# Moolabin Creek and Rocky Water Holes Creek Flood Study

## Volume 1 of 2

# Flood Study Report

**Prepared by Brisbane City Council's, City Projects Office**

**June 2015**



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**Note:** *The Moolabin Creek and Rocky Water Holes Creek Flood Study is a joint initiative of Brisbane City Council and the Queensland Government.*

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# Executive Summary

## Introduction

Brisbane City Council (BCC) is in the process of updating all of its flood studies to reflect the current conditions of the catchment and best practice flood modelling techniques. The most recent flood study for the combined catchments was undertaken in 2000 by Connell Wagner (now Aurecon). This report was effectively reformatted and republished in 2006 by BCC City Design (now BCC City Projects Office).

Moolabin and Rocky Water Holes Creek Catchments are located within the greater Oxley Creek Catchment, approximately 8 km south of the Brisbane CBD. The combined catchment area of both creeks is approximately 11.8 km$^2$, of which Rocky Water Holes Creek is the larger with a catchment area of 6.2 km$^2$. The total combined catchment area encompasses the suburbs of Annerley, Moorooka, Salisbury, Yeerongpilly and Tennyson.

## Project Objectives

The primary objectives of the project were as follows:

- Update the Moolabin Creek and Rocky Water Holes Creek flood models (hydrologic and hydraulic) to represent the current catchment conditions and best practice flood modelling techniques.
- Adequately calibrate and verify the flood models to historical storm events to confirm that the models are suitable for the purposes of simulating design flood events.
- Estimation of design and rare / extreme flood magnitudes.
- Determination of flood levels for the design and rare / extreme events.
- Quantify the impacts of Minimum Riparian Corridor (MRC) and filling / development outside the Conveyance Corridor.
- Produce flood extent mapping for the selected range of design and rare / extreme events.
- Quantify the impacts of climate change as well as hydraulic structure blockages on flooding within the catchment.

## Project Elements

The flood study consists of two main components, as follows:

**Calibration and Verification Modelling**

Hydrologic and hydraulic models of the Moolabin Creek and Rocky Water Holes Creek Catchment have been developed using the XP-RAFTS and TUFLOW modelling software, respectively.

The hydrologic model simulates the catchment rainfall-runoff and runoff-routing processes. The hydrologic model also utilises high-level routing methodology to simulate the flow of floodwater in the major waterways within the catchment. The hydraulic model uses more sophisticated routing to simulate the movement of this floodwater through these waterways in order to predict flood levels, flood discharges and velocities. The hydraulic model takes into account the effects of the channel / floodplain topography; downstream tailwater conditions and hydraulic structures.

Calibration is the process of refining the model parameters to achieve a good agreement between the modelled results and the historical / observed data. Model calibration is achieved when the model simulates the historical event to within specified tolerances. Verification is then undertaken on additional flooding events to confirm the calibrated model is suitable for use in simulating synthetic design storm events.

Calibration of the XP-RAFTS and TUFLOW models was undertaken utilising three historical storms; namely January 2013, February 2010 and March 2001. Verification of the XP-RAFTS and TUFLOW models utilised the November 2004 historical storm event.

An acceptable correlation was achieved between the simulated and historical records for all three calibration events. At the Maximum Height Gauges (MHGs), the simulated peak levels were generally within the specified tolerance of ± 0.3 m.

Utilising the adopted parameters from the calibration process, the verification was undertaken. Similar to the calibration results, the verification achieved an acceptable correlation between the simulated and historical records for both of the verification events.

Given the results of the calibration and verification process were quite reasonable, the XP-RAFTS and TUFLOW models were considered acceptable for use in the second part of the flood study, in which design flood levels were estimated.

**Design and Extreme Event Modelling**

The calibrated hydrologic and hydraulic models were then used to simulate a range of synthetic design flood events. Design and extreme flood magnitudes were estimated for the full range of events from 2-yr ARI to PMF. These analyses assumed ultimate catchment hydrological conditions.

Three waterway scenarios were considered, as follows:

- Scenario 1 – Existing Waterway Conditions: Based on the current waterway conditions. Some minor modifications were made to the TUFLOW model developed as part of the calibration / verification phase.
- Scenario 2 – Minimum Riparian Corridor (MRC): Includes an allowance for a riparian corridor along the edge of the channel.
- Scenario 3 – Ultimate Conditions: Includes an allowance for the minimum riparian corridor (as per Scenario 2) and also assumes development infill to the boundary of the Conveyance Corridor in order to simulate potential development.

The results from the TUFLOW modelling were used to determine / produce the following:

- Peak flood discharges
- Critical storm durations at selected locations
- Peak flood levels at 100 m intervals along the AMTD line and well as model cross-sections
- Peak flood extent mapping
- Hydraulic structure flood immunity

A sensitivity analysis was undertaken to understand the impacts of the following:

- Climate change for two planning horizons; namely 2050 and 2100.
- Hydraulic Structure Blockages

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# Glossary of Terms

| Term | Definition |
|---|---|
| Annual Exceedance Probability(AEP) | The probability that a given rainfall total or flood flow will be exceeded in any one year. |
| Average Recurrence Interval (ARI) | The long-term average number of years between the occurrence of a flood as big as (or larger than) the selected event. For example, floods with a discharge as great as (or greater than) the 20 year ARI design flood will occur on average once every 20 years. |
| AHD | Australian Height Datum (AHD) is the reference level for defining reduced levels adopted by the National Mapping Council of Australia. The level of 0.0 m AHD is approximately mean sea level. |
| Brisbane Bar | Location at the mouth of the Brisbane River |
| Catchment | The area of land draining through the main stream (as well as tributary streams) to a particular site. It always relates to an area above a specific location. |
| Digital Elevation Model (DEM) | A three-dimensional model of the ground surface elevation. |
| Design Event, Design Storm | A hypothetical flood/storm representing a specific likelihood of occurrence (for example the 100 year ARI). |
| ESTRY | TUFLOW 1D engine. |
| Floodplain | Area of land subject to inundation by floods up to and including the probable maximum flood (PMF) event |
| Flood Frequency Analysis (FFA) | Method of predicting flood flows at a particular location by fitting observed values at the location to a standard statistical distribution. |
| HEC-RAS | Hydraulic modelling software package. |
| Hydrograph | A graph showing how the discharge or stage/flood level at any particular location varies with time during a flood. |
| Manning's 'n' | The Gauckler–Manning coefficient, used to represent roughness in 1D/2D flow equations. |
| MIKE11 | Hydraulic modelling software package. |
| Minimum Riparian Corridor (MRC) | An area of (maximum) 15m width either side of the main flow channel. |
| Probable Maximum Flood (PMF) | An extreme flood deemed to be the largest flood that could conceivably occur at a specific location. |
| Probably maximum Precipitation (PMP) | Probable Maximum Precipitation. The maximum precipitation (rainfall) that is reasonably estimated to not be exceeded. |
| WBNM | Hydrologic modelling software package. |
| XP-RAFTS | Hydrologic modelling software package. |

# List of Abbreviations

| Abbreviation | Definition |
|---|---|
| 1d | One dimensional, in the context of hydraulic modelling |
| 2d | Two dimensional, in the context of hydraulic modelling |
| AMTD | Adopted Middle Thread Distance |
| ALS | Airborne Laser Scanning |
| AR&R | Australian Rainfall and Runoff (1999) |
| BCC | Brisbane City Council |
| CBD | Central Business District |
| CL | Continuing rainfall loss (mm/hr) |
| IFD | Intensity Frequency Duration |
| IL | Initial rainfall loss (mm) |
| m AHD | metres above AHD |
| MHG | Maximum Height Gauge |
| MRC | Minimum Riparian Corridor |
| MSQ | Maritime Safety Queensland |
| POT | Peak Over Threshold |
| RCBC | Reinforced Concrete Box Culvert |
| RCP | Reinforced Concrete Pipe |
| QUDM | Queensland Urban Drainage Manual (2013) |
| WC | Waterway Corridor |
| WQA | Water Quantity Assessment |

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# 1.0 Introduction

## 1.1 Catchment Overview

Moolabin and Rocky Water Holes Creek Catchments are located within the greater Oxley Creek Catchment, approximately 8 km south of the Brisbane CBD. The combined catchment area of both creeks is approximately 11.8 $km^2$, of which Rocky Water Holes Creek is the larger with a catchment area of 6.2 $km^2$. The total combined catchment area encompasses the suburbs of Annerley, Moorooka, Salisbury, Yeerongpilly and Tennyson.

Figure 1.1 indicates the locality of the catchment.

## 1.2 Study Background

BCC is in the process of updating all of its flood studies to reflect the current conditions of the catchment and best practice flood modelling techniques. This flood study has been undertaken in accordance with the current BCC flood study procedures.[1]

The most recent flood study for the combined catchments was undertaken in 2000 by Connell Wagner (now Aurecon). This report was effectively reformatted and republished in 2006 by BCC City Design (now BCC City Projects Office). [2]

## 1.3 Study Objectives

The primary objectives of the project are as follows:

- Update the Moolabin Creek and Rocky Water Holes Creek flood models (hydrologic and hydraulic) to represent the current catchment conditions and best practice flood modelling techniques.
- Adequately calibrate and verify the flood models to historical storm events to confirm that the models are suitable for the purposes of simulating design flood events.
- Estimation of design and rare / extreme flood magnitudes.
- Determination of flood levels for the design and rare / extreme events.
- Quantify the impacts of Minimum Riparian Corridor (MRC) and filling in accordance with the planning requirements.
- Produce flood extent mapping for the selected range of design and rare / extreme events.
- Quantify the impacts of climate change as well as hydraulic structure blockages on flooding within the catchment.

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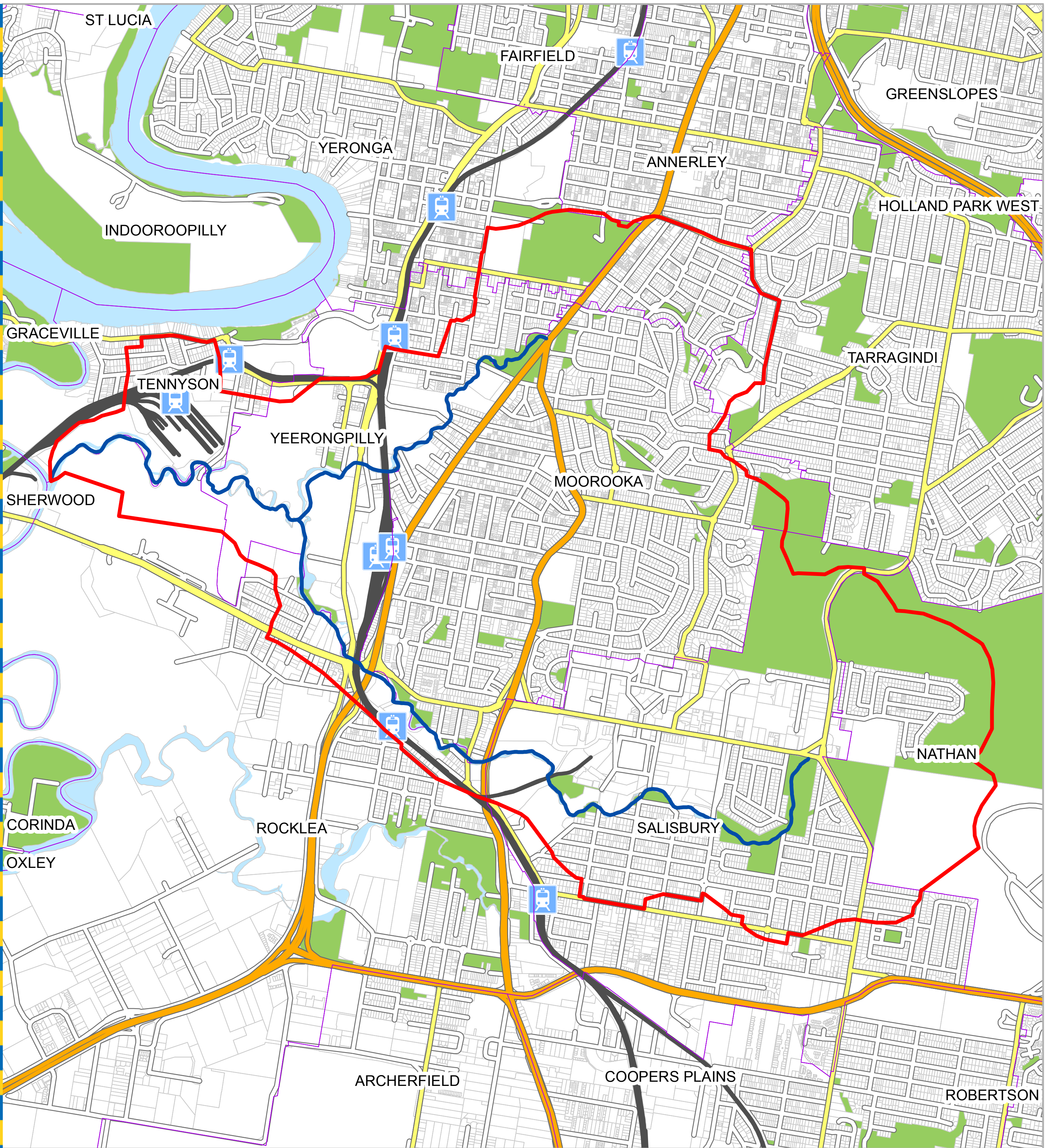
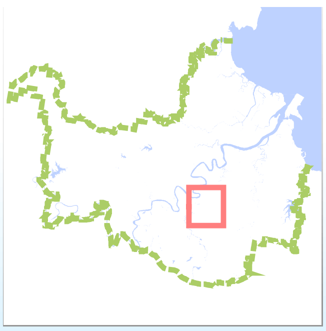
[1] Brisbane City Council 2015, *Creek Flood Study Procedure Document Version 7.0*
[2] Brisbane City Council 2006, *Moolabin Creek and Rocky Waterholes Flood Study*, prepared by BCC City Design, Brisbane



Legend

| | |
|---|---|
| Catchment Area | Park Boundaries |
| Creek Centreline | Suburb Boundaries |
| Railway Station | Property Holdings |
| Railway Line | Water |
| Freeways/Highways | |
| Major Roads | |
| Streets | |

0 300 600 900
Metres

N W E S

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Checked : NB
Revision : 0
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Project Number : 150339

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**Figure 1.1: Locality Plan**

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## 1.4 Scope of the Study

The following tasks were undertaken to achieve the project objectives as outlined in Section 1.3:

- Development of an XP-RAFTS hydrologic model of the catchment, superseding the previous WBNM model.
- Develop a 1-dimensional (1d) / 2-dimensional (2d) TUFLOW hydraulic model of the creek system to replace the existing 1d MIKE11 hydraulic model.
- Calibrate the hydrologic and hydraulic models to the January 2013, February 2010 and March 2001 historical flood events.
- Verify the hydrologic and hydraulic models against the November 2004 historical flood event.
- Estimate the design and extreme flood magnitudes for the full range of events from 2-yr ARI to PMF.
- Simulate synthetic Australian Rainfall and Runoff (AR&R) design storms for multiple durations to determine the critical duration at various locations within the catchment.
- Utilise the calibrated flood models to determine peak design flood levels for the design and rare / extreme events.
- Make adjustments to the hydraulic model to simulate the impacts of MRC and filling outside the Conveyance Corridor.
- Combine the modelling results for the various storm durations to produce peak results throughout the catchment for each ARI.
- Produce flood extent mapping for the selected range of design and rare / extreme events.
- Undertake climate change modelling for the 100-yr, 200-yr and 500-yr ARI events to determine the impacts.

## 1.5 Study Limitations

In utilising the flood models it is important to be aware of their limitations which can be summarised as follows:

- The models have only been calibrated / verified at locations where stream gauge and MHG records exist. This should be taken into account when considering the accuracy of results outside the influence of the gauge locations.
- These models are catchment scale and have been developed to simulate the flooding characteristics at a broad scale. As a result, smaller more localised flooding characteristics may not be apparent in the results.
- BCC 2009 ALS data has been used to represent the hydraulic model floodplain topography. Detailed checks have not been undertaken on the accuracy of the ALS data, it is assumed that the data is representative of the topography and "fit for purpose."
- The accuracy of the model results is directly linked to the following:
  - The accuracy limits of the data used to develop the model (e.g. ALS, survey information, bridge data, etc).
  - The accuracy and quality of the hydrometric data used to calibrate / verify the models.
  - The number of historical stream gauge / MHG locations throughout the catchment.
  - The purpose of the study (i.e. catchment / broad-scale or detailed).

# 2.0 Catchment Description

## 2.1 Catchment and Waterway Characteristics

The combined catchment area of Moolabin Creek and Rocky Water Holes Creek is approximately 11.8 $km^2$. Rocky Water Holes Creek is the larger with a catchment area of 6.2 $km^2$ compared with 5.6 $km^2$ for Moolabin Creek. Rocky Water Holes Creek joins Moolabin Creek approximately 2 km upstream of its outfall into Oxley Creek. The Moolabin Creek outfall is located 1.8 km upstream of the mouth of Oxley Creek.

### 2.1.1 Moolabin Creek

Moolabin Creek Catchment is bounded by Norman Creek Catchment (north-east and east) and Rocky Water Holes Creek Catchment (south). Local catchments of the Brisbane River and Oxley Creek comprise the remainder.

The highest elevation in the catchment is approximately 125 m AHD and is situated along the eastern catchment boundary within the Toohey Forest Conservation Park. The catchment headwaters are in the Toohey Forest Conservation Park, of which this area is characterised by steep slopes (up to 1v:4h) and dense / forested vegetation.

Moolabin Creek is an open waterway from just downstream of Ipswich Road to its confluence with Oxley Creek, a length of approximately 4.6 km. From review of a 1946 aerial image, it would appear that the creek (or tributary thereof) originally extended about 1.5 km further upstream of Ipswich Road, however this reach has been since been culverted / piped as part of the development of the area. The open waterway has an upstream invert level of 13.6 mAHD at Ipswich Road and an average bed slope of approximately 0.35 %. The upper section of the open waterway is the steepest section with an average bed slope of 0.8 % for the first 1 km length.

The creek corridor is quite heavily vegetated in the upper and lower reaches. The middle reach traverses the Brisbane Golf Course, between Fairfield Road and Curzon Street. Within this reach the creek has been heavily modified to suit the layout of the golf course. These modifications include numerous fairway bridges, culverts, online and offline ponds as well as channel realignment and bifurcation.

The creek tributaries generally consist of large stormwater drainage pipes / box culverts which drain heavily developed land parcels on both sides of the creek. There are no large open waterway tributaries within the catchment.

The creek is subject to downstream hydraulic interaction from a number of sources including:

- Brisbane River, and
- Oxley Creek

The middle and lower sections of the creek are also subject to tidal interaction.

### 2.1.2 Rocky Water Holes Creek

Rocky Water Holes Creek Catchment is bounded by Norman Creek Catchment (north-east), Moolabin Creek Catchment (north), Bulimba Creek Catchment (east) and Stable Swamp Creek Catchment (south and west). The catchment is reasonably elongated and uniform in shape, with an average length to width ratio of approximately 3 to 1.

The highest elevation in the catchment is approximately 110 m AHD and is situated along the eastern catchment boundary within the property of Griffith University and the greater Toohey Forest Conservation Park. The catchment headwaters are in the Toohey Forest Conservation Park, which is characterised by steep slopes (up to 1v:4h) and dense / forested vegetation.

Rocky Water Holes Creek is an open waterway from just downstream of Evans Road to its confluence with Moolabin Creek, a length of approximately 4.7 km. The open waterway has an upstream invert level of 24.5 mAHD and an average bed slope of approximately 0.5 %. The upper section of the open waterway is quite steep with an average bed slope of 1 % for the first 1 km length.

The creek corridor is quite heavily vegetated in the middle to upper reaches. In the lower reach the creek corridor is less vegetated and opens out into the Brisbane Golf Course prior to the confluence with Moolabin Creek. The natural creek has been heavily modified over time with 13 bridge / culvert crossings as well as straightening in some areas.

Similar to Moolabin Creek, the creek tributaries generally consist of large stormwater drainage pipes / box culverts which drain heavily developed land parcels. The largest of these piped tributaries drains approximately 8 % of the total catchment area. There are no large open waterway tributaries within the catchment.

The middle to lower reach of the creek is subject to downstream hydraulic interaction from a number of sources including:

- Brisbane River
- Oxley Creek, and
- Moolabin Creek

In the vicinity of the confluence with Moolabin Creek, the creek is also subject to tidal interaction.

## 2.2 Land Use

The total catchment area is effectively fully developed with the primary land-use being low / low to medium density residential development. There are also areas of significant industrial development adjacent to both Moolabin and Rocky Water Holes Creeks.

There are scattered green space areas (e.g. urban parks) throughout the catchment. The largest urban green space is the Brisbane Golf Course, which occupies a significant portion of the lower catchment area.

The upper region of the Rocky Water Holes Catchment lies within the heavily forested Toohey Forest Conservation Park.

A number of significant transportation corridors cross both creeks, including:

- Ipswich Road
- Beaudesert Road
- Fairfield Road, and
- Railway Corridor including the Gold Coast and Grafton lines.

Appendix C provides a figure indicating the catchment land-use, which is based upon BCC City Plan 2014.

# 3.0 Hydrometric Data and Storm Selection

## 3.1 Selection of Historical Storm Events

Table 3.1 indicates the more significant flooding events which have occurred within the catchment over the previous 36 years. This table includes the peak flood level in Rocky Water Holes Creek just downstream of Fairfield Road as well as the availability of stream gauge / MHG information. The table indicates that the March 2001 event is the largest flood to have occurred within the catchment in recent history.

Table 3.1 – Historical Peak Levels downstream of Fairfield Road on Rocky Water Holes Creek

| Event | Peak Flood Level (m AHD) | Recorded Hydrograph at Stream Gauge | Number of MHGs and/or recorded levels |
|---|---|---|---|
| December 1978 | 4.84 | No | 4 |
| May 1980 | 4.80 | No | 4 |
| December 1980 | 4.80 | No | 2 |
| November 1981 | 4.90 | No | 5 |
| January 1982 | 4.50 | No | 2 |
| June 1983 | 4.93 | No | 6 |
| April 1984 | 5.06 | No | 5 |
| July 1988 | - | No | 7 |
| March 1992 | 4.75 | No | 6 |
| January 1996 | 4.54 | No | 5 |
| May 1996 | 4.84 | No | 8 |
| January 1998 | 4.91 | No | 6 |
| January 1999 | 4.98 | No | 9 |
| March 2001 | 5.31 | No | 12 |
| November 2004 | 4.67 | No | 8 |
| February 2010 | 4.63 | Yes | 6 |
| January 2013 | 4.79 | Yes | 8 |
| November 2014 | 4.75 | Yes | 7 |

The selection of specific historical events for calibration and verification was based upon the following criteria:

- Higher priority for those events with consistent rainfall throughout the catchment.
- Higher priority for those events which had readily available recorded hydrograph data at the Stream Gauge.

- Higher priority for events where the catchment / creek conditions are similar to the present.
- Higher priority for larger events.
- Higher priority for events which had the greatest number of MHGs in operation.

On the basis of these selection criteria, the following events were selected for calibration and verification:

- Calibration
  - January 2013
  - February 2010
  - March 2001

- Verification
  - November 2004

The November 2014 event was considered for calibration / verification. However, it was not chosen because there were no MHG records within Moolabin Creek and there was considerable variation in rainfall across the catchment.

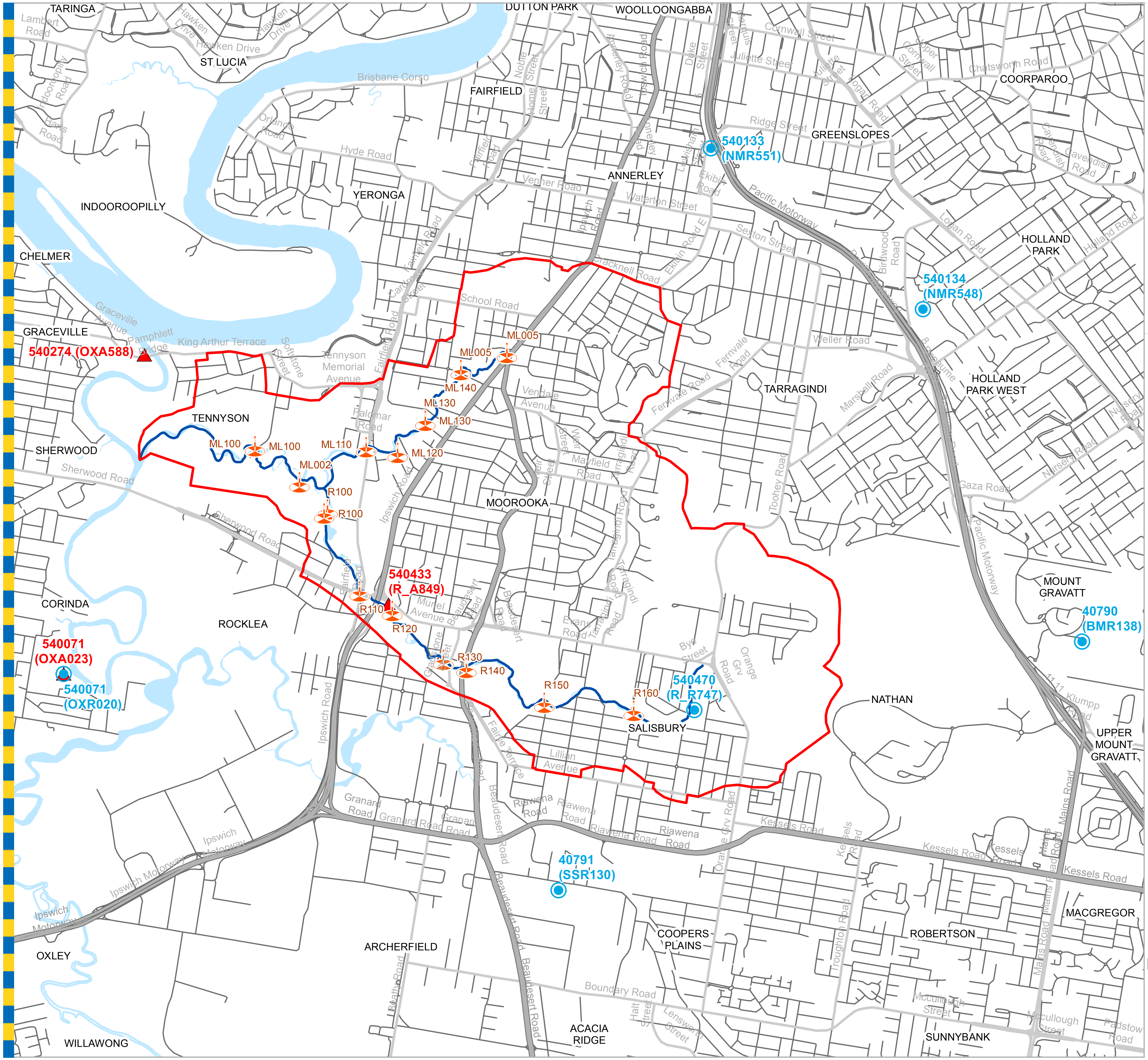
## 3.2 Availability of Historical Data for Selected Storms

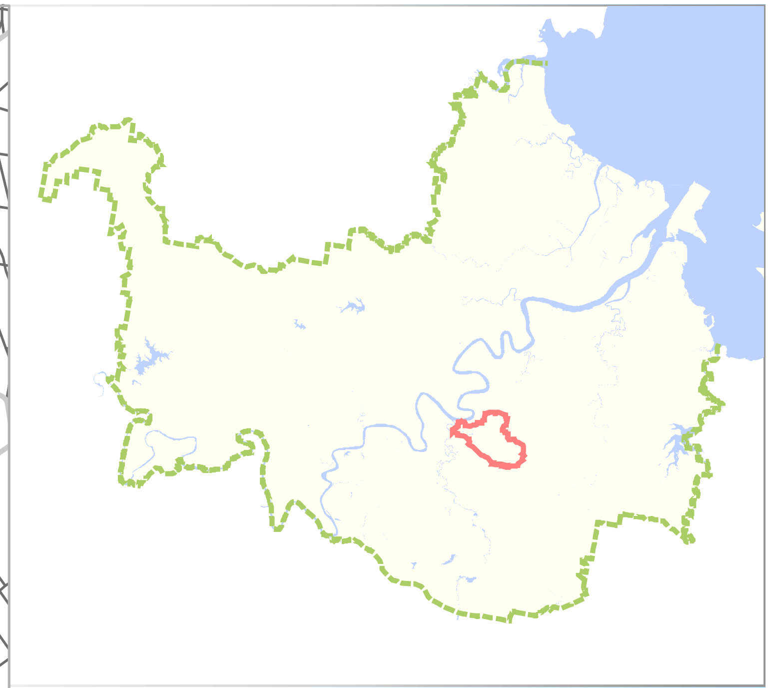
### 3.2.1 Continuous Recording Rainfall Stations

Six rainfall stations were utilised for the calibration and verification events. Figure 3.1 and Table 3.2 indicate the location and current status of each rainfall station. One rainfall station (540470) is located at Dulcie Street within the study area catchment with the remaining five outside. Two of these gauges (540133 and 40791) have been closed, however they were operational for the March 2001 event.

Table 3.2 – Rainfall Station details

| Gauge ID | Old BCC ID | Catchment | Location | Current Status |
|---|---|---|---|---|
| 540470 | R_R747 | Rocky Water Holes Creek | Dulcie Street, Salisbury | Open |
| 540133 | NMR551 | Norman Creek | SE Freeway, Green Slopes | Closed |
| 540134 | NMR548 | Norman Creek | Joachim Street, Holland Park | Open |
| 540071 | OXR020 | Oxley Creek | Corinda High, Corinda | Open |
| 40790 | BMR138 | Bulimba Creek | Griffith Uni, Mount Gravatt | Open |
| 40791 | SSR130 | Stable Swamp Creek | Musgrave Road, Coopers Plains | Closed |



**Legend**

- Maximum Height Gauges
- Pluviograph Stations
- Continuous Recording Stream Gauges
- Creek Centreline
- Freeways/Highways
- Major Roads
- Streets
- Catchment Area

N W E S

0 320 640 960
Metres

Prepared : 089958
Checked : NC
Revision : 0
Publication Date : 22 Jan 2015
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# Figure 3.1: Catchment Map and Gauge Location

Location: G:\BICD\Proj15\150339_Moolabin_Rocky_Waterholes_CK\ArcGIS\GIM_150339_002_A3L.mxd

GIM - 150339 - 002

Table 3.3 indicates the availability of the rainfall station data for each of the selected storm events.

Table 3.3 – Rainfall Station data availability

| Gauge ID | Old BCC ID | Location | Data Availability | | | |
|---|---|---|---|---|---|---|
| | | | January 2013 | February 2010 | November 2004 | March 2001 |
| 540470 | R_R747 | Dulcie Street, Salisbury | ✓ | ✓ | ✓ | × |
| 540133 | NMR551 | SE Freeway, Green Slopes | × | × | × | ✓ |
| 540134 | NMR548 | Joachim Street, Holland Park | ✓ | ✓ | ✓ | ✓ |
| 540071 | OXR020 | Corinda High, Corinda | ✓ | ✓ | ✓ | ✓ |
| 40790 | BMR138 | Griffith Uni, Mount Gravatt | ✓(1) | ✓(1) | ✓(1) | ✓ |
| 40791 | SSR130 | Musgrave Road, Coopers Plains | × | × | × | ✓ |

(1) Data available but not used

### 3.2.2 Continuous Recording Stream Gauges

Continuous recording stream height gauges collect instantaneous water level information over time. There is one stream gauge (540433 - R_A849) operational within the total catchment area. This gauge is located on Rocky Water Holes Creek, upstream of Ipswich Road and has been in operation since 2006. This results in data only being available for those events after 2006.

At the location of the gauge, the creek invert level is approximately 2.2 mAHD. At this level the gauge is not subject to tidal interaction, based on a normal tidal range. However, the location of the gauge is such that it can be subject to backwater effects from Moolabin Creek, Oxley Creek and the Brisbane River. The location of the gauge is indicated in Figure 3.1.

### 3.2.3 Maximum Height Gauges (MHGs)

Maximum Height Gauges (MHGs) record the maximum water level experienced in a flooding event at the gauge location. MHG data is manually read by the BCC Hydrometric Officer following the flooding event. In some instances where the gauge has malfunctioned during the event, the maximum water level has been based upon a nearby debris mark.

Table 3.4 indicates the period of operation for the MHGs on both Moolabin Creek and Rocky Water Holes Creek. There are 14 MHGs within the total catchment area of which three have been decommissioned since April 2001. Of the 11 operating MHGs, there are currently four on Moolabin Creek and seven on Rocky Water Holes Creek.

Table 3.5 indicates the availability of MHG data for each flooding event. It is apparent that the March 2001 event has the greatest number of readings, although most of the readings are from debris marks. Generally, there is limited MHG data for Moolabin Creek.

Table 3.4 – MHG period of record

| Creek | Gauge ID | Location | Records From | Records To |
|---|---|---|---|---|
| Moolabin | ML100 | d/s of Curzon Street Bridge | August 1977 | Present |
| | ML002 | Brisbane Golf Course | August 1977 | April 2001 |
| | ML110 | d/s Fairfield Road Culvert | August 1977 | April 2001 |
| | ML120 | u/s Evesham Street Culvert | August 1977 | Present |
| | ML130 | 75 m d/s Lucy Street Culvert | August 2010 | Present |
| | ML005 | u/s Gow Street Culvert | August 1977 | April 2001 |
| | ML140 | u/s Gow Street Culvert | August 2010 | Present |
| Rocky Water Holes | R100 | Brisbane Golf Course | August 1977 | Present |
| | R110 | d/s Fairfield Road Bridge | August 1977 | Present |
| | R120 | 65 m u/s Ipswich Road | August 1977 | Present |
| | R130 | d/s Gladstone Street Culvert | August 1977 | Present |
| | R140 | u/s Beaudesert Road Culvert | August 1977 | Present |
| | R150 | d/s Assembly Street Bridge | August 1977 | Present |
| | R160 | u/s McCarthy Road Culvert | August 1977 | Present |

Table 3.5 – Maximum Height Gauge data availability

| Creek | Gauge ID | Data Availability | | | |
|---|---|---|---|---|---|
| | | January 2013 | February 2010 | November 2004 | March 2001 |
| Moolabin | ML100 | ✓ | × | ✓ | ✓ |
| | ML002 | × | × | × | ✓(1) |
| | ML110 | × | × | × | ✓(1) |
| | ML120 | × | × | ✓(1) | ✓(1) |
| | ML130 | ✓ | × | × | × |
| | ML005 | × | × | × | ✓(1) |
| | ML140 | ✓ | × | × | × |
| Rocky Water Holes | R100 | ✓ | ✓ | ✓ | ✓ |
| | R110 | ✓ | ✓ | ✓ | ✓ |
| | R120 | × | × | ✓ | ✓(1) |
| | R130 | ✓ | ✓ | ✓ | ✓ |
| | R140 | ✓ | ✓ | × | ✓(1) |
| | R150 | × | - | ✓(1) | ✓ |
| | R160 | × | ✓ | ✓ | ✓ |

(1) Reading from debris mark

### 3.2.4 Downstream Boundary Information

There is no stream gauge at the mouth of Moolabin Creek from which to extract a downstream boundary. Therefore, it was required to utilise the closest gauges on Oxley Creek and develop a water level versus time boundary for each event.

The closest stream gauges are 540071 (OXA023) at Corinda High, approximately 3 km upstream and 540274 (OXA588) approximately 1.5 km downstream at the mouth of Oxley Creek. To create the boundary, a linear interpolation of the water level was undertaken between the two gauges. This is discussed further in Section 5.3.5.

## 3.3 Characteristics of Historical Events

### 3.3.1 January 2013 event

This event was a relatively small flooding event which produced a flood level of 5.11 m AHD at the stream gauge on Rocky Water Holes Creek. Minor flooding occurred in some localised areas in the middle and lower reaches of both creeks. Flooding of these lower areas was also a result of backwater due to Oxley Creek / Brisbane River, of which the peak flood levels occurred approximately a day after the peak in the Moolabin Creek / Rocky Water Holes Creek.

The event rainfall was consistent over the upper and middle portions of both catchments with approximately 170 mm being recorded on the 27$^{th}$ January. In the lower reaches the rainfall was less intense with only 134 mm being recorded at Rainfall Station 540071 (OXR020) at Corinda High. The most intense burst occurred over 6 hours between 12 noon and 6 pm on the 27$^{th}$ January, where approximately 112 mm of rainfall was recorded at Rainfall Station 540470 (R_R747) at Dulcie Street. The cumulative rainfall and rainfall hyetograph for each rainfall station is presented in Appendix A.

Table 3.6 indicates the 4-day and 14-day antecedent rainfall as well as statistics on the event rainfall at three rainfall stations. The catchment would have been quite saturated at the time of the event as it experienced approximately 80 mm of rainfall in the 4-day lead up to the event, with 49 mm occurring on the 26$^{th}$ January and 27 mm from midnight to 9am on the 27$^{th}$ January. The total rainfall experienced in 4 days from the 25$^{th}$ January was approximately 300 mm.

Due to the long continuous nature of the rainfall it is conceivable that there may have been some baseflow contribution to the total flow hydrograph.

Table 3.6 - Rainfall characteristics (January 2013 event)

| Gauge ID | Old BCC ID | Location | Antecedent Rainfall (mm) | | Event Rainfall (mm) | |
|---|---|---|---|---|---|---|
| | | | 14-day | 4-day | 27$^{th}$ January (peak 3hr burst) | 27$^{th}$ January (full day) |
| 540470 | R_R747 | Dulcie Street, Salisbury | 93 | 86 | 77 | 172 |
| 540134 | NMR548 | Joachim Street, Holland Park | 94 | 88 | 76 | 175 |
| 540071 | OXR020 | Corinda High, Corinda | 79 | 71 | 61 | 134 |

Figure 3.2 indicates the IFD curve for the three rainfall stations when compared to the AR&R IFD curve generated at the catchment centroid. The equivalent design rainfall ARI at Rainfall Station 540470 (R_R747) at Dulcie Street would have been as follows:

- 1 hour rainfall: < 1-yr ARI
- 2 hour rainfall: 2-yr ARI
- 3 hour rainfall: 2-yr to 5-yr ARI
- 6 hour rainfall: 10-yr to 20-yr ARI

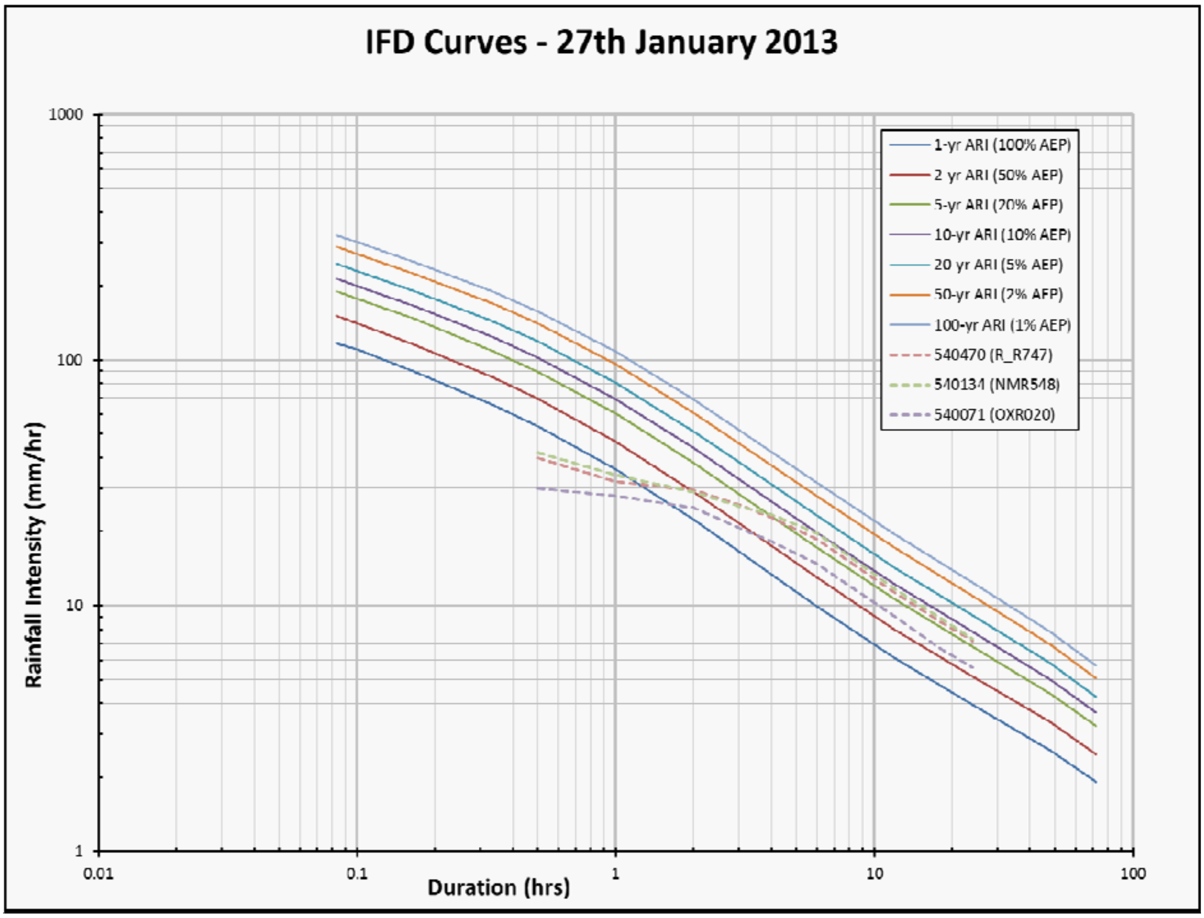



Figure 3.2: IFD Curve for January 2013 event.

### 3.3.2 February 2010 event

This event was a relatively small flooding event which produced a flood level of 5.04 m AHD at the stream gauge on Rocky Water Holes Creek. Minor flooding occurred in some localised areas in the middle and lower reaches of both creeks.

The event occurred from 11:30pm on the 6th February to around 9am on the 7th February. The most intense burst occurred over 4.5 hours between 12:30 am and 5 am, where approximately 80 mm to 110 mm of rainfall fell across the catchment. The event was more intense in the upper section of the catchment compared with the lower section of the catchment; with the recorded peak 3 hour burst up to 50 % higher. The cumulative rainfall and rainfall hyetograph for each rainfall station is presented in Appendix A.

Table 3.7 indicates the 4-day and 14-day antecedent rainfall as well as statistics on the event rainfall at three rainfall stations. The catchment experienced approximately 15 mm of rainfall in the 4-day lead up to the event and 55 mm in the preceding 14 days, meaning that the soil is unlikely to have been saturated when the event occurred.

Table 3.7 - Rainfall characteristics (February 2010 event)

| Gauge ID | Old BCC ID | Location | Antecedent Rainfall (mm) | | Event Rainfall (mm) | |
|---|---|---|---|---|---|---|
| | | | 14-day | 4-day | 7th February (peak 3hr burst) | 6th – 7th February |
| 540470 | R_R747 | Dulcie Street, Salisbury | 56 | 20 | 89 | 133 |
| 540134 | NMR548 | Joachim Street, Holland Park | 55 | 12 | 82 | 115 |
| 540071 | OXR020 | Corinda High, Corinda | 54 | 16 | 60 | 107 |

Figure 3.3 indicates the IFD curve for the three rainfall stations when compared to the AR&R IFD curve generated at the catchment centroid. The equivalent design rainfall ARI at Rainfall Station 540470 (R_R747) at Dulcie Street would have been as follows:

- 1 hour rainfall: 2-yr to 5-yr ARI
- 2 hour rainfall: 2-yr to 5-yr ARI
- 3 hour rainfall: 5-yr ARI

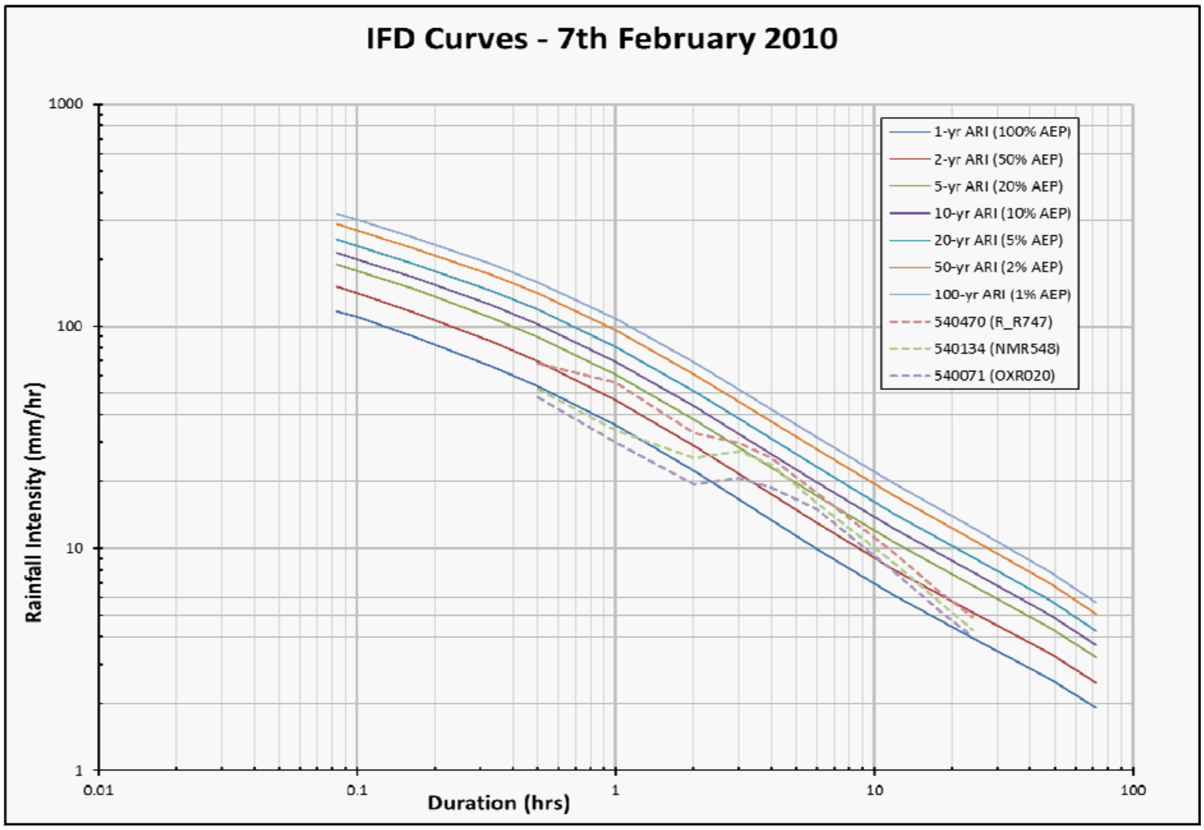


Figure 3.3: IFD Curve for February 2010 event.

### 3.3.3 November 2004 Event

This event recorded a flood level of 5.6 m AHD on Rocky Water Holes Creek at MHG R120; just upstream of the current location of the stream gauge.

The event occurred over 13 hours from midnight to around 1pm on the 7th November. The most intense burst occurred over 3.5 hours between 9:30 am and 1 pm, where approximately 95 mm to 160 mm of rainfall fell across the catchment. The event comprised highly variable rainfall with considerably more intense rainfall occurring towards the bottom of the catchment. This high spatial variability of the rainfall is not ideal for calibration as it leads to significant uncertainty with regards to the rainfall that actually fell on the catchment. The cumulative rainfall and rainfall hyetograph for each rainfall stations is presented in Appendix A.

Table 3.8 indicates the 4-day and 14-day antecedent rainfall as well as statistics on the event rainfall at three rainfall stations. The catchment experienced approximately 25 mm of rainfall in the 4-day lead up to the event, meaning that the soil is unlikely to have been saturated when the event occurred.

Table 3.8 - Rainfall characteristics (November 2004 event)

| Gauge ID | Old BCC ID | Location | Antecedent Rainfall (mm) | | Event Rainfall (mm) | |
|---|---|---|---|---|---|---|
| | | | 14-day | 4-day | 7th November (peak 3hr burst) | 6th – 8th November |
| 540470 | R_R747 | Dulcie Street, Salisbury | 34 | 34 | 109 | 180 |
| 540134 | NMR548 | Joachim Street, Holland Park | 26 | 16 | 94 | 145 |
| 540071 | OXR020 | Corinda High, Corinda | 23 | 23 | 146 | 238 |

Figure 3.4 indicates the IFD curve for the three rainfall stations when compared to the AR&R IFD curve generated at the catchment centroid. The equivalent design rainfall ARI at Rainfall Station 540470 (R_R747) at Dulcie Street would be as follows:

- 1 hour rainfall: 5-yr ARI
- 2 hour rainfall: 10-yr ARI
- 3 hour rainfall: 20-yr ARI
- 6 hour rainfall: 10-yr ARI

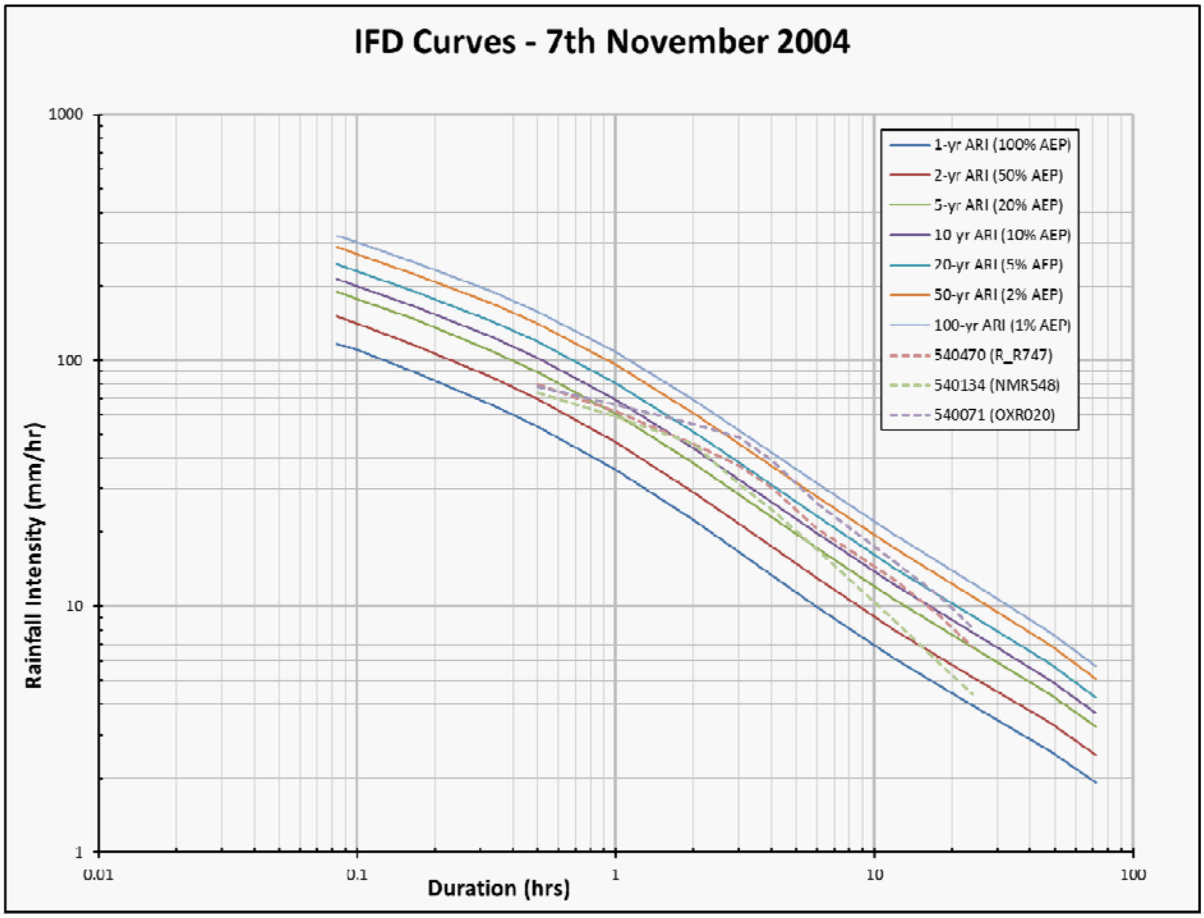


Figure 3.4: IFD Curve for November 2004 event.

### 3.3.4 March 2001 event

This event was the largest on record since gauges were installed in the catchment in 1977. A flood level of 6.61 mAHD was recorded from debris marks at the location of MHG R120; just upstream of the current location of the stream gauge. The majority of the MHG readings were from debris marks as most of the gauges were overtopped and / or destroyed.

The event occurred as one intense burst over a 3.5 hour period from 3:30pm to 7pm on the 9$^{th}$ March. During this period 135 mm to 215 mm fell across the catchment, with the more intense rainfall occurring towards the top of the catchment. This high spatial variability of the rainfall is not ideal for calibration as it leads to significant uncertainty with regards to the rainfall that actually fell on the catchment. The cumulative rainfall and rainfall hyetograph for each rainfall station is presented in Appendix A.

Table 3.9 indicates the 4-day and 14-day antecedent rainfall as well as statistics on the event rainfall at three rainfall stations. The catchment would have been virtually dry as there was effectively no rainfall in the weeks preceding the event.

