Euroa Post Flood Mapping and Intelligence Project

FINAL

Stage 1 - Detailed Report

NA49913546

Prepared for
Strathbogie Shire Council

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Cardno
Shaping the Future

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Executive Summary

The township of Euroa requires the development of additional flood intelligence and mapping for the Seven Creeks and Castle Creek catchments to manage their risk of flooding. This investigation follows on from the development of the Euroa Water Management Scheme (July, 2000). The proposed works from this Scheme were completed in 2012 and this investigation aims at examining the revised impacts of these works on the flooding for the Euroa Township and potential management options for the future flood protection of Euroa.

The primary objectives are categorised as Stage 1 and Stage 2.

> Stage 1 – Hydrologic and hydraulic modelling (calibration and validation); and the hydraulic assessment (performance) of the Castle Creek levee, including analysis for potential improvements.
>
> Stage 2 – The development of the flood intelligence, MFEP Appendices, municipal flood response plan and land use planning maps.

The report outlines the details of the study from the Stage 1 component of this study. Stage 2 focuses on the flood emergency response aspects of the study and is to be presented in a separate document.

Key Deliverables

The key objectives of this study include:

- Review the hydrology and flood modelling and prepare new flood inundation maps for emergency management and land-use planning purposes;
- Create new flood intelligence data (stage versus consequence). It is expected that this data is tied with the relevant flood maps;
- Augment floor level database through additional survey;
- Review the performance of the Castle Creek levee and investigate the appropriateness of the levee alignment and height;
- Assess the sediment transport and potential sediment removal programme of Castle Creek;
- Independently apply storm events over the township area to inform overland flow paths without riverine flooding;
- Provide information and prepare community information awareness and education brochures in line with the FloodSafe Initiative; and
- Augmentation of telephone alert system including opt-out system.
- Provide site specific flood chart information to assist with building community flood resilience.

Catchment Background

Euroa lies at the foot of the Strathbogie Ranges approximately 160 km north of Melbourne on the Hume Highway. Euroa has a population of around 4,000. Euroa has two major catchments that contribute to flooding including Seven Creeks and Castle Creek. Seven Creeks is the larger of the two catchments at 332 km^2 and Castle Creek at 80 km^2.

The catchments are mostly cleared with the largest land use for agriculture, cropping, sheep and cattle grazing and horse studs. Within the catchment there are areas set as National Parks including the Mt William Flora and Fauna Reserve.

Castle Creek has ongoing issues with sedimentation of sands along the system which migrate from the upper catchment. The investigation of the sedimentation and management options to mitigate this problem forms an important part of this investigation.

Hydrology

The hydrology was developed through a detailed process that calibrated the hydrologic model to 5 events. The design events were ultimately set to match the previous SKM assessment to ensure there was consistency in the planning controls and outputs from the project. The Castle Creek flows were developed using the calibrated hydrologic models and calibrated model parameters from Seven Creeks. The design event peak flow rates are summarised in Table i.

The hydrology is presented in detail in Section 4.

Table i Design peak inflow rates for Seven Creeks and Castle Creek

	Design Events (AEP and peak flows m^3/s)						
	20%	10%	5%	2%	1%	0.5%	0.2%
Seven Creeks	123	184	246	331	398	468	563
Castle Creek	26	40	55	73	90	107	131

Hydraulic Modelling

The hydraulic modelling simulated the 20%, 10%, 5%, 2%, 1%, 0.5%, 0.2% AEP and the PMF event for both Seven Creeks and Castle Creek. The model was calibrated to the 1993 and 2010 flood events. The hydraulic model was developed using the software package TUFLOW and included a detailed 4m grid for the main township of Euroa and a coarse 12m grid to cover the upstream and downstream areas.

The hydraulic modelling results are presented in detail in Section 5. For Castle Creek the levee did not overtop even up to the 0.2% AEP event however during this event there was no freeboard remaining.

The outputs from the hydraulic modelling are a key input into the Stage 2 Municipal Flood Emergency Plan documents.

Survey

For this investigation additional survey was captured. Additional cross sections were captured on Castle Creek upstream of the gauge at Telfords Bridge to impriove the model performance leading to the gauge. This survey information was also used to develop a rating table for the gauge to allow a stage-discharge realtionship to be developed for assessment of the levels at the gauge. The gauge relationship was developed to establish a gauge height and crest level relationship for emergency management purposes. The rating table is summarised in Section 5.8.

In addition to this survey an addiitonal 146 floor levels were captured and combined with the exisiting 1,369 floor levels already captured. Many of the 146 buildings are newly constructed dwellings.

Overland flowpath assessment

To assess the local drainage issue the hydraulic model was simulated using a direct rainfall on grid approach which included the local drainage network. This model process aimed at identifying the local issues which may cause flooding in high intensity rainfall events independent of the riverine flooding. The overland flows were assessed using a range of rainfall durations from 15 minute up to 2 hour events. Section 5.5 outlines the results.

Mapping

Key outputs from the hydraulic modelling process were a suite of maps outlining:

- Peak flood depths for all design flood events (as shown in Figure 5-9 to Figure 5-16).
- Flood extents for all design events.
- Flood planning controls (floodway overlays for the LSIO and FO).
- Velocity and hazard maps for the design events.
- Flood extents with peak water surface elevations at 200mm contours.
- Series of maps showing the peak depths and extents corresponding to gauge levels for Seven Creeks at Euroa at 200mm intervals (and one 100mm interval) between 4.6m and 6.5m on the gauge. Depth interval maps are to be produced for Castle Creek as well.
- Properties impacted during each flood event have been shown on each flood map, this includes properties with overfloor flooding and with water impacting the house below floor level.
- Historic calibration events showing depths and extents (the calibration events, 1993 and 2010).
- Municipal Flood Emergency Plan (MFEP) maps for inclusion in the MFEP appendices.
- Minor, moderate and major flood levels have been mapped for Seven Creeks at Euroa (minor 2.5m, moderate 4.0m and major 4.6m) and Castle Creek at Telfords Bridge (minor 1.2m, moderate 1.8m and major 2.4m).

Damage Assessment

The design events were used to develop a damage assessment for the catchment. The damage assessment estimated the Annual Average Damages at **$896,544** per annum. This is a high annual damage figure and reflects the widespread damage that can occur with flooding in Euroa. The Annual Average Damage calculation is shown in Figure i.

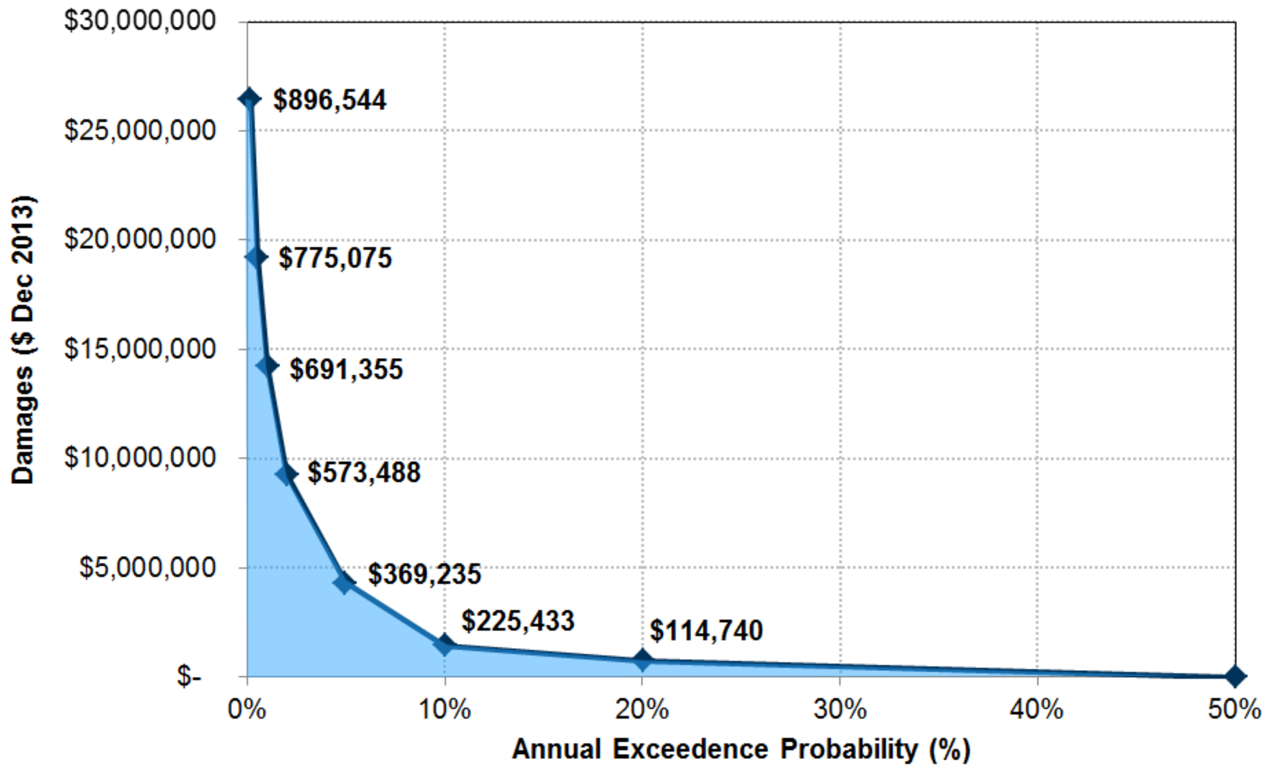

Figure i Annual average damage calculation for Euroa

For the 1% AEP event there are predicted to be 205 buildings with overfloor flooding and over $14m of damage. The largest proportion of this damage is to residential buildings. The full details of the damages are summarised in Table ii and full details are in Section 7.1.

Table ii Summary of the damages and properties impacted during the design flood events

Recurrence Interval	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
Property Damage							
Property Damages	$206,378	$517,914	$1,685,790	$3,487,573	$3,904,188	$4,790,497	$5,599,538
Inundated properties (> 10cm depth, > 1% area)	187	256	598	970	1137	1292	1513
Building Damage							
Residential	1	1	24	72	154	222	323
Industrial	0	1	8	10	14	14	14
Commercial	0	0	7	24	37	55	75
Total buildings with overfloor flooding	**1**	**2**	**39**	**106**	**205**	**291**	**412**
Residential	$29,394	$42,562	$938,394	$3,179,685	$6,802,439	$10,227,889	$15,718,954
Industrial	$0	$23,526	$164,160	$265,923	$327,932	$397,049	$446,606
Commercial	$0	$0	$127,112	$451,331	$1,003,364	$1,289,288	$1,852,650
Total overfloor damages	**$29,394**	**$66,088**	**$1,229,666**	**$3,896,939**	**$8,133,734**	**$11,914,225**	**$18,018,210**
Road Damage							
Major	$344,917	$568,708	$840,488	$1,128,443	$1,278,781	$1,452,690	$1,626,901
Minor	$127,883	$209,812	$431,216	$661,443	$784,818	$896,505	$1,029,560
Unsealed	$56,362	$86,404	$115,991	$139,338	$158,047	$174,536	$191,440
Total road damages	**$529,162**	**$864,924**	**$1,387,695**	**$1,929,224**	**$2,221,647**	**$2,523,731**	**$2,847,901**
Total	**$764,934**	**$1,448,926**	**$4,303,151**	**$9,313,736**	**$14,259,569**	**$19,228,453**	**$26,464,649**

Mitigation Assessment

A range of mitigation options were considered for the Castle Creek system, these ranged from physical modification of the levee through to management of the sediment within the system. The mitigation options were focussed more on the management of the system rather than to provide additional protection to the township.

The mitigation options 1 and 2 demonstrated that modifying the levee to utilise the additional railway culverts increases the flooding on a number of properties but does not reduce the peak flood depths upstream of the railway embankment sufficiently to benefit the buildings adjacent to the Euroa Main Road. Both mitigation options led to increased damages associated with flood events.

Mitigation Options 3a, 3b and 3c examined the impact of sedimentation and structure blockage. The assessment identified that if the structures block by up to 50% then there are some small areas of increased damages and the total damage increases. If channel clearing is undertaken then there is not expected to be a substantial change in the flood behaviour.

The final mitigation assessment examined the erosion and scour assessment for the range of design events. The velocity in Castle Creek is estimated to be sufficient to mobilise sediment accumulated in the main channel and structures assuming this accumulated sediment is not locked in via vegetation growth between events.

Velocities in the main channel and structures in flood events as frequent as the 20% AEP event are expected to exceed 1 m/s which is sufficient to mobilise coarse sand.

Recommended Mitigation Approach

Of the mitigation options assessed Mitigation options 1 and 2 examined levee realignment solutions, both of these options increased damages and are not appropriate for reducing damages on upstream properties. As such these are not recommended to implement.

The recommended mitigation approach is periodic assessment and clearing of the structures under Euroa Main Road and under the main railway embankment bridge. If checks are undertaken to ensure that there is no vegetation locking the sediment in place then during flood events the velocity of flood water is expected to scour and erode the built up sand pockets (mitigation option 4 assessed the mobilisation requirements). If the structures become excessively blocked (>50% blockage) it is recommended that they are cleared of sediment. See Section 7.3.2 for the recommended inspection strategy and clearing triggers.

Clearing of the channel away from the structures for Castle Creek has some impact on the flood behaviour but is likely to involve impacting on the natural system and is not recommended. Vegetation clearing and sediment clearing in these areas is not expected to change the flood behaviour and damages significantly. Periodic sand removal of the main channel may be required in order to control the sediment build up at the structures and this should be monitored in an ongoing manner.

The current levee for Castle Creek protects up to the 0.2% AEP event (albeit with no freeboard). Current standard practice for urban levees is a minimum of 600 mm freeboard above the 1% AEP peak level. For the Castle Creek level the majority of the levee meets this criteria, however there is a small section near the freeway end of the catchment which should be raised to meet these requirements. See Figure 7-4 for the current freeboard and location required additional levee freeboard.

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Glossary

1D	1D – One Dimensional. In this report 1D refers to a hydraulic model where the flow direction of water is only calculated in one direction. A 1D model is often used to reduce model run times.
2D	2D – Two Dimensional. In this report 2D refer to a hydraulic model where the flow direction of water is calculated in two directions. Two dimensional models are used to model floodplains and overland flows.
Annual Exceedence Probability (AEP)	Refers to the probability or risk of a flood of a given size occurring or being exceeded in any given year. A 90% AEP flood has a high probability of occurring or being exceeded; it would occur quite often and would be relatively small. A 1% AEP flood has a low probability of occurrence or being exceeded; it would be fairly rare but it would be relatively large.
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Catchment	The area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.
Design flood	A design flood is a hypothetical flood that is used to plan for floods. Design floods are described in terms of how likely they are to occur (see definition for AEP).
Development	The erection of a building or the carrying out of work; or the use of land or of a building or work; or the subdivision of land.
Digital Terrain Model (DTM)	A Digital Terrain Model is a representation of the ground surface excluding objects such as buildings, trees, grass etc. In this report this DTM is in the form of a grid with each grid cell representing the surface elevation at that location.
Discharge	The rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow, which is a measure of how fast the water is moving rather than how much is moving.
Flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or overland runoff before entering a watercourse.
Flood Frequency Analysis	The calculation of the statistical probability that a flood of a certain magnitude for a given river will occur in a certain period. This analysis is undertaken on recorded gauge data.
Floodplain	A floodplain is the low-lying land bordering a river, stream, lake or coastal zone over which water will flow during a flood. Flooding is caused by runoff from heavy or prolonged rainfall exceeding the capacity of rivers and drainage systems.
Geographical information systems (GIS)	A system of software and procedures designed to support the management, manipulation, analysis and display of spatially referenced data.
HECRAS	Hydrologic Engineering Centres River Analysis System. HEC-RAS is a computer program that models the hydraulics of water flow through natural rivers and other channels.
Hydraulics	The term given to the study of water flow in a river, channel or pipe, in particular, the evaluation of flow parameters such as stage and velocity.
Hydrograph	A graph that shows how the discharge changes with time at any particular location.

Hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
LiDAR	Light Detection and Ranging (LiDAR) is a technology that uses laser pulses to generate large amounts of data about terrain and landscape features.
Losses	For the hydrology, losses refer to the volumes of rainfall that are lost within a catchment prior to the runoff reaching the main flow paths through the catchment. This water is lost as evaporation, evapotranspiration, infiltration and surface storage.
Mathematical/computer models	The mathematical representation of the physical processes involved in runoff and stream flow. These models are often run on computers due to the complexity of the mathematical relationships. In this report, the models referred to are mainly involved with rainfall, runoff, pipe and overland stream flow.
MSS	Municipal Strategic Statement. A concise statement of the key strategic planning, land use and development objectives for a municipality and includes strategies and actions for achieving those objectives.
Planning Overlays	Planning overlays are used to control development within areas at risk of flooding. Four planning overlays are used in Victoria: Urban Floodway Zone (UFZ), Floodway Overlay (FO), Land Subject to Inundation Overlay (LSIO) and Special Building Overlay (SBO).
Pluviograph	A rainfall gauge that records rainfall depth at 6 minute intervals continuously.
Probability	A statistical measure of the expected frequency or occurrence of flooding. For a fuller explanation see Annual Exceedence Probability.
Rainfall excess	See definition of “Runoff”.
Risk	The possibility of something happening that impacts your objectives. It is the chance to either make a gain or a loss. It is measured in terms of likelihood and consequence (AS/NZ 4360). For this report risk is used to describe both likelihood and consequence of flooding.
RORB	RORB is a general runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs (Laurenson et al., 2005)
Roughness	The resistance of the surface to the flow of water over it. For the hydraulic model the resistance is measured using Manning’s Roughness.
Runoff	The amount of rainfall that actually ends up as stream or pipe flow, also known as rainfall excess. This is rainfall less losses equals rainfall excess.
Stage Discharge Relationship	A relationship between a known water level at a location and the corresponding flow rate. This is used to translate recorded flood depth to flow rates.
Topography	A surface which defines the ground level of a chosen area.
WBNM	WBNM is a hydrologic catchment model that generates flood hydrographs from rainfall and catchment parameters.
Zoning	Zoning is the process of planning for land use by a locality to allocate certain kinds of structures in certain areas. Zoning also includes restrictions in different zoning areas, such as height of buildings, use of green space, density (number of structures in a certain area), use of lots, and types of businesses.

Abbreviations

Abbreviation	Full Description
AAD	Average Annual Damage
AEP	Annual Exceedence Probability
AHD	Australian Height Datum
AMG	Australian Map Grid
ARI	Annual Recurrence Interval
AR&R	Australian Rainfall and Runoff
AWS	Automatic Weather Station
BoM or 'the Bureau'	Bureau of Meteorology
CFA	Country Fire Authority
CMA	Catchment Management Authority
DPI	Department of Primary Industry
DSE	Department of Sustainability and Environment
DTM	Digital Terrain Model
ERTS	Event-Reporting Radio Telemetry System
FFA	Flood Frequency Analysis
FO	Floodway Overlay
GEV	Generalised Extreme Value
GenPareto	Generalised Pareto Distribution
GIS	Geographical Information System
Goulburn Broken CMA	Goulburn Broken Catchment Management Authority
GSAM	Generalised Southeast Australia Method
GSDM	Generalised Short Duration Method
HECRAS	Hydrologic Engineering Centres River Analysis System
Hwy	Highway
IC	Incident Control
ISC	Index of Stream Conditions
LSIO	Land Subject to Inundation Overlay
LiDAR	Light Detection and Ranging
LPIII	Log Pearson Type III Distribution
MERO	Municipal Emergency Resource Officer
MFEP	Municipal Flood Emergency Plan
MGA	Map Grid of Australia
ML/d	Megalitres per day
MSS	Municipal Strategic Statement
PMF	Probable Maximum Flood
PSM	Permanent Survey Mark
RDO	Regional Duty Officer
RFA	Rainfall Frequency Assessment
SBO	Special Building Overlay

SMS	Short Message Service
SOP	Standard Operating Procedure
SWMP	Surface Water Monitoring Partnership
TBRG	Tipping Bucket Rain Gauge
TFWS	Total Flood Warning System
UFZ	Urban Flood Zone
VFD	Victorian Flood Database
VICSES	Victoria State Emergency Services
VPP	Victoria Planning Provisions

1 Introduction and Study Objectives

The township of Euroa requires the development of additional flood intelligence and mapping for the Seven Creeks and Castle Creek catchments to manage their risk of flooding. This investigation follows on from the development of the Euroa Water Management Scheme (July, 2000). The proposed works from this Scheme were completed in 2012 and this investigation aims at examining the revised impacts of these works on the flooding for the Euroa Township and potential management options for the future flood protection of Euroa.

The detailed study objectives are outlines in Section 1.3 but the primary objectives are categorised as Stage 1 and Stage 2.

> Stage 1 – Hydrologic and hydraulic modelling (calibration and validation); and the hydraulic assessment (performance) of the Castle Creek levee, including analysis for potential improvements.
>
> Stage 2 – The development of the flood intelligence, MFEP Appendices, municipal flood response plan and land use planning maps.

1.1 Catchment Background

Euroa lies at the foot of the Strathbogie Ranges approximately 160 km north of Melbourne on the Hume Highway. Euroa has a population of around 4,000. Euroa has two major catchments that contribute to flooding including Seven Creeks and Castle Creek. Seven Creeks is the larger of the two catchments at 332 km^2 and Castle Creek at 80 km^2. Figure 1-1 shows the catchments for both Seven Creeks and Castle Creek respectively.

The topography of the site ranges from 170 mAHD within Euroa up to in excess of 800 mAHD in the upper headwaters near the town of Creek Junction. The flooding characteristics are a result of the high rainfalls associated with the Strathbogie Ranges and flood warning times are less than a day.

The catchments are mostly cleared with the largest land use for agriculture, cropping, sheep and cattle grazing and horse studs. Within the catchment there are areas set as National Parks including the Mt William Flora and Fauna Reserve.

Castle Creek has ongoing issues with sedimentation of sands along the system which migrate from the upper catchment. The investigation of the sedimentation and management options to mitigate this problem forms part of this investigation.

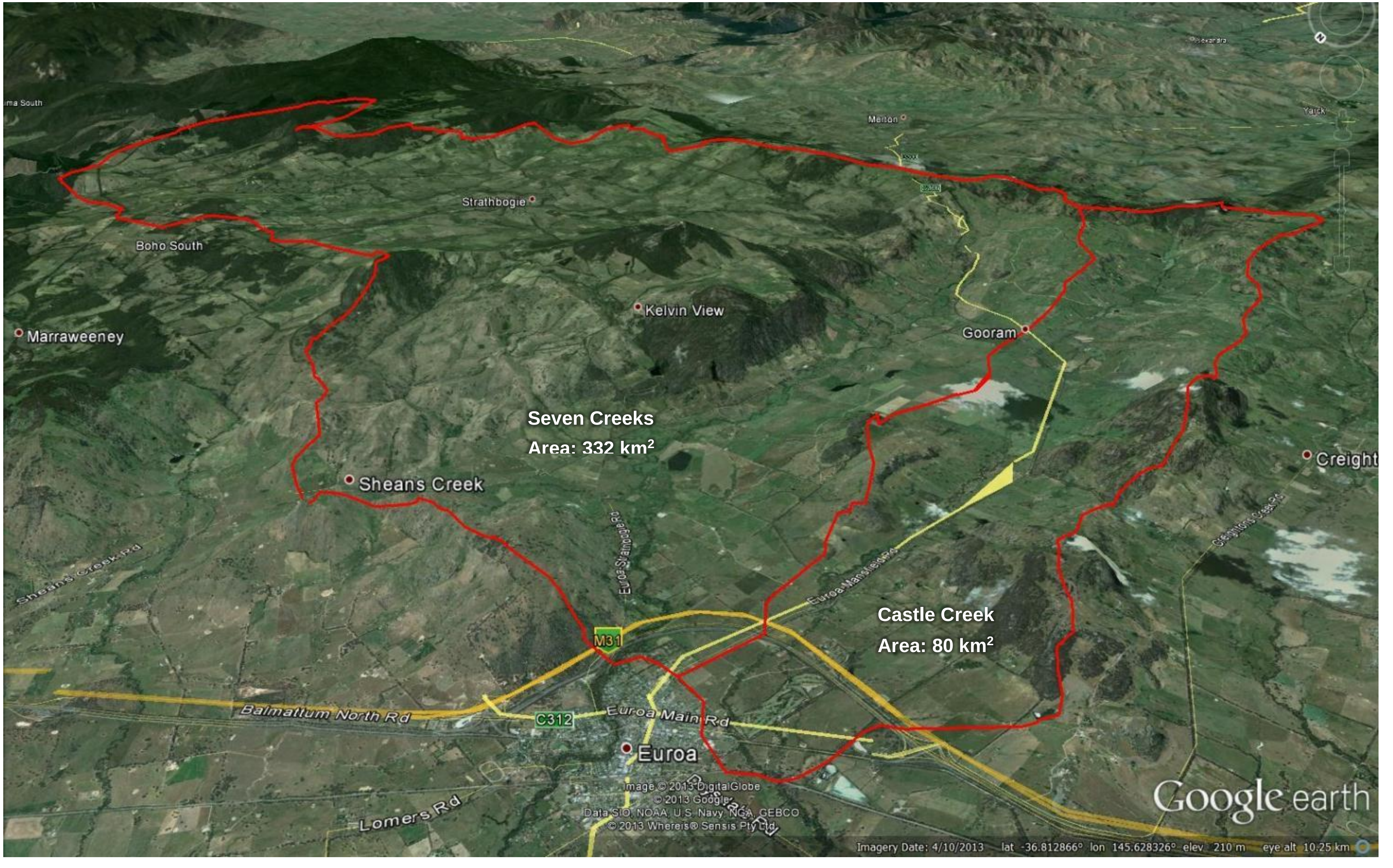


Figure 1-1 Seven Creeks and Castle Creek catchment boundaries

1.2 Recent Floods

Since the 1900s, there have been a number of large floods which have been ranked in order of magnitude in Table 1-1. The largest flood recorded at Euroa was the 1916 event.

Table 1-1 Peak historical flood events experienced at Euroa

Rank	Year	Peak flow in Seven Creeks		Comments
1	1916	~34,040 ML/d	~394 m^3/s	Largest flood on record for Euroa
2	1993	24,615 ML/d	285 m^3/s	Overfloor flooding of 150 habitable buildings and over 550 properties were impacted. Damages exceeded $1.4m.
3	1992	17,185 ML/d	199 m^3/s	
4	2010	16,407 ML/d	190 m^3/s	First major flood with the newly constructed Castle Creek levee in place. The levee freeboard was encroached and concerns over the blocking of railway culverts was raised. A handful of buildings experienced overfloor flooding.
5	1986	12,528 ML/d	145 m^3/s	
6	1984	12,286 ML/d	142 m^3/s	

Euroa sits between two catchments and during major flood events is inundated by both Seven Creeks and Castle Creek. There are a number on anabranches that flow through the township itself. The construction of the Castle Creek levee in recent times has led to a number of these anabranches being cut off and additional protection to be offered to the south-west side of Euroa.

In recent times the 1993 event is the largest event to have impacted Euroa. This flood caused overfloor flooding to over 150 habitable buildings and over 550 properties. Damages are estimated at greater than $1.4m.

The most recent event was the 2010 flood which resulted in a handful of overfloor flooding of buildings. However, concerns were raised during this event as the freeboard for the recently constructed Castle Creek levee was compromised and that has led to some of the objectives of this investigation to determine if the levee height is appropriate. In addition concerns were raised that the levee circumvents some culverts under the railway line which may need to be utilised.

1.3 Study Objectives

The key objectives of this study include:

- Review the hydrology and flood modelling and prepare new flood inundation maps (FIMs) for emergency management and land-use planning purposes;
- Create new flood intelligence data (stage versus consequence). It is expected that this data is tied with the relevant flood maps;
- Augment floor level database through additional survey;
- Review the performance of the Castle Creek levee and investigate the appropriateness of the levee alignment and height;
- Review the performance of sediment removal programme of Castle Creek;
- Independently apply storm events over the township area to inform overland flow paths without riverine flooding;

- Provide information and prepare community information awareness and education brochures in line with the FloodSafe Initiative; and
- Augmentation of telephone alert system including opt-out system.
- Provide site specific flood information for properties.

1.4 Study Area

The study area for the project includes a proposed detailed grid for the urban area of Euroa and a less detailed grid for the semi-rural surrounds. The topography was derived from DEPI LiDAR captured in 2010. The areas upstream of the Hume Freeway are included to ensure the Freeway bridges are modelled, including the gauge located on Castle Creek at Telfords Bridge.

The study area as per the brief is shown in Figure 1-2.

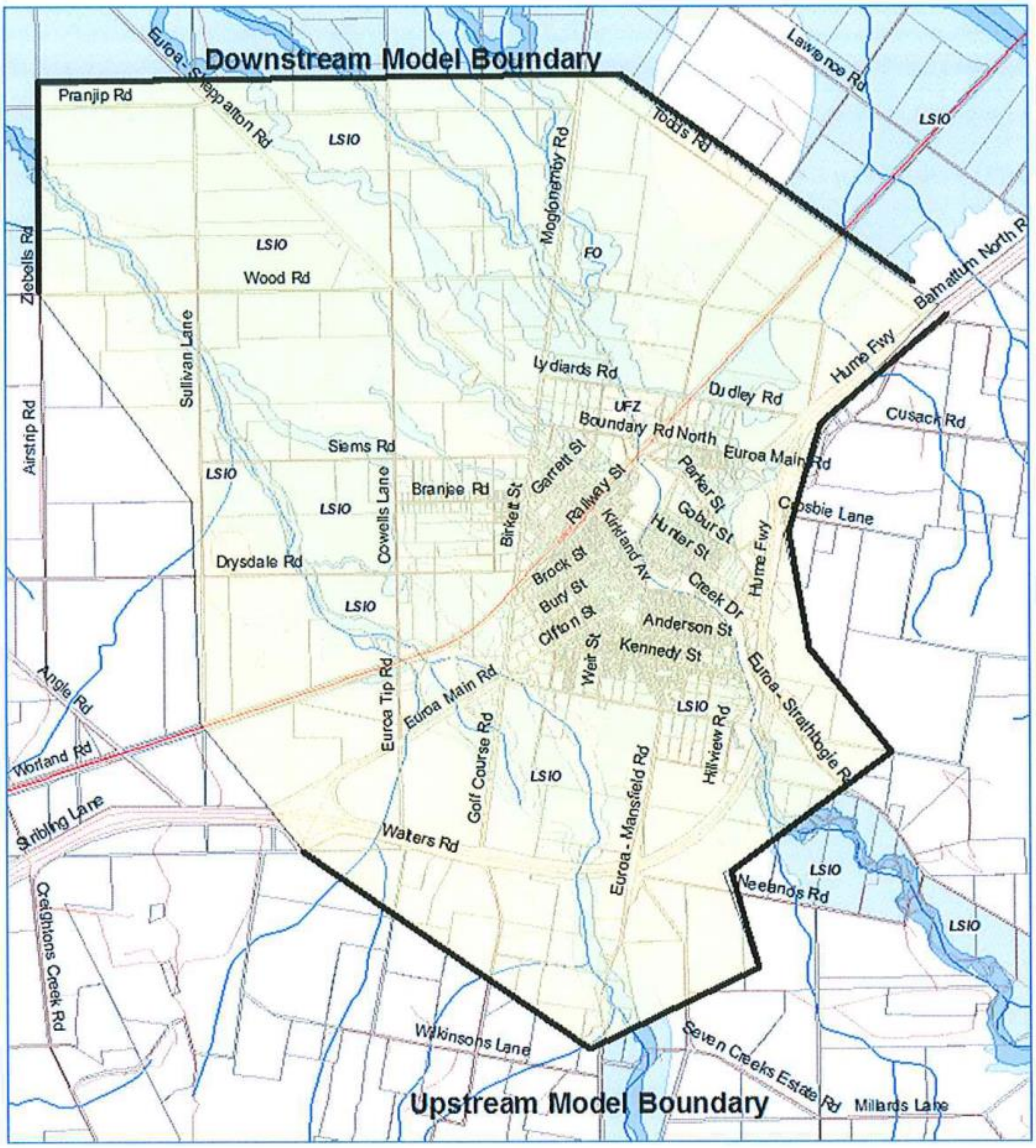


Figure 1-2 Preliminary study area as per the project brief

2 Available Data

A substantial amount of data was required for the development of the Euroa Post Flood Mapping and Intelligence Project. This section aims at summarising the data utilised for the project. The data has been supplied from both the Strathbogie Shire Council and the Goulburn Broken CMA.

The bulk of the data was supplied from the Goulburn Broken CMA in ArcGIS format.

2.1 Data supplied

2.1.1 Goulburn Broken CMA

The following datasets were provided from the Goulburn Broken CMA for this project and included:

- Aerial laser survey data (LiDAR, 2011)
- Planning scheme information.
- Peak flood level Geo Database.
- Cross section plans and cross sections for Castle Creek and Seven Creeks.
- Floor levels captured from the Euroa Floodplain Management Study (excluded approximately 300 buildings that may require survey).
- Flood Photography
    - Aerial photography for the 1993 flood event
    - Oblique photography for the 1992 event

2.1.2 Strathbogie Shire Council

The Strathbogie Shire Council supplied the following data to Cardno for this project:

- Aerial photography from 2009 (full study area) and 2011 (high detail).
- VicMap layers for the cadastre, roads and properties.
- As built Castle Creek feature survey.
- Extended information for the Castle Creek levee.
- Planning layers including the LSIO
- Key components of the Euroa Shire Council stormwater system.

2.1.3 Other Data Collected

Cardno also collated data from the following locations for this investigation:

- Streamflow levels and flows from the Victorian Data Warehouse
- Site visit photos of the catchment from the site visit
- Rainfall and pluviograph information from the BoM
- Land use mapping from the DEPI
- NASA SRTM broad scale contour data
- VicRoads plans for culverts and bridges under the Hume Freeway
- Reports including:
    - Euroa Floodplain Management Study: Final Report (SKM, 1997)
    - Euroa Water management Scheme (Euroa Floodplain Management Community Consultative Committee, 1999).
    - Euroa Water management Scheme: Technical Report (Euroa Floodplain Management Community Consultative Committee, 1999).
    - Euroa Water management Scheme: Environmental Report (Department of Natural Resources and Environment, 1999).
    - Shepparton Mooroopna Floodplain Management Study (SKM, 2002).
    - Violet Town Flood Study (Water Technology, 2007).
    - Images from the 2010 floods from various sources.

3 Survey

The survey information required included both additional cross sections near the Telfords Bridge streamflow gauge and additional floor levels. No survey was required to be captured to define the bridges and structures throughout the study area as these were sourced from plans made available for the study from the council, Goulburn Broken CMA and VicRoads. The data captured is summarised in the following sections and the survey has been supplied as part of the deliverables to this project.

3.1 Castle Creek – Upstream of Telfords Bridge

In order to obtain a reasonable estimate of flows approaching Telfords Bridge along Castle Creek, ground survey of the channel was required. This was due to insufficient LiDAR information being present upstream of the bridge to ensure boundary influences did not alter the results. The key driver of this survey was to enable a flood rating curve at Telfords Bridge to be developed. Figure 3-1 shows the cross section locations developed as part of this survey.

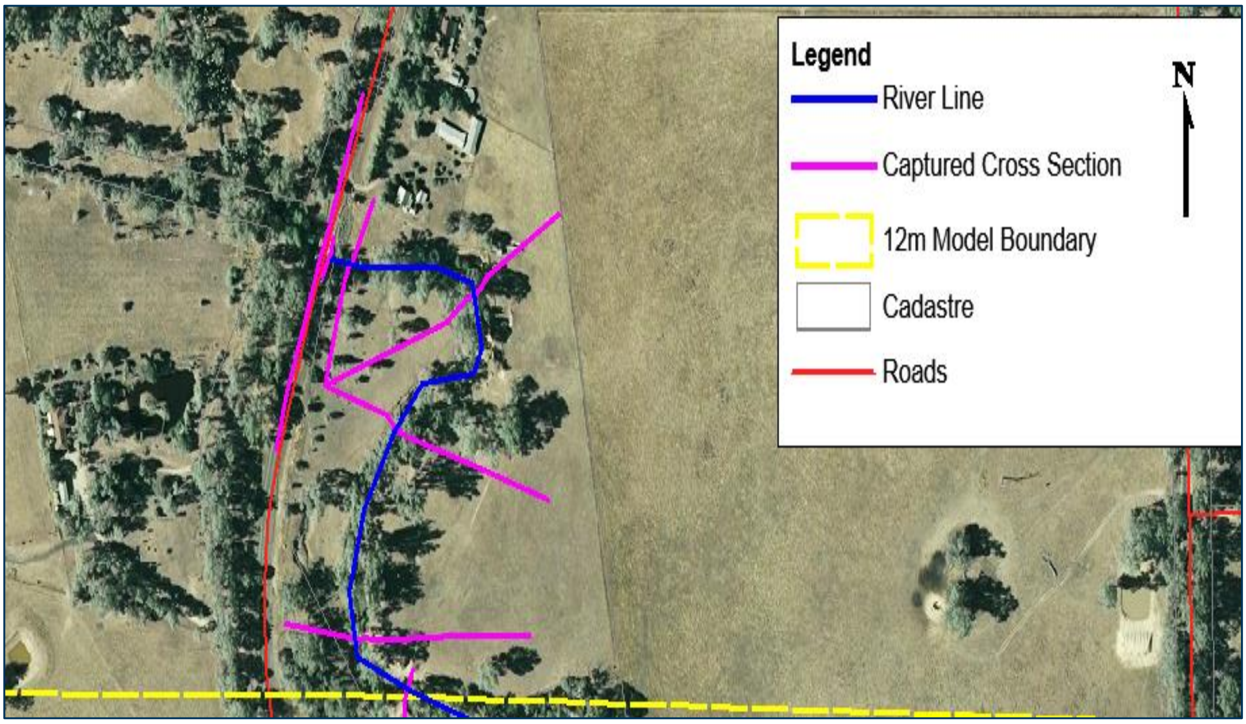


Figure 3-1 Surveyed cross sections and structures for the Telfords Bridge on Castle Creek gauge

3.2 Floor Level Survey

Additional floor level surveyed was captured as part of this flood investigation. This was required as missing data was identified, the flood results have changed and there have been additional dwellings constructed since the previous floor level survey was completed. At the start of the project Cardno received 1,369 data points for floor levels. One floor level data point was removed at 1-7 Simpsons Lane as this building has been removed.

Cardno obtained additional floor level survey based on the identification of properties that are within the 1% AEP flood extent. It was deemed too costly to capture all floor levels within the PMF flood extent. An additional 146 floor levels were captured as part of the survey. The locations of the existing floor level information and the additional captured floor levels are shown in Figure 3-2.

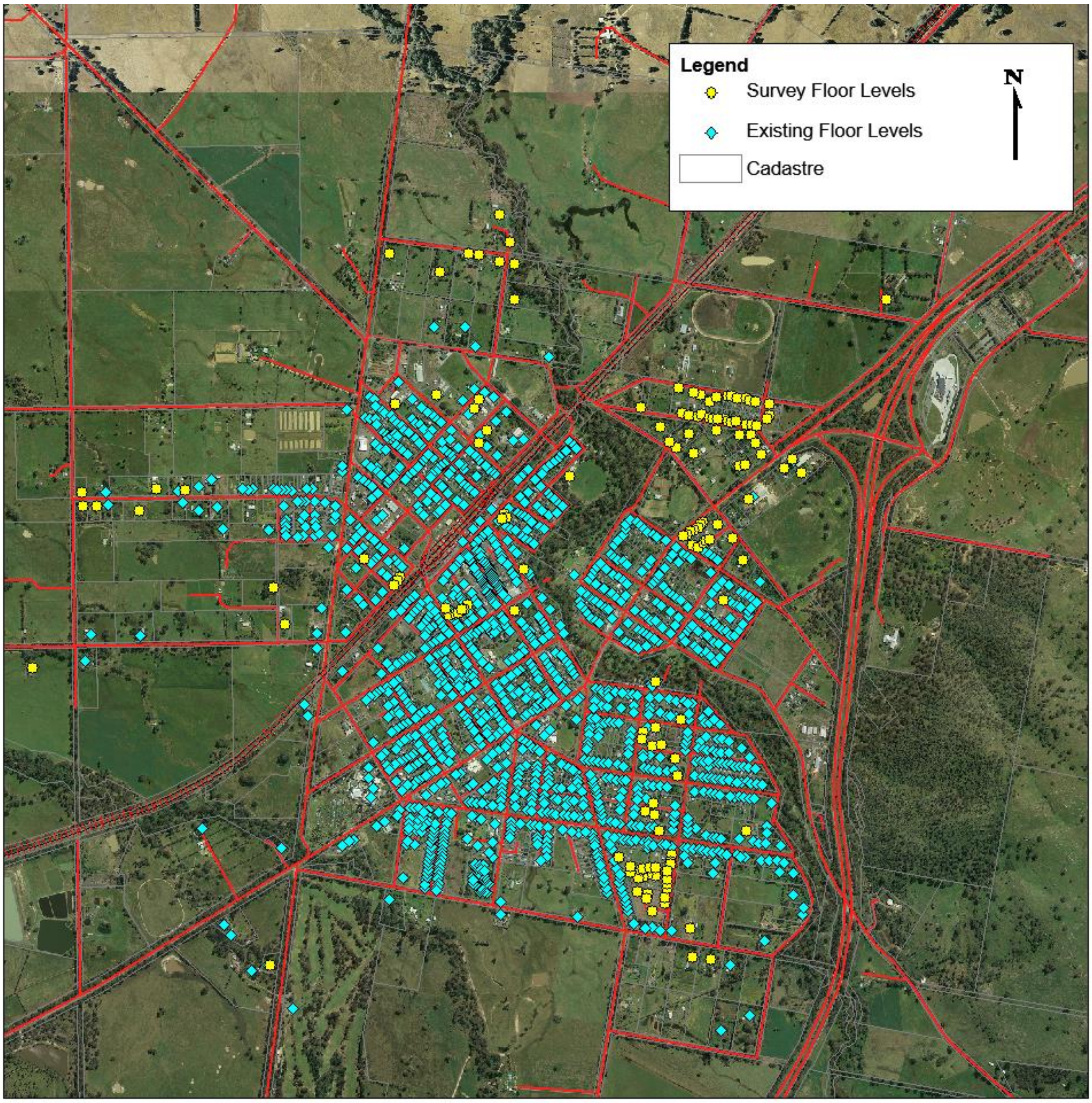


Figure 3-2 Existing and surveyed floor levels for buildings in Euroa

4 Hydrology

For this investigation the two systems, Seven Creeks and Castle Creek, were required to be represented using a hydrological model. Currently there is an URBS model in operation by the Bureau of Meteorology (BoM), however the BoM has advised Cardno on 9th April 2013 that these models would not be made available as the BoM have a policy of not providing forecasting models to external parties. As this model was unavailable Cardno developed a RORB model for both the Seven Creeks and Castle Creek systems upstream of Euroa.

This section outlines the development of the hydrological models, the calibration of these models and the development of the design events for the study.

This section of the report includes the following information, see Table 4-1.

Table 4-1 Outline of the hydrology development

Section	Title	Description
4.1	Available data	Outlining the available resources to develop the hydrology and design events.
4.2	Approach	Providing a synopsis of the approach for developing the design events.
4.3	RORB Model Development	This section outlines the development of the RORB models.
4.4	RORB Calibration	This section outlines the calibration of the RORB models.
4.5	Flood Frequency Analysis (FFA)	The assessment of the gauged streamflow records.
4.6	Design Events	A summary of the design events obtained using the FFA peaks and the calibrated RORB model.
4.7	Castle Creek Historic Analysis	An assessment of past events on Castle Creek to provide context to the historical flooding and an improved understanding of the hydrology through this system.
4.8	Sensitivity	An assessment of the sensitivity of the RORB model to variations in key model parameters to develop an understanding of the uncertainty within the hydrological modelling.
4.9	Climate Change	Outline of the development of the climate change for the catchments.
4.10	Probable Maximum Flood (PMF)	Outlines the development of the PMF as derived using the GSAM, GSDM and the Probable Maximum Precipitation (PMP) for the catchment.
4.11	Flood Warning Time	Outlines the approximate flood warning times from the assessment of the historical flood events and from flood modelling.

4.1 Available Data

4.1.1 Streamflow Data

Within the Seven Creeks and Castle Creek catchments upstream and within Euroa there are four streamflow gauges. The four streamflow gauges are summarised in Table 4-2 and shown in Figure 4-1.

Table 4-2 Streamflow gauges available for the study

Gauge	Gauge Name	Area (km^2)	Data	Start date	End Date
405233	Spring Creek at Strathbogie	28	Daily Avg.	May 1964	Nov 1982
			Inst. Flow	May 1974	Nov 1982
405234	Seven Creeks at D/S of Polly McQuinn Weir	153	Inst. Flow	Jun 1965	Present
405237	Seven Creeks at Euroa	332	Daily Avg.	1963	1973
			Inst. Flow	Nov 1973	Present
405269	Seven Creeks at Kialla West	1,505	Inst. Flow	Jun 1977	Present
405246	Castle Creek at Arcadia	164	Inst. Flow	Dec 1973	Present
405308	Castle Creek at Telfords Bridge	68	Station Level	May 2005	Jul 2012
			Daily Avg.	Oct 2006	Jul 2012
			Inst. Flow	Apr 2011	Jul 2012

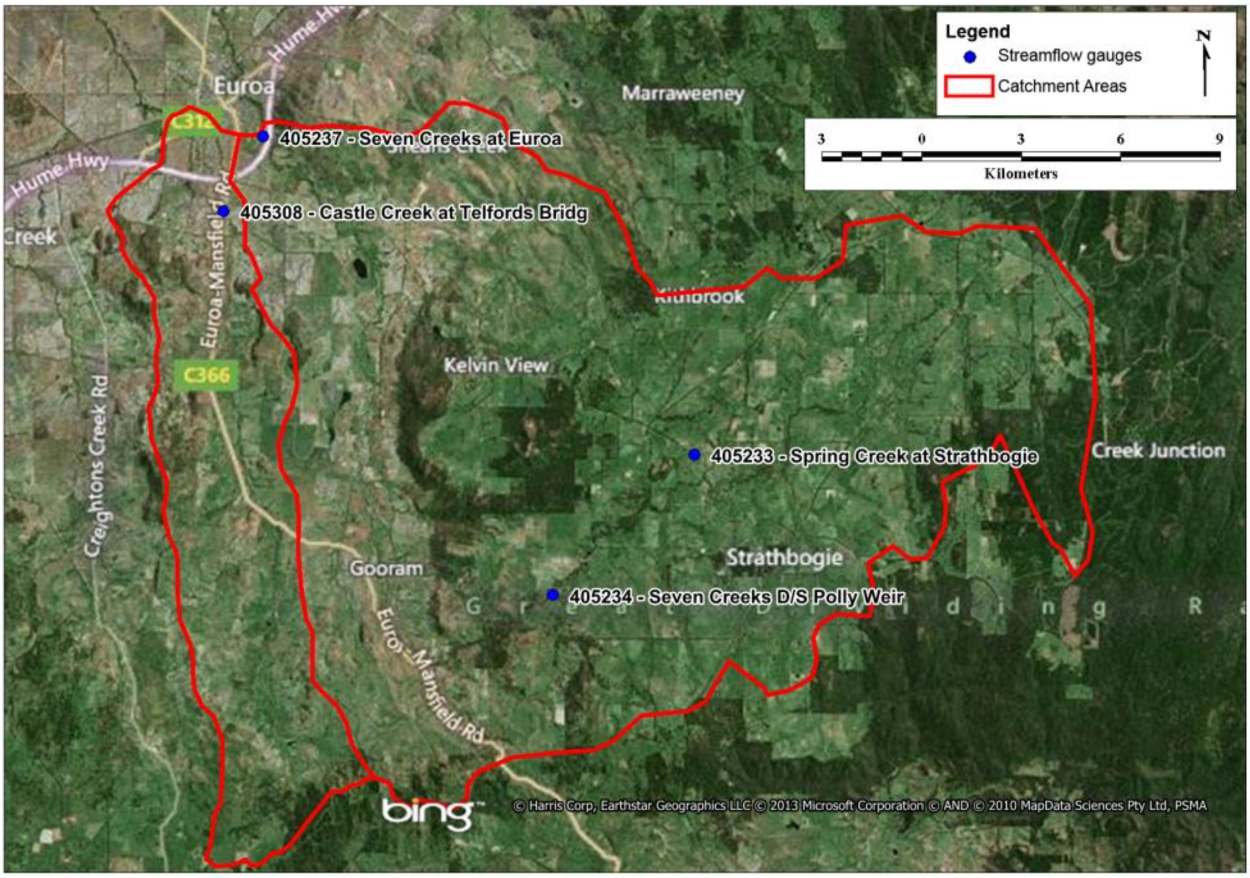


Figure 4-1 Available streamflow gauges for the Seven Creeks and Castle Creek models

The Seven Creeks catchment had the majority of the streamflow information with three gauges. The primary gauge for this catchment was the Seven Creeks at Euroa (405237), which had a record of 40 years. Importantly this record was an instantaneous time series which allows for the daily peak flows to be extracted more reliably.

Seven Creeks at D/S Polly McQuinn Weir (405234) had a similar length of record, however examination at this site revealed that the gauge had a maximum flow rating of approximately 60 m^3/s. Above this level the streamflow gauge is exceeded. The impact of the structure is also unknown as the weir is located upstream of a bridge structure and it is expected that during large flood events these structures interact.

The use of Spring Creek at Strathbogie (405233) gauge was not proposed to be used due to the short record available and due to the limited size of the contributing catchment.

For the Castle Creek catchment the only available gauge was the Castle Creek at Telfords Bridge (405308). This gauge is relatively new and has a limited daily average record from 2006 to 2012. The use of the daily average is difficult in the assessment of the peak flows for Castle Creek as the daily average flows are often less than the instantaneous maximum peak flows for any given day. The station level information is a more reliable measure of the peaks reached during events but as there is no known reliable rating table this level cannot be converted to a flow rate.

4.1.2 Rainfall and Pluviograph Stations

The rainfall and pluviograph records for the Seven Creeks and Castle Creek catchments were well represented with a good spatial distribution of long term rainfall records. The two pluviograph records both covered the streamflow records. The rainfall gauges and pluviographs used within this study are summarised in Table 4-3 and shown spatially in Figure 4-2.

Table 4-3 Rainfall and pluviograph gauges for the Seven Creeks and Castle Creek catchments

Number	Name	Data Type	Start Date	End date
082016	Euroa	Rainfall	01/01/1883	Open
		Pluviograph	12/12/1967	Open
082042	Strathbogie	Rainfall	01/09/1902	Open
		Pluviograph	03/01/1972	Open
082043	Strathbogie North	Rainfall	01/11/1879	Open
082089	Terip Terip	Rainfall	01/01/1959	Open
082096	Baronga	Rainfall	01/10/1937	Open

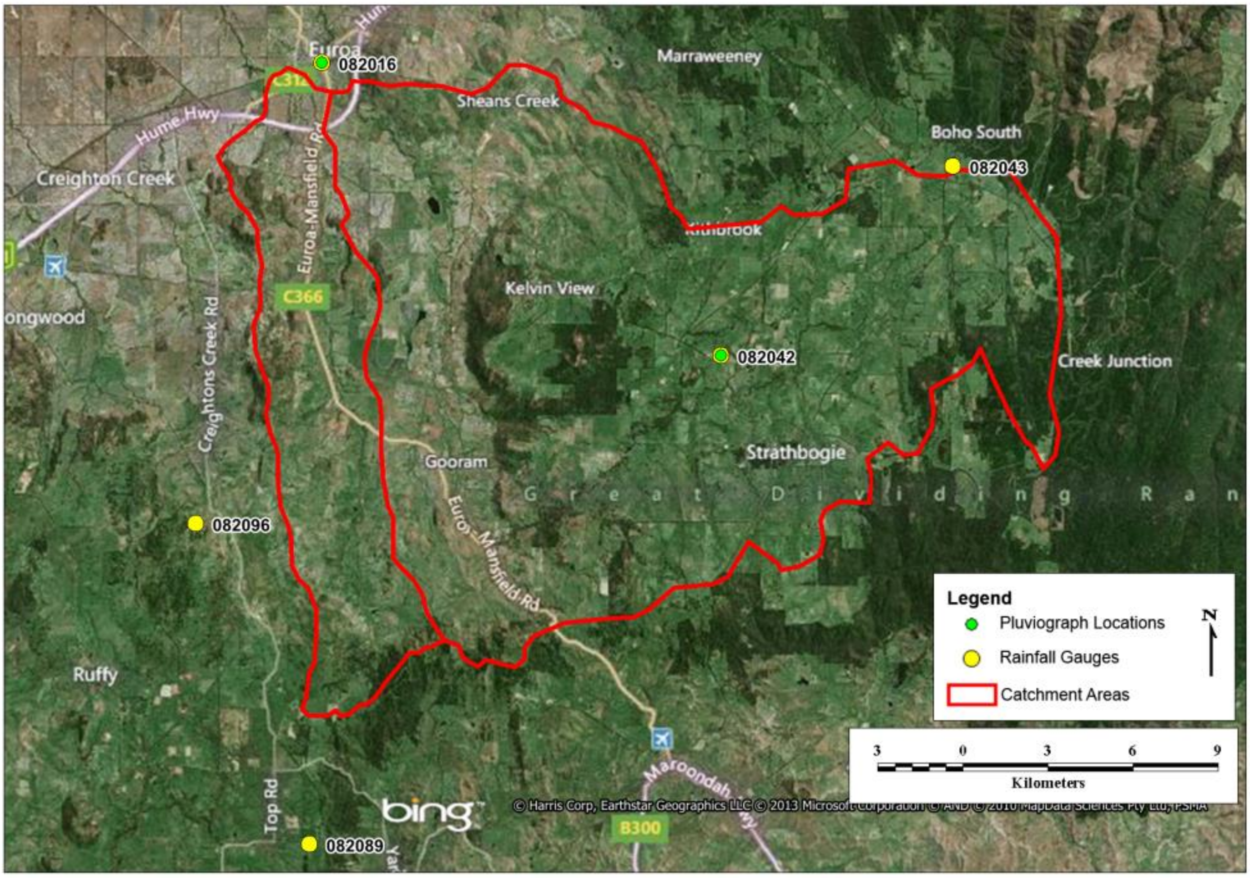


Figure 4-2 Rainfall and pluviograph locations for use in the study

The rainfall gauges cover the low flats near Euroa and also the Strathbogie Ranges. This is important for the hydrological modelling as it is likely that the Strathbogie Ranges have an impact on the rainfall patterns across the catchment. Similarly the available pluviographs within Euroa and at Strathbogie provide an improved understanding of the rainfall patterns during the calibration events.

Overall there is sufficient data to assess the hydrology using the rainfall, rainfall patterns and streamflows.

