Moggill Creek Flood Study

Volume 1 of 2

Flood Study Report

Prepared by Brisbane City Council's, City Projects Office

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Flood Study Report Disclaimer

BRISBANE CITY

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Brisbane City Council
City Projects Office
Level 1, 505 St Pauls Terrace
Fortitude Valley QLD 4006
GPO Box 1434
Brisbane QLD 4000

Telephone 07 3403 8888
Facsimile 07 3334 0071

Notice

Note: *The Moggill 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 BCC flood studies within the catchment were undertaken in 1994 (Moggill, Gold and Gap Creeks) and 1999 (McKay Brook).

The entire Moggill Creek Catchment has a total area of 65.8 km^2 of which the catchment centroid is located approximately 15 km west of the Brisbane CBD. Moggill Creek is the largest tributary with a total catchment area of 39 km^2, followed by Gold Creek (17.9 km^2), Gap Creek (6.6 km^2) and McKay Brook (2.4 km^2). The total combined catchment area includes the suburbs of Upper Brookfield, Brookfield and Kenmore Hills.

Project Objectives

The primary objectives of the project were as follows:

- Update the 1994 Moggill Creek and 1999 McKay Brook 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.
- Estimate design and rare / extreme flood magnitudes.
- Determine flood levels for the design and rare / extreme events.
- Quantify the impacts of Minimum Riparian Corridor (MRC) and filling / development outside the “Modelled Flood Corridor.”
- Produce flood extent mapping for the selected range of design and rare / extreme events.
- Quantify the sensitivity of climate variability on flooding within the catchment.

Project Elements

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

Model Set-up and Calibration

Hydrologic and hydraulic models of the Moggill Creek Catchment have been developed using the URBS 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 event(s) to confirm the calibrated model is suitable for use in simulating synthetic design storm events.

Calibration of the URBS and TUFLOW models was undertaken utilising three historical storms; namely, May 2015, May 2009 and November 2008. Verification of the URBS and TUFLOW models utilised the January 2013 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, the verification achieved an acceptable correlation between the simulated and historical records for the single verification event.

Given the results of the calibration and verification process were quite reasonable, the URBS 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 (50 % AEP) 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 "Modelled Flood Corridor" in order to simulate potential development.

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

- Peak design flood discharges
- Critical storm durations at selected locations
- Peak design flood levels at 100 m intervals along the AMTD line
- Peak design flood extent mapping (Scenario 1 only)
- Hydraulic structure flood immunity

As part of the required sensitivity analysis a climate variability 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 variability impacts within the catchment will increase the magnitude of flooding, for example:

- Based on current climatic projections, by the year 2050, the 100-yr ARI (1 % AEP) flood levels are likely to be of similar magnitude to the present day 200-yr ARI (0.5 % AEP) flood levels.
- Based on current climatic projections, by the year 2100, the 100-yr ARI (1 % AEP) flood levels are likely to be between the present day 200-yr ARI (0.2 % AEP) and 500-yr ARI (0.2 % AEP) flood levels.
- Based on current climatic projections, by the year 2100, the 200-yr ARI (0.5 % AEP) flood levels are likely to be of similar magnitude to the present day 500-yr ARI (0.2 % AEP) flood levels.

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Glossary of Terms

Term	Definition
2014 ALS Data	This dataset is part of the SEQ 2014 LiDAR capture project and covers an area of approximately 1392 km^2 over Brisbane City Council. This project was undertaken by Fugro Spatial Solutions Pty Ltd on behalf of the Queensland Government.
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.
Flood Planning Area (FPA)	Flood Planning Areas (FPAs) were introduced in BCC City Plan 2014. FPAs define the extent of development filling together with the Waterway Corridor (WC).
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.
Modelled Flood Corridor	The "Modelled Flood Corridor" is the greater extent of the Waterway Corridor (WC) and Flood Planning Areas (FPAs) 1, 2 and 3

Glossary of Terms (cont)

Term	Definition
Probable Maximum Flood (PMF)	An extreme flood deemed to be the largest flood that could conceivably occur at a specific location.
Probable Maximum Precipitation (PMP)	The maximum precipitation (rainfall) that is reasonably estimated to not be exceeded.
URBS	Hydrologic modelling software package.
WBNM	Hydrologic modelling software package.

Adopted ARI to AEP Conversion

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 magnitudes, the term AEP (%) is now preferred by AR&R, in lieu of ARI. The relationship between ARI and AEP can be expressed by the following equation:

$AEP = 1 - \exp(-1 / ARI)$

The use of this equation results in the “Actual AEP” as indicated in the table below. However, it is quite common to see the “Nominal AEP” ($AEP = 1 / ARI$) used for simplicity within the industry.

For the purpose of this study, the “Nominal AEP” has been used. The flood probability will be firstly expressed in ARI and then secondly in brackets by the equivalent “Nominal AEP.”

Event (ARI years)	Actual AEP (%)	Nominal AEP (%)
2	39	50
5	18	20
10	10	10
20	5	5
50	2	2
100	1	1
200	0.5	0.5
500	0.2	0.2
2000	0.05	0.05

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)
FPA	Flood Planning Area
IFD	Intensity Frequency Duration
IL	Initial rainfall loss (mm)
IWL	Initial Water Level (mAHD)
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

Moggill Creek Catchment comprises the major tributaries of Moggill, Gold and Gap Creeks as well as the minor tributary of McKay Brook. The entire Moggill Creek Catchment has a total area of 65.8 km^2 of which the catchment centroid is located approximately 15 km west of the Brisbane CBD. The catchment area includes the suburbs of Upper Brookfield, Brookfield and Kenmore Hills. 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 catchment conditions 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 studies undertaken by BCC are:

- Flood Study of Moggill, Gold and Gap Creeks in 1994 [2]
- Stormwater Management Plan for McKay Brook in 1999. [3]

For the purposes of this report these previous reports are termed the (i) 1994 Flood Study and (ii) 1999 SWMP.

1.3 Study Objectives

The primary objectives of the project are as follows:

- Update the 1994 Moggill Creek and 1999 McKay Brook 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.
- Estimate design and rare / extreme flood magnitudes.
- Determine flood levels for the design and rare / extreme events.
- Quantify the impacts of Minimum Riparian Corridor (MRC) and floodplain development / filling in accordance with planning requirements.
- Produce flood extent mapping for the selected range of design and rare / extreme events.
- Quantify the sensitivity of climate variability on flooding within the catchment.

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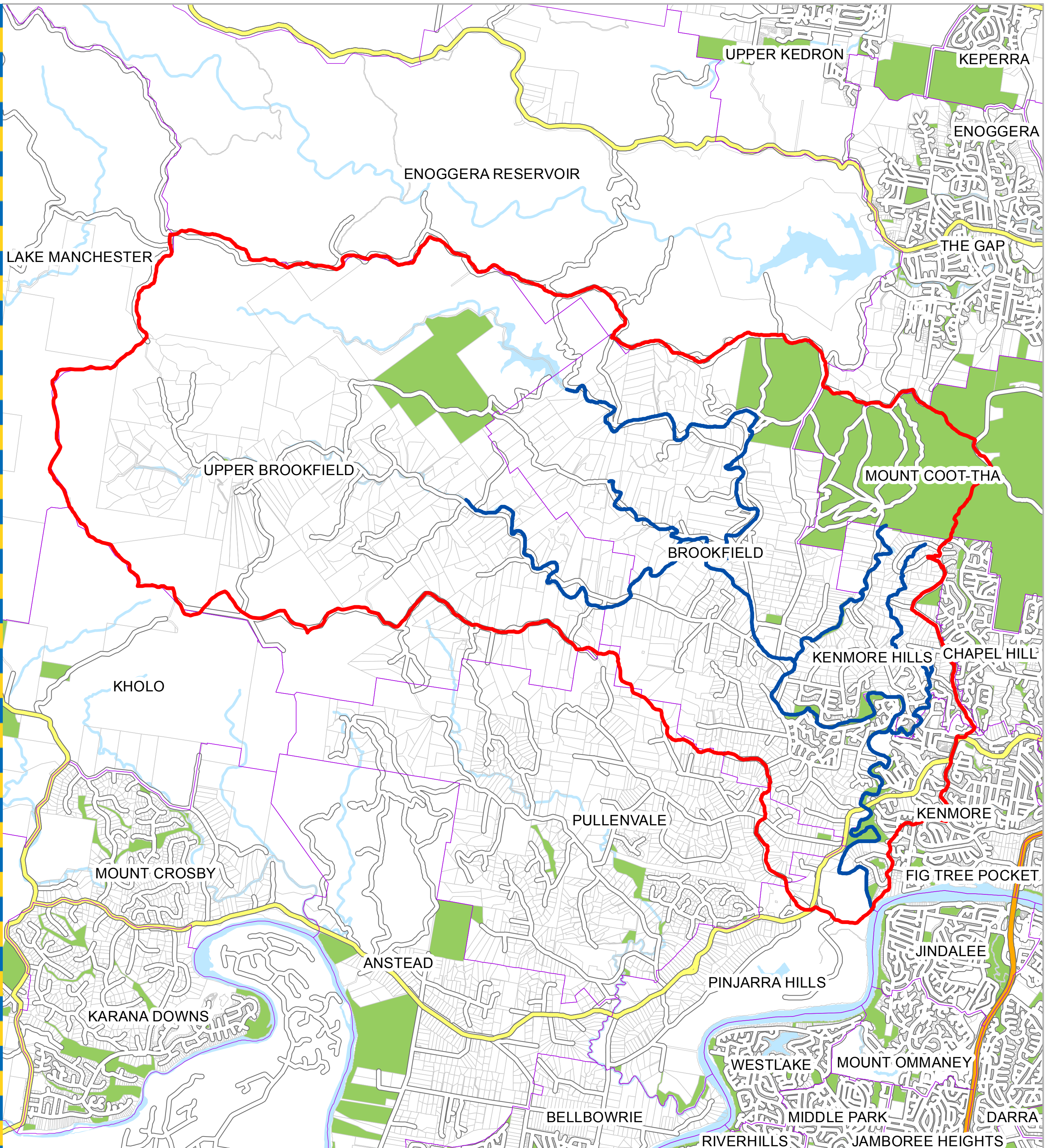
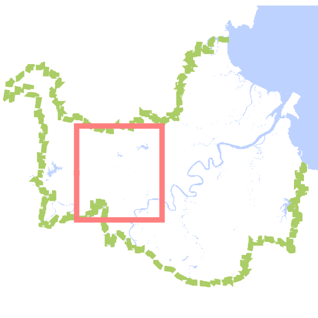
[1] Brisbane City Council 2015, *Creek Flood Study Procedure Document Version 7.1*
[2] Brisbane City Council Design Branch 1994, *Moggill Creek Flood Study*
[3] Brisbane City Council City Design 1999, *McKay Brook Stormwater Management Plan Technical Report*





0 300 600 900

Metres

N W E S

Legend

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

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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Moggill Creek Flood Study

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:

- Develop an URBS hydrologic model of the catchment, superseding the previous URBS model.
- Develop a 1-dimensional (1d) / 2-dimensional (2d) TUFLOW hydraulic model of the creek system to replace the existing 1d MIKE11 model (Moggill, Gold and Gap Creeks) and steady state HEC-RAS model (McKay Brook).
- Calibrate the hydrologic and hydraulic models to the May 2015, May 2009 and November 2008 historical flood events.
- Verify the hydrologic and hydraulic models against the January 2013 historical flood event.
- Estimate the design and extreme flood magnitudes for the full range of events from 2-yr ARI (50% AEP) 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 "Existing Condition" hydraulic model to simulate the impacts of MRC and filling outside the "Modelled Flood Corridor."
- Combine the modelling results for the various storm durations to produce peak results throughout the catchment for each AEP event.
- Produce flood extent mapping for the selected range of design and rare / extreme events.
- Undertake climate variability modelling for the 100-yr ARI (1% AEP), 200-yr ARI (0.5% AEP) and 500-yr ARI (0.2% AEP) events to determine the potential 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. Refer to Figure 3.1 for the hydrometric 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.

- 2014 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

2.1.1 General

The confluence of Moggill Creek and the Brisbane River is approximately 2 km upstream of the Centenary Highway Bridge at Kenmore. The total catchment area of the Moggill Creek Catchment is approximately 65.8 km^2, which comprises the following tributaries:

- Moggill Creek: 39 km^2
- Gold Creek: 17.9 km^2
- Gap Creek: 6.6 km^2
- McKay Brook: 2.4 km^2

2.1.2 Moggill Creek

Moggill Creek is the largest waterway within the catchment with a length of approximately 25 km from Upper Brookfield to its outfall at Kenmore. The catchment is bounded by Gold Creek Catchment (north); Lake Manchester Catchment (west); Kholo and Pullen Pullen Creek Catchments (south) and Gap Creek / McKay Brook (east).

The highest elevation in the catchment is approximately 420 m AHD and is situated along the western catchment boundary within the D'Aguilar Ranges. The catchment headwaters are in the D'Aguilar Ranges, an area which is characterised by steep slopes and dense / forested vegetation.

Moggill Creek is an open waterway and generally in a natural state along its entire length. The creek corridor is quite heavily vegetated with dense riparian vegetation for most of its length. The length of creek upstream of the hydraulic model extent is approximately 7.7 km with an average bed slope of approximately 1.2 %. Within the hydraulic model extents, the length of the creek is 17.1 km with an average bed slope of 0.4 %.

The lower section of the creek is subject to downstream hydraulic interaction from a number of sources including the Brisbane River and the ocean tidal cycle.

2.1.3 Gold Creek

Gold Creek has a length of over 15 km and is the second largest creek within the catchment. Gold Creek joins Moggill Creek in the middle section of the catchment, approximately 11 km upstream of the confluence with the Brisbane River.

Gold Creek contains a relatively small water supply reservoir (Gold Creek Reservoir), which is located approximately half way along the length of the creek. Gold Creek Reservoir is discussed further in Section 4.

Gold Creek Catchment is bounded by Breakfast Creek Catchment (north); Lake Manchester Catchment (west); Moggill Creek (south) and Gap Creek (east). The catchment is quite narrow and elongated, with an average length to width ratio of approximately 5 to 1.

The highest elevation in the catchment is approximately 365 m AHD and is situated along the north-western catchment boundary within the D'Aguilar Ranges. Similar to Upper Moggill Creek, the catchment headwaters are in the D'Aguilar Ranges, an area which is characterised by steep slopes and dense / forested vegetation.

Gold Creek is an open waterway and generally in a natural state along its entire length, apart from the 1 km section which contains the Gold Creek Reservoir storage area. Upstream of the reservoir the average bed slope of the creek is 1.9 %, whereas downstream the average bed slope is milder at 0.7 %.

2.1.4 Gap Creek

Gap Creek has a length of nearly 5 km and is the third largest creek within the catchment. Gap Creek joins Moggill Creek in the mid to lower section of the catchment, approximately 8.8 km upstream of the confluence with the Brisbane River. The average bed slope of the creek is 1.2 %.

Gap Creek Catchment is bounded by Breakfast Creek Catchment (north and east); Moggill and Gold Creek Catchments (west); Moggill Creek Catchment (south) and McKay Brook Catchment (east).

The highest elevation in the catchment is approximately 255 m AHD and is situated along the north-eastern catchment boundary within the Mount Coot-tha Forest. The catchment headwaters are in the Mount Coot-tha Forest, an area which is characterised by steep slopes and dense / forested vegetation.

2.1.5 McKay Brook

McKay Brook Catchment is a small catchment on the eastern boundary of the greater total catchment. The catchment is bounded by Cubberla Creek Catchment (north and east); Moggill and Gap Creek Catchments (north and west) and Moggill Creek Catchment (south).

McKay Brook has a length of approximately 4.5 km and joins Moggill Creek in the lower section of the catchment, approximately 5 km upstream of the confluence with the Brisbane River. The average bed slope of the creek is 1.3 %, which is of similar magnitude to Gap Creek.

The highest elevation in the catchment is approximately 95 m AHD and is situated along the north-eastern catchment boundary within Mount Coot-tha Forest. The catchment is very narrow and elongated, with an average length to width ratio of approximately 9 to 1.

2.2 Land Use

Land-use within the total catchment varies between creek catchments and also from upstream to downstream. The elevated catchment headwaters of Moggill, Gold and Gap Creeks are heavily forested and are typically zoned as environmental management and conservation areas.

Appendix C provides a figure indicating the catchment land-use, which is based upon BCC City Plan 2014. [4]

In the Upper Moggill Creek Catchment (downstream of the upstream hydraulic model extent), the zoning is typically rural adjacent to the creek and environment management further away from the creek. In the mid to lower areas of the Moggill Creek Catchment, where there is more development, the zoning is typically a mix of rural residential, low density residential and open space.

Downstream of Gold Creek Reservoir, the Gold Creek Catchment area is comprised primarily of environmental management, conservation and rural zoned areas. However, close to the confluence with Moggill Creek, there are small pockets of community purpose and rural residential zoned areas which adjoin the creek.

In the Gap Creek Catchment, nearly the entire catchment is zoned environmental management and conservation.

In the McKay Brook Catchment, the upper section of the catchment is zoned environmental management and conservation, whereas the mid to lower sections are typically a mix of rural residential, low density residential and open space.

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

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 35 years. This table includes the peak flood level in Moggill Creek at the Fortrose Street (540061) stream gauge in Kenmore. The table also indicates the availability of stream gauge / MHG information.

The May 2009 event is the largest flood to have occurred within the catchment in recent history, noting that the January 2011 flood may have recorded a higher flood level than the May 2009 event but the source of flooding included backwater from the Brisbane River.

Table 3.1 – Historical Peak Levels at Fortrose Street on Moggill Creek

Event	Peak Flood Level (m AHD)	Recorded Hydrograph at Stream Gauge	Number of MHGs and/or recorded levels	Approximate Size of Event
February 1981	8.26	Yes	13	2-yr to 5-yr ARI (50 % to 20 % AEP)
June 1983	8.43	Yes	13	2-yr to 5-yr ARI (50 % to 20 % AEP)
April 1984	8.17	Yes	15	2-yr to 5-yr ARI (50 % to 20 % AEP)
April 1988	8.60	Yes	14	~5-yr ARI (20 % AEP)
April 1989	9.02	Yes	28	~10-yr ARI (10 % AEP)
March 1992	8.07	Yes	11	2-yr to 5-yr ARI (50 % to 20 % AEP)
May 1996	8.57	Yes	16	2-yr to 5-yr ARI (50 % to 20 % AEP)
November 2008	9.18	Yes	18	10-yr to 20-yr ARI (10 % to 5 % AEP)
May 2009	10.91	Yes	18	~100-yr ARI (1 % AEP)
October 2010	7.83	Yes	8	< 2-yr ARI (50 % AEP)
January 2011	N/A*	N/A	15	N/A
January 2013	9.60	Yes	24	20-yr to 50-yr ARI (5 % to 2 % AEP)
May 2015	9.02	Yes	16	~10-yr ARI (10 % AEP)

**Telemetry station failed due to the equipment being damaged by flood water.*

The selection of specific historical events for calibration and verification was based upon the criteria as listed below.

- 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.

As well as these criteria, it was considered important to cover a wide range of flood magnitudes, if possible. On the basis of these selection criteria, the following events were selected for calibration and verification:

- Calibration
  - May 2015
  - May 2009
  - November 2008

- Verification
  - January 2013

The January 2011 event was considered for calibration / verification. However, it was not chosen because there was no continuous stream height information at Fortrose Street (540061) and the dominant source of flooding in the lower areas was due to backwater from the Brisbane River, not local catchment runoff.

The selection of these four events also represents a period of time where there have not been any changes to Gold Creek Reservoir.

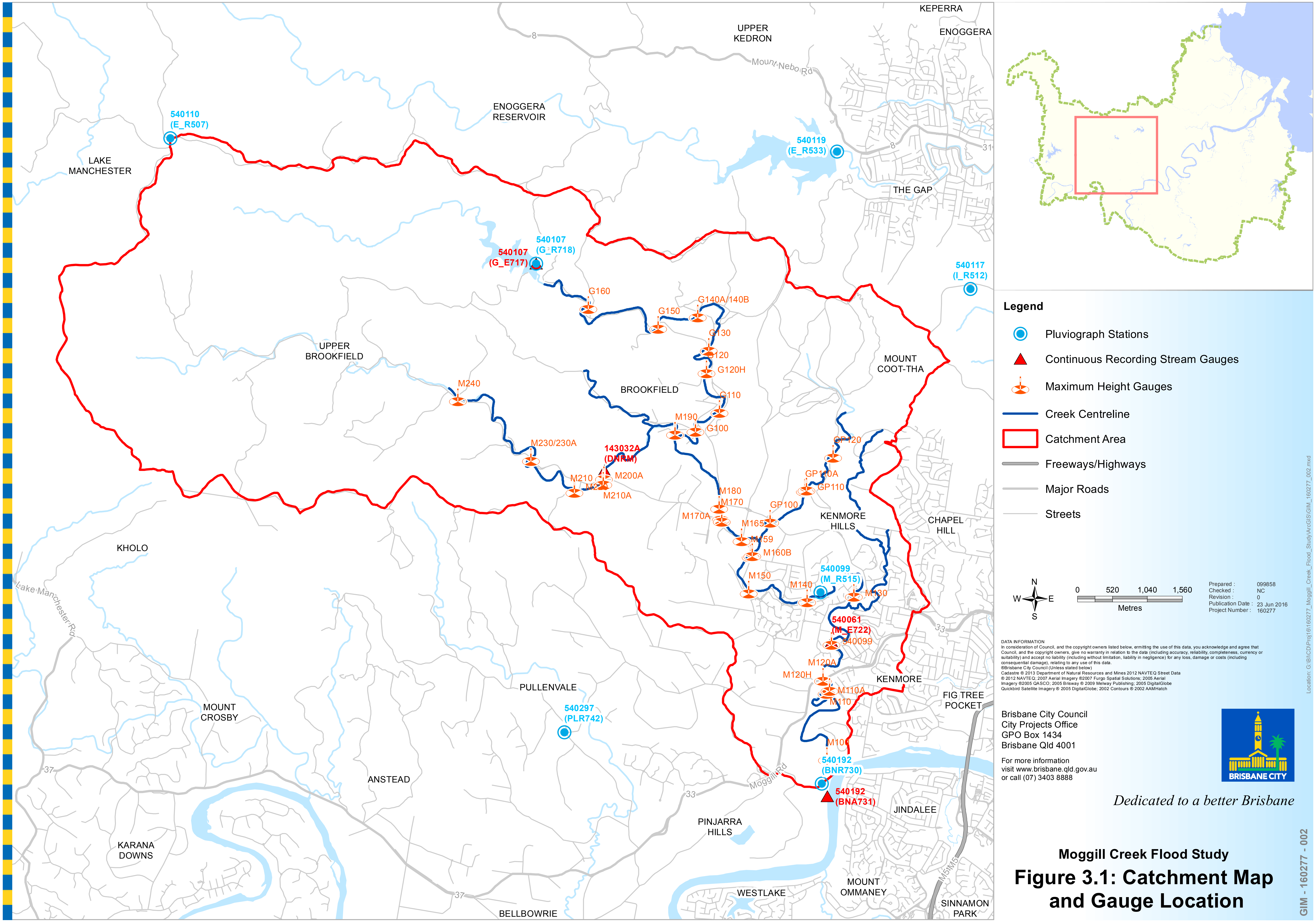
3.2 Availability of Historical Data for Selected Storms

3.2.1 Continuous Recording Rainfall Stations

Seven 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.

Table 3.2 – Rainfall Station details

Gauge ID	Old BCC ID	Catchment	Location	Current Status
540099	M_R515	Moggill Creek	Chadstone Close, Kenmore Hills	Open
540107	G_R718	Moggill Creek	Gold Creek Reservoir at Brookfield	Open
540110	E_R507	Breakfast Creek	Brisbane Forest Park, Mt. Nebo	Open
540117	I_R512	Breakfast Creek	Mt Coot-tha	Open
540119	E_R533	Breakfast Creek	Enoggera Creek Dam, The Gap	Open
540192	BNR730	Brisbane River	Brisbane River at Jindalee	Open
540297	PLR742	Pullen Pullen Creek	Pullenvale Hall, Pullenvale	Open



Moggill Creek Flood Study

Figure 3.1: Catchment Map and Gauge Location

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			
			May 2015	January 2013	May 2009	November 2008
540099	M_R515	Chadstone Close, Kenmore Hills	✓	✓	✓	✓
540107	G_R718	Gold Creek Reservoir at Brookfield	✓	✓	✓	✓
540110	E_R507	Brisbane Forest Park, Mt. Nebo	✓	✓	✓	✓
540117	I_R512	Mt Coot-tha	✓	✓	✓	✓
540119	E_R533	Enoggera Creek Dam, The Gap	✓	✓	✓	✓
540192	BNR730	Brisbane River at Jindalee	✓	✓	✓	✓
540297	PLR742	Pullenvale Hall, Pullenvale	✓	✓	✓	✓

3.2.2 Continuous Recording Stream Gauges

Continuous recording stream height gauges collect instantaneous water level information over time. There are three water level gauges operational within the total catchment area and these are listed in Table 3.4 below. All three gauges were operational during the calibration and verification events.

Table 3.4 – Continuous recording stream gauges

Gauge ID	Old BCC ID	Catchment	Owner	Location	Current Status
143032A	N/A	Moggill Creek	DNRM	Upper Brookfield Road, Upper Brookfield	Open
540061	M_E722	Moggill Creek	BCC	Fortrose Street, Kenmore	Open
540107	G_E717	Gold Creek	BCC	Gold Creek Reservoir at Brookfield	Open

The Upper Brookfield (143032A) gauge is owned by the Department of Natural Resources and Mines (DNRM) and is typically used for water quality monitoring purposes.

For the purposes of the 1994 Flood Study, this gauge was not used as "*there was considerable uncertainty in relation to datums (even with the new survey information) and the validity of available rating curves*."

The data from this gauge is not received automatically by BCC, however is accessible via the DNRM web portal. Typically, this gauge records a reading every hour and more regularly during a flooding event.

Our review of the data indicated that readings were recorded for the calibration / verification events as follows:

- May 2015 – recorded hourly
- January 2013 – sub-hourly readings during the main peak
- May 2009 – recorded hourly
- November 2008 – sub-hourly readings during the main peak

Generally, at the location of the Upper Brookfield gauge (143032A), sub-hourly readings would be required to ensure the accuracy of the peak water level and hydrograph shape. For the two events where only hourly recordings are available (i.e. May 2015 and May 2009), it is likely that the recorded hydrograph does not fully represent the actual hydrograph. This should be considered when viewing the results of the calibration and verification.

Also, the gauge zero datum reported by DNRM is 36.426 mAHD. As part of the 1994 Flood Study, this level was surveyed and found to be 36.766 mAHD. As this 1993 survey is the latest available information, it was decided to also adopt this same gauge zero level of 36.766 mAHD for this study.

The location of this gauge is such that it provides valuable information on the hydrologic / flooding characteristics for Upper Moggill Creek. Although there is some uncertainty with respect to this gauge, it was decided to include it as part of this flood study.

At the Fortrose Street gauge (540061), the creek invert level is approximately 2.5 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 the Brisbane River.

The Gold Creek Reservoir gauge (540107) monitors the reservoir water level and is also a valuable tool in understanding the hydrologic / flooding characteristics of Upper Gold Creek.

All gauges have recorded data available for all calibration and verification events. The locations of these gauges are 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 3.5 indicates the period of operation for the MHGs on Moggill, Gold and Gap Creeks. There are 30 MHGs within the total catchment area and all are currently operational. Of the 30 operating MHGs, there are currently 19 on Moggill Creek, 8 on Gold Creek and 3 along Gap Creek. There are currently no MHGs within McKay Brook.

Table 3.6 indicates the availability of MHG data for each flooding event. It is apparent that the January 2013 event has the greatest number of recorded levels. However, this event was not as large as the May 2009 event, where many of the MHG gauges were destroyed and surveyed debris levels were acquired in lieu of the MHG record.

Table 3.5 – Maximum Height Gauge period of record

Creek	Gauge ID	Location	Records From	Records To
Moggill	M100	U/S Moggill Creek Mouth	March 2004	Present
	M110	D/S Moggill Rd	February 1981	Present
	M120	U/S Moggill Rd (Low)	February 1981	Present
	M120H	D/S Moggill Rd (High)	February 2010	Present
	M130	D/S Branton St Footbridge	February 1981	Present
	M140	End of Kailua St	February 1981	Present
	M150	U/S Willunga St	February 1981	Present
	M150H	D/S Willunga St / D/S Rafting Ground Rd	February 2010	Present
	M159	D/S Rafting Ground Rd	February 2010	Present
	M160	U/S Rafting Ground Rd	February 1981	Present
	M165	D/S Boscombe Rd	May 2009	Present
	M170	Brookfield Showgrounds	February 1981	Present
	M180	U/S Brookfield Rd	February 1981	Present
	M190	Bundaleer Rd	January 1979	Present
	M200	D/S Upper Brookfield Rd	February 1981	Present
	M210	U/S Upper Brookfield Rd	February 1981	Present
	M220	Haven Rd	April 1978	Present
	M230	U/S Upper Brookfield Rd	February 1981	Present
	M240	U/S Kittani St	October 2010	Present
Gold	G100	U/S Savages Rd	February 1982	Present
	G110	179 Gold Creek Rd	February 1982	Present
	G120	U/S 274 Gold Creek Road Driveway (Low)	February 1982	Present
	G120H	U/S 274 Gold Creek Road Driveway (High)	October 2010	Present
	G130	U/S Gold Creek Rd / Jones Rd intersection	February 1982	Present
	G140	U/S Jones Rd	March 2001	Present
	G150	408 Gold Creek Rd Driveway	May 1980	Present
	G160	U/S 581 Gold Creek Rd	January 1979	Present
Gap	GP100	U/S Brookfield Rd @ Deerhurst Rd	January 1979	Present
	GP110	End of Kookaburra St	January 1979	Present
	GP120	U/S Gap Creek Rd	January 1979	Present

Table 3.6 – Maximum Height Gauge data availability

Creek	Gauge ID	Data Availability			
		May 2015	January 2013	May 2009	November 2008
Moggill	M100	×	✓	✓	✓
	M110	✓	✓	×	✓
	M120	✓	×	✓(1)	✓
	M120H	✓	✓	×	×
	M130	×	×	✓	×
	M140	×	✓	✓	✓(1)
	M150	✓	×	✓(1)	✓
	M150H	✓	✓	×	×
	M159	✓	✓	×	×
	M160	✓	✓	×	×
	M165	✓	×	✓(1)	×
	M170	✓	✓	×	✓
	M180	✓	✓	✓	✓
	M190	×	✓	✓	✓
	M200	×	✓	✓	×
	M210	✓	✓	✓	✓
	M220	×	✓(1)	✓(1)	✓(1)
	M230	×	✓	×	✓
	M240	✓	✓	×	×
Gold	G100	×	✓	✓	×
	G110	×	×	✓	×
	G120	✓	✓	×	✓
	G120H	×	✓	×	×
	G130	×	×	✓	×
	G140	✓	✓	×	✓
	G150	✓	✓	✓	✓
	G160	✓	✓	✓	✓
Gap	GP100	✓	✓	✓	✓
	GP110	✓	✓	×	✓
	GP120	✓	✓	✓	✓

(1) Reading from debris mark

3.2.4 Downstream Boundary Information

There are two stream gauges located on the Brisbane River near the mouth of Moggill Creek; as indicated in Table 3.7. These gauges are situated approximately 400 m upstream of the mouth of Moggill Creek on opposing banks of the Brisbane River. The Seqwater owned gauge (540192) has recorded data from November 1994, whereas the BCC gauge (540682) was installed more recently in May 2014 for redundancy purposes.

Table 3.7 – Nearby Brisbane River Stream Gauges

Gauge ID	Old BCC ID	Catchment	Owner	Location	Current Status
540192	BNA731	Brisbane River	Seqwater	Brisbane River at Jindalee	Open
540682	BNA765	Brisbane River	BCC	Mount Ommaney Dr, Jindalee	Open

The continuous water level data from the Seqwater gauge (540192) was used as the downstream boundary conditions for the May 2015 and January 2013 events. Sufficient data was also available for the May 2015 event for the BCC owned gauge (540682); however the differences in levels were negligible so a consistent approach was taken.

For both May 2009 and November 2008, there were no records available for both gauges; refer to Section 5.3.5 for further details on the adoption of downstream boundary conditions.

3.3 Characteristics of Historical Events

3.3.1 May 2015 event

This event was a relatively small flooding event which produced a flood level of 9.02 m AHD at the stream gauge on Moggill Creek at Fortrose Street. Minor flooding occurred in some localised areas in the middle and lower reaches of the creek.

The event rainfall was consistent over the entire catchment with approximately 170 mm being recorded in 24 hours on the 1st May. In the lower reaches, the rainfall was less intense with only 19 mm being the peak recorded 30 minute rainfall at the Jindalee Alert station 540192 (BNR730). The most intense burst occurred over 6 hours between 1:30 pm and 7:30 pm on the 1st May, where approximately 132 mm of rainfall was recorded at Rainfall Station 540099 (M_R515) at Chadstone Place, Kenmore Hills. The cumulative rainfall for each rainfall station 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 seven rainfall stations. The catchment experienced approximately 30 mm of rainfall in the 4-day lead up to the event and 50 mm in the preceding 14 days, meaning that the soil is unlikely to have been saturated when the event occurred.

Table 3.8 - Rainfall characteristics (May 2015 event)

Gauge ID	Old BCC ID	Location	Antecedent Rainfall (mm)		Event Rainfall (mm)	
			14-day	4-day	1st May (peak 3hr burst)	1st May (full day)
540099	M_R515	Chadstone Close, Kenmore Hills	54	29	85	178
540107	G_R718	Gold Creek Reservoir at Brookfield	45	33	82	174
540110	E_R507	Brisbane Forest Park, Mt. Nebo	47	31	77	177
540117	I_R512	Mt Coot-tha	51	43	91	178
540119	E_R533	Enoggera Creek Dam, The Gap	57	45	84	165
540192	BNR730	Brisbane River at Jindalee	40	27	88	165
540297	PLR742	Pullenvale Hall, Pullenvale	34	24	75	164

Figure 3.2 indicates the IFD curve for the seven rainfall stations when compared to the AR&R IFD curve generated at the catchment centroid. The equivalent design rainfall ARI at Rainfall Station 540099 (M_R515) at Chadstone Close would have been as follows:

- 1 hour rainfall: 2-yr ARI (50 % AEP) to 5-yr ARI (20 % AEP)
- 2 hour rainfall: 5-yr ARI (20 % AEP) to 10-yr ARI (10 % AEP)
- 3 hour rainfall: 10-yr ARI (10 % AEP)
- 6 hour rainfall: 20-yr ARI (5 % AEP)

3.3.2 January 2013 event

This event was a relatively long duration flooding event which produced a flood level of 9.60 m AHD at the stream gauge on Moggill Creek at Fortrose Street, causing minor flooding in the middle and upper reaches of the creek. However, flooding in the lower reach during this event was more significant due to backwater effects from the Brisbane River with the Jindalee Alert gauge recording a level of 4.98 mAHD.

The event occurred from 6 pm on the 26th January to around 8 am on the 28th January. The most intense burst occurred on the 27th January over a 10 hour period between 9:30 am and 7:30 pm, where approximately 160 mm to 180 mm of rainfall fell across the catchment. The event was more intense in the upper sections of the Gap Creek and McKay Brook Catchments compared with the upper and lower sections of the Moggill and Gold Creek Catchments. The cumulative rainfall 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 seven rainfall stations. The catchment experienced between 103 and 197 mm of rainfall in the 14 day lead up to the event with between 99 mm and 192 mm falling in the 4 days prior. Therefore the soil would have been fairly saturated due to the rainfall in the days prior to the main storm event.

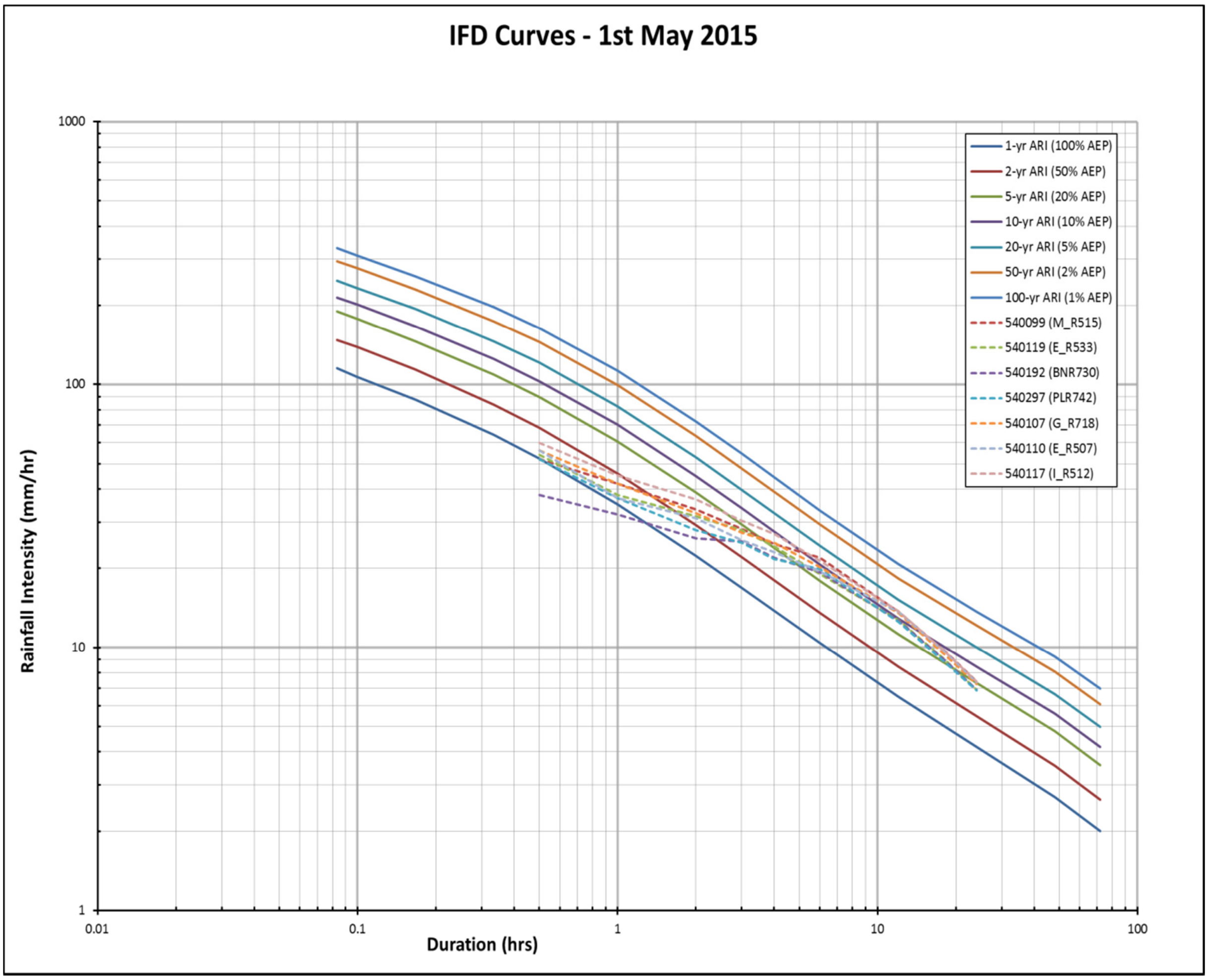


Figure 3.2: IFD Curve for May 2015 event.

Table 3.9 - Rainfall characteristics (January 2013 event)

Gauge ID	Old BCC ID	Location	Antecedent Rainfall (mm)		Event Rainfall (mm)	
			14-day	4-day	27th January (peak 3hr burst)	27th January (full day)
540099	M_R515	Chadstone Close, Kenmore Hills	143	138	101	262
540107	G_R718	Gold Creek Reservoir at Brookfield	164	157	81	285
540110	E_R507	Brisbane Forest Park, Mt. Nebo	197	192	84	325
540117	I_R512	Mt Coot-tha	173	165	110	285
540119	E_R533	Enoggera Creek Dam, The Gap	151	145	95	268
540192	BNR730	Brisbane River at Jindalee	103	99	85	228
540297	PLR742	Pullenvale Hall, Pullenvale	130	125	80	234

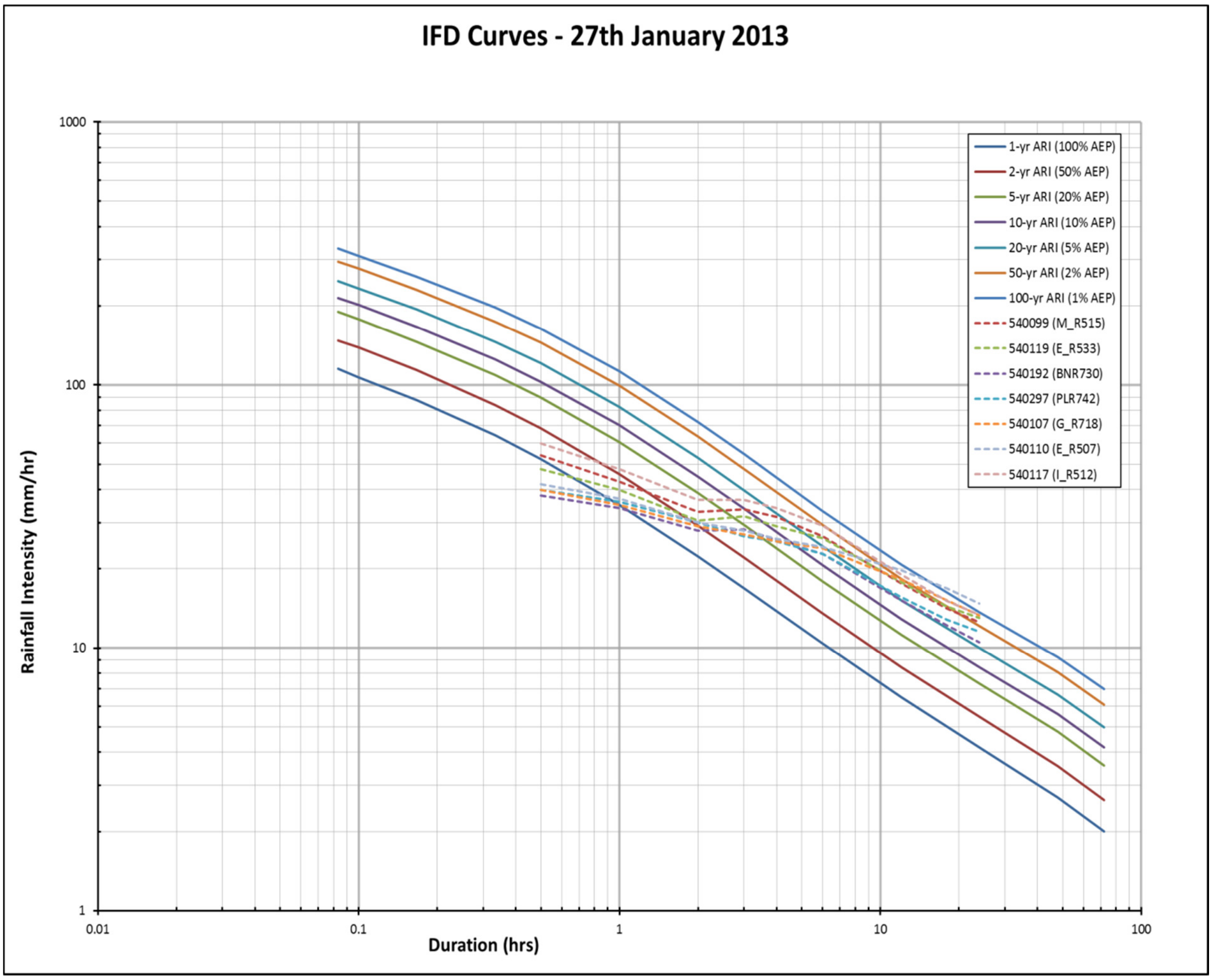


Figure 3.3: IFD Curve for January 2013 event.

Figure 3.3 indicates the IFD curve for the seven rainfall stations when compared to the AR&R IFD curve generated at the catchment centroid. The equivalent design rainfall ARI at Rainfall Station 540099 (M_R515) at Chadstone Close would have been as follows:

- 1 hour rainfall: 2-yr ARI (50 % AEP) to 5-yr ARI (20 % AEP)
- 2 hour rainfall: 5-yr (20 % AEP) to 10-yr ARI (10 % AEP)
- 3 hour rainfall: 10-yr ARI (10 % AEP)
- 6 hour rainfall: 20-yr ARI (5 % AEP) to 50-yr ARI (2 % AEP)

3.3.3 May 2009 Event

This event was the highest recorded event for the Moggill Creek Catchment in recent times and produced a flood level of 10.91 m AHD at the stream gauge on Moggill Creek at Fortrose Street. Moderate flooding occurred in the upper and middle reaches of the creek.

The event occurred over a 13 hour period starting at approximately 8 am on the 20th May. The event consisted of two significant bursts of rainfall as evidenced by the recorded stream gauge data showing two distinct flood peaks. The first burst of rainfall fell between 11:30 am and 3 pm where approximately 120 mm to 160 mm of rainfall fell across the catchment, causing the larger of the two flood peaks. The second burst lasted approximately 1.5 hours starting around 6:30 pm with an average of 70 mm rainfall falling across the catchment.

The event comprised variable rainfall with considerably more intense rainfall occurring within the upper reaches of the catchment. This 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 for each rainfall station is presented in Appendix A.

Table 3.10 indicates the 4-day and 14-day antecedent rainfall as well as statistics on the event rainfall at seven rainfall stations. The catchment experienced between 65 and 129 mm of rainfall in the 14-day lead up to the event with practically all occurring within the 4 days prior. Therefore it is likely that the soil would have had a reasonable degree of saturation prior to the main storm event.

Table 3.10 - Rainfall characteristics (May 2009 event)

Gauge ID	Old BCC ID	Location	Antecedent Rainfall (mm)		Event Rainfall (mm)	
			14-day	4-day	20th May (peak 3hr burst)	20th May (full day)
540099	M_R515	Chadstone Close, Kenmore Hills	87	87	141	343
540107	G_R718	Gold Creek Reservoir at Brookfield	118	115	123	336
540110	E_R507	Brisbane Forest Park, Mt. Nebo	116	114	71	229
540117	I_R512	Mt Coot-tha	127	122	143	320
540119	E_R533	Enoggera Creek Dam, The Gap	135	129	105	268
540192	BNR730	Brisbane River at Jindalee	65	65	109	259
540297	PLR742	Pullenvale Hall, Pullenvale	79	79	105	303

Figure 3.4 indicates the IFD curve for the seven rainfall stations when compared to the AR&R IFD curve generated at the catchment centroid. The equivalent design rainfall ARI at Rainfall Station 540099 (M_R515) at Chadstone Close would have been as follows:

- 1 hour rainfall: 5-yr ARI (20 % AEP)
- 2 hour rainfall: 20-yr ARI (5 % AEP)
- 3 hour rainfall: 50-yr ARI (2 % AEP)
- 6 hour rainfall: 50-yr ARI (2 % AEP)

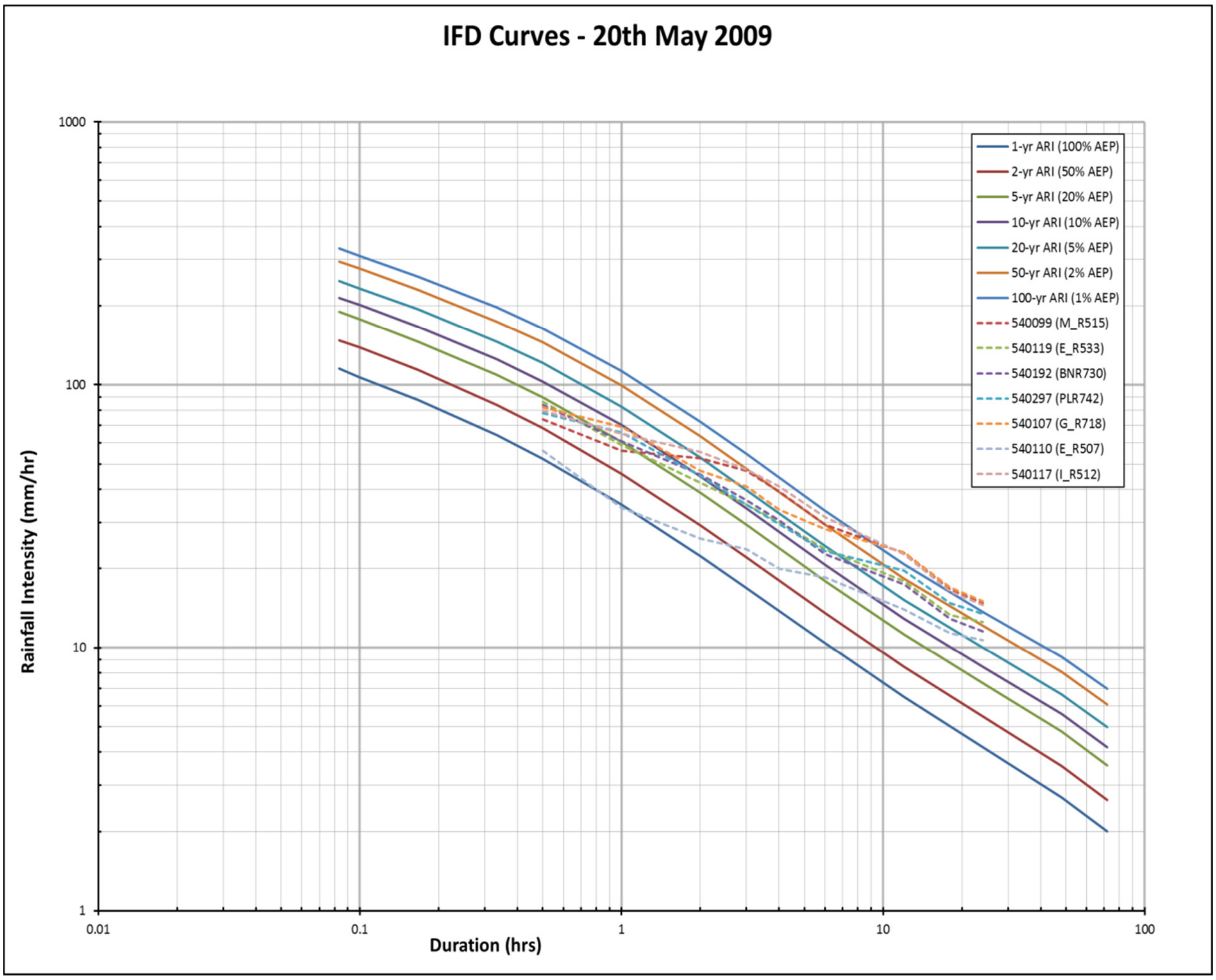


Figure 3.4: IFD Curve for May 2009 event.

3.3.4 November 2008 event

This event was a relatively small flooding event which produced a flood level of 9.18 m AHD at the stream gauge on Moggill Creek at Fortrose Street. Minor flooding occurred in some localised areas in the middle and lower reaches of the creek.

The event occurred as one intense burst over a 4 hour period from 10 pm on the 19th November to 2 am on the 20th November. During this period, an average of 90 mm of rain fell on the middle and upper reaches of the catchment with only 48 mm recorded in the lower reaches at the Jindalee Alert station. The most intense rainfall was experienced in the upper reaches of Gap Creek and McKay Brook with a peak 124 mm of rain falling in the 4 hour period.

The large 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 for each rainfall station is presented in Appendix A.

Table 3.11 indicates the 4-day and 14-day antecedent rainfall as well as statistics on the event rainfall at seven rainfall stations. The catchment experienced between 127 mm and 190 mm of rainfall in the 14-day lead up to the event with between 108 mm and 172 mm falling in the 4 days prior. Therefore the soil would have been saturated due to the rainfall in the days prior to the main storm event.

Table 3.11 - Rainfall characteristics (November 2008 event)

Gauge ID	Old BCC ID	Location	Antecedent Rainfall (mm)		Event Rainfall (mm)	
			14-day	4-day	20th Nov (peak 3hr burst)	19th-20th Nov (two full days)
540099	M_R515	Chadstone Close, Kenmore Hills	180	161	88	122
540107	G_R718	Gold Creek Reservoir at Brookfield	182	157	90	114
540110	E_R507	Brisbane Forest Park, Mt. Nebo	182	156	123	153
540117	I_R512	Mt Coot-tha	183	166	124	149
540119	E_R533	Enoggera Creek Dam, The Gap	190	172	93	107
540192	BNR730	Brisbane River at Jindalee	127	108	48	74
540297	PLR742	Pullenvale Hall, Pullenvale	159	144	91	114

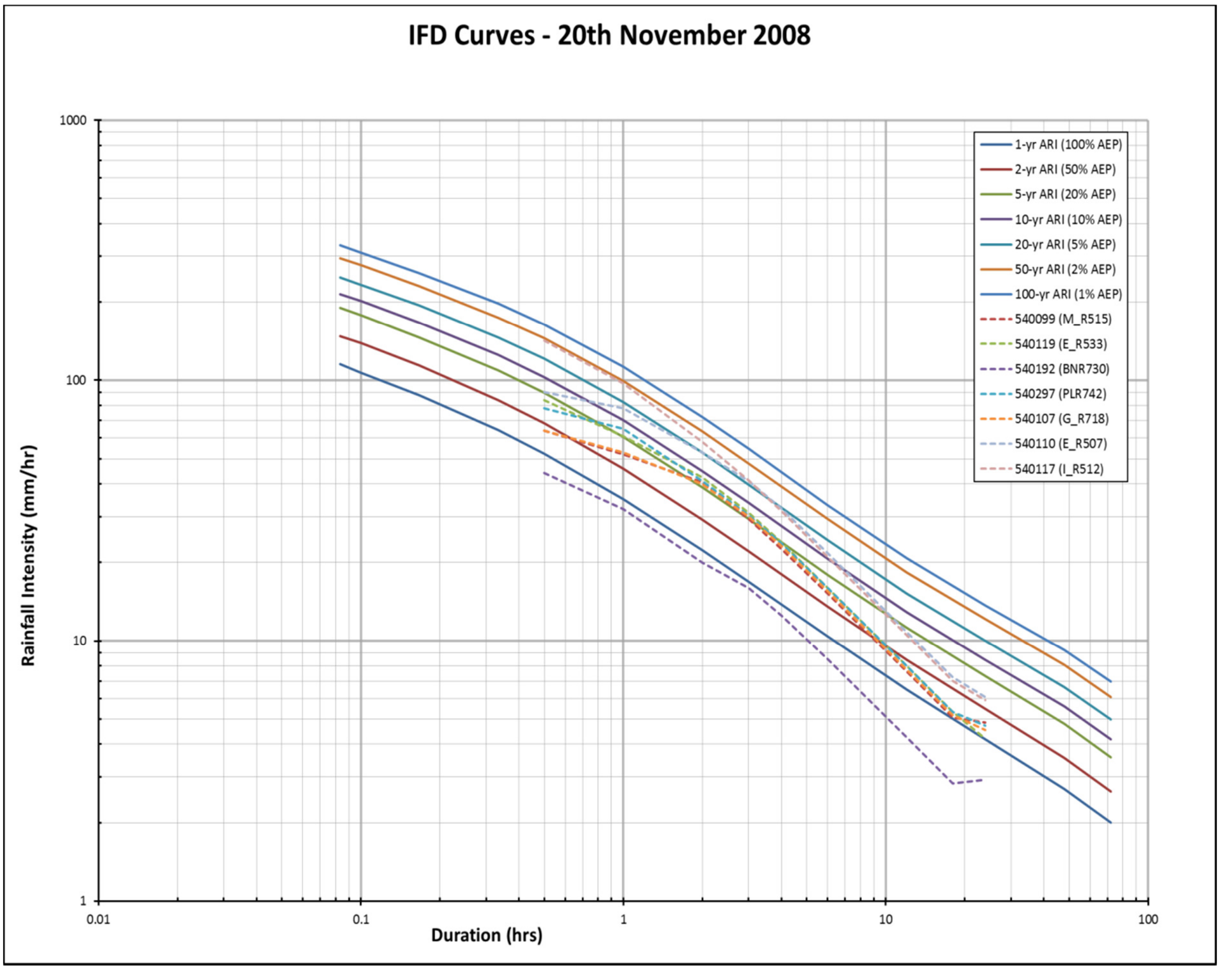


Figure 3.5: IFD Curve for November 2008 event.

Figure 3.5 indicates the IFD curve for the seven rainfall stations when compared to the AR&R IFD curve generated at the catchment centroid. The equivalent design rainfall ARI at Rainfall Station 540099 (M_R515) at Chadstone Close would have been as follows:

- 1 hour rainfall: 2-yr ARI (50 % AEP) to 5-yr ARI (20 % AEP)
- 2 hour rainfall: 5-yr ARI (20 % AEP) to 10-yr ARI (10 % AEP)
- 3 hour rainfall: 5-yr ARI (20 % AEP) to 10-yr ARI (10 % AEP)
- 6 hour rainfall: 2-yr ARI (50 % AEP) to 5-yr ARI (20 % AEP)

4.0 Hydrologic Model Development and Calibration

4.1 Overview

The hydrologic model simulates the rainfall-runoff process within the catchment and calculates a flow hydrograph at the outlet of each sub-catchment.

An URBS (version 5.85a) model was developed for the total catchment area including Moggill Creek, Gold Creek, Gap Creek and McKay Brook as well as some other major tributaries. The “Split” modelling approach was used whereby the catchment and channel routing are separated. The rainfall on a sub-catchment is routed through the catchment to the creek/river channel and then the inflow is routed along a reach using the non-linear Muskingum method.

Sub-catchment routing is undertaken by routing through a non-linear reservoir, of which the storage-discharge relationship is based upon the following equation:

$$S_{catch} = \{\beta \sqrt{A}(1 + F)^2 / (1 + U)^2\}Q^m$$

where:

S_{catch} = catchment storage
β = catchment lag parameter
A = area of sub-catchment
U = fraction urbanisation of sub-catchment
F = fraction of sub-catchment forested
m = catchment non-linearity parameter
Q = outflow

Routing of all major open waterways and tributaries utilised the Muskingum methodology, which is based on the following equation:

$$S_{chnl} = \alpha f(\mathrm{n}L / \sqrt{S_c})(xQ_u + (1 - x)Q_d)^n$$

where:

S_{chnl} = channel storage
α = channel lag parameter
f = reach length factor
L = length of reach
S_c = slope of reach
Q_u = inflow at upstream end of the reach
Q_d = inflow at downstream end of the reach
x = Muskingum translation parameter
n = Muskingum non-linearity parameter
n = Manning’s ‘n’ or channel roughness

For further details on this modelling approach refer to the URBS User Manual.[5]

[5] *URBS A Rainfall Runoff Routing Model for Flood Forecasting and Design Version 5.00*, DG Carroll 2012

4.2 Sub-catchment Data

4.2.1 General

This section describes the sub-catchment parameters used in the URBS model. URBS allows the user to define the sub-catchment with differing levels of detail depending on the type of catchment and requirements for the study.

For this study the following parameters were utilised:

Area – sub-catchment area
UL – Urban Low Density
UM – Urban Medium Density
UH – Urban High Density
UR – Urban Rural
CS – Catchment Slope

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

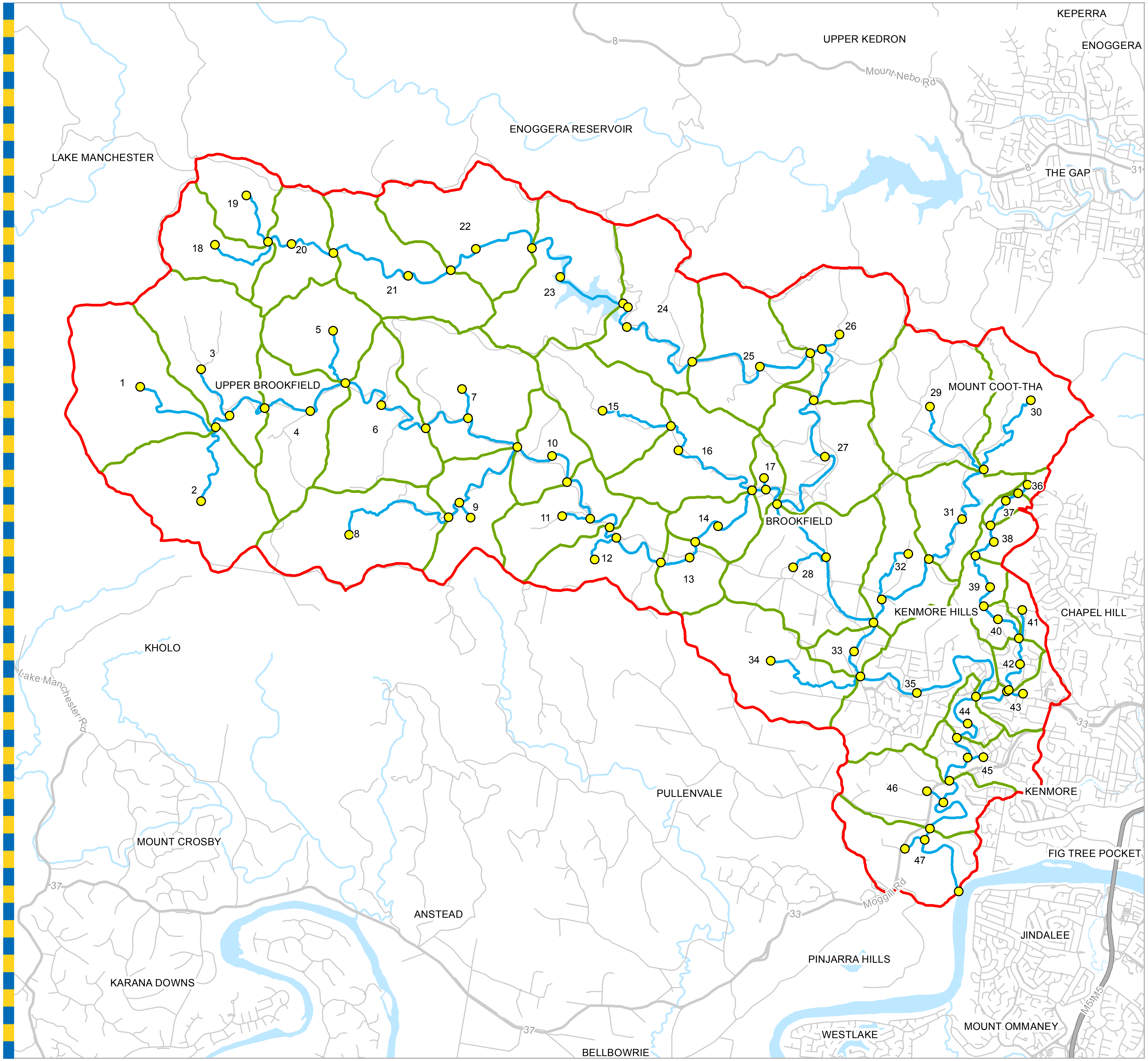
The URBS model comprised 47 sub-catchments and the layout is indicated in Figure 4.1. Based on a total catchment area of 65.8 km^2, this results in an average sub-catchment size of 1.4 km^2. The sub-catchment delineation was based upon the 2014 ALS contours and considered the location of major tributaries and hydrometric gauges, as well as man-made boundaries such as Gold Creek Reservoir and major road crossings.

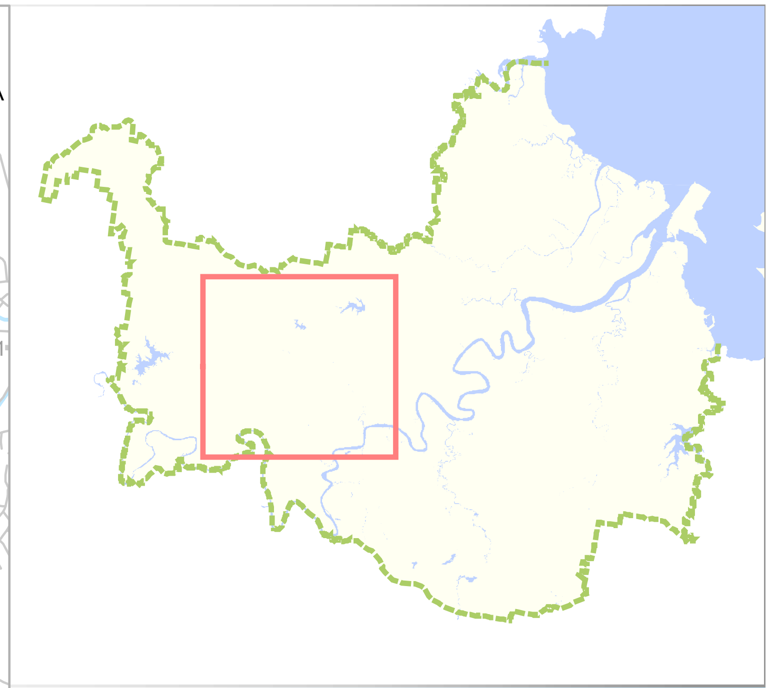
4.2.3 Sub-catchment Slope

Sub-catchment slopes have been calculated from the topography by identifying indicative flow paths and associated equal area slopes. The sub-catchment slope is used to determine the time it takes for flow to travel from the sub-catchment perimeter to the centroid of the sub-catchment. The sub-catchment slopes ranged from over 20 % at the catchment headwaters to less than 3 % in the lower catchment.

4.2.4 Impervious Area

The major development and urban areas are located in the lower section of the catchment. The degree of impervious area occupied by buildings, roads, carparks, etc was determined by using both BCC City Plan 2014 and aerial photography.





Legend

- URBS Nodes
- URBS Links/Routes
- Freeways/Highways
- Major Roads
- Streets
- URBS Subcatchments (1 - 47)
- Catchment Area

N W E S

0 520 1,040 1,560
Metres

Prepared : 099858
Checked : NC
Revision : 0
Publication Date : 23 Jun 2016
Project Number : 160277

Location: G:\BI\CD\Proj16\160277_Moggill_Creek_Flood_Study\ArcGIS\GIM_160277_003.mxd

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Figure 4.1: URBS Model Schematisation

GIM - 160277 - 003

Using BCC City Plan 2014, a percentage impervious for each land-use type was adopted and the corresponding impervious area determined. Aerial photography was then used to cross-check that this value appeared representative from a visual perspective.

The land-use and impervious areas were identified 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.

4.3 Gold Creek Reservoir

Gold Creek Reservoir is a water supply reservoir managed by Seqwater. The reservoir is earth-filled (clay puddle core) with un-regulated spillway at a level of 95.75 mAHD. At a level of 92.75 mAHD, there is a gated 900 mm diameter outflow pipe; which can be used to regulate the water level in the dam between 92.75 and 95.75 mAHD. The major characteristics of the reservoir are indicated in Table 4.1. [6]

Table 4.1 – Gold Creek Reservoir Characteristics

Component	Details
Full Supply Level (FSL)	92.75 mAHD
Piped Outlet (900 mm dia) Invert Level	92.75 mAHD
Full Supply Capacity (92.75 mAHD)	801 ML
Surface Area at FSL (92.75 mAHD)	15.84 ha (1)
Spillway Weir Crest Level	95.75 mAHD
Spillway Weir Length	51.7 m
Main Dam Crest Level	100.15 mAHD

(1) From BCC calculations

The construction of Gold Creek Reservoir was completed in 1886. Since this time, numerous changes have been made, with the most relevant in recent times including:

- Year 2005 - filling of the spillway slot (to 95.75 mAHD) with a concrete structure containing a 900 mm diameter outlet pipe (with slide gate and trash screen) at a nominal invert level of 92.75 mAHD.
- Year 2003 - lowering of the spillway level from 95.75 mAHD to 92.75 mAHD; through construction of a 4 m wide (base) two stage slot in the spillway.
- Year 1997 - lowering of the spillway level from 96.25 mAHD to 95.75 mAHD
- Year 1975 - lowering of the spillway level from 97.45 mAHD to 96.25 mAHD

To enable the reservoir to be incorporated into the URBS hydrologic model, the hydraulic characteristics of the reservoir as well as event specific operational procedures and initial conditions were required to be obtained.

[6] Gold Creek Dam Emergency Action Plan – Seqwater (2014)

The stage-storage-discharge data for the reservoir was obtained from Seqwater. This table is provided in Appendix B and represents the condition when the gate for the 900 mm diameter outlet pipe is open.

