Dunolly Flood Investigation

Study Report

July 2014

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PROJECT DETAILS

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Michael Cawood & Associates Pty Ltd

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GLOSSARY OF TERMS

Annual Exceedance 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. Introduced in 1971 to eventually supersede all earlier datum's.
Average Recurrence Interval (ARI)	Refers to the average time interval between a given flood magnitude occurring or being exceeded. A 10 year ARI flood is expected to be exceeded on average once every 10 years. A 100 year ARI flood is expected to be exceeded on average once every 100 years. The AEP is the ARI expressed as a percentage.
Cadastre, cadastral base	Information in map or digital form showing the extent and usage of land, including streets, lot boundaries, water courses etc.
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 significant event to be considered in the design process; various works within the floodplain may have different design standards. A design flood will generally have a nominated AEP or ARI (see above).
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.
Flash flooding	Flooding which is sudden and often unexpected because it is caused by sudden local heavy rainfall or rainfall in another area. Often defined as flooding which occurs within 6 hours of the rain which causes it.
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 and/or coastal inundation resulting from elevated sea levels and/or waves overtopping coastline defences.
Flood damage	The tangible and intangible costs of flooding.
Flood frequency analysis	A statistical analysis of observed flood magnitudes to determine the probability of a given flood magnitude.
Flood hazard	Potential risk to life and limb caused by flooding. Flood hazard combines the flood depth and velocity.
Flood mitigation	A series of works to prevent or reduce the impact of flooding. This includes structural options such as levees and non-structural options such as planning schemes and flood warning systems.
Floodplain	Area of land which is subject to inundation by floods up to the probable maximum flood event, i.e. flood prone land.
Flood storages	The parts of the floodplain that are important for the temporary storage, of floodwaters during the passage of a flood.

Freeboard	A factor of safety above design flood levels typically used in relation to the setting of floor levels or crest heights of flood levees. It is usually expressed as a height above the level of the design flood event.
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.
Intensity frequency duration (IFD) analysis	Statistical analysis of rainfall, describing the rainfall intensity (mm/hr), frequency (probability measured by the AEP), duration (hrs). This analysis is used to generate design rainfall estimates.
MIKE FLOOD	A hydraulic modelling tool used in this study to simulate the flow of flood water through the floodplain. The model uses numerical equations to describe the water movement.
Ortho-photography	Aerial photography which has been adjusted to account for topography. Distance measures on the ortho-photography are true distances on the ground.
Peak flow	The maximum discharge occurring during a flood event.
Probability	A statistical measure of the expected frequency or occurrence of flooding. For a fuller explanation see Average Recurrence Interval.
Risk	Chance of something happening that will have an impact. It is measured in terms of consequence and likelihood. For this study, it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
RORB	A hydrological modelling tool used in this study to calculate the runoff generated from historic and design rainfall events.
Runoff	The amount of rainfall that actually ends up as stream or pipe flow, also known as rainfall excess.
Stage	Equivalent to 'water level'. Both are measured with reference to a specified datum.
Stage hydrograph	A graph that shows how the water level changes with time. It must be referenced to a particular location and datum.
Topography	A surface which defines the ground level of a chosen area.
1D (one dimensional)	Refers to the hydraulic modelling where creeks and hydraulic structures are modelled using 1 dimensional methods. Using surveyed cross-sections to represent the path of water flow, the model calculates how high and how fast the water will flow for the specified flow path.
2D (two dimensional)	Refers to the hydraulic modelling where the floodplain is modelled using 2 dimensional methods. Using a grid of topography data the model will estimate not only how high and how fast water will flow but will also calculate the direction of flow across the 2D grid.

EXECUTIVE SUMMARY

Introduction

Following the recent flood events in September 2010 and January 2011, Dunolly was identified as a high flood risk community and funding was approved for a flood investigation of the township. The Dunolly Flood Study was run by the North Central CMA in conjunction with Central Goldfields Shire Council. The study included detailed hydrological and hydraulic modelling of Burnt Creek, flood mapping of the Dunolly area and the upstream catchment and also provided recommendations for flood mitigation works.

Study Area

Dunolly is situated in Central Victoria, approximately 20 km north of Maryborough and less than 50 km west-south-west of Bendigo. The township has a population of approximately 700 people and is situated in the Central Goldfields Shire Council. It is one of a number of towns in the shire that was seriously impacted in the 2010-2011 floods.

Burnt Creek flows through Dunolly with a catchment area of approximately 117 km^2, with its headwaters to the north-west near Moliagul. A small tributary flows from north-east of the town with a catchment area of approximately 14 km^2, flowing into Burnt Creek to the north-west of town. A contour levee and channel protects the town from local runoff from the forested hills immediately east of the township. The levee and drain is falling into disrepair but stills functions as flood protection. This channel drains the eastern slopes between Bridgewater-Dunolly Road and Dunolly-Eddington Road, discharging to Burnt Creek to the south-east of town. Burnt Creek flows into Bet Bet Creek approximately 10 km to the south-east, with Bet Bet flowing into Laanecoorie Reservoir a further 8 km downstream. There are currently no river gauging stations located on Burnt Creek, and Dunolly receives very little flood warning.

The study area and its features can be seen in Figure 1.

In large flood events Dunolly becomes isolated as access to the town is prevented by inundation of major roads both within and outside of the study area.

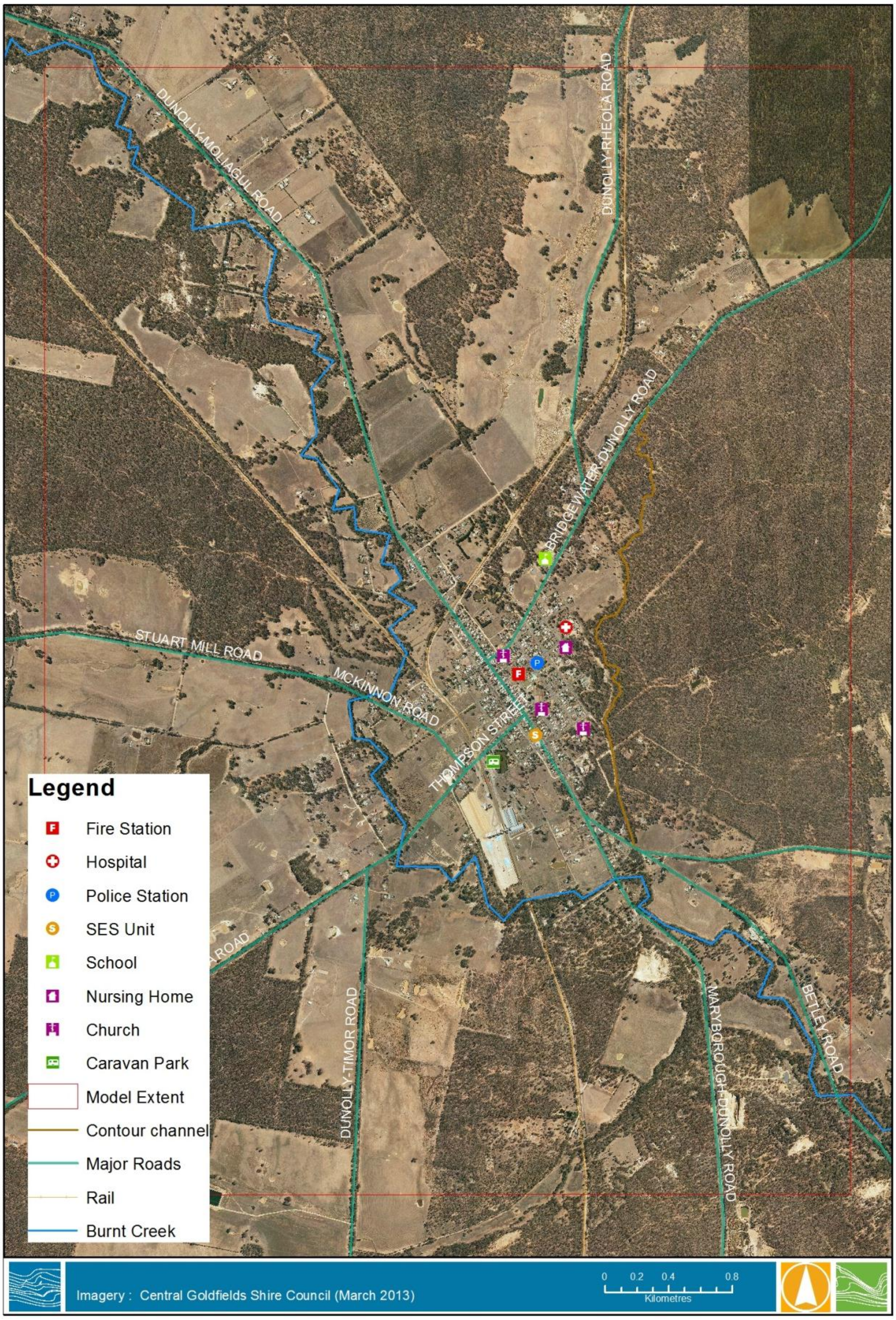

Figure 1 Study area features

Data Collation and Review

As part of the initial scoping work, the data required for modelling and mapping was collated and reviewed. This included:

- Streamflow data for Bet Bet Creek (no streamflow data exists for Burnt Creek)
- Rainfall data at 25 nearby daily rainfall stations; 7 tipping bucket rainfall stations; and 3 pluviographs (instantaneous rainfall data)
- Digital elevation models of the study area (i.e. topography)
- Feature survey of key hydraulic structures (commissioned during the study)
- Floor level survey (commissioned during the study)
- Surveyed flood marks from the January 2011 flood event

The data was supplemented by a significant amount of anecdotal evidence provided by Dunolly SES and community members.

An initial site visit was undertaken by Water Technology on 19th February 2013. During this visit no waterways were flowing, and there was no remnant water in the creek beds. A number of subsequent site visits were carried out during the study, with the full length of the contour channel walked as well as a number of sections of creek, all potential flood mitigation sites and the Old Lead Reservoir and its contour channels all inspected. On one of the site visits, Council, CMA, VICSES and community members accompanied the study team and described various stages of the January 2011 floods, pointing out sites of interest.

Community Consultation

Throughout the study, a range of community consultation activities were undertaken, including community drop-in sessions, media releases and questionnaires to ensure that community issues were heard and the ideas of the community were considered in the development of potential flood mitigation options. It must be noted that the community participation was very helpful, with flood observations, local information and feedback on the study greatly improving the outcomes for the study.

Model Schematisation / Development

A hydrological model of the catchment was developed for the purpose of estimating historic flood flows for calibration and design events. These flows were used as boundary conditions to the MIKE FLOOD hydraulic model, which comprised of:

- A one dimensional (1D) hydraulic model of key hydraulic structures and the downstream boundary of Burnt Creek; and
- A two dimensional (2D) hydraulic model of Burnt Creek, its tributaries and the broader floodplain.

As there are no streamflow gauges on Burnt Creek the hydrology and hydraulic models were verified in series, with the results of the hydraulic model feeding back into revised hydrology, as an iterative process.

The predicted flows from the RORB model (preliminarily calibrated to Bet Bet Creek gauge) were used as input to the hydraulic model and the resulting extent and levels compared to those observed during the January 2011 event. An overview of the modelling approach is given in Figure 2.

Verification of flows with this methodology was an iterative process. Where significant over and/or underestimations occurred in water level and extent in the hydraulic model, the hydrological model parameters were refined. Once a close agreement was achieved in both the anecdotal timing of the peak flow and the modelled water levels and extent within Dunolly the hydraulic model parameters were then used to fine tune the model results.

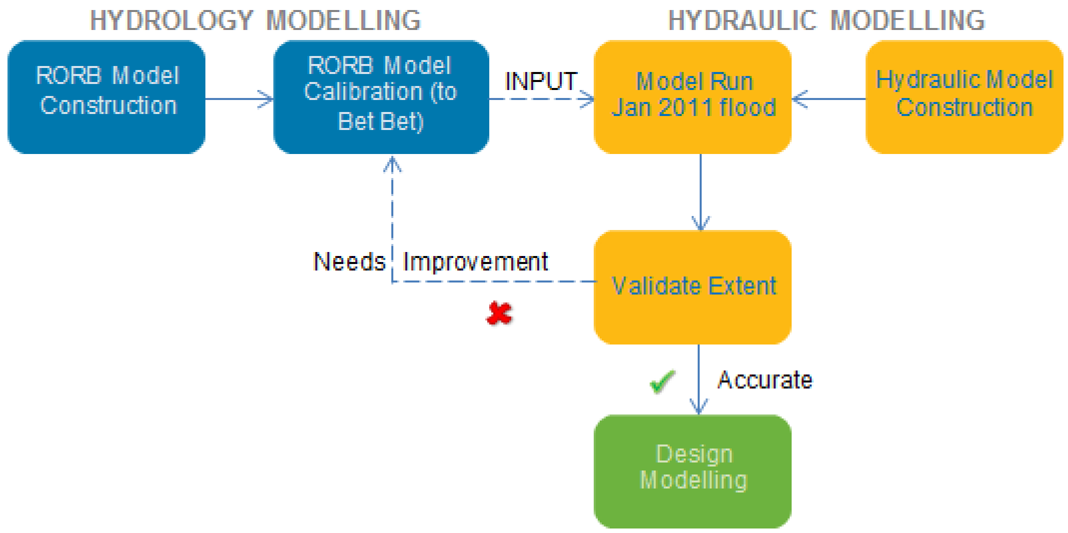

Figure 2 Modelling approach flow diagram

The resulting flood extent for the calibrated January 2011 flood event received support from community members at the drop in session held on the 26th June 2013.

Design Event Modelling

Following on from the successful RORB model and hydraulic model verification, a series of design events were modelled. This required the adoption of various design parameters to be included within RORB to generate design hydrographs for input to the hydraulic model. For this study the 20%, 10%, 5%, 2%, 1% and 0.5% AEP events were required.

The model considered temporal and spatial distributions of rainfall. A sensitivity analysis was undertaken on the model parameters adopted, and resulting flows were compared against estimates using other methods. A panel of technical experts from DEPI have reviewed and approved the methodology undertaken by Water Technology to derive the design flood estimates.

From an assessment of the range of flood events modelled and the resulting flood extent it was apparent that Burnt Creek itself inundates a very small number of buildings within Dunolly with the majority of the flood risk attributed to local runoff from the catchment to the north-east of town which is currently captured by a contour channel drain which diverts these flows around town.

The 1% annual exceedance probability flood extent can be seen in Figure 3.

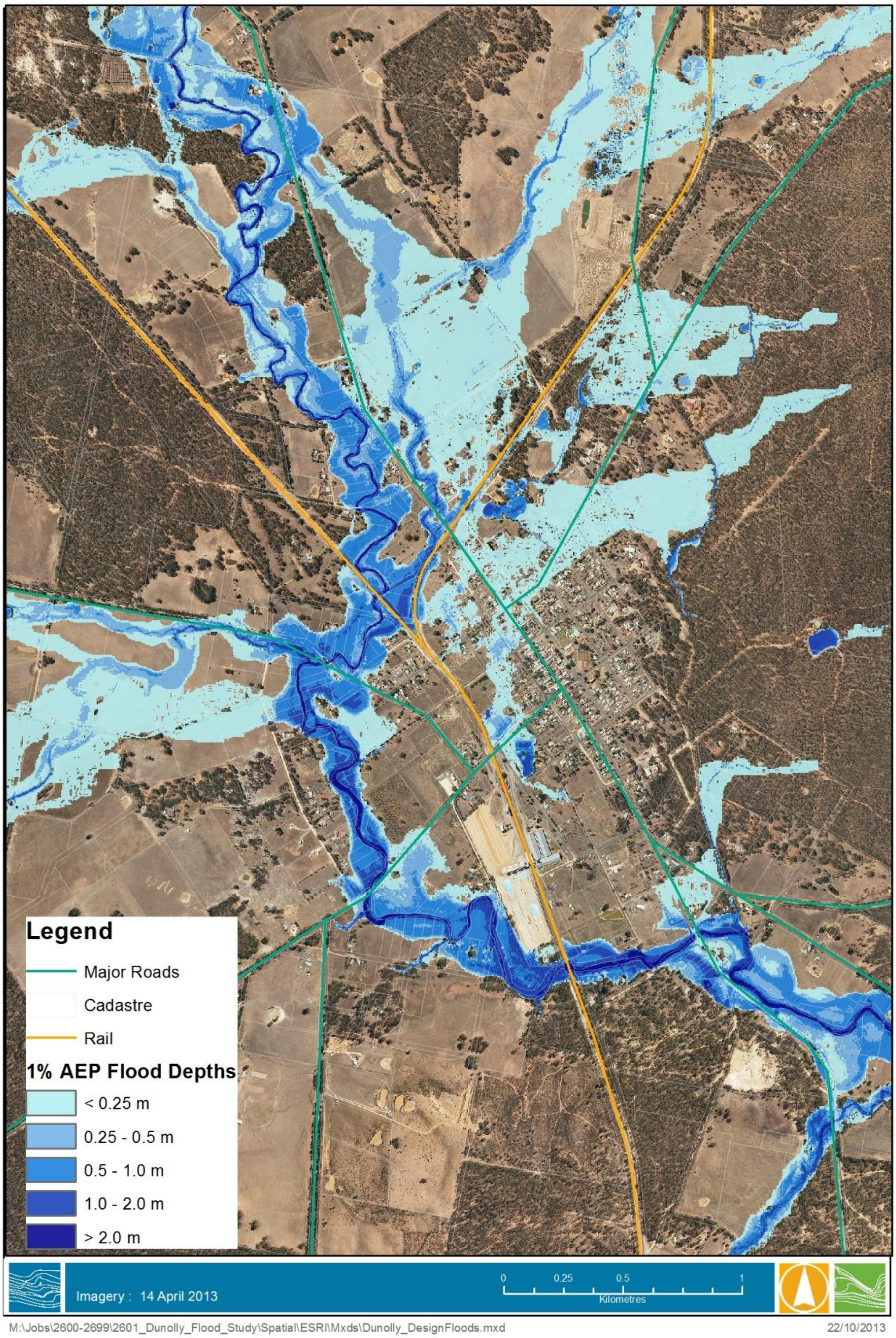

Figure 3 1% AEP Design Flood Extent

Flood Mitigation

A number of the properties impacted by Burnt Creek are isolated and would therefore require individual mitigation such as a private levee to protect single dwellings. An initial assessment found that individual property protection is unlikely to a cost effective solution for the community, and the Steering Committee determined not to pursue this option.

An initial prefeasibility assessment of 16 structural mitigation options was undertaken. From this assessment, three options were selected for further analysis using the developed hydraulic model. These included a levee near the cemetery and a levee on Broadway Road that protected a number of properties impacted in one area, and an upgrade of the contour channel.

While the levees were able to improve flooding impacts at a number of properties they redirected flood waters and increased flood levels at other properties and were therefore not considered feasible. An upgrade of the contour channel to improve capacity, and the incorporation of a retarding basin was the preferred mitigation option for the Dunolly Flood Study.

The option to upgrade the contour channel and incorporate a retarding basin to slow the rate of flow returned a very high benefit to cost ratio of 3.2. The contour channel is very important to the flood protection of Dunolly, it was originally constructed for a very good reason, but over the years has fallen into disrepair. Substantial flood protection can be provided with a relatively modest investment and upgrade, and ongoing maintenance of the channel.

Key Concerns Addressed

The Old Lead Reservoir was a major discussion point in all community consultation with many members of the community believing that it either posed a risk to the town if it was to fail and/or that it should be drained down so to capture runoff in a large storm event. The Old Lead Reservoir was subject to specific analysis which showed that the storage volume of the dam was far too small to reduce flood flows in a large event like January 2011, and in fact the volume of water that flowed through it in that event was approximately 4.5 times its storage capacity.

Another key area of concern for residents is access in and out of the town during floods. Critical information developed by this study (e.g. timings and flows) has been incorporated into the flood emergency response plan, and will aid in providing improved flood warning and response including evacuation.

Recommendations

Following significant consultation with the Dunolly Community, the Dunolly Flood Study Steering Committee recommends the following actions:

- Amendment of the planning scheme for Dunolly to reflect the flood risk identified by this project.
- Mitigation Package 3 (an upgrade of the contour channel and retarding basin) to be submitted for funding for detailed design and construction.
- The updated Municipal Flood Emergency Plan be used during a flood event to improve the emergency response.
- In any future bridge upgrade projects, consideration be given to elevating bridges to provide access during a major flood because currently the town becomes completely isolated by road.
- Installation of a gauge board within town to base future observations on and to tie the flood maps back to the gauge. This gauge could also be linked to a flash flood warning system should that be considered in the future.

Water Technology would like to thank the Steering Committee for their diligence in delivering a quality study in a timely manner.

It should be noted that this document does not represent policy of North Central CMA, Central Goldfields Shire or Government. This is a technical report produced as part of the Dunolly Flood Study. There are many considerations that must be made following the completion of this study by all stakeholders and Government prior to implementing any of the recommendations.

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1. INTRODUCTION

1.1 Background

Following the recent flood events of September 2010 and January 2011 where the Dunolly Community was significantly impacted, Water Technology was commissioned by the North Central CMA to undertake the Dunolly Flood Study. This study included detailed hydrological and hydraulic modelling of Burnt Creek, flood mapping of the Dunolly area and the upstream catchment and also provided recommendations for flood mitigation works.

As part of the investigation process there were several reporting stages to ensure the study was reviewed and approved by the Steering Committee. This report is the Study Report, a compilation of all the previous staged reports:

- Data Collation, Review and Model Scoping (8 August 2013)
- Hydrology and Hydraulics Report (21 June 2013)
- Design Hydrology and Hydraulics Report (1 November 2013)
- Mitigation Options Prefeasibility Assessment Memo (20 December 2013)
- Mitigation Package 2 Detailed Results Memo (11 April 2014)
- Mitigation Package 3 Detailed Results Memo (16 May 2014).

A Municipal Flood Emergency Plan has also been produced in collaboration with Michael Cawood & Associates. This report has been submitted to VicSES and North Central CMA independently.

1.2 Study Area

Dunolly is situated in Central Victoria, approximately 20 km north of Maryborough and less than 50 km west-south-west of Bendigo. The township has a population of approximately 700 people and is situated in the Central Goldfields Shire Council. It is one of a number of towns in the shire that was seriously impacted in the 2010-2011 floods.

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The study area and its features can be seen in Figure 1-1.

In large flood events Dunolly becomes isolated as access to the town is prevented by inundation of major roads both within and outside of the study area.

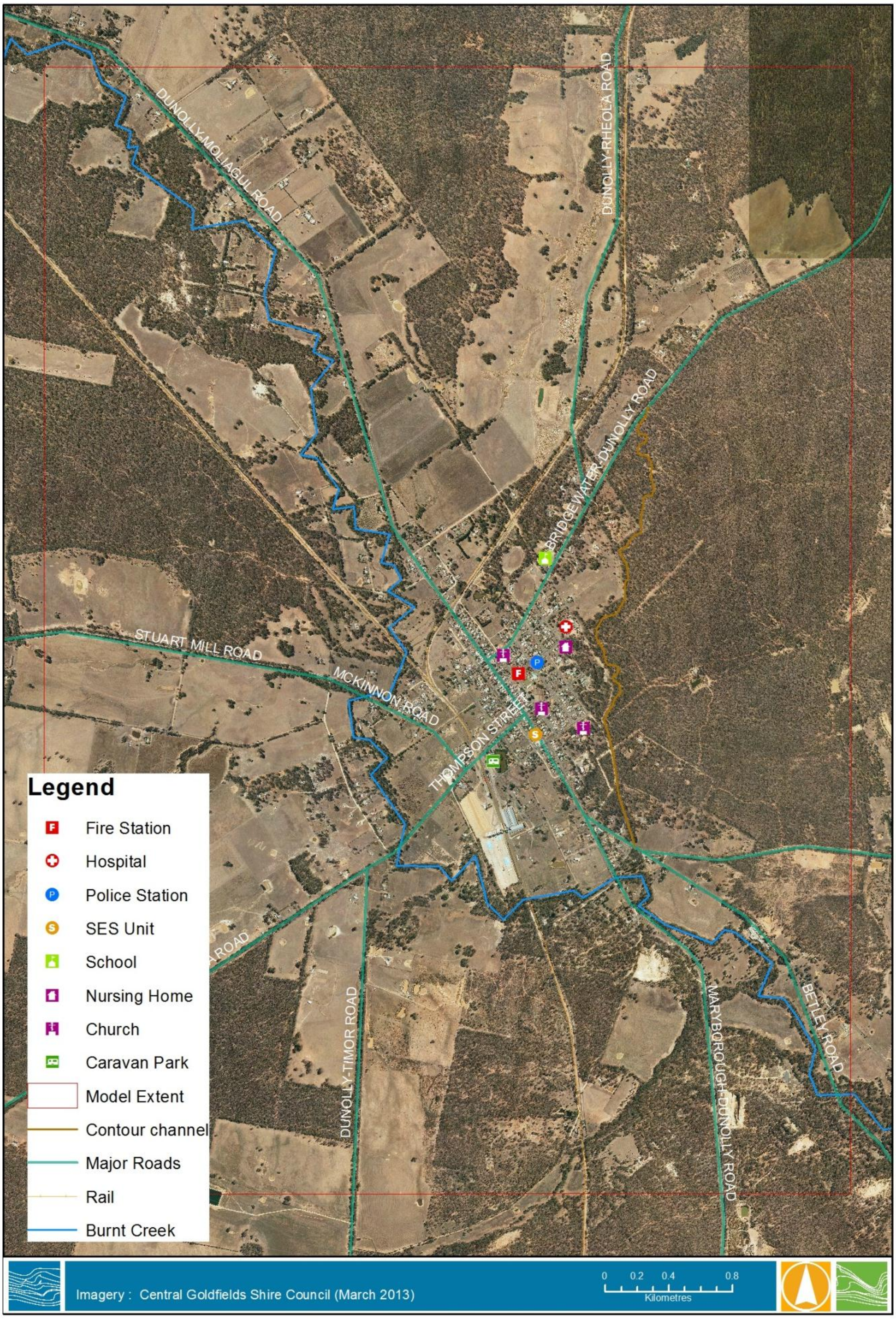


Figure 1-1 Study area features

2. DATA COLLATION AND REVIEW

2.1 Site Visit

An initial site visit was undertaken by Water Technology on 19th February 2013. During this visit no waterways were flowing, and there was no remnant water in the creek beds.

During the site visit, a number of photos were taken of Burnt Creek and its tributaries, drainage structures and floodplain features. These photos are shown in Appendix A. The dimensions of all structures located along the creek were roughly surveyed using a tape measure, measuring back to the road deck. These field measurements were used in combination with feature survey as part of the hydraulic model development.

A number of subsequent site visits were carried out during the study. The full length of the contour channel was walked as well as a number of sections of creek, all potential flood mitigation sites and the Old Lead Reservoir and its contour channels were all inspected. On one of the site visits, Council, CMA, VICSES and community members accompanied the study team and described various stages of the January 2011 floods, pointing out sites of interest.

2.2 Current Planning Scheme

Burnt Creek is currently covered by a Land Subject to Inundation Overlay (LSIO) within the Central Goldfields Shire Council planning scheme. The LSIO shows the area along Burnt Creek, a tributary to the north and other small gullies to the south-west as prone to flooding, as seen in Figure 2-1. It is not certain what this LSIO is based on, but it is likely that it has considered an investigation carried out by the Rural Water Commission in 1986, in which they estimated historic peak flows and design flows and estimated flood levels.

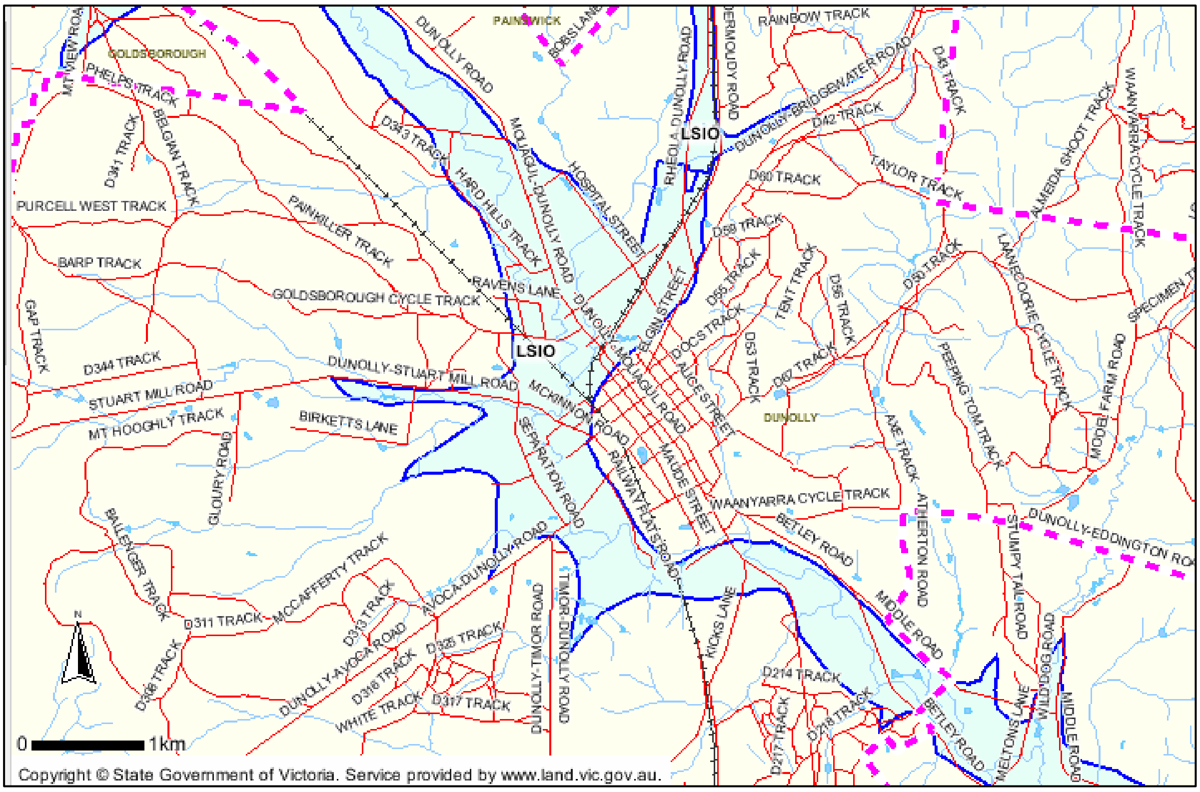


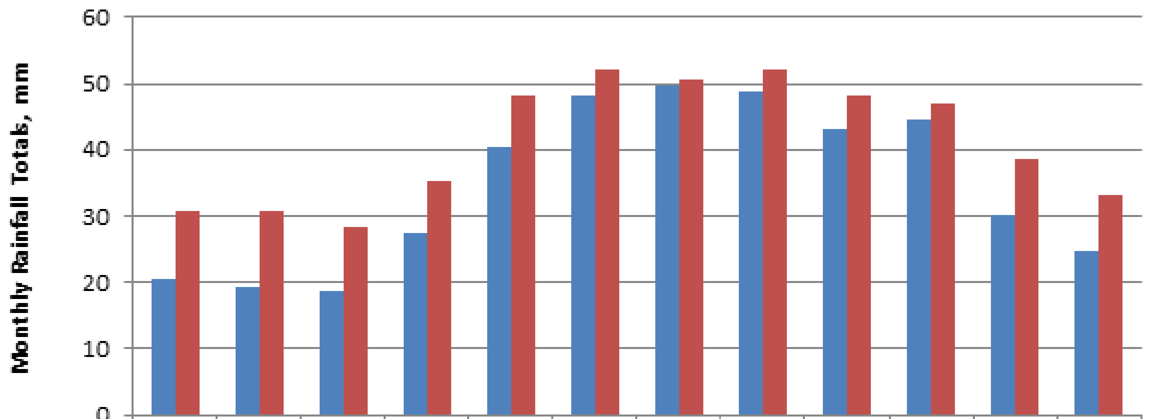
Figure 2-1 Land Subject to Inundation Overlay

2.3 Historical Flooding

Dunolly has a long history of flooding and has been impacted by several large events. Historical documents from the Shire of Bet Bet and anecdotal evidence from newspaper clippings and community member interviews indicates that major flood events occurred during the following years:

- February 1873
- November 1893
- September 1983
- April 1959 (used as a basis for developing the 1% probability flood profile at the time)
- September 2010
- January 2011

Figure 2-2 shows the mean and median monthly rainfall totals for the entire length of record at the Dunolly rainfall gauge (1882-2012). The wettest months are typically in winter with June, July and August recoding the highest mean values.



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Median	20.4	19.2	18.6	27.3	40.4	48.3	49.8	48.8	43.0	44.7	30.2	24.6
Mean	30.7	30.7	28.4	35.2	48.1	52.2	50.6	52.0	48.2	46.9	38.5	33.2

Figure 2-2 Historical rainfall records for the Dunolly rainfall gauge

2.3.1 January 2011

The January 2011 flood was a very significant flood in living memory, one of the largest, perhaps only being exceeded by the 1959 flood. Over 200 mm of rainfall was recorded over a 5 day period with maximum daily rainfall totals exceeding 90 mm. The heavy rainfall led to the town being completely isolated with only air or rail access. Forty homes were evacuated with areas along Burnt Creek and Broadway seriously impacted.

Floodwaters began impacting the township in the early hours of Friday the 14th of January, peaked at about 10:30 am and receded later in the day with residents returning to their homes in the afternoon. It is estimated that approximately 20 houses were flooded.

Streamflow data for this event is not available for either the Burnt Creek catchment or nearby Bet-Bet Creek catchment.

2.3.2 September 2010

The September 2010 flood was a result of heavy rain in Dunolly falling over a 5 day period starting on the 4th September. The town received 85 mm, with much of the rain falling at the beginning of this period.

Instantaneous streamflow data is available for this event for the nearby Bet Bet Creek catchment, however there is little available data relating to the timing of this flood event and extents within the Burnt Creek catchment.

2.3.3 September 1983

Flooding in September 1983 was as a result of 94 mm of rainfall falling on the catchment over a 6 day period, from the 25th to the 30th September. Rainfall data for this event is incomplete and little data relating to the flood extents and timing is available. No streamflow data is available for either the Burnt Creek or nearby Bet Bet Creek catchment.

2.3.4 April 1959

Historical documents from the Rural Water Commission of Victoria and the Shire of Bet Bet (provided by the North Central CMA) indicate the April 1959 flood to be of significant magnitude, with some houses recording flood levels at window height.

2.4 Hydrological Data

2.4.1 Streamflow

There are no streamflow gauges in the Burnt Creek catchment upstream of the Dunolly township. The nearest existing gauge is on Bet Bet Creek at Bet Bet (407211) just upstream of the confluence of Burnt Creek and Bet Bet Creek. The gauge has records from 24/09/1943 to current; however some data records are missing, including data during the time of the January 2011 flood event.

This stream flow gauge has a rating curve coded as reliable for flows between 0 and 33,900 ML/d, with the curve extrapolated out to flows of 57,400 ML/d[1].

2.4.2 Rainfall

There are numerous daily rainfall gauges in close proximity to Dunolly as well as seven instantaneous tipping bucket gauges and three pluviographs. Rainfall gauges that were of relevance to the Dunolly Flood Study are shown in Figure 2-3 and Table 2-1.

[1] Data.water.vic.gov.au/monitoring.htm (accessed 31 July 2014)

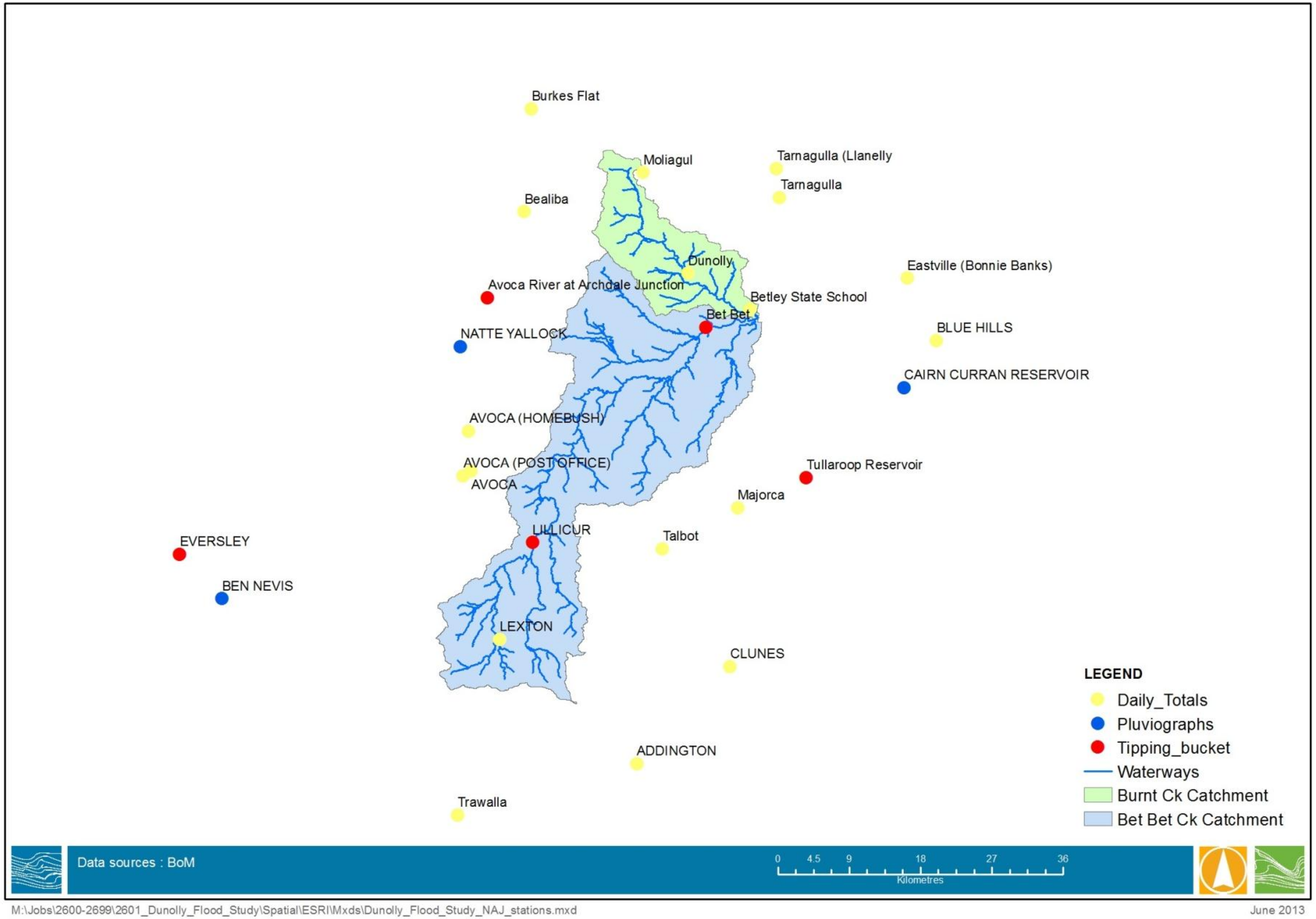


Figure 2-3 Rainfall Station Locations

Table 2-1 Rainfall gauges relevant to Burnt Creek and Bet Bet Creek

Name	Gauge Number	Length of Record	Type	January 2011 rainfall depth (mm)*	September 2010 rainfall depth (mm) #
Eversley	79014	1888-2012	Daily	144.2	69.3
Ben Nevis	79101	2007-2013	Daily	157.4	70.0
Avoca (Post Office)	81000	1884-2012	Daily	211.4	92.0
Bealiba	81002	1891-2012	Daily	206.8	77.0
Burkes Flat	81006	1902-2012	Daily	178.8	69.4
Natte Yallock	81038	1898-2012	Daily	200.6	79.6
Tarnagulla	81047	1888-2012	Daily	204.0	no data
Avoca	81063	1889-2012	Daily	no data	no data
Dunolly	81085	1882-2012	Daily	206.4	85.2
Moliagul	81090	1968-2012	Daily	228.9	86.0
Eastville (Bonnie Banks)	81092	1969-2012	Daily	182.0	50.0
Avoca (Homebush)	81122	1986-2012	Daily	201.6	81.6
Avoca River at Archdale Junction	81127	2001-2013	Daily	191.8	72.6
Tarnagulla (Llanelly)	81128	2011-2012	Daily	no data	no data
Betley State School	88006	1928-1944	Daily	no data	no data
Cairn Curran Reservoir	88009	1949-2012	Daily	182.2	49.2
Clunes	88015	1879-2013	Daily	208.4	88.6
Lexton	88038	1903-2012	Daily	79.0	62.0
Tullaroop Reservoir	88052	1881-2012	Daily	127.4	83.6
Talbot	88056	1898-2012	Daily	233.8	90.6
Blue Hills	88132	1972-2012	Daily	184.2	43.8
Lillicur	88137	2002-2012	Daily	234.0	81.4
Majorca	88160	1987-2012	Daily	217.8	115.6
Trawalla	89030	1888-2012	Daily	218.4	52.8
Addington	89106	1956-2012	Daily	245.4	68.8
Avoca @ Archdale	408206	1998-2012	Tipping Bucket	192.2	56.4

Junction					
Avon Wimmera Highway	415220	1989-2012	Tipping Bucket	130.0	37.6
Bet Bet	407211	1990-2012	Tipping Bucket	203.8	72.0
Laanecoorie Reservoir	407240	1997-2012	Tipping Bucket	194.0	74.0
Lillicur	407288	1990-2012	Tipping Bucket	223.0	56.4
Tularoop Creek	407222	1994-2012	Tipping Bucket	186.8	99.6
Wimmera Eversley	415207	1992-2012	Tipping Bucket	70.8	33.2
Laanecoorie Weir	81026	1973-2004	Pluviograph	no data	no data
Natte Yallock	81038	1974-2010	Pluviograph	no data	no data
Cairn Curran Reservoir	88009	2004-2010	Pluviograph	no data	no data

* January 2011 total is a 5 day total summed over the period 10/1/2011 to 14/1/2011

September 2010 total is a 4 day total summed over the period 4/9/2010 to 7/9/2010

2.5 Survey Data

Components of this study are based on topographic Light Detection And Ranging (LiDAR) data. LiDAR is an aerial laser survey technique which captures high resolution survey from fixed wing aircraft. As part of the investigation the LiDAR data was verified against feature survey data. Analysis was undertaken to verify the levels predicted by the LiDAR against feature survey to ensure its accuracy for input to the hydraulic model.

Three sources of topographic/survey data were obtained to prepare the hydrological and hydraulic models used in the Dunolly Flood Study:

- Vicmap Elevation DTM 20 m (a raster representation of Victoria's elevation at a 20 m grid resolution as provided by DEPI, note this has a low vertical accuracy);
- Light Detection And Ranging (LiDAR) data (provided by the North Central CMA on the 11 February 2013; and
- Field survey (undertaken by Tomkinson).

2.5.1 Field Survey

Information (dimensions, inverts) of the key hydraulic structures along Burnt Creek and its tributaries was required for input into the hydraulic model. Figure 2-4 shows the location of the key waterway structures within the study area, with Table 2-2 providing details of the structure.

Field survey was also used to confirm the reliability of the LiDAR data. This is discussed in Section 2.5.2 in further detail.

