# Western Geelong Growth Area Flood Impact Assessment and Stormwater Management Strategy

## Volume 1: Existing Conditions Report

January 2019

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## Project Details

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# 1 INTRODUCTION

## 1.1 Background

The Northern and Western Geelong Growth Areas have evolved from a long-term planning process that was set-out in the G21 Regional Growth Plan (G21 RGP), produced by the Geelong Regional Alliance in April 2013. The RGP and subsequent Implementation Plan Background Report (November 2013), set out a logical planning process for addressing the urban growth requirements for the region to 2050. This included the identification of two broad locations where future urban expansion for Geelong may be considered, these were known as the Northern and Western Further Investigation Areas. These areas are now referred to as the Northern and Western Geelong Growth Areas.

The Northern and Western Geelong Growth Areas Project – Context Report (April 2016) sets out the overall planning process and defines where this project sits within the broader program and how it will provide key information to the IIDP and the Framework Plan. The Context Report will be an important reference point for all parties to understand the overall direction that the City of Greater Geelong (CoGG) seeks to follow through this process. It details the vision, principles, governance arrangements, underlying assumptions and importantly the key steps in developing the IIDP and Framework Plan. This report focuses on the flood impact assessment (under existing conditions) for the Western Growth Area.

## 1.2 Study Area

The Western Geelong Growth Area covers a total area of approximately 3,200 hectares. The subject site is in a favourable position for residential development, encompassing Batesford and the area north-west of Fyansford, in proximity to transport links and with access to the existing Geelong city area. The Western Geelong Growth Area contains several unique features including the extensive former quarry pit and the Moorabool River. Figure 1-1 below shows the extent of the Western Geelong Growth Area. The boundary of the study area is defined to the north by Cowies Creek and to the South by the Barwon River. It extends from the Geelong Ring Road in the east to Dog Rocks Road in the west.

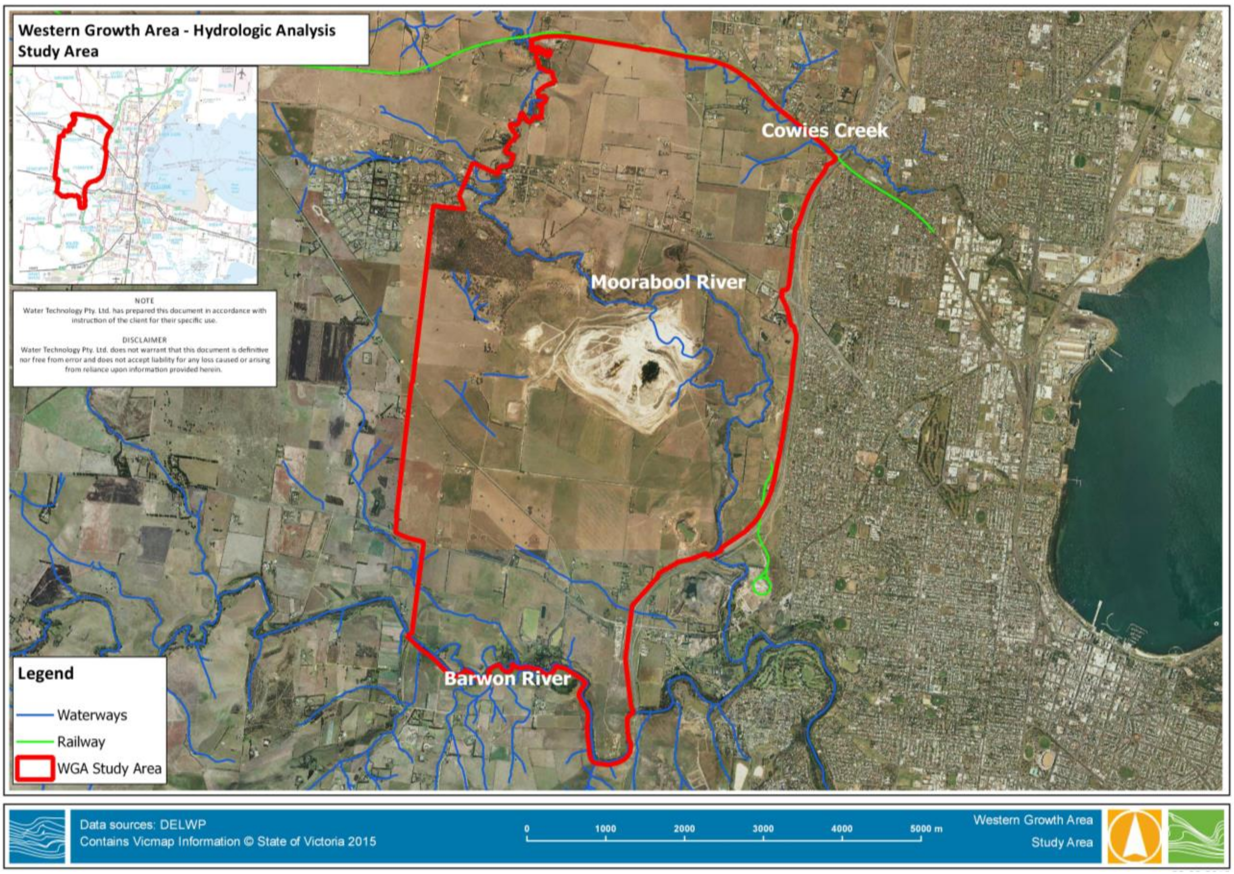


*Figure 1-1 Study Area*

## 1.3 Project Scope

In accordance with the Project Brief (CoGG, May 2016), the objective of the study is to "develop a clear strategy for managing floodwater and stormwater in the Study Area, in the context of a broader Integrated Water Cycle Management (IWCM) approach". We understand that ultimately the aims of this study must fit within the broader vision of the G21 RGP to deliver a:

"…liveable, vibrant and cohesive community..."

It has become well recognised over the last 20 years that the way in which water is managed in the urban environment has a significant impact on the social, economic and environmental values of communities. This has driven the transition to an integrated water cycle management approach and presents a significant opportunity for areas of future urban growth, such as the WGGA, to deliver innovative, more sustainable, and resilient water solutions.

In line with the outlined scope of works from the City of Greater Geelong, the study includes the preparation of three assessments that will inform the Flood Management, Drainage and Stormwater Strategic Plan, these assessments include:

- Flood Impact Assessment;
- Drainage Feasibility Assessment; and
- Stormwater Quality Impact Assessment.

In this first phase, the study focuses on the establishment of existing flood and drainage conditions at the site for the 1% and 10% AEP as well as determination of concept stormwater management options for the area. The study takes into account expectations of both the Corangamite Catchment Management Authority (CCMA) and Council in relation to the development of detailed flooding and drainage assessments. The three Phases of the flooding and stormwater investigations for this study are shown in Figure 1-2.

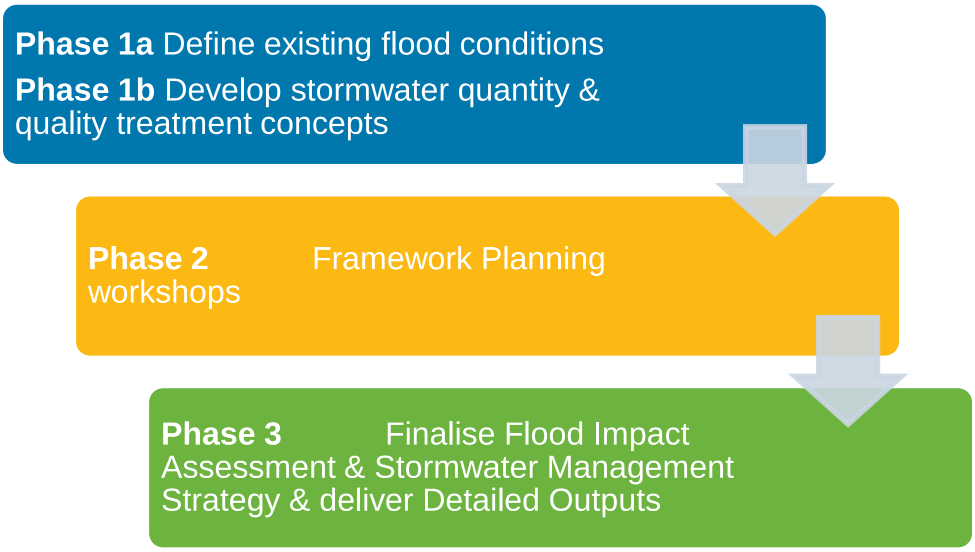


*Figure 1-2 Key Project Phases - Flooding and Stormwater*

## 1.4 Existing Conditions

The Phase 1 of the project includes the development of existing conditions with respect to drainage and flooding. This assessment considers the combined impacts of riverine and stormwater flooding over the study area. The assessment of existing conditions included two main parts, the development of site and catchment hydrological inputs and the development of a hydraulic model which defines the extent of flooding under design flood conditions.

This report presents the development of the catchment hydrology and existing flood extents. It includes the completion of flood frequency analyses for the Barwon River at Pollocksford and the Moorabool River at Batesford, the development and results of three RORB hydrologic models, and the setup and results of three TUFLOW hydraulic models for three catchments covering the study area.

## 1.5 Methodology Overview

Figure 1-3 outlines the adopted process for undertaking the assessment of the existing conditions hydrology and hydraulics of the study area.

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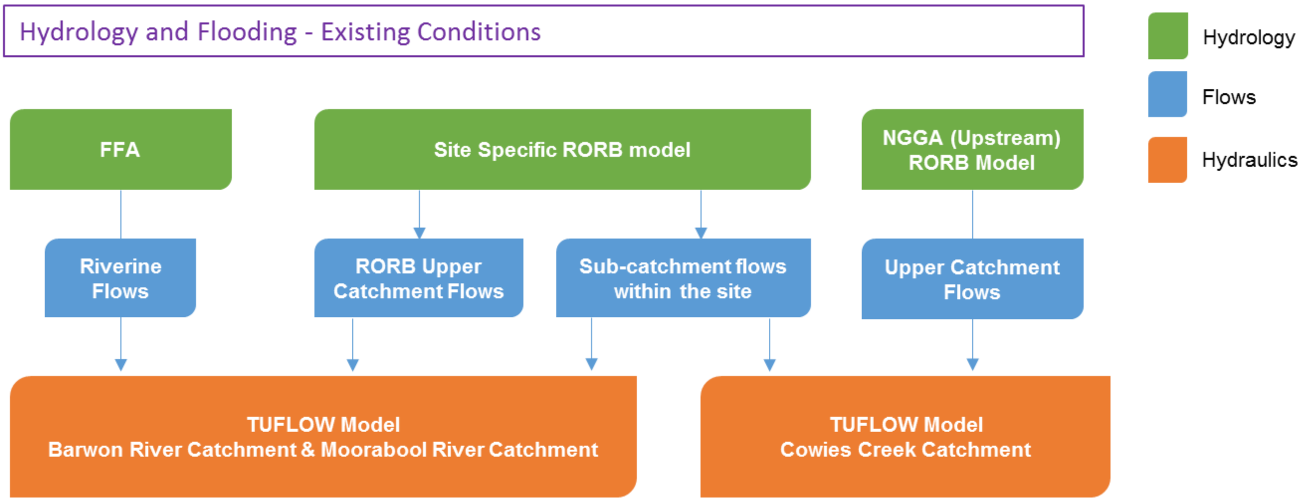


*Figure 1-3 Adopted Process for Developing Hydrology and Flooding - Existing Conditions*

The main tasks related to the site hydrology and hydraulics are described step-by-step below and reference both the Melbourne Water Guidelines and Technical Specifications for Flood Mapping Projects (2016)[1] and the RORB manual (2010)[2].

1- Setup of undiverted RORB models
   a. RORB models were developed to replicate catchment assumptions in the VicRoads Rational Method (fraction impervious = 0.05, no storages or diversions and reach type = natural).
2- Reconciliation of RORB peak design flows to VicRoads Rational Method
   a. A selected range of design flows for differing catchment sizes have been plotted against the VicRoads Rational Method results. RORB's Kc parameter was used to find the best fit across the catchment within a ±30% range in peak flow.
3- Setup of diverted RORB models
   a. The RORB models were updated to include the fraction impervious values representative of existing conditions as well as reach types and any significant storages (such as areas behind road embankments). Kc:Dav ratios were maintained to preserve the reconciliation to the VicRoads Rational Method peak flow estimates.
   b. The diverted model RORB has two functions:
      i. Provide inflows to the TUFLOW hydraulic model for Stage 1 existing conditions flood modelling. Print locations will be added at all required inflow locations as well as points of comparison to the TUFLOW modelling.
      ii. Enable the modelling of developed conditions to determine the location and sizing of basins and other water management assets in Stage 2.

[1]Melbourne Water, 2016. Melbourne Water Specifications for Flood Mapping Projects.
[2] Laurenson, E. M. and Russell, G. M. and Nathan, R.J., 2010. RORB Version 6. Runoff Routing Program: User Manual. Monash University Department of Civil Engineering, 1990 - Flood dams and reservoirs.

4- Setup of TUFLOW models with inflows from the RORB models.

    a. Inflows were added to the hydraulic models from the existing conditions diverted RORB models via TUFLOW SA Polygons. That is, distributed at inflow locations in the upstream sections of each waterway catchment.

5- Comparison of RORB and TUFLOW modelled flows and recommendations.

    a. Comparisons between RORB and TUFLOW results have been made at pre-determined locations throughout each of the catchments. Where flows do not match, explanations are provided or recommendations given for a way forward for the developed conditions assessment.

# 2 AVAILABLE DATA

## 2.1 Overview

The following sub-sections describe the data that was used as inputs to the study, including previous report and base terrain and hydrologic data.

## 2.2 Previous Studies and Investigations

The following previous studies were reviewed to provide background information and context to the study:

- A series of hydrologic and hydraulic investigations undertaken for the proposed Geelong Bypass Project Planning Study:
  - Geelong Bypass - Hydrology and Hydraulics- Stage 1 Assessment. Section 1 – Princes Freeway, Corio Interchange to Midland Highway by Water Technology and Neil Craigie, 2004.
  - Geelong Bypass - Hydrology and Hydraulics- Stage 1 Assessment, Section 2 – Midland Highway to Hamilton Highway by Water Technology and Neil Craigie, 2004.
  - Geelong Bypass - Hydrology and Hydraulics- Stage 1 Assessment Section 3 – Hamilton Highway to Surf Coast Highway by Water Technology and Neil Craigie, 2004.
- Bell Post Hill, Geelong - Pre-Feasibility Report on the Potential for Future Residential Development by Spiire, 2013.
- Batesford Quarry Groundwater Level Recovery Investigation by Nolan-ITU, 2002.
- Batesford Quarry Rehabilitation - Water Quality Study by Nolan-ITU, 2005.

## 2.3 GIS data

City of Greater Geelong provided the following georeferenced spatial data:

- Study area boundary
- Cadastre (property parcels, road reserves and easements)
- Drainage pipe and pit alignments
- Drainage - open drain alignments
- Roads
- Waterways
- Contours (0.5 m, derived from LiDAR)

The study team also utilised data provided from DELWP, including the full VicMap dataset and LiDAR data.

## 2.4 Digital Elevation Model

A digital elevation model (DEM) was developed from the available LiDAR survey with key features such as diversions, the quarry and main river channel specifically incorporated into the DEM. CoGG have also provided updated 1 metre horizontal resolution LiDAR data for the study area. Figure 2-1 shows the extent of the available LiDAR information. The LiDAR data was captured in 2014/15 and is hence reasonably current. LiDAR typically has a vertical accuracy of 10-20 cm which is appropriate for the hydrologic and hydraulic modelling being undertaken for this study.

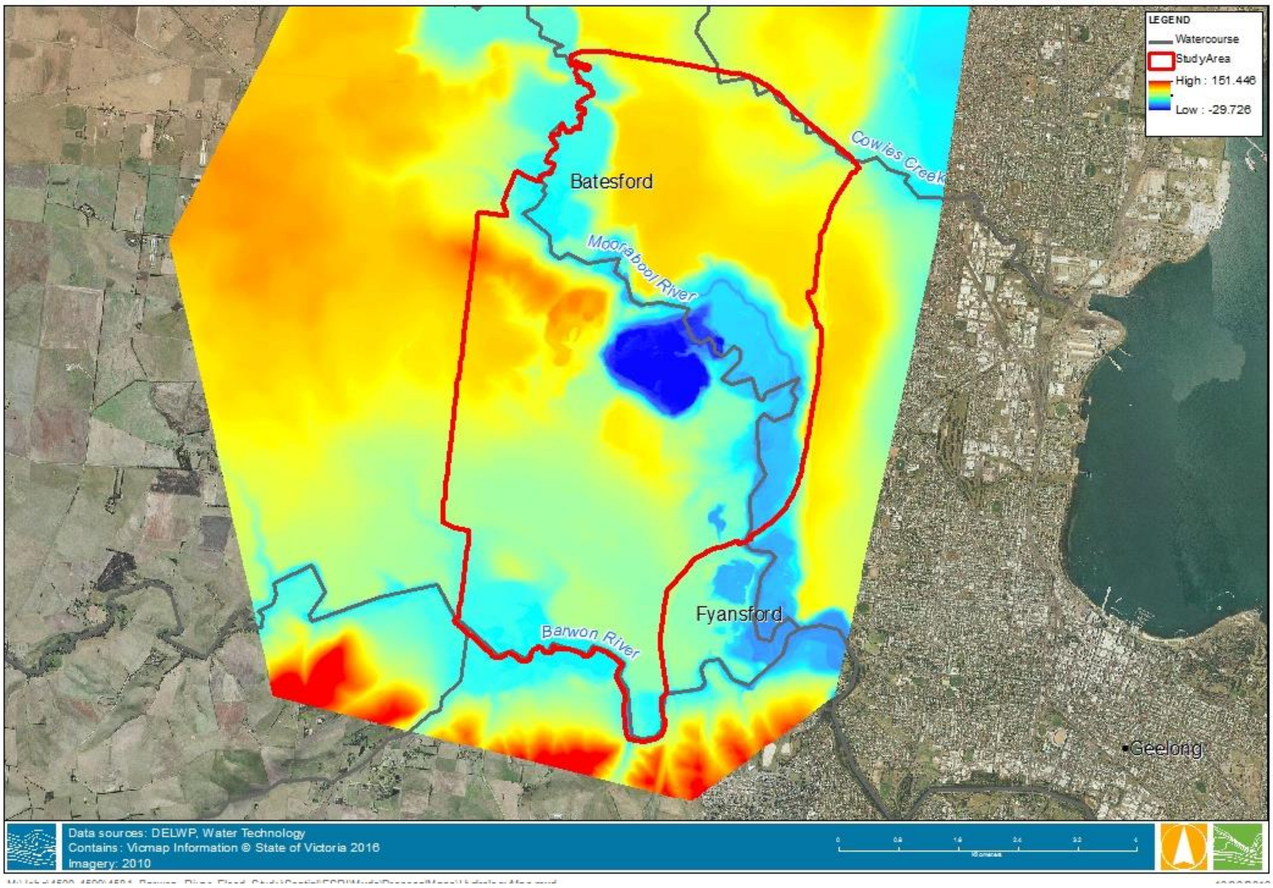


*Figure 2-1 Available LiDAR Information*

## 2.5 Structures

The data collection stage included collation of information on hydraulic structures across the study area. The results from draft hydraulic modelling indicated a number of structures appeared to be missing in the model. Additional structure information was then gathered from the City of Greater Geelong and VicRoads. It is noted that both organisations pointed to uncertainty in the accuracy of most of the provided data. Subsequently, information was checked and verified by the study team during several site visits. The details of the inspected structures were checked and incorporated into the existing conditions modelling. Figure 2-2 shows the location of hydraulic structures incorporated into the existing conditions modelling.

Figure 2-3 presents the location and characteristics of the key structures specifically around McCanns Lane and Hamilton Highway.

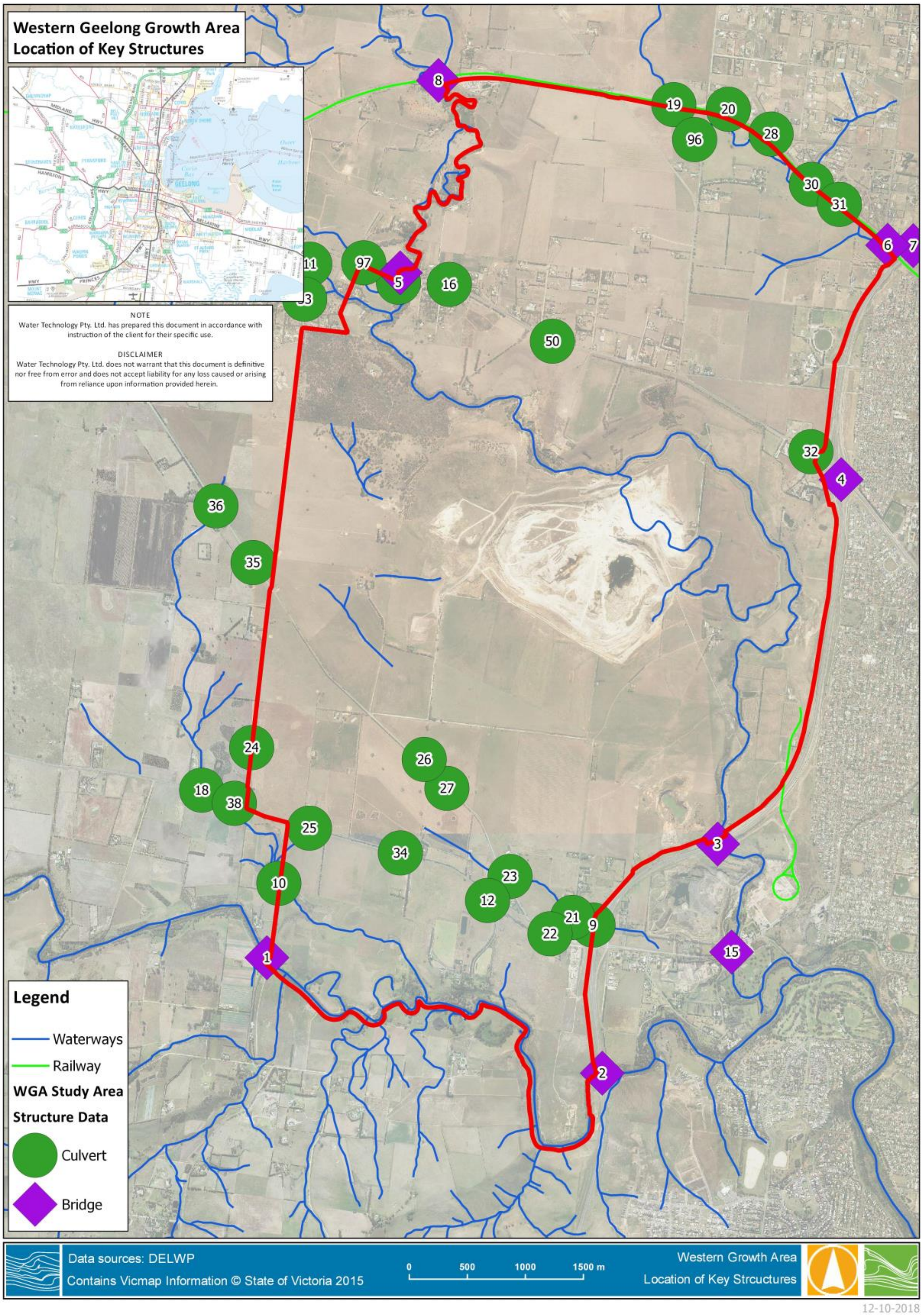


*Figure 2-2 Location of Key Structures*

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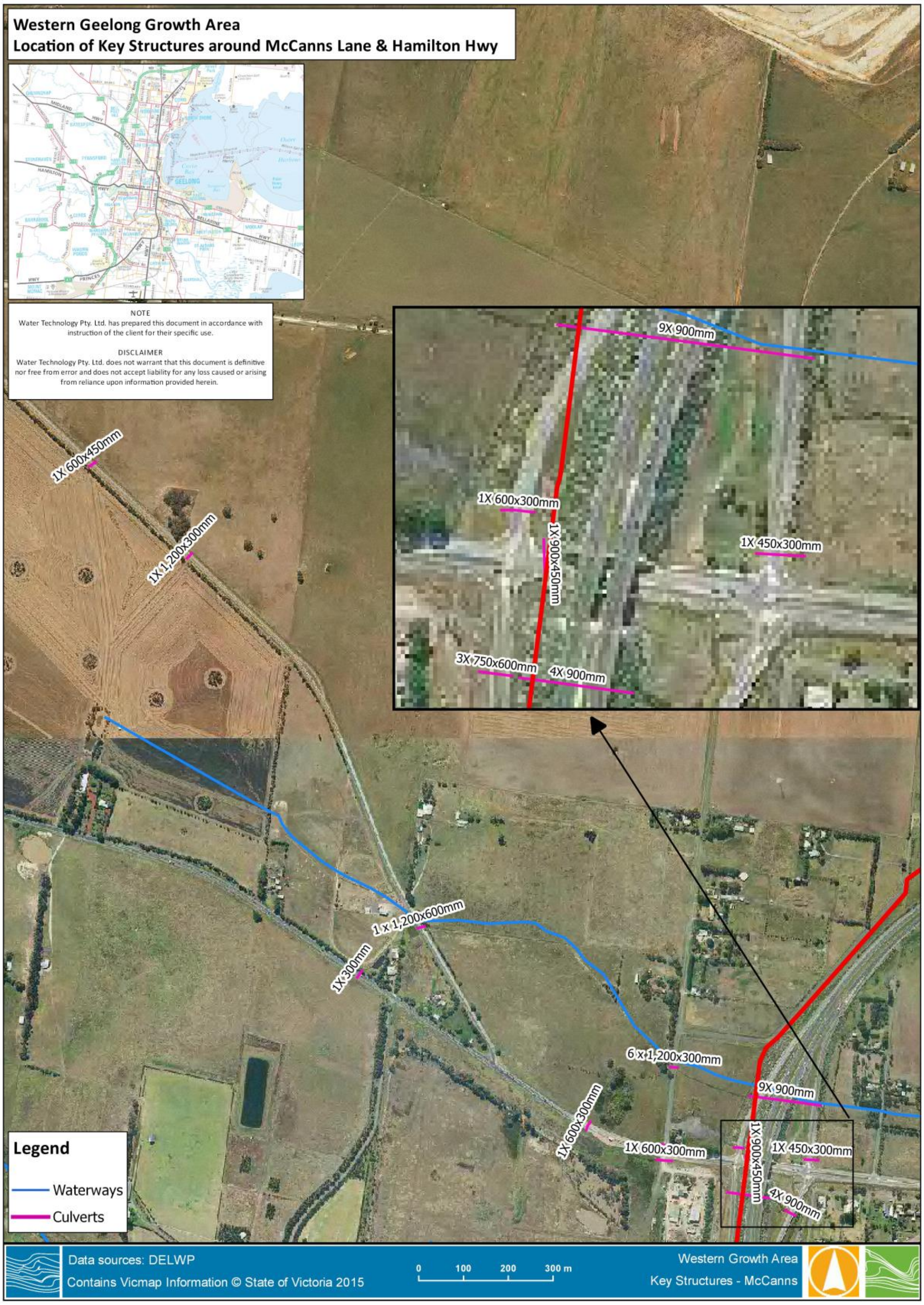


*Figure 2-3 Location of Key Structures around McCanns Lane and Hamilton Highway*

# 3 RIVERINE FLOWS

The catchment and waterways draining through and within the site are complex. This includes the main waterways of the Moorabool River, Barwon River and Cowies Creek. There are numerous small tributary catchments that flow into these waterways. In addition, there are some small catchments that flow directly to the existing quarry pit on the site. The approach to defining design hydrology for each of the main waterways as well as the smaller internal catchments, varied based on the availability of information.

Good streamflow data was available for the Moorabool River at Batesford gauge (232202) immediately upstream of the study area and for the Barwon River at Pollocksford gauge (233200). The data available for these gauges was used as the basis of flood frequency analysis (FFA) to derive design flow estimates.

The FFA used the FLIKE software package, which is the current industry standard tool for this analysis. FLIKE was applied using the peak above threshold method for ungauged events, with anecdotal information and low flow filtering.

A flood volume FFA was also undertaken using the available data for gauged floods. The average flood event duration was determined and, based on this, the annual series of flood volumes across the available record was calculated. For the peak flow analysis, flood durations and volumes for a range of events were assessed.

Water Technology is also undertaking the Barwon and Moorabool Flood Study on behalf of the Corangamite CMA. This project covers an expansive study area which includes the Western Growth Area. The timelines for this study are such that the hydrological assessment completed as part of the study is not yet available for inclusion as part of the Phase 1.

The peak flood estimates determined for the Barwon and Moorabool rivers in this study are considered appropriate for the current planning process. It is considered that the overall flood extents on the Moorabool and Barwon floodplains, within the study area, will not be significantly sensitive to modest changes in peak design flows, as the floodplains are relatively well defined through the study area. Any future changes that may occur in main river flood extents will not impact on the development and specification of site stormwater management options.

## 3.1 Flood Frequency Analysis

The period of gauge record at the Barwon River (Pollocksford) and Moorabool River (Batesford) gauges were used to develop a flood frequency analysis for the estimate of peak design flood flows. The FFA had the following key characteristics:

- FFA for the Barwon River at Pollocksford gauge was based on:
  - Maximum instantaneous flows to determine the peak annual flow series.
  - Four-day accumulated volume to determine design event volumes.
- FFA for the Moorabool River at Batesford gauge was based on:
  - Maximum instantaneous flows to determine the peak annual flow series.
  - Four-day accumulated volume to determine design event volumes.
- A ratio of design event peak flow to design event four-day volume was determined for the two assessments.
- For each river a historic event was chosen with a “peak flow to four-day volume ratio” similar to that determined across the design events. This was then used as the basis for the design flood hydrograph shape.

Design hydrographs were developed based on peak flows and relevant flood volumes estimated from the FFA.

A comparison of the FFA results from various distributions of raw and censored annual series was undertaken and the LP3 determined to provide the best fit for both gauge records. The adopted peak flows and associated confidence limits from this assessment are shown in Table 3-1 and Table 3-2. A detailed description of the FFA, available data and the distribution outputs is provided in Appendix A.

*Table 3-1 Adopted Peak Flows – Barwon River at Pollocksford*

| Design Event (AEP) | Log Pearson III with Low Flow Censoring (m³/s) | | |
|---|---|---|---|
| | Peak Flow | Lower Confidence Limit | Upper Confidence Limit |
| 20% | 262.4 | 190.0 | 373.9 |
| 10% | 415.5 | 299.7 | 617.9 |
| 5% | 586.9 | 413.1 | 921.8 |
| 2% | 835.8 | 562.6 | 1,502.4 |
| 1% | 1,037.1 | 666.9 | 2,074.3 |

*Table 3-2 Adopted Peak Flows – Moorabool River at Batesford*

| Design Event (AEP) | Log Pearson III with Low Flow Censoring (m³/s) | | |
|---|---|---|---|
| | Peak Flow | Lower Confidence Limit | Upper Confidence Limit |
| 20% | 116.5 | 81.4 | 177.4 |
| 10% | 196.0 | 138.5 | 292.4 |
| 5% | 280.8 | 198.9 | 421.7 |
| 2% | 392.7 | 273.2 | 641.2 |
| 1% | 473.1 | 321.3 | 912.2 |

# 4 CATCHMENT HYDROLOGY

## 4.1 Overview

RORB models were developed for the internal catchments draining to the Moorabool River, Barwon River and Cowies Creek. This included the catchments adjoining the site such as C236 (Barwon Catchment). Further comparison to previous investigations on Cowies Creek was undertaken. Previous assessments at this location were undertaken for the Geelong Ring Road and downstream development investigations (east of the Ring Road).

RORB is a non-linear rainfall runoff and streamflow routing model for calculation of flow hydrographs in drainage and stream networks. The model requires catchments to be divided into subareas, connected by a series of conceptual reach storages. Observed or design storm rainfall is input to the centroid of each subarea. Specific losses are then deducted, and the excess routed through the reach network.

The primary aims of the RORB analysis undertaken for this Stage of the project included:

- Determine design event peak flow and hydrograph shapes for input to the hydraulic model at the determined model boundaries and internal catchment inflows at each subarea. Design events include the 1%, 2%, 5%, 10% and 20% AEP.
- Comparison of the undiverted RORB derived flow with the VicRoads Rational Method.
- Comparison of the validated and diverted RORB existing conditions flows with those generated from the TUFLOW modelling.

Design hydrographs were developed based on peak flows and relevant flood volumes estimated from the FFA analysis and the RORB models. These peak flows were compared to each other and to alternative hydrology estimate methods.

## 4.2 Hydrology Undiverted RORB Model Setup

The RORB models were developed using MiRORB (MapInfo RORB tools), RORB GUI and RORBWIN V6.15.

Catchment and sub-catchment delineation was undertaken using ArcHydro software (a plugin to ESRI ArcMAP) with further manual alterations made by the modeller where required and through consultation with Council. Figure 4-1 shows the delineation of the three waterway catchment areas within the study area. It can be seen that the Moorabool RORB model is the largest of the 3 areas.

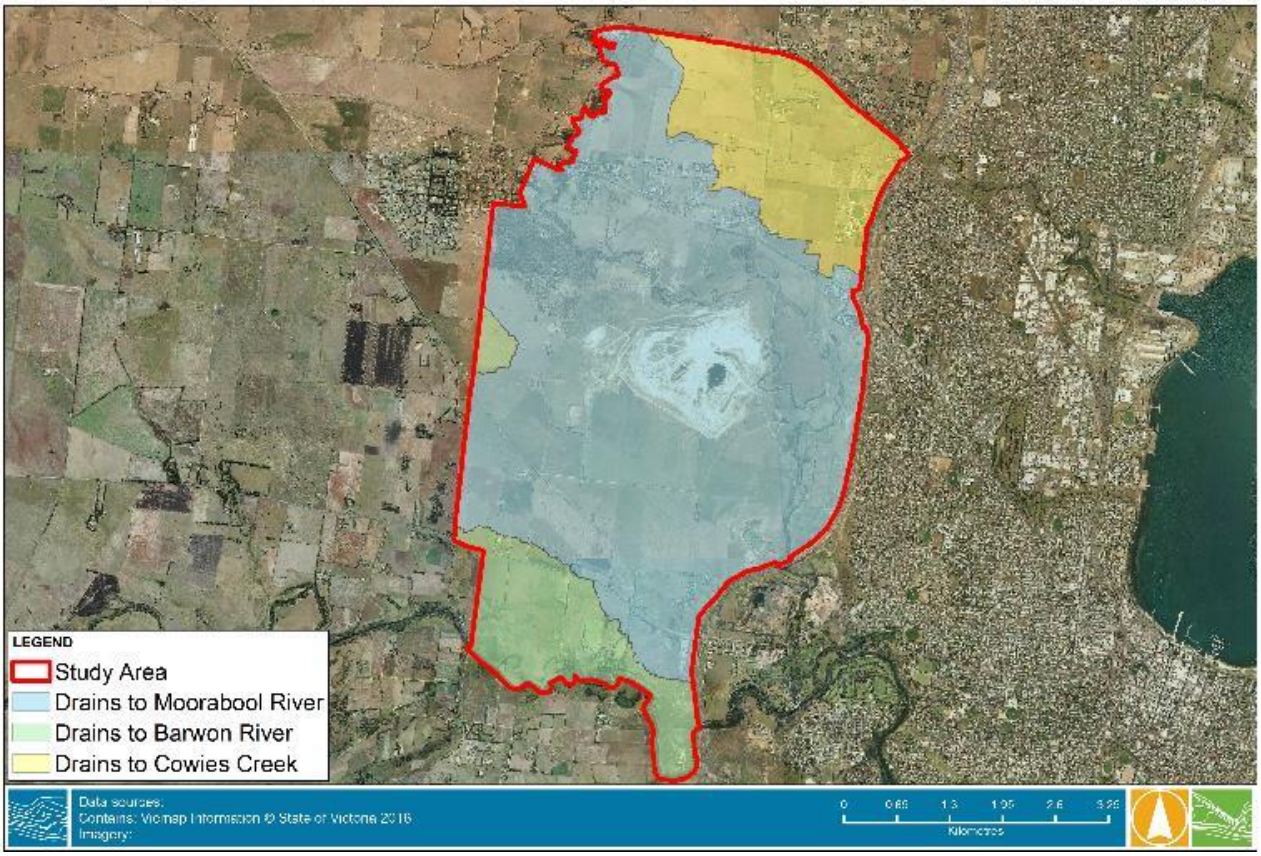


*Figure 4-1 Study Area Drainage Delineation*

### 4.2.1 Sub-area and Reach Delineation

The delineation of the subareas and reaches was refined by visual assessment of the terrain (1 m LiDAR) and further revised based on feedback from Council.

In some instances, where small sub-catchments drain directly to a main river, the catchment was defined with only one subarea. While this is not strictly in accordance with best practice, it was adopted as the estimated catchment flows in such areas are minimal compared to the river flows and routing through a series of sub-catchment would have little impact.

Nodes were placed at areas of interest, the centroid of each sub-area and the junction of any two reaches. Nodes were then connected by RORB reaches, each representing the length, slope and reach type.

To be consistent with the VicRoads Rational Method, an undiverted model based on the natural conditions of the site was built for model reconciliation. The undiverted RORB model has all reach type set to Type '1 – Natural'.

A view of the undiverted RORB model set is presented in Figure 4-2, Figure 4-3 and Figure 4-4.