BCC undertook some independent checks of the Seqwater stage-storage data using 2014 ALS data (at elevations above 92 mAHD) for which there was good correlation. Independent checks have not been undertaken on the stage-discharge relationship.

Advice from Seqwater indicates that both the approach channel (spillway slot) and the trash screen in front of the outlet pipe are frequently prone to blockage. Seqwater was unable to confirm whether the gate for the 900 mm diameter outlet pipe was open or closed during the four calibration / verification events. However, they believe that it should have been open, but was probably blocked or partially blocked during these events.

Our review of the design discharge results from the 2013 Gold Creek Dam Safety Review Hydrology Report [7] indicates that the status of the gate (i.e. open or closed) does not significantly change the outflow from the reservoir. Therefore, for the purposes of modelling the four calibration / verification events, the gate for the 900 mm diameter outlet pipe has been assumed as open. Refer also to Section 6.2.3.

Table 4.2 indicates the starting levels and volume above / below FSL adopted for the four historical events. For three out of four of the events, the reservoir was already above RL 92.75 mAHD at the commencement of the URBS simulation.

Table 4.2 – Gold Creek Reservoir at Commencement of URBS Simulation

Event	Date / Time	Water Level (mAHD)	Volume above / below FSL 92.75 mAHD (ML)
November 2008	19/11/08 22:00	95.05	478.1
May 2009	19/05/09 18:00	94.62	400.2
January 2013	26/01/13 18:00	92.62	-26.3
May 2015	01/05/15 06:00	92.84	19.8

4.4 Event Rainfall

4.4.1 Observed Rainfall

Recorded rainfall data from each calibration and verification event was incorporated into the URBS model at five minutes intervals, noting that the rainfall gauge only records information when 1 mm or more of rain has fallen.

Thiessen Polygons were utilised for each event to enable the gauged rainfall to be apportioned to each of the sub-catchments in the URBS model. Those sub-catchments which fell totally within a polygon were fully assigned to the respective rainfall station. Those sub-catchments which bridged

[7] Seqwater 2013, *Gold Creek Dam Safety Review Hydrology Report*

across two of more polygons were generally apportioned a weighted average of the total rainfall depth based on the respective rainfall gauges. The Thiessen Polygon distributions for the four events are presented in Appendix A for reference.

4.4.2 Rainfall Losses

The Initial Loss (IL) and Continuing Loss (CL) methodology was used to simulate the rainfall losses. For impervious areas, the URBS model assumes by default that there is no initial loss and 100 % runoff. Therefore, rainfall losses are only subtracted from the pervious portion of the sub-catchment.

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.

4.5 Stream Gauge Rating Curve

In order to undertake the hydrological calibration, the following three stream gauges were utilised:

- 540061 (Moggill Creek at Fortrose Street)
- 143032A (Moggill Creek at Upper Brookfield)
- 540107 (Gold Creek Reservoir Spillway)

To convert gauged water levels into discharge, it was necessary to establish a rating curve at two of the three sites; namely Fortrose Street (540061) and Upper Brookfield (143032A). As mentioned previously, at Gold Creek Reservoir (540107) the Seqwater stage-discharge rating curve of the spillway was adopted, considering an open outlet pipe. BCC Hydrometrics does not keep records of rating curves for stream gauges; therefore it was required to generate a rating curve at Fortrose Street (540061) using the TUFLOW hydraulic model. Similarly, for the Upper Brookfield stream gauge owned by DNRM, the TUFLOW model was used to generate the rating curve. For further discussions on the TUFLOW model refer to Section 5.

The location of the Upper Brookfield (143032A) stream gauge is not ideal to generate a rating curve using a hydraulic model as it is positioned upstream of a bridge structure. Rating curves upstream of hydraulic structures such as bridges that are generated by hydraulic models (e.g. TUFLOW or HEC-RAS) can be subject to sharp changes in water level once the energy grade line (or water level) becomes in contact with the low chord (or soffit) of the structure. This is because hydraulic models generally change the equation used to represent the bridge once the bridge opening becomes pressurised. This rapid change in water level may or may not be realistic and it is difficult to confirm without stream gaugings both upstream and downstream of the structure. Therefore, there is some inherent uncertainty in the rating curve at this location, which should be considered when reviewing the results at this location.

Figure 4.2 indicates the rating curve used at Fortrose Street (540061) and Figure 4.3 indicates the rating curve used at Upper Brookfield (143032A). These rating curves were used for all hydrologic calibration and verification events.

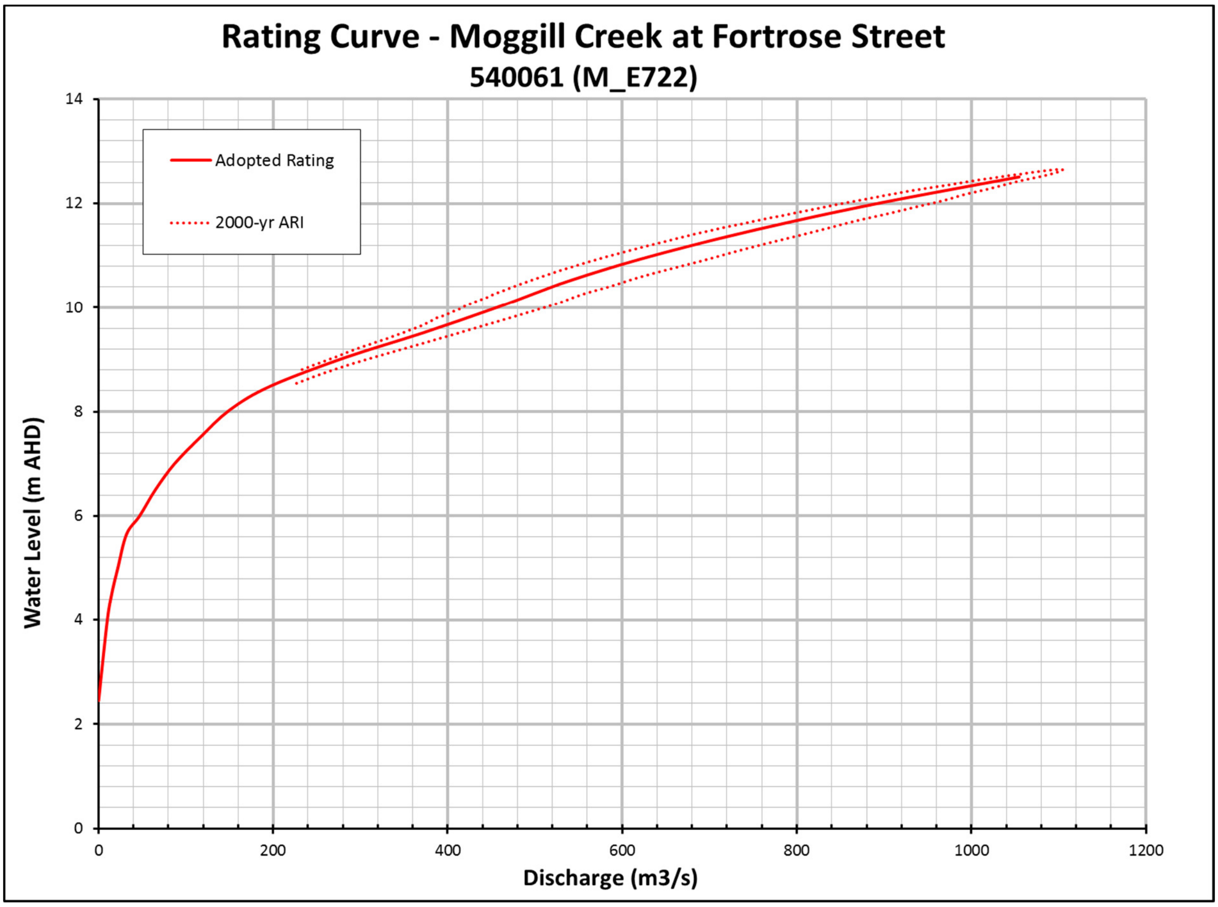

Figure 4.2: Rating Curve – Fortrose Street (540061)

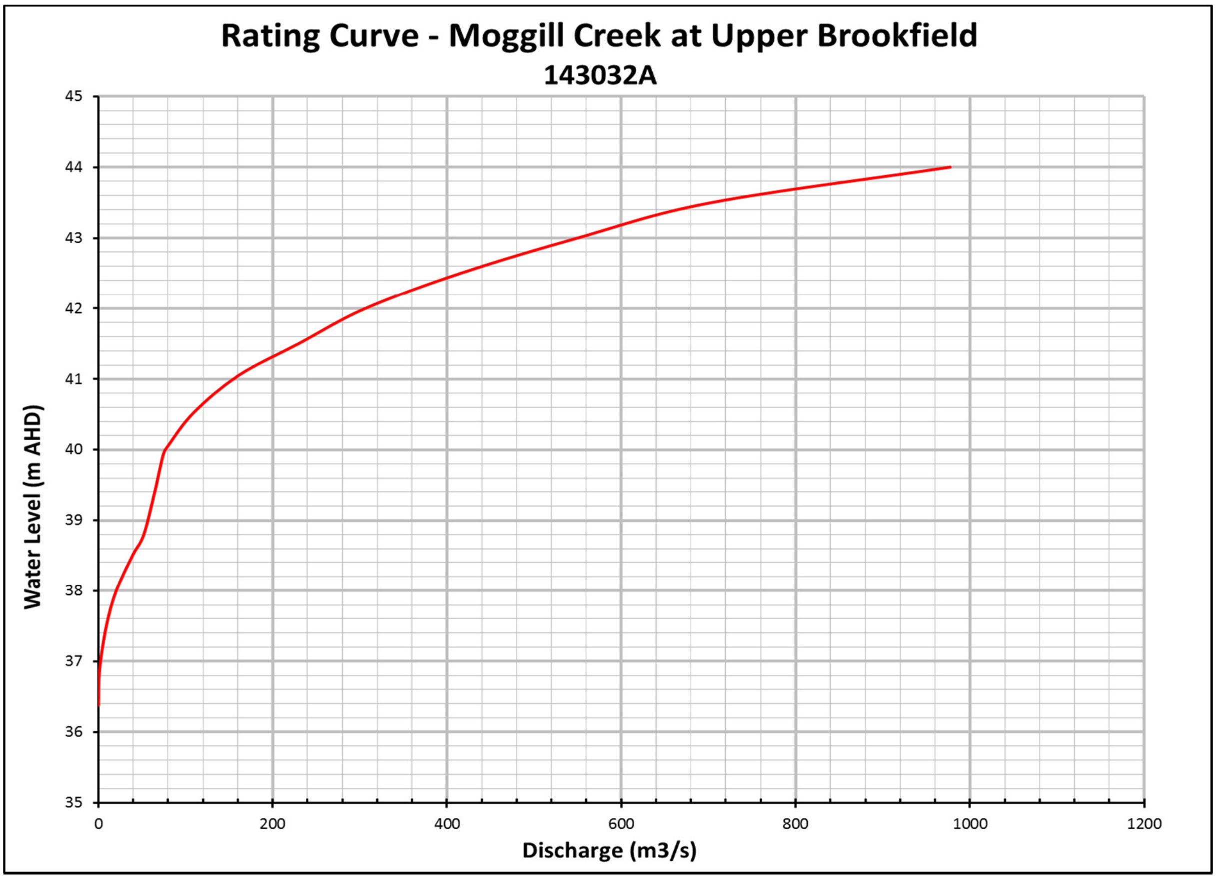

Figure 4.3: Rating Curve – Upper Brookfield (143032A)

At Fortrose Street (540061), there is considerable hysteresis (looping of the rating curve), which can result in quite different rated flows depending on whether the rising limb, falling limb or average of both is used. The hysteresis loop for the 2000-yr ARI (0.05 % AEP) is indicated in Figure 4.2. For this location, the rating curve derivation was undertaken using a gradually increasing flow of which the resultant rating curve lies between the rising limb and falling limb rating curve.

At Upper Brookfield (143032A), there are minimal hysteresis effects; however, the rating curve jumps quite sharply at around 39 m AHD which corresponds with when the flood level reaches the soffit level of the downstream bridge. As noted previously, it is difficult to determine whether this sharp rise is realistic without more detailed gauging at the site.

4.6 Calibration and Verification Procedure

4.6.1 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 URBS model was suitable to be used as a "standalone" model. The general approach adopted for the calibration and verification is indicated in Section 4.6.3.

4.6.2 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 (especially the rising limb)
- Good replication of the timing of peaks and troughs.

4.6.3 Methodology

The methodology applied to the calibration and verification of the URBS 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 4.4.
2) Establish an appropriate rating curve(s) at the stream gauges and convert the stage recordings to flow. This was detailed in Section 4.5.
3) Run the calibration events (i.e. May 2015, May 2009 and November 2008) through the URBS 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); channel lag parameter (α); catchment lag parameter (β) and catchment non-linearity parameter (m).

5) Adopt a single set of model parameters (typically CL, α, β and n) based on the calibration results.

6) Run the verification event (i.e. January 2013) through the calibrated URBS 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. If required, adjust the reach length factor (f) to better replicate the results of the hydraulic model. Refer to Section 5 for more detail on the hydraulic modelling.

4.7 Simulation Parameters

Table 4.3 indicates the start and finish times of the hydrologic simulations as well as the time step used.

Table 4.3 – Hydrologic Simulation Parameters

Event	Start Time	Finish Time	Duration (hours)	Time Step (min)
November 2008	19/11/08 22:00	20/11/08 10:00	12	5
May 2009	19/05/09 18:00	21/05/09 8:00	38	5
January 2013	26/01/13 18:00	28/01/13 18:00	48	5
May 2015	01/05/15 06:00	02/05/15 06:00	24	5

4.8 Hydrologic Model Calibration Results

4.8.1 May 2015

Figures 4.4 to 4.7 provide a comparison of the URBS results and the rated flows (established using the adopted rating curves) at the three gauges. The results indicate a good fit at two of the three gauges; being Fortrose Street (540061) and Gold Creek Reservoir (540107). At these two gauges there is a good replication of the hydrograph shape and timing as well as the peak flow and volume. At Fortrose Street (540061), the simulated peak is approximately 2 % lower than the rated peak flow. At Gold Creek Reservoir (540107), the peak flow is approximately 20 % higher than the rated spillway peak flow and the simulated peak water level 0.07 m higher than the recorded.

At Upper Brookfield (143032A), it was not possibly to obtain a good fit to the observed hydrograph. Contributing factors could include the following:

- The adopted Thiessen polygon distribution of rainfall across the URBS sub-catchments did not mirror reality, resulting in the simulation of more intense rainfall and higher flows than actually occurred.
- Inaccuracies in the rating curve, especially at levels around the bridge deck, as noted previously in Section 4.5
- Missing sub-hourly recorded data, resulting in inaccuracies in the recorded hydrograph shape, as noted previously in Section 3.2.2.

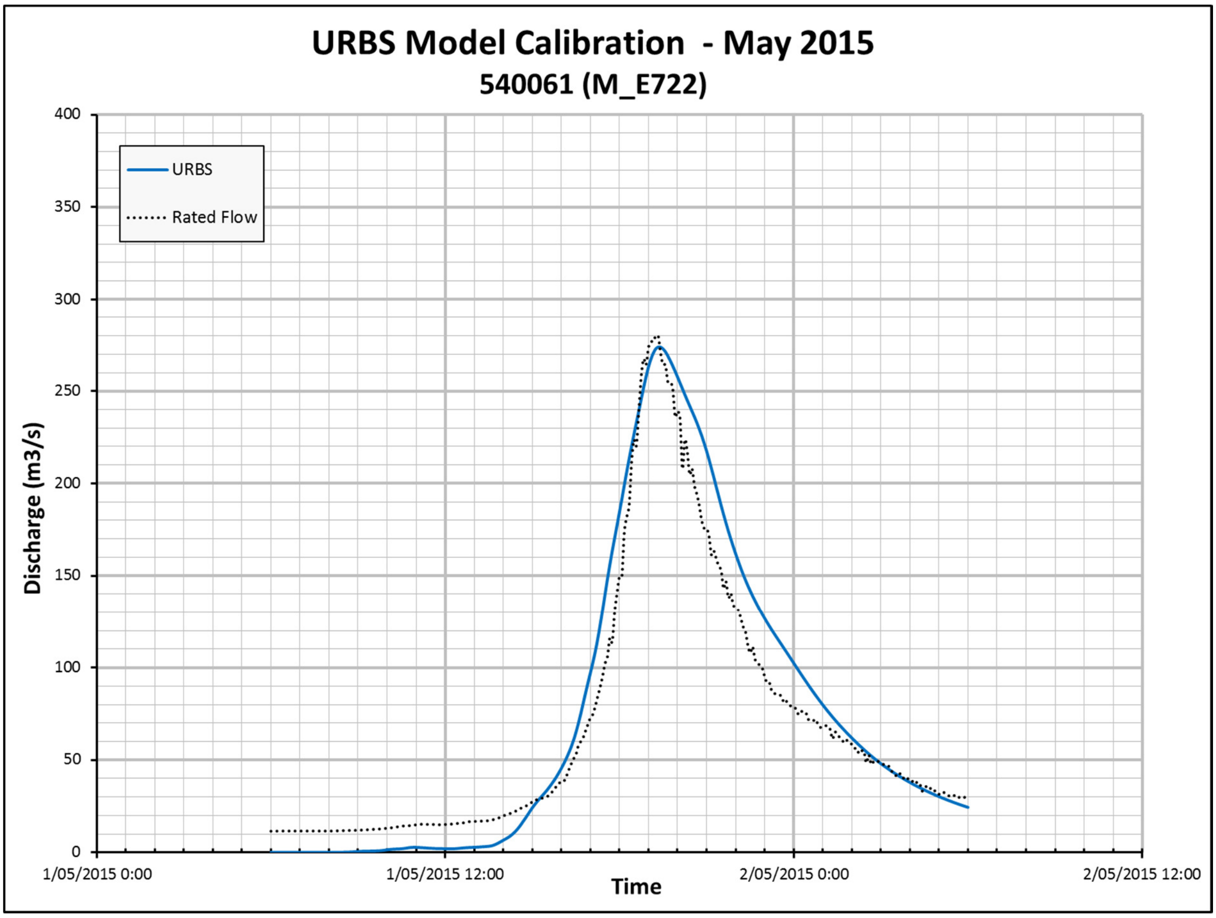

Figure 4.4: May 2015 URBS Model Calibration at 540061 (M_E722)

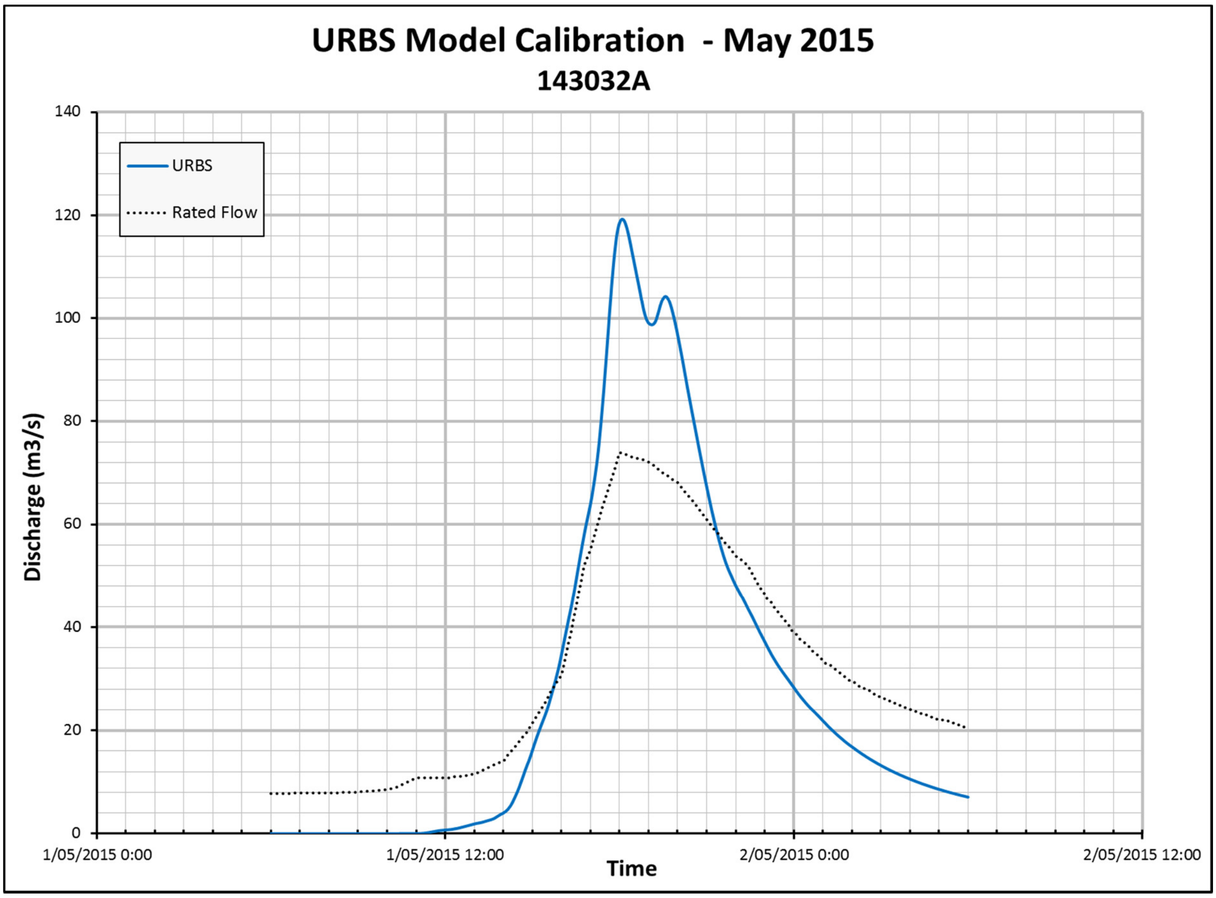

Figure 4.5: May 2015 URBS Model Calibration at 143032A

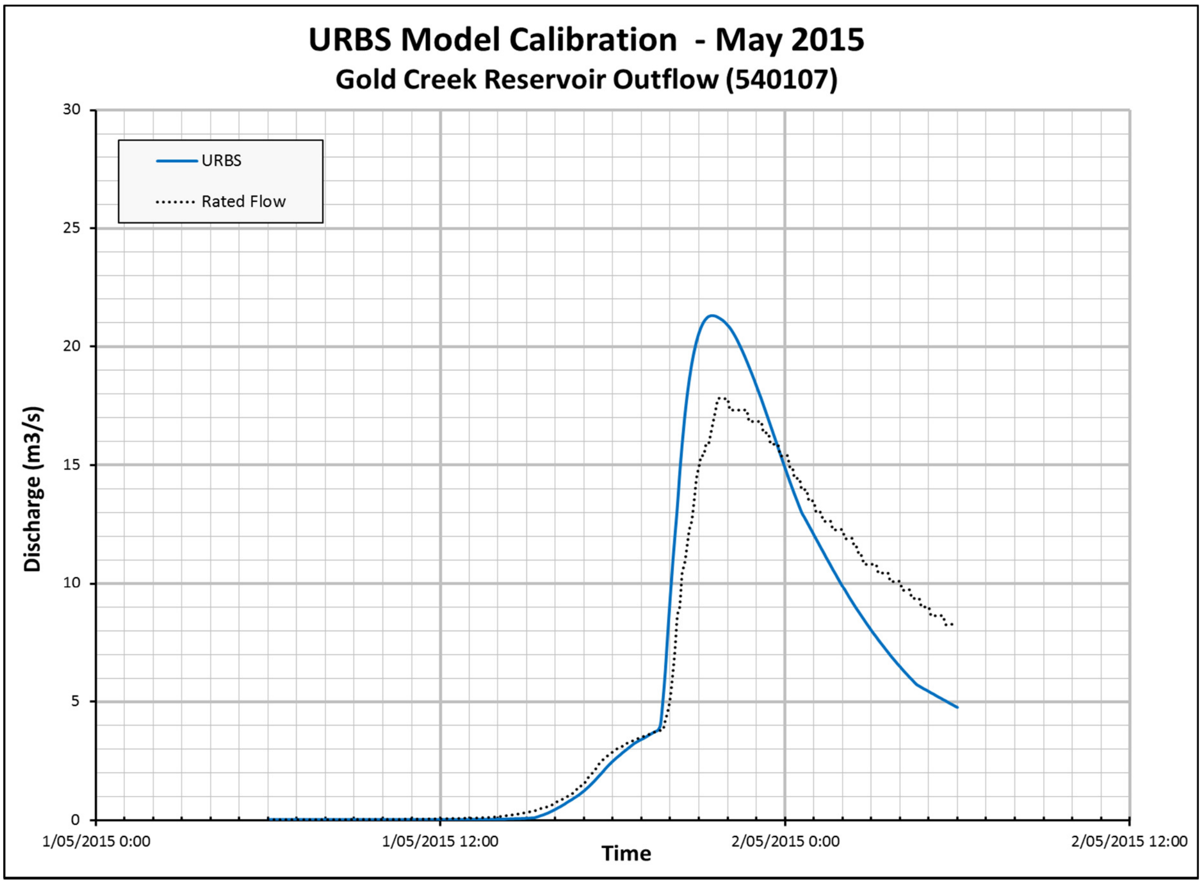

Figure 4.6: May 2015 URBS Model Calibration at 540107 (Flow)

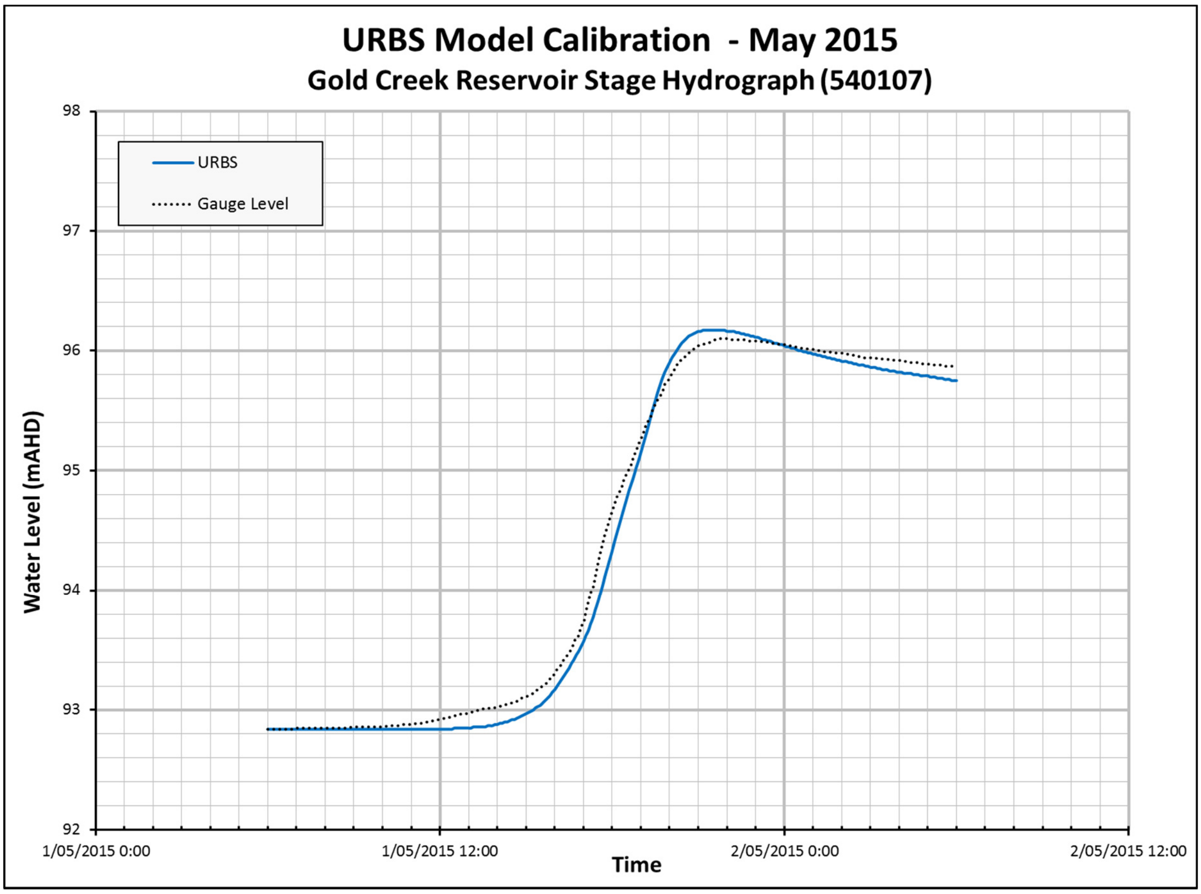

Figure 4.7: May 2015 URBS Model Calibration at 540107 (Stage)

The adopted URBS parameters as part of the calibration were as follows:

- Impervious Area: IL = 0 mm, CL = 0 mm/hr (URBS default)
- Pervious Area: IL = 35 mm, CL = 2.5 mm/hr
- Catchment lag parameter (β) = 5
- Channel lag parameter (α) = 0.008
- Catchment non-linearity parameter (m) = 0.65

Further discussion on the calibration is provided in Section 5.5.

4.8.2 May 2009

Figures 4.8 to 4.11 provide a comparison of the URBS results and the rated flows (established using the adopted rating curves) at the three gauges. The results indicate a good replication of the shape and timing at all three gauges; however the flows are consistently lower than the rated flow. At Upper Brookfield (143032A), the simulated peak flow is approximately 15 % lower than the rated peak flow. At Fortrose Street (540061), the simulated peak flow is approximately 20 % lower than the rated peak flow. At Gold Creek Reservoir (540107), the simulated peak flow is approximately 8 % lower than the rated spillway peak flow and the simulated peak water level 0.06 m lower than the recorded.

Peak flow and volume are typically low at all three gauges, of which contributing factors could include:

- The adopted Thiessen polygon distribution of rainfall across the URBS sub-catchments did not mirror reality, resulting in the simulation of less intense rainfall and lower flows than actually occurred.
- Rainfall gauge recordings not capturing the entire volume of rain which fell.
- Continuing rainfall losses too high – a better fit would be achieved if the continuing loss was set lower than 2.5 mm/hr for this event. However, this would adversely affect the results of the other calibration events.

The adopted URBS parameters as part of the calibration were as follows:

- Impervious Area: IL = 0 mm, CL = 0 mm/hr (URBS default)
- Pervious Area: IL = 10 mm, CL = 2.5 mm/hr
- Catchment lag parameter (β) = 5
- Channel lag parameter (α) = 0.008
- Catchment non-linearity parameter (m) = 0.65

Further discussion on the calibration is provided in in Section 5.5.

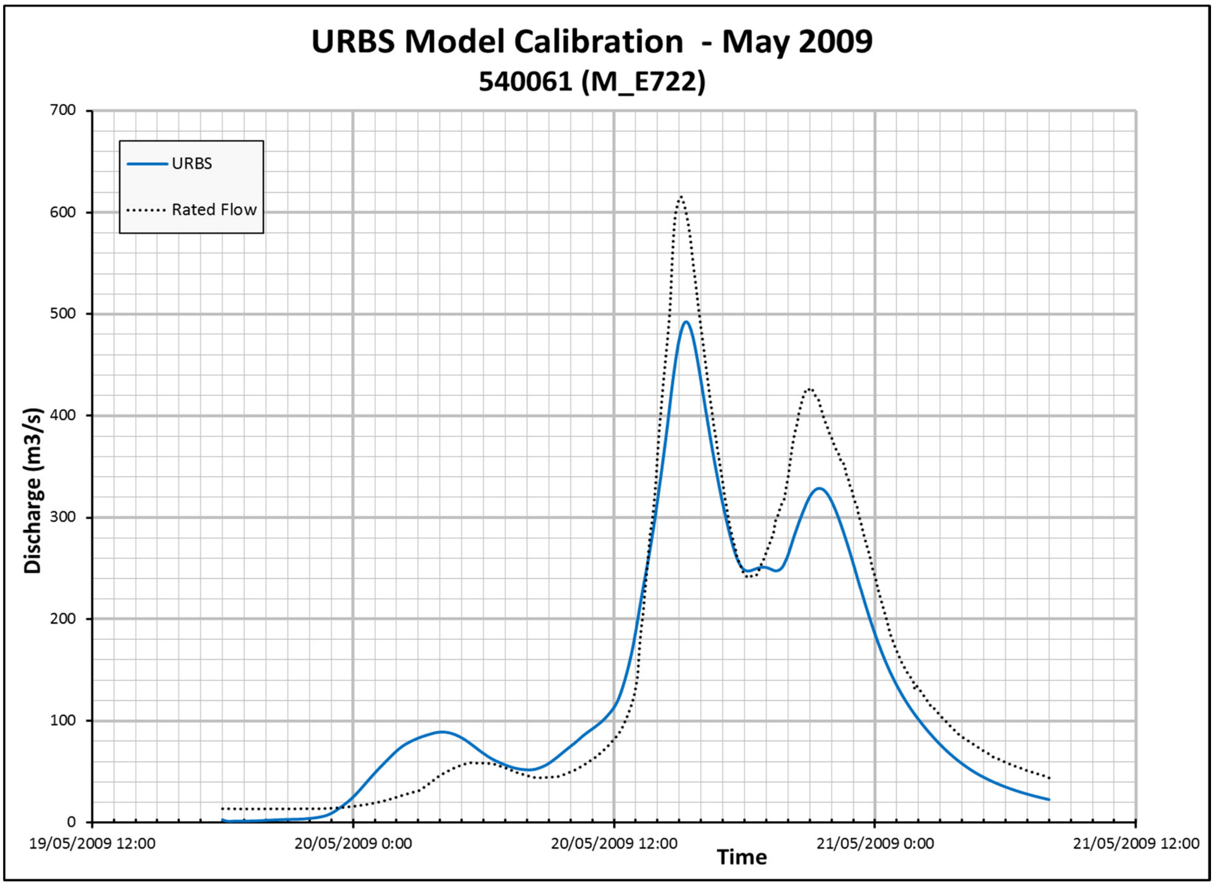

Figure 4.8: May 2009 URBS Model Calibration at 540061 (M_E722)

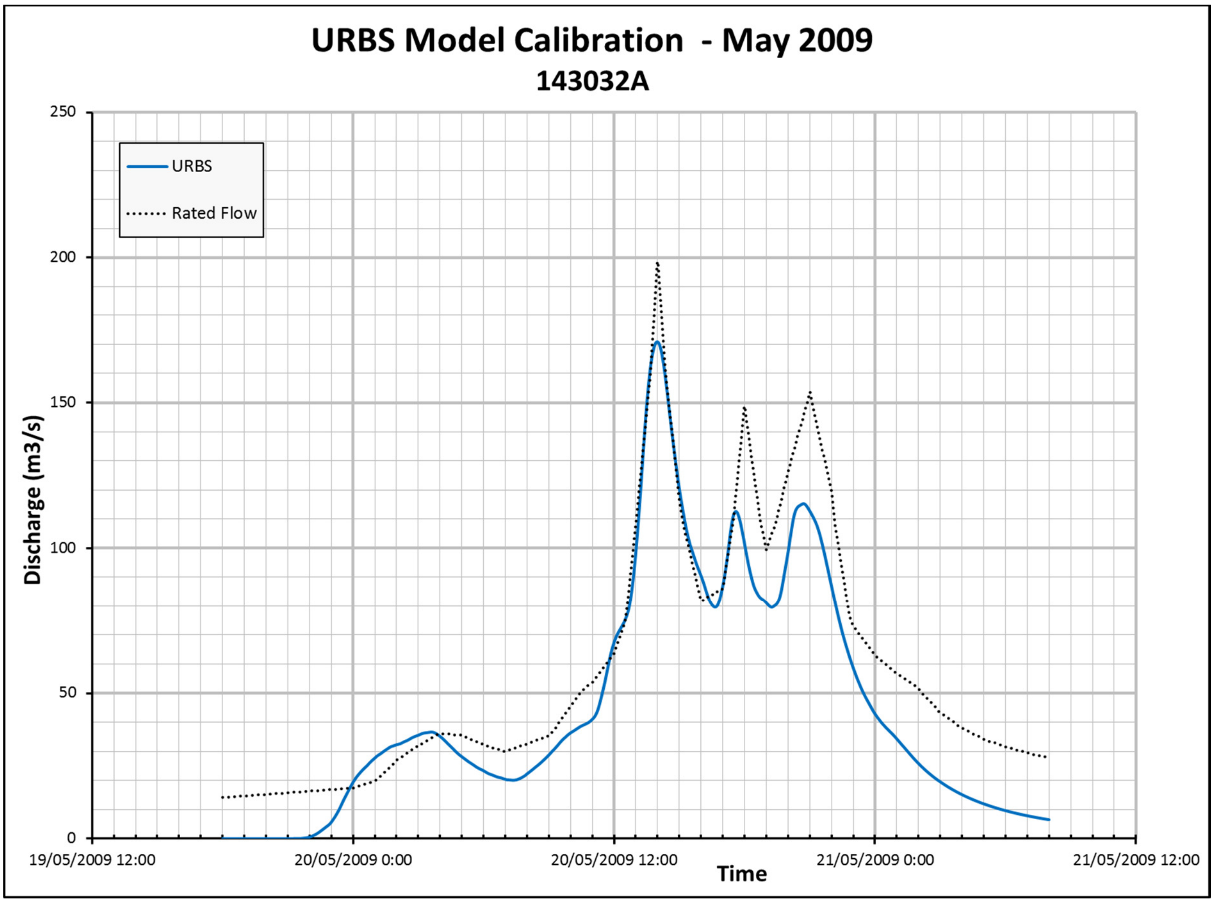

Figure 4.9: May 2009 URBS Model Calibration at 143032A

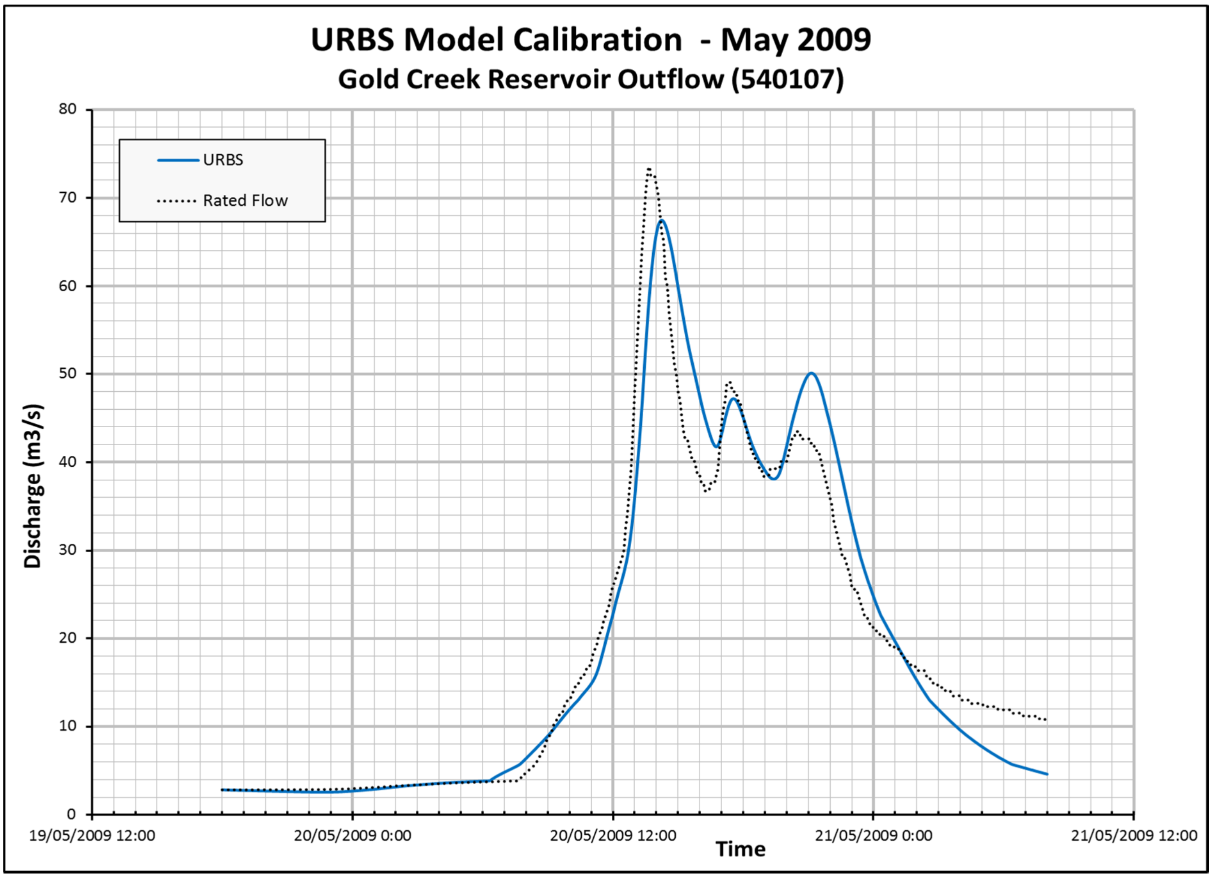

Figure 4.10: May 2009 URBS Model Calibration at 540107 (Flow)

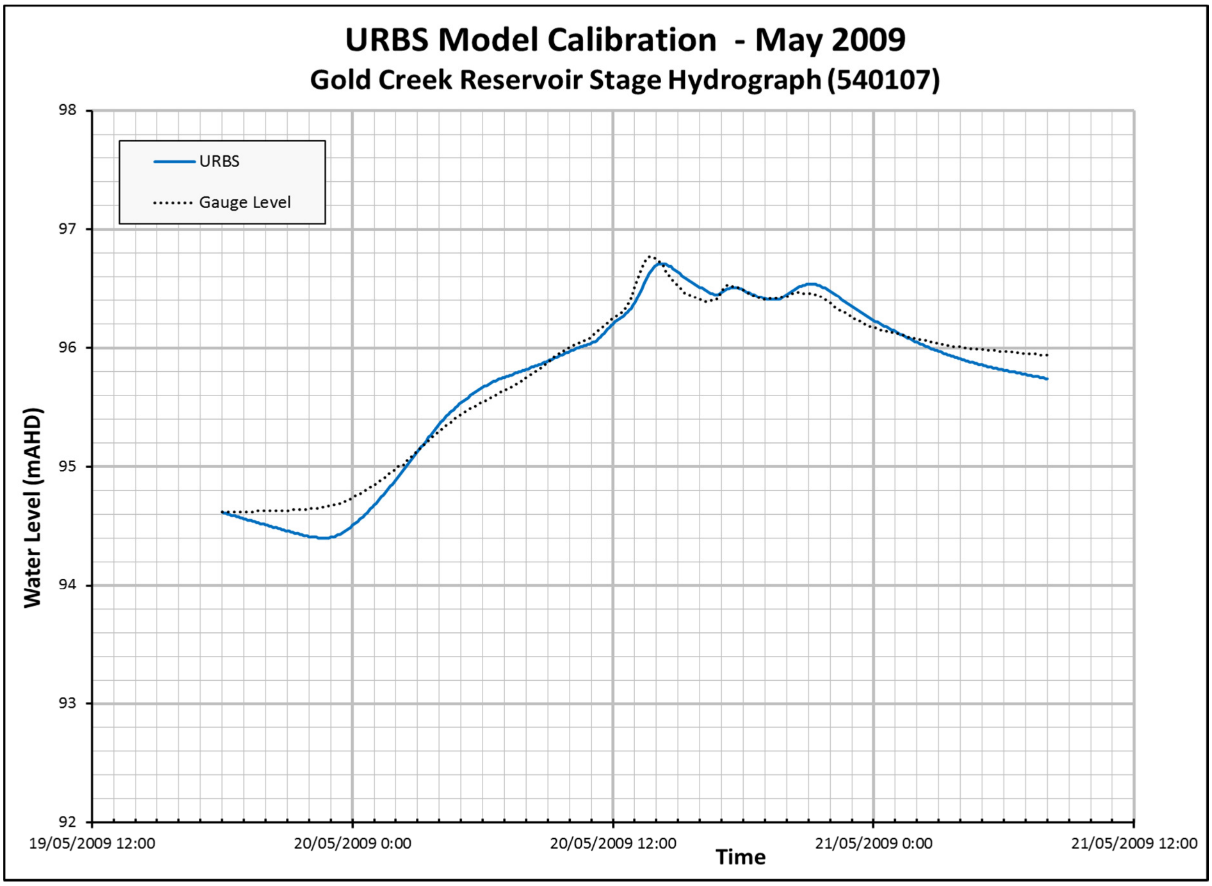

Figure 4.11: May 2009 URBS Model Calibration at 540107 (Stage)

4.8.3 November 2008

Figures 4.12 to 4.15 provide a comparison of the URBS results and the rated flows (established using the adopted rating curves) at the three gauges. The results indicate a reasonable fit at all three gauges with respect to the timing and shape of the hydrograph. At Upper Brookfield (143032A), the simulated peak flow is approximately 14 % lower than the rated peak flow. At Fortrose Street (540061), the simulated peak flow is approximately 1 % higher than the rated peak flow. At Gold Creek Reservoir (540107), the simulated peak flow is approximately 17 % higher than the rated spillway peak flow and the simulated peak water level 0.07 m higher than the recorded.

The adopted URBS parameters as part of the calibration were as follows:

- Impervious Area: IL = 0 mm, CL = 0 mm/hr (URBS default)
- Pervious Area: IL = 0 mm, CL = 2.5 mm/hr
- Catchment lag parameter (β) = 5
- Channel lag parameter (α) = 0.008
- Catchment non-linearity parameter (m) = 0.65

Further discussion on the calibration is provided in Section 5.5.

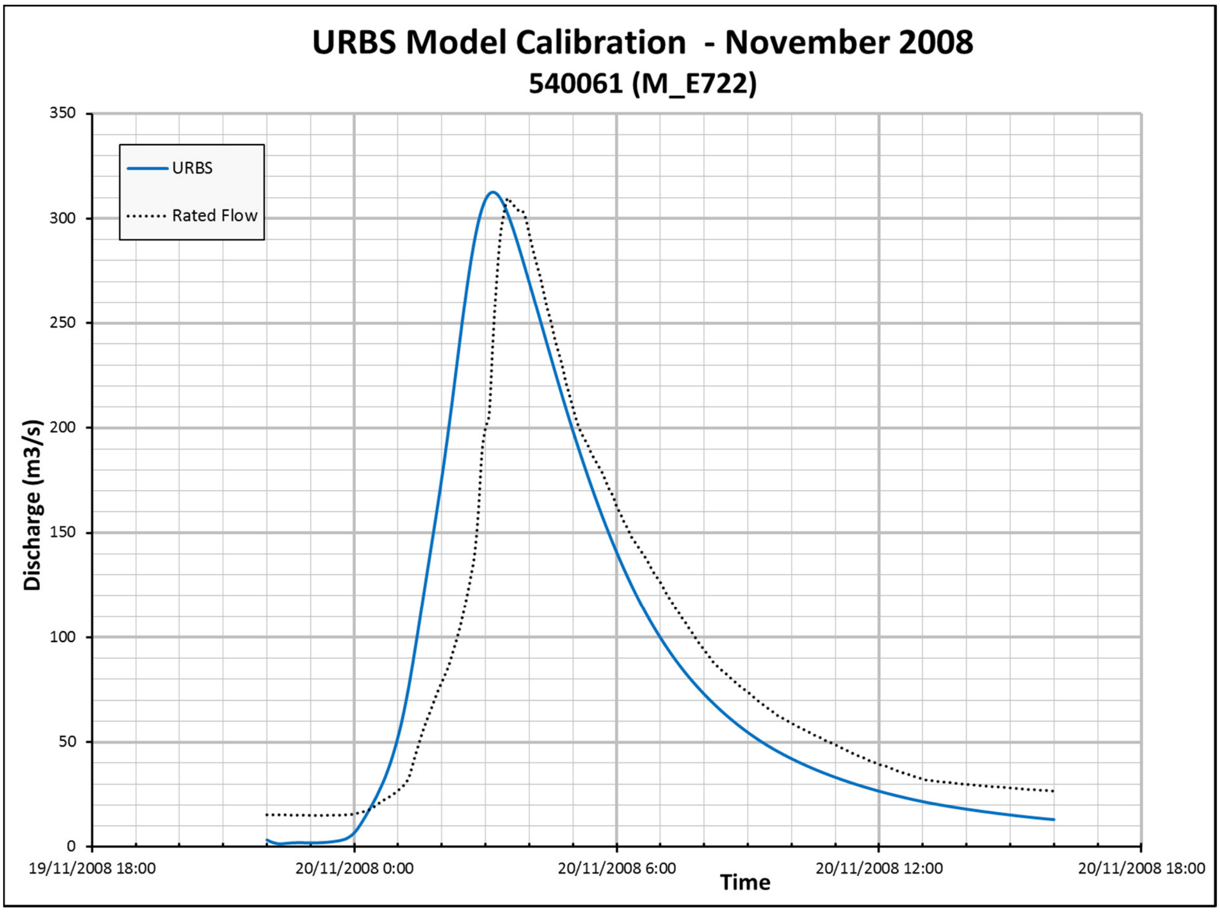

Figure 4.12: November 2008 URBS Model Calibration at 540061 (M_E722)

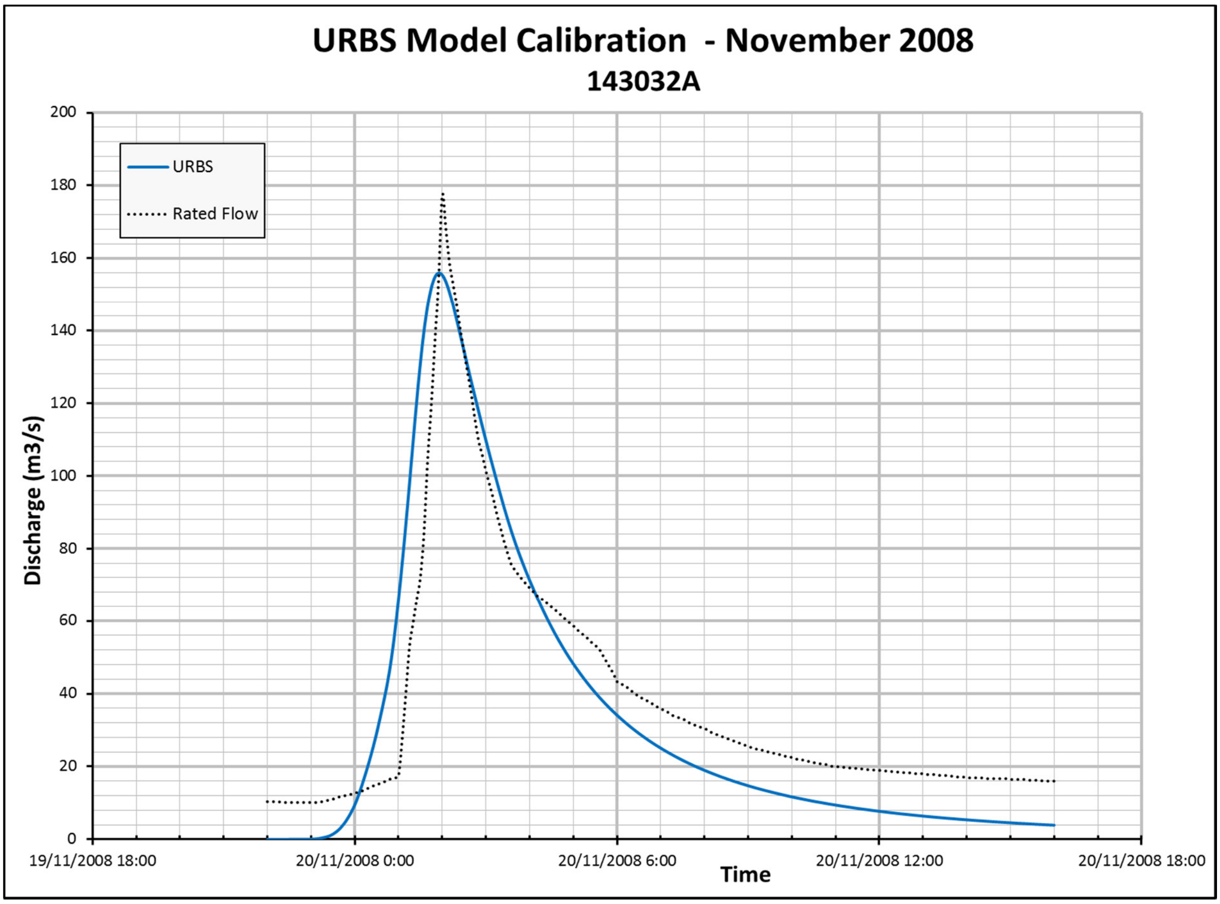

Figure 4.13: November 2008 URBS Model Calibration at 143032A

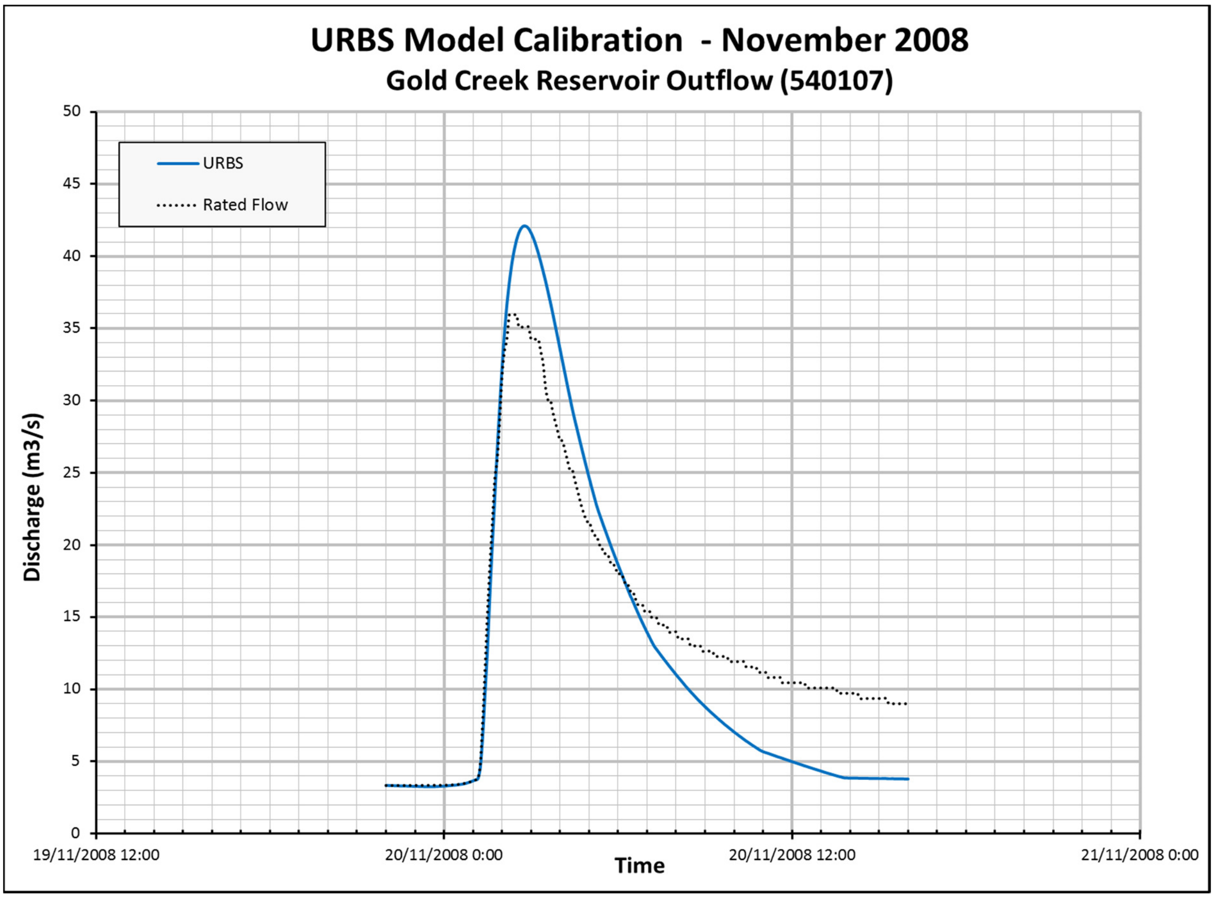

Figure 4.14: November 2008 URBS Model Calibration at 540107 (Flow)

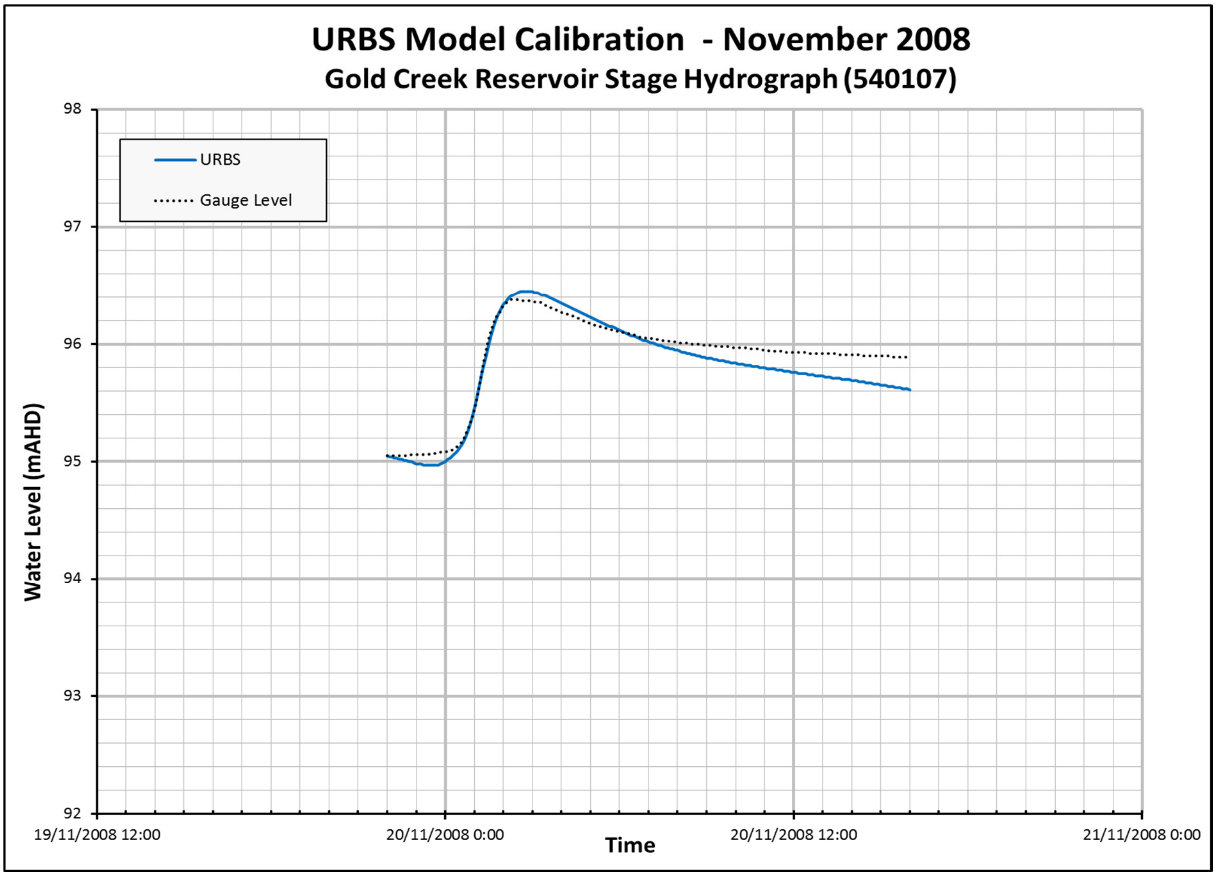

Figure 4.15: November 2008 URBS Model Calibration at 540107 (Stage)

4.9 Hydrologic Model Verification Results

4.9.1 Adopted model parameters

Table 4.4 indicates the parameters adopted from the hydrologic calibration of the three historical events. These parameters were used to verify the URBS model to the one verification event (i.e. January 2013).

Table 4.4 – Adopted URBS parameters

Parameter	Description	Adopted Value
Imp CL	Impervious Area Continuing Loss (mm/hr)	0
Perv CL	Pervious Area Continuing Loss (mm/hr)	2.5
α	Channel lag parameter	0.008
β	Catchment lag parameter	5
m	Catchment non-linearity parameter	0.65

4.9.2 January 2013

Using the adopted model parameters, the January 2013 event was simulated in URBS. Figures 4.16 to 4.19 provide a comparison of the URBS results and the rated flows (established using the adopted rating curves) at the three gauges. The results indicate a reasonable fit at all three gauges with respect to the timing and shape of the hydrograph. At Upper Brookfield (143032A), the simulated peak flow is approximately 14 % higher than the rated peak flow. At Fortrose Street (540061), the simulated peak flow is approximately 1 % lower than the rated peak flow. At Gold Creek Reservoir (540107), the simulated peak flow is approximately 9 % higher than the rated spillway peak flow and the simulated peak water level 0.05 m higher than the recorded.

The adopted URBS rainfall loss parameters adopted for this simulation were as follows:

- Impervious Area: IL = 0 mm, CL = 0 mm/hr (URBS default)
- Pervious Area: IL = 15 mm, CL = 2.5 mm/hr

Further discussion on the verification is provided in Section 5.6.

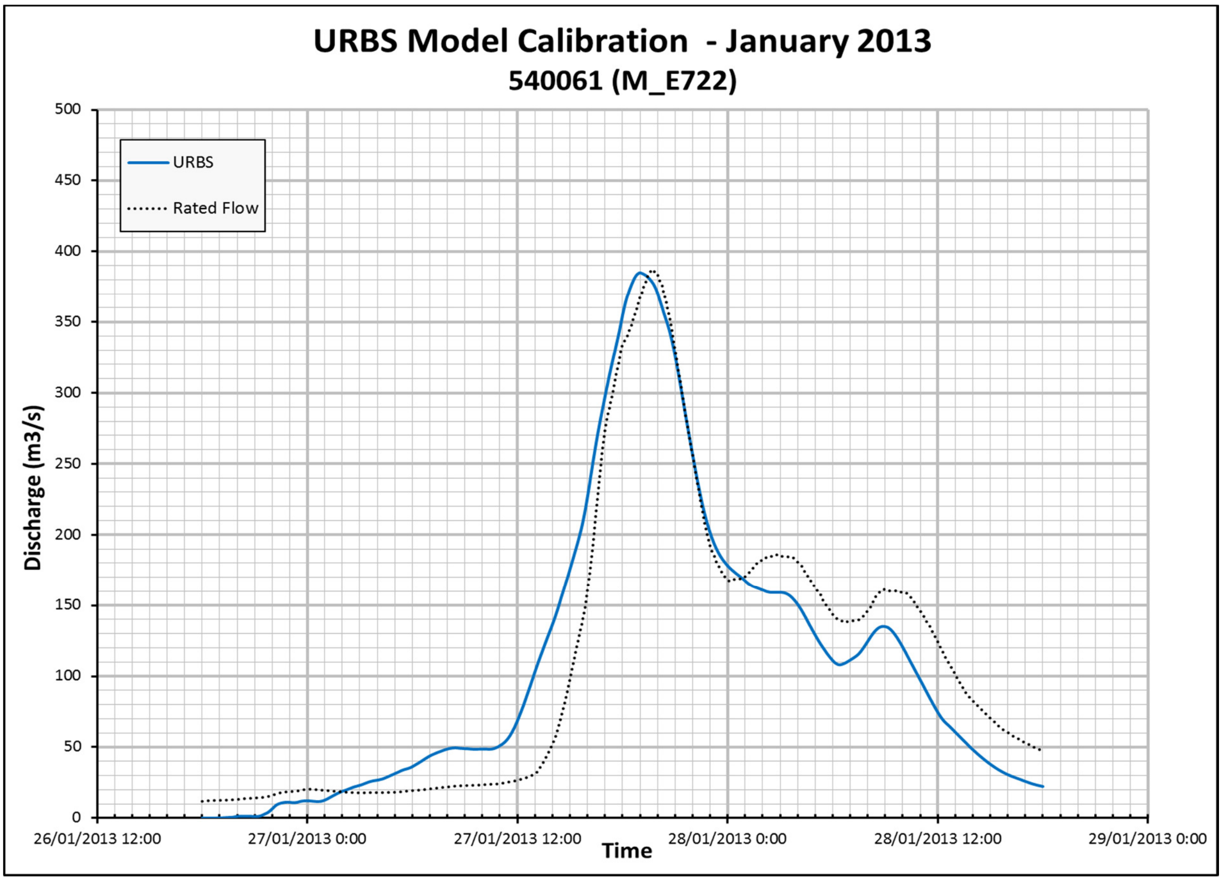

Figure 4.16: January 2013 URBS Model Calibration at 540061 (M_E722)

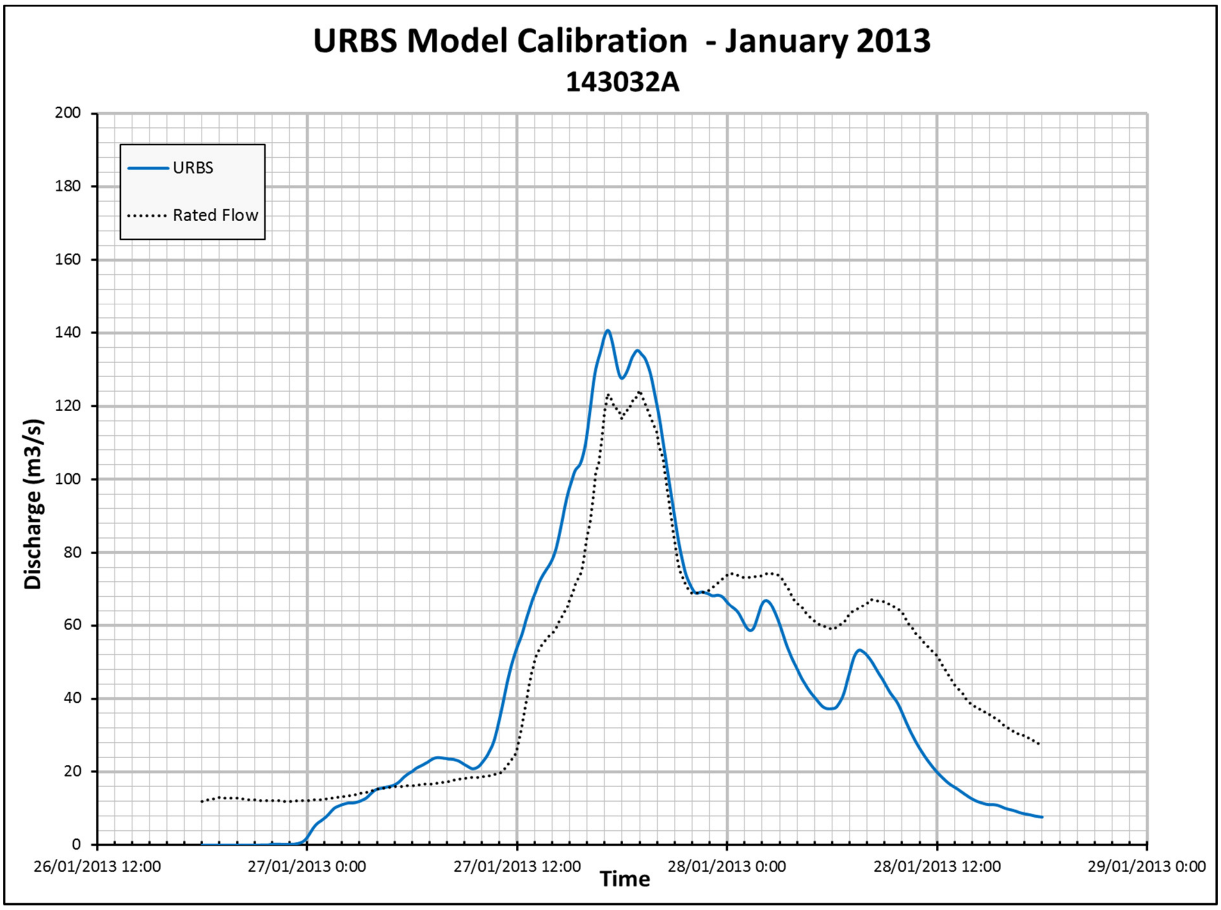

Figure 4.17: January 2013 URBS Model Calibration at 143032A

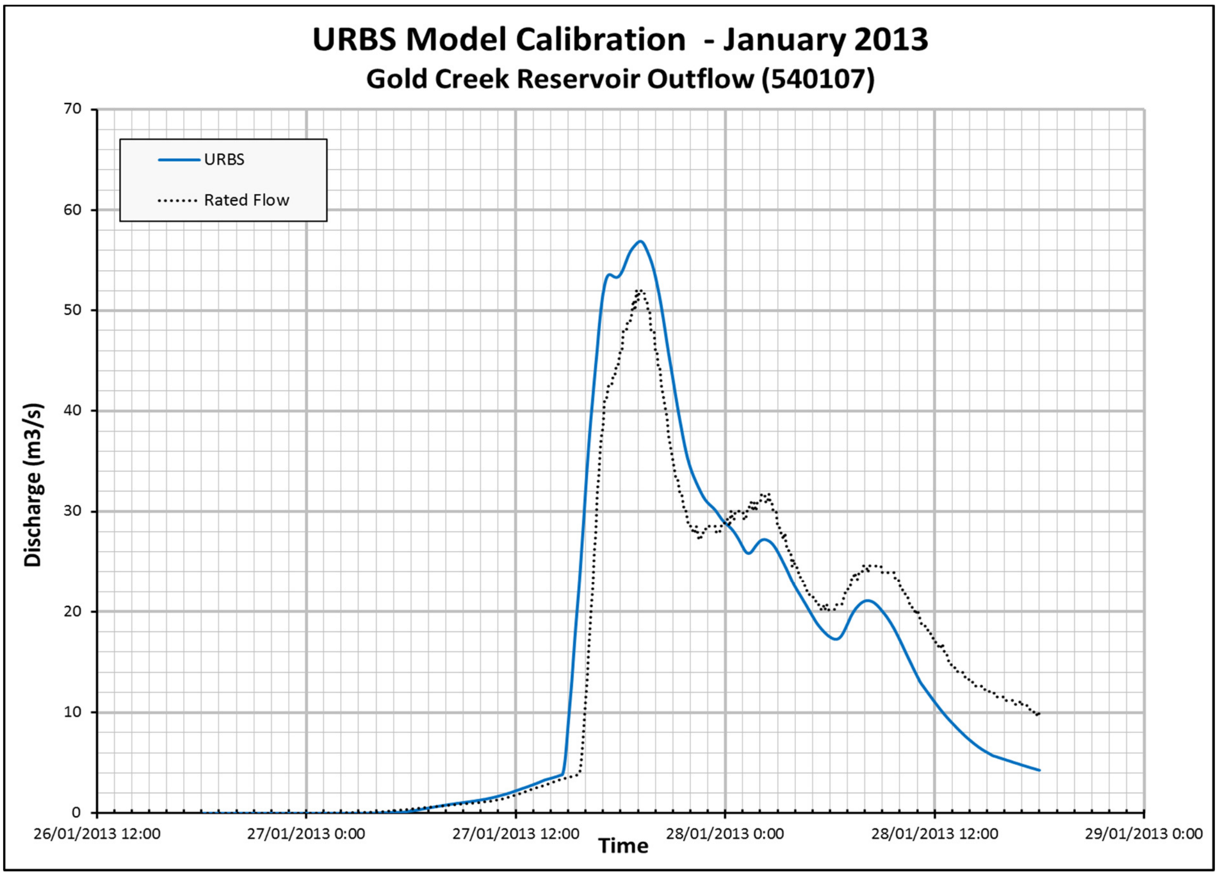

Figure 4.18: January 2013 URBS Model Calibration at 540107 (Flow)

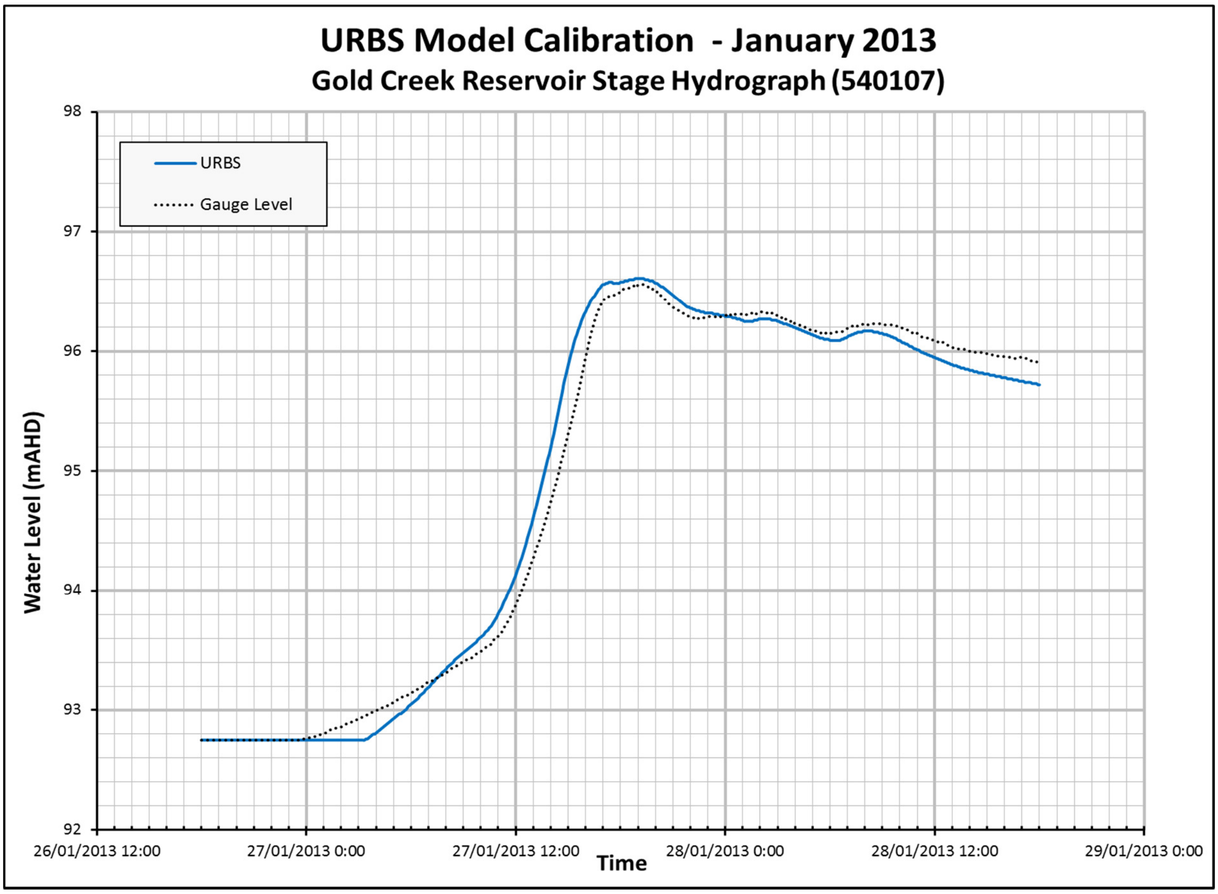

Figure 4.19: January 2013 URBS Model Calibration at 540107 (Stage)

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5.0 Hydraulic Model Development and Calibration

5.1 Overview

The previous hydraulic model of Moggill Creek was a 1d MIKE11 model, developed for the 1994 Flood Study. The previous McKay Brook model was a 1d HEC-RAS model, developed for the 1999 SWMP. To achieve best practice, it was considered appropriate to upgrade and combine the two 1d models into a single 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 the Moggill Creek Catchment.

