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WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Warragul Flood Study & Modelling Project

Project Report

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PROJECT DETAILS

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Today - February 7, 2011" http://www.abc.net.au/worldtoday/content/2011/s3131682.htm.

GLOSSARY OF TERMS

Annual Exceedance Probability (AEP)	Refers to the probability or risk of a rainfall event of a given magnitude (intensity and duration) occurring or being exceeded in any given year. A 90% AEP event has a high probability of occurring or being exceeded; it would occur quite often and would be a relatively minor rainfall event. A 1% AEP event has a low probability of occurrence or being exceeded; it would be rare but it would be likely to cause extensive damage.
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level. Introduced in 1971 to eventually supersede all earlier datums.
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. Generally 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.
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 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	Those 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.
TUFLOW	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 for 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.

EXECUTIVE SUMMARY

The overall project objective of this study was to improve Baw Baw Shire Council's understanding of the hydraulics and hydrology of the Warragul drainage catchment. The project study area was broadly covered the Warragul Urban Growth Boundary, but in situations where the storm water catchment extended beyond the Growth Boundary this area beyond the Growth Boundary was considered.

Data was requested from a group of stakeholders identified by BBSC and Water Technology with the following data provided for review in this study:

Table A-1 **Data provided by stakeholders**

Stakeholder	Data provided
WGCMA	Spatial Data – LiDAR topographic data (1m DEM)
Gippsland Water	Report - Warragul Wastewater System Plan (May 2011) Spatial Data – GIS layers representing Facilities, Manholes and Sewer Pipes
VicRoads	Survey Data – Used to validate LiDAR data
BoM	Flood History – Historical flood warnings 1999 -2012
BBSC	Spatial Data – VicMap, aerial images (2), pit and pipe network data Historical reporting, modelling and results – *Hazel and Spring Creeks, Hydrology and Hydraulics Review,* (Earthtech 2004), *Waterford Rise Estate (Warragul North West) – Surface Water Management Strategy (Version 3 Final),* Neil M Craigie Pty Ltd (2010), *Warragul Urban Drainage Strategy,* Sinclair Knight Merz (SKM 2007) *Rural City of Warragul – Hazel and Spring Creek Drainage Study* (CMPS&F 1994)
Water Technology	Historical reporting, modelling and results

The study was broken into 3 focus areas /tasks:

Hydrological Modelling (RORB) – Improve Existing Basin Performance

A previous SKM RORB model (SKM 2007) included 58 sub-catchments covering the Hazel and Spring Creek systems to the confluence of the Hazel Creek, and the Moe River East of Warragul near Bloomfield Road. Specific regional parameters used in the 2012 RORB modelling were adopted directly from the SKM (2007) RORB modelling following review by Water Technology.

After a thorough review of the SKM RORB model it was deemed to provide a quality representation of the Hazel Creek and Spring Creek catchment hydrology. In the absence of quality calibration data the model applied appropriate catchment parameters which produce results close to the empirical peak flow estimations. After the RORB model review Water Technology put forward some recommended system conditions and input changes which would likely increase the accuracy of the hydrological outputs.

The 2012 RORB modelling adopted the 'future developed conditions' modelling as described by SKM, with some modifications applied by Water Technology to include the Waterford Rise residential estate RORB modelling (Neil M Craigie, 2010) for a development located in the northwest area of Warragul (west of Tarwin Street, north of Princes Highway).

In Addition to these changes Water Technology recommended adjusting the way basin storage-discharge relationships were represented the SKM RORB modelling. It was recommended that the outlet features be included as discrete pipes within the RORB model as opposed to a simple storage-discharge table, using this option streamlined the basin optimisation process. Due to the irregular nature of the Brooker park outlet features, a specific storage-discharge table was modelled in a separate package (XP-STORM) before being incorporated into the RORB model.

Basin optimisation was discussed with BBSC, following this an approved set of general criteria were adopted in regard to improving basin efficiency. It was generally considered that minor modifications to outlet structures along with minimising basin volume augmentation would provide substantial reductions in peak basin outflows for shorter duration rainfall events while the modified basins should not increase flows for any of the modelled events.

The Tarwin Street basin is situated west of the intersection of Tarwin Street / Pharaoh's Road / Sutton Street in Warragul. The following optimised basin characteristics were recommended:

- Raising embankment height to 121.2m AHD (1.38m increase);
- Raising spillway height to 120.0m AHD (0.8m increase) and introducing 3 x 2750mm W x 600m H box culverts to form the spillway under the future road;
- Installing an additional 750mm orifice plate onto one of the existing 900mm orifice plates on the upstream culvert assembly; and
- Constructing an internal coffer wall (115.75m AHD crest) around the upstream invert location of the existing box culverts, with a single 750mm circular culvert at the base of the coffer dam wall.

The Landsborough Road basin is situated south west of the corner of Landsborough Road and Butlers Track in Warragul. The following optimised basin characteristics were recommended:

- Raising embankment height to 114.1m AHD (0.9m increase);
- Raising spillway height to 113.4m AHD (0.68m increase);
- Installing a 750mm orifice plate onto the US end of one of the existing 900mm culvert pipes; and
- Constructing an internal coffer wall (110.8m AHD crest) around the upstream invert location of the existing culvert pipes, with a single circular 525mm culvert at the base of the coffer dam wall.

The Brooker Park Basin (also referred to as Sutton Street basin) is situated north of Sutton Street in Warragul, and is bounded by Charles Street (east) and Bowen Street (west). Water Technology project staff reviewed the potential improvements that could be made to the Brooker Park configuration. As the existing basin is considered to be suitably functional, it was concluded that altering the Brooker Park basin is unnecessary.

Hydraulic Modelling (Rainfall on Grid) Flood Modelling to improve overland flow paths through town

Once built, calibrated and verified the Rainfall on Grid (RoG) model was used to create flood maps within the study area for the 1%, 2%, 5%, 10%, 20%, and 50%, AEP flood events and as a tool to determine possible solutions and estimated costs to address identified inundation issues.

The following conditions were noted in the 5 year ARI modelling:

- Minimal flooding of residential parcels occurs in the 5 year event.;

- All major overland flow paths are engaged in the 5 year event. This suggests, as expected, that the pipe network does not have 5 year ARI capacity. In this case, the pipe network is generally full and surcharging onto the 2D domain;
- Key Locations were flooding is noted include:
  - Downstream of Brooker Park Basin (upstream and downstream of Sutton Street);
  - End of Helen Court;
  - Ryan Court;
  - Downstream of civic park;
  - Along Normanby Street between Albert road and Queen Street
  - Bottom end of Phoenix and Pearse Streets.
  - Downstream of Churchill Street

Results generated in the RoG study covered the urbanised portion of the Warragul township but did not cover the Hazel Creek floodplain south of the main township area.

During the 100 Year ARI event, there were a significant number of localised flooding 'hot spots', The following conditions were noted as area of concern in the 100 year ARI modelling:

- The piped designated waterway which moves water from Sutton Street to the outlet at the intersection of Queen and Normanby Street is of major concern with major flooding along through residential properties adjacent to Normanby Rd.
- Significant flooding in the industrial area between Albert Road and Queen Street;
- The overland flow path which moves water from the Warragul CBD to toward the outlet at the intersection of Queen and Normanby Street reaches depths above 50cms;
- Significant flooding at the corner of Gladstone Street and Vermont Avenue.
- Flooding along Queen Street between Normanby Street and North Road (Most of the flooding in this area is found in Phoenix and Pearse Streets);
- Localised depression along Western Point Drive (near Pioneer Street);
- The designated waterway Downstream of Churchill Street is exceeded and flooding into residential properties is evident;
- Flooding of residential properties located downstream of Waratah drive (upstream of the Brooker Park Basin); and
- Overland flow paths through residential properties upstream of Stoddarts Road along Ellen Close;

A workshop presenting RoG modelling results (and Basin Optimisation works) was held at the BBSC offices on the 26th of June 2012. Possible mitigation works to relieve identified and historically known impacts of flooding within the study region were discussed in this meeting. Investigation of the implementation of additional retarding basins at four locations within the RoG study area was agreed upon with basic basin features integrated into the 2D component of the TUFLOW model, with outlet structures of the retarding basins were designed to fit in with the existing stormwater network system

The conceptual modelling showed there are significant benefits to be gained through the use of retarding basins within Warragul. The mitigation investigation highlighted the need for further detailed modelling and costing prior to detailed design and construction. The attenuation of flows within the retarding basins located at Civic Park (Basin 2) and Eisenhower Court (Basin 3) have a positive impact on the downstream flooding with a widespread reduction in depth of 2-5cm along much of the Normanby Street flow path, of which was shown as a major flooding issue under existing conditions.

The full benefit of the four retarding basins is difficult to quantify given the lack of floor level survey data for the affected properties. Nevertheless, based on the reductions in maximum flood depths and flood extents, shown in the conceptual modelling, the CBD block (Retarding Basin 1) area and Civic Park (Basin 2) are seen as the most appropriate sites for retarding basins. For the CBD block (Retarding Basin 1) option to be realised several issues will need to be addressed. Another feasible option for the CBD block (Retarding Basin 1) area is to construct an underground storage in the vicinity of the low lying area. Further analysis will be required before this option can proceed.

Additional concept design and costing was also undertaken for a piped solution to mitigate flooding if the CBD Block (Retarding Basin 1) is not constructed. A pipe was sized to carry flows from the northern existing pipe crossing of Mason Street south to Queen Street and then east along Queen Street to the outlet. This option was roughly costed at $2,848,700 (Total Cost with Design & Contingency). If upgrading the pipe network or constructing a retarding basin are not feasible options for controlling the overland flow that runs between Mason and Gladstone Streets, an easement may be an alternative option used to convey the 100 year ARI flow of 3.3 m^3/s through private property.

Recommendations on the LSIO and FO and flood emergency response.

Impacts from the flash flooding were investigated in the Urban Rain on Grid (RoG) modelling task. The Impacts from riverine flooding were to be determined in a separate project known as the Warragul Waterway Modelling Project (*Water Technology 2012*) funded by the WGCMA.

A new hydraulic model was constructed for much of the Hazel and Spring Creek floodplain. As much of the preliminary model set up is consistent with the (RoG) modelling covering the urbanised portion of the system, the TUFLOW hydraulic modelling program was recommended for the waterway modelling to produce recommendations on the LSIO and FO and flood emergency response. Hydrology for the Warragul Waterways Flood Modelling project considered 3 sources: RoG modelling, modified SKM RORB model and an independent RORB model constructed by Water Technology for the design of the Stoddarts Rd Basin (North of the Warragul Township).

Water Technology recommended the following shapes to be considered as LSIO and FO layers within the Warragul study area.

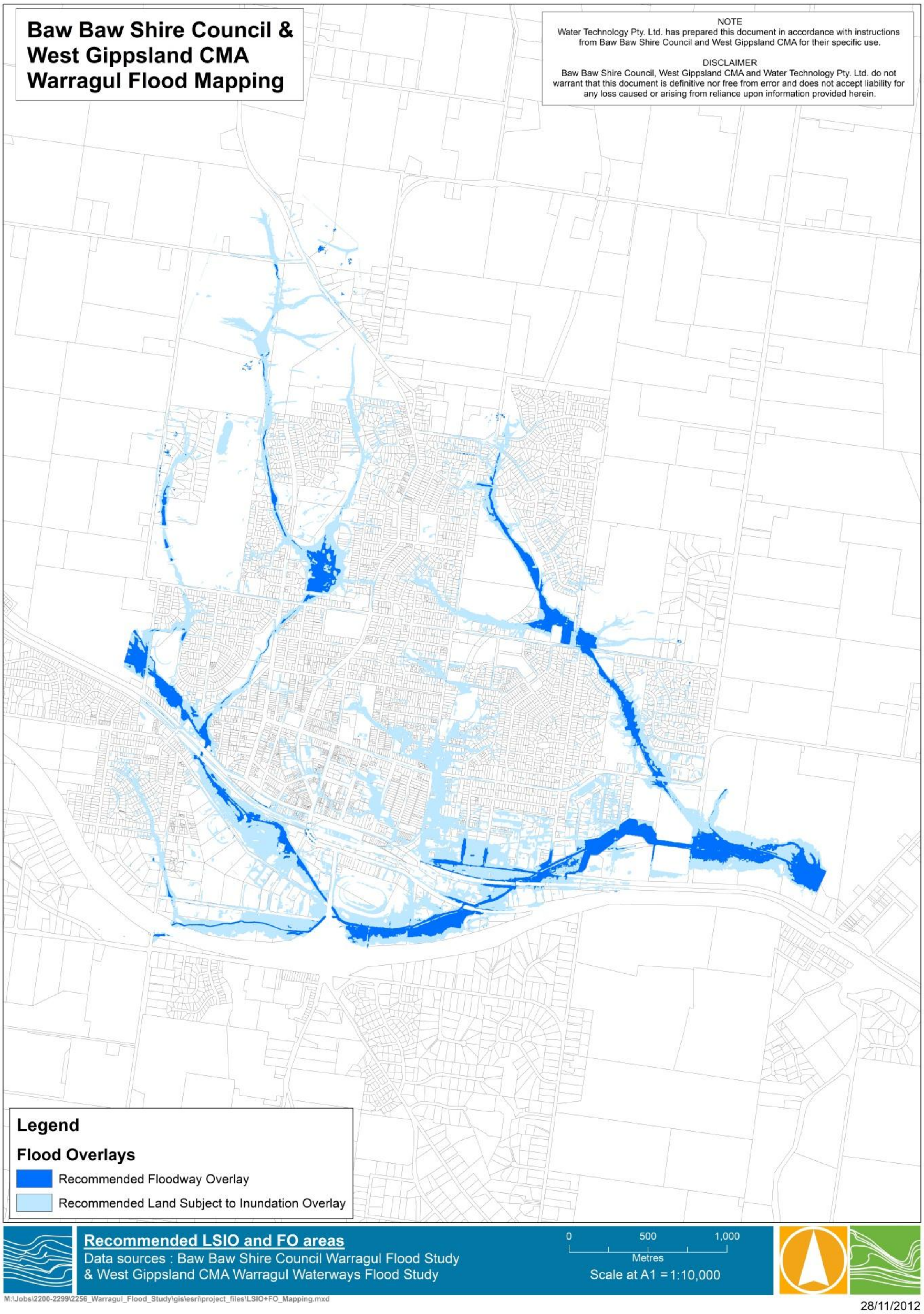


Figure ES 1 Recommended LSIO and FO shape from the Warragul Flood study and modelling project

A meeting was held with VicSES staff, where the existing conditions from the urban (RoG) modelling results were presented including critical duration and flood hazard risk outputs for the catchment. Two additional maps and MapInfo tables were requested by VicSES (Detailing of land parcels inundated above 0.1m & a map linking the critical duration and corresponding rainfall intensity and the maximum 100yr ARI flood extent). In addition to the two maps requested, Water Technology offered to provide assistance in the analysis of hazard mapping generated from both hydraulic models (urban RoG and waterway direct inflow).

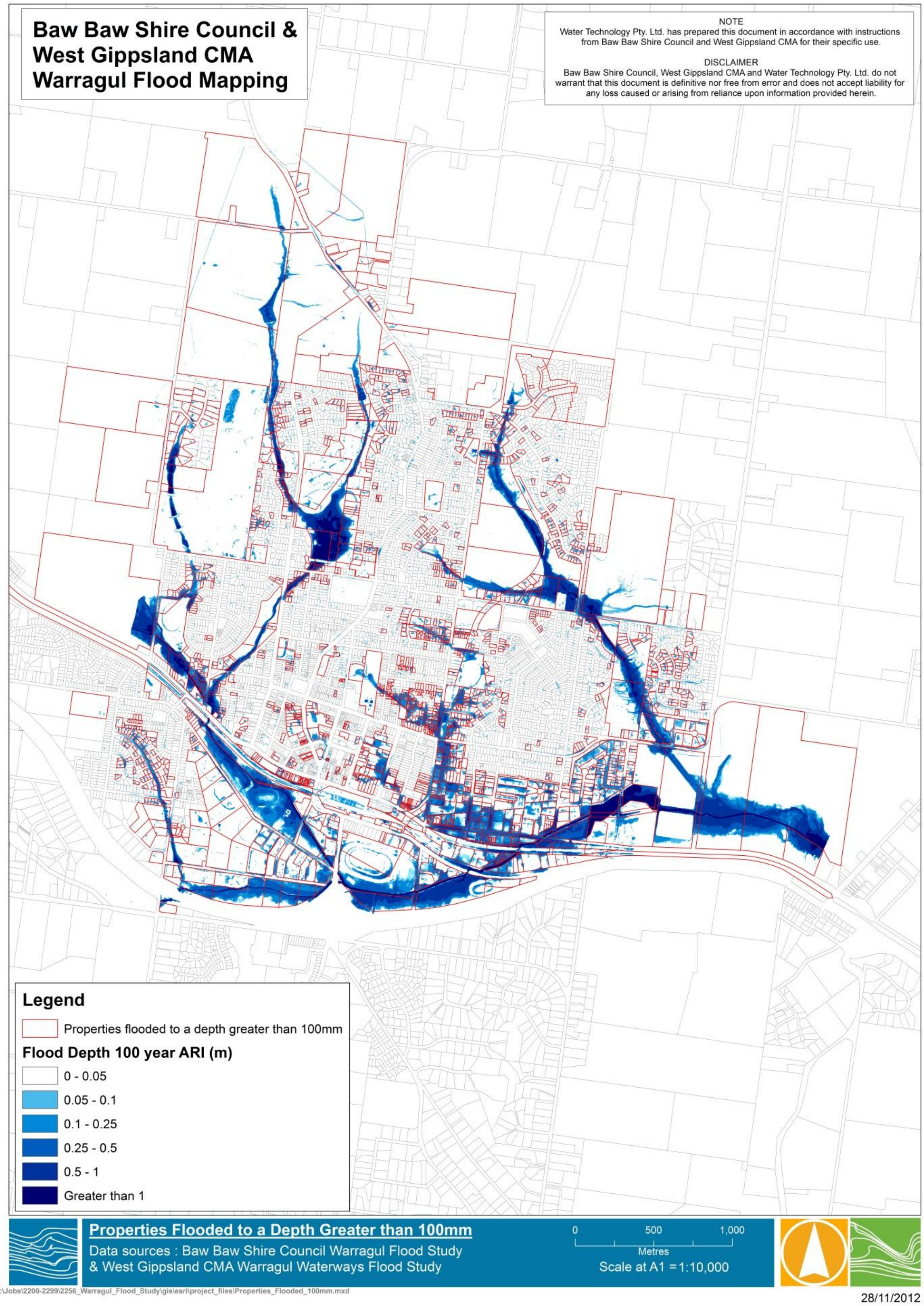


Figure ES 2 Parcels inundated above 0.1m in the Warragul study area

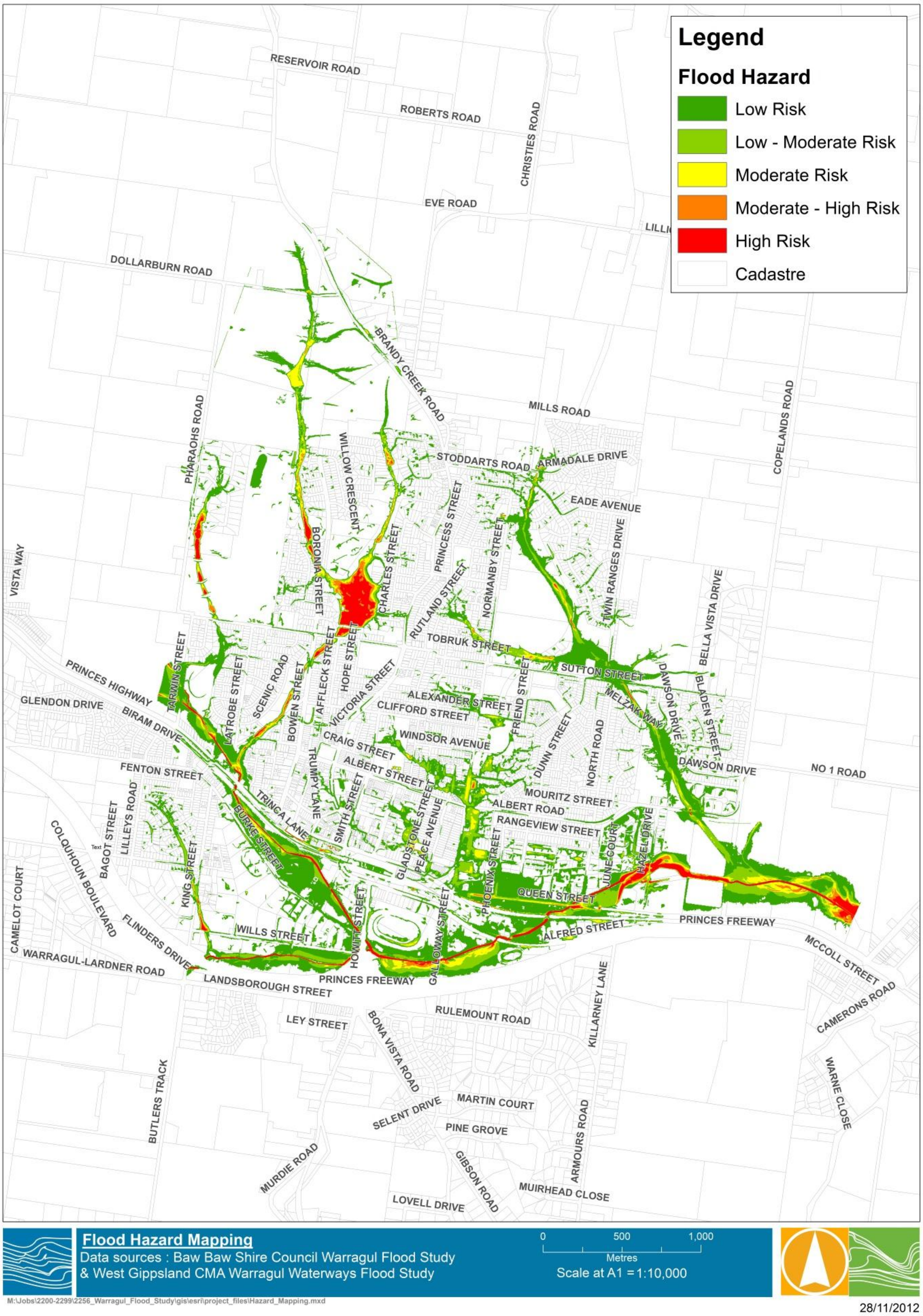
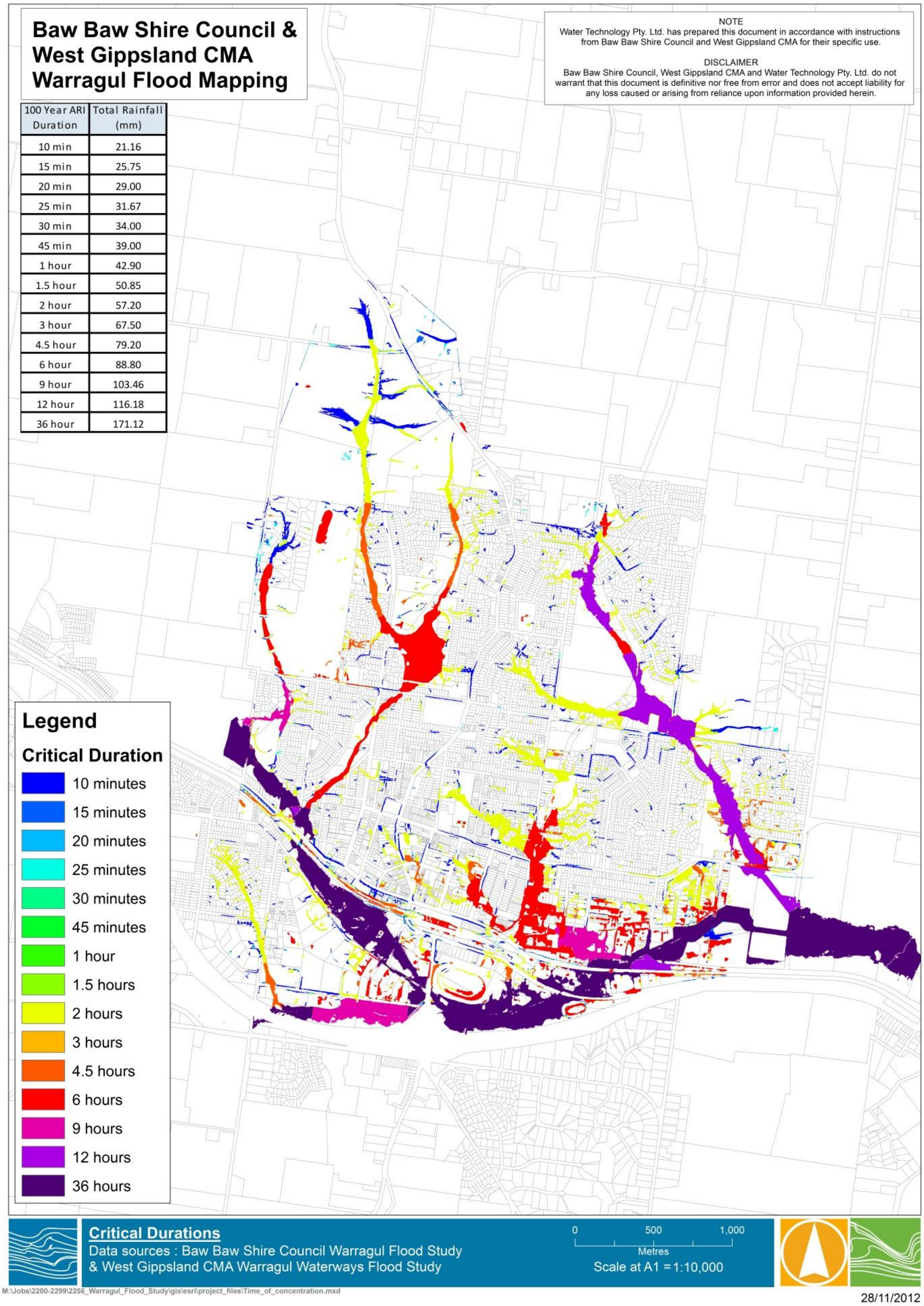


Figure ES 3 Hazard mapping within the Warragul flood study and modelling project study area



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1. INTRODUCTION AND BACKGROUND

In February of 2012 Baw Baw Shire Council (BBSC) released a project brief *(BBSC 2012)* focused on a series of objectives outlined below;

- The development of a flood study for the Warragul township to understand and improve the overland flow paths which occur through the town centre during large rainfall events;
- To improve the performance of the three existing retardation basins located within the catchment
- Gain a better understanding of the flooding issues in Queen Street,
- To update the information on existing Land Subject to Inundation Overlays and Flood Overlays and to improve planning and flood response time for VicSES.

The overall project objective was to improve councils understanding of the hydraulics and hydrology of the Warragul drainage catchment.

The project study area is broadly covered by the Warragul Urban Growth Boundary, but in situations where the stormwater catchment extends beyond the Growth Boundary, these areas are also included. BBSC identified two previous flood studies covering the study area which needed to be considered as part of the study:

1. *Hazel and Spring Creek Drainage Study* in 1994 by CMPS&F commissioned by the Rural City of Warragul. Subsequent to the study, three large retardation basins were constructed to manage peak flows in the waterways, these basins are known as Tarwin Street, Landsborough Road, and Brooker Park Basins.

2. *Urban Drainage Strategy for the township of Warragul* in 2007 by Sinclair Knight Merz (SKM) commissioned by BBSC. SKM's strategy identifies the drainage infrastructure required to adequately service land capable of being developed.

In addition to these key studies Water Technology considered the following documents in data review phase of the project:

3. Earthtech (2004), *Hazel and Spring Creeks, Hydrology and Hydraulics Review*
4. Neil M Craigie Pty Ltd (2010). *Waterford Rise Estate (Warragul North West) – Surface Water Management Strategy (Version 3 Final)*
5. Water Technology (2009). Queen Street Warragul Functional Waterway Design Report
6. Water Technology (2011a). Logan Park Soccer Fields - Surface Water Management Strategy
7. Water Technology (2011b). Lot 22M Howitt Street Warragul - Flooding Investigations Water Technology (2011c). Stoddarts Road Retarding Basin / Wetland Functional Design
8. Water Technology (2012a). 11-13 Ryan Court Warragul – Flooding Investigation

The Warragul Flood Study & Modelling Project has three key focus areas:

- Improving the flood mitigation infrastructure and provide adequate drainage infrastructure and overland flow paths within the study area;
- Improve the performance of three existing retardation basins for different storm events; and
- Update information on existing LISO and FO, and to assist emergency response in major storm and flood events.

Each of the above focus areas are detailed within this report.

2. OBJECTIVES & METHODOLOGY

The main objective of the study is to improve Councils understanding of the hydraulics and hydrology of the Warragul drainage catchment. A comprehensive methodology was presented to the BBSC in the Water Technology project proposal. The methodology was discussed and approved with BBSC staff in the project inception phase of the study. Project tasks and their appropriate method were described as Items 1 – 6 in the project proposal. Items 1 to 4 are presented below and relate to the three key focus areas in this study.

Item 1 : Data Collation and Review	
1.1	Project Inception Meeting
1.2	Site Visit
1.3	Data collation and review
1.4	Available data review
1.4	Undertake a range of consultations with relevant authorities, and other stakeholders.

Item 2: Hydrological Modelling (RORB) – Improve Existing Basin Performance	
2.1	Re-establish existing RORB model (SKM Warragul Drainage Strategy 2007)
2.2	Validate Model Assumptions
2.3	Update RORB model to reflect current system (if required)
2.4	Optimise basin performance and determine the impact that changes in outflow characteristics may have on the hydraulics of the major watercourses downstream
2.5	Provision of conceptual plans (functional design), and cost estimates for the works.

Item 3: Hydraulic Modelling (Rainfall on Grid) Flood Modelling to improve overland flow paths through town	
3.1	Hydraulic model established for the old Warragul township catchment
3.2	Identify key overland flow paths focused on areas identified by BBSC which experience inundation in heavy storm events.
3.3	Flood risk investigation
3.4	Mitigation options within the existing Warragul township.

Item 4: Recommendations on the LSIO and FO and flood emergency response.	
4.1	Hazel Creek catchment Hydrology review
4.2a	Re-establishment of 3 existing Hydraulic models of the urbanised portion of the Hazel Creek
4.2b	Establishment of a new Hydraulic model of the urbanised portion of the Hazel Creek Floodplain
4.3	Review of hydrological assumptions in previous modelling
4.4	Amalgamation of flooding data from existing (updated) hydraulic models
4.5	Development of suitable emergency response data for VicSES

At the beginning of the project the WGCMA were contacted to discuss streamlining of Council, VicSES and WGCMA modelling requirements within the Warragul Flood Study area. Due to project budget constraints, Item 4 (Recommendations on the LSIO and FO and flood emergency response task) was compromised, this was communicated to the WGCMA and they agreed to be a financial contributor to the project to improve the outputs from this task. Additional project outputs were requested by the WGCMA included:

WGCMA. – Warragul Waterways Flood Study
Augment existing LiDAR 2D grid
Additional Survey of Crossings / Bridges (ultimately not required after data review was complete)
Hydraulic Model Development and Calibration
High / Low Flood Hazard Levels
100 year Flood Extent and Depth Mapping
100 year ARI Hazard Mapping and Determination of Floodway / Land Subject to Inundation Overlays (for the expanded study area)

The objectives for the WGCMA are detailed in a separate report "Warragul Waterways Flood Study, Water Technology 2012" with findings from the waterways study relevant to the BBSC study presented within this report.

3. DATA COLLATION AND REVIEW

Information provided by stakeholders was compiled and reviewed during a 'Data Collation and Review' phase of the project. The information provided was split into several catagories; GIS data, 'as constructed' drainage design plans, industry standards/manuals and histoical reports relevant to the study area.

3.1 Consultation with relevant authorities and stakeholders

During the project proposal and project start-up phase of the study a Project Communication Plan was developed and discussed. This plan identified relevant stakeholders to the study and set out protocols for communication between these groups. During the project inception meeting a discussion was held around what Water Technology and BBSC believed each stakeholder offered the study, this information was then formalised into a data request memo circulated to the following stakeholders:

- Baw Baw Shire Council (Various departments);
- West Gippsland Catchment Management Authority (WGCMA);
- Gippsland Water;
- VicRoads;
- VicSES; and
- The Australian Bureau of Meteorology (BoM);

Table 3-1 Data provided by stakeholders (other than BBSC)

Stakeholder	Data provided
WGCMA	Spatial Data – LiDAR topographic data (1m DEM)
Gippsland Water	Report - Warragul Wastewater System Plan (May 2011) Spatial Data – GIS layers representing Facilities, Manholes and Sewer Pipes
VicRoads	Survey Data – Used to validate LiDAR data
VicSES	No data was provided by VicSES
BoM	Flood History – Historical flood warnings 1999 -2012

Data which was significant to the study has been discussed in detail in following sections.

3.2 GIS Data

Water Technology worked closely with the BBSC spatial services department insuring the best data held within the BBSC was made available for this project. At the request of Water Technology following GIS data was provided for this study:

- VicMAP data including:
    - Watercourses (designated waterways);
    - Current Overlays and Planning Zones;
    - Township Land Parcel (approved and proposed);
    - Details of the road and rail network;
- Topography (1m contour data);
- Aerial photography from 2006 & 2011;
- Details of the Warragul pit and pipe network; and,
- Details of major road crossings throughout the study area.

In addition to the spatial data provided, the West Gippsland Catchment Management Authority (WGCMA) provided 1m grid resolution LiDAR (Light detection and Aerial Ranging) data accurate to +/-0.1m vertically. Available spatial data was used extensively throughout the project to verify data, construct hydraulic models and present assumptions, results and recommendations to BBSC.

3.2.1 LiDAR Data

LiDAR data was provided by (WGCMA) for the study area. This data was converted to a suitable format and used extensively for the RORB model review, basin optimisation and in the hydraulic model construction.

Prior to the use of the LiDAR for modelling, an extensive analysis was undertaken to assess the quality of data provided. Detailed analysis of the LiDAR information was undertaken by the spatial team at Water Technology, this included a validation process to compare the WGCMA LiDAR information with representative sets of on-ground survey data provided by BBSC and VicRoads. The BBSC data was located in the north east corner of the study area returned a mean difference of + 0.12m (12cms), while the VicRoads survey (in the south west of the study area) produced a difference of - 0.06m (- 6cms). With a stated vertical accuracy of the LiDAR information of +/- 0.1m, this review supports the accuracy and suitability of the LiDAR information for use in this project.

It is critical to the accuracy of the models to ensure that the erroneous LiDAR data is removed prior to flood modelling, while other localised sudden elevation changes are understood and incorporated in the hydraulic modelling where appropriate. While the majority of the LiDAR data was suitable for use within the hydraulic model, there were several areas that required the application of TUFLOW ‘z shapes’ to smooth over abnormalities within the data prior to use in the hydraulic modelling of the study area. This involved identifying the areas of concern, of which were generally a result of LiDAR processing where built structures are removed from the LiDAR to more accurately depict the natural surface. Other areas of concern included localised areas of depression where the LiDAR depicts a false hole that may not actually be present.

These areas were then analysed using aerial images provided by BBSC along with *Google Street View,* to gain a better understanding of the existing conditions and where necessary additional data was requested from BBSC. **Figure 3-1** and **Figure 3-2** are examples of LiDAR errors where ‘z shapes’ were applied to the hydraulic model and eliminates water from ponding in false depressions and model instabilities.

Figure 3-1 10-14 Gladstone St, Warragul, -building prior to renovation in 2012 and 100yr flood extent before proposed mitigation works, Image Source: Google Street View (2012).

Figure 3-2 10-14 Gladstone St, Warragul, new building with raised floor level in 2012 and 100yr flood extent after proposed mitigation works, Image Source: Water Technology Site visit (2012).

Figure 3-2 shows the impact of the new floor (raised) level of the *Furniture Plus* store (10-14 Gladstone Street Warragul) the constructed floor level was supplied to Water Technology by BBSC. The new floor level is approximately 1m higher than the original building floor level. The flood extents in the lower portion of Figure 3-2 show the difference in flooding at 10-14 Gladstone St with the floor level raised. This is a good example of how TUFLOW 'z-shapes' can be used to modify the LiDAR to better represent the topography of the study area.

Figure 3-3 shows another location were a TUFLOW 'z-shape' has been applied to the hydraulic model to better represent the study area topography. The original LiDAR failed to show a series of shops and car parks constructed on piers above the natural surface which falls away steeply to the south. During a rainfall event, storm water runoff from the area falls north towards Queen Street and into the drainage network. The original LiDAR did not accurately depict this, which lead to ponding behind the buildings on the south side of Queen Street as well as some minor model instability due to the large slope. The addition of TUFLOW 'z-shapes' reduced this ponding and provided a better representation of the true drainage conditions.

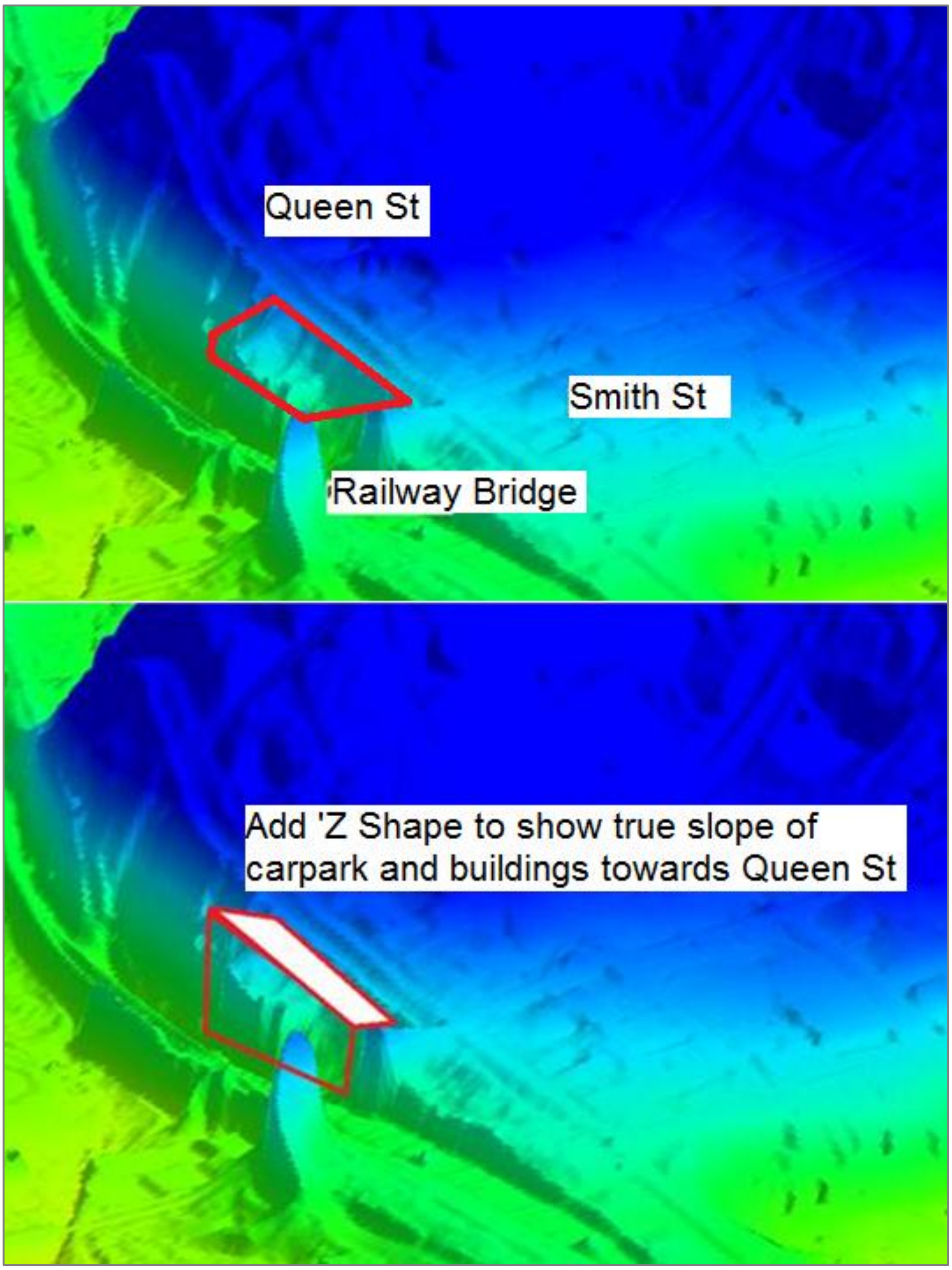

Figure 3-3 **(Upper Left) Raw LiDAR data (Upper Right) Location of the application of a TUFLOW 'z-shape' to the model**

3.2.2 Pit and Pipe network Data

A GIS dataset representing the location and details of existing stormwater infrastructure throughout the study area was provided to Water Technology by BBSC for review. Minimal feature details such as invert levels and pipe diameters were populated in this dataset. As such a comprehensive review of this dataset was required, involving the population of missing information prior to its use in the hydraulic modelling of this study. The review used the best data available including CAD drawings and hard copy plans provided by BBSC to verify pipe inverts and features sizes. The final pit and pipe data used in the hydraulic modelling will be made available to the BBSC spatial team.

3.3 Industry Standards and Technical documents considered

The following industry standards were reviewed/considered throughout the project.

- *Australian Rainfall and Runoff, a Guide to Flood Estimation Volume 1*. Institution of Engineers, Australia, 1987.
- MUSIC Guidelines, *Recommended input parameters and modelling approaches for MUSIC users*. Melbourne Water Corporation, 2010
- Flood Mapping Projects, Guidelines and Technical Specifications, Melbourne Water Corporation November 2010
- *RORB Version 5, Runoff Routing Program, User Manual*. Monash University Department of Civil Engineering, in conjunction with Sinclair Knight Merz Pty. Ltd. and the support of Melbourne Water Corporation, Laurenson E. M., Mein R. G. and Nathan, R. J. August 2005

3.4 Histocal reports reviewed

Documents identified earlier (Section 1) were reviewed as part of the Warragul Flood Study & Modelling Project. The following points were noted:
Catchment Characteristics/Historical flooding:

- The Hazel and Spring Creek catchment is approximately 47km^2;
- The catchment is ungauged; and,
- Limited historical flooding information is available. 1934 is assumed to be the biggest flood event on record as per the following paragraph from the 2004 Earth Tech report.

Historical recorded flood heights at Warragul documented in the Flood Data Transfer Project database are limited to four spot heights recorded for the 1934 flood event. Anecdotal evidence would suggest that the 1934 flood is the largest Hazel Creek flood experienced. The 1994 Hazel and Spring Creek Drainage Study report refers to an article in the Warragul Gazette of 4 December, which stated that 261 mm of rainfall was recorded during a 33-hour period. Assuming the article account to be accurate, the equivalent intensity of 7.9 mm/hr significantly exceeds the 100-year ARI, 33-hour intensity of 5.2 mm/hour (i.e. this would suggest the 1934 event was an extreme event).

Hydrology:

- Two RAFTS models have been created by CMPS&F and EarthTech in 1994 and 2003 respectively. The area covered by both models includes the greater Hazel Creek and Spring Creek catchments down to their confluence with the Moe River near Bloomfield Rd (east of the Warragul CBD);
- A study by SKM in 2007 involved building a third hydrological model for the Spring and Hazel Creek catchments. This model used the RORB hydrology Rainfall Runoff Routing software package. This model by SKM was assumed to be the most accurate representation of the catchment, and was therefore recommended for use in this study following a thorough review; and,
- Generally all historical hydrology studies appear to be well developed using the best data available at the time of model generation. Differences in historical peak flow estimates represent changes in best practice hydrological parameter estimation and actual changes in the catchment from increased urbanisation.

Hydraulics:

- A 1D hydraulic model (HEC-RAS) was constructed by Earth Tech in 2004 as part of the *Hazel and Spring Creeks – Flood Risk Study*. The study area included the Hazel and Spring Creek floodplains from Tarwin Street west of the township through to Bloomfield Road, east of the Warragul township;

- 15 hydraulic structures were included in the 2004 EarthTech HEC-RAS modelling, the details of these structures were considered suitable for use in this study. Cross-sections of the River channels used in the EarthTech modelling were reviewed against LiDAR obtained from BBSC and where appropriate the cross section data was used to augment the LiDAR to generate a better representation of the Hazel Creek channel morphology;
- Water Technology has undertaken 4 hydraulic studies (linked 1D-2D modelling) within the study area for various private land developers and BBSC. Each of these studies considered the impacts of proposed developments within the existing LSIO overlay (derived from the EarthTech 2004 study);

4. CATCHMENT AND DRAINAGE CHARACTERISTICS

4.1 Catchment and Drainage Description

Warragul is located approximately 100km east of Melbourne, topography in the area varies but can be described as moderately sloping. The Warragul township is located within the Hazel Creek catchment, with the Hazel Creek running to the south of the CBD and is the major waterway in the Warragul area. The Hazel Creek catchment is approximately 47km² and is fed by a number of smaller waterways including Spring Creek (to the south) with a catchment area of approximately 14km² which discharges to Hazel Creek to the South of the CBD in the low lying areas of the Hazel Creek floodplain. Hazel Creek eventually discharges into the Moe River east of the Warragul Township, with the Moe River, major a tributary of the Latrobe River one of the major waterways in the Gippsland region. Significant disturbance throughout these catchments since European settlement, including changed land use have significantly changed the hydrology and hydraulic conditions of the waterways. As a result of changed hydrology, some areas throughout the catchment may become more susceptible to flooding as a result of shorter more intense storms within the catchment compared to traditional flooding from longer storm durations. Therefore it was necessary when completing this flood study to assess a range of flood durations throughout the study area.

Drainage in the Warragul Township follows the natural topography and generally moves water from the higher reaches in north through the township to the south with most of the townships floodplain located between the railway line and Princess Freeway region. Elevations range from approximately 100m AHD to 160m AHD within the ROG study area as shown below in **Figure 4-1** .