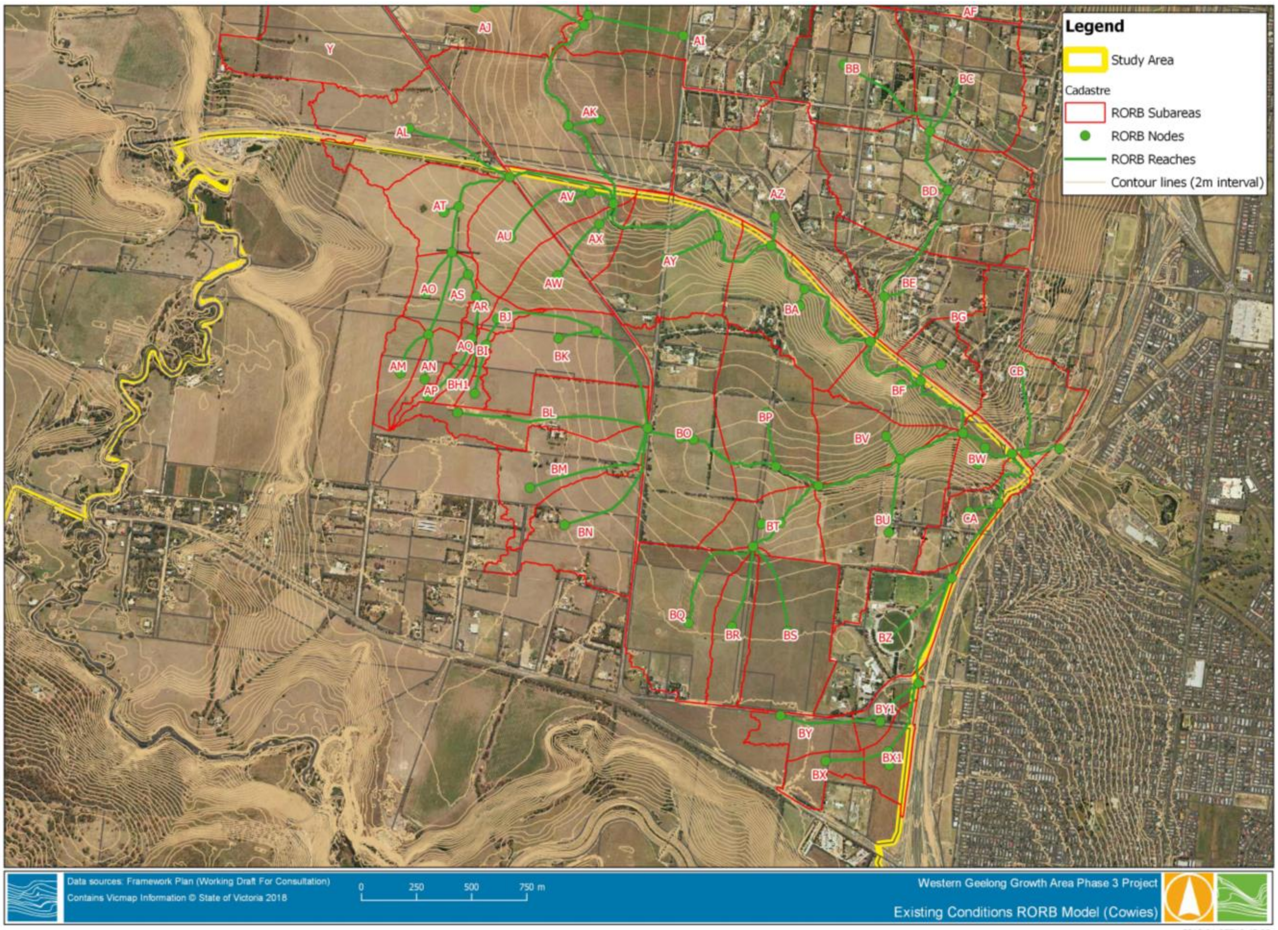


*Figure 4-2 Cowies Creek undiverted RORB Model*

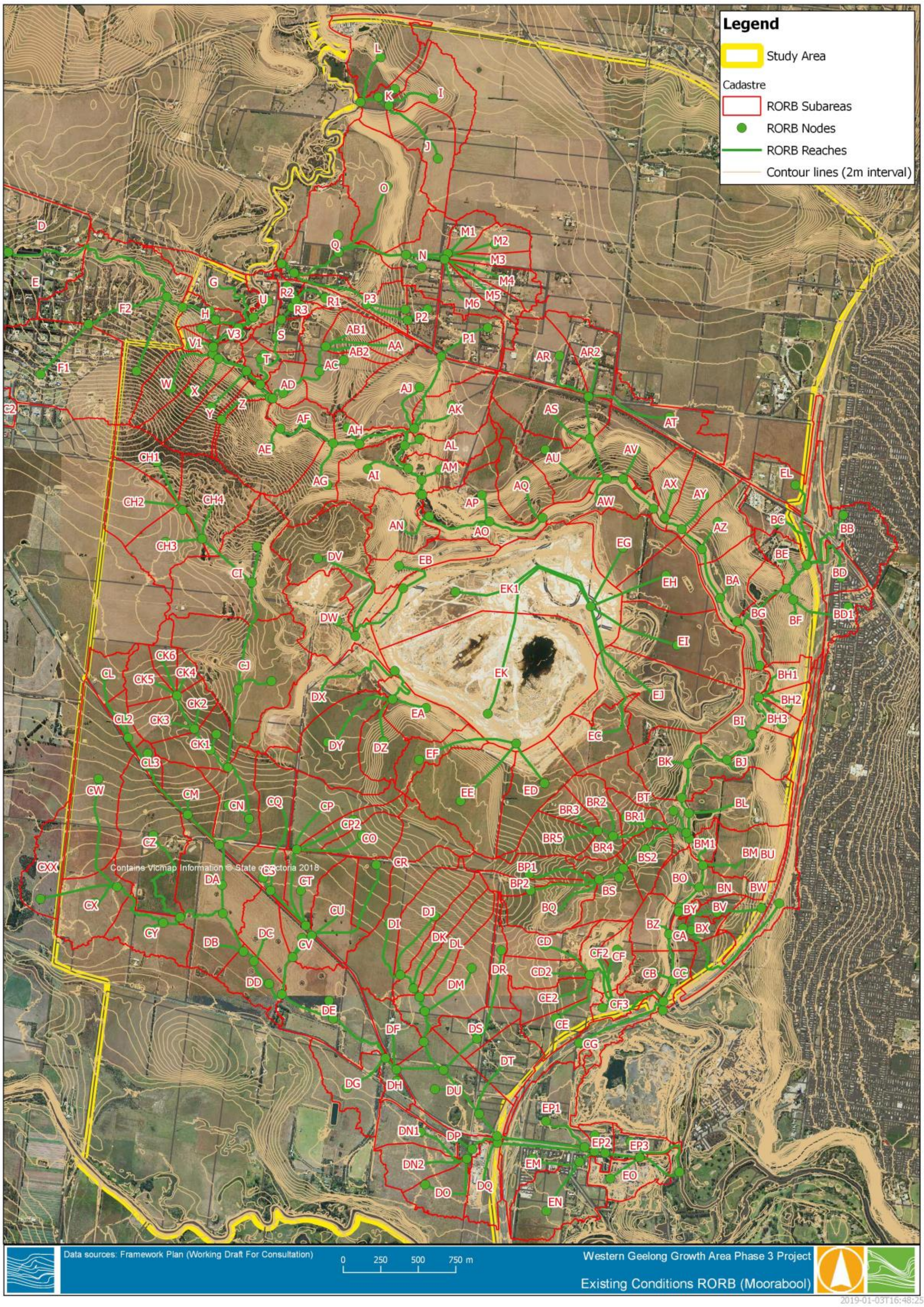


*Figure 4-3 Moorabool River undiverted RORB Model*

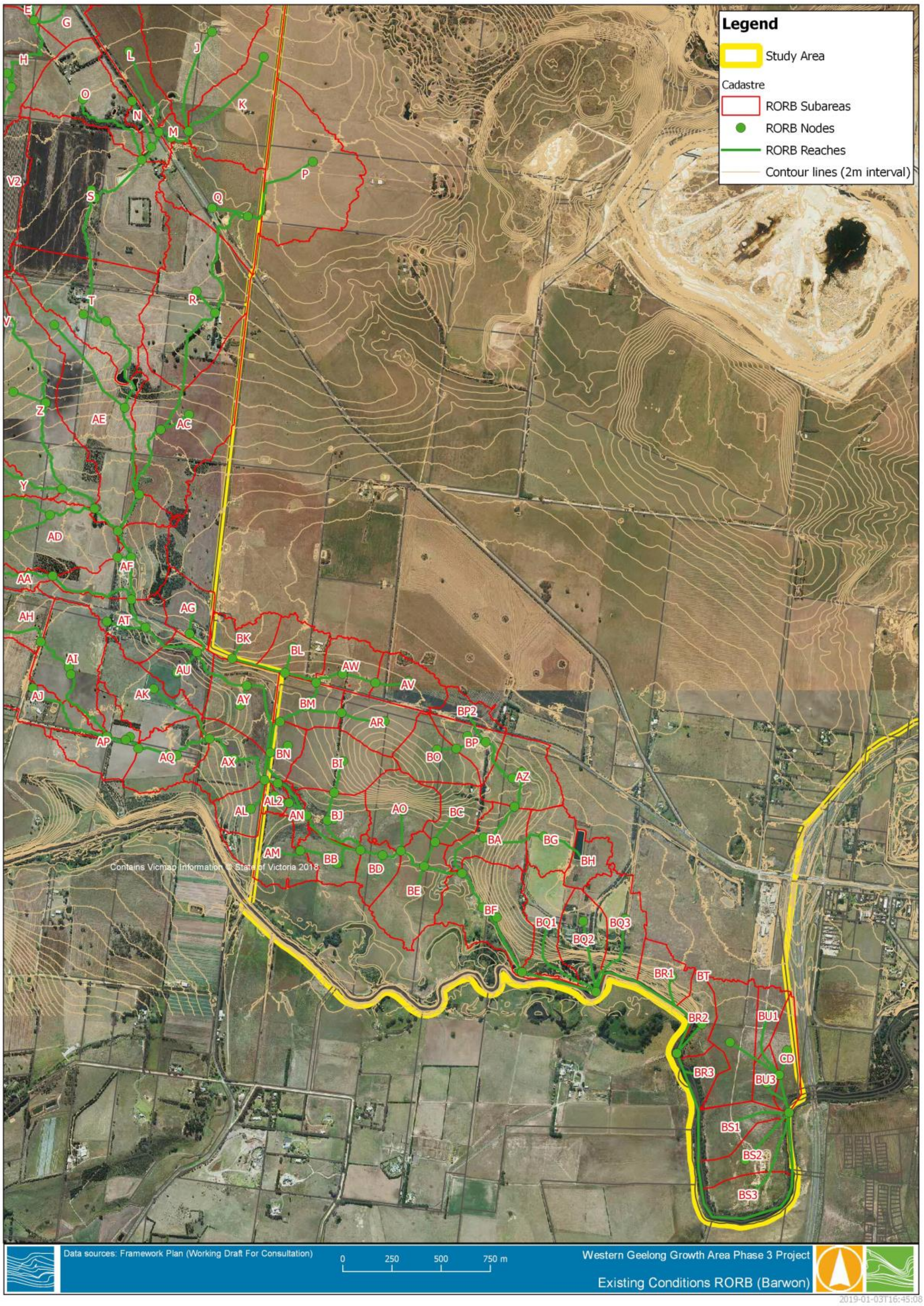


*Figure 4-4 Barwon River undiverted RORB Model*

To reflect the existing conditions of the site, diverted models were created by

- updating the fraction impervious information (refer to Section 4.2.2 below),
- updating the reach types

  Five different reach types are available in RORB (1 = natural, 2= excavated & unlined, 3= lined channel or pipe, 4= drowned reach, 5= dummy reach). The reach types in the undiverted model were all set as "natural" to be consistent with the VicRoads Rational Method. In the diverted model, the reaches types and slopes were updated to reflect existing conditions.
- adding in storages (formal and defacto) and diversions.

  Flood storage nodes were added in the Moorabool River diverted RORB model to account for flood retention within the three farm dams west of the Moorabool Quarry (refer to Figure 5-2) and water ponding between the Hamilton Highway and the Fyansford-Gheringhap Road.

  One diversion was added to the Cowies Creek diverted RORB to account for the current overland runoff being diverted under the Geelong Ring Road at Creamery Road.

### 4.2.2 Fraction Impervious

RORB subarea fraction impervious (FI) values in the undiverted model were set to 0.05 for all subareas to match the pre-development assumptions of the VicRoads Rational Method.

In the diverted model the fraction impervious values were updated to reflect the existing conditions based on the fraction impervious typical values presented in Table 4-1 and aerial photography. The typical fraction impervious values below were used in most cases unless aerial photography suggested that the value was not appropriate. For instance, the Idyll Wine Co winery is set on 'FZ' land. However, the land use is highly impervious and closer to the industrial land use type. Therefore, the fraction impervious value for this parcel was adjusted to 0.90 upon review of the aerial photography.

*Table 4-1 Existing Conditions Fraction Impervious Values*

| Zone Code | Land Use Type | Typical Fraction Impervious value |
|---|---|---|
| FZ | Farm Zone | 0.05 |
| PCRZ | Public Conservation and Resource Zone | 0.05 |
| PPRZ | Public Park and Recreation Zone | 0.10 |
| RCZ | Rural Conservation Zone | 0.05 |
| RDZ1 | Road Zone | 0.50 – 0.70 |
| RLZ | Rural Living Zone | 0.20 |

### 4.2.3 IFD

Design rainfall depths were determined using the Bureau of Meteorology online IFD tool and were based on 1987 data[3]. The rainfall Intensity Frequency Duration (IFD) parameters were generated for a location in the approximate centre of the Western Growth Study Area (144.300E, 38.100S) and are shown in Table 4-2 below.



[3] During the course of the project, new IFD information was released as part of the 2016 update to Australian Rainfall and Runoff.

*Table 4-2 Design Rainfall Intensity Frequency Duration Parameters*

| $2I_1$ (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 |
|---|---|---|---|---|---|---|---|---|
| 17.94 | 3.3 | 0.89 | 35.01 | 6.11 | 1.78 | 0.41 | 4.29 | 14.84 |

### 4.2.4 Areal Reduction Factors

Areal reduction factors were used to convert point rainfall to areal estimates and are used to account for the variation of rainfall intensities over a large catchment. Siriwardena and Weinmann (1996)[4] areal reduction factors were applied to the catchment area as recommended in Australian Rainfall and Runoff (1987)[5]. It is understood that these have not changed significantly for Victoria in the recent 2016 ARR edition.

### 4.2.5 Flow Validation/ Calibration

The RORB model flows were compared to the VicRoads Rational Method estimates. There are several model parameters used in RORB that control the modelled peak flow rate and volume of runoff; these are kc, m, initial and continuing losses.

#### Rainfall Losses

CoGG has advised that credible hydrographs for the nearby Hovells Creek catchment were derived by using estimated losses related to soil types, within a coupled hydrological-hydraulic model, with validation against gauged flood levels. Following this advice, it was agreed that this approach should be applied for the current study. Relevant geomorphic and soil landform maps were then extracted from the Victorian Resources Online[6] website (Figure 4-5) for the study area.

The losses were adopted based on the soil type. Table 4-3 below shows the predominant geomorphic and soil landform unit for Moorabool River catchment, its associated predominant soil type, and the adopted losses.

*Table 4-3 Derivation of RORB Rainfall Loss Parameters*

| Catchment | Predominant geomorphic and soil landform unit | Predominant soil type | Initial Loss (IL)/ Continuing Loss (CL) (mm) |
|---|---|---|---|
| Moorabool River | 172 | fine sandy loam | 20/3 |
| Cowies Creek | 119 | light clay | 15/2 |
| Barwon River | 119 | light clay | 15/2 |

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[4] Siriwardena and Weinmann, 1996 - Derivation of Areal Reduction Factors For Design Rainfalls (18 - 120 hours) in Victoria.
[5] ARR 1987 – Australian Rainfall and Runoff.
[6] http://vro.agriculture.vic.gov.au/dpi/vro/coranregn.nsf/pages/soil_landform_map_geelong

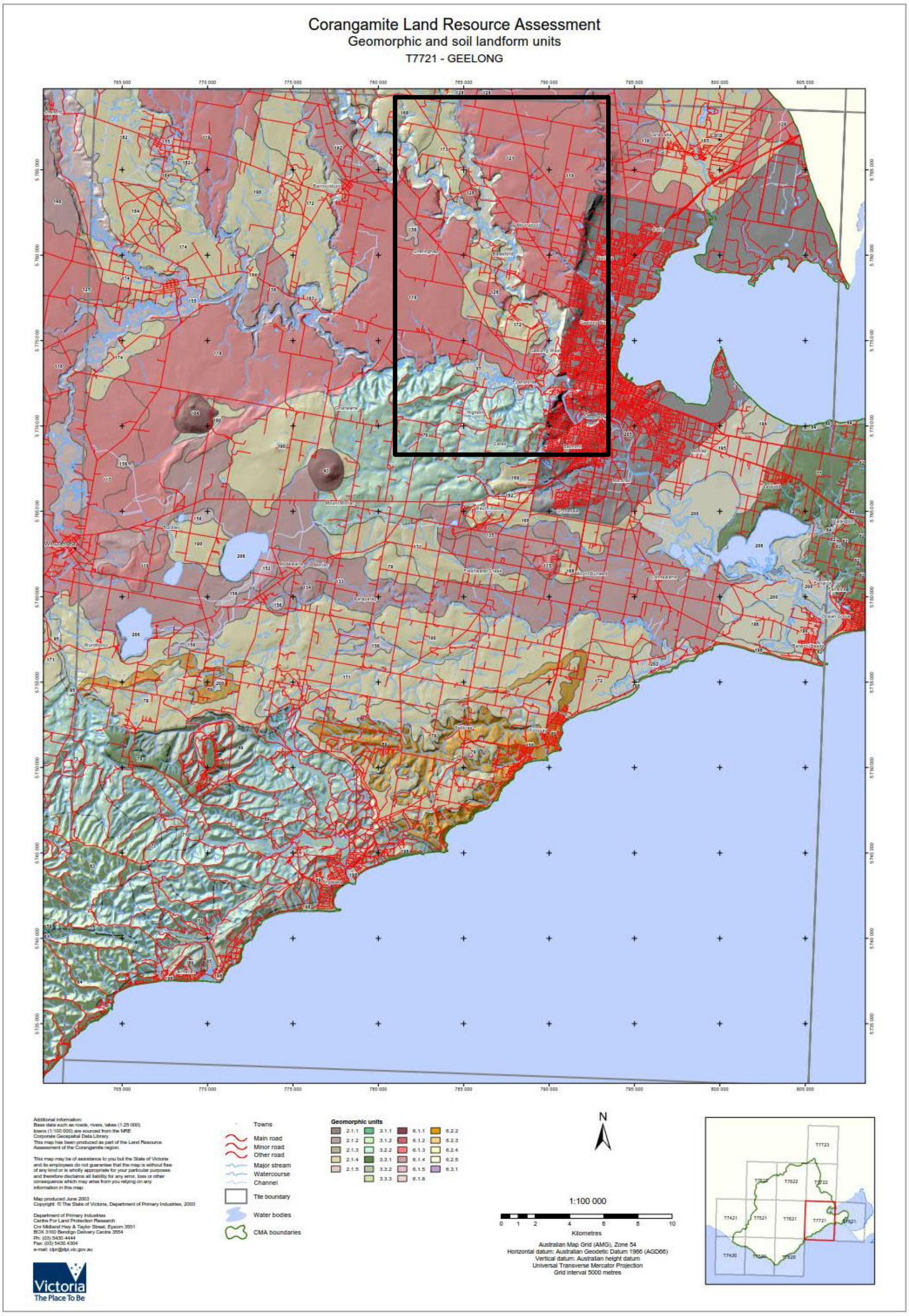


*Figure 4-5 Australian Map Grid (AMG), Zone 54 – Geomorphic and Soil Landform Map. The black rectangle indicates the area of interest.*

*(Source: http://vro.agriculture.vic.gov.au/dpi/vro/coranregn.nsf/pages/soil_landform_map_geelong)*

### m

The RORB m value is a measure of catchment non-linearity and is typically set at 0.8. This is an acceptable value for the degree of non-linearity of catchment response (Australian Rainfall and Runoff, 1987)[7]. There are alternate methods for determining m, such as Weeks (1980),[8] which uses multiple calibration events to select kc and m. However, unless there are strong reasons to modify this value it is generally left unchanged.

### kc

The RORB model kc was reconciled to the VicRoads Rational Method estimate.

The VicRoads Rational Method Estimate is based on a statistical interpretation of the Rational Method with design discharge factors determined by geographic location.

$$Q_{100} = \frac{(P_{100} \times I_{100} \times A)}{360}$$

Where:

$Q_{100}$ = Estimated maximum discharge for 1% AEP in $m^3/s$

$P_{100}$ = Design discharge factor $P_{100} = P_{10}\ F_{100}\ F_A$

$P_{10}$ = 10% AEP factor as read from AR&R maps

$F_{100}$ = Average Recurrence Interval Factor – Table 5.4 ARR 1987

$F_A$ = Area size factor (1.0, A>5000ha) (1.6-0.6, Area 1001-500ha) (2.1, A 301 -1000ha)

$I_{100}$ = average rainfall intensity (mm/h) for 1% AEP at catchment tc (time of concentration); and,

*A* = Catchment area in $km^2$.

### Flow Verification

Flows from the undiverted RORB models were reconciled to the VicRoads Rational Method at a number of locations throughout the catchments with varying upstream catchment areas. Figure 4-6, Figure 4-7 and Figure 4-8 present the locations for the flow comparisons between the undiverted RORB and VicRoads Rational Method peak design flows, as well as additional points at critical locations for the comparison of the diverted RORB model with the hydraulic model. The RORB results were plotted showing the relationship between the upstream catchment area and the flow rate at the outlet of each catchment for a given kc value. On the same graph, the results from the VicRoads Rational Method estimates are shown with a ± 30% range either side. Kc values in the RORB model were varied to achieve the best fit to the VicRoads Rational Method results whilst trying to achieve the most appropriate kc value for the catchment. The final **kc values of 3.2, 15 and 13** were adopted for the Moorabool River, Cowies Creek and Barwon River catchments respectively. The results from the analysis for the final adopted kc values in the 1% and 10% AEP events are shown from Figure 4-9 to Figure 4-14 below.



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

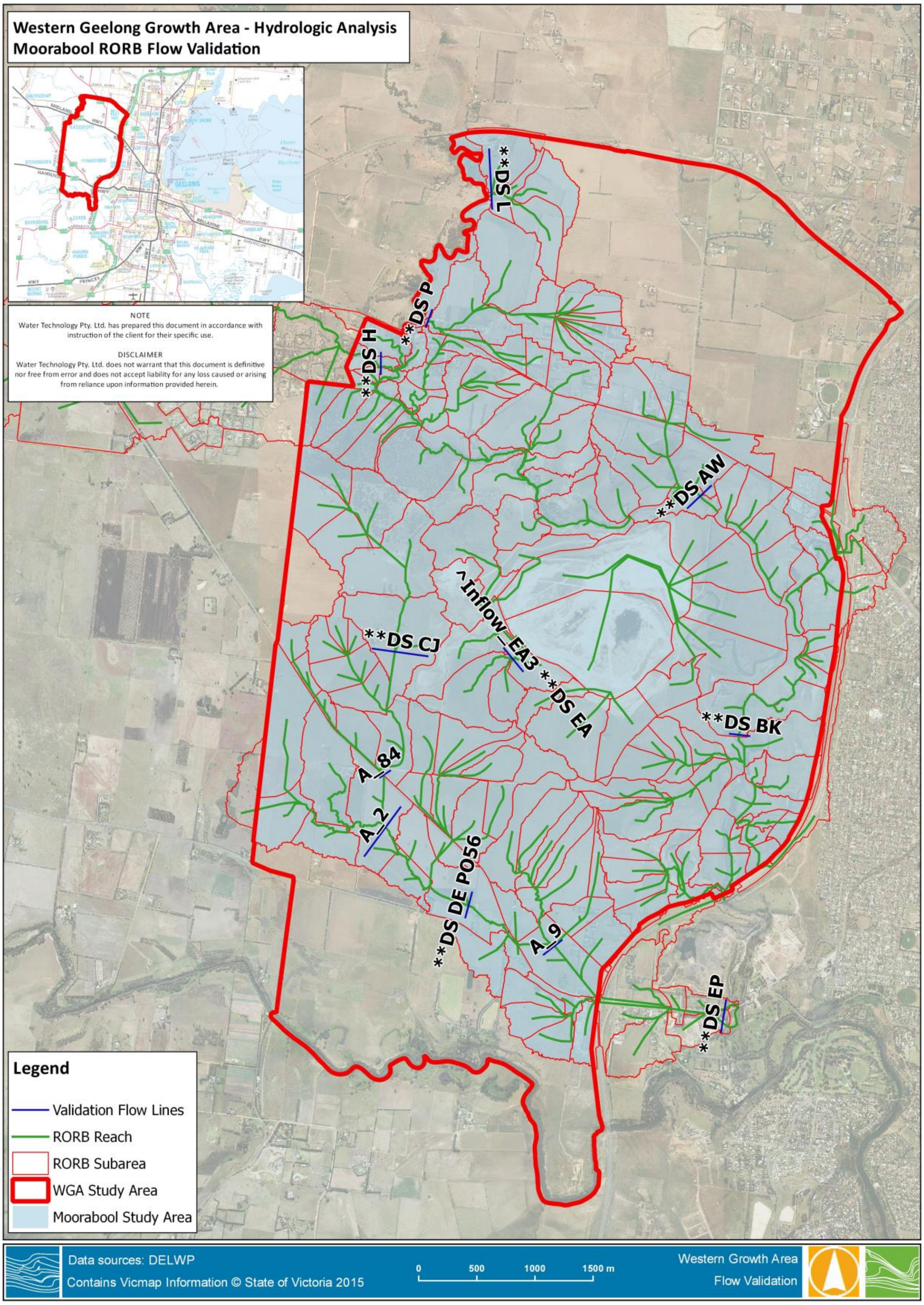


*Figure 4-6 Moorabool River Catchment – Flow Verification and Reporting Locations*

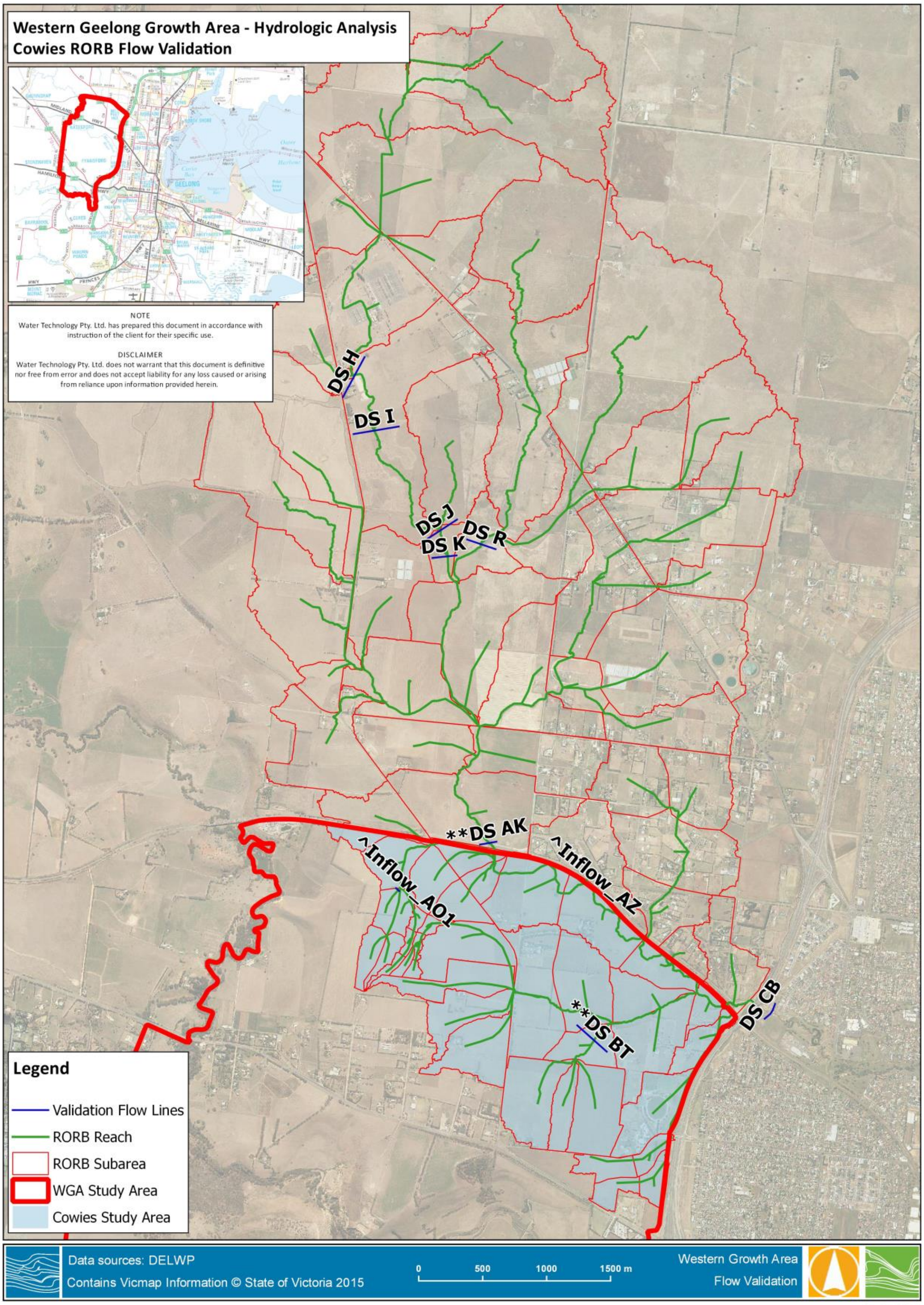


*Figure 4-7 Cowies Creek Catchment – Flow Verification and Reporting Locations*

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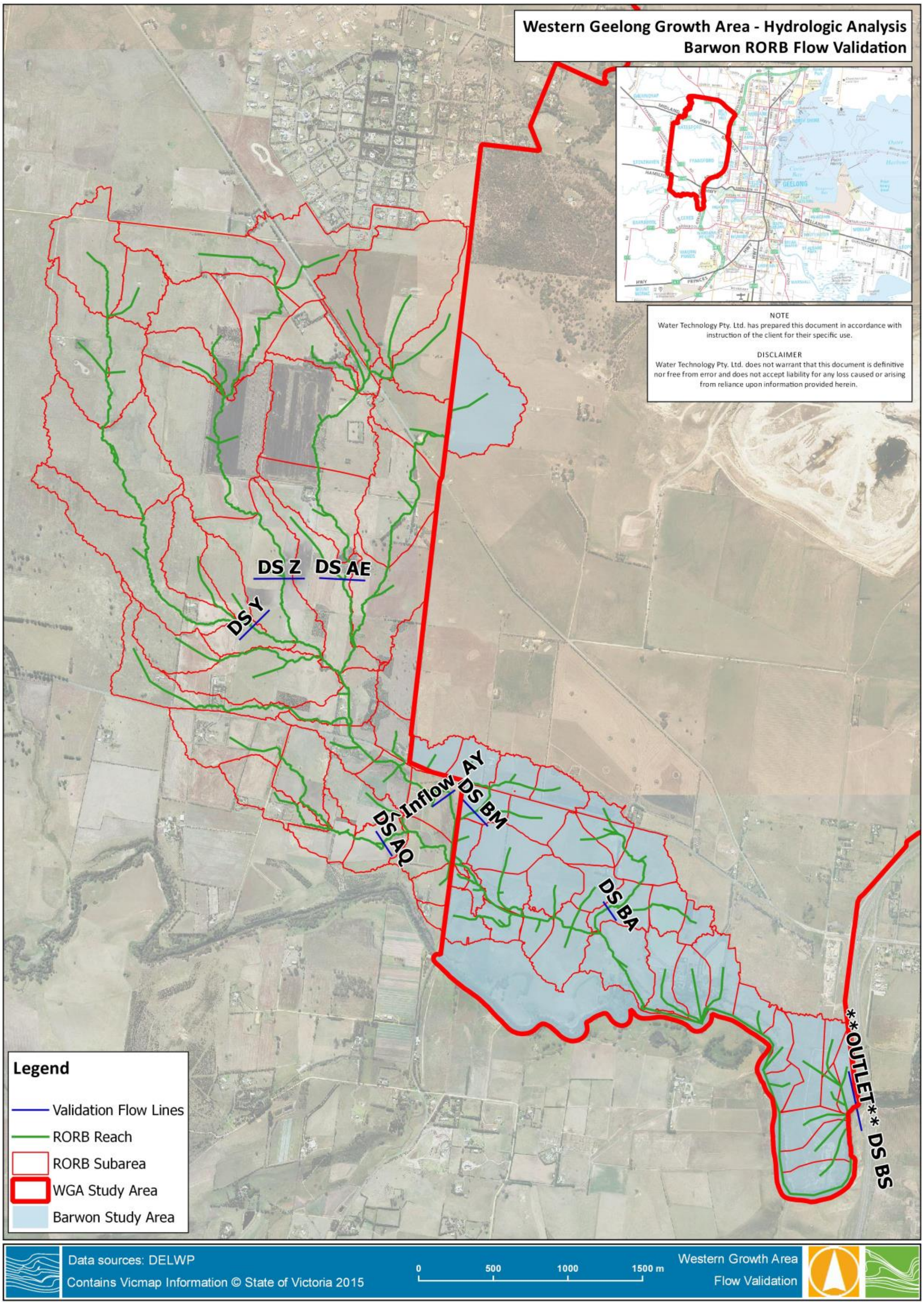