5.2 Available Data

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

- MIKE11 model – 1994 Flood Study
- HEC-RAS model – 1999 McKay Brook SWMP
- BCC 1983 cross-section survey of Moggill, Gap and Gold Creeks
- BCC 1993 hydraulic structure survey of Moggill, Gap and Gold Creeks
- BCC 1997 cross-section survey of McKay Brook
- BCC December 2015 cross-section survey (forty cross-sections)
- Aerial photography – 1997 to 2015
- 2014 Airborne Laser Scanning (ALS) data
- BCC City Plan 2014
- 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 seven major sections on the basis of the creek / drainage type and the modelling methodology as follows:

- Moggill Creek (Upper Reach - Kittani Street to confluence with Gold Creek) – this reach extends from upstream of Kittani Street to the confluence with Gold Creek; a length of approximately 6 km. The reach is open waterway surrounded by rural properties. The Savages Road Tributary feeds into this reach approximately 500 m upstream of the confluence. This reach has been modelled as 1d / 2d and includes all major hydraulic structures.

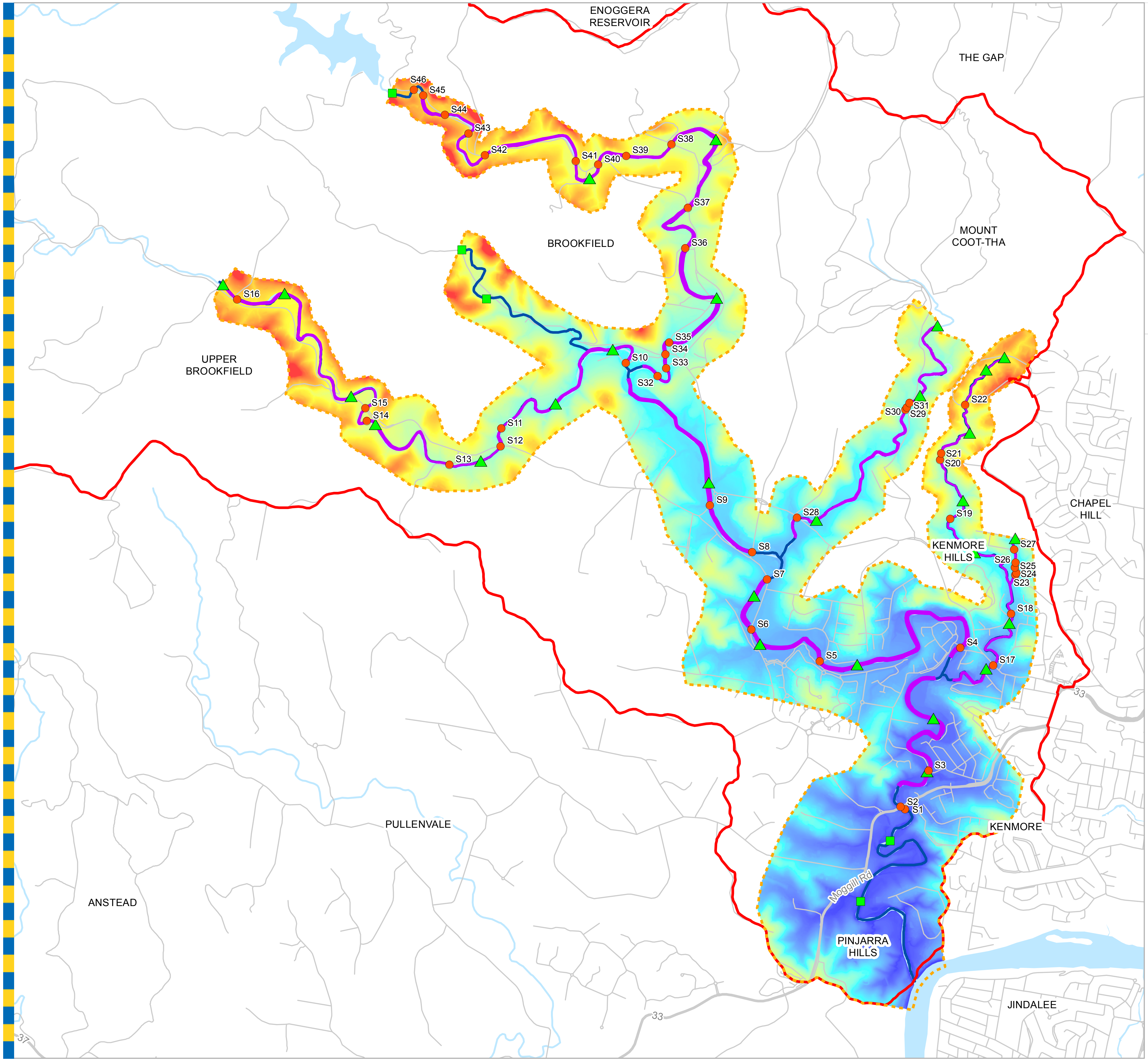
- Savages Road Tributary – this reach is modelled from downstream of a private crossing at 293 Savages Road and extends to the confluence with Moggill Creek; a length of approximately 1.8 km. This reach is located in a rural area with several private driveway crossings and numerous crossings of Savages Road. The entire length has been modelled as part of the 2d grid and is based on 2014 ALS data. The modelling has not included hydraulic structures such as driveway and road crossings. Surveyed spot levels were obtained of the channel invert at a number of locations, from which the channel invert was locally adjusted (if required).

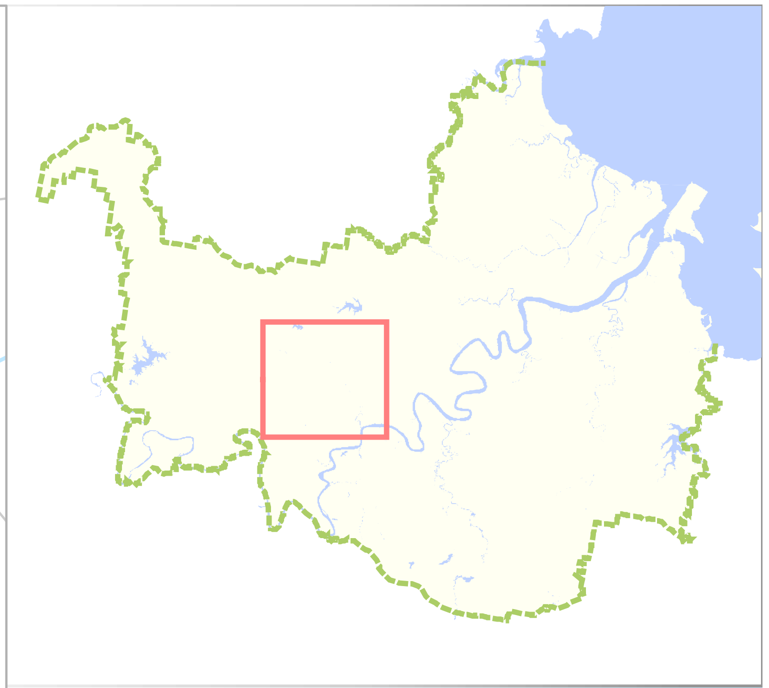
- Gold Creek – this reach begins from downstream of the Gold Creek Reservoir and extends to the confluence with Moggill Creek (immediately downstream of the Bundaleer Road crossing); a length of approximately 7.3 km. This reach is typically surrounded by rural properties and natural forest and is crossed eight times by Gold Creek Road. All major road crossings and most private crossings have been modelled along this reach; a total of 15 crossings. This reach has been modelled as predominantly 1d / 2d, with the exception of a 350 m long section immediately downstream of reservoir, which is modelled as 2d because of the highly sinuous bends.

- Moggill Creek (Middle Reach – Gold Creek confluence to McKay Brook confluence) – this reach extends from downstream of Bundaleer Road to the confluence with McKay Brook; a length of approximately 5.8 km. This reach is surrounded predominantly by rural properties from the upstream extent to the confluence with Gap Creek. Downstream of the Gap Creek confluence, the channel is surrounded by parkland, sports facilities and low density residential properties. This reach is typically modelled as 1d / 2d with the exception of the confluences of Gold and Gap Creeks which are modelled purely as 2d to cater for the complex flow interactions. All major road crossings along this reach have been modelled.

- Gap Creek – this reach is modelled from downstream of the Gap Creek reserve parking area to the confluence with Moggill Creek, approximately 3.1 km in length. This reach is largely surrounded by dense vegetation and bushland, with few significant hydraulic structures. The reach has been modelled as 1d / 2d with the exception of the Gap Creek Road crossing structures which are modelled only in 2d to cater for the complex flow interactions.

- McKay Brook – this catchment only covers a small area of the entire model in comparison with the other larger creeks. The main branch begins downstream of the northern section of Tinarra Crescent and flows into Moggill Creek at the northern side of Kenmore State High School; a length of approximately 4.3 km. The second smaller tributary (0.4 km in length) begins downstream of Elwood Street and discharges into the main McKay Brook branch behind private property at the corner of Billabong and Advanx Streets. The upper reach of the main branch (upstream of the junction with the smaller tributary) is surrounded by dense bushland and is a very steep and incised channel. The smaller tributary and lower reach are surrounded by medium density residential properties with dense vegetation along the banks of the channel. Both tributaries have been modelled as 1d / 2d, with the exception of the confluence with Moggill Creek, which has been modelled as purely 2d to cater for the complex flow interactions. All major road crossing and most private driveway crossings have been modelled.





Legend

- Hydraulic Structures
- 1d Inflow Locations
- 2d Inflow Locations
- Model Boundary
- Catchment Area
- 1d Channel Boundary
- Creek CL
- Freeways/Highways
- Major Roads
- Streets

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Prepared : 099858
Checked : NC
Revision : 0
Publication Date : 23 Jun 2016
Project Number : 160277

Location: G:\BI\CD\Proj16\160277_Moggill_Creek_Flood_Study\ArcGIS\GIM_160277_005.mxd

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Moggill Creek Flood Study

Figure 5.1: TUFLOW Model Schematisation

GIM - 160277 - 005

- Moggill Creek (Lower Reach – McKay Brook confluence to the Brisbane River) – this reach extends from the upstream side of Kenmore State High School to the mouth of Moggill Creek, where it meets the Brisbane River; a length of approximately 5 km. Upstream of Kilkivan Avenue, this reach is surrounded by parkland, sports facilities and low density residential properties. From downstream of this crossing to the Brisbane River, the creek is surrounded by rural properties. All significant hydraulic structures have been modelled along this reach, with the most significant being the Moggill Road crossing. This reach has been modelled in 1d / 2d from Kenmore State High School to upstream of Moggill Road and typically in 2d for the remainder of the reach

5.3.2 Topography

1d Domain

The 1d open channel was generally represented by utilising the channel cross-sectional information from the previous MIKE11 and HEC-RAS models. The cross-sections for Moggill, Gold and Gap Creeks were surveyed in 1983 to enable the development of the MIKE11 model. The cross-sections for McKay Brook were surveyed in 1997 for development of the 1999 SWMP HEC-RAS model.

The 1983 and 1997 survey information was supplemented with forty cross-sections from survey undertaken in December 2015. The location of the December 2015 surveyed cross-sections was selected at sites where the previously surveyed cross-sections appeared least representative of the channel shape compared to the 2014 ALS data. Survey of several structures (typically private) was also undertaken based on the limited available information.

Due to the highly sinuous nature of the main creeks within the catchment, head-losses due to bends of at least 90 degrees were included and added as a form loss to the 1d channel. The methodology used to determine the bend-loss coefficient is as outlined in Section 9.3.6 of the Queensland Urban Drainage Manual. [8] The loss coefficient is a function of the bend radius and channel width:

$$k_b = 2B/R_c$$

where:

k_b = bend loss coefficient
B = channel width
R_c = centreline radius of bend

2d Domain

The 2d bathymetry consisted of a 5 m grid which was created from a 1 m ASCII grid file (MGA Zone 56) of the 2014 ALS data.

The 2014 ALS data was captured as part of the SEQ 2014 LiDAR Capture Project, undertaken by Fugro Spatial Solutions Pty Ltd on behalf of the Queensland Government. The ALS data was acquired from a fixed wing aircraft over Brisbane City Council area on the 28th October 2014.

[8] QLD Department of Energy and Water Supply 2013, *Queensland Urban Drainage Manual (Provisional)*

The SEQ 2014 LiDAR Capture Project's technical processes and specifications were designed to achieve the following data accuracies:

- Vertical data: 0.3 m @ 95 % threshold accuracy
- Horizontal data: 0.8 m @ 95 % threshold accuracy

As part of this flood study, detailed validation checks have not been undertaken on the accuracy of the 2014 ALS data. It is assumed that the data is representative of the topography and "fit for purpose."

Some minor reaches of creek have been represented as fully 2d. For these reaches, the TUFLOW 2d "z-shape" function was used to better represent the creek invert levels. The "z-shape" approach utilised invert levels based on the best available cross-sectional information. These reaches include:

- Savages Road Tributary – entire reach
- Confluence of Moggill and Gold Creeks
- Confluence of Moggill and Gap Creeks
- Confluence of Moggill Creek and McKay Brook
- Moggill Creek - from upstream of Moggill Road to the Brisbane River confluence

Downstream of the Kilkivan Avenue causeway (500 m upstream of Moggill Road) there was a small area identified where the 2014 ALS data had picked up the height of the tall grass and not the ground level. At this location, the DEM was modified to represent the actual ground level using the "z-shape" function in TUFLOW.

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 discretisation of the land-use and topographical areas was undertaken utilising a combination of aerial photography, BCC City Plan 2014 and a number of site visits.

Typically, in the upper and middle reaches of the catchment (rural areas), detailed discretisation of the vegetation layers was required to represent the riparian vegetation and vegetated areas within close proximity of the creek. The use of global BCC City Plan Manning's 'n' roughness values was not suitable in these areas.

In the 1d ESTRY section, the Manning's 'n' values ranged from 0.03 to 0.15, depending on the type of channel material and degree of vegetation.

Table 5.1 – Adopted roughness parameters

Topographical feature / Land-use	**Adopted Manning's 'n'**
Land-use BCC City Plan 2014	
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	0.15
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

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, private access crossings and the more significant footbridge crossings. Many of the bridge structures throughout the catchment were complex and not perpendicular to the flow direction. At these locations, a skew angle was used to better represent the total flow area.

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).

The TUFLOW "z-shape" function was utilised to more accurately model the road deck and handrail levels for structures with a 2d representation of the overtopping (weir).

Upper Brookfield Road Crossing 2 (S15)

This crossing incorporates an old causeway, which is situated approximately 15 m downstream of the bridge structure. The causeway is aligned at 45 degrees to the channel flow direction and is the remnants of the original Upper Brookfield Road crossing at this location.

It was initially considered to model this causeway as a 1d weir structure; however this caused significant model stability issues, which proved problematic. It was decided to represent the head-losses from this minor structure using the form loss option within ESTRY. The form loss factor was derived from comparison to a steady flow HEC-RAS model of the Upper Brookfield Road structure, which incorporated the causeway.

Moggill Road Structures (S1 and S2)

The Moggill Road bridge crossing and the series of bulk water supply pipe crossings downstream were represented in TUFLOW as two separate structures using the "2d layered flow constriction" approach. This approach produced a reasonable representation of the head-losses when compared to the steady flow HEC-RAS model and MHG's as part of the calibration process.

Table 5.2 – Hydraulic Structures represented in the TUFLOW model

Creek	Structure ID	AMTD	Structure location	Structure details	Modelled structure representation	Origin of data used for coding the structure
Moggill	S1	2980	D/S Moggill Road	3 x bulk water supply pipelines	2d layered flow constriction	BCC records plus onsite measurements
Moggill	S2	3000	Moggill Road	Three span bridge	2d layered flow constriction	Design drawings plus onsite measurements
Moggill	S3	3550	Kilkivan Avenue	Low level causeway	2d weir only	2014 ALS Data
Moggill	S4	5370	Branton Street	Single span footbridge	1d bridge / 1d weir	Design drawings plus 2015 survey of the creek
Moggill	S5	7300	Creekside Street	Single span footbridge	2d weir only	2014 ALS Data
Moggill	S6	8100	Rafting Ground Road	3 / 3000 x 2400 mm RCBC	1d culvert / 2d weir	Design Drawings plus 12d road design TIN
Moggill	S7	8610	Rafting Ground Road	4 / 3600 x 2700 mm RCBC	1d culvert / 2d weir	Design Drawings
Moggill	S8	9100	Boscombe Road	3 / 300 mm RCP causeway	2d weir only	2014 ALS Data
Moggill	S9	9650	Brookfield Road	Four span bridge	1d bridge / 2d weir	Design drawings plus 1993 Field Book survey
Moggill	S10	11190	Bundeleer Road	Single span bridge	1d bridge / 2d weir	BCC records
Moggill	S11	12900	185 Upper Brookfield Road	Single span private bridge	1d bridge / 2d weir	2015 survey
Moggill	S12	13050	Upper Brookfield Road	Two span bridge	1d bridge / 2d weir	Design drawings plus 2015 survey of creek
Moggill	S13	13530	Haven Road	3 / 1500 mm RCP	1d culvert / 2d weir	1993 Field Book survey plus 2014 ALS Data
Moggill	S14	14530	455 Upper Brookfield Road	Single span private bridge	1d bridge / 1d weir	1993 Field Book survey
Moggill	S15	14750	Upper Brookfield Road	Two span bridge	1d bridge / 2d weir	Design drawings plus 2015 survey of creek

Creek	Structure ID	AMTD	Structure location	Structure details	Modelled structure representation	Origin of data used for coding the structure
Moggill	S16	16920	Kittani Street	3 / 750 mm RCP	1d culvert / 1d weir	2015 survey plus 2014 ALS Data
McKay Brook	S17	490	Brookfield Road	3 / 1800 mm RCP	1d culvert / 2d weir	1997 Field Book survey
McKay Brook	S18	1082	Mirbelia Street	5 / 3000 x 1308 mm RCBC	1d culvert / 2d weir	1997 Field Book survey
McKay Brook	S19	2180	389 Brookfield Road	2 / 1500 mm diameter corrugated iron	1d culvert / 1d weir	1997 Field Book survey
McKay Brook	S20	2832	23-24 Hillcrest Place	2 / 1500 mm RCP	1d culvert / 2d weir	1997 Field Book survey plus 2015 survey
McKay Brook	S21	2881	18 Hillcrest Place	2 / 1500 mm RCP	1d culvert / 1d weir	1997 Field Book survey plus 2015 survey
McKay Brook	S22	3445	Tinarra Crescent	1 / 1350 mm RCP	1d culvert / 2d weir	1997 Field Book survey plus BCC records
McKay Brook Tributary 1	S23	95	6 Billabong Street	1 / 900 mm RCP and 1 / 1050 mm RCP	1d culvert / 1d weir	1997 Field Book survey
McKay Brook Tributary 1	S24	105	10 Billabong Street	Single span private bridge	1d bridge / 1d weir	1997 Field Book survey
McKay Brook Tributary 1	S25	155	16 Billabong Street	1 / 2400 x 750 mm RCBC	1d culvert / 1d weir	1997 Field Book survey
McKay Brook Tributary 1	S26	195	20 Billabong Street	1 / 2400 x 750 mm RCBC	1d culvert / 1d weir	1997 Field Book survey plus 2015 survey
McKay Brook Tributary 1	S27	305	Wexford Street	2 / 1200 mm RCP	1d culvert / 2d weir	1997 Field Book survey
Gap	S28	400	Brookfield Road	Single span bridge	1d bridge / 2d weir	Design drawings
Gap	S29	2010	152 Gap Creek Road	Low level private driveway	2d weir only	2014 ALS Data
Gap	S30	2030	160 Gap Creek Road	Low level private driveway	2d weir only	2014 ALS Data
Gap	S31	2080	Gap Creek Road	Low level causeway	2d weir only	2014 ALS Data

Creek	Structure ID	AMTD	Structure location	Structure details	Modelled structure representation	Origin of data used for coding the structure
Gold	S32	275	132 Gold Creek Road	Low level private causeway	2d weir only	2014 ALS Data
Gold	S33	505	130 Gold Creek Road	Low level private causeway	2d weir only	2014 ALS Data
Gold	S34	620	Savages Road	Two span bridge	1d bridge / 2d weir	Design drawings
Gold	S35	750	Adavale Street	3 / 3200 x 1500 mm RCBC	1d culvert / 2d weir	1993 Field Book survey
Gold	S36	1990	272 Gold Creek Road	Single span arch bridge	1d bridge / 2d weir	Design drawings plus 2015 survey of creek
Gold	S37	2690	Gold Creek Road	Single span bridge	1d bridge / 2d weir	Design drawings
Gold	S38	3775	Jones Road	Low level causeway	2d weir only	2014 ALS Data
Gold	S39	4200	379 Gold Creek Road	2 / 1800 mm RCP	1d culvert / 2d weir	1993 Field Book survey
Gold	S40	4500	Gold Creek Road	1 / 2700 x 1800 mm RCBC	1d culvert / 2d weir	1993 Field Book survey
Gold	S41	4895	Gold Creek Road	1 / 2700 x 1800 mm RCBC	1d culvert / 2d weir	1993 Field Book survey
Gold	S42	5819	Gold Creek Road	3 / 600 mm RCP	2d weir only	2014 ALS Data
Gold	S43	6307	Gold Creek Road	2 / 750 mm RCP	2d weir only	2014 ALS Data
Gold	S44	6655	Gold Creek Road	1 / 1800 x 600 mm RCBC	1d culvert / 2d weir	Design drawings plus 2014 ALS Data
Gold	S45	6955	Gold Creek Road	1 / 1800 x 600 mm RCBC	1d culvert / 2d weir	Design drawings plus 2014 ALS Data
Gold	S46	7100	Gold Creek Road	1 / 1200 x 600 mm RCBC	1d culvert / 2d weir	Design drawings plus 2014 ALS Data

Gap Creek Road Structures (S29 to S31)

There is a series of three low-level structures within a 100 m reach of Gap Creek which include the Gap Creek Road crossing and two private driveway crossings. Due to the low height and small size of the structures (culverts) in comparison to the upstream catchment, it was decided to model these structures as a series of 2d weirs.

McKay Brook Structures

McKay Brook is a significantly smaller catchment than the other three catchments, meaning that flows are also small in comparison. As a result, a relatively smaller sized structure is likely to have a more significant impact on flood levels within this catchment, in comparison to the larger catchments. Consequently, the majority of the smaller private driveway type structures have been included in the model.

Gold Creek Structures

There are a large number of small hydraulic structures (culverts) within Gold Creek downstream of Gold Creek Dam. These structures are subject to large flows whereby the majority of the flow would be comprised of weir flow across the road. As such, most of the smaller causeway culverts have been modelled as weir only. It is also worth noting that many small private property access roads were not modelled within this reach.

Savages Road Tributary Structures

The structures along this tributary are typically low-level crossings and have not been represented in the TUFLOW model as per the agreed flood study scope. The omission of these structures is unlikely to significantly affect the accuracy of flood levels in the larger events, due to the high proportion of flow which overtops the road (in lieu of through the culvert).

5.3.5 Boundary Conditions

Inflow Boundaries

Inflows to the hydraulic model were taken from the URBS 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 URBS model sub-catchment schematisation.

Downstream Boundary

A varying water level versus time (H-T) downstream boundary was typically used to represent the downstream boundary conditions at the mouth of Moggill Creek.

For the May 2015 and January 2013 events, the H-T boundary was based on the Jindalee Alert Gauge (540192); owned by Seqwater.

For the November 2008 events, the H-T boundary was derived from the upstream gauge (540200) at Moggill and the downstream gauge (540274) at the mouth of Oxley Creek; as the gauges in the vicinity of the mouth of Moggill Creek were not working.

For the May 2009 event, time varying data was not available; therefore a fixed water level of 1.27 mAHD was used for the downstream boundary. This is representative of the Mean High Water Springs (MHWS) level reported at the Jindalee Alert Gauge location from the 2016 Queensland Tide

Tables publication. The adoption of MHWS does not impact on the results of the calibration, refer to Section 5.5.2 for further details.

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

At the majority of locations within the 1d-2d linked sections of the model, the 1d channel was linked to the 2d domain using the "HX" type boundary condition.

There are only two exceptions to this methodology, the first being the upstream boundary of the upstream culvert on Rafting Ground Road (S6), where an "SX" type flow boundary condition was used when transitioning from fully 2d into the 1d domain. Similarly, the second being the downstream boundary of the Bundaleer Bridge (S10) when transitioning from 1d to fully 2d.

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 URBS 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 URBS 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 Brisbane River. 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 May 2015

The May 2015 flood was simulated in TUFLOW for 24 hours from 6 am on the 1st May 2015. Figure 5.2 provides a comparison between the TUFLOW (and URBS) results and the gauged flood level at Fortrose Street (540061) in Kenmore.

Figure 5.3 provides a comparison between the TUFLOW (and URBS) results and the gauged flood level at the DNRM stream gauge (143032A) located at Upper Brookfield. Table 5.3 provides a comparison between the TUFLOW results and the recorded peak flood levels at the stream gauges and MHGs which were working during the event.

From review of the peak level / MHG results, it was apparent that at 13 out of 18 locations the desired peak flood level tolerance was able to be achieved. In the higher populated areas of Mid and Lower Moggill Creek, the simulated peak flood level at 9 out of 9 gauges was within the desired tolerance. The five locations where the simulated peak flood level was not able to meet the desired tolerances were upstream of hydraulic structures; where there is inherently considerable uncertainty due to blockages, guard rail / handrail effects, bridge / culvert losses, etc.

At both MHG G120, upstream of the private arch bridge location (S36), and MHG GP100, upstream of the single span bridge on Brookfield Road (S28), it is conceivable that blockage occurred resulting in a considerably higher upstream flood level than the simulated results. It is important to note that the simulated results have not included blockage at structures.

At Fortrose Street (540061), the simulated peak flood level was within the ± 0.15 m tolerance. The simulated rising limb achieved a good fit with the recorded hydrograph; however the simulated falling limb generally did not recede as quickly as the observed. At Upper Brookfield (143032A), the simulated peak flood level was not able to be calibrated to within the ± 0.15 m tolerance. The shape and timing of the hydrograph was good, however the peak was considerably higher than the observed, which was discussed previously in Section 4.8.1.

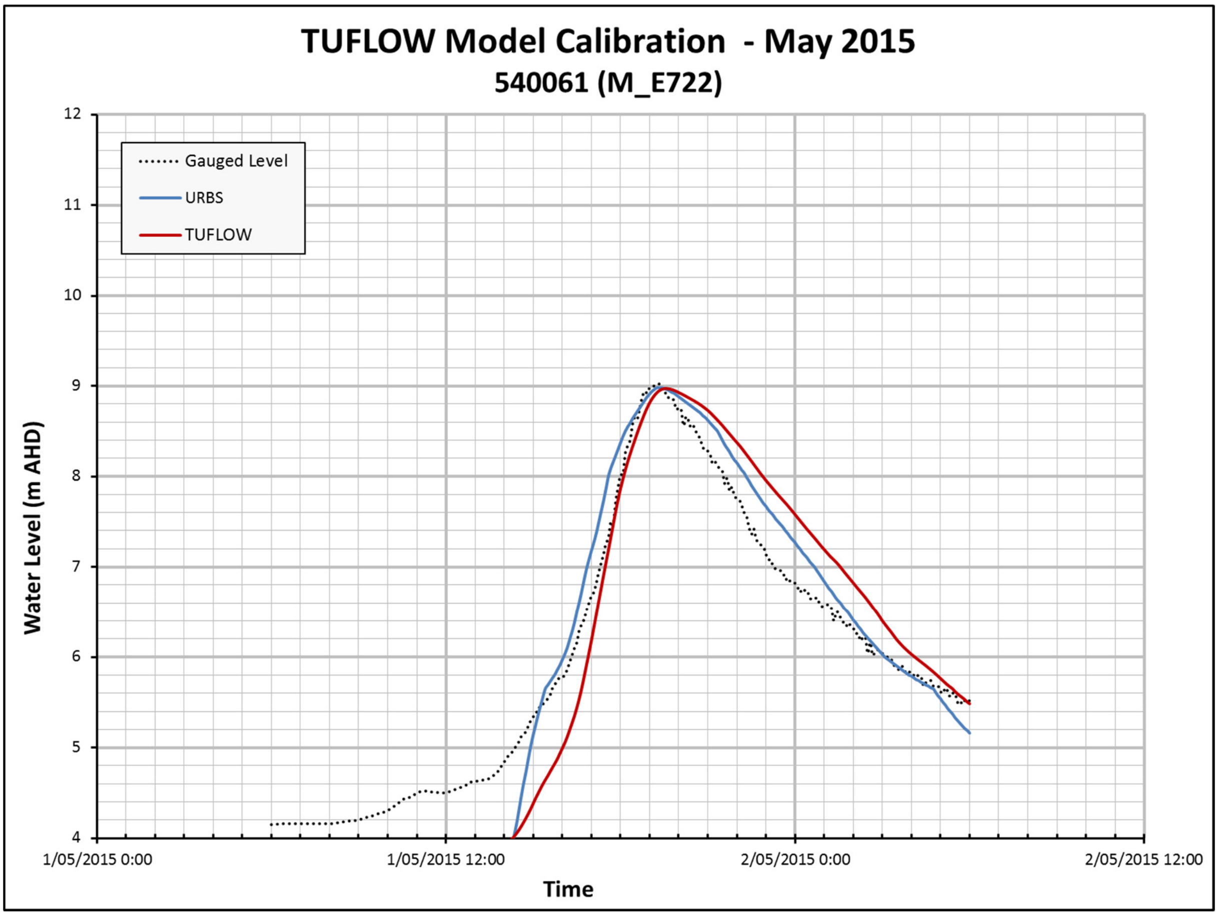

Figure 5.2: TUFLOW Model Calibration at Fortrose Street (May 2015)

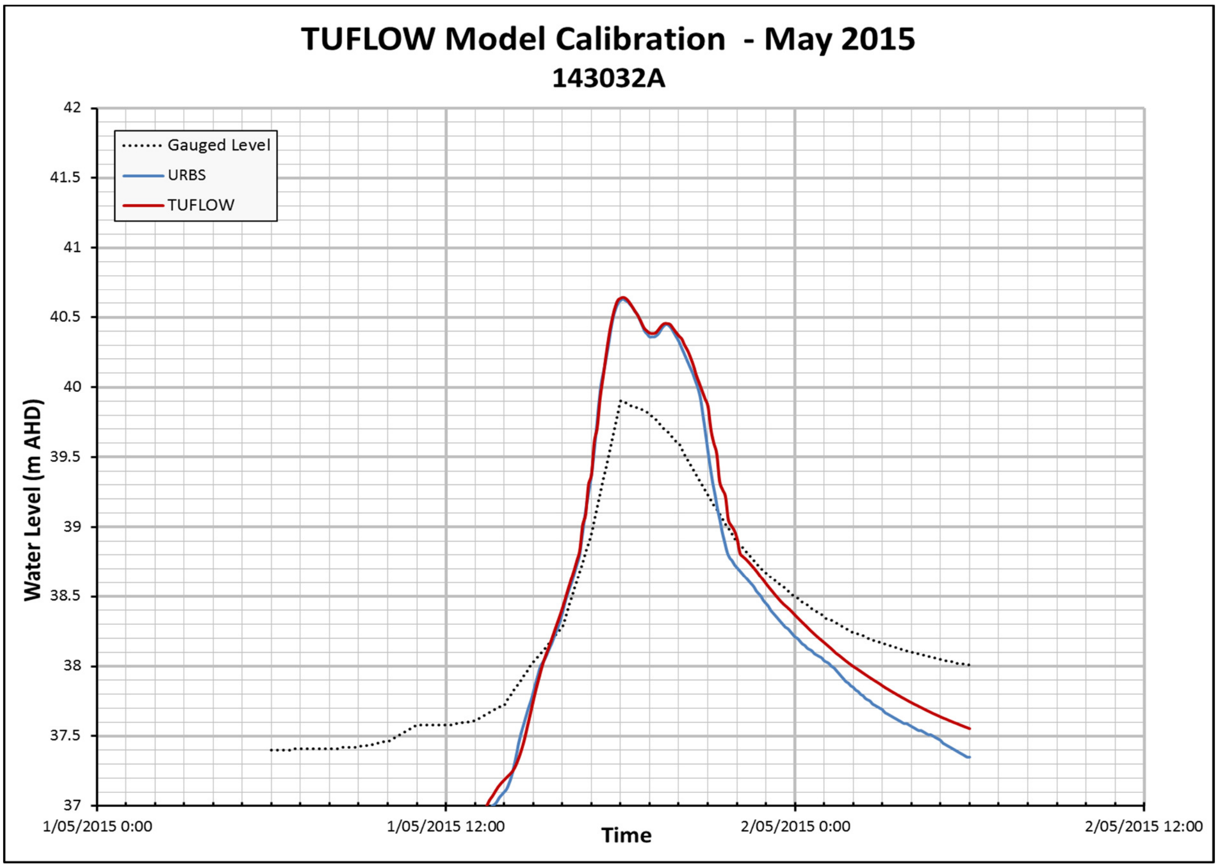

Figure 5.3: TUFLOW Model Calibration at Upper Brookfield (May 2015)

Table 5.3 – Calibration to Peak Flood Level Data (May 2015)

Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Difference (m)
Moggill Creek				
M100	U/S Moggill Creek Mouth	-	-	-
M110	D/S Moggill Rd	6.07	5.99	-0.08
M120	U/S Moggill Rd (Low)	7.11	7.19	0.08
M120H	U/S Moggill Rd (High)	7.04	7.10	0.06
540061	Fortrose Street	9.02	8.97	-0.05
M130	D/S Branton St Footbridge	-	-	-
M140	End of Kailua St	-	-	-
M150	U/S Willunga St	20.61	20.48	-0.13
M159	D/S Rafting Ground Rd	22.48	22.32	-0.16
M160	U/S Rafting Ground Rd	22.55	22.75	0.20
M165	U/S Boscombe Rd	-	-	-
M170	Brookfield Showgrounds	25.51	25.63	0.12
M180	U/S Brookfield Rd	26.22	26.52	0.30
M190	Bundaleer Rd	-	-	-
143032A	Upper Brookfield Road	39.90	40.69	0.79
M200	D/S Upper Brookfield Rd	-	-	-
M210	U/S Upper Brookfield Rd	-	-	-
M220	Haven Rd	-	-	-
M230	U/S Upper Brookfield Rd	-	-	-
M240	U/S Kittani St	66.87	67.36	0.49
Gold Creek				
G100	U/S Savages Rd	-	-	-
G110	179 Gold Creek Rd	-	-	-
G120	U/S 272 Gold Creek Road Driveway (Low)	41.36	40.48	-0.88
G120H	U/S 272 Gold Creek Road Driveway (High)	-	-	-
G130	U/S Gold Creek Rd / Jones Rd intersection	-	-	-
G140	U/S Jones Rd	-	-	-
G150	408 Gold Creek Rd Driveway	56.30	56.47	0.17
G160	U/S 581 Gold Creek Rd	68.25	68.60	0.35
540107	Gold Creek Reservoir	96.10	96.17 (URBS)	0.07

Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Difference (m)
Gap Creek				
GP100	U/S Brookfield Rd @ Deerhurst Rd	25.17	24.55	-0.62
GP110	End of Kookaburra St	30.67	30.43	-0.24
GP120	U/S Gap Creek Rd	37.37	37.15	-0.22

5.5.2 May 2009

The May 2009 flood was simulated in TUFLOW for 38 hours from 6 pm on the 19th May 2009. Figure 5.4 provides a comparison between the TUFLOW (and URBS) results and the gauged flood level at Fortrose Street (540061) in Kenmore. Also presented on this figure (for comparative purposes) is the TUFLOW hydrograph where the actual rated discharge from Gold Creek Reservoir is used in lieu of the modelled outflow from URBS.

Figure 5.5 provides a comparison between the TUFLOW (and URBS) results and the gauged flood level at the DNRM stream gauge (143032A) located in Upper Brookfield. Table 5.4 provides a comparison between the TUFLOW results and the recorded peak flood levels at the stream gauges and MHGs which were working during the event. Also presented in this table (for comparative purposes) are the TUFLOW peak flood levels where the actual rated discharge from Gold Creek Reservoir is used in lieu of the modelled outflow from URBS.

Many of the observed MHG readings were from debris marks as many of the gauges were overtopped due to the large magnitude of the event.

From review of the peak level / MHG results, it was apparent that at 12 out of 21 locations the desired peak flood level tolerance was able to be achieved. When using the actual rated discharge from Gold Creek Reservoir, it was apparent that at 14 out of 20 locations the desired peak flood level tolerance was able to be achieved. For Upper Moggill and Gap Creeks, the simulated peak flood level at all locations was within the desired tolerances.

For Gold Creek and Mid to Lower Moggill Creek, the simulated peak flood levels were typically lower than the recorded. In most of these locations there would appear to be scope to increase Manning's 'n' roughness values to increase flood levels. However, it is considered that the Manning's 'n' values are close to the upper limit of what would be considered reasonable and that insufficient flow would appear to be the main contributing factor for the consistency in low flood levels.

At Fortrose Street (540061), the simulated peak flood level was not able to be calibrated to within the ± 0.15 m tolerance. The shape and timing of the hydrograph was good, however the simulated peak flood level was considerably lower than the observed, which was discussed previously in Section 4.8.2.

At Upper Brookfield (143032A), the simulated peak flood level was within the ± 0.15 m tolerance. The simulated rising limb of the main peak achieved a good fit with the observed; however the subsequent two peaks were around 0.3 m too low.

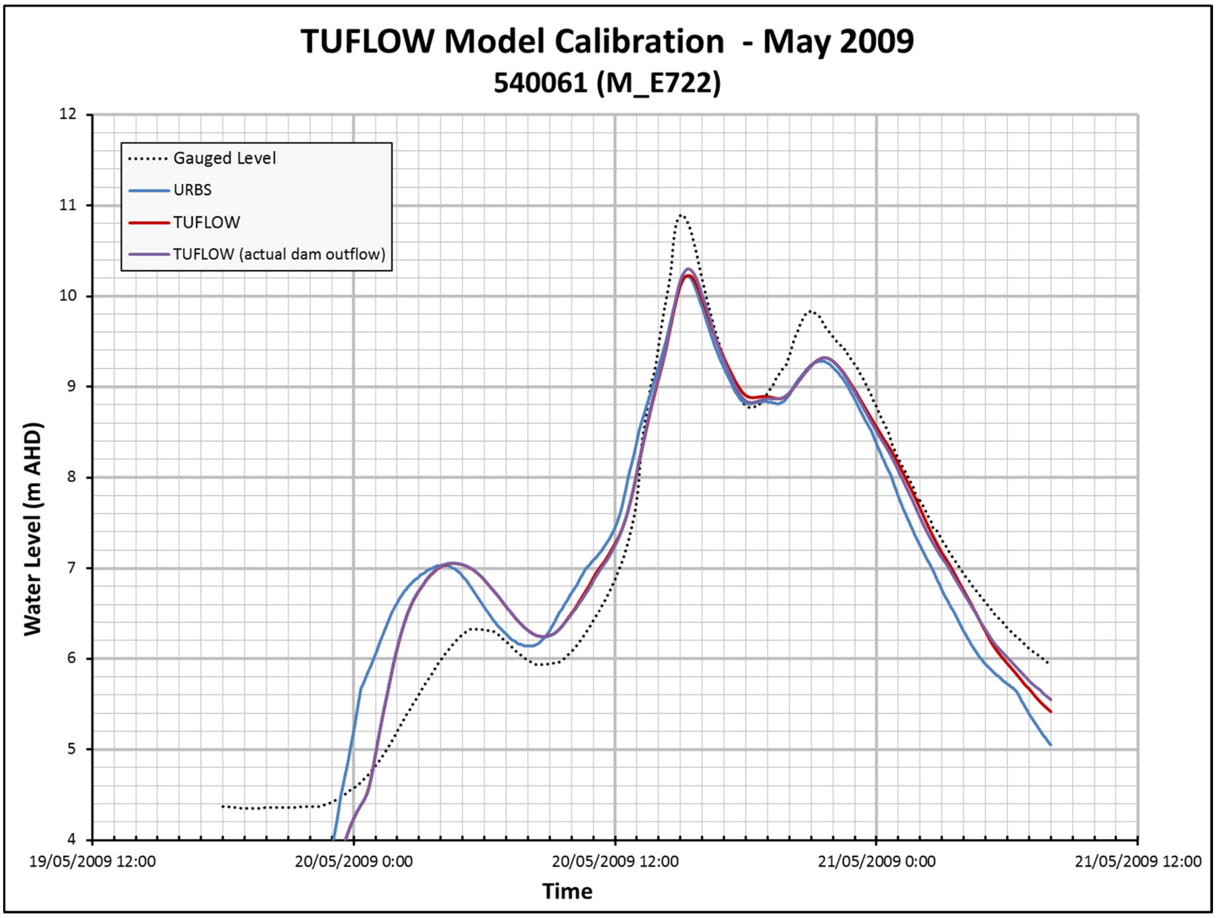

Figure 5.4: TUFLOW Model Calibration at Fortrose Street (May 2009)

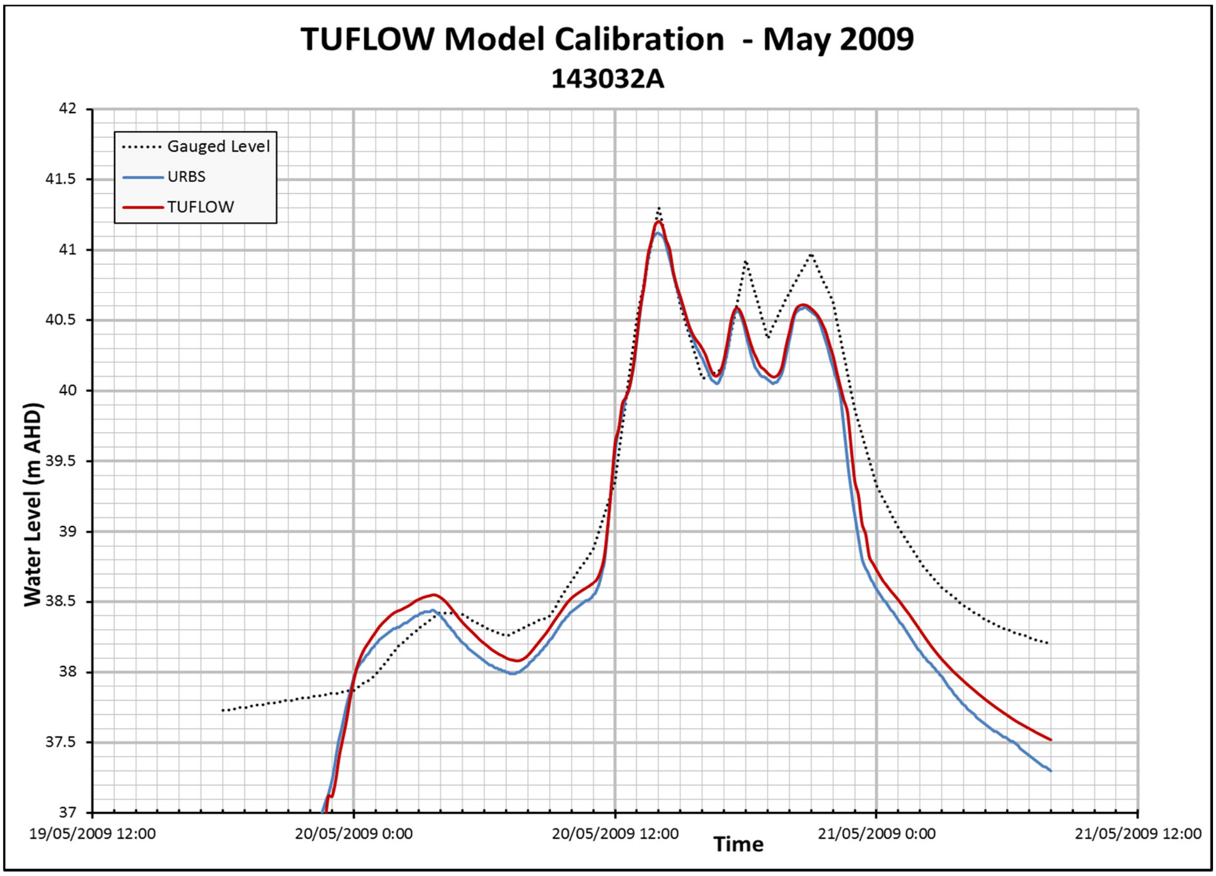

Figure 5.5: TUFLOW Model Calibration at Upper Brookfield (May 2009)

Table 5.4 – Calibration to Peak Flood Level Data (May 2009)

Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Diff. (m)	Simulated Peak WL (m AHD) [Actual Dam]	Diff. (m)
Moggill Creek						
M100	U/S Moggill Creek Mouth	3.54	3.48	-0.06	3.53	-0.01
M110	D/S Moggill Rd	-	-	-	-	-
M120	U/S Moggill Rd (Low)	8.90	9.21	0.31	9.28	0.38
M120H	U/S Moggill Rd (High)	-	-	-	-	-
540061	Fortrose Street	10.91	10.23	-0.68	10.31	-0.60
M130	D/S Branton St Footbridge	14.48	13.97	-0.51	14.02	-0.46
M140	End of Kailua St	18.26	18.05	-0.21	18.10	-0.16
M150	U/S Willunga St	21.95 (Debris)	21.16	-0.79	21.20	-0.75
M159	D/S Rafting Ground Rd	-	-	-	-	-
M160	U/S Rafting Ground Rd	-	-	-	-	-
M165	U/S Boscombe Rd	24.34 (Debris)	24.60	0.26	24.65	0.31
M170	Brookfield Showgrounds	-	-	-	-	-
M180	U/S Brookfield Rd	27.51	27.50	-0.01	27.54	0.03
M190	Bundaleer Rd	32.90	32.61	-0.29	32.62	-0.28
143032A	Upper Brookfield Road	41.30	41.25	-0.05	41.25	-0.05
M200	D/S Upper Brookfield Rd	41.90	41.65	-0.25	41.65	-0.25
M210	U/S Upper Brookfield Rd	42.54	42.50	-0.04	42.50	-0.04
M220	Haven Rd	45.27 (Debris)	45.41	0.14	45.41	0.14
M230	U/S Upper Brookfield Rd	-	-	-	-	-
M240	U/S Kittani St	-	-	-	-	-
Gold Creek						
G100	U/S Savages Rd	36.61	35.82	-0.79	35.99	-0.62
G110	179 Gold Creek Rd	38.86	38.50	-0.36	38.70	-0.16

Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Diff. (m)	Simulated Peak WL (m AHD) [Actual Dam]	Diff. (m)
G120	U/S 272 Gold Creek Road Driveway (Low)	Over Topped (No Survey)	-	-	-	-
G120H	U/S 272 Gold Creek Road Driveway (High)	-	-	-	-	-
G130	U/S Gold Creek Rd / Jones Rd intersection	46.61	46.08	-0.53	46.33	-0.28
G140	U/S Jones Rd	-	-	-	-	-
G150	408 Gold Creek Rd Driveway	57.64	57.21	-0.43	57.32	-0.32
G160	U/S 581 Gold Creek Rd	69.40	69.30	-0.10	69.37	-0.03
540107	Gold Creek Reservoir	96.77	96.71 (URBS)	-0.06	-	-
Gap Creek						
GP100	U/S Brookfield Rd @ Deerhurst Rd	25.74	25.52	-0.22	25.53	-0.21
GP110	End of Kookaburra St	-	-	-	-	-
GP120	U/S Gap Creek Rd	37.53	37.58	0.05	37.58	0.05

5.5.3 November 2008

The November 2008 flood was simulated in TUFLOW for 12 hours from 10 pm on the 19th November 2008.

Figure 5.6 provides a comparison between the TUFLOW (and URBS) results and the gauged flood level at Fortrose Street (540061) in Kenmore. Figure 5.7 provides a comparison between the TUFLOW (and URBS) results and the gauged flood level at the DNRM stream gauge (143032A) located in Upper Brookfield. Table 5.5 provides a comparison of the TUFLOW results and the recorded peak flood levels at the stream gauges and MHGs which were working during the event.

From review of the peak level / MHG results, it was apparent that at 17 out of 20 locations, the desired peak flood level tolerance was able to be achieved. The MHG recording at M150 was not considered, as there were some significant inconsistencies that warranted omission. For example, the MHG level at M150 for November 2008 is considerably lower (~0.8 m) than the next smallest event (May 2015), which is not consistent with the trend at other MHG gauges when comparing these two events.

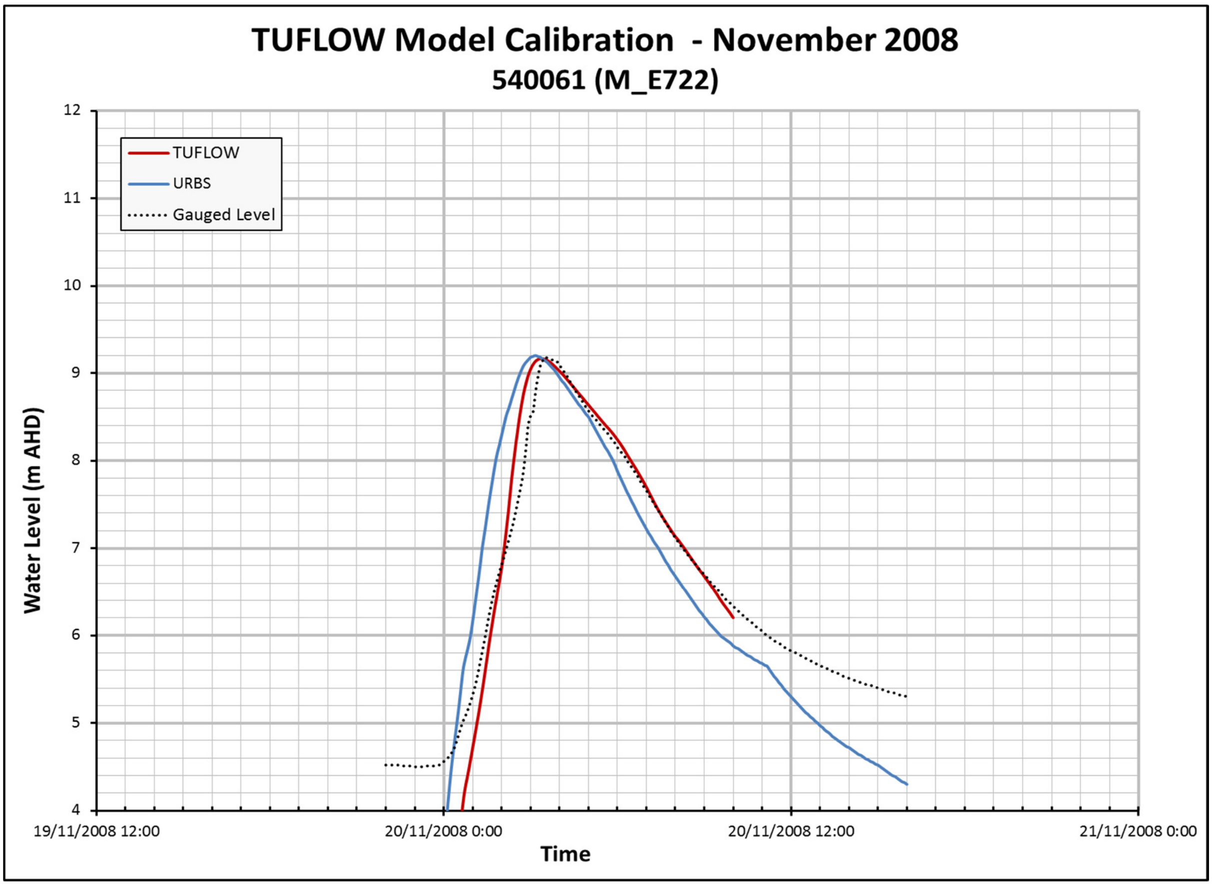

Figure 5.6: TUFLOW Model Calibration at Fortrose Street (November 2008)

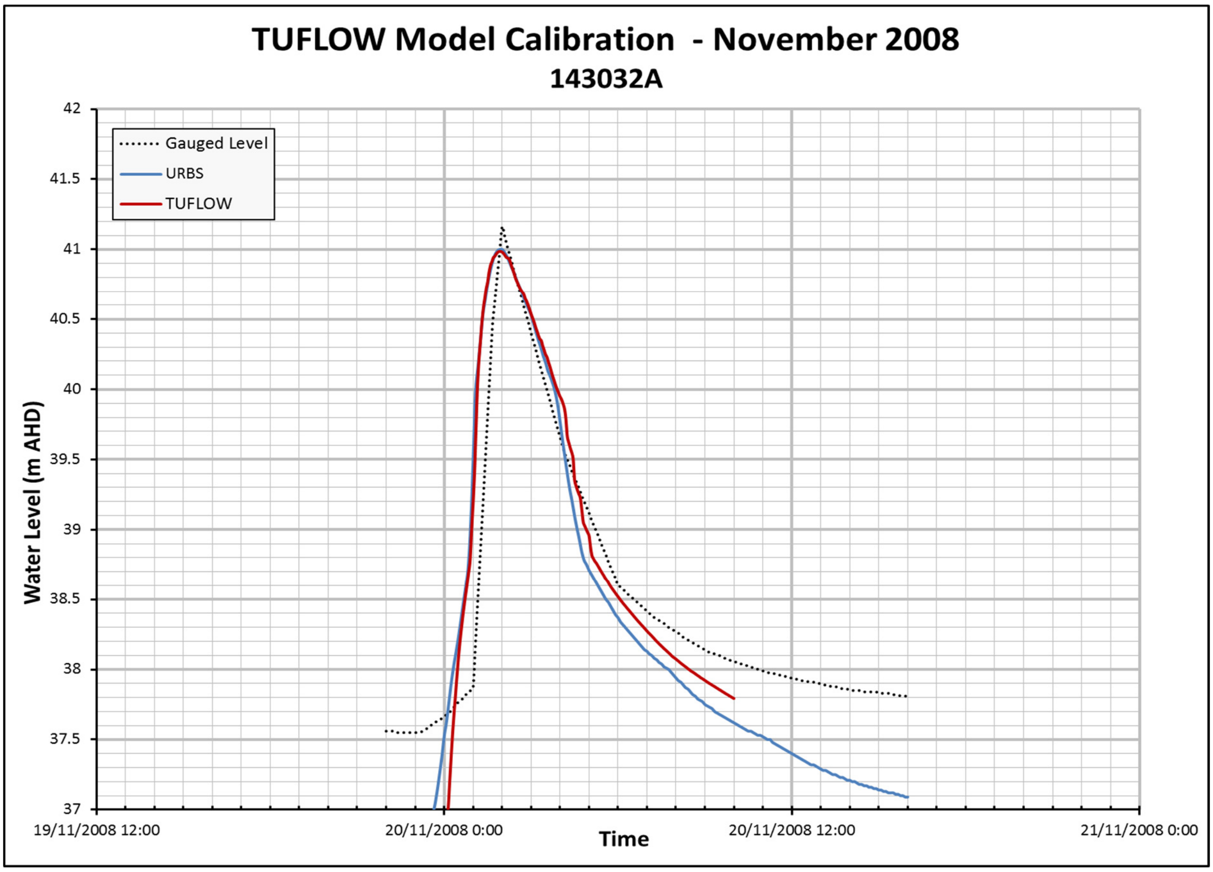

Figure 5.7: TUFLOW Model Calibration at Upper Brookfield (November 2008)

Table 5.5 – Calibration to Peak Flood Level Data (November 2008)

Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Difference (m)
Moggill Creek				
M100	U/S Moggill Creek Mouth	2.74	2.84	0.10
M110	D/S Moggill Rd	6.17	6.10	-0.07
M120	U/S Moggill Rd (Low)	7.38	7.56	0.18
M120H	U/S Moggill Rd (High)	-	-	-
540061	Fortrose Street	9.18	9.16	-0.02
M130	D/S Branton St Footbridge	-	-	-
M140	End of Kailua St	17.18 (debris)	17.42	0.24
M150	U/S Willunga St	not used due to likely reading error		
M159	D/S Rafting Ground Rd	-	-	-
M160	U/S Rafting Ground Rd	-	-	-
M165	U/S Boscombe Rd	-	-	-
M170	Brookfield Showgrounds	26.43	26.15	-0.28
M180	U/S Brookfield Rd	27.07	26.99	-0.08
M190	Bundaleer Rd	32.45	32.32	-0.13
143032A	Upper Brookfield Road	41.16	41.04	-0.12
M200	D/S Upper Brookfield Rd	-	-	-
M210	U/S Upper Brookfield Rd	42.77	42.25	-0.52
M220	Haven Rd	45.17 (debris)	45.25	0.08
M230	U/S Upper Brookfield Rd	54.36	54.07	-0.29
M240	U/S Kittani St	-	-	-
Gold Creek				
G100	U/S Savages Rd	-	-	-
G110	179 Gold Creek Rd	-	-	-
G120	U/S 272 Gold Creek Road Driveway (Low)	41.67	41.54	-0.13
G120H	U/S 272 Gold Creek Road Driveway (High)	-	-	-
G130	U/S Gold Creek Rd / Jones Rd intersection	-	-	-
G140	U/S Jones Rd	52.06	51.96	-0.10
G150	408 Gold Creek Rd Driveway	56.84	56.82	-0.02
G160	U/S 581 Gold Creek Rd	68.64	68.93	0.29
540107	Gold Creek Reservoir	96.38	96.45 (URBS)	0.07

Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Difference (m)
Gap Creek				
GP100	U/S Brookfield Rd @ Deerhurst Rd	25.67	25.09	-0.58
GP110	End of Kookaburra St	31.43	31.26	-0.17
GP120	U/S Gap Creek Rd	38.11	37.44	-0.67

For Moggill and Gold Creeks, the simulated peak flood level at 16 out of 17 gauges was within the desired tolerance. The only location where the desired tolerance could not be achieved was at M210, which is upstream of the complex skewed bridge (S12) on Upper Brookfield Road. At this location there appears to be some inconsistencies with the MHG readings, as the November 2008 reading is 0.23 m higher than the May 2009 reading, yet at MHG 220 (470 m upstream), the November 2008 reading is 0.1 m lower than the May 2009 reading. As it is unlikely that the relative flows would have changed significantly over this short length, it is likely that localised bridge impacts (e.g. blockage) occurred during the November 2008 event.

For Gap Creek, the simulated peak flood levels were typically lower than the observed, with only 1 out of 3 gauges falling within the desired tolerance. From review of the rainfall distribution there was considerable differences between the rainfall at the headwaters of Gap Creek (Mt Coot-tha - 540117) and that further towards the mid to lower sections (Chadstone Cl - 540099). It is likely that the adopted Thiessen polygon distribution did not mirror reality, resulting in the simulation of less intense rainfall and lower flows than actually occurred.

At Fortrose Street (540061), the simulated peak flood level was within the ± 0.15 m tolerance. Both the rising limb and receding limb of the hydrograph achieved a very good fit with the recorded hydrograph.

At Upper Brookfield (143032A), the simulated peak flood level was within the ± 0.15 m tolerance. Both the rising limb and receding limb of the hydrograph achieved a very reasonable fit with the recorded hydrograph.

5.6 Hydraulic Model Verification Results

5.6.1 January 2013

The January 2013 flood was simulated in TUFLOW for 48 hours from 6 pm on the 26th January 2013. Figure 5.8 provides a comparison between the TUFLOW (and URBS) results and the gauged flood level at Fortrose Street (540061) in Kenmore.

Figure 5.9 provides a comparison between the TUFLOW (and URBS) results and the gauged flood level at the DNRM stream gauge (143032A) located in Upper Brookfield. Table 5.6 provides a comparison between the TUFLOW results and the recorded peak flood levels at the stream gauges and MHGs which were working during the January 2013 event.