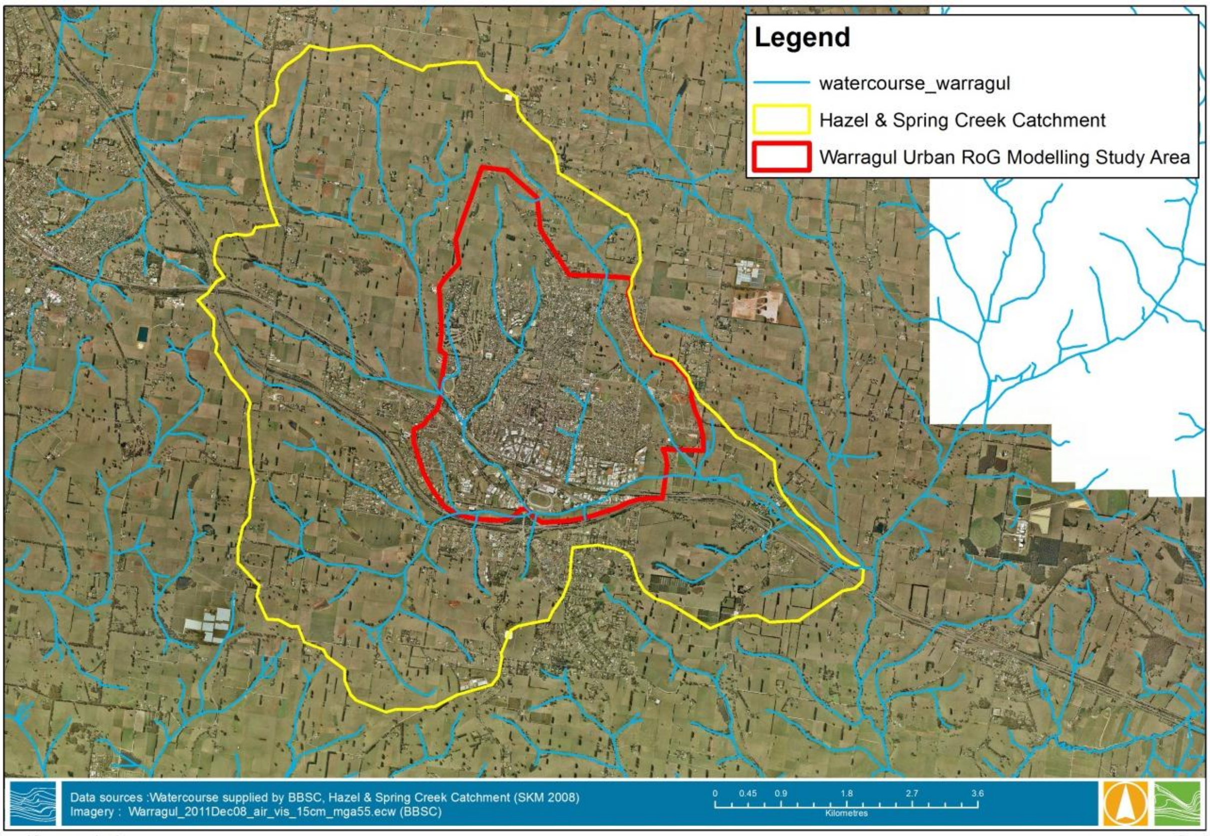

Figure 4-1 Hazel & Spring Creek catchment (Yellow) and study boundary of the urban RoG modelling (Red)

4.2 Land Use Zonings and Overlays

A 'Land use Zones' review for this study was undertaken from the BBSC Planning Scheme. The major zone types inside the study area are Residential (55%) and Farm Zone (11%). It is not expected that the quantity of residentially zoned land within the catchment will reduce in the future. The composition of zoning throughout the study area is shown in Table 4-1.

Table 4-1 Zoning Summary

Zone Type	Zone Codes	Percentage of Total Area
Residential	R1Z	55%
Farming Zone	FZ	11%
Public Park and Recreation Zone	PPRZ	9%
Industrial	IN1Z	8%
Business	B1Z, B4Z	5%
Public Use	PUZ7	4%
Other	LDRZ, MUZ, RAZ, RDZ1 and UFZ	8%

4.3 Known Flooding Issues

Existing Flood Overlay (FO) and Land Subject to Inundation (LSIO) layers supplied by the BBSC can provide information on areas where flooding may be expected. As shown below in Figure 4-2, issues arising within waterway assets are present across the study area. The areas of investigation within this report will be used to bridge the knowledge gap with regards to urban overland flow paths (outside designated waterways) and flooding from the underground drainage network and low points in the stormwater network where the capacity of stormwater infrastructure is exceeded causing significant ponding and inundation.

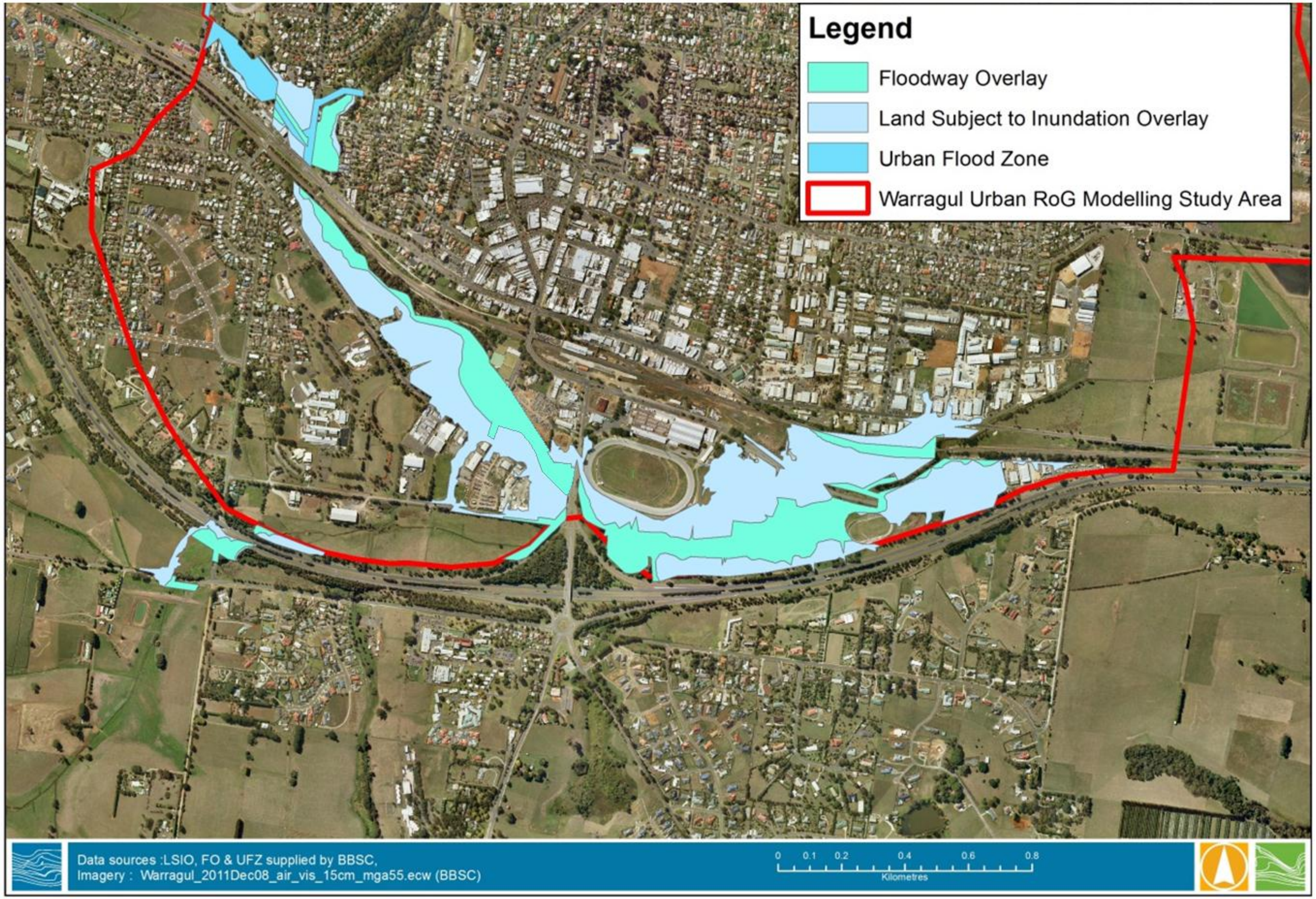

Figure 4-2 Existing Land Subject to Inundation Overlay (LSIO) & Floodway Overlay (FO)

5. EXISTING RORB MODEL REVIEW

5.1 Overview

A major component of the Warragul Flood study project is the review and update (as required) of the existing RORB hydrology model built by SKM in 2007 for the Warragul Urban Drainage Strategy project. The following section presents the findings of the review.

As noted in Section 3.4, various past studies have been reviewed for this study, based on this literature review the hydrological conditions summarised in Table 5-1 have been noted.

Table 5-1 Historical Peak flow estimates throughout Hazel Creek and Spring Creek systems

Location	Peak 100y ARI Design Flow (m³/s)					
	1994 RAFTS - CMPS&F Study Flows (2020 Conditions)	2003 RAFTS - Earth Tech Study Flows (Without Basins)	2003 RAFTS - Earth Tech Study Flows (With 3 Basins)	2007 RORB - SKM Current Conditions	2007 RORB - SKM Future Developed	2007 RORB - SKM Future Retarded
Tarwin St Basin Inflow	-	29	29	21	25.5	25.5
Tarwin St Basin Outflow	-	-	15	11.7	16.4	11.9
Sutton St Basin Inflow	14	11	11	13.4	13.9	13.9
Sutton St Basin Outflow	-	-	5	5.8	5.8	5.8
Landsborough Basin Inflow	24	21	21	22.3	25.9	25.9
Landsborough Basin Outflow	-	-	9.1	12.8	13	13
Junction of Hazel & Spring Creeks	127	100	65	66.4	82.2	71.7
Hazel Creek @ Bloomfield Rd	150	120	89	85.4	104.3	90

5.2 SKM RORB models and report – Data available

SKM RORB models for the three conditions investigated (Table 5-1 above, last 3 columns) were made available for this study. Reporting by SKM included the catchment parameters applied and the sub-catchment delineation. No spatial data of sub-catchment areas or reach definitions was available for review. As recommended in the detailed methodology of the Water Technology tender submission, nominal percentages (10 – 20%) of physical parameters were checked. The hard copy maps of sub-catchment delineation were geo-referenced and compared to physical values inside the catchment files of the respective models and found to be appropriate for modelling use.

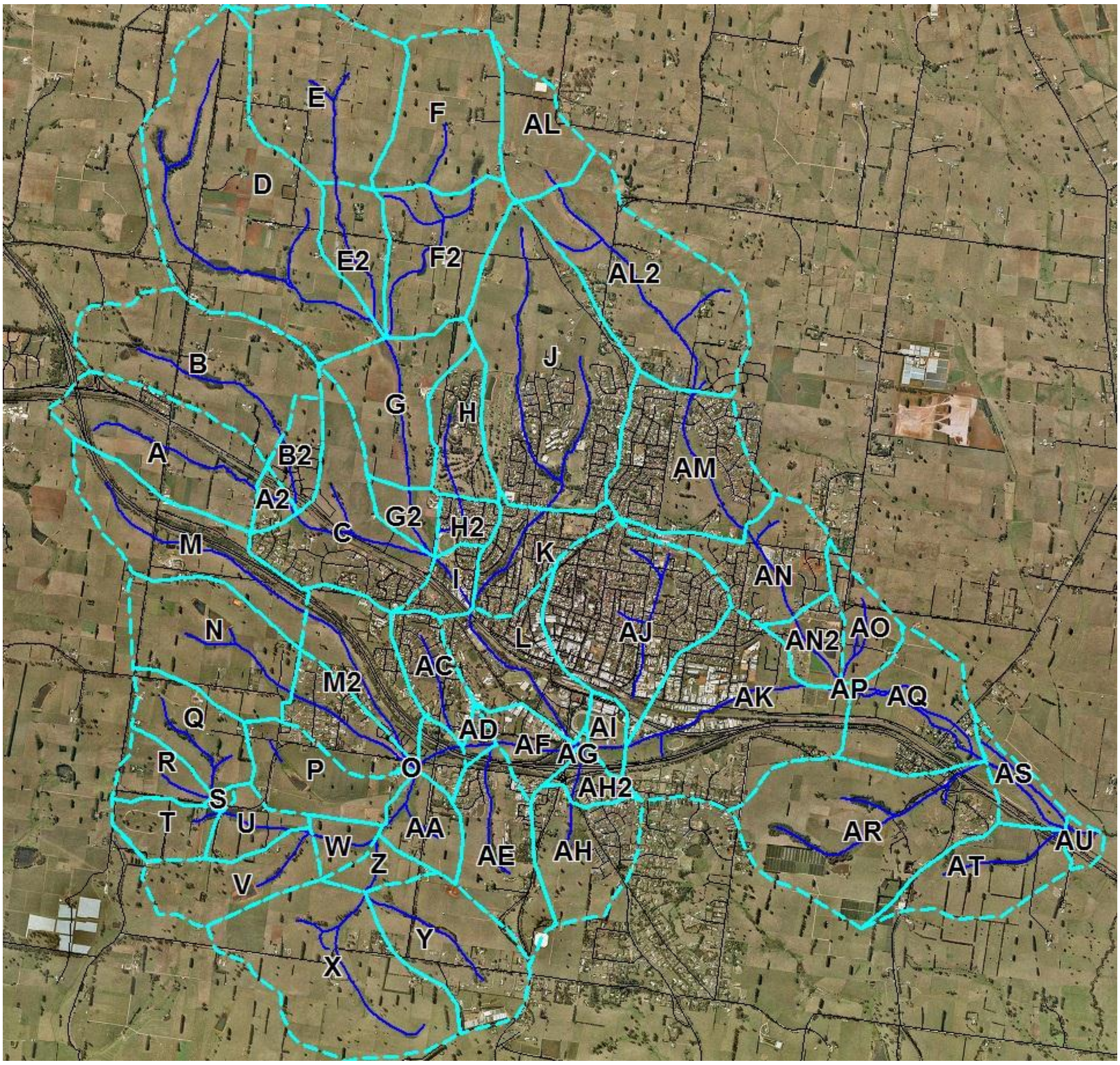

Figure 5-1 SKM RORB model, sub catchments and Vic Hydro GIS layer

Regional parameters used in SKM RORB modelling are shown in Table 5-2. As stated in the 2007 SKM report, the catchment is un-gauged the values have been "*decided upon by way of past experience, field observation, previous hydrology studies and an empirical formula useful for the region*". No regional peak flow estimates (empirical formulae) were presented in the SKM report supplied to Water Technology.

Table 5-2 SKM Routing parameters

	5y ARI	100y ARI
Initial Loss	25mm	25mm
Runoff Coefficient	0.05	0.68
m Value	0.8	0.8
Kc Value	14.53	14.53

5.3 Hydrology (RORB Modelling)

5.3.1 Overview

A hydrologic model of the Hazel Creek and Spring Creek catchments was developed / reconstructed for the purpose of extracting flows to be used as boundary conditions and source nodes throughout subsequent hydraulic modelling and for optimisation of basin performance within the study area. The rainfall-runoff program, RORB was utilised for this study.

RORB is a non-linear rainfall runoff and stream flow 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. Observed or design storm rainfall is assigned to the centroid of each subarea. Specific rainfall losses are then deducted, and the excess routed through the reach network. The following methodology will be applied in the Warragul Flood Study to review / update the SKM RORB modelling:

- Revision of the existing RORB model (SKM 2007) and parameters used, considering their appropriateness to this study;
- RORB modelling reconciled to accepted BBSC existing model results;
- Present results to BBSC (hold point);
- Update the RORB model as directed / approved by BBSC;
- Use the RORB model as a tool to undertake retarding basin optimisation and potential mitigation option analysis;
- Design flood events run for multiple storm durations; and
- Hydrographs extracted from RORB for use as inflow boundaries and source nodes in subsequent hydraulic modelling.

5.3.2 RORB Review

5.3.2.1 *SKM 2007RORB Hydrological Modelling*

Water Technology was supplied with the RORB (SKM, 2007) hydrological model used for design flood estimates for the Hazel Creek and Spring Creek catchments. The model report by SKM was also provided for review. The RORB model included 58 sub-catchments covering the Hazel and Spring Creek systems to the confluence of the Hazel Creek and the Moe River East of Warragul near Bloomfield Road. As a first step, the RORB model was deconstructed, with a proportion of the physical parameters (catchment size, reach length / type and fraction impervious applied) verified using spatial software. The model had a nominal 10-20% of its physical features checked. Generally the model showed a high quality representation of the Hazel & Spring Creek system.

5.3.2.2 *Physical Parameters*

Table 5-3 2007 RORB model physical parameter check Sub-Catchment Area & FI

Sub-Catchment	A	G	J	AJ	S	Y	Z	AU
RORB Area (km²)	1.3057	1.0075	2.6061	1.7618	0.0299	1.0952	0.2126	0.1081
Check Area (km²)*	1.3098	1.0086	2.6099	1.7469	0.0322	1.0979	0.2092	0.1077
FI Ex Condition	0.1	0.10	0.26	0.55	0.10	0.11	0.10	0.45
FI F Dev Condition	0.1	0.39	0.34	0.55	0.10	0.40	0.26	0.45
FI P Retard Condition	0.1	0.39	0.27	0.55	0.42	0.16	0.42	0.45
Check FI (visual - Existing)**	0.1	0.1	0.29	0.59	0.1	0.19	0.1	0.19

*Sub-catchment area based on geo-referenced image not a high quality representation of catchment delineation
**Based on aerial images from 2006 and 2011 supplied by BBSC & FI values of: Commercial 0.8, reserve 0.1, roads 0.85 & residential lots 0.6.

Table 5-4 2007 RORB model physical parameter check Reach Length & Type (existing Conditions)

SKM Comment	Length (km)	Check WT 2012 (km)	RORB Slope (%)	Check WT 2012 Slope (%)	Type RORB*	Type Check WT (aerial image 2006 & 2011)
Route to A2 input	0.21	0.22	0.67	0.66	2	✓
Route to G outlet	0.67	0.67	0.84	0.39	2	✓
Route to J outlet	1.45	1.4	0.48	0.51	2	✓
Route to AI input	0.17	0.18	0.25	0.28	2	✓
Route to S input	0.07	0.06	-		1	✓
Route to Y outlet	0.85	0.86	1.56	2.04	2	✓
Route to Z input	0.26	0.25	1.56	0.48	2	✓
Route to AU input	0.16	0.16	-		1	✓

*RORB reach types: 1 natural condition, 2 excavated but unlined, 3 lined channel or pipe, 4 drowned.

5.3.2.3 *Rainfall depths*

To check the rainfall inputs from the SKM 2007 RORB modelling, design rainfall depths were determined using the IFD methodology outlined in AR&R Volume 2, 1987. The IFD parameters were generated for the location of Warragul (406535.85E, 5776618.74N Zone 55, corresponding to the Catchment Centroid) and are shown in Table 5-5

Table 5-5 Catchment IFD parameters

IFD Parameter	$2I_1$	$2I_{12}$	$2I_{72}$	$50I_1$	$50I_{12}$	$50I_{72}$	G	F2	F50
	(mm/hr)	(mm/hr)	(mm/hr)	(mm/hr)	(mm/hr)	(mm/hr)			
Warragul	18.15	4.03	1.13	34.63	7.86	2.31	0.37	4.25	15.06

Data supplied by SKM included the storm files for each storm duration modelled (1hr to 72hrs). Rainfall event depths were checked against IFD depths generated using the catchment IFD parameters in Table 5-5. Comparisons were suitable for the scope of this study and differences are thought to be due to varied locations of IFD generation. Details of this analysis are shown in Table 5-6.

Table 5-6 IFD rainfall depths

100 year ARI Event Duration (hr)	SKM Depth (mm)	Water Technology Depth (mm)
2	56.49	56.41
4.5	78.56	78.29
9	104.16	103.63
12	117.16	116.49
36	165.17	173.27

5.3.2.4 *Areal reduction factor*

Areal reduction factors convert point rainfall to areal estimates and are used to account for the spatial variation of rainfall intensities across a large catchment. The SKM RORB model used the areal reduction methodology described by Siriwardena and Weinmann (1996). The areal reduction factor used in the SKM modelling ranged from 0.83 (1hr) and 0.97 (72hr) (based on Siriwardena and Weinmann for a catchment of 46.98km²).

5.3.2.5 *Routing Parameters (Kc)*

The routing parameter kc was checked against various regional estimates. It was found that SKM used the Vic Mean Average Rainfall greater than 800 mm - Equation 3.21 ARR (BkV) equation to estimate kc for the Hazel and Spring Creek catchments. This is considered a suitable approach/estimate of the catchment parameter kc for the Hazel and Spring Creek catchments.

Table 5-7 Regional estimates of RORB Kc parameter

Method	Equation*	Predicted kc
Default RORB	kc = 2.2A^0.5	15.08
Vic MAR>800 mm - Eq 3.21 ARR (BkV)	kc=2.57*A^0.45	14.53
Victoria data (Pearse *et al*, 2002)	kc=1.25*Dav	10.78
Aust wide Dyer (1994) (Pearse *et al* 2002)	kc=1.14*Dav	9.83
Aust wide Yu (1989) (Pearse *et al* 2002)	kc=0.96*Dav	8.28

*where: A (Catchment Area) = 46.98km² & Dav (Average flow distance) = 8.62km

5.3.2.6 *Catchment Fraction Impervious*

Discrete fraction impervious values modelled for each land use type were not presented in the SKM reporting. Weighted averages for each sub-catchment were extracted from the three system conditions modelled by SKM (from the RORB catchment files). The physical parameter checks undertaken in this study (on fraction impervious values in the existing conditions model) suggested that the approach taken by SKM was appropriate. Thematic maps showing the relative fraction impervious conditions modelled by SKM are shown in Figure 5-2, Figure 5-3 and Figure 5-4. Differences between existing conditions and future developed conditions appear consistent with Warragul's urban growth strategies. It is unclear however why "proposed retarded conditions" RORB model does not have Fraction Impervious values common to either the existing or future developed conditions.

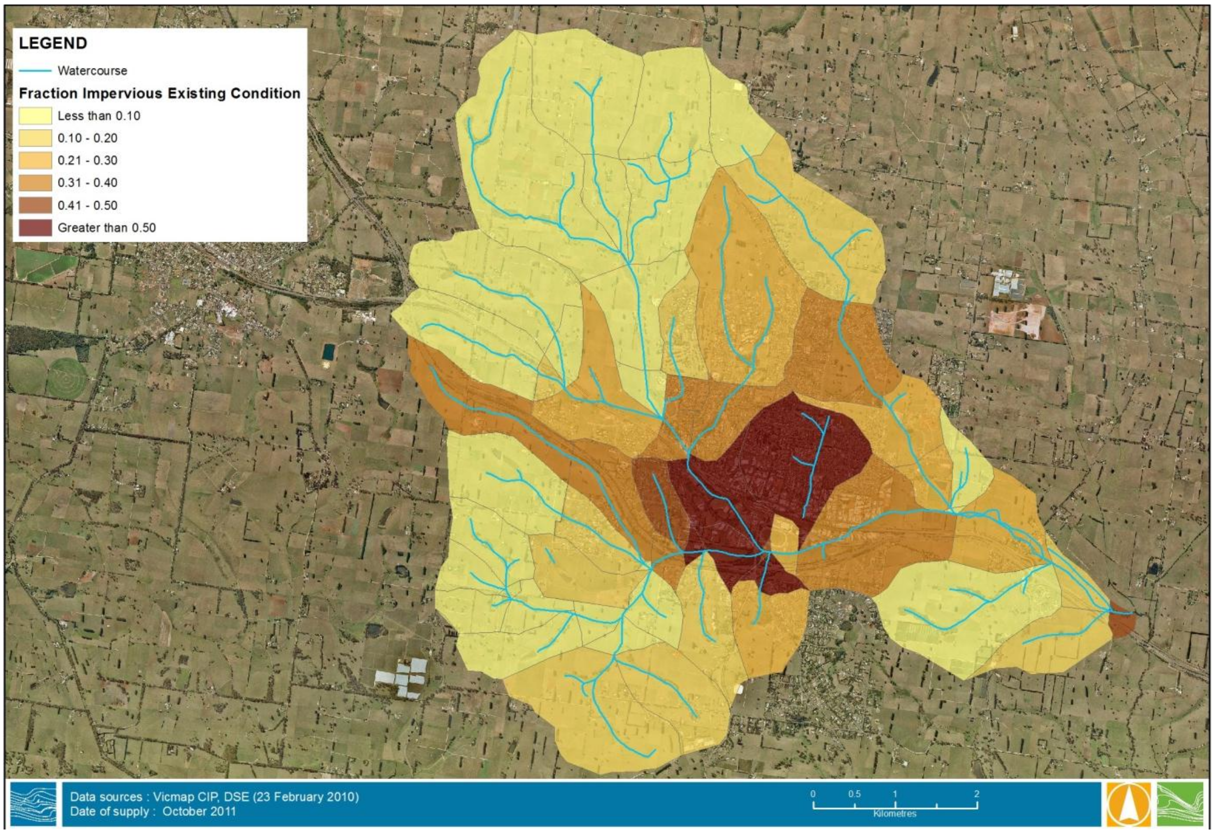

Figure 5-2 Fraction impervious values modelled in SKM existing conditions RORB model 2007

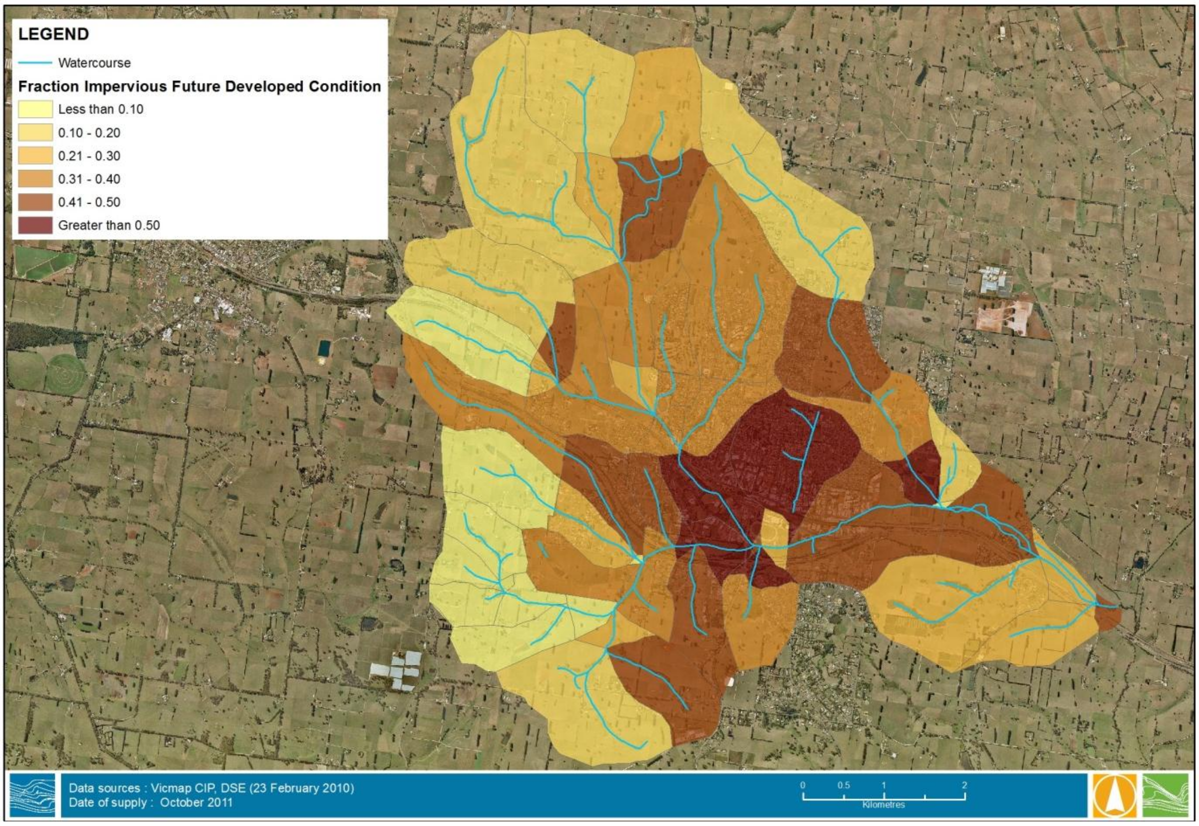

Figure 5-3 Fraction impervious values modelled in SKM Future Developed conditions RORB model 2007

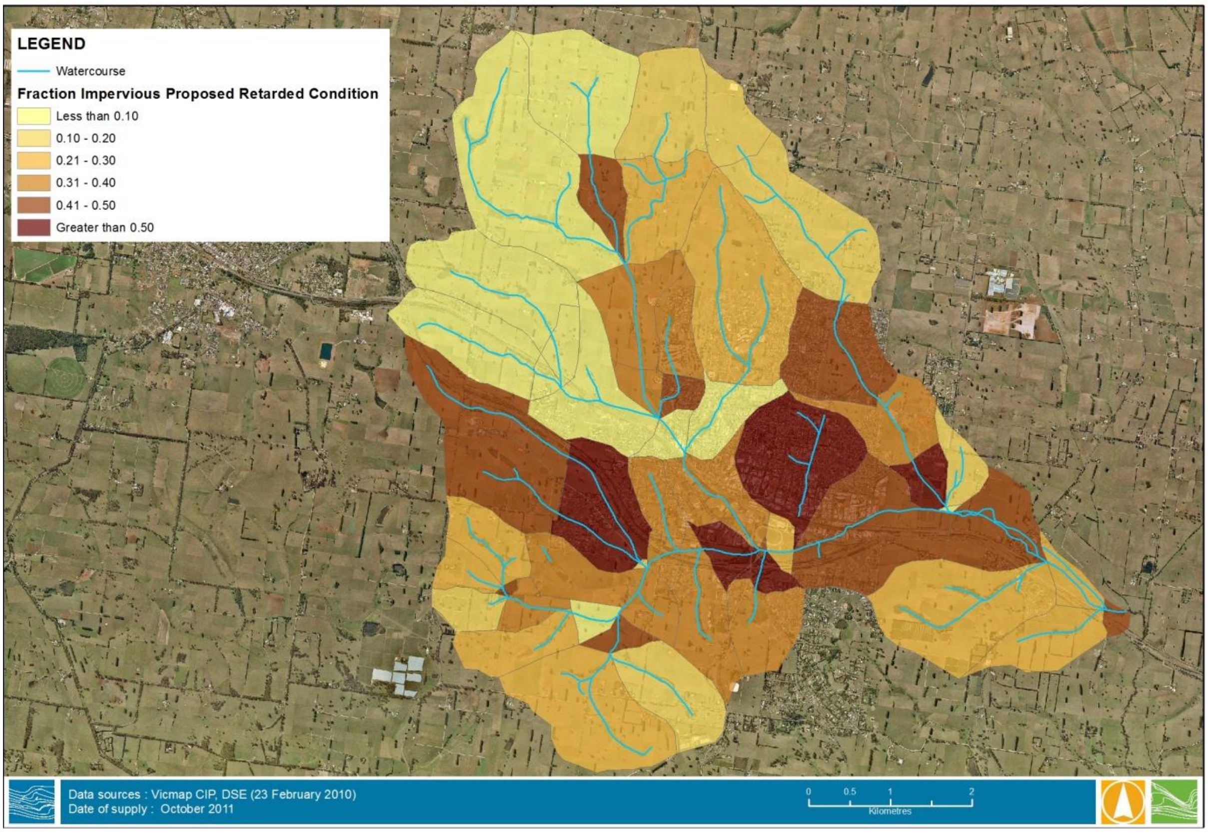

Figure 5-4 Fraction impervious values modelled in SKM Proposed retarded conditions RORB model 2007

5.3.2.7 *Retardation Basin Volumes*

RORB modelling by SKM in 2007 included modelling three existing and four proposed retarding basins. The SKM reporting noted that the stage storage relationships for the three existing basins (Tarwin, Landsborough and Sutton) were "*derived from previous studies together with information from the BBSC*". As a sanity check for these assumptions, the recently captured LiDAR data was analysed using 12d civil design software to produce a stage-storage relationship from the 1m Digital Elevation Model (DEM). These results were then compared to the 'as constructed' survey by Earth Tech in August of 2005 and the SKM RORB stage storage relationships, results of this analysis are presented in Table 5-8.

This analysis suggested that the basin volumes used by SKM for the existing basins were appropriate. Stage Storage relationships extracted from 12d by Water Technology provide a higher resolution representation (i.e. consistent incremental stage increases and basin volumes) of the basin dynamics, and may represent a more appropriate data set for use in future modelling works. Data extracted from the 12d investigation was provided to BBSC as a separate addendum to this report.

Table 5-8 Existing Storage Analysis

	Tarwin Street RB (R2)			**Sutton Street RB (R3)**			**Landsborough Road RB (R7)**		
	Depth (Cht Ht)	Depth (est. AHD)	Storage (m^3)	Depth (cht ht)	Depth (est. AHD)	Storage (m^3)	Depth (cht ht)	Depth (est. AHD)	Storage (m^3)
SKM 2007 RORB Details	0	112.9	0	0	113.4	0	0	108.35	0
	0.60	113.5	40	0.2	113.6	256	0.65	109	50
	1.10	114	190	0.4	113.8	493	1.65	110	980
	1.60	114.5	520	0.6	114	780	2.15	110.5	2,430
	2.10	115	1,970	0.9	114.3	8,264	2.73	111.08	9,130
	2.60	115.5	6,490	1.2	114.6	16,580	3.26	111.61	32,300
	3.10	116	14,770	1.6	115	28,500	3.88	112.23	81,000
	3.60	116.5	26,870	1.7	115.1	36,900	4.48	112.83	143,000
	4.10	117	44,370	1.8	115.2	45,300	4.98	113.33	213,513
	4.60	117.5	68,970	1.9	115.3	53,700	5.65	114	303,000
	5.10	118	100,660	2	115.4	62,100			
	5.60	118.5	139,000	2.1	115.5	70,500			
	6.10	119	183,800	2.6	116	112,500			
	6.20	119.1	193,167	2.8	116.2	134,850			
	6.30	119.2	202,533						
	6.40	119.3	211,900						
	6.50	119.4	221,267						
	6.60	119.5	230,633						
	6.70	119.6	240,000						
	6.80	119.7	250,000						
	6.90	119.8	260,357						
	7.00	119.9	270,000						
	7.30	120.2	300,000						

<table>
<tr><td></td><td colspan="3">Tarwin Street RB (R2)</td><td colspan="3">Sutton Street RB (R3)</td><td colspan="3">Landsborough Road RB (R7)</td></tr>
<tr><td rowspan="4">Earth Tech As Constructed Survey</td><td></td><td>~ Chart Height</td><td>mAHD</td><td></td><td>~ Chart Height</td><td>mAHD</td><td></td><td>~ Chart Height</td><td>mAHD</td></tr>
<tr><td>Basin empty (based on U/S pipe invert)</td><td>0</td><td>112.99</td><td>Basin empty (based on U/S pipe invert)</td><td>0</td><td>113.42</td><td>Basin empty (based on U/S pipe invert)</td><td>0</td><td>108.66</td></tr>
<tr><td>Spillway</td><td>6.21</td><td>119.2</td><td>Spillway</td><td>2.39</td><td>115.81</td><td>Spillway</td><td>4.06</td><td>112.72</td></tr>
<tr><td>Top of Embankment</td><td>6.83 - 7.07</td><td>119.82 - 120.06</td><td>Top of Embankment</td><td>2.73</td><td>116.15</td><td>Top of Embankment</td><td>4.55 - 4.72</td><td>113.21 - 113.38</td></tr>
<tr><td rowspan="4">LiDAR (12d Analysis)</td><td></td><td>mAHD</td><td>m³</td><td></td><td>mAHD</td><td>m³</td><td></td><td>mAHD</td><td>m³</td></tr>
<tr><td rowspan="3">Approx. Top of Embankment</td><td>120.1</td><td rowspan="3">272,313.6</td><td rowspan="3">Approx. Top of Embankment</td><td>116.2</td><td rowspan="3">112,323.1</td><td rowspan="3">Approx. Top of Embankment</td><td>113.4</td><td rowspan="3">235,682.4</td></tr>
<tr><td>~ Chart Height</td><td>~ Chart Height</td><td>~ Chart Height</td></tr>
<tr><td>7.11</td><td>2.78</td><td>4.74</td></tr>
</table>

5.4 Design Flow Verification

Given the absence of any relevant calibration data, the RORB model results were compared to several regional peak flow estimates. While it is widely accepted that significant error is inherent in many of the regional estimations of peak flow they have been included to broadly verify the hydrological results determined in this study. The design flows are largely dependent on the adopted RORB model design parameters.

5.4.1 Rational Method

As a useful sanity check, Rational Method estimates for specific sub-catchments regions of the SKM RORB model were undertaken. While not expected to reproduce the RORB results accurately the Rational Method estimates offer a broad scale sanity check of the model results. A Rational Method analysis was undertaken for the contributing catchments in accordance with the methodology outlined in Book 2 of Australian Rainfall and Runoff (AR&R, 1987). The basic equation is as follows:

$Q_{100} = C. I_{100}.A/360$

Where:

- Q_{100} is the flow in m^3/s for the 100 year ARI design event;
- C is the runoff coefficient;
- I_{100} is the rainfall intensity specific to the area, corresponding to the t_c (time of concentration of the catchment); and,
- A is the area of the catchment in hectares.

Rainfall parameters used in this study were derived from the AusIFD program, with parameters generated for the location of Warragul (406535.85E, 5776618.74N Zone 55, corresponding to the Catchment Centroid) and are shown in Table 5-5. Table 5-9 below shows the input parameters used in the Rational Method estimate with results shown in Table 5-10.

Table 5-9 Rational Method input parameters

	Sub-Catchment J (Sutton St Basin Inflow)	Sub-Catchments E,F,E2,F2, G, D & G2) (Tarwin St Basin Inflow)
A = catchment area (km^2)	2.61 km^2	8.94 km^2
L = mainstream length (km)	2.7 km	5.1 km
S_e = stream slope (m/km)	17.5 m/km	15 m/km
t_c = time of concentration (h)	60 min (Bransby Williams)	100 min (Area Estimate)
FI = Fraction Impervious	0.26 (SKM RORB)	0.1 (SKM RORB)
C_{100} = runoff coefficient	0.385	0.234

Table 5-10 Rational Method peak flow estimates and SKM RORB results

Peak Flow by ARI Rational / RORB	Sub-Catchment J (Sutton St Basin Inflow)	Sub-Catchments E,F,E2,F2, G, D & G2) (Tarwin St Basin Inflow)
Q_5 (m^3/s)	5.1 / 4.0	7.8 / 5.3
Q_{100} (m^3/s)	11.82 / 13.4	18.3 / 21.0

5.4.2 Regional Estimate

As a final check, the Greater Hazel Creek catchment (Hazel and Spring Creek to the confluence with the Moe River – a study area of 46.98km²) was checked in the regional prediction equation developed by Grayson (1996), which provides estimates of peak flow for catchments adjacent to the Great Dividing Range. While this equation does not consider the impacts of urbanisation in a catchment it should provide a suitable estimation of the likely magnitude of a peak flow from the system. It is expected that the Hazel and Spring Creek catchments would produce a higher peak flow from the system than that estimated by the regional prediction equation, as areas of the catchment are highly urbanised. Grayson's equation is described by:

$$Q_{100} = 4.67A^{0.763}$$

Where: A is the area of the catchment in kilometres square.

Table 5-11 Regional Method peak flow estimate and SKM RORB results

Peak Flow	Regional Estimate (m³/s)	RORB model (Existing Conditions) (m³/s)
Q_{100} (m³/s)	88.1	95.7

5.4.3 Neil M Craigie 2010 RORB Hydrological Modelling

In 2010, Neil M Craigie Pty Ltd were engaged to develop a Surface Water Management Strategy for a future development in the northwest area of Warragul (west of Tarwin Street, north of Princes Highway) for a residential development known as the Waterford Rise Estate. One component of this study included the determination of appropriate on site detention (flood attenuation) to mitigate the increased amount of runoff from the development. This work included some hydrological modelling incorporating utilising / updating the previous RORB modelling by SKM in 2007.

Neil M Craigie Pty Ltd modified the proposed retarded conditions model to include the following items:

- Northwest Railway retention basin (RB) deleted (but retained in model for this study as per original SKM design);
- Subareas A-C in SKM model re-subdivided to better simulate runoff at critical locations;
- Northern Tarwin Street RB retained as by SKM;
- New wetland (WL) / RB upstream of Princes Hwy in UGB area (referred to as Sutton Street West WL / RB-final design); and
- New WL/RB in Tarwin Street floodplain (Tarwin Street Lower WL / RB 2.55 ha wetland area at 111.20 m NTWL);

As the model is broadly consistent with the SKM Model a separate deconstruction process was not considered necessary for this model. Stage storage relationships for the two proposed basins are shown in Table 5-12. No functional design drawings for the proposed basins were available for review.

Table 5-12 Proposed Basins by N Craigie (2010) - Waterford Rise Estate

	Sutton St West WL/RB		Tarwin Street Lower WL/RB	
Neil M Craigie RORB 2010	Stage mAHD	Storage (m³)	Stage mAHD	Storage (m³)
	119.5	0	111.2	0
	120	2278	111.7	13,000
	120.5	5500	112	25,000
	121	9516	112.25	35,000
	121.5	15300	112.5	45,300
	122	23216	112.75	60,700
	123	43929	112.8	63,800
	124	60000	112.85	66,000
			112.9	67,500
			113	74,500

5.4.4 Water Technology 2011 RORB Hydrological Modelling

Water Technology was engaged by BBSC to design and model a proposed combined Retarding Basin and Wetland located north of Stoddarts Road. This proposed works (Stoddarts Road Retardation Basin and wetland) formed part of Council's overall Development Contributions Plan for the Shire as recommended from the 2007 SKM study. The work by Water Technology comprised the following elements:

- Design of a detention system consisting of two storages connected in series;
- Design of three Sedimentation Basins, treating both the Chesterfield development and greater catchment flows;
- Design of a single wetland cell of approximately 1.0 ha in size, treating both the Chesterfield development and greater catchment flows; and
- Realignment of the unnamed waterway.

To design the retarding basin system, a new RORB hydrological model was developed. The existing conditions model was calibrated to a Rational Method peak flow estimate. The RORB model was then updated to represent developed conditions, and the basin system was designed. Ultimately the stage storage relationship shown in Table 5-13 was determined from this study. For further details of this study see *Stoddarts Road Retarding Basin / Wetland Functional Design Report by Water Technology, October 2011*.

Table 5-13 Functional design Basin stage storage relationship by Water Technology 2011

	SR RB nth*		SR RB sth*	
	Stage mAHD	Storage (m^3)	Stage mAHD	Storage (m^3)
Water Technology RORB 2011	113.5	0	112.0	0
	113.6	160.31	112.1	0.1
	113.7	328.76	112.2	0.2
	113.8	505.55	112.3	985
	113.9	690.87	112.4	2125
	114.0	884.91	112.5	3,291
	114.1	1,087.9	112.6	4,486
	114.2	1,300	112.7	5,708
	114.3	1,521.4	112.8	7,061
	114.4	1,752.3	112.9	8,442
	114.5	1,993	113.0	9,851
	114.6	2,243.6	113.1	11,290
	114.7	2,504.2	113.2	12,757
	114.8	2,775.2	113.3	14,253
	114.9	3,056.7	113.4	15,778
	115.0	3,348.9	113.5	17,333
	115.1	3,651.9	113.6	18,917
	115.2	3,966.1	113.7	20,531
	115.3	4,291.6	113.8	22,174
	115.4	4,628.8	113.9	23,849
	115.5	4,978.8	114.0	25,554
	115.6	5,345.4	114.1	27,293
	115.7	5,722.4	114.2	29,075
	115.8	6,110.0	114.3	30,889
	115.9	6,508.7	114.4	32,734
	116.0	6,919.4		

* SR RB nth is a smaller retardation basin located at the northern section of the composite RB
SR RB sth is a larger retardation basin located at the southern section of the composite RB

5.5 Summary

Following a thorough review of the SKM RORB model it was deemed appropriate to provide a quality representation of the Hazel Creek and Spring Creek catchment hydrology. In the absence of quality calibration data (gauging stations) the model applied appropriate catchment parameters which produced results close to the empirical peak flow estimations. Other subsequent modelling has either revisited and modified the SKM RORB model, or has been completed by Water Technology.

Following the RORB model review, Water Technology provided recommendations for system conditions and input changes which would likely increase the accuracy of the hydrological outputs. These conditions (below) were approved by BBSC project manager on the 4th of April 2012.

- BBSC formally acknowledged to Water Technology that it accepts the results of the SKM RORB modelling from 2007;
- BBSC decided to adopt the SKM developed conditions model with the retardation basins in place as the base case modelling platform for this study;
- BBSC formally acknowledged that Stage-Storage relationships for the existing basins (Tarwin, Landsborough and Booker Park) be determined from analysis of the LiDAR topography dataset;
- Recent modelling work by N Craigie (2010) for the Waterford Rise estate was included in the base case modelling; and
- Modelling work by Water Technology for the Stoddarts Road retarding basins would be included in basin optimisation (RORB) modelling by Water Technology in 2012;

6. EXISTING RETARDING BASIN OPTIMISATION

6.1 Overview

The RORB hydrology model (SKM, 2007) review was presented to the BBSC in April 2012. Following on from the main RORB model review and update presented in Section 5.1, this Section presents the findings of the review of the basin optimisation process undertaken for the Landsborough Road, Tarwin Street and Sutton Street (Brooker Park) basins. The proposed modifications were presented to Baw Baw Shire for comments and approval in July 2012. Following receipt of comments, the functional designs for the basin optimisation measures were completed. The functional designs are included in Appendix A of this report.

The RORB modelling detailed in Section 5.1 adopted the 'future developed conditions' modelling as described by SKM, with some modifications applied by Water Technology to include the Waterford Rise residential estate RORB modelling (Neil M Craigie, 2010) for a development located in the northwest area of Warragul (west of Tarwin Street, north of Princes Highway). As detailed in Section 5.1, the fraction impervious map for the study area was updated and applied to the 2012 RORB modelling for the basin optimisation process.

Following discussion with the BBSC, it was determined that the following general criteria would be applied to basin optimisation across all three major basins (Landsborough / Tarwin / Brooker Park):

- Where practical, major structural modifications to the existing outlets would be avoided;
- Basin volume augmentation (i.e. excavation of the basin footprint) would be avoided where possible;
- Basin optimisation should aim specifically to provide substantial reductions in peak basin outflows for shorter duration (thunderstorm) rainfall events (less than / equal to ~2hours); and
- Notwithstanding the focus on short duration events, the modified basins should not increase flows for any of the modelled events.

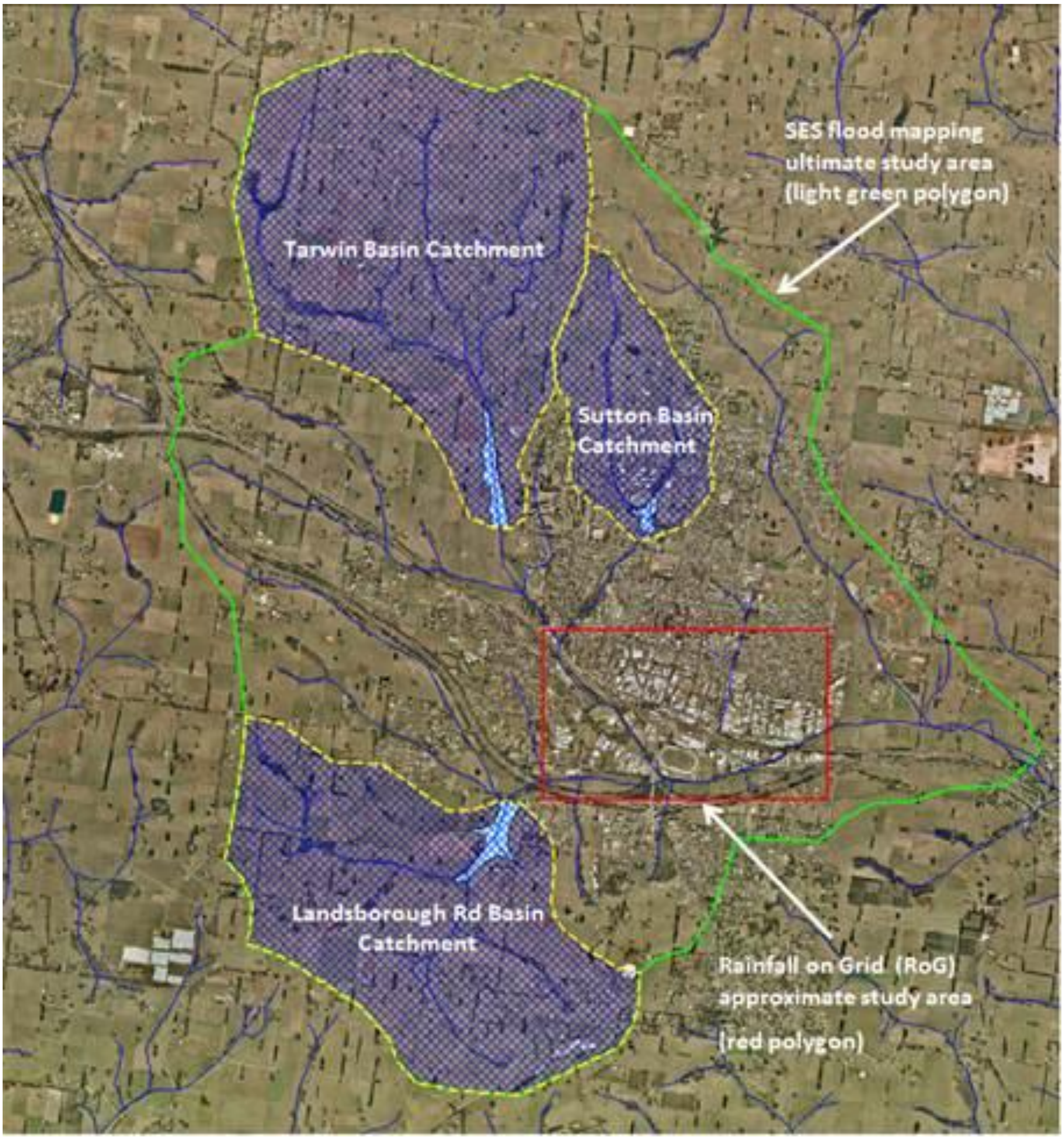

Figure 6-1 Three existing retarding basins in the Warragul Study area

6.2 Landsborough Road Basin

6.2.1 Basin Overview

The Landsborough Road basin is situated south west of the corner of Landsborough Road and Butlers Track in Warragul. The basin is located on the south eastern edge of the current urbanised area of Warragul, and the upstream land use is predominantly agricultural. A spillway approximately 260m in length runs east to west between Butlers Track and Lot 139 Warragul-Lardner Road.