*Figure 4-8 Barwon River Catchment – Flow Verification and Reporting Locations*

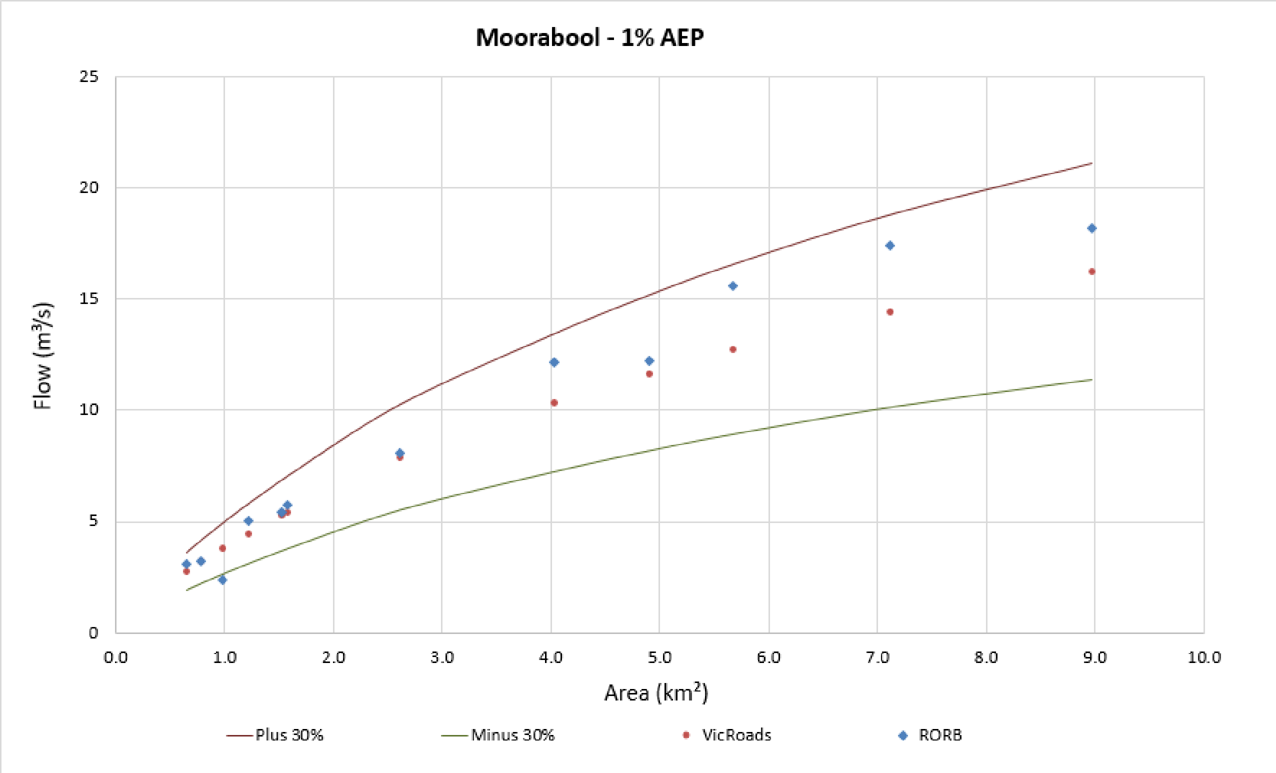


*Figure 4-9 Moorabool River Catchment Flow Validation – 1% AEP*

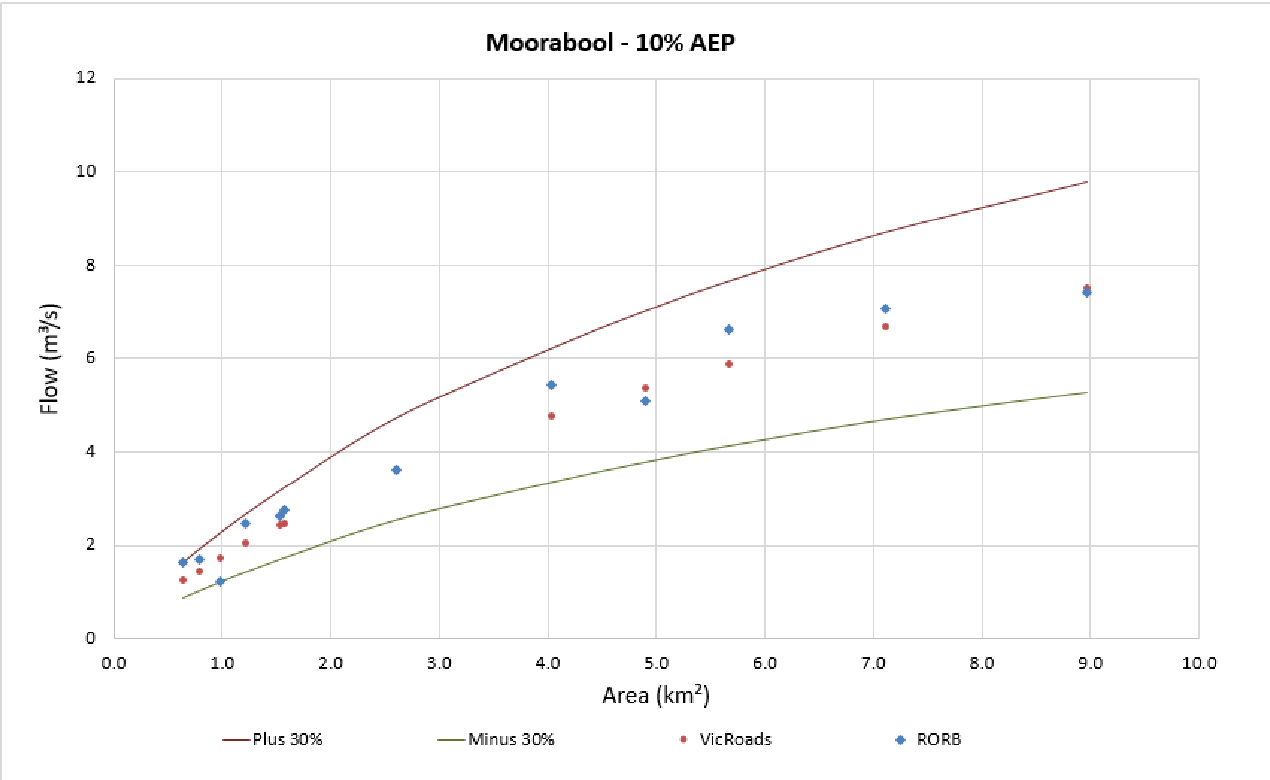


*Figure 4-10 Moorabool River Catchment Flow Validation – 10% AEP*

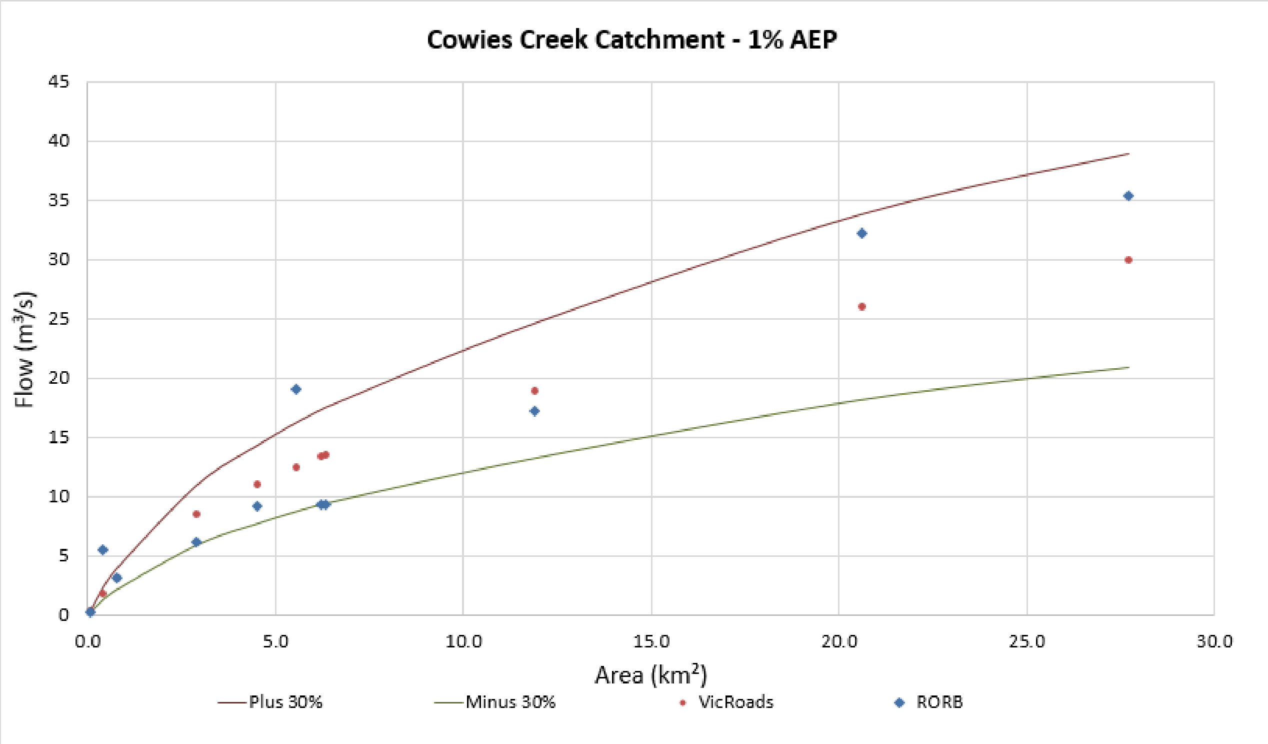


*Figure 4-11 Cowies Creek Catchment Model Flow Validation – 1% AEP*

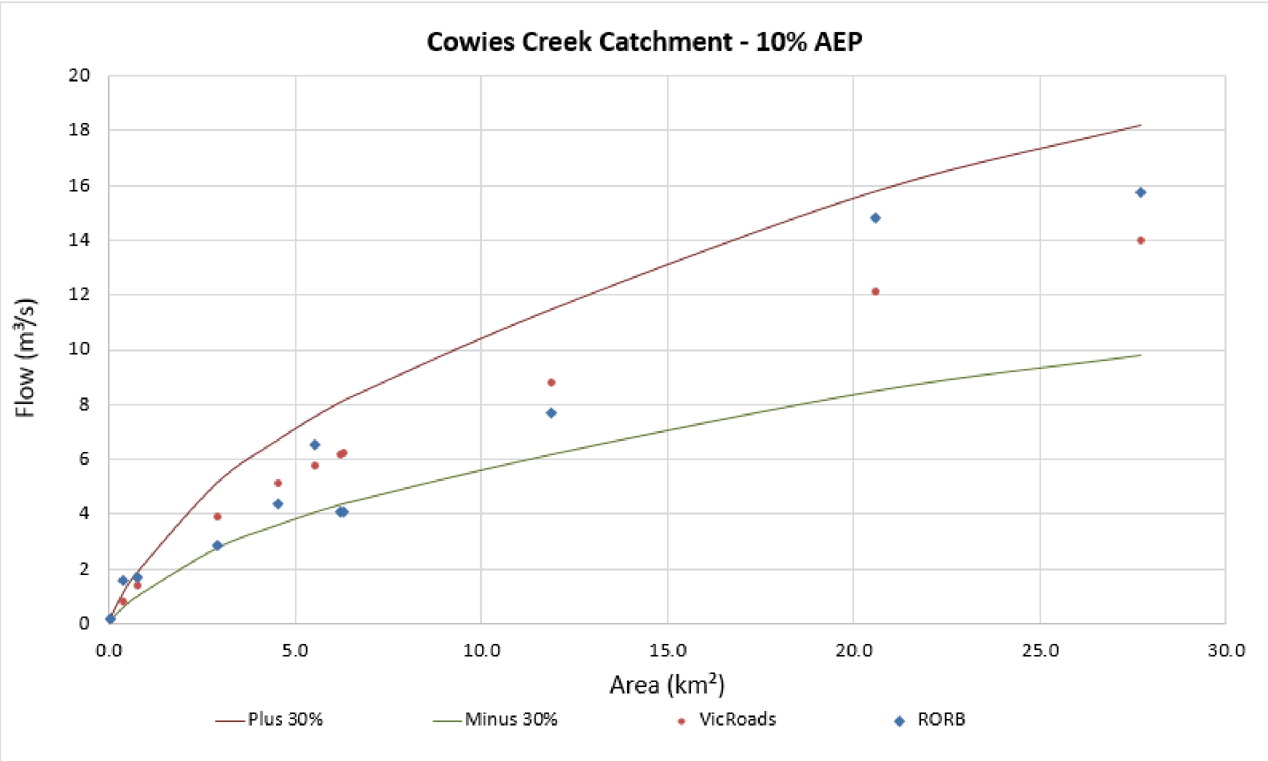


*Figure 4-12 Cowies Creek Catchment Model Flow Validation – 10% AEP*

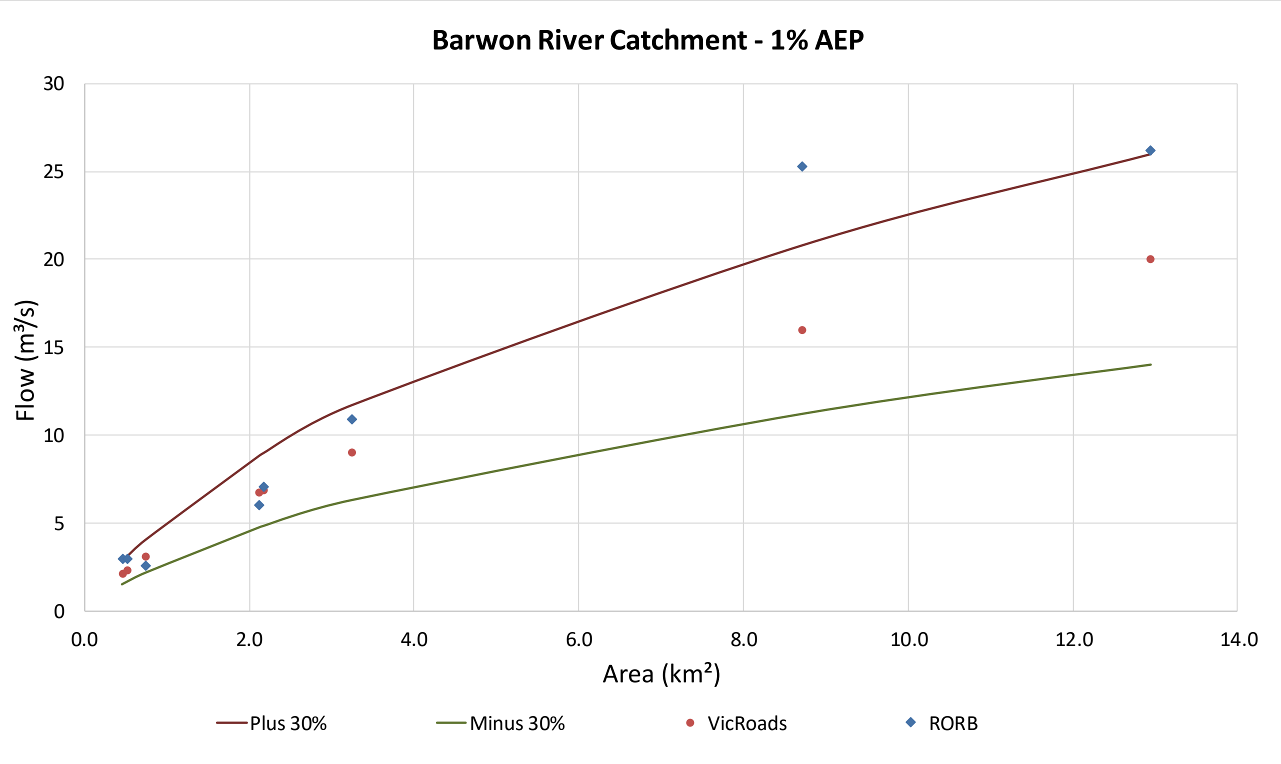


*Figure 4-13 Barwon River Catchment Model Flow Validation – 1% AEP*

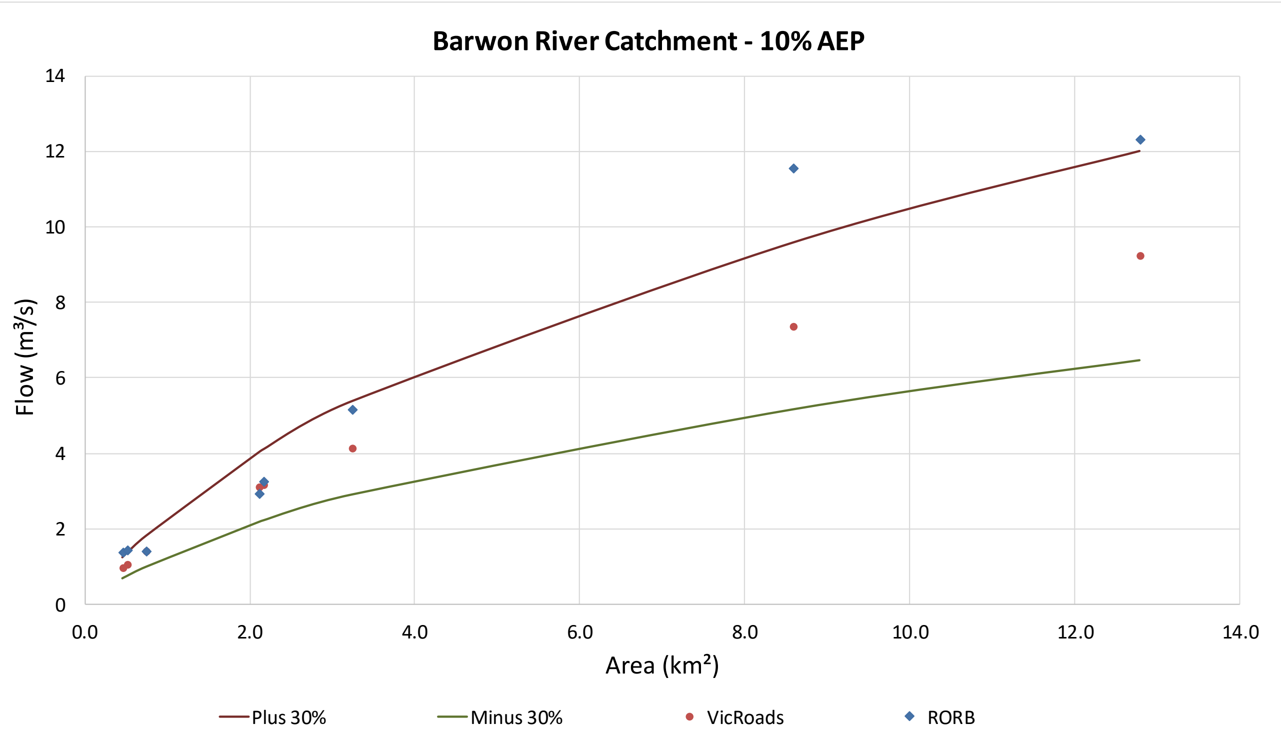


*Figure 4-14 Barwon River Catchment Model Flow Validation – 10% AEP*

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### 4.2.6 Results

The peak design flows for the 1% and 10% AEP floods for the undiverted RORB model and VicRoads Rational Method are presented in Table 4-4 to Table 4-6.

*Table 4-4 Moorabool River Catchment Flow Comparison*

| Location | 1% AEP Flow Rates (m³/s) | | 10% AEP Flow Rates (m³/s) | |
|---|---|---|---|---|
| **Print Location (RORB)** | **RORB** | **VicRoads** | **RORB** | **VicRoads** |
| **DS L | 3.1 | 2.8 | 1.7 | 1.2 |
| ^Inflow_EA3 **DS EA | 3.2 | 3.2 | 1.7 | 1.5 |
| **DS BK | 2.4 | 3.8 | 1.2 | 1.7 |
| **DS AW | 5.1 | 4.5 | 2.5 | 2.0 |
| **DS P | 5.4 | 5.3 | 2.6 | 2.4 |
| **DS CJ | 5.8 | 5.4 | 2.8 | 2.5 |
| A_84 | 8.1 | 7.9 | 3.6 | 3.6 |
| A_2 | 12.2 | 10.3 | 5.4 | 4.8 |
| **DS H | 12.2 | 11.7 | 5.1 | 5.4 |
| **DS DE PO56 | 15.6 | 12.7 | 6.6 | 5.9 |
| A_9 | 17.4 | 14.4 | 7.1 | 6.7 |
| **DS EP | 18.1 | 16.2 | 7.4 | 7.5 |

*Table 4-5 Cowies Creek Catchment Flow Comparison*

| Location | | 1% AEP Flow Rates (m³/s) | | 10% AEP Flow Rates (m³/s) | |
|---|---|---|---|---|---|
| **Print Location (RORB)** | **Area ($km^2$)** | **RORB** | **VicRoads** | **RORB** | **VicRoads** |
| ^Inflow_AO1 | 0.05 | 0.3 | 0.4 | 0.2 | 0.2 |
| ^Inflow_AZ | 0.39 | 5.6 | 1.9 | 0.8 | 1.6 |
| DS BT | 0.75 | 3.2 | 3.1 | 1.4 | 1.7 |
| DS R | 2.90 | 6.2 | 8.5 | 3.9 | 2.9 |
| DS H | 4.53 | 9.2 | 11.1 | 5.1 | 4.4 |
| DS I | 5.53 | 19.2 | 12.5 | 5.8 | 6.6 |
| DS J | 6.23 | 9.4 | 13.4 | 6.2 | 4.1 |
| DS K | 6.32 | 9.4 | 13.5 | 6.3 | 4.1 |
| S2 | 11.90 | 17.3 | 19.0 | 8.8 | 7.7 |
| DS AK | 20.61 | 32.2 | 26.1 | 12.1 | 14.8 |
| DS CB | 27.71 | 35.3 | 30.0 | 14.0 | 15.7 |

*Table 4-6 Barwon River Catchment Flow Comparison*

| Location | 1% AEP Flow Rates ($m^3/s$) | | 10% AEP Flow Rates ($m^3/s$) | |
|---|---|---|---|---|
| **Print Location (RORB)** | **RORB** | **VicRoads** | **RORB** | **VicRoads** |
| DS BM | 3.0 | 2.1 | 1.4 | 1.0 |
| DS BA | 3.0 | 2.3 | 1.4 | 1.0 |
| DS AQ | 2.6 | 3.1 | 1.4 | 1.4 |
| DS Y | 6.0 | 6.7 | 2.9 | 3.1 |
| DS Z | 7.1 | 6.9 | 3.3 | 3.2 |
| DS AE | 10.9 | 9.0 | 5.2 | 4.1 |
| ^Inflow_AY | 25.3 | 16.0 | 11.5 | 7.4 |
| **OUTLET** DS BS | 26.2 | 20.0 | 12.3 | 9.2 |

The results presented above indicate the undiverted RORB and VicRoads Rational Method flows match quite well for most catchments. This provides confidence in the RORB model results for use in the existing conditions hydraulic modelling.

# 5 PRELIMINARY HYDRAULIC MODELLING

## 5.1 Overview

A one-dimensional/two-dimensional TUFLOW hydraulic model was setup for each of the three main catchments covering the subject site: Cowies Creek, Moorabool River and Barwon River. Separating the study area in this manner reduced model run times and enabled focused analysis of the separate catchment flow contributions. A 5 m grid resolution was used for the Moorabool and Barwon River models and a 3 m grid resolution was used for the Cowies Creek model. The adopted grid sizes are considered appropriate for the level of detail required to establish overland flow paths and define existing conditions flood characteristics.

Each of the three models was simulated for three storm durations (2hr, 12hr and 24hr) based on the RORB results using the latest TUFLOW model build at the time of writing (2016-03-AB). For the main waterways, a 10% AEP level was set as the tailwater condition for all local runoff floods. Structure details, where available, were incorporated into the model.

## 5.2 Model Parameters

### 5.2.1 Boundary Conditions

Major inflow boundaries were applied at the upstream ends of the study area on the Moorabool River and Barwon River. These inflows were provided through the hydrologic modelling described in Section 3. Inflow boundaries for the Cowies Creek model were adopted from the Northern Growth Area model as supplied by SMEC through consultation with Council. **Note: 10 year ARI flow rates were not available from SMEC at the time of undertaking the modelling. In the 100-year and 10-year ARI modelling presented, the 10 year ARI Cowies Creek flood level was used as a tailwater level. For this version of the modelling, hydrographs were extracted from the Water Technology RORB modelling and used as boundary flows.** Within the study area, inflows were adopted from the RORB modelling. Hydrographs were extracted from print locations and applied as an SA (source over area) type boundary in TUFLOW.

The downstream boundary of the study area was defined by the confluence of the Barwon and Moorabool Rivers. The Barwon River flood study may also provide updated tailwater levels during later modelling stages. At this point, based on the timelines of the two studies, updated information will not be available for inclusion into the Phase 1 flood modelling. This is not expected to have any significant impact on flood extents for the main rivers, or for upstream tributary catchments. The Cowies Creek downstream tailwater level was set based on the expected hydraulic grade line at the outlet of the model.

To determine the 100 year ARI design flood extents for the major waterways (Barwon River, Moorabool River and Cowies Creek), separate TUFLOW models were created and run. For the Barwon and Moorabool River, design inflows came from the FFA. For the Cowies Creek catchment, inflows came from the SMEC investigation for the NGGA (a hydrograph with a peak 12-hour duration flow of 45.9 $m^3$/s was provided via email 11 November 2016)

Figure 5-1 shows the locations of the main riverine inflows to the hydraulic models as well as the SA Inflows across the study area. Note that the SA Inflows were based on the RORB sub-catchments and were further subdivided in places to provide a better representation of the expected flow paths after a review of some preliminary model runs.

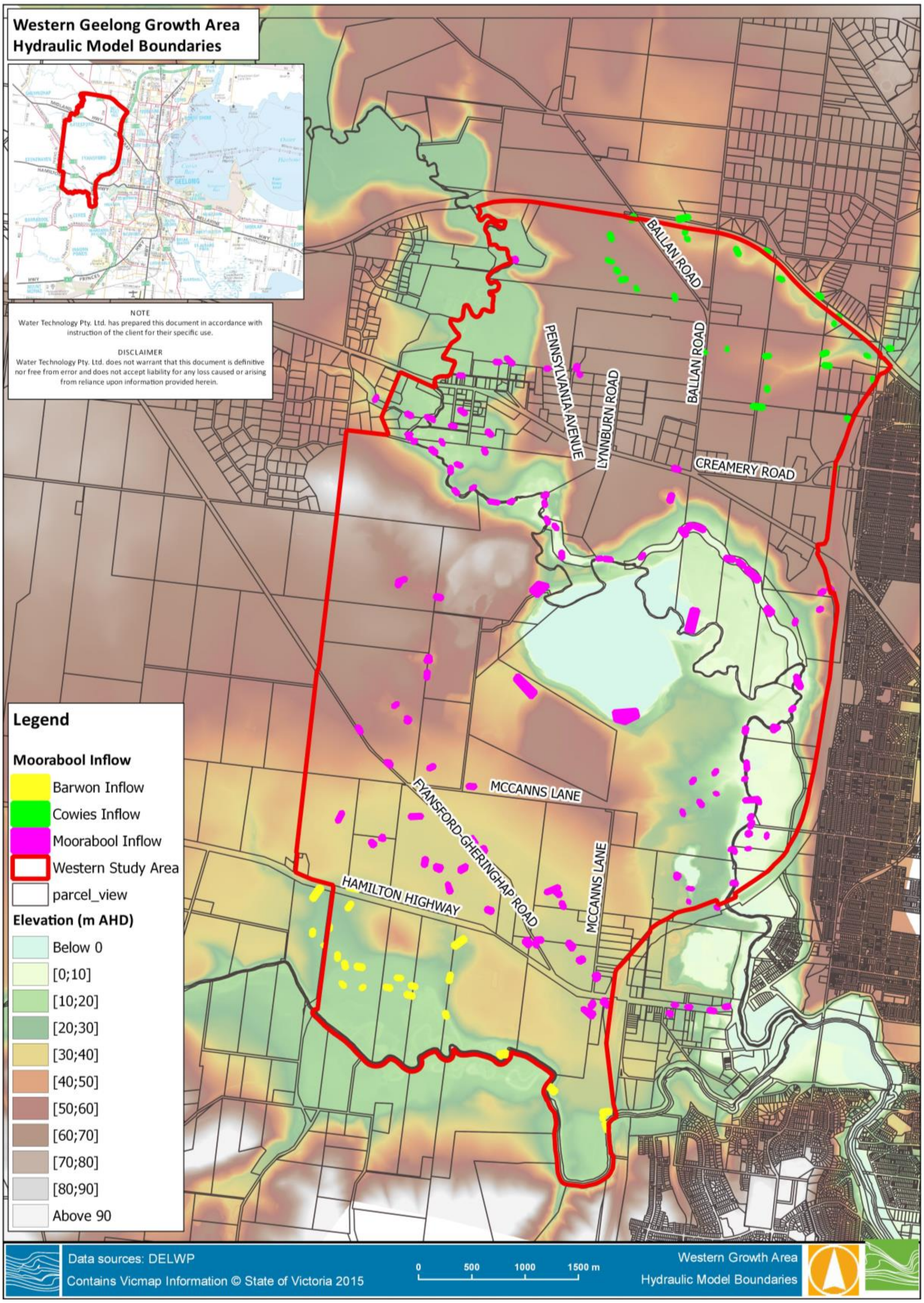


*Figure 5-1 TUFLOW Model Inflow Boundaries*

Three major farm dams were identified west of the Moorabool and east of Dog rocks Road as shown in Figure 5-2. Preliminary investigations of water levels and storage volumes, including looking through historical aerial photography, concluded that the two most upstream farm dams are unlikely to have enough water in them at the start of a rainfall even such that they would spill during an event. However, the most downstream farm dam is more likely to spill and therefore was assumed full at the start of the simulation (initial water level set at 55.0 m AHD).

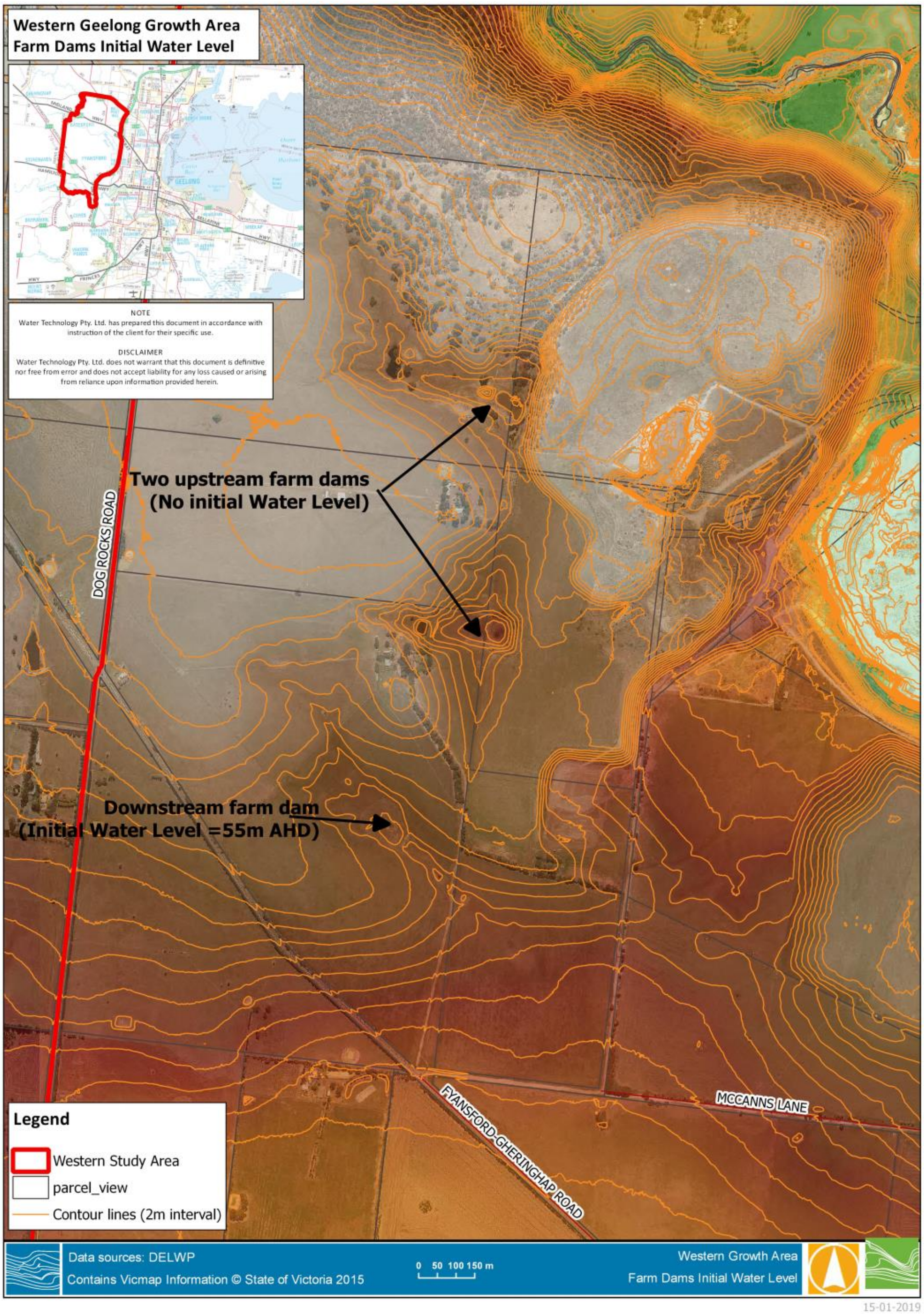


*Figure 5-2 Farm Dams Initial Water Level Assumptions*

### 5.2.2 Model Roughness

A depth-varying model roughness was adopted for grassed areas as discussed and agreed with Council. Figure 5-3 below shows the variation of Manning's roughness values as a function of the flood depth.

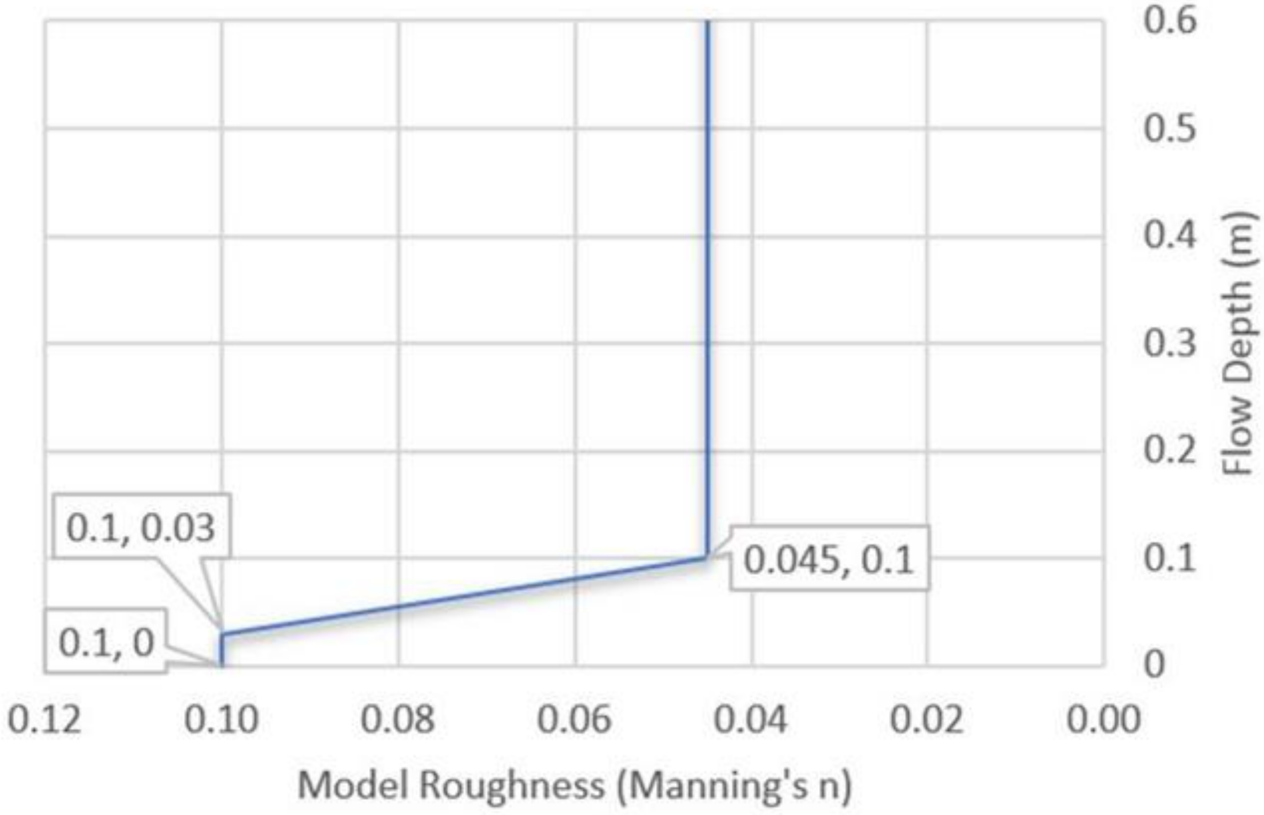


*Figure 5-3 Adopted Range of Model Roughness with Varying Depth*

For the remaining areas, the model hydraulic roughness was defined based on existing planning zone information and aerial imagery of the site. The model roughness was altered to better reflect the existing land use as opposed to the existing land zoning. A comparison of the existing zoning and the aerial imagery below in Figure 5-4 clearly shows areas where adopting a standard Manning's roughness value based on zoning alone is not accurate. Figure 5-5 shows the adopted Manning's roughness values for the existing conditions TUFLOW modelling. Depth varying roughness was applied to grassed area with Manning's value (n) following the following settings;

- Minimal vegetation; n = 0.1 for depths up to 0.03m, n= 0.045 for depths = to 0.1 m or greater, n linearly varying from 0.1 to 0.045 as depth increases from 0.03 m to 0.1
- Moderate vegetation; n = 0.1 for depths up to 0.03m, n= 0.06 for depths = to 0.1 m or greater, n linearly varying from 0.1 to 0.06 as depth increases from 0.03 m to 0.1
- Dense vegetation; n = 0.1 for depths up to 0.03m, n= 0.09 for depths = to 0.1 m or greater, n linearly varying from 0.1 to 0.09 as depth increases from 0.03 m to 0.1

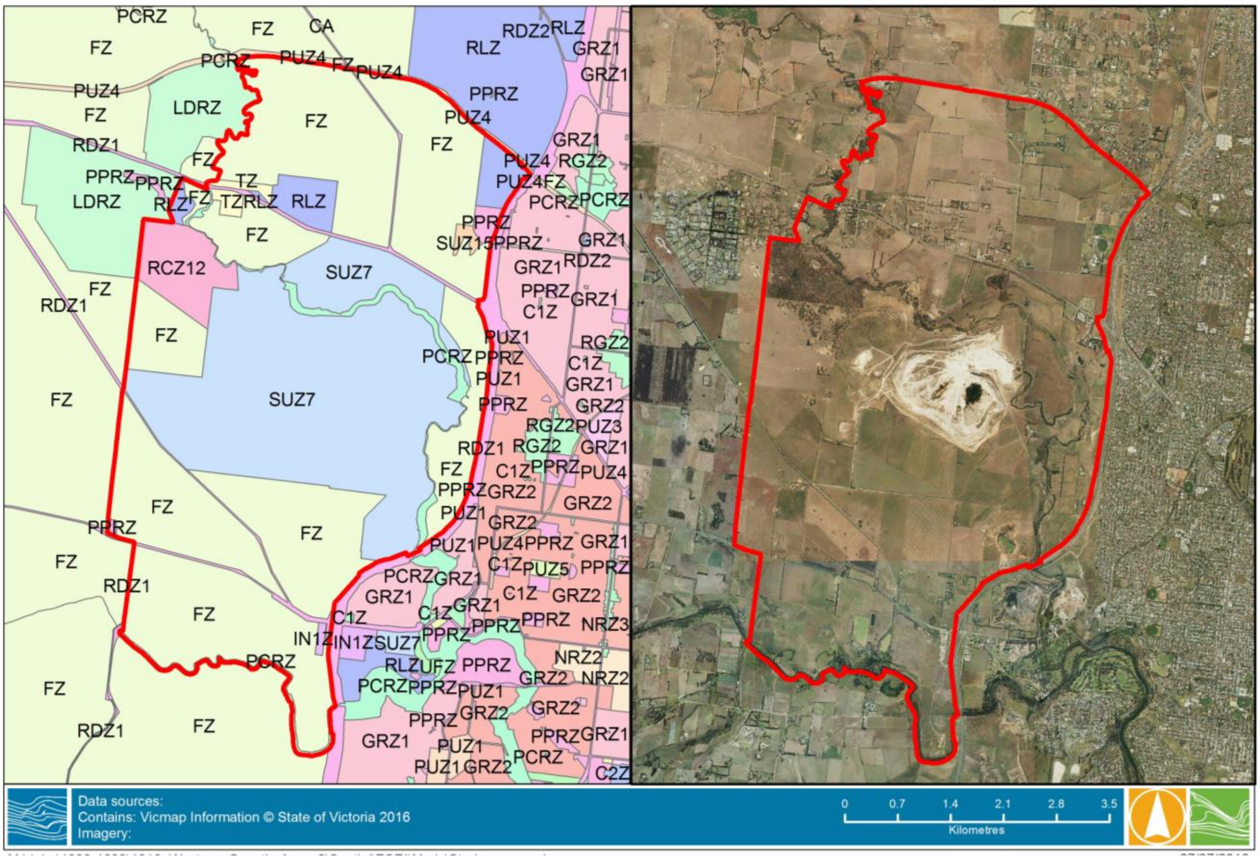


*Figure 5-4 Zonings compared with aerial imagery*

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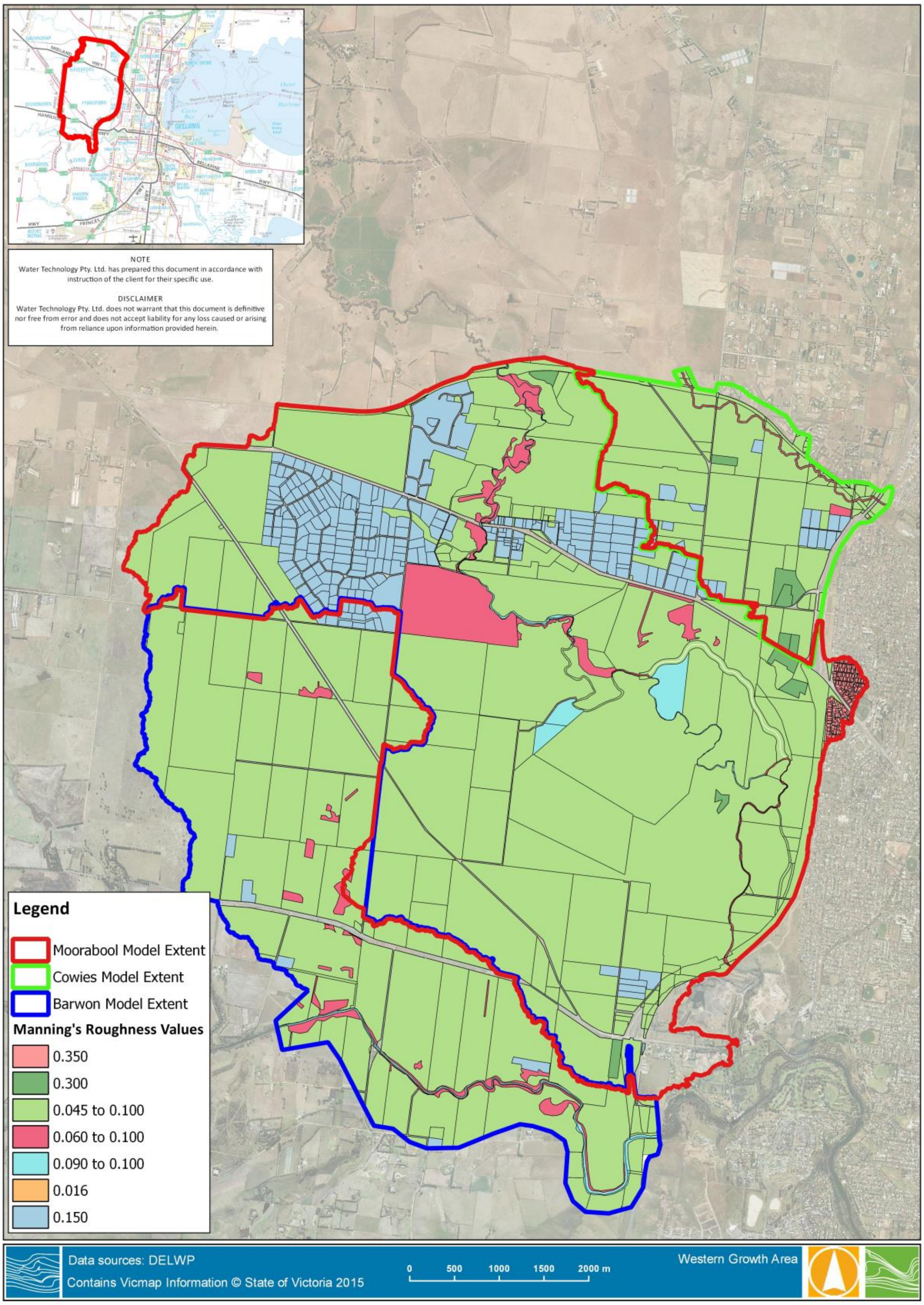


*Figure 5-5 Manning's Roughness Values Adopted in Existing Conditions Model; Not valid for Grassed Areas (See Figure 5-5)*

### 5.2.3 Structures

Key hydraulic structures were incorporated into the model using appropriate flow constrictions and embedded structures. These were based on available structure plans, site visits and information provided by Council and VicRoads. A discussion on structures was provided in Section 2.5. Each structure was added to the model as a one-dimensional link.

## 5.3 Preliminary Results

### 5.3.1 Comparison of RORB to TUFLOW Results

RORB results were compared with the preliminary results produced for the three TUFLOW models for both the 1% and 10% AEP design floods and are shown in Table 5-1 to Table 5-6. Differences between the TUFLOW and RORB results are provided and have been grouped into three categories, those with differences less than 15%, differences of 15–30% and differences greater than 30% as presented in Figure 5-6 to Figure 5-13. Commentary has also been provided throughout to add further explanation around the results, particularly in areas where the differences are greater.

#### Moorabool River Catchment

Throughout the Moorabool catchment, comparisons are generally very good in the catchments leading to the River immediately surrounding the quarry, particularly in the 1% AEP event. Differences do increase in the 10% AEP event, however these differences are seen to be acceptable for the purposes of the investigation and may be due to the influences of the depth varying roughness values in the shallower depths seen in this event. In all cases, the TUFLOW model results are lower than the RORB model results, potentially suggesting that the TUFLOW model is representing the catchment travel time at shallower depths in more detail than RORB.

Outputs for the area south of the quarry flowing towards Hamilton Highway show varied results. The upper reaches generally show very good matches, however areas between Fyansford-Gheringhap Road and Hamilton Highway are showing a poorer match. This was discussed prior to the modelling process and was expected to be an issue in this location. Flood flows through this area are very complex with shallow sheet flow moving between the defined RORB reach alignments, indicating that a hydrological model such as RORB is not the most appropriate model to use in this area. As discussed with Council at the meeting of 16 March, 2017, it is proposed to use TUFLOW modelling for the developed conditions options assessment for the catchment area upstream of Hamilton Highway for these reasons. The range of differences in this area of the catchment is consistent between the 1% and 10% AEP events. In the 1% AEP design flood, the greatest difference is seen at locations “A_2” and “**SouthWest1**”, located immediately upstream the junction between Fyansford-Gheringhap Road and Hamilton Highway. The respective difference seen at “**SouthWest1**” is 40%, with a RORB peak flow rate of 10.7 $m^3/s$ and a TUFLOW peak flow rate of 6.4 $m^3/s$. The RORB model was calibrated at the outlet upstream the Geelong Ring Road northern culverts to match the TUFLOW model 1% AEP peak flow. Calibration was achieved by adding RORB storage nodes upstream of some flow restrictions accounting for some of the 1% AEP overland flooding occurring between Fyansford-Gheringhap Road and Hamilton Highway. This provided a good match in peak 1% AEP flow between RORB and TUFLOW downstream of Fyansford-Gheringhap Road at Hamilton Highway.