Table 3.9 - Rainfall characteristics (March 2001 event)

| Gauge ID | Old BCC ID | Location | Antecedent Rainfall (mm) | | Event Rainfall (mm) | |
|---|---|---|---|---|---|---|
| | | | 14-day | 4-day | 9th March (peak 3hr burst) | 9th March |
| 40790 | BMR138 | Griffith Uni, Mount Gravatt | 8 | 7 | 200 | 217 |
| 540134 | NMR548 | Joachim Street, Holland Park | 6 | 5 | 194 | 206 |
| 540071 | OXR020 | Corinda High, Corinda | 5 | 4 | 136 | 137 |
| 540133 | NMR551 | SE Freeway, Green Slopes | 6 | 5 | 163 | 171 |
| 40791 | SSR130 | Musgrave Road, Coopers Plains | 5 | 4 | 171 | 175 |

Figure 3.5 indicates the IFD curve for the three rainfall stations when compared to the AR&R IFD curve generated at the catchment centroid. The equivalent design rainfall ARI at Rainfall Station 540133 (NMR551) at the South-east Freeway (Greenslopes) would be as follows:

- 1 hour rainfall: > 100-yr ARI
- 2 hour rainfall: > 100-yr ARI
- 3 hour rainfall: 100-yr ARI

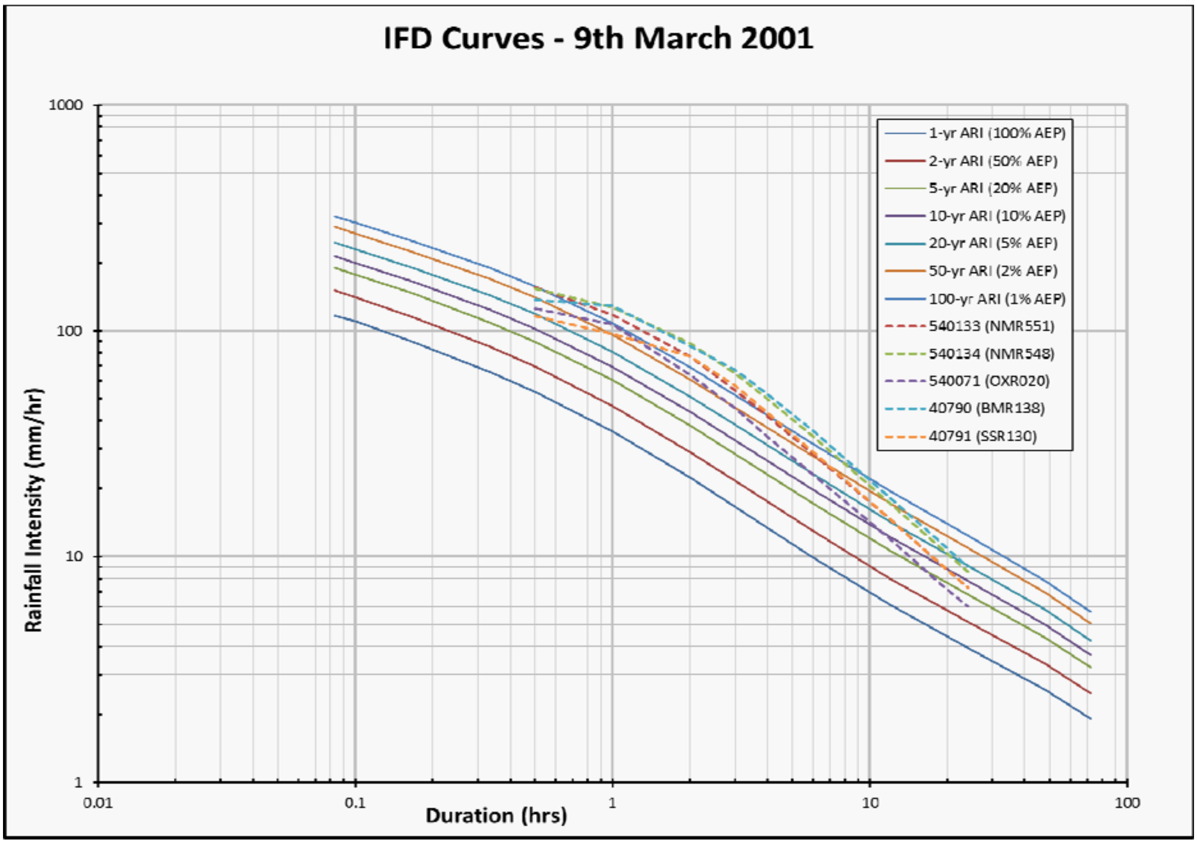



Figure 3.5: IFD Curve for March 2001 event.

# 4.0 Hydrologic Model Development and Calibration

## 4.1 Overview

The hydrologic model simulates the rainfall-runoff process within the catchment and calculates the flow hydrograph at the outlet of each sub-catchment. An XP-RAFTS (version 2013) model was developed for the total catchment area including both Moolabin Creek and Rocky Water Holes Creek.

## 4.2 Sub-catchment Data

### 4.2.1 General

This section describes the sub-catchment parameters used in the XP-RAFTS model. The "two sub-catchment" approach was used to separately define the impervious and pervious sub-catchments. This approach is recommended for highly urbanised catchment areas such as this study area.

The adopted sub-catchment parameters for the calibration and verification events are presented in Appendix B. The same sub-catchment parameters have been used for all events due to the relatively recent age of the calibration and verification events and the minimal changes in catchment / channel topography and development during this period.

### 4.2.2 Sub-catchment Delineation

The XP-RAFTS model comprises 33 sub-catchments and the layout is indicated in Figure 4.1. Based on a total catchment area of 11.8 $km^2$, this results in an average sub-catchment size of 0.36 $km^2$. The sub-catchment delineation was based upon the 2009 ALS contours and considered the major piped tributary locations as well as man-made boundaries such as the railway, roads and creek crossings.

### 4.2.3 Sub-catchment Slope

Sub-catchment slopes have been calculated from the topography by identifying indicative flow paths and associated equal area slopes. The sub-catchment slopes ranged from 5.3 % for Sub-catchment G1 to 0.3 % for Sub-catchment A8.

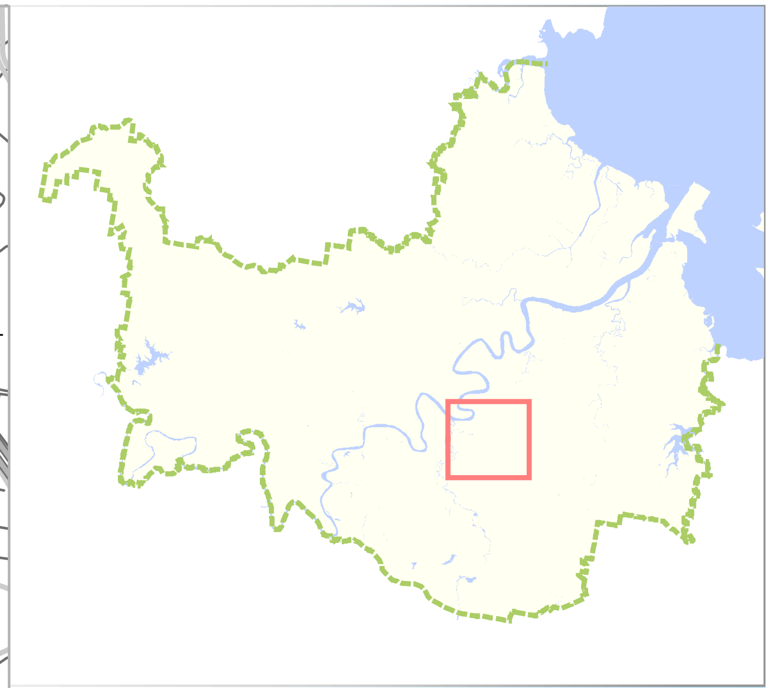
### 4.2.4 Percentage Impervious

The percentage impervious values were generally derived from the catchment land-use types, by assuming a percentage impervious for each land-use type. Where XP-RAFTS sub-catchments contained more than one type of land-use, weighted averages of the percentage imperviousness were applied for each sub-catchment.

The land-use and impervious areas were identified using BCC aerial photography and BCC City Plan[3] as indicated by the maps in Appendix C. The assumed impervious area per land-use type is also shown in a table in Appendix C.

The total catchment is considered to be fully urbanised, with the predominant land-use being low-density residential and to a lesser degree industrial. It is highly unlikely that the undeveloped

[3] *Brisbane City Plan 2014,* Brisbane City Council

**Legend**

- RAFTS Nodes
- RAFTS Links/Routes
- Freeways/Highways
- Major Roads
- Streets
- RAFTS Subcatchments
- Catchment Area

N W E S

0 225 450 675
Metres

Prepared : 089958
Checked : NC
Revision : 0
Publication Date : 22 Jan 2015
Project Number : 150339

**DATA INFORMATION**



Brisbane City Council
City Projects Office
GPO Box 1434
Brisbane Qld 4001

For more information
visit www.brisbane.qld.gov.au
or call (07) 3403 8888

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**Moolabin Creek and Rocky Water Holes Creek Flood Study**

# Figure 4.1: XP-RAFTS Model Schematisation

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GIM - 150339 - 003

forested areas within the catchment headwaters would be developed, as these are conservation areas.

### 4.2.5 Hydrologic Roughness (PERN)

The hydrologic roughness parameter (PERN) is input as a Manning's 'n' representation of the average sub-catchment roughness. Generally, a value of n = 0.015 was used for the impervious sub-catchment and a value of n = 0.04 for the pervious sub-catchment. However, in the heavily forested areas, a higher value was used for the pervious sub-catchment to reflect the significantly denser vegetation.

### 4.2.6 Link and Routing Parameters

Routing of the open waterway of both creeks was undertaken using the Muskingum-Cunge methodology, whereby the program calculates the Muskingum K and X values based on the channel cross-sectional and longitudinal characteristics. The cross-sectional shape was obtained from the previous 1997 survey sections. During the calibration process, the Brisbane Golf Course area was changed to the detention storage approach, as it provided a better representation of the significant storage effects within this area.

Link lags were used to represent the smaller tributaries, where the flow mechanism was typically a combination of piped drainage and overland flow. The link lag approach translates the base of the hydrograph (without attenuation) based on the input lag time. In reality, the link lag time will change depending on the size of the event and the capacity of the piped conduit. For example, where flow is fully contained within the conduit, the link lag time will typically be considerably quicker than if the flow is a combination of pipe flow and overland flow.

The lag time was initially calculated assuming an average travel time of 1 m/s. This value was chosen as it is more representative of larger flow events, where there is considerable overland flow as a result of the pipe capacity being exceeded. Checks were undertaken within the piped section of Moolabin Creek (upstream of Ipswich Road) and this value was found to be a good approximation of the lag time during a large flooding event.

## 4.3 Event Rainfall

### 4.3.1 Observed Rainfall

Recorded data from each calibration and verification event was incorporated into the XP-RAFTS model using a standard HYDSYS database format. The HYDSYS rainfall database which was used in the hydrological modelling incorporates recorded rainfall at five minutes intervals, noting that the rainfall gauge only records information when 1 mm or more of rain has fallen. This enabled the full rainfall period for each of the events to be modelled using a fast and reliable method.

Thiessen Polygons were utilised for each event to enable the gauged rainfall to be apportioned to each of the sub-catchments in the XP-RAFTS model. Those sub-catchments which fell totally within a polygon were fully assigned to the respective rainfall station. Those sub-catchments which bridged across two of more polygons were apportioned to the respective rainfall station based on the proportion of area within each polygon. The Thiessen Polygon distributions for the four events are presented in Appendix A for reference.

### 4.3.2 Rainfall Losses

The Initial Loss (IL) and Continuing Loss (CL) methodology was used to simulate the rainfall losses.

The IL (mm) is known to be the amount of rainfall that occurs before the start of surface runoff. The initial loss comprises factors such as interception storage (e.g. tree leaves); depression storage (e.g. ditches, surface puddles, etc.) and the initial infiltration capacity of the soil, whereby a dry soil has a larger capacity than a saturated soil.

The CL (mm/hr) is assumed to be the average loss rate throughout the remainder of the rainfall event and is predominantly dependant on the underlying soil type and porosity.

## 4.4 Stream Gauge Rating Curve

In order to undertake the hydrological calibration to the stream gauge for the January 2013 and February 2010 events, it was necessary to establish an appropriate rating curve. BCC Hydrometrics does not keep records of rating curves for stream gauges; therefore it was required to generate the rating curve using the TUFLOW hydraulic model, developed for this flood study. For further discussion on the TUFLOW model, refer to Section 5.

The location of the stream gauge is not ideal for deriving a rating curve as it is immediately upstream of the very complex hydraulic structure arrangement at Muriel Avenue. Similarly, the location has the potential to be subject to backwater in larger events from Moolabin Creek, Oxley Creek and the Brisbane River, as noted previously in Section 2.1.2.

From review of the observed flood levels for both the January 2013 and February 2010 events, it became apparent that a single rating curve would not be appropriate for both events. This is because in the January 2013 event there appears to be some form of constriction / blocking of the waterway downstream of Curzon Street, which is causing downstream controlled floodwater at the stream gauge during larger flows. In the February 2010 event, this phenomenon is not present and the rating is therefore different to January 2013 at higher flows.

Figure 4.2 indicates the rating curves used for the January 2013 and February 2010 events.

## 4.5 Calibration and Verification Procedure

### 4.5.1 General

The calibration and verification process was adopted to suit the study objectives and requirements. The general requirements were to produce a hydrologic model sufficiently robust to accurately predict design discharges without the need to run the hydraulic model. This requirement meant that the approach adopted was to undertake a separate hydrologic calibration to ensure the XP-RAFTS model was suitable to be used as a "standalone" model. The general approach adopted for the calibration and verification is indicated in Section 4.5.3.

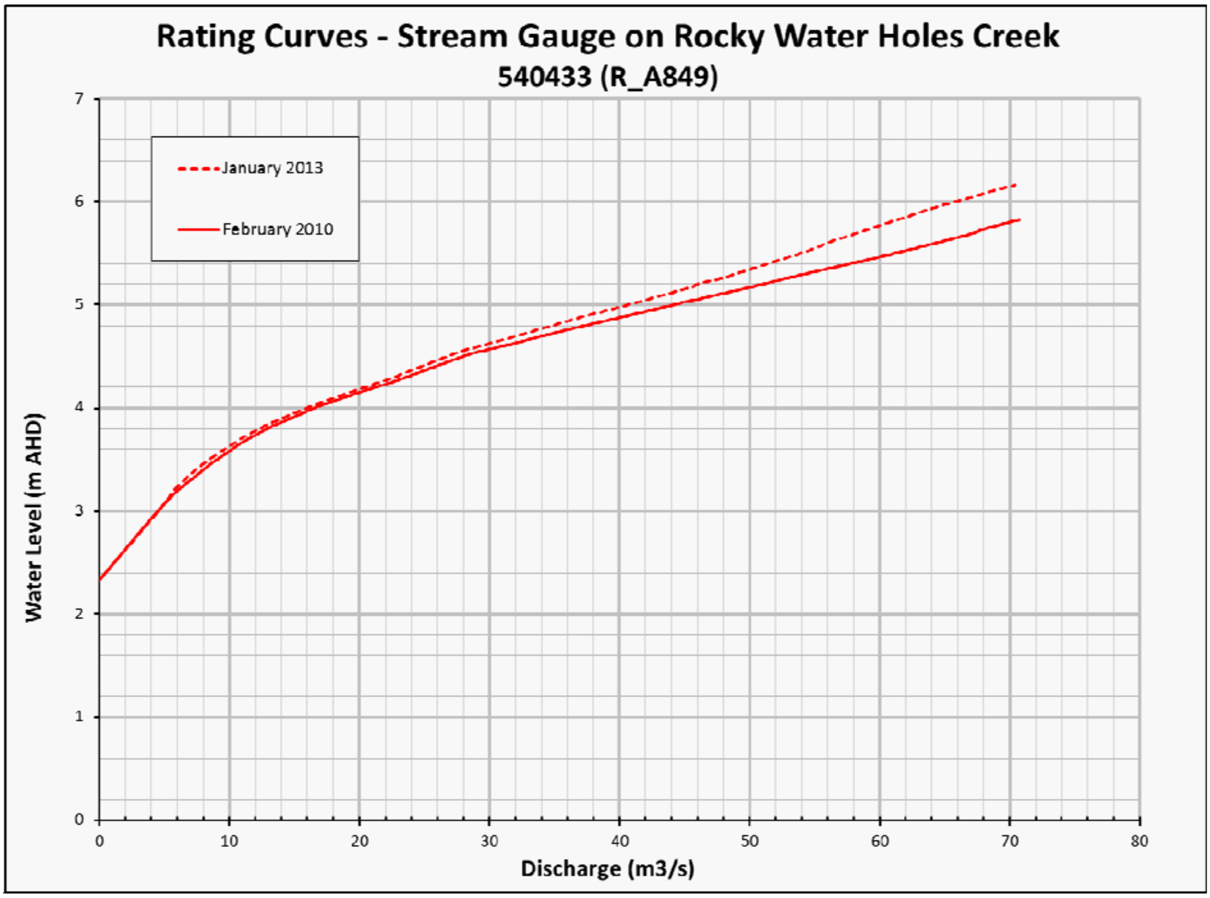


Figure 4.2: Rating Curves - Stream Gauge 540433 (R_A849)

### 4.5.2 Tolerances

The current flood study procedure document is not prescriptive in relation to the ideal hydrologic calibration and verification tolerances. For the purposes of this study, the calibration and verification process has aimed to achieve the following tolerances:

- Volume - within +20 % to -10 %
- Peak Flow - within +25 % to -15 %
- Good replication of the hydrograph shape.
- Good replication of the timing of peaks and troughs.

### 4.5.3 Methodology

The methodology applied to the calibration and verification of the XP-RAFTS model was as follows:

1) Input the observed rainfall data and apportion the rainfall to each sub-catchment. This was undertaken using the Thiessen Polygon methodology as described in Section 4.3.
2) Establish an appropriate rating curve(s) at the stream gauge and convert the stage recordings to flow. This was detailed in Section 4.4.
3) Run the calibration events (i.e. January 2013, February 2010 and March 2001) through the XP-RAFTS model and compare the simulated results against the observed flow records, if observed records are available.

4) Iteratively adjust the model parameters and re-run the model to achieve the best possible fit with the observed data. The predominant model parameters adjusted included the IL (mm), CL (mm/hr) and the storage delay time coefficient multiplier (Bx). However, the link-lag timing and also the Manning's 'n' value of the routing link were also adjusted, if considered appropriate.
5) Adopt model parameters (typically CL and Bx) based on the calibration results.
6) Run the verification event (i.e. November 2004) through the calibrated XP-RAFTS model and with use of the TUFLOW model compare the simulated flood levels against the observed flood levels at the MHGs.
7) Make adjustments to the initial loss (as required) to represent the event specific rainfall lost at the start of the event.
8) Repeat steps 2 to 7 (as necessary) following the results of the hydraulic model simulations. Refer to Section 5 for more detail on the hydraulic modelling.

## 4.6 Simulation Parameters

Table 4.1 indicates the start and finish times of the hydrologic simulations as well as the time step used.

Table 4.1 – Hydrologic Simulation Parameters

| Event | Start Time | Finish Time | Duration (hours) | Time Step (min) |
|---|---|---|---|---|
| January 2013 | 27/01/2013 0:00 | 28/01/2013 00:00 | 24 | 1 |
| February 2010 | 6/02/2010 18:00 | 7/02/2010 15:00 | 21 | 1 |
| November 2004 | 7/11/2004 6:00 | 7/11/2004 18:00 | 12 | 1 |
| March 2001 | 9/03/2001 15:00 | 10/03/2001 0:00 | 9 | 1 |

## 4.7 Hydrologic Model Calibration Results

### 4.7.1 January 2013

Figure 4.3 provides a comparison of the XP-RAFTS results and the rated flows (established using the adopted rating curve) at the stream gauge on Rocky Water Holes Creek. The results indicate a good fit to the peak flow, shape and timing of the hydrograph. However, the simulated flow and volume is generally lower than the rated flow for the entire event. This could be due to a number of factors including: lower recorded rainfall than actually fell; the presence of baseflow; inaccuracies in the rating curve at low flows; backwater effects at the stream gauge; etc. Further discussion on the calibration is provided in Section 5.5.

The adopted XP-RAFTS parameters as part of the calibration were as follows:

- Impervious Area: IL = 0 mm, CL = 0 mm/hr
- Pervious Area: IL = 0 mm, CL = 0 mm/hr
- Bx = 2.5

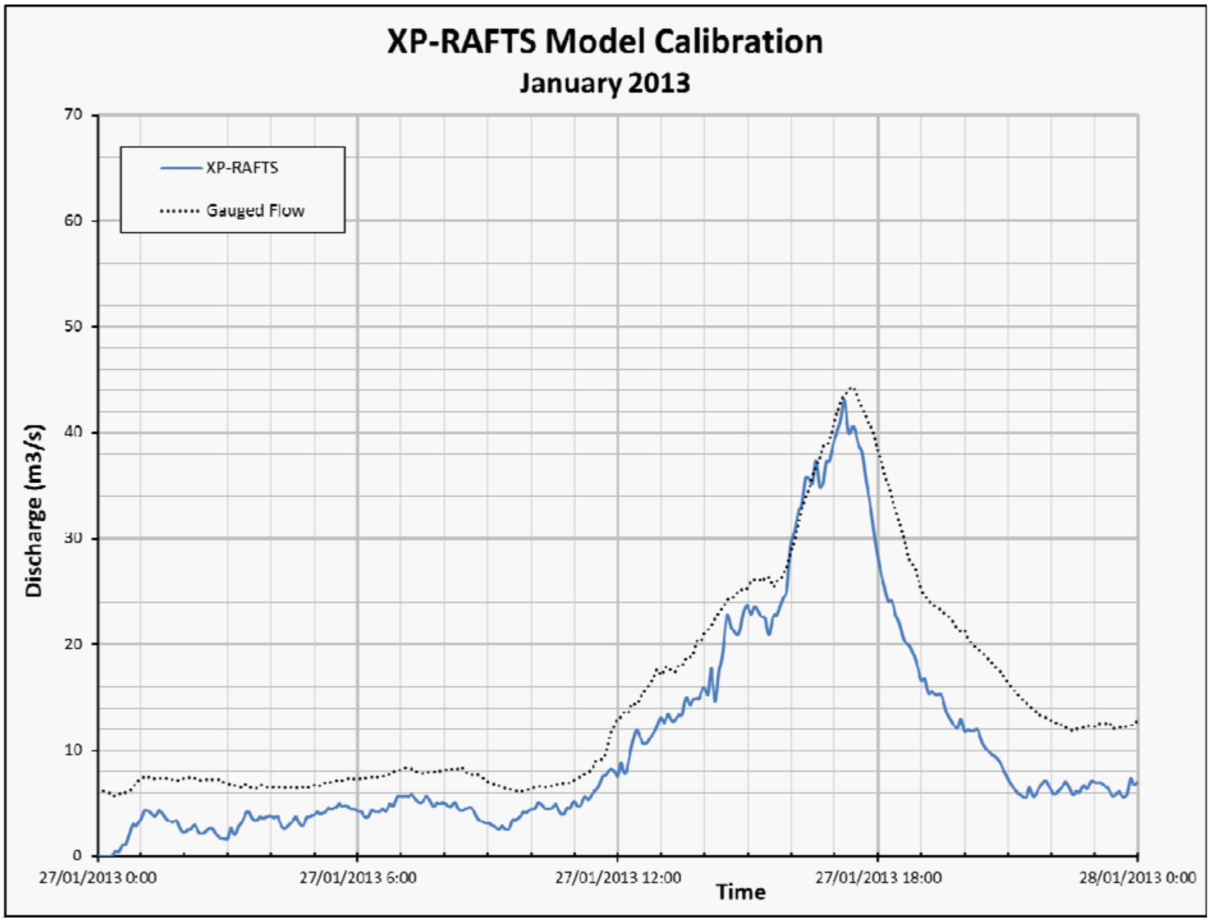


Figure 4.3: XP-RAFTS Model Calibration (January 2013)

### 4.7.2 February 2010

Figure 4.4 provides a comparison of the XP-RAFTS results and the rated flows (established using the adopted rating curve) at the stream gauge on Rocky Water Holes Creek. The results indicate a reasonable replication of the shape and timing of the rated hydrograph. However, the simulated peak flow is considerably higher (+33 %) than the rated peak flow. This is most likely due to the high spatial variability of the rainfall not being fully captured and represented within the hydrology model. Further discussion on the calibration is provided in in Section 5.5.

The adopted XP-RAFTS parameters as part of the calibration were as follows:

- Impervious Area: IL = 0 mm, CL = 0 mm/hr
- Pervious Area: IL = 20 mm, CL = 0 mm/hr
- Bx = 2.5

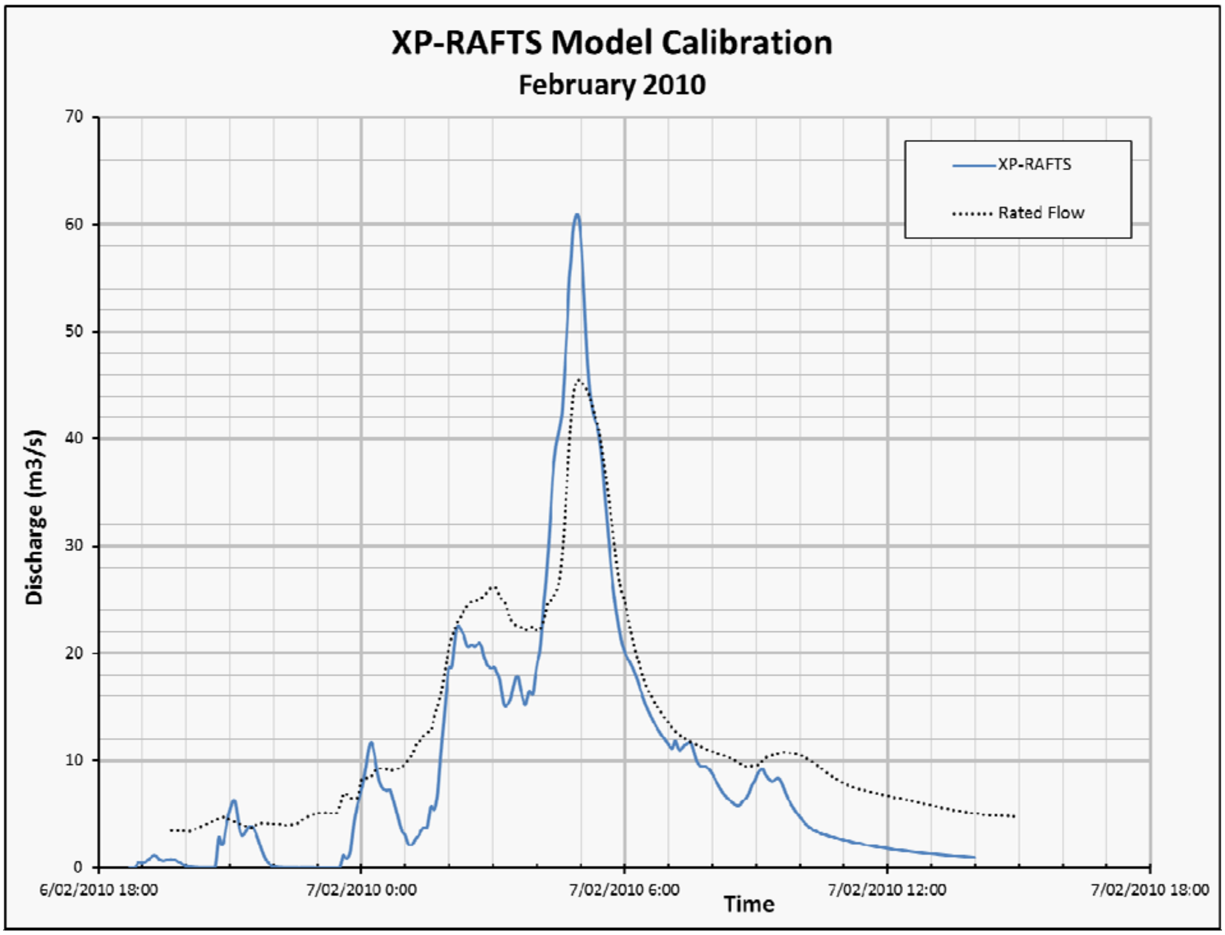



Figure 4.4: XP-RAFTS Model Calibration (February 2010)

### 4.7.3 March 2001

As there are no continuous stream gauge records for the March 2001 event, the hydrologic calibration to an observed hydrograph was unable to be undertaken. As such, there was no means of calibrating the hydrologic model to the observed flood volume and shape. Instead, a joint calibration with the hydraulic model was undertaken, whereby parameters were adjusted with the aim of achieving a good fit to the peak flood (MHG) levels. Refer to in Section 5.5 for the results of this joint calibration.

The adopted XP-RAFTS parameters as part of the calibration were as follows:

- Impervious Area: IL = 0 mm, CL = 0 mm/hr
- Pervious Area: IL = 10 mm, CL = 0 mm/hr
- Bx = 2.5

## 4.8 Hydrologic Model Verification Results

### 4.8.1 Adopted model parameters

Table 4.2 indicates the parameters adopted from the hydrologic calibration of the three historical events. These parameters were used to verify the XP-RAFTS model to the one verification event (i.e. November 2004).

Table 4.2 – Adopted XP-RAFTS parameters

| Parameter | Description | Adopted Value |
|---|---|---|
| n | Storage non-linearity exponent | -0.285 |
| Bx | Storage delay time coefficient multiplier | 2.5 |
| Imp CL | Impervious Area Continuing Loss (mm/hr) | 0 |
| Perv CL | Pervious Area Continuing Loss (mm/hr) | 0 |

### 4.8.2 November 2004

Similar to the March 2001 event, there are no continuous stream gauge records to allow the hydrologic verification to an observed hydrograph to be undertaken. Therefore, a joint verification with the hydraulic model was required to enable the verification of the XP-RAFTS parameters. The results of this verification are provided in Section 5.6.

The adopted XP-RAFTS parameters as part of the verification were as follows:

- Impervious Area: IL = 0 mm, CL = 0 mm/hr
- Pervious Area: IL = 40 mm, CL = 0 mm/hr
- Bx = 2.5

# 5.0 Hydraulic Model Development and Calibration

## 5.1 Overview

The previous hydraulic model of Moolabin Creek and Rocky Water Holes Creek was a 1d MIKE11 model, developed for the previous flood study(s). To achieve best practice, it was considered appropriate to upgrade the 1d model to a 1d / 2d model. This would provide better representation of the floodplain flooding characteristics in the middle to lower sections of the creek as well as a more efficient tool to produce flood mapping products.

The TUFLOW hydrodynamic model (version 2013-12-AD) was selected for the hydraulic analysis of Moolabin Creek and Rocky Water Holes Creek.

## 5.2 Available Data

The following data was utilised in the development of the TUFLOW model:

- MIKE11 model – 2006 Moolabin Creek and Rocky Water Holes Creek Flood Study
- BCC 1997 cross-section survey
- BCC November 2014 cross-section survey (eight cross-sections)
- BCC aerial photography – 1997 to 2013
- BCC 2009 Airborne Laser Scanning (ALS) data
- 2014 BCC City Plan
- Hydraulic structure drawings / reference sheets. Refer to Appendix H for further details.
- BCC Cadastre and GIS databases

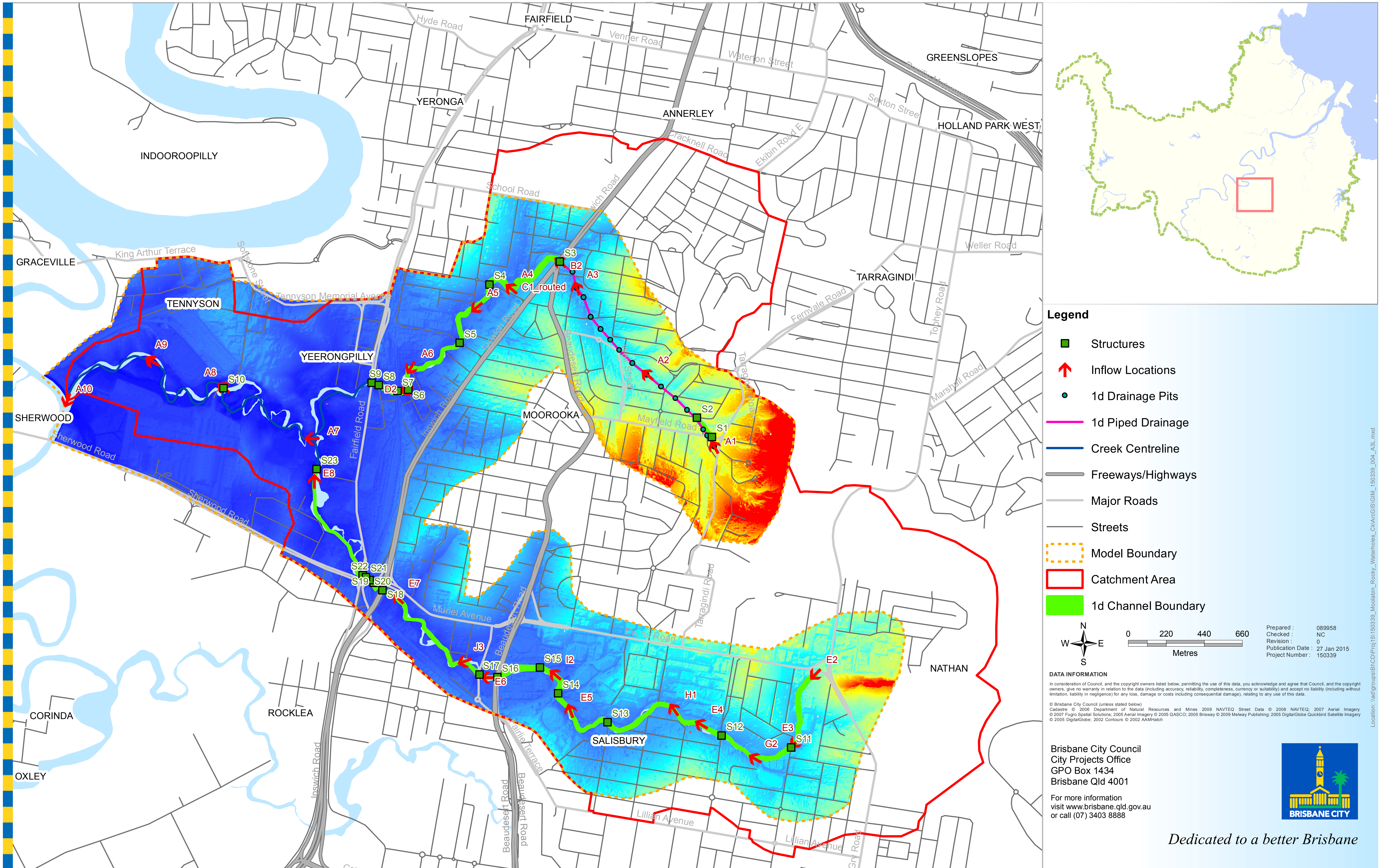
## 5.3 Model Development

### 5.3.1 Model Schematisation

Figure 5.1 indicates the extents of the TUFLOW model, as well as the inflow locations and the hydraulic structures included in the model. The model consists largely of a 1d / 2d linked schematisation, with the 1d domain modelled in ESTRY and the 2d domain in TUFLOW.

The hydraulic model can be broken up into four major sections on the basis of the creek / drainage type and the modelling methodology as follows:

- Moolabin Creek (Piped Reach) – this reach extends from Mayfield Road to Ipswich Road, a length of approximately 1.5 km. The reach is primarily piped drainage and has been modelled as 1d pipe work / 2d overland flow. The modelling approach is simplistic and only models the single trunk drainage pipeline, with the Engelhund methodology used to approximate pit / manhole losses. Overland flow can occur once the pipe capacity has been exceeded. Overland flow can re-enter the pipeline once there is excess pipe capacity and on the basis that the flow comes into contact with the 2d grid cell which contains the pit / manhole of the trunk drainage pipeline. Again, this is a simplistic approach as in reality overland flow would re-enter the trunk drainage pipeline through transverse drainage pipelines connected to the trunk drainage pipeline.




**Moolabin Creek and Rocky Water Holes Creek Flood Study**

**Figure 5.1: TUFLOW Model Schematisation**

- Moolabin Creek (Ipswich Road to the Brisbane Golf Course) – this reach extends from downstream of Ipswich Road to downstream of Fairfield Road at the upstream extent of the Brisbane Golf Course, a length of approximately 1.7 km. The reach is open waterway, includes all major hydraulic structures and has been modelled as 1d / 2d.

- Rocky Water Holes Creek (Evans Road to the Brisbane Golf Course) – this reach extends from downstream of Evans Road to the upstream extent of the Brisbane Golf Course, approximately 200 m upstream of the confluence with Moolabin Creek. This reach is open waterway, includes all major hydraulic structures and has been modelled as 1d / 2d for the entire 4.5 km length.

- Moolabin Creek (Brisbane Golf Course to Oxley Creek) – this reach includes the Brisbane Golf Course and the 1.4 km section of creek from downstream of Curzon Street to the confluence with Oxley Creek. The reach is open waterway and has been modelled as fully 2d.

### 5.3.2 Topography

<u>1d Domain</u>

The 1d open channel was generally represented by utilising the channel cross-sectional information from the previous MIKE11 model. These cross-sections were surveyed in 1997 to enable the development of this model.

The 1997 survey was supplemented with eight cross-sections from survey undertaken in November 2014. The location of the November 2014 surveyed cross-sections was selected at sites where the 1997 surveyed cross-sections appeared least representative of the channel shape.

<u>2d Domain</u>

The 2d bathymetry consisted of a 4 m grid which was created from 2009 BCC ALS data. The ALS data was triangulated to create a 12da TIN (MGA Zone 56) which was then able to be read into the TUFLOW model. Detailed checks have not been undertaken on the accuracy of the ALS data. It is assumed that the data is representative of the topography and "fit for purpose."

Within the Brisbane Golf Course, a review was undertaken of the available data and flooding characteristics which identified the following:

- The 2009 ALS data was representative of the water surface level (in lieu of bathymetry) for the creek channel and ponds.
- The previous modelling results identified that the flood surface gradient was very flat within this area, with Curzon Street acting as a hydraulic control, especially in the larger events.
- From review of the aerial photography, it was apparent that there had been some changes to the golf course layout since the 1997 survey, resulting in some changes to the creek alignment, pond alignment and fairway structures. Data was not readily available to accurately represent these changes which had occurred since the 1997 survey.
- The principle means of flood conveyance was the overbank areas, not the channel.
- Brisbane River flooding is the dominant flooding mechanism with respect to flood planning levels.

Based on this review, it was concluded that it was not warranted to model this area in significant detail, as more detail would not necessarily produce a more accurate flood level. Therefore, it was decided to utilise the ALS data and model the area as fully 2d, with the "z-line" function in TUFLOW to represent the low-flow channel.

From Curzon Street to the confluence with Oxley Creek the creek channel was represented in 2d utilising the "z-shape" function in TUFLOW. The "z-shape" function utilised invert levels based on the 1997 surveyed cross-sections.

### 5.3.3 Land Use

The Manning's 'n' values shown in Table 5.1 were adopted within the 2d section of the TUFLOW model. The assignment of the appropriate roughness values to the land-use / topographical feature was based upon experience with similar studies and relevant hydraulic literature.

The discretation of the land-use and topographical areas was undertaken utilising a combination of BCC aerial photography, BCC City Plan and a number of site visits.

Both Moolabin Creek and Rocky Water Holes Creek have areas of significant industrial development adjoining the creek. These areas are typically characterised by very large industrial buildings / sheds / warehouses surrounded by significant paved areas. Rather than adopt a single Manning's 'n' value to represent both the building and the building envelope, it was decided to adopt separate roughness values. This methodology required the industrial buildings to be digitised from the aerial photography and a high roughness value to be applied (typically n = 1.00). Manning's 'n' values for the building envelope typically ranged between 0.02 and 0.04.

In the 1d ESTRY section, the Manning's 'n' values ranged from 0.015 to 0.07, depending on the type of channel material and degree of vegetation.

### 5.3.4 Hydraulic Structures

Culverts and Bridges

The major bridge and culvert structures within the model domain were represented in the TUFLOW model. These structures generally consisted of road crossings, rail crossings and the more significant footbridge crossings. Table 5.2 indicates the location and details of the structures as well as the modelling approach used. The modelled head-loss across selected structures was checked utilising the HEC-RAS modelling software, as recommended in the TUFLOW manual. Refer to Section 5.7 for further details.

In the 1d / 2d section of the model, either of the following two approaches was used:

- 1d representation of the waterway opening with a 1d representation of the overtopping (weir).
- 1d representation of the waterway opening with a 2d representation of the overtopping (weir).

In the 2d section of the model,

- 1d representation of the waterway opening with a 2d representation of the overtopping (weir).
- 2d "layered flow constriction" approach (for bridges only).

Table 5.1 – Adopted roughness parameters

| Topographical feature / Land-use | Adopted Manning's 'n' |
|---|---|
| *City Plan Land-use* | |
| Low Density Residential | 0.12 |
| Low – Medium Density Residential | 0.15 |
| High Density Residential | 0.15 |
| Tourist Accommodation | 0.15 |
| Neighbourhood Centre | 0.15 |
| District Centre | 0.15 |
| Industrial | Refer Section 5.3.3 |
| Sport And Recreation | 0.04 |
| Open Space | 0.04 |
| Conservation | 0.08 |
| Emerging Communities | 0.06 |
| Rural | 0.04 |
| Rural Residential | 0.06 |
| Community Facilities (Community Purposes) | 0.10 |
| Community Facilities (Education Purposes) | 0.10 |
| Community Facilities (Emergency Services) | 0.15 |
| Community Facilities (Health Care Purposes) | 0.15 |
| Specialised Centres | 0.12 |
| Special Purpose (Transport Infrastructure) | 0.04 |
| Special Purpose (Utility Services) | 0.04 |
| Multi-Purpose Centre Convenience Centre | 0.15 |
| Multi-Purpose Centre Suburban Centre | 0.15 |
| *Additional Roughness* | |
| Road pavement | 0.02 |
| Road verge | 0.03 |
| Channel – concrete lined | 0.015 |
| Vegetation – light to high density | 0.035 to 0.15 |
| Buildings | 1.00 |
| Minimum Riparian Corridor (MRC) | 0.15 |

Table 5.2 – Hydraulic Structures represented in the TUFLOW model

| Creek | Structure ID | Structure location | Structure details | Modelled structure representation | Origin of data used for coding the structure |
|---|---|---|---|---|---|
| Moolabin (piped) | S1 | Mayfield Road | 3 / 900 mm RCP | 1d culvert / 2d weir | BCC Stormwater database plus onsite survey |
| Moolabin (piped) | S2 | Goodwin Terrace | 3 / 1050 mm RCP | 1d culvert / 2d weir | BCC Stormwater database plus onsite survey |
| Moolabin (piped) | - | Goodwin Terrace to Ipswich Road | 3 / 1050 mm RCP to 3 / 1650 mm RCP | 1d pipe / 2d overland flow | BCC Stormwater database |
| Moolabin (piped) | S3 | Ipswich Road | 3 / 1500 mm RCP<br>4 / 1575 mm RCP | 1d culvert / 2d weir | BCC Stormwater database plus design drawings |
| Moolabin | S4 | Gow Street | 4 / 1800 mm RCP | 1d culvert / 2d weir | 1997 Survey |
| Moolabin | S5 | Lucy Street | 5 / 1800 mm RCP | 1d culvert / 2d weir | 1997 Survey |
| Moolabin | S6 | Evesham Street | 4 / 3550 x 2400 mm RCBC | 1d culvert / 2d weir | 1997 Survey |
| Moolabin | S7 | Railway Bridges | 3 / various sized bridges | 1d bridge / 1d weir | Design drawings (part) |
| Moolabin | S8 | Chale Street | 3 / 3000 x 2150 mm RCBC | 1d culvert / 1d weir | 1997 Survey |
| Moolabin | S9 | Fairfield Road | 3 / 3350 x 1000 mm RCBC<br>1 / 3700 x 3250 mm RCBC | 1d culvert / 2d weir | 1997 Survey |
| Moolabin | S10 | Curzon Street | Three span bridge | 2d layered flow constriction | 1997 Survey plus design drawings |
| Rocky Water Holes | S11 | Ainsworth Street | Single span footbridge | 1d bridge / 1d weir | 1997 Survey plus design drawings |
| Rocky Water Holes | S12 | McCarthy Road | 3 / 1650 mm RCP<br>2 / 3000 x 1800 mm RCBC | 1d culvert / 2d weir | 1997 Survey |
| Rocky Water Holes | S13 | Assembly Street | Two span footbridge | 1d bridge / 1d weir | 1997 Survey plus design drawings |
| Rocky Water Holes | S14 | TAFE_Railway | Six span rail bridge | 1d bridge / 1d weir | 1997 Survey |
| Rocky Water Holes | S15 | TAFE_Footbridge | Single span footbridge | 1d bridge / 1d weir | Onsite measurements |

| Creek | Structure ID | Structure location | Structure details | Modelled structure representation | Origin of data used for coding the structure |
|---|---|---|---|---|---|
| Rocky Water Holes | S16 | Beaudesert Road | 6 / 2140 x 2170 mm RCBC | 1d culvert / 2d weir | 1997 Survey |
| Rocky Water Holes | S17 | Gladstone Street | 6 / 2130 x 2180 mm RCBC | 1d culvert / 2d weir | 1997 Survey |
| Rocky Water Holes | S18 | Ipswich Road | 4 / 3150 mm Armco | 1d culvert / 2d weir | 1997 Survey |
| Rocky Water Holes | S19 | Muriel Avenue | 7 / 2130 x 2250 mm RCBC | 1d culvert / 2d weir | 1997 Survey |
| Rocky Water Holes | S20 | Railway Bridges | 3 / various sized bridges | 1d bridge +<br>2d layered flow constriction | Design drawings (part) |
| Rocky Water Holes | S21 | Muriel Ave. Footbridge | Single span footbridge | 1d bridge / 1d weir | 1997 Survey |
| Rocky Water Holes | S22 | Fairfield Road | Two span footbridge | 1d bridge / 2d weir | 1997 Survey plus design drawings |
| Rocky Water Holes | S23 | Brisbane Golf Course | 3 / 900 mm RCP | 1d culvert / 2d weir | 1997 Survey |

Muriel Avenue Structures

The configuration of hydraulic structures in the vicinity of Muriel Avenue on Rocky Water Holes Creek is very complex. Within a 200 m length of creek (concrete-lined channel) there are seven waterway crossings, which include the following as listed in order from upstream to downstream:

- Ipswich Road Culverts
- Muriel Avenue Culverts
- 3 / Rail Bridges
- Muriel Avenue Footbridge
- Fairfield Road Bridge

The rail bridges are high level, considerably larger than the other four structures and span both the channel and overbank areas. For this reason, it was initially considered that the most suitable modelling approach for these large structures would be to represent them in TUFLOW as fully 2d using the “2d layered flow constriction” approach. This approach was initially trialled by modelling the section of creek between Ipswich Road and Fairfield Road as fully 2d. However, this approach encountered significant model instabilities, which were not able to be resolved satisfactorily.

The second approach trialled was to model the concrete channel from Ipswich Road to Fairfield Road as 1d and the overbanks / road embankments as 2d. This approach provided a far more stable solution, however it meant that the large rail bridges spanned both the 1d domain and also the 2d domain. This required that the rail bridges were modelled in the both the 1d and 2d domains. This representation is not ideal, however was adopted because of the instability issues with the previous method.

The head-loss results when compared to the gauged records appear to be reasonable and are discussed further in Section 5.7.

Brisbane Golf Course Structures

There are numerous fairway bridges spanning the creek within the Brisbane Golf Course. Because of the flooding characteristics of the area (as discussed previously in Section 5.3.2), these bridges were not represented in the hydraulic model.

### 5.3.5 Boundary Conditions

Inflow Boundaries

Inflows to the hydraulic model were taken from the XP-RAFTS hydrologic model. All inflows were represented as a discharge v time (Q-T) relationship, with the inflow locations as indicated in Figure 5.1. The inflow locations were generally adopted to match the XP-RAFTS model sub-catchment schematisation.

Downstream Boundary

A varying water level versus time (H-T) downstream boundary was used to represent the downstream boundary conditions at the mouth of Moolabin Creek. As noted previously in Section 3.2.4, this information was derived from the Oxley Creek stream gauges, located upstream and downstream of the mouth of the creek.

Table 5.3 provides a comparison of the derived flood level in Oxley Creek (at the mouth of Moolabin Creek) for each event with respect to the recorded upstream and downstream gauge levels. The table indicates that for the January 2013 event there was a considerably higher flood level in Oxley Creek at the mouth of Moolabin Creek (at the time of the peak flow) compared with the other events.

Table 5.3 – Comparison of Downstream Boundary Conditions

| Event | Peak Flood Level in Oxley Creek (mAHD) | | | | | |
|---|---|---|---|---|---|---|
| | Time of Peak Flow at Mouth of Moolabin Creek | | | Within Simulation Period | | |
| | Moolabin Mouth (derived) | 540071 (OXA023) | 540274 (OXA588) | Moolabin Mouth (derived) | 540071 (OXA023) | 540274 (OXA588) |
| January 2013 | 1.94 | 2.58 | 1.60 | 3.52 | 4.01 | 3.47 |
| February 2010 | 0.98 | 1.59 | 0.67 | 1.00 | 1.80 | 0.95 |
| November 2004 | 0.90 | 2.63 | -0.01 | 1.54 | 2.80 | 0.91 |
| March 2001 | 0.74 | 1.93 | 0.12 | 2.59 | 1.69 | 1.24 |

<u>1d-2d Boundaries</u>

On Moolabin Creek, the 1d-2d linked model was joined to the fully 2d model at the downstream side of Fairfield Road using an “SX” type flow boundary condition. On Rocky Water Holes Creek, the 1d-2d linked model was joined to the fully 2d model at the upstream extent of the Brisbane Golf Course using an “SX” type flow boundary condition.

Within the 1d-2d linked sections of the model, the 1d channel was linked to the 2d domain using the “HX” type boundary condition

### 5.3.6 Run Parameters

<u>Time Step</u>

The 1d ESTRY component was run using a 1 second time step and 2d TUFLOW component using a 1 second time step.

<u>Eddy Viscosity</u>

The Smagorinsky method was used for specifying the eddy viscosity in the 2d domain. This method is recommended in the TUFLOW manual and the default approach, in lieu of the Constant method. This method uses the Smagorinsky formula with a “Constant Coefficient” of 0.1 and “Smagorinsky Coefficient” of 0.2.

## 5.4 Calibration Procedure

### 5.4.1 Tolerances

BCC flood studies aim to achieve the following tolerances with regard to the hydraulic model calibration / verification:

- Continuous recording stream gauges - within ± 0.15 m of the peak flood level.
- MHGs - within ± 0.30 m of the peak flood level.
- Debris marks - within ± 0.40 m of the peak flood level.
- Good replication of the timing of peaks and troughs.

### 5.4.2 Methodology

The methodology applied to the calibration and verification of the TUFLOW model was as follows:

1) Run a large slowing increasing flow through the TUFLOW model to enable hydraulic structure head-loss checks to be undertaken against the HEC-RAS model(s).
2) Iteratively adjust the bridge loss parameters (as required) and re-run the model to establish a reasonable correlation with the HEC-RAS model(s).
3) Using the flow inputs from the XP-RAFTS model, run the calibration events through the TUFLOW model and compare the simulated results against the observed flood levels at both the stream gauge and the MHGs.
4) Iteratively adjust the model parameters and re-run the model with the aim of achieving a good fit with the observed data. The predominant model parameters adjusted included Manning's 'n' and the hydraulic structure losses.
5) Adopt model parameters based on the calibration results.
6) Using the flow inputs from the XP-RAFTS model, run the single verification event through the calibrated TUFLOW model and compare the simulated results against the observed flood levels at the MHGs.

As the creek conditions for all historical events are generally similar, the exact same model schematisation and parameters have been used for all four historical events. The only difference between the hydraulic modelling of the historical events is with the hydrologic flow inputs and the downstream boundary conditions at Oxley Creek. This methodology ensures that the TUFLOW model is sufficiently robust to be utilised for the design and extreme event modelling.

## 5.5 Hydraulic Model Calibration Results

### 5.5.1 January 2013

Figure 5.2 provides a comparison between the TUFLOW results and the gauged flood level at the stream gauge for the January 2013 event. The flood was simulated in TUFLOW for 3 days from midnight on the 27$^{th}$ January 2013. This simulation time comprised of a considerable period after the rainfall had effectively finished in order to observe the backwater effects due to Oxley Creek / Brisbane River.

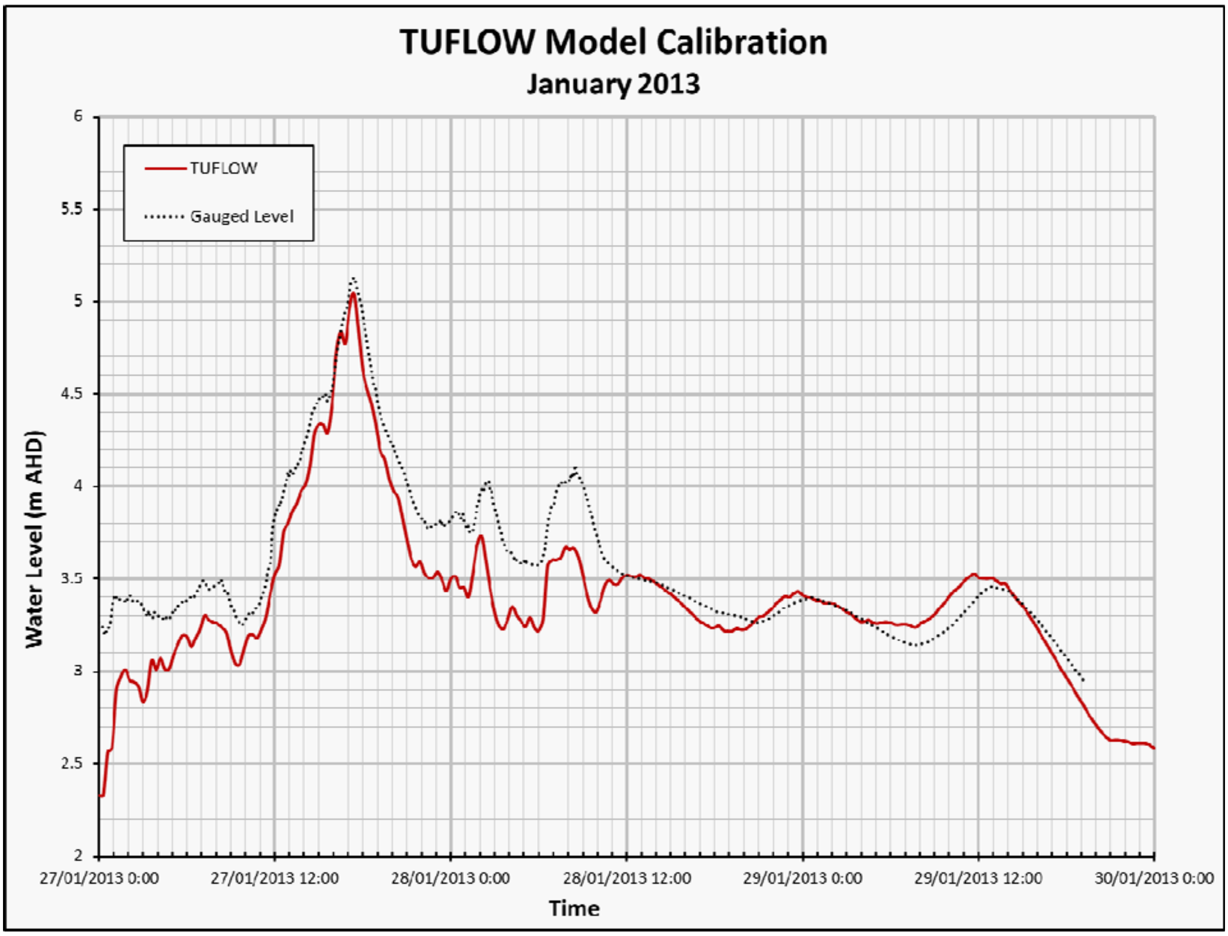



Figure 5.2: TUFLOW Model Calibration (January 2013)

The results indicate a good fit to the peak flood level, shape and timing of the hydrograph. However, the simulated flood level at lower flows is generally of the order of 0.1 to 0.3 m lower than the observed flood level within that part of the hydrograph where the flood level is a result of stream flow in lieu of backwater from Oxley Creek / Brisbane River (i.e. preceding 12 noon on the 28th January). It is conceivable that in lower flows (less than 20 $m^3/s$) that the model is slightly under predicting flood levels across the complex Muriel Avenue hydraulic structure arrangement.