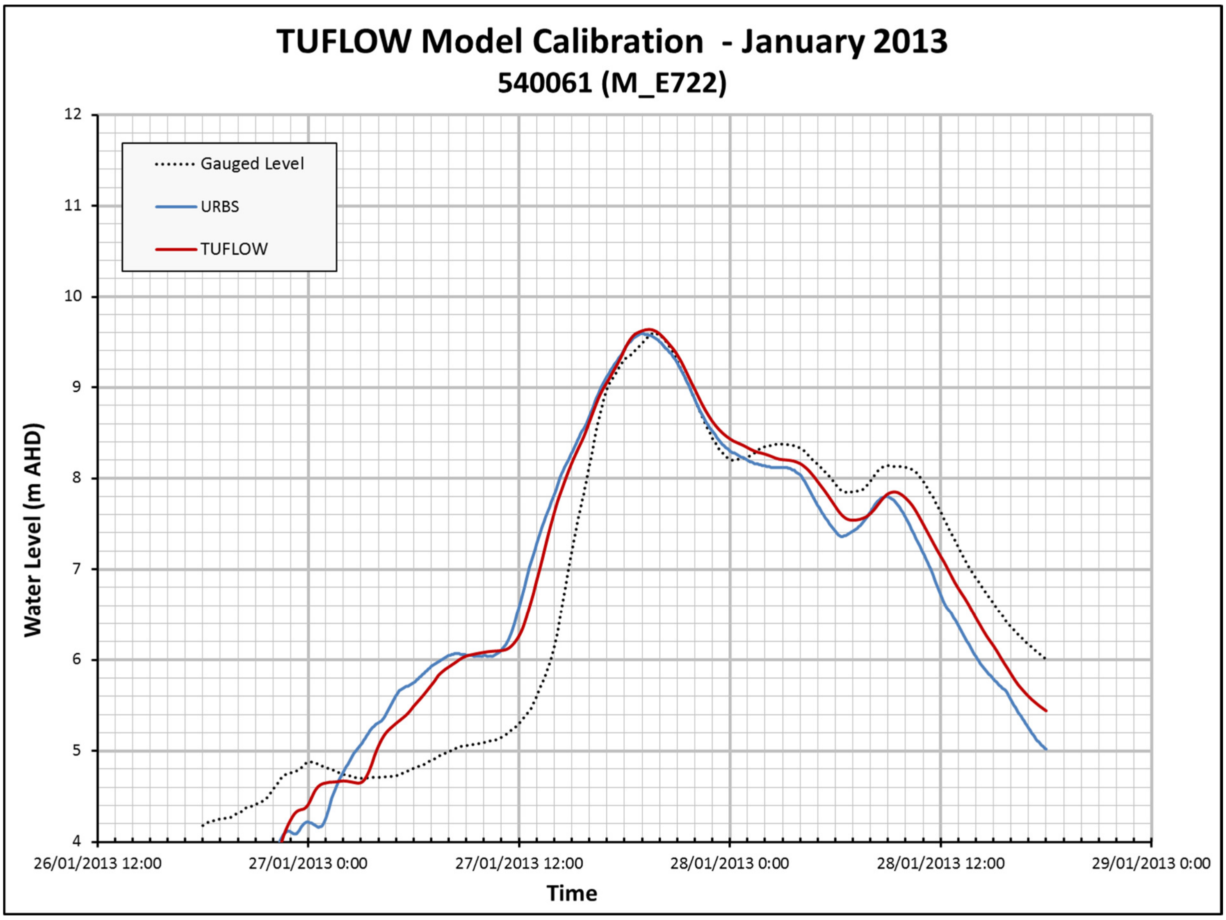

Figure 5.8: TUFLOW Model Calibration at Fortrose Street (January 2013)

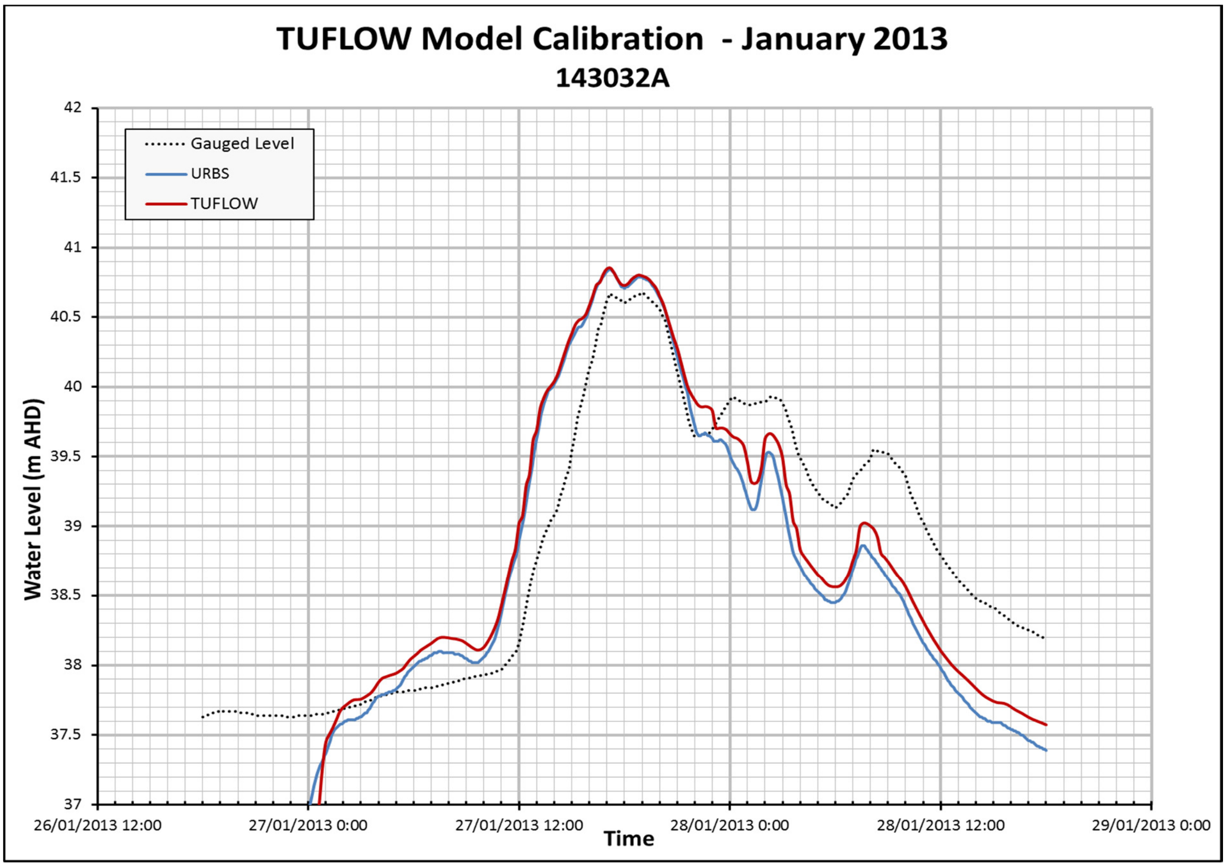

Figure 5.9: TUFLOW Model Calibration at Upper Brookfield (January 2013)

Table 5.6 – Verification to Peak Flood Level Data (January 2013)

Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Difference (m)
Moggill Creek				
M100	U/S Moggill Creek Mouth	4.53	4.61	0.08
M110	D/S Moggill Rd	6.76	6.68	-0.08
M120	U/S Moggill Rd (Low)	-	-	-
M120H	U/S Moggill Rd (High)	7.88	8.37	0.49
540061	Fortrose Street	9.60	9.64	0.04
M130	D/S Branton St Footbridge	-	-	-
M140	End of Kailua St	17.53	17.67	0.14
M150	U/S Willunga St	21.25	20.90	-0.35
M159	D/S Rafting Ground Rd	23.02	22.79	-0.23
M160	U/S Rafting Ground Rd	22.90	23.18	0.28
M165	U/S Boscombe Rd	-	-	-
M170	Brookfield Showgrounds	26.04	26.15	0.11
M180	U/S Brookfield Rd	26.94	27.25	0.31
M190	Bundaleer Rd	32.15	32.26	0.11
143032A	Upper Brookfield Road	40.68	40.91	0.23
M200	D/S Upper Brookfield Rd	41.56	41.34	-0.22
M210	U/S Upper Brookfield Rd	42.26	42.14	-0.12
M220	Haven Rd	45.32 (debris)	45.13	-0.19
M230	U/S Upper Brookfield Rd	53.56	53.76	0.20
M240	U/S Kittani St	67.47	67.54	0.07
Gold Creek				
G100	U/S Savages Rd	35.14	35.52	0.38
G110	179 Gold Creek Rd	-	-	-
G120	U/S 272 Gold Creek Road Driveway (Low)	42.76	42.25	-0.51
G120H	U/S 272 Gold Creek Road Driveway (High)	42.59	42.25	-0.34
G130	U/S Gold Creek Rd / Jones Rd intersection	-	-	-
G140	U/S Jones Rd	52.19	52.27	0.08
G150	408 Gold Creek Rd Driveway	57.18	57.06	-0.12
G160	U/S 581 Gold Creek Rd	69.19	69.15	-0.04
540107	Gold Creek Reservoir	96.56	96.61 (URBS)	0.05

Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Difference (m)
Gap Creek				
GP100	U/S Brookfield Rd @ Deerhurst Rd	25.00	24.97	-0.03
GP110	End of Kookaburra St	30.51	30.79	0.28
GP120	U/S Gap Creek Rd	37.15	37.34	0.19

From review of the peak level / MHG results, it was apparent that at 21 out of 27 locations the desired peak flood level tolerance was able to be achieved. For Moggill and Gap Creeks, there was generally a good correlation between simulated and observed peak levels throughout the entire length of each creek.

At MHG G120, upstream of the private arch bridge location (S36), it is conceivable that blockage occurred, resulting in a considerably higher upstream flood level than the simulated results. There also appears to be some inconsistencies with the MHG readings, as at G140 the difference in the MHG levels between January 2013 and November 2008 was 0.13 m, yet at G120 (1.8 km downstream) the difference is 1.09 m. As it is unlikely that the relative flows would have changed significantly over this length, it is likely that localised bridge impacts (e.g. blockage) occurred during the January 2013 event. It is important to note that the simulated results have not included blockage at structures.

At Fortrose Street (540061), the simulated peak flood level was within the ± 0.15 m tolerance. The simulated rising limb was not able to achieve a good fit with the recorded hydrograph; however the simulated receding limb achieved a better fit.

At Upper Brookfield (143032A), the simulated peak flood level was just outside the desired ± 0.15 m tolerance and the shape / timing of the hydrograph was reasonable. It is conceivable that the differences in shape and timing at both gauges could be attributed to the differences in the adopted and actual rainfall distribution.

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 more accurately representing hydraulic structures such as bridges. Many of the hydraulic structures within the catchment(s) are culverts, of which the

TUFLOW and HEC-RAS algorithms would be reasonably 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:

- Moggill Road (S2)
- Branton Street Footbridge (S4)
- Brookfield Road – Moggill Creek (S9)
- 185 Upper Brookfield Road (S11)
- Upper Brookfield Road crossing 1 (S12)
- Upper Brookfield Road crossing 2 (S15)
- Brookfield Road – Gap Creek (S28)
- Savages Road (S34)
- 272 Gold Creek Road (S36)
- Gold Creek Road crossing 1 (S37)

Many of the bridge structures were quite complex with the bridge decks not perpendicular to the flow direction. Others, such as S12 (Upper Brookfield Road #1) and S28 (Brookfield Road) had skewed bridge decks and were also on sharp bends, adding to the complexity.

Table 5.7 provides a comparison of the head-loss across the structure between TUFLOW and the HEC-RAS model. 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.

Table 5.7 – HEC-RAS Bridge Modelling Checks

Flow (m^3/s)	HEC-RAS Head-loss (m)	TUFLOW Head-loss (m)	Difference (m)
Structure S1 and S2 – Moggill Road Bridges			
57	0.51	0.36	-0.15
184	0.40	0.40	0.00
313	0.97	0.87	-0.10
426	1.34	1.17	-0.17
538	1.21	1.47	0.26
650	1.22	1.57	0.35
763	1.15	1.61	0.46
Structure S4 – Branton Street Footbridge			
40	0.02	0.01	-0.01
108	0.04	0.14	0.10
217	0.05	0.16	0.11
313	0.04	0.15	0.11
406	0.04	0.14	0.10
497	0.04	0.13	0.09
610	0.04	0.13	0.09

Flow (m^3/s)	HEC-RAS Head-loss (m)	TUFLOW Head-loss (m)	Difference (m)
699	0.05	0.14	0.09
816	0.05	0.15	0.10
Structure S9 – Brookfield Road (Moggill Creek)			
108	0.25	0.04	-0.21
200	0.59	0.46	-0.13
296	0.84	0.81	-0.03
406	0.89	0.88	-0.01
498	0.91	0.90	-0.01
608	0.92	0.90	-0.02
717	0.93	0.89	-0.04
793	0.89	0.87	-0.02
Structure S11 – 185 Upper Brookfield Road			
56	0.11	0.19	0.08
99	0.61	0.71	0.10
198	0.56	0.63	0.07
304	0.53	0.64	0.11
405	0.40	0.59	0.19
503	0.48	0.53	0.05
602	0.65	0.50	-0.15
Structure S12 – Upper Brookfield road crossing 1			
59	0.21	0.24	0.03
144	0.64	0.44	-0.20
229	0.89	0.70	-0.19
308	1.44	0.94	-0.50
388	1.52	1.68	0.16
473	1.73	1.88	0.15
550	1.62	2.00	0.38
632	1.74	2.05	0.31
714	1.80	2.05	0.25
Structure S15 – Upper Brookfield road crossing 2			
61	0.50	0.62	0.12
123	0.71	0.71	0.00
205	1.10	1.15	0.05
410	1.09	1.36	0.27
499	1.07	1.26	0.19
643	0.87	1.07	0.20
Structure S28 – Brookfield Road (Gap Creek)			
28	0.22	0.05	-0.17

Flow (m^3/s)	HEC-RAS Head-loss (m)	TUFLOW Head-loss (m)	Difference (m)
60	0.21	0.18	-0.03
95	0.54	0.62	0.08
128	0.82	0.72	-0.10
162	0.83	0.66	-0.17
Structure S34 – Savages Road			
54	0.54	0.2	-0.34
105	0.70	0.71	0.01
144	0.80	0.83	0.03
192	0.82	0.84	0.02
290	0.77	0.70	-0.07
393	0.45	0.22	-0.23
Structure S36 – 272 Gold Creek Road			
48	0.12	0.11	-0.01
97	0.27	0.40	0.13
161	0.59	0.54	-0.05
242	0.45	0.46	0.01
304	0.35	0.36	0.01
397	0.20	0.21	0.01
Structure S37 – Gold Creek Road crossing 1			
48	0.06	0.11	0.05
82	0.05	0.13	0.08
123	0.05	0.17	0.12
164	0.88	0.77	-0.11
247	1.09	1.37	0.28
401	1.26	1.48	0.22

5.8 Hydrologic-Hydraulic Model Consistency Check (Historical Events)

5.8.1 General

Comparison checks were undertaken between the URBS and TUFLOW models to understand how closely the hydrologic and hydraulic models were matching. Figures 5.10 to 5.13 provide comparative plots of the URBS and TUFLOW flow results for the historical events at the following three locations:

(i) MHG M165 – Moggill Creek (U/S of Boscombe Road)

(ii) Gold Creek - Confluence with Moggill Creek

(iii) Gap Creek - Confluence with Moggill Creek

Table 5.8 provides a comparison of the peak flows at these three locations.

Table 5.8 – Peak Flow Comparison, URBS and TUFLOW

Event	Peak Flow (m^3/s)					
	MHG M165		Gold Creek - Confluence with Moggill Creek		Gap Creek - Confluence with Moggill Creek	
	URBS	TUFLOW	URBS	TUFLOW	URBS	TUFLOW
Nov 2008	244.7	246.0	70.9	72.0	55.7	55.4
May 2009	348	357.6	120.3	122.6	74.0	74.0
Jan 2013	284.2	291.4	105.5	106.3	53.1	52.4
May 2015	198	195.4	43.5	42.2	39.8	39.7

The results of the comparison indicate that the URBS and TUFLOW models show a good correlation with peak flow and hydrograph timing / shape throughout the model. This is consistent with the results of the calibration and verification at the stream gauges.

Based on the good correlation between URBS and TUFLOW, it is considered that the URBS model would be suitable for use as a 'standalone' model on the basis that there are not considerable backwater effects from the Brisbane River. If there are backwater effects, then the hydraulic model would be more suitable for generating accurate flows / flood levels.

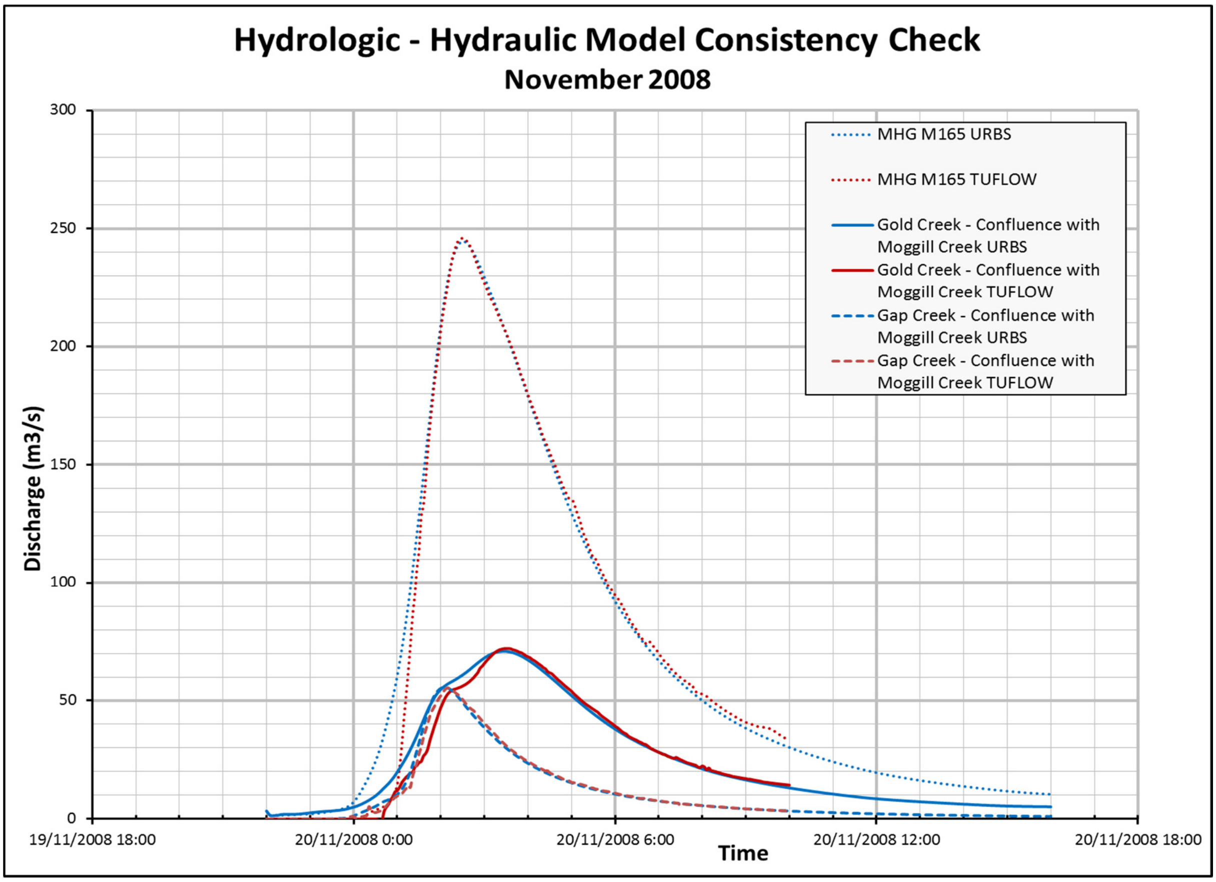

Figure 5.10: Model Consistency Check (November 2008)

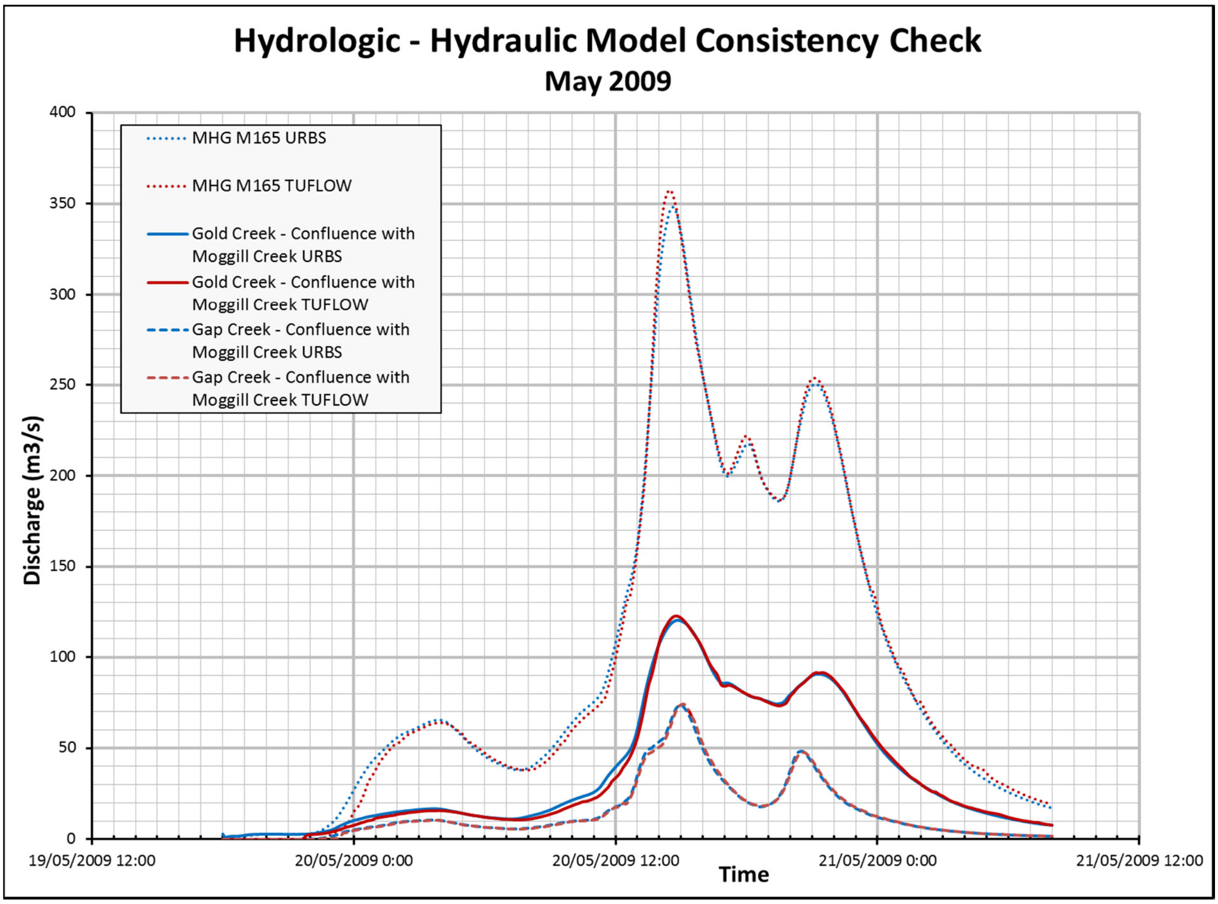

Figure 5.11: Model Consistency Check (May 2009)

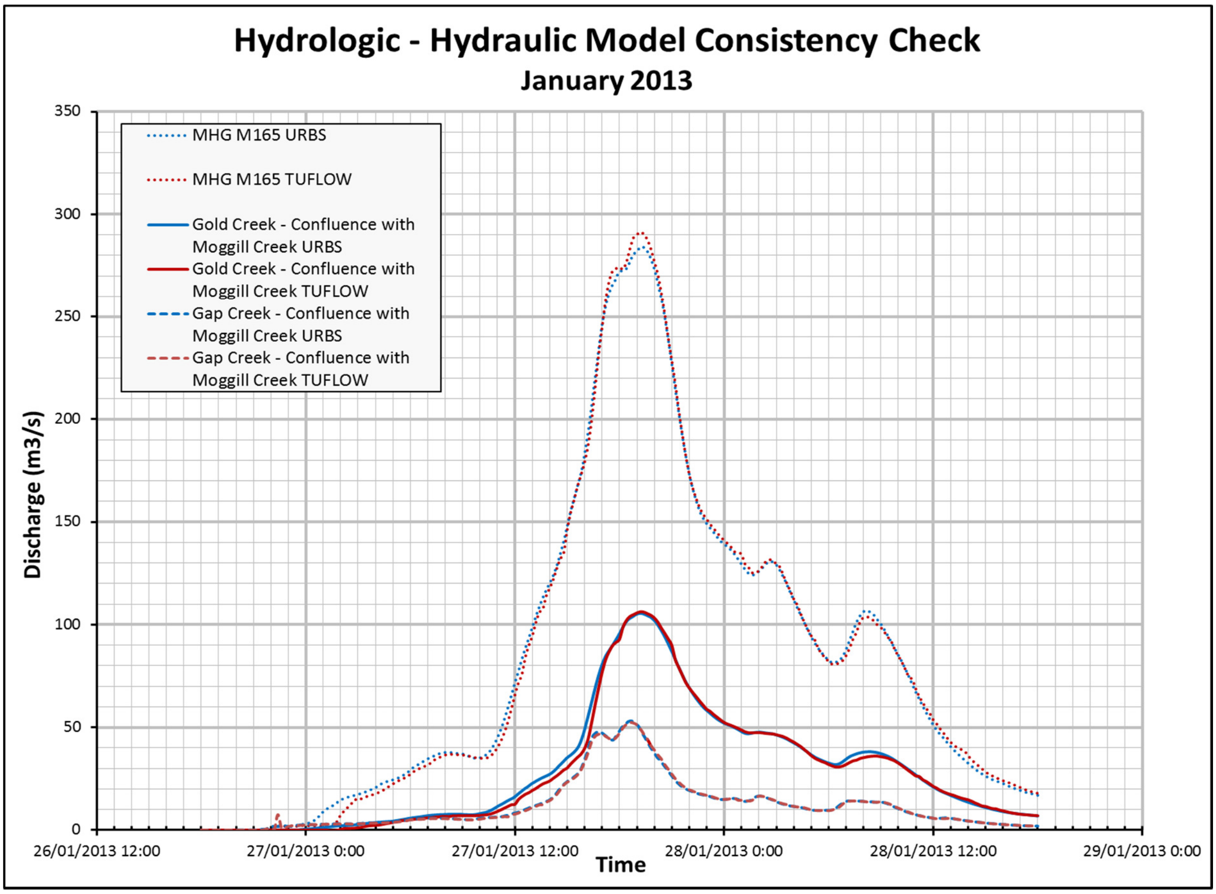

Figure 5.12: Model Consistency Check (January 2013)

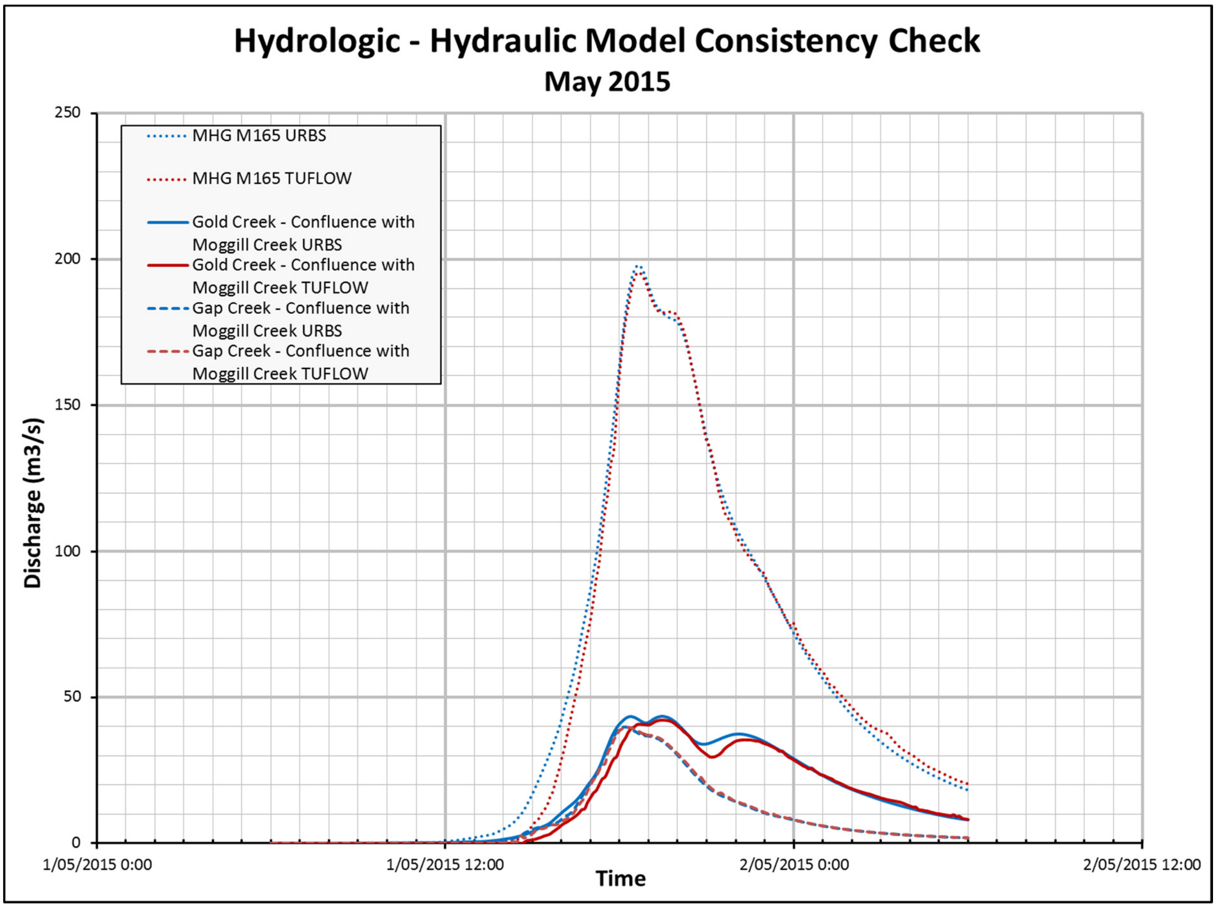

Figure 5.13: Model Consistency Check (May 2015)

5.9 Discussion on Calibration and Verification

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

- May 2015 – good fit at two out of three continuous recording stream gauges. At 13 out of 18 MHG locations, the desired peak flood level tolerance was able to be achieved. The five MHG locations where the simulated peak flood level was not able to meet the desired tolerances were upstream of hydraulic structures; where there is inherently considerable uncertainty due to blockages, guard rail / handrail effects, bridge / culvert losses, etc.
- May 2009 – good fit at two out of three continuous recording stream gauges. At 12 out of 21 MHG locations, the desired peak flood level tolerance was able to be achieved. Peak flood levels are typically low throughout the catchment, of which contributing factors could include:
    - The adopted Thiessen polygon distribution of rainfall across the URBS sub-catchments did not mirror reality, resulting in the simulation of less intense rainfall and lower flows than actually occurred.
    - Rainfall gauge recordings not capturing the entire volume of rain which fell.
    - Continuing rainfall losses too high – a better fit would be achieved if the continuing loss was set lower than 2.5 mm/hr for this event. However, this would adversely affect the results of the other calibration events.
- November 2008 – good fit at all three continuous recording stream gauges. At 17 out of 20 MHG locations, the desired peak flood level tolerance was able to be achieved.
- January 2013 – reasonable fit at all three continuous recording stream gauges. At 21 out of 27 MHG locations, the desired peak flood level tolerance was able to be achieved.

There are 18 MHGs upstream of hydraulic structures and of those 10 are upstream of bridge structures. In comparison, there are only four MHGs downstream of hydraulic structures. Given that the upstream catchment areas are heavily forested, the likelihood of significant woody debris and partial (or full) blockage of structures is considered high. This high risk of blockage can add further uncertainty to the calibration when considering those MHGs located upstream of hydraulic structures. To aid future calibration, there should be an even balance of MHGs upstream and downstream of hydraulic structures.

From the calibration results, it is apparent that the largest event (May 2009) produced the least successful calibration when compared with the other three historical events. As mentioned previously, this is most likely due to inconsistencies in the assumed rainfall distribution used in the hydrologic modelling. However, it would be prudent to further verify the hydrologic and hydraulic models, once another large flooding event occurs.

Given that the results of the calibration and verification are reasonable and that the events ranged in magnitude from small (~2-yr to 5-yr ARI) to large (~50-yr to 100-yr ARI), there is confidence that the hydrologic and hydraulic models would be suitable for producing accurate flood levels for the full range of design event modelling.

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6.0 Design Event Analysis

6.1 Design Event Scenarios

Table 6.1 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 (50 % AEP) to 100-yr ARI (1 % AEP).

Table 6.1 – Design Event Scenarios

ARI (year)	AEP (%)	Scenario 1	Scenario 2	Scenario 3
2	50	✓	×	✓
5	20	✓	×	✓
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 "Modelled Flood Corridor."

Scenario 3: Filling to the Modelled Flood Corridor + Minimum Riparian Corridor (MRC)
The "Modelled Flood Corridor" is the greater extent of the Waterway Corridor (WC) and Flood Planning Areas (FPAs) 1, 2 and 3. Figure 6.1 indicates the "Modelled Flood Corridor" for all creeks.

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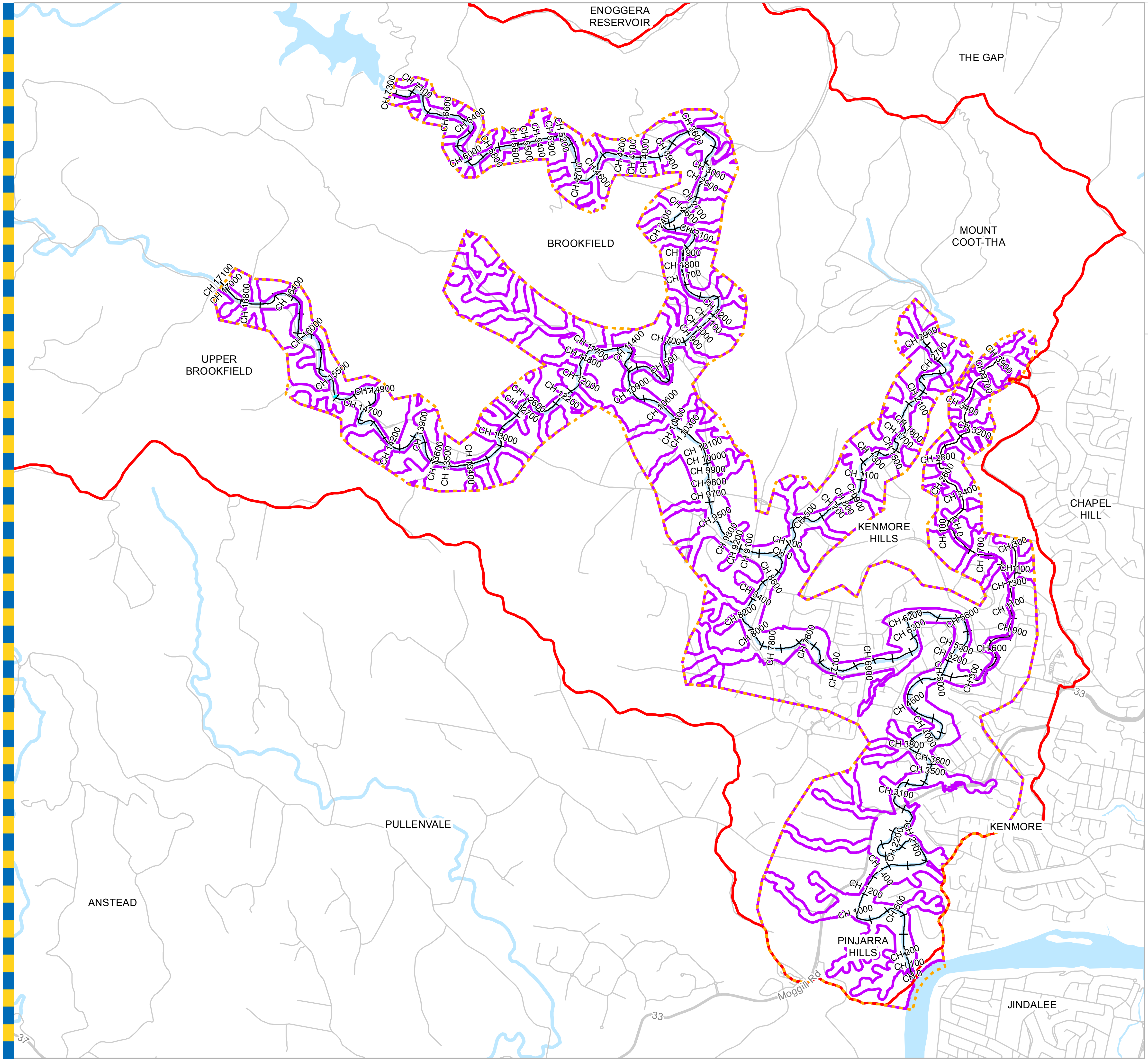

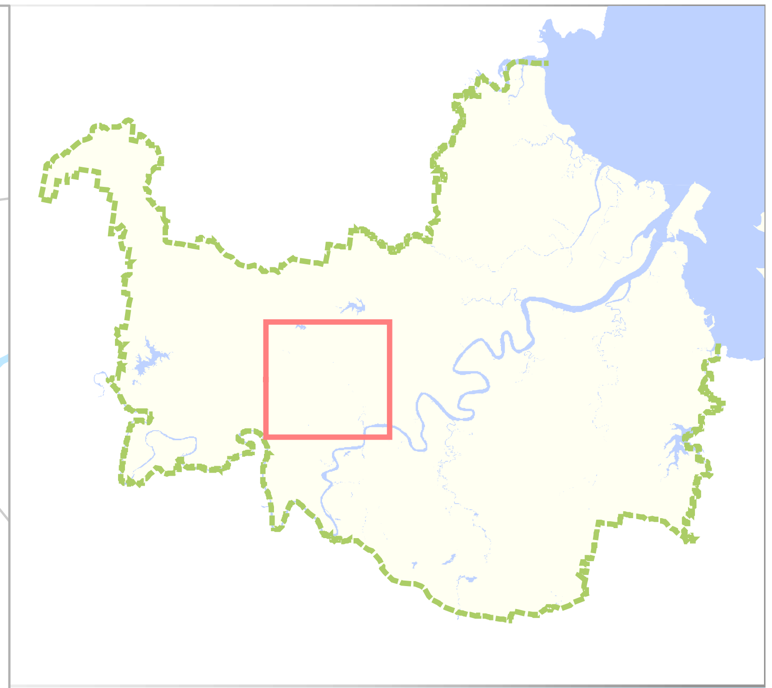

Legend

- Model Boundary
- Catchment Area
- Flood Corridor
- Freeways/Highways
- Major Roads
- Streets
- Waterways
- AMTD Line

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Metres

Prepared : 099858
Checked : NC
Revision : 0
Publication Date : 23 Jun 2016
Project Number : 160277

Location: G:\BI\CD\Proj16\160277_Moggill_Creek_Flood_Study\ArcGIS\GIM_160277_008.mxd

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Figure 6.1: Flood Corridor

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Scenario 3 assumes filling to the "Modelled Flood Corridor" boundary to represent potential development. In the design events, 2-yr ARI (50 % AEP) to 100-yr ARI (1 % AEP), the filling acts as a barrier and the "Modelled Flood 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 BCC City Plan.

6.2 Design Event Hydrology

6.2.1 Selection of Design Flood Estimation Methodology

Design flood estimation is generally best determined by undertaking some form of flood frequency analysis (FFA) of annual maximum and / or peak over threshold (POT) series from observed long-term stream flow records. If FFA is not suitable, then the other alternative to estimate the design flood is to use the rainfall based synthetic design storm concept from AR&R (1987).

<u>Suitability of Flood Frequency Analysis</u>

FFA is best performed on homogeneous catchments where there has been little change over the period of record. For example, a rural catchment with little change is potentially very suitable for FFA, whereas a catchment which has experienced considerable urbanisation over the period of record is not ideal for FFA. Similarly, FFA is not easily applied to catchments containing reservoirs / dams, due to inconsistencies in storage effects when considering the variability of initial dam water levels.

FFA has a number of advantages over the rainfall based synthetic design storm methodology; however it should only be used when: [9]

- A long record exists
- The flood record is homogenous or can be adjusted to a near homogenous state
- A reliable rating curve exists, and
- The probability of the event to be derived does not require extrapolation too far beyond the observed record length.

Table 6.2 [10] indicates some guidance for length of record versus expected error rate for FFA.

[9] WMAWater 2011, *Queensland Flood Commission of Inquiry, Brisbane River 2011 Flood Event – Flood Frequency Analysis, Final Report*

[10] University Corporation for Atmospheric Research (USA) 2010, *Flood Frequency Analysis*

Table 6.2 – Guidance for Length of Record versus Expected Error Rate using FFA

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

The most suitable location to undertake FFA would be at the Upper Brookfield (143032A) stream gauge. On closer examination of this gauge, the following was apparent:

- Continuous records are available from 1976 until the present, which equates to approximately 40 years of data.
- The upstream catchment is rural and is virtually unchanged over the period of record.
- The location of the gauge is such that it is upstream of the confluence with Gold Creek, meaning that it does not receive flow from Gold Creek Reservoir.
- The catchment area upstream of the gauge is 22.6 km^2, which represents approximately one third of the catchment area.
- As noted in Section 4.5, there are some uncertainties with the rating curve, with inconsistencies found with the published zero datum and being located upstream of a bridge structure.
- The period of record omits the 1974 event, which is the largest event in modern times.

<u>Adopted Methodology for Design Flood Estimation</u>

Based on the review of the suitability of FFA, it was decided that due to the limitations with the approach, the most appropriate methodology was to utilise the synthetic design storm concept from AR&R (1987) and undertake comparative checks against a FFA at Upper Brookfield (143032A). This is in lieu of adopting the results of the FFA and scaling the URBS hydrographs to match the FFA.

The 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 12 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.
- Comparative checks on the design flood estimates undertaken against FFA at the Upper Brookfield (143032A) stream gauge.

6.2.2 Flood Frequency Analysis

A flood frequency analysis of annual maximum flows (based on Log Pearson III distribution) was undertaken at the Upper Brookfield (143032A) stream gauge for the period from 1976 to 2015. For the purposes of this analysis, a water year was defined from July to June, as this incorporates the wet season, when nearly all flood events occur in Brisbane.

Figure 6.2 and Table 6.3 indicate the fitted Log Pearson III distribution as well as the confidence limits. As there is only 40 years of data, the confidence limits are noticeably wider for the 50-yr ARI (2 % AEP) and 100-yr ARI (1 % AEP) due to the greater uncertainty.

Flood estimates for the 2-yr ARI (50 % AEP) and 5-yr ARI (20 % AEP) are not presented in the table, as they are better derived by a POT analysis rather than Annual Maxima. An estimate is provided for the 100-yr ARI (1 % AEP), although it is noted that the probability of the event, requires considerable extrapolation beyond the observed record length.

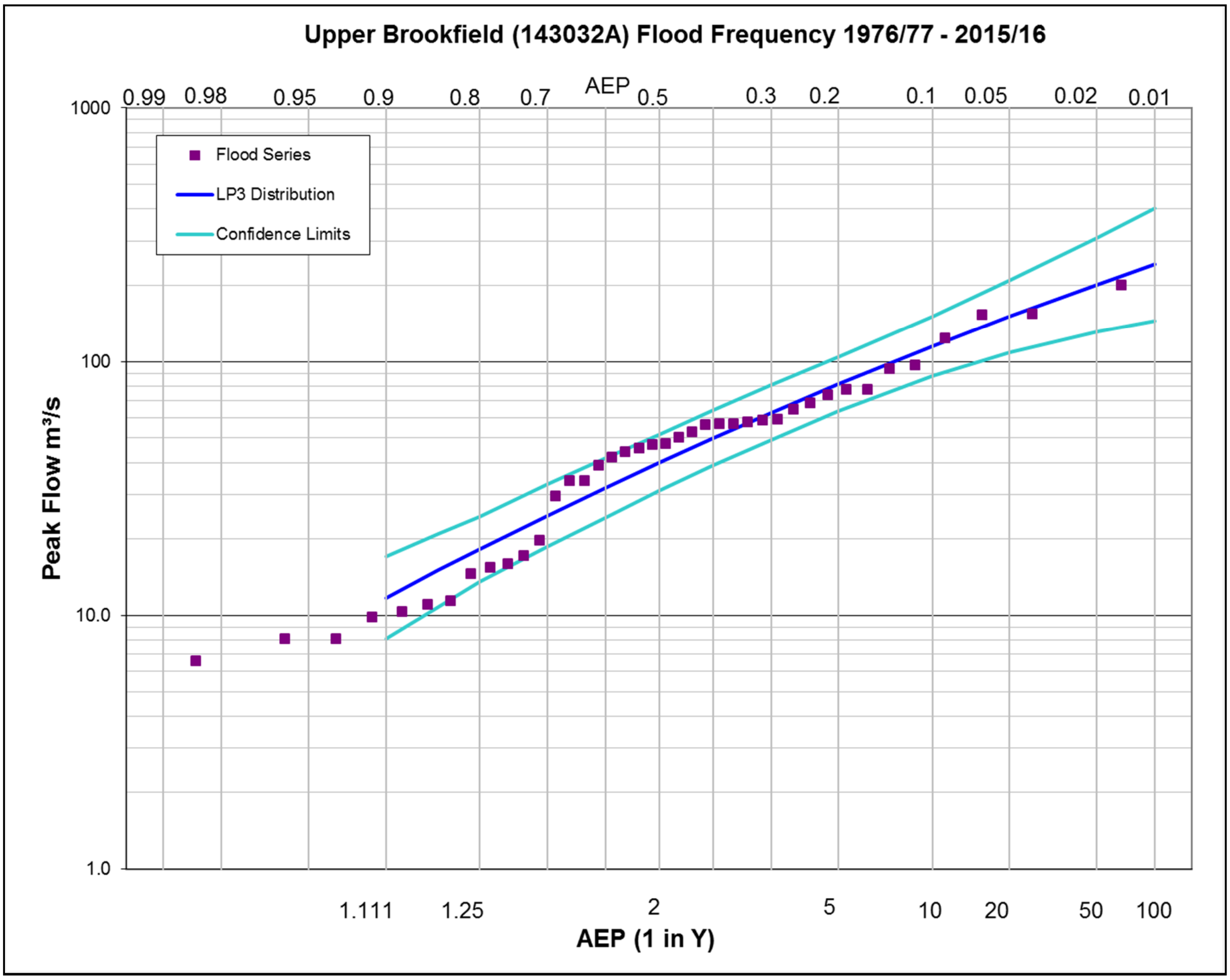

Figure 6.2: Flood Frequency Curve for Upper Brookfield (143032A)

Table 6.3 – Flood Frequency Analysis for Upper Brookfield (143032A)

ARI	AEP (%)	Fitted Log-Pearson III Distribution		
		95 % confidence limit	Q_y	5 % confidence limit
10	10	88	115	151
20	5	109	151	209
50	2	132	201	308
100	1	145	242	404

6.2.3 URBS Model Set-up

The calibrated URBS 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 typically increase the impervious land-use factors.

Appendix B presents the URBS 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.

When compared to the existing catchment development, the ultimate catchment development resulted in small increases in impervious area for Sub-catchments 27, 28, 32, 33, 34, 37, 40, 41, 44 and 47; all of which are towards the lower end of the catchment.

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 slightly conservative, although a sensitivity analysis on the value of the IL has not been undertaken.

A CL of 0 / 2.5 mm/hr was adopted for the impervious / pervious areas within the catchment respectively. These values were determined from the results of the calibration and verification process and are within the recommended ranges of AR&R (1987).

Design IFD Data

Design rainfall depth / intensity data was obtained from the Bureau of Meteorology (BOM) website, based on AR&R (1987). Table 6.4 indicates the adopted design IFD data, which was extracted at the centroid of the catchment.

Checks were undertaken at some selected locations around the catchment, from which it was ascertained that there was only a small variation in design rainfall depth throughout the catchment. On this basis, it was deemed appropriate to adopt a consistent design rainfall depth throughout the catchment.

Table 6.4 – Adopted Design Event IFD Data

Duration (hrs)	Rainfall Intensity (mm/hr)					
	2-yr ARI (50 % AEP)	5-yr ARI (20 % AEP)	10-yr ARI (10 % AEP)	20-yr ARI (5 % AEP)	50-yr ARI (2 % AEP)	100-yr ARI (1 % AEP)
0.5	68.3	89.7	103	121	145	164
1	45.9	60.8	70.2	82.6	99.4	113
2	29.2	38.8	44.9	52.8	63.7	72.3
3	22.1	29.3	33.8	39.8	48.0	54.5
6	13.5	17.9	20.7	24.3	29.3	33.2
12	8.46	11.2	12.9	15.2	18.3	20.8
18	6.58	8.75	10.09	11.89	14.36	16.36
24	5.47	7.31	8.46	10	12.1	13.7

Design hyetographs

Design hyetographs were derived from the techniques in AR&R (1987). Hyetographs were created for the 2-yr ARI (50 % AEP), 5-yr ARI (20 % 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, 2 hours, 3 hours, 6 hours, 12 hours, 18 hours and 24 hours.

Gold Creek Reservoir

To enable the reservoir to be modelled in URBS, there was a requirement to adopt an initial water level (IWL) for the reservoir as well as a gate open / closed status. The major considerations for the adoption of these parameters are as follows:

- The current Seqwater operational procedures for the gate for the outlet pipe.
- The likelihood of the outlet pipe being blocked prior to and / or during an event.
- The likelihood of the IWL in the reservoir being above 92.75 mAHD due to recent rainfall events.
- The impact on downstream flows from differing IWLs and gate open / closed status.

Seqwater advised that the normal operational procedure is to leave the gate for the outlet pipe open; meaning that in periods of dry weather the reservoir level would be at or below 92.75 mAHD prior to the commencement of a rainfall event.

As noted previously, advice from Seqwater also indicates that both the approach channel (spillway slot) and the trash screen in front of the outlet pipe are frequently prone to blockage. This would suggest the likelihood of blockage of the outlet pipe is high.

Drawdown calculations indicate that it would take over 3 days for the reservoir to drain from the level of the spillway (95.75 mAHD) to a level of around 93 mAHD (0.25 m above the invert level of the outlet pipe) with a fully open gate. This is on the basis of no flow into the reservoir within this period. However, given that there is likely to be some rainfall and baseflow within this period, this duration is likely to be higher. Our review of the January 2013 and May 2015 events indicated that it took nearly 8 days and over 6 days respectively for the reservoir to drain from the spillway level (95.75 mAHD) to a level of around 93.25 mAHD (0.5 m above the invert level of the outlet pipe). Likewise, our review of a wet month, (i.e. January 2011) indicated that the water level in the reservoir was above 94 mAHD for more than half of the month.

To understand the impact that the initial water level (IWL) and gate open / closed status has on downstream flows; a comparison was undertaken for the 2-yr ARI (50 % AEP) to 100-yr ARI (1 % AEP) considering three dam configurations:

- Configuration 1: IWL at outlet pipe invert level (92.75 mAHD) and gate open
- Configuration 2: IWL at outlet pipe invert level (92.75 mAHD) and gate closed / blocked by debris
- Configuration 3: IWL at spillway level (95.75 mAHD) and gate closed / blocked by debris

Table 6.5 indicates the results of this comparison at (i) immediately downstream of the reservoir, and (ii) upstream extent of the TUFLOW model. The results indicate that Configuration 1 and Configuration 2 produce similar results; however Configuration 3 results in higher flows downstream, particularly in the lower order events.

Table 6.5 – Comparison of Reservoir Outflows for differing Configurations

ARI (years)	Peak Discharge (m^3/s)					
	Configuration 1		Configuration 2		Configuration 3	
	Reservoir Downstream	Upstream TUFLOW Model	Reservoir Downstream	Upstream TUFLOW Model	Reservoir Downstream	Upstream TUFLOW Model
2	3.6	6.9	5.6	6.8	23.3	27.9
5	16.8	20.3	19.9	24.0	35.9	43.1
10	25.1	30.2	26.0	31.4	44.4	53.3
20	32.5	39.2	32.8	39.5	56.2	67.5
50	46.2	55.8	46.9	56.7	72.5	87.2
100	59.3	71.6	59.6	72.0	86.8	104.3

On the basis of this analysis, it was decided to adopt the more conservative Configuration 3 conditions as it may be conceivable that the water level in the reservoir prior to the commencement a storm event could be above 92.75 mAHD due to a combination of recent rainfall and / or blockage of the outlet pipe.

6.2.4 Comparison of FFA to URBS at Upper Brookfield (143032A)

Table 6.6 presents a comparison of the peak flows between the URBS model and the FFA for the 10-yr ARI (10 % AEP) to the 100-yr ARI (1 % AEP). The results indicate a very good correlation between the URBS and FFA peak flows for all four ARIs, with the largest difference being 6 % in the 10-yr ARI (10 % AEP) event.

Table 6.6 – Flood Frequency Table for Upper Brookfield (143032A)

AEP (1 in Y)	AEP (%)	Peak Flow (m^3/s)		Difference (%)
		FFA	URBS	
10	10	115	122	6.0
20	5	151	153	1.3
50	2	201	196	-2.4
100	1	242	233	-3.7

As noted previously, there are some limitations with the FFA approach; however this good correlation at Upper Brookfield (143032A) would appear to add some credibility to the URBS design flow estimation.

6.3 Design Event Hydraulic Modelling

6.3.1 Overview

The TUFLOW model was used to determine design flows and flood levels for those scenarios as detailed in Table 6.1 for the 2-yr ARI (50 % AEP) to the 100-yr ARI (1 % AEP) events. These events were simulated for durations from 30 minutes to 24 hours.

6.3.2 TUFLOW model extents

The Scenario 1, 2 and 3 TUFLOW model extents were the same as the TUFLOW model developed for the calibration and verification events.

6.3.3 TUFLOW model roughness

The hydraulic roughness in the calibrated TUFLOW model was updated (as required) to represent the ultimate catchment conditions; which included MRC for Scenarios 2 and 3.

6.3.4 TUFLOW model boundaries

Design Inflows

The design inflow (Q-T) boundaries to the TUFLOW model were taken from the URBS model for each ARI and duration. The inflow locations were the same as for the TUFLOW model developed for the calibration and verification events.

Design Tailwater Boundary

The design event TUFLOW model utilised a fixed Mean High Water Springs (MHWS) water level (H-T) boundary at the downstream boundary with the Brisbane River. At this location the value of MHWS is 1.27 mAHD.

6.4 Results and Mapping

6.4.1 Critical Durations

A full range of durations (30 minutes to 24 hours) were simulated for the 2-yr ARI (50 % 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 6.7. For this purpose, the critical duration is the storm duration which produces the peak flood level.

Table 6.7 – Critical Durations at Key Locations

Key Location	Critical Duration (minutes)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
Moggill Creek						
U/S Model Extent	120	120	120	120	120	120
Gold Creek Confluence	180	120	120	120	120	120
Gap Creek Confluence	180	180	180	180	120	120
McKay Brook Confluence	180	180	180	180	180	180
Moggill Road (S2)	180	180	180	180	180	180
Brisbane River Confluence	180	180	180	180	180	180
Gold Creek						
Dam In	120	120	120	120	120	120
Dam Out	180	180	180	180	180	180
U/S Model Extent	180	180	180	180	180	180
Gold Creek Road (S37)	180	180	180	180	180	180
Savages Road (S34)	360	360	180	180	180	180
Gap Creek						
U/S Model Extent	120	120	120	120	120	120
Gap Creek Road (S31)	120	120	120	120	120	120
Brookfield Road (S28)	120	120	120	120	120	120
McKay Brook						
U/S Model Extent	60	60	60	60	60	60
Hillcrest Place	60	60	60	60	60	60
Brookfield Road (S17)	60	60	60	60	60	60

The results indicate that along Moggill Creek, the 120-minute and 180-minute durations produce the peak flood levels. Within Gold Creek, the critical duration varies between events due to the influence of Gold Creek Reservoir. Within Gap Creek and McKay Brook, the critical durations are 120-minute and 60-minute, respectively.

6.4.2 Peak Discharge Results

Table 6.8 provides peak flow results at selected major hydraulic structures for the Scenario 1 conditions. This information is from the URBS hydrologic model.

Table 6.8 – Design Event Peak Discharge at Selected Major Structures (Scenario 1)

Location	Peak Discharge (m^3/s)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
Moggill Creek						
Upper Brookfield Road (S15)	59.8	90.5	111.4	139.8	179.6	213.3
Upper Brookfield Road (S12)	65.7	99.2	122.1	153.2	196.2	232.9
Brookfield Road (S9)	120.9	180.9	220.8	276.2	351.3	415.1
Rafting Ground Road (S7)	138.6	207.0	252.3	315.0	399.9	472.3
Rafting Ground Road (S6)	139.5	208.1	253.6	316.7	401.8	474.3
Moggill Road (S2)	153.7	226.2	274.9	341.4	430.8	507.1
Gold Creek						
Gold Creek Road (S46)	27.9	43.1	53.3	67.6	87.2	104.3
Gold Creek Road (S40)	31.7	48.7	60.2	76.0	98.1	117.2
Gold Creek Road (S37)	36.1	55.4	68.4	86.4	111.6	133.2
Savages Road (S34)	39.9	60.5	74.5	93.8	120.8	143.7
Gap Creek						
Gap Creek Road (S31)	16.2	24.5	30.2	37.9	48.6	57.7
Brookfield Road (S28)	21.1	31.7	38.8	48.5	61.9	73.3
McKay Brook						
Tinarra Crescent (S22)	1.4	2.1	2.6	3.2	4.1	4.8
Mirbelia Street (S18)	8.5	12.5	15.2	18.9	23.9	28.2
Brookfield Road (S17)	11.1	16.3	19.7	24.4	30.5	35.9

6.4.3 Peak Flood Levels

Tabulated peak flood level results for the design events are provided at the following locations for all creeks:

- Scenario 1: 2-yr ARI (50 % AEP) to 100-yr ARI (1 % AEP) events – Appendix D
- Scenario 3: 2-yr ARI (50 % 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 24 hours. The peak flood levels are extracted along the current AMTD line for all creeks.

6.4.4 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 within the catchment. These flood frequency curves were based on the Scenario 1 modelling and are indicated in Figure 6.3 and Figure 6.4. It is noted that at locations downstream of Gold Creek Reservoir there is greater uncertainty in the estimation due to the differences in the initial reservoir water level between the synthetic design events and the historical events.

Table 6.9 indicates the estimated return period of the historical events at the selected locations; based on the flood frequency curves.

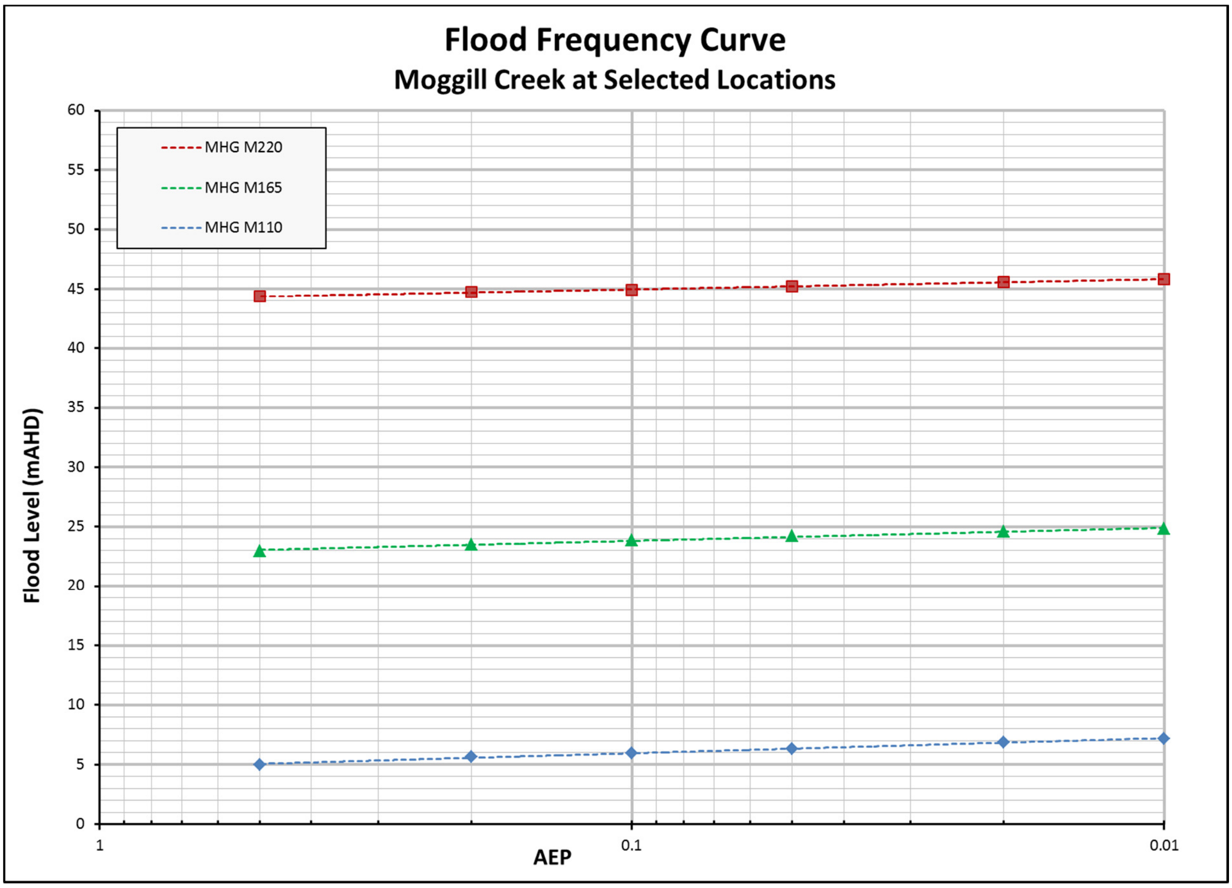

Figure 6.3: Flood Frequency Curve – Moggill Creek at Selected Locations

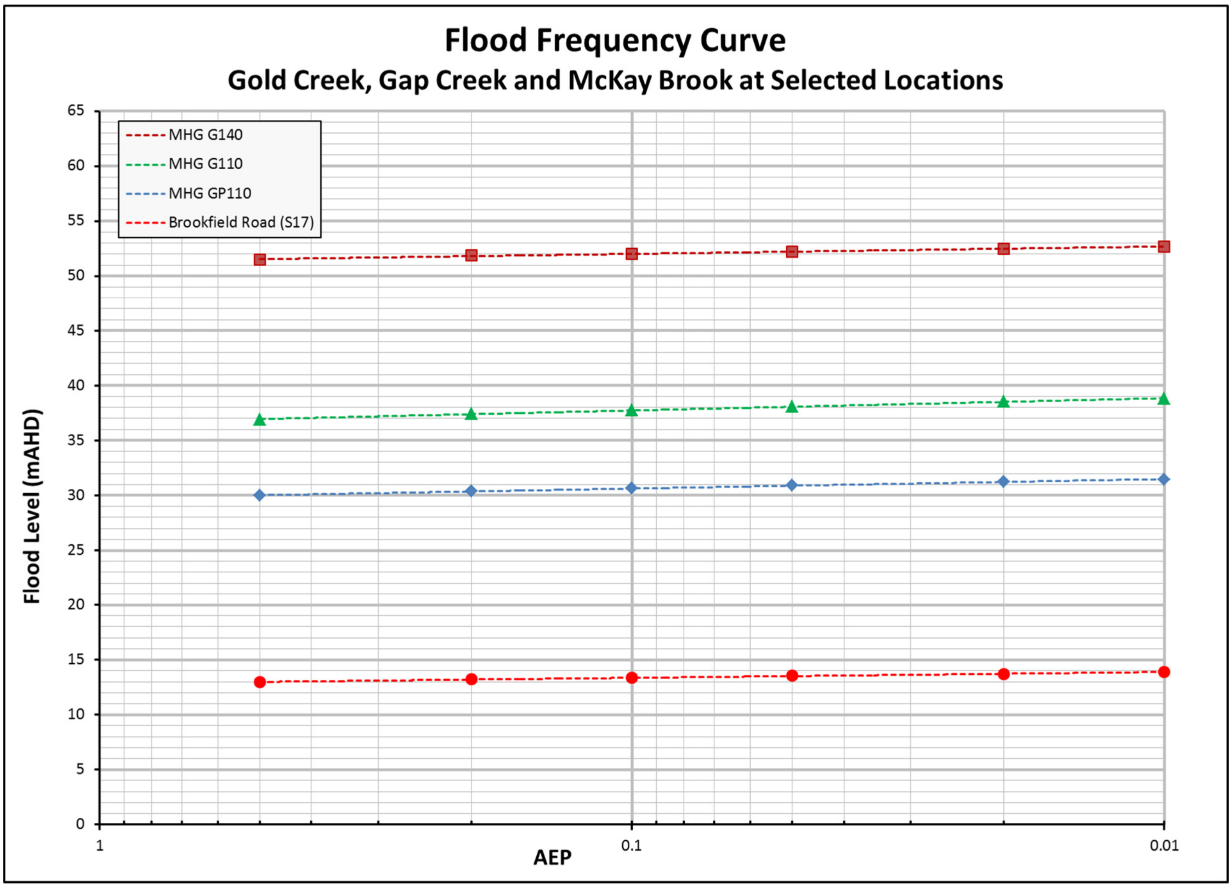

Figure 6.4: Flood Frequency Curve – Gold Creek, Gap Creek and McKay Brook at Selected Locations

Table 6.9 – Estimated Magnitude of Historical Events

Location	Event Magnitude			
	May 2015	Jan 2013	May 2009	Nov 2008
Moggill Creek				
MHG M220	10-yr ARI (10 % AEP)	10-yr to 20-yr ARI (10 % to 5 % AEP)	20-yr to 50-yr ARI (5 % to 2 % AEP)	20-yr ARI (5 % AEP)
MHG M165	5-yr to 10-yr ARI (20 % to 10 % AEP)	20-yr to 50-yr ARI (5 % to 2 % AEP)	50-yr ARI (2 % AEP)	10-yr to 20-yr ARI (10 % to 5 % AEP)
MHG M110	10-yr ARI (10 % AEP)	20-yr to 50-yr ARI (5 % to 2 % AEP)	50-yr to 100-yr ARI (2 % to 1 % AEP)	10-yr to 20-yr ARI (10 % to 5 % AEP)
Gold Creek				
MHG G140	2-yr ARI (50 % AEP)	20-yr ARI (5 % AEP)	50-yr ARI (2 % AEP)	5-yr to 10-yr ARI (20 % to 10 % AEP)
MHG G110	2-yr to 5-yr ARI (50 % to 20 % AEP)	20-yr to 50-yr ARI (5 % to 2 % AEP)	50-yr ARI (2 % AEP)	5-yr to 10-yr ARI (20 % to 10 % AEP)
Gap Creek				
MHG GP110	10-yr to 20-yr ARI (10 % to 5 % AEP)	20-yr to 50-yr ARI (5 % to 2 % AEP)	100-yr ARI (1 % AEP)	50-yr ARI (1 % AEP)
McKay Brook				
Brookfield Road (S17)	5-yr ARI (20 % AEP)	10-yr to 20-yr ARI (10 % to 5 % AEP)	100-yr ARI (1 % AEP)	10-yr ARI (10 % AEP)

6.4.5 Rating Curves

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

- Upper Brookfield Road (S15) – Moggill Creek
- Upper Brookfield Road (S12) – Moggill Creek
- Brookfield Road (S9) – Moggill Creek
- Rafting Ground Road (S6) – Moggill Creek
- Moggill Road (S1 & S2) – Moggill Creek
- Gold Creek Road #1 (S37) – Gold Creek
- Brookfield Road (S28) – Gap Creek
- Mirbelia Street (S16) – McKay Brook
- Brookfield Road (S17) – McKay Brook

The rating curves were developed by simulating a slowly increasing flow over a period of 60 hours, with a constant tailwater level in the Brisbane River of 1.5 m AHD. In the lower reach of both Moggill Creek and McKay Brook, care should be taken if utilising the rating curves, as they have the potential to change depending on the flow conditions in the Brisbane River.

6.4.6 Flood Immunity of Existing Crossings

The flood immunity of the existing waterway crossings under Scenario 1 conditions is presented in Table 6.10. The value indicated is the ARI of the largest flood which does not fully overtop the road / structure, when considering the 2-yr ARI (50 % AEP) to 100-yr ARI (1 % AEP) events. Interpolation between ARIs to ascertain an intermediate ARI value has not been undertaken.

Table 6.10 – Flood Immunity at Major Structures

Location	Flood Immunity (ARI)
Moggill Creek	
Upper Brookfield Road (S15)	50-yr ARI (2 % AEP)
Upper Brookfield Road (S12)	> 100-yr ARI (1 % AEP)
Brookfield Road (S9)	5-yr ARI (20 % AEP)
Rafting Ground Road (S7)	< 2-yr (50 % AEP)
Rafting Ground Road (S6)	< 2-yr (50 % AEP)
Moggill Road (S2)	20-yr ARI (5 % AEP)
Gold Creek	
Gold Creek Road #8 (S46)	< 2-yr (50 % AEP)
Gold Creek Road #7 (S45)	< 2-yr (50 % AEP)
Gold Creek Road #6 (S44)	< 2-yr (50 % AEP)
Gold Creek Road #5 (S43)	< 2-yr (50 % AEP)

Location	Flood Immunity (ARI)
Gold Creek Road #4 (S42)	< 2-yr (50 % AEP)
Gold Creek Road #3 (S41)	< 2-yr (50 % AEP)
Gold Creek Road #2 (S40)	< 2-yr (50 % AEP)
Gold Creek Road #1 (S37)	> 100-yr ARI (1 % AEP)
Adavale Street (S35)	< 2-yr (50 % AEP)
Savages Road (S34)	10-yr ARI (2 % AEP)
Gap Creek	
Gap Creek Road (S31)	< 2-yr (50 % AEP)
Brookfield Road (S28)	> 100-yr ARI (1 % AEP)
McKay Brook	
Tinarra Crescent (S22)	> 100-yr ARI (1 % AEP)
Mirbelia Street (S16)	> 100-yr ARI (1 % AEP)
Brookfield Road (S17)	> 100-yr ARI (1 % AEP)
Wexford Street (S27)	> 100-yr ARI (1 % AEP)

6.4.7 Hydrologic-Hydraulic Model Consistency Check (Design Events)

Comparison checks on flow were undertaken between the URBS and TUFLOW models for the 5-yr ARI (20 % 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 120-minute duration storm event.

