# Hamilton Flood Investigation

RM2238 v1.0 Final

Prepared for the Glenelg Hopkins CMA

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# EXECUTIVE SUMMARY

The primary aim of the Hamilton Flood Investigation was to undertake definitive flood investigations for Hamilton and to undertake a comprehensive analysis with all available data to determine a robust 1% Annual Exceedence Probability (AEP) flood extent for the flood plains of the Grange Burn and other minor tributaries in and around Hamilton. The study area for this project is shown in Figure 1.1.

## Key Deliverables

The primary outcomes from the Flood investigation included:

- Report summarising the Hamilton Flood Investigation;
- Digital floodplain maps showing both floodplain and floodway areas;
- Economic damage assessment;
- Mitigation option assessment and risk assessment;
- Victoria Flood Data (VFD) compliant datasets;
- Draft Planning Scheme Amendment documentation; and
- Municipal Emergency Management Plan Appendices.

## Hydrology

For the study area is was evident from the rainfall and streamflow analysis that antecedent conditions within the catchment play an important role in the translation of rainfall to runoff with significant differences in loss rates for rainfall events between the wet and dry periods. The hydrological assessment for the Hamilton catchment was restricted by the limited availability of streamflow data with only four years of streamflow record upstream of hamilton. The gauges within the system are summarised in Table i.

Table I Available streamflow gauges

| Gauge No. | Gauge Name | Area | Start Date | End Date |
|---|---|---|---|---|
| 238239 | Grange Burn at Hamilton | 222 $km^2$ | May -1981 | Apr-1985 |
| 238219 | Grange Burn at Morgiana | 964 $km^2$ | Jul -1963 | Present |

Flood frequency assessment was undertaken on both gauges to determine the peak flow rates associated with the design Annual Exceedence Probabilities (AEP). The design events were simulated using the method specified in Australian Rainfall and runoff (AR&R, 1987) and using the rainfall runoff program RORB. The Probable Maximum Flood (PMF) was developed using the Generalised Southeast Australia Method (GSAM) in accordance with the Bureau of Meteorology (BoM, 2003). The peak design events are summarised in Table ii.

Table ii Design peak flow rates for the Hamilton Investigation

| AEP (%) | Design Flow Rates | | | |
|---|---|---|---|---|
| | Grange Burn at Hamilton | Petschels Lane Tributary[1] | Marshalls Road Tributary[1] | Kennys Road Tributary[1] |
| 20% | 35.7 | 4.9 | 2.8 | 1.8 |
| 10% | 57.1 | 7.8 | 4.0 | 2.5 |
| 5% | 96.8 | 10.8 | 5.7 | 3.4 |
| 2% | 153.2 | 15.8 | 8.0 | 5.0 |
| 1% | 200.5 | 20.7 | 10.1 | 6.3 |
| 0.5% | 241.0 | 25.9 | 12.4 | 7.7 |
| 0.2% | 314.7 | 33.5 | 15.8 | 9.8 |
| PMF | 2,266 | 215.7 | 91.8 | 61.6 |

[1] Flows derived at RORB model outlets, these will be distributed within the hydraulic model.

Climate change was assessed with a 10%, 20% and 32% increase in rainfall intensity explored. Under the 32% increase scenario for the Grange Burn, the 1% AEP flood event increased to be greater than the 0.2% AEP event (an increase of 89%). It was observed that the percentage increase of peak flow rates for the smaller, more frequent events was greater than for the rarer more extreme events. For example, for the Grange Burn catchment the 20% AEP event is expected to increase by 229%, whereas the 0.2% AEP event is expected to increase by 81%.

### Hydraulic Modelling

The hydraulic modelling for the project was undertaken using the WL|Delft 1D2D modelling system, SOBEK. Three models were developed to represent the study area (see Section 5 for details). Structures within the model are represented using 1D model elements and the topography was represented using a 5m x 5m grid. Within the Grange Burn system there is one major active storage, Lake Hamilton. This acts as a control structure upstream of the township of Lake Hamilton. Lake Hamilton was previously assessed in the *Report on the Lake Hamilton Spillway / Grange Burn Flooding Investigations* (GHD, 1987) which identified that the Lake Hamilton spillway was undersized.

One difference between the GHD (1987) report and the current topography was the height of the Lake Hamilton embankment. The embankment in the GHD report was assumed to be 180 mAHD, whereas the embankment within the current model was found to be within 179.5 and 179.75 mAHD. It should be noted that the maximum elevations for the embankment were extracted from 1 m LiDAR elevation data sets to ensure the top of the embankment was accurately captured. This difference of between 300 and 500 mm between the GHD report and the current LiDAR implies that the dam wall may be overtopped in the current design runs sooner than GHD predicted in the 1987 report. It is recommended that the dam wall of Lake Hamilton be surveyed in detail and an assessment completed on the appropriate sizing of the spillway to meet large storage requirements.

The hydraulic model was calibrated to the 1983 and 2010 flood events. Overall the model was well calibrated and validated to these events. The hydraulic model was used to assess the 20%, 10%, 5%, 2%, 1%, 0.5% AEP and PMF flood events. The results of these model runs are summarised in Section 5.6.

Sensitivity analysis was undertaken to assess the variability in flood extent and depth due to a number of parameters. The purpose of the sensitivity was to demonstrate the variability of the model results to critical input parameters and to provide some guidance to the importance of each parameter. The sensitivity assessment examined:

- Hydrology sensitivity – tested through varying the hygrologic loss rates.
- ‘Low’ and ‘High’ roughness – this was achieved through decreasing and increasing the manning’s roughness by +/- 20% respectively.
- Individual buildings included in the roughness – this assessment modified the approach to roughness from a lumped roughness approach for properties and buildings to a method which delineated the buildings and reduced the property roughness accordingly.
- Climate change assessment for the 32% increase in rainfall intensity.

### Planning

The recommended flood controls to be put in place are a FO and LSIO. The method of deriving the FO was to use the 10% AEP extent. The LSIO included all areas inside the 1% AEP flood extent that are not covered by the final FO shape. It is recommended that the area within Model C covered by the PPRZ (Public Park and Recreation Zone) be excluded from the FO and included in the LSIO as this area already has planning restrictions and is not intended for development. This section of the model is also impacted by the man-made channel to the old Reservoir which has not been accurately surveyed and included within the model in detail. Planning Amendment documentation has been prepared in conjunction with this investigation.

## Economic Damage Assessment

An economic damage assessment was undertaken which included an assessment of the 20%, 10%, 5%, 2%, 1%, 0.5% AEP and PMF flood events. Buildings within the 1% AEP had their floor levels surveyed in order to assess the damages. Rating curves were developed using the Department of Culture and heritage (DECCW) damage curves adapted to Hamilton. Damages were estimated based on building, property and road damage.

The calculated Annual Average Damage (AAD) for Hamilton was $ 208,912 per annum. Details of the number of properties inundated and buildings with overfloor flooding is summarised in Section 7.3.

## Mitigation Option Assessment

High risk flood areas were highlighted at the area of Holden Street and Apex park for the Grange Burn and on King Street near Coleraine Road on Marshalls Road Tributary. The mitigation options were proposed and assessed included:

- Option A1 - Levees within King Street park
- Option A2 - Additional culverts under Coleraine Road at King Street Park
- Option B1 - Levee upstream of Ballarat Road (west side of the Grange Burn).
- Option B2 - Upgrading Apex Park Road to act as a raised road levee bank.
- Option B3 - Extending a levee from the Apex Park Road upgrade to Mt Napier Road (west side of Grange Burn)
- Option B4 - Removing the existing pedestrian bridge (at Apex Park)

The cost / benefit assessment indicated the payback periods as specified in Table iii.

Table iii Cost / Benefit analysis results

| Model Run | Mitigation option applied | AAD (restricted to 0.5% AEP) | Reduction in AAD ($) | Option Estimated Cost ($ 2012) | Payback Period (years) |
|---|---|---|---|---|---|
| *Existing* | *Existing* | *$ 183,772* | | | |
| 1 | A1, A2 | $ 107,753 | $ 76,019 | $ 928,000 | 12 |
| 2 | B1, B2, B3, B4 | $ 141,195 | $ 42,577 | $ 1,152,000 | 27 |
| All | A1, A2, B1, B2, B3, B4 | $ 65,100 | $ 118,672 | $ 2,080,000 | 18 |

## Recommendations

Following this study the following actions are recommended:

- Implement a stream flow monitoring upstream of Lake Hamilton at the old Robsons Road gauge location for the purpose of additional flood warning and for use in future flood studies.
- Possibly develop a temporary (or permanent) gauge location that could be used for periods where large rainfall events are expected at Tarrington-Strathkellar Road to give additional warning times.
- Develop a gauge within Hamilton, possible location includes at Portland Road. This would allow verification of the peak flows during large events within Hamilton excluding the influence of Lake Hamilton.
- Undertake Community awareness programs to highlight the information generated within this study to the community to improve flood awareness within the community.
- Consider undertaking a dam break assessment on Lake Hamilton as this was identified as being undersized compared to the original design specification based on the revised hydrology.
- Implement the flood overlays as suggested in this study for future planning control within the catchment. Incorporate the flood overlays into the Council's future development plans.
- Consider implementing detailed assessments of the mitigation options for development (if these are to be developed in the future via funding).
- Flood maps as generated by this project should be made available to emergency response agencies to assist with the response within Hamilton.
- Ensuring that flood information such as inundated properties, peak flood heights, timing of flood events, flood depths etc are captured post each event for future studies.
- Implementing a plviograph station within the Hamilton catchment would assist future flood investigations as this would aid the calibration of hydrologic models within the catchment. This gauge could be located within Hamilton or upstream within the catchment.

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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. |
| 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. |
| 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. |
| 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. |
| 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. |
| 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. |

# 1 ESTABLISH CONTEXT

The primary aim of the Hamilton Flood Investigation is to undertake definitive flood investigations for Hamilton and to undertake a comprehensive analysis with all available data to determine a robust 1% Annual Exceedence Probability (AEP) flood extent for the flood plains of the Grange Burn and other minor tributaries in and around Hamilton.

## 1.1 Project Objectives

The project objectives include:

(a) Hydrological assessment for the catchment draining to Hamilton using all available meteorological, topographical, geological, soils and other relevant data;
(b) Assess the robust probabilities of historic flood events including 1983 and 1946;
(c) Develop the Flood Frequency Assessment (FFA) to determine the full range of design flood events for the 20%, 10%, 5%, 2%, 1% and 0.5% AEP flood flows;
(d) Identify and define any additional survey data requirements;
(e) Produce a Digital Terrain Model (DTM) of suitable quality;
(f) Produce and calibrate a hydraulic model against at least 3 historic flood events, including the flood of 1946;
(g) Determine design flood levels and extents;
(h) Perform sensitivity analysis on all hydraulic modelling results;
(i) Produce flood planning maps based on sound rationale;
(j) Develop flood intelligence for updating the Municipal Emergency Management Plan (MEMP) Flood Sub-Plan.

The outputs at each stage of the project will include:

- Summary of literature review and consultation undertaken;
- Stage reports at each hold point of the project;
- Draft report summarising the Hamilton Flood Investigations;
- Final report summarising the Hamilton Flood Investigations;
- Digital floodplain maps showing both floodplain and floodway areas;
- All model files, data and outputs pertaining to the Hamilton Flood Investigations, this includes fully attributed Victoria Flood Data (VFD) compliant datasets;
- Draft Planning Scheme Amendment documentation, this includes the following as required:
  - Municipal Strategic Statement (MSS) amendments
  - local policies
  - zoning amendments
  - overlay amendments
    - land subject to inundation overlay
    - floodway overlay
  - associated mapping
  - Relevant flood intelligence for inclusion in the Flood Sub-Plan

## 1.2 Study Area

The Study Area is shown in Figure 1.1 and covers approximately 26.3 $km^2$. It includes the majority of the residential and commercial areas of Hamilton. The area is bounded by Beveridges Road to the North; West Boundary Road to the West; Petschel's Lane to the South; and Kutzes Road and Robsons Road to the East. The study area incorporates the reach of the Grange Burn between the stream gauging station at Robsons Road down to West boundary Road, as well as a number of smaller tributaries. These tributaries join the Grange Burn downstream of the study area.

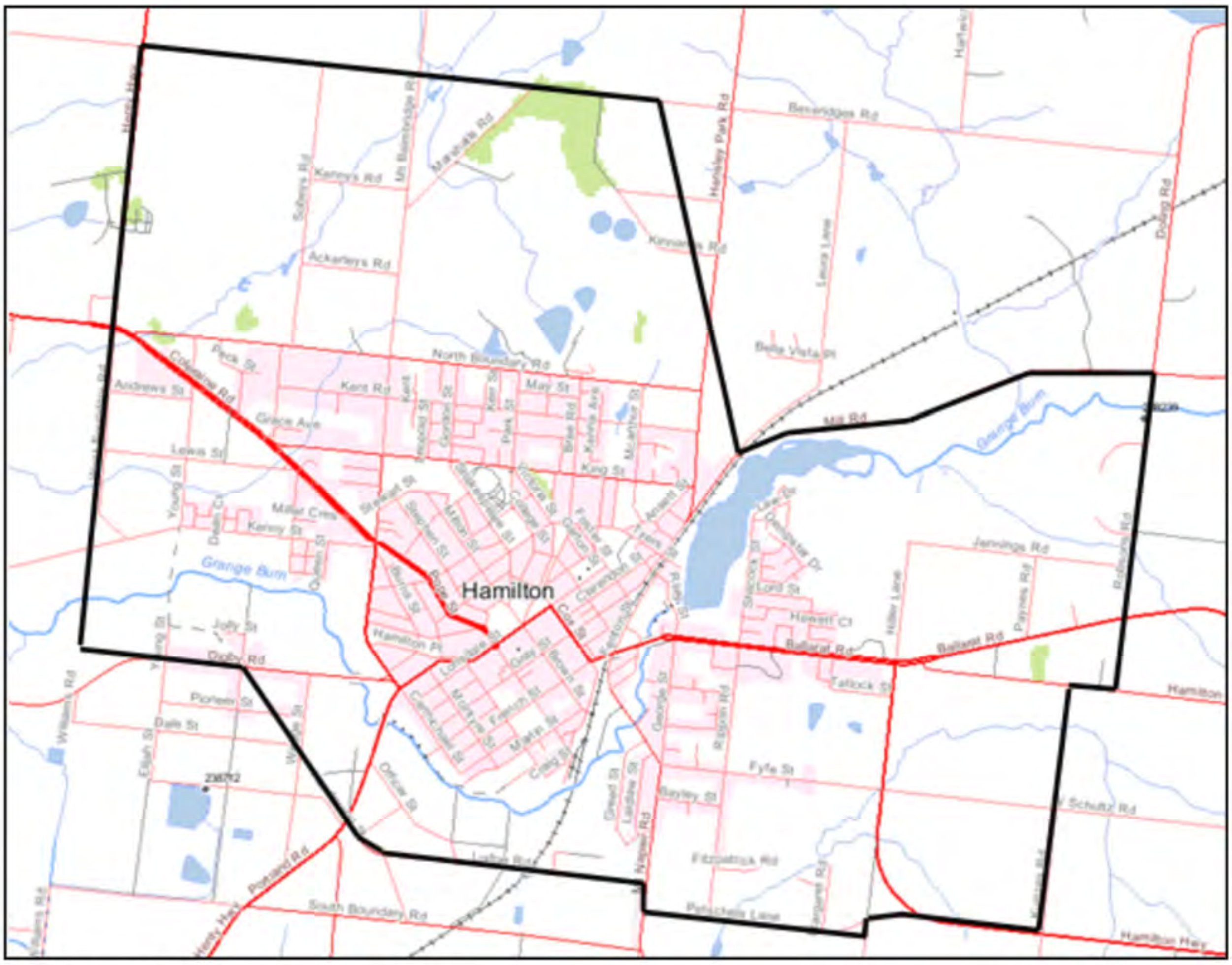



Figure 1.1 Study Area

## 1.3 Scope of Works

In accordance with the current risk management procedures as set out in the Australian/New Zealand Standard AS/NZS 4360:2004 Risk Management, Cardno have implemented the following key risk management steps interpreted for the flood management task:

- Establish context,
- Identify risks from key sections,
- Provide Datasets and Mapping,
- Analyse and advise on treatment of risks.

For further information on the risk management process as applied in flood management, Cardno have consulted the Emergency Risk Management – Application Guide (EMA, 2000).

In this context, Cardno have provided the following services:

- Undertaken a comprehensive review of the relevant policies and guidelines and ensure that our approach is guided by them.
- Researched and documented existing available information of relevance to the study as part of a Gap Analysis in consultation with the Project Manager. This step identified the key data required to develop robust hydrological and hydraulic models. Include any information provided by the community to give context for local flood events for model calibration.
- Collated existing aerial and ground surveys, and undertaken additional ground / river survey as required to provide a detailed database to facilitate the preparation of a calibrated hydraulic model and flood inundation maps that meet the study requirements.
- Specified, set-up, calibrated and validated a suitable hydrologic model for application to the study area. The model was subject to appropriate sensitivity analysis, design flood hydrographs were produced.
- Specified, set-up, calibrated and validated a suitable hydraulic model for application to the study area. The model was subjected to appropriate sensitivity analysis, flood levels and extents have been determined, and floodway areas were also delineated. Cardno has liaised closely with the Project Manager throughout model development, calibration and validation.
- Delivered all flood related information collected and developed through the study as fully attributed VFD compliant datasets in MapInfo / ArcGIS Version 9.* format ready for upload to the Master Datasets. The deliverables followed the VFD update protocols and have been reviewed through the QA checks of Cardno. Cardno advised DSE's Floodplain Management Unit (FMU) through the Technical Steering Committee of the datasets to be prepared for VFD upload and have liaised with DSE's VFD Data Manager on appropriate attribute values.
- Undertaken a Flood Risk Assessment for the study area. This involved developing a flood damage assessment model to determine flood damage potential at varying flood intervals, socio-economic benefits and costs, and environmental impacts. Appropriate land use planning and building controls have been discussed, as well as the potential need for and benefits of a flood warning system. Cardno investigated potential Mitigation works and provide outputs to the MEMP Flood sub-plan.

# 2 COMMUNITY CONSULTATION

Community consultation forms an integral part of this flood investigation with the community holding important information regarding past flood events and also being the main stakeholder impacted by the results from the flood investigation. This flood investigation not only examines the development of the Planning Scheme Amendment documentation, but also critically examines the emergency management procedures and emergency response information that protect the community during large flood events. This information must be communicated to the community and developed with their involvement.

As part of this flood investigation Cardno has liaised with the community to capture and utilise the information that is made available. The community consultation session was held was during Sheepvention (2011) in conjunction with the GHCMA display. During this 2 day consultation session Cardno spoke with numerous members of the community and captured information regarding historic peak flood heights and extents. Photography was gathered from the community of some peak flows and flood events that was used in the calibration and verification of the hydraulic models. A summary of the photographs obtained is summarised in Table 2.1. In addition to these photographs Cardno captured numerous images of the sub-catchment via a site inspection.

All photos from the Glenelg Hopkins CMA and from Roger Thompson (community member) are shown in Appendix D.

Table 2.1 Photograhy obtained for the flood investigation

| Year of Flood | Data Type | Source | Number |
|---|---|---|---|
| August 1983 | Flood photographs | GHCMA | 2 |
| 2003 | Flood photographs | GHCMA | 11 |
| August 2004 | Flood photographs | GHCMA | 14 |
| August 2004 | Aerial flood photographs | GHCMA | 8 |
| August 2004 | Flood photographs | Roger Thompson | 28 |

A second community consultation meeting was held on the Wednesday 2$^{nd}$ May. This session was aimed at allowing the community to view the preliminary flood extents and to ask any questions or comments. VicSES attended the meeting to present Floodsafe information to the community.

# 3 SURVEY AND MAPPING

In order to develop the models to represent the area at Hamilton detailed survey and mapping was required. This data was sourced from a number of agencies and was of varying degrees of accuracy. This section outlines the available data, the sources of this data and the validation of all data used in the study. The following sections outline the data and approach adopted:

- LiDAR data sets
  - LiDAR data Validation
- Historic flood event heights
- Structure and culverts information.

## 3.1 LiDAR Data

GHCMA has provided Cardno with the 2009 LiDAR data of the modelling area, however the data was not sufficient to cover the entire modelling area. The extent of this LiDAR data is summarised in Figure 3.1. The 2009 LiDAR data was developed as part of the DSE River Health Index of Stream Conditions (ISC) LiDAR data capture and was specified as having a vertical accuracy of +/- 0.10 m.

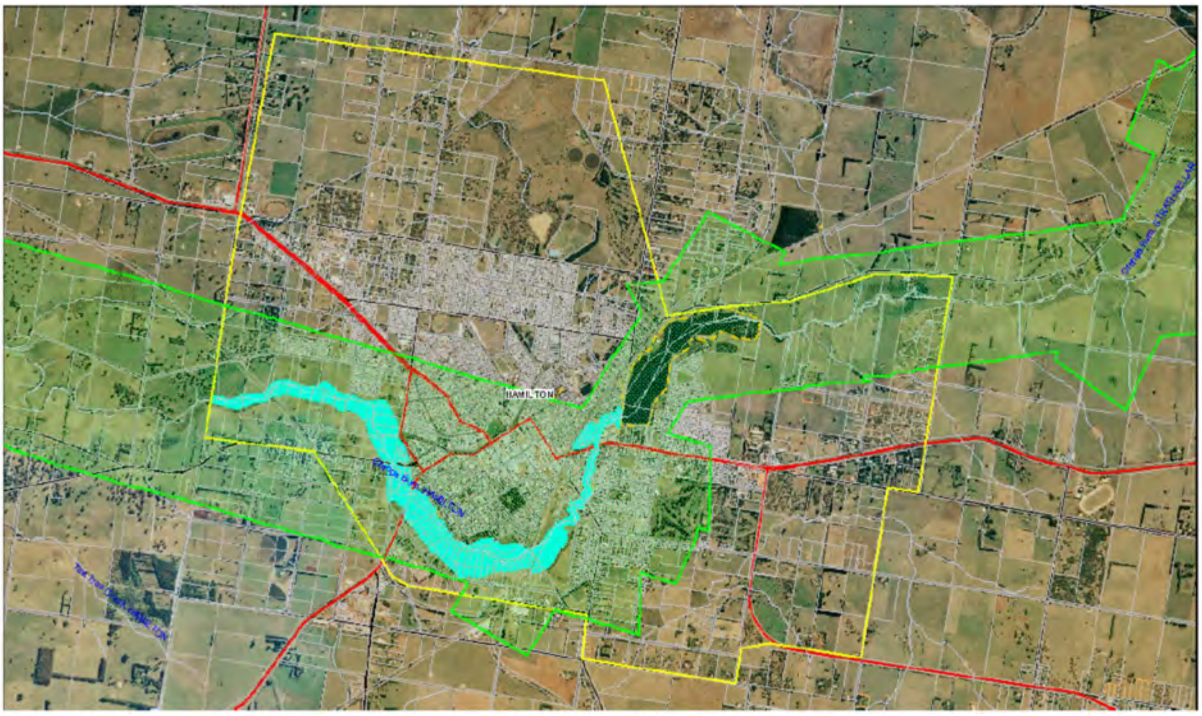



Figure 3.1 2009 LiDAR data extent for Hamilton

In order to undertake the study, LiDAR data was required to cover the full study area. AAM Group was commissioned by the Department of Sustainability and Environment to conduct additional LiDAR survey over the full study area. This set of data was captured on the 14th September 2011. The supplied data had a vertical accuracy of +/- 0.10 m to one sigma and was supplied in GDA94 (MGA Zone 54) projection.

### *3.1.1 LiDAR Data Validation*

As a first pass of validation the 2009 and 2011 LiDAR datasets were overlayed and compared directly with each other. From this comparison there was a visible constant shift in the 2009 LiDAR data as compared to the 2011 LiDAR data with the 2011 LiDAR data set being approximately 300 mm lower through the entire data set. An example of the comparison is shown in Figure 3.2.

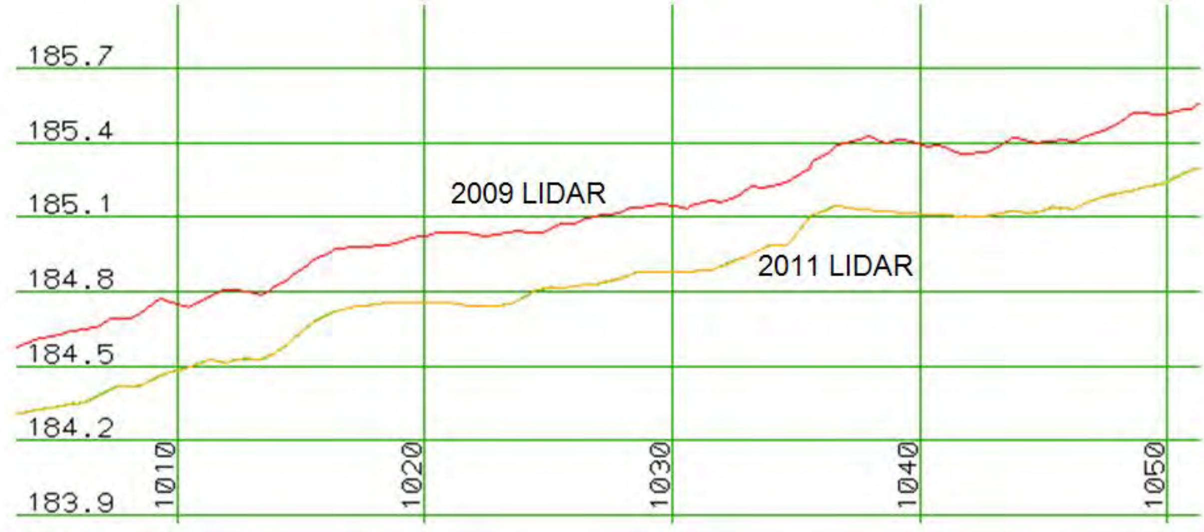



Figure 3.2 2009 LiDAR data versus the 2011 LiDAR data

In order to verify which LiDAR data set was more accurate, a check was undertaken against known permanent survey marks (PSM) and surveyed points captured during the field survey by Cardno. A summary of the findings from the assessment of both the 2009 and 2011 data sets are shown in Table 3.1. The comparison shows that the number of points that were assessed for the 2011 LiDAR data was greater than for the 2009 LiDAR data. This was due to the fact that the 2009 LiDAR data extent did not cover the full area of the Hamilton Study Area. The field survey points that were captured as part of this study form the more reliable data source for comparisons with the LiDAR data. The main reason for this is that PSMs can have the following issues:

- Located in pits or on curbs.
- May not account for road resurfacing and changes in topography over time.
- May be below the ground surface (and there is no way to check this without survey at the same location).

Although the PSMs are of a lesser quality than the field survey, the analysis has been included for comparison purposes.

Table 3.1 2009 and 2011 LiDAR data checks vs survey points and PSMs

| Parameter | 2009 LiDAR Assessment | | 2011 LiDAR Assessment | |
|---|---|---|---|---|
| | Survey Checks | PSM Checks | Survey Checks | PSM Checks |
| Count | 73 | 24 | 135 | 35 |
| Minimum (m) | + 0.113 | - 0.207 | - 0.092 | - 0.443 |
| Maximum (m) | + 0.350 | + 0.587 | + 0.117 | + 0.320 |
| Average (m) | + 0.271 | + 0.353 | + 0.024 | + 0.118 |
| St. Dev. (m) | + 0.043 | + 0.157 | + 0.042 | + 0.146 |

The analysis in Table 3.1 shows the number of points, minimum, maximum, average and standard deviation of the difference between the LiDAR data level and the survey or PSM levels. From the data the 2009 LiDAR data set is clearly higher than both the survey checks and the PSMs with the average differences being + 27.1 cm and + 35.3 cm respectively. This is clearly much higher than the suggested error bands for the 2009 LiDAR data.

The 2011 LiDAR data set was the most accurate LiDAR data set with the average difference for the survey checks and PSMs being + 0.024 m with a 95% confidence interval (2 sigma) of + 0.107 m and – 0.060 m. This is within the stated accuracy of the LiDAR data of +/- 0.1 m to one sigma.

An analysis of the error observed in the 2011 LiDAR data is summarised in Figure 3.3. The figure illustrates the histogram of the error between the 2011 LiDAR data and the field survey points with sample bands of 0.01 m. This has also been presented on the secondary axis as a cumulative histogram which illustrates the mean, as well as the 95% confidence intervals for the error range. For a LiDAR data set this is a good representation of the true ground surface and overall the 2011 LiDAR data provides an acceptable representation of the ground surface.

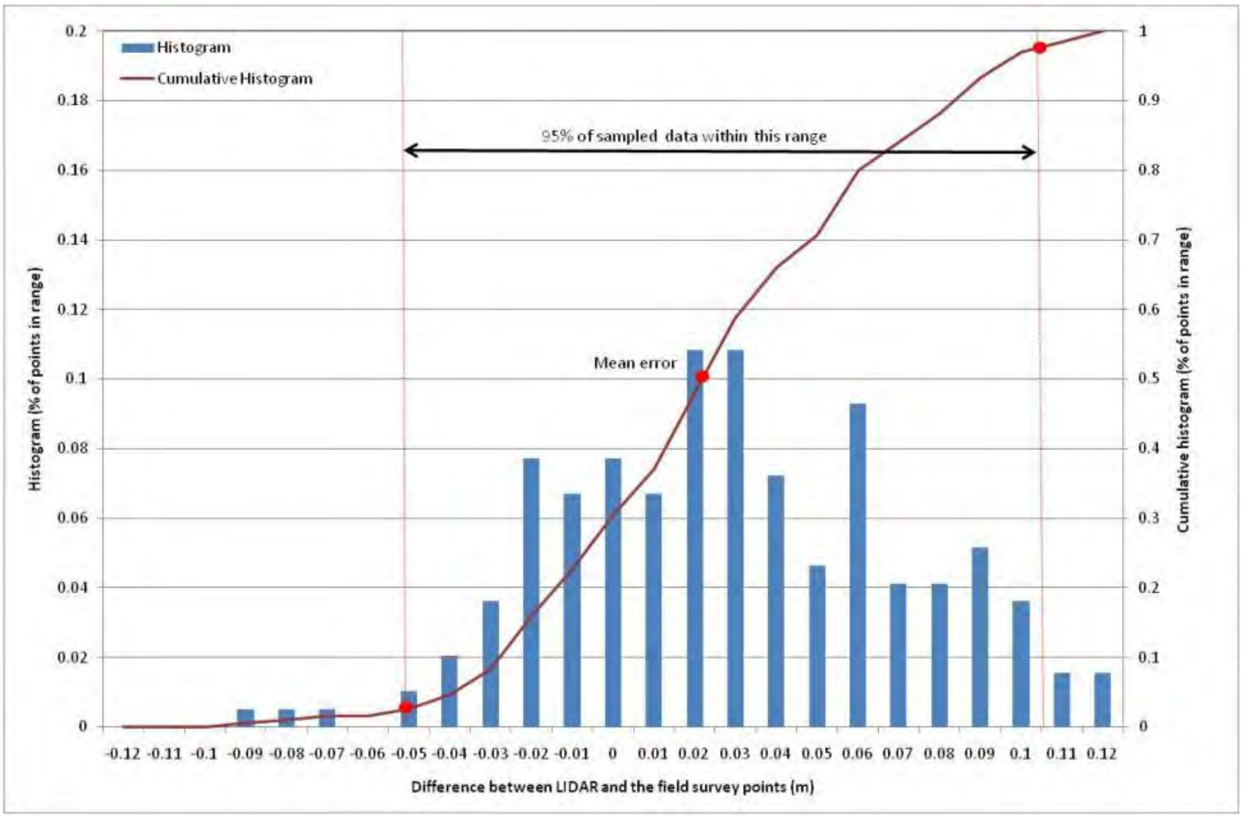



Figure 3.3 Analysis of the difference between the 2011 LiDAR data and field survey

A graphical representation of the field survey points compared to the 2011 LiDAR data is shown in Figure 3.4 that illustrates the observed differences on a spatial scale.

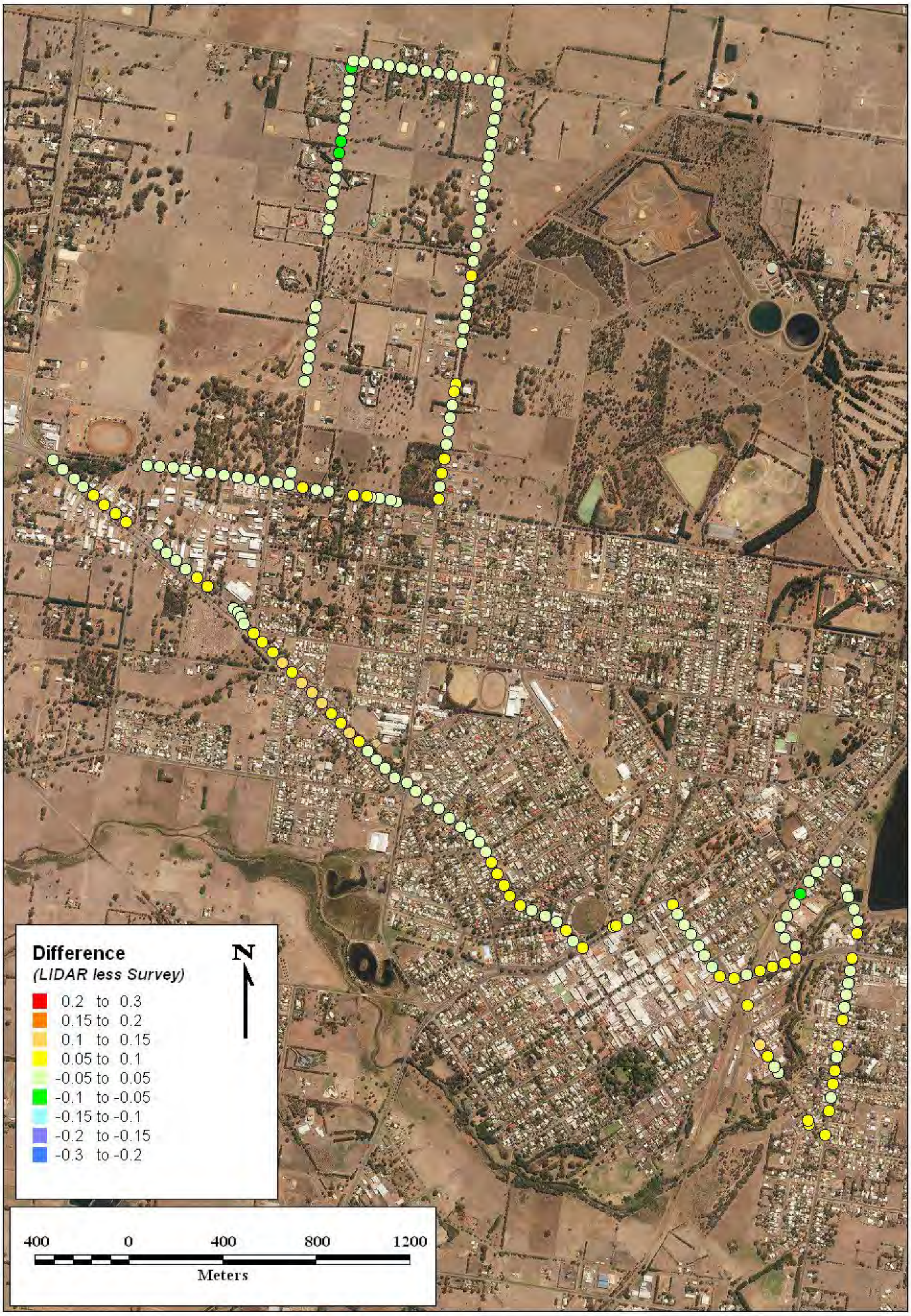



Figure 3.4 Differences between the 2011 LiDAR data and the survey field checks

## 3.2 Historic Flood Events

As part of this flood investigation historic events were required to be calibrated within the hydraulic model. This calibration can only be undertaken using observed historic flood heights. These flood heights have been supplied by the GHCMA, extracted from the *Report on Lake Hamilton Spillway/Grange Burn Flooding Investigations* (GHD, 1987) and from VicRoads design plans (REF).

There are 3 different periods of flood height data provided by GHCMA, namely August 2010 flood marks, March 1946 flood marks and September 1983 flood marks. The locations of these data points are shown in Figure 3.5. The quality and accuracy of these observations is largely unknown but they provide the basis for the calibration of these events.

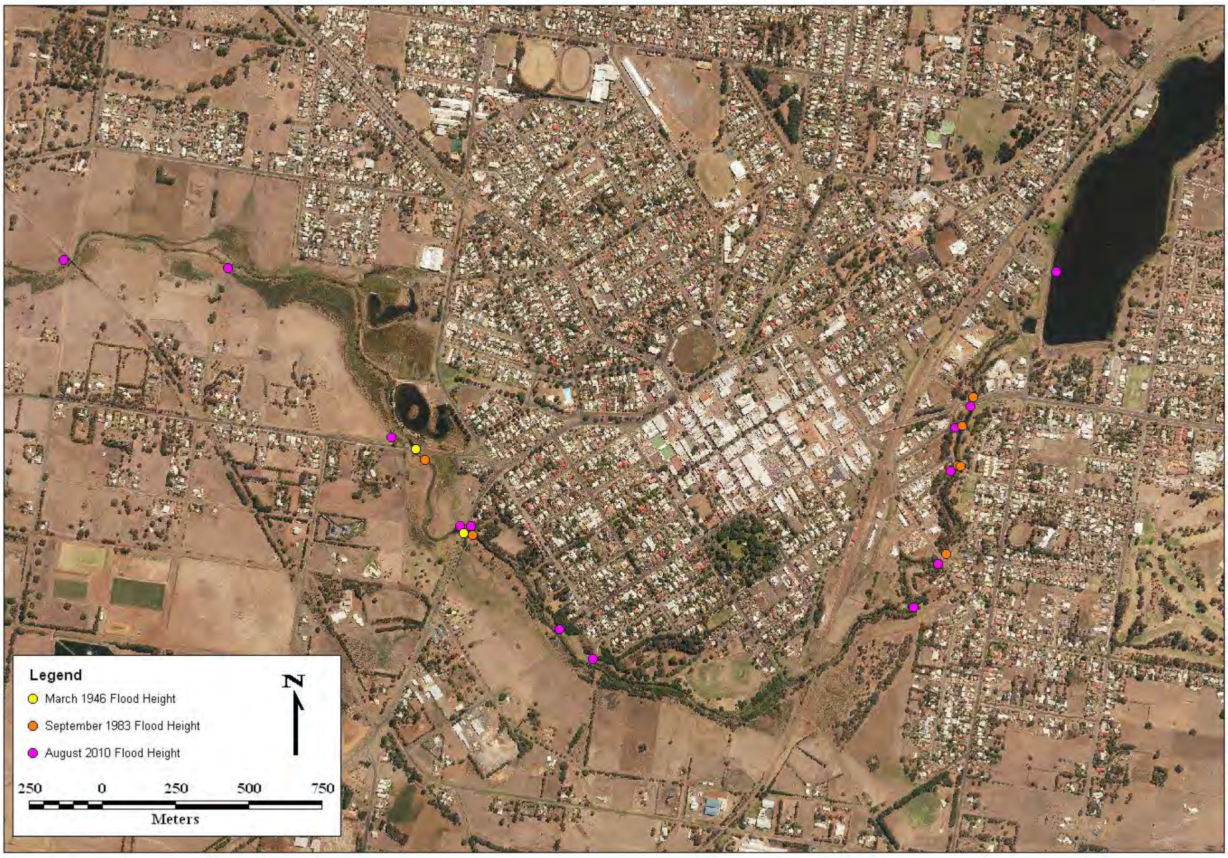



Figure 3.5 Known flood marks for calibration to the 1946, 1983 and 2010 flood events

Subsequent sources of information have been found with data points being sourced for the March 1946 event from the VicRoads plans. These height observations are summarised in Table 3.2. The difficulty with using this data is that the datum of the VicRoads plans is unknown in some cases. It is likely that these plans have been drawn using feet and the Hamilton Sewerage Authority (HSA) datum. The adjustment back to meters AHD is by converting the feet back to meters directly and then to subtract 0.31 m to convert the HSA datum to the AHD datum. This level shift was supplied by the GHCMA.

Table 3.2 Observed flood peaks as determined from the VicRoads bridge plans

| Location | March 1946 level (feet) | Datum | March 1946 level (m) | Datum |
|---|---|---|---|---|
| Ballarat Road Hamilton Hwy Bridge | 576.0 | HSA | 174.79 | AHD |
| Mt Napier Road Bridge | 569.0 | HSA | 172.67 | AHD |
| Dartmoor Hamilton Road | 556.1 | HSA | 168.73 | AHD |
| Portland Road Hamilton Hwy Bridge | 556.3 | HSA | 168.81 | AHD |

The final source of information obtained for the calibration was from the Lake Hamilton Spillway/Grange Burn Flooding Investigations (GHD, 1987). Within this report the flood heights were obtained and are summarised in Table 3.3. Again this data was required to be converted from the HSA datum to the AHD datum for the purpose of this study. It should be noted that for the 1946 event the two observed levels for the Mt Napier road Bridge (VicRoads and from the GHD report) differ by 0.77 m which is a considerable discrepancy. The two data sources were assessed against the structure and observed flood heights through the model to determine which level is the more reliable estimate of the flood heights. The results of this assessment are discussed further in Section 5.

Table 3.3 Observed flood peaks as determined from the GHD Hamilton Spillway Report

| Location | Event | Level (m) | Datum | Level (m) | Datum |
|---|---|---|---|---|---|
| House No. 4 Holden Street | March 1946 | 174.55 | HSA | 174.24 | AHD |
| Mt Napier Road Bridge | March 1946 | 172.21 | HSA | 171.90 | AHD |
| | | | | | |
| Ballarat Road Bridge | Sept 1983 | 174.26 | HSA | 173.95 | AHD |
| House No. 4 Holden Street | Sept 1983 | 174.27 | HSA | 173.96 | AHD |
| Mt Napier Road Bridge | Sept 1983 | 172.50 | HSA | 172.19 | AHD |
| Apex Park BBQ Structure | Sept 1983 | 174.15 | HSA | 173.84 | AHD |
| Apex park Toilet Block | Sept 1983 | 174.07 | HSA | 173.76 | AHD |

## 3.3 Structures and culverts

In order to model the system within a hydraulic model, information about the structures was required. The structure information was extracted from five sources; Hamilton City Council, VicRoads, VicTrack, from additional survey undertaken by Cardno and from the GHCMA. A summary of the structures that were captured as part of this study is summarised in Figure 3.6.

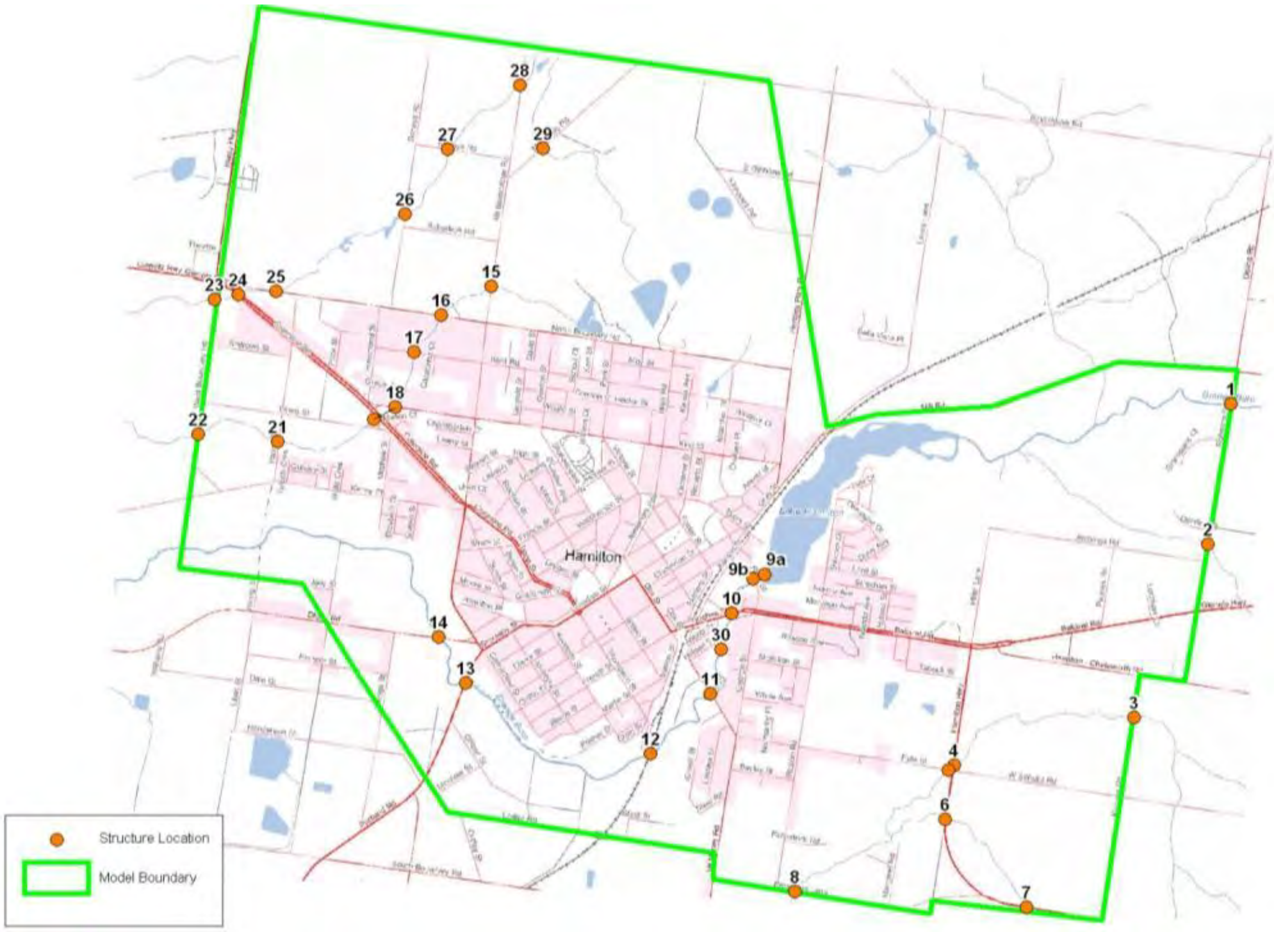



Figure 3.6 Structure survey locations

The details of these structures are summarised in Table 3.4. This table summarises the location of the structure, the type of structure, source of the structure and plans that have been received from various sources. The survey and plans have been used to develop the structure information within the hydraulic model.

Table 3.4 Culvert and bridge summary within the hydraulic model

| No. | Name | Type | Source | Plan Number | Survey | | LiDAR Surface Level | |
|---|---|---|---|---|---|---|---|---|
| | | | | | U/S Invert | D/s Invert | U/S Surface | D/S Surface |
| 1 | Robsons Road | Bridge | Hamilton Council | D11 27997 | Not used within model | | | |
| 2 | Robsons Road | 1 cell 1200 x900 mm Culvert | N/A | No plans | Not used within model | | | |
| 3 | Kurtzes | Culvert | Hamilton Council | HPSC0036 | Not used within model | | | |
| 4 | Hamilton Highway | 1 cell 3048 x 1016 mm Culvert | VicRoads | SN3660 | 187.305 | 187.295 | 187.75 | 187.99 |
| 5 | Fyfe St | 3 cell 914 x 1219 mm Culvert | Hamilton Council | D 11 27998 | 186.9 | 186.8 | 187.45 | 187.54 |
| 6 | Hamilton Highway | 1 cell 1200 x 900 mm Culvert | VicRoads | SN2783 | 188.1 | 179.9 | 188.56 | 188.29 |
| 7 | Hamilton Highway | 1 cell 1200 x 900 mm Culvert | VicRoads | SN2783 | | | | |
| 8 | Petschells lane | 1 cell 2467 x 1542 mm Culvert | Hamilton Council | D 11 28000 | Not used within model | | | |
| 9a | Hamilton Spillway | Spillway | Cardno survey | | 177.86 | N/A | 177.86 | N/A |
| 9b | Riley St | 6 x 800 mm pipes, 1 x 650 mm pipe, 1 x 800 mm arch, 1 x 250 mm arch | GHCMA | | 171.5 | 171.49 | 170.92 | 170.36 |
| 10 | Ballarat Road | Bridge | VicRoads | SN2101 | 170.34 | 170.3 | 170.37 | 170.31 |
| 11 | MtNapier Road | Bridge | Hamilton Council | D 11 27999 | 168.8 | 168.79 | 168.8 | 168.77 |
| 12 | Railway Line | Bridge | VicTrack | 737_80_01V7 | 166.47 | 166.46 | 166.41 | 166.40 |
| 13 | Portland Road | Bridge | VicRoads | SN3240 | 165.3 | 165.25 | 165.32 | 165.2 |
| 14 | Digby Road | Bridge | Hamilton Council | D 11 28001 | 164.4 | 164.39 | 164.51 | 164.6 |
| 15 | Mt Baimbridge Rd | Twin 1200 mm pipe Culvert | Hamilton Council | HPSC0035 | 190.71 | 190.66 | 190.90 | 190.80 |
| 16 | North Boundary Rd | Twin cell 1500 x 900 mm Culvert | Cardno survey | | 187.5 | 187.46 | 187.97 | 187.66 |
| 17 | Kent Road | 3 cell 750 mm pipe Culvert | Cardno survey | | 184.94 | 184.8 | 185.34 | 184.97 |
| 18 | King St | Twin cell 1200 mm pipe Culvert | Cardno survey | | 180.3 | 179.3 | 181.96 | 179.84 |
| 21 | Young St | Twin cell 1200 x 900 mm Culvert | Cardno survey | | 175.36 | 175.03 | 176.11 | 175.96 |
| 22 | West Boundary Rd | Twin cell 375 mm pipe Culvert | Cardno survey | | Not used within model | | | |
| 23 | West Boundary Rd | 4 cell 1200 x 900 mm Culvert | Hamilton Council | D 11 27996 | Not used within model | | | |
| 24 | Coleraine Rd | 4 cell 1200 x 900 mm Culvert | Cardno survey | | 182.31 | 181.1 | 182.58 | 182.51 |
| 25 | Nth Boundary Rd | Twin cell 1200 x 600 mm Culvert | Cardno survey | | 184.68 | 184.63 | 185.47 | 185.35 |
| 26 | Sobeys Rd | 1 cell 1540 x 900 mm Culvert | Hamilton Council | D 11 27993 | 195.0 | 194.9 | 195.35 | 195.3 |
| 27 | Kennys Rd | 1 cell 1200 x 900 mm Culvert | Cardno survey | | 201.1 | 201.05 | 201.83 | 201.28 |
| 28 | Mt Baimbridge Rd | 1 cell 900 mm pipe Culvert | Cardno survey | | Not used within model | | | |
| 29 | Marshalls Rd | Twin cell 300 x 900 mm Culvert | Cardno survey | | Not used within model | | | |
| 30 | Crean Street | Weir and Footbridge | Cardno survey | | 171.2 | 171.19 | 171.2 | 170.69 |

## 3.4 Summary

The LiDAR, historic flood heights and structure information all form an integral component of the hydraulic development process. The data allows for a robust and accurate representation of the study area to be developed and calibrated to known historic flood heights. The information proved as part of this study is shown to be of sufficient quality to develop the hydraulic model to represent the Hamilton Study Area appropriately.

# 4 HYDROLOGY

## 4.1 Previous Studies

Information from previous studies was used as part of the development of the hydrology for the Grange Burn catchment. The following reports and hydrologic analyses:

- Lake Hamilton Spillway/Grange Burn Flooding Investigation (GHD 1987)
- Flood Frequency Analysis – Grange Burn @ Morgiana (GHCMA 2009)
- RORB Model Calibrations: Grange Burn, Henty Creek, Dundas River, Wando River (Cardno 2010)
- Grange Burn RORB model design runs (GHCMA 2010)
- Glenelg Hopkins Catchment Flows spreadsheet (GHCMA 2010)

## 4.2 Review of Available Rainfall Data

### *4.2.1 Rainfall Frequency Analysis*

The rainfall data was required for a number of purposes including:

- Rainfall depths are required for calibrating the RORB model
- Pluviograph information is required for providing a distribution for the rainfall volume for calibrating RORB
- A rainfall frequency analysis (RFA) for providing some background information on the recurrence intervals for historic rainfall events as this was a specific aim of the flood investigation.
- Exploring if there was some correlation between the AEP of large rainfall events and the historic large flood events that have occurred on the Grange Burn.

Typically a RFA would not be required to be undertaken during a hydrology assessment of a catchment for a flood investigation, however, the Grange Burn at Hamilton has very limited flow data available and as such the flows cannot be analysed using the standard approaches adopted in AR&R. A RFA was undertaken to determine if this could provide some guidance on the AEP of historic rainfall events and to determine if there was any correlation between these AEPs and the AEPs of historic flood events. The following section outlines the derivation of the AEPs for historic rainfall events and provides some discussion on the likely correlation between these recurrence intervals and the flood event AEPs.

The rainfall frequency analysis was undertaken on the Hamilton gauge and the gauge was taken as an amalgamation of 090044 (Hamilton) and 090173 (Hamilton Airport). This enabled the record to run from 1889 to 2011 with minimal missing data. Also within the Grange Burn catchment were the 089022 (Moutajup) and 090088 (Yatchaw) gauges. The length of the daily rainfall gauge records are summarised in Table 4.1. The gauge locations in reference to the Grange Burn catchment are shown in Figure 4.1.

Table 4.1 Rainfall Data in the Hamilton Region

| Gauge No. | Gauge Name | Start Date | End Date |
|---|---|---|---|
| 090044 | Hamilton | 01/1889 | 06/1983 |
| 090173 | Hamilton Airport | 07/1983 | Present |
| 089022 | Moutajup | 01/1899 | Present |
| 090088 | Yatchaw (Amaroo) | 07/1903 | Present |

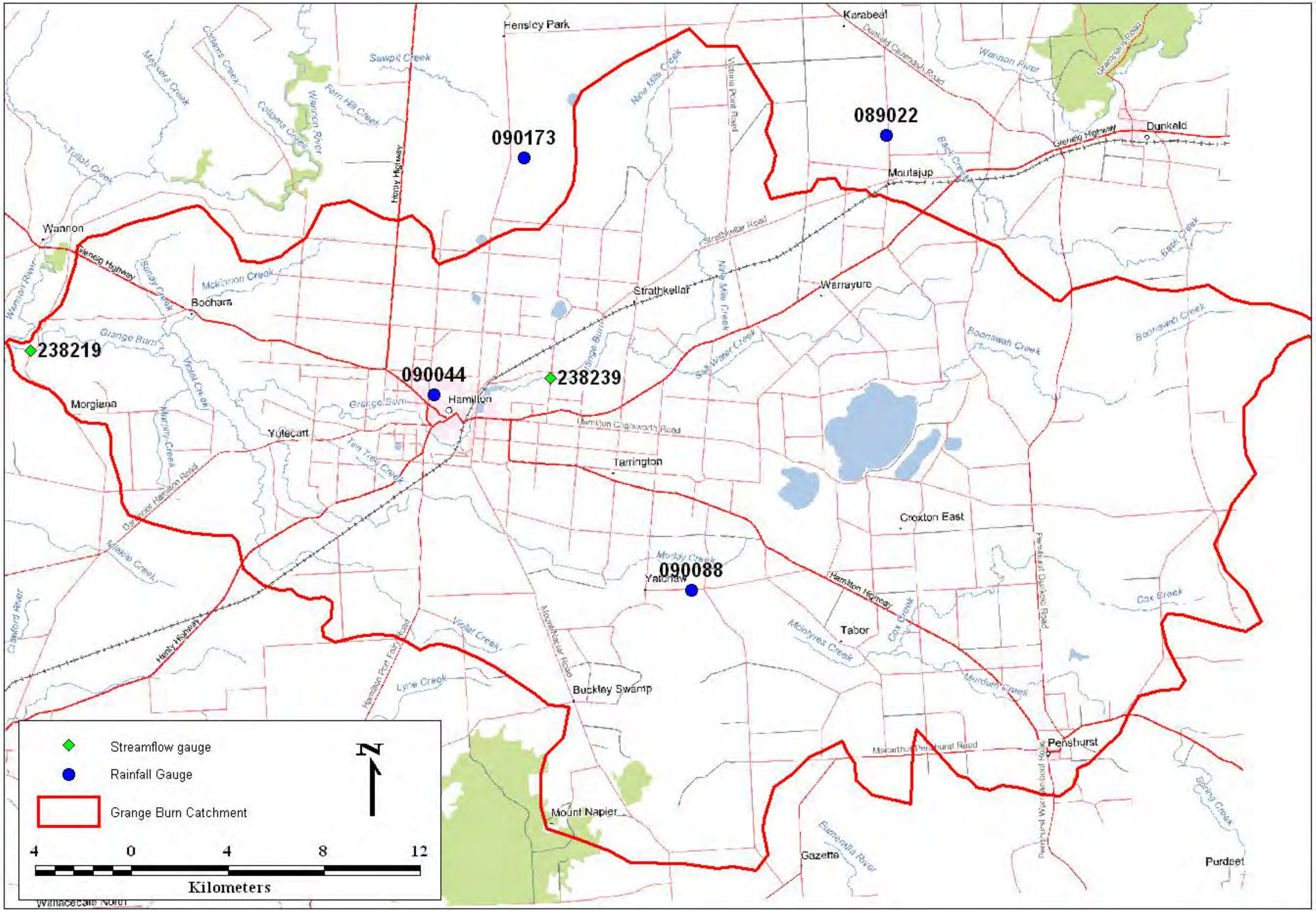



Figure 4.1 Steamflow and rainfall gauge locations for Hamilton

The primary rainfall gauge for the Hamilton flood investigation is the amalgamated Hamilton gauge. The Moutajup and Yatchaw gauges have been included to verify the distribution of the rainfall across the catchments in the large rainfall events.