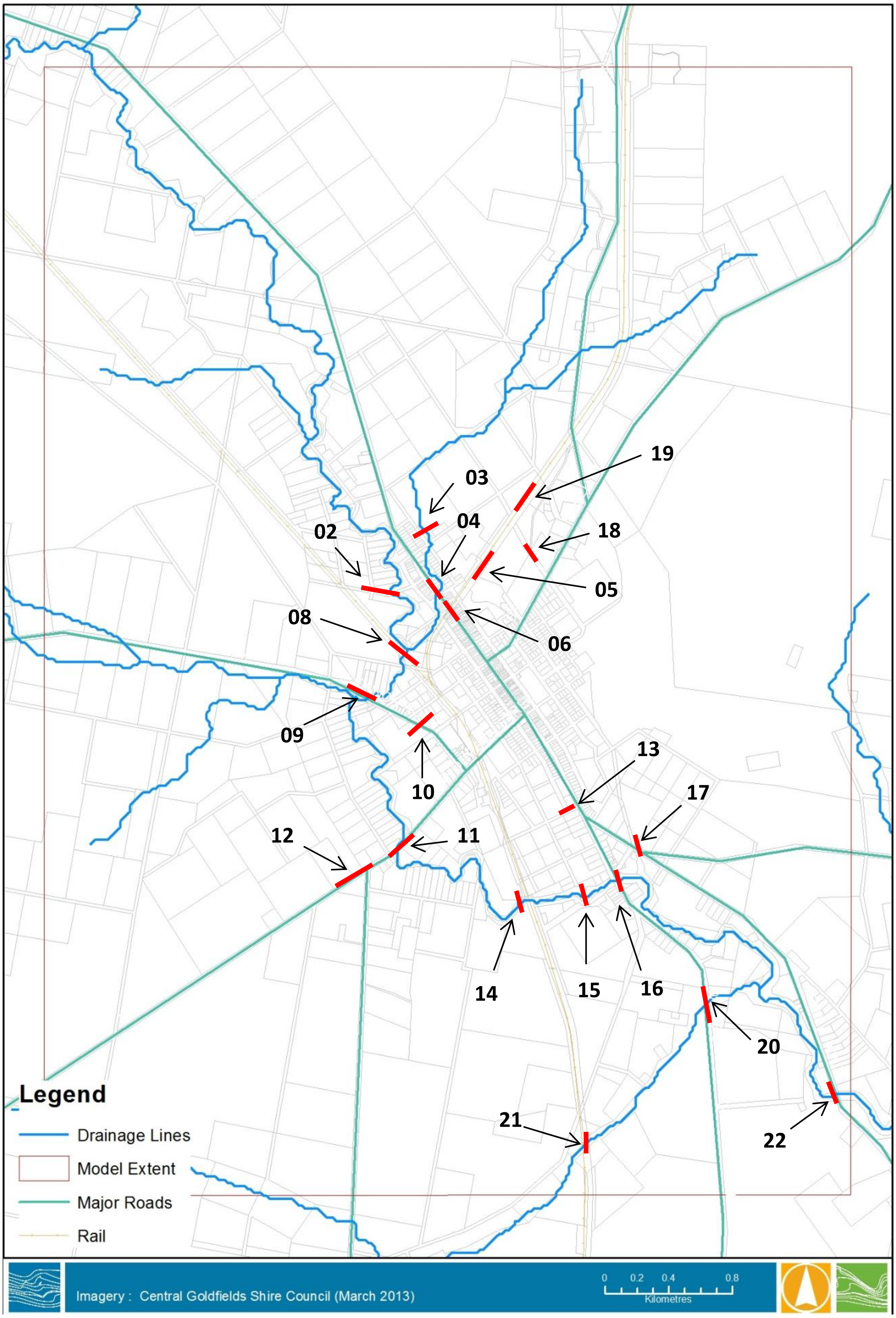


Figure 2-4 Key hydraulic structures in the study area

Table 2-2 Details of key hydraulic structures within the study area

Structure	Waterway	Crossing	Structure Details
01	Burnt Creek	Burnt Creek Ln	(1) 1.2m
02	Burnt Creek	Short St	(3) 2.1m x 4.2m
03	Tributary	Raglan St	(3) 0.5m x 1m
04	Tributary	Dunolly-Moliagul Rd	(5) 1.2m x 1.8m
05	Local	Railway crossing	Bridge
06	Local	Dunolly-Moliagul Rd	(5) 0.9m x 1.2m
07	Local	Hospital St	(1) 800mm
08	Burnt Creek	Railway crossing	2 x bridges
09	Burnt Creek	McKinnon St	Bridge
10	Local	McKinnon St	(1) 700mm
11	Burnt Creek	Dunolly Avoca Rd	Bridge
12	Local	Dunolly Avoca Rd	(1) 600mm
13	Local	Clark St	(1) 500mm
14a	Local	Railway crossing	Arch
14b	Burnt Creek	Railway crossing	Bridge
15	Burnt Creek	Maude St	(2) 600mm
16	Burnt Creek	Maryborough-Dunolly Rd	Bridge
17	Local	Dunolly Rd	(2) 1200mm
18	Local	Hospital St	Drain
19	Local	Railway crossing	Bridge
20	Local	Maryborough-Dunolly Rd	(1) 1200mm
21	Tributary	Railway	Bridge, 7 piers
22	Burnt Creek	Betley Road	Bridge, single pier

2.5.2 LiDAR data

LiDAR data for the study area was provided by the North Central CMA. The LiDAR was flown in August 2011 with a vertical accuracy of 0.1 m and a horizontal accuracy of 0.2 m.

Survey Comparison

A comparison between LiDAR elevations and surveyed elevations was made to determine the consistency of the data. Two 100 m transects, with points surveyed at 10 m intervals were taken along McKinnon Road and Market Street for this purpose.

As seen in Figure 2-5 and Figure 2-6, the LiDAR elevations were consistently higher than the surveyed elevations. The differences are summarised in Table 2-3. The discrepancy (mean difference of 54 mm) was well within the nominal vertical accuracy of LiDAR (+/- 200 mm) and as such no adjustment to the LiDAR was made.

Table 2-3 Statistics for difference between LiDAR and surveyed elevations

DIFFERENCE	Transect 1 (McKinnon Rd)	Transect 2 (Market St)
MIN	-5.290 mm	-21.635 mm
MAX	-78.294 mm	-73.039 mm
MEAN	-51.144 mm	-56.135 mm
STDEV	24.732 mm	14.013 mm

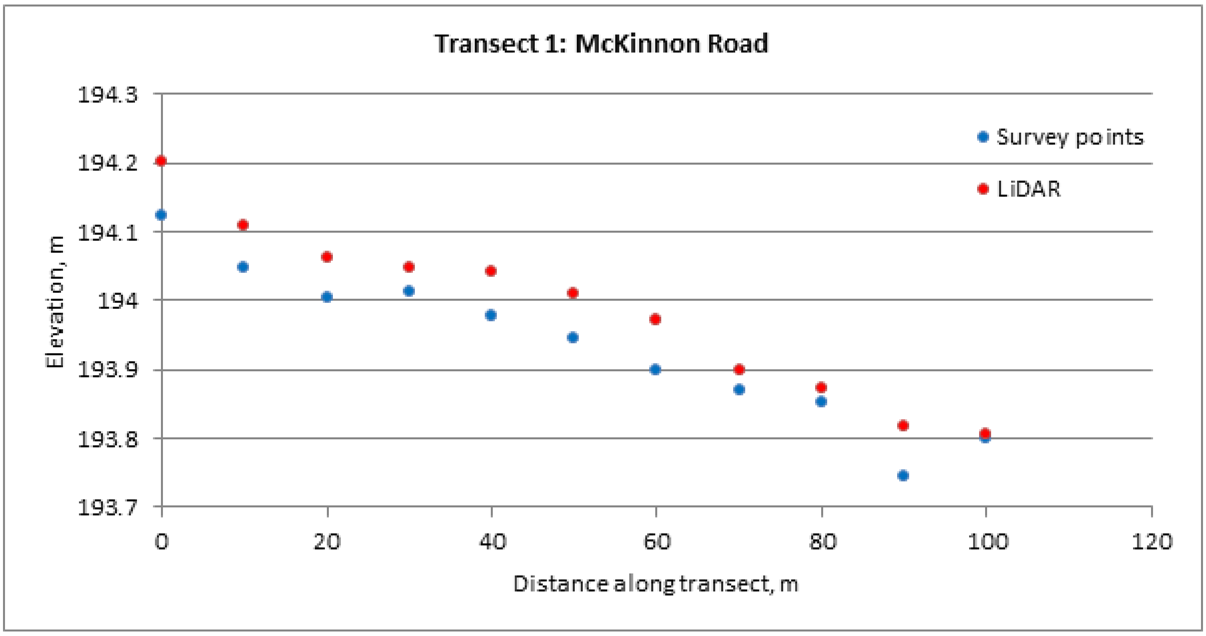


Figure 2-5 LiDAR and survey elevations along McKinnon Road

Figure 2-6 LiDAR and survey elevations along Market Street

2.6 Other Background Data

A substantial amount of anecdotal evidence was provided by community members, after a request for information was published in the local newspaper, the Welcome Record, on the 6th March 2013. This information included: rainfall data; hand marked maps of flood extents; date and time stamped photographs of flood events; and historic newspaper articles.

Other background data available for the study included:

- Floor level survey (commissioned during the study);
- Cadastral information sourced from DEPI;
- 1% AEP flow estimates for Burnt Creek calculated by the State Rivers and Water Supply Commission in 1986;
- A memorandum from the Rural water Commission of Victoria regarding 1% AEP Flood levels for Burnt Creek at Dunolly dated 27 June 1986;
- A letter from the Shire of Bet Bet outlining flood levels for Sections 4A and 4C Dunolly dated January 14 1986;
- Dunolly SES debrief report for the January 2011 floods; and,
- Rapid Impact Assessment report by the Office of the Emergency Services Commissioner for the January 2011 flood.

3. MODEL DEVELOPMENT

3.1 Overview

A hydrological model of the catchment was developed for the purpose of estimating historic flood flows for calibration and design events. These flows were used as boundary conditions to the MIKE FLOOD hydraulic model.

3.2 RORB Model

The rainfall-runoff program RORB (Version 6) was used for this study. RORB is a non-linear rainfall runoff and streamflow routing model for calculation of flow hydrographs in drainage and stream networks. The model requires catchments to be divided into subareas; connected by a series of conceptual reach storages. Storm rainfall is input to the centroid of each subarea. Specific losses are then deducted, and the excess routed through the reach network.

A new RORB hydrological model was developed using MiRORB (MapInfo RORB tools). The following methodology was applied to construct the RORB model:

- Delineation of the Burnt Creek catchment area upstream of Dunolly;
- Division of the catchment into subareas based on the site's topography and required hydrograph print (result) locations;
- Construction of the RORB model using appropriately selected parameters including reach types; fraction impervious values and rainfall information;
- Calibrated the model parameters to the selected historical flood events. Given no streamflow gauge exists on Burnt Creek, it was necessary to incorporate the catchment for Bet Bet Creek (on which a gauge exists) into the model to enable preliminary calibration of the resulting hydrograph. This involved catchment delineation and incorporation in the RORB model for the Bet Bet Creek catchment upstream of the gauge and confluence with Burnt Creek. The calibration involves matching the modelled hydrograph to the observed levels at Bet Bet and reported time to peak flow at Dunolly.
- RORB model was further verified by the calibration of the hydraulic model to observed levels and extents through Dunolly for historic flood events.
- Once calibrated, the RORB model was run for the 20%, 10%, 5%, 2%, 1% and 0.5% year AEP and PMF events. Key design flows were extracted for input into the hydraulic model.
- RORB design flows were verified using a number of alternative design flow estimates.

Key input from the Steering Committee was required for the calibration of the hydrologic (and hydraulic) models, along with an independent expert review by a DEPI review panel.

3.2.1 Subarea and Reach Delineation

Given no streamflow gauge exists on Burnt Creek, it was necessary to incorporate the catchment for Bet Bet Creek (on which a gauge exists) into the model to enable preliminary calibration of the resulting hydrograph. The downstream outlet of the RORB model was therefore at the confluence of Bet Bet Creek and Burnt Creek, with the model covering the entire upstream area of both catchments (712 km^2, and 163 km^2 respectively). While the total catchment size for Bet Bet Creek is considerably larger than for Burnt Creek, preliminary calibration to this catchment is considered to be a good approximation because of the similar RORB sub-area sizes (between approximately 3 – 6 km^2 for both catchments), catchment slope and land use.

The RORB model was constructed using MiRORB (MapInfo RORB tools), RORB GUI and RORBWIN V6.15. A catchment boundary was delineated from the 20 m VicMap Elevation Digital Terrain Model (DTM) of the area. Sub-area boundaries were delineated using ARC Hydro and revised as necessary to allow flows to be extracted at the points of interest. The RORB model was delineated into 213 sub-areas. Figure 3-1 shows the RORB sub area delineation for the study area.

Nodes were placed at areas of interest (i.e. at the streamflow gauge at Bet Bet Creek and on Burnt Creek adjacent to Dunolly) and the junction of any two reaches. Nodes were then connected by RORB reaches, each representing the length, slope and reach type.

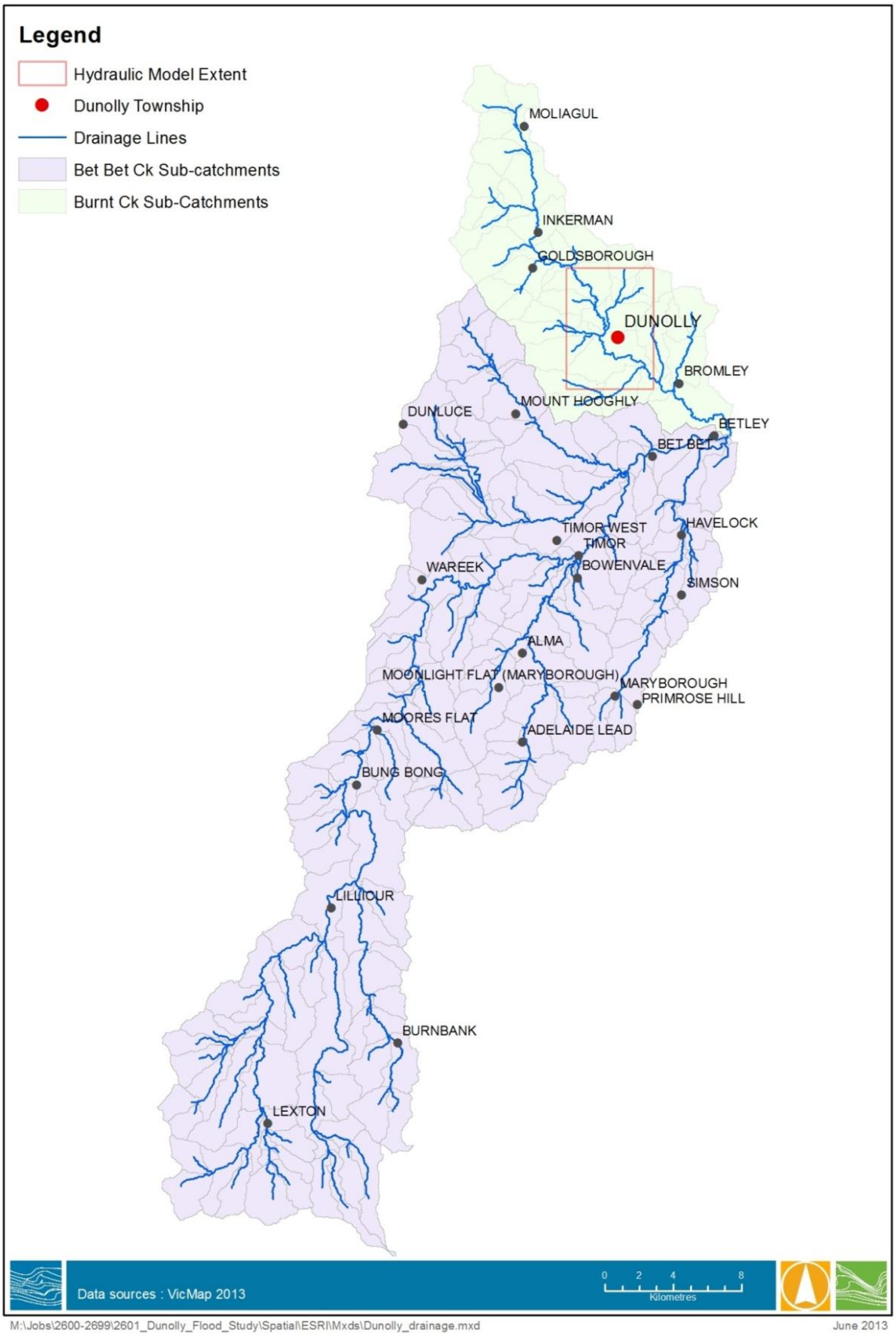


Figure 3-1 RORB Catchment Delineation (Bet Bet and Burnt Creek catchments)

Reach types were set to be consistent with land use across that catchment. Five different reach types are available in RORB (1 = natural, 2 = excavated & unlined, 3 = lined channel or pipe, 4 = drowned reach, 5 = dummy reach). All reaches were set to natural, representative of the open grassed areas and natural waterways in the catchment. Design hydrographs were extracted at the boundaries and local catchment points as shown in Figure 3-2.

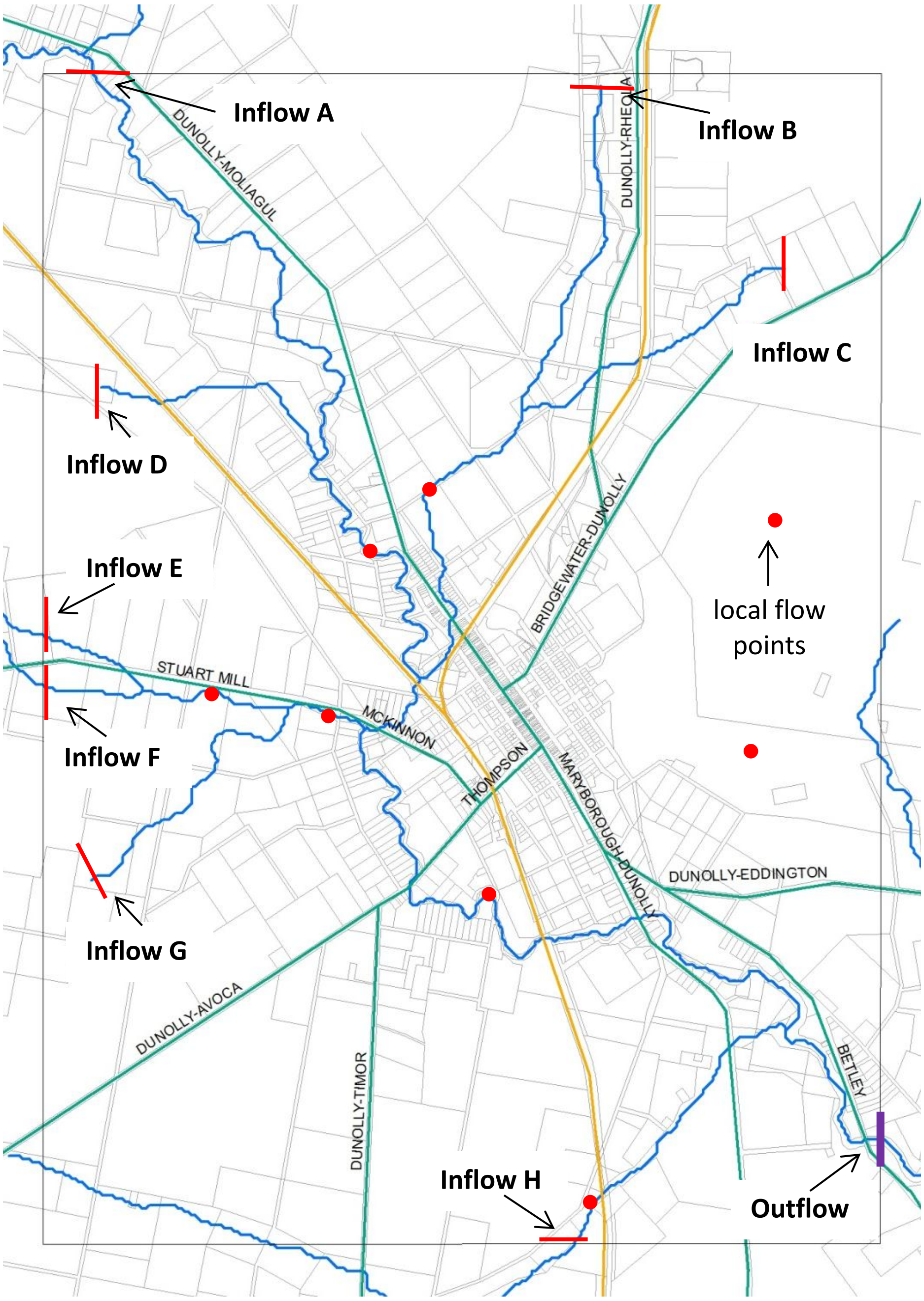


Figure 3-2 Hydraulic model boundary conditions (inflows from RORB)

3.2.2 Fraction Impervious Data

Fraction Impervious values were allocated to each of the RORB model subareas. These were an approximation of the land use based on Land Use Zoning in the area. The zones found within the catchment and the adopted fraction impervious values can be seen in Table 3-1.

A graphical representation of the sub-area delineation and the applied fraction impervious is shown in Figure 3-3. This figure shows the majority of the catchment to have a fraction impervious between 0.05 and 0.3, representative of farming and low residential rural living. The upper areas of the catchment are less pervious and more representative of denser bushland and conservation areas. At this scale, the finer details of roads and pockets of higher density residential areas are negligent.

Table 3-1 Land use zones and adopted fraction impervious[2]

Land Use Zone	Fraction Impervious
Business	0.9
Farming	0.1
Industrial	0.9
Low density residential & rural living	0.2
Medium density residential	0.45
High density residential	0.6
Mixed use	0.7
Public conservation & resource, rural conservation	0
Public park and recreation	0.1
Public use – service and utility	0.05
Public use – education, health and community, transport, local government	0.7
Public use – cemetery, other	0.6
Major roads	0.7
Minor roads	0.6
Special use	0.6
Township	0.55

[2] Melbourne Water, 2010 – Music Guidelines, Recommended input parameters and modelling approaches for MUSIC users

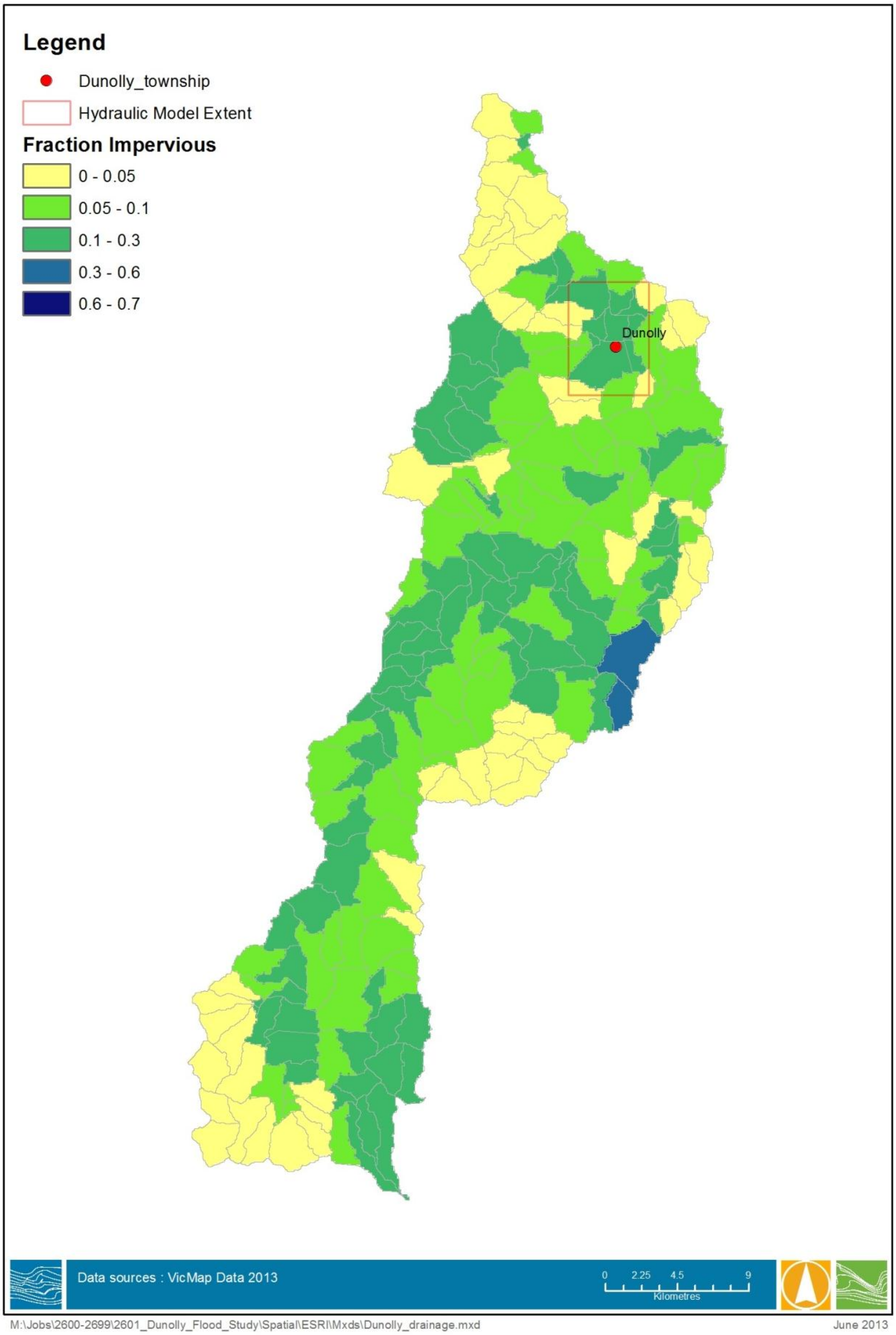


Figure 3-3 RORB Fraction Impervious

3.2.3 Storage Basins

The Old Lead Reservoir (no longer in use) is the only known storage in the Burnt Creek catchment. The reservoir has a storage capacity of only 120 ML. In our experience of similar flood studies, storages of this limited size are unlikely to have any impact on the flood peak and timing in large events. Furthermore, calculations based on preliminary hydrologic modelling for the January 2011 event indicated that the Old Lead Reservoir, if initially empty, would have filled 4.5 times during the flood event, with the initial 120 ML capacity being reached prior to the first streamflow peak on the morning of the 12th January 2011.

Feedback from the community at the drop in session (held on the 26 June 2013), indicated a strong concern regarding the management of the Old Lead Reservoir with respect to flood consequences in the town. To alleviate community concern, the reservoir was incorporated into a revised hydrologic model, and its impact analysed in more detail.

The Old Lead Reservoir is situated near the intersection of Dunolly-Rheola Road and Dunolly-Orville Road, just north of the study extent. The reservoir capacity is 120 ML. Contour channels divert runoff from the adjacent catchments to the reservoir, as indicated in Figure 3-4.

As a result of the contour channels feeding the reservoir, the sub-area catchment delineation was altered in the RORB hydrology model to ensure its full catchment was routed through the reservoir. The reservoir was incorporated, with discharge modelled by a height-storage relationship (extracted from the topography) and a Weir Formula modelling the spillway.

Figure 3-4 shows the revised sub-catchment delineation, the location of the reservoir and feeding channels with respect to the hydraulic model extent. While the RORB hydrology model extends further than depicted in Figure 3-4, no changes to the model were made outside the area shown.

Figure 3-4 also shows the location of two extraction points for hydrograph comparison. The first is directly downstream of the inflow from sub-catchments associated with the Old Lead Reservoir (i.e. the sub-catchments that have been altered from the previous hydrology model). A comparison of the resulting hydrograph for the two models can be seen in Figure 3-5. The graph indicates very minor differences in the shape and peak of the two hydrographs. Further downstream in the model, approximately 3 km downstream of the Dunolly Township, a second hydrograph was extracted, and is shown in Figure 3-6. There is negligible difference in the shape, timing and peak of the hydrographs at this location, indicating that the Old Lead Reservoir has no significant impact on flood hydrographs at Dunolly.

The model has been tested with various initial drawdown conditions in the reservoir. These initial conditions have had negligible impact on the resulting hydrographs and hence flood impacts. Note that no dambreak scenarios were modelled. The hydraulic model developed for this study could however test this scenario if it was required in the future for any planning decisions regarding the reservoir.

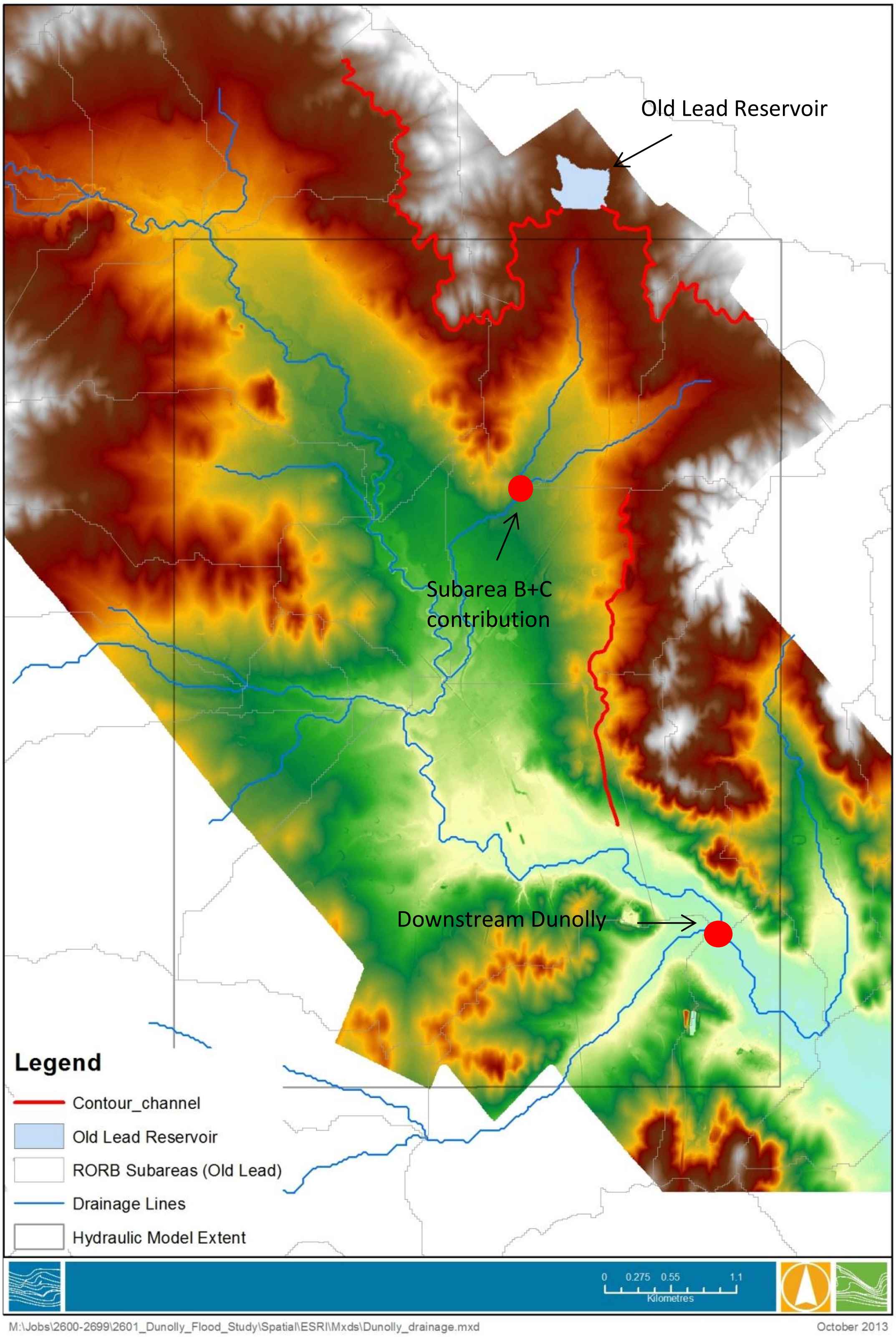


Figure 3-4 Location of Old Lead Reservoir and contour channels with respect to hydraulic study area

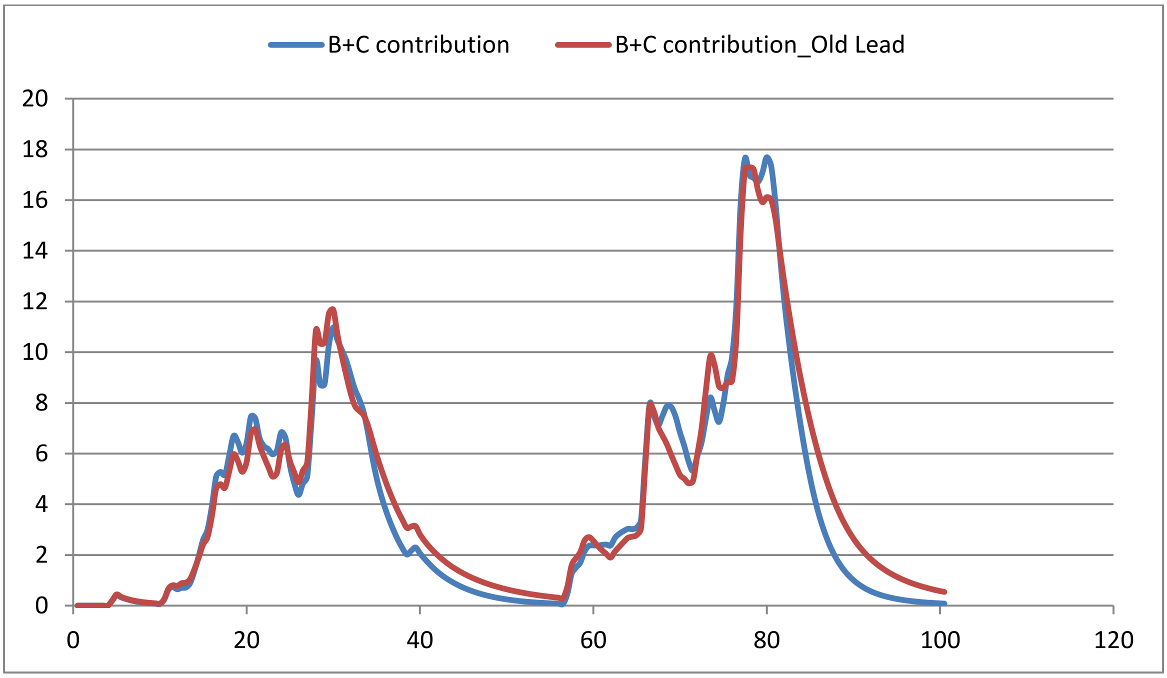


Figure 3-5 **Extracted hydrographs directly downstream of the Old Lead Reservoir for models with and without the reservoir incorporated**

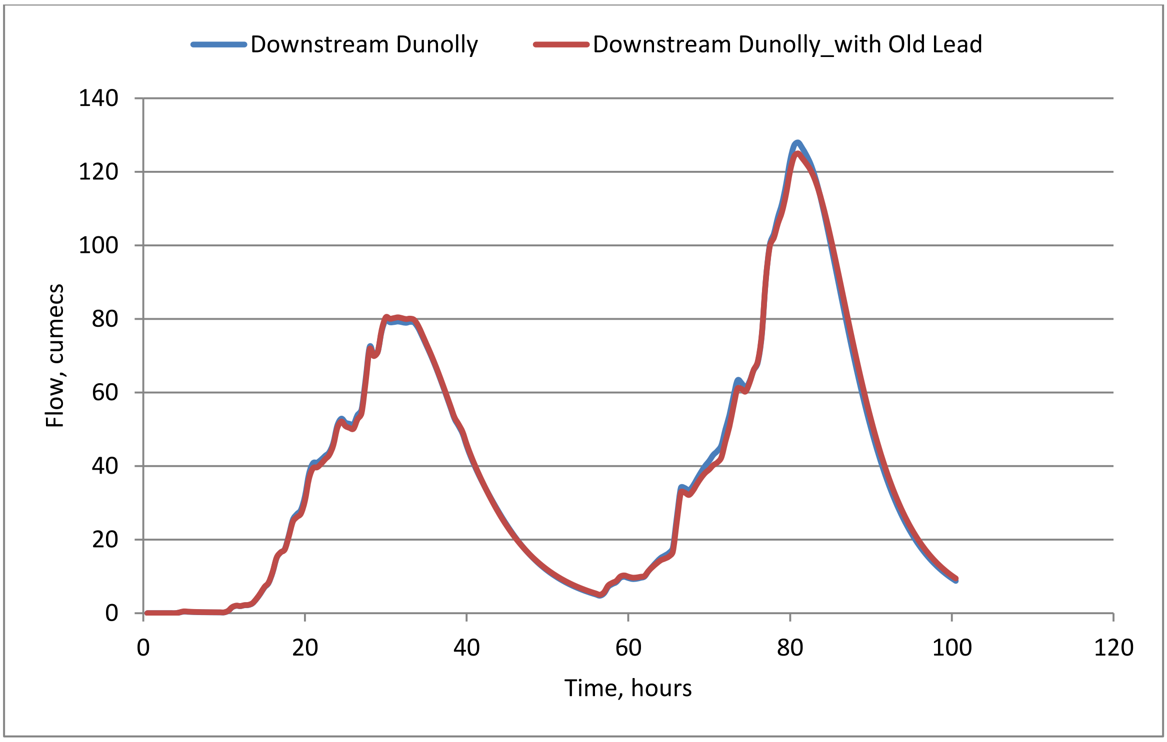


Figure 3-6 **Extracted hydrographs 3 km downstream of Dunolly for models with and without the reservoir incorporated**

3.3 Hydraulic Model

A combined 1D-2D hydraulic model was constructed.

- A one dimensional (1D) hydraulic model of key hydraulic structures and the downstream boundary of Burnt Creek; and
- A two dimensional (2D) hydraulic model of Burnt Creek, its tributaries and the broader floodplain.

The hydraulic modelling suite MIKE11 (1D), MIKE21 (2D) and MIKE FLOOD, developed by the Danish Hydraulic Institute (DHI) was applied in this study. MIKE FLOOD is a tool for floodplain modelling that combines the dynamic coupling of the one-dimensional MIKE 11 river model and MIKE 21 fully two-dimensional model system. Through coupling of these two systems it is possible to accurately represent river and floodplain processes.

3.3.1 Model Schematisation

1D Model Component

The MIKE11 model was used to control flow through all major floodplain and drainage structures on Burnt Creek and its tributaries. The details of these structures are discussed individually in the Data Review and Model Scoping Report previously completed as part of this study. There were a total of 23 structures modelled (culverts, bridges etc.). These structures were dynamically coupled with the two dimensional model.

A 1D representation of Burnt Creek downstream of the 2D model boundary was also developed. This ensured that results through the Dunolly township were not impacted by the tailwater condition and also allows a Q-H relationship to be used as a boundary condition rather than setting a constant water level boundary in the 2D model.

Hydraulic roughness within the 1D network branches is expressed through Manning's n. All structures were assigned a Manning's n value representative of their structure type and material.

2D Model Component

The 2D hydraulic model component consisted of a single model domain of Burnt Creek and the floodplain. The key items considered in schematising the 2D model were the model extent, boundary conditions, grid size and hydraulic roughness.

Model Extent

The model extent adopted covers the entirety of the Dunolly Township. The model extent is shown in Figure 3-2; it covers an area of approximately 1.5 km x 2.3 km.

Grid Size

The selection of grid size was critically important as it dictates the model's ability to represent particular floodplain features such as levees, waterways and roads. The selected grid size also dictates the model simulation times. For this study, a 5 m grid size was adopted to represent the key topographic features while allowing for reasonable model simulation time.

A 5 m grid size for the aforementioned extent yields a model with approximately 1,000 x 1,400 grid cells.

Time Step

A time step of 0.5 seconds was adopted for this study. This means that during the model simulation the model will run the full suite of hydrodynamic calculations on every active wet cell, every 0.5 seconds of model time.

Hydraulic Roughness

Variations in hydraulic roughness across the floodplain can be represented spatially as a 2D map. The hydraulic roughness (Manning's n) values for the floodplain were based on aerial photography, property parcel overlays and observations from the site inspection. Roughness categories used for the Dunolly catchment are shown in Table 3-2 and Figure 3-7. These roughness values are well within standard ranges expected for the relevant floodplain features.

Table 3-2 2D hydraulic model roughness parameters[3]

Floodplain Element	Manning's 'n' value
Local and major roads	0.02
Farm / crops / grassed areas / parks / rural living	0.04
Residential / commercial / industrial buildings	0.08
Defined waterways (creek beds)	0.03
Riparian fringe (dense vegetation)	0.05

[3] Chow, 1959 – Open Channel Hydraulics

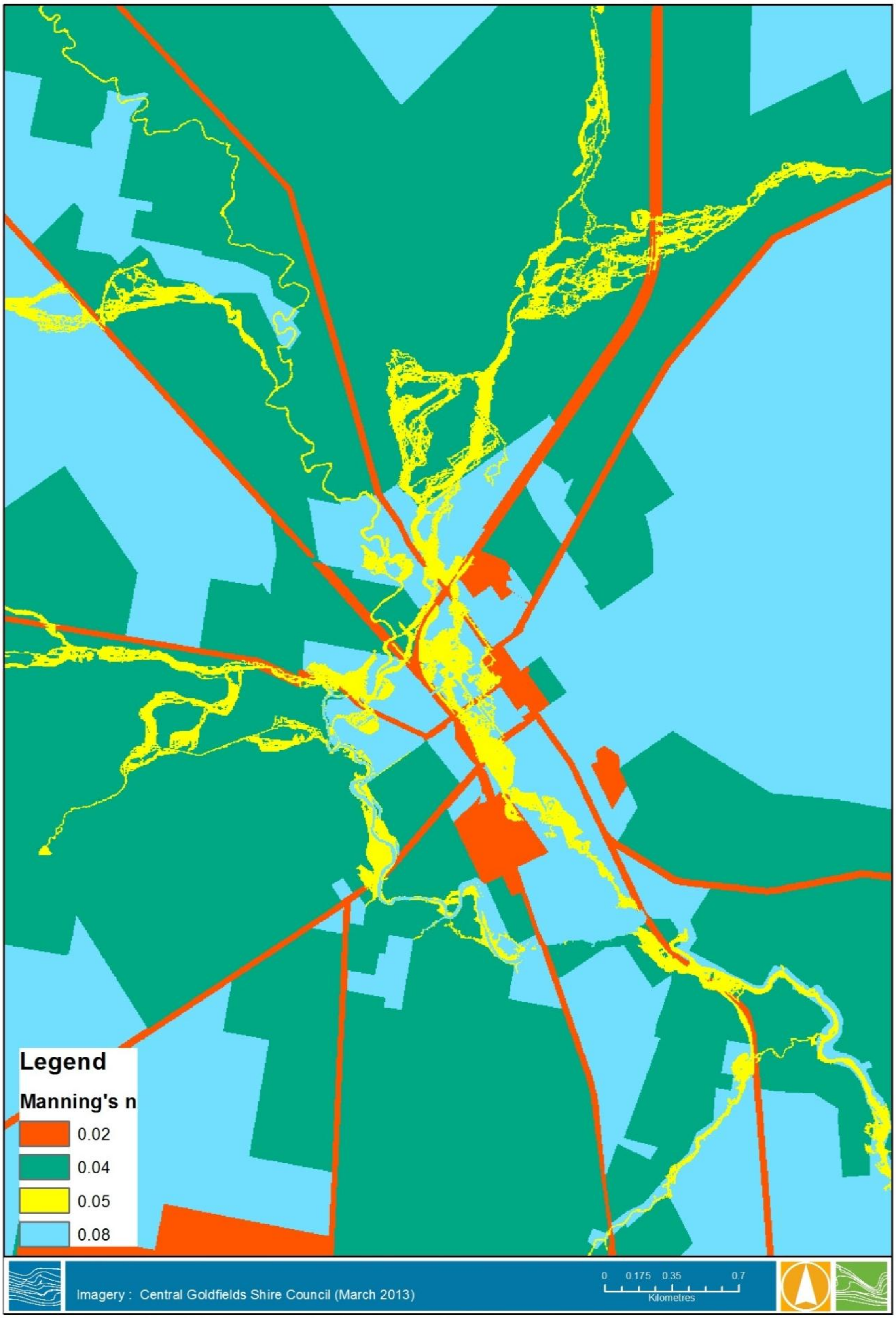


Figure 3-7 Manning's 'n' (roughness) values for the model extent

4. MODEL VERIFICATION

As there are no streamflow gauges on Burnt Creek the hydrology and hydraulic models were verified in series, with the results of the hydraulic model feeding back into revised hydrology, as an iterative process.

The predicted flows from the RORB model (preliminarily calibrated to Bet Bet Creek gauge) were used as input to the hydraulic model and the resulting extent and levels compared to those observed during the January 2011 event. An overview of the modelling approach is given in Figure 4-1. The model boundary can be seen in Figure 3-2.

Verification of flows with this methodology was an iterative process. Where significant over and/or underestimations occurred in water level and extent in the hydraulic model, the hydrological model parameters were refined. Once a close agreement was achieved in both the anecdotal timing of the peak flow and the modelled water levels and extent within Dunolly the hydraulic model parameters were then used to fine tune the model results.