Table 5.4 provides a comparison between the TUFLOW results and the recorded peak flood levels at the stream gauge and MHGs which were working during the event. The results indicate that the simulated peak flood levels were within the specified tolerance at all the MHGs, apart from R140. Further discussion on R140 is provided in Section 5.9. At the stream gauge, the difference of -0.04 m was also within the ideal ± 0.15 m tolerance. The simulated peak flood level at all MHGs is consistently lower than the observed. This could be explained by the presence of base flow associated with the prolonged wet conditions and / or insufficient recorded rainfall volume. The average difference between the simulated and recorded peak levels is -0.16 m. It is conceivable that insufficient rainfall was recorded as there were extremely strong winds during the peak rainfall burst, which may have affected the rainfall volume recorded by the rain gauges. The results of a sensitivity run (where the rainfall was increased by 10 %) revealed an average difference between the simulated and recorded peak levels is -0.08 m.

This event recorded a peak flood level of 4.14 mAHD at ML100 on the downstream side of the Curzon Street Bridge. This peak flood level is the same as that recorded for the March 2001 event (refer to Table 5.6), yet the size of the rainfall event was significantly smaller. From review of the stream gauge records for Oxley Creek, it does not appear that this MHG reading is a result of peak flood levels (within Oxley Creek) which occurred on the 28th January, around a day after the local catchment peak. The recorded peak at stream gauge 540071 (OXA023) at Corinda High was 4.01 mAHD on the 28th January. The flood level in Oxley Creek at the mouth of Moolabin Creek would have been approximately 1.94 mAHD at the time of the peak at Curzon Street, which would not influence the peak flood level at Curzon Street. The observed MHG reading at ML100 would appear to be accurate because the flood levels recorded at the upstream MHGs including R100, R110 and the stream gauge also appear to be downstream controlled.

The hydraulic modelling has not been able to replicate flood levels in this vicinity as it is likely that the downstream controlled flood levels are due to some form of constriction to the creek channel / floodplain during the event (e.g. blockage). It is noted that some development of the Toll Holdings property (immediately downstream of Curzon Street) occurred in 2012. Review of the approved development drawings did not reveal any potential constrictions / blockages.

Table 5.4 – Calibration to Peak Flood Level Data (January 2013)

| Creek | MHG ID | Location | Recorded Peak WL (m AHD) | Simulated Peak WL (m AHD) | Difference (m) |
|---|---|---|---|---|---|
| Moolabin | ML100 | d/s of Curzon Street Bridge | 4.14 | 3.87 | -0.27 |
| | ML002 | Brisbane Golf Course | - | - | - |
| | ML110 | d/s Fairfield Road Culvert | - | - | - |
| | ML120 | u/s Evesham Street Culvert | - | - | - |
| | ML130 | 75 m d/s Lucy Street Culvert | 8.63 | 8.37 | -0.26 |
| | ML005 | u/s Gow Street Culvert | - | - | - |
| | ML140 | u/s Gow Street Culvert | 11.38 | 11.25 | -0.13 |
| Rocky Water Holes | R100 | Brisbane Golf Course | 4.34 | 4.26 | -0.08 |
| | R110 | d/s Fairfield Road Bridge | 4.79 | 4.64 | -0.15 |
| | Stream Gauge | u/s Ipswich Road Culvert | 5.11 | 5.07 | -0.04 |
| | R120 | 65 m u/s Ipswich Road Culvert | - | - | - |
| | R130 | d/s Gladstone Street Culvert | 7.68 | 7.50 | -0.18 |
| | R140 | u/s Beaudesert Road Culvert | 8.25 | 7.78 | -0.47 |
| | R150 | d/s Assembly Street Bridge | - | - | - |
| | R160 | u/s McCarthy Road Culvert | - | - | - |

### 5.5.2 February 2010

Figure 5.3 provides a comparison between the TUFLOW results and the gauged flood level at the stream gauge for the February 2010 event. Table 5.5 provides a comparison between the TUFLOW results and the recorded peak flood levels at the stream gauge and MHGs which were working during the event.

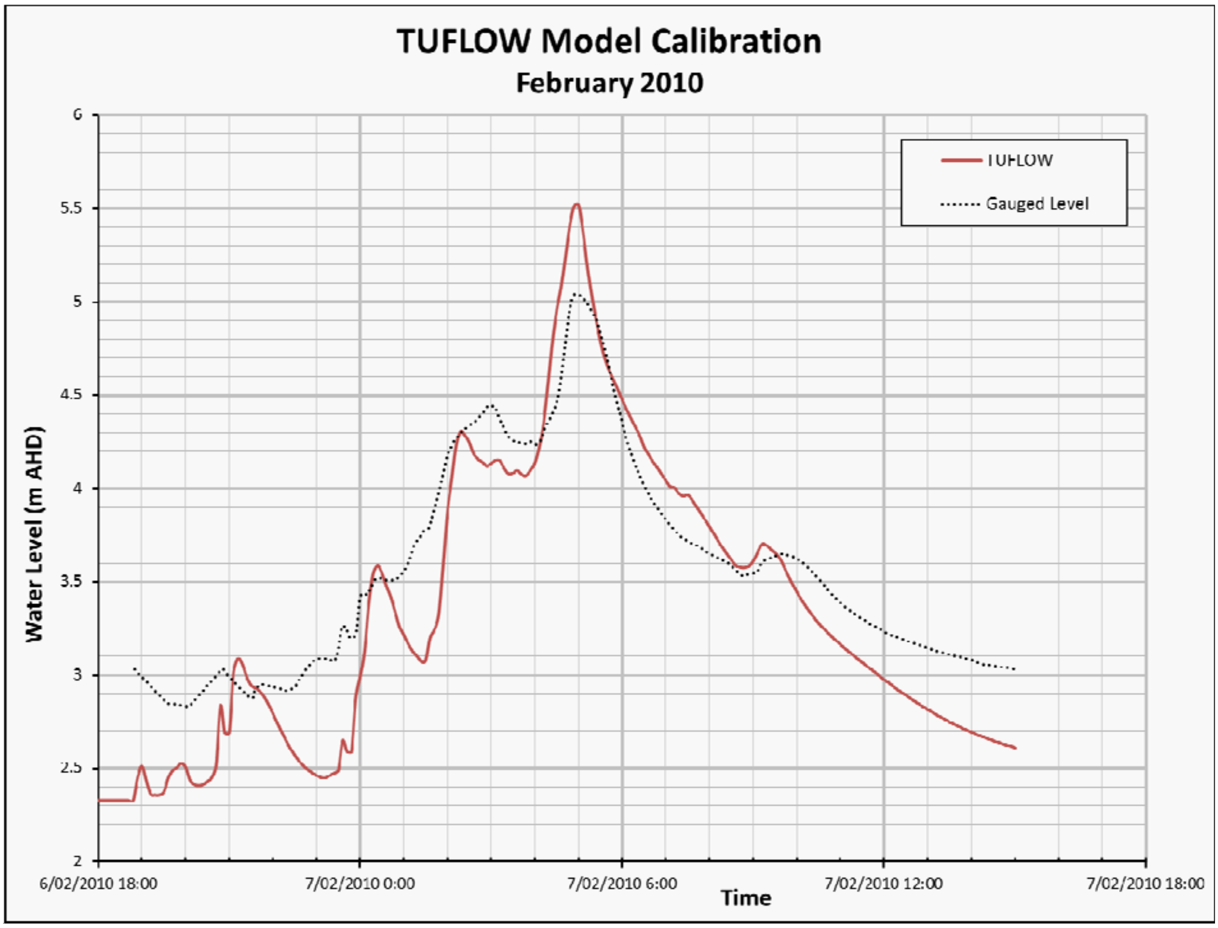


Figure 5.3: TUFLOW Model Calibration (February 2010)

The simulated results at the stream gauge indicate a reasonable replication of the shape and timing of the observed hydrograph; however the peak flood level has been considerably overestimated. From review of the MHG results, there is a general overestimation of peak flood levels along the entire length of Rocky Water Holes Creek, apart from at R140. The average difference between the simulated and recorded peak levels is +0.21 m. This overestimation is more noticeable in the lower reach of Rocky Water Holes Creek between the stream gauge and R100. This is opposed to the January 2013 event, where the simulated peak flood levels were low throughout this lower reach.

The simulated peak flood levels are outside the ideal calibration tolerances at both R100 and the stream gauge. The simulated flood volume is approximately 10 % lower than the observed, around the main burst of the event; therefore it was not considered justified to further increase pervious area rainfall losses to reduce the peak flow. The most likely reason for the consistently high flows / flood levels is that the spatial variation in rainfall across the catchment has not been fully captured and represented within the hydrology model. This has resulted in greater rainfall being applied than would have most likely fallen.

Table 5.5 – Calibration to Peak Flood Level Data (February 2010)

| Creek | MHG ID | Location | Recorded Peak WL (m AHD) | Simulated Peak WL (m AHD) | Difference (m) |
|---|---|---|---|---|---|
| Moolabin | ML100 | d/s of Curzon Street Bridge | - | - | - |
| | ML002 | Brisbane Golf Course | - | - | - |
| | ML110 | d/s Fairfield Road Culvert | - | - | - |
| | ML120 | u/s Evesham Street Culvert | - | - | - |
| | ML130 | 75 m d/s Lucy Street Culvert | - | - | - |
| | ML005 | u/s Gow Street Culvert | - | - | - |
| | ML140 | u/s Gow Street Culvert | - | - | - |
| Rocky Water Holes | R100 | Brisbane Golf Course | 3.87 | 4.29 | 0.42 |
| | R110 | d/s Fairfield Road Bridge | 4.63 | 4.85 | 0.22 |
| | Stream Gauge | u/s Ipswich Road Culvert | 5.04 | 5.52 | 0.48 |
| | R120 | 65 m u/s Ipswich Road Culvert | - | - | - |
| | R130 | d/s Gladstone Street Culvert | 7.67 | 7.73 | 0.06 |
| | R140 | u/s Beaudesert Road Culvert | 8.25 | 8.16 | -0.09 |
| | R150 | d/s Assembly Street Bridge | - | - | - |
| | R160 | u/s McCarthy Road Culvert | 17.13 | 17.31 | 0.18 |

### 5.5.3 March 2001

Table 5.6 provides a comparison of the TUFLOW results and the recorded peak flood levels at the MHGs which were working during the March 2001 event. Many of the observed MHG readings were from debris marks as most of the gauges were overtopped due to the large magnitude of the event.

The simulated peak flood levels within both Moolabin Creek and Rocky Water Holes Creek show a very good correlation with the observed results, with 9 out of 11 gauges falling within the ideal tolerance. There appears to an error with the debris reading at R140, therefore this result has been disregarded (refer to Section 5.9 for further discussion). The average difference between the simulated and recorded peak levels is -0.05 m.

The flood levels upstream and downstream of the very complex Muriel Avenue area indicate an extremely good correlation, with a peak head-loss of 1.3 m being both modelled and observed.

As this event is of similar magnitude to a 100-yr ARI (1 % AEP) flood, the results give confidence that the model is accurately replicating large flooding events.

Table 5.6 – Calibration to Peak Flood Level Data (March 2001)

| Creek | MHG ID | Location | Recorded Peak WL (m AHD) | Simulated Peak WL (m AHD) | Difference (m) |
|---|---|---|---|---|---|
| Moolabin | ML100 | d/s of Curzon Street Bridge | 4.14 | 4.38 | 0.24 |
| | ML002 | Brisbane Golf Course | 4.9 (debris) | 5.02 | 0.12 |
| | ML110 | d/s Fairfield Road Culvert | 6.93 (debris) | 6.45 | -0.48 |
| | ML120 | u/s Evesham Street Culvert | 7.51 (debris) | 7.40 | -0.11 |
| | ML130 | 75 m d/s Lucy Street Culvert | - | - | - |
| | ML005 | u/s Gow Street Culvert | 12.91 | 12.97 | 0.06 |
| | ML140 | u/s Gow Street Culvert | - | - | - |
| Rocky Water Holes | R100 | Brisbane Golf Course | 4.91 | 5.04 | 0.13 |
| | R110 | d/s Fairfield Road Bridge | 5.31 | 5.30 | -0.01 |
| | Stream Gauge | u/s Ipswich Road Culvert | - | - | - |
| | R120 | 65 m u/s Ipswich Road Culvert | 6.61 (debris) | 6.60 | -0.01 |
| | R130 | d/s Gladstone Street Culvert | 8.13 | 8.00 | -0.13 |
| | R140 | u/s Beaudesert Road Culvert | 9.96 (debris) | 8.83 | -1.13 |
| | R150 | d/s Assembly Street Bridge | 13.30 | 13.38 | 0.08 |
| | R160 | u/s McCarthy Road Culvert | 18.17 | 17.72 | -0.45 |

## 5.6 Hydraulic Model Verification Results

### 5.6.1 November 2004

Table 5.7 provides a comparison between the TUFLOW results and the recorded peak flood levels at the MHGs which were working during the November 2004 event. The simulated peak flood levels show a good correlation with 7 out of 7 gauges falling within the ideal tolerance. There appears to an error with the debris reading at ML120, therefore this result has been disregarded.

Table 5.7 – Verification to Peak Flood Level Data (November 2004)

| Creek | MHG ID | Location | Recorded Peak WL (m AHD) | Simulated Peak WL (m AHD) | Difference (m) |
|---|---|---|---|---|---|
| Moolabin | ML100 | d/s of Curzon Street Bridge | 3.76 | 3.97 | 0.21 |
| | ML002 | Brisbane Golf Course | - | - | - |
| | ML110 | d/s Fairfield Road Culvert | - | - | - |
| | ML120 | u/s Evesham Street Culvert | 5.37 (debris) | 6.28 | 0.91 |
| | ML130 | 75 m d/s Lucy Street Culvert | - | - | - |
| | ML005 | u/s Gow Street Culvert | - | - | - |
| | ML140 | u/s Gow Street Culvert | - | - | - |
| Rocky Water Holes | R100 | Brisbane Golf Course | 4.21 | 4.38 | 0.17 |
| | R110 | d/s Fairfield Road Bridge | 4.67 | 4.87 | 0.20 |
| | Stream Gauge | u/s Ipswich Road Culvert | - | - | - |
| | R120 | 65 m u/s Ipswich Road Culvert | 5.60 | 5.65 | 0.05 |
| | R130 | d/s Gladstone Street Culvert | 7.62 | 7.71 | 0.09 |
| | R140 | u/s Beaudesert Road Culvert | - | - | - |
| | R150 | d/s Assembly Street Bridge | 12.84 (debris) | 13.16 | 0.32 |
| | R160 | u/s McCarthy Road Culvert | 17.23 | 17.23 | 0.00 |

From review of the MHG results, there is a general overestimation of peak flood levels, apart from at R160. The average difference between the simulated and recorded peak levels is +0.15 m. This value could be reduced by increasing the pervious area initial loss across the catchment. However, when compared to the other events, there doesn't appear sufficient justification to further increase this loss value, especially when considering calibration to volume was unable to be undertaken (as the continuous recording stream gauge was yet to be installed). Similar to the February 2010 event, the small over estimation appears to be a result of the spatial variation in rainfall across the catchment not being fully captured and represented within the hydrology model

## 5.7 Hydraulic Structure Verification

The TUFLOW manual recommends confirming the head-loss across hydraulic structures as follows:

*It is strongly recommended that the losses through a structure be validated through:*

- *Calibration to recorded information (if available).*
- *Cross-checked using desktop calculations based on theory and/or standard publications (e.g. Hydraulics of Bridge Waterways, US FHA 1973).*
- *Cross-checked with results using other hydraulic software.*

It is common practice in BCC flood studies to cross-check structure head-losses against results from the HEC-RAS hydraulic modelling software. Generally, HEC-RAS is regarded as one of the better hydraulic modelling packages when it comes to accurately representing hydraulic structures such as bridges. The majority of the hydraulic structures within the catchment(s) are culverts, of which the TUFLOW and HEC-RAS algorithms are similar. Therefore, it was considered more important to check the head-loss at a number of the bridge structures.

The bridge structures where HEC-RAS checks were undertaken included:

- Ainsworth Street Footbridge
- TAFE_Railway Bridge
- Fairfield Road Bridge (Rocky Water Holes Creek)
- Railway Bridges (Moolabin Creek)
- Curzon Street Bridge

As discussed in Section 5.3.4, the reach of Rocky Water Holes Creek in the vicinity of Muriel Avenue is very complex with seven waterway crossings within a 200 m length. Fortunately, there are gauges both upstream and downstream which allow the modelled head-losses to be checked against the recorded head-losses. Table 5.8 presents the results of the head-loss comparison, from which it is apparent that there is good correlation between the modelled and recorded results. The largest difference of 0.26 m (February 2010) can be attributed to differences in flow as discussed previously in Section 5.5.2.

Table 5.8 – Headloss Checks for Muriel Avenue Structures

| Event | TUFLOW Flow ($m^3/s$) | Recorded Head-loss (m) | TUFLOW Head-loss (m) | Difference (m) |
|---|---|---|---|---|
| Jan 2013 | 43.6 | 0.32 | 0.43 | 0.11 |
| Feb 2010 | 61.6 | 0.41 | 0.67 | 0.26 |
| Mar 2001 | 98 | 1.30 | 1.30 | 0.00 |
| Nov 2004 | 63.5 | 0.93 | 0.78 | -0.15 |

Table 5.9 provides a comparison of the head-loss across the structure between TUFLOW and the HEC-RAS model.

Table 5.9 – HEC-RAS Bridge Modelling Checks

| Flow ($m^3/s$) | HEC-RAS Head-loss (m) | TUFLOW Head-loss (m) | Difference (m) |
|---|---|---|---|
| **Structure S7 – Railway Bridges (Moolabin Creek)** | | | |
| 10 | 0.20 | 0.08 | -0.12 |
| 20 | 0.12 | 0.04 | -0.08 |
| 40 | 0.08 | 0.04 | -0.04 |
| 60 | 0.06 | 0.03 | -0.03 |
| 80 | 0.08 | 0.11 | 0.03 |
| 100 | 0.15 | 0.17 | 0.02 |
| **Structure S10 – Curzon Street Bridge** | | | |
| 10 | 0.00 | 0.03 | 0.03 |
| 20 | 0.01 | 0.01 | 0.00 |
| 40 | 0.03 | 0.04 | 0.01 |
| 60 | 0.07 | 0.08 | 0.01 |
| 80 | 0.13 | 0.2 | 0.07 |
| 120 | 0.28 | 0.35 | 0.07 |
| 160 | 0.46 | 0.61 | 0.15 |
| 220 | 0.70 | 0.85 | 0.15 |
| 300 | 0.59 | 0.75 | 0.16 |
| **Structure S11 – Ainsworth Street Footbridge** | | | |
| 10 | 0.01 | 0.01 | 0.00 |
| 20 | 0.01 | 0.01 | 0.00 |
| 40 | 0.01 | 0.01 | 0.00 |
| 60 | 0.12 | 0.39 | 0.27 (1) |
| 80 | 0.24 | 0.60 | 0.36 (1) |
| **Structure S14 – TAFE_Railway Bridge** | | | |
| 10 | 0.13 | 0.13 | 0.00 |
| 20 | 0.12 | 0.10 | -0.02 |
| 40 | 0.12 | 0.10 | -0.02 |
| 60 | 0.12 | 0.11 | -0.01 |
| 80 | 0.15 | 0.12 | -0.03 |
| 120 | 0.35 | 0.36 | 0.01 |
| **Structure S22 – Fairfield Road (Rocky Water Holes Creek)** | | | |
| 10 | 0.00 | 0.00 | 0.00 |
| 20 | 0.00 | 0.00 | 0.00 |
| 40 | 0.00 | 0.01 | 0.01 |
| 60 | 0.01 | 0.01 | 0.00 |
| 80 | 0.02 | 0.01 | -0.01 |
| 100 | 0.05 | 0.02 | -0.03 |
| 110 | 0.36 | 0.39 | 0.03 |

Note (1) - At higher flows, where there is complex flooding behaviour, a simple extended cross-section HEC-RAS model can struggle to accurately represent the flooding characteristics. This is particularly relevant where there is a difference in channel and floodplain water levels. At these locations, the results of the comparison should be disregarded.

Generally, the TUFLOW head-losses for the bridge structures checked were within ± 0.3 m of the HEC-RAS values for the full range of flows at which checks were undertaken. This is considered reasonable and gives credence to the TUFLOW results.

A HEC-RAS model was also developed for the Assembly Street Footbridge, however the flow characteristics are quite complex and it quickly became apparent that the HEC-RAS model was unable to model the complex flow behaviour. A comparative analysis for this structure was discontinued.

## 5.8 Hydrologic-Hydraulic Model Consistency Check (Historical Events)

### 5.8.1 General

Comparison checks on flow were undertaken between the XP-RAFTS and TUFLOW models for all four historical events in the middle and lower sections of the catchment, to understand how closely the hydrologic and hydraulic models were matching.

Figures 5.4 to 5.7 provide comparative plots of the XP-RAFTS and TUFLOW flow results for the historical events at the following three locations:

(i) Rocky Water Holes Creek at Muriel Avenue

(ii) Moolabin Creek at the Railway Bridges

(iii) Moolabin Creek at Curzon Street

Table 5.10 provides a comparison of the peak flows at these three locations.

Table 5.10 – Peak Flow Comparison, XP-RAFTS and TUFLOW

| Event | Peak Flow ($m^3/s$) | | | | | |
|---|---|---|---|---|---|---|
| | Muriel Avenue | | Railway (Moolabin) | | Curzon Street | |
| | XP-RAFTS | TUFLOW | XP-RAFTS | TUFLOW | XP-RAFTS | TUFLOW |
| Jan 2013 | 43.1 | 43.6 | 31.8 | 34.5 | 63.3 | 64.3 |
| Feb 2010 | 61 | 61.6 | 48.3 | 51.9 | 64.7 | 67.2 |
| Mar 2001 | 100.1 | 98 | 97.8 | 82.4 | 121.8 | 117.2 |
| Nov 2004 | 59.2 | 63.5 | 47 | 51 | 70.2 | 71.9 |

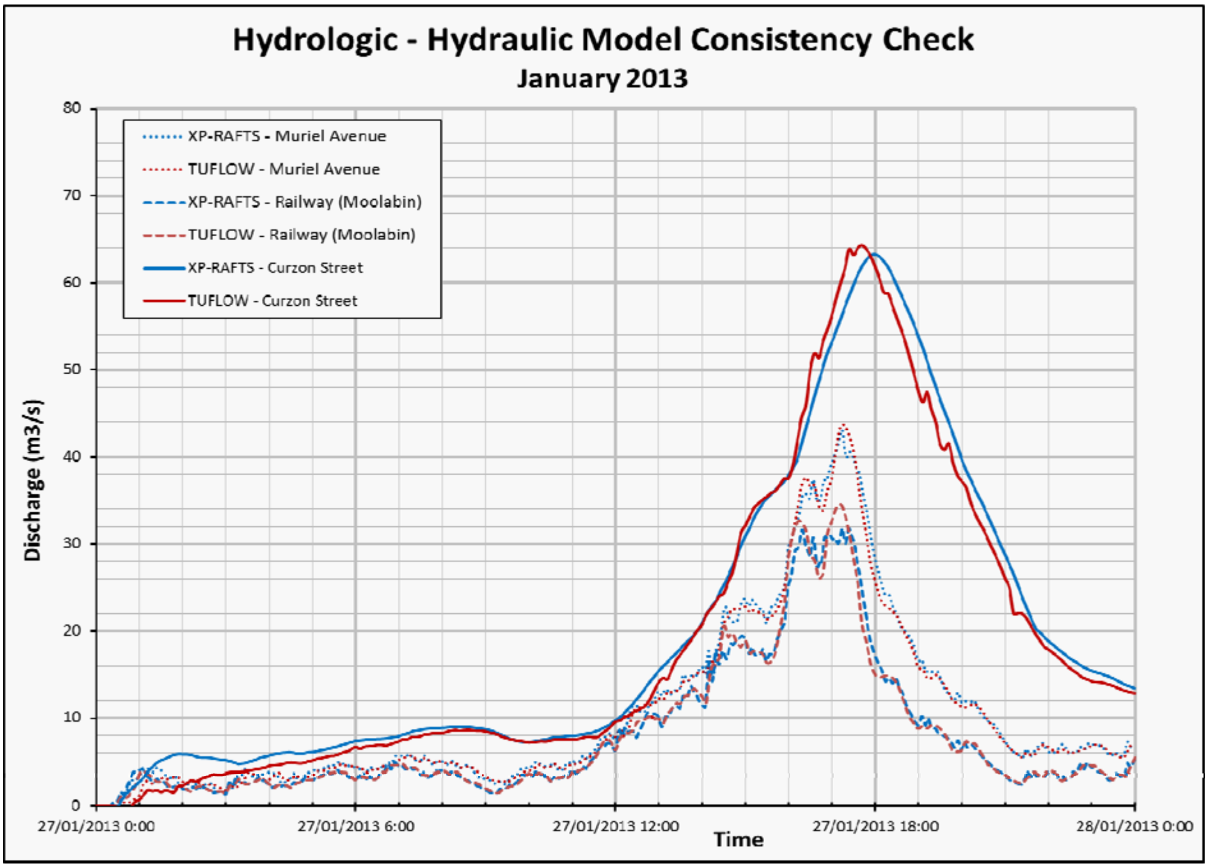



Figure 5.4: Model Consistency Check (January 2013)

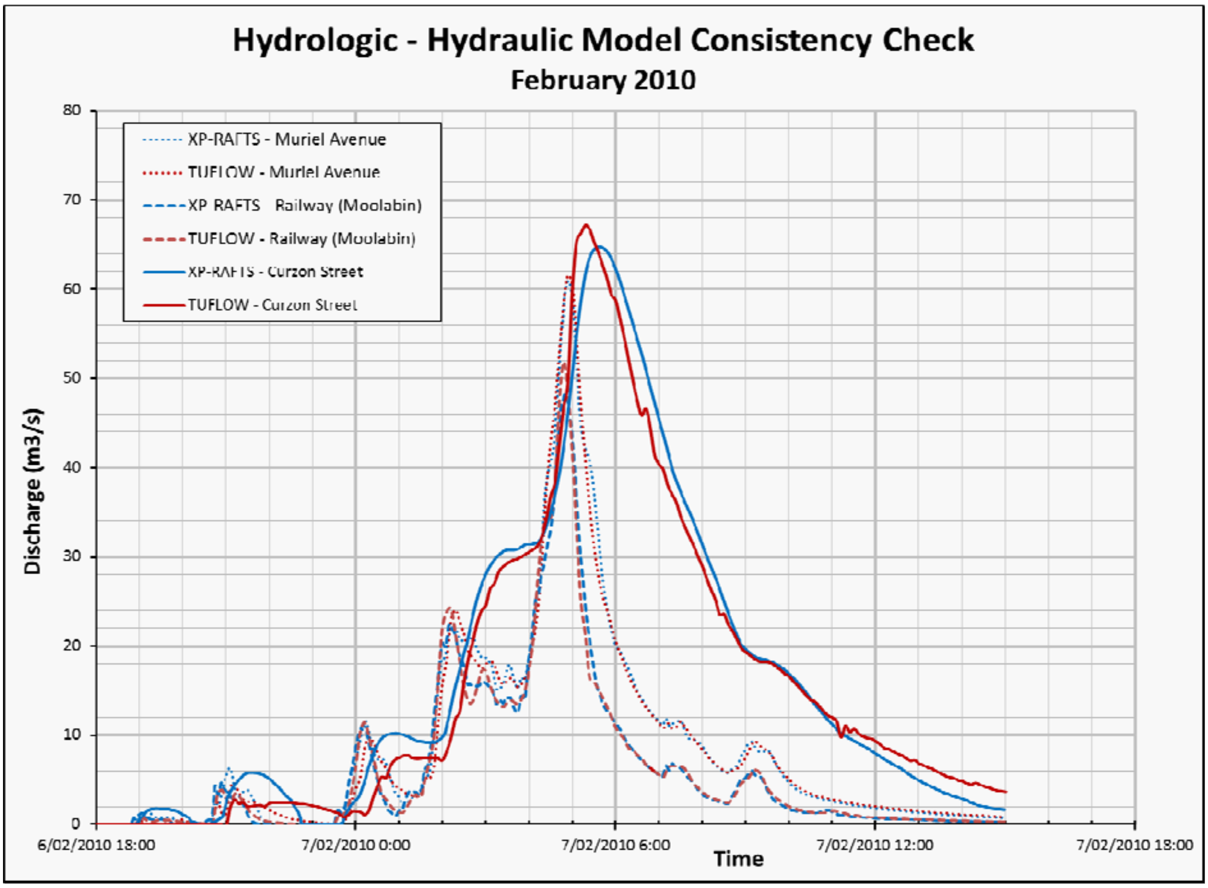



Figure 5.5: Model Consistency Check (February 2010)

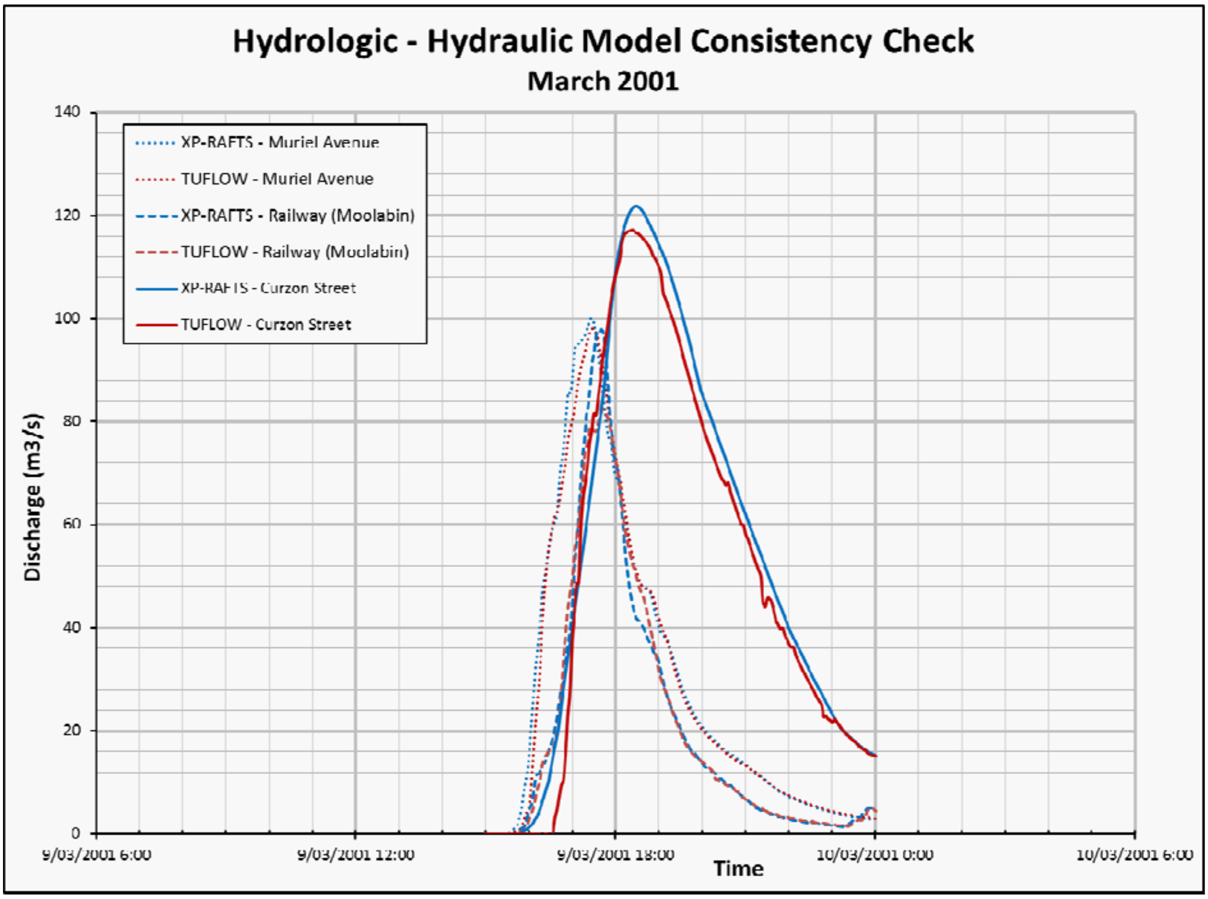



Figure 5.6: Model Consistency Check (March 2001)

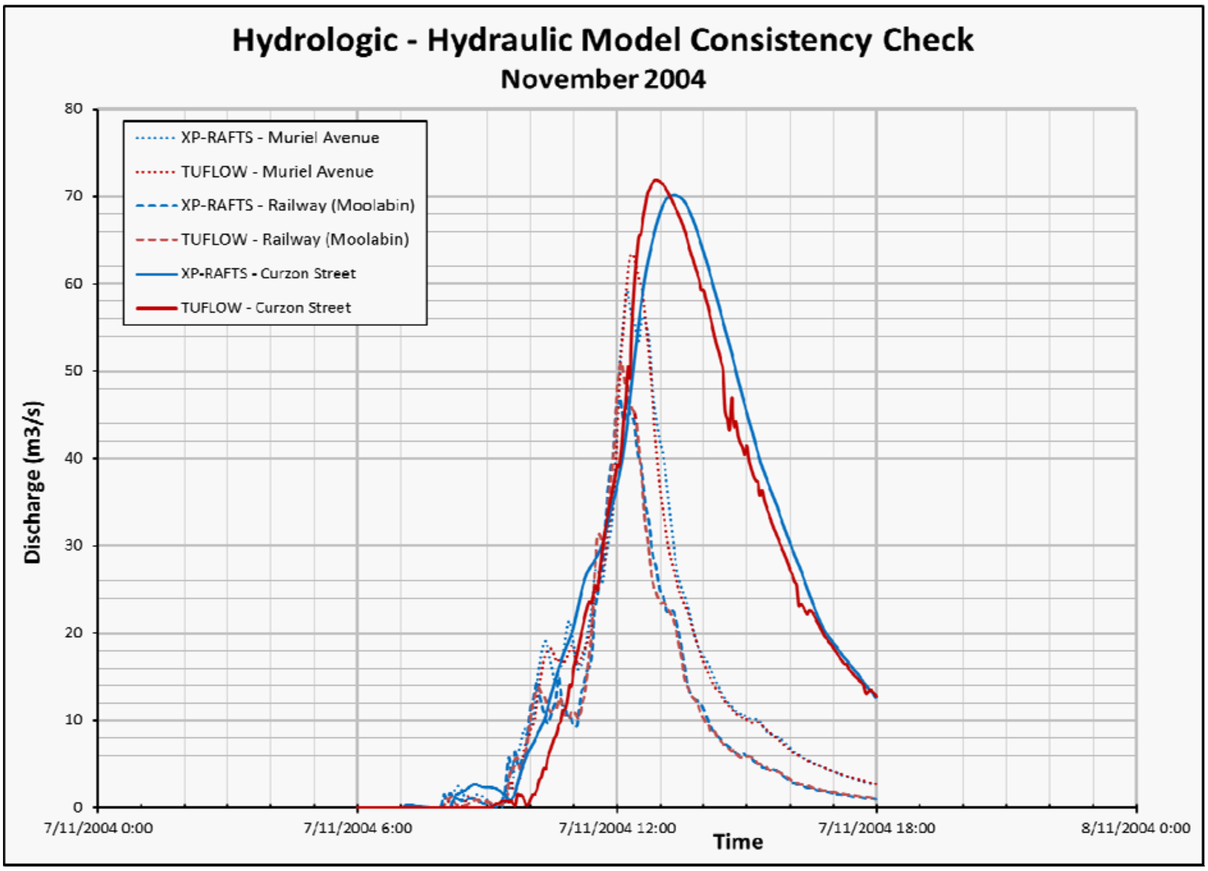



Figure 5.7: Model Consistency Check (November 2004)

The results of the comparison indicate that the XP-RAFTS and TUFLOW models generally match very well on both Moolabin Creek and Rocky Water Holes Creek. This is even apparent at Curzon Street where there are significant storage effects because of the Brisbane Golf Course.

Initially, the golf course area was represented in XP-RAFTS by the Muskingum-Cunge reach routing approach. However, the storage effects were unable to be accurately replicated using this methodology and the detention storage approach was utilised. This methodology involved creating a detention storage area between Fairfield Road and Curzon Street to better represent the golf course storage effects. This required that a stage-discharge relationship was extracted from the TUFLOW model at Curzon Street, as well as a stage-storage relationship from the 2009 ALS data.

Based on the good correlation between XP-RAFTS and TUFLOW, it is considered that the XP-RAFTS model would be suitable for use as a 'standalone' model on the basis that there are not backwater effects from Oxley Creek and / or the Brisbane River. If there are backwater effects, then the hydraulic model would be the only model suitable of generating accurate flows / flood levels.

## 5.9 Discussion on Calibration and Verification

The results of the calibration and verification of the four historical events are quite reasonable and can be summarised as follows:

- January 2013 – good shape and timing of the flood hydrograph at the stream gauge, with the peak flood level within the ideal tolerance. All peak flood levels at MHGs within the ideal tolerance. A slight under prediction of flood levels, which is consistent throughout the catchment.
- February 2010 – good shape and timing of the flood hydrograph at the stream gauge, with the peak flood level outside the ideal tolerance. Peak flood levels at 4 out of 5 MHGs within the ideal tolerance. A general over prediction of flood levels, which is consistent throughout the catchment.
- March 2001 – peak flood levels at 9 out of 11 MHGs within the ideal tolerance. Generally, a very good fit to an event of around 100-yr ARI (1 % AEP).
- November 2004 – all peak flood levels at MHGs within the ideal tolerance. Generally, a slight over prediction of flood levels, which is consistent throughout the catchment.
- Good correlation of bridge head-losses between the TUFLOW model and HEC-RAS.
- Good correlation between the XP-RAFTS and TUFLOW models for all historical events

From the calibration and verification results, it is apparent that these two small, steep and highly urbanised catchments are particularly sensitive to variations in the rainfall distribution. Modelled peak flood levels for two of the more spatial events (February 2010 and November 2004) could be considered slightly high, whereas modelled peak flood levels for January 2013 could be considered slightly low, although no rainfall losses have been applied to the latter.

Observed results from the January 2013 event indicate downstream controlled flood levels from Ipswich Road to downstream of Curzon Street. This phenomenon is not apparent in the modelled results or the observed results for any of the other historical events. It would appear that there may have been some form of constriction in the channel / floodplain downstream of Curzon Street during the January 2013 event.

Given that the results of the calibration and verification are quite reasonable and that the events ranged in magnitude from small (~1-yr to 2-yr ARI) to large (~100-yr ARI), there is confidence that the flood models would be suitable for producing accurate flood levels for the full range of design event modelling.

The observed peak flood level results at MHG R140 are considered questionable. R140 is bolted onto the wing-wall at the upstream of the Beaudesert Road Culvert. From review onsite, it is considered highly likely that the MHG readings would be subject to localised effects such as turbulence, which would not be able to be re-produced by the hydraulic model. This is supported by the modelling results which consistently underestimate flood levels at this location.

# 6.0 Design Event Analysis

## 6.1 Design Event Terminology

The use of the terms "*recurrence interval*" and "*return period*" has been criticised as leading to confusion in the minds of some decision-makers and members of the public. Therefore, the current update of AR&R will utilise different terminology.

Generally, for the larger flood magnitude discharges, the term AEP (%) is now preferred by AR&R, in lieu of ARI.

Table 6.1 indicates the equivalent AEP value (rounded to a whole number) with respect to ARI. The relationship can be expressed by the following equation:

$AEP = 1 – exp (-1 / ARI)$

Table 6.1 – ARI versus AEP

| ARI (year) | AEP (%) |
|---|---|
| 2 | 39 |
| 5 | 18 |
| 10 | 10 |
| 20 | 5 |
| 50 | 2 |
| 100 | 1 |

It is common to see the 50 % AEP being equated to the 2-yr ARI and also the 20 % AEP being equated to the 5-yr ARI. This is not technically correct; however the use of $AEP = 1 / ARI$ is very prevalent within the industry and often used for simplicity.

For the purpose of this technical report, the correct values indicated in Table 6.1 will be utilised. The flood probability will be firstly expressed firstly in ARI and then secondly in the equivalent AEP, for example 2-yr ARI (39 % AEP).

However, as the mapping products in Volume 2 will likely be viewed by a wider audience, for ease of common understanding the simplified $AEP = 1 / ARI$ will be utilised. The 2-yr ARI and 5-yr ARI will be referred to as 50 % AEP and 20 % AEP respectively.

## 6.2 Design Event Scenarios

Table 6.2 indicates the three scenarios utilised in the modelling of the design events, noting that all design event scenarios were modelled using ultimate hydrological conditions.

For the purpose of this report, the term “design events” refers to those events from 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP).

Table 6.2 – Design Event Scenarios

| ARI (year) | AEP (%) | Scenario 1 | Scenario 2 | Scenario 3 |
|---|---|---|---|---|
| 2 | 39 | ✓ | ✗ | ✓ |
| 5 | 18 | ✓ | ✗ | ✓ |
| 10 | 10 | ✓ | ✗ | ✓ |
| 20 | 5 | ✓ | ✗ | ✓ |
| 50 | 2 | ✓ | ✗ | ✓ |
| 100 | 1 | ✓ | ✓ | ✓ |

The following describes the design event scenarios:

***Scenario 1: Existing Waterway Conditions***
Scenario 1 is based on the current waterway conditions. Some minor modifications were made to the TUFLOW model developed as part of the calibration / verification; refer to Section 6.4 for further details.

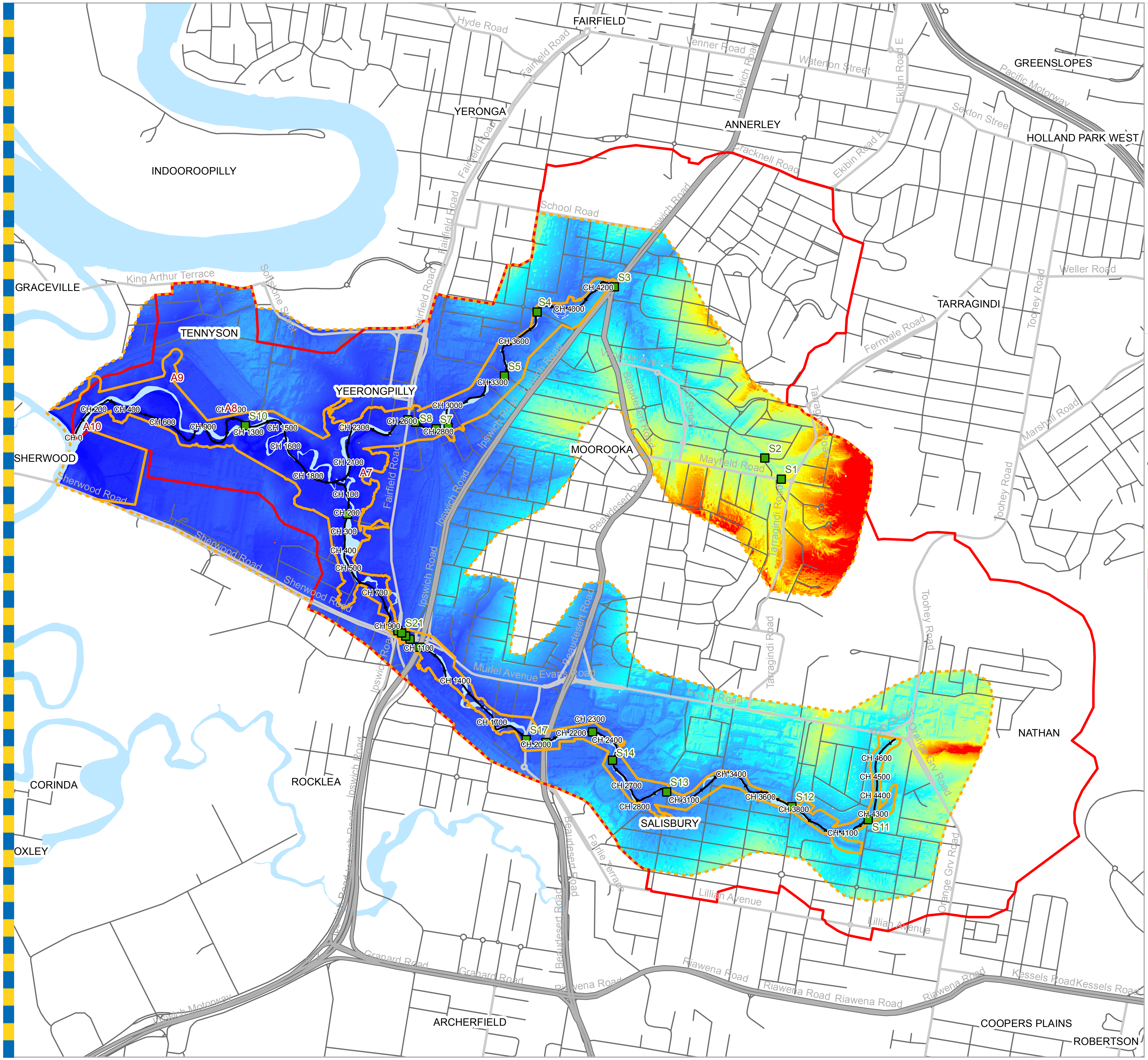
***Scenario 2: Minimum Riparian Corridor (MRC)***
Scenario 2 includes an allowance for a riparian corridor along the edge of the channel. This involved firstly reviewing the existing vegetation and land-use adjacent to the channel to determine an appropriate Manning's 'n' roughness value for the riparian corridor. In most locations the default value of n = 0.15 was used, however where the existing manning's 'n' is higher than n = 0.15, the manning's 'n' was left unchanged.

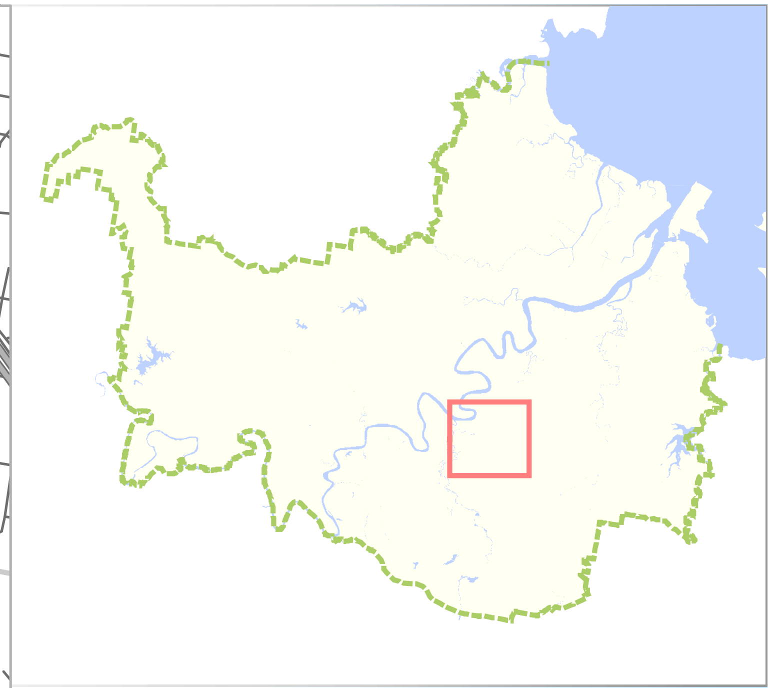
A 30 m wide corridor (15m wide each side from the low flow channel) was defined by changing the Manning's n of the 1d cross sections (as applicable) and a new 2d materials layer within the TUFLOW model. In areas where the 15 m width was not available, the MRC was set to the maximum possible width (i.e. less than 15 m) up to the boundary of the conveyance corridor.

***Scenario 3: Filling to the Conveyance Corridor + Minimum Riparian Corridor (MRC)***
Figure 6.1 indicates the conveyance corridor for the both creeks. The conveyance corridor is the greater extent of Flood Planning Area 3 (FPA3) and the waterway corridor. FPA3 is the greater extent of 0.6 - 1.2 m depth in the 100-yr ARI (1 % AEP) event and 0.6 $m^2/s$ < VxD < 1.2 $m^2/s$ in the 100-yr ARI (1 % AEP) event.

Scenario 3 assumes filling to the conveyance corridor boundary to represent potential development. In the design events, 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP), the filling acts as a barrier and the conveyance corridor can be modelled simplistically as a glass-wall of infinite height.

This is a simple and conservative assumption used to develop design planning levels. It does not necessarily reflect allowable development assumptions under City Plan.



**Legend**

- Structures
- AMTD Line
- Conveyance Corridor
- Freeways/Highways
- Major Roads
- Streets
- Model Boundary
- Catchment Area

N E S W

0 220 440 660
Metres

Prepared : 089958
Checked : NC
Revision : 0
Publication Date : 02 Jun 2015
Project Number : 150339

Location: G:\BI\CD\Proj15\150339_Moolabin_Rocky_Waterholes_Ck\ArcGIS\GIM_150339_018_A3L Figure6_1.mxd

**DATA INFORMATION**

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**Moolabin Creek and Rocky Water Holes Creek Flood Study**

# Figure 6.1: TUFLOW Model Conveyance Corridor

GIM - 150339 - 018

## 6.3 Design Event Hydrology

### 6.3.1 Overview

Design flood estimation is generally best determined by undertaking a flood frequency analysis of annual maximum and / or peak over threshold (POT) series from observed long-term stream flow records. In the Brisbane City Council region, however, the period of record is typically insufficient to enable sufficient confidence to warrant undertaking flood frequency methods. Table 6.3 [4] indicates some guidance for length of record versus expected error rate for flood frequency analysis.

On the basis that the one continuous recording stream gauge on Rocky Water Holes Creek (540433 - R_A849) has only approximately 9 years of records, it has been deemed unsuitable to undertake flood frequency analysis for this study.

Table 6.3 – Guidance for Length of Record versus Expected Error Rate

| ARI (year) | Required Length of Record (years) | |
|---|---|---|
| | ± 10 % Error Level | ± 25 % Error Level |
| 10 | 90 | 18 |
| 25 | 105 | 31 |
| 50 | 110 | 39 |
| 100 | 115 | 48 |

This study utilises the synthetic design storm concept from AR&R (1987) to estimate the design ARI flood. This methodology is as follows:

- Design Intensity Frequency Duration (IFD) estimates are determined from AR&R for the full range of storm ARIs (2-yr to 100-yr) and durations (30 minutes to 4.5 hours).
- Design temporal patterns are determined and design hyetographs produced for the full range of ARIs and durations.
- Appropriate design rainfall loss parameters are adopted by reference to the calibration and industry standard techniques.
- Using the calibrated models, design storms are simulated and the peak discharges and critical durations established within the model domain.

[4] University Corporation for Atmospheric Research 2010, *Flood Frequency Analysis*, UCAR, USA

### 6.3.2 XP-RAFTS Model Set-up

The calibrated XP-RAFTS model was used to simulate the design storm rainfall-runoff and sub-catchment routing process. The following describes the adjustments made to the model in order to simulate the design events.

***Catchment Development***

The design events were modelled using ultimate catchment hydrological conditions. These conditions assume that the state of development within the catchment is at its ultimate condition, with reference to the current adopted planning scheme. Depending on the developed state of the catchment, an increase in development will generally affect the percentage impervious and the PERN hydrologic roughness values.

Appendix B presents the XP-RAFTS catchment parameters that were adopted for the design event modelling scenarios. The current adopted version of BCC City Plan (2014) was used to establish the ultimate catchment hydrological conditions. The adopted land-use for the ultimate catchment development is shown on a catchment map in Appendix C.

***Rainfall Losses***

The Initial Loss (IL) and Continuing Loss (CL) approach was used to simulate the rainfall losses in order to determine the rainfall excess.

An IL of 0 mm was adopted for both the impervious and pervious areas within the catchment. This value is typically used in BCC flooding studies and is considered a conservative approach.

A CL of 0 mm/hr was also adopted for both the impervious and pervious areas within the catchment. This value was determined from the results of the calibration and verification process. As noted previously, a CL of 0 mm/hr has been used for a number of recently completed flood studies such as Norman Creek, Cabbage Tree Creek, Wynnum Creek and Oxley Creek.

***Design hyetographs***

Design hyetographs were derived from the techniques in AR&R (1987). Hyetographs were created for the 2-yr ARI (39 % AEP), 5-yr ARI (18 % AEP), 10-yr ARI (10 % AEP), 20-yr ARI (5 % AEP), 50-yr ARI (2 % AEP) and 100-yr ARI (1 % AEP) events, considering durations of 30 minutes, 1 hour, 1.5 hours, 2 hours, 3 hours and 4.5 hours.

## 6.4 Design Event Hydraulic Modelling

### 6.4.1 Overview

The TUFLOW model was used to determine design flows and flood levels for those scenarios as detailed in Table 6.2 for the 2-yr ARI (39 % AEP) to the 100-yr ARI (1 % AEP) events. These events were simulated for durations from 30 minutes to 4.5 hours.

### 6.4.2 TUFLOW model extents

The Scenario 1 and 2 TUFLOW model utilised the same model extents as the TUFLOW model developed for the calibration and verification events. However, the Scenario 3 TUFLOW model was truncated at the upstream extent of the Moolabin Creek open waterway, to coincide with the commencement of the conveyance corridor. This truncation meant that the 1.5 km long pipework / overland flow path was not incorporated into the Scenario 3 TUFLOW model.

### 6.4.3 TUFLOW model roughness

The hydraulic roughness in the calibrated TUFLOW model was updated as required to represent the ultimate catchment conditions.

### 6.4.4 TUFLOW model boundaries

***Design Inflows***

The design inflow (Q-T) boundaries to the TUFLOW model were taken from the XP-RAFTS model for each ARI and duration.

For the Scenario 1 and 2 simulations, the inflow locations did not change from those used in the calibration and verification.

For Scenario 3, the modelling of the conveyance corridor necessitated a change to the inflows at the upstream extent of Moolabin Creek, immediately downstream of Ipswich Road. At this location, a lumped XP-RAFTS inflow comprising sub-catchments A1, A2, A3, B1 and B2 was required, as the Scenario 3 hydraulic model was truncated at this location (as noted in Section 6.4.2).

***Design Tailwater Boundary***

The design event TUFLOW model utilised a water level (H-T) boundary at its downstream extent at Oxley Creek.

The study brief required that Mean High Water Springs (MHWS) conditions be utilised for all design events (where applicable). Review of the Oxley Creek Flood Study indicated that during a shorter duration storm event (localised to the lower Oxley Creek Catchment), it is likely that a MHWS level of 1.22 mAHD would be exceeded at the mouth of Moolabin Creek due to the magnitude of the fluvial discharge in Oxley Creek.