The basin crest is set at 113.21m AHD, some 4.55m above the basin floor level. A 5.0m wide broad-crested weir assembly is located approximately 70m west of Butlers Track, and consists of a smooth concrete weir set at 112.72m AHD discharging onto a stepped rock filled gabion spillway. At the floor level of the basin (108.66m AHD), approximately 45m further west of the spillway, a set of twin

900mm circular culvert pipes provides for release of low flow events. Ultimately, the Landsborough Road basin discharges into the waterway and runs beneath the Warragul-Lardner Road/ Landsborough Road before flowing north under the Princes Freeway and east along the line of Spring Creek.

Table 6-1 provides a summary of the key physical parameters for the Landsborough Road basin. Where Water Technology values differ from the SKM (2007) numbers, both values are stated for comparison. Water Technology (2012) basin volumes are derived from interrogation of LiDAR information provided by the WGCMA.

Table 6-1 As-constructed physical parameters for Landsborough Road basin

Basin Feature	Water Technology (2012)	SKM (2007)
Basin floor level (m AHD)	108.66	
Spillway height (m AHD)	112.72	
Spillway width (m)	5.0	
Embankment height (m AHD)	113.21	
Low flow pipe US invert (m AHD)	108.66	
Low flow pipe dia. / length (m)	0.9 / 34 (twin)	
Estimated basin volume at spillway level (m^3)	~151,000	~130,000

Figure 6-2 shows the main weir and spillway for the Landsborough Road basin viewed from both upstream (left frame) and downstream (right frame) locations. Figure 6-3 shows the location of the low flow pipes exiting the basin approximately 45m west of the spillway.

Figure 6-2 Landsborough Road basin spillway – view DS towards Landsborough Road (left frame) and US to spillway and basin crest (right frame)

Figure 6-3 Landsborough Road basin low flow culvert outlet – view US to basin crest

6.2.2 As-constructed Basin Storage Volumes

To determine the effective basin volume, the LiDAR information for the site was interrogated in 12d software and a more up to date stage - storage relationship was established. The LiDAR derived stage – storage data is considered more accurate than the previous (SKM, 2007) storage data, and has been adopted for all basin modelling.

Table 6-2 provides a comparative summary of the stage – storage relationship obtained from LiDAR (Water Technology, 2012) and the storage data used in the SKM (2007) RORB modelling. Figure 6-4 provides an aerial view of the key basin features and effective storage area footprint of the Landsborough Road basin

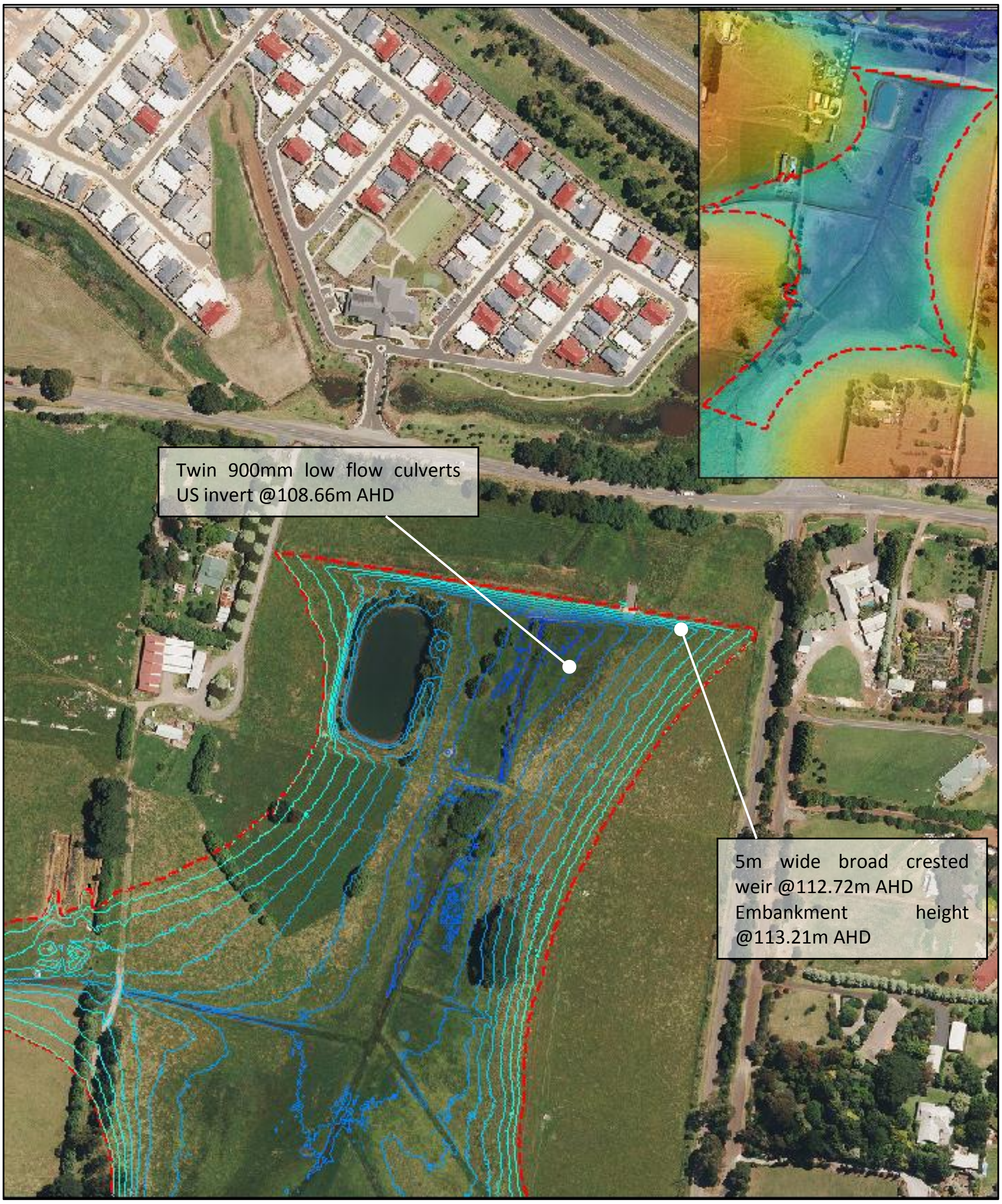

Figure 6-4 Location and key features of the Landsborough Road basin (as-constructed)

Table 6-2 Comparison of LiDAR derived (2012) vs. SKM (2007) storage volume data for Landsborough Road basin

Basin Stage (m AHD)	Water Technology (2012)	SKM (2007)
108.35		0
108.66	0	0
108.8	0.002	
109.0	0.2	50
109.2	14	
109.4	70	

Basin Stage (m AHD)	Water Technology (2012)	SKM (2007)
109.6	261	
109.8	755	
110.0	1,599	980
110.1	2,283	
110.3	4,214	
110.5	7,089	2,430
110.7	11,368	
110.9	17,626	
111.08		9,130
111.1	26,453	
111.3	37,145	
111.5	49,451	
111.6	56,200	
111.61		32,300
111.8	70,827	
112.0	86,653	
112.1	94,957	
112.2	103,523	
112.23		81,000
112.4	121,492	
112.6	140,796	
112.8	161,661	
112.83		143,000
113.0	184,379	
113.2	209,051	
113.3	222,129	
113.33		213,513
113.5	249,669	
113.7	278,847	
113.9	309,684	
114.0	325,730	308,000
114.1	341,990	
114.2	358,282	

The LiDAR derived volume indicates an increase of approximately 17,730 cubic metres (~6%) at the 114.0m AHD contour level, relative to the 2007 data. The LiDAR derived values have been utilised for all current RORB model runs for the Warragul flood study.

6.2.3 RORB Modelling – Basin Optimisation Methodology

The (2012) RORB model was run for the 100 year ARI storm for a range of storm event durations from 15 minutes through to 72 hours. For the Landsborough Road basin, the SKM (2007) RORB model used an externally generated Storage – Discharge (S-Q) relationship determined by SKM. RORB is capable of modelling basin behaviour using either weir and pipe equations, or assigned S-Q curves. Given the availability of high quality LiDAR data, it was decided that the 2012 RORB modelling would utilise the internal weir/pipe flow functionality with the RORB engine. The iterative design function within RORB was also utilised to test various outlet configurations across multiple storm durations.

6.2.4 Basin Optimisation

The (2012) RORB model was initially run to generate revised baseline results for the full set of 100 year ARI storm durations. The results of the baseline model runs were compared to the SKM (2007) model results to assess the impact of utilising updated (LiDAR derived) basin volume data and changing from externally generated S-Q curves to weir and pipe based calculations. Table 6-3 shows the results of the (2012) RORB baseline modelling for the as-constructed basin.

Table 6-3 Landsborough Road baseline RORB model run 100 year ARI

Landsborough Road Basin – Baseline Run			
100yr ARI duration	**Max. water elevation (m AHD)**	**Freeboard to basin embankment (m)**	**Peak discharge (m^3/s)**
15min	110.03	3.18	2.99
20min	110.14	3.07	3.30
25min	110.42	2.79	3.94
30min	110.62	2.59	4.29
45min	110.99	2.22	4.94
1hr	111.22	1.99	5.30
1.5hr	111.54	1.67	5.73
2hr	111.75	1.46	6.04
3hr	112.03	1.18	6.39
4.5hr	112.30	0.91	6.71
6hr	112.47	0.74	6.91
9hr	112.76	0.45	7.36
12hr	112.90	0.31	8.01
18hr	112.94	0.27	8.30
24hr	112.97	0.24	8.51
30hr	113.17	0.04	10.13
36hr	113.23	-0.02	10.63
48hr	113.18	0.03	10.17
72hr	112.50	0.71	6.95

The SKM (2007) modelling returned a peak basin outflow of 13 m^3/s for the 30 hour duration storm event. Results of 2012 baseline modelling indicate that the 36hr storm event produces a peak flow of 10.63m^3/s with a maximum water elevation of 113.23m AHD. Given the increase in effective basin volume (refer Table 6-2) the difference between the 2007 and 2012 model results are not considered to be anomalous. The 2012 modelling also indicates overtopping of the basin embankment during 100 year ARI peak flow conditions.

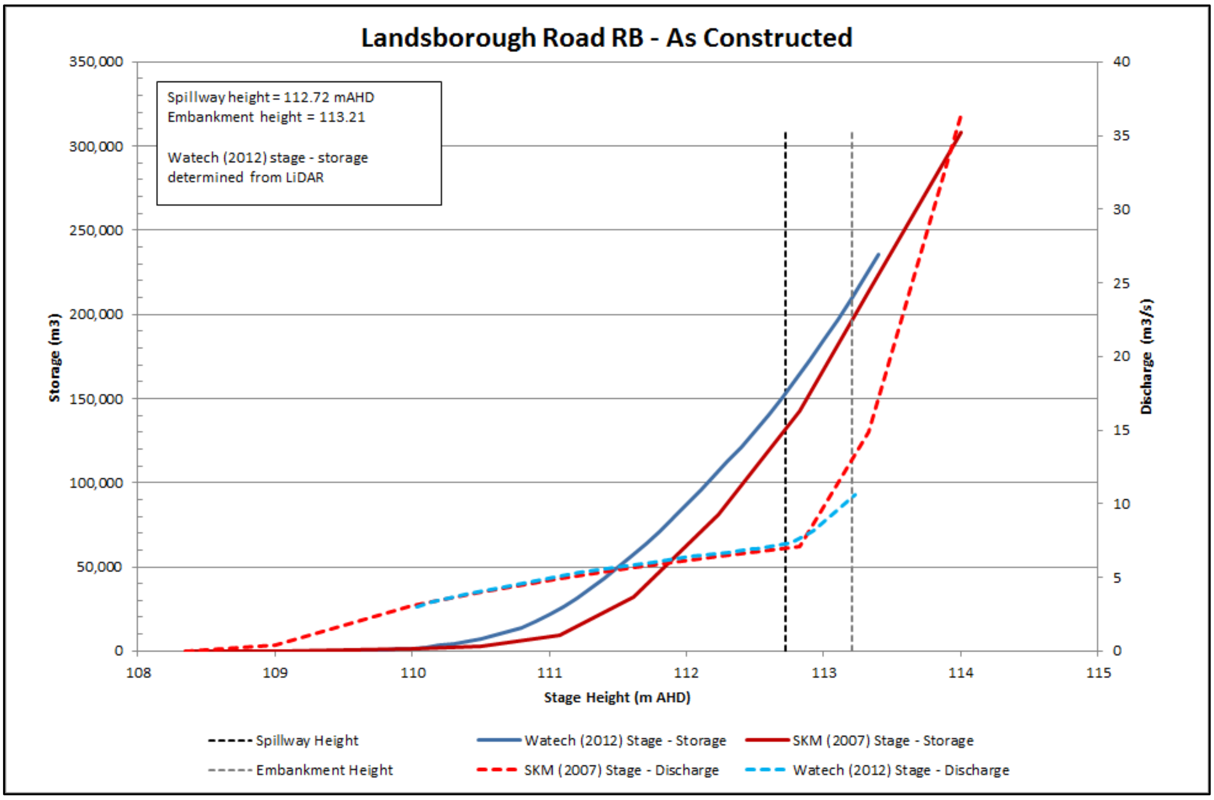

Figure 6-5 As-constructed basin performance curves – Landsborough Road basin

Figure 6-5 provides a graphical summary of the as-constructed basin performance curves for the Landsborough Road basin. Key points of note include:

- The increased LiDAR derived storage volume profile (solid blue line) for 2012;
- Confirmation of the 2012 stage – discharge curve exhibiting expected behaviour at the spillway engagement height (vertical dashed black line); and
- Anomalous behaviour of the SKM (2007) stage – discharge curve whereby the increases in discharges do not coincide with the spillway / embankment heights.

An iterative design process was then undertaken to assess the effects of varying embankment heights, spillway configurations and effective culvert pipe diameters. Initially, a scenario providing for an increase in embankment height of 0.9m, and raising the spillway height by 0.68m was modelled, with the inclusion of an additional 750mm diameter orifice plate on one of the low flow 900mm culvert pipes. While this scenario provided for between 12-20% reduction in peak outflows (20% reduction for the 1 hour event), Water Technology considered that the inclusion of an internal coffer dam conceptually represented as a 2.15m high, 10m x 10m x 10m wall around the upstream invert of the low flow culverts, and with a single 525mm culvert at the base, would further assist in reducing the peak outflows for sub 1 hour events.

The concept of an internal coffer dam was discussed with BBSC at a project meeting on the 13th June 2012, where it was agreed that such a configuration could be considered. The proposed (optimised) basin configuration for the Landsborough Road basin therefore consists of:

- Raising embankment height to 114.1m AHD (0.9m increase);
- Raising spillway height to 113.4m AHD (0.68m increase);
- Installing a 750mm orifice plate onto the US end of <u>one</u> of the existing 900mm culvert pipes; and

- Constructing an internal coffer wall (110.8m AHD crest) around the upstream invert location of the existing culvert pipes, with a single circular 525mm culvert at the base of the coffer dam wall.

Table 6-3 shows the results of the (2012) optimised conditions basin modelling for the Landsborough Road basin.

Table 6-4 Landsborough Road optimised RORB model run 100 year ARI

Landsborough Road Basin – Optimised Run				
100yr ARI duration	**Max. water elevation (m AHD)**	**Freeboard to basin embankment (m)**	**Peak discharge (m^3/s)**	**Optimised conditions % of baseline flows**
15min	110.63	3.57	0.82	* 27%
20min	110.73	3.47	0.85	* 26%
25min	110.8	3.4	0.87	* 22%
30min	110.89	3.31	1.60	* 37%
45min	110.82	3.38	3.97	80%
1hr	111.03	3.17	4.26	80%
1.5hr	111.42	2.78	4.73	83%
2hr	111.71	2.49	5.08	84%
3hr	112.1	2.1	5.49	86%
4.5hr	112.5	1.7	5.87	87%
6hr	112.78	1.42	6.13	89%
9hr	113.19	1.01	6.50	88%
12hr	113.44	0.76	6.80	85%
18hr	113.52	0.68	7.12	86%
24hr	113.59	0.61	7.51	88%
30hr	113.75	0.45	8.64	85%
36hr	113.82	0.38	9.17	86%
48hr	113.74	0.46	8.53	84%
72hr	112.96	1.24	6.29	91%

** Represents events effectively retarded by the coffer dam*

The optimised conditions (2012) modelling provides for substantial reductions in the sub 1 hour duration 100 year ARI events. With the proposed configuration in place, the peak basin outflow is 9.17m^3/s during the 36hr 100 year ARI storm event. Minimum 100 year ARI freeboard under optimised conditions is estimated at 0.38m.

Figure 6-6 provides a graphical summary of the optimised basin performance curves for the Landsborough Road basin. Key points of note include:

- Engagement of the coffer dam up 110.8m AHD (vertical red dashed line);
- Transition from weir to orifice flow represented as rapid increase once the coffer dam is flooded; and
- Confirmation of the 2012 stage – discharge curve exhibiting expected behaviour at the spillway engagement height (vertical dashed black line).

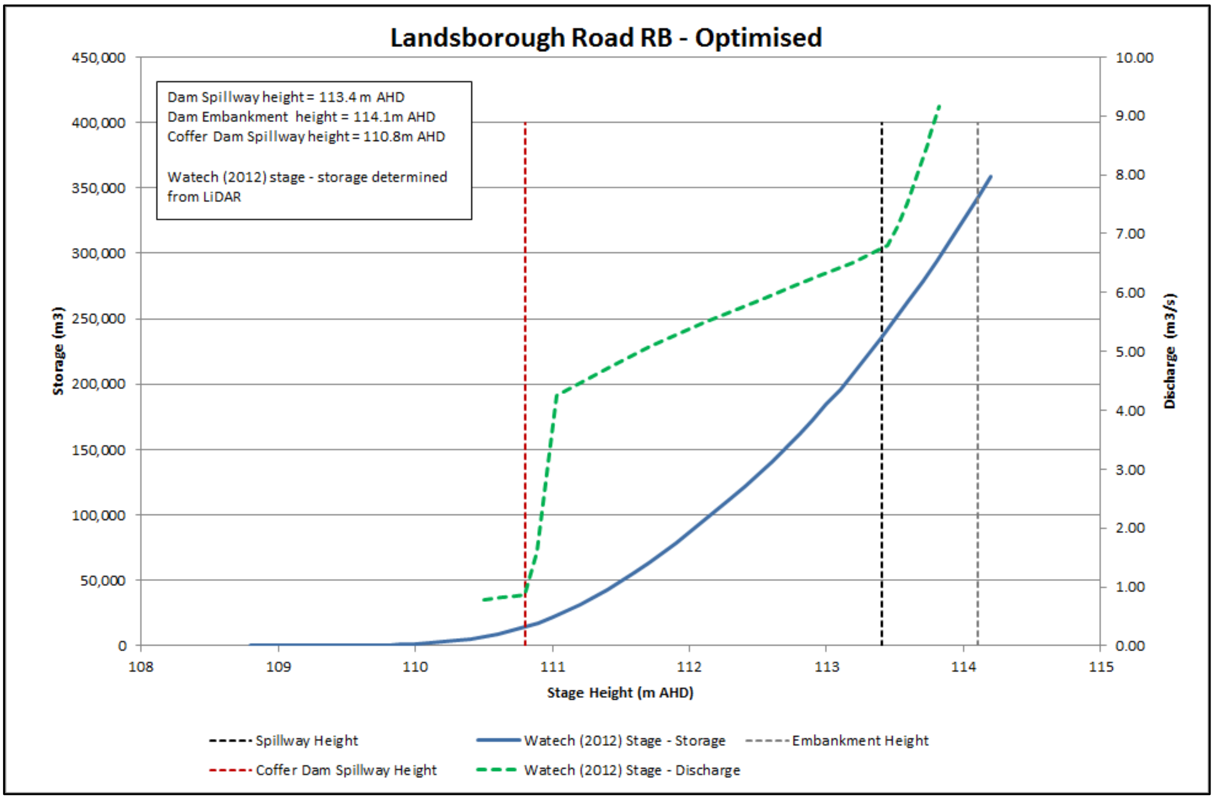

Figure 6-6 Optimised basin performance curves – Landsborough Road basin

6.3 Tarwin Street Basin

6.3.1 Basin Overview

The Tarwin Street basin is situated west of the intersection of Tarwin Street/ Pharaoh's Road/ Sutton Street in Warragul. The basin is located on the western edge of the developed area of Warragul, with upstream land use predominantly for agricultural purposes. A spillway approximately 170m in length runs east to west across the natural gully to the west of Pharaoh's Road.

The basin crest is set at 119.82m AHD, some 6.83m above the basin floor level. A 25.0m wide broad-crested weir assembly is located at the western end of the embankment, and consists of a rock filled gabion weir set at 119.2m AHD discharging onto a stepped rock filled gabion spillway. At the floor level of the basin (112.99m AHD), approximately 55m further east of the spillway, a set of twin box culverts (estimated 1200mm x 900mm) with 900mm circular orifice plates on the upstream end, provides for release of low flow events.

Table 6-5 provides a summary of the key physical parameters for the Tarwin Street basin. Where Water Technology values differ from the SKM (2007) numbers, both values are stated for comparison. Water Technology (2012) basin volumes are derived from interrogation of LiDAR information provided by the WGCMA.

Table 6-5 As-constructed physical parameters for Tarwin Street basin

Basin Feature	Water Technology (2012)	SKM (2007)
Basin floor level (m AHD)	112.99	
Spillway height (m AHD)	119.2	
Spillway width (m)	25.0	
Embankment height (m AHD)	119.82	
Low flow pipe US invert (m AHD)	112.99	
Low flow pipe dia. / length (m)	0.9 / 41.5 (twin)	
Estimated basin volume at spillway level (m^3)	~183,823	~202,533

Figure 6-7 shows the main weir and spillway for the Tarwin Street basin viewed from both upstream (left frame) and downstream (right frame) locations. Figure 6-8 shows the location of the low flow pipes exiting the basin approximately 55m east of the spillway.


Figure 6-7 Tarwin Street basin spillway – view east towards Pharaoh's Road (left frame) and US to spillway and basin crest (right frame)

Figure 6-8 Tarwin Street basin low flow culvert outlet – view of US outlet area. Note the significant ponding of water on the US side at time of site visit

Figure 6-9 provides an aerial view of the key basin features and effective storage area footprint of the Tarwin Street basin.

6.3.2 As-constructed Basin Storage Volumes

To determine the effective basin volume, the LiDAR information for the site was interrogated in 12d software and stage - storage relationship was established. The LiDAR derived stage – storage data is considered more accurate than the previous (SKM, 2007) storage data, and has been adopted for all basin modelling.

Table 5-2 provides a comparative summary of the stage – storage relationship obtained from LiDAR (Water Technology, 2012) and the storage data used in the SKM (2007) RORB modelling.

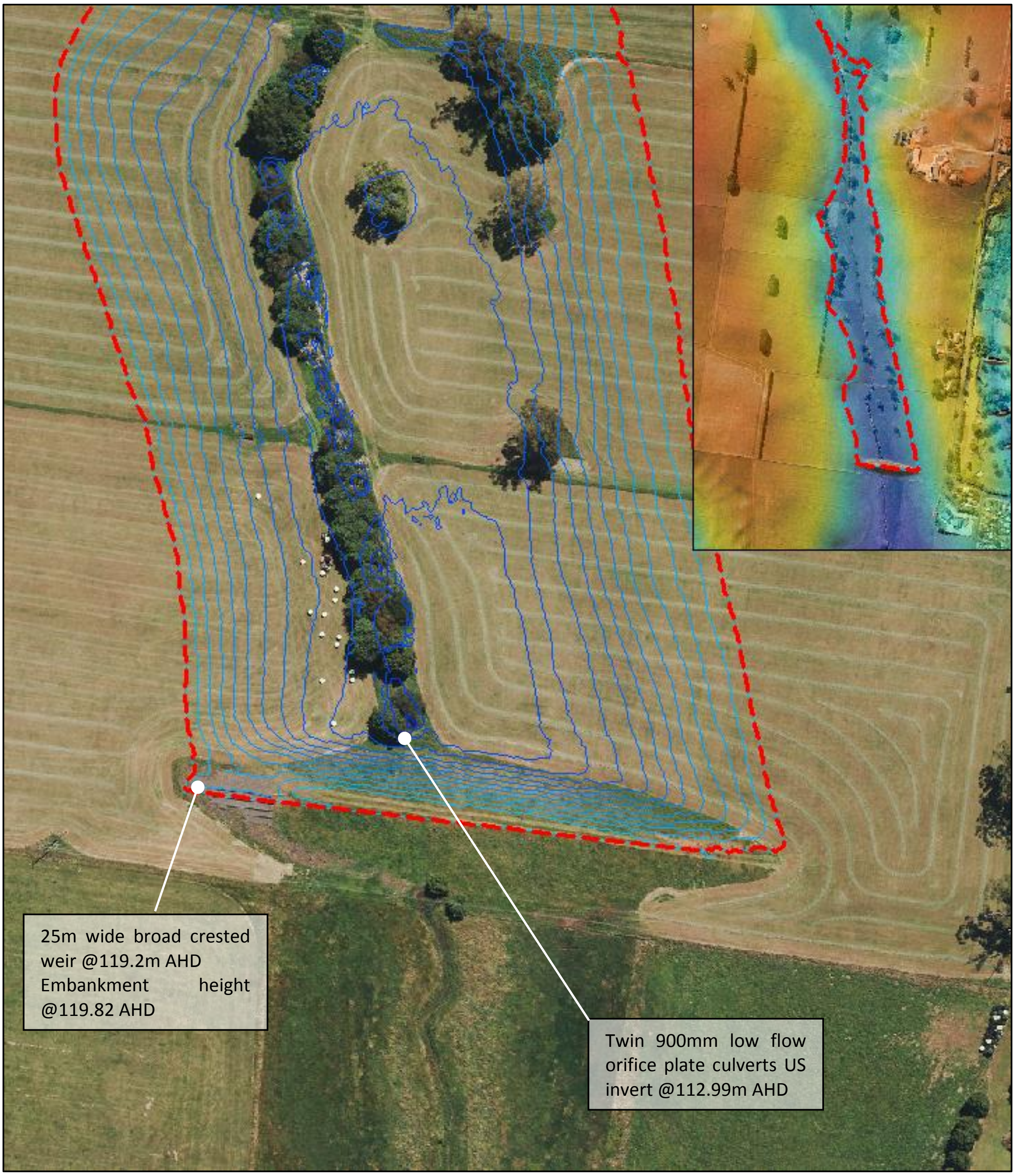

Figure 6-9 Location and key features of the Tarwin Street basin (as-constructed)

Table 6-6 Comparison of LiDAR derived (2012) vs. SKM (2007) storage volume data for Tarwin Street basin

Basin Stage (m AHD)	Water Technology (2012)	SKM (2007)
112.9	0	0
113.5	0	40
113.9	0.03	
114.0	0.40	190
114.1	1.80	
114.3	11.4	
114.5	33.9	520
114.7	88	
114.9	346	
115.0	595	1,970
115.1	919	
115.3	1,914	
115.5	3,597	6,490
115.7	5,837	
115.9	8,652	
116.0	10,281	14,770
116.3	15,986	
116.5	20,609	26,870
116.8	29,187	
117.0	36,138	44,370
117.3	48,715	
117.5	58,560	68,970
117.8	75,330	
118.0	87,677	100,660
118.5	122,984	139,000
119.0	165,142	183,800
119.1	174,344	193,167
119.2	183,823	202,533
119.3	193,591	211,900
119.4	203,644	221,267
119.5	213,973	230,633
119.6	224,606	240,000
119.7	235,552	250,000
119.8	246,875	260,357
119.9	258,561	270,000
120.2	295,327	300,000
120.4	320,097	
120.6	344,883	
120.8	369,669	
121.0	394,455	
121.2	419,240	

The LiDAR derived volume indicates a decrease of approximately 4,673 cubic metres (~2%) at the 120.2m AHD contour level, relative to the 2007 data; while the same comparison at the spillway level (119.2m AHD) shows a decrease of approximately 18,710 cubic metres (~10%). The LiDAR derived values have been utilised for all current RORB model runs for the Warragul flood study.

6.3.3 RORB Modelling – Basin Optimisation Methodology

The (2012) RORB model was run for the 100 year ARI storm for storm event durations of 15 minutes through to 72 hours. For the Tarwin Street basin, the SKM (2007) RORB model used an externally generated Storage – Discharge (S-Q) relationship determined by SKM. RORB is capable of modelling basin behaviour using either weir and pipe equations, or assigned S-Q curves. Given the availability of high quality LiDAR data, it was decided that the 2012 RORB modelling would utilise the internal weir/pipe flow functionality with the RORB engine. The iterative design function within RORB was also utilised to test various outlet configurations across multiple storm durations.

6.3.4 Basin Optimisation

The (2012) RORB model was initially run to generate revised baseline results for the full set of 100 year ARI storm durations. The results of the baseline model runs were compared to the SKM (2007) model results to assess the impact of utilising updated (LiDAR derived) basin volume data and changing from externally generated S-Q curves to weir and pipe based calculations. Table 6-7 shows the results of the (2012) RORB baseline modelling for the as-constructed basin.

Table 6-7 Tarwin Street baseline RORB model run 100 year ARI

Tarwin Street Basin – Baseline Run			
100yr ARI duration	**Max. water elevation (m AHD)**	**Freeboard to basin embankment (m)**	**Peak discharge (m^3/s)**
15min	114.66	5.16	4.00
20min	114.75	5.07	4.29
25min	114.74	5.08	4.28
30min	114.91	4.91	4.52
45min	115.23	4.59	5.07
1hr	115.61	4.21	5.56
1.5hr	116.55	3.27	6.73
2hr	117.1	2.72	7.32
3hr	117.73	2.09	7.93
4.5hr	118.31	1.51	8.47
6hr	118.71	1.11	8.82
9hr	119.31	0.51	10.70
12hr	119.46	0.36	14.87
18hr	119.46	0.36	14.74
24hr	119.43	0.39	13.80
30hr	119.55	0.27	18.04
36hr	119.57	0.25	18.63
48hr	119.53	0.29	16.97
72hr	118.87	0.95	8.96

The SKM (2007) modelling reported a peak basin outflow of 11.9m^3/s for the 36 hour duration storm event. The difference between the 2012 peak (18.63m^3/s) and the 2007 reported peak (11.9m^3/s) initially indicated a modelling error until the baseline RORB model was run with an increased embankment height of 500mm (119.7m AHD). The SKM (2007) report recommended that the embankment height be raised by 500mm and it appears that the tabled results in the SKM report assumed that the embankment increase had already taken place. When run with an embankment height of 119.7m AHD, the SKM 36hr peak outflow was reduced to 16.42m^3/s, still less than the results obtained from the revised Water Technology modelling.

Results of 2012 true baseline (embankment at 119.2m AHD) modelling indicate that the 36hr storm event produces a peak flow of 18.63m^3/s with a maximum water elevation of 119.57m AHD. Given the decrease in effective basin volume (refer Table 5-2) the difference between the 2007 and 2012 model results are not considered to be anomalous. Furthermore, review of the Tarwin Street basin performance curves (refer Figure 6-10) confirm that the 2007 SKM stage–discharge curve assumes an embankment height of 119.7m AHD (red dashed line in Figure 6-10). Water Technology believes that Table 6-7 provides a more appropriate estimate of current basin outflows.

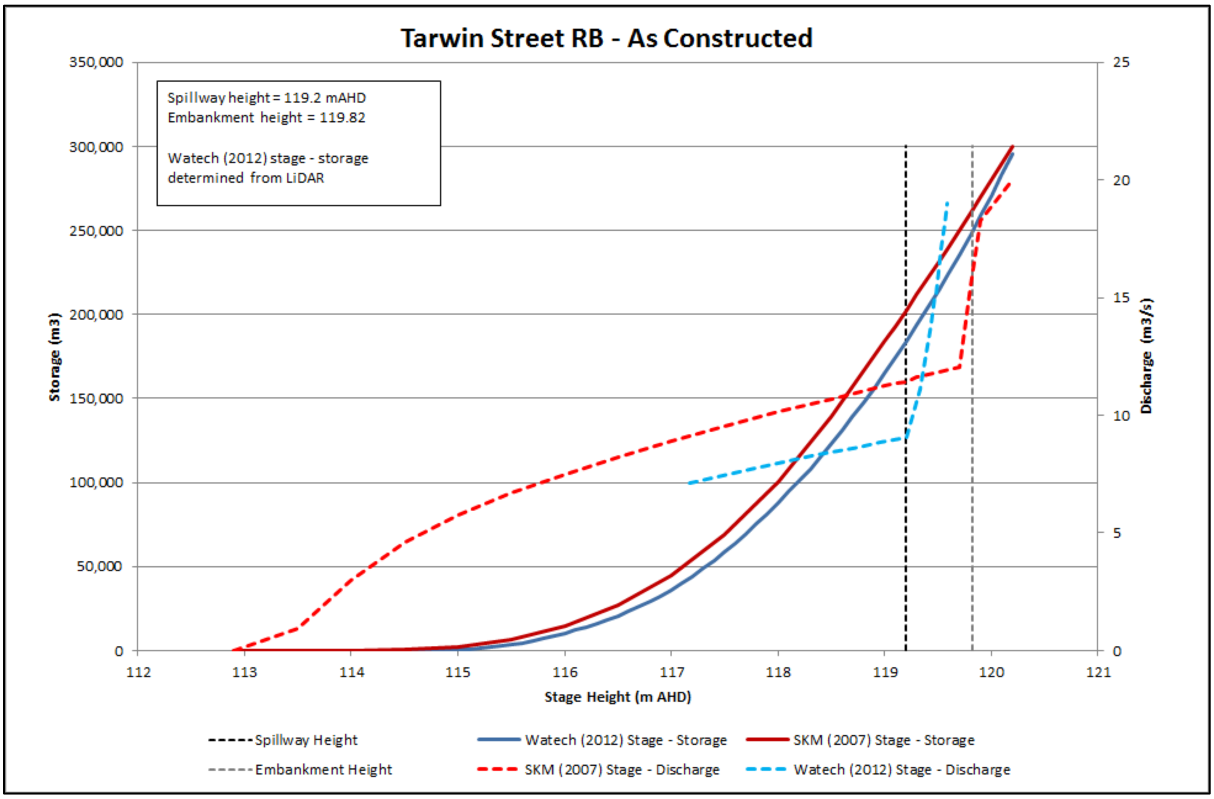

Figure 6-10 As-constructed basin performance curves – Tarwin Street basin

Figure 6-10 provides a graphical summary of the as-constructed basin performance curves for the Tarwin Street basin. Key points of note include:

- The decreased LiDAR derived storage volume profile (solid blue line) for 2012;
- Confirmation of the 2012 stage – discharge curve exhibiting expected behaviour at the spillway engagement height (vertical dashed black line); and
- Anomalous behaviour of the SKM (2007) stage – discharge curve whereby the increases in discharges do not coincide with the current as-constructed spillway height.

An iterative design process was then undertaken to assess the effects of varying embankment heights, spillway configurations and effective culvert pipe diameters. Initially, a scenario providing for an increase in embankment height of 0.5m (in line with the SKM 2007 recommendation), and similarly raising the spillway height by 0.5m was modelled. While this scenario provided for reduction in peak outflows for longer duration events (12% reduction for the 36 hour event), Water Technology considered that the inclusion of an internal coffer dam conceptually represented as a 2.75m high, 10m x 10m x 10m wall around the upstream invert of the low flow culverts, and with a single 750mm culvert at the base, would further assist in reducing the peak outflows for sub 1 hour events. Additional increases in embankment height were also considered, recognising that a future road is being proposed across the Tarwin Street basin embankment and represents an opportunity for potential upgrades.

The details of the planned future road were discussed with BBSC in July 2012 and the following concept design parameters were discussed:

- Top road width to be set at 24m;
- Spillway design to comprise a suitable slot arrangement (box culvert or free span) underneath the future road to act as the primary spillway; and
- Water Technology to conduct a preliminary assessment of the 500 year ARI rainfall event.

RORB was utilised to simulate the 500 year ARI event. AR&R presents temporal patterns for conversion of design rainfall depths to equivalent design floods. RORB extrapolates the IFD curves from AR&R to generate rainfall for the 500 year event. This approach is entirely consistent with current best practice. The reader is referred to the RORB manual for further information if required.

The concept of an internal coffer dam was discussed with BBSC at a project meeting on the 13th June 2012, where it was agreed that such a configuration could be considered. The proposed (optimised) basin configuration for the Street basin therefore consists of:

- Raising embankment height to 121.2m AHD (1.38m increase, as opposed to 0.5m recommended by SKM);
- Raising spillway height to 120.0m AHD (0.8m increase) and introducing 3 x 2750mm W x 600m H box culverts to form the spillway under the future road;
- Installing an additional 750mm orifice plate onto <u>one</u> of the existing 900mm orifice plates on the US culvert assembly; and
- Constructing an internal coffer wall (115.75m AHD crest) around the upstream invert location of the existing box culverts, with a single 750mm circular culvert at the base of the coffer dam wall.

Given the multiple box culverts proposed as the primary spillway, it was also decided that XPSTORM would be utilised to generate the optimised scenario stage – discharge (S-Q) curve for the Tarwin Street basin.

Table 6-8 shows the results of the (2012) optimised) basin modelling for the Tarwin Street basin.

Table 6-8 Tarwin Street optimised RORB model run 100 year ARI

Tarwin Street Basin – Optimised Run				
100yr ARI duration	**Max. water elevation (m AHD)**	**Freeboard to basin embankment (m)**	**Peak discharge (m^3/s)**	**Optimised conditions % of baseline flows**
15min	115.27	5.68	1.92	* 48%
20min	115.37	5.58	1.97	* 46%
25min	115.44	5.51	2.01	* 47%
30min	115.76	5.19	2.37	* 52%
45min	115.88	5.07	4.09	81%
1hr	116.21	4.99	4.61	83%

1.5hr	117.04	4.16	5.31	79%
2hr	117.53	3.67	5.68	78%
3hr	118.14	3.06	6.12	77%
4.5hr	118.74	2.46	6.51	77%
6hr	119.15	2.05	6.77	77%
9hr	119.76	1.44	7.13	67%
12hr	120.12	1.08	7.79	52%
18hr	120.29	0.91	9.10	62%
24hr	120.41	0.79	10.18	74%
30hr	120.6	0.6	12.08	67%
36hr	120.64	0.56	12.90	69%
48hr	120.59	0.61	11.93	70%
72hr	119.75	1.45	7.13	80%

** Represents events effectively retarded by the coffer dam*

The optimised conditions (2012) modelling provides for substantial reductions in the sub 1 hour duration 100 year ARI events. With the proposed configuration in place, the peak basin outflow is 12.90m^3/s during the 36hr 100 year ARI storm event. Minimum 100 year ARI freeboard (to the bank height of 121.2m AHD) under optimised conditions is estimated at 0.56m.

Figure 6-11 provides a graphical summary of the optimised basin performance curves for the Tarwin Street basin. Key points of note include:

- Engagement of the coffer dam up 115.75m AHD (vertical red dashed line);
- Transition from weir to orifice flow represented as rapid increase once the coffer dam is flooded; and
- Confirmation of the 2012 stage – discharge curve exhibiting expected behaviour at the new spillway engagement height (vertical dashed black line).

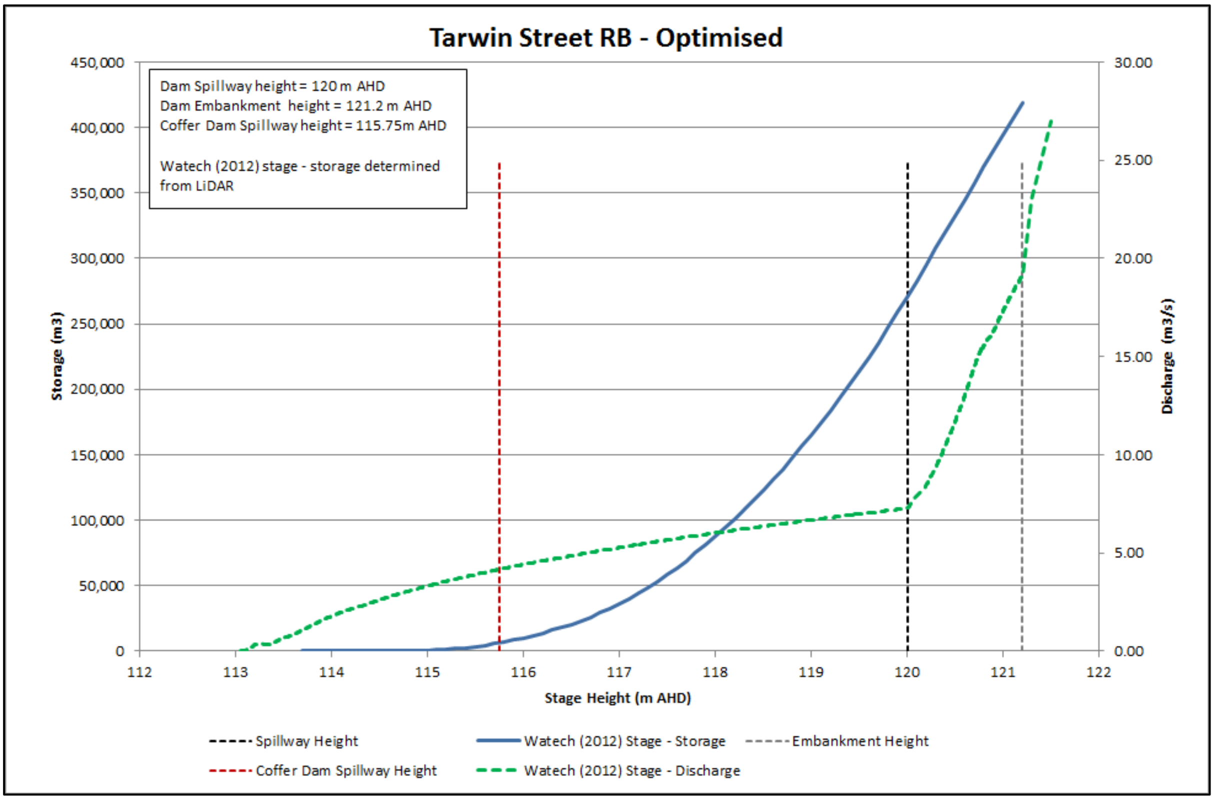

Figure 6-11 Optimised basin performance curves – Tarwin Street basin

Table 6-9 Tarwin Street optimised RORB model run 500 year ARI

Tarwin Street Basin – Optimised Run			
100yr ARI duration	**Max. water elevation (m AHD)**	**Freeboard to basin embankment (m)**	**Peak discharge (m^3/s)**
1hr	117.62	3.58	5.74
1.5hr	118.34	2.86	6.25
2hr	118.83	2.37	6.57
3hr	119.47	1.73	6.96
4.5hr	120.10	1.1	7.66
6hr	120.46	0.74	10.91
9hr	120.91	0.29	15.86
12hr	121.04	0.16	19.90
18hr	121.04	0.16	21.43
24hr	121.10	0.1	18.40
30hr	121.15	0.05	26.74
36hr	121.40	-0.2	33.00
48hr	121.17	0.03	30.93
72hr	120.79	0.41	14.28

6.4 Brooker Park Basin

6.4.1 Basin Overview

The Brooker Park (also referred to as Sutton Street) basin is situated north of Sutton Street in Warragul, and is bounded by Charles Street (east) and Bowen Street (west). The basin is located within the current urbanised area of Warragul. To the north of Sutton Street, a spillway approximately 150m in length runs east to west and defines the basin area.

The basin crest is set at 116.15m AHD, some 2.73m above the basin floor level. A 5.0m wide broad-crested concrete weir assembly is located on the western end of the embankment, and consists of a smooth concrete weir set at 115.81m AHD discharging into an engineered trapezoidal channel. At the floor level of the basin (113.42m AHD), close to the spillway, a relatively complex basin outlet configuration routes stormwater into a 1200mm culvert pipe running under the embankment. The inlet configuration for the 1200mm pipe essentially comprises:

- A 900mm x 600mm entry culvert pipe set at the upstream invert = 113.42m AHD (basin floor) – with a steel orifice plate fixed to the culvert entry which effectively restricts the entry capacity to 550mm x 600mm;
- A second (and linked) 900mm x 1200mm grated pit inlet set at invert = ~114.6m AHD - effective entry capacity assumed = 600mm x 750mm;
- A connecting chamber to direct flows into the 1200mm main culvert; and
- An energy dissipation area at the downstream end of the main culvert with deflection grates.

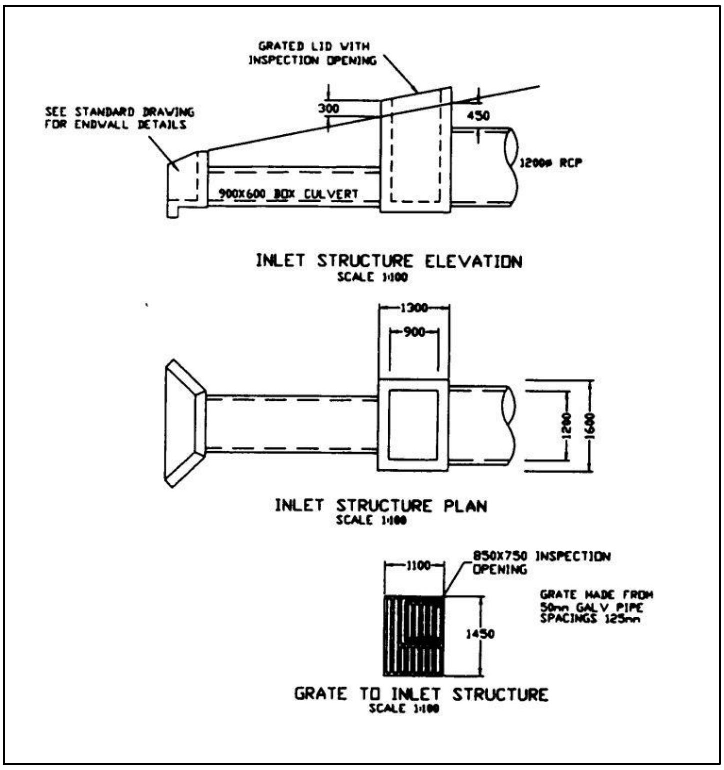

Figure 6-12 Excerpt from Miles Civil Design drawing (R-47-97/906502.DWG) from March 1997 showing inlet zone configuration for the Brooker Park basin

Table 6-10 provides a summary of the key physical parameters for the Brooker Park basin. Where Water Technology values differ from the SKM (2007) numbers, both values are stated for comparison. Water Technology (2012) basin volumes are derived from interrogation of LiDAR information provided by the WGCMA.