4.2 Approach

The approach has largely been shaped by the data available. The following process was undertaken to develop the design events for Euroa:

- A Rainfall-Runoff model is to be developed (RORB) for the Seven Creeks and Castle Creek catchment.
  - Calibration of the Seven Creeks model to five (5) events.
  - Obtain a set of parameters suited to the calibrations for the k_c and 'm' parameters.
  - Translate these parameters from the Seven Creeks catchment to the poorly gauged Castle Creek catchment.
- Flood Frequency Analysis (FFA) on the gauge records to determine the statistically derived peak flow rates for the 20%, 10%, 5%, 2%, 1% and 0.5% AEP flood events. These peak flow rates will be used as the targets for the peaks from the design events simulated in RORB.
- The calibrated RORB model, the Intensity Frequency Duration (IFD) parameters (from BoM) for the catchments and the FFA peak flow estimates will be used to generate the design hydrographs. The design hydrographs are developed using the 10 minute to 72 hour durations based on Australian rainfall and Runoff (AR&R, Volume 2). The design events have been assessed against previous flood investigations.

- Sensitivity is to be undertaken on the RORB model to determine the impact of the key parameters on the design events. This involves varying the initial and continuing loss rates and the critical k_c parameter.
- Climate change is assessed for the hydrology through the modification of the IFD parameters. For this investigation design events will be rerun using the increased rainfall intensities.
- The Probable Maximum Flood (PMF) will be derived using the Generalised Southeast Australian Method (GSAM) and Generalised Short Duration method (GSDM). These methods predict the Probable Maximum Precipitation (PMP) which is then temporally distributed using rainfall patterns from AR&R for the 1 to 72 hour duration events. These rainfall events are run through the RORB model to generate the PMF at Euroa.

The details of this method are presented in the subsequent sections.

4.3 RORB Model Development

RORB models were developed for the Seven Creeks and Castle Creek catchments using the MiRORB MapInfo tool in conjunction with RORB v6.15. The sub-catchment delineation was established using NASA SRTM elevations which are a coarse representation for the catchment but are at sufficient detail to define the broad scale hydrologic rainfall-runoff models. The grid can be seen in Figure 4-3.

The Seven Creeks catchment has an area of 337 km^2 upstream of the streamflow gauge 405237 within Euroa. The catchment was divided into 19 sub-catchments as shown in Figure 4-3 and contained one interstation area at gauge 405234. The Castle Creek catchment had an area of 80 km^2 and this was divided into 8 sub-catchments.

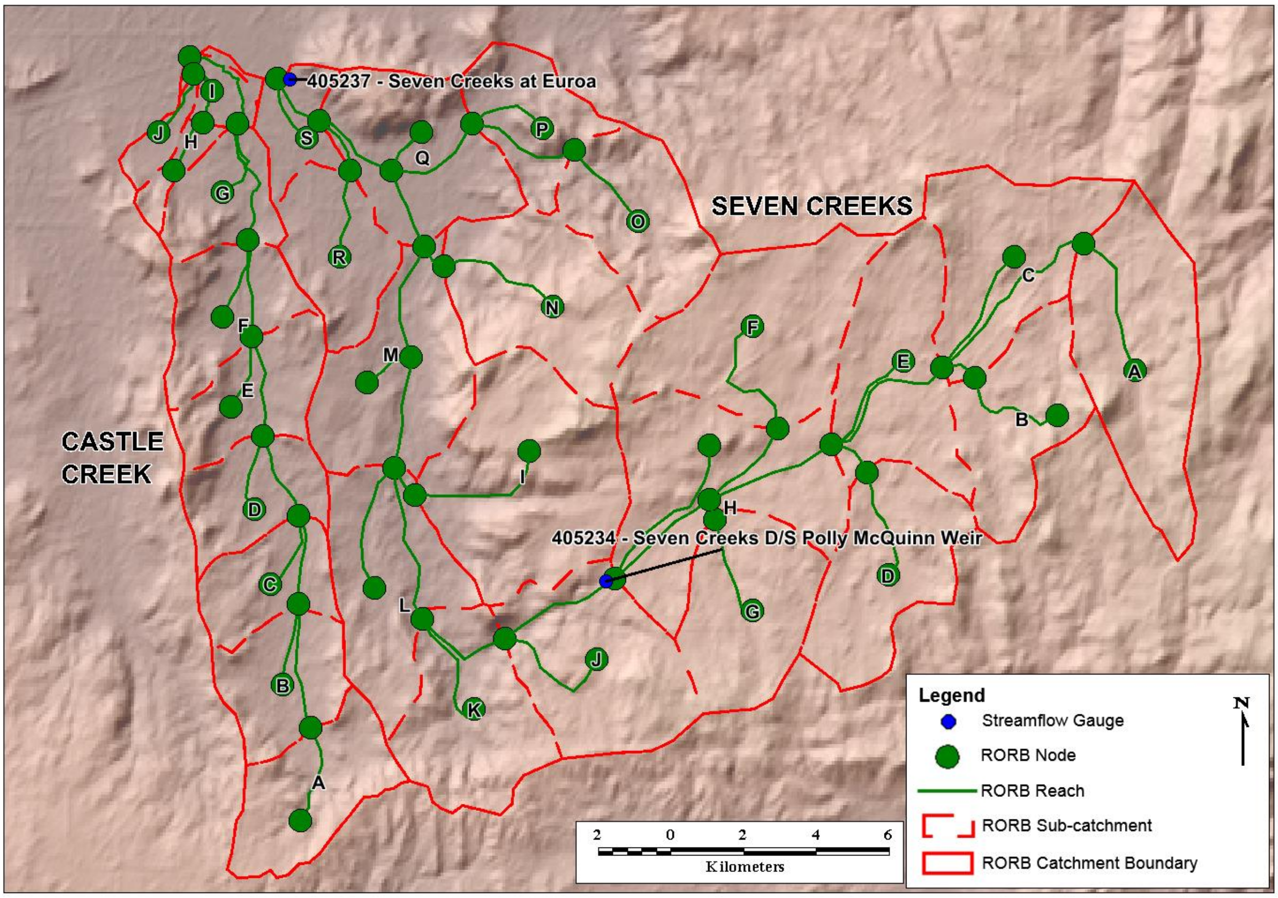


Figure 4-3 RORB models developed for Seven Creeks and Castle Creek

4.4 RORB Calibration Events

The calibration of the RORB model was restricted to the Seven Creeks catchment as the Castle Creek gauge at Telfords Bridge is not reliably rated for flow and has insufficient record to derive calibration events. A discussion and assessment of flood events within Castle Creek has been included for completeness in Section 4.7. The primary streamflow gauge for the calibration was the Seven Creeks at Euroa (405237) gauge. A summary of the gauge is shown in Table 4-4 outlining the data available and the top five recorded events. The peak years included 1993, 1992, 2010, 1986 and 1984. These five events form the basis for the calibration.

Table 4-4 Summary of the streamflow gauge at Euroa for RORB calibration

Site and Code	Seven Creeks at Euroa, 405237		
Years of Gauged Data	40 Years		
Dates	1973 – 2013 (no gaps)		
Data Type	Instantaneous Maximum Flows		
Source	Data Warehouse		
Area	332 km^2		
Highest 5 events		Lowest 5 events	
Year	Flow (m^3/s)	Year	Flow
1993	284.9	1982	2.3
1992	198.9	2006	4.1
2010	189.9	2008	4.2
1986	145.0	2002	4.3
1984	142.2	2009	7.1

For the interstation area upstream of Polly McQuinn Weir, the gauge data (405234) has been extracted for the matching years to the Euroa peaks. This information is summarised in Table 4-5. Unfortunately the gauge at Polly McQuinn Weir has not captured the peak flows at this location as it is likely that the streamflow gauge has been exceeded at approximately 60 m^3/s. This location has still been used in the calibration process as the rising and falling limb of the event has been captured and this provides some guidance on the catchments response to the peak flow events. However, due to the missing peak flow information more focus was placed on the Euroa streamflow gauge in the calibration.

Table 4-5 Summary of the streamflow gauge at Polly McQuinn Weir

Site and Code	Seven Creeks at D/S of Polly McQuinn Weir, 405234		
Years of Gauged Data	48 Years		
Dates	1965 – 2013 (gaps where rating table exceeded)		
Data Type	Instantaneous Maximum Flows		
Source	Data Warehouse		
Area	153 km^2		
Highest 5 Events (matching Euroa years)		Lowest 5 events (matching Euroa years)	
Year	Flow (m^3/s)	Year	Flow
1993	57.5[1]	1982	2.6
1992	61.2[1]	2006	2.2
2010	62.1[1]	2008	3.8
1986	55.0[1]	2002	3.6
1984	61.3[1]	2009	5.8

[1] The gauge was exceeded for all events and the peak flow rate was not captured.

4.4.2 October 1993

The October 1993 event was the largest on record for Seven Creeks at Euroa and this was also the most difficult model to calibrate in RORB. The flood event contained two rainfall bursts which caused Seven Creeks to rise on the 2nd October 1993, the hydrograph falls away over the subsequent days before the main flood event occurs on the 4th October 1993. The hydrograph is shown in Figure 4-4.

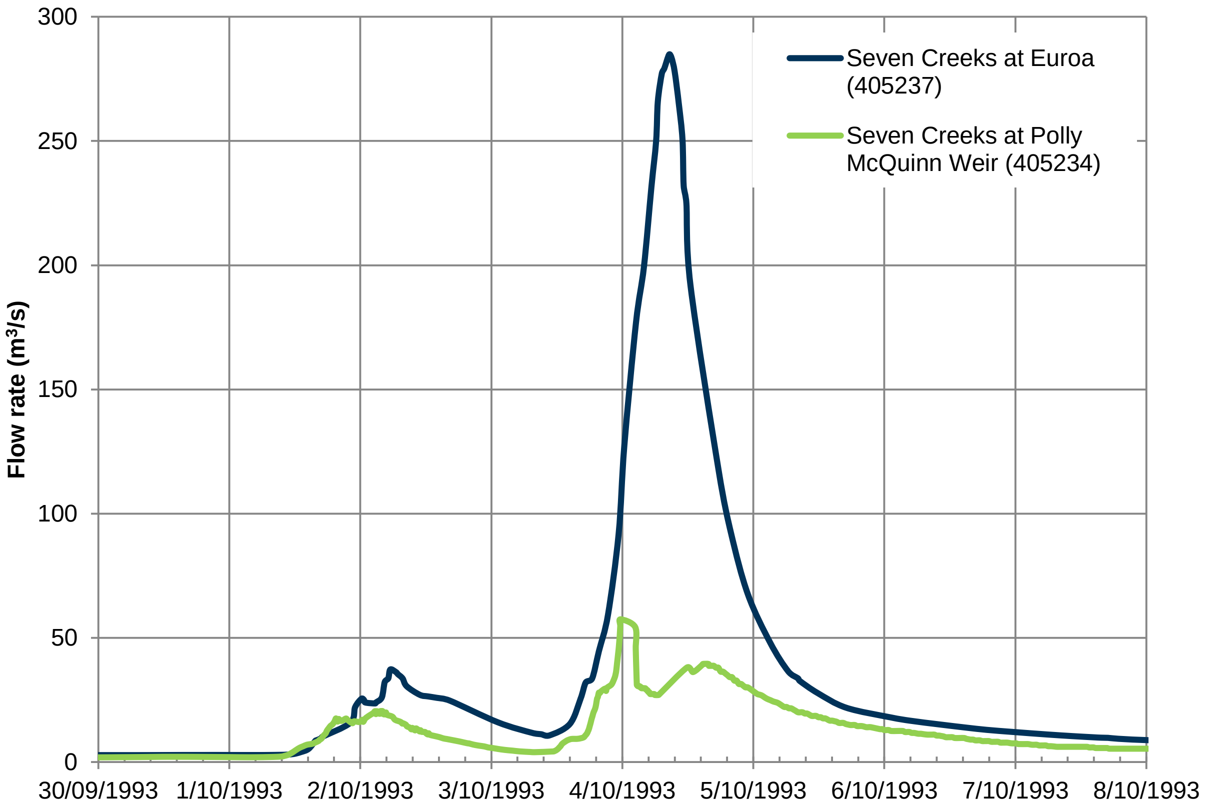


Figure 4-4 October 1993 recorded hydrographs

The rainfall across the catchment was concentrated over the Strathbogie Ranges with rainfall totals in excess of 150 mm, comparatively on the plains the rainfall volumes were below 130 mm over the 5 days for the event. For gauge 82043 is expected that the rainfall gauge was not working correctly on the 3rd October as the rainfall has been accumulated to the 4th October.

Table 4-6 October 1993 rainfall depths

Gauge	30/09	1/10	2/10	3/10	4/10	5/10	Total
82016 - Euroa	2.6	10.0	22.6	4.2	90.4	0	**129.8**
82042 – Strathbogie	2.4	10.0	37.2	3.2	96.4	0.2	**149.4**
82043 – Strathbogie North	2.6	11.0	39.0	0	122.6	4	**179.2**
82096 – Baronga	2.0	9.6	30.4	10.2	78.0	0	**130.2**
82089 – Terip Terip	1.8	16.6	27.6	6.2	75.2	0	**127.4**

As the rainfall is a daily total captured from 9am to 9am, it was distributed using the two pluviographs within the catchment. As for rainfall gauge 82043, the pluviograph at this location was not recording information from 9 am on the 2nd October to 9 am on the 3rd October. For this period the accumulated pluviograph volume was linearly distributed over the non-recording period. For pluviograph 82016 in Euroa the pluviograph was not working from 9 am on the 30th September until 9 am on the 1st October. The record also has no data for the 2nd October.

Overall the pluviograph information for the event was poor with the critical rainfall periods not adequately captured.

In order to improve the calibration the event was simulated within RORB with multiple bursts. The resulting calibrated hydrographs are shown in Figure 4-5. The Polly McQuinn Weir gauge was poorly calibrated, however it appears that this streamflow record is directly impacted by the Polly McQuinn Weir structure. As the primary focus of this calibration was for the peak flows at the Euroa township a greater importance was put on matching the recorded hydrograph at this location.

Table 4-7 Calibrated parameters for the 1993 event

Parameter	Polly McQuinn Weir	Euroa
k_c	25	
m	0.8	
IL (burst 1) (mm)	40.0	40.0
CL (burst 1) (mm/hr)	0.0	0.0
IL (burst 2) (mm)	40.0	35.0
CL (burst 2) (mm/hr)	0.6	0.0

The hydrograph at Euroa is well matched with the peak flow rate being exactly matched. The timing of the peak is good and is within an hour of the recorded hydrograph peak. The absence of the pluviograph information (as previously discussed) causes the hydrograph to deviate from the recorded both before the main peak and following this peak but overall this calibration was successful. It should be noted that no baseflow was removed from the recorded hydrograph and this is not included in the RORB model.

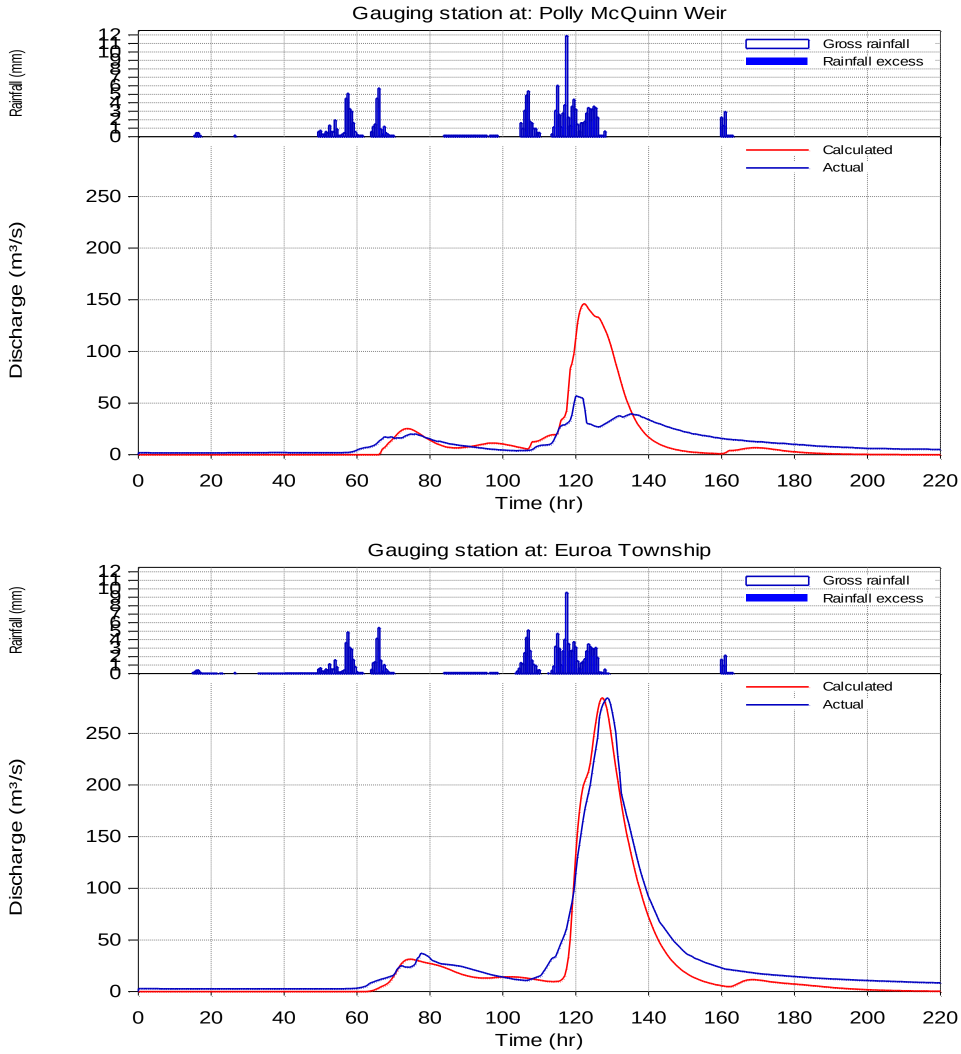


Figure 4-5 Calibrated RORB results for the October 1993 flood event

4.4.3 October 1992

The October 1992 event peaked at 200 m³/s, the peak at Polly McQuinn Weir was unknown as the gauge was exceeded. The hydrographs for these events are shown in Figure 4-6. The event was characterised by a sharp rising limb to the peak on the 18th October which then fell away at a reduced rate on the 19th October.

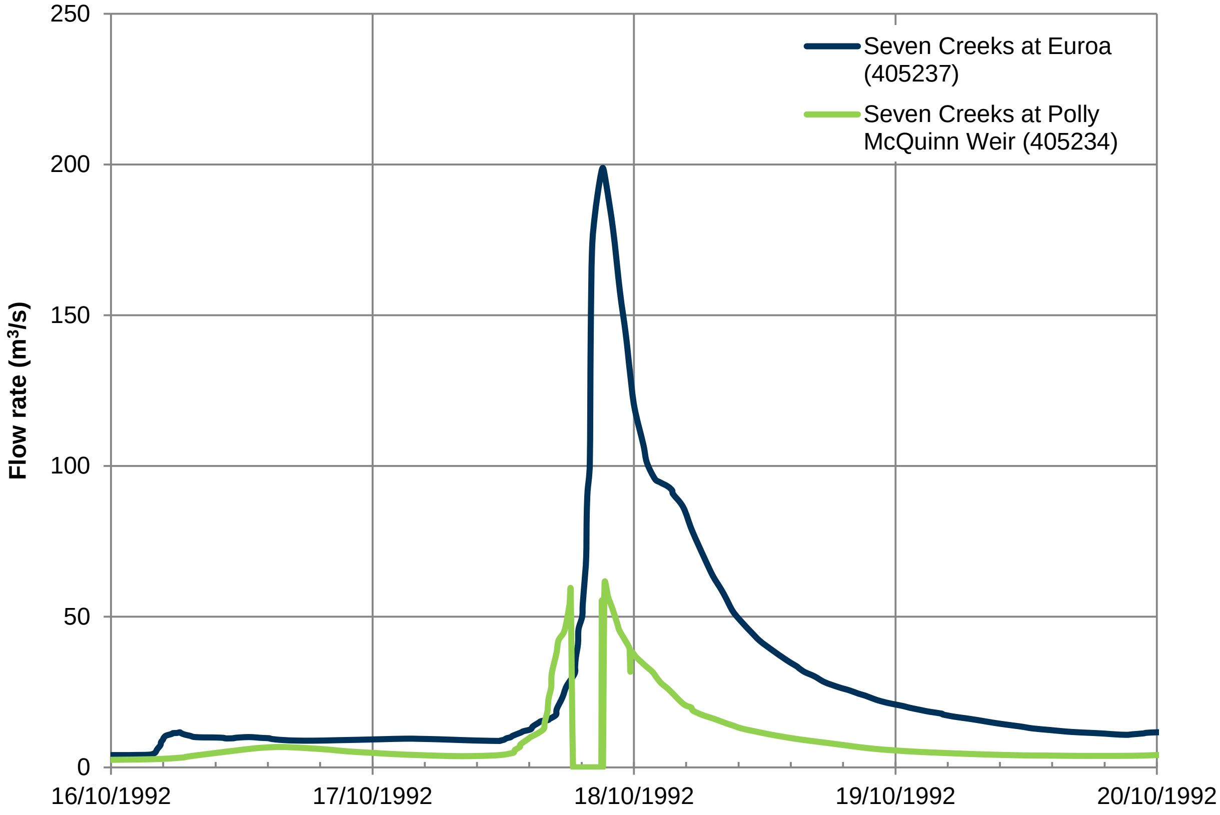


Figure 4-6 October 1992 recorded hydrographs

The rainfall for this event was low with a total over the event for the Euroa gauge and the Strathbogie gauge of approximately 50 mm. This suggests the antecedent conditions for the catchment were wet and the rainfall was converted to runoff with low losses. There was some rainfall in the days leading up to the event.

Table 4-8 October 1992 rainfall depths

Gauge	15/10	16/10	17/10	18/10	19/10	20/10	Total
82016 - Euroa	0	8	4.0	33.0	0.4	2.8	**48.2**
82042 – Strathbogie	0	12.8	8.0	27.6	0	4.6	**53.0**
82043 – Strathbogie North	0	13.4	7.0	24.6	0	6.6	**51.6**
82096 – Baronga	0	12.8	18.2	36.4	0.2	3.0	**70.6**
82089 – Terip Terip	27.2	0	16.4	43.6	0.2	3.2	**90.6**

The pluviograph data for this event was poor at Euroa with the pluviograph not functioning from 9 am on the 17th October to 9 am on the 19th October. This pluviograph missed the main rainfall patterns for the event and was not used in the calibration. The Strathbogie pluviograph captured the majority of days but missed the 18th October. However, although there was some missing periods within the pluviograph data the rainfall depths captured at the Strathbogie gauge matched the rainfall gauged with 49.5 mm captured at the Strathbogie pluviograph. All periods of cumulated rainfall (i.e. missing period followed by a recorded rainfall depth) were uniformly distributed.

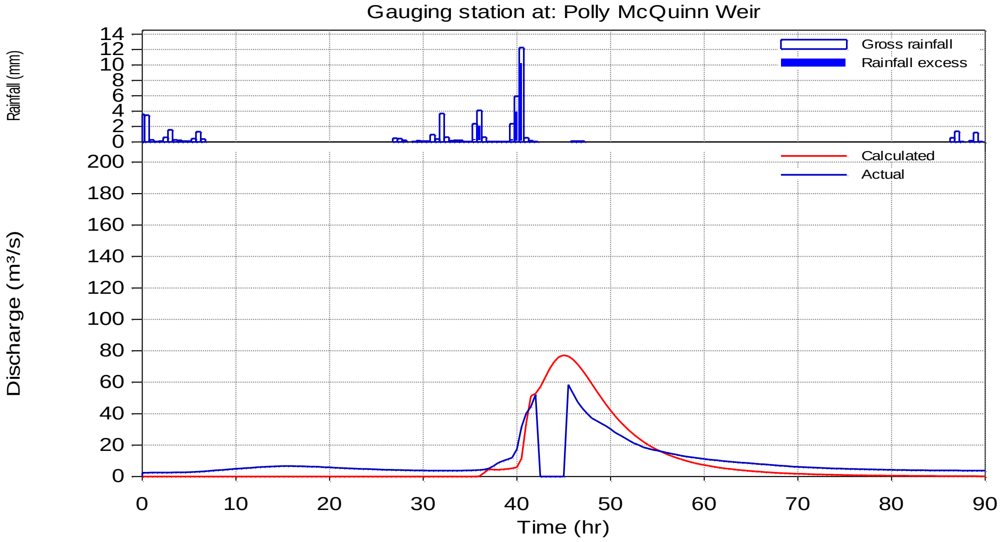
The calibrated RORB parameters are summarised in Table 4-9. The k_c and 'm' parameters were consistent with the 1993 calibration. The losses for the catchment were within reasonable levels, with a high continuing loss for the upper catchment (although the calibration to Polly McQuinn Weir is unreliable).

Table 4-9 Calibrated parameters for the 1992 event

Parameter	Polly McQuinn Weir	Euroa
k_c	25	
m	0.8	
IL (mm)	20	25
CL (mm/hr)	4.0	0.4

Although the pluviograph data missed some of the event, the calibration was still reasonable. The peak at Polly McQuinn Weir is unreliable as this is impacted by the Weir structure and the gauge did not manage to capture the peak flow rate. However, the hydrograph shape was reasonably well matched.

The hydrograph shape at Euroa was well matched. The peak was within 1% of the recorded peak. The RORB peak arrived faster than the recorded peak but this may have been caused by the restricted pluviograph data and the use to only the Strathbogie pluviograph. The peak arrived within 1 hour of the recorded peak and this was considered reasonable. The volume of the event was under predicted but this is attributed to the baseflow component of the recorded hydrograph. The RORB model does not included any baseflow component.



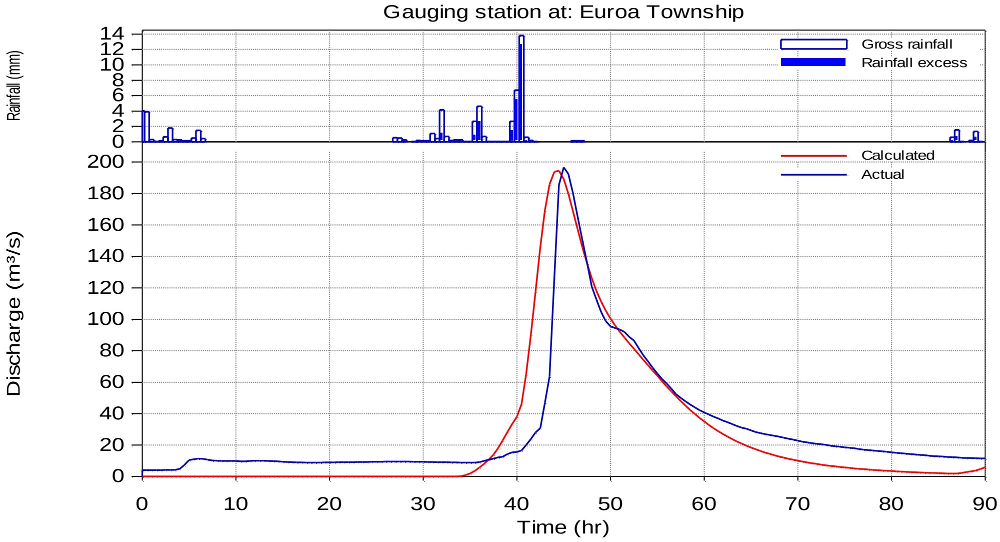


Figure 4-7 Calibrated RORB results for the October 1992 flood event

4.4.4 September 2010

The most recent large event at Euroa was in September 2010, this had a peak flow rate of 187 m³/s at Euroa. The majority of the recorded hydrograph during this event was not recorded at Polly McQuinn Weir. The hydrographs are shown in Figure 4-8.

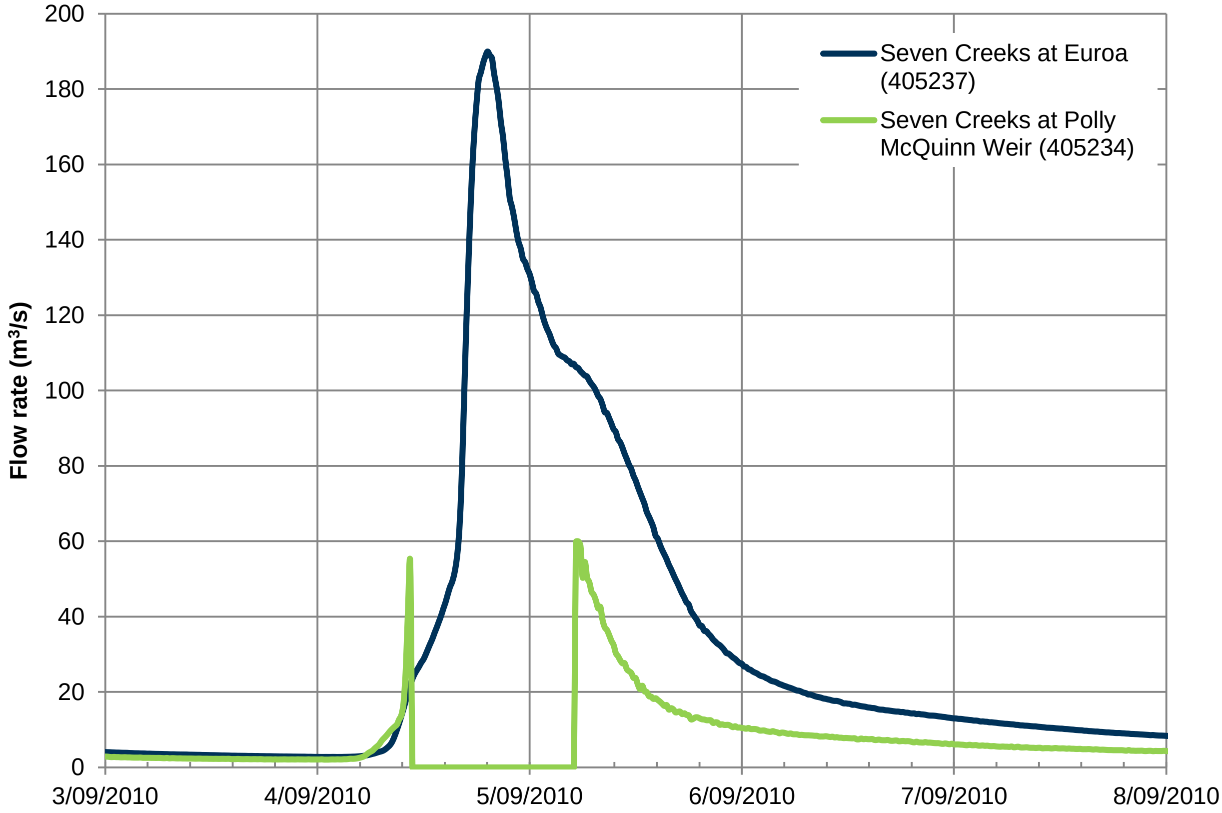


Figure 4-8 September 2010 recorded hydrographs

The rainfall for this event was concentrated on the 4th and 5th September with significantly more rainfall falling on the upper catchment relative to the lower part of the catchment. This can be observed in the recorded hydrographs with the peak flow rates at Polly McQuinn Weir being elevated for a longer period (as compared to the 1992 event which was larger at Euroa). There was some rainfall recorded in the days leading up the event so the catchment antecedent conditions will reflect this.

Table 4-10 September 2010 rainfall depths

Gauge	1/9	2/9	3/9	4/9	5/9	6/9	Total
82016 - Euroa	0.4	5	0	34	23	3.2	**65.6**
82042 – Strathbogie	1.4	6.6	0	43	54.4	2.4	**107.8**
82043 – Strathbogie North	1.4	7.6		87.2	52	3.2	**151.4**
82096 – Baronga	1.4	8.6	0	28.6	42.6	2.2	**83.4**
82089 – Terip Terip	2	10.2	0	24	60	2	**98.2**

The pluviograph data for the event was complete for this event at both Euroa and at Strathbogie. This allows for the rainfall to be distributed within the RORB model with confidence that the rainfall pattern is well matched.

The calibrated parameters are summarised in Table 4-11, the k_c and 'm' were consistent with the previously calibrated events.

Table 4-11 Calibrated parameters for the 2010 event

Parameter	Polly McQuinn Weir	Euroa
k_c	25	
m	0.78	
IL (mm)	30	50
CL (mm/hr)	2.0	1.5

The calibration for the upper catchment to Polly McQuinn Weir is well matched where the recorded information is available but as previously stated the majority of the peak flow is unknown. The peak may be too high here but there is no way to confirm this.

The calibration at Euroa is reasonable. The rising limb of the event does not seem to be well represented by the available rainfall pattern as there is a gradual rise followed by a sharp increase in flow rate to the peak. The pluviograph data has a more uniform shape and hence there is constant rise to the peak for the Euroa gauge. The peak flow rate is well matched and is within 1% of the recorded peak. The timing of the RORB peak was 1 hour after the recorded peak, which is the opposite of the 1992 flood event. As for the other events the volume for the event is approximately 15% below the recorded, against this is primarily due to the baseflow component of the hydrograph.

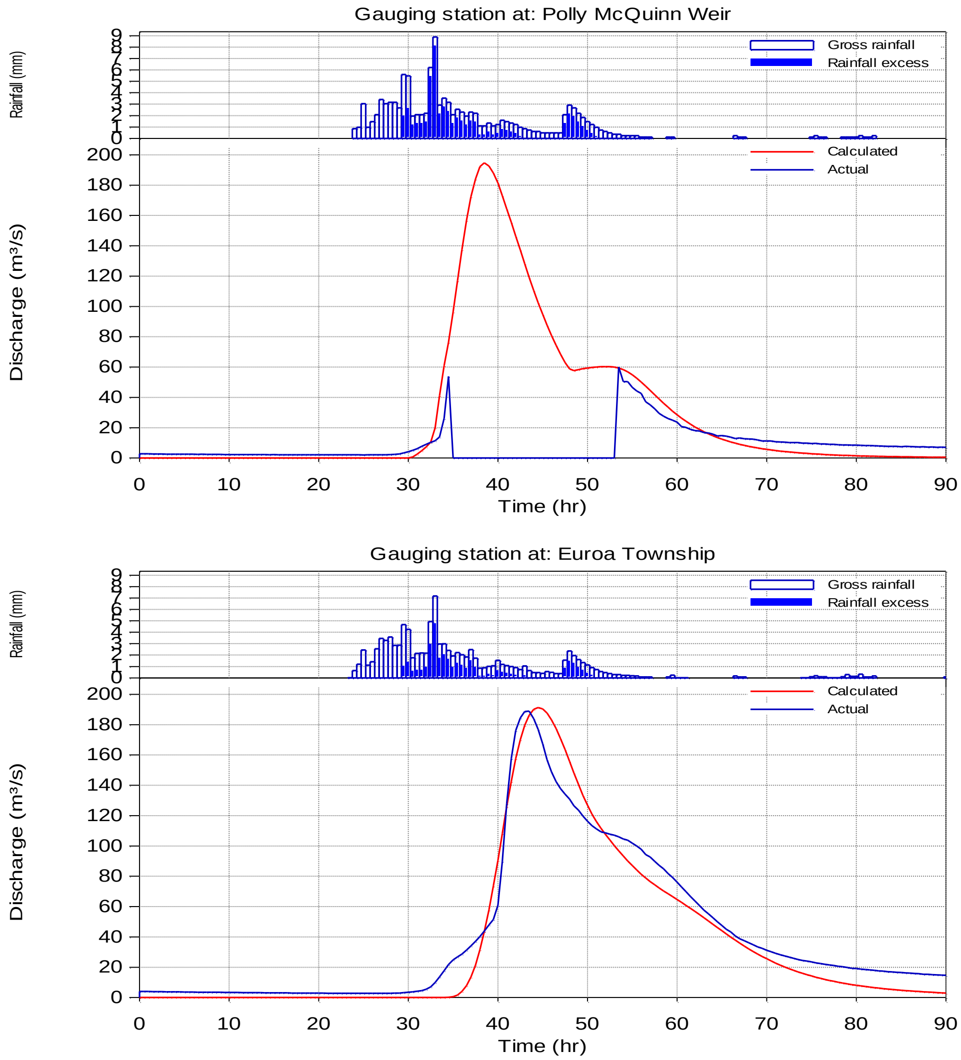


Figure 4-9 Calibrated RORB results for the September 2010 flood event

4.4.5 July 1986

The July 1986 event had a peak of just over 140 m^3/s. The Polly McQuinn Weir was largely recorded but for the peak which exceeded the gauge. The hydrographs for this event are summarised in Figure 4-10.

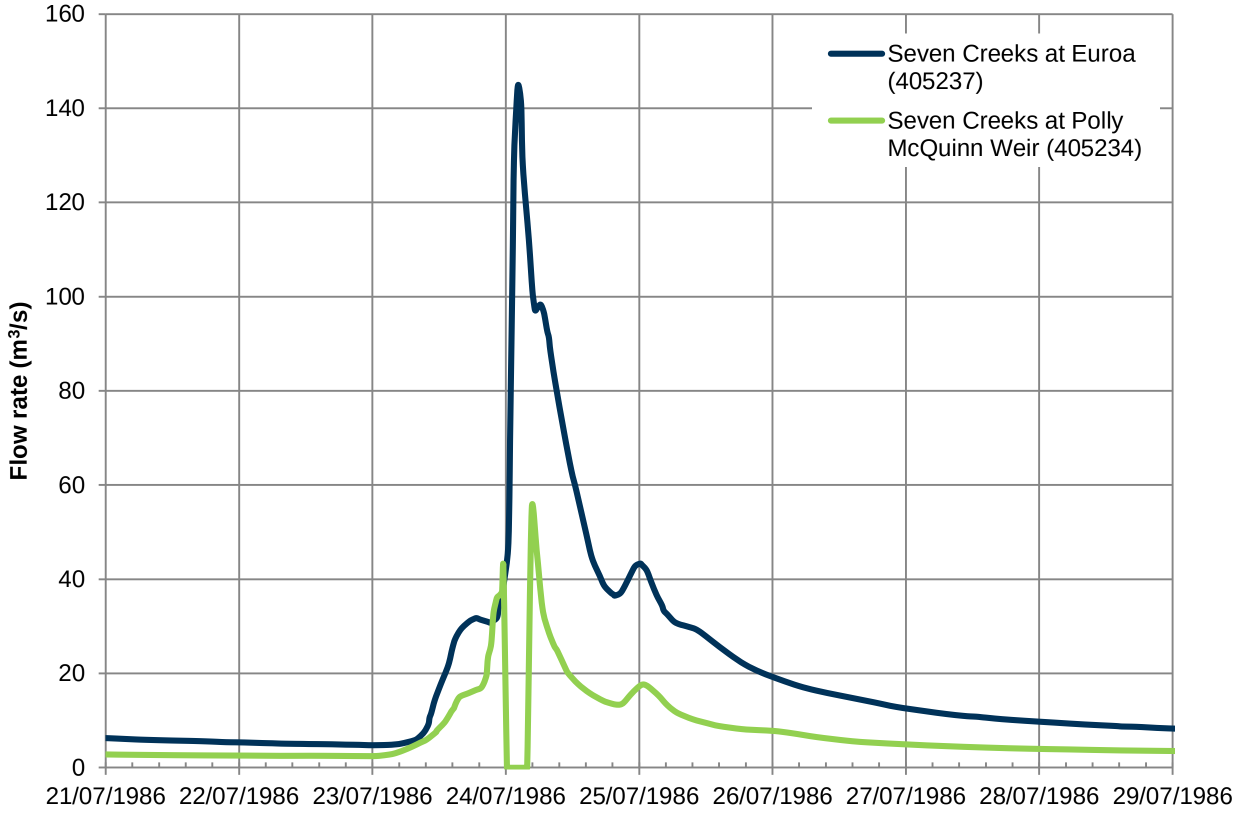


Figure 4-10 July 1986 recorded hydrographs

The rainfall for this event was concentrated in the Strathbogie Ranges near Strathbogie (see Table 4-12). The majority of the rainfall fell on the 24th July. The rainfall depths across the five rainfall gauges suggests the rainfall intensity and depth varied across the catchment which may make it difficult to match the rainfall depths to the recorded pluviograph information.

Table 4-12 July 1986 rainfall depths

Gauge	22/7	23/7	24/7	25/7	Total
82016 - Euroa	0	15.4	25.0	4.4	**44.8**
82042 – Strathbogie	0	23.6	53.7	9.0	**86.3**
82043 – Strathbogie North	0	27.0	27.0	11.0	**65.0**
82096 – Baronga	0	18.2	44.6	13.8	**76.6**
82089 – Terip Terip	0.2	18.6	20.2	11.2	**50.2**

The pluviograph data at Euroa represented the full event with no missing record, however as the lowest recorded rainfall total was also recorded at Euroa this pluviograph record may not be representative of the rainfall pattern across the catchment. The Strathbogie pluviograph was also well represented, however there was a missing period on the 23rd July from 6:30 am to 8:30 am. The cumulated rainfall reported during this period was distributed uniformly over the 2 hour period.

The calibrated parameters for the July 1986 event are summarised in Table 4-13. Again the k_c and 'm' were consistent with the other calibrated events.

Table 4-13 Calibrated parameters for the 1986 event

Parameter	Polly McQuinn Weir	Euroa
k_c	25	
m	0.8	
IL (mm)	20	20
CL (mm/hr)	2.5	0.5

The calibration within RORB is shown in Figure 4-11. The calibration at Polly McQuinn Weir is reasonable given that the structure is not modelled and the peak is unknown.

At Euroa the modelled hydrograph arrives before the recorded hydrograph, however the shape of the hydrograph matches the recorded event well. The peak is within 2% of the recorded peak of the event and this arrives 1 hour before the recorded peak. The volume of the event is not well matched, largely due to the baseflow component of the flows not being included in the rainfall-runoff model. Overall the shape of the hydrograph and the peak are well matched.

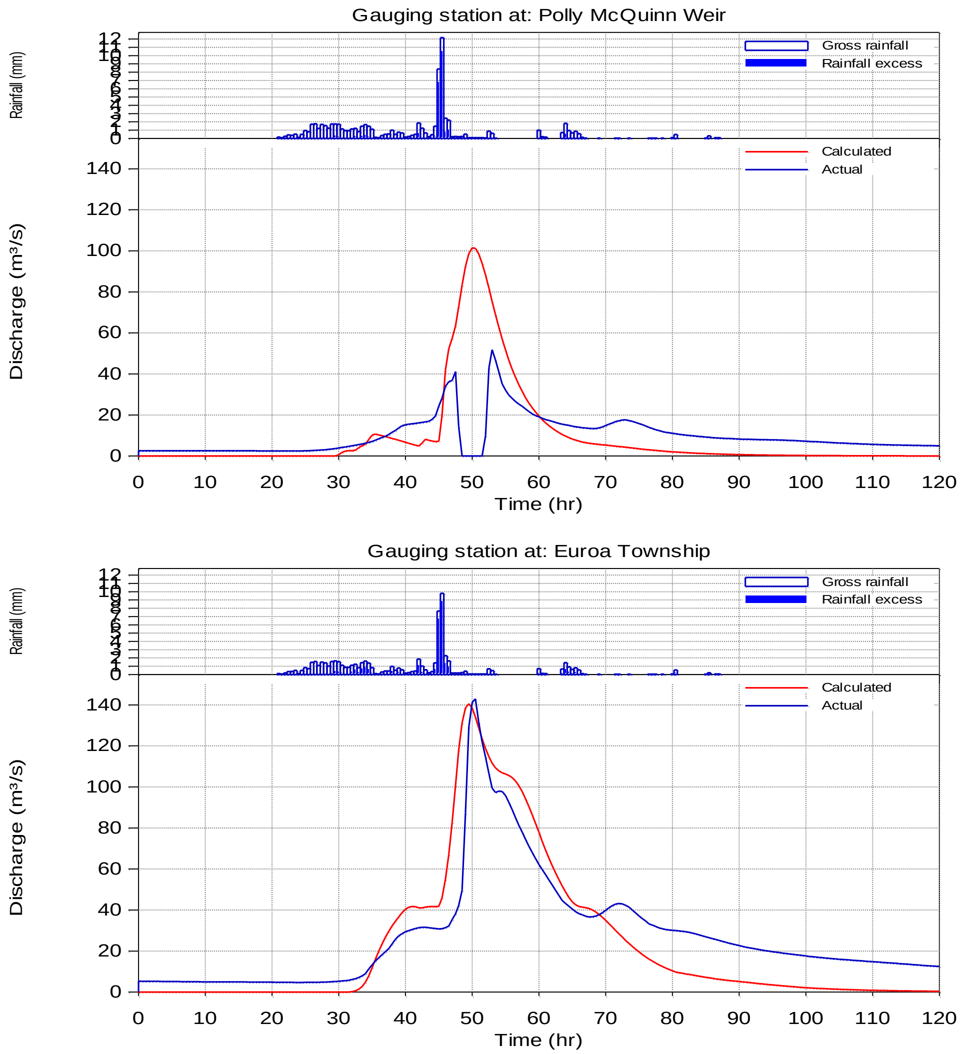


Figure 4-11 Calibrated RORB results for the July 1986 flood event

4.4.6 October 1984

The October 1984 event had a peak of just over 140 m^3/s. The Polly McQuinn Weir was largely recorded but for the peak which exceeded the gauge. The hydrographs for this event are summarised in Figure 4-12.

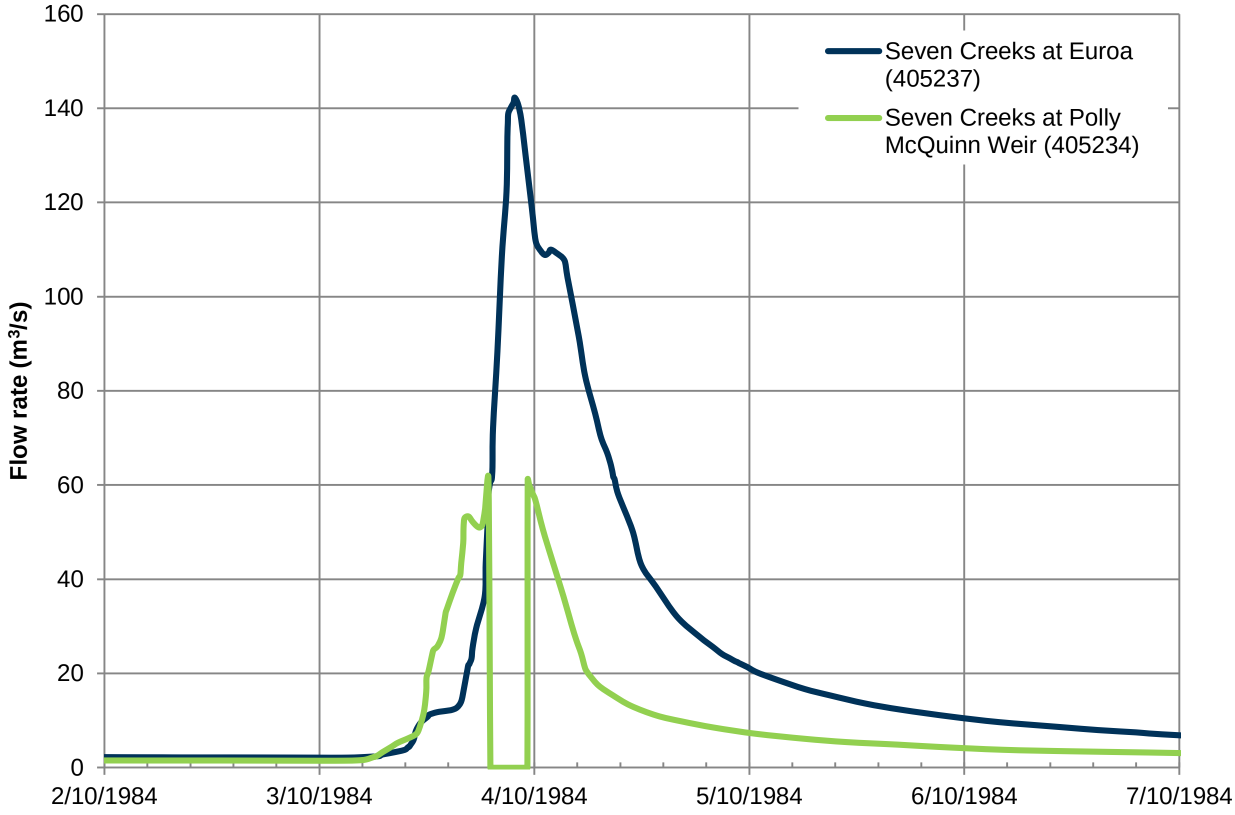


Figure 4-12 October 1984 recorded hydrographs

The rainfall for this event was similar to the pattern observed in 2010 with the majority of the rainfall falling on the upper catchment in the Strathbogie Ranges with lower rainfall volumes on the lower catchment. Approximately 120 mm fell over the upper catchment within a 48 hour period which resulted in the sharp peak being observed at Euroa. The rainfall depths are summarised in Table 4-14.

Table 4-14 October 1984 rainfall depths

Gauge	2/10	3/10	4/10	5/10	Total
82016 - Euroa	0	21.0	31.4	0	**52.4**
82042 – Strathbogie	0	36.2	45.6	0	**81.8**
82043 – Strathbogie North	0	68.0	52.4	0	**120.4**
82096 – Baronga	0	19.6	29.6	0	**49.2**
82089 – Terip Terip	27.0	38.0	0	0	**65.0**

The pluviograph information for this event was well captured with the Euroa pluviograph recording the rainfall pattern on the 3rd October. From the analysis of the pluviograph it appears that the full 52.4 mm fell during this period. For the Strathbogie pluviograph data was recorded for the 3rd and 4th October which is when the recorded rainfall depth of 81.8 mm fell.

The calibrated parameters are summarised in Table 4-15, the k_c and 'm' parameters are the same ad for the other events.

Table 4-15 Calibrated parameters for the 1984 event

Parameter	Polly McQuinn Weir	Euroa
k_c	25	
m	0.8	
IL (mm)	25	44
CL (mm/hr)	4.5	0.2

The calibration to the 1984 event was good at both Polly McQuinn Weir and at Euroa (see Figure 4-13). No comparison has been discussed at Polly McQuinn Weir as the peak was not captured, however the hydrograph recorded at this location matches the RORB model well.

At Euroa the RORB peak was within 1% of the recorded hydrograph. The timing of the peak was also matched well with the peak occurring at the same time as the recorded hydrograph. The modelled overall event volume was 25% less than the recorded event but the calibration was run over 200 hours and the baseflow was not included in the RORB model. The volume for the recorded peak of the event was well matched.

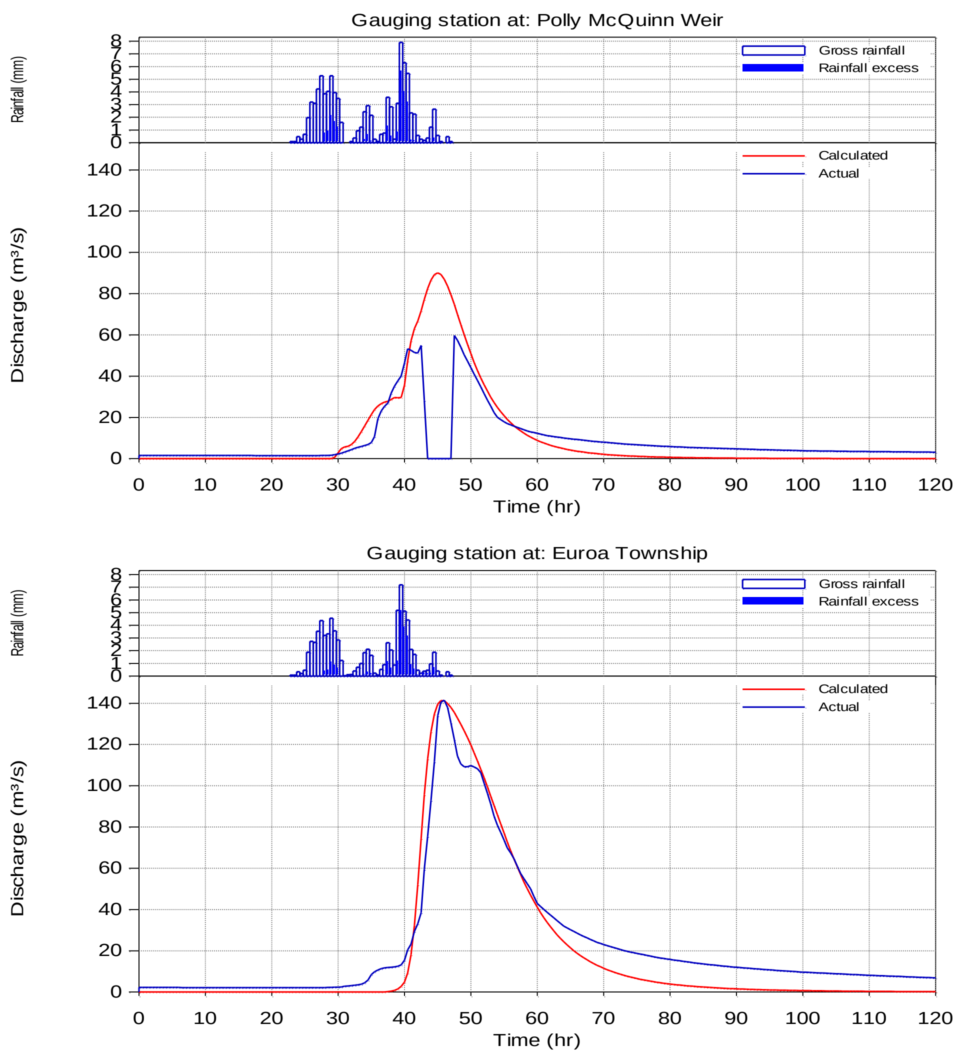


Figure 4-13 Calibrated RORB results for the October 1984 flood event

4.5 Flood Frequency Analysis

The flood frequency analysis (FFA) is required to determine the statistical peak flow rates for the design events as determined from a fitted distribution to the streamflow gauge at Euroa. For this assessment the distribution fitting program FLIKE has been used and the gauge was assessed using three distributions. Three distributions have been used to ensure that the uncertainty of the fitted distribution is understood for the streamflow gauge at Euroa. The distributions used to fit the annual streamflow data include:

- Log Pearson Type III (LPIII)
- Generalised Extreme Value (GEV)
- Generalised Pareto (GenPar)

The FFA has been undertaken using the available data on the Victorian Data warehouse for Seven Creeks at Euroa which has 40 years of continuous record from 1973 to 2013. Two additional sources have also been utilised to extent this gauge record for the FFA. The additional sources included:

- Estimates of the 1916 event at 394 m^3/s (CMPSF, 1993)
- Peak flow rates from 1963 to 1973 (SKM, 1997)

The 1916 flood event has been included in this assessment as this is the largest known flood event recorded for Euroa. The secondary source, the SKM 1997 report, included an additional 10 years of record that was not available on the Victorian Data Warehouse. Initial assessment of the concurrent annual peak flows from the SKM study against the current Victorian Data Warehouse peak flows showed that since the SKM 1997 study the rating table had been adjusted. The peak flow rates are shown in Figure 4-14. The concurrent period between 1974 and 1995 indicates that it is likely that a rating table modification was made and the peak flows were adjusted down for the current peak flow estimates.