The primary aim of this rainfall analysis was to determining the AEP of the 1946, 1983, 2004 and 2011 events and subsequently use this information as a guide to the likely flood AEP for these events. The reason this is required is because the only historic flood to be captured at the Grange Burn at Robsons Road gauge was the 1983 event and due to this extremely limited amount of data alternative method for predicting the flood AEPs was required. This assessment will consider if the rainfall events were classified as 24, 48 or 72 hour rainfall events as only daily rainfall totals are available for this preliminary assessment of rainfall.

The rainfalls at each gauge are recorded as 9am to 9am totals and as such are restricted totals. A suitable adjustment factor has been developed for 24 hour durations and the factor to be applied was 1.15 (Boughton et al. 2008). This factor accounts for the temporal restrictions that are present in the recorded rainfall totals.

Figure 4.2, Figure 4.3 and Figure 4.4 show the 24, 48 and 72 hour Rainfall Frequency Analysis (RFA) for the amalgamated Hamilton gauge. The analysis was undertaken using the annual maximum rainfall total for the period of record from 1889 to 2011 and was fitted using a Log Pearson Type III (LPIII) distribution.

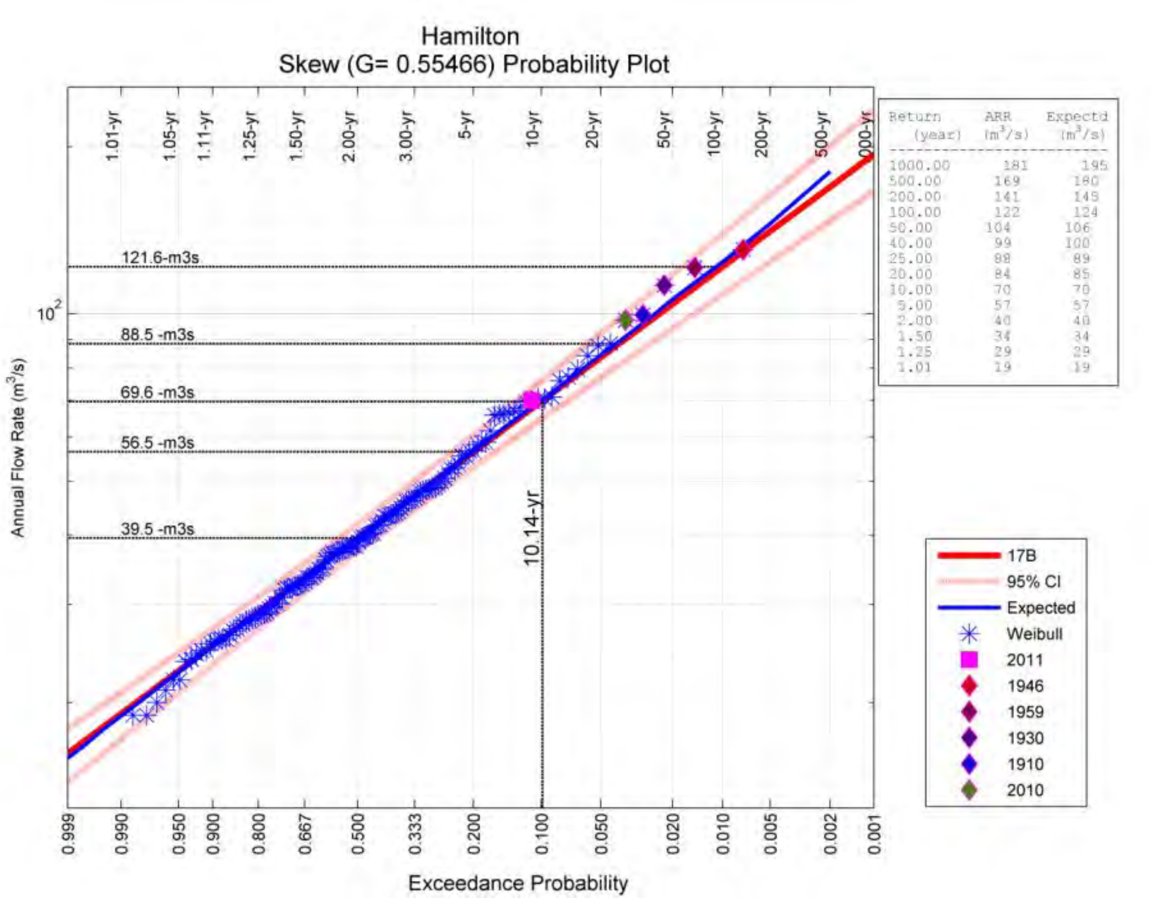



Figure 4.2 Rainfall Frequency Analysis for 24 hour events

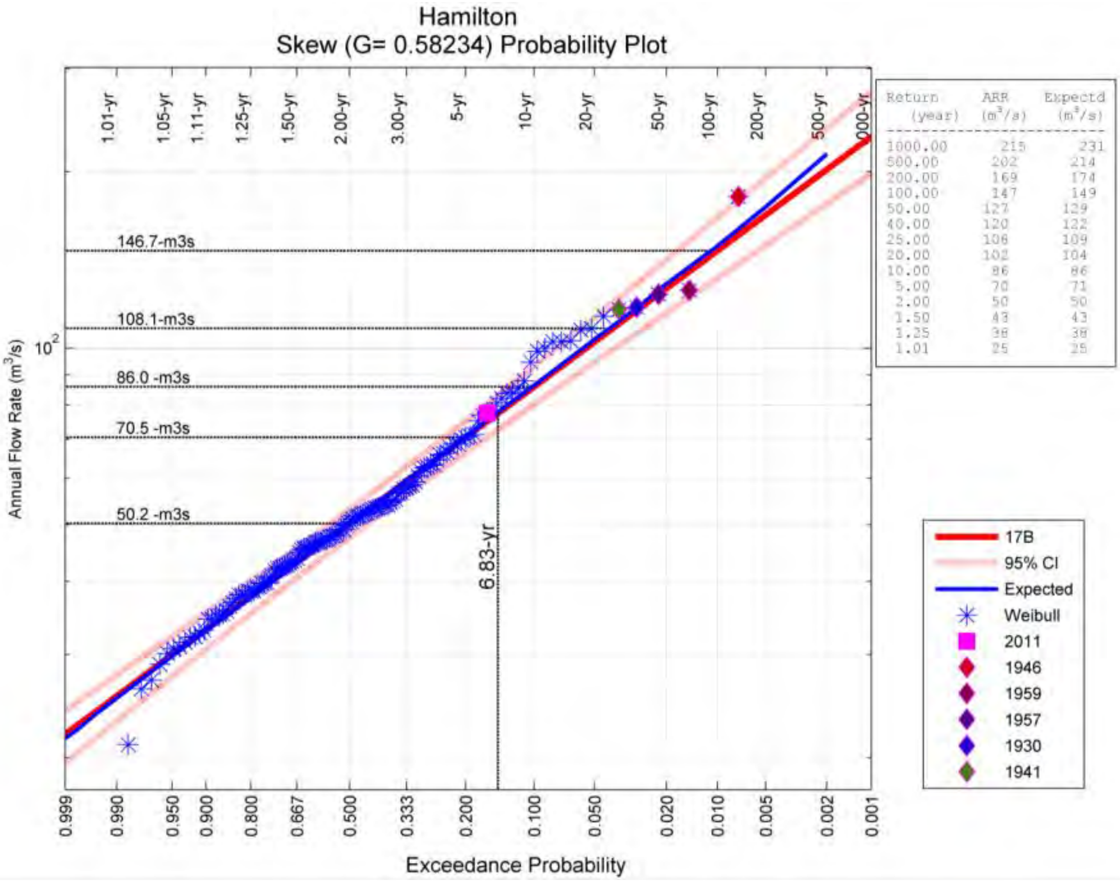



Figure 4.3 Rainfall Frequency Analysis for 48 hour events

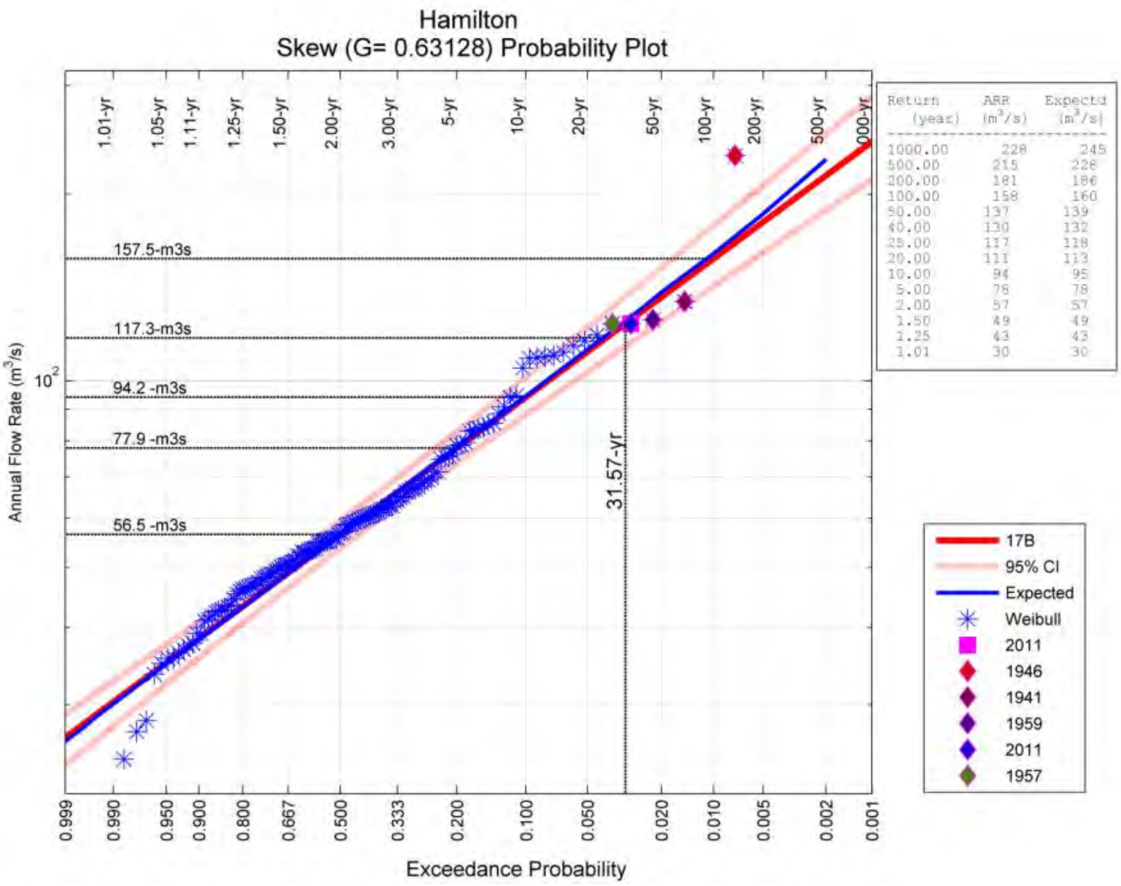



Figure 4.4 Rainfall Frequency Analysis for 72 hour events

The predicted rainfall totals for the 50%, 20%, 10%, 5%, 2% and 1% AEPs are summarised in Table 4.2.

Table 4.2 Rainfall depths estimated for return periods for the 24, 48 and 72 hour duration events (non-restricted)

| AEP (%) | 24 Hour Rainfall Depth (mm) | 48 Hour Rainfall Depth (mm) | 72 Hour Rainfall Depth (mm) |
|---|---|---|---|
| 50% | 39.5 | 50.2 | 56.5 |
| 20% | 56.5 | 70.5 | 77.9 |
| 10% | 69.6 | 86.0 | 94.2 |
| 5% | 83.7 | 102.5 | 111.5 |
| 2% | 104.3 | 126.5 | 136.5 |
| 1% | 121.6 | 146.7 | 157.5 |

In addition to the RFA predicted AEPs of historic rainfall events, the AEPs of these events can be estimated using the IFD parameters as derived from AR&R. The IFD parameters used for the design events are summarised in Table 4.3.

Table 4.3 Intensity Frequency Duration Parameters (Coordinates 37.575 S, 142.200 E)

| IFD | Value |
|---|---|
| ${}^{2}I_{1}$ | 18.09 |
| ${}^{2}I_{12}$ | 3.52 |
| ${}^{2}I_{72}$ | 0.9 |
| ${}^{50}I_{1}$ | 39.63 |
| ${}^{50}I_{12}$ | 6.04 |
| ${}^{50}I_{72}$ | 1.66 |
| Skew | 0.49 |
| F2 | 4.36 |
| F50 | 14.73 |
| Zone | 6 |

Table 4.4 shows a summary of the RFA predicted rainfall totals for the 50%, 20%, 10%, 5%, 2% and 1% AEPs events and for the estimated AR&R rainfall depths. AR&R derives these rainfall depths using a more comprehensive method that involves a regional skew coefficient, as well as daily and intra-daily rainfall records.

Table 4.4 Summary of the RFA and IFD predicted rainfasll depths for the Hamilton catchment

| AEP (%) | 24 Hour Rainfall Depth (mm) | | 48 Hour Rainfall Depth (mm) | | 72 Hour Rainfall Depth (mm) | |
|---|---|---|---|---|---|---|
| | RFA | IFD | RFA | IFD | RFA | IFD |
| 50% | 39.5 | 50 | 50.2 | 58.9 | 56.5 | 62.9 |
| 20% | 56.5 | 62.5 | 70.5 | 74.7 | 77.9 | 80.4 |
| 10% | 69.6 | 70.7 | 86.0 | 85.1 | 94.2 | 92.0 |
| 5% | 83.7 | 82.3 | 102.5 | 99.5 | 111.5 | 108.0 |
| 2% | 104.3 | 98.4 | 126.5 | 120.0 | 136.5 | 130.8 |
| 1% | 121.6 | 111.6 | 146.7 | 136.7 | 157.5 | 149.6 |

Analysis of the rainfall time series was undertaken to determine the relative AEPs of the large flood events that have occurred historically. The events assessed include the 1946, 1983, 2004, 2010 and 2011 events. Although the 2010 event is not a significant event it has been included in the assessment as flood height data has been recorded during this event that will later be used for calibration.

Table 4.5 shows the results of the AEP assessment for each event assessed over a 24, 48 and 72 hour period. The 24 hour period was the 9am to 9am total rainfall volume which was subsequently adjusted to account for the restricted nature of this time series. The RFA and the AR&R methods were used to determine the relative AEP of these events. The AR&R AEP estimates are likely to be more representative as they are derived from methods that account for additional sources of information and apply a region skew coefficient rather than a point source skew coefficient.

From the assessment, the 1946 event was the largest event to occur in recent history with the 24 hour event being above a 1% AEP and with the 72 hour event being approximately a 0.5% AEP event. The 2 events occurring in December 2010 and January 2011 were within the range of a 4 to 5% AEP rainfall events. The December 2010 event was more intense over a 24 hour period, whereas the January 2011 event was more significant over a 72 hour period. August 2004, which is a calibration event, is a frequent event with AR&R estimating it at approximately a 100% AEP. Similarly, the RORB calibration events of September 1983 and 1984 were estimated as 100% AEP rainfall events.

The peak flow event in September 1983 was not caused by the peak rainfall event for that year. The peak rainfall occurred during May, whereas the peak flow event occurred in September. From analysis of the concurrent streamflow and rainfall records it appears that the antecedent catchment conditions play an important role in the catchment response to rainfall events. Smaller rainfall events occurring after the winter period often cause the peak flow events, whereas large rainfall events during the summer period lead to relatively small flood events. This is examined further in Section 4.3.1. However, the fact that the antecedent conditions influence the peak flow rates suggests that the RFA or IFD assessment may not be a suitable method for inferring the AEP for the peak flow events.

In particular, the 1946 rainfall event occurred in March 1946 following the summer period. This may imply that although this event was a significant amount of rainfall over a three day period it may not have caused an extreme flow response from the catchment. This is reinforced by the December 2010 and January 2011 events where rainfall depths were near 100 mm but the catchment runoff response was not significant (compared to large streamflow events such as September 1983).

Overall, the purpose of the RFA was to establish if there was a suitable correlation between the AEP of the rainfall events and known peak flood events with the aim of using the rainfall AEPs to approximate the peak flood AEPs for use in deriving the design flood events. Following the conclusion of this assessment it appears that the antecedent conditions within the Grange Burn catchment have a large impact on the rates of runoff and as such there is no consistent correlation between the AEP of rainfall events and the historic flood events. This is by no way uncommon for regional Victorian catchments as antecedent conditions often can vary loss rates substantially. It is important to observe that the catchment antecedent conditions play an important role in the Grange Burn catchment to assist in the understanding of catchment behaviour for future flood warning and to aid understanding of catchment response to rainfall events.

Table 4.5 Annual Exceedence Probabilities for the 1946, 1983, 1984, 2004, 2010 and 2011 rainfall events

| Event | Duration | Hamilton (amalgamated) | | | Comment |
|---|---|---|---|---|---|
| | | Rainfall (mm) | AEP (FFA) [ % ] | AEP (AR&R) [ % ] | |
| Mar-46 | 24 | 113.5 | 1.6 % | 0.9 % | Largest event on record |
| | 48 | 157.7 | 0.7 % | 0.4 % | |
| | 72 | 200.9 | 0.3 % | < 0.2 % | |
| May-83 | 24 | 61.2 | 14 % | 20 % | Largest rainfall event in 1983 |
| | 48 | 63.2 | 25 % | 33 % | |
| | 72 | 64.4 | 33 % | 50 % | |
| Sep-83 | 24 | 29.8 | 100 % | > 100 % | Calibration event in RORB, largest flow event recorded at Hamilton. |
| | 48 | 50.2 | 50 % | 100 % | |
| | 72 | 56.0 | 50 % | 100 % | |
| Sep-84 | 24 | 16.4 | 100 % | > 100 % | Calibration event in RORB |
| | 48 | 25.6 | 100 % | > 100 % | |
| | 72 | 25.8 | 100 % | > 100 % | |
| Aug-04 | 24 | 29.8 | 100 % | > 100 % | Recent flood event |
| | 48 | 51.4 | 50 % | 100 % | |
| | 72 | 51.4 | 50 % | 100 % | |
| Dec-10 | 24 | 84.8 | 4.8 % | 4.0 % | Recent flood event |
| | 48 | 93.4 | 7.1 % | 6.3 % | |
| | 72 | 95.2 | 10 % | 8.3 % | |
| Jan-11 | 24 | 60.8 | 14 % | 20 % | Recent flood event |
| | 48 | 67.4 | 25 % | 25 % | |
| | 72 | 107.6 | 5.6 % | 5.0 % | |

## 4.3 Review of Available Flow Data

### 4.3.1 Flood Frequency Analysis

The Grange Burn has limited streamflow gauges with historical records. On the system there are only two suitable streamflow gauges. This limits the options available for developing the peak flood events and design flood events at Hamilton. This discussion outlines these restrictions and the methods that have been used to develop the flood frequency assessment (FFA) and design flood events.

The available flow gauges on the Grange Burn include the Grange Burn at Hamilton (238239) and the Grange Burn at Morgiana (238219). The location of the streamflow gauges are shown on Figure 4.1 and the data availability is summarised in Table 4.6.

Table 4.6 Streamflow Data in the Hamilton Region

| Gauge No. | Gauge Name | Area | Start Date | End Date |
|---|---|---|---|---|
| 238239 | Grange Burn at Hamilton | 222 $km^2$ | May-1981 | Apr-1985 |
| 238219 | Grange Burn at Morgiana | 964 $km^2$ | Jul-1963 | Present |

The Grange Burn at Hamilton gauge is upstream of Lake Hamilton and had limited data. The data was not used to develop a FFA due to the limited data availability. The gauged data was used to calibrate the RORB model for events that occurred during the 1983 and 1984 years.

At the termination of the Grange Burn is the Grange Burn at Morgiana gauge which is approximately 1.2 km upstream of the confluence of the Grange Burn and the Wannon River. This gauge captures the entire Grange Burn system and had a long term record of 48 years. The gauged data was used to undertake a FFA and a Log Normal, Log Pearson Type III (LPIII), Generalised Extreme Value (GEV) and Generalised Pareto distribution were fitted to the data. The distributions were fitted using a Bayesian approach and the resultant FFA distributions are shown in Figure 4.5.

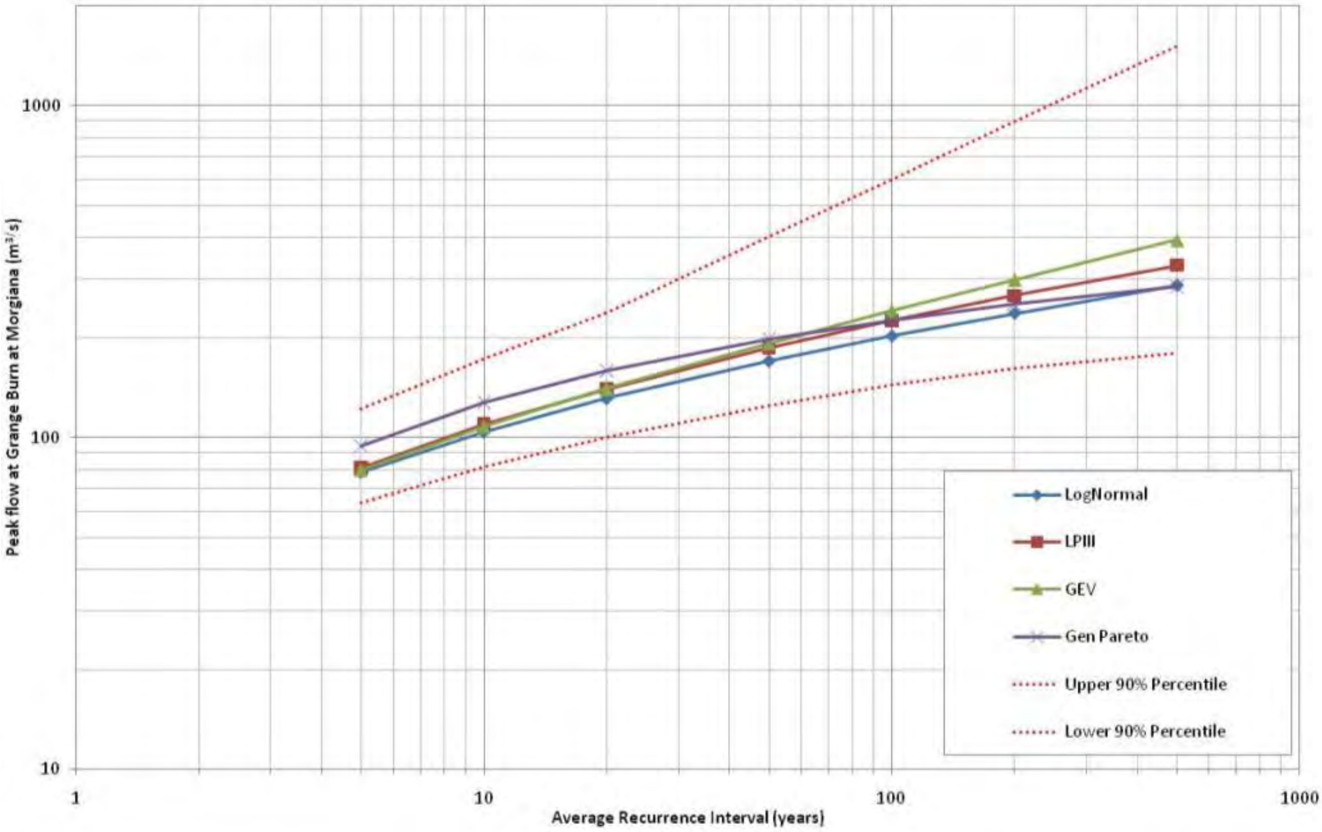



Figure 4.5 FFA for Grange Burn at Morgiana

Details of the fitting of each distribution are shown in Appendix C. The details of the predicted peak flows for the associated AEPs are shown in Table 4.7.

Table 4.7 Estimated flows for the fitted distributions for the FFA at Grange Burn at Morgiana

| | Annual Exceedence Probability (%) | | | | | | |
|---|---|---|---|---|---|---|---|
| Distributions | 20% | 10% | 5% | 2% | 1% | 0.5 % | 0.2 % |
| Log-Normal | 79 | 104 | 131 | 170 | 202 | 237 | 287 |
| Lower 90% Conf. | 64 | 82 | 100 | 124 | 144 | 165 | 193 |
| Upper 90% Conf. | 102 | 142 | 187 | 258 | 319 | 388 | 493 |
| LPIII | 81 | 109 | 140 | 186 | 225 | 267 | 330 |
| Lower 90% Conf. | 64 | 84 | 103 | 127 | 145 | 162 | 184 |
| Upper 90% Conf. | 106 | 155 | 220 | 342 | 469 | 636 | 940 |
| GEV | 80 | 108 | 140 | 193 | 241 | 299 | 394 |
| Lower 90% Conf. | 64 | 83 | 102 | 126 | 145 | 165 | 190 |
| Upper 90% Conf. | 105 | 159 | 238 | 405 | 600 | 892 | 1509 |
| Generalised Pareto | 94 | 128 | 159 | 198 | 226 | 252 | 285 |
| Lower 90% Conf. | 71 | 95 | 117 | 142 | 157 | 167 | 179 |
| Upper 90% Conf. | 122 | 173 | 236 | 342 | 447 | 572 | 785 |

The distributions fitted to the streamflow data show that the peak flows at Grange Burn at Morgiana exhibits a shape that is difficult to represent using a fitted distribution. The FFA indicates that the catchment has some complex interaction with storage and runoff that cause three clear groups of peak flows relationships, those for peak flows:

- Below 30 $m^3/s$
- Between 30 $m^3/s$ and 55 $m^3/s$
- Greater than 55 $m^3/s$.

This change in runoff relationship at Morgiana is likely to be caused by the complex nature of the catchment which involves large areas of swamps and storage. The main concern with the relationship fitted to the FFA is that it cannot be adequately fitted by a single distribution for the low, moderate and high flow events. This leads to considerable uncertainty around the return periods associated with the peak flows. For example the 90% confidence intervals around the predicted 1% AEP event are between 144 and 600 $m^3/s$ for all durations. This is a wide range of flows given the length of record used for the FFA.

Out of the four fitted distributions, the LPIII, GEV and Generalised Pareto distributions all predicted the 1% AEP peak flow to be between 225 and 241 $m^3/s$. This provides some measure of confidence that the 1% AEP estimate at Morgiana is within this range. The log-normal distribution was not considered for this assessment as this was the poorest fitting distribution. The GEV distribution provided the highest estimate for the extreme events.

To highlight the complexity of the catchment response at Grange Burn at Morgiana a number of peak flow events have been extracted along with the 24, 48 and 72 hour rainfall totals at Hamilton. The rainfall totals at Hamilton were checked against the rainfalls experienced at the Mountajup and Yatchaw and were found to have similar rainfall depths. Where possible the Grange Burn at Hamilton peak flow was also extracted. This data is summarised in Table 4.8.

Table 4.8 Comparison of rainfall and peak flow events at Morgiana and Hamilton

| Event | Rainfall (Hamilton) [mm] | Period of rainfall | Grange Burn at Hamilton Peak Flow | Grange Burn at Morgiana Peak Flow |
|---|---|---|---|---|
| Oct 1976 | 35.0<br>40.6<br>43.8 | 24 hrs<br>48 hrs<br>72 hrs | | 95.8 m³/s |
| May 1983 | 61.2<br>63.2<br>64.4 | 24 hrs<br>48 hrs<br>72 hrs | 8.0 m³/s | 15.8 m³/s |
| Sept 1983 | 29.8<br>50.2<br>56.0 | 24 hrs<br>48 hrs<br>72 hrs | 105.2 m³/s | 181.9 m³/s |
| Sept 1984 | 16.4<br>25.6<br>25.8 | 24 hrs<br>48 hrs<br>72 hrs | 33.6 m³/s | 55.2 m³/s |
| Oct 1992 | 18.2<br>28.6<br>38.0 | 24 hrs<br>48 hrs<br>72 hrs | | 102.4 m³/s |
| Mar 2003 | 39.2<br>39.8<br>41.0 | 24 hrs<br>48 hrs<br>72 hrs | | 48.4 m³/s |
| Aug 2004 | 29.8<br>51.4<br>51.4 | 24 hrs<br>48 hrs<br>72 hrs | | 157.8 m³/s |
| Nov 2007 | 64.0<br>70.8<br>80.8 | 24 hrs<br>48 hrs<br>72 hrs | | 53.1 m³/s |
| Dec 2010 | 84.8<br>93.4<br>95.2 | 24 hrs<br>48 hrs<br>72 hrs | | 31.2 m³/s |
| Jan 2011 | 60.8<br>67.4<br>107.6 | 24 hrs<br>48 hrs<br>72 hrs | | 32.8 m³/s |

From the analysis it is clear that the antecedent conditions within the catchment play a major part in the peak flows observed at Morgiana. For the largest event in the Morgiana record in September 1983 of 181.9 m³/s, the rainfall in the 72 hours leading up to the peak flow was 64.4 mm. For the events in December 2010 and January 2011 the 72 hour rainfall totals were 95.2 mm and 107.6 mm respectively and the peak flows seen at Morgiana were 31.2 and 32.8 m³/s respectively. Throughout the record it is evident that during wet antecedent periods the catchment has a much larger peak flow response at Morgiana, however, as seen during the summer period in 2010/11 during dry antecedent conditions the catchment has the capacity to store large volumes of runoff. This is evident through other events that have been extracted.

The outcome of this analysis is that the Morgiana FFA has a reasonably high degree of uncertainty and this limits the ability to use this gauge (via a regression relationship) as a suitable proxy for directly determining the flows at

Hamilton. However, due to the extremely limited data sources for this catchment this method for assessing the design flows at Hamilton was explored and used to provide some guidance for the design flow estimates.

The lack of data makes developing an estimate of the peak flood events and return periods at Hamilton particularly difficult. In particular, the estimation of peak flow rates for the key flood events such as the 1946, 1983, 2010 and 2011 events is not possible by directly using a FFA. The RFA can provide some guidance on the expected AEP of these events (see Table 4.5) however, as discussed the antecedent conditions heavily weight the catchment response to rainfall events and hence small rainfall events following wet periods can lead to large flow events.

## 4.4 Hydrologic Model Development

The Grange Burn hydrological model was recently calibrated by Cardno for the GHCMA. Much of the discussion and background of the hydrologic model development has been extracted from the *RORB Model Calibrations – Grange Burn, Henty Creek, Dundas River and Wando River* (Cardno, 2010). A Hamilton specific RORB model and three additional small RORB models have been developed for this Flood Investigation which represents the flows for the small tributaries of the Grange Burn that are within the hydraulic model study area. The $k_c$, m and loss rates were adapted from the Hamilton and Grange Burn models. These models have been developed using the original Grange Burn model as a guide.

The models created are denoted:

- “Grange Burn model” - full area model from upper reached down to the termination of the Grange Burn system. This model was developed and supplied to Cardno prior to this project but was updated to adjust the upstream catchment boundaries. This model is shown in Figure 4.6. This model was not used to develop any design flows for this flood investigation, it has been supplied as part of this section because the existing catchments were used as a basis for the revised Hamilton specific model. Exploratory runs were also undertaken using this model as part of the project.
- “Hamilton model” – this model terminated upstream of Lake Hamilton and does not include lake Hamilton. This model provides the inputs to the hydraulic model for the Grange Burn. This model is shown in Figure 4.7.
- “Petschels Lane model” – this model is for the unnamed tributary to the south east of the Grange Burn. This hydrologic model overlaps the hydraulic model. The inflows to the hydraulic model are via the upstream boundary for the non-overlapping areas, and the routed sub-catchment inflows where the models overlap. The routed sub-catchment inflows are input into the hydraulic model via lateral inflow nodes. This model is shown in Figure 4.8.
- “Marshalls Road model” & “Kennedys Road model” - these models are for the unnamed tributaries to the north of the Grange Burn. This hydrologic models overlaps the hydraulic model. The inflows to the hydraulic model are via the upstream boundary for the non-overlapping areas, and the routed sub-catchment inflows where the models overlap. The routed sub-catchment inflows are input into the hydraulic model via lateral inflow nodes. These models are shown in Figure 4.8.

Each model will be discussed in turn to provide background information of the model development for the Grange Burn as well as to outline the approach to the hydrology for this study.

## Grange Burn model

The Grange Burn catchment has an area of 964 $km^2$ and consists mostly of rural land. The largest town in the Grange Burn catchment is Hamilton. Lake Hamilton is a significant hydrological feature in the system, however the attenuation of large flow events is not significant due to the limited storage capacity of the lake. Lake Linlithgow and Buckley Swamp have also been identified as significant hydrological/hydraulic features of the catchment.

The Grange Burn hydrological (RORB) model was originally provided by the GHCMA. For this model update some significant changes to the model were implemented, particularly upstream of Hamilton. The major change is that Lake Linlithgow now discharges fully to Muddy Creek rather than through the Grange Burn at Hamilton. This reduced the catchment area to Hamilton significantly. Additional modifications were introduced to terminate a sub-area at the Grange Burn at Hamilton gauge. The revised RORB model is shown in Figure 4.6. These modifications were undertaken as a result of improved topography becoming available for this project that allowed for improved catchment delineation. The original RORB model did not represent the Grange Burn at Hamilton catchment appropriately.

In addition to the sub-catchment updates, Lake Hamilton was included in the Grange Burn RORB model. The spillway dimensions and stage-discharge curve were developed from a rating curve in the *Report on Lake Hamilton Spillway/Grange Burn Flooding Investigations* (GHD, 1987). The GHD report contained a spillway rating curve that covered flows over the spillway up to 410 $m^3/s$. This relationship was utilised as the known stage-discharge relationship.

The GHD report provided some background information regarding the Lake Hamilton spillway. Lake Hamilton was originally designed using an assumed 0.001% AEP design event of 142 $m^3/s$. In this report and the GHD report, it is clear that this estimate for the 0.001% AEP was an underestimate based on the additional hydrology available for the GHD study as well as this study. The maximum bank full capacity of Lake Hamilton when it was designed and built was 250 $m^3/s$. The spillway rating curve covers this full range of design flows and extends this further to account for the expected increase in flow rates for the current hydrological estimates for design flood events.

In order for the storage to be modelled in RORB the volume-depth relationship was required to be developed. As the volume-depth relationship was not known explicitly (no construction drawings were available), a volume-depth relationship was developed utilising the known surface area of the lake and an assumed edge slope. A linear relationship was used to estimate the volume for the top 3 metres of storage in Lake Hamilton (the top 3 m up to the spillway crest) and a 1:8 (vert:horiz) edge slope was assumed for levels exceeding the spillway level. The side slope was introduced to account for the likely increase in surface area and storage during flood events. The full Lake Hamilton volume was not required to be represented within the hydrological model as RORB only requires the active storage volume and the flow rate over the spillway to be defined. The spillway was set at 3.3 m, equating to 178 mAHD (GHD, 1987).

A depth-volume relationship was established to simulate the operation of Lake Hamilton as shown in Table 4.9.

Table 4.9 Stage, volume and discharge for Lake Hamilton

| Assumed Water Level (m) | Volume ($m^3$)[1] | Discharge ($m^3/s$)[2] | Comments |
|---|---|---|---|
| 0.0 | 0 | 0 | |
| 1.0 | 361,800 | 0 | |
| 2.0 | 723,600 | 0 | |
| 3.0 | 1,085,400 | 0 | |
| 3.3 | 1,196,964 | 0 | Spillway Level |
| 3.5 | 1,274,700 | 10 | |
| 3.7 | 1,355,124 | 23 | |
| 3.9 | 1,438,,236 | 40 | |
| 4.1 | 1,524,036 | 60 | |
| 4.3 | 1,612,524 | 85 | |
| 4.5 | 1,703,700 | 110 | |
| 4.7 | 1,797,564 | 140 | |
| 4.9 | 1,894,116 | 175 | |
| 5.1 | 1,993,356 | 210 | |
| 5.3 | 2,095,284 | 247 | |
| 5.5 | 2,199,900 | 280 | |
| 5.7 | 2,307,204 | 325 | |
| 5.9 | 2,417,196 | 360 | |
| 6.1 | 2,529,876 | 410 | |

[1] Assumed volume using surface area and estimated storage side slope.
[2] Depth-discharge taken from the GHD spillway discharge relationship (1987).

Due to the lack of detailed information on Buckley Swamp and Lake Linlithgow, inter-station areas were used to enable the model calibration parameters to vary between the area upstream of Lake Hamilton and the remainder of the catchment. The two inter-station areas are shown in Figure 4.6 and due to the inclusion of Buckley Swamp, Lake Hamilton and Lake Linlithgow in the downstream catchment, a higher stream lag is expected for this area as compared to the upstream section of the catchment.

All inflows for the Grange Burn hydraulic model were derived using the Hamilton hydrologic model. The full Grange Burn model was used only for exploratory purposes within this study.

### Hamilton Model

In addition to the improvement to the Grange Burn RORB model, a Hamilton specific model was developed to better represent the catchment upstream of Hamilton. This RORB model consisted of 13 sub-catchments and had an area of 223 $km^2$. The RORB model consisted of 2 inter-station areas, one terminating at the Grange Burn at Hamilton gauge and the other at Lake Hamilton. The catchment layout for this RORB model is shown in Figure 4.7.

The inflows to the hydraulic model are taken at 2 locations from this model. The primary location is immediately downstream of the previously utilised Robsons Road streamflow gauge and the second is the tributary that joins downstream of this location. Two inputs were used to ensure that there was no overlapping area from the hydrologic models and hydraulic models.

### Petschels Lane, Marshalls Road and Kennys Road Models

Figure 4.8 shows the small RORB models developed for the tributaries that are required in the hydraulic model.

The three hydrologic models that have been derived for the smaller tributaries within the Grange Burn study area overlap the hydraulic model due to the limited size of the models. The majority of the catchment contributing to the runoff within these catchments is derived within the hydraulic model itself and as such must be counted within the hydrology. The method used to deliver the runoff to the hydraulic model was to allow the non-overlapping areas to contribute the full flows at the upstream boundary of the model. For the overlapping areas the hydraulic model received inflows from the hydrologic model using the routed inflows within RORB. These were input into the hydraulic model via lateral inflow nodes within the hydraulic model.

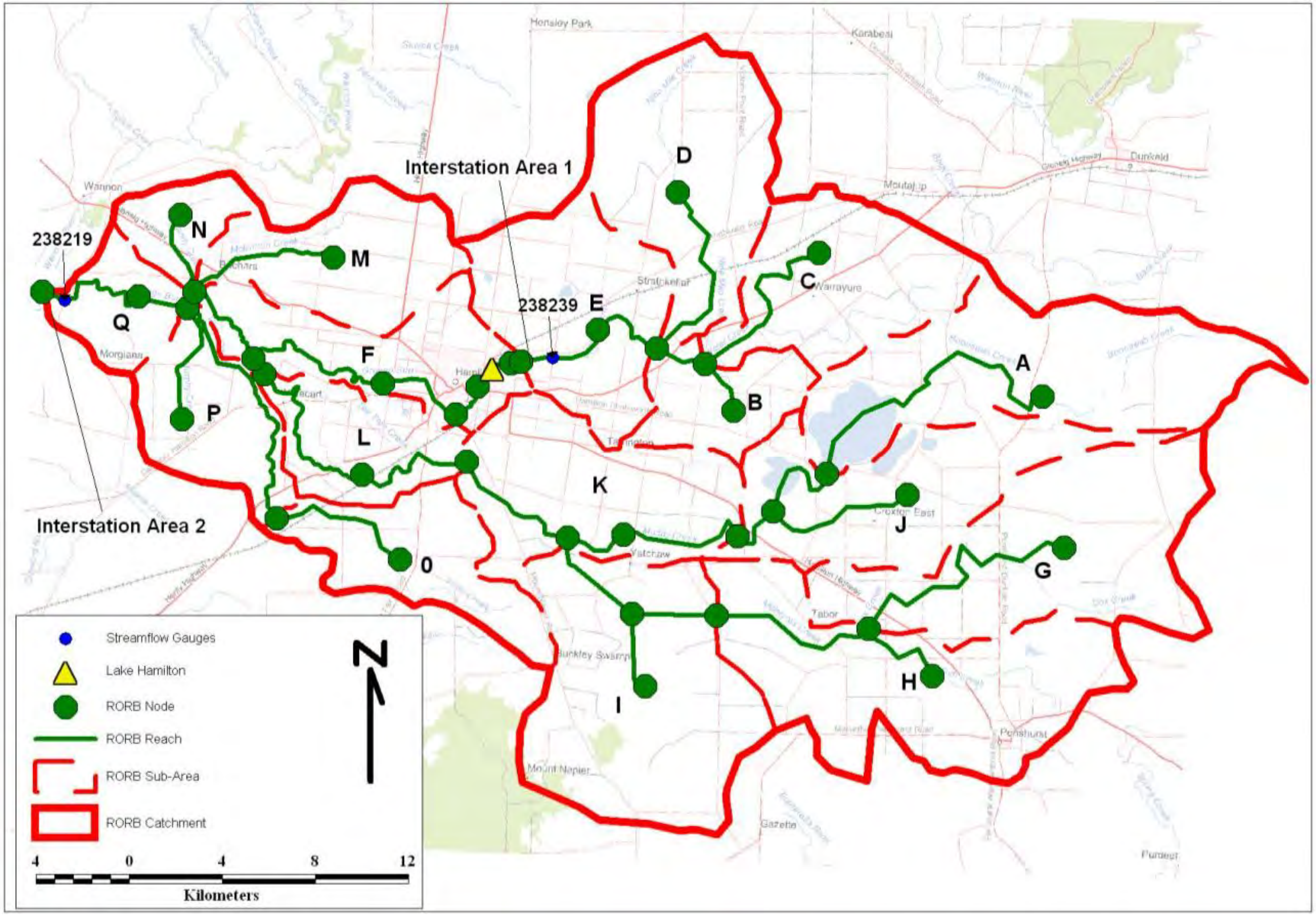



Figure 4.6 RORB model for the Grange Burn catchment (original RORB model)

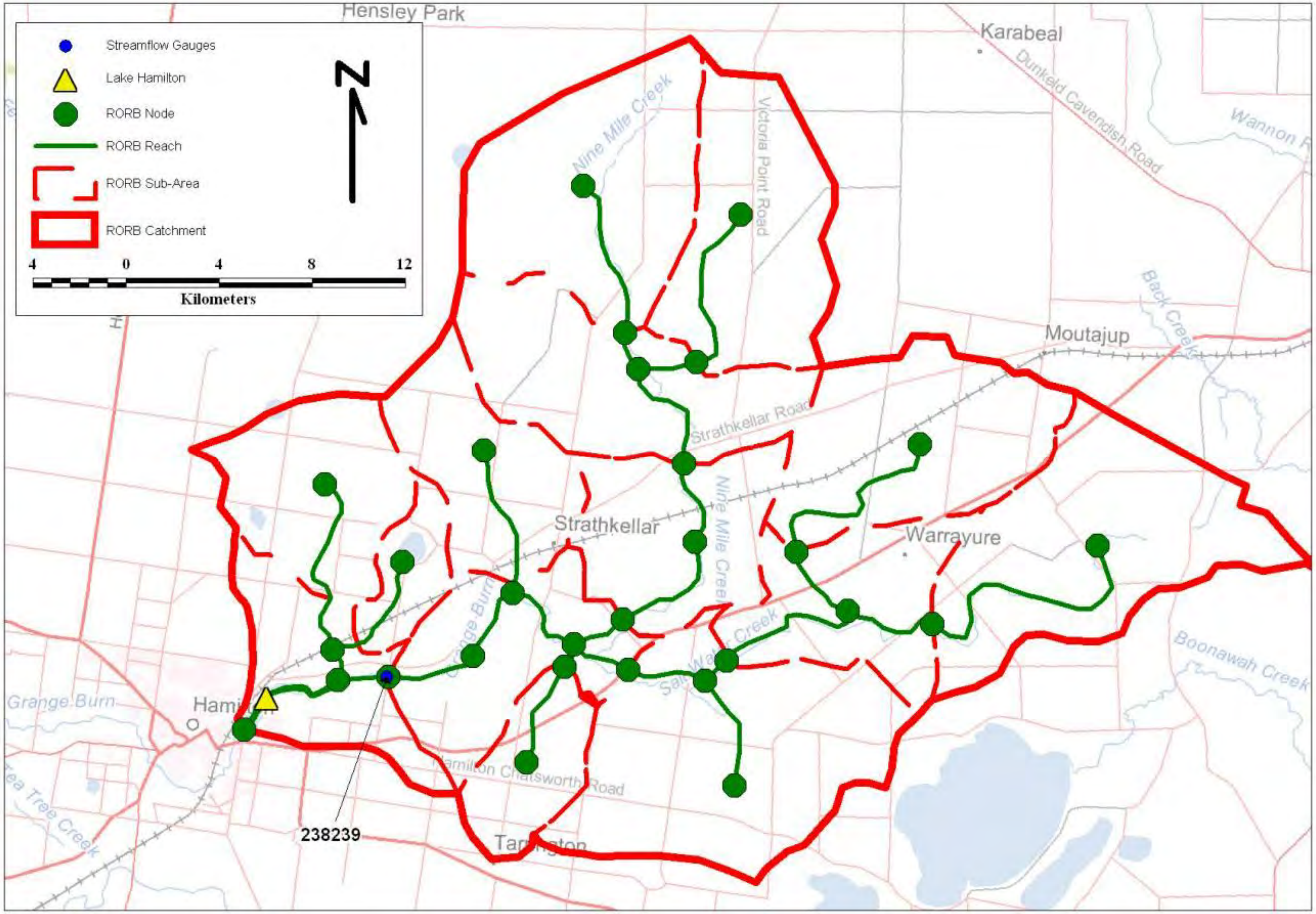



Figure 4.7 RORB model for Hamilton

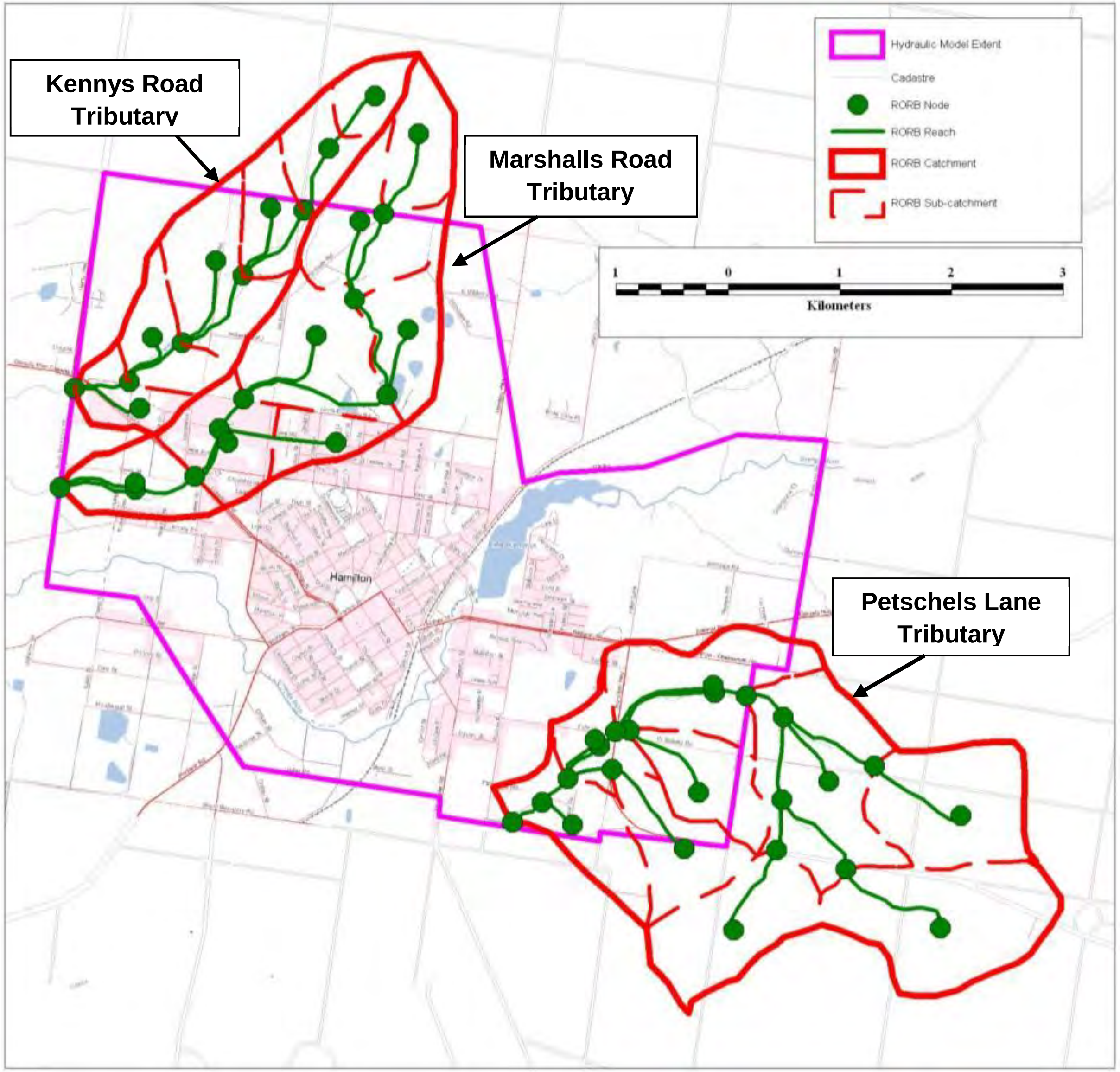



Figure 4.8 RORB models for the Petschels Lane, Marshalls Road and Kennys Road tributaries

#### *4.4.1 Model Calibration*

The model calibration process undertaken as part of this study involved a number of steps, including:

- Review the existing RORB model calibration parameters for the Grange Burn model (from Cardno, 2010).
- Adjust the adopted $k_c$ for Interstation Area 1 as a starting point for the Hamilton RORB model calibration.
- Calibrate the Hamilton RORB model for the September 1983 and September 1984 flood events.

##### *4.4.1.1 Review of existing Grange Burn model*

To calibrate the Grange Burn RORB model, three events were selected for which rainfall, pluviograph and hourly flow data were available. These events include September 1983, September 1984 and August 2004. Calibration was carried out at two locations, Hamilton and Morgiana. Instantaneous stream flow data from stations 238239 (Hamilton) and 238219 (Morgiana) were obtained from the Victorian Water Resources Data Warehouse. Daily rainfall totals were taken from Hamilton Research Station (90103) and Mountajup (89022) and used to estimate the total storm volume for each event. The Casterton Showground (90135) pluviograph (6 min) data was used as a basis for the temporal distribution of the total rainfall volume for each event.

It should be noted that there is no pluviograph station within a reasonable distance from Hamilton catchment and as such any temporal distributions applied are likely to introduce inaccuracies and uncertainty to the calibration process. It is recommended that a pluviograph station be implemented in the region around Hamilton to ensure future flood studies can distribute daily rainfall to a 6 min timestep. It is understood that half hourly rainfall data is available from Hamilton Airport, however this rainfall data has only been recorded for recent years and does not overlap the Grange Burn at Hamilton streamflow record.

The results of the calibration for each storm event are found in Appendix B along with the actual and modelled hydrographs at each gauge. It should be noted that the 2004 event had no gauging data at Hamilton. This event was used as a cross check for the $k_c$ parameter chosen to represent the Morgiana gauge. The main change in the calibration was due to the modification of the Grange Burn model and this resulted in the $k_c$ for Grange Burn increasing from 35 to 40. The loss rates for each storm event were also modified.

The main information that was taken from the Grange Burn model was the $k_c$ for the Hamilton Interstation area ($k_c$ of 25) and this was used as a starting point for the calibration of the Hamilton specific RORB model.

##### *4.4.1.2 Hamilton RORB Model*

The Hamilton RORB model was calibrated to the September 1983 and September 1984 flood events (the same events as for the Grange Burn RORB model). These are the only events where there is concurrent pluviograph, rainfall, and streamflow data.

The primary focus of the calibration is to determine an appropriate $k_c$ value for the Hamilton catchment. Within RORB there are suggested $k_c$ parameters for various regions across Australia. The suggested range of $k_c$ parameters varies considerably but the standard formulas from RORB do provide a working range for $k_c$. For the Hamilton catchment the recommended values are:

- Upstream of Grange Burn at Hamilton
  - Grange Burn RORB model – 25.0
  - Eqn 2.4 RORB Manual – 32.4
  - Victorian Region MAR < 800 mm – 16.2

The calibrated parameters are summarised in Table 4.10. The calibrated $k_c$ at Hamilton was higher than for the Grange Burn at Morgiana catchment calibration (a $k_c$ of 28 for Hamilton compared to a $k_c$ of 25 for the Grange Burn at Morgiana caliobration) but some adjustment must be made for the change in average reach length ($D_{av}$) as this parameter interacts with the $k_c$ parameter for the runoff routing within the catchment. The relationship shown in Equation 1 shows that the change in $k_c$ parameter is directly proportional to the change in $d_{av}$.

$$\frac{D_{av\,1}}{D_{av\,2}} = \frac{k_{c\,1}}{k_{c\,2}} \qquad \text{Equation 1}$$

Where:

$D_{av\,1}$ = Average stream length for the Hamilton RORB Model (14.09 km).
$k_{c\,1}$ = $k_c$ parameter for the Hamilton catchment.
$D_{av\,2}$ = Average stream length for Interstation area 1 from the Grange Burn RORB Model (13.08 km).
$k_{c\,2}$ = $k_c$ parameter for Interstation area 1 from the Grange Burn RORB Model (25).

Based on Equation 1, the adjusted $k_c$ for the Hamilton catchment from the Grange Burn model would be around 27 to account for the change in $D_{av}$. The final calibrated $k_c$ was 28 for the Hamilton catchment, which is slightly higher than for the Grange Burn model calibration but essentially very close to the same calibrated parameters obtained for the Grange Burn RORB calibration. This $k_c$ value also lies within the expected range of the AR&R (Eqn 2.4) predicted $k_c$ of 32.4 and the Victorian region MAR < 800 mm estimated $k_c$ of 16.2.

