Cubberla Creek Flood Study

Volume 1 of 2

Flood Study Report

Prepared by Brisbane City Council's, City Projects Office

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Note: *The Cubberla 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 creek flood studies to reflect the current conditions of the catchment and best practice flood modelling techniques. The most recent studies undertaken of Cubberla Creek were the Cubberla Creek Water Quantity Assessment (2001) and Cubberla Creek Flood Study (1996).

Cubberla Creek Catchment has a total area of 10.5 km^2 and the catchment centroid is located approximately 9 km south-west of the Brisbane CBD. The major creeks / tributaries within the catchment are: Cubberla Creek; Boblynne Street Branch; Gubberley Creek; Akuna Street Branch and Tributary C. The catchment area is quite elongated and includes the suburbs of Chapel Hill, Kenmore and Fig Tree Pocket. The lower section of the catchment is dominated by flooding originating from the Brisbane River.

Project Objectives

The primary objectives of the project were as follows:

- Update the Cubberla Creek flood models (hydrologic and hydraulic) to represent the current catchment conditions and best practice flood modelling techniques.
- Adequately calibrate and verify the flood models to historical storm events to confirm that the models are suitable for the purpose 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, rare and 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 Cubberla 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 URBS model incorporated 43 sub-catchments and the sub-catchment delineation was based upon the 2014 ALS contours. The sub-catchment delineation considered the location of major tributaries, hydrometric gauges as well as man-made boundaries such as the Western Freeway.

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. The hydraulic model consists largely of a 1d / 2d linked schematisation, with the 1d domain modelled in ESTRY and the 2d domain in TUFLOW. The model incorporated Cubberla Creek; Boblynne Street Branch; Gubberley Creek; Akuna Street Branch and Tributaries A, B and C.

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 flood levels were all within the specified tolerance of ± 0.3 m. There were no continuous recording stream gauges within the catchment.

Utilising the adopted parameters from the calibration process, the verification was undertaken. Similar to the calibration, the verification achieved a good 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 in accordance with BCC City Plan 2014. A fixed tidal boundary was used at the downstream model extent to represent the Brisbane River.

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 to update the hydraulic roughness (as required) based on City Plan 2014.
- 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 “Modelled Flood Corridor” is the greater extent of the Waterway Corridor and Flood Planning Areas (FPAs) 1, 2 and 3.

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

- Critical storm durations at selected locations (Section 6.4.1)
- Peak design flood discharges (Section 6.4.2)
- Peak design flood levels at 100 m intervals along the AMTD line (Appendices E,F,G and H)
- Scenario 1 peak design flood extent mapping (Volume 2 of 2)
- Hydraulic structure flood immunity (Section 6.4.6)

The lower section of the catchment is dominated by flooding originating from the Brisbane River; as such the reported peak flood levels in this area will be lower than the Brisbane River peak flood levels for each respective ARI (AEP).

As part of the required sensitivity analysis, a climate variability analysis was then undertaken to determine the impacts for four climate futures; namely Year 2050 RCP4.5; Year 2050 RCP8.5; Year 2100 RCP4.5 and Year 2100 RCP8.5. This included making allowances for increased rainfall intensity and increased mean sea level. 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 indicated that climate variability impacts within the catchment will increase the magnitude of flooding. The following observations were made from the results:

- Flood level increases are greater under RCP8.5 climate projections when compared with RCP4.5 climate projections.
- 2050 RCP8.5 and 2100 RCP4.5 flood levels are almost identical for those areas not affected by projected sea level increases.
- Based on RCP8.5 climatic projections, by the year 2100, 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 for those areas not affected by projected sea level increases.
- Based on RCP8.5 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 for those areas not affected by projected sea level increases.

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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. This project was undertaken by Fugro Spatial Solutions Pty Ltd on behalf of the Queensland Government.
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 mAHD is approximately mean sea level.
Annual Exceedance Probability(AEP)	The probability that a given rainfall total or flood flow will be exceeded in any one year.
AR&R 2016 Data Hub (Beta)	The Australian Rainfall and Runoff Data Hub is a tool that allows for easy access to the design inputs required to undertake flood estimation in Australia. Background on the development and use of this data can be found in Australian Rainfall and Runoff (2016).
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.
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	ESTRY is the 1d hydrodynamic engine used by TUFLOW.
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 to better advise on the susceptibility of flooding.
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 hydraulic roughness in 1d / 2d flow equations.
MIKE11	Hydraulic modelling software package.

Glossary of Terms (cont)

Term	Definition
Minimum Riparian Corridor (MRC)	An area where future revegetation of the creek riparian zone has been assumed for modelling purposes. Modelled as dense vegetation (nominal Manning's n=0.15) and typically extending for a maximum of 15 m on either side of the low-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, 3 and represents a zone of assumed no filling.
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 theoretical greatest depth of precipitation that is physically possible over a particular catchment
URBS	Hydrologic modelling software package developed by Don Carroll
WBNM	Hydrologic modelling software package developed by the University of Wollongong

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. The recently updated AR&R 2016 utilises different terminology whereby for the larger flood magnitudes the term AEP (%) is now preferred to 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 1987	Australian Rainfall and Runoff (1987)
AR&R 2016	Australian Rainfall and Runoff (2016)
BCC	Brisbane City Council
CBD	Central Business District
CL	Continuing rainfall loss (mm/hr)
DTMR	Department of Transport and Main Roads (Queensland)
FPA	Flood Planning Area
IFD	Intensity Frequency Duration
IL	Initial rainfall loss (mm)
IWL	Initial Water Level (mAHD)
mAHD	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
RCP4.5	Representative Concentration Pathway 4.5
RCP8.5	Representative Concentration Pathway 8.5
QUDM	Queensland Urban Drainage Manual (Draft 2013)
WC	Waterway Corridor
WQA	Water Quantity Assessment

1.0 Introduction

1.1 Catchment Overview

Cubberla Creek Catchment is located approximately 9 km south-west of the Brisbane CBD and includes the suburbs of Chapel Hill, Kenmore and Fig Tree Pocket. The catchment has a total area of 10.5 km^2 and features the main Cubberla Creek plus the major tributaries of the Boblynne Street Branch; Gubberley Creek and the Akuna Street Branch as well as a number of minor tributaries. 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:

- Cubberla Creek Water Quantity Assessment in 2001 [2]
- Cubberla Creek Flood Study in 1996. [3]

For the purposes of this report these previous reports are termed the (i) 2001 WQA and (ii) 1996 Flood Study.

1.3 Study Objectives

The primary objectives of the project are as follows:

- Update the Cubberla Creek flood models (hydrologic and hydraulic) to represent the current catchment conditions and best practice flood modelling techniques.
- Adequately calibrate and verify the flood models to historical storm events to confirm that the models are suitable for the purposes of simulating design flood events.
- Estimate design and rare / extreme flood magnitudes.
- Determine flood levels for the design and rare / extreme events, accounting for the effects of Minimum Riparian Corridor (MRC) and floodplain development / filling in accordance with current planning policy.
- Produce flood extent mapping for the selected range of design and rare / extreme events.
- Investigate the sensitivity of climate variability on flooding within the catchment.

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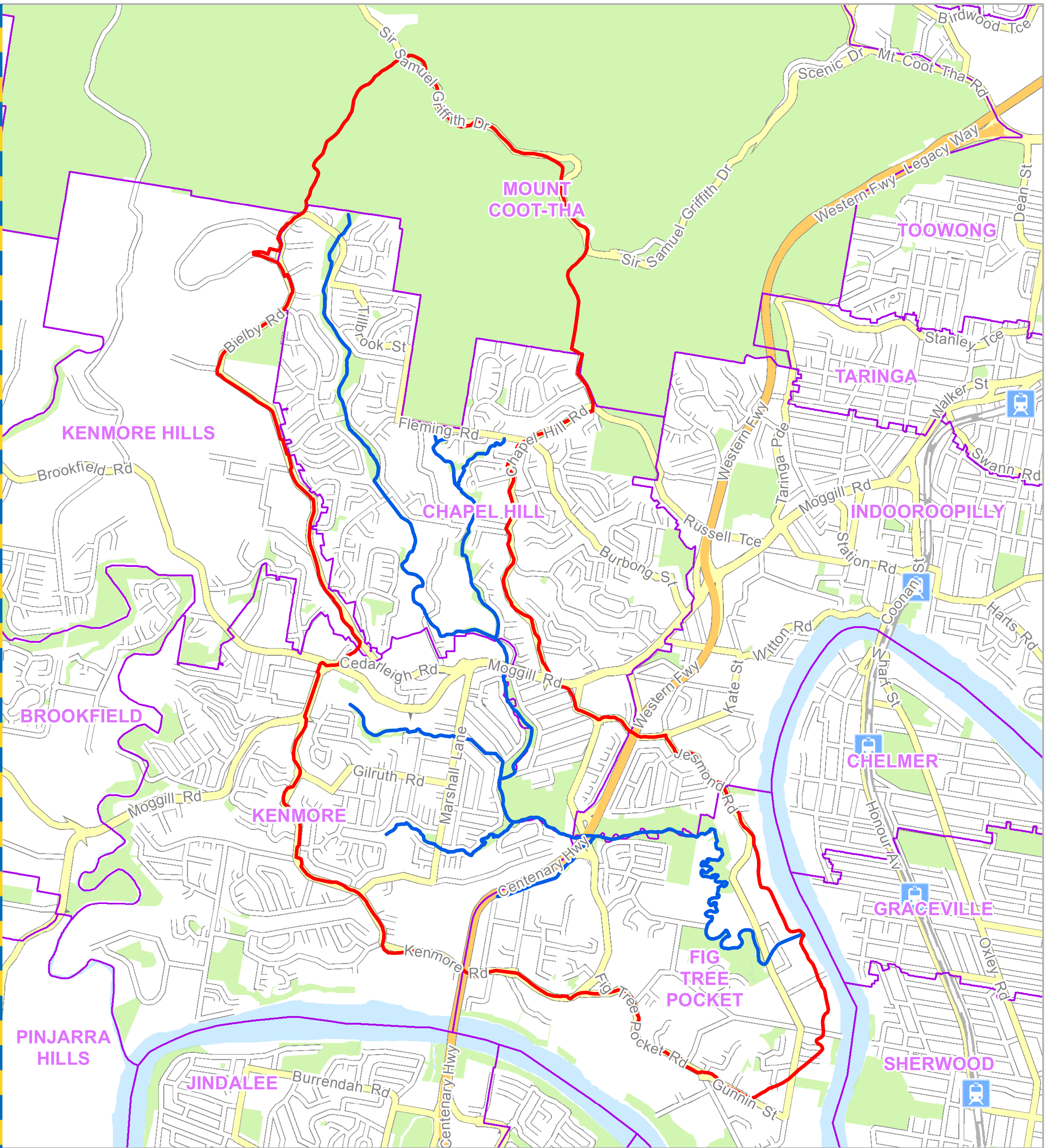
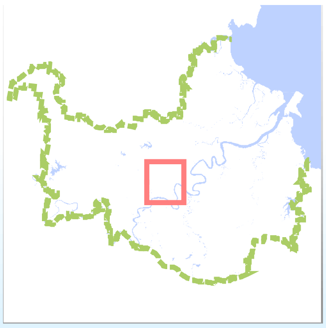
[1] Brisbane City Council 2015, *Creek Flood Study Procedure Document Version 7.1*
[2] Brisbane City Council Water and Environment 2001, *Cubberla Creek Water Quantity Assessment (Draft)*
[3] Sinclair Knight Merz for Brisbane City Council 1996, *Cubberla Creek Flood Study*





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Legend

- Creek Centreline
- Catchment Area
- Railway Stations
- Railway Lines
- Freeways/Highways
- Major Roads
- Streets
- Suburb Boundaries
- Parks
- Waterways

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Figure 1.1: Locality Plan

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1.4 Scope of the Study

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

- 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.
- 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 1987) 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.
- 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 changes to the flood behaviour within the catchment.

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 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 observed records, including MHG locations throughout the catchment.

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2.0 Catchment Description

2.1 Catchment and Waterway Characteristics

2.1.1 General

The confluence of Cubberla Creek and the Brisbane River is 2.2 km upstream of the Walter Taylor Bridge at Indooroopilly. The total catchment area of the Cubberla Creek Catchment is approximately 10.5 km^2, which comprises the following tributaries:

- Cubberla Creek: 6.83 km^2
- Boblynne Street Branch: 1.32 km^2
- Gubberley Creek: 0.8 km^2
- Akuna Street Branch: 0.73 km^2
- Tributary C: 0.84 km^2

Figure 2.1 indicates the major creeks and tributaries within the catchment.

2.1.2 Cubberla Creek

Cubberla Creek is the largest waterway within the catchment with a length of approximately 8.6 km from the upstream extent of development in Chapel Hill to the Brisbane River at Fig Tree Pocket. The catchment is bounded by Enoggera Creek Catchment (north); McKay Brook / Gap Creek / Moggill Creek (west); Brisbane River (south) and Witton Creek (east).

The catchment headwaters are within the Mount Coot-tha Forest, an area characterised by steep slopes and dense / forested vegetation. The highest elevation in the catchment is approximately 262 mAHD.

Cubberla Creek is an open waterway for the majority of its length, apart from two sections in which a low-flow pipe replaces the low-flow creek channel. These two locations are in the vicinity of Greenford Street and Goolman Street in the upper catchment and total approximately 830 m in length. During the urbanisation of the catchment, the natural waterway has been significantly modified in numerous areas, which has included: channelisation / straightening; channel relocation; drop structures; low-flow piping; culverts / bridges; floodplain filling; etc. The average bed slope of the creek over its entire 8.6 km length is approximately 0.7 %, with the most upstream 1 km of creek having an average bed slope of approximately 2 %.

There are two major arterial road crossings of Cubberla Creek, namely Moggill Road (AMTD 4350) and the Western Freeway (AMTD 2700). Between Moggill Road and the Western Freeway the major easterly draining sub-catchments join Cubberla Creek. Downstream of the Western Freeway, the topography changes and is characterised by wide open grassed floodplain areas underlain by alluvium.

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 Boblynne Street Branch

The Boblynne Street Branch is situated in the north-east section of the Cubberla Creek Catchment and contains two minor tributaries; named Tributary A and Tributary B for the purpose of this study. The reach has a length of approximately 2.3 km from the upstream extent of development in Chapel Hill to its outfall at Kenmore and is the second longest creek within the catchment. The Boblynne Street Branch joins Cubberla Creek in the middle section of the catchment, approximately 5.2 km upstream of the confluence with the Brisbane River and 190 m upstream of Moggill Road. The Boblynne Street Branch is an open waterway for the majority of its length, apart from the developed section upstream of Fleming Road, where the waterway has been fully piped. The average bed slope of the creek over the 1.6 km open waterway section is approximately 1.3 %.

The highest elevation in the catchment is approximately 234 mAHD and is situated along the northern catchment boundary within Mount Coot-tha Forest. The creek joins Cubberla Creek at an invert level of approximately 17.8 mAHD.

2.1.4 Gubberley Creek

Gubberley Creek is one of three eastward flowing tributaries and has a length of nearly 1.5 km. Gubberley Creek joins Cubberla Creek in the mid to lower section of the catchment, approximately 4.1 km upstream of the confluence with the Brisbane River. The bed slope is relatively consistent over the entire length of creek, with an average bed slope of 1.6 %.

The most upstream and downstream sections of the creek are fully piped and the middle section consist of open waterway, which includes a small detention basin; which is discussed further in Section 4.4. The downstream piped section traverses through a low-density residential subdivision prior to outfalling to Cubberla Creek.