The Oxley Creek TUFLOW model was utilised to generate a H-T boundary for all events and durations modelled. Simulations were undertaken with this model utilising a fixed MHWS (1.22 m AHD) boundary at the Oxley Creek / Brisbane River confluence. From the results of each simulation, a water level hydrograph was extracted at the Moolabin Creek / Oxley Creek confluence. It is considered that this methodology would create a more realistic downstream boundary, rather than using purely a fixed MHWS boundary at Oxley Creek.

## 6.5 Results and Mapping

### 6.5.1 Critical Durations

A full range of durations (30 minutes, 1 hour, 1.5 hours, 2 hours, 3 hours and 4.5 hours) were simulated for the 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP) events. From the results, the critical duration at key locations within the catchment was extracted and is provided in Table 6.4. For this purpose, the critical duration is the storm duration which produces the peak flood level.

The results indicate the 60-minute duration storm is critical for both creeks upstream of the Brisbane Golf Course. In the vicinity of Curzon Street the 120-minute duration storm produces the peak flood level.

Table 6.4 – Critical Durations at Key Locations

| Key Location | Critical Duration (minutes) | | | | | |
|---|---|---|---|---|---|---|
| | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| Moolabin Creek | | | | | | |
| Ipswich Road | 60 | 60 | 60 | 60 | 60 | 60 |
| Gow Street | 60 | 60 | 60 | 60 | 60 | 60 |
| Lucy Street | 60 | 60 | 60 | 60 | 60 | 60 |
| Railway Bridges | 60 | 60 | 60 | 60 | 60 | 60 |
| Fairfield Road | 60 | 60 | 60 | 60 | 60 | 60 |
| Curzon Street | 120 | 120 | 120 | 120 | 120 | 120 |
| Rocky Water Holes Creek | | | | | | |
| Ainsworth Street | 60 | 60 | 60 | 60 | 60 | 60 |
| McCarthy Road | 60 | 60 | 60 | 60 | 60 | 60 |
| TAFE_Railway | 60 | 60 | 60 | 60 | 60 | 60 |
| Beaudesert Road | 60 | 60 | 60 | 60 | 60 | 60 |
| Ipswich Road | 60 | 60 | 60 | 60 | 60 | 60 |
| Fairfield Road | 60 | 60 | 60 | 60 | 60 | 60 |

### 6.5.2 Peak Discharge Results

The provision of tabulated peak flow information throughout the creek extents is not a required output for this flood study. However, it is considered good practice to provide peak flows at the major hydraulic structures within the creek extents. The following Table 6.5 provides peak flows at selected major hydraulic structures for the Scenario 1 conditions.

Table 6.5 – Design Event Peak Discharge at Selected Major Structures (Scenario 1)

| Key Location | Peak Discharge ($m^3/s$) | | | | | |
|---|---|---|---|---|---|---|
| | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| Moolabin Creek | | | | | | |
| Gow Street | 35.3 | 42 | 46 | 51.6 | 57.3 | 66.4 |
| Lucy Street | 37.5 | 45.1 | 49.4 | 55.3 | 60.2 | 66 |
| Railway Bridges | 46.5 | 56.9 | 61.5 | 68.6 | 76.4 | 84.5 |
| Curzon Street | 41.2 | 57.3 | 65.5 | 76.1 | 90.3 | 104.6 |
| Rocky Water Holes Creek | | | | | | |
| Ainsworth Street | 16.6 | 23.2 | 27.1 | 32.5 | 37.2 | 42.3 |
| McCarthy Road | 23 | 30.7 | 34.8 | 40.2 | 46.5 | 52.5 |
| Railway_TAFE | 27.5 | 34.9 | 40.4 | 48.8 | 58.5 | 67.1 |
| Beaudesert Road | 34.1 | 45.3 | 50.4 | 59.4 | 72.4 | 83.2 |
| Ipswich Road | 48.3 | 63.5 | 70.2 | 79.7 | 92.1 | 104.9 |

The results indicate that the peak flow generally increases in the downstream direction of both creeks, as would typically be expected. An exception to this is at Curzon Street where the discharge is substantially below the combined upstream peak flows for Moolabin Creek and Rocky Water Holes. This is a primarily a result of substantial attenuation within the Brisbane Golf Course.

### 6.5.3 Peak Flood Levels

Tabulated peak flood level results for the design events are provided at the following locations for the open waterway sections of both creeks:

- Scenario 1: 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP) events – Appendix D
- Scenario 2: 100-yr ARI (1 % AEP) events – Appendix B of the Model Handover Guide
- Scenario 3: 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP) events – Appendix E

The peak flood levels are the maximum flood level when considering the full range of durations from 30-minute to 4.5 hours. The peak flood levels are extracted along the current AMTD line for both Moolabin Creek and Rocky Water Holes Creek.

### 6.5.4 Return Periods of Historic Events

In order to estimate the return period of the historical events modelled, a flood frequency curve was developed at a number of locations along both creeks. These flood frequency curves were based on the Scenario 1 modelling and are indicated in Figure 6.2 and Figure 6.3.

Table 6.6 indicates the estimated return period of the historical events at the selected locations; based on the flood frequency curves.

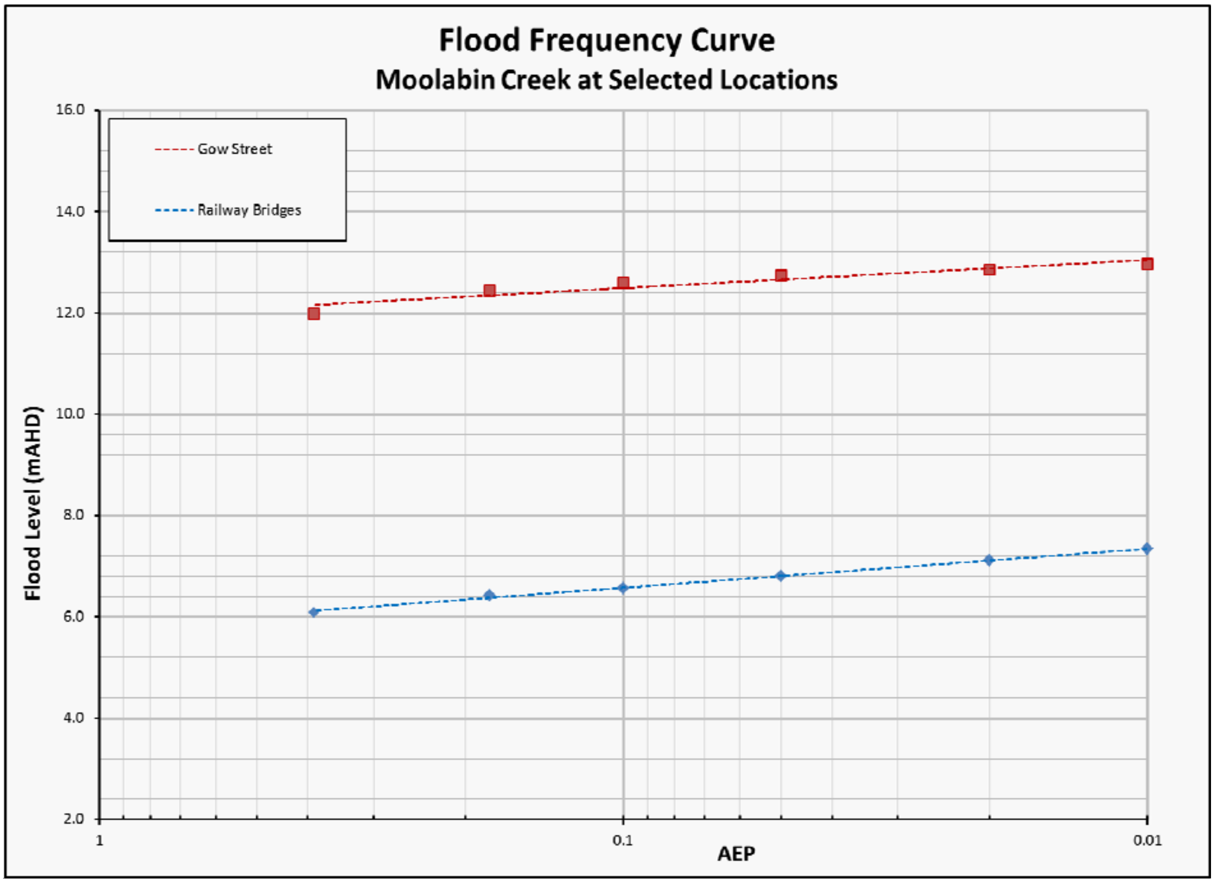


Figure 6.2: Flood Frequency Curve – Moolabin Creek at Selected Locations

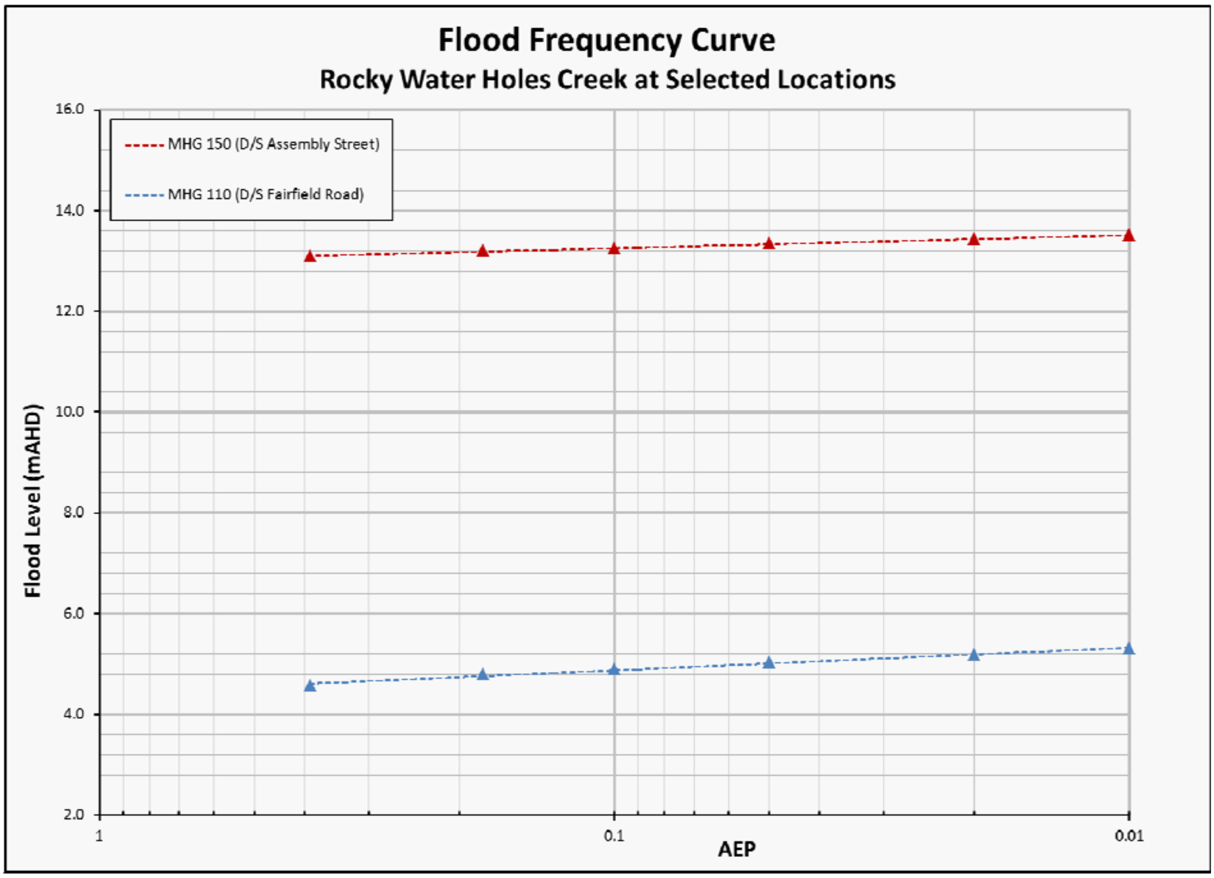


Figure 6.3: Flood Frequency Curve – Rocky Water Holes Creek at Selected Locations

Table 6.6 – Estimate Return Period of Historical Events

| Location | Return Period (ARI years) | | | |
|---|---|---|---|---|
| | Jan 2013 | Feb 2010 | Nov 2004 | Mar 2001 |
| Moolabin Creek | | | | |
| Gow Street | < 2-yr | 2 to 5-yr | < 2-yr | 50 to 100-yr |
| Railway Bridges | < 2-yr | 2 to 5-yr | 2 to 5-yr | 50 to 100-yr |
| Rocky Water Holes Creek | | | | |
| MHG 150 (d/s Assembly Street) | < 2-yr | 2 to 5-yr | < 2-yr | ~ 20-yr |
| MHG 110 (d/s Fairfield Road) | 5-yr (1) | 2 to 5-yr | 2 to 5-yr | 100-yr |

Note (1) – appears to be backwater affected

### 6.5.5 Rating Curves

Rating curves (H-Q) have been derived at a number of locations along both creeks and are provided in Appendix G. These locations are generally in the vicinity of hydraulic structures and include:

- Gow Street
- Lucy Street
- Railway Bridges (Moolabin)
- McCarthy Road
- Railway (TAFE)
- Beaudesert Road
- Ipswich Road

The rating curves were developed by simulating a slowly increasing flow over a period of 30 hours, with a constant tailwater level in Oxley Creek of 1.5 m AHD. In the lower reaches of both creeks, care should be taken if utilising the rating curves, as they have the potential to change depending on the flow conditions in Oxley Creek and the Brisbane River.

### 6.5.6 Flood Immunity of Existing Crossings

The flood immunity of the existing waterway crossings under Scenario 1 conditions is presented in Table 6.7. The value indicated is the ARI of the largest flood which does not fully overtop the road / structure, when considering the 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP) events. Interpolation between ARIs to ascertain an intermediate ARI value has not been undertaken.

Table 6.7 – Flood Immunity at Major Structures

| Location | Flood Immunity (ARI) |
|---|---|
| Moolabin Creek | |
| Gow Street | 2-yr (39 % AEP) |
| Lucy Street | < 2-yr (39 % AEP) |
| Evesham Street | < 2-yr (39 % AEP) |
| Railway Bridges | 100-yr (1 % AEP) |
| Chale Street | < 2-yr (39 % AEP) |
| Fairfield Road | 20-yr (5 % AEP) |
| Curzon Street | 100-yr (1 % AEP) |
| Rocky Water Holes Creek | |
| Ainsworth Street (Footbridge) | 100-yr (1 % AEP) |
| McCarthy Road | 5-yr (18 % AEP) |
| Assembly Street (Footbridge) | 2-yr (39 % AEP) |
| TAFE_Railway | 100-yr (1 % AEP) |
| TAFE_Footbridge | 100-yr (1 % AEP) |
| Beaudesert Road | 100-yr (1 % AEP) |
| Gladstone Street | 20-yr (5 % AEP) |
| Ipswich Road | 100-yr (1 % AEP) |
| Muriel Avenue | < 2-yr (39 % AEP) |
| Railway Bridges | 100-yr (1 % AEP) |
| Muriel Ave. Footbridge | < 2-yr (39 % AEP) |
| Fairfield Road | 50-yr (2 % AEP) |

### 6.5.7 Hydrologic-Hydraulic Model Consistency Check (Design Events)

Comparison checks on flow were undertaken between the XP-RAFTS and TUFLOW models for the 5-yr ARI (18 % AEP), 20-yr ARI (5 % AEP) and 100-yr ARI (1 % AEP) events at selected locations to understand how closely the hydrologic and hydraulic models were matching. Comparisons were undertaken utilising the 60-minute storm event.

Figures 6.4 to 6.8 provide comparative plots of the XP-RAFTS and TUFLOW flow results at the following five locations. Table 6.8 provides a comparison of the peak flows at these same five locations.

(i) Moolabin Creek at Gow Street

(ii) Moolabin Creek at the Railway Bridges

(iii) Moolabin Creek at Curzon Street

(iv) Rocky Water Holes Creek at Railway_TAFE

(v) Rocky Water Holes Creek at Muriel Avenue

Table 6.8 – Peak Flow Comparison, XP-RAFTS and TUFLOW

| Location | Peak Flow ($m^3/s$) | | | | | |
|---|---|---|---|---|---|---|
| | 5-yr ARI (18 % AEP) | | 20-yr ARI (5 % AEP) | | 100-yr ARI (1 % AEP) | |
| | XP-RAFTS | TUFLOW | XP-RAFTS | TUFLOW | XP-RAFTS | TUFLOW |
| Gow Street | 45.1 | 42.0 | 60.6 | 51.6 | 78.9 | 66.4 |
| Railway (Moolabin) | 54.2 | 56.9 | 74.4 | 68.6 | 99.1 | 84.5 |
| Curzon Street | 54.9 | 52.3 | 72.2 | 71.3 | 97.5 | 92.6 |
| Railway_TAFE | 37.2 | 34.9 | 52.0 | 48.8 | 69.8 | 67.1 |
| Muriel Avenue | 58.1 | 63.5 | 84.2 | 79.7 | 118.1 | 104.9 |

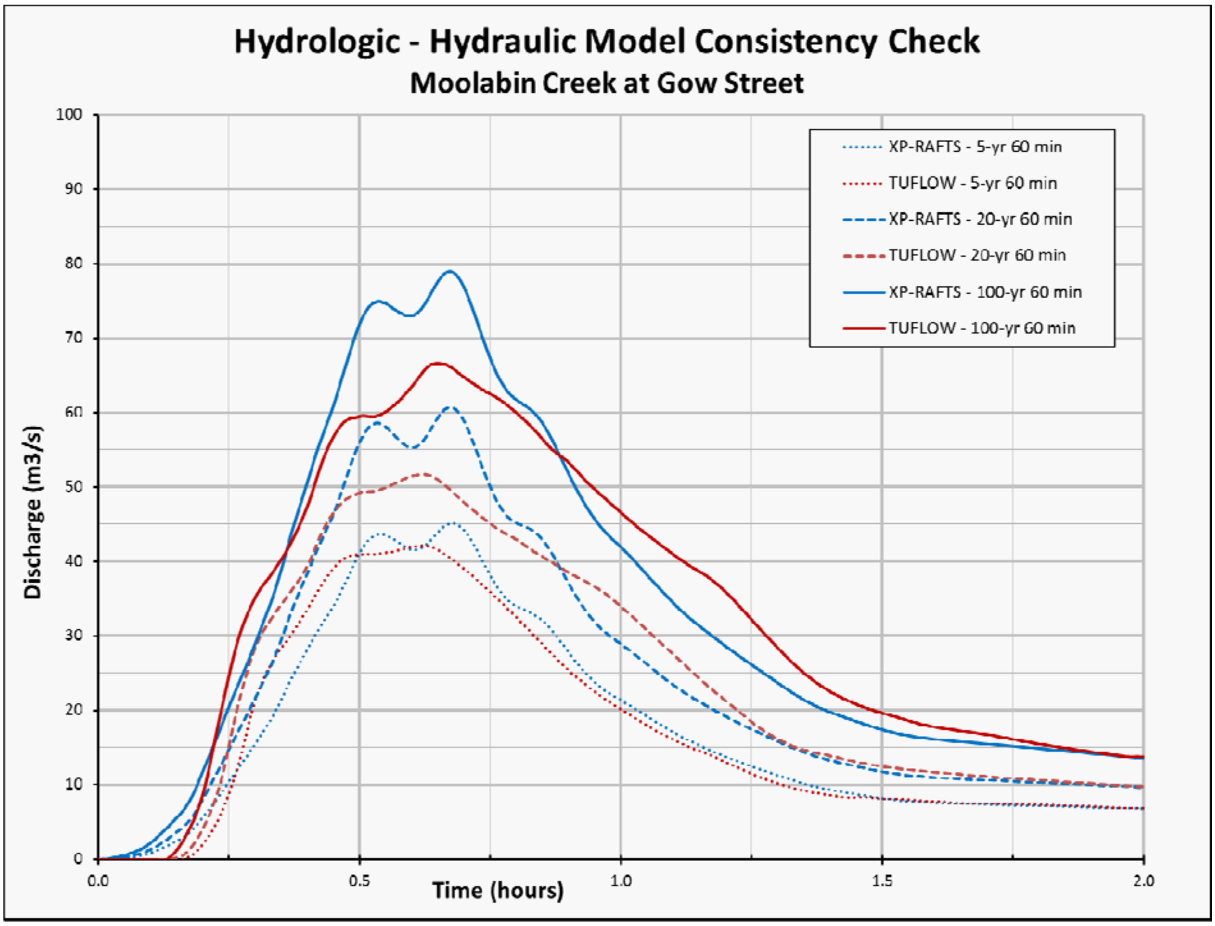

Figure 6.4: Hydrologic-hydraulic comparison at Gow Street

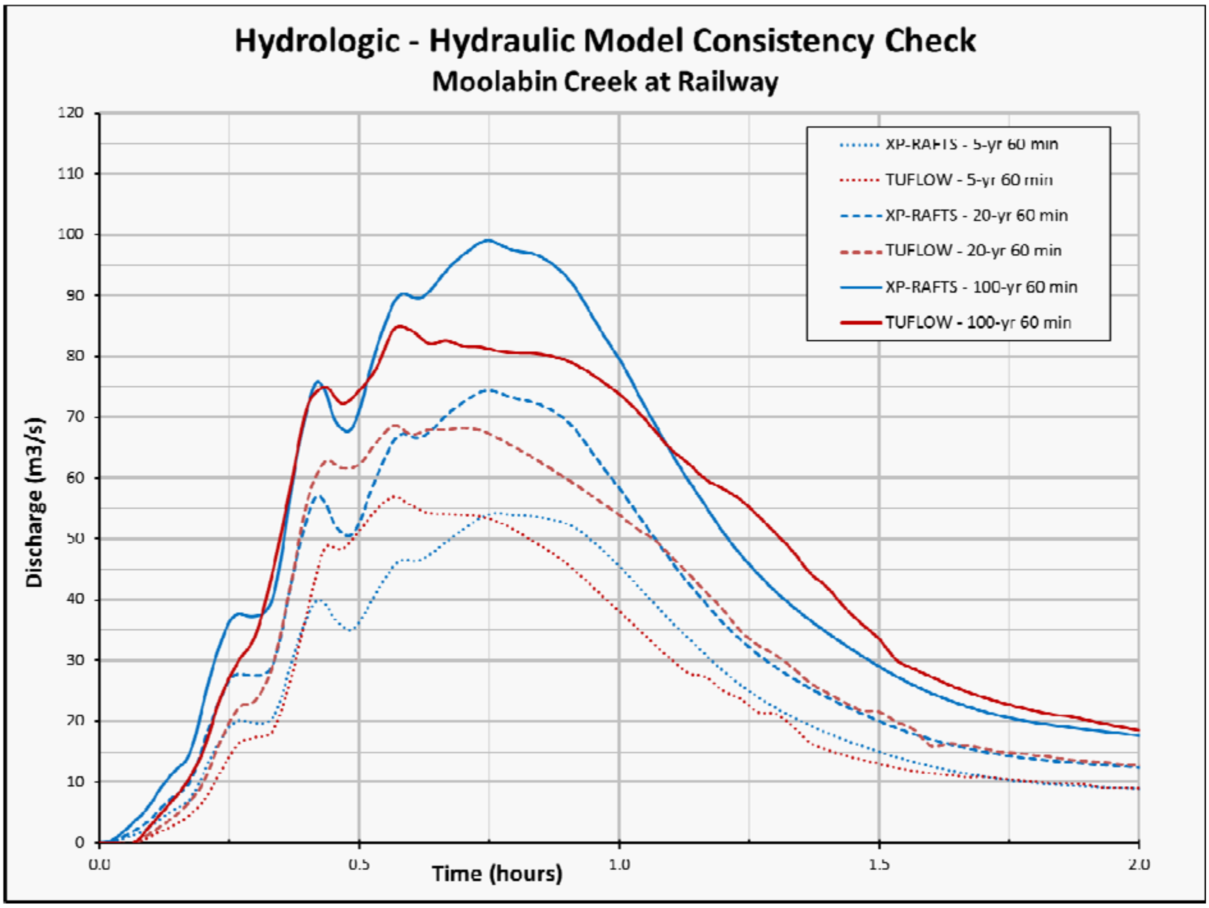

Figure 6.5: Hydrologic-hydraulic comparison at Railway (Moolabin Creek)

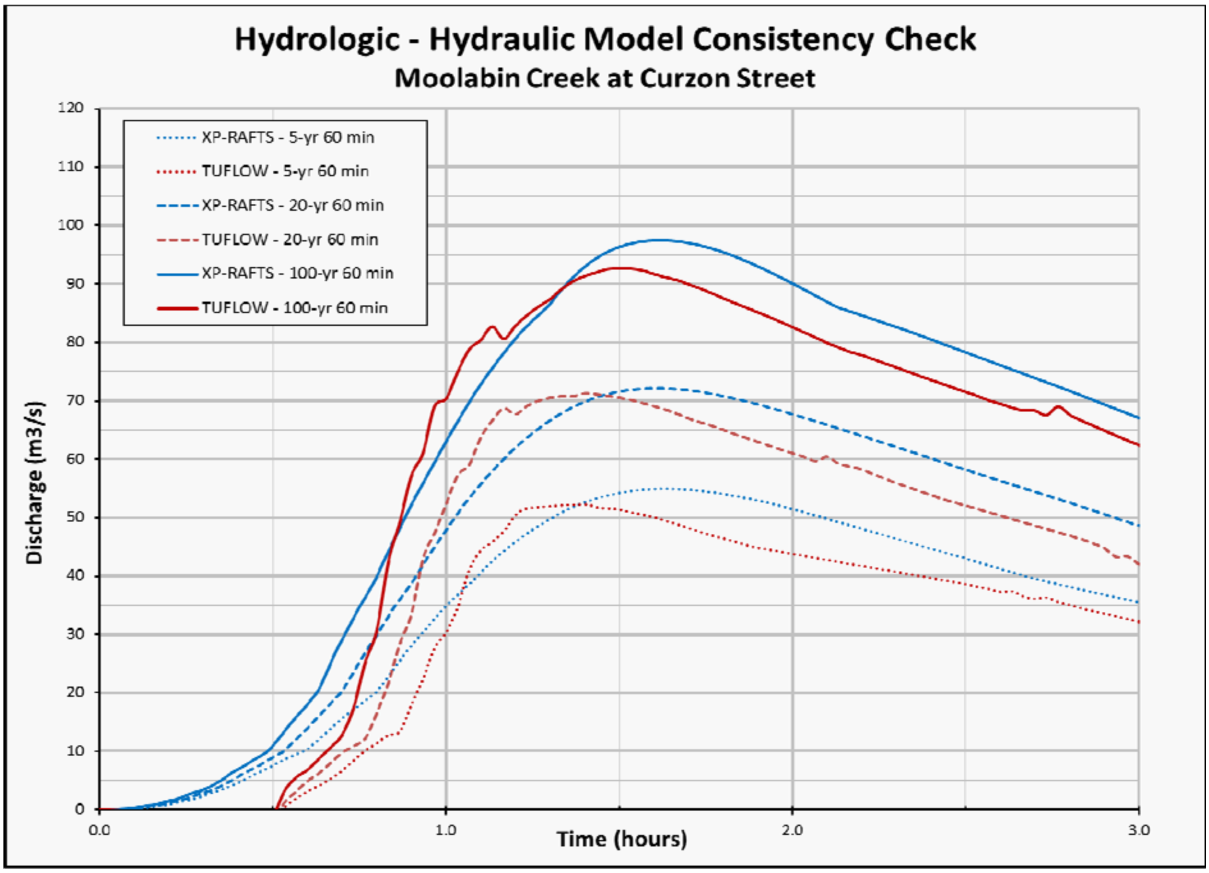

Figure 6.6: Hydrologic-hydraulic comparison at Curzon Street

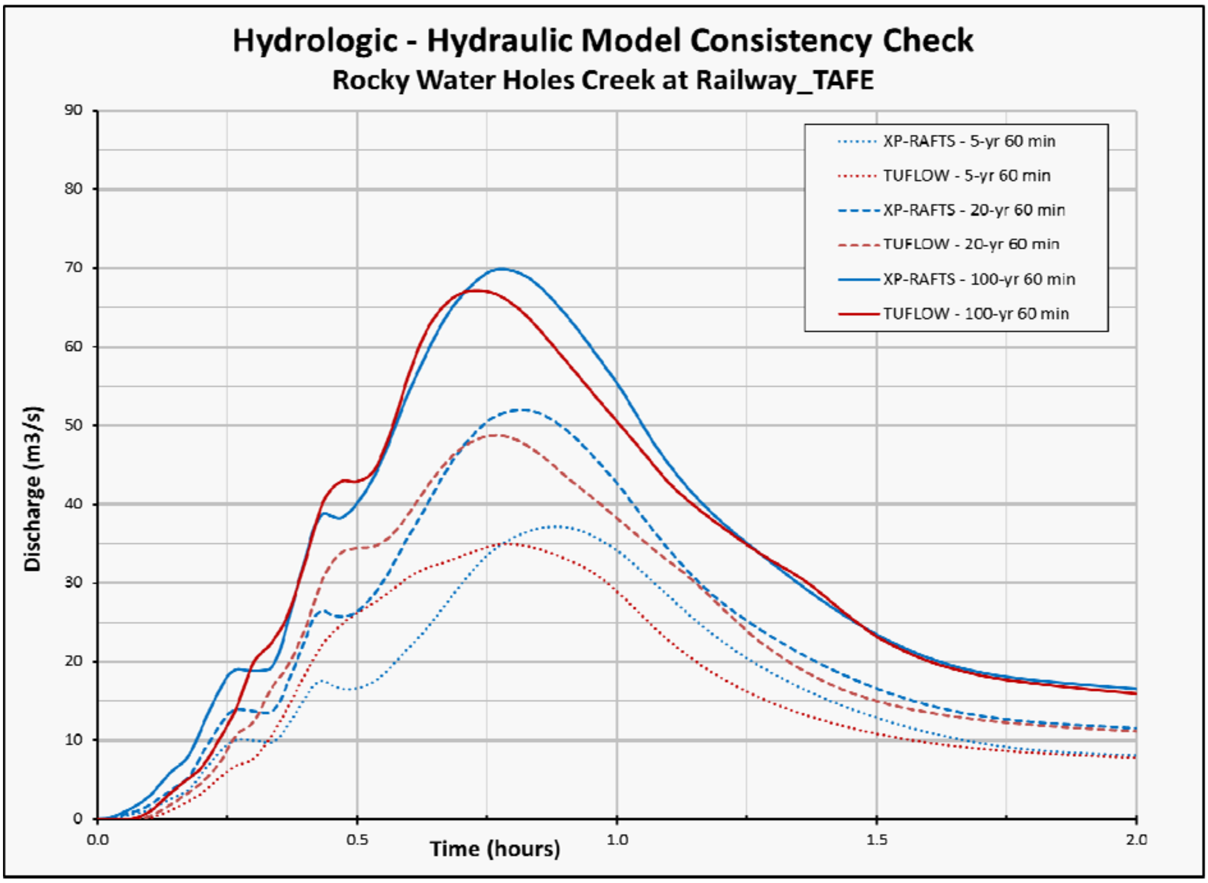

Figure 6.7: Hydrologic-hydraulic comparison at Railway_TAFE

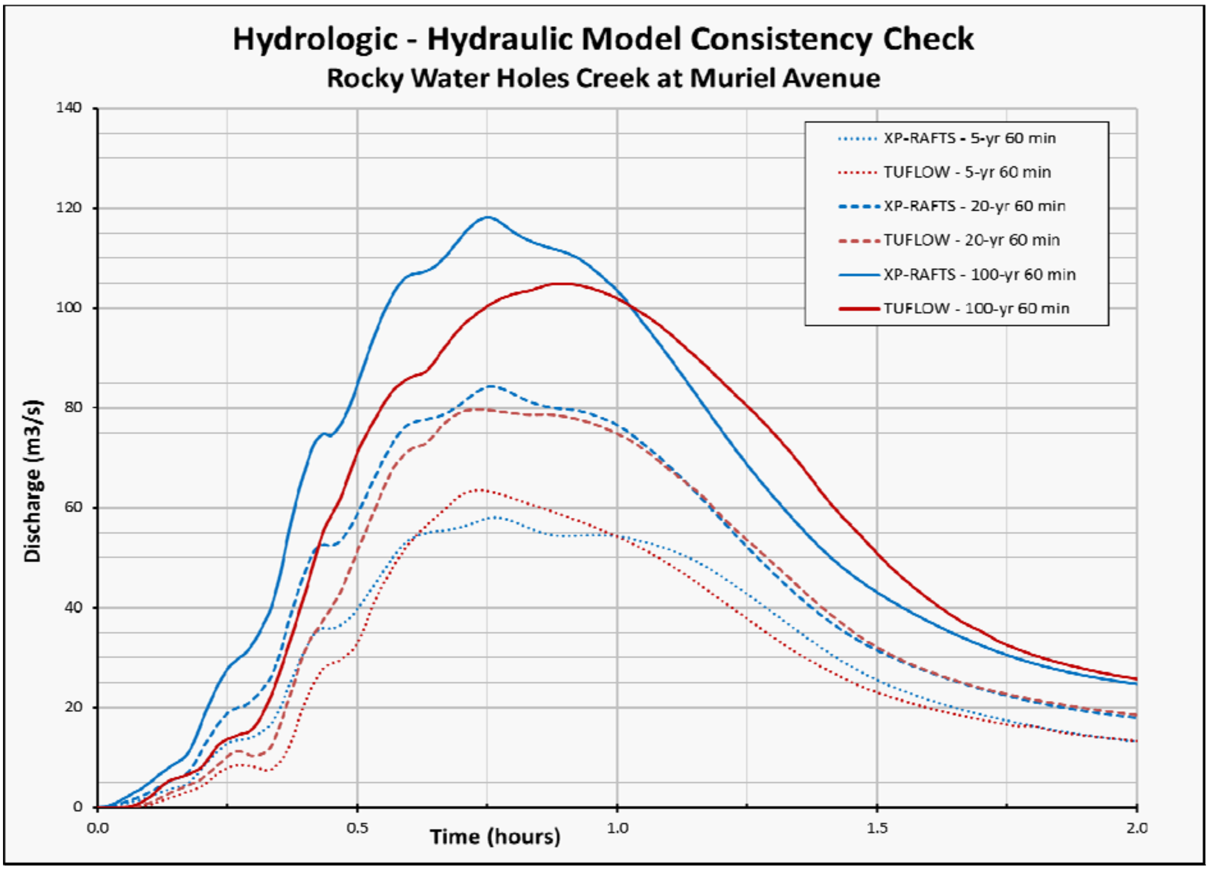

Figure 6.8: Hydrologic-hydraulic comparison at Muriel Avenue

The results indicate a reasonable comparison between the XP-RAFTS and TUFLOW models results at the selected locations. The peak flow is generally within ±10 % and the shape and timing of the hydrographs are relatively consistent.

The largest differences appear to be at both Gow Street and the Railway Bridge(s) on Moolabin Creek. These differences are relatively consistent at both locations and can be largely attributed to the upstream modelling of the 1.5 km long overland flow path above Ipswich Road. The XP-RAFTS model utilises the simplified the link-lag approach to model this reach, whereas the TUFLOW model utilises a more superior 1d pipe / 2d overland flow routing approach. In the smaller 5-yr ARI (18 % AEP) event, where the majority of the flow in this reach would be in the pipe, the XP-RAFTS and TUFLOW flows are similar. However, in the larger events (where above ground storage effects attenuate the flow), the TUFLOW model flows are less than the XP-RAFTS flows, as would be expected. These differences will also contribute to differences between the Scenario 1 / 2 and Scenario 3 flood level results.

### 6.5.8 Hydraulic Structure Reference Sheets

Details of flood level and flow data derived for the hydraulic structure crossings modelled are summarised in the Hydraulic Structure Reference Sheets and included in Appendix H.

### 6.5.9 Flood Mapping

The flood mapping products are provided in Volume 2 and include the following:

- Scenario 1
    - Flood Extent Mapping: 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP)

# 7.0 Rare and Extreme Event Analysis

## 7.1 Rare and Extreme Event Scenarios

Table 7.1 indicates the events and scenarios modelled as part of the rare and extreme event analysis. These scenarios have been previously described in Section 6.2. All rare and extreme event modelling was undertaken using ultimate hydrological conditions.

Table 7.1 – Extreme Event Scenarios

| ARI (year) | AEP (%) | Scenario 1 | Scenario 2 | Scenario 3 |
|---|---|---|---|---|
| 200 | 0.5 | ✓ | ✗ | ✓ |
| 500 | 0.2 | ✓ | ✗ | ✓ |
| 2000 | 0.05 | ✓ | ✗ | ✗ |
| PMF | | ✓ | ✗ | ✗ |

For the modelling of the Scenario 3 events, the fill height outside of the conveyance corridor is set to the Scenario 3 100-yr ARI (1 % AEP) flood level plus an additional height allowance of 0.3 m. The "100-yr ARI (1 % AEP) plus 0.3 m flood surface" is then required to be stretched, of which the methodology is detailed below.

## 7.2 Flood Extent Stretching Process

With the move to two-dimensional flood models, the production of flood levels, extents and depth-velocity products is inherent in simulating a model, i.e. a flood map is a direct output from a model simulation removing the requirement to apply a separate process. For the Scenario 1 "existing" simulations, the model is run and the direct output is able to be mapped or referenced in a GIS environment. In order to simulate the "ultimate" scenario, the model topography must be modified to represent filling associated with development. This in turn affects the resulting flood mapping with the flood extent limited to the edge of the filled floodplain. Post processing of the model output is required to represent the modelled flood levels against the current floodplain conditions.

In order to create the "stretched" flood surface(s), the Scenario 3 "ultimate" flood level surfaces were firstly required to be generated. As previously discussed in Section 6.2, the ultimate scenario involves modifying the flood model topography to represent a fully developed (filled) floodplain in accordance with City Plan and in most instances making further allowances for a riparian corridor.

WaterRIDE was utilised for the purpose of stretching the Scenario 3 "ultimate" case results and producing the "stretched" flood surface(s). The WaterRIDE 'buffer width' tool was used, whereby the surface is extended by an equal number of grid cells (or TIN triangles) as a buffer around the current wet cells. A minimum depth threshold is used to determine what surrounding cells (within the buffer width) are considered 'available' for stretching. For this purpose, a value of 500 was used for the buffer width and -5 for the minimum depth threshold. Using these high values / tolerances ensured the flood surface was initially stretched far beyond the realistic limit of stretching. The stretched flood surface was then mapped onto the ground surface terrain grid to produce the mapped flood extents of the stretched flood surface.

From experience to date, it is known that there are inherent anomalies with the stretching process and some degree of manual intervention is typically required by an experienced / skilled practitioner to produce a more realistic stretched flood surface. To facilitate this process, a comparison of the mapped extent against the "existing" flooding extents (including larger events) was undertaken. In areas where there were obvious anomalies, some minor adjustments were made to the mapped extents of the stretched flood surface.

## 7.3 Rare and Extreme Event Hydrology

### 7.3.1 Overview

Rare and extreme event flood hydrology was determined for the following events, as detailed further in Sections 7.3.2 to 7.3.3.

(i) 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) events
(ii) 2000-yr ARI (0.05 % AEP) event, and
(iii) Probable Maximum Precipitation (PMP)

### 7.3.2 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) Events

The 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) design IFD rainfall data was obtained using the CRC-Forge method for the events.

Table 7.2 indicates the adopted 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) design rainfall intensities with comparison to the adopted 100-yr ARI (1 % AEP). The 2-hour and 4.5-hour values were interpolated as CRC-Forge does not produce results for these intermediate values. The interpolation was based by plotting a graph (i.e. 200-yr and 500-yr ARI) and estimating the values at the time of interest.

The 100-yr ARI (1 % AEP) AR&R design temporal pattern was adopted for both these events to create the hyetograph.

Table 7.2 – Adopted IFD (200-yr ARI and 500-yr ARI)

| Duration (hr) | Rainfall Intensity (mm/hr) | | |
|---|---|---|---|
| | 100-yr ARI (1 % AEP) | 200-yr ARI (0.5 % AEP) | 500-yr ARI (0.2 % AEP) |
| 0.5 | 156.3 | 171.9 | 201.2 |
| 1 | 110.1 | 120.5 | 141.1 |
| 2 | 68.5 | 75.5 (1) | 88.4 (1) |
| 3 | 51.5 | 55.8 | 65.4 |
| 4.5 | 38.7 | 41.8 (1) | 49 (1) |

Note (1) - Interpolated value

### 7.3.3 2000-yr ARI (0.05 % AEP) and PMP

Table 7.3 indicates the adopted super-storm temporal pattern and hyetographs for the 2000-yr ARI (0.05 % AEP) and the PMP.

Table 7.3 – Adopted Super-storm Hyetographs

| Time (hr) | Rainfall (%) | Rainfall (mm) | | Time (hr) | Rainfall (%) | Rainfall (mm) | |
|---|---|---|---|---|---|---|---|
| | | 2000-yr ARI (0.05 % AEP) | PMP | | | 2000-yr ARI (0.05 % AEP) | PMP |
| 0.00 | 0 | 0.00 | 0.00 | 3.17 | 58 | 41.00 | 75.08 |
| 0.17 | 1 | 4.33 | 9.92 | 3.33 | 70 | 41.00 | 75.08 |
| 0.33 | 3 | 4.33 | 9.92 | 3.50 | 75 | 16.00 | 38.25 |
| 0.50 | 4 | 4.33 | 9.92 | 3.67 | 77 | 7.58 | 27.63 |
| 0.67 | 5 | 4.33 | 9.92 | 3.83 | 80 | 7.58 | 27.63 |
| 0.83 | 6 | 4.33 | 9.92 | 4.00 | 82 | 7.58 | 27.63 |
| 1.00 | 8 | 4.33 | 9.92 | 4.17 | 84 | 7.58 | 18.42 |
| 1.17 | 9 | 4.33 | 13.46 | 4.33 | 86 | 7.58 | 18.42 |
| 1.33 | 10 | 4.33 | 13.46 | 4.50 | 89 | 7.58 | 18.42 |
| 1.50 | 11 | 4.33 | 13.46 | 4.67 | 90 | 4.33 | 13.46 |
| 1.67 | 14 | 7.58 | 18.42 | 4.83 | 91 | 4.33 | 13.46 |
| 1.83 | 16 | 7.58 | 18.42 | 5.00 | 92 | 4.33 | 13.46 |
| 2.00 | 18 | 7.58 | 18.42 | 5.17 | 94 | 4.33 | 9.92 |
| 2.17 | 20 | 7.58 | 27.63 | 5.33 | 95 | 4.33 | 9.92 |
| 2.33 | 23 | 7.58 | 27.63 | 5.50 | 96 | 4.33 | 9.92 |
| 2.50 | 25 | 7.58 | 27.63 | 5.67 | 97 | 4.33 | 9.92 |
| 2.67 | 30 | 16.00 | 38.25 | 5.83 | 99 | 4.33 | 9.92 |
| 2.83 | 34 | 16.00 | 38.25 | 6.00 | 100 | 4.33 | 9.92 |
| 3.00 | 46 | 41.00 | 75.08 | | | | |

The 2000-yr ARI (0.05 % AEP) IFD rainfall was determined using the CRC-Forge method. To avoid the need to simulate all of the different storm durations, a simplified super-storm method was used. This same methodology has also been used on other BCC flood studies currently being undertaken.

The rationale for adopting this approach is that world-wide research indicates that as storm rainfall depths increase during short duration storms, the rainfall intensity becomes more uniform. For this reason, the multi-peaked AR&R temporal pattern (as used for the 200-yr ARI and 500-yr ARI) was not considered suitable for the analysis of this more extreme event.

A 6-hr super-storm was developed to represent all storm durations up to 6 hours. The super-storm was developed in 30 minute blocks and incorporates the 0.5-hr, 1-hr, 1.5-hr, 2-hr and 3-hr storm bursts. Durations less than 30 minutes were not considered. The total rainfall depth of the super-storm was set equal to the 6-hr 2000-yr ARI (0.05 % AEP) CRC-Forge rainfall depth (representative across the Brisbane Region) which was determined as 340 mm.

For the PMP scenario, the 6-hr super-storm approach was also undertaken using the same temporal pattern as the 2000-yr ARI (0.05 % AEP) event.

The total PMP depth was derived from the 6-hr storm duration using the Generalised Short Duration Method (GSDM). For the tropical and sub-tropical coastal areas it is recommended that this method is to be used to estimate the PMP over areas up to 520 $km^2$ and for durations up to 6 hours. To apply a consistent methodology across the majority of BCC an average catchment size of 60 $km^2$ and moisture adjustment factor of 0.85 were adopted.

The total rainfall depth of the super-storm was set equal to the 6-hr GSDM PMP rainfall depth, which was determined as 816 mm.

## 7.4 Hydraulic Modelling

### 7.4.1 General

The TUFLOW model was used to simulate the scenarios as detailed in Section 7.1 to enable design flood levels and flood mapping products to be determined / produced.

### 7.4.2 TUFLOW model extents

No changes were made from the design event TUFLOW model(s).

### 7.4.3 TUFLOW model roughness

No changes were made from the design event TUFLOW model(s).

### 7.4.4 TUFLOW model boundaries

***Design Inflows***

The rare and extreme event inflow (Q-T) boundaries to the TUFLOW model were taken from the results of the XP-RAFTS model for each ARI and duration. The inflow locations did not change from the design event TUFLOW model(s).

***Design Tailwater Boundary***

The rare and extreme event TUFLOW model utilised a water level (H-T) boundary at its downstream extent at Oxley Creek and also a second H-T boundary to represent the high-level bypass culverts at Sherwood Road. In larger flooding events, the high-level bypass channel conveys a considerable flow from Oxley Creek, therefore the inclusion of this boundary is considered more realistic for the larger events.

The study brief required that Highest Astronomical Tide (HAT) conditions be utilised for all events (where applicable). Review of the Oxley Creek Flood Study indicated that during a shorter duration storm event (localised to the lower Oxley Creek Catchment), it is likely that a HAT level of 1.83 mAHD would be exceeded at the mouth of Moolabin Creek due to the magnitude of the fluvial discharge in Oxley Creek.

The Oxley Creek TUFLOW model was utilised to generate a H-T boundary at both boundary locations for all events and durations modelled. Simulations were undertaken with this model utilising a fixed HAT (1.83 mAHD) boundary at the Oxley Creek / Brisbane River confluence. From the results of each simulation, a water level hydrograph was extracted at both of the boundary locations. It is considered that this methodology would create a more realistic downstream boundary, rather than using purely a fixed HAT boundary at Oxley Creek.

The Oxley Creek TUFLOW model was not able utilised to determine the boundary conditions for the PMF event, as there were instability problems of which resolving were outside the scope of this project. Therefore, the PMF event utilised the 2000-yr (0.05 % AEP) boundary conditions at both boundary locations.

### 7.4.5 Hydraulic Structures

The TUFLOW model(s) for the 200-yr ARI (0.5 % AEP), 500-yr ARI (0.2 % AEP) and 2000-yr ARI (0.05 % AEP) events incorporated the same hydraulic structures as the design event TUFLOW model(s).

To limit issues with model instabilities, the TUFLOW model for the PMF effectively excluded the following structures:

- Railway Bridges (Moolabin Creek)
- Fairfield Road (Moolabin Creek)
- Fairfield Road (Rocky Water Holes Creek)

Similarly, the TUFLOW model for the 2000-yr ARI (0.05 % AEP) and PMF events excluded bridge handrails at the following structures:

- Gow Street
- McCarthy Road
- Gladstone Street

## 7.5 Results and Mapping

### 7.5.1 Peak Flood Levels

Tabulated peak flood level results for the rare and extreme events are provided at the following locations for the open waterway sections of both creeks:

- Scenario 1: 200-yr ARI (0.5 % AEP) to 2000-yr ARI (0.05 % AEP) events – Appendix A of the Model Handover Guide
- Scenario 3: 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) events – Appendix F

### 7.5.2 Flood Mapping

The flood mapping products are provided in Volume 2 and include the following:

- Scenario 1
  - Flood Extent Mapping: 200-yr ARI (0.5 % AEP), 500-yr ARI (0.2 % AEP) and 2000-yr ARI (0.05 % AEP)

### 7.5.3 Discussion of Results

A longitudinal plot of the Scenario 1 100-yr ARI (1 % AEP) to PMF flood profiles for Moolabin Creek and Rocky Water Holes Creek is provided in Figure 7.1 and Figure 7.2 respectively.

The flood profile for both the 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) events are observed to follow a very similar trend when compared to the 100-yr ARI (1 % AEP) flood profile along both creeks. The hydraulic gradient across the Brisbane Golf Course is virtually flat and there is a considerable head difference across Curzon Street.

The Moolabin Creek flood profiles for the 100-yr ARI (1 % AEP) to 500-yr ARI (0.2 % AEP) event are observed to rise sharply within a short length of creek immediately downstream of Fairfield Road. At this location the floodplain appears to be quite constrained by development on both sides of the creek.

The results indicate a significant head difference from the upstream of Ipswich Road (ch.1150) to downstream of Fairfield Road (ch.950) for all flood profiles on Rocky Water Holes Creek. There is also a large head difference across Beaudesert Road in the more extreme 2000-yr ARI (0.05 % AEP) and PMF events.

The average increase in flood level along the length of both creeks when compared to the 100-yr ARI (1 % AEP) flood profile is indicated in Table 7.4. The results indicate the average increase in flood level is consistent for both creeks.

Table 7.4 – Average Increase in Flood Level

| Event | Average Increase in Flood Level (m) with reference to the 100-yr ARI (1 % AEP) flood level | |
|---|---|---|
| | Moolabin Creek | Rocky Water Holes Creek |
| 200-yr ARI (0.5 % AEP) | 0.15 | 0.12 |
| 500-yr ARI (0.2 % AEP) | 0.39 | 0.32 |
| 2000-yr ARI (0.05 % AEP) | 1.09 | 0.89 |
| PMF | 2.05 | 2.03 |

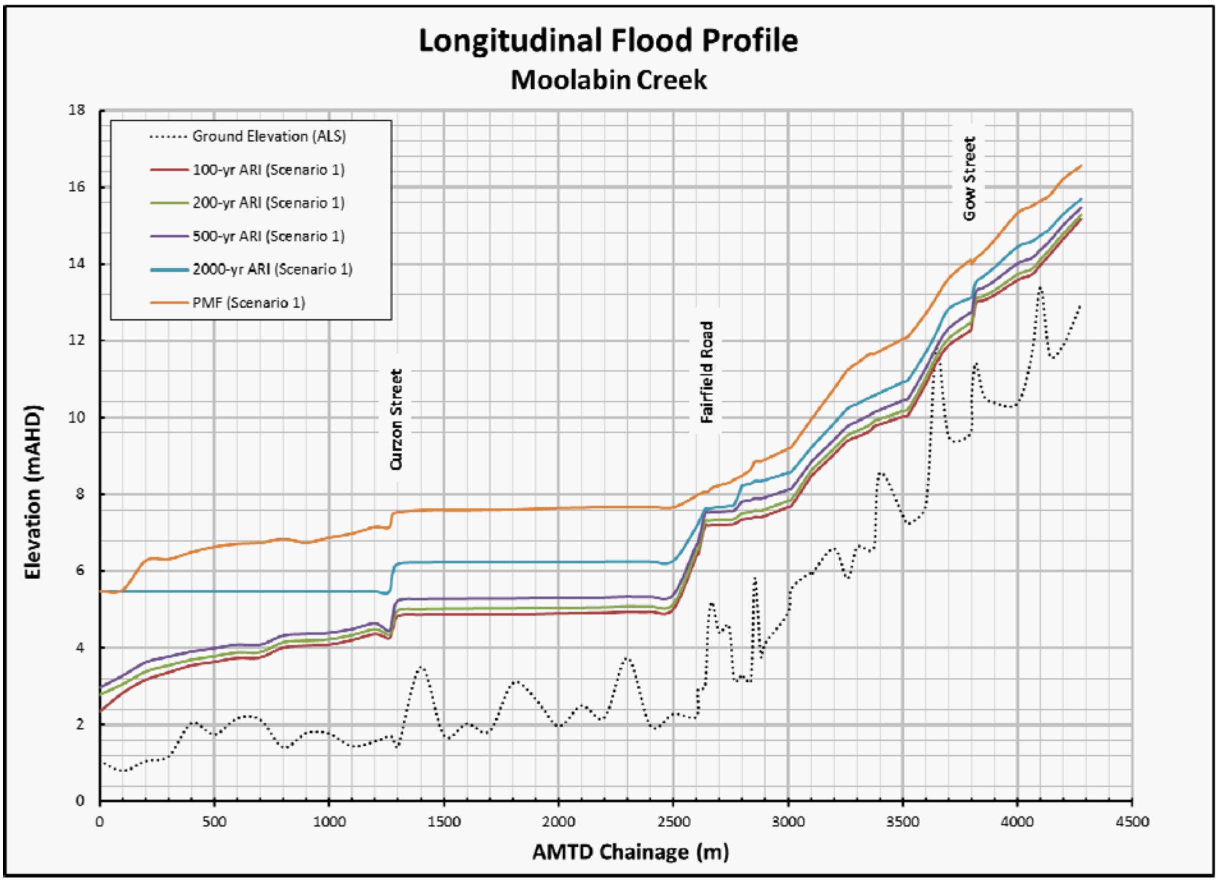


Figure 7.1: Longitudinal Flood Profile – Moolabin Creek

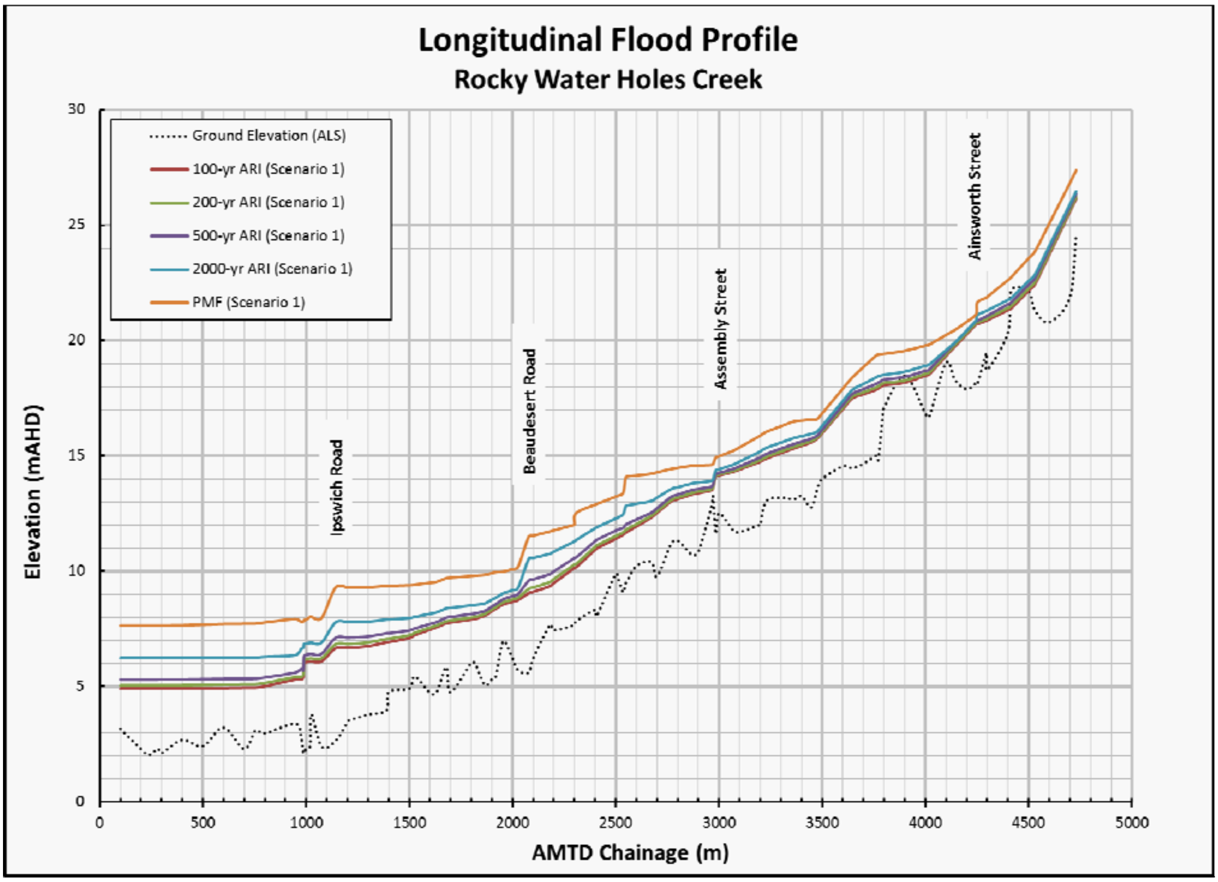


Figure 7.2: Longitudinal Flood Profile – Rocky Water Holes Creek

# 8.0 Climate Change and Structure Blockage

## 8.1 Overview

To enable comprehensive strategic planning to be undertaken, BCC flood studies are required to undertake a sensitivity analysis to address the following:

- Climate change
- Hydraulic structure blockage

The following sections provide the details of these analyses.