Table 4.10 Hamilton RORB Model Calibration Parameters

| Event | $k_c$ | m | IL (mm) | CL (mm) | Comments |
|---|---|---|---|---|---|
| September-1983 (extended) | 28 | 0.8 | 13.0 | 0.12 | A good calibration was obtained, the kc was increased to match the peak hydrograph shape more appropriately. |
| September-1983 (short) | 28 | 0.8 | 12.0 | 0.12 | This run modelled the large peak in isolation. The initial loss was reduced but the calibration was good. |
| September-1984 | 28 | 0.8 | 0.0 | 0.50 | Although this event is smaller it is reasonably calibrated. |

Table 4.11 September 1983 (extended) calibration results

| Hamilton | Hydrograph | | Error | |
|---|---|---|---|---|
| | Modelled Value | Recorded Value | Absolute Difference | Percentage Difference |
| Peak discharge,m³/s | 104.9 | 105.2 | -0.3 | -0.3 |
| Time to peak,h | 88 | 87 | 1 | 1.1 |
| Volume,m³ | 1.32E+07 | 1.14E+07 | 1.86E+06 | 16.4 |
| Time to centroid,h | 71.7 | 73.5 | -1.7 | -2.3 |
| Lag (c.m. to c.m.),h | 14.5 | 16.3 | -1.7 | -10.5 |
| Lag to peak,h | 30.8 | 29.8 | 1 | 3.4 |

Table 4.12 September 1983 (short) calibration results

| Hamilton | Hydrograph | | Error | |
|---|---|---|---|---|
| | Modelled Value | Recorded Value | Absolute Difference | Percentage Difference |
| Peak discharge,m³/s | 104.9 | 105.2 | -0.3 | -0.3 |
| Time to peak,h | 38 | 39 | -1 | -2.6 |
| Volume,m³ | 7.88E+06 | 8.38E+06 | -4.97E+05 | -5.9 |
| Time to centroid,h | 47.7 | 42.1 | 5.6 | 13.3 |
| Lag (c.m. to c.m.),h | 13.4 | 7.8 | 5.6 | 71.2 |
| Lag to peak,h | 3.75 | 4.75 | -1 | -21 |

Table 4.13 September 1984 calibration results

| Hamilton | Hydrograph | | Error | |
|---|---|---|---|---|
| | Modelled Value | Recorded Value | Absolute Difference | Percentage Difference |
| Peak discharge,m³/s | 22.59 | 22.07 | 0.52 | 2.4 |
| Time to peak,h | 50 | 49 | 1 | 2 |
| Volume,m³ | 2.05E+06 | 2.34E+06 | -2.96E+05 | -12.6 |
| Time to centroid,h | 52.7 | 51.5 | 1.2 | 2.3 |
| Lag (c.m. to c.m.),h | 14.1 | 12.9 | 1.2 | 9.1 |
| Lag to peak,h | 11.4 | 10.4 | 1 | 9.6 |

A good calibration was reached at Hamilton for the September 1983 and 1984 events. For the September 1983 event two runs were calibrated, one which included a longer duration event and one focusing on the large peak event. Both calibrations achieved a strong calibration with the peak flow rate being predicted to within 0.3% of the observed. The timing to the occurrence of the peak was within three hours of the observed but given that the Casterton Pluviograph was used to derive the rainfall pattern for this event, this is within acceptable limits. Similarly the volume of the September 1983 event varied between the two calibration runs but they were within 15% of the observed event. The observed streamflows would typically have some residual baseflow that was not derived from the event runoff and hence would be likely to have a larger volume that the predicted event. This is the primary calibration event for the Hamilton gauge and the event was reasonably represented by the Hamilton RORB model. For the smaller magnitude September 1984 event a good calibration was reached with the peak, timing and volume all being reasonably matched by the RORB model.

Overall, the model is considered suitably calibrated to estimate flows at Hamilton for design flood purposes given that there are considerable unknowns regarding the temporal distribution of the rainfall as well as limited streamflow events for calibration. Other methods of verification of the design flows will be required as the calibration was based on limited data.

## 4.5 1946 Event Analysis

For the Grange Burn model the 1946 event was a significant event and as such as part of this project the magnitude and flood extent associated with this event was required to be estimated.

As there is no streamflow data available for the 1946 event, there was no method of developing a Hamilton specific FFA to assess the AEP of this event. An alternative method was developed to assess the AEP for the 1946 event. This process used the RFA estimate of a 0.5% AEP (see Table 4.5) for the rainfall in conjunction with the AR&R design rainfall events and the RORB model to generate a flow hydrograph for Hamilton. This hydrograph was verified through the hydraulic model against known flood heights recorded during the 1946 flood event. The results of this assessment are summarised in Section 5.5.

It should be noted that there is a lot of uncertainty in this method for developing the 1946 flood hydrograph due to the issues associated with the antecedent conditions within the catchment. The proportion of rainfall that is translated into runoff varies significantly between the summer and winter periods for the Grange Burn catchment. As the 1946 event occurred in March following the summer period it is understood from other observed rainfall events that large volumes of the rainfall is likely to have been lost during this event and not translated to runoff (i.e. see Dec 2010 where 85 mm of rain fell in 24 hours and the flows registered at Morgiana were less than a 1% AEP event (31 $m^3$/s) –Table 4.8). In contrast, the September 1983 event was caused by a rainfall volume of only 30 mm falling within a 24 hour period and this cause a peak flow of 181.9 $m^3$/s (see Table 4.8).

Estimating the runoff from the large rainfall event in 1946 is difficult and highly uncertain due to the variability in catchment response discussed. Cardno have used a range of AR&R design flow hydrographs within the developed and calibrated hydraulic model and used this to determine which flow rates replicate the 1946 flood marks. This has given some indication as to the recurrence interval of the streamflows during the 1946 event. The results of this assessment are shown in Section 5.5.

## 4.6 Hydrologic Model Results

In order to assess the variability of the design flows to the loss rates a sensitivity analysis was undertaken on the RORB model. The sensitivity analysis aims at identifying the likely variability in design flows across the expected range of initial loss (IL) and continuing loss (CL) rates.

For the Hamilton region AR&R specifies the loss rates as summarised in Table 4.14. The suggested range of initial loss rates varies considerably from 15 to 35 mm. The continuing loss has been estimated at the Second Wannon River from 17 known events at a rate of between 3.3 to 3.6 mm/h, however this catchment and gauge location is within the Grampians National Park and may exhibit different characteristics to the Hamilton catchment. The suggested loss rate for catchments south of the Great Dividing Range in Victoria is 2.5 mm/h.

Table 4.14 Predicted loss rates for Hamilton (AR&R, 1987)

| Source | Description | | Comments |
|---|---|---|---|
| AR&R – Book 2: Table 3.5 | Victoria - South and east of the Great Dividing Range | Initial Loss | 15 – 35 mm |
| | | Continuing Loss | 2.5 mm/h (median) |
| AR&R – Book 2: Table 3.1 | Second Wannon River (238214) data from 17 events. | Continuing Loss | 3.3 mm/h (median)<br>3.6 mm/h (mean) |

In order to determine if the predicted flows at Hamilton are adequately represented, a relationship between the Morgiana gauge and the Hamilton gauge was developed that compared the peak flows during the concurrent records between 1981 and 1985. The comparison uses the top 35 events at Grange Burn at Hamilton and compares this to the corresponding flow at Grange Burn at Morgiana. This analysis aims at developing a likely magnitude of the 1% AEP event at Hamilton based on the relationship between Hamilton and Morgiana. This relationship is shown in Figure 4.9.

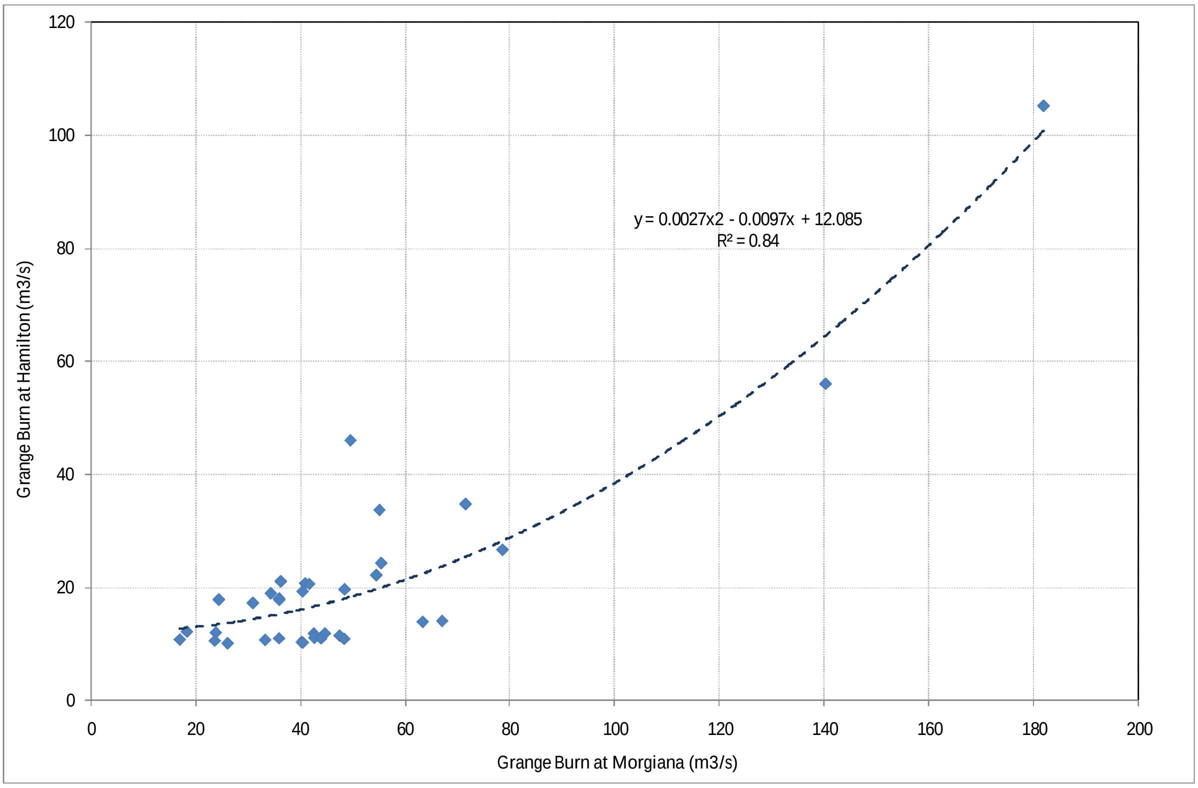



Figure 4.9 Peak flow relationship between Hamilton and Morgiana

The relationship was restricted to the peak 35 events as the primary purpose was to establish if there was a suitable relationship between Morgiana and Hamilton during large events. The data had a second order polynomial relationship fitted as this had the highest correlation to the data. A second order polynomial relationship was deemed appropriate as the relationship was only used to extrapolate to the peak 1% AEP flow estimates at Grange Burn at Morgiana of 225 to 241 $m^3/s$. It should be noted that the upper part of this relationship was fitted based on only 2 extreme events and as such has a high degree of uncertainty. The curve estimates a 1% AEP flow at Hamilton of 147 to 167 $m^3/s$. Considering that the highest recorded event at Hamilton was the 1983 flood event with a peak flow of 105 $m^3/s$ this estimate for the 1% AEP appears appropriate.

In order to cover the full variability of the loss parameters and to understand the uncertainties associated with the predicted design flows a range of loss rates were run through the RORB model. The ILs examined were 15, 25 and 35 mm as this covers the full range suggested by AR&R. The CLs covered a wider range and the values examined included 1.5, 2.5 and 3.5 mm/h. Initially only the 1% AEP event was examined at Hamilton to provide an understanding of the sensitivity of the peak flows to the losses, the results of this analysis are summarised in Table 4.15. The RORB model was run assuming unfiltered rainfall distribution, with a uniformly distributed rainfall volume across all catchments and it used the Siriwardena and Weinmann (AR&R, 1987) areal reduction factors.

Table 4.15 Predicted RORB peak flows based on a range of loss rates for Hamilton

| 1% AEP peak flow rates ($m^3/s$) | | | |
|---|---|---|---|
| Continuing Loss | Initial Loss | | |
| | 15 mm | 25 mm | 35 mm |
| 1.5 mm/h | 230.3 | 200.5 | 162.4 |
| 2.5 mm/h | 188.9 | 153.9 | 116.7 |
| 3.5 mm/h | 154.5 | 119.9 | 83.6 |

From the sensitivity analysis the loss rates that produced a 1% AEP flow estimate within the recommended range were the IL of 25 mm and CL of 1.5 to 2.5 mm/hour. These loss rates also corresponded to the median recommended design loss rates for this area from AR&R. Ultimately the 1.5 mm/hour loss rate was selected as this provides some conservatism in the estimated peak design flow rates which is important due to the uncertainties associated with the flows at Hamilton due to the poor gauged record.

Using this information and the preliminary sensitivity runs for the 1% AEP for Hamilton a detailed design run was undertaken using the IL of 25 mm and a CL of 1.5 mm/h. The results of the detailed sensitivity analysis are summarised in Table 4.16. The results show the predicted range of peak flows which occur as a result of the expected range of initial and continuing loss rates.

Table 4.16 Predicted RORB peak flows at Hamilton based on the selected Hamilton loss rates

| AEP (%) | IL / CL |
|---|---|
| | 25 mm / 1.5 mm/h |
| 20% | 35.7 |
| 10% | 57.1 |
| 5% | 96.8 |
| 2% | 153.2 |
| 1% | 200.5 |
| 0.5% | 241.0 |
| 0.2% | 314.7 |

The final selected $k_c$, m and loss rates are summarised in Table 4.17.

Table 4.17 Final RORB parameters for Hamilton

| Location | $k_c$ | m | Initial Loss | Continuing Loss |
|---|---|---|---|---|
| Grange Burn at Hamilton | 28 | 0.8 | 25 mm | 1.5 mm/h |

### *4.6.1 Estimated AEPs for Hamilton Events*

The estimated AEPs for the Hamilton and Morgiana catchments are summarised in Table 4.18. The AEPs are predicted from a variety of sources. The AEPs for the historic events were requested to be generated by the GHCMA to provide some guidance as to the relative magnitude of past flood events. The Morgiana AEPs are estimated directly from the FFA developed on the Morgiana gauge for the full flow record. These AEPs have a reasonable basis with four distributions fitted to the data, the LPIII distribution was used to predict the recurrence intervals in Table 4.18.

The Hamilton AEPs for the key events have been predicted from a range of sources. The 1946 event AEP estimate was developed as a result of the RFA as there were no streamflow records that captured this event and as discussed, there was a high degree of uncertainty for the relationship between the rainfall volume and the expected runoff due to the antecedent conditions. The design flows were used to estimate the peak flow for this event with the 0.2% AEP design flow being approximately 275 m$^3$/s. The peak flow rates were verified through the use of the hydraulic model as the analysis of the Grange Burn catchment shows that the relationship between rainfall depths and runoff is highly dependent on antecedent conditions. The known flood heights were used to modify the flow estimate and AEP as appropriate. See Section 5.5.

The 2004, 2010 and 2011 events were assessed in a similar way as these events had their AEPs predicted from the Morgiana gauge and the flows were estimated from the design events for Hamilton. The hydraulic model was used to verify the flow rates against the known flood heights.

Table 4.18 Predicted AEP of events in 1946, 1983, 1984, 2004, 2010 and 2011

| Event | Peak Flow Hamilton (m$^3$/s) | Predicted AEP [%] | Source | Peak Flow Morgiana (m$^3$/s) | Predicted AEP (LPIII) [%] | Source |
|---|---|---|---|---|---|---|
| Mar 1946 | 274.8 [1] | 0.2 % | Hamilton RFA | | | |
| Sep 1983 | 105.2 | 2.4 % | Design Events | 181.9 | 2.1 % | Morgiana FFA |
| Sep 1984 | 33.6 | 14.3 % | Design Events | 55.2 | 33 % | Morgiana FFA |
| Aug 2004 | 76.9 [1] | 4.5 % | Morgiana FFA & Design Events | 157.8 | 32 % | Morgiana FFA |
| Dec 2010 | 14.4 [2] | 50 % | Fitted Relationship [2] | 31.2 | 50 % | Morgiana FFA |
| Jan 2011 | 14.7 [2] | 50 % | | 32.8 | 50 % | Morgiana FFA |

[1] Assessed in the hydraulic model to verify the flow rates produce known flood heights.

[2] FFA was not appropriate to develop a 50% AEP event flow rate as the FFA was fitted excluding lowest 10 events and matched the low events poorly (this was undertaken to provide more accurate representation of the more extreme events). The fitted relationship in Figure 4.9 was used instead but these flow estimates should be considered highly uncertain.

### *4.6.2 Design Flows*

Based on the analysis and discussion undertaken in Section 4.6, the IL and CL for the design runs were set at 25 mm and 1.5 mm/h respectively. The RORB model was run assuming unfiltered rainfall distribution, with a uniformly distributed rainfall volume across all catchments and it used the Siriwardena and Weinmann areal reduction factors. The resulting design flows at Grange Burn at Hamilton are summarised in Table 4.19.

Table 4.19 Design flows for Grange Burn at Hamilton

| AEP | Grange Burn upstream of Lake Hamilton | |
|---|---|---|
| | IL 20 mm / CL 1.5 mm/h | Duration |
| 20% | 35.7 | 72 h |
| 10% | 57.1 | 72 h |
| 5% | 96.8 | 72 h |
| 2% | 153.2 | 72 h |
| 1% | 200.5 | 72 h |
| 0.5% | 241.0 | 72 h |
| 0.2% | 314.7 | 36 h |

The critical duration for the 5 to 0.2% AEP events was caused by the 36 and 72 hour duration design events. The IL of 25 mm and CL of 1.5 mm/h scenario predicted the 1% AEP at 200.5 m$^3$/s. From previous reports the 1% AEP flow rate was estimated to be 200 m$^3$/s at Lake Hamilton by GHD (1987) as part of the Lake Hamilton spillway project and the current estimate correlates well with this peak flow.

Further verification will be undertaken using recorded flood heights for the 1946 and 1983 event using the hydraulic model. The verification will use the predicted hydrograph from the design events to simulate these events and the flood heights from the hydraulic model will be critically assessed against the known flood heights. This analysis will provide some guidance on the accuracy of the predicted design flood events.

In order to develop the design flows that are associated with the minor tributaries around Hamilton the $k_c$ parameter has been adjusted from the Hamilton RORB model using Equation 1. This equation uses the fact that a proportional relationship exists between the average flow path length ($D_{average}$) for each sub-catchment to the outlet and the $k_c$ parameter of a model. The $k_c$ for each of the models has been summarised in Table 4.20. The resulting design flows are summarised in Table 4.21. It should be noted that these flows are at the model outlets which are located at the downstream end of the hydraulic model, the flows from these models will be distributed throughout the hydraulic model during the model runs.

Table 4.20 Calculations to determine $k_c$ for the tributaries at Hamilton

| Parameter | Petschels Lane Tributary[1] | Marshalls Road Tributary[1] | Kennys Road Tributary[1] |
|---|---|---|---|
| $k_c$ Hamilton | 28 | 28 | 28 |
| $D_{av}$ Hamilton (km) | 16.25 | 16.25 | 16.25 |
| Area ($km^2$) | 10.7 | 5.5 | 3.0 |
| $D_{av}$ Tributary (km) | 3.6 | 3.51 | 2.31 |
| $k_c$ Tributary | 5.34 | 6.06 | 3.98 |

Table 4.21 Design flows for the Hamilton minor tributaries

| AEP | Petschels Lane Tributary[1] | Marshalls Road Tributary[1] | Kennys Road Tributary[1] |
|---|---|---|---|
| 20% | 4.9 | 2.8 | 1.8 |
| 10% | 7.8 | 4.0 | 2.5 |
| 5% | 10.8 | 5.7 | 3.4 |
| 2% | 15.8 | 8.0 | 5.0 |
| 1% | 20.7 | 10.1 | 6.3 |
| 0.5% | 25.9 | 12.4 | 7.7 |
| 0.2% | 33.5 | 15.8 | 9.8 |

[1] Flows derived at RORB model outlets, these will be distributed within the hydraulic model.

## 4.7 Probable Maximum Flood

The Probable Maximum Flood (PMF) for the Hamilton catchment was estimated using both the Generalised Short-Duration Method (GSDM) and Generalised Southeast Australia Method (GSAM) (BoM, 2003). This allows estimation of the Probable Maximum Precipitation (PMP) for durations from 15 minutes up to 96 hours and ensures that the PMF is adequately captured for Hamilton.

The factors shown in Table 4.22 and Table 4.23 were used to develop the PMP. The runoff coefficient for the PMP flows was assumed to be 1 (i.e. all rainfall becomes runoff).

Table 4.22 PMP Parameters (GSDM)

| Parameter | Value |
|---|---|
| Catchment area | 212 $km^2$ |
| Duration limit | 3 hrs |
| Portion of area considered smooth | 1 |
| Portion of area considered rough | 0 |
| Mean Elevation | ~200 mAHD |
| Elevation adjustment factor | 1 |
| Moisture adjustment factor | 0.575 |

Table 4.23 PMP Parameters (GSAM)

| Parameter | Value |
|---|---|
| Catchment area | 212 $km^2$ |
| Topographical Adjustment Factor | 1.14 |
| $EPW_{\text{summer catchment average}}$ | 57.63 |
| $EPW_{\text{autumn catchment average}}$ | 46.45 |
| $EPW_{\text{summer standard}}$ | 80.80 |
| $EPW_{\text{autumn standard}}$ | 71.00 |
| $MAF_{\text{summer}}$ | 0.713 |
| $MAF_{\text{autumn}}$ | 0.654 |

Table 4.24 shows the total estimated rainfall depth from the GSDM and GSAM for various flow durations. These flow depths were converted to hyetographs (records of rainfall depth over time) and used as storm files in RORB to estimate the peak flow at the required locations through the catchment. The temporal distribution was utilised for a coastal zone with a catchment area close to 100 $km^2$ as per the GSDM and GSAM recommendation.

Table 4.24 PMP Rainfall Estimates

| Method | Duration (hours) | Rounded PMP Estimate (mm) | Estimated PMF upstream of Lake Hamilton ($m^3/s$) |
|---|---|---|---|
| GSDM | 0.25 | 90 | 205 |
| | 0.5 | 130 | 500 |
| | 0.75 | 160 | 680 |
| | 1 | 200 | 939 |
| | 1.5 | 230 | 1139 |
| | 2 | 260 | 1344 |
| | 2.5 | 270 | 1479 |
| | 3 | 290 | 1614 |
| GSAM | 12* | 500* | 2266 |
| | 24 | 630 | 1987 |
| | 36 | 700 | 1523 |
| | 48 | 740 | 1489 |
| | 72 | 780 | 1319 |
| | 96 | 810 | 920 |

* Extrapolated from 24 to 96 hour PMP curve

The PMF event has been estimated as a 12 hour duration event with a peak flow rate of 2,266 $m^3/s$. This compares well to the estimated PMF of 2,300 $m^3/s$ estimated by GHD (1987) as part of the Lake Hamilton spillway project. Multiple durations (3 hours, 12 hour and 24 hour events) were run through the hydraulic model to ensure the full PMF flood extent was captured. See Section 5.6.7.

## 4.8 Climate Change Assessment

In order to assess the possible future impact of climate change an analysis of the impacts of increased rainfall intensities ranging from 10% to 32% was explored. This section outlines the derivation and summary of the IFD parameters applied and the associated flow rates predicted from RORB.

The selected percentage increases in rainfall intensity are inline with the Melbourne Water (MW) Technical Specifications for Climate Change within Victoria. The basis for the rainfall intensity increases is the IPCC Fourth Assessment Report. This report assumes a 5% increase in rainfall intensity for every 1 ˚C rise in average temperature. The report has an upper limit of the expected temperature rise of 6.4 ˚C which gives the upper limit to the increase in rainfall intensity of 32%.

The increased rainfall intensities of 10%, 20% and 32% correspond to increases in average temperature of 2 ˚C, 4 ˚C and 6.4 ˚C respectively and have been examined to show the incremental predicted impact of rainfall intensity on the catchment runoff rates and peak flows for the Hamilton area. The increase in temperature has been applied to correspond with the recommendations within the MW Technical Specifications. The 10%, 20% and 32% increases in rainfall intensity have been related to approximate time periods of 2030, 2060 and 2110 respectively.

It should be noted that this climate change assessment only focusses on the change in rainfall intensity and does not explicitly assess any changes in catchment antecedent conditions and predicted loss rates. Losses may change in line with the expectation that the catchment may be drier for longer between more frequent intense rainfall events.

The modified Intensity Frequency Duration (IFD) parameters are summarised in Table 4.25 for the current conditions, 10%, 20% and 32% increases in rainfall intensity.

Table 4.25 IFD Parameters (Coordinates 37.575 S, 142.200 E) for the climate change scenarios

| IFD | Current | 10% Climate Change | 20% Climate Change | 32% Climate Change |
|---|---|---|---|---|
| ${}^{2}I_{1}$ | 18.09 | 19.90 | 21.71 | 23.88 |
| ${}^{2}I_{12}$ | 3.52 | 3.87 | 4.22 | 4.65 |
| ${}^{2}I_{72}$ | 0.9 | 0.99 | 1.08 | 1.19 |
| ${}^{50}I_{1}$ | 39.63 | 43.59 | 47.56 | 52.31 |
| ${}^{50}I_{12}$ | 6.04 | 6.64 | 7.25 | 7.97 |
| ${}^{50}I_{72}$ | 1.66 | 1.83 | 1.99 | 2.19 |
| Skew | 0.49 | 0.49 | 0.49 | 0.49 |
| F2 | 4.36 | 4.36 | 4.36 | 4.36 |
| F50 | 14.73 | 14.73 | 14.73 | 14.73 |
| Zone | 6 | 6 | 6 | 6 |

The RORB models presented in Section 4.4 were used in conjunction with the revised IFD parameters to determine the revised climate change peak flows from each of the catchments. The results are presented in Table 4.26 to Table 4.29 for the Grange Burn, Petschels Lane Tributary, Kennys Road Tributary and Marshalls Road Tributary respectively.

Table 4.26 Climate change assessment for the Grange Burn hydrology

| AEP (%) | Design Flow Rates | 10% increase rainfall intensity | 20% increase rainfall intensity | 32% increase rainfall intensity | |
|---|---|---|---|---|---|
| | Grange Burn at Hamilton | Grange Burn at Hamilton | Grange Burn at Hamilton | Grange Burn at Hamilton | Increase (% from current) |
| 20% | 35.7 | 69.8 | 89.3 | 117.3 | 229% |
| 10% | 57.1 | 100.0 | 126.6 | 159.2 | 179% |
| 5% | 96.8 | 148.9 | 180.0 | 217.8 | 125% |
| 2% | 153.2 | 214.1 | 252.4 | 303.2 | 98% |
| 1% | 200.5 | 273.9 | 321.7 | 379.8 | 89% |
| 0.5% | 241.0 | 342.6 | 396.9 | 462.7 | 92% |
| 0.2% | 314.7 | 442.6 | 505.5 | 570.1 | 81% |

Table 4.27 Climate change assessment for the Petchels Lane catchment hydrology

| AEP (%) | Design Flow Rates | 10% increase rainfall intensity | 20% increase rainfall intensity | 32% increase rainfall intensity | |
|---|---|---|---|---|---|
| | Petschels Lane | Petschels Lane | Petschels Lane | Petschels Lane | Increase (% from current) |
| 20% | 4.9 | 8.1 | 9.6 | 12.0 | 145% |
| 10% | 7.8 | 10.4 | 12.8 | 15.7 | 101% |
| 5% | 10.8 | 14.7 | 17.5 | 20.7 | 92% |
| 2% | 15.8 | 20.4 | 24.0 | 28.4 | 80% |
| 1% | 20.7 | 25.9 | 30.1 | 35.1 | 70% |
| 0.5% | 25.9 | 32.0 | 36.6 | 42.3 | 63% |
| 0.2% | 33.5 | 40.7 | 46.2 | 52.1 | 56% |

Table 4.28 Climate change assessment for the Kennys Road catchment hydrology

| AEP (%) | Design Flow Rates | 10% increase rainfall intensity | 20% increase rainfall intensity | 32% increase rainfall intensity | |
|---|---|---|---|---|---|
| | Kennys Road Tributary | Kennys Road Tributary | Kennys Road Tributary | Kennys Road Tributary | Increase (% from current) |
| 20% | 1.8 | 2.4 | 2.8 | 3.5 | 94% |
| 10% | 2.5 | 3.1 | 3.7 | 4.5 | 80% |
| 5% | 3.4 | 4.3 | 5.1 | 6.1 | 79% |
| 2% | 5.0 | 6.0 | 7.0 | 8.2 | 64% |
| 1% | 6.3 | 7.5 | 8.6 | 9.9 | 57% |
| 0.5% | 7.7 | 9.1 | 10.4 | 11.9 | 55% |
| 0.2% | 9.8 | 11.4 | 12.8 | 14.6 | 49% |

Table 4.29 Climate change assessment for the Marshalls Road catchment hydrology

| AEP (%) | Design Flow Rates | 10% increase rainfall intensity | 20% increase rainfall intensity | 32% increase rainfall intensity | |
|---|---|---|---|---|---|
| | Marshalls Road Tributary | Marshalls Road Tributary | Marshalls Road Tributary | Marshalls Road Tributary | Increase (% from current) |
| 20% | 2.8 | 4.0 | 4.7 | 5.8 | 107% |
| 10% | 4.0 | 5.1 | 6.2 | 7.5 | 88% |
| 5% | 5.7 | 7.1 | 8.4 | 10.0 | 75% |
| 2% | 8.0 | 9.8 | 11.4 | 13.3 | 66% |
| 1% | 10.1 | 12.2 | 14.0 | 16.3 | 61% |
| 0.5% | 12.4 | 14.9 | 16.9 | 19.5 | 57% |
| 0.2% | 15.8 | 18.8 | 21.3 | 24.0 | 52% |

For the Grange Burn the hydrological model assessment for the climate change indicated that the magnitude shift was that the current 0.2% AEP event would be likely to become the new 2% AEP event under a 32% increase to rainfall intensity. The 1% AEP flood event increased from 200 $m^3/s$ up to 380 $m^3/s$ under the 32% climate change scenario. For the Grange Burn model the current 1% AEP event increased to be greater than the 0.2% AEP event due to a 32% increase in rainfall intensity.

The remaining events showed increased peak flow rates of between 81% to 229%. It is important to note that the percentage increase of peak flow rates for the smaller, more frequent events was larger than for the rarer more extreme events. For example for the Grange Burn catchment the 20% AEP event is expected to increase by 229%, whereas the 0.2% AEP event is expected to increase by 81%. This trend is observed for the smaller tributaries as well. This indicates that the more frequent events are expected to increase in magnitude at a greater rate that the large extreme flood events.

For the three tributary models the percentage increase due to the 32% increase in rainfall intensity was a 49% to 145% increase in peak flow rates. For these models the current 1% AEP event increased to be approximately the equivalent of the 0.2% AEP event due to a 32% increase in rainfall intensity.

The detailed hydraulic assessment of the climate change scenarios are summarised in Section 5.8.

## 4.9 Summary of Selected Flows

Overall limited data availability makes producing a reliable estimate of the design flows for the Hamilton Flood Investigation difficult. A number of methods were explored to determine the appropriate flows for Hamilton, including rainfall frequency assessment, flood frequency assessment and regional regression relationships with downstream gauges. The result of this analysis is shown in Table 4.30. These flows will be verified using the hydraulic model and flood marks to reduce uncertainty around the design flow estimates.

Table 4.30 Design flows for the hydraulic model at Hamilton

| AEP (%) | Design Flow Rates | | | |
|---|---|---|---|---|
| | Grange Burn at Hamilton | Petschels Lane Tributary[1] | Marshalls Road Tributary[1] | Kennys Road Tributary[1] |
| 20% | 35.7 | 4.9 | 2.8 | 1.8 |
| 10% | 57.1 | 7.8 | 4.0 | 2.5 |
| 5% | 96.8 | 10.8 | 5.7 | 3.4 |
| 2% | 153.2 | 15.8 | 8.0 | 5.0 |
| 1% | 200.5 | 20.7 | 10.1 | 6.3 |
| 0.5% | 241.0 | 25.9 | 12.4 | 7.7 |
| 0.2% | 314.7 | 33.5 | 15.8 | 9.8 |
| PMF | 2,266 | 215.7 | 91.8 | 61.6 |

[1] Flows derived at RORB model outlets, these will be distributed within the hydraulic model.

# 5 HYDRAULIC MODELLING

## 5.1 Hydraulic Model Establishment

The WL|Delft 1D2D modelling system, SOBEK, was used to compute the channel (1D) and overland flow (2D) components of the study. SOBEK is a professional software package developed by WL|Delft Hydraulics Laboratory, which is one of the largest independent hydraulic institutes in Europe (situated in The Netherlands) and is world-renowned in the fields of hydraulic research and consulting (WL|Delft, 2005).

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 automatically coupled at 1D-calculation points (such as manholes) whenever they overlap each other. The model commences with the 1D component operating as the inflow increases until such time as the pipe or channel is full and overflows, with the flow then moving to the 2D domain. The 1D network and the 2D grid hydrodynamics are solved simultaneously using the robust Delft scheme that handles steep fronts, wetting and drying processes and subcritical and supercritical flows (Stelling et al., 1999).

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. This system employs a unique implicit coupling between the one and two-dimensional hydraulic components that provides high accuracy and stability within the computation.

## 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 Hamilton Flood Investigation the Grange Burn was the primary River, however the other minor tributaries were modelled within the Hamilton Study Area. For this reason three distinct models were created to manage model size and computational run times. This was permissible as it is known that there are no cross catchment flows between the Grange Burn and any of the modelled tributaries occur during major flood events. This can be observed from assessing the topography of the Hamilton Study area.

The three models are shown in Figure 5.1. Creating three stand-alone models provides the following benefits:

- Decreases the required model grid sizes, which decreases model run times and size of result files.
- Allows for the Grange Burn model to be calibrated independently of the tributaries.
- Future assessment of specific areas around Hamilton can be explored in a targeted way through the use of the site specific model.

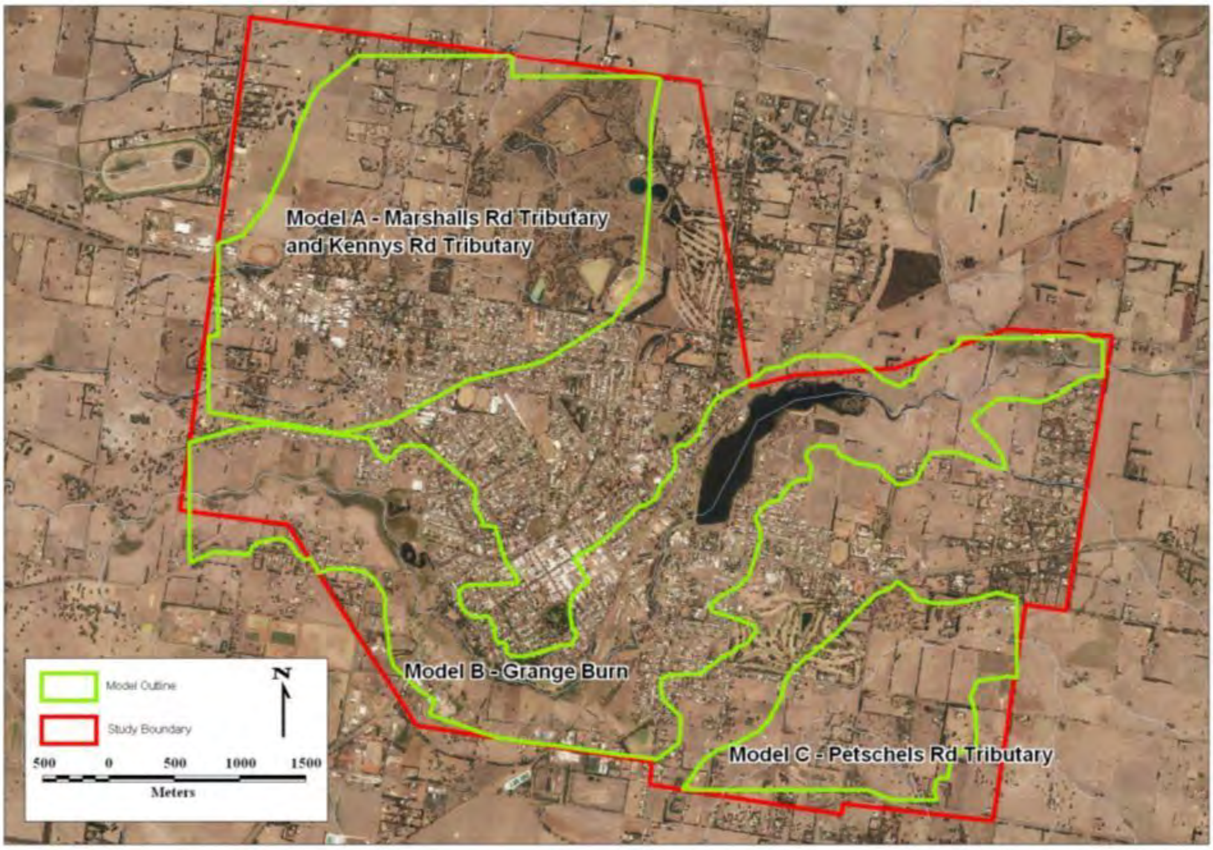



Figure 5.1 Hydraulic model boundaries for the Hamilton flood investigation

### *5.2.1 Channel and Structure System (1D)*

Survey was undertaken on specific structures within the Hamilton study area. This information is summarised in Section 3.3. This information was used to define the structures within the 1D domain of the hydraulic models that required specific representation within the simulation software. All structures were represented in the 1D domain and the locations of these structures is summarised in Figure 5.2.

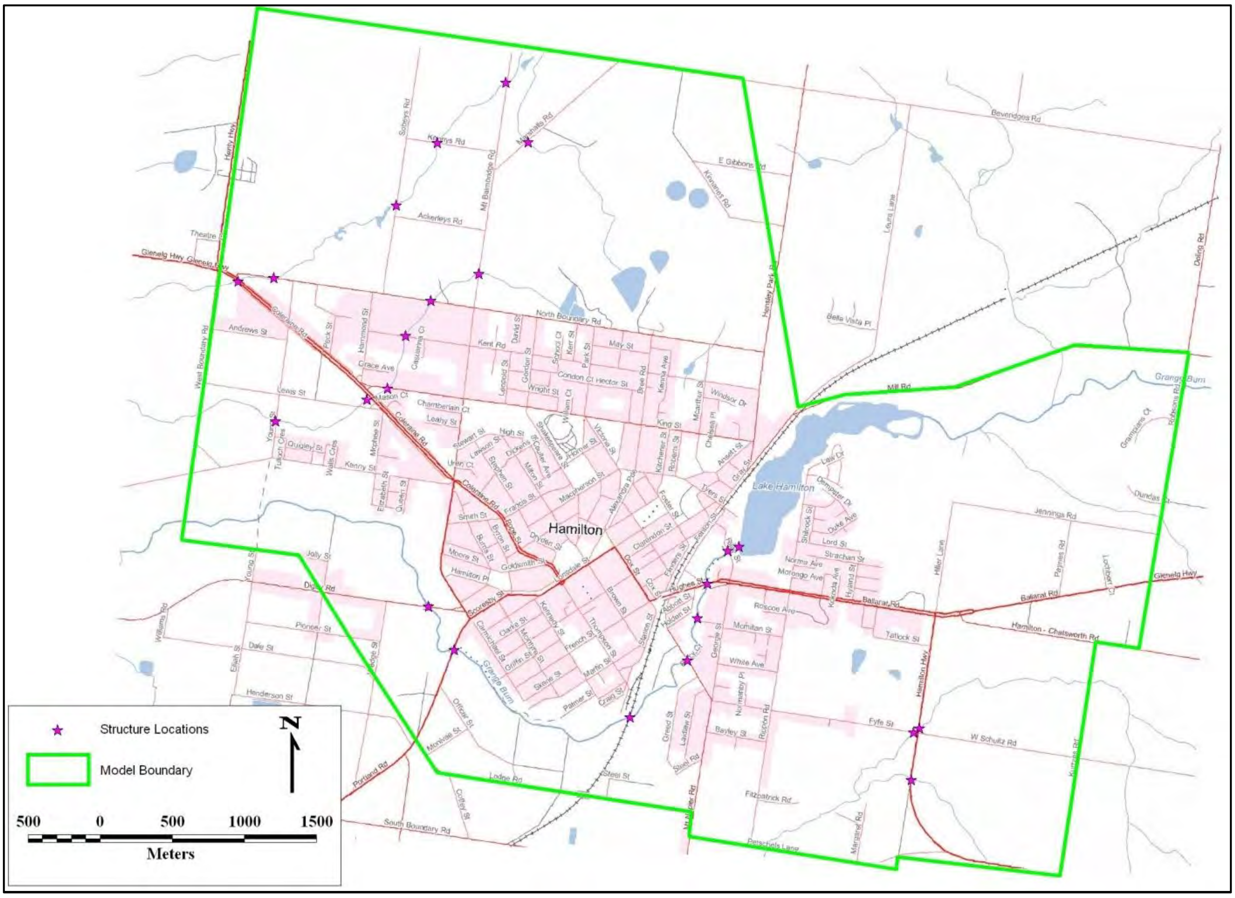



Figure 5.2 Structure locations within the hydraulic models

### *5.2.2 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.

The dimensions of the grids are summarised in Table 5.1. Each topography layer was set using the same grid cell size of 5m x 5m as this provided enough detail to capture the surface elevation details without causing computation run times and size of results to be excessive. The DTMs are shown in Figure 5.5.

The grid cell size selected was the finest detail possible without causing runtimes of multiple days while also replicating the known surface appropriately. Major bridge openings along the Grange Burn are approximately 25 to 30 m wide at most locations and this corresponds to 5 to 6 grid cells within the model. This is an adequate number of gridcells for the interaction at critical infrastructure.

An example cross section is shown in Table 5.2 and Table 5.3 shows that for the full flood plain cross section the 5 m grid cell is appropriate given that the floodplain is over a hundred metres (> 20 grid cells) wide and the main channel is approximately 40 m wide (8 grid cells). This grid cell size allows the main channel and the main flood plain to be well represented within the model.

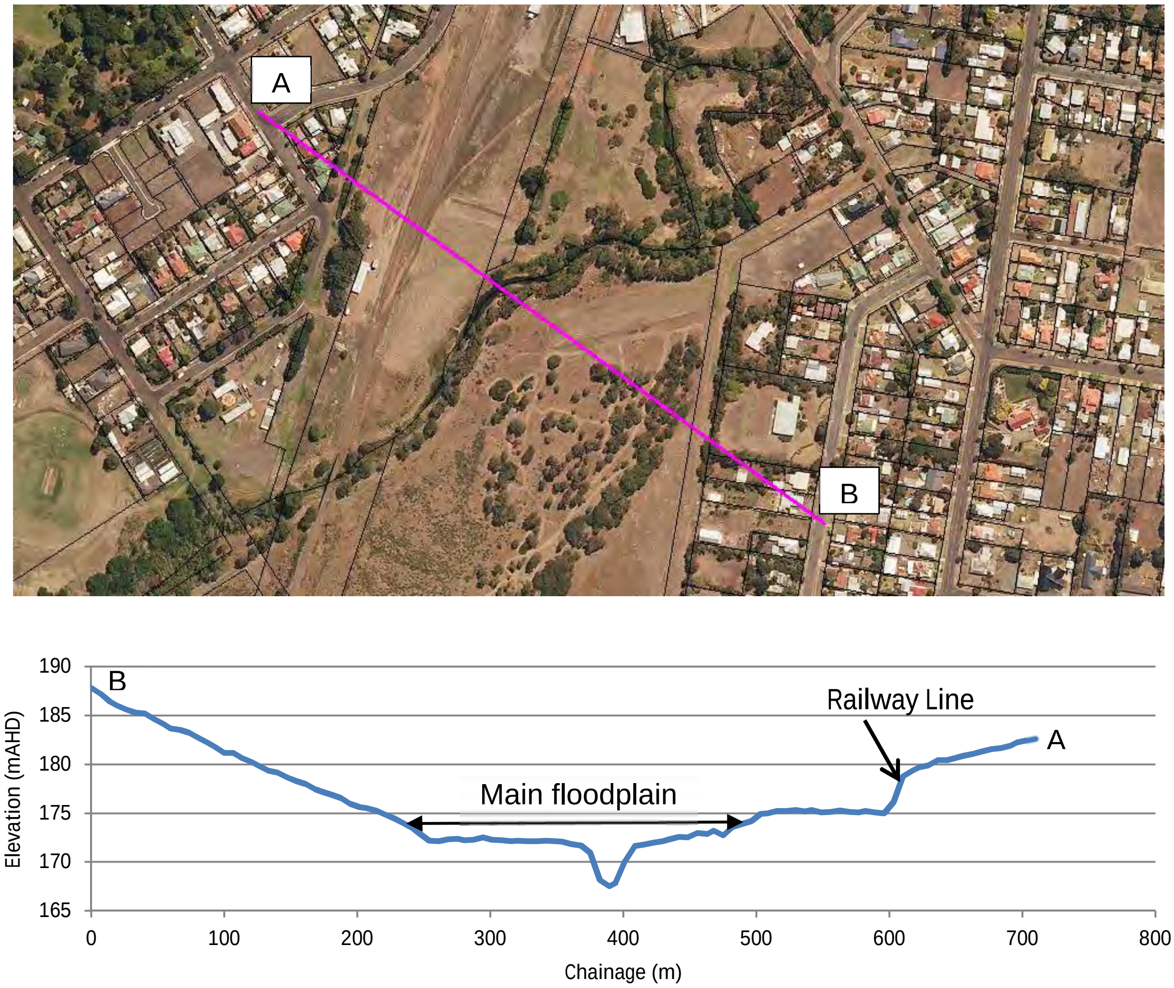



Figure 5.3 Example cross section for the Grange Burn catchment

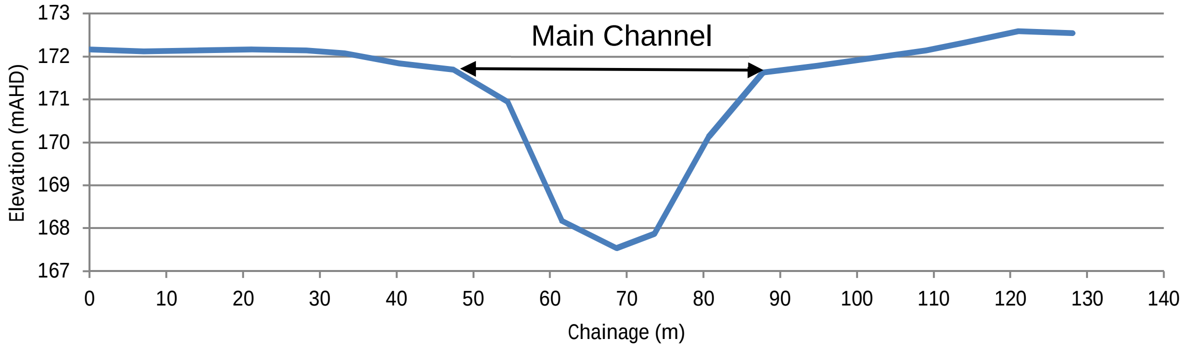



Figure 5.4 Example cross section for the main channel of the Grange Burn catchment

Modifications to the topography along the river in the 2D grid were undertaken to account for the fact that when the LiDAR was flown it could not capture the river bed level due to the existing water level. This leads to the reported surface elevation in the LiDAR being higher than reality which removes storage within the floodplain and creates inconsistent levels between the 2D bed level and the 1D structures. The reduction was based on the understanding of the water levels when the LiDAR was flows as well as the GHD survey information from the Hamilton Spillway study (1987). Cross checks were made between the main channel shape within this study and survey undertaken within the Hamilton Spillway Study (GHD, 1987). These cross checks and adjustments to the LiDAR ensured that the model was representing the channel and floodplain accurately.

Table 5.1 Topography grid size

| Parameter | Grid A | Grid B | Grid C |
|---|---|---|---|
| Cell size | 5m x 5m | 5m x 5m | 5m x 5m |
| Grid Cells (x direction) | 915 | 1422 | 638 |
| Grid Cells (y direction) | 890 | 664 | 406 |

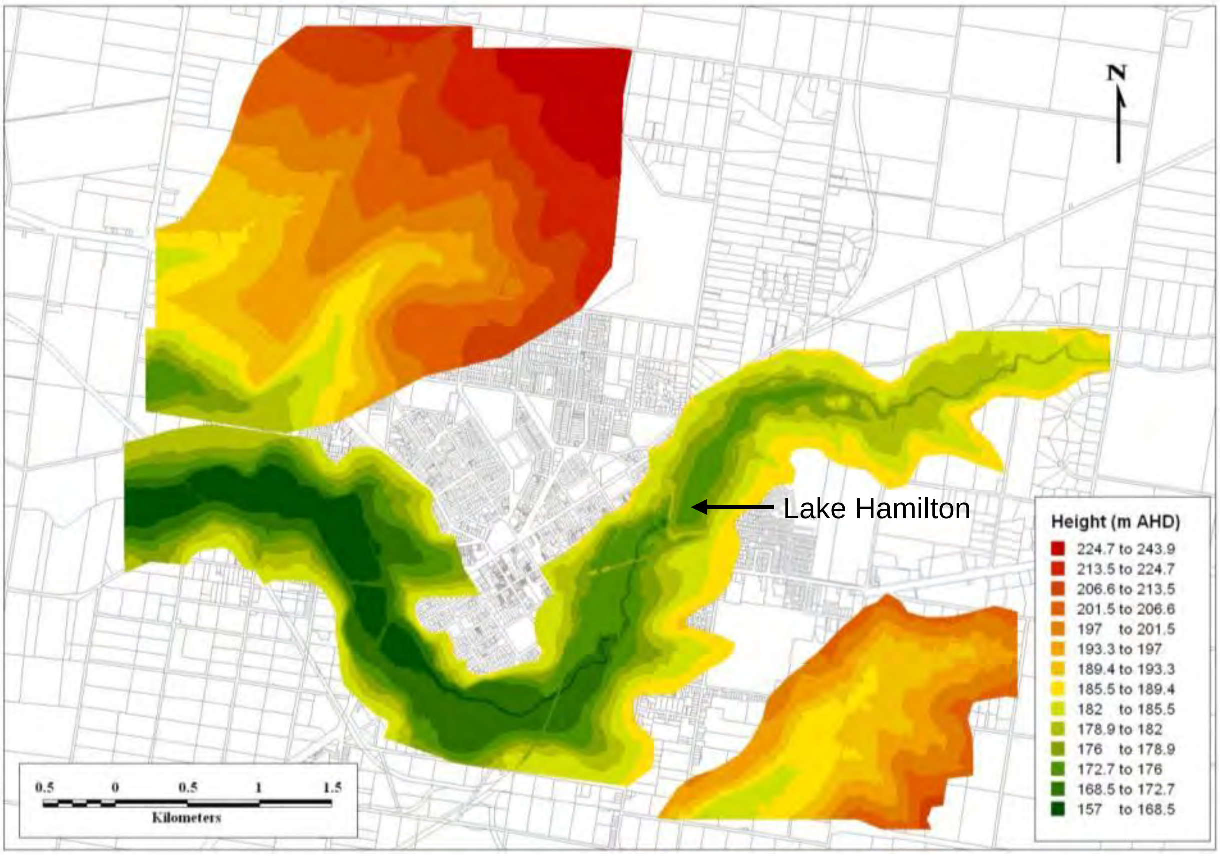



Figure 5.5 Digital Terrain Models (DTM) for Models A, B and C

### *5.2.2.1 Lake Hamilton*

Lake Hamilton is a large feature within the Grange Burn system and has been the focus of previous studies, namely the Study of Lake Hamilton Spillway / Grange Burn Flooding Investigation (GHD, 1987). As Lake Hamilton was full of water during the LiDAR capture the reported ground surface elevations within this area are not captured. The depth of Lake Hamilton was adjusted so that there was sufficient depth for SOBEK to replicate the frictions that would be typically experienced across the lake. Lake Hamilton will be started full in each of the scenarios so these assumptions regarding the bathymetry of Lake Hamilton will not adversely impact the model runs, and in particular will not volumetrically impact the model runs. Lake Hamilton is not used as a flood mitigation structure and as such is often maintained as close as possible at full level as this provides the greatest amenity value. During flood events during drier periods there may be additional storage available within Lake Hamilton but this study is examining planning controls and emergency response and as such is based on Lake Hamilton being at full capacity for the simulations.

Lake Hamilton was not included within the hydrological model within RORB (the Hamilton RORB model). The inflows are input into the hydraulic model upstream of Lake Hamilton and this ensures that no double counting of volume and inflows occurs.

One difference between the GHD (1987) report and the current topography was the height of the Lake Hamilton embankment. The embankment in the GHD report was assumed to be 180 mAHD, whereas the embankment within the current model was found to be within 179.5 and 179.75 mAHD. It should be noted that the maximum elevations for the embankment were extracted from the 1 m LiDAR elevation data sets to ensure the top of the embankment was accurately captured. This difference of between 300 and 500 mm between the GHD report and the current LiDAR implies that the dam wall may be overtopped in the current design runs sooner than GHD predicted in the 1987 report.

Historically Lake Hamilton was designed assuming a peak flow rate of 142 $m^3/s$ which was the estimate for the 0.001% AEP event. In addition the peak flow rate that the spillway was designed for was 250 $m^3/s$ when Lake Hamilton was at bank full capacity. From the GHD study and the current investigation we know the 0.001% AEP event is much larger than the 1987 estimate and that the Lake Hamilton spillway and embankment is not sufficient to pass the 0.001% AEP event without the embankments overtopping. GHD estimated that if the embankment was at the 180 mAHD level then the 1% AEP event could be passed by Lake Hamilton with approximately 300 mm freeboard remaining in the lake (GHD estimated a peak flow rate at the 1% AEP was 200 $m^3/s$).

The main difference between the current model for the Grange Burn and the GHD model (1987) is that the Lake Hamilton embankment is now shown to be between 300 and 500 mm lower than the stated 180 mAHD using the LiDAR captured as part of this study. This implies that the current 1% AEP estimate of 200 $m^3/s$ may overtop Lake Hamilton, whereas in the GHD study the 1% AEP was successfully passed by the Lake Hamilton spillway.

## 5.3 Hydraulic Model Calibration Setup

The hydraulic model was calibrated using the known flood heights as supplied from the GHCMA, GHD report (1987) and VicRoads design plans. Two events were calibrated and validated as part of this process, the 1983 and 2010 events. The 1983 event was the primary calibration event as this was a reasonably large flood event and also had a large number of recorded flood heights from the GHCMA and the GHD report (1987). Importantly this event had a peak flow rate recorded at the upstream gauge at Robsons Road. This was the only event in the record which had a known peak flow rate coupled with recorded flood heights. The known flow rate and known levels allows for the model to be calibrated accurately to this event.

Following the 1983 event the 2010 event was used to verify the final calibrated model to ensure that the model was accurately representing the full range of flood events. Each event is presented in the following sections. It is important to note that the reliability of the recorded flood heights is unknown and often this information has been captured post-event via debris lines and water marks on structures. It is for these reasons that there can be some inconsistencies between these recorded flood heights, especially from varied sources.

Cardno was unable to use the August 2004 flood event as a calibration or validation event as the only information available was photographs (see Appendix D). There was no known flow rate for the event and no recorded levels to match the model to. The photographs were not date and time stamped so could not be linked to peak depths and extents. If the flow rate for this event was known then an attempt could have been made to replicate some of the levels in the photography but this was not the case and the 2004 event was not used.

For this study oblique aerial photography was only available for the 2004 flood event from the GHCMA. As stated above this event did not have an associated flow rate and was not used as part of the calibration and validation of the hydraulic model.

### 5.3.1 1983 Event

The 1983 flood event occurred in August and had a peak flow rate of 105 m³/s at the Robsons Road gauge. This is one of the few events in the Hamilton catchment where the flow rate has been recorded. The recorded flood heights used for the calibration were supplied by the GHCMA and also extracted from the GHD (1987) report. Some inconsistencies were noted between the GHCMA and GHD reported flood peaks. These inconsistencies were noted at:

- Mt Napier Road Bridge – GHD 172.2 mAHD & GHCMA 172.1 mAHD – *Difference 0.10 m.*
- Portland Road Bridge – GHCMA recorded two values 169 mAHD & 168.96 mAHD – *Difference 0.04 m.*
- Dartmoor Hamilton Road Bridge – GHCMA recorded two values 168.32 mAHD and 168.3 mAHD – *Difference 0.02 m.*

For these locations the bold and underlined recorded flood heights were used, however it should be noted that the differences in peak flood height are quite small with the maximum difference being 0.10 m. It should also be noted that some recorded peak flood levels are not consistent within this event, for example the upstream and downstream Ballarat Road levels are 174.1 and 173.95 mAHD respectively, whereas the next downstream location states a level of 173.8 mAHD between Ballarat Road and the Weir. Then at the Weir itself the level is back up at 174 mAHD. It is unlikely that the peak flood level decrease and subsequently increase with the span of 200 m by ~20 cm. This does not imply the levels cannot be used but merely that there is some uncertainty in the exact levels reached during the flood event.