Table 6-10 As-constructed physical parameters for Brooker Park basin

Basin Feature	Water Technology (2012)	SKM (2007)
Basin floor level (m AHD)	113.42	
Spillway height (m AHD)	115.81	
Spillway width (m)	5.0	
Embankment height (m AHD)	116.15	
Low flow pipe invert (m AHD)	113.42	
Main culvert pipe dia. / length	1.2 / 28	
Estimated basin volume at spillway level (m^3)	~73,800	~87,000

Figure 6-13 shows the basin storage area and spillway for the Brooker Park basin. Figure 6-14 shows the grated entry to the secondary inlet zone of the basin outlet assembly.

Figure 6-13 Brooker Park basin storage area viewed from embankment (left frame) and view Upstream to spillway (right frame)

Figure 6-14 Brooker Park basin grated entry to main culvert (left) & Downstream outlet area (right)

Figure 6-15 provides an aerial view of the key basin features and effective storage area footprint of the Brooker Park basin.

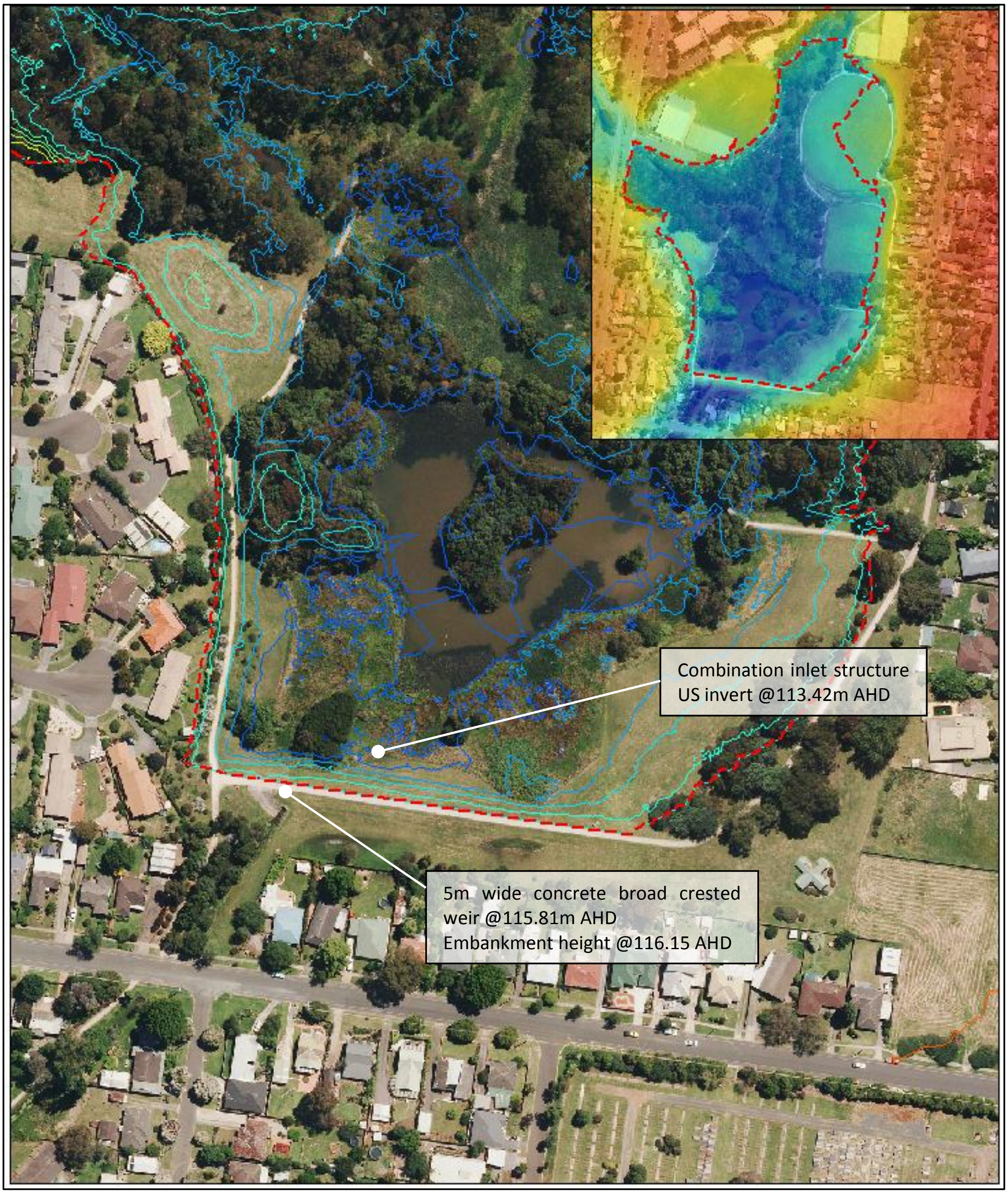

Figure 6-15 Location and key features of the Brooker Park basin (as-constructed)

6.4.2 As-constructed Basin Storage Volumes

To determine the effective basin volume, the LiDAR information for the site was interrogated in 12d software and a stage-storage relationship was established. The LiDAR derived stage–storage data is considered more accurate than the previous (SKM, 2007) storage data, and has been adopted for all basin modelling. Table 6-11 provides a comparative summary of the stage–storage relationship obtained from LiDAR (Water Technology, 2012) and the storage data used in the SKM (2007) RORB modelling.

Table 6-11 Comparison of LiDAR derived (2012) vs. SKM (2007) storage volume data for Brooker Park basin

Basin Stage (m AHD)	Water Technology (2012)	SKM (2007)
113.4	-	
113.6	-	256
113.8	-	493
113.9	-	
114	0	780
114.1	2	
114.2	18	
114.3	153	8,264
114.4	595	
114.5	1,480	
114.6	2,946	16,580
114.7	5,126	
114.8	8,176	
114.9	12,103	
115	16,766	28,500
115.1	22,088	36,900
115.2	27,992	45,300
115.3	34,469	53,700
115.4	41,458	62,100
115.5	48,894	70,500
115.6	56,744	
115.7	65,024	
115.8	73,717	
115.9	82,836	
116	92,326	112,500
116.1	102,163	
116.2	112,323	134,850
116.3	122,677	

The LiDAR derived volume indicates a decrease of approximately 22,527 cubic metres (~20%) at the 116.20m AHD contour level, relative to the 2007 data. The LiDAR derived values have been utilised for all current RORB model runs for the Warragul flood study.

6.4.3 RORB Modelling – Basin Optimisation Methodology

The (2012) RORB model was run for the 100 year ARI storm for storm event durations of 15 minutes through to 72 hours. For the Brooker Park basin, the SKM (2007) RORB model used an externally generated Storage–Discharge (S-Q) relationship determined by SKM. RORB is capable of modelling basin behaviour using either weir and pipe equations, or assigned S-Q curves.

Given the availability of high quality LiDAR data, and the relative complexity of the outlet structure, it was decided that the 2012 RORB modelling would review the S-Q relationship for the basin to generate a more accurate estimate of the S-Q curve. The hydraulic software package XP-STORM was utilised to simulate the basin and outlet structure and generate a revised S-Q curve for Brooker Park.

An outlet structure matching the configuration outlined in Section 6.1 was modelled in XP-STORM, and the basin was gradually filled to create a (S-Q) relationship to be used in RORB. Figure 6-16 shows the 2012 S-Q curve from the XP-STORM analysis.

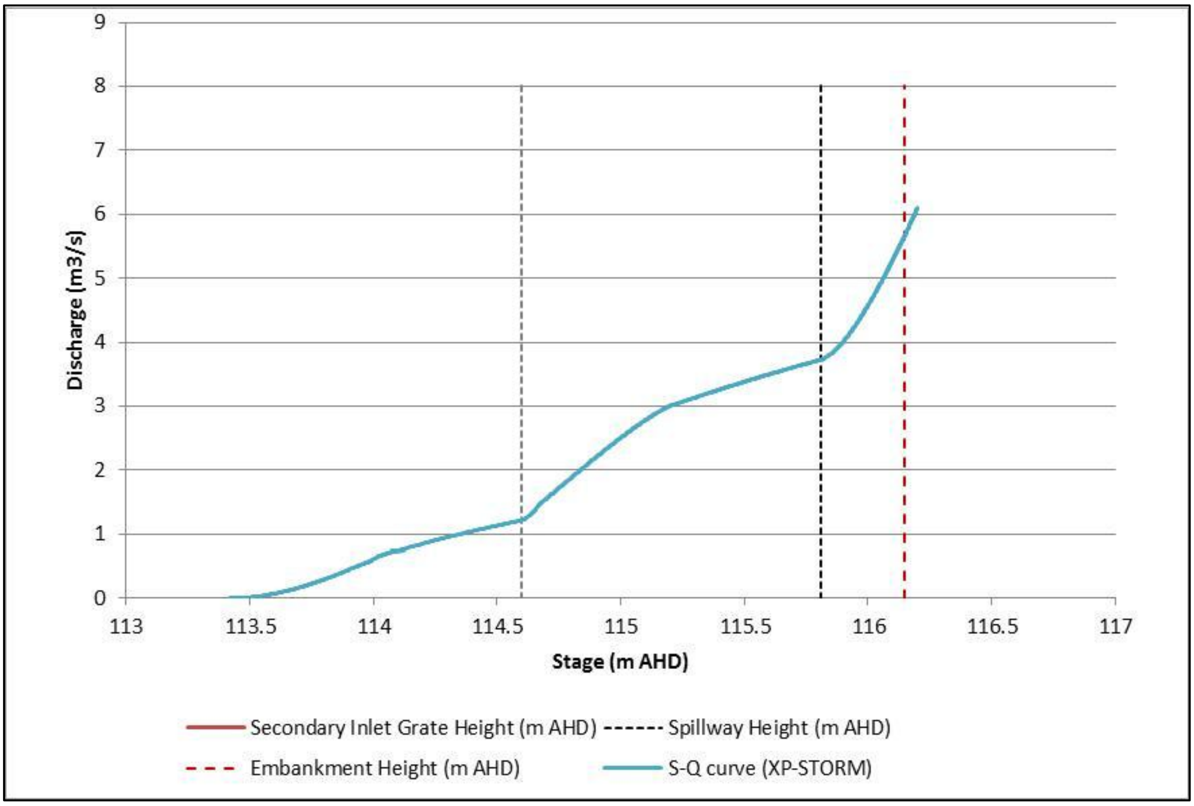

Figure 6-16 Brooker Park basin S-Q curve from XP-STORM analysis (Water Technology, 2012)

6.4.4 Basin Optimisation

The (2012) RORB model was initially run to generate revised baseline results for the full set of 100 year ARI storm durations. The results of the baseline model runs were compared to the SKM (2007) model results to assess the impact of using updated (LiDAR derived) basin volume data and changing from externally generated S-Q curves to weir and pipe based calculations. Table 6-12 shows the results of the (2012) RORB baseline modelling for the as-constructed basin.

Table 6-12 Brooker Park baseline RORB model run

Brooker Park Basin – Baseline Run			
100yr ARI duration	Max. water elevation (m AHD)	Freeboard to basin embankment (m)	Peak discharge (m^3/s)
15min	114.68	1.47	1.48
20min	114.72	1.43	1.61
25min	114.75	1.4	1.72
30min	114.82	1.33	1.94
45min	114.95	1.2	2.36
1hr	115.04	1.11	2.62
1.5hr	115.21	0.94	3.01
2hr	115.32	0.83	3.15
3hr	115.45	0.7	3.32

4.5hr	115.58	0.57	3.47
6hr	115.66	0.49	3.57
9hr	115.84	0.31	3.82
12hr	115.91	0.24	4.05
18hr	115.88	0.27	3.94
24hr	115.83	0.32	3.79
30hr	115.93	0.22	4.15
36hr	115.97	0.18	4.39
48hr	115.89	0.26	3.96
72hr	115.44	0.71	3.30

The SKM (2007) modelling returned a peak basin outflow of 5.8m^3/s for the 12, 36 and 48 hour duration storm events. Results of 2012 baseline modelling indicate that the 36hr storm event produces a peak flow of 4.39m^3/s with a maximum water elevation of 115.97m AHD (0.18m freeboard to the embankment height).

Given the reduction in effective basin volume (refer Table 6-11) and the revised S-Q curve used in the 2012 analysis, the difference between the 2007 and 2012 model results are not considered to be anomalous. The 2012 baseline modelling would therefore indicate an appropriately configured basin. Water Technology project staff reviewed potential improvements that could be made to the Brooker Park configuration. The main constraints on carrying out additional engineering works were considered to include:

- The urbanised context of the basin;
- The existing orifice inlet restriction and the grates on the secondary inlet already providing substantial reductions in the intake capacity of the outlet structure; and
- The overall existing level of functionality of the Brooker Park basin.

Given these factors, it was determined that basin volume augmentation (increases) may be considered expensive and impractical. Additionally, it was viewed that further restricting flows entering the system (via instalment of smaller orifice plates or similar) would present an unwarranted blockage risk to the basin given the existing constricted (grated) inlets. As the existing basin is considered to be highly functional, it was therefore concluded that altering the Brooker Park basin is unnecessary. No changes are therefore recommended in this report.

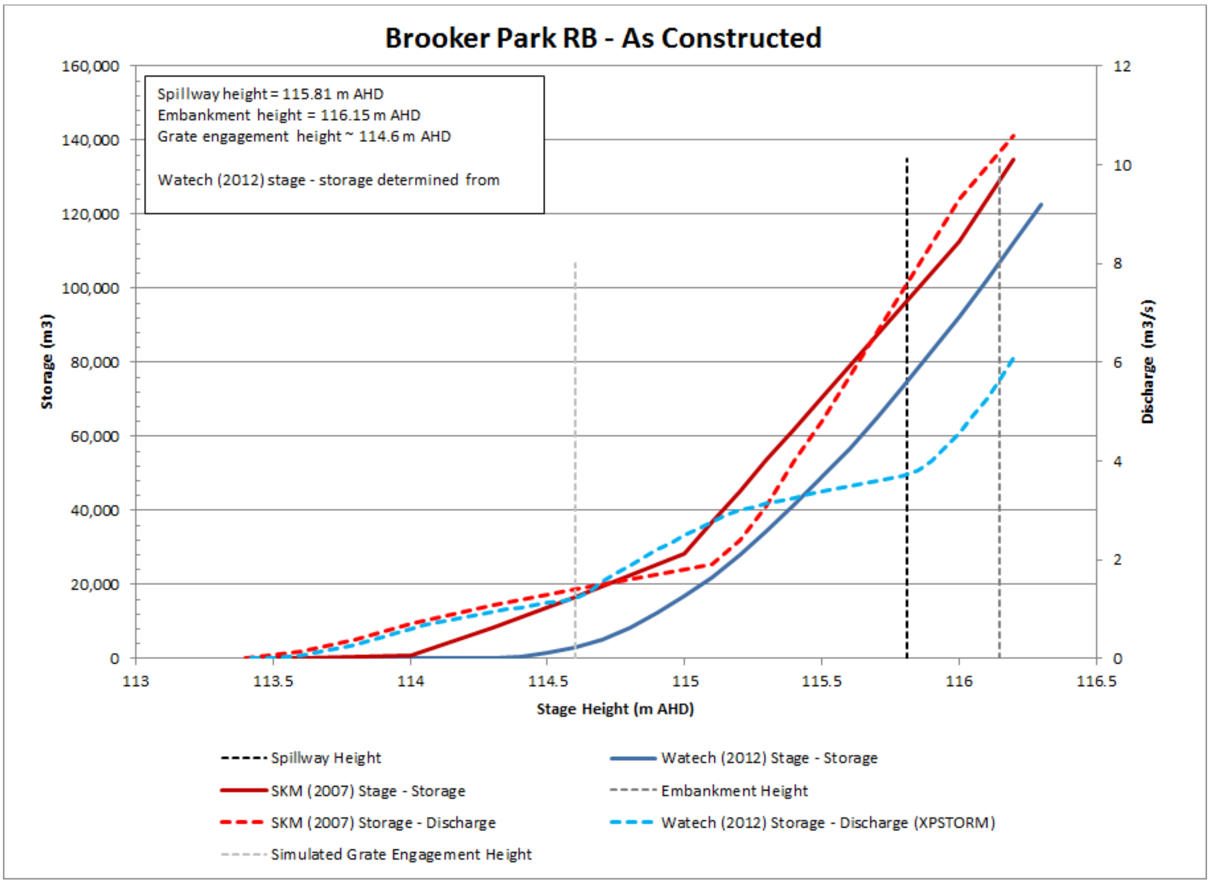

Figure 6-17 As-constructed basin performance curves – Brooker Park basin

Figure 6-17 provides a graphical summary of the as-constructed basin performance curves for the Brooker Park basin, including a comparison of the 2007 and 2012 S-Q curves. Key points of note include:

- The reduced LiDAR derived storage volume profile (solid blue line) for 2012;
- Confirmation of the 2012 stage–discharge curve exhibiting expected behaviour at the secondary grate and spillway engagement heights (vertical dashed grey / black line); and
- Anomalous behaviour of the SKM (2007) stage–discharge curve whereby the increases in discharges do not coincide with the secondary grate spillway/embankment heights.

6.5 Recommendations

The Water Technology review of basin performance for the Landsborough Road, Tarwin Street and Brooker Park basins recommends the following changes/improvements:

6.5.1 Landsborough Road RB

The proposed (optimised) basin configuration for the Landsborough Road basin consists of:

- Raising embankment height to 114.1m AHD (0.9m increase);
- Raising spillway height to 113.4m AHD (0.68m increase);
- Installing a 750mm orifice plate onto the US end of one of the existing 900mm culvert pipes; and
- Constructing an internal coffer wall (110.8m AHD crest) around the upstream invert location of the existing culvert pipes, with a single circular 525mm culvert at the base of the coffer dam wall.

6.5.2 Tarwin Street RB

The proposed (optimised) basin configuration for the Street basin consists of:

- Raising embankment height to 121.2m AHD (1.38m increase rather than the 0.5m increase as recommended by SKM);
- Raising spillway height to 120.0m AHD (0.8m increase) and installing 3 x 2750mm W x 600m H box culverts to form the spillway under the future road;
- Installing an additional 750mm orifice plate onto one of the existing 900mm orifice plates on the US culvert assembly; and
- Constructing an internal coffer wall (115.75m AHD crest) around the upstream invert location of the existing box culverts, with a single 750mm circular culvert at the base of the coffer dam wall.

6.5.3 Brooker Park RB

Water Technology project staff reviewed the potential improvements that could be made to the Brooker Park configuration. The main constraints on carrying out additional engineering works were considered to include:

- The urbanised context of the basin;
- The existing orifice inlet restriction and the grates on the secondary inlet already providing substantial reductions in the intake capacity of the outlet structure as well as present risk of blockage; and
- The overall existing level of functionality of the Brooker Park basin to be adequate.

As the existing basin is considered to be highly functional, it was concluded that altering the Brooker Park basin is unnecessary. No changes have therefore been recommended in this report.

A cost-benefit analysis is recommended to determine which upgrade would be most suitable for construction first.

7. HYDRAULIC MODELLING – WARRAGUL URBAN CATCHMENT

7.1 Overview

A key component of the Warragul Flood study project was the task of "Flood Modelling to improve overland flow paths in the Warragul Township". The RoG methodology approach was presented to Baw Baw Shire Council during the project inception meeting and approved as the most appropriate approach to establishing existing flow paths within the Warragul Township, giving the best opportunity to create improvements to overland flow paths via appropriate mitigation strategies.

Once built and verified the RoG model was used to create flood maps within the study area for the 1%, 2%, 5%, 10%, 20% and 50% AEP flood events and as a tool to determine possible solutions and estimated costs to address identified inundation issues. These results are presented in the following Section.

7.2 Technical Methodology

The assessment of flooding impacts within the study were analysed with the aid of hydrologic analysis and hydraulic models using the Direct Rain on Grid methodology. Using AusIFD and in-house Microsoft Excel tools a basic hydrologic model was constructed and provided rainfall hyetographs across all catchment areas for a suite of design ARI events. The hydraulic model then applied the rainfall directly to the catchment and routed the flow into and along the underground drainage infrastructure as well as overland across the 2D Domain.

This modelling methodology was verified by BBSC and adheres to the methodology outlined in the Project Brief supplied by BBSC along with the Melbourne Water, Flood Mapping Guidelines and Technical Specifications (Melbourne Water 2010b)

7.2.1 Hydrologic Analysis

The Direct Rain on Grid Method utilises the capability of the hydraulic modelling software to incorporate rainfall directly into the hydraulic model. This means only a basic hydrologic model is required which produces hyetographs for the range of desired events. These hyetographs are then applied directly onto the 2D domain in the hydraulic model. Initial loss (IL), Fraction Impervious (FI) and Runoff Coefficients (RoC) values are applied inside the hydraulic model.

7.2.2 Hydraulic Modelling – TUFLOW Rainfall on Grid

TUFLOW is a widely used hydraulic modelling software program that is suitable for the analysis of overland flows in urban areas. When used as part of the direct rainfall on grid method, TUFLOW has five main inputs:

- Topography data;
- Rainfall data;
- Catchment losses;
- Site roughness; and,
- Boundary conditions.

The TUFLOW model was used to apply rainfall and then route flows through the catchment both overland in a 2D domain and underground through the 1D pipe network. Where the capacity of the underground drainage network is exceeded, flows surcharge back to the surface and are routed overland in the 2D domain again across a topographic surface to create a series flood extents along with maximum depth and velocity values.

Further details of the TUFLOW modelling are provided in Section 7.4.

Rainfall on grid modelling has a number of distinct advantages over the traditional hydrograph (Rational Method or RORB) approach, including:

- A rainfall runoff hydrologic model such as RORB is not required, nor is a detailed analysis of sub-catchments.
- Flows are applied to the model at all points – removing the reliance on empirical relationships.
- Non-formal catchment storage areas are more accurately defined when compared to traditional approaches.
- Flood mapping covers the entire catchment, whereas the traditional approach starts mapping at a point where a flow hydrograph can be generated. This ensures results in areas around the catchment boundaries having the same detailed mapping as the remainder of the catchment, eliminating 'holes' in the mapping.
- All routing is completed in the hydraulic model. The routing methodology in TUFLOW is far superior to the methods employed by RORB and Rational Method. Resulting in flows arriving to locations based on the true topography with a better understanding of overland travel times. In traditional approaches, this is often controlled by the modeller and can be a subjective process.
- Two sets of mapping are produced: The first set is the raw model results which will show mapping across the entire catchment from main flow paths through to localised puddles/ponding in backyards providing an understanding of realistic ponding areas throughout the catchment. The second set of maps produced are the thinned maps which show the main flow paths and look very similar to maps produced via the traditional approach. Figure 7-1 below shows an example of the pre-thinned and thinned model results for a recent rainfall on grid project completed by Water Technology. A flow chart of the typical rainfall on grid methodology is shown in Figure 7-2.

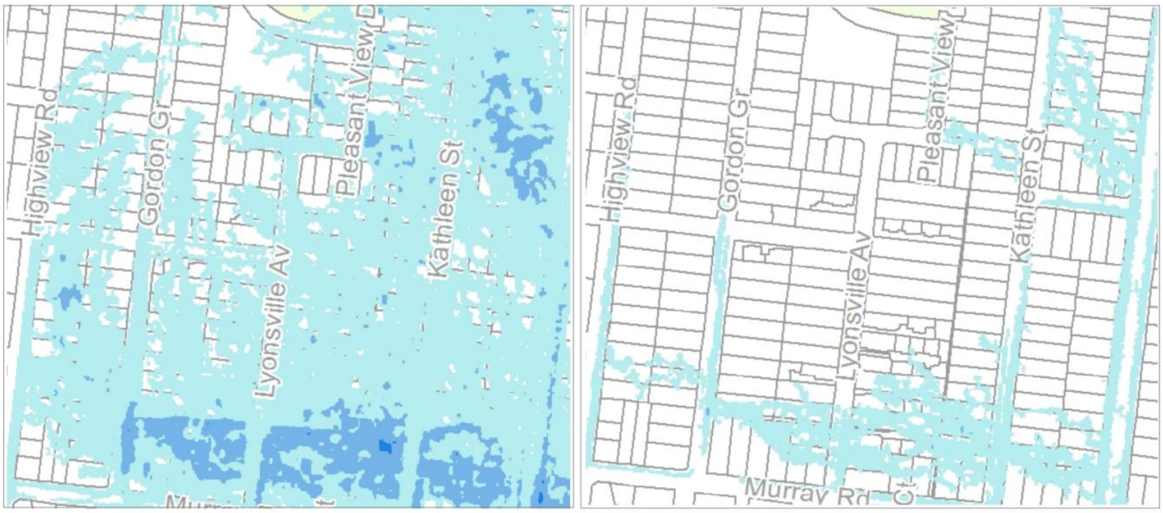

Figure 7-1 Example Rain on Grid Output Maps – Un-thinned (left) and Thinned (right)

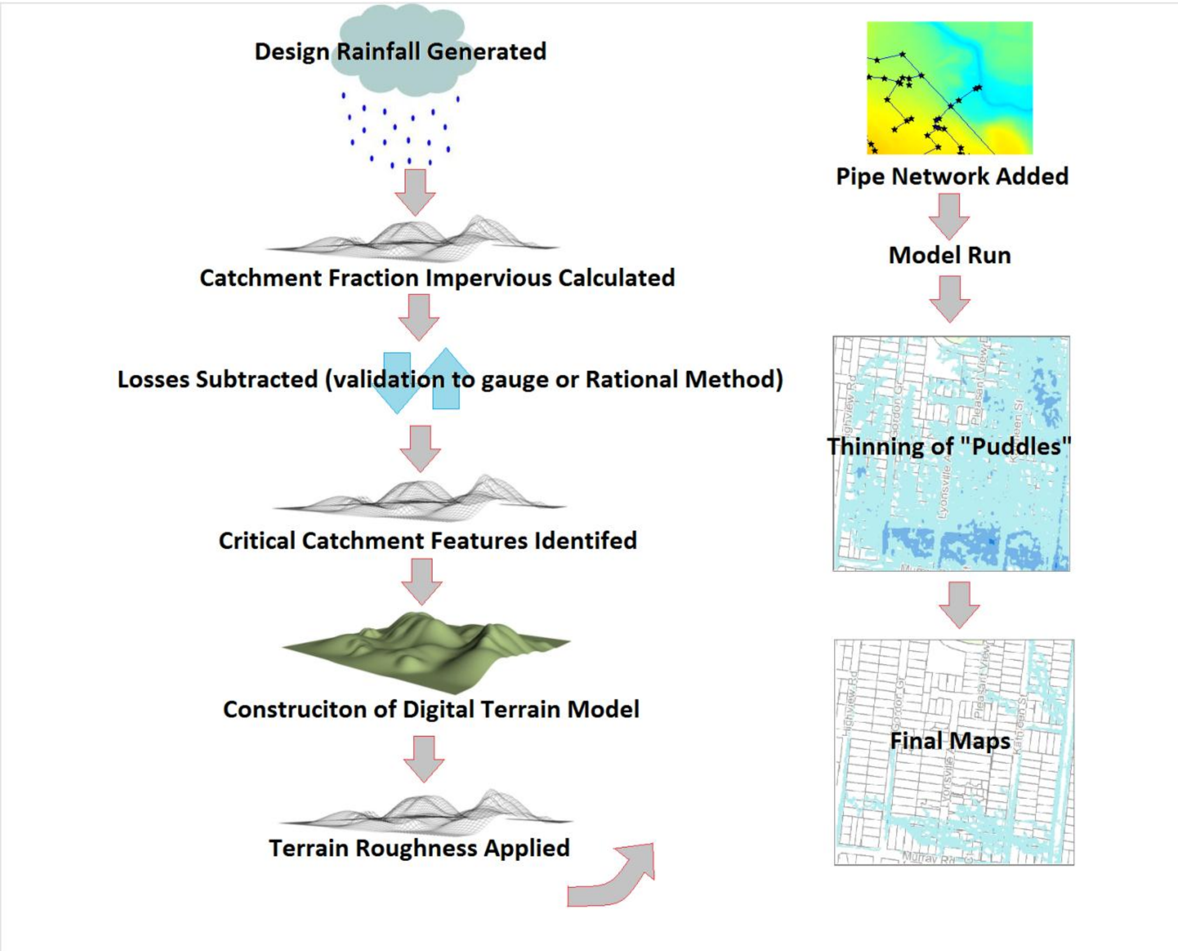

Figure 7-2 Rainfall on Grid (Direct Rainfall) Methodology

7.3 Hydrological Modelling

7.3.1 Overview

The basic hydrologic model provided design rainfall event hyetographs for input to the hydraulic modelling as part of the Direct Rainfall on Grid method. AusIFD Software and Excel Spread sheets were employed as the principal tools for the hydrologic modelling. Table 7-1 shows the modelled catchment conditions and the ARI events examined for this project.

Table 7-1 Modelled catchment conditions

Catchment Conditions	Modelling Scenario	ARI Event					
		2	5	10	20	50	100
Existing FI	Base Case	✓	✓	✓	✓	✓	✓

7.3.2 IFD Parameters

Intensity Frequency Duration (IFD) Parameters were determined at the centroid of the Warragul catchment using the Bureau of Meteorology IFD Program. The adopted parameters are shown below.

Table 7-2 IFD Parameters

IFD Parameter	$2I_1$ (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
Warragul	18.15	4.03	1.13	34.63	7.86	2.31	0.37	4.25	15.06

AusIFD Software and in-house Microsoft Excel tools were then used to produce hyetographs for each required ARI Event and duration for the catchment. These were converted to an appropriate format for input into the TUFLOW hydraulic model.

7.3.3 Fraction Impervious

Fraction Impervious (FI) values across all catchments were determined according to BBSC planning zones as per Melbourne Water's MUSIC Guidelines (2010a) and are shown in Table 7-3 . These values were then used to calculate runoff coefficients as detailed in Section 7.4.2.6.

Table 7-3 Planning Zone and Adopted FI Values)

Zone	Zone Code	FI
Residential Zones	R1Z	0.45
	R1Z (Allotment size 500m2 – 800m2)	0.45-0.55
	R1Z (Allotment size 350m2 – 500m2)	0.55-0.65
	R1Z (Allotment size <350m2)	0.65-0.70
	R3Z	0.60
Mixed Use Zone	MUZ	0.60
Industrial Zones	IN1Z	0.90
Business Zones	B1Z	0.90
	B2Z	0.90
	B3Z	0.90
	B4Z	0.90
Public Land Zones	PUZ1	0.05
	PUZ2	0.70
	PUZ3	0.70
	PUZ7	0.60
	PPRZ	0.10
	PCRZ	0.00
	RDZ1	0.70
	RDZ2	0.60
Special Purpose Zones	SUZn	0.60
	CDZn	0.50
	UFZ	0.00

FI values were determined using the above method and verified by review of aerial imagery and site visits of the catchment area. Values were judicially adjusted if they were deemed inappropriate based on the imagery.

7.3.4 Losses

Initial Loss was applied directly to the rainfall hyetograph and Runoff Coefficients were applied within the hydraulic model. An Initial Loss of 10mm was used across all of the study area and across all ARI events modelled. After verification with the Rational Method, a Runoff Coefficient was applied for the 2, 5, 10, 20, 50 and 100 year ARI events. The values used are shown below in Table 5-4.

Table 7-4 Loss Coefficients

ARI Event (years)	Initial Losses (mm)	Runoff Coefficient
2	10	0.20
5	10	0.25
10	10	0.35
20	10	0.45
50	10	0.55
100	10	0.60

7.4 Hydraulic Modelling

7.4.1 Overview

A TUFLOW model was developed for this study. Input parameters were taken from the best available data including recent LiDAR, planning information and council drainage assets and matched with appropriate industry standards.

7.4.2 Hydraulic model construction and parameters

- The TUFLOW model was constructed using MapInfo V11.0 and text editing software. This section details key elements and parameters of the TUFLOW model that adhere to the Melbourne Water 2D Modelling Guidelines (Melbourne Water 2011).
- The double precision version of the latest TUFLOW release (as of May 2012) was used for all simulations (TUFLOW Version: 2012-05-AA-iDP).

7.4.2.1 *2D Grid Size and Topography*

- The 2D domain grid size was set to 3 metres; based on the total catchment size and to ensure catchment characteristics including natural surface, waterways and roads were defined. The 2d_zpt file was populated with elevations from the 1m DEM grid provided by WGCMA.
- Erroneous areas within the DEM were identified and smoothed with the surrounding terrain through the use of z-shapes in TUFLOW.

7.4.2.2 *1d Network*

- All pipes, culverts, spillways and other structures were modelled in a 1D network using the council plans and drawings provided by BBSC. These plans were converted to electronic MapInfo tables for their use in hydraulic modelling.
- Pipe and pit specifications were obtained from the council plans provided and inverts were also added to the MapInfo tables. Where inverts were missing, an assumed cover depth was applied as shown below in Table 7-5

Table 7-5 Assumed Depth of Cover where no further information was available

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

*If this cover could not be achieved it was discussed with and approved by BBSC

7.4.2.3 *Roughness*

- For the 2D domain, “2d_mat” files were produced based on land use zones to represent catchment roughness (Manning’s roughness) characteristics, with further refinement through the use of aerial photographs and site visits. The Manning’s values are specified in the .tmf TUFLOW model file.
- Throughout the central business district of the township as well as within key / major flow paths, building footprints were digitised and applied to the Manning’s roughness layer for residential, commercial and industrial buildings to provide a better understanding of overland flow paths through these areas.
- Where building footprints were not available the entire land parcel was specified a separate roughness value.
- For the 1D domain, Manning’s values are defined in the “1d_nwk: file.
- Manning’s ‘n’ roughness coefficients are listed in Table 7-6 below.

Table 7-6 Manning's n Roughness Coefficients

Land Use	Manning's n Roughness Coefficient
Roads	0.018
Car Parks (Paved and Gravel)	0.030
Parkland	0.080
Industrial / Commercial	0.350
Grassy Park	0.035
Residential Property (No Building Footprint)	0.350
Railway Line	0.125
Residential Property (With Building Footprint)	0.100
Building Footprint	0.400

7.4.2.4 *Pit Configuration*

- Pits along the 1D pipe section were connected to the 2D using the "SXL" option for the 1d_nwk pit Conn_2D attribute. This option automatically lowered the 2D cells connected to the pits by 0.1m, ensuring overland flow is able to adequately enter the pipe network.
- Some pits were inspected during the site visited to verify the information provided through BBSC plans and input to the TUFLOW model accordingly. Where house connections needed to be accounted for, weir type pits were used in TUFLOW to allow water to enter and exit the pipe network as required.

7.4.2.5 *Boundary Conditions*

The 1D components conveyed flows out of the catchment as defined in the 1d_nwk. Where overland flow out of the catchment was present, 'HQ' and 'HT' boundaries were used to convey the overland flow out the catchment in a steady manner. Where required, 10year tail water levels within Hazel and Spring Creek were determined through discussion with the BBSC and previous flood modelling within the Warragul area (Earth Tech 2004).The Use of 10yr ARI tailwater levels for design events of =<100yr ARI is consistent with current Melbourne Water recommendations.

Specific catchment boundary information and initial water levels can be found in Table 7-7 and Figure 7-3.

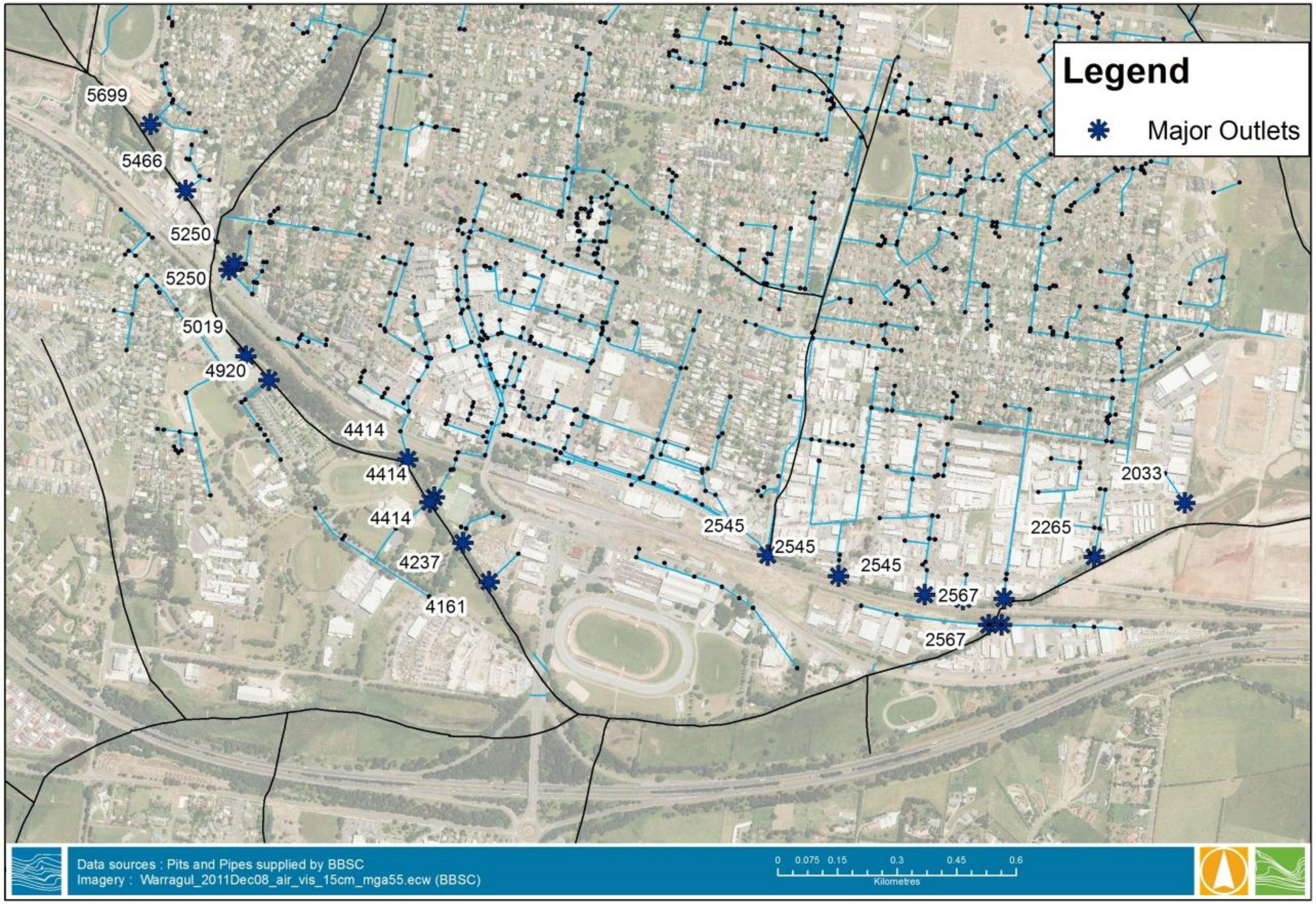

Figure 7-3 Location of major outlets and chainage used in TUFLOW modelling

Table 7-7 1d Boundary conditions applied to major outlets in the Warragul Urban Flood Modelling

Chainage	Water Level (mAHD)*
2033	102.25
2265	102.88
2545	103.85
2567	103.86
4161	106.72
4237	107.02
4414	107.46
4920	109.33
5019	109.64
5250	110.66
5466	111.54
5699	111.80

* 10yr ARI WSE extracted from results of the 2004 Hazel & Spring Creek Flood Study by Earth Tech

7.4.2.6 *Rainfall and Total Runoff Coefficients*

A "2d_rf" rainfall file was produced in MapInfo for all ARI events which consisted of rainfall polygons. Each polygon contains a field listing the desired hyetograph and a final runoff coefficient calculated from the FI value for that area and the runoff coefficients detailed previously. The FI value was determined from the Planning Scheme zones as detailed previously. The final runoff coefficients for each rainfall polygon were calculated using the following equation:

$$ROC_{total} = (FI \times 0.9) + ((1 - FI) \times ROC_{x\ years\ ARI})$$

Where:

ROC_{total} = Total runoff coefficient for ARI of x years

FI = Fraction Impervious of rainfall polygon

$ROC_{x\ years\ ARI}$ = Runoff Coefficient for ARI of x years

- Hyetographs
  - Hyetograph .csv files were created for both rainfall ARI events and durations using AusIFD software and in-house Excel tools. They were then applied to the TUFLOW model as appropriate.

7.4.2.7 *TUFLOW model checks*

The following checks were undertaken on TUFLOW model parameters and outputs and are based on Melbourne Water recommendations:

- 2D grid size: A 2D grid size of 3 meters, within the recommended range of 2-3 meters for urban catchments.
- 2D timestep: The 2D timestep for the model was 0.75, equal to ¼ of the grid size and hence within the recommended range.
- 1D timestep: The 1D timestep was set to equal the 2D timestep and is hence within the recommended range.
- Model mass errors: The cumulative mass error for all models does not exceed 1% and in most cases is maintained at less than 0.1% and is hence within the recommended range.
- Errors and warning messages: None
- Pipe flow: A majority of pipes flow full in all models, as was expected.
- 2D Model extent: All produced flood extents are not impacted by the edge of the TUFLOW model's active area.

7.4.3 Rational Method Checks

The Rational Method was used to provide an estimate of 100 year ARI flows at key points within the catchment. The Rational Method flow estimate was compared to the combined pipe and overland flows from the TUFLOW model in order to validate the appropriateness of loss and runoff coefficient values used. Checks were made at key locations in the catchment to ensure the modelled flow values fell within a 10% range of the Rational Method estimates. These results suggest the runoff coefficients and loss parameters employed were appropriate for use.

Reconciliation of the TUFLOW model flow results to an estimate from the Rational Method is important to ensure that catchment losses and conveyance parameters are appropriately accounted for in the TUFLOW model. As the TUFLOW model is able to take many more catchment characteristics into account, it is not expected that the results from the TUFLOW model should exactly match the estimate from the Rational Method. Instead, the Rational Method is used as a check to ensure that the flows seen in the TUFLOW model are of the order expected. As the TUFLOW models are quite sensitive to the runoff coefficients and initial loss parameters applied, the Rational Method estimate also serves as a good check that appropriate values are being used.

Three reference areas were identified within the study area that were representative of the catchment and suitable for Rational Method calculation and TUFLOW model reconciliation. The selected areas had well defined catchment boundaries and a single and distinctive outlet where comparison between the estimated flow rate from the Rational Method calculation could be compared to the TUFLOW model output.

Figure 7-4, Figure 7-5 and Figure 7-6 show the catchment areas where Rational Method estimates of peak flow were taken.

Figure 7-4 Location of Rational Method estimate Area 1

Figure 7-5 Location of Rational Method estimate Area 2

Figure 7-6 Location of Rational Method estimate Area 3

Overland and pipe flows (where applicable) were extracted from the TUFLOW models at the same locations as the Rational Method estimates. The PO_line command in TUFLOW was utilised to record overland flow rates. Where the flow location comprised of overland flow and underground pipe flow, the TUFLOW flow rate was calculated as the sum of the 1D and 2D flows for the critical storm duration.

- A Rational Method calculation was completed for each catchment to estimate the peak 100 year ARI flow at the point of interest. The Rational Method flow rate was calculated at the flow comparison location through the use of the Rational Method shown below:

$$Q_{100} = \frac{C.I_{100}.A}{360}$$

Where Q_{100} = 100 year ARI peak flow rate (m^3/s)

C= Runoff coefficient, based on FI values and ARI storm events.

A=Catchment area (ha)

I_{100} =Rainfall intensity of the storm for the time of concentration

The Fraction Imperious (FI) value of the selected area was determined using the same methodology as outlined earlier in Section 7.3.3.

- The Time of Concentration (tc) was calculated from the time taken for water to travel the longest path from within the catchment which is based on: (Overland flow, based on assumed velocities verified within the hydraulic model;
- Piped flow, based on pipe velocities from the hydraulic model with pipes flowing full; and,
- Initiation time, taken as 7 minutes.

7.4.3.1 *TUFLOW Model Reconciliation*

The Rational Method flow estimates were compared to the TUFLOW outputs for 100 year ARI storms. Successful reconciliation was judged to be no more than ±10% difference between the TUFLOW and Rational Method peak flows.

The flow calculations and comparison are shown in **Table 7-8** below.

Table 7-8 TUFLOW to Rational Method Comparisons

Location	Area (Ha)	Fraction Impervious (FI)	Rational Peak Flow (m^3/s)	TUFLOW Flow Line Peak Flow (m^3/s)	% Diff
Area 1	39	0.65	6.96	6.76	-2.96%
Area 2	12	0.70	2.73	2.36 (Overland) 0.25 (Piped) = 2.61	-4.40%
Area 3	92.5	0.45	7.49	6.96	-7.08%

The results shown in Table 7-8 indicate that the results extracted from the TUFLOW model reconcile well to the Rational Method Estimates completed. Water Technology considers the reconciliation to be successful based on the results above.

7.4.4 GIS Processing

The raw model output data was processed in order for it to be easily viewed in GIS. Processing occurred in two stages – firstly processing the raw data using TUFLOW utilities and then processing the resulting data within a GIS environment. These processes are detailed below.

7.4.4.1 *TUFLOW Data Processing*

TUFLOW contains a number of utilities for processing output data. The following utilities were used:

- Dat_to_dat.exe: This utility has a number of functions and in this instance was used to extract the maximum value for depth, velocity and water elevation at each grid point across the twelve durations for each event. The maximums values are then placed into a new data file.
- TUFLOW_to_GIS.exe: This utility converts TUFLOW data into GIS formats and in this instance was used to convert TUFLOW data into the MapInfo mid/mif interchange format.

7.4.4.2 *Results Processing*

MapInfo was used to import and then compile the data into an appropriate format. Initially the depth, velocity, water surface elevation and duration layers were amalgamated into a single layer for each event. Final maps were produced from ASCII plots in Arc-GIS v10.

7.4.4.3 *Data Integrity Checks*

The results were checked to ensure that larger events corresponded with increased depths, flood level and velocity in each cell.

Depth results must conform to the following: 100 year > 50 year >20 year>10 year>5 year>2 year

7.4.4.4 *Filtering of Results*

The model results were filtered according to the following criteria, in accordance with BBSC:

- Minimum Depth Threshold – any flooded cells with depths less than 0.02 m were removed; and
  Velocity * Depth Criteria – The results were filtered to remove any cells where both the depth is less than 0.10 m and the V*D is less than 0.008.

All cells considered as flooded after the application of the above filters (1 and 2) are then combined into a flood extent that connects neighbouring cells. Any flooded areas that are less than 100 m^2 are then removed.

7.4.5 Hydraulic model application

The Hydraulic model software TUFLOW was used to model the catchment using a direct rainfall on grid approach. The outlet of the TUFLOW model was extended around the entire catchment to allow water to freely drain out of the catchment in the 2D domain. This was achieved through the use of a ‘HQ’ boundary with a slope applied to match the surrounding terrain Figure 7-8 shows the TUFLOW model structure adopted for Warragul RoG Urban Study area with Figure 7-7 also showing the fraction impervious values for the model.