## 8.2 Climate Change

### 8.2.1 Overview

To enable BCC to undertake future land-use planning from an informed perspective, there is a requirement to understand the impacts of climate change on flooding. BCC flood studies are therefore required to utilise the latest statutory guidelines in order to assess the impacts of climate change.

To enable BCC to understand and plan for the impacts of climate change on flooding, a number of climate change scenarios were undertaken, as outlined below. These scenarios are consistent with the most recently completed BCC flood studies and the latest statutory guidelines.

- 2050 Planning Horizon
  - 10 % increase in rainfall intensity
  - 0.3 m increase in mean sea level
- 2100 Planning Horizon
  - 20 % increase in rainfall intensity
  - 0.8 m increase in mean sea level

### 8.2.2 Modelled Scenarios

Modelling was undertaken to determine the climate change impacts for the 100-yr ARI (1 % AEP), 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) events. Table 8.1 indicates the events modelled and the respective climate change modifications undertaken.

Table 8.1 – Climate Change Modelling Scenarios

| ARI (year) | AEP (%) | Planning horizon | Rainfall Intensity | Tailwater Condition | Scenario 1 | Scenario 3 |
|---|---|---|---|---|---|---|
| 100 | 1 | 2050 | + 10 % | MHWS + 0.3 m | ✓ | ✓ |
| | | 2100 | + 20 % | MHWS + 0.8 m | ✓ | ✓ |
| 200 | 0.5 | 2050 | + 10 % | HAT + 0.3 m | ✓ | × |
| | | 2100 | + 20 % | HAT + 0.8 m | ✓ | × |
| 500 | 0.2 | 2100 | + 20 % | HAT + 0.8 m | ✓ | × |

### 8.2.3 Hydraulic Modelling

The climate change TUFLOW model(s) incorporated the same model set-up as the design event TUFLOW model(s), apart from the boundary conditions.

The XP-RAFTS model was utilised to derive the inflow boundary conditions for the +10 % rainfall intensity and +20 % rainfall intensity scenarios. The inflow boundary locations did not change from the design event modelling.

Similar to the design and extreme events, the downstream boundary conditions were derived by utilising the Oxley Creek TUFLOW model. Simulations were undertaken with this model utilising a fixed boundary at the Oxley Creek / Brisbane River confluence. The fixed boundary condition used at the confluence with the Brisbane River corresponded to that indicated in Table 8.1. It is considered that this methodology would create a more realistic downstream boundary, rather than using purely a fixed HAT boundary at Oxley Creek.

### 8.2.4 Tabulated Results

Tabulated peak flood level results for the climate change events are provided at the following locations for the open waterway sections of both creeks:

- Scenario 1 (2050): 100-yr ARI (1 % AEP) and 200-yr ARI (0.5 % AEP) events - Appendix C of the Model Handover Guide
- Scenario 1 (2100): 100-yr ARI (1 % AEP) to 500-yr ARI (0.2 % AEP) events - Appendix C of the Model Handover Guide
- Scenario 3 (2050): 100-yr ARI (1 % AEP) event - Appendix D of the Model Handover Guide
- Scenario 3 (2100): 100-yr ARI (1 % AEP) event - Appendix D of the Model Handover Guide

### 8.2.5 Impacts of Climate Change

Tables 8.2 to 8.4 indicate a comparison of the peak flood levels for the Scenario 1 climate change conditions. The flood level results are provided at selected locations along both creeks for the 100-yr ARI (1 % AEP), 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) events. The results indicate the greatest change in flood level is generally in the lower reaches where the projected sea-level rise has the greatest impact.

The results indicate that climate change impacts within the catchment will increase the magnitude of flooding, for example:

- By the year 2050, the 100-yr ARI (1 % AEP) flood levels will be of similar magnitude to the current day 200-yr ARI (0.5 % AEP) flood levels.
- By the year 2100, the 200-yr ARI (0.5 % AEP) flood levels will be of similar magnitude to the current day 500-yr ARI (0.2 % AEP) flood levels.

Table 8.2 – 100-yr ARI (1 % AEP) Climate Change Impacts at Selected Locations (Scenario 1)

| Structure Location | 100-yr ARI (1 % AEP) Flood Level (m AHD) | | |
|---|---|---|---|
| | Existing | 2050 | 2100 |
| Moolabin Creek | | | |
| Gow Street | 12.97 | 13.09 | 13.20 |
| Lucy Street | 9.78 | 9.93 | 10.05 |
| Railway Bridges | 7.34 | 7.52 | 7.68 |
| Fairfield Road | 7.18 | 7.32 | 7.44 |
| Curzon Street | 4.48 | 4.59 | 4.70 |
| Rocky Water Holes Creek | | | |
| Ainsworth Street | 20.72 | 20.78 | 20.83 |
| McCarthy Road | 18.06 | 18.16 | 18.25 |
| Railway_TAFE | 11.68 | 11.80 | 11.92 |
| Beaudesert Road | 9.05 | 9.25 | 9.44 |
| Ipswich Road | 6.66 | 6.85 | 7.00 |
| Fairfield Road | 5.33 | 5.44 | 5.70 |

Table 8.3 – 200-yr ARI (0.5 % AEP) Climate Change Impacts at Selected Locations (Scenario 1)

| Structure Location | 200-yr ARI (0.5 % AEP) Flood Level (m AHD) | | |
|---|---|---|---|
| | Existing | 2050 | 2100 |
| Moolabin Creek | | | |
| Gow Street | 13.08 | 13.20 | 13.30 |
| Lucy Street | 9.91 | 10.05 | 10.18 |
| Railway Bridges | 7.50 | 7.69 | 7.85 |
| Fairfield Road | 7.31 | 7.44 | 7.55 |
| Curzon Street | 4.59 | 4.72 | 4.86 |
| Rocky Water Holes Creek | | | |
| Ainsworth Street | 20.78 | 20.83 | 20.88 |
| McCarthy Road | 18.16 | 18.25 | 18.33 |
| Railway_TAFE | 11.80 | 11.92 | 12.06 |
| Beaudesert Road | 9.24 | 9.44 | 9.66 |
| Ipswich Road | 6.84 | 7.00 | 7.16 |
| Fairfield Road | 5.43 | 5.70 | 5.87 |

Table 8.4 – 500-yr ARI (0.2 % AEP) Climate Change Impacts at Selected Locations (Scenario 1)

| **Structure Location** | **500-yr ARI (0.2 % AEP) Flood Level (m AHD)** | |
|---|---|---|
| | **Existing** | **2100** |
| Moolabin Creek | | |
| Gow Street | 13.28 | 13.46 |
| Lucy Street | 10.14 | 10.41 |
| Railway Bridges | 7.81 | 8.14 |
| Fairfield Road | 7.52 | 7.69 |
| Curzon Street | 4.79 | 5.09 |
| Rocky Water Holes Creek | | |
| Ainsworth Street | 20.86 | 21.14 |
| McCarthy Road | 18.31 | 18.49 |
| Railway_TAFE | 12.02 | 12.35 |
| Beaudesert Road | 9.60 | 10.11 |
| Ipswich Road | 7.11 | 7.49 |
| Fairfield Road | 5.77 | 6.30 |

## 8.3 Hydraulic Structure Blockage

### 8.3.1 Overview

Blockage of hydraulic structures is a common cause of increasing flood risk over and above the risk due to the intensity and duration of the rainfall. Current guidance recommends that designers of hydraulic structures should make allowances for the risk of blockage in the design. However, current guidance does not stipulate that blockage is required to be included as part of the determination of the overall design flood level.

BCC has taken the approach to include the blockage of selected hydraulic structures as part of a sensitivity analysis. This approach will provide an understanding of the potential impacts should the selected hydraulic structure(s) become blocked during an event.

### 8.3.2 Selection of Hydraulic Structures

The following six hydraulic structures were selected for the blockage analysis:

- Moolabin Creek
  - Gow Street (4 / 1800 mm RCP)
  - Fairfield Road (3 / 3350 x 1000 mm RCBC + 1 / 3700 x 3250 mm RCBC)

- Rocky Water Holes Creek
  - McCarthy Road Culverts (3 / 1650 mm RCP + 2 / 3000 x 1800 mm RCBC)
  - Beaudesert Road (6 / 2140 x 2170 mm RCBC)
  - Ipswich Road (4 / 3150 mm Armco)
  - Muriel Avenue (7 / 2130 x 2250 mm RCBC)

These structures were primarily selected based on limiting the size of the bridge / culvert dimensions. However, other factors were considered including the following:

- the predominant upstream catchment use;
- availability of woody debris;
- existing submergence of the inlet;
- flood risk of upstream properties; and
- flooding characteristics of the reach

### 8.3.3 Blockage Scenarios

The blockage analysis has been carried out with the existing case scenario (Scenario 1) for the 100-yr ARI (1 % AEP) design event only. Individual structures were blocked and modelled separately to ensure that the blockage impacts would not be masked by the effect of blocking other upstream structures.

The Queensland Urban Drainage Manual (QUDM) was used as guidance for the degree of blockage for each structure; refer to Table 10.4.1 of this manual. For the purposes of this sensitivity analysis "severe" blockage conditions have been assumed. "Severe" blockage is defined as the level of blockage considered possible during the design life of the structure. Given that the sensitivity analysis is only being undertaken for a low probability large flooding event (i.e. 100-yr ARI), which is only likely to occur one or two times during the design life of the structure, this level of blockage is considered more appropriate than the "design" blockage.

### 8.3.4 Impacts of Structure Blockage

Table 8.5 indicates the 100-yr ARI (1 % AEP) flood level and afflux immediately upstream of the hydraulic structure for each of six blockage simulations. The 2000-yr ARI (0.05 % AEP) flood level is also shown for comparative purposes. The flood level results for the entire length of both creeks are provided in Appendix E of Volume 2.

The results indicate that at three locations the full blockage of the structure results in an upstream flood level greater than the 2000-yr ARI (0.05 % AEP) flood level. These locations are at Fairfield Road (Moolabin Ck), McCarthy Road and Beaudesert Road.

The most severe blockage impacts occur at Beaudesert Road on Rocky Water Holes Creek. At this location the blocked upstream flood level is considerably higher than the 2000-yr ARI (0.05 % AEP) flood level. The flooding extents of the blocked scenario are significantly greater than the un-blocked scenario resulting in substantially worse flooding to road infrastructure and properties.

At Muriel Avenue, the full blockage of the structure results in negligible afflux. This is a result of the flood level being controlled by the downstream structures.

Table 8.5 – 100-yr ARI Blockages (Scenario 1)

| Blockage Scenario | Structure Location | Blockage Factor Applied (%) | Flood Level (m AHD) | | | Afflux (m) |
|---|---|---|---|---|---|---|
| | | | 100-yr ARI (1% AEP) | | 2000-yr ARI (0.05 % AEP) | |
| | | | Existing | Blockage | | |
| BL1 | Gow Street | 100 | 12.96 | 13.33 | 13.52 | 0.37 |
| BL2 | Fairfield Road | 100 | 7.18 | 7.78 | 7.61 | 0.60 |
| BL3 | McCarthy Road | 100 | 18.06 | 18.65 | 18.51 | 0.59 |
| BL4 | Beaudesert Road | 100 | 9.05 | 11.15 | 10.54 | 2.10 |
| BL5 | Ipswich Road | 100 | 6.66 | 7.76 | 7.79 | 1.10 |
| BL6 | Muriel Avenue | 100 | 6.10 | 6.12 | 6.91 | 0.02 |

# 9.0 Summary of Study Findings

This flood study report details the calibration and verification, design events, extreme events and sensitivity modelling for Moolabin Creek and Rocky Water Holes Creek. New hydrologic and hydraulic models have been developed for the study using the XP-RAFTS and TUFLOW modelling software respectively.

Hydrometric data was sourced from the available recorded rainfall data. Numerous MHG's are present within the catchment, however only one continuous stream gauge exists. Calibration of the XP-RAFTS and TUFLOW models was undertaken for the January 2013, February 2010 and March 2001 events. Verification of the XP-RAFTS and TUFLOW models was undertaken for the November 2004 event.

The results of the hydraulic calibration and verification indicated that the XP-RAFTS and TUFLOW models were able to satisfactorily replicate the historical flooding events to within the specified tolerances. On this basis, it was concluded that the XP-RAFTS and TUFLOW models were sufficiently robust to be used to accurately simulate design flood events.

Cross-checks of the TUFLOW structure head-losses were undertaken at selected structures using the HEC-RAS software, from which it was confirmed that the model was representing the structures adequately.

Design and extreme flood magnitudes were estimated for the full range of events from 2-yr ARI (39% AEP) to PMF. These analyses assumed hydrologic ultimate catchment development conditions in accordance with BCC City Plan (2014).

Three waterway scenarios were considered as follows:

- Scenario 1 is based on the current waterway conditions. No further modifications were made to the TUFLOW model developed as part of the calibration / verification phase.
- Scenario 2 includes an allowance for a riparian corridor along the edge of the channel.
- Scenario 3 includes an allowance for the riparian corridor (as per Scenario 2) and also assumes filling to the Conveyance Corridor boundary to simulate potential development.

The results from the TUFLOW modelling were used to produce the following:

- Peak flood discharges at selected locations
- Critical storm durations at selected locations
- Peak flood levels at 100 m intervals along the AMTD line and well as model cross-sections
- Peak flood extent mapping
- Hydraulic structure flood immunity data

As part of the required sensitivity analysis a climate change analysis was then undertaken to determine the impacts for two planning horizons; namely 2050 and 2100. This included making allowances for increased rainfall intensity and increased mean sea level rise. This analysis was undertaken for the 100-yr ARI (1% AEP), 200-yr ARI (0.5% AEP) and 500-yr ARI (0.2% AEP) events.

The results indicate that climate change impacts within the catchment will increase the magnitude of flooding, for example:

- By the year 2050, the 100-yr ARI (1 % AEP) flood levels will be of similar magnitude to the current day 200-yr ARI (0.5 % AEP) flood levels.
- By the year 2100, the 200-yr ARI (0.5 % AEP) flood levels will be of similar magnitude to the current day 500-yr ARI (0.2 % AEP) flood levels.

The sensitivity analysis also included analyses of blockages on significant hydraulic structures. Two structures on Moolabin Creek and four structures on Rocky Water Holes Creek were blocked as per the recommendations in QUDM. Individual structures were blocked and modelled separately to ensure that the blockage impacts would not be masked by the effect of blocking other upstream structures. The most severe blockage impacts occur at Beaudesert Road on Rocky Water Holes Creek. At this location the blocked upstream flood level is considerably higher than the 2000-yr ARI (0.05 % AEP) flood level. The flooding extents of the blocked scenario are significantly greater than the un-blocked scenario resulting in substantially worse flooding to road infrastructure and properties.

Hydraulic Structure Reference Sheets (HSRS) for all major crossings within the TUFLOW model area were also prepared. The HSRS provide data for each hydraulic structure and include data relating to the structure description, location, hydraulic performance and history.

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# APPENDICES

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# Appendix A: Rainfall Distribution

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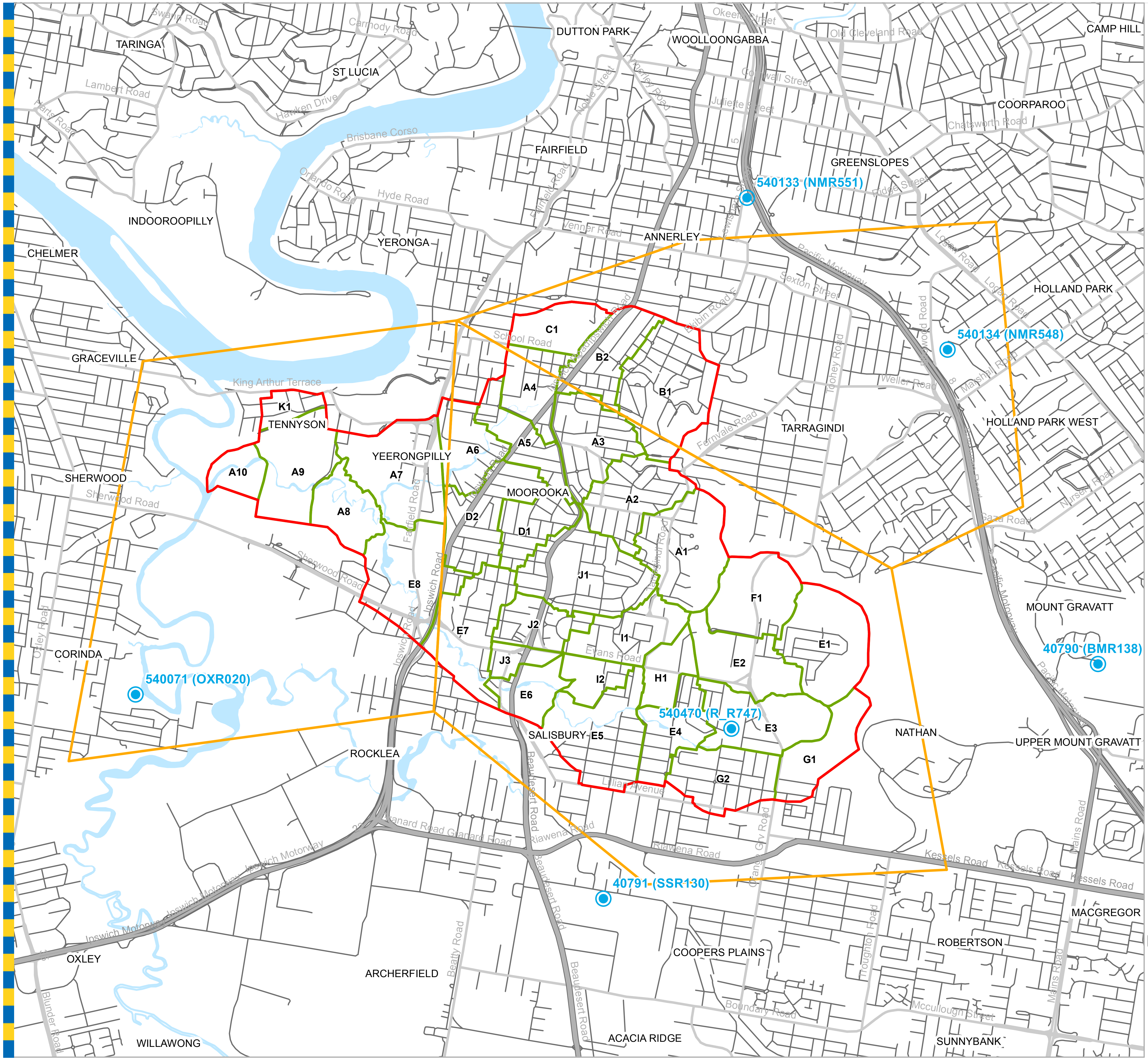

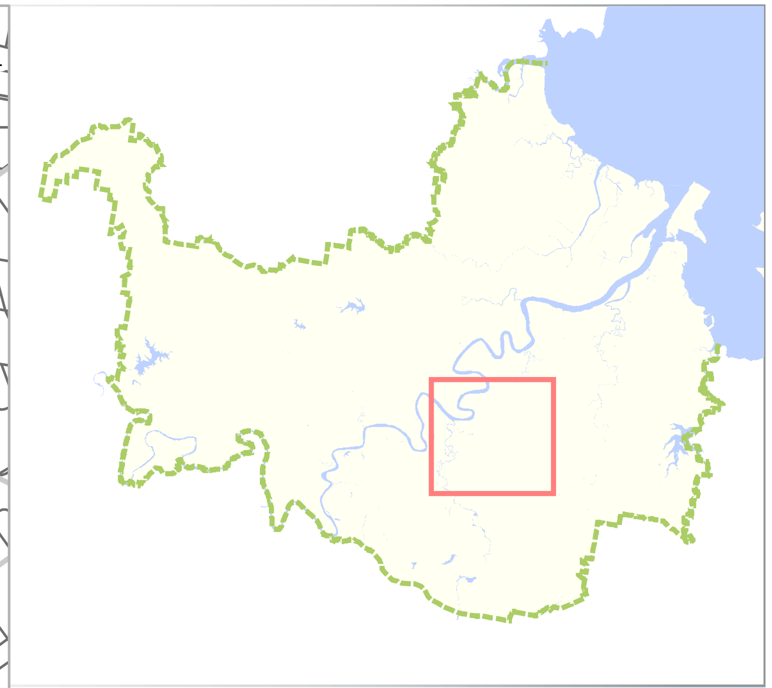

## Legend



| | |
|---|---|
| Prepared : | 089958 |
| Checked : | NC |
| Revision : | 0 |
| Publication Date : | 22 Jan 2015 |
| Project Number : | 150339 |

**DATA INFORMATION**




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or call (07) 3403 8888


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**Moolabin Creek and Rocky Water Holes Creek Flood Study**

# Figure A.1: Thiessen Polygon January 2013, February 2010 and November 2004

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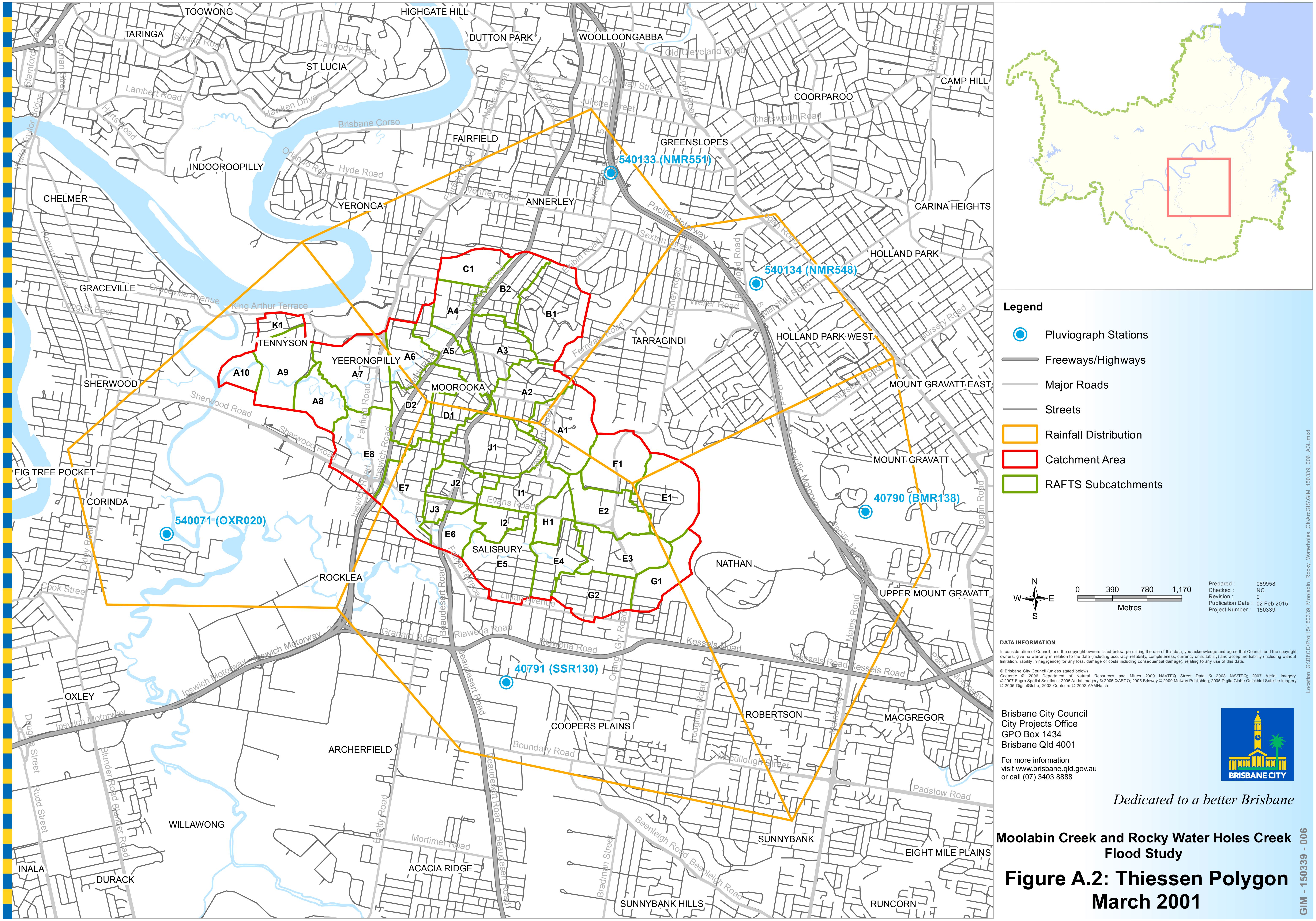


## Legend

- Pluviograph Stations
- Freeways/Highways
- Major Roads
- Streets
- Rainfall Distribution
- Catchment Area
- RAFTS Subcatchments

N / E / S / W

0 390 780 1,170

Metres

Prepared : 089958
Checked : NC
Revision : 0
Publication Date : 02 Feb 2015
Project Number : 150339

Location: G:\BICD\Proj\15\150339_Moolabin_Rocky_Waterholes_Ck\ArcGIS\GIM_150339_006_A3L.mxd

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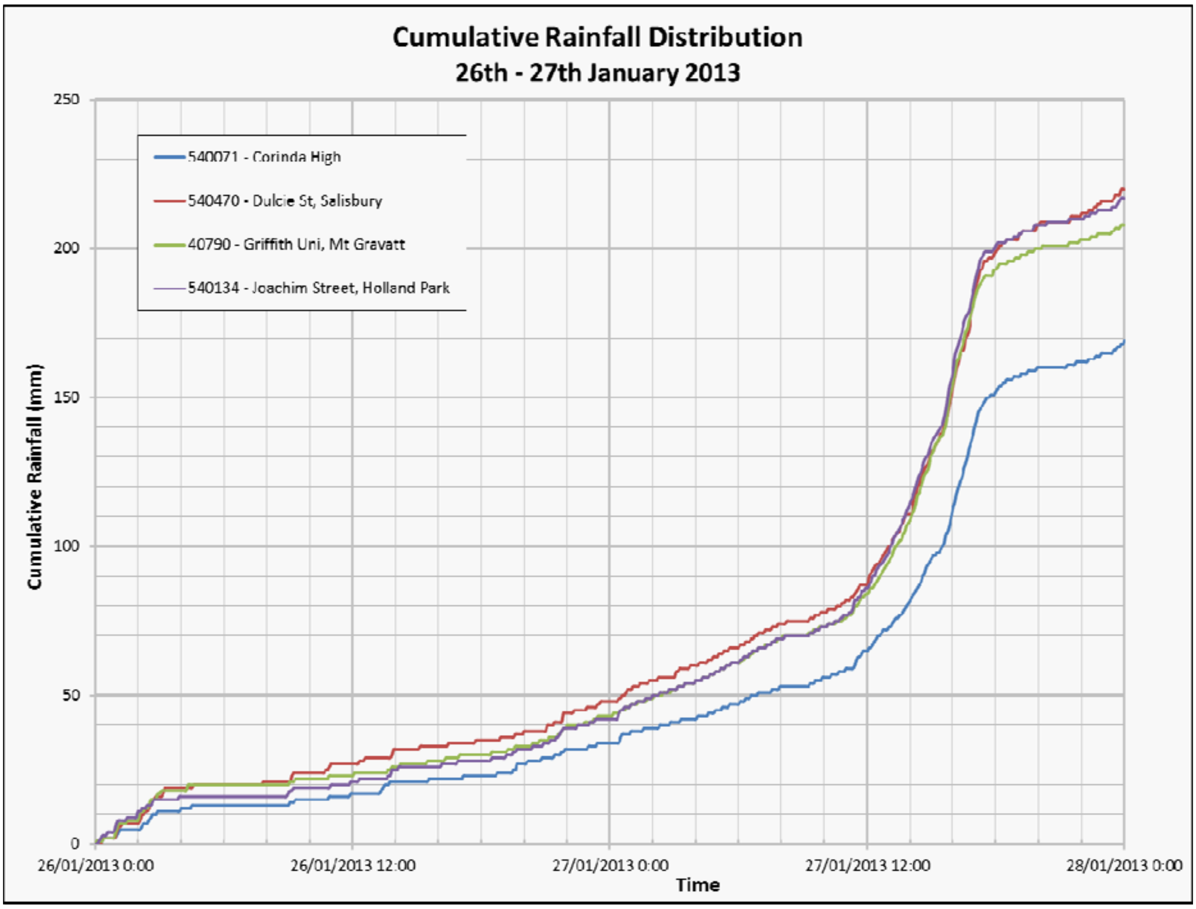
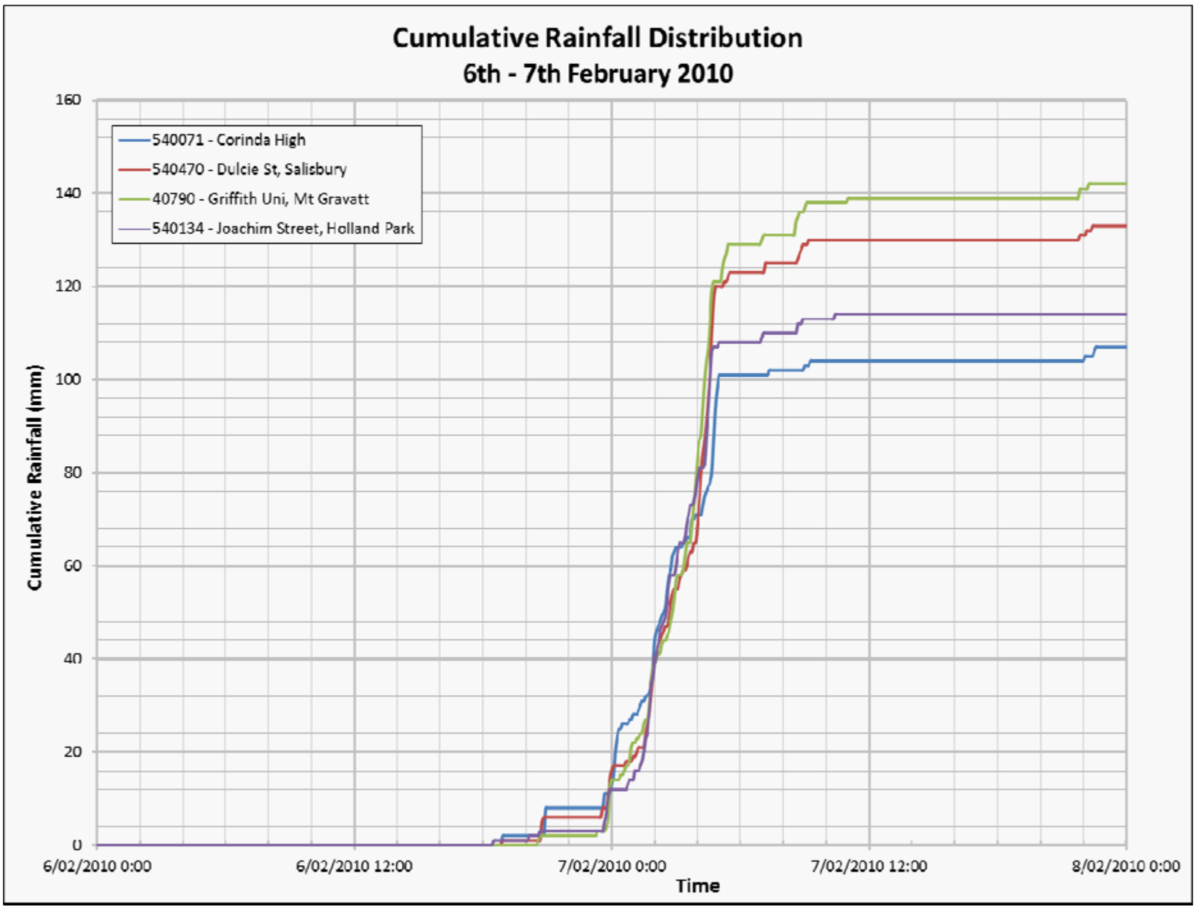
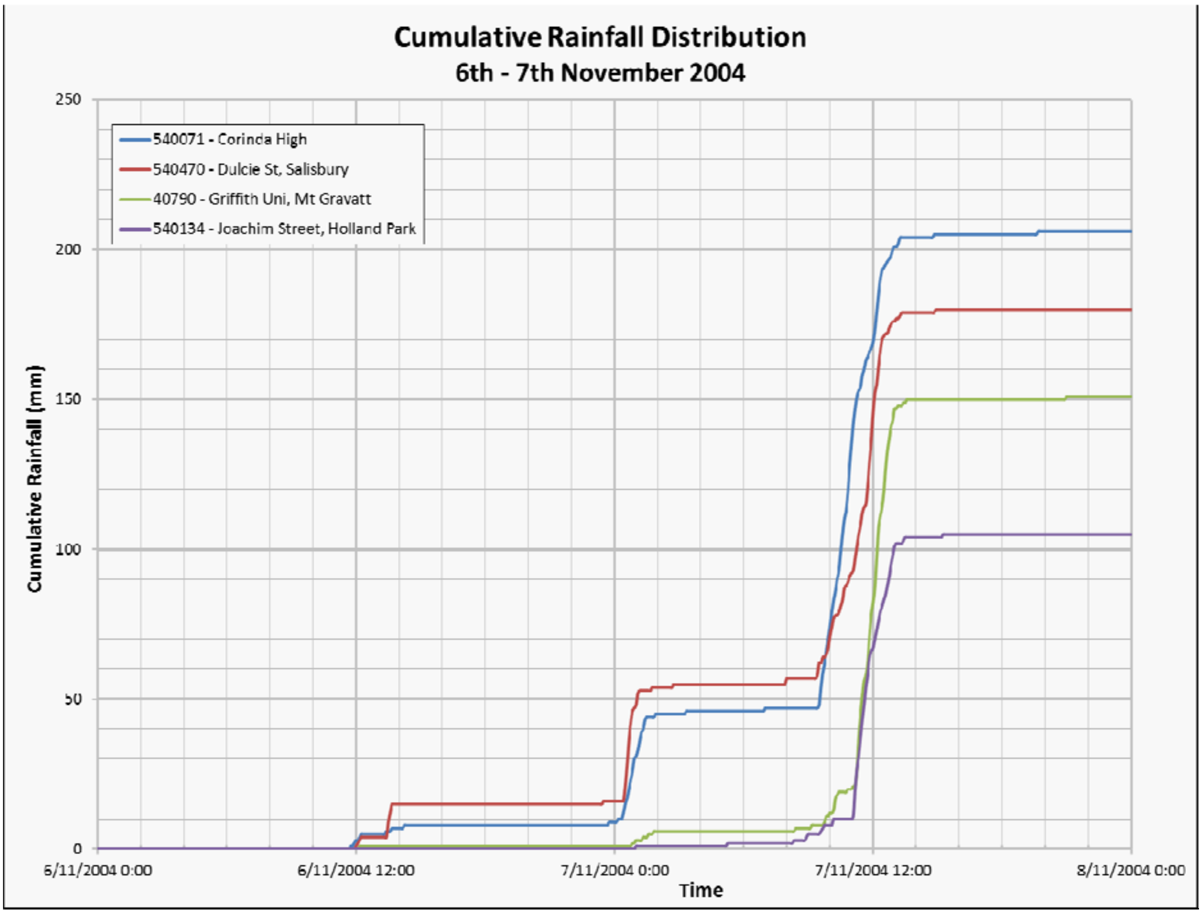
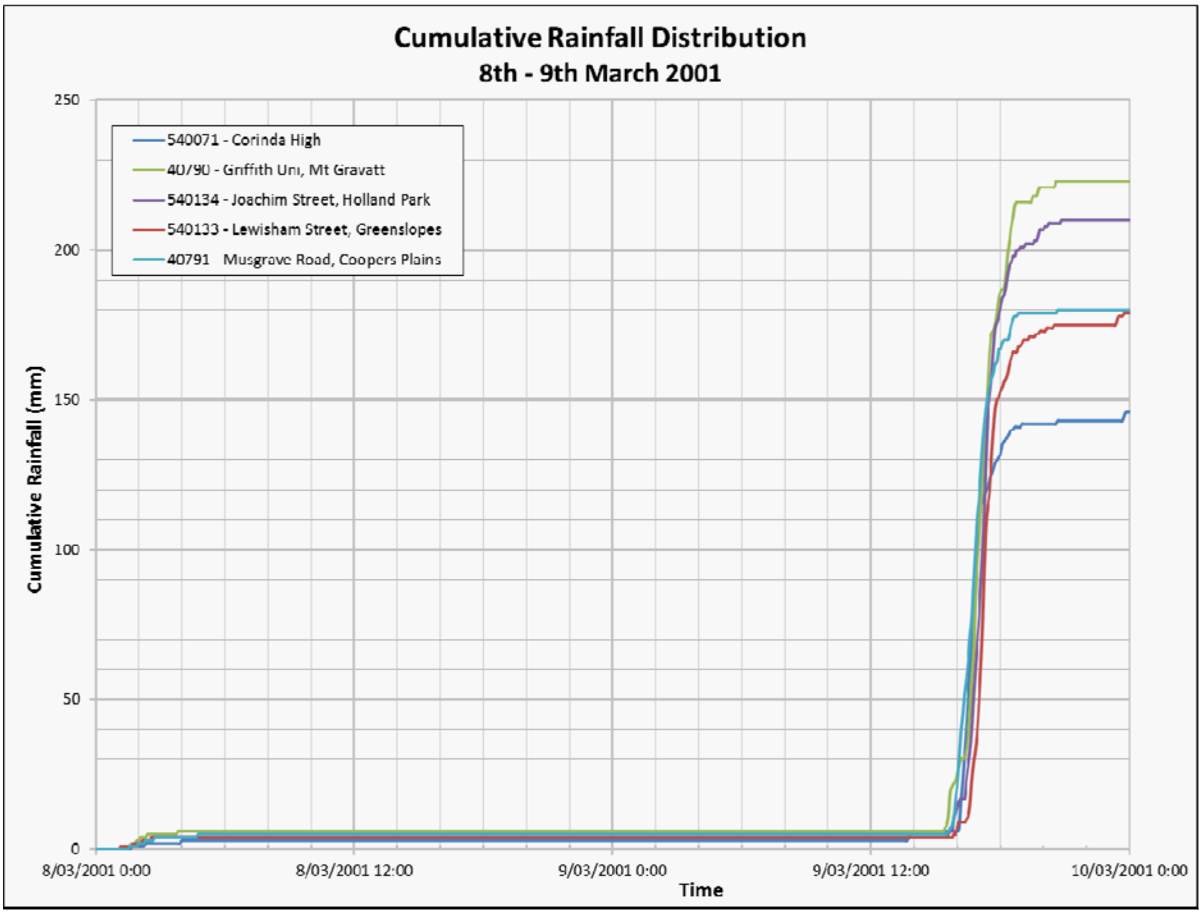
# Figure A.2: Thiessen Polygon March 2001

GIM - 150339 - 006


Cumulative Rainfall Distribution
26th - 27th January 2013
540071 - Corinda High
540470 - Dulcie St, Salisbury
40790 - Griffith Uni, Mt Gravatt
540134 - Joachim Street, Holland Park
Cumulative Rainfall (mm)
250
200
150
100
50
0
26/01/2013 0:00
26/01/2013 12:00
27/01/2013 0:00
27/01/2013 12:00
28/01/2013 0:00
Time

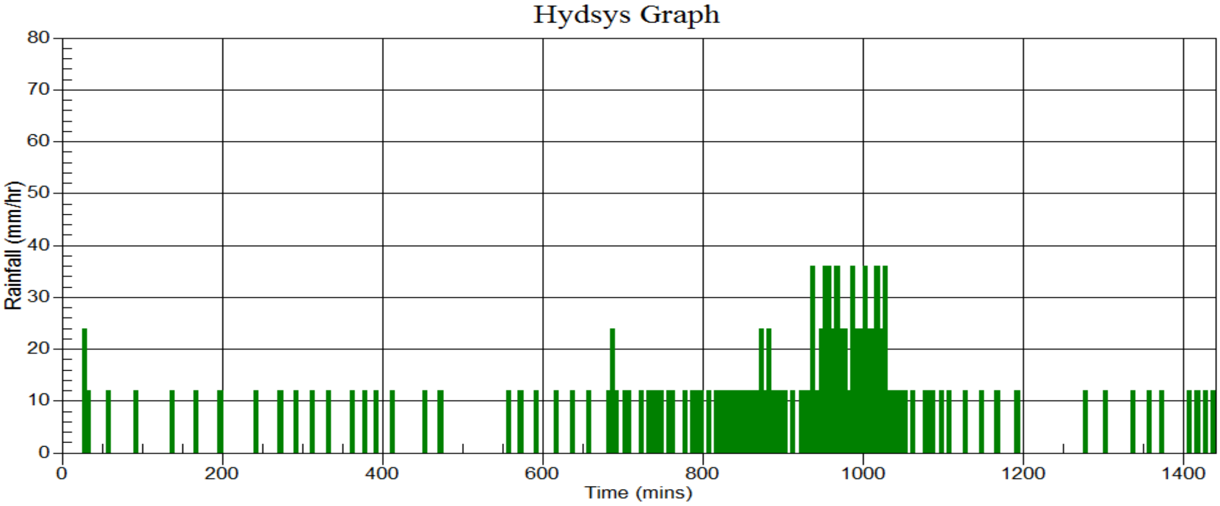

Cumulative Rainfall Distribution
6th - 7th February 2010
540071 - Corinda High
540470 - Dulcie St, Salisbury
40790 - Griffith Uni, Mt Gravatt
540134 - Joachim Street, Holland Park
Cumulative Rainfall (mm)
160
140
120
100
80
60
40
20
0
6/02/2010 0:00
6/02/2010 12:00
7/02/2010 0:00
7/02/2010 12:00
8/02/2010 0:00
Time


Cumulative Rainfall Distribution
6th - 7th November 2004
540071 - Corinda High
540470 - Dulcie St, Salisbury
40790 - Griffith Uni, Mt Gravatt
540134 - Joachim Street, Holland Park
Cumulative Rainfall (mm)
0
50
100
150
200
250
6/11/2004 0:00
6/11/2004 12:00
7/11/2004 0:00
7/11/2004 12:00
8/11/2004 0:00
Time

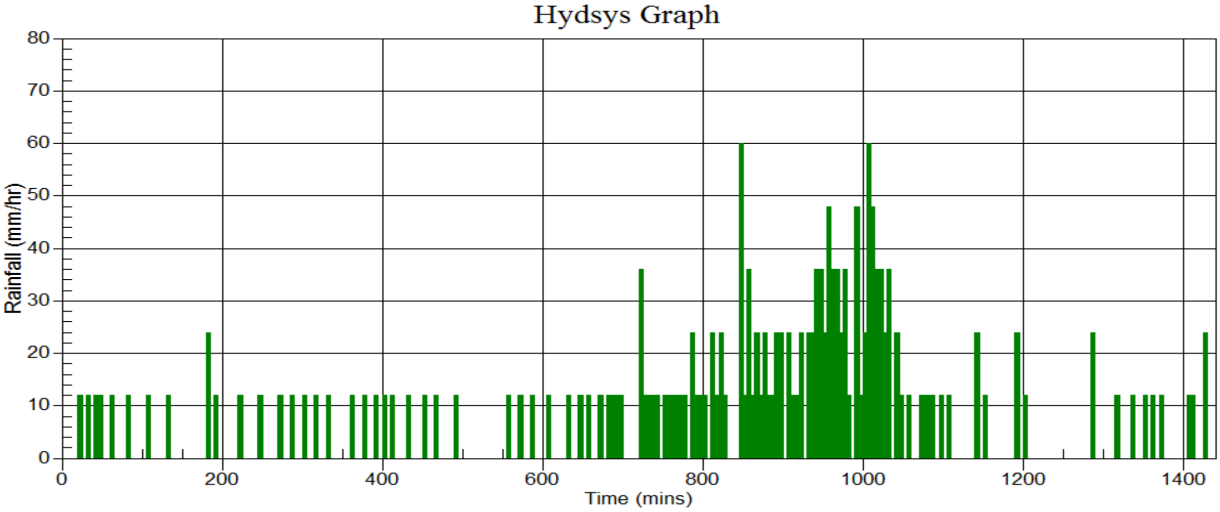

Cumulative Rainfall Distribution
8th - 9th March 2001
540071 - Corinda High
40790 - Griffith Uni, Mt Gravatt
540134 - Joachim Street, Holland Park
540133 - Lewisham Street, Greenslopes
40791 Musgrave Road, Coopers Plains
Cumulative Rainfall (mm)
0
50
100
150
200
250
8/03/2001 0:00
8/03/2001 12:00
9/03/2001 0:00
9/03/2001 12:00
10/03/2001 0:00
Time