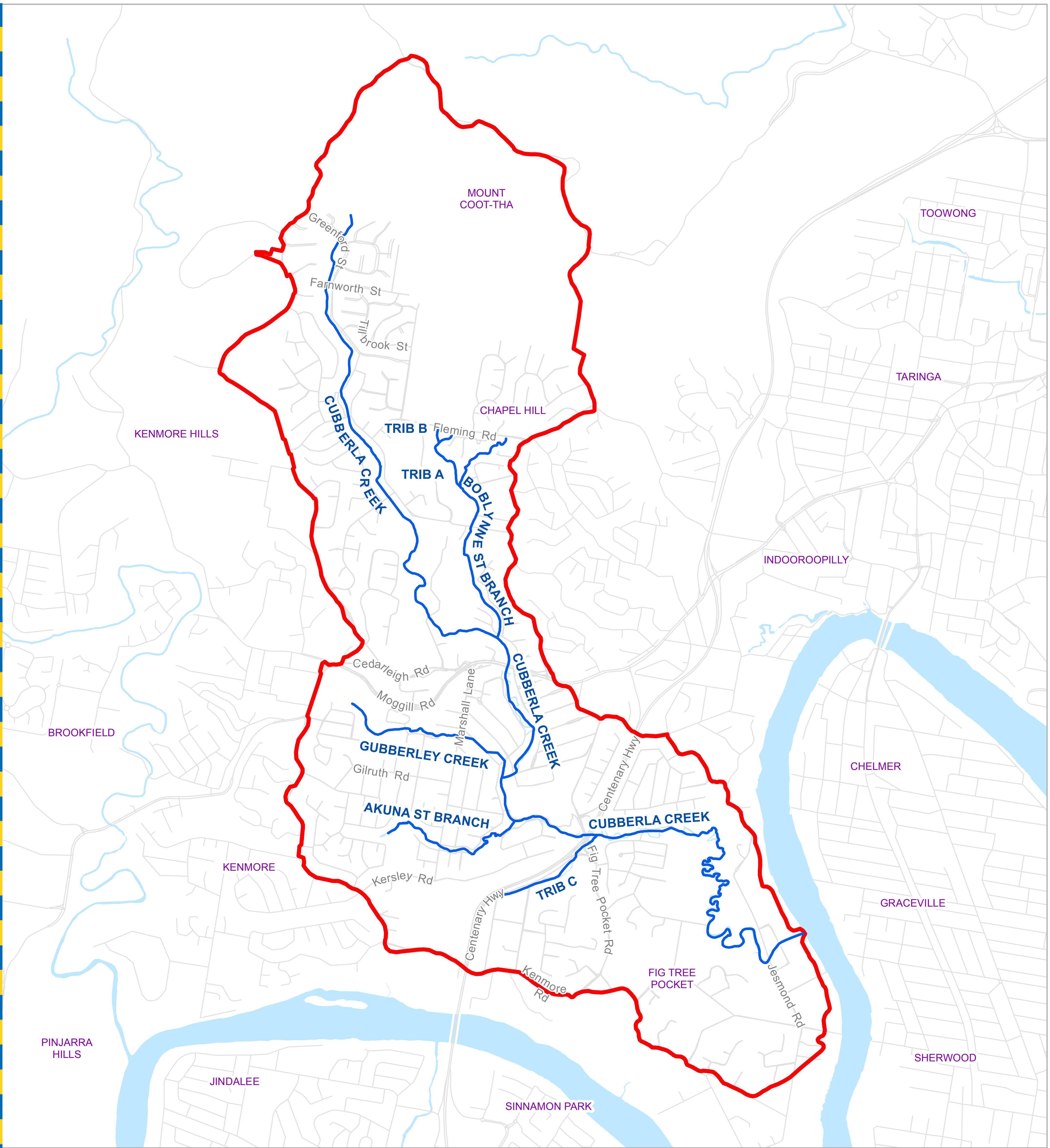
The highest elevation in the catchment is approximately 61 mAHD and is situated along the western boundary on Kenmore Road. The creek joins Cubberla Creek at an invert level of approximately 10.5 mAHD.

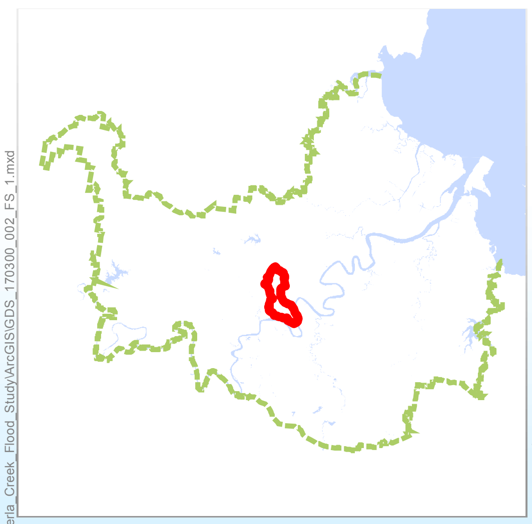
2.1.5 Akuna Street Branch

The Akuna Street Branch flows in an easterly direction over a length of approximately 1.8 km and joins Cubberla Creek in the mid to lower section of the catchment, approximately 3.9 km upstream of the confluence with the Brisbane River.

The most upstream section of the creek is fully piped and the remainder of the creek is open waterway. The average bed slope of the piped section is approximately 4 %, whereas the open waterway is less steep at an average bed slope of 1.7 %; which is similar to the nearby Gubberley Creek.

The highest elevation in the catchment is approximately 61 mAHD and is situated along the western boundary on Kenmore Road. The creek joins Cubberla Creek at an invert level of approximately 10.3 mAHD.





Legend

- Creek Centreline
- Catchment Area
- Streets

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Figure 2.1: Major Creeks and Tributaries

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2.1.6 Tributary C

Tributary C flows in an easterly direction adjacent to the Western Freeway over a length of approximately 1.5 km. The reach joins Cubberla Creek in the lower section of the catchment, approximately 3.3 km upstream of the confluence with the Brisbane River and immediately downstream of the Western Freeway Bridge.

The creek is an open waterway over its entire length with an average bed slope of approximately 2.6 %. The lower section of the creek has been significantly modified with a number of culverts over a short length, resulting from the Fig Tree Pocket Road – Western Freeway interchange. The creek is bisected by the Western Freeway with the upstream section being considerably steeper than the downstream section. The average bed slope of the upstream section is approximately 4.3 %, whereas the downstream section is approximately 1.4 %. The highest elevation in the catchment is approximately 63 mAHD and is situated along the southern boundary on Kenmore Road. The creek joins Cubberla Creek at an invert level of approximately 7.5 mAHD.

2.2 Land Use

There is significant development throughout the catchment with the predominant land-use zoning being "Low Density Residential", which occupies just over 40 % of the catchment area. The next largest is "Environmental Management and Conservation" (19.9 %) and then "Road Reserve" (15.5 %). Figure 2.2 provides a breakdown of the catchment land-use by percentage and Appendix C provides a map indicating the distribution of the land-use throughout the catchment. Both figures are based upon BCC City Plan 2014. [4]

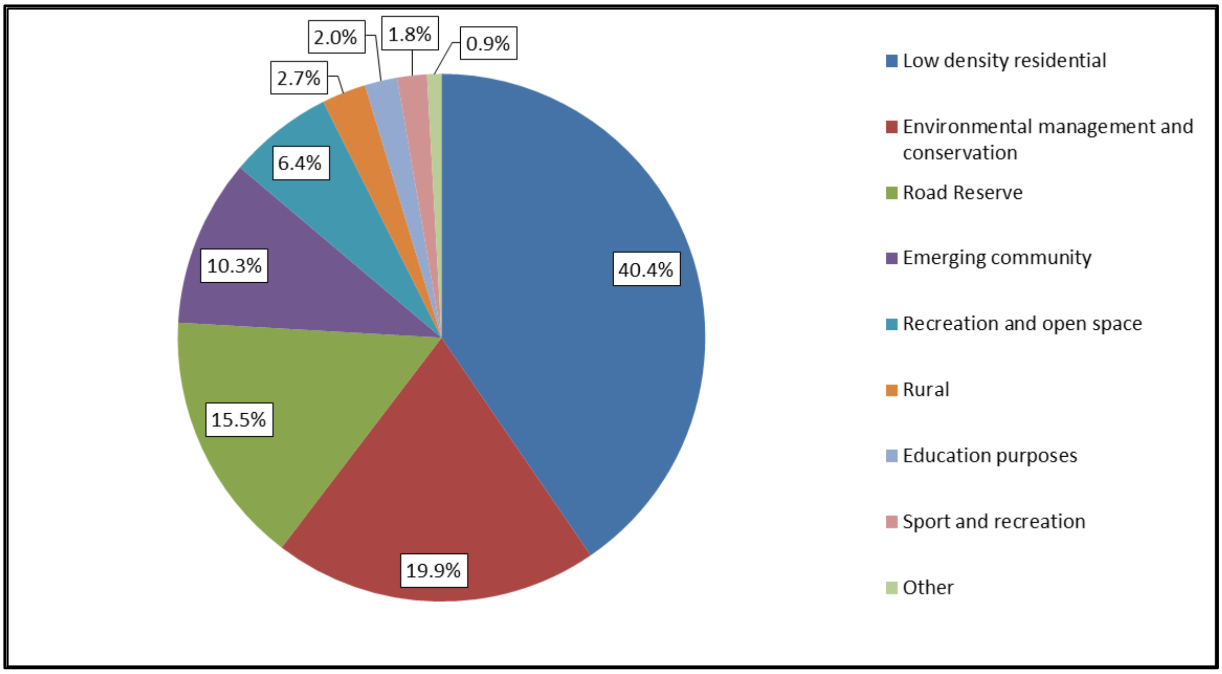


Figure 2.2: Cubberla Creek Catchment Land-use

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

The “Environmental Management and Conservation” areas are primarily within the catchment headwaters in the Mount Coot-tha Forest and are characterised by dense forest on steep slopes.

The “Emerging Community” zone is typically for land that would become urban development in the future. There are pockets of “Emerging Community” zoned land spread throughout the catchment. The value of 10.3 % indicates that there is only just over 1 km^2 of land remaining in the catchment for the purpose of urban development.

Downstream of the confluence with the Akuna Street Branch (where the floodplain widens) is where the majority of “Sport and Recreation”, “Open Space” and “Rural” zoned areas are located.

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 38 years. This table includes the peak flood level in Cubberla Creek at both MHG CB120 (U/S Western Freeway) and MHG CB130 (confluence of Akuna Street Branch). This table also indicates the availability of MHG information as well as the approximate size of the event.

Table 3.1 – Historical Peak Levels on Cubberla Creek

Event	Peak Flood Level (mAHD)		Number of MHGs and/or recorded levels	Approximate Size of Event
	MHG CB120	MHG CB130		
April 1978	10.03	-	1	< 2-yr ARI (50% AEP)
June 1979	9.67	11.64	3	< 2-yr ARI (50% AEP)
May 1980	10.02	11.93	3	< 2-yr ARI (50% AEP)
February 1981	10.84	12.29	5	2-yr ARI (50% AEP) to 5-yr ARI (20% AEP)
January 1982	10.30	12.06	2	< 2-yr ARI (50% AEP)
May 1983	-	12.04	1	< 2-yr ARI (50% AEP)
June 1983	-	11.72	3	< 2-yr ARI (50% AEP)
January 1985	9.69	12.29	2	2-yr ARI (50% AEP) to 5-yr ARI (20% AEP)
April 1988	10.20	12.03	3	< 2-yr ARI (50% AEP)
July 1988	10.95	12.50	5	10-yr ARI (10% AEP) to 20-yr ARI (5% AEP)
April 1989	11.61	12.67	6	20-yr ARI (5% AEP) to 50-yr ARI (2% AEP)
February 1992	10.20	12.12	5	< 2-yr ARI (50% AEP)
May 1996	-	11.98	4	< 2-yr ARI (50% AEP)
November 2008	11.25	12.47	8	10-yr ARI (10% AEP) to 20-yr ARI (5% AEP)
May 2009	11.27	12.36	9	5-yr ARI (50% AEP) to 10-yr ARI (20% AEP)
February 2010	-	12.14	6	< 2-yr ARI (50% AEP)
January 2011	-	11.91	7	< 2-yr ARI (50% AEP)
January 2013	-	12.32	9	2-yr ARI (50% AEP) to 5-yr ARI (20% AEP)
May 2015	-	12.28	9	2-yr ARI (50% AEP) to 5-yr ARI (20% AEP)
May 2016	-	12.03	8	< 2-yr ARI (50% AEP)

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 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

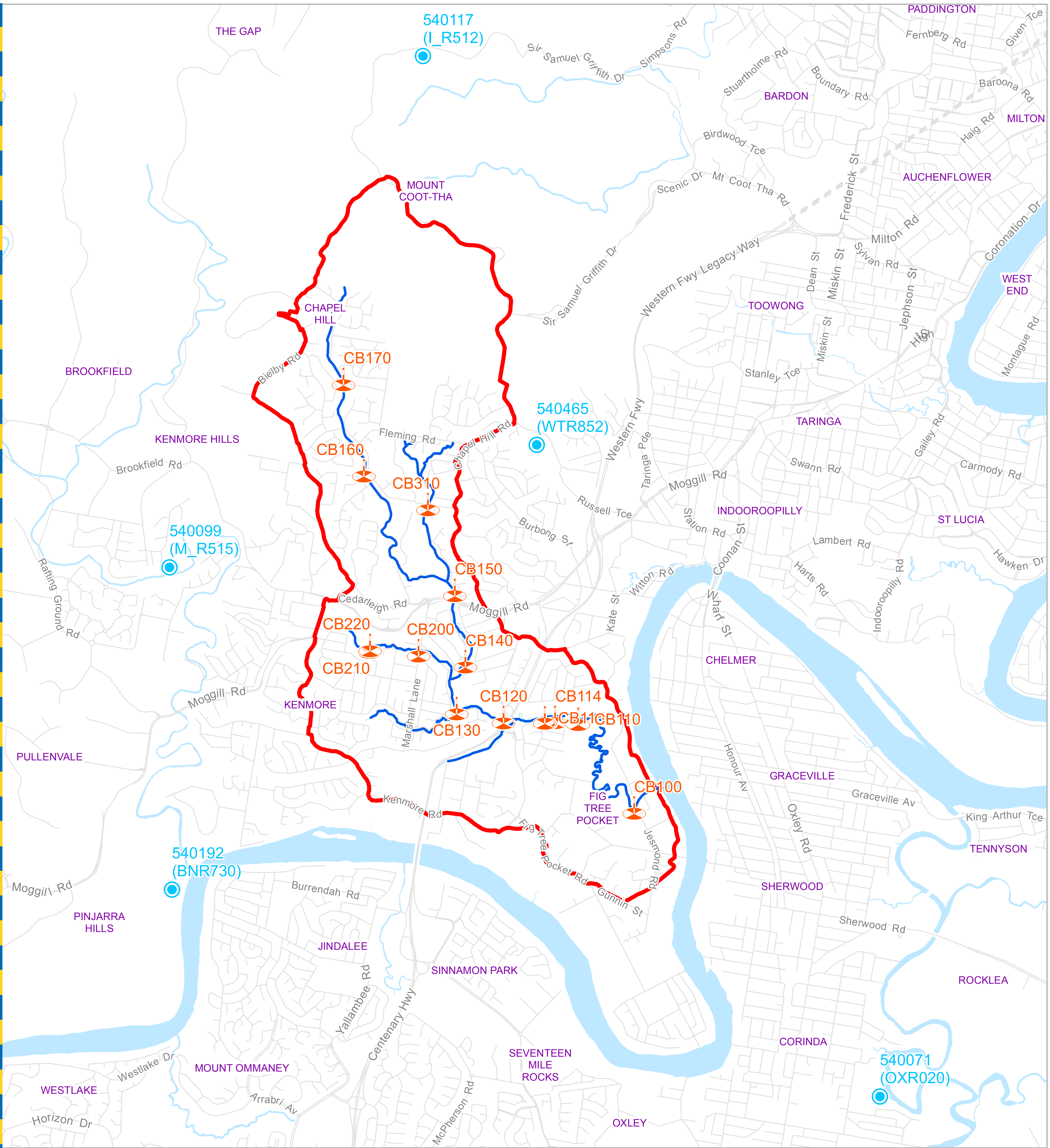
3.2 Availability of Historical Data for Selected Storms

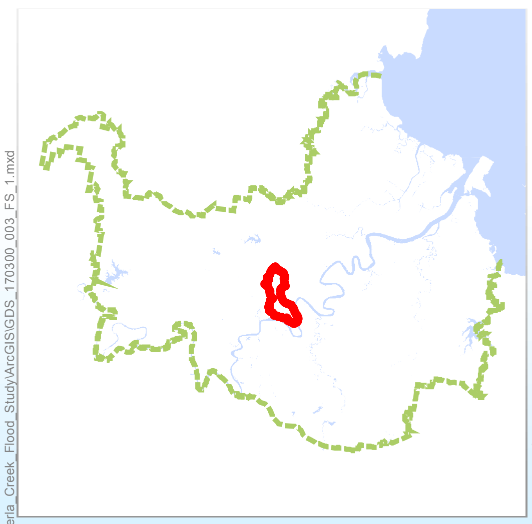
3.2.1 Continuous Recording Rainfall Stations

Five 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
540465	WTR852	Witton Creek	Green Hill Reservoir, Chapel Hill	Open
540117	I_R512	Breakfast Creek	Mt Coot-tha	Open
540192	BNR730	Brisbane River	Brisbane River at Jindalee	Open
540071	OXR020	Oxley Creek	Corinda High, Corinda	Open





Legend

- Pluviograph Stations
- Maximum Height Gauges
- Catchment Area
- Creek Centreline
- Streets

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Metres

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Prepared : 081335
Checked : JS
Revision : 1
Publication Date : 17 May 2017
Project Number : 170300

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Cubberla Creek Flood Study

Figure 3.1: Catchment Map and Gauge Location

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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	✓	✓	✓	✓
540465	WTR852	Green Hill Reservoir, Chapel Hill	✓	✓	×	×
540117	I_R512	Mt Coot-tha	✓	✓	✓	✓
540192	BNR730	Brisbane River at Jindalee	✓	✓	✓	✓
540071	OXR020	Corinda High, Corinda	✓	✓	✓	✓

3.2.2 Continuous Recording Stream Gauges

Continuous recording stream height gauges collect instantaneous water level information over time. They are important for calibration purposes as they provide important information on the timing of the flood as well as the total shape and volume of the flood hydrograph. Unfortunately, there are none of these stream gauges within the Cubberla Creek Catchment.

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 BCC staff following the flooding event. However, if the gauge has malfunctioned during the event and there is a nearby debris mark, then the recorded water level is typically based on this debris level.

There are 14 MHGs within the total catchment area and all are currently operational. Of the 14 operating MHGs, there are currently 10 located on Cubberla Creek, three located on Gubberley Creek and one located on the Boblynne Street Branch. There are currently no MHGs located on the Akuna Street Branch or on Tributary C. Table 3.4 indicates the period of operation for the MHGs on Cubberla and Gubberley Creeks as well as the Boblynne Street Branch.