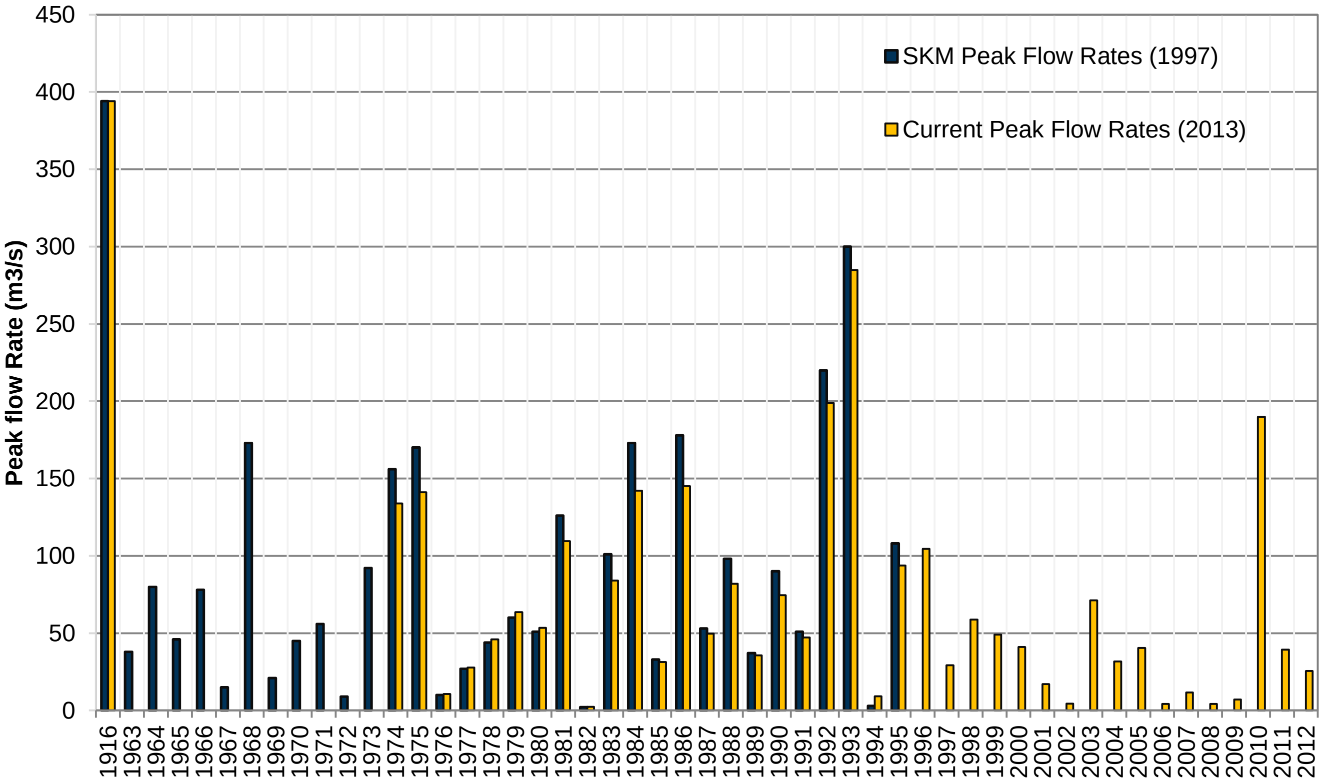


Figure 4-14 Peak flow rate comparison – SKM 1997 values versus current values

In order to utilise the peak flows from 1963 to 1973 from the SKM Study the peak flows were adjusted down by 20% which was the average reduction in peak flow rate observed from 1974 to 1995 for peak flow rates greater than 50 m^3/s. Peak flow rates below 50 m^3/s were maintained at the peak flow rate stated in the SKM Study.

As summarised in Section 4.4 - Table 4-4, Seven Creeks at Euroa has 40 years of continuous instantaneous maximum flows from 1973 to 2013. This has been extended to include the 1916 event and the period from 1963 to 1974. The full infilled record was used in the FFA process using the annual maximum peaks. The optimisation method for fitting the distributions was based on the Bayesian approach.

For the LPIII and GenPar distributions the full record was used to fit the distributions. For the GEV distribution the flow threshold of 10 m^3/s was used to censor the years where the peak flow was lower than this value (that is to ignore the peak flow but include the year in the length of record count to estimate the ARI of the recorded events). This censored 7 years of the gauged record. The purpose of censoring the record is to increase the weighting of the fit to the larger events for the GEV distribution. This gives a more reliable fit to the peak flows that are of interest for this investigation.

The results of the FFA are summarised in Table 4-16. The three distributions are summarised for the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEPs. The lower and upper 90% Monte Carlo percentile bounds are reported to show the range of uncertainty for the fitted distributions. The plots of the fitted distributions are shown in Appendix A.

Table 4-16 Flood Frequency Analysis results (1916, 1963 – 2012)

ARI (years)	5	10	20	50	100	200	500
AEP	20%	10%	5%	2%	1%	0.5%	0.2%
Log Pearson Type III	**107**	**159**	**214**	**290**	**350**	**410**	**490**
Lower 90% Monte Carlo Probability	84	125	167	222	261	296	338
Upper 90% Monte Carlo Probability	138	205	282	412	532	666	881
Generalised Extreme Value	**104**	**150**	**201**	**281**	**353**	**437**	**570**
Lower 90 Monte Carlo Probability	84	119	156	206	245	284	338
Upper 90% Monte Carlo Probability	131	192	275	433	604	830	1271
Generalised Pareto	**114**	**166**	**219**	**294**	**353**	**414**	**499**
Lower 90% Monte Carlo Probability	89	131	174	228	265	300	341
Upper 90% Monte Carlo Probability	140	205	281	405	530	675	918

The three fitted distributions produce similar peak flow estimates for the various return intervals. The 1% AEP event is estimated at 350 to 353 m^3/s. The 90% Monte Carlo Probability indicates the lower and upper bounds for which it is estimated that there is a 90% chance of the real peak flow for that AEP occurring. It is a measure of the uncertainty associated with the fitted distributions and the smaller the range the more reliable the estimated values. As the events become more extreme and rare, they require extrapolation from the recorded annual dataset which increases the uncertainty.

For the 1% AEP event the lower and upper 90% probabilities were from 245 to 604 m^3/s. This range of uncertainty is expected as there is only 50 years of record. The fitted distributions are summarised in Figure 4-15 along with the 90% percentile bounds.

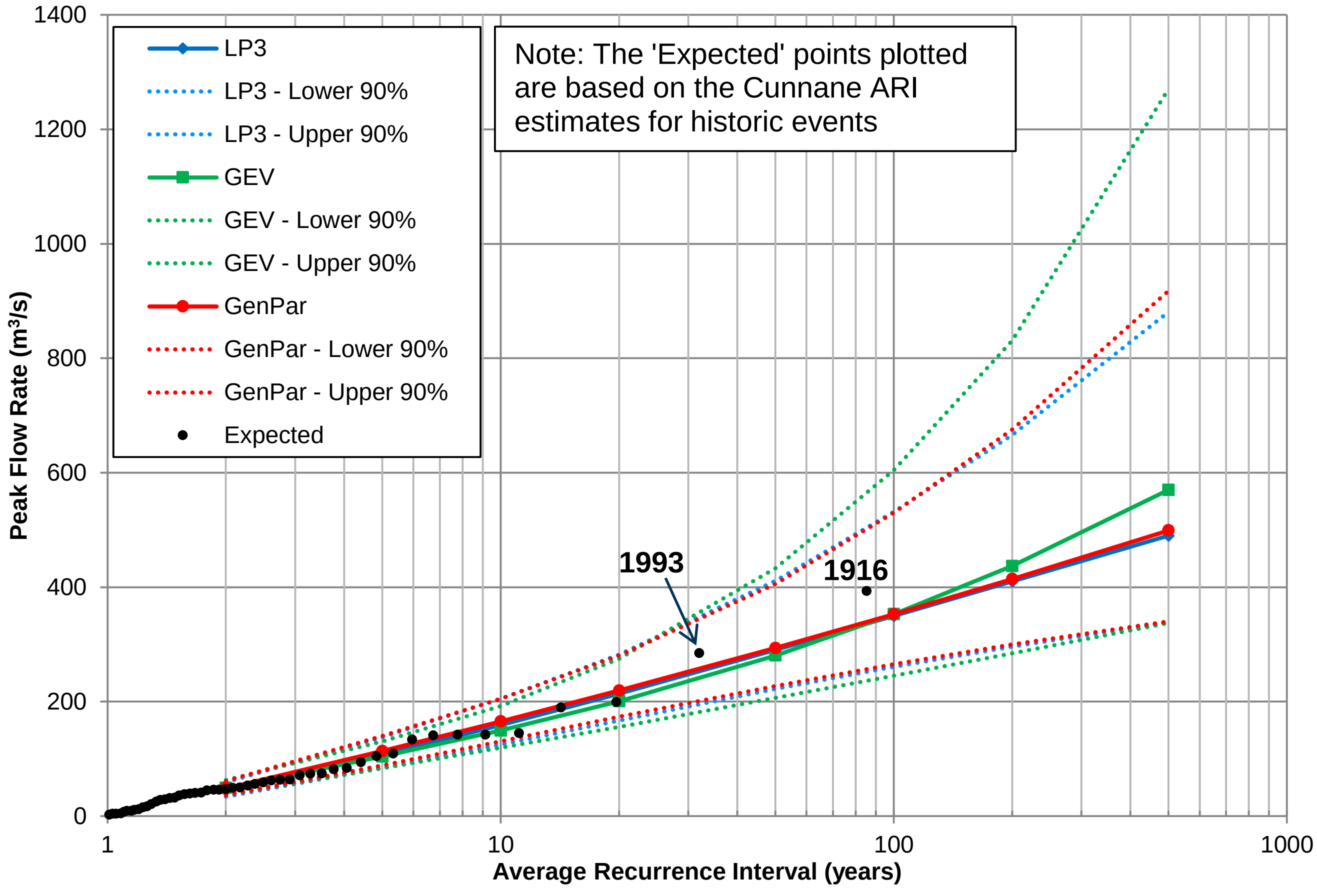


Figure 4-15 Flood Frequency Analysis results

Overall, the three fitted distributions produce similar peak flood estimates for the required 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEPs. As the LPIII distribution is the primary recommended distribution in AR&R this has been selected to provide the design peak flow targets. This is also consistent with previous flood investigations in the area.

4.5.2 <u>Sensitivity on the FFA</u>

Due to the unknown accuracy of the 1916 flood event and the SKM data from 1963 to 1973 the FFA has been run using solely the data available from 1973 to 2013 from the Victorian Data Warehouse. This process aims to explore the differences between the FFA peak flow estimates generated by the inclusion of the 1916 event and the additional data from the SKM Study.

For the LPIII and GenPar distributions the full 40 year record was used to fit the distributions. For the GEV distribution the flow threshold of 40 m^3/s was used to censor the years where the peak flow was lower than this value (that is to ignore the peak flow but include the year in the length of record count to estimate the ARI of the recorded events). This censored 16 years of the record. The purpose of censoring the record is to increase the weighting of the fit to the larger events for the GEV distribution. This gives a more reliable fit to the peak flows that are of interest for this investigation.

The summary of the FFA results are summarised in Table 4-17. The 1% AEP peak has now been estimated at between 337 and 344 m^3/s. This is only a 2.6% reduction as compared with the extended FFA assessment. However, the uncertainty of these predicted values has increased as the 90% confidence limits are now 230 to 707 m^3/s (as compared with 245 to 604 m^3/s from the extended FFA assessment). It is clear from this assessment that although the extension of the FFA assessment data does not impact the final FFA values significantly, it does act to reduce the uncertainty associated with the estimate and should be used for this investigation.

Table 4-17 Flood Frequency Analysis results (1973 to 2012)

ARI (years)	5	10	20	50	100	200	500
AEP	20%	10%	5%	2%	1%	0.5%	0.2%
Log Pearson Type III	**109**	**161**	**216**	**288**	**341**	**393**	**460**
Lower 90% Probability	82	121	159	204	233	257	283
Upper 90% Probability	147	224	324	501	670	872	1210
Generalised Extreme Value	**118**	**168**	**219**	**286**	**337**	**388**	**456**
Lower 90 Probability	94	133	168	207	232	252	277
Upper 90% Probability	147	229	330	521	707	950	1341
Generalised Pareto	**115**	**166**	**218**	**289**	**344**	**400**	**477**
Lower 90% Probability	85	123	159	203	230	253	277
Upper 90% Probability	148	225	327	517	707	946	1400

4.5.3 Historic Event Recurrence Intervals

Following the development of the FFA it is possible to assess the historic events for Seven Creeks at Euroa and to determine their estimated recurrence intervals. For this assessment the extended FFA fitted distribution using the Log Pearson Type III distribution was used.

The predicted recurrence intervals are summarised in Table 4-18 for the largest 11 peak flood events at Seven Creeks at Euroa.

Table 4-18 Recurrence Intervals for Seven Creeks at Euroa

Rank	Year	Peak Flow Rate (m^3/s)	AEP (%)	ARI (years)
1	1916	394.0	0.6%	174
2	1993	284.9	2%	48
3	1992	198.9	6%	17
4	2010	189.9	6%	16
5	1986	145.0	12%	9
6	1984	142.2	12%	8
7	1968	142.0	12%	8
8	1975	141.1	12%	8
9	1974	133.8	13%	8
10	1981	109.4	19%	5
11	1996	104.6	21%	5

The largest recorded flood event of 394 m^3/s in 1916 has been estimated as a 0.6% AEP event (~ 1 in 174 year ARI). The revised FFA predicts that the 1916 event was rarer than within the SKM report with the AEP decreasing to 0.6% AEP (SKM predicted this event was approximately the 1% AEP). This change is a combination of the generally reduced peak design flow rates due to the re-rating of the gauge, but also the addition of 18 years to the gauge record (see Section 4.6.3).

The next largest flood event for the catchment occurred in 1993 and has an estimated recurrence interval of 2% AEP (~1 in 50 years). The recurrence interval for this event was commensurate with the previous SKM Study (1997).

4.6 Design Events

The design events have been derived using the calibrated RORB model and the peak flows derived using the FFA. The design events have been developed within RORB using the AR&R87 Volume 2 design generated storms using the Intensity Frequency Duration (IFD) parameters for the catchment. The Seven Creeks design flows were run using the calibrated parameters and the Castle Creek model was run using the translated parameters from this model.

During the derivation of the design events Cardno had discussions with the Steering Committee and it was decided that the design events should aim to match the previous SKM study. This was to ensure that the flood extents and depths generated were commensurate between the investigations and allow for the uncertainty in the flood estimates and the change in rating curve. It should be noted that this produces peak flow rates for the 1% AEP event which are approximately 15% higher than the FFA predicts. All design event loss rates were adjusted to meet these requirements of an increase of 15% to the FFA predicted peak.

4.6.1 Seven Creeks

The Seven Creeks IFD parameters are summarised in Table 4-19. The design events have been generated using filtered patterns, a uniform rainfall distribution and using the Siriwardene and Weinmann formulation for the areal reduction factors. The design losses were adjusted until the peak flow from each recurrence interval matched the design targets as shown in Table 4-20.

Table 4-19 Intensity-Frequency-Duration parameters for the Seven Creeks catchment

IFD Coefficient	Value
${}^{2}I_{1}$	25.58
${}^{2}I_{12}$	5.05
${}^{2}I_{72}$	1.54
${}^{50}I_{1}$	45.64
${}^{50}I_{12}$	9.18
${}^{50}I_{72}$	2.9
G (skew)	0.22
F2	4.31
F50	15.11
Zone	2
Location	
Latitude	36.850 S
Longitude	145.725 E

Table 4-20 RORB design parameters and peak flow targets for Seven Creeks

RORB Parameters	
k_c	25
'm'	0.8

Design Events (AEP and peak flows m^3/s)						
20%	**10%**	**5%**	**2%**	**1%**	**0.5%**	**0.2%**
123	184	246	331	398	468	563

The initial loss was set at a constant 20 mm for the design runs and the continuing loss was adjusted until the peak flows matched Table 4-20. The initial loss was in the mid-range of the recommended AR&R loss rate for north of the divide of 10 - 35 mm. Durations from 1 hour to 72 hours were run within RORB using the losses in Table 4-21. The critical duration for the catchment ranged from the 24 hour event down to the 9 hour event.

Table 4-21 Design events generated for the Seven Creeks catchment

AEP	Design Peak (m^3/s)	Design Event Target (m^3/s)	Duration	Initial Loss	Continuing Loss
20%	123	123	24h	20.0	1.88
10%	184	183	18h	20.0	1.65
5%	246	247	12h	20.0	1.70
2%	331	329	12h	20.0	1.90
1%	398	400	12h	20.0	2.00
0.5%	468	467	12h	20.0	2.10
0.2%	563	557	9h	20.0	2.30

4.6.2 Castle Creek

The design events for Castle Creek have been developed using the calibrated parameters from the Seven Creeks catchment using an adjustment factor for the k_c using the average distance from the sub-catchments to the outlet (D_{av}). This adjustment is possible as there is a proportional relationship between k_c and D_{av} for catchments. This relationship is shown in Equation 5.1. The adjusted k_c for Castle Creek is 13.8, the 'm' factor remained unchanged at 0.8.

$$k_{c\ Castle} = \frac{D_{av\ Castle}\ x\ k_{c\ Seven}}{D_{av\ Seven}} = 13.8$$ **Equation 5.1**

Where: $k_{c\ Castle}$ is the k_c for the castle Creek catchment (unknown)
$k_{c\ Seven}$ is the k_c for the Seven Creeks catchment (25)
$D_{av\ Castle}$ is the average distance from the sub-catchments to the outlet for Castle Creek (13.22)
$D_{av\ Seven}$ is the average distance from the sub-catchments to the outlet for Seven Creeks (23.99)

The Castle Creek IFD parameters are summarised in Table 4-22. The design events have been generated using filtered patterns, a uniform rainfall distribution and using the Siriwardene and Weinmann formulation for the areal reduction factors.

Table 4-22 Intensity-Frequency-Duration parameters for the Castle Creek catchment

IFD Coefficient	Value
$^{2}I_{1}$	24.56
$^{2}I_{12}$	4.18
$^{2}I_{72}$	1.29
$^{50}I_{1}$	45.99
$^{50}I_{12}$	7.17
$^{50}I_{72}$	2.03
G (skew)	0.21
F2	4.31
F50	15.09
Zone	2
Location	
Latitude	36.800 S
Longitude	145.575 E

The design events were run using the adjusted k_c value of 13.8 and the loss rates as for Seven Creeks. The resulting peak flow rates are summarised in Table 4-23. As for Seven Creeks durations from 1 hour to 72 hours were assessed. The critical duration for Castle Creek at Euroa were the 6 to 9 hour events. The shorter durations for Castle Creek (compared to Seven Creeks) reflects the smaller catchment area.

Table 4-23 Design events generated for the Castle Creek catchment

AEP	Design Peak (m^3/s)	Duration	Initial Loss	Continuing Loss
20%	25.5	9h	20.0	1.88
10%	39.8	9h	20.0	1.65
5%	54.9	9h	20.0	1.70
2%	73.4	9h	20.0	1.90
1%	89.7	9h	20.0	2.00
0.5%	106.9	9h	20.0	2.10
0.2%	130.8	6h	20.0	2.30

4.6.3 Comparisons with Previous Investigations

The design events have been estimated for Seven Creeks in two previous flood investigations, in 1993 by CMPSF and in 1997 by SKM. To assess the validity of the current design flow estimates they have been compared in Table 4-24. For Castle Creek the design peaks have only been compared to the SKM study in 1997, these are shown in Table 4-25.

Table 4-24 Design events generated for the Seven Creeks catchment

AEP	Design Peak (m^3/s)	Lower 90% Conf. Limit	Upper 90% Conf. Limit	SKM (1997)	Difference (SKM to current)	CMPSF (1993)
20%	**123**	84	138	**135**	10%	**121**
10%	**184**	125	205	**190**	3%	
5%	**246**	167	282	**250**	2%	**184**
2%	**331**	222	412	**330**	0%	
1%	**398**	261	532	**400**	1%	**228**
0.5%	**468**	296	666	**470**	0%	
0.2%	**563**	338	881	**560**	-1%	

Table 4-25 Design events generated for the Castle Creek catchment

AEP	Design Peak (m^3/s)	SKM (1997)	Difference (SKM to current)
20%	**26**		
10%	**40**	36	11%
5%	**55**	54	2%
2%	**73**	73	1%
1%	**90**	97	-8%
0.50%	**107**	122	-12%
0.20%	**131**	159	-18%

The SKM design peaks were substantially higher than the CMPSF (1993) peak flow estimates. From the SKM Study this was explained to be due to the fact that the large 1916, 1992 and 1993 flood events were not included in this assessment (SKM, 1997). The current design estimates also exceed the CMPSF (1993) estimated design events and the reasoning is the same as for the SKM Study. The CMPSF estimates are too low for the catchment.

Since the SKM study was completed there have been few floods as most of Victoria has been in drought like conditions. This lead to the peak flow estimates obtained using the FFA approach being lower than the SKM study. The following factors impacted on the reduction in predicted FFA peak flow rates for flood events at Euroa:

- There has been a rating table correction in the current gauge which has adjusted the large flood events down by approximately 5 to 23%. Some examples of these adjustments include:
    - the 1993 flood peak was reduced 5% from 300 m^3/s down to 285 m^3/s
    - the 1992 flood peak was reduced 11% from 220 m^3/s down to 199 m^3/s
    - the 1986 event was reduced 23% from 178 m^3/s down to 145 m^3/s
    - the 1984 event was reduced 22% from 174 m^3/s down to 142 m^3/s
- The gauge record has been extended from 1996 to 2013. Within this period there has only been one significant event in 2010. The average annual peak flow rate over this period (1996 to 2013) was 43 m^3/s whereas the average from 1973 to 1995 was 85 m^3/s.

Overall the reduction in the peak flow rates is expected from the FFA due to the rating table reductions and the extension of the peak flow record by a further 18 years (which were below the long term average for flooding). The inclusion of the 1916 event allows for the direct comparison with the SKM estimated design flows and including past historic events where information is known is desirable.

In order to ensure that the flood extents and depths obtained from this study are commensurate with the previous planning advice and outputs from the SKM study it was decided that the design events should be increased to match the previous study. Table 4-24 shows the peak design estimated determined by SKM were between -1% and 10% larger than the current peak flow estimates. The primary reason that the differences are low is due to the requirement of the Steering Committee for this study to match the SKM assessment hydrology to allow for the uncertainty in the flood estimates and to manage the uncertainty due to the changes to the rating table (since the 1997 SKM report).

4.7 Castle Creek Historic Analysis

Although the Castle Creek catchment has no long term streamflow gauge to assess for a RORB calibration there is recent rainfall and level information for recent flood events in 2010. These events have been assessed to provide some additional verification and validation of the hydrology for the catchment.

Examination has been undertaken on the rainfall records and Telfords Bridge flow records for the September and December 2010 flood events. For clarity, the September flood event occurred between 3-7 September and the December flood event occurred between the 7-9 December. Assessment of the records from the rainfall stations in the vicinity of Euroa in addition to the Telfords Bridge flow gauge has been included:

- Sevens Creek at Euroa
- Strathbogie North
- HoneySuckle Ck U/S Violet Town
- Heronslea
- Waterhouse Reservoir
- Hillside
- Morella
- Enderlee
- Moroko Park
- Castle Ck at Telfords Bridge
- Sevens Cks at Strathbogie
- Balquhain
- Seven Creeks at D/S Polly McQuinns Weir
- Mount Wombat

The locations of each gauge are shown in Figure 4-16 with the Seven Creeks and Castle Creek catchment areas. The key gauges for the Castle Creek catchment are Castle Creek at Telfords Bridge, Hillside and Morella. These gauges are located in the Castle Creek catchment and provide the best estimate of rainfall distribution across the catchment.

Gauge plots obtained from BoM state that the gauge heights for the September 2010 and December 2010 events as 2.58 m and 2.75 m respectively. The rainfall and predicted peak flows for each event are discussed further in the following sections.

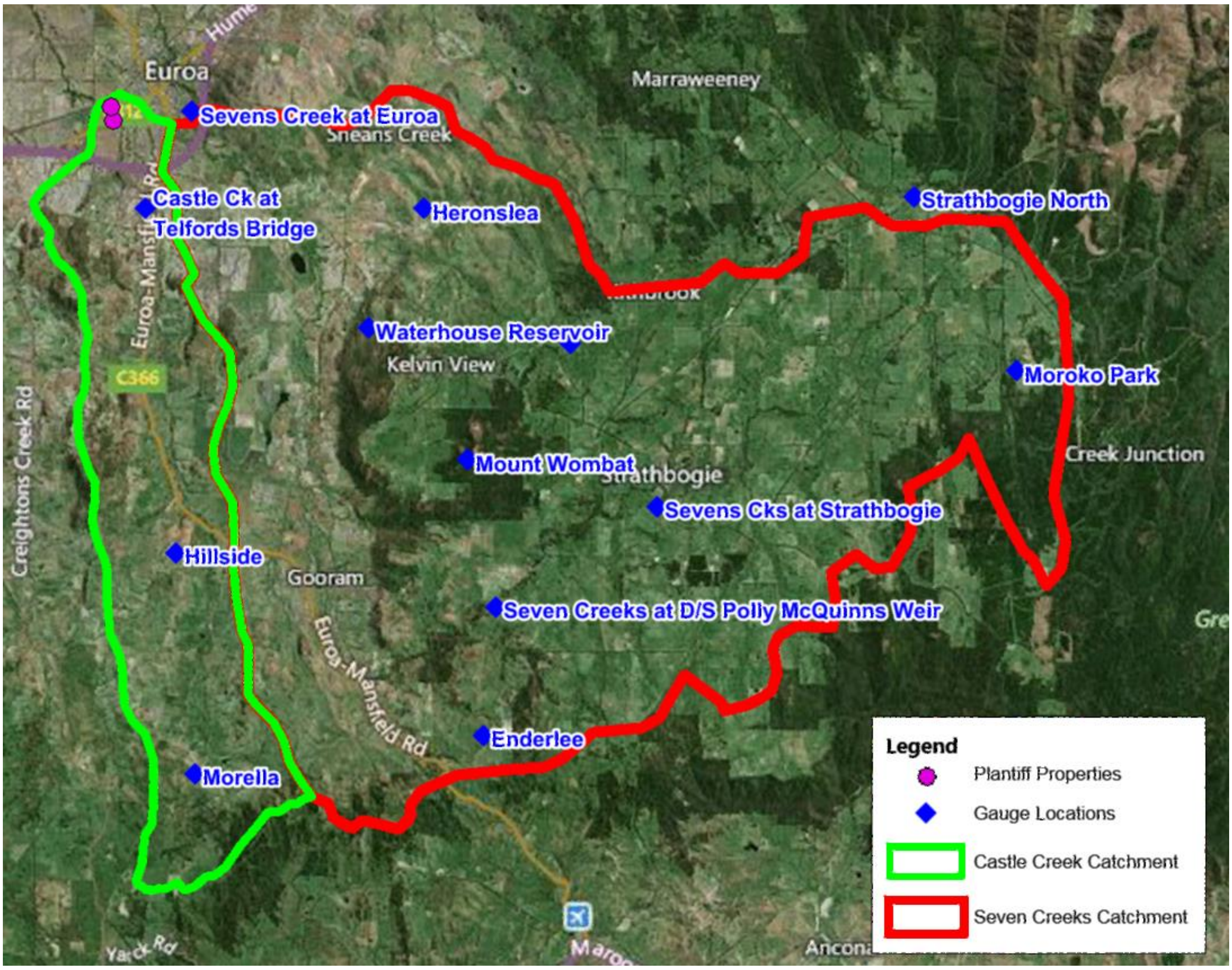


Figure 4-16 Location of the assessed gauges

4.7.1 September 2010

The September 2010 flood event was of regional significance, with a peak flow rate of 187 m^3/s recorded at the Sevens Creek gauge at Euroa. This assessment estimates that the 2010 flood event was approximately equivalent to the 5% AEP event on Sevens Creek. The rainfalls recorded for the September 2010 flood event in the Castle Creek catchment are shown in Table 4-26.

Table 4-26 Recorded Rainfall Castle Creek, September 2010

Date	Telfords Bridge (mm)	Hillside (mm)	Morella (mm)
3/09/2010	0	0	0
4/09/2010	49.6	69.4	80.2
5/09/2010	3.4	4.6	6.4
6/09/2010	2.8	0.8	0.4
7/09/2010	0	0.2	0

It should be noted that these values are for the actual 24 hour period (midnight to midnight) on the date shown and are a sum of values from the hourly data obtained from the Bureau of Meteorology. Standard daily rainfall data is reported from 9am to 9am.

The rainfall was distributed across the catchment, with higher intensity rainfall occurring in the upstream reaches of the catchment. The average rainfall total for the three stations on 4 September was 66.4 mm. For comparison purposes, the average rainfall recorded at gauges in the Seven Creeks catchment was 78.2 mm, with a maximum rainfall of 114.0 mm and a minimum rainfall 48.6 mm.

The analysis of the rainfall data for the 2010 event shows a consistent period of heavy rain especially on the 4th December. There are no major intense periods of rainfall, and the average hourly rainfall intensity is between 2.5 and 3.5 mm per hour, approximately equivalent to the 20% AEP rainfall for a 24 hour period.

The RORB model was used to assess the expected flows in Castle Creek in the September 2010 flood event. The model includes the recorded rainfall at the Telfords Bridge, Hillside and Morella gauges on an hourly basis and a number of loss rate scenarios were assessed to determine the range of expected flow rates at Euroa as shown in Table 4-27. The initial loss rate is the expected adsorption into the soil at the start of a flood event, prior to runoff occurring, and the continuing loss rate is effectively the amount of rainfall per hour that is percolated into the soil.

Table 4-27 Model flow rates at Telfords Bridge, September 2010

<table>
<tr><th>Initial Loss (mm)</th><th>Continuing Loss (mm/h)</th><th>Flow at Telfords Bridge (m^3/s)</th><th>Comment</th></tr>
<tr><td>0</td><td>0</td><td>81.2</td><td>Maximum possible flow</td></tr>
<tr><td>0</td><td>2</td><td>38.0</td><td rowspan="4">Estimated loss rates in this range</td></tr>
<tr><td>20</td><td>1.5</td><td>35.8</td></tr>
<tr><td>10</td><td>2</td><td>33.3</td></tr>
<tr><td>20</td><td>2</td><td>28.2</td></tr>
<tr><td>30</td><td>2</td><td>20.8</td><td></td></tr>
<tr><td>50</td><td>1.5</td><td>12.8</td><td>High initial loss case</td></tr>
</table>

At the most likely loss rates (0 to 20 mm initial loss, 1.5 to 2 mm continuing), the flow rates are in the order of 28 - 38 m^3/s. The recurrence interval of this peak flow rates is discussed in Section 4.7.4.

4.7.2 December 2010

A second event occurred in December 2010 that caused flooding in the area around Euroa. This event was of a smaller magnitude than the September on the Seven Creeks system event but was still significant in a historical sense. At Seven Creeks, the magnitude of the flood was estimated at the Euroa gauge as 118 m^3/s, equivalent to a 20% AEP. This flow rate is lower than that recorded in the September flood event at the Seven Creeks gauge. Although rare, the experience of having two major flood events in the same year is not impossible. It should be noted that on the Castle Creek system, this event was of a larger magnitude than the September event.

The rainfalls recorded for the December flood event in the Castle Creek catchment are shown in Table 4-28.

Table 4-28 Recorded Rainfall Castle Creek, December 2010

Date	Telfords Bridge (mm)	Hillside (mm)	Morella (mm)
2/12/2010	12.8	24.8	19
3/12/2010	21.2	22.2	13.4
4/12/2010	0.6	1	0.4
5/12/2010	0	0	0
6/12/2010	0	0	0
7/12/2010	25.4	31.6	31.2
8/12/2010	45	44	58.8
9/12/2010	1	1.8	0.6

It should be noted that these values are for the actual 24 hour period (midnight to midnight) on the date shown and are a sum of values from the hourly data obtained from the Bureau of Meteorology. The rainfall was distributed across the catchment, with higher intensity rainfall occurring in the upstream reaches of the catchment. The average rainfall total for the three stations on 7th December was 29.4 mm and on 8th December was 49.2 mm.

The analysis of the rainfall data for the December 2010 event shows an event that had two significant, very intense, rainfall peaks occurring between 3 and 5 pm on the 7th of December and 1 and 2 pm on the 8th of December.

The expected flows in Castle Creek in the December 2010 flood event have been assessed using a range of loss rates using the existing RORB model. A number of loss rates were assessed to determine the range of expected flow rates at Euroa as shown in Table 4-29.

Table 4-29 Model flow at Telfords Bridge, December 2010

Initial Loss (mm)	Continuing Loss (mm/h)	Flow at Telfords Bridge (m^3/s)	Comments
0	0	80.3	Maximum possible flow rate
10	1.5	60.4	Expected losses around these levels
0	2	55.7	
10	2	55.7	
20	2	55.6	
30	2	55.4	
50	1.5	41.9	High initial loss case

At the most likely loss rates (0 to 20 mm initial loss, 1.5 to 2 mm continuing), the flow rates are in the order of 55 - 60 m^3/s. The December event, due to the intense rainfall peaks and the flow timing was less sensitive to changes in the rainfall loss than the September event. In particular the peak flow estimates were not impacted by the initial losses, but more so by the continuing loss. The recurrence interval associated with this event is discussed further in Section 4.7.4.

4.7.3 October 1992 and October 1993 floods on Castle Creek

Although the Castle Creek catchment could not be calibrated using gauged data for the 1992 and 1993 events there was an assessment of these events undertaken in the Euroa Floodplain Management Study: Final Report Volume 2 (SKM, 1997). This has also been verified through hydrological modelling in this study.

The assessment of these two events for Castle Creek was undertaken by SKM constructing a hydrological model extending down to Castle Creek at Arcadia for calibration. The October 1992 event was a short duration event (estimated as a less than 2 hour burst) which had a concentrated burst upstream of Euroa. The estimated depth of rainfall for this event ranges from 32 to 75 mm (daily total). SKM estimated that the initial loss was 20 mm and the continuing loss was 2 mm/h for this calibrated event. The resulting peak estimated by SKM at Euroa was 85 m^3/s based on their assessment. The October 1992 event was noted as being higher than the October 1993 event which was attested by residents in Euroa (SKM, 2007). For this current assessment the 1992 event was not directly assessed as there was insufficient pluviograph information to adequately define the peak burst of rainfall.

The October 1993 flood event was associated with a longer distribution of rainfall. The recorded depth of rainfall for this event was approximately 130 mm over a 5 day period. The extended distribution of the 1993 event rainfall resulted in a peak flow rate predicted by SKM of 45 m^3/s. For this study Cardno have also assessed the 1993 event through the developed RORB model. For this event there was a reliable pluviograph distribution of the rainfall which allowed for a more detailed assessment. The RORB model was used to assess a range of loss rates (including the SKM adopted values), the results are shown in 0.

Table 4-30 Model flow rates, October 1993

Initial Loss (mm)	Continuing Loss (mm/h)	Flow at Euroa (m^3/s)	Comment
0	0	106.2	Maximum possible runoff
10	1.5	75.2	
0	2	69.9	
10	2	68.2	
20	2	66.1	
30	2	61.5	
15	2.5	60.1	SKM adopted loss rates
30	2.5	54.7	

If the SKM calibrated loss rates are adopted then the peak flow rate estimated for this event is 60 - 70 m^3/s, this is higher than the SKM predicted peak of 45 m^3/s. Both estimates of the peak flow rate are within the realistic expectation for the peak flow during an event of this magnitude through the Castle Creek catchment. SKM note that there is a high level of uncertainty associated with their assessment. Cardno agrees that as these events are ungauged there is some uncertainty in the estimates but based on the rainfall depths the peak flow rates predicted seem reasonable.

It should be noted that this event is more sensitive to continuing loss than initial loss due to the length and total depth associated with the rainfall event.

Overall the estimated peak flow rates associated with each of these events was 85 m^3/s and 45 to 60 m^3/s for the October 1992 and the October 1993 events respectively. The recurrence interval of these events is discussed further in Section 4.7.4.

4.7.4 Recurrence Interval Assessment

In order to assess the peak flow rates estimated for the 1992, 1993 and 2010 events for Castle Creek the estimated design peak flow rates have been utilised. These have been developed as part of the hydrology assessment and are presented in Section 4.6.2. The estimated recurrence intervals for the historic Castle Creek events are summarised in Table 4-31. The range of exceedence probabilities has been estimated for the full range of estimated peak flow rates.

Table 4-31 Estimated recurrence intervals for the historical Castle Creek events

Event	Estimated Peak Flow at Euroa (m^3/s)	Estimate	AEP estimate	ARI estimate
October 1992	85	SKM, 1997	1.3% AEP	78 year ARI
October 1993	45	SKM, 1997	8.3% AEP	12 year ARI
	60	Cardno, 2015	4.2% AEP	24 year ARI
	70	Cardno, 2015	2.6% AEP	39 year ARI
September 2010	28	Cardno, 2015	18.3% AEP	5 year ARI
	38	Cardno, 2015	11.3% AEP	9 year ARI
December 2010	55	Cardno, 2015	5.0% AEP	20 year ARI
	60	Cardno, 2015	4.2% AEP	24 year ARI

The estimated peak flow rate for the 1992 event was just below the 1% AEP peak estimate. This event is the largest in recent history for Castle Creek and it was associated with a known short intense burst of rainfall over this area. There is some uncertainty associated with the estimated peak flow rate but this event is estimated be between the 1% and 2% AEPs.

The estimate for the October 1993 event has been assessed using the SKM estimate and the revised Cardno estimate. These estimates provide an expected range for the recurrence interval in the range of a 3 to 10%

AEP event. This event is predicted to be higher than that reported by SKM, it is understood that the SKM estimate for the peak flow for this event was argues by residents of Euroa to be too low.

September 2010 was less rare than the other events assessed with an estimate AEP of 10% to 20%. The event in December 2010 is expected to be rarer with flow rates similar to the October 1993 event. This again places the event in the range of a 5% AEP.

The flood extent for each event can be compared to the relevant design event in the hydraulic modelling section to identify the approximate flood behaviour for the historic event.

4.8 Sensitivity

Sensitivity of the RORB model was focussed on the k_c parameter as this is the primary control of the catchment response. The initial and continuing losses are arbitrarily assigned based on the matching of the FFA peak flow targets so these have not been assessed in the sensitivity assessment.

The k_c factor was adjusted by +/- 20% for each of the catchments to determine the impact on the peak flow rate. The results of this assessment are summarised in Table 4-32 and Table 4-33. For both of the catchments, decreasing the k_c parameter increased the flow rate as expected (this increases the response rate for the catchment). In both instances the increase was approximately 20-30%. When the k_c parameter was increased (decreasing the response rate for the catchment) the peak flows dropped by approximately 10-20%.

Table 4-32 Sensitivity on the design events for the Seven Creeks catchment

AEP	Design Peak (m^3/s)	Design Peak (m^3/s)	Difference	Design Peak (m^3/s)	Difference
	kc = 25	**kc = 20 (-20%)**		**kc = 30 (+20%)**	
20%	123	146	+19%	103	-16%
10%	184	220	+20%	160	-13%
5%	246	299	+22%	217	-12%
2%	331	390	+18%	295	-11%
1%	398	475	+19%	357	-10%
0.5%	468	564	+21%	421	-10%
0.2%	563	687	+22%	503	-11%

Table 4-33 Sensitivity on the design events for the Castle Creek catchment

AEP	Design Peak (m^3/s)	Design Peak (m^3/s)	Difference	Design Peak (m^3/s)	Difference
	kc = 13.8	**kc = 11.0 (-20%)**	**%**	**kc = 16.6 (+20%)**	**%**
20%	25.5	32.2	+26%	20.3	-20%
10%	39.8	47.5	+19%	32.0	-20%
5%	54.9	67.0	+22%	44.6	-19%
2%	73.4	89.2	+22%	59.9	-18%
1%	89.7	109.8	+22%	73.8	-18%
0.5%	106.9	131.8	+23%	88.4	-17%
0.2%	130.8	161.8	+24%	108.2	-17%

4.9 Climate Change

The climate change modelling was undertaken assuming a low, medium and high change in rainfall intensity. This was modelled by assuming an increase in the intensities in the IFD parameters by 10%, 20% and 30% respectively. This range of increases allows for an examination of the possible impacts of climate change in the short, medium and long term.

The intensity increases for Seven Creeks are summarised in Table 4-34 for the 10%, 20% and 30% climate change scenarios. The resulting changed in the peak flow rates are summarised in Table 4-35.

The low climate change scenario (10% intensity increase) increased the peak flow rates by between 17 to 28%. The medium climate change scenario (20% intensity increase) resulted in increases in peak flow rates from 34 to 59%. The high climate change scenario (30% intensity increase) resulted in increases in peak flow rates from 52 to 91%.

The more frequent events (i.e. 20% and 10% AEP) showed proportionally larger increases than the rarer events as a result of climate change for the low, moderate and high climate change scenarios.

Table 4-34 Intensity-Frequency-Duration parameters with climate change for Seven Creeks

IFD Coefficient	Value	10% CC	20% CC	30% CC
${}^{2}I_{1}$	25.58	28.14	30.70	33.25
${}^{2}I_{12}$	5.05	5.56	6.06	6.57
${}^{2}I_{72}$	1.54	1.69	1.85	2.00
${}^{50}I_{1}$	45.64	50.20	54.77	59.33
${}^{50}I_{12}$	9.18	10.10	11.02	11.93
${}^{50}I_{72}$	2.9	3.19	3.48	3.77
G (skew)	0.22			
F2	4.31			
F50	15.11			
Zone	2			
Location				
Latitude	36.850 S			
Longitude	145.725 E			

Table 4-35 Climate change events generated for the Seven Creeks catchment

AEP	Design Peak (m^3/s)	Climate Change Peaks (m^3/s)					
	Existing	10% Increase	Difference	20% Increase	Difference	30% Increase	Difference
20%	123	157	28%	195	59%	235	91%
10%	184	228	24%	272	48%	320	74%
5%	246	300	22%	355	44%	411	67%
2%	331	394	19%	460	39%	520	57%
1%	398	469	18%	541	36%	616	55%
0.5%	468	548	17%	633	35%	716	53%
0.2%	563	658	17%	757	34%	857	52%

The intensity increases for Castle Creek are summarised in Table 4-36 for the low (10% increase), medium (20% increase) and high (30% increase) in intensity. The resulting increases in the peak flows for castle Creek are summarised in Table 4-37.

The low climate change scenario (10% intensity increase) increased the peak flow rates by between 17 to 36%. The medium climate change scenario (20% intensity increase) resulted in increases in peak flow rates from 36 to 76%. The high climate change scenario (30% intensity increase) resulted in increases in peak flow rates from 56 to 111%.

As for Seven Creeks, the more frequent events (i.e. 20% and 10% AEP) showed proportionally larger increases than the rarer events as a result of climate change for the low, moderate and high climate change scenarios.

Table 4-36 Intensity-Frequency-Duration parameters with climate change for Castle Creek

IFD Coefficient	Value	10% CC	20% CC	30% CC
$^{2}I_{1}$	24.56	27.02	29.47	31.93
$^{2}I_{12}$	4.18	4.60	5.02	5.43
$^{2}I_{72}$	1.29	1.42	1.55	1.68
$^{50}I_{1}$	45.99	50.59	55.19	59.79
$^{50}I_{12}$	7.17	7.89	8.60	9.32
$^{50}I_{72}$	2.03	2.23	2.44	2.64
G (skew)	0.21			
F2	4.31			
F50	15.09			
Zone	2			
Location				
Latitude	36.800 S			
Longitude	145.575 E			

Table 4-37 Climate change events generated for the Castle Creek catchment

AEP	Design Peak (m^3/s)	Climate Change Peaks (m^3/s)					
	Existing	10% Increase	Difference	20% Increase	Difference	30% Increase	Difference
20%	25.5	34.8	36%	44.8	76%	53.9	111%
10%	39.8	50.7	27%	60.7	52%	72.5	82%
5%	54.9	66.5	21%	80.0	46%	93.8	71%
2%	73.4	88.8	21%	105.3	43%	122.2	66%
1%	89.7	107.2	20%	125.7	40%	144.7	61%
0.5%	106.9	126.5	18%	147.0	37%	169.1	58%
0.2%	130.8	153.5	17%	178.2	36%	203.9	56%

The implications of the climate change assessment on both catchments is that in the future it is likely that increased rainfall intensity will lead to increases in peak flow rates across the catchments. The most significant increases are likely to be experienced in the more frequent events with a possible doubling of the current peak flow rates if the full 30% increase in intensity is observed. The impact of the increased flow rates will be assessed in the hydraulic model to determine the impact on flood extent, depth, hazard and duration. This has been assessed in the hydraulic modelling chapter.

4.10 Probable Maximum Flood

The Probable Maximum Flood (PFM) has been determined using the Generalised South Australian Method (GSAM) and the Generalised Short Duration Method (GSDM). The durations considered ranged from the 1 hour event up to the 72 hour event. The details of the Probable Maximum Precipitation (PMP) is summarised in Appendix B.

The RORB models for Seven Creeks and Castle Creek were used to generate the PMF events using the temporal distributions and PMP estimates for the catchments as specified in the GSDM and GSAM methods. Each model was run with the calibrated k_c and 'm' parameters. The loss rates were specified at 20 mm and 2.0 mm/hour for the initial loss and continuing loss rates respectively.

The peak flows rates for the PMF event are summarised in Table 4-38. The critical duration for each catchment is the 12 hour event which coincides with the design critical durations.

Table 4-38 Probable Maximum Flood peaks

Duration	Seven Creeks at Euroa	Castle Creek at Euroa
	Peak flow (m^3/s)	**Peak flow (m^3/s)**
6 hour	4,085	1,170
12 hour	5,140	1,230
24 hour	4,260	950
36 hour	3,860	910
48 hour	3,310	740
72 hour	2,390	590

4.11 Flood Timing Assessment

An examination of the five peak flood events for the Seven Creeks catchment was undertaken using firstly the recorded data and secondly the hydrological model. For the Castle Creek catchment no analysis was possible for the recorded data as this is very limited.

Table 4-39 shows the largest 5 events recorded at Euroa for Seven Creeks. The table identifies the approximate time the rainfall started and when the peak burst was recorded. It should be noted that this is approximate as each event has a different preceding rainfall pattern. The pluviographs from both Euroa and Strathbogie were used to identify the peak rainfall bursts. The approximate timing of the peak rainfall and the peak flow occurring in Euroa is estimated based on this information.

Table 4-39 Flood timing for the large events in Seven Creeks

Event	Rainfall		Seven Creeks at Euroa		Travel time	
	Start of Rainfall	**Peak burst**	**Peak (m^3/s)**	**Date and time**	**Start Rain to Euroa**	**Burst to Euroa**
Oct 1993	~5:30pm	~9:00pm	284.9	4/10/1993 8:34	15 h	11 h
Oct 1992	~2:30pm	~4:30pm	198.9	17/10/1992 21:09	7 h	5 h
Sep 2010	~3/09/2010 11:00pm	~9:00am	189.9	4/09/2010 19:15	20 h	10 h
Jul 1986	~4:00pm	~9:30pm	145.0	24/07/1986 2:15	10.5 h	5 h
Oct 1984	~ 11:30am	~3:00pm	142.2	3/10/1984 21:49	9.5 h	6 h

[1] The gauge was exceeded for all events and the peak flow rate was not captured.

The second component of this assessment was to use the RORB model to assess the approximate travel times based on the calibrated models. This was assessed for the five calibrated models. The results of this assessment are summarised in Table 4-40. The table is in the form of the calibrated peak for Seven Creeks at Polly McQuinn Weir and for Euroa and the RORB model time this occurred. The delay between the peaks has then been calculated. Overall the results indicate that the peaks can range from coincident (likely due to rainfall patterns) to a delay of 6 hours. It should be noted that the reliability of this analysis is very low because the Polly McQuinn Weir peak flow is an estimate only based on the hydrological model as the peak flows are not captured at this gauge.

Table 4-40 Flood timing for the RORB modelled events in Seven Creeks

Event	Seven Creeks at Polly McQuinn Weir		Seven Creeks at Euroa		Travel time
	Peak (m^3/s)	RORB model time	Peak (m^3/s)	RORB model time	Polly McQuinn Weir to Euroa
Oct 1993	146.1	122 h	284.4	128 h	6 h
Oct 1992	77.2	45 h	194.4	45 h	0 h
Sep 2010	194.7	38.5 h	191.3	44.5 h	6 h
Jul 1986	93.7	50 h	140.5	50 h	0 h
Oct 1984	90.0	45.5 h	141.3	46.0 h	0.5 h

Overall it is evident that there is approximately a 5 to12 hour warning window from the time large rainfall bursts occur on the Seven Creeks catchment to the time the peak flow is observed in Euroa for large flood events.

This catchment response rate is expected for this catchment, particularly due to the elevated upper catchment within the Strathbogie Ranges. Currently the Bureau of Meteorology (BoM) run an URBS model for Seven Creeks, however the primary purpose of this model is to provide inputs into Goulburn River for flood warning for Shepparton.

Currently the BoM operate streamflow monitoring at the following locations within the Seven Creeks and Castle Creek catchments to Euroa:

- Strathbogie Fld (82151)
- Polly McQuinn Weir (82154)
- Galls Gap Road (82150)
- Euroa Fld (582017)
- Telfords Bridge (82149).

The locations provide updates on the current warning levels based on the current river heights. Each of these locations has minor, moderate and major flood warning levels. Additional details of the flood warning are discussed in the Flood Intelligence section of this report. This assessment was focussed on a broad assessment of the hydrology only.

5 Hydraulic Modelling

5.1 Hydraulic Model Establishment

The BMT WBM 1D2D modelling system, Tuflow, was used to compute the channel (1D) and overland flow (2D) components of the study.

This combined package allows for the computation of channel and pipe flow (including structures such as culverts, weirs, gates and pumps, and pipe details such as inverts, obverts, pipe sizes and pipe material) by the 1D module, which is then dynamically linked to the 2D overland flow module. The 1D and 2D domains are coupled at 1D-calculation points (such as manholes) whenever they overlap each other.

The advantages of this system are that the channel/pipe system is explicitly modelled as a sub-system within the two-dimensional overland flow computation. This means that generalised assumptions regarding the capacity of the channel/pipe system are not required.

5.2 Hydraulic Model Development

The hydraulic models consist of two main hydraulic components:

- The channel network for structures (1D); and
- 2D grid of the surface topography.

The establishment of these two components of the model is described in the following sections.

For the Euroa Post Flood Mapping and Intelligence project the Castle Creek and Seven Creeks are the main tributaries which contribute to flooding within the study area. The model area is shown in Figure 5-1.

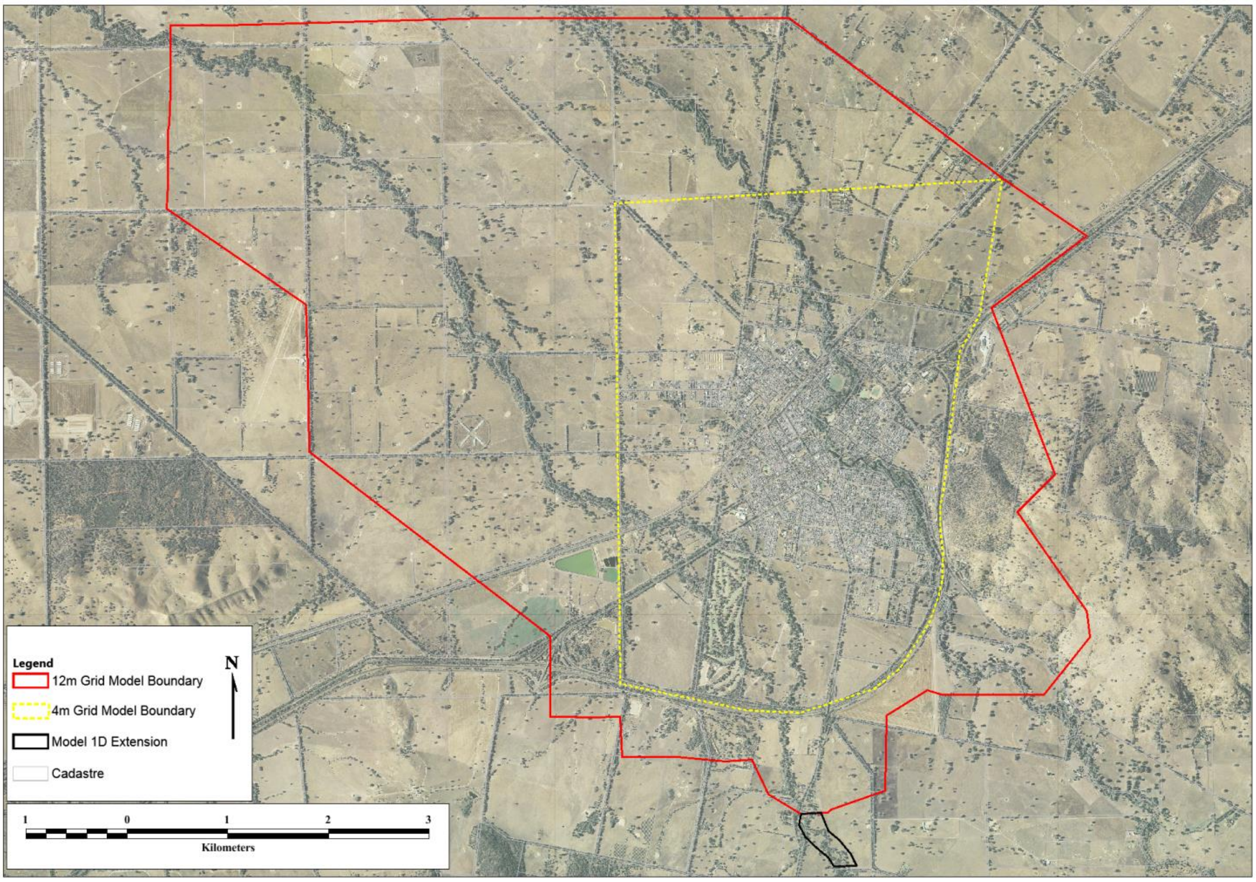


Figure 5-1 Model boundary for the Euroa Hydraulic Model

5.2.2 Channel and Structure System (1D)

Survey was captured for the study area and this information has been presented in Section 3. The locations of the cross sections captured are summarised in Figure 3-1. In addition to this survey, significant existing information of structures within the study area was provided. Overall there were 8 cross sections surveyed. Some cross sections were omitted from the hydraulic model as they did not provide additional resolution.

5.2.3 Topography (2D)

The topography was defined using a Digital Terrain Model (DTM) of the region. The DTM was derived from the 2011 LiDAR data within the software package 12D. A 12 m grid for areas outside the area of interest, coupled with a 4m, high resolution grid was selected for use within the hydraulic model. The dimensions of the grids are summarised in Table 5-1.