It should be noted that flows along the Moorabool River itself were not modelled in RORB, with the inflow to the TUFLOW modelling coming from the FFA. Comparisons along the river have hence not been made and are not considered to be relevant for this study.

Flow comparison between the TUFLOW and RORB models show a close match for the flow location north of Fyansford-Gheringhap Road. However, the downstream areas located between Hamilton Highway and

Fyansford-Gheringhap Road are not so good. Flows from the catchment upstream of Fyansford-Gheringhap Road overtops the road near the intersection between Fyansford-Gheringhap Road and Friend Hand Road as shown in the TUFLOW model results. This shallow sheet flow, moving towards the south, is not represented or accounted for in the hydrological RORB model. Storage nodes were added to the RORB model as part of the calibration to prevent larger discrepancies in flow further downstream. Due to the limitations of the RORB model, we recommend using the TUFLOW model where the discrepancies in flows are not appropriate (between Fyansford-Gheringhap Road and Hamilton Highway). That is why we calibrated the RORB model to the TUFLOW peak 1% AEP flow at the outlet to Geelong Ring Road.

Hence it is proposed that the northern portion of the catchment, that discharges directly to the Moorabool River or the quarry, is modelled in RORB for developed conditions. All catchment areas upstream of the Hamilton Highway are proposed to be modelled in TUFLOW for developed conditions. In each case, a performance based approached will be applied to determine the most appropriate design method.

### Cowies Creek Catchment

For the purposes of this version of the report, flow comparisons have not been made along the main reach alignment of Cowies Creek – these will be made following receipt of inflow information from Council/SMEC.

A number of flow comparison locations were selected within the Cowies Creek catchment to compare the RORB model results to the TUFLOW modelling. There are three main branches which define the flow paths moving through the subject site to the creek. The most north-western branch immediately north of Batesford shows a very good match between the models, with flows differences less than 15% in the 1% event (2% at location "1_A"). The central main branch flowing south-east from the area north of Batesford towards Geelong along the Geelong-Ballan Road, also shows good matches in the 1% AEP event. Location "**DS_BT" shows the greatest difference (27%) with a RORB peak flow of 3.3 $m^3/s$ and a TUFLOW peak flow of 4.2 $m^3/s$. Similar to the Moorabool River results, the 10% AEP results show greater difference in most places. The exception is location "DS_BT" mentioned above, which improves to be a 3% difference (1.8 $m^3/s$ RORB vs 1.7 $m^3/s$ TUFLOW).

The southern branch which heads north towards the creek along the Geelong Ring Road is not shown. The flows generated from this catchment leave the catchment under the Geelong Ring Road at Creamery Road. This is represented in RORB as a diversion and was also naturally reflected in the TUFLOW modelling. No flows were seen in either model downstream of this location.

### Barwon River Catchment

The results of the Barwon River modelling show that large portions of the site are inundated by the large flows from the Barwon River catchment. This was represented in RORB by drowned reaches and in TUFLOW by having the Barwon River flows dynamically running with the sub-catchment flows. It is not expected that good matches would be seen in this area, and it is not expected to be an issue as most of the impacted low lying land would likely be considered un-developable.

Given the above, a small number of locations were used to compare the flows between the two models. The two western catchments which drain the elevated land north of the Barwon River (Locations **DS_BM and DS_AN) show very good matches in the 1% AEP event (5% and 9% respectively). The flow path flowing southerly from the centre of the study area, south of Hamilton Highway, shows a greater difference of 41% in the 1% AEP event (2.8 $m^3/s$ RORB vs 3.9 $m^3/s$ TUFLOW). Unlike the Moorabool and Cowies catchments, the 10% AEP results generally show closer results than the 1% AEP with the exception of location "DS_BM" (5% in 1% AEP vs 18% in 10% AEP), however the differences are minor and generally the matches are considered to be good.

It should be noted that flows along the Barwon River itself were not modelled in RORB, with the inflow to the TUFLOW modelling coming from the FFA. Comparisons along the River have hence not been made and are not considered to be relevant for the requirements of the study.

### 5.3.2 Flood Extents

Hydraulic modelling indicative flood extents, depths and WSE plots are provided in Figure 5-14 to Figure 5-19.

#### Moorabool River Catchment

Outside of inundation caused by the Moorabool River, flooding from local catchments is limited to a few defined areas.

Breakaway flows from the River combine with local catchment based overland flow heading west, north of Batesford. It is expected that some of this flooding from the local catchment may be able to be controlled through development. This is to be determined in later stages of the project. Patches of flooding are seen in the catchments leading to and surrounding the quarry. Many are minor and are considered likely to be able to be controlled through development.

Significant flooding is seen in the catchment upstream of Hamilton Highway. This area will be particularly difficult to manage during development. It is likely that a linear and dispersed quantity and quality control solution will be required in this area. This may take the form of a linear floodway, interspersed with storage. The slope of the catchment will also add to the difficulty here. Most of the flooding in this area is relatively shallow, generally less than 250 mm.

Between the 1% and 10% AEP results, only minor differences in extents are predicted. This is particularly noticeable in the area north of Batesford, and in the southern catchment in the area upstream of Fyansford – Gheringhap Road. Volumes entering the quarry are also greater in the 1% AEP event, as expected.

#### Cowies Creek Catchment

In the northern portion of the catchment, north of Batesford, spill through flow over Geelong-Ballan Road is predicted to head directly north towards the creek. The residual flow in this area passes in a south-easterly direction beside Geelong-Ballan Road then, just past Evans Road, flows overland in an easterly direction to Cowies Creek. The depth plots show that the large ponding area upstream of Geelong–Ballan Road is very shallow, less than 20 mm (see differences between the depth map which is filtered to 0.02 m and the WSE maps). It is thought that this shallow sheet flow will be readily managed through development.

The flow path moving easterly through the catchment towards the Geelong Ring Road is very well confined in both the 1% and 10% AEP events. It is expected that developed conditions modelling through this area will need to focus on how to achieve storage and treatment in these well-defined reaches.

Differences in predicted flood extents between the 1% and 10% AEP events are minor.

#### Barwon River Catchment

Flooding in the Barwon River catchment is largely defined by the flooding from the Barwon River itself. Two main flowpaths which show flooding from the local catchment are seen. The first is the flowpath from the area around Merrawarp Road which flows in a southerly direction towards the Barwon River. The second is a small flow path heading directly south within private property, south of Hamilton Highway.

Differences in flood extents between the 1% and 10% AEP events are minor.

*Table 5-1 Preliminary 1% AEP peak flow comparison – Moorabool River Catchment*

| Location | Diverted RORB | | TUFLOW | | | |
|---|---|---|---|---|---|---|
| **Print Location (RORB) PO Line (TUFLOW)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **% Difference (absolute value)** | **Comments** |
| **DS CJ | 2h | 0.0 | 2h | 0.0 | 0% | |
| A_2 | 9h | 7.7 | 24h | 5.6 | 27% | Breakaway flow south of Fyansford-Gheringhap Road causing flow discrepancies downstream - hence Q TUFLOW < Q RORB |
| **DS H | 9h | 19.7 | 2h | 18.0 | 9% | |
| **DS DE PO56 | 9h | 7.3 | 12h | 7.1 | 3% | |
| A_9 | 12h | 8.1 | 12h | 8.3 | 3% | |
| **DS EP | 2h | 11.0 | 12h | 10.9 | 1% | |
| ^Inflow_CX2 | 9h | 3.4 | 2h | 3.3 | 3% | Immediately downstream of an inflow |
| A_1 | 9h | 4.4 | 12h | 3.9 | 11% | |
| **SouthWest1** | 9h | 10.7 | 12h | 7.2 | 33% | RORB not accounting fully for storage as overland flooding is wide and shallow - hence Q TUFLOW < Q RORB |
| ^Inflow_AS2 | 2h | 2.8 | 2h | 2.8 | 0% | Immediately downstream of an inflow |
| A_55 | 2h | 3.6 | 2h | 4.1 | 14% | |
| ^Inflow_BF2 | 2h | 3.3 | 2h | 3.3 | 0% | Immediately downstream of an inflow |
| ^Inflow_BF | 2h | 3.1 | 2h | 3.0 | 3% | Immediately downstream of an inflow |
| ^Inflow_BP1 | 9h | 1.6 | 2h | 1.5 | 6% | Immediately downstream of an inflow |
| A_57 | 1.5h | 2.1 | 2h | 2.5 | 19% | |
| A_59 | 9h | 4.0 | 2h | 5.5 | 38% | Small flows - discrepancies around 1.5 $m^3/s$ – RORB not taking account of slope and roughness |
| ^Inflow_BR25 | 2h | 1.7 | 2h | 1.7 | 0% | Immediately downstream of an inflow |
| A_60 | 2h | 1.7 | 2h | 2.1 | 24% | Small flows - discrepancies around 0.4 $m^3/s$ – RORB not taking account of slope and roughness |

| Location | Diverted RORB | | TUFLOW | | | |
|---|---|---|---|---|---|---|
| ^Inflow_BV | 1.5h | 1.6 | 2h | 1.5 | 6% | Immediately downstream of an inflow |
| ^Inflow_BX2 | 9h | 1.0 | 2h | 0.9 | 10% | Immediately downstream of an inflow |
| ^Inflow_DK | 2h | 2.3 | 2h | 2.3 | 0% | Immediately downstream of an inflow |
| A_61 | 9h | 3.7 | 2h | 3.8 | 3% | |
| A_62 | 9h | 3.4 | 2h | 3.8 | 12% | |
| ^Inflow_CP2 | 2h | 1.7 | 2h | 1.7 | 0% | Immediately downstream of an inflow |
| A_85 | 2h | 2.0 | 24h | 1.9 | 5% | |
| C_8 | 9h | 2.8 | 12h | 2.8 | 0% | |
| A_83 | 9h | 3.8 | 2h | 3.1 | 18% | Breakaway flow south of Fyansford-Gheringhap Road causing flow discrepancies downstream - hence Q TUFLOW < Q RORB |
| ^Inflow_CL | 1.5h | 3.4 | 2h | 3.3 | 3% | Immediately downstream of an inflow |
| ^Inflow_CK | 2h | 2.1 | 2h | 1.7 | 19% | Immediately downstream of an inflow, over farm dam IWL (not representative flow line) |
| A_63 | 1.5h | 1.5 | 12h | 1.5 | 0% | |
| A_64 | 1h | 0.9 | 12h | 1.5 | 67% | Not representative flow line (not perpendicular to the flow path) - Good flow match immediately U/S and D/S |
| A_67 | 2h | 10.2 | 12h | 9.4 | 8% | |
| ^Inflow_AA1 | 2h | 0.8 | 2h | 0.8 | 0% | Immediately downstream of an inflow |
| A_69 | 2h | 2.1 | 12h | 0.8 | 62% | Breakaway flow upstream in Batesford causing flow discrepancies downstream - hence Q TUFLOW < Q RORB |
| A_71 | 2h | 2.6 | 12h | 1.9 | 27% | Some flood retention U/S Pennsylvania Av (RORB not taking into account storage and flood attenuation) - hence Q TUFLOW < Q RORB |
| A_74 | 2h | 5.8 | 2h | 10.2 | 76% | Breakaway flow from Moorabool River contributing to the flow increase - hence Q TUFLOW > Q RORB |
| A_75 | 10m | 0.0 | 2h | 0.0 | 0% | |
| ^Inflow_CH | 9h | 3.3 | 2h | 3.1 | 6% | Immediately downstream of an inflow |
| ^Inflow_DN1 | 9h | 2.0 | 2h | 1.8 | 10% | |

| Location | Diverted RORB | | TUFLOW | | | |
|---|---|---|---|---|---|---|
| ^Inflow_F2 | 9h | 19.4 | 2h | 17.7 | 9% | Immediately downstream of an inflow |
| A_77 | 9h | 19.5 | 2h | 19.6 | 1% | |
| ^Inflow_CG | 2h | 0.7 | 2h | 0.7 | 0% | |
| A_82 | 2h | 1.9 | 2h | 1.6 | 16% | |
| A_86 | 2h | 3.3 | 2h | 2.7 | 18% | Breakaway flow south of Fyansford-Gheringhap Road causing flow discrepancies downstream - hence Q TUFLOW < Q RORB |

*Table 5-2 Preliminary 1% AEP peak flow comparison – Cowies Creek Catchment*

| Location | Diverted RORB | | TUFLOW | | | |
|---|---|---|---|---|---|---|
| **Print Location (RORB) PO Line (TUFLOW)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **% Difference (absolute value)** | **Comments** |
| ^Inflow_AO1 | 1h | 0.3 | 2h | 0.3 | 0% | Flow cannot be compared as it is influenced by the upstream flow and flow line is not wide enough. |
| ^Inflow_AZ | 1.5h | 5.6 | 2h | 5.5 | 1% | |
| **DS BT | 2h | 3.3 | 2h | 4.2 | 27% | Discrepancies may be caused by RORB model not accounting for slope and roughness unlike the TUFLOW model. |
| **DS AK | 12h | 32.1 | 12h | 31.6 | 2% | |
| 1_A | 1.5h | 2.2 | 2h | 2.2 | 2% | Flow cannot be compared as it is influenced by the upstream flow and flow line is not wide enough. |
| 14_A | 9h | 8.2 | 12h | 9.3 | 14% | |
| 18_A | 2h | 7.7 | 2h | 7.2 | 6% | |
| 28_A | 2h | 2.4 | 2h | 2.3 | 5% | |
| DS CB | 18h | 31.6 | 12h | 40.8 | 29% | Discrepancies may be caused by RORB model not accounting for slope and roughness unlike the TUFLOW model. |

*Table 5-3 Preliminary 1% AEP peak flow comparison – Barwon River Catchment*

| Location | Diverted RORB | | TUFLOW | | | |
|---|---|---|---|---|---|---|
| **Print Location (RORB) PO Line (TUFLOW)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **% Difference (absolute value)** | **Comments** |
| DS BM | 2h | 3.2 | 2h | 3.1 | 5% | |
| DS BA | 2h | 2.8 | 2h | 3.9 | 41% | Small flows - discrepancies around 1.5 $m^3/s$ – RORB not taking account of slope and roughness |
| ^Inflow_AY | 9h | 24.4 | 12h | 20.7 | 15% | |
| DS AN | 9h | 26.9 | 12h | 24.6 | 9% | |

*Table 5-4 Preliminary 10% AEP peak flow comparison – Moorabool River Catchment*

| Location | Diverted RORB | | TUFLOW | | | |
|---|---|---|---|---|---|---|
| **Print Location (RORB) PO Line (TUFLOW)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **% Difference** | **Comments** |
| **DS CJ | 2h | 0.0 | 2h | 0.0 | 0% | |
| A_2 | 9h | 3.7 | 12h | 2.2 | 41% | Breakaway flow south of Fyansford-Gheringhap Road causing flow discrepancies downstream - hence Q TUFLOW < Q RORB |
| **DS H | 9h | 9.9 | 12h | 7.5 | 24% | |
| **DS DE PO56 | 9h | 3.0 | 12h | 2.3 | 23% | |
| A_9 | 12h | 3.5 | 12h | 2.4 | 31% | |
| **DS EP | 12h | 4.1 | 12h | 4.2 | 2% | |
| ^Inflow_CX2 | 9h | 1.8 | 12h | 1.3 | 28% | Immediately downstream of an inflow |
| A_1 | 9h | 2.2 | 12h | 1.6 | 27% | |

| Location | Diverted RORB | | TUFLOW | | | |
|---|---|---|---|---|---|---|
| **SouthWest1** | 9h | 4.1 | 12h | 2.3 | 44% | RORB not accounting fully for storage as overland flooding is wide and shallow - hence Q TUFLOW < Q RORB |
| ^Inflow_AS2 | 9h | 1.4 | 12h | 1.1 | 21% | Immediately downstream of an inflow |
| A_55 | 9h | 1.8 | 12h | 1.6 | 11% | |
| ^Inflow_BF2 | 9h | 1.6 | 2h | 1.4 | 13% | Immediately downstream of an inflow |
| ^Inflow_BF | 9h | 1.1 | 2h | 0.9 | 18% | Immediately downstream of an inflow |
| ^Inflow_BP1 | 9h | 0.8 | 12h | 0.6 | 25% | Immediately downstream of an inflow |
| A_57 | 9h | 1.1 | 12h | 1.0 | 9% | |
| A_59 | 9h | 2.1 | 12h | 2.2 | 5% | |
| ^Inflow_BR25 | 9h | 0.9 | 12h | 0.7 | 22% | Immediately downstream of an inflow |
| A_60 | 9h | 0.9 | 12h | 0.8 | 11% | |
| ^Inflow_BV | 9h | 0.6 | 12h | 0.5 | 17% | Immediately downstream of an inflow |
| ^Inflow_BX2 | 9h | 0.5 | 24h | 0.4 | 20% | Immediately downstream of an inflow |
| ^Inflow_DK | 9h | 1.3 | 12h | 1.0 | 23% | Immediately downstream of an inflow |
| A_61 | 9h | 2.0 | 12h | 1.5 | 25% | |
| A_62 | 9h | 1.8 | 12h | 1.5 | 17% | |
| ^Inflow_CP2 | 9h | 0.9 | 12h | 0.7 | 22% | Immediately downstream of an inflow |
| A_85 | 9h | 0.9 | 12h | 0.8 | 11% | |
| C_8 | 12h | 0.7 | 12h | 0.7 | 0% | |
| A_83 | 9h | 2.0 | 12h | 0.9 | 55% | Breakaway flow south of Fyansford-Gheringhap Road causing flow discrepancies downstream - hence Q TUFLOW < Q RORB |
| ^Inflow_CL | 9h | 0.9 | 12h | 0.8 | 11% | Immediately downstream of an inflow |
| ^Inflow_CK | 9h | 1.1 | 12h | 0.7 | 36% | Immediately downstream of an inflow, over farm dam IWL (not representative flow line) |
| A_63 | 9h | 0.5 | 12h | 0.6 | 20% | |
| A_64 | 9h | 0.4 | 12h | 0.6 | 50% | Not representative flow line (not perpendicular to the flow path) - Good flow match immediately U/S and D/S |

| Location | Diverted RORB | | TUFLOW | | | |
|---|---|---|---|---|---|---|
| A_67 | 12h | 3.6 | 12h | 3.9 | 8% | |
| ^Inflow_AA1 | 9h | 0.4 | 12h | 0.3 | 25% | Immediately downstream of an inflow |
| A_69 | 9h | 1.0 | 12h | 0.2 | 80% | Breakaway flow upstream in Batesford causing flow discrepancies downstream - hence Q TUFLOW < Q RORB |
| A_71 | 9h | 1.4 | 24h | 0.8 | 43% | Some flood retention U/S Pennsylvania Av (RORB not taking into account storage and flood attenuation) - hence Q TUFLOW < Q RORB |
| A_74 | 9h | 2.8 | 2h | 7.4 | 164% | Breakaway flow from Moorabool River contributing to the flow increase - hence Q TUFLOW > Q RORB |
| A_75 | 10m | 0.0 | 2h | 0.0 | 0% | |
| ^Inflow_CH | 9h | 1.8 | 12h | 1.2 | 33% | Immediately downstream of an inflow |
| ^Inflow_DN1 | 9h | 1.1 | 12h | 0.7 | 36% | |
| ^Inflow_F2 | 9h | 10.0 | 12h | 7.6 | 24% | Immediately downstream of an inflow |
| A_77 | 9h | 10.0 | 12h | 7.5 | 25% | |
| ^Inflow_CG | 9h | 0.3 | 12h | 0.3 | 0% | |
| A_82 | 9h | 1.0 | 12h | 0.7 | 30% | |
| A_86 | 9h | 1.7 | 12h | 1.0 | 41% | Breakaway flow south of Fyansford-Gheringhap Road causing flow discrepancies downstream - hence Q TUFLOW < Q RORB |

*Table 5-5 Preliminary 10% AEP peak flow comparison – Cowies Creek Catchment*

| Location | Diverted RORB | | TUFLOW | | | |
|---|---|---|---|---|---|---|
| **Print Location (RORB) PO Line (TUFLOW)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **% Difference (absolute value)** | **Comments** |
| ^Inflow_AO1 | 9h | 0.2 | 12h | 0.1 | 34% | Flow cannot be compared as it is influenced by the upstream flow and flow line is not wide enough. |
| ^Inflow_AZ | 2h | 2.4 | 2h | 2.7 | 14% | |
| **DS BT | 9h | 1.8 | 2h | 1.7 | 3% | |

| Location | Diverted RORB | | TUFLOW | | | |
|---|---|---|---|---|---|---|
| **DS AK | 12h | 14.9 | 12h | 14.3 | 4% | |
| 1_A | 9h | 0.9 | 12h | 0.8 | 6% | Flow cannot be compared as it is influenced by the upstream flow and flow line is not wide enough. |
| 14_A | 9h | 3.9 | 12h | 4.8 | 22% | Discrepancies may be caused by RORB model not accounting for slope and roughness unlike the TUFLOW model. |
| 18_A | 9h | 2.7 | 2h | 2.6 | 5% | |
| 28_A | 9h | 1.0 | 2h | 0.8 | 17% | |
| DC CB | 12h | 14.5 | 12h | 18.6 | 29% | Discrepancies may be caused by RORB model not accounting for slope and roughness unlike the TUFLOW model. |

*Table 5-6 Preliminary 10% AEP peak flow comparison – Barwon River Catchment*

| Location | Diverted RORB | | TUFLOW | | | |
|---|---|---|---|---|---|---|
| **Print Location (RORB) PO Line (TUFLOW)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **Critical Storm Duration (hr)** | **Peak Flow ($m^3/s$)** | **% Difference (absolute value)** | **Comments** |
| DS BM | 9h | 1.4 | 12h | 1.2 | 18% | Small flows - discrepancies around 0.2 $m^3/s$ – RORB not taking account of slope and roughness |
| DS BA | 9h | 1.3 | 12h | 1.5 | 13% | |
| ^Inflow_AY | 9h | 11.2 | 12h | 11.1 | 1% | |
| DS AN | 12h | 12.3 | 12h | 12.8 | 4% | |

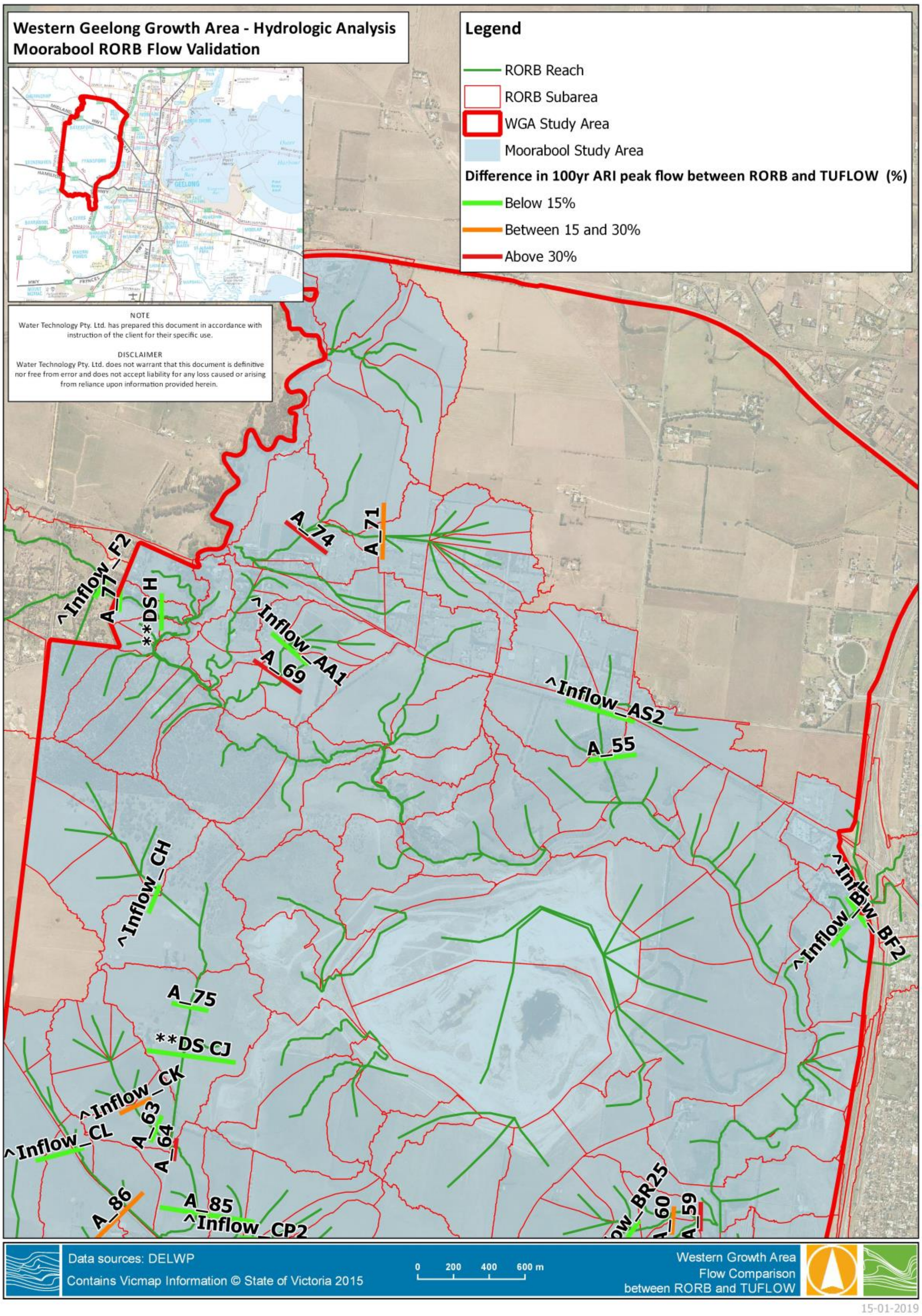


*Figure 5-6 Flow Estimation Points for Comparison (1%) – Moorabool River Catchment (Northern section)*

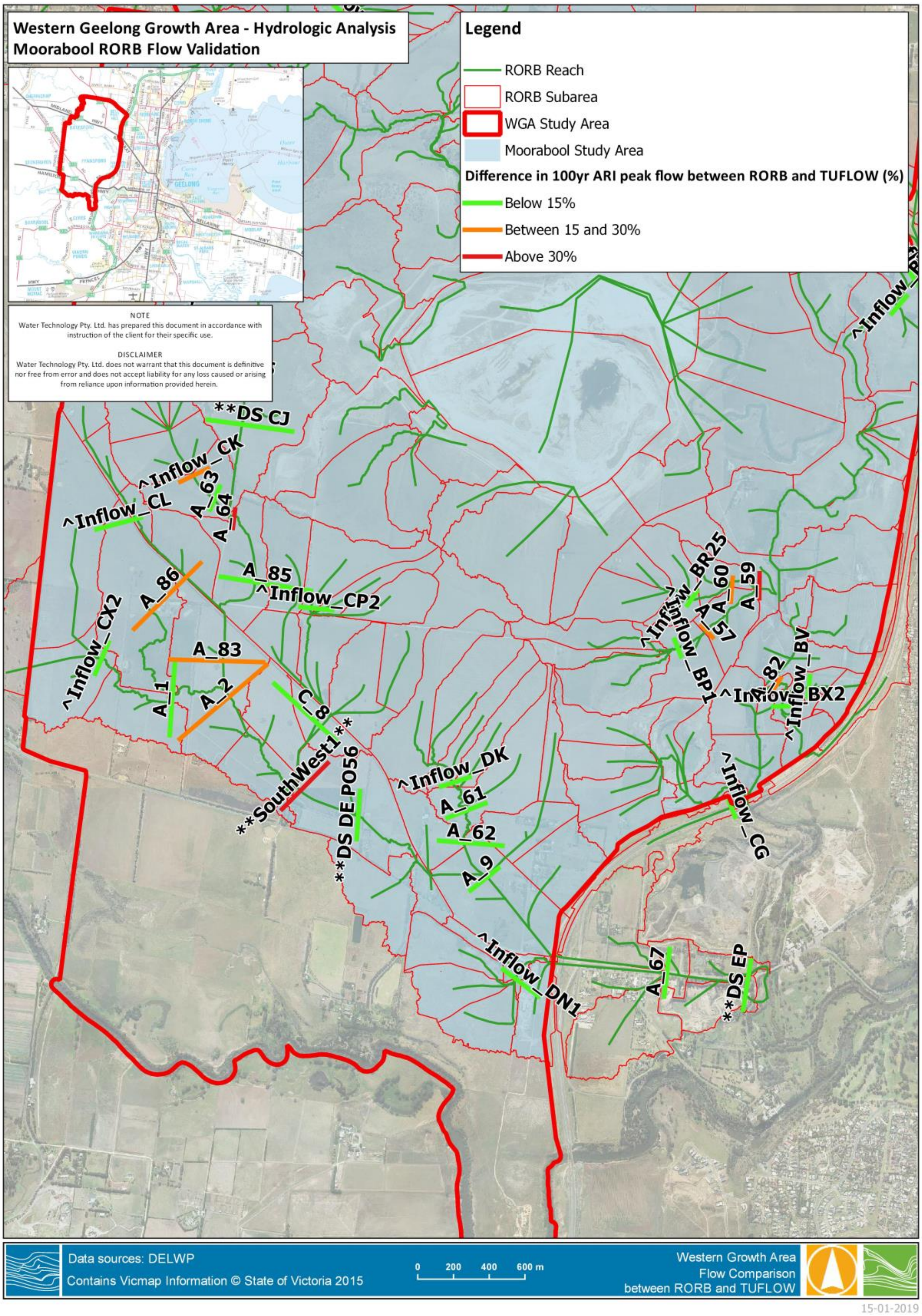


*Figure 5-7 Flow Estimation Points for Comparison (1%) – Moorabool River Catchment (Southern section)*

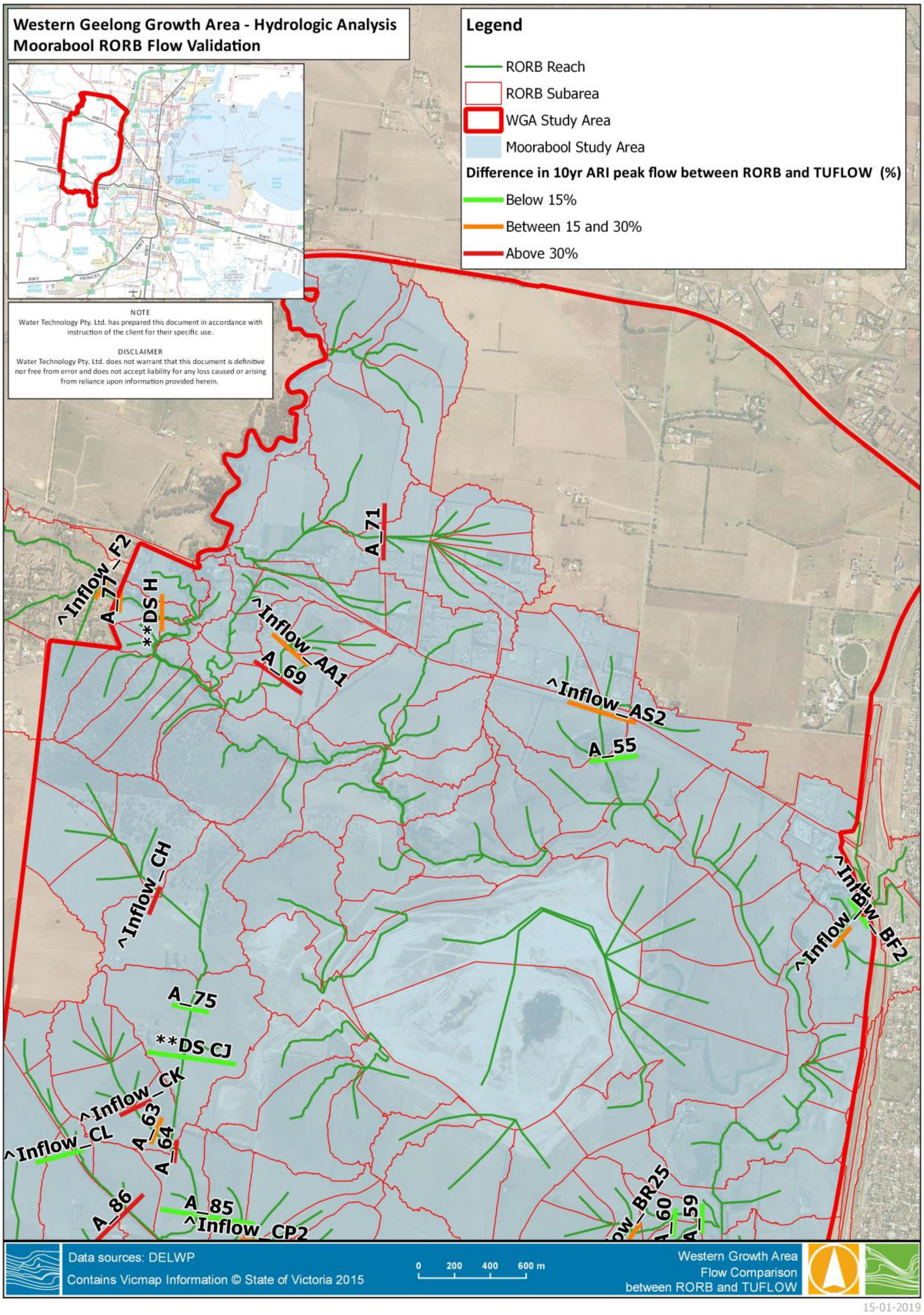


*Figure 5-8 Flow Estimation Points for Comparison (10%) – Moorabool River Catchment (Northern section)*

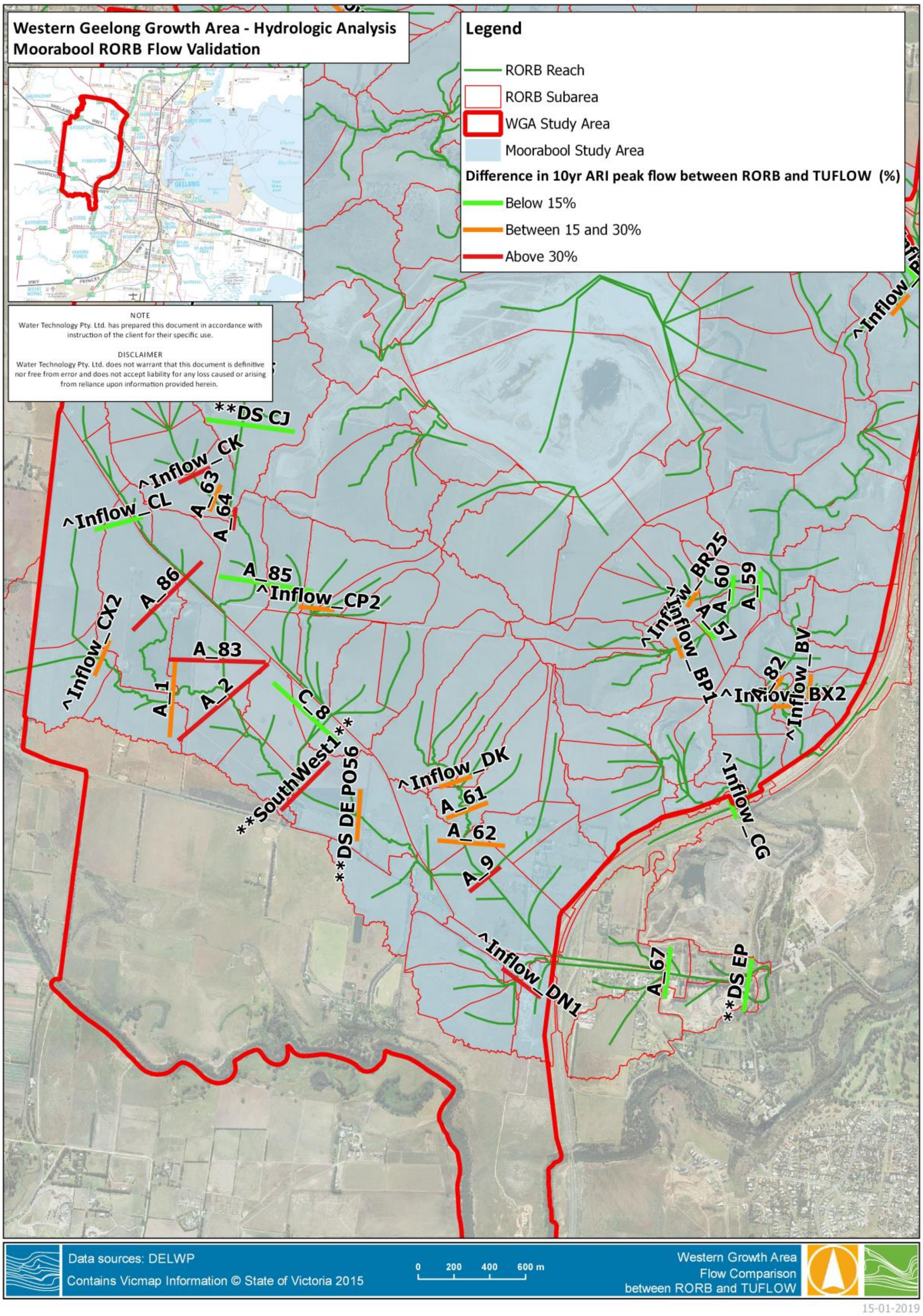


*Figure 5-9 Flow Estimation Points for Comparison (10%) – Moorabool River Catchment (Southern section)*

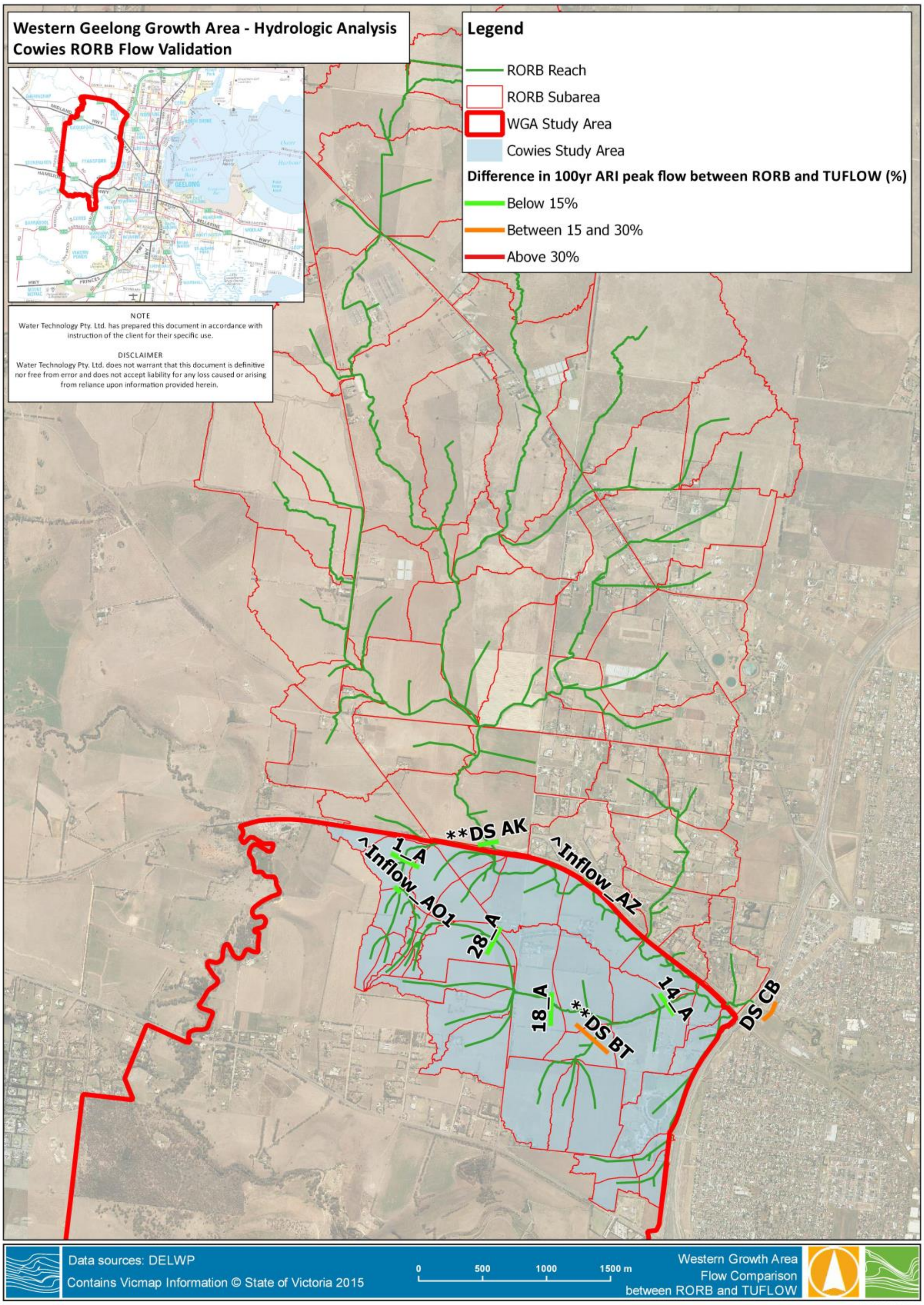


*Figure 5-10 Flow Estimation Points for Comparison (1%) – Cowies Creek Catchment*