Table 3.5 indicates the availability of MHG data for each flooding event. It is apparent that May 2015, January 2013 and May 2009 each have 9 recorded levels and November 2008 has 8 recorded levels. Two of the recorded levels for the May 2009 event were from debris marks.

Table 3.4 – Maximum Height Gauge period of record

Creek	Gauge ID	Location	Records From	Records To
Cubberla	100	U/S Jesmond Rd	August 1977	Present
	110	280 m D/S of Dobell St Footbridge	August 1977	Present
	114	D/S Dobell Street Footbridge	October 2009	Present
	115	U/S Dobell Street Footbridge	October 2009	Present
	120	U/S Western Freeway	August 1977	Present
	130	Confluence of Akuna Street Branch	April 1978	Present
	140	Adjacent 95 Sutling Street	April 1978	Present
	150	U/S Moggill Road Culvert	August 1977	Present
	160	130 m U/S of Goolman Street	October 2010	Present
	170	Adjacent 29 Greenford Street	October 2010	Present
Boblynne	310	U/S Brymer Street	October 2010	Present
Gubberley	200	U/S Marshall Lane	November 1991	Present
	210	Gubberley Creek Detention Basin	September 1990	Present
	220	Gubberley Creek Detention Basin	September 1990	Present

Table 3.5 – Maximum Height Gauge data availability

Creek	Gauge ID	Data Availability			
		May 2015	January 2013	May 2009	November 2008
Cubberla	100	×	✓	✓	✓
	110	✓	✓	✓	✓
	114	✓	✓	×	×
	115	×	✓	×	×
	120	×	×	✓	✓
	130	✓	✓	✓	✓
	140	✓	×	✓	✓
	150	×	×	✓ (d)	×
	160	✓	✓	×	×
	170	✓	✓	×	×
Boblynne	310	✓	✓	×	×
Gubberley	200	×	×	✓	✓
	210	✓	✓	✓	✓
	220	✓	×	✓ (d)	✓

(d) Reading from debris mark

3.2.4 Brisbane River Stream Gauges

Brisbane River stream gauges are used to generate downstream boundary conditions for the hydraulic model in the calibration and verification events.

Table 3.6 indicates the details of the nearest upstream and downstream gauges to the mouth of Cubberla Creek. There are two stream gauges located at Jindalee 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. For consistency, the Seqwater stream gauge was used in preference to the BCC gauge due to its longer period of operation.

Table 3.6 – Nearest Brisbane River Stream Gauges

Gauge ID	Old BCC ID	Owner	BNE AMTD (km)	Location	Current Status
540274	OXA588	BCC	38.7	Mouth of Oxley Creek	Open
540192	BNA731	Seqwater	52.1	Jindalee	Open
540682	BNA765	BCC	52.2	Mount Ommaney Drive, Jindalee	Open
540200	BNA755	BOM / Seqwater	72.2	Moggill	Open

Table 3.7 indicates the availability of stream gauge data for the four calibration / verification events. For 3 out of 4 events there was both upstream and downstream stream gauge data; however for the May 2009 event there was only downstream stream gauge information available; refer to Section 5.3.8 for further details on the adoption of downstream boundary conditions.

Table 3.7 – Brisbane River Stream Gauge data availability

Gauge ID	Old BCC ID	Data Availability			
		May 2015	January 2013	May 2009	November 2008
540274	OXA588	✓	✓	✓	✓
540192	BNA731	✓	✓	×	×
540682	BNA765	✓	✓	×	×
540200	BNA755	✓	✓	×	✓

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 12.28 mAHD at MHG CB130 on Cubberla Creek at the confluence with the Akuna Street Branch. Minor flooding occurred in the middle and lower reaches of the creek.

The total event rainfall was consistent over the entire catchment with approximately 180 mm being recorded in 24 hours on the 1st May. The most intense burst occurred over 6 hours between 1:30 pm and 7:30 pm on the 1st May, where approximately 148 mm of rainfall was recorded at Rainfall Station 540071 (OXR020) at Corinda High. 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 the five rainfall stations. The catchment experienced between 27 to 50 mm of rainfall in the 4-day lead up to the event and between 40 to 67 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	Peak 1hr burst	Peak 6hr burst
540099	M_R515	Chadstone Close, Kenmore Hills	61	36	42	132
540465	WTR852	Green Hill Reservoir, Chapel Hill	67	43	39	133
540117	I_R512	Mt Coot-tha	58	50	45	128
540192	BNR730	Brisbane River at Jindalee	40	27	32	116
540071	OXR020	Corinda High, Corinda	55	40	42	148

Figure 3.2 provides a comparison of the IFD curve for the five rainfall stations against the AR&R 1987 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: Less than 2-yr ARI (50 % AEP)
- 2 hour rainfall: 2-yr ARI (50 % AEP) to 5-yr ARI (20 % AEP)
- 3 hour rainfall: 5-yr ARI (10 % AEP)
- 6 hour rainfall: 10-yr ARI (10 % AEP) to 20-yr ARI (5 % AEP)

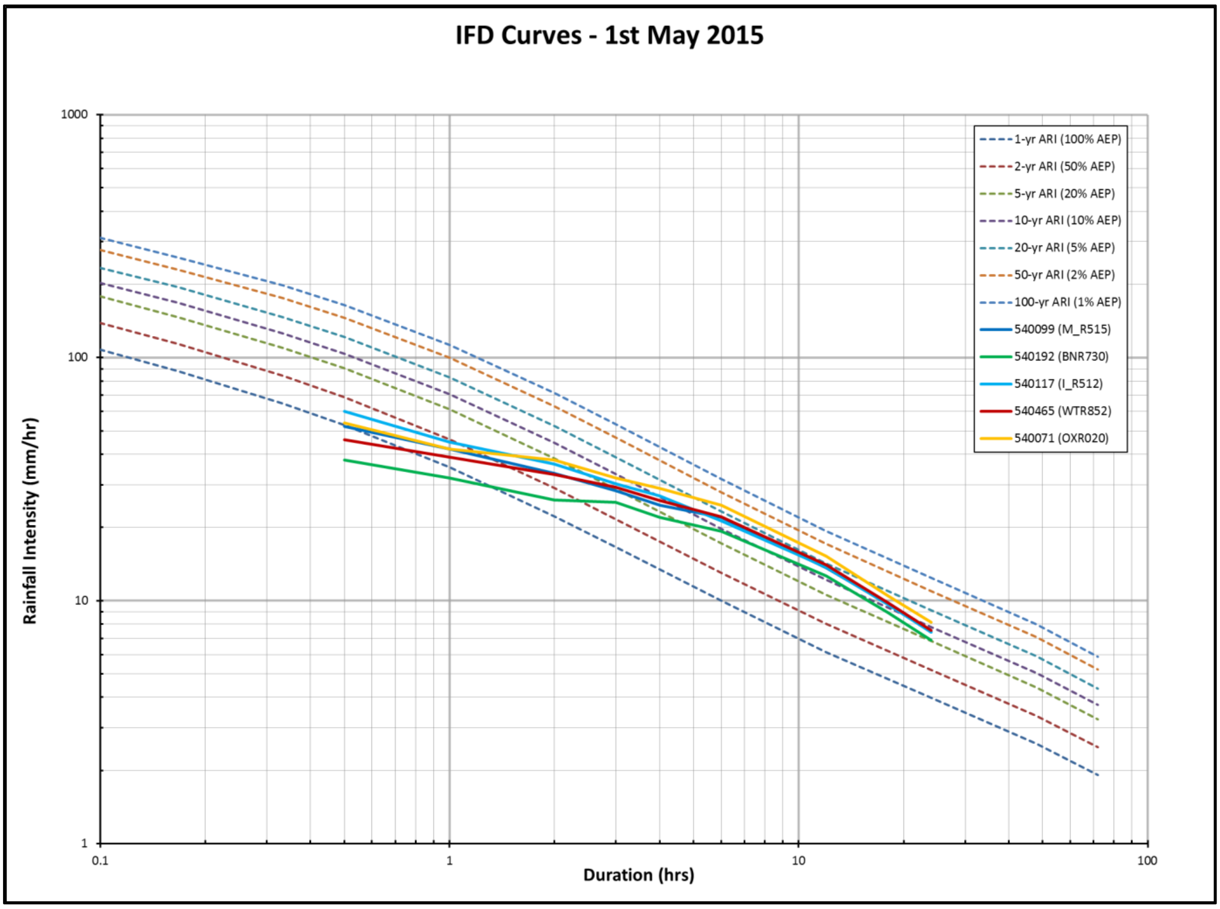


Figure 3.2: IFD Curve for May 2015 event.

3.3.2 January 2013 event

This event was a relatively long duration flooding event which produced a flood level of 12.32 mAHD at MHG CB130 on Cubberla Creek at the confluence with the Akuna Street Branch. Minor flooding occurred in the middle and lower reaches of the creek.

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 170 mm to 215 mm of rainfall fell across the catchment. The event was more intense in the upper sections of the catchment with Rain Gauge 540117 (I_R512) at Mount Coot-tha recording the most intense bursts. 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 the five rainfall stations. The catchment experienced between 68 and 147 mm of rainfall in the 14 day lead up to the event with between 60 mm and 139 mm falling in the 4 days prior. Therefore the soil would have been quite saturated due to the rainfall in the days prior to the main storm event.

Figure 3.3 provides a comparison of the IFD curve for the five rainfall stations against the AR&R 1987 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: Less than 2-yr ARI (50 % AEP)
- 2 hour rainfall: 2-yr ARI (50 % AEP) to 5-yr ARI (20 % AEP)
- 3 hour rainfall: 10-yr ARI (10 % AEP)
- 6 hour rainfall: 20-yr ARI (5 % AEP) to 50-yr ARI (2 % AEP)

Table 3.9 - Rainfall characteristics (January 2013 event)

Gauge ID	Old BCC ID	Location	Antecedent Rainfall (mm)		Event Rainfall (mm)	
			14-day	4-day	Peak 1hr burst	Peak 6hr burst
540099	M_R515	Chadstone Close, Kenmore Hills	109	104	43	160
540465	WTR852	Green Hill Reservoir, Chapel Hill	105	98	44	163
540117	I_R512	Mt Coot-tha	147	139	48	175
540192	BNR730	Brisbane River at Jindalee	103	99	34	137
540071	OXR020	Corinda High, Corinda	68	60	28	88

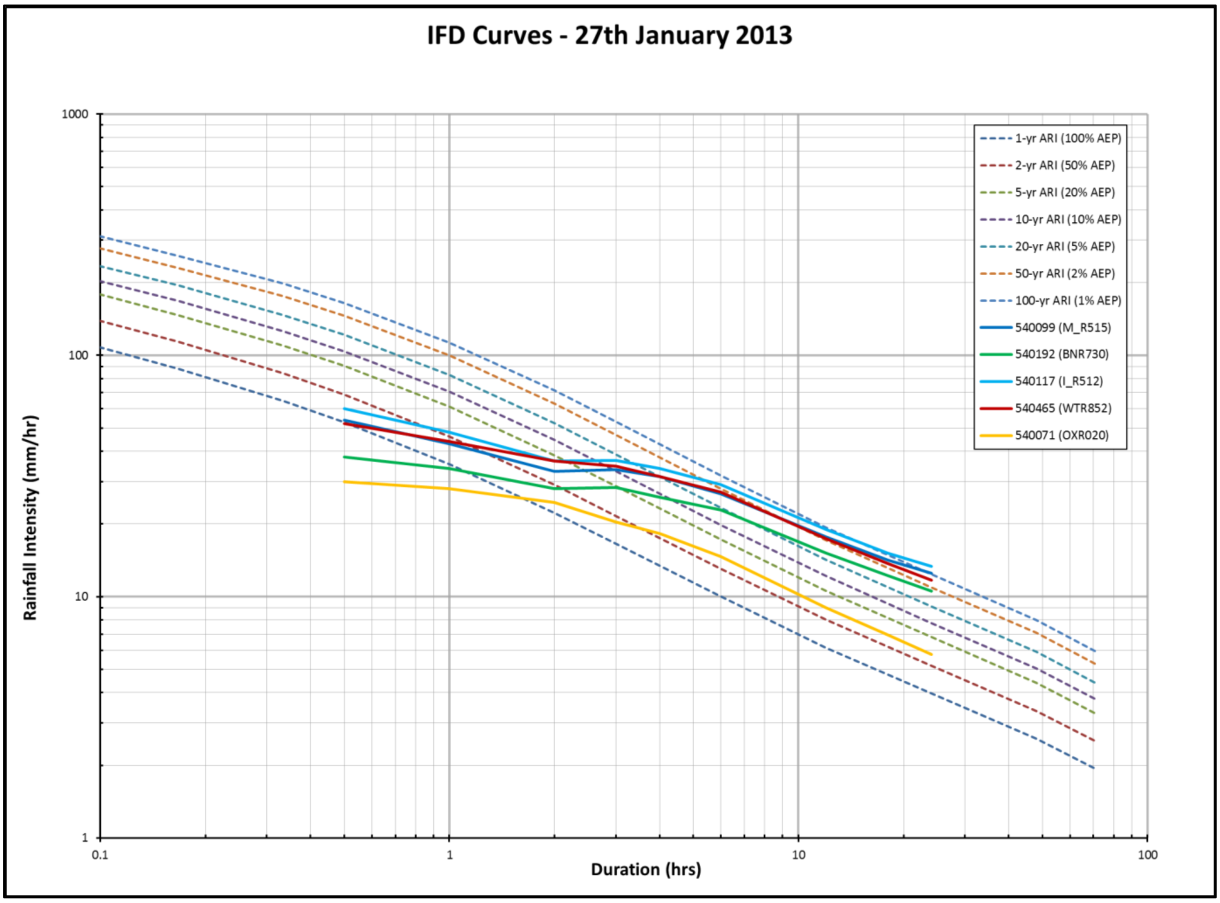


Figure 3.3: IFD Curve for January 2013 event.

3.3.3 May 2009 Event

This event was one of the largest in recent times and produced a flood level of 12.36 mAHD at MHG CB130 on Cubberla Creek at the confluence with the Akuna Street Branch. Moderate flooding occurred in the middle and lower reaches of the creek.

The event occurred over a 13 hour period starting at approximately 8 am on the 20th May and consisting of two significant bursts of rainfall. The first burst occurred 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 at 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 the four rainfall stations. The catchment experienced between 51 and 89 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	Peak 1hr burst	Peak 6hr burst
540099	M_R515	Chadstone Close, Kenmore Hills	55	55	56	176
540117	I_R512	Mt Coot-tha	89	84	65	188
540192	BNR730	Brisbane River at Jindalee	65	65	61	136
540071	OXR020	Corinda High, Corinda	51	51	15	58

Figure 3.4 provides a comparison of the IFD curve for the five rainfall stations against the AR&R 1987 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: 20-yr ARI (5 % AEP)
- 3 hour rainfall: 50-yr ARI (2 % AEP)
- 6 hour rainfall: 50-yr ARI (2 % AEP) to 100-yr ARI (1 % AEP)

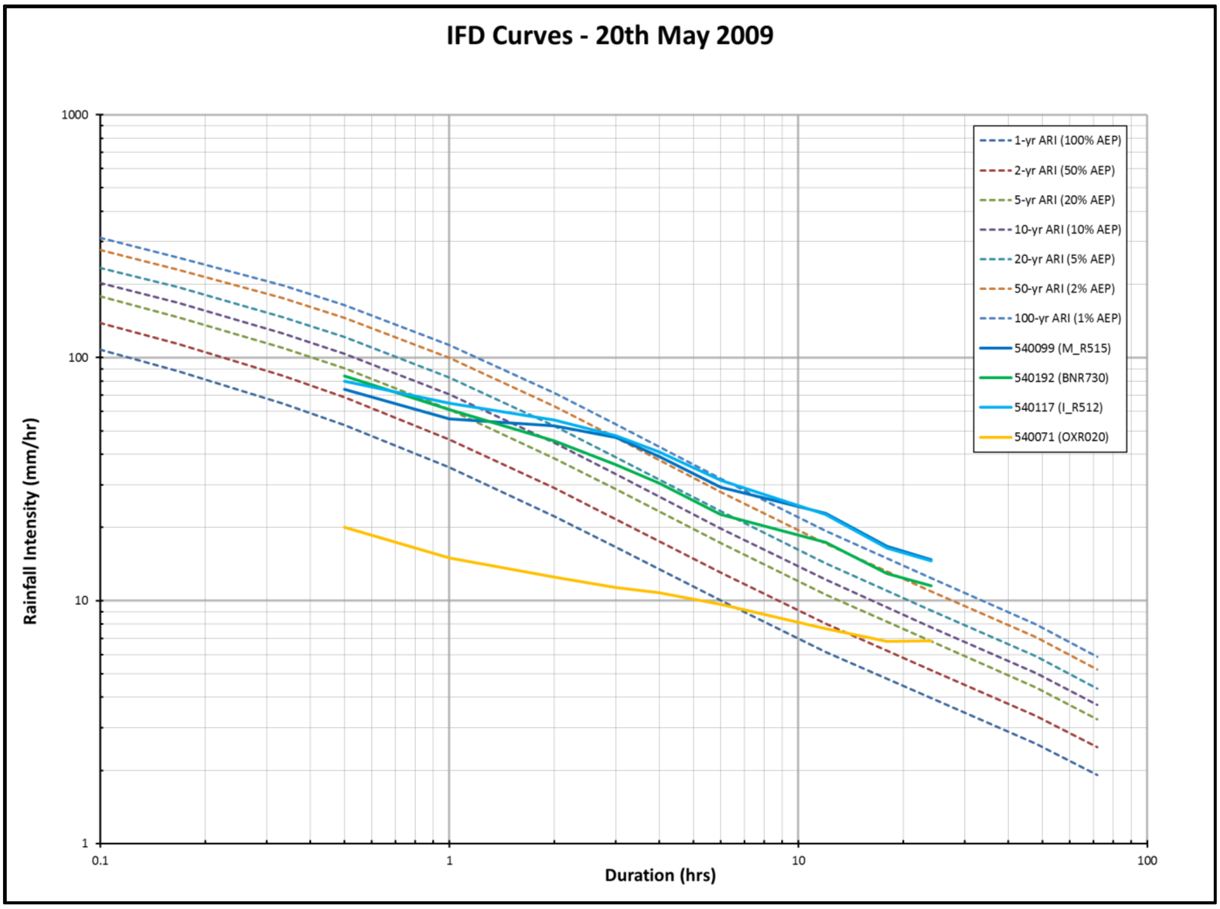


Figure 3.4: IFD Curve for May 2009 event.