Table 5.2 and Figure 5.6 shows the locations of the recorded flood peaks, their source and the final comparison of the recorded peaks versus the modelled 1983 flood peaks.

Table 5.2 Hydraulic model calibration for the August 1983 flood event

| Location | Source | Recorded value | Recorded peak flood depth (mAHD) | Model peak flood depth (mAHD) | Difference (m) |
|---|---|---|---|---|---|
| U/S Ballarat Rd | GHCMA | 174.10 mAHD | 174.10 | 174.04 | -0.06 |
| D/S Ballarat Rd | GHCMA | 173.95 mAHD | 173.95 | 173.87 | -0.08 |
| Between Ballarat Rd & Weir | GHCMA | 173.80 mAHD | 173.80 | 173.93 | 0.13 |
| U/S weir | GHCMA | 174.00 mAHD | 174.00 | 173.91 | -0.09 |
| D/S weir | GHCMA | 173.70 mAHD | 173.70 | 173.78 | 0.08 |
| Apex Park BBQ shelter | GHD | 174.15 mHSA | 173.84 | 173.82 | -0.02 |
| Apex park toilet block | GHD | 174.07 mHSA | 173.76 | 173.75 | -0.01 |
| 4 Holden Street | GHD | 174.27 mHSA | 173.96 | 173.84 | -0.12 |
| Mt Napier Rd Bridge | GHD | 172.50 mHSA | 172.19 | 172.41 | 0.22 |
| Portland Rd Bridge | GHCMA | 169.00 mAHD | 169.00 | 169.16 | 0.16 |
| Dartmoor-Hamilton Rd Bridge | GHCMA | 168.32 mAHD | 168.32 | 168.57 | 0.25 |
| | | | | Average model difference | 0.042 |

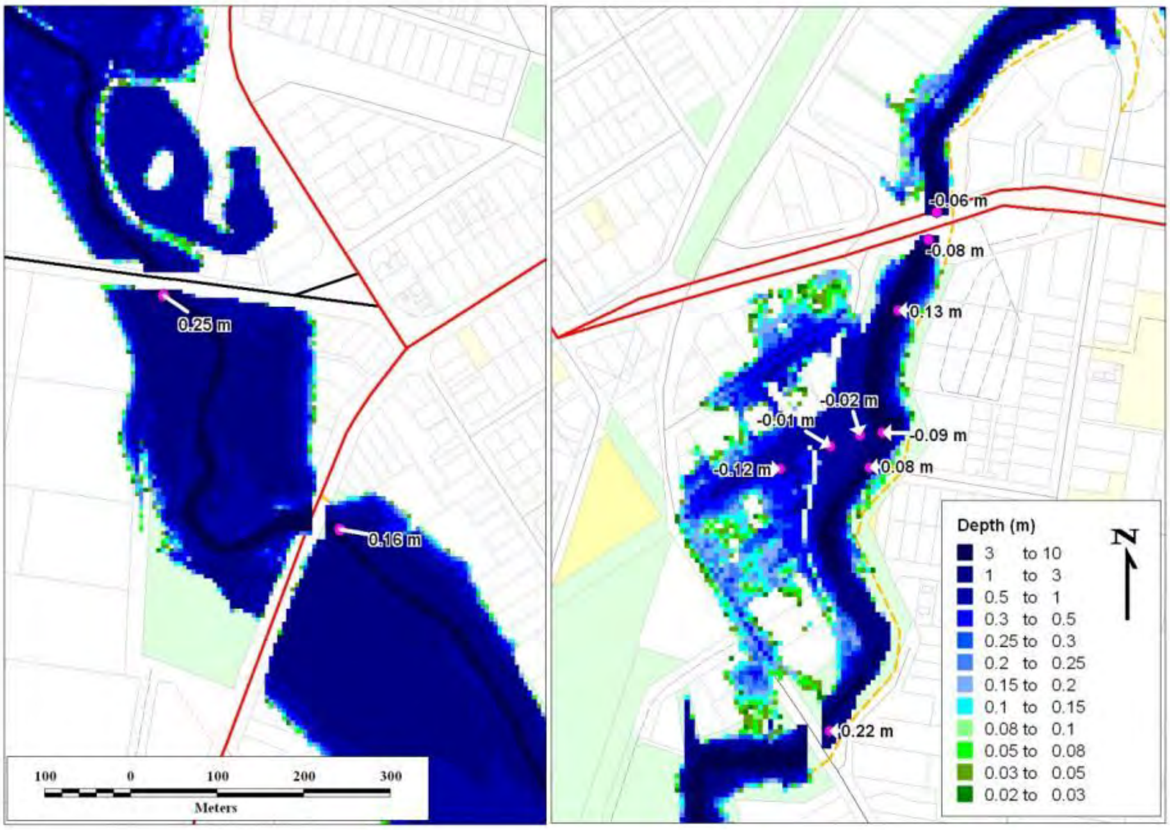



Figure 5.6 Hydraulic model calibration for the 1983 flood event

The calibration was achieved primarily through modification of the model roughness. Structure losses were also modified to replicate expected head losses throughout the model. Structures were assumed to have no blockage factor as it was unknown if any structures were blocked during this event. The overall calibration was reasonable with the average difference between the model and the recorded flood heights being 0.04 m. All flood depths were between -0.12 m and +0.25 m which is an acceptable calibration range given the uncertainties of the model and the recorded flood heights. The majority of the recorded flood heights were in the Apex Park area and the model predicts the majority of these levels well.

During this flood event the following roads were inundated:

- Abbott Street
- Holden Street
- Apex Drive
- Mount Napier Drive

No other roads were inundated in the calibrated event. Properties on Holden Street and Abbott Street were inundated by water. It should be noted that no sandbagging was included in the model as it is unknown where and when this sandbagging occurred.

The two flood photographs of this event, Figure 5.7 and Figure 5.8, seem to match the flood extents produced by the model, however as these photographs are not time stamped there is some uncertainty as to whether these were captured at the peak of the event. As previously stated the sand bagging that saved a number of properties was not modelled but this would be expected to have a minimal impact on the flood behaviour and flood shape.


Figure 5.7 Flooding during the August 1983 event at Apex Park


Figure 5.8 Flooding during the August 1983 event (Holden Street)

### 5.3.2 2010 Event

The final calibration event was the 2010 event. This event had a large number of calibration points, however as for the 1946 event there was no known peak flow rate or flood hydrograph. This event was also a much lower magnitude than the 1946 and 1983 flood events. A known peak flood height was recorded near the Grange Burn at Hamilton Robsons Road gauge and this peak water level has been used to set the upstream boundary of the model. This inputs the approximate peak flow rate into the model at a steady state, the main concern with this method is that if there is any error or uncertainty in the recording of the peak flood heights upstream of Lake Hamilton then this will introduce errors into the assumed peak flow rate.

The model was run solely using the parameters setup from the 1983 calibration event. This was a verification event only. The 2010 event was used as a validation event to test the calibration that was achieved under the 1983 flood event. The performance of this validation are summarised in Figure 5.9 and Table 5.3.

Table 5.3 Hydraulic model calibration for the 2010 flood event

| Location | Source | Recorded peak flood depth (mAHD) | Model peak flood depth (mAHD) | Difference (m) |
|---|---|---|---|---|
| Behind bluestone building at Pretonholme Nursery | GHCMA | 181.31 | 181.32 | 0.01 |
| Across from Abbot St. intersection | GHCMA | 172.79 | 172.99 | 0.20 |
| Downstream of APEX Park footbridge | GHCMA | 172.01 | 172.33 | 0.32 |
| Downstream of Mt. Napier Rd. Bridge | GHCMA | 170.52 | 170.49 | -0.03 |
| At bend in street | GHCMA | 170.05 | 170.23 | 0.18 |
| Downstream of footbridge | GHCMA | 168.53 | 168.73 | 0.20 |
| End of French St. | GHCMA | 168.44 | 168.60 | 0.16 |
| Upstream of bridge | GHCMA | 168.39 | 168.11 | -0.28 |
| Downstream of bridge | GHCMA | 168.04 | 168.08 | 0.04 |
| Downstream of bridge | GHCMA | 167.70 | 167.42 | -0.28 |
| Grange St. on fenceline with Keith Noltes property | GHCMA | 163.81 | 163.91 | 0.10 |
| Downstream of railway bridge | GHCMA | 162.58 | 162.57 | -0.01 |
| | | | Average model difference | 0.05 |

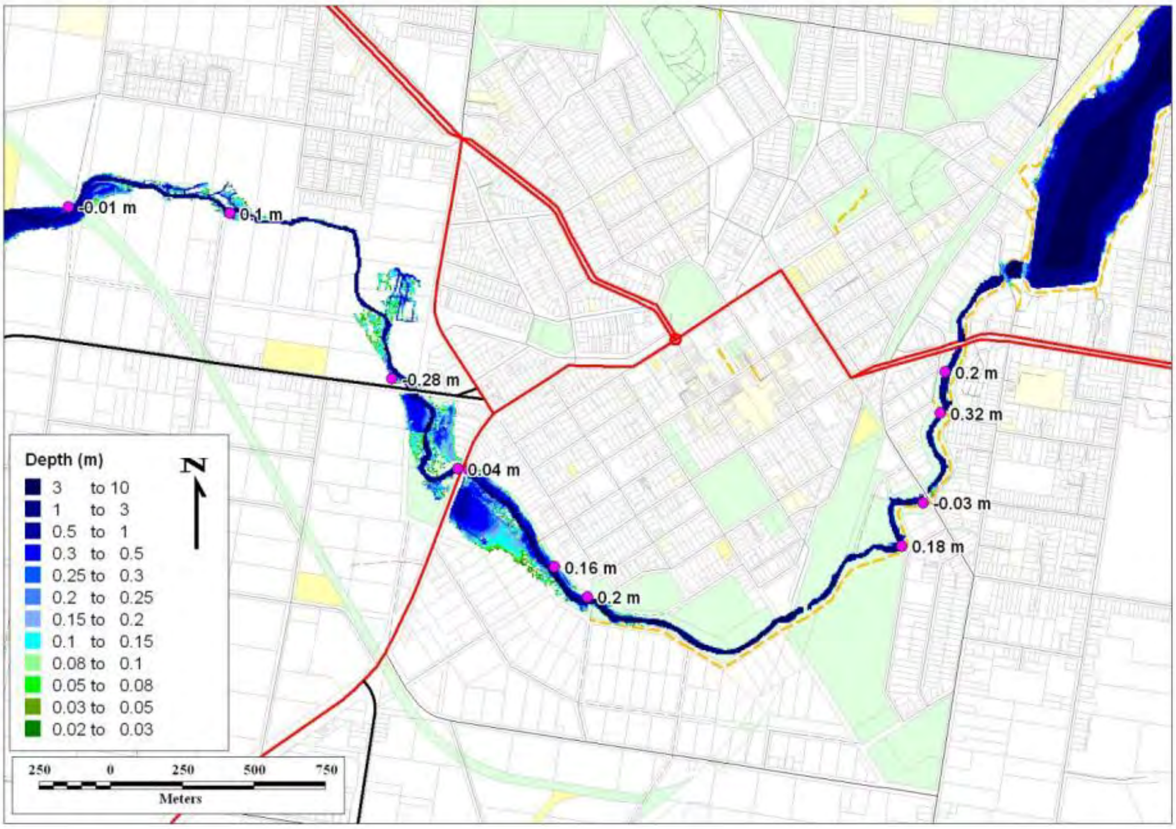



Figure 5.9 Hydraulic model validation for the 2010 flood event

Overall the model is well validated with all of the peak flood heights within +/- 0.30 m for the model area. However, the main purpose of this model is to assess the larger flood events and the model is fit for that purpose. The average difference between the model and the recorded peak flood heights was 0.05 m and the recorded flood heights and locations matched well to the model results. One complexity for this event is that there was no recorded streamflow information and as such the model was run at a steady state conditions with a static inflow based on the upstream recorded level.

This implies that the flows going through the system are not represented as a flood hydrograph (as would occur in reality) as this information was not captured and is not available. In light of this limited information the validation of the 2010 flood event demonstrates that the hydraulic model is representing Grange Burn flood plain appropriately.

## 5.4 Calibrated Model Parameters

The topography and roughness grids are as summarised in Sections 5.2.2 and 5.4.1 respectively. These layers were developed during the calibration process. From the calibration events the upstream boundary conditions were set as 2D line boundaries and these can be used to input the design flow hydrographs as required.

### *5.4.1 Hydraulic Roughness*

The hydraulic roughness for the overland flow model was described using a two-dimensional roughness map of Manning's 'n' values. This was developed by digitising different land-use zones from the digital aerial images within a GIS environment (MapInfo). The roughness values were set to the values as shown in Table 5.4. The final roughness grids are shown in Figure 5.10.

The roughness parameters are consistent with the values specified in *Open Channel Hydraulic* by Chow (Chow, 1973), the manning's 'n' for the roads, residential and commercial are consistent with previous modelling experience and practices. The roughness parameters were used to calibrate the hydraulic model.

For the model the main river channel had a roughness of 0.04 and 0.045 for the tributaries. This roughness range is within the range of a clean straight and clean winding stream (Chow, 1973). The floodplain has been set mainly at 0.05 which is consistent with light brush and weeds.

Sensitivity analysis of the roughness parameters has been undertaken in Section 5.7.

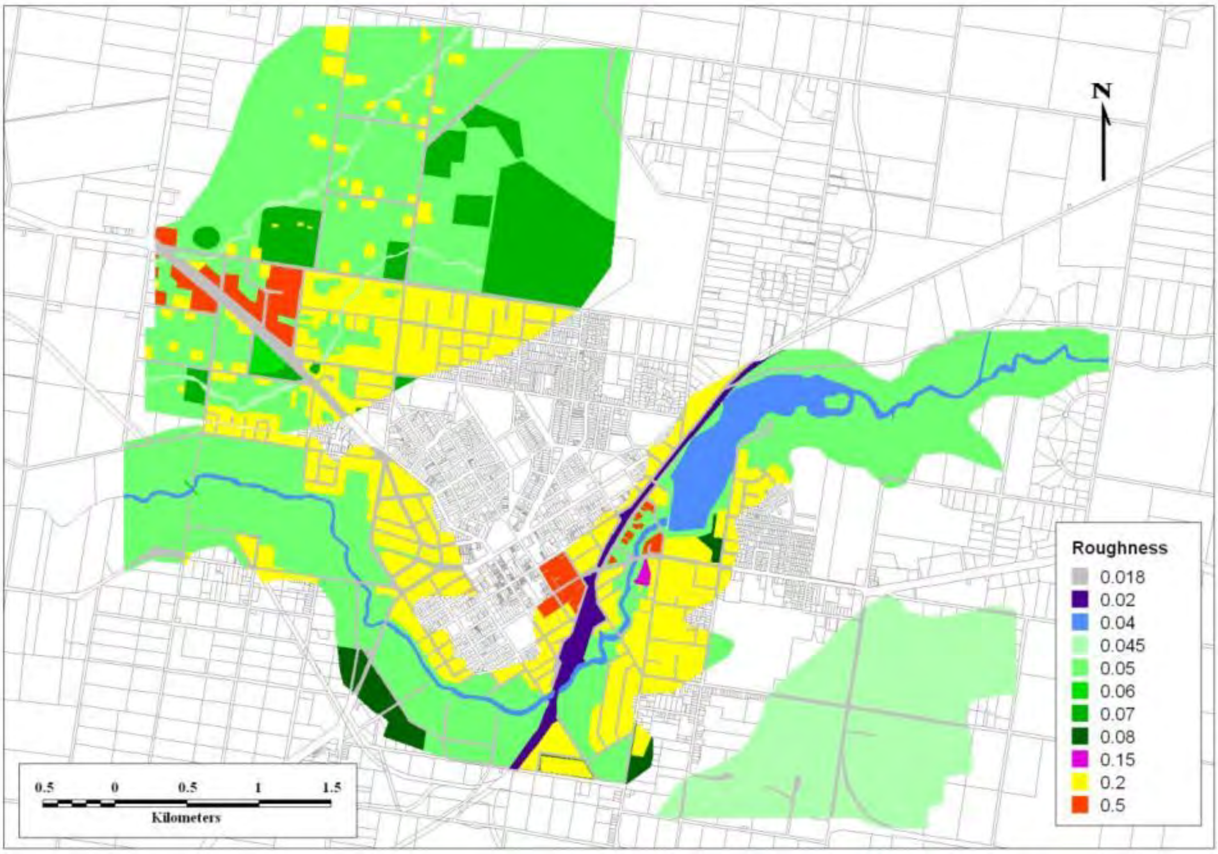



Figure 5.10 Roughness grid for Models A, B and C

The Manning's 'n' value of 0.05 for the flood plain corresponds to light brush and weeds which is considered reasonable for the Grange Burn and tributaries floodplain. The roughness parameter of 0.06 was adopted for the upper reaches of the Russell Creek floodplain as there is likely to be less flow path delineation within these sections of Russell Creek and images of the creek and surrounds shows medium to dense brush.

Table 5.4 Calibrated Roughness Parameters, Mannings 'n'

| Parameter | Roughness Manning's 'n' |
|---|---|
| Roads | 0.018 |
| Railway Line and Embankments | 0.02 |
| Main river channel | 0.04 |
| Minor river channel | 0.045 |
| Main floodplain | 0.05 |
| Moderate floodplain | 0.06 |
| Dense floodplain | 0.07 |
| Partly Residential | 0.15 |
| Residential | 0.2 |
| Commercial | 0.5 |

### *5.4.2 Boundary Conditions*

Boundary conditions were established at the upstream boundary of the Grange Burn model to account for the inflows from the Grange Burn catchment upstream of Lake Hamilton. Two inflow locations were required, one to represent the inflows from the Grange Burn catchment at the Robsons Road gauge and one for the remaining inflows downstream of the Robson Road gauge to Lake Hamilton. These boundaries were established as 2D line boundaries and inflows were introduced to the model as hydrographs (flow varying over time).

Each of the three tributaries had one upstream boundary but included numerous local inflow points throughout the model as lateral inflows. These were included as the majority of the inflows are generated within the local tributaries. The upstream inflows were input using a 2D line boundary and the local inflows were input using lateral inflow nodes. Again, design events were modelled as flow hydrographs over time.

The downstream model boundaries were setup as a static water level boundary as the catchments are low grade and the downstream boundaries are sufficiently far downstream to not impact the model results. The level of each boundary was set by assessing the typical flood depths during an event using a cross section with the Manning's Equation to calculate the flow rate and expected depth. The details are presented in Section 5.4.3.

The locations of the inflow locations for Model A. B and C is presented in Figure 5.11.

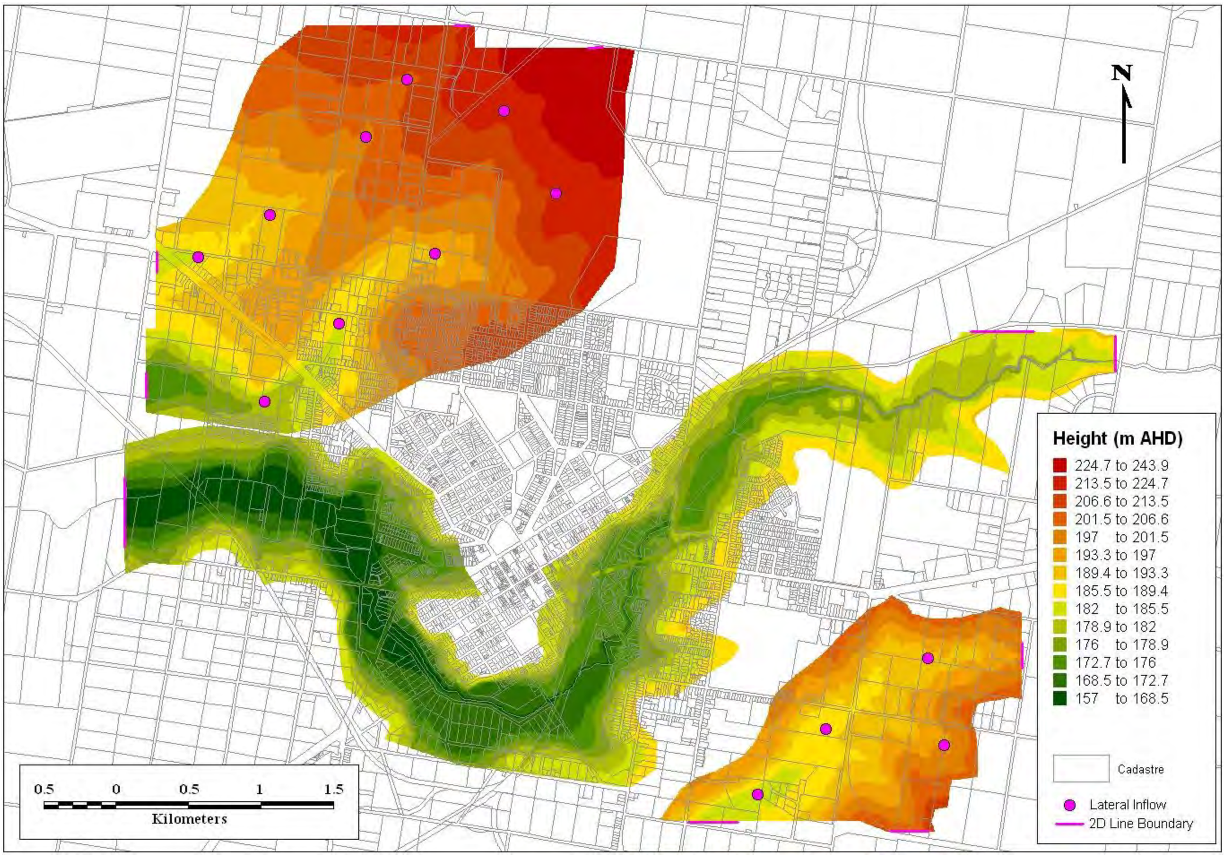



Figure 5.11 Boundary inputs to SOBEK

### 5.4.3 Downstream Conditions

The downstream boundary was set as a 2D level boundary. The boundary conditions in Models A and C are sufficient to convey the flows for the full range of durations. Within Model B (Grange Burn) the boundary was set using the Manning's Equation which uses the cross sectional area, channel slope and roughness to predict flow rates at the model boundary as the depths varied considerably during the full range of flood events. The levels are summarised in Table 5.5.

Table 5.5 Downstream boundary conditions within Models A, B and C

| Model | Location | Durations | Boundary Level | Comment |
|---|---|---|---|---|
| Model A | Marshalls Road Tributary | 20%, 10%, 5%, 2%, 1%, 0.5% | 173.0 mAHD | |
| | Kennys Road Tributary | 20%, 10%, 5%, 2%, 1%, 0.5% | 183.0 mAHD | |
| Model B | Grange Burn | 20% | 160.6 mAHD | Based on Manning's calculation. Uses channel slope, roughness and cross section at model boundary. |
| | | 10% | 161.1 mAHD | |
| | | 5% | 161.9 mAHD | |
| | | 2% | 162.3 mAHD | |
| | | 1% | 162.5 mAHD | |
| | | 0.5% | 162.7 mAHD | |
| | | PMF | 167.2 mAHD | |
| Model C | Petchels Road Tributary | 20%, 10%, 5%, 2%, 1%, 0.5% | 182.2 mAHD | |

## 5.5 1946 Event Assessment

As part of this flood investigation the GHCMA has requested that the 1946 event (the largest event in recent history) be qualified and a peak flow rate determined. The hydrology was unable to determine the peak flow rate as there was no streamflow recorded for this period. It was proposed that the calibrated hydraulic model would be used to estimate the peak flow rate, and hence the return period for the 1946 event.

Following the satisfactory calibration of the 1983 and 2010 flood events, the model was used to develop and assess the March 1946 event. From the hydrology section it was evident that there was no peak flow estimate for this event and that the peak flow rate would have to be determined using the hydraulic model. This also provides a guide as to the behaviour of the calibrated hydraulic model.

The recorded peak flood heights were obtained from the GHCMA, VicRoads design plans and from the GHD report (1987). The variety of sources produced a number of peak flood heights that were in clear conflict even once the various datum were adjusted to mAHD. These locations included:

- Mt Napier Road Bridge – GHD 171.9 m AHD & VicRoads 172.7 mAHD – *Difference 0.8 m*
- Portland Road Bridge – GHCMA 170.1 mAHD & VicRoads 168.8 mAHD – *Difference 1.3 m*
- Dartmoor-Hamilton Rd Bridge – GHCMA 169.2 mAHD & VicRoads 168.7 mAHD – *Difference 0.5 m*

The selected recorded peak flood height for the assessment is shown in bold in the dot points above and was based on examining the full set of recorded peak flood heights in conjunction with the reliability of the information obtained. Overall, it is clear that there is some uncertainty associated with these levels as this was a large flood event, and the method of determining peak flood heights is not documented along with the recorded peak flood heights.

The process of determining the peak flow rate for the March 1946 event was to run the model assuming a range of flow rates until the known peak flood heights were adequately matched within the model. The model was run on a steady state basis with a constant inflow. The peak flow rate that was required to generate the approximate 1946 event was 200 $m^3/s$. This is approximately equal to the 1% AEP flood event. The peak flood heights are summarised and shown graphically in Table 5.6 and Figure 5.12 respectively.

Table 5.6 Hydraulic model calibration for the March 1946 flood event

| Location | Source | Recorded value | Recorded peak flood depth (mAHD) | Model peak flood depth (mAHD) | Difference (m) |
|---|---|---|---|---|---|
| Ballarat Road | VicRoads | 175.1 mHSA | 174.79 | 174.50 | -0.29 |
| 4 Holden Street | GHD | 174.55 mHSA | 174.24 | 174.33 | 0.09 |
| Mt Napier Rd Bridge | VicRoads | 569.0 ft HSA | 172.67 | 173.22 | 0.55 |
| Portland Road | GHCMA | 170.07 mAHD | 170.07 | 169.72 | -0.35 |
| Dartmoor-Hamilton Rd | GHCMA | 169.19 mAHD | 169.16 | 169.01 | -0.15 |

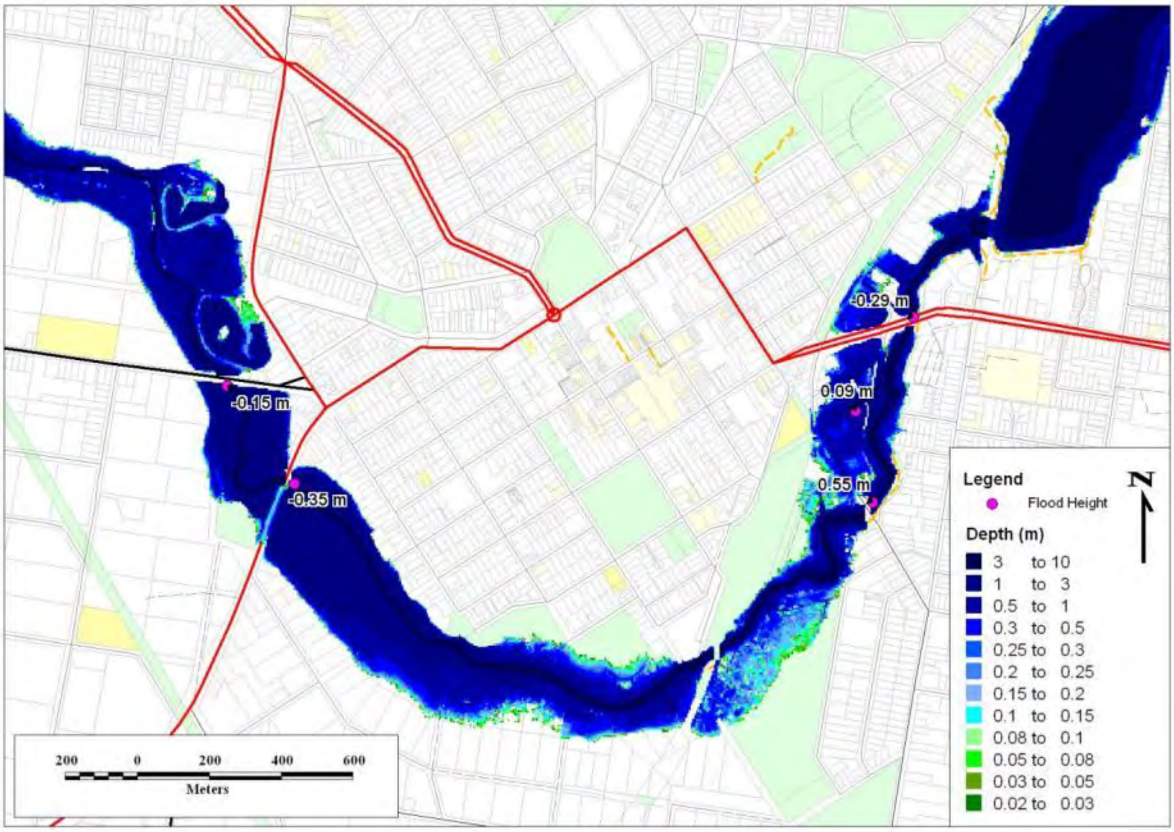



Figure 5.12 Hydraulic model calibration for the 1946 flood event

Overall, the 1946 event is reasonably well represented, however, the fact that the levels at Mt Napier Road Bridge appear inconsistent with the other known levels within the model makes it difficult to achieve a strong match to all historic peak flood heights. The peak flow estimate is in the right range, but the quality and consistency of the historic flood heights makes the calibration difficult to improve.

## 5.6 Design Event Results

The design model runs have been completed for the 20%, 10%, 5%, 2%, 1% and 0.5% AEP events. For each event the peak flow duration was run through the model. Where the peak flow duration varied throughout the catchment both event durations were run to ensure that the peak flood event was captured.

The peak flood depths have been presented in Figure 5.13 to Figure 5.30 for Model A, B and C for each AEP respectively. A discussion of the increasing impacts of the floods has been discussed in the following sections.

### *5.6.1 20% AEP Event*

The 20% AEP flood depths and extents are shown in Figure 5.13, Figure 5.14 and Figure 5.15 for Model A, B and C respectively.

Model A indicated that the 20% AEP event was largely contained within the existing channels and the majority of the culverts and structures conveying the full flood flows. The exception to this was at Young Street where the road was overtopped. Some roads within the Kennys Road Tributary had shallow water over the road during the peak of this event.

Model B showed that the Grange Burn was largely containing the 20% AEP flows within the river banks. Flows only enter the floodplain upstream of the Dartmoor-Hamilton Bridge, however this area is understood to be a floodplain area that would inundate during frequent flood events such as the 20% AEP event. No roads are inundated during this event.

Model C shows large areas inundated to a shallow level along the existing flow path as well as numerous natural storages filling along the branches of the Tributary. No water is over the main Hamilton Hwy, there was some water over the minor roads within the catchment.

### 5.6.2 10% AEP Event

The 10% AEP flood depths and extents are shown in Figure 5.16, Figure 5.17 and Figure 5.18 for Model A, B and C respectively.

Model A indicates that the flood extent under the 10% AEP flood event was largely the same as the 20% AEP flood event, however flood depths have increased. For the Kennys Road Tributary water was over North Boundary Road adjacent to Coleraine Road, Sobeys Road, Kennys Road and Mount Bainbridge Road. Depths over these roads were shallow at less than 10 cm. For Marshalls Road Tributary only Kings Road and Young Street have water over the road.

Model B shows that the Grange Burn begins to break out of the main channel during the 10% AEP flood event adjacent to Holden Street. Flood waters of depth up to 30 cm are over Holden Street. The remaining flows are at bank full capacity until downstream of the railway bridge where they break the banks and inundate the floodplain. The area of floodplain inundation is increased as compared to the 20% AEP event.

Model C indicates that during the 10% AEP flood event some water cross the Hamilton Hwy and Fyfe Street. This water was shallow but does cross the road at the peak of the event. The flood extent is slightly larger than the 20% AEP flood event, however, there are no major changes to the areas inundated.

### 5.6.3 5% AEP Event

The 5% AEP flood depths and extents are shown in Figure 5.19, Figure 5.20 and Figure 5.21 for Model A, B and C respectively.

Model A indicates a relatively unchanged flood extent under the 5% AEP flood event, however depths throughout the model increased. Peak flows overtopped North Boundary Road adjacent to Coleraine Road.

During the 5% AEP event the Grange Burn was shown to break the banks significantly adjacent to Apex Park. Properties are impacted during this event, mainly along Holden Street and Abbott Street. In this model run the levee banks were not sealed to represent the limited warning scenario, however the levees were not overtopped. Flood waters cross Mount Napier Road to a depth of approximately 10 to 30 cm. The flood waters break out of the main channel downstream of the railway bridge as per the lower flow events.

Model C shows no significant increase in flood extent as compared to the 10% AEP, however the depth of water over the Hamilton Hwy and Fyfe Street have increase.

### 5.6.4 2% AEP Event

The 2% AEP flood depths and extents are shown in Figure 5.22, Figure 5.23 and Figure 5.24 for Model A, B and C respectively.

Within Model A the 2% AEP flood event increased the overland flows upstream of Hamilton. Within the township some properties began to get inundated along the main flow channels. Water has now overtopped Kent Road on the Marshalls Road Tributary in addition to the roads overtopped in smaller flood events. Coleraine Road begins to be overtopped during this event, although at very shallow depths.

In the 2% AEP flood event there is substantial flooding downstream of Lake Hamilton and upstream of Ballarat Road. Some of the commercial sites have been inundated in this area. Downstream of Ballarat Road there was increased flooding around Abbott Road and Holden Street. This extended as far back as the railway line. Mount Napier Road was between 30 and 50 cm under flood waters. Flows were well outside the main river channel along the entire length of the model area.

The flood extent for Model C was not increased significantly as compared to the smaller flood events, however the depths of water over the Hamilton Highway had increased to between 10 and 30 cm.

#### 5.6.5 1% AEP Event

The 10% AEP flood depths and extents are shown in Figure 5.25, Figure 5.26 and Figure 5.27 for Model A, B and C respectively.

Model A shows increased flood depths and small increases in flood extent under the 1% AEP event. Depths over Coleraine Road have increased but are still below 10 cm.

Model B shows that additional areas upstream of Ballarat Road are inundated during the 1% AEP event and the Apex Park and Holden Street areas are further inundated. In this scenario the Lake Hamilton dam wall was briefly overtopped to the north of the spillway (refer to Section 5.2.2.1 for discussion), although this occurs only for a short time and for a small volume of flood waters. Additional areas of floodplain are inundated during this event as compared to the 2% AEP event.

Model C shows increased depths for the 1% AEP over smaller flood events and an additional overland flow path adjacent to Fyfe Street. The depth of water across the Hamilton Hwy is not significantly increased under this scenario, however the length of road inundated has increased.

#### 5.6.6 0.5% AEP Event

The 0.5% AEP flood depths and extents are shown in Figure 5.28, Figure 5.29 and Figure 5.30 for Model A, B and C respectively. For Model A most roads were overtopped during the 0.5% AEP event. The flood extent has not significantly increased over the 1% AEP event. Depths over Coleraine Road have increased and the Highway would be closed during an event of this magnitude.

For Model B the 0.5% AEP flood event causes additional overtopping of Lake Hamilton, on both the northern and southern embankments (relative to the spillway). Water also crosses the railway embankment to Ballarat Road. The water depth over Portland Road has also increased and this road would be closed during this event. The flood extent did not change significantly during this event as compared to the 1% AEP event.

For Model C the Hamilton Highway was still only overtopped at the one location. The 0.5% AEP event increased the floodplain depths but did not increase the flood extent significantly. The additional flow path stemming from Fyfe Road downstream of the Hamilton Highway was more evident during the larger 0.5% AEP event as compared to the 1% AEP event.