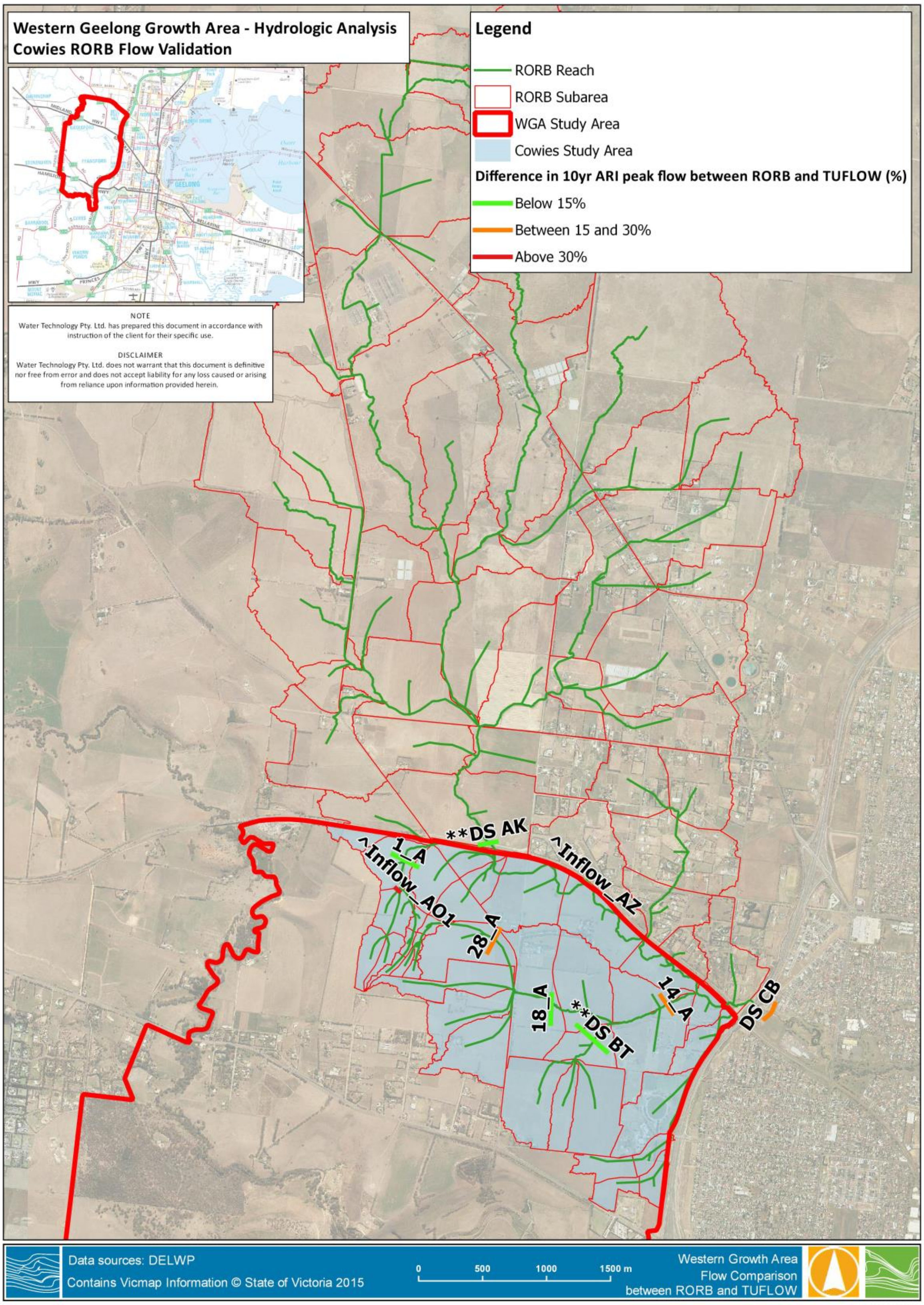


*Figure 5-11 Flow Estimation Points for Comparison (10%) – Cowies Creek Catchment*

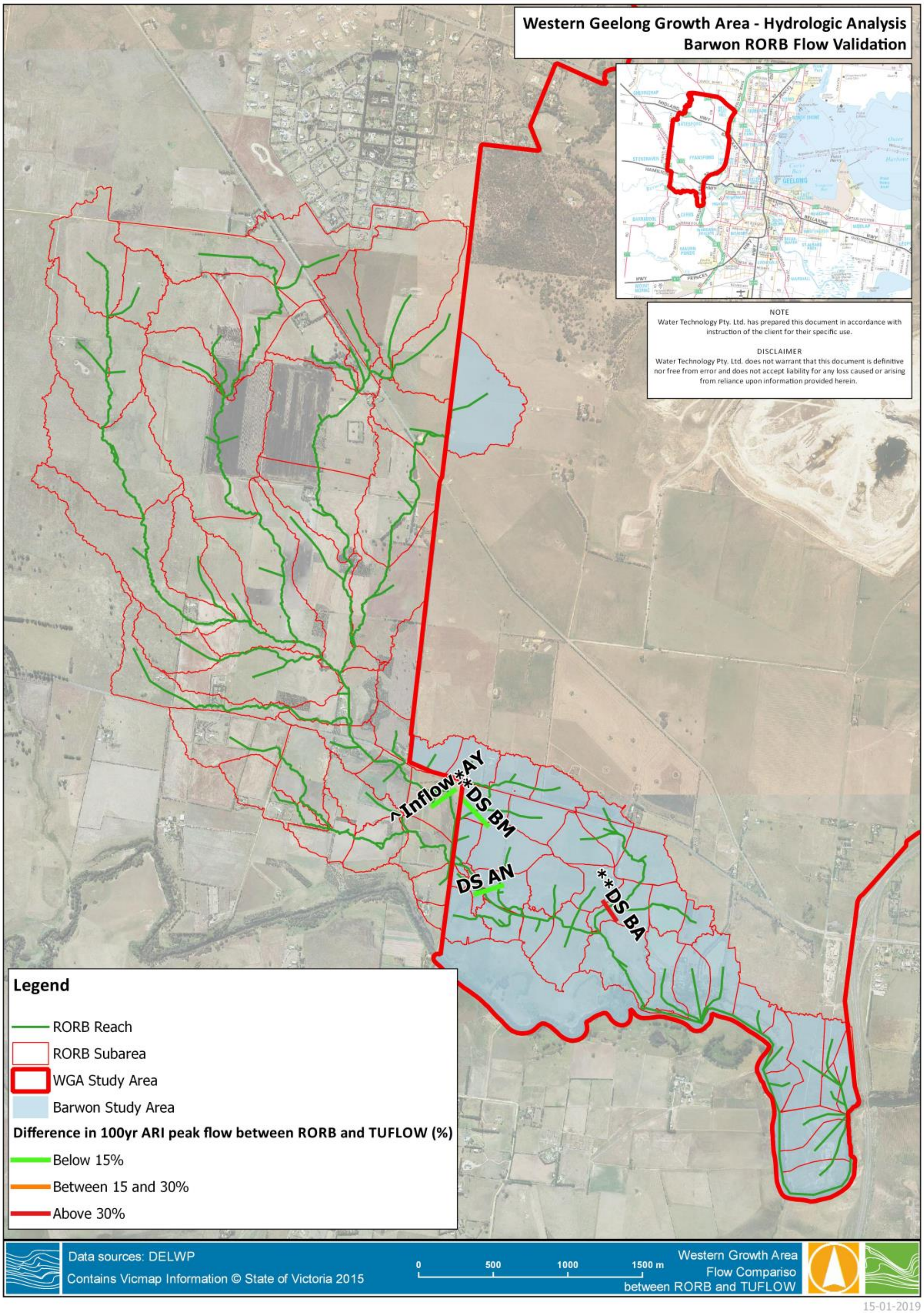


*Figure 5-12 Flow Estimation Points for Comparison (1%) – Barwon River Catchment*

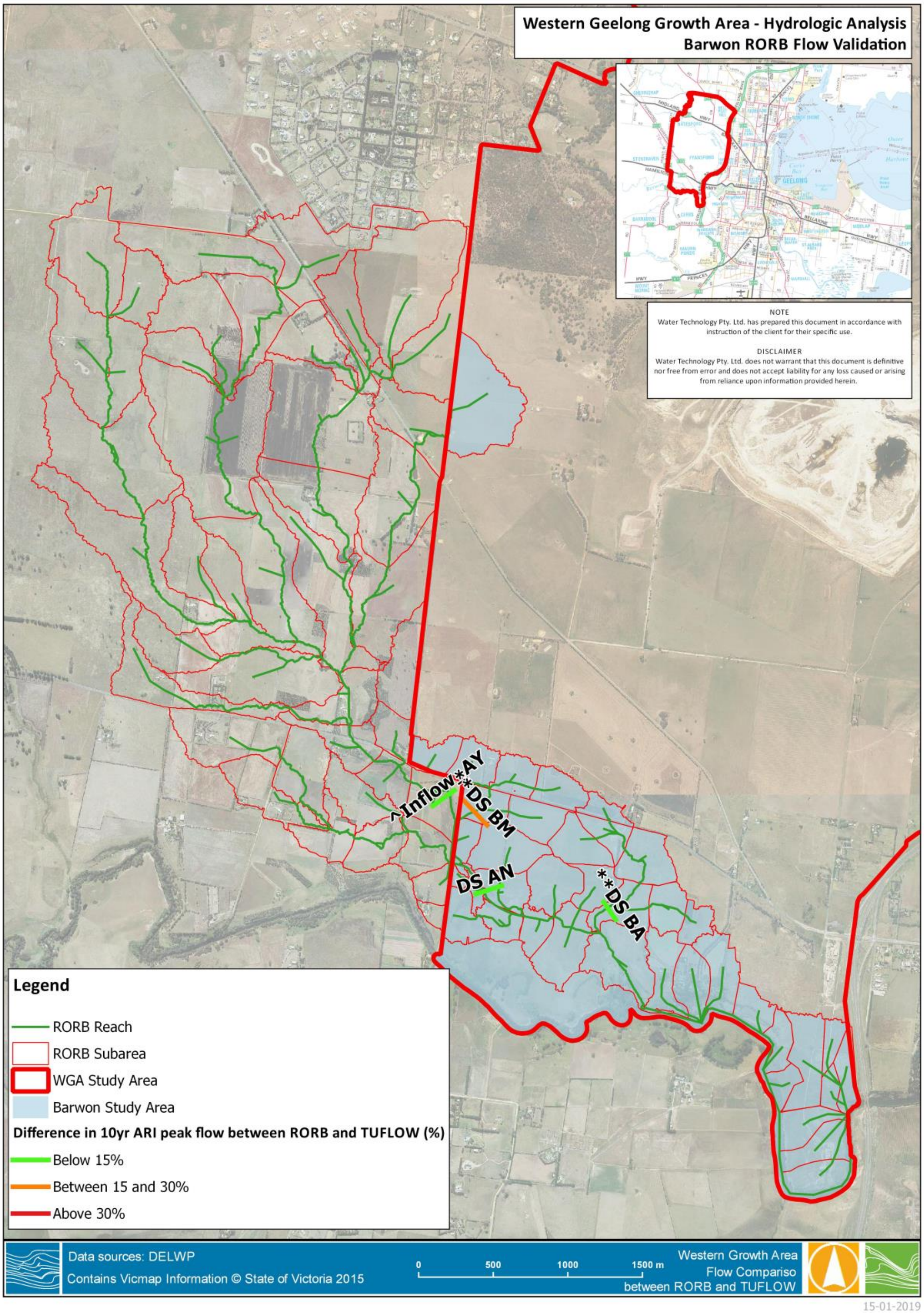


*Figure 5-13 Flow Estimation Points for Comparison (10%) – Barwon River Catchment*

### 5.3.3 Flood Results

Figure 5-14 to Figure 5-19 show the indicative results from the hydraulic modelling for the existing conditions.

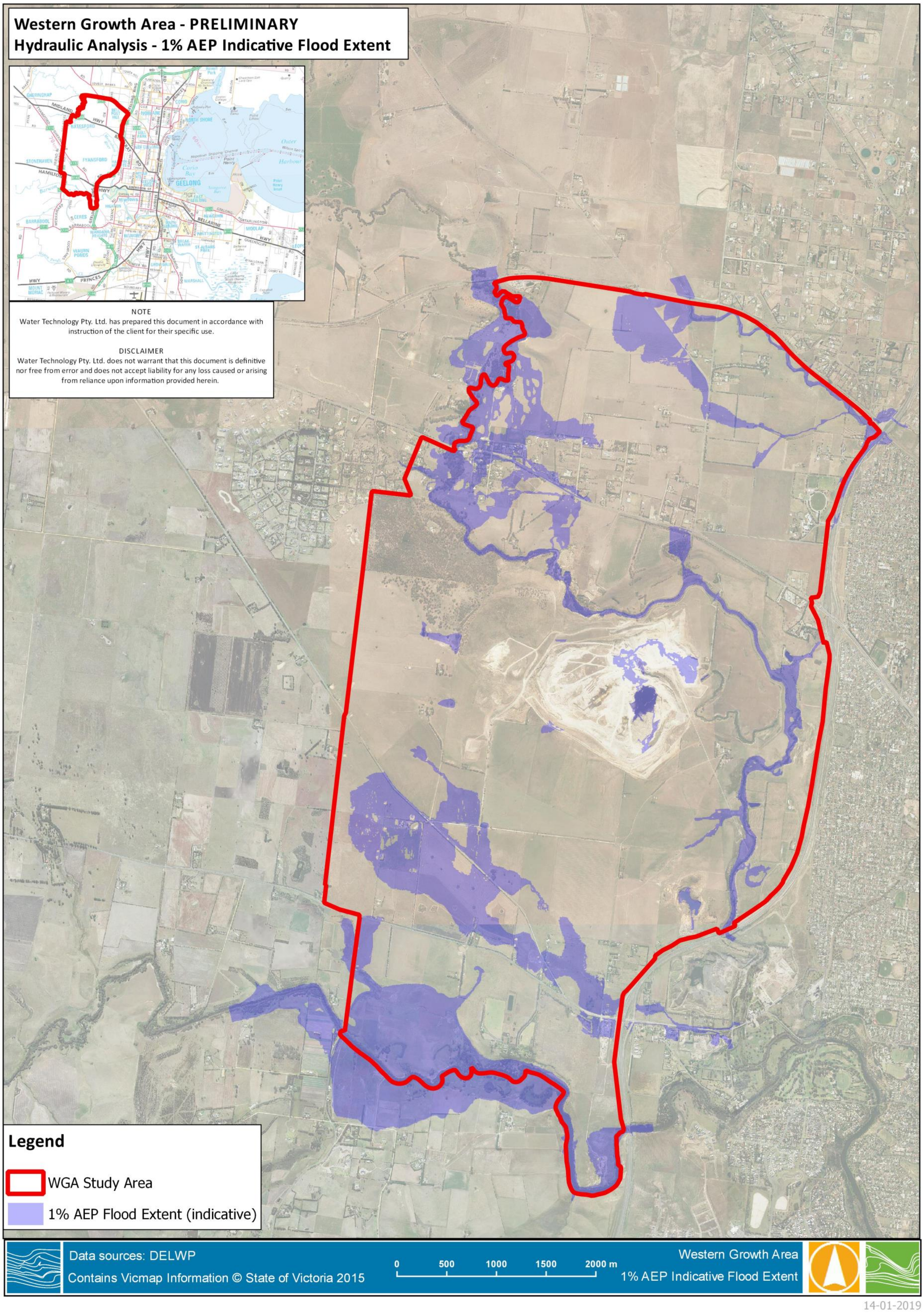


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*Figure 5-14 Hydraulic Model Indicative Flood Extent Results 1% AEP Extent*

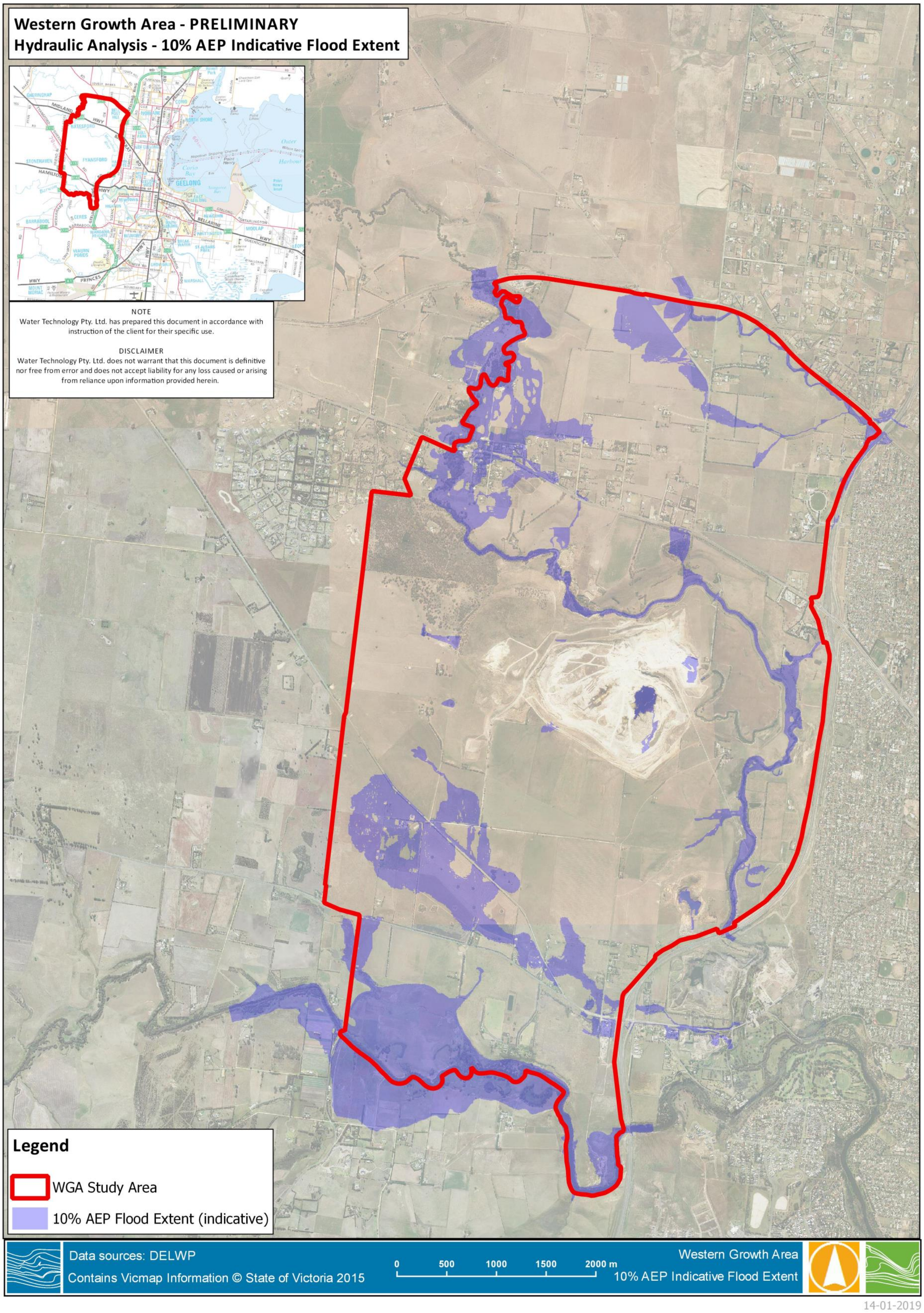


*Figure 5-15 Hydraulic Model Indicative Flood Extent Results 10% AEP Extent*

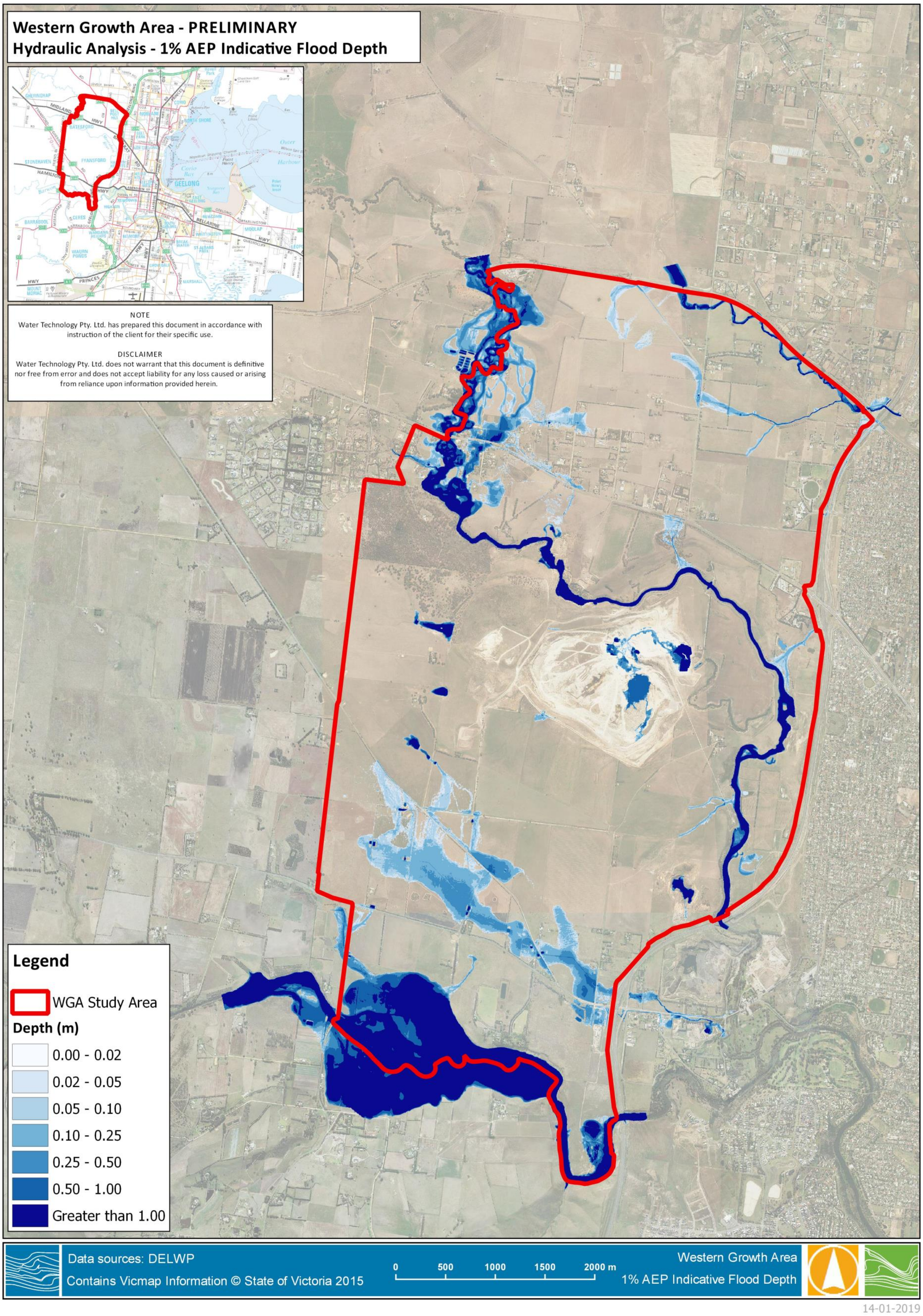


*Figure 5-16 Hydraulic Model Indicative Flood Depth Results 1% AEP Extent*

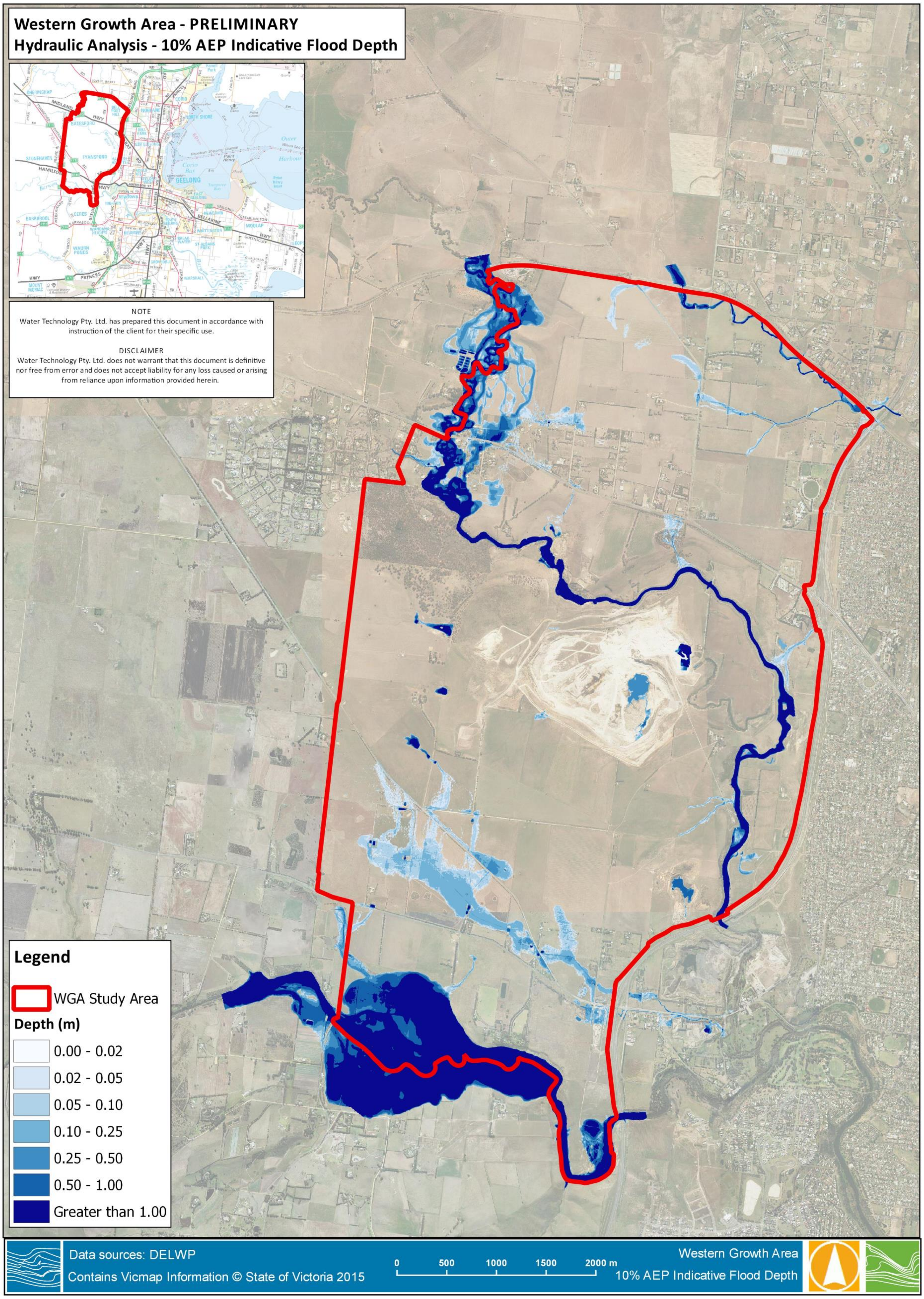


*Figure 5-17 Hydraulic Model Indicative Flood Depth Results 10% AEP Extent*

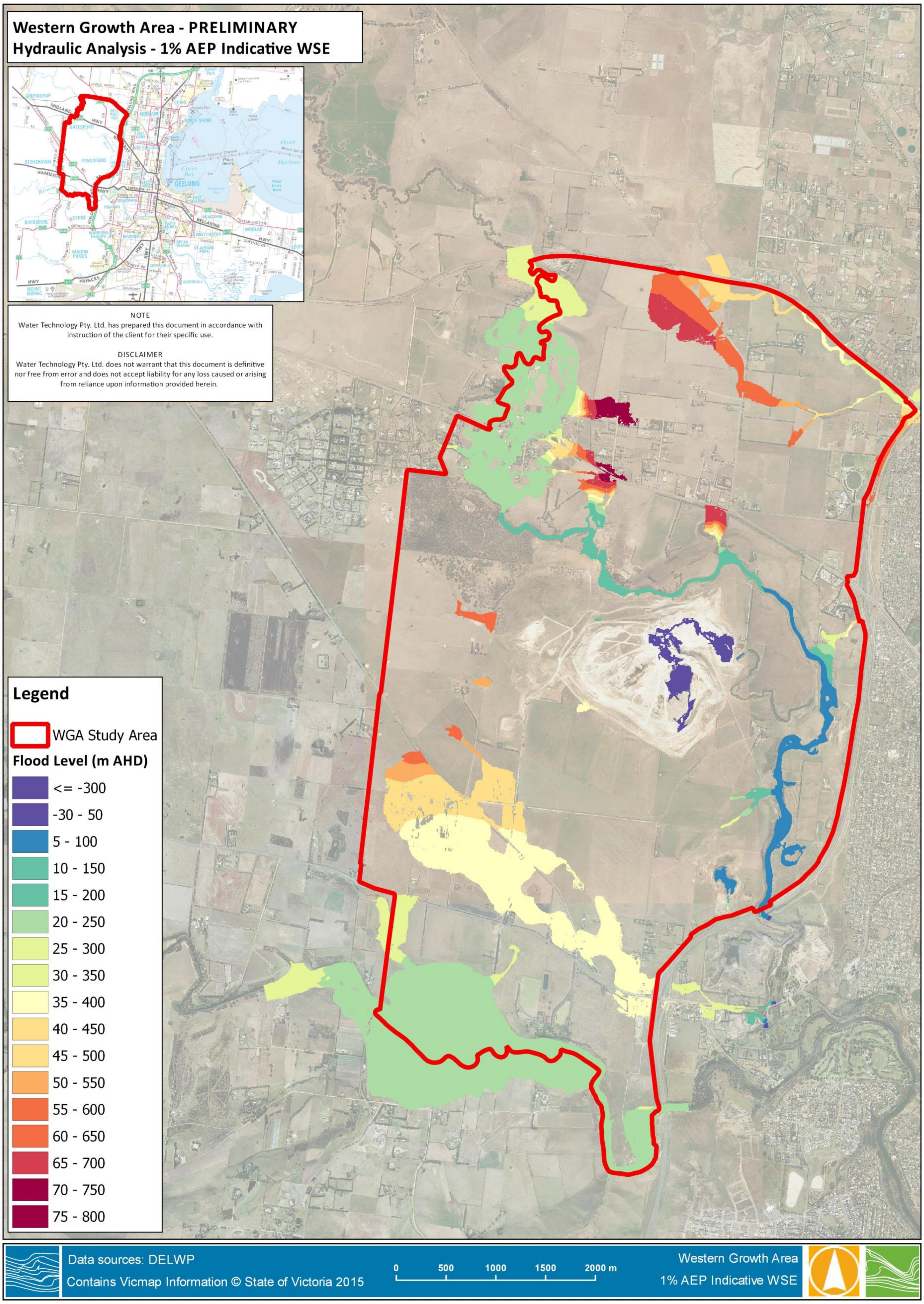


*Figure 5-18 Hydraulic Model Water Surface Elevation (WSE) Indicative Results 1% AEP Extent*

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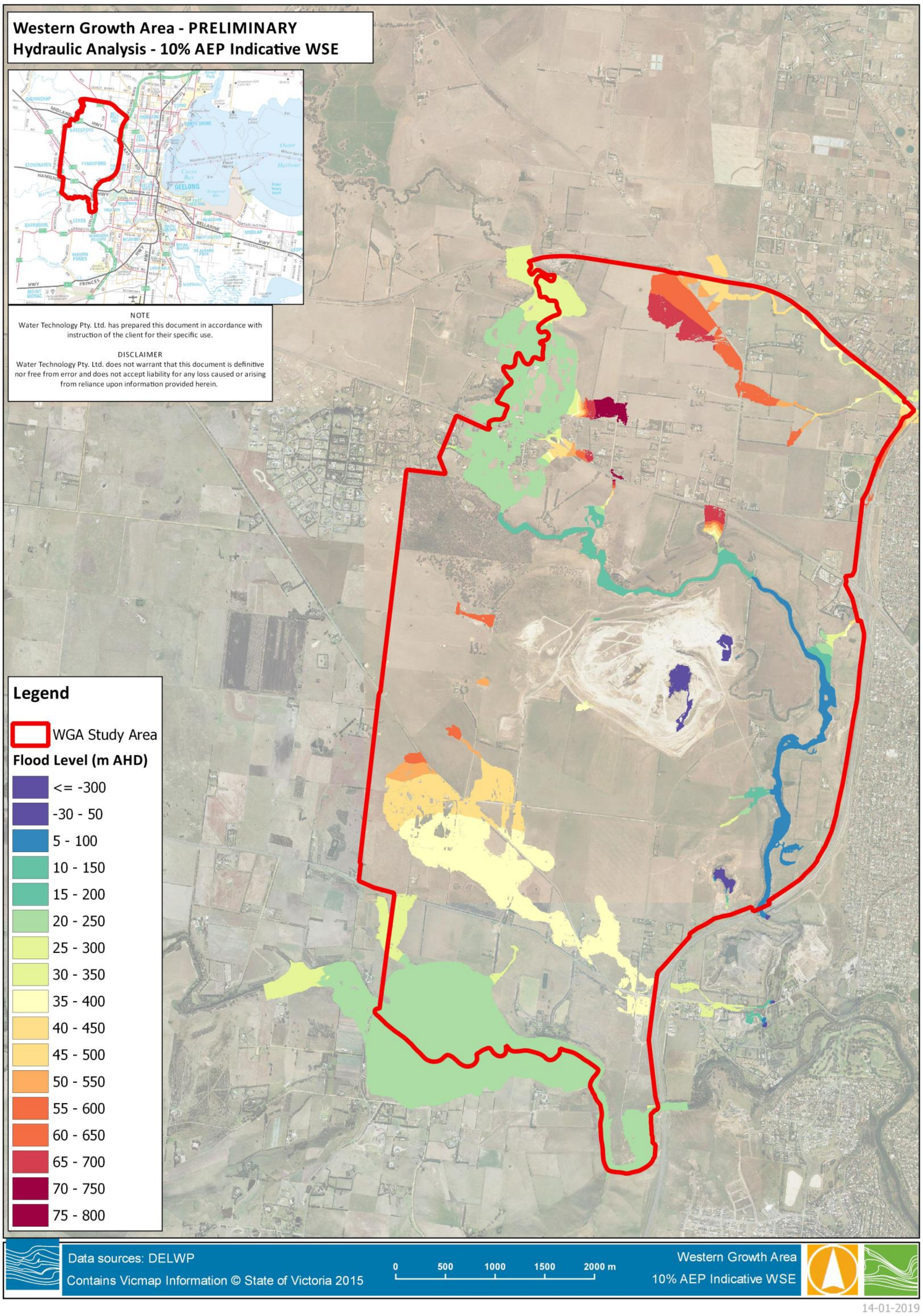


*Figure 5-19 Hydraulic Model Water Surface Elevation (WSE) Indicative Results 10% AEP Extent*

# 6 SUMMARY AND CONCLUSIONS

Existing design flood conditions within the Geelong WGA have been defined through detailed hydrologic and hydraulic modelling.

Three RORB hydrologic models covering the study area have been developed and reconciled to VicRoads Rational Method peak design flows. These models are based around the three main receiving waterways; the Barwon River, Moorabool River and Cowies Creek. The RORB models are considered to provide reliable estimates of existing conditions design flood inflows across the study area catchments.

Three TUFLOW hydraulic models have been developed for the study area, with coverage corresponding to the three hydrologic models for the Barwon River, Moorabool River and Cowies Creek. A depth varying roughness approach has been used in the hydraulic model to simulate the much higher relative roughness experienced across grassed areas under very shallow overland flow depths. This approach allows for a more realistic representation of overland flow routing in areas of shallow depths by slowing the flood propagation speed.

The preliminary hydraulic model peak design flood flows have been compared to the hydrologic model peak design flows. Overall, the two methods match well, however there are certain areas within each catchment where the peak design flows are not well matched. In these areas there is typically a physical driver and logical explanation for such discrepancies (such as breakout flows that are not able to be accurately defined within the hydrologic model). These instances and explanations are described in the report.

The preliminary hydraulic model results have been used to produce existing conditions indicative flood inundation maps for the study area. This mapping can help inform the selection of proposed stormwater management options for developed conditions. It identifies where current main overland flow paths exist and in which areas development may be constrained.

It is noted that Council has adopted some performance-based criteria for outcomes within the developed conditions assessment for the next part of the Phase 1 investigations. Key performance criteria include:

- Assets to be sized considering the potential loss in existing floodplain storage.
- Downstream impacts for volume to be assessed to the point of waterway connection.
- Drainage strategy and developed conditions to be reflective of designated waterways and buffer requirements and the point where piped infrastructure commences.

The RORB hydrologic models that have been developed are generally considered to be appropriate to enable these criteria to be met. That is, developed-conditions RORB models will be suitable in most cases to locate and size stormwater treatment measures across the catchment. However, it is recognised that in some situations hydraulic modelling will be required. This may be anything from simple one-dimensional tools to full two-dimensional hydraulic modelling in TUFLOW.

One particular example of this is the catchment that runs along the south side of the Hamilton Highway. In this catchment there is widespread, shallow, overland flow. In this area, RORB is not able to accurately predict existing flow characteristics or the likely impacts of potential future development in that area. However, the RORB model was calibrated to match the TUFLOW peak 1% AEP flow at the outlet to the Geelong Ring Road in order to size retarding basins appropriately in the Phase 3 Flood Impacts Assessment.

The developed conditions assessment being prepared is a separate document that will refer to this existing conditions report.

# APPENDIX A
# FLOOD FREQUENCY ANALYSIS

# A-1 Barwon River

## A-1-1 Barwon River at Pollocksford Gauge

The Barwon River gauge at Pollocksford (site No. 233200) is located within a relatively confined section of the Barwon River. At this location, the size of the upstream catchment is estimated to be approximately 2,713 km². Gauging at this location commenced in 1906, with the gauge itself having moved several times since records began. Table A-1 shows, based on the period of record, the various locations and types of structures for this gauge. Table A-2 identifies the variation in regulating weir height across the various sites.

*Table A-1 Stream Gauge Site Record*

| Record Period | Location | Control Structure | Recorder |
|---|---|---|---|
| **1906 - 1922** | Site A | Gravel and Clay | Manual Daily Readings |
| **1922 - 1969** | No Record | No Record | No Record |
| **1969 - 1972** | Site B | Rock and Boulders | Recorder |
| **1972 - Current** | Site B | Concrete Hump | Recorder |

*Table A-2 Gauge Weir Height Record*

| Record Period | Gauge Height at Zero |
|---|---|
| **May 1969 to July 1986** | 24.074 Meters AHD |
| **July 1986 to October 1986** | 24.156 Meters AHD |
| **October 1986 to Current** | 24.074 Meters AHD |

While the gauge record at this location extends from 1906 to current day, there is a significant period of 'no record' from 1922 to 1969. This is observed in the continuous height and flow data as displayed on the Victorian water data warehouse and is clearly below.