An initial review of the 5 year ARI modelling results was undertaken by BBSC. Identified flooding was then discussed internally at council (including with the Urban Operations team) to determine if the results bear resemblance to historical flooding observations.

Comments were supplied back to Water Technology from this process allowing for refinement of the model (specifically the pit and pipe network) and in certain cases augmented, before the revised results were again provided to the BBSC. The process was then repeated a second time before BBSC nominated they were comfortable with the 5 year ARI results. With the 5yr results approved, flood mapping for all ARI required was undertaken.

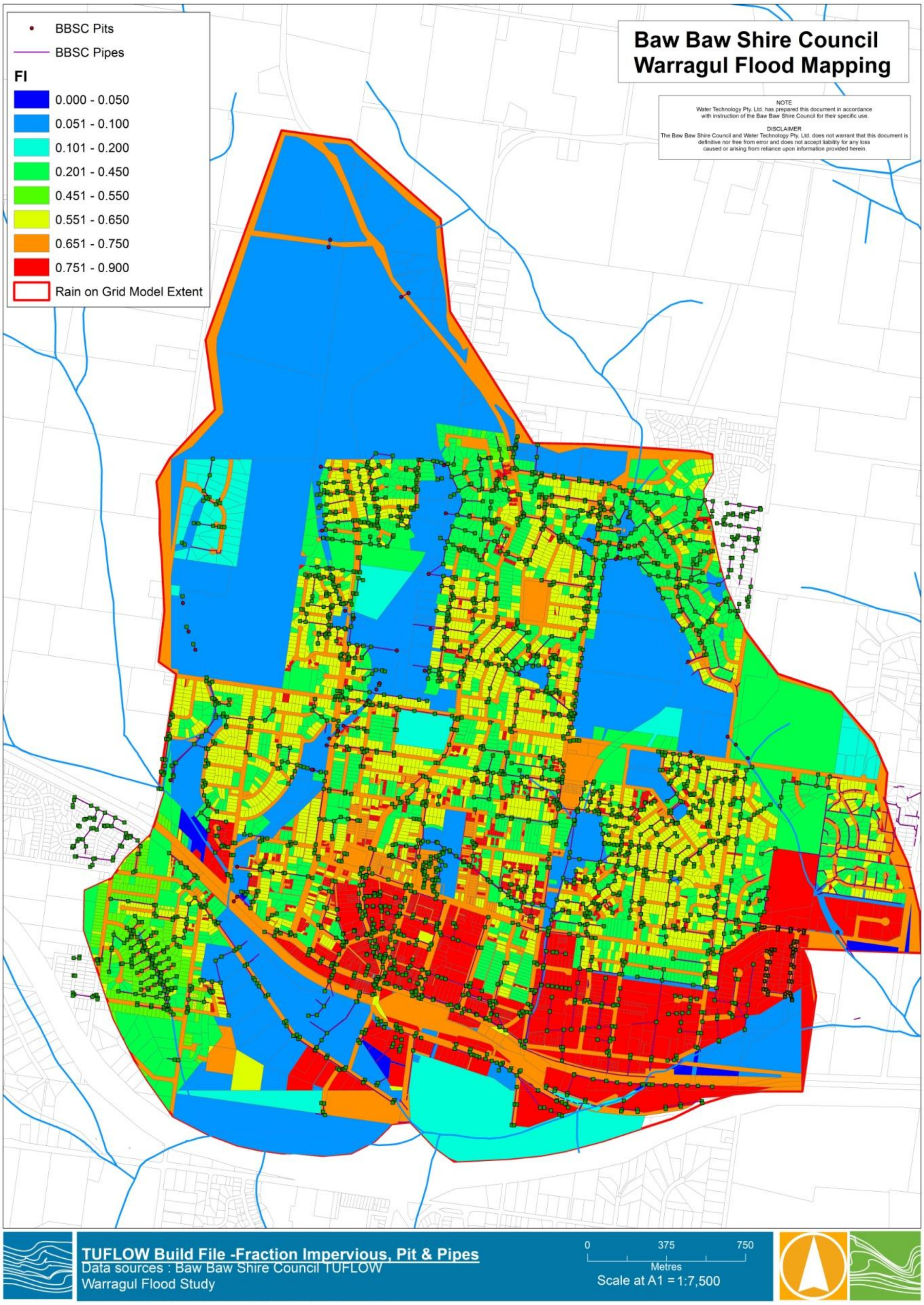

Figure 7-7 Warragul RoG Fraction Impervious Values*, pit and pipe network

* A range of FI values were used for a sensitivity analysis in a small catchment within the model area.

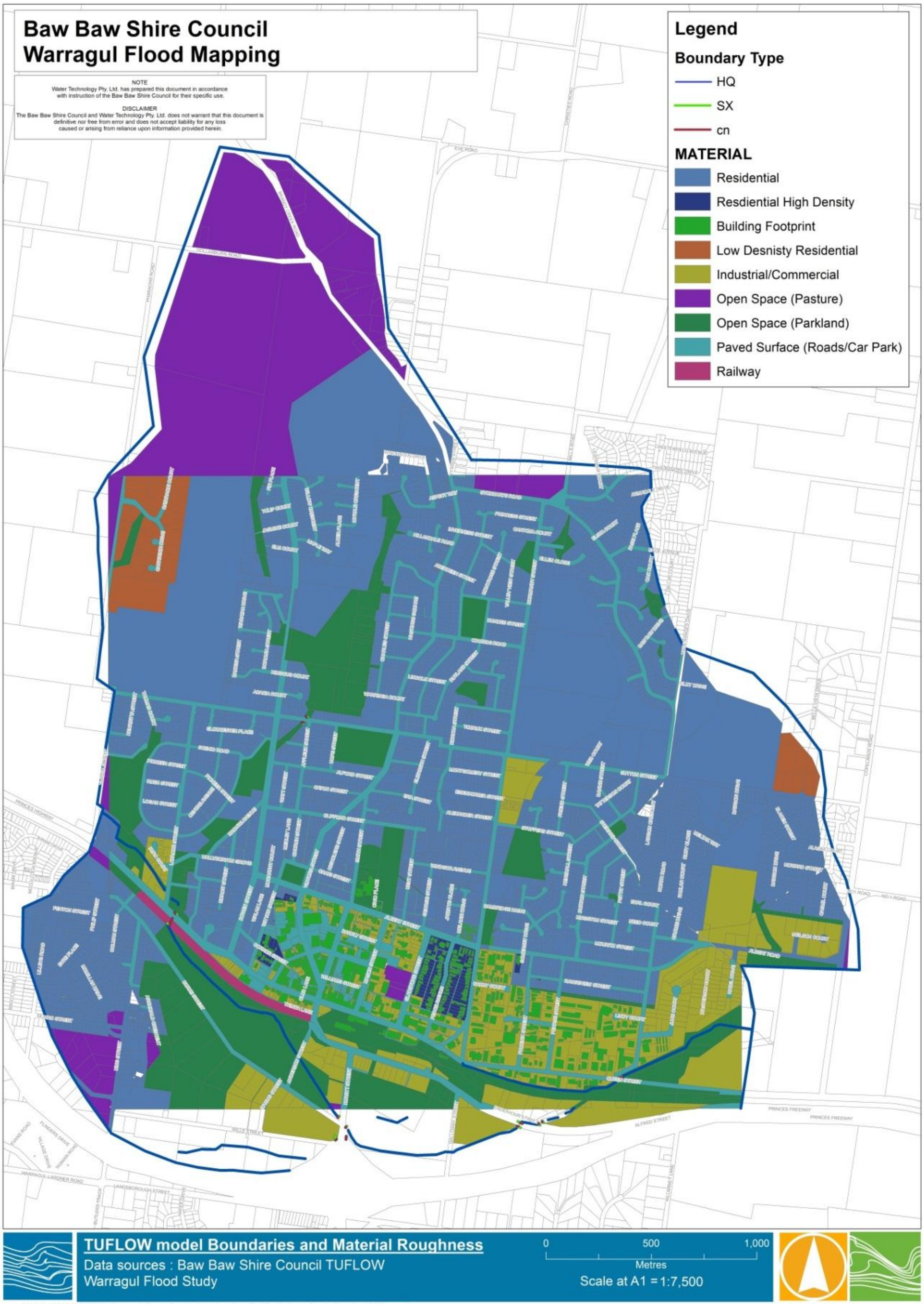

Figure 7-8 TUFLOW model boundaries and Manning's Roughness values

7.5 Sensitivity Analysis

A sensitivity analysis was undertaken in small localised areas where the fraction impervious was analysed using recent aerial photography rather than planning zones which may represent future development. This involved the modification of the fraction impervious values for several residential zoned properties originally given a value of 0.1 (to confer with the existing undeveloped land) updated to 0.45 to reflect the possible developed conditions as shown bordered in red in Figure 7-9 below. As shown in Figure 7-10, there was minimal difference (1-2cm) in the maximum flow depth through and downstream of the sub catchment in question. The difference shown at the southern end of the area modelled is likely the result of changed tailwater conditions as the whole catchment was not included in the sensitivity analysis modelling and is not relevant for this section.

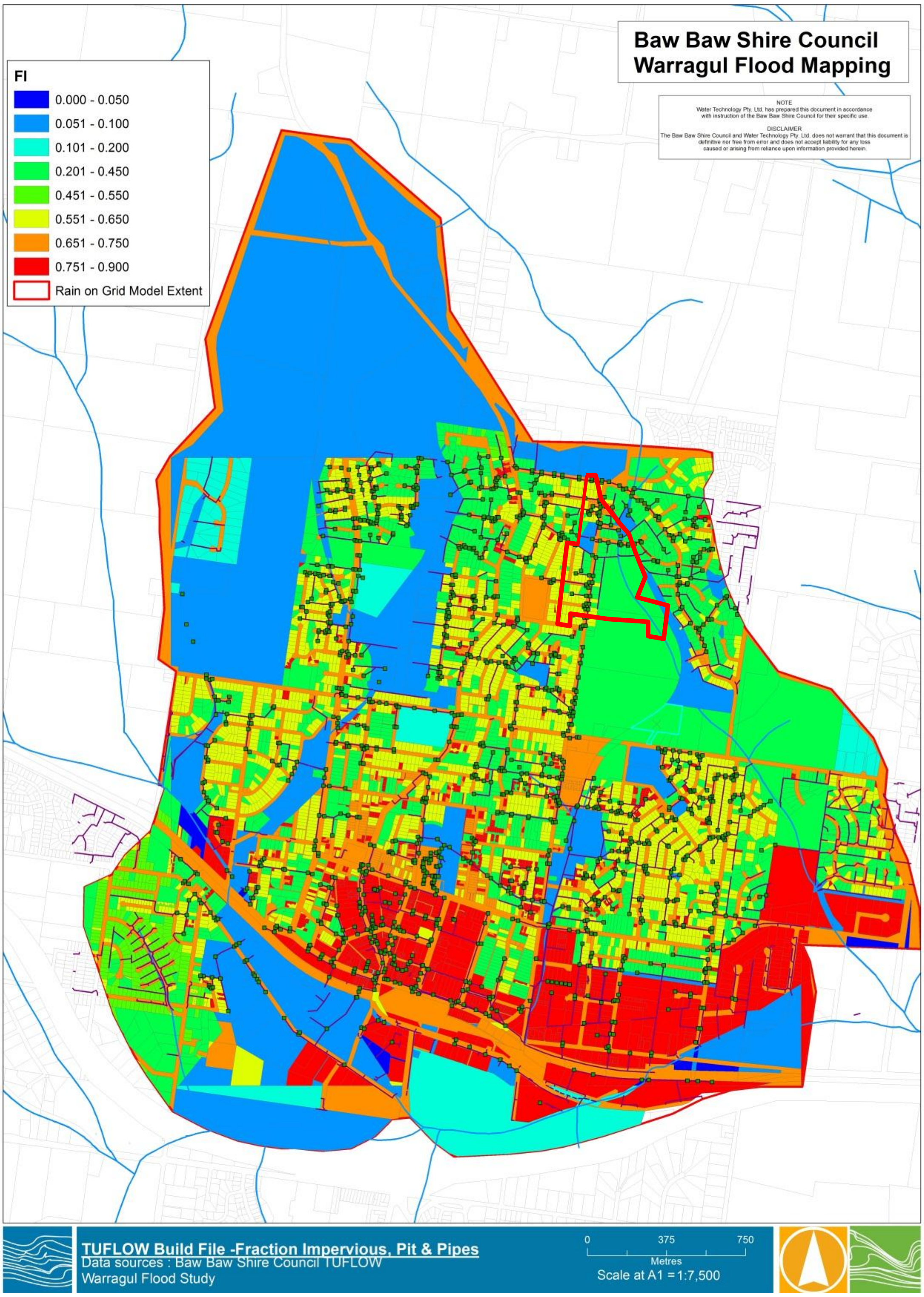

Figure 7-9 FI Values Used for Sensitivity Analysis

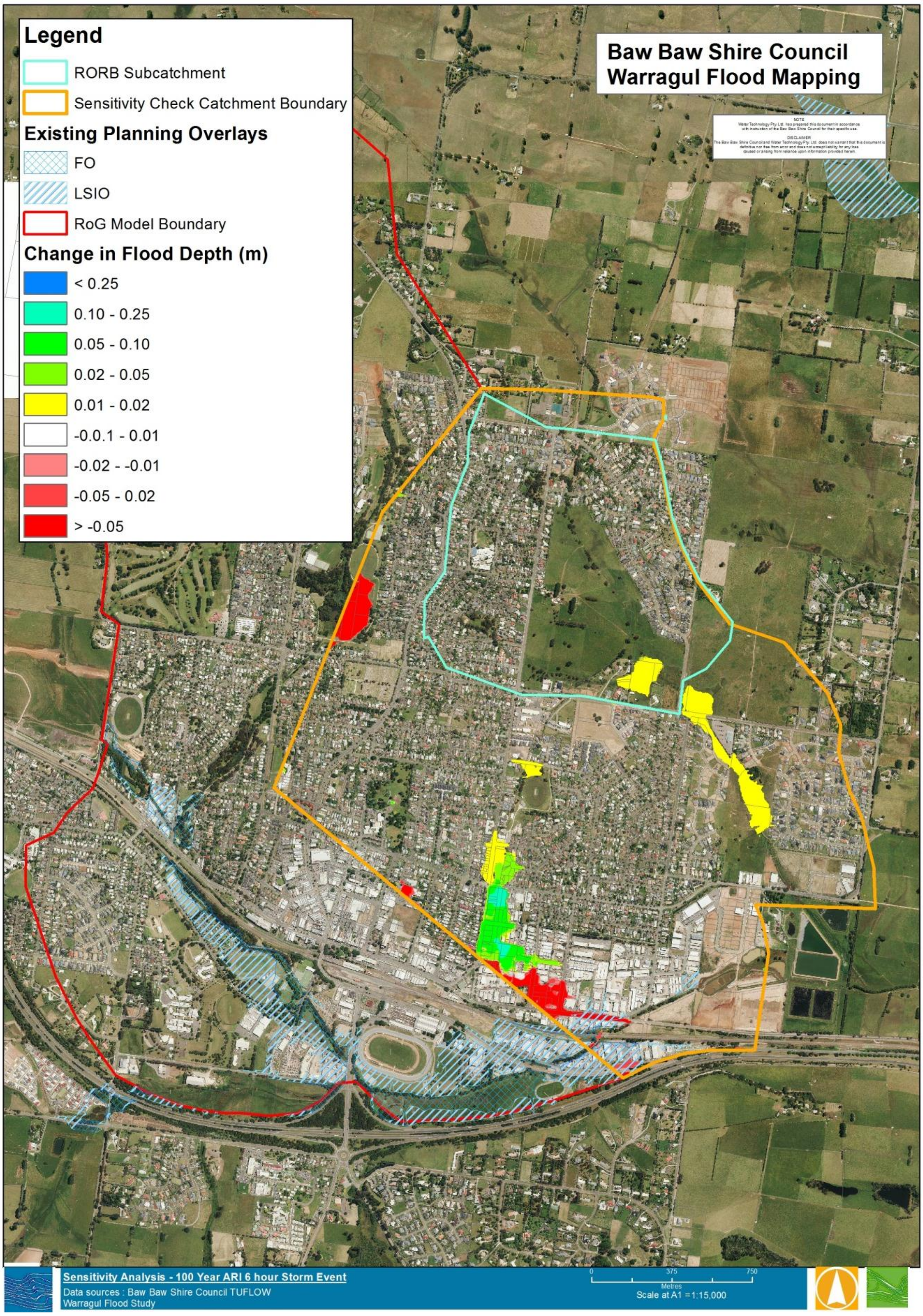

Figure 7-10 Sensitivity Analysis for 100yr 6 hour storm event (localised sub catchment only)

8. HYDRAULIC MODELLING RESULTS

8.1 Overview

The flood mapping deliverables consisted of a series of flood extents along with maximum depth, velocity and hazard plots for a range of ARI events. Figure 8-1 shows a comparison of flood extents for all ARI scenarios modelled with Figure 8-2 showing the flood hazard across the catchment in the 100 year ARI. Further flood depth and water surface elevation plots are shown in Appendix B for the scenarios and events listed in Table 7-1.

8.2 5 year ARI

The 5 year ARI event was used as verification tool by BBSC to confirm historical flood events throughout the catchment with preliminary results from the hydraulic model. Results were agreed upon with BBSC based on historical flooding, local knowledge of the area and the existing drainage infrastructure. While not a technical process, this task helped BBSC understand flood magnitude and dynamics resulting from a 20% AEP storm (10 minute to 6hr duration) in the Warragul urban area. The following flooding conditions were noted:

Minimal flooding of residential parcels occurs in the 5 year event. All major overland flow paths are engaged in the 5 year event. This suggests, as expected, that the pipe network does not have 5 year ARI capacity. In this case, the pipe network is generally full and surcharging onto the 2D domain;

- Key Locations where flooding is noted include:
  - Downstream of Brooker Park Basin (upstream and downstream of Sutton Street);
  - End of Helen Court;
  - Ryan Court;
  - Downstream of Civic Park;
  - Along Normanby Street between Albert Road and Queen Street
  - Bottom end of Phoenix and Pearse Streets.
  - Downstream of Churchill Street

8.3 100 year ARI

100 year ARI flooding results are used to make informed decisions regarding planning controls within local council. Results generated in this portion of the study cover the urbanised portion of the Warragul township but <u>do not</u> cover the Hazel Creek floodplain south of the main township area. This area is to be mapped during the *Warragul Waterways Flood Mapping Project* being completed by Water Technology (2012). All tailwater levels for this investigation were adopted from the previous flood modelling discussed earlier.

In an event as significant as a 1 in 100 year ARI, ponding and overland flow paths are activated, resulting in flooding impacts across much of the study area. Some significant flooded areas identified include (but not limited too):

- The piped designated waterway which moves water from Sutton Street to the outlet at the intersection of Queen and Normanby Street;
- Significant flooding in the industrial area between Albert Road and Queen Street;
- The flow path which moves water from the Warragul CBD to toward the outlet at the intersection of Queen and Normanby Street;
- Significant flooding at the corner of Gladstone Street and Vermont Avenue.
- Flooding along Queen Street between Normanby Street and North Road;(Most of the flooding in this area is found in Phoenix and Pearse Streets);

- Low point along Western Point Drive (near Pioneer Street);
- Downstream of Churchill Street;
- Down Stream of Waratah Drive (upstream of the Brooker Park Basin); and
- Upstream of Stoddarts Road and Ellen Close;

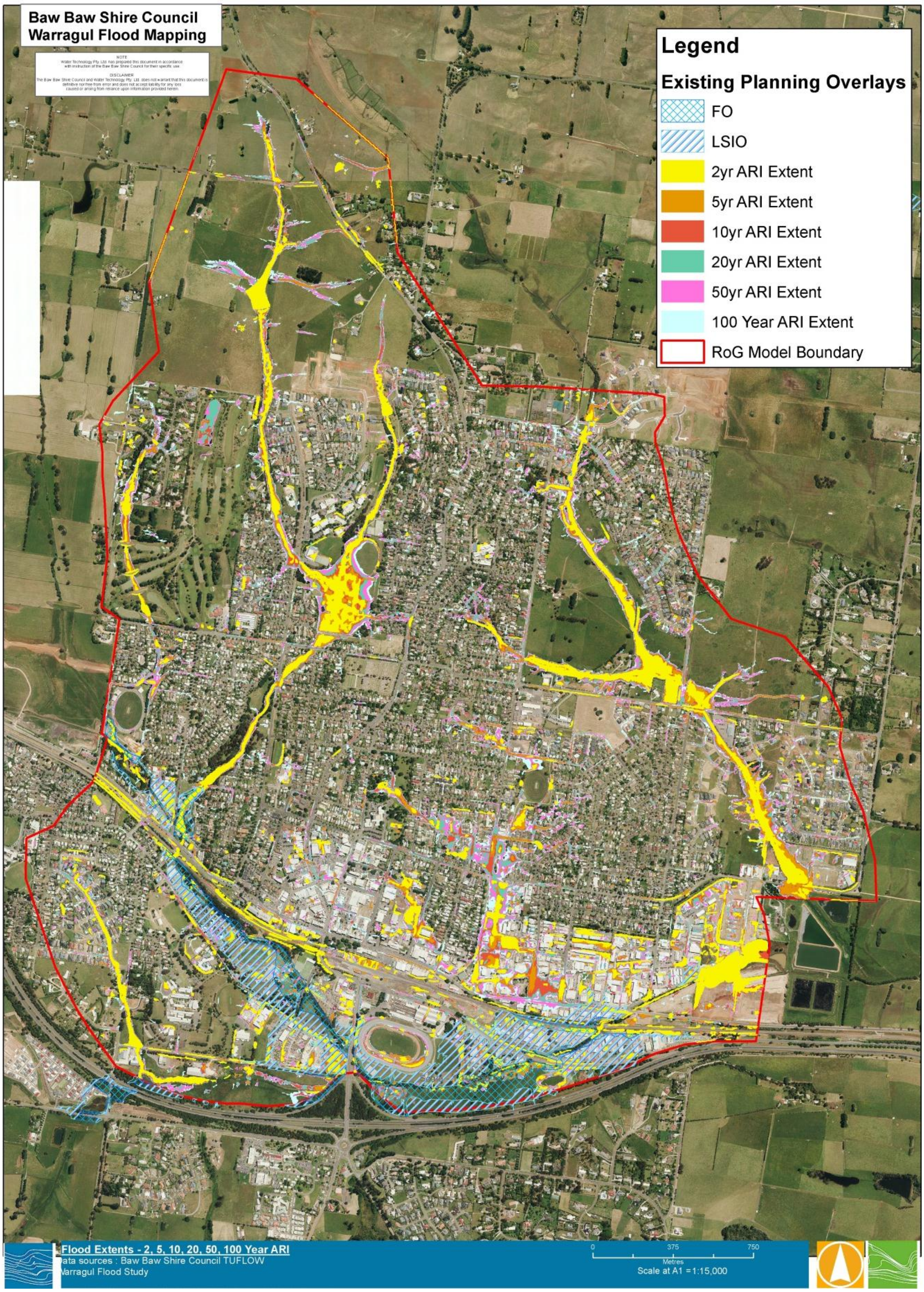

Figure 8-1 2, 5, 10, 20, 50 & 100 Year ARI Flood Extent

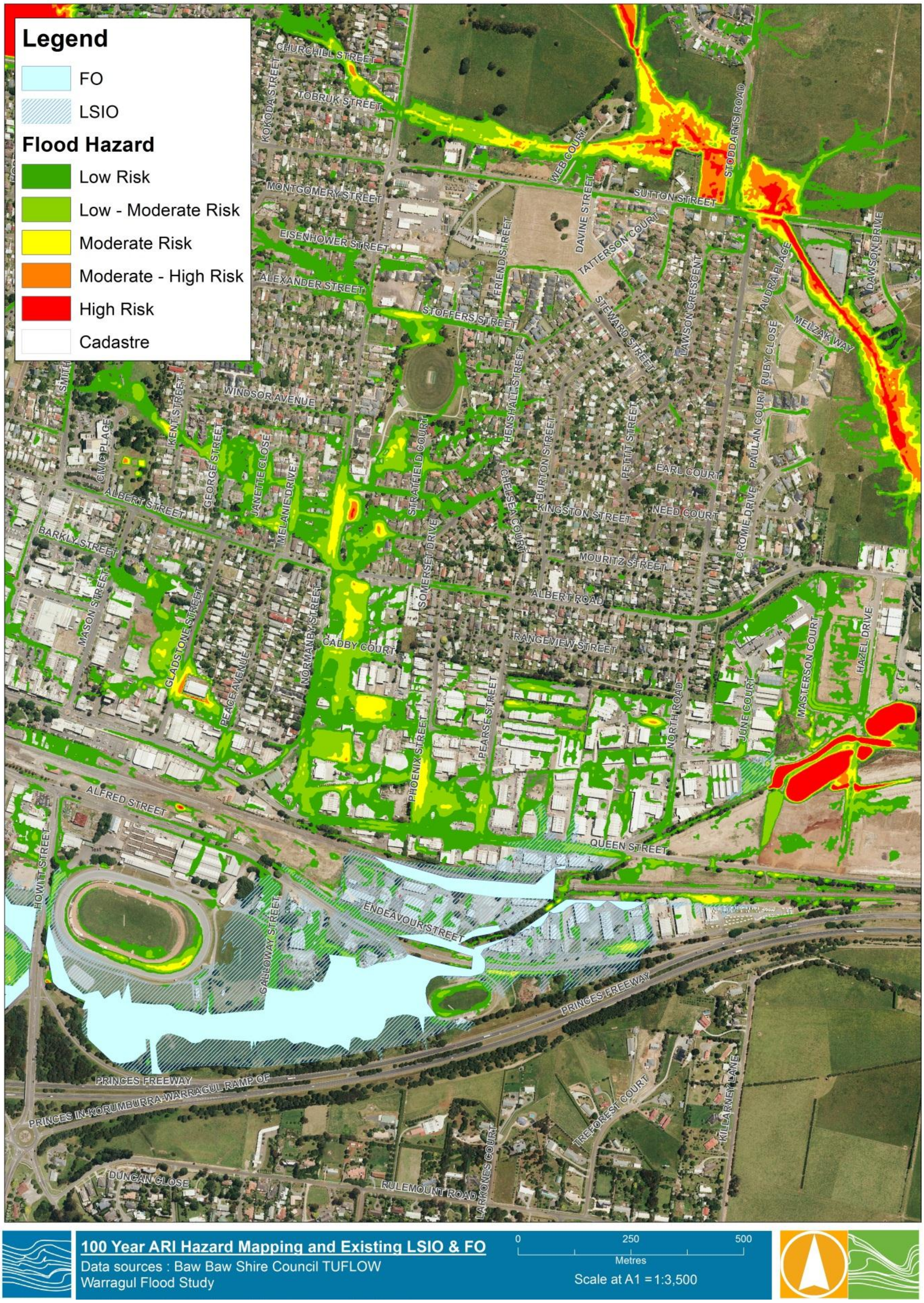

Figure 8-2 Urban study area 100 Year ARI Flood Hazard Risk

9. MITIGATION

9.1 Overview

A workshop presenting existing conditions RoG modelling results (and basin optimisation works) was held at the BBSC offices on the 26th of June 2012. Representatives from Council, the WGCMA and VicSES were in attendance. After the project team had briefed the attendees of the modelling undertaken to establish the existing conditions results, possible mitigation works to relieve identified and historically known impacts of flooding within the study region were discussed.

As outlined in the BBSC tender documentation key flooding “hotspots” were identified and were a focus of this project. Therefore discussions during the workshop identified that mitigation works should not focus solely on these specific locations but instead investigate the reduction of widespread flooding through the implementation of additional retarding basins at four locations within the RoG study area. These areas are shown in Figure 9-1.

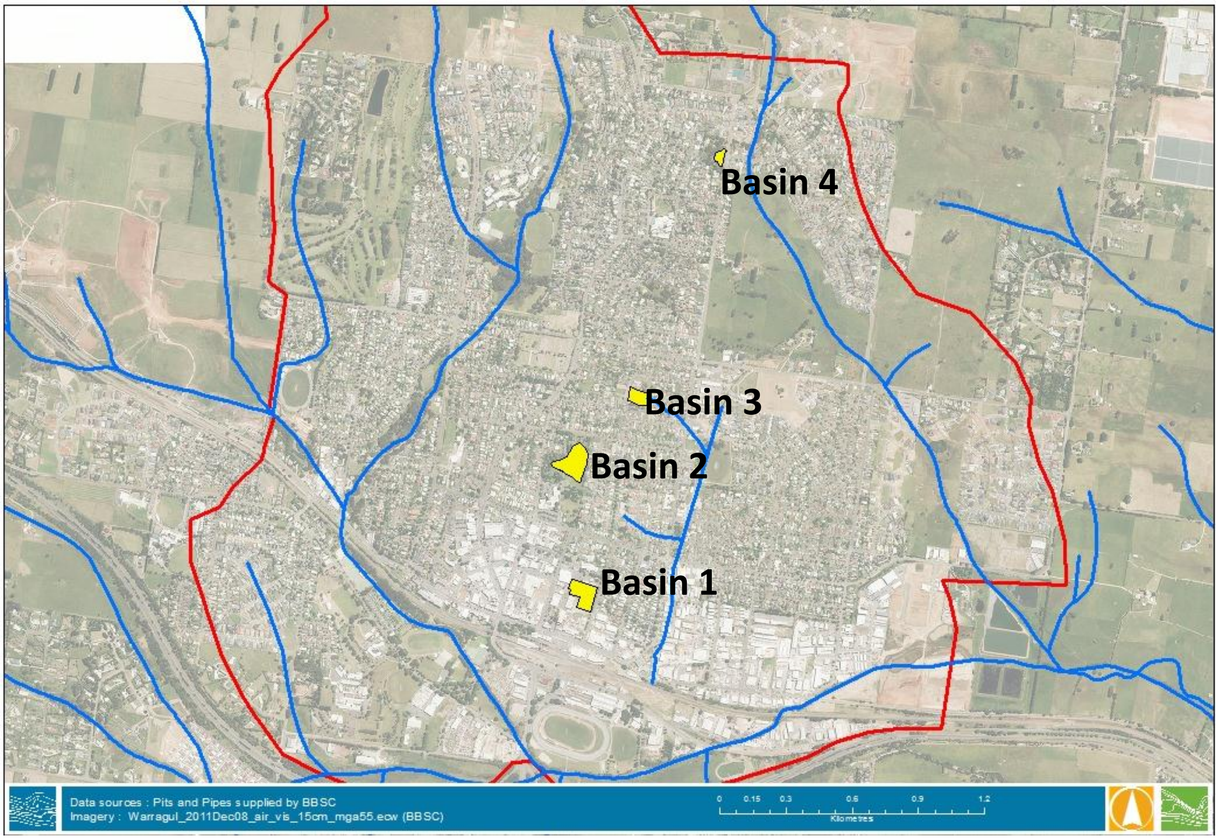

Figure 9-1 Location of modelled flood mitigation basins

With the four areas identified, LiDAR topography data was reviewed and manipulated in the civil design package 12d to generate a conceptual representation of a basin feature. Potential basin volumes were estimated considering a very simple approach. In the first instance the full footprint of each reserve area was converted into a basin feature. Basin depths were capped at 1m below natural surface. Batters slopes were set to 1 in 5 (considering safety criteria) and basin inlet and outlet structure locations and sizes were designed to fit in with the existing stormwater network system.

9.2 Basin 1 and other options – CBD Block (Between Gladstone & Mason Streets)

A significant amount of stormwater travels overland from the Warragul CBD precinct towards the industrial area between Gladstone and Mason Streets and as such this area is frequently inundated. There are 3 options available to minimise flooding in this area. These options are discussed below:

1. To construct a major underground drainage pipe to convey major flow to the open waterway in Queen Street. A mitigation pipe solution may be to upgrade or duplicate the existing pipe travelling south on Gladstone Street to Queen Street and then east along Queen Street to the outlet. One potential issue with a proposed pipe upgrade is protecting the outfall from tail water levels in the creek. If water levels in the receiving creek drown the outlet pipe, the effectiveness of the mitigation pipe to relieve flows in the area is greatly reduced. If a piped option is favoured, further hydraulic modelling would be recommended. Further information regarding a piped option is discussed in Section 9.7.
2. To obtain drainage easements on private properties to carry runoff to the open waterway in Queen Street. This option will need to be investigated further to determine the flow paths and width of easements required.
3. To purchase land and construct a retardation basin on a vacant block of land in Gladstone Street. However, Council Officers advised that this is prime real estate located within the central town area, which has been identified in the Warragul Town Centre Urban Design Framework for potential large format retail development. Further, Council officers commented that this is not considered a favourable option and that the use of this land for a retarding basin would significantly compromise Council's strategic intent for the future use of this land.

Another feasible option is to construct an underground storage in the vicinity of the low lying area. Further analysis will be required before this option can proceed

The area were the basin was sited is not currently developed. Table 9-1 outlines conceptual design elements of the basin modelled in this study.

Table 9-1 Basin 1 details

Name / Location	Parcels	Foot print available (m^2)	Max Storage (no freeboard) (m^3)	Outlet size* (mm)	Inlet size* (mm)
Basin 1 (CDB Block) Between Gladstone and Mason Streets	Lot 1 (TP675924) Lot 2 (TP668063) Lot 1 (TP663421)	~11,340	8,678	450	450

* utilising existing drainage network sizes

9.3 Basin 2 – Civic Park (at the northern end of Civic Place)

Stormwater (piped and overland flows) travels east from the residential area (north of the Warragul CBD) through the Civic Park precinct between Smith and Kent Streets before meeting with other overland flows along Normandy Street and moving south towards the receiving water body. This site was identified during the workshop at BBSC (26th June 2012) as having potential to attenuate flows from the upstream catchment, potentially reducing flooding impacts downstream. It is understood that the land where the proposed basin was modelled is currently council owned. Table 9-2 outlines conceptual design elements of the basin modelled.

Table 9-2 Basin 2 details

Name / Location	Parcels	Foot print available (m²)	Max Storage (no freeboard) (m³)	Outlet size* (mm)	Inlet size* (mm)
Basin 2 (Civic Park) Between Smith and Kent Streets	Allot. 5A Sec. 6 Lot 1 TP903844	~15,560	5,595	450	450

* utilising existing drainage network sizes

9.4 Basin 3 – Eisenhower Court (at the Western end of the Court)

Currently stormwater (piped and overland flows) in this system flows east from the residential area (north of the Warragul CBD) through the small parkland at the end of Eisenhower Court between Normandy and Macarthur Streets before travelling south along Normanby Street and Stoffers Street. It is understood that the land where the proposed basin was modelled is currently council owned. Table 9-3 outlines the conceptual design elements of the proposed basin.

Table 9-3 Basin 3 details

Name / Location	Parcels	Foot print available (m²)	Max Storage (no freeboard) (m³)	Outlet size* (mm)	Inlet size* (mm)
Basin 3 (Eisenhower Court) Between Normandy and Macarthur Streets	Lot RES1 LP110740 Lot RES1 LP110644	~7,795	3,400	375	525

* utilising existing drainage network sizes

9.5 Basin 4 – Valleyview Park (West of the intersection between Normanby Street and Ellen Close)

An existing wetland within Valleyveiw Park between Normandy and Princess Streets was identified as an area that may be used to attenuate flows and reduce flooding within Ellen Close. While this site is relatively small, during the workshop this site was identified as having potential to manage flows from the relatively small upstream catchment potentially reducing flooding impacts downstream. It is understood that the land where the proposed basin was modelled is currently council owned. Table 9-4 outlines the conceptual design elements of the proposed basin.

Table 9-4 Basin 4 details

Name / Location	Parcel	Foot print available (m²)	Max Storage (no freeboard) (m³)	Outlet size* (mm)	Inlet size* (mm)
Basin 4 (Valleyview Park) Between Normandy & Princess Streets	Lot 1 TP906058	~2,465	1476	600	225

* utilising existing drainage network sizes

9.6 Mitigation Modelling Results

As shown in Figure 9-2 below there is a reduction in maximum depth downstream of all retarding basins modelled. The benefits of attenuating flows at each basin were completed in a single run to show the maximum benefit possible through the use of all retarding basins. All basins were designed to be at a maximum depth of 1 metre, however only the CBD basin reached close to its capacity, and results showed it was likely that pits immediately downstream of the basin may surcharge during a large event. While this highlights the need for further modelling, for costing, detailed design and construction, the conceptual modelling does show there are significant flood protection benefits to be gained through the use of retarding basins within the Warragul township. The attenuation of flows within the retarding basins located at Civic Park and Eisenhower Court have a positive impact on the downstream flooding with a widespread reduction in depth of 2-5cm along the Normanby Street flow path, of which was shown as a major flooding issue under existing conditions.

From the conceptual modelling of all four retarding basins, the It is recommended that further design work is completed to provide a better understanding of the costs and benefits associate with the mitigation solutions once floor level surveys within the existing flood extent are captured.

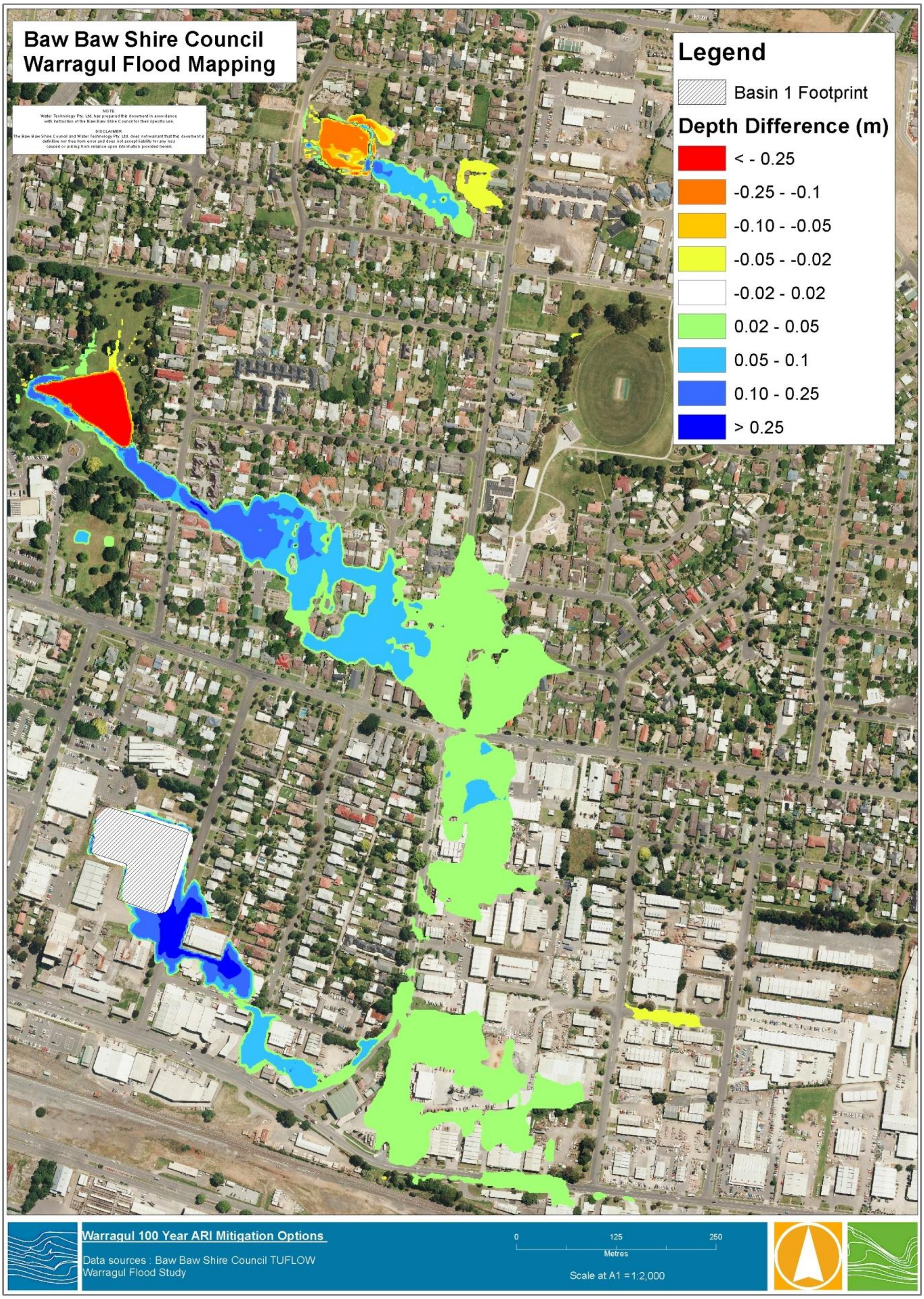

Figure 9-2 **Depth Difference Plot (Existing – Mitigated Scenario) showing reductions in flood depth for basins 1,2 & 3. Refer to Figure 9-5 below for basin 4 results.**

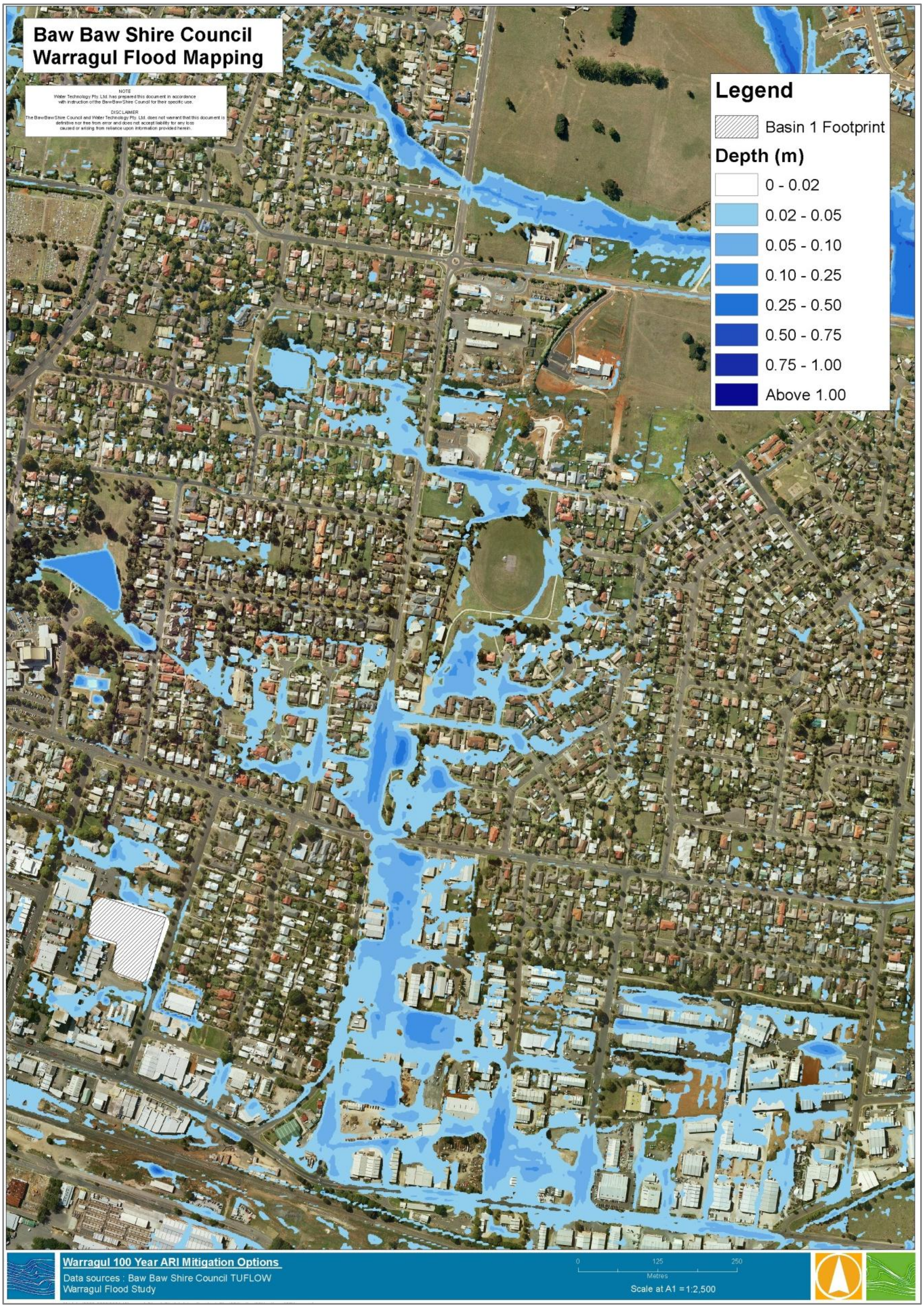

Figure 9-3 Mitigation Scenario 100 Year ARI Flood Depths for basins 1,2 & 3.

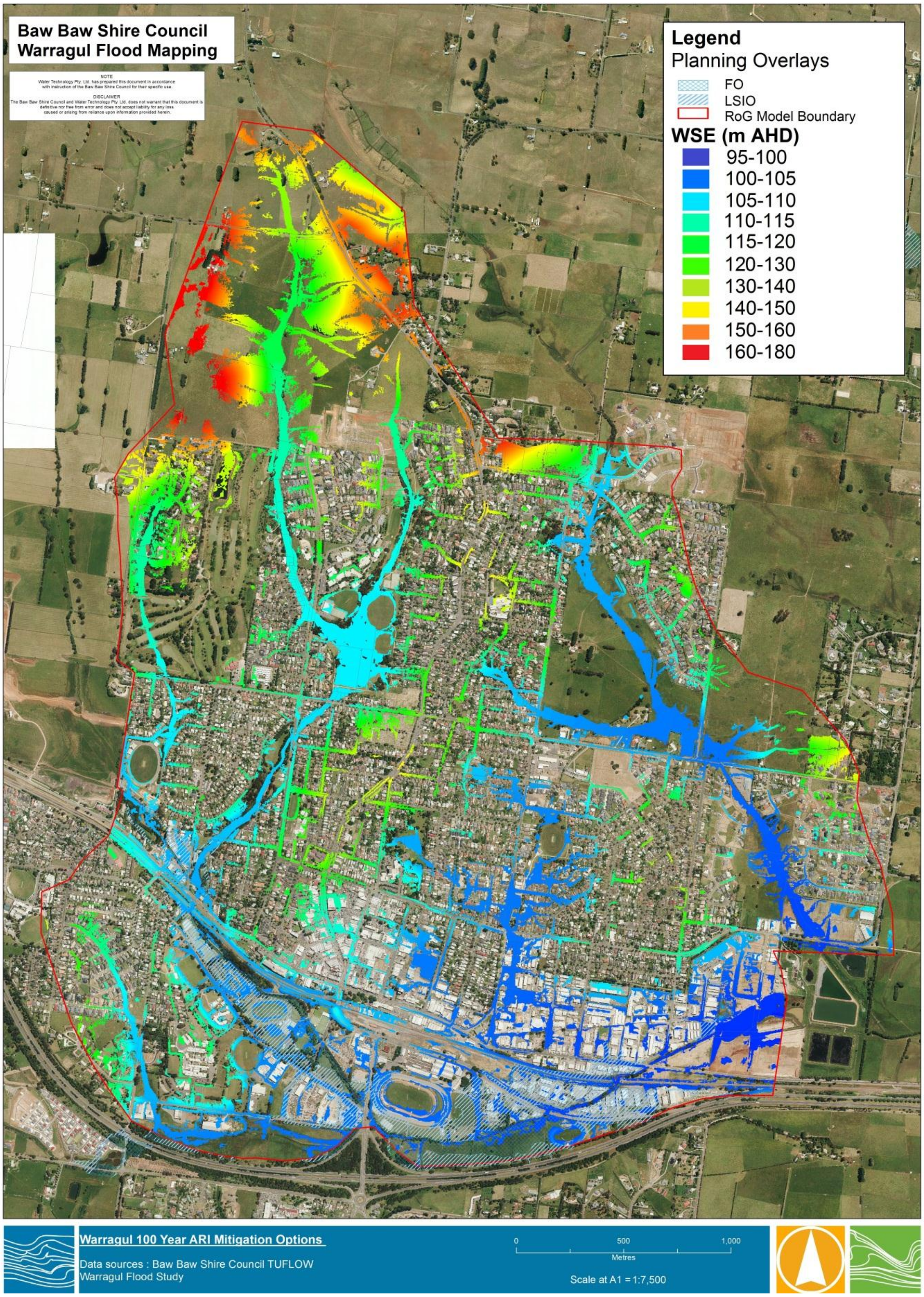

Figure 9-4 Mitigation Scenario 100 Year ARI Flood Levels

9.6.1 Basin 1 – CBD Block (Between Gladstone & Mason Streets)

The retarding basin located in the vacant block at Gladstone Street provides a significant reduction in flooding immediately downstream of the basin along Gladstone Street, removing the majority of flooding at Peace Avenue and across to Normanby Place. During the modelling, the basin filled close to 1 metre in depth and would also provide hydraulic efficiency in providing a head water level to push the water through at the outlets. However, there are issues regarding the construction of this basin as previous discussed under Section 9.2

9.6.2 Basin 2 – Civic Park (at the northern end of Civic Place)

The retarding basin modelled within Civic Park shows a significant reduction in flooding immediately downstream of up to 10cm and further downstream through residential properties to Normanby Street. The Civic Park basin filled up to 60cm maximum depth suggesting that further detailed modelling may reduce the footprint of the basin. Immediately downstream there is water ponding up behind Kent Street, likely a result of a surcharging pit. Under existing conditions a number of properties downstream of Civic Park are subject to flooding where a defined overland path conveys flows through residential areas at George Street, Janette Close and Melanie Drive to Normanby Street. The mitigation results show that a large number of properties would have a significant reduction in overland flows while many will have reduced all flooding as a result of the retarding basin.