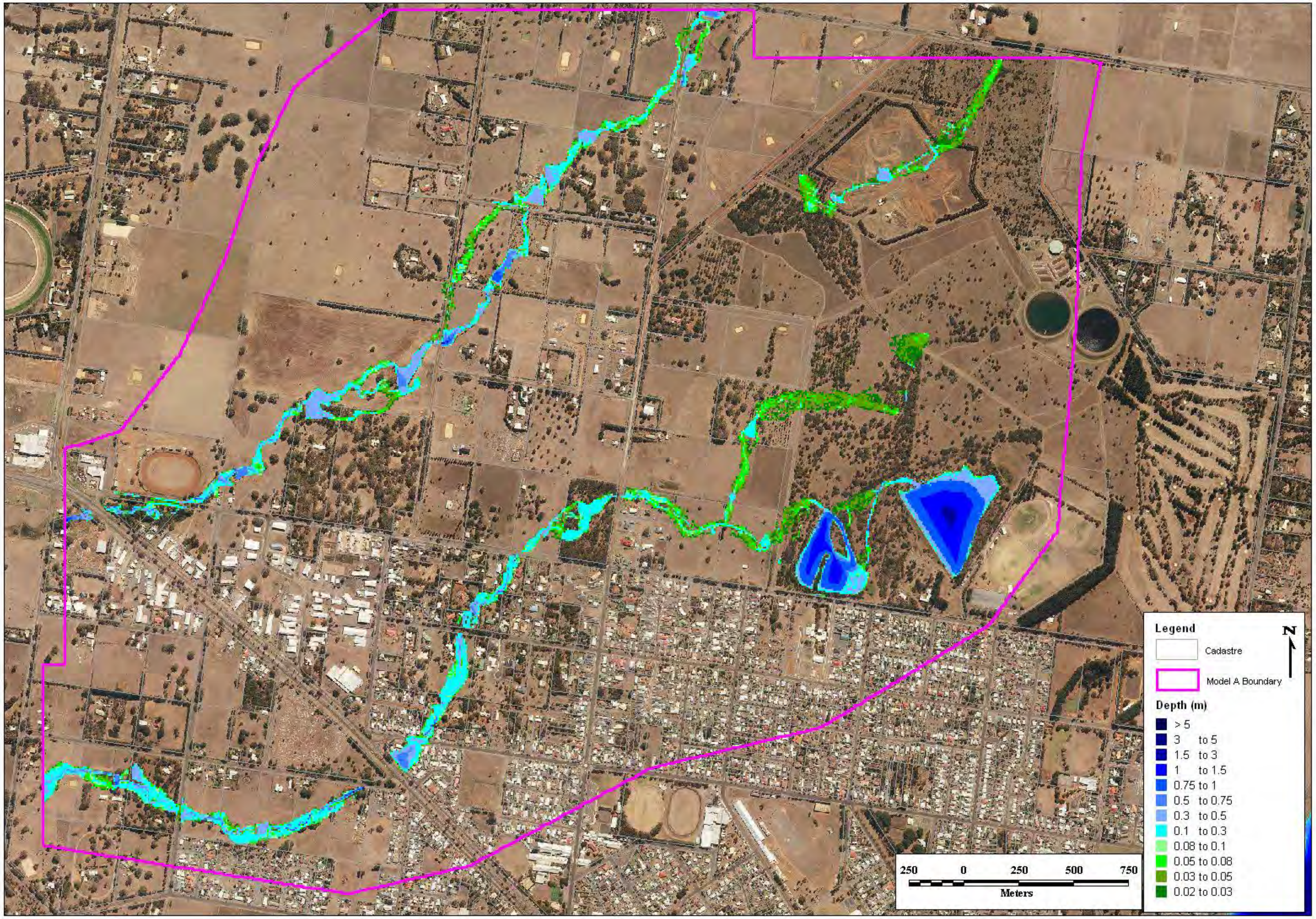



Figure 5.13 Model A – 20% AEP peak flood depths

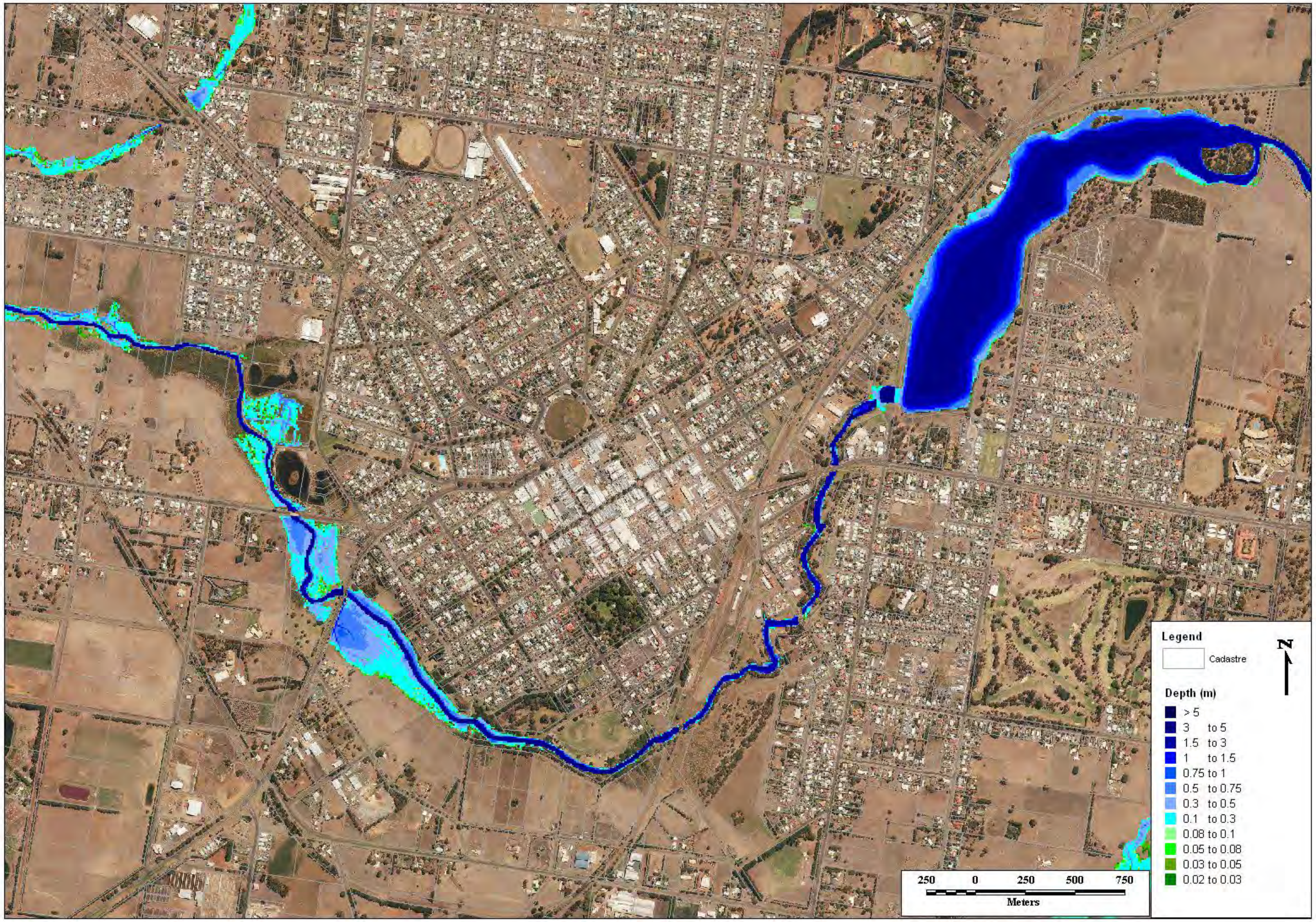



Figure 5.14 Model B – 20% AEP peak flood depths

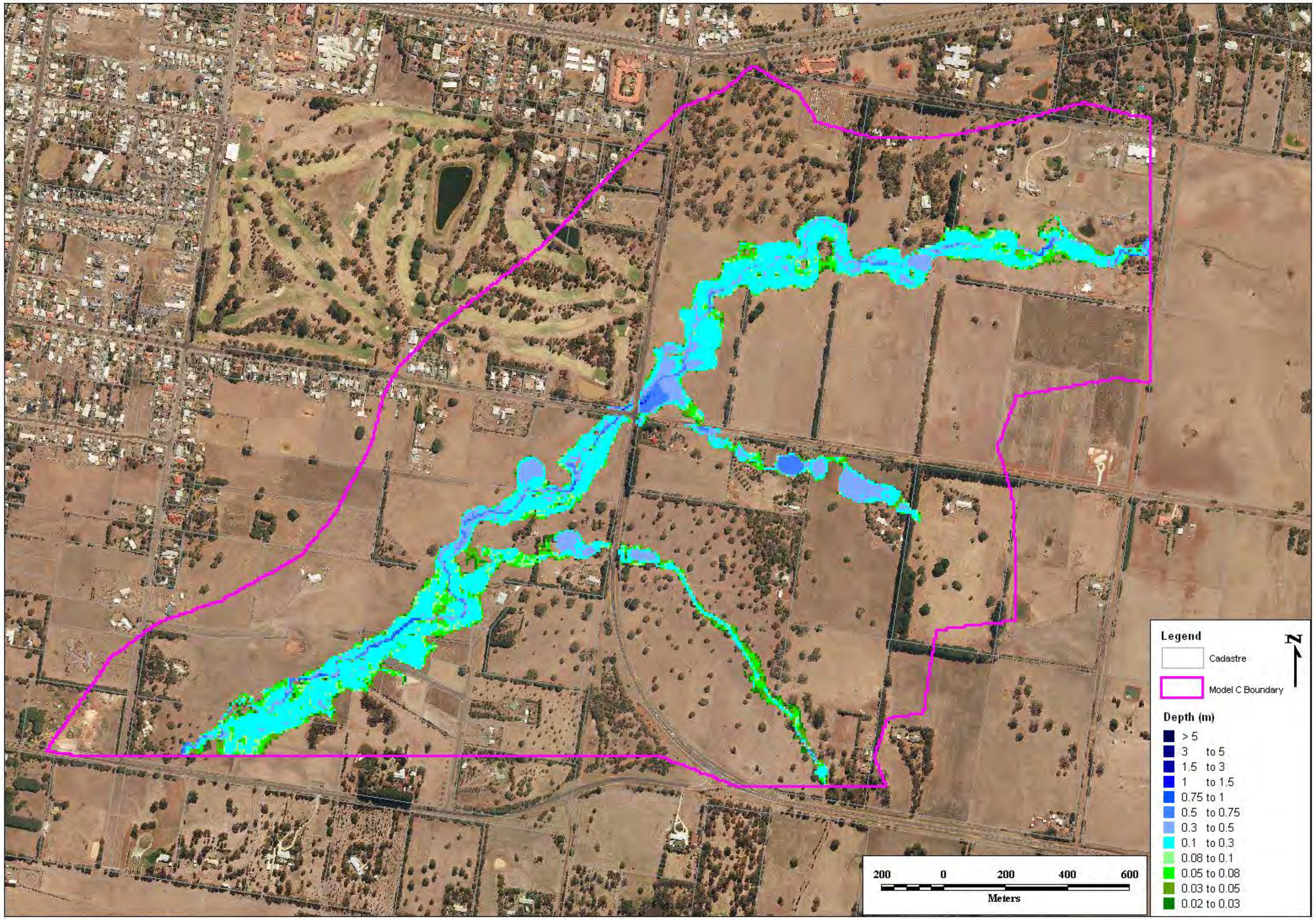



Figure 5.15 Model C – 20% AEP peak flood depths

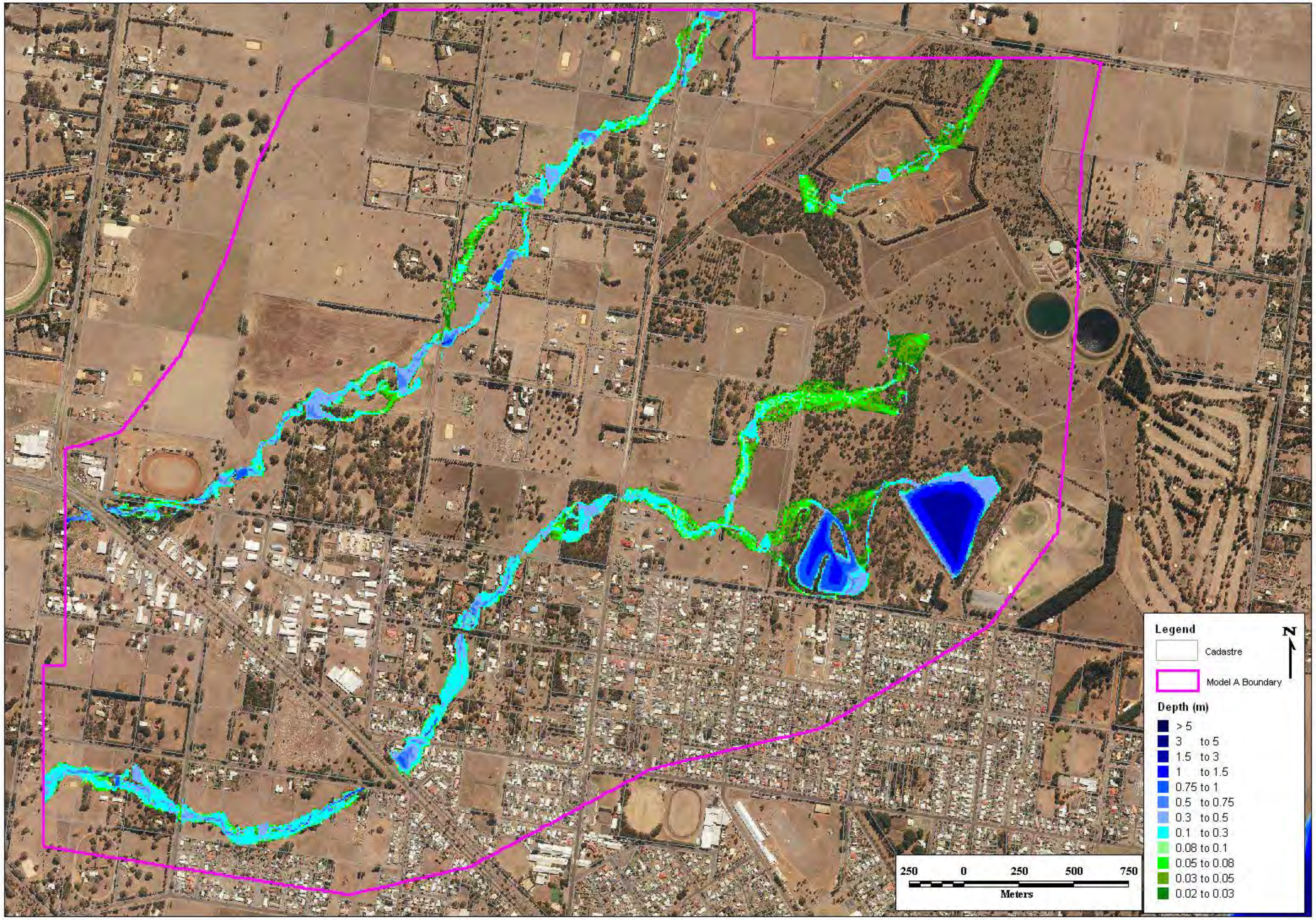



Figure 5.16 Model A – 10% AEP peak flood depths

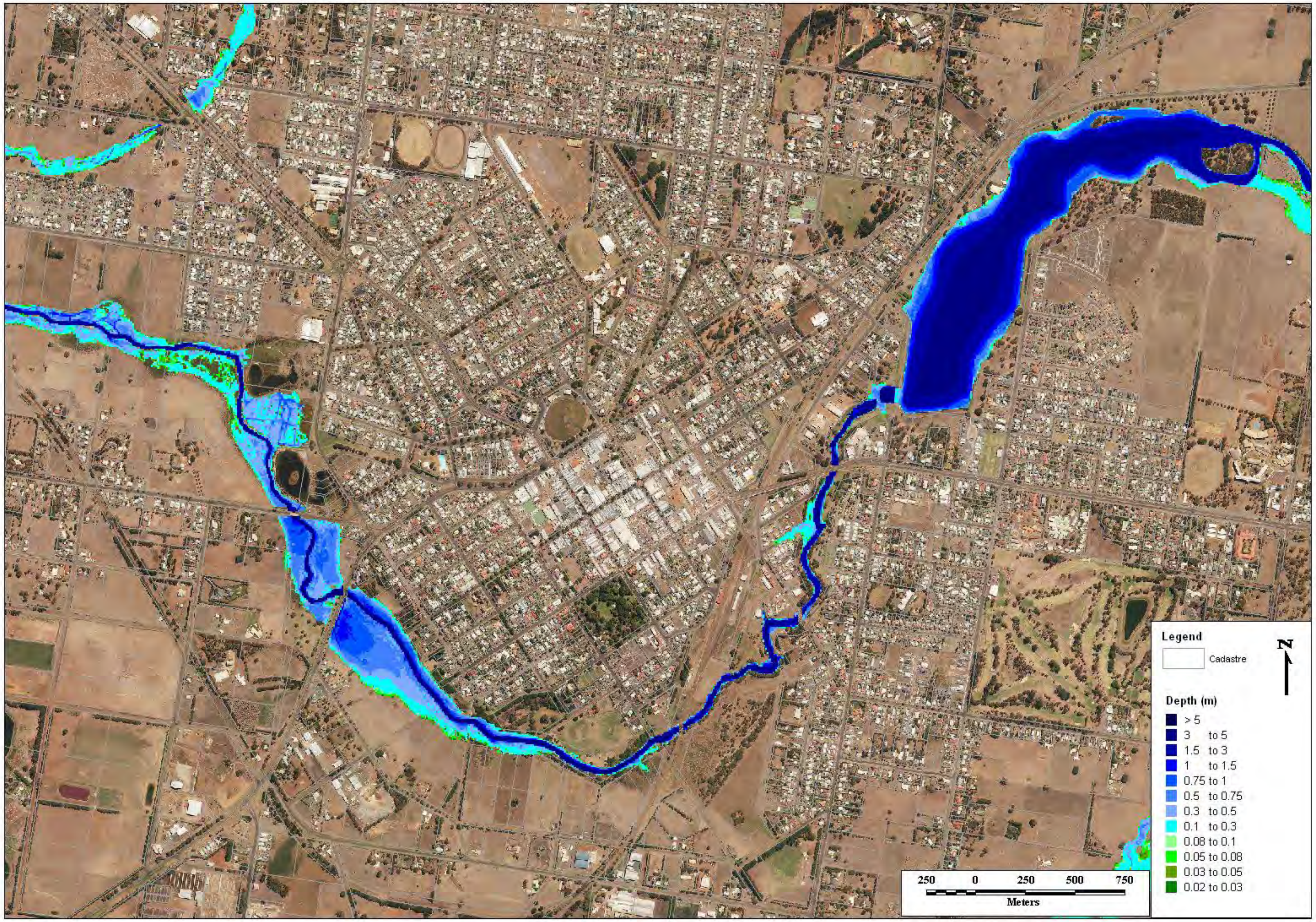



Figure 5.17 Model B – 10% AEP peak flood depths

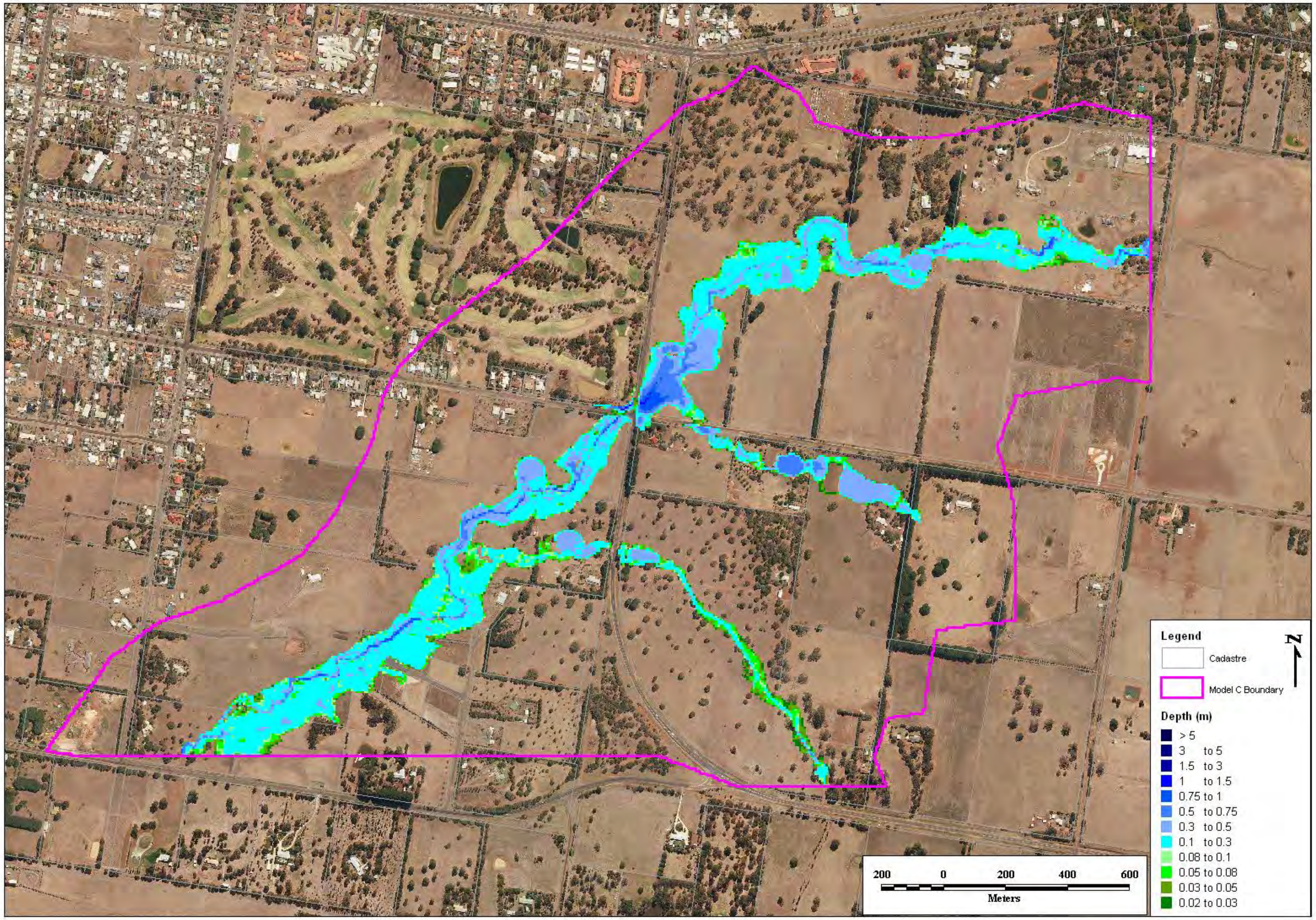



Figure 5.18 Model C – 10% AEP peak flood depths

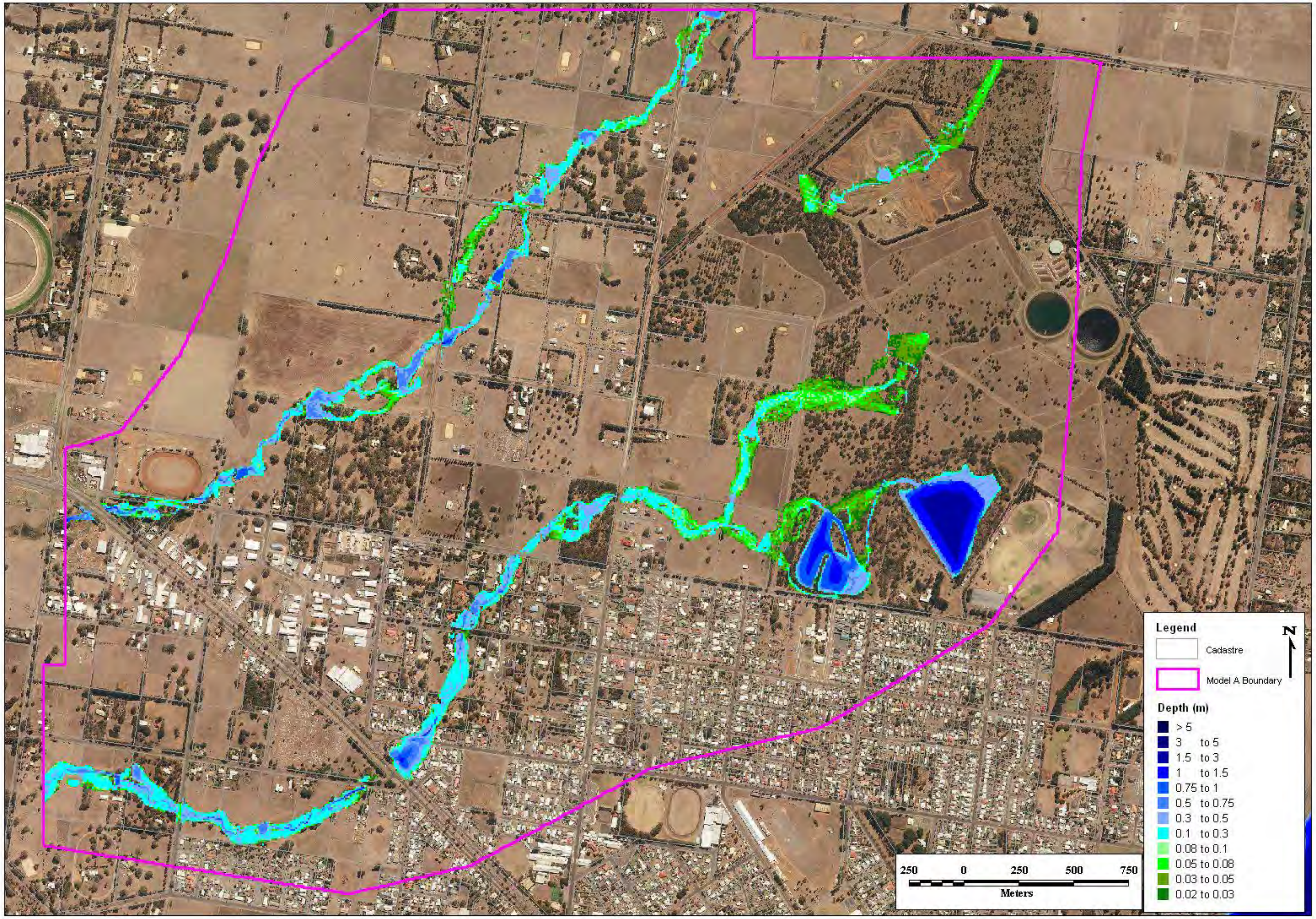



Figure 5.19 Model A – 5% AEP peak flood depths

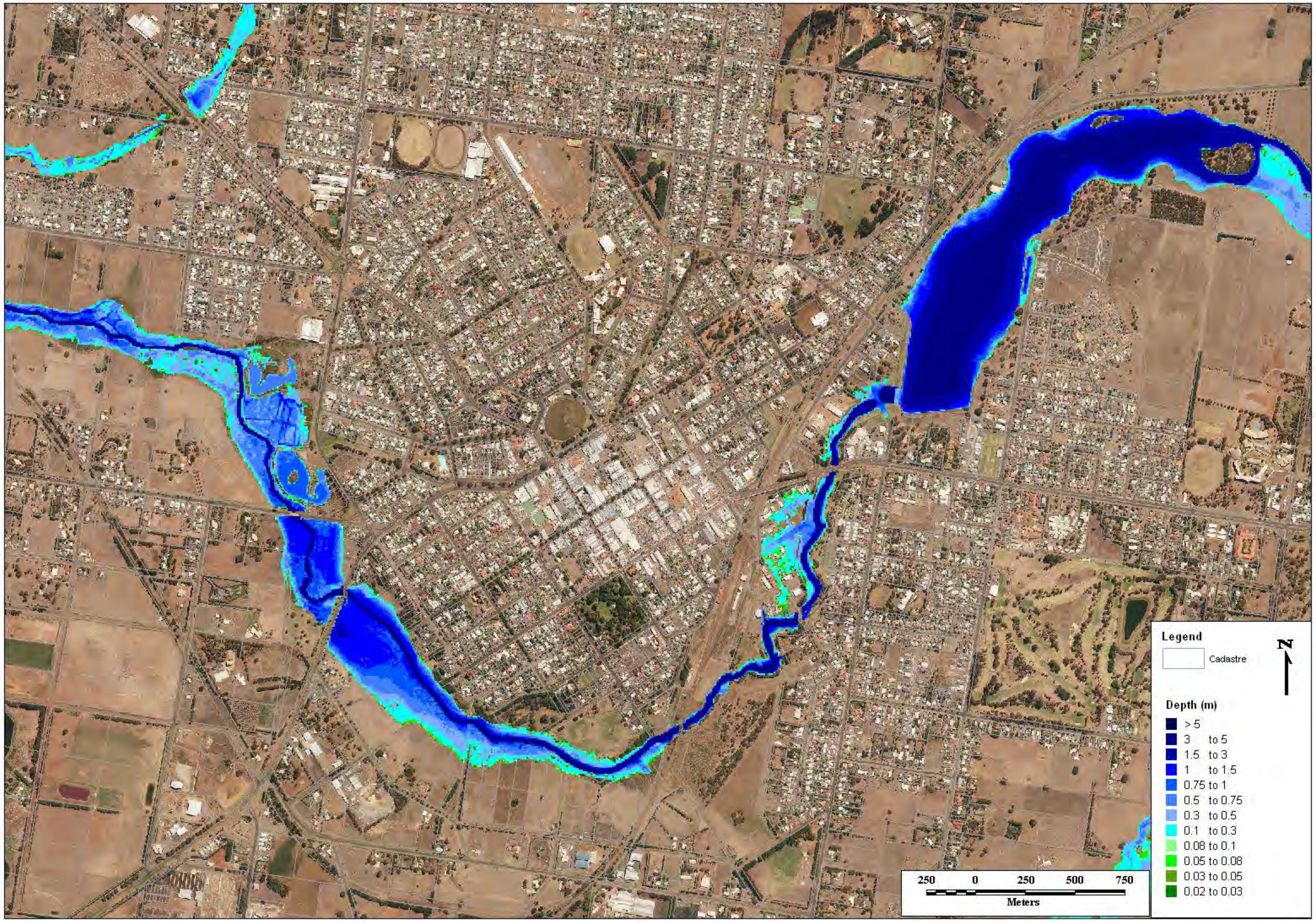



Figure 5.20 Model B – 5% AEP peak flood depths

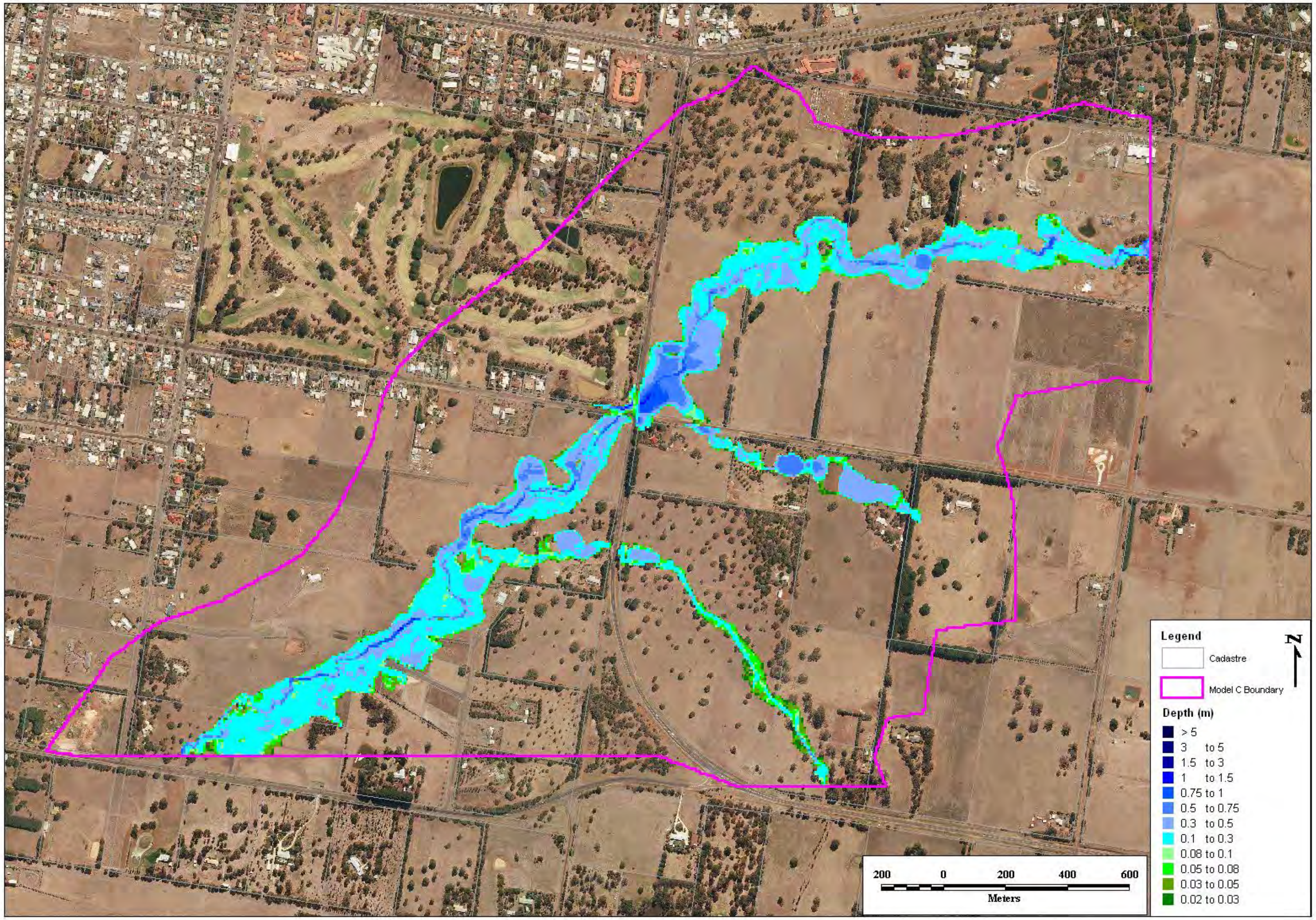



Figure 5.21 Model C – 5% AEP peak flood depths

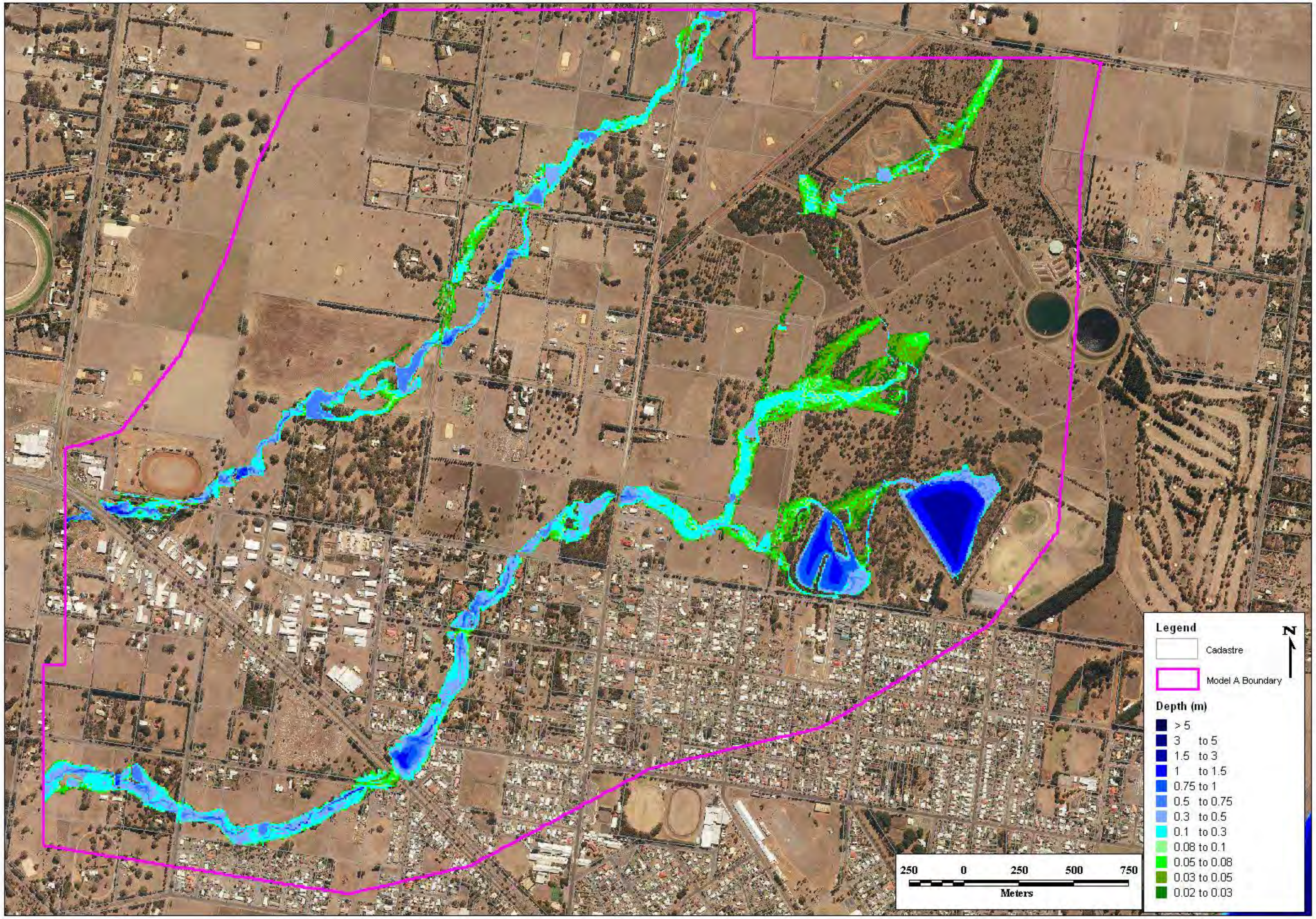



Figure 5.22 Model A – 2% AEP peak flood depths

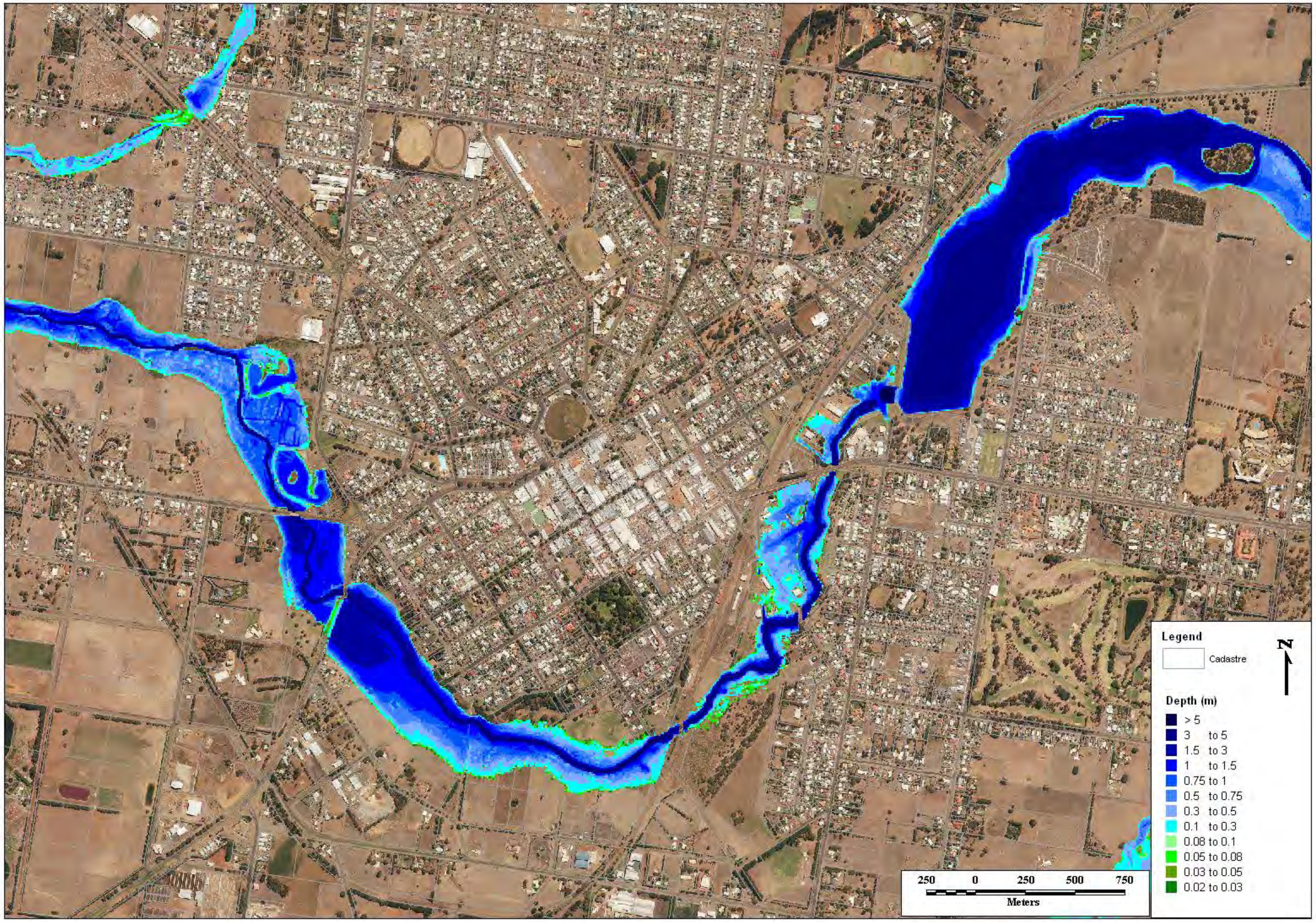



Figure 5.23 Model B – 2% AEP peak flood depths

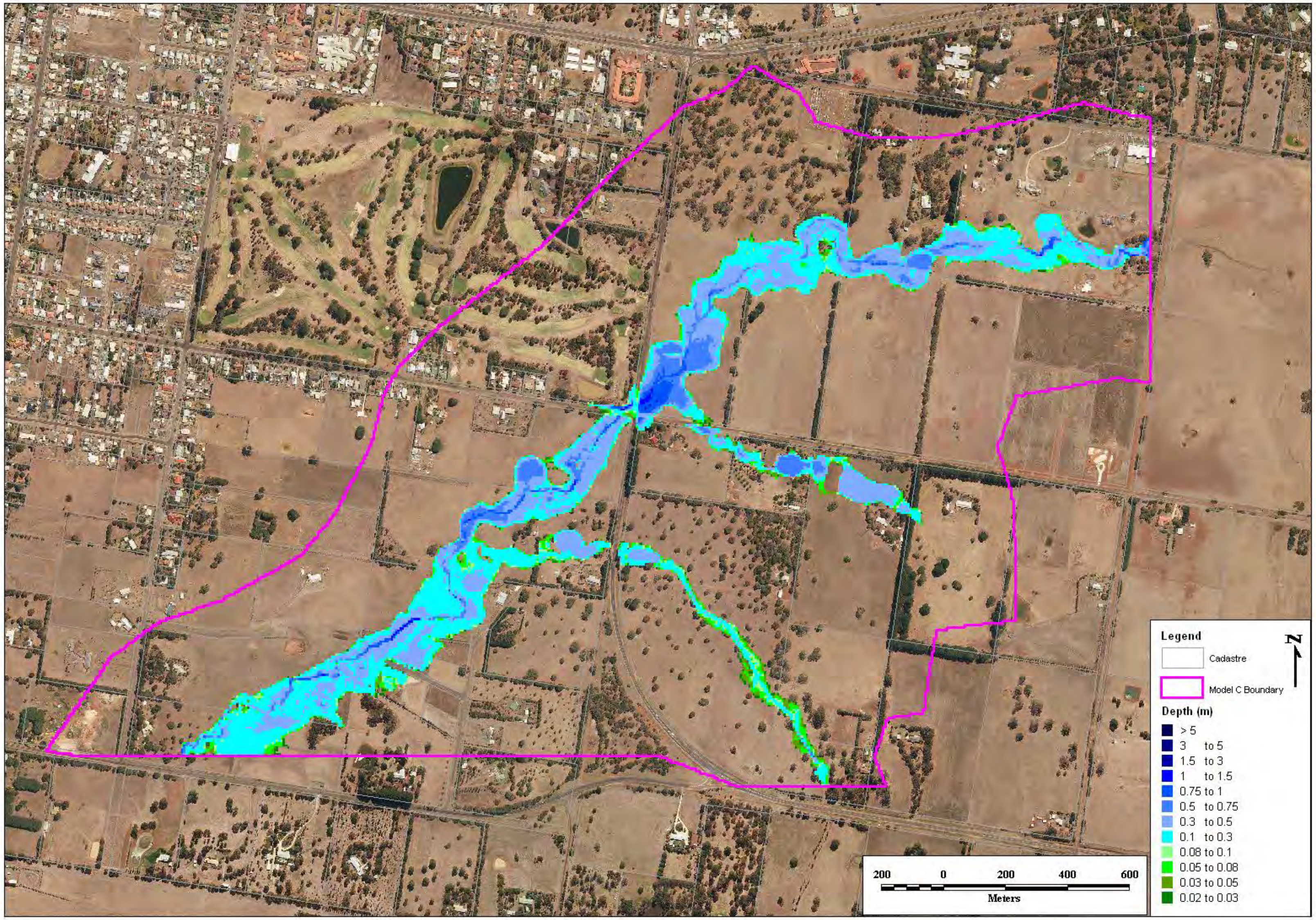



Figure 5.24 Model C – 2% AEP peak flood depths

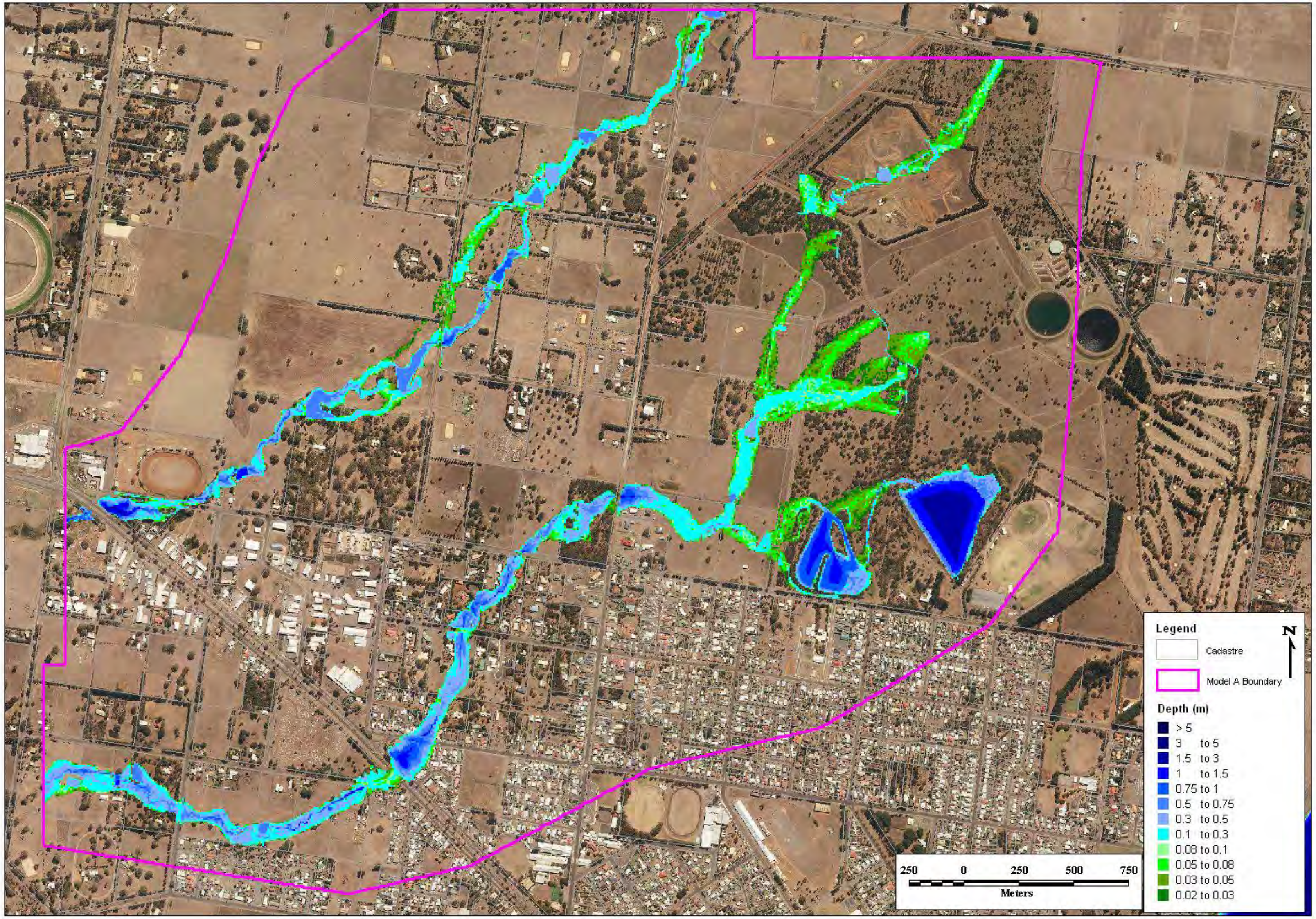



Figure 5.25 Model A – 1% AEP peak flood depths

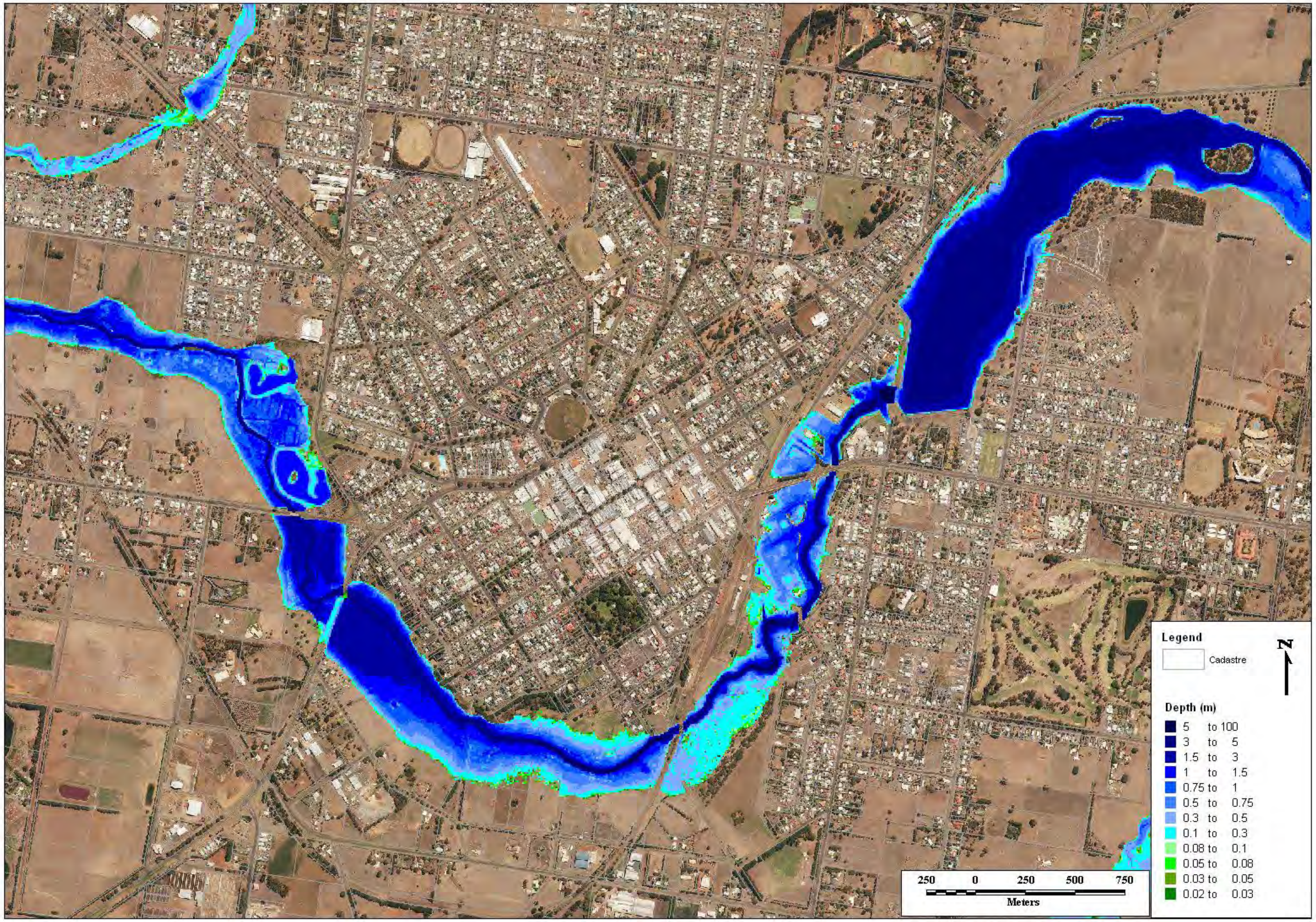



Figure 5.26 Model B – 1% AEP peak flood depths

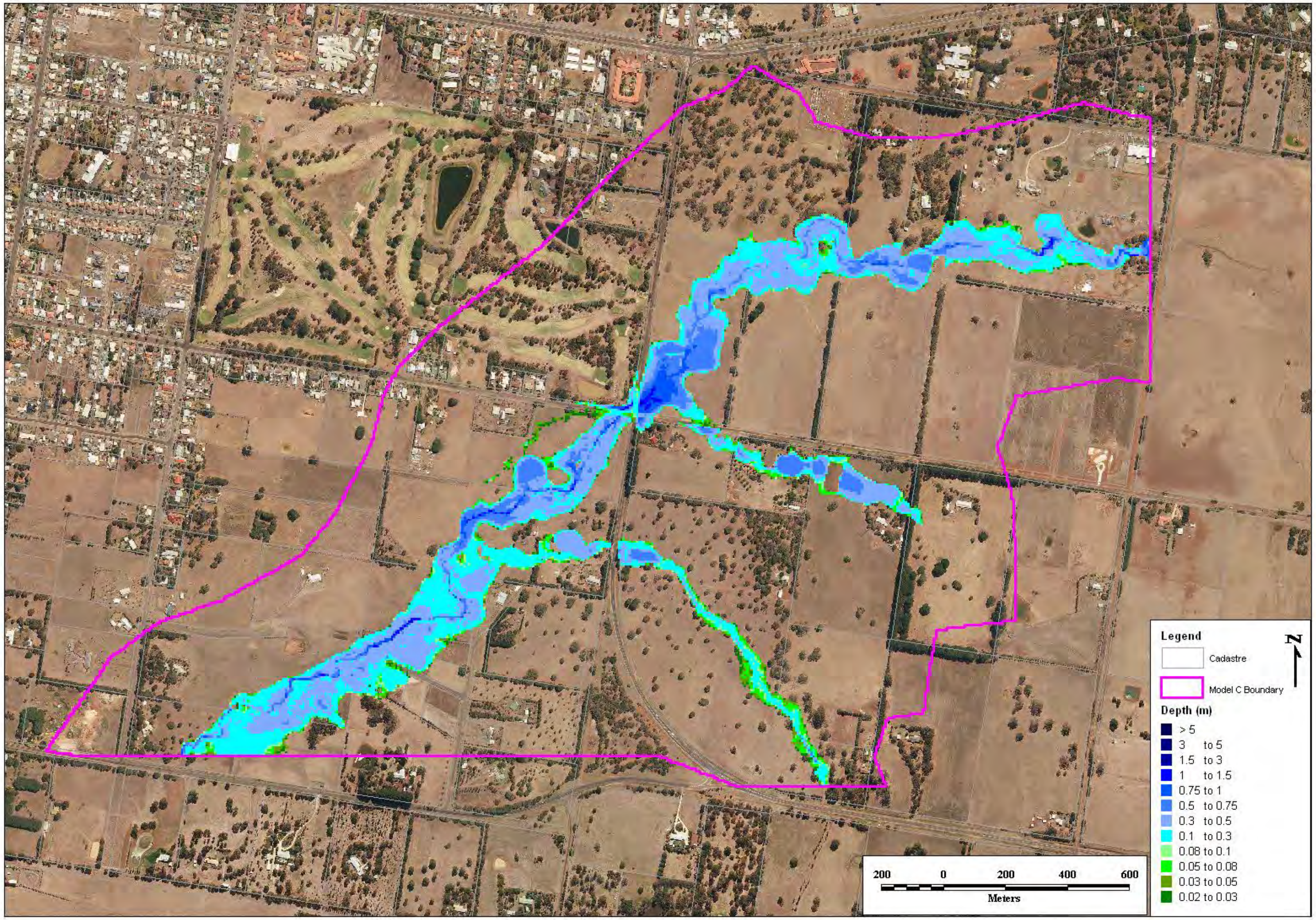



Figure 5.27 Model C – 1% AEP peak flood depths

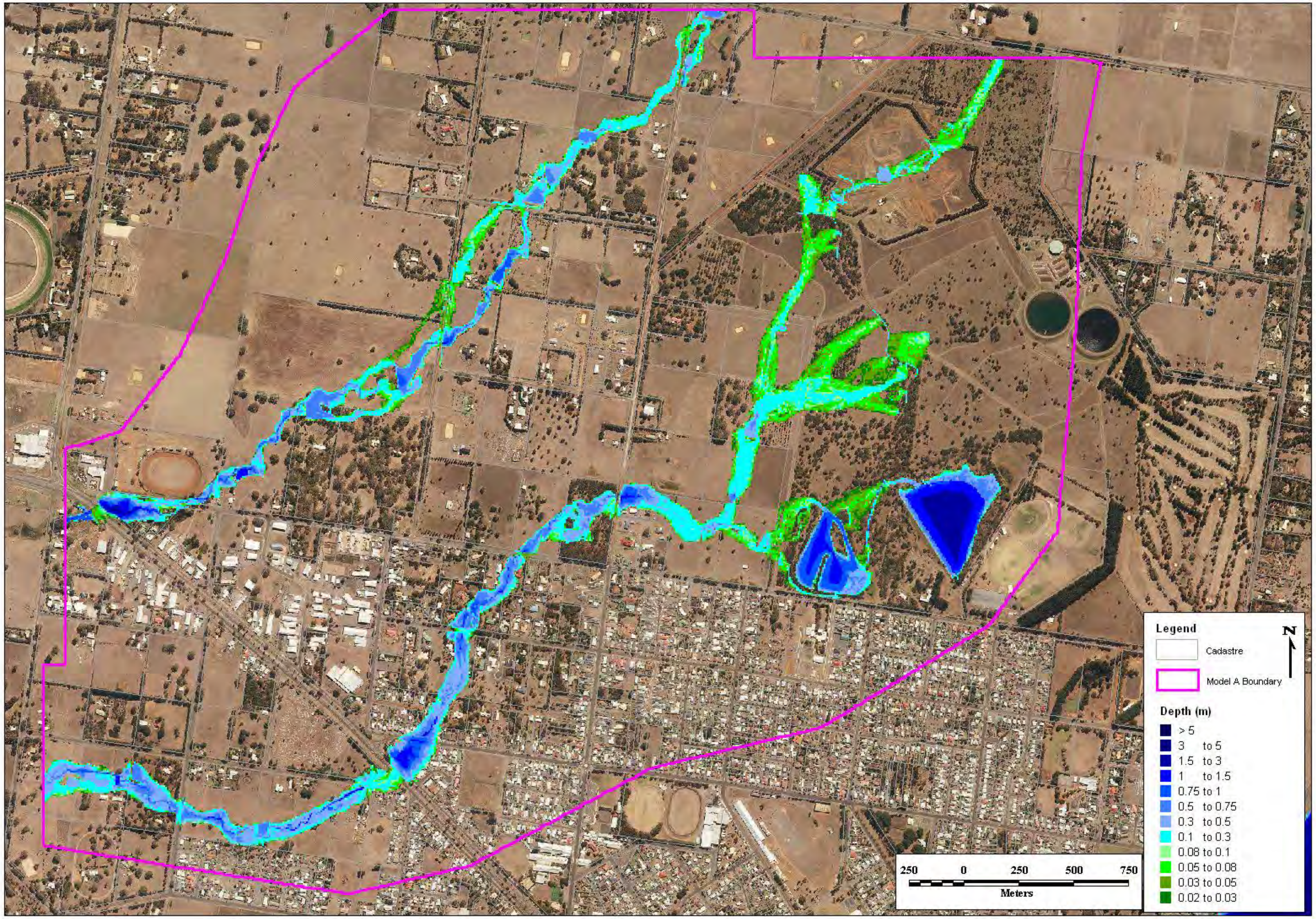



Figure 5.28 Model A – 0.5% AEP peak flood depths

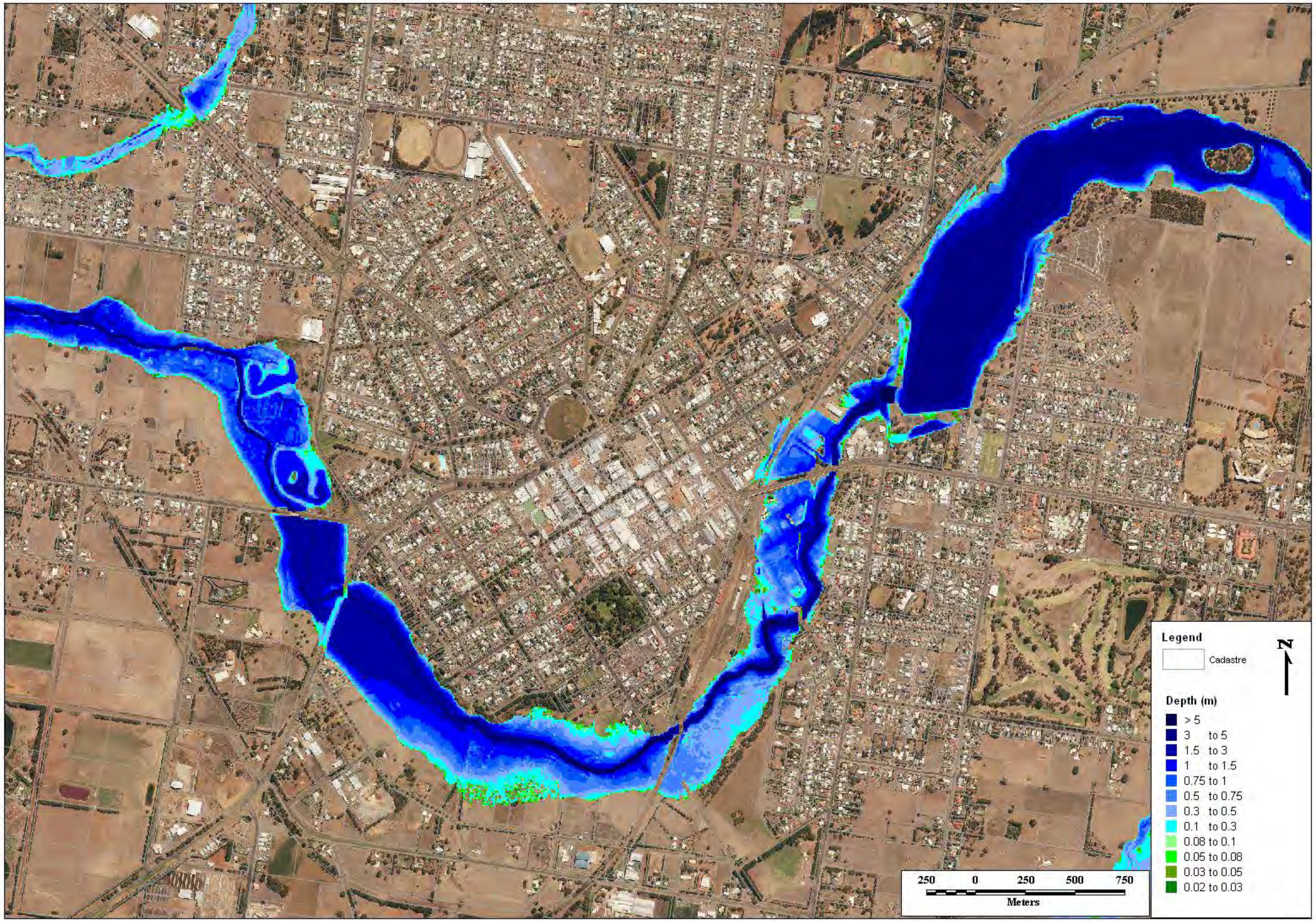



Figure 5.29 Model B – 0.5% AEP peak flood depths

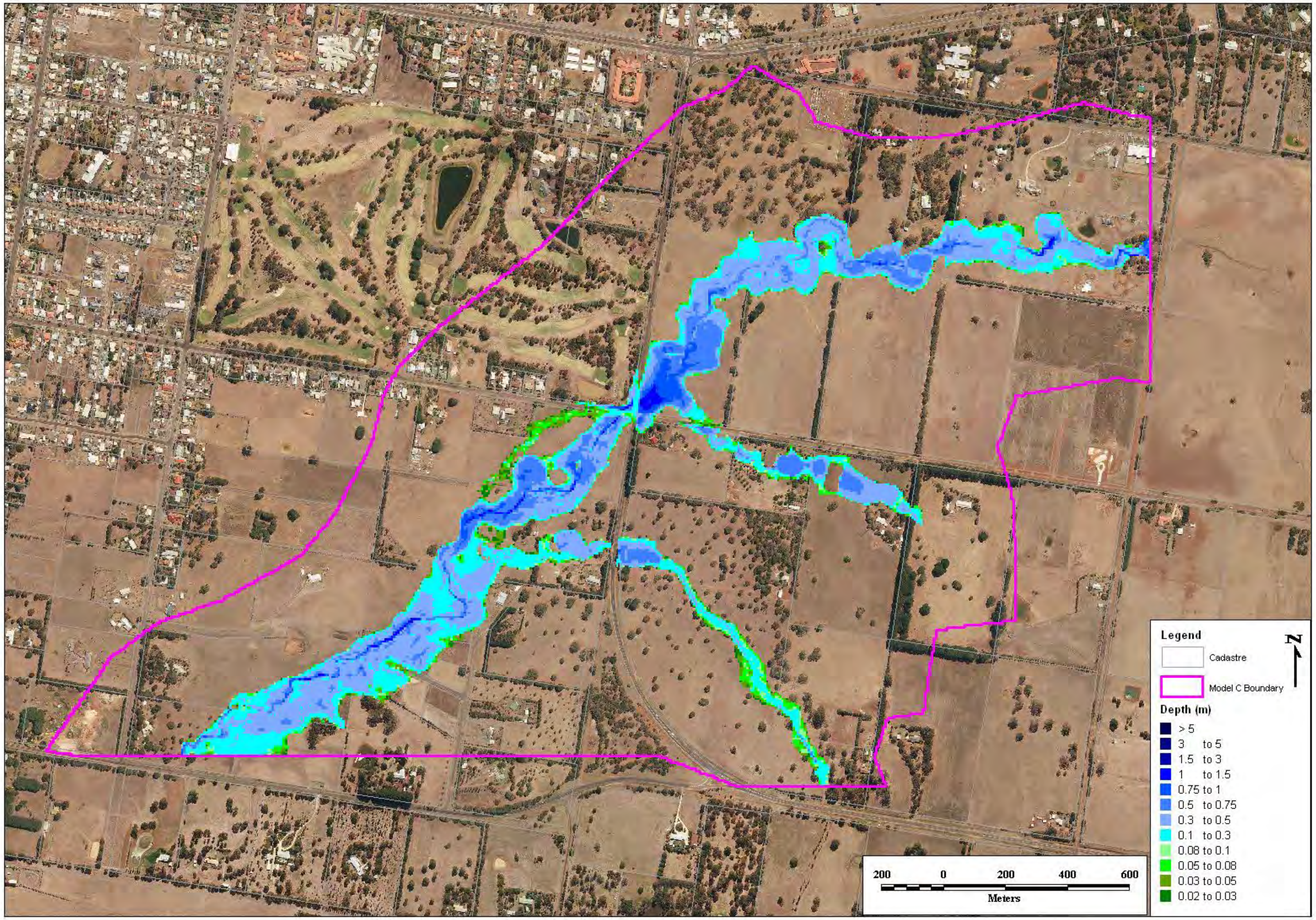



Figure 5.30 Model C – 0.5% AEP peak flood depths

### 5.6.7 PMF Event

The probable maximum flood (PMF) was run for Model A, B and C using the hydrographs as developed in Section 4.7. The critical duration for the three areas was the 3 hour event (the 3 hour, 12 hour and 24 hour events were modelled). The resulting flood depths and extents are shown in Figure 5.31, Figure 5.32 and Figure 5.33 for Model A, B and C respectively.

The PMF gives the likely maximum flood extent that may be possible within the study area although the chances of this event happening area extremely rare. It is an important tool for estimating the range of impacts and extents that may be expected during a catastrophic event.

In all of the modelled areas the roads and bridges have been overtopped during the Probable Maximum Flood. Many properties and businesses are impacted by the flood and this will be further assessed in the damage assessment and risk assessment components of this study. Area A shows some cross catchment flows from Marshalls Road Tributary to the Grange Burn adjacent to Walls Crescent with a maximum flow rate of 3.85 $m^3/s$. This cross catchment flow was accounted for using an additional downstream boundary within the model.

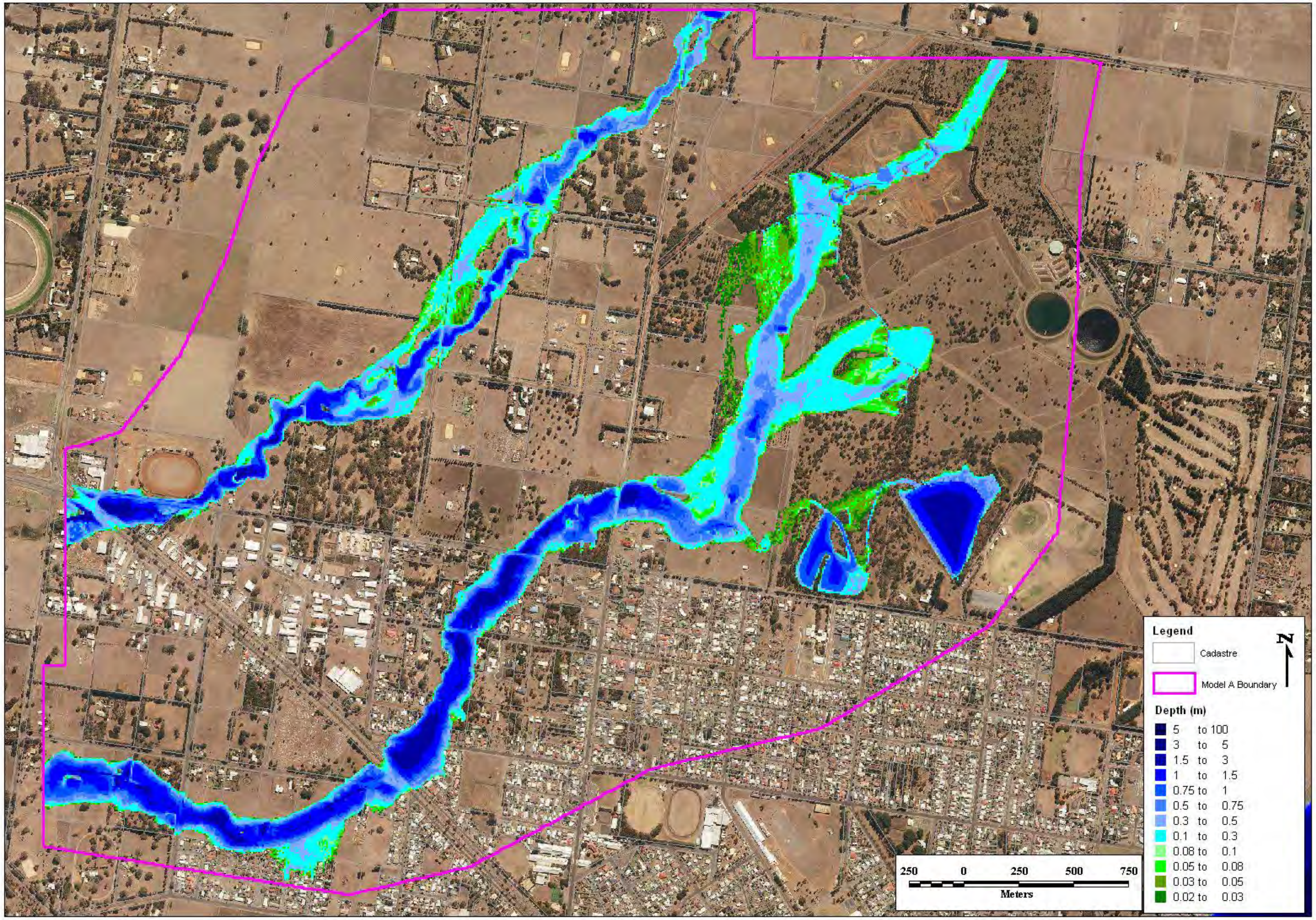



Figure 5.31 Model A – PMF peak flood depths

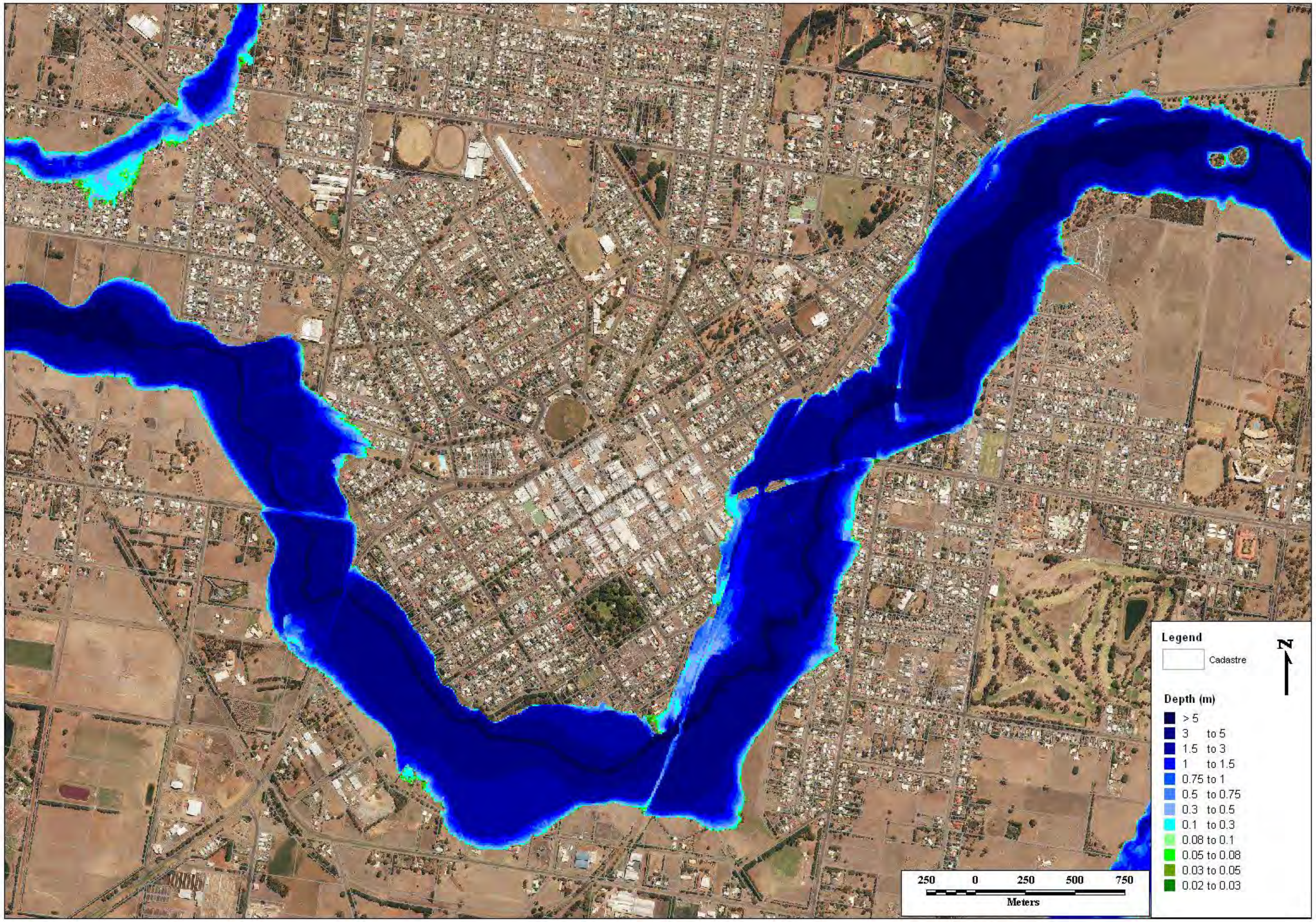



Figure 5.32 Model B – PMF peak flood depths

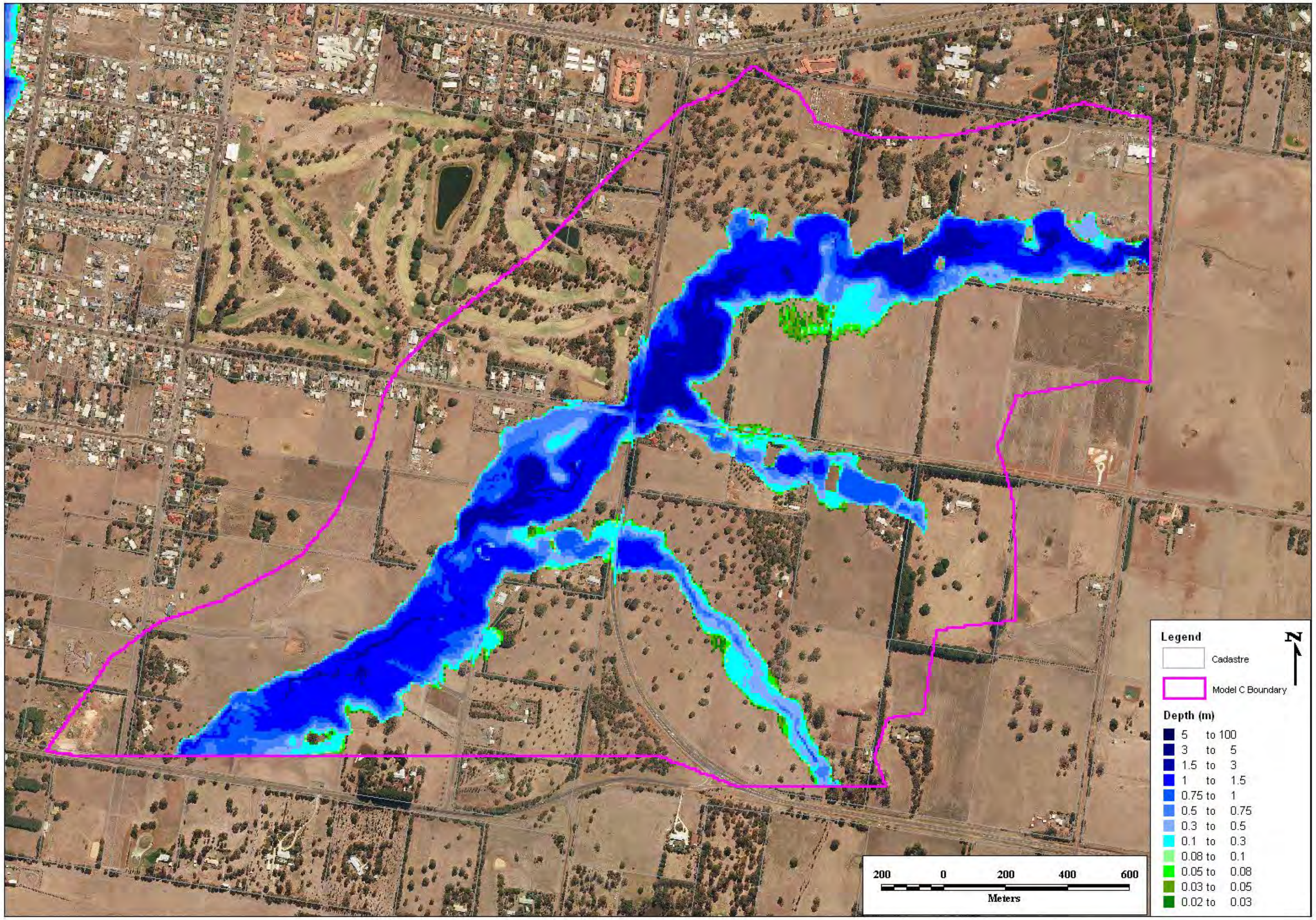



Figure 5.33 Model C – PMF peak flood depths

## 5.7 Sensitivity Analysis

Three sets of sensitivity runs were undertaken in order to assess the uncertainty within the model. The uncertainty was assessed based on the hydrology, as well as the hydraulic roughness. These assessments included:

- Hydrology sensitivity – tested through varying the hygrologic loss rates.
- 'Low' and 'High' roughness – this was achieved through decreasing and increasing the manning's roughness by +/- 20% respectively.
- Individual buildings included in the roughness – this assessment modified the approach to roughness from a lumped roughness approach for properties and buildings to a method which delineated the buildings and reduced the property roughness accordingly.