3.3.4 November 2008 event

This event was also one of the largest in recent times and produced a flood level of 12.47 mAHD at MHG CB130 on Cubberla Creek at the confluence with the Akuna Street Branch. Moderate flooding occurred 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. The event was more intense in the upper section of the catchment with Rain Gauge 540117 (I_R512) at Mount Coot-tha recording a burst of 124 mm. During this 4 hour period, an average of 105 mm of rain fell on the middle and upper reaches of the catchment, compared with only 48 mm recorded in the lower reaches at the Jindalee Alert station.

The large spatial variability of the rainfall is not ideal for calibration as it leads to significant uncertainty with regard 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 the four rainfall stations. The catchment experienced between 127 mm and 171 mm of rainfall in the 14-day lead up to the event with between 96 mm and 148 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	Peak 1hr burst	Peak 6hr burst
540099	M_R515	Chadstone Close, Kenmore Hills	165	140	52	91
540117	I_R512	Mt Coot-tha	171	148	97	126
540192	BNR730	Brisbane River at Jindalee	127	108	32	51
540071	OXR020	Corinda High, Corinda	102	96	43	84

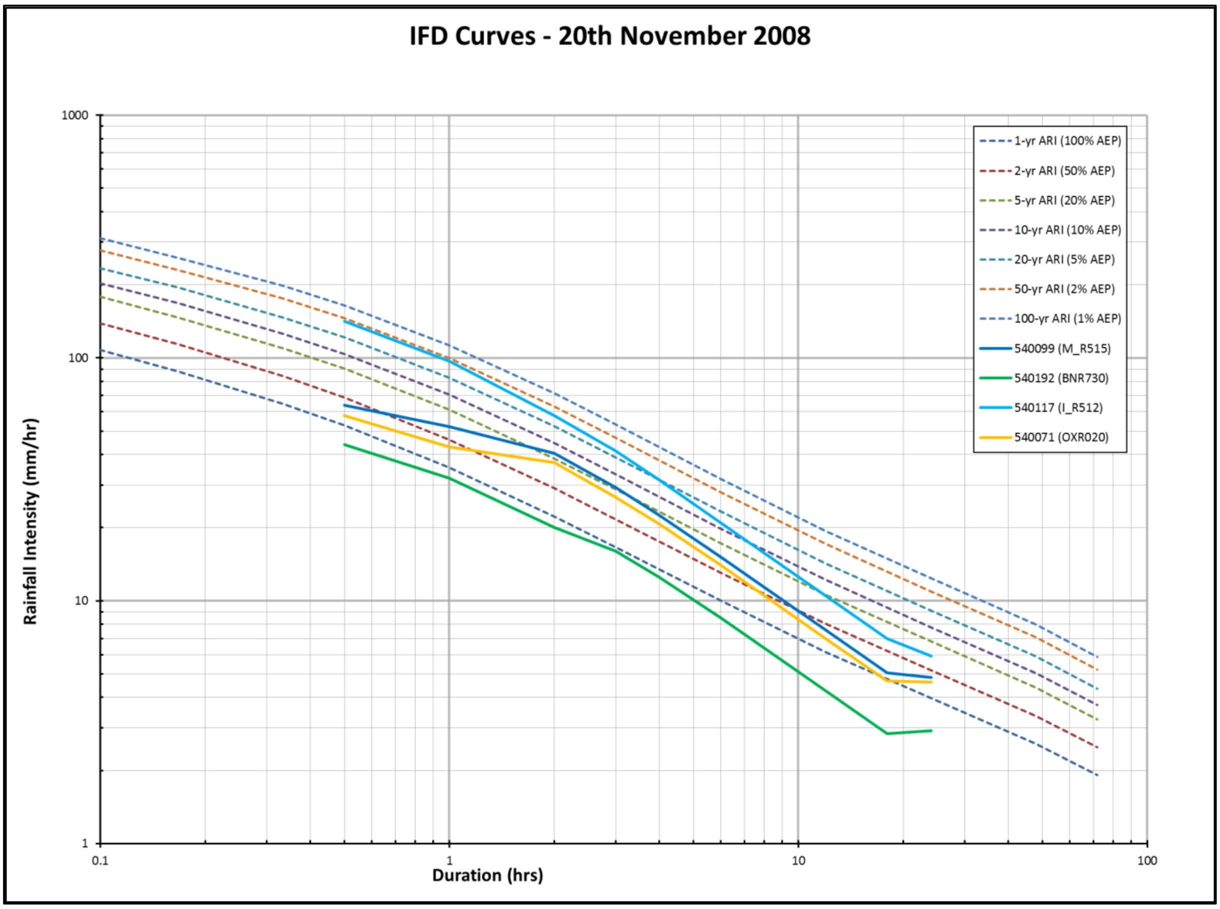


Figure 3.5: IFD Curve for November 2008 event.

Figure 3.5 provides a comparison of the IFD curve for the five rainfall stations against the AR&R 1987 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-routing process within the catchment. Hydrologic modelling for this study was performed using the URBS (version 5.85a) software. URBS allows the effects of development / urbanisation to be assessed, which makes it suitable for largely urbanised catchments such as Cubberla Creek. URBS also provides the option of modelling the sub-catchment and channel routing separately by selecting the “Split” modelling approach. This approach allows better compatibility with the hydraulic model, as the channel routing component can be matched to the hydraulic model, while varying the sub-catchment routing parameters to achieve calibration to recorded events.

An URBS model was previously developed for the Cubberla Creek Catchment as part of the 1996 Flood Study. This model was developed to be used in conjunction with the previous MIKE11 hydraulic model; which only modelled both Cubberla Creek and the Boblynne Branch. As this current study involves the hydraulic modelling of considerably more tributaries, the previous URBS model was considered unsuitable, which necessitated the development of a new URBS model.

Sub-catchment routing using the “Split” modelling approach 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]

4.2 URBS Sub-catchment Data

4.2.1 General

This section describes the sub-catchment information 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 (mandatory)
UL:	Urban Low Density Index
UM:	Urban Medium Density Index
UH:	Urban High Density Index
UR:	Urban Rural Index
I:	Impervious Fraction

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.

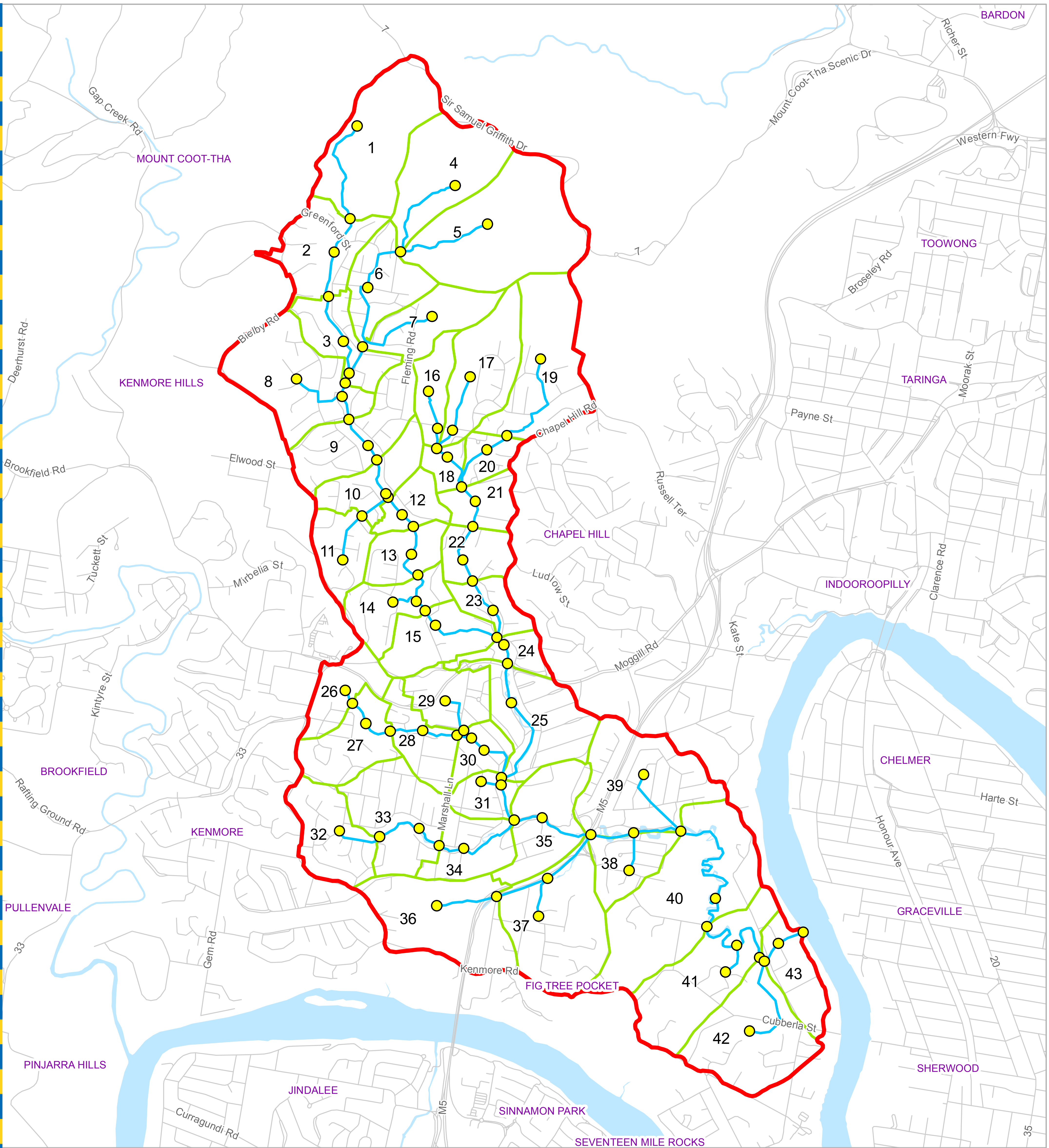
4.2.2 Sub-catchment Delineation

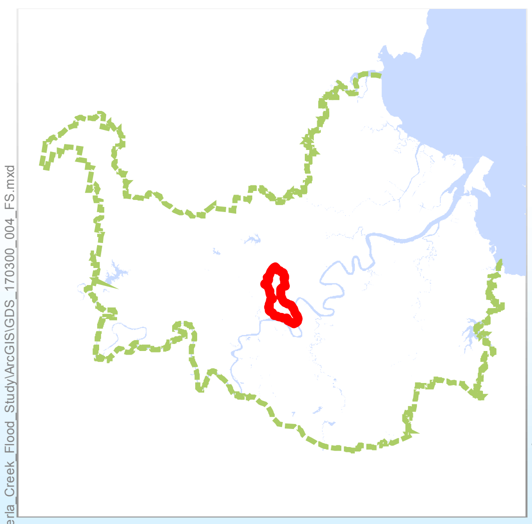
The URBS model was divided into 43 sub-catchments as indicated in Figure 4.1. Based on a total catchment area of 10.5 km^2, the resultant average sub-catchment size was 0.24 km^2. The sub-catchment delineation was based upon the 2014 ALS contours and considered the location of major tributaries and hydrometric gauges, as well as man-made boundaries such as the Western Freeway.

4.2.3 Land-use and Impervious Area

The effect of development / urbanisation is modelled in URBS using an Urbanisation Index (U) and Impervious Fraction (I). The Urbanisation Index (U) is used to determine the decrease in catchment lag and the Impervious Fraction (I) is used to determine the increase in runoff volume as a result of development. The Urbanisation Index (U) for each sub-catchment is determined with respect to the urbanisation indices; namely UL, UM, UH and UR for this study. These represent the fraction of the sub-catchment area occupied by that specific URBS urbanisation category. For example, a value of UL = 0.1 equates to 10 % of the sub-catchment area being occupied by the Urban Low Density (UL) urbanisation category.

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





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Legend

- URBS Nodes
- URBS Links/Routes
- URBS Subcatchments (1 - 43)
- Catchment Area
- Streets

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Figure 4.1: URBS Model Schematisation

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To determine the value of UL, UM, UH and UR for each sub-catchment it was firstly required to adopt impervious fractions for each and secondly determine the total impervious area.

Impervious Fractions

The urbanisation indices were assigned the following impervious fractions: UL (0.15), UM (0.5), UH (0.9) and UR (0.0 - default). The threshold Urban Impervious Fraction (UI) was assigned the default value of 0.5.

Total Impervious Area

Using the catchment land-use map from BCC City Plan 2014 and the adopted land-use percentage impervious (refer Appendix C); the total impervious area for the sub-catchment was able to be determined. The impervious fraction for the road reserve was assigned on a sub-catchment to sub-catchment basis to reflect the actual conditions. From this, the Impervious Fraction (I) for each sub-catchment was able to be determined.

Once the Impervious Fractions were assigned and the Total Impervious Area determined the following process was used to assign values to the urbanisation indices (UL, UM, UH and UR):

(i) Each BCC City Plan 2014 land-use category within the catchment was assigned to the most appropriate urbanisation index (UL, UM, UH or UR) and the respective area of each determined.
(ii) The impervious area for each sub-catchment was calculated using the adopted fraction impervious for each urbanisation index.
(iii) This calculated impervious area was compared to the total impervious area for each sub-catchment.
(iv) The values of the urbanisation indices were adjusted (as required) so that this calculated impervious area matched the total impervious area for each sub-catchment.

4.3 URBS Channel Data

URBS allows the user to define the channel with differing levels of detail depending on the type of catchment and requirements for the study. For this study the following parameters were utilised:

L: Channel length (mandatory)
Sc: Channel slope

The channel length was determined using GIS software and the channel slope from channel survey or 2014 ALS (at locations where channel survey was not available).

4.4 Gubberley Creek Detention Basin

4.4.1 General Description

Gubberley Creek Detention Basin is a small detention basin located approximately 900 m upstream of the confluence with Cubberla Creek. The detention basin was constructed in 1990 with the objective of reducing flood risk in the downstream Marshall Lane area. The bed level of the basin is approximately 23 mAHD and lowest elevation along the spillway is approximately 27.66 mAHD.

The detention basin consists of an unregulated low-flow pipe together with an unregulated overflow spillway (weir). The low-flow pipe is 900 mm in diameter and is able to fully drain the detention basin via a grated inlet as indicated in Figure 4.2.

Figure 4.2: Detention Basin Low-flow Grated Inlet

The grated inlet would appear to be at high risk from blockage by plant / leaf litter originating from within the basin. BCC Field Service Group (FSG) confirmed that following a sizeable storm event the approach channel and inlet grate for the low-flow piped outlet are inspected and debris removed (as necessary). FSG confirmed that there is no debris / vegetation removal undertaken within the greater basin storage area outside of this localised area.

The detention basin spillway (weir) is approximately 18 m long; constructed of gabions and stepped on the downstream side. Survey along the spillway and embankment crest was undertaken in 2015 as part of the Asset Maintenance Management Plan Level One Assessment (2016 AMMP). [6]

The major characteristics of the detention basin are indicated in Table 4.1.

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[6] Memorandum Gubberley Creek Detention Basin – AMMP Level One Assessment, BCC Flood Management 14th April 2016

Table 4.1 – Gubberley Creek Detention Basin Characteristics

Component	Details
Low-flow piped outlet size	900 mm dia
Low-flow piped outlet upstream invert level	22.99 mAHD
Low-flow piped outlet downstream invert level	22.58 mAHD
Spillway Weir Crest Level	27.66 mAHD (varies)
Spillway Weir Length	18 m (approx.)
Storage Capacity at 27.66 mAHD	8504 m^3
Surface Area at 27.66 mAHD	5833 m^2

4.4.2 Storage – Discharge Relationship

To enable the detention basin to be incorporated into the URBS hydrologic model, the storage-discharge relationship for the basin was required to be determined / sourced. The 2016 AMMP provided a stage-storage relationship which appeared to be derived in 1988 on the basis of digitising 1 m contours from a hardcopy survey plan. Given the more precise calculation methods available today, it was considered good practice to undertake a comparison using 2014 ALS data.

Table 4.2 provides a comparison of the stage-storage relationship indicating that at the spillway level (27.66 mAHD) the 2014 ALS storage is approximately 20% less than the circa 1988 storage. Given the likelihood of sediment accumulation (since 1988) together with the more precise calculation methods, it was decided to adopt the 2014 ALS stage-storage relationship.