## January 2013 Event (00:00 27/1/13 to 24:00 27/1/13)

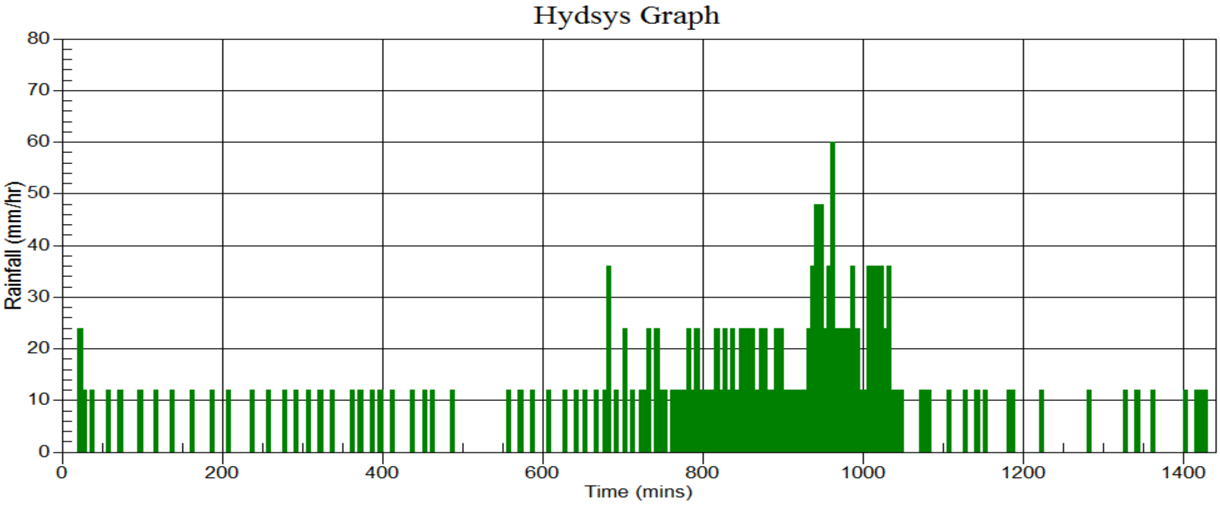
540071 - Corinda High




540470 - Dulcie Street, Salisbury

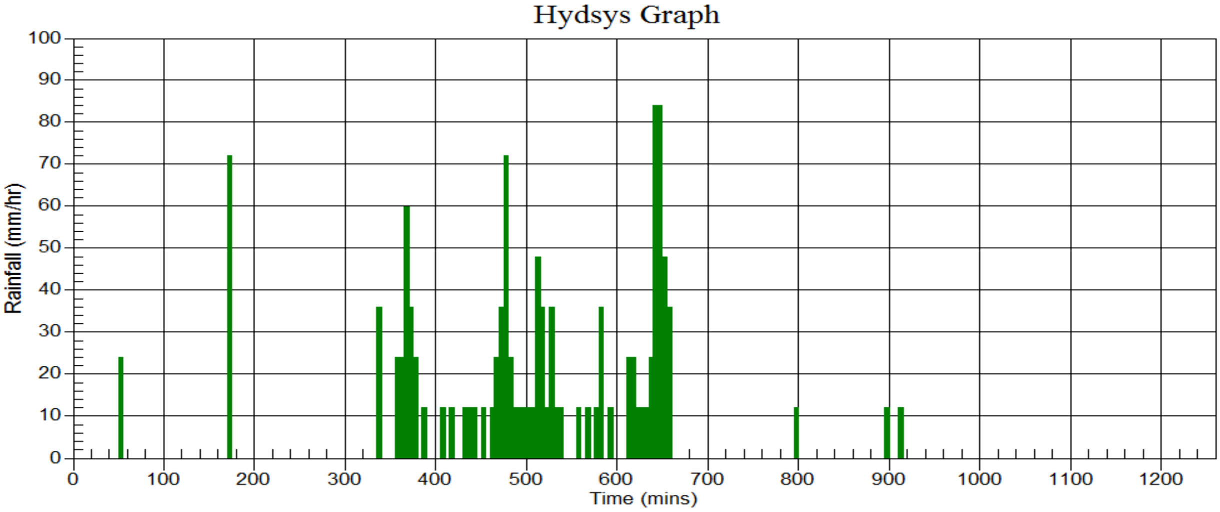



540134 - Joachim Street, Holland Park

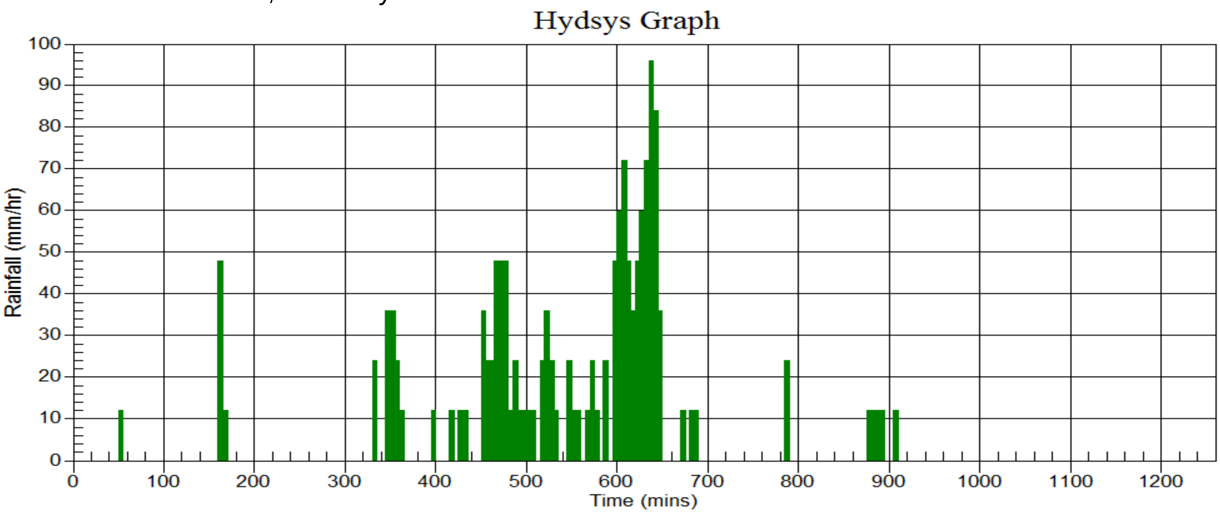


## February 2010 Event (18:00 6/2/10 to 15:00 7/2/10)

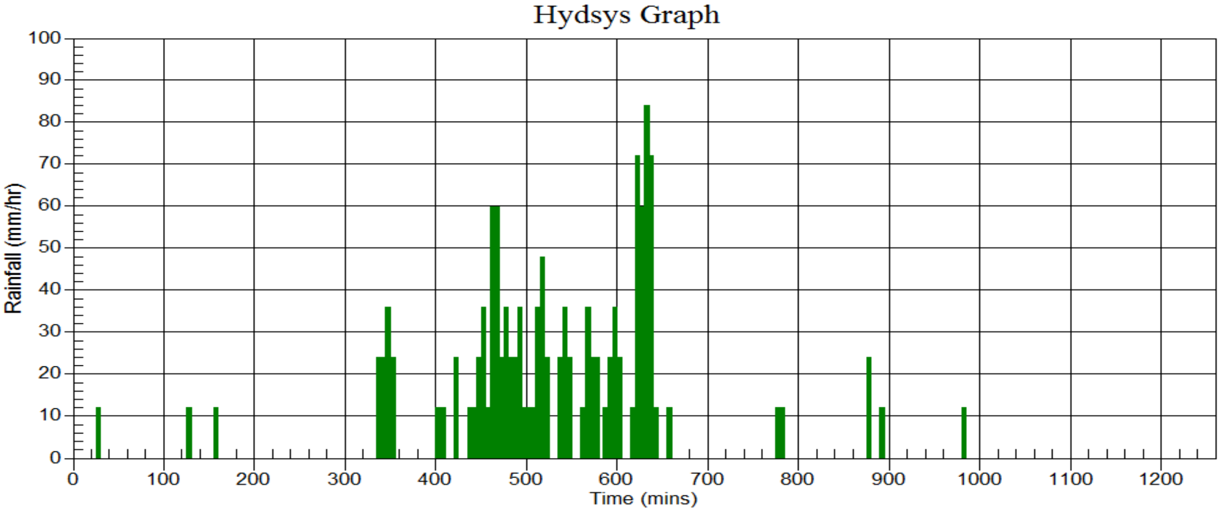
540071 - Corinda High




540470 - Dulcie Street, Salisbury

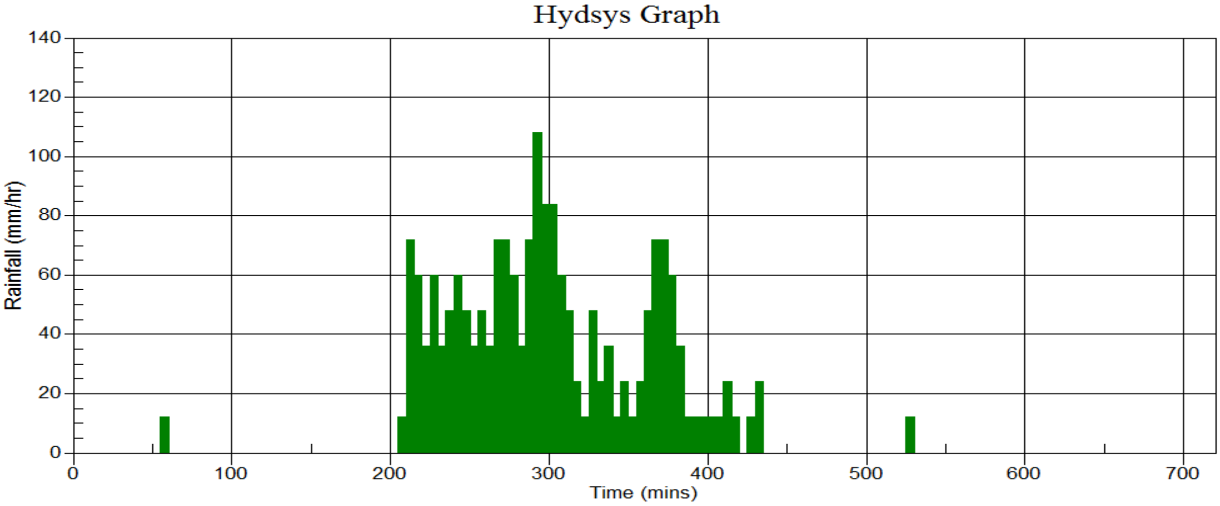



540134 - Joachim Street, Holland Park

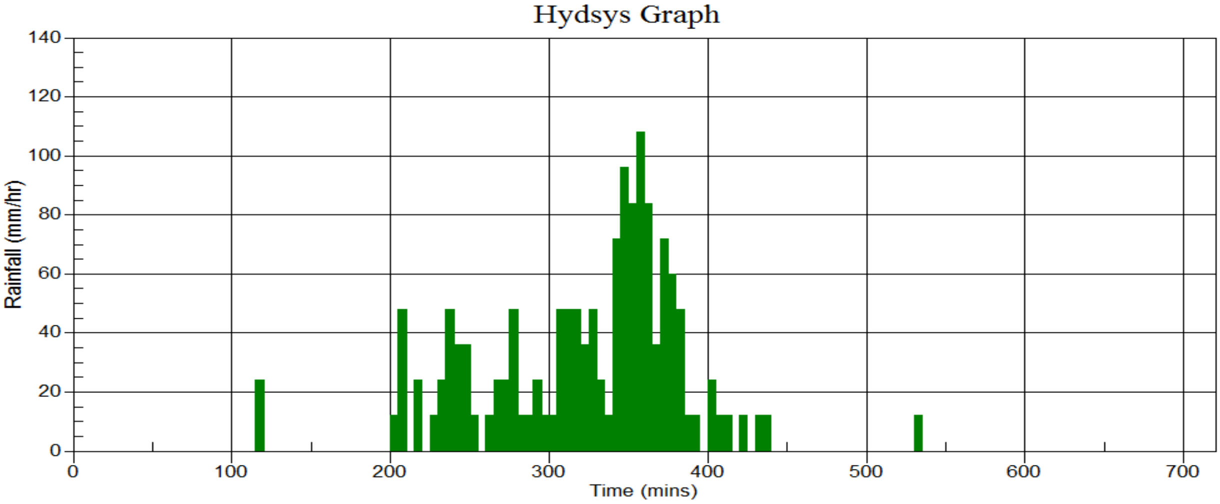


## November 2004 Event (06:00 7/11/04 to 18:00 7/11/04)

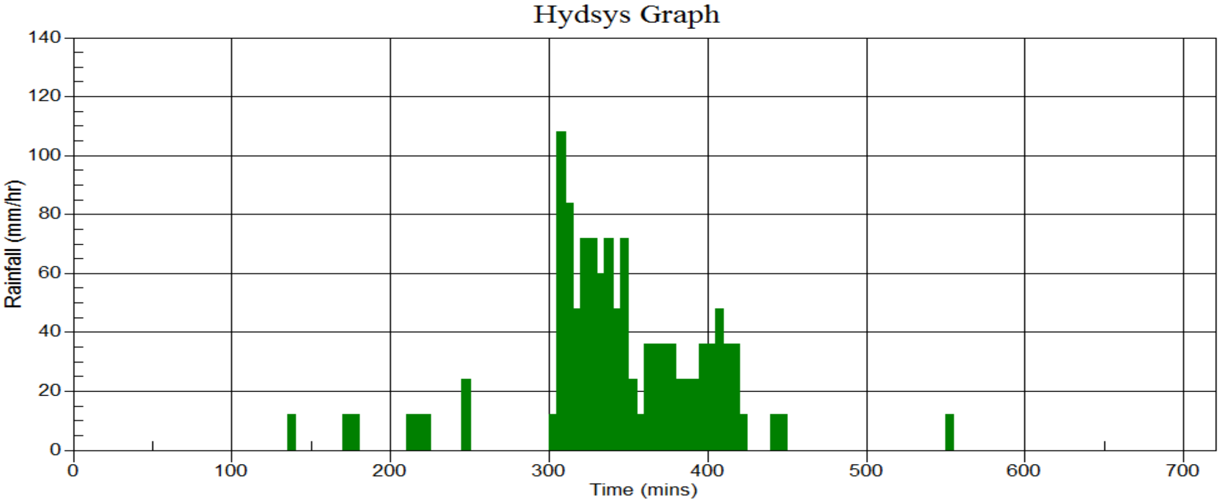
540071 - Corinda High




540470 - Dulcie Street, Salisbury

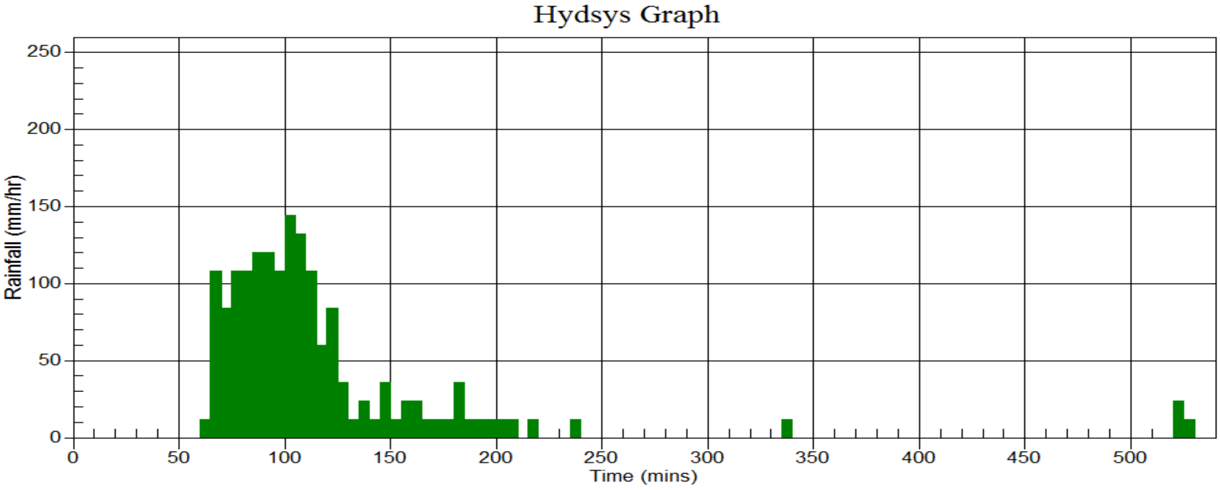



540134 - Joachim Street, Holland Park

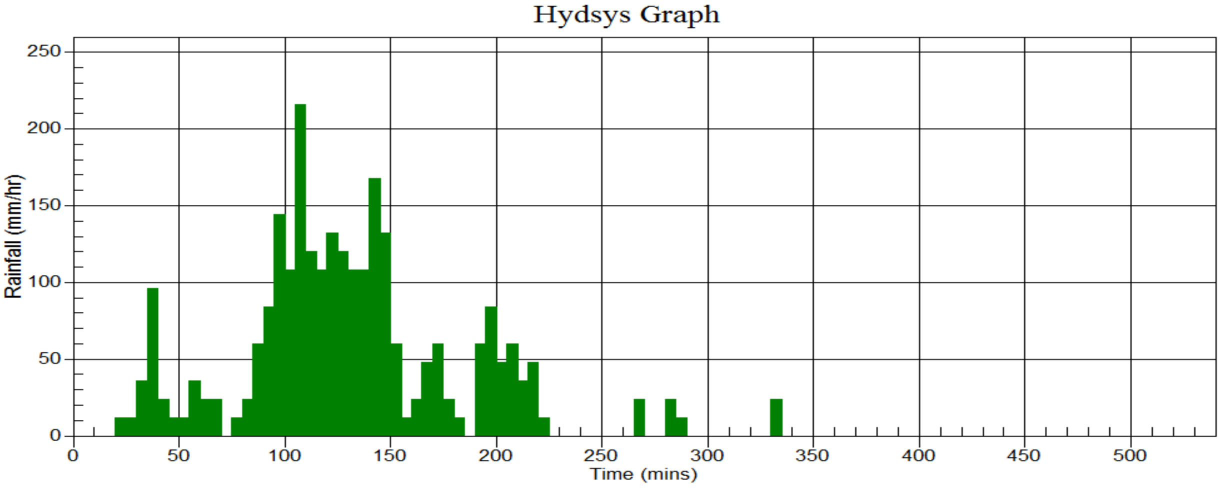


## March 2001 Event (15:00 9/3/01 to 24:00 9/3/01)

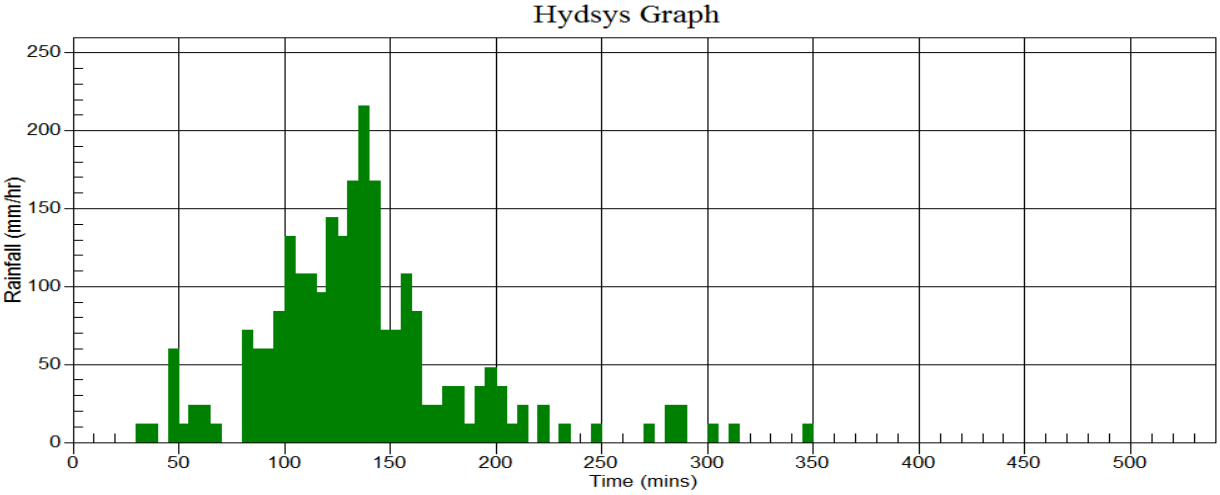
540071 - Corinda High




40790 - Griffith Uni, Mt Gravatt

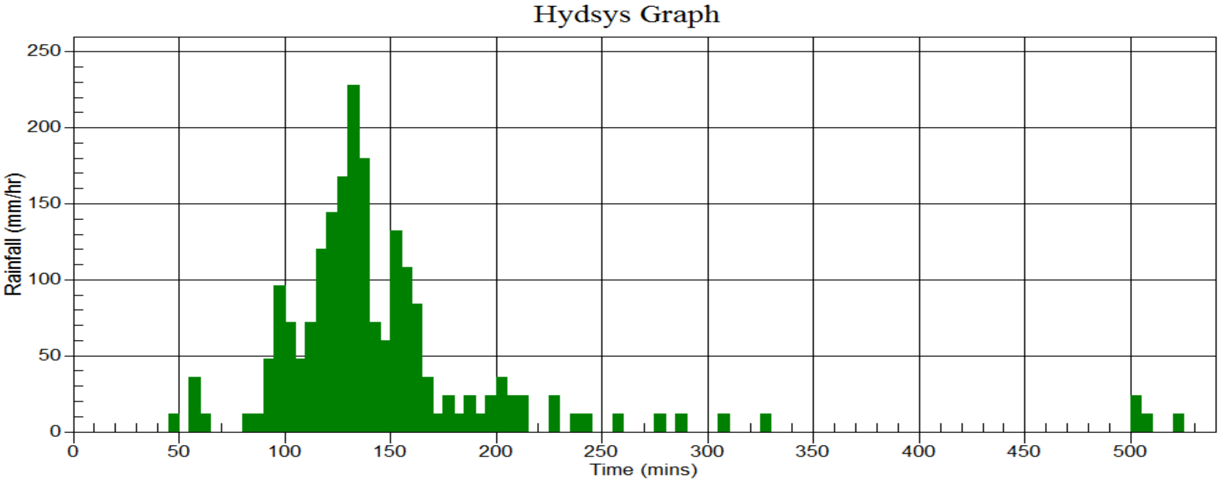



540134 - Joachim Street, Holland Park

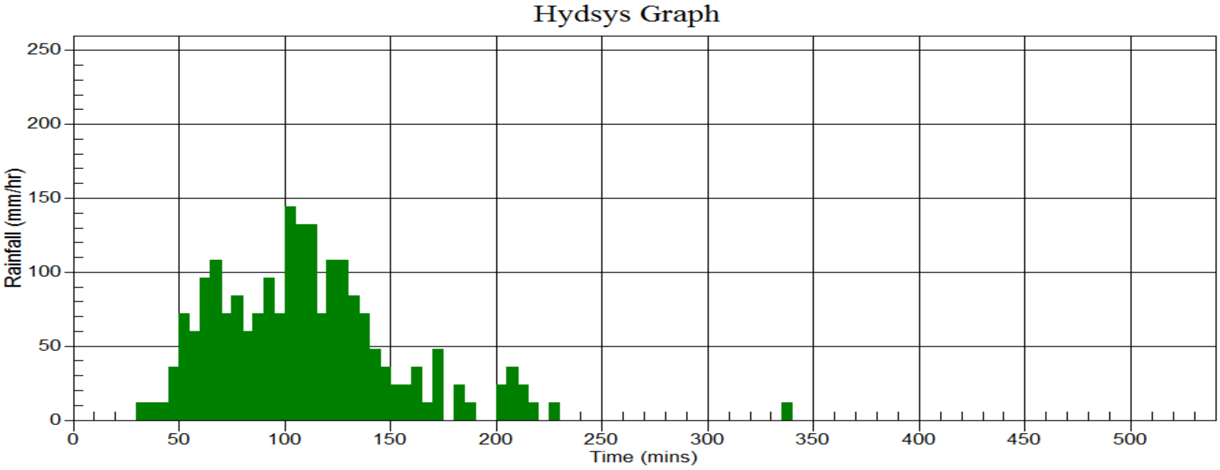


## March 2001 Event (15:00 9/3/01 to 24:00 9/3/01)

540133 - Lewisham Street, Greenslopes




40791 - Musgrave Road, Coopers Plains



# Appendix B: XP-RAFTS Sub-catchment Parameters

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| Catchment | Total Area [ha] | Percentage Impervious [%] | Catchment Mannings 'n' | Catchment Slope [%] |
|---|---|---|---|---|
| A1 [Subcatch 1] | 21.46 | 100 | 0.015 | 4.4 |
| A1 [Subcatch 2] | 31.17 | 0 | 0.06 | 4.4 |
| A2 [Subcatch 1] | 23.53 | 100 | 0.015 | 2.7 |
| A2 [Subcatch 2] | 17.36 | 0 | 0.04 | 2.7 |
| A3 [Subcatch 1] | 24.15 | 100 | 0.015 | 1.4 |
| A3 [Subcatch 2] | 15.69 | 0 | 0.04 | 1.4 |
| B1 [Subcatch 1] | 36.42 | 100 | 0.015 | 2.6 |
| B1 [Subcatch 2] | 27.18 | 0 | 0.04 | 2.6 |
| B2 [Subcatch 1] | 20.52 | 100 | 0.015 | 1.2 |
| B2 [Subcatch 2] | 13.42 | 0 | 0.04 | 1.2 |
| C1 [Subcatch 1] | 8.31 | 100 | 0.015 | 2.9 |
| C1 [Subcatch 2] | 16.02 | 0 | 0.04 | 2.9 |
| A4 [Subcatch 1] | 15.15 | 100 | 0.015 | 1 |
| A4 [Subcatch 2] | 12.97 | 0 | 0.04 | 1 |
| A5 [Subcatch 1] | 18.8 | 100 | 0.015 | 3.6 |
| A5 [Subcatch 2] | 7.81 | 0 | 0.04 | 3.6 |
| D2 [Subcatch 1] | 29.64 | 100 | 0.015 | 2.3 |
| D2 [Subcatch 2] | 12.13 | 0 | 0.04 | 2.3 |
| A7 [Subcatch 1] | 18.4 | 100 | 0.015 | 0.7 |
| A7 [Subcatch 2] | 38.2 | 0 | 0.04 | 0.7 |
| A8 [Subcatch 1] | 5.58 | 100 | 0.015 | 0.3 |
| A8 [Subcatch 2] | 20.89 | 0 | 0.04 | 0.3 |
| A6 [Subcatch 1] | 22.51 | 100 | 0.015 | 1.2 |
| A6 [Subcatch 2] | 6.69 | 0 | 0.04 | 1.2 |
| D1 [Subcatch 1] | 17.17 | 100 | 0.015 | 4 |
| D1 [Subcatch 2] | 8.76 | 0 | 0.04 | 4 |
| A9 [Subcatch 1] | 33.81 | 100 | 0.015 | 2.1 |
| A9 [Subcatch 2] | 10.65 | 0 | 0.04 | 2.1 |
| A10 [Subcatch 1] | 12.62 | 100 | 0.015 | 0.4 |
| A10 [Subcatch 2] | 2.43 | 0 | 0.04 | 0.4 |
| K1 [Subcatch 1] | 5.17 | 100 | 0.015 | 1.8 |
| K1 [Subcatch 2] | 5.87 | 0 | 0.04 | 1.8 |
| E1 [Subcatch 1] | 11.6 | 100 | 0.015 | 3.2 |
| E1 [Subcatch 2] | 41.06 | 0 | 0.08 | 3.2 |

| Catchment | Total Area [ha] | Percentage Impervious [%] | Catchment Mannings 'n' | Catchment Slope [%] |
|---|---|---|---|---|
| F1 [Subcatch 1] | 4.58 | 100 | 0.015 | 5.2 |
| F1 [Subcatch 2] | 36.36 | 0 | 0.09 | 5.2 |
| E2 [Subcatch 1] | 28.13 | 100 | 0.015 | 3.8 |
| E2 [Subcatch 2] | 9.81 | 0 | 0.04 | 3.8 |
| E3 [Subcatch 1] | 13.79 | 100 | 0.015 | 3.2 |
| E3 [Subcatch 2] | 30.74 | 0 | 0.07 | 3.2 |
| G2 [Subcatch 1] | 21.98 | 100 | 0.015 | 2.3 |
| G2 [Subcatch 2] | 16.9 | 0 | 0.04 | 2.3 |
| E4 [Subcatch 1] | 17.24 | 100 | 0.015 | 2.4 |
| E4 [Subcatch 2] | 13.38 | 0 | 0.04 | 2.4 |
| H1 [Subcatch 1] | 16.46 | 100 | 0.015 | 3.6 |
| H1 [Subcatch 2] | 2.72 | 0 | 0.04 | 3.6 |
| G1 [Subcatch 1] | 4.02 | 100 | 0.015 | 5.3 |
| G1 [Subcatch 2] | 27.09 | 0 | 0.085 | 5.3 |
| E5 [Subcatch 1] | 32.87 | 100 | 0.015 | 1.1 |
| E5 [Subcatch 2] | 20.32 | 0 | 0.04 | 1.1 |
| I2 [Subcatch 1] | 13.81 | 100 | 0.015 | 3.2 |
| I2 [Subcatch 2] | 2.34 | 0 | 0.04 | 3.2 |
| I1 [Subcatch 1] | 25.49 | 100 | 0.015 | 3.6 |
| I1 [Subcatch 2] | 8.38 | 0 | 0.04 | 3.6 |
| J1 [Subcatch 1] | 36.3 | 100 | 0.015 | 3.7 |
| J1 [Subcatch 2] | 23.03 | 0 | 0.04 | 3.7 |
| J2 [Subcatch 1] | 13.37 | 100 | 0.015 | 3.2 |
| J2 [Subcatch 2] | 11.15 | 0 | 0.04 | 3.2 |
| J3 [Subcatch 1] | 2.18 | 100 | 0.015 | 2.2 |
| J3 [Subcatch 2] | 8.19 | 0 | 0.04 | 2.2 |
| E6 [Subcatch 1] | 16.88 | 100 | 0.015 | 1.1 |
| E6 [Subcatch 2] | 4.27 | 0 | 0.04 | 1.1 |
| E7 [Subcatch 1] | 29.93 | 100 | 0.015 | 3 |
| E7 [Subcatch 2] | 17.64 | 0 | 0.04 | 3 |
| E8 [Subcatch 1] | 34.46 | 100 | 0.015 | 0.6 |
| E8 [Subcatch 2] | 19.63 | 0 | 0.04 | 0.6 |

# Appendix C: Adopted Land-use

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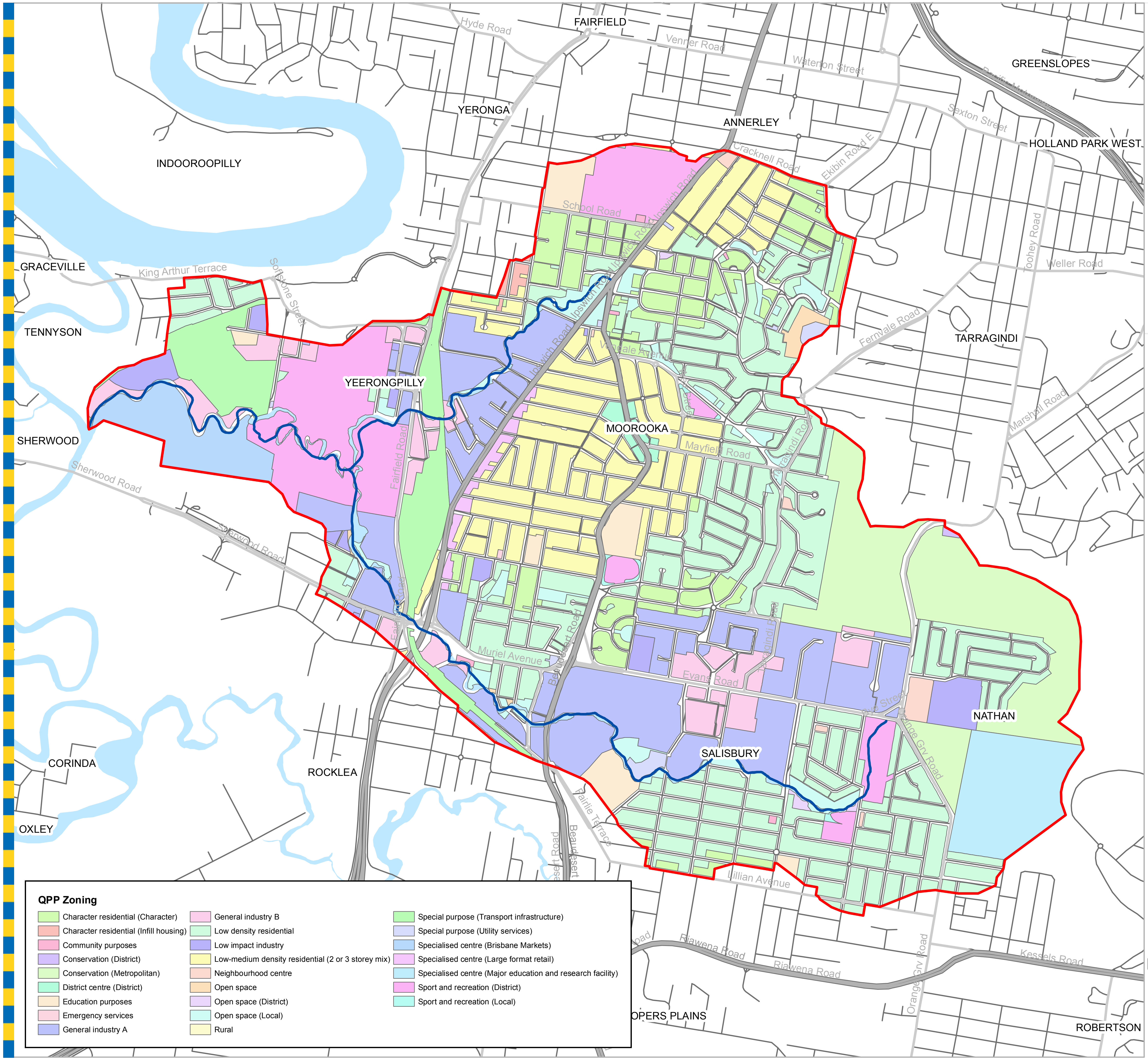


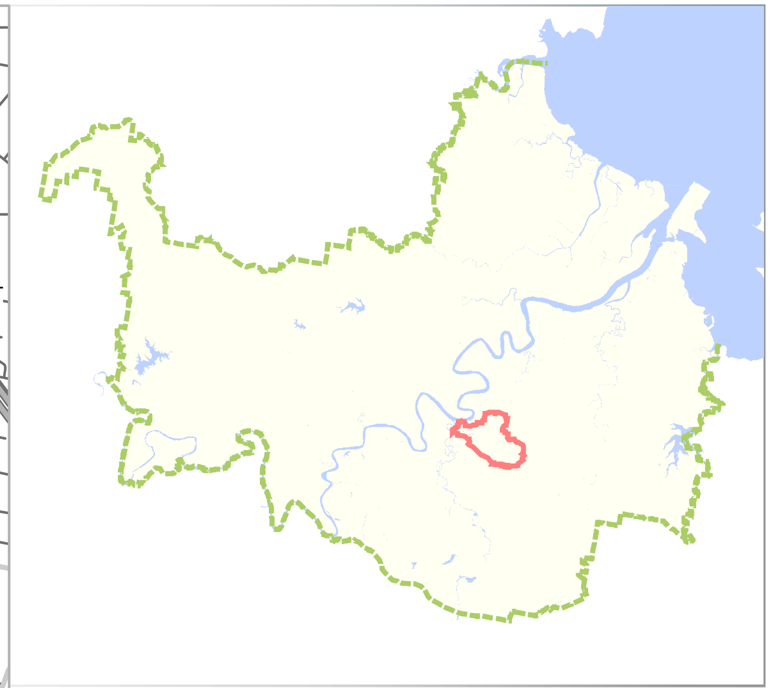

**Legend**

- Creek Centreline
- Freeways/Highways
- Major Roads
- Streets
- Catchment Area

N W E S

0 225 450 675
Metres

Prepared : 089958
Checked : NC
Revision : 0
Publication Date : 22 Jan 2015
Project Number : 150339

Location: G:\BI\CD\Proj15\150339_Moolabin_Rocky_Waterholes_Ck\ArcGIS\GIM_150339_007_A3.mxd



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BRISBANE CITY

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**Moolabin Creek and Rocky Water Holes Creek Flood Study**

# Figure C.1: Catchment Land Use

GIM - 150339 - 007


**Legend**

- Creek Centreline
- Freeways/Highways
- Major Roads
- Streets
- Catchment Area

N W E S

0 225 450 675
Metres

Prepared : 089958
Checked : NC
Revision : 0
Publication Date : 22 Jan 2015
Project Number : 150339

**DATA INFORMATION**



Brisbane City Council
City Projects Office
GPO Box 1434
Brisbane Qld 4001

For more information
visit www.brisbane.qld.gov.au
or call (07) 3403 8888

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**Moolabin Creek and Rocky Water Holes Creek Flood Study**

# Figure C.2: Catchment Aerial Image

Location: G:\BI\CD\Proj15\150339_Moolabin_Rocky_Waterholes_Ck\ArcGIS\GIM_150339_008_A3.mxd

GIM - 150339 - 008

| Land-use Type | % Impervious |
|---|---|
| Low density residential | 60 |
| Character residential (Character) | 70 |
| Character residential (Infill housing) | 70 |
| Low-medium density residential (2 storey mix) | 70 |
| Low-medium density residential (2 or 3 storey mix) | 70 |
| Low-medium density residential (Up to 3 storeys) | 70 |
| Medium density residential | 80 |
| High density residential (Up to 8 storeys) | 90 |
| High density residential (Up to 15 storeys) | 90 |
| Tourist accommodation | 80 |
| Neighbourhood centre | 90 |
| District centre (District) | 90 |
| District centre (Corridor) | 90 |
| Major centre | 90 |
| Principal centre (City centre) | 90 |
| Principal centre (Regional centre) | 90 |
| Low impact industry | 90 |
| Industry (General industry A) | 90 |
| Industry (General industry B) | 90 |
| Industry (General industry C) | 90 |
| Special industry | 90 |
| Industry investigation | 90 |
| Sport and recreation | 20 |
| Sport and recreation (Local) | 20 |
| Sport and recreation (District) | 20 |
| Sport and recreation (Metropolitan) | 20 |
| Open space | 5 |
| Open space (Local) | 5 |
| Open space (District) | 5 |
| Open space (Metropolitan) | 5 |
| Environmental management | 5 |
| Conservation | 0 |
| Conservation (Local) | 0 |
| Conservation (District) | 0 |
| Conservation (Metropolitan) | 0 |

| Land-use Type | % Impervious |
|---|---|
| Emerging community | 70 |
| Extractive industry | 90 |
| Mixed use (Inner city) | 90 |
| Mixed use (Centre frame) | 90 |
| Mixed use (Corridor) | 90 |
| Rural | 20 |
| Rural residential | 30 |
| Township | 80 |
| Community facilities (Major health care) | 70 |
| Community facilities (Major sports venue) | 60 |
| Community facilities (Cemetery) | 40 |
| Community facilities (Community purposes) | 70 |
| Community facilities (Education purposes) | 70 |
| Community facilities (Emergency services) | 70 |
| Community facilities (Health care purposes) | 70 |
| Specialised centre (Major education and research facility) | 90 |
| Specialised centre (Entertainment and conference centre) | 90 |
| Specialised centre (Brisbane Markets) | 90 |
| Specialised centre (Large format retail) | 90 |
| Specialised centre (Mixed industry and business) | 90 |
| Specialised centre (Marina) | 80 |
| Special purpose (Defence) | 80 |
| Special purpose (Detention facility) | 80 |
| Special purpose (Transport infrastructure) | 75 |
| Special purpose (Utility services) | 75 |
| Special purpose (Airport) | 60 |
| Special purpose (Port) | 60 |

# Appendix D: Design Events (Scenario 1) - Peak Flood Levels

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| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| **Moolabin Creek** | | | | | | | |
| ML 0 | - | 1.42 | 1.55 | 1.67 | 1.86 | 2.14 | 2.34 |
| ML 100 | - | 1.82 | 2.10 | 2.22 | 2.43 | 2.63 | 2.82 |
| ML 200 | - | 2.05 | 2.37 | 2.51 | 2.75 | 2.95 | 3.17 |
| ML 300 | - | 2.33 | 2.63 | 2.75 | 2.96 | 3.18 | 3.36 |
| ML 400 | - | 2.66 | 2.94 | 3.07 | 3.22 | 3.39 | 3.54 |
| ML 500 | - | 2.80 | 3.05 | 3.17 | 3.32 | 3.49 | 3.64 |
| ML 600 | - | 2.93 | 3.17 | 3.28 | 3.43 | 3.59 | 3.74 |
| ML 700 | - | 2.98 | 3.22 | 3.32 | 3.46 | 3.61 | 3.75 |
| ML 800 | - | 3.15 | 3.41 | 3.53 | 3.69 | 3.86 | 4.01 |
| ML 900 | - | 3.20 | 3.47 | 3.59 | 3.74 | 3.91 | 4.06 |
| ML 1000 | - | 3.24 | 3.50 | 3.62 | 3.77 | 3.94 | 4.09 |
| ML 1100 | - | 3.37 | 3.64 | 3.76 | 3.91 | 4.07 | 4.21 |
| ML 1200 | - | 3.49 | 3.78 | 3.90 | 4.06 | 4.22 | 4.36 |
| ML 1260 | - | 3.53 | 3.79 | 3.89 | 4.02 | 4.15 | 4.25 |
| Curzon Street | | | | | | | |
| ML 1276 | - | 3.54 | 3.90 | 3.95 | 4.15 | 4.33 | 4.48 |
| ML 1300 | - | 3.68 | 4.01 | 4.16 | 4.38 | 4.64 | 4.83 |
| ML 1400 | - | note (1) | 4.02 | 4.17 | 4.40 | 4.67 | 4.86 |
| ML 1500 | - | 3.74 | 4.05 | 4.20 | 4.41 | 4.68 | 4.87 |
| ML 1600 | - | 3.76 | 4.06 | 4.21 | 4.42 | 4.68 | 4.87 |
| ML 1700 | - | 3.77 | 4.07 | 4.21 | 4.43 | 4.69 | 4.88 |
| ML 1800 | - | 3.77 | 4.07 | 4.21 | 4.42 | 4.69 | 4.88 |
| ML 1900 | - | 3.79 | 4.09 | 4.23 | 4.44 | 4.70 | 4.89 |
| ML 2000 | - | 3.83 | 4.12 | 4.25 | 4.45 | 4.71 | 4.90 |
| ML 2100 | - | 3.84 | 4.13 | 4.26 | 4.46 | 4.72 | 4.90 |
| ML 2200 | - | 3.84 | 4.13 | 4.26 | 4.46 | 4.73 | 4.91 |
| ML 2300 | - | 3.88 | 4.17 | 4.30 | 4.51 | 4.75 | 4.94 |
| ML 2400 | - | 3.96 | 4.17 | 4.31 | 4.51 | 4.76 | 4.93 |
| ML 2500 | - | 4.59 | 4.69 | 4.75 | 4.83 | 4.91 | 5.01 |
| ML 2600 | - | 5.61 | 5.80 | 5.84 | 5.88 | 6.03 | 6.39 |
| ML 2607 | - | 5.64 | 5.83 | 5.87 | 5.93 | 6.09 | 6.43 |
| Fairfield Road | | | | | | | |

| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| ML 2638 | XS2895_ML170 | 5.97 | 6.31 | 6.45 | 6.71 | 6.96 | 7.18 |
| ML 2651 | XS2907_ML179 | 5.97 | 6.31 | 6.45 | 6.71 | 6.96 | 7.18 |
| Chale Street | | | | | | | |
| ML 2669 | XS2925_ML | 6.00 | 6.33 | 6.48 | 6.73 | 6.98 | 7.20 |
| ML 2700 | - | 6.02 | 6.35 | 6.49 | 6.74 | 6.99 | 7.20 |
| ML 2741 | XS2999_ML200 | 6.03 | 6.36 | 6.51 | 6.76 | 7.00 | 7.22 |
| ML 2766 | XS3024_ML210 | 6.06 | 6.38 | 6.53 | 6.78 | 7.02 | 7.23 |
| Railway Bridges | | | | | | | |
| ML 2798 | XS3056_ML230 | 6.09 | 6.42 | 6.56 | 6.81 | 7.11 | 7.34 |
| ML 2800 | - | 6.09 | 6.42 | 6.56 | 6.82 | 7.12 | 7.34 |
| ML 2838 | XS3093_ML231 | 6.13 | 6.46 | 6.61 | 6.85 | 7.15 | 7.38 |
| Evesham Street | | | | | | | |
| ML 2855 | XS3109_ML233 | 6.14 | 6.48 | 6.63 | 6.88 | 7.18 | 7.41 |
| ML 2881 | XS3125_ML240 | 6.16 | 6.49 | 6.64 | 6.88 | 7.17 | 7.40 |
| ML 2900 | - | 6.25 | 6.56 | 6.71 | 6.94 | 7.22 | 7.44 |
| ML 3000 | - | 6.78 | 7.02 | 7.15 | 7.31 | 7.50 | 7.67 |
| ML 3015 | XS3249_ML250 | 6.87 | 7.10 | 7.22 | 7.38 | 7.55 | 7.71 |
| ML 3100 | - | 7.76 | 7.99 | 8.10 | 8.23 | 8.35 | 8.47 |
| ML 3101 | XS3337_ML260 | 7.78 | 8.00 | 8.11 | 8.24 | 8.36 | 8.48 |
| ML 3200 | - | 8.36 | 8.59 | 8.70 | 8.83 | 8.94 | 9.05 |
| ML 3258 | XS3498_ML270 | 8.72 | 8.94 | 9.06 | 9.19 | 9.29 | 9.39 |
| ML 3300 | - | 8.80 | 9.03 | 9.15 | 9.29 | 9.39 | 9.50 |
| ML 3353 | XS3594_ML280 | 8.90 | 9.14 | 9.27 | 9.42 | 9.52 | 9.64 |
| Lucy Street | | | | | | | |
| ML 3376 | XS3617_ML | 9.18 | 9.39 | 9.49 | 9.60 | 9.69 | 9.78 |
| ML 3400 | - | 9.22 | 9.43 | 9.53 | 9.65 | 9.73 | 9.82 |
| ML 3500 | - | 9.40 | 9.62 | 9.72 | 9.84 | 9.93 | 10.02 |
| ML 3523 | XS3767_ML | 9.44 | 9.66 | 9.76 | 9.88 | 9.97 | 10.07 |
| ML 3600 | - | 10.20 | 10.42 | 10.53 | 10.67 | 10.77 | 10.88 |
| ML 3646 | XS3896_ML320 | 10.72 | 10.93 | 11.06 | 11.20 | 11.31 | 11.43 |
| ML 3700 | - | 11.11 | 11.34 | 11.47 | 11.63 | 11.74 | 11.88 |
| ML 3794 | XS4052_ML330 | 11.44 | 11.69 | 11.83 | 11.99 | 12.12 | 12.26 |
| ML 3800 | - | 11.44 | 11.71 | 11.85 | 12.00 | 12.14 | 12.37 |
| Gow Street | | | | | | | |

| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| ML 3818 | XS4076_ML350 | 11.99 | 12.44 | 12.60 | 12.74 | 12.85 | 12.97 |
| ML 3851 | XS4106_ML351 | 12.06 | 12.50 | 12.66 | 12.80 | 12.92 | 13.05 |
| ML 3900 | - | 12.21 | 12.62 | 12.77 | 12.92 | 13.05 | 13.19 |
| ML 4000 | - | 12.67 | 13.00 | 13.13 | 13.28 | 13.41 | 13.58 |
| ML 4062 | XS4381_ML370 | 12.86 | 13.15 | 13.28 | 13.42 | 13.54 | 13.72 |
| ML 4100 | - | 13.19 | 13.43 | 13.55 | 13.68 | 13.80 | 13.98 |
| ML 4140 | XS4461_ML370 | 13.51 | 13.71 | 13.82 | 13.94 | 14.05 | 14.24 |
| ML 4200 | - | 14.00 | 14.16 | 14.27 | 14.39 | 14.50 | 14.66 |
| ML 4277 | XS4592_ML380 | 14.62 | 14.76 | 14.84 | 14.96 | 15.07 | 15.17 |
| **Rocky Water Holes Creek** | | | | | | | |
| R 100 | - | 3.84 | 4.12 | 4.26 | 4.46 | 4.72 | 4.90 |
| R 200 | - | 3.85 | 4.13 | 4.26 | 4.46 | 4.72 | 4.91 |
| R 237 | XS214_R29 | 3.85 | 4.13 | 4.26 | 4.46 | 4.72 | 4.90 |
| R 283 | XS259_R30 | 3.86 | 4.13 | 4.26 | 4.46 | 4.72 | 4.91 |
| R 300 | - | 3.86 | 4.13 | 4.26 | 4.46 | 4.72 | 4.91 |
| R 400 | - | 3.87 | 4.14 | 4.27 | 4.47 | 4.73 | 4.91 |
| R 500 | - | 3.91 | 4.16 | 4.29 | 4.49 | 4.74 | 4.93 |
| R 600 | - | 3.95 | 4.18 | 4.31 | 4.50 | 4.76 | 4.94 |
| R 700 | - | 4.03 | 4.23 | 4.34 | 4.52 | 4.77 | 4.95 |
| R 755 | XS725_R60 | 4.07 | 4.27 | 4.38 | 4.53 | 4.78 | 4.96 |
| R 800 | - | 4.20 | 4.41 | 4.52 | 4.65 | 4.86 | 5.02 |
| R 947 | XS904_R80 | 4.57 | 4.79 | 4.89 | 5.03 | 5.18 | 5.31 |
| Fairfield Road | | | | | | | |
| R 977 | XS934_R100 | 4.58 | 4.80 | 4.90 | 5.04 | 5.20 | 5.33 |
| R 986 | XS943_R | 4.58 | 4.84 | 4.95 | 5.09 | 5.26 | 5.39 |
| Muriel Avenue Footbridge | | | | | | | |
| R 989 | XS946_R | 4.82 | 5.12 | 5.27 | 5.48 | 5.75 | 5.96 |
| R 997 | XS957_R115 | 4.82 | 5.17 | 5.34 | 5.56 | 5.85 | 6.06 |
| R 1000 | - | 4.83 | 5.18 | 5.34 | 5.56 | 5.85 | 6.07 |
| Railway Bridges | | | | | | | |
| R 1020 | XS976_R | 4.85 | 5.20 | 5.36 | 5.58 | 5.87 | 6.09 |
| R 1025 | XS988_R120 | 4.85 | 5.20 | 5.36 | 5.58 | 5.87 | 6.09 |
| Muriel Avenue | | | | | | | |
| R 1075 | XS1036_R140 | 4.88 | 5.21 | 5.37 | 5.60 | 5.88 | 6.10 |

| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| Ipswich Road | | | | | | | |
| R 1142 | XS1104_R150 | 5.03 | 5.46 | 5.71 | 6.02 | 6.39 | 6.66 |
| R 1200 | - | 5.08 | 5.51 | 5.75 | 6.05 | 6.41 | 6.68 |
| R 1210 | XS1174_R170 | 5.09 | 5.52 | 5.76 | 6.06 | 6.42 | 6.68 |
| R 1300 | - | 5.38 | 5.76 | 5.96 | 6.22 | 6.52 | 6.76 |
| R 1390 | XS1359_R180 | 5.90 | 6.18 | 6.31 | 6.51 | 6.73 | 6.93 |
| R 1400 | - | 5.92 | 6.20 | 6.33 | 6.51 | 6.74 | 6.93 |
| R 1500 | - | 6.37 | 6.59 | 6.69 | 6.82 | 6.97 | 7.11 |
| R 1526 | XS1508_R190 | 6.60 | 6.81 | 6.90 | 7.01 | 7.13 | 7.24 |
| R 1600 | - | 6.87 | 7.08 | 7.16 | 7.27 | 7.38 | 7.47 |
| R 1632 | XS1614_R200 | 6.99 | 7.19 | 7.27 | 7.38 | 7.49 | 7.57 |
| R 1679 | XS1664_R202 | 7.14 | 7.34 | 7.43 | 7.54 | 7.65 | 7.75 |
| R 1700 | - | 7.18 | 7.37 | 7.46 | 7.57 | 7.68 | 7.78 |
| R 1800 | - | 7.37 | 7.53 | 7.60 | 7.70 | 7.82 | 7.91 |
| R 1818 | XS1800_R210 | 7.41 | 7.56 | 7.63 | 7.73 | 7.84 | 7.94 |
| R 1865 | XS1848_R215 | 7.57 | 7.73 | 7.79 | 7.88 | 7.99 | 8.08 |
| R 1900 | - | 7.66 | 7.85 | 7.92 | 8.03 | 8.16 | 8.27 |
| R 1918 | XS1899_R220 | 7.70 | 7.91 | 7.98 | 8.10 | 8.25 | 8.37 |
| Gladstone Street | | | | | | | |
| R 1954 | XS1934_R240 | 7.71 | 7.95 | 8.04 | 8.19 | 8.41 | 8.56 |
| R 1989 | XS1969_R250 | 7.77 | 8.02 | 8.11 | 8.27 | 8.49 | 8.65 |
| R 2000 | - | 7.79 | 8.04 | 8.14 | 8.30 | 8.52 | 8.68 |
| R 2026 | XS2004_R260 | 7.84 | 8.10 | 8.20 | 8.36 | 8.59 | 8.75 |
| Beaudesert Road | | | | | | | |
| R 2078 | XS2058_R281 | 7.86 | 8.13 | 8.27 | 8.48 | 8.79 | 9.05 |
| R 2100 | - | 7.92 | 8.19 | 8.33 | 8.54 | 8.85 | 9.11 |
| R 2181 | XS2159_R290 | 8.15 | 8.44 | 8.58 | 8.80 | 9.11 | 9.36 |
| R 2200 | - | 8.32 | 8.59 | 8.73 | 8.95 | 9.25 | 9.49 |
| R 2300 | - | 9.12 | 9.37 | 9.49 | 9.70 | 9.95 | 10.15 |
| R 2301 | XS2280_R298 | 9.12 | 9.37 | 9.50 | 9.70 | 9.95 | 10.16 |
| Footbridge (TAFE) | | | | | | | |
| R 2308 | XS2287_R299 | 9.14 | 9.39 | 9.51 | 9.72 | 9.97 | 10.18 |
| R 2400 | - | 9.94 | 10.19 | 10.31 | 10.51 | 10.75 | 10.94 |
| R 2407 | XS2388_R320 | 10.00 | 10.25 | 10.37 | 10.56 | 10.80 | 10.99 |

| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| R 2500 | - | 10.39 | 10.63 | 10.78 | 11.00 | 11.23 | 11.41 |
| R 2514 | XS2499_R | 10.46 | 10.69 | 10.85 | 11.06 | 11.29 | 11.48 |
| R 2536 | XS2520_R330 | 10.57 | 10.81 | 10.97 | 11.18 | 11.41 | 11.59 |
| Railbridge (TAFE) | | | | | | | |
| R 2548 | XS2531_R | 10.65 | 10.89 | 11.05 | 11.27 | 11.50 | 11.68 |
| R 2560 | XS2544_R350 | 10.74 | 10.97 | 11.13 | 11.34 | 11.56 | 11.74 |
| R 2600 | - | 11.06 | 11.27 | 11.41 | 11.60 | 11.79 | 11.95 |
| R 2672 | XS2656_R360 | 11.67 | 11.83 | 11.94 | 12.08 | 12.22 | 12.33 |
| R 2700 | - | 11.89 | 12.06 | 12.16 | 12.30 | 12.43 | 12.54 |
| R 2768 | XS2755_R370 | 12.45 | 12.61 | 12.71 | 12.83 | 12.94 | 13.03 |
| R 2800 | - | 12.55 | 12.71 | 12.81 | 12.93 | 13.04 | 13.13 |
| R 2871 | XS2861_R380 | 12.74 | 12.91 | 13.01 | 13.14 | 13.25 | 13.34 |
| R 2900 | - | 12.85 | 13.00 | 13.08 | 13.20 | 13.31 | 13.39 |
| R 2962 | XS2953_R390 | 13.09 | 13.20 | 13.24 | 13.34 | 13.43 | 13.51 |
| R 2971 | XS2963_R | 13.36 | 13.43 | 13.44 | 13.50 | 13.57 | 13.63 |
| Assembly Street Footbridge | | | | | | | |
| R 2984 | XS2971_R410 | 13.38 | 13.78 | 13.90 | 14.00 | 14.07 | 14.12 |
| R 3000 | - | 13.44 | 13.81 | 13.94 | 14.04 | 14.11 | 14.16 |
| R 3070 | XS3056_R420 | 13.66 | 13.96 | 14.07 | 14.18 | 14.26 | 14.32 |
| R 3100 | - | 13.74 | 14.04 | 14.15 | 14.26 | 14.35 | 14.42 |
| R 3199 | XS3184_R430 | 14.03 | 14.30 | 14.42 | 14.54 | 14.66 | 14.75 |
| R 3200 | - | 14.03 | 14.31 | 14.42 | 14.55 | 14.66 | 14.75 |
| R 3228 | XS3213_R432 | 14.15 | 14.41 | 14.53 | 14.66 | 14.78 | 14.88 |
| R 3300 | - | 14.42 | 14.66 | 14.77 | 14.90 | 15.03 | 15.12 |
| R 3364 | XS3349_R440 | 14.65 | 14.88 | 14.98 | 15.11 | 15.23 | 15.33 |
| R 3400 | - | 14.78 | 15.01 | 15.11 | 15.24 | 15.35 | 15.44 |
| R 3446 | XS3432_R450 | 14.95 | 15.18 | 15.28 | 15.40 | 15.51 | 15.59 |
| R 3475 | XS3460_R460 | 15.17 | 15.39 | 15.48 | 15.59 | 15.67 | 15.74 |
| R 3500 | - | 15.37 | 15.61 | 15.71 | 15.83 | 15.92 | 15.99 |
| R 3600 | - | 16.26 | 16.55 | 16.68 | 16.82 | 16.93 | 17.03 |
| R 3646 | XS3633_R470 | 16.64 | 16.97 | 17.11 | 17.26 | 17.39 | 17.50 |
| R 3700 | - | 16.84 | 17.13 | 17.27 | 17.42 | 17.56 | 17.68 |
| R 3760 | XS3747_R481 | 17.06 | 17.31 | 17.44 | 17.60 | 17.74 | 17.87 |
| R 3771 | XS3757_R | 17.09 | 17.36 | 17.49 | 17.65 | 17.80 | 17.93 |

| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| McCarthy Road | | | | | | | |
| R 3795 | XS3780_R500 | 17.13 | 17.42 | 17.58 | 17.77 | 17.92 | 18.06 |
| R 3800 | - | 17.13 | 17.43 | 17.58 | 17.78 | 17.93 | 18.07 |
| R 3900 | - | note (1) | note (1) | note (1) | note (1) | note (1) | 18.18 |
| R 4000 | - | 17.86 | 18.07 | 18.16 | 18.28 | 18.39 | 18.48 |
| R 4018 | XS4009_R | 17.96 | 18.16 | 18.25 | 18.35 | 18.46 | 18.54 |
| R 4100 | - | 18.64 | 18.88 | 19.00 | 19.13 | 19.24 | 19.34 |
| R 4143 | XS4134_R530 | 18.99 | 19.25 | 19.38 | 19.54 | 19.65 | 19.75 |
| R 4200 | - | 19.48 | 19.75 | 19.90 | 20.06 | 20.18 | 20.28 |
| R 4245 | XS4234_R540 | 19.86 | 20.15 | 20.30 | 20.47 | 20.60 | 20.71 |
| Ainsworth Street Footbridge | | | | | | | |
| R 4251 | XS4240_R | 19.87 | 20.16 | 20.31 | 20.49 | 20.62 | 20.72 |
| R 4296 | XS4281_R560 | 19.97 | 20.27 | 20.43 | 20.61 | 20.74 | 20.86 |
| R 4300 | - | 20.00 | 20.30 | 20.45 | 20.63 | 20.77 | 20.89 |
| R 4400 | - | 20.48 | 20.75 | 20.89 | 21.06 | 21.19 | 21.32 |
| R 4416 | XS4406_R570 | 20.57 | 20.83 | 20.98 | 21.14 | 21.27 | 21.40 |
| R 4500 | - | 21.41 | 21.65 | 21.78 | 21.93 | 22.04 | 22.17 |
| R 4530 | XS4522_R580 | 21.71 | 21.94 | 22.06 | 22.21 | 22.32 | 22.44 |
| R 4600 | - | 23.06 | 23.26 | 23.38 | 23.51 | 23.62 | 23.73 |
| R 4700 | - | 25.01 | 25.18 | 25.29 | 25.40 | 25.49 | 25.59 |
| R 4730 | XS4723_R590 | 25.58 | 25.77 | 25.86 | 25.98 | 26.06 | 26.16 |

