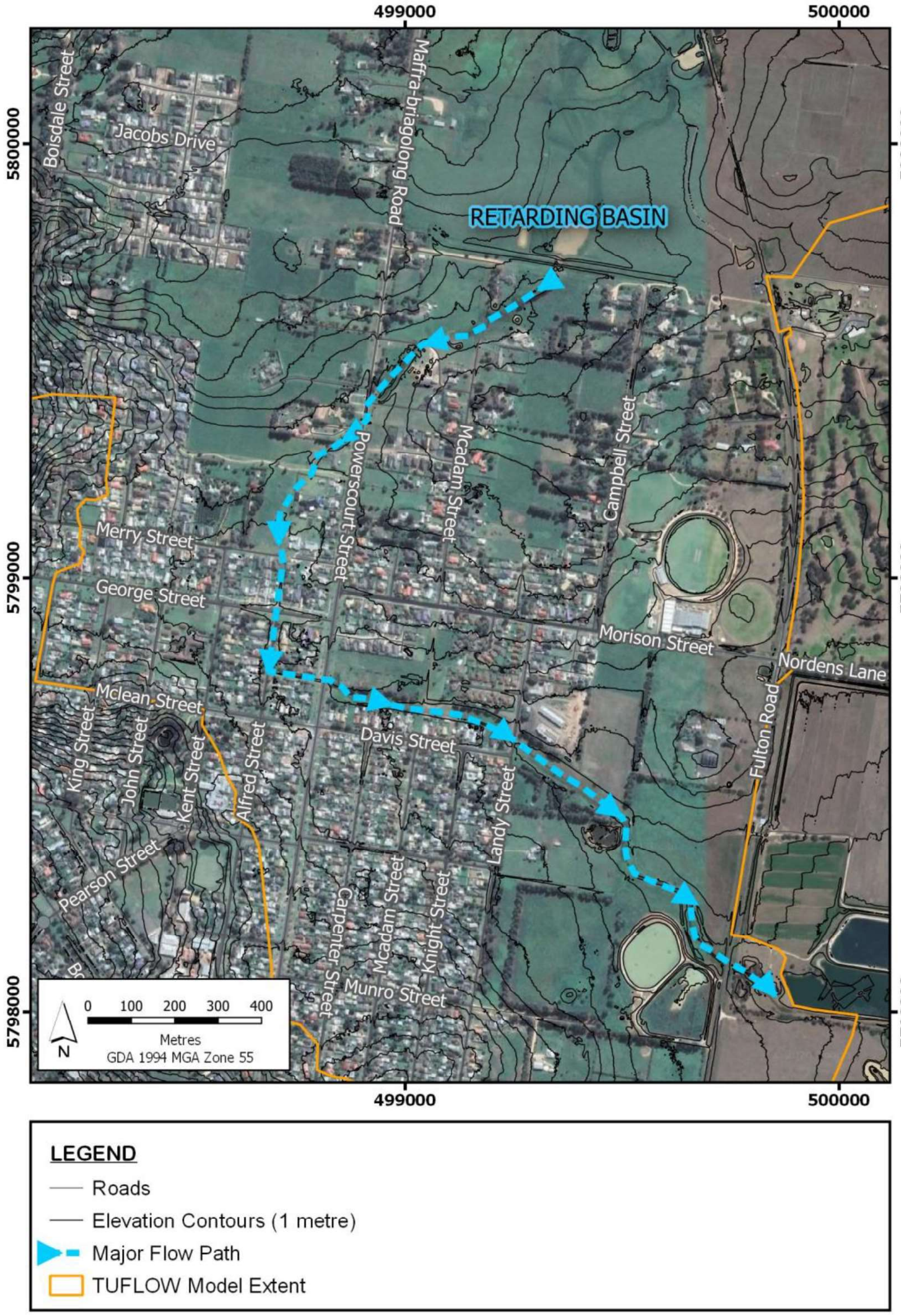


Figure 1-2 Retarding Basin and Major Flow Path Location

2 AVAILABLE DATA

The following section details the data sources used in the development of the hydraulic modelling.

2.1 LIDAR

LiDAR data was provided by West Gippsland Catchment Management Authority (WGCMA) and has the following features:

- Captured as part of the Southern Rural Water Macalister River Irrigation District project between July 28th to August 3rd 2008
- GDA 1994 MGA Zone 55
- Provided in 1m gridded DEM format
- Stated accuracy is ± 0.1 m vertically and ± 0.25 m horizontally

2.2 STORMWATER NETWORK

The stormwater network was provided in GIS format as two layers – the first was a polyline layer representing the pipes, and the second was a points layer representing pits, outlets and manholes.

2.3 STRUCTURE DETAILS

The details of several standalone culverts were provided by WSC, including plans with dimensions and invert levels for major culverts and dimensions for culverts where no plans were provided. Culvert data without plans are included in Appendix B.

2.4 FLOOD OBSERVATIONS

Observations from the 1988 and 1993 flood events were provided as photographs annotated with location notes.

2.5 SITE VISIT

A site visit was undertaken by Water Modelling Solutions and Alluvium on Friday 6th March 2020. The purpose of the site visit was primarily to obtain a high-level overview of the catchment and key features such as the ephemeral waterway, wetlands, the detention basin, proposed basin locations and structures such as culverts and bridges. No structure measurements were undertaken during this site visit.

2.6 GENERAL GIS DATA

The following spatial data layers were obtained from VicSpatial

- Cadastral / Lot boundaries
- Waterways
- Land Use / Planning Scheme Zones
- Road Centrelines

3 HYDRAULIC MODELLING

The hydraulic model was undertaken as a 1-dimensional / 2-dimensional combined hydraulic model in the industry standard software TUFLOW. TUFLOW is a numerical model used to simulate the hydrodynamic behaviour of rivers, floodplains and urban drainage environments (BMT Group Ltd, 2007 – 2018). The software is ideal for large scale catchment studies such as the Maffra D&IWMS, as it is equally capable of modelling riverine and floodplain environments as well as the urban drainage environment, such as the overland flow and stormwater flooding, including pits, pipes and culverts.

Furthermore, the latest TUFLOW HPC (Heavily Parallelised Compute) model was used, which delivers a 10-100 times simulation speed increase compared to the standard CPU version (BMT Group Ltd, 2007 – 2018). A feature of the latest release is Sub-Grid-Sampling (SGS), which allows topographic features on a smaller scale than the model cell size to be represented in the calculations and results. This is useful in urban environments where hydraulic behaviour of small features such as roadside drains can be accurately represented using a larger cell size than has traditionally been used.

The model used the 'rainfall excess approach' where rainfall-excess hydrographs are applied directly to the model terrain. This is a similar approach to direct-rainfall, with the difference being that flows (in m^3/s) are applied, as opposed to rainfall hyetographs (in mm/hr). These flows are spread evenly over the sub-catchment area in the model.

The methodology is illustrated in Figure 3-1.

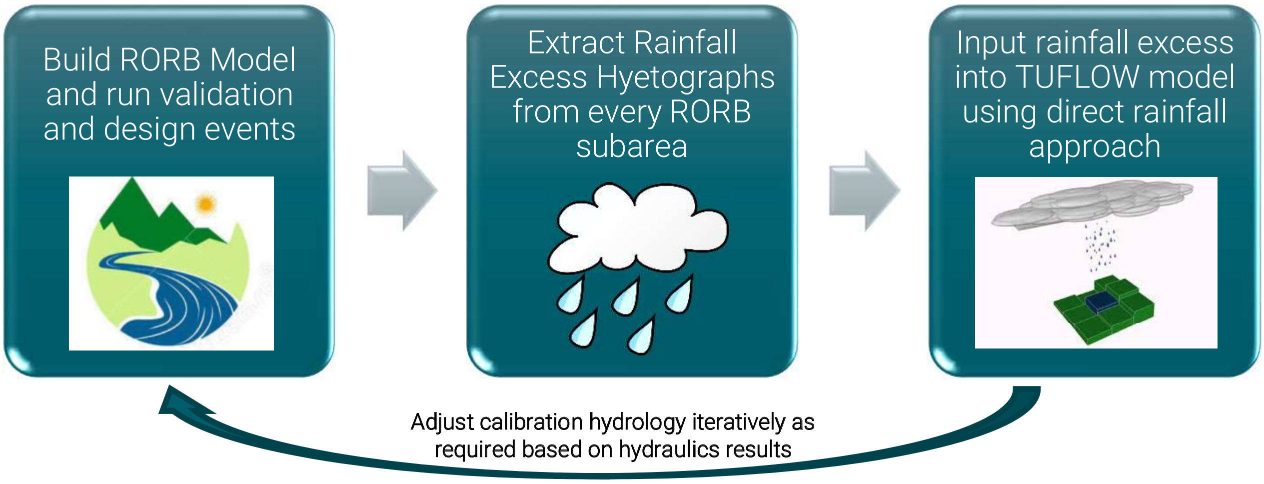


Figure 3-1 Rainfall Excess Approach

3.1 KEY PARAMETERS

Key TUFLOW model parameters are listed in Table 3-1.

Table 3-1 Key Model Parameters

Parameter	Value	Derivation/Reason for Use
Model Version	2020-01-AB	Latest version at time of project
Guidelines	ARR2019, others as referenced	Latest version at time of project
Solution Scheme	HPC (2^{nd} order – default)	Run-time efficiency
Timestep	variable	Artefact of using HPC
Sub-Grid Sampling	1 metre sampling to match LiDAR resolution	Better representation of sub-grid scale flow paths
Cell Size	3 metres	Good model topography representation without exorbitant run time

Parameter	Value	Derivation/Reason for Use
Projection	GDA94 Zone 55	Relevant location for Maffra
Inflows	RORB Excess input via 2d_sa polygons	Discussed in detail in Section 3.2.6
Downstream Boundary Conditions	Based on Terrain Slope	
1d-2d connections	SX / CN lines and 1d nodes	Standard TUFLOW practice
Manning's Roughness Values	Discussed in detail in Section 3.2.4	
Pits, pipes and structures	Discussed in detail in Section 3.2.2and 3.2.3	
Model Health and Log File Output	Model health checks show that minimum dT does not drop significantly below 0.6 for the majority of the model or display multiple significant jumps. This is considered to be acceptable for a model with 3m grid size. The timestep for a 3m grid would typically be 0.75s – 1.5s, therefore a drop to 0.6s using adaptive timestepping is within acceptable range.	Graph provided in Appendix C

3.2 EXISTING CASE

The setup of the existing case scenario is described over the following sections. The existing case model setup is shown in Figure 3-2.

3.2.1 Topography

Topography was based on LiDAR. Sub-grid sampling (SGS) was used. The LiDAR was sampled at a distance of 1 metre, and all z shapes were sampled at a distance of half a metre.

3.2.2 Standalone Structures

Standalone culverts were incorporated as 1D elements linked to the 2D domain. The geometry and inverts of major culverts were provided by Wellington Shire Council. Some small additional culverts were identified, and were implemented as small culverts with a high blockage to allow for free-drainage.

3.2.3 Stormwater (Pit and Pipe) Network

The pit and pipe network was included as a one-dimensional model. Pipe sizes were taken from the Council-provided dataset. All pipes were assumed to be circular unless otherwise stated. Where diameter data was missing – the diameter was assumed from upstream and downstream pipes. Where invert data was missing the depth of cover values in Table 3-2 were assumed to calculate pipe invert levels, where IL = terrain surface – cover – diameter. This data was converted into the appropriate format for use in TUFLOW. Where the stormwater network exits the 2D domain, 1D water level boundaries were used and the obvert of the pipe was the assumed tailwater level.

Table 3-2 Assumed Pipe Cover to determine pipe invert levels

Pipe Diameter (mm)	Assumed Depth of Cover (mm)
Less than or equal to 900 mm	600
Greater than 900 mm	750

Pit inlet curves were adopted from the Sutherland Shire Council curves. These curves are well researched through physical testing and are one of the recommend curves provided by BMT WBM (2019) for use in the TUFLOW pit and pipe network modelling.

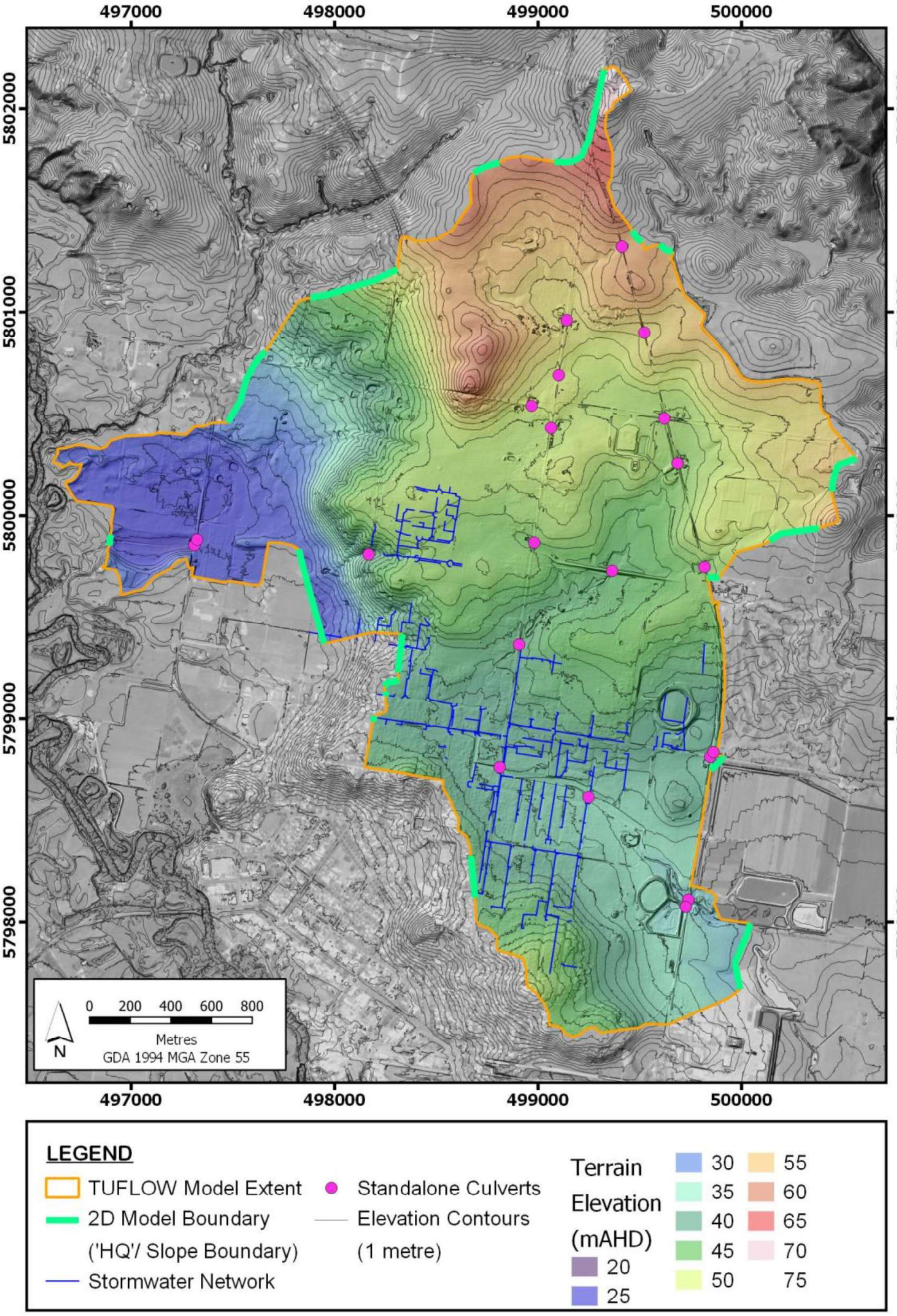


Figure 3-2 TUFLOW Model Setup (Existing Case)

3.2.4 Mannings Roughness

Manning's roughness values used throughout the model are listed in Table 3-3 and the roughness delineation is shown in Figure 3-3. Manning's values that have been used throughout the model were developed to comply with the ARR2019 Guidelines as per Table 6.2.1 – Values of Roughness Coefficient n for different channel conditions (Sellin 1961) and Table 6.2.2 Valid Manning 'n' Ranges for Different Land Use Types, with both tables and references cited in (Lambert, M. B. Cathers & R Keller, 2019).

Table 3-3 Roughness (Manning's 'n') Values

Description	'n' value
Open Areas and Parks	0.04
Farming	0.05
Dense Vegetation	0.08
Rural Residential	0.1
Low Density Residential	0.2
Industrial / Commercial	0.3
Roads	0.02
Cemetery	0.15
Waterway	0.045
Maintained Channel	0.04
Water Body	0.03

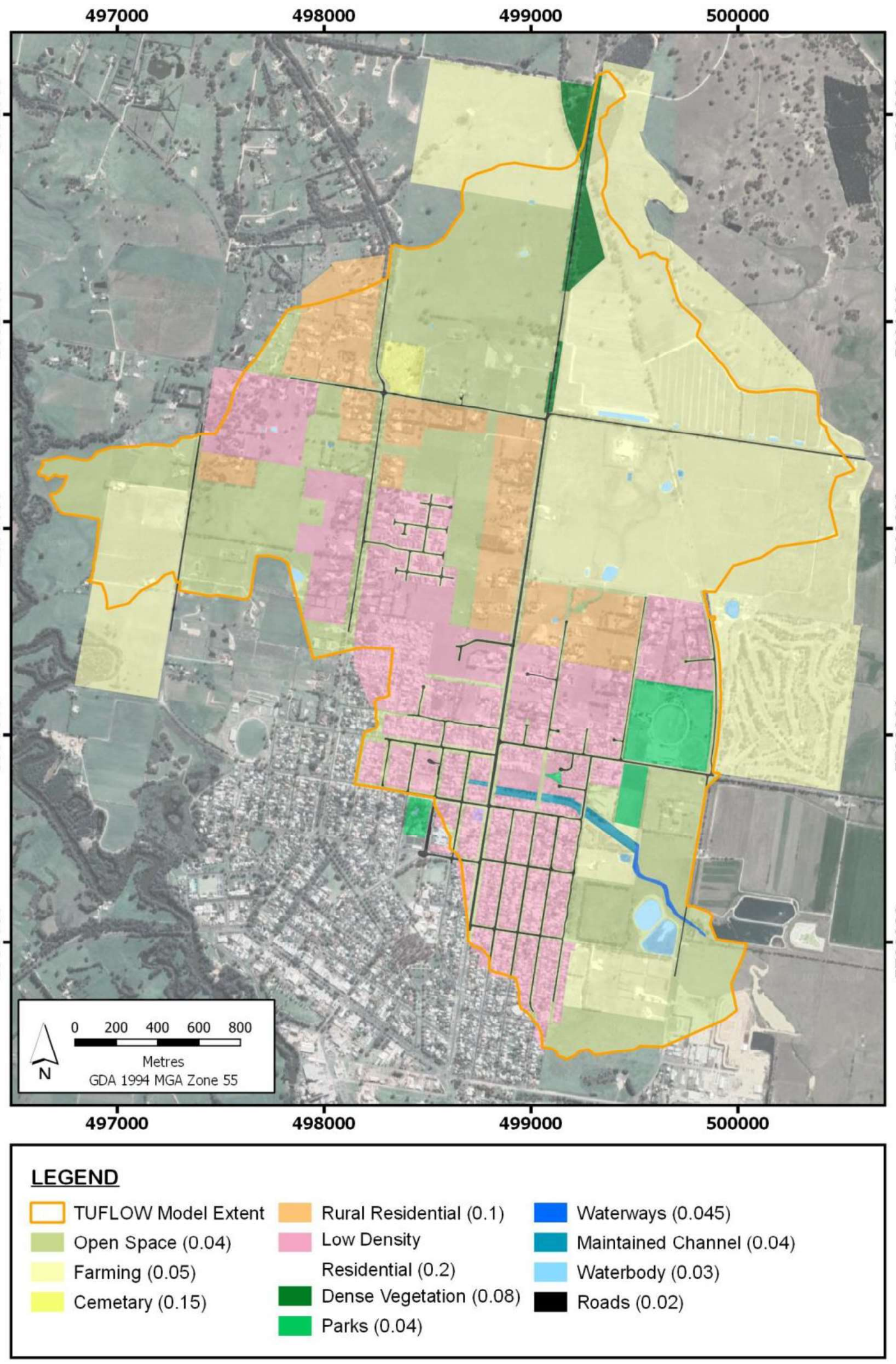


Figure 3-3 Existing Case Roughness Delineation

3.2.5 Initial Water Levels

A gridded water level was applied as an initial water level. This grid was developed by extracting the final water level from a simulation after all of the free-draining cells had emptied. The purpose of applying an initial water level grid is to fill any DEM depressions that are artefacts of the LiDAR rather than being true depressions in the terrain.

3.2.6 Hydrologic Inputs

Rainfall excess hydrographs were extracted from the RORB model at each node. These were applied as source-area (2d_SA) inflows, evenly distributed over each sub-catchment as illustrated in Figure 3-4.

Where sub-catchments were not entirely covered by the TUFLOW model domain, the flows were scaled by the proportion of the sub-catchment covered by the TUFLOW domain.

For each AEP, three indicative ARR 2019 ensemble temporal patterns were identified to broadly represent an ARR2019 ensemble approach, whilst maintaining modelling efficiency. The temporal patterns chosen correspond to a front-loaded, mid-loaded, and rear-loaded pattern. The storms identified from the RORB modelling were run through the hydraulic model as listed in Table 3-4.

Table 3-4 Temporal Patterns

AEP	Duration	Temporal Patterns
1%	20 minutes	2, 4, 8
	45 minutes	2, 3, 10
	1 hour	1, 5, 10
	1.5 hours	5, 9, 10
	4.5 hours	4, 5, 7
	9 hours	3, 5, 10
	12 hours	3, 5, 8
20%	20 minutes	1, 3, 5
	45 minutes	3, 7, 9
	1 hours	2, 7, 9
	1.5 hours	3, 5, 10
	4.5 hours	2, 5, 9
	9 hours	3, 9, 10
	12 hours	2, 9, 10

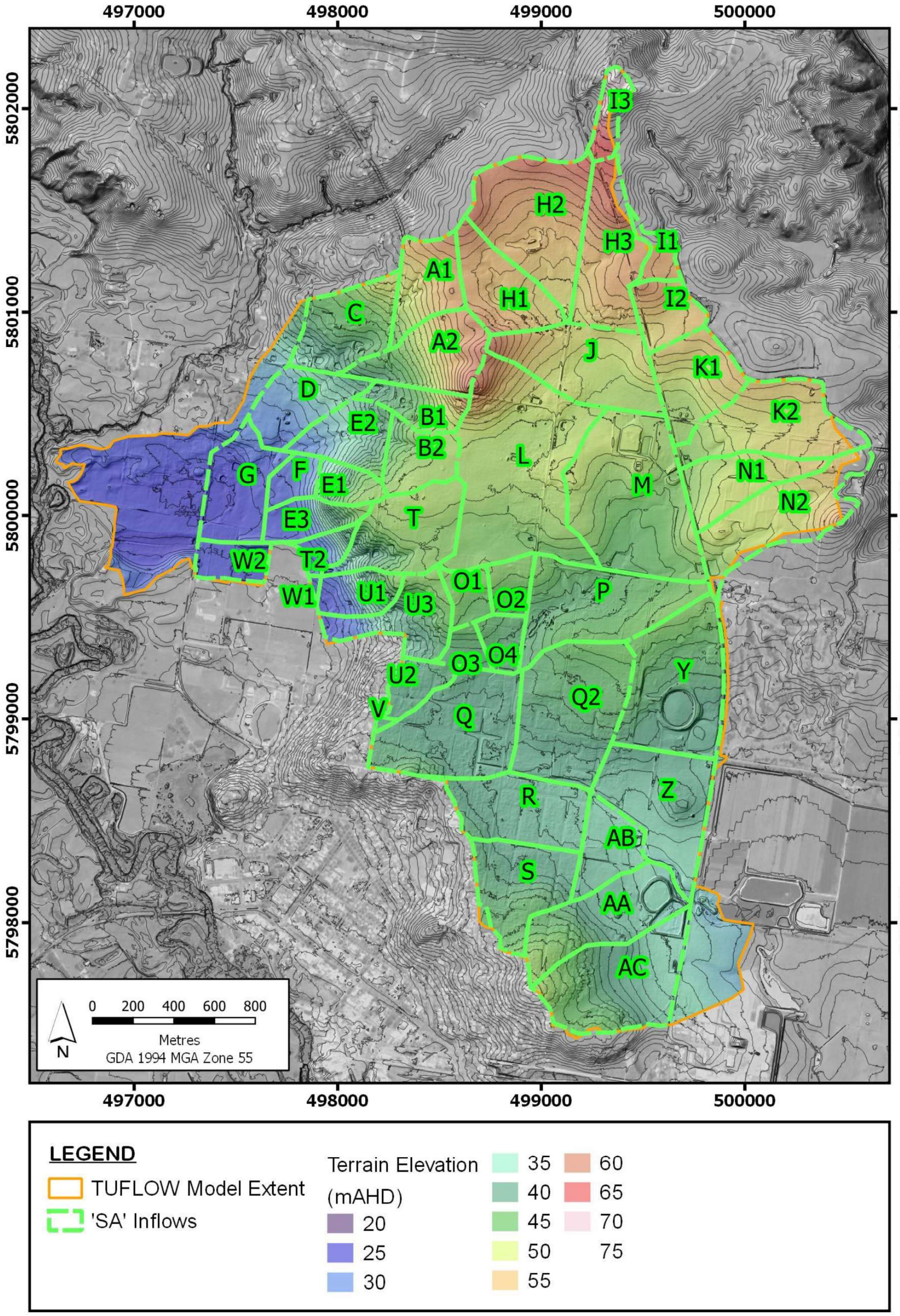


Figure 3-4 Rainfall Excess Inflow Extents

3.3 DESIGN SCENARIO

The design scenario incorporates three major changes: implementation of the basin and channel design, an update of the hydrology and an update of the model roughness to match increased fraction impervious due to ultimate land use according to the zoning. The implementation of these aspects into the hydraulic model is described in the following sections.

3.3.1 Basin and Channel Upgrade Implementation

Basin topography was implemented as a .dem exported from 12D as provided by Alluvium and illustrated in Figure 3-5. The design scenario culverts and pipes are listed in Table 3-5 and these were copied from the design scenario RORB model. Inter-basin pipes have not been finalised for this preliminary stage of design, and were implemented using a nominal 525 mm diameter structure.

Table 3-5 Design Case Culvert Details

Culvert Location	Geometry
East Catchment Wetland outfall	525 mm RCP Upstream Invert 43.6 mAHD Downstream Invert 43.5 mAHD
Catchment 'O' wetland outfall	600 mm RCP Upstream Invert 40.0 mAHD Downstream Invert 39.5 mAHD
Catchment 'O' sediment basin – macrophyte zone pipe	525 mm RCP Upstream Invert 40.2 mAHD Downstream Invert 40.0 mAHD

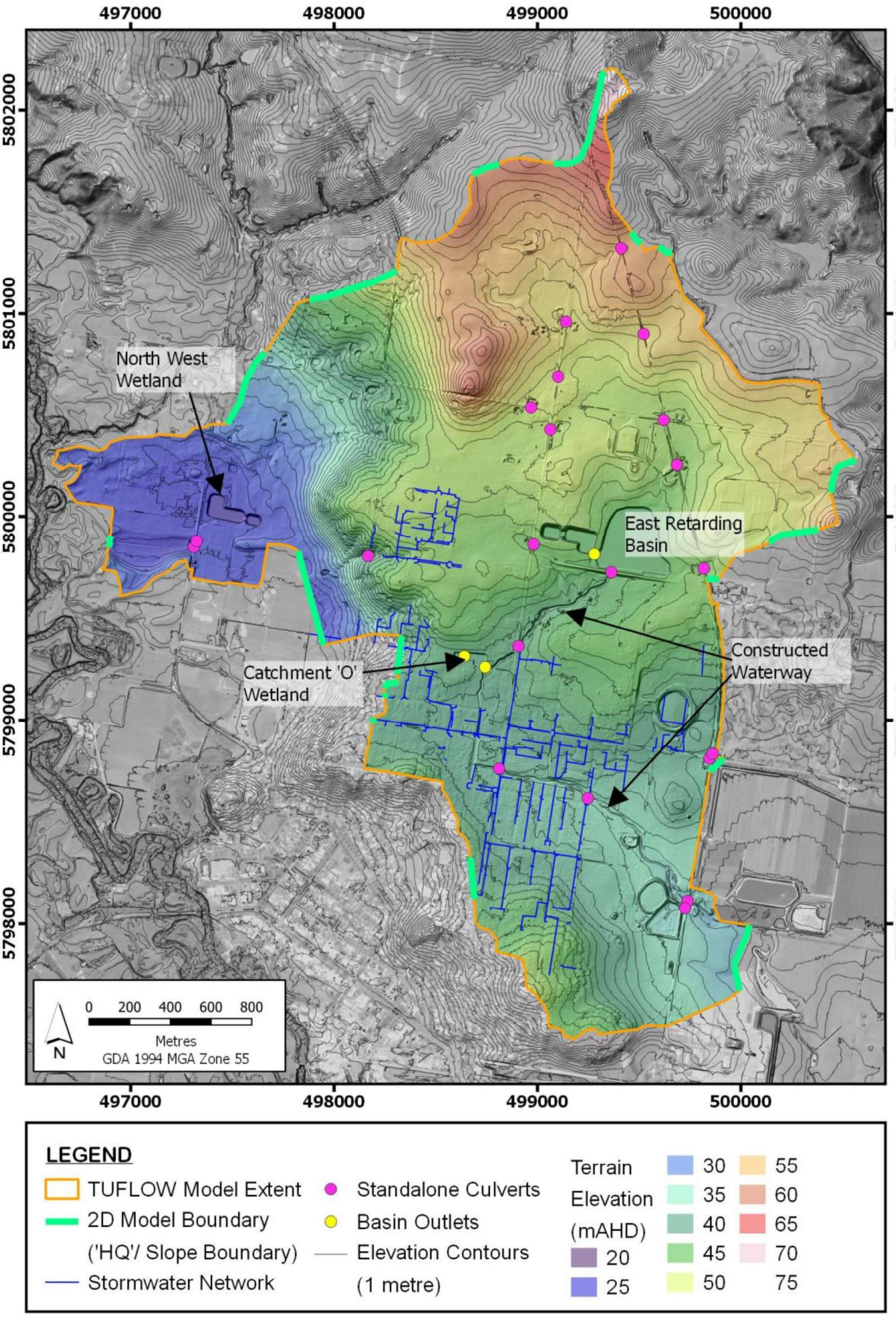


Figure 3-5 TUFLOW Model Setup (Developed Scenario)

3.3.2 Roughness

Roughness values for the design scenario are listed in Table 3-6. A depth-varying roughness was applied to the constructed channel, from 0.02 to 0.05 at depths of half a metre or higher. The smooth roughness at lower depths was chosen to represent the bare-earth case immediately post-construction before vegetation has had a chance to establish in the low flow channel.

Melbourne Water recommends manning's 'n' values of between 0.018 for straight earth channels to 0.06 for established vegetated high-flow channels (Constructed Waterways Design Manual Part D: Technical Requirements, 2019).

It is recommended that roughness values be revisited in the detailed design phase.

The delineation of roughness is shown in Figure 3-6, and incorporates extended areas of rural residential which were previously open space, to represent the ultimate development according to council zoning.

Table 3-6 Roughness (Manning's 'n') Values for Design

Description	'n' value
Constructed Waterway	Depth varying from 0.02 at 0.2 m to 0.05 at 0.5 m and above
Sediment Basin	0.02
Macrophyte Zone	0.05

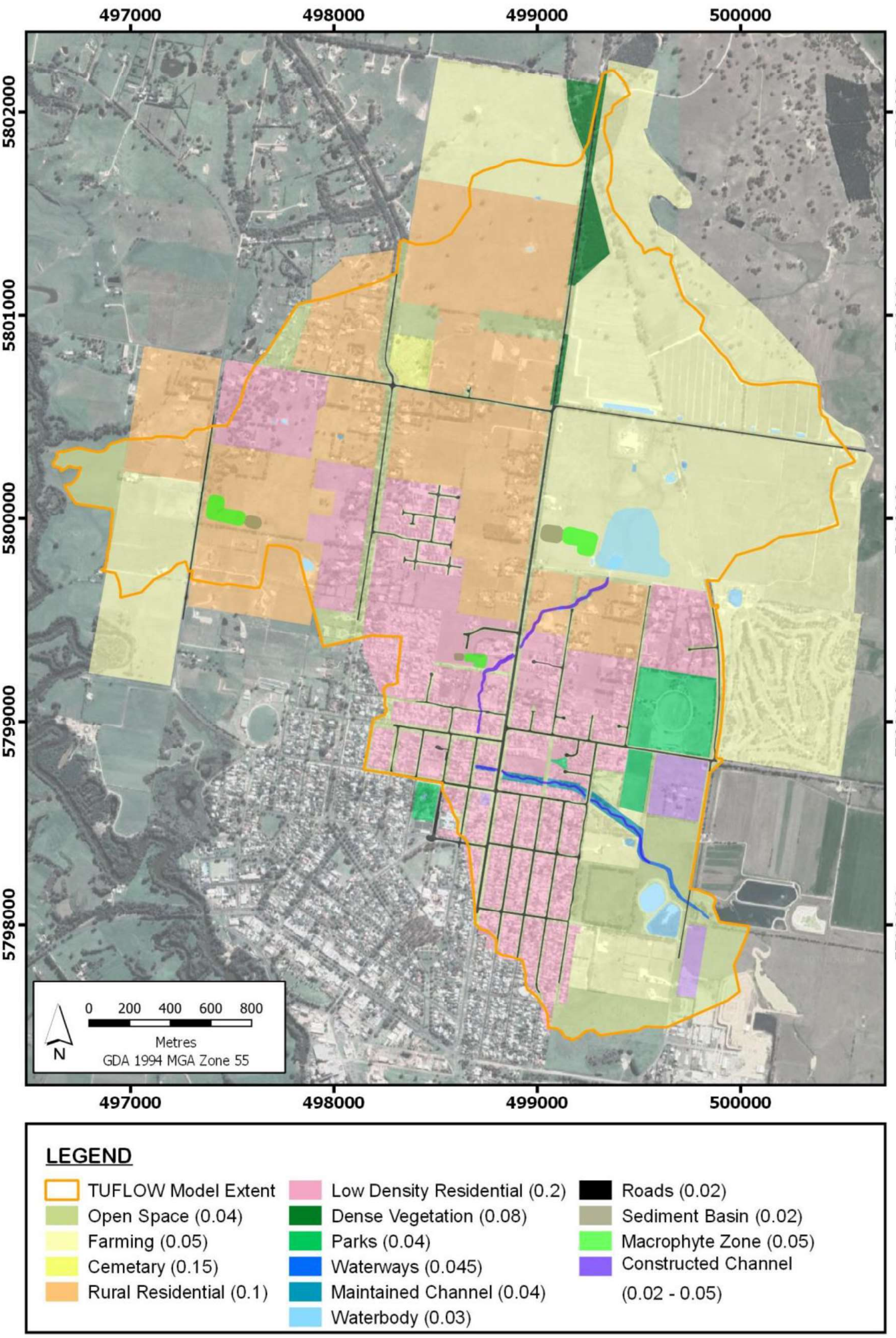


Figure 3-6 Developed Case Roughness Delineation

3.3.3 Hydrologic Inputs

Developed scenario RORB excess hydrographs were provided by Alluvium for use in the design scenario modelling. The increase in fraction impervious was incorporated into the RORB model, and therefore these hydrographs produce a higher volume of runoff than the existing case hydrographs for the same storms. The same temporal patterns identified for use in the existing scenario were adopted for the design scenario.

3.4 SUMMARY OF MODELLED EVENTS

The existing and design scenarios were both modelled for 1% and 20% AEP events. In addition, the design scenario was modelled for climate sensitivity and Probable Maximum Flood (PMF) events. The hydrologic inputs for the climate change and PMF runs were provided as RORB rainfall excess hydrographs. All simulated existing scenario events are listed in Table 3-7 and all simulated design scenario events are listed in Table 3-8.

Table 3-7 Existing Scenario Modelled Events

Probability	Durations	Temporal Patterns
20%	20 minute 45 minute 1 hour 1.5 hour 4.5 hour 9 hour 12 hour	As outlined in Table 3-4
1%	20 minute 45 minute 1 hour 1.5 hour 4.5 hour 9 hour 12 hour	As outlined in Table 3-4

Table 3-8 Design Scenario Modelled Events

Probability	Durations	Temporal Patterns
20% AEP	20 minute 45 minute 1 hour 1.5 hour 4.5 hour 9 hour 12 hour	As outlined in Table 3-4
1% AEP	20 minute 45 minute 1 hour 1.5 hour 4.5 hour 9 hour 12 hour	As outlined in Table 3-4

Probability	Durations	Temporal Patterns
Climate Change RCP 4.5 – 1% AEP	20 minute 45 minute 1 hour 1.5 hour 4.5 hour 9 hour 12 hour	Same as for 1% AEP design runs
Climate Change RCP 8.5 – 1% AEP	20 minute 45 minute 1 hour 1.5 hour 4.5 hour 9 hour 12 hour	Same as for 1% AEP design runs
PMF	15 minute 30 minute 45 minute 1 hour 1.5 hour 2 hour 3 hour	ARR 1987 GSDM Temporal Pattern

3.5 POST-PROCESSING

Peak value envelope surfaces were calculated for depth, water level, and velocity result types for each modelled event. These were derived by firstly finding the median water value (out of three modelled temporal patterns/storms) at each cell for each storm duration. The maximum of these 'median' values was then calculated at each cell to produce the peak value envelope surface. This process is shown in Figure 3-7.