Figures 6.5 to 6.11 provide comparative plots of the URBS and TUFLOW flow results at the following seven locations. Table 6.11 provides a comparison of the peak flows at these same seven locations.

(i) Moggill Creek at Upper Brookfield (143032A)

(ii) Moggill Creek at Boscombe Road (MHG M165)

(iii) Moggill Creek at Fortrose Street (540061)

(iv) Gold Creek at Gold Creek Road (MHG G150)

(v) Gold Creek at the Confluence with Moggill Creek

(vi) Gap Creek at Brookfield Road (S28)

(vii) McKay Brook at Brookfield Road (S17)

Table 6.11 – Peak Flow Comparison, URBS and TUFLOW

Location	Peak Flow (m^3/s) – 120 minute duration					
	5-yr ARI (20 % AEP)		20-yr ARI (5 % AEP)		100-yr ARI (1 % AEP)	
	URBS	TUFLOW	URBS	TUFLOW	URBS	TUFLOW
Moggill Creek						
Upper Brookfield (143032A)	99.2	100.6	153.2	155.5	232.9	234
MHG M165 (Boscombe Road)	172.7	164.6	264.8	265.3	402.5	408.3
Fortrose Street (540061)	212.9	192.4	321.6	309.3	481.8	475.4
Gold Creek						
MHG G150 (Gold Creek Road)	44.6	45.5	70.5	70.5	110.8	111.3
Confluence with Moggill Creek	54.9	56.1	85.5	87.6	132.4	136.9
Gap Creek						
Brookfield Road (S28)	31.7	31.8	48.5	48.6	73.3	73.3
McKay Brook						
Brookfield Road (S17)	15.9	15.3	23.7	23.1	33.2	33.1

The results indicate an acceptable comparison between the URBS and TUFLOW models. The peak flow is generally within ±10 % and the shape and timing of the hydrographs are consistent at the majority of locations.

In the upper and middle sections of Moggill Creek, there is a very good comparison between the URBS and TUFLOW hydrographs for all three events; refer to Figure 6.5 and Figure 6.6. However, in the lower section of Moggill Creek, there are some differences in the shape and timing. The comparison of peak flow is reasonable; however the URBS model is unable to accurately replicate the shape of the TUFLOW hydrograph.

In the upper and middle sections of Gold Creek, there is a reasonable comparison between the URBS and TUFLOW hydrographs for all three events; refer to Figure 6.8. However, similar to Moggill Creek, there are some differences in shape and timing in the lower section.

For both Gap Creek and McKay Brook, there is a good comparison between the URBS and TUFLOW models for all three events; refer to Figure 6.10 and Figure 6.11.

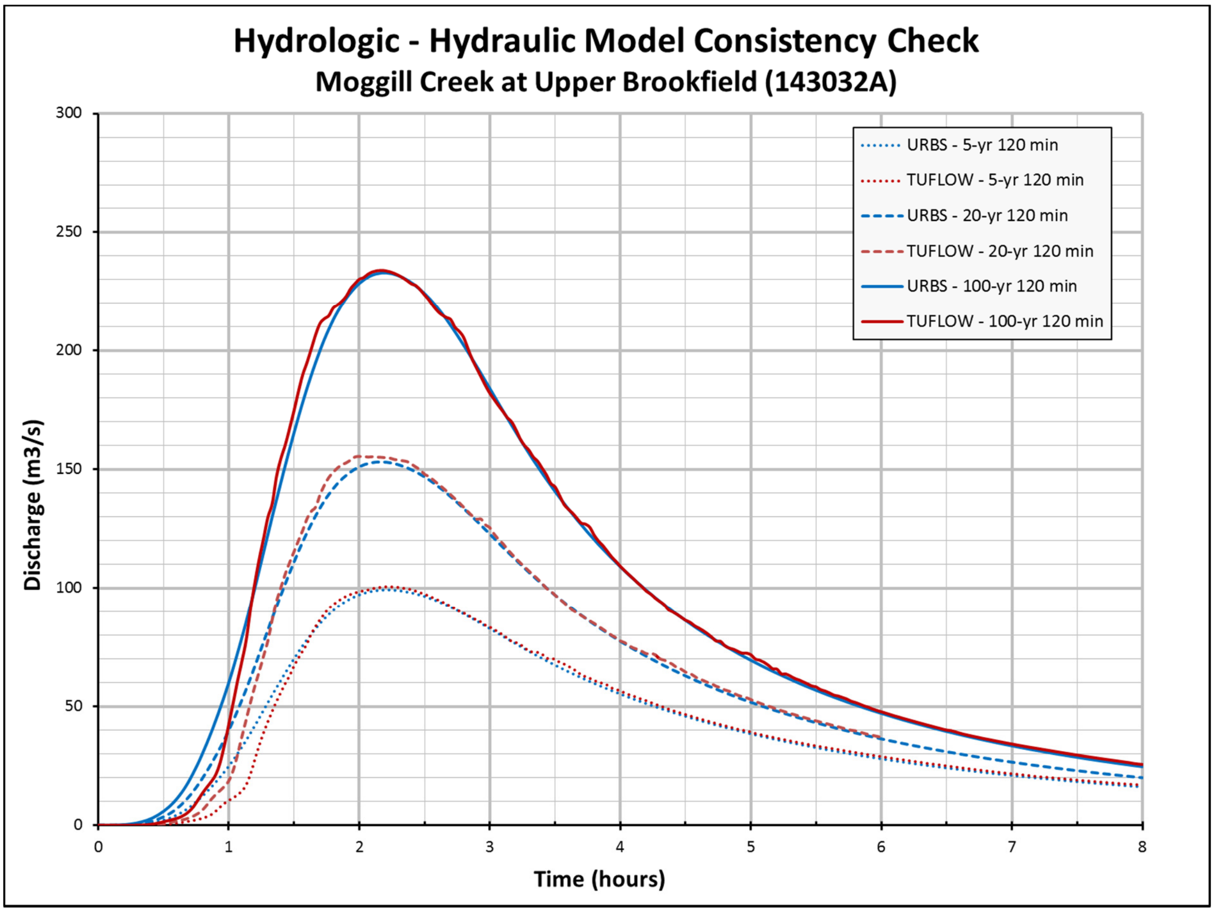

Figure 6.5: Hydrologic-hydraulic comparison at Upper Brookfield (143032A)

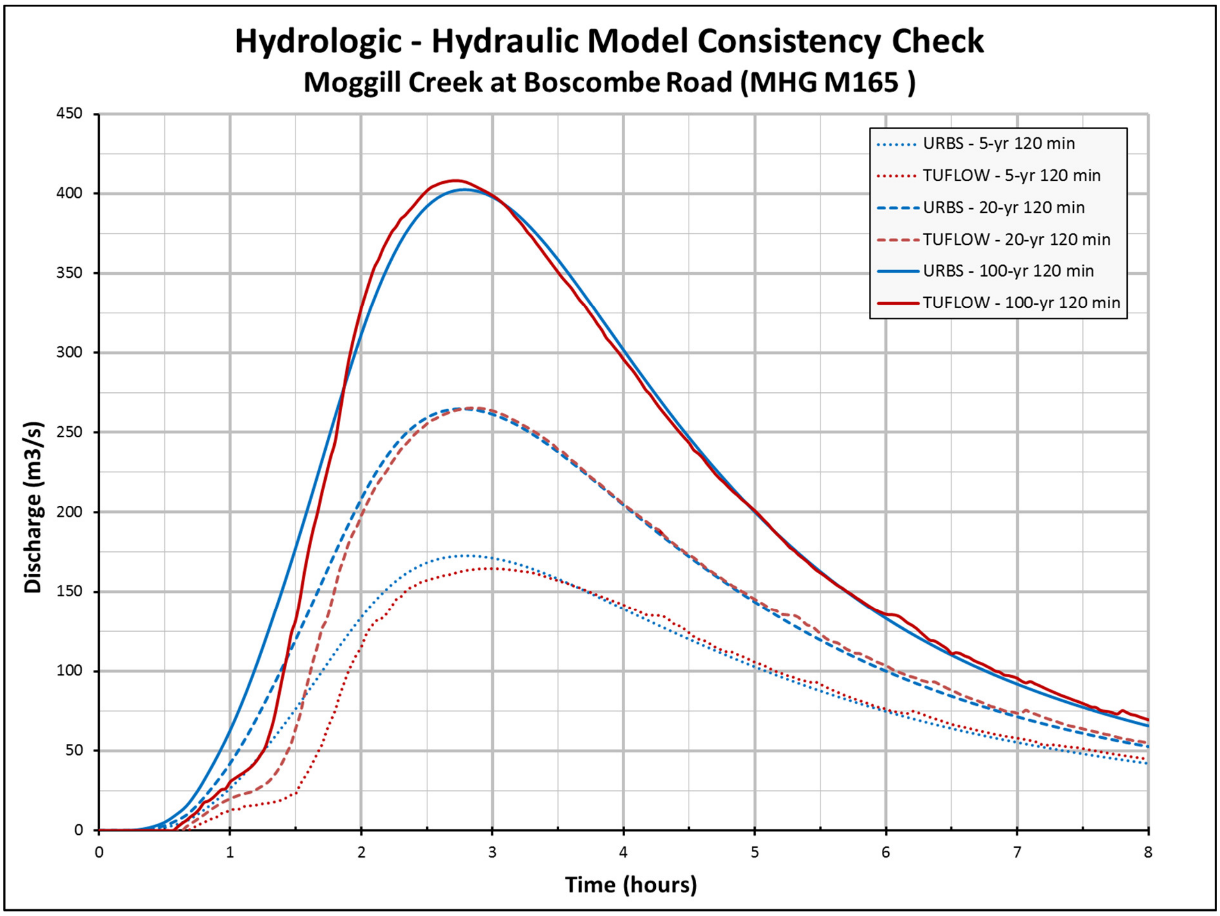

Figure 6.6: Hydrologic-hydraulic comparison at Boscombe Road (MHG M165)

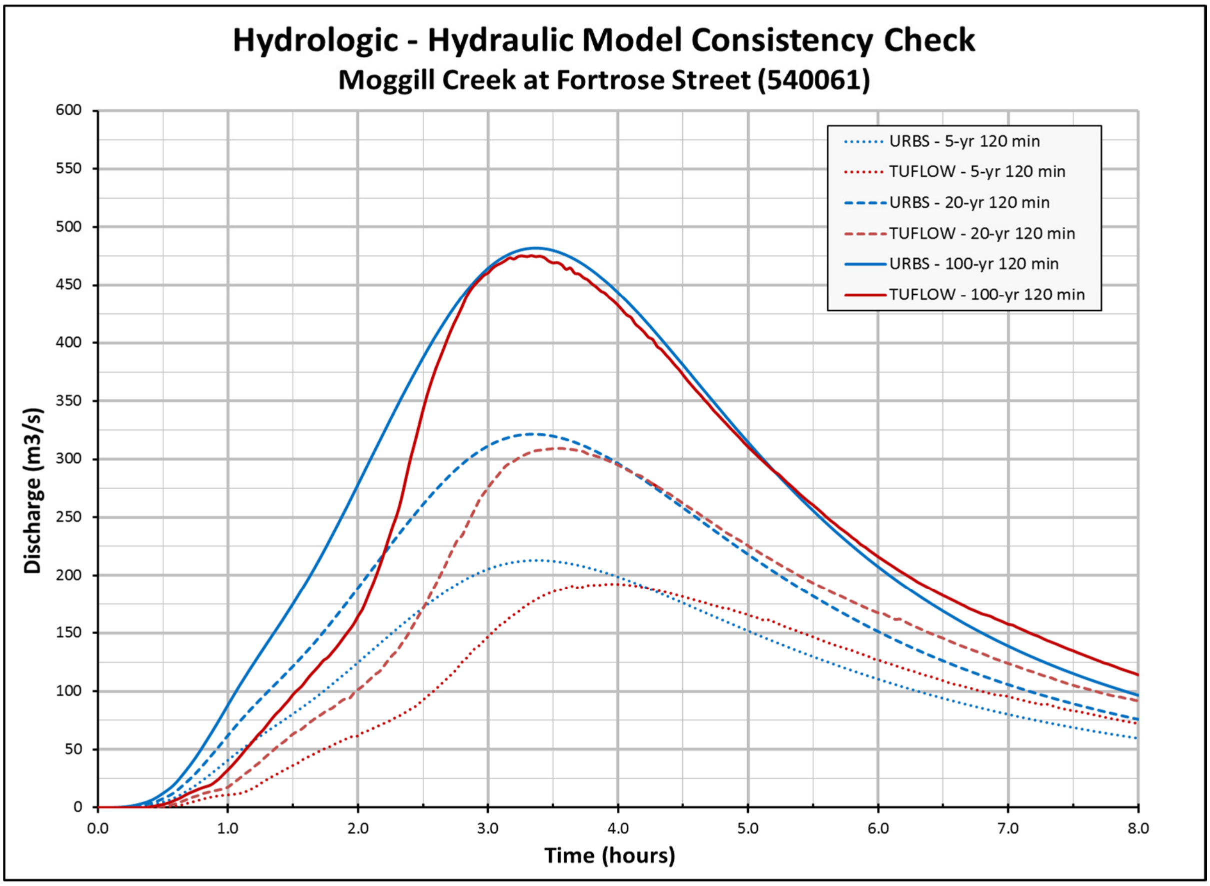

Figure 6.7: Hydrologic-hydraulic comparison at Fortrose Street (540061)

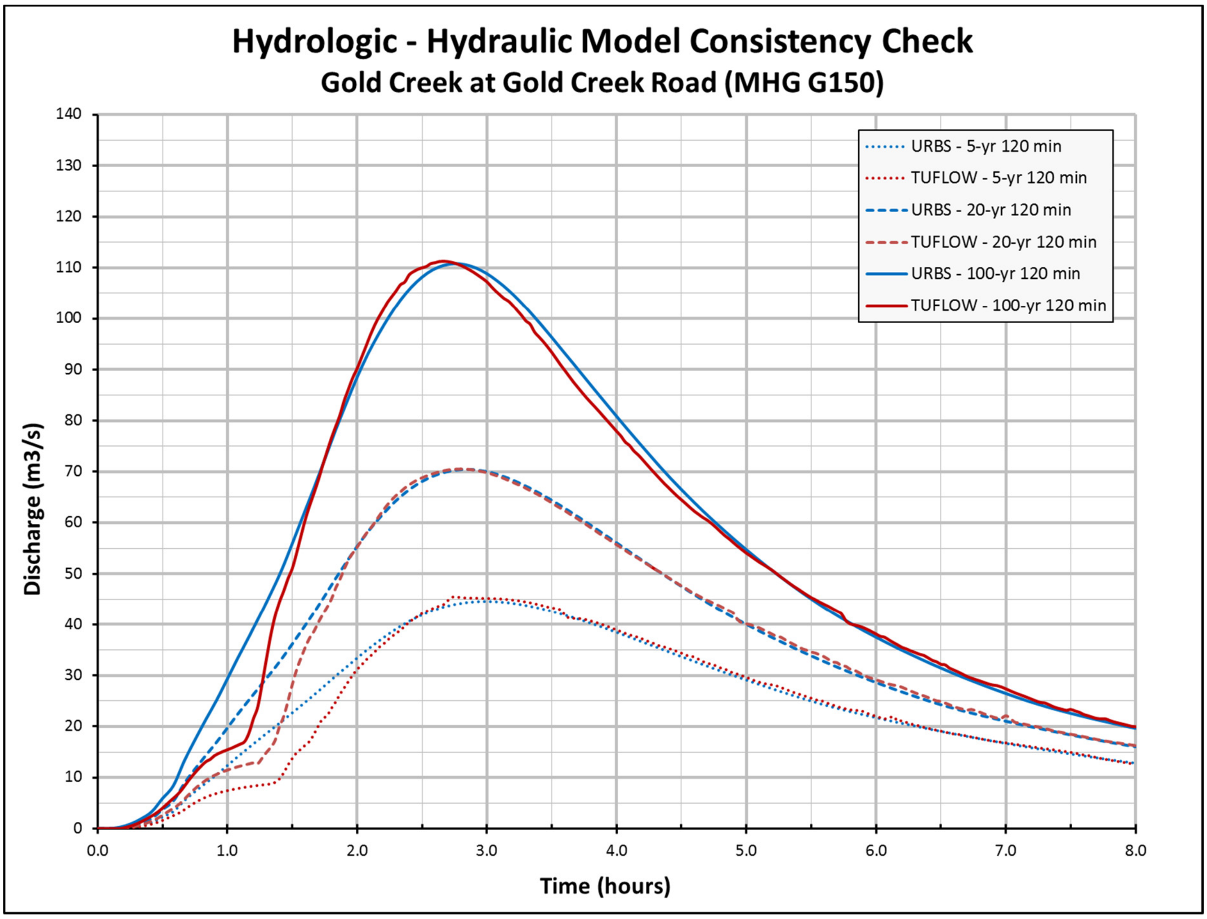

Figure 6.8: Hydrologic-hydraulic comparison at Gold Creek (MHG G150)

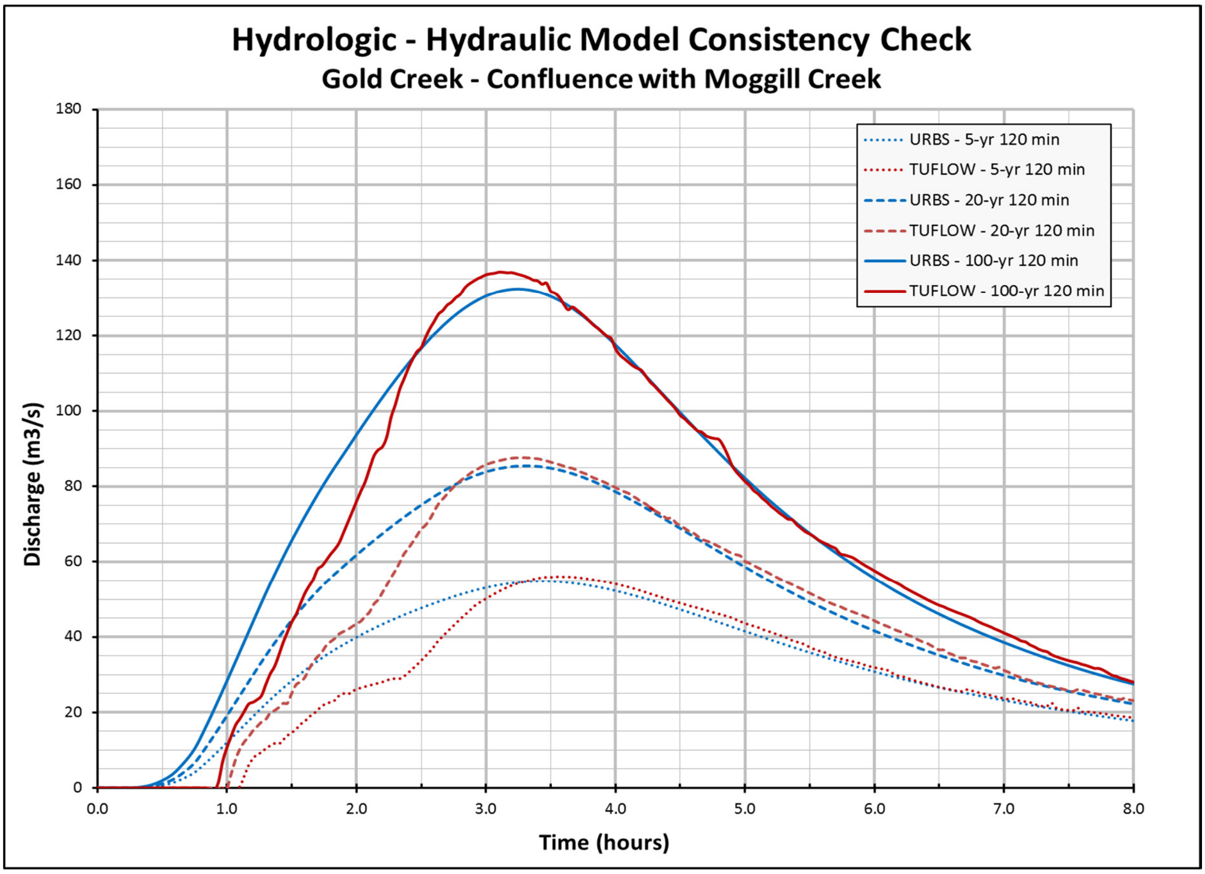

Figure 6.9: Hydrologic-hydraulic comparison at Gold Creek (Confluence with Moggill Creek)

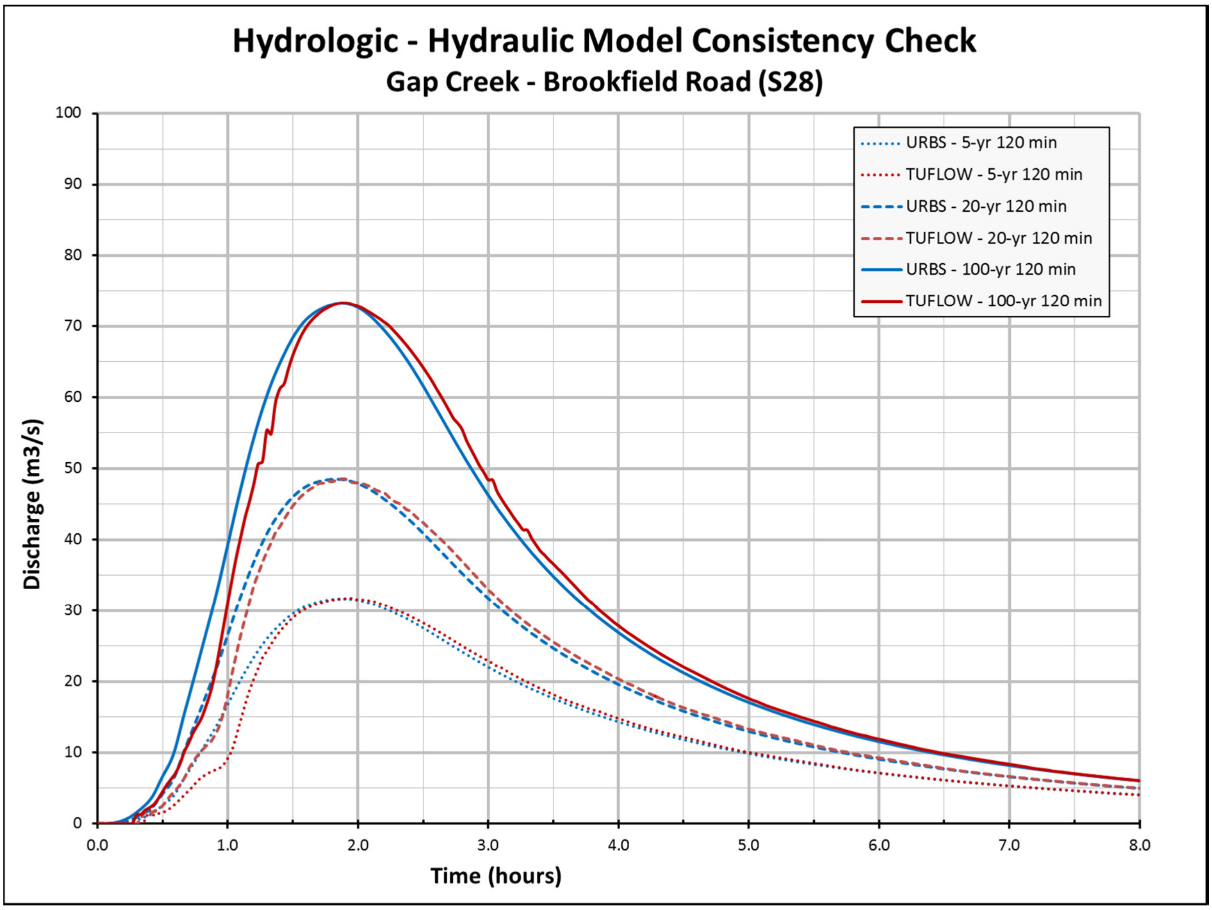

Figure 6.10: Hydrologic-hydraulic comparison at Gap Creek (Brookfield Road)

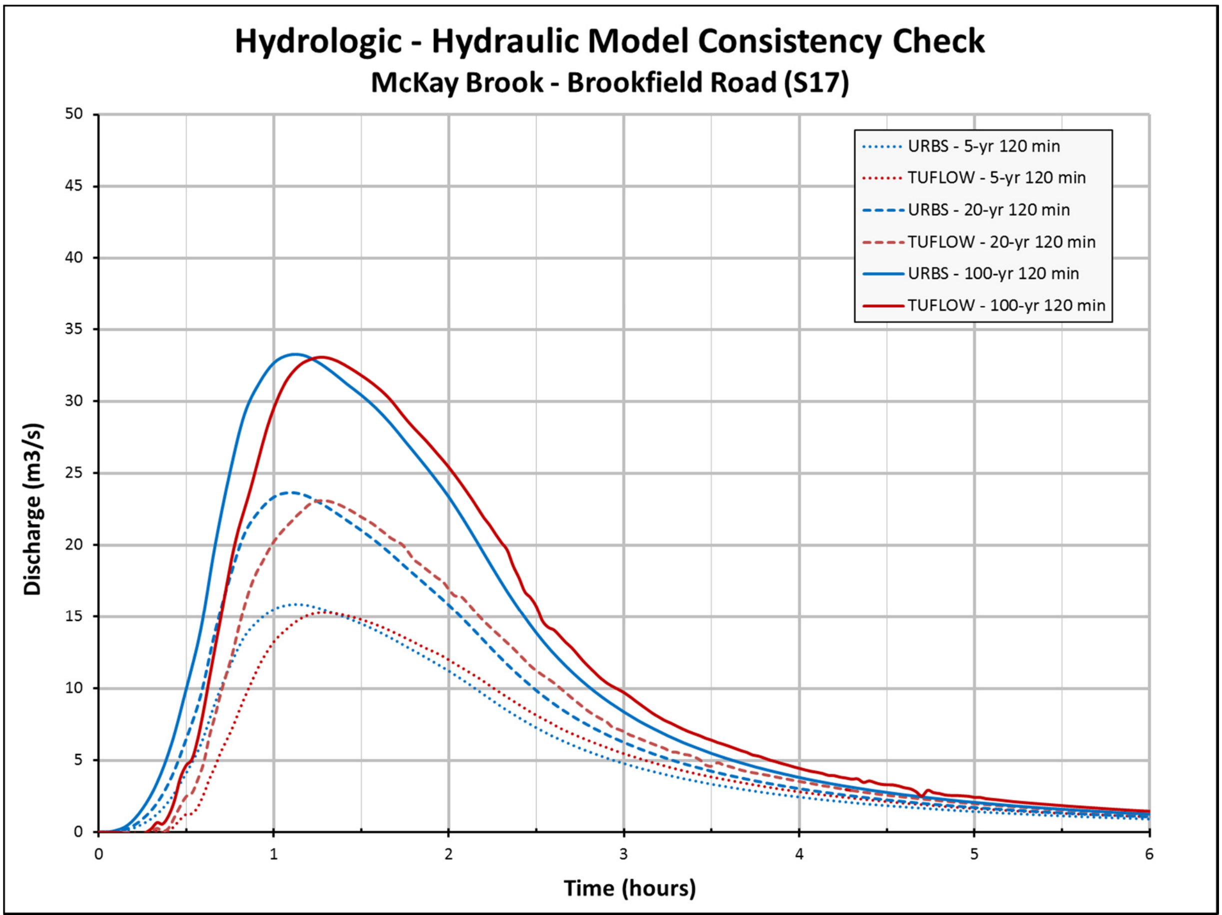

Figure 6.11: Hydrologic-hydraulic comparison at McKay Brook (Brookfield Road)

6.4.8 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.

6.4.9 Flood Mapping

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

- Scenario 1
    - Flood Extent Mapping: 2-yr ARI (50 % 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 6.1. 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 “Modelled Flood 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, for 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 6.1, the ultimate scenario involves modifying the flood model topography to represent a fully developed (filled) floodplain in accordance with BCC City Plan 2014 and in most instances making further allowances for a riparian corridor.

The WaterRIDE™ Flood Manager software 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 automated 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 values were interpolated as CRC-Forge does not produce results for these intermediate values. The interpolation was based on 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 design hyetograph.

Table 7.2 – Adopted Large Event IFD Data

Duration (hrs)	Rainfall Intensity (mm/hr)		
	100-yr ARI (1 % AEP)	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)
0.5	164	179.2	208.8
1	113	126.2	147.1
2	72.3	79.83 (1)	93.03 (1)
3	54.5	59.55	69.38
6	33.2	36.75	42.82
12	20.8	22.73	26.49

Note (1) - Interpolated value

7.3.3 2000-yr ARI (0.05 % AEP) and Probable Maximum Precipitation (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	**TOTAL**		**340**	**816**

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 methodology was documented in the memorandum "Technical Memorandum for Adopted Methodology – Extreme Events Modelling" from BCC Flood Management to BCC Natural Environment Water and Sustainability Branch (NEWS) on the 15th March 2013. This same methodology has also been used on other BCC flood studies recently 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 URBS 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 fixed Highest Astronomical Tide (HAT) water level (H-T) boundary at the downstream boundary with the Brisbane River. At this location the value of HAT is 1.87 mAHD.

7.4.5 Hydraulic Structures

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

To limit issues with model instabilities generated by extreme flows, the TUFLOW model for the 2000-yr ARI (0.05 % AEP) and PMF events excluded the following hydraulic structures:

- Gold Creek Road Culvert (S44)
- Savages Road Bridge (S34) – PMF only

Similarly, the TUFLOW model for the PMF event excluded handrail blockage for the Savages Road Bridge (S34) over Gold Creek.

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 all creeks:

- Scenario 1: 200-yr ARI (0.5 % AEP) to 2000-yr ARI (0.05 % AEP) events – Appendix J
- 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 each creek is provided in Figure 7.1 to Figure 7.4.

The flood profiles for the 200-yr ARI (0.5 % AEP), 500-yr ARI (0.2 % AEP) and 2000-yr ARI (0.05 % AEP) events are observed to follow a very similar trend when compared to the 100-yr ARI (1 % AEP) flood profile along all four creeks. Typically, as the bed slope (gradient) of the creek increases, the relative differences in flood level between events decreases. The largest differences in relative flood level for the three tributaries occur at the confluence with Moggill Creek, which is primarily due to backwater effects from Moggill Creek.

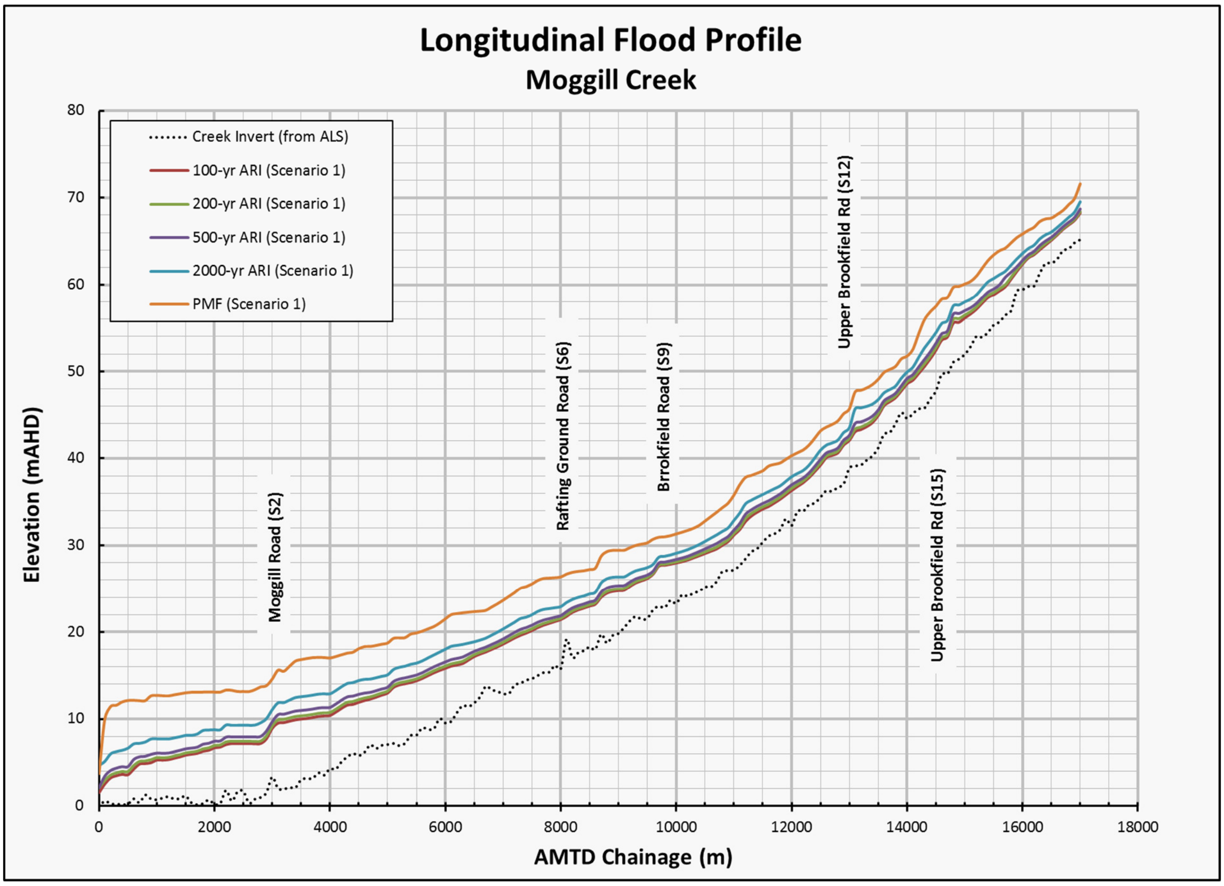

Figure 7.1: Longitudinal Flood Profile – Moggill Creek

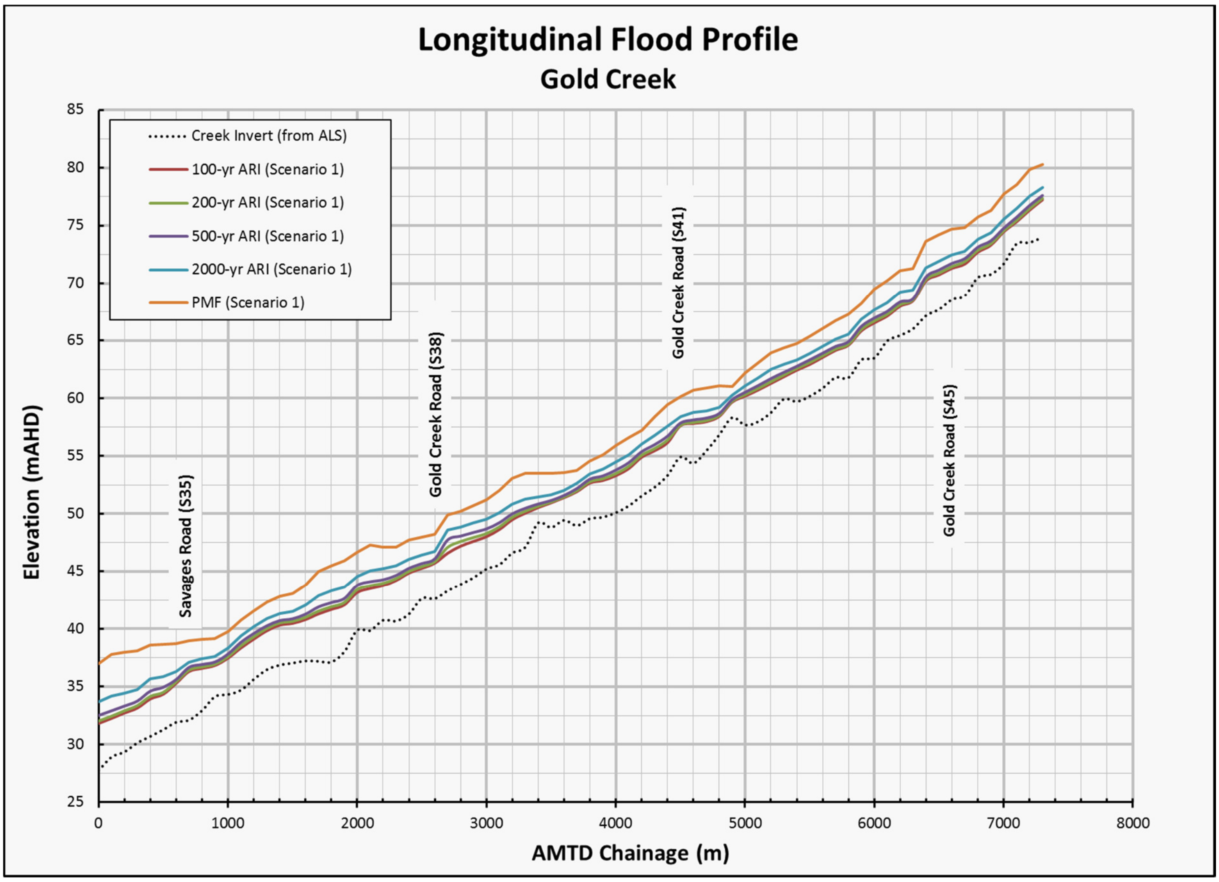

Figure 7.2: Longitudinal Flood Profile – Gold Creek

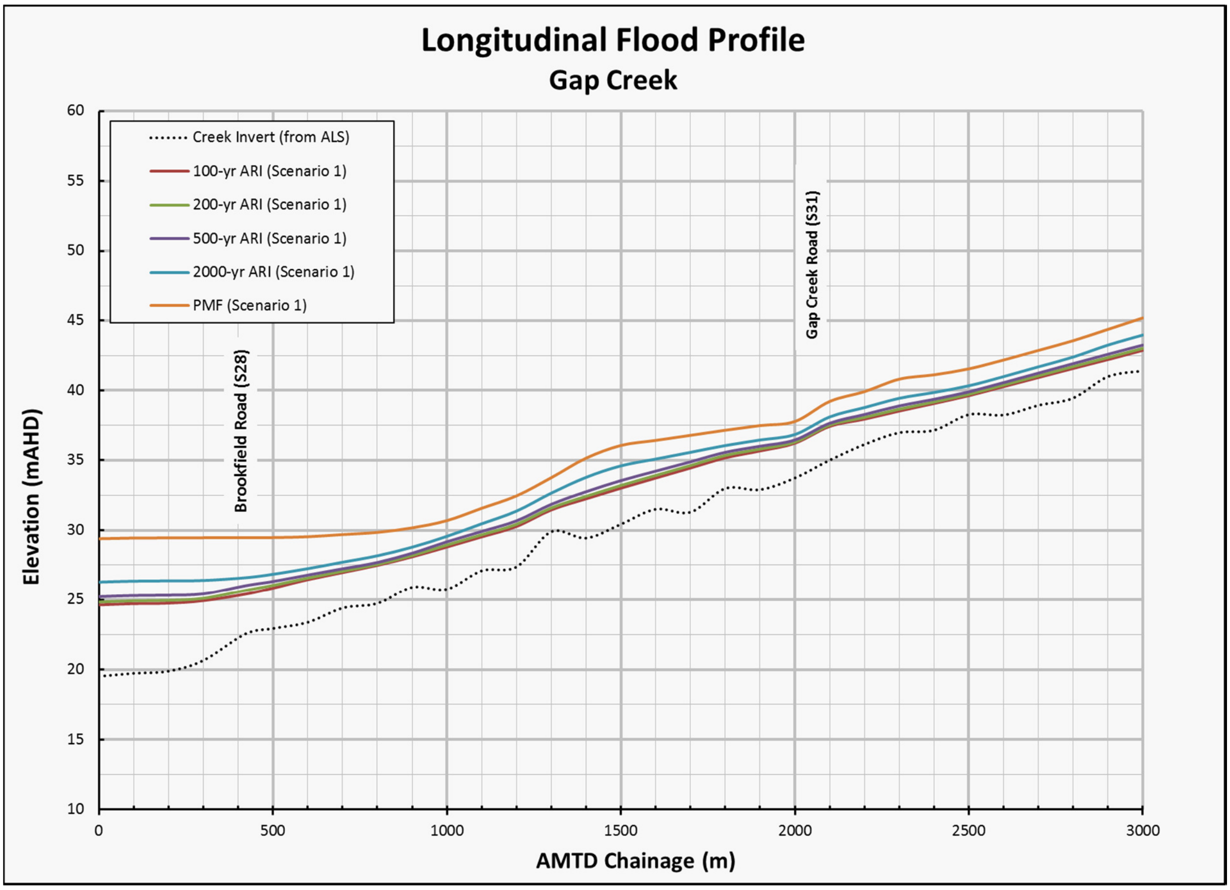

Figure 7.3: Longitudinal Flood Profile – Gap Creek

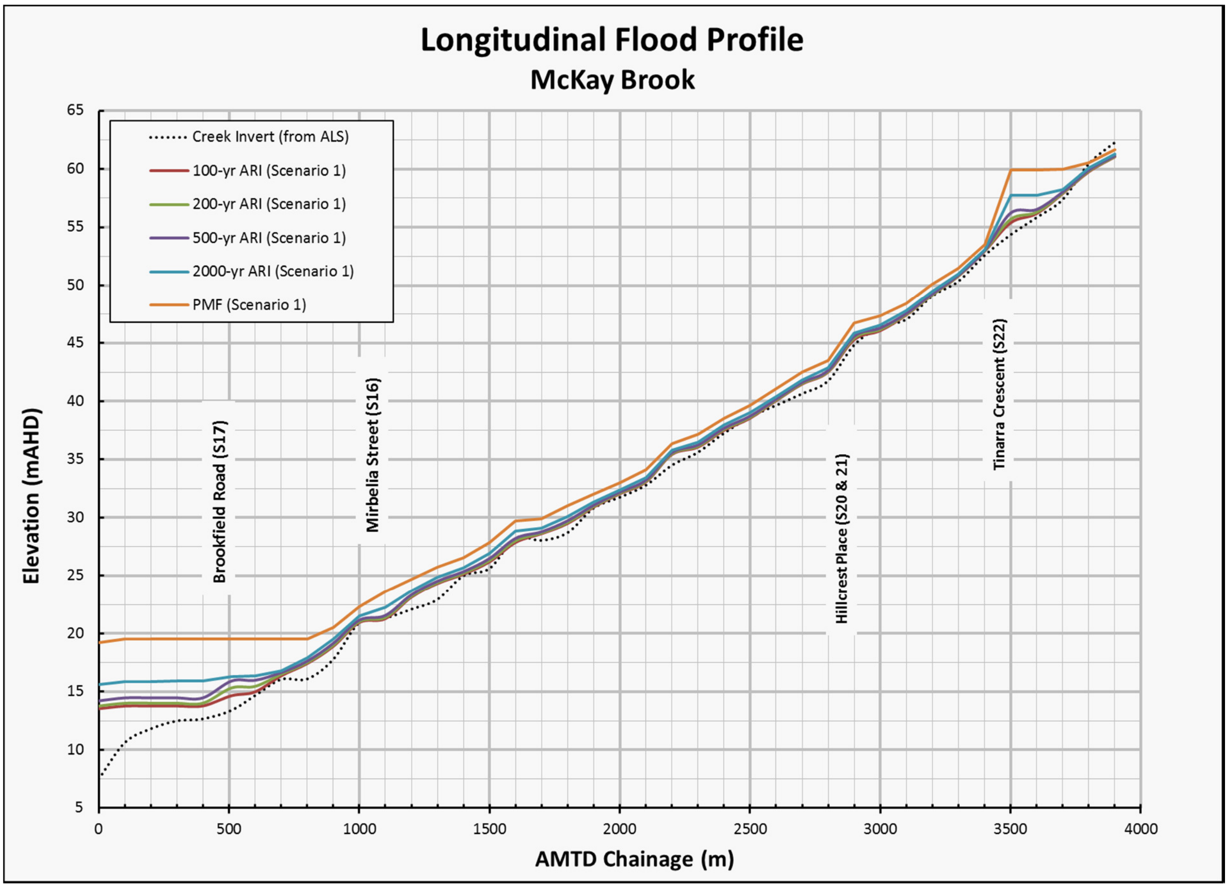

Figure 7.4: Longitudinal Flood Profile – McKay Brook

The McKay Brook flood profile for the PMF indicates a significant increase in flood level upstream of Tinarra Crescent, which is a result of the very high road embankment being overtopped in the PMF only.

The average increase in flood level along the length of each creek, when compared to the 100-yr ARI (1 % AEP) flood profile, is indicated in Table 7.4. The results indicate the largest differences are in Moggill Creek and the smallest in McKay Brook; which is largely a result of the differences in flow due to the relative size of the catchment.

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			
	Moggill Creek	Gold Creek	Gap Creek	McKay Brook
200-yr ARI (0.5 % AEP)	0.25	0.17	0.15	0.13
500-yr ARI (0.2 % AEP)	0.67	0.44	0.39	0.32
2000-yr ARI (0.05 % AEP)	1.86	1.16	1.02	0.79
PMF	5.02	3.01	2.63	2.19

8.0 Climate Variability

8.1 Overview

BCC flood studies are required to undertake a sensitivity analysis to assess climate variability. The following sections provide the details of these analyses.

8.2 Climate Variability

8.2.1 Overview

In order for BCC to undertake informed future land-use planning, there is a requirement to understand the impacts of climate variability on flooding. BCC flood studies are therefore required to utilise the latest statutory guidelines in order to assess the impacts of climate variability.

As part of this climate variability assessment, a number of climate scenarios were modelled, 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 variability 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 variability modifications undertaken.

Table 8.1 – Climate Modelling Scenarios

ARI (year)	AEP (%)	Planning horizon	Rainfall Intensity	Tailwater Condition	Scenario 1	Scenario 3
100	1	2050	+ 10 %	MHWS + 0.3 m = 1.57mAHD	✓	✓
		2100	+ 20 %	MHWS + 0.8 m = 2.07mAHD	✓	✓
200	0.5	2050	+ 10 %	HAT + 0.3 m = 2.17mAHD	✓	×
		2100	+ 20 %	HAT + 0.8 m = 2.67mAHD	✓	×
500	0.2	2100	+ 20 %	HAT + 0.8 m = 2.67mAHD	✓	×

8.2.3 Hydraulic Modelling

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

The URBS 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.

8.2.4 Impacts of Climate Variability

Tables 8.2 to 8.4 indicate a comparison of the peak flood levels for the Scenario 1 climate conditions. The flood level results are provided at selected locations along all 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 variability impacts within the catchment will increase the magnitude of flooding, for example:

- Based on current climatic projections, by the year 2050, the 100-yr ARI (1 % AEP) flood levels are likely to be of similar magnitude to the present day 200-yr ARI (0.5 % AEP) flood levels.
- Based on current climatic projections, by the year 2100, the 100-yr ARI (1 % AEP) flood levels are likely to be between the present day 200-yr ARI (0.2 % AEP) and 500-yr ARI (0.2 % AEP) flood levels.
- Based on current climatic projections, by the year 2100, the 200-yr ARI (0.5 % AEP) flood levels are likely to be of similar magnitude to the present day 500-yr ARI (0.2 % AEP) flood levels.

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

Structure Location	100-yr ARI (1 % AEP)				
	Existing WL (mAHD)	2050		2100	
		WL (mAHD)	Afflux (m)	WL (mAHD)	Afflux (m)
Moggill Creek					
Upper Brookfield Road (S15)	55.51	55.93	0.42	56.29	0.78
Upper Brookfield Road (S12)	42.98	43.27	0.29	43.56	0.58
Brookfield Road (S9)	27.62	27.76	0.14	27.89	0.27
Rafting Ground Road (S7)	23.64	23.81	0.17	23.96	0.32
Rafting Ground Road (S6)	22.03	22.21	0.18	22.39	0.36
Moggill Road (S2)	9.37	9.78	0.41	10.12	0.75
Gold Creek					
Gold Creek Road (S46)	75.44	75.58	0.14	75.72	0.28
Gold Creek Road (S40)	57.70	57.79	0.09	57.90	0.2
Gold Creek Road (S37)	46.53	47.07	0.54	47.50	0.97
Savages Road (S34)	36.08	36.20	0.12	36.32	0.24
Gap Creek					
Gap Creek Road (S31)	37.40	37.49	0.09	37.55	0.15
Brookfield Road (S28)	25.40	25.66	0.26	25.87	0.47
McKay Brook					
Tinarra Crescent (S22)	55.36	55.62	0.26	55.89	0.53
Mirbelia Street (S18)	21.27	21.36	0.09	21.46	0.19
Brookfield Road (S17)	14.66	15.31	0.65	15.65	0.99

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

Structure Location	200-yr ARI (0.5 % AEP)				
	Existing WL (mAHD)	2050		2100	
		WL (mAHD)	Afflux (m)	WL (mAHD)	Afflux (m)
Moggill Creek					
Upper Brookfield Road (S15)	55.95	56.34	0.39	56.65	0.70
Upper Brookfield Road (S12)	43.29	43.61	0.32	44.19	0.90
Brookfield Road (S9)	27.75	27.89	0.14	28.03	0.28
Rafting Ground Road (S7)	23.80	23.96	0.16	24.14	0.34
Rafting Ground Road (S6)	22.20	22.40	0.20	22.60	0.40
Moggill Road (S2)	9.75	10.13	0.38	10.48	0.73
Gold Creek					
Gold Creek Road (S46)	75.57	75.72	0.15	75.89	0.32
Gold Creek Road (S40)	57.78	57.91	0.13	58.03	0.25
Gold Creek Road (S37)	47.05	47.51	0.46	47.77	0.72
Savages Road (S34)	36.19	36.32	0.13	36.43	0.24
Gap Creek					
Gap Creek Road (S31)	37.49	37.56	0.07	37.65	0.16
Brookfield Road (S28)	25.67	25.90	0.23	26.06	0.39
McKay Brook					
Tinarra Crescent (S22)	55.67	55.98	0.31	56.33	0.66
Mirbelia Street (S18)	21.38	21.49	0.11	21.59	0.21
Brookfield Road (S17)	15.33	15.75	0.42	15.98	0.65

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

Structure Location	500-yr ARI (0.2 % AEP)		
	Existing WL (mAHD)	2100	
		WL (mAHD)	Afflux (m)
Moggill Creek			
Upper Brookfield Road (S15)	56.54	57.03	0.49
Upper Brookfield Road (S12)	43.95	44.97	1.02
Brookfield Road (S9)	27.98	28.28	0.3
Rafting Ground Road (S7)	24.08	24.47	0.39
Rafting Ground Road (S6)	22.53	22.97	0.44
Moggill Road (S2)	10.35	11.04	0.69
Gold Creek			
Gold Creek Road (S46)	75.83	76.16	0.33
Gold Creek Road (S40)	57.99	58.25	0.26
Gold Creek Road (S37)	47.69	48.09	0.4
Savages Road (S34)	36.40	36.57	0.17
Gap Creek			
Gap Creek Road (S31)	37.62	37.79	0.17
Brookfield Road (S28)	26.01	26.29	0.28
McKay Brook			
Tinarra Crescent (S22)	56.21	57.05	0.84
Mirbelia Street (S18)	21.55	21.80	0.25
Brookfield Road (S17)	15.93	16.18	0.25

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9.0 Summary of Study Findings

This flood study report details the calibration and verification, design event, extreme event and sensitivity modelling for the Moggill Creek Catchment, including Moggill Creek, Gold Creek, Gap Creek and McKay Brook. New hydrologic and hydraulic models have been developed for the study using the URBS and TUFLOW modelling software, respectively.

Hydrometric information was sourced from the available recorded rainfall, stream gauge and reservoir data. Calibration of the URBS and TUFLOW models was undertaken for the May 2015, May 2009 and November 2008 events. Verification of the URBS and TUFLOW models was undertaken for the January 2013 event.

The results of the hydraulic calibration and verification indicated that the URBS 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 URBS 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 (50% 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 “Modelled Flood 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
- Peak flood extent mapping (Scenario 1 only)
- Hydraulic structure flood immunity data

As part of the required sensitivity analysis a climate variability 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 variability impacts within the catchment will increase the magnitude of flooding, for example:

- Based on current climatic projections, by the year 2050, the 100-yr ARI (1 % AEP) flood levels are likely to be of similar magnitude to the present day 200-yr ARI (0.5 % AEP) flood levels.
- Based on current climatic projections, by the year 2100, the 100-yr ARI (1 % AEP) flood levels are likely to be between the present day 200-yr ARI (0.2 % AEP) and 500-yr ARI (0.2 % AEP) flood levels.
- Based on current climatic projections, by the year 2100, the 200-yr ARI (0.5 % AEP) flood levels are likely to be of similar magnitude to the present day 500-yr ARI (0.2 % AEP) flood levels.

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.

APPENDICES

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Appendix A: Rainfall Distribution

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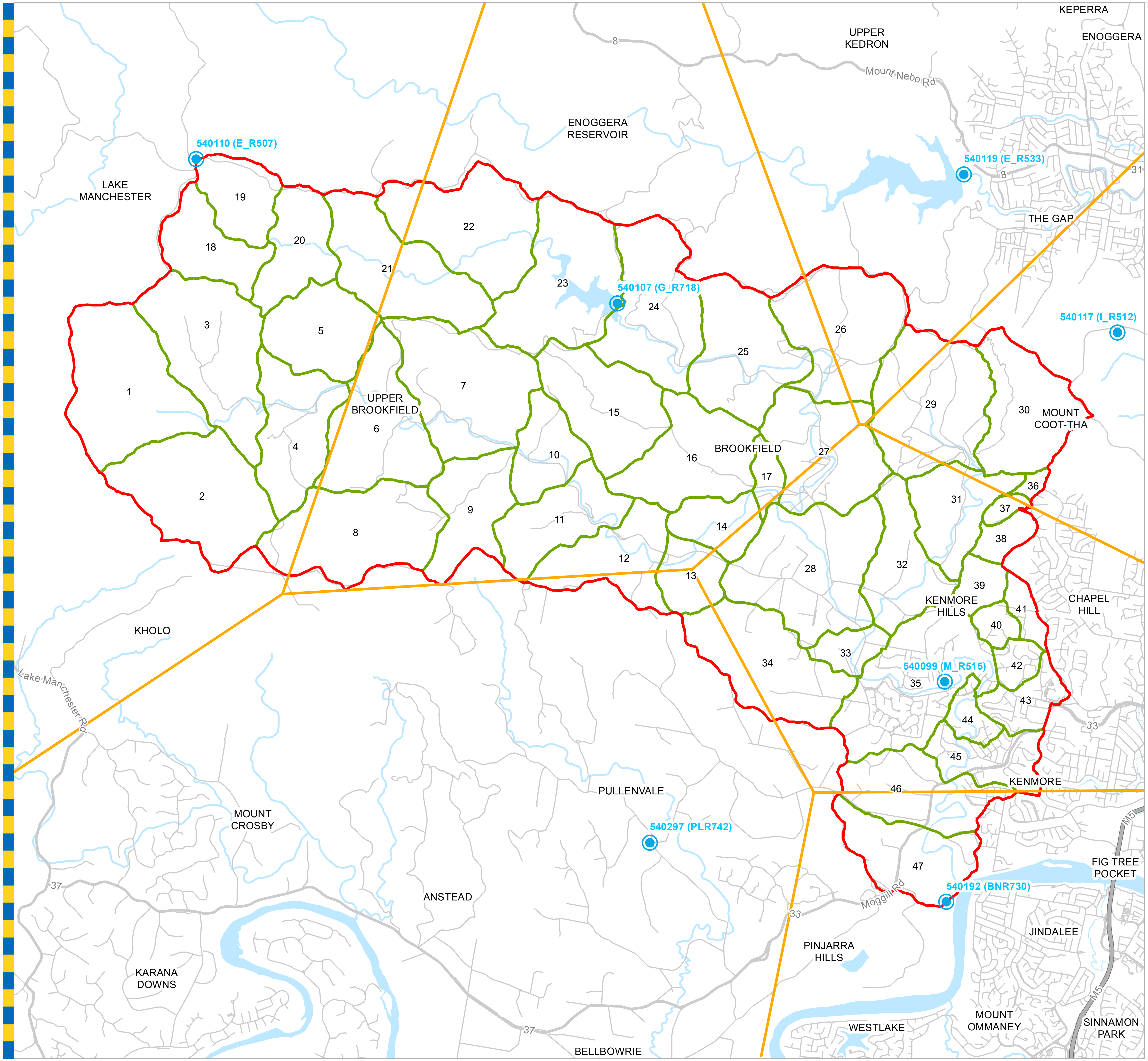

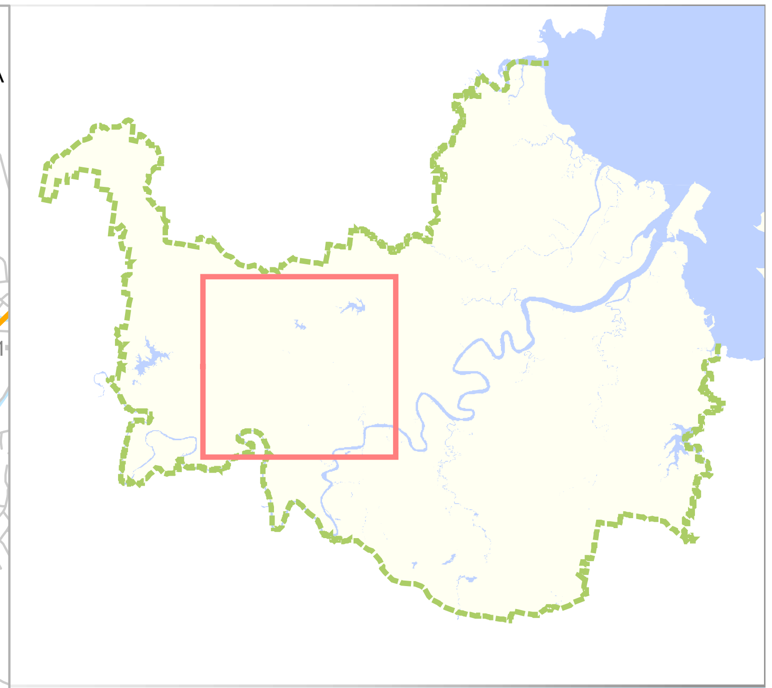

Legend

- Pluviograph Stations
- Freeways/Highways
- Major Roads
- Streets
- Rainfall Distribution
- Catchment Area
- URBS Subcatchments (1 - 47)

N W E S

0 520 1,040 1,560
Metres

Prepared : 099858
Checked : NC
Revision : 0
Publication Date : 23 Jun 2016
Project Number : 160277

Location: G:\B\CD\Proj16\160277_Moggill_Creek_Flood_Study\ArcGIS\GIM_160277_004.mxd

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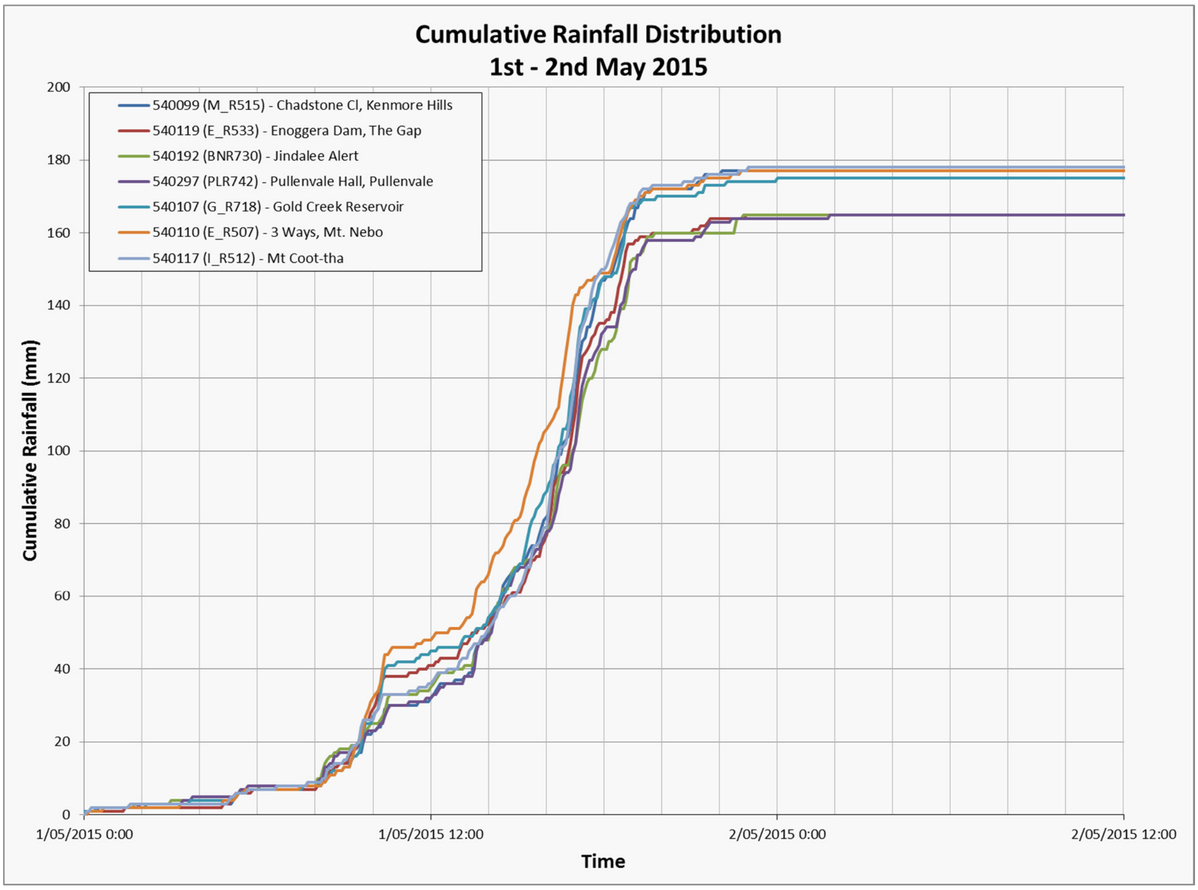
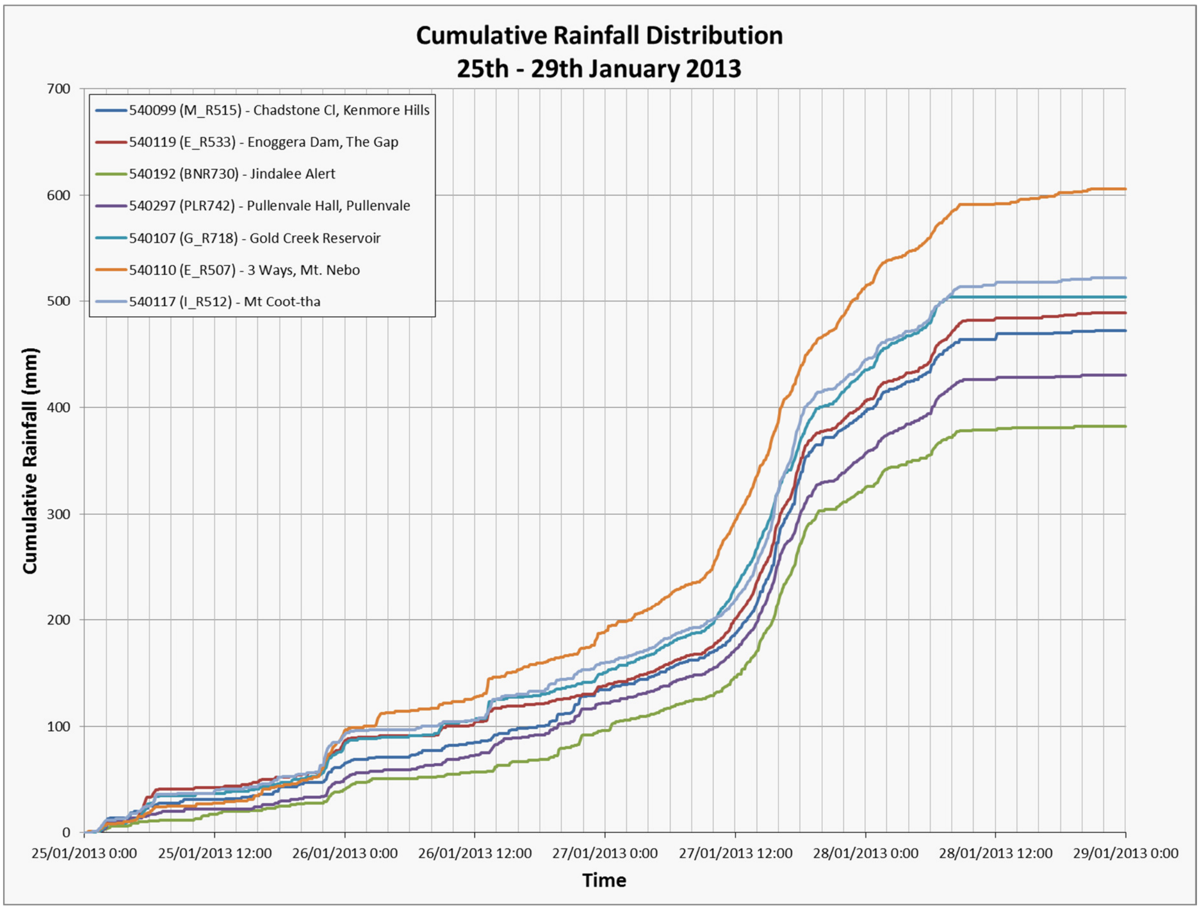
Figure A: Thiessen Polygon May 2015, January 2013, May 2009 and November 2008