9.6.3 Basin 3 – Eisenhower Court (at the Western end of the Court)

There is a significant reduction in flooding through the residential properties downstream of the Eisenhower Court retarding basin, as flows are attenuated within the basin. Properties along Eisenhower and Alexander Street show a reduction in flood depth of up to 10cm as much of the overland flow is retained within the Eisenhower Street road reserve. This option offers further potential to direct any overflow from the retarding basin along Eisenhower Court reducing overland flow through residential properties. The results also showed a localised area of increased flood depth as a pit downstream of the basin surcharges in Eisenhower Court due to the increased hydraulic grade line associated with increased water surface elevations within the proposed retarding basin. Further detailed modelling should address this and possibly look at sealing this pit should the basin be constructed.

9.6.4 Basin 4 – Valleyview Park (West of the intersection between Normanby Street and Ellen Close)

The small basin located within Valleyview Park provides attenuation up to depths of 50cm within the basin and provides minor flood depth reduction of up to 5cm along Ellen Close. Site generated runoff from the catchment only fills the retarding basin to approximately 50% full therefore with more detailed modelling may reduce the footprint area required. There is still significant flooding along Ellen Close of which most is confined to the roadway and waterway. At present the existing 600 mm pipe is used as the outlet structure. An alternative smaller outlet pipe configuration which connects directly to the waterway to the east of Valleyview Park could be used to further reduce downstream flooding.

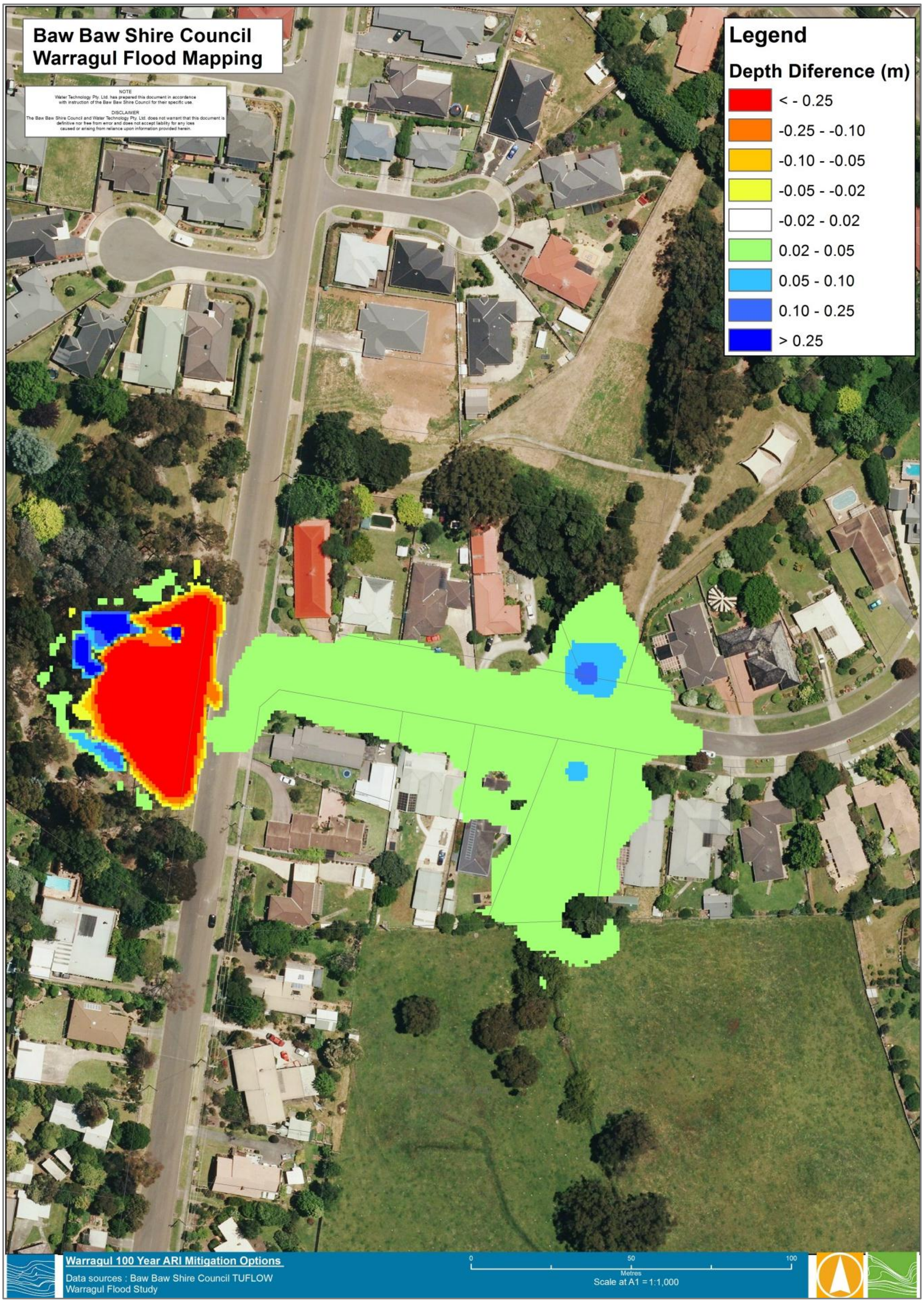

Figure 9-5 Valleyview Park Retarding Basin Difference Plot

9.7 Costing and Recommendations

The full benefit of the four retarding basins is difficult to quantity given the lack of floor survey data for the affected properties. Nevertheless, based on the reductions in flood levels and flood extents the Civic Park basin is seen as the most appropriate site for a retarding basin. To assist with further assessment of the four retarding basin options presented, a rough cost of each retarding basin was calculated (Table 9-5). The estimated cost includes a 15% engineering fee, 9% administration fee and 30% contingency fee. A Land acquisition cost has also been included for the CBD block basin, this has been based on a land acquisition cost of $1,500,000 per hectare of developable land for the CBD block, Basin 1 (refer to Section 9.2 regarding issues with the construction of Basin 1). As the land acquisition cost makes up the majority of the cost of the basin, it is important that more detailed costing estimates of this land are completed during functional design. Please note, all cost estimates are based on preliminary design and excavation volumes. Many items have not been accounted for such as site specific obstructions, tree removal, site establishment, etc. and as such all cost estimates should be treated as rough estimates only and used for comparison between the construction costs of each of the four basins presented.

Table 9-5 Estimated Cost for Conceptual Retarding Basins

Retarding Basin	Estimated Cost
CDB Block	$ 2,404,365
Civic Park	$ 619,000
Eisenhower Court	$ 185,000
Valleyview Park	$ 169,000

Additional concept design and costing has been completed for a piped solution to mitigate flooding if the CBD Block (Retarding Basin 1) is not constructed. A pipe was sized to carry flows from the northern existing pipe crossing of Mason Street south to Queen Street (Segment 1) and then east along Queen Street to the outlet (Segment 2). Table 9-6 below shows the details of each pipe to convey the peak 1 in 100 year ARI overland flow rate of $3.3m^3/s$.

Table 9-6 Details of Potential Mitigation Pipe

Segment	Required Length (m)	Required Slope (1 in x)	Pipe Diameter (mm)
1	314	89	1200
2	552	162	1200

Table 9-7 below shows a conceptual costing of the proposed pipe upgrade. The pipe has been designed to follow the road network rather than through private property. As such, invert depths are up to 6.9m below the natural surface level, particularly along Mason Street. It is generally accepted that at such depths, pipe jacking or tunnelling is a preferred option for constructability and construction safety reasons. Tunnelling would also minimise disruptions to the public on the busy Mason Street and Drouin-Warragul Road, although jacking/tunnelling launching and monitoring pits would be expected to cause some significant disruption.

If upgrading the pipe network or constructing a retarding basin are not feasible options for controlling the overland flow that runs between Mason and Gladstone Streets, an easement may be appropriate through the private property to convey the 100 year ARI flow of 3.3 m^3/s through the site. Widths of the easement would vary depending on the maximum depth Council will allow through the site. As an example, at 1 in 100 grade and a maximum water depth of 350mm, a 3m wide easement (with works to ensure flood waters stay within the easement) would be expected to convey the 100 year ARI flow through the site. Further detailed calculations should be completed prior to setting of any easement widths.

Table 9-7 Pipe Mitigation Costing – Mason Street to Outlet

Location	Works Description	Pipeline Diameter (mm)	Length (m)	Factored Unit Cost ($/m)	Cost ($)	TOTAL Cost With Design & Contingency ($)
Mason and Palmerston Street Intersection	Large Inlet Pit			$ 10,000	$ 10,000	$ 15,535
Mason and Palmerston Street Intersection to Williams Street	Pipe Augmentation	1200	193	$ 2,096	$ 404,528	$ 628,434
Mason Street/Williams Street Intersection	Junction Pit			$ 5,000	$ 5,000	$ 7,768
Williams St to Queen Street	Pipe Augmentation	1200	121	$ 2,096	$ 253,616	$ 393,992
Queen Street to Outlet	Pipe Augmentation	1200	552	$ 2,096	$ 1,156,992	$ 1,797,387
Headwall at outlet	Headwall	1200		$ 1,800	$ 1,800	$ 2,796
Non-return flap gate	Gate	1200		$ 1,800	$ 1,800	$ 2,796
Total Cost					**$ 1,833,736**	**$ 2,848,709**

10. RECOMMENDATIONS ON THE LSIO AND FO AND FLOOD EMERGENCY RESPONSE

10.1 Overview

A component of the Warragul Flood study project was the task of "Recommendations on the Land Subject to Inundation Overlay (LSIO) and Floodway Overlay (FO) and flood emergency response." This task was discussed with key stakeholders (BBSC, Bureau of Meteorology (BoM) and the Victorian Sate Emergency Service) during the project inception phase. Additional funding was secured through the support of the WGCMA, this enabled more detailed hydraulic modelling of the Hazel and Spring Creek floodplains reducing the reliance on historical flood studies and previous development based modelling work.

Anecdotal evidence and preliminary hydrological analysis of the Warragul catchment suggested that flooding which impacted the highly populated areas (inside the Warragul urban growth boundary) came from two unique mechanisms, flash flooding (or stormwater) resulting from intense rainfall falling on the highly impervious area within the urban growth boundary, and riverine flooding of the Hazel and Spring creek floodplains resulting from rain in the upper catchments moving through the catchment and inundating low lying developed areas around Warragul.

Impacts from the flash flooding scenario were intensively investigated in the Urban Rain on Grid modelling task (see Section 7). While impacts from riverine flooding are to be determined as part of this task and the Warragul Waterway Modelling Project (*Water Technology 2012*) funded by the WGCMA.

Due to the moderately steep nature and relatively small size of the upstream catchment, the Hazel and Spring creek system could be considered a flashy catchment (i.e. flooding are caused by flash flood), suggesting the travel times between a significant rain event in the upper catchment and the flooding which impacts Warragul township is minimal.

While mapping the LSIO and FO conditions throughout the study area was a relatively simple task involving interpretation of the hydraulic modelling results, development of suitable emergency response data for VicSES was more of a challenging task. At the beginning of the project the Bureau of Meteorology (BoM) were contacted to provide advice on a possible flood warning system to assist VicSES and BBSC in flood emergency response. The following comments were offered by Mrs Elma Kazazic of the Flood Warning team (Victoria):

- To her knowledge no flood warnings have been issued for Warragul;
- She suggested contacting the DSE Floodplain Management Group to discuss the DSE's views / records etc. on flood warning systems in flashy catchments such as Warragul;
- She noted that Melbourne Water are currently doing some work in the Brushy Creek catchment (a similarly flashy catchment) where warnings and emergency alerts are sent to residents within the catchment via SMS to warn of impeding flooding danger. The Brushy Creek alert system has 3 tiers;
    - A severe weather alert warning residents to be prepared for possible flooding,
    - A link to rain gauges in the upper catchment, which trigger a warning once rainfall intensity exceeds a certain level,
    - A series of stream gauges throughout the lower catchment, again which trigger warnings once stream level heights are exceeded.
- Mrs Kazazic was also aware of several Councils in NSW that have similar flashy catchments – including Coffs Harbour as one example;

- Mrs Kazaziac made the comment that generally, the installation of warning gauges is driven by assessment of risk and consequences, and that it is typically a 'beneficiary pays' approach;

Considering this advice, the project budget / key outputs and the unique flood emergency response challenges were identified, and outputs for the VicSES were restricted to additional mapping and GIS analysis processes instead of recommendations for specific flood warning infrastructure e.g. gauges or similar. This was discussed with BBSC and VicSES and agreed to be a suitable approach. It is noted that this modelling work may be revisited in the near future during the development of the Warragul Flood Emergency Plan. Water Technology, BBSC and VicSES will need to work together to assist with this project.

10.2 Data Inputs

As discussed above, the formulation of LSIO/FO as well as advice on flood emergencies is based on two main studies.

1) Within the Warragul Urban catchment, the flood modelling presented in this report is used for the generation of the LSIO/FO as well as the preparation of flood emergency advice. The full set of ARI scenarios were investigated from the 2 year ARI through to the 100 year ARI.
2) Along the Spring and Hazel Creek floodplain, the findings from the Warragul Waterway Modelling Project (*Water Technology 2012*) was used for the generation of the LSIO/FO as well as the preparation of flood emergency advice. Only the 100 year ARI flood event was investigated in this study.

Project backgrounds, hydrology and hydraulic specifics from each study can be found in their associated report.

10.3 Land Subject to Inundation Overlay and Floodway Overlay

The delineation of flood overlays is set out in accordance with the guidelines developed by Department of Infrastructure (DOI) in 2000. The guidelines (DOI, 2000) suggest the following tools to assist in managing the risk of flooding:

Urban Floodway Zone: The UFZ applies to mainstream flooding in urban areas where the primary function of the land is to convey active flood flows. It applies to urban floodway areas where the potential flood risk is high due to the presence of existing development or to pressures for new or more intensive development.

The UFZ restricts the use of such land, as the risk associated with flooding renders it unsuitable for any further intensification of use or development. The land use is therefore restricted to activities such as agriculture, animal husbandry and recreational activities. Most other uses are prohibited.

Sometimes the UFZ can cover the full extent of land subject to inundation, including situations where the floodplain is relatively narrow and deep.

The UFZ is not widely used due to its restrictive nature. As an alternative, a flood overlay can be used in conjunction with an appropriate zone (such as the Floodway Overlay and the Public Park and Recreation Zone) to enable the primary use of the land to be recognised at the same time as acknowledging its flooding characteristics.

Floodway Overlay: The FO applies to mainstream flooding in both rural and urban areas. These areas convey active flood flows or store floodwater in a similar way to the UFZ, but with a lesser flood risk. The FO is suitable for areas where there is less need for control over land use, and the focus is more on control of development.

As with the UFZ, in some cases the FO can cover the full extent of land subject to inundation, for example, in situations where the floodplain is relatively narrow and deep.

Land Subject to Inundation Overlay: The LSIO applies to mainstream flooding in both rural and urban areas. In general, areas covered by the LSIO have a lower flood risk than UFZ or FO areas.

Special Building Overlay: The SBO applies to stormwater flooding in urban areas only. Before 1975, drainage systems were designed to a lower standard than those used today. Often they were designed for a five-year ARI storm capacity, and sometimes for a lesser standard. Usually no provision was made for overland flows, so land is often flooded when the capacity of the underground drainage system is exceeded.

With the redevelopment of existing urban areas and the proposed development of new areas, there will be pressure to develop within overland flow-path areas. The purpose of the SBO is to manage development in these areas. While the SBO is primarily intended for overland flow path areas in the Melbourne metropolitan area, it can also be applied to urban areas affected by stormwater flooding in regional towns.

The guidelines outline the flood risk factors to be considered in the delineation of flooding overlays. From these guidelines, the following three approaches to the delineation of overlays were assessed:

- **Flood frequency**: Department of Natural Resources and Environment, DNRE (1998) suggest areas which flood frequently and for which the consequences of flooding are moderate or high should generally be regarded as floodway. The 10 year ARI flood extent was considered an appropriate floodway delineation option based on flood frequency.

- **Flood depth:** Regions with a flood depth in the 1 in 100 year ARI event greater than 0.5 m were considered as FO based on the flood depth delineation option.

- **Flood hazard:** Flood hazard combines the flood depth and flow velocity for a given design flood event. DNRE (1998) suggest the use of Figure 10-1 for delineating the floodway based on flood hazard. The flood hazard for the 1 in 100 year ARI event was considered for this study.

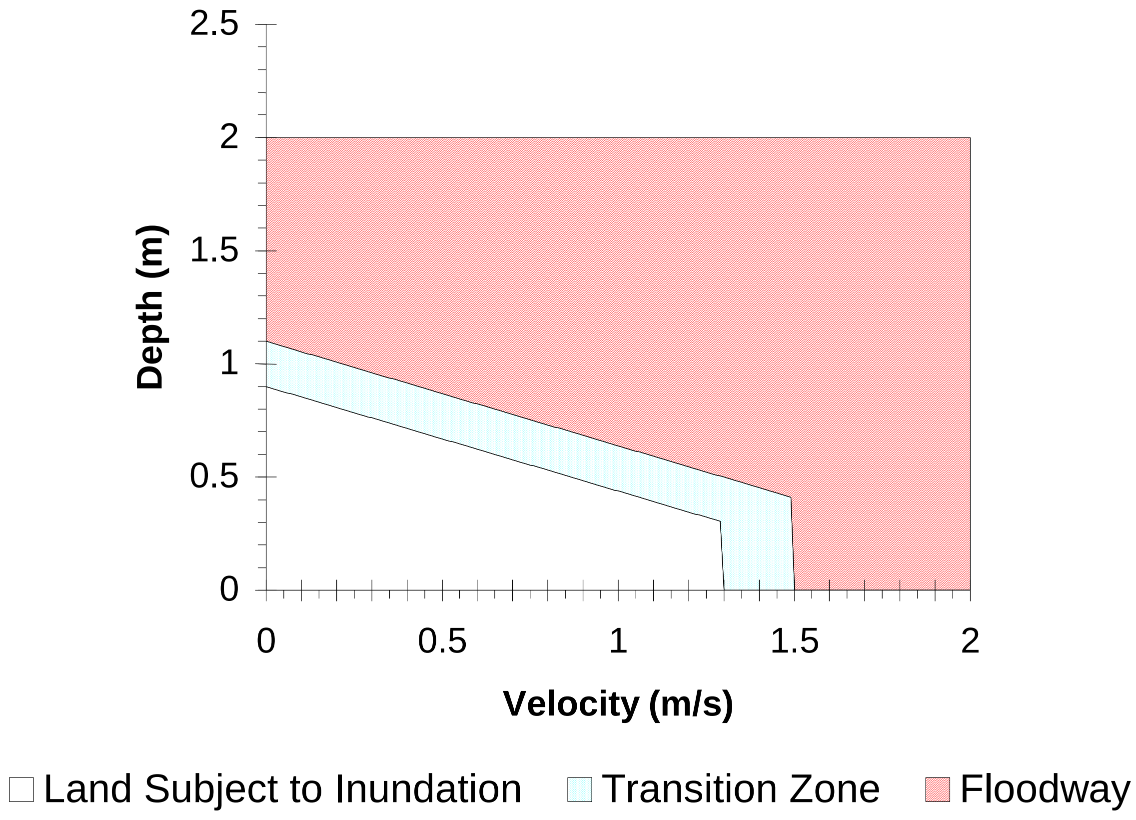

Figure 10-1 Floodway overlay flood hazard criteria

Special building overlays (SBOs) are generally not supported by the Victorian CMAs and hence are not considered appropriate for use in Warragul.

Based on the comprehensive and detailed methodology presented in this study (Section 7 and Warragul Waterway Modelling Project, WT 2012) Water Technology recommends the following shapes (**Figure 10-2**) to be considered as LSIO and FO layers within the Warragul study area. Current LSIO and FO shapes are shown in Figure 10-3.

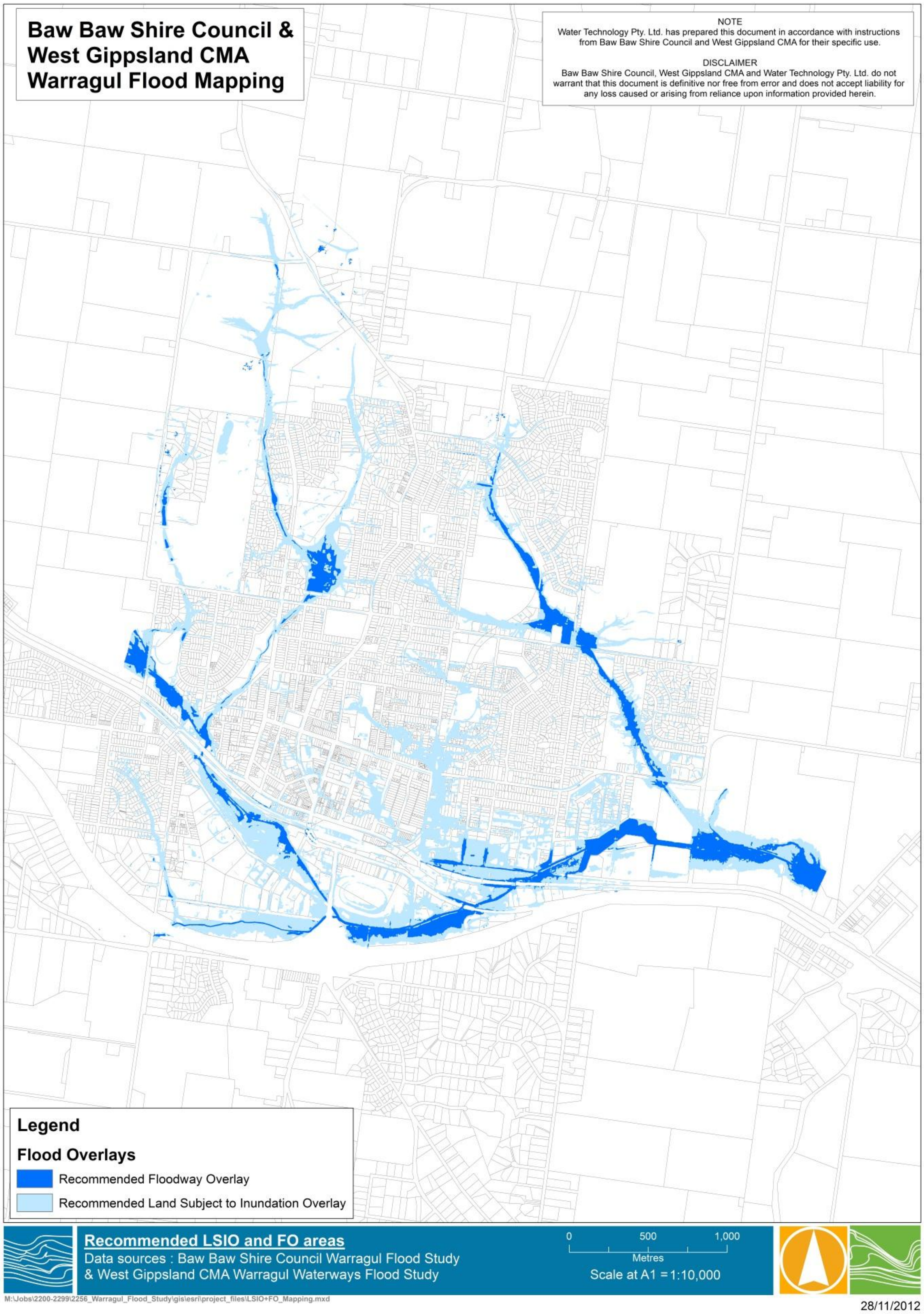

Figure 10-2 Recommended LSIO and FO shape from the Warragul Flood study and modelling project

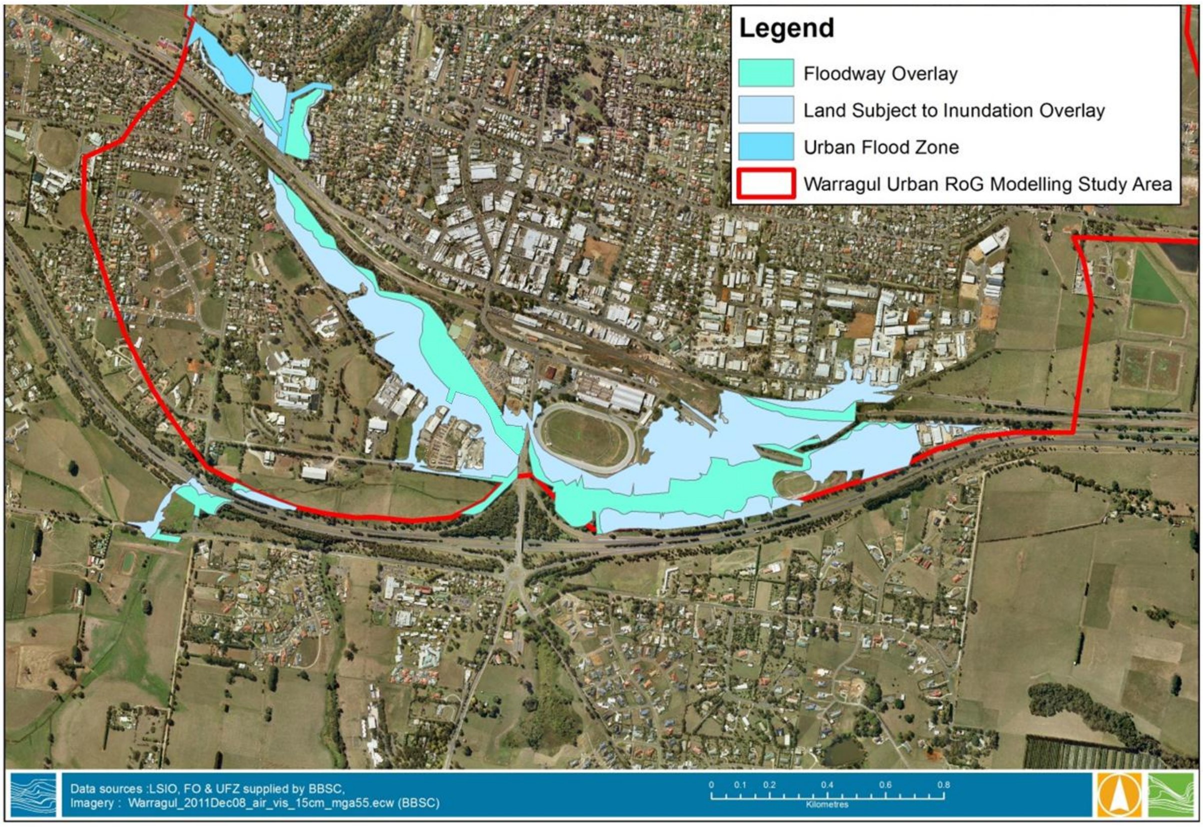

Figure 10-3 Current (2012) Land Subject to Inundation Overlay (LSIO) & Floodway Overlay (FO)

The shape of the planning layers (LSIO/FO) derived from this study is similar to the old planning layers, with a few differences as follows:

- The current FO has a slightly larger extent than the proposed FO layer from this study;
- The old planning layers (LSIO, FO and UFZ) were based on older topography data and do not cover the urbanised section of Warragul; and
- While the old layers cover most of Hazel Creek, the layers are very coarse and don't include sections of Spring Creek and parts of Hazel Creek downstream of Alfred Street.

10.4 Development of suitable emergency response data for VicSES

As discussed earlier, specific flood warning processes or systems were not developed as part of this study, instead a series of maps and tables resulting from post processing flood modelling results were generated at the request of VicSES. A meeting was held on the 22nd of August 2012 with VicSES staff (Jane Rowe & Dave Walker), where existing conditions urban rain on grid modelling results were presented including catchment critical duration and hazard outputs. Suitable outputs from this project were discussed with two additional maps and tables being identified as being useful to the VicSES.

Details of land parcels inundated are a key parameter to VicSES emergency management and planning processes. As no floor level data is available for the Warragul area, making maps of inundation above floor level (a typical output from this type of study) was not possible. In the absence of this data it was proposed to provide a map and table of land parcels which experience inundation greater than 0.1m. This data could then be revisited if and when floor level data is collected in the Warragul area. Results of this analysis are shown in Figure 10-4.

Hazard mapping is another key tool which VicSES can used to understand flood risk. While a simple flood risk map was produced for the urban area in the Rain on Grid modelling report it did not cover all of the Warragul area.

Figure 10-5 uses the combined results of the urban rain on grid modelling and the waterway direct inflow modelling and presents flood risk for all of the Warragul flood study and modelling project study area.

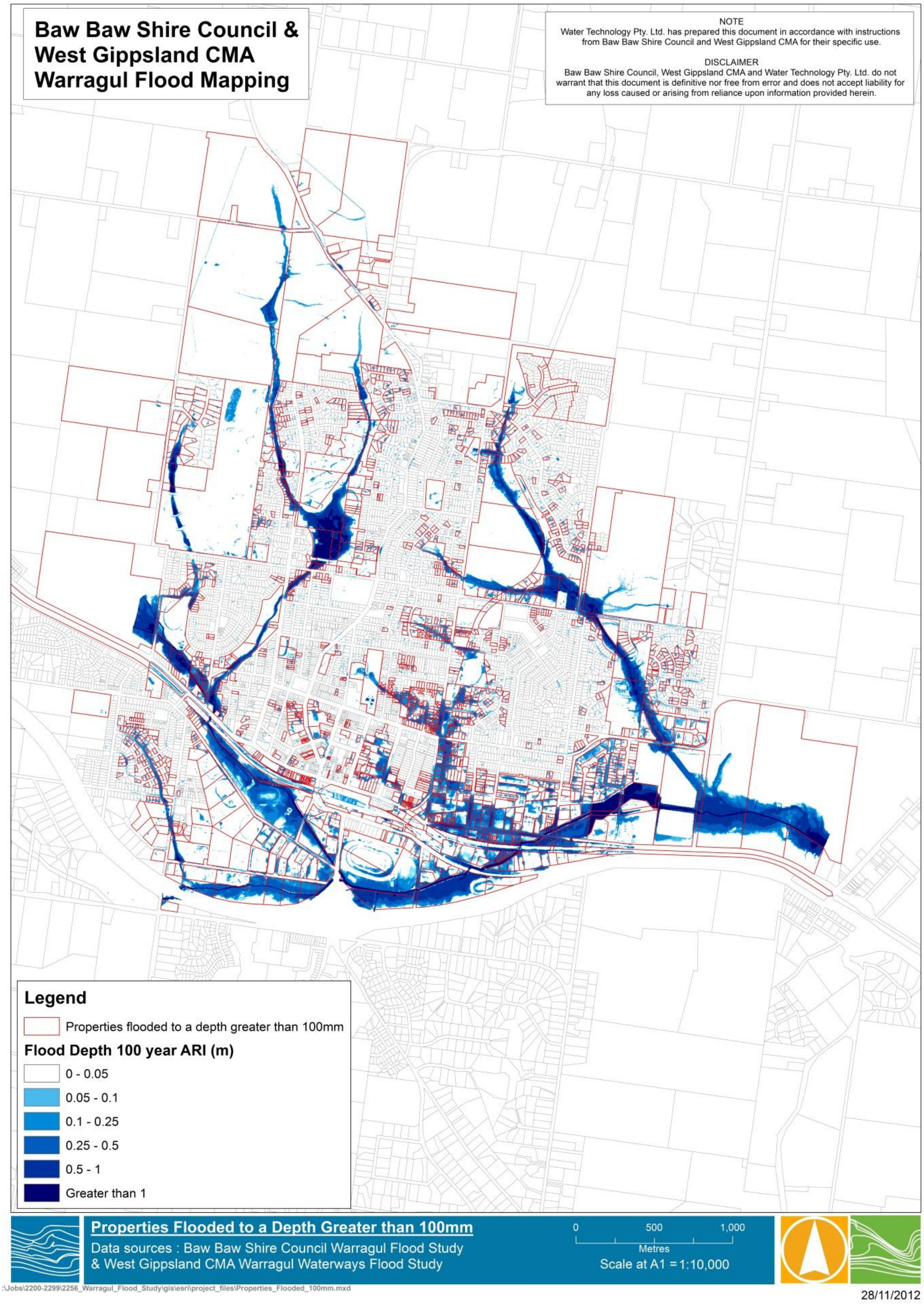

Figure 10-4 Parcels inundated above 0.1m in the Warragul study area

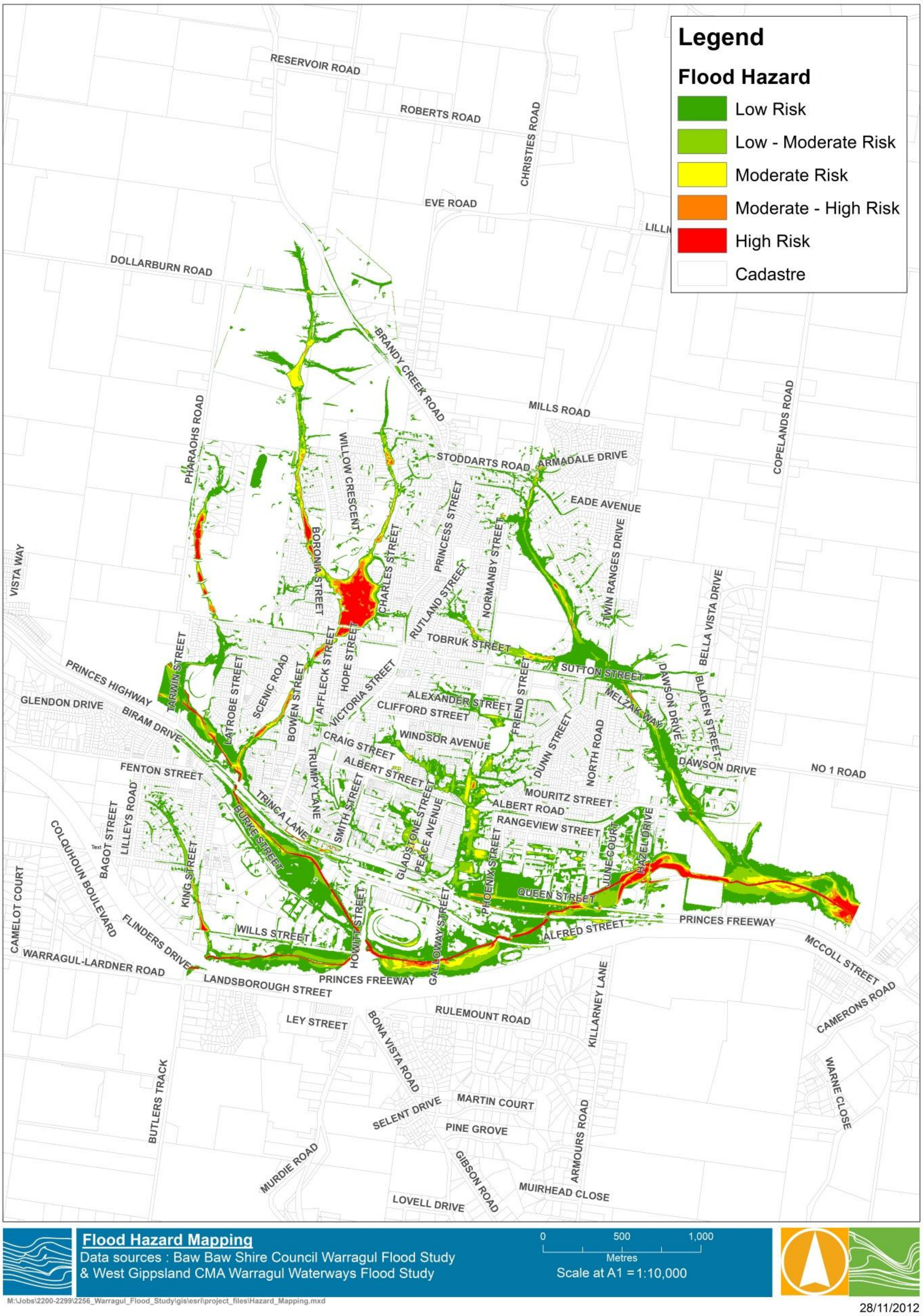

Figure 10-5 Hazard mapping within the Warragul flood study and modelling project study area

The final output of interest to the VicSES arising from the meeting on the 22nd of August 2012 was a map which linked the critical duration (and corresponding rainfall intensity) and the maximum 100yr ARI flood extent. Using this map SES staff and volunteers would be able to consider weather warnings provided by the BoM and where impacts may occur based on the type of weather predicted (e.g. a short burst storm event compared to a long duration rain event over many days).

Figure 10-6 shows the 100 year ARI critical duration used to generate the 100 year flood mapping results (depths), it also shows where the critical 100 year ARI durations occur throughout the catchment.

The Table embedded in the figure shows the maximum 100 year ARI rainfall intensity by duration.

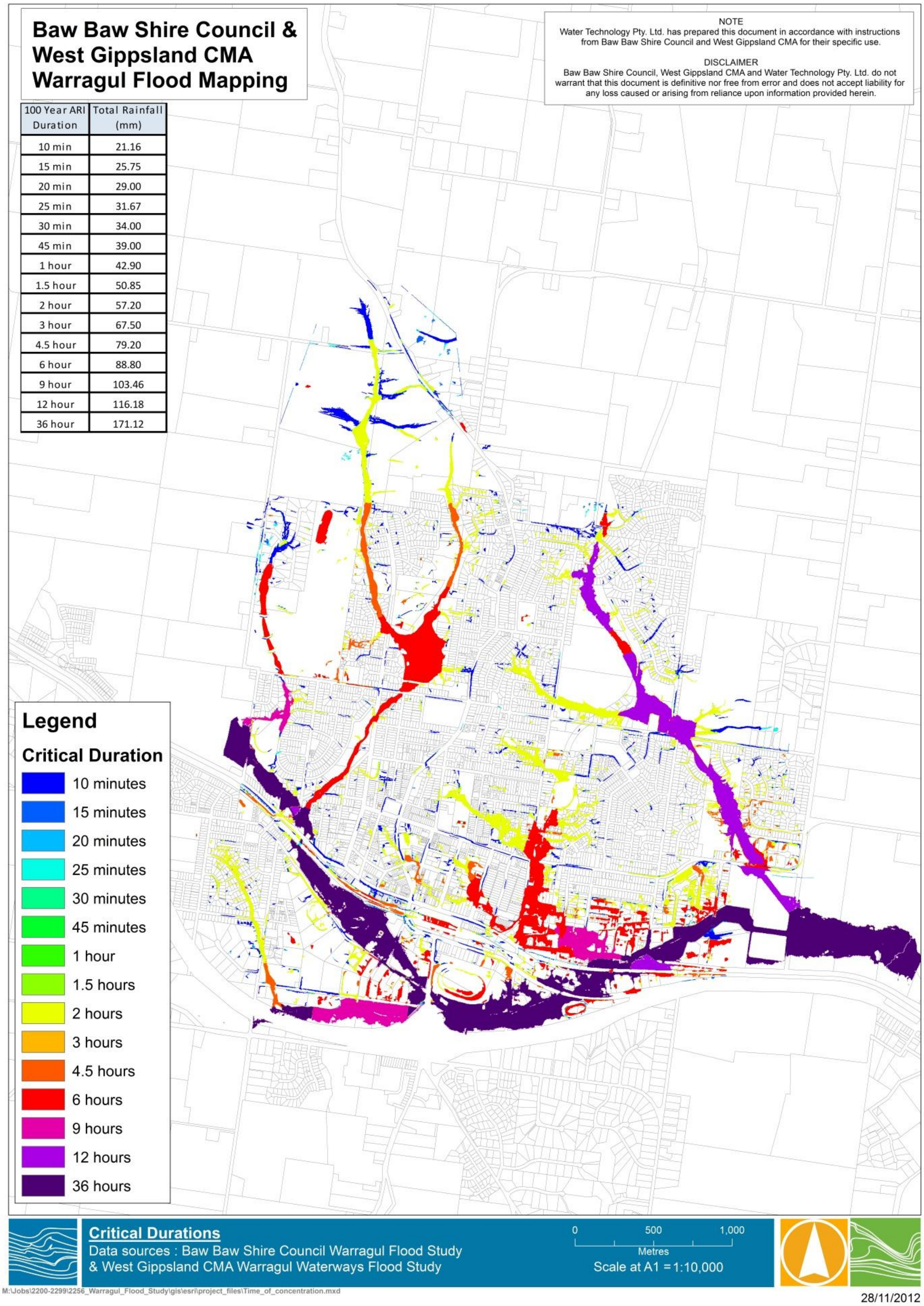

Figure 10-6 Critical Duration Map

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Water Technology (2012a). 11-13 Ryan Court Warragul – Flooding Investigation

Water Technology (2011c). Stoddarts Road Retarding Basin / Wetland Functional Design