Table 5-1 Topography Grid Size

Parameter	Grid
Grid Size	12m x 12m
Grid Cells (x direction)	805
Grid Cells (y direction)	693
Grid Size	4m x 4m
Grid Cells (x direction)	998
Grid Cells (y direction)	1347

The nested topography layer was set using the grid cell size of 4m x 4m as this provided enough detail to capture the surface elevation details without causing computation run times and size of results to be excessive. The DTM is shown in Figure 5-3. The grid cell size selected was the finest detail possible without causing runtimes of multiple days while also replicating the known surface appropriately. The major channel definition is approximately 16 m wide at the base and 100 m wide at the top of bank which corresponds to 25 grid cells within the model. This is an adequate number of cells for accurate definition of the channel within the 2D domain. An example cross section of the floodplain is shown in Figure 5-2.

5.2.4 Roughness

The hydraulic roughness was determined from the calibration presented in Section 5.3. This has been shown in Figure 5-4. The values used in the roughness are summarised in Table 5-2.

Table 5-2 Manning's Roughness

Description	Manning's Roughness
Roads and Impervious Surfaces	0.018
Railway line and embankment	0.02
Farmland	0.055
Rivers / Creeks	0.06
Dense trees / township area	0.08
Buildings / Commercial / Industrial	0.5

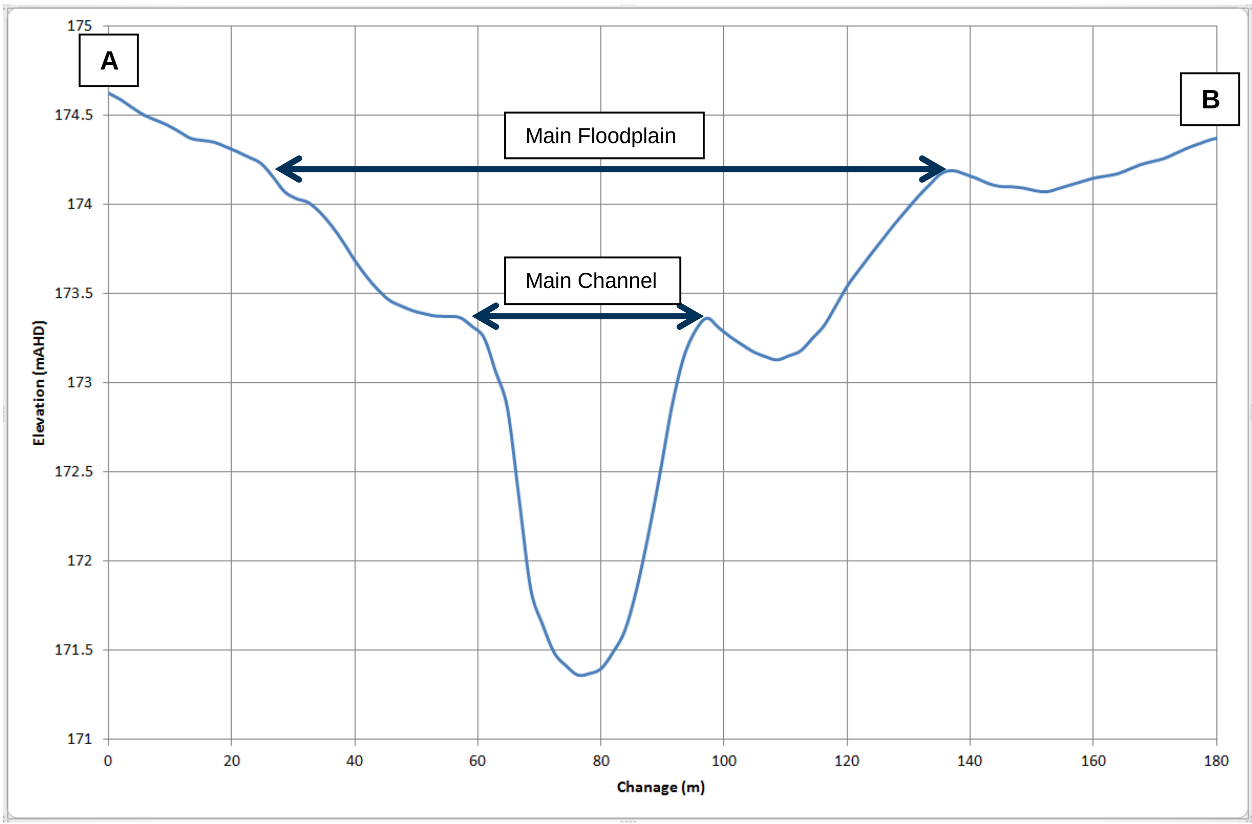


Figure 5-2 Example Cross Section of Seven Creeks near Euroa

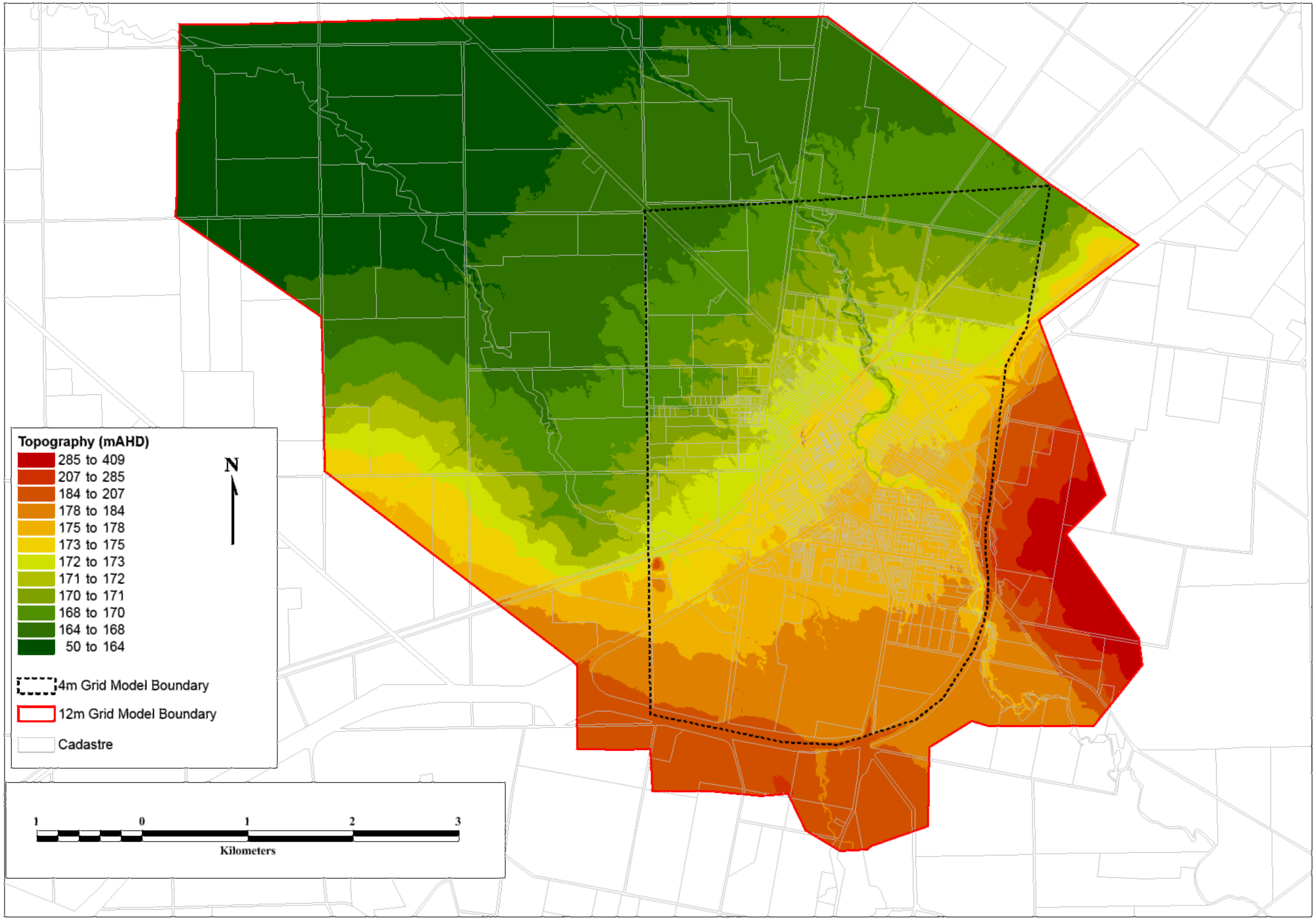


Figure 5-3 Digital Terrain Models (DTM) for the Euroa Model

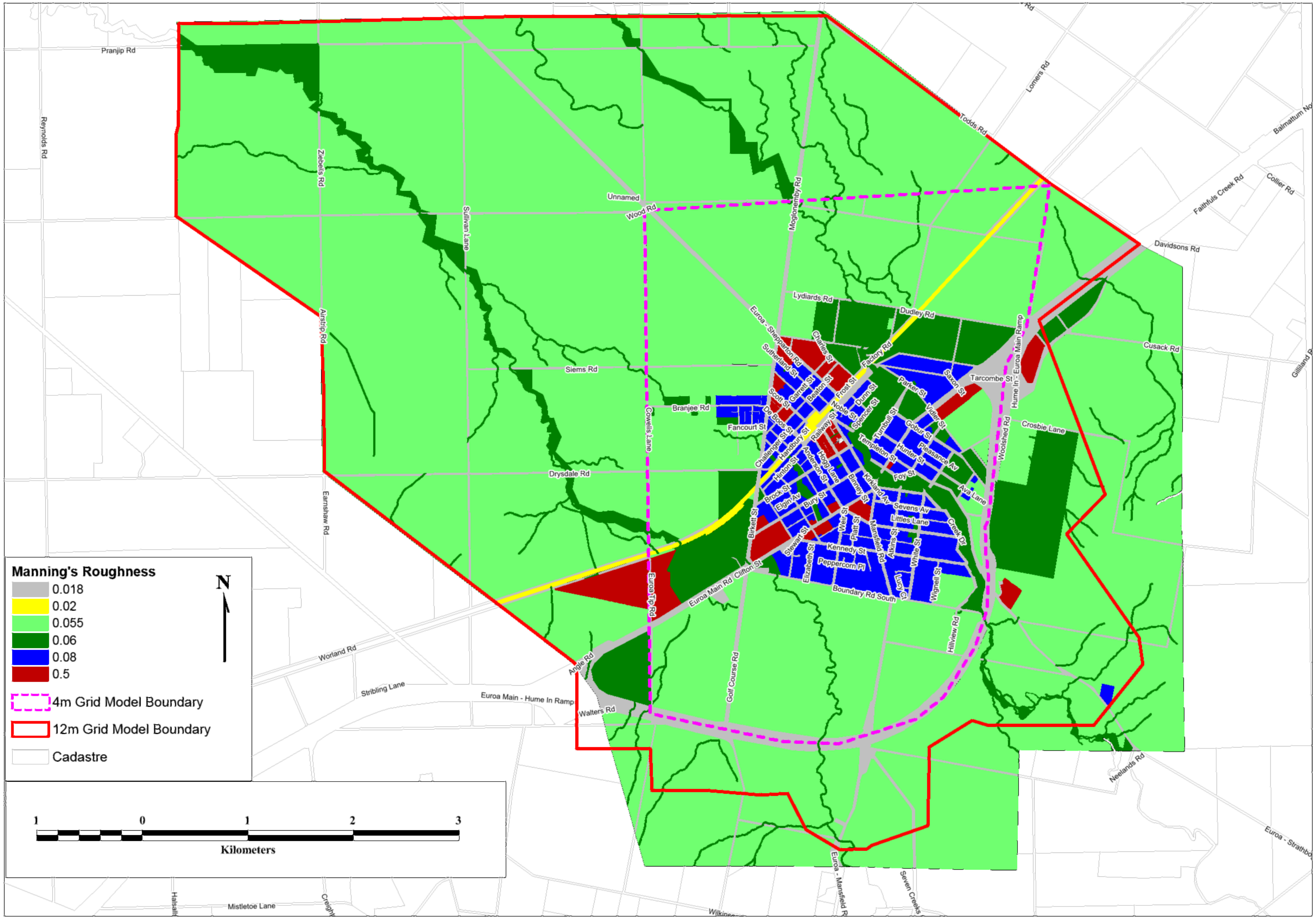


Figure 5-4 Hydraulic Roughness for the Euroa Model

5.3 Hydraulic Model Calibration

Two major flood events have occurred in recent history at Euroa that had peak streamflow levels, peak flood levels and extents captured that can be used to calibrate the hydraulic model. The two events where levels have been captured included the September 2010 and October 1993 flood events. Of these events the September 2010 flood event had a significantly larger set of recorded flood heights that captured the peak flows at Euroa. Each of these calibration events have been used in order to calibrate the hydraulic model. The main calibration variables within the hydraulic model include the roughness parameters and the losses across the structures.

5.3.1 October 1993

The October 1993 flood event was the largest flood event with recorded flood peak information. The event has been presented and discussed in detail in Section 4.4.2. The event had a peak flow rate of 284 m^3/s recorded at the Seven Creeks at Euroa gauge. The flood event contained two rainfall bursts which caused Seven Creeks to rise on the 2^{nd} October 1993, the hydrograph falls away over the subsequent days before the main flood event occurred on the 4^{th} October 1993. To calibrate the model the inflow to Seven Creeks was set at the gauge and was controlled by the recorded hydrograph at this location.

Castle Creek did not have a streamflow record for this event and the inflows were derived from the calibrated RORB model for castle Creek. The kc, m and loss rates were taken from the Seven Creeks RORB model which was calibrated to the 1993 event. The peak flow rate generated for Castle Creek was estimated at approximately 95 m^3/s.

The Goulburn Broken CMA had captured 257 recorded and observed flood heights of varying accuracy. This information was supplied to Cardno as a GIS layer that provided a peak flood level and a brief description of the location and method utilised in deriving the peak flood height. For this calibration the Castle Creek levee was excluded as this levee was not constructed in 1993, the partial levee constructed in the 1950s was retained in the topography. The captured points were all on the Seven Creeks floodplain.

The process undertaken to calibrate the hydraulic model was to utilise the recorded points and compare the modelled results to the points. The manning's roughness was the primary method of adjusting the model behaviour, however some checks and changes were required to the 1D elements of the model.

From the 257 recorded calibration points there were 4 points which were clearly in significant error due to the peak flood levels being over 3 metres different to the modelled results. These points were excluded from the calibration (Ids 714, 770, 772 and 773 from the Goulburn Broken CMA calibration point database). There were also 33 points which did not intersect the flood extent, many of these points may have been local drainage flooding which was not captured in this model.

For the remaining points the following statistics were calculated from the assessment of the recorded flood heights against the modelled 1993 event, this information is also shown graphically in Figure 5-5.

- Number of points assessed: 220
- Average difference: - 0.029 m
- Minimum difference: - 0.537 m
- Maximum difference: 0.49 m
- Standard Deviation of points: 0.139 m.

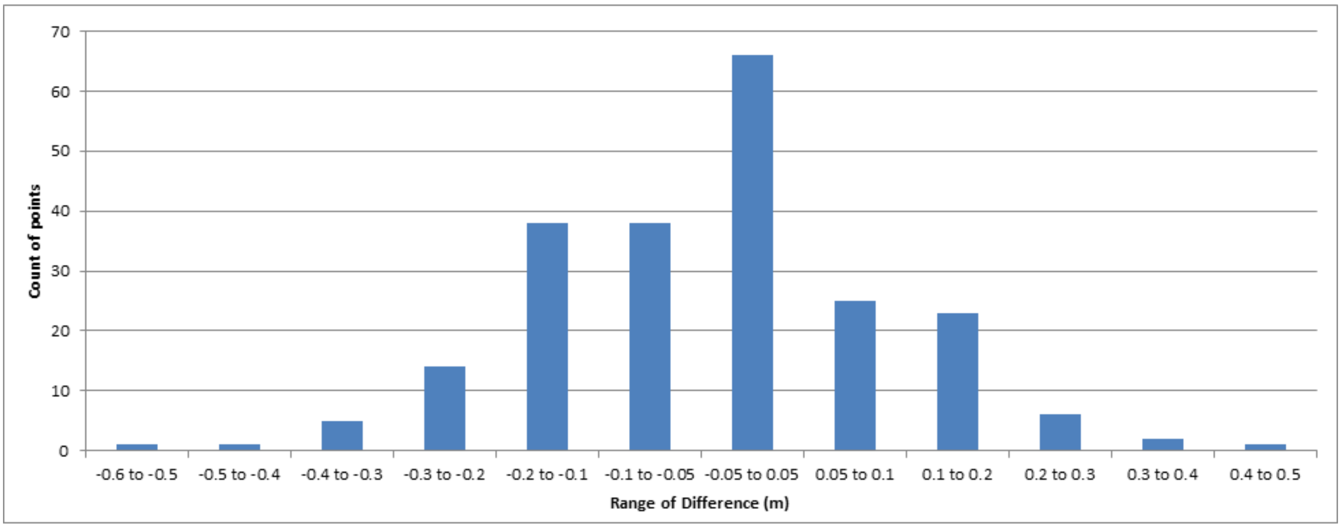


Figure 5-5 Statistics from the calibrated 1993 event at Euroa

From a large sample of calibration points, the average difference between the model and the recorded points was -0.03 m. That is the modelled points are marginally lower than the recorded points. Figure 5-5 shows the distribution of the differences. The range of differences may be partly attributed to the fact that the local drainage issues are not modelled and no direct rainfall is applied to the hydraulic model.

There were no points for the Castle Creek area in 1993 for comparison. The only location where there were some calibration points were on the anabranch adjacent to Castle Creek. There were about 23 points in this location and the average difference was +0.08 m.

It was noted that across the floodplain many of the recorded peak flood heights for the 1993 event were not consistent with nearby recorded peak flood heights varying by significant amounts over small distances. Some variability in the captured peak of the event is expected due to the fact that it is difficult to capture the peak level during the event and that most points would have been captured from debris lines and water marks on objects following the event. Points also would be sourced from various sources which add to the uncertainty in some of the recorded peak heights. The water marks and debris lines are influenced by wave action and turbulence during the flood and as such there may be differences in the order of +/- 30 cm for the recorded levels. An example of wave action causing differences in peak flood heights is from the wake of vehicles driving through the floodwaters or flow pushing against an object at high velocity. These examples do not aim to discredit the recorded information but more to make it clear that the surface of floodwaters are not flat and many factors can raise and lower recorded information across short distances.

Figure 5-6 shows the differences between the modelled 1993 event and the recorded points. In this figure it is evident that some of the comparisons between the recorded and modelled points are in direct conflict with the model under and over predicting the peak levels in similar areas. This shows that there is some variability in the recorded data that cannot be resolved and the use of the statistical approach to determine the calibration is the most appropriate method for measuring the calibration. The approach adopted at calibrating the event shows that the peak flood levels are largely well matched.

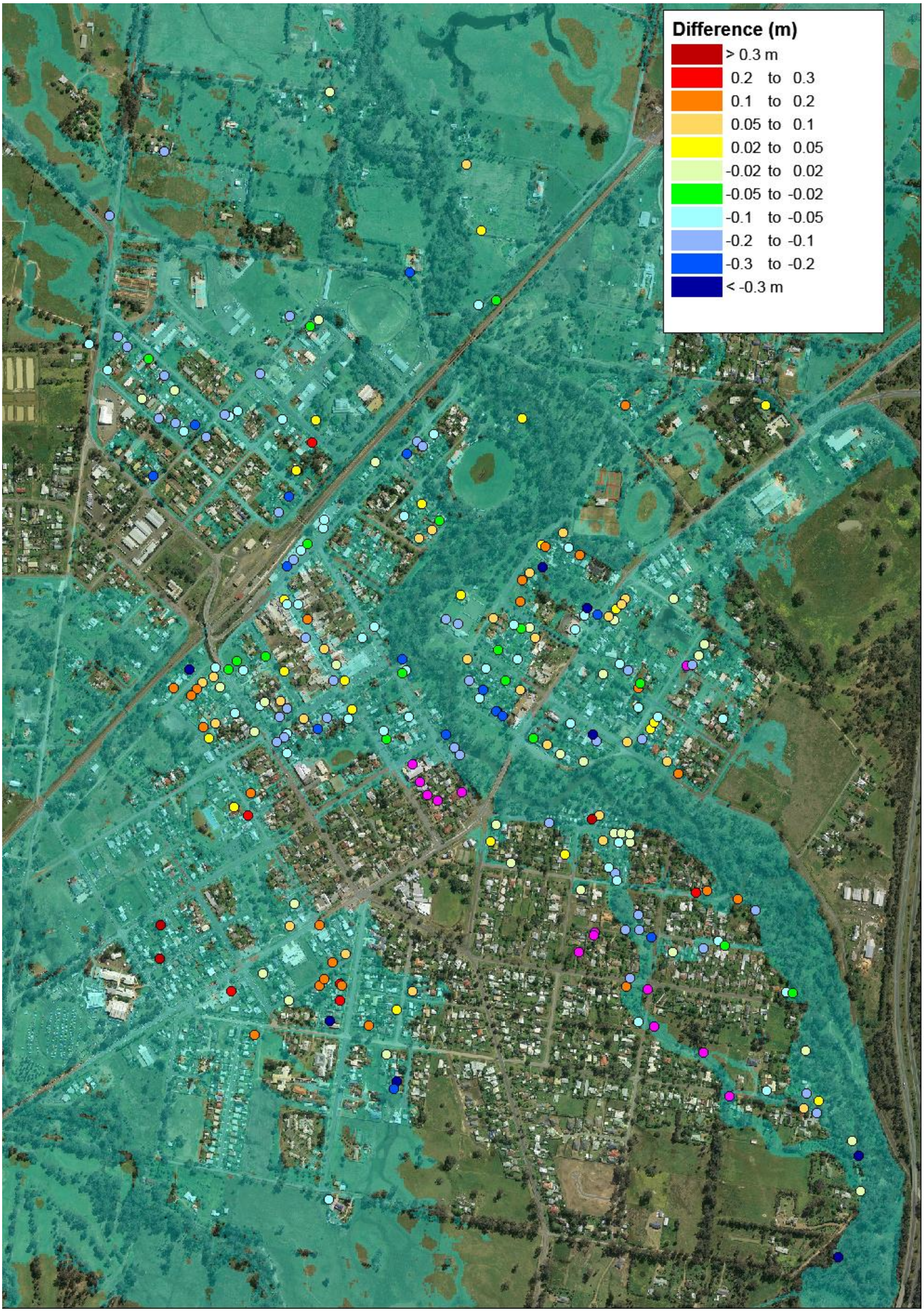


Figure 5-6 Difference plot of the calibrated points for the 1993 event at Euroa (modelled less the recorded 1993 flood heights)

Overall the 1993 event was well represented across the floodplain. Also obtained from the Goulburn Broken CMA was an image captured during the peak of the 1993 event. This image has been overlayed with the flood extent from the calibrated TUFLOW model in Figure 5-7. Although it is hard to discern the exact flood extent from the aerial image is it clear that the model is reproducing similar extents.

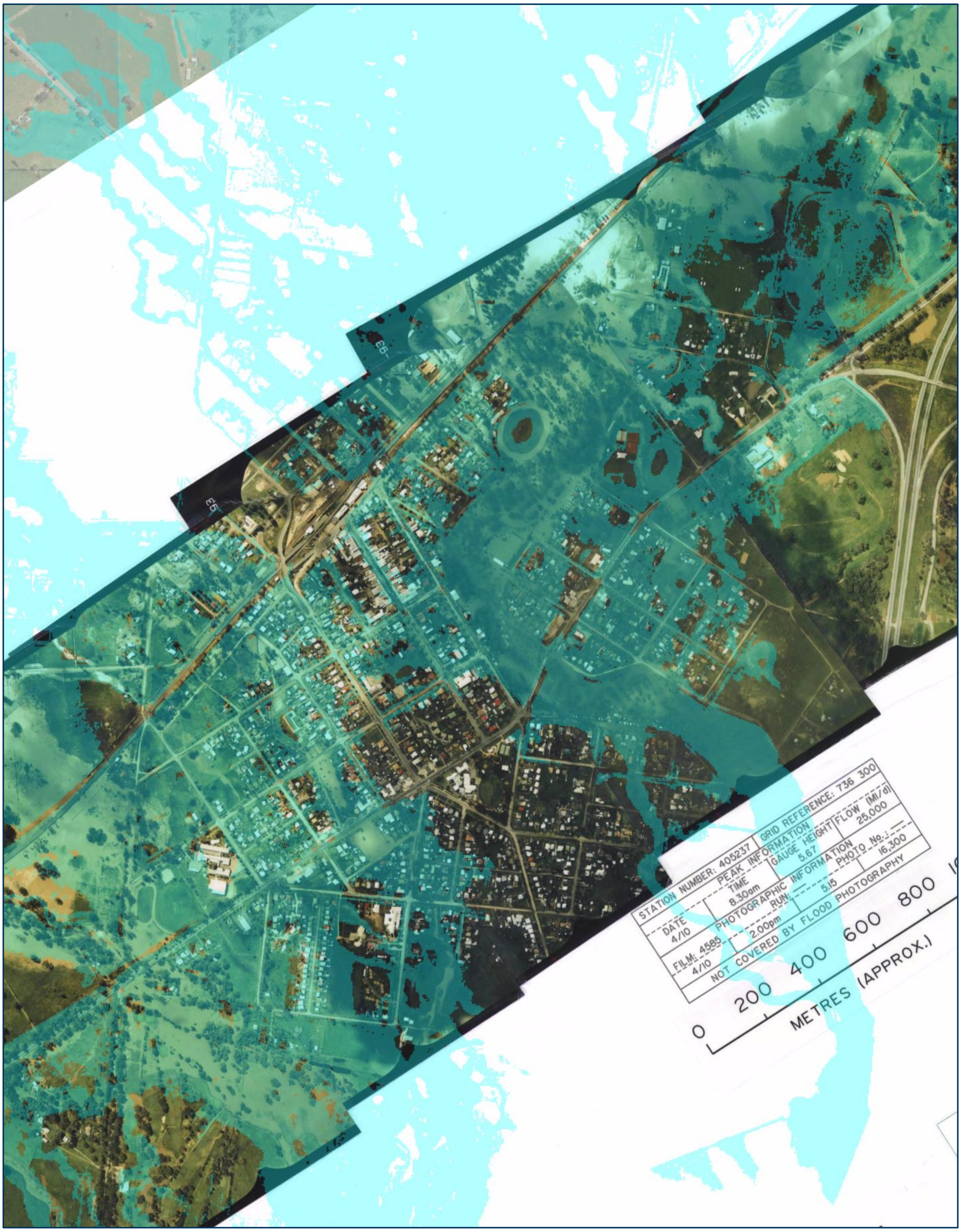


Figure 5-7 Modelled 1993 extent overlayed on the peak flood image

5.3.2 September 2010

The September 2010 flood event had a peak flow rate of 187 m^3/s. The event was driven by a large amount of rainfall falling on the upper catchment within the Strathbogie Ranges. As such the peak of this event had a steep rising limb. This is the most recent large flood event occurring at Euroa. Castle Creek was run with a peak flow rate of 24 m^3/s (derived from RORB using parameters from Seven Creeks). The details of this event are presented in Section 4.4.4. It should be noted that the peak flow rate for Castle Creek had a high degree of uncertainty and from sensitivity on the loss rates the peak flow rate was likely to be in the range of 24 to 55 m^3/s for this event.

For this event there were 29 recorded flood levels, 11 of these were along the levee along Castle Creek and the remaining 18 points were from the Seven Creeks gauge at Euroa to past the Buttery Factory. Along Seven Creeks many of the points were clustered around areas which limited the spatial distribution of the calibration points. The levels on Castle Creek were not a good guide for the calibration of the flood model as there was no indication of the flood extent across the broader floodplain, rather these were levels against the levee. Given the uncertainty of the peak flow rate the focus of the calibration was for Seven Creeks.

For this event Cardno had two sources of information for Seven Creeks regarding the peak flow, including,

- Victorian Data Warehouse – 187 m^3/s peak
- Goulburn Broken CMA – 225 m^3/s peak.

From the initial assessment of the information it was evident that a change in the rating table had been introduced since the 2010 event. The rating table adjustment introduced some uncertainty in the flow rates and to the recorded gauge heights.

The recorded gauge height for this event was 177.88 mAHD at the peak of the event (sourced from GBCMA). In addition to the recorded gauge heights there was a point recorded at this location from the 2010 recorded level dataset that also had a peak at 177.88 mAHD which is stated as have high reliability.

To ensure a reliable calibration for the model the calibration run was undertaken using the peak level set at the recorded level at the gauge of 177.88 mAHD. This run produced a good calibration using the same catchment parameters as per the 1993 calibrated run. The results of the final calibration run are shown in Figure 5-8 for Seven Creeks In general the flood levels are between +/- 0.10 m of the recorded levels across the study area.

Near the Butter Factory on Seven Creeks the peak flood depths were not well matched with the model being lower than the recorded flood levels. There is a recorded height on the western side of this property that was not inundated in the current model run but matches the maximum flood extent. There are two points on the eastern side of the site that are stated as 172.99 mAHD and 172.63 mAHD (0.36 m difference) and the points are only 10m apart, so both of these points cannot be matched. Downstream of the Butter Factory the next locations are well matched (-0.09 m and +0.13 m) by the model.

Overall the 2010 event was well represented by the hydraulic model through the floodplain. There were some concerns with recorded flood levels and the peak level at the gauge but overall the model is producing levels consistent with the majority of the recorded flood levels using the same roughness parameters as the 1993 event.

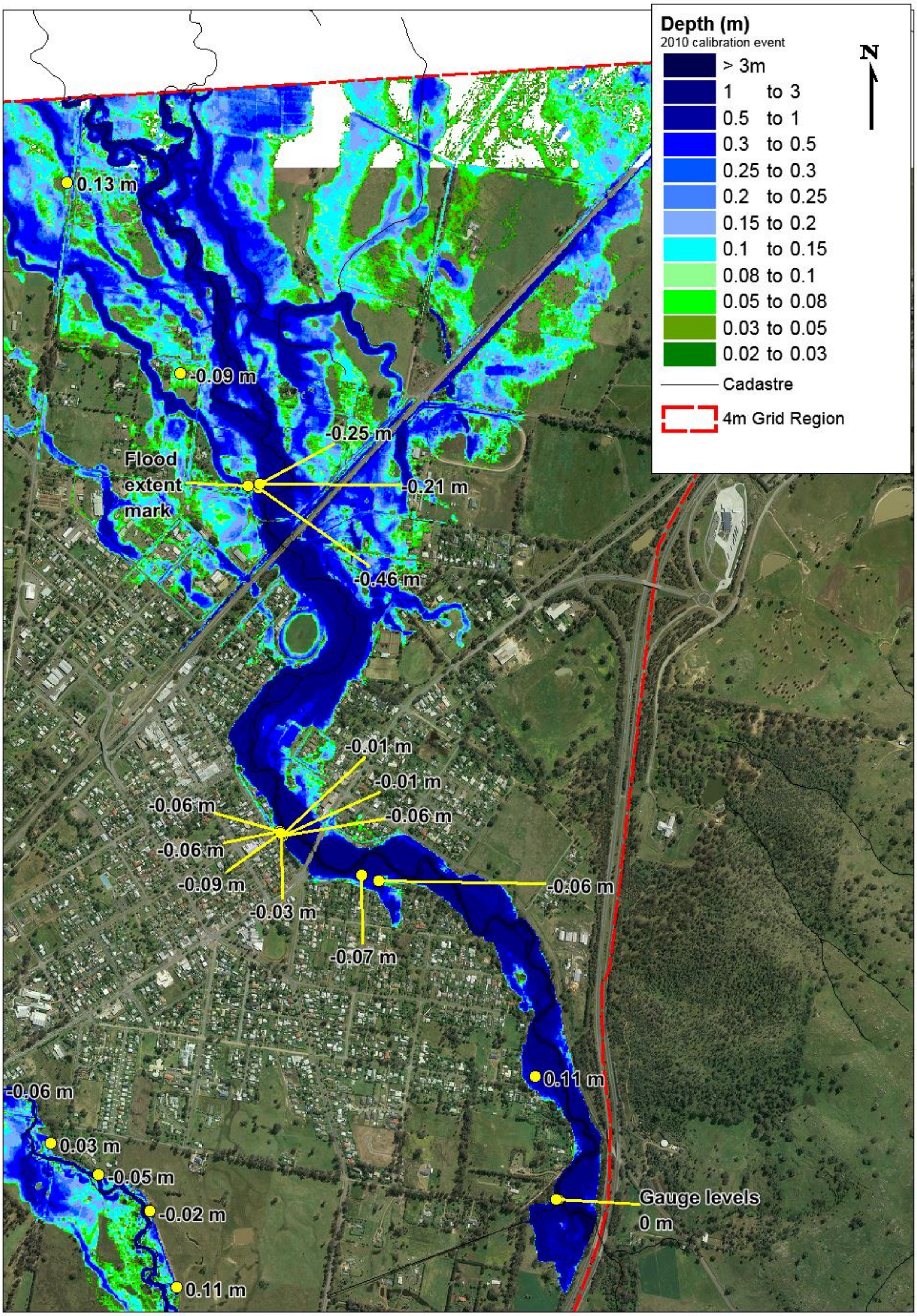


Figure 5-8 Calibrated 2010 event for the Seven Creeks based on level boundary (modelled less recorded flood heights)

5.4 Modelling of Design Events

For this study design events were simulated through the hydraulic model for the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP and the PMF events. The details of the derivation of the design events is summarised in Section 4.6. For each event the 6 and 9 hour durations were simulated, aside from the PMF event which had a critical duration of 12 hours.

The design event peak depth plots are shown in Figure 5-9 to Figure 5-16 for the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP and the PMF events respectively. The results grids have been supplied electronically as part of this project.

5.4.1 20% AEP Event

The 20% AEP is generally contained in the main channel and floodplain throughout the Seven Creeks system. The Caravan Park on Kirkland Avenue is completely inundated during this event as this is located in the main floodplain. The Seven Creeks system shows some overbank flows from the main channel downstream of the main township area near Parker Street and Factory Road. This area experiences overbank flows is relatively frequent events. From Factory Road there are numerous overland flow paths that are activated as the floodwaters spread out to cover a large area. The railway embankment acts as a brier to flows even in the 20% AEP event.

Flows along Castle Creek are out of bank in the 20% AEP event. Downstream of the Hume Freeway Castle Creek has an activated anabranch through the Golf Course. Downstream of this location the floodwaters spill out of the main channel and begin to inundate floodplain to the west of the main Creek. The levee acts as a barrier to these flows to prevent the flows entering the Euroa township. The Euroa Main Road acts as a barrier to the peak flows, as does the railway embankment. The Main Road into Euroa is not overtopped.

Upstream of the railway embankment there is an opening in the Castle Creek levee which acts as a spillway. Figure 5-9 shows this is active even in the 20% AEP event. This directs excess flow towards the small railway culverts in the 20% AEP event and for larger events.

There is no interaction between Castle Creek and Seven Creeks floodplains in this event.

5.4.2 10% AEP Event

The 10% AEP event is contained within the main floodplain of Seven Creeks with some minor breakouts into the township occurring. As for the 20% AEP flood event the Caravan Park is substantially inundated. Additional breakouts into the township occur at:

- Across Kirkland Avenue adjacent to the Caravan Park
- Near Templeton Street and Turnbull Street
- Around the Memorial Oval and Dunn Street.

The main breakout from the main floodplain around Parker Street is more substantial as are the breakouts across the floodplain near Factory Road and downstream of this location. There are additional flow paths along Beaton Street and around the Euroa-Shepparton Road.

For Castle Creek the flood extent has increased to the west of the main Creek, with addition flood storage and flows over the Golf Course. Flows briefly overtop the Euroa Main Road. Flows are retarded by the railway embankment but downstream of this location the flows begin to spread widely across the floodplain.

During this event there is no interaction between the Seven Creeks and Castle Creek floodplains.

5.4.3 5% AEP Event

In the 5% AEP event many of the streets through Euroa become inundated from breakout flows. The anabranch near Kennedy Street is activated during this event and flows begin to inundate the drain known as the “Suez Canal” near Boundary Road South. Additional flow paths are activated to the east of Seven Creeks via Foy Street to Hunter Street, Pleasance Ave and Gobur Street. During this event Brock Street, Binney Street and Kirkland Avenue are inundated.

The flood extent for Castle Creek increased within areas already inundated. The levee was still protecting Euroa effectively with a large amount of freeboard. Additional water has now overtopped Euroa Main Road to a depth of approximately 10 cm. Additional water depths are observed upstream of Euroa Main Road and the railway embankment.

During this event the interaction between the Seven Creeks and Castle Creek floodplains begins with a flow path forming via Brock Street and through the railway culverts near Handbury Street. There is also a connection via the Euroa-Shepparton Road flowpath near Wood Road.

5.4.4 2% AEP Event

A large number of additional flowpaths impacting Euroa are observed in the 2% AEP event as compared to the 5% AEP event for Seven Creeks. A main flowpath that is activated during this event extends from the Boundary Road South through the township via Kennedy Street and Atkins Street. This flowpath connects back to Seven Creeks adjacent to the Caravan Park. Additional flowpaths are activated between Creek Drive and the old Anabranch in this area. The breakouts that were observed in more frequent events begin to inundate a large number of properties as they flow overland. Much of the Euroa township is now impacted by floodwaters.

Castle Creek has increase flood extents and depths but the levee in not breached and has suitable freeboard remaining. Euroa Main Road is now overtopped over a length of 300m with a maximum depth of approximately 15 cm.

Substantial cross catchment flows are observed during this event.

5.4.5 1% AEP Event

During the 1% AEP flood event additional areas of the Euroa township are inundated. Most flowpaths were active during the 2% AEP event, however in the 1% AEP event the peak flood depths have increased. The active flowpath via Boundary Road South begins to inundate a large area through the township.

The flood extent for Castle Creek has increased to the west of the main channel with peak depths behind the Euroa Main Road increasing. The levee is not breached and has approximately 300 mm freeboard remaining.

The flowpath between Seven Creeks and Castle Creek via Brock Street and Anderson Street now experiences significant overland flows.

5.4.6 0.5% AEP Event

The depths and extents across the floodplain have increased for this design event. An additional flowpath is activated through the centre of Euroa along Kennedy Street, across Weir Street and Howitt Avenue. This follows an old overland flowpath past the swimming pool and over Brock Street where it joins floodwaters on Hinton Street near the railway line.

The depths and flood extent along Castle Creek increased but the levee is not overtopped.

5.4.7 0.2% AEP Event

Generally the flood depths and extents increase throughout the floodplain. There are some additional flow paths through the main township near Kennedy Street and the Euroa Main Road.

On Castle Creek, the levee is not breached but it has no freeboard remaining, any increase in flows down Castle Creek or areas of weakness in the levee will cause the levee to fail.

5.4.8 PMF Event

The PMF event is the theoretical maximum flood that could occur across the Castle Creek and Seven Creeks catchments. The flood extent for this event is significant with the entire township of Euroa being inundated, much of it with depths over 1 metre. The Castle Creek levee is significantly overtopped during this event.

Overall this event shows the likely maximum flood extent that would be experienced within Euroa during any flood.