Table 4.2 – Stage versus Storage Comparison

Stage (mAHD)	Area (m^2)		Volume (m^3)		Volume Difference (%)
	2014 ALS	Circa 1988	2014 ALS	Circa 1988	
23	0	0	0	0	0
24	65	233	19	78	-76
25	760	1214	348	737	-53
26	2475	2773	1955	2678	-27
27	4219	5241	5181	6620	-22
28	6940	8328	10647	13345	-20
29	10254	11661	19098	23293	-18
30	14497	15536	31569	36845	-14
31	19119	19431	48292	54293	-11

The stage-discharge relationship for the basin was derived from the TUFLOW hydraulic model and checked against a HEC-RAS model. This relationship makes allowance for decreased hydraulic efficiency due to the trash screen at the inlet of the low-flow pipe (refer QUDM page 7-90 (Eq. 7.26)).

Appendix B provides the adopted stage-storage-discharge relationship for the detention basin. As the likelihood of blockage of the low-flow piped outlet is high, discharges are provided for both a (i) fully open and (ii) fully blocked scenario. Considering a fully open low-flow pipe, the discharge through the pipe would be approximately 3.5 m^3/s when the basin water level was at the spillway crest level (27.66 mAHD).

4.5 Event Rainfall

4.5.1 Observed Rainfall

Recorded rainfall data from each calibration and verification event was incorporated into the URBS model at five minute 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 across two or 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.

4.5.2 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.

4.6 Calibration and Verification Procedure

4.6.1 General

The calibration and verification process was adopted to suit the study objectives in conjunction with the hydrometric data limitations. The general requirements were to produce a hydrologic model sufficiently robust to be used as a “standalone” model to accurately predict design discharges without the need to run the hydraulic model.

As there are no stream gauges within the catchment it was not possible to calibrate and verify the hydrologic model to observed hydrographs. This meant that it was not possible to calibrate and verify the volume and shape of the hydrograph, which are two important elements in a robust calibration process. As a result, the calibration and verification of the URBS model was required to be undertaken iteratively in conjunction with the TUFLOW model.

4.6.2 Methodology

The methodology undertaken for the hydrologic calibration and verification is as follows, noting that the results of the hydraulic calibration are presented in Section 5.

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 4.5.
2) Run the calibration events (i.e. May 2015, May 2009 and November 2008) through the URBS model to provide inflows for the TUFLOW model.
3) Using the URBS inflows, run the TUFLOW model and compare the modelled peak flood levels at the MHGs against the observed flood levels.
4) Iteratively adjust the URBS and TUFLOW model parameters and re-run the models to achieve the best possible match with the MHG data. The predominant URBS model parameters adjusted included the IL (mm); CL (mm/hr); catchment lag parameter (β) and catchment non-linearity parameter (m).
5) Compare the URBS and TUFLOW hydrographs for all events at a number of locations within the model extents. Adjust the URBS channel lag parameter (α) and the reach length factor (f) to replicate the results of the TUFLOW model.
6) Repeat Steps 2 to 5 as necessary.
7) Adopt a single set of URBS model parameters (typically CL, α, β and m) based on the calibration results.
8) Run the verification event (i.e. January 2013) through the calibrated URBS and TUFLOW models and compare the peak flood levels at the MHGs against the observed flood levels. Make adjustments to the URBS IL (mm) to represent the event specific rainfall lost at the start of the event.

The hydraulic calibration and verification tolerances are indicated in Section 5.4. In terms of the URBS model successfully replicating the TUFLOW model, the following tolerances were adopted:

- 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.

4.7 Simulation Parameters

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

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	0.5
May 2009	19/05/09 18:00	21/05/09 8:00	38	0.5
January 2013	26/01/13 18:00	28/01/13 18:00	48	0.5
May 2015	01/05/15 06:00	02/05/15 06:00	24	0.5

4.8 Hydrologic Model Calibration Results

As the URBS model calibration and verification was required to be undertaken in conjunction with the TUFLOW model, the peak flood level results are presented in Sections 5.5 and 5.6. Consistency checks between the URBS and TUFLOW models are presented in Section 5.8.

The first calibration run used URBS parameters that were based on the adjacent and recently completed 2015 Moggill Creek Flood Study.[7] The calibration and verification of the Moggill Creek URBS model used the same historical events, however was fortunate to have three stream gauges from which to better assess the shape and volume of the flood hydrograph. The initial parameter values used were as follows:

- Catchment lag parameter (β) = 5
- Channel lag parameter (α) = 0.008
- Catchment non-linearity parameter (m) = 0.65
- Impervious Area: IL = 0 mm, CL = 0 mm/hr (URBS default)
- Pervious Area: CL = 2.5 mm/hr
- May 2015 Pervious Area: IL = 35 mm
- May 2009 Pervious Area: IL = 10 mm
- November 2008 Pervious Area: IL = 0 mm

Using the methodology outlined previously in Section 4.6, the calibration was undertaken until the results were considered satisfactory. During the calibration process, the catchment lag parameter (β) was required to be decreased from the initial value to better match the peak flood levels. However, the remainder of the parameters were able to be kept at the initial values to achieve a satisfactory calibration.

[7] Brisbane City Council 2015, *Moggill Creek Flood Study*

Table 4.4 indicates the parameters adopted from the hydrologic calibration of the three historical events.

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	2
m	Catchment non-linearity parameter	0.65

4.9 Hydrologic Model Verification Results

The adopted URBS parameters were used to verify the URBS model to the one verification event (i.e. January 2013). The URBS pervious area Initial Loss (IL) value was adopted as 15 mm, which is also the same as used for the 2015 Moggill Creek Flood Study.

A satisfactory verification was achieved for the January 2013 event. The peak flood level results are presented in Section 5.6 and consistency checks in Section 5.8.

4.10 URBS Model Consistency Checks (Historical Events)

As noted above, the results of the hydrologic – hydraulic model consistency checks are presented in Section 5.8. As part of these consistency checks, the URBS model channel routing was adjusted in order to better replicate the shape and timing of the TUFLOW model hydrograph. This was undertaken by using one of the following means:

- Increasing the reach length factor (*f*); or
- Using Level-pool (reservoir) routing in lieu of Muskingum channel routing

There were two areas for which level-pool routing was used in lieu of Muskingum channel routing to better represent the storage effects. For both of these areas, the stage–storage relationship was derived using the 2014 ALS data and the stage–discharge relationship from the TUFLOW model results. These areas were as follows:

- Upstream of the Western Freeway incorporating the sporting fields on the left-hand side floodplain.
- Between the Western Freeway and the Brisbane River incorporating the wide expansive floodplain areas (AMTD 2100 to AMTD 0).

The reach length factor was increased to better match the TUFLOW routing for the majority of the waterways as indicated in Table 4.5.

Table 4.5 – Adopted Reach Length Factor (*f*)

Creek	Adopted Value
Cubberla	1.0 to 2.0
Boblynne Street Branch	1.3
Gubberley Creek	1.0 to 1.5
Akuna Street Branch	1.0 to 2.0
Tributary C	1.0

5.0 Hydraulic Model Development and Calibration

5.1 Overview

The previous hydraulic model of Cubberla Creek was a 1d MIKE11 model, developed for the 1996 Flood Study. To achieve best practice, it was considered appropriate to upgrade this 1d model into a 1d / 2d model. This would provide better representation of the floodplain flooding characteristics in the middle to lower sections of the creek as well as a more efficient tool to produce flood mapping products.

The TUFLOW hydrodynamic model (version 2016-03-AC) was selected for the hydraulic analysis of the Cubberla Creek Catchment.

5.2 Available Data

5.2.1 Utilised Data

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

- MIKE11 model – 1996 Flood Study
- HEC2 model – 1996 Flood Study
- 1995 cross-section survey of Cubberla Creek and the Boblynne Street Branch
- 2009, 2010 and 2011 detailed survey for proposed Cubberla Creek Bikeway
- 2016 cross-section survey (35 x cross-sections)
- Aerial photography – 1995 to 2015
- 2014 Airborne Laser Scanning (ALS) data
- BCC City Plan 2014
- Hydraulic structure drawings / reference sheets. Refer to Appendix J for further details.
- QLD Digital Cadastre Database (DCDB)
- BCC GIS databases

5.2.2 Cadastre Issues

In the upper catchment area, there appears to be a mismatch between the property boundary alignment from the Cadastre and the Aerial Photography. It would appear that the problem is with the Cadastre and not the Aerial Photography. At locations where there are obvious issues, the Aerial Photography has been used in lieu of the Cadastre for determining the approximate location of property boundaries.

5.3 Model Development

5.3.1 Model Schematisation

Figure 5.1 indicates the extent 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:

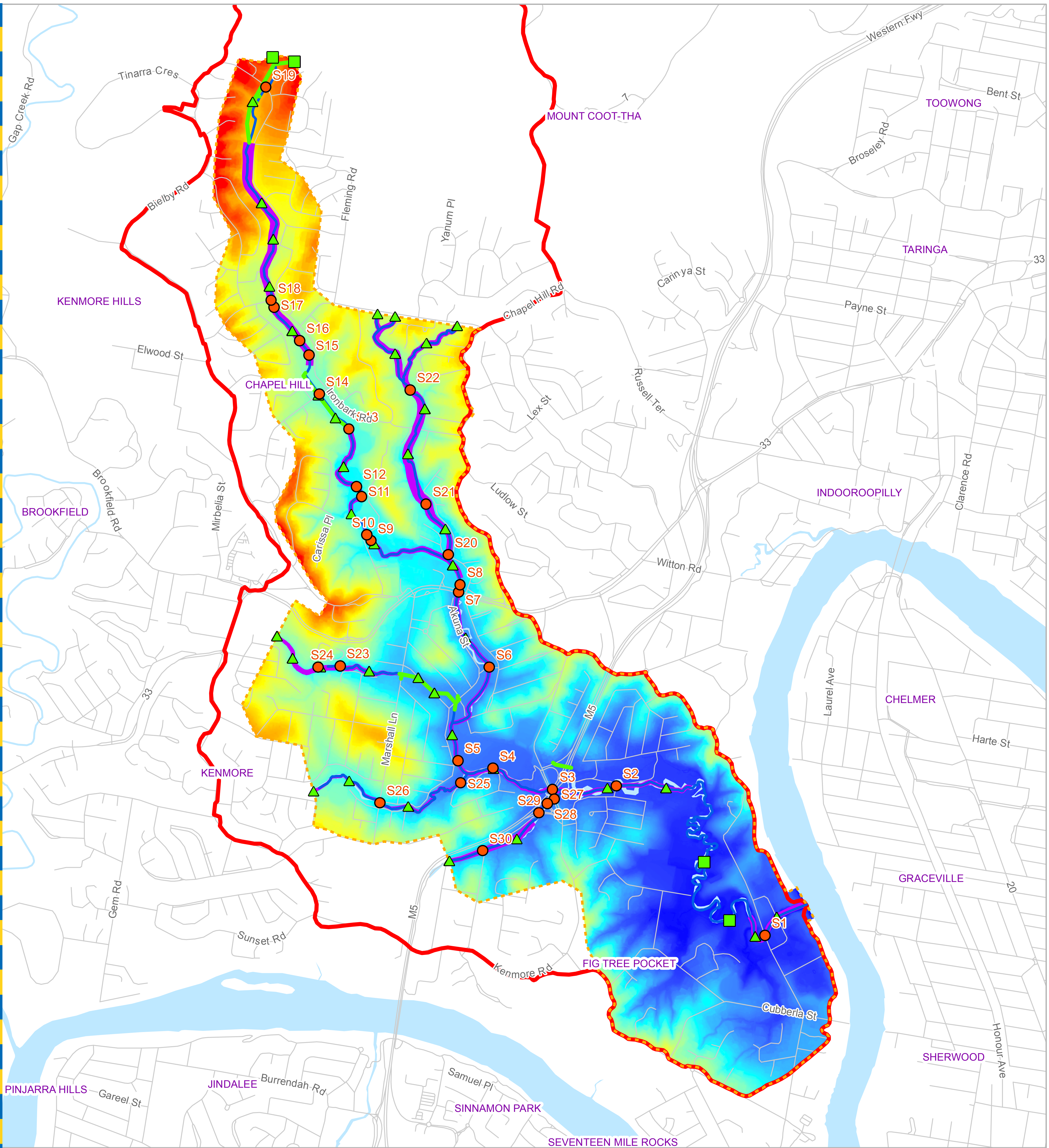
- Cubberla Creek (Upper Reach – Greenford Street to Moggill Road) – the modelled reach extends from upstream of Greenford Street to Moggill Road; a length of approximately 3.6 km. The upstream reach typically flows through dedicated public land, whereas from downstream of the Chapel Hill State School, a significant portion of the reach flows through private property without a designated easement (or waterway reserve). The reach is typically open channel; however there are two significant sections which include a low-flow pipe in conjunction with high-flow channel (discussed further in Section 5.3.5). The lower section of the reach passes close to Kenmore Plaza, where the creek is highly constrained and as a result has been engineered significantly to convey high flows. There are four main road crossings which include Greenford Street; Dillingen Street; Goolman Street and Tristania Road. This reach has been typically modelled as 1d / 2d throughout its entire length.

- Boblynne Street Branch – the modelled reach extends from downstream of Fleming Road to the confluence with Cubberla Creek; a length of approximately 1.6 km. The upper section is highly incised, which is similar for Tributaries A and B, that join the reach approximately 500 m downstream of Fleming Road. The upstream section is heavily vegetated, which tends to reduce in the downstream direction along the entire reach. There are three hydraulic structures within the reach, one of which is the crossing of the channel by two large bulk water supply pipelines owned by Seqwater. The Boblynne Street Branch including Tributaries A and B have all been modelled as 1d / 2d.

- Cubberla Creek (Middle Reach – Moggill Road to Western Freeway) – this reach extends from Moggill Road to the Western Freeway, a length of just over 1.6 km. The upper section flows through parkland / reserve, which is up to 115 m in width. The lower section opens out into a wide floodplain and incorporates a number of sporting ovals. Both Gubberley Creek and the Akuna Street Branch join Cubberla Creek within the mid to lower section of this reach. The reach is modelled entirely as 1d / 2d, with the 2d representation in the lower section being particularly important to model the complex floodplain hydraulics. The two major hydraulic structures are Moggill Road and the Western Freeway, which are discussed further in Section 5.3.4. Apart from these two major hydraulic structures, the others are minor pedestrian bridge crossings.

- Gubberley Creek – this reach has been modelled from downstream of Kenmore State School to the confluence with Cubberla Creek (including the Gubberley Creek Detention Basin), a length of approximately 1250 m. The reach is open waterway from the upstream extent to Marshall Lane, and has been modelled as 1d / 2d. Downstream of Marshall Lane, the trunk drainage pipework and overland flow paths have been both incorporated to more accurately represent the flow routing through this urbanised area. Modelling of the detention basin is discussed further in Section 5.3.6.

- Akuna Street Branch – this reach flows in an easterly direction and has been modelled for a length of 1.1 km. The reach flows predominantly through parklands, which include: Henry Clarkson Park; Wallawa Street Park and Katunga Street Park. The major waterway crossing is Marshall Lane, which is located approximately 640 m upstream of the confluence with Cubberla Creek. The reach is modelled as 1d / 2d apart from a small section adjacent to Cubberla Creek, which is fully 2d.



Legend

- 2d Inflow Locations
- Hydraulic Structures
- 1d Inflow Locations
- 1d Piped Drainage
- Creek Centreline
- Model Boundary
- 1d Channel Boundary
- Catchment Area
- Streets

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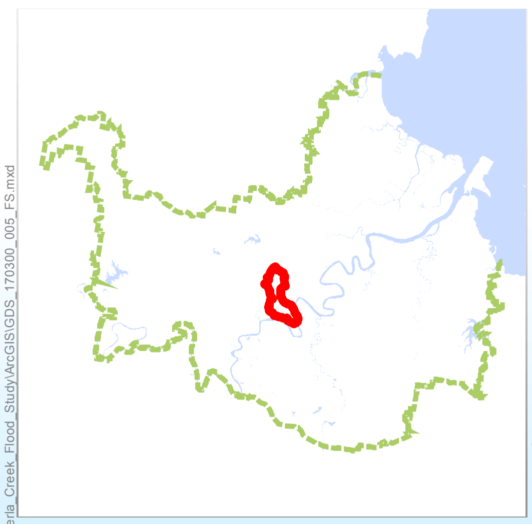

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DATA INFORMATION

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Cubberla Creek Flood Study

Figure 5.1: TUFLOW Model Schematisation

GDS - 170300 - 005

- Tributary C – this reach flows parallel to the Western Freeway and has been modelled for a length of 740 m. This reach joins Cubberla Creek at the downstream side of the Western Freeway Bridge crossing. The modelled reach has one pedestrian bridge crossing and three culvert crossings. The upstream and middle sections of the modelled reach are reasonably uniform in cross-section and have been modelled as 1d / 2d. The lower section is complex and is crossed by Fig Tree Pocket Road as well as the On and Off ramps of the freeway. The lower section has been modelled in 2d to allow for better representation of the complex flow patterns.

- Cubberla Creek (Lower Reach – Western Freeway to Brisbane River) – this reach extends from downstream of the Western Freeway to the confluence with the Brisbane River; a length of approximately 3.3 km. The creek traverses predominantly parkland and sporting fields and the floodplain is up to 400 m wide in places; which provides significant flood storage. The middle section of this reach is highly sinuous, whereas the upper and lower sections are quite straight. As a result the middle section has been modelled as fully 2d, whereas the upper and lower sections as 1d / 2d. The lower section passes through the natural levee of the Brisbane River which results in quite an incised cross-section with bank heights of over 10 m. The three span Jesmond Road Bridge is the major hydraulic structure within this reach.