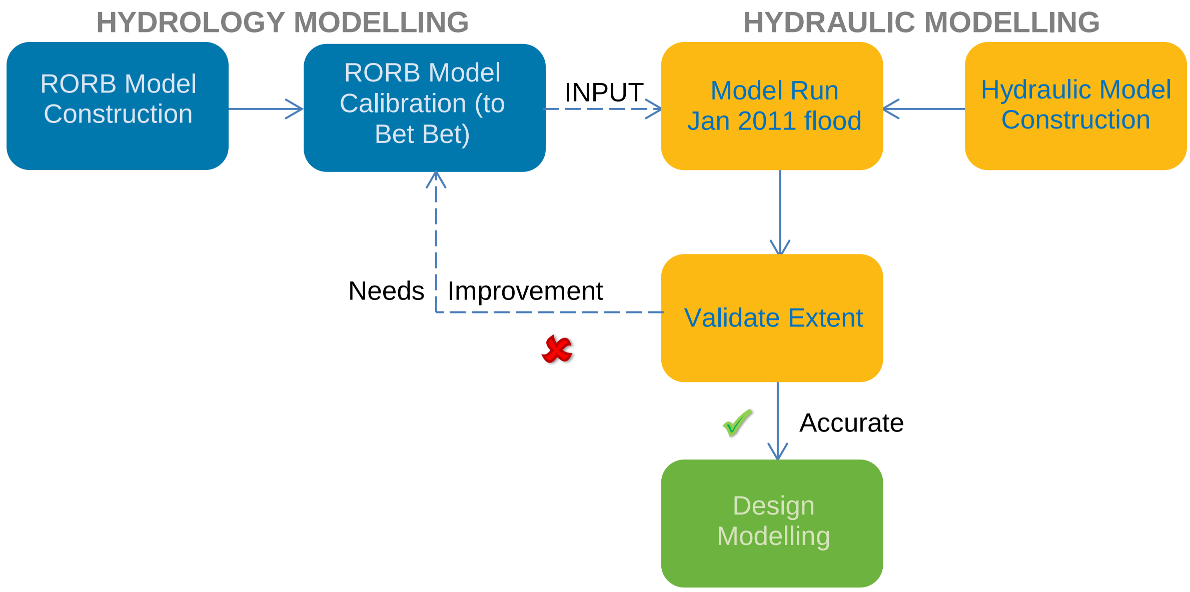


Figure 4-1 Modelling approach flow diagram

4.1 RORB Model

Due to the lack of available stream flow information on Burnt Creek the model was in the first instance calibrated to gauged flows in Bet Bet Creek for the September 2010 and January 2011 events. These same parameters were used to estimate flows through Burnt Creek in the January 2011 event and were routed through the hydraulic model to compare the predicted and observed flood heights and extents.

The focus of the RORB model verification was the determination of kc and loss values for the entire catchment.

The hydrological model was calibrated to the recent large flood event of September 2010 and January 2011. These events were chosen due to the quality of information available for rainfall and streamflow and as they were recent they therefore reflect the most up to date approximation of the current catchment conditions and behaviour.

Bet Bet Creek streamflow data, surveyed flood heights in Dunolly (January 2011 event only), ground photography (January 2011 event only) and anecdotal evidence was available for the calibration. A number of community observations of the January 2011 event aided verification of the RORB model and assisted the final hydraulic model calibration.

4.1.1 Observed Rainfall

Both pluviograph and daily rainfall records were required for the hydrological analysis. The daily rainfall gauges record the 24 hour rainfall total prior to 9am on any given day, whereas pluviograph/tipping bucket rainfall gauges record rainfall on a continuous basis, measuring the rainfall intensity and pattern across the event.

The pluviograph rainfall data was used to define the temporal distribution of rainfall during an event while daily rainfall data provided an understanding of the spatial variation. Figure 2-3 shows the locations of daily rainfall and pluviograph stations in the region.

Pluviograph records were available at Avoca (408206), Avon Wimmera Highway (415220), Bet Bet (407211), Laanecoorie Reservoir (407240), Lillicur (407222), Tullaroop Creek (407222) and Wimmera Eversley (415207) gauges. Daily rainfall records are available for a number of stations spread out across the catchment; these are listed in Table 2-1.

Temporal Distribution of Rainfall

Temporal patterns can only be developed from instantaneous rainfall data, of which there were seven potential gauges (pluviograph or tipping bucket data) available for use. Two of these gauges were adopted for use to develop temporal patterns (Lillicur and Bet Bet) as these were the only gauges within the catchment.

Sub-areas were assigned the temporal pattern of the nearest of these two gauges. While the northern sub-areas of the Bet Bet catchment are closer to the Avoca River (at Archdale Junction) gauge than to the Bet Bet gauge, rainfall experienced at this station was found to be sufficiently different from gauges within the catchment, and therefore the use of this gauge in assigning temporal patterns was not adopted.

September 2010

The September 2010 flood event was as a result of heavy rainfall early in the month, with maximum intensities recorded in the early hours of the morning on the 4th September. The rainfall observed correlates roughly to between a 20% and 10% AEP rainfall event using the ARR1987 intensity-frequency-duration curves available on the Bureau of Meteorology website (early June 2013).

Rainfall over this period observed at the Bet Bet and Lillicur gauges can be seen in Figure 4-2 and Figure 4-3. Rainfall observed at the Lillicur gauge had a less intense peak.

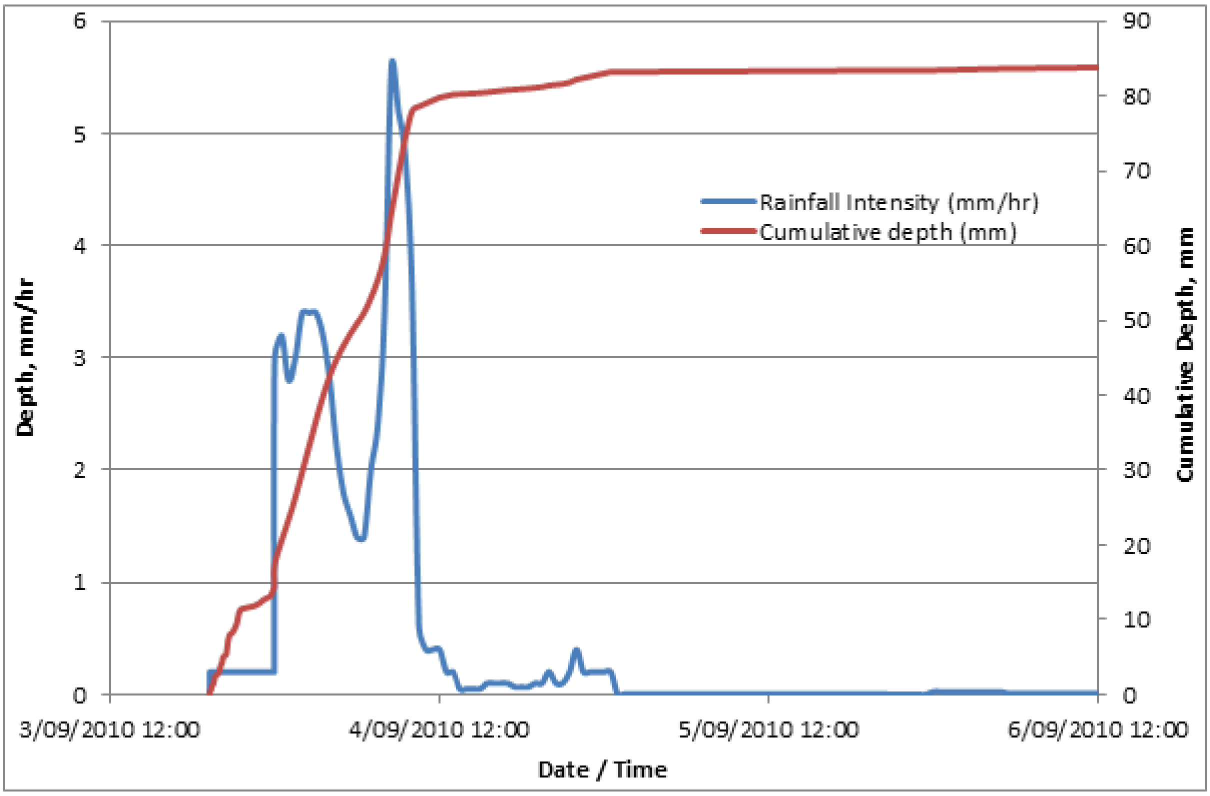


Figure 4-2 Temporal rainfall distribution at the Bet Bet gauge for September 2010 flood event

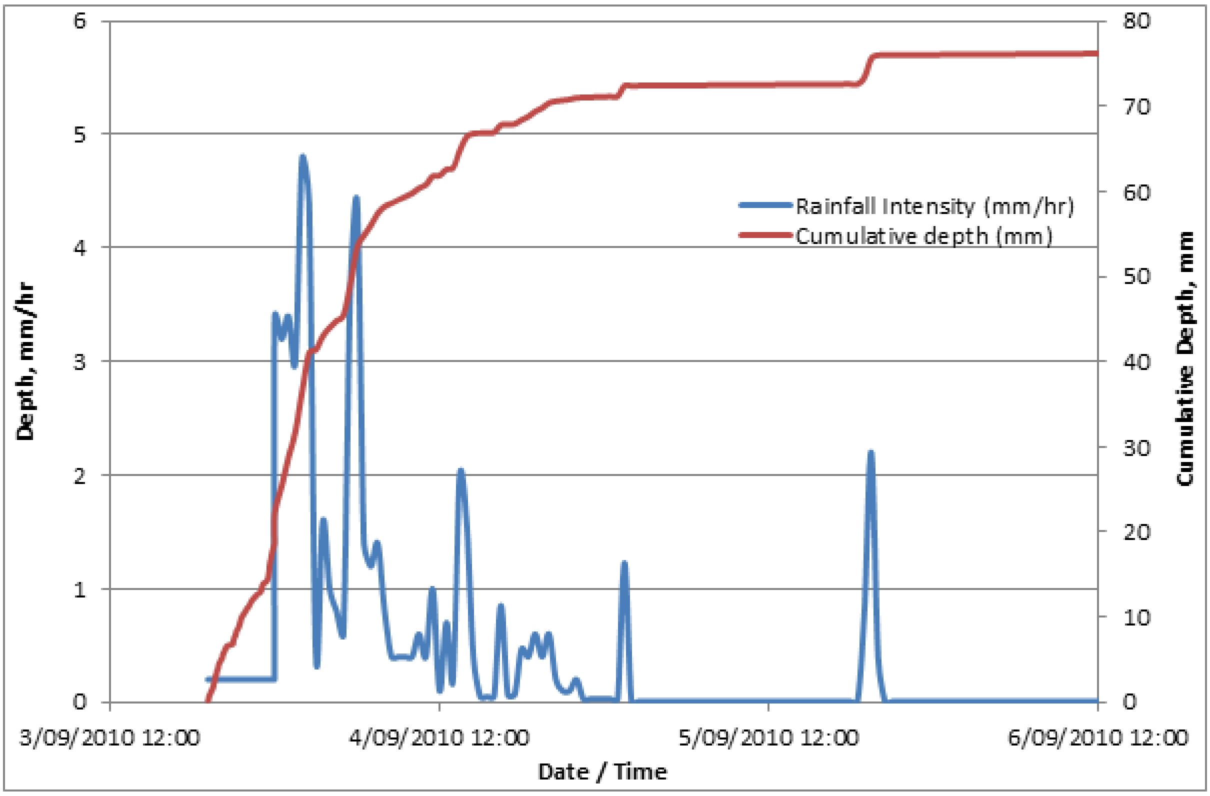


Figure 4-3 Temporal rainfall distribution at the Lillicur gauge for September 2010 flood event

January 2011

The January 2011 event was defined in three separate bursts, the first on the morning of the 10th January, lasting only a couple of hours. The second and third bursts, beginning on the 11th and 13th January respectively were more prolonged, with rainfall occurring over periods of approximately 36 and 24 hours respectively. Over 200 mm of rainfall fell over the total 5 day period, with maximum daily rainfall totals exceeding 90 mm.

Anecdotal evidence from Dunolly SES indicates that maximum water levels occurred at 9:30 am on the 14th January, approximately 12 hours after the onset of the third rainfall burst.

The temporal distribution of rainfall for the two pluviograph stations used for this study can be seen in Figure 4-4 and Figure 4-5.

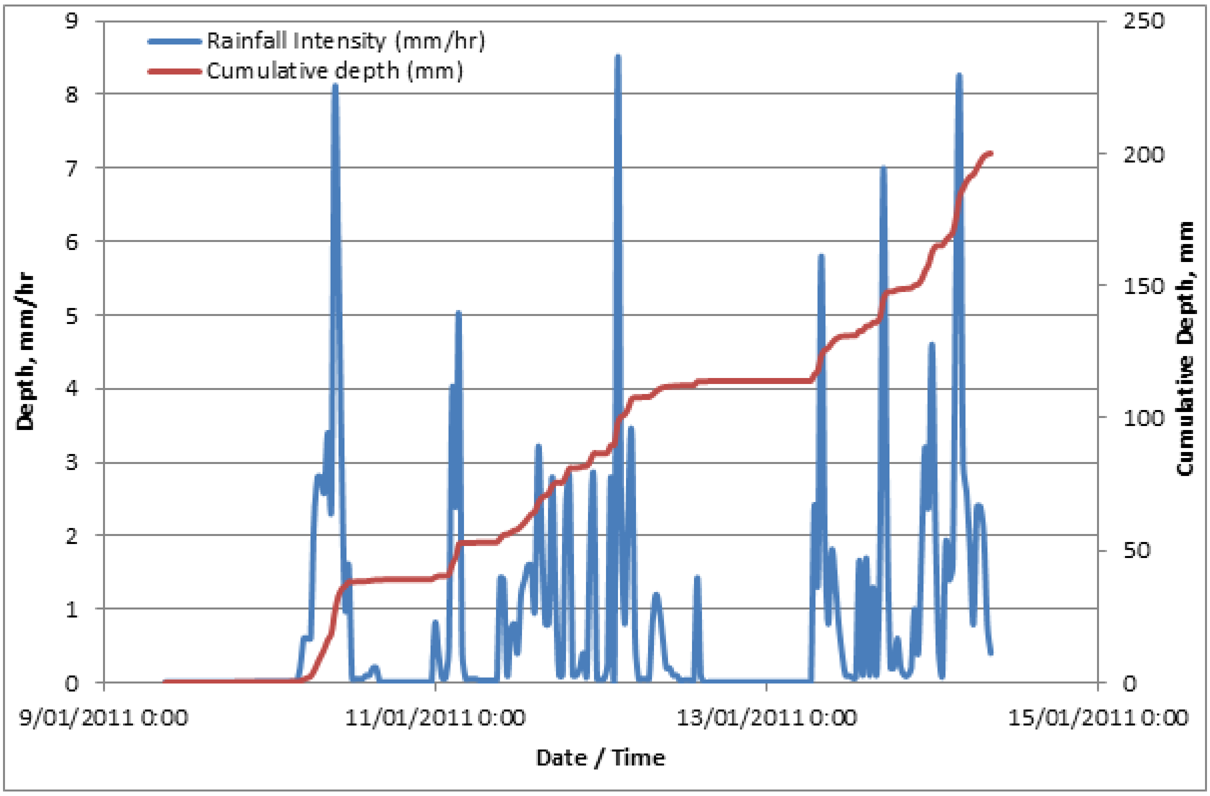


Figure 4-4 Temporal rainfall distribution at the Bet Bet gauge for January 2011 flood event

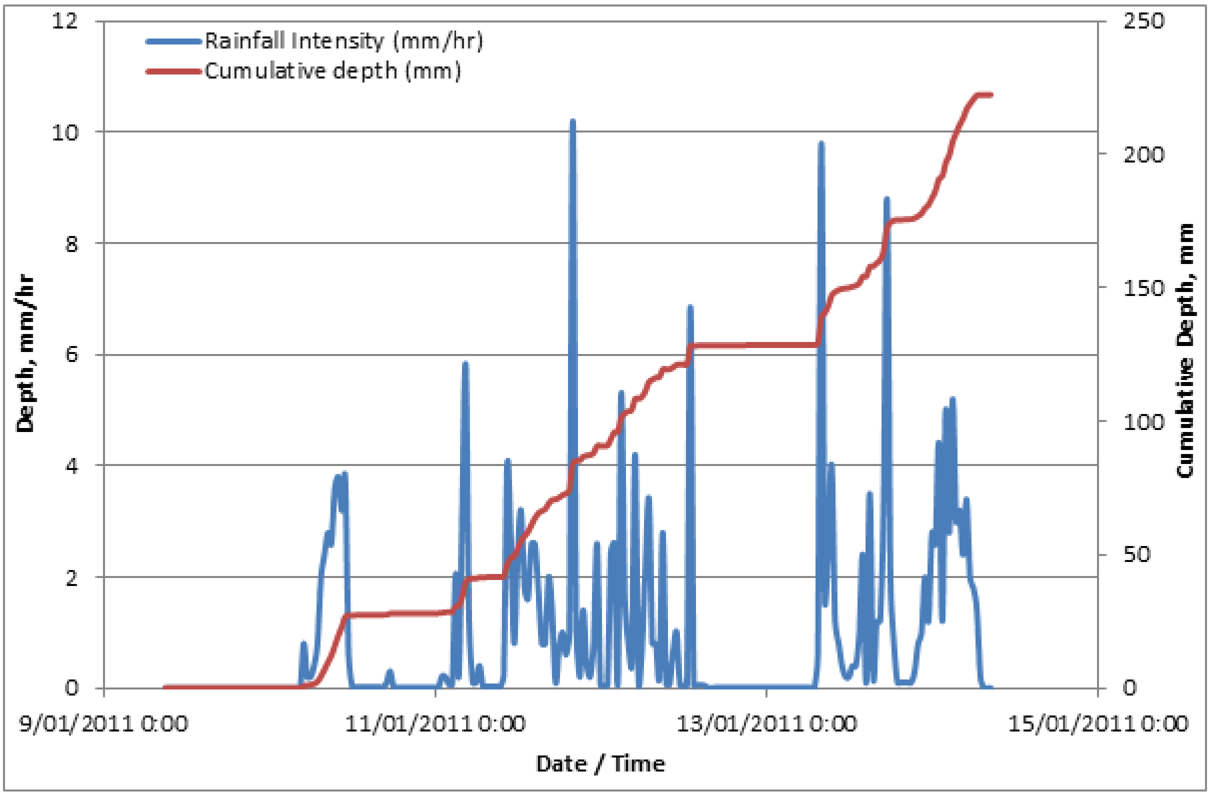


Figure 4-5 Temporal rainfall distribution at the Lillicur gauge for January 2011 flood event

Spatial Distribution of Rainfall

To determine the spatial distribution of rainfall for the verification events, the rainfall totals from each daily rainfall gauge, as shown in Figure 2-3 and Table 2-1 was used to create a Triangulated Irregular Network (TIN). The TIN provides an estimate of the spatial variation in depth of rainfall covering the entire catchment during the two events. The triangulated rainfall values are shown in Figure 4-6 and Figure 4-7 respectively.

There is more spatial variability in the January 2011 rainfall event, particularly around the eastern reaches of the Bet Bet Creek catchment. Total rainfall depths for the January 2011 event were slightly higher in the upper reaches of the Bet Bet Creek catchment as compared to the Burnt Creek catchment.

While there is less variability for the September 2010 rainfall event, rainfall still varies by 15 mm across the Burnt Creek catchment. Rainfall was highest on the eastern side of the catchments.

From the TIN of rainfall depths, a total depth for the event is determined for each RORB subarea. This depth is then distributed over the duration of the event according to the temporal distribution recorded at the chosen pluviograph rainfall station.

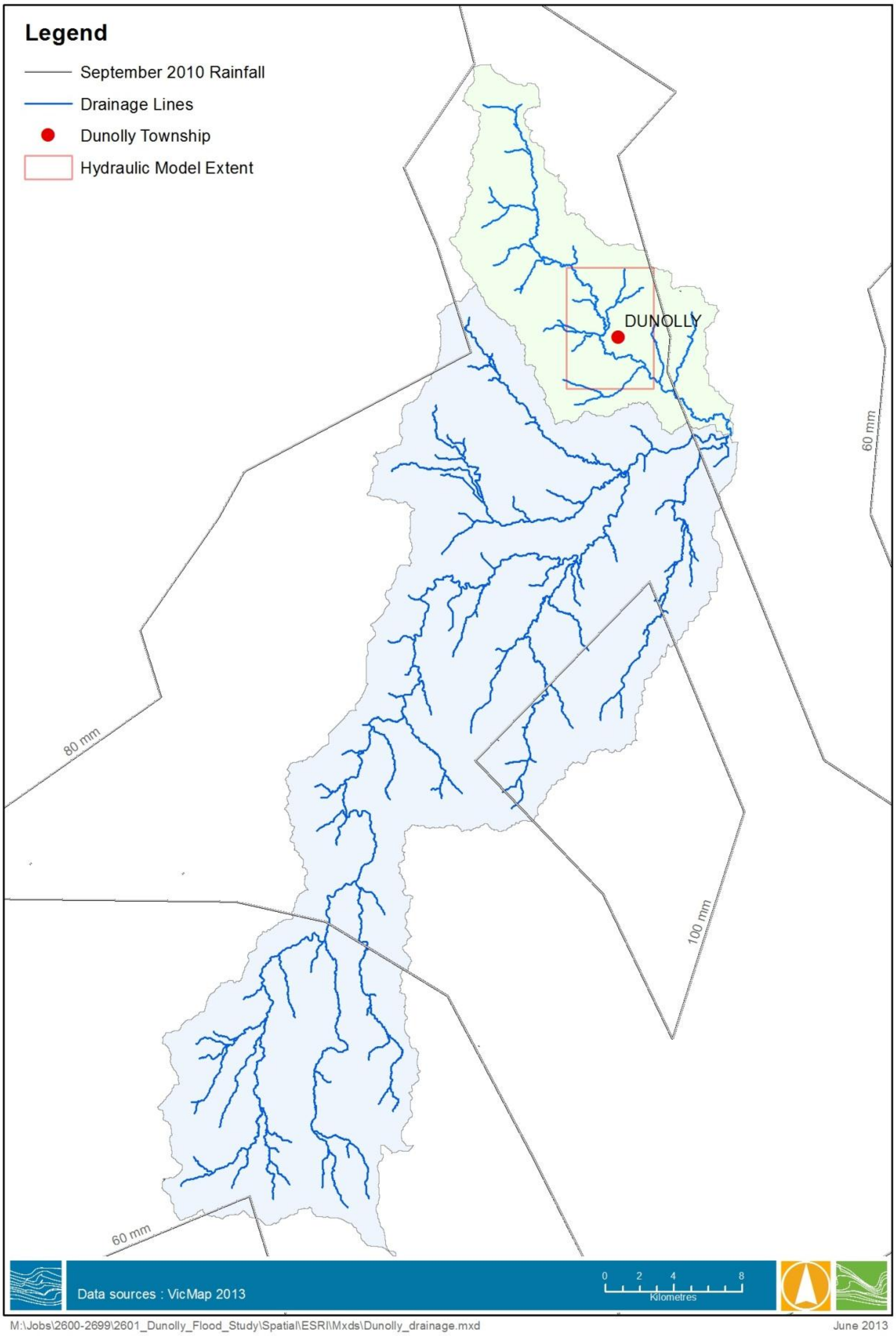


Figure 4-6 September 2010 triangulated rainfall totals

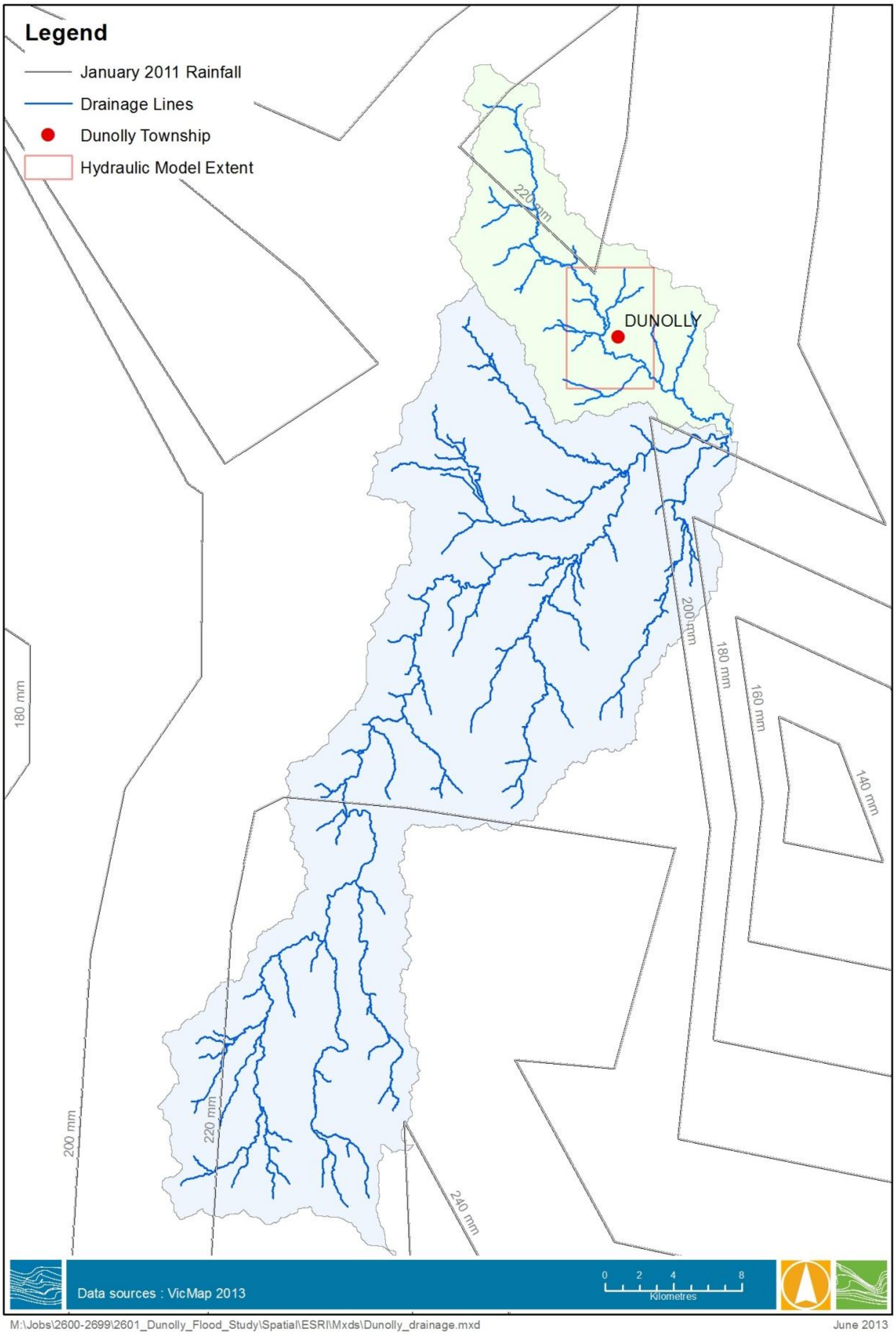


Figure 4-7 January 2011 triangulated rainfall totals

4.1.2 RORB Model Verification Parameters

There are several model parameters used in RORB that control the resulting peak flow rate and volume of runoff – kc, m and initial and continuing losses (IL and CL). These parameters can be adjusted to fit to observed information.

Losses

The loss model chosen for the two catchments (Bet Bet and Burnt Creeks) was the initial and continuing loss model. This model has been chosen because both catchments are predominately rural. The catchments are likely to have high rainfall infiltration at the beginning of an event when the ground is dry, which will then reduce to a constant loss rate over the remainder of the event.

m

The RORB m value is typically set at 0.80. This value remains unchanged and is an acceptable value for the degree of non-linearity of catchment response (Australian Rainfall and Runoff, 1987)[4]. There are alternative methods for determining m, such as Weeks (1980)[5], which uses multiple calibration events to select k_c and m. However, given the absence of streamflow data, a change to the m value from 0.8 would be difficult to justify.

kc

A range of methods for predicting kc are available, some of which are built into RORB. In this case, the kc value was initially estimated using the Dyer (1994) method (Pearse et al 2002). This method assumes the Kc value to be a function of the average flow distance in the channel network of sub area inflows. The initial Kc value was used as a starting point for selecting the model parameters to optimise the fit between the resulting and actual hydrographs at Bet Bet Creek for the September 2010 flood event.

4.1.3 Verification Results

Streamflow data at Bet Bet was not available for the January 2011 event, and as such, the RORB model was preliminarily calibrated to the September 2010 event only. Fifteen sets of model parameters were used in the calibration, as outlined in Table 4-1, with the final parameters highlighted. The calculated and actual hydrograph for the September 2010 event for the finalised calibration parameters can be seen in Figure 4-8.

[4] AR&R, 1987 – Australian Rainfall and Runoff
[5] Weeks, W.D. (1980). Using the Laurenson model: traps for young players. Hydrology and Water Resources Symposium, Adelaide, Institution of Engineers Australia

Table 4-1 RORB model parameters for calibration with data from Bet Bet Creek @ Bet Bet gauge

Kc	IL	CL	Q peak	Error	Time, h	Error
45	16	1.5	360.5	5.6	14	-3
40	16	1.5	406.6	51.7	13	-4
50	16	1.5	321.9	-33	14.5	-2.5
45	20	2	307.2	-47.7	14	-3
50	20	2	273	-81.9	15	-2
48	20	2	273	-81.9	15	-2
39	20	2	357.2	2.3	13.5	-3.5
47	15	1.5	350.3	-4.6	14	-3
46	16	1.5	352.4	-2.5	14	-3
50	16	1	354.5	-0.4	14.5	-2.5
48	16	1.2	355.9	1	14	-3
49	**16**	**1.1**	**355.1**	**0.2**	**14.5**	**-2.5**
48	18	1	358.8	3.9	14.5	-2.5
49	18	1	351.1	-3.8	14.5	-2.5
52	10	1.3	351.9	-3	14.5	-2.5

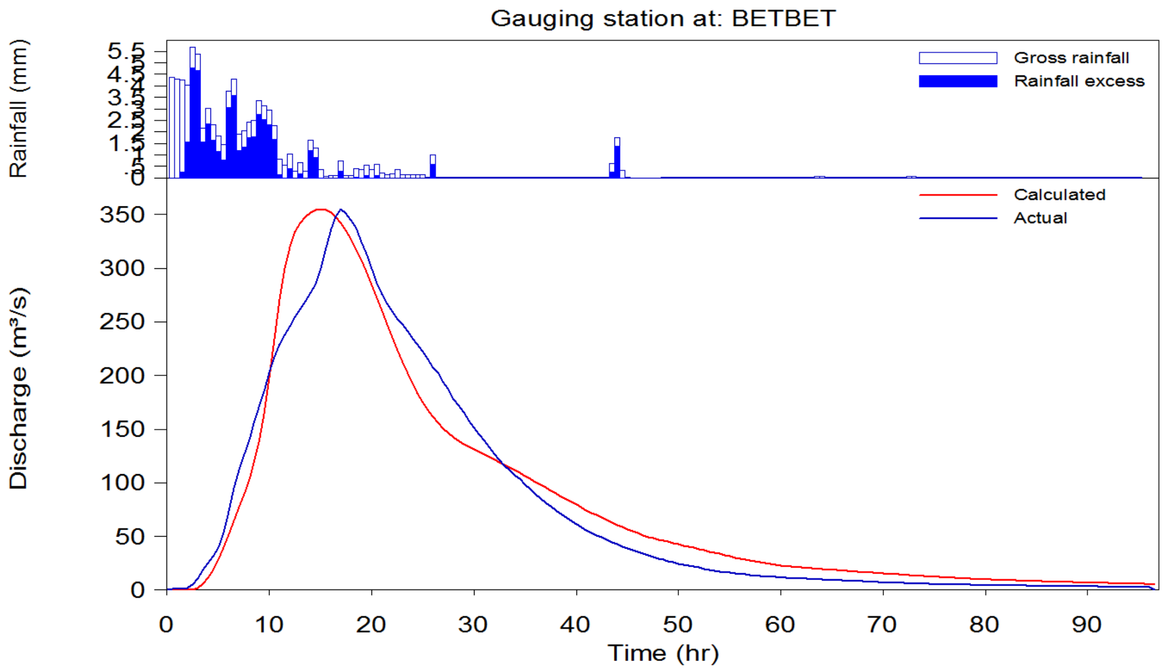


Figure 4-8 September 2010 RORB calibration results

Based on the loss parameters used for the calibration of the September 2010 flood event, the losses outlined in Table 4-2 were adopted as a first pass to generate the hydrographs for Burnt Creek as input to the hydraulic model.

Note that the initial burst was combined with the first in the development of the hydrographs.

Table 4-2 RORB model calibration parameters – January 2011 event

January 2011	Kc	m	Burst 1+2		Burst 3	
			IL	CL	IL	CL
	49	0.8	16	1.1	10	1.1

4.2 Hydraulic Model

A preliminary modelled flood extent for the January 2011 event was presented at the Steering Committee Meeting (held on 21 May 2013) for discussion. The extent generally aligned well along Burnt Creek with survey of the flood peaks provided by North Central CMA. The time of the peak water levels from the model simulation was at approximately 8am on 14th January 2011. This result aligns well with the anecdotal 9:30am time of the peak water levels in Dunolly provided by local VICSES volunteers.

The modelled extent for the January 2011 event can be seen in Figure 4-10.

Discussion with Steering Committee Members provided confidence that the extent produced as a result of flooding from Burnt Creek was relatively accurate, but that the extent produced by local flows to the east of the township required further refinement.

It was highlighted that a contour channel, which runs approximately north to south along the eastern side of the township, prevented much of the local flows from flooding the town. This contour channel was not adequately represented in the model topography, and hence adjustments were made to incorporate this. A revised model extent can be seen in Figure 4-11.

These modelled flood results were closely aligned with the survey data, with differences between surveyed and modelled flood levels in the order of 20 cm. The differences are summarised in Table 4-3 and can be seen in Figure 4-9. Although the results showed that the model generally over predicted water level, it is unknown due to the ungauged nature of the catchment if the error lies in the hydrology or the hydraulics, and it was felt that within 20 cm was a good result. The calibration flows and hydraulic model results were adopted and the models deemed appropriate for consideration of design events.

The final calibrated January 2011 modelled flood extent showed large areas of the township of Dunolly impacted by local runoff, with flows spilling over the contour channel. The modelled depths are generally lower than 100 mm through most of this area and in reality are probably dealt with through roadside drainage and small swale drains etc. Although there is little evidence for this shallow depth of flooding through properties, residents did report water in the streets. There is also evidence in some sections of the contour channel of water overtopping and eroding some sections of the bank, particularly toward the northern end, in the area where local runoff flowed around the primary school and sporting ovals. Further reports of a significant amount of water flowing from the Caravan Park lake through the currently vacant block upstream and over Clark Street and Watt Road, before entering Burnt Creek near the corner of Gooseberry Hill Road and Maryborough-Dunolly Road were observed in the hydraulic model. There are areas along this flow path where water banks up behind roads with little or no culvert capacity to significant depths. Note that this flow path is volume dependant and will be of significance in long duration high volume storm events like January

2011. The area of recent developments north of the railway and Dunolly Road, near Raglan and Cardigan Streets is impacted by water backing up from Burnt Creek directly as well as from water flowing overland from the Old Lead Reservoir and another northern flow path, as well as breakout flows from Burnt Creek from further upstream near the cemetery. Residents reported very close agreements between model results and observations in this location.

Table 4-3 Surveyed and modelled flood elevations for the January 2011 event

Survey Peg ID	Survey elevation (m AHD)	Modelled flood elevation (m)	Difference (m)
DunT0001	195.85	196.08	+0.23
DunT0002	195.58	195.82	+0.24
DunF0003	193.58	193.90	+0.32
DunT0004	193.79	193.82	+0.03
DunF0007	190.80	191.00	+0.20
DunT0006	191.71	191.62	-0.09
DunT0005	188.94	189.14	+0.20
DunF0007	187.75	187.93	+0.18
		AVERAGE DIFFERENCE	**+0.16**

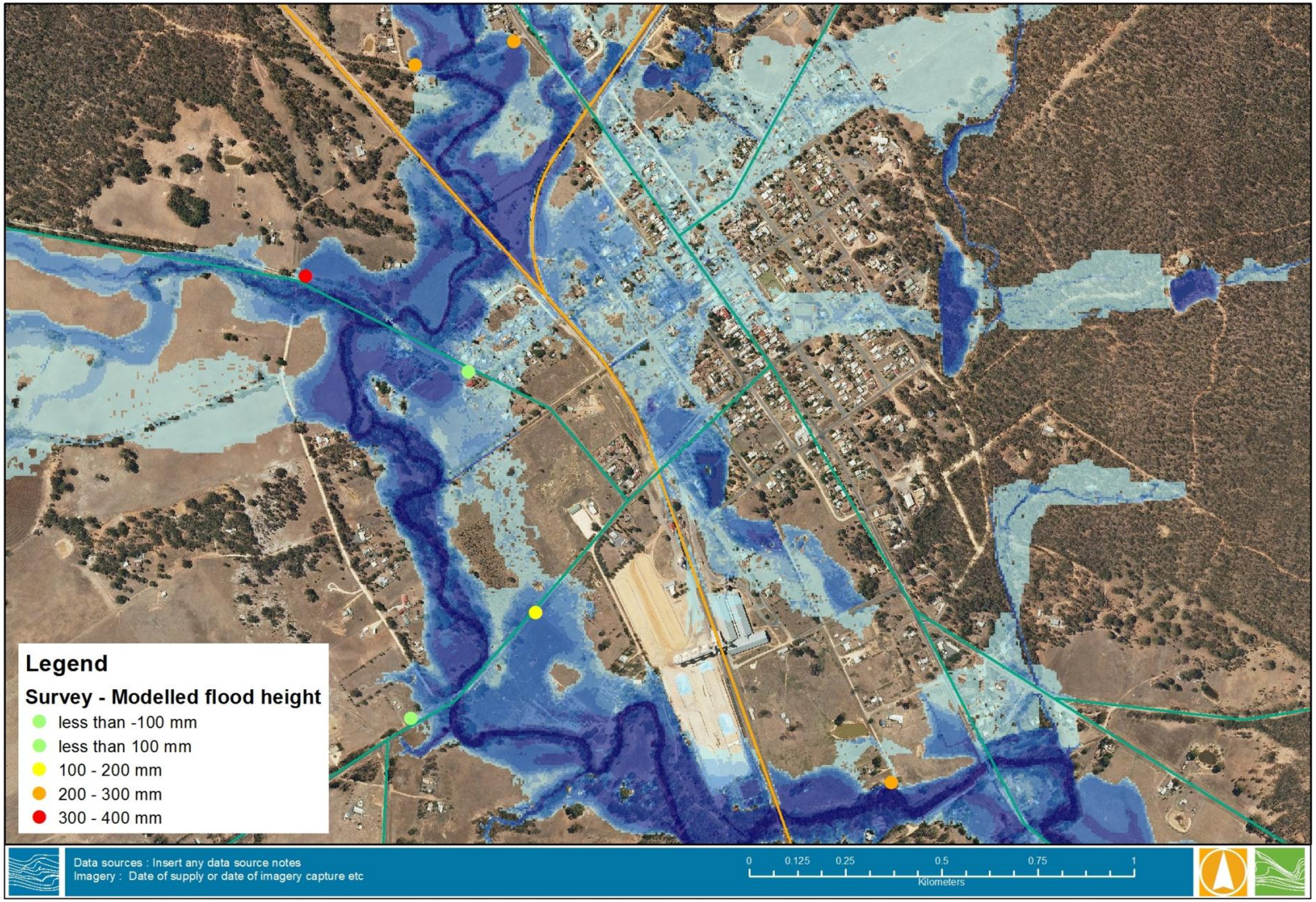


Figure 4-9 Difference in surveyed and modelled flood heights

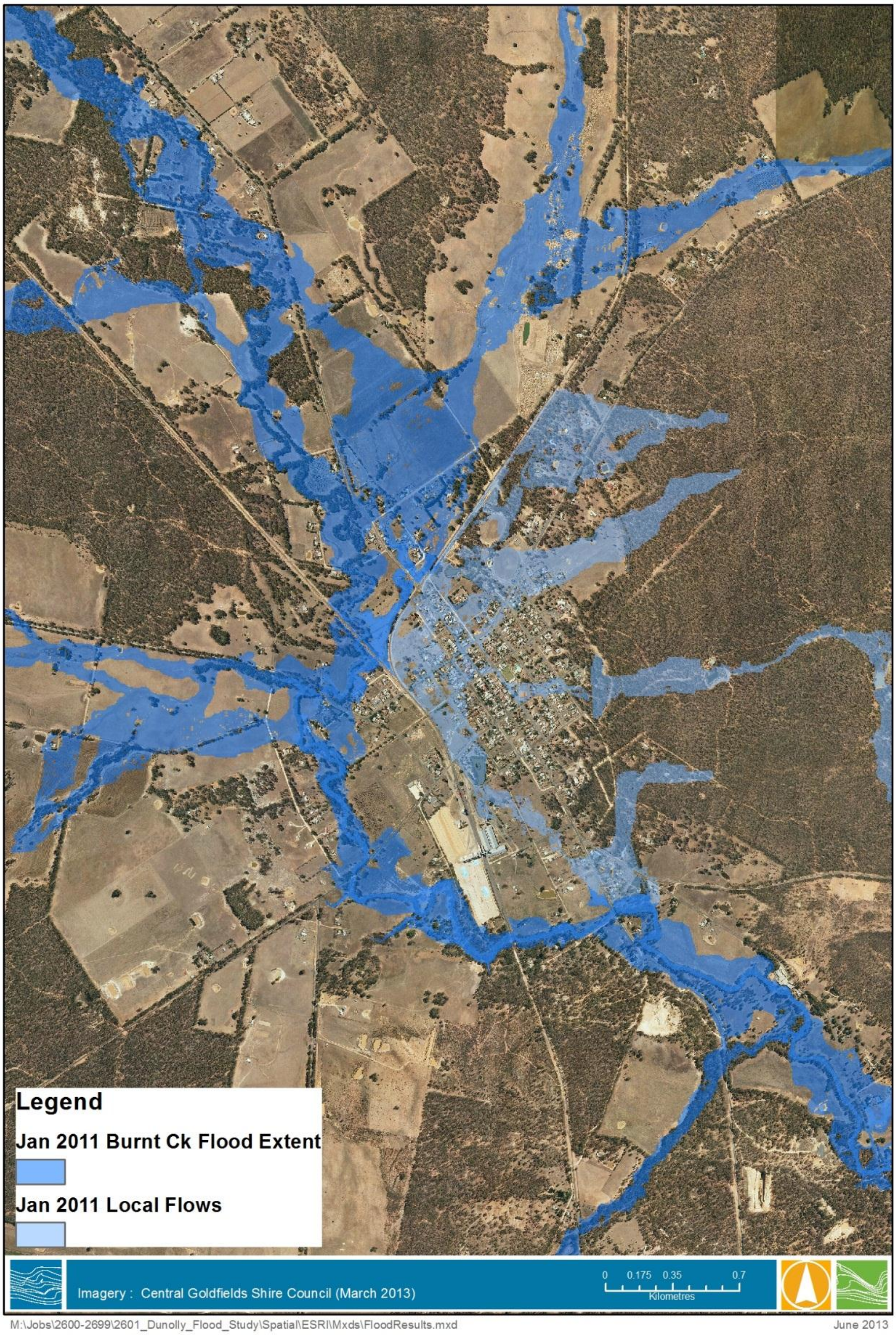


Figure 4-10 Preliminary modelled flood extent for January 2011 flood event

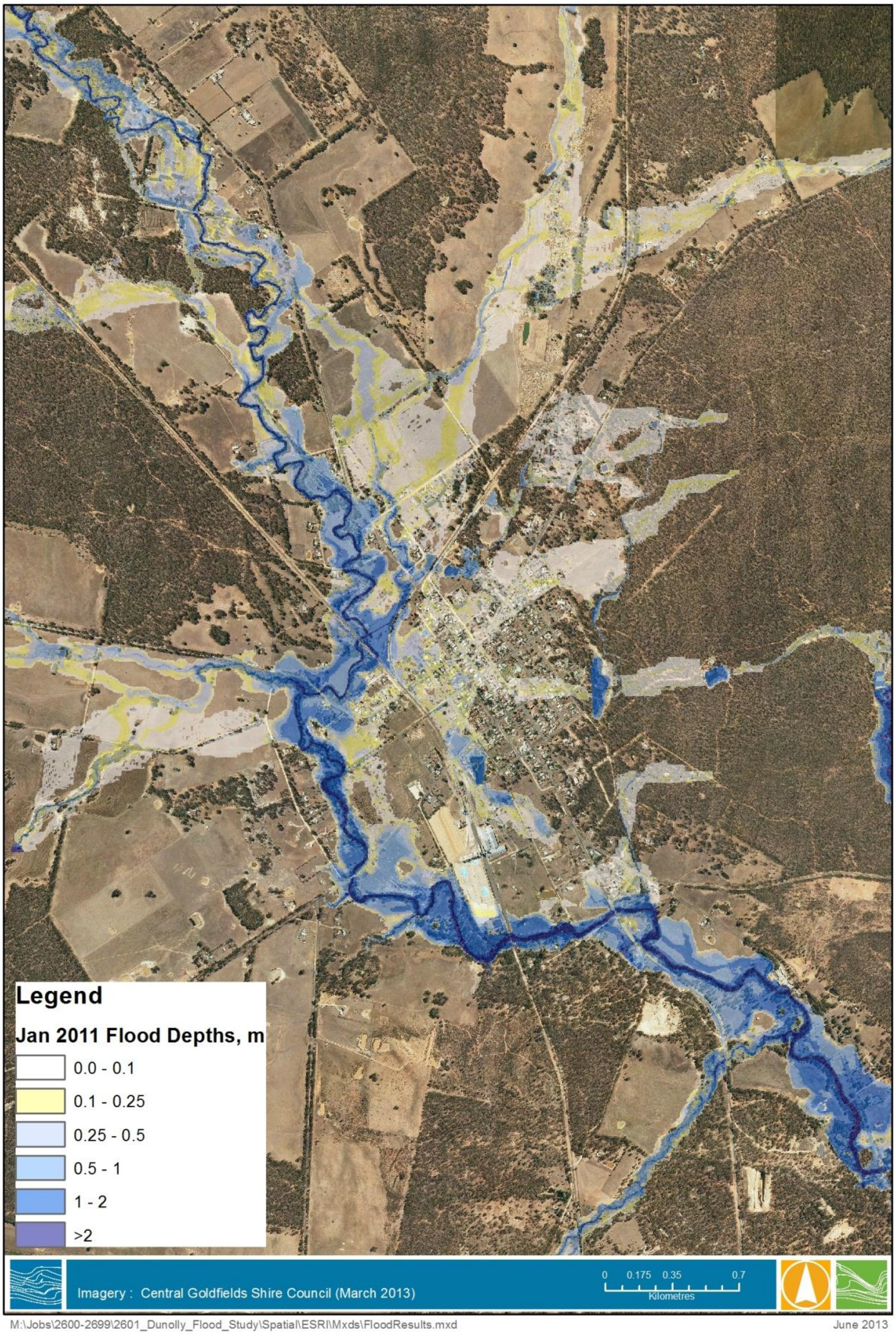


Figure 4-11 Revised flood extent for January 2011 event

5. DESIGN EVENT MODELLING

5.1 Overview

Following on from the successful RORB model and hydraulic model verification, a series of design events were modelled. This required the adoption of various design parameters to be included within RORB to generate design hydrographs for input to the hydraulic model.