Note (1) – the current BCC AMTD line does not intersect the flood surface

# Appendix E: Design Events (Scenario 3) - Peak Flood Levels

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| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| **Moolabin Creek** | | | | | | | |
| ML 0 | - | 1.42 | 1.55 | 1.67 | 1.86 | 2.14 | 2.34 |
| ML 100 | - | 1.79 | 2.01 | 2.14 | 2.34 | 2.58 | 2.79 |
| ML 200 | - | 1.99 | 2.29 | 2.44 | 2.70 | 2.97 | 3.20 |
| ML 300 | - | 2.35 | 2.68 | 2.80 | 3.03 | 3.30 | 3.50 |
| ML 400 | - | 2.70 | 3.03 | 3.16 | 3.34 | 3.58 | 3.77 |
| ML 500 | - | 2.84 | 3.13 | 3.25 | 3.42 | 3.66 | 3.84 |
| ML 600 | - | 2.95 | 3.22 | 3.33 | 3.50 | 3.74 | 3.92 |
| ML 700 | - | 3.02 | 3.29 | 3.40 | 3.57 | 3.80 | 3.98 |
| ML 800 | - | 3.24 | 3.55 | 3.67 | 3.86 | 4.09 | 4.26 |
| ML 900 | - | 3.31 | 3.61 | 3.74 | 3.93 | 4.15 | 4.32 |
| ML 1000 | - | 3.36 | 3.66 | 3.78 | 3.97 | 4.20 | 4.36 |
| ML 1100 | - | 3.51 | 3.82 | 3.93 | 4.11 | 4.32 | 4.48 |
| ML 1200 | - | 3.63 | 3.94 | 4.06 | 4.24 | 4.46 | 4.61 |
| ML 1260 | - | 3.67 | 3.96 | 4.07 | 4.24 | 4.43 | 4.57 |
| Curzon Street | | | | | | | |
| ML 1276 | - | 3.71 | 3.99 | 4.14 | 4.33 | 4.56 | 4.71 |
| ML 1300 | - | 3.77 | 4.10 | 4.29 | 4.52 | 4.78 | 4.97 |
| ML 1400 | - | 3.78 | 4.11 | 4.30 | 4.53 | 4.80 | 5.00 |
| ML 1500 | - | 3.81 | 4.13 | 4.32 | 4.54 | 4.81 | 5.00 |
| ML 1600 | - | 3.82 | 4.14 | 4.32 | 4.55 | 4.81 | 5.00 |
| ML 1700 | - | 3.83 | 4.14 | 4.33 | 4.55 | 4.82 | 5.01 |
| ML 1800 | - | 3.83 | 4.14 | 4.33 | 4.55 | 4.82 | 5.01 |
| ML 1900 | - | 3.84 | 4.16 | 4.34 | 4.56 | 4.82 | 5.01 |
| ML 2000 | - | 3.88 | 4.17 | 4.35 | 4.57 | 4.83 | 5.02 |
| ML 2100 | - | 3.88 | 4.18 | 4.35 | 4.58 | 4.84 | 5.03 |
| ML 2200 | - | 3.88 | 4.18 | 4.36 | 4.58 | 4.84 | 5.03 |
| ML 2300 | - | 3.92 | 4.21 | 4.38 | 4.60 | 4.87 | 5.05 |
| ML 2400 | - | 3.94 | 4.22 | 4.38 | 4.60 | 4.87 | 5.05 |
| ML 2500 | - | 4.57 | 4.69 | 4.76 | 4.85 | 4.93 | 5.09 |
| ML 2600 | - | 5.50 | 5.80 | 5.84 | 5.91 | 6.22 | 6.54 |
| ML 2607 | - | 5.53 | 5.82 | 5.87 | 5.96 | 6.27 | 6.58 |
| Fairfield Road | | | | | | | |

| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| ML 2638 | XS2895_ML170 | 5.83 | 6.29 | 6.48 | 6.79 | 7.10 | 7.33 |
| ML 2651 | XS2907_ML179 | 5.83 | 6.29 | 6.48 | 6.79 | 7.10 | 7.33 |
| Chale Street | | | | | | | |
| ML 2669 | XS2925_ML | 5.87 | 6.32 | 6.51 | 6.81 | 7.12 | 7.35 |
| ML 2700 | - | 5.89 | 6.33 | 6.53 | 6.82 | 7.13 | 7.36 |
| ML 2741 | XS2999_ML200 | 5.91 | 6.35 | 6.55 | 6.83 | 7.14 | 7.37 |
| ML 2766 | XS3024_ML210 | 5.94 | 6.36 | 6.57 | 6.85 | 7.16 | 7.39 |
| Railway Bridges | | | | | | | |
| ML 2798 | XS3056_ML230 | 5.97 | 6.40 | 6.60 | 6.93 | 7.26 | 7.53 |
| ML 2800 | - | 5.98 | 6.40 | 6.60 | 6.93 | 7.26 | 7.54 |
| ML 2838 | XS3093_ML231 | 6.02 | 6.44 | 6.64 | 6.97 | 7.29 | 7.57 |
| Evesham Street | | | | | | | |
| ML 2855 | XS3109_ML233 | 6.03 | 6.46 | 6.66 | 7.00 | 7.32 | 7.60 |
| ML 2881 | XS3125_ML240 | 6.05 | 6.47 | 6.67 | 6.99 | 7.32 | 7.60 |
| ML 2900 | - | 6.14 | 6.55 | 6.75 | 7.05 | 7.37 | 7.64 |
| ML 3000 | - | 6.71 | 7.02 | 7.19 | 7.40 | 7.64 | 7.88 |
| ML 3015 | XS3249_ML250 | 6.80 | 7.10 | 7.27 | 7.46 | 7.69 | 7.92 |
| ML 3100 | - | 7.71 | 8.01 | 8.17 | 8.33 | 8.50 | 8.71 |
| ML 3101 | XS3337_ML260 | 7.72 | 8.02 | 8.18 | 8.34 | 8.51 | 8.72 |
| ML 3200 | - | 8.30 | 8.61 | 8.79 | 8.96 | 9.12 | 9.34 |
| ML 3258 | XS3498_ML270 | 8.65 | 8.97 | 9.15 | 9.33 | 9.49 | 9.72 |
| ML 3300 | - | 8.74 | 9.06 | 9.25 | 9.43 | 9.60 | 9.82 |
| ML 3353 | XS3594_ML280 | 8.84 | 9.17 | 9.37 | 9.56 | 9.73 | 9.95 |
| Lucy Street | | | | | | | |
| ML 3376 | XS3617_ML | 9.12 | 9.43 | 9.59 | 9.73 | 9.87 | 10.07 |
| ML 3400 | - | 9.16 | 9.47 | 9.63 | 9.78 | 9.92 | 10.12 |
| ML 3500 | - | 9.34 | 9.66 | 9.83 | 9.98 | 10.13 | 10.34 |
| ML 3523 | XS3767_ML | 9.38 | 9.71 | 9.88 | 10.03 | 10.18 | 10.40 |
| ML 3600 | - | 10.15 | 10.47 | 10.65 | 10.82 | 10.97 | 11.20 |
| ML 3646 | XS3896_ML320 | 10.66 | 10.99 | 11.18 | 11.35 | 11.52 | 11.75 |
| ML 3700 | - | 11.06 | 11.41 | 11.62 | 11.80 | 11.96 | 12.20 |
| ML 3794 | XS4052_ML330 | 11.40 | 11.77 | 11.99 | 12.20 | 12.34 | 12.59 |
| ML 3800 | - | 11.40 | 11.79 | 11.99 | 12.32 | 12.43 | 12.62 |
| Gow Street | | | | | | | |

| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| ML 3818 | XS4076_ML350 | 11.91 | 12.54 | 12.74 | 12.93 | 13.06 | 13.19 |
| ML 3851 | XS4106_ML351 | 11.99 | 12.60 | 12.81 | 13.00 | 13.14 | 13.29 |
| ML 3900 | - | 12.15 | 12.72 | 12.93 | 13.13 | 13.30 | 13.45 |
| ML 4000 | - | 12.65 | 13.10 | 13.30 | 13.53 | 13.72 | 13.89 |
| ML 4062 | XS4381_ML370 | 12.87 | 13.26 | 13.46 | 13.68 | 13.87 | 14.04 |
| ML 4100 | - | 13.20 | 13.56 | 13.74 | 13.95 | 14.13 | 14.29 |
| ML 4140 | XS4461_ML370 | 13.54 | 13.86 | 14.02 | 14.22 | 14.40 | 14.55 |
| ML 4200 | - | 14.02 | 14.33 | 14.50 | 14.70 | 14.87 | 15.03 |
| ML 4277 | XS4592_ML380 | 14.63 | 14.92 | 15.07 | 15.26 | 15.44 | 15.60 |
| **Rocky Water Holes Creek** | | | | | | | |
| R 100 | - | 3.88 | 4.18 | 4.35 | 4.58 | 4.84 | 5.02 |
| R 200 | - | 3.89 | 4.18 | 4.35 | 4.58 | 4.84 | 5.02 |
| R 237 | XS214_R29 | 3.89 | 4.18 | 4.35 | 4.58 | 4.84 | 5.02 |
| R 283 | XS259_R30 | 3.89 | 4.18 | 4.36 | 4.58 | 4.84 | 5.03 |
| R 300 | - | 3.89 | 4.18 | 4.36 | 4.58 | 4.84 | 5.03 |
| R 400 | - | 3.90 | 4.20 | 4.37 | 4.59 | 4.85 | 5.04 |
| R 500 | - | 3.98 | 4.24 | 4.40 | 4.62 | 4.88 | 5.07 |
| R 600 | - | 4.08 | 4.31 | 4.44 | 4.65 | 4.91 | 5.10 |
| R 700 | - | 4.18 | 4.41 | 4.54 | 4.70 | 4.95 | 5.14 |
| R 755 | XS725_R60 | 4.23 | 4.47 | 4.59 | 4.75 | 4.98 | 5.16 |
| R 800 | - | 4.35 | 4.59 | 4.72 | 4.87 | 5.06 | 5.24 |
| R 947 | XS904_R80 | 4.68 | 4.93 | 5.07 | 5.22 | 5.42 | 5.58 |
| Fairfield Road | | | | | | | |
| R 977 | XS934_R100 | 4.69 | 4.94 | 5.08 | 5.23 | 5.43 | 5.59 |
| R 986 | XS943_R | 4.71 | 4.97 | 5.11 | 5.27 | 5.48 | 5.65 |
| Muriel Avenue Footbridge | | | | | | | |
| R 989 | XS946_R | 4.86 | 5.20 | 5.38 | 5.61 | 5.89 | 6.10 |
| R 997 | XS957_R115 | 4.89 | 5.24 | 5.43 | 5.67 | 5.97 | 6.19 |
| R 1000 | - | 4.90 | 5.25 | 5.43 | 5.67 | 5.98 | 6.19 |
| Railway Bridges | | | | | | | |
| R 1020 | XS976_R | 4.91 | 5.27 | 5.45 | 5.69 | 6.00 | 6.22 |
| R 1025 | XS988_R120 | 4.92 | 5.27 | 5.45 | 5.69 | 6.00 | 6.22 |
| Muriel Avenue | | | | | | | |
| R 1075 | XS1036_R140 | 4.93 | 5.28 | 5.46 | 5.70 | 6.00 | 6.23 |

| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| Ipswich Road | | | | | | | |
| R 1142 | XS1104_R150 | 5.06 | 5.49 | 5.76 | 6.08 | 6.45 | 6.71 |
| R 1200 | - | 5.11 | 5.54 | 5.80 | 6.11 | 6.47 | 6.73 |
| R 1210 | XS1174_R170 | 5.12 | 5.55 | 5.80 | 6.11 | 6.47 | 6.73 |
| R 1300 | - | 5.37 | 5.76 | 5.98 | 6.26 | 6.58 | 6.82 |
| R 1390 | XS1359_R180 | 5.88 | 6.21 | 6.38 | 6.61 | 6.86 | 7.07 |
| R 1400 | - | 5.91 | 6.24 | 6.40 | 6.63 | 6.88 | 7.08 |
| R 1500 | - | 6.45 | 6.71 | 6.83 | 6.99 | 7.18 | 7.36 |
| R 1526 | XS1508_R190 | 6.64 | 6.89 | 7.00 | 7.15 | 7.31 | 7.49 |
| R 1600 | - | 6.93 | 7.17 | 7.28 | 7.42 | 7.57 | 7.72 |
| R 1632 | XS1614_R200 | 7.06 | 7.29 | 7.40 | 7.54 | 7.69 | 7.83 |
| R 1679 | XS1664_R202 | 7.21 | 7.45 | 7.56 | 7.69 | 7.84 | 7.98 |
| R 1700 | - | 7.27 | 7.50 | 7.61 | 7.74 | 7.89 | 8.03 |
| R 1800 | - | 7.51 | 7.73 | 7.84 | 7.97 | 8.11 | 8.24 |
| R 1818 | XS1800_R210 | 7.56 | 7.78 | 7.88 | 8.01 | 8.15 | 8.29 |
| R 1865 | XS1848_R215 | 7.68 | 7.90 | 8.00 | 8.13 | 8.27 | 8.41 |
| R 1900 | - | 7.75 | 7.98 | 8.08 | 8.22 | 8.39 | 8.53 |
| R 1918 | XS1899_R220 | 7.79 | 8.02 | 8.12 | 8.27 | 8.45 | 8.58 |
| Gladstone Street | | | | | | | |
| R 1954 | XS1934_R240 | 7.81 | 8.06 | 8.18 | 8.37 | 8.59 | 8.74 |
| R 1989 | XS1969_R250 | 7.85 | 8.11 | 8.23 | 8.42 | 8.65 | 8.81 |
| R 2000 | - | 7.87 | 8.12 | 8.25 | 8.45 | 8.67 | 8.83 |
| R 2026 | XS2004_R260 | 7.91 | 8.16 | 8.30 | 8.50 | 8.73 | 8.89 |
| Beaudesert Road | | | | | | | |
| R 2078 | XS2058_R281 | 7.93 | 8.21 | 8.37 | 8.61 | 8.93 | 9.18 |
| R 2100 | - | 7.98 | 8.26 | 8.42 | 8.67 | 8.98 | 9.23 |
| R 2181 | XS2159_R290 | 8.19 | 8.47 | 8.63 | 8.88 | 9.18 | 9.43 |
| R 2200 | - | 8.34 | 8.62 | 8.77 | 9.01 | 9.31 | 9.55 |
| R 2300 | - | 9.10 | 9.35 | 9.48 | 9.68 | 9.93 | 10.14 |
| R 2301 | XS2280_R298 | 9.11 | 9.35 | 9.48 | 9.68 | 9.93 | 10.15 |
| Footbridge (TAFE) | | | | | | | |
| R 2308 | XS2287_R299 | 9.12 | 9.37 | 9.50 | 9.70 | 9.95 | 10.16 |
| R 2400 | - | 9.93 | 10.17 | 10.30 | 10.47 | 10.70 | 10.90 |
| R 2407 | XS2388_R320 | 9.98 | 10.23 | 10.35 | 10.53 | 10.76 | 10.95 |

| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| R 2500 | - | 10.39 | 10.61 | 10.76 | 10.96 | 11.19 | 11.38 |
| R 2514 | XS2499_R | 10.45 | 10.67 | 10.82 | 11.03 | 11.25 | 11.44 |
| R 2536 | XS2520_R330 | 10.57 | 10.79 | 10.94 | 11.15 | 11.37 | 11.55 |
| Railbridge (TAFE) | | | | | | | |
| R 2548 | XS2531_R | 10.65 | 10.87 | 11.03 | 11.24 | 11.46 | 11.64 |
| R 2560 | XS2544_R350 | 10.74 | 10.95 | 11.10 | 11.30 | 11.52 | 11.70 |
| R 2600 | - | 11.07 | 11.26 | 11.40 | 11.58 | 11.78 | 11.94 |
| R 2672 | XS2656_R360 | 11.67 | 11.83 | 11.95 | 12.10 | 12.25 | 12.37 |
| R 2700 | - | 11.92 | 12.08 | 12.19 | 12.33 | 12.47 | 12.59 |
| R 2768 | XS2755_R370 | 12.52 | 12.68 | 12.78 | 12.90 | 13.02 | 13.12 |
| R 2800 | - | 12.61 | 12.77 | 12.88 | 13.00 | 13.12 | 13.22 |
| R 2871 | XS2861_R380 | 12.78 | 12.95 | 13.06 | 13.20 | 13.32 | 13.42 |
| R 2900 | - | 12.89 | 13.04 | 13.15 | 13.28 | 13.40 | 13.50 |
| R 2962 | XS2953_R390 | 13.14 | 13.24 | 13.34 | 13.46 | 13.57 | 13.67 |
| R 2971 | XS2963_R | 13.38 | 13.44 | 13.49 | 13.60 | 13.70 | 13.78 |
| Assembly Street Footbridge | | | | | | | |
| R 2984 | XS2971_R410 | 13.41 | 13.85 | 13.97 | 14.06 | 14.15 | 14.22 |
| R 3000 | - | 13.46 | 13.88 | 14.01 | 14.10 | 14.19 | 14.27 |
| R 3070 | XS3056_R420 | 13.68 | 14.01 | 14.13 | 14.24 | 14.34 | 14.43 |
| R 3100 | - | 13.76 | 14.08 | 14.20 | 14.32 | 14.42 | 14.51 |
| R 3199 | XS3184_R430 | 14.05 | 14.32 | 14.44 | 14.57 | 14.69 | 14.79 |
| R 3200 | - | 14.05 | 14.33 | 14.44 | 14.57 | 14.69 | 14.80 |
| R 3228 | XS3213_R432 | 14.17 | 14.43 | 14.54 | 14.68 | 14.81 | 14.92 |
| R 3300 | - | 14.44 | 14.69 | 14.80 | 14.95 | 15.07 | 15.18 |
| R 3364 | XS3349_R440 | 14.66 | 14.91 | 15.03 | 15.17 | 15.29 | 15.40 |
| R 3400 | - | 14.80 | 15.05 | 15.16 | 15.30 | 15.42 | 15.52 |
| R 3446 | XS3432_R450 | 14.98 | 15.22 | 15.33 | 15.46 | 15.58 | 15.67 |
| R 3475 | XS3460_R460 | 15.19 | 15.43 | 15.54 | 15.68 | 15.79 | 15.89 |
| R 3500 | - | 15.40 | 15.65 | 15.77 | 15.90 | 16.02 | 16.13 |
| R 3600 | - | 16.27 | 16.56 | 16.68 | 16.85 | 17.00 | 17.12 |
| R 3646 | XS3633_R470 | 16.66 | 16.97 | 17.10 | 17.27 | 17.43 | 17.57 |
| R 3700 | - | 16.91 | 17.19 | 17.31 | 17.48 | 17.64 | 17.79 |
| R 3760 | XS3747_R481 | 17.18 | 17.43 | 17.55 | 17.71 | 17.87 | 18.02 |
| R 3771 | XS3757_R | 17.22 | 17.47 | 17.59 | 17.76 | 17.91 | 18.06 |

| AMTD (m) | 1d Cross-section ID | Design Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-yr ARI (39% AEP) | 5-yr ARI (18% AEP) | 10-yr ARI (10% AEP) | 20-yr ARI (5% AEP) | 50-yr ARI (2% AEP) | 100-yr ARI (1% AEP) |
| McCarthy Road | | | | | | | |
| R 3795 | XS3780_R500 | 17.25 | 17.53 | 17.67 | 17.86 | 18.03 | 18.18 |
| R 3800 | - | 17.26 | 17.54 | 17.68 | 17.87 | 18.03 | 18.19 |
| R 3900 | - | note (1) | note (1) | note (1) | note (1) | 18.14 | 18.29 |
| R 4000 | - | 17.91 | 18.14 | 18.25 | 18.41 | 18.54 | 18.66 |
| R 4018 | XS4009_R | 18.00 | 18.23 | 18.34 | 18.49 | 18.61 | 18.73 |
| R 4100 | - | 18.68 | 18.91 | 19.02 | 19.17 | 19.30 | 19.42 |
| R 4143 | XS4134_R530 | 19.03 | 19.26 | 19.38 | 19.53 | 19.65 | 19.78 |
| R 4200 | - | 19.52 | 19.77 | 19.89 | 20.06 | 20.19 | 20.32 |
| R 4245 | XS4234_R540 | 19.90 | 20.17 | 20.30 | 20.47 | 20.61 | 20.75 |
| Ainsworth Street Footbridge | | | | | | | |
| R 4251 | XS4240_R | 19.91 | 20.18 | 20.32 | 20.49 | 20.63 | 20.77 |
| R 4296 | XS4281_R560 | 20.02 | 20.29 | 20.43 | 20.61 | 20.75 | 20.89 |
| R 4300 | - | 20.04 | 20.32 | 20.46 | 20.63 | 20.77 | 20.92 |
| R 4400 | - | 20.53 | 20.77 | 20.89 | 21.05 | 21.19 | 21.32 |
| R 4416 | XS4406_R570 | 20.62 | 20.85 | 20.97 | 21.13 | 21.26 | 21.40 |
| R 4500 | - | 21.45 | 21.66 | 21.77 | 21.91 | 22.03 | 22.16 |
| R 4530 | XS4522_R580 | 21.75 | 21.95 | 22.05 | 22.19 | 22.31 | 22.43 |
| R 4600 | - | 23.09 | 23.27 | 23.37 | 23.50 | 23.61 | 23.72 |
| R 4700 | - | 25.03 | 25.19 | 25.28 | 25.39 | 25.48 | 25.58 |
| R 4730 | XS4723_R590 | 25.62 | 25.77 | 25.85 | 25.96 | 26.05 | 26.15 |

Note (1) – the current BCC AMTD line does not intersect the flood surface

# Appendix F: Rare Events (Scenario 3) - Peak Flood Levels

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| AMTD (m) | 1d Cross-section ID | Rare Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | |
|---|---|---|---|
| | | 200-yr ARI (0.5% AEP) | 500-yr ARI (0.2% AEP) |
| **Moolabin Creek** | | | |
| ML 0 | - | 2.78 | 2.97 |
| ML 100 | - | 3.01 | 3.21 |
| ML 200 | - | 3.39 | 3.64 |
| ML 300 | - | 3.67 | 3.91 |
| ML 400 | - | 3.93 | 4.16 |
| ML 500 | - | 4.00 | 4.24 |
| ML 600 | - | 4.08 | 4.31 |
| ML 700 | - | 4.13 | 4.37 |
| ML 800 | - | 4.39 | 4.59 |
| ML 900 | - | 4.44 | 4.64 |
| ML 1000 | - | 4.48 | 4.69 |
| ML 1100 | - | 4.59 | 4.79 |
| ML 1200 | - | 4.73 | 4.92 |
| ML 1260 | - | 4.66 | 4.82 |
| Curzon Street | | | |
| ML 1276 | - | 4.85 | 5.11 |
| ML 1300 | - | 5.14 | 5.41 |
| ML 1400 | - | 5.17 | 5.45 |
| ML 1500 | - | 5.17 | 5.45 |
| ML 1600 | - | 5.17 | 5.45 |
| ML 1700 | - | 5.18 | 5.45 |
| ML 1800 | - | 5.18 | 5.45 |
| ML 1900 | - | 5.18 | 5.46 |
| ML 2000 | - | 5.19 | 5.46 |
| ML 2100 | - | 5.20 | 5.47 |
| ML 2200 | - | 5.20 | 5.47 |
| ML 2300 | - | 5.21 | 5.48 |
| ML 2400 | - | 5.21 | 5.48 |
| ML 2500 | - | 5.26 | 5.54 |
| ML 2600 | - | 6.65 | 6.78 |
| ML 2607 | - | 6.69 | 6.81 |
| Fairfield Road | | | |

| AMTD (m) | 1d Cross-section ID | Rare Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | |
|---|---|---|---|
| | | 200-yr ARI (0.5% AEP) | 500-yr ARI (0.2% AEP) |
| ML 2638 | XS2895_ML170 | 7.46 | 7.63 |
| ML 2651 | XS2907_ML179 | 7.46 | 7.63 |
| Chale Street | | | |
| ML 2669 | XS2925_ML | 7.48 | 7.65 |
| ML 2700 | - | 7.49 | 7.66 |
| ML 2741 | XS2999_ML200 | 7.51 | 7.68 |
| ML 2766 | XS3024_ML210 | 7.52 | 7.70 |
| Railway Bridges | | | |
| ML 2798 | XS3056_ML230 | 7.71 | 7.97 |
| ML 2800 | - | 7.72 | 7.98 |
| ML 2838 | XS3093_ML231 | 7.75 | 8.02 |
| Evesham Street | | | |
| ML 2855 | XS3109_ML233 | 7.79 | 8.06 |
| ML 2881 | XS3125_ML240 | 7.79 | 8.06 |
| ML 2900 | - | 7.82 | 8.10 |
| ML 3000 | - | 8.05 | 8.30 |
| ML 3015 | XS3249_ML250 | 8.09 | 8.33 |
| ML 3100 | - | 8.84 | 9.07 |
| ML 3101 | XS3337_ML260 | 8.85 | 9.08 |
| ML 3200 | - | 9.49 | 9.74 |
| ML 3258 | XS3498_ML270 | 9.87 | 10.13 |
| ML 3300 | - | 9.97 | 10.24 |
| ML 3353 | XS3594_ML280 | 10.11 | 10.37 |
| Lucy Street | | | |
| ML 3376 | XS3617_ML | 10.21 | 10.45 |
| ML 3400 | - | 10.26 | 10.51 |
| ML 3500 | - | 10.49 | 10.75 |
| ML 3523 | XS3767_ML | 10.55 | 10.80 |
| ML 3600 | - | 11.36 | 11.58 |
| ML 3646 | XS3896_ML320 | 11.90 | 12.10 |
| ML 3700 | - | 12.34 | 12.45 |
| ML 3794 | XS4052_ML330 | 12.72 | 12.85 |
| ML 3800 | - | 12.72 | 12.81 |
| Gow Street | | | |

| AMTD (m) | 1d Cross-section ID | Rare Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | |
|---|---|---|---|
| | | 200-yr ARI (0.5% AEP) | 500-yr ARI (0.2% AEP) |
| ML 3818 | XS4076_ML350 | 13.29 | 13.42 |
| ML 3851 | XS4106_ML351 | 13.39 | 13.55 |
| ML 3900 | - | 13.57 | 13.75 |
| ML 4000 | - | 14.03 | 14.25 |
| ML 4062 | XS4381_ML370 | 14.18 | 14.41 |
| ML 4100 | - | 14.43 | 14.65 |
| ML 4140 | XS4461_ML370 | 14.67 | 14.90 |
| ML 4200 | - | 15.16 | 15.39 |
| ML 4277 | XS4592_ML380 | 15.73 | 15.97 |
| **Rocky Water Holes Creek** | | | |
| R 100 | - | 5.19 | 5.47 |
| R 200 | - | 5.19 | 5.47 |
| R 237 | XS214_R29 | 5.19 | 5.47 |
| R 283 | XS259_R30 | 5.20 | 5.47 |
| R 300 | - | 5.20 | 5.47 |
| R 400 | - | 5.20 | 5.48 |
| R 500 | - | 5.24 | 5.51 |
| R 600 | - | 5.28 | 5.54 |
| R 700 | - | 5.31 | 5.57 |
| R 755 | XS725_R60 | 5.33 | 5.59 |
| R 800 | - | 5.40 | 5.65 |
| R 947 | XS904_R80 | 5.68 | 5.95 |
| Fairfield Road | | | |
| R 977 | XS934_R100 | 6.07 | 6.37 |
| R 986 | XS943_R | 6.06 | 6.39 |
| Muriel Avenue Footbridge | | | |
| R 989 | XS946_R | 6.39 | 6.65 |
| R 997 | XS957_R115 | 6.45 | 6.70 |
| R 1000 | - | 6.46 | 6.70 |
| Railway Bridges | | | |
| R 1020 | XS976_R | 6.48 | 6.73 |
| R 1025 | XS988_R120 | 6.49 | 6.73 |
| Muriel Avenue | | | |
| R 1075 | XS1036_R140 | 6.49 | 6.74 |

| AMTD (m) | 1d Cross-section ID | Rare Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | |
|---|---|---|---|
| | | 200-yr ARI (0.5% AEP) | 500-yr ARI (0.2% AEP) |
| Ipswich Road | | | |
| R 1142 | XS1104_R150 | 6.94 | 7.25 |
| R 1200 | - | 6.95 | 7.27 |
| R 1210 | XS1174_R170 | 6.96 | 7.27 |
| R 1300 | - | 7.02 | 7.33 |
| R 1390 | XS1359_R180 | 7.24 | 7.53 |
| R 1400 | - | 7.25 | 7.55 |
| R 1500 | - | 7.51 | 7.78 |
| R 1526 | XS1508_R190 | 7.63 | 7.88 |
| R 1600 | - | 7.87 | 8.12 |
| R 1632 | XS1614_R200 | 7.97 | 8.22 |
| R 1679 | XS1664_R202 | 8.12 | 8.37 |
| R 1700 | - | 8.16 | 8.41 |
| R 1800 | - | 8.37 | 8.59 |
| R 1818 | XS1800_R210 | 8.41 | 8.63 |
| R 1865 | XS1848_R215 | 8.53 | 8.74 |
| R 1900 | - | 8.64 | 8.84 |
| R 1918 | XS1899_R220 | 8.70 | 8.89 |
| Gladstone Street | | | |
| R 1954 | XS1934_R240 | 8.84 | 9.00 |
| R 1989 | XS1969_R250 | 8.92 | 9.10 |
| R 2000 | - | 8.94 | 9.12 |
| R 2026 | XS2004_R260 | 9.00 | 9.18 |
| Beaudesert Road | | | |
| R 2078 | XS2058_R281 | 9.40 | 9.80 |
| R 2100 | - | 9.45 | 9.85 |
| R 2181 | XS2159_R290 | 9.65 | 10.03 |
| R 2200 | - | 9.76 | 10.14 |
| R 2300 | - | 10.33 | 10.65 |
| R 2301 | XS2280_R298 | 10.33 | 10.65 |
| Footbridge (TAFE) | | | |
| R 2308 | XS2287_R299 | 10.35 | 10.67 |
| R 2400 | - | 11.06 | 11.34 |
| R 2407 | XS2388_R320 | 11.11 | 11.38 |

| AMTD (m) | 1d Cross-section ID | Rare Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | |
|---|---|---|---|
| | | 200-yr ARI (0.5% AEP) | 500-yr ARI (0.2% AEP) |
| R 2500 | - | 11.52 | 11.78 |
| R 2514 | XS2499_R | 11.58 | 11.84 |
| R 2536 | XS2520_R330 | 11.67 | 11.90 |
| Railbridge (TAFE) | | | |
| R 2548 | XS2531_R | 11.78 | 12.04 |
| R 2560 | XS2544_R350 | 11.84 | 12.10 |
| R 2600 | - | 12.06 | 12.28 |
| R 2672 | XS2656_R360 | 12.46 | 12.62 |
| R 2700 | - | 12.68 | 12.83 |
| R 2768 | XS2755_R370 | 13.21 | 13.35 |
| R 2800 | - | 13.30 | 13.45 |
| R 2871 | XS2861_R380 | 13.50 | 13.64 |
| R 2900 | - | 13.58 | 13.72 |
| R 2962 | XS2953_R390 | 13.75 | 13.89 |
| R 2971 | XS2963_R | 13.85 | 13.97 |
| Assembly Street Footbridge | | | |
| R 2984 | XS2971_R410 | 14.28 | 14.38 |
| R 3000 | - | 14.33 | 14.43 |
| R 3070 | XS3056_R420 | 14.51 | 14.63 |
| R 3100 | - | 14.59 | 14.72 |
| R 3199 | XS3184_R430 | 14.88 | 15.02 |
| R 3200 | - | 14.88 | 15.02 |
| R 3228 | XS3213_R432 | 15.01 | 15.15 |
| R 3300 | - | 15.27 | 15.40 |
| R 3364 | XS3349_R440 | 15.49 | 15.61 |
| R 3400 | - | 15.60 | 15.72 |
| R 3446 | XS3432_R450 | 15.75 | 15.87 |
| R 3475 | XS3460_R460 | 15.97 | 16.09 |
| R 3500 | - | 16.21 | 16.34 |
| R 3600 | - | 17.21 | 17.36 |
| R 3646 | XS3633_R470 | 17.66 | 17.82 |
| R 3700 | - | 17.89 | 18.05 |
| R 3760 | XS3747_R481 | 18.13 | 18.30 |
| R 3771 | XS3757_R | 18.17 | 18.34 |

| AMTD (m) | 1d Cross-section ID | Rare Events – Scenario 3 (Ultimate Conditions) Peak Water Levels (mAHD) | |
|---|---|---|---|
| | | 200-yr ARI (0.5% AEP) | 500-yr ARI (0.2% AEP) |
| McCarthy Road | | | |
| R 3795 | XS3780_R500 | 18.29 | 18.47 |
| R 3800 | - | 18.30 | 18.48 |
| R 3900 | - | 18.41 | 18.59 |
| R 4000 | - | 18.76 | 18.91 |
| R 4018 | XS4009_R | 18.83 | 18.97 |
| R 4100 | - | 19.52 | 19.64 |
| R 4143 | XS4134_R530 | 19.87 | 19.99 |
| R 4200 | - | 20.42 | 20.55 |
| R 4245 | XS4234_R540 | 20.86 | 21.01 |
| Ainsworth Street Footbridge | | | |
| R 4251 | XS4240_R | 20.89 | 21.25 |
| R 4296 | XS4281_R560 | 21.02 | 21.36 |
| R 4300 | - | 21.04 | 21.37 |
| R 4400 | - | 21.43 | 21.70 |
| R 4416 | XS4406_R570 | 21.51 | 21.76 |
| R 4500 | - | 22.26 | 22.45 |
| R 4530 | XS4522_R580 | 22.52 | 22.70 |
| R 4600 | - | 23.81 | 23.97 |
| R 4700 | - | 25.66 | 25.81 |
| R 4730 | XS4723_R590 | 26.23 | 26.37 |

# Appendix G: Rating Curves

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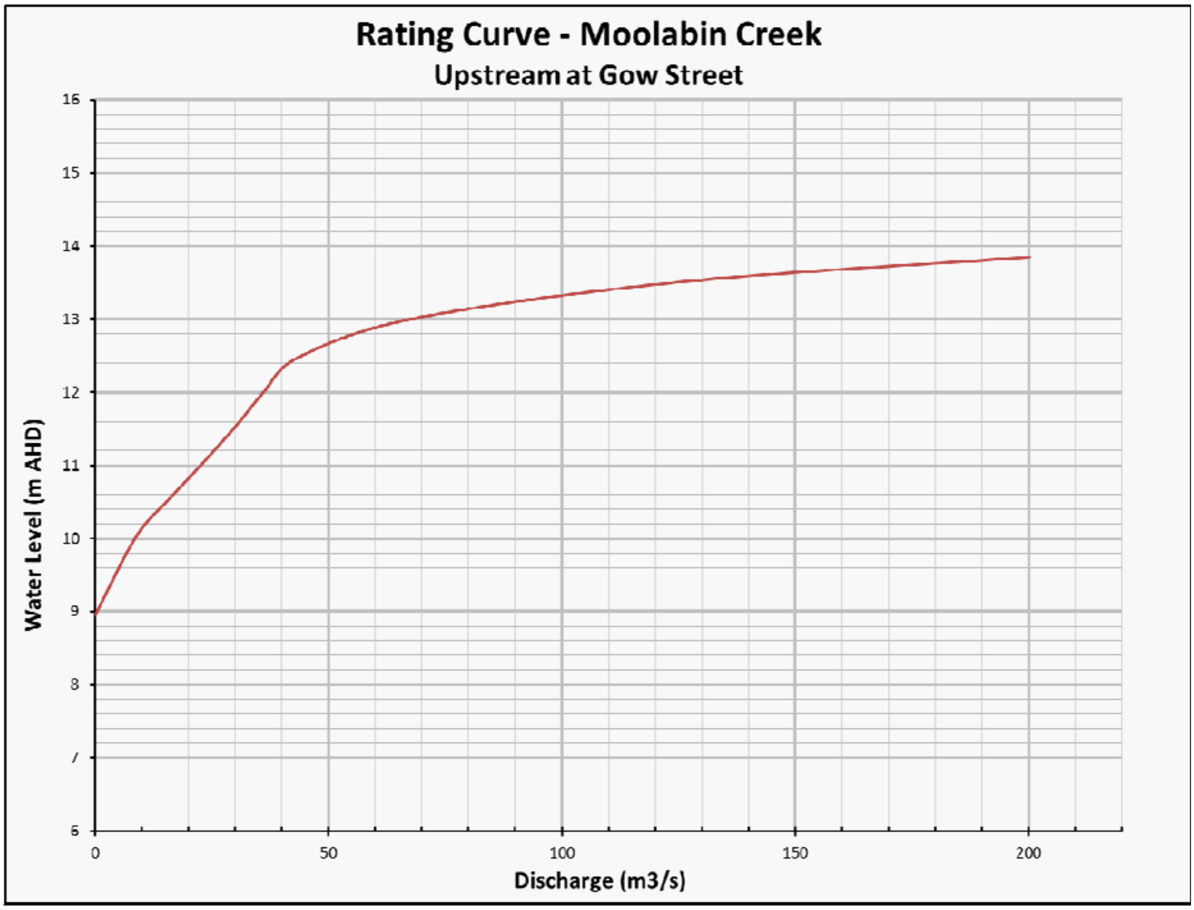
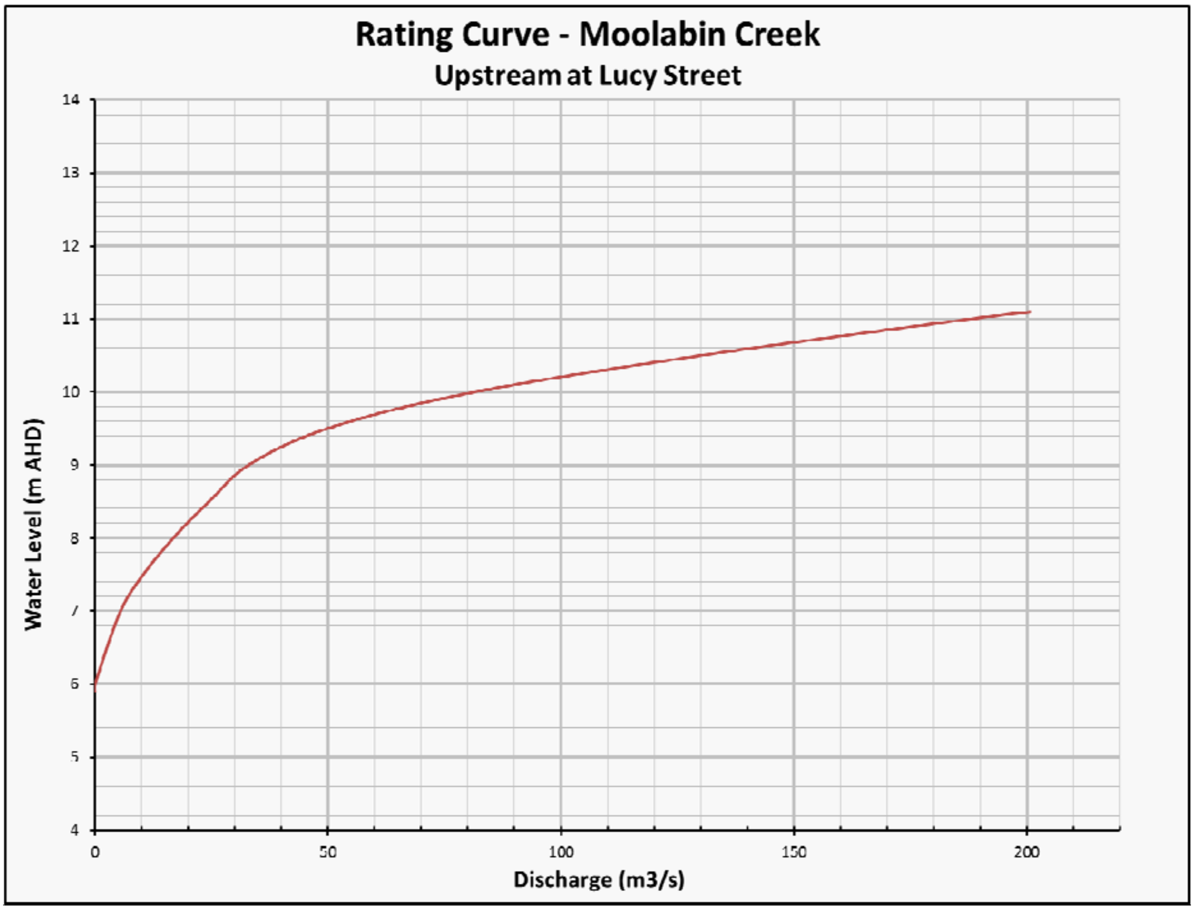
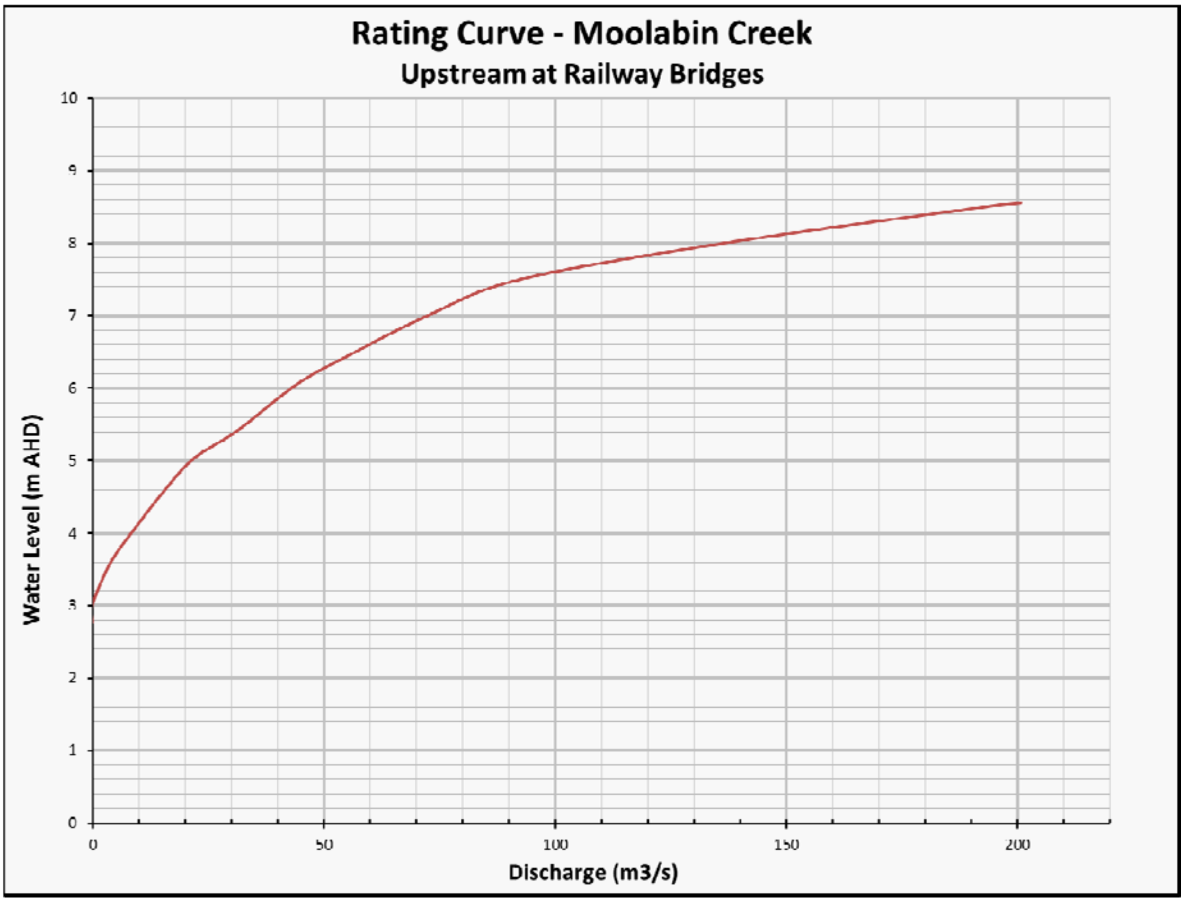
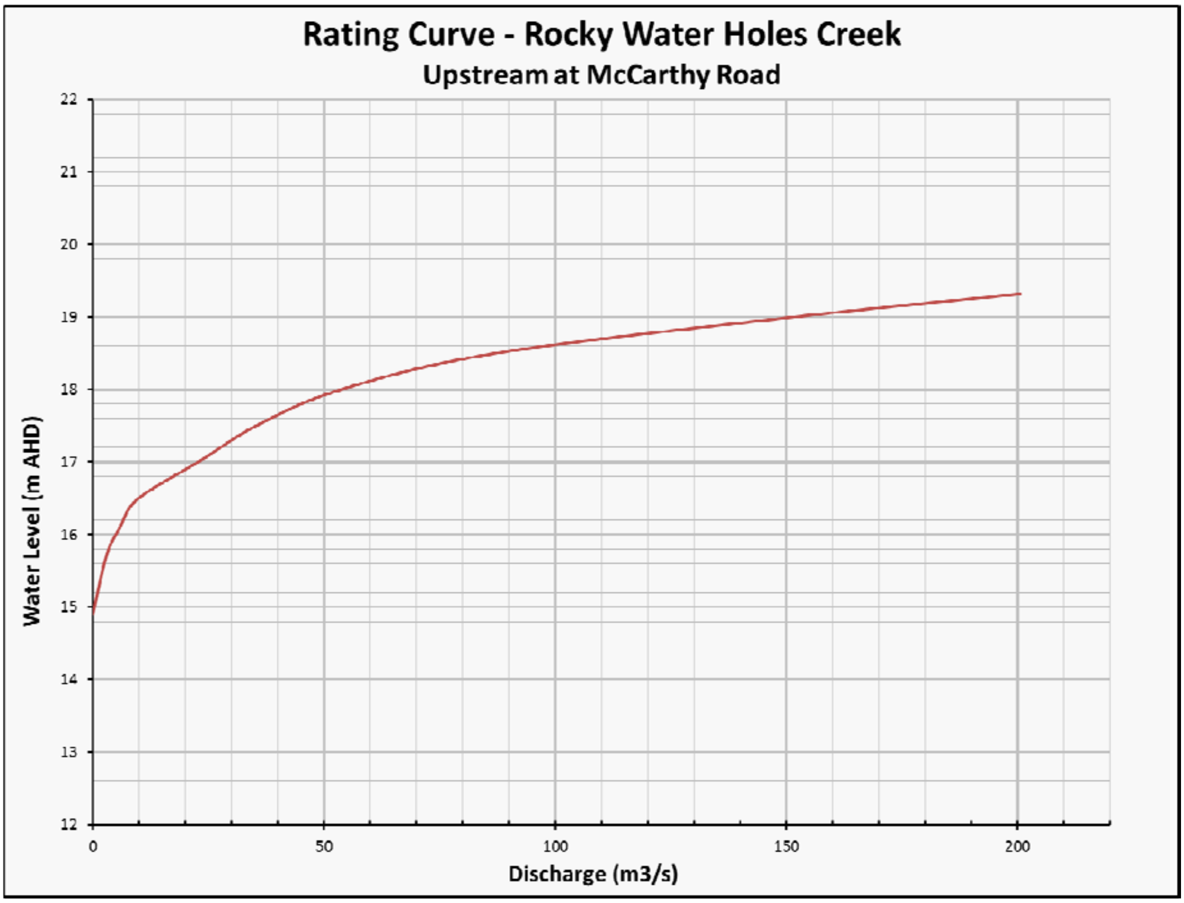

Rating Curve - Moolabin Creek
Upstream at Gow Street
Water Level (m AHD)
Discharge (m3/s)
16
15
14
13
12
11
10
9
8
7
6
0
50
100
150
200

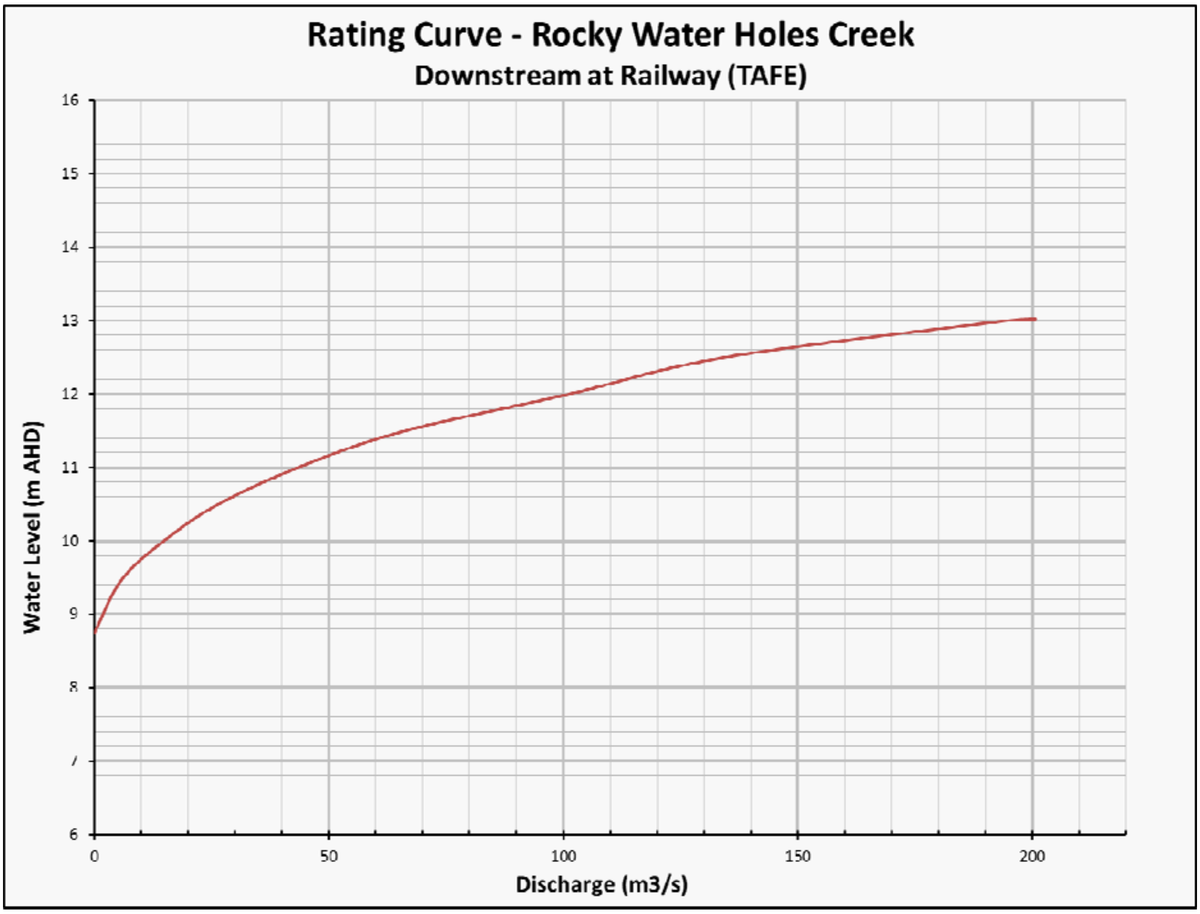

Rating Curve - Moolabin Creek
Upstream at Lucy Street
Water Level (m AHD)
Discharge (m3/s)
14
13
12
11
10
9
8
7
6
5
4
0
50
100
150
200

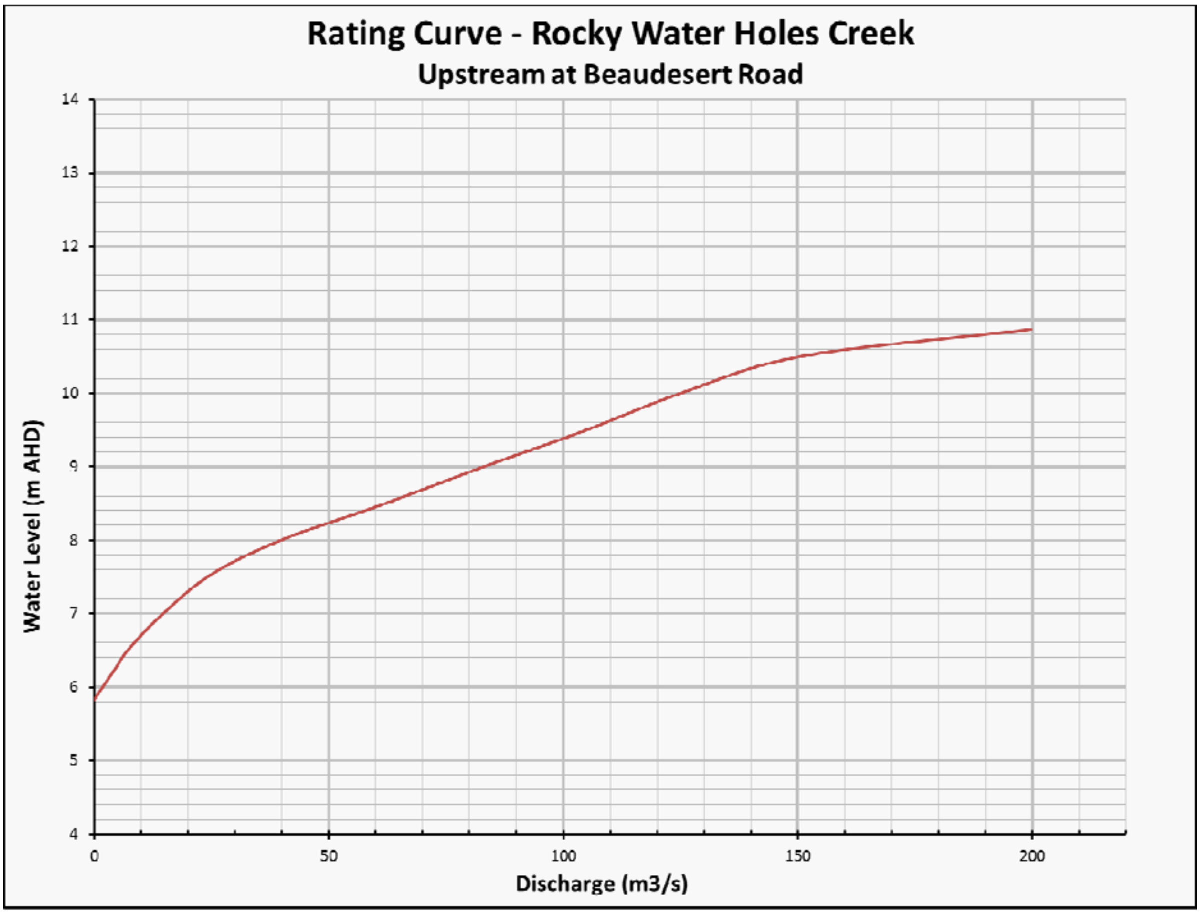


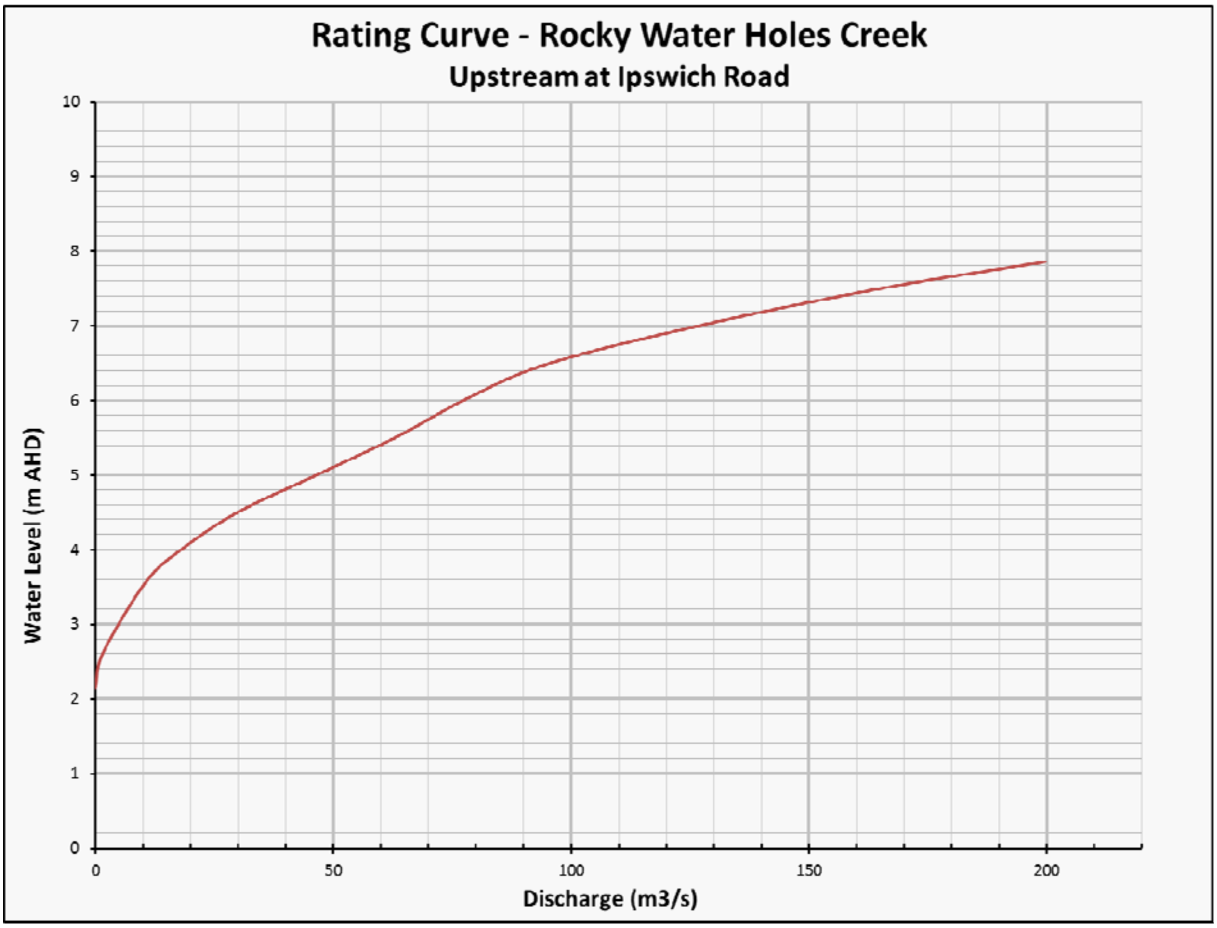
Rating Curve - Moolabin Creek
Upstream at Railway Bridges
Water Level (m AHD)
Discharge (m3/s)