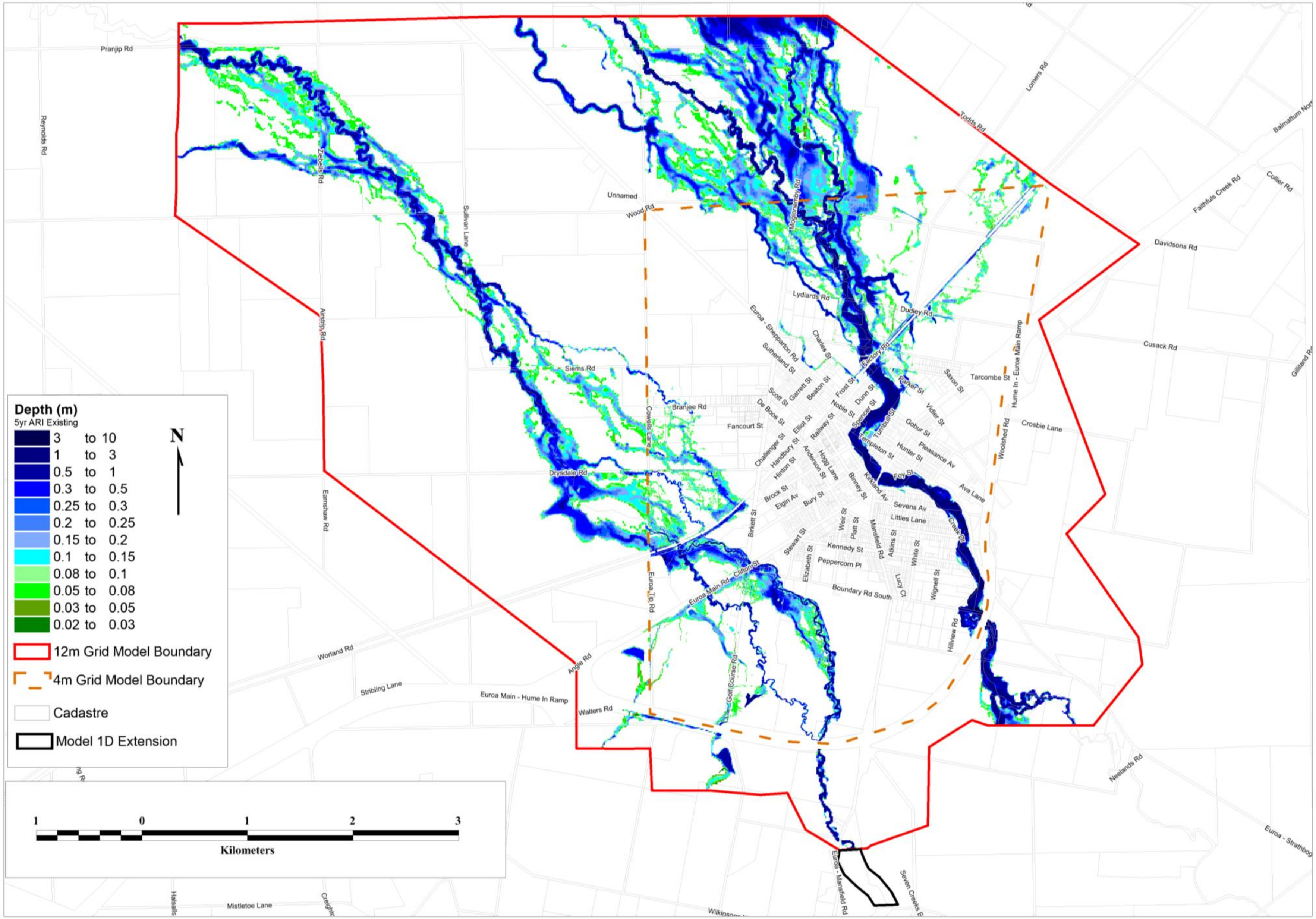


Figure 5-9 Design event peak depths – 20% AEP

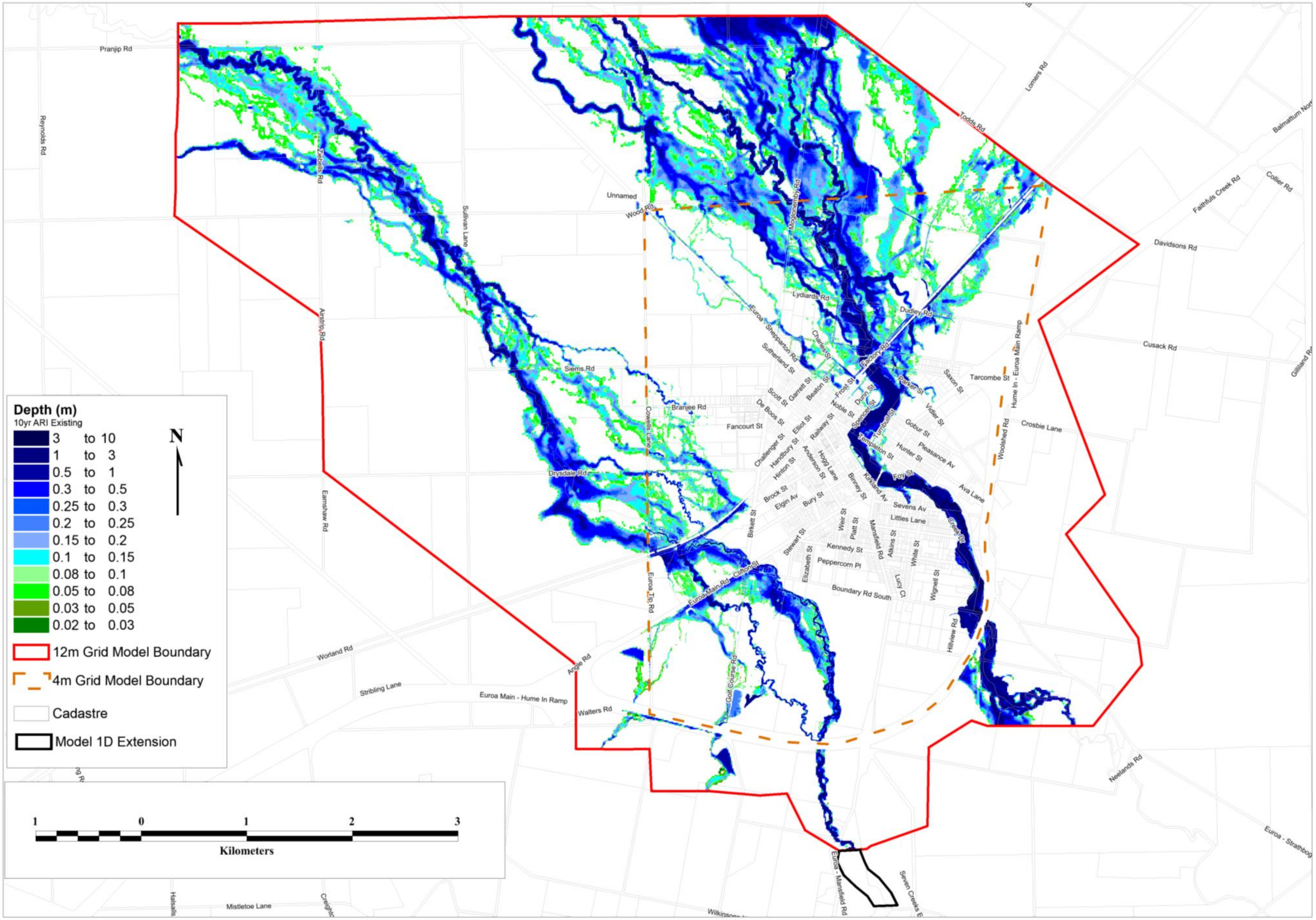


Figure 5-10 Design event peak depths – 10% AEP

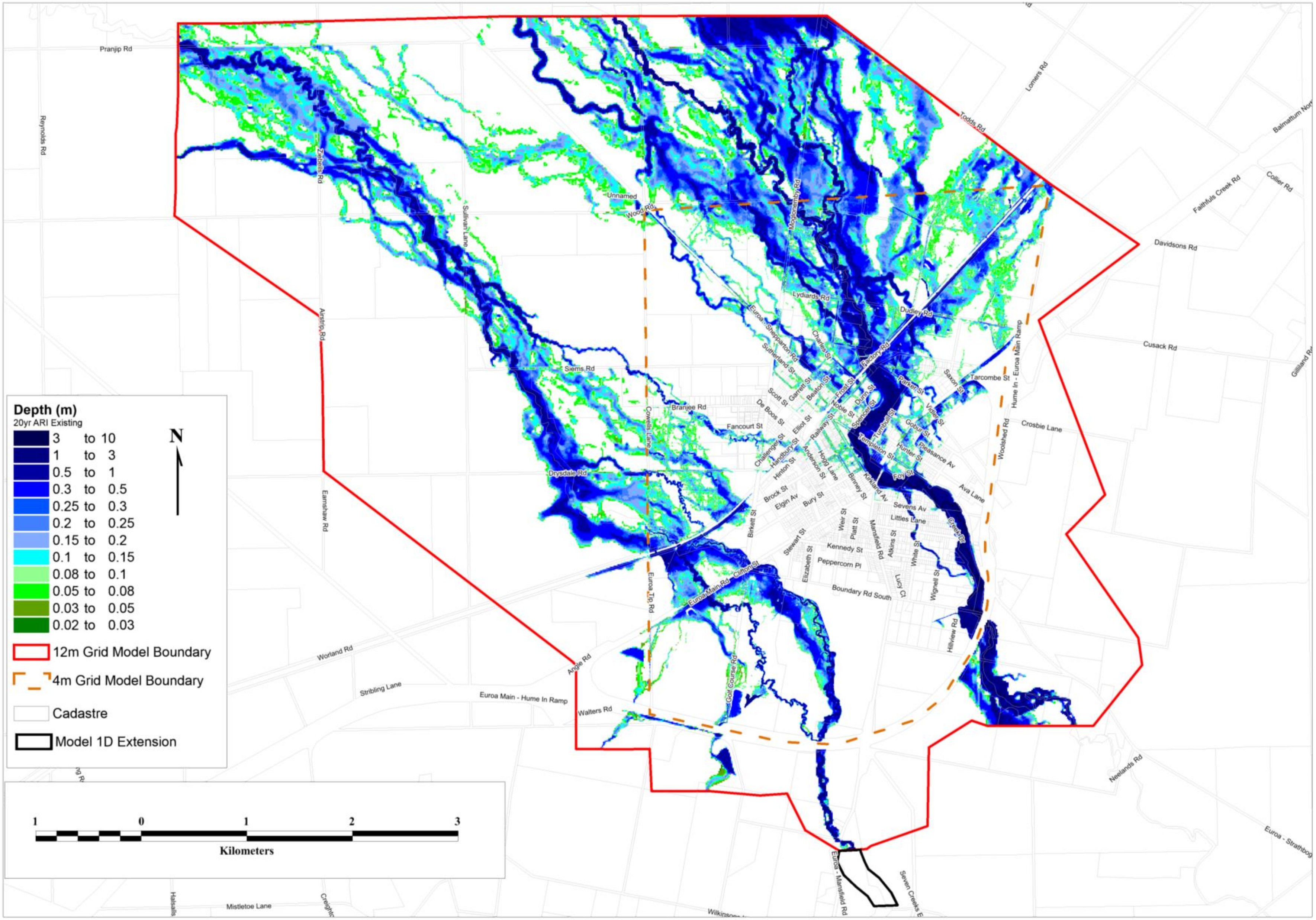


Figure 5-11 Design event peak depths – 5% AEP

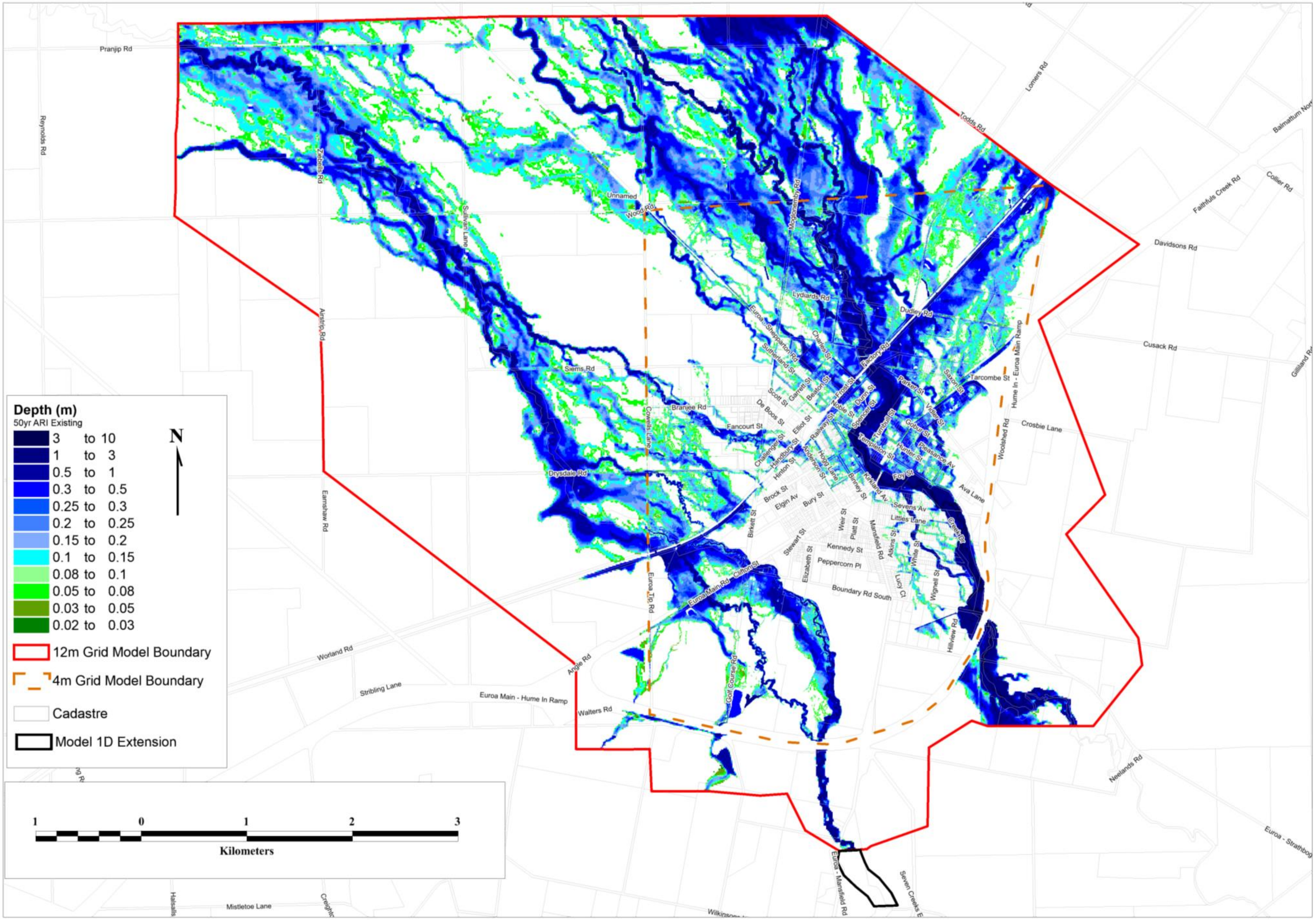


Figure 5-12 Design event peak depths – 2% AEP

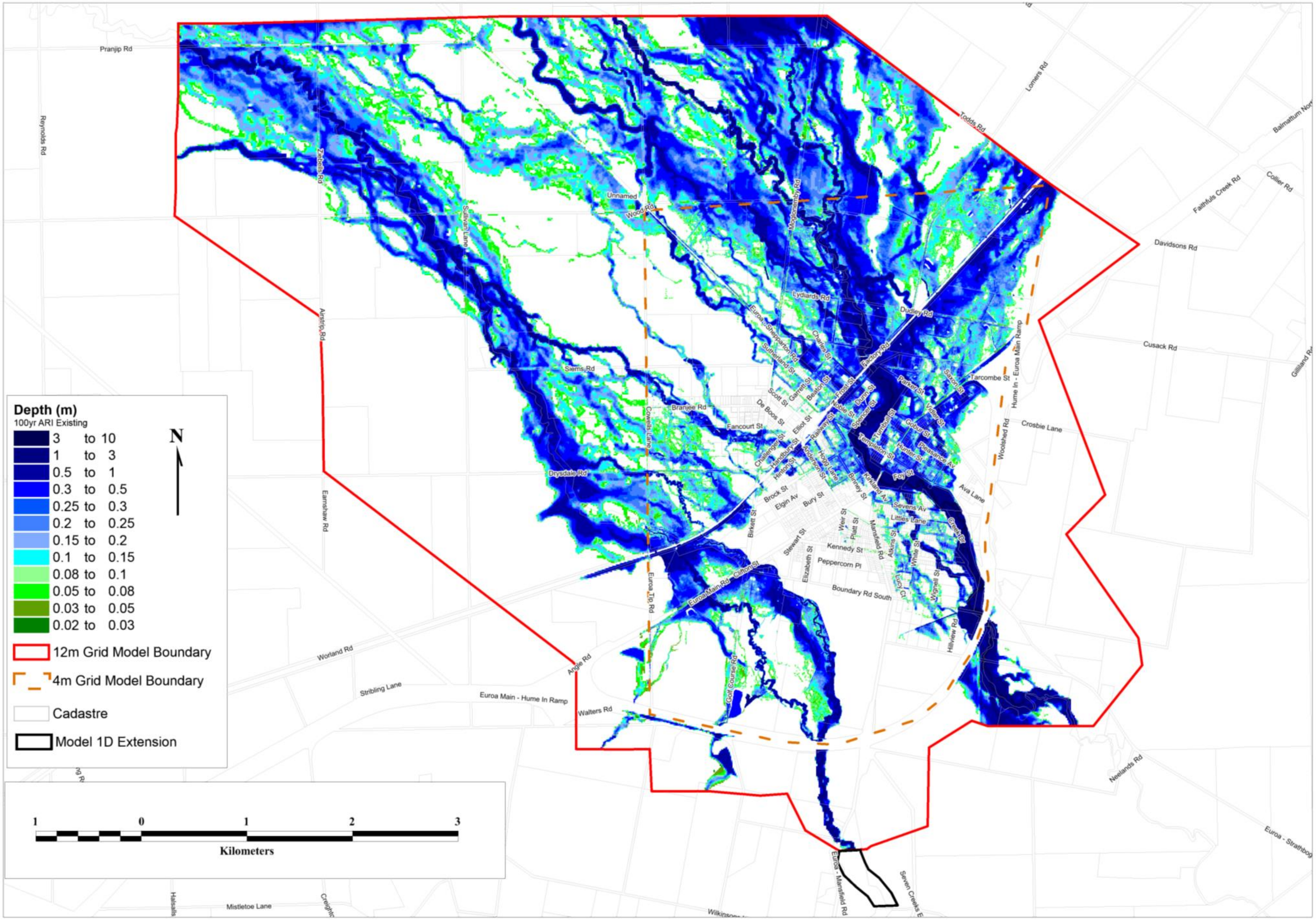


Figure 5-13 Design event peak depths – 1% AEP

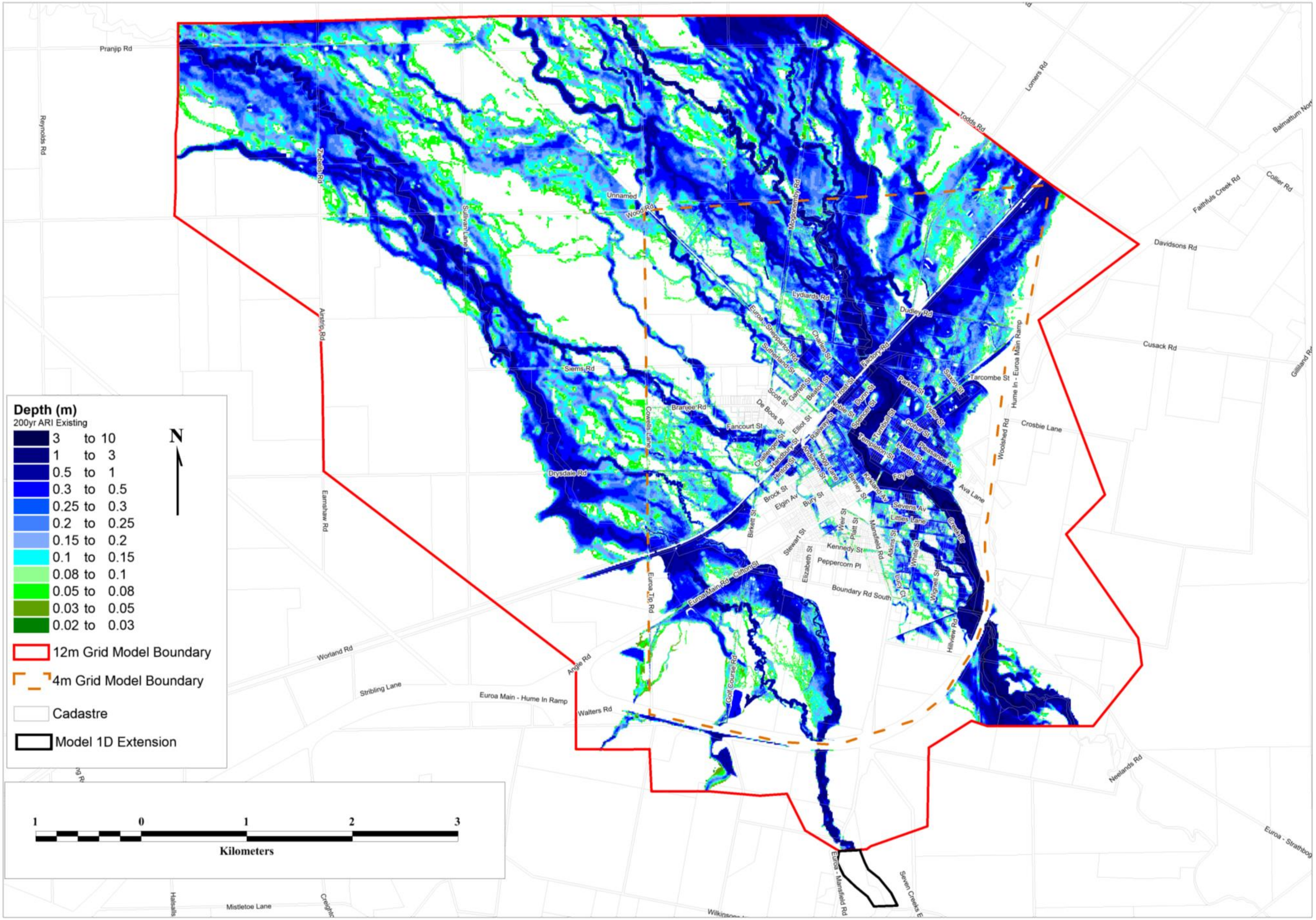


Figure 5-14 Design event peak depths – 0.5% AEP

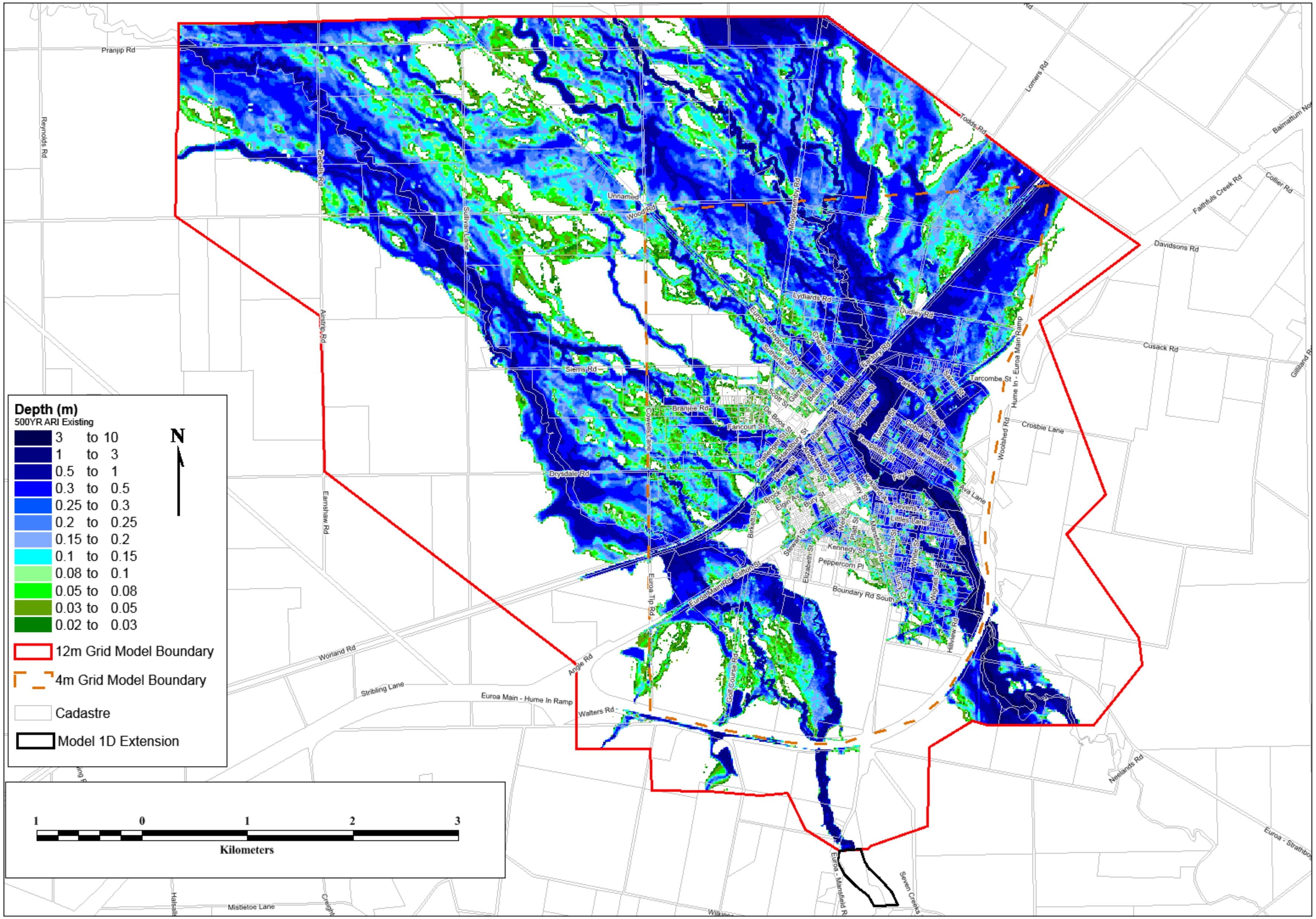


Figure 5-15 Design event peak depths – 0.2% AEP

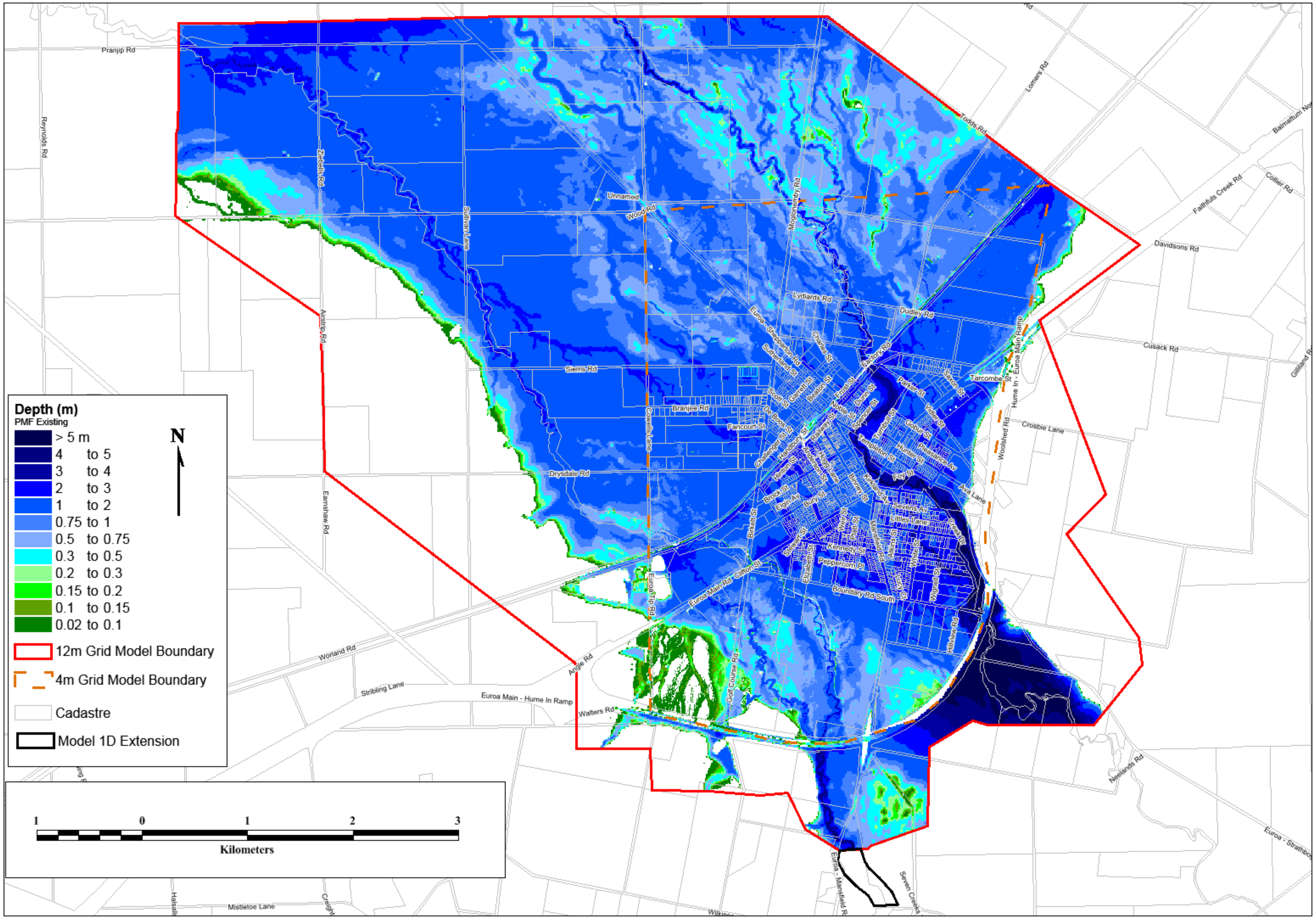


Figure 5-16 Design event peak depths – Probable Maximum Flood

5.5 Modelling of Overland Flow Paths within Euroa

As local drainage was not assessed directly in this assessment as the focus was on riverine flooding, an initial investigation was undertaken to identify local drainage paths. The method for assessing the local drainage was to run the hydraulic model with the rainfall directly applied to the study area and excluding the flows in Seven and Castle Creeks. This shows the flow paths within the study area independently of riverine flooding. It is noted that this is required to be a first pass investigation, identifying areas that may require additional information.

The 5%, 2% and 1% AEP event were assessed for six durations, from the 15 minute event up to the 2 hour event for the inner (4m x 4m grid) model area only. These are presented in Figure 5-17, Figure 5-18 and Figure 5-19 respectively. The local pipe network was included as shown in these figures.

The flood depths have been filtered to a depth of 10 cm as the purpose of this investigation is to identify the key urban runoff routes. Each of the figures gives an indication of the runoff paths present in Euroa.

For Euroa the main flow path highlighted is along the old anabranch extending from Castle Creek (now the levee blocks this flow) through the centre of the township. The flow path crosses Boundary Road South, Kennedy Street, Howitt Ave, Bury Street and then banks up behind the railway embankment along Hinton Street. This flow path is not as active in riverine dominated flood events.

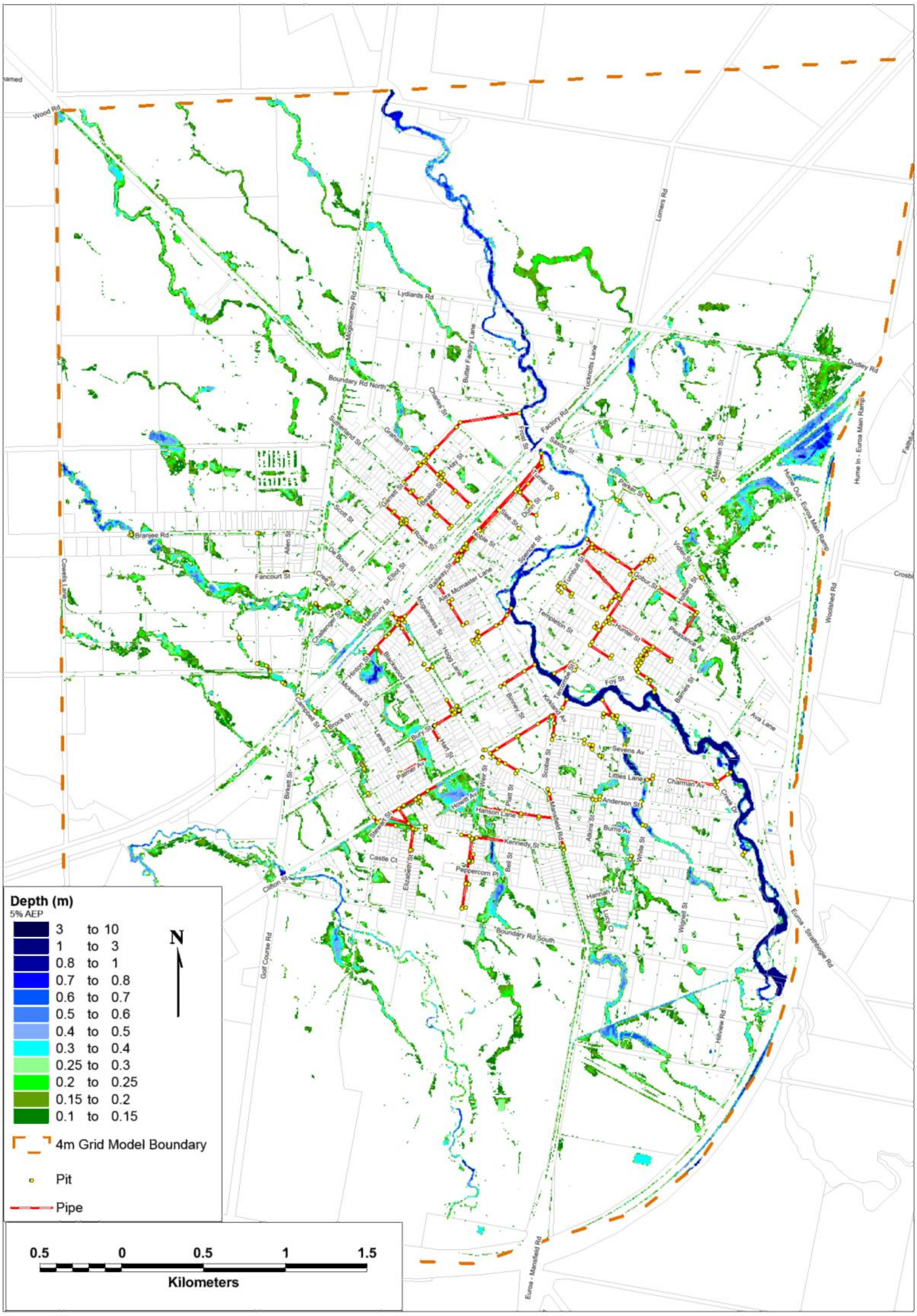


Figure 5-17 Overland flow path modelling for the 5% AEP event

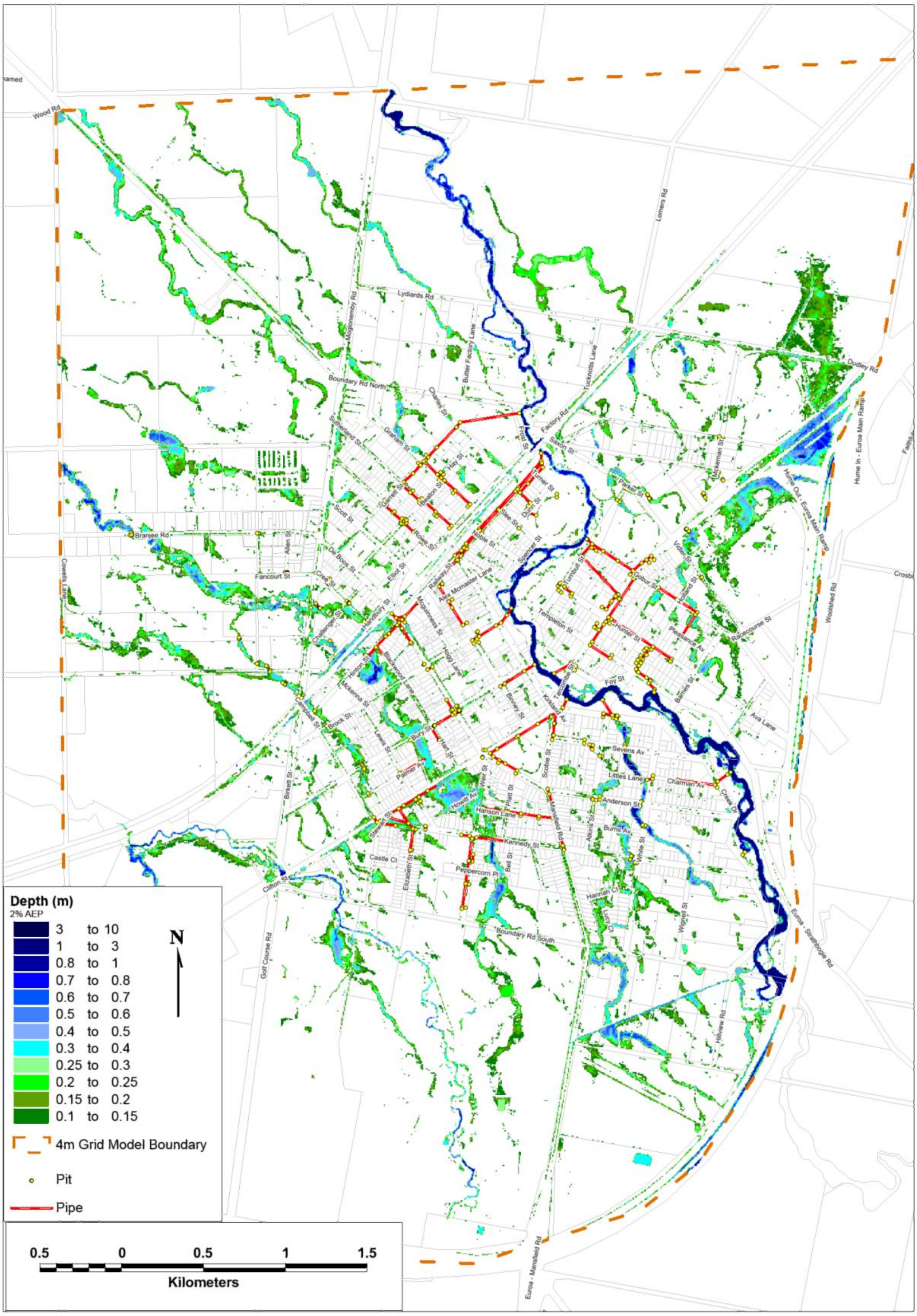


Figure 5-18 Overland flow path modelling for the 2% AEP event

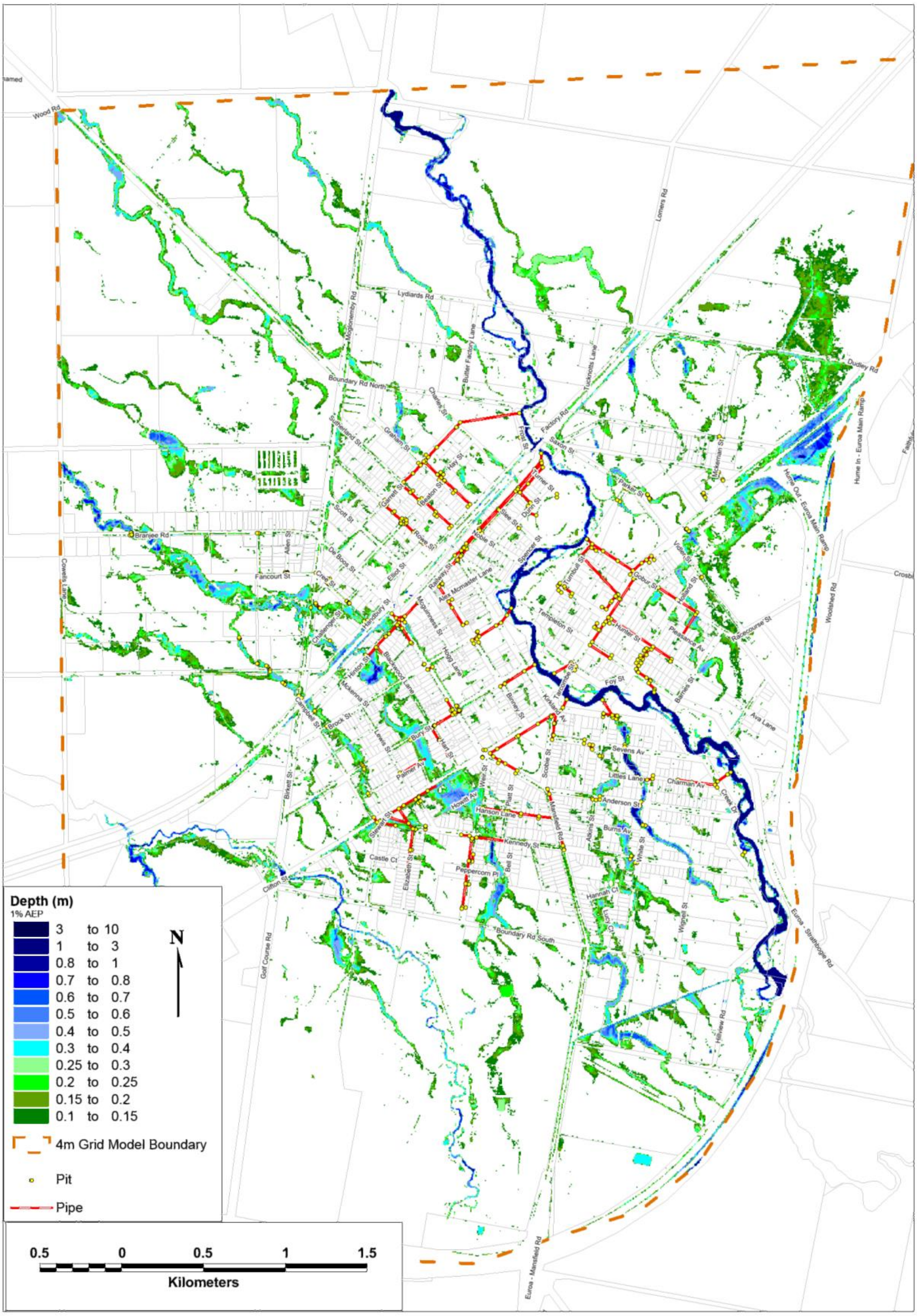


Figure 5-19 Overland flow path modelling for the 1% AEP event

5.6 Climate Change Assessment

The climate change hydrology is discussed in detail in Section 4.9. The climate change scenarios that were run through the hydraulic model included the 1% AEP event for the 10%, 20% and 30% increase in rainfall intensity. The purpose of hydraulically modelling these events was to demonstrate the potential for increased flooding as a result of increased rainfall intensity in the future.

The 10%, 20% and 30% increase in intensity rainfall runs are presented in the form of depth plots and difference plots against the existing 1% AEP event in Figure 5-20 to Figure 5-25.

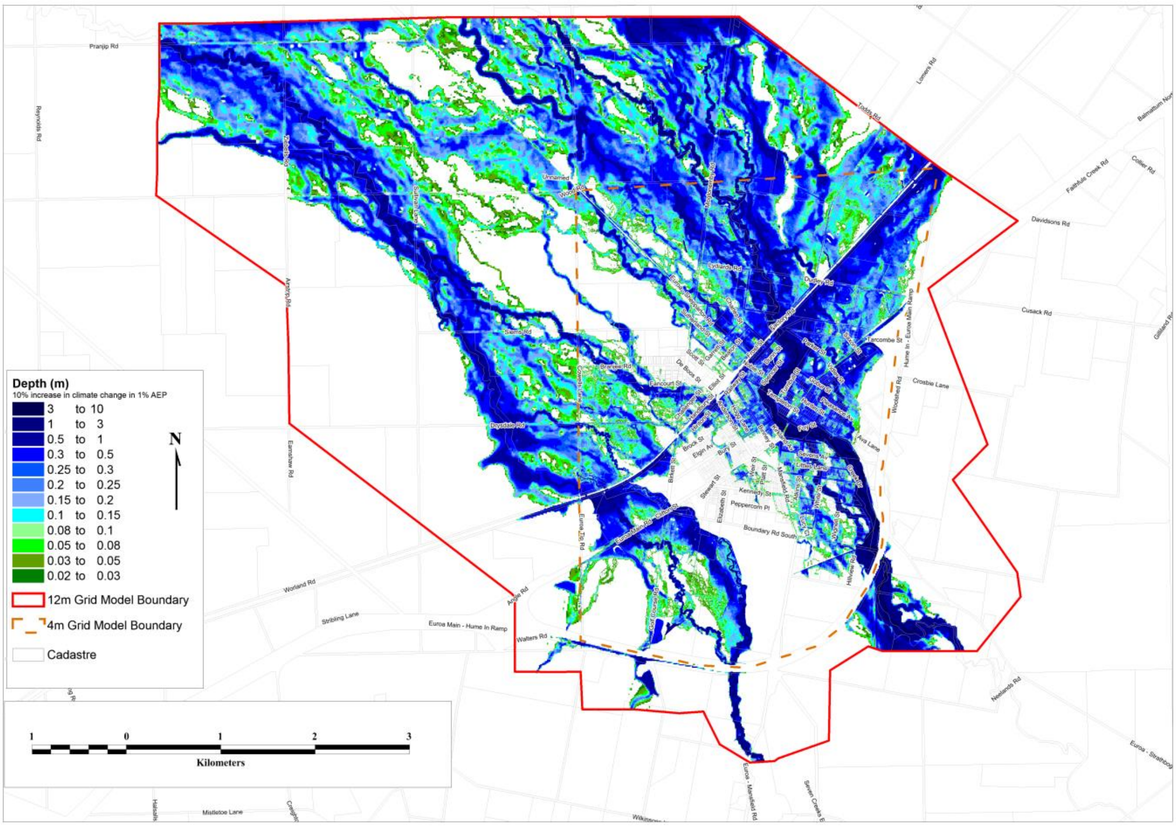


Figure 5-20 Climate change depths, 1% AEP – 10% increase in rainfall intensity

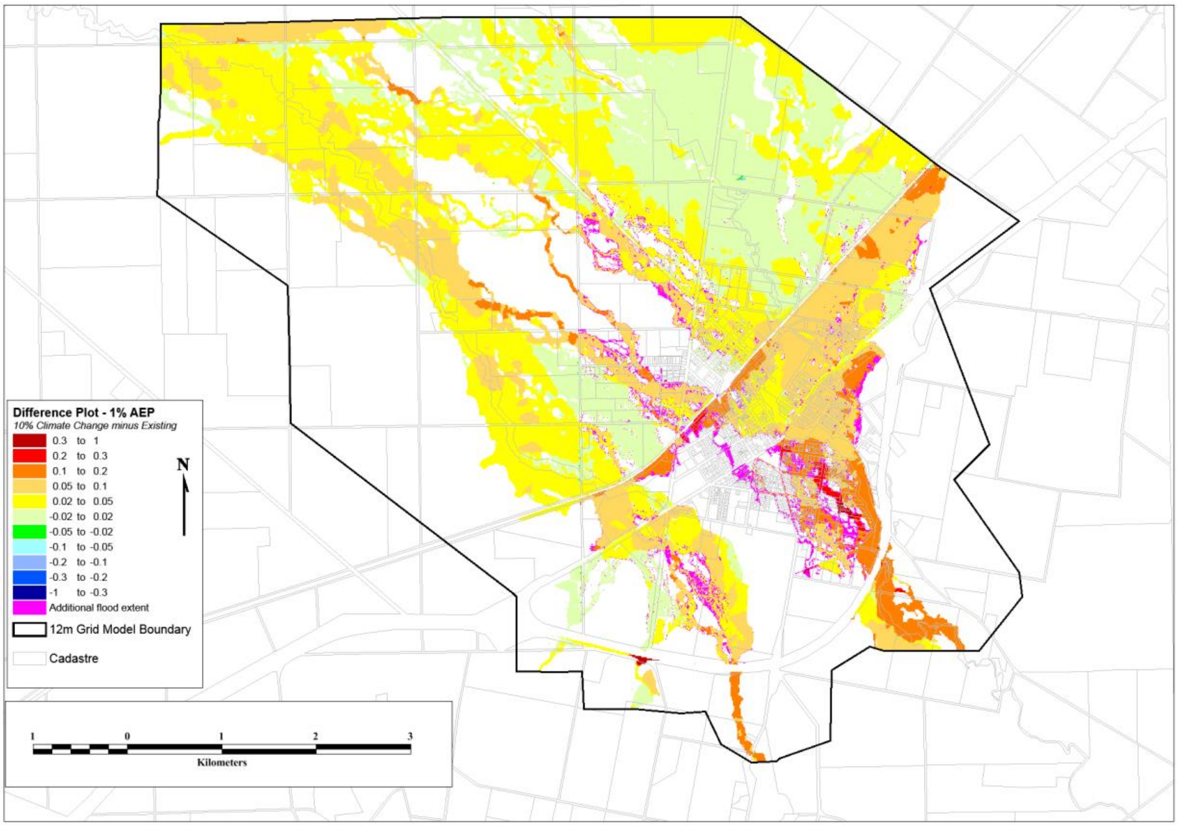


Figure 5-21 Climate change differences, 1% AEP – 10% increase in rainfall intensity less existing conditions

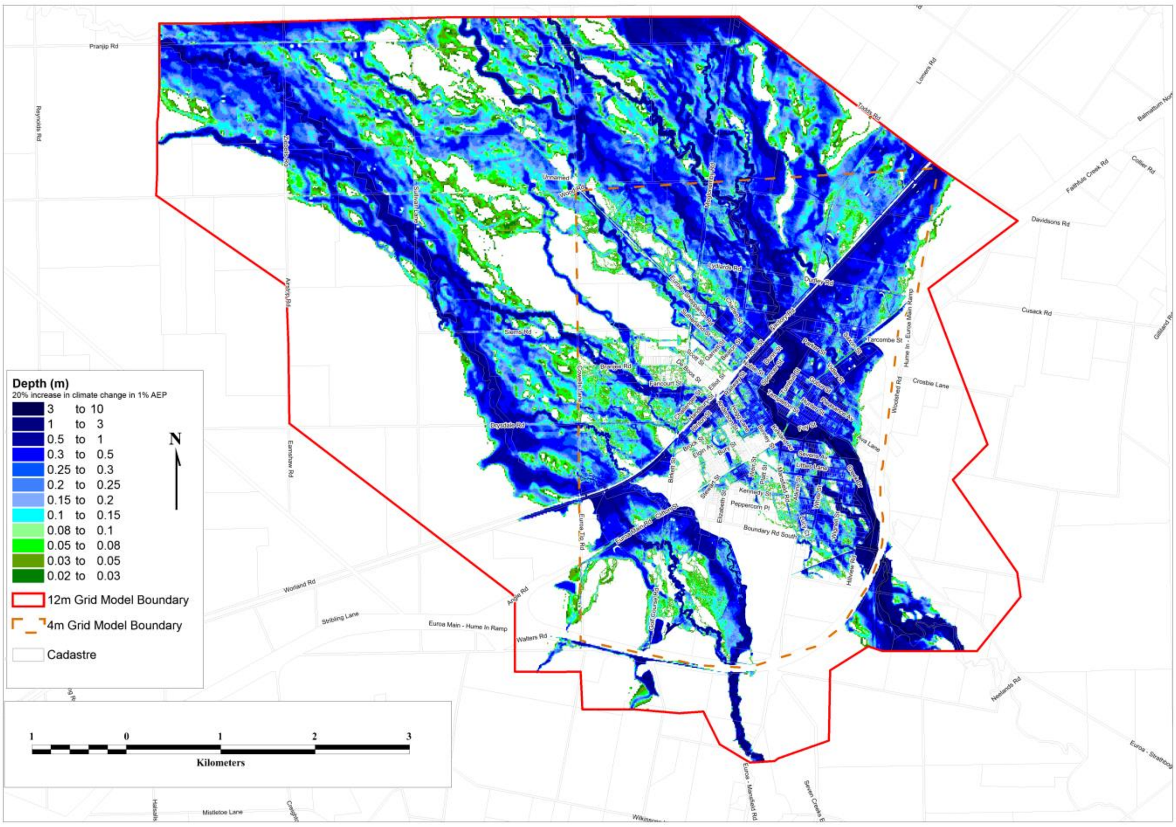


Figure 5-22 Climate change depths, 1% AEP – 20% increase in rainfall intensity

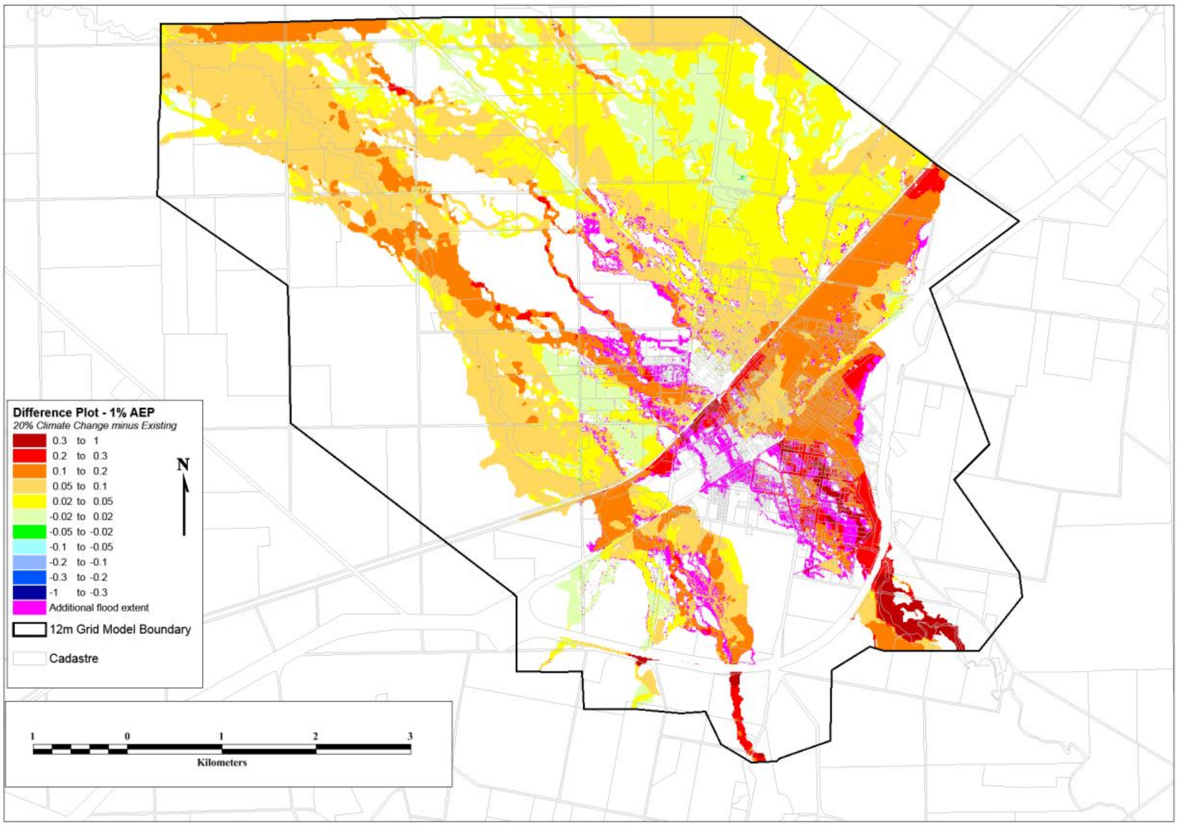


Figure 5-23 Climate change differences, 1% AEP – 20% increase in rainfall intensity less existing conditions

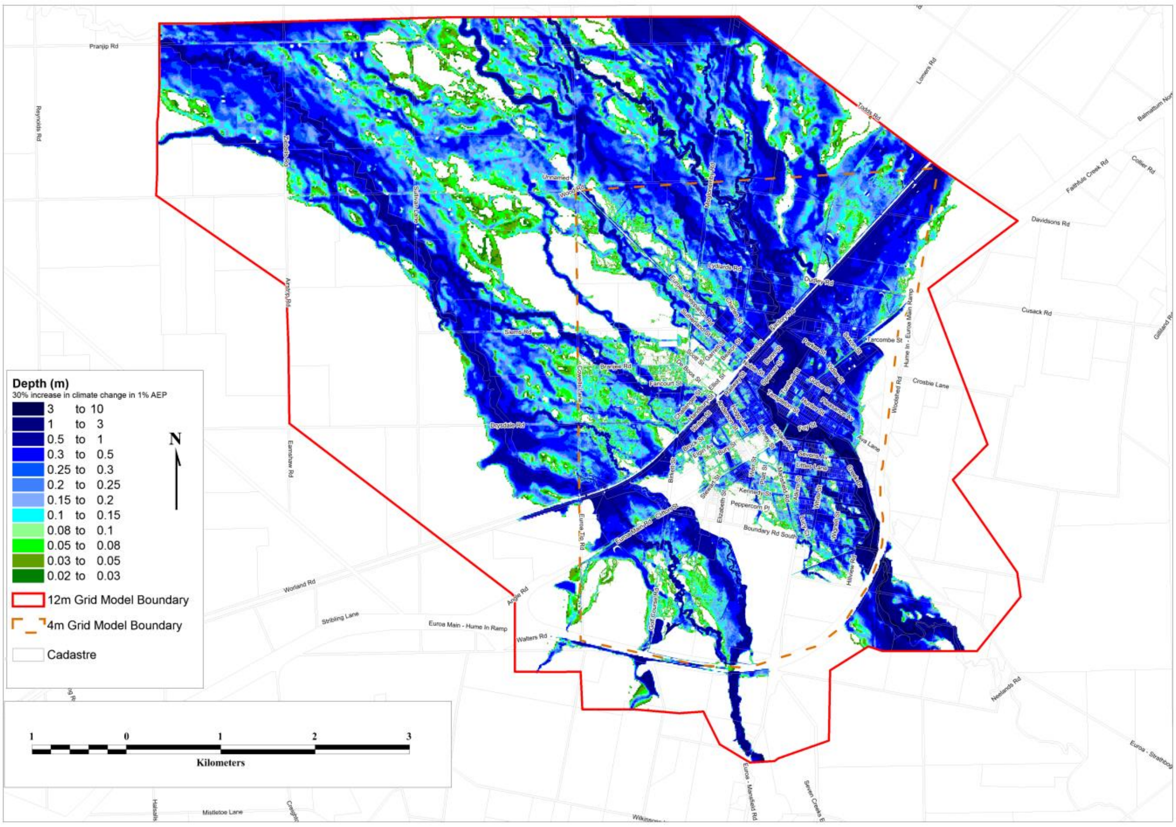


Figure 5-24 Climate change depths, 1% AEP – 30% increase in rainfall intensity

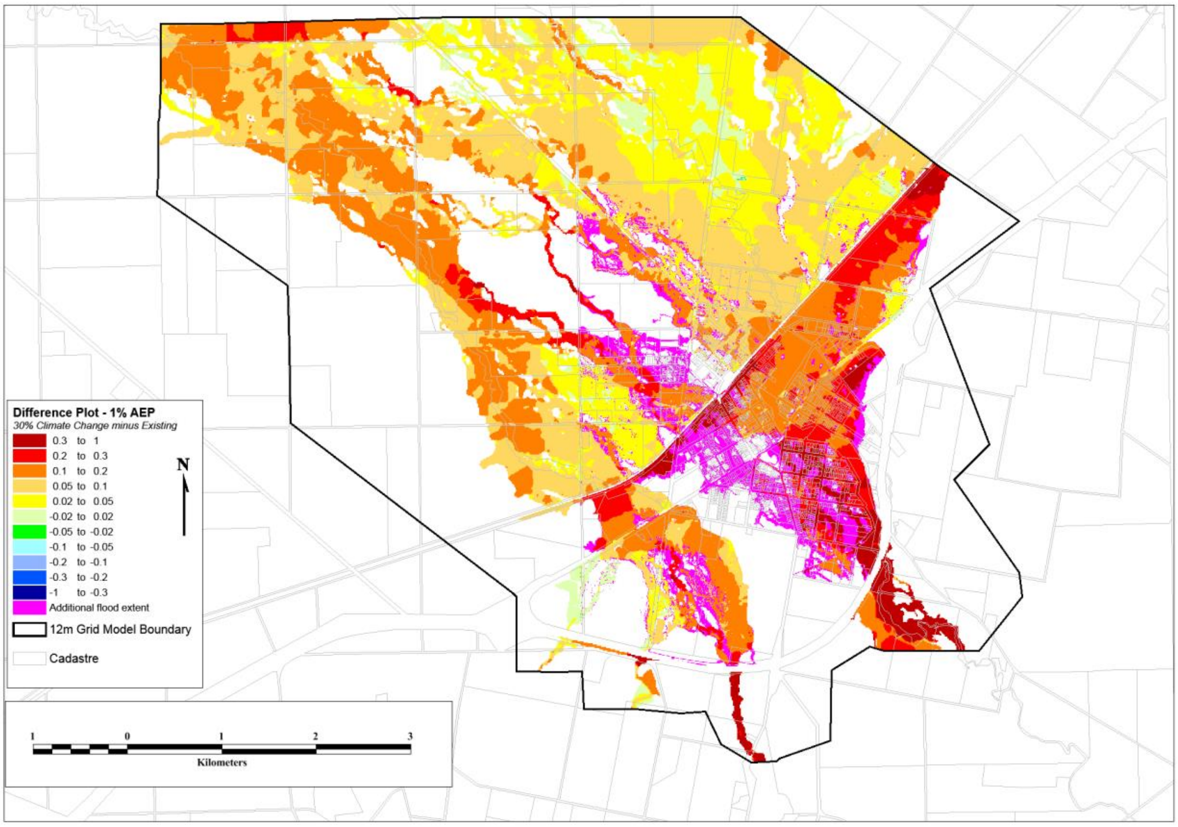


Figure 5-25 Climate change differences, 1% AEP – 30% increase in rainfall intensity less existing conditions

5.7 Sensitivity Model Runs

During the development of the hydraulic model a number of sensitivity model runs were undertaken. These sensitivity runs focused on the main components of the models that impact on the flood depths and flood behaviour, the roughness and the inflow hydrographs. Sensitivity on the peak flow rate was undertaken using the 1% AEP design event (peak of 398 m^3/s) and this following peak flow rates:

- Reduction in peak and hydrograph to 350 m^3/s (15% decrease)
- Increase in peak and hydrograph to 468 m^3/s (18% increase)

The low flow scenario difference plot has been presented in Figure 5-26 and the high flow scenario id presented in Figure 5-27.

The low flow had a 15% lower flow rate and resulted in peak depths reducing across the floodplain. On average the peak depth reductions were in the range of 0 to 0.3 m. The change across the larger floodplain were between 2 and 5 cm. The flood extent was reduced on the fringes of the flood extent. Through the Euroa township there was a reduction in the flooding (see the magenta area in Figure 5-26).

The increased flow scenario had an 18% higher flow rate than the current 1% AEP event. This resulted in increased in flood extent and depths across the catchment. Depths ranged from no change to + 30 cm across the catchment. There were some areas where the depth changes exceeded 30 cm.

Generally across the catchment a +18% and -15% of the flow rates resulted in a +/- 30 cm change to the peak flood depths across the catchment. The flood extents did change but these changes were relatively minor. For the increased peak flow rate scenario there was an additional flow path activated through the township of Euroa.

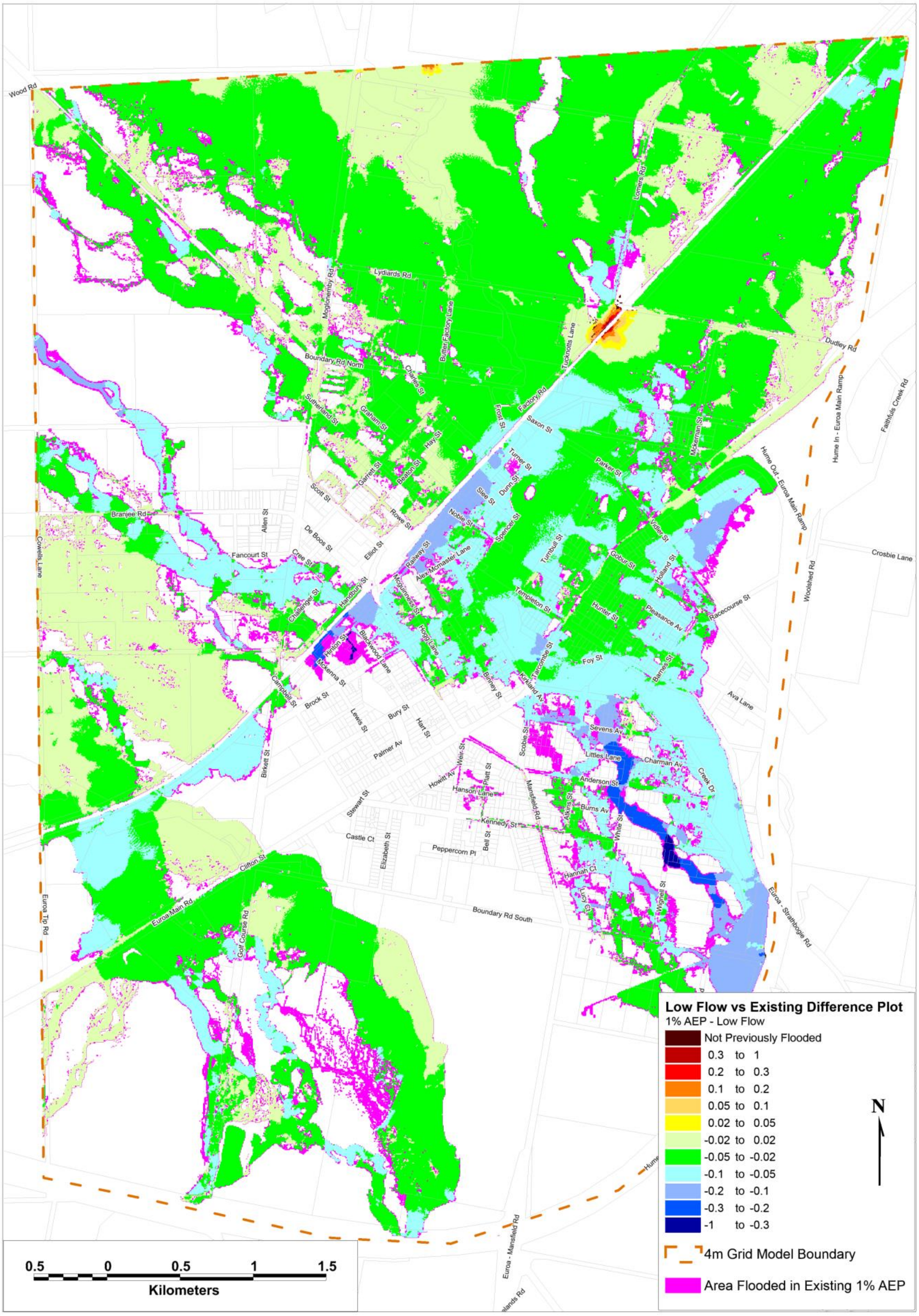


Figure 5-26 Low flow sensitivity run

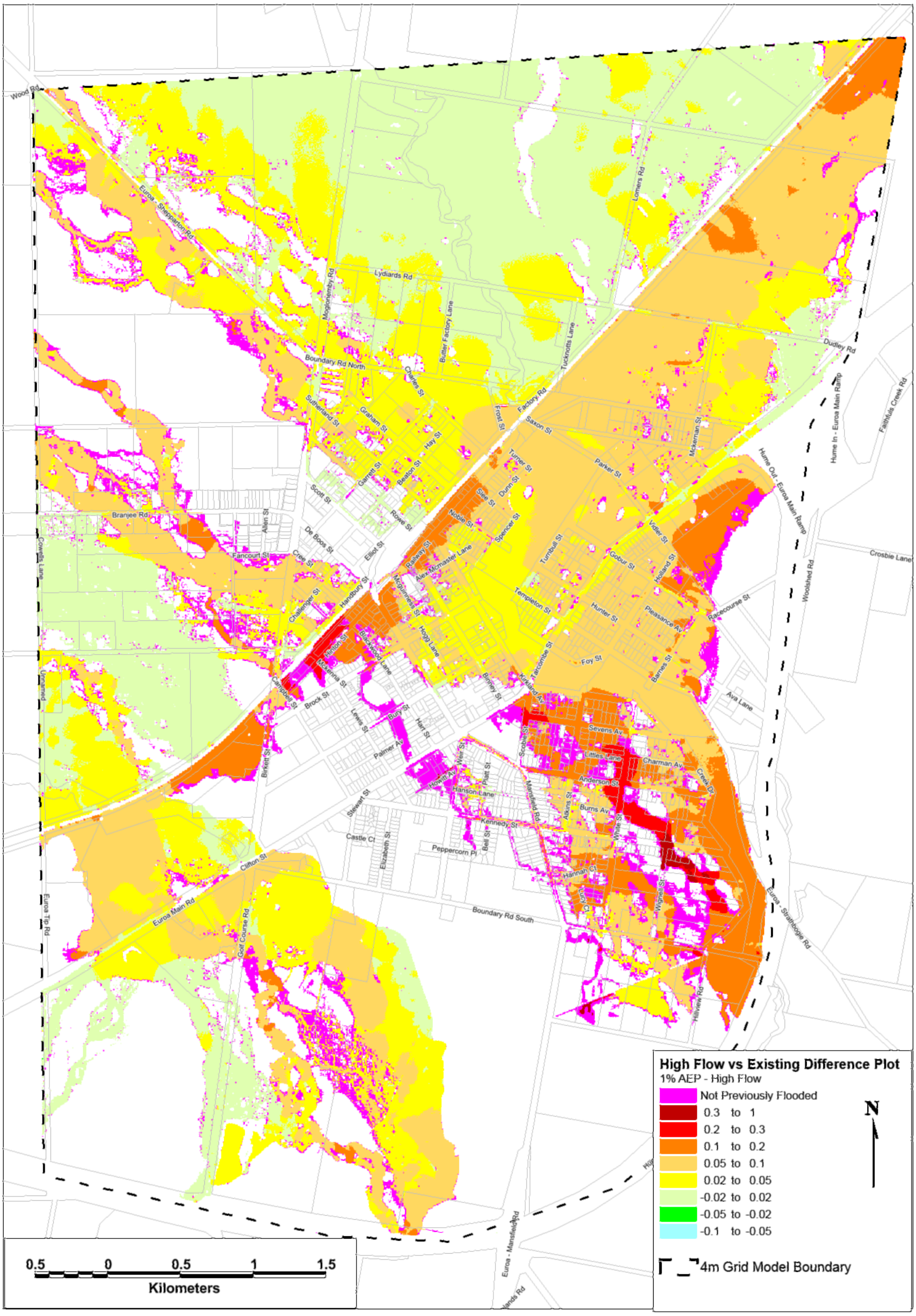


Figure 5-27 High flow sensitivity run

5.8 Telfords Bridge Rating Table

During the investigation additional survey was gathered for the Telfords Bridge streamflow gauge. This information was gathered to determine a rating table linking the gauge height to predicted streamflow. Currently there is no rating table at this gauge and as such all that is reported is gauge depths. The survey linked the board gauge heights to the Australian Height Datum (AHD). The hydraulic model was then used to develop a relationship between the depth at the gauge and the flow rate.