5.3.2 Topography

1d Domain

The 1d open channel for Cubberla Creek was typically represented by utilising the channel cross-sectional information from a number of sources. Those sources included:

- 1995 cross-section survey of Cubberla Creek and the Boblynne Street Branch
- 2009, 2010 and 2011 detailed survey for proposed Cubberla Creek Bikeway
- 2016 cross-section survey

The 1995 cross-sectional survey is the most comprehensive dataset available and extends from downstream of Dillingen Street to the Brisbane River. From upstream of Greenford Street to Dillingen Street, new cross-sectional survey was undertaken (comprising 12 x cross-sections) in August 2016.

For the Boblynne Street Branch, the 1995 cross-sectional data was supplemented with August 2016 survey and 2014 ALS data. 2014 ALS was considered suitable for the steep incised upper section of the Boblynne Street Branch as well as Tributaries A and B. This is because these sections have significant capacity and floodwater is contained within the channel in extreme events.

Existing surveyed cross-sectional data was not available for the section of Gubberley Creek from the detention basin to Marshall Lane. Therefore, five new cross-sections were acquired as part of the August 2016 survey.

The Akuna Street Branch was extended upstream of Marshall Lane by five new cross-sections acquired as part of the August 2016 survey. Downstream of Marshall Lane the 1995 cross-sectional information was utilised.

Tributary C was not previously modelled as part of the 1996 Flood Study; therefore four new cross-sections were acquired as part of the August 2016 survey.

2d Domain

The 2d bathymetry consisted of a 4 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 on the 28th October 2014.

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."

The predominant location where the channel was represented as fully 2d is the 2.1 km long section upstream of Jesmond Road. For this sinuous reach, the TUFLOW gully line approach was utilised whereby one 4 m grid cell is lowered at the centre of the channel using linear interpolation between the invert levels of the upstream and downstream cross-sections. For this purpose, there were five 1995 survey cross-sections available within this reach. Using this approach, if the elevation of the grid cell is lower than the linearly interpolation level then the elevation is not changed; only higher elevations are lowered to the interpolated level.

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.

In the 1d ESTRY section, the Manning's 'n' values ranged from 0.015 to 0.15, depending on the type of channel material and degree of vegetation.

5.3.4 Hydraulic Structures – Culverts and Bridges

The major bridge and culvert structures within the model extents were represented in the TUFLOW model. These structures generally consisted of road crossings, footbridges and a small number of private access crossings. The most significant structure is the Western Freeway – Fig Tree Pocket Road Interchange, which comprises a bridge (and bikeway) crossing of Cubberla Creek and three culvert crossings of Tributary C. This interchange is quite complex and is discussed separately as part of this section.

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.

Table 5.1 – Adopted TUFLOW 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

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
Cubberla	S1	283	Jesmond Road	Three span road bridge	1d bridge / 2d weir	As-constructed drawings + creek survey (1995)
Cubberla	S2	2376	Dobell Street	Single span footbridge	1d bridge / 1d weir	Design drawings + creek survey (circa 2009)
Cubberla	S3	2718	Western Freeway	Single span road bridge	1d bridge / 2d weir	DTMR design drawings + creek survey (1995 & 2016) + 2014 ALS
Cubberla	S4	3075	Garaboo Street	Single span footbridge	1d bridge / 1d weir	1996 HEC2 + onsite measurements + creek survey (1995)
Cubberla	S5	3297	Akuna Street	Single span footbridge	1d bridge / 1d weir	Design drawings + onsite measurements + creek survey (circa 2011)
Cubberla	S6	3888	Henry Street	Single span footbridge	1d bridge / 1d weir	Detailed survey (circa 2011) + onsite measurements
Cubberla	S7	4336	Moggill Road	1 / 7.92 x 5.38 m RCBC	1d culvert / 2d weir	Design drawings + onsite measurements + 2014 ALS
Cubberla	S8	4376	Moggill Road (Upstream)	2 / 3.66 x 3.34 m RCBC	1d culvert / 2d weir	1996 HEC2 and HSRS
Cubberla	S9	4968	D/S Tristania Road	2 x Bulk water mains	1d bridge / 1d weir	BCC records + 1996 HSRS + creek survey (1995)
Cubberla	S10	5006	Tristania Road	1 / 3.05 x 3.01 m RCBC	1d culvert / 2d weir	1996 HEC2 and HSRS + onsite measurements
Cubberla	S11	5251	56 Tristania Road	Multi-span private bridge	1d bridge / 1d weir	Creek survey (2016) + onsite measurements
Cubberla	S12	5309	70 Tristania Road	Three span private bridge	1d bridge / 1d weir	Creek survey (2016) + onsite measurements
Cubberla	S13	5692	Chapel Hill State School	4 / 2.4 x 1.8 m RCBC	1d culvert / 2d weir	1996 HEC2 & HSRS + 2014 ALS
Cubberla	S14	5937	Goolman Street	3 / 3.05 x 1.22 m RCBC + varying size single culvert	1d culvert / 2d weir	Design drawings

Creek	Structure ID	AMTD	Structure location	Structure details	Modelled structure representation	Origin of data used for coding the structure
Cubberla	S15	6159	57 Ironbark Road	Drop structure	1d weir	Creek survey (1995)
Cubberla	S16	6249	75 Ironbark Road	Drop structure	1d weir	Creek survey (1995)
Cubberla	S17	6474	93 Ironbark Road	Drop structure	1d weir	Creek survey (1995)
Cubberla	S18	6512	Dillingen Street	3 / 2.7 x 1.8 m RCBC + 1 / 3 x 2.64 m RCBC	1d culvert / 2d weir	Design drawings + 2014 ALS
Cubberla	S19	N/A	Greenford Street	1 / 1.8 m diameter RCP	1d culvert / 2d weir	Design drawings + onsite measurements + 2014 ALS
Boblynne	S20	20	St. James Estate Access	2 / 3.34 x 3.05 m RCBC	1d culvert / 1d weir	1996 HEC2 & HSRS + onsite measurements + 2014 ALS
Boblynne	S21	330	80 Boblynne Street	2 x Bulk water mains	1d bridge / 1d weir	BCC records + 1996 HSRS + creek survey (1995)
Boblynne	S22	N/A	8 Alana Circuit	2 / 1.65 m diameter RCP	1d culvert / 2d weir	Design drawings + 2014 ALS
Gubberley	S23	N/A	Cedar Xing	2 / 1.65 m diameter RCP	1d culvert / 2d weir	BCC records + creek survey (2016) + 2014 ALS
Gubberley	S24	N/A	60 Gubberley Street	Detention Basin	1d / 2d storage - 1d culvert - 1d spillway - 1d / 2d weir	Design drawings + 2014 Pipe survey + 2014 ALS
Akuna	S25	62	Katunga Street	2 / 1.5 m diameter RCP	1d culvert / 1d weir	1996 HEC2 & HSRS + 2014 ALS
Akuna	S26	N/A	Marshall Lane	1 / 1.5 m diameter RCP	1d culvert / 2d weir	BCC Records + 2014 ALS
Tributary C	S27	N/A	Fig Tree Pocket Road	2 / 1.8 m diameter RCP	1d culvert / 2d weir	DTMR design drawings + 2014 ALS
Tributary C	S28	N/A	Western Freeway Off Ramp	2 / 1.8 m diameter RCP	1d culvert / 2d weir	DTMR design drawings + 2014 ALS
Tributary C	S29	N/A	Western Freeway On Ramp	3 / 1.5 x 1.2 m RCBC	1d culvert / 2d weir	DTMR design drawings + 2014 ALS
Tributary C	S30	N/A	Norman Street	Single span footbridge	1d bridge / 1d weir	DTMR design drawings + creek survey (2016)

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).

Three of the more complex hydraulic structures are discussed as follows:

Goolman Street Crossing (S14)

This crossing is quite complex as it incorporates a four cell box culvert of which the most western cell allows flow transfer to a separate 1.8 m diameter low-flow pipe (discussed in Section 5.3.5) via a large manhole / chamber located in Goolman Street. This large chamber has been represented in 1d and allows connection to both the 1d pipe and 1d box culvert. The remaining three cells of the box culvert only receive flow from the high-flow channel, which is represented as fully 2d.

Moggill Road Crossing (S7 and S8)

This crossing consists of two inline culverts with a total length of just over 82 m. The upstream culvert is approximately 56 m in length and consists of 2 / 3.66 x 3.34 m RCBCs. The second culvert (underneath Moggill Road) consists of 1 / 7.92 x 5.38 m RCBC and is approximately 26 m long. The join between the culverts consists of a large chamber, located immediately upstream of Moggill Road. The chamber is open in both the horizontal and vertical planes and allows 2d overflows from the upstream culvert to enter the downstream culvert if there is sufficient hydraulic capacity. The flow interchange at the chamber between the 2d channel and the 1d culvert was modelled to occur “freely” whereby the control will be the limiting size of the downstream culvert and not the size of the chamber inlet.

Western Freeway Crossing (S3)

This crossing consists of a single span bridge in combination with a very complex overflow arrangement. The bridge has been modelled in 1d and the overflow in 2d in order to best represent the complex hydraulic behaviour that occurs once there is flooding of the freeway.

There are numerous obstructions to flow across the freeway, such as Armco crash barriers, concrete impact barriers and noise barriers. For the purposes of modelling, these barriers have been assumed to be impervious and able to withstand the impact forces from the flow. This is an assumption and in reality it is likely that some of these barriers would withstand the force of the flood water and some would not. As a result, a sensitivity analysis of design and extreme flood levels with respect to the barrier assumptions is provided in Section 6.4.3 and Section 7.5.2.

From upstream to downstream, the following barriers have been incorporated into the hydraulic model using the TUFLOW “z-shape” function, which typically alters the elevation of the base grid cell.

- Upstream Noise Barrier – this barrier is located on the right side of the channel (looking downstream) and the level at the top of the barrier has been taken as 3.5 m above the ground level, which is based on 2014 ALS.
- Concrete Median Barrier – this barrier provides a continuous obstruction along the entire width of the floodplain. The level at the top of the barrier varies and was taken from DTMR design drawings.
- Downstream Concrete Crash Barrier – this barrier is located within the left floodplain (looking downstream) and runs intermittently on the westbound carriageway shoulder until the Fig Tree Pocket Bridge. The level at the top of the barrier has been taken as 0.82 m above the ground level (based on the 2d grid).

- Downstream Noise Barrier – this barrier is located on the left side of the channel (looking downstream) and the level at the top of the barrier has been taken as 3.35 m above the ground level, which is based on 2014 ALS. This barrier is not fully continuous and there are two openings where the bikeway alignment leaves (and returns) to the shoulder of the westbound carriageway. These openings have been provided in the TUFLOW model.
- Armco barriers – numerous Armco barriers have been included in the vicinity of the Freeway On / Off ramps.

5.3.5 Piped Drainage

Although this flood study is essentially of open channel / creek systems, it was considered necessary to include piped drainage in three areas to more accurately determine flood levels. In all three areas the flow interchange between the 2d channel and the 1d pipe network was assumed to occur "freely" at the inlet pits. This assumes that the hydraulic control will be the limiting size of the pipe and not the size of the pit inlet.

These three areas where piped drainage has been included are discussed below:

<u>Cubberla Creek - Greenford Street</u>

From approximately 150 m upstream of Greenford Street to 320 m downstream, the reach consists of a low-flow pipe in conjunction with a high-flow open channel. The low-flow pipe size ranges between 1.2 m and 1.5 m diameter. The low-flow pipe was modelled in 1d and the high-flow channel in 2d. Inlet pits in the park downstream of Greenford Street were included to allow the transfer of flow between the 1d piped network and 2d channel.

<u>Cubberla Creek - Goolman Street</u>

From approximately 130 m upstream of Goolman Street to 220 m downstream, the reach consists of a low-flow pipe in conjunction with a high-flow open channel. The low-flow pipe size is 1.8 m diameter and has been represented in the model as 1d with the high-flow channel in 2d. As noted previously, flows are able to transfer between the Goolman Street culvert and this pipe via a large manhole / chamber at Goolman Street.

The outlet of this pipe is upstream of the Chapel Hill State School Culvert. At this location, the invert level of the pipe is lower than the invert level of the box culvert by more than 0.5 m; which results in water constantly sitting in the bottom of the pipe, as was witnessed during a site visit in January 2017.

<u>Gubberley Creek – downstream of Marshall Lane</u>

From Marshall Lane to the confluence with Cubberla Creek (a length of approximately 430 m), the creek is piped through a low-density residential area. This reach is typical of an urban drainage network whereby the minor flow is conveyed by pipework and the overland flow by the road reserve and / or a designated overland flow path. The low-flow pipe was modelled in 1d and the overland flow areas in 2d.

5.3.6 Gubberley Creek Detention Basin

The details of the Gubberley Creek Detention Basin have been discussed previously in Section 4.4. The basin has also been included in the TUFLOW hydraulic model as the hydraulic model was used to derive the outlet rating curve which was required for the URBS hydrologic model.

The basin was modelled in TUFLOW as 1d / 2d meaning the storage volume was derived from both the storage in the 1d channel and the 2d grid. The 900 mm diameter low-flow pipe was incorporated as a 1d pipe using invert levels from the design drawings.

The spillway was modelled using a 1d weir (in lieu of 2d) as it was considered important to use the actual dimensions of the spillway, which is not always possible using the 2d approach. The spillway dimensions were adopted from the 2015 survey undertaken for the 2016 AMMP.

5.3.7 Drop Structures

There are three drop structures (small weirs) on Cubberla Creek between Dillingen Street and Goolman Street. These were surveyed in 1995 and do not appear to have changed since this time. Each drop structure has been represented in TUFLOW as a 1d weir.

5.3.8 Boundary Conditions

Inflow Boundaries

Inflows to the hydraulic model were taken from the URBS hydrologic model. All inflows were represented as a discharge versus 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) boundary was used to represent the downstream boundary conditions at the mouth of Cubberla Creek. As there is no stream gauge at the mouth of Cubberla Creek, the H-T boundary was derived based on interpolation between the closest upstream and downstream river gauges. The mouth of Cubberla Creek is located along the Brisbane River at AMTD 43.8 km, resulting in the closest stream gauges being upstream at Jindalee (540192) AMTD 52.1 km and downstream at the mouth of Oxley Creek (540274) AMTD 38.7 km.

For the May 2015 and January 2013 events, the H-T boundary was interpolated based on the recorded data from the upstream Jindalee Alert Gauge (540192) and the downstream Oxley Creek Mouth Gauge (540274).

For the November 2008 event, data was not available for the upstream gauge(s) at Jindalee. The H-T boundary was interpolated based on the recorded data from the Moggill Gauge (540200) further upstream and the downstream Oxley Creek Mouth Gauge (540274).

For the May 2009 event, upstream data was not available to use in the interpolation. The H-T boundary was derived by adding 0.1 m to the downstream Oxley Creek Mouth Gauge (540274). This value is an estimate of the difference between the two locations at the peak of the tidal cycle and is based on observations from the other historical events.

5.3.9 Run Parameters

Time Step

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

Eddy Viscosity

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. The method uses the Smagorinsky formula with a “Constant Coefficient” of 0.1 and “Smagorinsky Coefficient” of 0.2. This method has been successfully used on other similar BCC flood studies.

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 (not applicable for this study as there are no stream gauges).
- 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 (not applicable for this study as there are no stream gauges).

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 the MHGs.
4) Iteratively adjust the TUFLOW 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. Table 5.3 provides a comparison between the TUFLOW results and the recorded peak flood levels at the operational MHGs.

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

Gauge ID	Location	Recorded Peak WL (mAHD)	Simulated Peak WL (mAHD)	Difference (m)
Cubberla Creek				
CB100	U/S Jesmond Rd	-	3.19	-
CB110	280 m D/S of Dobell St Footbridge	6.83	6.76	-0.07
CB114	D/S Dobell Street Footbridge	7.6	7.47	-0.13
CB115	U/S Dobell Street Footbridge	-	8.14	-
CB120	U/S Western Freeway	-	10.49	-
CB130	Confluence of Akuna Street Branch	12.28	12.11	-0.17
CB140	Adjacent 95 Sutling Street	14.32	14.27	-0.05
CB150	U/S Moggill Road Culvert	-	19.61	-
CB160	130 m U/S of Goolman Street	30.64	30.70	0.06
CB170	Adjacent 29 Greenford Street	43.18	42.98	-0.20
Boblynne Street Branch				
CB310	U/S Brymer Street	26.82	26.92	0.13
Gubberley Creek				
CB200	U/S Marshall Lane	-	16.79	-
CB210	Gubberley Creek Detention Basin	26.88	26.99	0.11
CB220	Gubberley Creek Detention Basin	26.90	26.99	0.09

From review of the peak level / MHG results, it was apparent that at each operational MHG the simulated flood level was within the desired peak flood level tolerance.

In the upper portion of the catchment, the simulated flood levels were both higher and lower of the respective MHG levels. Downstream of MHG CB140, the simulated flood levels were consistently slightly lower than the MHG levels.

For the purposes of modelling, the low-flow pipe in the Gubberley Creek Detention Basin was assumed to be fully open (i.e. 0 % blockage) and the simulated flood level was within the desired tolerance.