Appendix A EXISTING RETARDING BASIN OPTIMISATION RESULTS

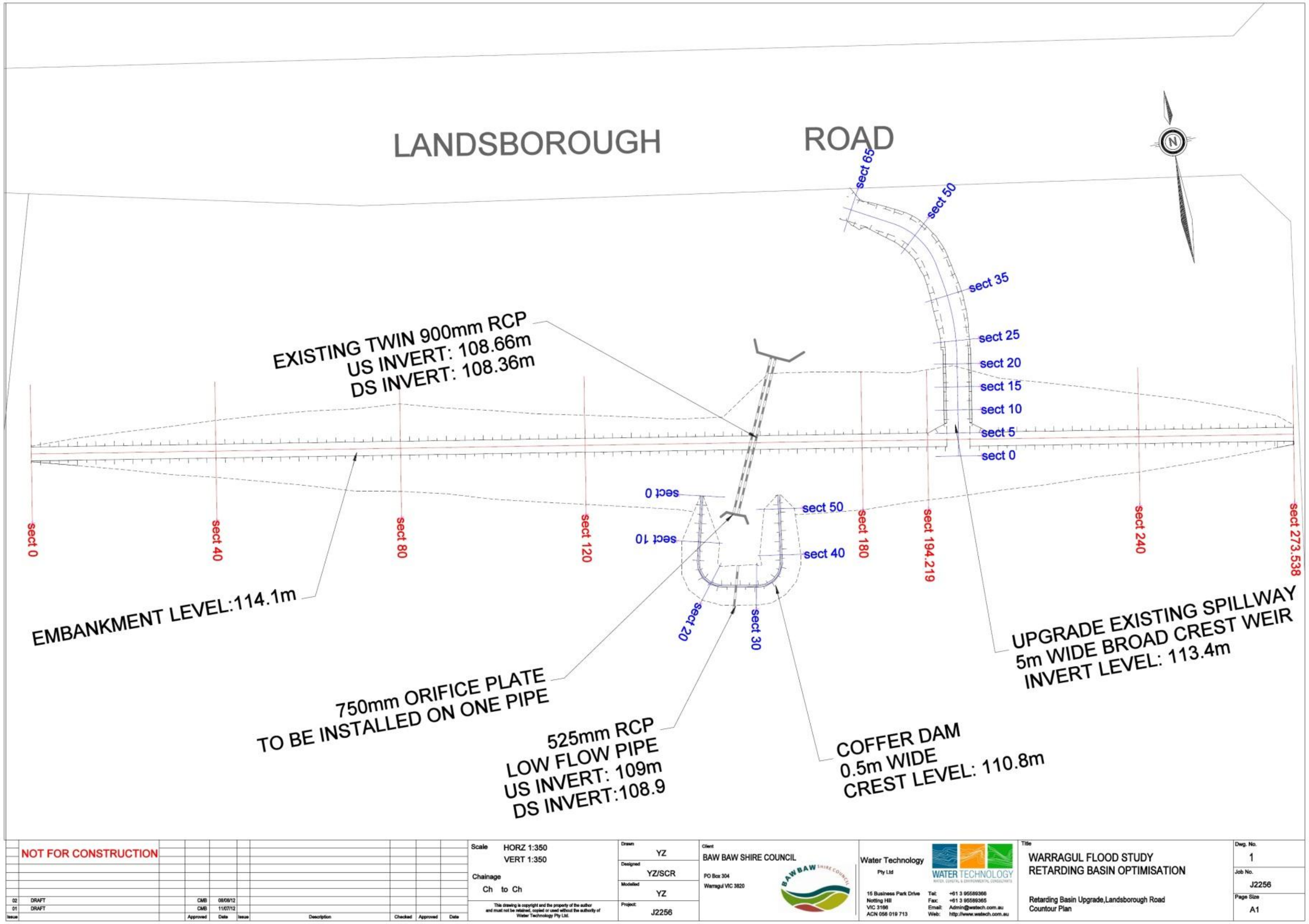


Figure A - 1 **Landsborough Road Existing Basin**

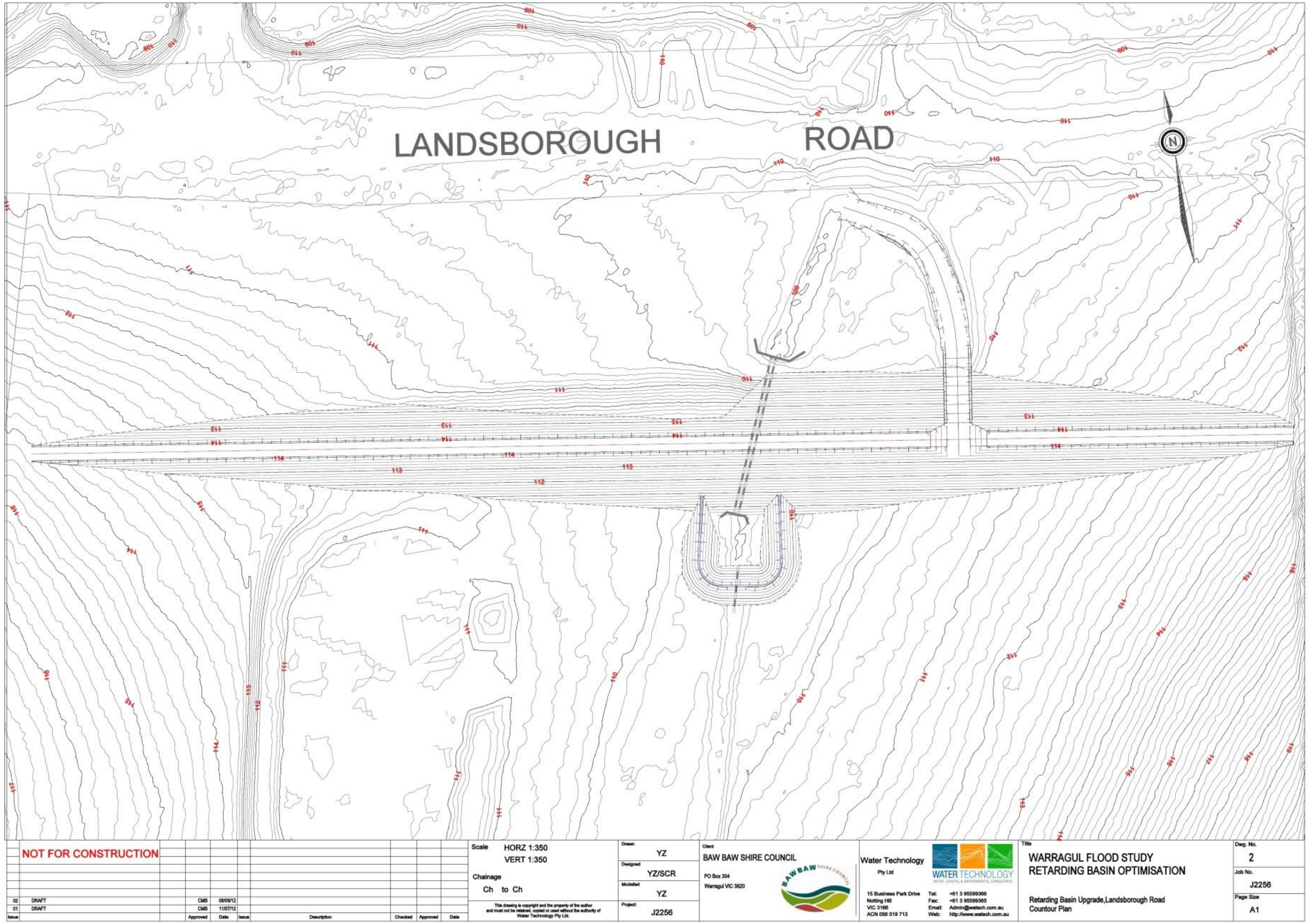
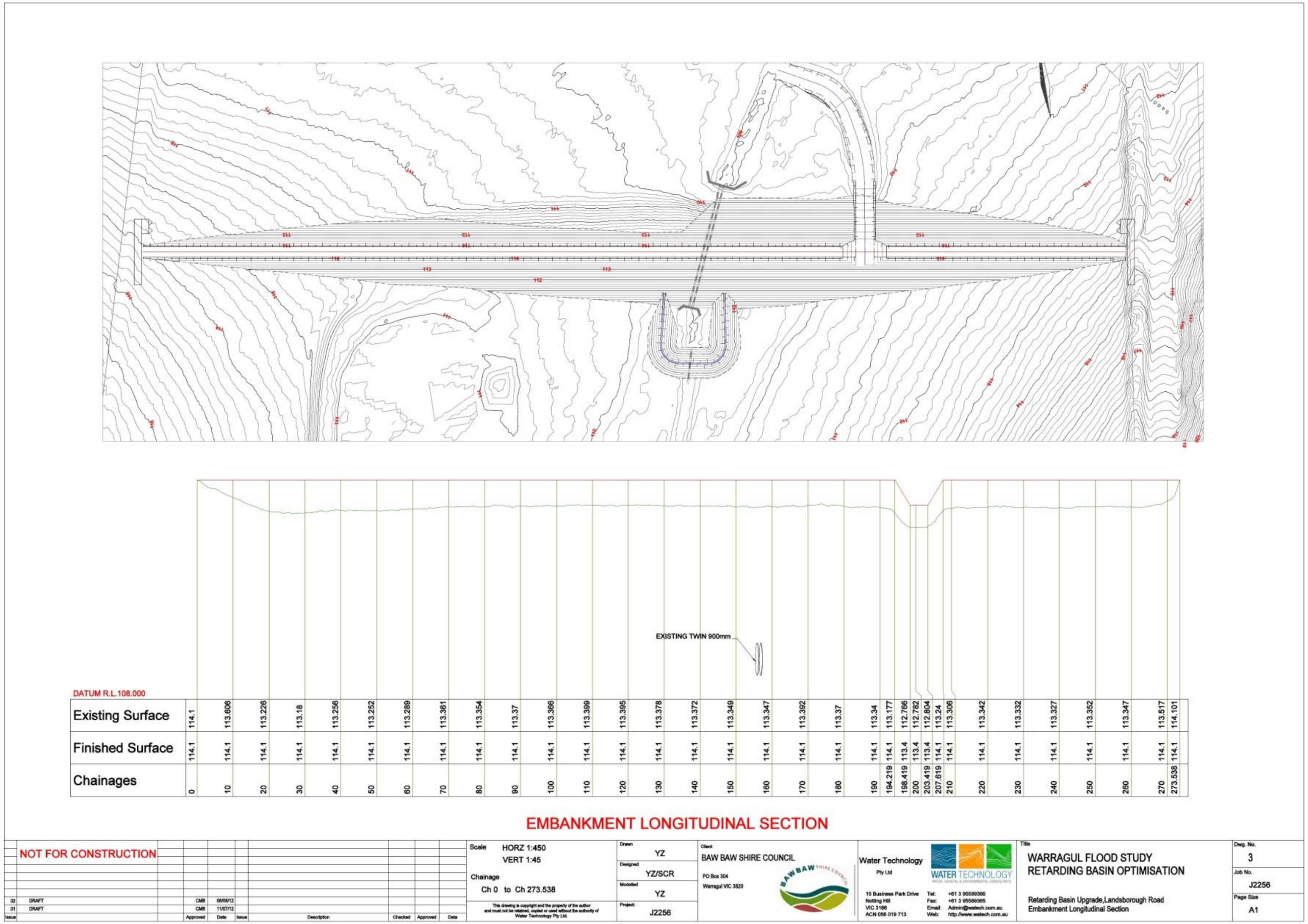


Figure A - 2 Landsborough Road Existing Basin



Chainages	0	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160
Existing Surface	114.1	113.606	113.226	113.18	113.256	113.252	113.289	113.361	113.354	113.37	113.366	113.399	113.395	113.378	113.372	113.349	113.347
Finished Surface	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1

Chainages	170	180	190	194.219	198.419	200	203.419	207.619	210	220	230	240	250	260	270	273.538
Existing Surface	113.392	113.37	113.34	113.177	112.766	112.782	112.804	113.24	113.306	113.342	113.332	113.327	113.352	113.347	113.517	114.101
Finished Surface	114.1	114.1	114.1	114.1	113.4	113.4	113.4	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1	114.1

Figure A - 3 Landsborough Road Embankment Longitudinal Section

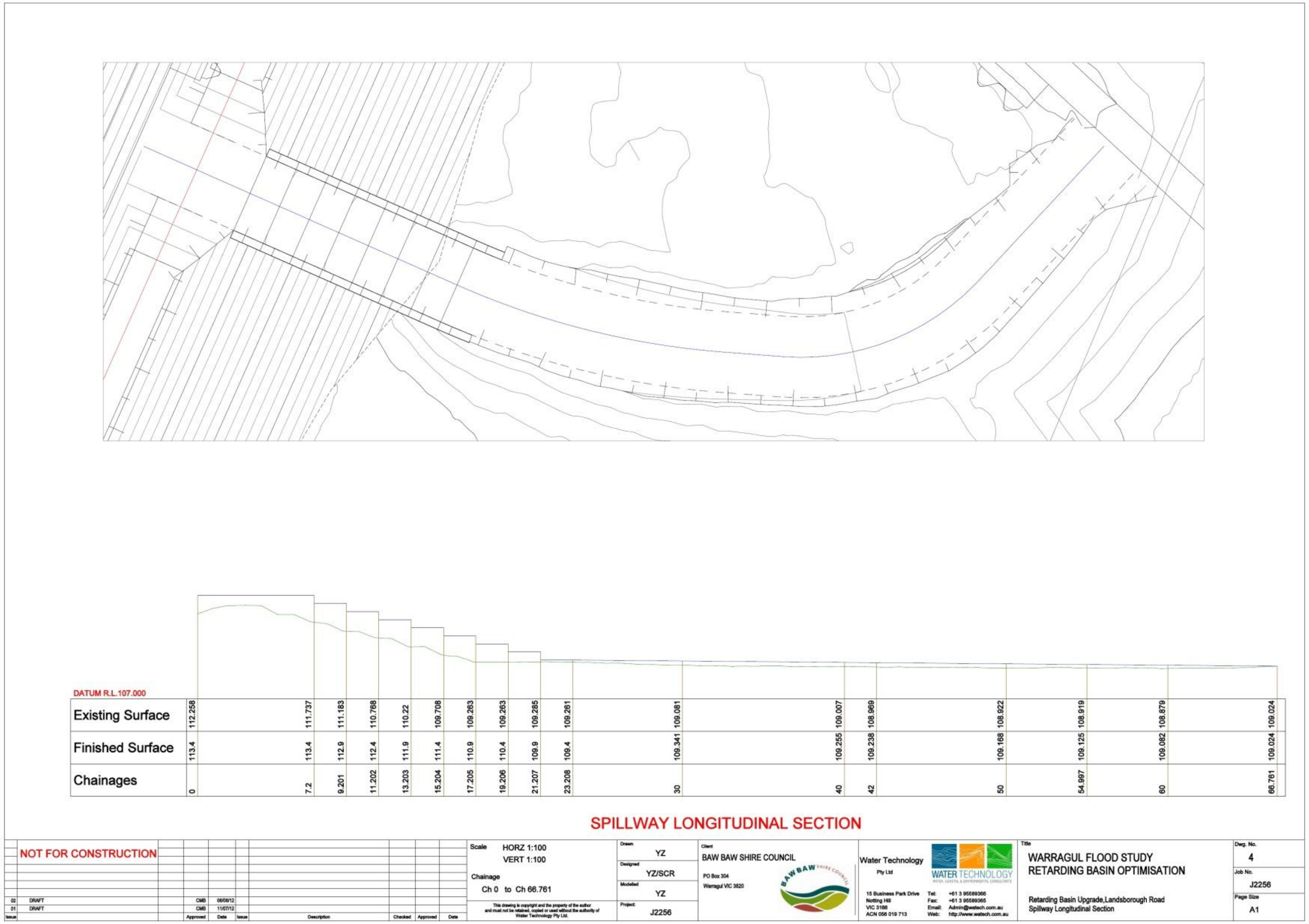


Figure A - 4 Landsborough Road Spillway Longitudinal Section

Figure A - 5 Landsborough Road Coffer Dam Longitudinal Section

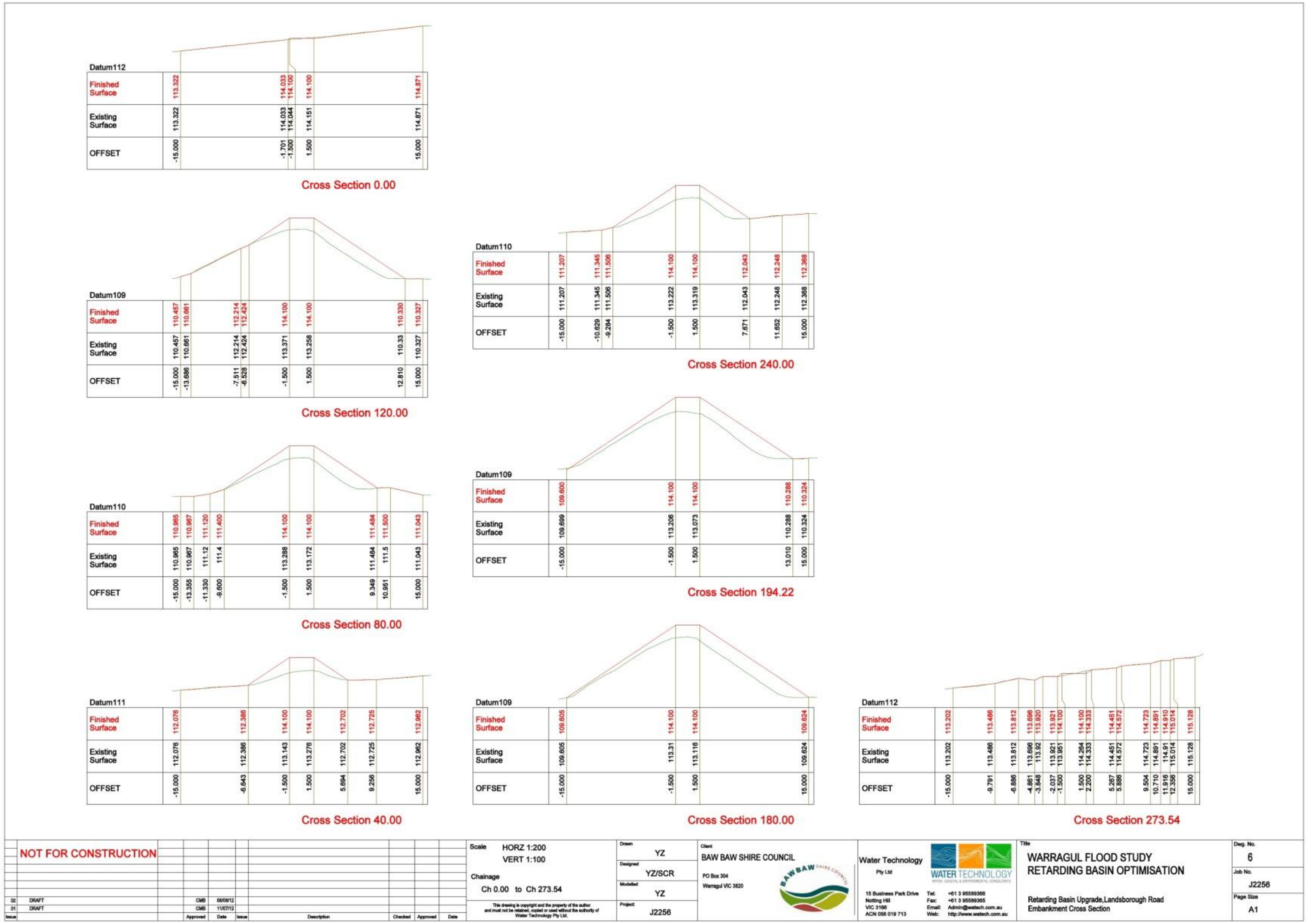


Figure A - 6 Landsborough Road Cross Sections 1/3

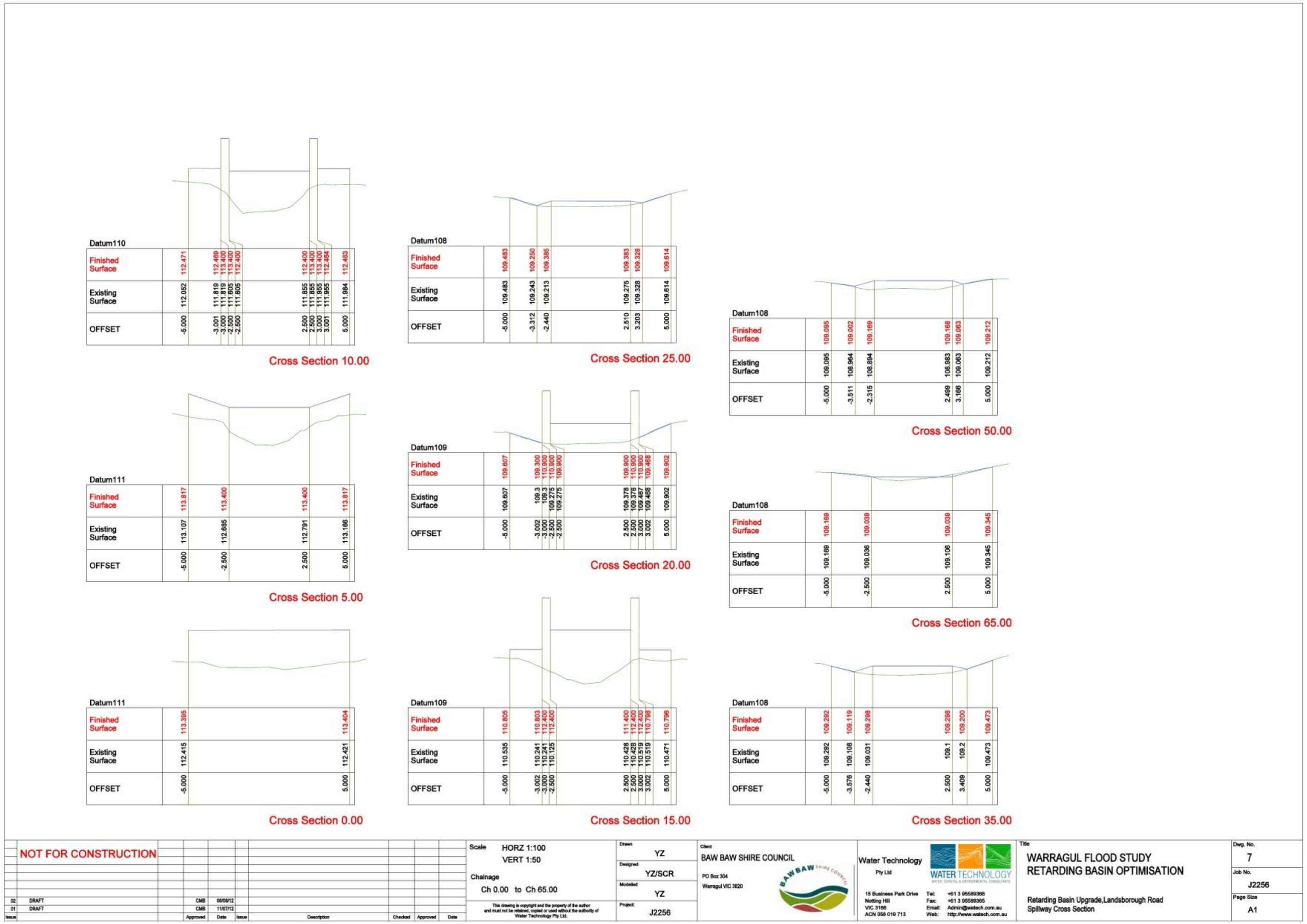


Figure A - 7 Landsborough Road Cross Sections 2/3

Figure A - 8 Landsborough Road Cross Sections 3/3

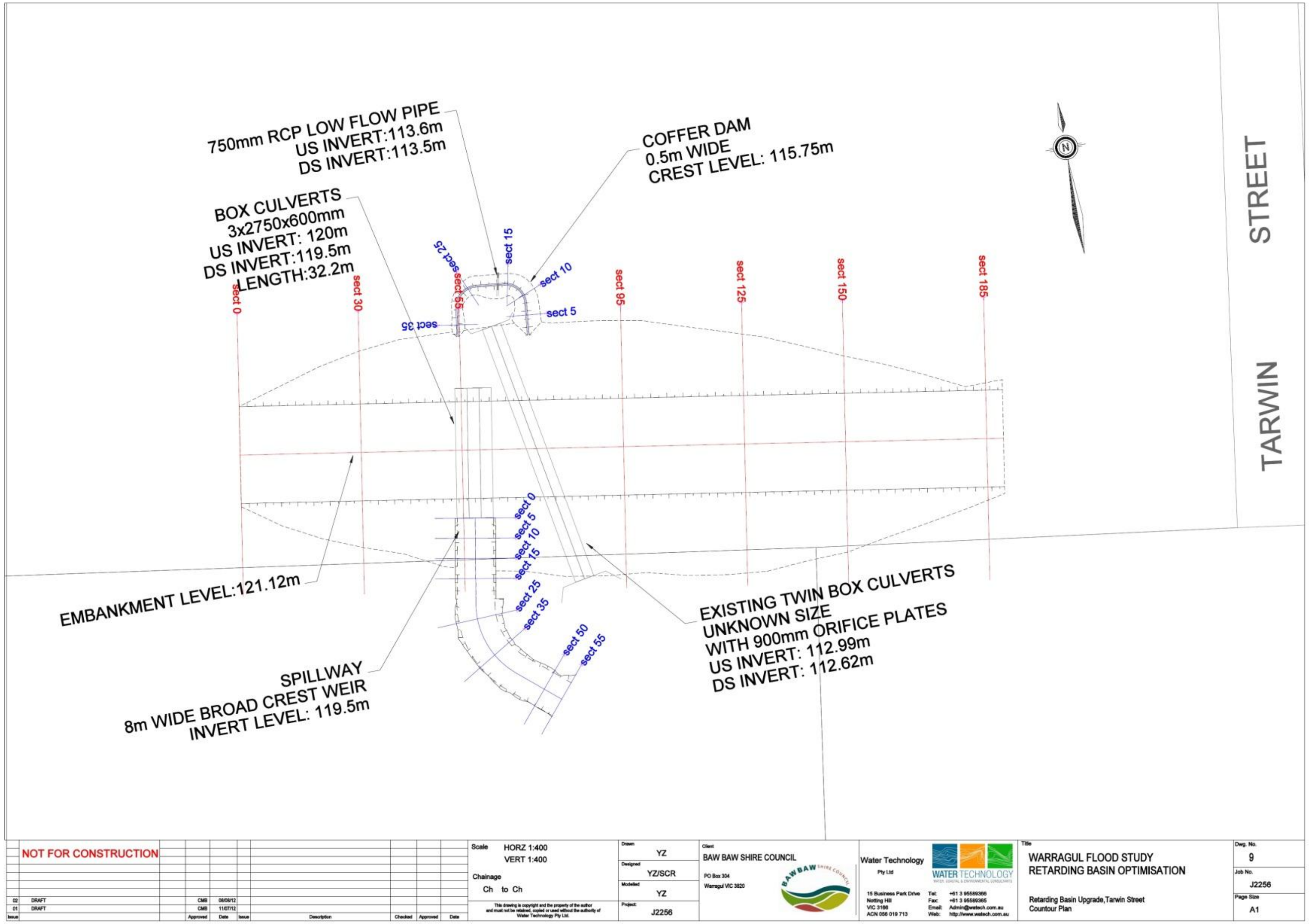


Figure A - 9 Tarwin Street Existing Basin

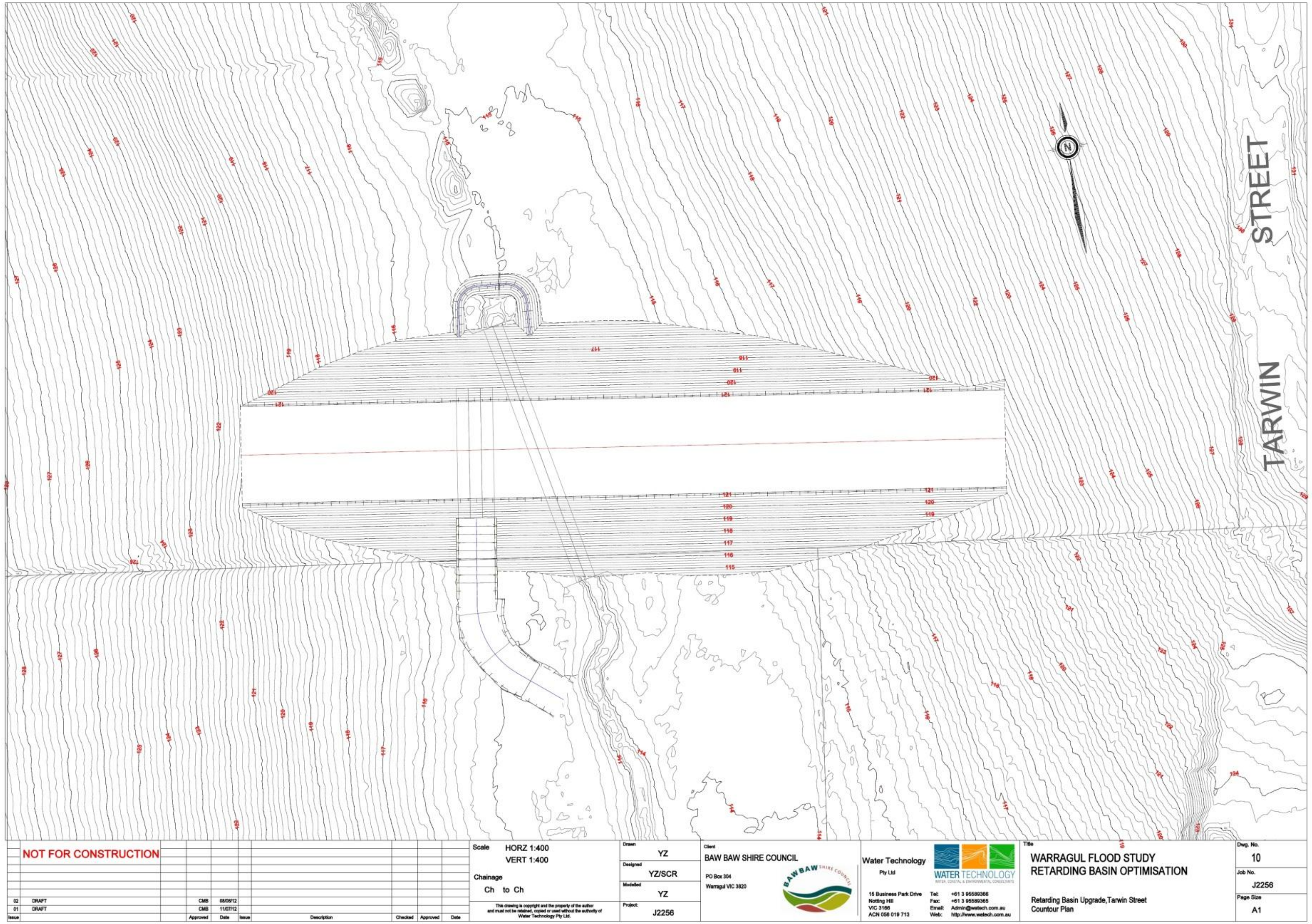
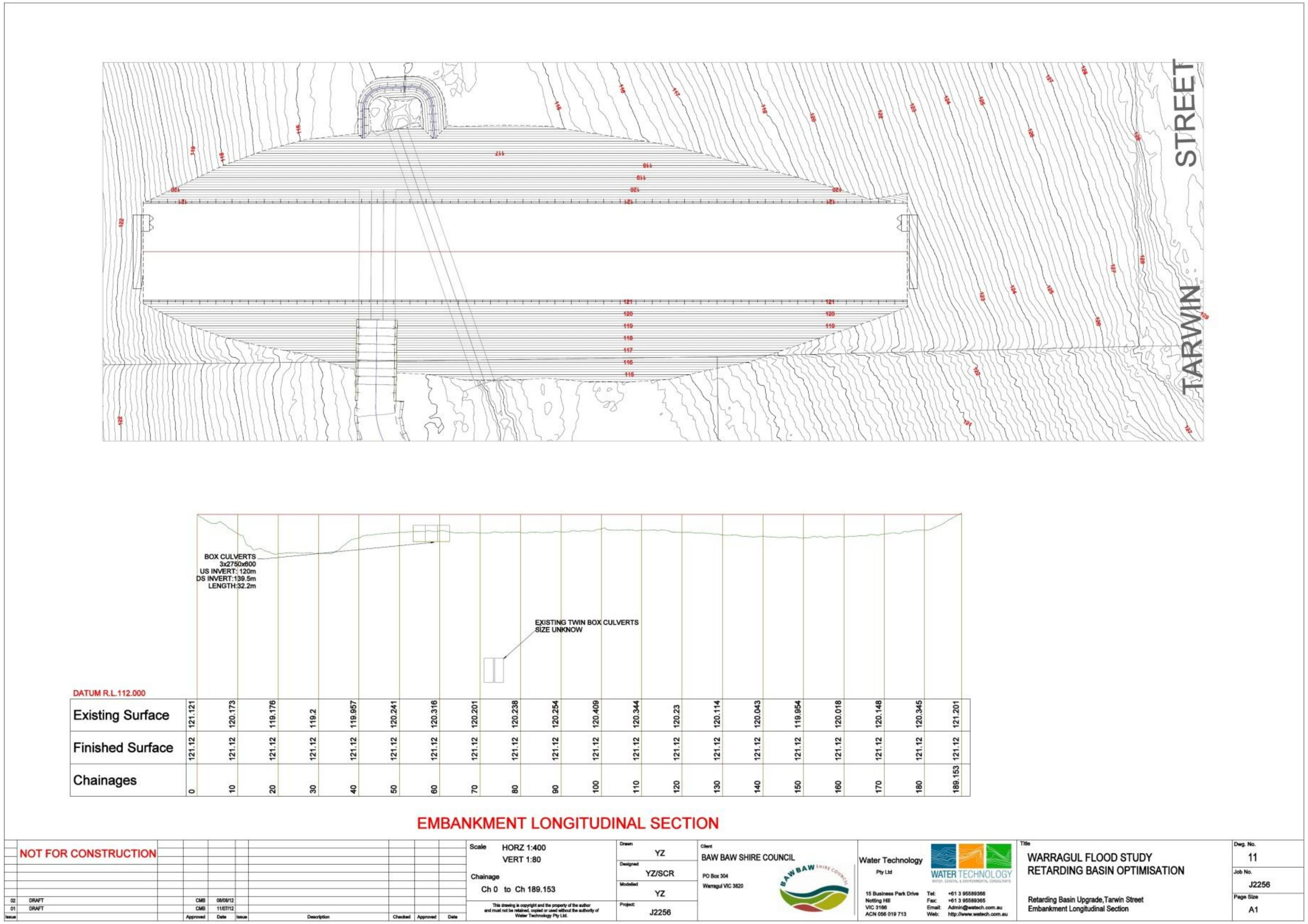


Figure A - 10 Tarwin Street Existing Basin Wall



Existing Surface	121.121	120.173	119.176	119.2	119.957	120.241	120.316	120.201	120.238	120.254	120.409	120.344	120.23	120.114	120.043	119.954	120.018	120.148	120.345	121.201
Finished Surface	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12	121.12
Chainages	0	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170	180	189.153