This section presents the design parameter selection and subsequent flows generated within RORB and the hydraulic model results.

For this study the 20%, 10%, 5%, 2%, 1% and 0.5% AEP events were required. The inputs for design flood estimation are described below.

5.2 RORB Design Methodology

5.2.1 Design Rainfall Depths

Design rainfall depths were determined using the BoM online IFD Tool[6]. The ARR87 IFD parameters were generated for a location north of Dunolly, at the centroid of the Burnt Creek catchment upstream of Dunolly (-36.825 S, 143.675 E) and are shown in Table 5-1 below. The resulting IFD curves can be seen in Figure 5-1.

Table 5-1 Catchment IFD parameters for Burnt Creek

$2I_1$ (mm/hr)	$2I_{12}$ (mm/hr)	$2I_{72}$ (mm/hr)	$50I_1$ (mm/hr)	$50I_{12}$ (mm/hr)	$50I_{72}$ (mm/hr)	G	F2	F50	Zone
18.6	3.43	0.89	41.30	7.29	1.90	0.20	4.35	14.92	2

The approximate rainfall intensities and storm durations for the September 2010 and January 2011 flood events are plotted on the IFD curve below. Their placement suggests that the September 2010 event was between a 20% and 10% AEP rainfall event, while the January 2011 event was closer to a 2% AEP rainfall event.

[6] BoM Online IFD Tool - http://www.bom.gov.au/hydro/has/cdirswebx/cdirswebx.shtml Accessed: June 2013

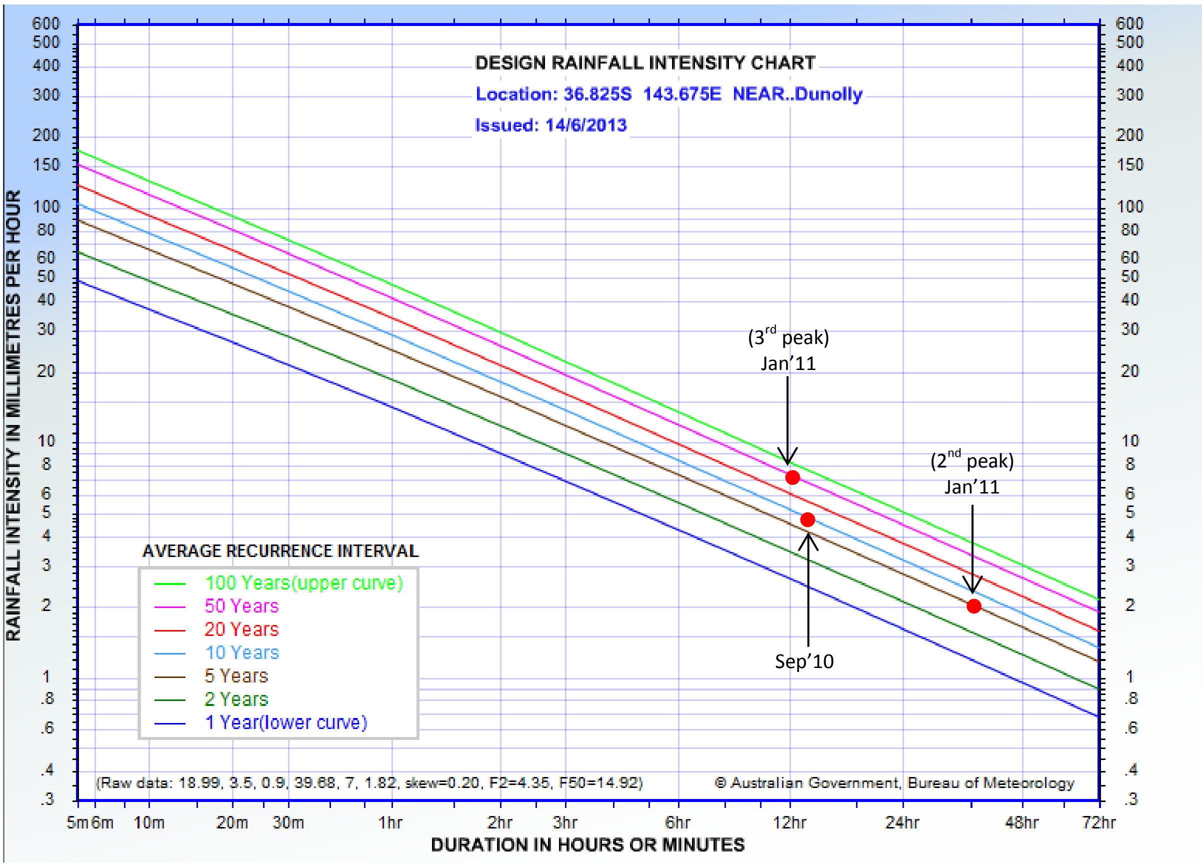


Figure 5-1 IFD curves extracted from the BoM online IFD tool for Dunolly catchment[6]

5.2.2 Design Temporal Pattern

Design temporal patterns were taken from the Generalised Short Duration Method (GSDM) and Generalised South East Australian Method (GSAM) as well as AR&R (1987) in order to understand the sensitivity of the flood estimates to the temporal pattern. GSDM patterns were used for durations up to and including 3 hours and unsmoothed GSAM patterns for durations greater than 24 hours.

The Burnt Creek catchment is located within Zone 2 of the temporal pattern map as defined in AR&R (1987). The temporal pattern described in AR&R (1987) was applied to all duration events.

The AR&R (1987) temporal pattern resulted in a critical duration of 18 hours. Given the magnitude of the critical storm duration, the shorter duration events are not critical and therefore the GSDM (which applies for durations less than 3 hours) was not investigated further.

The 24 hour duration storm was selected for comparison between the GSAM and AR&R (1987) temporal patterns. The GSAM pattern resulted in lower peak flows for both the 24 hour and critical duration (18 hour) storm. A comparison of peak flow results can be seen in Table 5-2.

Table 5-2 GSAM and ARR (1987) temporal pattern comparison for 24 hour duration storm

	Peak Flow, m^3/s		
ARI, years	**24hr, GSAM**	**24hr, AR&R (1987)**	**18hr, AR&R (1987)**
5	12.77	18.626	19.601
10	28.42	36.921	34.334
20	53.22	62.096	64.102
50	90.33	98.963	105.660
100	119	135.587	145.040
200	151.8	175.801	187.594
500	195.9	235.878	248.853

Given that the GSAM temporal pattern didn't result in larger peak flows than those produced for the same duration using the ARR (1987) Zone 2 temporal patterns, the ARR (1987) Zone 2 temporal patterns were adopted for design estimation as a conservative measure.

5.2.3 Design Spatial Pattern

A uniform spatial rainfall pattern (i.e. the same rainfall depths applied to the entire catchment) was adopted for the generation of design flood hydrographs. This is thought to be a reasonable assumption because:

- The Burnt Creek catchment size is small;
- Only 5-15 mm variability in rainfall was experienced over the Burnt Creek catchment for both the January 2011 and September 2010 events; and
- There are no large topographical features which would lead to orographic rainfall variations.

5.2.4 Areal Reduction Factors

Areal reduction factors convert point rainfall to areal estimates and are used to account for the variation of rainfall intensities over a large catchment. Siriwardena and Weinmann (1996) reduction factors were applied to the catchment area of 163.3 km^2. It is understood that revised areal reduction factors are being released in the revision of ARR, a review of the draft revised chapter revealed that any changes in Victoria for small catchments are likely to be quite minor.

5.2.5 Design Model Parameters

Routing Parameters

Various estimates of kc were trialled for the calibration process and a value of 49.0 was found to provide a good fit of the observed and modelled information available for the January 2011 flood event. A final k_c value of 49.0 and m value of 0.8 was adopted as routing parameters for the design flood estimation. This is in line with Australian Rainfall and Runoff (1987) and initial estimates using the Dyer (1994) method.

Design Losses

The study adopted an initial loss of 25 mm and a continuing loss of 2.5 mm/hr as the design loss parameters. The loss parameters were applied across all ARI events and durations. The loss parameters are consistent with design loss parameters set out in AR&R (1987) and correlate well to those described by Hill et al (1998). The method proposed by Hill et al (1998) uses a baseflow index (30% for the Dunolly catchment, based on regional maps) and mean annual potential evaporation to calculate the initial and continuing losses, as per Equation 5-1 below.

Equation 5-1 Loss Equations as described by Hill, Mein and Siriwardena (1998)

$$Initial\ Loss = (-25.8\ \times BFI) + 33.8$$

$$Continuing\ Loss = (7.97\ \times BFI) + (0.00659\ \times PET) - 6$$

Where: BFI is the baseflow index

PET is the mean annual potential evaporation (mm)

A summary of the initial and continuing losses predicted by each method (and comparison to nearby Carisbrook catchment) can be seen in Table 5-3.

Table 5-3 Initial and continuing losses for various methods

Method	Initial Loss (mm)	Continuing Loss (mm/h)
Jan 2011 flood event calibration (2nd peak)	16.0	1.1
Jan 2011 flood event calibration (3rd peak)	10.0	1.1
Hill et al (1998)	26.1	5.0
AR&R (1987)	10 – 35	2.5
Carisbrook Flood Study	25	2.5

It should be noted that the design losses were not based on the losses adopted in the calibration events. Losses applied for the September 2010 and January 2011 events are highly dependent on antecedent catchment conditions and are not suitable for design flood estimation.

Given the variability of loss parameters between methods of estimation, a sensitivity analysis was undertaken. Figure 5-2 below shows the change in peak flow rate downstream of the Dunolly township with increasing initial losses at a range of design events. The sensitivity of the peak flow rates to continuing loss can be seen in Figure 5-3. As the initial and continuing losses increase the peak flow rate decreases. The rate of this decrease varies over the loss values trialled as well as across design events.

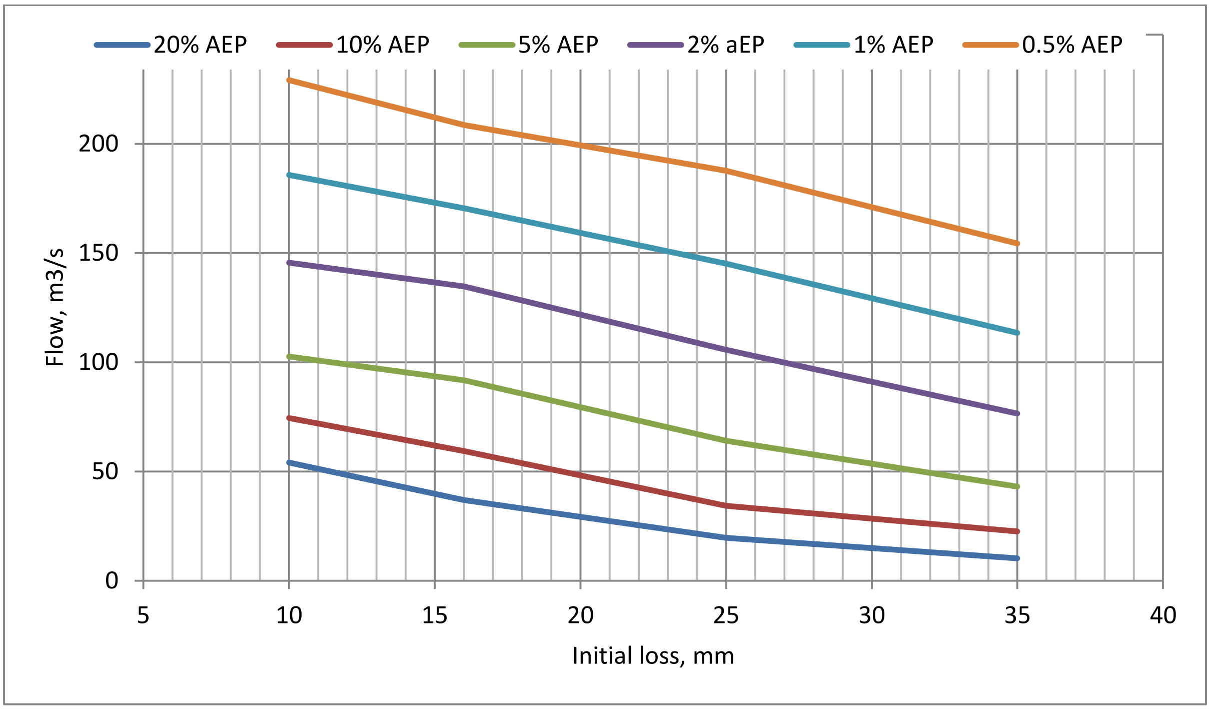


Figure 5-2 **Initial Losses and their corresponding flow rate at a range of ARIs for critical storm duration (18 hours)**

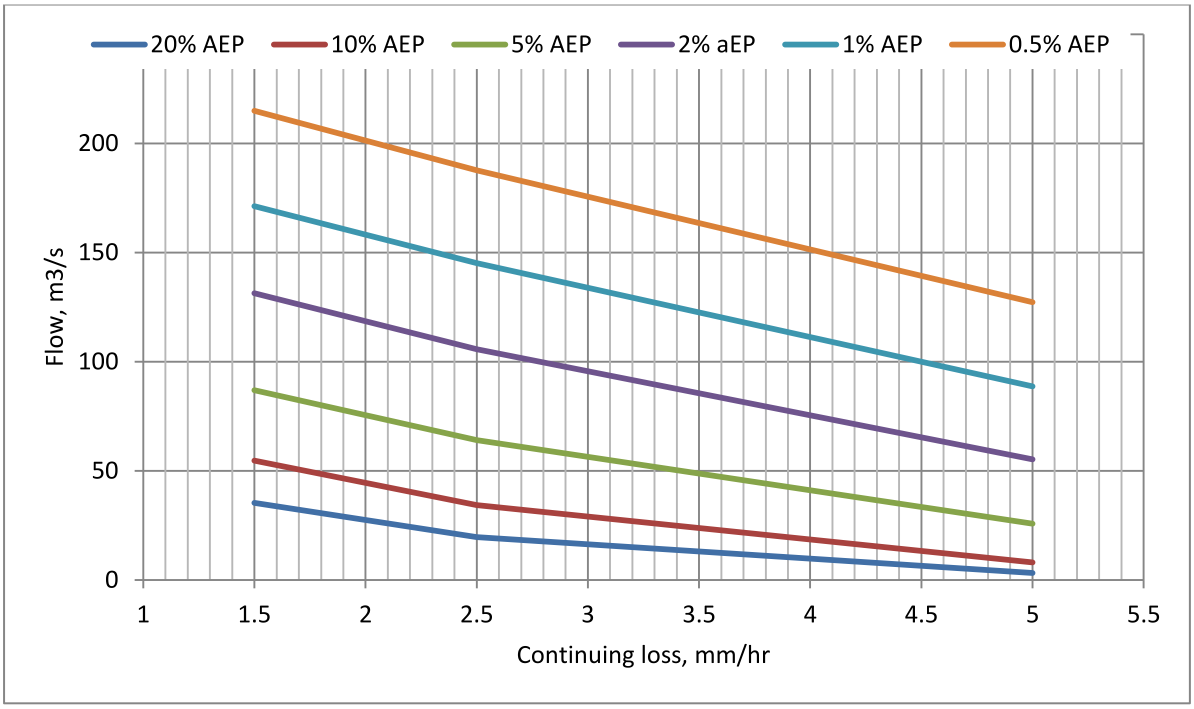


Figure 5-3 **Continuing losses and their corresponding flow rate at a range of ARIs for critical storm duration (18 hours)**

The selection of loss parameters for design purposes in an ungauged catchment is a complicated process that requires an understanding of the sensitivity of the assumptions. The initial losses trialled resulted in 1% AEP design flow estimates with a range of 60 m^3/s. The continuing loss trials showed a similar flow range.

The recommended design loss parameters of 25 mm initial loss and 2.5 mm continuing loss represent losses that are well within the bounds of what could be deemed reasonable, and given that they correspond to losses adopted in nearby Carisbrook, provide a consistency across studies. Further verification of the adopted values will be discussed later with respect to the hydraulic design modelling results.

5.2.6 Design Flood Hydrographs

Using the proposed RORB parameters, design flood hydrographs were determined for input locations into the hydraulic model. A range of storm durations were run (10min – 72hrs) to ensure the critical storm durations were determined. Table 5-4 displays the calculated design peak flows and critical storm durations for various design events for the inflow from Burnt Creek and outflow from the model (for comparison only).

Table 5-4 RORB model design peak flows and critical storm durations at selected locations

	Burnt Creek Inflow		Burnt Creek Outflow*	
ARI	**Peak flow (m^3/s)**	**Duration (hrs)**	**Peak flow (m^3/s)**	**Duration (7hrs)**
5	14	18	20	18
10	28	72	42	72
20	48	72	71	72
50	70	18	106	18
100	93	18	145	18
200	119	18	188	18

* the outflow at the location of the hydraulic model downstream boundary is represented here only for information, and has not been incorporated into the model.

It is noted that for the 10% and 5% AEP events, the critical duration storm was calculated to be 72 hours. It is thought that the 72 hour duration storm would not actually be the critical duration storm given the small size of the catchment. This issue has occurred in previous studies and it relates to the 72 hour temporal pattern from ARR (1987) Zone 2. Figure 5-4 and Figure 5-5 show the hydrographs for the 10% and 5% AEP events for storm durations upwards of 3 hours for the main Burnt Creek inflow to the hydraulic model.

The hydrographs show that the 72 hour duration event produces a hydrograph quite different than the other durations. The 18 hour duration for the 10% and 5% AEP events produces peak flows within 2 m^3/s of the other peak flows discarding the 72 hour duration. It was decided that the 18, 24 and 32 hour peak flows were all quite similar and would be run for each design scenario through the hydraulic model and the results enveloped to produce a maximum water surface elevation for each design event.

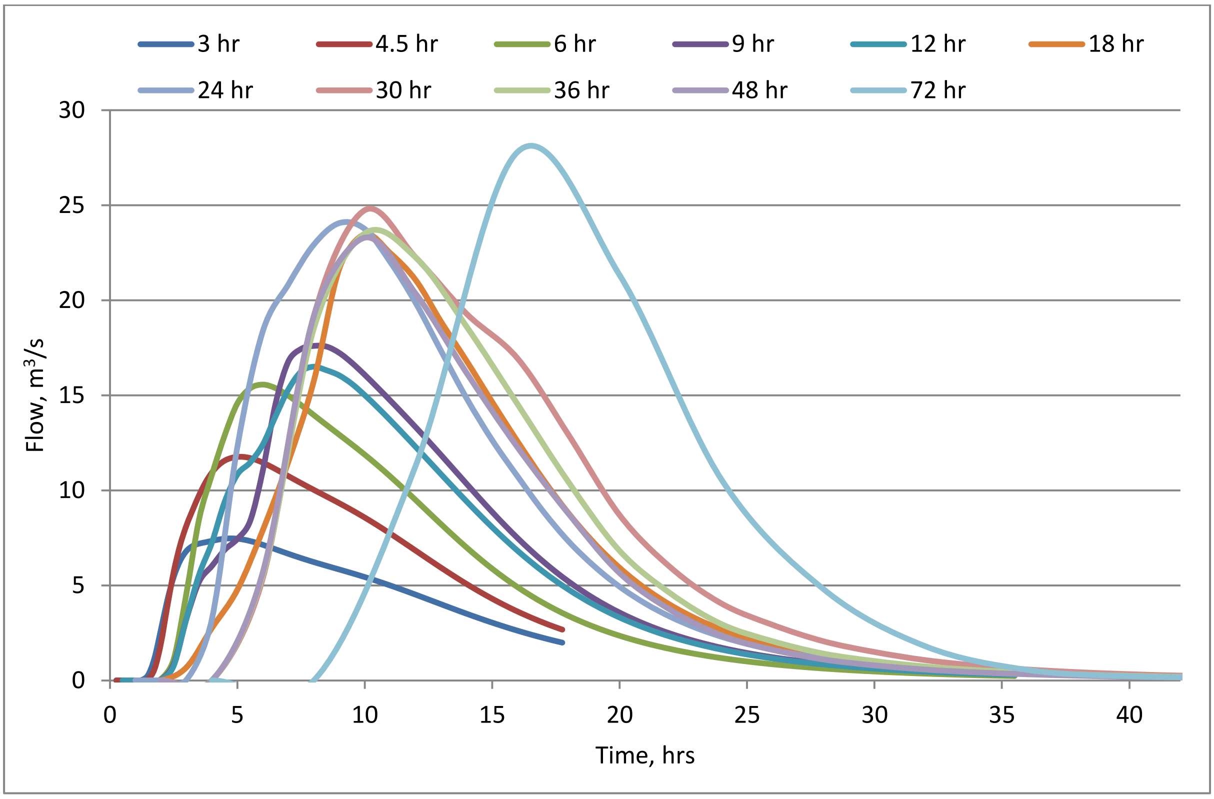


Figure 5-4 10% AEP hydrographs for various durations

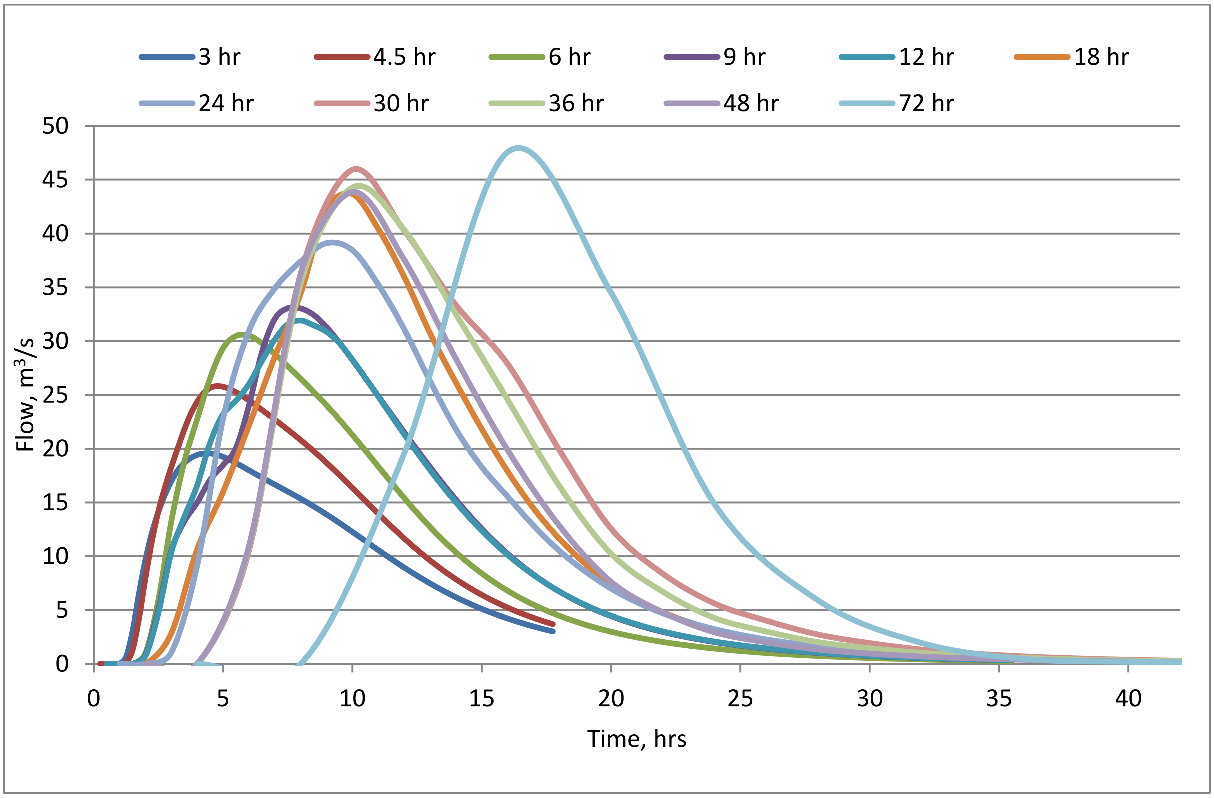


Figure 5-5 5% AEP hydrographs for various durations

5.2.7 Design Flow Verification

The design flows are largely dependent on the adopted RORB model design parameters. A number of checks were undertaken to verify the generated design flows.

Flood Frequency Analysis

A flood frequency analysis allows the estimation of peak flows based on statistical analysis. FLIKE was used to perform the flood frequency analysis for flows at the Bet Bet gauge on Bet Bet Creek, providing an estimate of the 1% AEP flow at this location from data for the period 1943-2012.

FLIKE uses a different fitting procedure to that outlined in AR&R (1987). AR&R (1987) recommends the 'method of moments' fitting algorithm while FLIKE offers a choice of either the Global Probablistic or Quasi-Newton fitting algorithms. The new AR&R (currently being revised) has updated the recommendations regarding fitting procedure, moving away from that previously recommended method and adopting the probabilistic method within FLIKE.

There are a number of probability distributions which can be used to undertake a flood frequency analysis. The 'Log Pearson III' distribution was adopted as the best fit. The 1% AEP flow estimated from this fit (360 m^3/s) was greater than the 1% AEP design flow of 295 m^3/s as estimated by the RORB model.

The difference between the flood frequency analysis and the RORB model results at the Bet Bet Creek gauge is significant, however given that the gauge is outside the Burnt Creek catchment this has not been given a high weighting, but must still be considered.

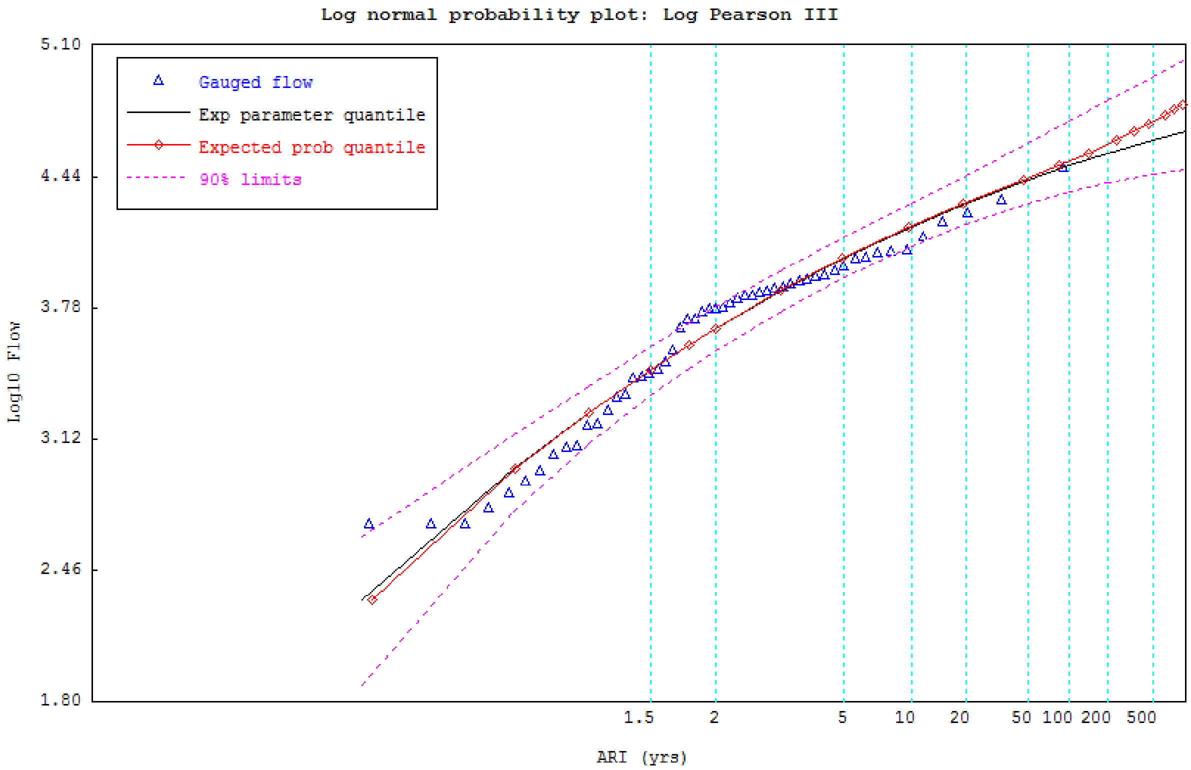


Figure 5-6 Log Pearson III flood frequency analysis – Bet Bet Creek.

Rational Method

Rational Method calculations were performed as an additional check of the design flows at the lower end of Burnt Creek. The results are shown in Table 5-5 and demonstrate a variable difference across events, with consistency improving for larger events. The Rational Method is generally used for estimating peak flows from small catchments (ARR recommends catchments less than 25 km²), and is not designed to be used for large rural catchments such as Burnt Creek (132.1 km²).

Table 5-5 Comparison between design flows and rational method estimates at hydraulic model downstream boundary

ARI	Rational Method flow (m^3/s)	Design flow (m^3/s)	Difference, m^3/s
20%	57	20	37 (185%)
10%	70	42	28 (67%)
5%	87	71	16 (23%)
2%	116	106	10 (9%)
1%	139	145	-6 (-4%)

Draft Australian Regional Flood Frequency Method

As a part of the current Project 5 update to AR&R, a new regional flood frequency method has been developed to replace the Rational Method. This method is still under development, but details of the method can be found at the AR&R website[7]. Using the same coordinates and IFD intensity for the 12 hour, 2 year ARI in Section 5.2.1 and a catchment area of 132 km², the regional flood frequency method produces the results below in Table 5-6. The flows using the regional flood frequency method are generally lower than those estimated using RORB, with the RORB flow fitting within the confidence limits of the regional flood frequency estimates.

Table 5-6 Comparison between design flows and Australian Regional Flood Frequency Method estimates at hydraulic model downstream boundary

AEP	ARFF method flow (m^3/s) 5% and 95% confidence limits in brackets	Design flow (m^3/s)	Difference, m^3/s
20%	26 (11-60)	20	6 (30%)
10%	39 (17-91)	42	-3 (-7%)
5%	53 (23-125)	71	-18 (-25%)
2%	73 (30-175)	106	-33 (-31%)
1%	88 (36-215)	145	-57 (-39%)

Regional Method

Design flows were also verified against methods described in Hydrological Recipes – Estimation Techniques in Australian Hydrology (Grayson et al, 1996). This method utilises a regional equation for the 1% AEP event in rural catchments. The regional method estimated a 1% AEP flow of

[7] http://www.arr.org.au/revision-projects/project-list/project-5/

194 m^3/s, with a catchment area of 132 km^2 to Dunolly (as per the Equation 5-2). The estimated design flow using the regional method is higher than the design estimate of 145 m^3/s from RORB.

Equation 5-2 Regional method flow estimate

$$Q_{100} = 4.67 \times (area^{0.763})$$

Previous Estimates

In a letter from the Rural Water Commission (1986)[8], a 1% AEP flow was calculated using RORB and was verified with calculated 1983 and 1959 flows. The 1% AEP flow was estimated to be between 150 and 207 m^3/s, with an 18 hour storm duration being critical. The estimated 1% AEP design flow calculated for this project is 145 m^3/s, at the lower end of this previous estimate by RWC. It is worth noting that the same critical duration was adopted. It is noted that the continuing loss adopted for these previous estimates is very low at 0.08 mm/hr, with an initial loss of 20 mm. This low continuing loss contributes to the higher design estimate by RWC.

The same author of the above mentioned letter also performed additional design flow estimation using the adopted 1% AEP levels (based on 1959 and 1983 observed flood levels) and calculating a flow using a Manning's equation approach. This approach came up with a 1% AEP flow of 160 m^3/s, reasonably close to the recommended 145 m^3/s from this study.

The RWC estimated that the 1983 event had a peak flow of between 42 and 45 m^3/s, with the larger 1959 event having a peak flow of between 128 and 192 m^3/s (they adopted 160 m^3/s as the average). They then concluded that the 1959 event was representative of a 1% AEP event.

Scaled Design Flows

A very simplistic method for comparing design flood events is to look at similar catchments and compare design peak flows by the ratio of catchment area; the following equation can be applied.

Equation 5-3 Scaled peak flow by catchment area

$$Q_X = Q_Y \times \left(\frac{A_X}{A_Y}\right)^{0.8}$$

Recent flood studies were investigated for similar catchments north of the Great Dividing Range, but most catchments were either much larger riverine systems or steeper catchments on the slopes of the Great Dividing Range. The most similar catchment was thought to be that of Natimuk Creek. Although this catchment is further west in the Wimmera, its catchment is approximately 114 km^2 and is very flat. The peak 1% AEP flow estimated for Natimuk Creek was 100 m^3/s (Water Technology, 2012)[9]. The peak flow calculated for Dunolly with a catchment area of 132 km^2, using the above equation is 112 m^3/s, slightly lower than the adopted design flow of 145 m^3/s for this study.

Historic and Design Comparison

From the RWC work described above, estimates of the 1983 and 1959 flood can be taken as 45 m^3/s and 160 m^3/s respectively. The RORB modelling from this study estimates that the January 2011 flood event had a peak flow of approximately 107 m^3/s.

Comparing the above flows to the adopted design flows in Dunolly from the RORB modelling allows a frequency to be assigned to these historic events. This provides some context for the design estimates.

[8] Correspondence No. 83/2847, 27th June 1986
[9] Water Technology (2012) Natimuk Flood Investigation, *Wimmera CMA*

Table 5-7 Design estimates and historic event comparison

Event	Peak Flow (m^3/s)
20% AEP	20
10% AEP	42
September 1983	45
5% AEP	71
2% AEP	106
January 2011	107
1% AEP	145
April 1959	160
0.5% AEP	188

Summary of Design Verification

A number of methods for verifying the adopted design flows were employed for the Burnt Creek catchment. A summary of the various methods and their corresponding 1% AEP flow can be seen in Table 5-8. The adopted design flow for the 1% AEP event is within the range represented by the five verification methods. The average flow from all the methods is 140 m^3/s.

Table 5-8 Verification methods and their corresponding 1% AEP flow

Approach	1% AEP Flow (m^3/s)
Adopted RORB modelling	145
Rational Method	139
Draft Australian Regional Flood Frequency Method	88 (5% and 95% confidence intervals of 36 and 215)
Regional Method	194
RWC estimate	160 (range from 150 to 207)
Scaled flow (from Natimuk)	112

5.2.8 Probable Maximum Flood

The Probable Maximum Flood (PMF) was determined using the rapid assessment method. This method is obtained from a study by Nathan et al (1994)[10], and uses a prediction equation based on a sample of 56 catchments in South Eastern Australia, ranging in size from 1 km^2 to 10,000 km^2. The equation derived by Nathan et al (1994) was as follows:

$$Q_p = 129.1\ A^{0.616}$$

$$V = 497.7\ A^{0.984}$$

$$T_P = 1.062 \times 10^{-4}\ A^{-1.057}\ V^{1.446}$$

[10] Nathan. R. J., Weinmann, P. E. and Gato, S. A. (1994), 'A Quick Method for Estimating Probable Maximum Flood in South Eastern Australia', Water Down Under 94 Conference Proc., Adelaide, November, 1994, pp. 229-234.

Where Q_p is the PMF peak flow (m^3/s), A is the catchment area (km^2), V is the hydrograph volume (ML) and T_P is the time to peak of the hydrograph (h).

This method was considered appropriate given the uncertainty associated with the extrapolation of an uncalibrated model beyond the credible limit. It is also considered appropriate as we note that PMF estimates, by their nature, have an extreme degree of uncertainty. It should be noted that the total catchment area and location of the catchment are within the specified range for application of this equation.

The Nathan et al (1994)[10] method includes regression equations that can be used to obtain preliminary estimates of the peak, volume, and time to peak of Probable Maximum Floods (PMFs). Following calibration of the hydraulic model, some consideration was given to the timing and hydrograph shape of PMF flows.

The estimated peak flow rate for Burnt Creek catchment (downstream of Dunolly) was 2,420 m^3/s. Hydrographs for each inflow location to the hydraulic model were developed for the PMF based on the contributing catchment areas. No flow was distributed locally within the hydraulic model, as the magnitude and timing of the PMF inflow hydrographs are such that any local runoff is negligible.

5.2.9 Adopted Design Hydrology Summary

Based on the hydrological analysis undertaken the RORB model design results were adopted using the following parameters and assumptions:

- Design rainfall depths from IFD analysis of Dunolly location;
- Zone 2 design temporal patterns for all design events;
- Siriwardena and Weinmann areal reduction factors for upstream of Dunolly;
- Uniform spatial rainfall pattern across the entire catchment for all design events;
- Design losses; an initial loss of 25 mm and a continuing loss of 2.5 mm;
- *kc* of 49.0 and *m* of 0.8; and
- Storm durations from 18, 24 and 30 hours modelled.

The peak design flows generated and adopted for use in the hydraulic modelling are presented above in Table 5-7. Full design hydrographs from RORB were utilised as boundary conditions in the hydraulic modelling.

5.3 Hydraulic Design Modelling

Utilising the updated hydraulic model, the design flood events were mapped for the 20%, 10%, 5%, 2%, 1% and 0.5% AEP and PMF events. Each design event (except the PMF) was run for the critical duration events for Burnt Creek; this included the 18, 24 and 30 hour events. The results for each event were then combined taking the maximum water levels for each event. A suite of flood maps were produced, as shown in Appendix A. Figure 5-7 shows all design flood extents overlayed on the one figure for comparison.

A long-section of Burnt Creek was developed to show the water level profile and the impact of structures on the range of events. The length of the creek was split into two (for enhanced visualisation), and can be seen in Figure 5-8 and Figure 5-9 below. The long section was extracted along the centreline of the creek. The horizontal line at each marked road crossing denotes the level of the road/bridge deck at that location.

Figure 5-10 shows the maximum velocity for the 1% AEP design event. The velocity does not exceed 1 m/s outside of the main creek line. So other than along Burnt Creek and its major tributaries, flood velocity is relatively low.