When developing rating tables it should be noted that there is some variability in the peak flow rate and depth at the gauge. To facilitate this variability the peak flow rate during the rising limb of the design events (20%, 10%, 5%, 2%, 1%, 0.2% and 0.5% AEP and the PMF) were plotted against the depth at the gauge. This provided a range of data points to help assign a typical rating curve for the gauge. These flow / depth pairs are shown in Figure 5-28 with a preliminary rating curve shown.

The derived rating curve is stated shown in tabular format in Table 5-3. The gauge zero was derived from the field survey.

Level data was extracted for the Telfords Bridge gauge to assess the peak flow rates using the derived rating table however peak levels are only available from 13th February 2013 and no significant event have occurred since that date on Castle Creek. Additional data may exist for Telfords Bridge but it was not available via the Department of Environment and Primary Industry (DEPI) water monitoring website (formerly Victorian Data Warehouse).

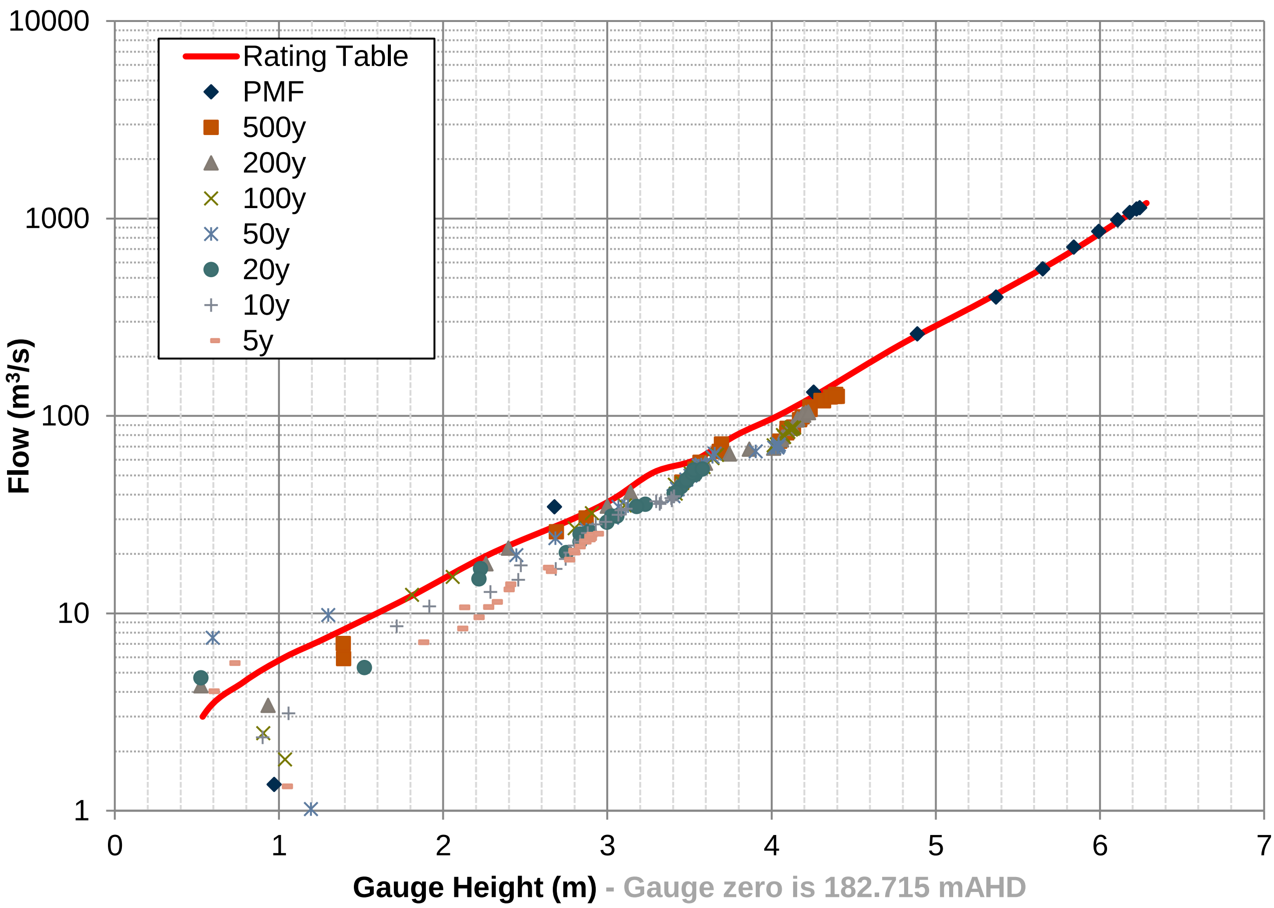


Figure 5-28 Preliminary rating table for Telfords Bridge at Castle Creek

GBCMA stated that the December 2010 event had a gauge height of 2.75 m which corresponds to a gauge height of 185.5 mAHD. From the derived rating curve this corresponds to a predicted peak flow rate of 34 m^3/s, this is below the predicted peak flow rate of 55 - 60 m^3/s. At this stage the rating curve is preliminary so the difference is not unexpected, there is also high uncertainty with the peak flow rate for this event as there is no formalised and validated rating curve.

The peak level for the September 2010 event had a gauge height of 2.58 m which corresponds to a gauge height of 185.3 mAHD. This corresponds to a peak flow rate of 30 m^3/s. This matches the predicted peak flow rate from the assessment well. This provides some confirmation to the derived rating table, but it should be noted that without a long recorded history of peak flow rates and levels recorded at this location there will always be some uncertainty in the estimates.

It is recommended that further work on the rating table be undertaken in the future. The gauge is currently located near the edge of the hydraulic model, and while additional survey was undertaken to prove the cross sections in this location, the fact remains that the gauge is located near the model boundary in an area where no LiDAR exists and there is a structure immediately downstream. Preferably additional LiDAR should be captured for the upstream catchment to make the model continuous in the 2D domain so that the floodplain is accurately captured and additional assessment on the hydraulic behaviour of the structure is assessed in detail. Cardno notes that immediately downstream of the gauge there is a fence spanning Castle Creek, this is likely to trap debris and as such may impact peak flood levels.

The gauge zero from the gauge boards and survey is 182.715 m^3/s, however as Castle Creek was flowing during the study and survey capture it was not clear at what level the cease to flow occurs. Once Castle Creek ceases to flow it would be beneficial to assess the gauge site to determine the relationship with the gauge zero and the cease to flow gauge height. For Seven Creeks at Euroa the cease to flow height is 0.63 m. This will impact the low flow predictions and associated gauge heights for the derived gauge relationship.

Table 5-3 Derived rating table for Telfords Bridge

	Derived Rating Table			Derived Rating Table – Cont'd		
	Height	Height	Q	Height	Height	Q
	mAHD	m	m^3/s	mAHD	m	m^3/s
Gauge Zero ->	**182.715**	**0**	**0**			
	183	0.285	1	186	3.285	54
	183.25	0.535	3	186.25	3.535	68
	183.5	0.785	4.5	186.5	3.785	86
	183.75	1.035	8	186.75	4.035	105
	184	1.285	10	187	4.285	130
	184.5	1.785	15	187.5	4.785	230
	185	2.285	22	188	5.285	380
	185.5	2.785	35	188.5	5.785	650
	185.75	3.035	44	189	6.285	1200

5.9 Seven Creeks at Euroa Rating Table Assessment

The relationship between the modelled gauged levels, modelled local levels, modelled peak flow rates recorded and the corresponding rating table flow estimate for the design events are summarised in Table 5-4. This information links the local gauge height to the expected peak flow rate and AHD level, which has then been linked back to the current rating table for the gauge. The rating table has been sourced from the Victorian State Government Water Monitoring website (http://data.water.vic.gov.au/monitoring.htm). The reported gauge zero for the site is 172.9 mAHD and the cease to flow level is 0.6 m.

Table 5-4 Design events compared to the rating curve flow rates

Event	Modelled Level (mAHD)	Modelled Level (m)	Modelled Flow (m^3/s)	Rating Table Flow (m^3/s)	Difference (Model flow to rating table flow)
Gauge Zero	172.9				
20% AEP	177.64	4.74	123	114	8%
10% AEP	178.04	5.14	184	159	15%
5% AEP	178.39	5.49	246	217	13%
2% AEP	178.73	5.83	331	294	13%
1% AEP	178.94	6.04	398	353	13%
0.5% AEP	179.09	6.19	468	403	16%
0.2% AEP	179.26	6.36	563	Above rating	
PMF	181.46	8.56	5140	Above rating	

The current rating table for Seven Creeks at Euroa (405237) is shown in Figure 5-29 with the design events compared to the curve. The design event flows sit higher than the rating curve flows for a given gauge level. This suggests that the hydraulic model is passing a higher flow at a given gauge level than is predicted by the rating table. The design event flow rates are approximately 13% higher than the rating table estimated flow rates. There have been few large events that can validate the rating curve at high flow rates and due to the flow estimation difficulties associated with high flow events, this levels of difference is within a reasonable error range.

It is noted that since the SKM study in 1997 the rating curve at Seven Creeks at Euroa Township has been adjusted. This periodic adjustment is expected for gauges to ensure the relationships are maintained. In this instance comparison of the peak flow rates under the revised rating curve are observed to be lower than the same events under the previous rating curve (see section 4.5 for the comparison). It appears that the current Cardno derived design event peak flow rates and predicted levels at the gauge more closely match the older rating table.

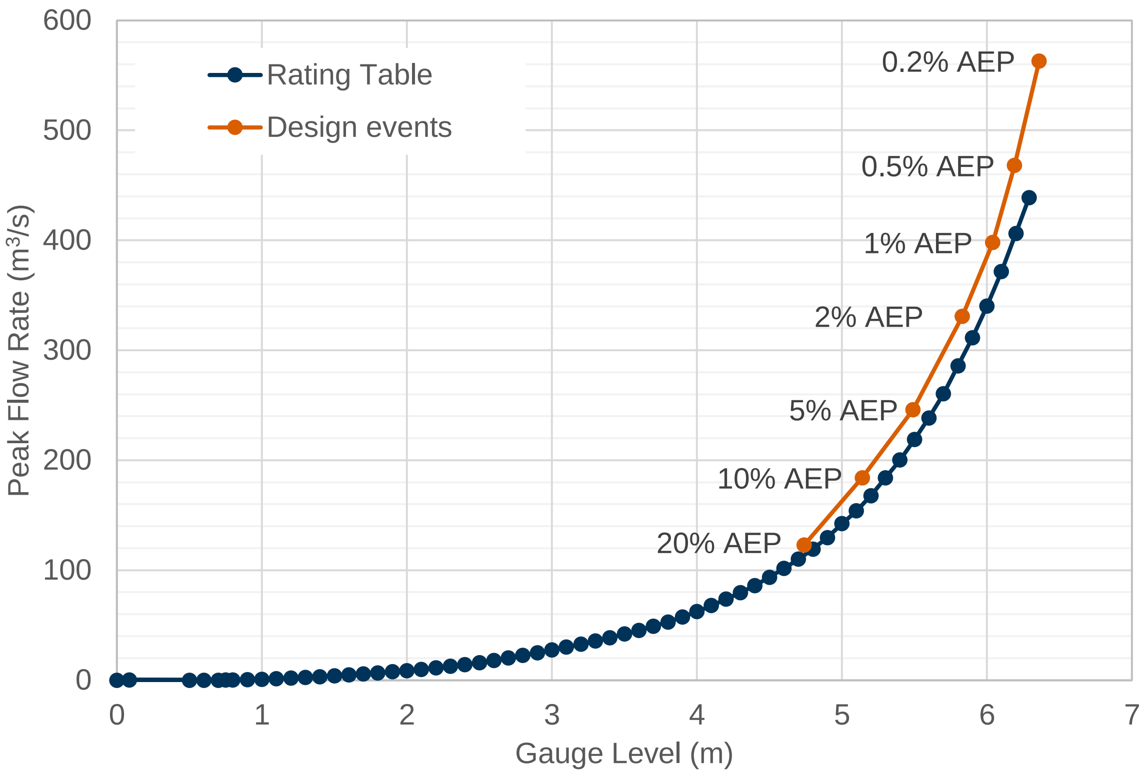


Figure 5-29 Existing rating curve for Seven Creeks at Euroa with design events

6 Datasets and Mapping

The calibrated TUFLOW model for Euroa was used to analyse the extent, location and depths for the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP events and the Probable Maximum Flood. Key outputs from the project have been developed as a result of the detailed hydraulic modelling. This section outlines the datasets and mapping that are to be supplied as part of this process. Key outputs include:

- Peak flood depths for all design flood events (as shown in Figure 5-9 to Figure 5-16).
- Flood extents for all design events.
- Flood planning controls (floodway overlays for the LSIO and FO).
- Velocity and hazard maps for the design events.
- Flood extents with peak water surface elevations at 200mm contours.
- Series of maps showing the peak depths and extents corresponding to gauge levels for both the Seven Creeks at Euroa at 200mm intervals (and one 100mm interval) between 4.6m and 6.5m on the gauge.
- Properties impacted during each flood event have been shown on each flood map, this includes properties with overfloor flooding and with water impacting the house below floor level.
- Historic calibration events showing depths and extents (the calibration events, 1993 and 2010).
- Municipal Flood Emergency Plan (MFEP) maps for inclusion in the MFEP appendices.
- Minor, moderate and major flood levels have been mapped for Seven Creeks at Euroa (minor 2.5m, moderate 4.0m and major 4.6m) and Castle Creek at Telfords Bridge (minor 1.2m, moderate 1.8m and major 2.4m).
- All gridded datasets of results from the design events.
- All model files.

All datasets and mapping have been supplied along with the final report as the final deliverables to the project. The mapping has been supplied as a regional scale map (at an approximate scale of 1:20,000) for the inner urban areas of Euroa (at an approximate scale of 1:5,000).

6.1 Design flood extents

The final flood extents for the design flood events for Euroa have been derived from the hydraulic model with some adjustments applied. The adjustments to the final model grid results included:

- A filter was applied to the final flood depth for each recurrence interval and depths less than 2 cm were removed. Floodwaters below this depth are nuisance waters and are not expected to cause any damage within the floodplain.
- Wet and dry islands were removed from the floodplain using the assumptions:
    - For the 4m grid any wet or dry island over 6 cells in size was removed.
    - For the 12m grid any wet or dry island over 4 cells in size was removed.
- The gridded model output was combined and smoothed using AutoCAD to generate a more realistic flood shape for final viewing. This process removes the square edges of the grid cells from the proposed flood extent.

The modifications to the flood extent are designed to produce a clear consistent flood shape for each of the flood events. The final flood extents are shown in Figure 6-1.

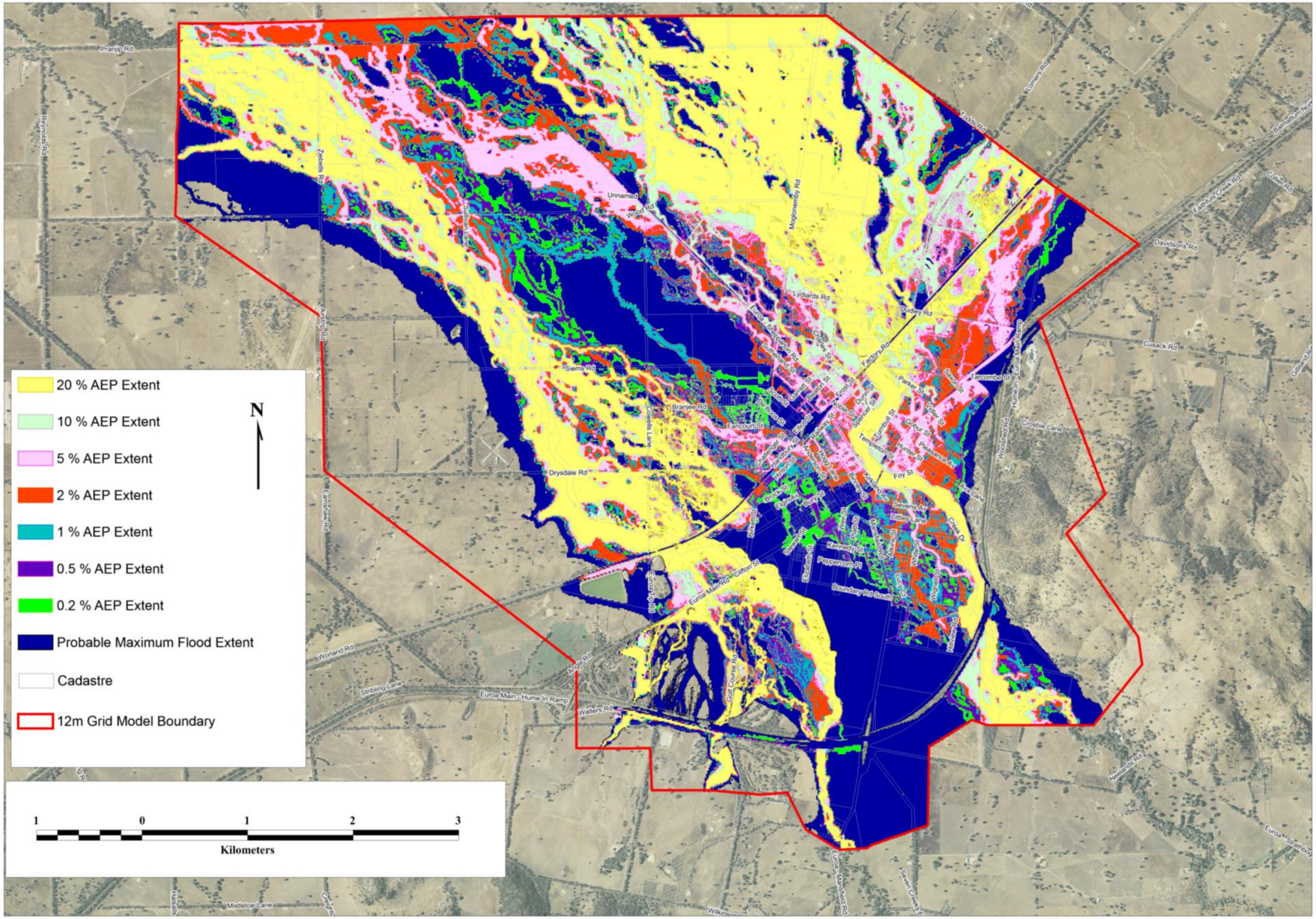


Figure 6-1 Flood extents for the 20%, 10%, 5%, 2%, 1%, 0.5%, 0.2% AEP and the PMF

6.2 Flood Related Planning Zones and Overlays

The planning scheme allows for a number of flood related overlays to identify land liable to flooding and flood characteristics. In general, the nature of the flood risk and available flood information will determine to what extent the provisions are applied in the planning scheme. The flood zone and overlay provisions allow for control of the land use and development through the use of a planning process to ensure that development is in-line with the level of flood risk.

There are four flood zones and overlays available for use:

- Urban Floodway Zone (UFZ)
- Floodway Overlay (FO)
- Land Subject to Inundation Overlay (LSIO)
- Special Building Overlay (SBO).

Each of these zones and overlays are defined more clearly in the following sections. Currently the Strathbogie Shire Council operates utilising the Urban Floodway Zone (UFZ), Floodway Overlay (FO) and Land Subject to Inundation (LSIO) controls. This study aims at providing updated UFZ, FO and LSIO layers. A description of the potential controls is provided for background information.

6.2.1 Urban Floodway Zone (UFZ)

The Urban Floodway land use zoning is intended to protect land in urban areas that has a primary function of floodwater conveyance. It applies to urban areas where the potential flood hazard is high due to the presence of existing development or to pressures from new or more intensive development. The UFZ restricts, to a very limited number, the use of land to those that are consistent with the primary function of flood conveyance.

6.2.2 Land Subject to Inundation Overlay (LSIO)

The LSIO aims to include land which is likely to be inundated by overland flow during the 1% AEP flood. The LSIO is covered under Clause 44 of the VPPF.

The purpose of the Land Subject to Inundation Overlay as described in the planning scheme is as follows:

- To implement the State Planning Policy Framework and the Local Planning Policy Framework, including the Municipal Strategic Statement and local planning policies.
- To identify land in a flood storage or flood fringe area affected by the 1% AEP flood or any other area determined by the floodplain management authority.
- To ensure that development maintains the free passage and temporary storage of floodwaters, minimises flood damage, is compatible with the flood hazard and local drainage conditions and will not cause any significant rise in flood level or flow velocity.
- To protect water quality in accordance with the provisions of relevant State Environment Protection Policies, particularly in accordance with Clauses 33 and 35 of the State Environment Protection Policy (Waters of Victoria).
- To ensure that development maintains or improves river and wetland health, waterway protection and flood plain health.

A planning permit is required to construct a building or to construct or carry out works, including fences and roadworks on land covered by the LSIO, with some exemptions for public infrastructure works. Any subdivision of land requires a planning permit and the number of lots can be increased.

As the LSIO defines flood areas which carry lower risk due to the frequency of inundation and impacts of flooding it is typically defined as the extent of less significant events. The LSIO covers areas that are not included within the FO or UFZ but are still exposed to flood risk. For the Euroa township the LSIO has historically been defined by the 1% AEP event and this was used in the current definition for the LSIO. This extent is shown in Figure 6-2.

6.2.3 Floodway Overlay (FO)

The purpose of the Floodway Overlay, as described in the planning scheme, is as follows:

- To implement the State Planning Policy Framework and the Local Planning Policy Framework, including the Municipal Strategic Statement and local planning policies.
- To identify waterways, major flood paths, drainage depressions and high hazard areas, which have the greatest risk and frequency of being affected by flooding.
- To ensure that any development maintains the free passage and temporary storage of floodwater, minimises flood damage and is compatible with flood hazard, local drainage conditions and the minimisation of soil erosion, sedimentation and silting.
- To reflect any declarations under Division 4 of Part 10 of the Water Act, 1989 if a declaration has been made.
- To protect water quality and waterways as natural resources in accordance with the provisions of relevant State Environment Protection Policies, and particularly in accordance with Clauses 33 and 35 of the State Environment Protection Policy (Waters of Victoria).

Possible methods for development of the FO are outlined in the “Advisory Notes for Delineating Floodways” (NRE, 1998). These methods include:

- Flood frequency
- Flood hazard
- Flood depth

For the flood frequency the advisory notes (Appendix A1) suggest that areas which have a high consequence of flooding, have flood depths that are moderate or high and flood frequently should generally be regarded as floodway.

There are also a range of ways that flood fringe and floodway can be defined. It needs to be tailored to the circumstances that exist for the town and location of the assessment, considerations include:

- Any existing built-up and open areas
- Flow connectivity
- Flood depths greater than 0.3 m
- Flood depths greater than 0.5 m
- Flood hazard greater than 0.35 m^2/s.

As part of this flood investigation some example approaches to defining the Floodway Overlay have been considered however the Floodway Overlay is set at the discretion of the Goulburn Broken CMA and Council.

Three Floodway Overlay methods were considered for this assessment. These have been developed for consideration by the Goulburn Broken CMA and the Council. The three examples provided include the Floodway Overlay defined by:

- The 10% AEP extent (Figure 6-2)
- Hazard approach (depth greater than 0.4m and hazard greater than 0.4 m^2/s) (Figure 6-3)
- Depths greater than 0.5m (Figure 6-4)

For this investigation three methods were developed and assessed in order to generate example Floodway Overlay layers for consideration of the Goulburn Broken CMA and Council.

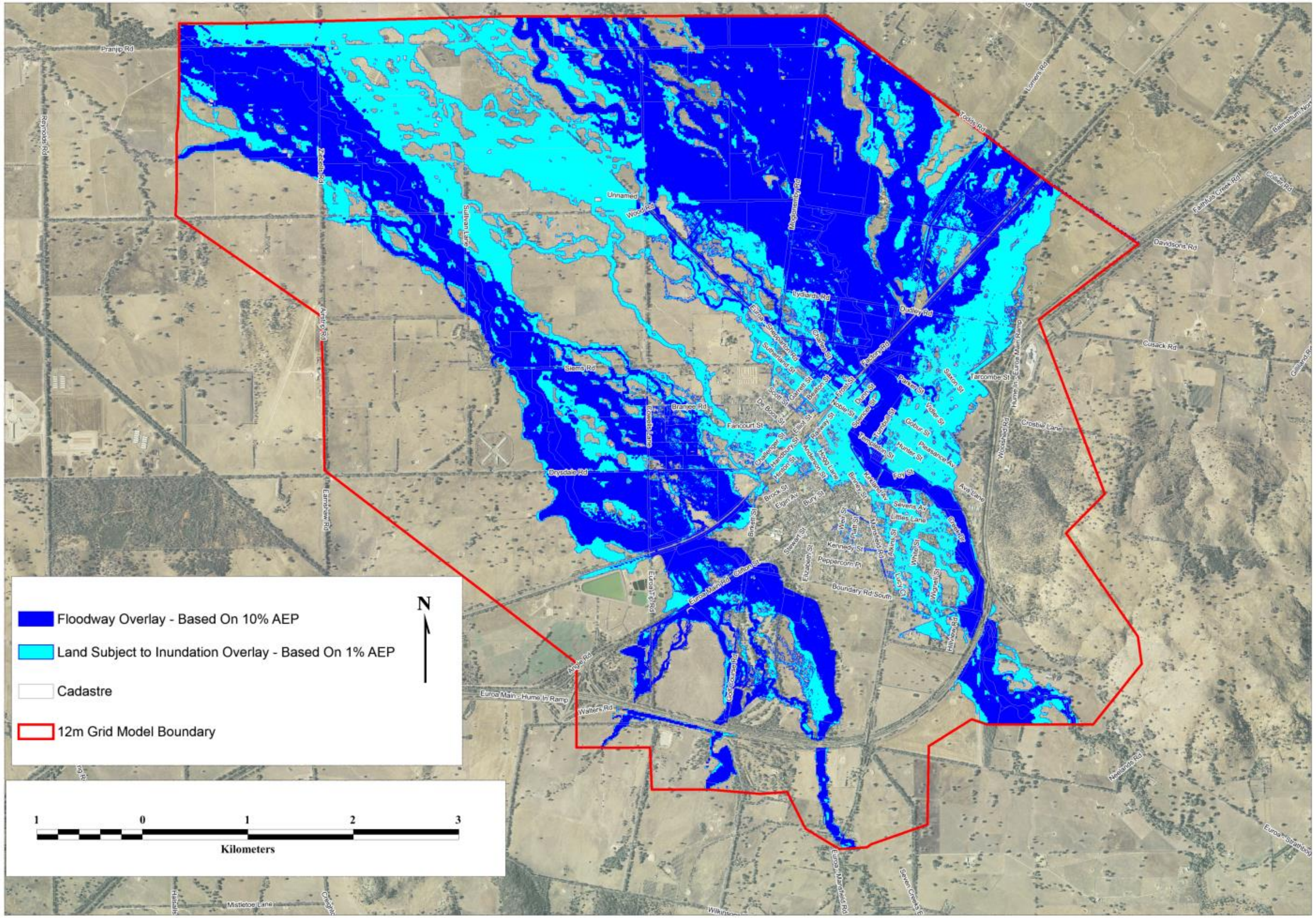


Figure 6-2 Floodway Overlay as defined by the 10% AEP flood extent

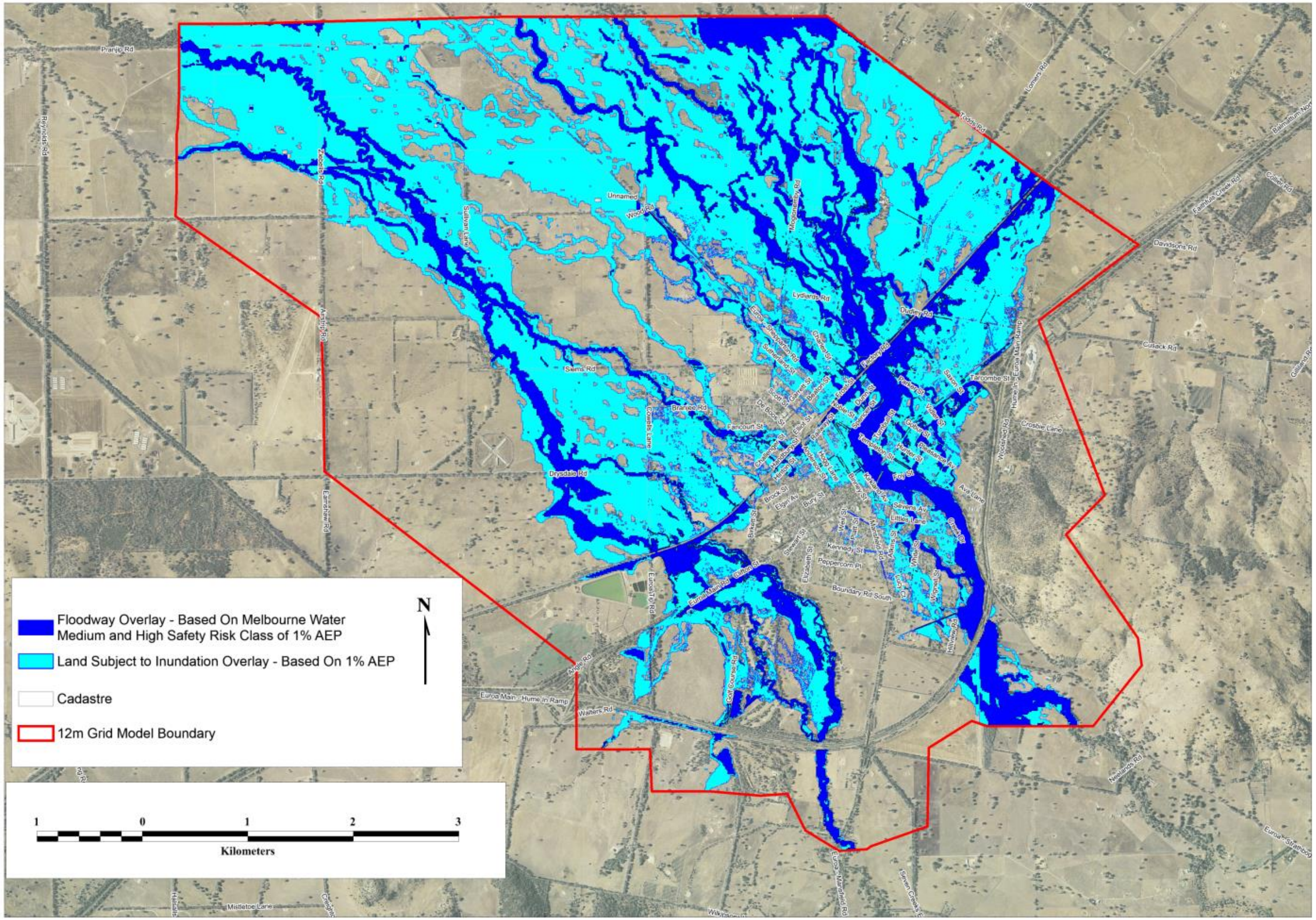


Figure 6-3 Floodway Overlay as based on the hazard criteria

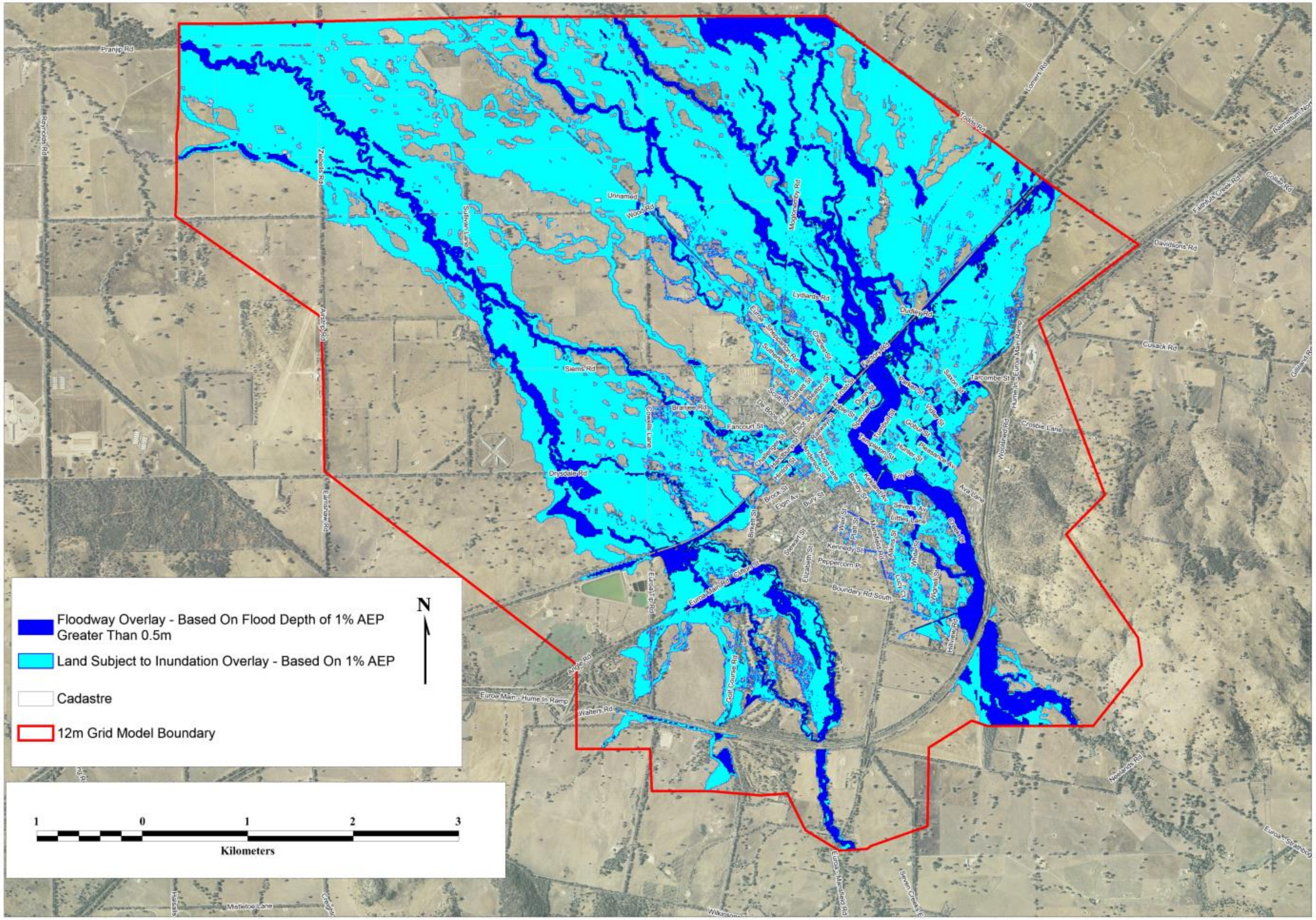


Figure 6-4 Floodway Overlay as based 1% AEP depths greater than 0.5m

7 Assessment of Risk

7.1 Flood Damage Assessment

The economic impact of flooding can be defined by what is commonly referred to as 'flood damages'. These flood damages can be defined as being direct, indirect or intangible as defined in Figure 7-1.

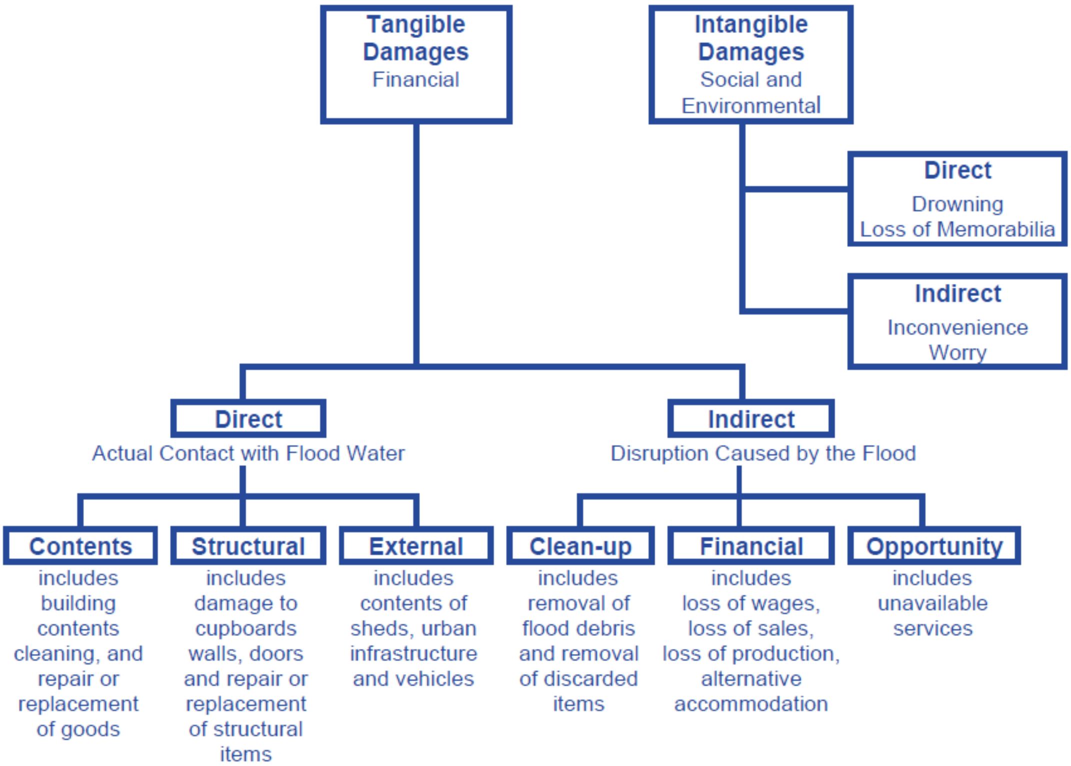


Figure 7-1 Types of flood damage (Floodplain Development Manual (NSW Gov, 2005))

The direct damage costs are just one part of the flood damage overall cost. The flood damages are broken down into two distinct groups, tangible and intangible damages. The damage assessment in this report is restricted to the tangible damages and makes no estimate of the costs associated with the 'intangible' costs, such as social distress and loss of memorabilia.

The 'tangible' damages are further divided into direct and indirect damages. The indirect damages are damages caused by the disruptions of the flooding (such as clean-up costs and accommodation costs), whereas the direct damages are caused by contact with the flood waters directly (such as damage to carpets and household contents).

For Euroa it has been assumed that the residents will have little to no warning time and hence no allowance has been made for the residents protecting or removing their valuables. This assumption has been made as it gives a more conservative estimate of flood damages as the maximum 'potential' damage is assessed. At present there is no working warning system for the residents of Euroa.

Flood damages can be assessed by a number of methods including the use of computer programs such as FLDAMAGE, ANUFLOOD or via more generic methods such using spreadsheets. For the purposes of this project, generic spreadsheets have been used based on experience by Cardno in this area. The use of both the Floodplain Management Manual (NSW Gov, 2005) and The Rapid Appraisal Method for Floodplain Management (NRE, 2000) were utilised in this flood damage assessment.

7.1.2 Damage Analysis

A flood damage assessment has been undertaken for the existing catchment and floodplain as part of the current study. The assessment is based on damage curves that relate to the depth of flooding on a property to the likely damage to a property.

Ideally, the damage curves would be calibrated to the specific catchment for which the study was undertaken, however, damage data in most catchments is not available and as a result damage curves from other catchments are utilised. The Department of Environment, Climate Change and Water NSW (DECCW) has carried out research and prepared a methodology to develop damage curves based on state-wide historical data. This methodology is only for residential properties and does not cover industrial or commercial properties.

The DECCW methodology is only a recommendation and there are currently no strict guidelines regarding the use of damage curves in Victoria. The Rapid Appraisal Method (RAM) suggests specific damage values for residential, commercial and industrial buildings, however, these values are not specific to Victoria and the flood damage curves developed by DECCW are based on a more robust methodology.

The following sections provide an overview of the methodology applied for the determination of damages within the floodplain at Euroa.

7.1.2.1 Damage Curves

Residential Damage Curves

The *Floodplain Management Guideline No. 4 Residential Flood Damage Calculation* prepared by DIPNR (now DECCW) (DIPNR, 2004) has been used in this residential damage assessment. These guidelines include a template spreadsheet program that determines damage curves for three types of residential buildings;

- Single storey, slab on ground,
- Two storey, slab on ground, and
- Single storey, high-set.

The floor level survey data collected by Cardno during this study did not specify the residential property construction, however from site visits and street view (Google) it has been identified that all residential properties in Wickliffe are slab on ground. This is the most conservative estimate of damages for the residential properties.

Damages are generally incurred on a property prior to any over floor flooding. There are two possibilities:

- The flooding overtops the representative ground level but does not necessarily reach the base of the house. When this representative ground level is exceeded by a depth of 10 cm.
- The flooding overtops the garden and also reaches the base of the house. The DECCW curves allow for a damage of $11,189 (Dec 2013 dollars) to be incurred when the water level reaches the base of the house (the base of the house is determined by the floor level less 0.3 m for slab on ground houses in this instance). This accounts for the garden damage as specified in the point above, but also includes some damage to cars and structures.

Residential damage associated with the building was only applied when the flooding reached 0.3 m below the floor level of the house using the DECCW damage curves (adjusted to current dollar values). When the flood waters overtop the floor level the DECCW damage curves are used to determine the economic damage.

The residential damage curve is shown in Figure 7-2.

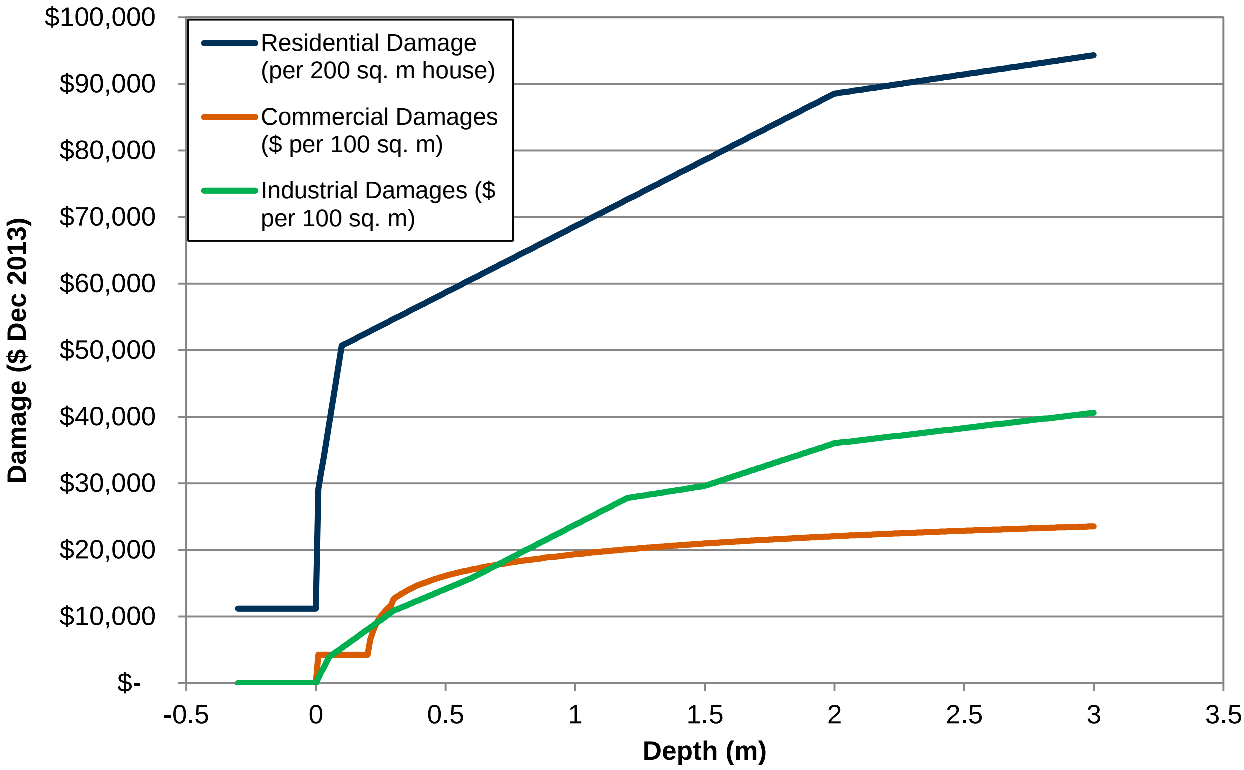


Figure 7-2 Damage curves applied to the Euroa flood investigation

The DECCW curves are derived for late 2001 and have been adjusted to represent current dollar terms. General recommendations by DECCW are to adjust values in residential damage curves by the increase in Average Weekly Earnings (AWE), rather than by the inflation rate as measured by the Consumer Price Index (CPI). DECCW proposes that AWE is a better representation of societal wealth, and hence an indirect measure of the building and contents value of the home. The most recent data for AWE from the Australian Bureau of Statistics (ABS) was in December 2013. Therefore all ordinates in the residential flood damage curves were updated to the Dec 2013 dollars. In additional, all damage curves include GST as per the DECCW recommendations.

While not specified, it was assumed that these curves were derived in November 2001, which therefore assumes the use of the November 2001 AWE (issued quarterly) would be appropriate. November 2001 and December 2013 AWE statistics were obtained from the ABS website (www.abs.gov.au). The AWE figures and percentage adjustment factor is summarised in Table 7-1.

Table 7-1 Residential damage curve adjustment factor

Month	Year	AWE
November	2001	$ 898.50
December	2013	$ 1,496.9
Change	66.7 %	

Consequently, all ordinates on the damage curves were increased by 66.7 %. It has been assumed that Dec 2013 values are representative of current dollars.

There are a number of input parameters required for the DECCW curves, such as the area of the floor of houses in the floodplain and level of flood awareness. The damage assessment adopted values within the recommended range specified by the DECCW guidelines. The average house size for Euroa was estimated at 200 m^2. This area reflects the ground floor only.

Conservatively, the Effective Warning Time has been assumed to be zero. A long Effective Warning Time allows residents to prepare for flooding by moving valuable household contents (e.g. the placement of valuables on top of tables and benches).

The Euroa Community has access to Shepparton, Wangaratta, Seymour and Melbourne via the Hume Freeway access routes and as a result it is assumed that there are no post-flood inflation costs. These inflation costs are generally experienced in regional areas where re-construction resources are limited and large floods can cause a strain on these resources.

Commercial Damage Curves

Within the township of Euroa there are a large number of Commercial businesses. To estimate the damages to these businesses during periods of flooding a damage curve was required linking flood depth to damage to the business.

The commercial damage curve has been developed based on the FLDamage User Manual (1992). When developing the damage curve for a commercial business the following parameters were considered:

- Direct damage
    - Internal building damage
    - External building damage
    - Structural damage
    - Stock and equipment damage
- Indirect financial costs
    - Loss of trading days
    - Clean-up costs
    - Loss of opportunity costs

For these direct and indirect damages a range of assumptions have to be made to facilitate the development of the commercial damage curve.

- For Euroa the warning time is assumed to be zero. This is the worst case scenario but as Euroa has no warning system in place (formalised) then there may be little or no warning.
- It is assumed that there is no structural damage for properties, it may be that some damage is higher if a buildings structural integrity is compromised.
- It is assumed that all commercial properties are privately owned and there are no lost opportunity costs.
- Loss of trading is estimated at approximately 5 days for Euroa as the flood peak generally passes within a day and this allows for clean-up time.

The derived commercial damage curve is shown as a damage cost per 100 m^2 of floor area and hence each business was delineated using aerial photography. The derived curve has damages beginning as floodwaters overtop the floor level. The curve is shown in Figure 7-2. The original curve was developed based on June 1990 dollars and this was adjusted using the Consumer Price Index (CPI) which is a good measure for commercial properties for the increasing impact of inflation. This is summarised in Table 7-2.

Table 7-2 Consumer Price Index adjustment for the Commercial damage curve

Month	Year	AWE
June	1990	56.20
December	2013	104.80
Change	86.5 %	

Industrial Damage Curves

The industrial damage curves were developed based on the concepts as per the FLDamage User Manual (1992) and previous project experience. Industrial damage was calculated on the same basis as the commercial damage curve. The general principles of the FLDamage User Manual were considered. The developed damages were adjusted to current dollar terms using a CPI adjustment.

The curve is shown in Figure 7-2. The original curve was developed based on June 1990 dollars and this was adjusted using the Consumer Price Index (CPI) which is a good measure for commercial properties for the increasing impact of inflation. This is summarised in Table 7-3.

Table 7-3 Consumer Price Index adjustment for the Industrial damage curve

Month	Year	AWE
June	1998	67.80
December	2013	104.80
Change	54.6 %	

7.1.2.2 Road damages

Road damage was assessed based on the Rapid Appraisal Method (RAM) which assigns a damage value for major roads, minor roads and unsealed roads. The RAM was developed in May 2000 and the damages are quoted in May 2000 dollars. To convert these to December 2013 dollars, the CPI was used to adjust for inflation. The adjustment factor is shown in Table 7-4.

Table 7-4 Roads damage adjustment factor

Month	Year	CPI
Mar	2000	69.7
Dec	2013	104.8
Change	50.4 %	

The RAM uses a single estimate cost per km for roads which are inundated and includes:

- Initial repairs to roads
- Subsequent additional maintenance to roads
- Initial repairs to bridges (based on 1/3 of road damages)
- Subsequent additional maintenance to bridges.

The RAM estimates of the costs per km of inundated road are shown in Table 7-5. These unit damages were adjusted using the CPI adjustment factor. The RAM also states that the damages to roads and bridges generally outweighs the costs associated with other infrastructure such as water, electricity, gas and sewerage services and is a good approximation for the overall damage to the regional infrastructure.

Table 7-5 Unit damages for roads and bridges (dollars per km inundated)

Road Type	Initial road repair	Subsequent accelerated deterioration of roads	Initial bridge repair and increased maintenance	Total cost applied per km to inundated roads (May 2000 $)	Total cost applied per km to inundated roads (Dec 2013 $)
Major sealed roads	*$ 32,000*	*$ 16,000*	*$ 11,000*	$ 59,000	**$88,712**
Minor sealed roads	*$ 10,000*	*$ 5,000*	*$ 3,500*	$ 18,500	**$27,816**
Unsealed roads	*$ 4,500*	*$ 2,250*	*$ 1,600*	$ 8,350	**$12,555**

7.1.2.3 Property Damages

Property damage has been applied to account for damage that is expected to occur to a property due to flood waters impacting the site, during the event and post-event. This damage includes damages such as garden damage, fence damage, damage due to extended inundation etc. This damage is only applied to properties if the building on that property is not impacted. This is because this damage is included in the derived damage curves and when the damage curves are activated the property damage is included in the building damage.

Property damage was applied to any delineated property that experienced flooding to a depth greater than 10 cm deep and covering over 1% of the property area but did not have a building that was impacted. These factors have been applied as flood depths less than 10 cm and for an area of less than 1% will not generally cause significant damage to a property.

In order to provide a more robust assessment of the likely property damage the land use types were used to determine the property zone for the impacted properties. This information was obtained from the Department of Sustainability and Environment (DSE) land use section of land.vic.gov.au.

The assigned economic damages were $1,000 per property for all properties impacted to greater than 10 cm of depth and for more than 1% of the area.

7.1.3 Annual Average Damage

Annual Average Damage (AAD) is calculated on a probability approach, using the flood damages calculated for each design event. Flood damages (for a design event) are calculated using the ‘damage curves’ described in the sections above. These damage curves approximate the damage occurring on a property for varying depths of flooding. The total damages in the summation of the damage to all houses and properties within the flood extent for that design event.

The AAD attempts to quantify flood damage that a floodplain would receive on average during a single year. It does this by using a probability approach. A probability curve is drawn, based on the flood damages calculated for each design event. This is shown in Figure 7-3. The 1% AEP design event has a 1% chance of occurring in any given year, and as such the 1% AEP damage is plotted at this point on the AAD curve. AAD is then calculated by determining the area under the curve.

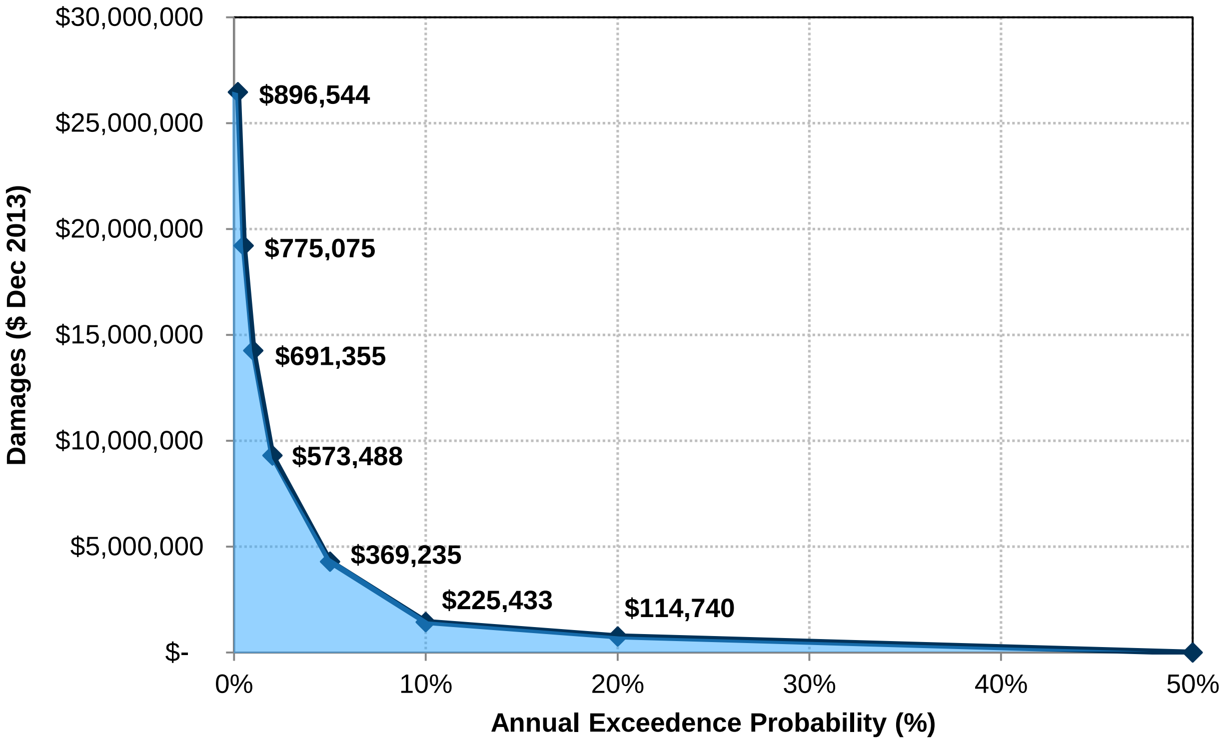


Figure 7-3 Flood damages used to estimate the Average Annual Damages

Further information on the calculation of the AAD can be found in the Floodplain Development Manual (NSW Government, 2005). For Euroa the AAD was calculated up to the 0.2% AEP event. The PMF event was not used as this event is expected to have a damage assigned which will be an order of magnitude greater than the 0.5% AEP event (i.e. $100m+). It is difficult to estimate the magnitude of damage in the PMF event and as this is also an extremely rare event it has low impact on the AAD calculation.