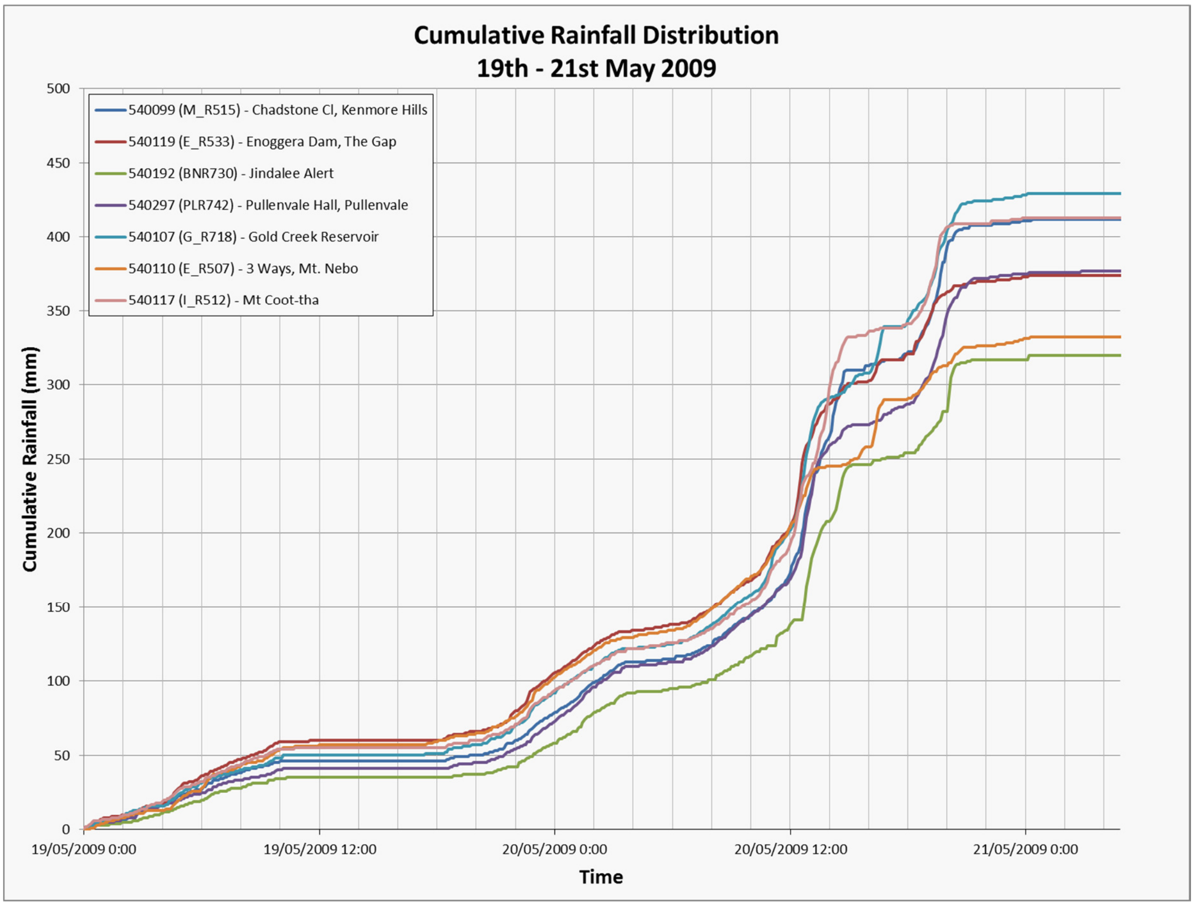
GIM - 160277 - 004



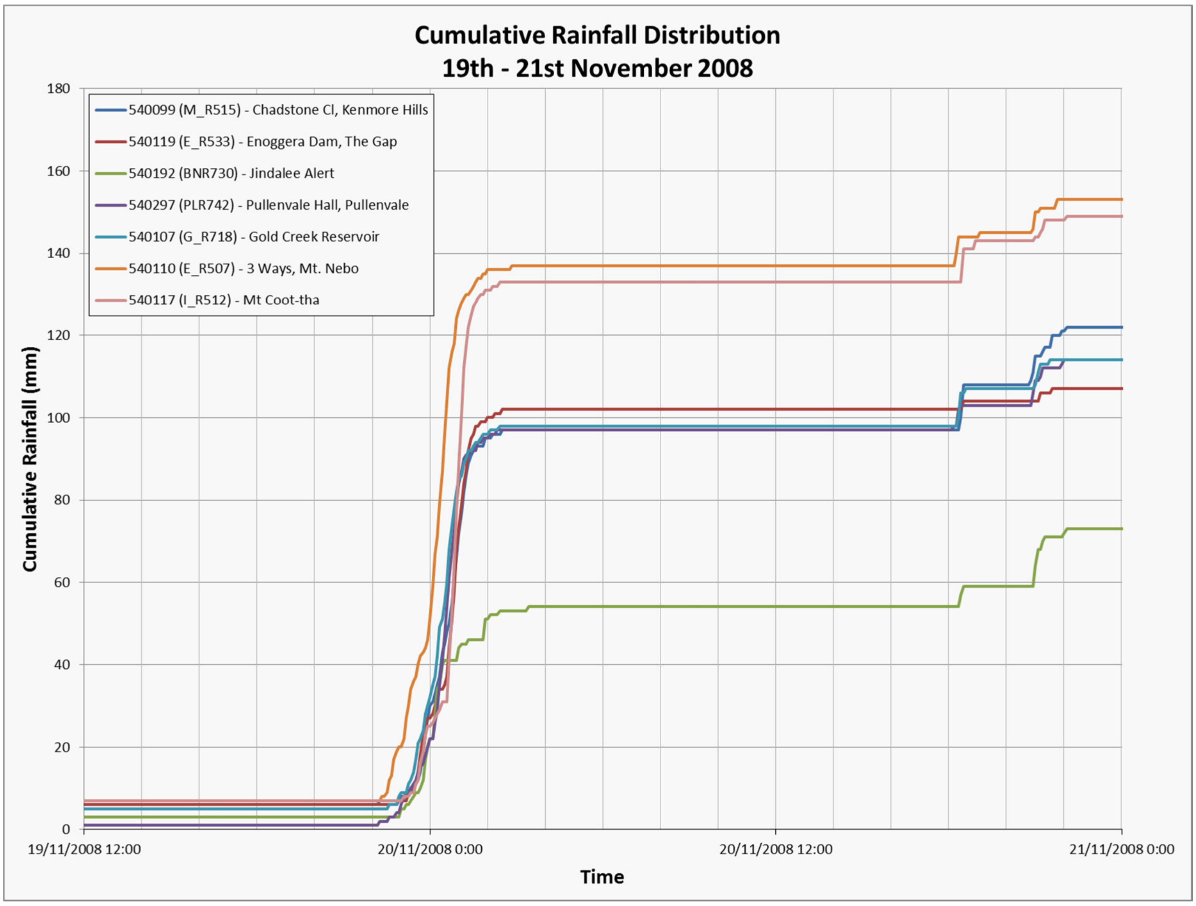
Cumulative Rainfall Distribution
1st - 2nd May 2015
540099 (M_R515) - Chadstone Cl, Kenmore Hills
540119 (E_R533) - Enoggera Dam, The Gap
540192 (BNR730) - Jindalee Alert
540297 (PLR742) - Pullenvale Hall, Pullenvale
540107 (G_R718) - Gold Creek Reservoir
540110 (E_R507) - 3 Ways, Mt. Nebo
540117 (I_R512) - Mt Coot-tha
Cumulative Rainfall (mm)
0
20
40
60
80
100
120
140
160
180
200
1/05/2015 0:00
1/05/2015 12:00
2/05/2015 0:00
2/05/2015 12:00
Time



Cumulative Rainfall Distribution
25th - 29th January 2013
540099 (M_R515) - Chadstone Cl, Kenmore Hills
540119 (E_R533) - Enoggera Dam, The Gap
540192 (BNR730) - Jindalee Alert
540297 (PLR742) - Pullenvale Hall, Pullenvale
540107 (G_R718) - Gold Creek Reservoir
540110 (E_R507) - 3 Ways, Mt. Nebo
540117 (I_R512) - Mt Coot-tha
Cumulative Rainfall (mm)
0
100
200
300
400
500
600
700
25/01/2013 0:00
25/01/2013 12:00
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
28/01/2013 12:00
29/01/2013 0:00
Time



Cumulative Rainfall Distribution
19th - 21st May 2009
540099 (M_R515) - Chadstone Cl, Kenmore Hills
540119 (E_R533) - Enoggera Dam, The Gap
540192 (BNR730) - Jindalee Alert
540297 (PLR742) - Pullenvale Hall, Pullenvale
540107 (G_R718) - Gold Creek Reservoir
540110 (E_R507) - 3 Ways, Mt. Nebo
540117 (I_R512) - Mt Coot-tha
Cumulative Rainfall (mm)
500
450
400
350
300
250
200
150
100
50
0
19/05/2009 0:00
19/05/2009 12:00
20/05/2009 0:00
20/05/2009 12:00
21/05/2009 0:00
Time



Cumulative Rainfall Distribution
19th - 21st November 2008
540099 (M_R515) - Chadstone Cl, Kenmore Hills
540119 (E_R533) - Enoggera Dam, The Gap
540192 (BNR730) - Jindalee Alert
540297 (PLR742) - Pullenvale Hall, Pullenvale
540107 (G_R718) - Gold Creek Reservoir
540110 (E_R507) - 3 Ways, Mt. Nebo
540117 (I_R512) - Mt Coot-tha
Cumulative Rainfall (mm)
180
160
140
120
100
80
60
40
20
0
19/11/2008 12:00
20/11/2008 0:00
20/11/2008 12:00
21/11/2008 0:00
Time

Appendix B: URBS Model Parameters

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URBS Calibration / Verification Event Sub-catchment Parameters

Subcatchment	Area (km^2)	Imp (%)	UL	UM	UH	UR	CS
1	2.539	0.0	0.000	0.000	0.000	1.000	0.087
2	2.399	0.0	0.000	0.000	0.000	1.000	0.210
3	2.394	0.0	0.000	0.000	0.000	1.000	0.067
4	1.601	0.0	0.000	0.000	0.000	1.000	0.153
5	1.382	0.0	0.000	0.000	0.000	1.000	0.152
6	2.005	0.0	0.000	0.000	0.000	1.000	0.066
7	2.466	0.0	0.000	0.000	0.000	1.000	0.064
8	2.257	0.0	0.000	0.000	0.000	1.000	0.122
9	1.271	0.0	0.000	0.000	0.000	1.000	0.086
10	0.913	1.0	0.000	0.000	0.011	0.989	0.069
11	1.314	1.0	0.000	0.000	0.011	0.989	0.073
12	1.384	1.0	0.000	0.000	0.011	0.989	0.090
13	0.900	2.0	0.000	0.000	0.022	0.978	0.056
14	0.520	0.0	0.000	0.000	0.000	1.000	0.053
15	1.783	0.0	0.000	0.000	0.000	1.000	0.047
16	1.776	1.0	0.000	0.000	0.011	0.989	0.060
17	0.301	2.0	0.000	0.000	0.022	0.978	0.135
18	1.070	0.0	0.000	0.000	0.000	1.000	0.092
19	0.931	0.0	0.000	0.000	0.000	1.000	0.126
20	1.145	0.0	0.000	0.000	0.000	1.000	0.118
21	2.066	0.0	0.000	0.000	0.000	1.000	0.041
22	2.174	0.0	0.000	0.000	0.000	1.000	0.076
23	2.341	0.0	0.000	0.000	0.000	1.000	0.079
24	2.109	0.0	0.000	0.000	0.000	1.000	0.071
25	1.958	1.0	0.000	0.000	0.011	0.989	0.122
26	2.182	0.0	0.000	0.000	0.000	1.000	0.039
27	1.918	8.0	0.000	0.123	0.021	0.857	0.024
28	2.611	10.0	0.494	0.017	0.019	0.470	0.044
29	2.383	0.0	0.000	0.000	0.000	1.000	0.039
30	1.701	0.0	0.000	0.000	0.000	1.000	0.227
31	1.218	3.0	0.101	0.000	0.016	0.883	0.020
32	1.302	8.0	0.455	0.000	0.013	0.532	0.044
33	0.429	10.0	0.527	0.000	0.023	0.450	0.047

Subcatchment	Area (km^2)	Imp (%)	UL	UM	UH	UR	CS
34	2.068	8.0	0.499	0.000	0.006	0.496	0.194
35	2.446	30.0	0.515	0.348	0.054	0.083	0.020
36	0.103	5.0	0.097	0.000	0.039	0.864	0.060
37	0.150	6.0	0.160	0.000	0.040	0.800	0.150
38	0.337	5.0	0.309	0.000	0.004	0.687	0.037
39	0.396	5.0	0.308	0.000	0.004	0.688	0.069
40	0.261	20.0	0.383	0.257	0.015	0.345	0.057
41	0.289	10.0	0.457	0.000	0.035	0.509	0.058
42	0.308	48.1	0.000	0.377	0.325	0.299	0.063
43	0.613	55.1	0.000	0.604	0.277	0.119	0.054
44	0.342	34.9	0.000	0.594	0.058	0.348	0.037
45	0.891	40.0	0.202	0.456	0.157	0.185	0.045
46	1.531	22.0	0.607	0.128	0.072	0.193	0.029
47	1.435	15.0	0.425	0.122	0.028	0.425	0.029

URBS Design Event Sub-catchment Parameters

Subcatchment	Area (km^2)	Imp (%)	UL	UM	UH	UR	CS
1	2.539	0.0	0.000	0.000	0.000	1.000	0.087
2	2.399	0.0	0.000	0.000	0.000	1.000	0.210
3	2.394	0.0	0.000	0.000	0.000	1.000	0.067
4	1.601	0.0	0.000	0.000	0.000	1.000	0.153
5	1.382	0.0	0.000	0.000	0.000	1.000	0.152
6	2.005	0.0	0.000	0.000	0.000	1.000	0.066
7	2.466	0.0	0.000	0.000	0.000	1.000	0.064
8	2.257	0.0	0.000	0.000	0.000	1.000	0.122
9	1.271	0.0	0.000	0.000	0.000	1.000	0.086
10	0.913	1.0	0.000	0.000	0.011	0.989	0.069
11	1.314	1.0	0.000	0.000	0.011	0.989	0.073
12	1.384	1.0	0.000	0.000	0.011	0.989	0.090
13	0.900	2.0	0.000	0.000	0.022	0.978	0.056
14	0.520	0.0	0.000	0.000	0.000	1.000	0.053
15	1.783	0.0	0.000	0.000	0.000	1.000	0.047
16	1.776	1.0	0.000	0.000	0.011	0.989	0.060
17	0.301	2.0	0.000	0.000	0.022	0.978	0.135
18	1.070	0.0	0.000	0.000	0.000	1.000	0.092
19	0.931	0.0	0.000	0.000	0.000	1.000	0.126
20	1.145	0.0	0.000	0.000	0.000	1.000	0.118
21	2.066	0.0	0.000	0.000	0.000	1.000	0.041
22	2.174	0.0	0.000	0.000	0.000	1.000	0.076
23	2.341	0.0	0.000	0.000	0.000	1.000	0.079
24	2.109	0.0	0.000	0.000	0.000	1.000	0.071
25	1.958	1.0	0.000	0.000	0.011	0.989	0.122
26	2.182	0.0	0.000	0.000	0.000	1.000	0.039
27	1.918	12.0	0.357	0.095	0.021	0.527	0.024
28	2.611	15.0	0.762	0.037	0.019	0.182	0.044
29	2.383	0.0	0.000	0.000	0.000	1.000	0.039
30	1.701	0.0	0.000	0.000	0.000	1.000	0.227
31	1.218	3.0	0.101	0.000	0.016	0.883	0.020
32	1.302	9.0	0.522	0.000	0.013	0.465	0.044

Subcatchment	Area (km^2)	Imp (%)	UL	UM	UH	UR	CS
33	0.429	16.0	0.927	0.000	0.023	0.050	0.047
34	2.068	10.0	0.632	0.000	0.006	0.362	0.194
35	2.446	30.0	0.462	0.363	0.054	0.120	0.020
36	0.103	5.0	0.097	0.000	0.039	0.864	0.060
37	0.150	8.0	0.293	0.000	0.040	0.667	0.150
38	0.337	5.0	0.309	0.000	0.004	0.687	0.037
39	0.396	5.0	0.308	0.000	0.004	0.688	0.069
40	0.261	26.0	0.551	0.327	0.015	0.107	0.057
41	0.289	12.0	0.592	0.000	0.035	0.373	0.058
42	0.308	48.1	0.000	0.377	0.325	0.299	0.063
43	0.613	55.1	0.000	0.604	0.277	0.119	0.054
44	0.342	39.0	0.000	0.675	0.058	0.267	0.037
45	0.891	40.0	0.202	0.456	0.157	0.185	0.045
46	1.531	22.0	0.607	0.128	0.072	0.193	0.029
47	1.435	31.0	0.425	0.442	0.028	0.105	0.029

Gold Creek Reservoir: Stage - Storage - Discharge Relationship (with outlet pipe open)

Stage (mAHD)	Storage (ML)	Discharge (m^3/s)	Stage (mAHD)	Storage (ML)	Discharge (m^3/s)	Stage (mAHD)	Storage (ML)	Discharge (m^3/s)
79.2	3	0.00	87.2	267	0.00	95.2	1307	3.46
79.4	10	0.00	87.4	273	0.00	95.4	1343	3.62
79.6	16	0.00	87.6	279	0.00	95.6	1379	3.78
79.8	23	0.00	87.8	285	0.00	95.8	1417	5.72
80.0	30	0.00	88.0	292	0.00	96.0	1460	13.00
80.2	37	0.00	88.2	301	0.00	96.2	1501	22.60
80.4	43	0.00	88.4	313	0.00	96.4	1550	37.60
80.6	49	0.00	88.6	326	0.00	96.6	1604	56.00
80.8	55	0.00	88.8	340	0.00	96.8	1659	76.80
81.0	62	0.00	89.0	355	0.00	97.0	1713	100.00
81.2	69	0.00	89.2	370	0.00	97.2	1767	126.40
81.4	76	0.00	89.4	384	0.00	97.4	1821	154.00
81.6	82	0.00	89.6	398	0.00	97.6	1875	183.60
81.8	89	0.00	89.8	412	0.00	97.8	1929	215.20
82.0	95	0.00	90.0	426	0.00	98.0	1984	248.00
82.2	102	0.00	90.2	442	0.00	98.2	2043	283.20
82.4	109	0.00	90.4	461	0.00	98.4	2105	320.20
82.6	115	0.00	90.6	484	0.00	98.6	2169	358.60
82.8	122	0.00	90.8	510	0.00	98.8	2232	398.40
83.0	128	0.00	91.0	538	0.00	99.0	2295	440.00
83.2	135	0.00	91.2	565	0.00	99.2	2357	484.00
83.4	142	0.00	91.4	591	0.00	99.4	2420	529.20
83.6	148	0.00	91.6	617	0.00	99.6	2483	575.60
83.8	155	0.00	91.8	643	0.00	99.8	2547	623.40
84.0	161	0.00	92.0	670	0.00	100.0	2614	673.00
84.2	167	0.00	92.2	700	0.00	100.2	2686	724.20
84.4	174	0.00	92.4	734	0.00	100.4	2759	777.20
84.6	180	0.00	92.6	771	0.00	100.6	2831	831.60
84.8	187	0.00	92.8	812	0.02	100.8	2903	887.20
85.0	194	0.00	93.0	857	0.11	101.0	2975	944.00
85.2	200	0.00	93.2	902	0.53	101.2	3053	1003.20
85.4	207	0.00	93.4	946	0.92	101.4	3132	1063.00
85.6	213	0.00	93.6	990	1.30	101.6	3210	1124.60
85.8	220	0.00	93.8	1035	1.68	101.8	3287	1188.00
86.0	227	0.00	94.0	1079	2.00	102.0	3366	1252.00
86.2	233	0.00	94.2	1121	2.32	102.2	3450	1312.80
86.4	240	0.00	94.4	1160	2.58	102.4	3535	1374.20
86.6	246	0.00	94.6	1198	2.82	102.6	3619	1436.60
86.8	253	0.00	94.8	1234	3.06	102.8	3704	1500.00
87.0	260	0.00	95.0	1270	3.30	103.0	3788	1564.00

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Appendix C: Adopted Land-use

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Figure C-1: BCC City Plan 2014 Zones

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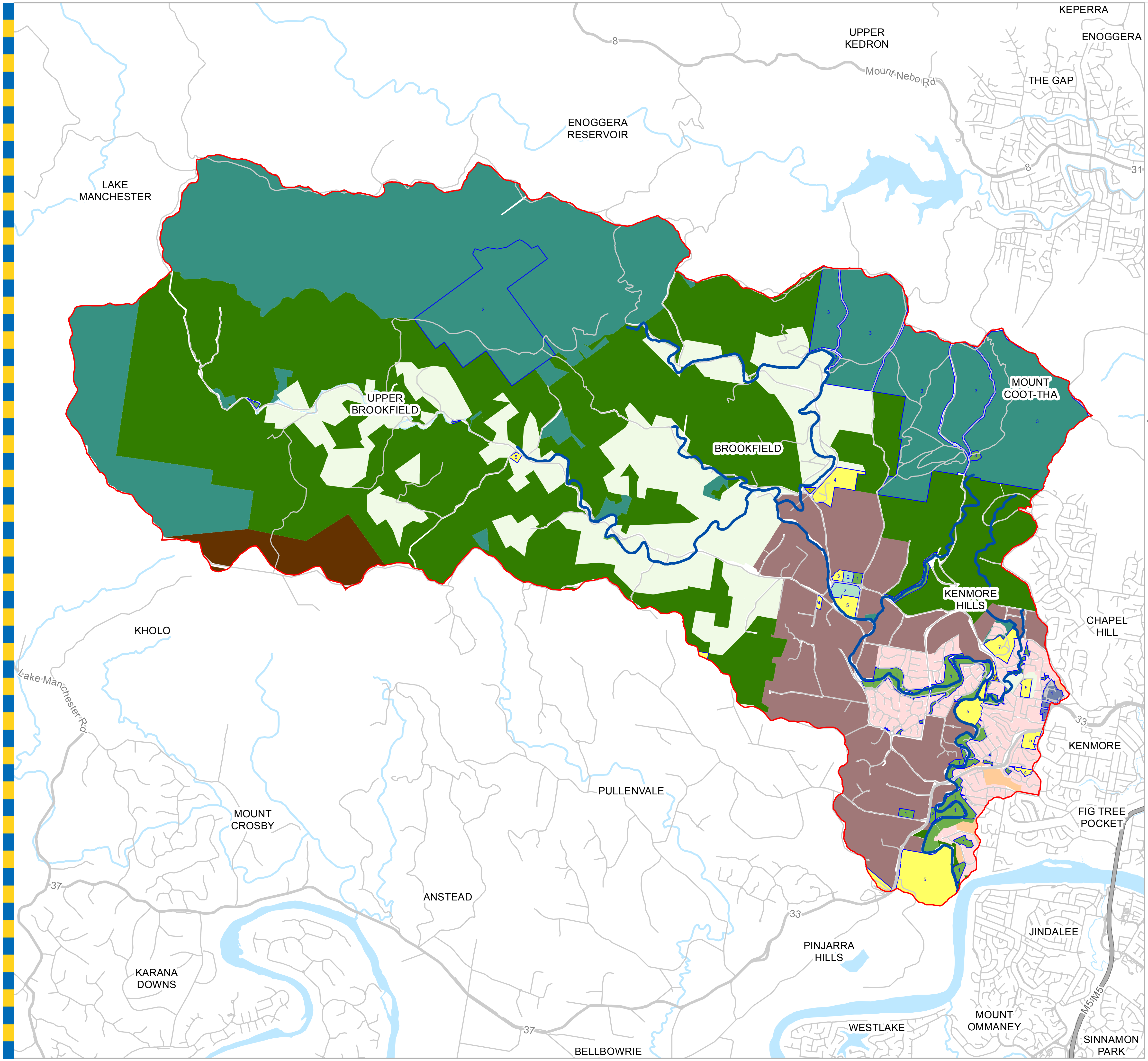

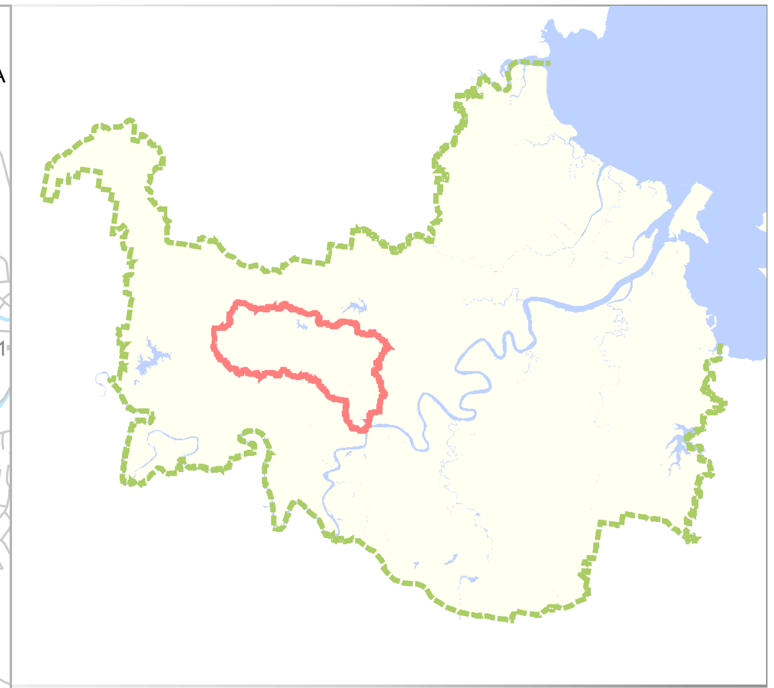

Legend

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

- LDR Low density residential
- DC1 District centre (District)
- SR2 Sport and recreation (District)
- OS Open space
- OS1 Open space (Local)
- OS2 Open space (District)
- OS3 Open space (Metropolitan)
- EM Environmental management
- CN Conservation
- CN2 Conservation (District)
- CN3 Conservation (Metropolitan)
- EC Emerging community
- EI Extractive industry
- RR Rural residential
- RU Rural
- CF3 Cemetery
- CF4 Community purposes
- CF5 Education purposes
- CF7 Health care purposes
- SP4 Utility services

0 520 1,040 1,560
Metres

Prepared : 099858
Checked : NC
Revision : 0
Publication Date : 23 Jun 2016
Project Number : 160277

Location: G:\BI\CD\Proj16\160277_Moggill_Creek_Flood_Study\ArcGIS\GIM_160277_006.mxd

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

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Moggill Creek Flood Study

Figure C - 1: 2014 City Plan Zones

GIM - 160277 - 006

Figure C-2: 2015 Aerial Photo

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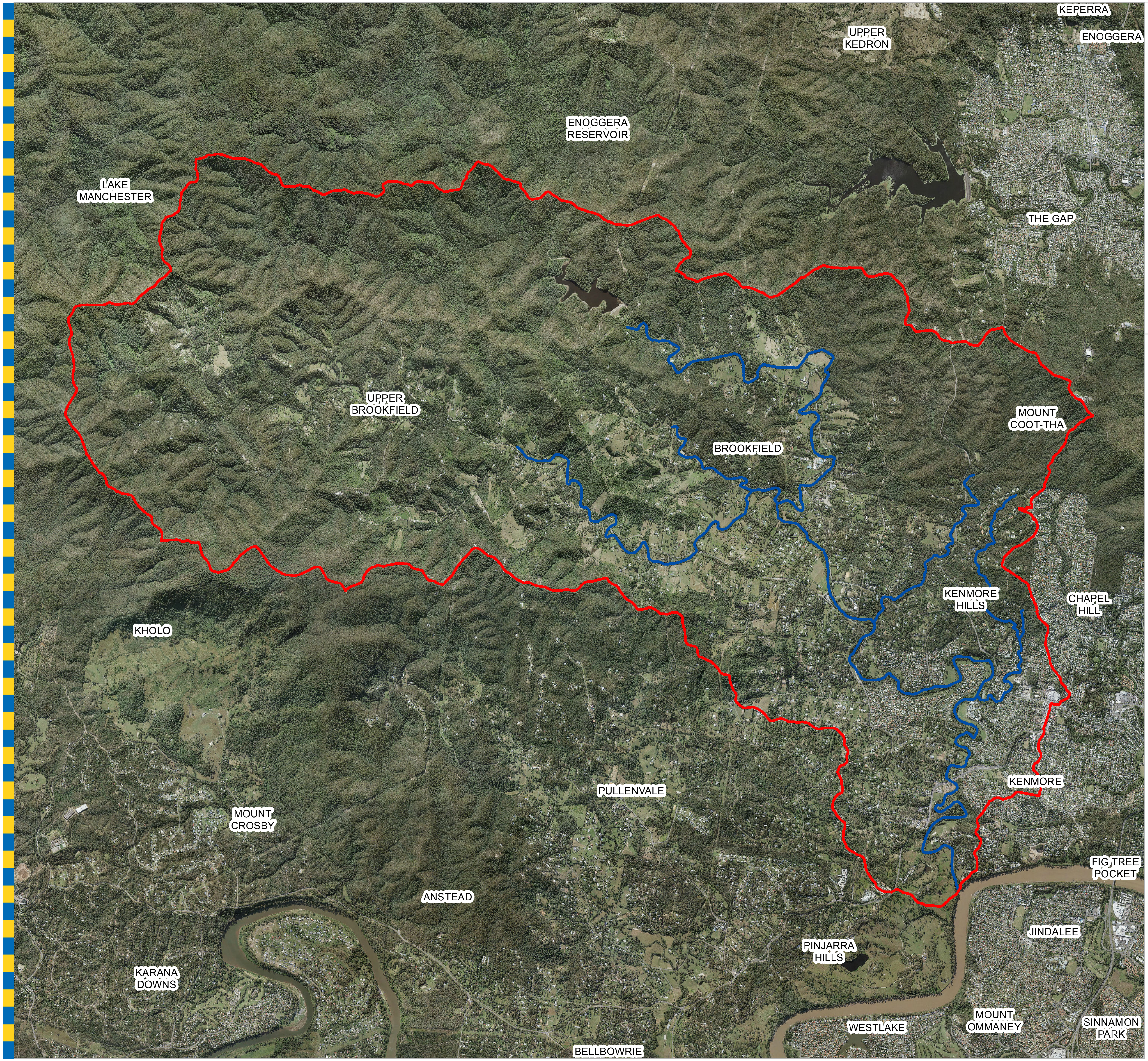

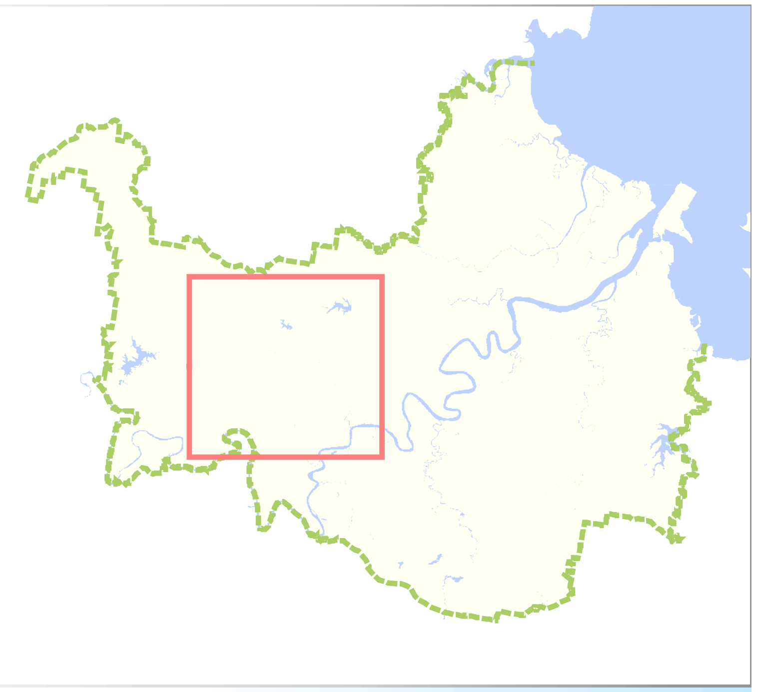

Legend

Catchment Area

Creek Centreline

0 520 1,040 1,560

Metres

Prepared : 099858
Checked : NC
Revision : 0
Publication Date : 23 Jun 2016
Project Number : 160277

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

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Moggill Creek Flood Study

Figure C - 2:
2015 Aerial Photo

GIM - 160277 - 007

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	5
Mixed use (Inner city)	90
Mixed use (Centre frame)	90
Mixed use (Corridor)	90
Rural	5
Rural residential	15
Township	80
Community facilities (Major health care)	70
Community facilities (Major sports venue)	60
Community facilities (Cemetery)	40
Community facilities (Community purposes)	50
Community facilities (Education purposes)	50
Community facilities (Emergency services)	70
Community facilities (Health care purposes)	50
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

The flood level data presented in this Appendix has been extracted (in part) from the results of a 2-dimensional flood model. Levels presented have been extracted generally at selected points along the centreline of the waterway with the intent of demonstrating general flood characteristics. The applicability of this data to locations on the floodplains adjacent should be determined by a suitably qualified professional. It is recommended for any detailed assessment of flood risk associated with the waterway that complete flood model results be accessed and interrogated.

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AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
Moggill Creek						
0	1.27	1.29	1.29	1.50	1.53	1.55
100	1.29	1.35	1.43	1.68	2.23	2.55
200	1.44	1.70	1.93	2.29	2.87	3.24
300	1.53	1.89	2.15	2.52	3.11	3.49
400	1.64	2.07	2.35	2.73	3.28	3.66
500	1.68	2.10	2.36	2.71	3.23	3.61
600	1.84	2.43	2.73	3.14	3.72	4.26
700	2.19	2.97	3.33	3.80	4.41	4.83
800	2.31	3.09	3.45	3.90	4.47	4.90
900	2.43	3.15	3.49	3.91	4.52	4.99
1000	2.59	3.39	3.78	4.29	4.91	5.29
1100	2.64	3.43	3.82	4.33	4.92	5.29
1200	2.72	3.51	3.90	4.41	4.99	5.34
1300	2.92	3.72	4.09	4.57	5.14	5.50
1400	3.05	3.92	4.25	4.73	5.31	5.66
1500	3.11	4.05	4.44	4.98	5.52	5.86
1600	3.33	4.20	4.57	5.10	5.62	5.97
1700	3.47	4.29	4.67	5.18	5.72	6.07
1800	3.64	4.47	4.85	5.38	5.93	6.30
1900	3.84	4.67	5.05	5.56	6.07	6.42
2000	4.09	4.99	5.38	5.86	6.37	6.70
2100	4.30	5.15	5.52	5.98	6.47	6.79
2200	4.38	5.21	5.61	6.13	6.70	7.08
2300	4.53	5.41	5.81	6.31	6.84	7.19
2400	4.76	5.53	5.88	6.33	6.85	7.19
2500	4.88	5.59	5.91	6.34	6.85	7.19
2600	4.91	5.60	5.91	6.34	6.85	7.19
2700	4.96	5.62	5.92	6.33	6.83	7.17
2800	4.99	5.64	5.93	6.34	6.84	7.20
2900	5.18	5.89	6.23	6.71	7.28	7.74
Moggill Road (S1 & S2)						
3020	5.61	6.57	7.04	7.91	8.73	9.39

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
3100	6.00	6.97	7.41	8.14	8.91	9.53
3200	6.23	7.18	7.59	8.26	8.99	9.60
3300	6.44	7.36	7.77	8.42	9.15	9.76
3400	6.65	7.55	7.95	8.57	9.31	9.92
3500	6.79	7.69	8.08	8.69	9.41	10.00
3600	6.94	7.83	8.22	8.82	9.52	10.07
3700	7.23	8.07	8.44	8.99	9.65	10.18
3800	7.45	8.28	8.66	9.17	9.79	10.30
3900	7.87	8.59	8.90	9.31	9.86	10.37
4000	8.08	8.78	9.07	9.44	9.95	10.42
4100	8.37	9.11	9.43	9.82	10.34	10.80
4200	8.71	9.49	9.84	10.27	10.81	11.24
4300	9.04	9.83	10.19	10.65	11.20	11.60
4400	9.29	9.97	10.32	10.76	11.30	11.70
4500	9.54	10.28	10.65	11.06	11.56	11.93
4600	9.76	10.55	10.94	11.34	11.80	12.14
4700	9.94	10.78	11.18	11.59	12.01	12.30
4800	10.14	10.99	11.41	11.83	12.24	12.53
4900	10.33	11.21	11.63	12.07	12.48	12.76
5000	10.53	11.42	11.85	12.31	12.73	13.00
5100	11.19	12.04	12.45	12.89	13.34	13.63
5200	11.59	12.41	12.77	13.19	13.63	13.93
5300	11.75	12.54	12.90	13.32	13.76	14.06
Branton Street Footbridge (S4)						
5400	11.95	12.71	13.06	13.47	13.91	14.21
5500	12.14	12.93	13.28	13.69	14.11	14.40
5600	12.34	13.16	13.53	13.95	14.38	14.67
5700	12.54	13.39	13.79	14.24	14.70	15.00
5800	12.74	13.62	14.05	14.52	15.01	15.32
5900	13.02	13.89	14.31	14.78	15.28	15.59
6000	13.35	14.20	14.58	15.03	15.52	15.84
6100	13.67	14.50	14.84	15.28	15.76	16.09
6200	14.01	14.74	15.05	15.44	15.90	16.22

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
6300	14.37	15.01	15.29	15.63	16.07	16.38
6400	14.86	15.47	15.73	16.08	16.50	16.78
6500	15.36	15.94	16.19	16.53	16.93	17.19
6600	15.69	16.26	16.50	16.83	17.21	17.47
6700	15.97	16.54	16.78	17.09	17.46	17.71
6800	16.24	16.82	17.06	17.38	17.75	18.00
6900	16.51	17.11	17.35	17.67	18.04	18.29
7000	16.81	17.41	17.66	17.99	18.36	18.62
7100	17.12	17.72	17.98	18.32	18.71	18.97
7200	17.43	18.03	18.31	18.66	19.05	19.33
7300	17.65	18.27	18.57	18.93	19.35	19.63
7400	17.88	18.54	18.85	19.21	19.61	19.88
7500	18.20	18.86	19.18	19.54	19.91	20.17
7600	18.60	19.26	19.56	19.91	20.27	20.52
7700	18.97	19.60	19.88	20.21	20.55	20.79
7800	19.33	19.90	20.16	20.47	20.79	21.01
7900	19.69	20.21	20.44	20.73	21.02	21.22
8000	19.89	20.41	20.65	20.95	21.25	21.44
Rafting Ground Road #1 (S6)						
8145	20.67	21.07	21.29	21.57	21.85	22.08
8200	20.79	21.24	21.48	21.76	22.05	22.27
8300	21.01	21.50	21.76	22.06	22.34	22.55
8400	21.23	21.72	21.98	22.28	22.56	22.77
8500	21.42	21.94	22.19	22.49	22.79	23.00
8595	21.60	22.14	22.39	22.70	23.01	23.23
Rafting Ground Road #2 (S7)						
8700	22.15	22.77	23.08	23.39	23.76	24.02
8800	22.39	23.06	23.40	23.79	24.21	24.49
8900	22.53	23.23	23.59	23.98	24.41	24.70
9000	22.84	23.46	23.77	24.12	24.52	24.78
9100	22.96	23.53	23.88	24.22	24.59	24.84
9200	23.57	24.07	24.38	24.72	25.07	25.29
9300	24.05	24.54	24.83	25.17	25.50	25.69

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
9400	24.30	24.86	25.16	25.48	25.77	25.94
9500	24.57	25.20	25.50	25.80	26.06	26.21
9600	25.05	25.68	25.99	26.31	26.57	26.71
Brookfield Road (S9)						
9700	25.24	26.23	26.73	27.15	27.45	27.63
9800	25.39	26.34	26.83	27.24	27.54	27.72
9900	25.63	26.53	26.97	27.36	27.66	27.85
10000	25.94	26.76	27.14	27.51	27.80	27.99
10100	26.25	26.99	27.30	27.65	27.95	28.14
10200	26.56	27.23	27.50	27.83	28.13	28.33
10300	26.87	27.49	27.76	28.09	28.38	28.58
10400	27.14	27.73	28.00	28.33	28.62	28.83
10500	27.33	27.92	28.20	28.54	28.85	29.08
10600	27.52	28.12	28.40	28.75	29.09	29.34
10700	27.73	28.33	28.61	28.97	29.35	29.61
10800	28.07	28.67	28.97	29.35	29.76	30.03
10900	28.40	29.01	29.33	29.73	30.17	30.46
11000	29.49	29.98	30.23	30.55	30.92	31.20
11100	30.08	30.55	30.80	31.11	31.52	31.83
Bundeleer Road (S10)						
11200	31.46	31.75	31.89	32.11	32.45	32.76
11300	31.77	32.20	32.43	32.75	33.11	33.38
11400	32.01	32.51	32.79	33.16	33.55	33.82
11500	32.25	32.79	33.10	33.51	33.92	34.20
11600	32.63	33.15	33.43	33.81	34.21	34.48
11700	33.01	33.56	33.85	34.23	34.63	34.89
11800	33.40	33.98	34.29	34.66	35.06	35.31
11900	33.98	34.53	34.82	35.18	35.57	35.82
12000	34.56	35.07	35.34	35.69	36.07	36.32
12100	35.10	35.58	35.86	36.19	36.55	36.78
12200	35.62	36.07	36.36	36.68	37.02	37.22
12300	36.18	36.64	36.93	37.26	37.60	37.80
12400	36.77	37.27	37.57	37.92	38.28	38.50

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
12500	37.40	37.93	38.24	38.62	39.01	39.25
12600	38.26	38.90	39.19	39.51	39.87	40.09
12700	38.56	39.19	39.48	39.79	40.14	40.35
12800	38.87	39.48	39.75	40.06	40.42	40.64
185 Upper Brookfield Road (S11)						
12907	39.65	40.44	40.69	40.98	41.33	41.55
13000	40.41	40.98	41.22	41.51	41.85	42.08
Upper Brookfield Road #1 (S12)						
13100	41.07	41.67	41.96	42.31	42.74	43.07
13200	41.46	42.02	42.28	42.60	43.01	43.32
13300	41.90	42.42	42.67	42.98	43.36	43.66
13400	42.42	42.91	43.17	43.47	43.84	44.12
13500	43.26	43.77	44.06	44.37	44.69	44.93
Haven Road (S13)						
13600	44.56	44.95	45.16	45.44	45.81	46.06
13700	45.03	45.43	45.66	45.92	46.27	46.52
13800	45.51	45.91	46.16	46.41	46.72	46.98
13900	46.10	46.59	46.87	47.16	47.50	47.77
14000	46.59	47.20	47.54	47.89	48.29	48.58
14100	47.00	47.57	47.92	48.27	48.69	48.99
14200	47.70	48.27	48.61	48.98	49.44	49.77
14300	48.43	48.99	49.33	49.71	50.20	50.56
14400	49.33	49.88	50.22	50.59	51.08	51.44
14500	50.29	50.84	51.18	51.56	52.07	52.44
14600	50.85	51.41	51.76	52.29	53.16	53.59
14700	51.40	51.93	52.27	52.73	53.53	53.97
Upper Brookfield Road #2 (S15)						
14800	52.52	53.15	53.51	53.97	54.66	55.54
14900	52.93	53.52	53.86	54.27	54.88	55.64
15000	54.08	54.55	54.85	55.20	55.66	56.15
15100	54.88	55.30	55.57	55.88	56.24	56.60
15200	55.42	55.86	56.14	56.45	56.81	57.15
15300	55.88	56.40	56.71	57.06	57.48	57.84

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
15400	56.34	56.93	57.26	57.64	58.12	58.50
15500	56.66	57.27	57.62	58.00	58.48	58.84
15600	57.27	57.83	58.15	58.51	58.96	59.29
15700	58.04	58.50	58.77	59.08	59.48	59.76
15800	59.05	59.49	59.73	60.01	60.38	60.64
15900	60.07	60.48	60.70	60.95	61.29	61.53
16000	61.11	61.53	61.72	61.97	62.21	62.38
16100	61.84	62.29	62.49	62.74	62.94	63.09
16200	62.06	62.54	62.77	63.04	63.29	63.46
16300	62.38	62.90	63.17	63.47	63.80	64.04
16400	62.97	63.48	63.76	64.06	64.37	64.59
16500	63.73	64.21	64.48	64.74	64.97	65.12
16600	64.40	64.88	65.14	65.36	65.58	65.72
16700	65.14	65.59	65.84	66.03	66.23	66.37
16800	65.70	66.10	66.32	66.52	66.73	66.89
16900	66.26	66.60	66.80	67.01	67.24	67.40
Kittani Street (S16)						
17000	67.05	67.38	67.57	67.79	68.06	68.25
17088	67.24	67.60	67.81	68.05	68.34	68.55
Gold Creek						
0	30.08	30.55	30.80	31.11	31.51	31.82
100	30.49	30.93	31.21	31.51	31.93	32.26
200	30.90	31.42	31.71	32.01	32.42	32.73
300	31.84	32.06	32.24	32.45	32.86	33.16
400	32.26	32.67	32.91	33.21	33.64	33.94
500	32.84	33.26	33.47	33.73	34.08	34.35
600	33.91	34.29	34.50	34.76	35.09	35.30
Savages Road (S34)						
700	34.43	34.91	35.16	35.65	36.06	36.32
Adavale Street (S35)						
800	35.03	35.52	35.71	36.05	36.38	36.59
900	35.38	35.88	36.10	36.40	36.67	36.84
1000	35.88	36.40	36.65	36.96	37.26	37.46

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
1100	36.50	37.03	37.32	37.65	38.06	38.35
1200	37.19	37.71	38.01	38.36	38.79	39.11
1300	37.91	38.42	38.72	39.06	39.50	39.83
1400	38.48	38.98	39.26	39.58	40.00	40.32
1500	38.86	39.36	39.61	39.84	40.19	40.48
1600	39.21	39.71	39.96	40.18	40.52	40.81
1700	39.48	40.04	40.31	40.59	40.98	41.30
1800	39.80	40.39	40.69	40.99	41.39	41.69
1900	40.14	40.75	41.07	41.41	41.81	42.08
272 Gold Creek Road (S36)						
2000	40.54	41.23	41.60	42.05	42.71	43.15
2100	41.22	41.79	42.11	42.52	43.10	43.51
2200	42.04	42.51	42.78	43.06	43.45	43.77
2300	42.77	43.24	43.49	43.71	43.97	44.21
2400	43.29	43.82	44.09	44.35	44.65	44.86
2500	43.79	44.27	44.52	44.76	45.06	45.28
2600	44.29	44.72	44.92	45.16	45.49	45.72
Gold Creek Road #1 (S37)						
2700	44.71	45.18	45.43	45.71	46.08	46.59
2800	45.23	45.76	46.04	46.36	46.78	47.20
2900	45.82	46.31	46.58	46.88	47.26	47.62
3000	46.39	46.86	47.12	47.39	47.73	48.03
3100	47.13	47.56	47.81	48.06	48.38	48.65
3200	48.08	48.46	48.68	48.91	49.24	49.50
3300	48.92	49.24	49.40	49.58	49.84	50.06
3400	49.63	49.95	50.08	50.20	50.39	50.54
3500	50.05	50.42	50.56	50.69	50.85	50.96
3600	50.49	50.89	51.03	51.16	51.31	51.41
3700	51.00	51.38	51.51	51.64	51.81	51.93
3800	51.48	51.87	52.05	52.24	52.49	52.67
3900	51.58	52.00	52.20	52.40	52.69	52.89
4000	51.88	52.30	52.52	52.75	53.07	53.31
4100	52.51	52.89	53.13	53.34	53.66	53.92

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
4217	54.18	54.55	54.72	54.73	54.83	55.05
4300	54.32	54.73	54.91	54.99	55.22	55.46
4400	54.80	55.22	55.42	55.59	55.92	56.17
Gold Creek Road #2 (S40)						
4517	56.89	57.13	57.26	57.40	57.58	57.71
4600	56.95	57.21	57.36	57.50	57.69	57.82
4700	57.09	57.37	57.51	57.66	57.85	57.98
4800	57.52	57.86	58.02	58.17	58.33	58.45
Gold Creek Road #3 (S41)						
4924	59.00	59.20	59.34	59.47	59.64	59.78
5000	59.14	59.42	59.59	59.78	60.02	60.20
5100	59.53	59.86	60.05	60.26	60.53	60.72
5200	60.01	60.39	60.59	60.84	61.12	61.30
5300	60.55	60.93	61.15	61.39	61.68	61.87
5400	61.11	61.49	61.70	61.93	62.22	62.44
5500	61.64	62.00	62.19	62.43	62.72	62.94
5600	62.26	62.61	62.81	63.04	63.34	63.55
5700	62.88	63.23	63.43	63.66	63.96	64.16
5790	63.43	63.78	63.98	64.21	64.51	64.71
Gold Creek Road #4 (S42)						
5900	64.69	65.02	65.17	65.39	65.63	65.86
6000	65.05	65.48	65.70	65.98	66.30	66.55
6100	65.87	66.24	66.44	66.68	66.94	67.14
6200	67.03	67.32	67.47	67.65	67.84	68.00
6274	67.78	68.04	68.16	68.29	68.44	68.57
Gold Creek Road #5 (S43)						
6400	68.99	69.34	69.54	69.79	70.06	70.22
6500	69.34	69.73	69.95	70.22	70.53	70.73
6600	69.88	70.28	70.51	70.78	71.08	71.30
Gold Creek Road #6 (S44)						
6700	70.60	70.87	71.00	71.15	71.42	71.68
6800	71.50	71.82	72.00	72.21	72.48	72.69
6900	72.23	72.54	72.70	72.90	73.12	73.30

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
Gold Creek Road #7 (S45)						
7000	73.53	73.77	73.90	74.05	74.30	74.46
7088	74.14	74.45	74.64	74.86	75.10	75.27
Gold Creek Road #8 (S46)						
7200	75.05	75.40	75.62	75.85	76.12	76.33
7300	76.00	76.37	76.55	76.78	77.04	77.23
7310	76.22	76.64	76.82	77.05	77.35	77.58
Gap Creek						
0	22.48	23.17	23.53	23.92	24.34	24.63
100	22.57	23.26	23.61	24.01	24.44	24.72
200	22.78	23.39	23.71	24.07	24.48	24.76
300	23.22	23.79	24.08	24.42	24.72	24.94
Brookfield Road (S28)						
421	23.65	24.17	24.47	24.77	25.22	25.41
500	24.23	24.73	25.03	25.34	25.68	25.85
600	25.00	25.47	25.76	26.06	26.30	26.46
700	25.57	25.98	26.23	26.51	26.79	26.97
800	26.14	26.49	26.71	26.96	27.28	27.48
900	26.77	27.11	27.32	27.56	27.88	28.09
1000	27.45	27.82	28.04	28.28	28.59	28.80
1100	28.13	28.51	28.74	28.99	29.31	29.53
1200	28.82	29.23	29.47	29.73	30.05	30.27
1300	29.98	30.40	30.64	30.91	31.25	31.46
1400	30.81	31.19	31.42	31.67	32.01	32.25
1500	31.59	31.96	32.17	32.42	32.76	33.01
1600	32.33	32.69	32.90	33.15	33.49	33.74
1700	33.05	33.41	33.62	33.87	34.20	34.45
1800	33.78	34.14	34.35	34.60	34.93	35.18
1900	34.38	34.73	34.93	35.15	35.49	35.68
2000	35.38	35.64	35.78	35.93	36.17	36.23
Gap Creek Road (S29, S30 & S31)						
2100	36.74	36.92	37.01	37.15	37.31	37.43
2200	36.99	37.26	37.40	37.58	37.79	37.95

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
2300	37.46	37.76	37.93	38.13	38.37	38.53
2400	38.10	38.38	38.54	38.73	38.94	39.07
2500	38.73	39.00	39.15	39.32	39.51	39.63
2600	39.35	39.62	39.77	39.94	40.14	40.28
2700	39.91	40.19	40.35	40.53	40.76	40.93
2800	40.48	40.77	40.93	41.13	41.38	41.58
2900	41.17	41.43	41.59	41.78	42.02	42.21
3000	41.98	42.20	42.33	42.49	42.71	42.87
3090	42.76	42.93	43.03	43.16	43.34	43.48
McKay Brook						
0	11.09	11.95	12.36	12.81	13.25	13.54
100	11.22	12.08	12.51	12.99	13.46	13.77
200	11.36	12.08	12.52	12.99	13.46	13.77
300	N/A refer Note (1)	N/A refer Note (1)	12.53	12.99	13.47	13.77
400	12.68	12.93	13.10	13.31	13.54	13.79
Brookfield Road (S17)						
510	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	13.97	14.35	14.66
600	13.92	14.18	14.34	14.57	14.80	14.99
700	15.94	16.04	16.11	16.20	16.30	16.37
800	16.89	17.04	17.12	17.22	17.34	17.42
900	18.27	18.44	18.53	18.65	18.80	18.90
1000	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	20.96
Mirbelia Street (S18)						
1100	20.66	20.81	20.90	21.02	21.17	21.28
1200	22.62	22.76	22.84	22.95	23.06	23.15
1300	23.79	23.94	24.02	24.13	24.25	24.33
1400	24.54	24.72	24.81	24.93	25.06	25.15
1500	25.45	25.66	25.78	25.93	26.09	26.21
1600	27.07	27.29	27.41	27.56	27.74	27.88
1700	28.16	28.29	28.36	28.45	28.56	28.63
1800	28.91	29.07	29.16	29.28	29.41	29.51
1900	30.51	30.62	30.68	30.76	30.85	30.92

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
2000	31.67	31.78	31.84	31.91	31.99	32.06
2100	32.64	32.77	32.84	32.92	33.01	33.07
2200	34.48	34.79	35.13	35.25	35.36	35.44
2300	35.59	35.72	35.80	35.88	35.99	36.07
2400	36.99	37.13	37.21	37.31	37.43	37.51
2500	38.15	38.26	38.32	38.39	38.48	38.55
2600	39.64	39.75	39.82	39.90	39.99	40.07
2700	41.02	41.13	41.20	41.30	41.40	41.49
2800	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)
Hillcrest Place (S20 & S21)						
2900	44.16	44.37	44.49	44.80	44.99	45.31
3000	45.42	45.53	45.60	45.75	45.89	46.08
3100	47.02	47.15	47.22	47.32	47.38	47.44
3200	48.63	48.76	48.84	48.96	49.11	49.18
3300	50.53	50.61	50.65	50.71	50.77	50.81
3400	52.66	52.74	52.78	52.83	52.89	52.93
Tinarra Crescent (S22)						
3500	54.49	54.63	54.73	54.85	55.10	55.38
3600	55.95	56.01	56.05	56.09	56.14	56.18
3700	57.59	57.67	57.72	57.77	57.84	57.89
3800	59.41	59.50	59.55	59.62	59.70	59.75
3900	60.87	60.91	60.93	60.97	61.00	61.04
3986	63.20	63.29	63.33	63.39	63.46	63.50
McKay Brook Tributary						
0	24.25	24.41	24.49	24.60	24.71	24.80
100	25.93	26.17	26.36	26.60	26.91	27.01
200	28.82	28.87	28.89	28.92	28.95	28.97
280	29.90	29.99	30.03	30.09	30.15	30.20
Wexford Street (S27)						
403	32.22	32.32	32.36	32.42	32.48	32.54

Note (1) – Current BCC AMTD Line does not intersect the flood surface

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

The flood level data presented in this Appendix has been extracted (in part) from the results of a 2-dimensional flood model. Levels presented have been extracted generally at selected points along the centreline of the waterway with the intent of demonstrating general flood characteristics. The applicability of this data to locations on the floodplains adjacent should be determined by a suitably qualified professional. It is recommended for any detailed assessment of flood risk associated with the waterway that complete flood model results be accessed and interrogated.

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AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
Moggill Creek						
0	1.27	1.29	1.29	1.48	1.53	1.81
100	1.29	1.35	1.42	1.67	2.16	2.50
200	1.44	1.69	1.90	2.27	2.82	3.17
300	1.53	1.86	2.12	2.50	3.05	3.41
400	1.63	2.04	2.31	2.70	3.23	3.60
500	1.67	2.08	2.33	2.69	3.19	3.56
600	1.83	2.39	2.69	3.11	3.65	4.16
700	2.19	2.91	3.29	3.77	4.39	4.85
800	2.30	3.04	3.42	3.89	4.49	4.97
900	2.42	3.12	3.48	3.91	4.53	5.05
1000	2.59	3.35	3.75	4.27	4.91	5.36
1100	2.64	3.39	3.79	4.31	4.95	5.36
1200	2.71	3.49	3.90	4.45	5.08	5.47
1300	2.92	3.73	4.13	4.67	5.27	5.66
1400	3.05	3.92	4.31	4.80	5.40	5.78
1500	3.11	4.03	4.48	5.00	5.57	5.93
1600	3.34	4.23	4.66	5.17	5.73	6.10
1700	3.48	4.33	4.77	5.27	5.85	6.23
1800	3.64	4.50	4.94	5.46	6.06	6.47
1900	3.84	4.69	5.11	5.62	6.18	6.57
2000	4.09	4.99	5.42	5.89	6.44	6.80
2100	4.31	5.17	5.57	6.03	6.55	6.91
2200	4.40	5.24	5.68	6.16	6.72	7.08
2300	4.55	5.43	5.85	6.33	6.87	7.23
2400	4.78	5.58	5.96	6.41	6.93	7.28
2500	4.91	5.67	6.02	6.44	6.95	7.29
2600	4.95	5.70	6.03	6.44	6.94	7.27
2700	5.07	5.81	6.14	6.54	7.02	7.34
2800	5.11	5.83	6.15	6.55	7.04	7.36
2900	5.26	6.03	6.42	6.86	7.41	7.83
Moggill Road (S1 & S2)						

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
3020	5.66	6.62	7.18	7.99	8.77	9.51
3100	6.05	7.00	7.51	8.22	8.97	9.65
3200	6.27	7.21	7.68	8.35	9.08	9.73
3300	6.51	7.44	7.92	8.58	9.30	9.92
3400	6.75	7.68	8.16	8.81	9.52	10.11
3500	6.92	7.85	8.32	8.96	9.66	10.23
3600	7.07	8.00	8.47	9.10	9.79	10.34
3700	7.33	8.21	8.66	9.26	9.94	10.47
3800	7.55	8.41	8.83	9.39	10.03	10.57
3900	7.95	8.72	9.06	9.52	10.11	10.63
4000	8.15	8.90	9.22	9.63	10.19	10.69
4100	8.44	9.22	9.56	9.98	10.53	10.99
4200	8.77	9.58	9.95	10.40	10.93	11.36
4300	9.10	9.92	10.30	10.76	11.29	11.68
4400	9.40	10.12	10.49	10.94	11.45	11.84
4500	9.62	10.39	10.76	11.20	11.68	12.05
4600	9.82	10.63	11.02	11.44	11.89	12.23
4700	9.99	10.84	11.24	11.66	12.08	12.40
4800	10.17	11.04	11.45	11.88	12.31	12.62
4900	10.36	11.24	11.66	12.11	12.54	12.86
5000	10.54	11.44	11.87	12.33	12.78	13.09
5100	11.18	12.05	12.47	12.93	13.40	13.73
5200	11.58	12.42	12.80	13.23	13.68	14.01
5300	11.78	12.59	12.96	13.39	13.85	14.18
Branton Street Footbridge (S4)						
5400	12.01	12.80	13.15	13.57	14.02	14.35
5500	12.19	13.00	13.35	13.77	14.21	14.53
5600	12.38	13.22	13.59	14.03	14.48	14.80
5700	12.59	13.46	13.86	14.32	14.80	15.14
5800	12.80	13.69	14.12	14.62	15.13	15.48
5900	13.07	13.96	14.37	14.87	15.39	15.75
6000	13.37	14.24	14.63	15.11	15.62	15.97
6100	13.67	14.53	14.89	15.34	15.84	16.20

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
6200	14.01	14.78	15.11	15.54	16.01	16.35
6300	14.37	15.05	15.36	15.76	16.21	16.53
6400	14.85	15.50	15.81	16.20	16.62	16.93
6500	15.34	15.97	16.27	16.64	17.05	17.33
6600	15.68	16.29	16.57	16.93	17.32	17.60
6700	15.97	16.58	16.84	17.17	17.56	17.83
6800	16.23	16.85	17.11	17.44	17.82	18.09
6900	16.50	17.13	17.39	17.71	18.09	18.36
7000	16.79	17.42	17.69	18.02	18.40	18.67
7100	17.10	17.73	18.01	18.35	18.74	19.02
7200	17.41	18.04	18.33	18.69	19.09	19.38
7300	17.67	18.32	18.63	19.00	19.41	19.72
7400	17.90	18.59	18.92	19.29	19.69	19.97
7500	18.25	18.96	19.30	19.65	20.04	20.31
7600	18.69	19.44	19.76	20.11	20.47	20.74
7700	19.06	19.79	20.09	20.42	20.77	21.03
7800	19.39	20.03	20.31	20.62	20.95	21.19
7900	19.72	20.28	20.53	20.83	21.14	21.36
8000	19.92	20.48	20.74	21.04	21.35	21.57
Rafting Ground Road #1 (S6)						
8145	20.67	21.09	21.32	21.59	21.91	22.16
8200	20.80	21.26	21.50	21.78	22.09	22.33
8300	21.02	21.53	21.79	22.08	22.38	22.60
8400	21.25	21.76	22.02	22.32	22.62	22.83
8500	21.44	21.98	22.23	22.53	22.84	23.07
8595	21.61	22.18	22.43	22.74	23.06	23.29
Rafting Ground Road #2 (S7)						
8700	22.20	22.89	23.20	23.55	23.92	24.19
8800	22.46	23.20	23.54	23.94	24.35	24.64
8900	22.61	23.37	23.73	24.15	24.56	24.84
9000	22.89	23.58	23.89	24.26	24.65	24.92
9100	22.98	23.67	23.99	24.35	24.72	24.98
9200	23.61	24.18	24.49	24.85	25.19	25.42

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
9300	24.14	24.68	24.97	25.31	25.61	25.81
9400	24.48	25.06	25.36	25.65	25.91	26.08
9500	24.84	25.45	25.76	26.01	26.22	26.37
9600	25.20	25.86	26.19	26.47	26.70	26.83
Brookfield Road (S9)						
9700	25.36	26.38	26.89	27.33	27.63	27.80
9800	25.50	26.50	26.99	27.42	27.71	27.89
9900	25.73	26.67	27.12	27.52	27.82	28.00
10000	26.03	26.88	27.26	27.64	27.94	28.13
10100	26.32	27.08	27.40	27.76	28.07	28.26
10200	26.64	27.33	27.62	27.95	28.26	28.46
10300	27.00	27.66	27.96	28.25	28.55	28.76
10400	27.29	27.93	28.23	28.52	28.82	29.04
10500	27.46	28.10	28.42	28.72	29.06	29.29
10600	27.64	28.29	28.61	28.93	29.30	29.56
10700	27.83	28.48	28.80	29.15	29.55	29.82
10800	28.14	28.78	29.11	29.49	29.91	30.19
10900	28.44	29.08	29.42	29.83	30.28	30.56
11000	29.49	30.04	30.32	30.68	31.10	31.39
11100	30.09	30.63	30.91	31.29	31.74	32.06
Bundeleer Road (S10)						
11200	31.58	31.86	32.06	32.35	32.72	33.10
11300	31.88	32.32	32.59	32.93	33.28	33.59
11400	32.11	32.63	32.95	33.32	33.71	34.00
11500	32.33	32.91	33.25	33.66	34.09	34.38
11600	32.67	33.23	33.55	33.94	34.36	34.65
11700	33.03	33.60	33.92	34.32	34.75	35.04
11800	33.39	33.98	34.32	34.72	35.16	35.44
11900	33.97	34.52	34.83	35.22	35.64	35.92
12000	34.54	35.05	35.34	35.70	36.12	36.39
12100	35.08	35.57	35.85	36.20	36.59	36.85
12200	35.60	36.08	36.35	36.70	37.07	37.30
12300	36.16	36.64	36.92	37.27	37.63	37.86

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
12400	36.75	37.25	37.55	37.92	38.30	38.54
12500	37.38	37.91	38.21	38.61	39.00	39.26
12600	38.23	38.86	39.18	39.56	39.97	40.24
12700	38.55	39.16	39.48	39.84	40.24	40.50
12800	38.87	39.46	39.76	40.12	40.51	40.78
185 Upper Brookfield Road (S11)						
12907	39.63	40.45	40.75	41.10	41.45	41.70
13000	40.39	40.98	41.24	41.56	41.91	42.18
Upper Brookfield Road #1 (S12)						
13100	41.05	41.66	41.95	42.31	42.76	43.11
13200	41.44	42.01	42.27	42.61	43.03	43.36
13300	41.88	42.41	42.66	42.98	43.37	43.69
13400	42.40	42.90	43.15	43.46	43.85	44.16
13500	43.24	43.75	44.04	44.35	44.68	44.95
Haven Road (S13)						
13600	44.61	45.01	45.23	45.54	45.90	46.18
13700	45.11	45.55	45.79	46.07	46.42	46.69
13800	45.61	46.10	46.35	46.61	46.95	47.21
13900	46.13	46.66	46.95	47.26	47.63	47.92
14000	46.59	47.23	47.59	47.99	48.47	48.81
14100	46.99	47.59	47.94	48.35	48.82	49.18
14200	47.68	48.27	48.61	49.02	49.50	49.87
14300	48.41	48.98	49.31	49.72	50.21	50.60
14400	49.31	49.87	50.19	50.59	51.08	51.46
14500	50.27	50.82	51.15	51.56	52.05	52.44
14600	50.83	51.40	51.74	52.16	53.15	53.60
14700	51.38	51.92	52.25	52.65	53.51	53.96
Upper Brookfield Road #2 (S15)						
14800	52.50	53.13	53.49	53.96	54.67	55.63
14900	52.91	53.50	53.85	54.28	54.92	55.76
15000	54.05	54.54	54.83	55.21	55.71	56.29
15100	54.89	55.34	55.62	55.95	56.35	56.76
15200	55.45	55.94	56.22	56.56	56.94	57.30

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
15300	55.90	56.44	56.75	57.12	57.56	57.93
15400	56.34	56.93	57.26	57.66	58.15	58.53
15500	56.67	57.27	57.62	58.02	58.51	58.88
15600	57.27	57.83	58.14	58.52	58.98	59.32
15700	58.04	58.49	58.76	59.08	59.48	59.78
15800	59.06	59.48	59.73	60.03	60.41	60.69
15900	60.07	60.48	60.72	61.01	61.36	61.63
16000	61.13	61.64	61.90	62.20	62.53	62.79
16100	61.88	62.45	62.73	63.03	63.35	63.60
16200	62.09	62.67	62.96	63.26	63.59	63.85
16300	62.40	62.98	63.29	63.62	64.00	64.29
16400	62.98	63.52	63.82	64.14	64.49	64.75
16500	63.73	64.23	64.50	64.77	65.05	65.25
16600	64.44	64.92	65.18	65.44	65.71	65.89
16700	65.20	65.68	65.92	66.16	66.41	66.59
16800	65.74	66.18	66.41	66.64	66.90	67.08
16900	66.29	66.67	66.89	67.11	67.39	67.57
Kittani Street (S16)						
17000	67.06	67.39	67.58	67.82	68.11	68.32
17088	67.25	67.63	67.84	68.09	68.41	68.64
Gold Creek						
0	30.09	30.63	30.91	31.28	31.73	32.05
100	30.53	31.01	31.29	31.67	32.17	32.49
200	30.93	31.44	31.74	32.10	32.56	32.88
300	31.86	32.06	32.25	32.53	32.99	33.30
400	32.28	32.66	32.91	33.25	33.71	34.04
500	32.93	33.34	33.58	33.90	34.30	34.57
600	33.91	34.28	34.50	34.79	35.16	35.39
Savages Road (S34)						
700	34.43	34.89	35.15	35.66	36.11	36.37
Adavale Street (S35)						
800	35.09	35.57	35.77	36.09	36.47	36.71
900	35.41	35.90	36.13	36.45	36.80	37.02

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
1000	35.89	36.39	36.66	36.99	37.36	37.61
1100	36.50	37.01	37.30	37.64	38.08	38.38
1200	37.19	37.68	37.98	38.33	38.79	39.12
1300	37.91	38.39	38.69	39.04	39.49	39.83
1400	38.48	38.96	39.25	39.58	40.01	40.34
1500	38.87	39.36	39.64	39.90	40.26	40.56
1600	39.21	39.72	40.00	40.26	40.61	40.90
1700	39.48	40.03	40.33	40.63	41.04	41.37
1800	39.80	40.37	40.69	41.02	41.46	41.79
1900	40.14	40.73	41.06	41.42	41.87	42.20
272 Gold Creek Road (S36)						
2000	40.54	41.20	41.59	42.05	42.75	43.26
2100	41.22	41.76	42.09	42.51	43.14	43.61
2200	42.04	42.48	42.77	43.08	43.52	43.90
2300	42.77	43.21	43.49	43.76	44.10	44.41
2400	43.29	43.79	44.10	44.42	44.83	45.13
2500	43.80	44.26	44.55	44.86	45.26	45.53
2600	44.29	44.70	44.95	45.24	45.62	45.86
Gold Creek Road #1 (S37)						
2700	44.71	45.16	45.43	45.74	46.23	46.78
2800	45.23	45.73	46.03	46.36	46.84	47.31
2900	45.82	46.28	46.57	46.87	47.32	47.72
3000	46.39	46.83	47.10	47.38	47.78	48.12
3100	47.13	47.53	47.79	48.05	48.42	48.71
3200	48.08	48.44	48.66	48.90	49.24	49.51
3300	48.92	49.24	49.42	49.61	49.89	50.11
3400	49.63	49.96	50.11	50.26	50.47	50.62
3500	50.05	50.42	50.56	50.70	50.87	50.98
3600	50.50	50.89	51.04	51.18	51.35	51.46
3700	51.01	51.41	51.58	51.74	51.94	52.10
3800	51.49	51.88	52.08	52.29	52.57	52.76
3900	51.59	52.00	52.22	52.45	52.76	52.98
4000	51.88	52.30	52.54	52.78	53.13	53.38

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
4100	52.51	52.88	53.13	53.35	53.70	53.97
4217	54.18	54.53	54.71	54.73	54.84	55.06
4300	54.34	54.72	54.93	55.00	55.26	55.51
4400	54.84	55.25	55.47	55.67	56.02	56.32
Gold Creek Road #2 (S40)						
4517	56.93	57.18	57.31	57.45	57.62	57.75
4600	57.00	57.27	57.41	57.58	57.76	57.90
4700	57.13	57.42	57.57	57.74	57.93	58.08
4800	57.55	57.89	58.06	58.23	58.41	58.55
Gold Creek Road #3 (S41)						
4924	59.02	59.23	59.36	59.49	59.68	59.82
5000	59.18	59.47	59.65	59.84	60.11	60.30
5100	59.57	59.92	60.12	60.35	60.63	60.83
5200	60.06	60.44	60.67	60.94	61.19	61.38
5300	60.58	60.97	61.20	61.47	61.74	61.95
5400	61.12	61.49	61.72	61.97	62.29	62.53
5500	61.69	62.06	62.27	62.52	62.85	63.09
5600	62.32	62.69	62.91	63.16	63.49	63.68
5700	62.95	63.33	63.55	63.80	64.14	64.28
5790	63.51	63.89	64.12	64.38	64.72	64.81
Gold Creek Road #4 (S42)						
5900	64.70	65.03	65.19	65.44	65.72	65.96
6000	65.07	65.49	65.73	66.03	66.40	66.66
6100	65.88	66.26	66.48	66.73	67.06	67.31
6200	67.06	67.37	67.53	67.72	67.97	68.18
6274	67.84	68.12	68.27	68.42	68.62	68.78
Gold Creek Road #5 (S43)						
6400	69.02	69.39	69.61	69.88	70.16	70.36
6500	69.37	69.78	70.02	70.31	70.63	70.87
6600	69.91	70.34	70.58	70.87	71.20	71.45
Gold Creek Road #6 (S44)						
6700	70.61	70.87	71.01	71.18	71.55	71.78
6800	71.50	71.84	72.03	72.25	72.54	72.77

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
6900	72.23	72.55	72.72	72.92	73.16	73.35
Gold Creek Road #7 (S45)						
7000	73.53	73.77	73.91	74.08	74.32	74.48
7088	74.15	74.47	74.68	74.91	75.16	75.35
Gold Creek Road #8 (S46)						
7200	75.05	75.41	75.62	75.86	76.13	76.34
7300	76.00	76.37	76.55	76.78	77.04	77.23
7310	76.22	76.64	76.82	77.06	77.35	77.58
Gap Creek						
0	22.56	23.32	23.67	24.08	24.49	24.78
100	22.66	23.39	23.75	24.17	24.58	24.86
200	22.80	23.49	23.83	24.23	24.62	24.91
300	23.23	23.84	24.15	24.50	24.80	25.05
Brookfield Road (S28)						
421	23.64	24.18	24.47	24.84	25.16	25.48
500	24.23	24.73	25.04	25.36	25.68	25.93
600	24.99	25.47	25.76	26.08	26.35	26.53
700	25.57	25.98	26.24	26.53	26.82	27.00
800	26.14	26.49	26.72	26.98	27.28	27.48
900	26.77	27.11	27.32	27.57	27.88	28.09
1000	27.45	27.82	28.04	28.29	28.59	28.81
1100	28.13	28.52	28.75	29.01	29.31	29.54
1200	28.82	29.23	29.47	29.74	30.05	30.29
1300	29.98	30.40	30.64	30.92	31.25	31.49
1400	30.81	31.20	31.42	31.69	32.01	32.27
1500	31.59	31.96	32.17	32.43	32.75	33.02
1600	32.31	32.68	32.90	33.15	33.47	33.74
1700	33.03	33.39	33.61	33.86	34.18	34.45
1800	33.75	34.12	34.34	34.59	34.90	35.17
1900	34.35	34.70	34.91	35.14	35.44	35.70
2000	35.36	35.62	35.77	35.92	36.14	36.27
Gap Creek Road (S29, S30 & S31)						
2100	36.77	36.95	37.06	37.20	37.38	37.50

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
2200	37.02	37.28	37.44	37.63	37.86	38.02
2300	37.47	37.77	37.95	38.16	38.40	38.58
2400	38.10	38.38	38.55	38.75	38.97	39.13
2500	38.74	39.00	39.16	39.34	39.55	39.70
2600	39.35	39.62	39.77	39.96	40.17	40.34
2700	39.91	40.19	40.35	40.55	40.78	40.96
2800	40.48	40.77	40.93	41.14	41.39	41.59
2900	41.17	41.43	41.59	41.78	42.03	42.22
3000	41.98	42.20	42.33	42.49	42.71	42.88
3090	42.76	42.93	43.03	43.16	43.34	43.48
McKay Brook						
0	11.08	11.96	12.38	12.84	13.31	13.63
100	11.21	12.09	12.53	13.01	13.51	13.85
200	11.36	12.09	12.53	13.01	13.51	13.85
300	N/A refer Note (1)	N/A refer Note (1)	12.55	13.02	13.52	13.86
400	12.69	12.96	13.13	13.36	13.60	13.86
Brookfield Road (S17)						
510	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	13.94	14.31	14.63
600	13.91	14.18	14.33	14.56	14.78	14.97
700	15.98	16.11	16.19	16.29	16.41	16.50
800	16.97	17.14	17.24	17.35	17.50	17.61
900	18.31	18.50	18.60	18.74	18.89	19.01
1000	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	20.96
Mirbelia Street (S18)						
1100	20.66	20.81	20.89	21.02	21.17	21.28
1200	22.64	22.80	22.89	23.00	23.13	23.23
1300	23.85	24.02	24.12	24.24	24.38	24.48
1400	24.57	24.77	24.88	25.01	25.16	25.27
1500	25.47	25.69	25.82	25.99	26.18	26.31
1600	27.08	27.31	27.44	27.62	27.83	28.00
1700	28.19	28.33	28.41	28.51	28.62	28.71
1800	28.96	29.13	29.22	29.35	29.49	29.60

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)					
	2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
1900	30.53	30.64	30.70	30.78	30.88	30.95
2000	31.68	31.79	31.85	31.93	32.02	32.09
2100	32.65	32.80	32.88	32.98	33.11	33.20
2200	34.48	34.79	35.13	35.25	35.37	35.45
2300	35.59	35.72	35.80	35.89	36.00	36.09
2400	36.99	37.13	37.21	37.31	37.43	37.52
2500	38.15	38.26	38.32	38.39	38.49	38.55
2600	39.64	39.75	39.82	39.91	40.00	40.07
2700	41.01	41.13	41.21	41.30	41.41	41.50
2800	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)
Hillcrest Place (S20 & S21)						
2900	44.16	44.37	44.49	44.80	44.98	45.31
3000	45.42	45.53	45.60	45.75	45.89	46.08
3100	47.02	47.15	47.22	47.32	47.38	47.44
3200	48.63	48.76	48.84	48.96	49.11	49.19
3300	50.53	50.61	50.65	50.71	50.77	50.82
3400	52.66	52.74	52.78	52.84	52.90	52.95
Tinarra Crescent (S22)						
3500	54.49	54.63	54.73	54.86	55.10	55.37
3600	55.96	56.03	56.06	56.12	56.17	56.21
3700	57.60	57.69	57.74	57.81	57.88	57.94
3800	59.41	59.49	59.55	59.61	59.68	59.74
3900	60.87	60.92	60.94	60.97	61.01	61.04
3986	63.20	63.28	63.33	63.39	63.45	63.50
McKay Brook Tributary						
0	24.29	24.48	24.57	24.70	24.84	24.94
100	25.93	26.18	26.36	26.60	26.91	27.02
200	28.82	28.87	28.89	28.92	28.95	28.97
280	29.90	29.99	30.03	30.09	30.16	30.21
Wexford Street (S27)						
403	32.23	32.33	32.37	32.44	32.51	32.56

Note (1) – Current BCC AMTD Line does not intersect the flood surface

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

The flood level data presented in this Appendix has been extracted (in part) from the results of a 2-dimensional flood model. Levels presented have been extracted generally at selected points along the centreline of the waterway with the intent of demonstrating general flood characteristics. The applicability of this data to locations on the floodplains adjacent should be determined by a suitably qualified professional. It is recommended for any detailed assessment of flood risk associated with the waterway that complete flood model results be accessed and interrogated.