The afflux result for each event was created by subtracting the existing scenario peak water level envelope surface from the developed scenario peak water level envelope surface.

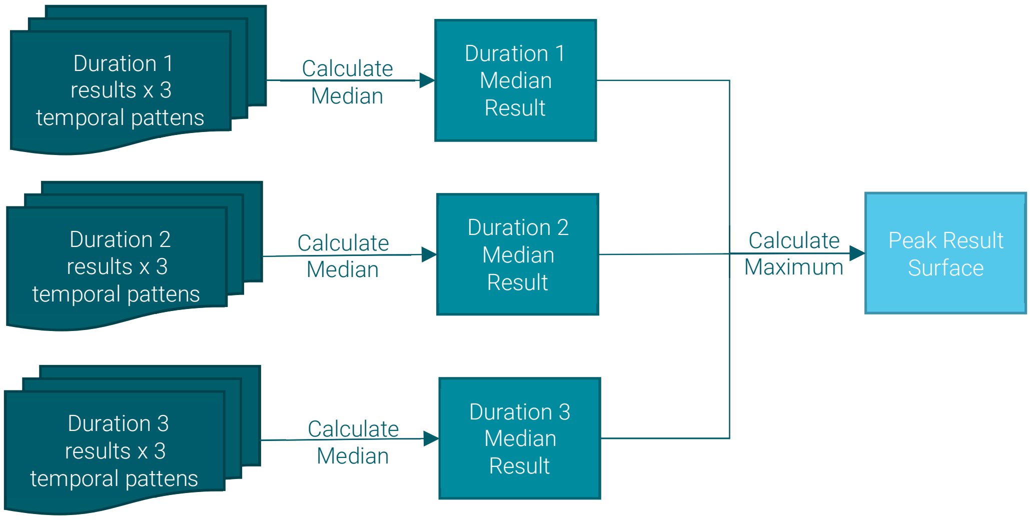


Figure 3-7 Peak Result Envelope Process

4 RESULTS

The following sections provide discussion of the hydraulic model results. Mapping of results is provided in Appendix A.

4.1 FLOOD EXTENT AND DEPTHS

Under existing conditions for the 1% AEP event, the majority of flooding is occurring from the north east along the ephemeral stream from the location of the basin. In some locations flood depths along the ephemeral stream exceed one metre. As the flooding reaches Merry Street, flows split and some travels west along Merry Street, whilst the remainder travels south along Alfred Street. Modelling also demonstrates that flows that travel west along Merry Street, then again split at McCubbin Street, where some flow travels south towards George Street, east along George Street and returning to the main flow path at Alfred Street. There is significant pooling of water along Alfred Street prior to the flows turning east and following the Davis Street Drain downstream to the outlet of the model at Fulton Road. The water joins the natural waterway at this location. Throughout the remainder of the township, flooding is relatively shallow overland flows due to local catchment rainfall with depths typically less than 100mm.

Flood behaviour under existing conditions for the 20% AEP event is similar with a lesser degree of severity.

The developed scenario results in a reduced extent of flooding more broadly across the township and due to the development of the upstream detention basin, as detailed within the Alluvium Maffra Drainage and Integrated Water Management Strategy, water levels are typically lower within the constructed channels. Under developed conditions it has been observed that there is an increase in flooding at a number of locations including the junction of George Street and Alfred Street and again along Powerscourt Street at the culvert crossing to the Davis Street Drain. This afflux is discussed in detail in Section 4.2 below.

Detailed water level, depth velocity and afflux maps are provided in Appendix A.

4.2 WATER LEVELS AND AFFLUX

In general, water levels are lower in the proposed constructed channel than along the existing channel due to flow being further retarded upstream by the increased detention basin size. There are a few areas where the water levels are higher in the developed case. These locations are discussed in sections 4.2.1 through 4.2.4 and the afflux plots are illustrated in detail in Appendix A.

4.2.1 Alfred Street / George Street Junction

The afflux at this location is showing approximately 20 – 50 mm across the residential properties. This is due to the deepening and widening of the upstream channel. Alfred Street then becomes the hydraulic control, as the formalised waterway cannot be continued down Alfred Street where it then meets the Davis Street drain. Figure 4-1, below, shows the smooth transition of the terrain from the drainage easement crossing George Street into Alfred Street under existing conditions (blue line), whereas, under developed conditions (red line), there is approximately 600mm rise between the residential easement and George Street. This jump in terrain is causing water to pond up behind the road before overtopping the road with a slightly greater depth and therefore spreading further along George Street and flowing down through the residential properties. A small portion of these flows, however, will be transferred along the George Street Drain – please refer to the Alluvium Maffra Drainage and Integrated Water Management Strategy for further discussion on the George Street Drain.

Figure 4-1 Terrain long section along drainage easement from Merry Street then along Alfred Street (Crossing George St) (blue represents existing case, red represents design case)

4.2.2 Powerscourt Street Along the Open Drain

There is noticeable afflux of 30-70mm in the 1% AEP event, across the residential properties in the vicinity of Powerscourt Street flowing down and along Davis Street. At this location the afflux is caused by the culvert. It is observed that without upgrading or changing the culvert inverts in conjunction with the deepening and widening of the channel at this location, the culvert becomes a hydraulic control. Figure 4-2 is a long section of the existing vs developed terrain within the channel. It can be seen that the difference in channel invert levels is approximately 600mm. Figure 4-3 shows that as a result of these changes, whilst the peak flow through the culvert at this location is approximately ½ - 1 cumec higher in the developed case, the volume is significantly lower and thus the flow is now ponding behind the culvert – overtopping the road, spreading more widely and flowing down to, and along Davis Street.

The design of culvert upgrades is required for the next phase of the study and will ensure no adverse impact on the flooding.

Figure 4-2 Long section through the Powerscourt Channel across Powerscourt Street (blue represents existing channel; red represents proposed channel)

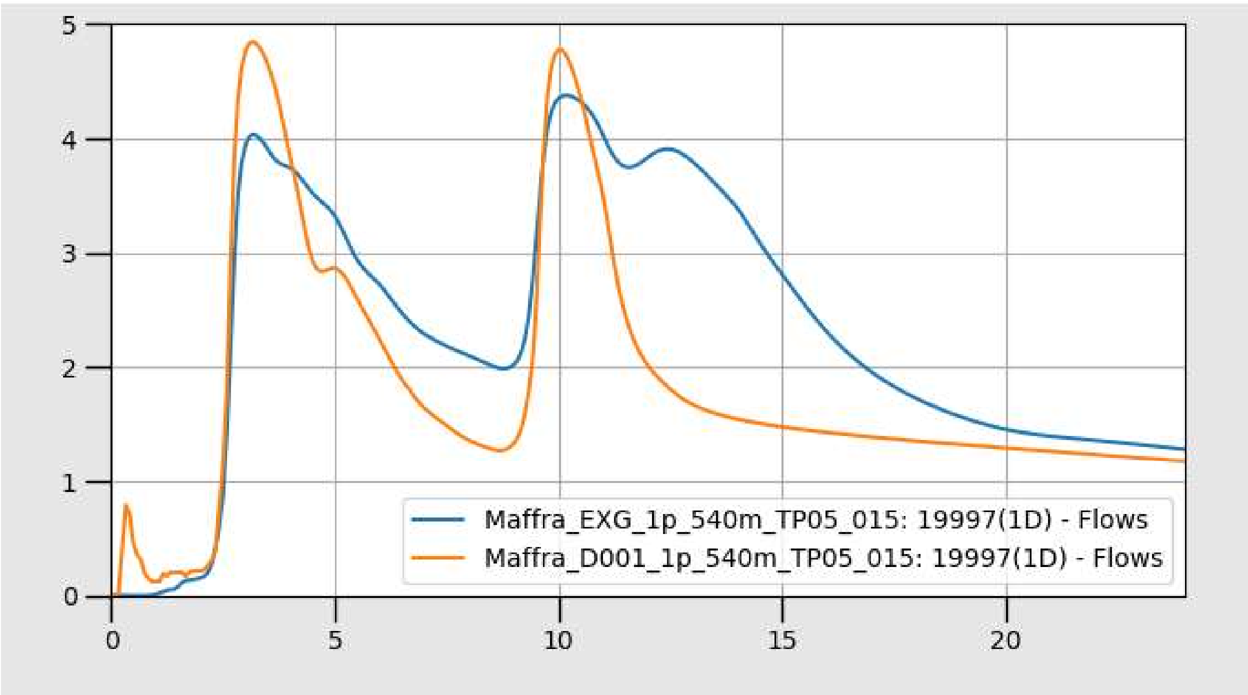


Figure 4-3 Culvert at Powerscourt Street (blue represents existing scenario hydrograph; orange represents developed scenario hydrograph)

4.2.3 Northern Powerscourt Street

Similarly, the culvert within the northern part of Powerscourt Street is showing a slightly higher peak flow but significantly less volume in the 20% AEP (Figure 4-5) and thus there is afflux caused due to flows overtopping the road. The location of the culvert in the northern part of Powerscourt Street is illustrated in Figure 4-4. As discussed in Section 4.2.2 above, culvert upgrades are due to be considered within the next phase of works.

Figure 4-4 Location of northern Powerscourt Street culvert

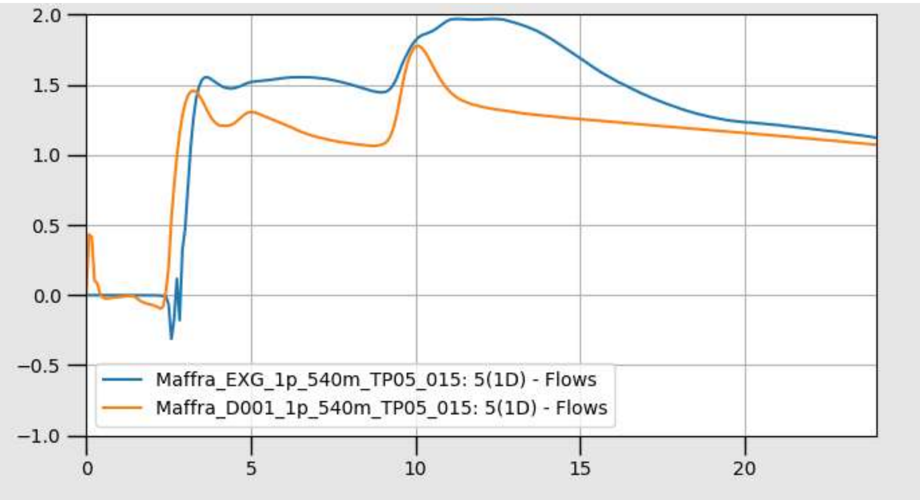


Figure 4-5 20% AEP flow Hydrograph through Culvert at northern section of Powerscourt Street (blue represents the existing scenario culvert hydrograph, orange represents the developed scenario culvert hydrograph)

4.2.4 Western Basin / Wetland

The afflux downstream of the western basin ranges between 75mm and 175mm. There are two possible reasons for this occurrence:

The basin outfall has only been designed from the basin itself at this stage. A pipeline or open channel will need to be incorporated all the way to the outfall at Macalister River, this has not, as yet, been incorporated into the modelling. A fully designed outfall will assist in alleviating the afflux that is demonstrated at this location. Secondly, there is an increase in flow volume due to the increased impervious fraction runoff. The increase in volume and corresponding increase in roughness for the ultimate development case is responsible for widespread low-level afflux over the western catchment.

4.2.5 Culverts

Detail of culvert upgrades are not part of the scope of this flood study. It is intended that culvert upgrades will be considered as part of the next phase of the study or within detailed design.

4.2.6 Eastern Model Outfall

The model illustrates approximately 60 – 80mm afflux downstream of the channel widening works at the model outfall on the Eastern side of the catchment. This afflux appears to be due to the channel widening works not continuing further downstream than they have, and there may also be some issues with the culvert crossing at Fulton Road.

4.3 PMF

The PMF Scenario has been modelled for the developed conditions. The flood extent for the PMF scenario is significant, with flows overtopping the channel and ponding at Merry Street. The depths in the channel from the detention basin all the way to the model outlet are greater than 1m. The depths are typically between 500mm and 1m for the flash flooding across significant parts of the residential area with the whole residential area covered by flash flooding to at least 20-50mm and much of it to depths of 100mm.

PMF flood maps are provided in Appendix A.

4.4 CLIMATE CHANGE

The RCP4.5 and RCP8.5 for 2100 climate change scenarios have been modelled for the developed case. Results of these scenarios show that the depths of flooding within the channel are greater than 1m. There is significant ponding at the junction of George Street and Alfred Street – continuing along Alfred Street. And flood flows along Merry Street are much greater than in the 1% AEP event. In addition, the flash flooding behaviour shows depths up to 200mm, where in the 1% AEP event flood depths were typically 100mm. The RCP8.5 event shows slightly greater depths of flooding that the RCP4.5 event.

Climate Change flood maps are provided in Appendix A.

4.5 FLOWS

A comparison has been undertaken of the RORB modelled flows with the TUFLOW model output flows for the purposes of supporting the hydrology that was undertaken by Alluvium. Flows were compared at two locations; upstream of the existing detention basin at sub area J, and within the western portion of the Maffra catchment at sub area D. The locations for the flow extraction are indicated in Figure 4-6 and Figure 4-9. The 1% AEP hydrographs for sub area J for existing and developed conditions respectively are illustrated in Figure 4-7 and Figure 4-8 for a critical duration of 1.5 hours. The 1% AEP hydrographs for sub area D for existing and developed conditions respectively are illustrated in Figure 4-10 and Figure 4-11 for a critical duration of 9 hours. It should be noted when considering these comparisons that outflow hydrographs from RORB and TUFLOW will never be exactly the same as the routing equations used by the respective software platforms are quite different. In addition, RORB routes through 1-dimension, whereas TUFLOW routes through 2-dimensions.

It can be seen that for sub area J the peaks are occurring at approximately the same time – only differing by approximately 5-6 minutes. The peak values are also quite similar, with peak values differing by only 1-3 cumecs. This is acceptable given the way the software handles the routing.

For sub area D, the hydrographs are offset by approximately 10 minutes with the differences in peak flows showing between 3-4 cumecs difference. Again, this is considered acceptable given the difference in the handling of flows within the respective software platforms.

A further difference can be due to the slightly differing locations of flow extract, with the TUFLOW flow being extracted in a slightly different location to the RORB flow, which was extracted at the sub area outlets.

In summary, the TUFLOW modelling supports the results of the hydrology undertaken by Alluvium.

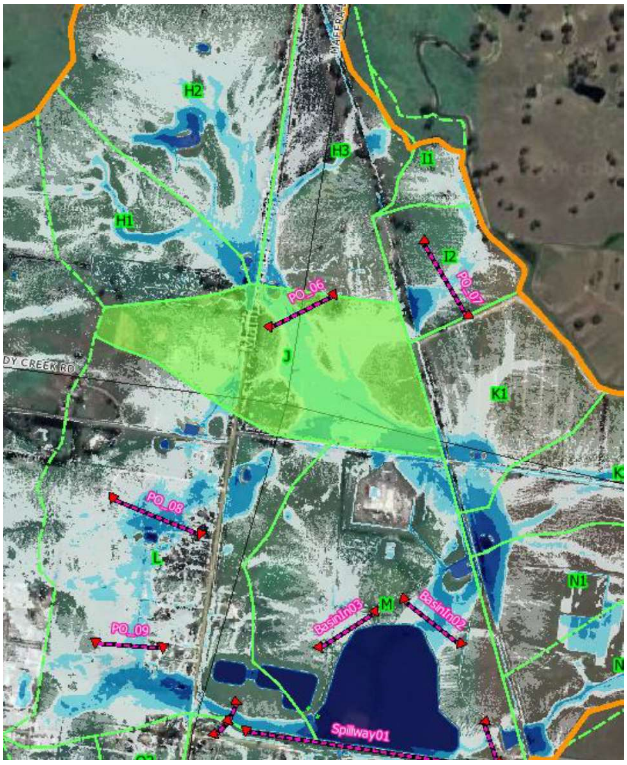


Figure 4-6 1% AEP sub area J flow extraction location

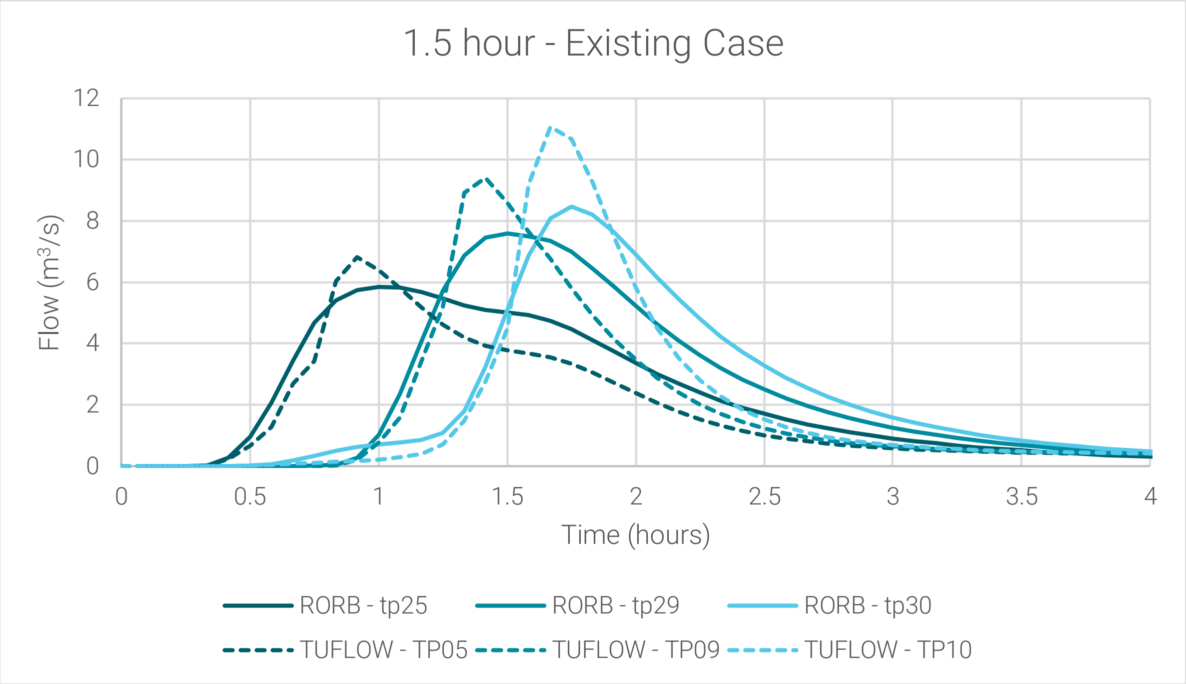


Figure 4-7 1% AEP hydrographs for sub area J – Existing Case

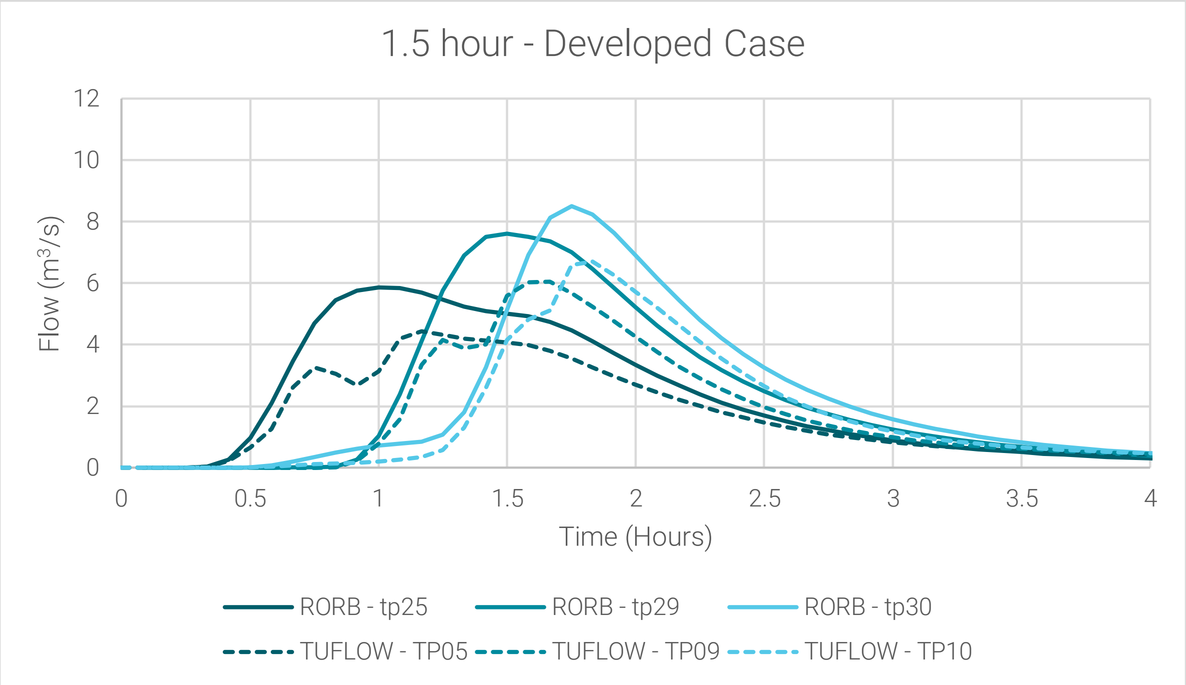


Figure 4-8 1% AEP hydrographs for sub area J – Developed Case

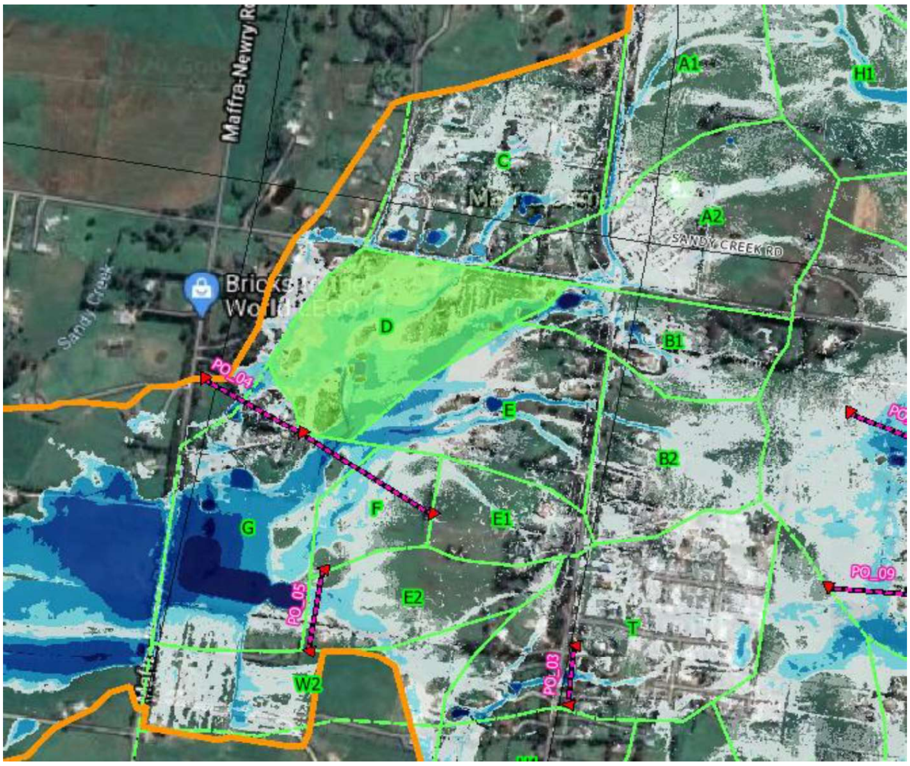


Figure 4-9 1% AEP sub area D flow extraction location

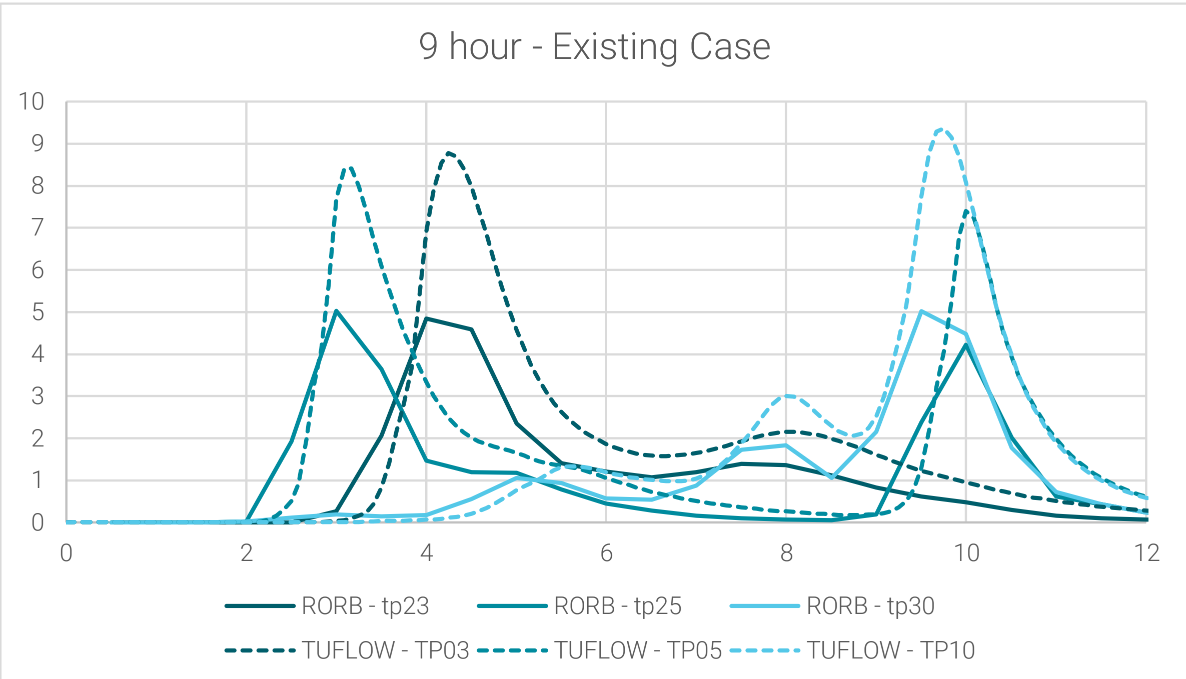


Figure 4-10 1% AEP hydrographs for sub area D – Existing Case

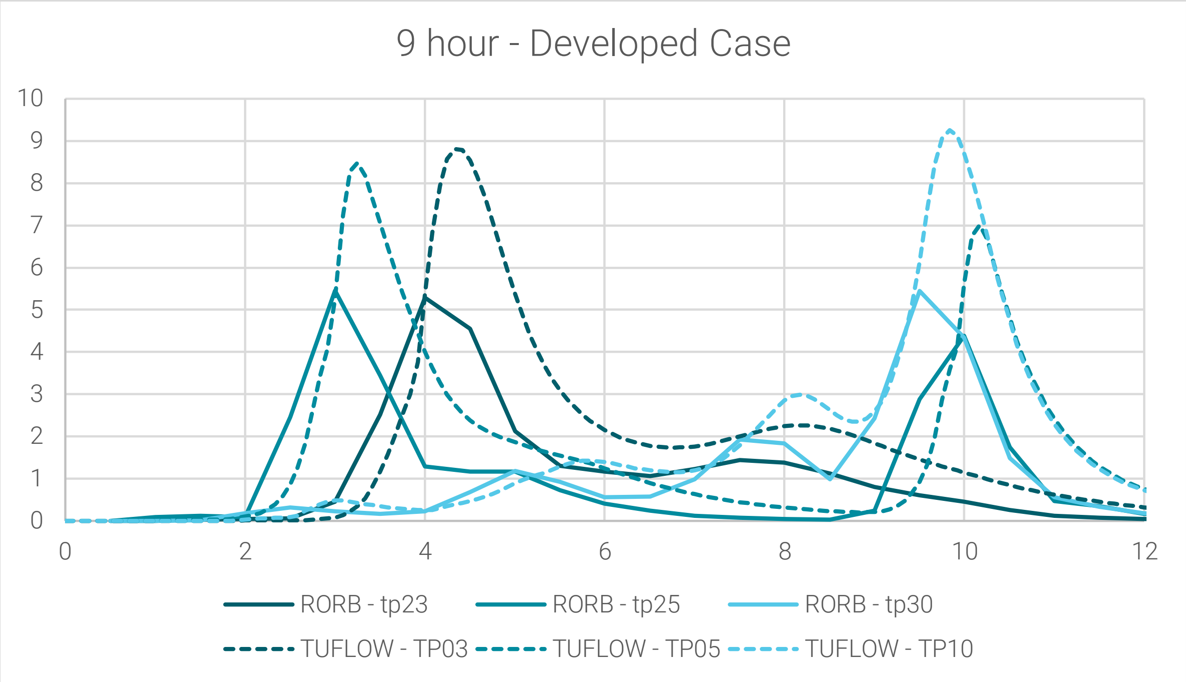


Figure 4-11 1% AEP hydrographs for sub area D outflows – Developed Case

4.6 COMPARISON TO OBSERVED FLOODING BEHAVIOUR

Historical photographs of the observed flooding from 1988 and 1993 at Powerscourt Street and at the intersection of Powerscourt and George Street are provided side-by-side to the results of the 1% AEP modelled flooding for the purposes of validation and comparison.

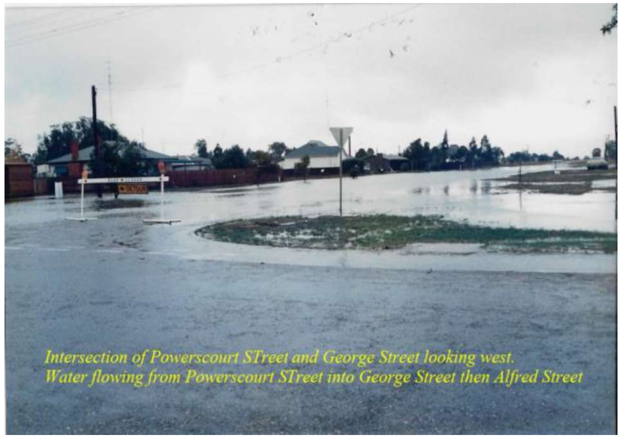
In Table 4-1 the historical photographs both indicate significant flooding along the floodway between Powerscourt Street and Alfred Street. This is confirmed by the 1% AEP modelling. Similarly, in

Table 4-2, both of the historical photographs indicate ponding of floodwaters at the intersection of Powerscourt Street and George Street. Similarly, this is confirmed by the 1% AEP modelling.

Table 4-1 Comparison of modelled to observed flooding at Powerscourt Street

1988	1993	1% AEP Modelling
		

Table 4-2 Comparison of modelled to observed flooding at the Intersection of Powerscourt Street and George Street

1988	1993	1% AEP Modelling
		

5 UNCERTAINTY

The following limitations are applicable to the data used as input to the investigation and the hydraulic modelling results and mapping deliverables. The modelling results should therefore be viewed in light of these limitations.

1. Due to insufficient data, the model was unable to be calibrated. The model has been compared and validated wherever possible to any known flooding or anecdotal evidence as supplied by WSC.

   If WSC are interested in collecting data for future calibration of models, it is recommended to install water level gauges at key points along the main flow paths, such as at the retarding basin, and at culverts where the flow path crosses Powerscourt Street, the Alfred Street/George Street junction or Fulton Road. In addition, the installation of a pluviograph rainfall gauge in the township would also allow for better calibration in future modelling exercises.

   In the absence of gauged data, a large and spatially-varied dataset of high water marks from a flood event, in conjunction with photographs would also be of use in model calibration.

2. The data supplied by WSC, in particular the pit and pipe network, was missing some of the meta data. Engineering judgement has been used to in-fill the missing data, as discussed in Section 2 and it is believed that for the purposes of this study, the data in-fill will be of acceptable quality.

6 SUMMARY AND CONCLUSIONS

Hydraulic modelling has been undertaken for the Maffra Township utilising rainfall-excess hydrology supplied by Alluvium. The modelling utilised the industry standard software, TUFLOW with a 1-dimensional drainage network connected to a 2-dimensional terrain.

A range of events were modelled for both the existing and developed scenarios including sensitivity scenarios for PMF and Climate Change for 2100 RCP4.5 and RCP8.5 for the developed scenario. Three indicative temporal patterns – front, mid and rear loaded, were chosen to represent the ensemble modelling as recommended in ARR2019. This methodology was adopted to ensure a combination of best practice with modelling efficiency.

Flood flow behaviour under existing conditions shows that flow travels from the north east at the location of the detention basin along the ephemeral stream, splitting at Merry Street with some flow travelling west along Merry Street and the remainder travelling south along Alfred Street. Of the flow that travels west along Merry Street, some is diverted south along McCubbin Street then George Street to re-join the Alfred Street flow path. Significant ponding of flows is occurring at the junction of Merry Street and the drainage reserve and along Alfred Street between Mclean Street and George Street. The township is also experiencing shallow sheet overland flows due to local catchment flash flooding.

Flood flow behaviour under proposed conditions shows that flood levels within the proposed channel is typically lower due to the upsizing of the detention basin. However, there are some areas of afflux due to the proposed design changes as previously discussed.

The flow behaviour is similar in the 20% AEP event with lower flood depths and lesser extent observed.

Flow behaviour and afflux has been discussed in detail and recommendations for detailed design have been provided.