Based on the analysis as described in the above section the annual average damages (AAD) for the floodplain under existing conditions is approximately **$ 896,544 per annum**.

Table 7-6 Summary of Economic Flood Damages

Recurrence Interval	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP
Property Damage							
Property Damages	$206,378	$517,914	$1,685,790	$3,487,573	$3,904,188	$4,790,497	$5,599,538
Inundated properties (> 10cm depth, > 1% area)	187	256	598	970	1137	1292	1513
Building Damage							
Residential	1	1	24	72	154	222	323
Industrial	0	1	8	10	14	14	14
Commercial	0	0	7	24	37	55	75
Total buildings with overfloor flooding	**1**	**2**	**39**	**106**	**205**	**291**	**412**
Residential	$29,394	$42,562	$938,394	$3,179,685	$6,802,439	$10,227,889	$15,718,954
Industrial	$0	$23,526	$164,160	$265,923	$327,932	$397,049	$446,606
Commercial	$0	$0	$127,112	$451,331	$1,003,364	$1,289,288	$1,852,650
Total overfloor damages	**$29,394**	**$66,088**	**$1,229,666**	**$3,896,939**	**$8,133,734**	**$11,914,225**	**$18,018,210**
Road Damage							
Major	$344,917	$568,708	$840,488	$1,128,443	$1,278,781	$1,452,690	$1,626,901
Minor	$127,883	$209,812	$431,216	$661,443	$784,818	$896,505	$1,029,560
Unsealed	$56,362	$86,404	$115,991	$139,338	$158,047	$174,536	$191,440
Total road damages	**$529,162**	**$864,924**	**$1,387,695**	**$1,929,224**	**$2,221,647**	**$2,523,731**	**$2,847,901**
Total	**$764,934**	**$1,448,926**	**$4,303,151**	**$9,313,736**	**$14,259,569**	**$19,228,453**	**$26,464,649**

7.1.4 Assumption and Qualifications

A significant assumption in the calculation of the AAD was the assumption that the damages below the 20% AEP were extrapolated with the assumption that there are no damages at the 50% AEP event. Assuming a different slope for this line or a different AEP for zero damages will result in a change in the AAD calculated value. A paper was presented at the 2006 Floodplain Management Conference (Thomson et al, 2006) highlighting the issues associated with this assumption.

7.2 Mitigation Options

The mitigation options for the Euroa Flood Investigation have focussed on the Castle Creek system. Currently there is a levee along Castle Creek that protects the township of Euroa up to the 0.2% AEP (with no freeboard). The Castle Creek system also has issues that arise from high volumes of sediment build up through the main channel, floodplain and at structures. This sediment is in the form of coarse sand. The proposed mitigation assessments aim at evaluating and assessing mitigation measures that make changes to the levee system and address the sedimentation / blockage concerns for the creek.

The primary options to be assessed include the changes to the levee system. The current levee system freeboard is shown in Figure 7-4. This has been shown to demonstrate that the current levee system performance is expected to be sufficient with regards to the revised design events. In this figure the 1% AEP event is summarised with the freeboard varying between a minimum of 300 mm and 800 mm. It should be noted that the recommended minimum freeboard for urban levees is 600 mm in Victoria. For the Castle Creek levee complies with this requirement aside from a 450 m section downstream of the Freeway. It is recommended that this section is raised to comply with the 600 mm freeboard requirements.

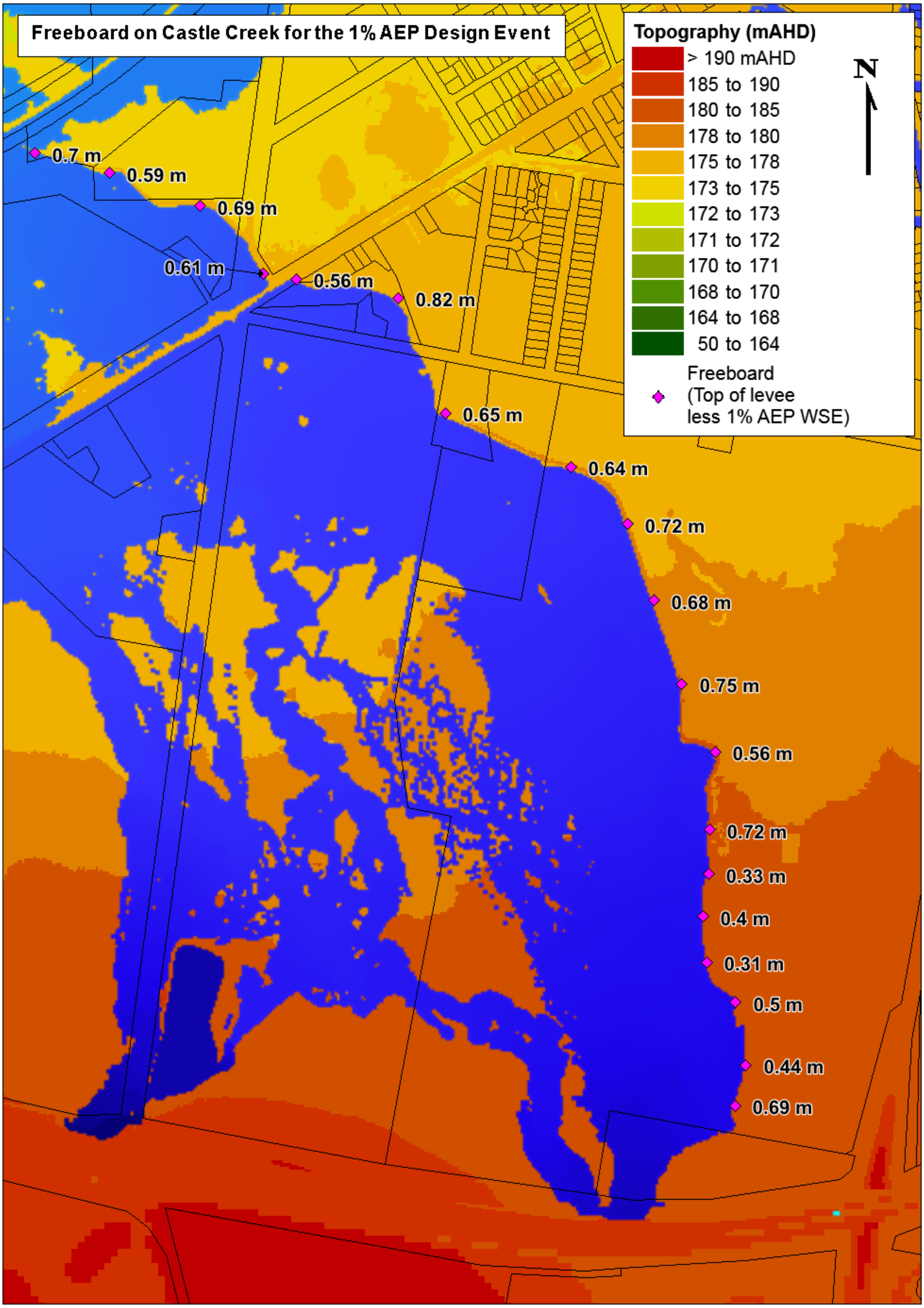


Figure 7-4 Current levee freeboard for the 1% AEP design event on Castle Creek

Overall six (6) mitigation options have been considered, ranging from modifications to the existing levee structure to cleaning and clearing of the system. The mitigation options have been summarised in Table 7-7. The mitigation options were run for the 20%, 5% and 1% AEP flood events.

Table 7-7 Proposed mitigation assessment scenarios

Mitigation Option	Description	Purpose
1	Removing the downstream end of the levee by 20-30m	To allow the floodwaters to pass under the additional culverts under the railway line that have been excluded by the current alignment of the levee.
2	Levee realignment along Birkett Street	To all additional flood storage in the area between Birkett Street and the levee (~100,000m^2) and to allow additional flow under the culverts along the railway line.
3a	Base Castle Creek scenario	This run aims to assess the performance of Castle Creek with the bridges and openings running at full capacity (no blockage) but the channel has standard roughness.
3b	Cleaned Castle Creek scenario	This run is as for Option 3a however the in bank channel has had some allowance for sediment clearing (deeper channel and reduced roughness). Structures are fully open.
3c	Blocked Castle Creek scenario	This scenario includes 50% blockage in all structures and standard roughness (i.e. fully vegetated and sedimented) along Castle Creek.
4	Castle Creek scour assessment	This assessment examines the velocities along castle Creek to determine if there is sufficient velocity to scour the sediment that builds up from the system.

7.2.1 Option 1 – Levee Shortening

Mitigation option 1 involved removing 30 metres of existing levee with the purpose of allowing additional floodwaters to flow north east through this opening to utilise the additional culverts under the railway in this area. The purpose of activating these culverts was to reduce the depths of flooding upstream of the railway embankment back up to the Euroa Main Road. The opening following the removal of the section of levee is approximately 45 metres from the railway line to where the levee starts, this is shown in Figure 7-5.

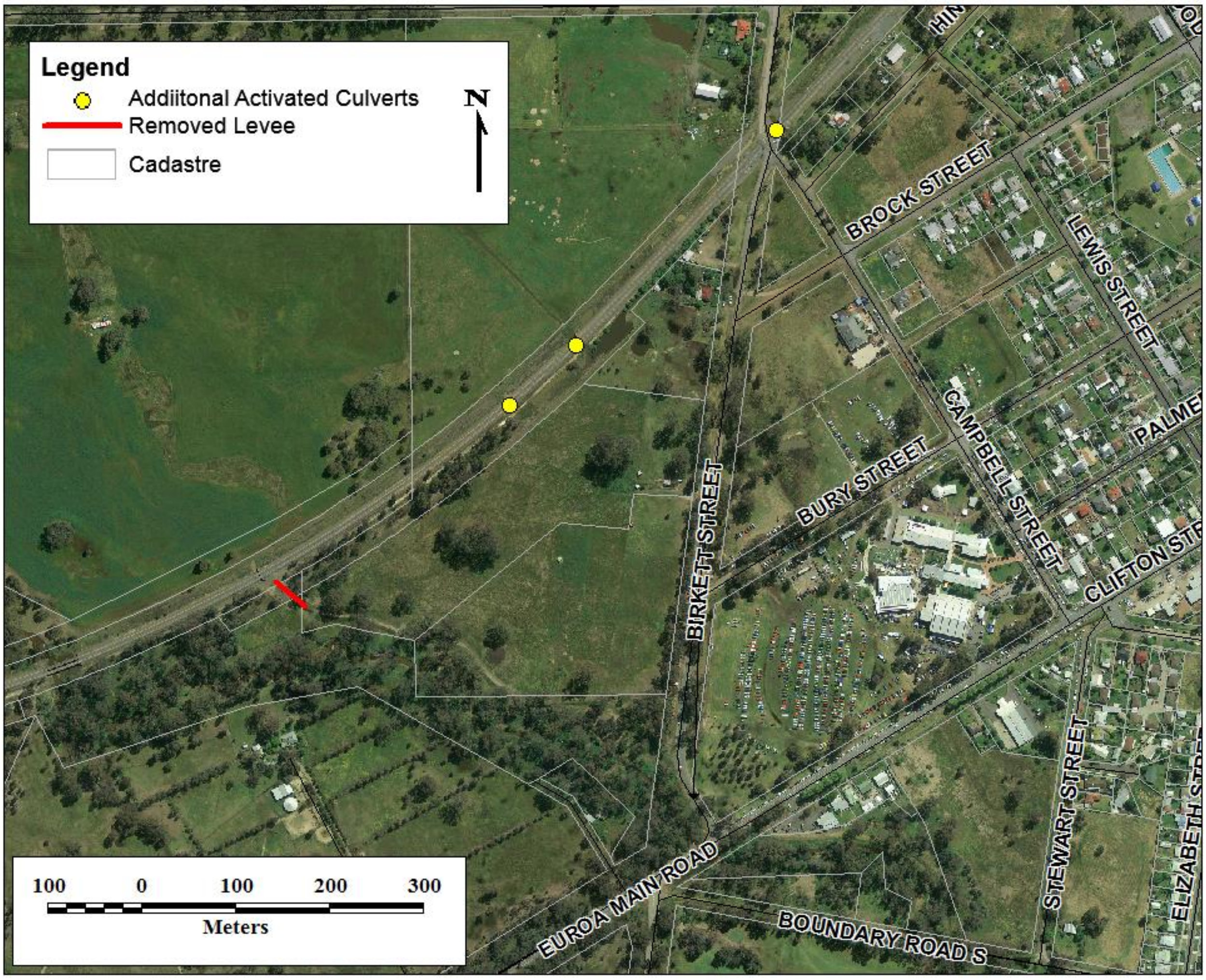


Figure 7-5 Mitigation option 1 – removal of 30m of existing levee

The results for the 1% AEP flood event are shown as a difference plot in Figure 7-6 (Option 1 less existing conditions). The results show that the increased opening allows for additional floodwaters to flow along the railway line up to the Birkett Street culvert under the railway. Downstream of this location there is additional flood extent and depths as a result of this increased flow. Some of this increased flow and depths are adjacent to existing buildings.

The benefit of this opening is minimal in the main channel and the reduction in peak depths is less than 3 cm upstream of the main railway opening and this only extends ~200m upstream. There are no properties or buildings in this area that benefit from this change in orientation. Downstream of the railway there are also small areas of minor reductions in peak flood depths.

The advantages and disadvantages of this options are discussed further in Section 7.3.

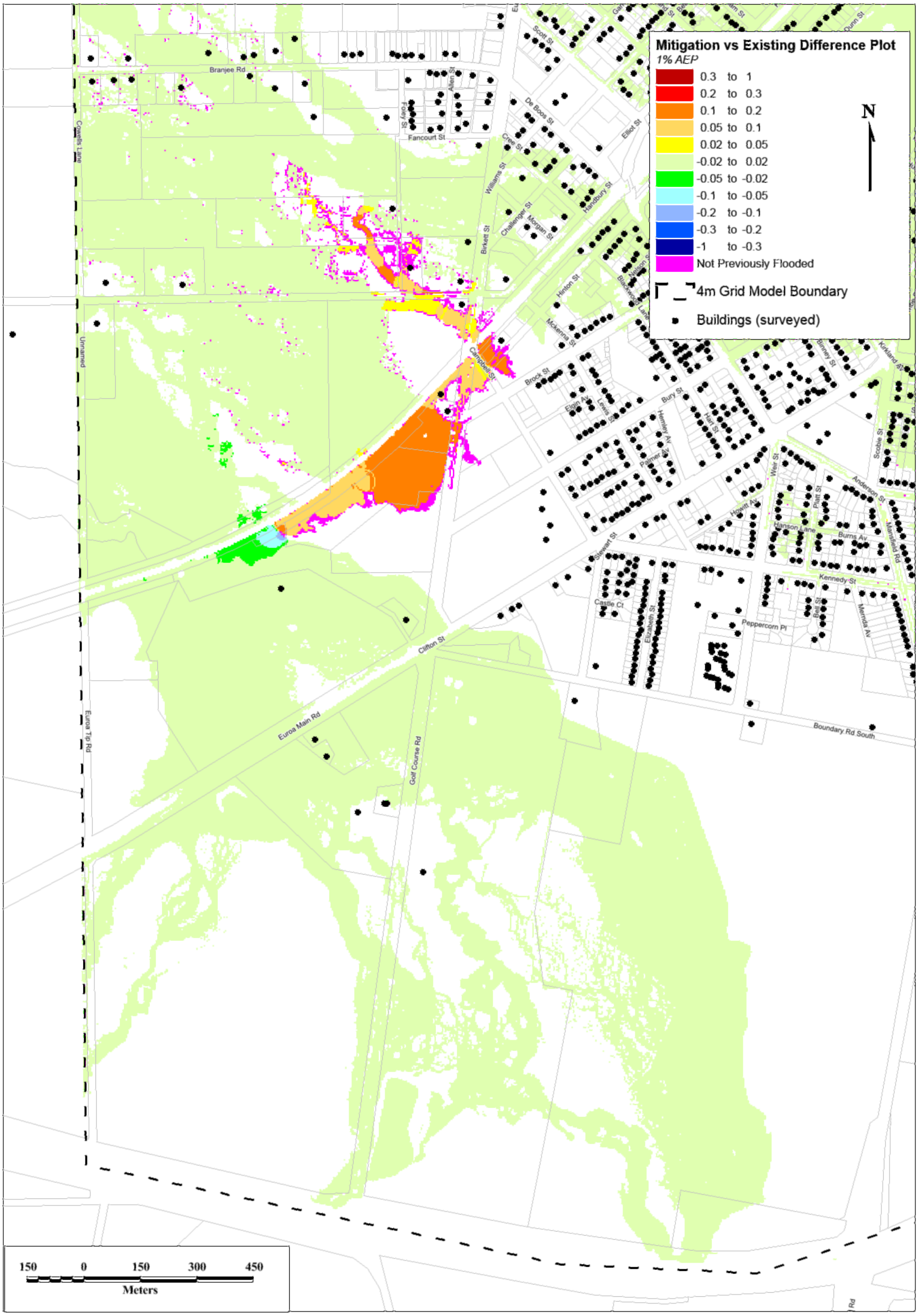


Figure 7-6 Mitigation option 1 difference plot for the 1% AEP event – Option 1 less existing conditions

7.2.2 Option 2 - Levee Realignment

The Option 2 mitigation involved removing a large section of the existing levee back to the Euroa Main Road and reinstating this along Birkett Street. The aim of this levee realignment was to activate the large area of unused land that is currently blocked by the levee (~100,000 m^2). This area may provide additional floodplain storage which has potential to change the flood behaviour in this area. The realigned levee is shown in Figure 7-7. The levee extends to Birkett Street but protects the existing property near the corner of Birkett Street and the railway line.

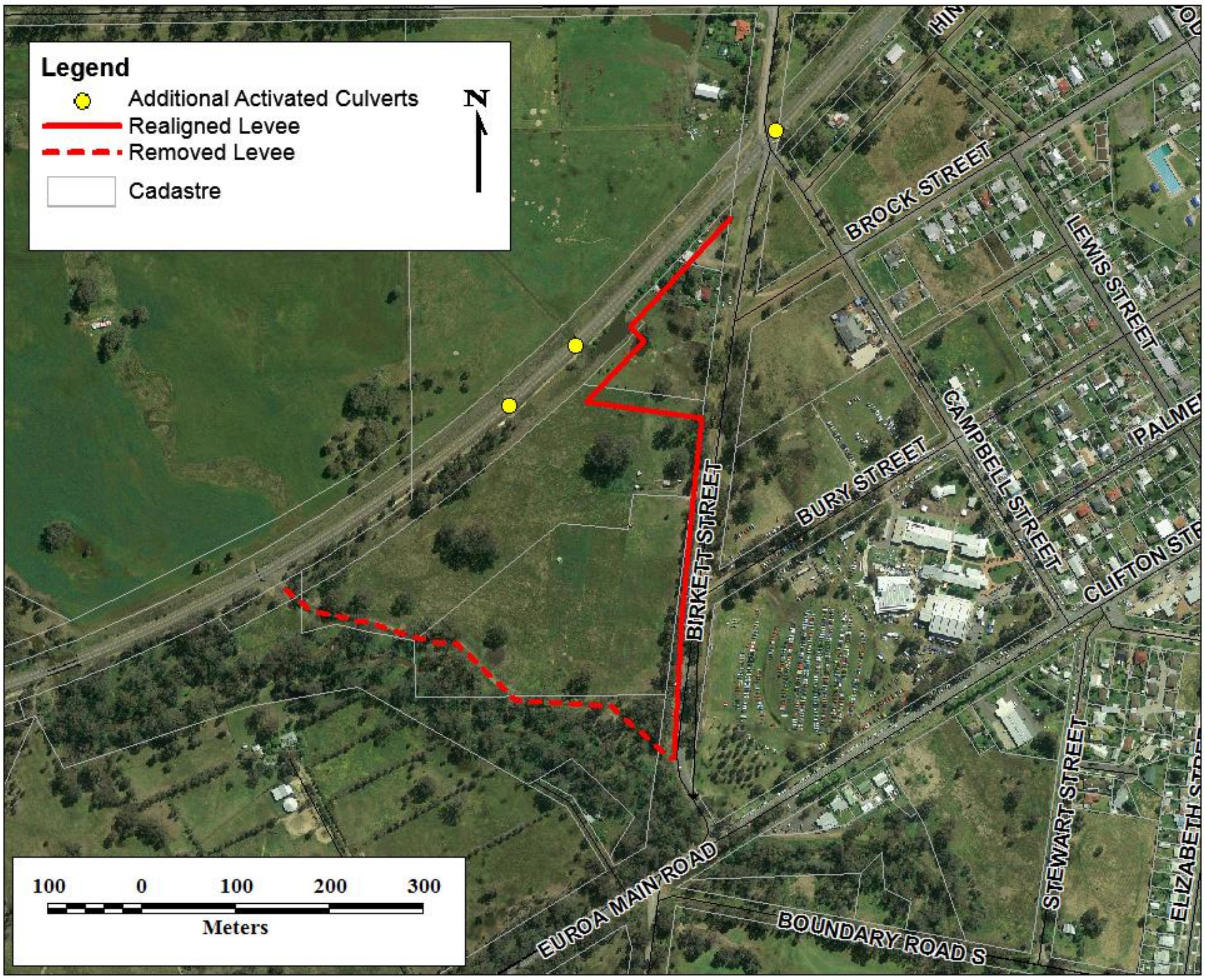


Figure 7-7 Mitigation Option 2 – Levee realignment

The difference between the mitigation option 2 and existing 1% AEP conditions are presented in Figure 7-8. Relocating the levee reduces the peak flood depths upstream of the main railway culvert along the main Castle Creek by up to 30 cm. These reductions reach approximately 400 m upstream from the railway, unfortunately these reductions do not reach the property at 207 Euroa Main Road. This property has the high damages and is the only property within Euroa to have overfloor flooding in the 20% AEP event.

Additional flooding has been introduced to the area of the Birkett Street railway crossing. Additional properties are impacted in this area with flood waters reaching depths of over 30 cm around some of the buildings in this area. Downstream of the railway culvert additional flooding is introduced across the properties downstream of this location. There is a significant area of increased flood depths downstream as a result of the three additional culverts being more heavily utilised during the event.

The advantages and disadvantages of this options are discussed further in Section 7.3.

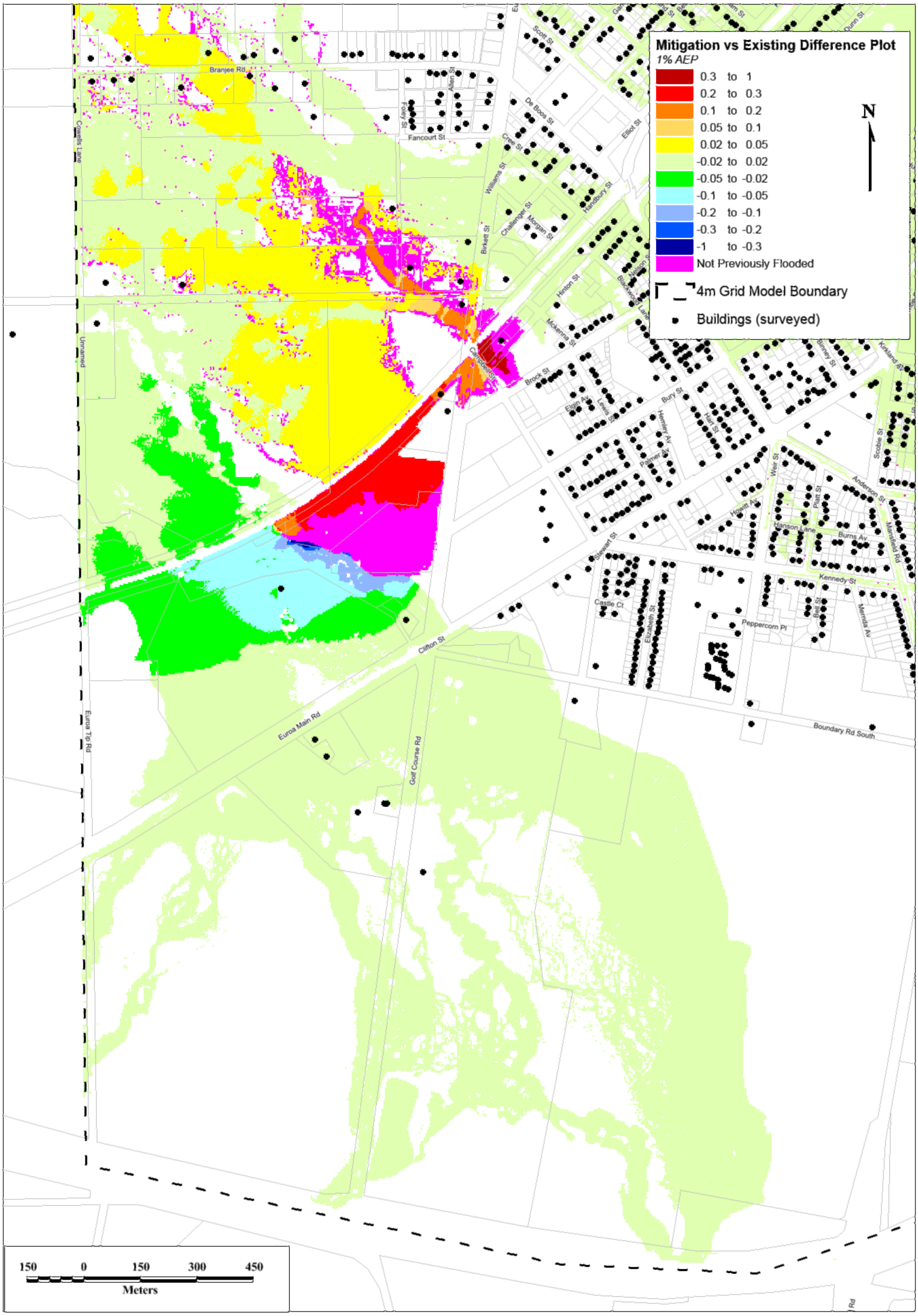


Figure 7-8 Mitigation option 2 difference plot for the 1% AEP event – Option 2 less existing conditions

7.2.3 Options 3a, 3b, 3c – Castle Creek sedimentation and blockage assessment

Castle Creek is subject to a high rate of sand sedimentation as a result of erosion from the upper catchment. This sand is a coarse grade sand that collects at sag points and structures when flows are insufficient to carry the sediment downstream. This has been an ongoing concern for the flow capacity of the Castle Creek main channel and the structures. These structures are particularly important as these are the control points at the roads and railway through the catchment.

The design runs assumed that the structures would be free of blockage and the channel was set at the LiDAR captured levels (includes some sedimentation). The 1% AEP design run forms the basis of the Mitigation Option 3a. This is likely to be the best case scenario for Castle Creek as it is subject to progressive sedimentation between large flood events.

Option 3b examines a system where the structures remain free from blockage and the main channel is cleared of some sediment (where this is possible and free from vegetation). This option aims to examine if periodic channel clearance and maintenance can reduce flood levels along Castle Creek. The hydraulic model for this scenario has the main channel deepened and the roughness reduced from the Golf Course down to the railway embankment.

Finally, Option 3c examines the worst case scenario where the structures are blocked up as a result of sedimentation. This is a more likely scenario of what the current and future situations would be as there is some sedimentation within the creek at present.

Difference plots have been generated between:

- Option 3b (reduced channel roughness) less option 3a for the 1% AEP event (Figure 7-9)
- Option 3c (structure blockage) less option 3a for the 1% AEP event (Figure 7-10).

Reducing the channel roughness through the portion of the main channel of Castle Creek from the Golf Course to the railway bridge resulted in almost reductions in peak flood depths up to 10 cm. The channel clearing assessed in this scenario was developed by reducing the channel roughness from a Manning's 'n' of 0.06 down to 0.035. This is a substantial reduction in roughness and this was initiated to get the likely maximum benefit of any channel clearing. As this was a major reduction in roughness it would be expected that channel clearing would have a maximum impact on the flood behaviour as shown in Figure 7-9, in reality the reductions in peak flood depths would only be a fraction of this depending on the works undertaken.

The benefits from the roughness reduction (clearing) are generally located at the cleared site only this is due to the fact that the peak flood depths caused in this area are primarily caused by the hydraulic controls through the floodplain of the Euroa Main Road and the railway line. This causes flows to back up behind these structures and reach their peak flood depths. Much of the flooding here is caused by the structure restrictions to the flow paths rather than the flow conveyance of the main Castle Creek channel.

For the structure blocking scenario there were some small changes in flood behaviour due to the reduced carrying capacity of the structures. The structures for this scenario were blocked at 50%. The main area where this caused increased depths was along the Euroa Main Road. This area is also where there are a number of impacted properties.

The mitigation assessment identified that ensuring the structures are not blocked and are operating at full capacity will result in more efficient transfer of floodwaters. The increases in depths observed are relatively minor however with the peak increase at 10 cm over a small area.

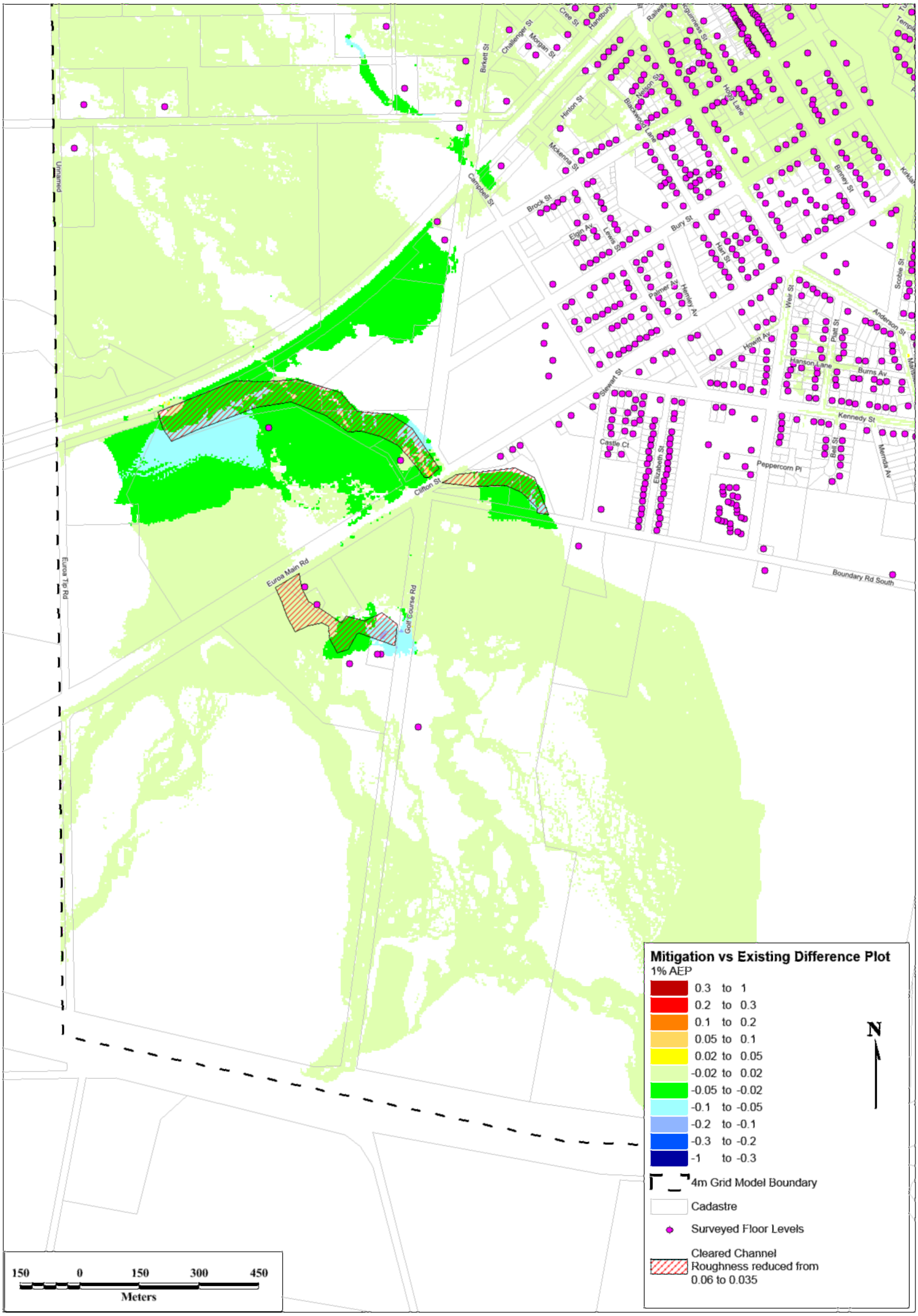


Figure 7-9 Difference between Option 3b (some channel clearing) and 3a – 1% AEP event

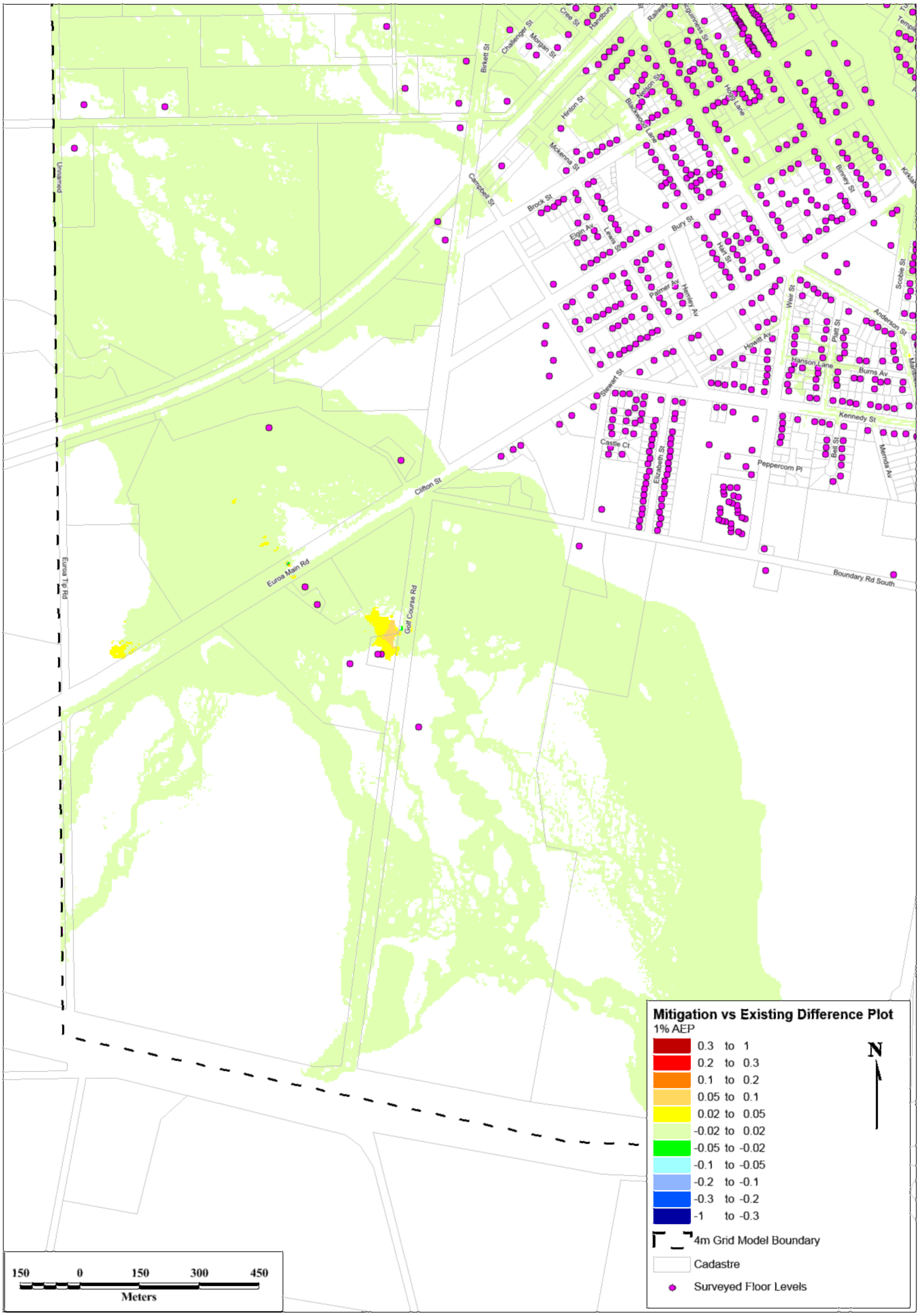


Figure 7-10 Difference between Option 3c (structure blockage) and 3a – 1% AEP event

7.2.4 Option 4 - Castle Creek scour assessment

This assessment aimed at examining the velocity that would be required within Castle Creek to clear the structures of sediment and to mobilise bed sediment within the system during a flood event. In order to assess the sediment transport potential, the peak velocities for the design events have been assessed to determine if they are sufficient to clear the system during a flood event.

Sediment transport for coarse sands is initiated between 0.2 and 1 m/s (U.S Department of Interior et al., 2006) and sediment deposition generally occurs when velocities are below the 0.2 m/s. This is dependent on the sediment sizing, with larger sediments being deposited at higher velocities than finer sediment. A chart showing a typical relationship between velocity, sediment transport, deposition and erosion is shown in Figure 7-11. For the coarse sand in Castle Creek the typical grain size is likely to range from a fine sand to a coarse gravel (from the site inspection) which make the grain size range from less than 1 mm up to approximately 8 mm with a mix of finer grained sediment and some smaller gravel components.

Figure 7-11 shows that for grain sizes ranging from sub mm in size up to approximately 8 mm in diameter will be eroded with velocities of 1 m/s or less.

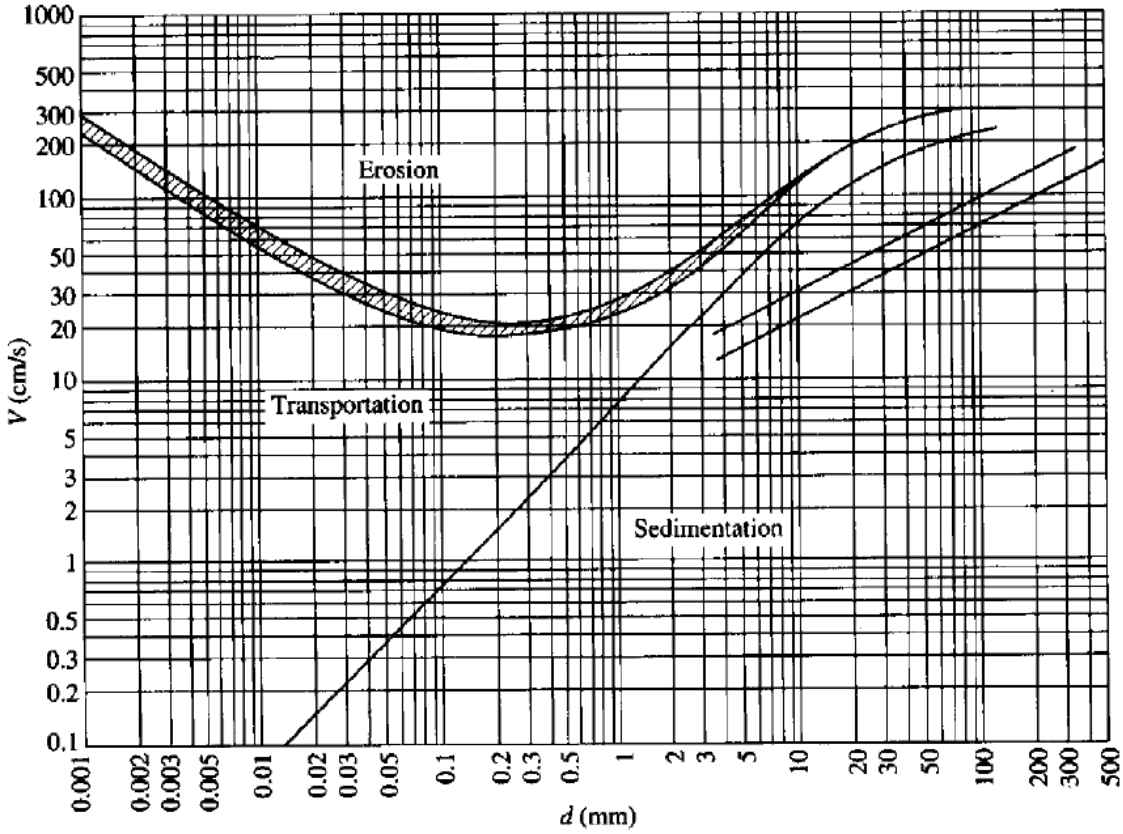


Figure 7-11 Erosion-deposition criteria for uniform particles (Hjulstrom, 1935)

For the structures on Castle Creek the three main areas where sedimentation occurs are on the bridge and culverts on the Euroa Main Road and under the main Railway bridge opening. These locations are shown in Figure 7-12.

Figure 7-12 Structure locations for scour assessment on Castle Creek

The peak velocities and flow rates through these structures are summarised in Table 7-8 for the three locations. At all three locations the maximum velocity in the 20% AEP is equal or greater than 1 m/s and increases as the events get rarer. The peak velocity at the railway bridge was 1.4 m/s which should scour and erode the built up sediment assuming that there is nothing stabilising the sediment i.e. vegetation. Velocities under Euroa Main Road are much higher and should provide suitable erosion to clear the structures in large flood events.

Table 7-8 Velocity and maximum flow rate through the Castle Creek structures

AEP	Euroa Railway Bridge		Castle Creek under Euroa Main Rd		Tributary Culverts	
	Max. Velocity (m/s)	Max Q (m^3/s)	Max. Velocity (m/s)	Max Q (m^3/s)	Max. Velocity (m/s)	Max Q (m^3/s)
20%	1.0	12.7	1.0	20.5	2.2	7.8
10%	1.1	19.2	1.2	25.9	2.9	17.4
5%	1.2	26.5	1.3	30.2	3.0	19.1
2%	1.2	35.6	1.5	34.5	3.1	20.8
1%	1.3	45.3	1.6	38.0	3.1	21.2
0.5%	1.3	53.2	1.7	41.4	3.1	21.2
0.2%	1.4	67.3	1.9	44.8	3.1	21.2

To assess the main channels through the Castle Creek catchment the peak velocities reached have been summarised in Figure 7-13, Figure 7-14 and Figure 7-15 for the 20%, 5% and 1% AEP events.

For the 20% AEP event much of the flow is maintained within the main channels. The peak velocity in this event exceeds 1 m/s through the main channel along the entire length of Castle Creek. This suggests that scour and sediment transport would be possible during events as frequent as a 20% AEP. The peak velocities over the floodplain however are relatively slow moving and are likely to deposit sediment.

For the 5% AEP events the velocity across the floodplain increases to 0.5 to 0.7 m/s which is likely to be able to mobilise some sediment. The main channel peak velocities are greater than 1 m/s but these exceed 2 m/s in some locations. The velocities for the 1% AEP event shows velocities across the floodplain reaching as high as 1 m/s, the main channel velocity increases as well.

Overall the potential for scour and sediment mobilisation is high for events even as frequent as the 20% AEP. For larger events such as the 5% and 1% AERP event there would be significant chance of erosion and scour of a coarse sand based sediment. As such it is expected that between large events sediment would accumulate but once a large event occurred that much of this sediment would be naturally cleared. It is more likely that this would be cleared around the structures as these have higher velocities than the main channel.

For the main channel and structures there is a risk that as sediment accumulates that this is stabilised via vegetation and hence a much higher velocity of floodwaters is required to erode and transport the sediment. Checks should be carried out on the system around the structure through the main channel to ensure large pockets of deposited sediment are not being stabilised by new vegetation otherwise the structure capacities may reduce and as well as the main channels carrying capacity.

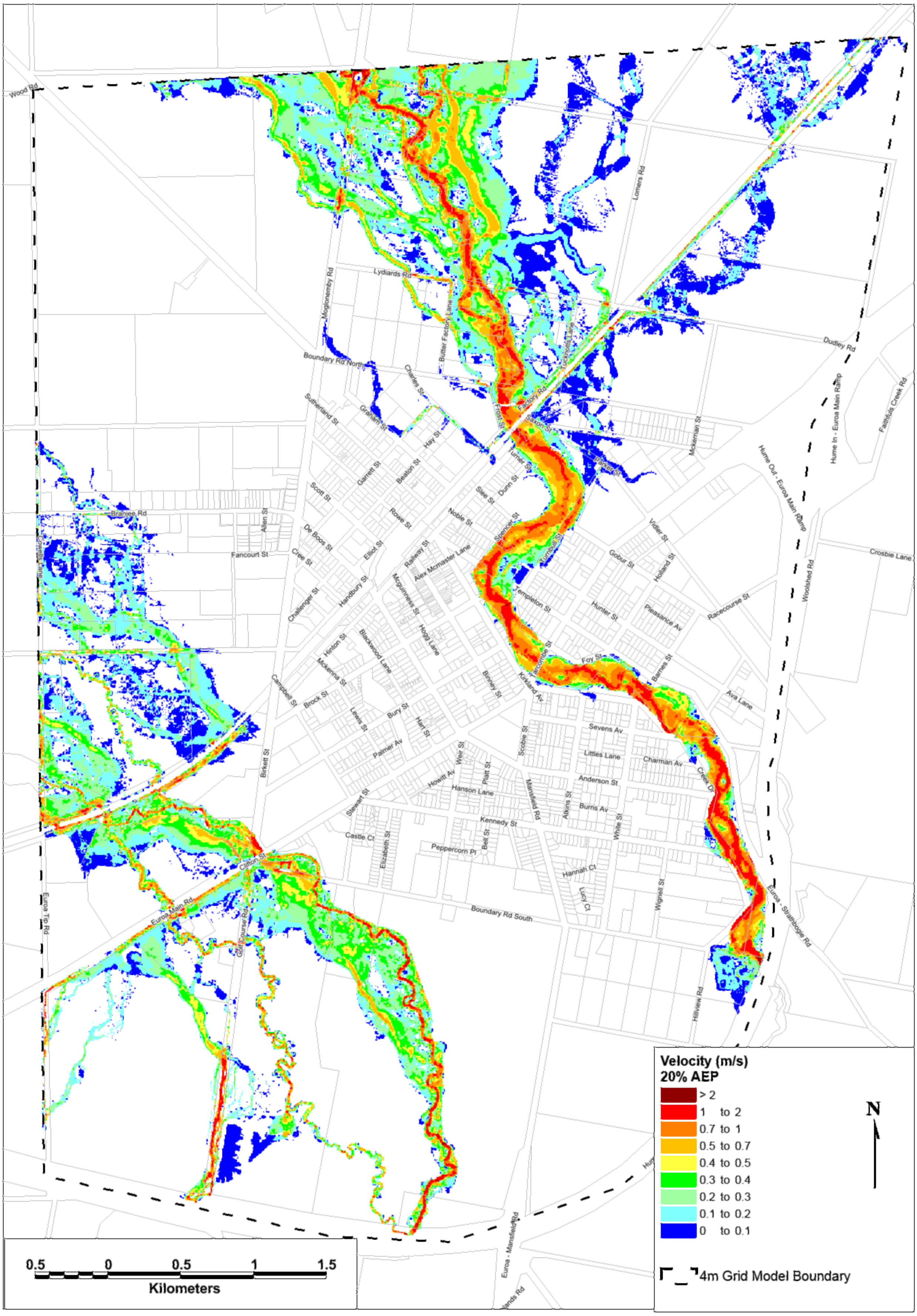


Figure 7-13 Peak velocities in the study area for the 20% AEP design event

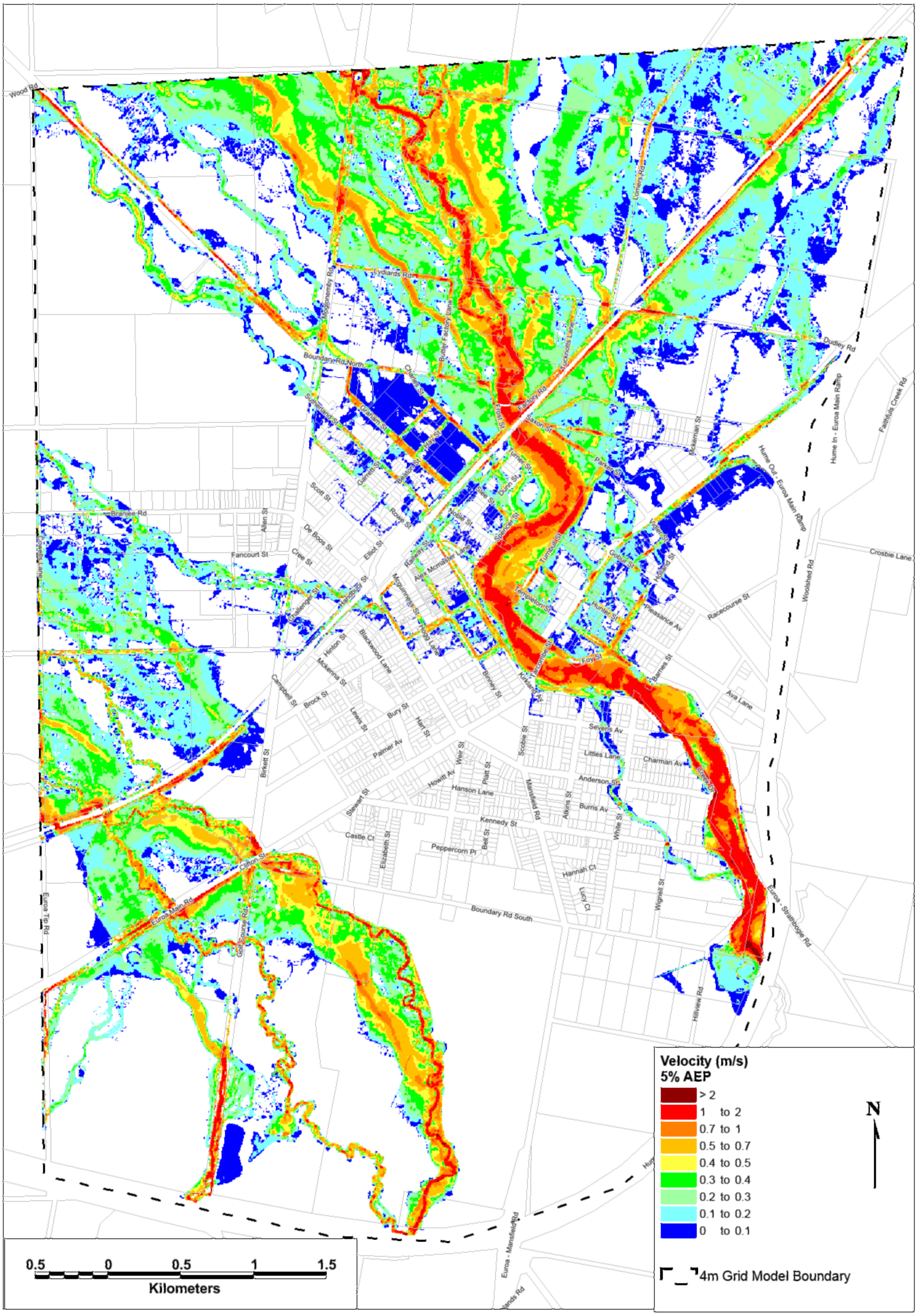


Figure 7-14 Peak velocities in the study area for the 5% AEP design event

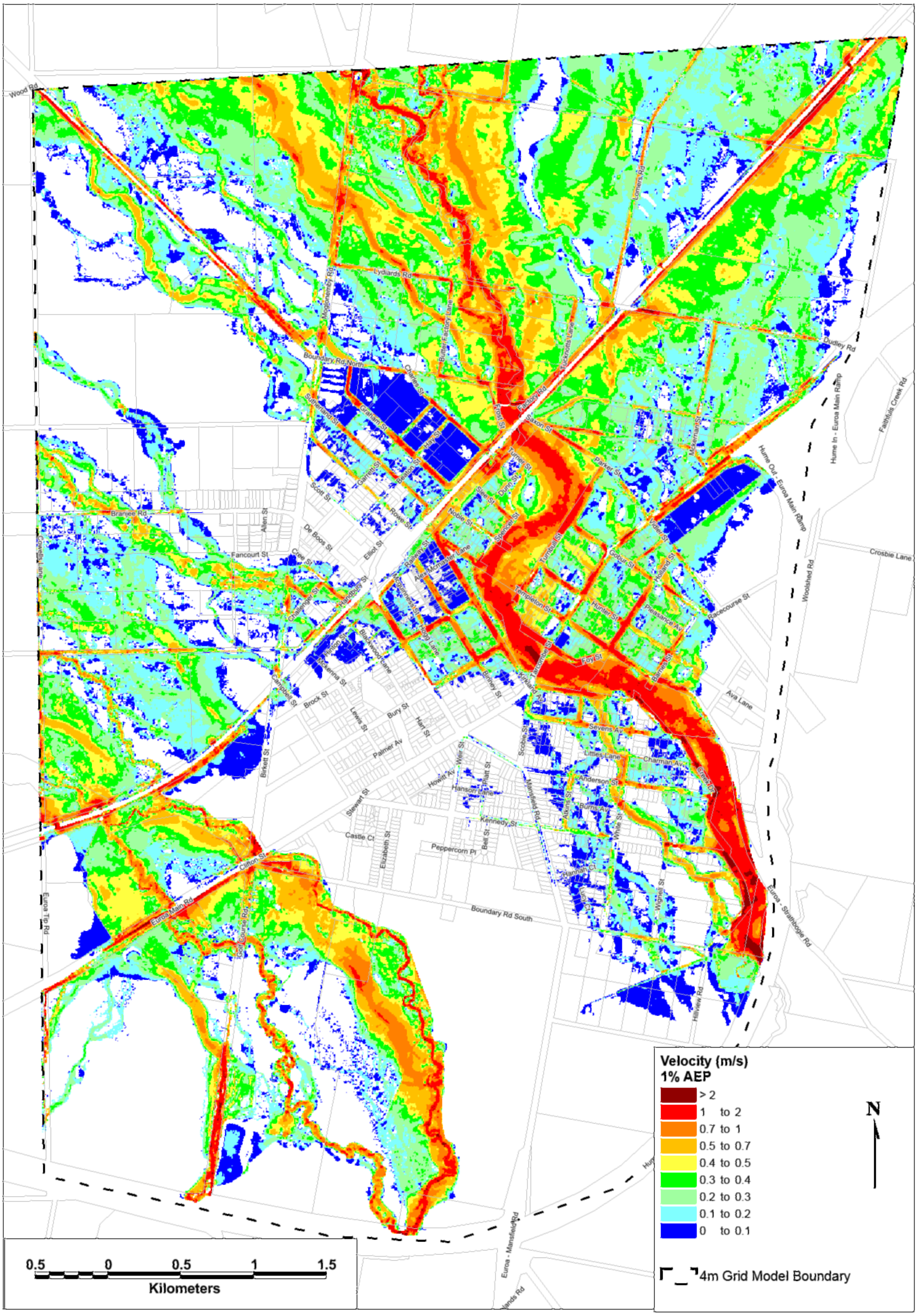


Figure 7-15 Peak velocities in the study area for the 1% AEP design event

7.3 Mitigation Assessment

The mitigation measures assessed for the Castle Creek catchment were not specifically aimed at reducing damages but more around managing the flood extents and flood behaviour. The damages have been assessed for the 20%, 5% and 1% AEP events in Table 7-9. None of the options reduced the overfloor flooded properties across Euroa.