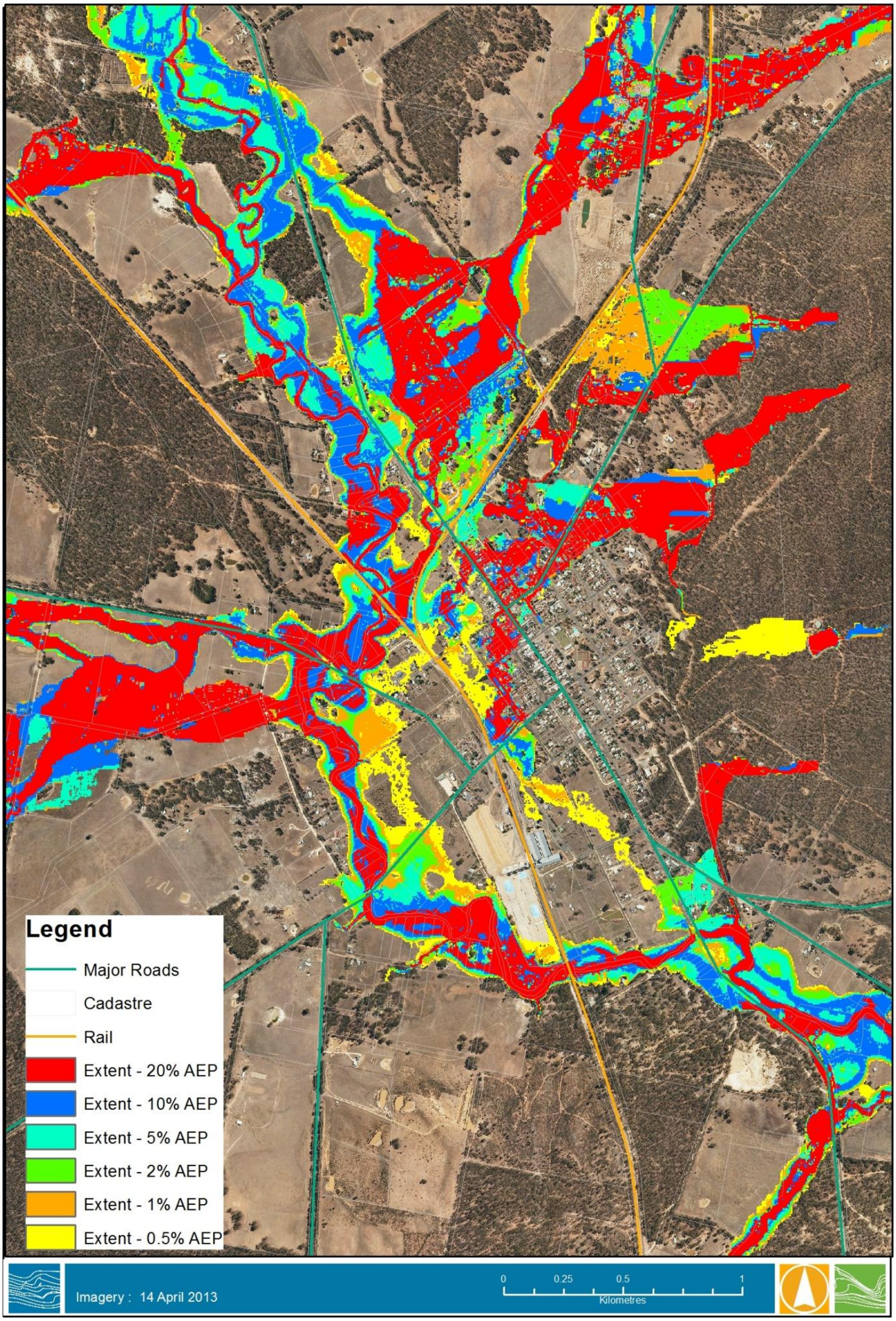


Figure 5-7 Design flood extents

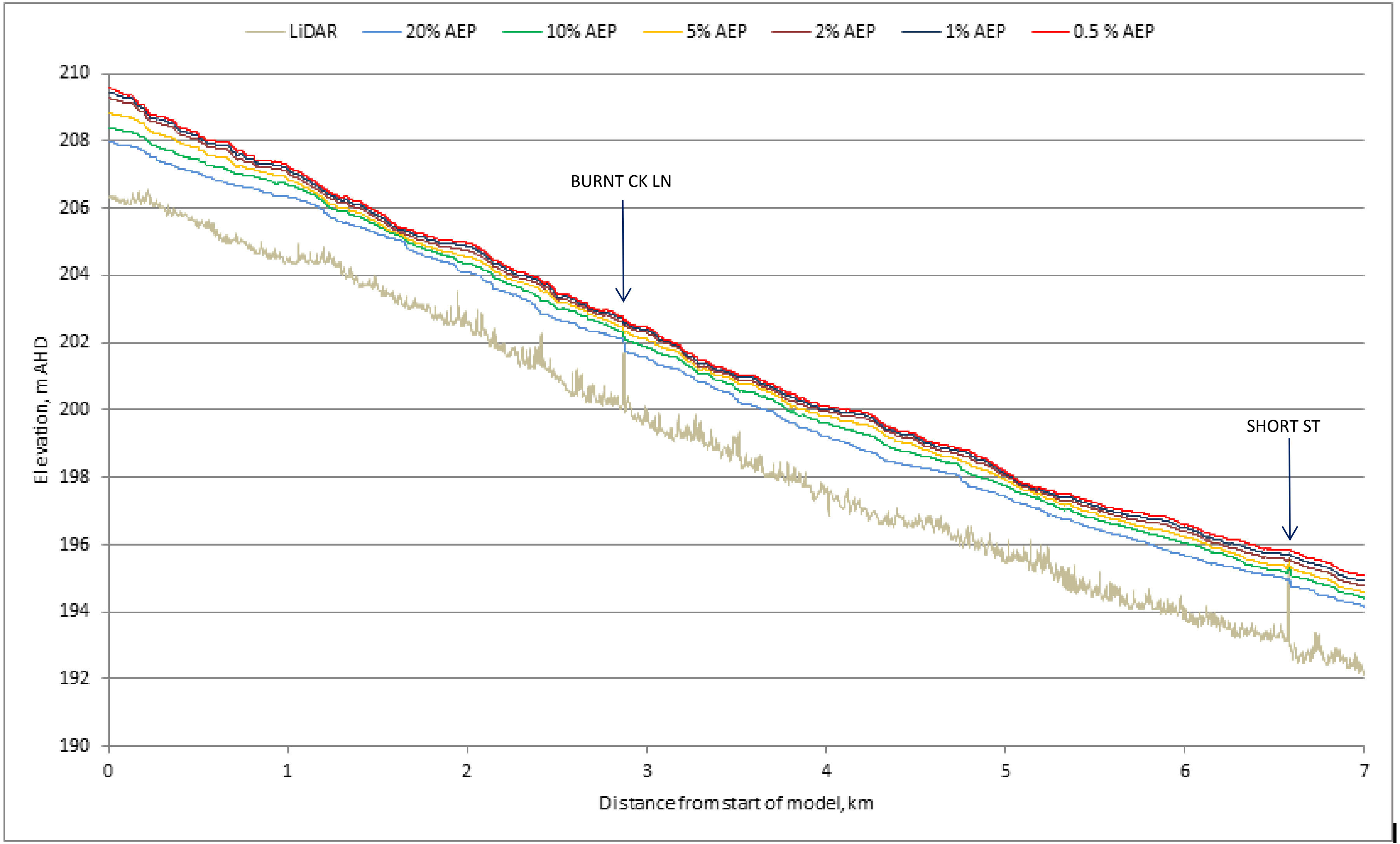


Figure 5-8 Long section of Burnt Creek model predictions for the range of design events (1 of 2)

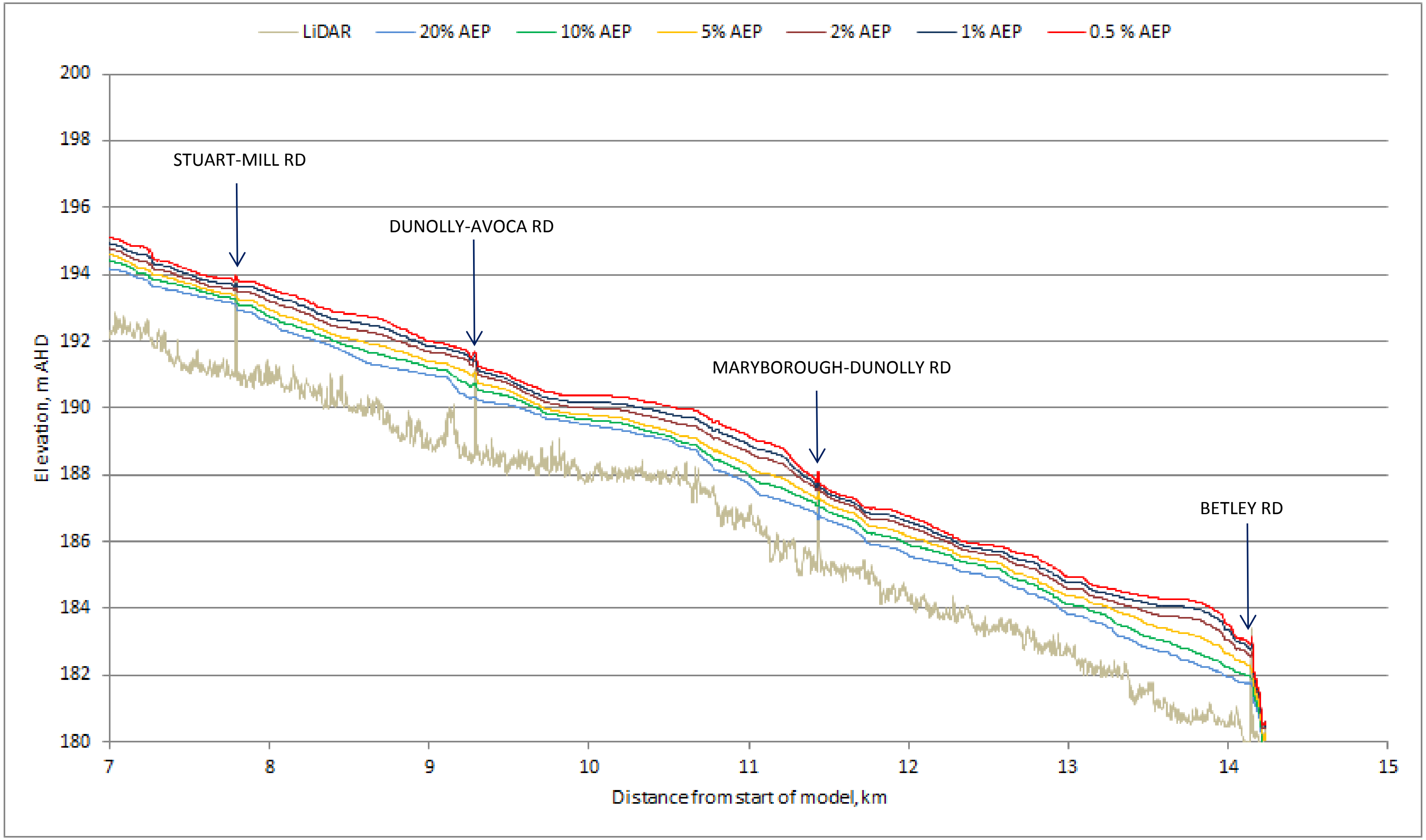


Figure 5-9 Long section of Burnt Creek model predictions for the range of design events (2 of 2)

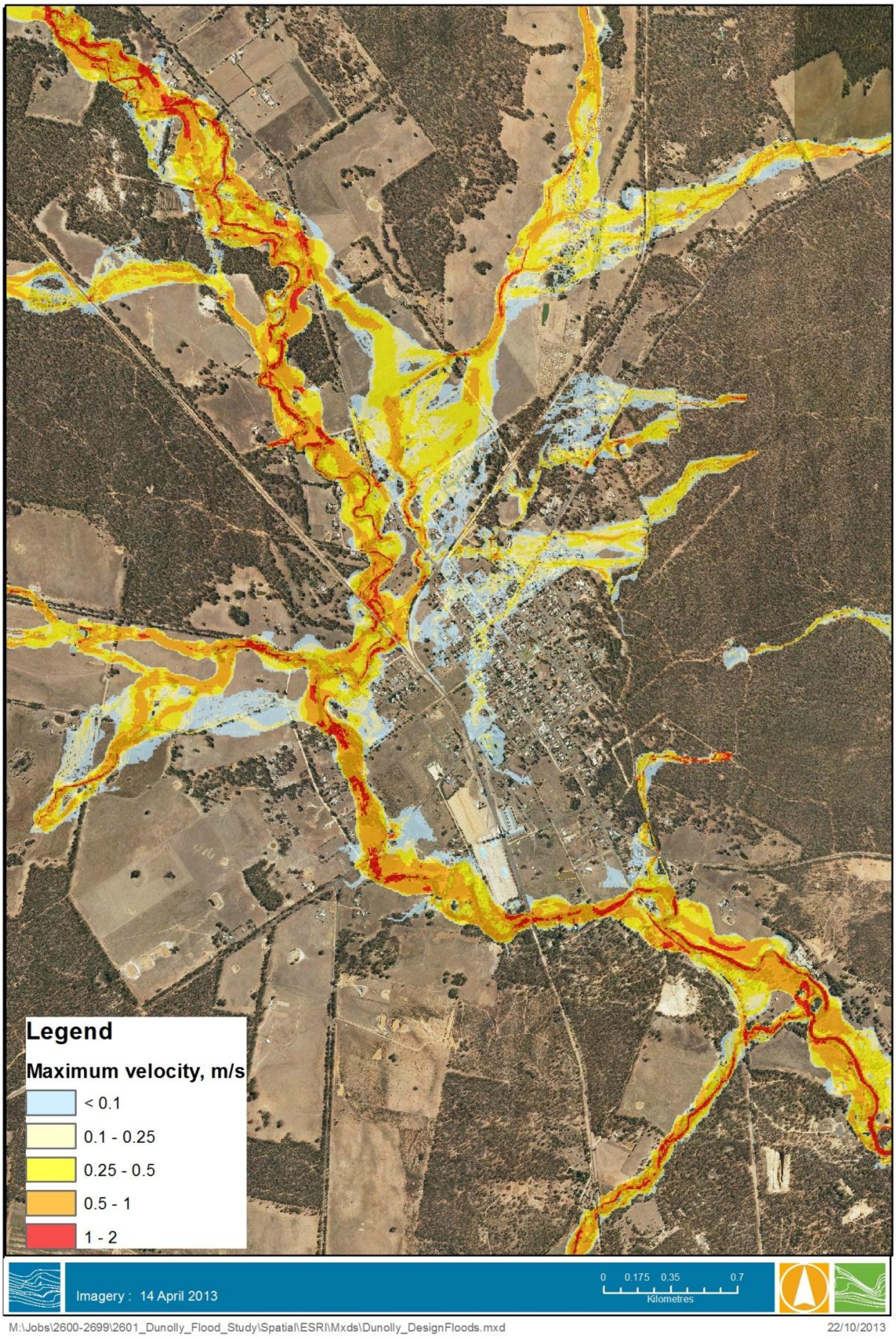


Figure 5-10 Maximum velocity for 1% AEP flood event

5.4 Design Flood Behaviour

The following comments describe the key flood characteristics along Burnt Creek for each design event.

5.4.1 20% AEP Event

- Overtops causeway at Burnt Creek Lane by approximately 400 mm
- Overtops Raglan Street by approximately 20 mm
- There is shallow, sheet flow through town streets
- Some water is conveyed by the Maryborough-Dunolly Road (approximately 50 mm deep)

5.4.2 10% AEP Event

- Overtops causeway at Burnt Creek Lane by approximately 400 mm
- Overtops Raglan Street by approximately 30 mm
- Shallow sheet flow through town streets
- Some water is conveyed by the Maryborough-Dunolly Road (approximately 100 mm deep)
- Water breaks out and begins to flow across approximately half of the football oval on Elgin Street

5.4.3 5% AEP Event

- Approximately 200 m of Betley Road is inundated (on the town side of Betley Road bridge) to depths between 200 mm and 700 mm
- Maryborough-Dunolly road conveys water up to 150 mm deep
- Water breaks out from the end of the contour channel across Dunolly Road (shallow flow only)
- Floodwaters enter the property at 200 Dunolly Road
- Floodwaters break out across Dunolly Road in the low point north of Raglan street (depths less than 100 mm)

5.4.4 2% AEP Event

- Approximately 500 m of Betley Road is inundated (on the town side of Betley Road bridge) to depths between 100 mm and 300 mm
- Maryborough-Dunolly road conveys water up to 200 mm deep
- Water breaks out from the end of the contour channel across Dunolly Road (shallow flow only)
- Floodwaters are approximately 150 mm deep at the property located at 200 Dunolly Road
- Floodwaters break out across Dunolly Road in the low point north of Raglan street (depths less than 250 mm)

5.4.5 1% AEP Event

- Approximately 600 m of Betley Road is inundated (on the town side of Betley Road bridge) to depths between 400 mm and 1.1 m
- Maryborough-Dunolly road conveys water up to 250 mm deep
- Water breaks out from the end of the contour channel across Dunolly Road (shallow flow only)
- Floodwaters are approximately 500 mm deep at the property located at 200 Dunolly Road
- Floodwaters break out across Dunolly Road in the low point north of Raglan street (depths less than 400 mm)
- Table 5-9 indicates approximate travel times of flood waters to various locations within the study extent.

Table 5-9 1% AEP event travel times at various locations

From	To	Approximate Travel Time
RAGLAN STREET		
Start of Rainfall	Start of Rise	3.5 hours
Start of Rainfall	Peak Inundation	9.5 hours
DUNOLLY-AVOCA ROAD		
Start of Rainfall	Start of Rise	3.5 hours
Start of Rainfall	Peak Inundation	10.5 hours
BETLEY ROAD BRIDGE		
Start of Rainfall	Start of Rise	3.5 hours
Start of Rainfall	Peak Inundation	10.5 hours

5.4.6 0.5% AEP Event

- Approximately 600 m of Betley Road is inundated (on the town side of Betley Road bridge) to depths between 600 mm and 1.3 m
- Maryborough-Dunolly road conveys water up to 300 mm deep
- Water breaks out from the end of the contour channel across Dunolly Road (shallow flow only)
- Floodwaters are approximately 700 mm deep at the property located at 200 Dunolly Road
- Floodwaters break out across Dunolly Road in the low point north of Raglan street (depths of approximately 500 mm)

It is worth noting that while Betley Road is inundated beyond the 5% AEP event, the floods modelled here do not overtop the bridge at Betley Road. Inundation occurs at a section with lower topography on the Dunolly side of the bridge.

The capacity of all rail structures (bridges and culverts) is sufficient for the 0.5% AEP flood.

This summary of flood behaviour for each design flood event was prepared along with a number of other flood intelligence outputs, with the aim of providing a condensed summary of flood behaviour to go into the Municipal Flood Emergency Plan to assist in emergency response when a flood occurs.

6. CATCHMENT RAINFALL-ON-GRID MODELLING

6.1 Overview

In order to model the entire Burnt Creek Catchment upstream of the Betley Road Bridge, a 'rainfall-on-grid' model was developed. The purpose of the model was to generate a coarse 1% AEP extent which could be used for planning purposes and flood response. The model results should not be used to set flood levels or floor levels, but should be used as a guide for areas likely to be impacted by overland flooding during a large storm event.

The model extent and estimated drainage lines are the same as those developed during the RORB model construction, and can be seen in Figure 6-1 below. Note however that some differences can occur in coarse rainfall-on-grid models and detailed hydraulic modelling results due to differences in the modelled storage characteristics of the catchment, model roughness values, etc. In flat areas, the coarseness of the grid and lack of 1D structures may also lead to slight differences in flow distribution over the floodplain.

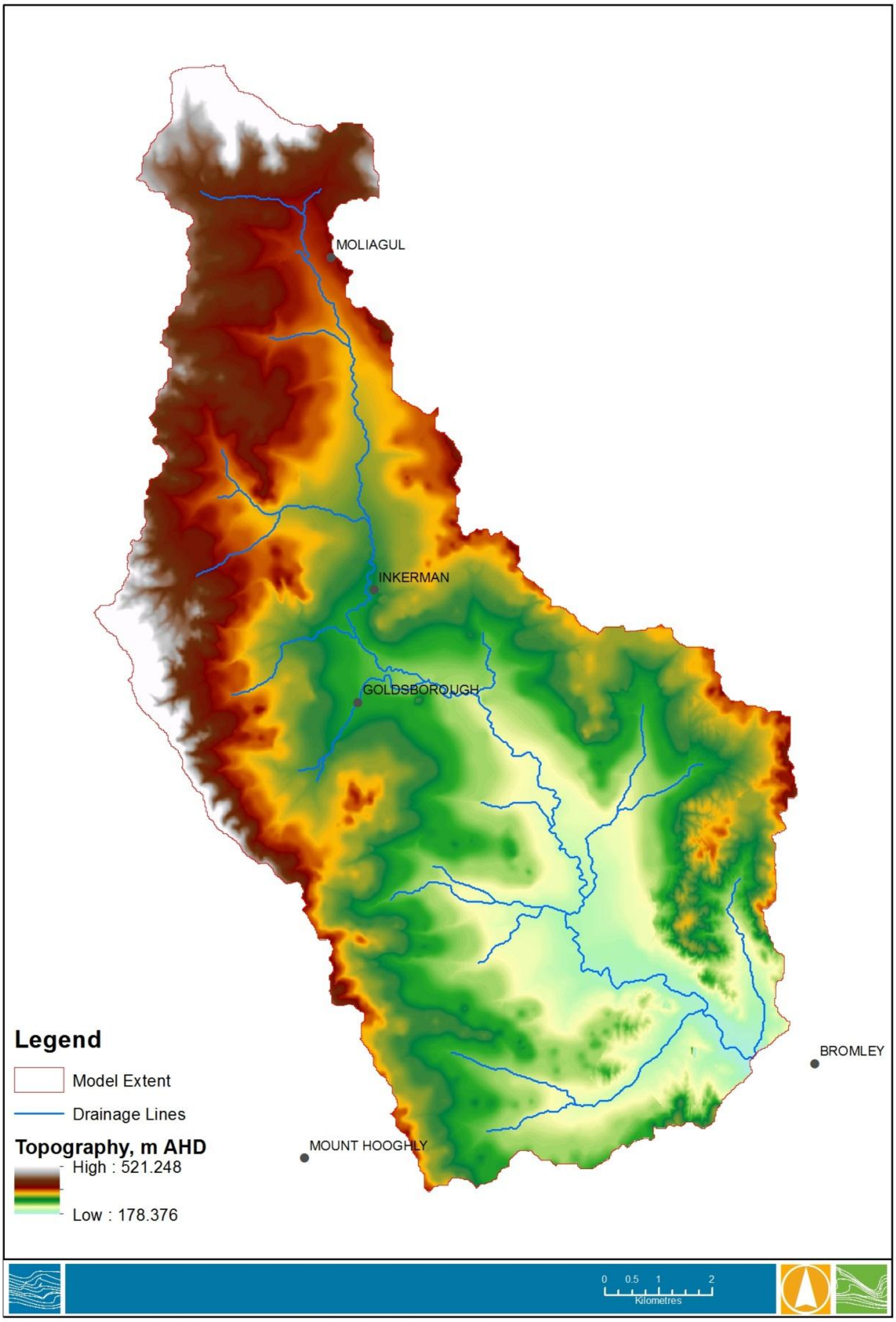


Figure 6-1 Burnt Creek catchment rain-on-grid model extent

6.2 Model Construction

6.2.1 Rainfall

The rainfall-on-grid model covering the Burnt Creek catchment was run for the 1% AEP event only. The response to rainfall can be different depending on the location within the catchment. The peak flood level higher in the catchment is typically generated by intense short duration rainfall events, whereas further downstream, the storm that produces the peak flood level typically has a longer duration. Hence, the 2 hour duration event was added to the 18 hour, 24 hour and 30 hour duration rainfall events simulated for the detailed hydraulic modelling of the township. The water levels were enveloped for the rainfall-on-grid modelling for the 1% AEP event.

Zone 2 temporal rainfall patterns were adopted from AR&R (1987), Book 2[11], and total rainfall depths were extracted using the BoM Online IFD Tool[12] extracted at Dunolly and discussed in Section 5.2.1. The rain-on-grid modelling was tested with and without losses incorporated into the rainfall hyetograph.

6.2.2 Boundary Conditions

The hydraulic model contained one boundary, approximately 500 m downstream of the Betley Road Bridge. This boundary was a fixed water level, set to 179.9 m AHD, simulating some water within the creek downstream, but not sufficient water to cause any significant back water conditions. Essentially it allows the free flow of water further downstream.

6.2.3 Topography

The hydraulic model topography was constructed using a combination of topographic data sets of variable grid size. The areas covered by each dataset are shown in Figure 6-2.

The 10 m grid (Vicmap Elevation DTM, provided by DSE) was available for a large part of the area of interest, with the 20 m grid available to cover gaps (Vicmap Elevation DTM, provided by DSE). The 1m LiDAR grid (provided by the North Central CMA) covered the least area of all three data sets but was the most accurate so was used in preference.

In areas of overlap, the 1 m grid was used in preference to the 10 m grid and 20 m grid respectively. The combined LiDAR data set was re-sampled to a 10 m grid resolution for the creation of the model topography to allow for reasonable run times and resolution of results.

[11] AR&R, 1987 – Australian Rainfall and Runoff

[12] BoM Online IFD Tool - http://www.bom.gov.au/hydro/has/cdirswebx/cdirswebx.shtml *Accessed: September 2013*

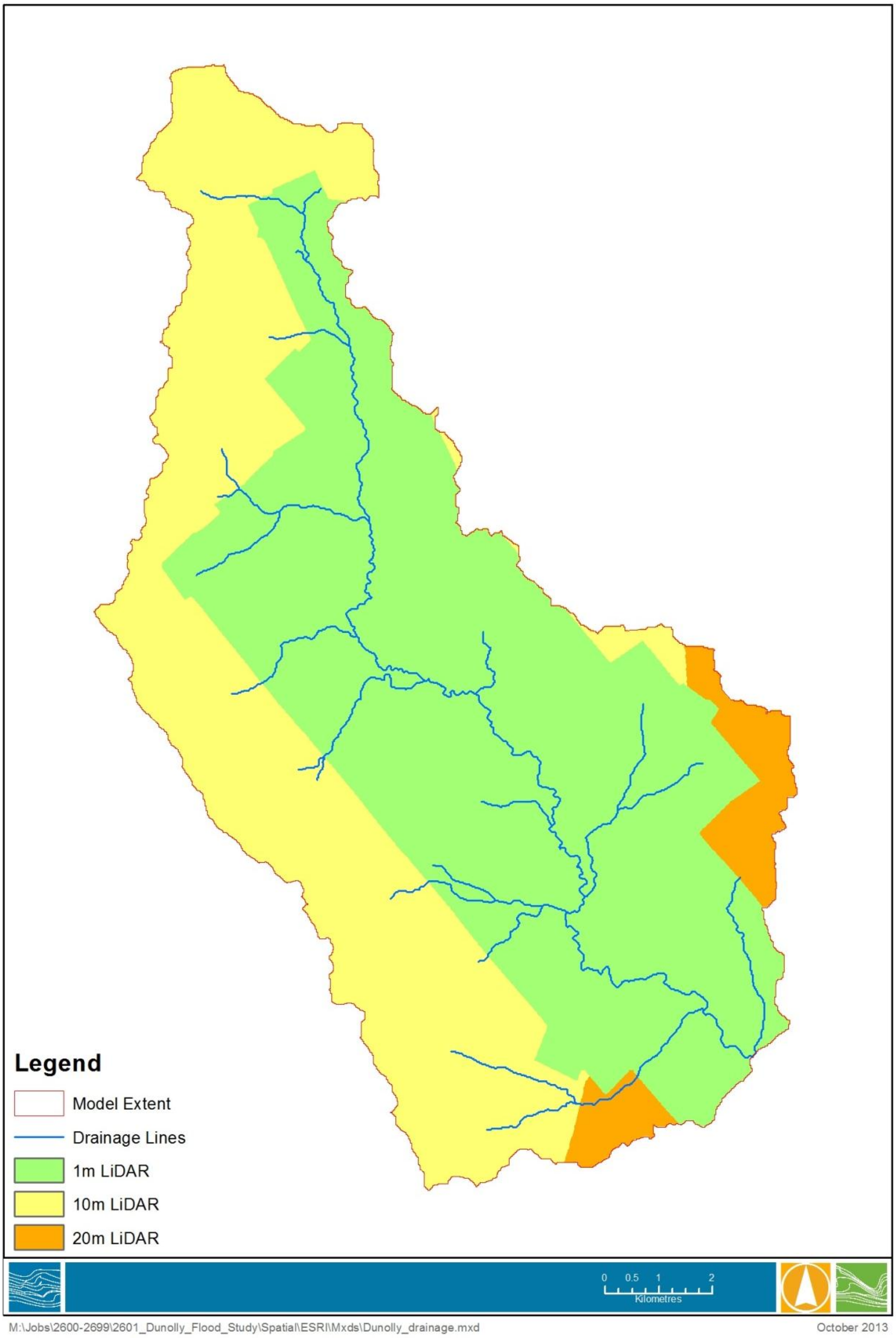


Figure 6-2 Available LiDAR data sets within the Burnt Creek catchment

6.3 Model Verification

6.3.1 Overview

The rainfall-on-grid model was verified by comparison to the 1% AEP flows and water levels produced during the RORB modelling and detailed hydraulic modelling of the Dunolly Township.

To verify the rainfall-on-grid model predictions, hydraulic roughness values applied to the model were altered until results reflected that of the detailed hydraulic modelling for the Dunolly Township.

6.3.2 Losses

The model was tested with and without the incorporation of losses in the rainfall hyetograph. Losses were applied to the rainfall-on-grid model in a similar fashion to the RORB model, using an initial and continuing loss model. An initial loss of 10 mm, followed by a continuing loss of 3 mm was found to produce results with reasonable fit to the detailed model.

It is worth noting that the initial loss for a rainfall-on-grid model will invariably be less than the initial loss applied in RORB. This is because the initial loss parameter in RORB accounts for the initial infiltration, storage and retention of water, whereas storage and retention is represented by the topographic grid in the rainfall-on-grid model.

6.3.3 Roughness

Hydraulic roughness was represented spatially across the catchment by assigning a roughness value to every 2D grid cell. These values were based on aerial photography and zone overlays.

The roughness values were iteratively adjusted, with the peak flow rate, timing and predicted water levels of the model results assessed until a model output was produced that was in agreement with RORB output and the detailed hydraulic modelling.

6.3.4 Results

Parameters within the 1% AEP 18 hour duration model were changed and resulting hydrographs analysed for the verification process. Three extraction points were chosen for comparison of the rain-on-grid model outputs with the detailed model results: Clark Street; Burnt Creek Lane and Kicks Lane (on a tributary that joins Burnt Creek). Hydrographs at these extraction points can be seen in Figure 6-3, Figure 6-4 and Figure 6-5 respectively. The results indicate that the rain on grid model with roughness scaled by a factor of 57% from initial estimates most closely aligned with the detailed model hydrographs. The incorporation of losses into the modelling further resulted in a closer fit. Spot checks on resulting water depths throughout the rain on grid model indicated close alignment with the detailed model also.

It is worth noting that the hydrographs at Kicks Lane show a slightly different response to the calibration than the hydographs at Clark Street and Burnt Creek Lane. This is a result of the flow passing Kicks Lane being from a local tributary rather than Burnt Creek itself. It has been included in this analysis to determine the impact/sensitivity of localised flow compared to creek flow.

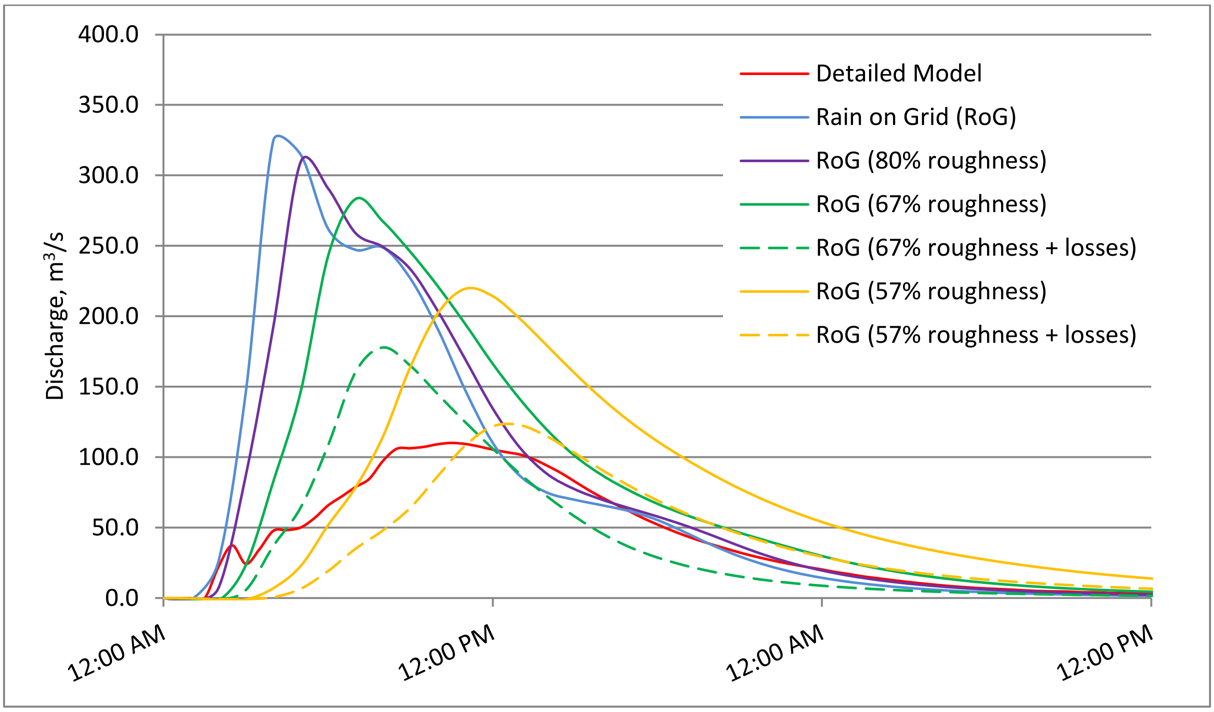


Figure 6-3 Model hydrographs at Clark Street

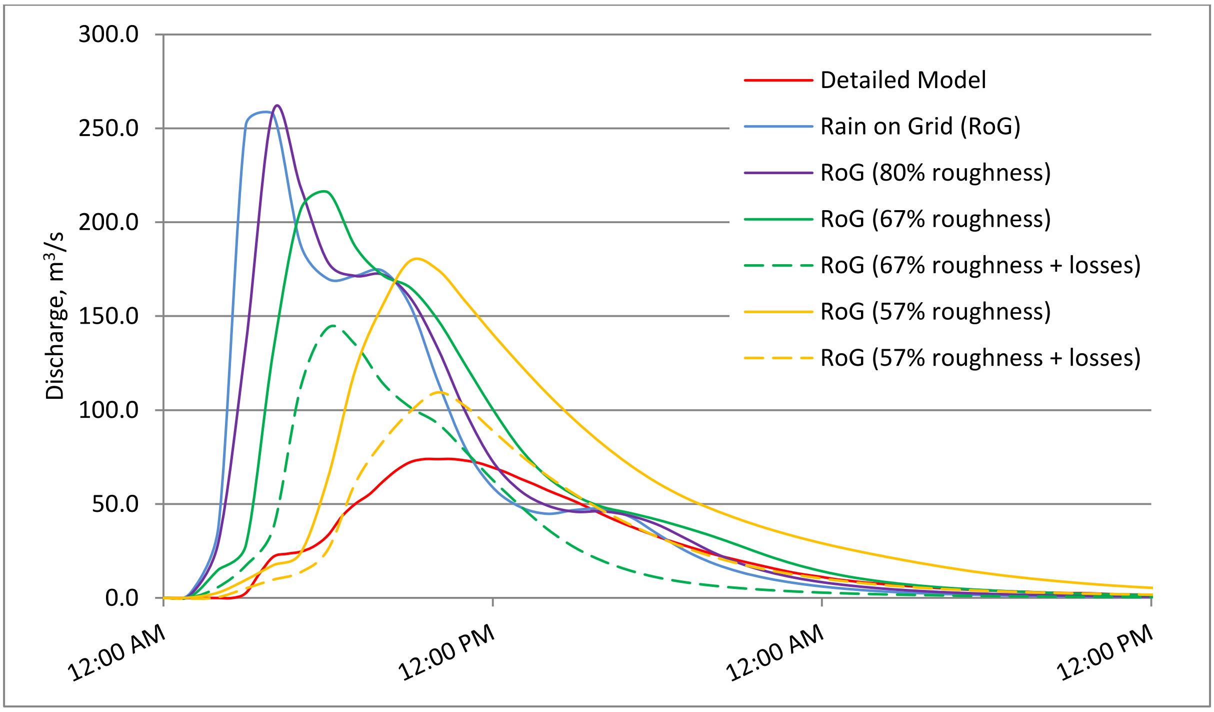


Figure 6-4 Model hydrographs at Burnt Creek Lane

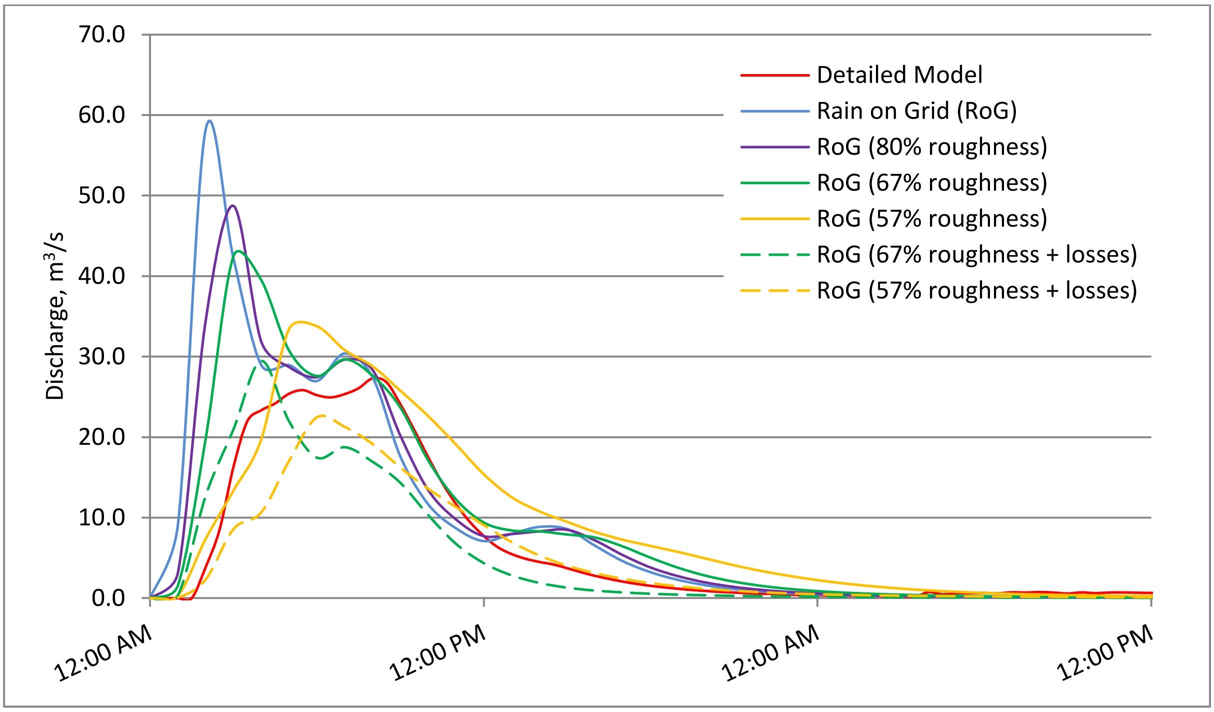


Figure 6-5 Model hydrographs at Kicks Lane

6.4 Results

The 1% AEP catchment rainfall-on-grid results can be seen in Figure 6-6 below. These results were generated using the original roughness grid, scaled by a factor of 57%, with an initial loss of 10 mm and continuing loss of 3 mm/h incorporated into the hydrograph.

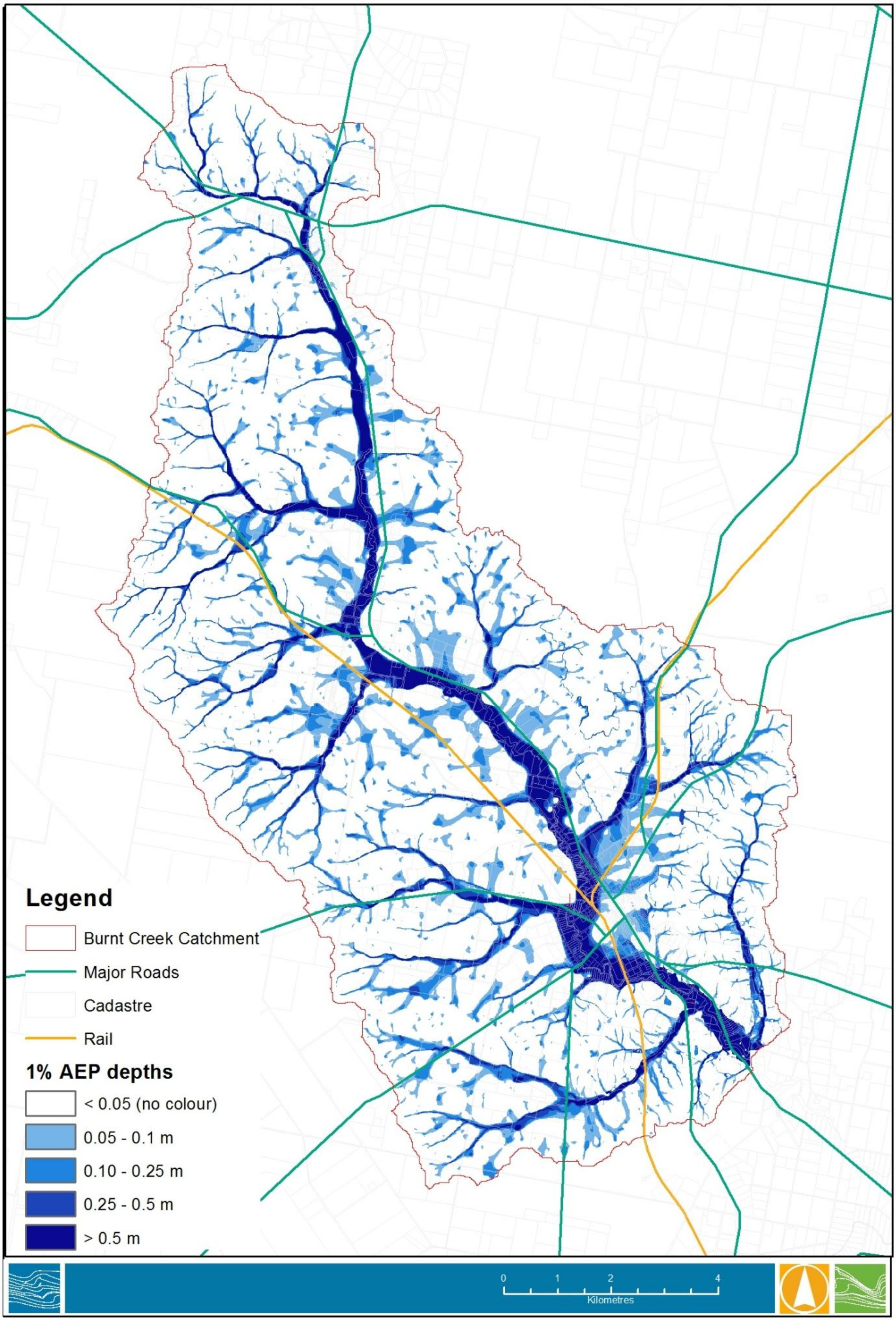


Figure 6-6 1% AEP catchment rain-on-grid model output

7. FLOOD MITIGATION

This section provides an overview of the mitigation options considered to reduce the flood risk and flood damages in Dunolly. The options are divided into structural (i.e. physical works) and non-structural mitigation options (i.e. planning, warning and response actions).

7.1 Structural Mitigation options

Possible mitigation options, detailed in Table 7-1 were derived from suggestions from community members during the drop-in session, discussion with the Steering Committee and inspection of the flood modelling results by Water Technology.

A number of these mitigation options address localised flood impacts (i.e. one or two properties). The Steering Committee discussed whether protection for individual sites was realistic in terms of benefit-cost and it was decided that individual site protection was not a feasible option for this study. It is not to say that individuals can not investigate protecting private property, but that in terms of a community based study, it was not feasible to include single property protection into the overall flood mitigation scheme.

Table 7-1 Potential mitigation options

Option No.	Detail	Source
1	Upgrade culvert under the Dunolly-Bealiba Road (just past Cardigan Street)	Community members
2	Increase channel volume through the golf course	Community members
3	Maintenance and management of the Old Lead Reservoir to attenuate flows	Community members
4	Clearing of the creek to reduce roughness and speed up flow	Community members
5	Construction of drain along Hospital Street and along the railway to intercept shallow sheet flow around the area	Community members
6	Raise Hospital Street to divert overland flows to drain (away from Thomas Street area)	Community members
7	Increase capacity of crossing at the railway near Russell St	Steering committee
8	Raise road or construct roadside levee at 246 Broadway Road and around 237 Broadway Rd (approx. 350m length, 0.5 – 1m high)	Water Technology
9	Ring levee around property at 200 Broadway Rd	Water Technology
10	Construct levee near Dunolly Cemetery to prevent tributary flow downstream	Water Technology
11	Increase capacity of culvert at Dunolly-Moliagul Rd (adjacent to 200 Broadway Rd	Water Technology
12	Construct small/shallow levee at 35 McKinnon Rd	Water Technology
13	Construct small levee/wall with floodgate around property boundary at 1787 Dunolly Maryborough Rd (Junction Hotel)	Water Technology
14	Construct culvert across Dunolly Maryborough Road at main flow path in front of Junction Hotel	Water Technology

15	Construct 500m levee at 171 Betley Road (1.5m high)	Water Technology
16	Upgrade of contour drain north-east of Dunolly	Water Technology

7.2 Structural Mitigation Option Prefeasibility Assessment

Each mitigation option was assessed against a number of criteria: potential reduction in flood damage; cost of construction; feasibility of construction; and environmental impact. The score for each criterion was based on a ranking system of 1 to 5, with 1 being the worst score and 5 the best. Each criteria score was then weighted according to the weighting shown in Table 7-2. The reduction in flood damage was the most heavily weighted criteria as this is really the main objective for all flood mitigation. Table 7-3 reviews and scores each mitigation option against the four criteria and calculates a total score for each option. The options with the higher scores indicate the most appropriate mitigation solutions for Dunolly. While these options were reviewed and scored individually it is important to consider a combination of options when developing a flood mitigation scheme.

Using the prefeasibility assessment above, the 15 identified mitigation options are listed in order of total weighted score as seen in Table 7-4.