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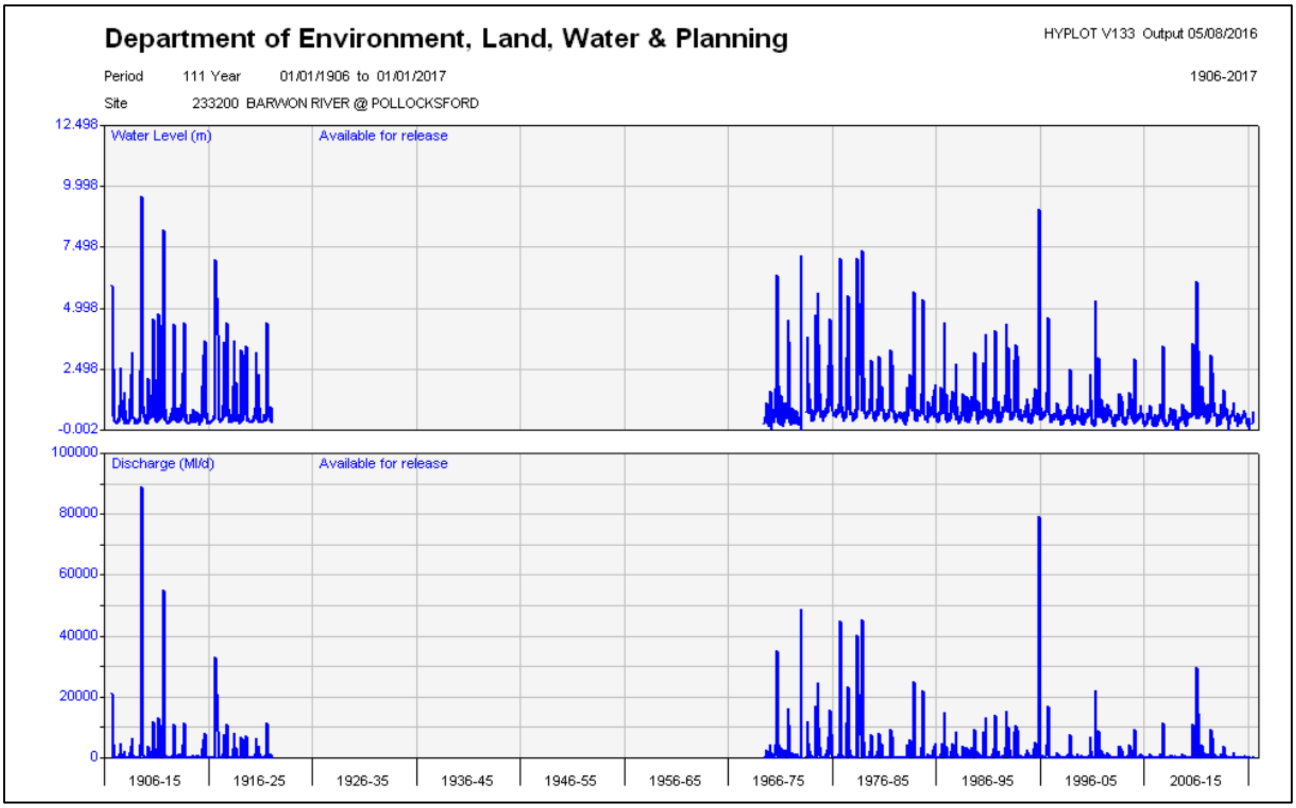


*Figure A-1 Stream Gauge Flows*

The current rating table at this location was last updated in October 2012 (Figure A-2). The plotted flows against the current rating table clearly show the change in gauge recording, as observed by the clear banding within the plotted recorded data as show in Figure A-3.

Based on the period of absent data and several catchment gauge alterations between 1969-1972, namely the movement of the gauge to its current location with constructed concrete hump and the construction of the West Barwon Dam, the period of record used for the flood frequency assessment (FFA) was decided to be 1973 – 2015 inclusive.

The use of the flow record from 1973 onwards for the FFA onwards is consistent with existing work undertaken by Tony Jones of CCMA[9] and has been selected due to being post-construction of the major upstream storages. This methodology was discussed in a Barwon flood study technical meeting attended by CCMA and CoGG at project inception and agreed as appropriate. Sensitivity testing using the full gauge records will be undertaken as part of the Barwon study.

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[9] Corangamite CMA. 2016. Barwon Consultancy Brief for Flood Risk Management Study Lower Barwon River and Lower Moorabool River Appendix B – Summary of Hydrology Analysis for the Lower Barwon River

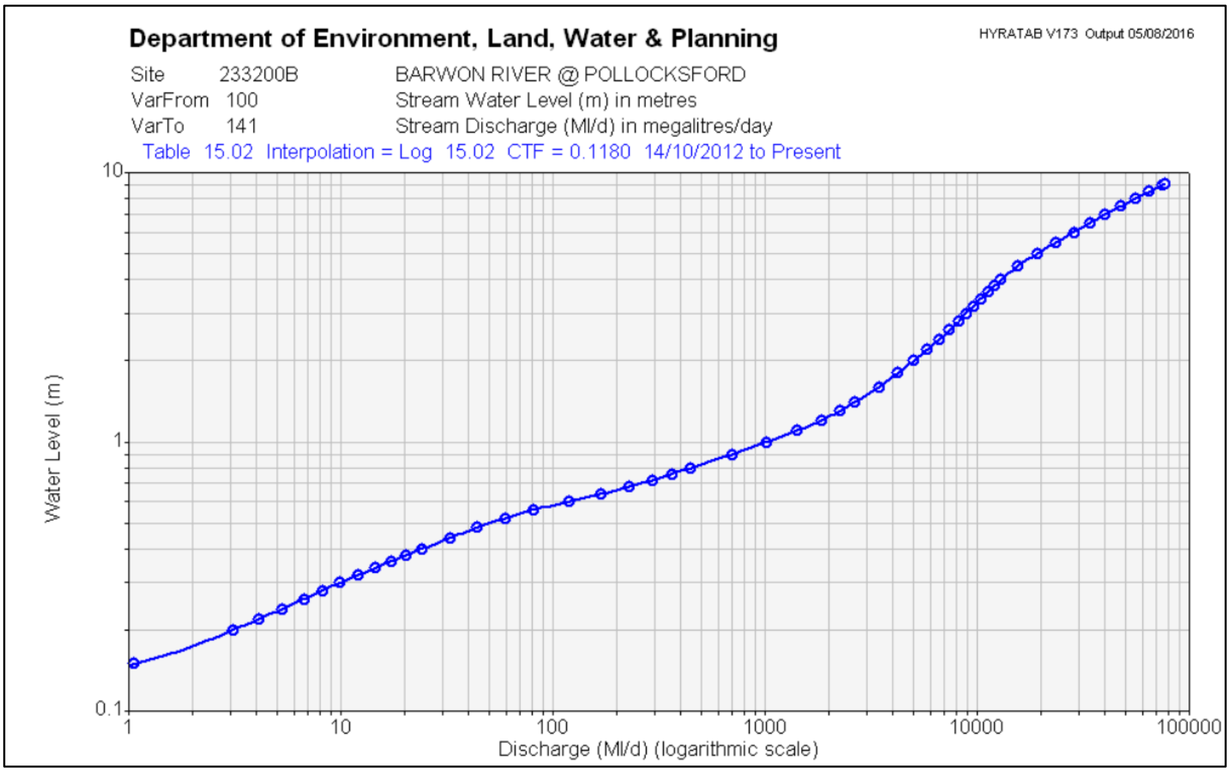



*Figure A-2 Barwon River at Pollocksford Gauge Rating Table*

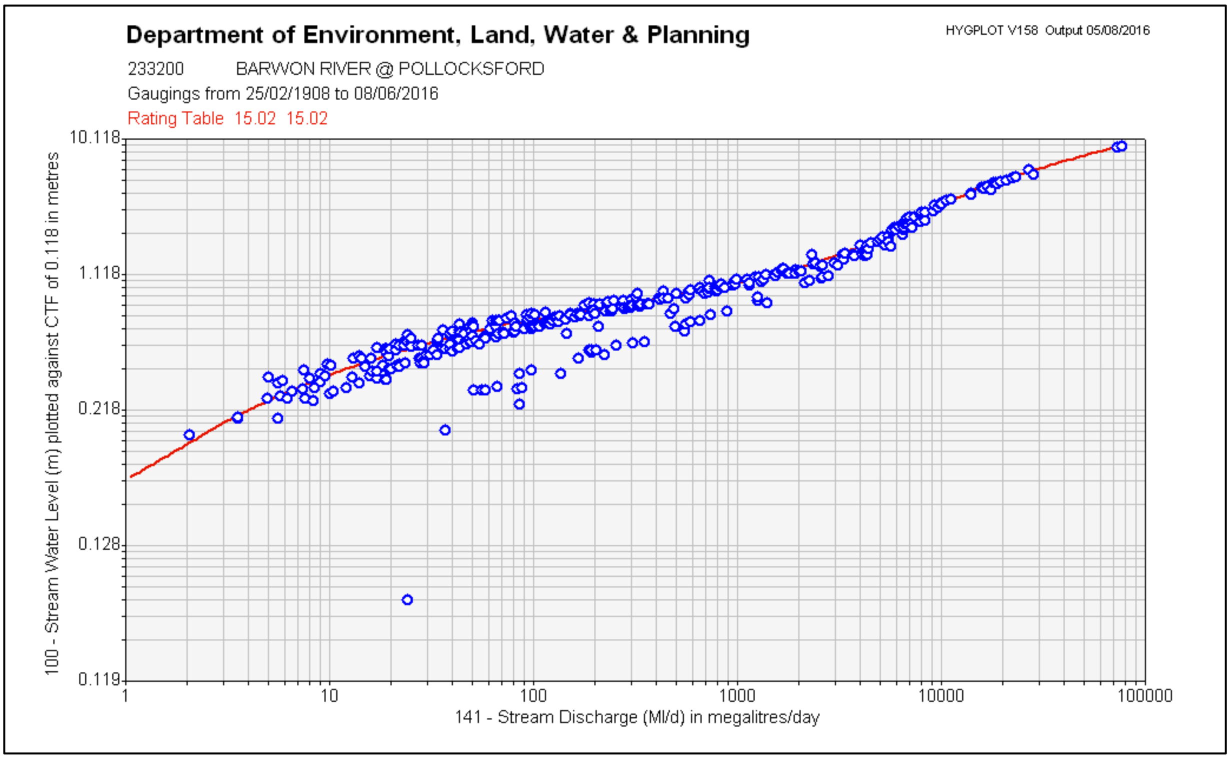



*Figure A-3 Plotted Rated Historical Flows*

## A-1-2 Design Flow Estimates

An annual series FFA was completed at the Barwon River at Pollocksford gauge as part of this study to determine design flow estimates for input into the hydraulic model. The available period of instantaneous record at this gauge is from 1906 to the present, however only the period from 1973-2015 will be used for this assessment as previously discussed.

This period of gauge record was used to complete an annual series FFA in FLIKE. The analysis was completed on raw annual peak flow series and a modified annual series with low flow censoring to remove years of low flow using the Multiple Grubbs Beck test. This determined a low flow threshold of 30.3 $m^3/s$, removing 9 years from the 43-year record.

The top 5 ranked (highest to lowest) flow events are shown in Table A-3 below.

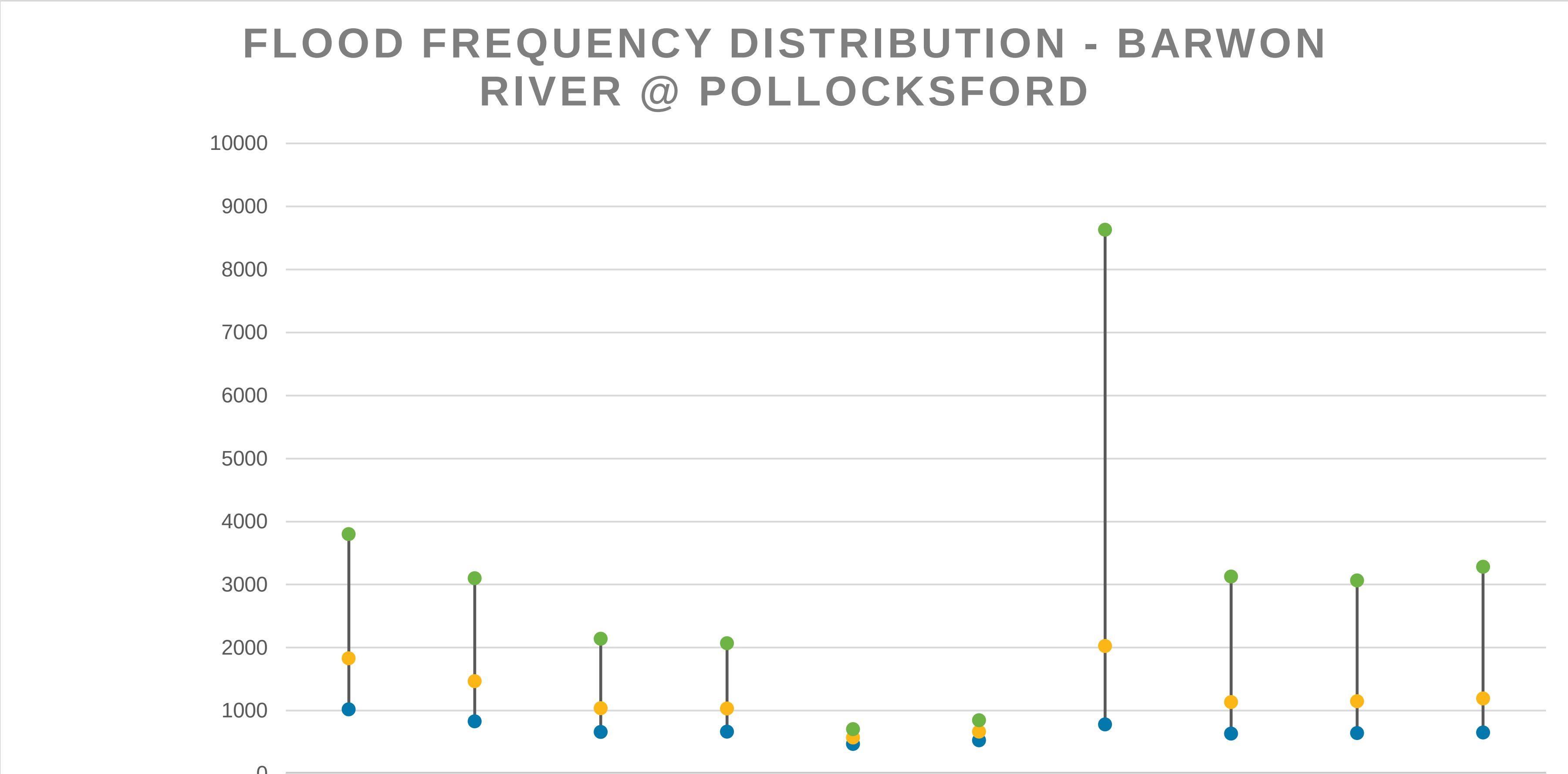
*Table A-3 Ranked Peak Historical Flow Events*

| Year | Peak Flow ($m^3/s$) | Peak Flow (ml/d) |
|---|---|---|
| **1995** | 915 | 79030 |
| **1973** | 564 | 48722 |
| **1978** | 524 | 45247 |
| **1976** | 521 | 45026 |
| **2011** | 468 | 29460 |

The Log Normal (LN), Log Pearson Type 3 (LP3), Generalised Extreme Value (GEV), Generalised Pareto GP) and Gumbel distributions were tested. Of these distributions, the LP3 and GP matched well for both the raw and censored annual series, while GEV matched better using the censored annual series. A comparison of the FFA results for all distributions for the raw and censored annual series are shown in Table A-4. The 1% AEP flow estimate and associated confidence limits are compared in Figure A-4 for the range of FFA distributions tested. The plot shows the GEV Raw and low flow censored data have the narrowest confidence limits, followed by the LP3 censored distribution. The FFA plots for the LP3 raw, LP3 censored, GP censored and GEV censored provide the best fit. The available gauge record and the resulting distributions are shown in Figure A-5, Figure A-6, Figure A-7 and Figure A-8 respectively.

*Table A-4 Flood Frequency Comparison*

| Design Event AEP | Log Normal | | LP3 | | Gumble | | GEV | | GP | |
|---|---|---|---|---|---|---|---|---|---|---|
| | **Raw** | **Censored** | **Raw** | **Censored** | **Raw** | **Censored** | **Raw** | **Censored** | **Raw** | **Censored** |
| **20%** | 255.7 | 247.0 | 264.2 | 262.4 | 248.22 | 266.5 | 246.7 | 256.3 | 273.6 | 271.9 |
| **10%** | 458.5 | 418.9 | 419.1 | 415.5 | 327.24 | 363.7 | 425.6 | 393.7 | 426.1 | 425.7 |
| **5%** | 742.5 | 647.9 | 591.8 | 587.0 | 403.03 | 456.9 | 697.5 | 562.8 | 605.2 | 610.5 |
| **2%** | 1277.5 | 1058.6 | 841.1 | 835.8 | 501.14 | 577.6 | 1291.15 | 852.5 | 891.1 | 912.8 |
| **1%** | 1834.3 | 1468.6 | 1041.36 | 1037.1 | 574.65 | 668.1 | 2027.82 | 1138.1 | 1151.66 | 1195.3 |




| | Log Normal Raw | Log Normal Censored | Log Pearson 3 Raw | Log Pearson 3 Censored | Gumbel Raw | Gumbel Censored | GEV Censored | Generalised Pareto Raw | Generalised Pareto Censored | |
|---|---|---|---|---|---|---|---|---|---|---|
| Lower Confidence Limit | 1021.47 | 833.86 | 666.3 | 666.86 | 472.58 | 533.14 | 782.91 | 637.39 | 646.27 | 657.38 |
| Mean | 1834.25 | 1468.56 | 1041.36 | 1037.09 | 574.65 | 668.05 | 2027.82 | 1138.12 | 1151.66 | 1195.31 |
| Upper Confidence Limit | 3803.1 | 3103.5 | 2143 | 2074.3 | 707.6 | 849.5 | 8633.4 | 3130.5 | 3070.1 | 3284.7 |

*Figure A-4 Flood Frequency Comparison*

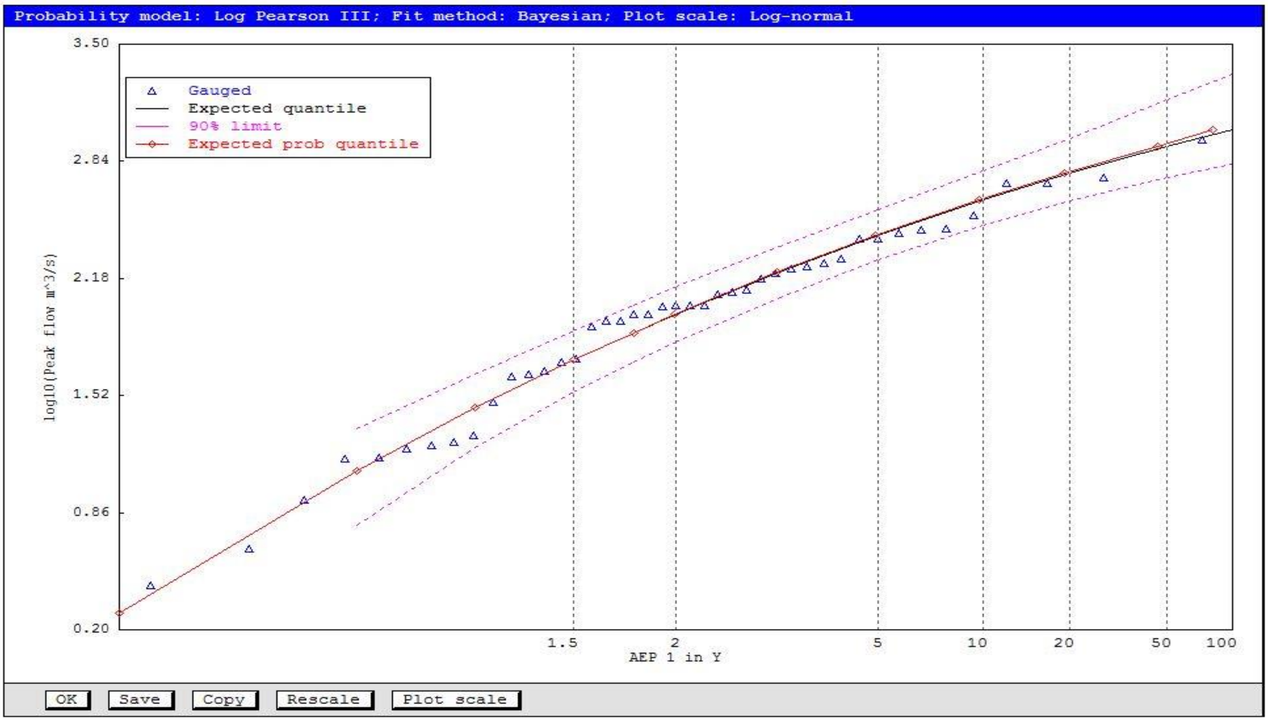



*Figure A-5 Log Pearson III (raw) Flood Frequency Distribution Plot*

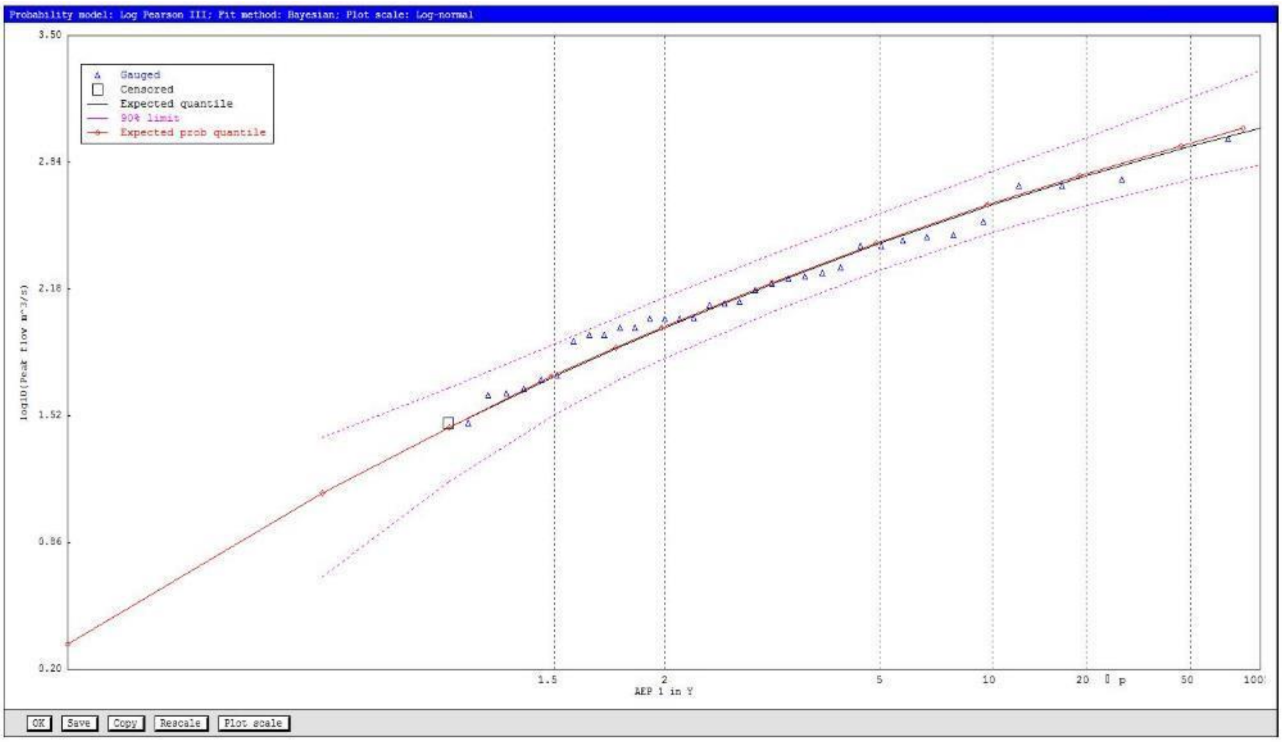



*Figure A-6 Log Pearson III (censored) Flood Frequency Distribution Plot*

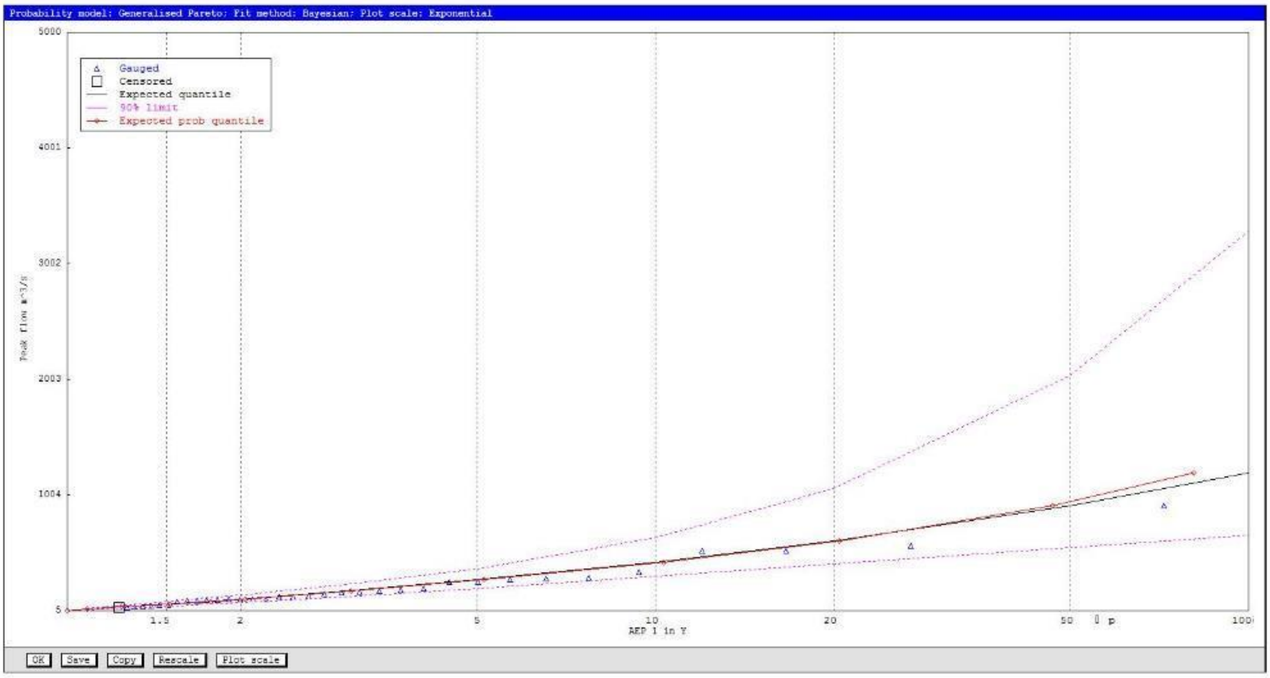



*Figure A-7 Generalised Pareto (censored) Flood Frequency Distribution Plot*

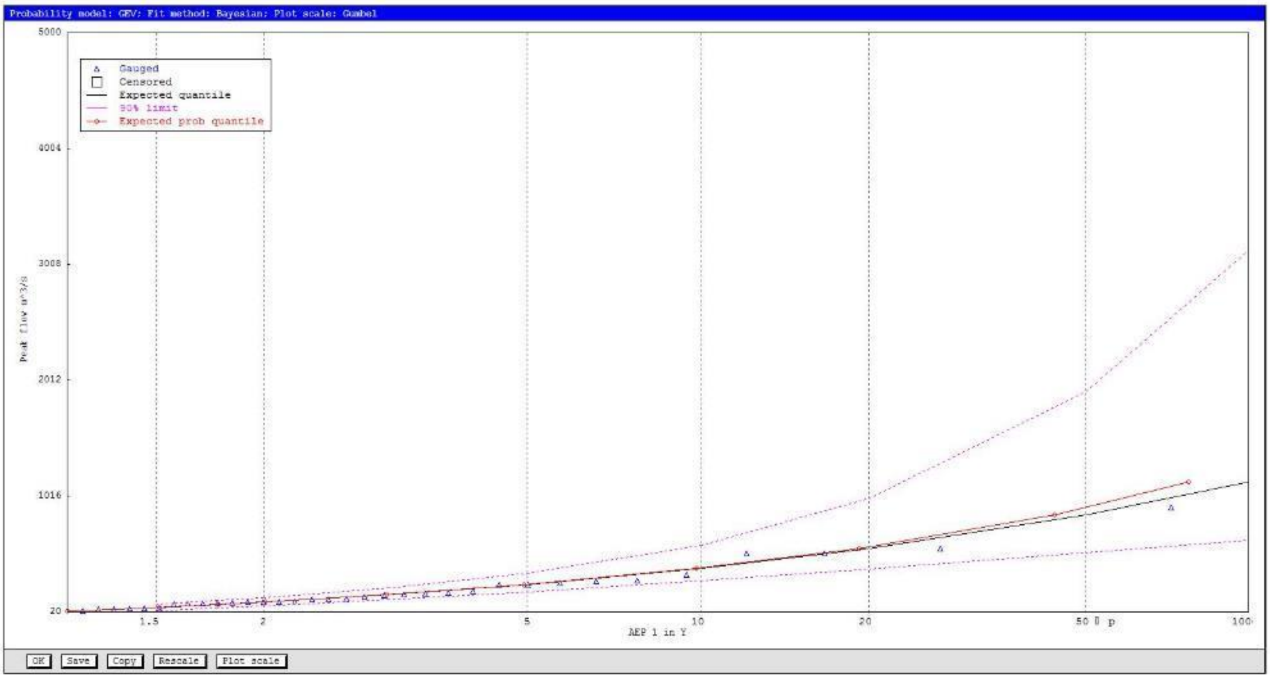



*Figure A-8 GEV (censored) Flood Frequency Distribution Plot*

## A-1-3 Design Flow Comparison

A number of methods were used to verify the flood frequency design flow estimates, with these outlined below. There is, in general, a good correlation between the flood frequency flows and the alternative methods.

The following regional prediction equation (Grayson et al, 1996) is based on the regression of 1% AEP flood flows for catchments on either side of the Great Dividing Range in Victoria:

$$Q_{100}(m^3/s) = 4.67A^{0.763}$$

The comparisons in Table A-5 indicate that the adopted 1% AEP design flow from the flood frequency analysis (using the LP3 distribution and low flow filtering) is of similar magnitude to the Rural Rational Method. The adopted design flows are therefore considered appropriate for this study.

Table A-5 *Regional Flow Estimate Comparison*

| Upstream Catchment Area (km²) | 1% AEP Estimated Peak Flow (m³/s) | | | |
|---|---|---|---|---|
| | **Water Technology FFA** | **CCMA FFA** | **Regional Equation** | **Rural Rational** |
| **2713** | 1037.1 | 1030.0 | 1945.5 | 648.2 |

## A-1-4 Historical Flow AEP Comparison

The table below (Table A-6) shows a comparison of the top 5 ranked flow events and the estimated AEP associated with the flood frequency assessment of the Barwon River at Pollocksford gauge. The highest of the recorded events (November 1995) has been estimated to have a AEP of 1.4% on the Barwon River at this location.

Table A-6 *Peak Flow AEP comparison*

| **Flood Event** | Peak Flow (ML/d) | Peak Flow (m³/s) | ARI (years) | AEP (%) |
|---|---|---|---|---|
| **November 1995** | 79030 | 915 | 72 | 1.4 |
| **February 1973** | 48722 | 564 | 27 | 3.7 |
| **November 1978** | 45247 | 524 | 17 | 5.9 |
| **October 1976** | 45026 | 521 | 12 | 8.3 |
| **January 2011** | 29460 | 341 | 9 | 11.1 |

## A-1-5 Final Adopted Design Peak Flows

As discussed earlier the Log Pearson III distribution with low flow filtering was determined to represent the best fit with the available historic data. The design flows in Table A-7 below were adopted as the final design flows for the Barwon River at Pollocksford for this study.

Table A-7 *Adopted Peak Flows*

| Design Event (AEP) | Log Pearson III with Low Flow Censoring ($m^3/s$) | | |
|---|---|---|---|
| | Peak Flow | Lower Confidence Limit | Upper Confidence Limit |
| **20%** | 262.4 | 190.0 | 373.9 |
| **10%** | 415.5 | 299.7 | 617.9 |
| **5%** | 586.9 | 413.1 | 921.8 |
| **2%** | 835.8 | 562.6 | 1502.4 |
| **1%** | 1037.1 | 666.9 | 2074.3 |

## A-1-6 Barwon River at Pollocksford Hydrograph Shape

Similar to the analysis of peak flows, an analysis on volume was undertaken to inform the choice of hydrograph shape for design flows at the Barwon River (at Pollocksford) streamflow gauge. In order to do this, the ratio of historic event peak flows to volumes was compared to the ratio of design peak flows to volumes determined by FFA.

The five largest events recorded at the Barwon River at Pollocksford gauge (November 1995, February 1973, November 1978, October 1976 and January 2011) are shown overlayed in Figure A-9. In all events the flood occurred over a three to five day period. The 1995, 1973, 1978 and 2011 events all have a similar shape, whilst the 1976 event seemed to rise initially and plateau prior to the main peak. Based on these hydrographs it was determined a four-day volume FFA would be completed. Further to this, based on the similar shape of the 1995, 1978 and 1976 events it was determined that a hydrograph shape similar to these events would likely be adopted.

The four-day volume FFA was completed using a variety of distributions, with the LP3 distribution the best fit of the recorded data. The four-day volume FFA results and comparisons for the design events are shown in Table A-8, with the historical events shown in Table A-9.

The volume to peak flow ratio of the 1976 historic events match the average ratio of the range of design events considerably better than some of the other historic event. The 1995 event provides an almost perfect fit to the 1% AEP ratio. Given that the 1995 hydrograph has a slightly lower ratio, and a better fit to the 1% AEP design ratio it was chosen as the donor hydrograph shape for design purposes.

The November 1995 hydrograph shape was scaled for each design event to match the peak and the volume of the design hydrograph from the FFA. Figure A-15presents the final design flood hydrographs.