Table 7-9 Damage comparison for the catchment management options

No.	Description	20% AEP	5% AEP	1% AEP
Existing	**Existing design events**	**$764,934**	**$4,303,151**	**$14,259,569**
1	Shortening of levee by 30m	$815,346	$4,395,890	$14,423,554
		+6.6%	**+2.2%**	**+1.2%**
2	Levee realignment along Birkett Street	$802,795	$4,401,634	$14,423,077
		+4.9%	**+2.3%**	**+1.1%**
3a	No structure blockage and standard roughness, same as existing design conditions.	$764,934	$4,303,151	$14,259,569
		N/A	**N/A**	**N/A**
3b	No structure blockage and reduced channel roughness	$761,131	$4,292,284	$14,263,682
		- 0.5%	**-0.3%**	**0.0%**
3c	50% Structure blockage and standard roughness	$800,438	$4,395,569	$14,418,671
		+4.6%	**+2.1%**	**+1.1%**

Mitigation option 1 examined shortening the levee to allow additional water to flow along the railway line and through the three small culverts. Ultimately this led to an increase in damage associated with the events due to additional properties being impacted, additional road area being impacted and the increase in flood depths against some buildings (below floor level). The sharpest increase was in the 20% AEP as in the existing scenario very little water passed through the current opening at the end of the levee. However the 5% and 2% AEP events already had some flooding in this area in the existing conditions.

Similarly the realignment of the levee option increased the damages in much the same way as Mitigation option 1 however this was somewhat offset by the reduction in levels upstream of the railway embankment. The damages still increased in all design events simulated.

Mitigation 3a was the same as the existing design run and assumed that the structures were unblocked with the current roughness through the Castle Creek catchment. This was the base case for this assessment.

Mitigation option 3b involved clearing vegetation and sediment from the main channel of Castle Creek from the Golf Course down to the railway line. These changes did not involve modifying the structures. Reducing the roughness in this area resulted in minor changes to the flood behaviour. This did result in a small reduction in property damage and peak flood depths against some houses in the immediate area. This small change in flood behaviour occurred as the peak flood depths in this area are controlled primarily to the hydraulic controls of the Euroa Main Road and railway line which cause flows to fill back up the system.

Mitigation option 3c examined the impact of blocking the structures. As expected, the blocking of the structures results in an increase in damages. The implication of this that is if the structures progressively block up to the modelled 50% blockage rate then the damage during flood events is expected to increase. The increase in flood damage is proportionally higher for the more frequent events.

7.3.2 Recommendations

Of the mitigation options assessed Mitigation options 1 and 2 examined levee realignment solutions, both of these options increased damages and are not appropriate for reducing damages on upstream properties. As such these are not recommended to implement.

The recommended mitigation approach is periodic assessment and clearing of the structures under Euroa Main Road and under the main railway embankment bridge. If checks are undertaken to ensure that there is no vegetation locking the sediment in place then during flood events the velocity of flood water is expected to scour and erode the built up sand pockets (mitigation option 4 assessed the mobilisation requirements). If the structures become excessively blocked (>50% blockage) it is recommended that they are cleared of sediment.

Cardno have developed some preliminary guidelines for the management of the structures under the Euroa Main Road and under the Railway Embankment. The locations of these structures are as per Figure 7-16 and they correspond to those assessed in the scour assessment.

Figure 7-16 Structure locations recommended for monitoring and clearing

The strategy for ensuring that the structures are not blocked or impeded in large flood events is recommended for two main reasons:

1. To ensure that the structures can operate at full capacity during flood events to reduce the depths of flooding upstream of these structures.
2. To ensure that any sediment that has built up at the structures has not been 'locked' in place by vegetation so that during larger events this can be mobilised to clear the structure.

The three main structures have a strategy as stated in Table 7-10. This strategy should initially aim at having checks undertaken on the structures at a monthly frequency. The inspections should aim at capturing:

- Photograph of the upstream and downstream sides of the structure.
- Approximate blockage percentage of the structure (measurement of the full opening versus the blocked depth.
- Inspection of the accumulated sediment for vegetation growth.

It is anticipated that the inspections are to be more important during the wetter periods and following rainfall events across the catchment, however it is during the periods of low rainfall activity that sediment can be 'locked' in place by vegetation and compaction. The inspections of these structures has a number of benefits in that it provides an improved understanding of the sediment build up within the system over time and documents this process. This documentation can then be used to reduce or increase the inspections on the structures.

The recommendations as stated in Table 7-10 are an initial guide for the inspection ratio and importance within the system and will require review following a period of inspections being undertaken (annual or six monthly review).

Clearing of the channel away from the structures for Castle Creek has some impact on the flood behaviour but is likely to involve impacting on the natural system and is not recommended. Vegetation clearing and sediment clearing in these areas is not expected to change the flood behaviour and damages significantly. Periodic sand removal of the main channel may be required in order to control the sediment build up at the structures and this should be monitored in an ongoing manner.

The current levee for Castle Creek protects up to the 0.2% AEP event (albeit with no freeboard). Current standard practice for urban levees is a minimum of 600 mm freeboard above the 1% AEP peak level. For the Castle Creek level the majority of the levee meets this criteria, however there is a small section (450 m) near the freeway end of the catchment which should be raised to meet these requirements. See Figure 7-4 for the current freeboard and location required additional levee freeboard.

Table 7-10 Recommended inspection and clearing strategy for sediment build-up at structures

Structure	Description	Priority	Frequency	Trigger	Action
Castle Creek Railway Bridge	This is the major flow path for Castle Creek under the railway embankment and as such need to be relatively free of sediment and it must be ensured that sediment is not being locked in place by vegetation.	High – large flow control structure	Monthly[1]	50% of opening filled with sediment Sediment showing signs of compaction or vegetation growth	Clear the sediment to a minimum of 25% of the opening height and ensure no vegetation is present in the sediment near the structure.
Tributary Culverts	The tributary culverts are undersized and as such it is important that they are clear and free of sediment. Periodic inspections should be undertaken.	Low – Not prone to as much sedimentation as other structures	Monthly[1]	50% Blockage or vegetation and obstructions at entrance or exit	Clear the sediment to a minimum of 10% of the opening height and ensure entrance and exits are clear of vegetation / obstructions
Castle Creek under Euroa Main Road	The Castle Creek bridge under the Main Road is prone to sedimentation, particularly upstream of the structure.	High – large flow control structure	Monthly[1]	50% of opening filled with sediment Sediment showing signs of compaction or vegetation growth	Clear the sediment to a minimum of 25% of the opening height and ensure entrance and exits are clear of vegetation / obstructions

[1] The inspections are recommended be made monthly initially with the sediment levels estimated against the structure, the change in sediment should be monitored over time and the inspection frequency adjusted accordingly. It is noted that frequent inspections would also facilitate a better understanding of the rates of accumulation of sediment within the Castle Creek system for future assessments.

8 Conclusions

The Euroa Flood Investigation was undertaken with the aim to revise the flood mapping for Euroa and to improve the understanding of the floodplain behaviour for the Seven Creeks and Castle Creek. The primary outcomes from the project include:

- Stage 1 – Hydrologic and hydraulic modelling (calibration and validation); and the hydraulic assessment (performance) of the Castle Creek levee, including analysis for potential improvements.
- Stage 2 – The development of the flood intelligence, MFEP Appendices, municipal flood response plan and land use planning maps.

The report outlines the details of the study from the Stage 1 component of this study. Stage 2 focuses on the flood emergency response aspects of the study and is to be presented in a separate document.

The hydrology was developed through a detailed process that calibrated the hydrologic model to 5 events. The design events were ultimately set to match the previous SKM assessment to ensure there was consistency in the planning controls and outputs from the project. The Castle Creek flows were developed using the calibrated hydrologic models and calibrated model parameters from Seven Creeks.

The hydraulic modelling simulated the 20%, 10%, 5%, 2%, 1%, 0.5%, 0.2% AEP and the PMF event for both Seven Creeks and Castle Creek. The model was calibrated to the 1993 and 2010 flood events.

To assess the local drainage issue the hydraulic model was simulated using a direct rainfall on grid approach which included the local drainage network. This model process aimed at identifying the local issues which may cause flooding in high intensity rainfall events independent of the riverine flooding.

Key outputs from the hydraulic modelling process were a suite of maps outlining:

- Peak flood depths for all design flood events (as shown in Figure 5-9 to Figure 5-16).
- Flood extents for all design events.
- Flood planning controls (floodway overlays for the LSIO and FO).
- Velocity and hazard maps for the design events.
- Flood extents with peak water surface elevations at 200mm contours.
- Series of maps showing the peak depths and extents corresponding to gauge levels for both the Seven Creeks at Euroa at 200mm intervals (and one 100mm interval) between 4.6m and 6.5m on the gauge.
- Properties impacted during each flood event have been shown on each flood map, this includes properties with overfloor flooding and with water impacting the house below floor level.
- Historic calibration events showing depths and extents (the calibration events, 1993 and 2010).
- Municipal Flood Emergency Plan (MFEP) maps for inclusion in the MFEP appendices.
- Minor, moderate and major flood levels have been mapped for Seven Creeks at Euroa (minor 2.5m, moderate 4.0m and major 4.6m) and Castle Creek at Telfords Bridge (minor 1.2m, moderate 1.8m and major 2.4m).

The outputs from the hydraulic modelling are a key input into the Stage 2 Municipal Flood Emergency Plan documents.

The design events were used to develop a damage assessment for the catchment. The damage assessment estimated the Annual Average Damages at $ 891,970 per annum. This is a high annual damage figure and reflects the widespread damage that can occur with flooding in Euroa.

A range of mitigation options were considered for the Castle Creek system, these ranged from physical modification of the levee through to management of the sediment within the system. The mitigation options were focussed more on the management of the system rather than to provide additional protection to the township.

The mitigation options 1 and 2 demonstrated that modifying the levee to utilise the additional railway culverts increases the flooding on a number of properties but does not reduce the peak flood depths upstream of the railway embankment sufficiently to benefit the buildings adjacent to the Euroa Main Road. Both mitigation options led to increased damages associated with flood events.

Mitigation Options 3a, 3b and 3c examined the impact of sedimentation and structure blockage. The assessment identified that if the structures block by up to 50% then there are some areas of increased damages and the total damage increases. If channel clearing occurs in isolation away from the structures it is expected that only minor changes will occur to the flood behaviour.

The final mitigation assessment examined the erosion and scour assessment for the range of design events. The velocity in Castle Creek is estimated to be sufficient to mobilise sediment accumulated in the main channel and structures assuming this accumulated sediment is not locked in via vegetation growth between events. Velocities in the main channel and structures in flood events as frequent as the 20% AEP event are expected to exceed 1 m/s which is sufficient to mobilise coarse sand.

Mitigation Recommendations

Of the mitigation options assessed Mitigation options 1 and 2 examined levee realignment solutions, both of these options increased damages and are not appropriate for reducing damages on upstream properties. As such these are not recommended to implement.

The recommended mitigation approach is periodic assessment and clearing of the structures under Euroa Main Road and under the main railway embankment bridge. If checks are undertaken to ensure that there is no vegetation locking the sediment in place then during flood events the velocity of flood water is expected to scour and erode the built up sand pockets (mitigation option 4 assessed the mobilisation requirements). If the structures become excessively blocked (>50% blockage) it is recommended that they are cleared of sediment. See Table 7-10 for the recommended inspection strategy and clearing triggers.

Clearing of the channel away from the structures for Castle Creek has some impact on the flood behaviour but is likely to involve impacting on the natural system and is not recommended. Vegetation clearing and sediment clearing in these areas is not expected to change the flood behaviour and damages significantly. Periodic sand removal of the main channel may be required in order to control the sediment build up at the structures and this should be monitored in an ongoing manner.

The current levee for Castle Creek protects up to the 0.2% AEP event (albeit with no freeboard). Current standard practice for urban levees is a minimum of 600 mm freeboard above the 1% AEP peak level. For the Castle Creek level the majority of the levee meets this criteria, however there is a small section near the freeway end of the catchment which should be raised to meet these requirements. See Figure 7-4 for the current freeboard and location required additional levee freeboard.

9 References

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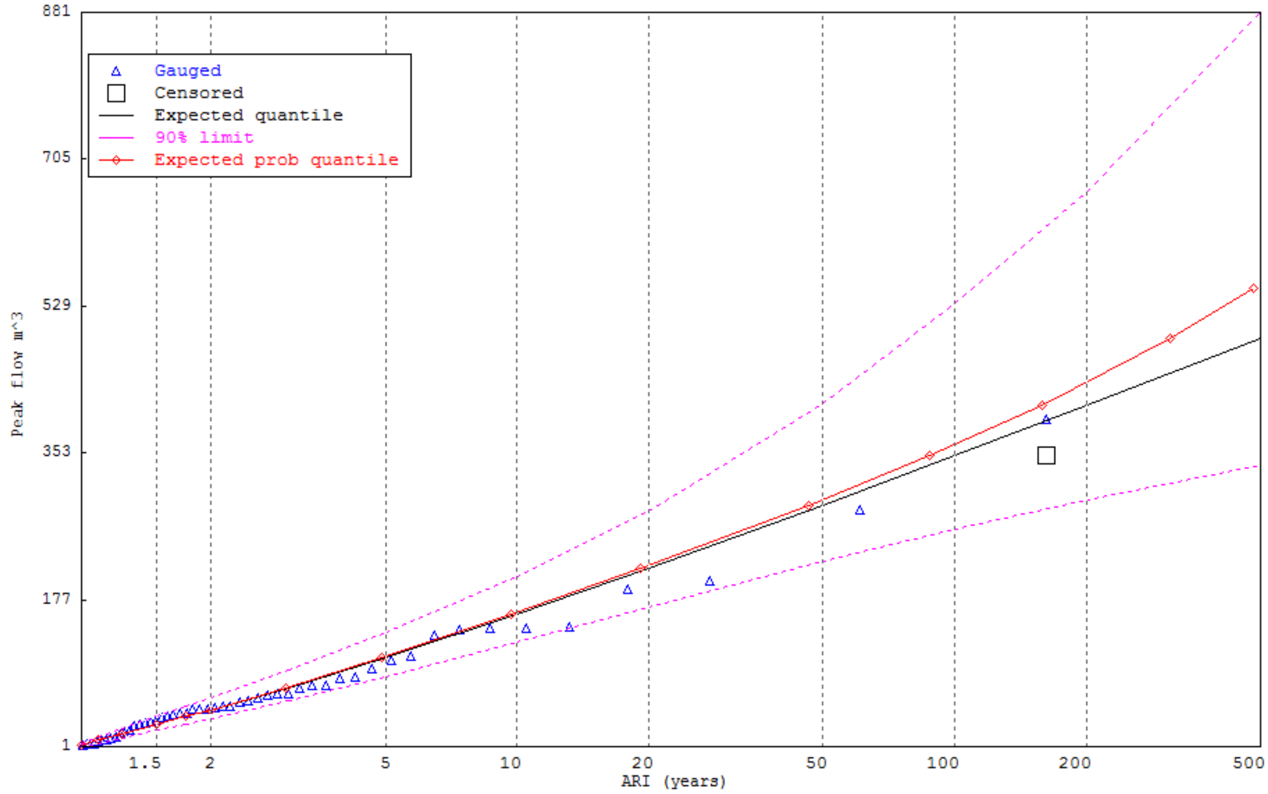
Stage 1 - Detailed Report

APPENDIX A

FLOOD FREQUENCY ANALYSIS

Cardno
Shaping the Future

Log Pearson Type III



Report created on 16/ 5/2013 at 15:04

FLIKE program version 4.50
FLIKE file version 3.00

Title:

Input Data for Flood Frequency Analysis for Model: Log Pearson III

Measurement Error Data

Group	Error coefficient of variation	Lower bound rated flow
1	0.000	0.00

Gauged Annual Maximum Discharge Data

Obs	Discharge	Year	Incremental error zone	Error coefficient of variation	Cunnane ARI,yrs*
1	393.00	1916	1	0.000	85.33
2	284.90	1993	1	0.000	32.00
3	198.90	1992	1	0.000	19.69
4	189.90	2010	1	0.000	14.22
5	145.00	1986	1	0.000	11.13
6	142.20	1984	1	0.000	9.14
7	142.00	1968	1	0.000	7.76
8	141.10	1975	1	0.000	6.74
9	133.80	1974	1	0.000	5.95
10	109.40	1981	1	0.000	5.33
11	104.60	1996	1	0.000	4.83
12	93.80	1995	1	0.000	4.41
13	84.10	1983	1	0.000	4.06
14	81.90	1988	1	0.000	3.76
15	74.50	1990	1	0.000	3.51
16	74.00	1973	1	0.000	3.28
17	71.30	2003	1	0.000	3.08
18	64.00	1964	1	0.000	2.91
19	63.50	1979	1	0.000	2.75
20	62.00	1966	1	0.000	2.61
21	58.80	1998	1	0.000	2.49

22	56.00	1971	1	0.000	2.37
23	53.40	1980	1	0.000	2.27
24	49.80	1987	1	0.000	2.17
25	49.10	1999	1	0.000	2.08
26	47.20	1991	1	0.000	2.00
27	46.00	1978	1	0.000	1.92
28	46.00	1965	1	0.000	1.86
29	45.00	1970	1	0.000	1.79
30	41.10	2000	1	0.000	1.73
31	40.30	2005	1	0.000	1.67
32	39.40	2011	1	0.000	1.62
33	38.00	1963	1	0.000	1.57
34	35.70	1989	1	0.000	1.52
35	31.80	2004	1	0.000	1.48
36	31.20	1985	1	0.000	1.44
37	29.20	1997	1	0.000	1.40
38	27.80	1977	1	0.000	1.36
39	25.50	2012	1	0.000	1.33
40	21.00	1969	1	0.000	1.29
41	17.00	2001	1	0.000	1.26
42	15.00	1967	1	0.000	1.23
43	11.70	2007	1	0.000	1.20
44	10.50	1976	1	0.000	1.17
45	9.20	1994	1	0.000	1.15
46	9.00	1972	1	0.000	1.12
47	7.10	2009	1	0.000	1.10
48	4.30	2002	1	0.000	1.08
49	4.20	2008	1	0.000	1.05
50	4.10	2006	1	0.000	1.03
51	2.30	1982	1	0.000	1.01

Note: Cunnane plotting position is based on gauged flows only

The following gauged flows were censored:

Obs	Discharge	Year
52	0.0	1951
53	0.0	1952
54	0.0	1954
55	0.0	1955
56	0.0	1953
57	0.0	1950
58	0.0	1958
59	0.0	1959
60	0.0	1960
61	0.0	1961
62	0.0	1962
63	0.0	1956
64	0.0	1957
65	0.0	1917
66	0.0	1918
67	0.0	1919
68	0.0	1920
69	0.0	1921
70	0.0	1922
71	0.0	1923
72	0.0	1924
73	0.0	1925
74	0.0	1926
75	0.0	1927
76	0.0	1928
77	0.0	1929
78	0.0	1930
79	0.0	1931
80	0.0	1932
81	0.0	1933
82	0.0	1934
83	0.0	1935
84	0.0	1936
85	0.0	1937
86	0.0	1938
87	0.0	1939
88	0.0	1940
89	0.0	1941
90	0.0	1942
91	0.0	1943
92	0.0	1944
93	0.0	1945

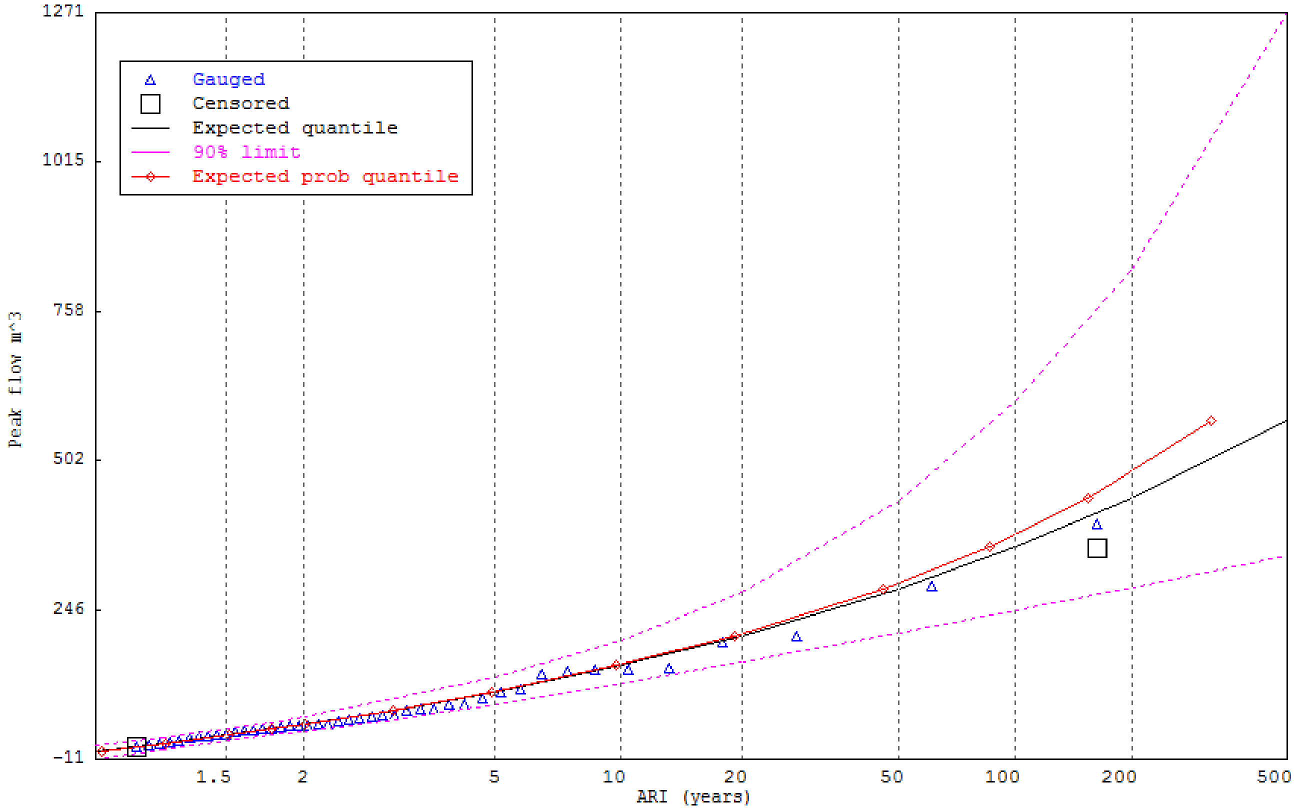
```
 94        0.0 1946
 95        0.0 1947
 96        0.0 1948
 97        0.0 1949


Censored Data
Obs   Threshold   Number of floods   Correlated   Error coefficient
                 Above     Below   error group       of variation
------------------------------------------------------------------
  1     350.00        0        46              1              0.000


-------------------------------------------------------------------------------
Posterior Parameter Results

Data file: Z:\Jobs\NA49913546_Euroa\Design_Analysis\Hydrology\FFA\FLIKE

Flood model: Log Pearson III

>>> Fitting algorithm: Global probabilistic search

Parameter   Lower bound   Upper bound
-------------------------------------
      1     -1.95535       9.41372
      2     -2.17427       2.43090
      3     -5.00000       5.00000

Incremental error model: Log-normal

Solution PROBABLY found in 2329 iterations

Maximized log-posterior density =   -267.317

No Parameter                    Initial value    Most probable value
--------------------------------------------------------------------
 1 Mean (loge flow)                 3.72919              3.71608
 2 loge [Std dev (loge flow)]         0.12831              0.10993
 3 Skew (loge flow)                -0.58984             -0.66458

--------------------------------------------------
Zero flow threshold:     0.0000
Number of gauged flows below flow threshold =    0

--------------------------------------------------
Parameter Moments based on Multi-normal Approximation to Posterior Distribution
No Most probable       Std dev       Correlation
--------------------------------------------------
 1      3.71608       0.18927  1.000
 2      0.10993       0.22900 -0.670  1.000
 3     -0.66458       0.49864  0.474 -0.901  1.000
--------------------------------------------------
Note: Parameters are roughly normally distributed.
      This approximation improves with sample size.


Summary of Posterior Moments from Importance Sampling
No         Mean       Std dev       Correlation
------------------------------------------------
 1      3.69386       0.16179  1.000
 2      0.13325       0.11213 -0.432  1.000
 3     -0.58241       0.26094 -0.065 -0.437  1.000
----------------
Note: Posterior expected parameters are the most
      accurate in the mean-squared-error sense.
      They should be used in preference to the most probable parameters


Upper bound =   2033.06

 Recurrence    Exp parameter     Monte Carlo 90% quantile Mean(log10(q)) Stdev(log10(q))
 interval      quantile            probability limits
    yrs
-------------------------------------------------------------------------------------
    1.010          1.74          0.61          3.71        0.2288        0.2473
    1.100          8.17          4.76         12.57        0.9091        0.1298
    1.250         16.10         10.98         22.74        1.2060        0.0973
```

1.500	27.10	19.88	36.78	1.4335	0.0818
1.750	36.54	27.47	48.46	1.5637	0.0761
2.000	44.89	34.18	59.21	1.6534	0.0732
3.000	71.37	55.43	92.81	1.8549	0.0685
5.000	107.04	83.98	137.98	2.0309	0.0658
10.000	158.94	125.11	204.78	2.2024	0.0658
20.000	214.01	167.17	282.32	2.3315	0.0706
50.000	290.28	222.08	411.59	2.4641	0.0835
100.000	349.70	260.65	531.65	2.5454	0.0968
200.000	409.97	296.27	665.80	2.6151	0.1120
500.000	490.10	338.46	881.22	2.6938	0.1337
1000.000	550.55	365.79	1078.95	2.7456	0.1508
2000.000	610.48	389.42	1289.22	2.7919	0.1682
5000.000	688.46	415.86	1621.50	2.8462	0.1913
10000.000	746.21	434.05	1911.00	2.8831	0.2087
20000.000	802.70	447.31	2216.08	2.9169	0.2260
50000.000	875.19	463.76	2681.82	2.9574	0.2486
100000.000	928.25	474.51	3084.20	2.9855	0.2654

Expected Probability Flood based on
Monte Carlo samples = 20000
Probability weight = 1.000
Scalng factor = 2.500

Flood magnitude	Expected probability	<----------ARI------------> yrs	95% limits	
1.74	0.01202	1.01	1.01	1.01
8.17	0.09253	1.10	1.10	1.10
16.10	0.20054	1.25	1.25	1.25
27.10	0.33287	1.50	1.50	1.50
36.54	0.42738	1.75	1.74	1.75
44.89	0.49823	1.99	1.99	2.00
71.37	0.66357	2.97	2.97	2.98
107.04	0.79639	4.91	4.90	4.93
158.94	0.89694	9.70	9.66	9.74
214.01	0.94777	19.15	19.04	19.25
290.28	0.97843	46.35	45.96	46.75
349.70	0.98861	87.76	86.76	88.78
409.97	0.99367	158.08	155.73	160.50
490.10	0.99678	311.02	304.90	317.40
550.55	0.99793	482.00	470.79	493.76
610.48	0.99859	707.97	689.03	727.98
688.46	0.99909	1099.51	1065.19	1136.12
746.21	0.99932	1469.72	1419.06	1524.15
802.70	0.99948	1904.89	1833.22	1982.40
875.19	0.99961	2578.56	2471.25	2695.61
928.25	0.99968	3159.74	3019.13	3314.08

Generalised Extreme Value



Report created on 16/ 5/2013 at 15:14

FLIKE program version 4.50
FLIKE file version 3.00

Title:

Input Data for Flood Frequency Analysis for Model: GEV

Measurement Error Data

Group	Error coefficient of variation	Lower bound rated flow
1	0.000	0.00

Gauged Annual Maximum Discharge Data

Obs	Discharge	Year	Incremental error zone	Error coefficient of variation	Cunnane ARI,yrs*
1	393.00	1916	1	0.000	73.67
2	284.90	1993	1	0.000	27.63
3	198.90	1992	1	0.000	17.00
4	189.90	2010	1	0.000	12.28
5	145.00	1986	1	0.000	9.61
6	142.20	1984	1	0.000	7.89
7	142.00	1968	1	0.000	6.70
8	141.10	1975	1	0.000	5.82
9	133.80	1974	1	0.000	5.14
10	109.40	1981	1	0.000	4.60
11	104.60	1996	1	0.000	4.17
12	93.80	1995	1	0.000	3.81
13	84.10	1983	1	0.000	3.51
14	81.90	1988	1	0.000	3.25
15	74.50	1990	1	0.000	3.03
16	74.00	1973	1	0.000	2.83
17	71.30	2003	1	0.000	2.66
18	64.00	1964	1	0.000	2.51
19	63.50	1979	1	0.000	2.38
20	62.00	1966	1	0.000	2.26
21	58.80	1998	1	0.000	2.15

22	56.00	1971	1	0.000	2.05
23	53.40	1980	1	0.000	1.96
24	49.80	1987	1	0.000	1.87
25	49.10	1999	1	0.000	1.80
26	47.20	1991	1	0.000	1.73
27	46.00	1978	1	0.000	1.60
28	46.00	1965	1	0.000	1.66
29	45.00	1970	1	0.000	1.55
30	41.10	2000	1	0.000	1.49
31	40.30	2005	1	0.000	1.44
32	39.40	2011	1	0.000	1.40
33	38.00	1963	1	0.000	1.36
34	35.70	1989	1	0.000	1.32
35	31.80	2004	1	0.000	1.28
36	31.20	1985	1	0.000	1.24
37	29.20	1997	1	0.000	1.21
38	27.80	1977	1	0.000	1.18
39	25.50	2012	1	0.000	1.15
40	21.00	1969	1	0.000	1.12
41	17.00	2001	1	0.000	1.09
42	15.00	1967	1	0.000	1.06
43	11.70	2007	1	0.000	1.04
44	10.50	1976	1	0.000	1.01

Note: Cunnane plotting position is based on gauged flows only

The following gauged flows were censored:

Obs	Discharge	Year
45	9.2	1994
46	9.0	1972
47	7.1	2009
48	4.3	2002
49	4.2	2008
50	4.1	2006
51	2.3	1982
52	0.0	1951
53	0.0	1952
54	0.0	1954
55	0.0	1955
56	0.0	1953
57	0.0	1950
58	0.0	1958
59	0.0	1959
60	0.0	1960
61	0.0	1961
62	0.0	1962
63	0.0	1956
64	0.0	1957
65	0.0	1917
66	0.0	1918
67	0.0	1919
68	0.0	1920
69	0.0	1921
70	0.0	1922
71	0.0	1923
72	0.0	1924
73	0.0	1925
74	0.0	1926
75	0.0	1927
76	0.0	1928
77	0.0	1929
78	0.0	1930
79	0.0	1931
80	0.0	1932
81	0.0	1933
82	0.0	1934
83	0.0	1935
84	0.0	1936
85	0.0	1937
86	0.0	1938
87	0.0	1939
88	0.0	1940
89	0.0	1941
90	0.0	1942
91	0.0	1943
92	0.0	1944
93	0.0	1945

94	0.0	1946
95	0.0	1947
96	0.0	1948
97	0.0	1949

Censored Data

Obs	Threshold	Number of floods Above	Number of floods Below	Correlated error group	Error coefficient of variation
1	350.00	0	46	1	0.000
2	10.00	0	7	1	0.000

Posterior Parameter Results

Data file: Z:\Jobs\NA49913546_Euroa\Design_Analysis\Hydrology\FFA\FLIKE

Flood model: GEV

>>> Fitting algorithm: Global probabilistic search

Parameter	Lower bound	Upper bound
1	-118.34203	207.63229
2	1.18165	5.78682
3	-2.50000	2.50000

Incremental error model: Log-normal

Solution PROBABLY found in 2240 iterations

Maximized log-posterior density = -250.983

No	Parameter	Initial value	Most probable value
1	Location u	44.64513	34.50626
2	loge (Scale a)	3.48423	3.64022
3	Shape k	-0.34356	-0.20643

Zero flow threshold: 0.0000

Number of gauged flows below flow threshold = 0

Parameter Moments based on Multi-normal Approximation to Posterior Distribution

No	Most probable	Std dev	Correlation		
1	34.50626	6.13108	1.000		
2	3.64022	0.17027	0.462	1.000	
3	-0.20643	0.12384	0.315	0.524	1.000

Note: Parameters are roughly normally distributed.
This approximation improves with sample size.

Summary of Posterior Moments from Importance Sampling

No	Mean	Std dev	Correlation		
1	34.00130	6.55129	1.000		
2	3.67248	0.17186	0.419	1.000	
3	-0.22651	0.12575	0.271	0.474	1.000

Note: Posterior expected parameters are the most
accurate in the mean-squared-error sense.
They should be used in preference to the most probable parameters

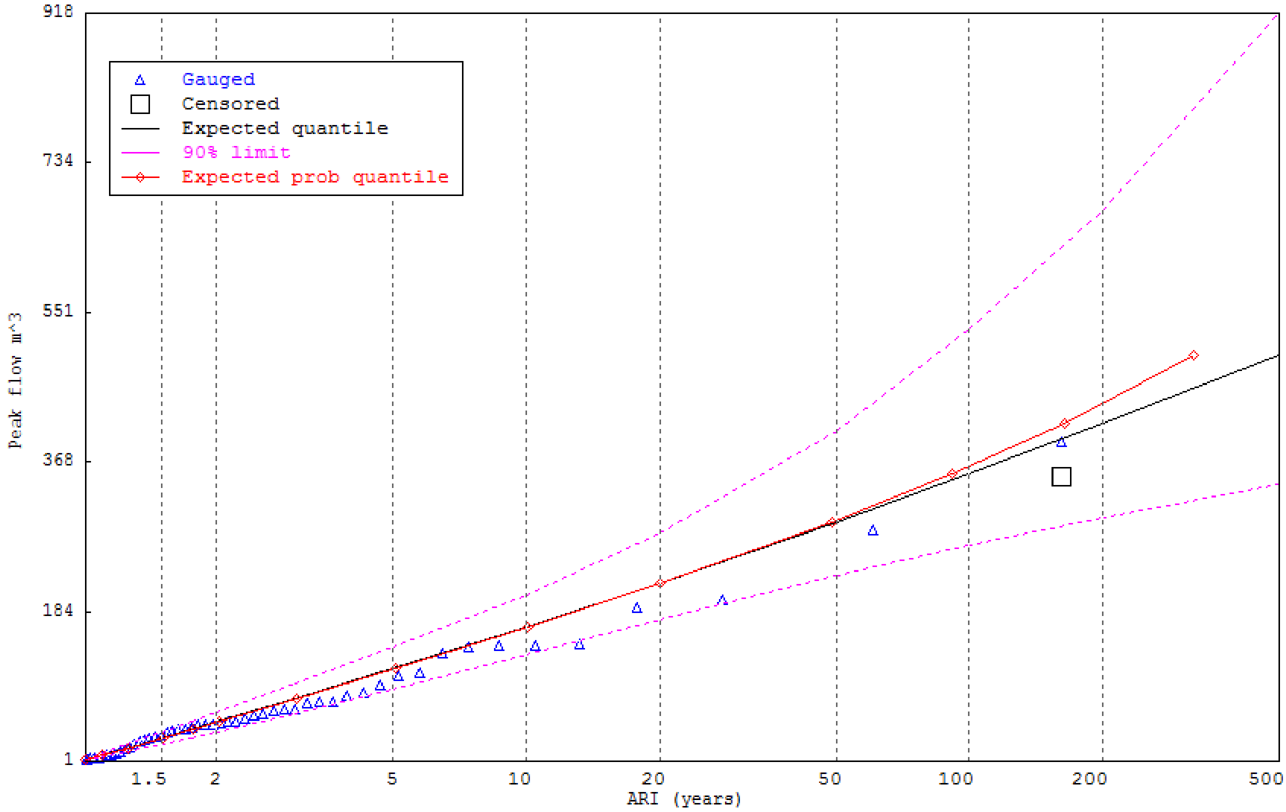
Lower bound = -139.718

Recurrence interval yrs	Exp parameter quantile	Monte Carlo 90% quantile probability limits		Mean(log10(q))	Stdev(log10(q))
1.100	2.78	-10.81	12.02	0.6490	0.4240
1.250	16.25	5.68	25.30	1.1699	0.2111
1.500	30.34	20.27	40.96	1.4712	0.0962
1.750	40.65	29.81	52.94	1.6033	0.0769
2.000	49.04	37.12	62.90	1.6867	0.0700
3.000	73.42	57.93	92.17	1.8643	0.0616
5.000	104.29	83.63	130.58	2.0181	0.0589
10.000	149.50	119.25	192.15	2.1760	0.0633
20.000	200.72	156.06	274.70	2.3054	0.0752
50.000	280.71	206.41	432.93	2.4533	0.0989
100.000	352.75	245.33	604.08	2.5543	0.1208
200.000	436.80	284.28	830.47	2.6490	0.1448
500.000	570.02	337.56	1270.83	2.7674	0.1792
1000.000	690.77	379.06	1757.12	2.8530	0.2066
2000.000	832.01	422.22	2407.77	2.9361	0.2350
5000.000	1056.19	479.73	3655.38	3.0427	0.2739
10000.000	1259.51	523.53	5017.94	3.1215	0.3042
20000.000	1497.39	569.04	6877.57	3.1989	0.3350
50000.000	1875.02	630.34	10377.46	3.2997	0.3767
100000.000	2217.53	679.32	14209.70	3.3748	0.4087

Expected Probability Flood based on
Monte Carlo samples = 20000
Probability weight = 1.000
Scalng factor = 2.500

Flood magnitude	Expected probability	<----------ARI------------> yrs	95% limits	
2.78	0.09504	1.11	1.10	1.11
16.25	0.20365	1.26	1.25	1.26
30.34	0.33690	1.51	1.51	1.51
40.65	0.43118	1.76	1.76	1.76
49.04	0.50151	2.01	2.00	2.01
73.42	0.66534	2.99	2.98	3.00
104.29	0.79735	4.93	4.92	4.95
149.50	0.89757	9.76	9.72	9.80
200.72	0.94800	19.23	19.12	19.34
280.71	0.97821	45.89	45.50	46.29
352.75	0.98832	85.63	84.69	86.60
436.80	0.99351	154.15	152.04	156.32
570.02	0.99685	317.13	311.58	322.88
690.77	0.99810	526.58	515.80	537.81
832.01	0.99882	849.28	829.28	870.26
1056.19	0.99935	1537.38	1494.69	1582.58
1259.51	0.99957	2349.33	2276.27	2427.23
1497.39	0.99972	3524.67	3402.90	3655.49
1875.02	0.99983	5880.55	5649.36	6131.46
2217.53	0.99988	8523.40	8156.21	8925.21

Generalised Pareto



Report created on 16/ 5/2013 at 15:09

FLIKE program version 4.50
FLIKE file version 3.00

Title:

Input Data for Flood Frequency Analysis for Model: Generalised Pareto

Measurement Error Data

Group	Error coefficient of variation	Lower bound rated flow
1	0.000	0.00

Gauged Annual Maximum Discharge Data

Obs	Discharge	Year	Incremental error zone	Error coefficient of variation	Cunnane ARI,yrs*
1	393.00	1916	1	0.000	85.33
2	284.90	1993	1	0.000	32.00
3	198.90	1992	1	0.000	19.69
4	189.90	2010	1	0.000	14.22
5	145.00	1986	1	0.000	11.13
6	142.20	1984	1	0.000	9.14
7	142.00	1968	1	0.000	7.76
8	141.10	1975	1	0.000	6.74
9	133.80	1974	1	0.000	5.95
10	109.40	1981	1	0.000	5.33
11	104.60	1996	1	0.000	4.83
12	93.80	1995	1	0.000	4.41
13	84.10	1983	1	0.000	4.06
14	81.90	1988	1	0.000	3.76
15	74.50	1990	1	0.000	3.51
16	74.00	1973	1	0.000	3.28
17	71.30	2003	1	0.000	3.08
18	64.00	1964	1	0.000	2.91
19	63.50	1979	1	0.000	2.75
20	62.00	1966	1	0.000	2.61

21	58.80	1998	1	0.000	2.49
22	56.00	1971	1	0.000	2.37
23	53.40	1980	1	0.000	2.27
24	49.80	1987	1	0.000	2.17
25	49.10	1999	1	0.000	2.08
26	47.20	1991	1	0.000	2.00
27	46.00	1978	1	0.000	1.92
28	46.00	1965	1	0.000	1.86
29	45.00	1970	1	0.000	1.79
30	41.10	2000	1	0.000	1.73
31	40.30	2005	1	0.000	1.67
32	39.40	2011	1	0.000	1.62
33	38.00	1963	1	0.000	1.57
34	35.70	1989	1	0.000	1.52
35	31.80	2004	1	0.000	1.48
36	31.20	1985	1	0.000	1.44
37	29.20	1997	1	0.000	1.40
38	27.80	1977	1	0.000	1.36
39	25.50	2012	1	0.000	1.33
40	21.00	1969	1	0.000	1.29
41	17.00	2001	1	0.000	1.26
42	15.00	1967	1	0.000	1.23
43	11.70	2007	1	0.000	1.20
44	10.50	1976	1	0.000	1.17
45	9.20	1994	1	0.000	1.15
46	9.00	1972	1	0.000	1.12
47	7.10	2009	1	0.000	1.10
48	4.30	2002	1	0.000	1.08
49	4.20	2008	1	0.000	1.05
50	4.10	2006	1	0.000	1.03
51	2.30	1982	1	0.000	1.01

Note: Cunnane plotting position is based on gauged flows only

The following gauged flows were censored:

Obs	Discharge	Year
52	0.0	1951
53	0.0	1952
54	0.0	1954
55	0.0	1955
56	0.0	1953
57	0.0	1950
58	0.0	1958
59	0.0	1959
60	0.0	1960
61	0.0	1961
62	0.0	1962
63	0.0	1956
64	0.0	1957
65	0.0	1917
66	0.0	1918
67	0.0	1919
68	0.0	1920
69	0.0	1921
70	0.0	1922
71	0.0	1923
72	0.0	1924
73	0.0	1925
74	0.0	1926
75	0.0	1927
76	0.0	1928
77	0.0	1929
78	0.0	1930
79	0.0	1931
80	0.0	1932
81	0.0	1933
82	0.0	1934
83	0.0	1935
84	0.0	1936
85	0.0	1937
86	0.0	1938
87	0.0	1939
88	0.0	1940
89	0.0	1941
90	0.0	1942
91	0.0	1943
92	0.0	1944

```
  93        0.0 1945
  94        0.0 1946
  95        0.0 1947
  96        0.0 1948
  97        0.0 1949


 Censored Data
 Obs   Threshold   Number of floods   Correlated   Error coefficient
               Above     Below   error group       of variation
 ------------------------------------------------------------------
   1     350.00       0       46           1            0.000


 ------------------------------------------------------------------------------
 Posterior Parameter Results

 Data file: Z:\Jobs\NA49913546_Euroa\Design_Analysis\Hydrology\FFA\FLIKE




 Flood model: Generalised Pareto

 >>> Fitting algorithm: Global probabilistic search

 Parameter   Lower bound   Upper bound
 ------------------------------------
        1       6.53265     653.26520
        2      -3.00000       3.00000
        3       0.00000       2.29900
 >>> PENALTIES were added to log-posterior density <<<

 Incremental error model: Log-normal

 Solution PROBABLY found in 2604 iterations

 Maximized log-posterior density =   -266.042

 No Parameter                   Initial value    Most probable value
 ---------------------------------------------------------------------
  1 Scale a                       65.32652             63.25356
  2 Shape k                       -0.05764             -0.03323
  3 Thresh                         0.49338              2.29900

--------------------------------------------------
Zero flow threshold:     0.0000
 Number of gauged flows below flow threshold =    0

--------------------------------------------------
 Parameter Moments based on Multi-normal Approximation to Posterior Distribution
 No Most probable       Std dev       Correlation
 --------------------------------------------------
  1     63.25356      14.36219  1.000
  2     -0.03323       0.15947  0.753  1.000
  3      2.29900       3.55330 -0.406 -0.252  1.000
 --------------------------------------------------
 Note: Parameters are roughly normally distributed.
      This approximation improves with sample size.


 Summary of Posterior Moments from Importance Sampling
 No         Mean       Std dev       Correlation
 ------------------------------------------------
  1     66.62479      12.46375  1.000
  2     -0.05760       0.12342  0.683  1.000
  3      1.50233       0.62084 -0.037 -0.031  1.000
 ----------------
 Note: Posterior expected parameters are the most
      accurate in the mean-squared-error sense.
      They should be used in preference to the most probable parameters


  Recurrence    Exp parameter     Monte Carlo 90% quantile Mean(log10(q)) Stdev(log10(q))
  interval      quantile            probability limits
     yrs
 -------------------------------------------------------------------------------------
```

1.010	2.17	0.93	2.96	0.3116	0.1556
1.100	7.87	5.80	9.98	0.8897	0.0731
1.250	16.47	12.09	21.06	1.2100	0.0720
1.500	28.83	21.32	36.68	1.4530	0.0710
1.750	39.39	29.46	49.82	1.5884	0.0694
2.000	48.62	36.54	61.43	1.6797	0.0679
3.000	77.06	59.07	96.08	1.8798	0.0634
5.000	113.86	88.89	140.39	2.0496	0.0595
10.000	165.55	130.74	204.89	2.2129	0.0590
20.000	219.35	173.53	280.59	2.3360	0.0650
50.000	293.84	227.50	405.48	2.4647	0.0810
100.000	352.87	265.26	529.90	2.5458	0.0969
200.000	414.30	299.91	675.29	2.6173	0.1151
500.000	499.36	340.58	917.69	2.7012	0.1412
1000.000	566.75	367.37	1154.67	2.7585	0.1621
2000.000	636.89	395.25	1448.26	2.8117	0.1838
5000.000	734.02	426.23	1946.93	2.8771	0.2134
10000.000	810.97	445.18	2399.98	2.9234	0.2364
20000.000	891.06	466.21	2964.77	2.9675	0.2599
50000.000	1001.97	488.22	3931.92	3.0230	0.2914
100000.000	1089.84	503.72	4873.92	3.0632	0.3157

Expected Probability Flood based on
Monte Carlo samples = 20000
Probability weight = 1.000
Scalng factor = 2.500

Flood magnitude	Expected probability	<----------ARI------------> yrs	95% limits	
2.17	0.01025	1.01	1.01	1.01
7.87	0.09362	1.10	1.10	1.10
16.47	0.20492	1.26	1.26	1.26
28.83	0.33944	1.51	1.51	1.52
39.39	0.43467	1.77	1.77	1.77
48.62	0.50571	2.02	2.02	2.03
77.06	0.67061	3.04	3.03	3.04
113.86	0.80216	5.05	5.04	5.07
165.55	0.90094	10.09	10.05	10.14
219.35	0.95025	20.10	19.99	20.21
293.84	0.97950	48.79	48.37	49.21
352.87	0.98909	91.65	90.59	92.74
414.30	0.99388	163.48	161.04	165.99
499.36	0.99689	321.78	315.50	328.31
566.75	0.99802	505.26	493.62	517.47
636.89	0.99868	758.59	738.38	779.93
734.02	0.99918	1225.42	1186.96	1266.46
810.97	0.99941	1698.31	1638.91	1762.18
891.06	0.99956	2292.60	2204.24	2388.34
1001.97	0.99970	3294.69	3152.32	3450.53
1089.84	0.99976	4241.06	4042.97	4459.57

Stage 1 - Detailed Report

APPENDIX B
PMF WORKSHEETS

Cardno®
Shaping the Future

Seven Creeks PMF

WORKSHEET 2: Generalised Southeast Australia Storm Method (GSAM)

LOCATION INFORMATION	
Catchment Name: Seven Creeks	State: Vic
GSAM zone: Inland GSAM	Area: 332 km^2

CATCHMENT FACTORS

Topographical Adjustment Factor **TAF** = 1.615 (1.0 – 2.0)

Annual Moisture Adjustment Factor

$$MAF = \frac{EPW_{seasonal\ catchment\ average}}{EPW_{seasonal\ standard}}$$

Season	$EPW_{seasonal\ catchment\ average}$	$EPW_{seasonal\ standard}$	MAF
Summer (Annual)	60.1	80.80	0.744 (0.60 - 1.05)
Autumn	48.98	71.00	0.690 (0.56 - 0.91)

Summer PMP values (mm)			Autumn PMP values (mm)		
Duration (hours)	Initial Depth (D_{summer})	PMP Estimate (D_sxTAFxMAF$_s$)	Duration (hours)	Initial Depth (D_{autumn})	PMP Estimate (D_axTAFxMAF$_a$)
24	880.9	880.9	24	516	575.2
36	984.7	984.7	36	646	720.2
48	1041.0	1041.0	48	763	849.6
72	1101.5	1101.5	72	963	1072.4
96	1155.7	1155.7	96	1033	1150.7

Final GSAM PMP Estimates

Duration (hours)	Maximum of the Seasonal Depths	Preliminary PMP Estimate (nearest 10mm)	Final PMP Estimate (from envelope)
1	Where applicable, calculate GSDM depths (Bureau of Meteorology, 2003)	180	180
2		260	260
3		310	310
4		360	360
5		380	390
6		410	430
12	(no preliminary estimates available)		650
24	880.9	880	880
36	984.7	980	980
48	1041.0	1040	1040
72	1101.5	1100	1110
96	1155.7	1160	1160

GSDM WORKSHEET

LOCATION INFORMATION

Catchment: Seven Cks State: Victoria

Duration Limit: ……………………....... (3-6) hours Area: ………………………….. km5

Approx. Centroid: Latitude: …………….° S Longitude: ………...……..° E

Portion of Area Considered:

Smooth, **S** = 30% (0.0 - 1.0) Rough, **R** = 70%.(0.0 -1.0)

ELEVATION ADJUSTMENT FACTOR (EAF)

Mean Elevation: ………N/A………..…. m required if greater than 1,500 m

Adjustment for Elevation: …N/A……… -0.05 per 300m above 1500 m

EAF = 1.0. (0.85 - 1.00)

GSDM MOISTURE ADJUSTMENT FACTOR (MAF)

$EPW_{catchment}$= ……………………… GSDM MAF=$EPW_{catchment}$/104.5

OR

read directly off GSDM Moisture Adjustment Factor chart at centroid

GSDM MAF = ……0.58…………..(0.46-1.19)

PMP VALUES (mm)

Duration (hours)	Initial Depth - Smooth (D_S)	Initial Depth - Rough (D_R)	PMP Estimate = (D_SHS + D_RHR) H MAF H EAF	Rounded PMP Estimate (nearest 10 mm)
0.25	138	138	80	80
0.50	200	200	116	120
0.75	255	255	148	150
1.0	310	310	180	180
1.5	405	355	226	230
2.0	463	405	258	260
2.5	525	442	290	290
3.0	570	475	314	310
4.0	645	535	355	360
5.0	692	580	382	380
6.0	743	617	409	410

Castle Creek PMF

WORKSHEET 2: Generalised Southeast Australia Storm Method (GSAM)

LOCATION INFORMATION	
Catchment Name: Castle Creek	State: Victoria
GSAM zone: Inland Zone	Area: 153 km^2

CATCHMENT FACTORS

Topographical Adjustment Factor **TAF** = 1.4057 (1.0 – 2.0)

Annual Moisture Adjustment Factor $$MAF = \frac{EPW_{seasonal\ catchment\ average}}{EPW_{seasonal\ standard}}$$

Season	$EPW_{seasonal\ catchment\ average}$	$EPW_{seasonal\ standard}$	MAF
Summer (Annual)	60.02	80.80	0.743 (0.60 - 1.05)
Autumn	48.93	71.00	0.689 (0.56 - 0.91)

Summer PMP values (mm)			Autumn PMP values (mm)		
Duration (hours)	Initial Depth (D_{summer})	PMP Estimate (D_SxTAFxMAF_S)	Duration (hours)	Initial Depth (D_{autumn})	PMP Estimate (D_AxTAFxMAF_A)
24	780	814.4	24	534	517.0
36	868	906.4	36	663	642.2
48	919	959.9	48	781	756.5
72	965	1008.1	72	990	959.3
96	1005	1049.7	96	1062	1028.7

Final GSAM PMP Estimates

Duration (hours)	Maximum of the Seasonal Depths	Preliminary PMP Estimate (nearest 10mm)	Final PMP Estimate (from envelope)
1	Where applicable, calculate GSDM depths (Bureau of Meteorology, 2003)	210	210
2		270	270
3		310	310
4		340	340
5		370	370
6		400	400
12	(no preliminary estimates available)		600
24	814.4	810	810
36	906.4	910	910
48	959.9	960	960
72	1008.1	1010	1010
96	1049.7	1050	1050

GSDM WORKSHEET

LOCATION INFORMATION

Catchment : Castle Creek State: Victoria

Duration Limit: ......3-6hrs................... (3-6) hours Area: ...153......................... km5

Approx. Centroid: Latitude: ....° S Longitude: ....° E

Portion of Area Considered:

Smooth, **S** = 100% (0.0 - 1.0) Rough, **R** = 0% (0.0 -1.0)

ELEVATION ADJUSTMENT FACTOR (EAF)

Mean Elevation: ....N/A.............. m required **if** greater than 1,500 m

Adjustment for Elevation: ...N/A........ -0.05 per 300m above 1500 m

EAF = 1.0. (0.85 - 1.00)

GSDM MOISTURE ADJUSTMENT FACTOR (MAF)

$EPW_{catchment}$= .... GSDM MAF=$EPW_{catchment}$/104.5

OR read directly off GSDM Moisture Adjustment Factor chart at centroid

GSDM MAF = ...0.58............ (0.46-1.19)

PMP VALUES (mm)

Duration (hours)	Initial Depth - Smooth (D_S)	Initial Depth - Rough (D_R)	PMP Estimate = (D_SHS + D_RHR) H MAF H EAF	Rounded PMP Estimate (nearest 10 mm)
0.25	155	N/A	90	90
0.50	230	N/A	133	130
0.75	292	N/A	169	170
1.0	355	N/A	206	210
1.5	405	N/A	235	240
2.0	460	N/A	267	270
2.5	492	N/A	285	290
3.0	528	N/A	306	310
4.0	590	N/A	342	340
5.0	640	N/A	371	370
6.0	682	N/A	396	400

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