Table 7-2 Ranking score for mitigation criteria

Score	Reduction in Flood Damages	Cost ($)	Feasibility/ Constructability	Environmental Impact
Weighting	**2**	**1**	**0.5**	**0.5**
5	Major reduction in flood damage	Less than $ 50,000	Excellent (Ease of construction and/or highly feasible option)	None
4	Moderate reduction in flood damage	$ 50,000 – $ 100,000	Good	Minor
3	Minor reduction in flood damage	$ 100,000 – $ 500,000	Average	Some
2	No reduction in flood damage	$ 500,000 – $ 1,000,000	Below Average	Major
1	Increase in flood damage	Greater than $ 1,000,000	Poor (No access to site and/or highly unfeasible option)	Extreme

Table 7-3 Mitigation option prefeasibility list

No.	Works Location	Mitigation Option	Criteria					Score
			Damage Reduction	Cost	Feasibility/ Constructability	Environmental Impact	Comments	
1	Dunolly / Broadway Rd	Upgrade culvert north of Cardigan St intersection with Dunolly / Broadway Rd *Purpose: increase flow under Dunolly Rd to prevent breakout across Cardigan St and into 200 Broadway Rd*	3	4	4	4	• Approximately 500 mm head difference through culvert indicates potential for increased capacity • Majority of flooding at 200 Broadway Rd is from overtopping of Cardigan St with flows from the north-west. • Flow at this site could be reduced by option 10	14
2	Golf Course	Increase channel capacity through golf course *Purpose: unclear*	2	5	4	3	• Unsure what the intent of this option is	12.5
3	Old Lead Reservoir	Maintenance/management of reservoir to attenuate flows *Purpose: controls peak flow and volume of water downstream*	2	5	3	4	• Modelling indicates the Old Lead Reservoir has no impact on downstream flood impacts in large floods	12.5
4	Burnt Ck	Clear woody debris and vegetation from creek bed along length of Burnt Creek *Purpose: reduce roughness, increase flow*	2	4	3	2	• Would require ongoing maintenance • Cost is approx. $50k annually • Experience indicates marginal difference to flood levels.	10.5
5	Hospital St	Construction of drain along Hospital St and along railway to intercept shallow sheet flow around the area *Purpose: protect properties north of the intersection of the railway with Dunolly Rd*	2	5	4	4	• Flow from this direction has minor effect on above floor flooding	13
6	Hospital St	Raise Hospital St to divert overland flows to drain (away from Thomas St area) *Purpose: protect properties north of the intersection of the railway with Dunolly Rd*	2	5	4	4	• Flow from this direction has minor effect on above floor flooding	13

7	Railway	Increase capacity of crossing at the railway near Russell St *Purpose: reduce backwater effects at railway*	2	3	2	4	• Approx. 300 mm head difference across railway indicates limited potential to improve capacity • Very expensive to upgrade railway crossings	10
8	Broadway Rd	Raise road or construct levee on western side of Broadway road, extending to and around 237 Broadway Rd *Purpose: protect 246 and 237 Broadway Rd*	3	5	4	4	• Approximately 350m long levee • 0.5 – 1m high • Protection of two properties only • Localised site specific option	15
9	Broadway Rd	Construct ring levee around property at 200 Broadway Rd *Purpose: protect 200 Broadway Rd*	3	5	4	4	• Up to 1m high • Localised site specific option • Option 10 may solve the problem cheaper	15
10	Dunolly Rd near cemetery	Construct levee near Dunolly cemetery to prevent breakout from Burnt Creek *Purpose: protect properties between Dunolly Rd and railway*	4	5	4	4	• Would need to occur upstream of cemetery to prevent any impacts there • Approximately 300 m long, up to 1m high	17
11	Dunolly / Broadway Rd	Upgrade culvert south of Cardigan Street, adjacent to 200 Broadway Rd *Purpose: increase capacity of flow under Dunolly Rd (protect 200 Broadway Rd)*	3	4	3	4	• Approximately 700 mm head difference through culvert indicates potential for increased capacity • Majority of flooding at 200 Broadway Rd is from overtopping of Cardigan St with flows from the north-west. • Localised, site specific option	13.5
12	Burnt Ck near McKinnon Rd	Construct small levee at McKinnon Rd *Purpose: protect property at 35 McKinnon Rd*	3	5	4	4	• Localised, site specific option	15
13	Junction Hotel	Construct small levee around property boundary with floodgate/sandbags on entrance *Purpose: protect property at 1787 Dunolly Maryborough Rd (Junction Hotel)*	3	5	4	4	• Localised, site specific option • Levee/wall would require significant height, may be obtrusive	15
14	Dunolly Maryborough Rd	Construct culvert across Dunolly Maryborough Rd at main flow path in front of Junction Hotel *Purpose: protect Junction Hotel*	3	4	3	4	• Approximately 1 m head difference across road • Flooding may be controlled by volume rather than flow rate • Very expensive, major road • Localised, site specific option	13.5
15	Betley Rd	Construct 500 m long levee at 171 Betley Rd (1.5 m high)	3	5	4	4	• Few other options available for protection of this	15

		Purpose: protect 171 Betley Rd					property • Localised, site specific option	
16	Contour channel	Upgrade to contour drain to the north-east of Dunolly *Purpose: protect properties within town from local flow (including up to 7 additional properties affected by shallow flow, not identified in this document)*	4	4	4	4	• Would require ongoing maintenance • Re-establish contour drain and repair breakouts • Need to assess impact on housing further downstream adjacent to channel and near outfall	16

Table 7-4 Ranked mitigation options

Rank	Mitigation Option	Weighted Score
1	Construct levee near Dunolly Cemetery to prevent tributary flow downstream	17
2	Upgrade to contour drain east of Dunolly	16
3	Raise road or construct roadside levee at 246 Broadway Road and around 237 Broadway Rd (approx. 350m length, 0.5 - 1m high)	15
4	Ring levee around property at 200 Broadway Rd	15
5	Construct small/shallow levee at 35 McKinnon Rd	15
6	Construct small levee/wall with floodgate around property boundary at 1787 Dunolly Maryborough Rd (Junction Hotel)	15
7	Construct 500m levee at 171 Betley Road (1.5m high)	15
8	Upgrade culvert under the Dunolly-Bealiba Road (just past Cardigan Street)	14
9	Increase capacity of culvert at Dunolly-Moliagul Rd (adjacent to 200 Broadway Rd)	13.5
10	Construct culvert across Dunolly Maryborough Road at main flow path in front of Junction Hotel	13.5
11	Construction of drain along Hospital Street and along the railway to intercept shallow sheet flow around the area	13
12	Raise Hospital Street to divert overland flows to drain (away from Thomas Street area)	13
13	Increase channel volume through the golf course	12.5
14	Maintenance and management of the Old Lead Reservoir to attenuate flows	12.5
15	Clearing of the creek to reduce roughness and speed up flow	10.5
16	Increase capacity of crossing at the railway near Russell Street	10

7.3 Structural Mitigation Options Modelled

A decision was made by the Steering Committee to focus on options that had potential to provide protection to multiple properties as preliminary cost estimates indicated that protection of individual properties would not be cost effective. The three highest ranking options were progressed to detailed modelling, that is:

- A levee near the Dunolly Cemetery to prevent breakout flow from Burnt Creek inundating properties downstream
- An upgrade to the contour drain east of Dunolly
- A levee on Broadway road for protection of properties at 246 and 237 Broadway Road.

Three mitigation packages were modelled, with various configurations/alignments of the above mitigation options.

7.3.1 Package 1

Package 1 included refinement of the contour channel (to reflect upgrade, management and maintenance) and two levees: one at the cemetery to prevent breakouts from Burnt Creek flowing towards the town; and a roadside levee in the vicinity of 246 and 237 Broadway Road in an attempt to prevent both backing up over Broadway Road from Burnt Creek and also inundation of two properties at the southern end of the levee. The alignment of these levees can be seen in Figure 7-1.

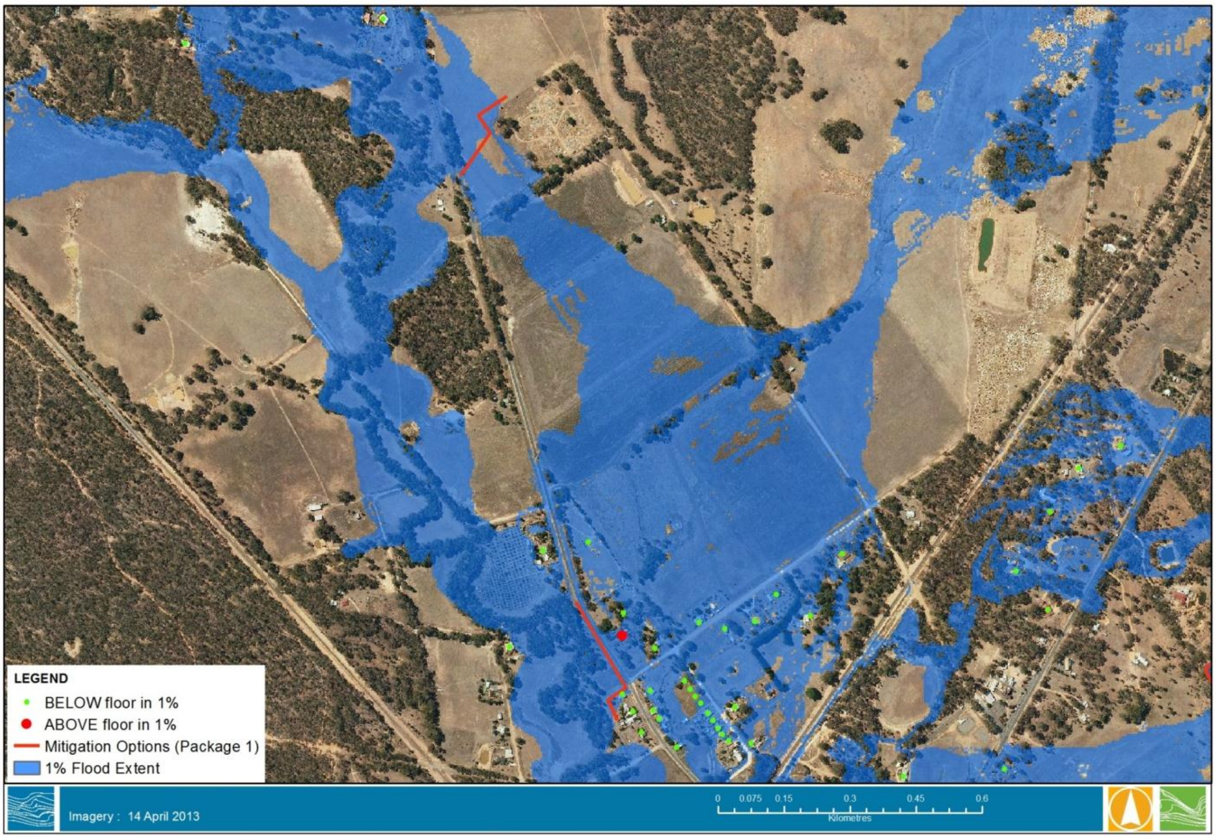


Figure 7-1 Mitigation package 1 levees (with 1% AEP existing conditions extent)

Results

The cemetery levee was somewhat successful in preventing breakout flow from Burnt Creek, although the levee was outflanked with this alignment. It did not provide relief to properties north east of the intersection between Dunolly Road and the railway. Modelled results identified that this area was still flooded as a result of local runoff from the reservoir, as seen in Figure 7-2. The darker blue colour shows the mitigation 1% AEP flood extent compared to the lighter blue existing conditions results.

The roadside levee to protect properties on Broadway road was successful in preventing breakout from Burnt Creek, but flooding at the properties continued, as a result of flow down Broadway Road coming from local runoff from the Old Lead Reservoir, as seen in Figure 7-3.

The difference plot shown in Figure 7-4 shows that the two levees make some difference to flood levels, but the property at 246 Broadway Road is still flooded above floor. An adjustment to the alignment and location of these levees was recommended to the Steering Committee for package 2.

Preliminary modifications to the contour channel in the hydraulic model (to reflect repairs to damaged banks) resulted in some reduction in overflow, as seen in Figure 7-5. The results indicated that preliminary channel modifications showed promise but further refinement of the channel upgrade was required in subsequent mitigation packages.

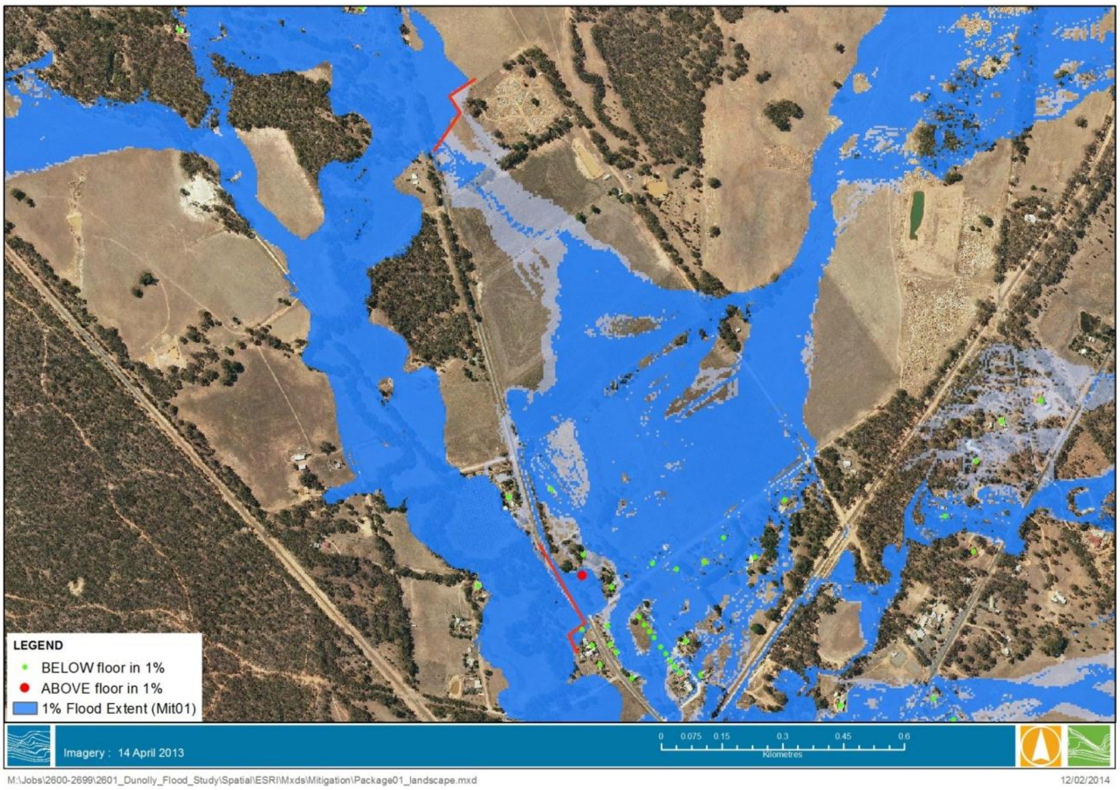


Figure 7-2 Package 1 levee results

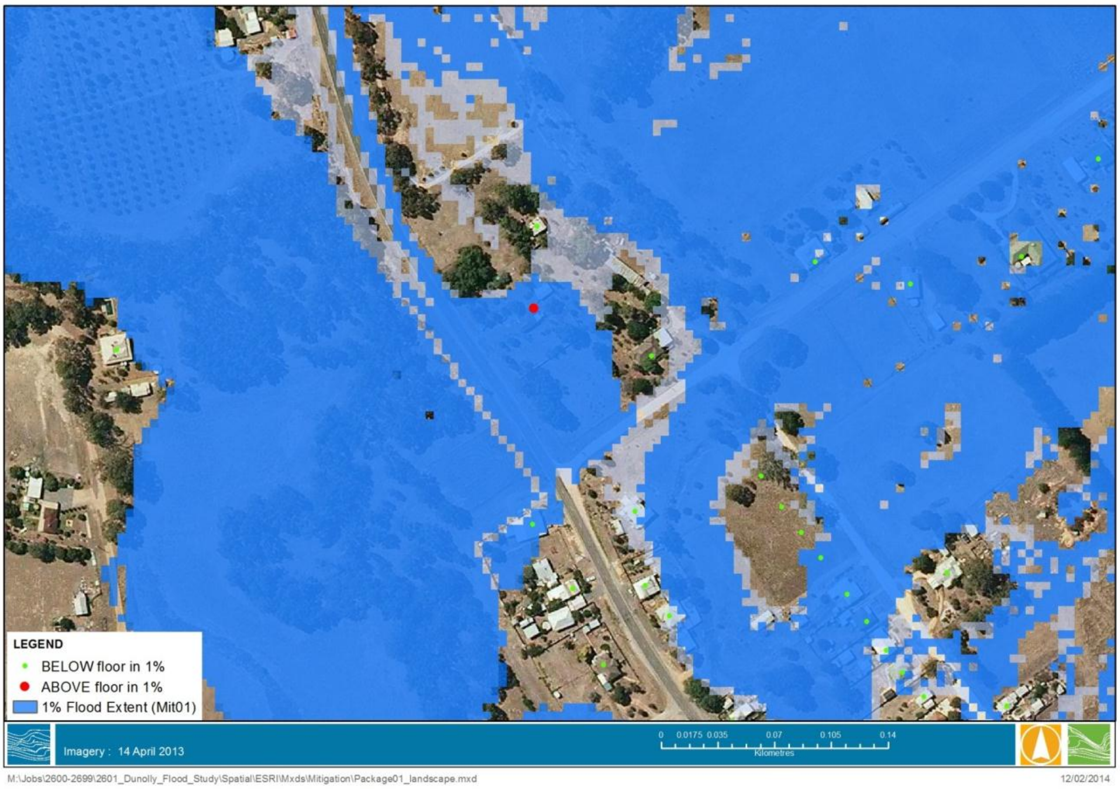


Figure 7-3 Package 1 roadside levee results

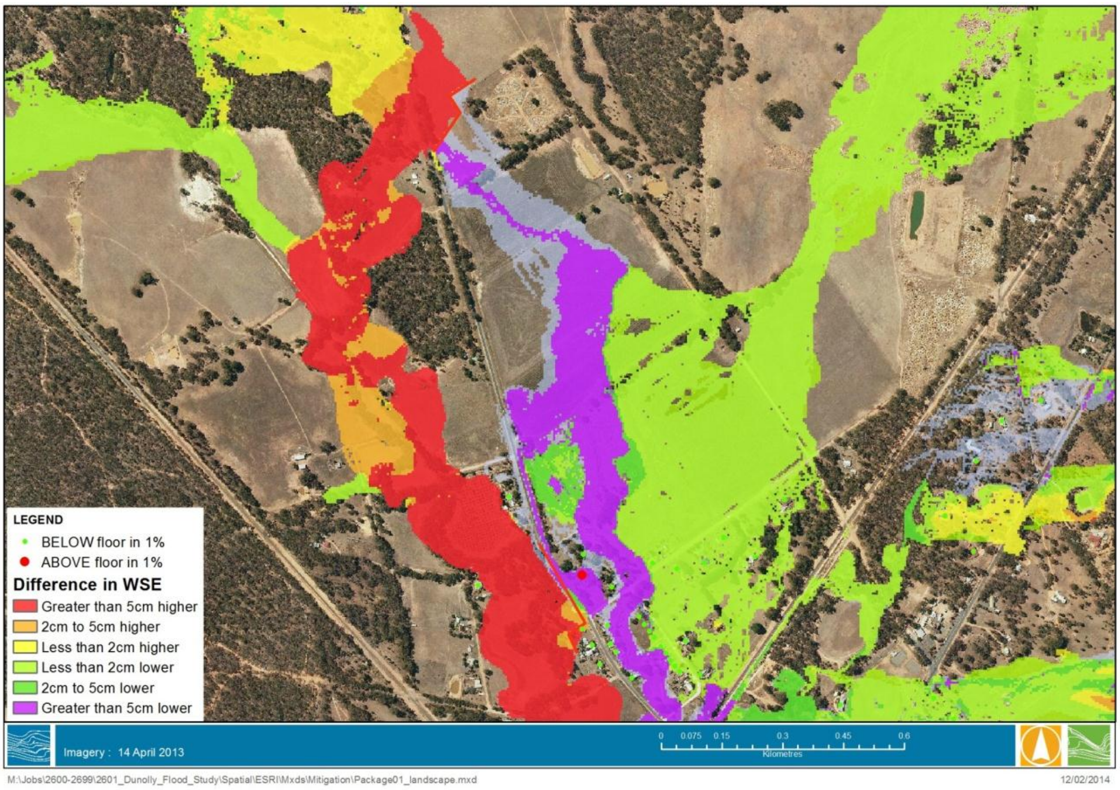


Figure 7-4 Difference in water surface elevation between mitigation package 1 and existing conditions (1% AEP)

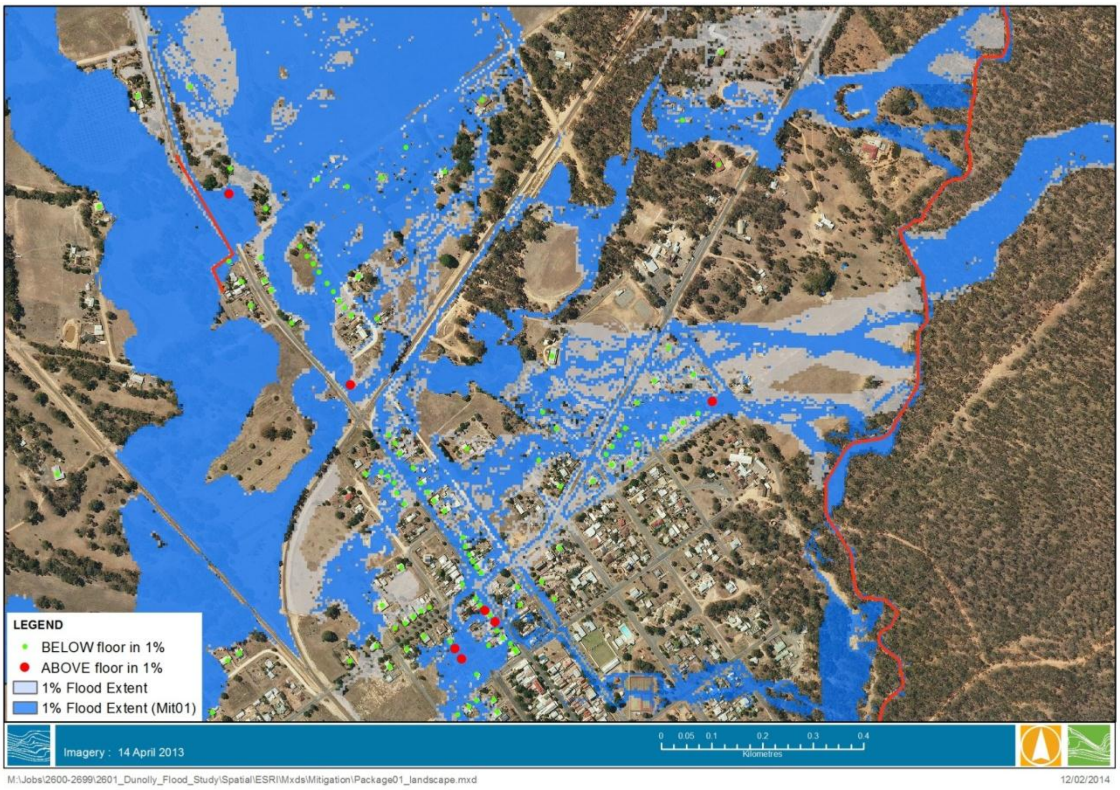


Figure 7-5 Package 1 contour channel results

7.3.2 Package 2

After reviewing results from package 1 the Steering Committee held a meeting and inspected the site. Subsequent to this meeting the second package was developed. Package 2 was very similar to Package 1 with some minor adjustments to levee alignments in an attempt to improve the performance of the cemetery and Broadway levees.

The alignments of the revised levees from Package 2 are shown below in Figure 7-6. The cemetery levee was extended to ensure that the levee was not out flanked and that the cemetery would not be inundated. The Broadway levee was changed slightly to protect only the impacted properties to the south-west of Broadway, as it was felt that preventing the above floor flooding of the property on the northern side of Broadway was very difficult owing to its low floor level and location in a very low area of the floodplain. This property is inundated from overland flooding from local runoff even if flooding from Burnt Creek is prevented.

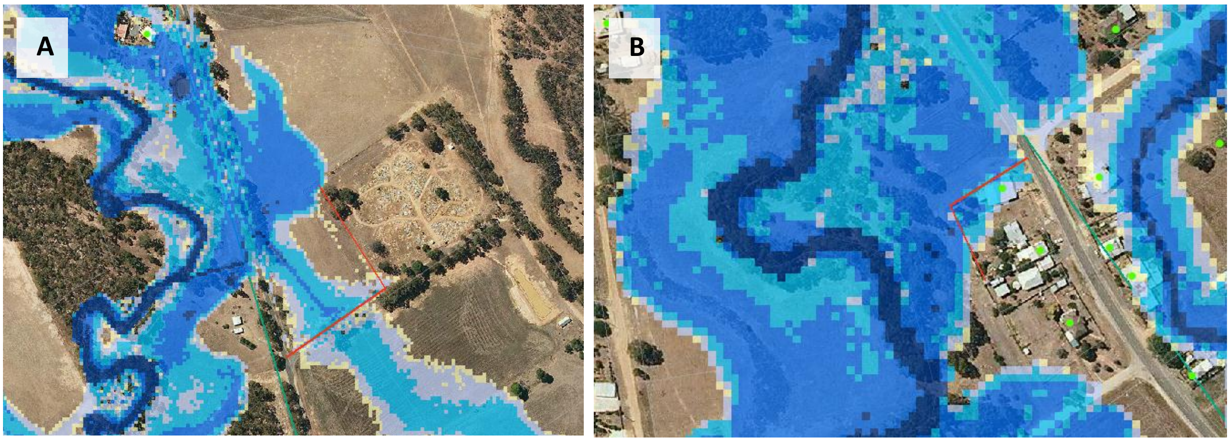


Figure 7-6 Package 2 levee arrangements (a) cemetery levee; (b) Broadway road levee

Results

The flood extent shown in Figure 7-7 demonstrates that all three mitigation measures successfully reduce flooding impacts at their intended location. Flooding in the township is reduced by increased integrity of the contour channel. Breakout from Burnt Creek near the cemetery is prevented by the cemetery levee, and flooding at 246 and 237 Broadway Road is alleviated by the adjacent levee.

The differences in flood levels caused by implementing Mitigation Package 2 can be seen in Figure 7-8. While the mitigation measures have a positive impact on their intended area, they also cause increased water levels in some areas. Water surface elevation is increased in the section of Burnt Creek between the two levees, as well as along the contour channel and at some properties at the downstream end of the contour channel. This is a result of redistributing the flow of water across the floodplain, essentially pushing the problem on to someone else. Properties impacted by this change are identified below.

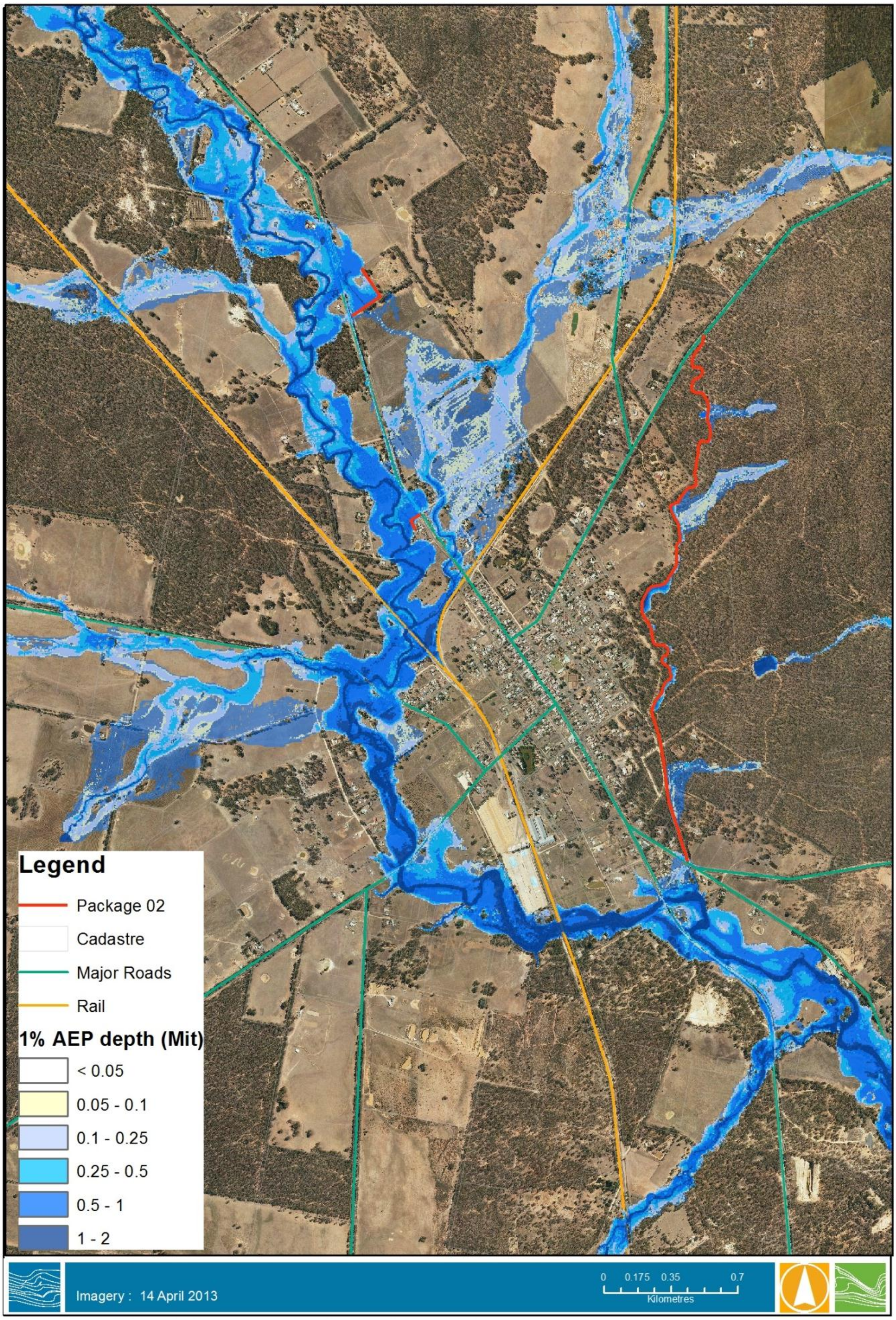


Figure 7-7 Mitigation package 2 – 1% AEP flood extent

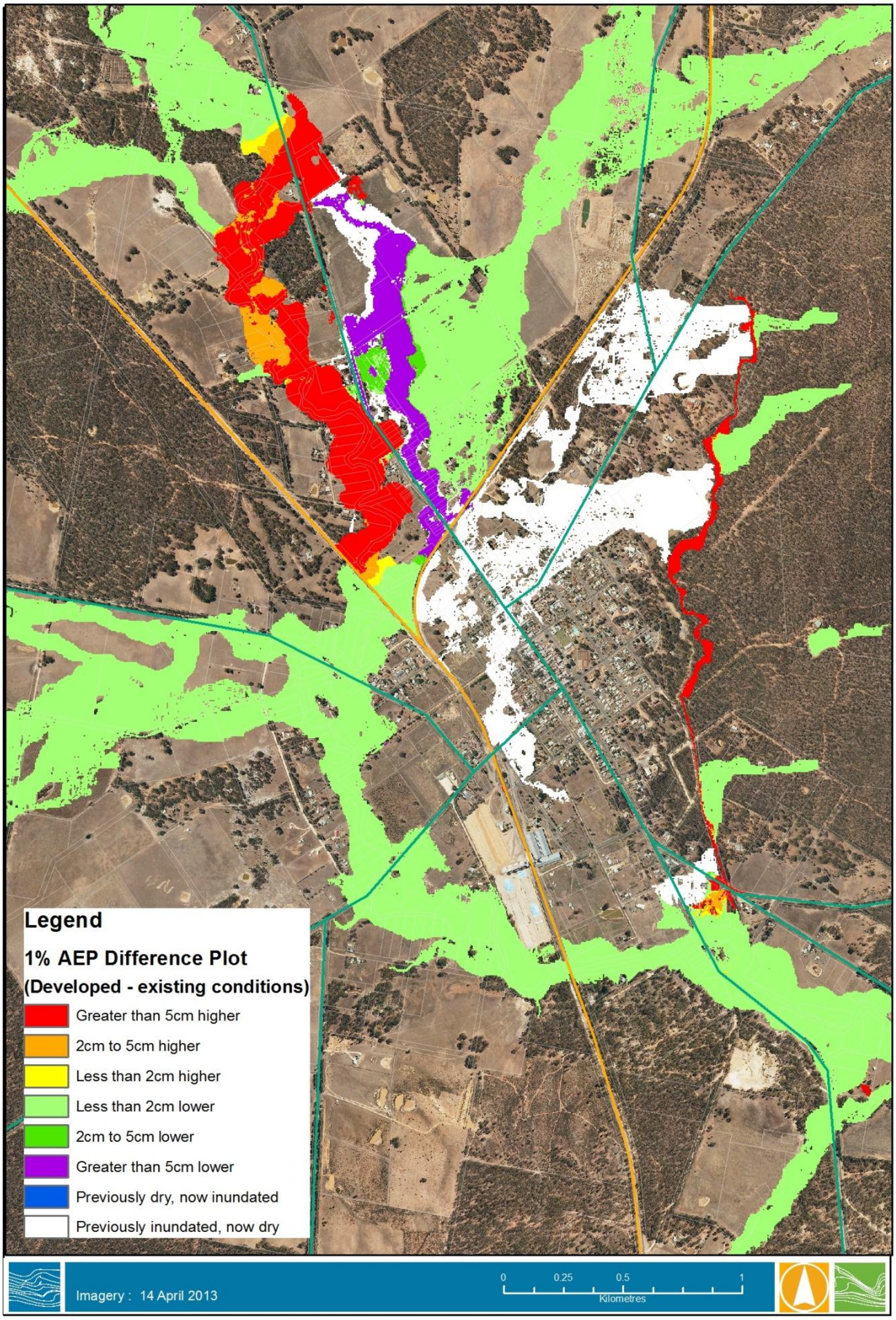


Figure 7-8 **Difference in water surface elevation between mitigation package 2 and existing conditions – 1% AEP flood extent**

A detailed analysis on the flood impact at affected properties for Package 2 was undertaken to determine the impact on individual properties. The results indicated that:

- 13 buildings would no longer be inundated above floor level;
- 49 properties that were flooded (below floor) would now not be inundated at all.
- 5 properties that weren't inundated would now be flooded below floor (all residential buildings).

Note that the mitigation measures of Package 2 did not cause any additional properties to be flooded above floor level. However some properties that are already flooded (below and / or above floor) in a 1% AEP event would likely be inundated to a higher level. Properties flooded above and below floor in mitigation Package 2 are identified in Figure 7-9. A summary of the affected properties is given in Table 7-5.

Table 7-5 Flood affected properties (1% AEP)

	Existing Conditions	Mitigation Package 2	Change
Flooded above floor	17	4	-13
Flooded below floor	69	25	-44
Was above floor, now below			-13
Was below floor, now above			0
Was below floor, now not flooded			-49
Was not flooded now flooded below floor			+5

The five properties that would be negatively impacted as a result of the mitigation Package 2, with a change in their flood status, and now experience below floor flooding are located at:

- 78 Dermoundy Road
- 1824/1 Dunolly Maryborough Road
- 1824/2 Dunolly Maryborough Road
- 5 Lea Kuribur Street
- 14 Lea Kuribur Street

Note that four of these properties are located at the downstream end for the contour channel.

The results of the Package 2 modelling indicated that this option achieves its purpose of effectively protecting the town from the local catchment flows, and reducing impacts from breakouts from Burnt Creek. It demonstrated significant improvements in the flood impacts experienced in a 1% AEP flood event. However there were 5 properties that were slightly worse off, an unacceptable result.

The two levees provided little benefit in terms of flood protection, were potentially cost prohibitive and raised flood levels at other properties. A decision was made by the Steering Committee that the levees would not be progressed further as a mitigation option and that the contour channel would be refined further.

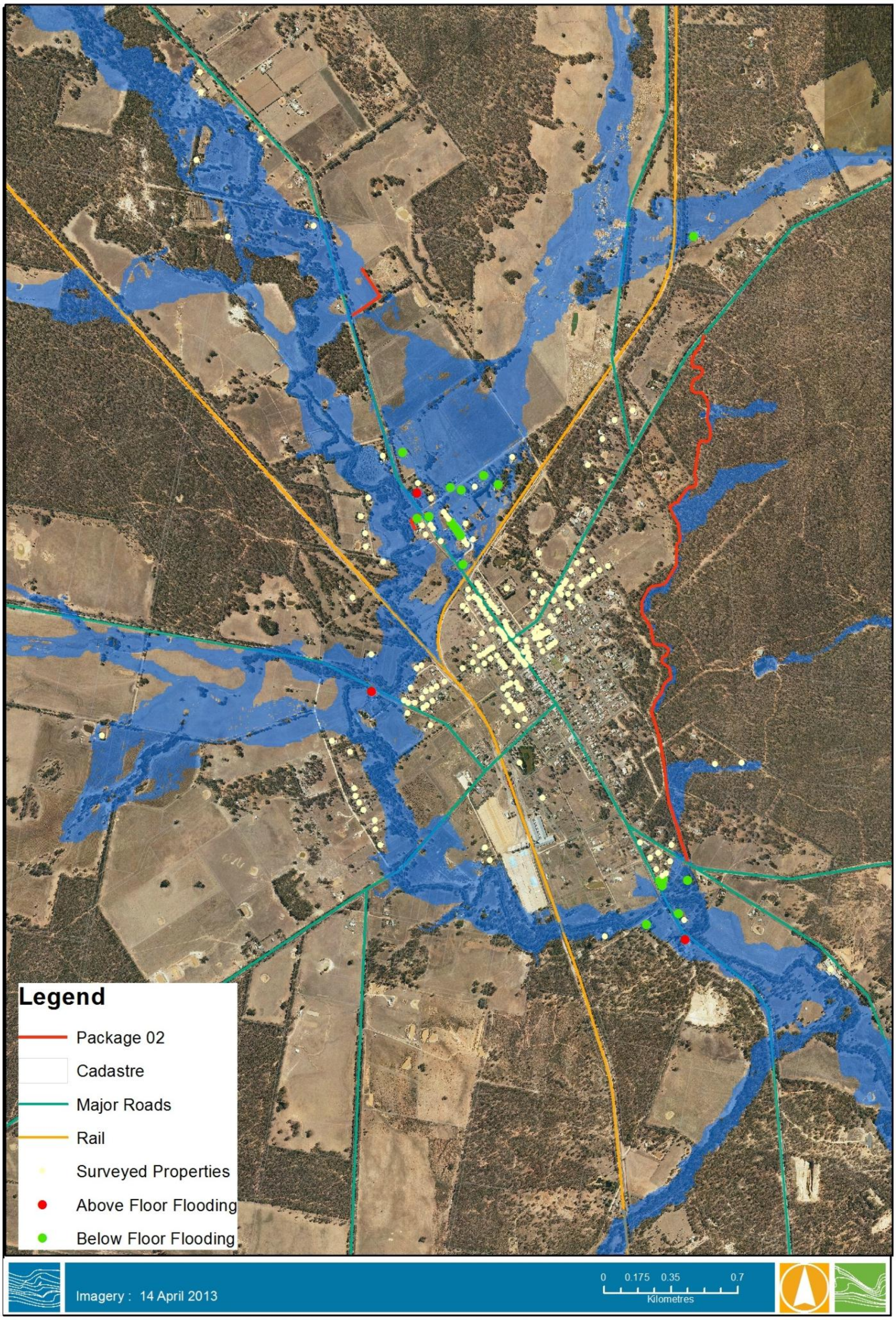


Figure 7-9 Mitigation Package 2 flooded properties – 1% AEP flood extent

7.3.3 Package 3

Mitigation Package 3 was developed with the intent of improving the impact of increased flow discharging from the contour channel onto properties downstream of the Dunolly Road Bridge. Previous modelling indicated an increased flood level at some properties downstream of the bridge, where the contour channel discharges into Burnt Creek.

Package 3 included:

- Refinement of contour channel (to reflect upgrade, management and maintenance)
- A retarding basin at the Council Depot to slow flow from the contour drain
- A low level levee at the downstream end of the contour channel, to a maximum height of 1 m, tying in with Dunolly Road

The location of the proposed retarding basin can be seen in Figure 7-10. It is understood that this land parcel is property of the Council, and is not currently serving any purpose. During the last site visit piles of debris and rubbish were left lying on the ground and the site looked unused.

Figure 7-10 Location of proposed retarding basin

The topography of the site, seen in Figure 7-11 indicates the site to be a naturally low lying basin. Minimal works would be required to convert the site, that is, only inflow and outflow points from the contour channel, and possible raising of some sections of the adjacent unsealed road. The proposed basin could potentially store up to 43 ML.

Minor earthworks would be required to configure the inlet and outlet connections to the contour channel. The inlet would require a reduction in height of the contour channel bank by approximately 1 m (this height was chosen to ensure minor flows will not be diverted through the retarding basin). At the outlet, a small overflow of up to 1 m depth would be required. These estimates will need to be confirmed in a subsequent detailed design phase.

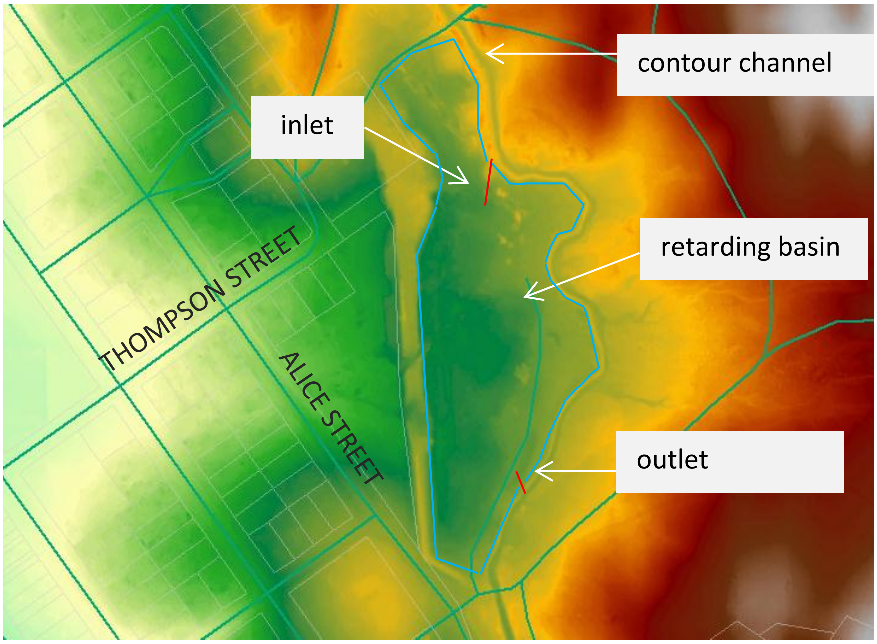


Figure 7-11 Topography at location of proposed retarding basin

Results

The flood extent shown in Figure 7-12 demonstrates that the contour channel is effective in reducing flood impacts in the township. The differences in flood levels caused by implementing Mitigation Package 3 can be seen in Figure 7-13. Water surface elevation would be increased along the contour channel, and within the retarding basin, but decreased throughout the township. There would be no difference in levels along Burnt Creek.

It should be noted that without the retarding basin (which acts to slow the release of water through the contour channel), there would be a slight increase in water levels downstream of the Dunolly Road Bridge. In previous packages this was found to exacerbate flooding at some properties. The retarding basin would prevent any increased impact downstream.

A detailed analysis on these changes was carried out, to determine the impact on individual properties. The results indicated that:

- 11 buildings would no longer be inundated above floor level;
- 54 properties that were flooded (below floor) would not be inundated
- No properties experience greater flood levels than existing conditions (i.e. none are worse off).

Properties that flooded (below and/or above floor) in Mitigation Package 3 are identified in Figure 7-14. A summary of the affected properties is given in Table 7-6. Note that there is a difference in the number of properties flood in existing conditions than previously reported due to a refined schematisation of the Dunolly Road Bridge.

Table 7-6 Flood affected properties (1% AEP)

	Existing Conditions	Mitigation Package 3	Change
Flooded above floor	17	6	-11
Flooded below floor	75	21	-54
Was above floor, now below			-11
Was below floor, now above			0
Was below floor, now not flooded			-54

The results of the Package 3 modelling indicated that this option achieves its purpose of effectively protecting the town from the local catchment flows, while preventing downstream impacts as a result of greater conveyance in the contour channel. It demonstrated significant improvements in the flood impacts experienced in a 1% AEP flood event.