7 REFERENCES

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*BMT WBM (2019), 1d Pits, [Online], August 14th, 2020, at <
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*Commonwealth of Australia (2019). 2016 Census QuickStats Maffra, Australian Bureau of Statistics. [Online]: November 11, 2019 at
https://quickstats.censusdata.abs.gov.au/census_services/getproduct/census/2016/quickstat/UCL215050*

Lambert, M, B Cathers, R. Keller (2019), Flood Hydraulics Book 6, in Australian Rainfall and Runoff – A Guide to Flood Estimation, Commonwealth of Australia

Smith, K (2020), Inception Meeting and Site Visit Photograph, 6/3/2020

*State Government of Victoria (2017), Spatial DataMart, [Online], August 14th, 2020, at <
http://services.land.vic.gov.au/SpatialDatamart/>*

APPENDIX A

RESULTS

solutions WATER MODELLING

Appendix A-1

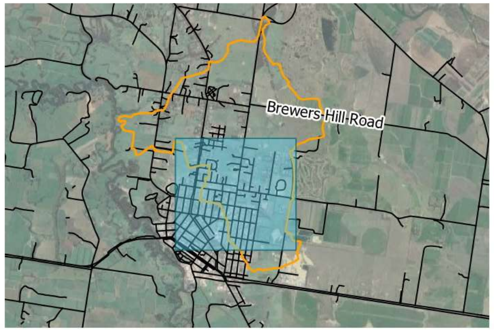
Town Centre
Existing Case
Peak Depth
20% AEP Event

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0

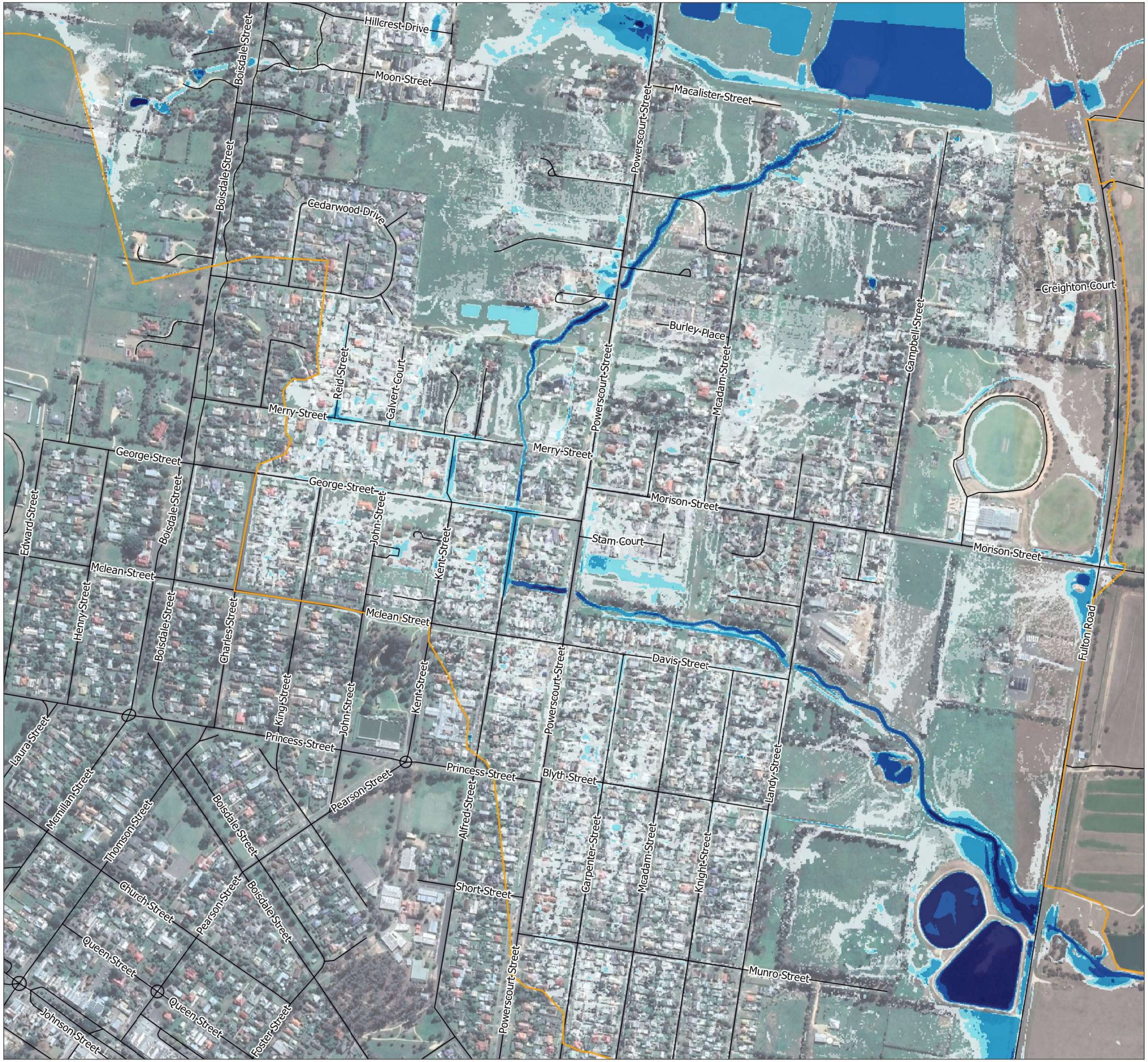


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-2

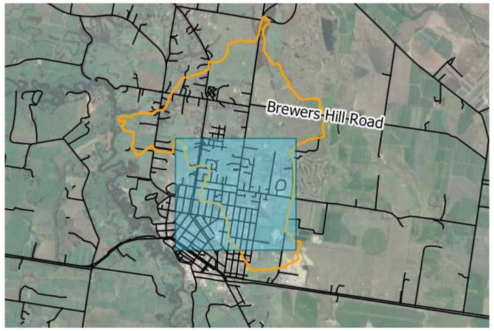
Town Centre
Developed Case
Peak Depth
20% AEP Event

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

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Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-3

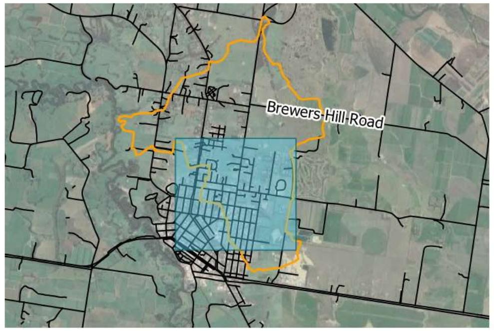
Town Centre
Existing Case
Peak Depth
1% AEP Event

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

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GDA 1994 / MGA Zone 55

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0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-4

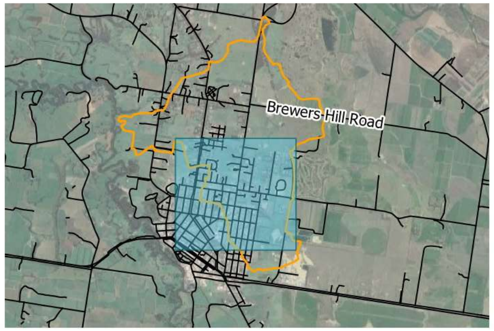
Town Centre
Developed Case
Peak Depth
1% AEP Event

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

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GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

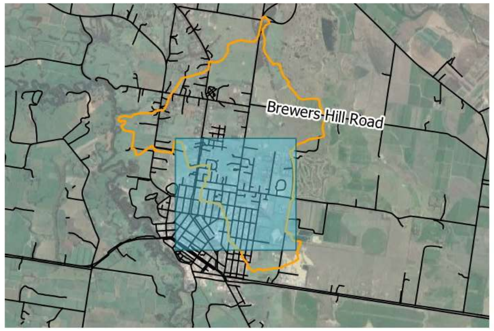
WATER MODELLING solutions

Appendix A-5

Town Centre
Existing Case
Peak Velocity
20% AEP Event

LEGEND

- Model Extent
- Velocity (m/s)
  - <= 0.25
  - 0.25 - 0.50
  - 0.5 - 1.00
  - 1.00 - 1.50
  - >1.5



Maffra Drainage Integrated Water Management Strategy

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Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

solutions
WATER MODELLING

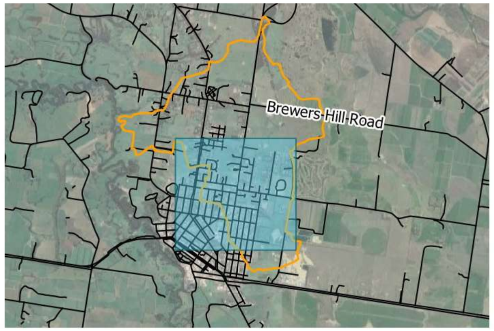
Appendix A-6

Town Centre
Developed Case
Peak Velocity
20% AEP Event

LEGEND

Model Extent

Velocity (m/s)
- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

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GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-7

Town Centre
Existing Case
Peak Velocity
1% AEP Event

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5

Brewers Hill Road

Maffra Drainage Integrated Water Management Strategy

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Job No: 30009
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0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-8

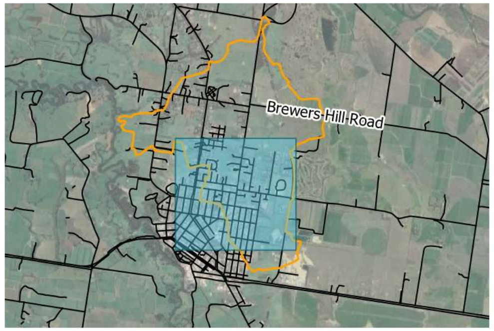
Town Centre
Developed Case
Peak Velocity
1% AEP Event

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



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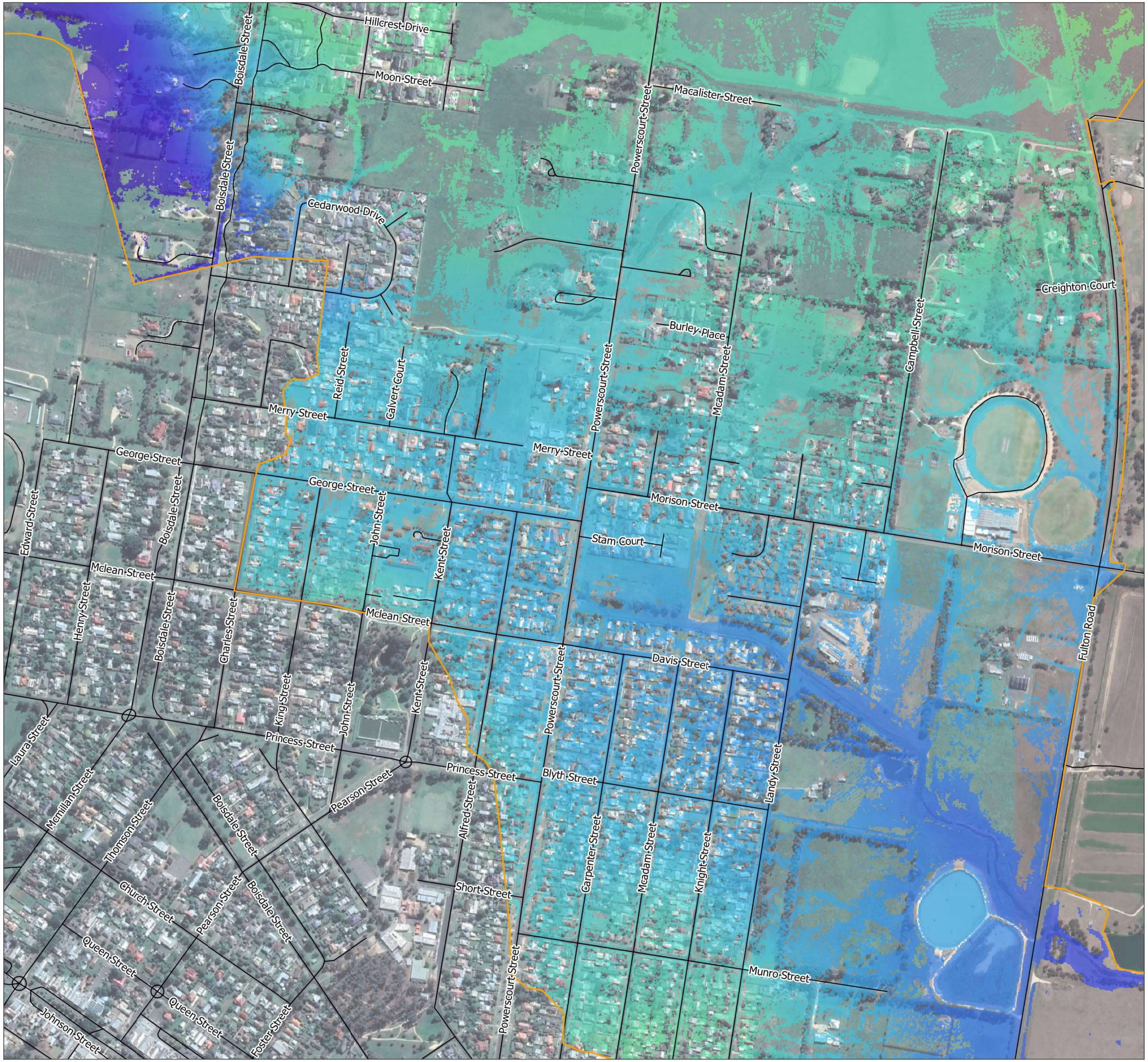
0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-9

Town Centre
Existing Case
Peak Water Surface Level
20% AEP Event

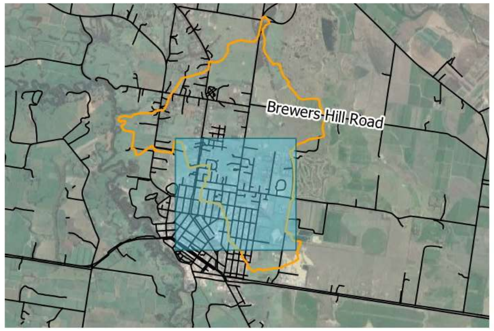


LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70



Maffra Drainage Integrated Water Management Strategy

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GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-10

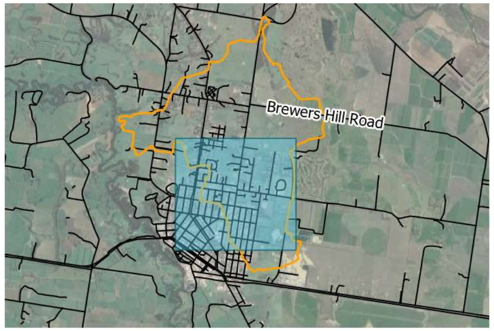
Town Centre
Developed Case
Peak Water Surface Level
20% AEP Event

LEGEND

Model Extent

Water Level (mAHD)

20
30
40
50
60
70



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0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-11

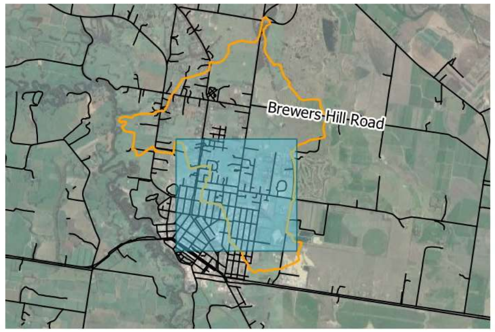
Town Centre
Existing Case
Peak Water Surface Level
1% AEP Event

LEGEND

Model Extent

Water Level (mAHD)

20
30
40
50
60
70



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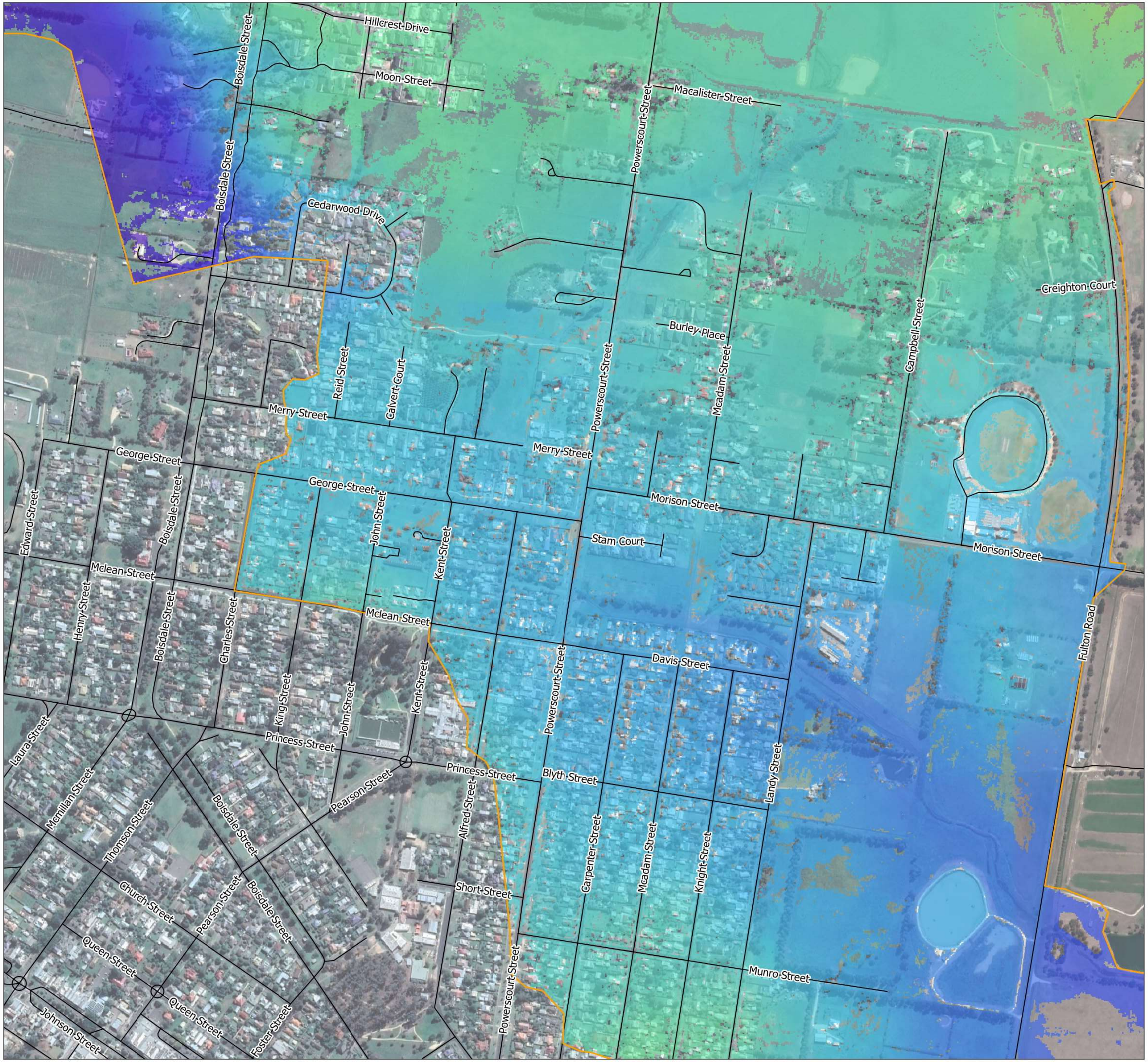
0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-12

Town Centre
Developed Case
Peak Water Surface Level
1% AEP Event

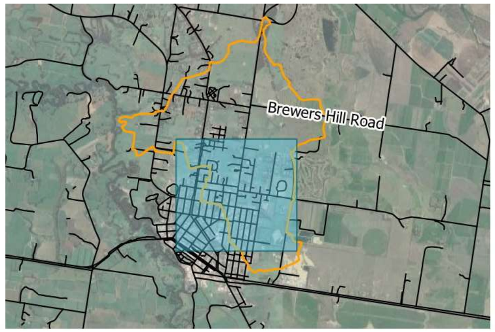


LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70

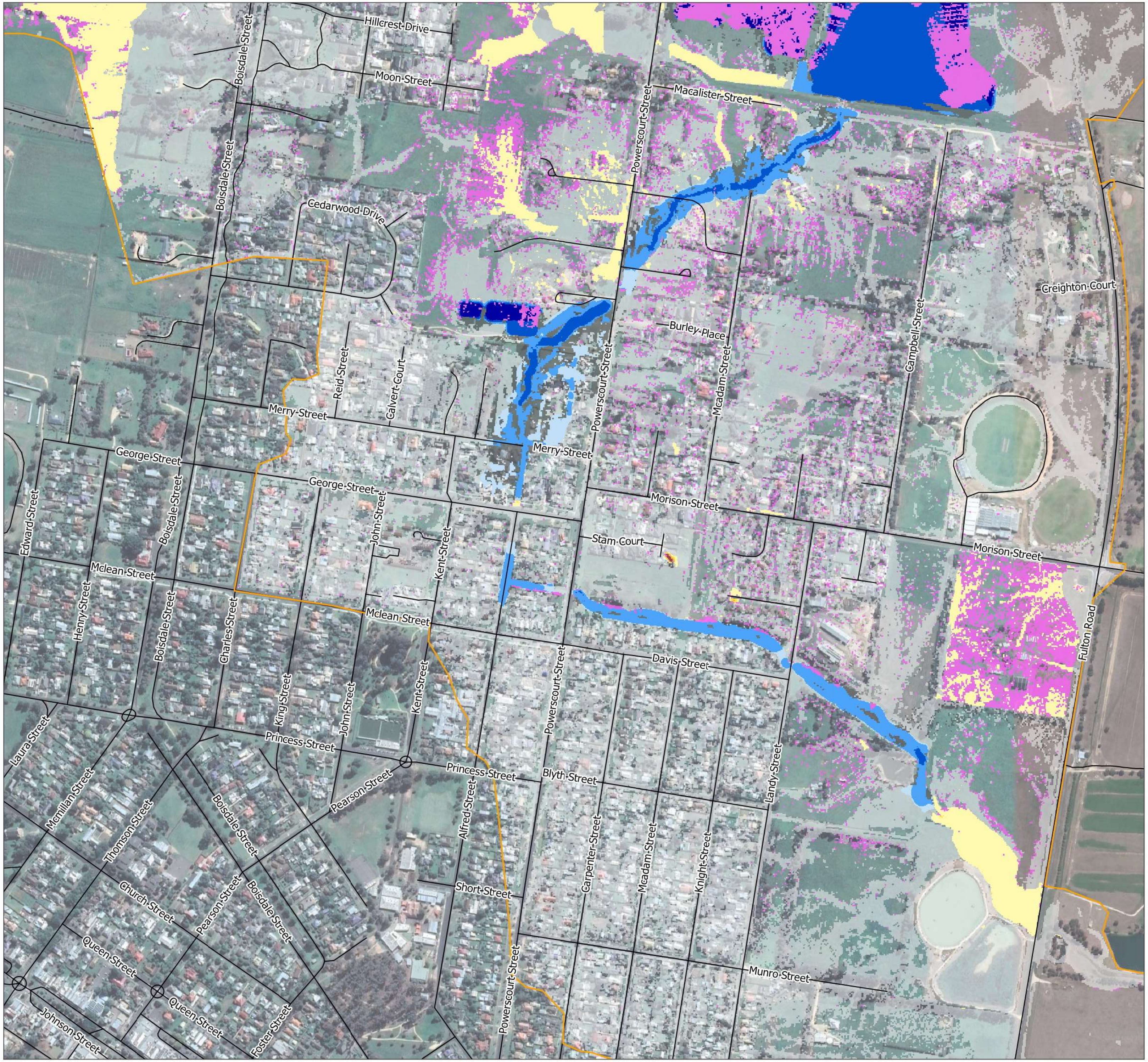


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



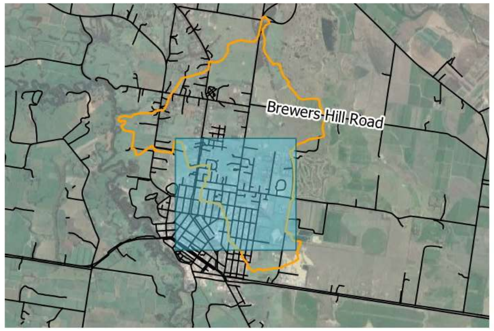
solutions
WATER MODELLING

Appendix A-13

Town Centre
Developed Case
Afflux
20% AEP Event

LEGEND

- Model Extent
- Change in Water Level (m)
- <= -1.0
- -1.0 to -0.50
- -0.50 to -0.10
- -0.10 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.10
- 0.10 to 0.25
- 0.25 to 0.50
- > 0.50
- Was Wet Now Dry
- Was Dry Now Wet

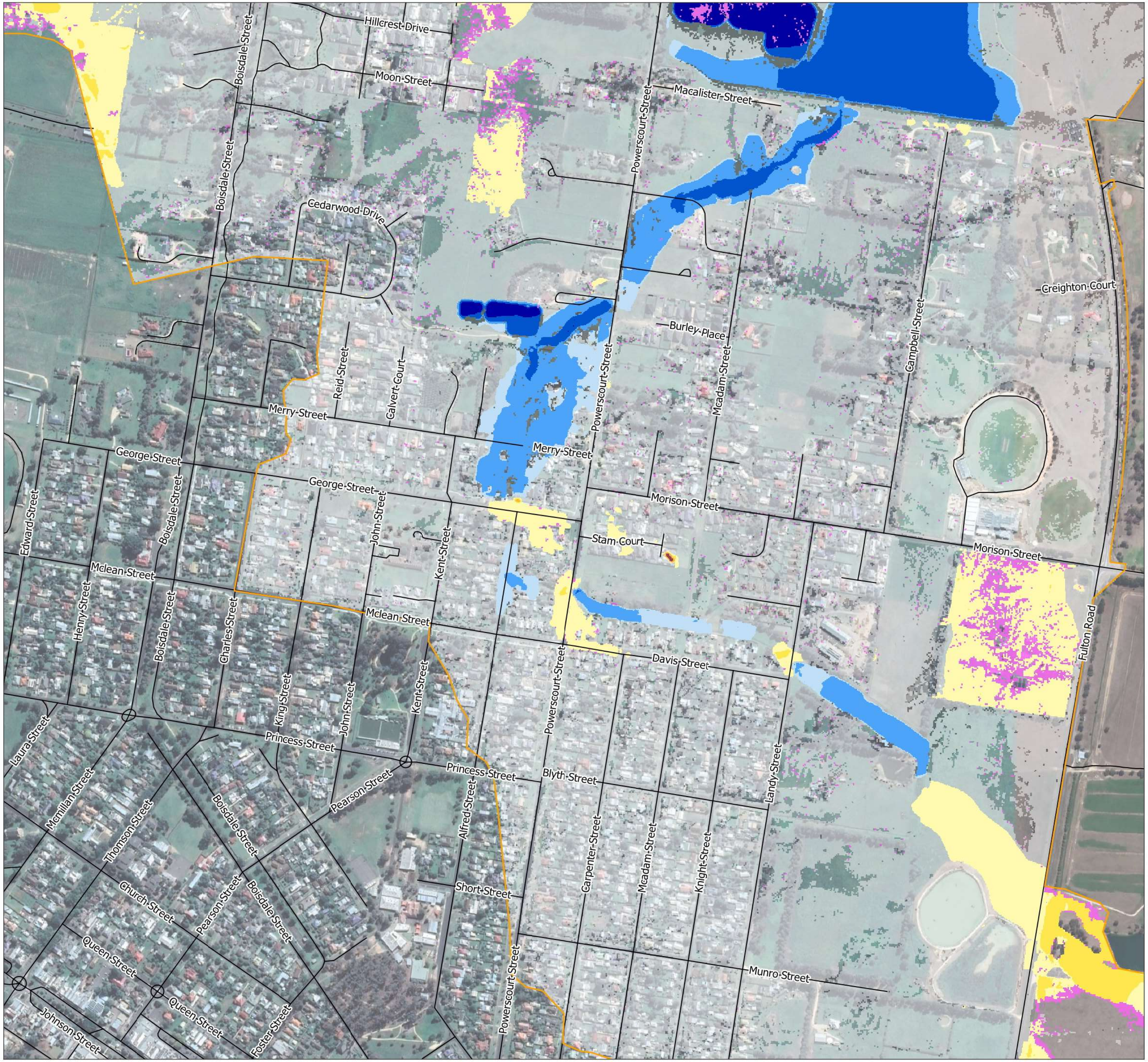


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



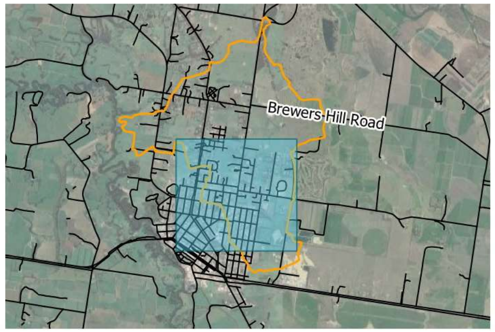
WATER MODELLING solutions

Appendix A-14

Town Centre
Developed Case
Afflux
1% AEP Event

LEGEND

- Model Extent
- Change in Water Level (m)
- <= -1.0
- -1.0 to -0.50
- -0.50 to -0.10
- -0.10 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.10
- 0.10 to 0.25
- 0.25 to 0.50
- > 0.50
- Was Wet Now Dry
- Was Dry Now Wet



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-15

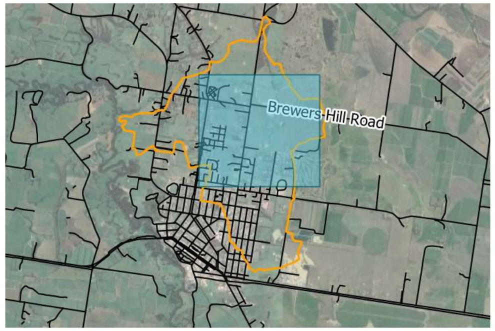
Retarding Basin
Existing Case
Peak Depth
20% AEP Event

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

solutions WATER MODELLING

Appendix A-16

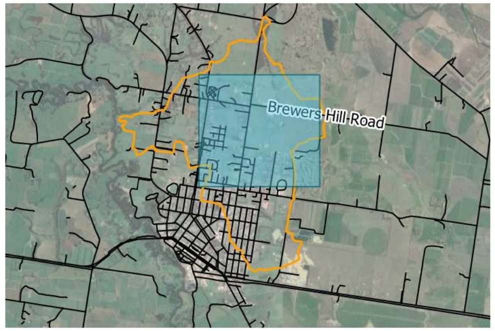
Retarding Basin
Developed Case
Peak Depth
20% AEP Event

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0

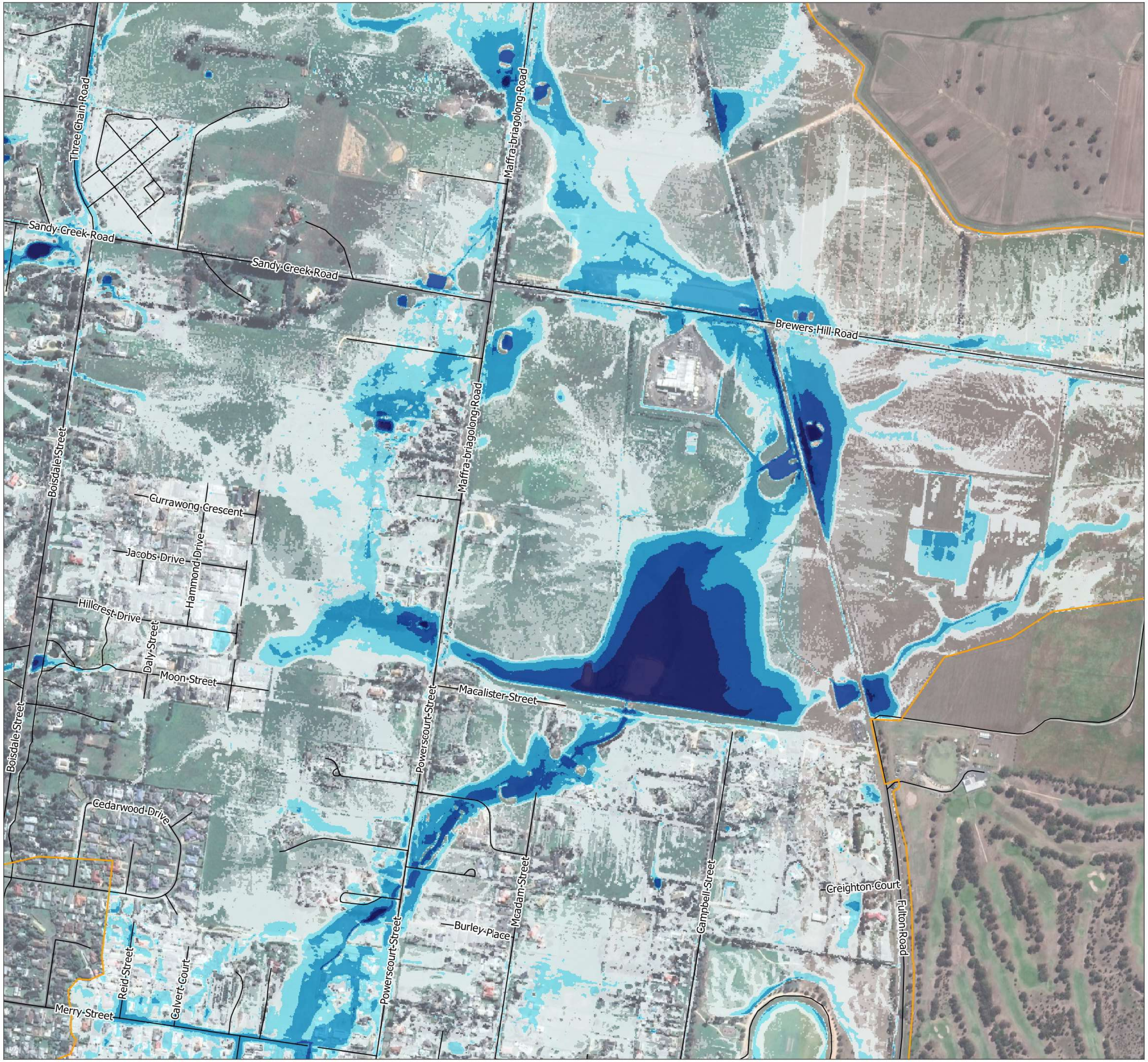


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



solutions
WATER MODELLING

Appendix A-17

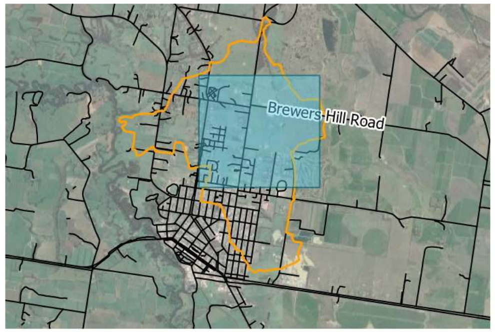
Retarding Basin
Existing Case
Peak Depth
1% AEP Event

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

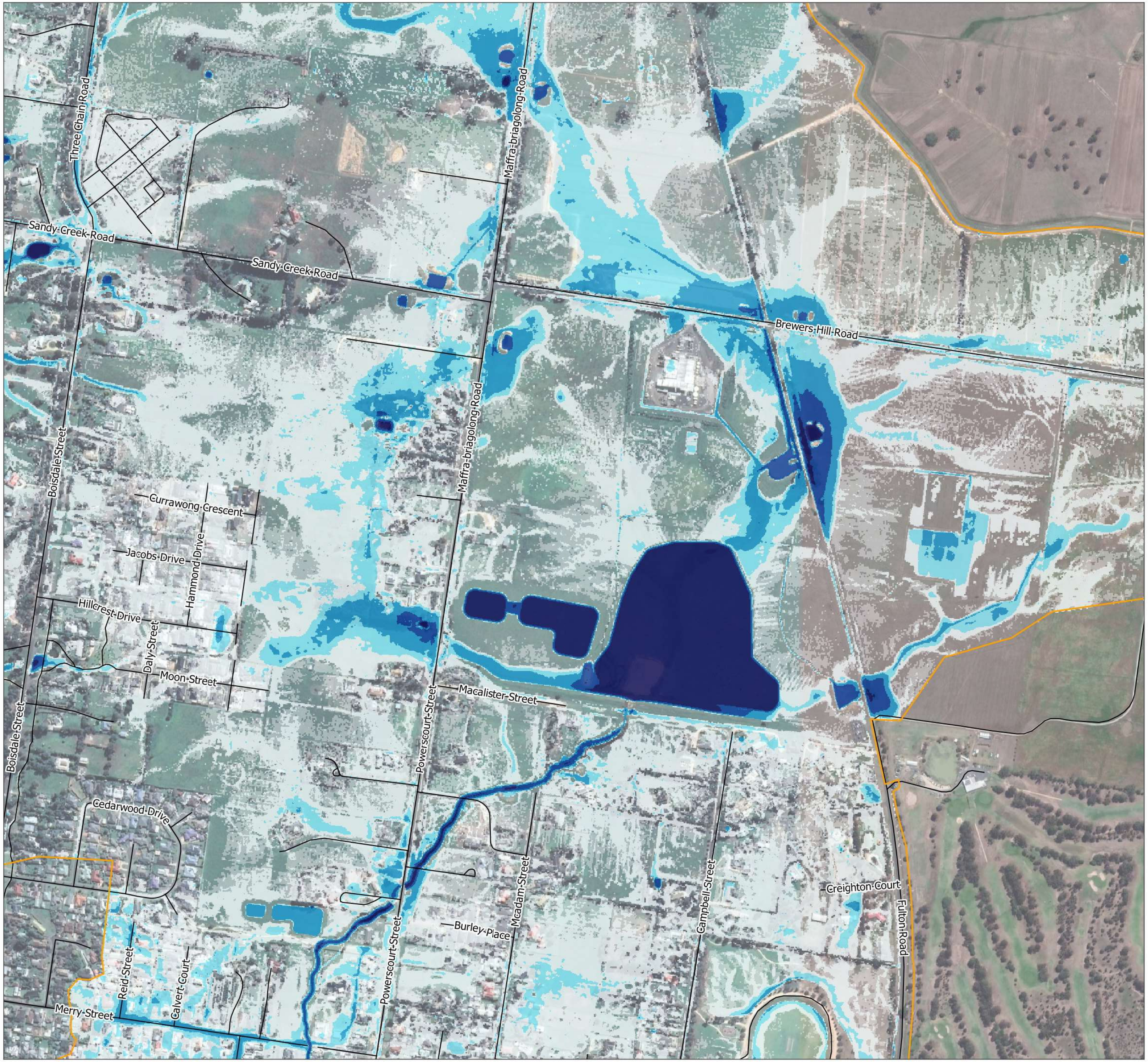
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

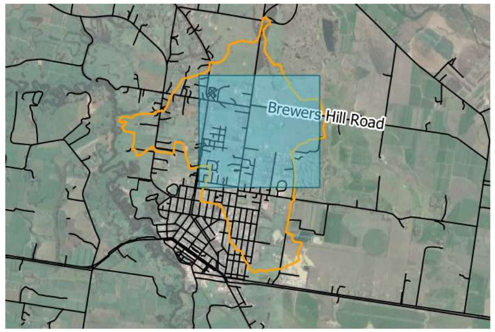
Appendix A-18

Retarding Basin
Developed Case
Peak Depth
1% AEP Event



LEGEND

- Model Extent
- Depth (m)
  - <= 0.02
  - 0.02 - 0.1
  - 0.1 - 0.2
  - 0.2 - 0.5
  - 0.5 - 1.0
  - >1.0



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
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0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-19

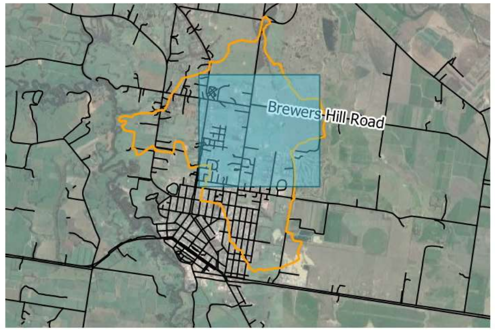
Retarding Basin
Existing Case
Peak Velocity
20% AEP Event