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AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)	
	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
Moggill Creek		
0	2.07	2.11
100	2.97	3.45
200	3.57	4.09
300	3.82	4.37
400	3.98	4.53
500	3.97	4.53
600	4.67	5.35
700	5.14	5.67
800	5.20	5.73
900	5.34	5.95
1000	5.57	6.09
1100	5.55	6.07
1200	5.60	6.10
1300	5.75	6.24
1400	5.92	6.41
1500	6.11	6.58
1600	6.21	6.67
1700	6.32	6.78
1800	6.58	7.11
1900	6.70	7.23
2000	6.97	7.46
2100	7.04	7.50
2200	7.39	7.93
2300	7.46	7.96
2400	7.46	7.96
2500	7.46	7.96
2600	7.46	7.96
2700	7.44	7.95
2800	7.49	8.01
2900	8.06	8.63
Moggill Road (S1 & S2)		
3020	9.77	10.40

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)	
	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
3100	9.91	10.52
3200	9.98	10.58
3300	10.13	10.75
3400	10.29	10.93
3500	10.37	11.00
3600	10.44	11.07
3700	10.54	11.16
3800	10.66	11.28
3900	10.72	11.33
4000	10.76	11.35
4100	11.13	11.72
4200	11.57	12.18
4300	11.94	12.55
4400	12.03	12.65
4500	12.25	12.85
4600	12.43	12.99
4700	12.56	13.08
4800	12.77	13.26
4900	13.00	13.46
5000	13.23	13.66
5100	13.87	14.31
5200	14.16	14.60
5300	14.30	14.74
Branton Street Footbridge (S4)		
5400	14.45	14.89
5500	14.63	15.06
5600	14.90	15.33
5700	15.23	15.67
5800	15.56	16.00
5900	15.84	16.30
6000	16.09	16.56
6100	16.35	16.83
6200	16.49	16.97

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)	
	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
6300	16.64	17.13
6400	17.02	17.45
6500	17.40	17.78
6600	17.67	18.03
6700	17.92	18.27
6800	18.20	18.57
6900	18.50	18.86
7000	18.82	19.20
7100	19.19	19.58
7200	19.55	19.96
7300	19.87	20.31
7400	20.10	20.51
7500	20.38	20.79
7600	20.73	21.13
7700	20.99	21.38
7800	21.19	21.54
7900	21.39	21.71
8000	21.60	21.89
Rafting Ground Road #1 (S6)		
8145	22.25	22.58
8200	22.44	22.77
8300	22.72	23.04
8400	22.93	23.24
8500	23.16	23.47
8595	23.40	23.72
Rafting Ground Road #2 (S7)		
8700	24.22	24.63
8800	24.70	25.10
8900	24.90	25.28
9000	24.98	25.35
9100	25.03	25.39
9200	25.47	25.81
9300	25.85	26.17

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)	
	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
9400	26.09	26.38
9500	26.34	26.60
9600	26.83	27.05
Brookfield Road (S9)		
9700	27.76	28.00
9800	27.86	28.10
9900	27.98	28.23
10000	28.13	28.39
10100	28.28	28.56
10200	28.48	28.76
10300	28.74	29.03
10400	29.00	29.31
10500	29.27	29.61
10600	29.53	29.91
10700	29.81	30.22
10800	30.24	30.63
10900	30.67	31.04
11000	31.40	31.78
11100	32.05	32.53
Bundeleer Road (S10)		
11200	33.11	33.59
11300	33.67	34.09
11400	34.09	34.49
11500	34.45	34.84
11600	34.73	35.13
11700	35.13	35.52
11800	35.55	35.92
11900	36.07	36.45
12000	36.58	36.97
12100	37.01	37.38
12200	37.42	37.76
12300	37.99	38.33
12400	38.72	39.09

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)	
	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
12500	39.49	39.89
12600	40.29	40.62
12700	40.55	40.88
12800	40.84	41.20
185 Upper Brookfield Road (S11)		
12907	41.76	42.13
13000	42.30	42.66
Upper Brookfield Road #1 (S12)		
13100	43.37	44.02
13200	43.60	44.20
13300	43.92	44.47
13400	44.37	44.86
13500	45.15	45.56
Haven Road (S13)		
13600	46.27	46.62
13700	46.73	47.05
13800	47.19	47.49
13900	47.99	48.34
14000	48.84	49.19
14100	49.25	49.63
14200	50.05	50.49
14300	50.86	51.36
14400	51.74	52.23
14500	52.75	53.24
14600	53.87	54.31
14700	54.22	54.66
Upper Brookfield Road #2 (S15)		
14800	55.98	56.58
14900	56.06	56.65
15000	56.48	56.99
15100	56.87	57.31
15200	57.40	57.81
15300	58.09	58.49

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)	
	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
15400	58.74	59.14
15500	59.10	59.51
15600	59.54	59.99
15700	59.99	60.85
15800	60.85	61.44
15900	61.73	62.06
16000	62.53	62.80
16100	63.21	63.45
16200	63.61	63.85
16300	64.22	64.53
16400	64.76	65.04
16500	65.25	65.46
16600	65.84	66.04
16700	66.49	66.66
16800	67.01	67.19
16900	67.52	67.72
Kittani Street (S16)		
17000	68.43	68.71
17088	68.74	69.04
Gold Creek		
0	32.03	32.51
100	32.45	32.92
200	32.92	33.33
300	33.37	33.76
400	34.17	34.61
500	34.52	34.95
600	35.43	35.63
Savages Road (S34)		
700	36.45	36.68
Adavale Street (S35)		
800	36.71	36.92
900	36.95	37.14
1000	37.60	37.82

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)	
	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
1100	38.54	38.83
1200	39.32	39.60
1300	40.03	40.26
1400	40.51	40.71
1500	40.66	40.87
1600	41.01	41.27
1700	41.54	41.90
1800	41.92	42.27
1900	42.29	42.63
272 Gold Creek Road (S36)		
2000	43.42	43.78
2100	43.75	44.10
2200	43.97	44.28
2300	44.38	44.65
2400	45.01	45.28
2500	45.43	45.69
2600	45.86	46.10
Gold Creek Road #1 (S37)		
2700	47.10	47.74
2800	47.60	48.08
2900	47.95	48.39
3000	48.30	48.70
3100	48.88	49.23
3200	49.71	50.00
3300	50.23	50.48
3400	50.66	50.85
3500	51.03	51.16
3600	51.47	51.60
3700	52.02	52.20
3800	52.80	53.01
3900	53.04	53.28
4000	53.48	53.79
4100	54.11	54.43

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)	
	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
4217	55.22	55.52
4300	55.65	55.99
4400	56.39	56.73
Gold Creek Road #2 (S40)		
4517	57.79	58.00
4600	57.92	58.14
4700	58.08	58.29
4800	58.53	58.69
Gold Creek Road #3 (S41)		
4924	59.87	60.02
5000	60.33	60.53
5100	60.86	61.10
5200	61.45	61.71
5300	62.03	62.26
5400	62.60	62.78
5500	63.11	63.34
5600	63.70	63.92
5700	64.29	64.50
5790	64.81	65.02
Gold Creek Road #4 (S42)		
5900	65.99	66.25
6000	66.71	66.97
6100	67.30	67.54
6200	68.13	68.37
6274	68.69	68.90
Gold Creek Road #5 (S43)		
6400	70.35	70.57
6500	70.89	71.15
6600	71.45	71.73
Gold Creek Road #6 (S44)		
6700	71.89	72.15
6800	72.86	73.13
6900	73.44	73.68

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)	
	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
Gold Creek Road #7 (S45)		
7000	74.55	74.76
7088	75.40	75.67
Gold Creek Road #8 (S46)		
7200	76.47	76.73
7300	77.36	77.61
7310	77.73	78.03
Gap Creek		
0	24.83	25.22
100	24.92	25.30
200	24.95	25.33
300	25.10	25.43
Brookfield Road (S28)		
421	25.68	26.02
500	26.04	26.32
600	26.57	26.78
700	27.06	27.24
800	27.55	27.69
900	28.19	28.35
1000	28.94	29.17
1100	29.68	29.91
1200	30.43	30.67
1300	31.60	31.85
1400	32.43	32.75
1500	33.20	33.55
1600	33.92	34.23
1700	34.63	34.89
1800	35.35	35.57
1900	35.81	36.01
2000	36.31	36.46
Gap Creek Road (S29, S30 & S31)		
2100	37.52	37.65
2200	38.07	38.28

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)	
	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
2300	38.66	38.89
2400	39.18	39.37
2500	39.73	39.90
2600	40.38	40.56
2700	41.05	41.24
2800	41.71	41.92
2900	42.36	42.59
3000	43.01	43.24
3090	43.60	43.82
McKay Brook		
0	13.77	14.22
100	14.01	14.47
200	14.01	14.47
300	14.01	14.47
400	14.02	14.48
Brookfield Road (S17)		
510	15.33	15.92
600	15.45	15.98
700	16.50	16.65
800	17.50	17.63
900	19.00	19.16
1000	21.03	21.16
Mirbelia Street (S18)		
1100	21.39	21.56
1200	23.22	23.34
1300	24.41	24.53
1400	25.23	25.36
1500	26.32	26.50
1600	28.02	28.23
1700	28.69	28.80
1800	29.60	29.74
1900	30.98	31.08
2000	32.11	32.20

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)	
	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
2100	33.14	33.23
2200	35.50	35.59
2300	36.15	36.25
2400	37.60	37.70
2500	38.63	38.73
2600	40.12	40.20
2700	41.55	41.64
2800	N/A refer Note (1)	N/A refer Note (1)
Hillcrest Place (S20 & S21)		
2900	45.45	45.60
3000	46.19	46.31
3100	47.49	47.57
3200	49.20	49.27
3300	50.85	50.90
3400	52.97	53.01
Tinarra Crescent (S22)		
3500	55.68	56.21
3600	56.29	56.52
3700	57.93	58.02
3800	59.82	59.89
3900	61.06	61.12
3986	63.55	63.61
McKay Brook Tributary		
0	24.87	24.99
100	27.07	27.15
200	29.00	29.03
280	30.24	30.29
Wexford Street (S27)		
403	32.58	32.65

Note (1) – Current BCC AMTD Line does not intersect the flood surface

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Appendix G: Rating Curves

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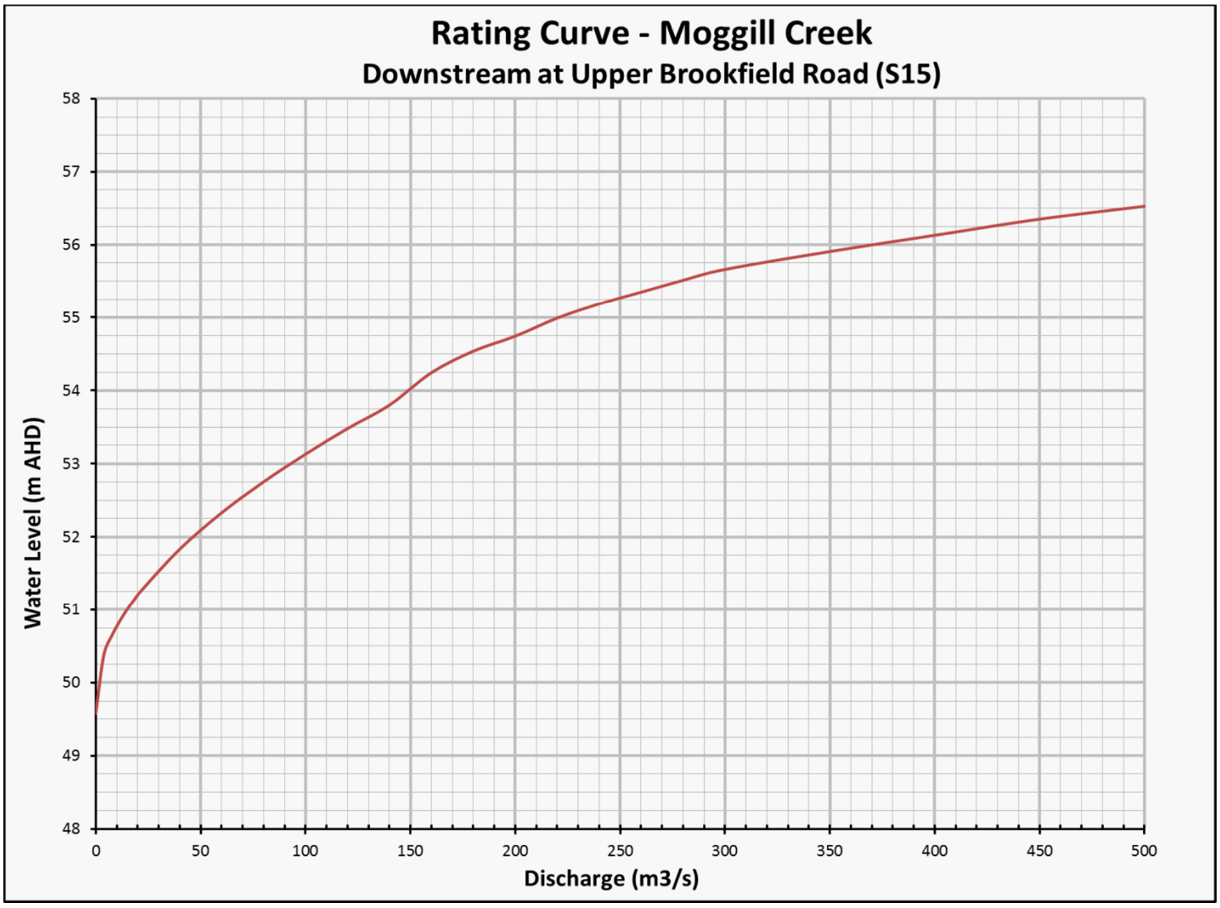
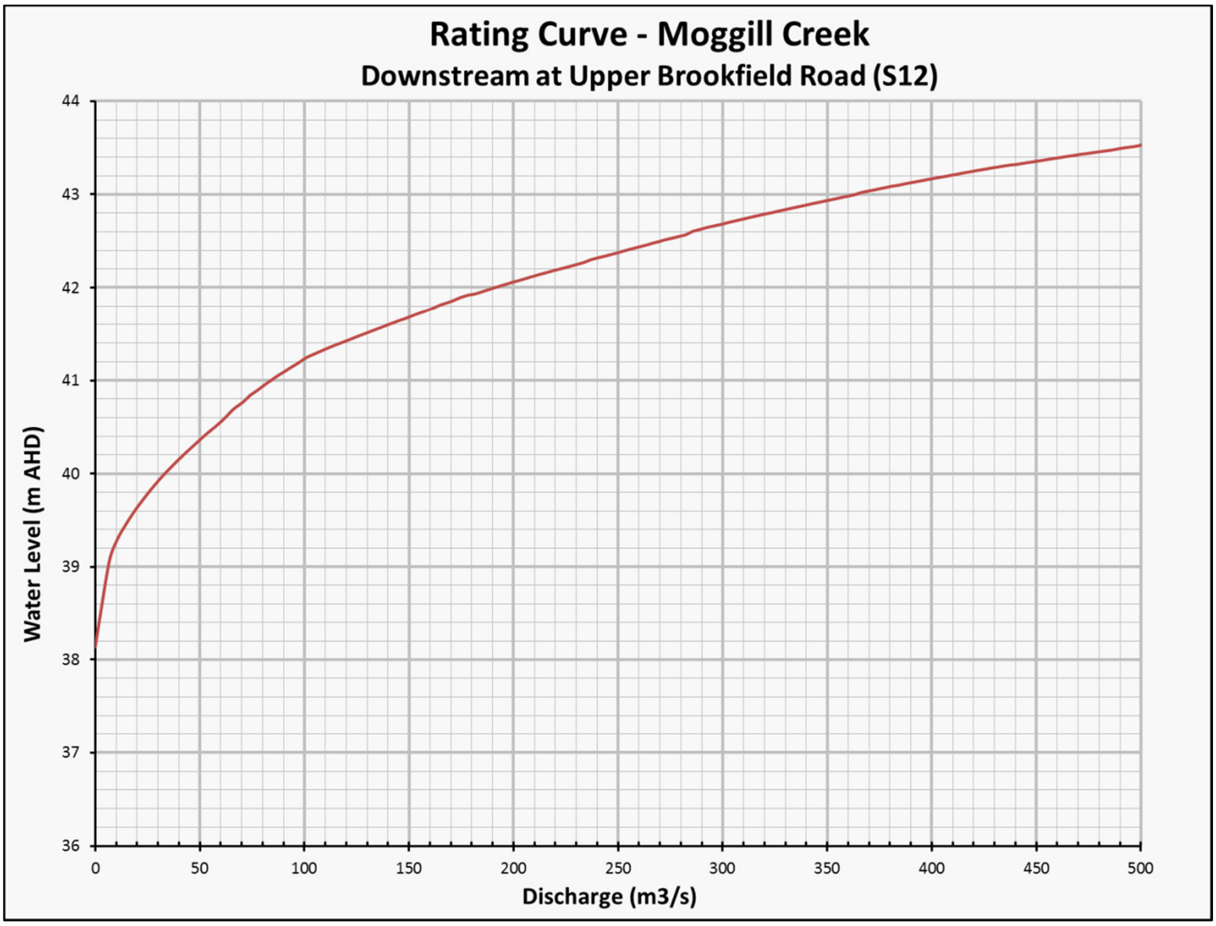
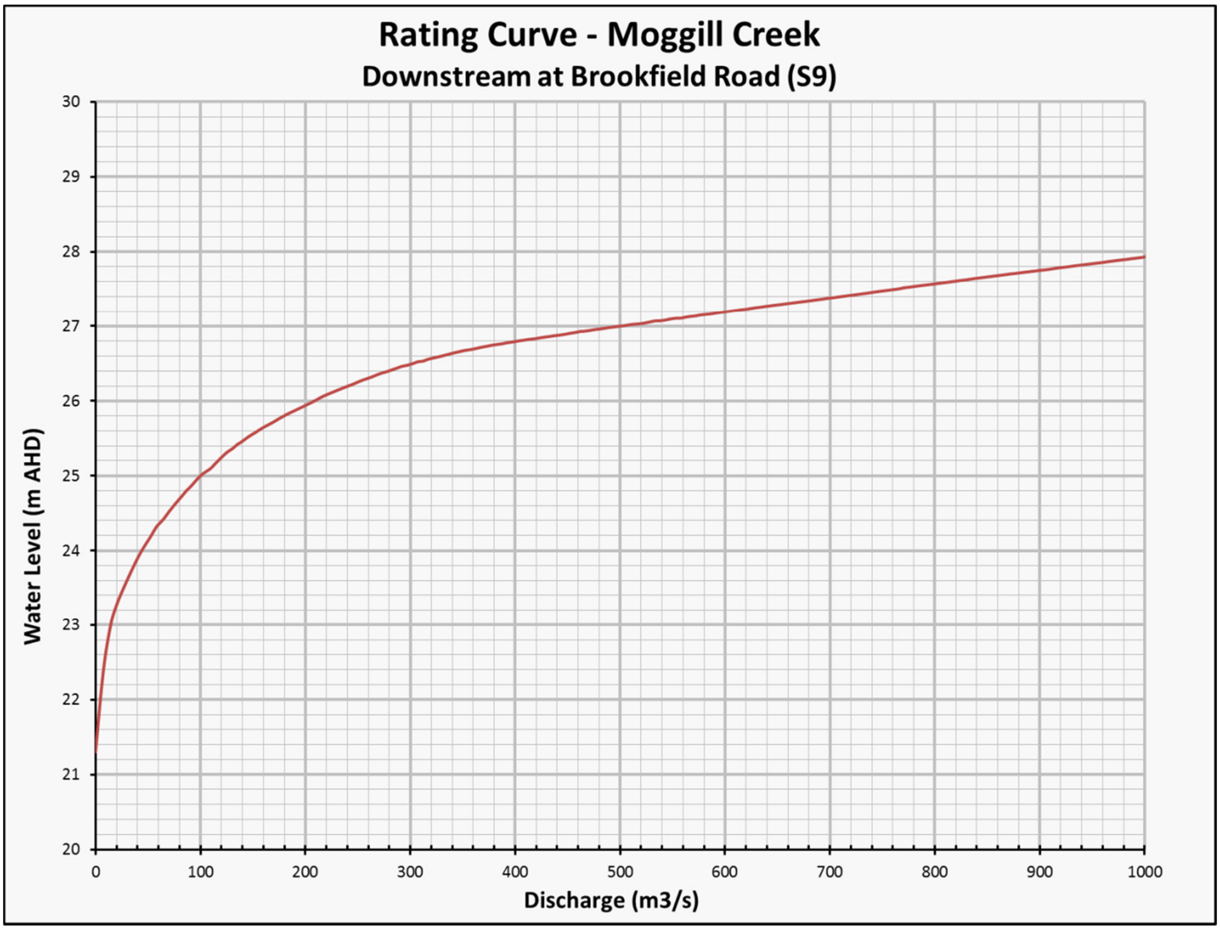
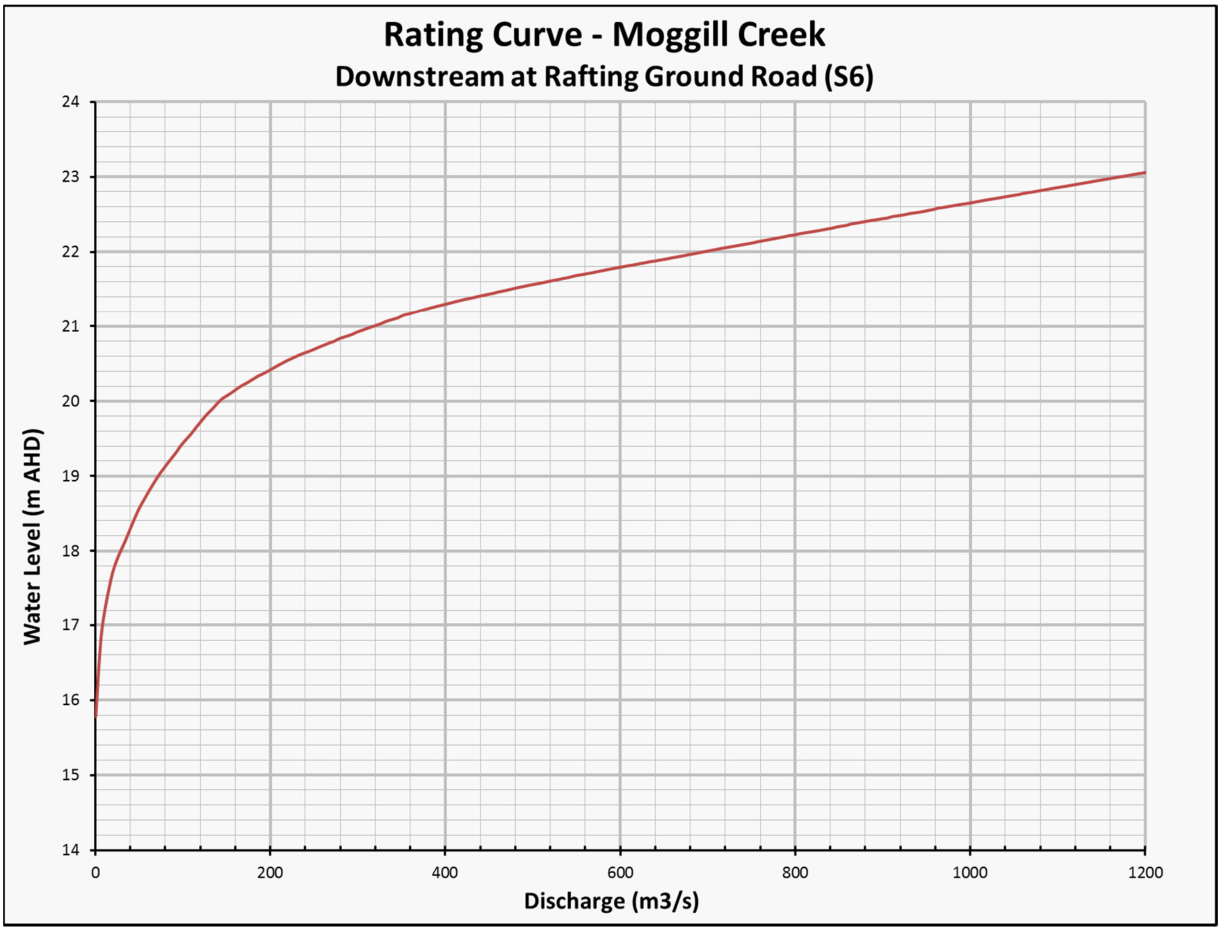
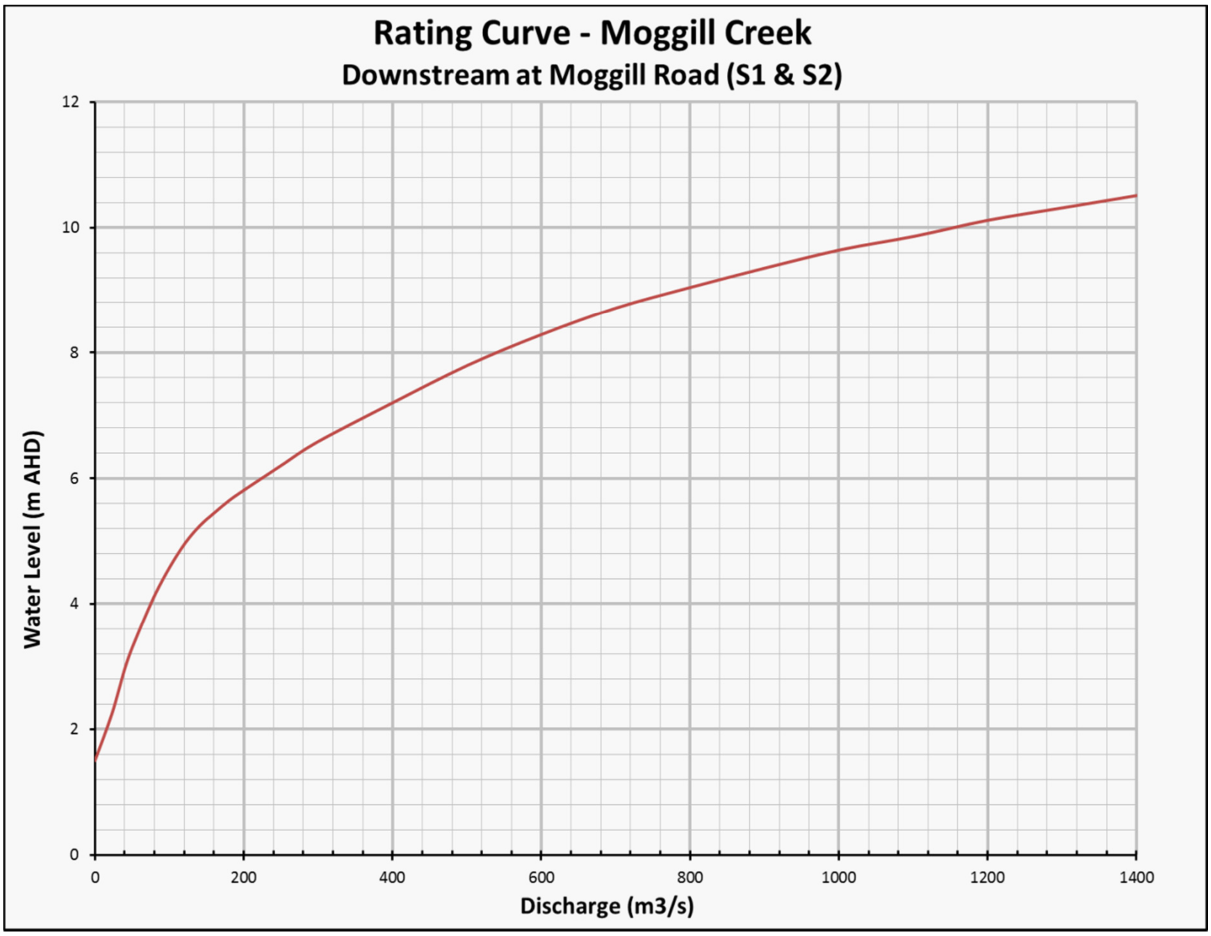
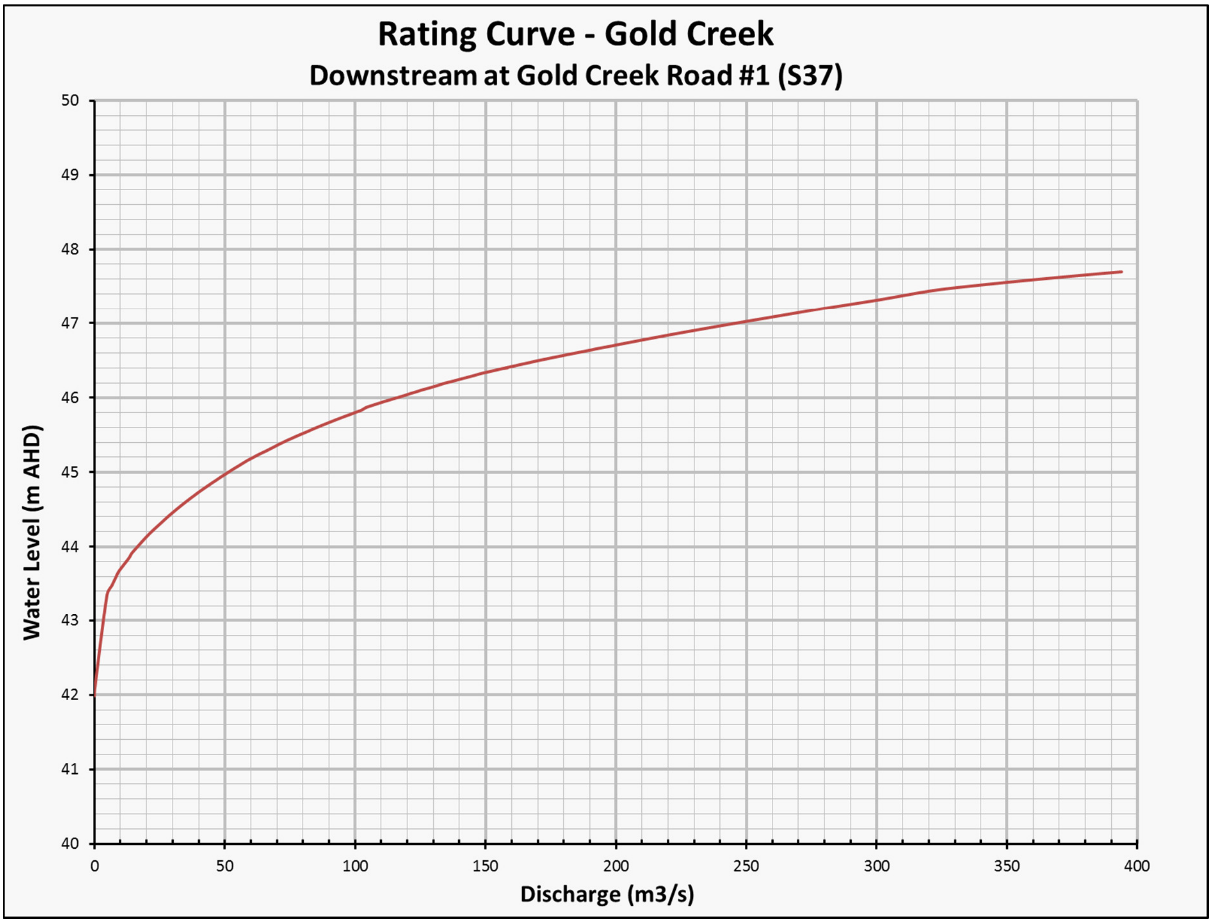

Rating Curve - Moggill Creek
Downstream at Upper Brookfield Road (S15)
Water Level (m AHD)
58
57
56
55
54
53
52
51
50
49
48
0
50
100
150
200
250
300
350
400
450
500
Discharge (m3/s)

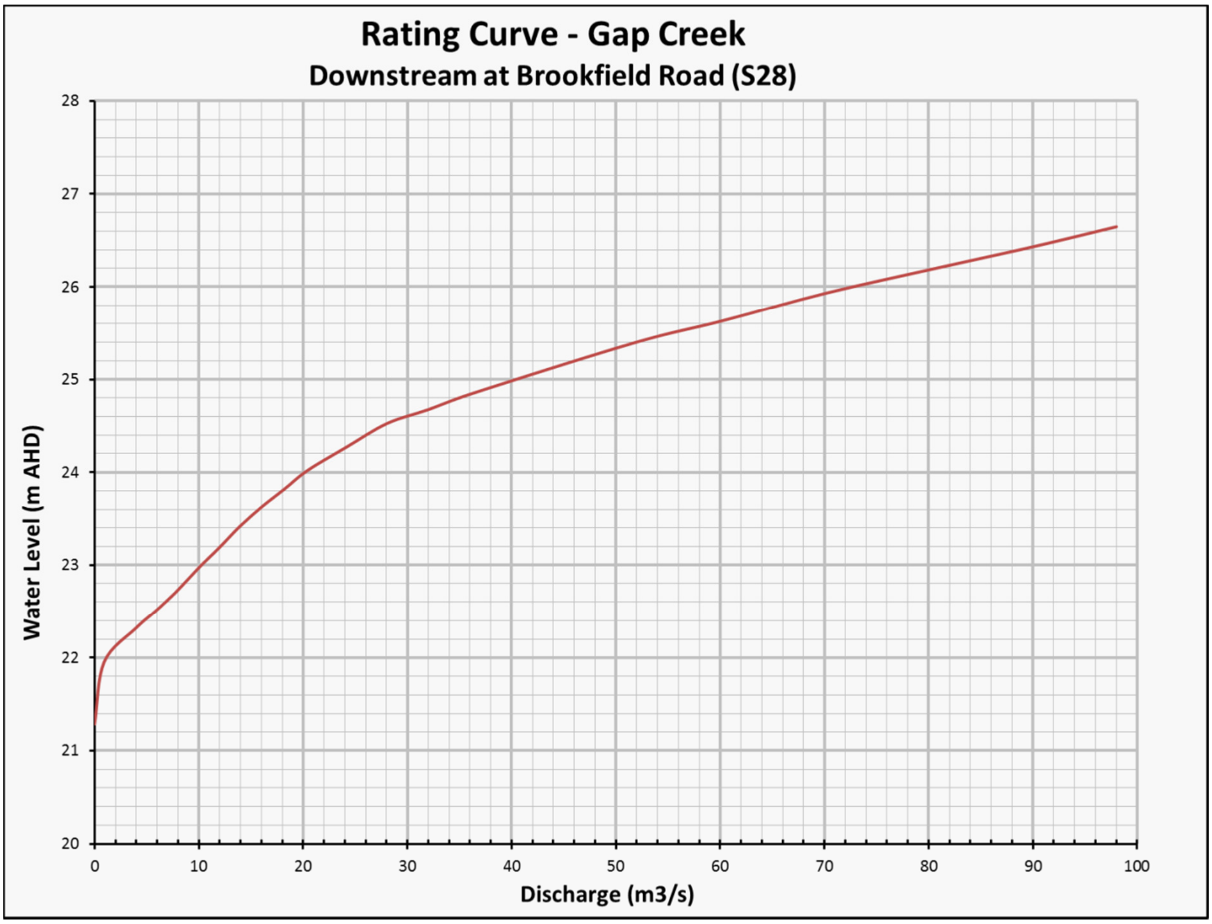

Rating Curve - Moggill Creek
Downstream at Upper Brookfield Road (S12)
Water Level (m AHD)
44
43
42
41
40
39
38
37
36
0
50
100
150
200
250
300
350
400
450
500
Discharge (m3/s)


Rating Curve - Moggill Creek
Downstream at Brookfield Road (S9)
Water Level (m AHD)
Discharge (m3/s)
30
29
28
27
26
25
24
23
22
21
20
0
100
200
300
400
500
600
700
800
900
1000

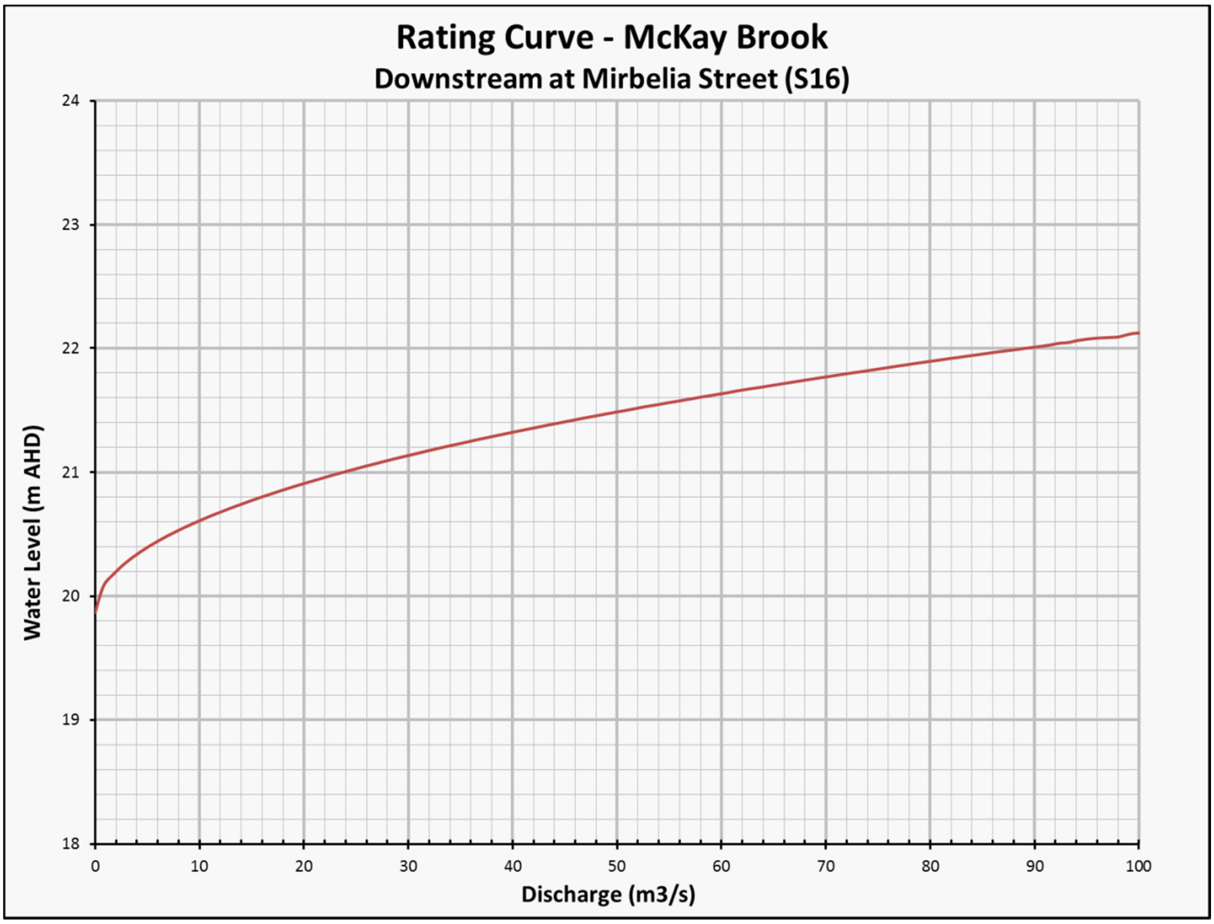

Rating Curve - Moggill Creek
Downstream at Rafting Ground Road (S6)
Water Level (m AHD)
Discharge (m3/s)
24
23
22
21
20
19
18
17
16
15
14
0
200
400
600
800
1000
1200


Rating Curve - Moggill Creek
Downstream at Moggill Road (S1 & S2)
Water Level (m AHD)
0
2
4
6
8
10
12
Discharge (m3/s)
0
200
400
600
800
1000
1200
1400

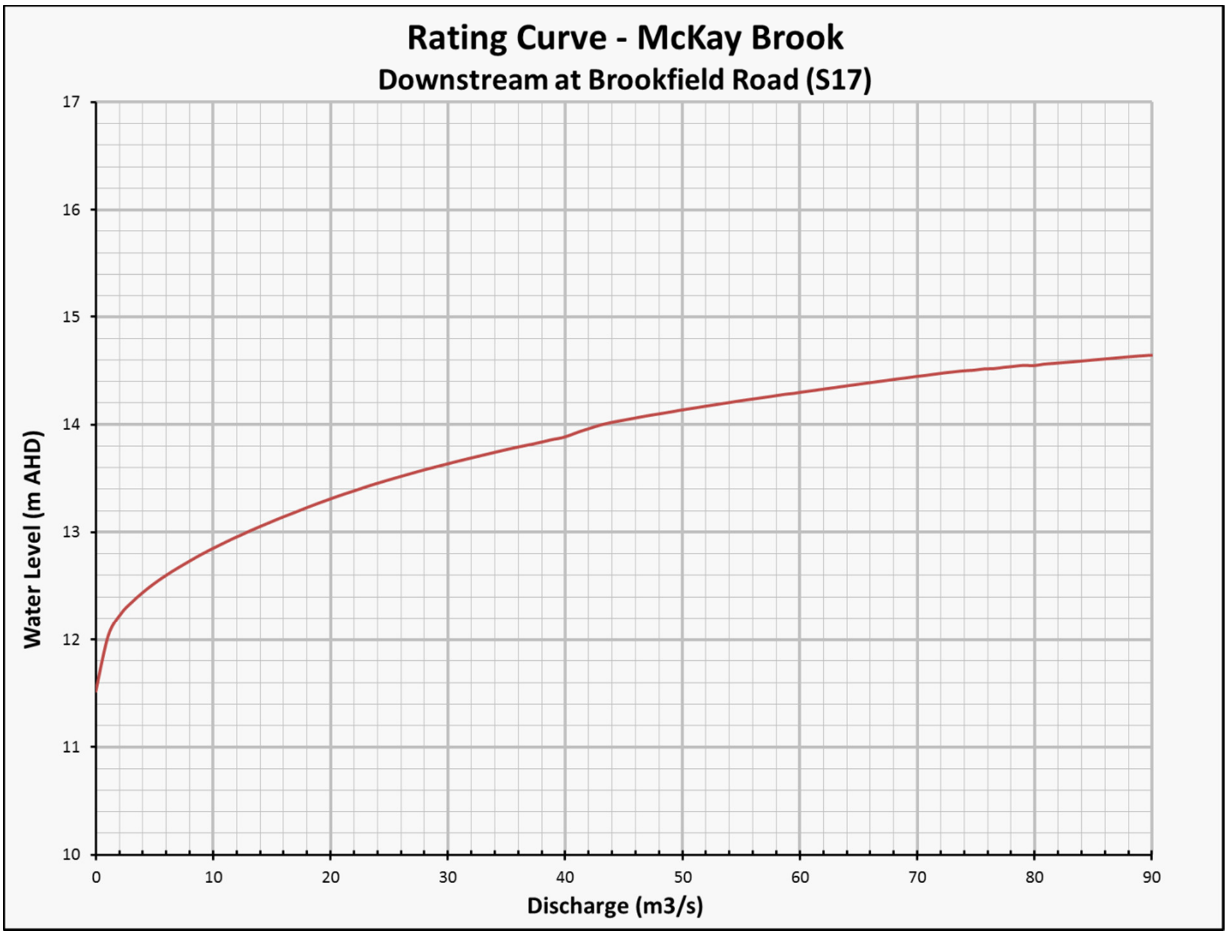

Rating Curve - Gold Creek
Downstream at Gold Creek Road #1 (S37)
Water Level (m AHD)
40
41
42
43
44
45
46
47
48
49
50
Discharge (m3/s)
0
50
100
150
200
250
300
350
400



Rating Curve - Gap Creek
Downstream at Brookfield Road (S28)
Water Level (m AHD)
Discharge (m3/s)
28
27
26
25
24
23
22
21
20
0
10
20
30
40
50
60
70
80
90
100



Rating Curve - McKay Brook
Downstream at Mirbelia Street (S16)
Water Level (m AHD)
Discharge (m3/s)
24
23
22
21
20
19
18
0
10
20
30
40
50
60
70
80
90
100


Rating Curve - McKay Brook
Downstream at Brookfield Road (S17)
Water Level (m AHD)
Discharge (m3/s)
17
16
15
14
13
12
11
10
0
10
20
30
40
50
60
70
80
90

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Appendix H: Hydraulic Structure Reference Sheets

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Creek:	Moggill Creek		**Immunity Rating:**	2% AEP
Location:	Moggill Road			50-yr ARI

DATE OF SURVEY: 2015 (U/S Cross-section)

UBD REF: 177 G12

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): N/A TMR

MODEL ID: S2

New AMTD (m): 3000

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: Three Span

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

U/S INVERT LEVEL (m) 1.36
U/S OBVERT LEVEL (m) 7.83 to 8.36

D/S INVERT LEVEL (m) 1.68 (ALS 2014)
D/S OBVERT LEVEL (m) 7.83 to 8.36

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? N/A N/A

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): 25.2
PIER WIDTH (m): 0.7

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

LOWEST POINT OF WEIR (m AHD): 8.53 (approx)

HEIGHT OF GUARDRAIL/HANDRAIL: varies

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

PLAN NUMBER: N/A TMR

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 2006 Duplication

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Moggill Creek
Location:	Moggill Road

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	1211.5	11.61	10.54	1.07	157	2.06	4.2	3.6
500-yr (0.2%)	704.1	10.26	9.45	0.81	146	0.86	3.5	3.2
100-yr (1%)	503.8	9.36	8.74	0.62	118	0.13	3.5	2.9
50-yr (2%)	431.5	8.68	8.31	0.37	0.0	0.0	0.0	2.5
20-yr (5%)	333.9	7.87	7.63	0.24	0.0	0.0	0.0	2.5
10-yr (10%)	263.3	6.98	6.73	0.25	0.0	0.0	0.0	2.4
5-yr (20%)	213.1	6.51	6.14	0.37	0.0	0.0	0.0	2.3
2-yr (50%)	137.8	5.55	5.35	0.20	0.0	0.0	0.0	2.2

Notes:

Max depth is taken at road centreline

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:	Moggill Creek
Location:	Moggill Road

Downstream of Moggill Road Bridge

Creek:	Moggill Creek	**Immunity Rating:**	>50% AEP
Location:	Branton Street Footbridge		<2-yr ARI

DATE OF SURVEY: 2015 (U/S Cross-section)

UBD REF: 177 J6

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): B7010

MODEL ID: S4

New AMTD (m): 5370

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: Single Span

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

U/S INVERT LEVEL (m) 7.19 U/S OBVERT LEVEL (m) 10.8 to 11.12

D/S INVERT LEVEL (m) 7.19 D/S OBVERT LEVEL (m) 10.8 to 11.12

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? N/A N/A

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): 3.3 PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 11.2

HEIGHT OF GUARDRAIL/HANDRAIL: 1.2

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

PLAN NUMBER: W110121

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 1999

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Moggill Creek
Location:	Branton Street Footbridge

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	1187.2	16.25	16.10	0.15	141	5.9	2.7	1.5
500-yr (0.2%)	613.8	14.83	14.77	0.07	130	4.5	1.9	1.4
100-yr (1%)	448.4	14.15	14.09	0.06	120	3.9	1.6	1.6
50-yr (2%)	394.3	13.84	13.78	0.06	115	3.6	1.6	1.6
20-yr (5%)	330.4	13.40	13.34	0.06	110	3.1	1.5	1.6
10-yr (10%)	281.1	12.99	12.92	0.07	100	2.7	1.5	1.6
5-yr (20%)	240.3	12.64	12.56	0.08	90	2.3	1.6	1.6
2-yr (50%)	161.0	11.89	11.78	0.11	80	1.6	1.3	1.7

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:	Moggill Creek
Location:	Branton Street Footbridge

Upstream of Branton Street Footbridge

Downstream of Branton Street Footbridge

Creek:	Moggill Creek
Location:	Rafting Ground Road 1

Immunity Rating:	>50% AEP
	<2-yr ARI

DATE OF SURVEY: N/A	UBD REF: 177 B6
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID (Gecko): C4747B
MODEL ID: S6	New AMTD (m): 8100

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 3 / 3000 x 2400mm SLBC

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

U/S INVERT LEVEL (m) 16.76   U/S OBVERT LEVEL (m) 19.16

D/S INVERT LEVEL (m) 16.14   D/S OBVERT LEVEL (m) 18.54

For culverts give floor level   For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 35.38

LENGTH OF CULVERT AT OBVERT (m): 35.38

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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 (on skew)   PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 19.05 (at culvert, not road alignment sag)

HEIGHT OF GUARDRAIL/HANDRAIL: 0.7 (armco)

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

PLAN NUMBER: CD070583

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 2009

HAS THE STRUCTURE BEEN UPGRADED? Yes   Upgraded from a causeway crossing

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

ADDITIONAL COMMENTS:

Creek:	Moggill Creek
Location:	Rafting Ground Road 1

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	686.7	23.55	23.01	0.53	255	4.2	1.1	5.0
500-yr (0.2%)	340.3	22.53	21.99	0.54	175	3.1	0.9	6.2
100-yr (1%)	216.8	22.03	21.56	0.47	120	2.6	0.8	6.1
50-yr (2%)	170.2	21.80	21.36	0.43	105	2.4	0.7	6.1
20-yr (5%)	119.9	21.51	21.05	0.45	95	2.1	0.4	6.1
10-yr (10%)	99.8	21.24	20.73	0.51	85	1.8	0.2	6.4
5-yr (20%)	92.7	21.02	20.48	0.54	75	1.5	0.2	6.5
2-yr (50%)	89.3	20.64	19.94	0.70	65	1.0	0.2	6.4

Notes:

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

Structure velocity is the peak within the culvert barrel

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	Moggill Creek
Location:	Rafting Ground Road 1

Upstream of Rafting Ground Road Culvert 1

Downstream of Rafting Ground Road Culvert 1

Creek:	Moggill Creek	**Immunity Rating:**	>50% AEP
Location:	Rafting Ground Road 2		<2-yr ARI

DATE OF SURVEY: N/A	UBD REF: 177 B4
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID (Gecko): C0698B
MODEL ID: S7	New AMTD (m): 8610

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 4 / 3600 x 2700mm RCBC

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

U/S INVERT LEVEL (m)	17.72	U/S OBVERT LEVEL (m)	20.42
D/S INVERT LEVEL (m)	17.61	D/S OBVERT LEVEL (m)	20.31

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 14.64

LENGTH OF CULVERT AT OBVERT (m): 14.64

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 11.9 (on skew) PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 21 (approx)

HEIGHT OF GUARDRAIL/HANDRAIL: 0.7 (armco)

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

PLAN NUMBER: W10033

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 1997

HAS THE STRUCTURE BEEN UPGRADED? Yes New road works circa 2014

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

ADDITIONAL COMMENTS:

Creek:	Moggill Creek
Location:	Rafting Ground Road 2

ARI (AEP %)	**PEAK DISCHARGE (m3/s)**	**Peak U/S Water Level**	**Peak D/S Water Level**	**AFFLUX (m)**	**MAX WIDTH OF WEIR FLOW (m)**	**MAX DEPTH OF WEIR FLOW (m)**	**VELOCITY (m/s)**	
		(m AHD)					**Weir**	**Structure**
2000-yr (0.05%)	701.7	24.92	24.66	0.26	130	3.3	2.7	3.4
500-yr (0.2%)	397.5	24.03	23.73	0.30	110	2.5	2.0	3.4
100-yr (1%)	295.2	23.59	23.23	0.36	100	2.1	1.6	3.4
50-yr (2%)	256.6	23.38	23.01	0.37	95	1.9	1.4	3.4
20-yr (5%)	208.2	23.08	22.70	0.38	90	1.8	1.1	3.4
10-yr (10%)	179.6	22.82	22.39	0.43	85	1.5	0.8	3.3
5-yr (20%)	156.6	22.56	22.14	0.42	80	1.3	0.6	3.2
2-yr (50%)	118.3	21.96	21.60	0.36	45	0.8	0.3	2.9

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:	Moggill Creek
Location:	Rafting Ground Road 2

Upstream of Rafting Ground Road 2 Culverts

Downstream of Rafting Ground Road 2 Culverts

Creek:	Moggill Creek	**Immunity Rating:**	20% AEP
Location:	Brookfield Road		5-yr ARI

DATE OF SURVEY:	2015 (U/S Cross-section)	UBD REF:	176 R2
SURVEYED CROSS SECTION ID:	N/A	BCC ASSET ID:	B0360
MODEL ID:	S9	AMTD (m):	9650

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: 4 x span

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

U/S INVERT LEVEL (m)	21.7	U/S OBVERT LEVEL (m)	25.36 to 25.52
D/S INVERT LEVEL (m)	21.7	D/S OBVERT LEVEL (m)	25.36 to 25.52

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? N/A N/A

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): 11.3 PIER WIDTH (m): varies

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

LOWEST POINT OF WEIR (m AHD): 26.42

HEIGHT OF GUARDRAIL/HANDRAIL: 0.74 (approx)

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

PLAN NUMBER: W10033

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 1988

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Moggill Creek
Location:	Brookfield Road

ARI (AEP %)	**PEAK DISCHARGE (m3/s)**	**Peak U/S Water Level**	**Peak D/S Water Level**	**AFFLUX (m)**	**MAX WIDTH OF WEIR FLOW (m)**	**MAX DEPTH OF WEIR FLOW (m)**	**VELOCITY (m/s)**	
		(m AHD)					**Weir**	**Structure**
2000-yr (0.05%)	691.2	28.65	27.93	0.72	283	2.2	1.4	3.3
500-yr (0.2%)	428.9	27.98	27.17	0.81	235	1.5	1.0	3.2
100-yr (0.1%)	337.9	27.62	26.85	0.77	203	1.2	0.8	3.1
50-yr (0.2%)	302.3	27.44	26.71	0.72	195	1.0	0.6	3.0
20-yr (5%)	266.8	27.14	26.45	0.69	98	0.8	0.9	3.0
10-yr (10%)	231.3	26.71	26.13	0.58	94	0.4	0.9	3.0
5-yr (20%)	182.8	26.21	25.81	0.40	0	0.0	0.0	2.9
2-yr (50%)	114.2	25.21	25.18	0.03	0	0.0	0.0	2.9

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

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	Moggill Creek
Location:	Brookfield Road

Downstream of Brookfield Road (Moggill Creek)

Downstream of Brookfield Road (Moggill Creek)

Creek:	Moggill Creek
Location:	Bundeleer Road

Immunity Rating:	>50% AEP
	<2-yr ARI

DATE OF SURVEY: N/A

UBD REF: 136 Q19

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): B0380

MODEL ID: S10

New AMTD (m): 11190

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: Single span with low flow culverts

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

U/S INVERT LEVEL (m) 28.65 (approx) U/S OBVERT LEVEL (m) 29.35 (approx)

D/S INVERT LEVEL (m) 28.60 (approx) D/S OBVERT LEVEL (m) 29.30 (approx)

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? N/A N/A

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): 4 (approx) PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 30.37

HEIGHT OF GUARDRAIL/HANDRAIL: 1.27

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

PLAN NUMBER:

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 2013

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Moggill Creek
Location:	Bundeleer Road

ARI (AEP %)	**PEAK DISCHARGE (m3/s)**	**Peak U/S Water Level**	**Peak D/S Water Level**	**AFFLUX (m)**	**MAX WIDTH OF WEIR FLOW (m)**	**MAX DEPTH OF WEIR FLOW (m)**	**VELOCITY (m/s)**	
		(m AHD)					**Weir**	**Structure**
2000-yr (0.05%)	649.9	34.77	34.82	-0.04	132	4.9	1.9	6.0
500-yr (0.2%)	378.8	33.49	33.53	-0.04	81	3.7	2.4	6.1
100-yr (1%)	271.2	32.71	32.70	0.02	71	2.9	2.5	6.1
50-yr (2%)	235.2	32.41	32.28	0.13	70	2.5	2.4	6.0
20-yr (5%)	184.2	32.08	31.74	0.34	64	2.0	2.5	6.0
10-yr (10%)	146.5	31.86	31.28	0.57	60	1.7	2.5	6.0
5-yr (20%)	118.9	31.73	31.04	0.69	59	1.5	2.2	6.0
2-yr (50%)	78.1	31.45	30.48	0.97	45	1.2	2.0	6.0

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:	Moggill Creek
Location:	Bundeleer Road

Upstream of Bundaleer Street

Downstream of Bundaleer Street

Creek:	Moggill Creek	**Immunity Rating:**	50% AEP
Location:	185 Upper Brookfield Road		2-yr ARI

DATE OF SURVEY: 2015

UBD REF: 136 N20

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): N/A Private

MODEL ID: S11

New AMTD (m): 12900

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: Single Span

For Culverts: Number of cells/pipes & sizes

For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 36.38

U/S OBVERT LEVEL (m) 39.25

D/S INVERT LEVEL (m) 36.38

D/S OBVERT LEVEL (m) 39.25

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? N/A N/A

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): 4.6

PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 39.7

HEIGHT OF GUARDRAIL/HANDRAIL: None

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

PLAN NUMBER: N/A Private

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Unknown

HAS THE STRUCTURE BEEN UPGRADED? Unknown

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

ADDITIONAL COMMENTS:

Creek:	Moggill Creek
Location:	185 Upper Brookfield Road

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	538.6	42.99	42.40	0.59	63	3.2	4.8	3.1
500-yr (0.2%)	325.9	42.11	41.47	0.65	51	2.3	4.4	3.2
100-yr (1%)	232.0	41.53	40.89	0.65	47	1.6	4.3	3.2
50-yr (2%)	197.0	41.32	40.69	0.63	43	1.5	4.1	3.2
20-yr (5%)	153.8	40.97	40.32	0.65	39	1.2	3.8	3.1
10-yr (10%)	123.5	40.67	40.00	0.67	36	0.9	3.4	3.1
5-yr (20%)	100.3	40.43	39.73	0.70	34	0.7	2.7	3.1
2-yr (50%)	66.2	39.63	39.14	0.49	0	0.0	0.0	2.9

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

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	Moggill Creek
Location:	185 Upper Brookfield Road

Upstream of 185 Upper Brookfield Road

Downstream of 185 Upper Brookfield Road

Creek:	Moggill Creek	**Immunity Rating:**	0.2% AEP
Location:	Upper Brookfield Road #1		500-yr ARI

DATE OF SURVEY: 2015 (U/S Cross-section)

UBD REF: 136 N20

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): B2090

MODEL ID: S12

New AMTD (m): 13050

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: Two span

For Culverts: Number of cells/pipes & sizes

For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 38.13

U/S OBVERT LEVEL (m) 43.09 to 43.60

D/S INVERT LEVEL (m) 38.13

D/S OBVERT LEVEL (m) 43.09 to 43.60

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? N/A N/A

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): 10.45

PIER WIDTH (m): 0.76

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

LOWEST POINT OF WEIR (m AHD): 44.04

HEIGHT OF GUARDRAIL/HANDRAIL: 1.2

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

PLAN NUMBER: W8233

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 1989

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Moggill Creek
Location:	Upper Brookfield Road #1

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level (m AHD)	Peak D/S Water Level (m AHD)	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s) Weir	VELOCITY (m/s) Structure
2000-yr (0.05%)	540.7	45.65	43.67	1.98	90	0.8	3.2	5.0
500-yr (0.2%)	328.3	43.95	42.83	1.12	0	0.0	0.0	4.4
100-yr (1%)	233.9	42.98	42.27	0.70	0	0.0	0.0	3.7
50-yr (2%)	198.4	42.64	42.05	0.59	0	0.0	0.0	3.4
20-yr (5%)	155.3	42.19	41.73	0.46	0	0.0	0.0	3.0
10-yr (10%)	124.4	41.83	41.47	0.37	0	0.0	0.0	2.7
5-yr (20%)	100.6	41.54	41.24	0.29	0	0.0	0.0	2.4
2-yr (50%)	66.3	40.92	40.69	0.23	0	0.0	0.0	2.1

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

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	Moggill Creek
Location:	Upper Brookfield Road #1

Upstream of Upper Brookfield Road #1

Downstream of Upper Brookfield Road #1

Creek:	Moggill Creek		**Immunity Rating:**	>50% AEP
Location:	Haven Road			<2-yr ARI

DATE OF SURVEY: N/A	UBD REF: 136 M20
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID (Gecko): C0305P
MODEL ID: S13	New AMTD (m): 13530

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 3 / 1500 mm dia RCP

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

U/S INVERT LEVEL (m) 41.72 — U/S OBVERT LEVEL (m) 43.22

D/S INVERT LEVEL (m) 41.48 — D/S OBVERT LEVEL (m) 42.98

For culverts give floor level — For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 9