Each sensitivity assessment is aimed at exploring the uncertainties around the parameter selection. For the Hamilton study this is particularly important due to the high uncertainty around the hydrology due to the very limited recorded streamflow data. Each of the sensitivity assessments are presented in the following sections.

### *5.7.1 Hydrology sensitivity*

The sensitivity assessment has been undertaken in the hydraulic model for three scenarios, based on varying loss parameters in the hydrological model. This is the primary focus of the sensitivity as this is primary uncertainty in the hydraulic model. The loss parameters assessed were:

- Initial loss of 25mm and a continuing loss of 1.5 mm/hr (base case)
- Initial loss of 25mm and a continuing loss of 2.5 mm/hr (case 1)
- Initial loss of 15mm and a continuing loss of 1.5 mm/hr (case 2)

These loss rates refer to the losses applied within RORB when generating the design hydrographs for input into the hydraulic model. The base case has been generated using the predicted 1% AEP peak flow rate at Hamilton as a result of the hydrological study. This is what the analysis would suggest the 1% AEP event should be based on following the detailed analysis of all data sources. However, as there is a high degree of uncertainty in the hydrology due to the extremely restricted quantity of flow data, lower and higher loss rates have been selected for sensitivity assessment to determine the likely impact on flood extents.

The loss rates selected for case 1 and 2 are within the recommended loss ranges for AR&R and are the likely lower and upper estimates for the loss rates. The loss rates have been selected to provide guidance to the likely increase / decrease in peak flows if losses are lower / higher than the base case and to determine the resulting increase or decrease in flood depths and extent in the Hamilton flood plain.

The hydraulic model receives the flows from each scenario at the upstream extent of the model at the Grange Burn at Hamilton (Robsons Road) gauge. A second set of inflows is also received from the catchments contributing to the Hamilton flows from downstream of this gauge. The hydrographs for each event are summarised in Figure 1. The peak flow at Lake Hamilton for each case (all inflows, taken at Lake Hamilton) is summarised below:

- Base Case – 200.5 $m^3/s$
- Case 1 – 153.9 $m^3/s$
- Case 2 – 230.3 $m^3/s$.

The comparison between the base case, case 1 and case 2 have been assessed in the form of flood depth difference plots as well as extent increase plots.

#### *5.7.1.1 Sensitivity Results*

The results of this assessment are presented in the figures following this discussion. The figures are as follows:

- Figure 5.34: Inflow rates for base case, case 1 and case 2
- Figure 5.35: The base case peak depth plot
- Figure 5.36: The base case peak water surface elevation plot
- Figure 5.37: Case 1 depth difference plot (case 1 *less* base case)
- Figure 5.38: Case 2 depth difference plot (case 2 *less* base case)
- Figure 5.39: Flood extents for the base case, case 1 and case 2.

Figure 5.34 outlines the peak flow rate entering the model area at the Hamilton gauge. This includes all inflows from upstream of the gauge. The second inflow is for the area that is located downstream of the gauge but upstream of Lake Hamilton.

Figure 5.35 and Figure 5.36 show the base case results including the maximum flood depth, peak water surface elevation and the extent of the flood. This provides the base to examine the impact of changing the loss rates for case 1 and case 2.

Figure 5.37 shows the results of the depth difference plot for case 1 in relation to the base case. The decrease in depths ranges was on average in the order of 20 to 30 cm across the floodplain. The flood extent was also reduced as compared to the base case. There are some noticeable decreases in flood extent in and around Ballarat Road, as well as upstream of the railway bridge.

Figure 5.38 shows the results of the depth difference plot for case 2 in relation to the base case. The increase in depths ranges was on average between 10 and 20 cm, with some areas exceeding a 30 cm increase. Noticeably in this case the railway is overtopped adjacent to Ballarat Road and Lake Hamilton is overtopped to the north and east of the main spillway.

Figure 5.39 shows the direct comparison of the three cases in the form of flood extents. The areas where case 1 is reduced against the base case and the areas where case 2 exceeds the base case are clearly shown. It is interesting to note that the change in flood extent between the base case and case 2 is not significant, with the key area of increase being upstream of Ballarat Road.

#### *5.7.1.2 Discussion*

From the analysis it is evident that reducing the loss rates for the Grange Burn at Hamilton catchment increases the flood extents and depths. The key observations from the sensitivity assessment include:

- Case 1 decreases the depths by approximately 20 - 30 cm on average.
- Case 1 decreases the 1% AEP flood extent as compared to the base case.
- Case 2 increases the depths by approximately 10 - 20 cm on average.
- Case 2 has a maximum increase of 30 cm in the floodplain.
- Case 2 causes Lake Hamilton to overtop to the north and east of the main spillway.
- Case 2 shows small increases in flood extent as compared to Case 1.
- Figure 5.39 shows the change in flood extent between the three cases.

For a comparison, the peak flow rate from the 1946 event has been estimated at approximately 200 $m^3/s$, which is equal to the flow estimates for the base case. The use of the base case loss rates imply that the March 1946 event is roughly equivalent to the 1% AEP event. As we have no understanding of the magnitude of this event, this may be an appropriate flow rate to define planning controls as the 1946 is the largest record flood for the Grange Burn. The rainfall associated with the March 1946 event was approximately a 0.2% AEP rainfall event (refer to RM2338 V0.2). Based on the analysis of antecedent conditions, the March 1946 event was likely to have occurred on a dry catchment with high losses, so it is not unreasonable that the 0.2% AEP rainfall event would generate a 1% AEP runoff response.

From the analysis it is observed that the case 2 loss rates produce a flood extent and a catchment response that may exceed the likely 1% AEP event. This observation is made as the Lake Hamilton dam wall was significantly overtopped during this event. The report on the Lake Hamilton Spillway / Grange Burn Flood Investigations (GHD, 1987) suggests that Lake Hamilton would not be likely to overtop significantly during the 1% AEP event. It should be noted that GHD reported the 1% AEP event at 200 $m^3/s$.

Overall the assessment indicates that the 1% AEP peak flow rate as defined by the base case is an appropriate set of loss rates for the development of the flood planning controls. The case 1 results show that a slight increase in loss rates can lead to a relatively large reduction in flood extent, whereas case 2 demonstrates that a further reduction in loss rates does not significantly increase the flood extent.

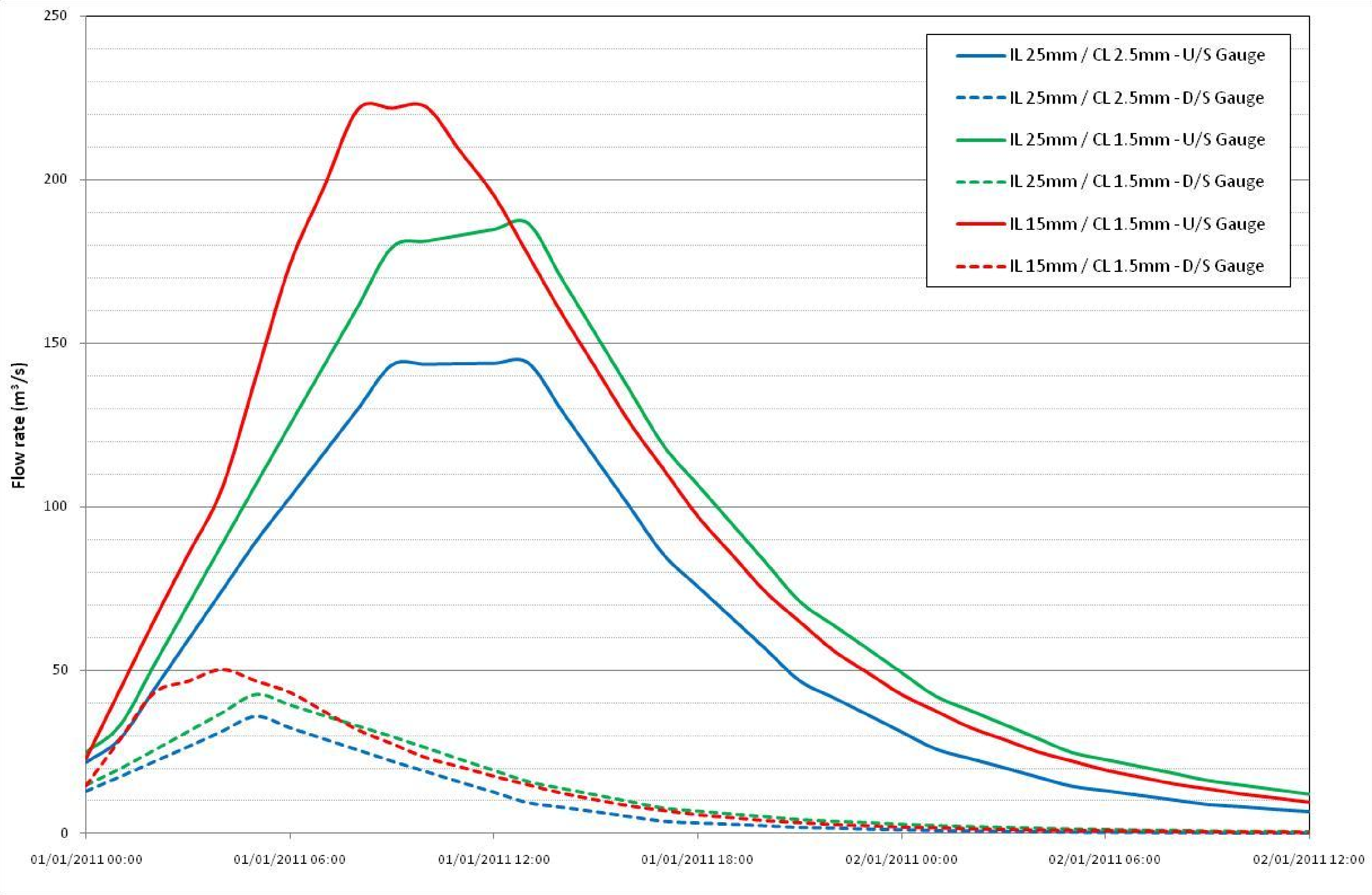



Figure 5.34 1% AEP design flows – Base case (IL 25mm / CL 1.5 mm/h, case 1 (IL 25mm / CL 2.5 mm/h) and case 2 (IL 15mm / CL 1.5 mm/h)

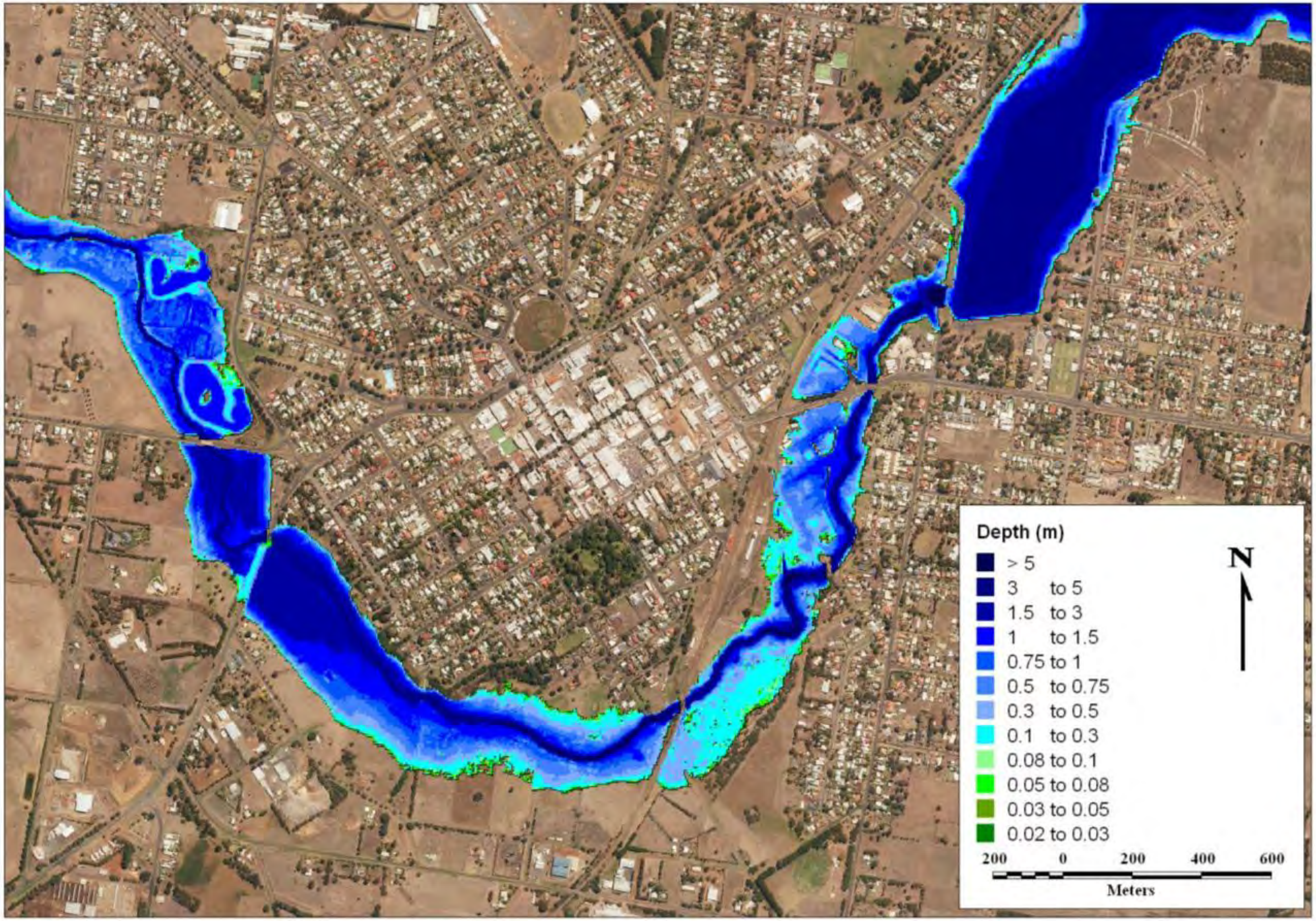



Figure 5.35 1% AEP peak depths using an initial loss of 25mm and a continuing loss of 1.5 mm/hr (base case)

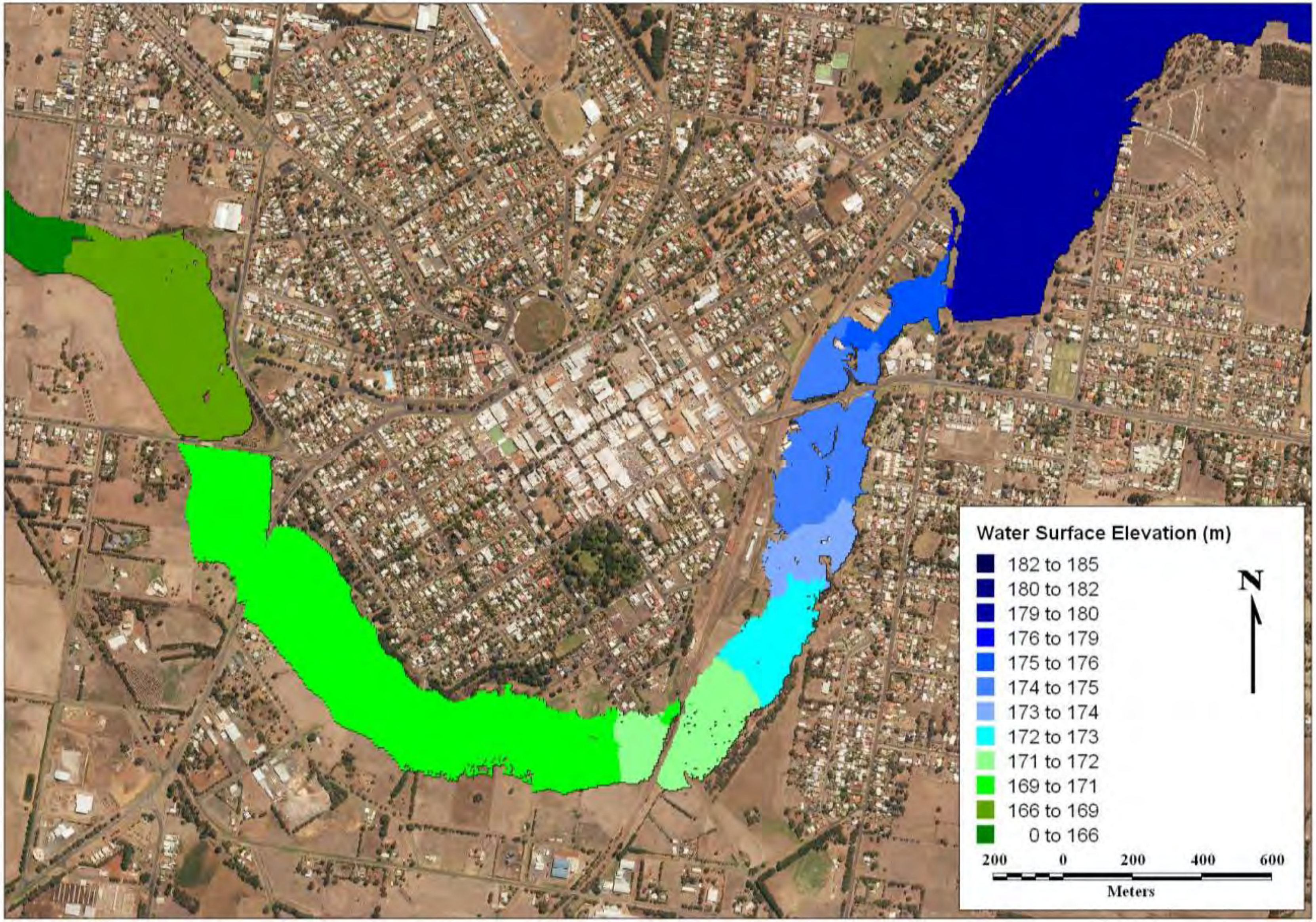



Figure 5.36 1% AEP peak water surface elevation using an initial loss of 25mm and a continuing loss of 1.5 mm/hr (base case)

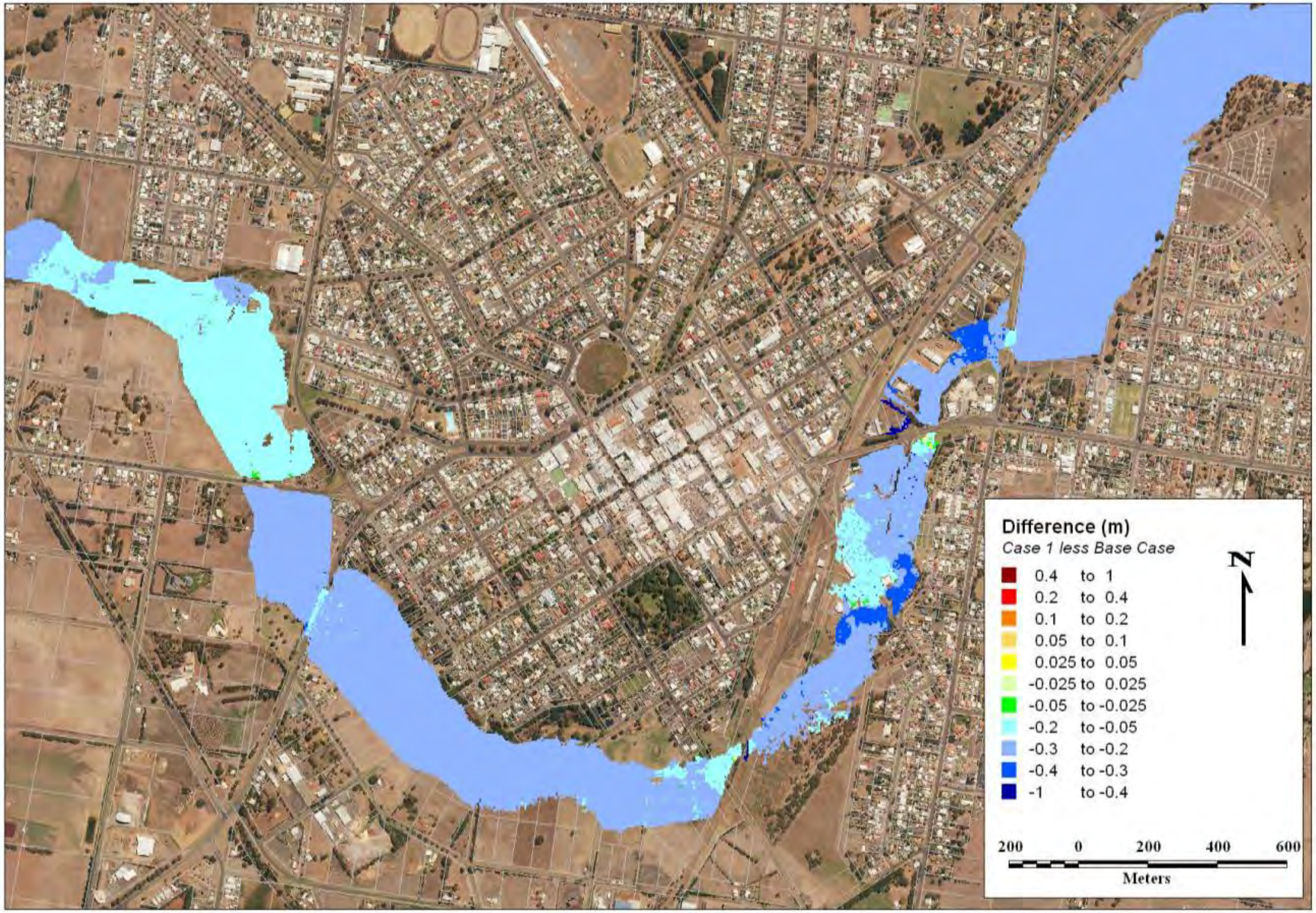



Figure 5.37 1% AEP peak water surface elevation difference plot (initial loss of 25mm and a continuing loss of 2.5 mm/hr [Case 1] *less* base case)

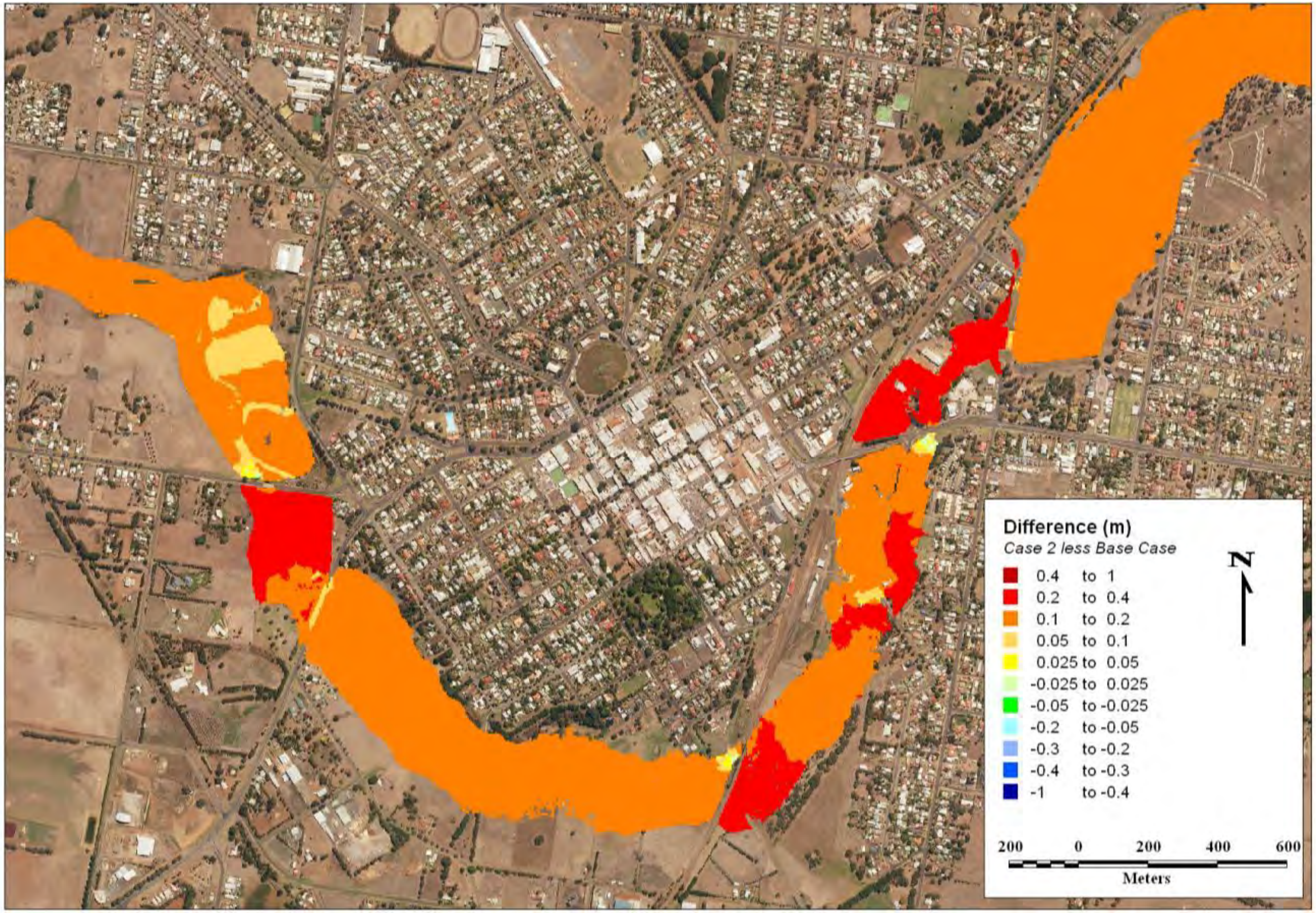



Figure 5.38 1% AEP peak water surface elevation difference plot (initial loss of 15mm and a continuing loss of 1.5 mm/hr [Case 2] *less* base case)

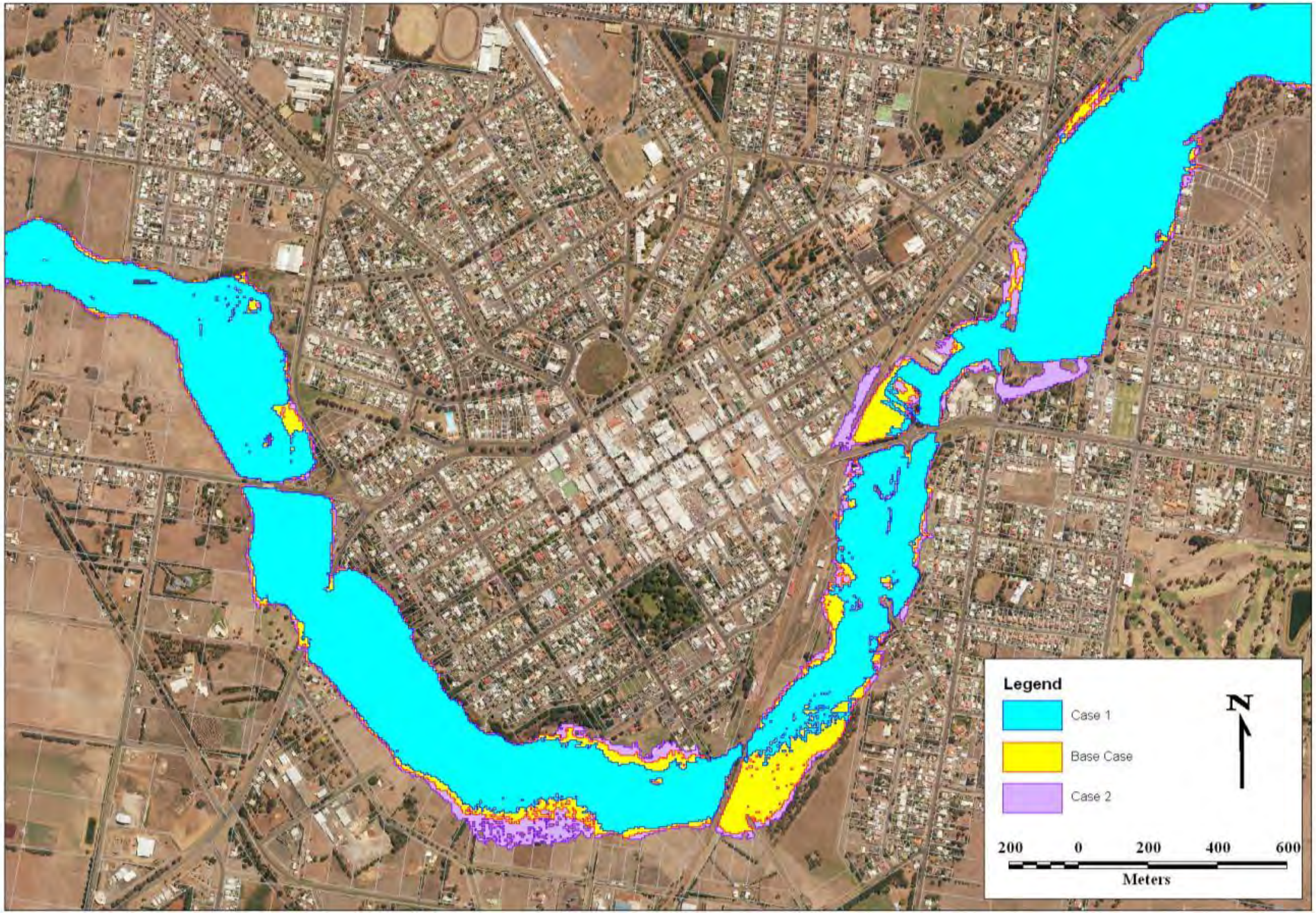



Figure 5.39 1% AEP peak water extents (base case, IL 25mm CL 1.5mm/h and IL 15mm CL 1.5mm/h)

### 5.7.2 Roughness sensitivity

An additional sensitivity assessment was undertaken on the roughness used within the hydraulic model in order to provide an upper and lower range for the uncertainty for the Manning's Roughness. For these model run the roughness was modified by +/- 20%. The roughness parameters used for the sensitivity assessment are summarised in Table 5.7.

Table 5.7 Calibrated Roughness Parameters, Mannings 'n'

| Parameter | Roughness Manning's 'n' | Low roughness (-20%) | High roughness (+20%) |
|---|---|---|---|
| Roads | 0.018 | 0.014 | 0.022 |
| Railway Line and Embankments | 0.02 | 0.032 | 0.048 |
| Main river channel | 0.04 | 0.036 | 0.054 |
| Minor river channel | 0.045 | 0.04 | 0.06 |
| Main floodplain | 0.05 | 0.048 | 0.072 |
| Moderate floodplain | 0.06 | 0.056 | 0.084 |
| Dense floodplain | 0.07 | 0.12 | 0.18 |
| Partly Residential | 0.15 | 0.16 | 0.24 |
| Residential | 0.2 | 0.4 | 0.6 |
| Commercial | 0.5 | 0.016 | 0.024 |

The results of the analysis are summarised in the form of peak water surface elevation plots. These plots are shown in Figure 5.40 and Figure 5.41 for the high and low roughness sensitivity runs respectively.

The high roughness case shows very small increases in the flood extent but on average an increase in flood depth of between 10 and 20 cm. The important observation is that the flood extent has not increased significantly (increase in flood extent shown with the magenta areas). This is important as it demonstrates that any uncertainty in the roughness will not result in any substantial changes to the flood extent for the Hamilton Flood Investigation.

The low roughness case shows that there is a slight decrease in the flood extent as compared to the existing conditions. Again this is not a substantial change and this demonstrates that any uncertainty in the roughness will not result in significant changes to the flood extent for Hamilton. The peak flood depths were reduced by on average 10 to 20 cm due to the reduction in the roughness by 20%.

Overall the high and low roughness scenarios demonstrate that a +/- 20% change in roughness will ultimately lead to changes in the flood depth of +/- 20 cm. Importantly the sensitivity analysis shows that the +/- 20% change in roughness does not significantly change the existing flood extent.

##### *5.7.2.1 Impacts of Vegetation Clearing on Flooding*

The roughness sensitivity provides an insight into the likely maximum benefit that clearing vegetation would have on the flood extent and depths within the system. It is important to note that clearing the vegetation within the main channel would have less of an impact that the -20% roughness case as this assessment reduced the roughness for the entire model by 20%. This discussion will provide some guidance on the likely benefit of clearing vegetation from the main channel as a mitigation option.

For the reduced roughness case (-20% roughness) the flood extent was only marginally reduced and the average reduction in flood depths was between 10 and 20 cm. If vegetation clearing was targeted at the main channel and floodplain for the full Grange Burn system then it would be expected that the reduction in flood depths would be much less than this 10 to 20 cm as only the main channel and riparian zone roughness have been reduced. Perhaps a change of less than 5 cm could be expected.

Overall it would be expected that removing vegetation from the main channel would have a very small impact on the flood depth and extent as the depths through the main floodplain during the 1% AEP event for example are approximately 5 m deep in the main channel and over 0.5 m deep on the majority of the floodplain.

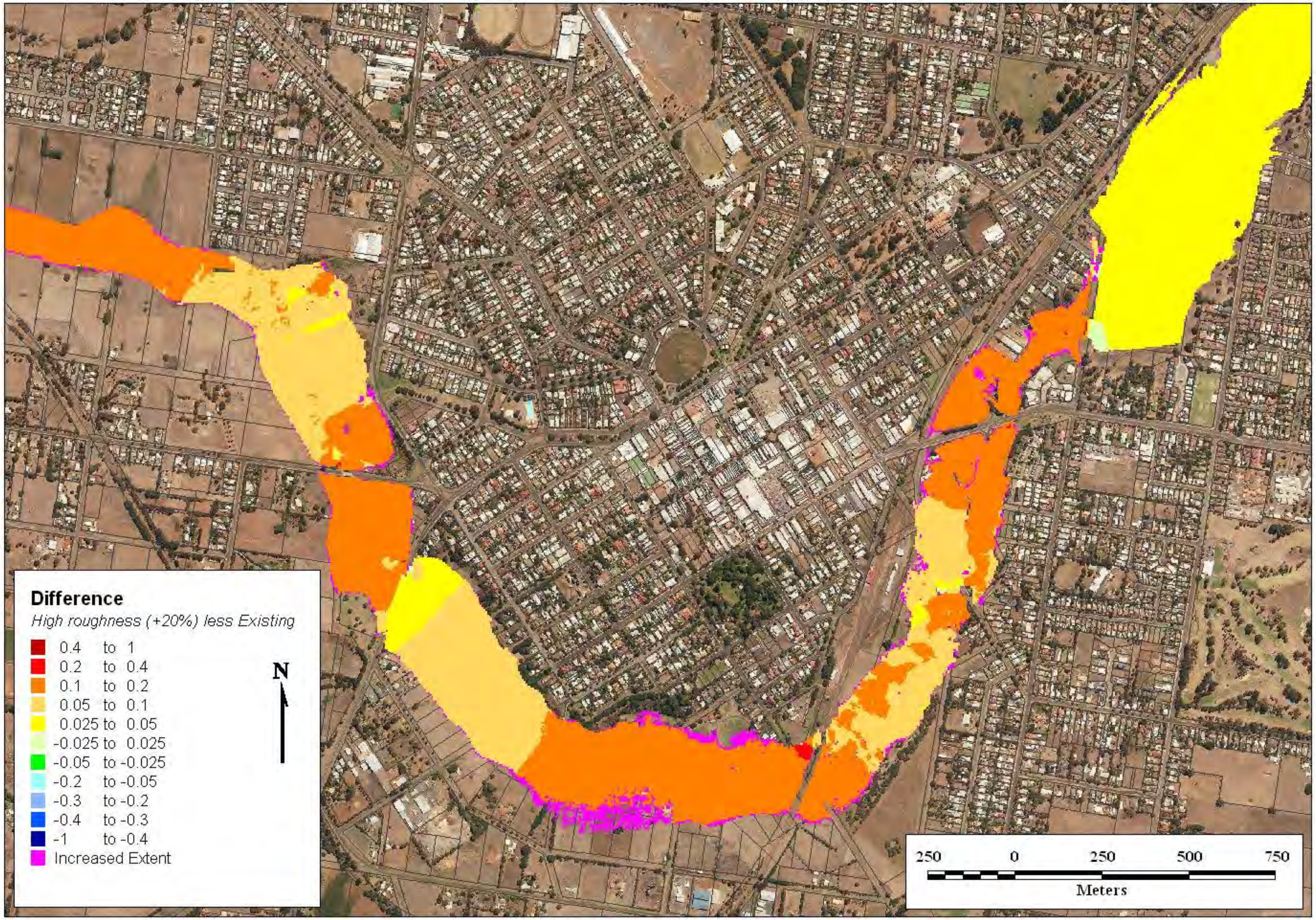



Figure 5.40 High roughness sensitivity run – 1% AEP water surface elevation difference (+20% roughness *less* existing conditions)

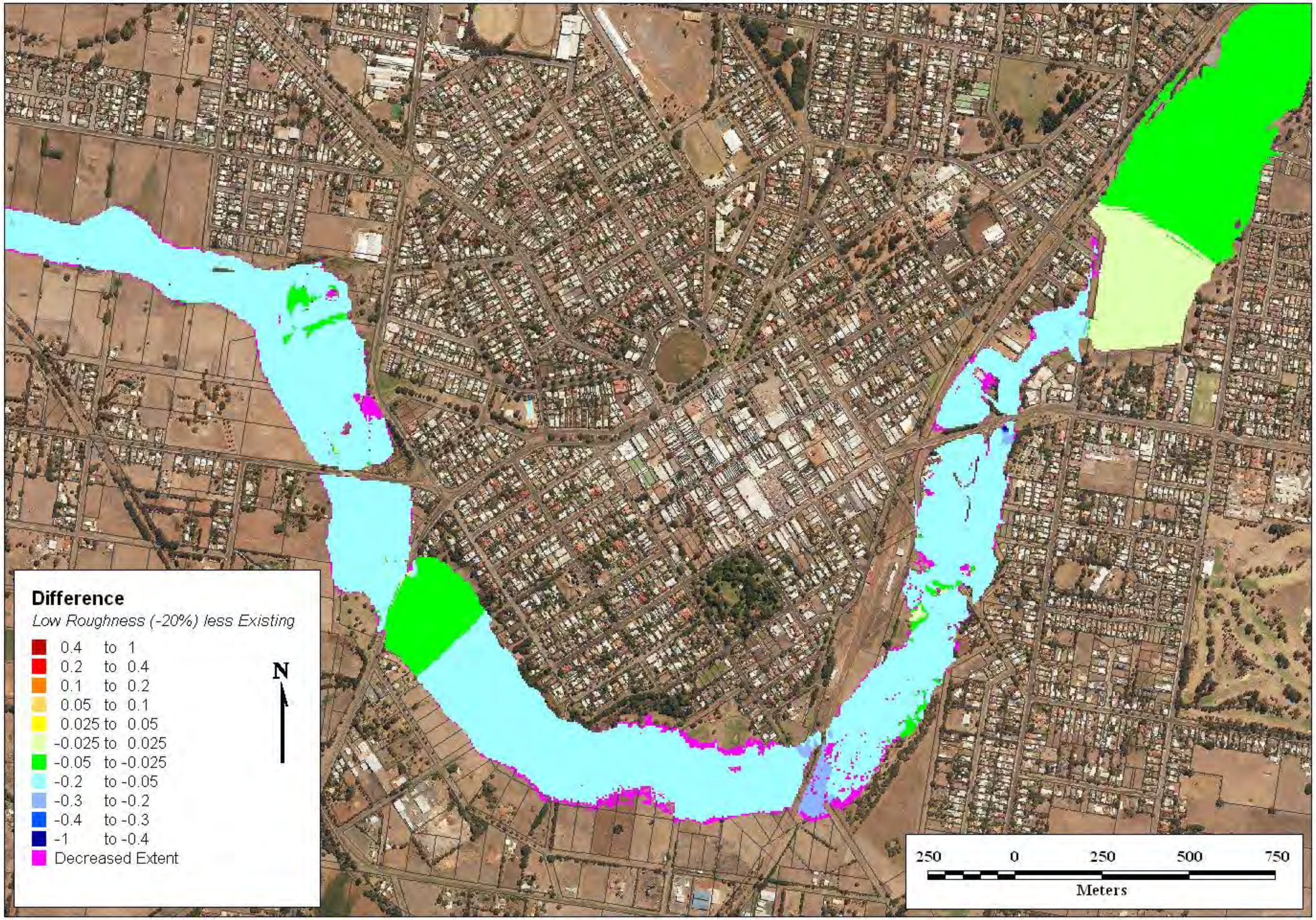



Figure 5.41 Low roughness sensitivity run – 1% AEP water surface elevation difference (-20% roughness *less* existing conditions)

### *5.7.3 Roughness – Delineated Buildings Assessment*

The final assessment for the roughness was to examine the influence of delineating the individual properties within the model and using these properties to modify the roughness. The current model uses a combined roughness approach which provides a roughness parameter across a property which encapsulates the fences, buildings, obstructions, gardens etc. This sensitivity analysis will modify this approach to delineate the buildings on an individual scale. This allows for the roughness of the building to increase and subsequently the remaining area of the property the roughness will decrease.

This run has been provided to determine if there will be any substantial impacts to the flood extent within the model by using the approach of a combined property roughness versus the delineated buildings approach.

The delineated building roughness layer is summarised in Figure 5.42 and the difference plot which compares the results between this model run and the current roughness results for the 1% AEP flood event is summarised in Figure 5.43.

The results of the sensitivity assessment indicate that for the majority of the floodplain there is no change to the peak flood extents or depths. This is expected as the majority of the Hamilton floodplain does not interact with buildings on properties.

Upstream of Ballarat Road there is an area where the flood depths increased by between 2.5 to 5 cm but this area is largely restricted to the main floodplain. Downstream of Ballarat Road there is some interaction with the properties around Holden Street and as a result there is a small change in peak flow behaviour. These changes were expected as the roughness in this area was modified. Overall the changes are within +/- 5 cm and were restricted to a small area of the floodplain.

This sensitivity assessment demonstrates that there was no change in flood extent and a small change in flow behaviour. Overall the modelling of the roughness using the delineated buildings did not change the model results significantly enough to change the approach that was ultimately adopted.

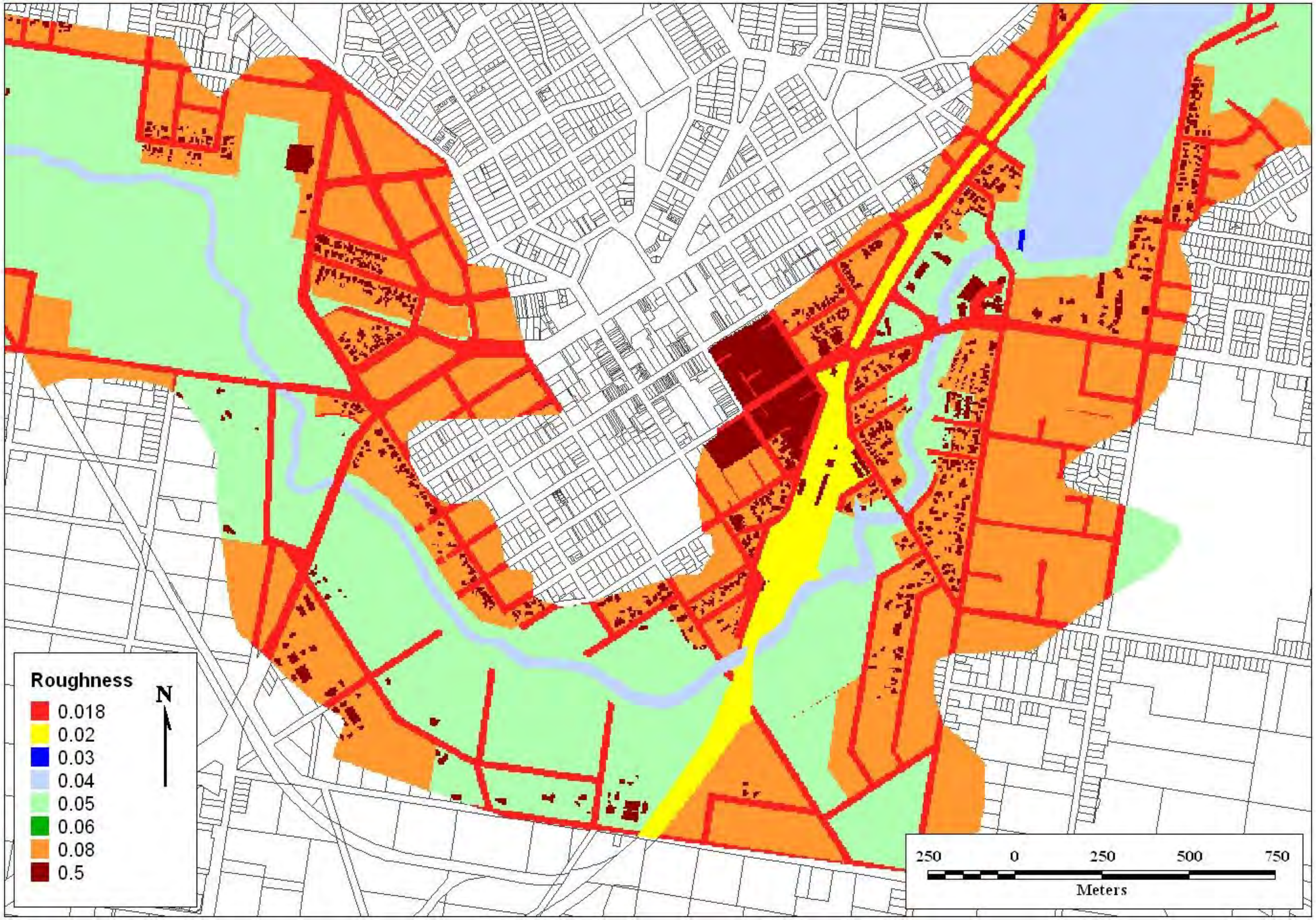



Figure 5.42 Roughness with individual buildings

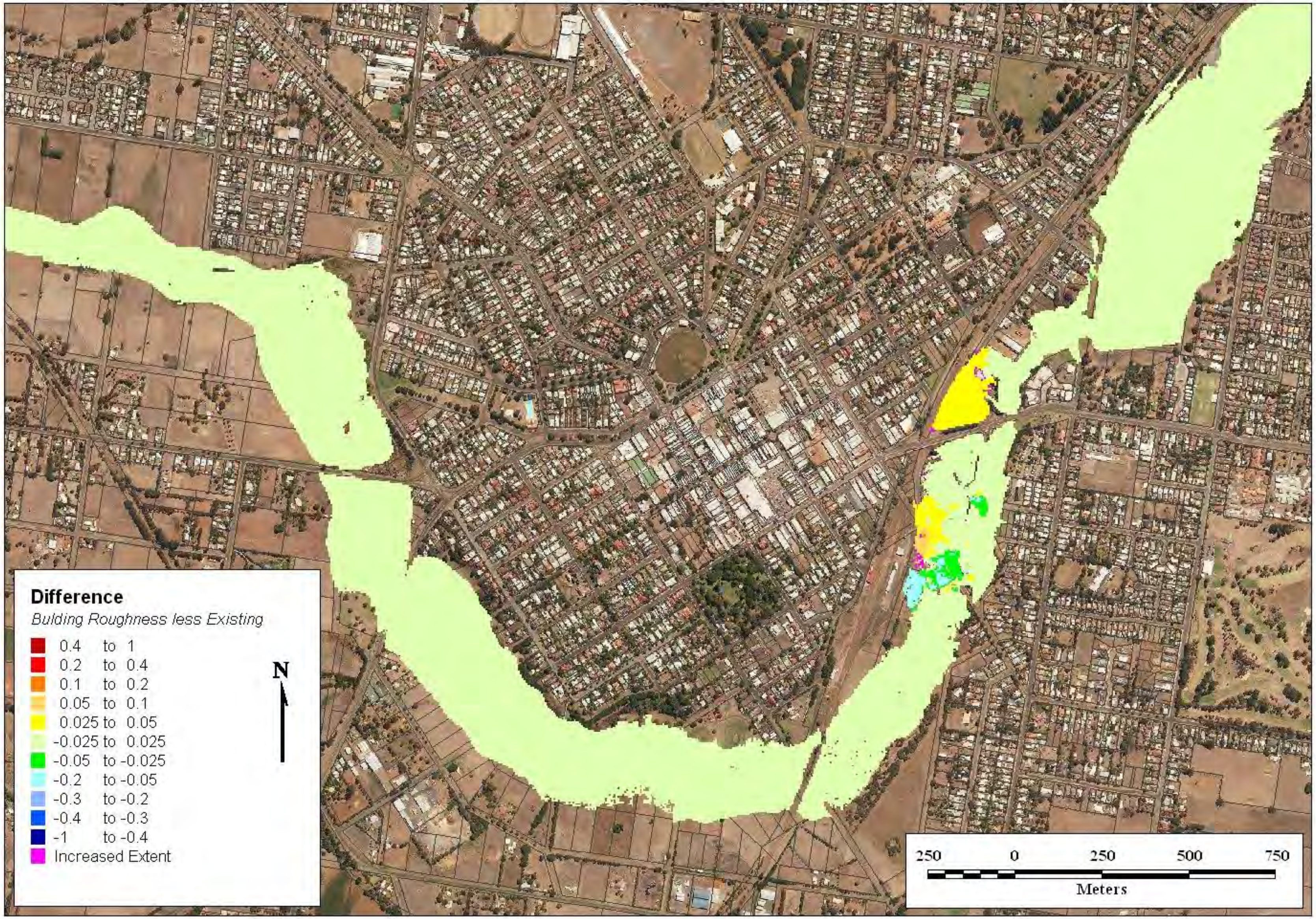



Figure 5.43 Roughness with individual buildings - 1% AEP water surface elevation difference (Individual building roughness *less* existing conditions)

## 5.8 Climate Change Assessment

The climate change hydrology was developed in Section 4.8 for the Grange Burn, as well as for the three tributaries modelled. The primary aim of the hydrological assessment was to determine the possible impact of predicted ranges of climate change on the peak flows passing through the Hamilton region. In order to determine the impact that the increased flood peaks have on the catchment floodplain the 32% increase in intensity climate change hydrograph was run through the hydraulic model for the 1% AEP flood event and compared to the existing 1% AEP model results. The assessment of the climate change scenario against the existing conditions is shown in the form of a depth difference plot for the North Eastern tributaries (Model A), Grange Burn (Model B) and Petschels Lane Tributary (Model C) in Figure 5.44, Figure 5.45 and Figure 5.46 respectively. These figures show the climate change results less the existing results with areas of increased flood extent shown in magenta.

For the North Eastern tributaries climate change is expected to increase the peak flows but this did not translate into a significant increase in flood extent. The peak depths due to the flood event increased but due to the shape of the flood plain the extent was largely the same as for the existing conditions. Peak depths through the main channel increased by approximately 10 – 20 cm through the Marshalls Road Tributary and by approximately 10 cm for the Kennys Road Tributary.

For the Marshalls Road Tributary, the key area regarding flood damages is near King Street upstream of Coleraine Road. Within the parkland the peak water level increased by approximately 12 cm. The peak depths across Coleraine Road are also expected to increase by approximately 10 cm due to climate change.