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

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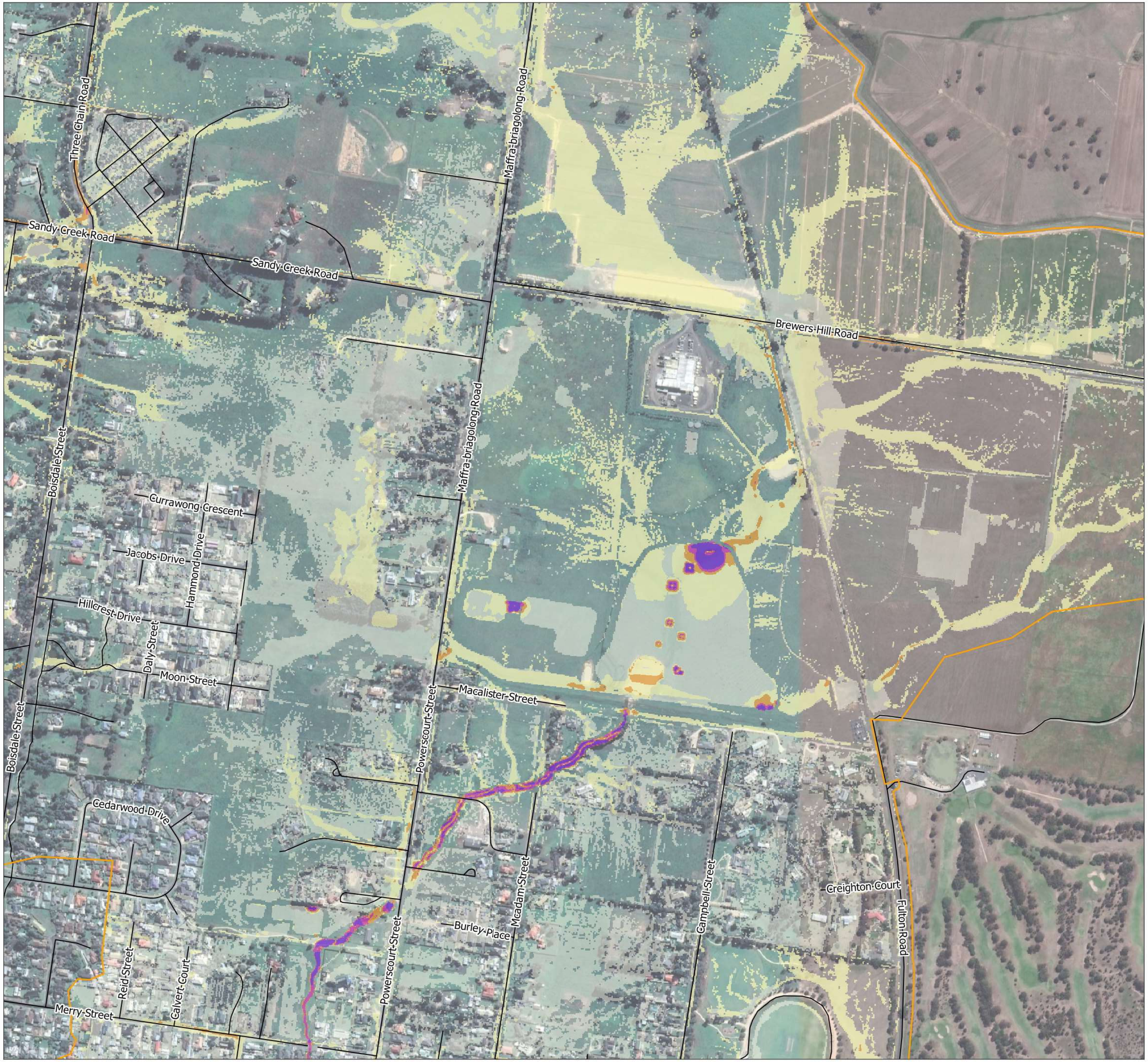
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-20

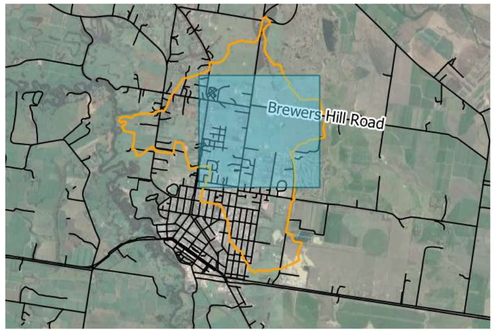
Retarding Basin
Developed Case
Peak Velocity
20% AEP Event

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

solutions
WATER MODELLING

Appendix A-21

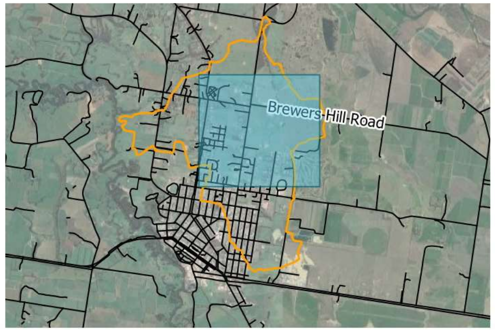
Retarding Basin
Existing Case
Peak Velocity
1% AEP Event

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

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0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-22

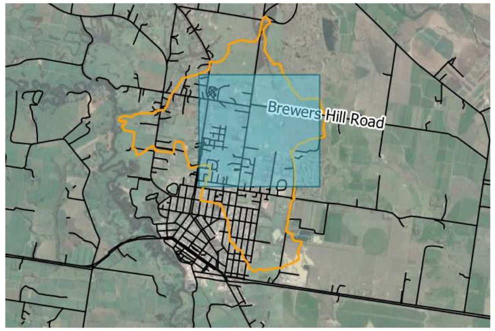
Retarding Basin
Developed Case
Peak Velocity
1% AEP Event

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

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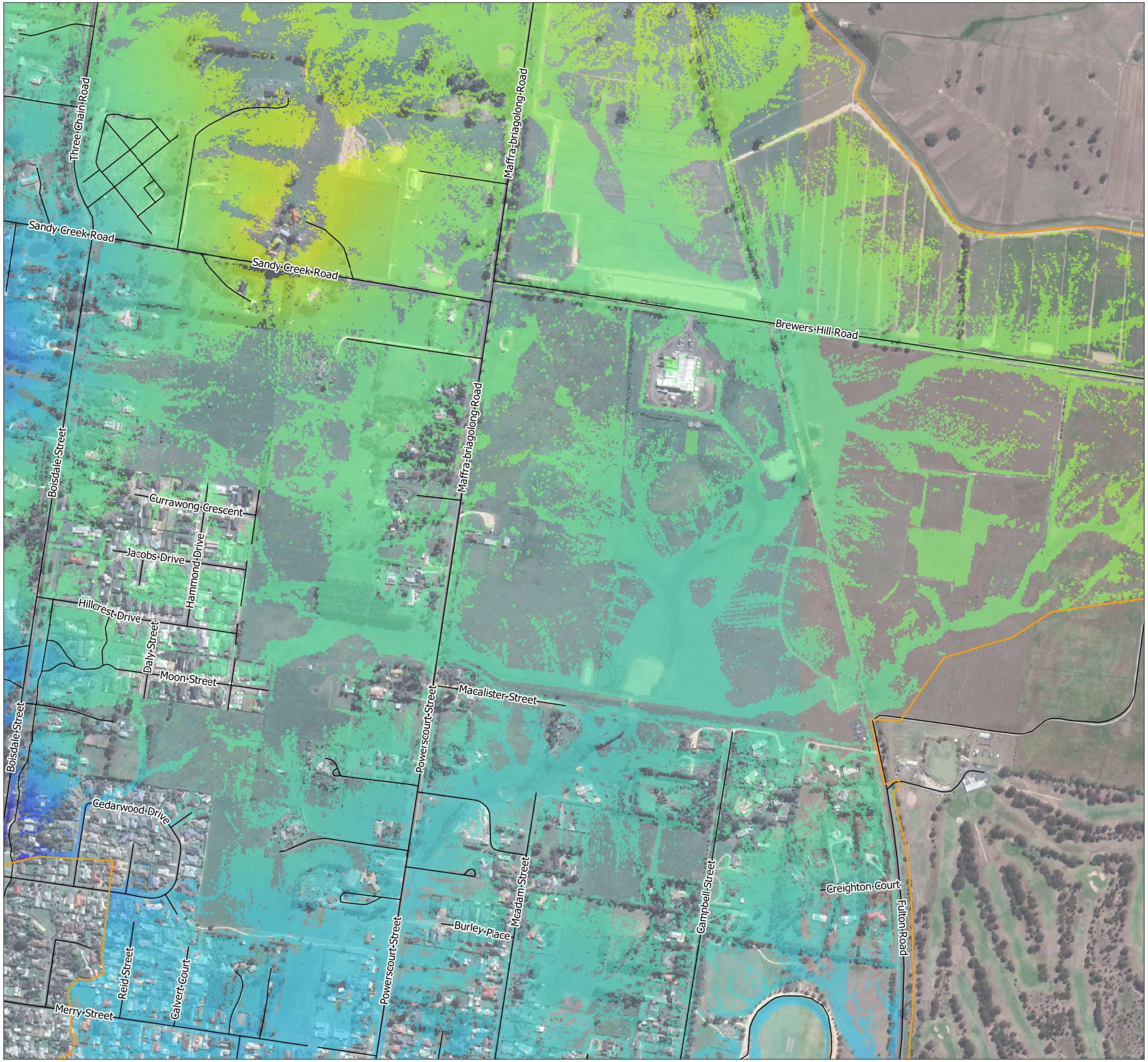
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-23

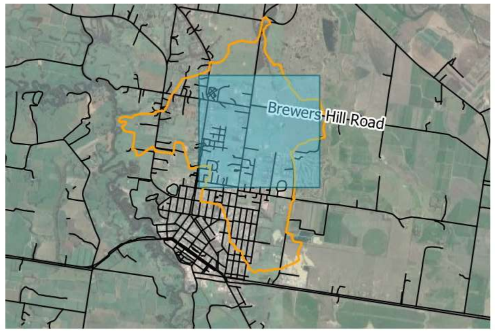
Retarding Basin
Existing Case
Peak Water Surface Level
20% AEP Event

LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70



Maffra Drainage Integrated Water Management Strategy

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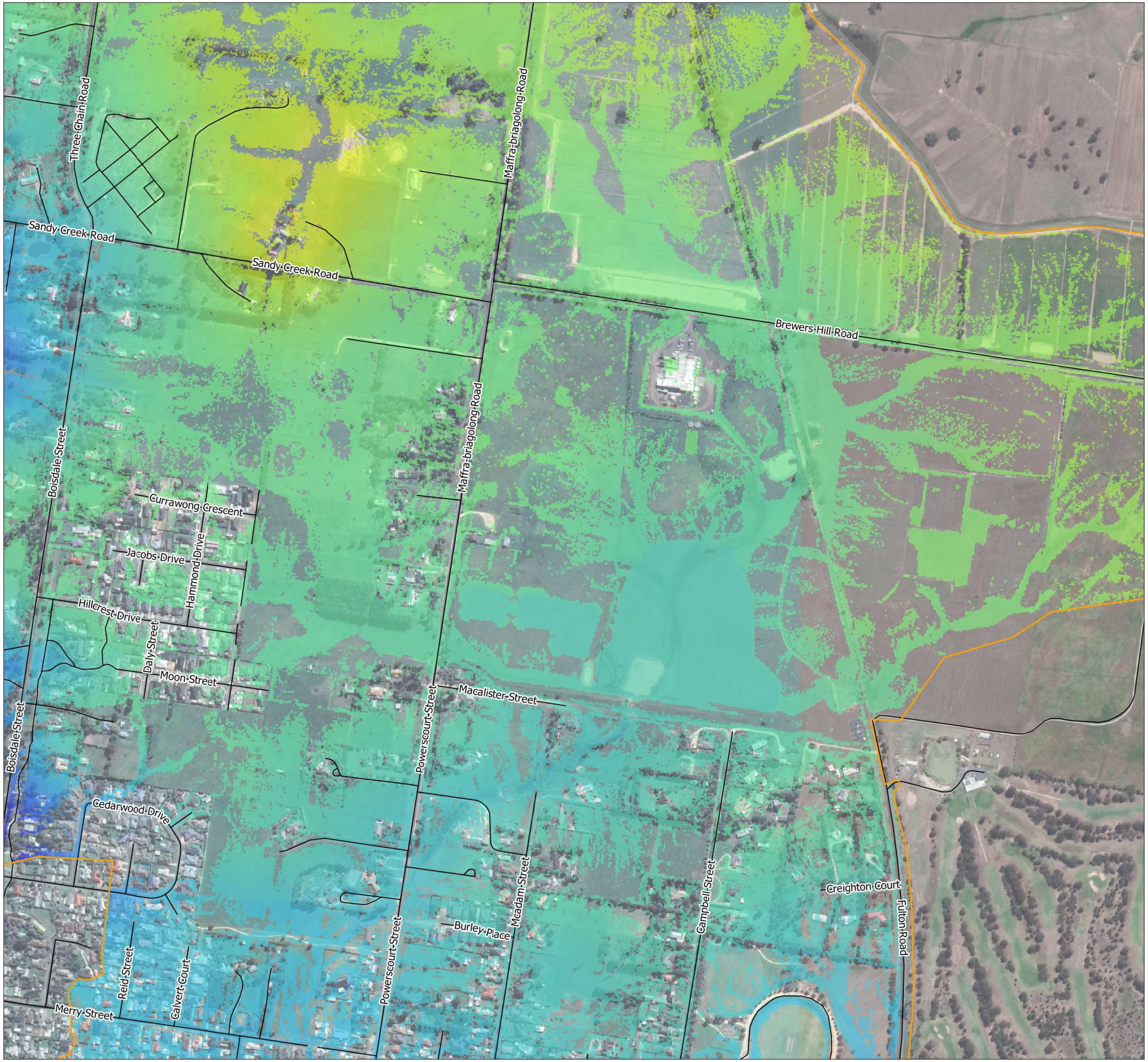
0 100 200 300 400 500 m

solutions WATER MODELLING

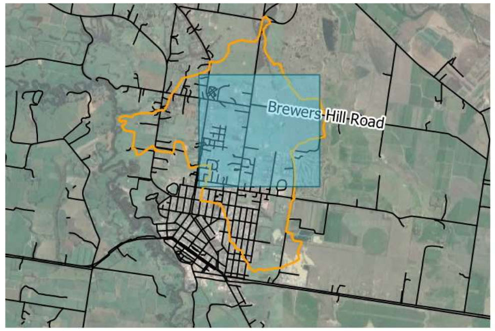
Appendix A-24

Retarding Basin
Developed Case
Peak Water Surface Level
20% AEP Event



LEGEND

- Model Extent
- Water Level (mAHD)
  - 20
  - 30
  - 40
  - 50
  - 60
  - 70



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
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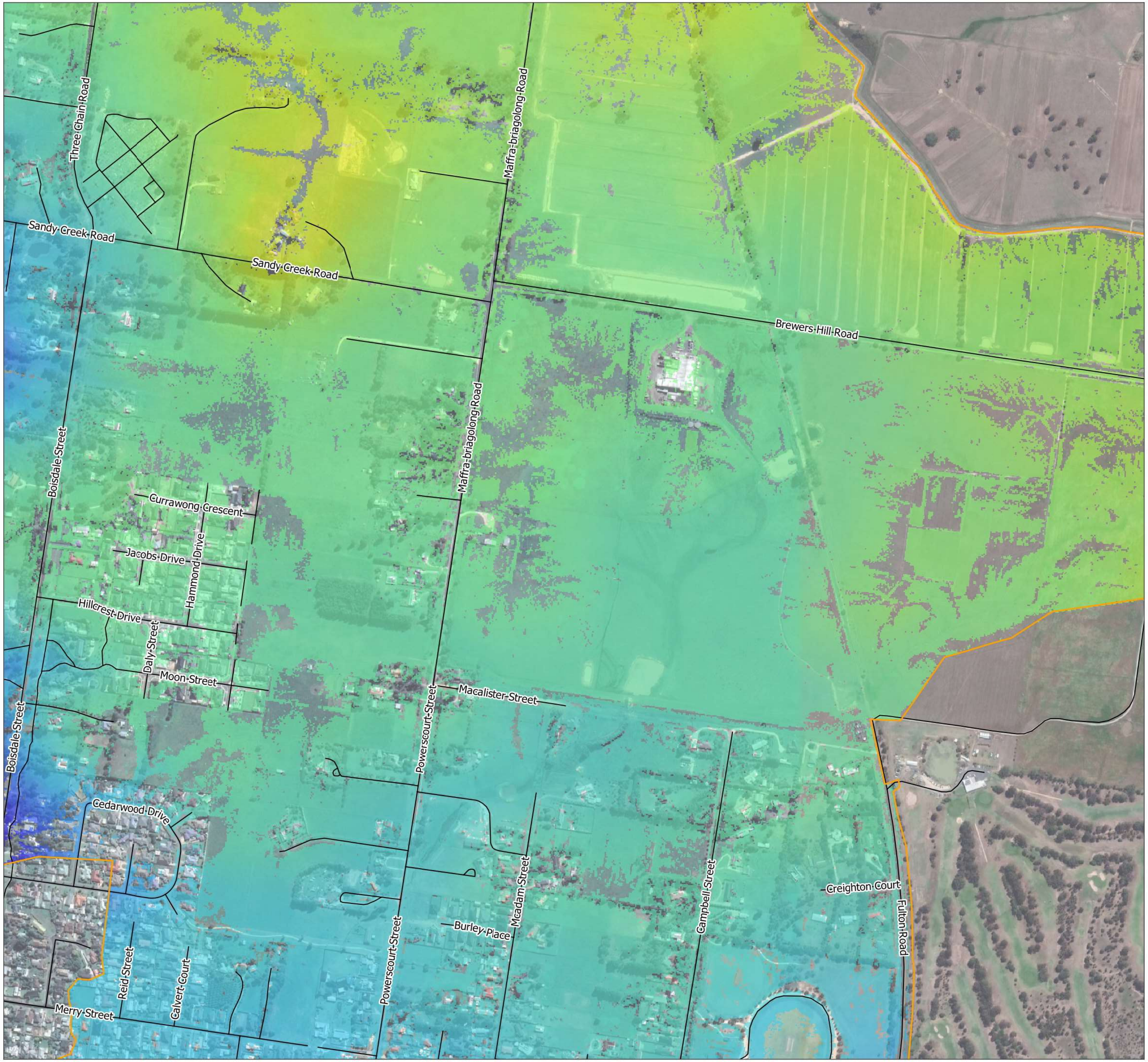
0 100 200 300 400 500 m

solutions WATER MODELLING

Appendix A-25

Retarding Basin
Existing Case
Peak Water Surface Level
1% AEP Event

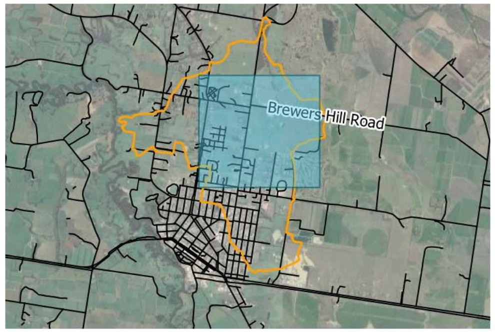


LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70



Maffra Drainage Integrated Water Management Strategy

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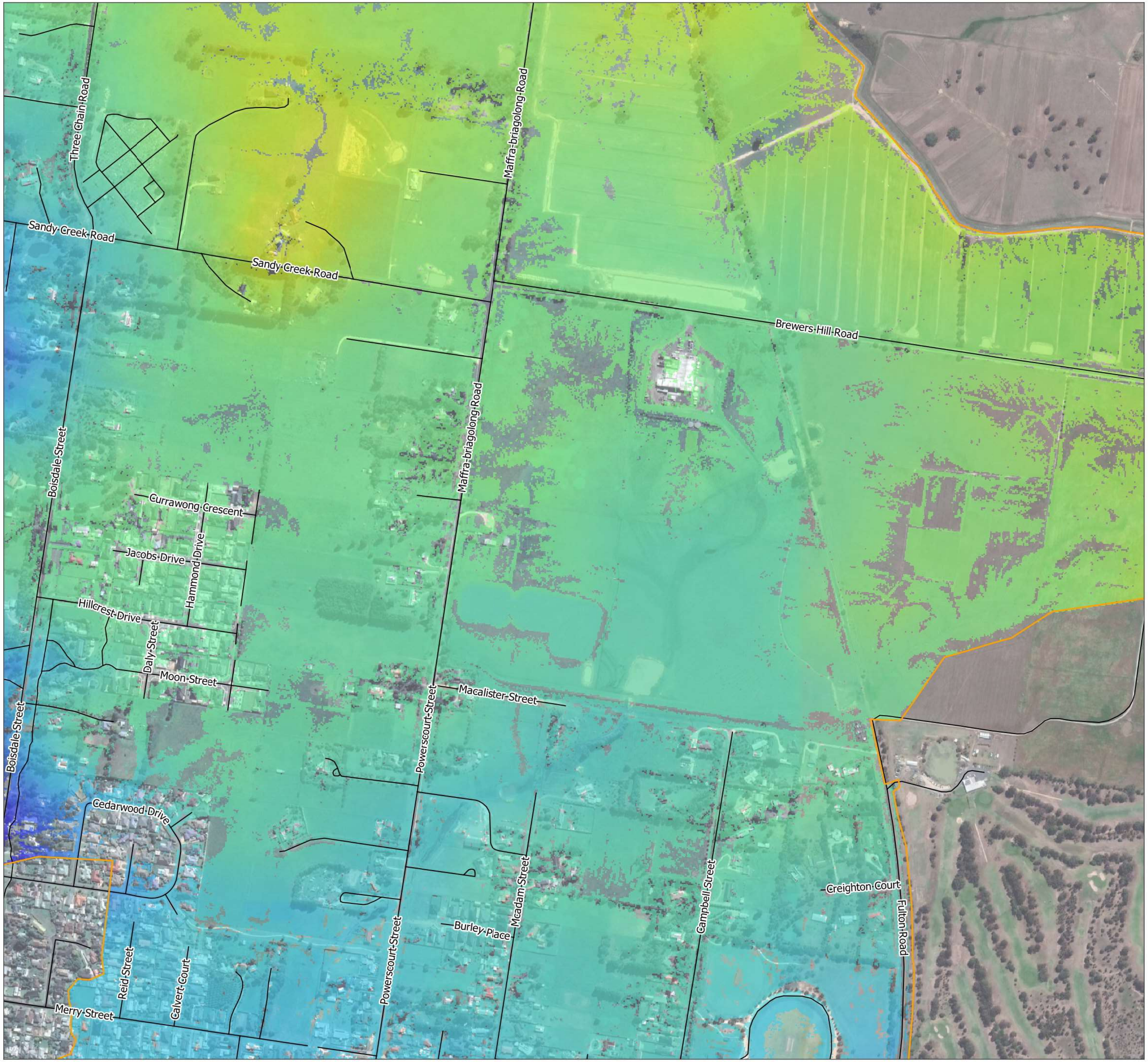
0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-26

Retarding Basin
Developed Case
Peak Water Surface Level
1% AEP Event

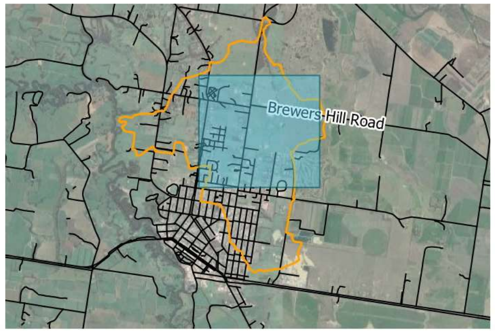


LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70



Maffra Drainage Integrated Water Management Strategy

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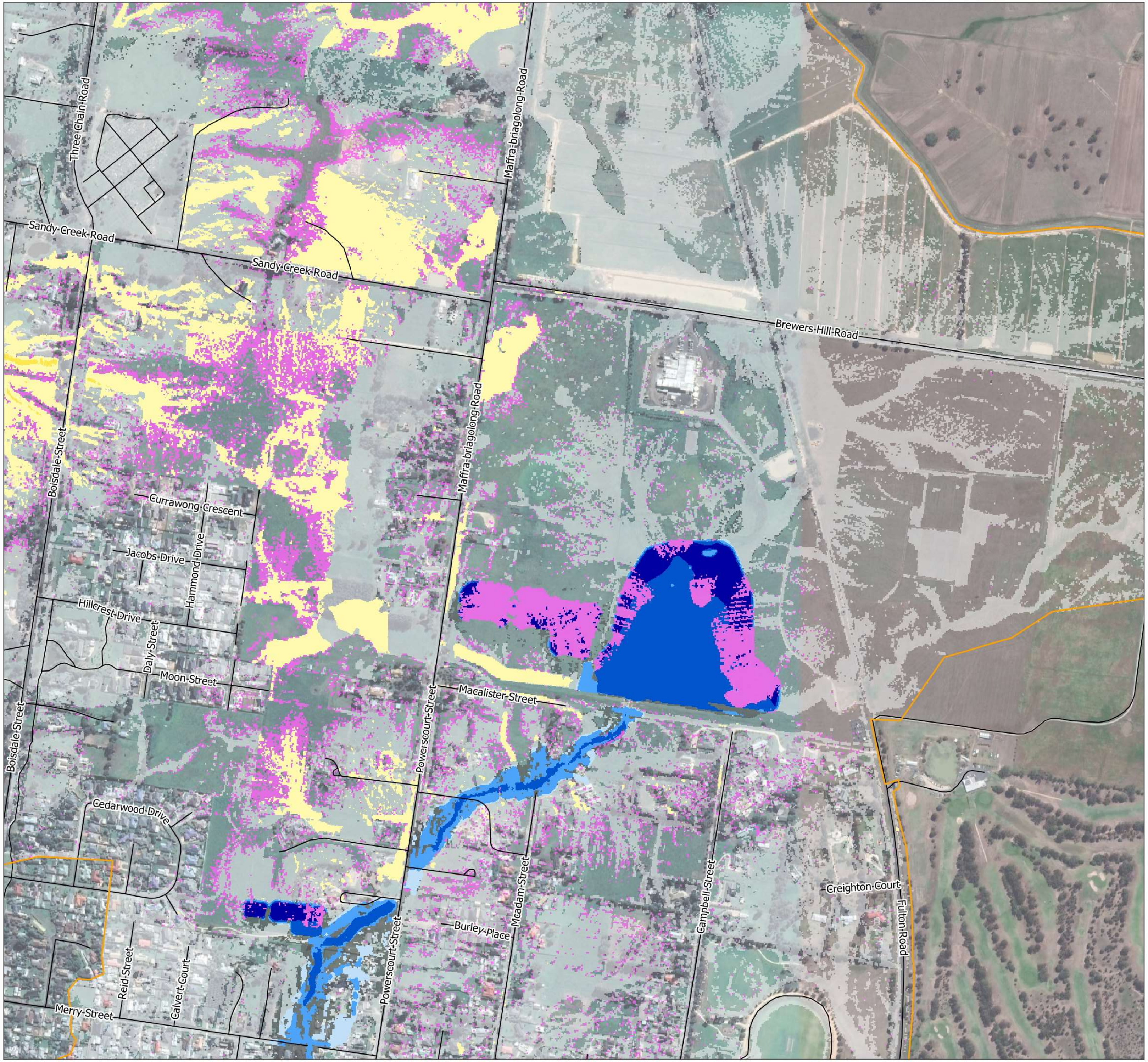
Date: 14/08/2020

0 100 200 300 400 500 m



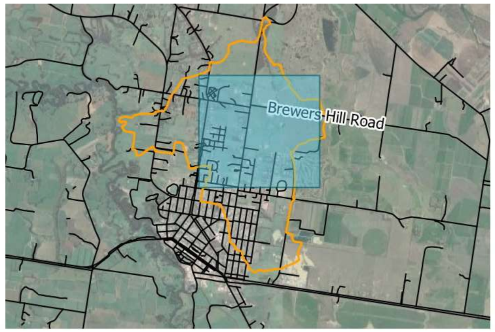
solutions WATER MODELLING

Appendix A-27

Retarding Basin
Developed Case
Afflux
20% AEP Event

LEGEND

- Model Extent
- Change in Water Level (m)
- <= -1.0
- -1.0 to -0.50
- -0.50 to -0.10
- -0.10 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.10
- 0.10 to 0.25
- 0.25 to 0.50
- > 0.50
- Was Wet Now Dry
- Was Dry Now Wet



Maffra Drainage Integrated Water Management Strategy

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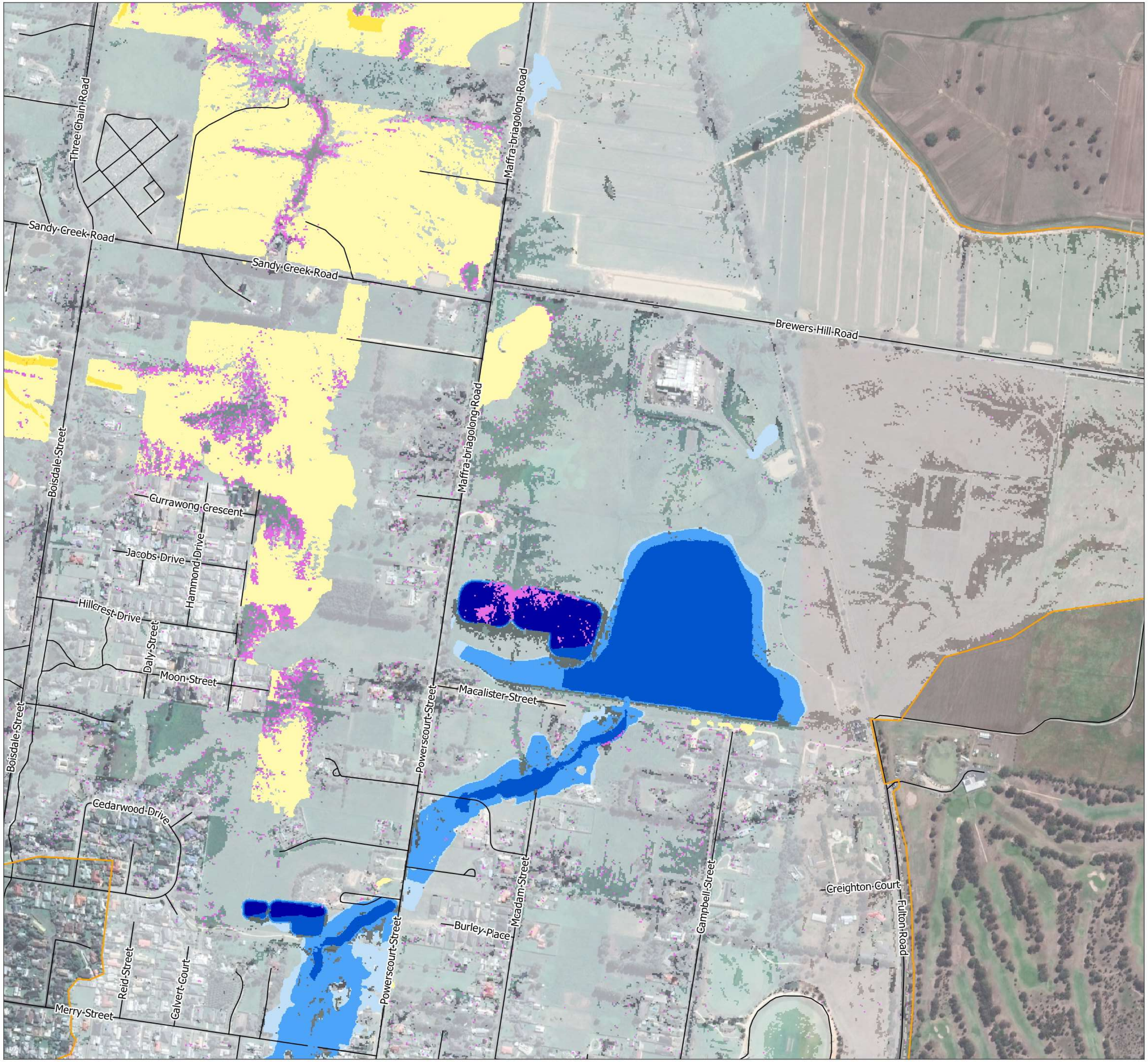
0 100 200 300 400 500 m



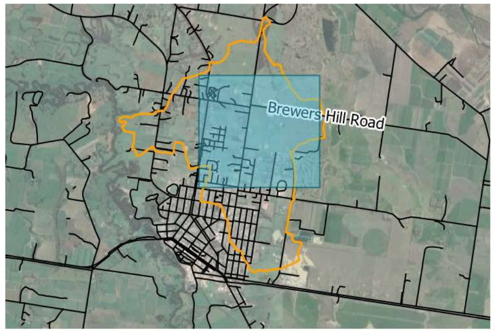
WATER MODELLING solutions

Appendix A-28

Retarding Basin
Developed Case
Afflux
1% AEP Event

LEGEND

	Model Extent
Change in Water Level (m)	
	<= -1.0
	-1.0 to -0.50
	-0.50 to -0.10
	-0.10 to -0.01
	-0.01 to 0.01
	0.01 to 0.05
	0.05 to 0.10
	0.10 to 0.25
	0.25 to 0.50
	> 0.50
	Was Wet Now Dry
	Was Dry Now Wet



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

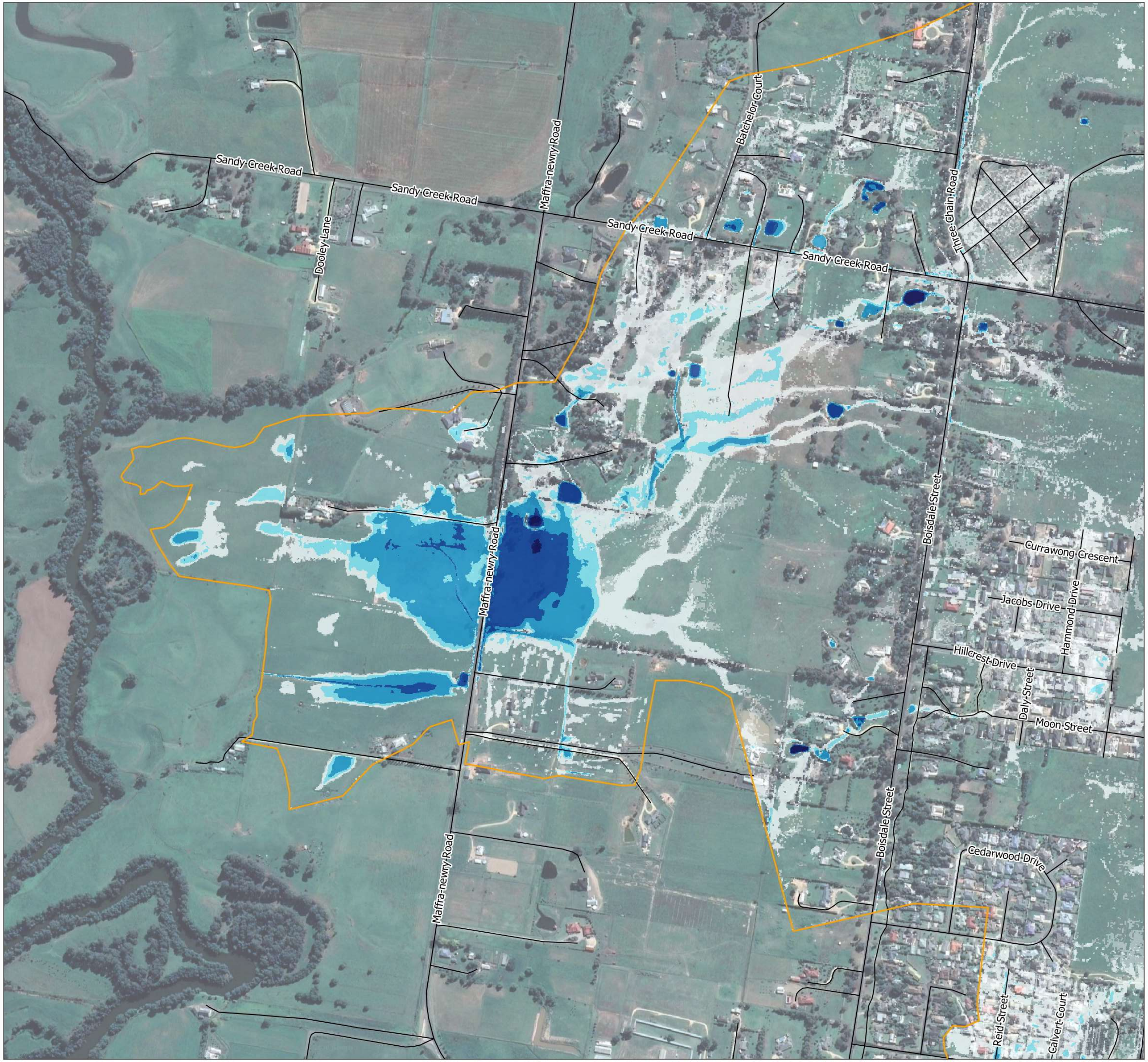
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-29

Western Catchment
Existing Case
Peak Depth
20% AEP Event

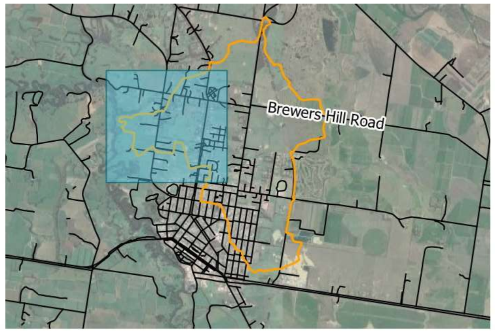


LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

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GDA 1994 / MGA Zone 55

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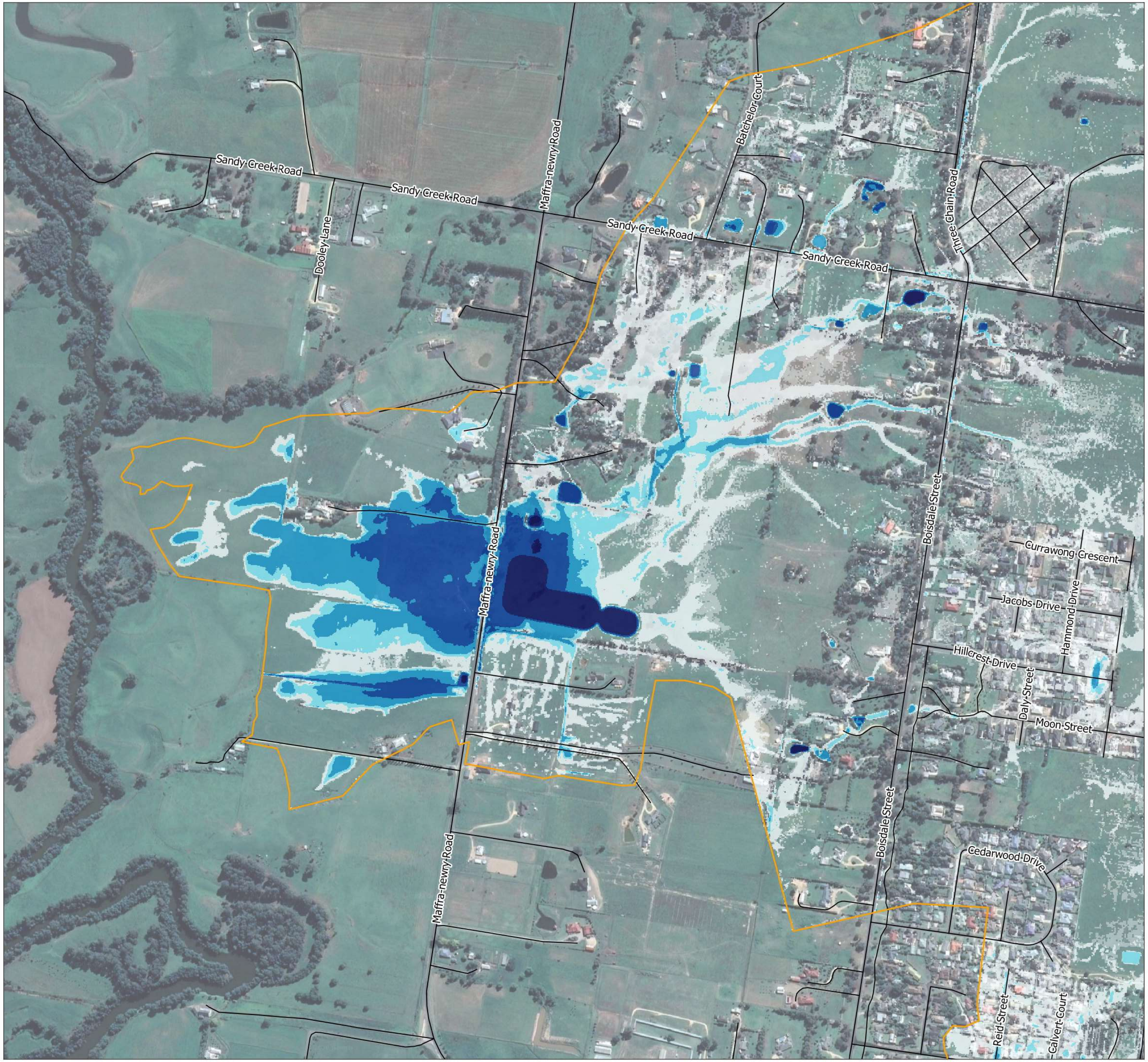
0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-30