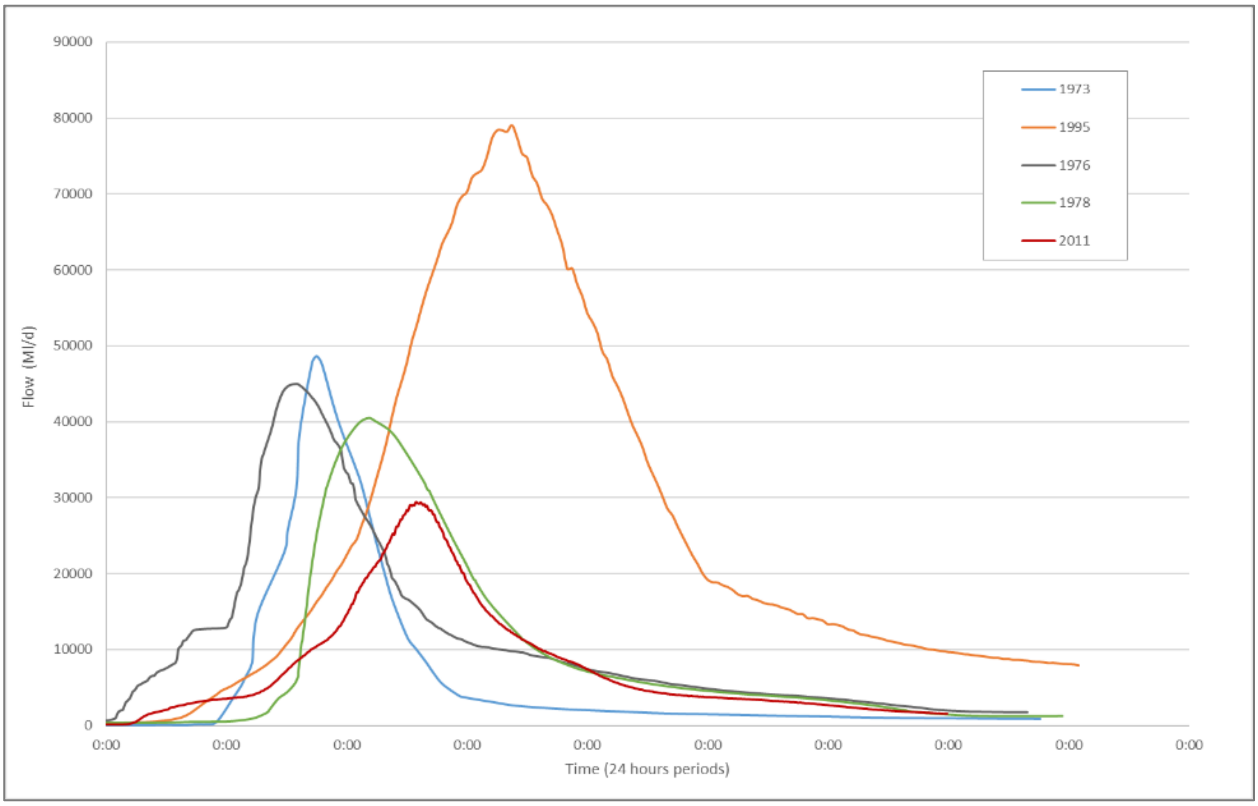



*Figure A-9 Historical Flow Events - Barwon River at Pollocksford*

Table A-8 *FFA Design Peak flow, Four-Day Volume and Ratios*

| AEP | 4 Day Volume (ML) | Peak Flow (ML/d) | Ratio |
|---|---|---|---|
| **20%** | 42711 | 22675 | 1.88 |
| **10%** | 59847 | 35900 | 1.67 |
| **5%** | 77394 | 50712 | 1.53 |
| **2%** | 101075 | 72217 | 1.40 |
| **1%** | 119233 | 89605 | 1.33 |

Table A-9 *Historical Peak Flow and Four-Day Volume Ratio Comparison*

| Historical Events | Peak Flow ML/day | 4 Day Volume (ML) | Ratio |
|---|---|---|---|
| **November 1995** | 79030 | 103523 | 1.31 |
| **February 1973** | 48722 | 46479 | 0.95 |
| **November 1978** | 45247 | 67119 | 1.48 |
| **October 1976** | 45026 | 71409 | 1.56 |
| **January 2011** | 29460 | 47795 | 1.62 |
| | | **Average** | 1.38 |

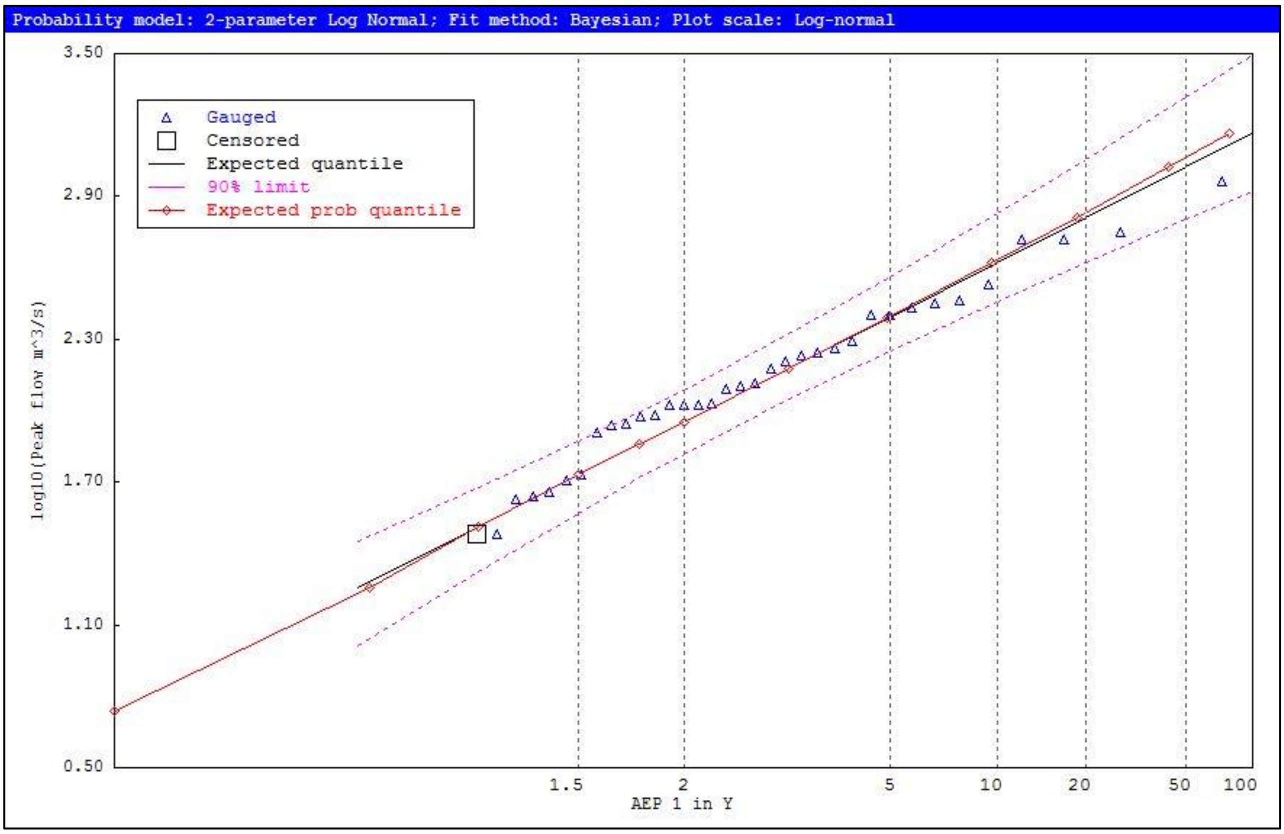



*Figure A-10 Log Normal – 4 Day Volume Distribution Plot*

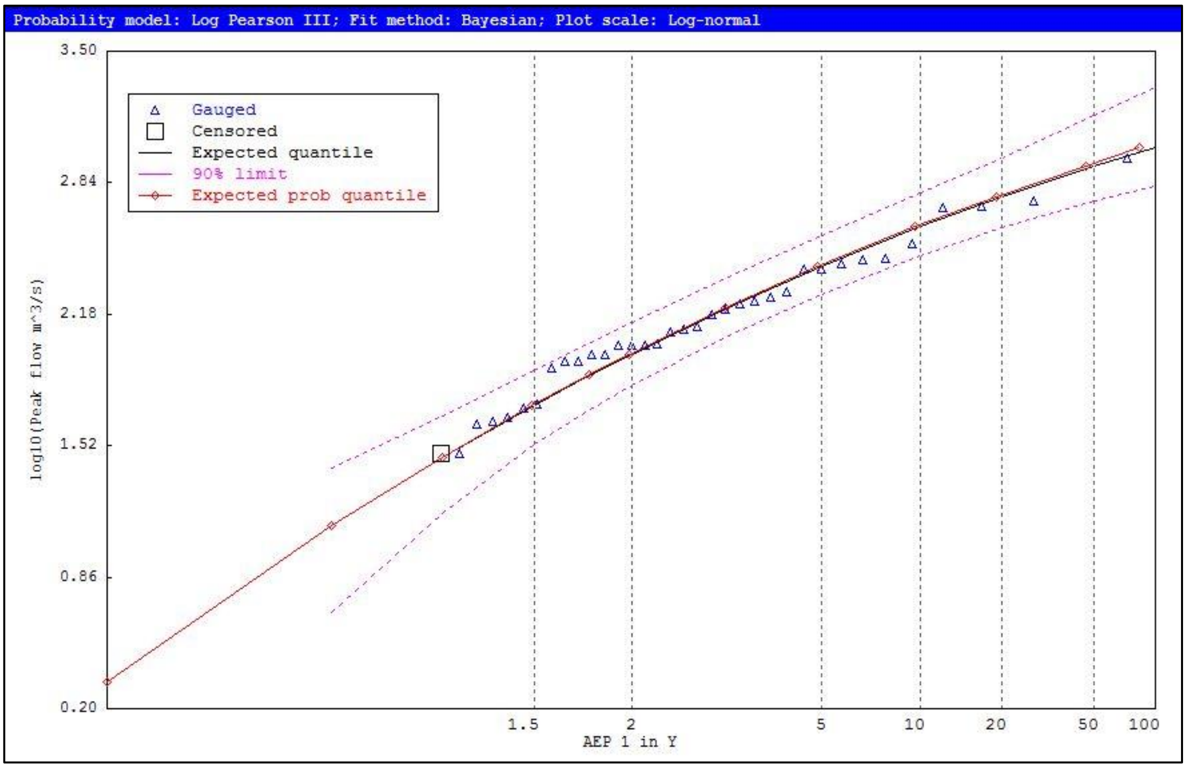



*Figure A-11 Log Pearson III – 4 Day Volume Distribution Plot*

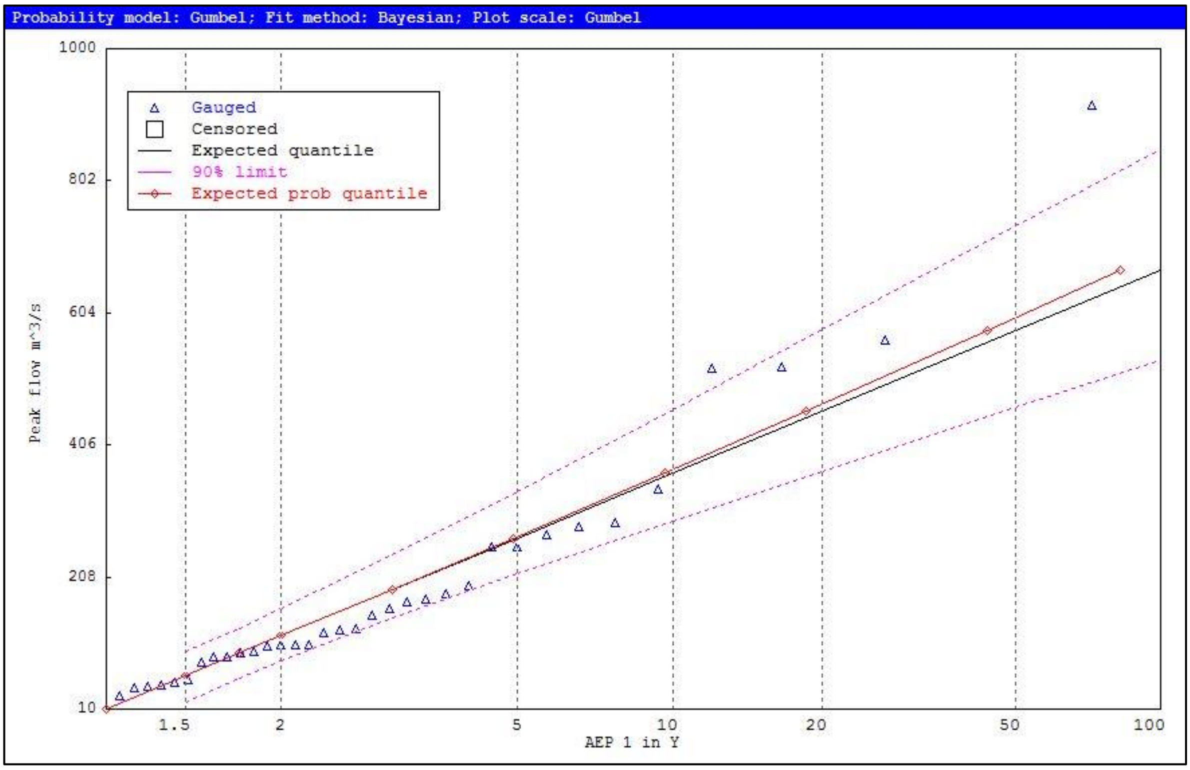



*Figure A-12 Gumble – 4 Day Volume Distribution Plot*

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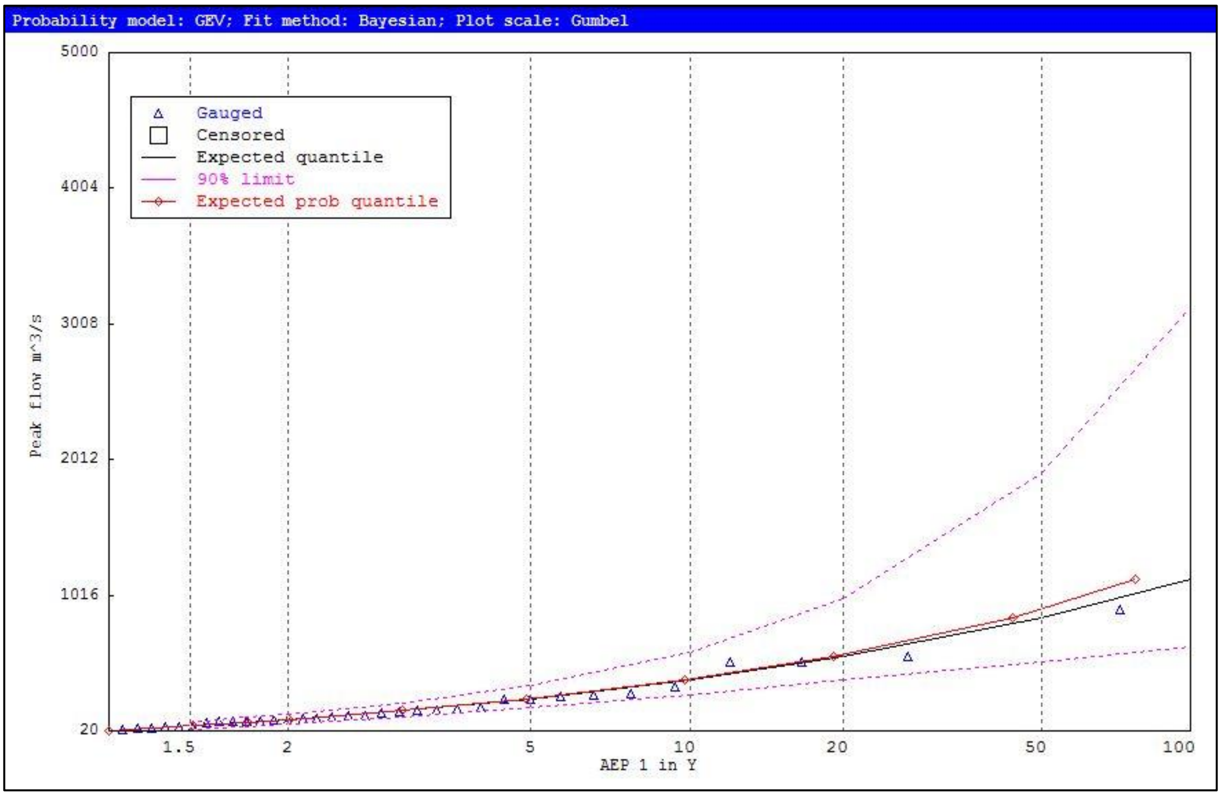


*Figure A-13 GEV - 4 Day Volume Distribution Plot*

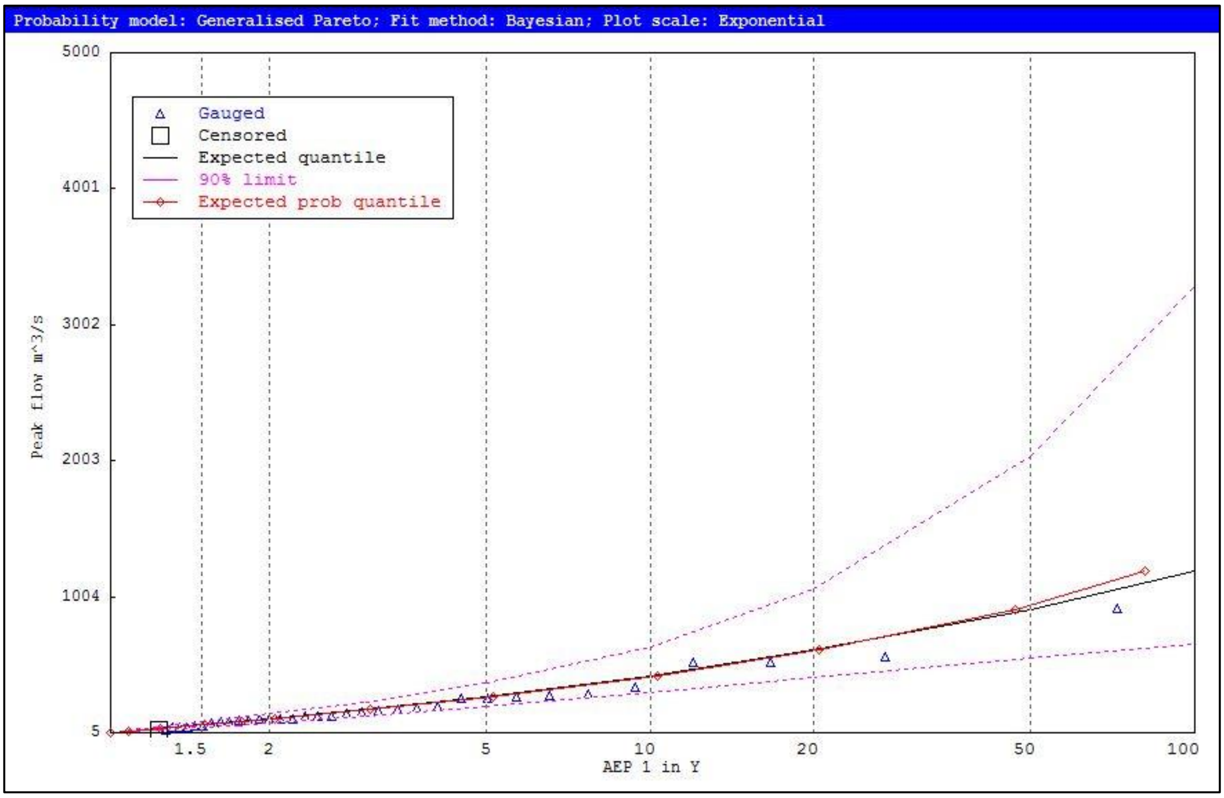


*Figure A-14 GP - 4 Day Volume Distribution Plot*

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*Table A-10 Adopted Design Peak flow and Four-Day Volume Ratio*

| AEP | 4 Day Volume (ML) | Peak Flow (ML/d) | Ratio |
|---|---|---|---|
| 20% | 31820 | 22675 | 1.40 |
| 10% | 50381 | 35900 | |
| 5% | 71167 | 50712 | |
| 2% | 101344 | 72217 | |
| 1% | 125746 | 89605 | |

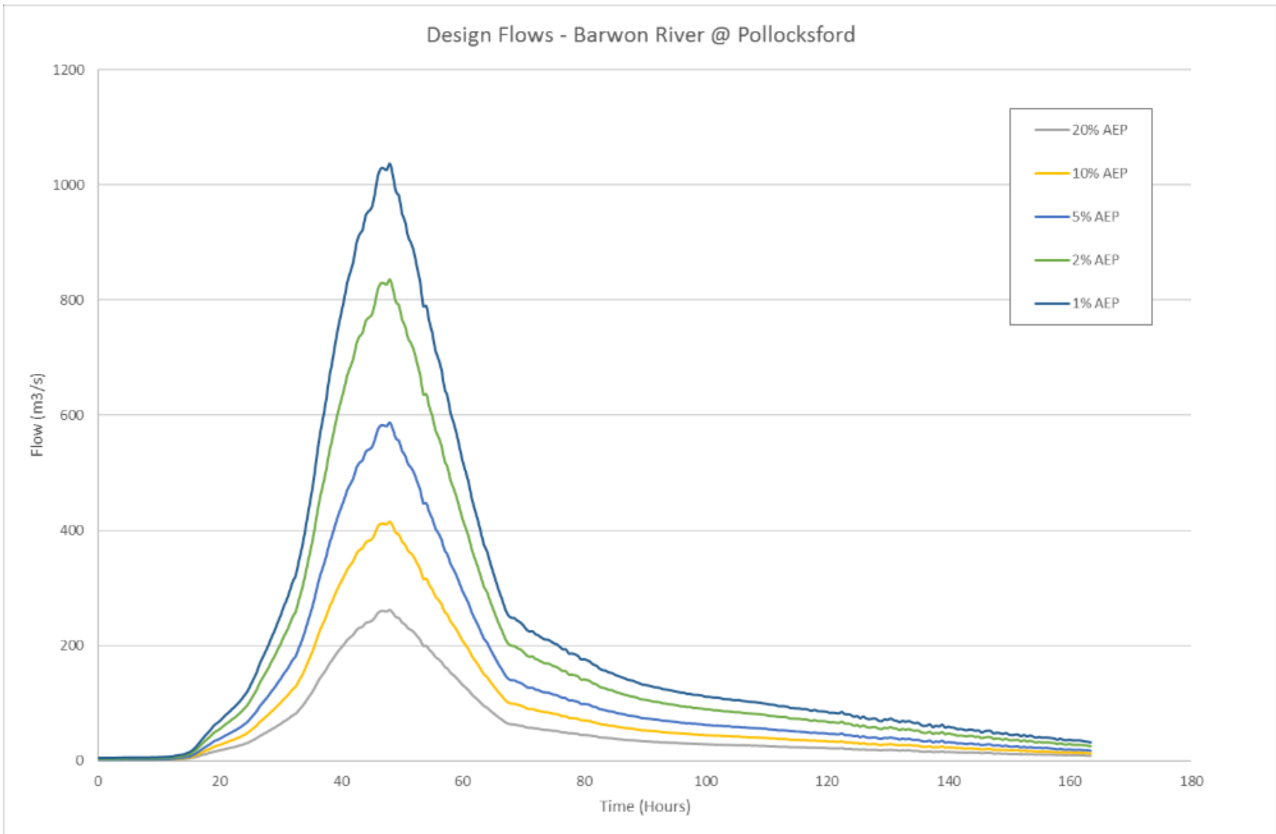


*Figure A-15 Adopted design hydrographs*

### A-1-7 Comparison of Previous FFA – Barwon River at Pollocksford

The Corangamite CMA has previous undertaken a Flood Frequency Analysis for this gauge based on a reduced record and partial series (1972-2014). A comparison of these results with the current adopted design flows at this location is shown in Table A-11. It is noted that the difference in the generated flows is low. The greatest difference of 2.8% for the 20% AEP (5 year ARI) is likely to as a result of the low flow censoring of the Water Technology method which was not undertaken as part of the CMA analysis.

*Table A-11 FFA Comparison – Barwon River at Pollocksford*

| AEP | CCMA (2015) ML/d | Water Technology (2016) ML/d | Difference % (+/-) |
|---|---|---|---|
| 20% | 23328 | 22675 | 2.80 |
| 10% | 35856 | 35900 | 0.12 |
| 5% | 50976 | 50712 | 0.51 |
| 2% | 71712 | 72217 | 0.70 |
| 1% | 88992 | 89605 | 0.70 |

## A-2 Moorabool River

### A-2-1 Moorabool River at Batesford Gauge

This section related to the assessment undertaken at the Moorabool River gauge (site No. 232202) in Batesford. At this location, the size of the upstream catchment is estimated to be approximately 1088 km$^2$.

Records indicate that the gauge at Batesford has moved at least once in the period of record. In addition to this, the type of control has also changed throughout the period of record. The site description of the two sites of record is referred to as Site A and Site B, with Site A being the existing site and original gauge immediately downstream of the Midland Highway bridge and Site B being 2.5 km downstream of Site A (referred to as "Lynnburn").

*Table A-12 Stream Gauge Location Record*

| Record Period | Location | Control | Recorder |
|---|---|---|---|
| **1908 - 1921** | Site A | Gravel | Manual Daily Readings |
| **1922-1944** | No Record | No Record | No Record |

| Record Period | Location | Control | Recorder |
|---|---|---|---|
| **1945-1953** | Site A | Gravel | Manual Daily Readings |
| **1953-1957** | Site B | Concrete Weir | Manual Daily Readings |
| **1959 – 1967** | Site A | Concrete Weir | Recorder |
| **1967- Current** | Site A | Concrete Weir | Recorder |

*Table A-13 Stream Gauge Height at Zero*

| Record Period | Gauge Height at Zero |
|---|---|
| **1959 to Current** | 17.11 Meters AHD |

The gauge record extends from 1908 to present, however a large amount of data from 1921 to 1945 and 1953 to 1959 is absent from the record. This is observed in the continuous height and flow data as displayed on the Victorian water data warehouse and can be observed in Figure A-16. While continuous flow records are available for the period back to 1959, only the record from 1973 to current will be used for this assessment. Another reason for this is that the Lal Lal reservoir, which is a major storage within the Moorabool catchment, was not completed until 1972.

The current gauge rating curve is shown in Figure A-17. The plotted historical flows can be observed in Figure A-18 and show clear banding which confirms the alterations to the site location and rating.

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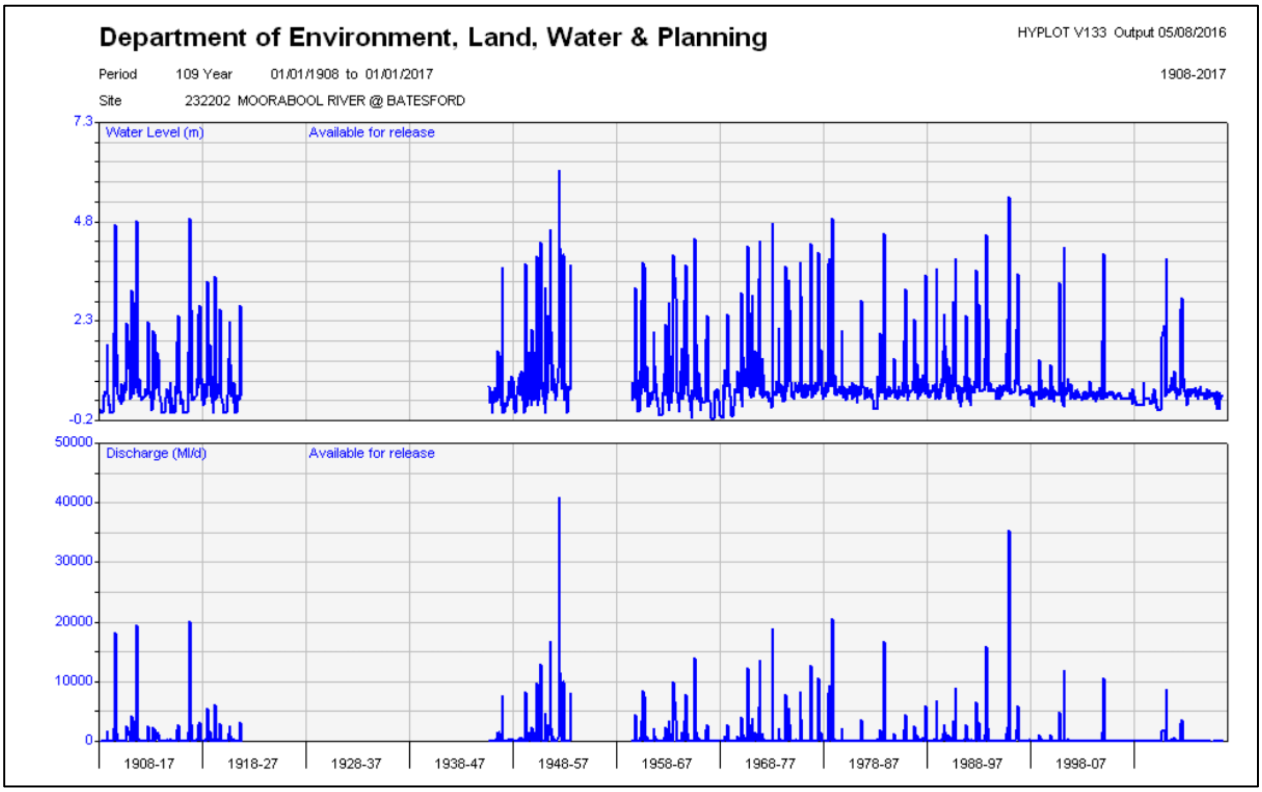


*Figure A-15 Batesford Stream Gauge Flow Record*

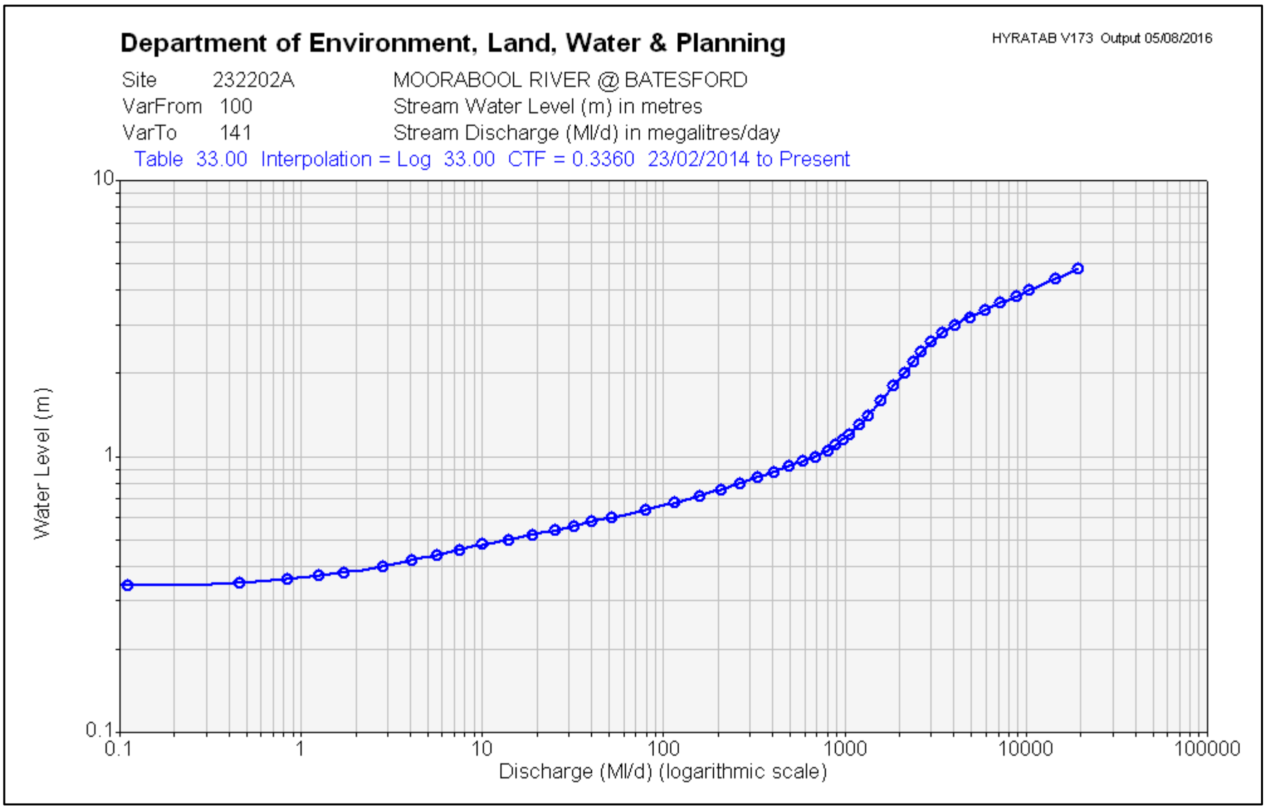


*Figure A-16 Batesford Stream Gauge Rating Table*

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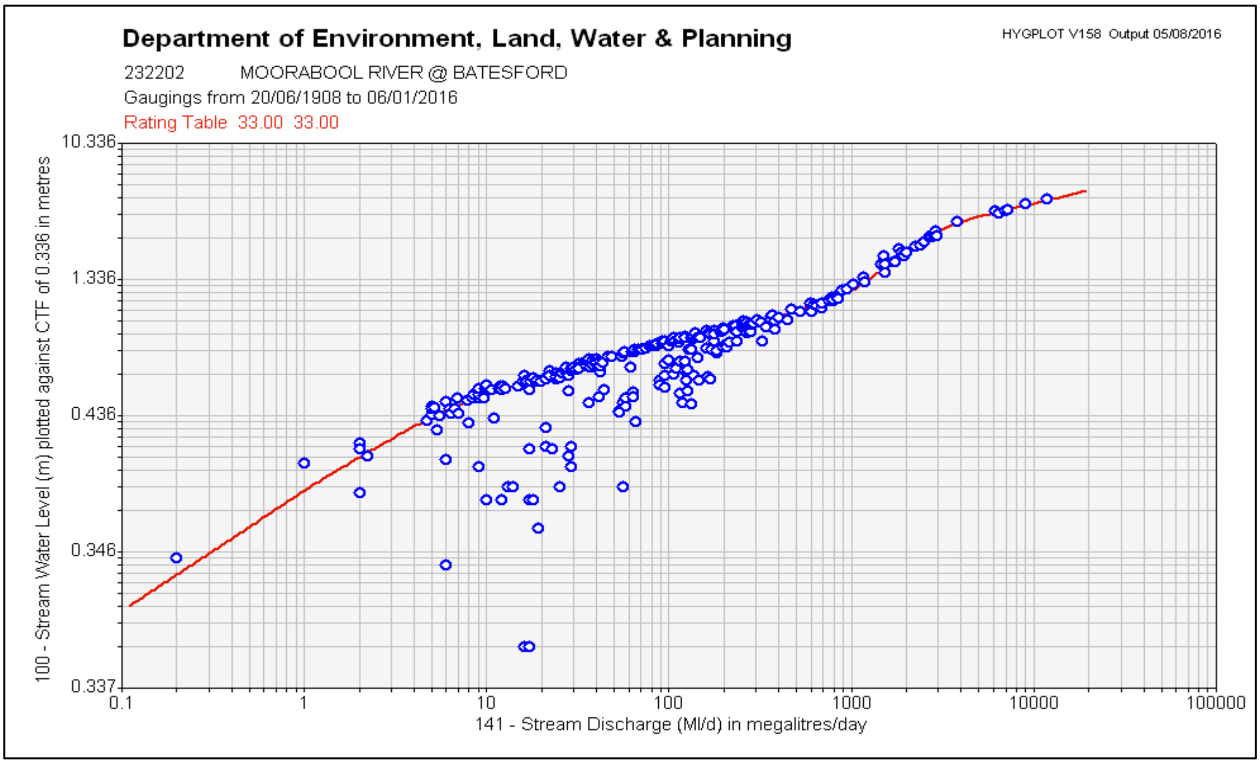



*Figure A-17 Batesford Gauge Plotted Flow Event*

## A-2-2 Design Flow Estimates

As discussed previously, the period of gauge record at the Moorabool River at Batesford gauge was used for the completion of a flood frequency analysis and was also used for the determination of peak design flows:

- Flood frequency analysis completed at the Moorabool River at Batesford gauge based on maximum instantaneous flows to determine the peak annual flow series.
- Flood frequency analysis completed at the Moorabool River at Batesford gauge based on four-day accumulated volume to determine design event volumes.
- A ratio of design event peak flow: design event four-day volume determined.
- A historic event chosen with a peak flow four-day volume ratio similar to that determined across the design events to be used as the basis for hydrograph shape.

An annual series FFA was completed at the Moorabool River at Batesford gauge as part of this study to determine design flow estimates. The available period of instantaneous record at this gauge includes 1969-current. The 232202 gauge at Batesford has daily flow data from 1906 to 1922. As mentioned previously, the record from 1973 to current time was selected to undertake the flood frequency.

The analysis was completed in an identical manner to the Barwon River assessment (for more information please refer to A-1-2). This determined a low flow threshold of 11.07 $m^3/s$, removing 12 years from the 42-year record.

The top 10 ranked flow events are shown in Table A-14 below, with the data ranked from highest to lowest.

*Table A-14 Ranked Historical Flow Events*

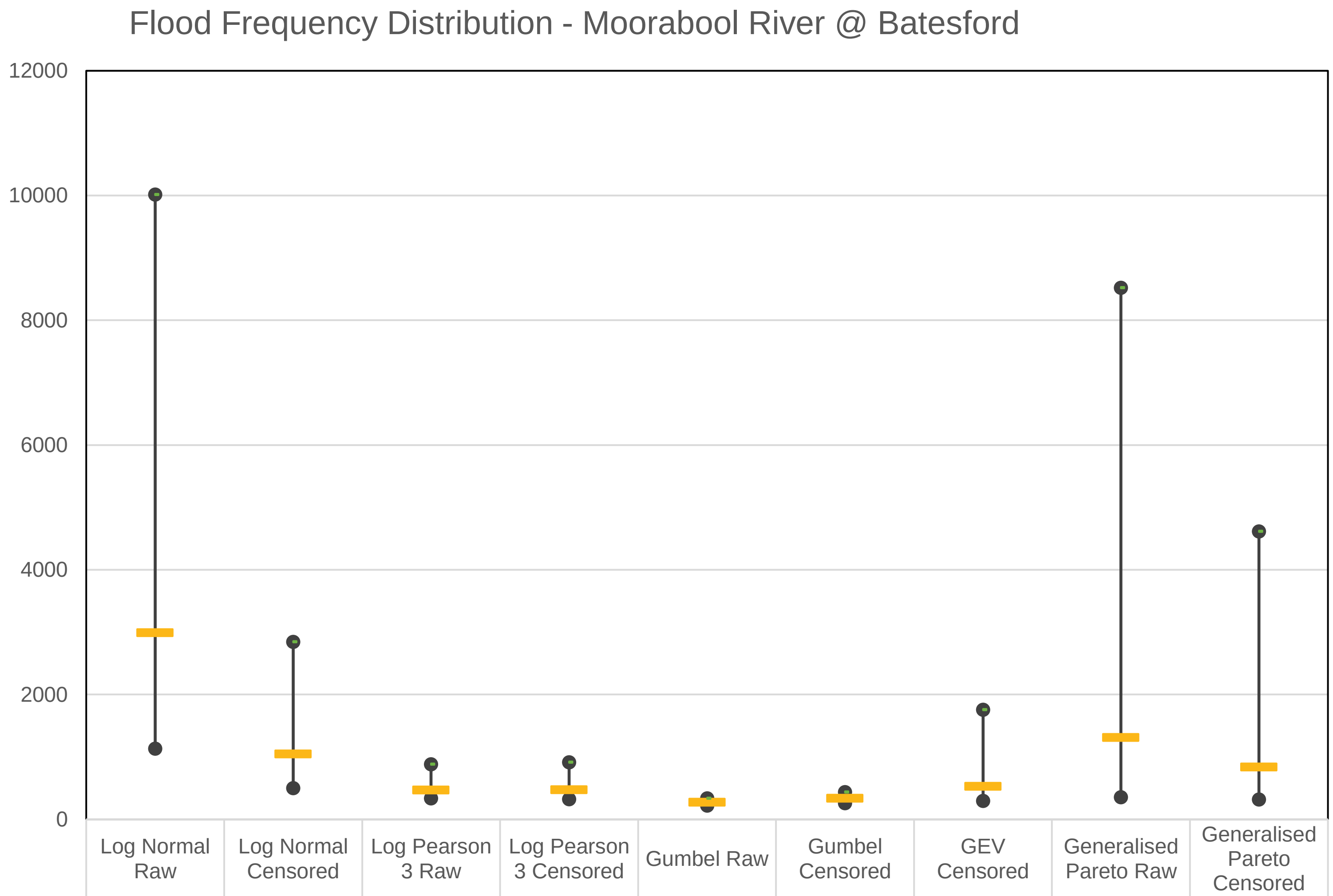
| Year | Peak Flow ($m^3/s$) | Peak Flow (ml/d) |
|---|---|---|
| **1995** | 410 | 35401 |
| **1978** | 238 | 20563 |
| **1973** | 219 | 18922 |
| **1983** | 193 | 16675 |
| **1993** | 183 | 15811 |
| **1976** | 146 | 12614 |
| **2001** | 137 | 11837 |
| **2005** | 123 | 10627 |
| **1977** | 122 | 10540 |
| **1990** | 102 | 8813 |

The Log Pearson Type 3 (LP3), Generalised Extreme Value (GEV), Generalised Pareto (GP) and Gumbel distributions were tested. Of these distributions, the LP3 and Gumbel matched well for both the raw and censored annual series, while GEV and GP matched better using the censored annual series.

A comparison of the FFA results for all distributions for the raw and censored annual series are shown in Table A-15. The 1% AEP flow estimate and associated confidence limits are also compared in Figure A-18 for the range of FFA distributions tested. The plot shows the LP3 Raw and censored, Gumbel Raw and Gumbel censored and the GEV censored data have the narrowest confidence limits. The FFA plots for the LP3 Raw, LP3 censored, Gumble censored and GEV censored distributions are shown in Figure A-19, Figure A-20, Figure A-21 and Figure A-22, respectively. Given the good fit for the distribution the LP3 censored distribution was adopted.