Rating Curve - Rocky Water Holes Creek
Upstream at McCarthy Road
Water Level (m AHD)
Discharge (m3/s)


Rating Curve - Rocky Water Holes Creek
Downstream at Railway (TAFE)
Water Level (m AHD)
Discharge (m3/s)
16
15
14
13
12
11
10
9
8
7
6
0
50
100
150
200


Rating Curve - Rocky Water Holes Creek
Upstream at Beaudesert Road
Water Level (m AHD)
Discharge (m3/s)
14
13
12
11
10
9
8
7
6
5
4
0
50
100
150
200


Rating Curve - Rocky Water Holes Creek
Upstream at Ipswich Road
Water Level (m AHD)
Discharge (m3/s)
0
1
2
3
4
5
6
7
8
9
10
0
50
100
150
200

# Appendix H: Hydraulic Structure Reference Sheets

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| | | | |
|---|---|---|---|
| **Creek:** | Moolabin Creek | **Immunity Rating:** | 2-yr ARI |
| **Location:** | Gow Street | | 50 % AEP |

| | |
|---|---|
| DATE OF SURVEY: Circa 1998 | UBD REF: N/A |
| SURVEYED CROSS SECTION ID: ML340 | BCC ASSET ID: C0486P |
| MODEL ID: S4 | AMTD (m): 3808 |

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 4 / 1800 mm RCP

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 8.99 U/S OBVERT LEVEL (m) 10.79

D/S INVERT LEVEL (m) 8.8 D/S OBVERT LEVEL (m) 10.6

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 22.3

LENGTH OF CULVERT AT OBVERT (m): 22.3

TYPE OF LINING: Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 22.3 PIER WIDTH (m): N/A - multi-cell culvert

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 12.15

HEIGHT OF GUARDRAIL/HANDRAIL: ~ 0.75

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: 60 dia tube steel Rails @ RL 9.95 & 9.45

PLAN NUMBER: W2365

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1963

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Gow Street |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level (m AHD) | PEAK D/S Water Level (m AHD) | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) Weir | VELOCITY (m/s) Structure |
|---|---|---|---|---|---|---|---|---|
| 2000-yr (0.05%) | 131.5 | 13.52 | 13.14 | 0.38 | 103 | 1.10 | 1.6 | 4.2 |
| 500-yr (0.2%) | 93.4 | 13.28 | 12.72 | 0.56 | 69 | 0.98 | 1.6 | 4.1 |
| 100-yr (0.1%) | 66.4 | 12.97 | 12.37 | 0.60 | 56 | 0.77 | 1.2 | 4.1 |
| 50-yr (0.2%) | 57.3 | 12.85 | 12.14 | 0.71 | 52 | 0.65 | 1.0 | 4.0 |
| 20-yr (5%) | 51.6 | 12.74 | 12.00 | 0.74 | 40 | 0.51 | 1.1 | 4.0 |
| 10-yr (10%) | 46.1 | 12.60 | 11.85 | 0.75 | 36 | 0.34 | 0.9 | 4.0 |
| 5-yr (20%) | 42.0 | 12.44 | 11.71 | 0.73 | 22 | 0.18 | 1.0 | 3.9 |
| 2-yr (50%) | 35.3 | 11.99 | 11.44 | 0.55 | 0 | 0.00 | 0.0 | 3.5 |

Notes:

Weir velocity is the average across the entire flooded width at peak flood level

Structure velocity is the peak within the culvert barrel

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Gow Street |


Upstream of Gow Street Culverts


Downstream of Gow Street Culverts

| | | | |
|---|---|---|---|
| **Creek:** | Moolabin Creek | **Immunity Rating:** | < 2-yr ARI |
| **Location:** | Lucy Street | | < 50 % AEP |

DATE OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: ML290

BCC ASSET ID: Unknown

MODEL ID: S5

AMTD (m): 3365

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 5 / 1800 mm RCP

For Culverts: Number of cells/pipes & sizes

For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 5.94

U/S OBVERT LEVEL (m) 7.74

D/S INVERT LEVEL (m) 5.89

D/S OBVERT LEVEL (m) 7.69

For culverts give floor level

For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 18.7

LENGTH OF CULVERT AT OBVERT (m): 18.7

TYPE OF LINING: Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 18.7

PIER WIDTH (m): N/A - multi-cell culvert

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 8.95

HEIGHT OF GUARDRAIL/HANDRAIL: ~ 1

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Steel Rails Top RL 6.71

PLAN NUMBER: W2811

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1970

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Lucy Street |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level (m AHD) | PEAK D/S Water Level (m AHD) | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) Weir | VELOCITY (m/s) Structure |
|---|---|---|---|---|---|---|---|---|
| 2000-yr (0.05%) | 140.4 | 10.58 | 10.51 | 0.07 | 83 | 1.75 | 1.5 | 2.7 |
| 500-yr (0.2%) | 97.2 | 10.14 | 10.06 | 0.08 | 76 | 1.32 | 1.4 | 3.0 |
| 100-yr (0.1%) | 66.0 | 9.78 | 9.64 | 0.14 | 64 | 0.94 | 1.2 | 3.0 |
| 50-yr (0.2%) | 60.2 | 9.69 | 9.52 | 0.17 | 57 | 0.85 | 1.2 | 3.0 |
| 20-yr (5%) | 55.3 | 9.60 | 9.42 | 0.18 | 54 | 0.76 | 1.2 | 3.0 |
| 10-yr (10%) | 49.4 | 9.49 | 9.27 | 0.22 | 52 | 0.65 | 1.0 | 2.9 |
| 5-yr (20%) | 45.1 | 9.39 | 9.14 | 0.25 | 48 | 0.56 | 0.9 | 2.9 |
| 2-yr (50%) | 37.5 | 9.18 | 8.90 | 0.28 | 35 | 0.37 | 0.6 | 2.7 |

Notes:
Weir velocity is the average across the entire flooded width at peak flood level
Structure velocity is the peak within the culvert barrel

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Lucy Street |


Upstream of Lucy Street Culverts


Downstream of Lucy Street Culverts

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Evesham Street |

| | |
|---|---|
| **Immunity Rating:** | < 2-yr ARI |
| | < 50 % AEP |

| | |
|---|---|
| DATE OF SURVEY: Circa 1998 | UBD REF: N/A |
| SURVEYED CROSS SECTION ID: ML232 | BCC ASSET ID: Unknown |
| MODEL ID: S6 | AMTD (m): 2847 |

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 4 / 3550mm w x 2400mm h RCBC

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 3.21 U/S OBVERT LEVEL (m) 5.61

D/S INVERT LEVEL (m) 3.16 D/S OBVERT LEVEL (m) 5.56

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 8.4

LENGTH OF CULVERT AT OBVERT (m): 8.4

TYPE OF LINING: Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 7.85 PIER WIDTH (m): N/A - multi-cell culvert

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 5.81

HEIGHT OF GUARDRAIL/HANDRAIL: 0.9

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS:

PLAN NUMBER: W7214

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1987

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Evesham Street |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level (m AHD) | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | Weir | Structure |
| 2000-yr (0.05%) | 168.7 | 8.34 | 8.28 | 0.06 | 380 | 2.49 | 0.2 | 1.8 |
| 500-yr (0.2%) | 115.4 | 7.89 | 7.85 | 0.04 | 308 | 2.05 | 0.2 | 1.4 |
| 100-yr (0.1%) | 84.5 | 7.41 | 7.38 | 0.03 | 158 | 1.56 | 0.4 | 1.2 |
| 50-yr (0.2%) | 76.4 | 7.18 | 7.15 | 0.03 | 135 | 1.33 | 0.4 | 1.2 |
| 20-yr (5%) | 68.6 | 6.88 | 6.85 | 0.03 | 97 | 1.03 | 0.5 | 1.2 |
| 10-yr (10%) | 61.5 | 6.63 | 6.61 | 0.02 | 83 | 0.79 | 0.6 | 1.2 |
| 5-yr (20%) | 56.9 | 6.48 | 6.46 | 0.02 | 62 | 0.64 | 0.9 | 1.2 |
| 2-yr (50%) | 46.5 | 6.14 | 6.13 | 0.01 | 37 | 0.31 | 1.6 | 1.2 |

Notes:
Weir velocity is the average across the entire flooded width at peak flood level
Structure velocity is the peak within the culvert barrel

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Evesham Street |


Upstream of Evesham Street Crossing


Exist of Evesham Street Crossing

| | | | |
|---|---|---|---|
| **Creek:** | Moolabin Creek | **Immunity Rating:** | > 100-yr ARI |
| **Location:** | Railway Bridges | | > 1 % AEP |

DATE OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: ML221

BCC ASSET ID: W0020 & W0021

MODEL ID: S7

AMTD (m): 2782

STRUCTURE DESCRIPTION: 3 / Multi-span Bridges (1 bridge redundant but still in place)

STRUCTURE SIZE: Various sizes

For Culverts: Number of cells/pipes & sizes　　For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) taken as 3 for structure group　　U/S OBVERT LEVEL (m) taken as 6.85 for structure group

D/S INVERT LEVEL (m) taken as 3 for structure group　　D/S OBVERT LEVEL (m) taken as 6.85 for structure group

For culverts give floor level　　For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING: N/A

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): ~ 25　　PIER WIDTH (m): Various sizes

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): taken as 8.21 for structure group

HEIGHT OF GUARDRAIL/HANDRAIL: Unknown

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Unknown

PLAN NUMBER: S23716 & S23717 & S23718

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Unknown

HAS THE STRUCTURE BEEN UPGRADED? Yes　　Circa 1996 for most downstream structure

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Modelled hydraulically as one structure

| **Creek:** | Moolabin Creek |
|---|---|
| **Location:** | Railway Bridges |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | (m AHD) | | | | | Weir | Structure |
| 2000-yr (0.05%) | 168.7 | 8.22 | 7.74 | 0.48 | 0 | 0.00 | 0.0 | 2.5 |
| 500-yr (0.2%) | 115.4 | 7.81 | 7.58 | 0.23 | 0 | 0.00 | 0.0 | 2.9 |
| 100-yr (0.1%) | 84.5 | 7.34 | 7.23 | 0.11 | 0 | 0.00 | 0.0 | 2.3 |
| 50-yr (0.2%) | 76.4 | 7.11 | 7.02 | 0.09 | 0 | 0.00 | 0.0 | 2.1 |
| 20-yr (5%) | 68.6 | 6.81 | 6.78 | 0.03 | 0 | 0.00 | 0.0 | 2.0 |
| 10-yr (10%) | 61.5 | 6.56 | 6.53 | 0.03 | 0 | 0.00 | 0.0 | 2.2 |
| 5-yr (20%) | 56.9 | 6.42 | 6.38 | 0.04 | 0 | 0.00 | 0.0 | 2.4 |
| 2-yr (50%) | 46.5 | 6.09 | 6.06 | 0.03 | 0 | 0.00 | 0.0 | 2.4 |

Notes:

Weir velocity is the average across the entire flooded width at peak flood level

Structure velocity is a peak average across the bridge opening

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Railway Bridges |

Upstream of Ralway Bridges (from previous Report)

Downstream of Ralway Bridges (from previous Report)

| | | | |
|---|---|---|---|
| **Creek:** | Moolabin Creek | **Immunity Rating:** | < 2-yr ARI |
| **Location:** | Chale Street | | < 50 % AEP |

DATE OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: ML180

BCC ASSET ID: C0127B

MODEL ID: S8

AMTD (m): 2660

STRUCTURE DESCRIPTION: 3 / 3000mm w x 2150mm h RCBC

STRUCTURE SIZE: 1 / 3700mm w x 3450mm h RCBC + 3 / 3350mm w x 1700mm h SLBC

For Culverts: Number of cells/pipes & sizes — For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 2.56 — U/S OBVERT LEVEL (m) 4.71

D/S INVERT LEVEL (m) 2.49 — D/S OBVERT LEVEL (m) 4.64

For culverts give floor level — For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 11

LENGTH OF CULVERT AT OBVERT (m): 11

TYPE OF LINING: Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 11 — PIER WIDTH (m): N/A - multi-cell culvert

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 4.84

HEIGHT OF GUARDRAIL/HANDRAIL: 1

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Steel Rails top RL 6.37 plus Armco

PLAN NUMBER: W5972

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1977

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Chale Street |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | (m AHD) | | | | | Weir | Structure |
| 2000-yr (0.05%) | 147.0 | 7.65 | 7.62 | 0.03 | 175 | 2.79 | 0.2 | 1.8 |
| 500-yr (0.2%) | 110.1 | 7.54 | 7.52 | 0.02 | 169 | 2.68 | 0.2 | 2.0 |
| 100-yr (0.1%) | 84.5 | 7.20 | 7.18 | 0.02 | 155 | 2.34 | 0.2 | 2.1 |
| 50-yr (0.2%) | 75.0 | 6.98 | 6.96 | 0.02 | 149 | 2.13 | 0.2 | 2.0 |
| 20-yr (5%) | 67.5 | 6.73 | 6.71 | 0.02 | 141 | 1.88 | 0.2 | 2.1 |
| 10-yr (10%) | 60.1 | 6.48 | 6.45 | 0.03 | 127 | 1.62 | 0.2 | 1.9 |
| 5-yr (20%) | 54.0 | 6.33 | 6.31 | 0.02 | 125 | 1.49 | 0.2 | 2.2 |
| 2-yr (50%) | 44.4 | 6.00 | 5.97 | 0.03 | 109 | 1.14 | 0.2 | 1.9 |

Notes:
Weir velocity is the average across the entire flooded width at peak flood level
Structure velocity is the peak within the culvert barrel

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Chale Street |


Upstream of Chale Street Culverts


Downstream of Chale Street Culverts

| | | | |
|---|---|---|---|
| **Creek:** | Moolabin Creek | **Immunity Rating:** | 20-yr ARI |
| **Location:** | Fairfield Road | | 5 % AEP |

DATE OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: ML160

BCC ASSET ID: C0520B

MODEL ID: S9

AMTD (m): 2623

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 1 / 3700mm w x 3450mm h RCBC + 3 / 3350mm w x 1700mm h SLBC

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 2.15 U/S OBVERT LEVEL (m) 3.71 & 5.5

D/S INVERT LEVEL (m) 2.05 D/S OBVERT LEVEL (m) 3.61 & 5.40

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 24.5 (3450h) 29.25 (1700h)

LENGTH OF CULVERT AT OBVERT (m): 24.5 (3450h) 29.25 (1700h)

TYPE OF LINING: Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 29.25 PIER WIDTH (m): N/A - multi-cell culvert

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 6.59

HEIGHT OF GUARDRAIL/HANDRAIL: 1.11

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: see sketch FB 8810/1

PLAN NUMBER: W6683

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1984

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Significant siltation in smaller culverts, which has been included in hydraulic model

Differing level information between drawings and field book sketches

| Creek: | Moolabin Creek |
|---|---|
| Location: | Fairfield Road |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | (m AHD) | | | | | Weir | Structure |
| 2000-yr (0.05%) | 147.0 | 7.62 | 7.19 | 0.43 | 141 | 0.84 | 1.2 | 3.5 |
| 500-yr (0.2%) | 110.1 | 7.52 | 6.72 | 0.80 | 125 | 0.54 | 1.0 | 3.5 |
| 100-yr (0.1%) | 84.5 | 7.18 | 6.43 | 0.75 | 102 | 0.37 | 0.5 | 3.5 |
| 50-yr (0.2%) | 75.0 | 6.96 | 6.09 | 0.87 | 76 | 0.21 | 0.3 | 3.4 |
| 20-yr (5%) | 67.5 | 6.71 | 5.93 | 0.78 | 0 | 0.00 | 0.0 | 3.2 |
| 10-yr (10%) | 60.1 | 6.45 | 5.87 | 0.58 | 0 | 0.00 | 0.0 | 2.8 |
| 5-yr (20%) | 54.0 | 6.31 | 5.83 | 0.48 | 0 | 0.00 | 0.0 | 2.5 |
| 2-yr (50%) | 44.4 | 5.97 | 5.64 | 0.33 | 0 | 0.00 | 0.0 | 2.1 |

Notes:

Weir velocity is the average across the entire flooded width at peak flood level

Structure velocity is the peak within the culvert barrel

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Fairfield Road |


Upstream of Fairfield Road Culverts


Downstream of Wynnum Road bridge/culverts

| | | | |
|---|---|---|---|
| **Creek:** | Moolabin Creek | **Immunity Rating:** | > 100-yr ARI |
| **Location:** | Curzon Street | | > 1 % AEP |

DATE OF SUSPENSION OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: ML80

BCC ASSET ID: Unknown

MODEL ID: S10

AMTD (m): 1268

STRUCTURE DESCRIPTION: Concrete Bridge

STRUCTURE SIZE: 3 spans each 6.15m

For Culverts: Number of cells/pipes & sizes — For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) -0.6 — U/S OBVERT LEVEL (m) 5.4

D/S INVERT LEVEL (m) -0.6 — D/S OBVERT LEVEL (m) 5.4

For culverts give floor level — For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING: N/A

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 12.15 — PIER WIDTH (m): 2 @ 0.36

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 5.31 (on approach road to structure)

HEIGHT OF GUARDRAIL/HANDRAIL: 1

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Top of Steel Handrail ~ RL 7.08

PLAN NUMBER: W4438

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1963

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Differing level information between drawings and field book sketches

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Curzon Street |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level (m AHD) | PEAK D/S Water Level (m AHD) | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) Weir | VELOCITY (m/s) Structure |
|---|---|---|---|---|---|---|---|---|
| 2000-yr (0.05%) | 207.0 | 5.94 | 5.45 | 0.49 | 161 | 0.9 | 0.4 | 3.5 |
| 500-yr (0.2%) | 137.2 | 4.79 | 4.44 | 0.35 | 0 | 0.0 | 0.0 | 3.1 |
| 100-yr (0.1%) | 104.6 | 4.48 | 4.25 | 0.23 | 0 | 0.0 | 0.0 | 2.5 |
| 50-yr (0.2%) | 90.3 | 4.33 | 4.15 | 0.18 | 0 | 0.0 | 0.0 | 2.3 |
| 20-yr (5%) | 76.1 | 4.15 | 4.02 | 0.13 | 0 | 0.0 | 0.0 | 2.0 |
| 10-yr (10%) | 65.5 | 3.95 | 3.89 | 0.06 | 0 | 0.0 | 0.0 | 1.7 |
| 5-yr (20%) | 57.3 | 3.90 | 3.79 | 0.11 | 0 | 0.0 | 0.0 | 1.6 |
| 2-yr (50%) | 41.2 | 3.54 | 3.53 | 0.01 | 0 | 0.0 | 0.0 | 1.3 |

Notes:
Weir velocity is the average across the entire flooded width at peak flood level
Structure velocity is a peak average across the bridge opening

| | |
|---|---|
| **Creek:** | Moolabin Creek |
| **Location:** | Curzon Street |


Upstream of Curzon Street Bridge


Downstream of Curzon Street Bridge

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Ainsworth Street |

| | |
|---|---|
| **Immunity Rating:** | 100-yr ARI |
| | 1 % AEP |

| | |
|---|---|
| DATE OF SURVEY: Circa 1998 | UBD REF: N/A |
| SURVEYED CROSS SECTION ID: R550 | BCC ASSET ID: B0040 |
| MODEL ID: S11 | AMTD (m): 4248 |

STRUCTURE DESCRIPTION: Steel Footbridge

STRUCTURE SIZE: Single Span of 24.4m

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

| | | | |
|---|---|---|---|
| U/S INVERT LEVEL (m) | 17.69 | U/S OBVERT LEVEL (m) | 20.86 |
| D/S INVERT LEVEL (m) | 17.69 | D/S OBVERT LEVEL (m) | 20.86 |

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING: N/A

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

| | | | |
|---|---|---|---|
| WEIR WIDTH (m): | 1.65 | PIER WIDTH (m): | N/A - single span |

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 21.69 (on bridge)<br>19.83 (on adjacent road)

HEIGHT OF GUARDRAIL/HANDRAIL: 1.05

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: 2 rails of 45 dia. GI Pipe

PLAN NUMBER: W4135

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1977

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Perched bridge at southern end

| Creek: | Rocky Waterholes |
|---|---|
| Location: | Ainsworth Street |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level (m AHD) | PEAK D/S Water Level (m AHD) | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) Weir | VELOCITY (m/s) Structure |
|---|---|---|---|---|---|---|---|---|
| 2000-yr (0.05%) | 62.3 | 21.11 | 20.88 | 0.23 | 222 | 0.6 | 0.1 | 2.1 |
| 500-yr (0.2%) | 55.2 | 20.86 | 20.85 | 0.01 | 188 | 0.4 | 0.1 | 2.1 |
| 100-yr (0.1%) | 42.3 | 20.72 | 20.71 | 0.01 | 0 | 0.0 | 0.0 | 1.8 |
| 50-yr (0.2%) | 37.2 | 20.62 | 20.60 | 0.02 | 0 | 0.0 | 0.0 | 1.7 |
| 20-yr (5%) | 32.5 | 20.49 | 20.47 | 0.02 | 0 | 0.0 | 0.0 | 1.6 |
| 10-yr (10%) | 27.1 | 20.31 | 20.30 | 0.01 | 0 | 0.0 | 0.0 | 1.4 |
| 5-yr (20%) | 23.2 | 20.16 | 20.15 | 0.01 | 0 | 0.0 | 0.0 | 1.4 |
| 2-yr (50%) | 16.6 | 19.87 | 19.86 | 0.01 | 0 | 0.0 | 0.0 | 1.2 |

Notes:
Weir velocity is the average across the entire flooded width at peak flood level
Structure velocity is a peak average across the bridge opening

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Ainsworth Street |

Upstream of Ainsworth Pedestrian Bridge

Downstream of  Ainsworth Pedestrian Bridge

| | | | |
|---|---|---|---|
| **Creek:** | Rocky Waterholes | **Immunity Rating:** | 5-yr ARI |
| **Location:** | McCarthy Road | | 18 % AEP |

DATE OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: R498

BCC ASSET ID: C0595B

MODEL ID: S12

AMTD (m): 3783

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 2 / 3050mm w x 1800mm h RCBC + 3 / 1680mm RCP

For Culverts: Number of cells/pipes & sizes — For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 14.98

U/S OBVERT LEVEL (m) 16.78 RCBC, 16.68 RCP

D/S INVERT LEVEL (m) 14.90 RCBC, 14.95 RCP

D/S OBVERT LEVEL (m) 16.70 RCBC, 16.63 RCP

For culverts give floor level — For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 18.4 RCP, 18.9 RCBC

LENGTH OF CULVERT AT OBVERT (m): 18.4 RCP, 18.9 RCBC

TYPE OF LINING: Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 18.9

PIER WIDTH (m): N/A - multi-cell culvert

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 17.36

HEIGHT OF GUARDRAIL/HANDRAIL: 1.03

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: 2 of 60 dia. GI Pipe

PLAN NUMBER: W5974

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1977

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | McCarthy Road |

| **ARI (AEP %)** | **PEAK DISCHARGE (m3/s)** | **PEAK U/S Water Level (m AHD)** | **PEAK D/S Water Level** | **AFFLUX (mm)** | **MAX WIDTH OF WEIR FLOW (m)** | **MAX DEPTH OF WEIR FLOW (m)** | **VELOCITY (m/s)** | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | **Weir** | **Structure** |
| 2000-yr (0.05%) | 89.1 | 18.51 | 18.46 | 0.05 | 80 | 0.9 | 1.5 | 2.3 |
| 500-yr (0.2%) | 66.5 | 18.31 | 18.17 | 0.14 | 72 | 0.7 | 0.9 | 2.5 |
| 100-yr (0.1%) | 52.5 | 18.06 | 17.93 | 0.13 | 60 | 0.5 | 0.7 | 2.5 |
| 50-yr (0.2%) | 46.5 | 17.92 | 17.80 | 0.12 | 52 | 0.4 | 0.5 | 2.5 |
| 20-yr (5%) | 40.2 | 17.77 | 17.65 | 0.12 | 44 | 0.2 | 0.2 | 2.3 |
| 10-yr (10%) | 34.8 | 17.58 | 17.49 | 0.09 | 20 | 0.01 | 0.1 | 2.1 |
| 5-yr (20%) | 30.7 | 17.42 | 17.36 | 0.06 | 0 | 0.0 | 0.0 | 1.8 |
| 2-yr (50%) | 23.0 | 17.13 | 17.09 | 0.04 | 0 | 0.0 | 0.0 | 1.4 |

Notes:
Weir velocity is the average across the entire flooded width at peak flood level
Structure velocity is the peak within the culvert barrel

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | McCarthy Road |

Upstream of McCarthy Road Culverts

Downstream of McCarthy Road Culverts

| | | | |
|---|---|---|---|
| **Creek:** | Rocky Waterholes | **Immunity Rating:** | 2-yr ARI |
| **Location:** | Assembly St | | 50 % AEP |

DATE OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: R400

BCC ASSET ID: B1130

MODEL ID: S13

AMTD (m): 2967

STRUCTURE DESCRIPTION: Concrete Bikeway Bridge

STRUCTURE SIZE: 2 Spans, 1 @ 9.9m and 1 @ 10.0m

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 10.87 U/S OBVERT LEVEL (m) 13.42

D/S INVERT LEVEL (m) 10.87 D/S OBVERT LEVEL (m) 13.42

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING: N/A

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 2.8 PIER WIDTH (m): 0.45 dia

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 13.68 (on bridge)
12.97 (on adjacent road)

HEIGHT OF GUARDRAIL/HANDRAIL: 1.06

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: 45mm dia. Steel pipe,

PLAN NUMBER: W4135

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1981

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Assembly St |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | (m AHD) | | | | | Weir | Structure |
| 2000-yr (0.05%) | 113.4 | 14.37 | 13.98 | 0.39 | 202 | 1.0 | 0.9 | 3.5 |
| 500-yr (0.2%) | 79.6 | 14.22 | 13.76 | 0.46 | 190 | 0.9 | 0.7 | 3.6 |
| 100-yr (0.1%) | 61.5 | 14.12 | 13.63 | 0.49 | 184 | 0.8 | 0.6 | 3.6 |
| 50-yr (0.2%) | 53.8 | 14.07 | 13.57 | 0.50 | 180 | 0.7 | 0.5 | 3.6 |
| 20-yr (5%) | 45.4 | 14.00 | 13.50 | 0.50 | 174 | 0.7 | 0.4 | 3.6 |
| 10-yr (10%) | 37.3 | 13.90 | 13.44 | 0.46 | 168 | 0.6 | 0.3 | 3.6 |
| 5-yr (20%) | 32.3 | 13.78 | 13.43 | 0.35 | 166 | 0.5 | 0.2 | 3.6 |
| 2-yr (50%) | 25.7 | 13.38 | 13.36 | 0.02 | 4 | 0.1 | 0.1 | 3.4 |

Notes:

Weir velocity is the average across the entire flooded width at peak flood level

Structure velocity is a peak average across the bridge opening

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Assembly St |

Upstream of Assembly Street Footbridge

Downstream of Assembly Street Footbridge

| | | | |
|---|---|---|---|
| **Creek:** | Rocky Waterholes | **Immunity Rating:** | > 100-yr ARI |
| **Location:** | Railway Bridge in TAFE | | > 1 % AEP |

DATE OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: R340

BCC ASSET ID: Unknown

MODEL ID: S14

AMTD (m): 2542

STRUCTURE DESCRIPTION: Timber Railway Bridge

STRUCTURE SIZE: 6 Spans @ 4.3m centres

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 8.79 U/S OBVERT LEVEL (m) 12.3

D/S INVERT LEVEL (m) 8.79 D/S OBVERT LEVEL (m) 12.3

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING: N/A

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 5.5 PIER WIDTH (m): 0.5

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 13.03

HEIGHT OF GUARDRAIL/HANDRAIL: 1

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: 50mm sq. Steel handrail

PLAN NUMBER: Unknown

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Unknown

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Railway Bridge in TAFE |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | (m AHD) | | | | | Weir | Structure |
| 2000-yr (0.05%) | 121.5 | 12.82 | 12.48 | 0.34 | 0 | 0.0 | 0.0 | 2.1 |
| 500-yr (0.2%) | 87.1 | 12.02 | 11.89 | 0.13 | 0 | 0.0 | 0.0 | 2.0 |
| 100-yr (0.1%) | 67.1 | 11.68 | 11.59 | 0.09 | 0 | 0.0 | 0.0 | 2.1 |
| 50-yr (0.2%) | 58.5 | 11.50 | 11.41 | 0.09 | 0 | 0.0 | 0.0 | 2.0 |
| 20-yr (5%) | 48.8 | 11.27 | 11.18 | 0.09 | 0 | 0.0 | 0.0 | 2.0 |
| 10-yr (10%) | 40.4 | 11.05 | 10.97 | 0.08 | 0 | 0.0 | 0.0 | 2.0 |
| 5-yr (20%) | 34.9 | 10.89 | 10.81 | 0.08 | 0 | 0.0 | 0.0 | 2.1 |
| 2-yr (50%) | 27.5 | 10.65 | 10.57 | 0.08 | 0 | 0.0 | 0.0 | 2.0 |

Notes:

Weir velocity is the average across the entire flooded width at peak flood level

Structure velocity is a peak average across the bridge opening

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Railway Bridge in TAFE |


Upstream of Railway Bridge in TAFE


Downstream of Railway Bridge in TAFE

| | | | |
|---|---|---|---|
| **Creek:** | Rocky Waterholes | **Immunity Rating:** | > 100-yr ARI |
| **Location:** | Footbridge in TAFE | | > 1 % AEP |

DATE OF SURVEY: N/A

UBD REF: N/A

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID: Unknown

MODEL ID: S15

AMTD (m): 2305

STRUCTURE DESCRIPTION: Metal Footbridge

STRUCTURE SIZE: Single span @ 20m

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 7.5 U/S OBVERT LEVEL (m) ~ 11.3

D/S INVERT LEVEL (m) 7.5 D/S OBVERT LEVEL (m) ~ 11.3

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING: N/A

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 2 PIER WIDTH (m): N/A - single span

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): Unknown

HEIGHT OF GUARDRAIL/HANDRAIL: 1.5

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Metal rail with fine mesh

PLAN NUMBER: Unknown

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Between 2001 to 2005

HAS THE STRUCTURE BEEN UPGRADED? Yes Between 2001 to 2005

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Footbridge in TAFE |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | (m AHD) | | | | | Weir | Structure |
| 2000-yr (0.05%) | 141.4 | 11.36 | 11.30 | 0.06 | 0.00 | 0.0 | 0 | 2.8 |
| 500-yr (0.2%) | 102.4 | 10.60 | 10.58 | 0.02 | 0.00 | 0.0 | 0 | 2.8 |
| 100-yr (0.1%) | 79.8 | 10.18 | 10.16 | 0.02 | 0.00 | 0.0 | 0 | 2.7 |
| 50-yr (0.2%) | 69.3 | 9.97 | 9.95 | 0.02 | 0.00 | 0.0 | 0 | 2.6 |
| 20-yr (5%) | 56.8 | 9.72 | 9.70 | 0.02 | 0.00 | 0.0 | 0 | 2.5 |
| 10-yr (10%) | 47.8 | 9.51 | 9.50 | 0.01 | 0.00 | 0.0 | 0 | 2.5 |
| 5-yr (20%) | 42.4 | 9.39 | 9.37 | 0.02 | 0.00 | 0.0 | 0 | 2.4 |
| 2-yr (50%) | 32.6 | 9.14 | 9.12 | 0.02 | 0.00 | 0.0 | 0 | 2.3 |

Notes:
Weir velocity is the average across the entire flooded width at peak flood level
Structure velocity is a peak average across the bridge opening

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Footbridge in TAFE |


Upstream of Footbridge in TAFE


Downstream of Footbridge in TAFE

| | | | | |
|---|---|---|---|---|
| **Creek:** | Rocky Waterholes | | **Immunity Rating:** | > 100-yr ARI |
| **Location:** | Beaudesert Road | | | > 1 % AEP |

DATE OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: R280

BCC ASSET ID: C0072B

MODEL ID: S16

AMTD (m): 2052

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 6 / 2140mm w x 2170mm h RCBC

For Culverts: Number of cells/pipes & sizes    For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 5.88    U/S OBVERT LEVEL (m) 8.05

D/S INVERT LEVEL (m) 5.74    D/S OBVERT LEVEL (m) 7.91

For culverts give floor level    For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 41.72

LENGTH OF CULVERT AT OBVERT (m): 41.72

TYPE OF LINING: Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 41.72    PIER WIDTH (m): N/A - multi-cell culvert

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 9.7

HEIGHT OF GUARDRAIL/HANDRAIL: 1

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: 2 of 150 sq steel handrails

PLAN NUMBER: MRD153841

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1972

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| **Creek:** | Rocky Waterholes |
|---|---|
| **Location:** | Beaudesert Road |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | (m AHD) | | | | | Weir | Structure |
| 2000-yr (0.05%) | 150.1 | 10.54 | 9.27 | 1.27 | 86 | 0.7 | 0.7 | 6.2 |
| 500-yr (0.2%) | 107.2 | 9.6 | 9.01 | 0.59 | 0 | 0 | 0.0 | 3.8 |
| 100-yr (0.1%) | 83.2 | 9.05 | 8.75 | 0.30 | 0 | 0 | 0.0 | 3.0 |
| 50-yr (0.2%) | 72.4 | 8.79 | 8.59 | 0.20 | 0 | 0 | 0.0 | 2.6 |
| 20-yr (5%) | 59.4 | 8.48 | 8.36 | 0.12 | 0 | 0 | 0.0 | 2.1 |
| 10-yr (10%) | 50.4 | 8.27 | 8.2 | 0.07 | 0 | 0 | 0.0 | 2.1 |
| 5-yr (20%) | 45.3 | 8.13 | 8.1 | 0.03 | 0 | 0 | 0.0 | 2.1 |
| 2-yr (50%) | 34.1 | 7.86 | 7.84 | 0.02 | 0 | 0 | 0.0 | 1.8 |

Notes:

Weir velocity is the average across the entire flooded width at peak flood level

Structure velocity is the peak within the culvert barrel

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Beaudesert Road |

Upstream of Beaudesert Road Culverts


Downstream of Beaudesert Road Culverts

| | | | |
|---|---|---|---|
| **Creek:** | Rocky Waterholes | **Immunity Rating:** | 20-yr ARI |
| **Location:** | Gladstone Road | | 5 % AEP |

DATE OF SUBVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: R230

BCC ASSET ID: C0071B

MODEL ID: S17

AMTD (m): 1936

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 6 / 2130 mm w x 2180mm h RCBC

For Culverts: Number of cells/pipes & sizes — For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 5.47 — U/S OBVERT LEVEL (m) 7.65

D/S INVERT LEVEL (m) 5.26 — D/S OBVERT LEVEL (m) 7.44

For culverts give floor level — For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 25.75

LENGTH OF CULVERT AT OBVERT (m): 25.75

TYPE OF LINING: Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 25.75 — PIER WIDTH (m): N/A - multi-cell culvert

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 8.00 (on southern approach road)

HEIGHT OF GUARDRAIL/HANDRAIL: 1

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: 2 rails of 100 sq steel

PLAN NUMBER: MRD153846

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1972

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Gladstone Road |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | (m AHD) | | | | | Weir | Structure |
| 2000-yr (0.05%) | 147.9 | 9.05 | 8.86 | 0.19 | 235 | 1.09 | 0.5 | 3.1 |
| 500-yr (0.2%) | 107.2 | 8.79 | 8.57 | 0.22 | 218 | 0.82 | 0.2 | 3.1 |
| 100-yr (0.1%) | 83.2 | 8.56 | 8.37 | 0.19 | 122 | 0.55 | 0.1 | 2.8 |
| 50-yr (0.2%) | 72.1 | 8.41 | 8.25 | 0.16 | 72 | 0.36 | 0.1 | 2.6 |
| 20-yr (5%) | 59.4 | 8.19 | 8.10 | 0.09 | 0 | 0.00 | 0.0 | 2.1 |
| 10-yr (10%) | 50.4 | 8.04 | 7.98 | 0.06 | 0 | 0.00 | 0.0 | 1.8 |
| 5-yr (20%) | 45.3 | 7.95 | 7.91 | 0.04 | 0 | 0.00 | 0.0 | 1.6 |
| 2-yr (50%) | 34.1 | 7.71 | 7.70 | 0.01 | 0 | 0.00 | 0.0 | 1.2 |

Notes:

Weir velocity is the average across the entire flooded width at peak flood level

Structure velocity is the peak within the culvert barrel

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Gladstone Road |

Upstream of Gladstone Road Culverts

Downstream of Gladstone Road Culverts

| | | | |
|---|---|---|---|
| **Creek:** | Rocky Waterholes | **Immunity Rating:** | > 100-yr ARI |
| **Location:** | Ipswich Road | | > 1 % AEP |

DATE OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: R150

BCC ASSET ID: C0235P

MODEL ID: S18

AMTD (m): 1108

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 4 / 3150mm dia Corrugated Iron Pipes

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 2.2 U/S OBVERT LEVEL (m) 5.35

D/S INVERT LEVEL (m) 2.08 D/S OBVERT LEVEL (m) 5.23

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 55.7

LENGTH OF CULVERT AT OBVERT (m): 55.7

TYPE OF LINING: Corrugated Iron

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 55.7 PIER WIDTH (m): N/A - multi-cell culvert

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 5.5 (Muriel Ave. underpass)

HEIGHT OF GUARDRAIL/HANDRAIL: N/A

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: N/A

PLAN NUMBER: W4857

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1974

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Ipswich Road |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | (m AHD) | | | | | Weir | Structure |
| 2000-yr (0.05%) | 192.7 | 7.79 | 6.91 | 0.88 | 30 | 1.63 | 3.9 | 3.2 |
| 500-yr (0.2%) | 139.1 | 7.11 | 6.43 | 0.68 | 28 | 1.13 | 3.1 | 2.9 |
| 100-yr (0.1%) | 104.9 | 6.66 | 6.10 | 0.56 | 24 | 0.88 | 2.1 | 2.7 |
| 50-yr (0.2%) | 92.1 | 6.39 | 5.88 | 0.51 | 20 | 0.69 | 1.7 | 2.6 |
| 20-yr (5%) | 79.7 | 6.02 | 5.60 | 0.42 | 10 | 0.44 | 1.4 | 2.5 |
| 10-yr (10%) | 70.2 | 5.71 | 5.37 | 0.34 | 7 | 0.18 | 0.8 | 2.3 |
| 5-yr (20%) | 63.5 | 5.46 | 5.21 | 0.25 | 0 | 0.00 | 0 | 2.1 |
| 2-yr (50%) | 48.3 | 5.03 | 4.88 | 0.15 | 0 | 0.00 | 0 | 1.7 |

Notes:

Weir flow is on Muriel Avenue, not Ipswich Road

Weir velocity is the average across the entire flooded width at peak flood level

Structure velocity is the peak within the culvert barrel

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Ipswich Road |

Upstream of Ipswich Road Culverts

Downstream of Ipswich Road Culverts

| | | | |
|---|---|---|---|
| **Creek:** | Rocky Waterholes | **Immunity Rating:** | < 2-yr ARI |
| **Location:** | Muriel Avenue | | < 50 % AEP |

DATE OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: R130

BCC ASSET ID: C1881B

MODEL ID: S19

AMTD (m): 1038

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 7 / 2130mm w x height (varies 2250mm to 1900mm) RCBC

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 2.15

U/S OBVERT LEVEL (m) 4.4 to 4.15

D/S INVERT LEVEL (m) 2.15

D/S OBVERT LEVEL (m) 4.4 to 4.15

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 20 to 23.5

LENGTH OF CULVERT AT OBVERT (m): 20 to 23.5

TYPE OF LINING: Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 23.5

PIER WIDTH (m): N/A - multi-cell culvert

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 4.25 (Railway underpass)

HEIGHT OF GUARDRAIL/HANDRAIL: 1

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: 25 dia bars @ 150 centres

PLAN NUMBER: W3256 & S12192

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1968

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Muriel Avenue |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | (m AHD) | | | | | Weir | Structure |
| 2000-yr (0.05%) | 192.7 | 6.91 | 6.91 | 0.00 | N/A | 2.71 | N/A | 1.1 |
| 500-yr (0.2%) | 139.1 | 6.43 | 6.42 | 0.01 | N/A | 2.23 | N/A | 1.5 |
| 100-yr (0.1%) | 104.9 | 6.10 | 6.09 | 0.01 | N/A | 1.90 | N/A | 1.4 |
| 50-yr (0.2%) | 92.1 | 5.88 | 5.87 | 0.01 | N/A | 1.69 | N/A | 1.3 |
| 20-yr (5%) | 79.7 | 5.60 | 5.58 | 0.02 | N/A | 1.40 | N/A | 1.3 |
| 10-yr (10%) | 70.2 | 5.37 | 5.36 | 0.01 | N/A | 1.18 | N/A | 1.3 |
| 5-yr (20%) | 63.5 | 5.21 | 5.20 | 0.01 | N/A | 0.99 | N/A | 1.3 |
| 2-yr (50%) | 48.3 | 4.88 | 4.85 | 0.03 | N/A | 0.67 | N/A | 1.2 |

Notes:

Structure velocity is the peak within the culvert barrel

Weir width and velocity not as shown as predominant weir flow is not perpendicular to the road

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Muriel Avenue |

Upstream of Muriel Avenue Culverts


Downstream of Muriel Avenue Culverts

| | | | |
|---|---|---|---|
| **Creek:** | Rocky Waterholes | **Immunity Rating:** | < 2-yr ARI |
| **Location:** | Muriel Avenue Footbridge | | < 50 % AEP |

DATE OF SURVEY: Circa 1998

UBD REF: N/A

SURVEYED CROSS SECTION ID: R110

BCC ASSET ID: B2770

MODEL ID: S21

AMTD (m): 988

STRUCTURE DESCRIPTION: Concrete Footbridge

STRUCTURE SIZE: Single Span of 20.29m

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 2.39

U/S OBVERT LEVEL (m) 4.16 to 5.21

D/S INVERT LEVEL (m) 2.39

D/S OBVERT LEVEL (m) 4.16 to 5.21

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m):

LENGTH OF CULVERT AT OBVERT (m):

TYPE OF LINING:

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 2.05

PIER WIDTH (m): N/A - single span

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 4.59

HEIGHT OF GUARDRAIL/HANDRAIL: 1

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: 25 dia bars @150 centres

PLAN NUMBER: W3256-793

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1968

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Muriel Avenue Footbridge |

| **ARI (AEP %)** | **PEAK DISCHARGE (m3/s)** | **PEAK U/S Water Level** | **PEAK D/S Water Level** | **AFFLUX (mm)** | **MAX WIDTH OF WEIR FLOW (m)** | **MAX DEPTH OF WEIR FLOW (m)** | **VELOCITY (m/s)** | |
|---|---|---|---|---|---|---|---|---|
| | | **(m AHD)** | | | | | **Weir** | **Structure** |
| 2000-yr (0.05%) | 116.0 | 6.83 | 6.67 | 0.16 | 26 | 2.27 | 2.4 | 3.1 |
| 500-yr (0.2%) | 118.6 | 6.27 | 5.70 | 0.57 | 19 | 1.79 | 2.2 | 3.2 |
| 100-yr (0.1%) | 103.6 | 5.96 | 5.39 | 0.57 | 13 | 1.47 | 1.9 | 3.0 |
| 50-yr (0.2%) | 91.9 | 5.75 | 5.26 | 0.49 | 9 | 1.25 | 1.8 | 2.8 |
| 20-yr (5%) | 79.1 | 5.48 | 5.09 | 0.39 | 3 | 0.96 | 1.5 | 2.6 |
| 10-yr (10%) | 69.9 | 5.27 | 4.95 | 0.32 | 3 | 0.74 | 1.3 | 2.3 |
| 5-yr (20%) | 62.8 | 5.12 | 4.84 | 0.28 | 3 | 0.58 | 1.1 | 2.1 |
| 2-yr (50%) | 47.9 | 4.82 | 4.58 | 0.24 | 0 | 0.23 | 0.5 | 1.7 |

Notes:

Weir velocity is the average across the 1d domain, which includes the structure

Structure velocity is a peak average across the bridge opening

Peak discharge is only at the structure, noting there is considerable bypass flow in extreme events

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Muriel Avenue Footbridge |


Upstream of Footbridge beside Fairfield Road


Downstream of Footbridge beside Fairfield Road

| | | | | |
|---|---|---|---|---|
| **Creek:** | Rocky Waterholes | | **Immunity Rating:** | 50-yr ARI |
| **Location:** | Fairfield Road Bridge | | | 2 % AEP |

| | |
|---|---|
| DATE OF SURVEY: Circa 1998 | UBD REF: N/A |
| SURVEYED CROSS SECTION ID: R90 | BCC ASSET ID: B2800 & B0720 |
| MODEL ID: S22 | AMTD (m): 962 |

STRUCTURE DESCRIPTION: Road Bridge

STRUCTURE SIZE: Upstream Carriageway: Two span bridge 1 @ 6.85 m and 1 @ 6.45mDownstream Carriage

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

| | | | |
|---|---|---|---|
| U/S INVERT LEVEL (m) | 2.24 | U/S OBVERT LEVEL (m) | 5.64 |
| D/S INVERT LEVEL (m) | 1.73 | D/S OBVERT LEVEL (m) | 5.65 |

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m):

LENGTH OF CULVERT AT OBVERT (m):

TYPE OF LINING:

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 19.9 PIER WIDTH (m): 0.4

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 5.75 (on southern approach)

HEIGHT OF GUARDRAIL/HANDRAIL: 1

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: 2 of 90 dia GI Pipe rails

PLAN NUMBER: W6086 & W6683

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1979

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

| Creek: | Rocky Waterholes |
|---|---|
| Location: | Fairfield Road Bridge |

| ARI (AEP %) | PEAK DISCHARGE (m3/s) | PEAK U/S Water Level | PEAK D/S Water Level | AFFLUX (mm) | MAX WIDTH OF WEIR FLOW (m) | MAX DEPTH OF WEIR FLOW (m) | VELOCITY (m/s) | |
|---|---|---|---|---|---|---|---|---|
| | | (m AHD) | | | | | Weir | Structure |
| 2000-yr (0.05%) | 176.8 | 6.65 | 6.38 | 0.27 | 392 | 1.02 | 0.3 | 2.4 |
| 500-yr (0.2%) | 131.6 | 5.77 | 5.61 | 0.16 | 66 | 0.58 | 0.5 | 2.5 |
| 100-yr (0.1%) | 102.4 | 5.33 | 5.31 | 0.02 | 28 | 0.28 | 0.3 | 2.3 |
| 50-yr (0.2%) | 91.9 | 5.20 | 5.18 | 0.02 | 0 | 0.00 | 0 | 2.2 |
| 20-yr (5%) | 79.1 | 5.04 | 5.03 | 0.01 | 0 | 0.00 | 0 | 2.4 |
| 10-yr (10%) | 69.8 | 4.90 | 4.89 | 0.01 | 0 | 0.00 | 0 | 2.2 |
| 5-yr (20%) | 62.8 | 4.80 | 4.79 | 0.01 | 0 | 0.00 | 0 | 2.0 |
| 2-yr (50%) | 47.8 | 4.58 | 4.57 | 0.01 | 0 | 0.00 | 0 | 2.4 |

Notes:
Weir velocity is the average across the entire flooded width at peak flood level
Structure velocity is a peak average across the bridge opening

| | |
|---|---|
| **Creek:** | Rocky Waterholes |
| **Location:** | Fairfield Road Bridge |


Upstream of Fairfield Road Bridge


Downstream of Fairfield Road Bridge

# Appendix I: External Peer Review Documentation

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1 June 2015

Brisbane City Council
City Projects Office
Green Square, Level 1
505 St Pauls Terrace
Fortitude Valley
Qld 4006

Attention: Scott Glover

Dear Scott

**RE: MOOLABIN AND ROCKY WATERHOLES CREEK FLOOD MODELLING PEER REVIEW**

## Background

BMT WBM was commissioned by Council to undertake a peer review of the Moolabin and Rocky Waterholes Creek flood modelling prepared as part of the Moolabin-Rocky Waterholes Creek Flood Study. This letter documents the outcomes of BMT WBM's review.

At the commencement of the review process, Council submitted the following data to BMT WBM:

- Hydrological models;
- Hydraulic models including all model output files;
- GIS data;
- Site photographs; and
- Initial reporting.

These data were reviewed and initial feedback on the calibration modelling was provided to Council by email (dated 24th February 2015). The design event modelling was subsequently provided for review and feedback was provided to Council by email (dated 20th April 2015).

Generally, no errors of significant consequence were identified in the models. Some minor errors were identified and rectified following initial feedback provided to Council – these are not discussed in this letter as they are minor and have been resolved. Residual matters arising from the review are discussed in this letter.

## Overview of the Modelling Approach

Hydrological models were developed using XP-RAFTS. The structure of the XP-RAFTS models and the sub-catchment parameters has been reviewed. Hydraulic models of Moolabin and Rocky Waterholes Creek were developed using TUFLOW. A 4m computational grid cell size was used. The upper and middle reaches of the creeks were modelled in 1D (i.e. upstream of the golf course at the Brisbane Golf

Club) and linked to the 2D model domain of the floodplain. The lower reach of the creek system from the vicinity of the golf course to the confluence with Oxley Creek was modelled in 2D.

**Model Performance**

The model performance has been checked in relation to: mass balance error, negative depth warnings, and instability. The model performance is considered suitable. It is noted that Council has also assessed the model performance in relation to replication of historical events (calibration and verification) and bridge structures have been compared to equivalent HEC-RAS models. Council's acceptable tolerance for calibration is 0.15m variance for peak flood levels at stream gauges and 0.3m variance for peak flood levels at maximum height gauges. This correlates with standard industry practice.

**Residual Matters**

A few matters have been identified during the review that should be borne in mind when using the models in future or interpreting results from the models. These are discussed below.

Moolabin Creek commences at Ipswich Road near the junction with Beaudesert Road. Overland flow has been modelled upstream of this location for the Existing Scenario. The main stormwater trunk line was modelled, but not any spur lines. Local sub-catchment flows were inserted directly into the main trunk line, with pits connecting the 1D trunk line to the 2D domain at manholes to enable surcharging and overland flow. This approach forces the sub-catchment runoff into the main trunk line without considering the location, capacity and efficiency of the spur line inlets that collect runoff from the sub-catchment and feed the main trunk line. As such, the overland flow behaviour has not been fully modelled and the flood risk from overland flow may be more extensive than the results of the model indicate. Council adopted this approach given that the mapped results will be compared against Council's overland flow mapping in this area, and will be incorporated with the existing overland flow mapping if considered an improvement on the existing mapping.

The overland flow and trunk drain upstream of Moolabin Creek is not modelled in TUFLOW for the Ultimate Scenario. Instead, the upstream inflow for Moolabin Creek was extracted from the lumped catchment flow at the corresponding location in the XP-RAFTS model. Whereas, the upper Moolabin Creek catchment was modelled in TUFLOW using local sub-catchment inflows from the XP-RAFTS model for the Existing Scenario. As such, there is an inconsistency in the application of the hydrologic model inflows at the upstream end of Moolabin Creek across the two scenarios. This inconsistency is unavoidable given the requirement to model overland flow upstream of Moolabin Creek in the Existing Scenario and not in the Ultimate Scenario. A comparison of TUFLOW and XP-RAFTS flows is provided in Council's report to give an indication of the difference in flow.

There are two areas where flood waters in the extreme (0.05% AEP and PMF) events strike the edge of the model extent. As such, the flow path is blocked causing ponding of flood water. This may lead to a small overestimate of flood level and flood inundation extent. Since this occurs at the edge of the catchment the flood inundation extent is not underestimated – unblocked flows would spill into the adjacent catchment. While this matter may impact on local catchment flood levels, the broader Oxley Creek flood levels are higher in the impacted areas. Therefore, Oxley Creek flood levels will be used for flood risk management decisions in the impacted areas and Council decided there was little value in undertaking further work to resolve this matter.

**Limitations of the Review**

This review focussed on scrutinising the design and performance of the models developed by Council. The scope of the review does not include the underlying data used to develop the model or the broader flood study methodology and procedure. For example, the accuracy of the topographic data, land use mapping (based on Brisbane City Council's City Plan and refined using aerial imagery), structure details and historic flood data has not been explicitly checked. If supplied information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions may change. As a consequence, BMT WBM provides no liability to the accuracy or the precision of the supplied data. All liability to do with the assumptions that rely on the accuracy or the precision of the supplied data rest with Brisbane City Council.

While the design and performance of the models used for calibration has been reviewed, the calibration and verification exercise has not been reviewed (for example, BMT WBM has not inspected modelled water levels at Maximum Height Gauge locations or reviewed comparisons of observed data versus modelled results).

**Conclusion**

The flood modelling undertaken as part of the Moolabin and Rocky Waterholes Creek Flood Study complies with current industry practice, and is considered suitable for the purposes of the study. Limitations to this endorsement as well as matters that should be considered when interpreting model results or future use of the model are discussed in this letter.

Yours Faithfully
**BMT WBM**

**Richard Sharpe**
Senior Flood Engineer

**Jo Tinnion RPEQ (11395)**

Supervising Engineer[1]:

*[1] Supervising engineer signoff is based on information provided by Richard Sharpe and confidence in Richard's ability to undertake the review. Trust has been placed in the validity and completeness of the information provided by Richard.*