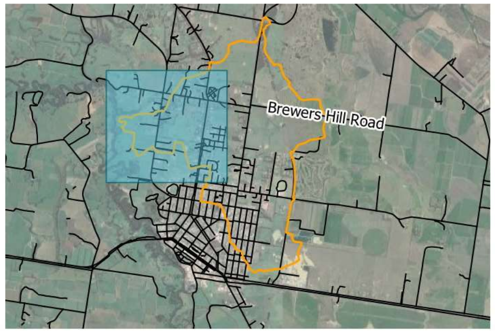
Western Catchment
Developed Case
Peak Depth
20% AEP Event

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

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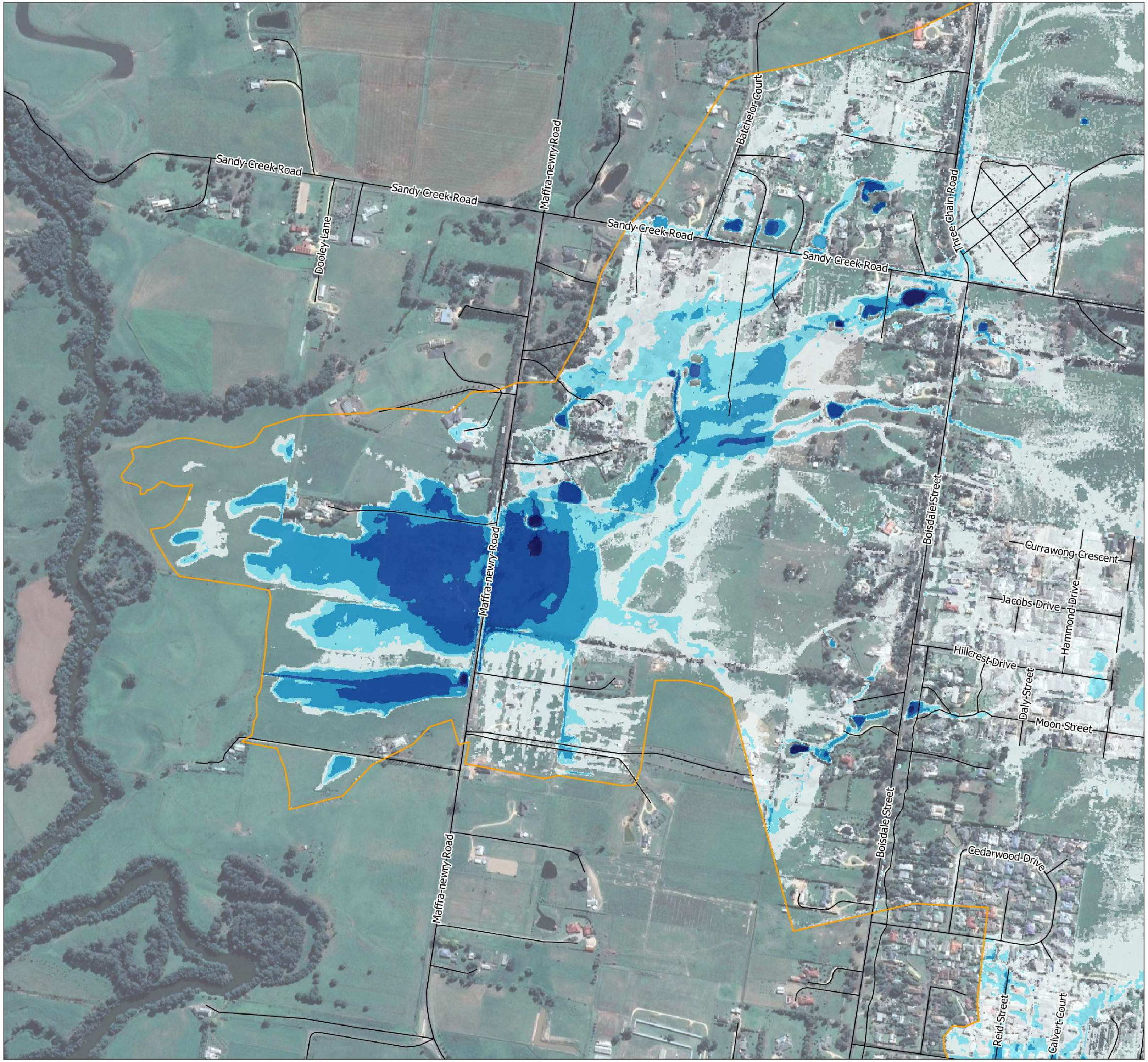
0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-31

Western Catchment
Existing Case
Peak Depth
1% AEP Event

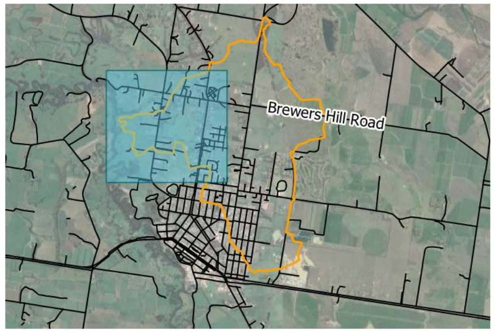


LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

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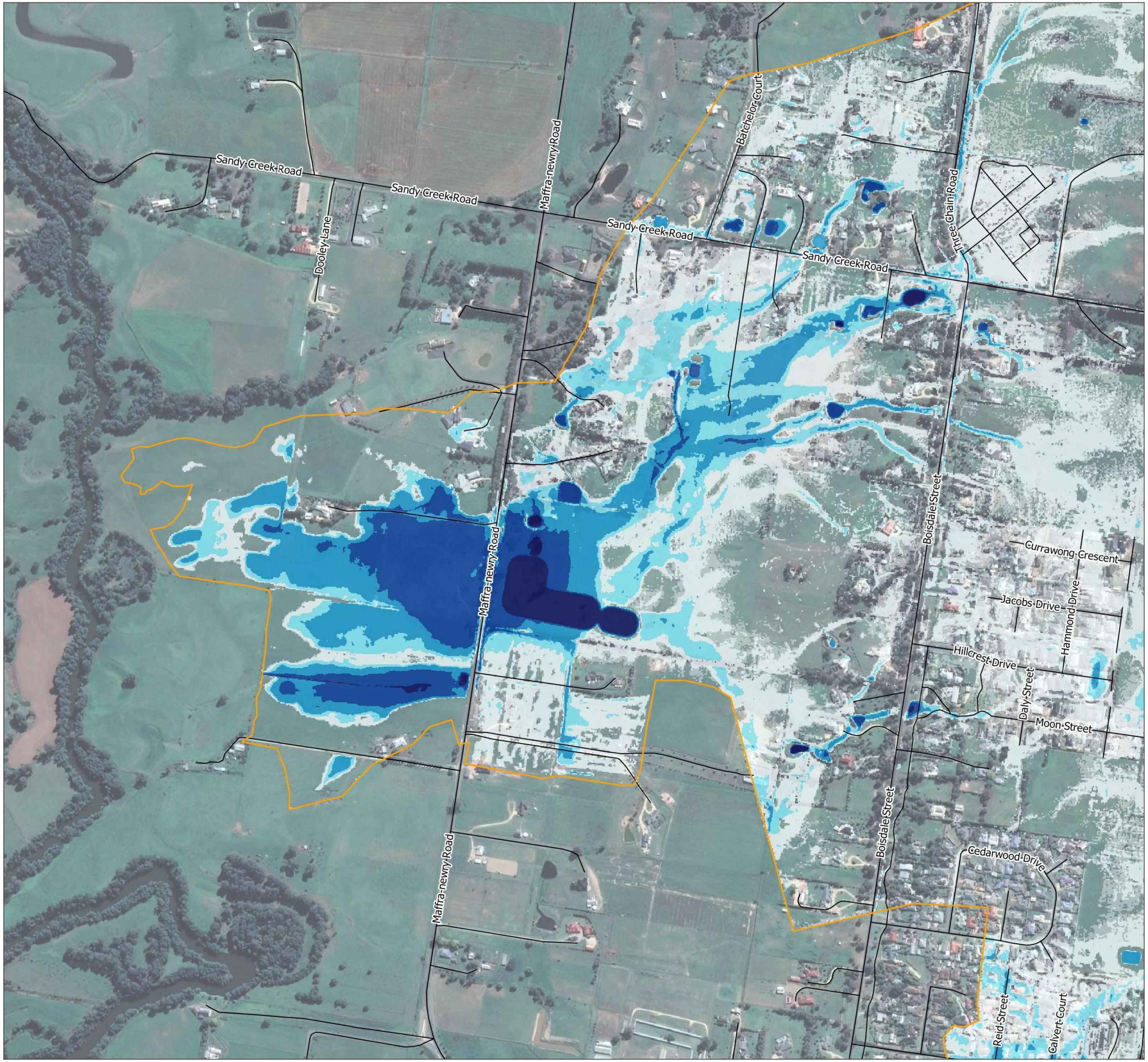
0 100 200 300 400 500 m

solutions
WATER MODELLING

Appendix A-32

Western Catchment
Developed Case
Peak Depth
1% AEP Event

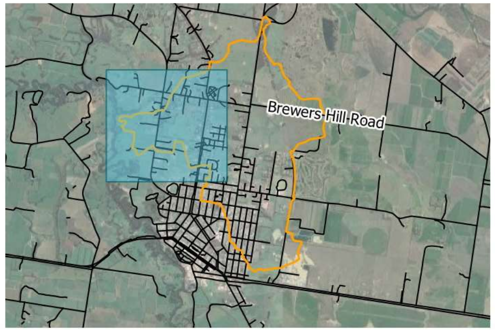


LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
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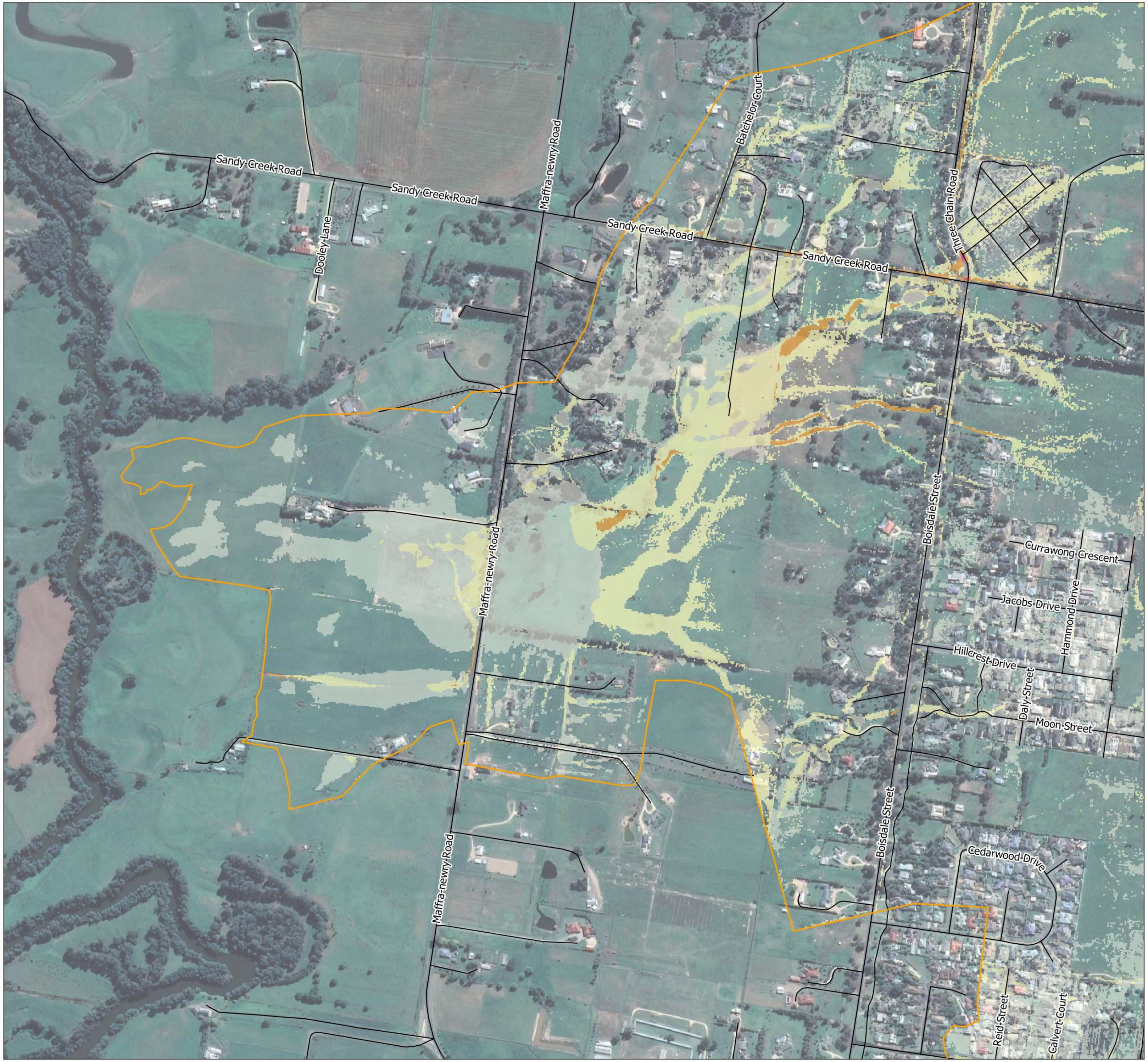
0 100 200 300 400 500 m



solutions WATER MODELLING

Appendix A-33

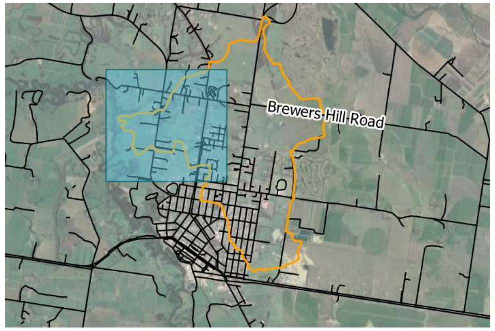
Western Catchment
Existing Case
Peak Velocity
20% AEP Event

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

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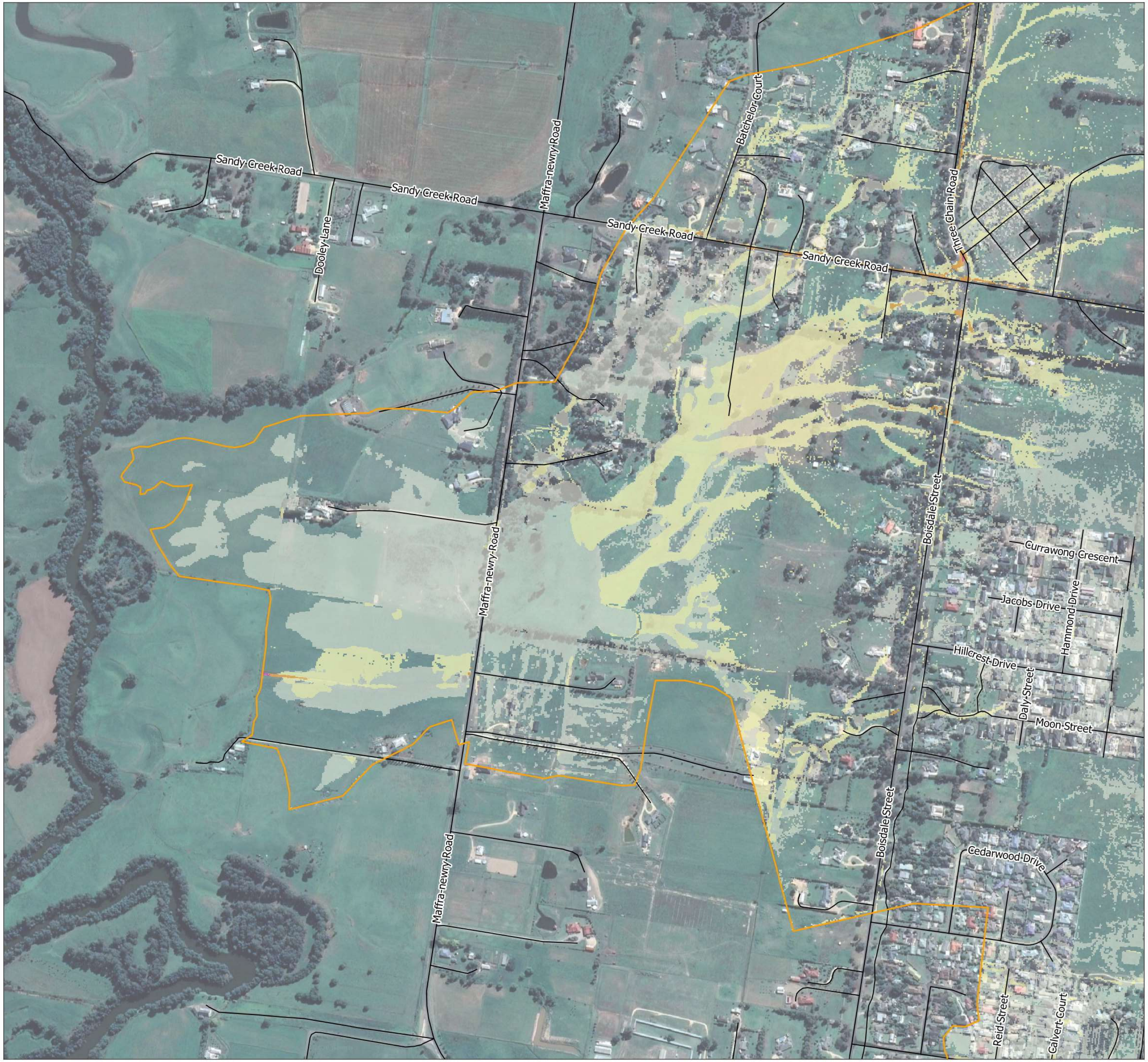
0 100 200 300 400 500 m

WATER MODELLING solutions

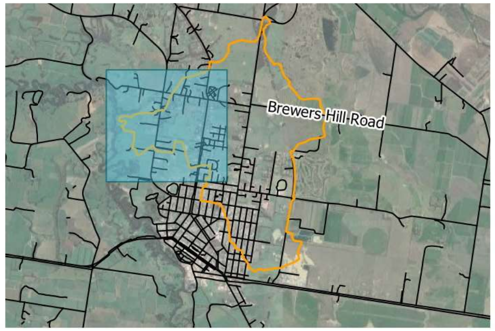
Appendix A-34

Western Catchment
Developed Case
Peak Velocity
20% AEP Event



LEGEND

- Model Extent
- Velocity (m/s)
  - <= 0.25
  - 0.25 - 0.50
  - 0.5 - 1.00
  - 1.00 - 1.50
  - >1.5



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

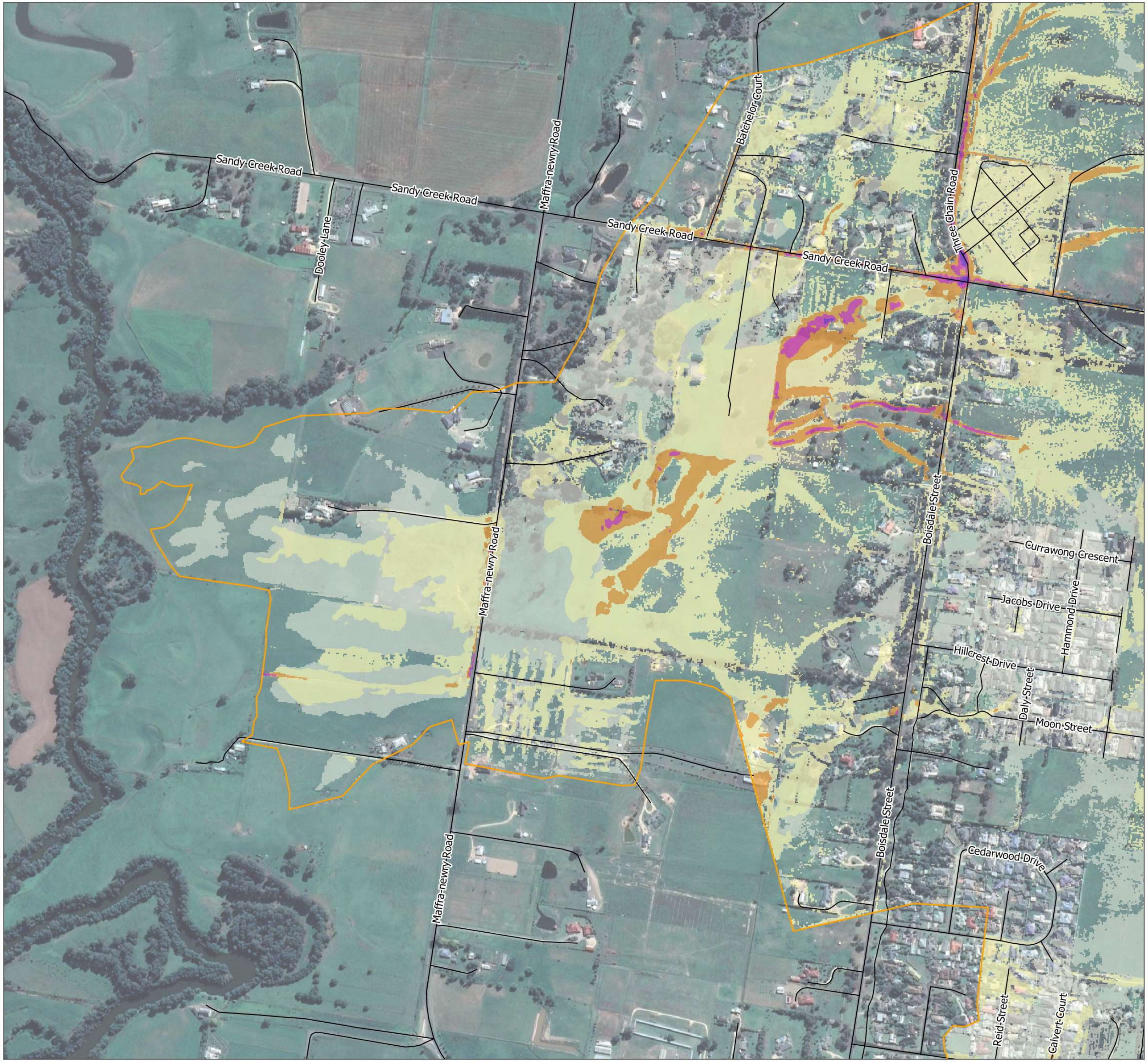
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

solutions
WATER MODELLING

Appendix A-35

Western Catchment
Existing Case
Peak Velocity
1% AEP Event

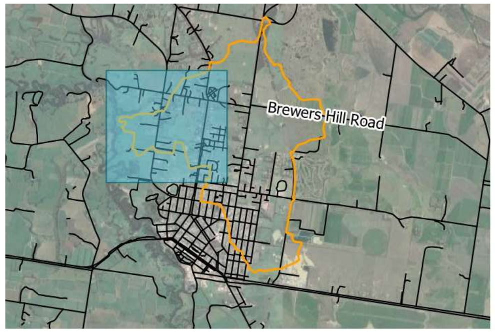


LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5

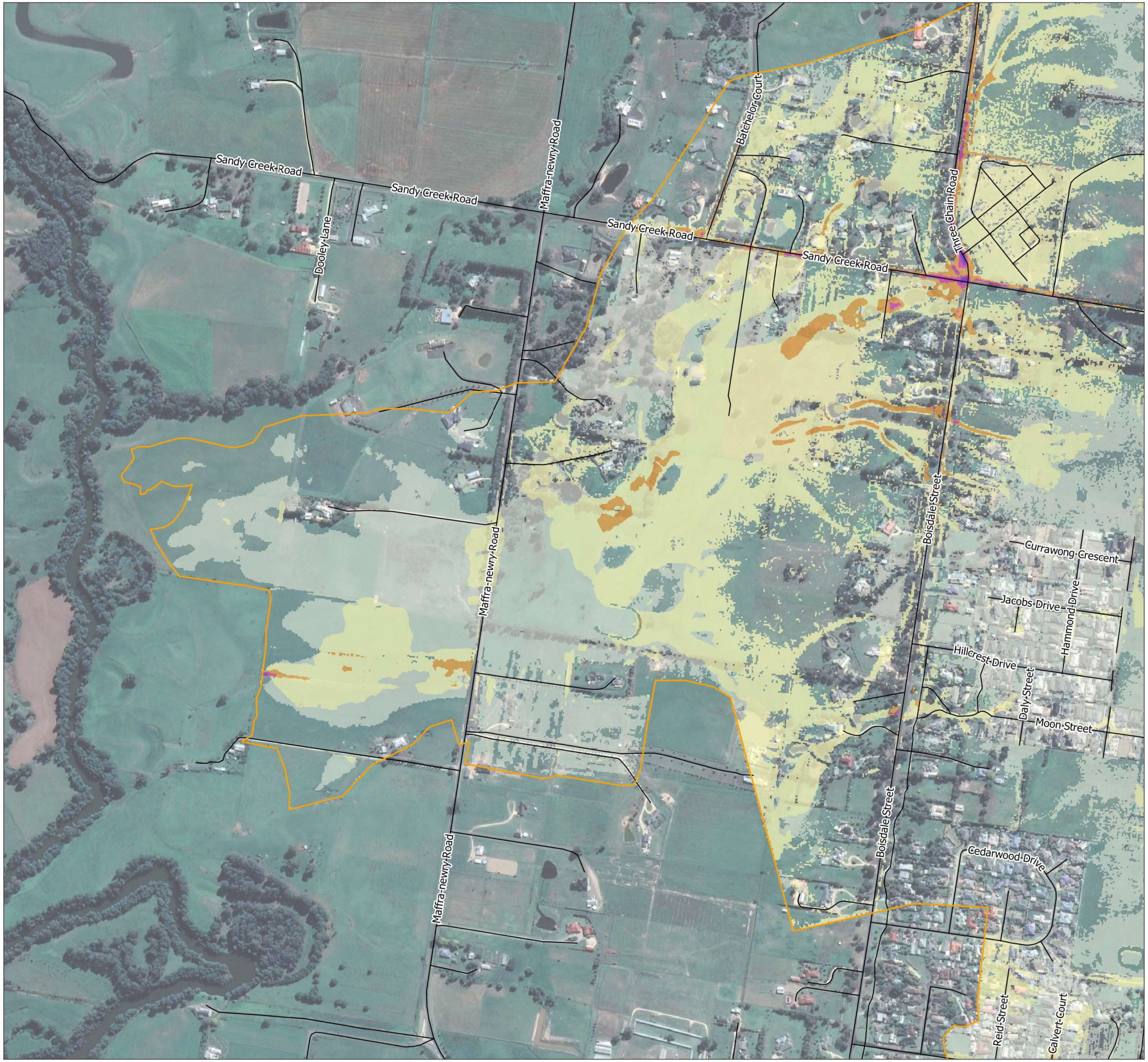


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-36

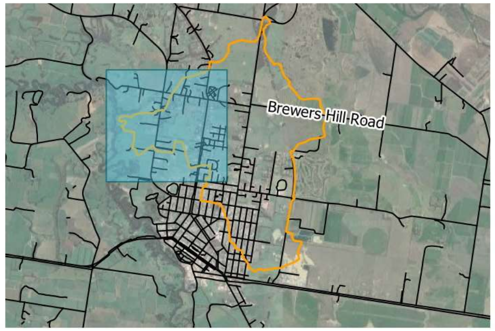
Western Catchment
Developed Case
Peak Velocity
1% AEP Event

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

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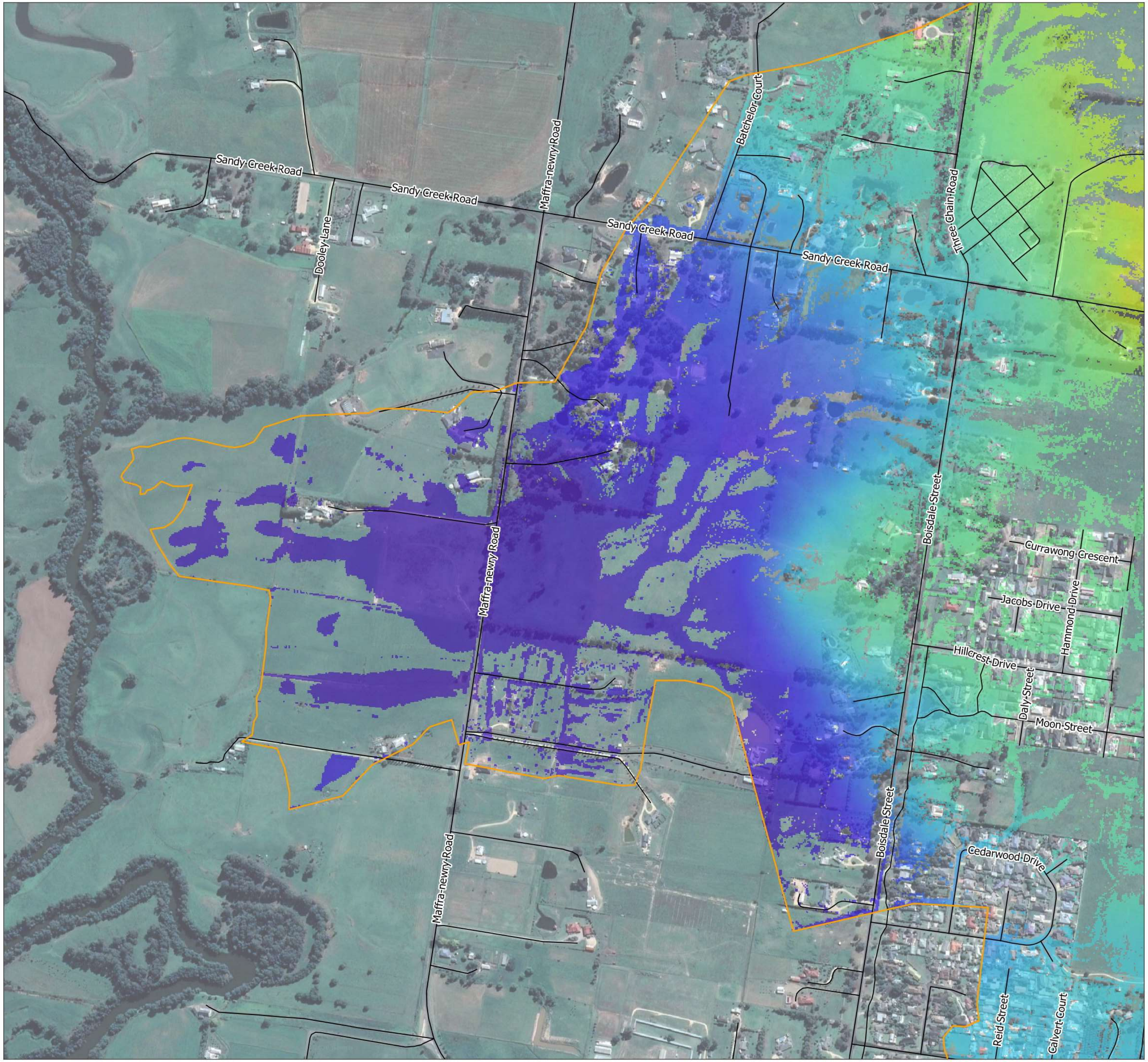
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-37

Western Catchment
Existing Case
Peak Water Surface Level
20% AEP Event

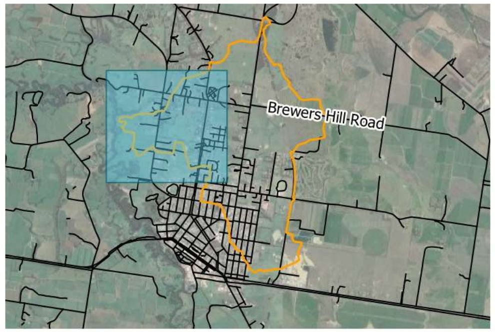


LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
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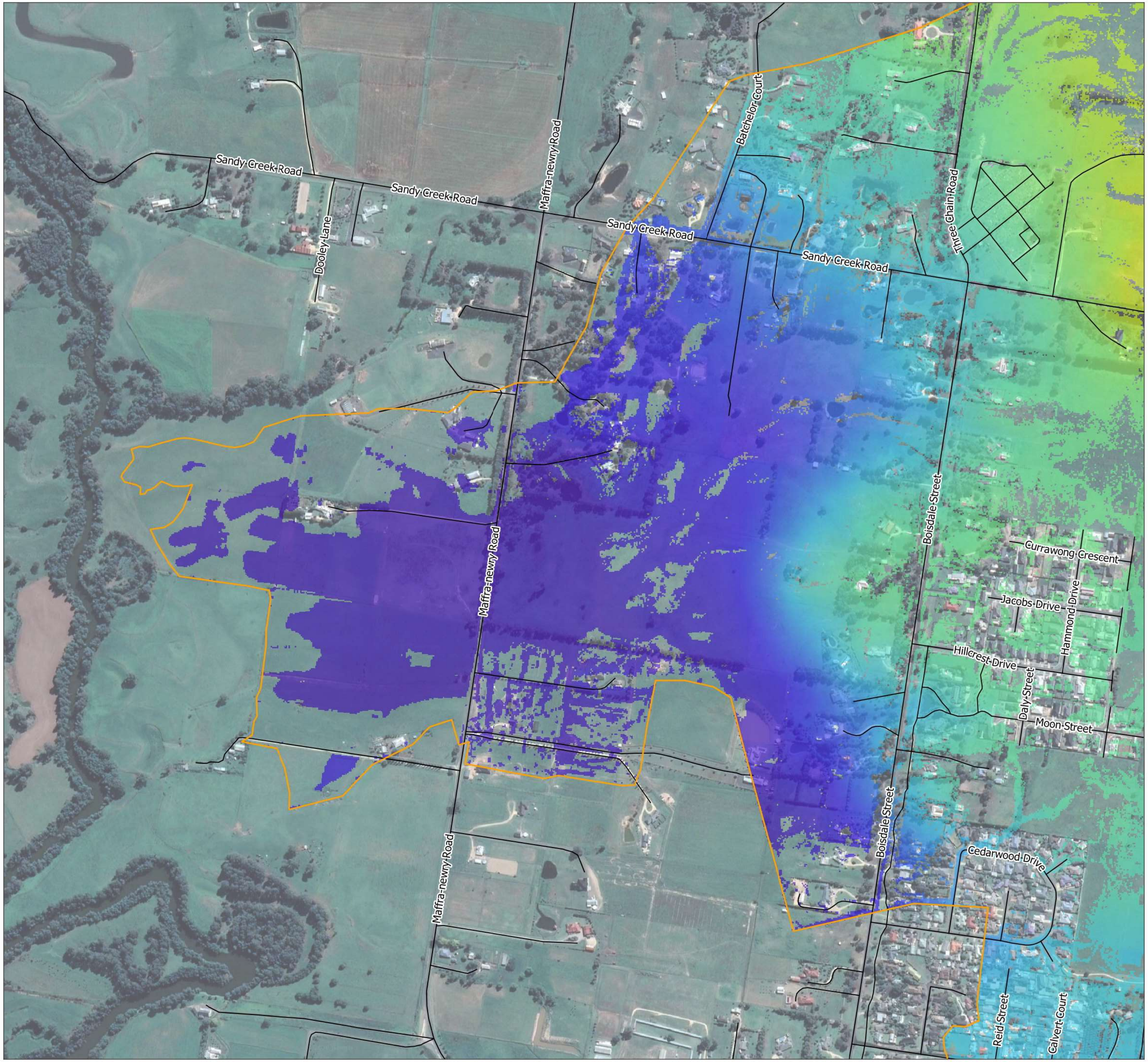
0 100 200 300 400 500 m



solutions
WATER MODELLING

Appendix A-38

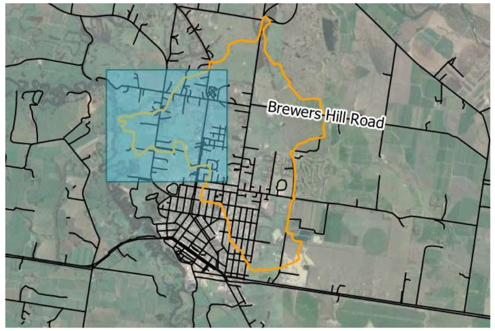
Western Catchment
Developed Case
Peak Water Surface Level
20% AEP Event