For Kennys Road Tributary the main change to the flood extent and depths was that at the downstream end of the model Coleraine Road is now overtopped in the climate change sceanrio (32% increase in rainfall intensity). The road is overtopped to a depth of approximately 10 cm, whereas under the existing 1% AEP event Coleraine Road was not overtopped.

For the Grange Burn climate change is predicted to increase the current 1% AEP flood event peak flow rate to an equivalent of approximately the current 0.2% AEP event. This caused the flood extent to increase throughout the entire flood plain. At Lake Hamilton the dam wall is now significantly overtopped during the 1% AEP climate change scenario both to the north and south of the spillway. Additional properties and building are impacted due to this overtopping of the dam wall. Upstream of Ballarat Road there are additional businesses that would be impacted during this event, including the cement refinery on the eastern side of the Grange Burn. The climate change 1% AEP extent would be expected to overtop the railway line.

Downstream of Ballarat Road in the Apex Park area the increase in the flood extent is unlikely to impact significantly more properties or buildings. However, the peak flood depth increases by approximately 50 cm which is likely to cause significantly more damage to buildings in this area that are inundated. Further downstream the peak depths over Portland Road would be expected to increase by approximately 40 cm over the road and bridge. The Dartmoor-Hamilton Road is also now expected to be overtopped during the climate change 1% AEP event to a depth of between 10 - 20 cm.

For the Petschels Lane Tributary there was an increase in flood depth by approximately 10 cm across the floodplain, however the flood extent did not increase significantly due to the expected impact of climate change. Peak depths across the Hamilton Hwy increased from approximately 20 cm to 30 cm which may lead to longer road closure times.

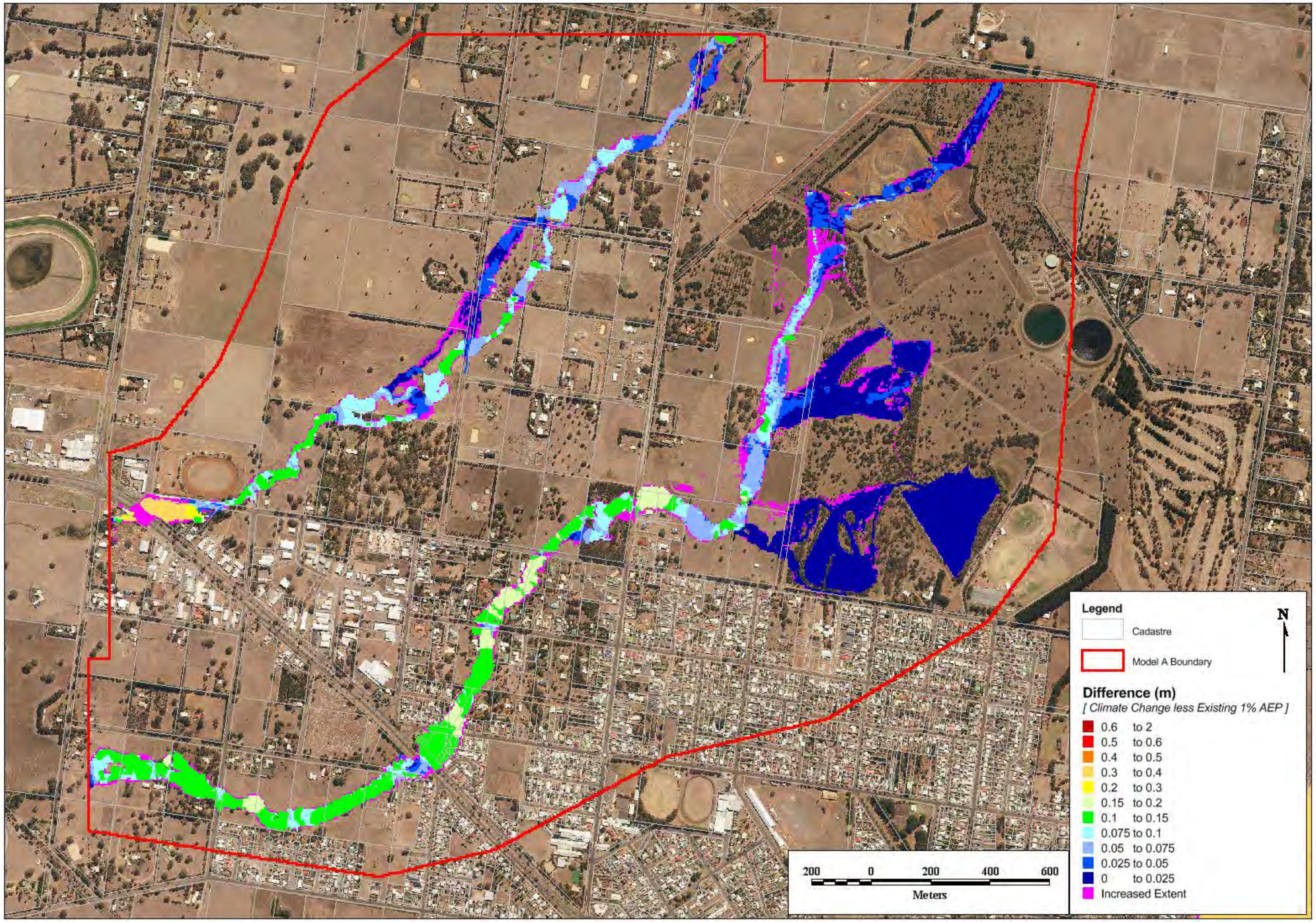



Figure 5.44 Model A – Climate change (32%) compared to existing conditions for the 1% AEP event

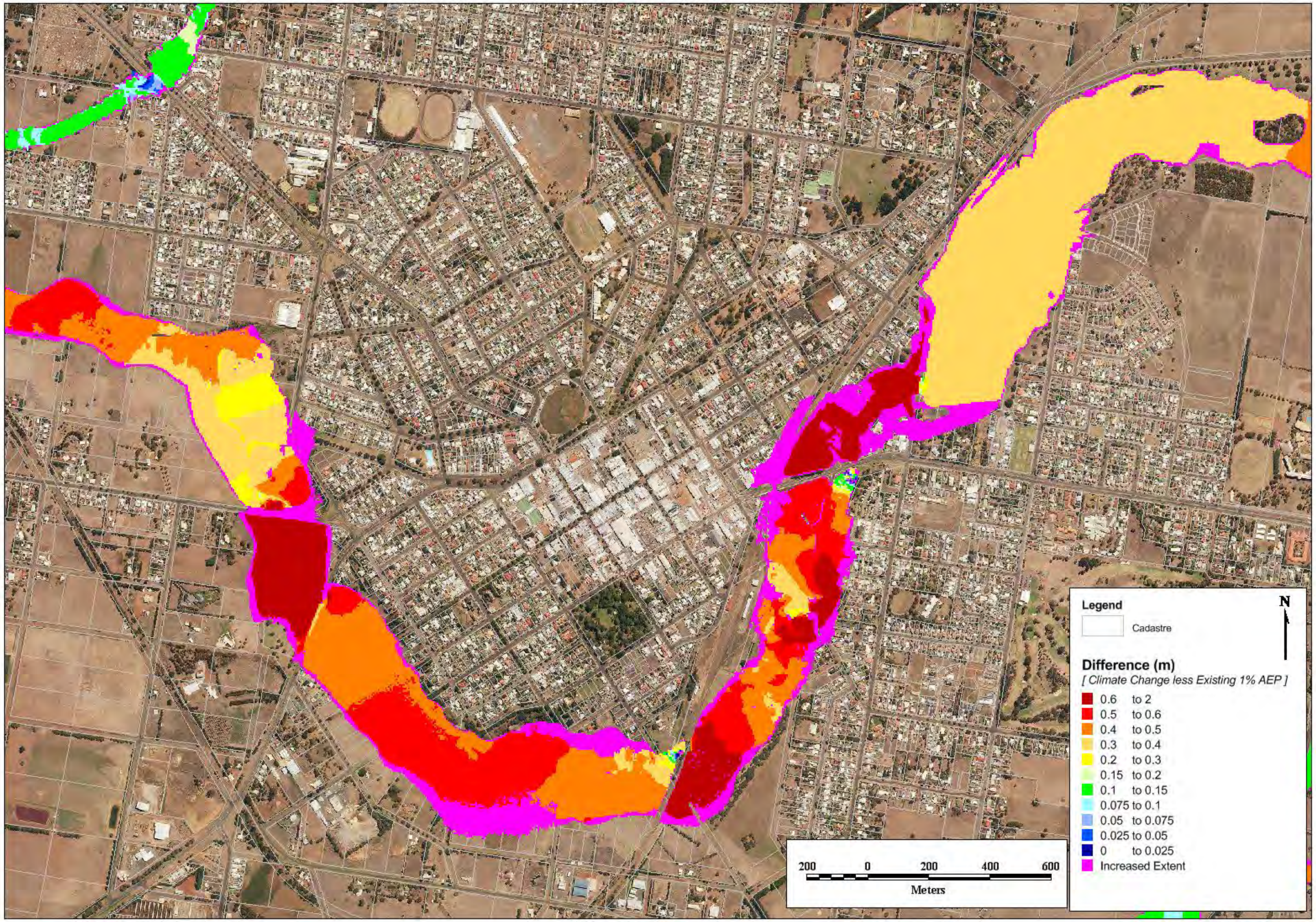



Figure 5.45 Model B – Climate change (32%) compared to existing conditions for the 1% AEP event

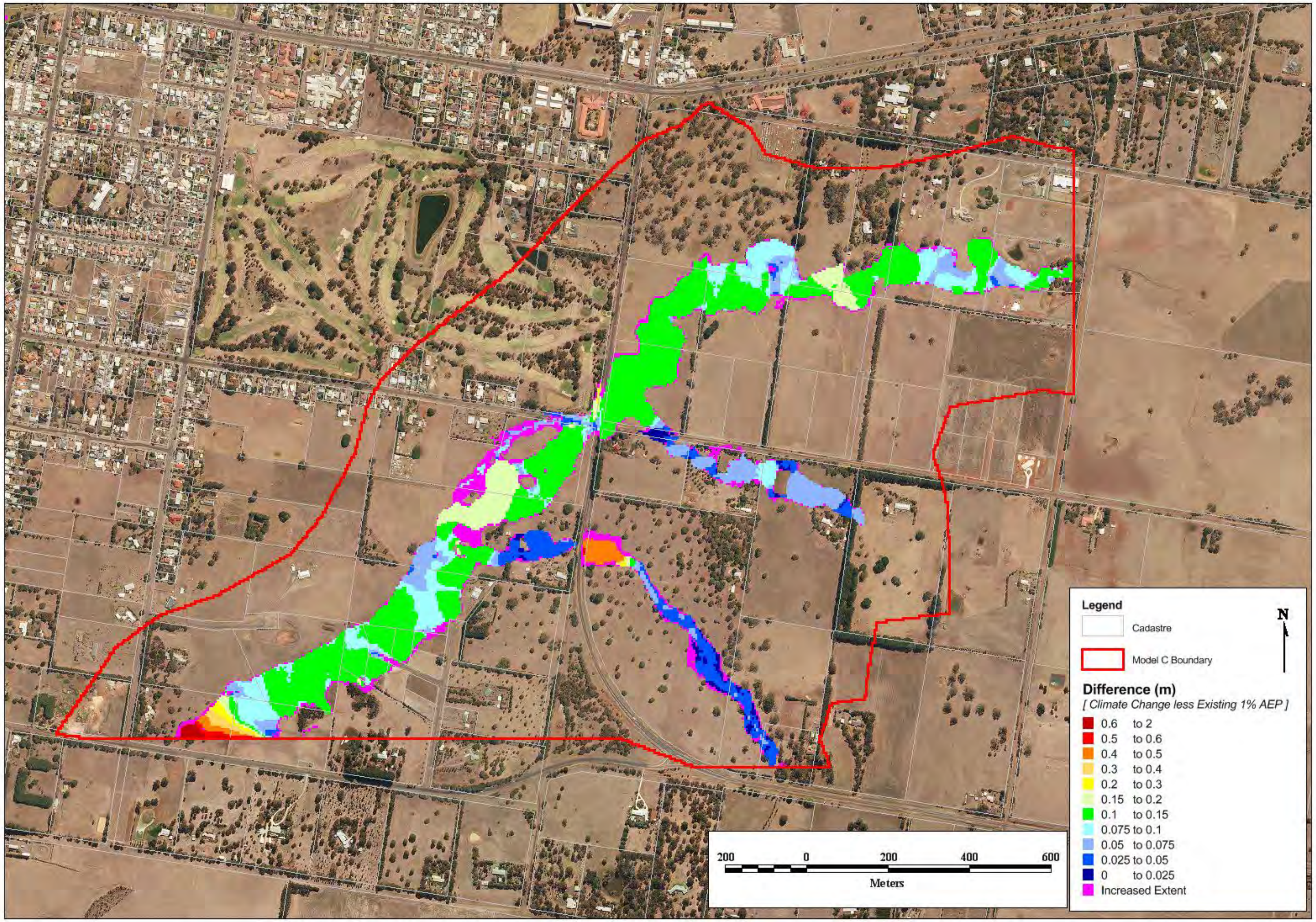



Figure 5.46 Model C – Climate change (32%) compared to existing conditions for the 1% AEP event

# 6 DATASETS AND MAPPING

The calibrated SOBEK model for Hamilton was used to analyse the extent, location and depths for the 20%, 10%, 5%, 2%, 1%, 0.5% AEP events and the Probable Maximum Flood. Key outputs from the project are 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:

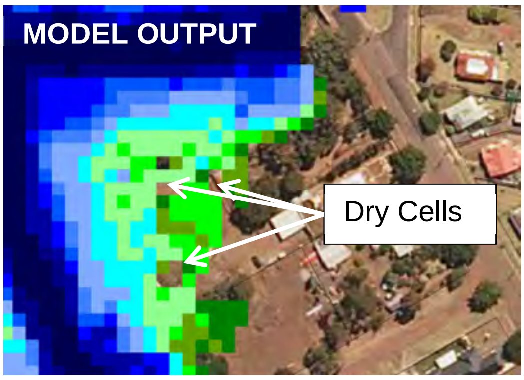
- Peak flood depths for all flood events (Figure 5.13 to Figure 5.33)
- Flood extents for all flood events
- Flood planning controls (flood overlays)
- 1% AEP event
    - Hazard class maps
    - Extent with water surface elevation contours (200mm contour intervals)
    - Velocity map
- Properties impacted during each flood event will be shown on each flood map.

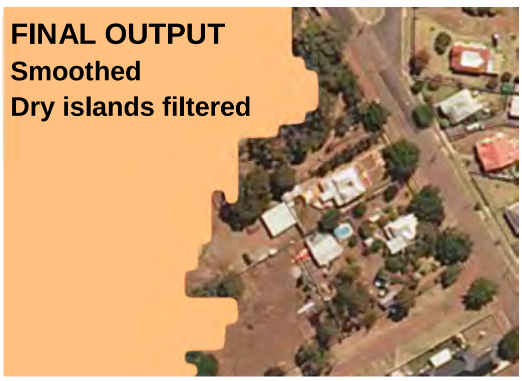
All datasets and mapping have been supplied along with the final report as the final deliverables to the project. The flood depths for all flood events have been summarised as part of the hydraulic model results in (Figure 5.13 to Figure 5.33).

## 6.1 Design flood extents

The final flood extents for the design flood events for Hamilton 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 merely nuisance waters and are not expected to cause any damage.
- Wet and dry islands were removed where they were less than 4 gridcells (125 m²). This ensures consistency and continuity in the mapped flood extent for planning purposed.
- 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 to Figure 6.3 for Model A, Model B and Model C respectively.

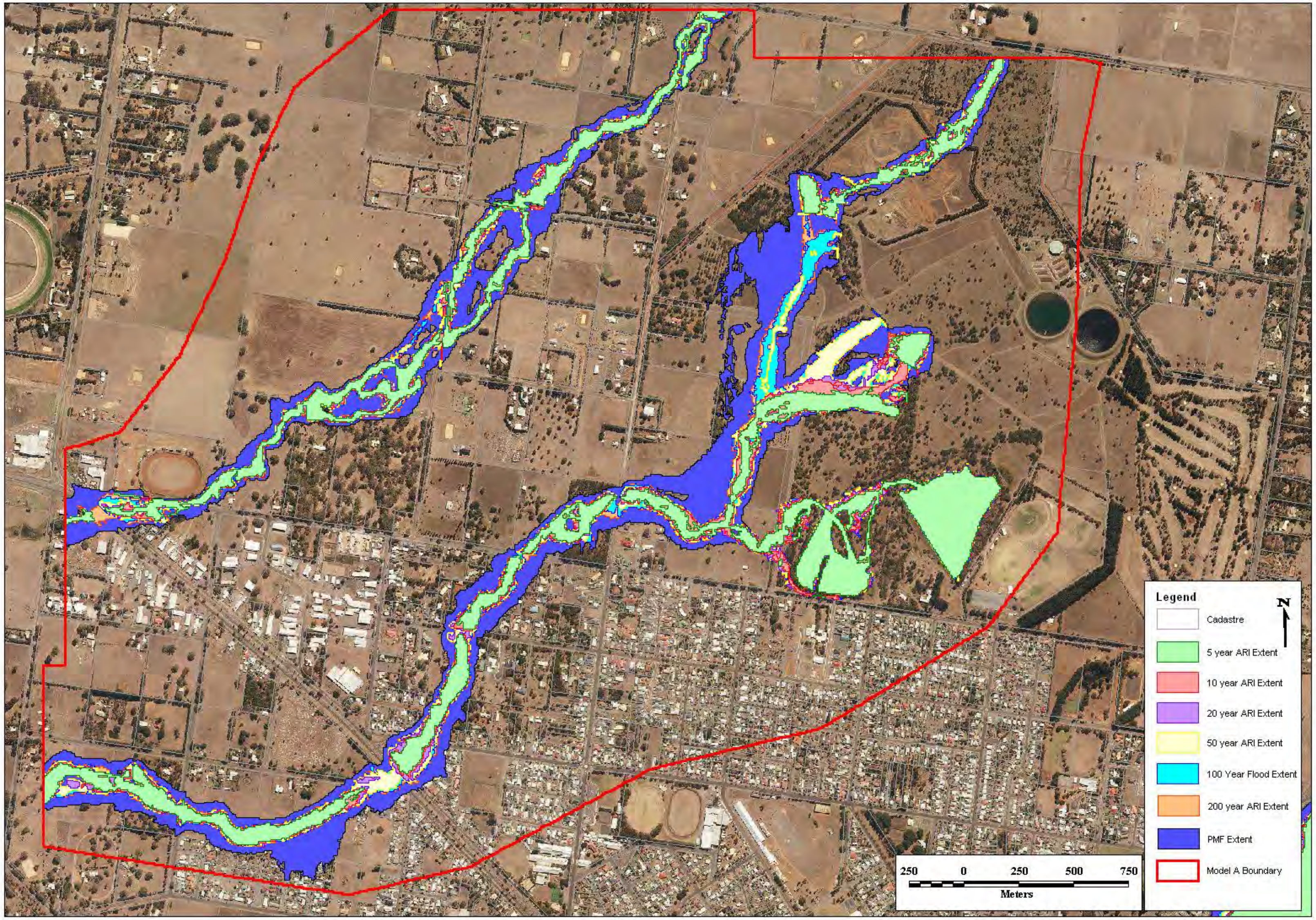



Figure 6.1 Model A - Flood extents for the 20%, 10%, 5%, 2%, 1%, 0.5% AEP and the PMF

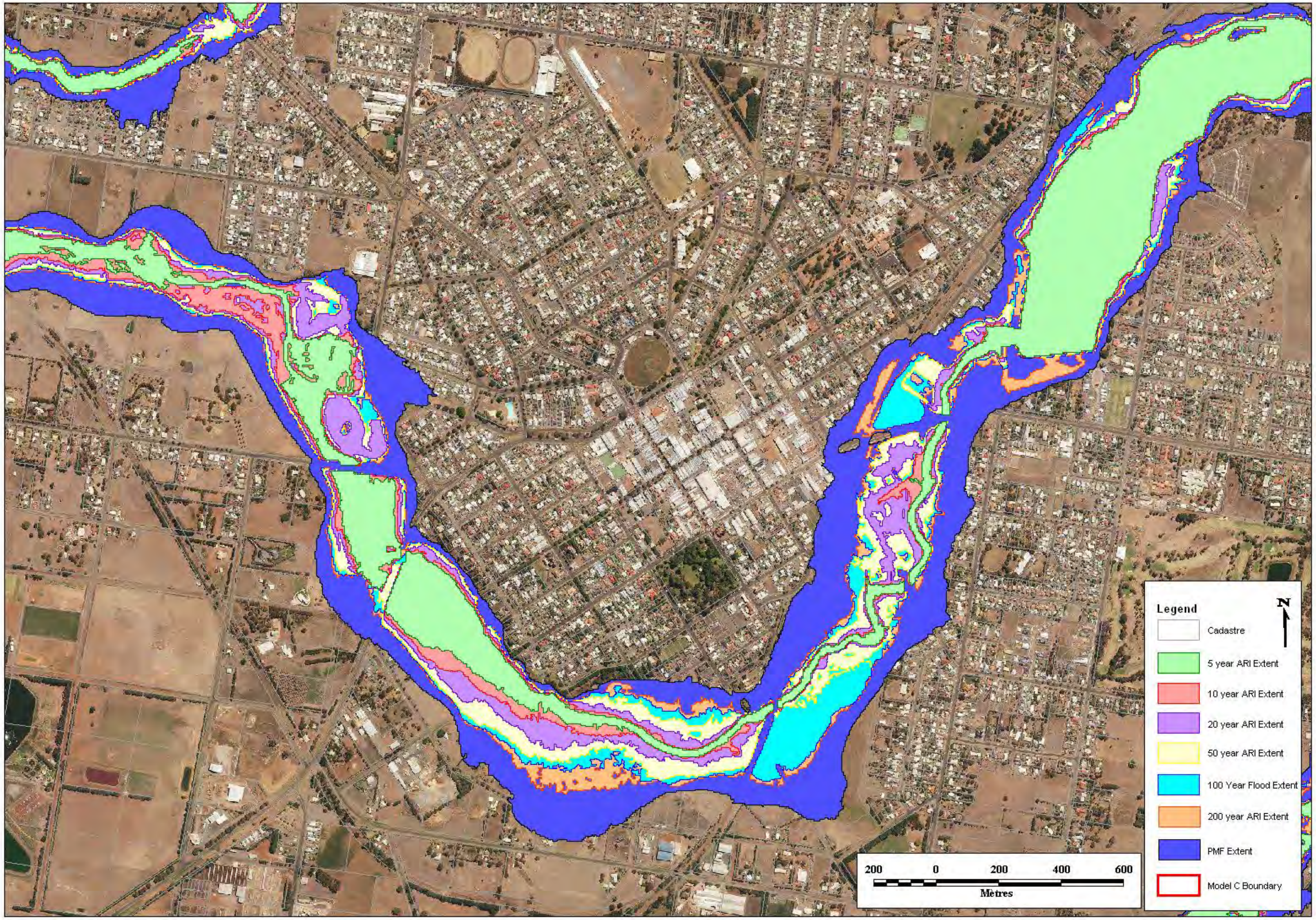



Figure 6.2 Model B - Flood extents for the 20%, 10%, 5%, 2%, 1%, 0.5% AEP and the PMF

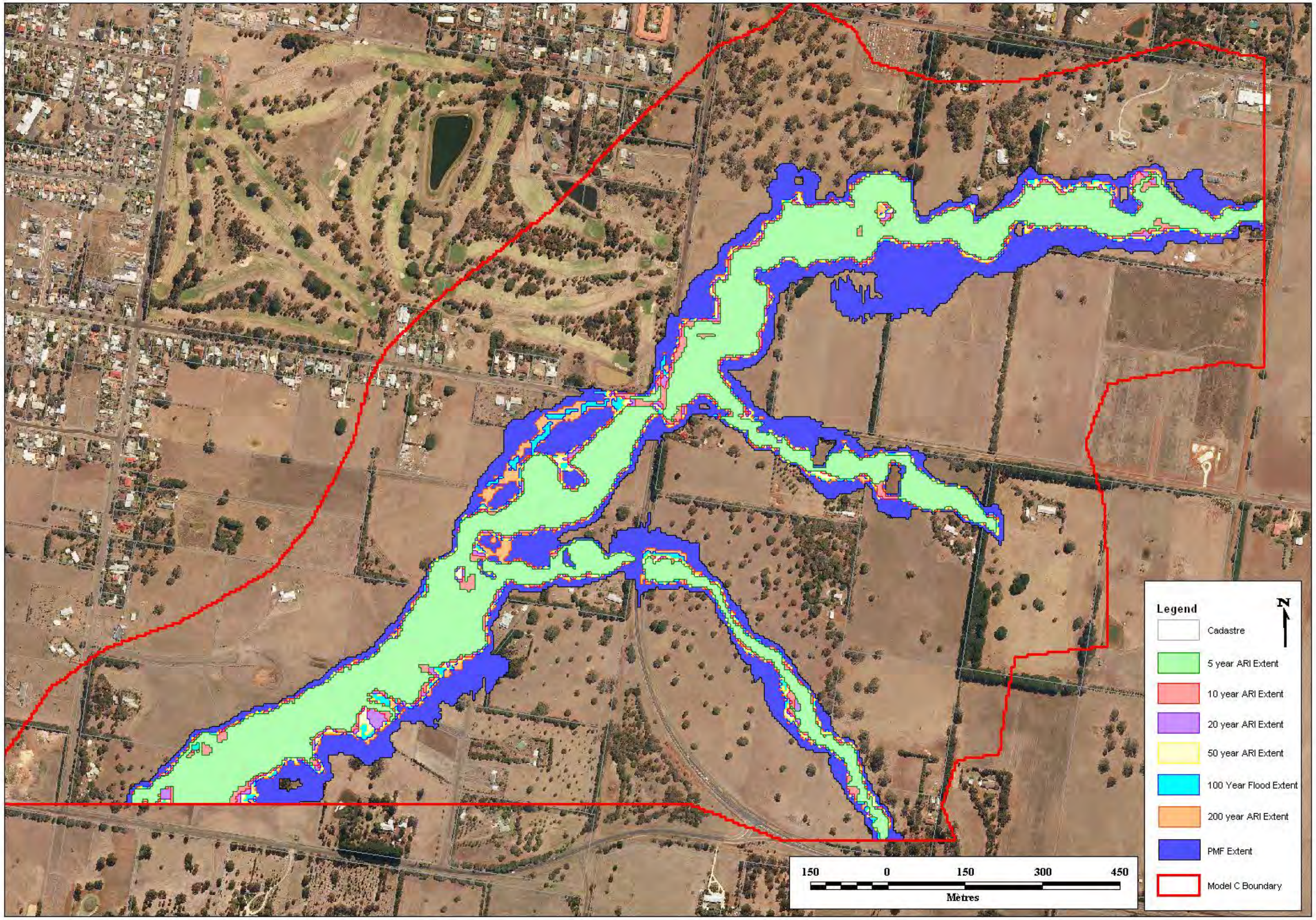



Figure 6.3 Model C - Flood extents for the 20%, 10%, 5%, 2%, 1%, 0.5% AEP and the PMF

## 6.2 Flood Planning Controls

The current planning framework for the floodplain is encapsulated in the *Southern Grampians Planning Scheme*. The Planning Scheme, prepared in accordance with Victorian State Planning Policy Framework (VPP), documents all planning controls in the study area. The scheme consists of a written document as well as maps, plans and related documents. It contains (as outlined in the accompanying User Guide):

- The objectives of planning in Victoria.
- Purposes of the planning scheme.
- A User Guide.
- The State Planning Policy Framework.
- The Local Planning Policy Framework.
- Zone and overlay requirements.
- Particular provisions.
- General provisions.
- Definitions.
- Incorporated documents.

The State Planning Policy Framework (SPPF) covers strategic issues of State importance. It lists policies under six headings: settlement, environment, housing, economic development, infrastructure, and particular uses and development. Every planning scheme in Victoria contains this policy framework, which is identical in all schemes.

The Local Planning Policy Framework (LPPF) contains a municipal strategic statement and local planning policies. The framework identifies long term directions for land use and development in the Hamilton region; presents a vision for its community and other stakeholders; and provides the rationale for the zone and overlay requirements and particular provisions in the scheme.

## 6.3 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.

### *6.3.1 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 risk 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.

In Urban Floodway Zone areas, the following uses are allowed without a permit:

- Apiculture - Must meet the requirements of the Apiary Code of Practice, May 1997.
- Extensive animal husbandry
- Informal outdoor recreation
- Mineral exploration
- Mining
- Natural systems
- Search for stone - Must not be costeaning or bulk sampling.
- Telecommunications facility.

The following uses are allowed with a planning permit:

- Agriculture (other than Apiculture and Extensive animal husbandry)
- Leisure and recreation (other than Informal outdoor recreation, Indoor recreation facility, and Motor racing track)
- Mineral, stone or soil extraction (other than Mineral exploration, Mining, and Search for stone)
- Road
- Utility installation (other than Telecommunications facility).

The following land uses are prohibited in an Urban Floodway Zone:

- Indoor recreation facility
- Motor racing track
- Any other use not listed above.

All planning permits and subdivisional applications are also subject to the same controls as required for an application on land covered by the Floodway Overlay described below.

### *6.3.2 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 floodpaths, 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).

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 floodway overlay, with some limited exemptions for public infrastructure works.

Subdivision of land covered by a FO/RFO can only be accomplished with a planning permit and under the following conditions:

- The subdivision does not create any new lots, which are entirely within this overlay. This does not apply if the subdivision creates a lot, which by agreement between the owner and the relevant floodplain management authority, is to be transferred to an authority for a public purpose.
- The subdivision is the re-subdivision of existing lots and the number of lots is not increased, unless a local floodplain development plan incorporated into this scheme specifically provides otherwise.

All planning applications where a local floodplain development plan has not been incorporated into the scheme require a flood risk study to be undertaken with regard to the following points:

- The State Planning Policy Framework and the Local Planning Policy Framework.
- The existing use and development of the land.
- Whether the proposed use or development could be located on flood-free land or land with a lesser flood hazard outside this overlay.
- The susceptibility of the development to flooding and flood damage.
- The potential flood risk to life, health and safety associated with the development.
- Flood risk factors to consider include:
  - The frequency, duration, extent, depth and velocity of flooding of the site and accessway
  - The flood warning time available
  - The danger to the occupants of the development, other floodplain residents and emergency personnel if the site or accessway is flooded.
  - The effect of the development on redirecting or obstructing floodwater, stormwater or drainage water and the effect of the development on reducing flood storage and increasing flood levels and flow velocities.
  - The effects of the development on environmental values such as natural habitat, stream stability, erosion, water quality and sites of scientific significance.

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, has flood depths that are moderate or high and flood frequently should generally be regarded as floodway. For the Hamilton study the frequency method used the 10% AEP flood extent.

The flood hazard is defined by combining the flood depth and flow speed to form a hazard category for a given design event. The advisory notes suggest using Figure 6.4 for delineating the floodway based on flood hazard.

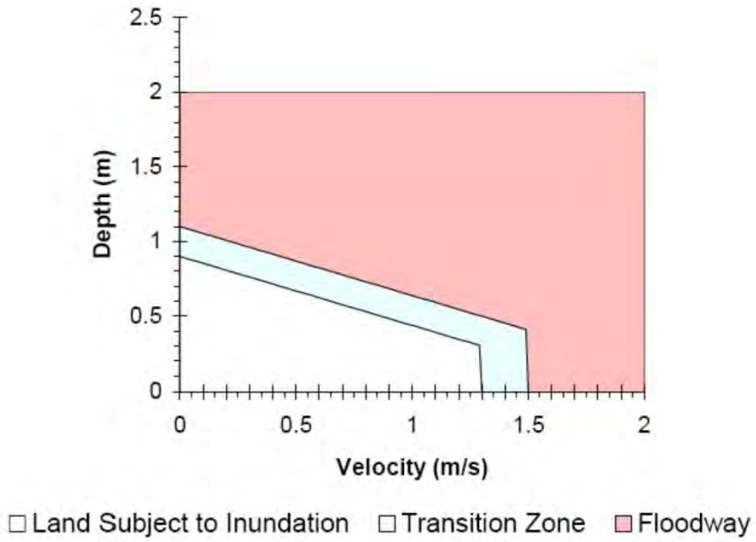



Figure 6.4 Floodway overlay flood hazard criteria (NRE, 1998)

An alternate definition of flood hazard (or safety risk) is provided by Melbourne Water based on both the velocity-depth product and the total flood depth. Melbourne Water defines 5 classes of safety risk as shown in Table 6.1. The Melbourne Water hazard approach was considered for this study. The flood overlay selection criteria was based on a hazard greater than 2.

Table 6.1 Melbourne Water Safety Risk Definition

| Safety Risk Category | | Definition | | |
|---|---|---|---|---|
| | | V*D | | Depth |
| High | 5 | > 0.84 m²/s | or | > 0.84 m |
| Moderate to High | 4 | 0.6 - 0.84 m²/s | | 0.6 - 0.84 m |
| Moderate | 3 | 0.4 - 0.6 m²/s | | 0.4 - 0.6 m |
| Low to Moderate | 2 | 0.2 - 0.4 m²/s | | 0.2 - 0.4 m |
| Low | 1 | < 0.2 m²/s | | < 0.2 m |

The final method for defining the flood overlay was the flood depth method. The flood overlay was set using this method where the flood depths were greater than 0.5 m within the 1% AEP flood event.

For this investigation all three methods were developed and assessed in order to generate the appropriate Flood Overlay. From the assessment the flood frequency method was determined to be the most appropriate as the flood hazard and depth methods generated inconsistent and incomplete Flood Overlays. The developed flood overlay is shown for Models A, B and C in Figure 6.5, Figure 6.6 and Figure 6.7 respectively.

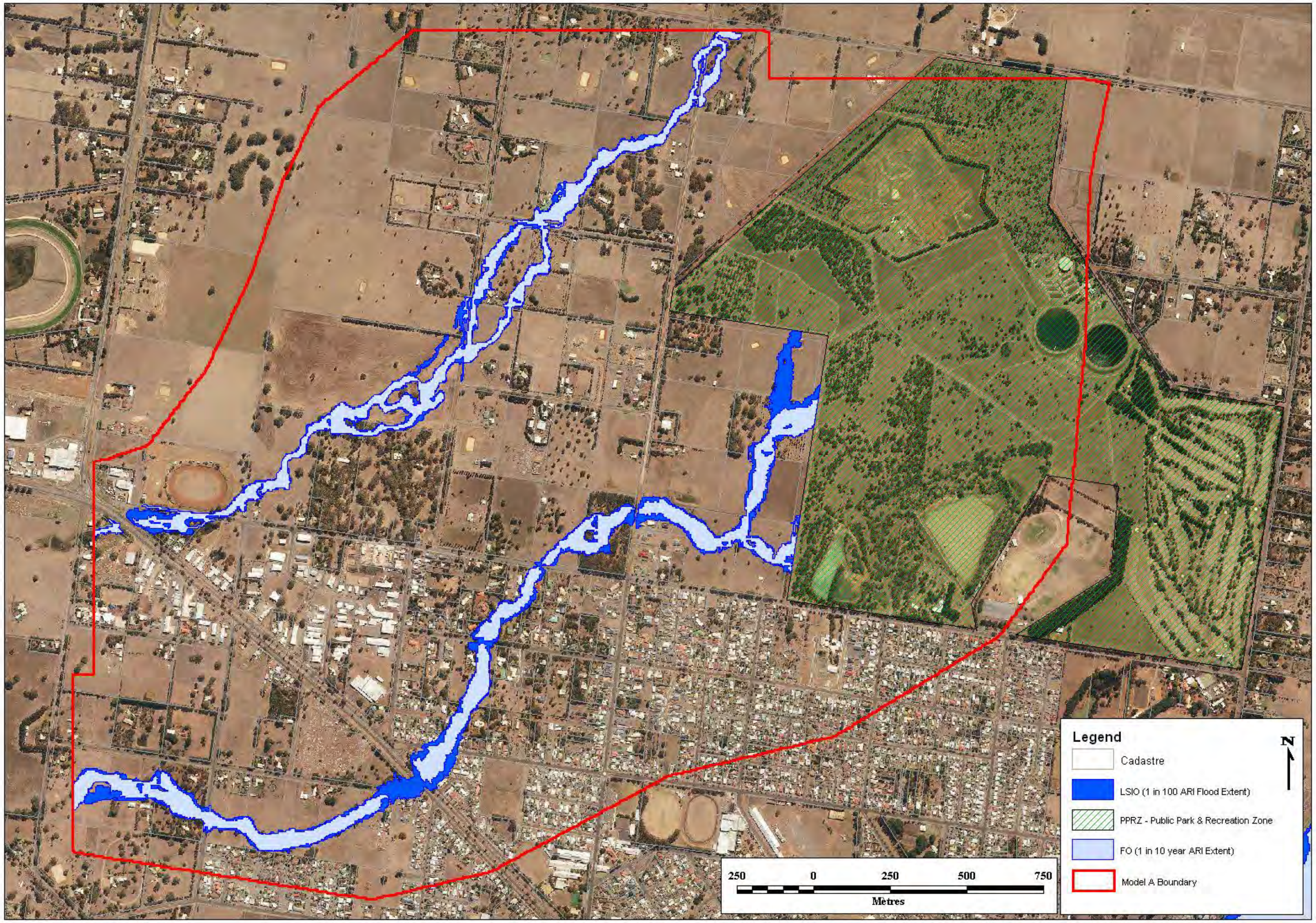



Figure 6.5 Model A – FO and LSIO definition using the 10% AEP flood extent and 1% AEP flood extent

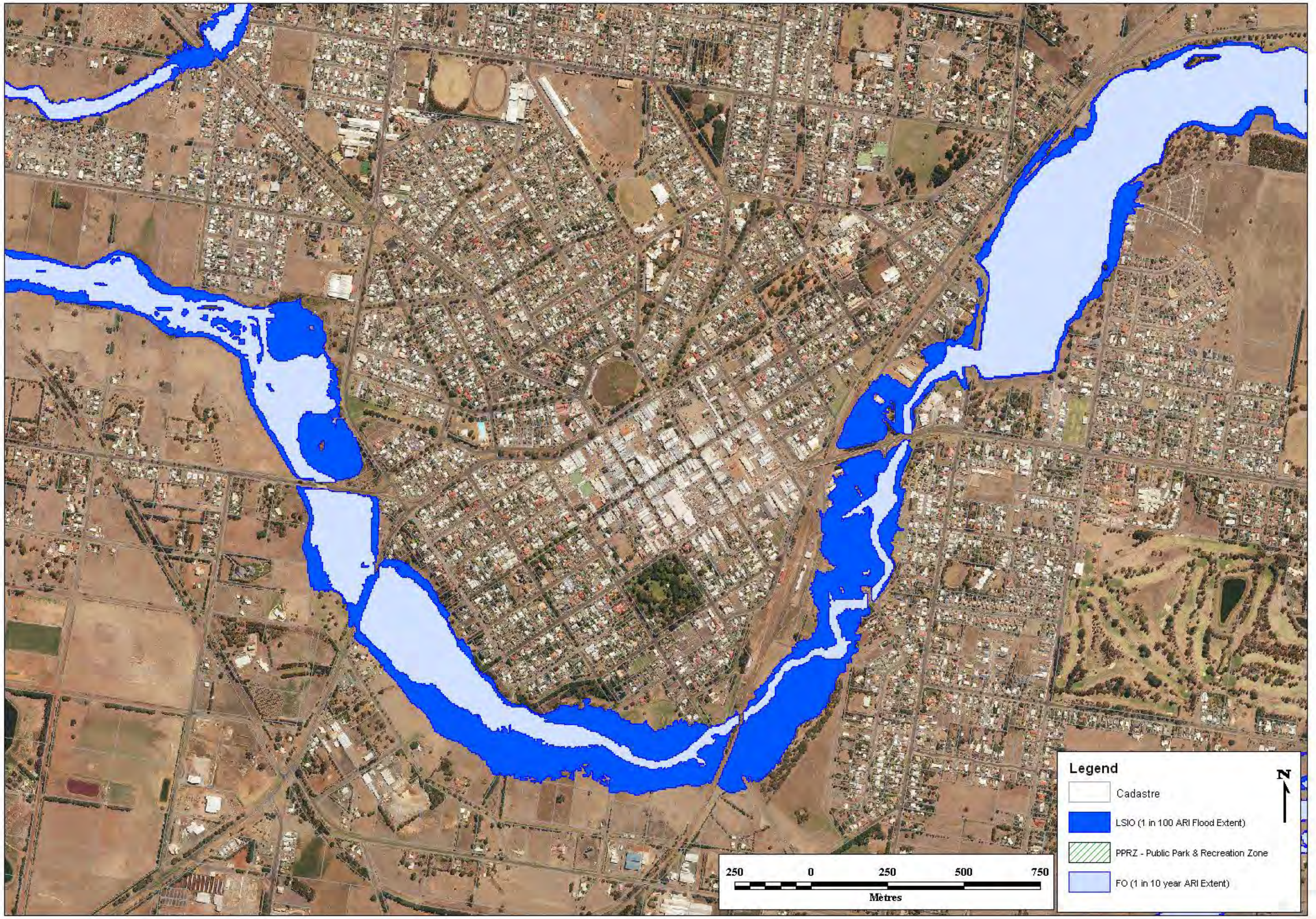



Figure 6.6 Model B – FO and LSIO definition using the 10% AEP flood extent and 1% AEP flood extent

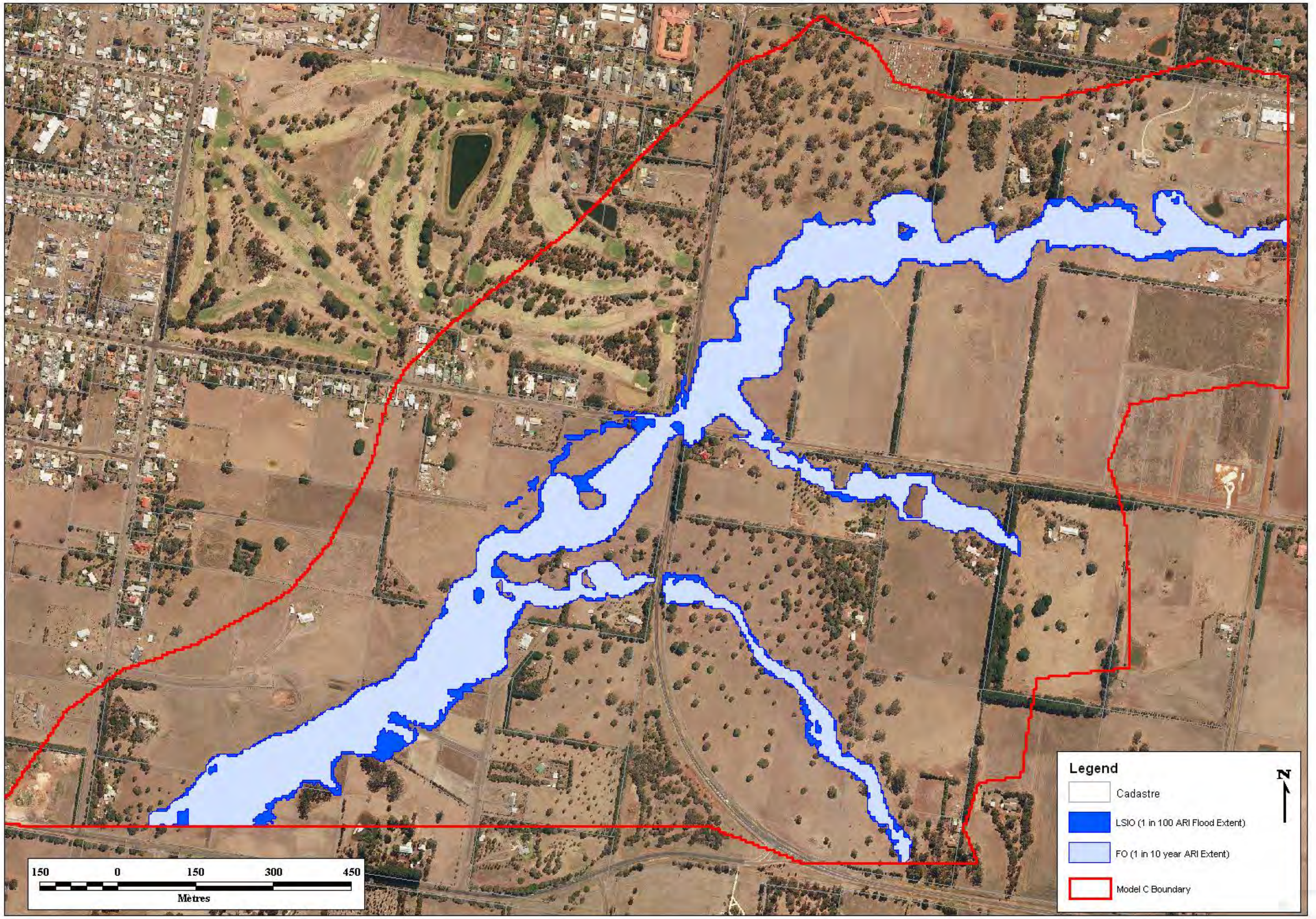



Figure 6.7 Model C – FO and LSIO definition using the 10% AEP flood extent and 1% AEP flood extent

### *6.3.3 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 for Hamilton.

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 reflect any declaration under Division 4 of Part 10 of the Water Act, 1989 where a declaration has been made.
- 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.

Applications for planning permits in areas covered by the LSIO have the following decision guidelines with respect to flooding:

- The State Planning Policy Framework and the Local Planning Policy Framework.
- Any local floodplain development plan.
- Any comments from the relevant floodplain management authority
- The existing use and development of the land.
- Whether the proposed use or development could be located on flood-free land or land with a lesser flood hazard outside this overlay.
- The susceptibility of the development to flooding and flood damage.
- The potential flood risk to life, health and safety associated with the development.
- Flood risk factors to consider include:
  - The frequency, duration, extent, depth and velocity of flooding of the site and accessway
  - The flood warning time available
  - The danger to the occupants of the development, other floodplain residents and emergency personnel if the site or accessway is flooded.
  - The effect of the development on redirecting or obstructing floodwater, stormwater or drainage water and the effect of the development on reducing flood storage and increasing flood levels and flow velocities.
  - The effects of the development on environmental values such as natural habitat, stream stability, erosion, water quality and sites of scientific significance.

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 Hamilton region it was considered appropriate to use the 1% AEP event as the extent for the LSIO.

#### *6.3.4 Special Building Overlay (SBO)*

The SBO applies to areas that are subject to stormwater flooding in urban areas. That is to say areas which are inundated due to the inability of the stormwater infrastructure to convey the flood flows. This overlay is considered as many stormwater systems were implemented prior to current design standards and there has been substantial development since the infrastructure was completed.

Stormwater systems and modelling was not included in this project and as such Cardno does not recommend any areas to be covered under a Special Building Overlay. Detailed modelling of the stormwater system of the Hamilton area would be required to accurately develop the SBO.

#### *6.3.5 Recommended Planning Controls*

As the Hamilton region is already largely well developed, we do not believe that there is a need to implement an Urban Floodway Zone in the catchment. Similarly, a SBO is unlikely to be required as the predominant flooding is from main channel flows rather than from stormwater flooding. Stormwater flooding was not specifically assessed as part of this project.

The recommended flood controls to be put in place are a FO and LSIO. The method of deriving the FO included using the 10% AEP extent, the hazard class exceeding 2 for the 1% AEP event and where the depths were greater than 0.5 m during the 1% AEP event. The three possible extents for the FO varied with each method protecting different areas. The main difference was that the FO included consistent coverage within the north west and south east tributaries under the 10% AEP flood extents as compared to the other methods of deriving the FO.

The results of the three methodologies were provided to the Glenelg Hopkins CMA and Council to determine a final Floodway Overlay shape. The LSIO would include all areas inside the 1% AEP flood extent that are not covered by the final FO shape. It is recommended that the area within Model C covered by the PPRZ (Public Park and Recreation Zone) be excluded from the FO and included in the LSIO as this area already has planning restrictions and is not intended for development. This section of the model is also impacted by the man-made channel to the old Reservoir which has not been accurately surveyed and included within the model in detail.

## 6.4 Planning Amendment Documentation

As part of this flood investigation Cardno have developed the planning amendment documentation including the following Planning Scheme Ordinances and Maps;

- A Schedule to Clause 44.03 Floodway Overlay if the GHCMA considers that any exemption should be provided from the permit requirements of Clause 44.03.
- A revised Schedule to Clause 44.04 Land Subject to Inundation Overlay if the GHCMA considers that any exemptions should be added to this Schedule. We consider that the current exemption given to the construction of fences should be removed from this Schedule.
- The relevant Planning Scheme Maps to implement the FO and LSIO mapping.

Cardno have developed the appropriate documentation to facilitate the implementation of the planning amendment.

# 7 ECONOMIC DAMAGES

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.

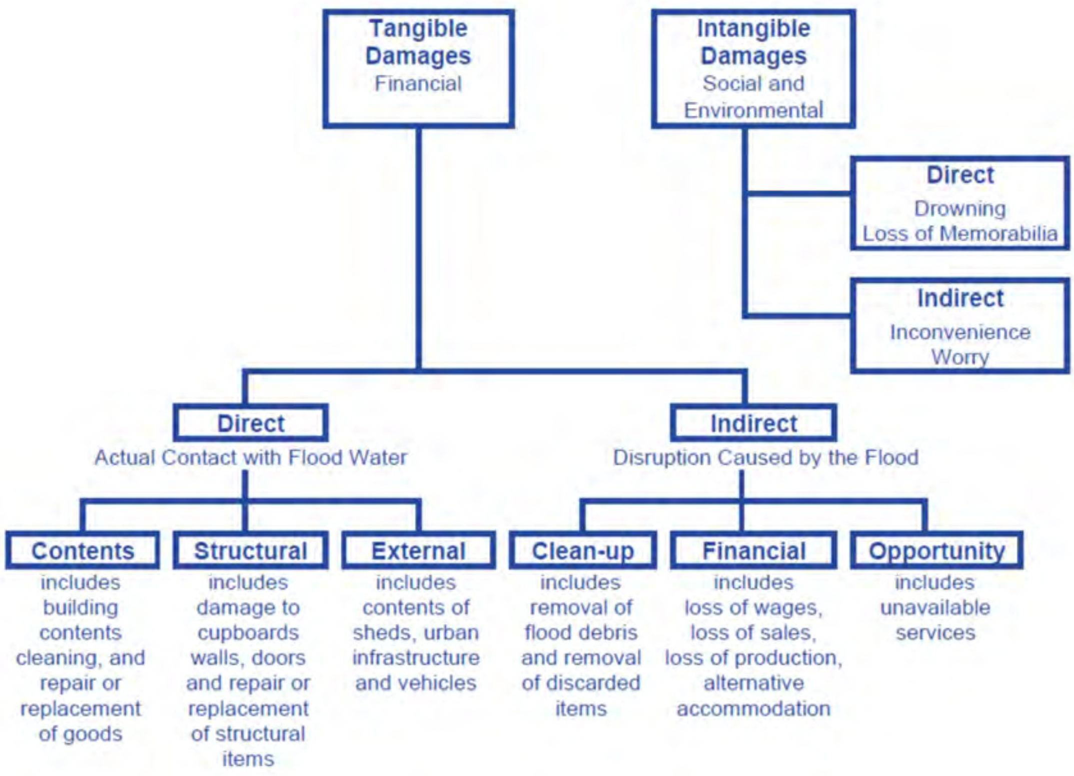



Figure 7.1 Types of flood damage (Floodplain Development Manual (NSW Gov, 2005))

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 Hamilton 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.

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 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 of the Grange Burn and associated tributaries.

### *7.1.1 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 assumed that all residential properties 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, a nominal property damage value was applied (see Section 7.1.5 for details).
- The flooding overtops the garden and also reaches the base of the house. The DECCW curves allow for a damage of $10,050 (Mar 2012 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). This accounts for the garden damage as specified in the point above, but also includes some damage to cars and structures.

Residential damages 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). This equates to

$10,050 (Mar 2012 dollars) for flooding depths between 0.3 m below the floor height, when the flood water 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. It should be noted that the damages for the residential curve are shown on a $ per $m^2$ basis, whereas the commercial and industrial damage curves are quoted on a $ per 100 $m^2$ basis.

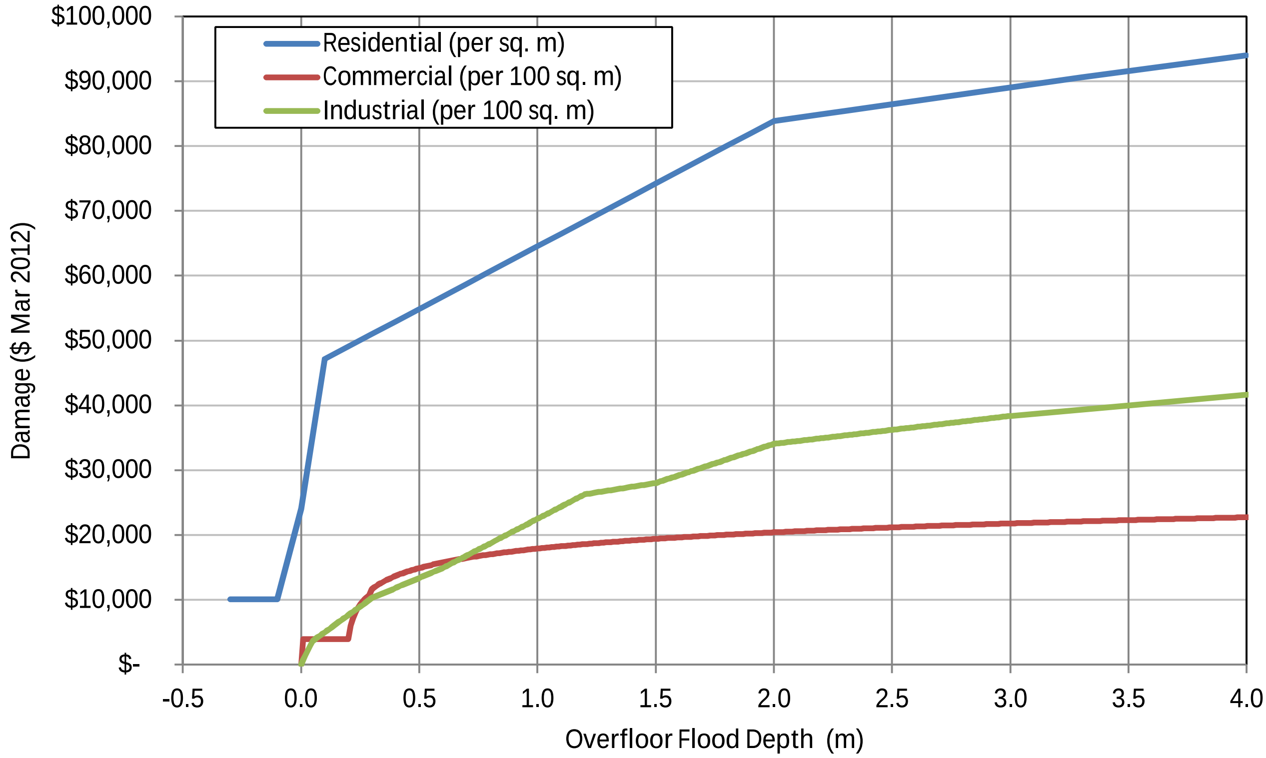



Figure 7.2 Damage curves applied to the Hamilton Flood Plain

### *7.1.1.1 Average Weekly Earnings*

The DECCW curves are derived for late 2001 and have been adjusted to represent March 2012 dollars.

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 March 2012. Therefore all ordinates in the residential flood damage curves were updated to the March 2012 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 March 2012 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 |
| March | 2012 | $ 1,345.20 |
| Change | 49.7 % | |

Consequently, all ordinates on the damage curves were increased by 49.7 %. It has been assumed that March 2012 values are representative of current dollars.

#### *7.1.1.2 Other Parameters*

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 Hamilton was estimated based on the delineated residential buildings within the Probable Maximum Flood extent. The average was approximately 200 $m^2$. This area reflects the ground floor only.