Figure A - 11 Tarwin Street Embankment Longitudinal Section

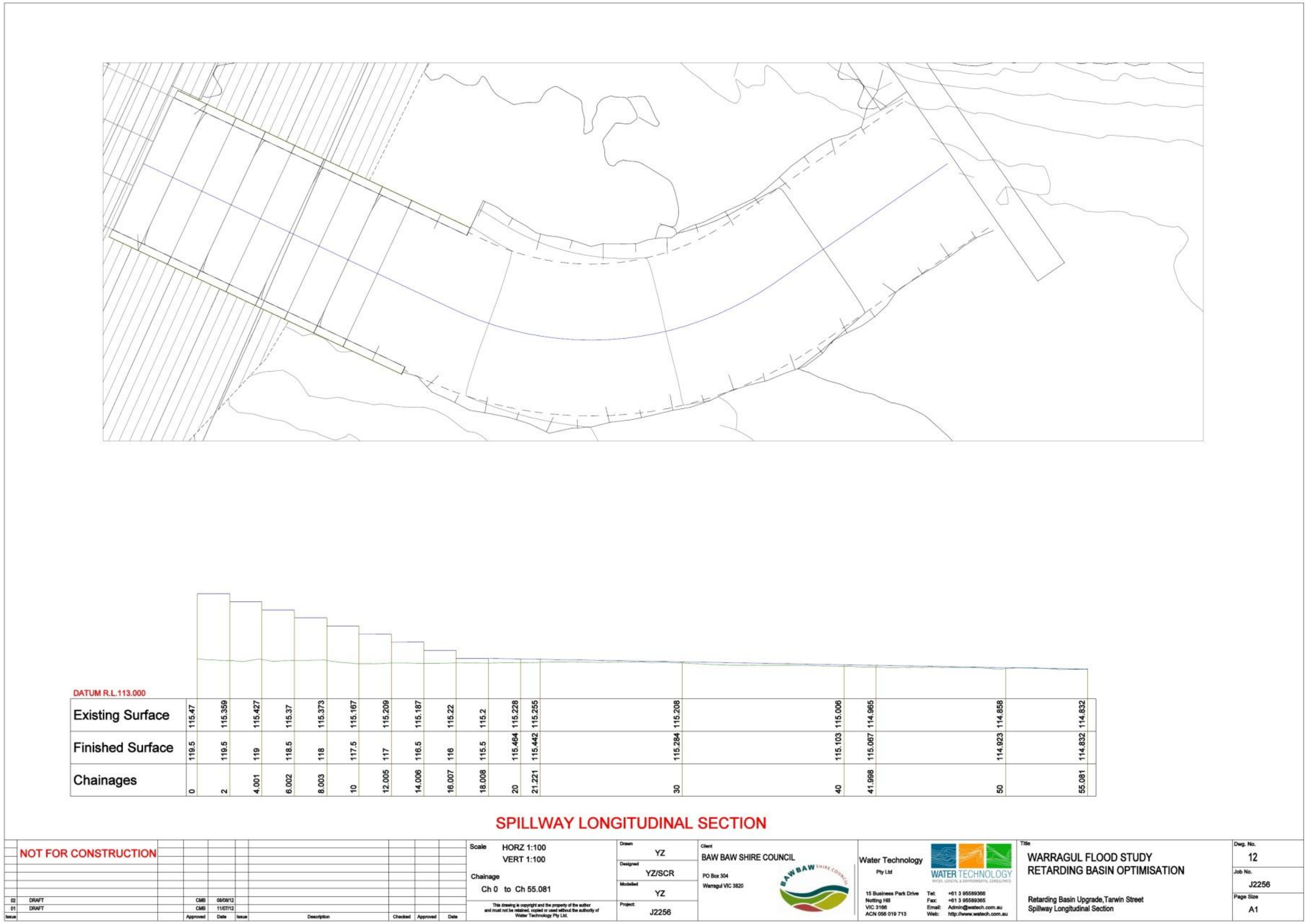


Figure A - 12 Spillway Longitudinal Section

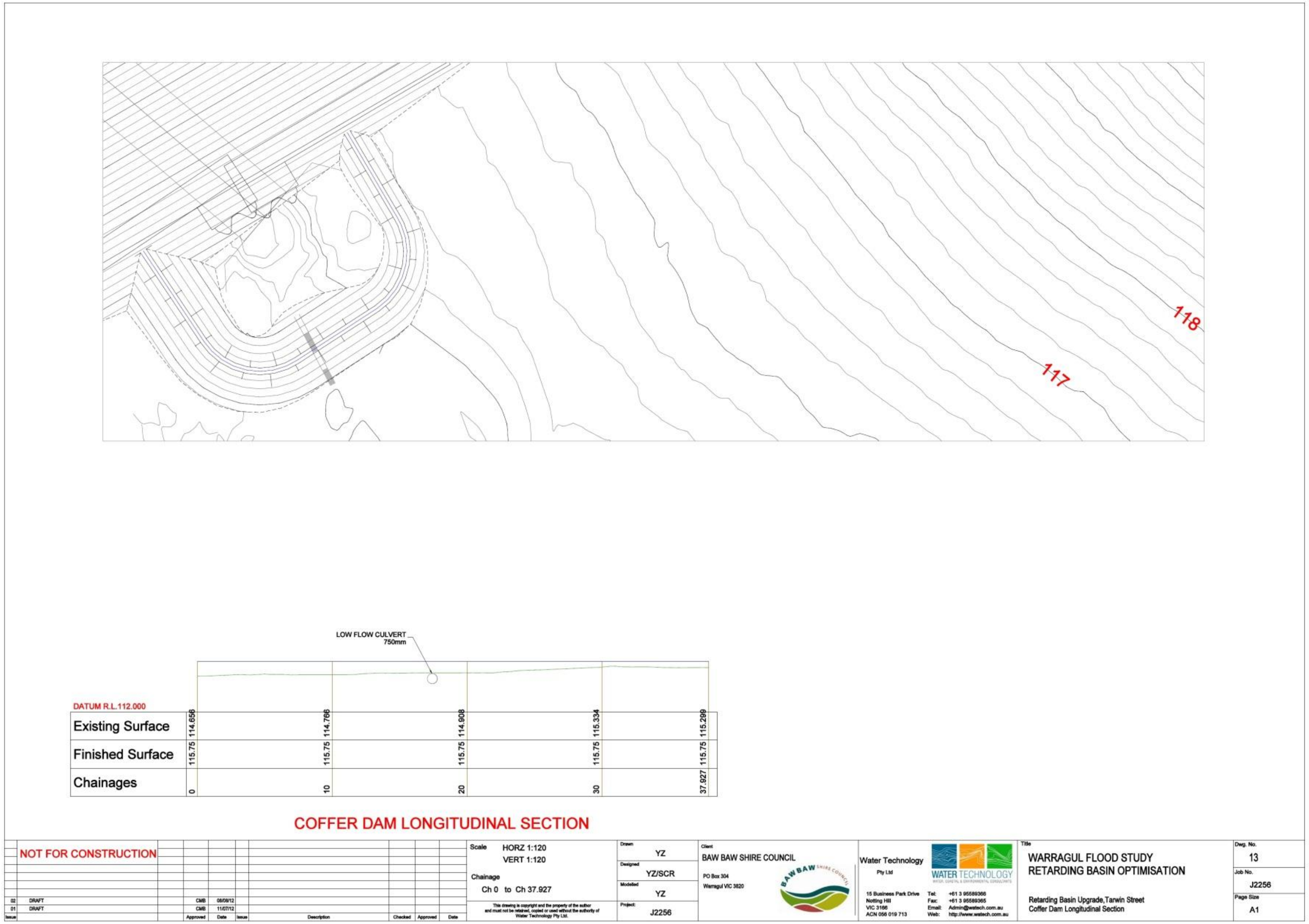


Figure A - 13 Tarwin Street Coffer Dam Longitudinal Section

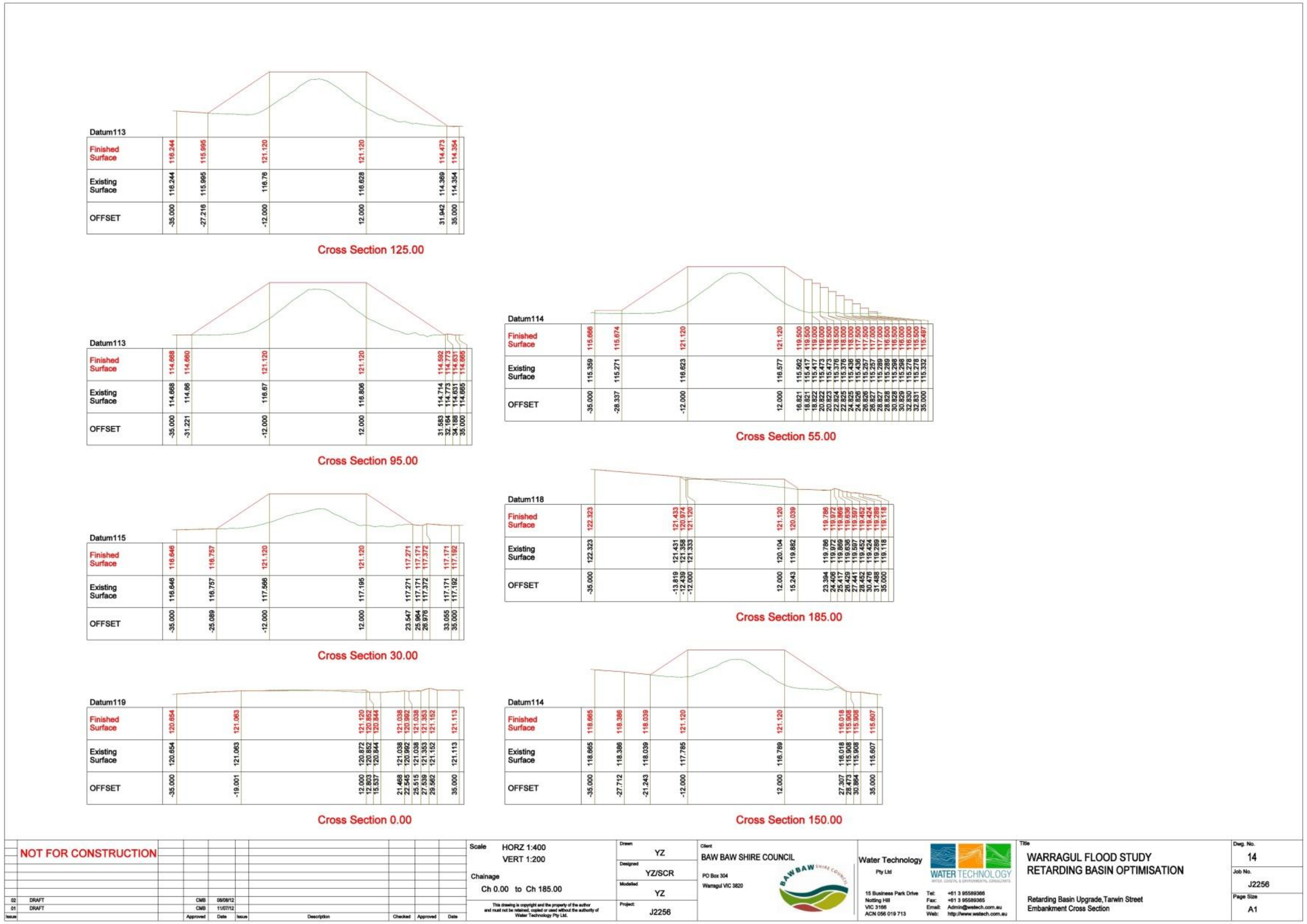


Figure A - 14 Tarwin Street Cross Sections 1/3

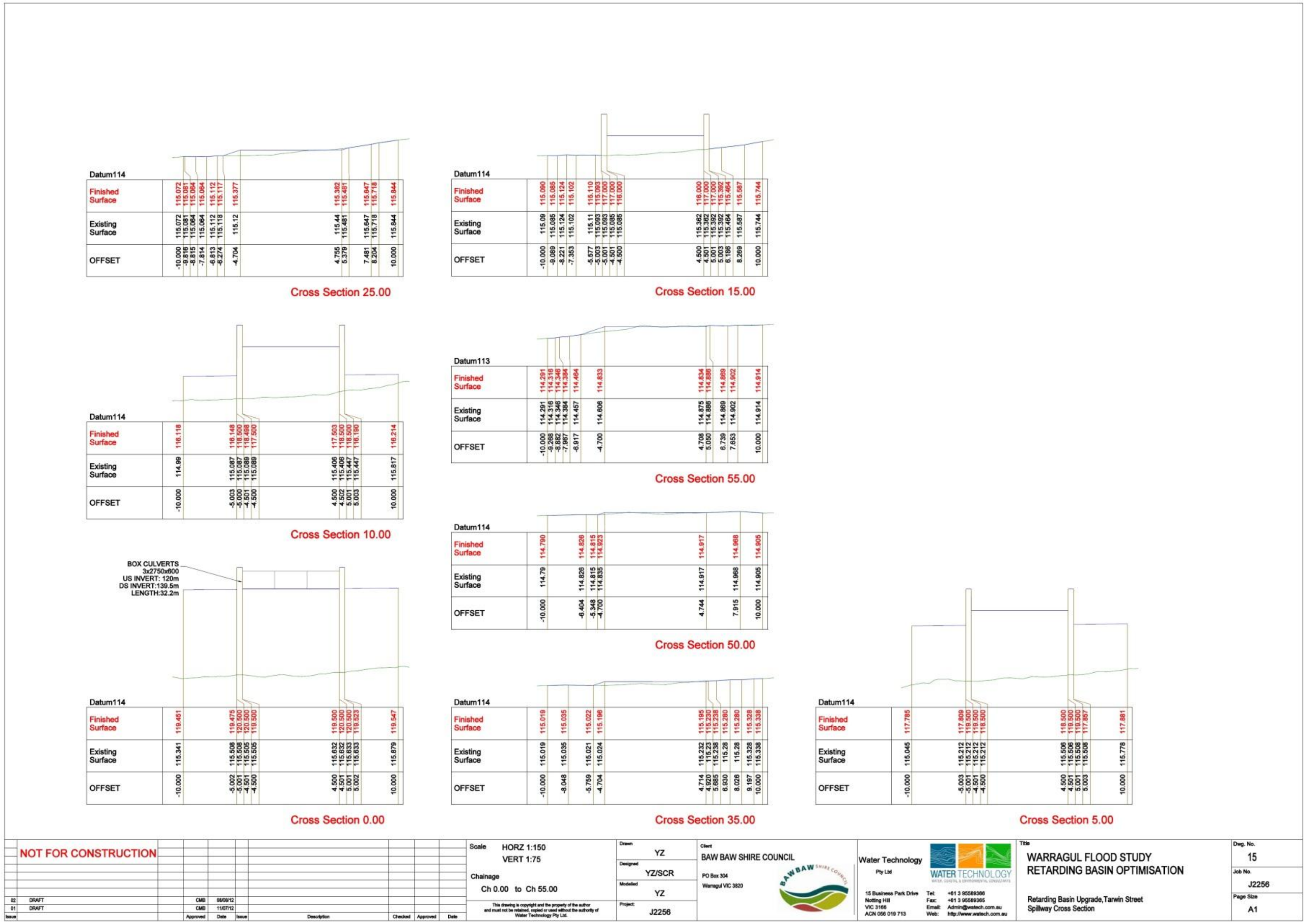


Figure A - 15 Tarwin Street Cross Sections 2/3

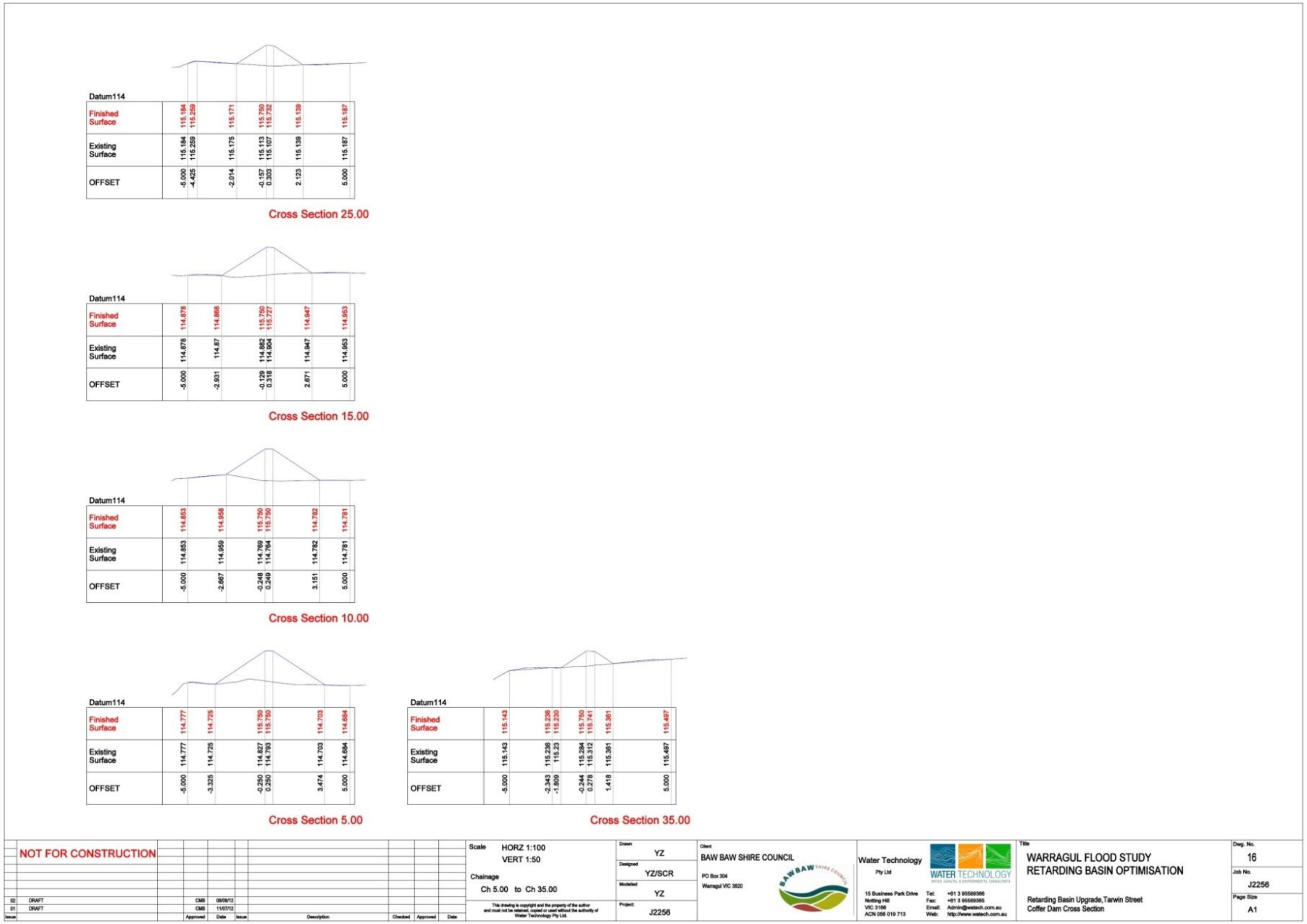


Figure A - 16 Tarwin Street Cross Sections 3/3

Appendix B WARRAGUL TOWNSHIP URBAN FLOOD MAPPING RESULTS

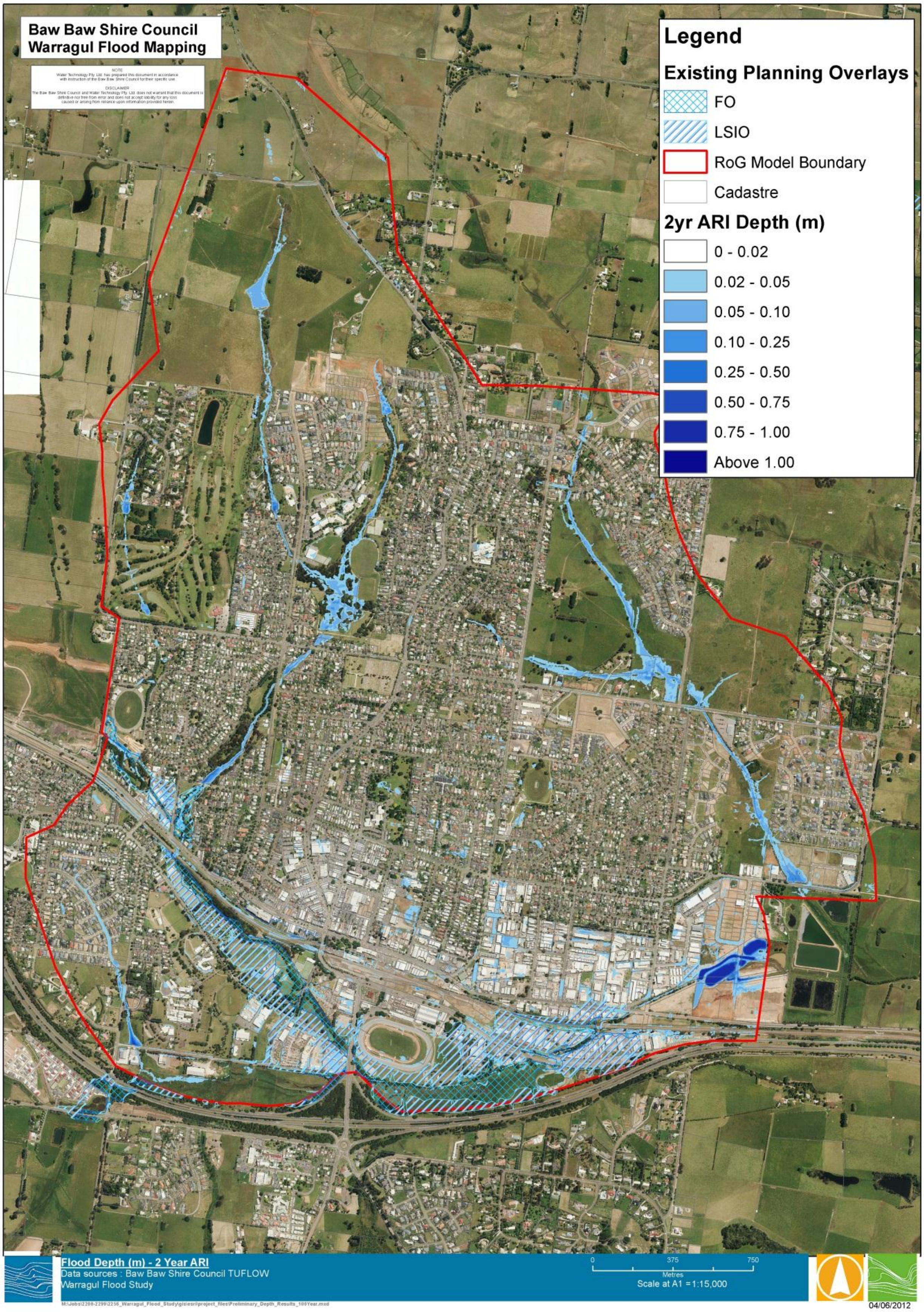


Figure B - 1 2 Year ARI Flood Maximum Depth Plot

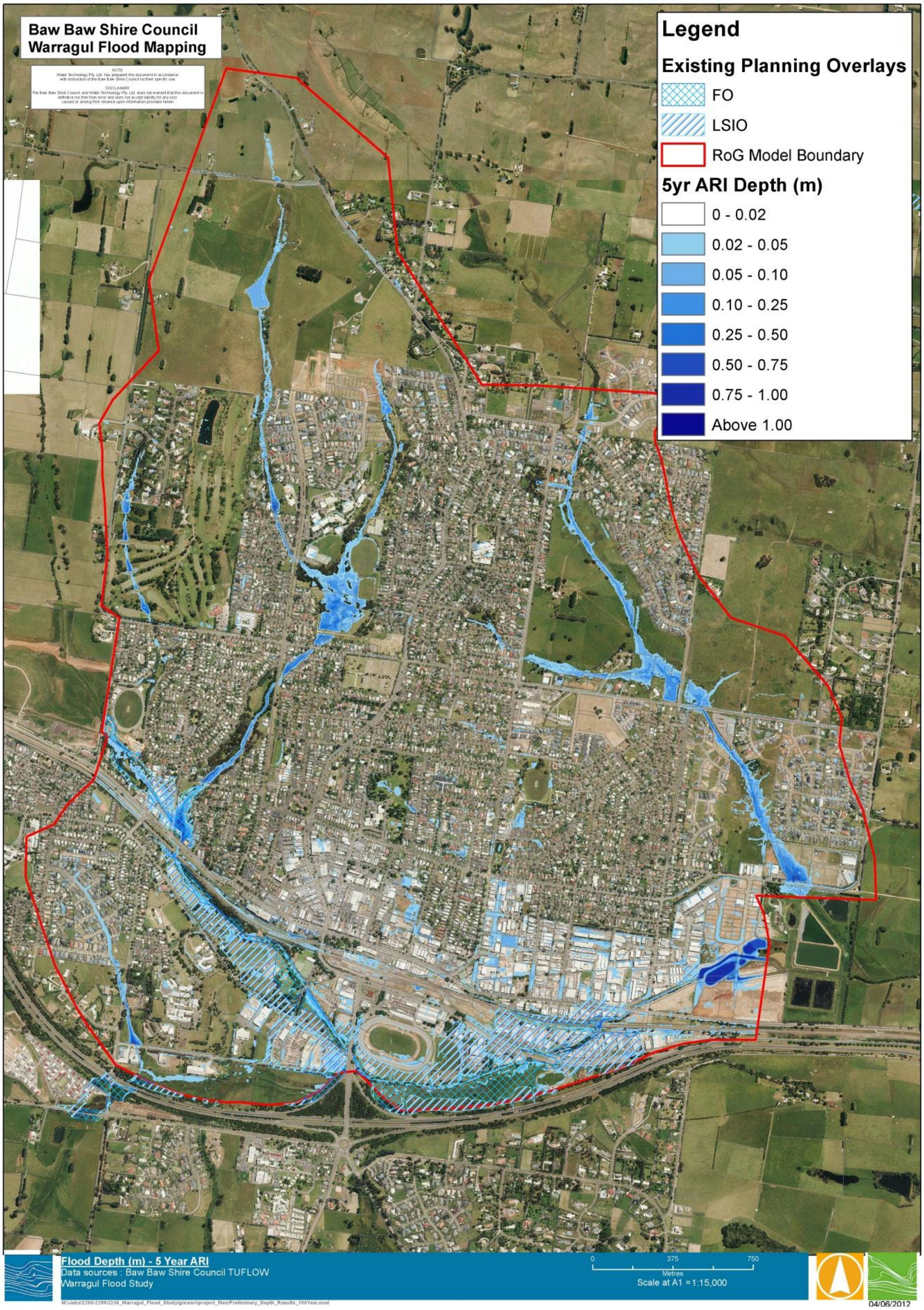


Figure B - 2 5 Year ARI Flood Maximum Depth Plot

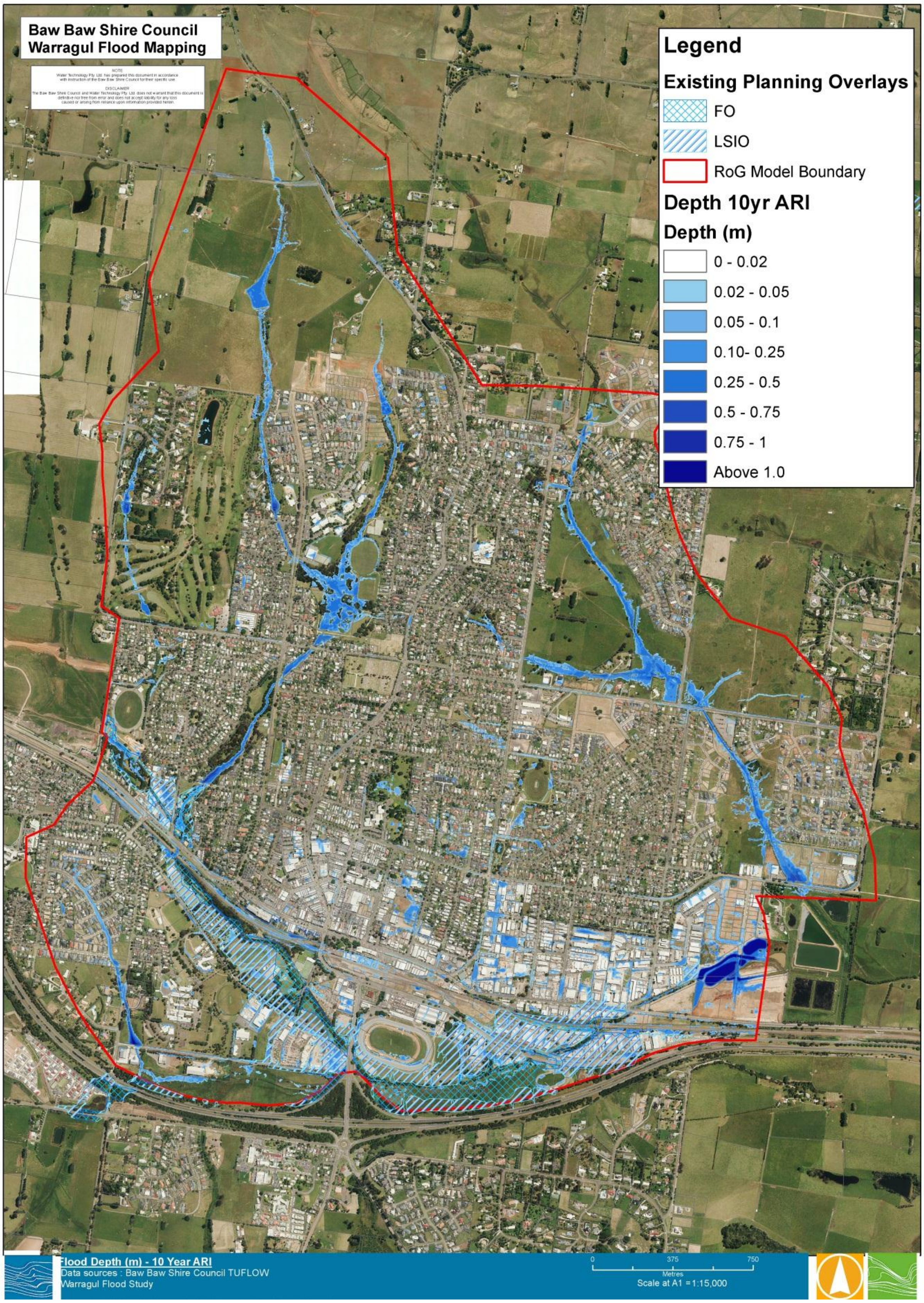


Figure B - 3 10 Year ARI Flood Maximum Depth Plot

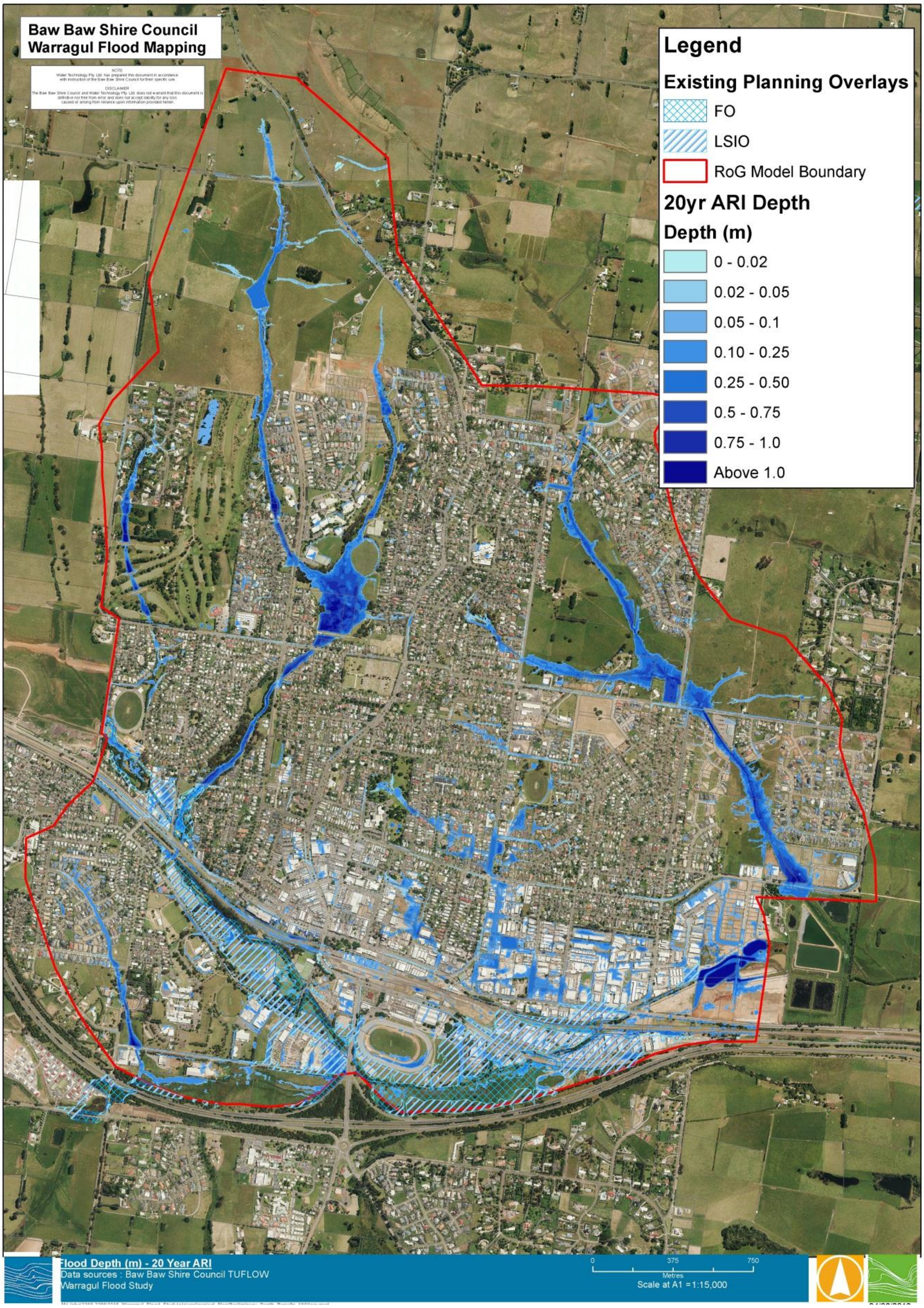


Figure B - 4 20 Year ARI Flood Maximum Depth Plot

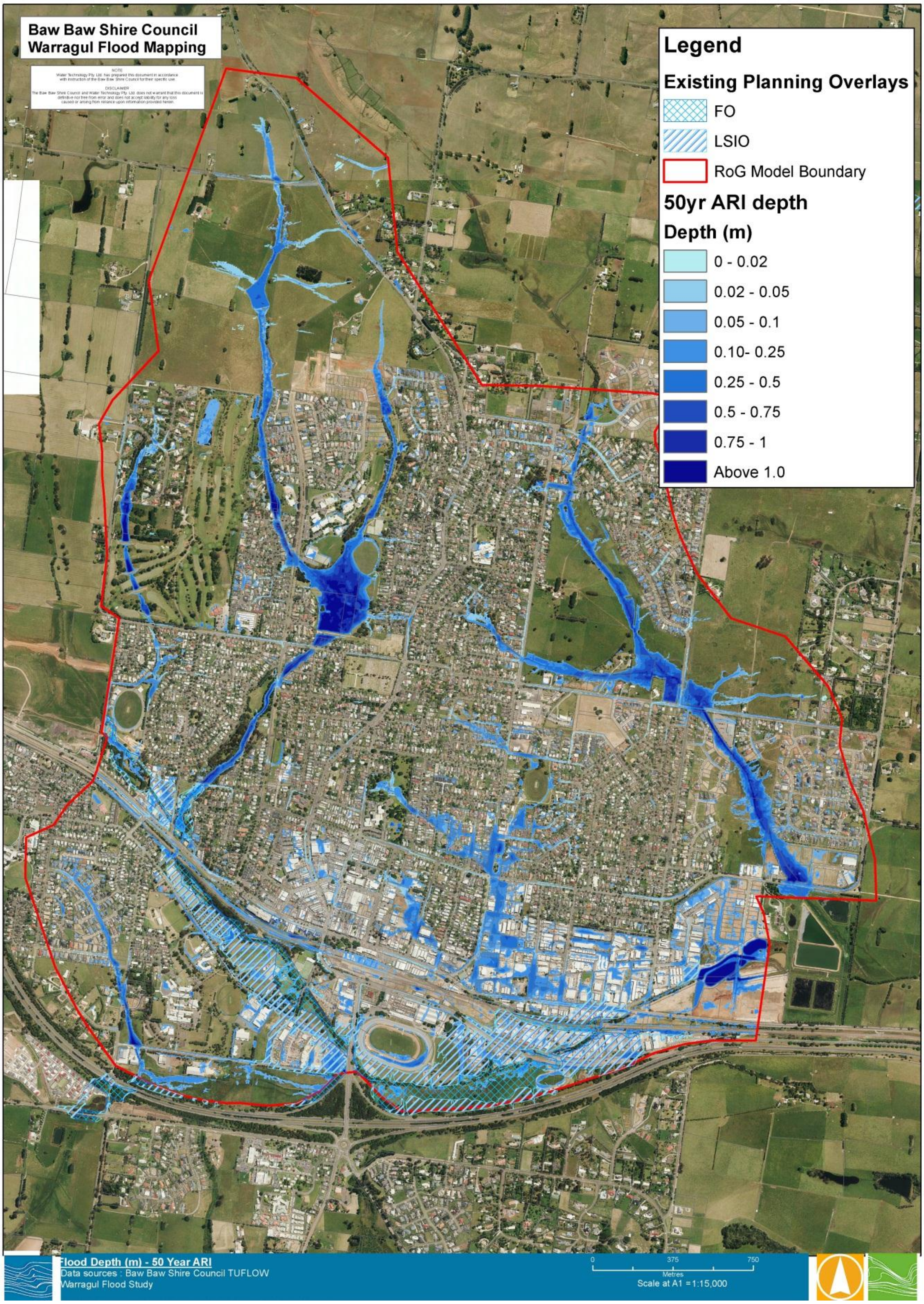


Figure B - 5 50 Year ARI Flood Maximum Depth Plot

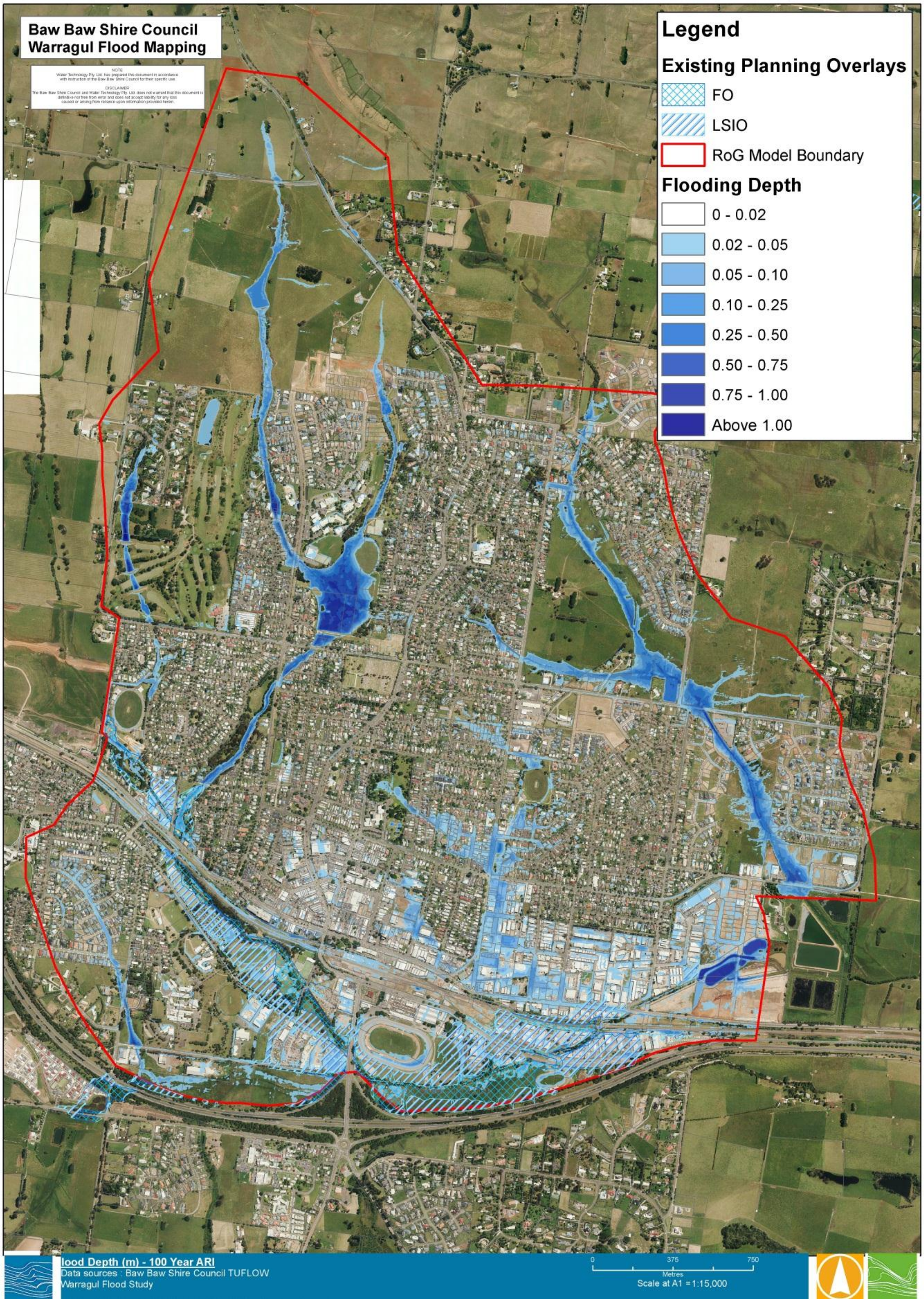


Figure B - 6 100 Year ARI Flood Maximum Depth Plot

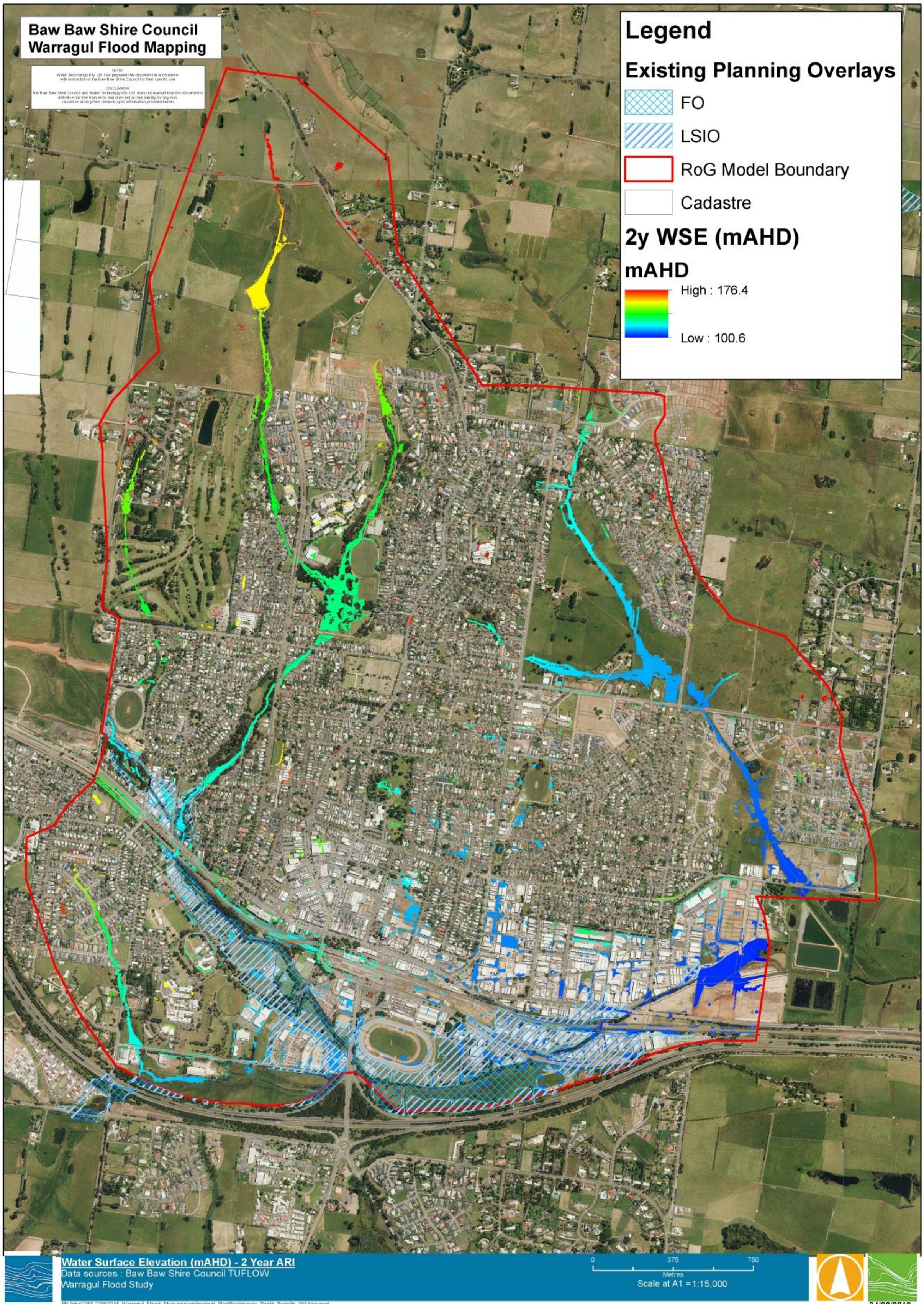


Figure B - 7 2 Year ARI Flood Maximum Water Surface Elevation Plot

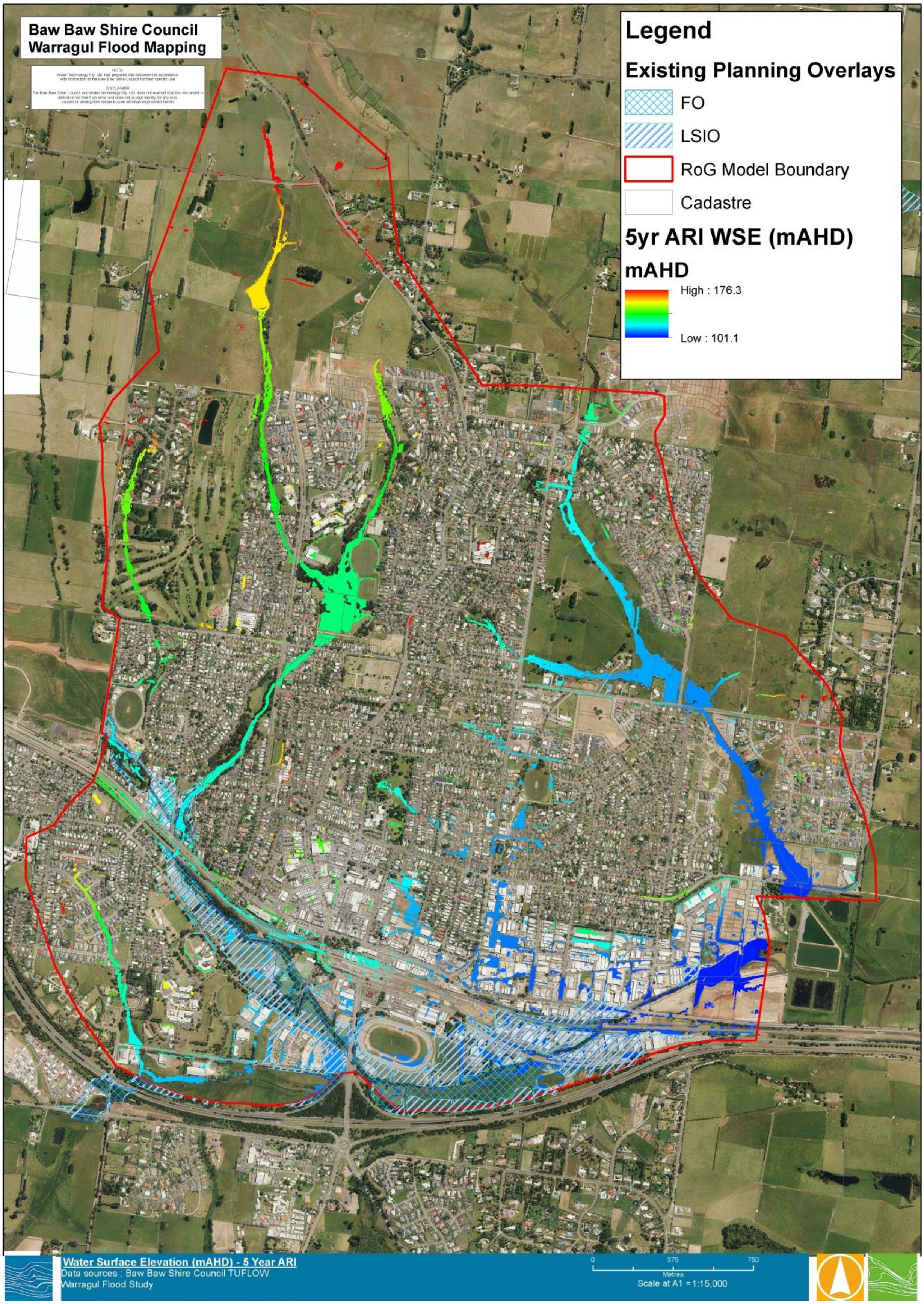


Figure B - 8 5 Year ARI Flood Maximum Water Surface Elevation Plot

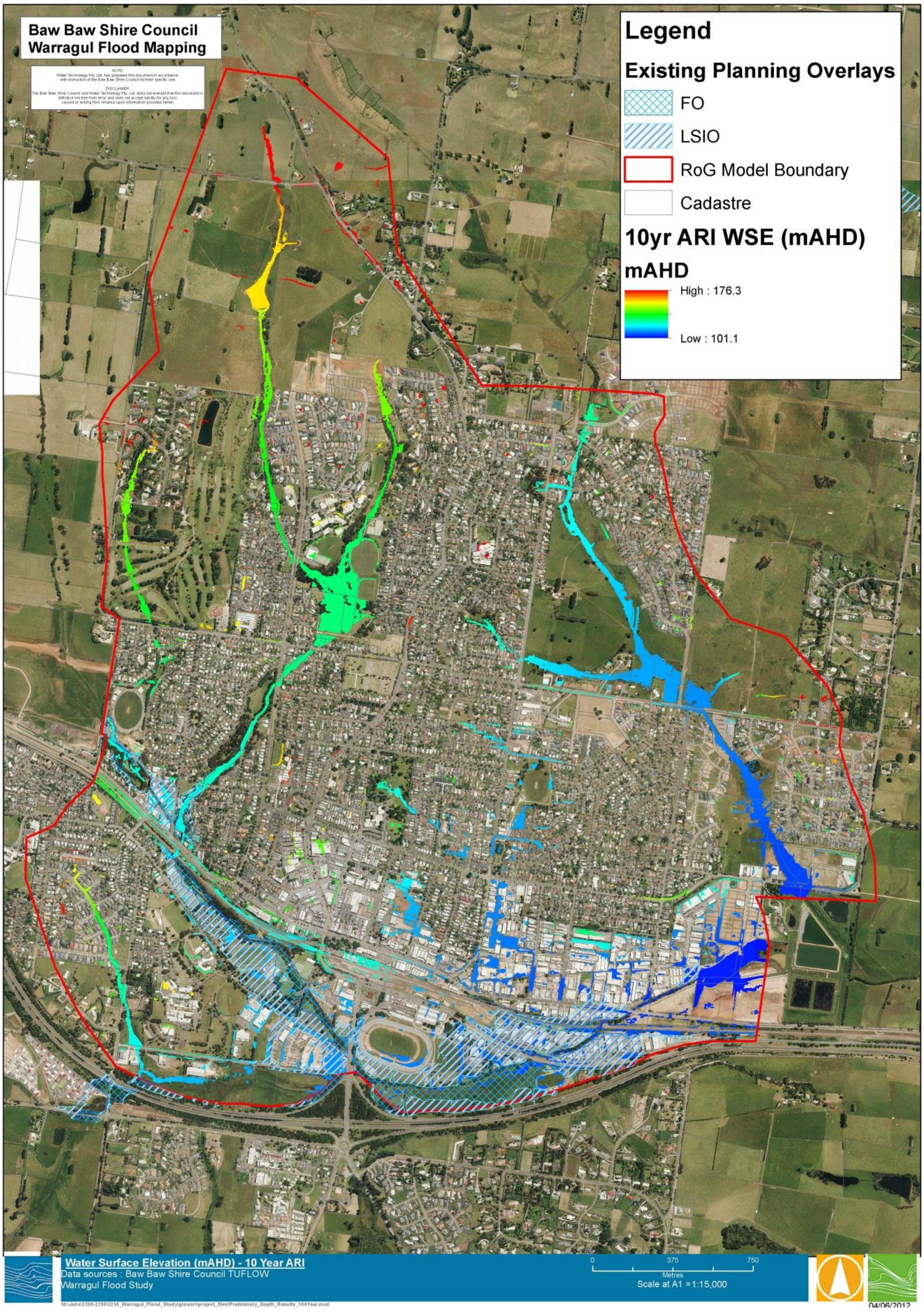


Figure B - 9 **10 Year ARI Flood Maximum Water Surface Elevation Plot**

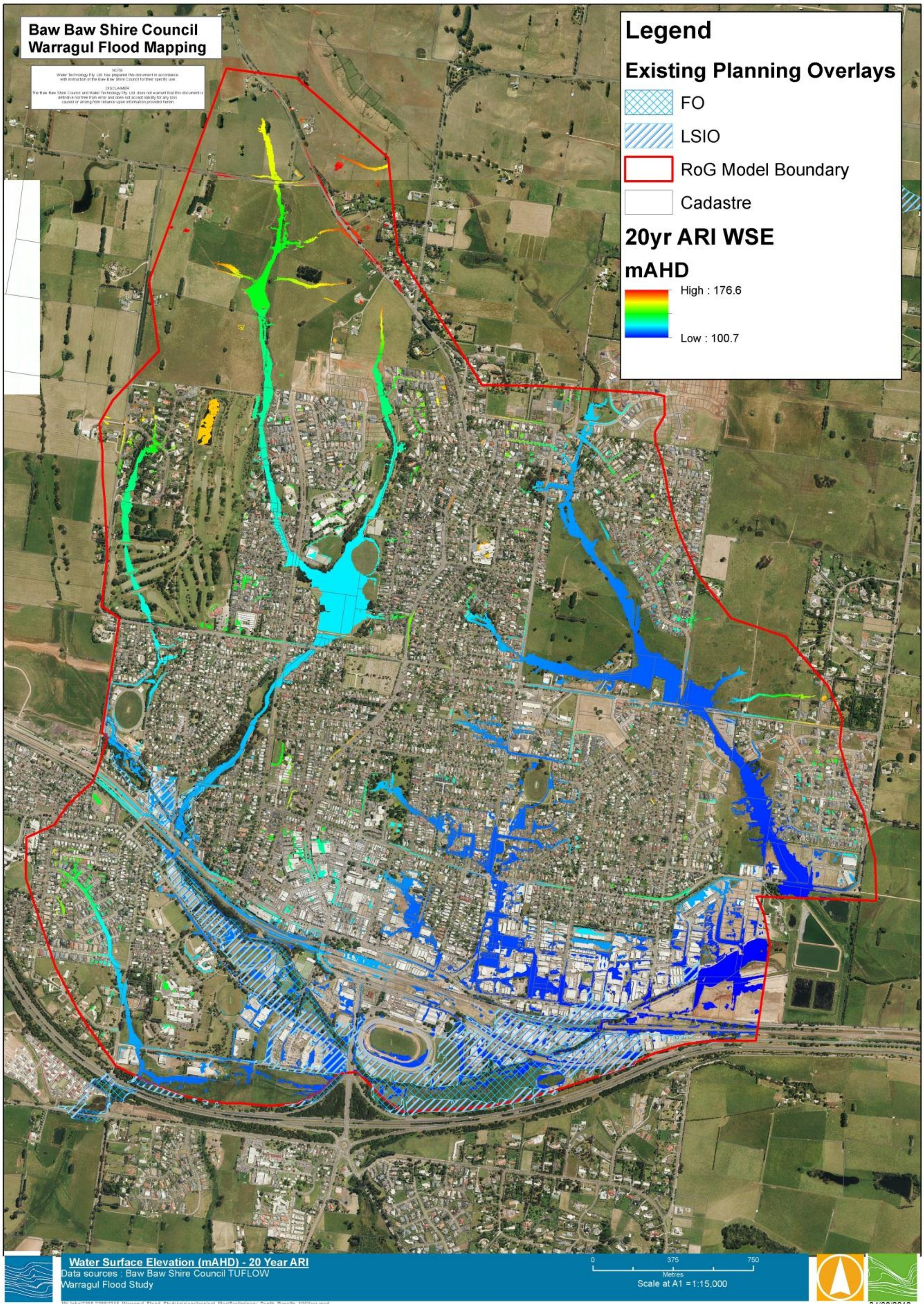


Figure B - 10 20 Year ARI Flood Maximum Water Surface Elevation Plot

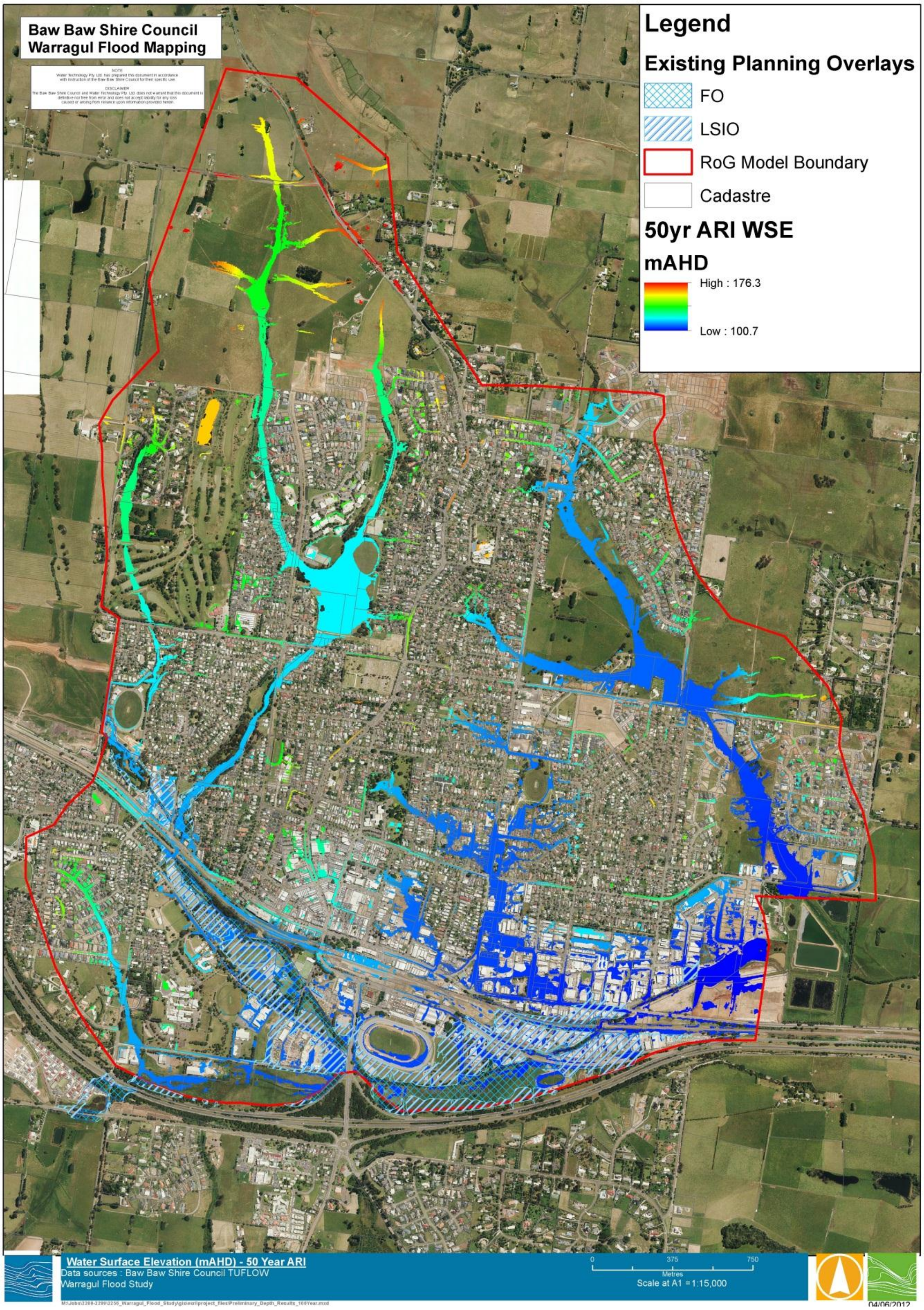


Figure B - 11 50 Year ARI Flood Maximum Water Surface Elevation Plot

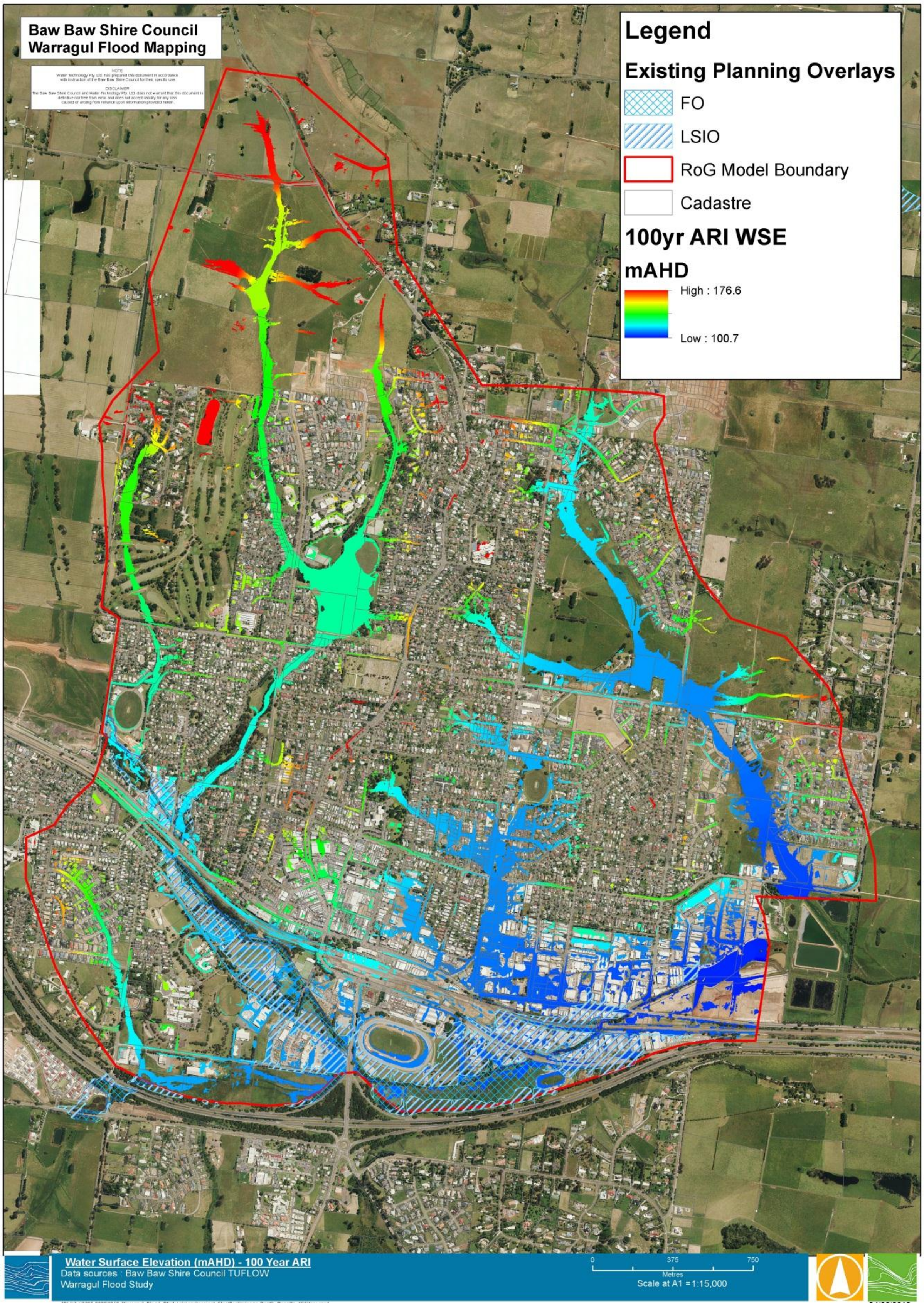


Figure B - 12 100 Year ARI Flood Maximum Water Surface Elevation Plot