LEGEND

Model Extent

Water Level (mAHD)

20
30
40
50
60
70



Maffra Drainage Integrated Water Management Strategy

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GDA 1994 / MGA Zone 55

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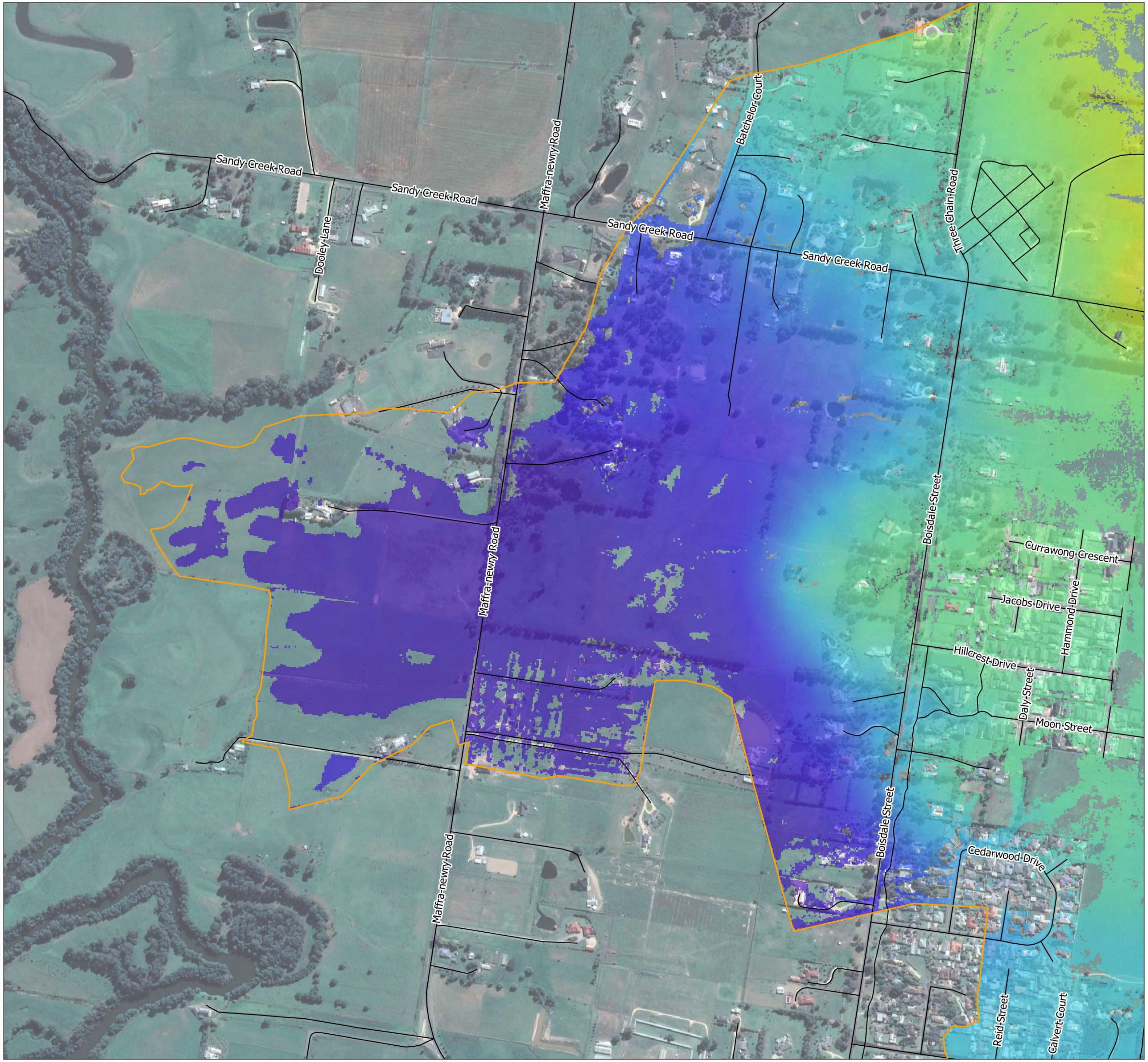
0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-39

Western Catchment
Existing Case
Peak Water Surface Level
1% AEP Event

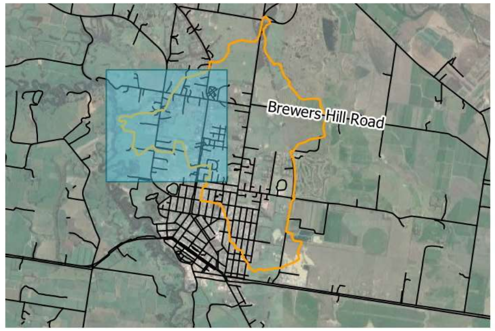


LEGEND

Model Extent

Water Level (mAHD)

20
30
40
50
60
70



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

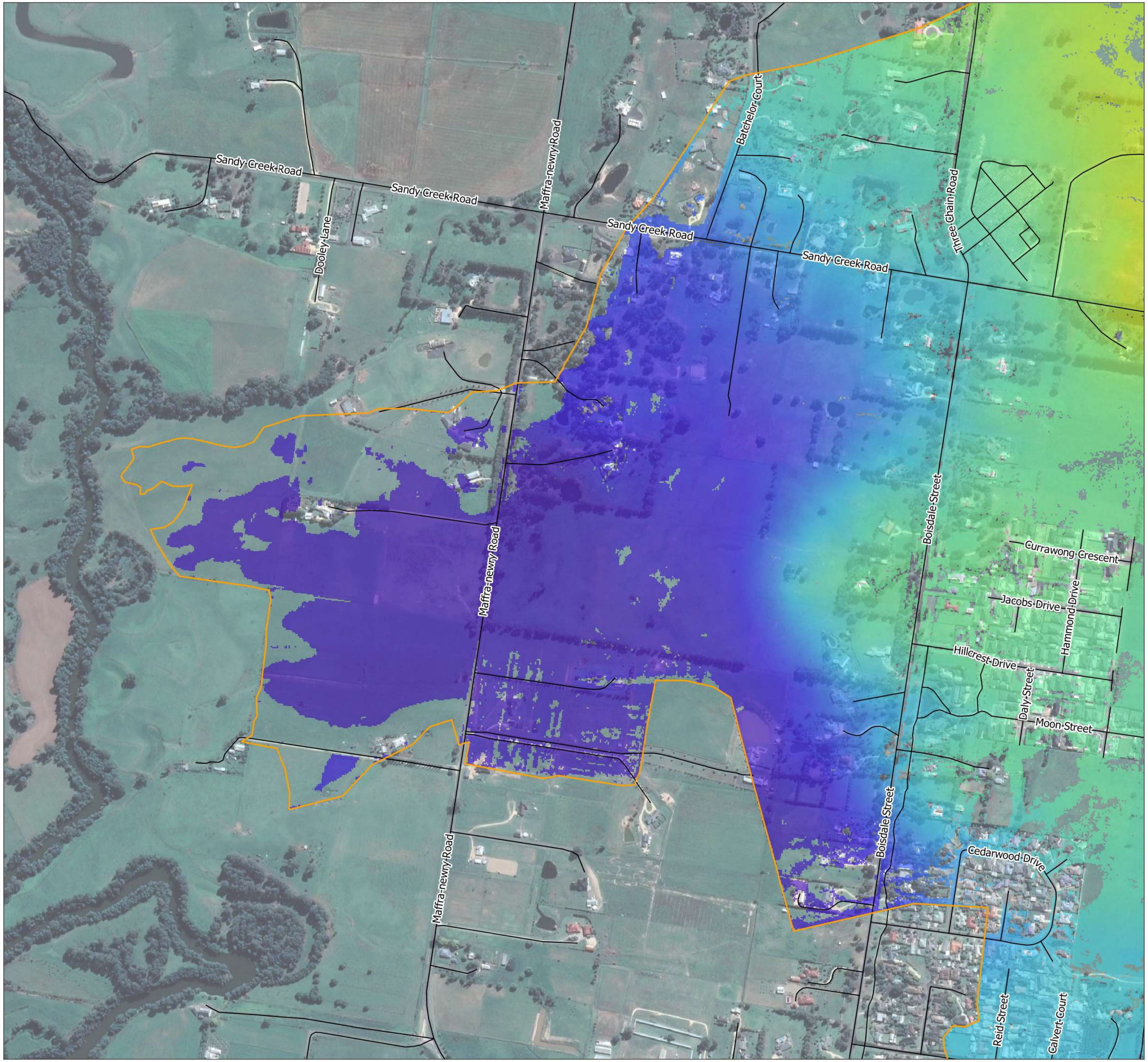
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-40

Western Catchment
Developed Case
Peak Water Surface Level
1% AEP Event

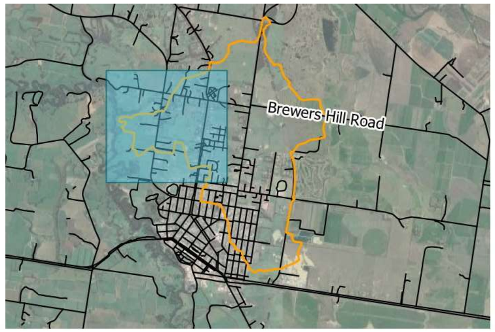


LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70

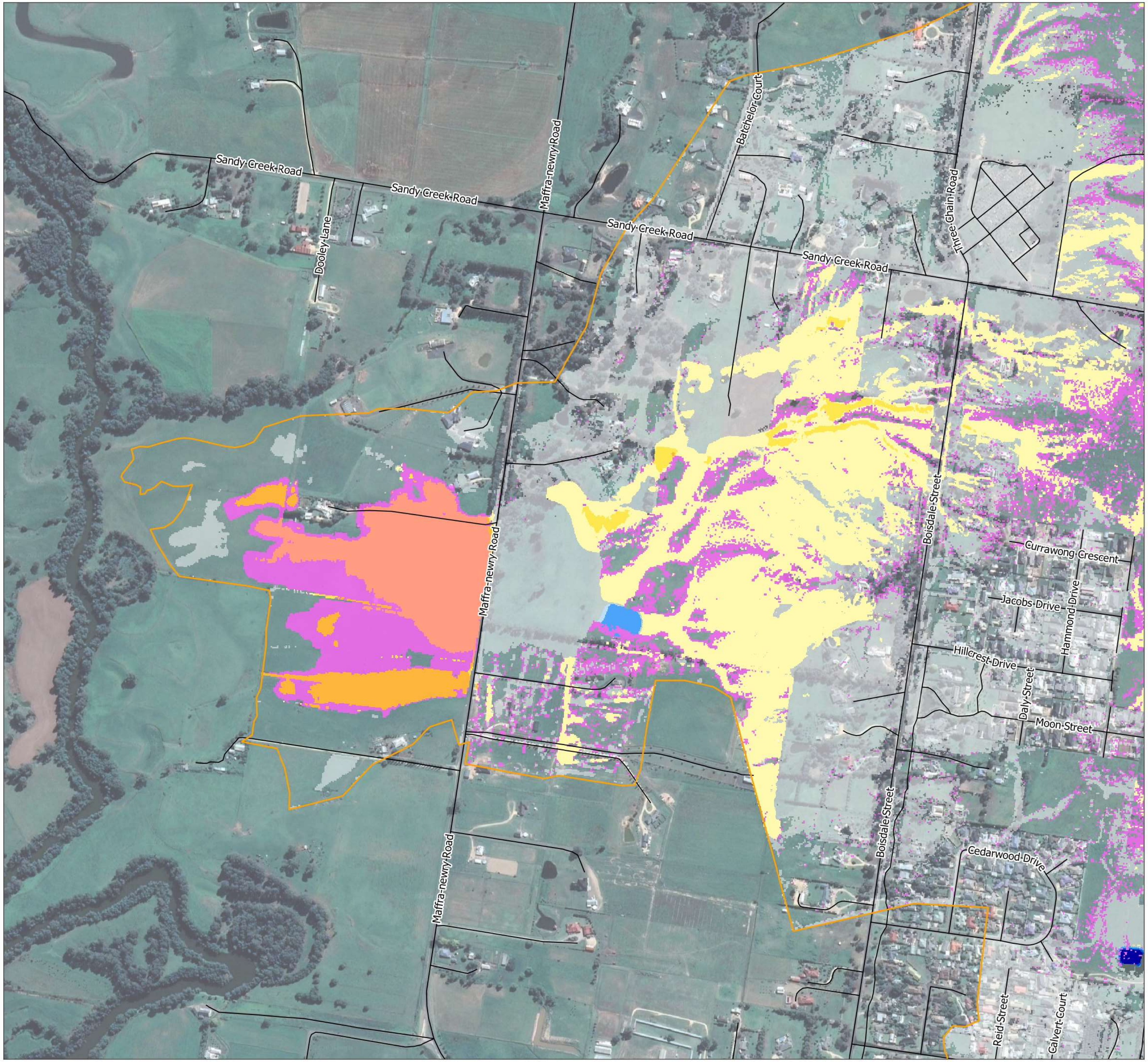


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-41

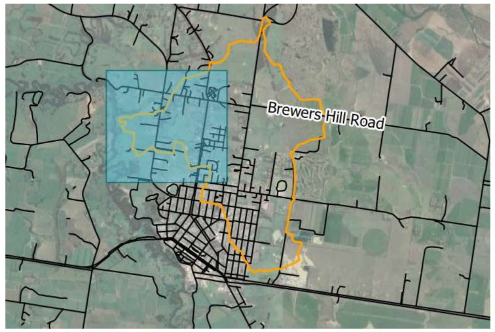
Western Catchment
Developed Case
Afflux
20% AEP Event

LEGEND

Model Extent

Change in Water Level (m)

- <= -1.0
- -1.0 to -0.50
- -0.50 to -0.10
- -0.10 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.10
- 0.10 to 0.25
- 0.25 to 0.50
- > 0.50
- Was Wet Now Dry
- Was Dry Now Wet



Maffra Drainage Integrated Water Management Strategy

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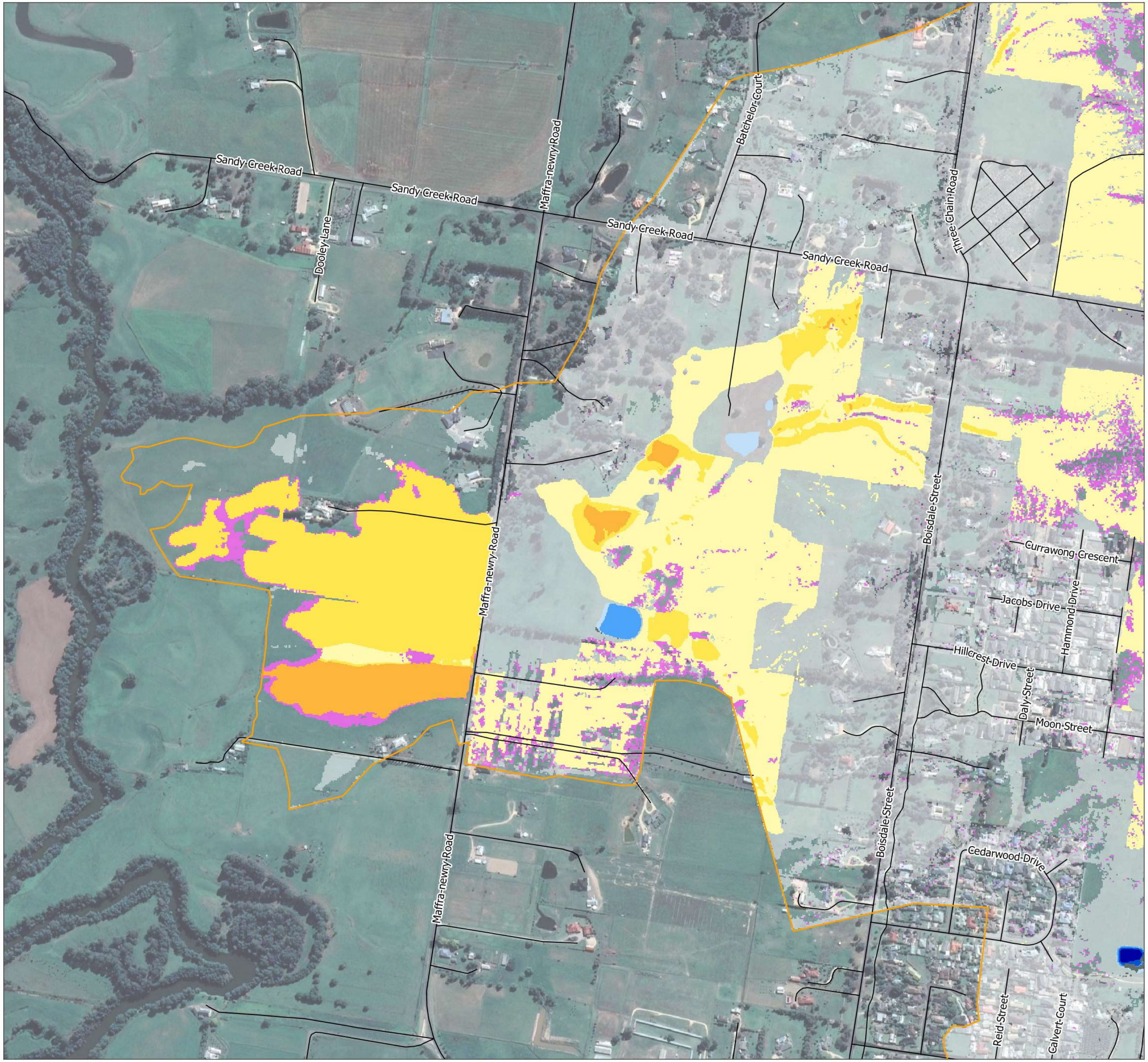
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-42

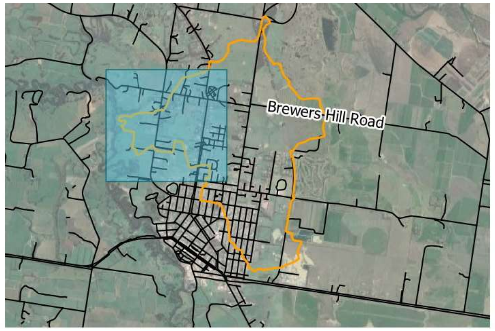
Western Catchment
Developed Case
Afflux
1% AEP Event

LEGEND

Model Extent

Change in Water Level (m)

- <= -1.0
- -1.0 to -0.50
- -0.50 to -0.10
- -0.10 to -0.01
- -0.01 to 0.01
- 0.01 to 0.05
- 0.05 to 0.10
- 0.10 to 0.25
- 0.25 to 0.50
- > 0.50
- Was Wet Now Dry
- Was Dry Now Wet



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

solutions
WATER MODELLING

Appendix A-43

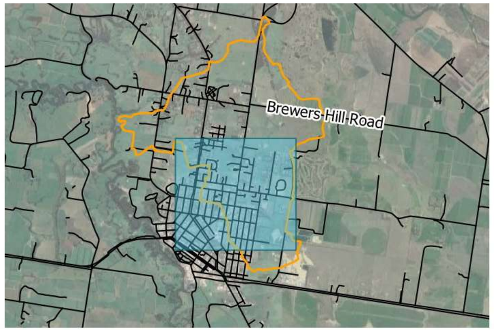
Town Centre
Developed Case
Peak Depth
1% AEP - RCP 4.5

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

solutions
WATER MODELLING

Appendix A-44

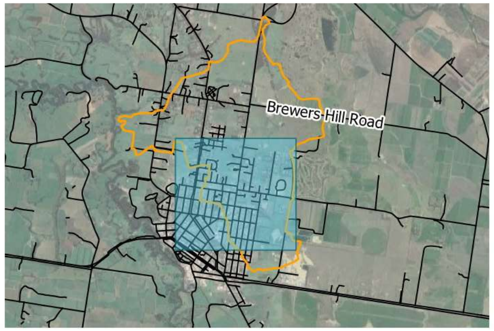
Town Centre
Developed Case
Peak Depth
1% AEP - RCP 8.5

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

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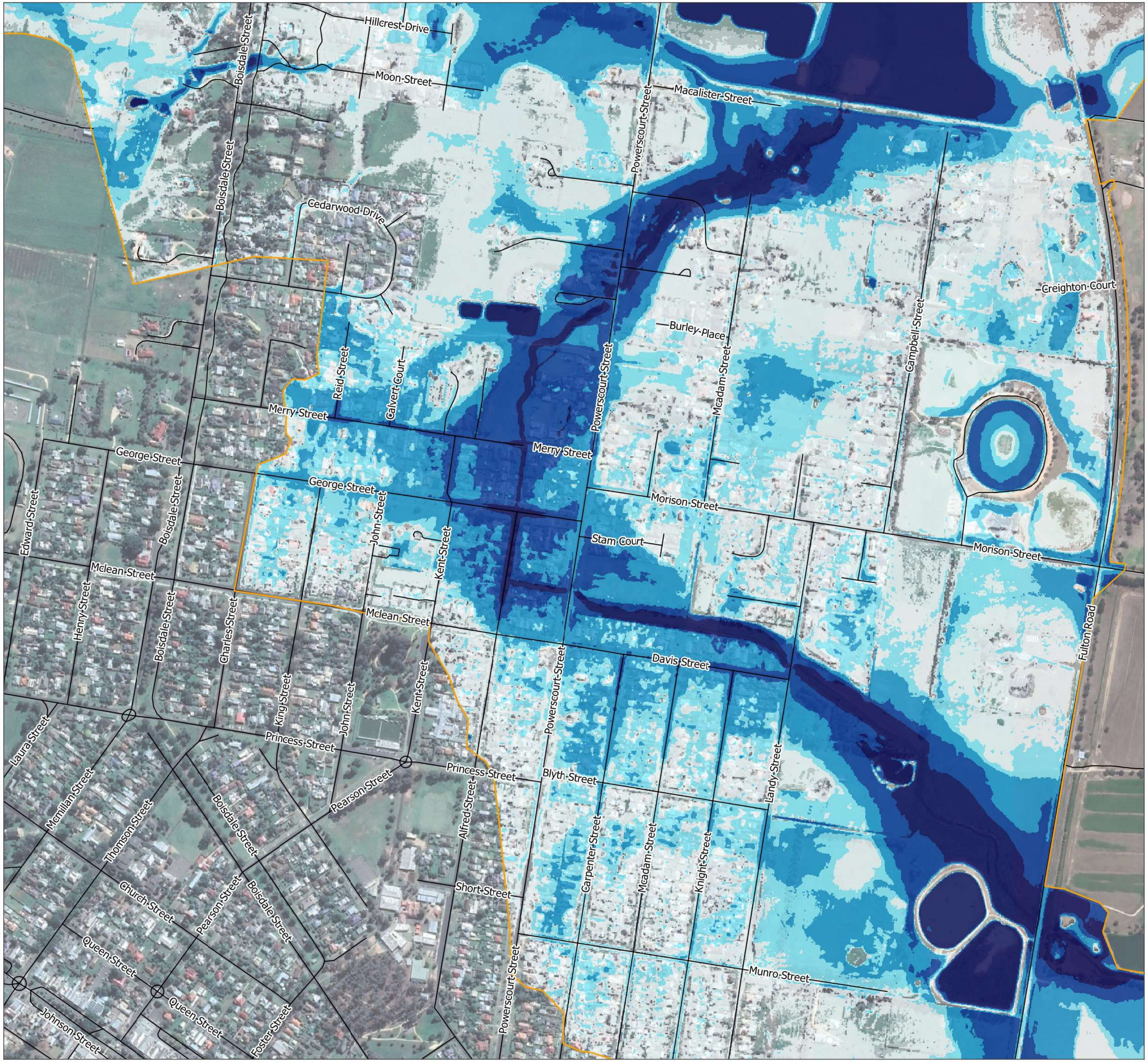
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-45

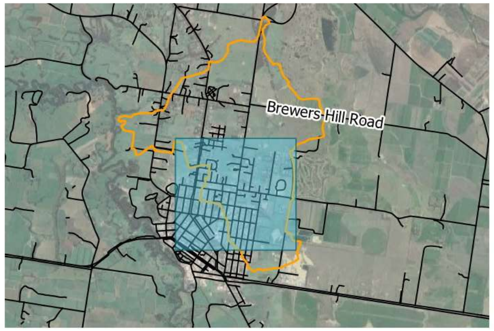
Town Centre
Developed Case
Peak Depth
PMF

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-46

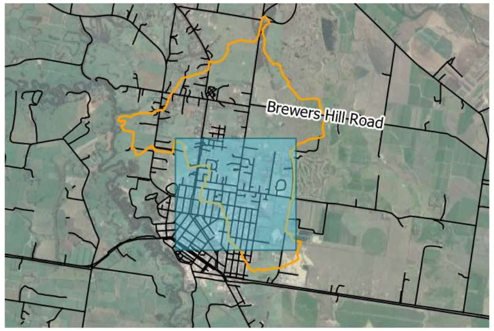
Town Centre
Developed Case
Peak Velocity
1% AEP - RCP 4.5

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-47

Town Centre
Developed Case
Peak Velocity
1% AEP - RCP 8.5

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5

Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

solutions
WATER MODELLING

Appendix A-48

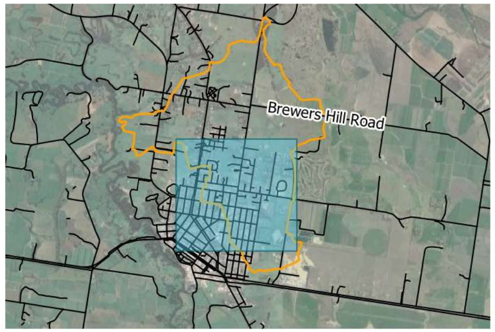
Town Centre
Developed Case
Peak Velocity
PMF

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-49

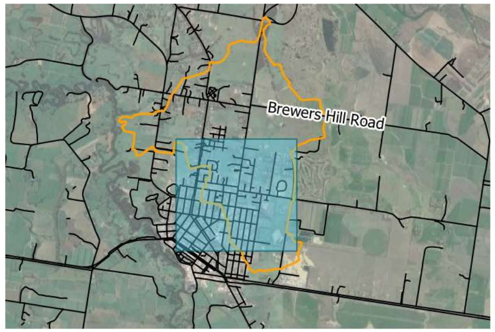
Town Centre
Developed Case
Peak Water Surface Level
1% AEP - RCP 4.5

LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

solutions WATER MODELLING

Appendix A-50

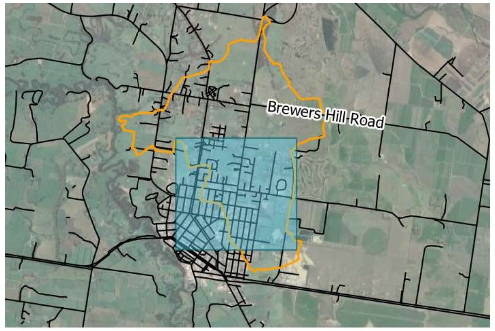
Town Centre
Developed Case
Peak Water Surface Level
1% AEP - RCP 8.5

LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-51

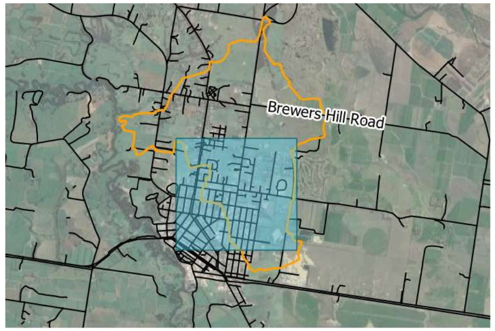
Town Centre
Developed Case
Peak Water Surface Level
PMF

LEGEND

Model Extent

Water Level (mAHD)

20
30
40
50
60
70



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

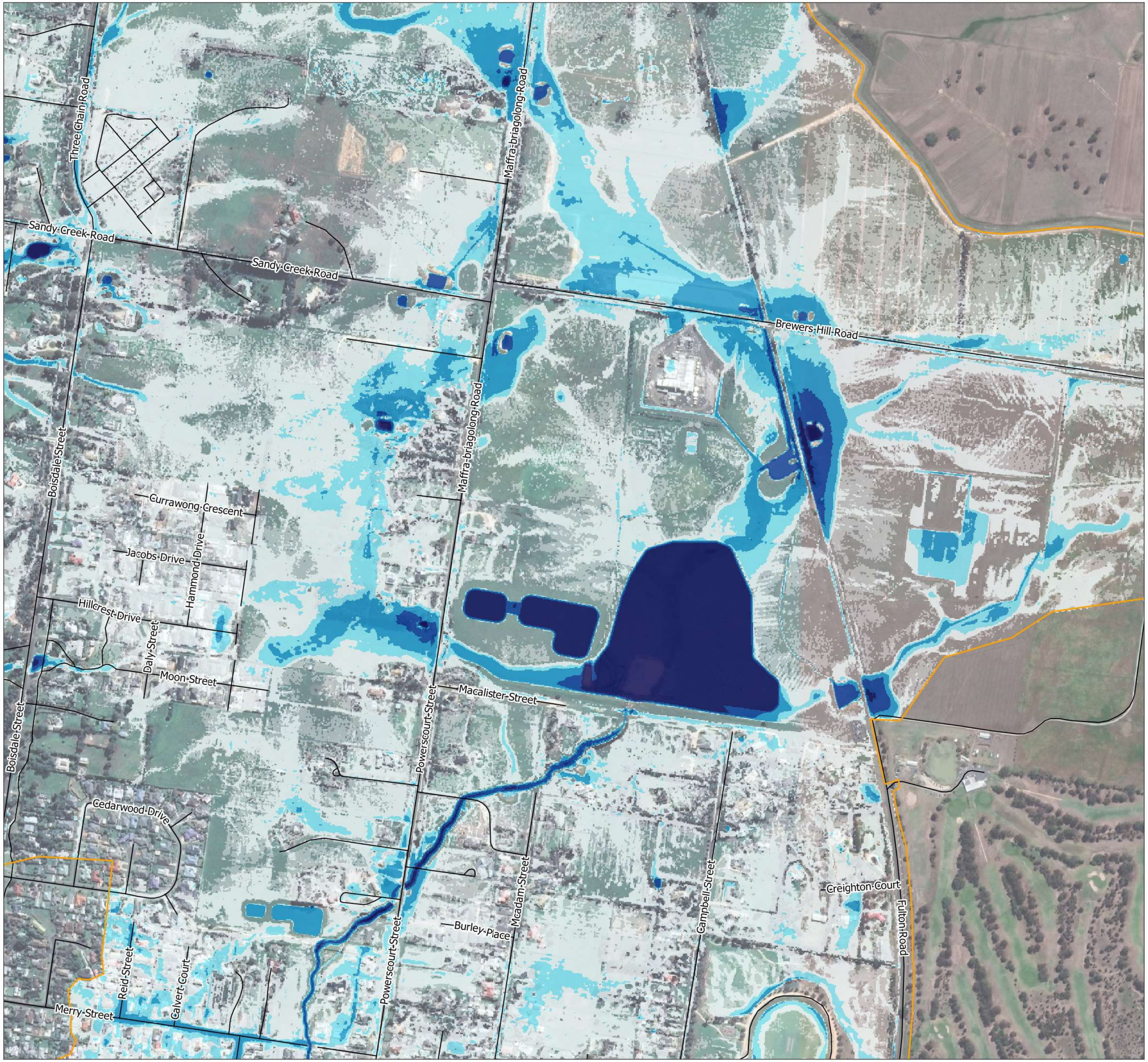
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

solutions
WATER MODELLING

Appendix A-52

Retarding Basin
Developed Case
Peak Depth
1% AEP - RCP 4.5

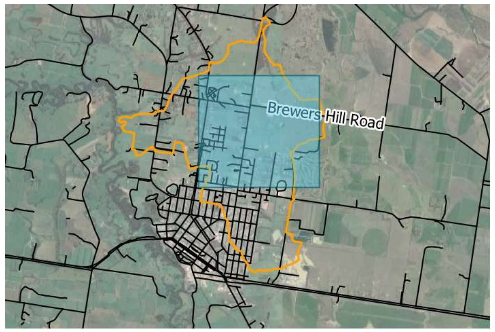


LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

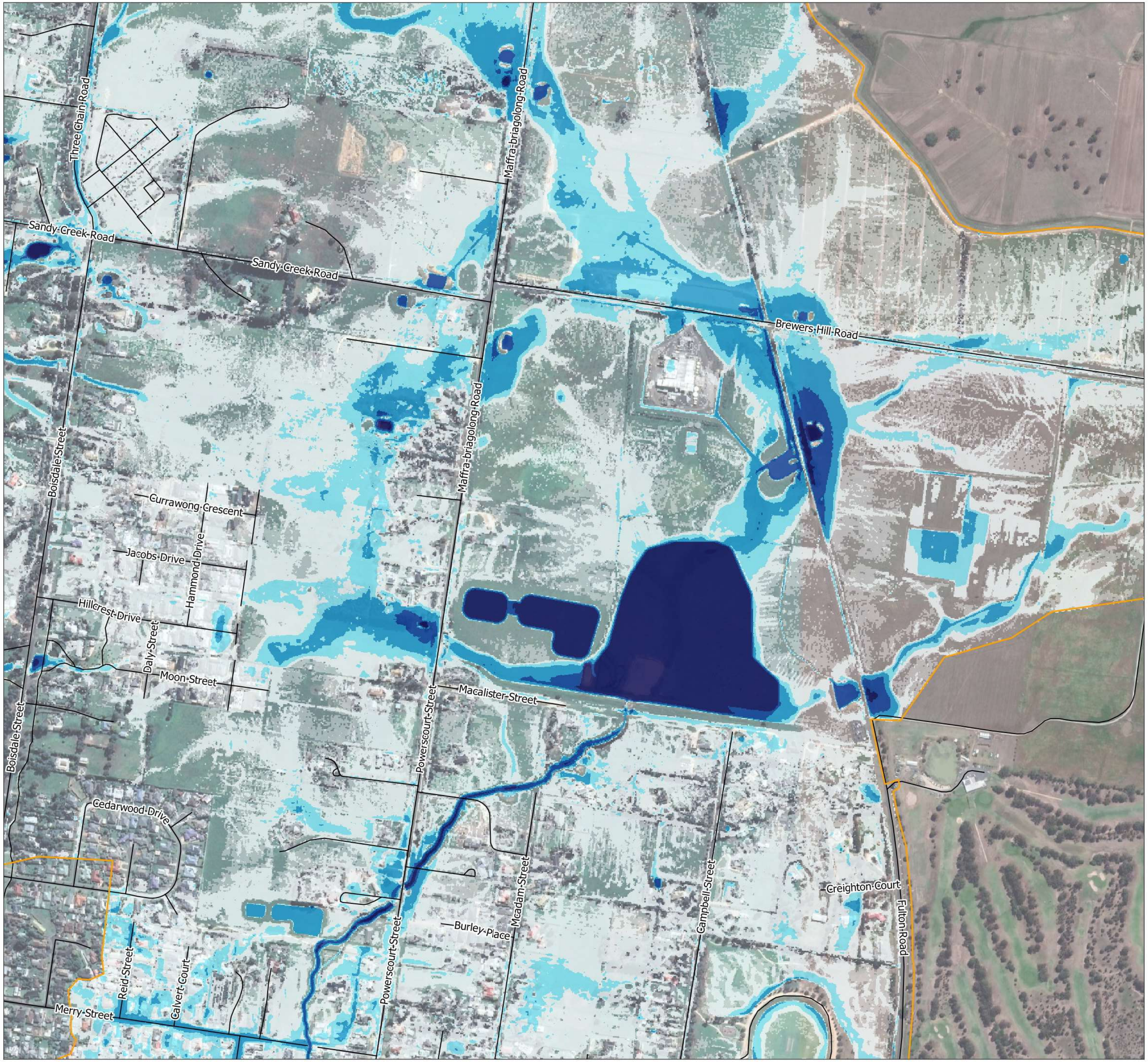
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-53