5.5.2 May 2009

The May 2009 flood was simulated in TUFLOW for 38 hours from 6 pm on the 19th May 2009. Table 5.4 provides a comparison between the TUFLOW results and the recorded peak flood levels at the operational MHGs.

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

Gauge ID	Location	Recorded Peak WL (mAHD)	Simulated Peak WL (mAHD)	Difference (m)
Cubberla Creek				
CB100	U/S Jesmond Rd	3.82	3.77	-0.05
CB110	280 m D/S of Dobell St Footbridge	7.11	7.02	-0.09
CB114	D/S Dobell Street Footbridge	-	7.60	-
CB115	U/S Dobell Street Footbridge	-	8.28	-
CB120	U/S Western Freeway	11.27	11.26	-0.01
CB130	Confluence of Akuna Street Branch	12.36	12.36	0.00
CB140	Adjacent 95 Sutling Street	14.66	14.50	-0.14
CB150	U/S Moggill Road Culvert	20.29 (d)	20.45	0.16
CB160	130 m U/S of Goolman Street	-	30.98	-
CB170	Adjacent 29 Greenford Street	-	43.26	-
Boblynne Street Branch				
CB310	U/S Brymer Street	-	27.01	-
Gubberley Creek				
CB200	U/S Marshall Lane	18.34	18.37	0.03
CB210	Gubberley Creek Detention Basin	28.07	28.07	0.00
CB220	Gubberley Creek Detention Basin	~~27.75~~ (d) 28.07	28.07	0.00

(d) Reading from debris mark

From review of the peak level / MHG results, it was apparent that at each operational MHG the simulated flood level was within the desired peak flood level tolerance.

The MHG level of 27.75 mAHD within the detention basin at CB220 was from a debris mark(s). As this level should be a similar value to MHG CB210 (as they are both within the detention basin adjacent to each other) it was revised to the more accurate reading of 28.07 mAHD.

During the calibration process it became apparent that to match the peak water level within the detention basin there needed to be an allowance for blockage of the low-flow outlet pipe. As mentioned previously, this is conceivable as the grated inlet would appear to be at high risk from blockage by plant / leaf litter originating from within the basin. For the purposes of modelling, the low-flow pipe was assumed to be fully blocked (i.e. 100 % blockage) and the modelled flood levels matched exactly with the MHG level(s).

5.5.3 November 2008

The November 2008 flood was simulated in TUFLOW for 12 hours from 10 pm on the 19th November 2008. Table 5.5 provides a comparison of the TUFLOW results and the recorded peak flood levels at the operational MHGs.

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

Gauge ID	Location	Recorded Peak WL (mAHD)	Simulated Peak WL (mAHD)	Difference (m)
Cubberla Creek				
CB100	U/S Jesmond Rd	3.36	3.38	0.02
CB110	280 m D/S of Dobell St Footbridge	7.02	6.97	-0.05
CB114	D/S Dobell Street Footbridge	-	7.60	-
CB115	U/S Dobell Street Footbridge	-	8.27	-
CB120	U/S Western Freeway	11.25	11.05	-0.20
CB130	Confluence of Akuna Street Branch	12.47	12.39	-0.08
CB140	Adjacent 95 Sutling Street	14.63	14.58	-0.05
CB150	U/S Moggill Road Culvert	-	20.77	-
CB160	130 m U/S of Goolman Street	-	31.17	-
CB170	Adjacent 29 Greenford Street	-	43.43	-
Boblynne Street Branch				
CB310	U/S Brymer Street	-	27.16	-
Gubberley Creek				
CB200	U/S Marshall Lane	18.16	18.30	0.14
CB210	Gubberley Creek Detention Basin	28.15	28.02	-0.13
CB220	Gubberley Creek Detention Basin	~~26.25~~ 28.15	28.02	-0.13

From review of the peak level / MHG results, it was apparent that at each operational MHG the simulated flood level was within the desired peak flood level tolerance.

Downstream of MHG CB140, the simulated flood levels were consistently slightly lower than the MHG levels, apart from MHG CB100 which was slightly higher.

The MHG level of 26.25 mAHD within the detention basin at CB220 appears to be in error and it was disregarded and the higher reading of 28.15 mAHD from CB210 adopted. Similar to May 2009, it became apparent during the calibration process that to match the peak water level within the detention basin there needed to be an allowance for blockage of the low-flow outlet pipe. For the purposes of modelling, the low-flow pipe was assumed to be fully blocked (i.e. 100 % blockage) and the resultant flood level was within the desired tolerance. However, it should be noted that there are still some doubts about the accuracy of the recorded MHG peak level in the detention basin of 28.15 mAHD. This results because of the inconsistencies between the MHG levels at CB210/220 and CB200 when comparing the May 2009 and November 2008 events. For example, the MHG level at CB200 (downstream of the detention basin) is 18.34 mAHD for May 2009 and 18.16 mAHD for November 2008. As there are no major tributaries between the detention basin and CB200, it would be expected that the flow from the detention basin is higher in May 2009 than November 2008. Review of the MHG peak flood levels in the detention basin reveals that the November 2008 flood level is higher than the May 2009 level, which appears counter-intuitive and possibly in error.

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. Table 5.6 provides a comparison between the TUFLOW results and the recorded peak flood levels at the operational MHGs.

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

Gauge ID	Location	Recorded Peak WL (mAHD)	Simulated Peak WL (mAHD)	Difference (m)
Cubberla Creek				
CB100	U/S Jesmond Rd	3.87	3.93	0.06
CB110	280 m D/S of Dobell St Footbridge	6.91	6.83	-0.08
CB114	D/S Dobell Street Footbridge	7.64	7.51	-0.13
CB115	U/S Dobell Street Footbridge	8.37	8.18	-0.19
CB120	U/S Western Freeway	-	10.66	-
CB130	Confluence of Akuna Street Branch	12.32	12.19	-0.13
CB140	Adjacent 95 Sutling Street	-	14.33	-
CB150	U/S Moggill Road Culvert	-	19.78	-
CB160	130 m U/S of Goolman Street	30.59	30.71	0.12
CB170	Adjacent 29 Greenford Street	43.21	42.99	-0.22
Boblynne Street Branch				
CB310	U/S Brymer Street	26.75	26.93	0.18
Gubberley Creek				
CB200	U/S Marshall Lane	-	17.67	-
CB210	Gubberley Creek Detention Basin	27.87	27.99	0.12
CB220	Gubberley Creek Detention Basin	-	27.99	-

From review of the peak level / MHG results, it was apparent that at each operational MHG the simulated flood level was within the desired peak flood level tolerance.

Downstream of MHG CB130, the simulated flood levels were consistently slightly lower than the MHG levels, apart from MHG CB100 which was slightly higher.

Similar to May 2009 and November 2008, it became apparent during the calibration process that to match the peak water level within the detention basin there needed to be an allowance for blockage of the low-flow outlet pipe. For the purposes of modelling, the low-flow pipe was assumed to be fully blocked (i.e. 100 % blockage). The resultant flood levels are slightly high indicating that the actual blockage was most likely between 0 and 100 %.

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:

- Jesmond Road (S1)
- Dobell Street Footbridge (S2)
- Western Freeway (S3)
- Garaboo Street Footbridge (S4)
- Akuna Street Footbridge (S5)
- Henry Street Footbridge (S6)

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.

There were a number of locations where HEC-RAS was not able to replicate the complex flow behaviour and / or there were some anomalies because of the different assumptions and algorithms used in TUFLOW and HEC-RAS structure routines. These locations were as follows:

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 – Jesmond Road Bridge			
55.0	0.41	0.24	-0.17
100.3	0.56	0.34	-0.22
140.9	0.69	0.41	-0.28
257.8	0.89	1.04	0.15
358.9	1.08	1.02	-0.06
464.8	0.97	1.04	0.07
Structure S2 – Dobell Street Footbridge			
21.1	0.53	0.41	-0.12
35.5	0.61	0.43	-0.19
50.0	0.71	0.49	-0.22
66.3	0.67	0.46	-0.21
84.9	0.62	0.38	-0.24
98.0	0.52	0.29	-0.23
Structure S3 – Western Freeway Bridge			
31.2	0.06	0.09	0.03
61.5	0.09	0.18	0.09
88.3	0.15	0.28	0.13
113.4	1.67	0.38	-1.29 (see note)
139.6	1.44	1.70	0.26
Structure S4 – Garaboo Street Footbridge			
34.6	0.01	0.00	-0.01
57.7	0.16	0.19	0.03
138.9	0.10	0.12	0.02
188.4	0.12	0.15	0.03
259.4	0.16	0.23	0.07
298.2	0.20	0.28	0.08
Structure S5 – Akuna Street Footbridge			
40.3	0.01	0.04	0.04
69.5	0.02	0.05	0.03
119.3	0.02	0.06	0.04
144.4	0.03	0.05	0.02
167.8	0.03	0.05	0.02
190.3	0.02	0.04	0.02
Structure S6 – Henry Street Footbridge			
47.5	0.36	0.23	-0.13
98.6	0.21	0.35	0.15
146.6	0.32	0.40	0.08
244.5	0.49	0.47	-0.02
342.4	0.59	0.47	-0.12
440.6	0.59	0.46	-0.13

Akuna Street Footbridge
At flows above 200 m^3/s flow recirculation starts to occur in the vicinity of the Akuna Street Footbridge. HEC-RAS is unable to accurately replicate the complex flow behaviour, therefore comparative checks have not been undertaken at flows greater than 200 m^3/s.

Garaboo Street Footbridge
During larger floods where the floodplain is fully engaged, head-losses due to the footbridge only occur in the immediate vicinity of the bridge. There are no head-losses due to the structure on the sports ovals on the left floodplain. An extended cross-section HEC-RAS model is unable to model differential head-losses across the channel and floodplain. As a result, comparative checks have only been undertaken within the 1d channel.

Western Freeway
Comparative checks have been undertaken to the point of overtopping the median barrier as after this point the flow is complex and HEC-RAS is unable to accurately replicate the complex flow patterns.

There is also a large discrepancy between HEC-RAS and TUFLOW at a flow of 113.4 m^3/s when the downstream water surface is just below the soffit level of the bridge. Differences around the bridge soffit level are common as each model uses different criteria for changing to pressurised flow. Once a model changes to pressurised flow there is typically a sharp increase in head-loss across the structure. HEC-RAS changes to pressurised flow when the upstream total energy line (or optionally the upstream water surface) comes in contact with the bridge soffit. Whereas, TUFLOW changes to pressurised flow when the downstream water surface comes in contact with the bridge soffit. At 113.4 m^3/s, HEC-RAS has changed to pressurised, whereas TUFLOW has not yet changed, which is why there is a substantial difference. Results better align when the flows are higher and both models operate under pressurised flow conditions through the bridge opening.

Dobell Street Footbridge
Similar to the Garaboo Street Footbridge, during floods where the floodplain is fully engaged, head-losses due to the footbridge only occur in the immediate vicinity of the bridge. There are no head-losses due to the structure within the parkland in the floodplain areas. An extended cross-section HEC-RAS model is unable to model differential head-losses across the channel and floodplain. As a result, comparative checks have only been undertaken within the 1d channel.

5.8 Hydrologic-Hydraulic Model Consistency Checks (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 and as a means of confirming whether the URBS model was adequately calibrated. The locations where comparative plots were undertaken are as follows:

(i) Cubberla Creek – Goolman Street
(ii) Cubberla Creek – Moggill Road
(iii) Cubberla Creek – Western Freeway
(iv) Cubberla Creek – Outlet at Brisbane River
(v) Boblynne Street Branch - Confluence with Cubberla Creek
(vi) Gubberley Creek – Detention Basin Outlet

Figure 5.2 to Figure 5.9 provide comparative plots at Moggill Road and the Western Freeway on Cubberla Creek. The remainder of the comparative plots are provided in Appendix D.

Table 5.8 provides a comparison of the peak flows at these six locations plus some additional locations.

Table 5.8 – Peak Flow Comparison, URBS and TUFLOW

Location	Model	Peak Flow (m^3/s)			
		May 2015	Jan 2013	May 2009	Nov 2008
Cubberla Creek at Dillingen Street	URBS	32.2	31.3	50.1	70.7
	TUFLOW	30.4	30.6	48.5	67.0
Cubberla Creek at Goolman Street	URBS	38.6	38.5	59.8	78.6
	TUFLOW	37.4	37.9	58.7	75.0
Cubberla Creek at Moggill Road	URBS	55.3	62.8	85.8	102.4
	TUFLOW	57.4	63.9	84.6	102.3
Cubberla Creek at the confluence with Gubberley Creek	URBS	64.3	74.1	99.0	109.6
	TUFLOW	67.1	76.1	101.9	113.0
Cubberla Creek at the confluence with the Akuna Street Branch	URBS	70.8	82.2	109.7	114.5
	TUFLOW	71.7	83.9	115.7	121.8
Cubberla Creek at Western Freeway	URBS	72.7	84.8	112.7	113.6
	TUFLOW	70.6	81.6	110.9	118.3
Cubberla Creek at the confluence with the Brisbane River	URBS	76.6	82.7	105.7	84.5
	TUFLOW	78.7	87.0	111.2	87.8
Boblynne Street Branch at the confluence with Cubberla Creek	URBS	16.0	16.9	23.3	30.0
	TUFLOW	15.9	17.3	25.1	29.5
Gubberley Creek Detention Basin Outflow	URBS	3.2	5.6	7.7	6.4
	TUFLOW	3.2	5.3	7.5	6.1
Gubberley Creek at Marshall Lane	URBS	6.0	8.1	11.0	9.2
	TUFLOW	5.8	7.3	10.6	9.0
Akuna Street Branch at Marshall Lane	URBS	9.8	8.8	12.8	9.9
	TUFLOW	8.8	8.0	11.1	9.3

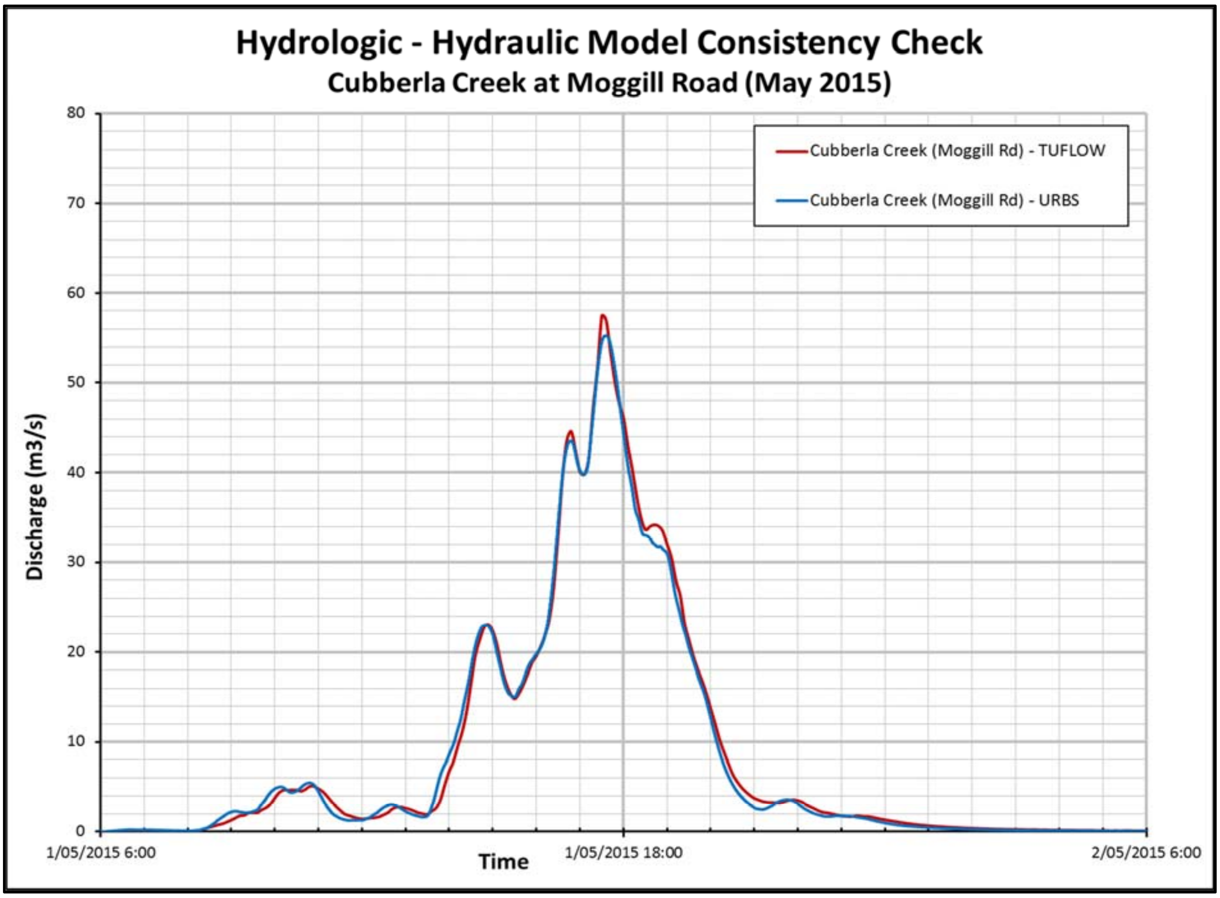


Figure 5.2: Cubberla Creek at Moggill Road (May 2015)