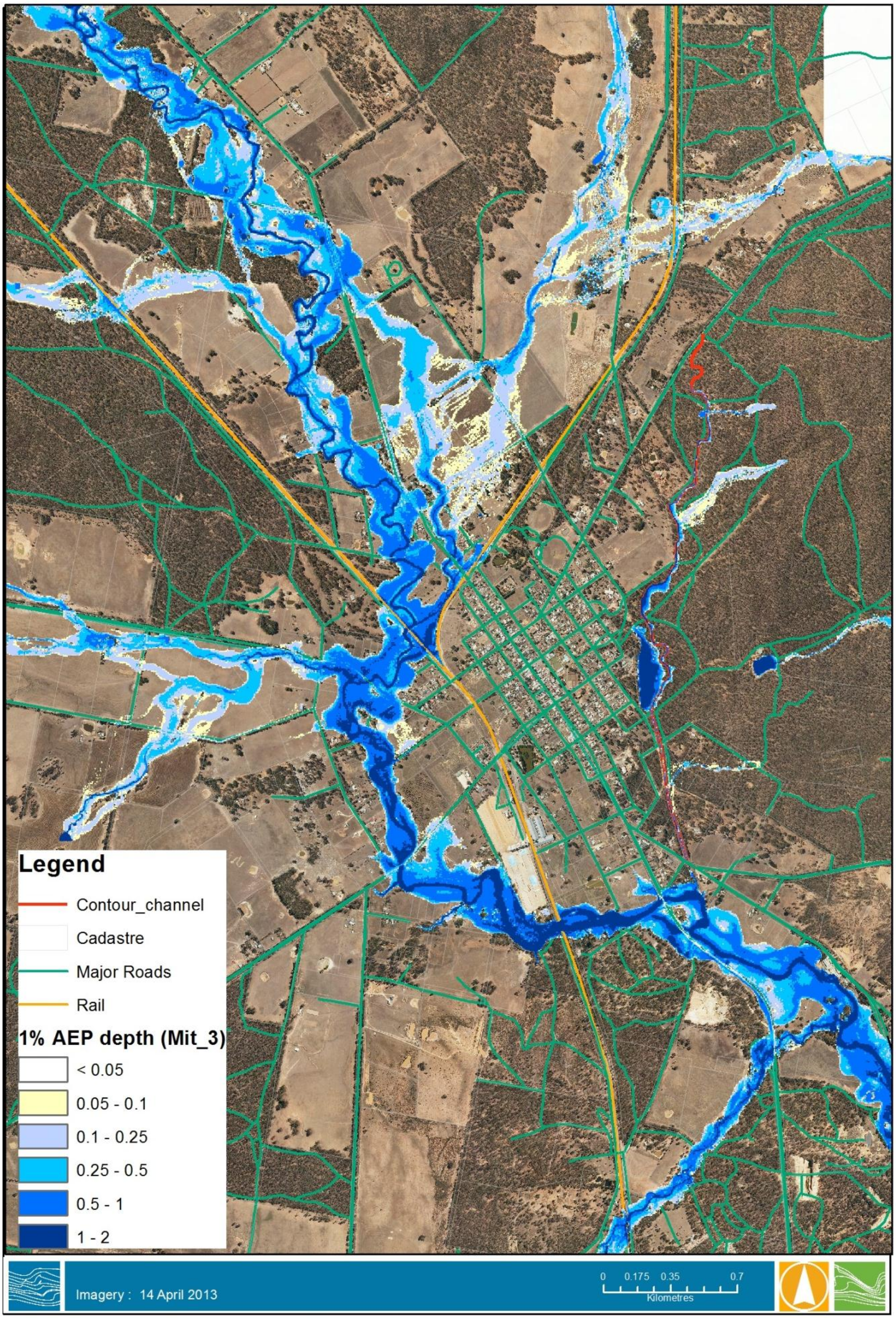


Figure 7-12 Mitigation Package 3 – 1% AEP flood extent

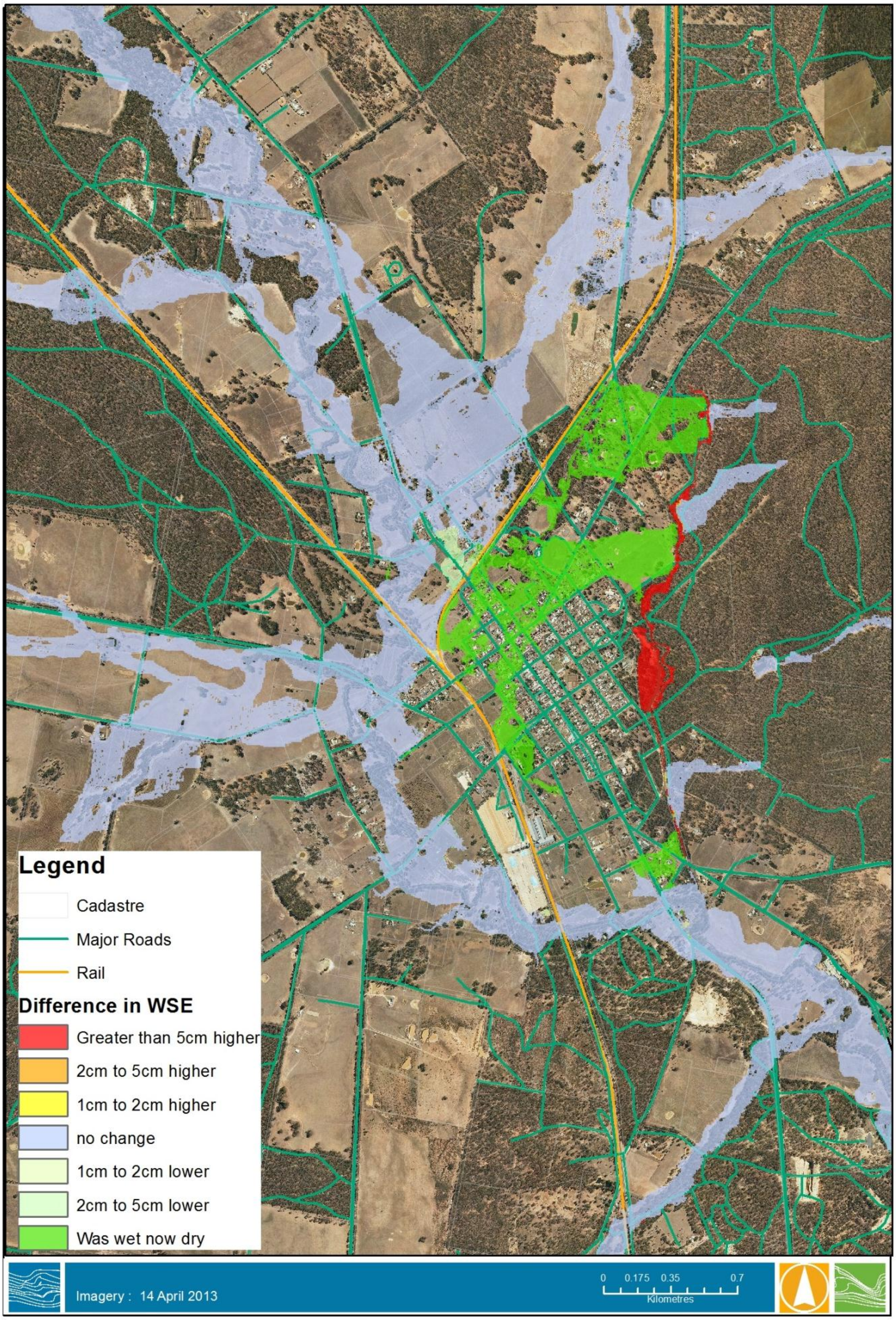


Figure 7-13 Difference in water surface elevation between mitigation package 3 and existing conditions – 1% AEP flood extent

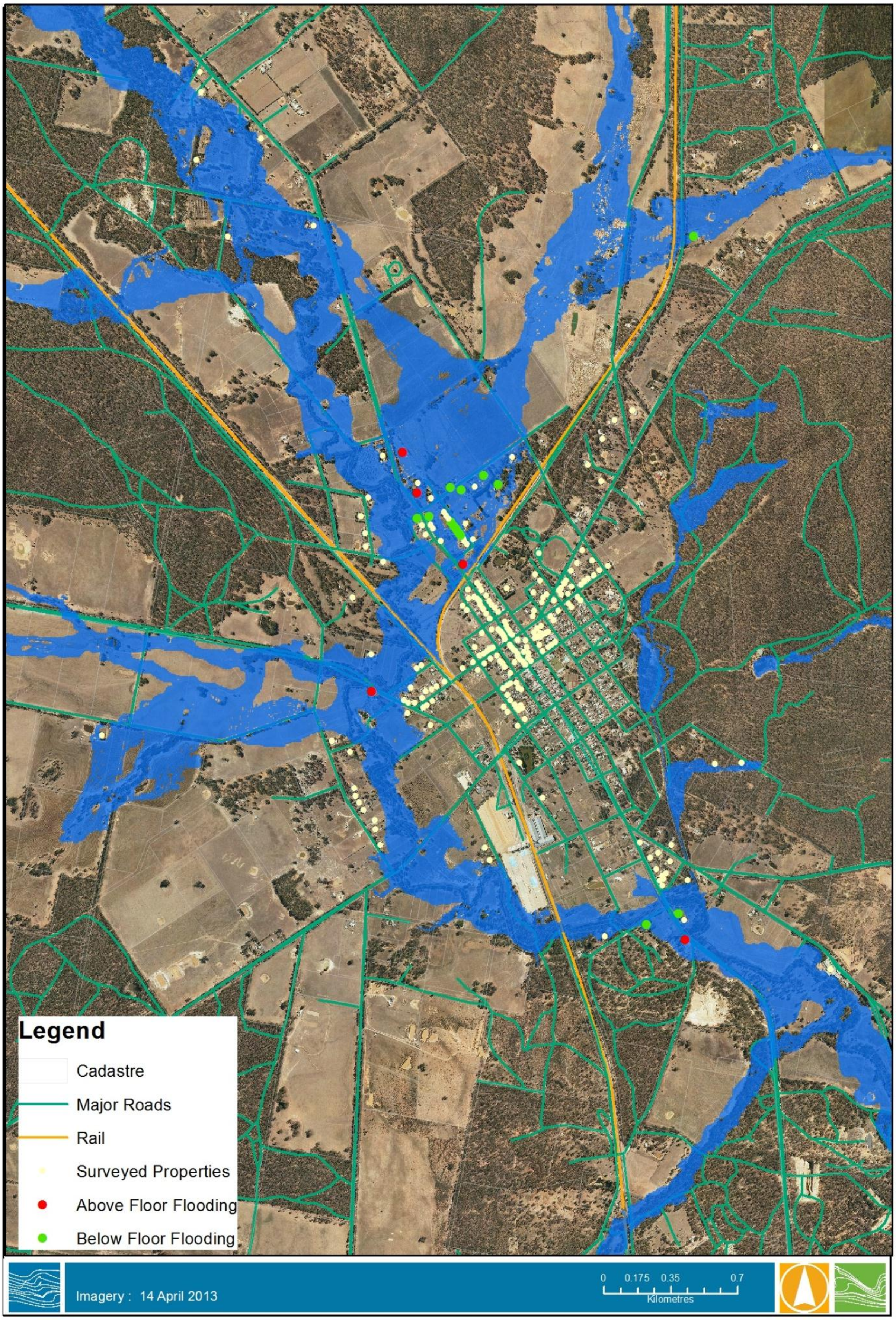


Figure 7-14 Mitigation Package 3 flooded properties – 1% AEP flood extent

7.4 Non Structural Mitigation Options

There are a range of non-structural mitigation options that can be implemented including land use planning, flood warning, flood response and flood awareness.

7.4.1 Land Use Planning

The Victoria Planning Provisions (VPPs) contain a number of controls that can be employed to provide guidance for the use and development of land that is affected by inundation from floodwaters. These controls include the Floodway Overlay (FO), the Land Subject to Inundation Overlay (LSIO), the Special Building Overlay (SBO), the Urban Floodway Zone (UFZ) and the Environmental Significance Overlay (ESO).

Section 6(e) of the Planning and Environment Act 1987 enables planning schemes to 'regulate or prohibit any use or development in hazardous areas, or likely to be hazardous'. As a result, planning schemes contain State planning policy for floodplain management requiring, among other things, that flood risk be considered in the preparation of planning schemes and in land use decisions.

Guidance for applying flood controls to Planning Schemes is available from the Department of Planning and Community Development's (DPCD) Practice Note on Applying Flood Controls in Planning Schemes.

Planning Schemes can be viewed online at http://services.land.vic.gov.au/maps/pmo.jsp. It is recommended that the planning scheme for Dunolly be amended to reflect the flood risk identified by this project. Figure 7-16 shows proposed FO and LSIO for consideration into such an amendment. The draft planning scheme map is based on the 'Advisory Notes for Delineating Floodways' (NRE, 1998), with three approaches considered.

Flood frequency - Appendix A1 of the advisory notes suggest areas which flood frequently and for which the consequences of flooding are moderate or high, should generally be regarded as floodway. The 10% AEP flood extent was considered an appropriate floodway delineation option for Dunolly.

Flood hazard - Combines the flood depth and flow speed for a given design flood event. The advisory notes suggest the use of Figure 7-15 for delineating the floodway based on flood hazard. The flood hazard for the 1% AEP event was considered for this study.

Flood depth - Regions with a flood depth in the 1% AEP event greater than 0.5 m were considered as FO based on the flood depth delineation option.

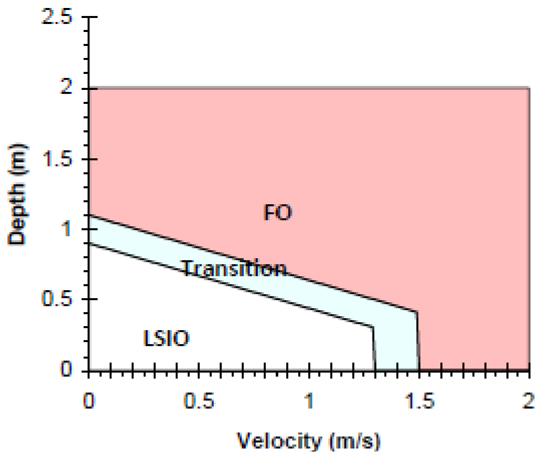


Figure 7-15 Flood Hazard Delineation of FO

All three of the above flood frequency, hazard and depth maps were enveloped to provide the final proposed FO maps as shown in Figure 7-16 below.

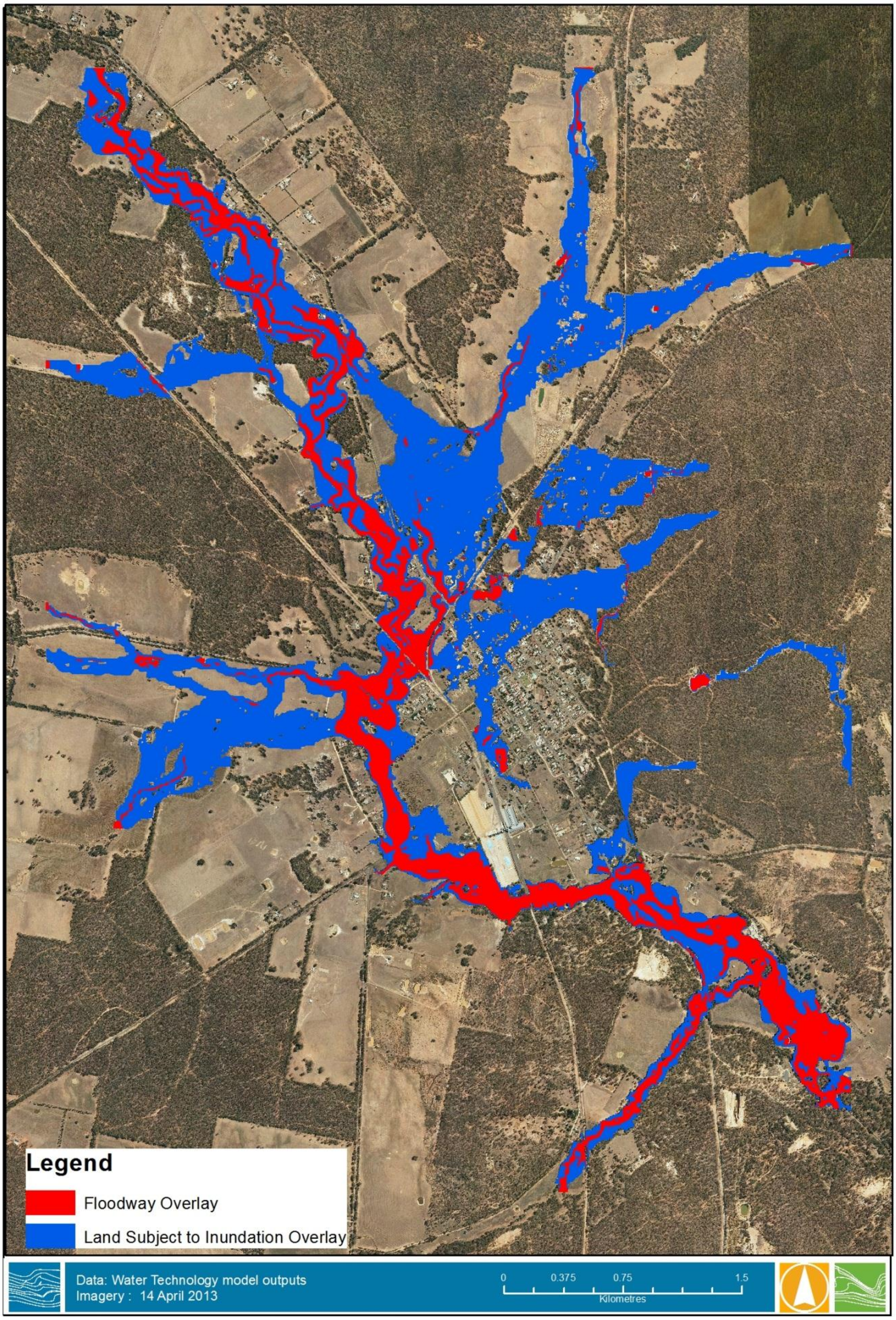


Figure 7-16 Draft LSIO and FO Map for Existing Conditions

7.4.2 Flood Warning, Response and Awareness

Flood Warning

There is currently no flood warning service provided by the Bureau of Meteorology at Dunolly, and given the short available warning time the Bureau would most likely classify this as flash flooding so would not be covered under the traditional flood warning service. The likely warning time available would place Dunolly somewhere between a flash flood warning and a traditional riverine flood warning service. We have assumed flash flood warning will be applied to Dunolly given current trends across other studies. The Flood Warning Arrangements for Victoria (VFWCC, 2001) report outlines the following principles for flash flood warning services:

- The Bureau of Meteorology has a responsibility to provide predictions of weather conditions likely to lead to flash flooding (e.g. thunderstorms);
- Local Government has prime responsibility for flash flood warning extending from system establishment and operation through to the provision of predictions of stream levels if required; and
- The Bureau of Meteorology will provide specialist technical assistance and advice to Local Government to assist in system establishment and in relation to flood prediction techniques.

This means that any flood warning system considered for Dunolly would be the responsibility of Central Goldfields Shire Council, with the Bureau of Meteorology providing assistance in the development of the system and the supply of software, as well as the supply of severe weather warnings and flood watches.

Any flood warning system should consider the eight building blocks of a flood warning system, these include:

- Data collection and collation
- Detection and prediction
- Interpretation
- Message construction
- Message dissemination
- Response
- Review
- Awareness

Failure to consider any one of these building blocks will considerably reduce the effectiveness of any flash flood warning system.

Given the relatively low level of flood risk for Dunolly during flood events, it is suggested that a very basic flood warning tool could be used to provide an indication of how various combinations of rainfall depths over different periods will translate into flooding at Dunolly. This tool can then provide a link between rainfall and the flood maps generated as part of the study. A tool of this nature was developed for Dunolly by Mike Cawood and Associates and is included in the Municipal Flood Emergency Plan.

Flood Response

The information and understanding gathered during this project regarding the flood behaviour at Dunolly for a range of events is critical to capture in order to improving the flood response at Dunolly. This includes areas that are likely to be impacted by floods of various magnitudes, the timing and behaviour of flooding through town, areas most at risk, identifying vulnerable communities, access and egress issues, buildings inundated above and below floor, areas that need to be evacuated as a priority, etc. This information has been summarised in the Municipal Flood Emergency Management Plan. It is suggested that a gauge board be installed at an appropriate location in town so that the outputs from this study can be tied back to a common gauge level. An appropriate location for a gauge board may be at Burnt Creek Lane or for easier access on the rail bridge.

Flood Awareness

A flood awareness and flood ready community stands a much better chance of reducing their flood damage than a community that is not aware of the flood risk before an event. There are many misconceptions commonly held regarding flooding that may prevent a person from preparing to and then evacuating prior to the arrival of a flood. A strong community awareness campaign will reduce these misconceptions, it will never eliminate them entirely, but it will ensure that a greater percentage of the community is aware and ready to act when a flood is imminent.

Flood awareness can be improved by making this study available to the public, as well as more condensed brochure style documents that clearly explain the risk and what is being done about it by the relevant agencies, but more importantly what individuals can do to best prepare themselves. Establishing an active community group that promotes flood related issues in the community, this can be run in conjunction with a more formal program such as VICSES' FloodSafe program. Installing flood markers of historic or potential design floods in suitable locations. This may include a town gauge board that may be part of a flash flood warning system, or at least linked to the outputs from this study in the flood response plans. Individual property flood intelligence cards have been prepared for some communities in Victoria. These generally link a flood level at a gauge to the commencement of flooding on the specific property, and the level at which above floor flooding is likely to occur, they also provide basic flood information including contact details and at what level on the gauge they should consider evacuating.

8. FLOOD DAMAGE ASSESSMENT

8.1 Overview

A flood damages assessment was undertaken for the study area under existing conditions. The flood assessment determined the monetary flood damages for design floods (20%, 10%, 5%, 2%, 1% and 0.5% AEP events). The flood damage assessment was also undertaken for the final mitigation package.

Water Technology has developed an industry best practice damage assessment methodology that has been utilised for a number of studies in Victoria, combining aspects of the Rapid Appraisal Method, ANUFLOOD, more recent damage curves from the NSW Office of Environment and Heritage and other relevant flood damage literature. The model results for all mapped flood events were processed to calculate the numbers and locations of properties affected. This included properties with buildings inundated above floor, properties with buildings inundated below floor and properties where the building was not impacted but the grounds of the property were. In addition to the flood affected properties, lengths of flood affected roads for each event were also calculated. Details of the flood damage assessment methodology are provided in Appendix D.

8.2 Existing Conditions

The 1% AEP flood damage estimate for existing conditions was calculated to be $870,000. A total of 65 properties are flooded in a 1% AEP event, with 17 of those properties flooded above floor level. The January 2011 event is estimated at approximately a 1% AEP event. The total number of properties flooded is consistent with that reported in VICSES rapid impact assessments. The Average Annual Damages (AAD) was determined as part of the flood damage assessment. The AAD is a measure of the flood damage per year averaged over an extended period. The AAD for existing conditions for the study is estimated at approximately **$127,000**. This is effectively a measure of the amount of money that must be put aside each year in readiness for the event that a flood may happen in the future.

Table 8-1 Flood damage assessment for existing conditions

AEP	0.50%	1%	2%	5%	10%	20%
Buildings Flooded Above Floor	23	17	16	10	9	6
Properties Flooded Below Floor	60	48	42	42	30	26
Total Properties Flooded	**83**	**65**	**58**	**52**	**39**	**32**
Direct Potential External Damage Cost						
Direct Potential Residential Damage Cost	$975,477	$643,485	$483,692	$286,379	$267,504	$191,051
Direct Potential Commercial Damage Cost	$38,748	$35,244	$29,584	$18,075	$12,523	$7,107
Total Direct Potential Damage Cost	**$1,014,225**	**$678,728**	**$513,276**	**$304,455**	**$280,028**	**$198,158**
Total Actual Damage Cost *(0.8*Potential)*	**$811,380**	**$542,983**	**$410,621**	**$243,564**	**$224,022**	**$158,526**
Infrastructure Damage Cost	**$231,892**	**$200,131**	**$170,952**	**$141,632**	**$105,859**	**$79,458**
Indirect Clean Up Cost	$125,794	$90,668	$84,813	$49,686	$46,046	$30,698
Indirect Residential Relocation Cost	$15,163	$10,374	$9,576	$4,788	$4,788	$3,192
Indirect Emergency Response Cost	$33,463	$25,097	$20,078	$15,058	$10,039	$10,039
Total Indirect Cost	**$174,420**	**$126,139**	**$114,467**	**$69,533**	**$60,873**	**$43,928**
Total Cost	**$1,217,691**	**$869,253**	**$696,040**	**$454,728**	**$390,755**	**$281,912**
Average Annual Damage (AAD)	$127,363					

Note that the number of properties inundated reported in Section 7.3.3 is slightly different to the numbers presented in this section. This is a results of a depth tolerance applied in the damages analysis as it was felt that the damage cost as a result of urban flooding was being over estimated due to the minor urban drainage infrastructure not being modelled in the riverine flood model. The depth used to calculate the external damage is the depth at the centre of the property, rather than the maximum parcel depth. This is also the case for the reported number of properties inundated as reported in Table 8-2.

8.3 Preferred Mitigation Option

The AAD for the preferred mitigation option (Package 3) was calculated to be approximately **$30,000**. During a 1% AEP event, the preferred option reduces the total number of properties inundated above floor level from 17 properties to 6 properties. Over a long period of time with a range of flood events, the AAD may be reduced by approximately **$97,000** per year by implementing mitigation package 3.

Table 8-2 Flood damage assessment for mitigation package 3

AEP	0.50%	1%	2%	5%	10%	20%
Buildings Flooded Above Floor	17	6	5	1	0	0
Properties Flooded Below Floor	32	14	11	9	4	4
Total Properties Flooded	**49**	**20**	**16**	**10**	**4**	**4**
Direct Potential External Damage Cost						
Direct Potential Residential Damage Cost	$624,924	$287,135	$133,156	$0	$0	$0
Direct Potential Commercial Damage Cost	$33,334	$18,272	$13,675	$3,656	$0	$0
Total Direct Potential Damage Cost	**$658,258**	**$305,407**	**$146,831**	**$3,656**	**$0**	**$0**
Total Actual Damage Cost *(0.8*Potential)*	**$526,606**	**$244,325**	**$117,465**	**$2,925**	**$0**	**$0**
Infrastructure Damage Cost	**$170,643**	**$125,050**	**$105,691**	**$81,730**	**$55,665**	**$40,234**
Indirect Clean Up Cost	$88,453	$32,912	$27,058	$3,640	$0	$0
Indirect Residential Relocation Cost	$9,576	$3,990	$3,192	$0	$0	$0
Indirect Emergency Response Cost	$33,463	$25,097	$20,078	$15,058	$10,039	$10,039
Total Indirect Cost	**$131,492**	**$61,999**	**$50,327**	**$18,698**	**$10,039**	**$10,039**
Total Cost	**$828,741**	**$431,375**	**$273,483**	**$103,353**	**$65,704**	**$50,273**
Average Annual Damage (AAD)	$29,893					

8.4 Non-Economic Flood Damages

The previous discussion relating to flood damages has concentrated on monetary damages, that is, damages that are easily quantified. In addition to those damages, it is widely recognised that individuals and communities also suffer significant non-monetary damage, i.e. emotional distress, health issues, etc. There has been extensive research undertaken and documented in the scientific literature relating to the individuals and communities response to natural disasters. A recent publication entitled *"Understanding floods: Questions and Answers"* by the Queensland Floods Science Engineering and Technology Panel, when discussing the large social consequences floods have on individuals and communities states:

Floods can also traumatise victims and their families for long periods of time. The loss of loved ones has deep impacts, especially on children. Displacement from one's home, loss of property and disruption to business and social affairs can cause continuing stress. For some people the psychological impacts can be long lasting.

The *"Disaster Loss Assessment Guidelines"* (EMA, 2002) make the following key points:

- *Intangibles are often found to be more important than tangible losses.*
- *Most research shows that people value the intangible losses from a flooded home—principally loss of memorabilia, stress and resultant ill-health—as at least as great as their tangible dollar losses.*
- *There are no agreed methods for valuing these losses.*

There is no doubt that the intangible non-monetary flood related damage in Dunolly is high. The benefit-cost analysis presented later in this report has not considered this cost. Any decisions made that are based on the benefit-cost ratios need to understand that the true cost of floods in Dunolly is far higher than the economic damages alone. This would have the effect of increasing the benefit cost ratio, improving the argument for approving a mitigation scheme at Dunolly.

It should also be noted the damages do not include the cost of road closures and repairs for roads outside of the modelled study boundary.

9. BENEFIT COST ANALYSIS

9.1 Overview

A benefit cost analysis was undertaken to assess the economic viability of the preferred mitigation option. Indicative benefit-cost ratios were based on the construction cost estimates and average annual damages. For the analysis, a net present value model was used, applying a 6% discount rate over a 30 year project life.

9.2 Mitigation Option Costs

The mitigation works were costed based on a number of key references:

- Melbourne Water's standard rates for earthworks and pipe/headwall construction costs.
- Rawlinsons Australian Construction Handbook Rates
- Comparison to cost estimates for similar mitigation works for other flood studies

Detailed costing was only carried out for mitigation package 3, as options in package 1 and 2 were found to be infeasible based on their increased flooding impacts at some properties.

The principal cost estimates for mitigation package 3 are the earthworks associated with improving the capacity of the contour channel (i.e. repairing channel banks and increasing channel bank heights in some areas) and rock chutes at the inlet and outlet of the retarding basin. The cost has been based on the estimated volume of excavation and fill.

A 30% contingency cost has been added along with engineering and administration costs. An annual maintenance cost of 2% of the works was also factored in for the channel and retarding basin works.

Note that the vegetation removal cost does not include native vegetation offsets, as it is thought that this will not be a significant cost to the project. It is recommended that a net gain assessment be undertaken during the detailed design phase to validate this assumption.

A summary of the costing can be seen in Table 9-1.

Table 9-1 Package 3 Mitigation Option Cost Breakdown

Option	Total Construction Cost	Annual Maintenance
Upgrade contour channel	$220,000	$4,400
Retarding basin	$36,000	$1,000
Vegetation Removal	$130,000	$,2600
TOTAL	**$386,000**	**$8,000**

9.3 Benefit-Cost Ratio

A benefit-cost analysis was undertaken for the preferred mitigation option (package 3). The ratio is based on a CPI of 6% over a 30 year period. The resulting ratio is significantly high, strongly justifying an upgrade of the contour channel. This reflects the fact that the contour channel was historically built for a very good reason, and is currently protecting the town from flooding. The channel is falling into disrepair, and for relatively minor expenditure could be upgraded to improve its performance, securing Dunolly's flood protection into the future.

Table 9-2 Benefit Cost Analysis

	Existing Conditions	Mitigation Package 3
Average Annual Damage	$127000	$30,000
Annual Maintenance Cost		$8,000
Annual Cost Saving		$89,000
Net Present Value (6%)		$1,252,000
Capital Cost of Mitigation		$386,000
Benefit – Cost Ratio		3.2

10. PROJECT CONSULTATION

10.1 Overview

A key element in the development of the Dunolly Flood Study was the active engagement of community members. This engagement was developed over the course of the study through community consultation sessions, public notices (in the Welcome Record and Maryborough Advertiser) and meetings with a Steering Committee containing community representatives. The community consultation sessions were jointly managed between the North Central CMA and Water Technology. The aims of the community consultation were as follows:

- To raise awareness of the study and to identify key community concerns; and
- To provide information to the community and seek their feedback/input regarding the study outcomes including the existing flood behaviour and proposed mitigation options for the township.

10.2 Steering Committee

The study was led by a Steering Committee consisting of representatives from North Central CMA, Central Goldfields Shire Council, Department of Environment Planning and Infrastructure (DEPI), Bureau of Meteorology (BoM), State Emergency Service (SES), Water Technology and the Dunolly community. Members of the Steering Committee and their respective organisations were as follows:

- Cr Barry Rinaldi (Central Goldfields Shire Council)
- Cr John Smith (Central Goldfields Shire Council)
- Cr Bob Henderson (Central Goldfields Shire Council)
- David Sutcliffe (Central Goldfields Shire Council)
- Sonny Neale (Central Goldfields Shire Council)
- Ken Coates (North Central CMA) – Steering Committee Chair
- Adrian Bathgate (North Central CMA)
- Sarah Stanaway (North Central CMA)
- Camille White (North Central CMA)
- Leila Macadam (North Central CMA)
- Peter Daly (Deledio Reserve Committee)
- Fiona Lindsay (Community representative)
- Tony Mullan (SES – Dunolly)
- Gary Lavars (SES – Dunolly)
- Barry Cann (SES – Dunolly)
- Jemma Nesbit-Sackville (VIC SES)
- Simone Wilkinson (DEPI)
- Elma Kazazic (Bureau of Meteorology)
- David Hildebrand (VicRoads)
- Matt Bunney (VicTrack)

The Steering Committee met on 7 occasions at key points throughout the study, to manage the development of the plan.

10.3 Community Consultation

Two formal information sessions were held throughout the course of the study. A drop in session was held on the 26th June 2013 providing an opportunity for interested members of the public to discuss their experiences of flooding in Dunolly and comment on the development of the study.

Community members were largely in agreement with the modelled January 2011 flood extent. Several residents were also able to provide flood related information (in the form of photos, anecdotes and records) which were instrumental in the validation of modelled flood extents.

Community feedback from this initial community information session indicated concern regarding the capacity of the Old Lead Reservoir upstream of Dunolly. As a result of these concerns being raised, the Old Lead Reservoir (which is outside of the study area) was incorporated into the modelling and a sensitivity test undertaken. The reservoir was found to have no impact on flooding within Dunolly for large flood events. The management of the reservoir (which is owned by the Loddon Shire Council) has been discussed with the Steering Committee. An additional concern raised by residents is access in and out of the town during floods. Critical information developed by this study (e.g. timings and flows) has been incorporated into the flood emergency response plan, and will aid in providing improved flood warning and response including evacuation.

At the second (final) community meeting, held 25th June 2014, the proposed mitigation option was presented, and residents were given the opportunity to ask questions and comment on their support (or otherwise). Attendance at the meeting was exceptional, with over 30 residents dropping in over the evening. Six formal responses were received (feedback forms were available on the night), with four indicating support for the proposed mitigation package; one not in support and one neutral response. A lack of response from the remaining attendees has been assumed as in support of or neutral to the proposed mitigation package, as was their indication on the night.

11. CONCLUSIONS AND RECOMMENDATIONS

Following the recent flood events in September 2010 and January 2011, Dunolly was identified as a high flood risk community and funding was approved for a flood investigation of the township. The Dunolly Flood study was run by the North Central CMA in conjunction with Central Goldfields Shire Council.

The study involved the development of a hydrologic model of the Burnt Creek catchment and hydraulic models of the township / study area. The models were successfully verified to the January 2011 flood event, and a number of design flood events were simulated along with the design of potential flood mitigation options.

Throughout the study, a range of community consultation activities were undertaken, including community drop-in sessions, media releases and questionnaires to ensure that community issues were heard and the ideas of the community were considered in the development of potential flood mitigation options. It must be noted that the community participation was very helpful, with flood observations, local information and feedback on the study greatly improving the outcomes for the study.

An initial prefeasibility assessment of 16 structural mitigation options was undertaken. From this assessment, three options were selected for further analysis using the developed hydraulic model. These included a levee near the cemetery; a levee on Broadway Road and an upgrade of the contour channel. The decision was made by the Steering Committee that this study would not recommend individual flood protection.

While the levees were able to improve flooding impacts at small number of properties they increased levels at other properties and were therefore not considered feasible. An upgrade of the contour channel to improve capacity, and the incorporation of a retarding basin is the preferred mitigation option for Dunolly.

The option to upgrade the contour channel and incorporate a retarding basin to slow the rate of flow has returned a very high benefit to cost ratio of 3.2. The contour channel is very important to the flood protection of Dunolly. Substantial flood protection can be provided with a relatively modest investment and upgrade, and ongoing maintenance of the channel.

Regardless of the benefit cost ratio, no option is likely to be considered unless it has the strong support of the community. At the final community meeting, where residents were given the opportunity to ask questions and comment on the proposed mitigation option, there was strong support for the upgrade of the contour channel. .

Following significant consultation with the Dunolly Community, the Dunolly Flood Study Steering Committee recommends the following actions:

- Amendment of the planning scheme for Dunolly to reflect the flood risk identified by this project;
- Mitigation Package 3 (an upgrade of the contour channel and retarding basin) to be submitted for funding for detailed design and construction.
- The updated Municipal Flood Emergency Plan be used during a flood event to improve the emergency response.
- In any future bridge upgrade projects, consideration be given to elevating bridges to provide access during a major flood because currently the town becomes completely isolated by road.
- Installation of a gauge board within town to base future observations on and to tie the flood maps back to the gauge. This gauge could also be linked to a flash flood warning system should that be considered in the future.

APPENDIX A PHOTOS FROM SITE VISIT

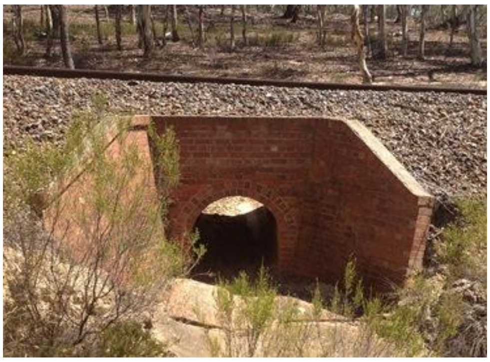
 **01 – Burnt Creek at Burnt Creek Lane**	 **02 – Burnt Creek at Short Street**
 **04 – Tributary at Dunolly-Moliagul Rd**	 **05 – Local drainage railway crossing**
 **06 – Local drainage at Dunolly-Moliagul Rd**	 **07 – Local drainage at Hospital Street**

08 – Burnt Creek railway crossing

09 – Burnt Creek at McKinnon St

10 – Local drainage at McKinnon St

11 – Burnt Creek at Dunolly-Avoca Rd



14a – Burnt Creek railway crossing

14b – Burnt Creek railway crossing

15 – Burnt Creek at Maude St

16 – Burnt Creek at Maryborough-Dunolly Rd

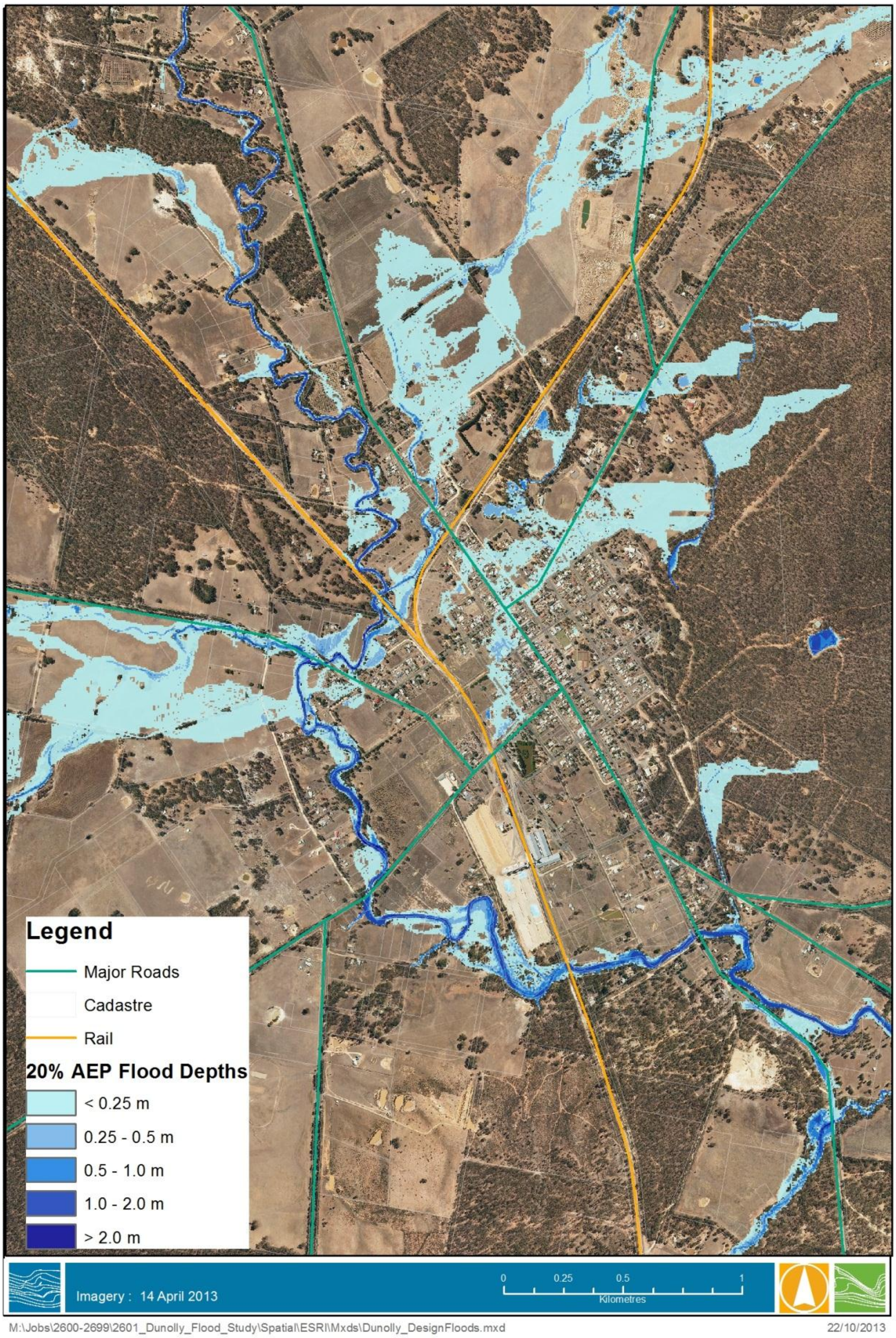
17 – Local drainage at Dunolly Rd

19 – Local drainage railway crossing

Contour channel

Contour channel

APPENDIX B DESIGN FLOOD MAPS (EXISTING CONDITIONS)


Legend
Major Roads
Cadastre
Rail
20% AEP Flood Depths
< 0.25 m
0.25 - 0.5 m
0.5 - 1.0 m
1.0 - 2.0 m
> 2.0 m
Imagery : 14 April 2013
0
0.25
0.5
1
Kilometres
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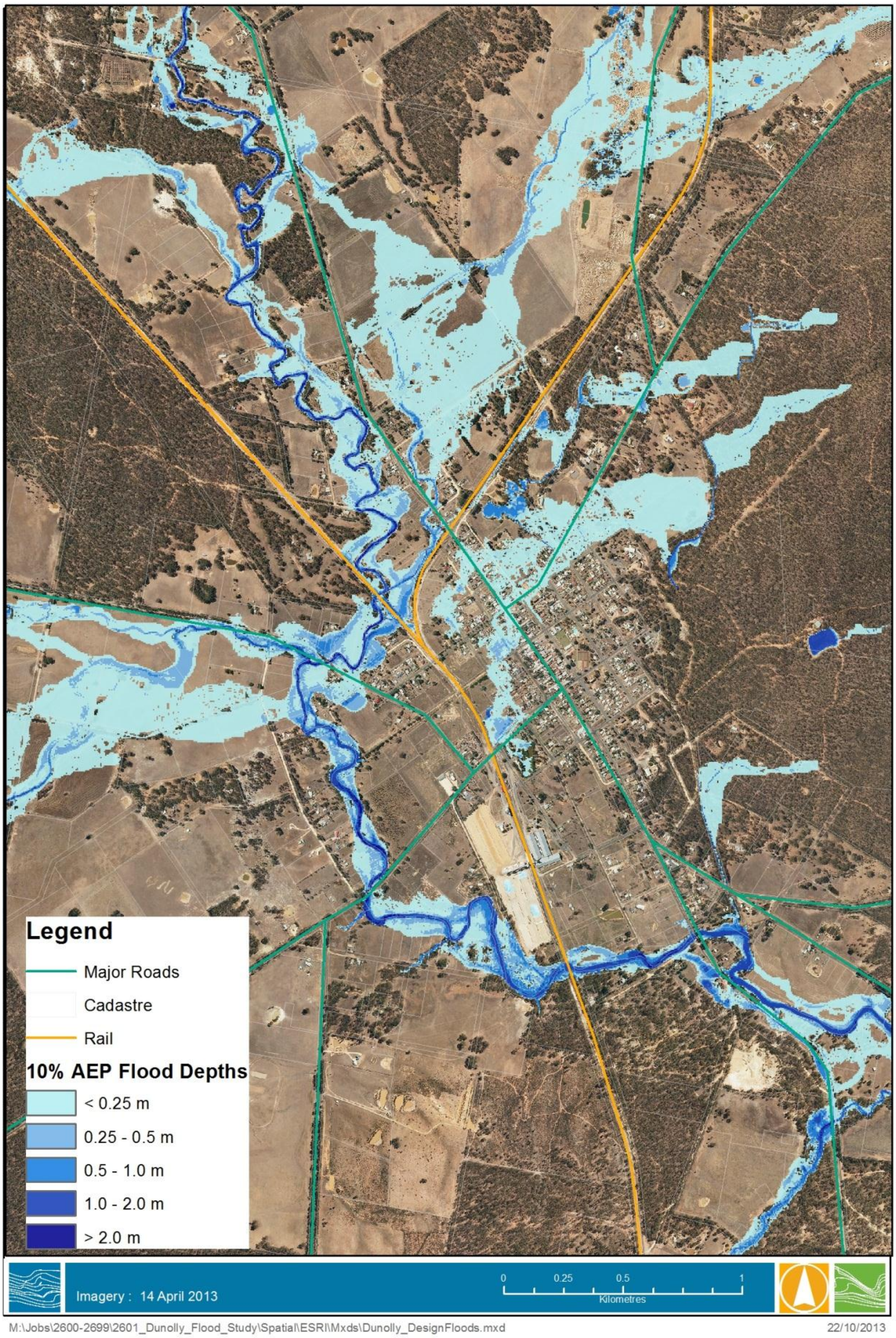
22/10/2013