Retarding Basin
Developed Case
Peak Depth
1% AEP - RCP 8.5

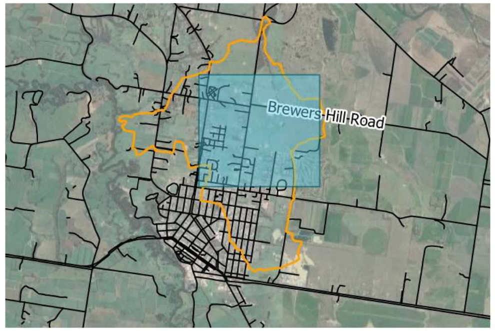


LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0

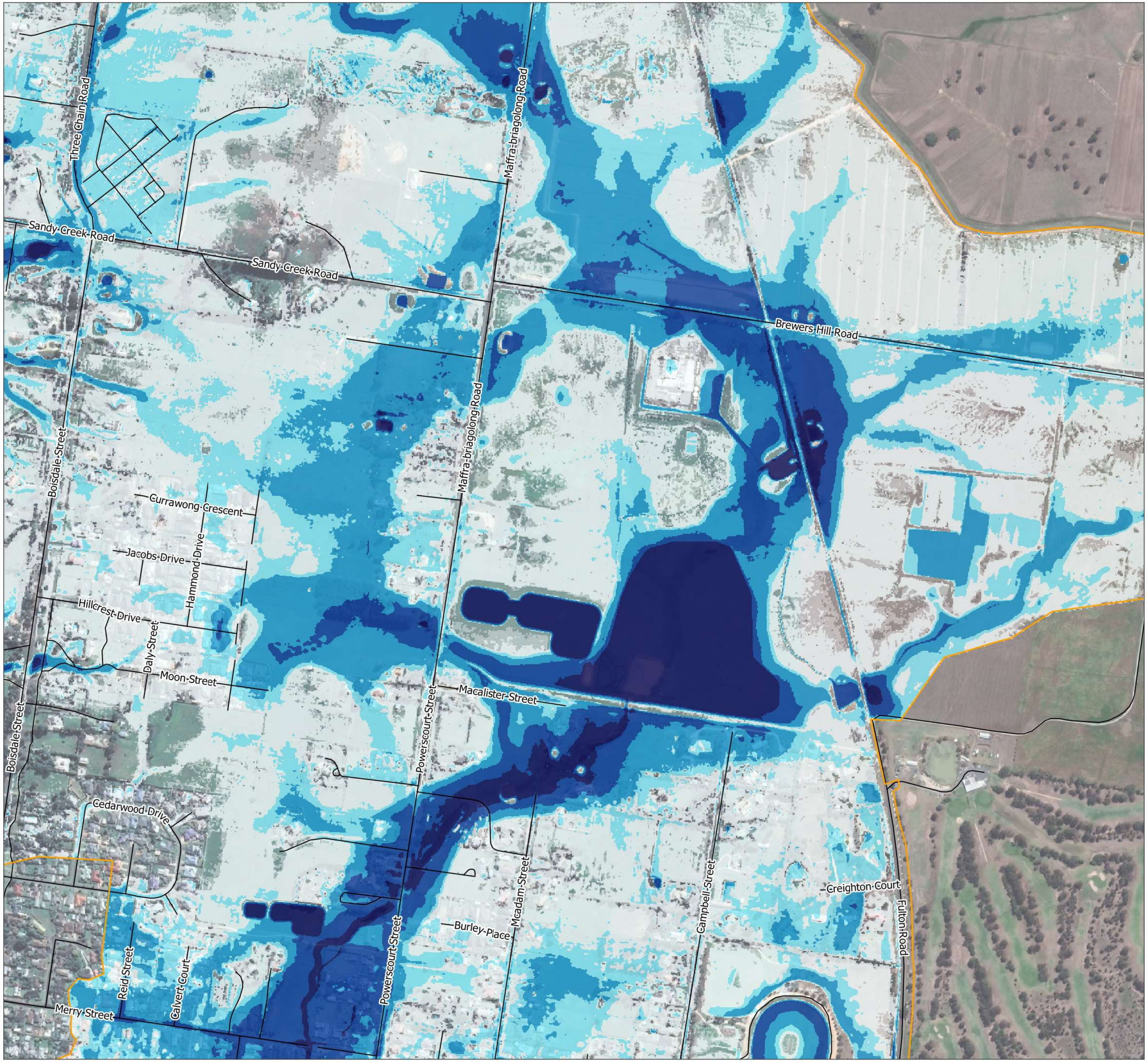


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



solutions
WATER MODELLING

Appendix A-54

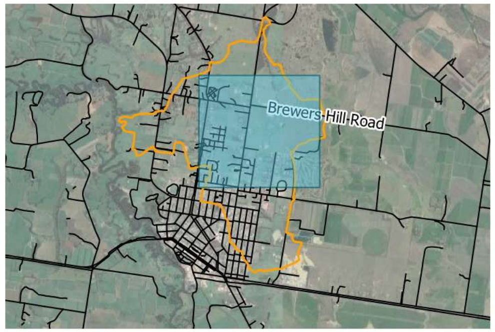
Retarding Basin
Developed Case
Peak Depth
PMF

LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-55

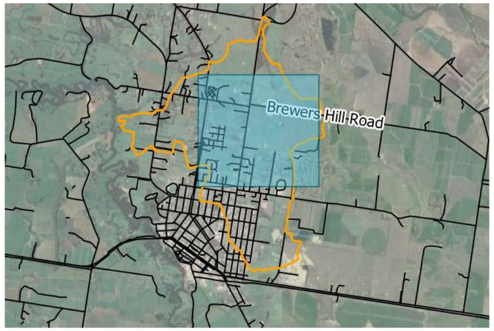
Retarding Basin
Developed Case
Peak Velocity
1% AEP - RCP 4.5

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-56

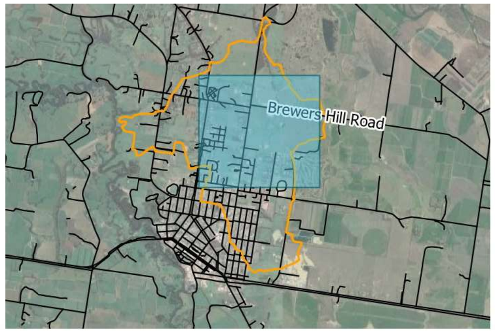
Retarding Basin
Developed Case
Peak Velocity
1% AEP - RCP 8.5

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5

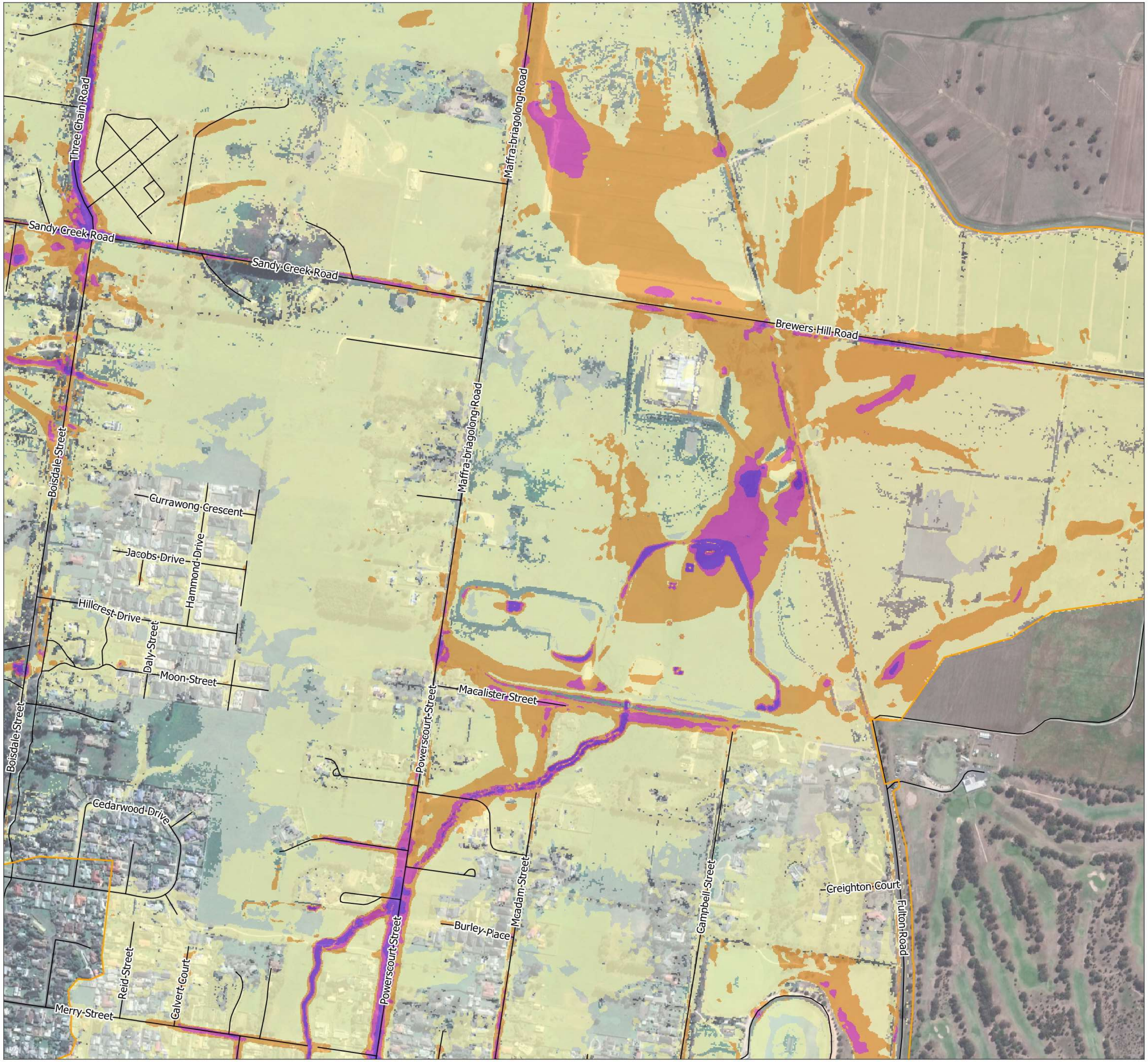


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



solutions
WATER MODELLING

Appendix A-57

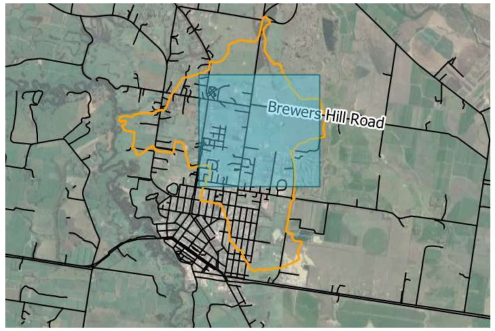
Retarding Basin
Developed Case
Peak Velocity
PMF

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5

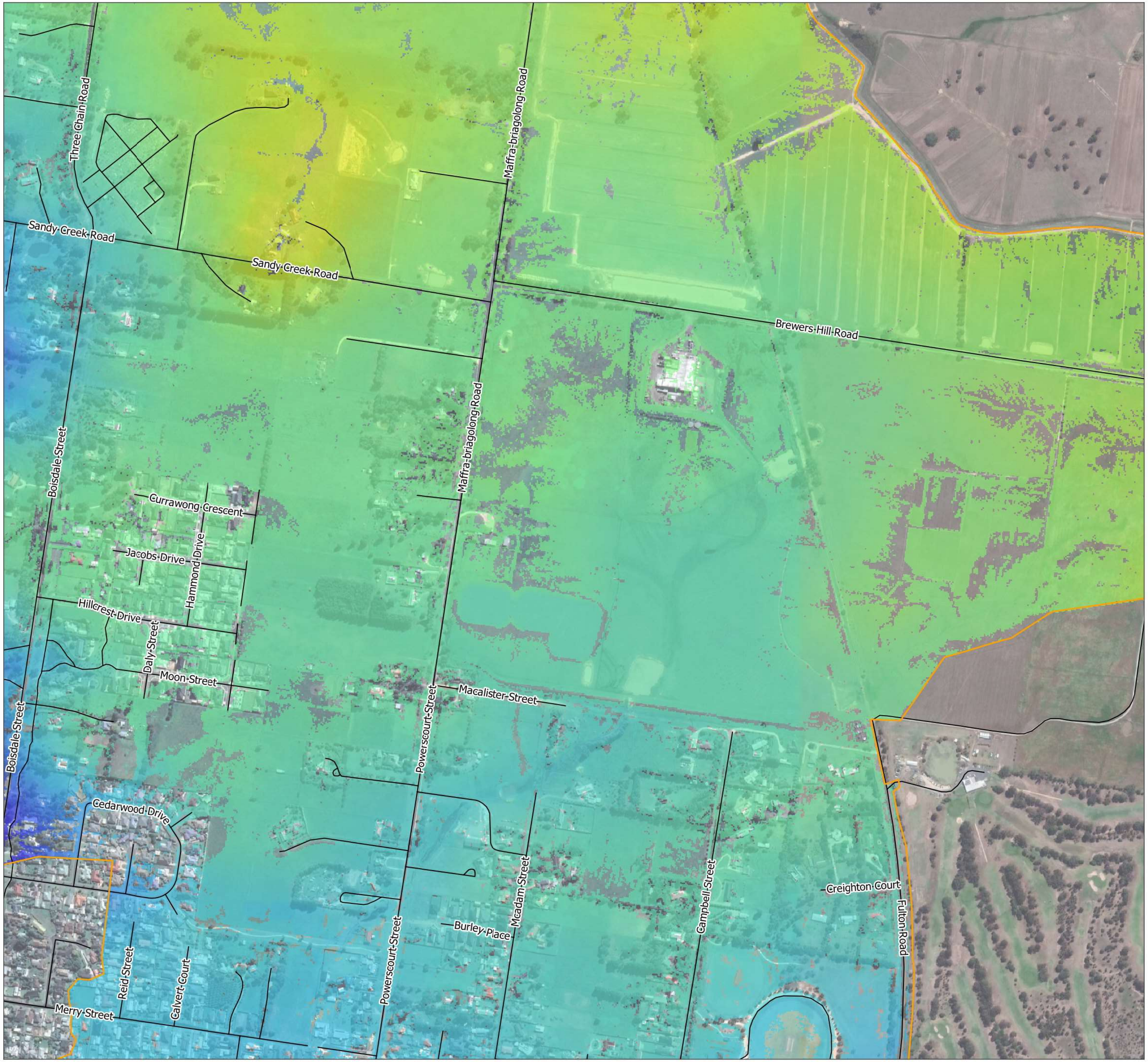


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-58

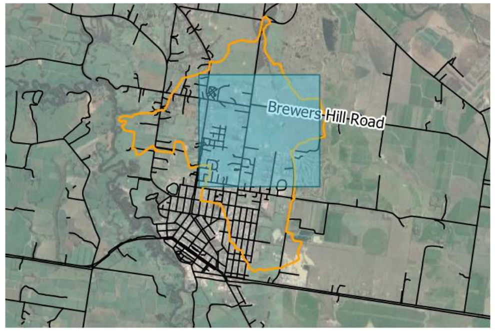
Retarding Basin
Developed Case
Peak Water Surface Level
1% AEP - RCP 4.5

LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70

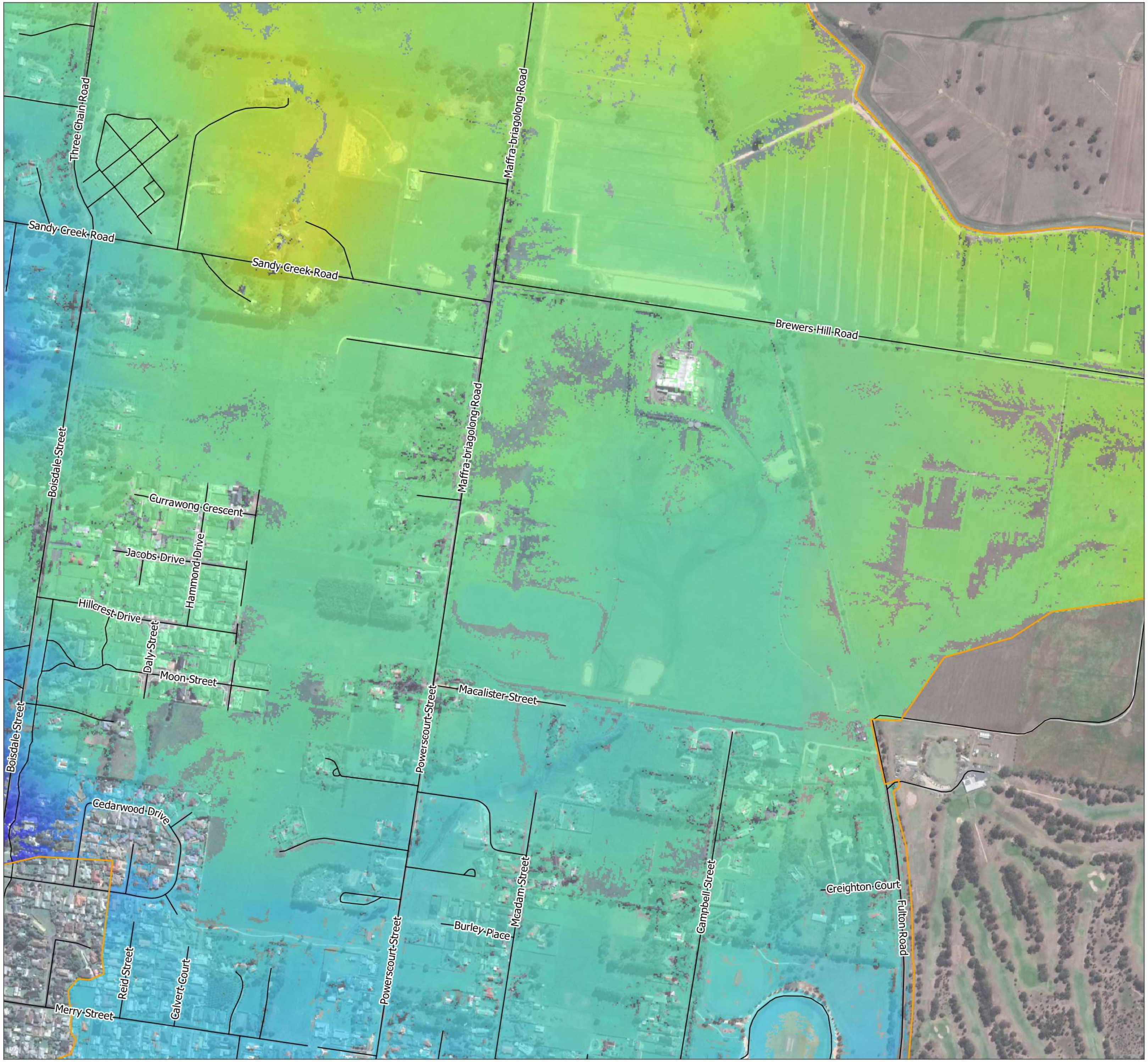


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



solutions
WATER MODELLING

Appendix A-59

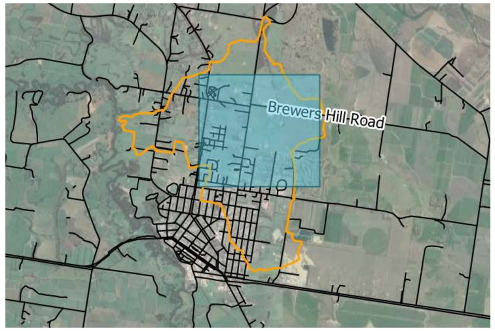
Retarding Basin
Developed Case
Peak Water Surface Level
1% AEP - RCP 8.5

LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

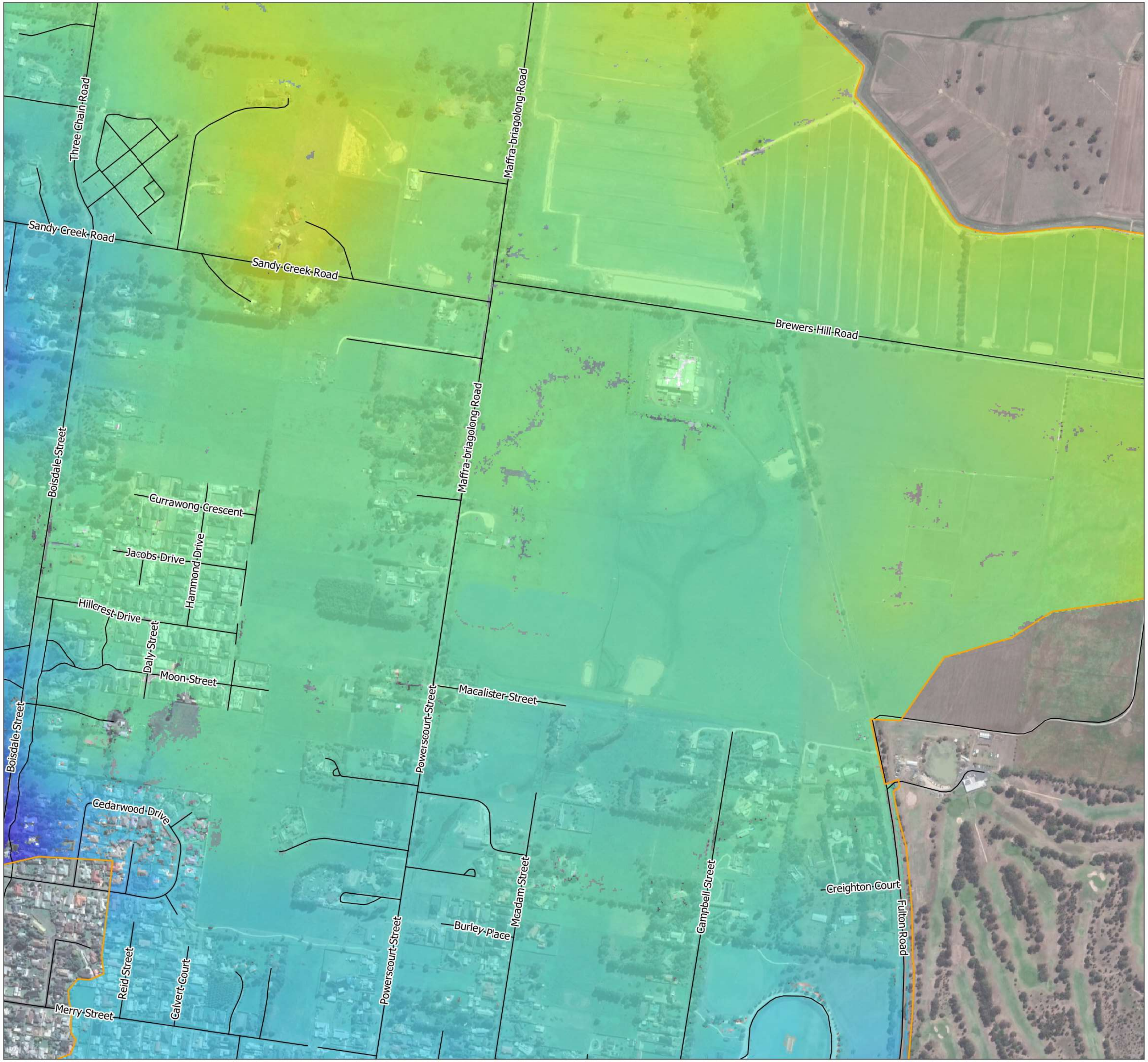
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-60

Retarding Basin
Developed Case
Peak Water Surface Level
PMF

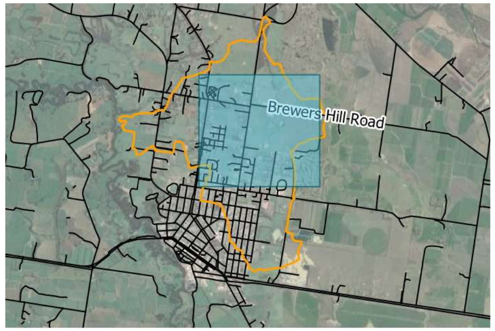


LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

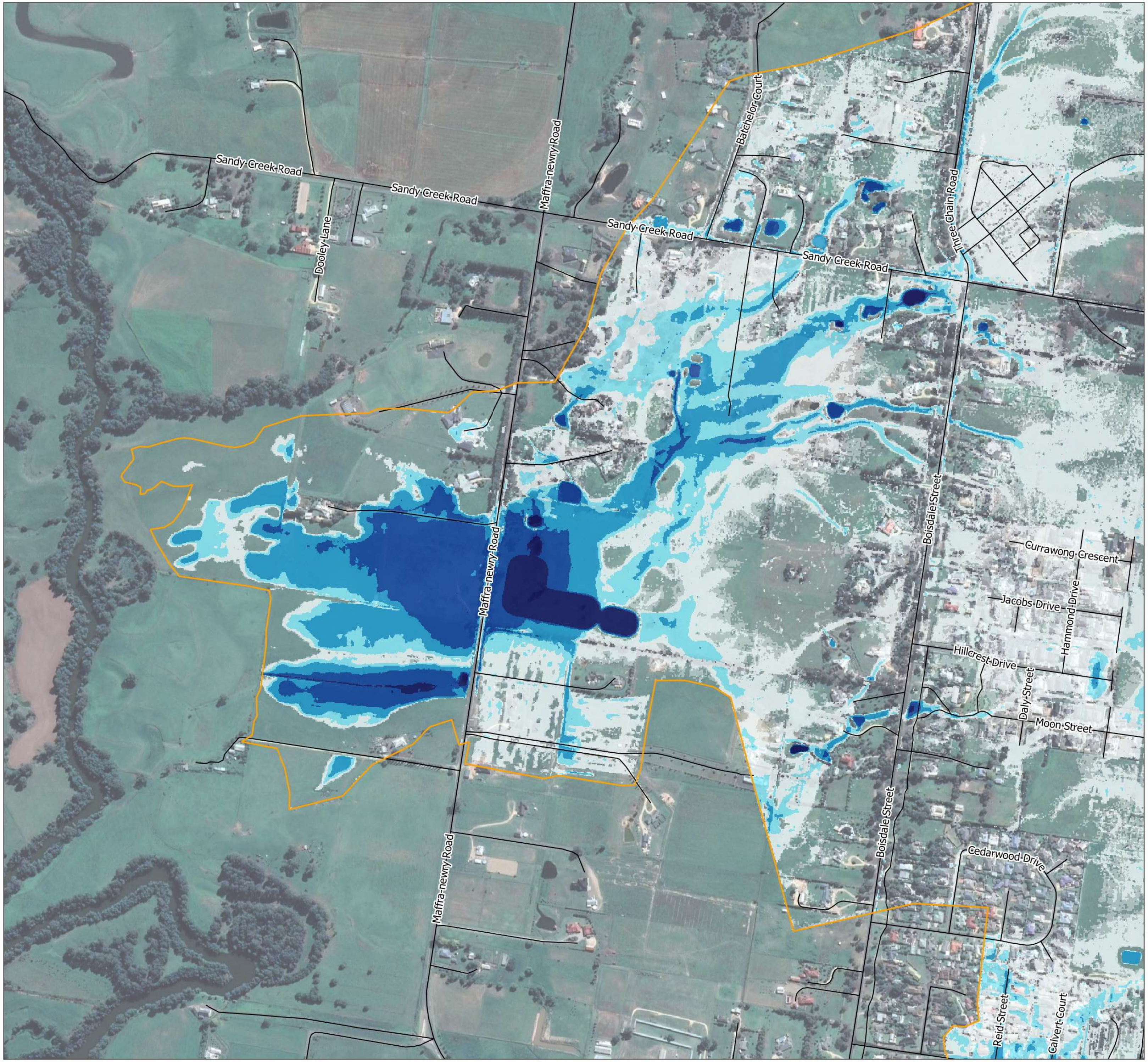
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-61

Western Catchment
Developed Case
Peak Depth
1% AEP - RCP 4.5

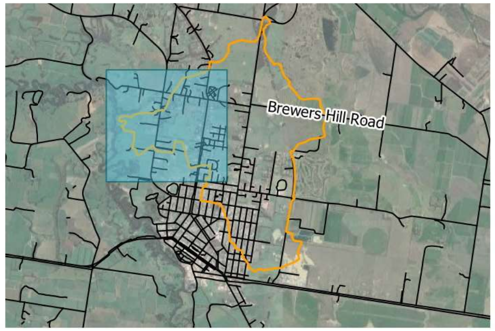


LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0

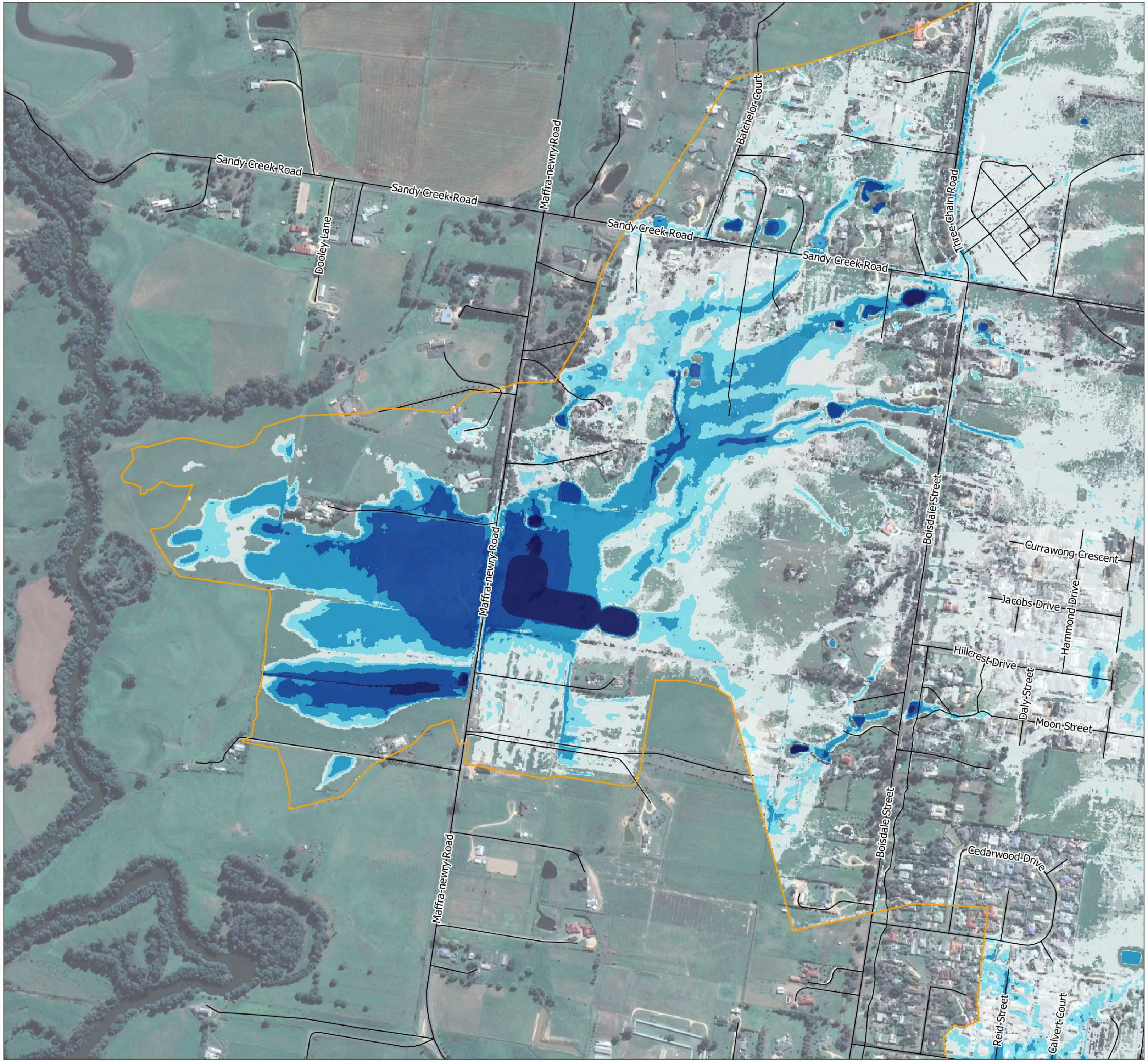


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



solutions
WATER MODELLING

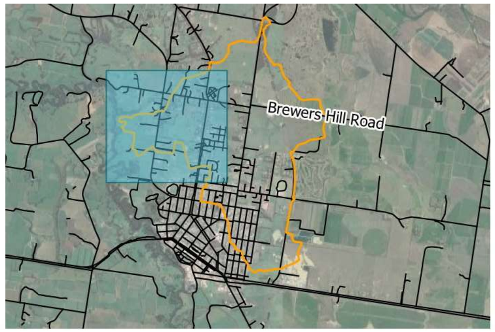
Appendix A-62

Western Catchment
Developed Case
Peak Depth
1% AEP - RCP 8.5

LEGEND

Model Extent

Depth (m)
- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

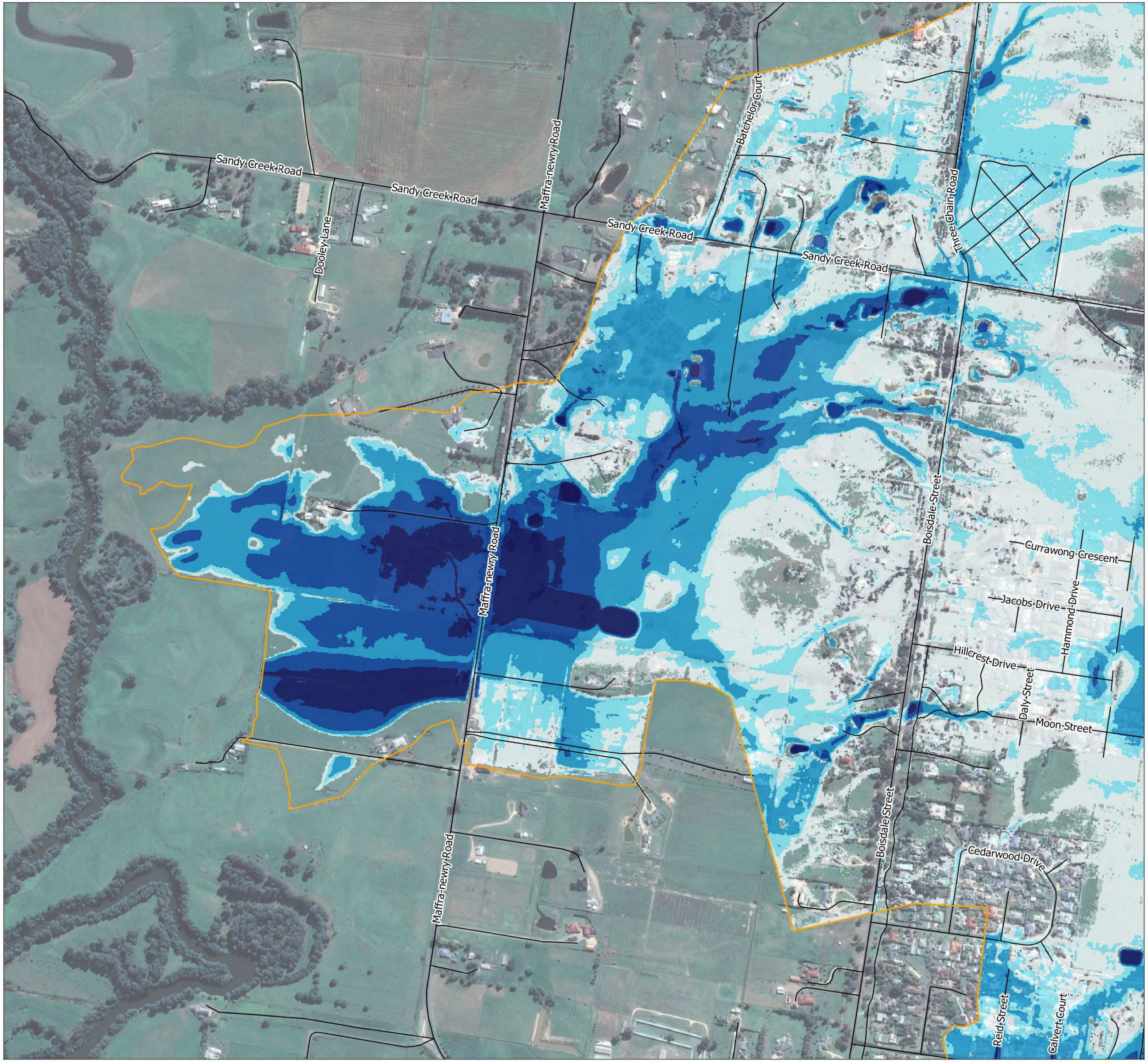
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-63

Western Catchment
Developed Case
Peak Depth
PMF

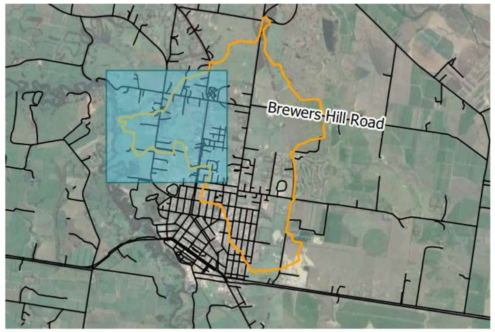


LEGEND

Model Extent

Depth (m)