*Table A-15 Flood Frequency Comparison*

| Design Event | Log Normal | | LP3 | | Gumble | | GEV | | GP | |
|---|---|---|---|---|---|---|---|---|---|---|
| **AEP** | **Raw** | **Censored** | **Raw** | **Censored** | **Raw** | **Censored** | **Raw** | **Censored** | **Raw** | **Censored** |
| **20%** | 118.5 | 107.8 | 118.8 | 116.5 | 111.2 | 122.4 | 122.3 | 119.6 | 115.9 | 116.8 |
| **10%** | 308.4 | 211.4 | 205.5 | 196.0 | 149.7 | 173.6 | 557.4 | 187.9 | 221.9 | 203.0 |
| **5%** | 679.4 | 368.9 | 293.2 | 280.8 | 186.6 | 222.8 | 2385.3 | 268.6 | 393.0 | 324.7 |
| **2%** | 1652.4 | 690.1 | 399.4 | 392.7 | 234.3 | 286.4 | 15655.3 | 401.0 | 789.8 | 565.8 |
| **1%** | 2988.4 | 1047.8 | 468.4 | 473.1 | 270.1 | 334.1 | 64113.1 | 525.8 | 1309.4 | 837.1 |




| | Log Normal Raw | Log Normal Censored | Log Pearson 3 Raw | Log Pearson 3 Censored | Gumbel Raw | Gumbel Censored | GEV Censored | Generalised Pareto Raw | Generalised Pareto Censored |
|---|---|---|---|---|---|---|---|---|---|
| Lower Confidence Limit | 1133.93 | 500.84 | 333.68 | 321.28 | 219.53 | 260.35 | 292.9 | 355.45 | 319.26 |
| Mean | 2988.35 | 1047.79 | 468.44 | 473.11 | 270.06 | 334.09 | 525.81 | 1309.38 | 837.07 |
| Upper Confidence Limit | 10015.5 | 2842.2 | 883.2 | 912.2 | 335.9 | 434.9 | 1753.8 | 8521 | 4614.3 |

*Figure A-18 Plotted Flood Frequency Analysis - Moorabool River at Batesford*

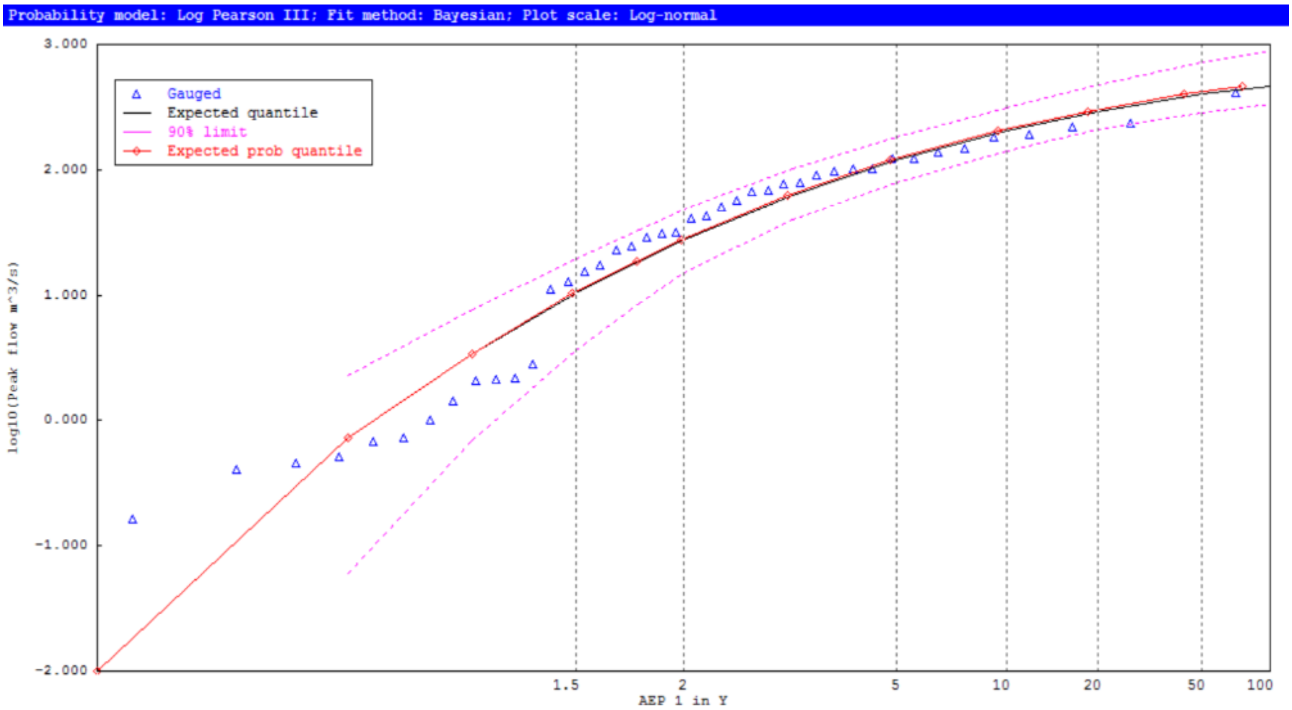



*Figure A-19 Log Peasron III (raw) Flood Frequency Distribution Plot*

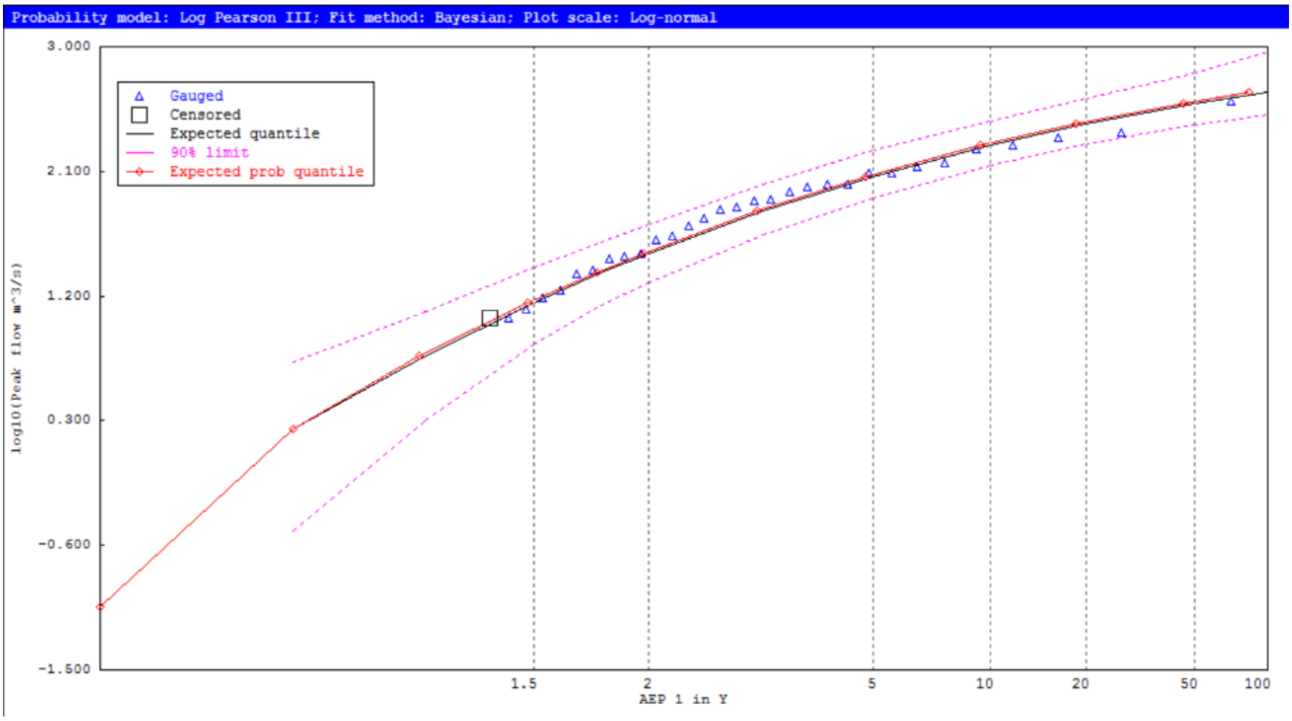



*Figure A-20 Log Pearson III Censored Flood Frequency Distribution Plot*

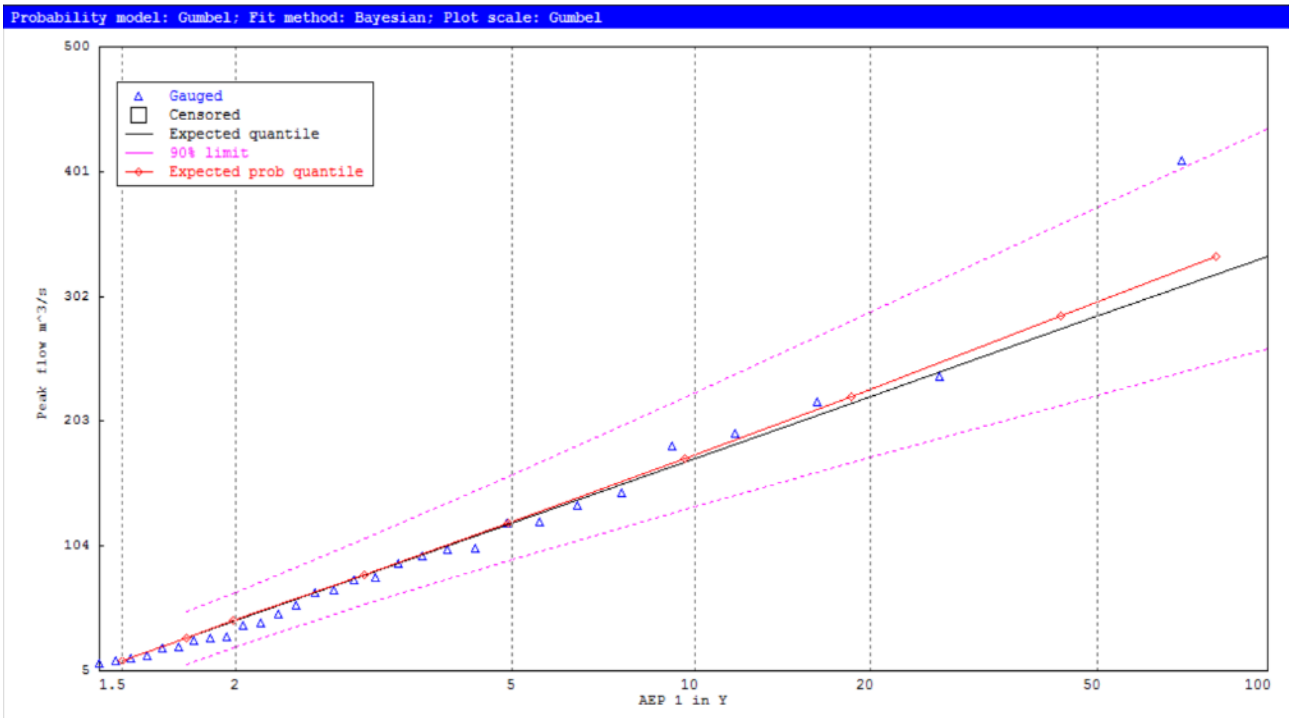



*Figure A-21 Gumble Censored Flood Frequency Distribution Plot*

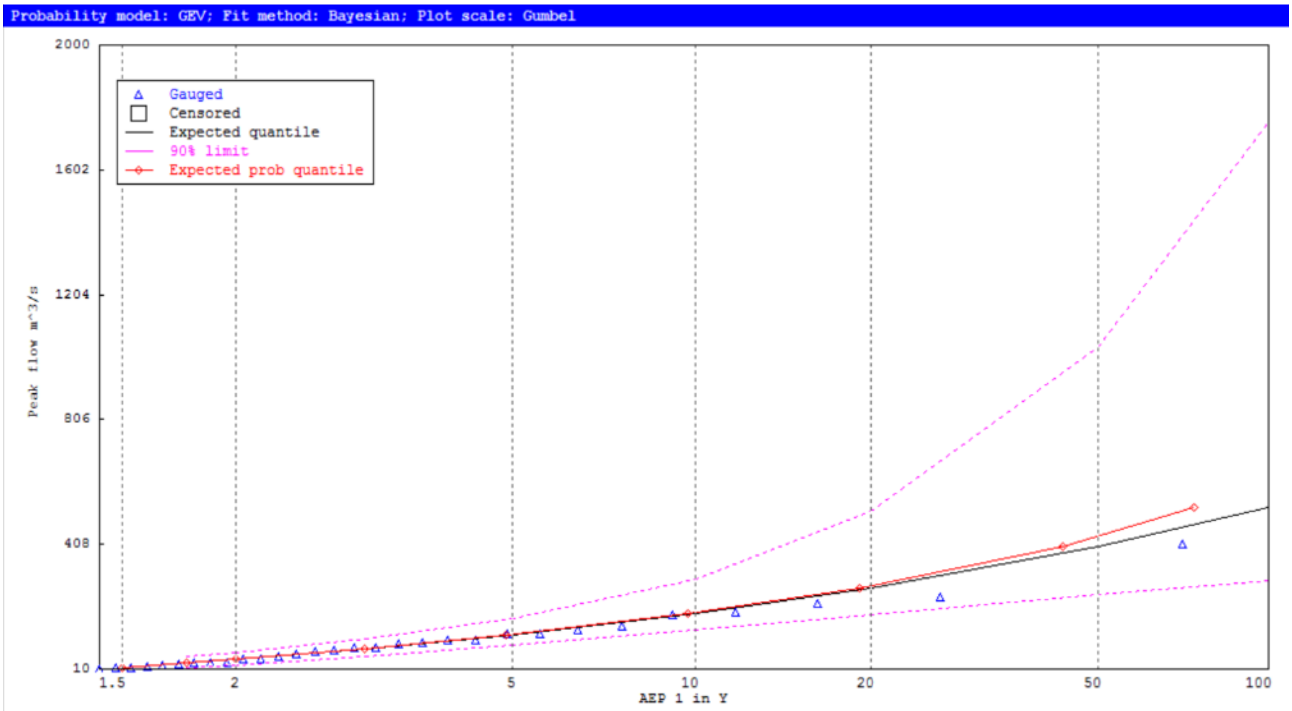



*Figure A-22 GEV Censored Flood Frequency Distribution Plot*

### A-2-3 Design Flow Comparison

Similarly to the Barwon River assessment, a number of methods were used to verify the flood frequency design flow estimates. For more information on these please refer to section A-1-3.

The comparisons in Table A-16 indicate that the adopted 1% AEP design flow from flood frequency analysis using the LP3 distribution and low flow filtering is of similar magnitude to the Rural Rational Method. Given this, the adopted design flows are considered appropriate for this study.

*Table A-16 Regional Flow Estimate Comparison*

| Upstream Catchment Area (km$^2$) | 1% AEP Estimated Peak Flow (m$^3$/s) | | | |
|---|---|---|---|---|
| | **Water Technology FFA** | **CCMA FFA** | **Regional Equation** | **Rural Rational** |
| **1,088** | 473.1 | 485 | 322.6 | 968.9 |

### A-2-4 Historical Flow AEP Comparison

Table A-17 shows a comparison of the top 5 ranked flow events and the estimated AEP associated with the flood frequency assessment at this gauge. As shown, the highest of the recorded events is November 1995, which was estimated to have a AEP of 1.3% on the Moorabool River at this location.

*Table A-17 Estimated AEPs of Historical Flows*

| **Flood Event** | Peak Flow (ML/d) | Peak Flow (m$^3$/s) | ARI (years) | AEP (%) |
|---|---|---|---|---|
| **November 1995** | 35,401 | 410 | 70.33 | 1.3 |
| **November 1978** | 20,578 | 219 | 26.37 | 3.8 |
| **February 1973** | 18,904 | 238 | 16.23 | 6.16 |
| **October 1983** | 16,670 | 193 | 11.72 | 8.53 |
| **September 1993** | 15,851 | 183 | 9.17 | 10.9 |

### A-2-5 Final Adopted Design Peak Flows

As discussed earlier, the Log Pearson III distribution with low flow filtering was determined to represent the best fit with the available historic data. The design flows in the table below represent the final design adopted flows for the Moorabool River at Batesford.

*Table A-18 Adopted Design Peak Flows*

| Design Event (AEP) | Log Pearson III with Low Flow Censoring ($m^3/s$) | | |
|---|---|---|---|
| | Peak Flow | Lower Confidence Limit | Upper Confidence Limit |
| **20%** | 116.5 | 81.4 | 177.4 |
| **10%** | 196.0 | 138.5 | 292.4 |
| **5%** | 280.8 | 198.9 | 421.7 |
| **2%** | 392.7 | 273.2 | 641.2 |
| **1%** | 473.1 | 321.3 | 912.2 |

## A-2-6 Moorabool River at Batesford Hydrograph Shape

Similar to the analysis of peak flows, an analysis on volume was undertaken to inform the choice of hydrograph shape for design flows at the Moorabool River at Batesford streamflow gauge. To determine the design hydrograph the ratio of historic event peak flows to volumes was compared to the ratio of design peak flows to volumes determined by FFA.

The five largest events recorded at the Moorabool River at Batesford gauge (November 1995, November 1978, February 1973, October 1983 and September 1993) are shown overlayed in Figure A-28. In all events the flood occurred over a three to five-day period. The 1995, 1973, 1978 and 1983 events all have a similar shape, while the 1993 event seemed to have two significant peaks within the time period. Based on these hydrographs it was determined a four-day volume FFA would be completed. Furthermore, based on the similar shape of the 1995, 1973 and 1983 events it was determined that a hydrograph shape similar to these events would be adopted.

The four-day volume FFA was completed using a variety of distributions, with the LP3 distribution the best fit of the recorded data. The four-day volume FFA results and comparisons for the design events are shown in Table A-19 , with the historical events shown in Table A-20.

The volume to peak flow ratio of the 1978 historic events match the average ratio of the range of design events considerably better than some of the other historic event. The 1995, 1993, 1983 and 1978 event provides a good fit to the 1% AEP ratio. Given that the smooth shape of the 1995 hydrograph it was chosen as the donor hydrograph shape for design purposes.

The November 1995 hydrograph shape was scaled for each design event to match the peak and the volume of the design hydrograph from the FFA. Figure A-29 presents the final design flood hydrographs.

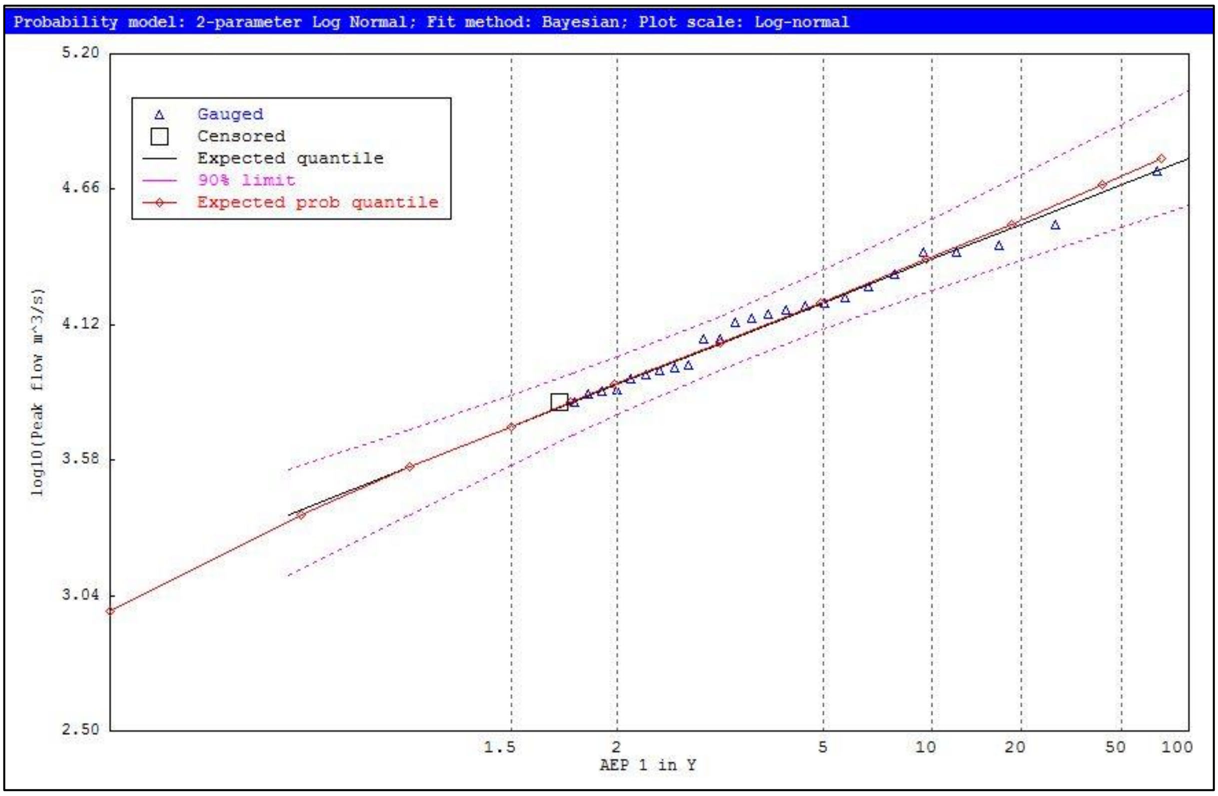



*Figure A-23 Log Normal - 4 Day Volume Distribution Plot*

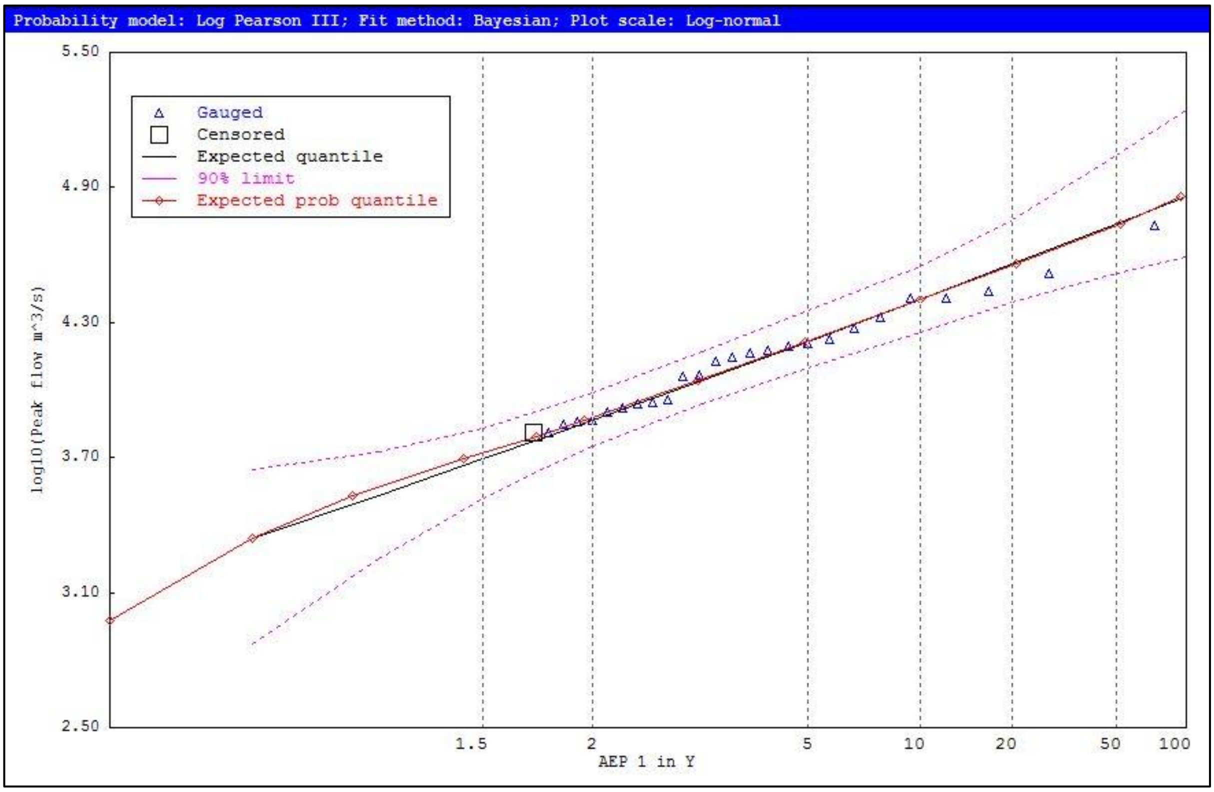



*Figure A-24 Lot Pearson III - 4 Day Volume Distribution Plot*

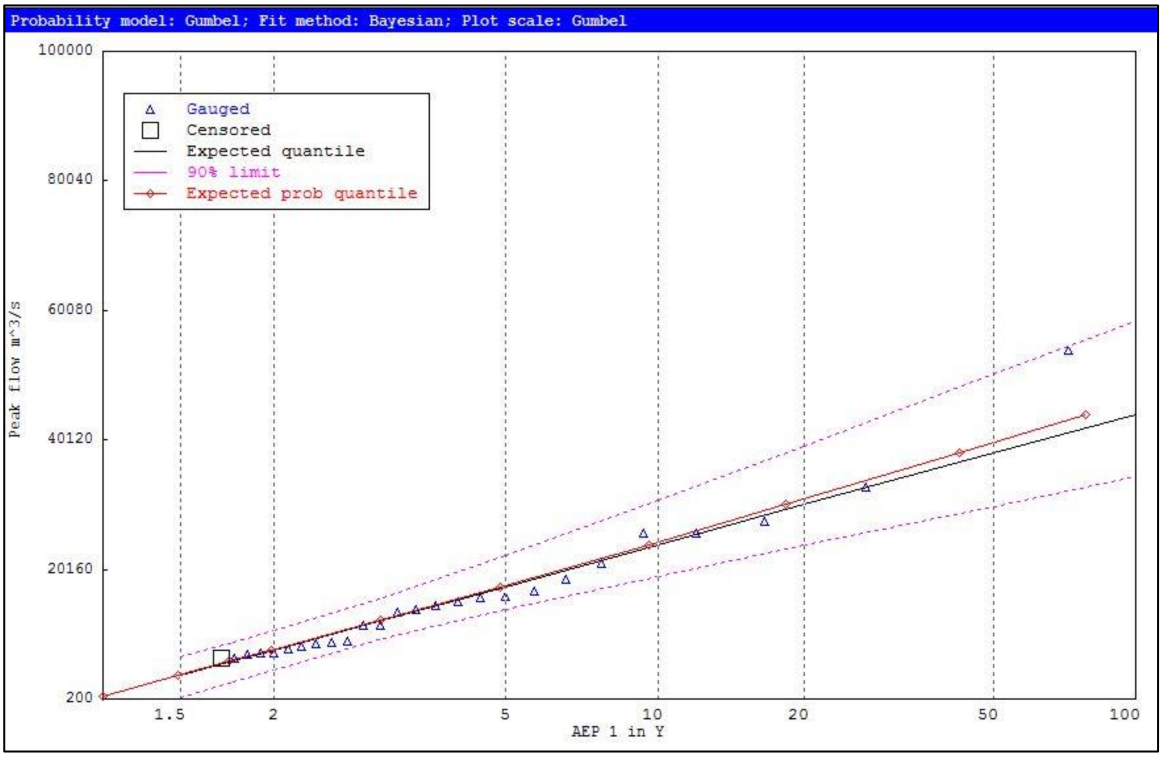



*Figure A-25  Gumble - 4 Day Volume Distribution Plot*

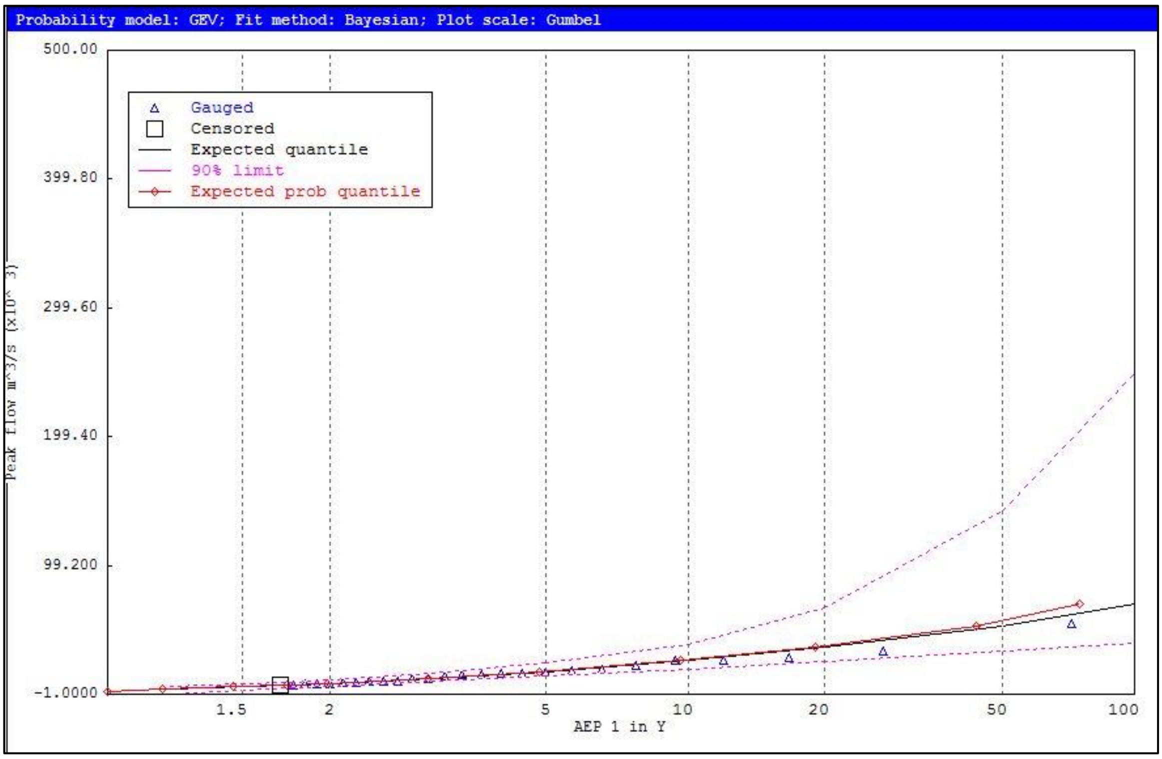



*Figure A-26  GEV - 4 Day Volume Distribution Plot*

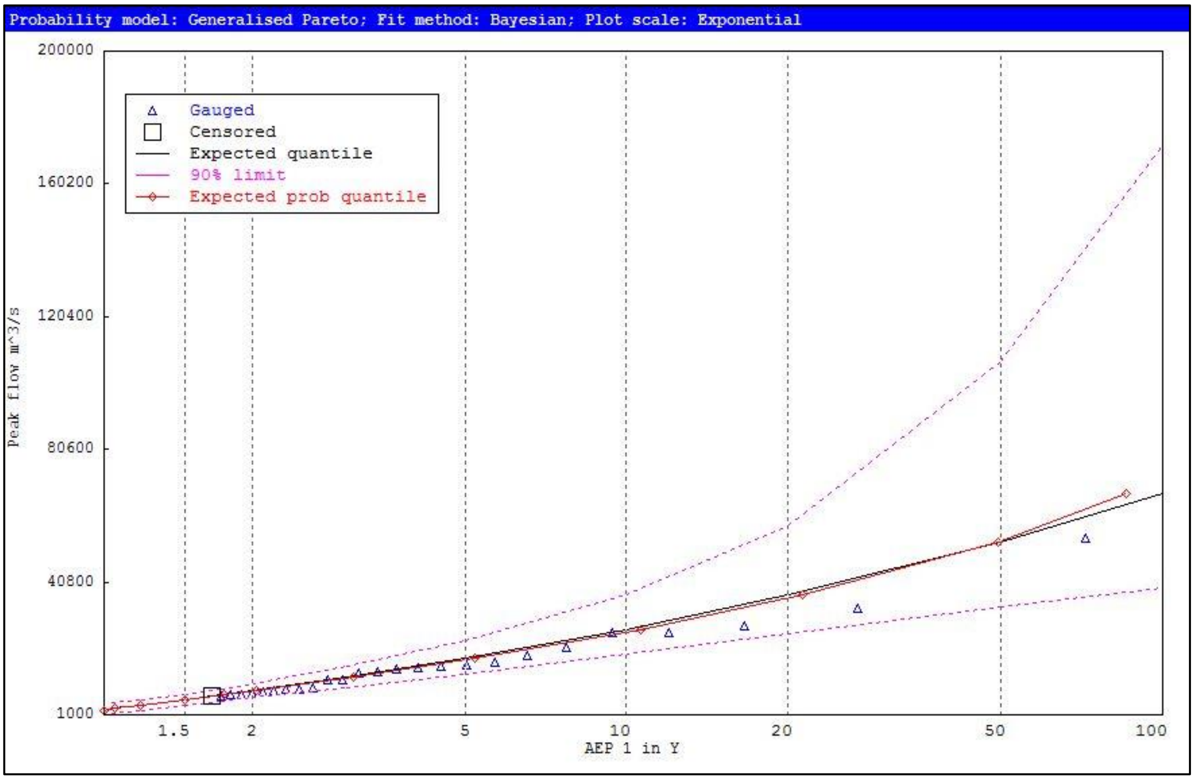



*Figure A-27 GP - 4 Day Volume Distribution Plot*

*Table A-19 FFA Four Day Volume and Peak Flow Comparison and Ratio*

| AEP | 4 Day Volume (ML) | Peak Flow (ML/d) | Ratio |
|---|---|---|---|
| **20%** | 16,401 | 10,063 | 1.63 |
| **10%** | 25,235 | 16,936 | 1.49 |
| **5%** | 36,259 | 24,263 | 1.49 |
| **2%** | 54,918 | 33,927 | 1.62 |
| **1%** | 72,741 | 40,876 | 1.78 |

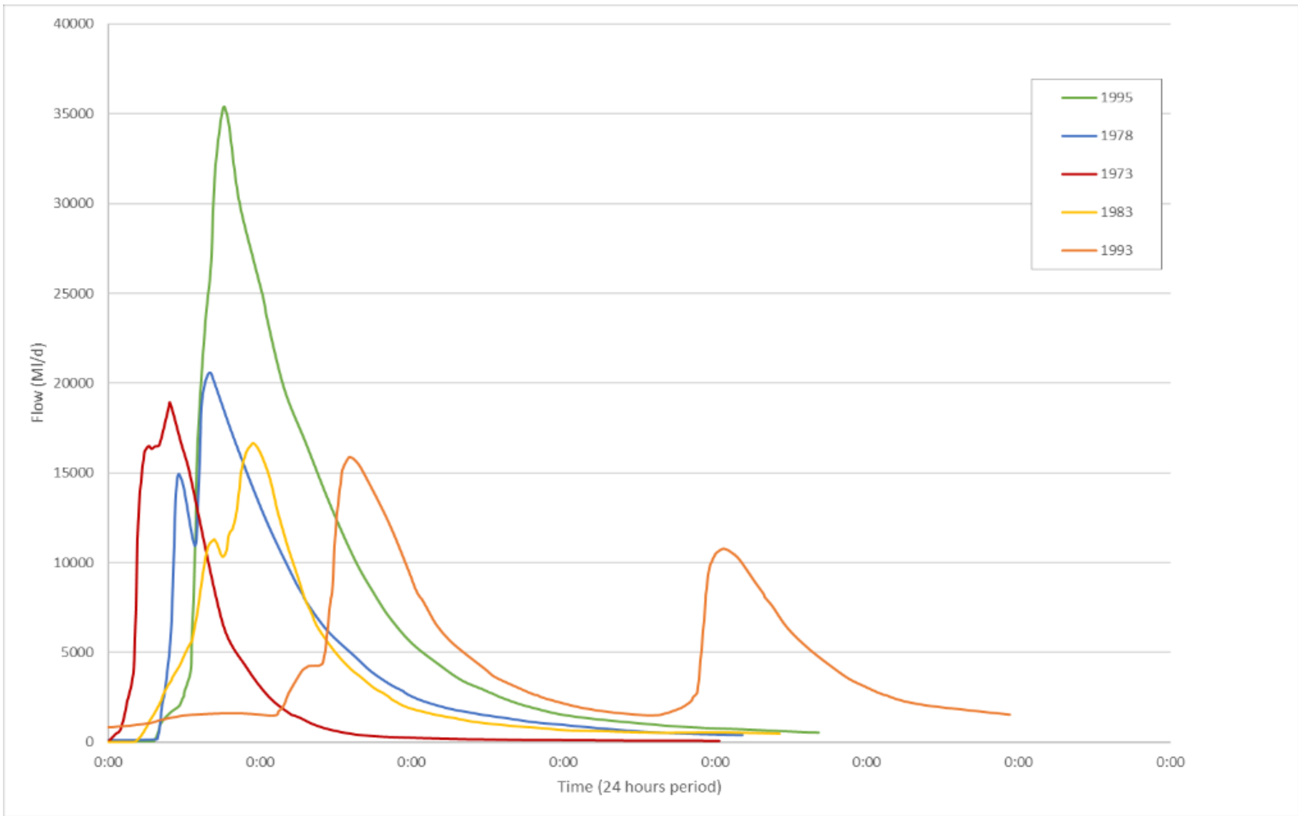



*Figure A-28 Historical flow Event Gauge Hydrograph*

*Table A-20 Historical Event Flow vs Four-Day Volume Ratio*

| Historical Events | Peak Flow ML/day | 4 Day Volume (ML) | Ratio |
|---|---|---|---|
| **November 1995** | 35,401 | 53,997 | 1.53 |
| **November 1978** | 20,578 | 32,937 | 1.60 |
| **February 1973** | 18,904 | 21,087 | 1.10 |
| **October 1983** | 16,670 | 25,756 | 1.54 |
| **September 1993** | 15,851 | 25,777 | 1.63 |
| | | **Average** | 1.48 |

*Table A-21 Adopted Four Day Volume and Peak Flow Comparison and Ratio*

| AEP | 4 Day Volume (ML) | Peak Flow (ML/d) | Ratio |
|---|---|---|---|
| **20%** | 14,852 | 10,063 | 1.48 |
| **10%** | 24,996 | 16,936 | |
| **5%** | 35,810 | 24,263 | |
| **2%** | 50,073 | 33,927 | |
| **1%** | 60,330 | 40,876 | |

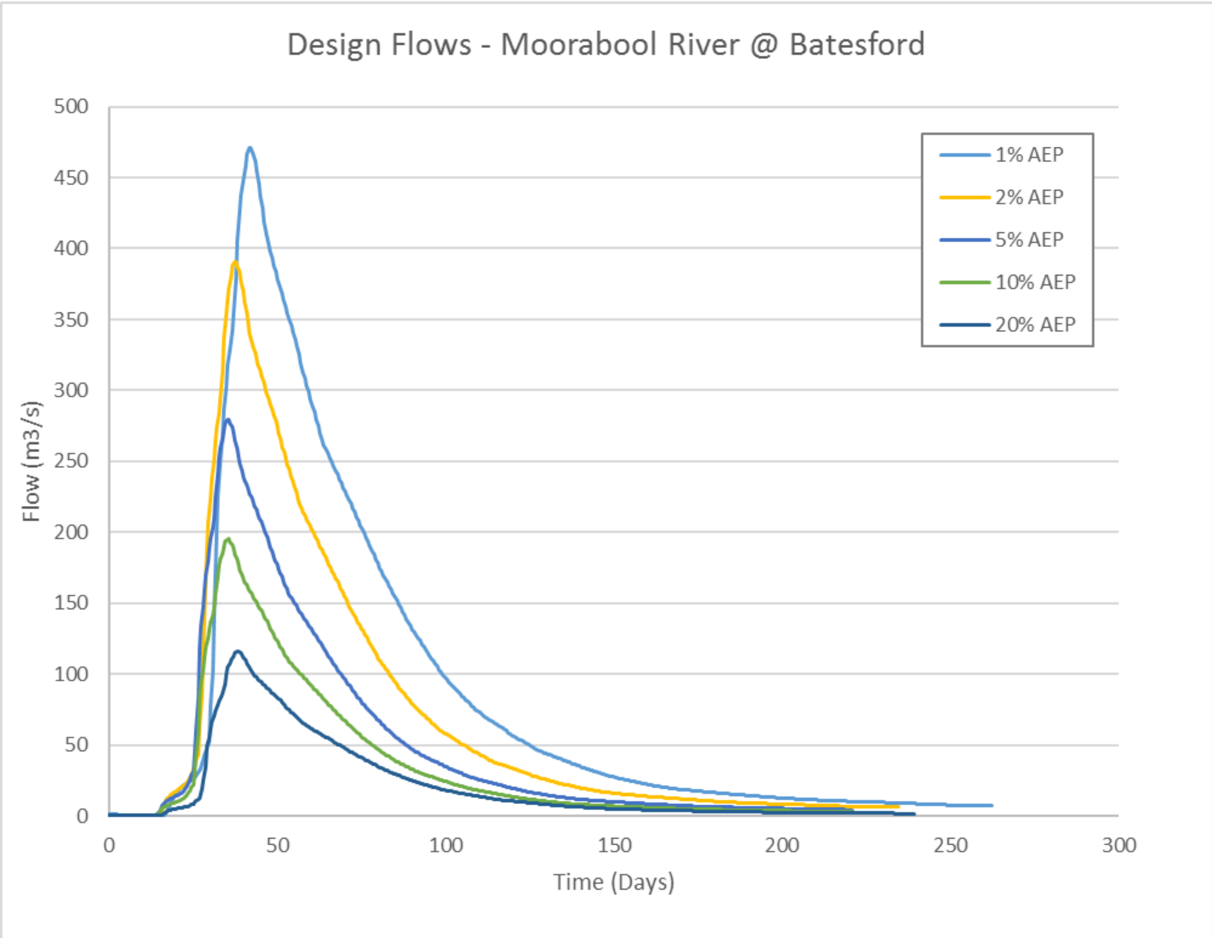



*Figure A-29 Adopted Design Hydrographs*

### A-2-7 Comparison of Previous FFA – Moorabool River at Batesford

The Corangamite CMA has previous undertaken a Flood Frequency Analysis for this gauge based on a reduced record and partial series (1972-2014). A comparison of these results with the current adopted design flows at this location is shown in the table below. It is noted that the difference in the generated flows is low. The greatest difference of 3% is for the 20% AEP (50 year ARI) is likely to as a result of the low flow censoring of the Water Technology method which was not undertaken as part of the CMA analysis Further analysis using a variety of methods will further test these differences as part of the major flood study investigation.

*Table A-22 FFA Comparison – Moorabool River at Batesford*

| AEP | CCMA (2015) | Water Technology (2016)<br>ML/d | Difference % (+/-) |
|---|---|---|---|
| 20% | 10,368 | 10,063 | 2.9 |
| 10% | 17,366 | 16,936 | 2.5 |
| 5% | 25,228 | 24,263 | 2.8 |
| 2% | 34,992 | 33,927 | 3.0 |
| 1% | 41,904 | 40,876 | 2.5 |

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