Within the Probable Maximum Flood extent there were 201 buildings which did not have floor levels captured (only buildings within the 1% AEP flood extent were surveyed). In order to estimate the floor levels of these properties, the average floor level to building topography level was examined. The average floor height above the topography elevation was found to be 280 mm. A relationship was determined between the surveyed floor levels and topography level for each building, this relationship was then used to estimate the floor level for the non-surveyed buildings. The details of this relationship are shown in Figure 7.3.

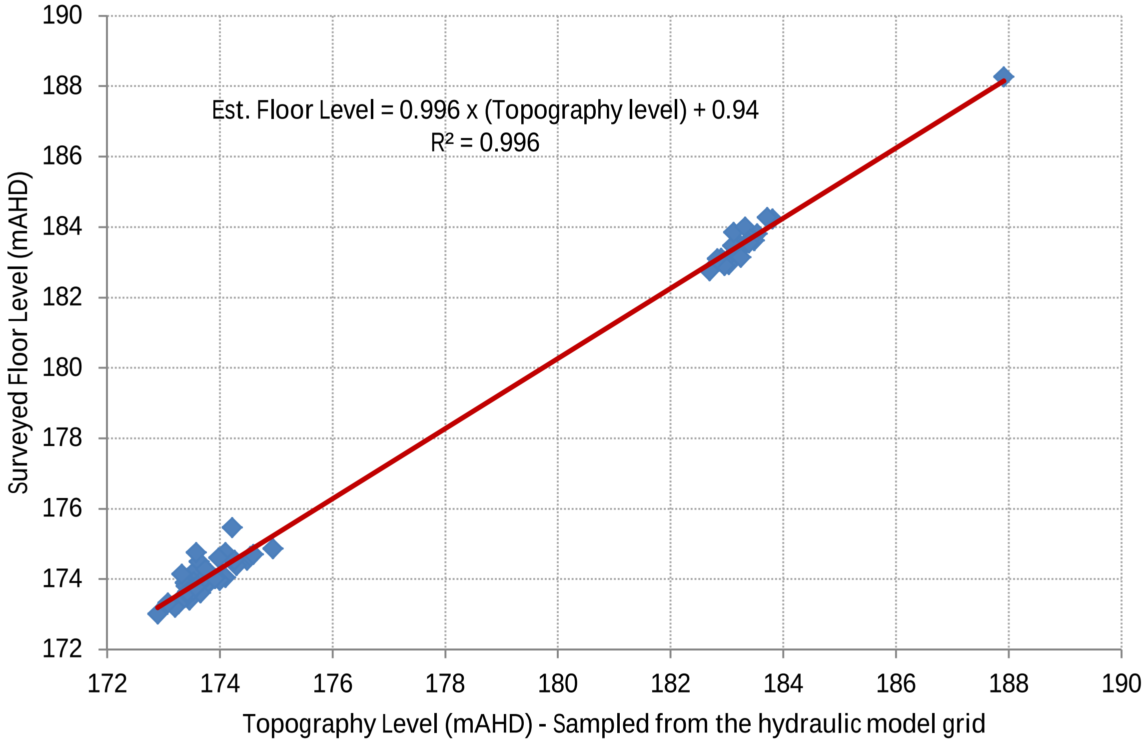



Figure 7.3 Infilling relationship for the buildings with no surveyed floor level (outside the 1% AEP flood extent)

Conservatively, the Effective Warning Time has been assumed to be zero as Hamilton has no currently working flow gauge. 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 Hamilton catchment, while rural, has access to Warrnambool, Geelong, Ballarat and Melbourne via multiple highways 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.

### *7.1.2 Commercial Damage Curves*

Commercial damage curves are determined based on those included in the *FLDamage Manual* (Water Studies, 1992). FLDamage allows for three types of commercial properties;

- Low Value Commercial,
- Medium Value Commercial,
- High Value Commercial.

In Hamilton in the Grange Burn floodplain all commercial has been assumed to be low value commercial based on FLDamage. In determining these damage curves, it has been assumed that the effective warning time is approximately zero, and the loss of trading days has been approximated at 10.

The commercial damage curve is linked to the floor area of the property and aerial imagery was used to estimate the floor area of the individual properties. These areas were used to factor these curves, the curves have been determined for a standardised 100 $m^2$. The damage curves can be seen in Figure 7.2.

The CPI was used to bring the 1990 data to March 2012 (CPI was obtained from the ABS www.abs.com.au). The CPI adjustment factor is shown in Table 7.2.

Table 7.2 Commercial damage curve adjustment factor

| Month | Year | CPI |
|---|---|---|
| June | 1990 | 102.5 |
| March | 2012 | 176.8 |
| Change | 72.5 % | |

Consequently, damages have been increased by 72.5% and GST has been included.

### *7.1.3 Industrial Damage Curves*

Industrial building damages were assessed using damage curves based on the *FLDamage Manual* (Water Studies, 1992). FLDamage allows for three types of industrial properties;

- Low Value Industrial,
- Medium Value Industrial,
- High Value Industrial.

The industrial properties within the Grange Burn floodplain have been assessed as low value industrial. This is with respect to the fact that a high value industrial site for example would be a BHP mining operation.

The CPI was used to bring the 1998 data to March 2012 (CPI was obtained from the ABS www.abs.com.au) . The CPI adjustment factor is shown in Table 7.2. The damage curves can be seen in Figure 7.2.

Table 7.3 Commercial damage curve adjustment factor

| Month | Year | CPI |
|---|---|---|
| June | 1998 | 121.0 |
| March | 2012 | 176.8 |
| Change | 46.1 % | |

Consequently, damages have been increased by 46.1% and GST has been included.

### *7.1.4 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 March 2012 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 |
|---|---|---|
| May | 2000 | 126.2 |
| March | 2010 | 176.8 |
| Change | 40.1 % | |

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)

| | *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 (Mar 2012 $) |
|---|---|---|---|---|---|
| Major sealed roads | *$ 32,000* | *$ 16,000* | *$ 11,000* | $ 59,000 | $82,656 |
| Minor sealed roads | *$ 10,000* | *$ 5,000* | *$ 3,500* | $ 18,500 | $25,918 |
| Unsealed roads | *$ 4,500* | *$ 2,250* | *$ 1,600* | $ 8,350 | $11,698 |

### 7.1.5 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 are summarised in Table 7.6 for each of the land use types.

Table 7.6 Assumed property damages (land use supplied from land.vic.gov.au)

| Land Use Zone | Description | Assumed Damage (if property has inundation >1% of area and at least 10cm of depth) |
|---|---|---|
| B1Z | Business 1 | $1,000 |
| FZ | Farming | $500 |
| FZ & R1Z | farming and Residential 1 | $800 |
| IN1Z | Industrial 1 | $1,000 |
| LDRZ | Low Density Residential | $500 |
| PCRZ | Public Conservation and Resource | $1,000 |
| PPRZ | Public park and Recreation | $1,000 |
| PUZ4 | Public Use, Transport | $1,000 |
| PUZ7 | Public Use, Other | $1,000 |
| R1Z | Residential 1 | $1,000 |
| SUZ1 | Special Use | $1,000 |
| SUZ3 | Special Use | $1,000 |

## 7.2 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 7.2. 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.

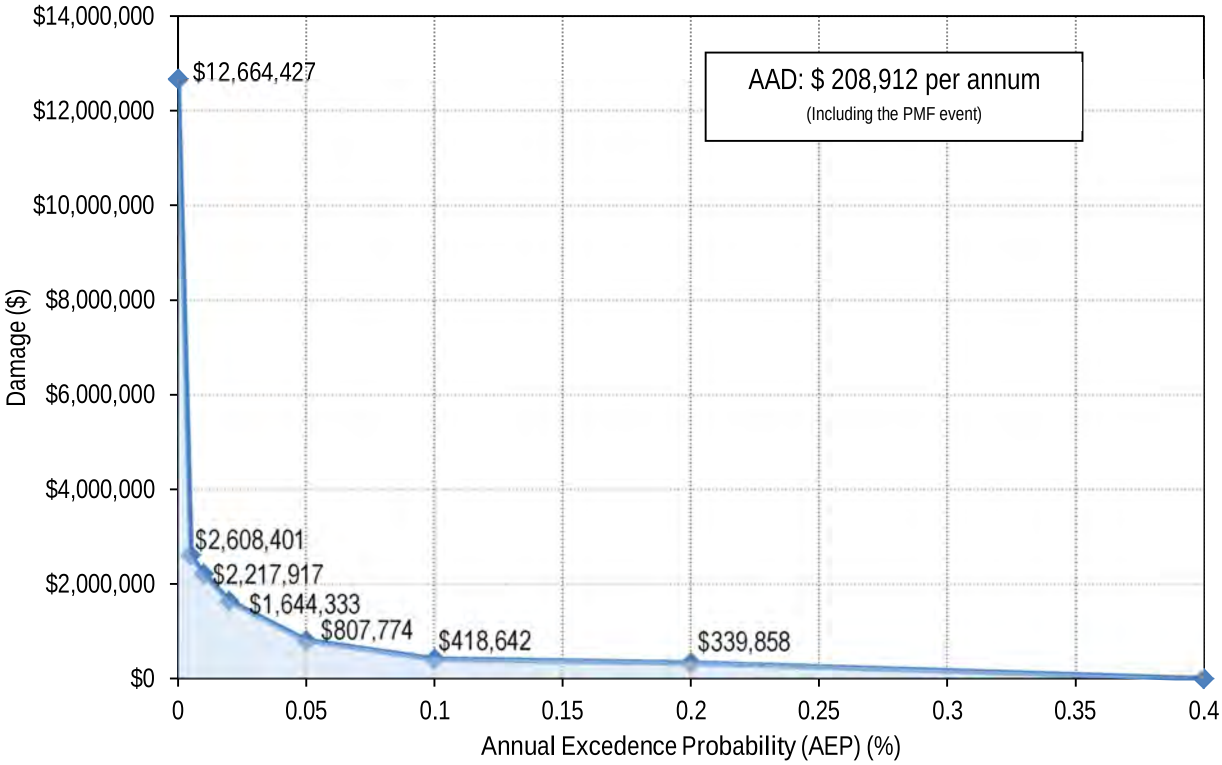



Figure 7.4 Flood damages used to estimate the Average Annual Damages

Further information on the calculation of the AAD can be found in the Floodplain Development Manual (NSW Government, 2005).

## 7.3 Results

The results of the flood damage assessment are shown in Table 7.7. Based on the analysis as described in the above section the annual average damages (AAD) for the floodplain under existing conditions is approximately $ 208,912 per annum.

Table 7.7 Summary of Economic Flood Damages

| Property Damage | | | | | | | |
|---|---|---|---|---|---|---|---|
| Recurrence Interval | 20% AEP | 10% AEP | 5% AEP | 2% AEP | 1% AEP | 0.5% AEP | PMF |
| Total property damages | $122,500 | $140,800 | $234,850 | $227,500 | $279,400 | $302,400 | $583,416 |
| Inundated properties (> 10cm depth, > 1% area) | 179 | 204 | 270 | 315 | 358 | 389 | 649 |
| **Building Damage** | | | | | | | |
| Recurrence Interval | 20% AEP | 10% AEP | 5% AEP | 2% AEP | 1% AEP | 0.5% AEP | PMF |
| Community Buildings | 0 | 0 | 0 | 1 | 1 | 1 | 2 |
| Caravan Park | 0 | 0 | 0 | 0 | 0 | 0 | 23 |
| Residential | 4 | 6 | 10 | 20 | 23 | 26 | 137 |
| Commercial | 0 | 0 | 0 | 5 | 6 | 6 | 13 |
| Industrial | 0 | 0 | 5 | 12 | 19 | 21 | 41 |
| Total buildings with overfloor flooding | 4 | 6 | 15 | 38 | 49 | 54 | 216 |
| Recurrence Interval | 20% AEP | 10% AEP | 5% AEP | 2% AEP | 1% AEP | 0.5% AEP | PMF |
| Community Buildings | $ - | $ - | $ 5,421 | $ 15,365 | $ 16,896 | $ 17,836 | $ 43,148 |
| Caravan Park | $ - | $ - | $ - | $ - | $ - | $ - | $ 221,235 |
| Residential | $ 203,488 | $ 251,042 | $ 503,546 | $ 1,062,783 | $ 1,262,404 | $ 1,472,160 | $ 8,739,764 |
| Commercial | $ - | $ - | $ - | $ 177,873 | $ 358,852 | $ 406,113 | $ 1,429,342 |
| Industrial | $ - | $ - | $ 18,939 | $ 80,340 | $ 195,750 | $ 272,889 | $ 1,221,464 |
| Total overfloor damages | $ 203,488 | $ 251,042 | $ 527,906 | $ 1,336,361 | $ 1,833,903 | $ 2,168,999 | $ 11,654,953 |
| **Road Damage** | | | | | | | |
| Recurrence Interval | 20% AEP | 10% AEP | 5% AEP | 2% AEP | 1% AEP | 0.5% AEP | PMF |
| Major | $ 1,680 | $ 4,740 | $ 7,157 | $ 21,736 | $ 26,813 | $ 36,095 | $ 178,313 |
| Minor | $ 12,190 | $ 22,060 | $ 37,861 | $ 58,736 | $ 77,801 | $ 100,907 | $ 247,744 |
| Total road damages | $ 13,870 | $ 26,800 | $ 45,018 | $ 80,472 | $ 104,614 | $ 137,002 | $ 426,057 |
| Total | $ 339,858 | $ 418,642 | $ 807,774 | $ 1,644,333 | $ 2,217,917 | $ 2,608,401 | $ 12,664,427 |

## 7.4 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 40% 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.

# 8 ASSESS AND TREAT RISK

The final section of this report uses the information gathered and developed as part of this flood investigation to critically assess the current flood warning and response system, as well as to propose mitigation options that can be assessed. The discussion will first present the critical areas that have been identified as part of the detailed flood modelling and subsequently provide some viable structural and non-structural options that could be explored to plan for and potentially mitigate flooding in these high risk areas.

## 8.1 Identification of High Risk Areas

Within the Hamilton region there are two key areas that have been highlighted as having a high flood risk and are expected to have properties and buildings inundated during large flood events.

The first of these areas is adjacent to the reserve at the corner of Coleraine Road and King Street. In large flood events the flows down the Marshalls Road Tributary exceed the capacity of the twin 1200 mm diameter pipes which have their inlet upstream of King Street and exit downstream of Lewis Street. When the capacity of the culverts are exceed, floodwaters pool within the reserve and threaten adjacent properties. The depths within the park are greater than 1 m during the 1% AEP flood event. There are 16 houses and units that are predicted to have overfloor flooding during the 1% AEP flood event in this area. These properties and the peak flood depths are shown in Figure 8.1.

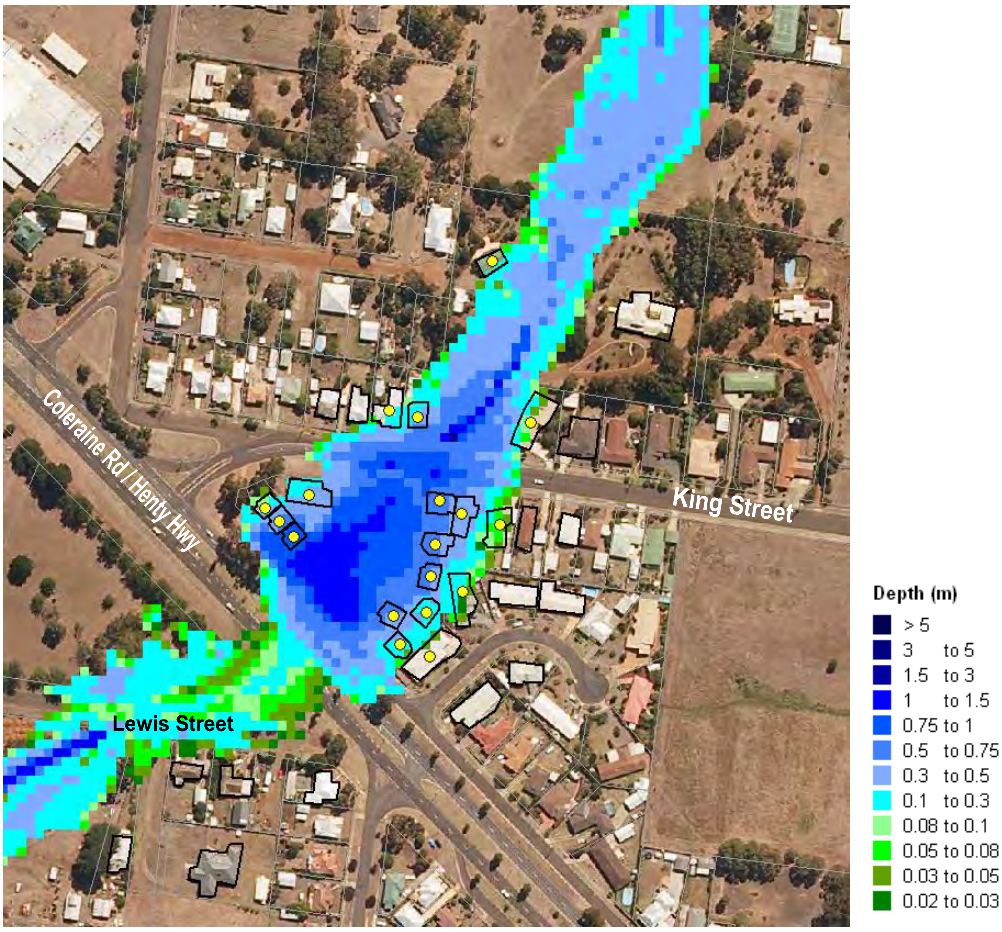



Figure 8.1 Predicted overfloor flooding for the 1% AEP flood event for Marshalls Road Tributary

The second area that has significant flooding concerns is the area adjacent to Apex Park on the Grange Burn. This area contains a large amount of industrial business however also contains a number of residential properties. During the 1% AEP design event there are 42 buildings expected to have overfloor flooding these buildings have been shown along with the peak 1% AEP flood depths for this area in Figure 8.2.

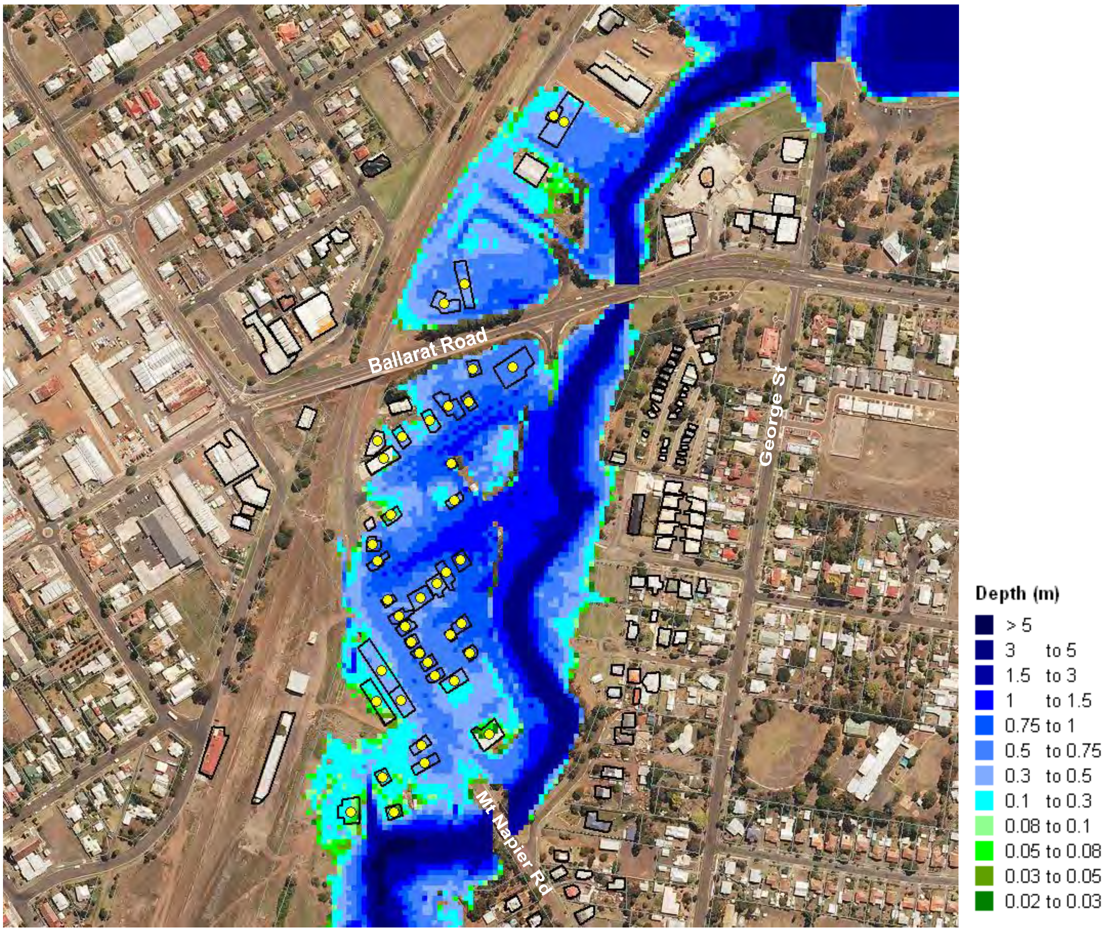



Figure 8.2 Predicted overfloor flooding for the 1% AEP flood event for the Grange Burn

These areas are the primary areas in the Hamilton Region which pose a current flood risk to properties and infrastructure. It is for these reasons that these locations are the primary focus of the mitigation options proposed as part of this assessment.

## 8.2 Non Structural Risk Management Options

Before considering the structural mitigation options it is important to outline and discuss the possible non-structural management options that are available to the GHCMA, Council and flood response agencies. These options are more economical to develop, however the benefits of the non-structural management methods are difficult to measure as there are no tangible outcomes i.e. no properties permanently protected.

Non-structural management options do not impact the likelihood of a flood occurring but aim to reduce the consequences associated with a flood. These management options achieve this reduction in consequences through increased warning, forward planning and raised awareness of the community and flood response agencies.

Methods for managing the consequences associated with flooding can include:

- Develop a flood response plan for the Hamilton Region.
- Assist the community in developing individual flood plans – identifying the risk of flooding to the individual properties and assist in them developing a flood plan.
- Plan evacuation routes and locations.
- Providing adequate upstream gauging (Robsons Road or Tarrington / Strathkellar Road) to provide warning for residents and businesses.
- Develop a temporary gauge location with rating curve so that the location can be used to place a portable automated streamflow gauge if one should be available for an event.
- Assist the VicSES in holding information sessions and providing information via Flood Safe information packs.
- Provide ongoing information sessions to residents and businesses that may be effected. It is important that this information is distributed periodically to capture changes of residents and businesses.
- Provide appropriate information signage in areas where there may be risk in the future, especially where transient populations are involved i.e. caravan parks and camping grounds.

## 8.3 Structural Risk Management Options

In conjunction with the Glenelg Hopkins CMA and Council Cardno has identified six (6) mitigation options that are to be explored within the hydraulic model. These mitigation options aim to reduce the flood impacts at the locations identified in Section 8.1. The mitigation options included additional culverts, levees and road reconstructions.

### *8.3.1 Model A Options*

Two mitigation options were considered for the high risk area identified at King Street along the Marshalls Road Tributary. The two options included levees (mitigation option A2) along the property boundaries adjacent to the park to provide flood protection and installing additional culverts (mitigation option A1) under Coleraine Road / Henty Hwy to pass additional floodwaters. The mitigation locations are shown in Figure 8.3.


Figure 8.3 Model A – Marshalls Road Tributary mitigation options

Mitigation Option A2 set the levees at 183.4 mAHD which results in a maximum levee height of 1 m above the existing surface level. The purpose of the levees is to allow the floodwaters that exceed the capacity of the existing culverts under Coleraine Road to collect to a higher level within the park reserve without inundating nearby properties to above floor levels during large flood events.

Mitigation Option A1 was a set of box culverts 3 x 1.2 m x 0.9 m (no. x W x H). These culverts were set with an upstream invert level of 181.8 mAHD and a downstream invert level of 181.7 mAHD. Within the park an entry pit would be required to pass water to the culverts and this would sit below the current surface level. The invert of the culverts was set below the lowest floor level of the adjacent properties. The aim of these culverts was to pass the peak flows that exceed the existing culverts through to the vacant land on the south western side of Coleraine Road. Floodwaters would then pass over Lewis Street and back into the main channel.

### *8.3.2 Model B Options*

Four mitigation options were considered for the Grange Burn (Model B). These options are shown in Figure 8.4. The four options included:

- Option B1 - Levee upstream of Ballarat Road (west side of the Grange Burn).
- Option B2 - Upgrading Apex Park Road to act as a raised road levee bank.
- Option B3 - Extending a levee from the Apex Park Road upgrade to Mt Napier Road (west side of Grange Burn)
- Option B4 - Removing the existing pedestrian bridge


Figure 8.4 Model B – Grange Burn mitigation options

#### *8.3.2.1 Mitigation Option B1*

Mitigation Option B1 protects the industrial sites located on the western side of the Grange Burn. The levee is approximately 155 m in length and has a crest height of 175.6 mAHD at the northern end of the levee, sloping to 175.2 mAHD at the southern end of the levee. The long section for the levee is shown in Figure 8.5. The levee is approximately 1 – 1.3 m above the existing surface. Freeboard of 300 mm has been included in the proposed levee height over the 1% AEP event and the levee can withstand the predicted 0.5 % AEP event at current levels (without freeboard).

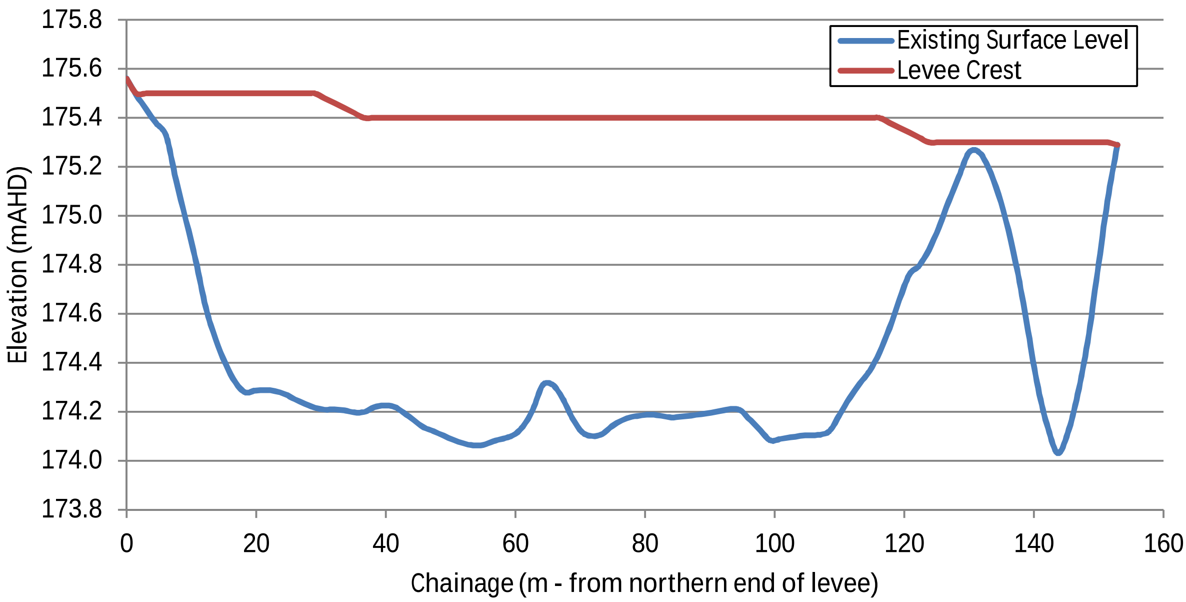



Figure 8.5 Mitigation Option B1 levee long section

#### *8.3.2.2 Mitigation Option B2*

Mitigation Option B2 was proposed by the Council as a possible solution to the flooding of properties and buildings to the west of Apex Park. These areas are currently protected by a levee system that runs from Ballarat Road to south of Holden Street. The current levee system has a number of deficiencies that this mitigation options aims to address. These deficiencies include:

- The levee is partly on private land, Council would prefer to have the asset on public / council land
- The levee breaks at road crossings (Holden Street and Apex Drive) which means without adequate flood warning the levees will remain open. Manual filling of these gaps is currently required during times of flood.
- The levees are no longer sufficient to protect against the 1% AEP flood event (the 1% flood event has been revised as part of this flood investigation).
- The levees are bypassed by floodwaters at the southern end which can result in floodwaters entering the Holden Street and Abbott Street areas from the south.

The proposed method of upgrading the current system was proposed by Council and involved redeveloping Apex Drive. This redevelopment would involve raising the road to approximately 175 mAHD. At the lowest section of road (near junction with Holden Street) this involves raising the road by approximately 1.9 m. The long section along Apex Drive is shown in Figure 8.6.

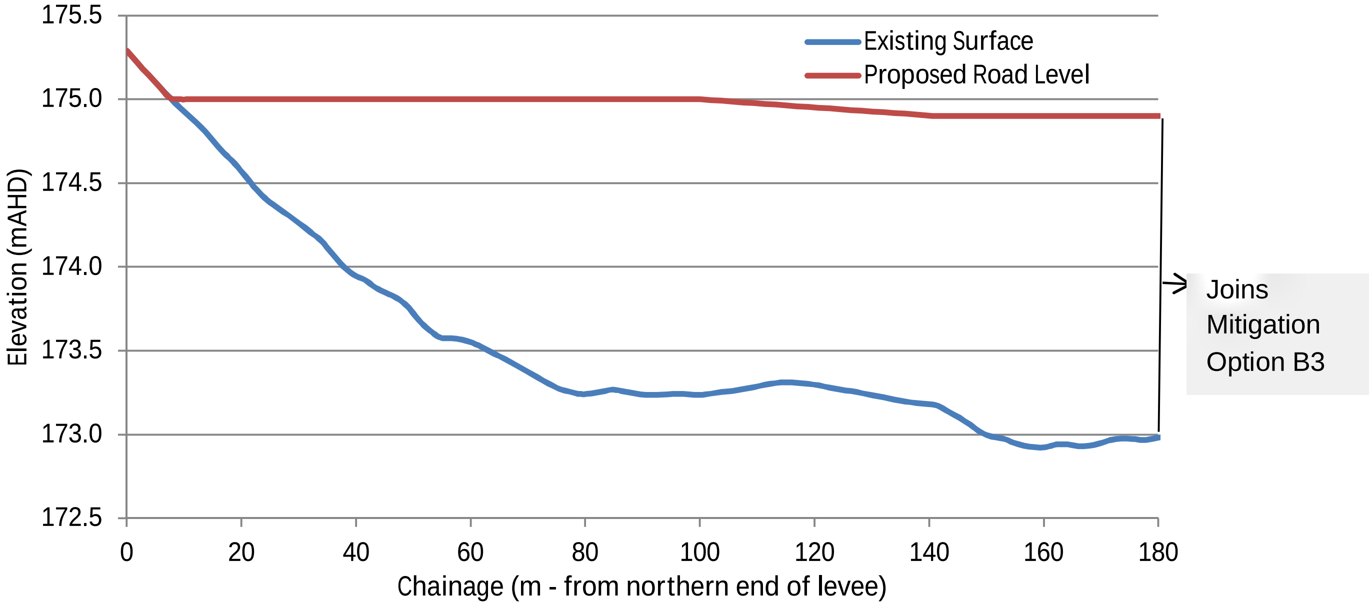



Figure 8.6 Mitigation Option B2 road long section

At the southern end of the Apex Drive redevelopment the road surface would tie into the proposed levee as part of Mitigation Option B3. For this preliminary mitigation assessment the grading of the developed Apex drive into Holden Street has not been considered, however it is anticipated that this would be possible within the space available. The long section shown in Figure 8.6 has freeboard of 300 mm over the 1% AEP included and is also sufficient to protect against the 0.5 % AEP flood event (without freeboard).

#### *8.3.2.3 Mitigation Option B3*

Mitigation Option B3 was a levee extending from the southern end of the Apex Drive road redevelopment down to Mt Napier Road. This levee was proposed to increase the current levee system and to stop the flows bypassing the existing levees to the south. This levee was proposed at 280 m in length and had a crest elevation of 174.9 mAHD at the northern end reducing to 173.7 mAHD at the southern end where it meets Mt Napier Road. The long section is shown in Figure 8.7.

The proposed levee follows the same path as the existing levee and some sections can be upgraded as part of the development of this option. The levee has been designed with a freeboard of 300 mm over the 1% AEP flood event. The levee is of sufficient height to protect against the 0.5% AEP event but without freeboard.

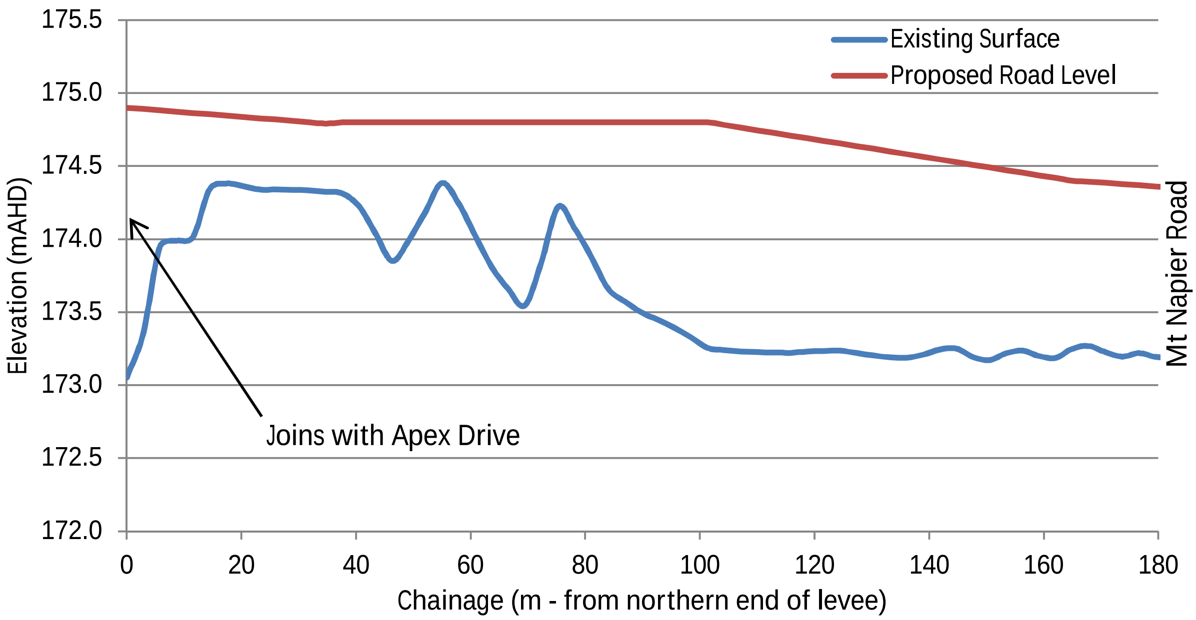



Figure 8.7 Mitigation Option B3 long section

### 8.3.2.4 *Mitigation Option B4*

Mitigation Option B4 involves removing the small pedestrian bridge downstream of the Grange Burn weir (old swimming pool). It was thought that this would reduce losses over this structure and reduce flood depths upstream. This option involves simply removing this structure. It is anticipated that a replacement walkway would be constructed but this has not been proposed at this stage. It is anticipated that this structure would be above the 1% AEP peak flood level.

### 8.3.3 *Model C Mitigation Options*

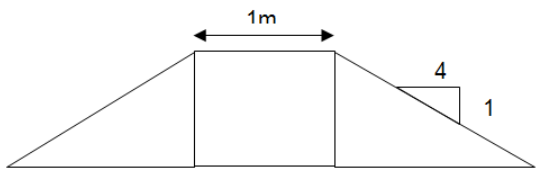
No mitigation options were considered for Model area C as this area is largely non-residential at this stage and no buildings were damaged during extreme flood events.

### 8.3.4 *Option Costing*

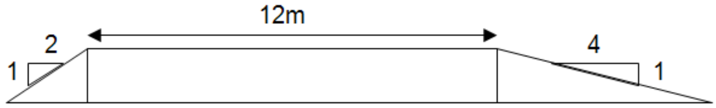
Each of the proposed mitigation options has been costed based on current prices. The costs estimates should be considered as estimates only and not be considered definitive costs for each of the options as the costing has been based on preliminary design only. The primary purpose of the option costing is to determine a relative cost benefit between each of the options.

This section will outline the assumptions and qualifications that are involved in the estimate of the costs for each option.

- All levees are assumed to have a 1m top width at the crest and have a 4:1 (h:v) batter slope. The only location where this is not the case was within the King Street park where the levee was constructed against the fence so only one batter was included.



- Apex Drive was assumed to require a road replacement of width 12m with the western batter slope being at 2:1 (v:h) and the eastern batter slope (Grange Burn side) at 4:1 (h:v). Additional width may be required if a footpath is required and to allow for drainage infrastructure.




- The removal of the pedestrian bridge assumes the structure is not replaced. No costing has been undertaken for a replacement structure.

The summary of the costs to develop each of the mitigation options is shown in Table 8.1. Details of the costing for each option are shown in Appendix E.

Table 8.1 Summary of the preliminary costing for the proposed mitigation options

| Option | Mitigation | Estimated Cost |
|---|---|---|
| A1 | Levee west side of King Street park | $ 30,217 |
| | Levee east side of King Street park | $ 20,095 |
| A2 | Culverts under Coleraine Road | $ 877,257 |
| | Total | $ 927,569 |
| | | |
| B1 | Levee upstream of Ballarat Road | $ 100,453 |
| B2 | Raising Apex Drive | $ 835,169 |
| B3 | Levee downstream of Apex Drive | $ 196,681 |
| B3 | Removing pedestrian bridge | $ 20,000 |
| | Total | $ 1,152,304 |

It should be noted that the following assumptions are included in this assessment:

- No depreciation has been applied for the costs and benefits into the future, all damages and benefits are assessed as 2012 dollars.
- Asset life is assumed at perpetuity, no allowances have been allocated for asset maintenance and replacement at the end of their useful life.
- No ongoing maintenance costs have been included in the estimated costs i.e. mowing levees, cleaning culverts, repairing levee wear etc.

## 8.4 Flood Mitigation Assessment

Two forms of assessment were considered for the mitigation options, a preliminary assessment of the options against the 1% AEP flood event followed by a detailed assessment of a selected set of mitigation options which was run for all flood durations. The preliminary assessment aimed at determining the effectiveness of each mitigation option and to determine if it should be included in the detailed mitigation assessment.

The preliminary runs are summarised in Table 8.2.

Table 8.2 Preliminary model runs for the mitigaion assessment

| Model Run | Model | Mitigation options included | Events | Purpose |
|---|---|---|---|---|
| 1 | Model A | A1 | 1% AEP | Assess culvert in isolation |
| 2 | Model A | A1, A2 | 1% AEP | Assess levees and culverts |
| 3 | Model B | B1 | 1% AEP | Assess u/s levees only |
| 4 | Model B | B4 | 1% AEP | Assess bridge removal only |
| 5 | Model B | B2, B3, B4 | 1% AEP | Assess levees d/s Ballarat Road |
| 6 | Model B | B1, B2, B3, B4 | 1% AEP | Assess all options in Grange Burn |

### 8.4.1 Results 1% AEP Preliminary Assessment

For the preliminary assessment the key indicators of the benefit include the number of properties and buildings impacted as well as the reduction in damages. The preliminary results are summarised in Table 8.4. It should be noted that only the buildings and property damage was included in the damage assessment, no damage to roads was considered as this was only a preliminary assessment. For Model B the 'existing' scenario is the 1% AEP model run with the existing levees remaining open on Apex Drive and Holden Street.

Table 8.3 Preliminary flood mitigation results

| Model Run | Scenario | Property (>1% and >10cm) | Overfloor Flooding | Property Damage | Reduction | Building damage | Reduction |
|---|---|---|---|---|---|---|---|
| Existing | Model A | 79 | 12 | $45,000 | | $696,747 | |
| 1 | A1 | 67 | 4 | $40,500 | 10% | $290,979 | 58% |
| 2 | A1, A2 | 64 | 3 | $41,000 | 9% | $207,437 | 70% |
| Existing | Model B | 245 | 37 | $168,700 | | $1,177,356 | |
| 3 | Original Levees | 207 | 34 | $169,700 | -1% | $1,079,583 | 8% |
| 4 | B4 | 245 | 35 | $168,700 | 0% | $1,069,474 | 9% |
| 5 | B1 | 234 | 30 | $160,700 | 5% | $861,139 | 27% |
| 6 | B2, B3, B4 | 207 | 7 | $152,700 | 9% | $251,534 | 79% |
| 7 | B1, B2, B3, B4 | 196 | 2 | $144,700 | 14% | $30,497 | 97% |

For Model A the use of additional culverts within the park reduced the expected number of properties with overfloor flooding to reduce down to 4 from 12. This in turn dropped the damages associated with building damage considerably by 58%. Adding the levees at either side of the park reduced the number of predicted overfloor flooded properties and also reduced the expected building damages. Overall, both mitigation options assist in reducing the damages near the King Street park and will be included in the final detailed mitigation assessment run.

For Model B it is evident for the 1% AEP model run that the existing levees are undersized for the revised 1% AEP flood event with flood water exceeding these levees. This resulted in property damage marginally increasing, however the levees did reduce peak water levels on some properties which reduced building damages.

Mitigation Option B4 (removing the pedestrian bridge) had little impact on the property damage but managed to reduce the building damage marginally. The pedestrian bridge removal would be expected to benefit the more frequent events as during the 1% AEP event this structure is well inundated at the flood peak and does not having a significant impact on the peak flows.

Mitigation Option B1 (levees upstream of Ballarat Road) reduced the building flood damage primarily due to maintaining the flood waters within the Grange Burn above Ballarat Road. A 27% reduction in building damages was expected due to this levee. The reduction in property damage was not significant.

The inclusion of mitigation option B2, B3 and B4 significantly reduced the impact on building due to the 1% AEP flood event. The full area to the west of Apex Drive was protected from floodwaters and this resulted in a 79% reduction in building damages. Property damage was reduced by 9%. When mitigation Option B1 is added to this suite of mitigation measures the building damage is almost removed completely with a reduction in damages of 97%. Property damages are reduced by 15% but much of this damage is associated with the property along the main floodway and cannot be mitigated. The combined mitigation options reduce the expected overfloor flooding to only 2 buildings, both of which are not residential.

For the detailed mitigation model run the combined set of options including B1, B2, B3 and B4 will be used.

### *8.4.2 Final Mitigation Cost / Benefit*

In order to assess the benefit of the detailed mitigation options the full suite of design flood events were simulated though the hydraulic model with the sets of mitigation options included as shown in Table 8.4. For the assessment of the options a costing has already been undertaken in Section 8.3.4, this section aims at determining the reduction in damages so that a cost / benefit assessment can be undertaken.

Table 8.4 Detailed model runs for the mitigaion assessment

| Model Run | Model | Mitigation options included | Events |
|---|---|---|---|
| 1 | Model A | A1 and A2 | 20%, 10%, 5%, 2%, 1% and 0.5% AEP |
| 2 | Model B | B1, B2. B3 and B4 | 20%, 10%, 5%, 2%, 1% and 0.5% AEP |

The results of the modelling for each of the runs has been summarised in Table 8.5 with the following summarised:

- Properties impacted – any property where inundated area is > 1% and to a depth of at least 10 cm.
- Overfloor flooding – where the peak flood depth exceeds the known floor level.
- Damages – Combination of the damages to buildings, property and roads.

It should be noted that the damages summarised are for the entire hydraulic model area i.e. they include Model A, B and C damages in each case.

Table 8.5 Results for the detailed mitigation runs for the Hamilton Region

| Model Run | Mitigation option applied | Properties Impacted ( > 1% inundated and > 10 cm depth) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 20% | 10% | 5% | 2% | 1% | 0.5% |
| *Existing* | *Existing* | *179* | *204* | *270* | *315* | *358* | *389* |
| 1 | A1, A2 | 171 | 195 | 257 | 298 | 344 | 377 |
| 2 | B1, B2, B3, B4 | 180 | 203 | 238 | 277 | 309 | 342 |
| All | A1, A2, B1, B2, B3, B4 | 172 | 194 | 225 | 260 | 295 | 330 |
| Model Run | Mitigation option applied | Overfloor Flooding | | | | | |
| *Existing* | *Existing* | *4* | *6* | *15* | *38* | *49* | *54* |
| 1 | A1, A2 | 0 | 0 | 7 | 31 | 40 | 48 |
| 2 | B1, B2, B3, B4 | 4 | 6 | 8 | 11 | 14 | 19 |
| All | A1, A2, B1, B2, B3, B4 | 0 | 0 | 0 | 4 | 5 | 13 |
| Model Run | Mitigation option applied | Damages ($ 2012) (each total includes the combined damages from Model A, B and C) | | | | | |
| *Existing* | *Existing* | *$203,488* | *$251,042* | *$527,906* | *$1,336,361* | *$1,833,903* | *$2,168,999* |
| 1 | A1, A2 | $0 | $13,798 | $156,613 | $902,232 | $1,354,643 | $1,766,295 |
| 2 | B1, B2, B3, B4 | $203,488 | $251,042 | $395,597 | $593,719 | $697,726 | $910,865 |
| All | A1, A2, B1, B2, B3, B4 | $0 | $13,798 | $24,304 | $159,590 | $218,466 | $508,161 |

The majority of the damages that occur in the 20% and 10% AEP flood events occur adjacent to King Street park in Model A. This is evident by the fact that in Model Run 1 where mitigation options A1 and A2 are included the 20% and 10% AEP damages are almost completely mitigated. For Model Run 2 which only includes the Model B mitigation measures the damages for the 20% and 10% AEP events remains unchanged as these damages occur within Model A.

The change in damages for the full range of modelled events is summarised graphically in Figure 8.8.

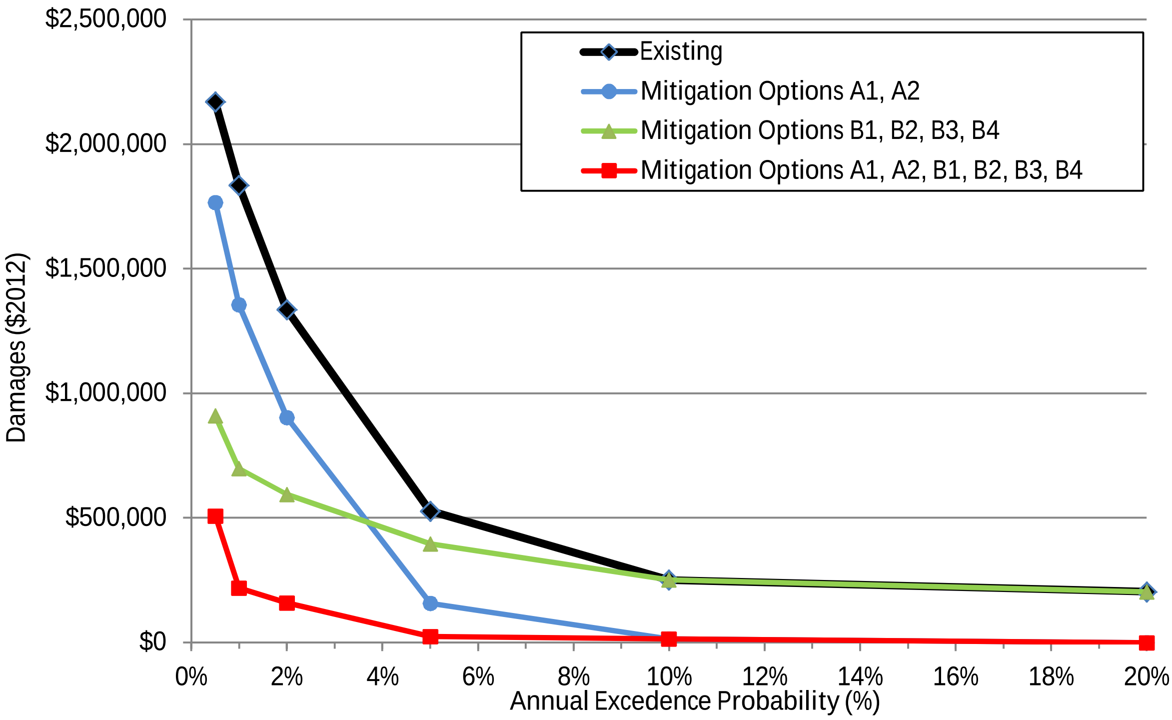



Figure 8.8 Reduction in expected damages for the full range of modelled recurrence intervals

The method applied to assess the cost / benefit of each set of options was a relative payback period as assessed using the Annual Average Damages (AAD). The AAD for the existing condition was recalculated for this assessment to be based on the curve up to the 0.5% AEP event only as the PMF event was not assessed using the mitigation options. This results in a lower AAD estimate but for the cost / benefit assessment this is a reasonable approach.

For each option the AADs were calculated and compared against the existing AADs. The reduction in damages can then be directly assessed against the estimated cost for constructing the option giving a relative period before the assets benefits outweigh the costs. The assumptions and qualifications should be noted from Section 8.3.4. The primary aim of this cost / benefit assessment is not to provide a definitive time period for a return on investment (as frequency and timing of extreme flood events are unknown) but is to determine the *relative* cost / benefit between the mitigations options considered. A summary of the cost benefit assessment is summarised in Table 8.6.

Table 8.6 Cost benefit assessment for the detailed mitigation options

| Model Run | Mitigation option applied | AAD (restricted to 0.5% AEP) | Reduction in AAD ($) | Option Estimated Cost ($ 2012) | Payback Period (years) |
|---|---|---|---|---|---|
| *Existing* | *Existing* | *$ 183,772* | | | |
| 1 | A1, A2 | $ 107,753 | $ 76,019 | $ 928,000 | 12 |
| 2 | B1, B2, B3, B4 | $ 141,195 | $ 42,577 | $ 1,152,000 | 27 |
| All | A1, A2, B1, B2, B3, B4 | $ 65,100 | $ 118,672 | $ 2,080,000 | 18 |

The mitigation measures applied in Model A had the greatest individual reduction in the AAD calculations. The primary reason for this was the fact that these mitigation options reduced the damages associated with the frequent flood events (20% and 10% AEP events). The more frequent flood events have a greater weighting in the AAD calculation due to their increased frequency. The reduction in AAD was approximately $76,000. When considering the cost for developing mitigation options A1 and A2 this implies a payback period of 12 years.

Although the mitigation options for the Grange Burn (Model B) reduced the overall damages for the larger more extreme events more significantly than for the Model A mitigation, the options did not reduce the AAD by as much as Model A with a reduction of approximately $44,500 per year. This resulted in a much longer payback period of 27 years when compared to mitigation options A1 and A2. Again the primary reason for this is that the Model A mitigation targets the more frequent flood event damage which has a greater weighting in the AAD calculation.

When all detailed mitigation options are included in the assessment the AAD is reduced significantly to $65,100 per year which is a reduction in damages of approximately $119,000 per year. This results in a payback period for all considered mitigation options of approximately 18 years. Mitigation options A1 and A2 have the greatest likelihood of providing immediate benefit to the Hamilton Community as they mitigate damages that occur in frequent events.

It should be noted that for the Marshalls Road Tributary the two reservoirs are assumed to be at full capacity in line with worst case scenario modelling for the catchment. In reality these storages may not be full at the time of a peak flood and this would reduce the expected damages downstream and hence decrease the benefit of the developed mitigation options. It should also be clearly noted that there is greater uncertainty around the flows associated with the tributaries as compared with the Grange Burn as the Grange Burn peak flows have been developed from gauged data whereas the tributaries are estimated based on best hydrological practice from this information. Further discussion of the uncertainty associated with the models can be found in Sections 4 and 5.

## 8.5 Discussion on Flood Warning Gauge Locations

Currently at Hamilton there is no formal flood warning system in place and residents rely on flood warnings supplied directly from the Victoria Police, VicSES, local CFA or word of mouth from upstream residents. There are currently no working gauges upstream of Hamilton on the Grange Burn and as such there is no automated warning system. Once a warning is received the spaces in the current levees at Apex Drive are blocked using sandbags and pumps are operated by VicSES to pass water across the temporary levees. Individual properties are protected as required. All tributaries within the Hamilton Region have no warning system in place.

The main risk associated with the current warning system are that the existing levees require hours of lead time prior to the flood arriving to be effective due to the manual sandbagging to complete the structure. If a flood occurs and here is no warning then the levees become ineffective. There is a strong case for an automated warning gauge to be operational upstream of Lake Hamilton to provide lead time and warning to Hamilton. For the tributaries it is not likely to be feasible to put flood warning gauges at these locations.

The feasible locations for an automated streamflow gauge would be at:

- Robsons Road – historic gauge location, gauge infrastructure present.
- Tarrington-Strathkellar Road (see Figure 8.9 for image of location)

Figure 8.9 Tarrington-Strathkellar Road Bridge (view upstream) (Google Maps, 2012)

The most suitable gauge location would be the Robsons Road historic gauge location. At this location the Grange Burn is constrained between a relatively steep floodplain such that rating curve could be established up to high depths to give readings during large flow events. It is recommended that the current flow gauge be upgraded to an automated gauge that would provide two functions. The primary function is to provide flood warning to Hamilton, with a secondary purpose of capturing flow data for use in future flood studies. The current estimates of the peak flows for the Grange Burn are restricted by the fact that there is only 4 years of gauged record upstream of Lake Hamilton.

The travel time for the peak to travel from the Robsons Road gauge to the Grange Burn in Hamilton is approximately 3 hours. This has been estimated based on the use of the hydrological model and anecdotal information as provided by Leo Vandooren (manager Engineering Services at Grampians Shire Council) who stated ‘The 1983 flood event had a peak travel time of 3 hours between Robsons Road and Fairburn Road’. This would allow approximately 2 hours of warning time for the levees at Apex Park to be blocked.

The location at Tarrington-Strathkellar Road could also be used as a temporary gauge location. It is unlikely that this location would be suitable for capturing long term flow information as the floodplain at this location is reasonably flat and the channel is not significantly incised. This implies that the flood waters would spread out during a large rainfall event and peak flows would be difficult to estimate. The suggestion to use this gauge as a temporary gauge location is such that a cross section would be determined at this location and rating table established and if a large rainfall event is predicted for the region with sufficient warning time then a temporary gauge can be placed at this location to provide additional flood warning.

# 9 RECOMMENDATIONS

The primary aim of the Hamilton Flood Investigation was to undertake definitive flood investigations for Hamilton and to undertake a comprehensive analysis with all available data to determine a robust 1% Annual Exceedence Probability (AEP) flood extent for the flood plains of the Grange Burn and other minor tributaries in and around Hamilton. This Flood Investigation has developed the hydrology for the Hamilton Region as well as detailed hydraulic models of the study area for generation of the key flood maps and outputs. The hydraulic models have been used to assess the design events, generate flood maps and to assess mitigation options as suggested by the Steering Committee and the Community. The estimated damage as caused by the design events was examined to determine the cost / benefit of the mitigation options.

Following this study the following actions are recommended:

- Implement a stream flow monitoring upstream of Lake Hamilton at the old Robsons Road gauge location for the purpose of additional flood warning and for use in future flood studies.
- Possibly develop a temporary (or permanent) gauge location that could be used for periods where large rainfall events are expected at Tarrington-Strathkellar Road to give additional warning times.
- Develop a gauge within Hamilton, possible location includes at Portland Road. This would allow verification of the peak flows during large events within Hamilton excluding the influence of Lake Hamilton.
- Undertake Community awareness programs to highlight the information generated within this study to the community to improve flood awareness within the community.
- Consider undertaking a dam break assessment on Lake Hamilton as this was identified as being undersized compared to the original design specification based on the revised hydrology.
- Implement the flood overlays as suggested in this study for future planning control within the catchment. Incorporate the flood overlays into the Council's future development plans.
- Consider implementing detailed assessments of the mitigation options for development (if these are to be developed in the future via funding).
- Flood maps as generated by this project should be made available to emergency response agencies to assist with the response within Hamilton.
- Ensuring that flood information such as inundated properties, peak flood heights, timing of flood events, flood depths etc are captured post each event for future studies.
- Implementing a plviograph station within the Hamilton catchment would assist future flood investigations as this would aid the calibration of hydrologic models within the catchment. This gauge could be located within Hamilton or upstream within the catchment.

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# Appendix A

# RORB Vector