- <= 0.02
- 0.02 - 0.1
- 0.1 - 0.2
- 0.2 - 0.5
- 0.5 - 1.0
- >1.0

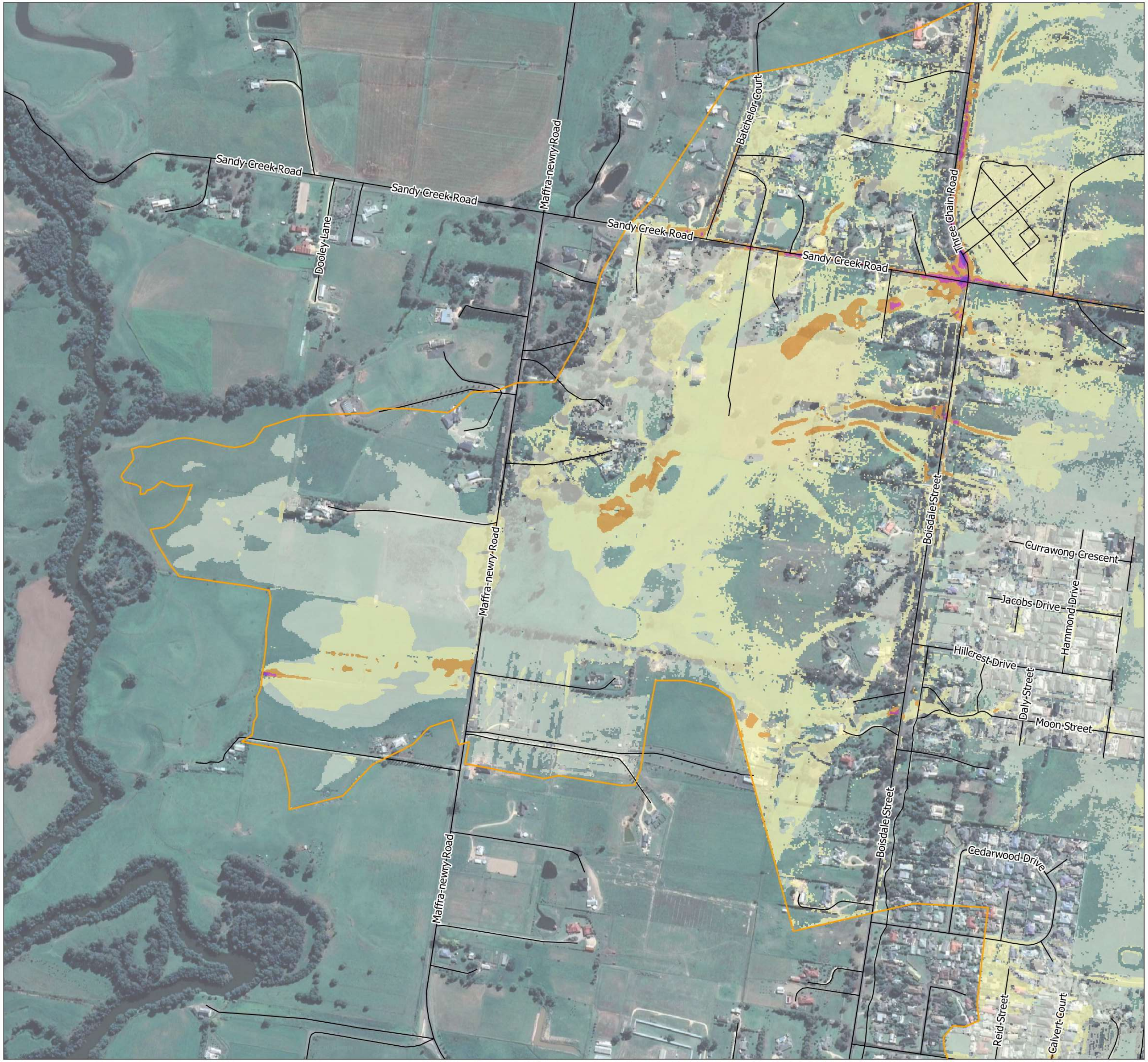


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-64

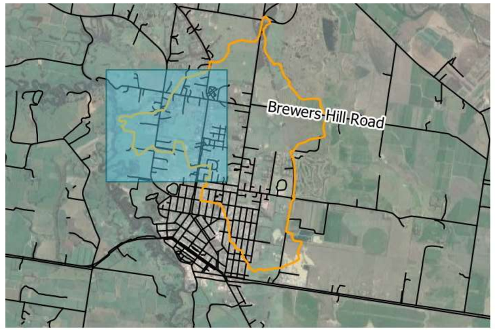
Western Catchment
Developed Case
Peak Velocity
1% AEP - RCP 4.5

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

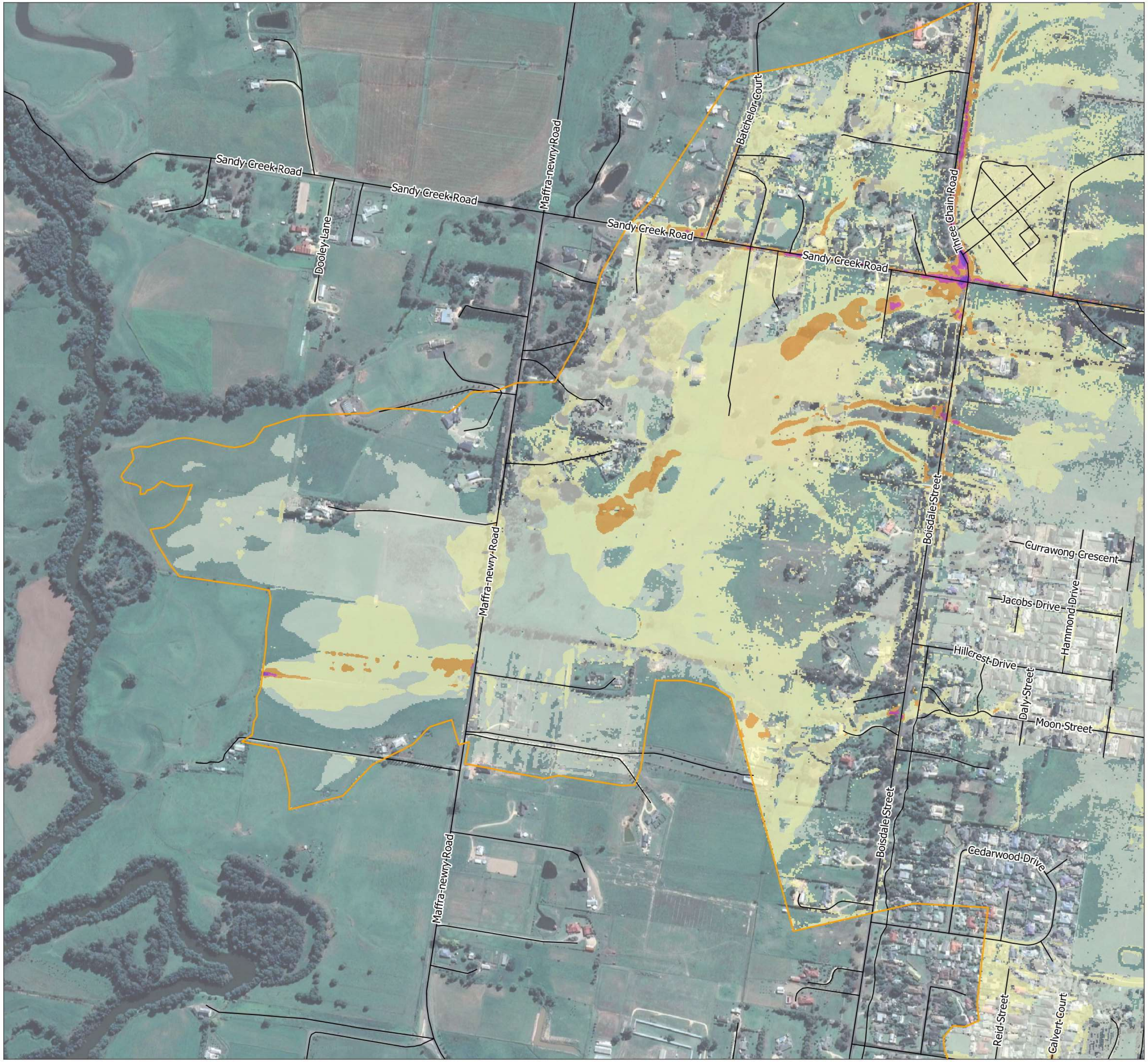
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-65

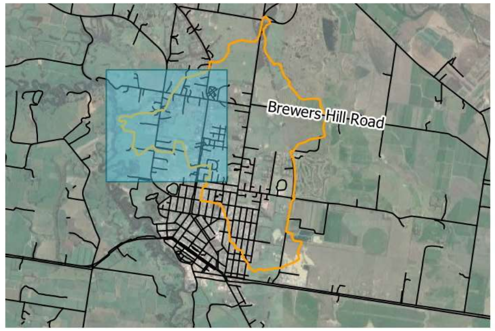
Western Catchment
Developed Case
Peak Velocity
1% AEP - RCP 8.5



LEGEND

Model Extent

Velocity (m/s)
- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5

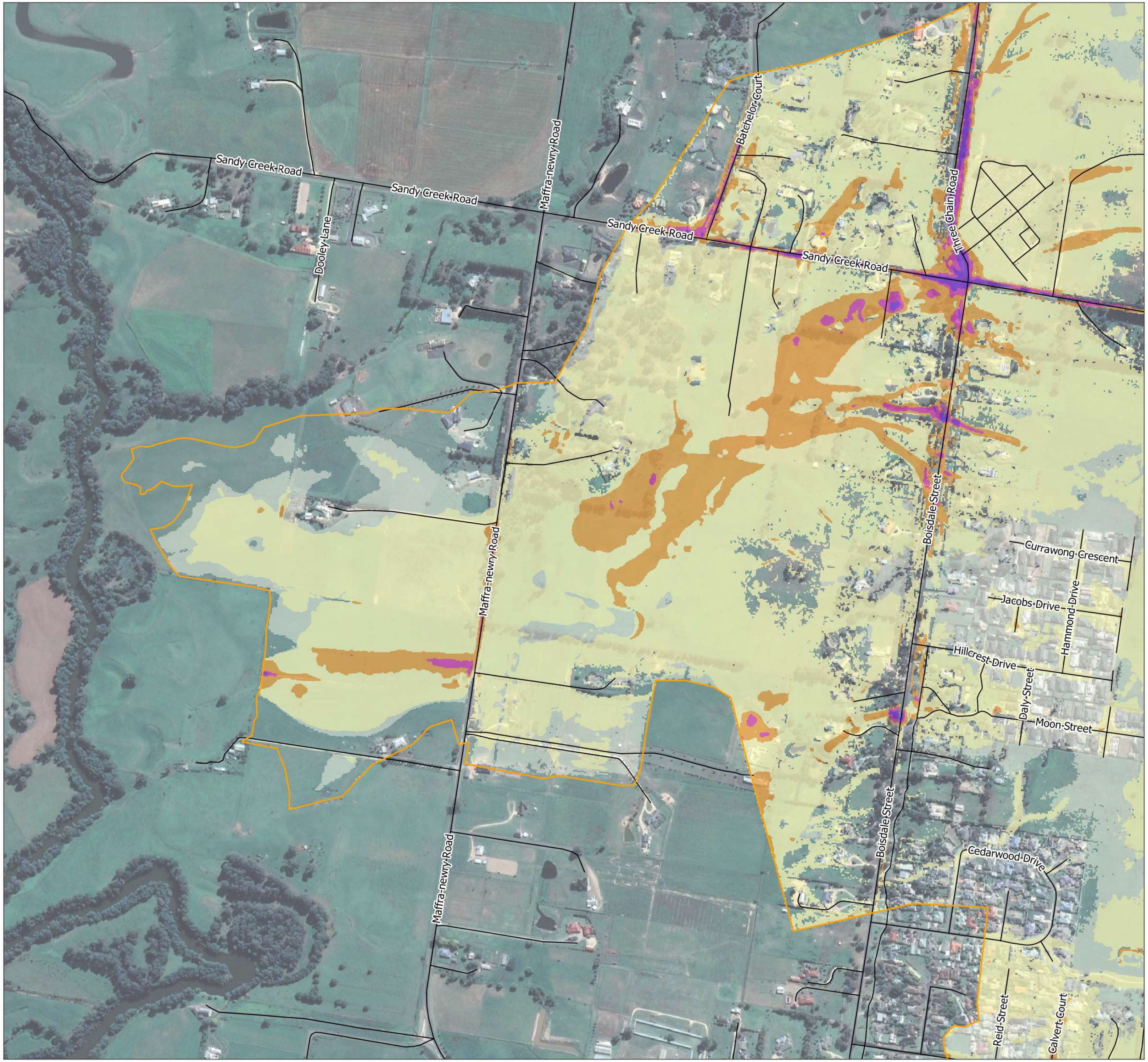


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-66

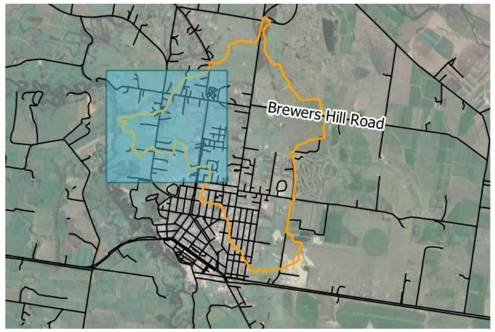
Western Catchment
Developed Case
Peak Velocity
PMF

LEGEND

Model Extent

Velocity (m/s)

- <= 0.25
- 0.25 - 0.50
- 0.5 - 1.00
- 1.00 - 1.50
- >1.5



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

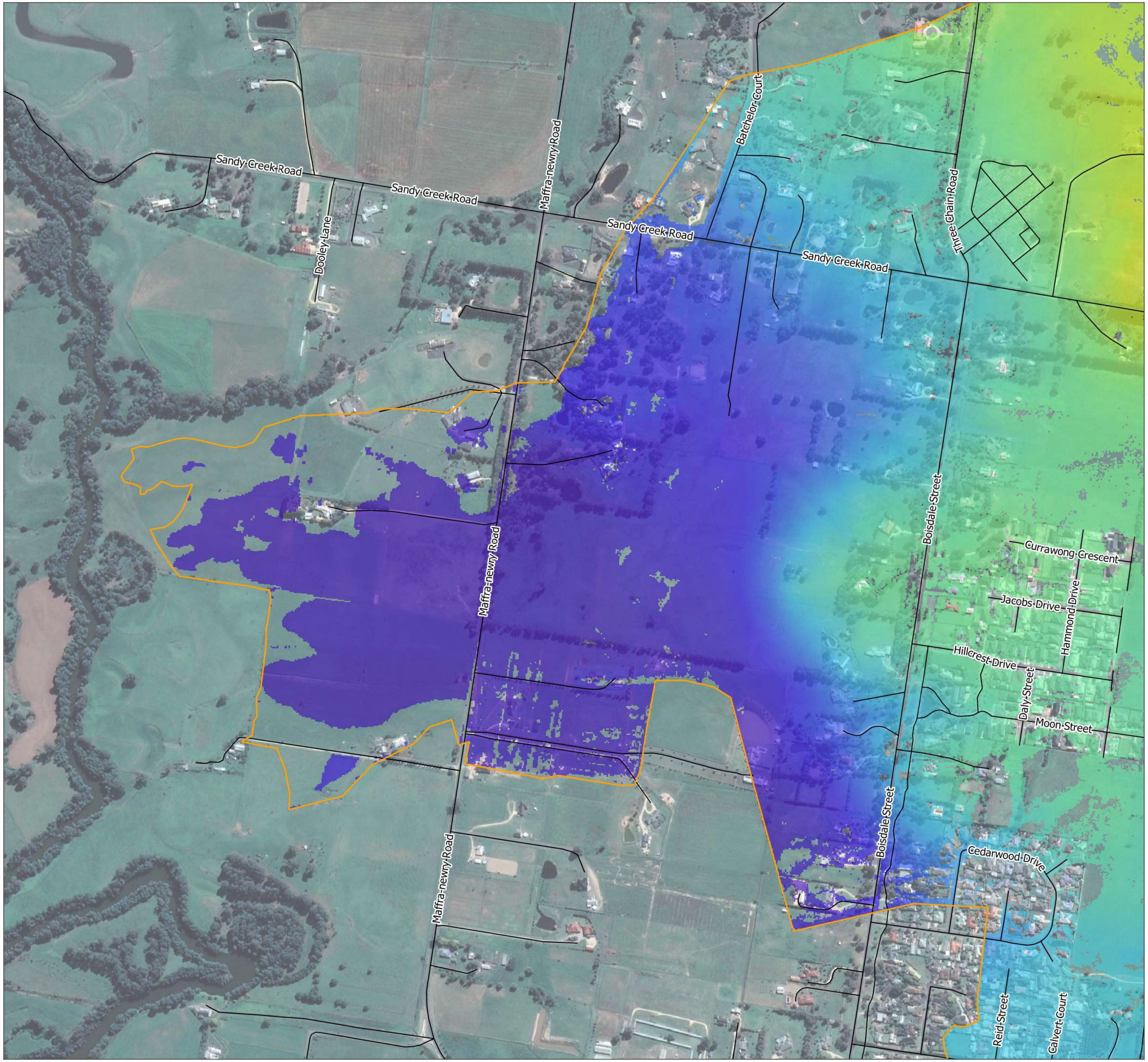
Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

WATER MODELLING solutions

Appendix A-67

Western Catchment
Developed Case
Peak Water Surface Level
1% AEP - RCP 4.5

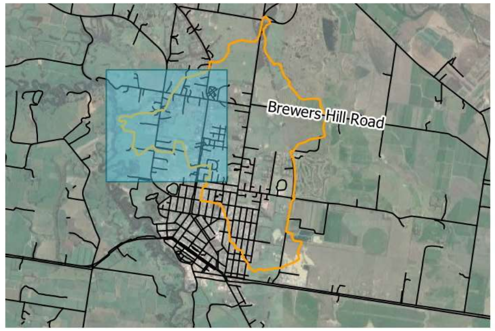


LEGEND

Model Extent

Water Level (mAHD)

- 20
- 30
- 40
- 50
- 60
- 70

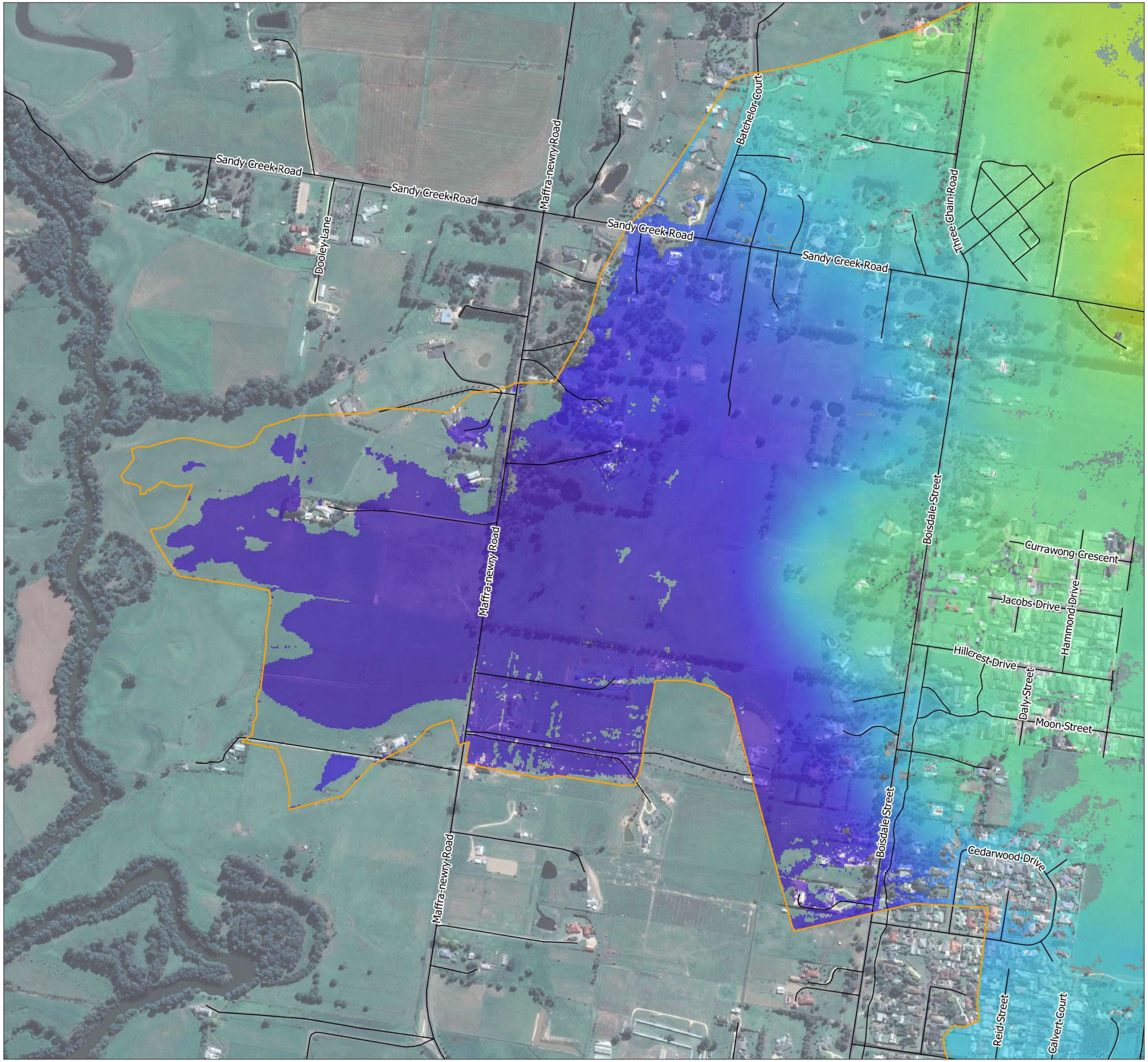


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-68

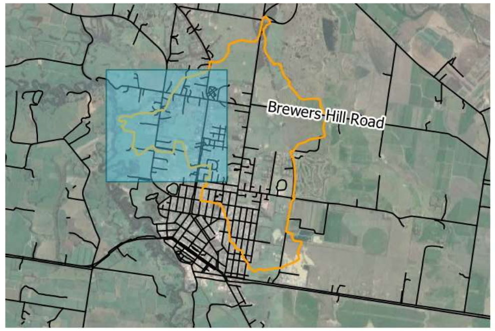
Western Catchment
Developed Case
Peak Water Surface Level
1% AEP - RCP 8.5

LEGEND

Model Extent

Water Level (mAHD)

20
30
40
50
60
70

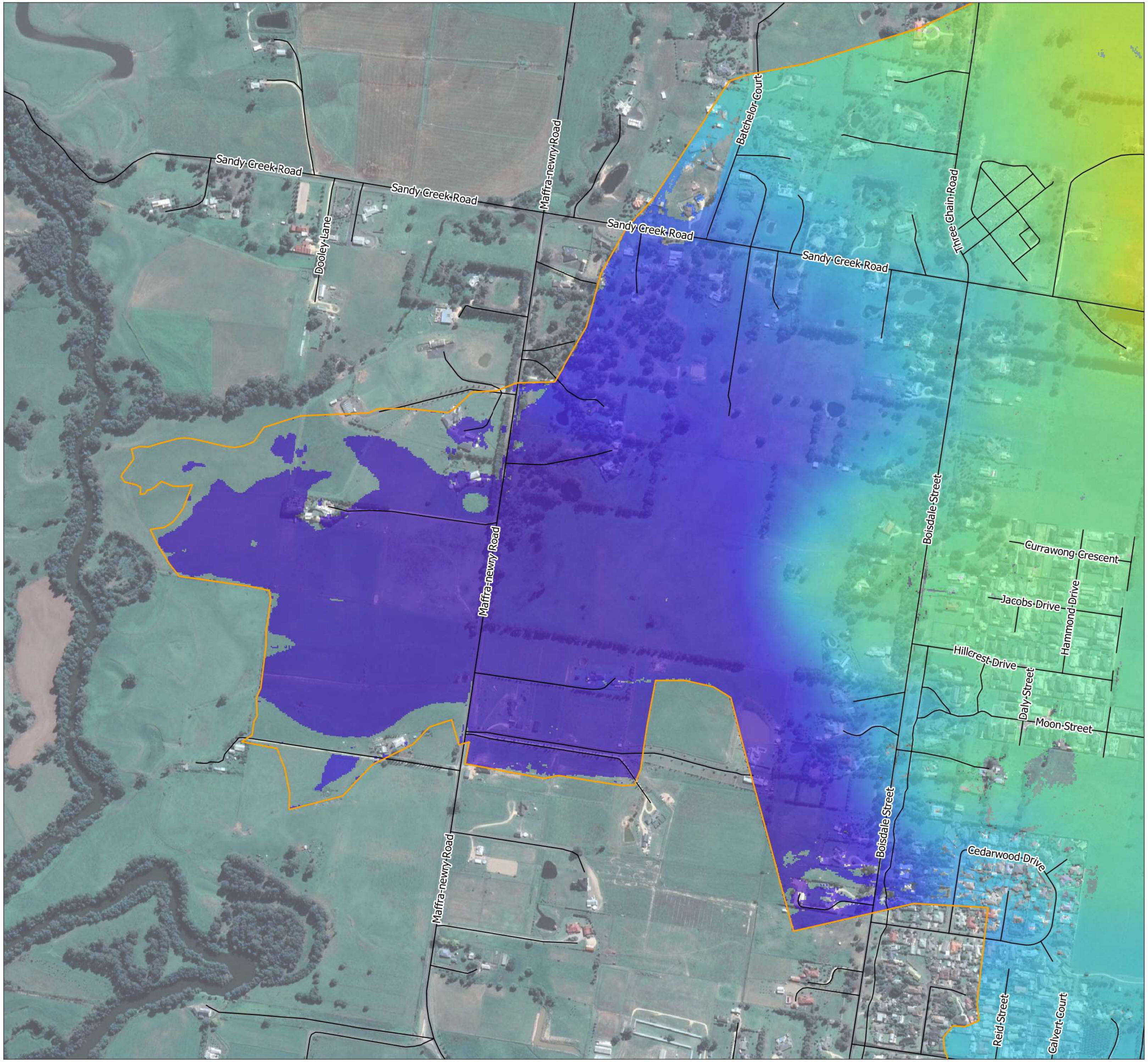


Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m



WATER MODELLING solutions

Appendix A-69

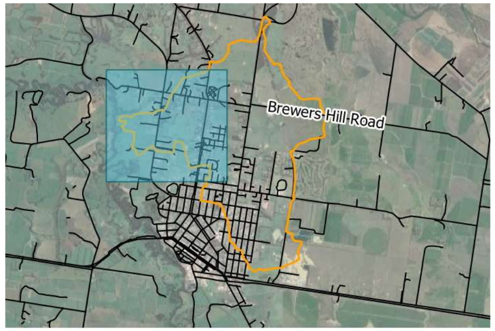
Western Catchment
Developed Case
Peak Water Surface Level
PMF

LEGEND

Model Extent

Water Level (mAHD)

20
30
40
50
60
70



Maffra Drainage Integrated Water Management Strategy

1:7500 @ A3
GDA 1994 / MGA Zone 55

Job No: 30009
Date: 14/08/2020

0 100 200 300 400 500 m

APPENDIX B

COUNCIL DATA

B.1 CULVERT DATA

Figure A-1 Additional Culvert information provided by WSC 30/4/2020

Tue 2/06/2020 9:28 AM

John Inglis <johni@wellington.vic.gov.au>

RE: Additional culvert info for Maffra

To Dan O'Halloran; Sam Pye

Cc Jenny Butcher; Kylee Smith

You forwarded this message on 2/06/2020 9:29 AM.

Hi Dan,

The culvert dimensions requested are as follows:

- Maffra Newry Road DN 600 RCP
- Boisdale Street DN 600 RCP
- Morison Street 1 900*300 Box Culvert
- Fulton Road 1 450*225 Box Culvert, this flows east a short distance then under Norden's Lane in a southerly direction
- Fulton Road 2 3/DN 900 RCP, ex Railway installation

It should be noted that in times of high rainfall and Macalister River in flood the road section immediately north of the culvert can be overtopped.

If you have any further questions please contact me.

Regards

John

John Inglis
Project Engineer

APPENDIX C

TUFLOW MODEL HEALTH

C.1 TUFLOW MODEL HEALTH CHECKS – MINIMUM DT

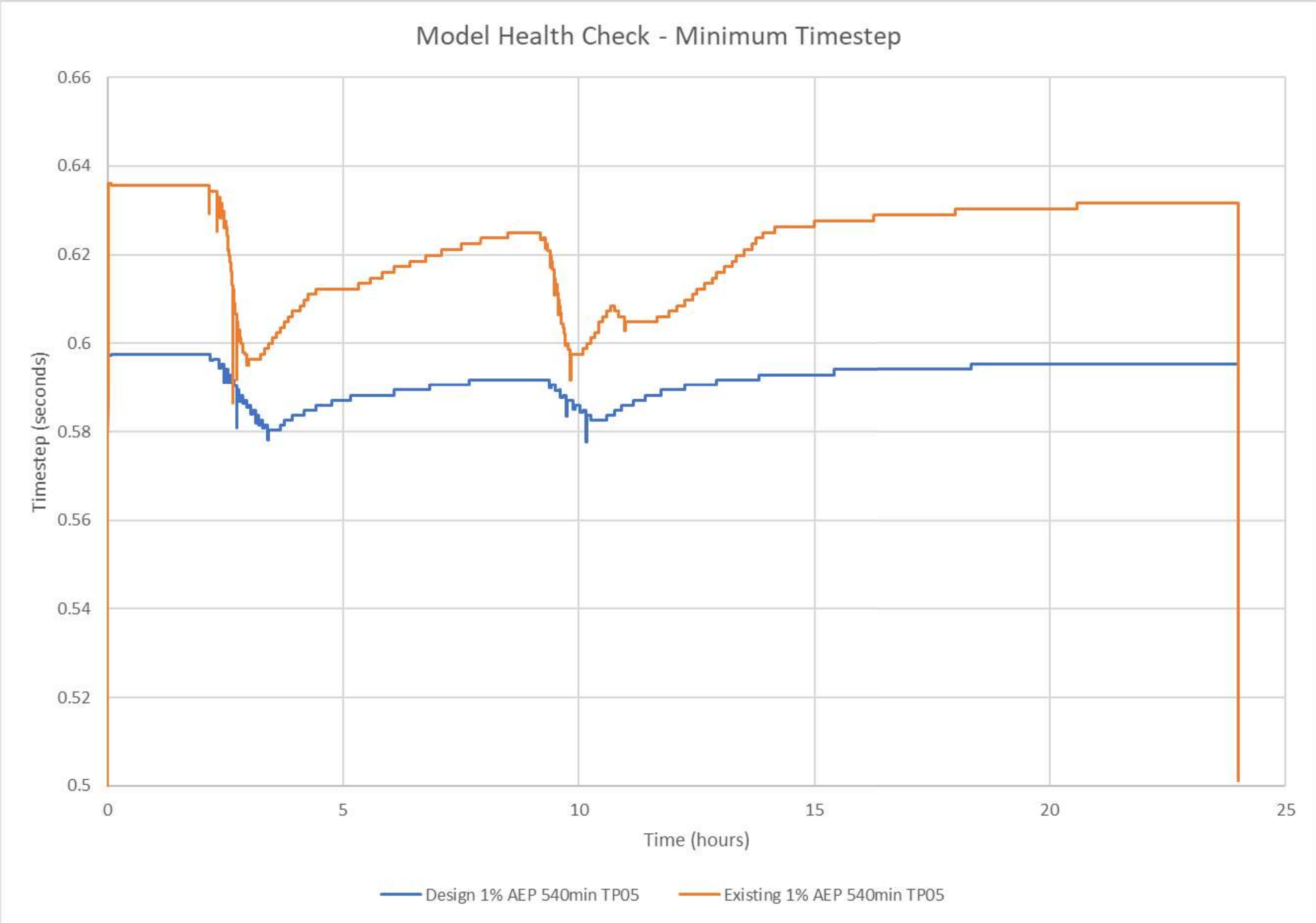
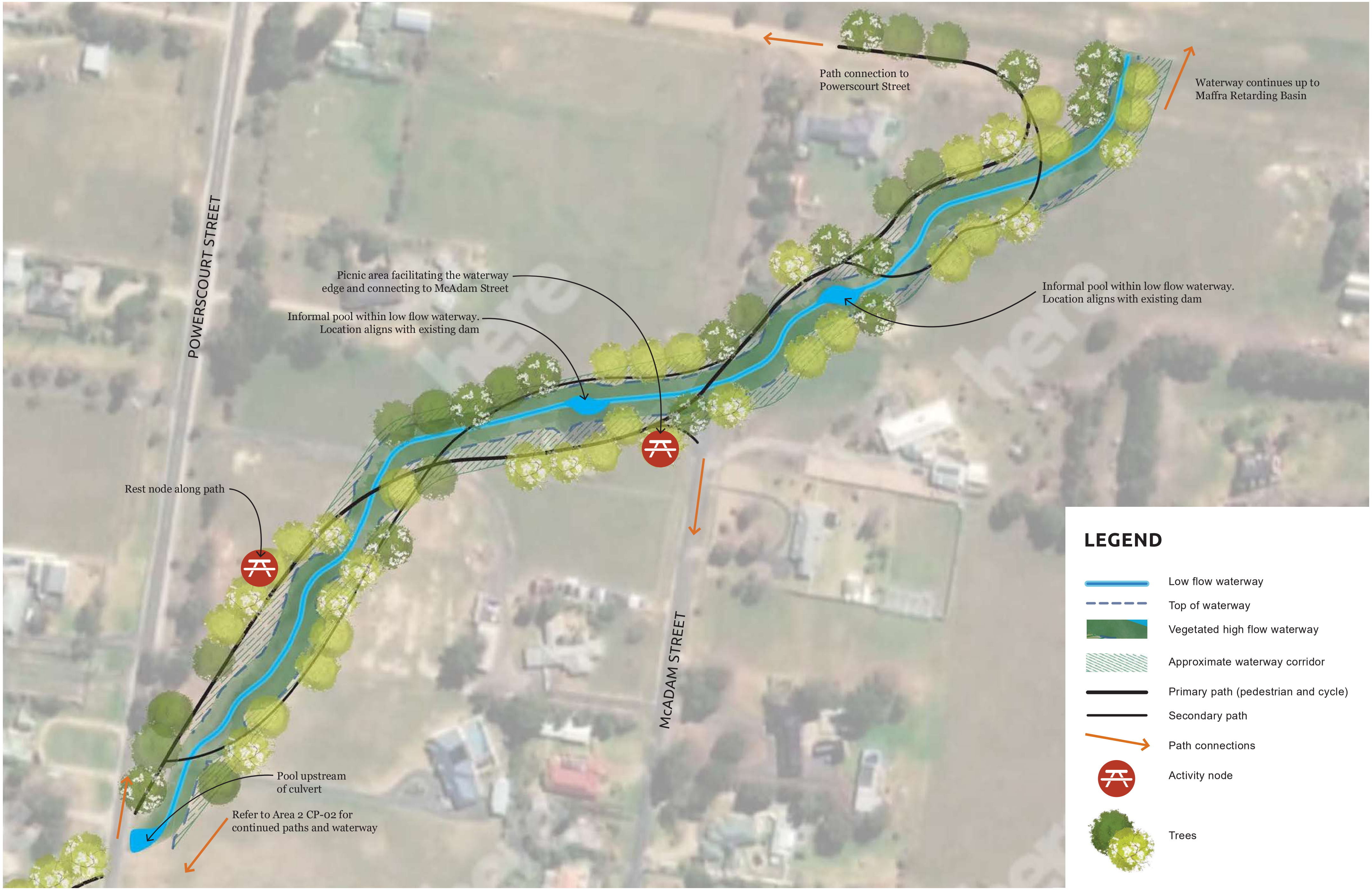


Figure A-2 Tuflow Model Health Check – Minimum dT plot

Appendix F
Landscape sketches



DRAWING TITLE AREA 1 - NORTH POWERSCOURT STREET DRAIN NATURALISATION

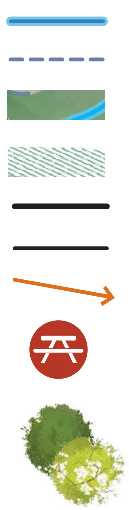
PROJECT TITLE MAFFRA DRAINAGE IWM STRATEGY
CLIENT ALLUVIUM LOCATION MAFFRA, VICTORIA
PROJECT NO. 2002 ISSUE DATE SEPTEMBER 2020 DRAWN/CHECKED BY JZ/HL DRAWING NO. CP-01 REVISION NO. A PAGE NO. 1 OF 3

NORTH

0 15 30 45 60 75m
Scale 1:1500 @ A3

T 0428 325 192 E design@joshbyrne.com.au W joshbyrne.com.au
A 293 Barkly Street Brunswick VIC 3056
JOSH BYRNE & ASSOCIATES LANDSCAPE SUSTAINABILITY COMMUNICATIONS

LEGEND



Potential nature play or public art at waterway bend.

Potential picnic area.

DRAWING TITLE AREA 2 - GEORGE STREET TO POWERSCOURT STREET WATERWAY TRANSFORMATION

PROJECT TITLE MAFFRA DRAINAGE IWM STRATEGY
CLIENT ALLUVIUM LOCATION MAFFRA, VICTORIA
PROJECT NO. 2002 ISSUE DATE SEPTEMBER 2020 DRAWN/CHECKED BY JZ/HL DRAWING NO. CP-02 REVISION NO. B PAGE NO. 2 OF 3

NORTH

0 15 30 45 60 75m
Scale 1:1500 @ A3

T 0428 325 192 E design@joshbyrne.com.au W joshbyrne.com.au
A 293 Barkly Street Brunswick VIC 3056
JOSH BYRNE & ASSOCIATES LANDSCAPE SUSTAINABILITY COMMUNICATIONS

LEGEND

- Low flow waterway
- Top of waterway
- Vegetated high flow waterway
- Approximate waterway corridor
- Vegetated swale
- Primary path (pedestrian and cycle)
- Secondary path
- Path connections
- Activity node
- Trees


Potential nature play elements scattered along waterway.


Potential billabong feature adjacent to activity node.

DRAWING TITLE AREA 3 - DAVIS STREET DRAIN NATURALISATION
PROJECT TITLE MAFFRA DRAINAGE IWM STRATEGY
CLIENT ALLUVIUM **LOCATION** MAFFRA, VICTORIA
PROJECT NO. 2002 **ISSUE DATE** SEPTEMBER 2020 **DRAWN/CHECKED BY** JZ/HL **DRAWING NO.** CP-03 **REVISION NO.** B **PAGE NO.** 3 OF 3

NORTH

0 25 50 75 100 125m
Scale 1:2500 @ A3

T 0428 325 192 E design@joshbyrne.com.au W joshbyrne.com.au
A 293 Barkly Street Brunswick VIC 3056
JOSH BYRNE & ASSOCIATES LANDSCAPE SUSTAINABILITY COMMUNICATIONS