LENGTH OF CULVERT AT OBVERT (m): 9

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 9 (on skew) — PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 43.5

HEIGHT OF GUARDRAIL/HANDRAIL: None

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

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Unknown

HAS THE STRUCTURE BEEN UPGRADED? Unknown

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

ADDITIONAL COMMENTS:

Creek:	Moggill Creek
Location:	Haven Road

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	529.4	47.37	46.77	0.59	53	3.5	5.3	6.2
500-yr (0.2%)	319.9	46.43	45.58	0.86	44	2.6	5.0	5.8
100-yr (1%)	228.5	45.85	44.95	0.90	42	2.1	4.6	5.5
50-yr (2%)	193.4	45.61	44.71	0.91	39	1.8	4.6	5.3
20-yr (5%)	151.4	45.23	44.39	0.84	36	1.5	4.7	5.0
10-yr (10%)	121.2	44.94	44.08	0.86	34	1.2	4.6	4.8
5-yr (20%)	97.8	44.72	43.79	0.93	32	1.0	4.5	4.7
2-yr (50%)	64.5	44.35	42.28	2.07	27	0.6	5.0	4.4

Notes:

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

Structure velocity is the peak within the culvert barrel

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	Moggill Creek
Location:	Haven Road

Upstream of Haven Road

Downstream of Haven Road

Creek:	Moggill Creek		**Immunity Rating:**	2% AEP
Location:	Upper Brookfield Road #2			50-yr ARI

DATE OF SURVEY: N/A

UBD REF: 136 L20

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): B2080

MODEL ID: S15

New AMTD (m): 14750

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: Two span

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

U/S INVERT LEVEL (m) 49.58 (approx) U/S OBVERT LEVEL (m) 54.73

D/S INVERT LEVEL (m) 49.58 (approx) D/S OBVERT LEVEL (m) 54.73

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? N/A N/A

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): 9.9 PIER WIDTH (m): 0.6

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

LOWEST POINT OF WEIR (m AHD): 55.4

HEIGHT OF GUARDRAIL/HANDRAIL: 1.25

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

PLAN NUMBER: W5428

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 1974

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Moggill Creek
Location:	Upper Brookfield Road #2

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	507.1	57.50	56.57	0.93	104	2.0	2.3	3.5
500-yr (0.2%)	299.1	56.56	55.66	0.90	93	1.0	0.8	3.4
100-yr (1%)	213.4	55.51	54.92	0.59	12	0.1	0.1	2.7
50-yr (2%)	179.8	54.60	54.55	0.05	0	0.0	0.0	2.4
20-yr (5%)	140.4	53.88	53.83	0.04	0	0.0	0.0	2.3
10-yr (10%)	112.7	53.40	53.36	0.04	0	0.0	0.0	2.2
5-yr (20%)	91.5	53.03	52.98	0.05	0	0.0	0.0	2.0
2-yr (50%)	60.8	52.39	52.35	0.04	0	0.0	0.0	1.8

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:	Moggill Creek
Location:	Upper Brookfield Road #2

Upstream of Upper Brookfield Road #2

Downstream of Upper Brookfield Road #2

Creek:	Moggill Creek
Location:	Kittani Street

Immunity Rating:	>50% AEP
	<2-yr ARI

DATE OF SURVEY: 2015 (U/S Cross-section)	UBD REF: 136 J18
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID (Gecko): C2061P
MODEL ID: S16	New AMTD (m): 16920

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 3 / 600mm dia RCP

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

U/S INVERT LEVEL (m) 64.5 U/S OBVERT LEVEL (m) 65.1

D/S INVERT LEVEL (m) 64.49 D/S OBVERT LEVEL (m) 65.09

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 5

LENGTH OF CULVERT AT OBVERT (m): 5

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 4.5 PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 65.66

HEIGHT OF GUARDRAIL/HANDRAIL: None

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

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Unknown

HAS THE STRUCTURE BEEN UPGRADED? Unknown

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

ADDITIONAL COMMENTS:

Creek:	Moggill Creek
Location:	Kittani Street

ARI (AEP %)	**PEAK DISCHARGE (m3/s)**	**Peak U/S Water Level**	**Peak D/S Water Level**	**AFFLUX (m)**	**MAX WIDTH OF WEIR FLOW (m)**	**MAX DEPTH OF WEIR FLOW (m)**	**VELOCITY (m/s)**	
		(m AHD)					**Weir**	**Structure**
2000-yr (0.05%)	525.1	69.24	68.41	0.83	68	3.5	4.3	5.9
500-yr (0.2%)	313.5	68.42	67.78	0.64	59	2.7	3.8	5.3
100-yr (1%)	211.4	67.98	67.46	0.53	53	2.3	3.4	5.0
50-yr (2%)	173.2	67.81	67.29	0.52	51	2.1	3.1	4.8
20-yr (5%)	130.4	67.56	67.06	0.50	47	1.9	2.8	4.5
10-yr (10%)	99.5	67.35	66.85	0.50	45	1.7	2.5	4.4
5-yr (20%)	80.6	67.19	66.65	0.54	41	1.5	2.5	4.4
2-yr (50%)	53.5	66.89	66.32	0.57	35	1.2	2.3	4.2

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:	Moggill Creek
Location:	Kittani Street

Upstream of Kittani Street

Downstream of Kittani Street

Creek:	McKay Brook		**Immunity Rating:**	1% AEP
Location:	Brookfield Road			100-yr ARI

DATE OF SURVEY: N/A

UBD REF: 177 K7

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): C0291P

MODEL ID: S17

New AMTD (m): 490

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 3 / 1800 mm RCP

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

U/S INVERT LEVEL (m) 11.79 U/S OBVERT LEVEL (m) 13.59

D/S INVERT LEVEL (m) 11.57 D/S OBVERT LEVEL (m) 13.37

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 21 (approx)

LENGTH OF CULVERT AT OBVERT (m): 21 (approx)

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 16 (approx) PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 15.29

HEIGHT OF GUARDRAIL/HANDRAIL: 0.7 (armco)

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

PLAN NUMBER: W3363

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 1966

HAS THE STRUCTURE BEEN UPGRADED? Unknown

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

ADDITIONAL COMMENTS:

Creek:	McKay Brook
Location:	Brookfield Road

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	72.3	16.32	15.90	0.42	115	0.9	0.5	5.7
500-yr (0.2%)	45.5	15.92	14.16	1.76	88	0.5	0.2	5.5
100-yr (1%)	34.7	14.65	13.92	0.73	0	0.0	0.0	4.5
50-yr (2%)	29.3	14.34	13.78	0.56	0	0.0	0.0	3.8
20-yr (5%)	22.9	13.96	13.56	0.40	0	0.0	0.0	3.0
10-yr (10%)	18.2	13.65	13.40	0.25	0	0.0	0.0	2.6
5-yr (20%)	15.1	13.43	13.25	0.18	0	0.0	0.0	2.4
2-yr (50%)	10.3	13.10	12.98	0.12	0	0.0	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

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	McKay Brook
Location:	Brookfield Road

Upstream of Brookfield Road (McKay Brook)

Downstream of Brookfield Road (McKay Brook)

Creek:	McKay Brook	**Immunity Rating:**	<0.05% AEP
Location:	Mirbelia Street		>2000-yr ARI

DATE OF SURVEY: N/A	UBD REF: 177 K5
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID (Gecko): C0385B
MODEL ID: S18	New AMTD (m): 1082

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 5 / 3000 x 1200mm SLBC

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

U/S INVERT LEVEL (m) 20.14   U/S OBVERT LEVEL (m) 21.34

D/S INVERT LEVEL (m) 19.94   D/S OBVERT LEVEL (m) 21.14

For culverts give floor level   For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 22.5 (approx)

LENGTH OF CULVERT AT OBVERT (m): 22.5 (approx)

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 16.5   PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 22.34

HEIGHT OF GUARDRAIL/HANDRAIL: None

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

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Unknown

HAS THE STRUCTURE BEEN UPGRADED? Unknown

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

ADDITIONAL COMMENTS:

Creek:	McKay Brook
Location:	Mirbelia Street

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	59.2	22.22	21.71	0.51	0	0.0	0.0	5.0
500-yr (0.2%)	38.2	21.54	21.36	0.18	0	0.0	0.0	3.0
100-yr (1%)	27.5	21.27	21.14	0.13	0	0.0	0.0	2.7
50-yr (2%)	23.5	21.16	21.04	0.12	0	0.0	0.0	2.6
20-yr (5%)	18.4	21.00	20.91	0.09	0	0.0	0.0	2.4
10-yr (10%)	14.5	20.88	20.80	0.08	0	0.0	0.0	2.2
5-yr (20%)	12.1	20.79	20.71	0.08	0	0.0	0.0	2.1
2-yr (50%)	8.2	20.64	20.56	0.08	0	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

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	McKay Brook
Location:	Mirbelia Street

Downstream of Mirbelia Street

Creek:	McKay Brook		**Immunity Rating:**	<0.05% AEP
Location:	Tinarra Crescent			>2000-yr ARI

DATE OF SURVEY: N/A | UBD REF: 157 J18

SURVEYED CROSS SECTION ID: N/A | BCC ASSET ID (Gecko): Unknown

MODEL ID: S22 | New AMTD (m): 3445

STRUCTURE DESCRIPTION: Culvert

STRUCTURE SIZE: 1 / 1350 mm RCP

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

U/S INVERT LEVEL (m) 53.35 — U/S OBVERT LEVEL (m) 54.7

D/S INVERT LEVEL (m) 52.93 — D/S OBVERT LEVEL (m) 54.28

For culverts give floor level — For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 37.5

LENGTH OF CULVERT AT OBVERT (m): 37.5

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 9 (approx) — PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 59.01

HEIGHT OF GUARDRAIL/HANDRAIL: None

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

PLAN NUMBER: W7590

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 1991

HAS THE STRUCTURE BEEN UPGRADED? Unknown

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

ADDITIONAL COMMENTS:

Creek:	McKay Brook
Location:	Tinarra Crescent

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	8.0	57.71	53.40	4.31	0	0.0	0.0	5.6
500-yr (0.2%)	6.2	56.21	53.35	2.86	0	0.0	0.0	4.4
100-yr (1%)	4.7	55.36	53.28	2.08	0	0.0	0.0	3.3
50-yr (2%)	4.0	55.06	53.24	1.82	0	0.0	0.0	2.8
20-yr (5%)	3.2	54.81	53.18	1.63	0	0.0	0.0	2.4
10-yr (10%)	2.5	54.61	53.13	1.48	0	0.0	0.0	2.4
5-yr (20%)	2.1	54.45	53.09	1.36	0	0.0	0.0	2.4
2-yr (50%)	1.4	54.24	53.01	1.23	0	0.0	0.0	2.4

Notes:
Weir velocity is the average across the entire flooded width at peak flood level
Structure velocity is the peak within the culvert barrel
U/S and D/S water levels have been taken at the 1d channel extents

Creek:	McKay Brook
Location:	Tinarra Crescent

Upstream of Tinarra Crescent

Creek:	McKay Brook Trib	**Immunity Rating:**	50% AEP
Location:	Wexford Street		2-yr ARI

DATE OF SURVEY: N/A	UBD REF: 177 K3
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID (Gecko): C2130P
MODEL ID: S27	New AMTD (m): 305

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 2 / 600 mm RCP

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

U/S INVERT LEVEL (m) 30.4 U/S OBVERT LEVEL (m) 31

D/S INVERT LEVEL (m) 29.9 D/S OBVERT LEVEL (m) 30.5

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 20 (approx)

LENGTH OF CULVERT AT OBVERT (m): 20 (approx)

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 9 (approx) PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 31.56

HEIGHT OF GUARDRAIL/HANDRAIL: None

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

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Unknown

HAS THE STRUCTURE BEEN UPGRADED? Unknown

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

ADDITIONAL COMMENTS:

Creek:	McKay Brook Trib
Location:	Wexford Street

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	13.3	32.30	30.55	1.75	25	0.5	1.8	3.7
500-yr (0.2%)	8.3	32.15	30.43	1.72	25	0.4	1.3	3.6
100-yr (1%)	5.8	32.05	30.33	1.72	25	0.3	0.9	3.5
50-yr (2%)	4.9	31.99	30.28	1.71	20	0.3	1.0	3.4
20-yr (5%)	3.9	31.94	30.22	1.72	20	0.3	0.8	3.3
10-yr (10%)	3.1	31.89	30.16	1.73	20	0.2	0.6	3.3
5-yr (20%)	2.6	31.84	30.11	1.73	15	0.2	0.5	3.2
2-yr (50%)	1.6	31.54	30.02	1.52	0	0.0	0.0	2.9

Notes:

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

Structure velocity is the peak within the culvert barrel

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	McKay Brook Trib
Location:	Wexford Street

Upstream of Wexford Street

Downstream of Wexford Street

Creek:	Gap Creek	**Immunity Rating:**	1% AEP
Location:	Brookfield Road		100-yr ARI

DATE OF SURVEY: 2015 (U/S Cross-section)	UBD REF: 177 C2
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID (Gecko): B0350
MODEL ID: S28	New AMTD (m): 400

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: Single span

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

U/S INVERT LEVEL (m) 21.76 — U/S OBVERT LEVEL (m) 25.1

D/S INVERT LEVEL (m) 21.76 — D/S OBVERT LEVEL (m) 25.1

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? N/A N/A

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): 15.7 — PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 25.95

HEIGHT OF GUARDRAIL/HANDRAIL: 1.15

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

PLAN NUMBER: W11131

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 2000

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Gap Creek
Location:	Brookfield Road

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level (m AHD)	Peak D/S Water Level (m AHD)	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s) Weir	VELOCITY (m/s) Structure
2000-yr (0.05%)	170.9	26.59	26.43	0.16	187	0.5	1.9	3.1
500-yr (0.2%)	99.6	26.01	25.51	0.50	166	0.2	0.5	2.8
100-yr (1%)	73.1	25.40	25.09	0.31	0	0.0	0.0	2.7
50-yr (2%)	62.7	25.21	24.97	0.24	0	0.0	0.0	2.7
20-yr (5%)	48.3	24.76	24.71	0.05	0	0.0	0.0	2.6
10-yr (10%)	38.9	24.46	24.41	0.05	0	0.0	0.0	2.6
5-yr (20%)	31.6	24.16	24.11	0.05	0	0.0	0.0	2.6
2-yr (50%)	21.2	23.65	23.61	0.04	0	0.0	0.0	2.5

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
U/S and D/S water levels have been taken at the 1d channel extents

Creek:	Gap Creek
Location:	Brookfield Road

Upstream of Brookfield Road (Gap Creek)

Downstream of Brookfield Road (Gap Creek)

Creek:	Gold Creek		**Immunity Rating:**	5% AEP
Location:	Savages Road			20-yr ARI

DATE OF SURVEY: N/A

UBD REF: 136 R19

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): B1750

MODEL ID: S34

New AMTD (m): 620

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: Two span

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

U/S INVERT LEVEL (m) 31.29 U/S OBVERT LEVEL (m) 34.82

D/S INVERT LEVEL (m) 31.29 D/S OBVERT LEVEL (m) 34.82

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? N/A N/A

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): 8.2 PIER WIDTH (m): 0.46

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

LOWEST POINT OF WEIR (m AHD): 35.28

HEIGHT OF GUARDRAIL/HANDRAIL: 0.7 (armco)

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

PLAN NUMBER: W2313

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 1962

HAS THE STRUCTURE BEEN UPGRADED? Unknown

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

ADDITIONAL COMMENTS:

Creek:	Gold Creek
Location:	Savages Road

ARI (AEP %)	**PEAK DISCHARGE (m3/s)**	**Peak U/S Water Level**	**Peak D/S Water Level**	**AFFLUX (m)**	**MAX WIDTH OF WEIR FLOW (m)**	**MAX DEPTH OF WEIR FLOW (m)**	**VELOCITY (m/s)**	
		(m AHD)					**Weir**	**Structure**
2000-yr (0.05%)	349.8	36.75	36.35	0.40	120	1.4	2.6	3.1
500-yr (0.2%)	203.9	36.46	35.71	0.75	105	1.1	1.4	3.1
100-yr (1%)	145.1	36.10	35.41	0.69	84	0.7	0.8	3.0
50-yr (2%)	121.6	35.82	35.21	0.61	67	0.5	0.6	2.8
20-yr (5%)	93.7	35.30	34.88	0.42	0	0.0	0.0	2.7
10-yr (10%)	75.1	34.65	34.61	0.04	0	0.0	0.0	2.0
5-yr (20%)	61.1	34.42	34.39	0.03	0	0.0	0.0	1.8
2-yr (50%)	39.3	34.00	33.98	0.02	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 a peak average across the bridge opening

Creek:	Gold Creek
Location:	Savages Road

Upstream of Savages Road

Downstream of Savages Road

Creek:	Gold Creek	**Immunity Rating:**	>50% AEP
Location:	Adavale Street		<2-yr ARI

DATE OF SURVEY: N/A	UBD REF: 136 R18
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID (Gecko): C0106B
MODEL ID: S35	New AMTD (m): 750

STRUCTURE DESCRIPTION: Multi-cell Culvert

STRUCTURE SIZE: 3 / 3200 x 1500 mm RCBC

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

U/S INVERT LEVEL (m) 32.49   U/S OBVERT LEVEL (m) 33.99

D/S INVERT LEVEL (m) 32.4   D/S OBVERT LEVEL (m) 33.9

For culverts give floor level   For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 10

LENGTH OF CULVERT AT OBVERT (m): 10

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 9.3   PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 34.42 (at structure)

HEIGHT OF GUARDRAIL/HANDRAIL: 0.7 (armco)

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

PLAN NUMBER: W6756

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Unknown

HAS THE STRUCTURE BEEN UPGRADED? Unknown

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

ADDITIONAL COMMENTS:

Creek:	Gold Creek
Location:	Adavale Street

ARI (AEP %)	**PEAK DISCHARGE (m3/s)**	**Peak U/S Water Level**	**Peak D/S Water Level**	**AFFLUX (m)**	**MAX WIDTH OF WEIR FLOW (m)**	**MAX DEPTH OF WEIR FLOW (m)**	**VELOCITY (m/s)**	
		(m AHD)					**Weir**	**Structure**
2000-yr (0.05%)	349.8	37.36	37.36	0.00	N/A	3.3	N/A	2.7
500-yr (0.2%)	203.9	36.86	36.86	0.00	N/A	2.8	N/A	2.7
100-yr (1%)	145.1	36.50	36.50	0.00	N/A	2.5	N/A	2.7
50-yr (2%)	121.6	36.28	36.28	0.00	N/A	2.2	N/A	2.7
20-yr (5%)	93.7	35.92	35.92	0.00	N/A	1.8	N/A	2.7
10-yr (10%)	75.1	35.57	35.53	0.04	N/A	1.4	N/A	2.7
5-yr (20%)	61.1	35.40	35.28	0.12	N/A	1.2	N/A	2.7
2-yr (50%)	39.3	34.90	34.77	0.13	N/A	0.6	N/A	2.3

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:	Gold Creek
Location:	Adavale Street

Upstream of Adavale Street

Downstream of Adavale Street

Creek:	Gold Creek
Location:	272 Gold Creek Road

Immunity Rating:	2% AEP
	50-yr ARI

DATE OF SURVEY: N/A

UBD REF: 136 R17

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): N/A Private

MODEL ID: S36

New AMTD (m): 1990

STRUCTURE DESCRIPTION: Arch Bridge

STRUCTURE SIZE: Single span

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

U/S INVERT LEVEL (m) 37.72
U/S OBVERT LEVEL (m) 39.85 to 42.8

D/S INVERT LEVEL (m) 37.72
D/S OBVERT LEVEL (m) 39.85 to 42.8

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? N/A N/A

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): 4.5
PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 42.54

HEIGHT OF GUARDRAIL/HANDRAIL: 1.2 (approx)

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

PLAN NUMBER: N/A Private

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 1998

HAS THE STRUCTURE BEEN UPGRADED? Unknown

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

ADDITIONAL COMMENTS:

Creek:	Gold Creek
Location:	272 Gold Creek Road

ARI (AEP %)	**PEAK DISCHARGE (m3/s)**	**Peak U/S Water Level**	**Peak D/S Water Level**	**AFFLUX (m)**	**MAX WIDTH OF WEIR FLOW (m)**	**MAX DEPTH OF WEIR FLOW (m)**	**VELOCITY (m/s)**	
		(m AHD)					**Weir**	**Structure**
2000-yr (0.05%)	321.3	44.55	44.23	0.32	89	1.5	3.7	2.8
500-yr (0.2%)	186.2	43.77	43.11	0.66	67	0.7	3.2	2.8
100-yr (1%)	132.6	43.14	42.49	0.65	57	0.2	1.8	3.3
50-yr (2%)	110.0	42.70	42.18	0.52	0	0.0	0.0	2.6
20-yr (5%)	84.7	42.04	41.75	0.29	0	0.0	0.0	2.2
10-yr (10%)	67.9	41.59	41.40	0.19	0	0.0	0.0	1.9
5-yr (20%)	55.5	41.21	41.08	0.13	0	0.0	0.0	1.7
2-yr (50%)	36.1	40.52	40.48	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 a peak average across the bridge opening

Creek:	Gold Creek
Location:	272 Gold Creek Road

Upstream of 272 Gold Creek Road

Creek:	Gold Creek	**Immunity Rating:**	1% AEP
Location:	Gold Creek Road #1		100-yr ARI

DATE OF SURVEY: N/A

UBD REF: 136 R16

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): B0850

MODEL ID: S37

New AMTD (m): 2690

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: Single span

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

U/S INVERT LEVEL (m) 41.99 U/S OBVERT LEVEL (m) 46.4

D/S INVERT LEVEL (m) 41.99 D/S OBVERT LEVEL (m) 46.4

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? N/A N/A

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): 8.6 PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 47.1

HEIGHT OF GUARDRAIL/HANDRAIL: 1.2 (approx)

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

PLAN NUMBER: W3595

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 1970

HAS THE STRUCTURE BEEN UPGRADED? Unknown

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

ADDITIONAL COMMENTS:

Creek:	Gold Creek
Location:	Gold Creek Road #1

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level (m AHD)	Peak D/S Water Level (m AHD)	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s) Weir	VELOCITY (m/s) Structure
2000-yr (0.05%)	325.6	48.55	47.47	1.08	106	1.5	1.9	3.8
500-yr (0.2%)	190.2	47.71	46.64	1.07	88	0.6	0.6	3.7
100-yr (1%)	134.1	46.54	46.19	0.35	0	0.0	0.0	3.0
50-yr (2%)	110.9	46.02	45.95	0.07	0	0.0	0.0	2.7
20-yr (5%)	85.0	45.66	45.60	0.06	0	0.0	0.0	2.4
10-yr (10%)	68.4	45.38	45.33	0.05	0	0.0	0.0	2.1
5-yr (20%)	55.6	45.13	45.10	0.03	0	0.0	0.0	1.9
2-yr (50%)	36.2	44.66	44.64	0.02	0	0.0	0.0	1.6

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:	Gold Creek
Location:	Gold Creek Road #1

Upstream of Gold Creek Road #1

Downstream of Gold Creek Road #1

Creek:	Gold Creek		**Immunity Rating:**	>50% AEP
Location:	Gold Creek Road #2			<2-yr ARI

DATE OF SUPERVEY: N/A

UBD REF: 136 Q16

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): Unknown

MODEL ID: S40

New AMTD (m): 4500

STRUCTURE DESCRIPTION: Culvert

STRUCTURE SIZE: 1 / 2700 x 1800 mm RCBC

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

U/S INVERT LEVEL (m) 53.62 U/S OBVERT LEVEL (m) 55.42

D/S INVERT LEVEL (m) 53.5 D/S OBVERT LEVEL (m) 55.3

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 12

LENGTH OF CULVERT AT OBVERT (m): 12

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 10.5 (approx) PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 55.95

HEIGHT OF GUARDRAIL/HANDRAIL: 0.7 (armco)

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

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Unknown

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Gold Creek
Location:	Gold Creek Road #2

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level (m AHD)	Peak D/S Water Level (m AHD)	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s) Weir	VELOCITY (m/s) Structure
2000-yr (0.05%)	285.7	58.60	58.14	0.46	70	1.9	3.9	7.7
500-yr (0.2%)	167.3	57.99	57.32	0.67	62	1.5	3.1	7.7
100-yr (1%)	117.2	57.70	56.74	0.96	61	1.4	2.2	7.7
50-yr (2%)	97.6	57.57	56.49	1.08	57	1.3	2.0	7.5
20-yr (5%)	75.5	57.39	56.13	1.26	56	1.2	1.7	7.3
10-yr (10%)	60.4	57.26	55.87	1.39	55	1.1	1.4	7.1
5-yr (20%)	49.0	57.13	55.67	1.46	54	1.0	1.1	6.9
2-yr (50%)	31.8	56.88	55.24	1.64	48	0.8	0.7	6.5

Notes:

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

Structure velocity is the peak within the culvert barrel

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	Gold Creek
Location:	Gold Creek Road #2

Upstream of Gold Creek Road #2

Downstream of Gold Creek Road #2

Creek:	Gold Creek	**Immunity Rating:**	>50% AEP
Location:	Gold Creek Road #3		<2-yr ARI

DATE OF SURVEY: N/A	UBD REF: 136 P16
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID (Gecko): C0102B
MODEL ID: S41	New AMTD (m): 4895

STRUCTURE DESCRIPTION: Culvert

STRUCTURE SIZE: 1 / 2700 x 1800 mm RCBC

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

U/S INVERT LEVEL (m) 56.24 | U/S OBVERT LEVEL (m) 58.04

D/S INVERT LEVEL (m) 56.24 | D/S OBVERT LEVEL (m) 58.04

For culverts give floor level | For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 17.6

LENGTH OF CULVERT AT OBVERT (m): 17.6

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 17.6 (on skew) | PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 58.3

HEIGHT OF GUARDRAIL/HANDRAIL: 0.7 (armco)

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

PLAN NUMBER: W8085

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Unknown

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Gold Creek
Location:	Gold Creek Road #3

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level (m AHD)	Peak D/S Water Level (m AHD)	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s) Weir	VELOCITY (m/s) Structure
2000-yr (0.05%)	249.5	60.42	59.42	1.00	115	1.7	2.4	5.4
500-yr (0.2%)	144.8	60.01	58.93	1.08	90	1.3	2.0	5.3
100-yr (1%)	102.2	59.77	58.72	1.05	80	1.2	1.7	5.1
50-yr (2%)	85.4	59.64	58.61	1.03	75	1.1	1.5	5.0
20-yr (5%)	66.3	59.46	58.47	0.99	70	0.9	1.3	4.9
10-yr (10%)	53.1	59.33	58.32	1.01	65	0.8	1.1	4.8
5-yr (20%)	43.1	59.20	58.15	1.05	60	0.7	0.9	4.7
2-yr (50%)	27.9	59.00	57.78	1.22	30	0.5	1.4	3.6

Notes:

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

Structure velocity is the peak within the culvert barrel

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	Gold Creek
Location:	Gold Creek Road #3

Upstream of Gold Creek Road #3

Downstream of Gold Creek Road #3

Creek:	Gold Creek	**Immunity Rating:**	>50% AEP
Location:	Gold Creek Road #6		<2-yr ARI

DATE OF SUBVEY: N/A

UBD REF: 136 M15

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): Unknown

MODEL ID: S44

New AMTD (m): 6655

STRUCTURE DESCRIPTION: Culvert

STRUCTURE SIZE: 1 / 1800 x 600 mm RCBC

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

U/S INVERT LEVEL (m) 68.7 U/S OBVERT LEVEL (m) 69.3

D/S INVERT LEVEL (m) 68.5 D/S OBVERT LEVEL (m) 69.1

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 13.2

LENGTH OF CULVERT AT OBVERT (m): 13.2

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 13.2 (on skew) PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 69.55

HEIGHT OF GUARDRAIL/HANDRAIL: None

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

PLAN NUMBER: W11563-1

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 2001

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Gold Creek
Location:	Gold Creek Road #6

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	248.6	72.57	72.57	0.00	65	2.9	2.6	N/A (not modelled)
500-yr (0.2%)	144.4	71.96	71.86	0.10	56	2.2	2.3	5.1
100-yr (1%)	103.2	71.48	71.45	0.03	32	1.7	3.7	5.1
50-yr (2%)	86.2	71.24	71.23	0.01	27	1.5	4.3	5.1
20-yr (5%)	67.3	70.96	70.94	0.02	23	1.1	5.4	5.1
10-yr (10%)	53.2	70.83	70.67	0.16	23	0.8	5.8	5.1
5-yr (20%)	42.8	70.71	70.44	0.27	22	0.6	5.9	5.1
2-yr (50%)	27.8	70.47	70.04	0.43	21	0.5	4.8	5.1

Notes:

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

Structure velocity is the peak within the culvert barrel

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	Gold Creek
Location:	Gold Creek Road #6

Upstream of Gold Creek Road #6

Downstream of Gold Creek Road #6

Creek:	Gold Creek		**Immunity Rating:**	>50% AEP
Location:	Gold Creek Road #7			<2-yr ARI

DATE OF SURVEY: N/A

UBD REF: 136 M14

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): Unknown

MODEL ID: S45

New AMTD (m): 6955

STRUCTURE DESCRIPTION: Culvert

STRUCTURE SIZE: 1 / 1800 x 600 mm RCBC

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

U/S INVERT LEVEL (m) 71.35 U/S OBVERT LEVEL (m) 71.95

D/S INVERT LEVEL (m) 71.2 D/S OBVERT LEVEL (m) 71.8

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 10.8

LENGTH OF CULVERT AT OBVERT (m): 10.8

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 10.8 (on skew) PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 72.4

HEIGHT OF GUARDRAIL/HANDRAIL: None

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

PLAN NUMBER: W11564-1

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 2000

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Gold Creek
Location:	Gold Creek Road #7

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	249.9	75.07	74.57	0.50	61	2.2	3.6	5.7
500-yr (0.2%)	145.1	74.34	73.88	0.46	56	1.5	3.3	5.8
100-yr (1%)	104.2	74.01	73.52	0.49	47	1.2	3.7	5.7
50-yr (2%)	87.0	73.86	73.35	0.51	46	1.0	3.5	5.7
20-yr (5%)	67.4	73.68	73.14	0.54	45	0.9	3.3	5.8
10-yr (10%)	53.2	73.55	72.95	0.60	41	0.8	3.2	5.7
5-yr (20%)	43.0	73.46	72.79	0.67	40	0.7	3.0	5.7
2-yr (50%)	27.8	73.29	72.47	0.82	32	0.5	2.9	5.6

Notes:

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

Structure velocity is the peak within the culvert barrel

U/S and D/S water levels have been taken at the 1d channel extents

Creek:	Gold Creek
Location:	Gold Creek Road #7

Upstream of Gold Creek Road #7

Downstream of Gold Creek Road #7

Creek:	Gold Creek		**Immunity Rating:**	>50% AEP
Location:	Gold Creek Road #8			<2-yr ARI

DATE OF SURVEY: N/A

UBD REF: 136 M14

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID (Gecko): C1454B

MODEL ID: S46

New AMTD (m): 7100

STRUCTURE DESCRIPTION: Culvert

STRUCTURE SIZE: 1 / 1200 x 600 mm RCBC

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

U/S INVERT LEVEL (m) 72.32 U/S OBVERT LEVEL (m) 72.92

D/S INVERT LEVEL (m) 72.16 D/S OBVERT LEVEL (m) 72.76

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 10.8

LENGTH OF CULVERT AT OBVERT (m): 10.8

TYPE OF LINING: concrete

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

IS THERE A SURVEYED WEIR PROFILE? N/A N/A

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): 10.8 (on skew) PIER WIDTH (m): N/A

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

LOWEST POINT OF WEIR (m AHD): 73.42

HEIGHT OF GUARDRAIL/HANDRAIL: None

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

PLAN NUMBER: W11565-1

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 including abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: Circa 2000

HAS THE STRUCTURE BEEN UPGRADED? Yes

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

ADDITIONAL COMMENTS:

Creek:	Gold Creek
Location:	Gold Creek Road #8

ARI (AEP %)	PEAK DISCHARGE (m3/s)	Peak U/S Water Level	Peak D/S Water Level	AFFLUX (m)	MAX WIDTH OF WEIR FLOW (m)	MAX DEPTH OF WEIR FLOW (m)	VELOCITY (m/s)	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	249.9	76.61	76.43	0.18	60	3.0	2.7	4.4
500-yr (0.2%)	145.1	75.88	75.67	0.21	52	2.3	2.4	4.5
100-yr (1%)	104.2	75.48	75.27	0.21	46	1.9	2.4	4.4
50-yr (2%)	87.0	75.29	75.10	0.19	40	1.7	2.5	4.4
20-yr (5%)	67.4	75.05	74.86	0.19	38	1.5	2.4	4.4
10-yr (10%)	53.2	74.85	74.64	0.21	36	1.2	2.3	4.4
5-yr (20%)	43.0	74.65	74.45	0.20	34	1.1	2.3	4.4
2-yr (50%)	27.8	74.36	74.15	0.21	30	0.8	2.4	4.4

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:	Gold Creek
Location:	Gold Creek Road #8

Upstream of Gold Creek Road #8

Downstream of Gold Creek Road #8

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Appendix I: External Peer Review Documentation

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BMT WBM Pty Ltd
Level 8, 200 Creek Street
Brisbane Qld 4000
Australia
PO Box 203, Spring Hill 4004

Tel: +61 7 3831 6744
Fax: + 61 7 3832 3627

ABN 54 010 830 421

www.bmtwbm.com.au

Our Ref: L.B20679.007.Moggill_Creek.docx

2 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: MOGGILL CREEK FLOOD MODELLING PEER REVIEW

Background

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

The review was undertaken at two stages, firstly following calibration and secondly following design event modelling. At the commencement of these two review stages, Council submitted the following data to BMT WBM:

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

These data were reviewed and initial feedback on the calibration modelling was provided to Council with minor suggestions. These suggestions were implemented, and the design event modelling was subsequently provided for review. Generally, no concerns with the models have been identified.

Overview of the Modelling Approach

Hydrological models were developed using URBS. The structure of the URBS models and the sub-catchment parameters has been reviewed. The URBS model parameters have been appropriately applied and are within the standard values for URBS models. The Design event rainfall IFD used in the URBS model is appropriate for the catchment. The CC1 and CC2 climate change scenarios are stated as being 10% and 20% increases in rainfall intensity which is consistent with their respective IFD tables when compared to the base case IDF table. The only comment in relation to the setup of the models is that in the pluviography rainfall files there is no need for the total rainfall depth of each subarea to been stated when they are all receiving the same total depth, in this case only one value is required.

Hydraulic models of the creeks in the study area were developed using TUFLOW. A 5m computational grid cell size was used. The upper and middle reaches of the creeks were mostly modelled in 1D and linked to the 2D model domain of the floodplain. The lower reach of Moggill Creek, south of Moggill Road, 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.

While there are some large discrepancies in modelled peak flood levels compared to MHG gauges (beyond the tolerances stated above), in the context of the overall comparison across the four historical events, spatial variations of discrepancies, potential for debris blockage at structures during the flood events and rainfall data limitations, the calibration appears reasonable.

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.

Conclusion

The flood modelling undertaken as part of the Moggill Creek Flood Study complies with current industry practice, and is considered suitable for the purposes of the study.

Yours Faithfully
BMT WBM

Richard Sharpe
Senior Flood Engineer

Ben Caddis RPEQ (9234)

Supervising Engineer[1]:

[1] *The review of the hydrologic modelling was undertaken by Eoghain O'Hanlon and the hydraulic modelling by Richard Sharpe. Both Eoghain and Richard were supervised by RPEQ Ben Caddis.*

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Appendix J: Rare Events (Scenario 1) - Peak Flood Levels

The flood level data presented in this Appendix has been extracted (in part) from the results of a 2-dimensional flood model. Levels presented have been extracted generally at selected points along the centreline of the waterway with the intent of demonstrating general flood characteristics. The applicability of this data to locations on the floodplains adjacent should be determined by a suitably qualified professional. It is recommended for any detailed assessment of flood risk associated with the waterway that complete flood model results be accessed and interrogated.

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AMTD (m)	Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)		
	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)	2000-yr ARI (0.05 % AEP)
Moggill Creek			
0	2.10	2.11	4.67
100	2.97	3.45	5.14
200	3.56	4.09	6.00
300	3.82	4.37	6.26
400	3.98	4.53	6.42
500	3.97	4.53	6.65
600	4.67	5.36	7.15
700	5.13	5.67	7.22
800	5.20	5.73	7.36
900	5.34	5.95	7.71
1000	5.57	6.09	7.74
1100	5.55	6.07	7.73
1200	5.60	6.10	7.72
1300	5.75	6.24	7.84
1400	5.92	6.41	7.98
1500	6.11	6.58	8.13
1600	6.21	6.67	8.14
1700	6.32	6.78	8.28
1800	6.58	7.11	8.69
1900	6.70	7.22	8.75
2000	6.97	7.46	8.78
2100	7.04	7.49	8.77
2200	7.39	7.93	9.33
2300	7.46	7.96	9.29
2400	7.46	7.96	9.29
2500	7.46	7.96	9.29
2600	7.46	7.96	9.29
2700	7.44	7.95	9.32
2800	7.49	8.01	9.58
2900	8.06	8.63	10.05
Moggill Road (S1 & S2)			
3020	9.77	10.38	11.72

AMTD (m)	Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)		
	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)	2000-yr ARI (0.05 % AEP)
3100	9.91	10.51	11.87
3200	9.98	10.56	11.89
3300	10.13	10.74	12.18
3400	10.29	10.92	12.47
3500	10.37	10.99	12.57
3600	10.44	11.06	12.65
3700	10.54	11.16	12.75
3800	10.66	11.27	12.87
3900	10.72	11.32	12.90
4000	10.76	11.34	12.90
4100	11.13	11.72	13.27
4200	11.57	12.17	13.72
4300	11.94	12.55	14.09
4400	12.03	12.65	14.20
4500	12.25	12.85	14.43
4600	12.43	12.99	14.57
4700	12.56	13.08	14.60
4800	12.77	13.26	14.75
4900	13.00	13.46	14.92
5000	13.23	13.66	15.09
5100	13.87	14.31	15.69
5200	14.16	14.59	15.95
5300	14.30	14.74	16.08
Branton Street Footbridge (S4)			
5400	14.45	14.89	16.29
5500	14.63	15.06	16.43
5600	14.90	15.33	16.69
5700	15.23	15.67	17.03
5800	15.56	16.00	17.37
5900	15.84	16.30	17.70
6000	16.09	16.56	18.03
6100	16.35	16.83	18.36
6200	16.49	16.97	18.47

AMTD (m)	Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)		
	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)	2000-yr ARI (0.05 % AEP)
6300	16.65	17.13	18.58
6400	17.02	17.45	18.73
6500	17.41	17.78	18.88
6600	17.67	18.03	19.08
6700	17.92	18.27	19.30
6800	18.20	18.57	19.64
6900	18.50	18.86	19.98
7000	18.82	19.20	20.35
7100	19.19	19.58	20.75
7200	19.55	19.96	21.14
7300	19.87	20.31	21.55
7400	20.10	20.51	21.72
7500	20.38	20.79	21.99
7600	20.73	21.13	22.38
7700	20.99	21.38	22.61
7800	21.19	21.54	22.70
7900	21.39	21.71	22.80
8000	21.60	21.89	22.93
Rafting Ground Road #1 (S6)			
8145	22.26	22.58	23.59
8200	22.44	22.77	23.76
8300	22.72	23.04	23.99
8400	22.93	23.24	24.16
8500	23.16	23.47	24.39
8595	23.40	23.72	24.65
Rafting Ground Road #2 (S7)			
8700	24.23	24.63	25.69
8800	24.70	25.10	26.17
8900	24.90	25.28	26.34
9000	24.98	25.35	26.37
9100	25.03	25.39	26.39
9200	25.47	25.81	26.77
9300	25.85	26.17	27.09

AMTD (m)	Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)		
	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)	2000-yr ARI (0.05 % AEP)
9400	26.09	26.38	27.27
9500	26.34	26.60	27.45
9600	26.83	27.05	27.83
Brookfield Road (S9)			
9700	27.76	28.00	28.66
9800	27.86	28.10	28.76
9900	27.98	28.23	28.91
10000	28.13	28.39	29.10
10100	28.29	28.56	29.30
10200	28.48	28.76	29.52
10300	28.74	29.03	29.81
10400	29.00	29.31	30.11
10500	29.27	29.61	30.48
10600	29.53	29.91	30.84
10700	29.82	30.22	31.21
10800	30.24	30.63	31.62
10900	30.67	31.04	32.03
11000	31.40	31.78	32.88
11100	32.05	32.53	33.73
Bundeleer Road (S10)			
11200	33.11	33.59	34.79
11300	33.68	34.09	35.20
11400	34.09	34.49	35.54
11500	34.46	34.84	35.86
11600	34.74	35.13	36.19
11700	35.14	35.52	36.52
11800	35.55	35.92	36.85
11900	36.07	36.45	37.38
12000	36.58	36.97	37.90
12100	37.01	37.37	38.27
12200	37.43	37.76	38.62
12300	38.00	38.33	39.22
12400	38.72	39.09	40.06

AMTD (m)	Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)		
	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)	2000-yr ARI (0.05 % AEP)
12500	39.49	39.89	40.95
12600	40.30	40.62	41.52
12700	40.56	40.88	41.78
12800	40.85	41.20	42.10
185 Upper Brookfield Road (S11)			
12907	41.76	42.13	43.00
13000	42.30	42.65	43.54
Upper Brookfield Road #1 (S12)			
13100	43.37	44.02	45.69
13200	43.61	44.20	45.81
13300	43.93	44.47	45.98
13400	44.38	44.86	46.24
13500	45.16	45.56	46.76
Haven Road (S13)			
13600	46.27	46.62	47.53
13700	46.73	47.05	47.92
13800	47.19	47.49	48.31
13900	48.00	48.34	49.21
14000	48.84	49.19	49.93
14100	49.26	49.63	50.48
14200	50.05	50.49	51.60
14300	50.86	51.36	52.68
14400	51.75	52.23	53.55
14500	52.75	53.24	54.46
14600	53.88	54.31	55.51
14700	54.22	54.66	55.86
Upper Brookfield Road #2 (S15)			
14800	55.98	56.58	57.53
14900	56.07	56.65	57.64
15000	56.48	56.99	58.00
15100	56.87	57.31	58.31
15200	57.40	57.81	58.82
15300	58.09	58.49	59.59

AMTD (m)	Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)		
	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)	2000-yr ARI (0.05 % AEP)
15400	58.74	59.14	60.32
15500	59.10	59.51	60.72
15600	59.54	59.99	61.15
15700	59.99	60.85	61.54
15800	60.85	61.44	62.25
15900	61.73	62.06	62.96
16000	62.53	62.80	63.60
16100	63.21	63.45	64.17
16200	63.61	63.85	64.55
16300	64.22	64.53	65.28
16400	64.76	65.04	65.75
16500	65.25	65.46	66.06
16600	65.84	66.04	66.59
16700	66.49	66.66	67.15
16800	67.01	67.19	67.75
16900	67.52	67.72	68.35
Kittani Street (S16)			
17000	68.43	68.71	69.55
17088	68.74	69.04	69.89
Gold Creek			
0	32.03	32.51	33.71
100	32.45	32.92	34.19
200	32.92	33.33	34.43
300	33.37	33.76	34.76
400	34.16	34.61	35.65
500	34.52	34.95	35.83
600	35.43	35.63	36.27
Savages Road (S34)			
700	36.45	36.68	37.08
Adavale Street (S35)			
800	36.71	36.92	37.40
900	36.95	37.14	37.61
1000	37.60	37.82	38.32

AMTD (m)	Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)		
	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)	2000-yr ARI (0.05 % AEP)
1100	38.54	38.83	39.39
1200	39.32	39.60	40.19
1300	40.03	40.26	40.87
1400	40.51	40.71	41.34
1500	40.66	40.87	41.53
1600	41.01	41.27	42.06
1700	41.54	41.90	42.91
1800	41.92	42.27	43.29
1900	42.29	42.63	43.64
272 Gold Creek Road (S36)			
2000	43.42	43.78	44.57
2100	43.75	44.10	45.06
2200	43.97	44.28	45.22
2300	44.38	44.65	45.51
2400	45.01	45.28	46.08
2500	45.43	45.69	46.43
2600	45.86	46.10	46.77
Gold Creek Road #1 (S37)			
2700	47.10	47.74	48.57
2800	47.60	48.07	48.83
2900	47.95	48.39	49.19
3000	48.30	48.70	49.55
3100	48.88	49.23	50.10
3200	49.71	50.00	50.86
3300	50.23	50.48	51.25
3400	50.66	50.85	51.45
3500	51.03	51.16	51.64
3600	51.47	51.60	52.02
3700	52.02	52.20	52.62
3800	52.80	53.01	53.47
3900	53.04	53.28	53.86
4000	53.48	53.79	54.49
4100	54.11	54.43	55.14

AMTD (m)	Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)		
	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)	2000-yr ARI (0.05 % AEP)
4217	55.22	55.52	56.18
4300	55.65	55.99	56.82
4400	56.39	56.73	57.59
Gold Creek Road #2 (S40)			
4517	57.79	58.00	58.61
4600	57.92	58.14	58.76
4700	58.08	58.29	58.92
4800	58.53	58.69	59.23
Gold Creek Road #3 (S41)			
4924	59.87	60.02	60.43
5000	60.33	60.53	61.08
5100	60.86	61.10	61.78
5200	61.45	61.71	62.51
5300	62.03	62.26	62.97
5400	62.60	62.78	63.33
5500	63.11	63.34	63.88
5600	63.70	63.92	64.49
5700	64.29	64.50	65.10
5790	64.81	65.02	65.64
Gold Creek Road #4 (S42)			
5900	65.99	66.25	66.87
6000	66.71	66.97	67.66
6100	67.30	67.54	68.30
6200	68.13	68.37	69.20
6274	68.69	68.90	69.61
Gold Creek Road #5 (S43)			
6400	70.35	70.57	71.33
6500	70.89	71.15	71.90
6600	71.45	71.73	72.44
Gold Creek Road #6 (S44)			
6700	71.89	72.15	72.77
6800	72.86	73.13	73.81
6900	73.44	73.68	74.36

AMTD (m)	Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)		
	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)	2000-yr ARI (0.05 % AEP)
Gold Creek Road #7 (S45)			
7000	74.55	74.76	75.54
7088	75.40	75.67	76.43
Gold Creek Road #8 (S46)			
7200	76.47	76.73	77.56
7300	77.36	77.61	78.31
7310	77.73	78.03	78.95
Gap Creek			
0	24.83	25.22	26.28
100	24.92	25.30	26.35
200	24.95	25.33	26.37
300	25.10	25.43	26.40
Brookfield Road (S28)			
421	25.68	26.02	26.60
500	26.04	26.32	26.84
600	26.57	26.78	27.25
700	27.06	27.24	27.71
800	27.55	27.69	28.17
900	28.19	28.35	28.79
1000	28.94	29.17	29.56
1100	29.68	29.91	30.46
1200	30.43	30.67	31.37
1300	31.60	31.85	32.65
1400	32.43	32.75	33.78
1500	33.20	33.55	34.61
1600	33.92	34.23	35.09
1700	34.63	34.89	35.57
1800	35.35	35.57	36.06
1900	35.81	36.01	36.45
2000	36.31	36.46	36.85
Gap Creek Road (S29, S30 & S31)			
2100	37.52	37.65	38.11
2200	38.07	38.28	38.78

AMTD (m)	Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)		
	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)	2000-yr ARI (0.05 % AEP)
2300	38.66	38.89	39.44
2400	39.18	39.37	39.86
2500	39.73	39.90	40.33
2600	40.38	40.56	40.99
2700	41.05	41.24	41.69
2800	41.71	41.92	42.39
2900	42.36	42.59	43.25
3000	43.01	43.24	43.97
3090	43.60	43.82	44.55
McKay Brook			
0	13.77	14.22	15.61
100	14.01	14.47	15.89
200	14.01	14.47	15.89
300	14.01	14.47	15.90
400	14.02	14.48	15.90
Brookfield Road (S17)			
510	15.33	15.93	16.32
600	15.46	15.98	16.39
700	16.50	16.65	16.80
800	17.51	17.63	17.89
900	19.00	19.16	19.55
1000	21.03	21.16	21.49
Mirbelia Street (S18)			
1100	21.39	21.56	22.24
1200	23.22	23.35	23.68
1300	24.41	24.54	24.85
1400	25.23	25.36	25.71
1500	26.32	26.50	26.95
1600	28.02	28.23	28.82
1700	28.69	28.80	29.09
1800	29.60	29.74	30.11
1900	30.98	31.08	31.34
2000	32.11	32.20	32.42

AMTD (m)	Scenario 1 (Existing Waterway Conditions) Peak Water Levels (mAHD)		
	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)	2000-yr ARI (0.05 % AEP)
2100	33.14	33.23	33.47
2200	35.50	35.59	35.82
2300	36.15	36.25	36.52
2400	37.60	37.70	37.96
2500	38.63	38.73	39.02
2600	40.12	40.20	40.39
2700	41.55	41.64	41.84
2800	N/A refer Note (1)	N/A refer Note (1)	N/A refer Note (1)
Hillcrest Place (S20 & S21)			
2900	45.45	45.60	45.84
3000	46.19	46.31	46.55
3100	47.49	47.57	47.78
3200	49.20	49.27	49.47
3300	50.85	50.90	50.98
3400	52.97	53.01	53.06
Tinarra Crescent (S22)			
3500	55.68	56.21	57.71
3600	56.29	56.52	57.72
3700	57.93	58.02	58.23
3800	59.82	59.89	60.09
3900	61.06	61.12	61.26
3986	63.55	63.61	63.78
McKay Brook Tributary			
0	24.87	24.99	25.32
100	27.07	27.15	27.35
200	29.00	29.03	29.12
280	30.24	30.29	30.41
Wexford Street (S27)			
403	32.58	32.65	32.78

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

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Appendix K: Modelling User Guide

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Moggill Creek Flood Study

Model User Guide

Prepared by Brisbane City Council's, City Projects Office

June 2016

BRISBANE CITY

Dedicated to a better Brisbane

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Table of Contents

List of Tables

1.0 Introduction

1.1 Moggill Creek Flood Study (2016)

This document is to be read in conjunction with the Moggill Creek Flood Study - Volume 1 (2016).

The Moggill Creek Flood Study (2016) incorporates the calibration and verification of the hydrologic and hydraulic models; design event modelling; extreme event modelling and sensitivity modelling. Hydrologic and hydraulic models have been developed using the URBS and TUFLOW modelling software respectively.

Calibration of the URBS and TUFLOW models was undertaken utilising three historical storms; namely May 2015, May 2009 and November 2008. Verification of the URBS and TUFLOW models utilised the January 2013 historical storm event.

Design and extreme flood magnitudes were estimated for the full range of events from 2-yr ARI (50 % AEP) to PMF. These analyses assumed hydrologic ultimate catchment development conditions in accordance with the current version of BCC City Plan.

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 "Modelled Flood Corridor" in order to simulate potential development.

A sensitivity analysis was undertaken to understand the impacts of climate variability for two planning horizons; namely 2050 and 2100.

1.2 Scope of this Document

This document provides a guide to users of the URBS hydrologic and TUFLOW hydraulic models that were developed as part of the flood study.

2.0 Hydrologic and Hydraulic Models

2.1 Hydrologic Models

2.1.1 General

The URBS modelling has been undertaken using Version 5.85a (beta), with simulations performed using the URBS Control Centre Version 2.2.0 in lieu of a batch file.

The name and location of the URBS Control Centre project is as follows:

..\URBS\Moggill\Moggill.prj

The URBS modelling has been separated into:

- Calibration / Verification, and
- Design / Extreme / Climate Variability

The following sections discuss each respectively.

2.1.2 Calibration Models

For the calibration / verification runs, a separate model for each of the historical events has been developed. These are discussed individually in the following sections:

Event 1 – May 2015

The name and location of the May 2015 event folder is as indicated below, with the URBS Control Centre settings indicated in Figure 2.1.

..\URBS\Moggill\Calibration\May_2015

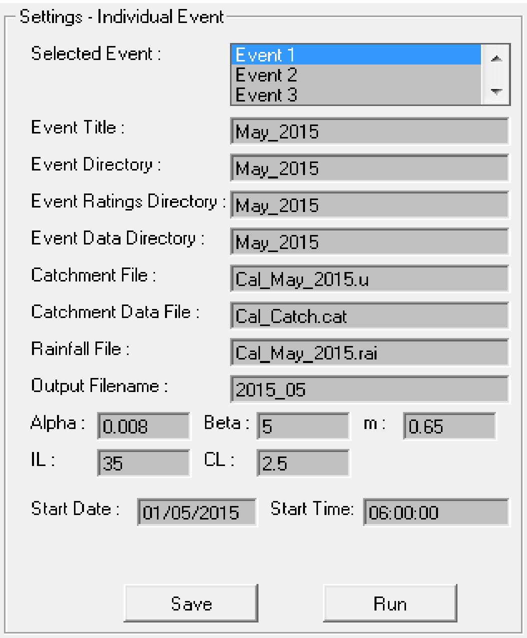

Figure 2.1: Event 1 (May 2015)

Event 2 – January 2013

The name and location of the January 2013 event folder is as indicated below, with the URBS Control Centre settings indicated in Figure 2.2.

..\URBS\Moggill\Calibration\Jan_2013

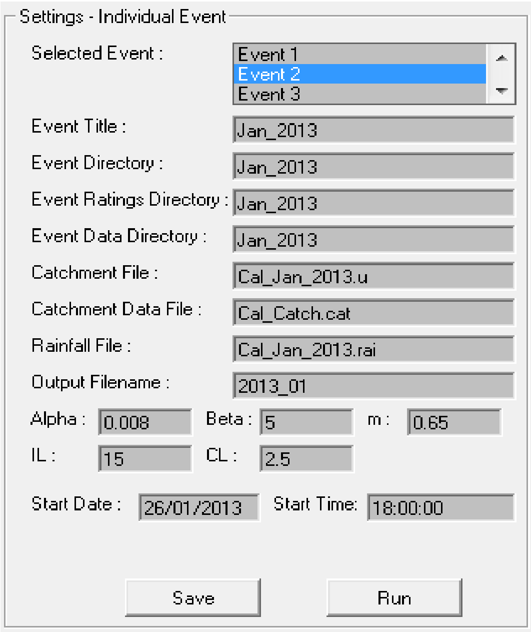

Figure 2.2: Event 2 (January 2013)

Event 3 – May 2009

The name and location of the May 2009 event folder is as indicated below, with the URBS Control Centre settings indicated in Figure 2.3.

..\URBS\Moggill\Calibration\May_2009

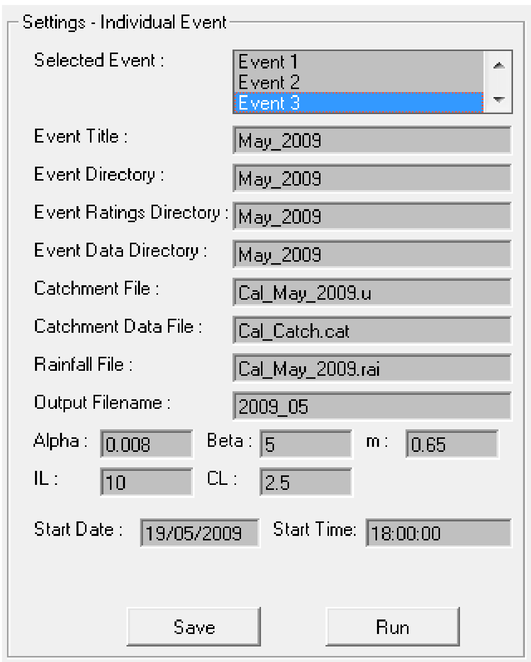

Figure 2.3: Event 3 (May 2009)

Event 4 – November 2008

The name and location of the November 2008 event folder is as indicated below, with the URBS Control Centre settings indicated in Figure 2.4.

..\URBS\Moggill\Calibration\Nov_2008

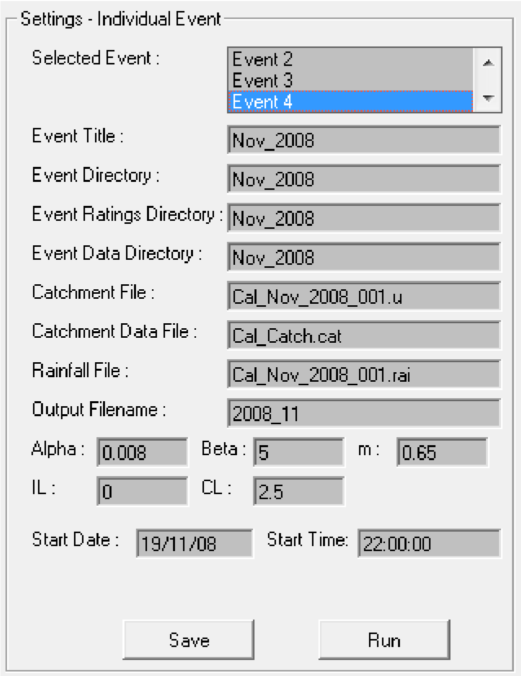

Figure 2.4: Event 4 (November 2008)

2.1.3 Design Model

For the design, extreme and climate variability events, one model has been developed. The name and location of the Design model folder is as indicated below, with the URBS Control Centre settings indicated in Figure 2.5.

..\URBS\Moggill\Design

For the Climate Variability runs, replace “IFD_Centroid.ifd” with those indicated below in order to generate the appropriate ARI files for the 100-yr to 500-yr ARI events:

- Climate Scenario 1 (2050): IFD_Centroid_CC1.ifd
- Climate Scenario 2 (2100): IFD_Centroid_CC2.ifd

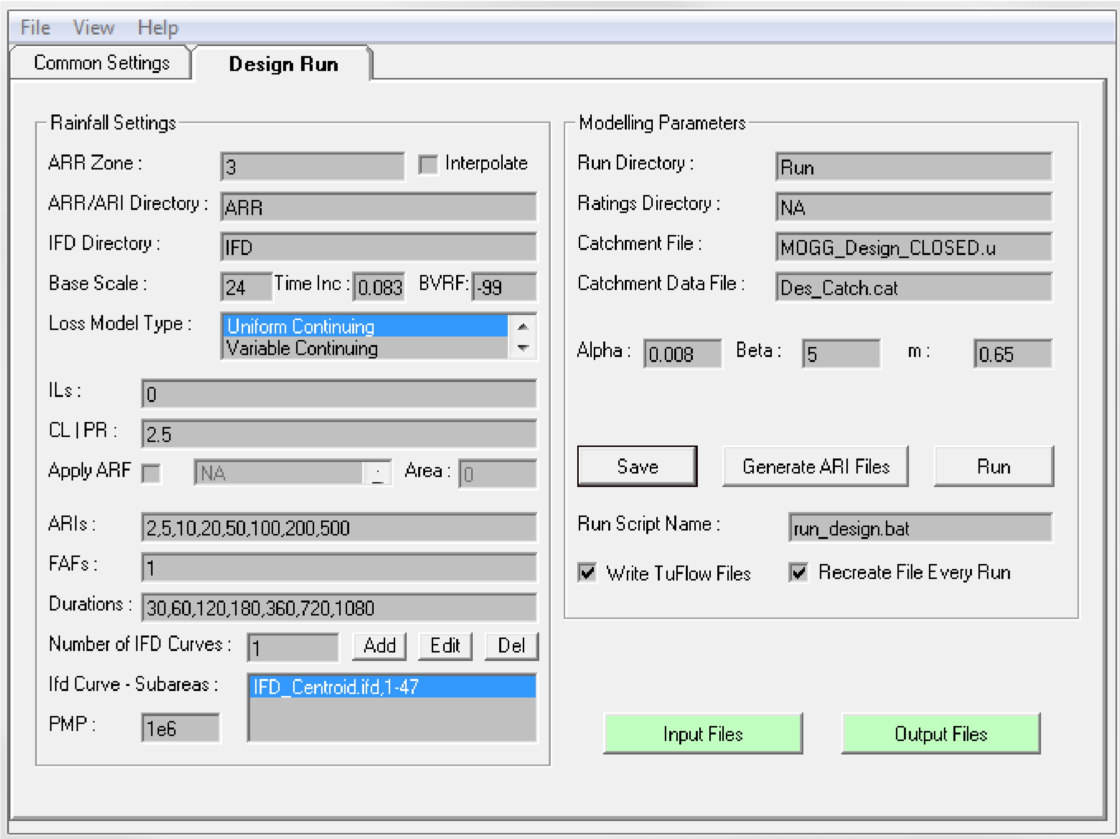

Figure 2.5: Design Run Settings – 2-yr to 500-yr ARI

In order to run the 2000-yr ARI and PMF events, the URBS Control Centre settings are as per Figure 2.6.

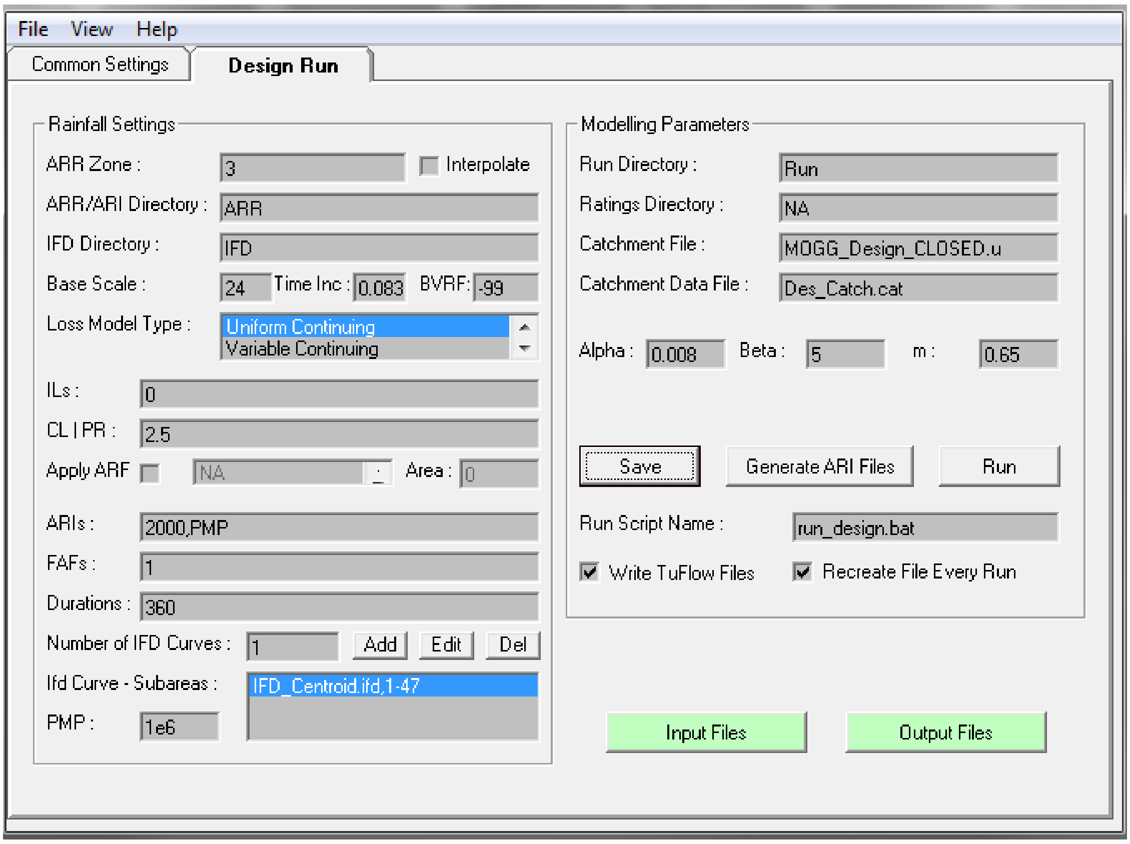

Figure 2.6: Design Run Settings – 2000-yr and PMF

2.2 Hydraulic Models

2.2.1 General

TUFLOW modelling was undertaken using build: 2013-12-AD-iSP-w64.

The TUFLOW modelling was undertaken using a single TUFLOW Control File (TCF), which was named: MCFS_~s~_~e1~_~e2~_033.tcf. The ESTRY Control File (ECF) is embedded into the TCF.

This TCF can be used to simulate all of the model runs undertaken as part of the flood study. The model is run using the appropriate TUFLOW batch command based on the required scenario and events.

2.2.2 TUFLOW Calibration and Verification Models

TUFLOW simulations were undertaken for all four historical events. The model is essentially the same for each, apart from the boundary conditions. Table 2.1 indicates the scenario and event codes to be used inside the TUFLOW batch file.

Table 2.1 – TUFLOW Calibration and Verification Batch Codes

Model Simulation	**Scenario (~s~)**	**Event 1 (~e1~)**	**Event 2 (~e2~)**
Calibration – May 2015	CAL	2015	05
Calibration – May 2009	CAL	2009	05
Calibration – November 2008	CAL	2008	11
Verification – January 2013	CAL	2013	01

As an example, the batch file command for January 2013 simulation would be as follows:

tuflow_iSP_w64.exe -b -s CAL -e1 2013 -e2 01 MCFS_~s~_~e1~_~e2~_033.tcf

2.2.3 TUFLOW Design Event Models

TUFLOW simulations were undertaken for all Scenario 1, Scenario 2 and Scenario 3 design events up to and including the 100-yr ARI (1 % AEP) event. Table 2.2 indicates the scenario and event codes to be used inside the TUFLOW batch file.

Table 2.2 – TUFLOW Design Event Batch Codes

Model Simulation	**Scenario (~s~)**	**Event 1 (~e1~)**	**Event 2 (~e2~)**
Design Events (Scenario 1)	S1_DES	002y 005y 010y 020y 050y 100y	030m 060m 120m 180m 360m 720m

Model Simulation	Scenario (~s~)	Event 1 (~e1~)	Event 2 (~e2~)
Design Events (Scenario 2)	S2_DES	100y	030m 060m 120m 180m 360m 720m
Design Events (Scenario 3)	S3_DES	002y 005y 010y 020y 050y 100y	030m 060m 120m 180m 360m 720m

As an example, the batch file command for Scenario 1 100-yr ARI 60-minute simulation would be as follows:

tuflow_iSP_w64.exe -b -s S1_DES -e1 100y -e2 060m MCFS_~s~_~e1~_~e2~_033.tcf

2.2.4 TUFLOW Extreme Event Models

TUFLOW simulations were undertaken for the Scenario 1 and Scenario 3 extreme events up to and including the PMF event. Table 2.3 indicates the scenario and event codes to be used inside the TUFLOW batch file.

Table 2.3 – TUFLOW Extreme Event Batch Codes

Model Simulation	Scenario (~s~)	Event 1 (~e1~)	Event 2 (~e2~)
Extreme Events (Scenario 1)	S1_EXT	200y 500y	030m 060m 120m 180m 360m 720m
	S1_EXT	2000y PMF	360m
Extreme Events (Scenario 3)	S3_EXT	200y 500y	030m 060m 120m 180m 360m 720m

As an example, the batch file command for Scenario 1 PMF simulation would be as follows:

tuflow_iSP_w64.exe -b -s S1_EXT -e1 PMF -e2 360m MCFS_~s~_~e1~_~e2~_033.tcf

2.2.5 TUFLOW Sensitivity Analysis Models

TUFLOW sensitivity simulations were undertaken for both climate variability and blockage. Table 2.4 indicates the scenario and event codes to be used inside the TUFLOW batch file.

Table 2.4 – TUFLOW Sensitivity Analysis Batch Codes

Model Simulation	Scenario (~s~)	Event 1 (~e1~)	Event 2 (~e2~)
Climate Variability (Scenario 1) Planning horizon 2050	S1_CC	100yCC1 200yCC1	030m 060m 120m 180m 360m 720m
Climate Variability (Scenario 1) Planning horizon 2100	S1_CC	100yCC2 200yCC2 500yCC2	030m 060m 120m 180m 360m 720m
Climate Variability (Scenario 3) Planning horizon 2050	S3_CC	100yCC1	030m 060m 120m 180m 360m 720m
Climate Variability (Scenario 3) Planning horizon 2100	S3_CC	100yCC2	030m 060m 120m 180m 360m 720m

As an example, the batch file command for Scenario 1 (2100) 100-yr 60-minute simulation would be as follows:

```
tuflow_iSP_w64.exe -b -s S1_CC -e1 100yCC2 -e2 060m MCFS_~s~_~e1~_~e2~_033.tcf
```