Legend
Major Roads
Cadastre
Rail
10% AEP Flood Depths
< 0.25 m
0.25 - 0.5 m
0.5 - 1.0 m
1.0 - 2.0 m
> 2.0 m
Imagery : 14 April 2013
0
0.25
0.5
1
Kilometres
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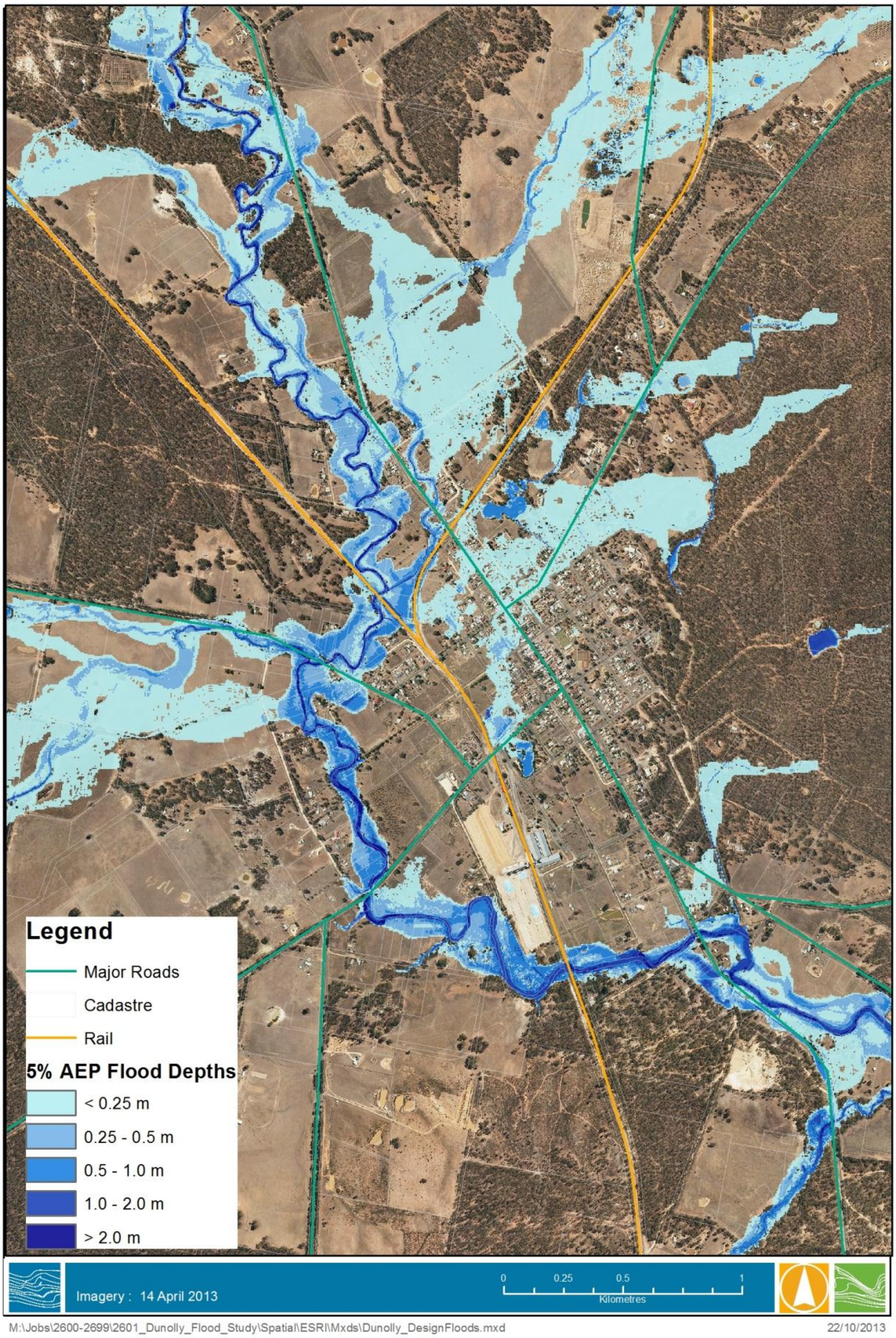
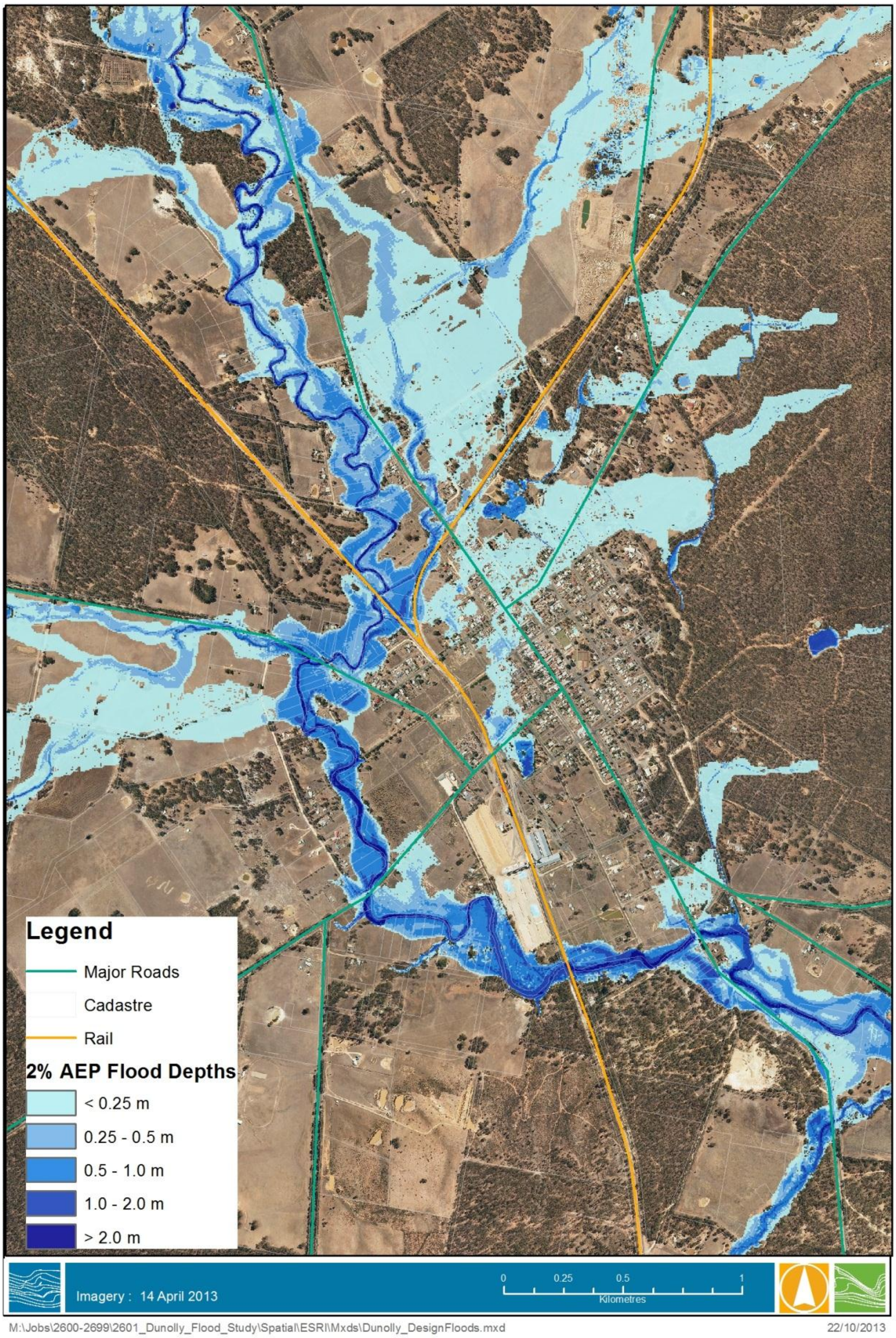
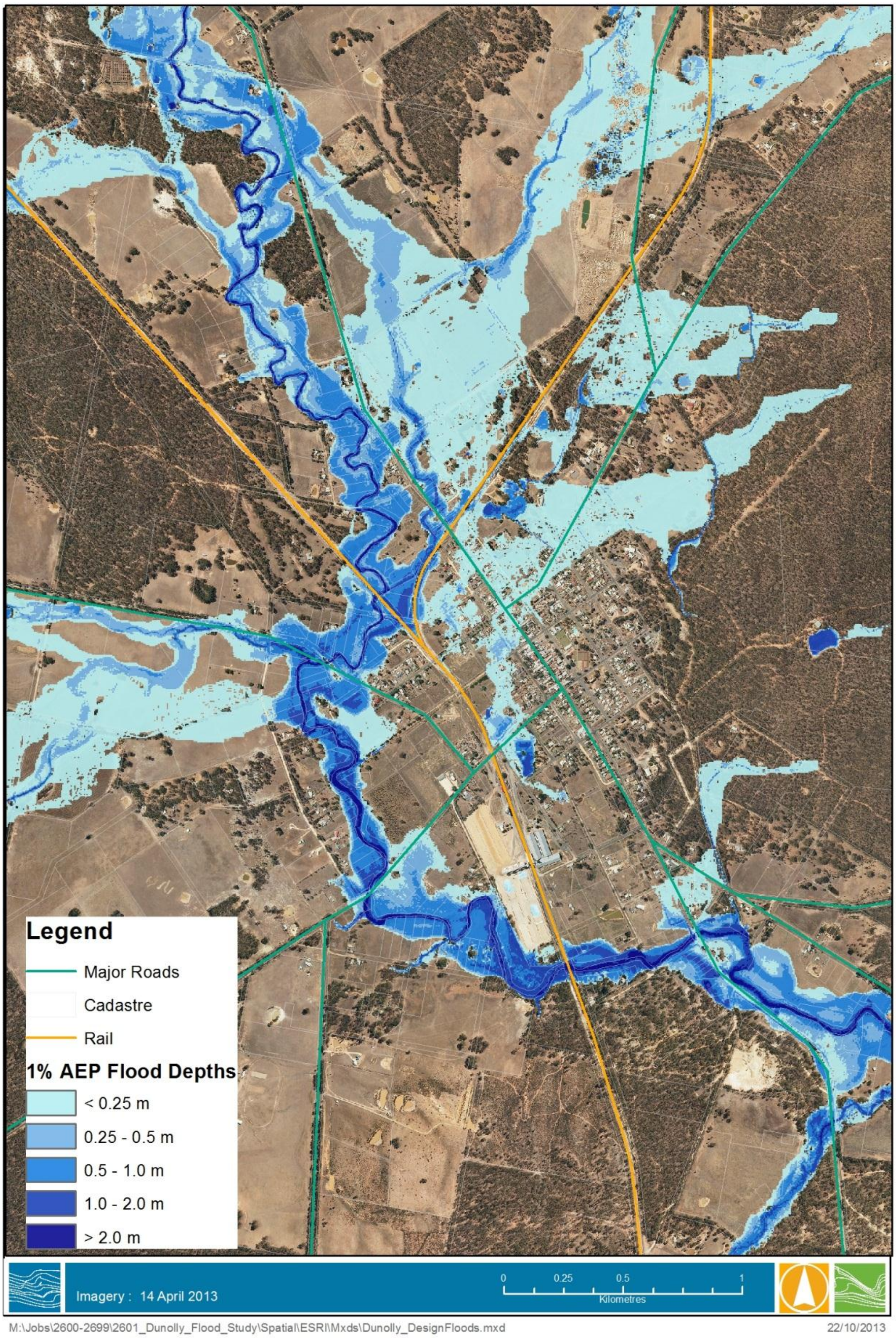
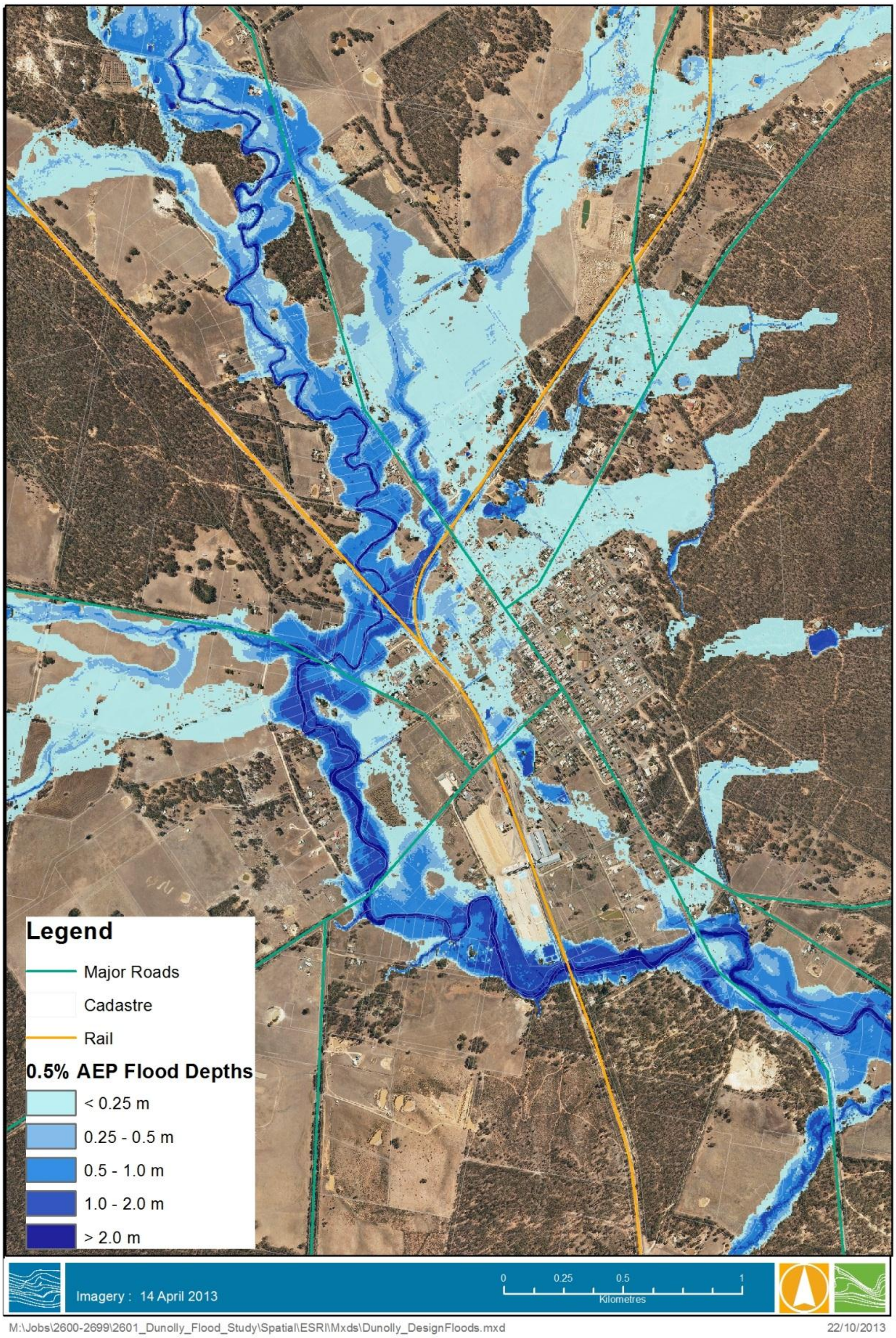
22/10/2013

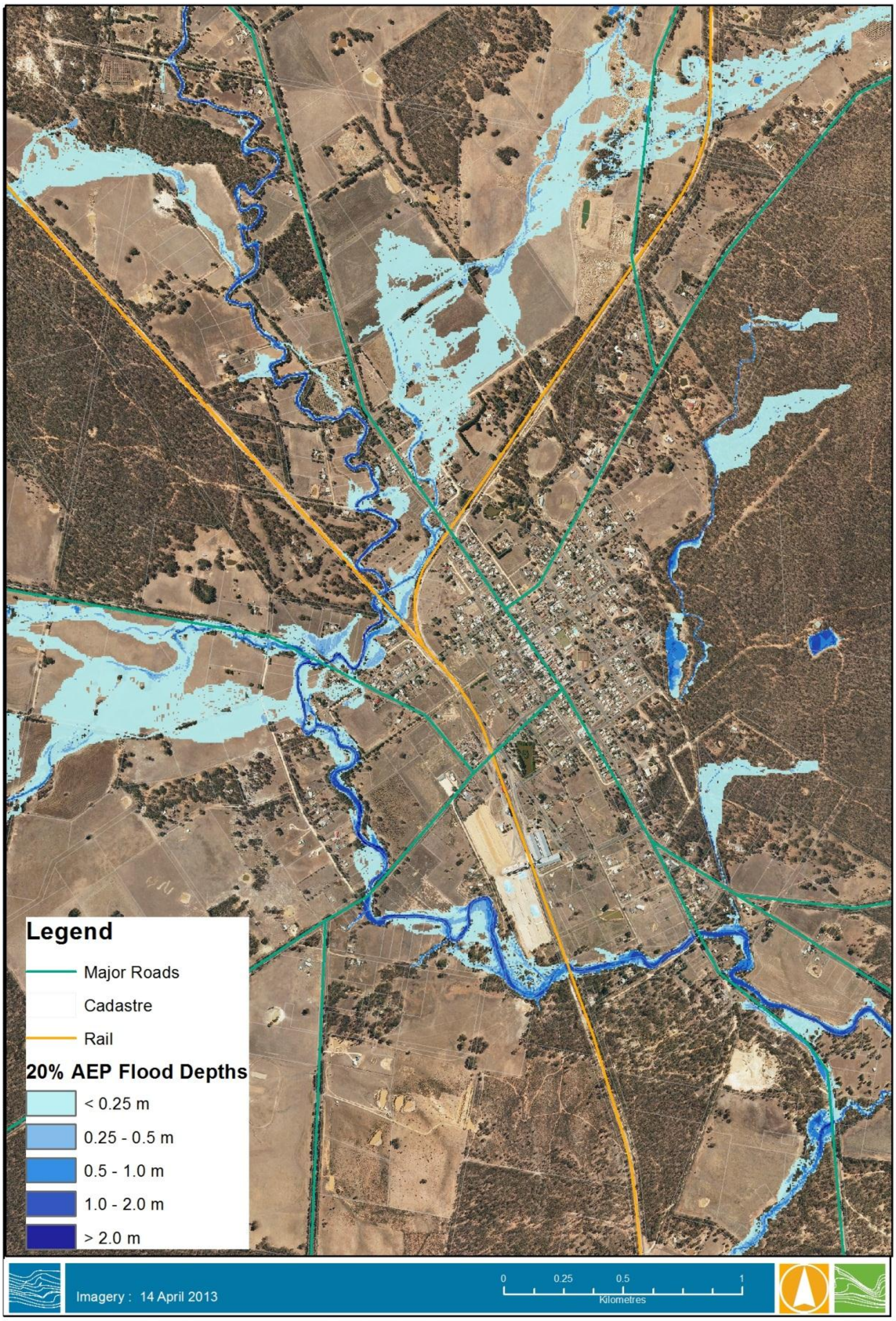

Legend
Major Roads
Cadastre
Rail
5% AEP Flood Depths
< 0.25 m
0.25 - 0.5 m
0.5 - 1.0 m
1.0 - 2.0 m
> 2.0 m
Imagery : 14 April 2013
0 0.25 0.5 1
Kilometres
M:\Jobs\2600-2699\2601_Dunolly_Flood_Study\Spatial\ESRI\Mxds\Dunolly_DesignFloods.mxd
22/10/2013


Legend
Major Roads
Cadastre
Rail
2% AEP Flood Depths
< 0.25 m
0.25 - 0.5 m
0.5 - 1.0 m
1.0 - 2.0 m
> 2.0 m
Imagery : 14 April 2013
0
0.25
0.5
1
Kilometres
M:\Jobs\2600-2699\2601_Dunolly_Flood_Study\Spatial\ESRI\Mxds\Dunolly_DesignFloods.mxd
22/10/2013


Legend
Major Roads
Cadastre
Rail
1% AEP Flood Depths
< 0.25 m
0.25 - 0.5 m
0.5 - 1.0 m
1.0 - 2.0 m
> 2.0 m
Imagery : 14 April 2013
0
0.25
0.5
1
Kilometres
M:\Jobs\2600-2699\2601_Dunolly_Flood_Study\Spatial\ESRI\Mxds\Dunolly_DesignFloods.mxd
22/10/2013


Legend
Major Roads
Cadastre
Rail
0.5% AEP Flood Depths
< 0.25 m
0.25 - 0.5 m
0.5 - 1.0 m
1.0 - 2.0 m
> 2.0 m
Imagery : 14 April 2013
0
0.25
0.5
1
Kilometres
M:\Jobs\2600-2699\2601_Dunolly_Flood_Study\Spatial\ESRI\Mxds\Dunolly_DesignFloods.mxd
22/10/2013

APPENDIX C PACKAGE 3 MITIGATION FLOOD MAPS



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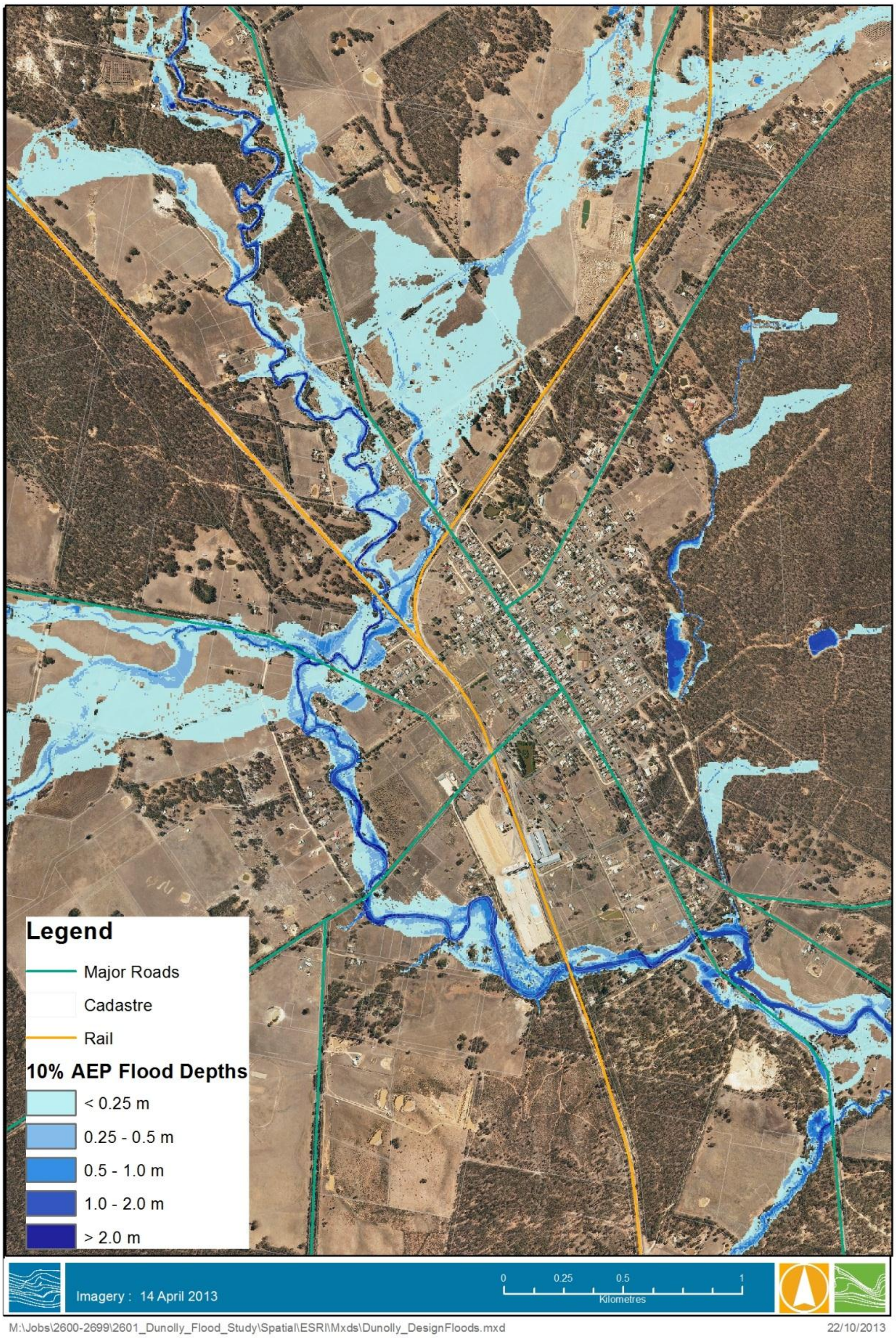
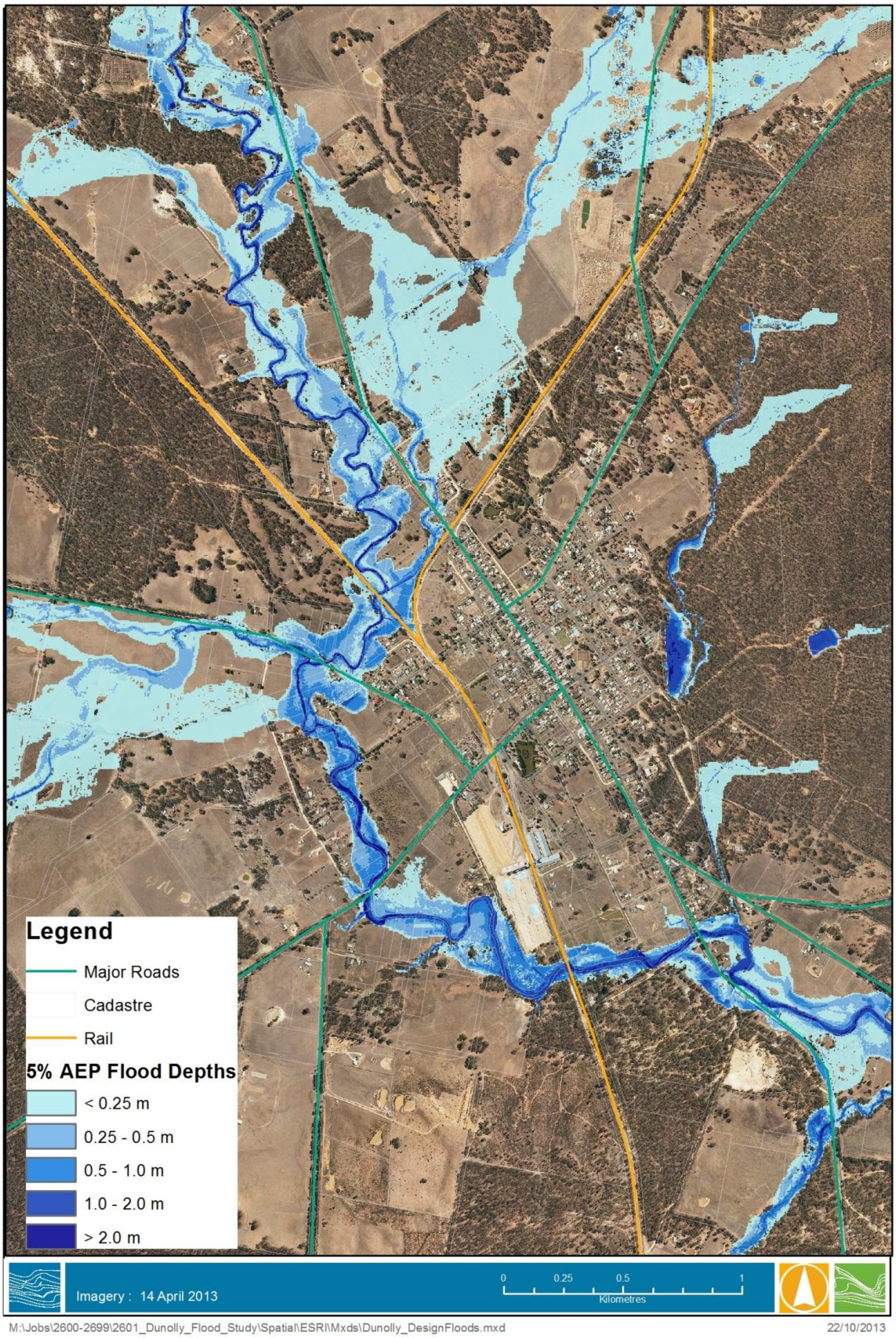
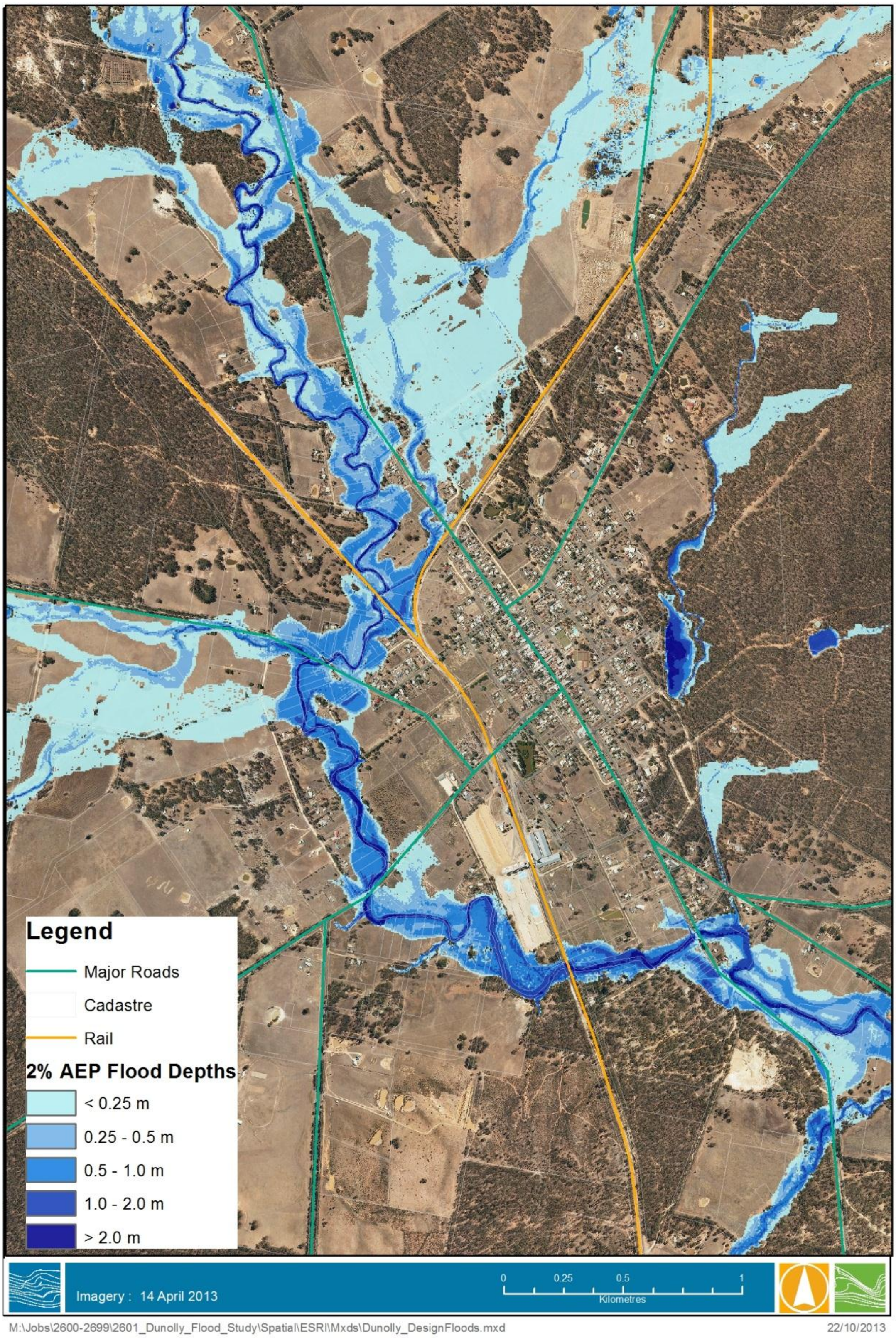
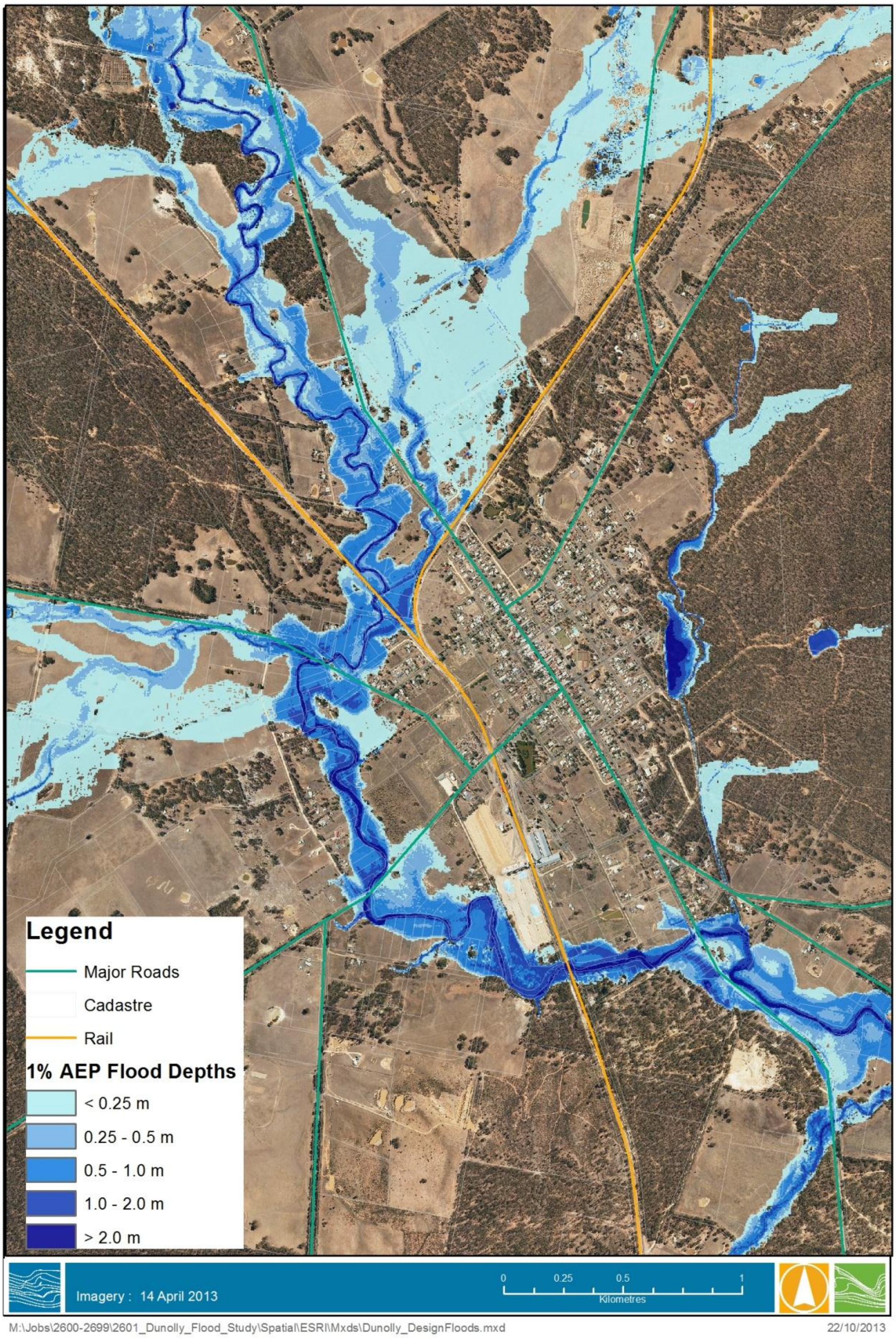
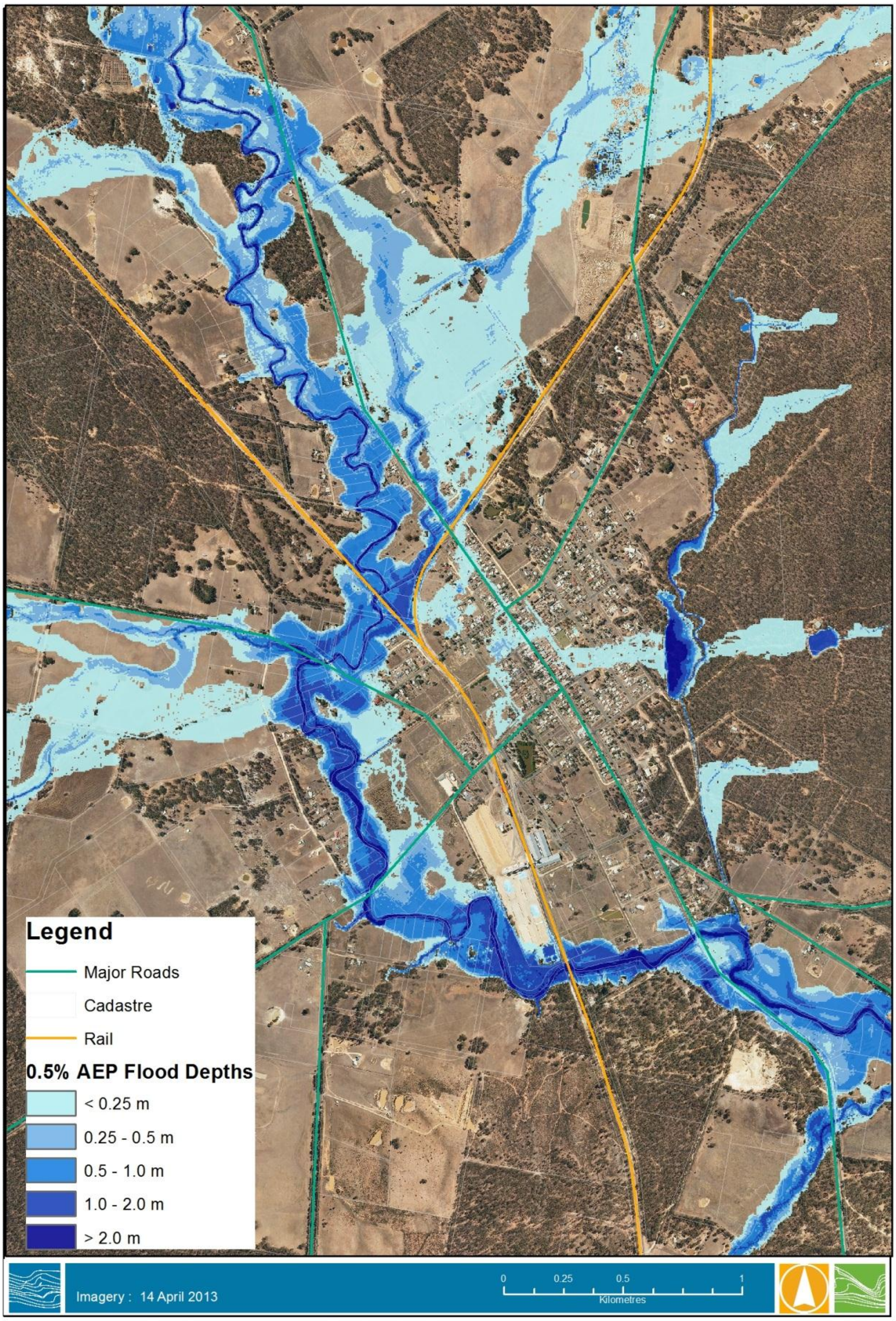

Legend
Major Roads
Cadastre
Rail
10% AEP Flood Depths
< 0.25 m
0.25 - 0.5 m
0.5 - 1.0 m
1.0 - 2.0 m
> 2.0 m
Imagery : 14 April 2013
0
0.25
0.5
1
Kilometres
M:\Jobs\2600-2699\2601_Dunolly_Flood_Study\Spatial\ESRI\Mxds\Dunolly_DesignFloods.mxd
22/10/2013


Legend
Major Roads
Cadastre
Rail
5% AEP Flood Depths
< 0.25 m
0.25 - 0.5 m
0.5 - 1.0 m
1.0 - 2.0 m
> 2.0 m
Imagery : 14 April 2013
0
0.25
0.5
1
Kilometres
M:\Jobs\2600-2699\2601_Dunolly_Flood_Study\Spatial\ESRI\Mxds\Dunolly_DesignFloods.mxd
22/10/2013


Legend
Major Roads
Cadastre
Rail
2% AEP Flood Depths
< 0.25 m
0.25 - 0.5 m
0.5 - 1.0 m
1.0 - 2.0 m
> 2.0 m
Imagery : 14 April 2013
0
0.25
0.5
1
Kilometres
M:\Jobs\2600-2699\2601_Dunolly_Flood_Study\Spatial\ESRI\Mxds\Dunolly_DesignFloods.mxd
22/10/2013


Legend
Major Roads
Cadastre
Rail
1% AEP Flood Depths
< 0.25 m
0.25 - 0.5 m
0.5 - 1.0 m
1.0 - 2.0 m
> 2.0 m
Imagery : 14 April 2013
0
0.25
0.5
1
Kilometres
M:\Jobs\2600-2699\2601_Dunolly_Flood_Study\Spatial\ESRI\Mxds\Dunolly_DesignFloods.mxd
22/10/2013



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APPENDIX D DAMAGES ASSESSMENT METHODOLOGY

Two primary sources for flood damage calculations were used, the original ANUFLOOD cost curves (CRES 1992) and the RAM methodology (Reed Sturgess and Associates (RSA) 2000). Further details on the ANUFLOOD methodology are provided in a guidance report produced by DNR (2002). ANUFLOOD cost curves cover residential and commercial direct costs applicable for townships. The RAM methodology incorporates the ANUFLOOD approach and extends it to include indirect and intangible costs resulting from flooding and provides guidance on costs for agricultural enterprises. A major study of the Economics of Natural Disasters in Australia by the Bureau of Transport Economics (BTE 2001) provides some further information on indirect costs and a recent study by Geoscience Australia (Middelmann-Fernandes 2010) provides information for accounting for the impact of velocity in flood damage assessments. These key references are described below.

- Bureau of Transport Economics (2001). Economic Costs of Natural Disasters in Australia. Report 103. Bureau of Transport Economics, Canberra.
- CRES (1992). ANUFLOOD : A field guide, prepared by D.I. Smith and M.A. Greenaway, Centre for Resource and Environmental Studies, ANU, Canberra.
- Department of Natural Resources and Mines (DNR) (2002). Guidance on assessment of Tangible Flood Damages. Queensland Department of Natural Resources and Mines, September 2002.
- Middelmann-Fernandes, M.H. (2010). Flood damage estimation beyond stage-damage functions: an Australian example. *Journal of Flood Risk Management* 3 (2010): 88-96.
- Reed Sturgess and Associates (2000). Rapid Appraisal Method (RAM) for floodplain management. May 2000. Report prepared for the Department of Natural Resources and Environment.

Before any stage damage curves from the literature were applied in the Rochester flood damage assessment they were adjusted to today's value by scaling using a ratio of today's CPI and the CPI at the time of development of the stage-damage curve. A number of stage damage curves are included below, representing the value at the time of development (i.e. no CPI adjustment).

This appendix does not include a detailed methodology of how the damage assessment was carried out but does include the majority of the source data sets that were used in the development of the methodology.

Table D1 Above floor level stage damage relationships for residential properties (from ANUFLOOD 1992; reproduced from DNR 2002)

		Small house (< 80 m2)	Medium house (80 – 140m2)	Large house (> 140m2)
Depth over flood level	0 m	$905	$2 557	$5 873
	0.1 m	$1 881	$5 115	$11 743
	0.6 m	$7 370	$13 979	$25 351
	1.5 m	$17 379	$18 585	$32 276
	1.8 m	$17 643	$18 868	$32 768

Table D2 Size categories for commercial properties (from ANUFLOOD 1992; reproduced from DNR 2002)

Size category	Guideline
Small	< 186 m2
Medium	186 – 650 m2
Large	650 m2

Table D3 ANUFLOOD Commercial properties cost curve (reproduced from DNR 2002)

	Small commercial properties (<186m²)					Medium commercial properties (186-650m²)					Large commercial properties (>650m²)*				
Value class	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
0.25	$2 202	$4 405	$8 809	$17 618	$35 237	$6 975	$13 948	$27 896	$55 791	$111 583	$7	$15	$32	$61	$122
0.75	$5 506	$11 011	$22 023	$44 046	$88 092	$16 884	$33 768	$67 537	$135 074	$270 147	$39	$78	$154	$308	$619
1.25	$8 258	$16 518	$33 034	$66 069	$132 137	$25 693	$51 387	$102 773	$205 574	$411 094	$81	$162	$326	$649	$1297Ł
1.75	$9 176	$18 352	$36 705	$73 410	$146 819	$28 445	$56 893	$113 785	$227 570	$455 140	$132	$267	$533	$1065	$2129
2	$9 726	$19 454	$38 907	$77 814	$155 628	$30 281	$60 564	$121 126	$242 252	$484 504	$159	$318	$636	$1 272	$2 545

* units of $/m²

Table D4 External / below floor damage per building (from DPIE Floodplain Management in Australia (1992))

Depth above ground (m)	External Damage ($)
0	0
0.065	0
0.26	$1 833
0.5	$4 000
0.75	$6 166
1	$8 333
2	$8 333

Table D5 Unit damages for roads and bridges (per kilometre of road inundated) (From DNR 2002)

	Initial road repair ($)	Subsequent accelerated deterioration of roads ($)	Initial bridge report and subsequent increased maintenance ($)	Total cost to be applied per km of road inundated ($)
Major sealed road	34, 860	17 430	11 985	64 275
Minor sealed road	10 895	5 450	3 815	20 160
Unsealed road	4 900	2 450	1 740	9 090

Table D6 Actual to Potential Damages Ratio from RAM (RSA 2002)

	Actual to Potential Damages Ratio	
Warning time (hrs)	Past Flood Experience	No Flood Experience
0	0.8	0.9
2	0.8	0.8
7	0.6	0.8
12	0.4	0.8
12	0.4	0.7
96	0.4	0.7

Table D7 Indirect costs following BTE (1999)

Indirect damages	Cost ($)	Note
Clean-up costs per Residential property		
-cost of materials	$330	
-cost of labour (40 hours)	$1,102	This is the 2007 average weekly wage from ABS
Clean-up costs per Commercial property		
-total cost to clean up	$2,400	
Alternative Housing per Residential property		
-relocation of household items	$53	
-alternative accommodation	$473	Based on 2.6 ppl per household & 7 nights
Emergency Response Costs		
-cost of labour	$4,000 - $20,000	Different magnitude events require different responses