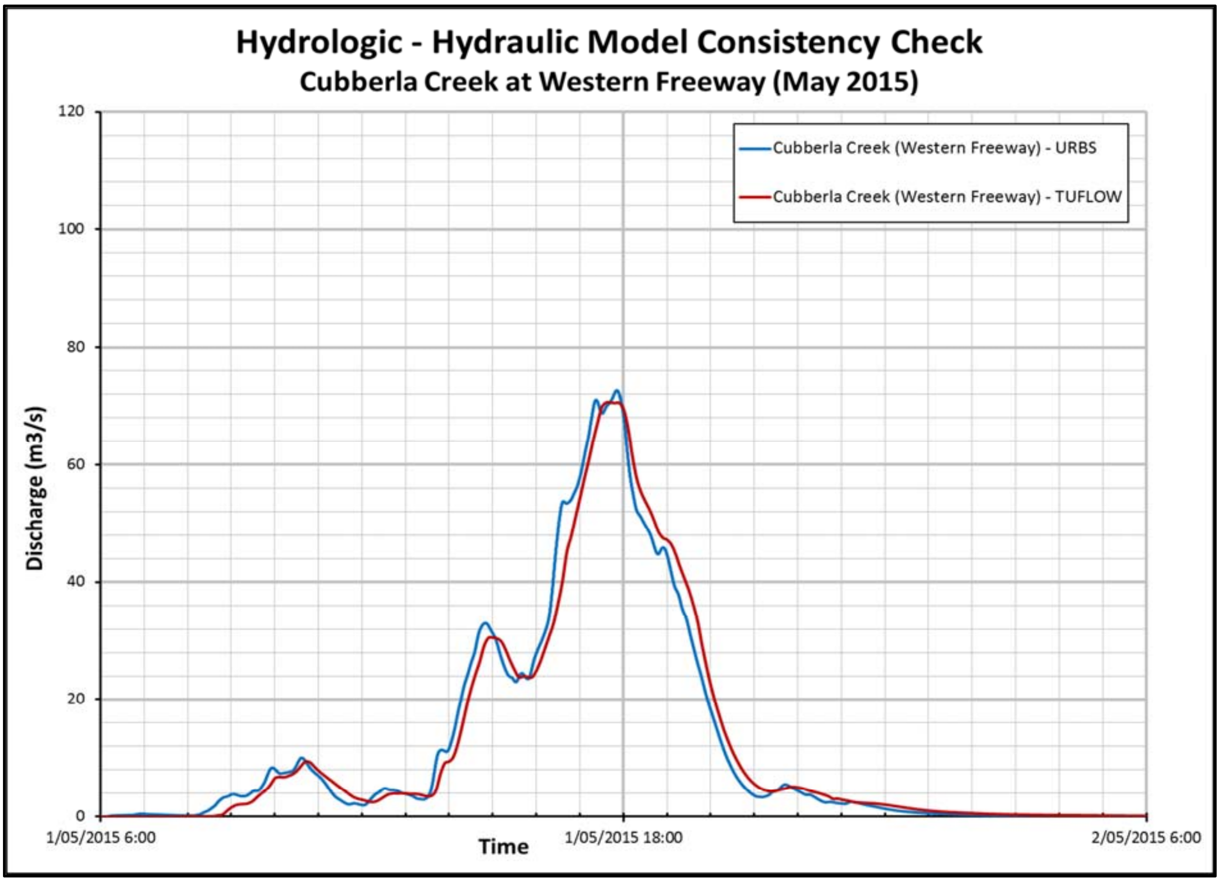


Figure 5.3: Cubberla Creek at Western Freeway (May 2015)

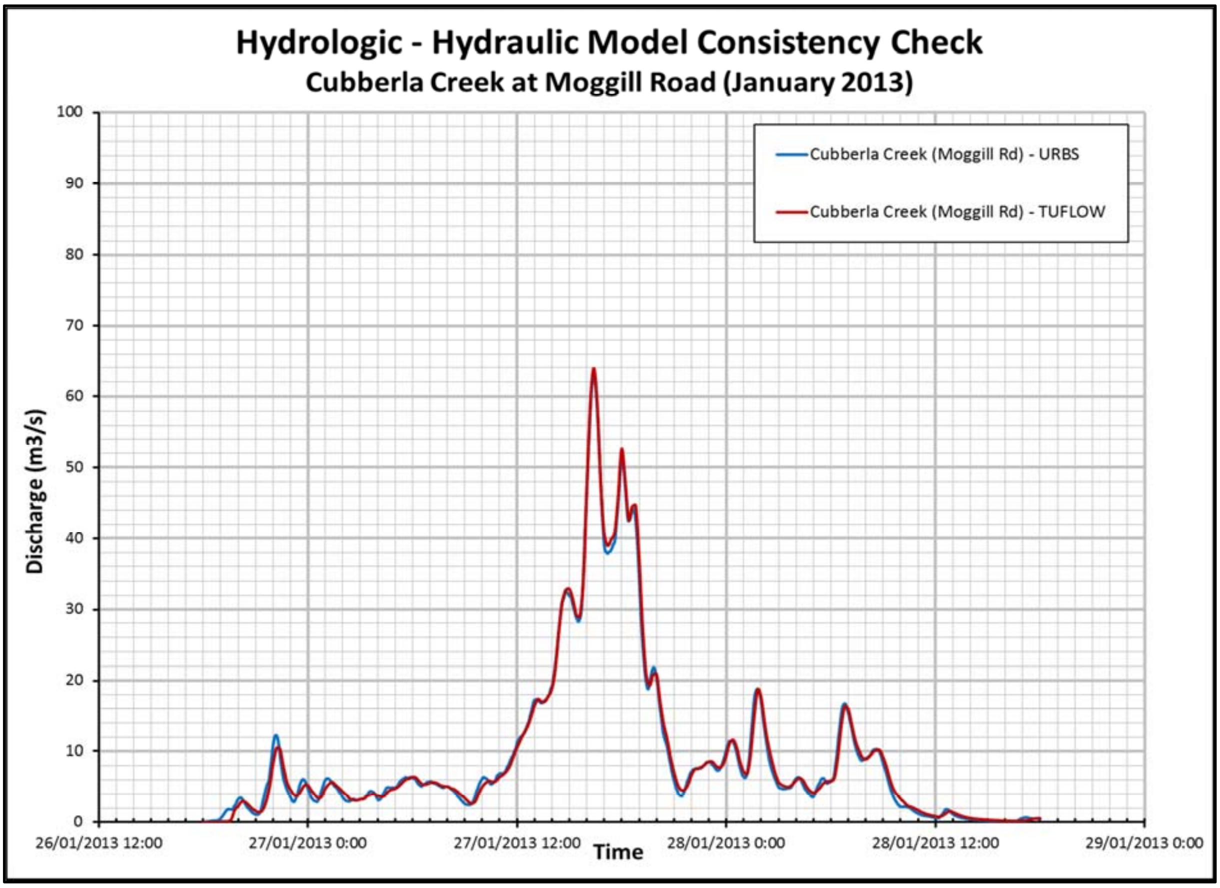


Figure 5.4: Cubberla Creek at Moggill Road (January 2013)

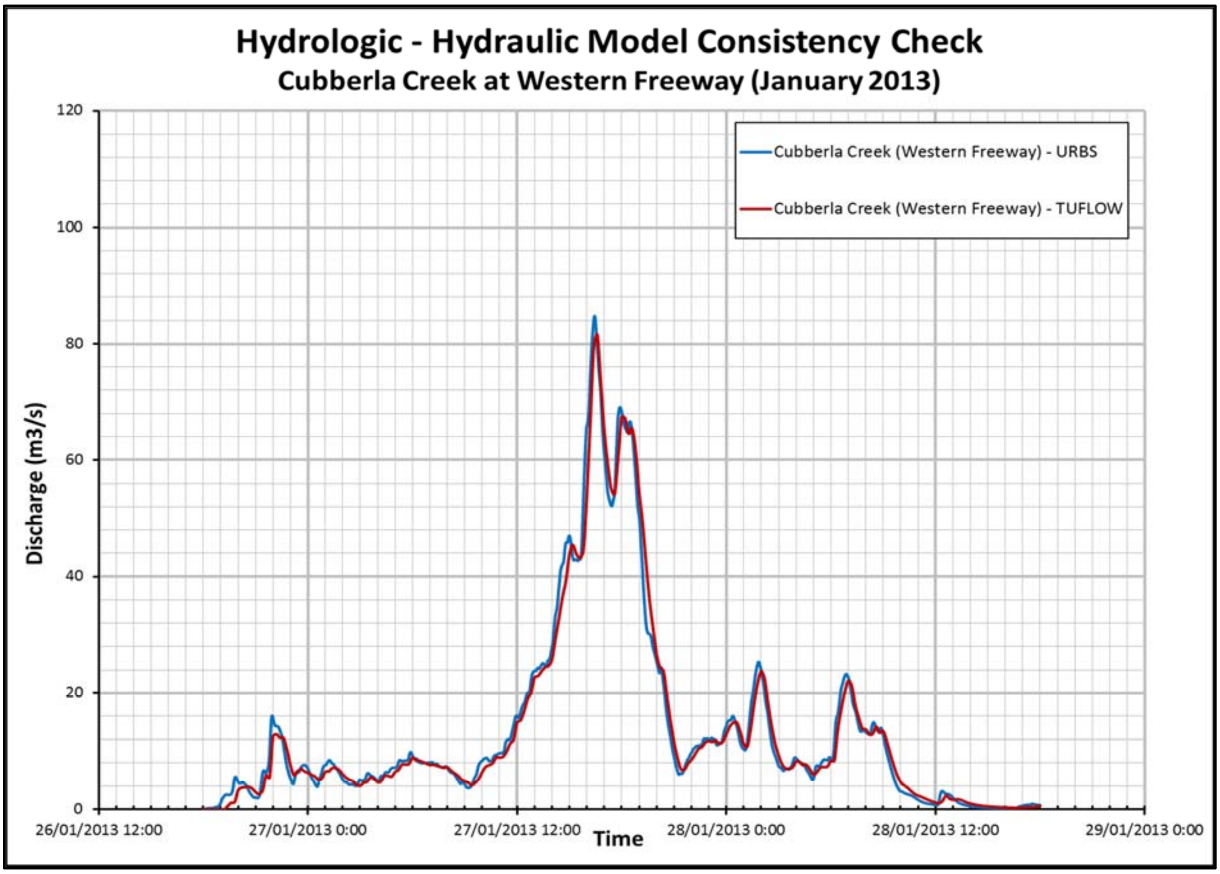


Figure 5.5: Cubberla Creek at Western Freeway (January 2013)

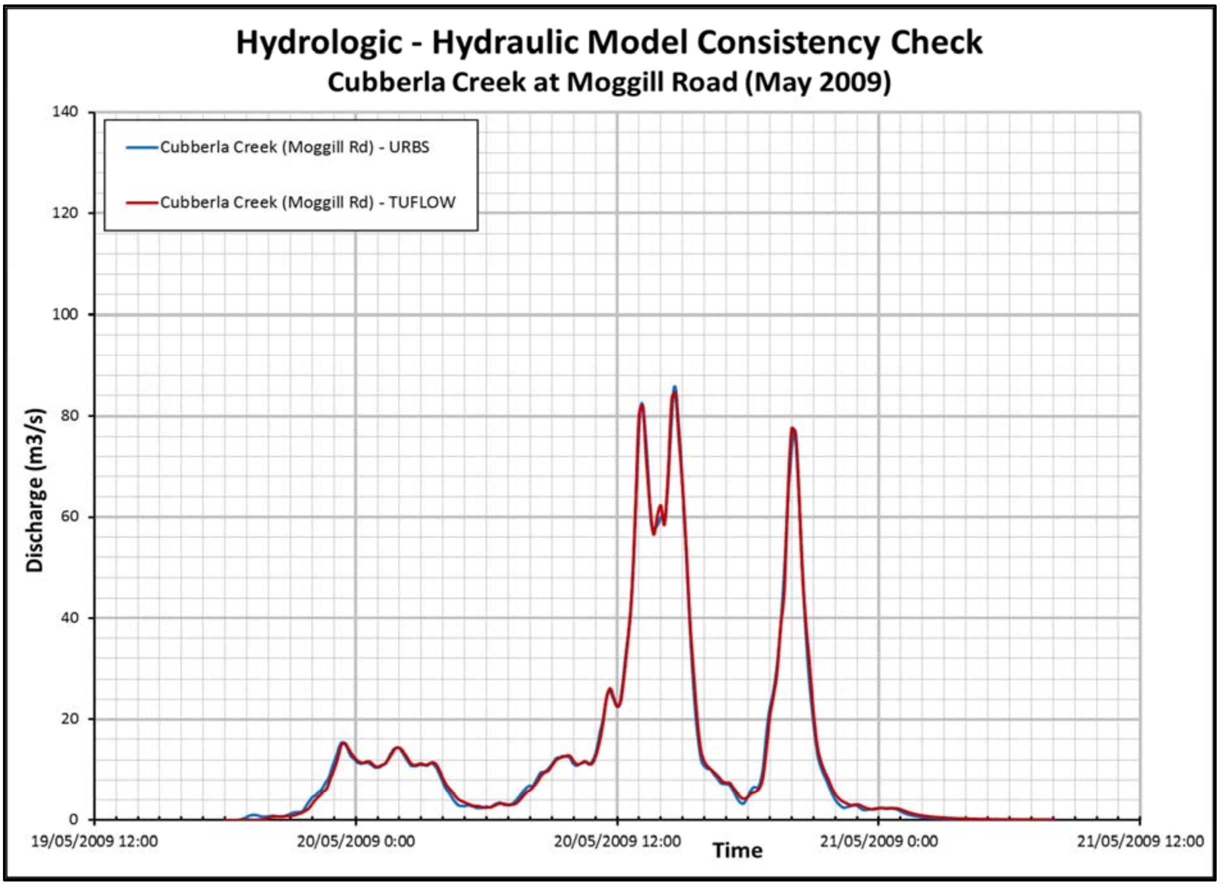


Figure 5.6: Cubberla Creek at Moggill Road (May 2009)

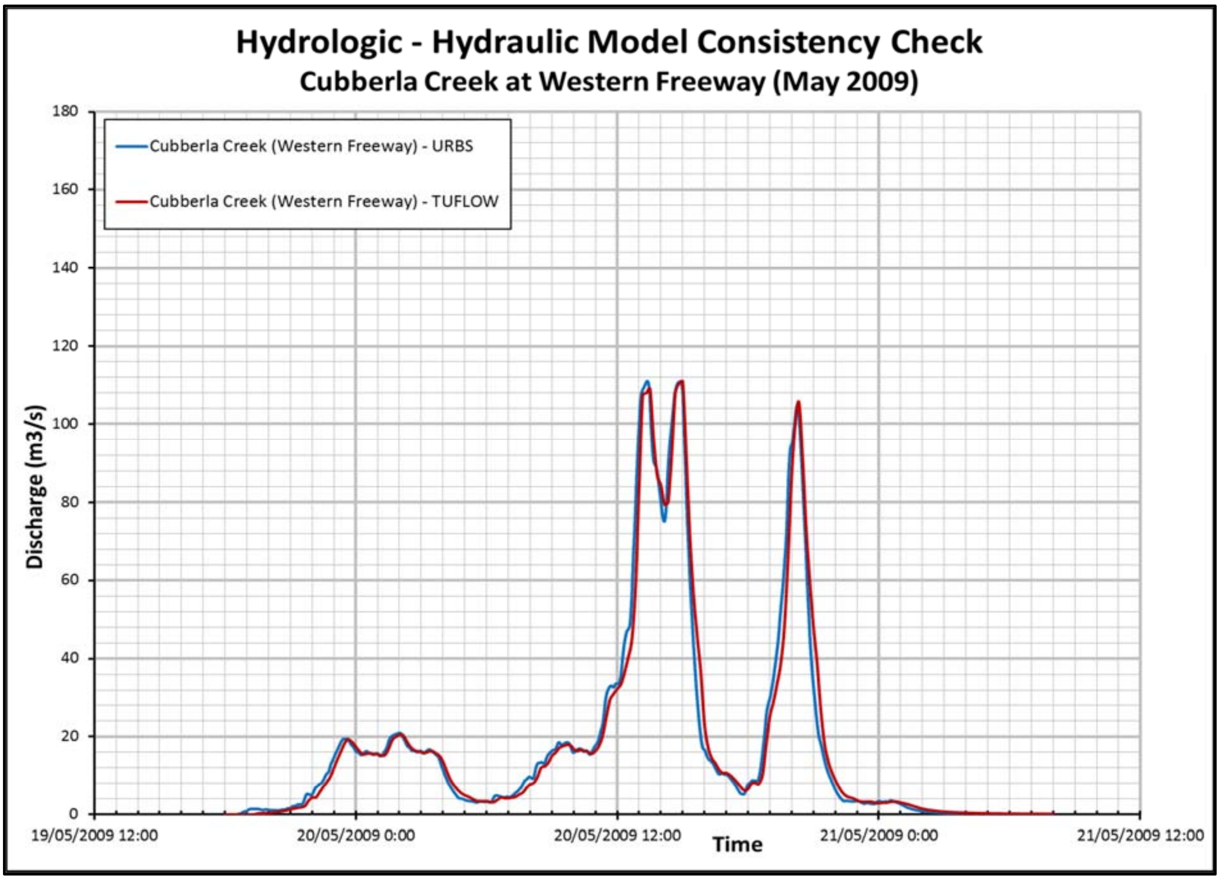


Figure 5.7: Cubberla Creek at Western Freeway (May 2009)

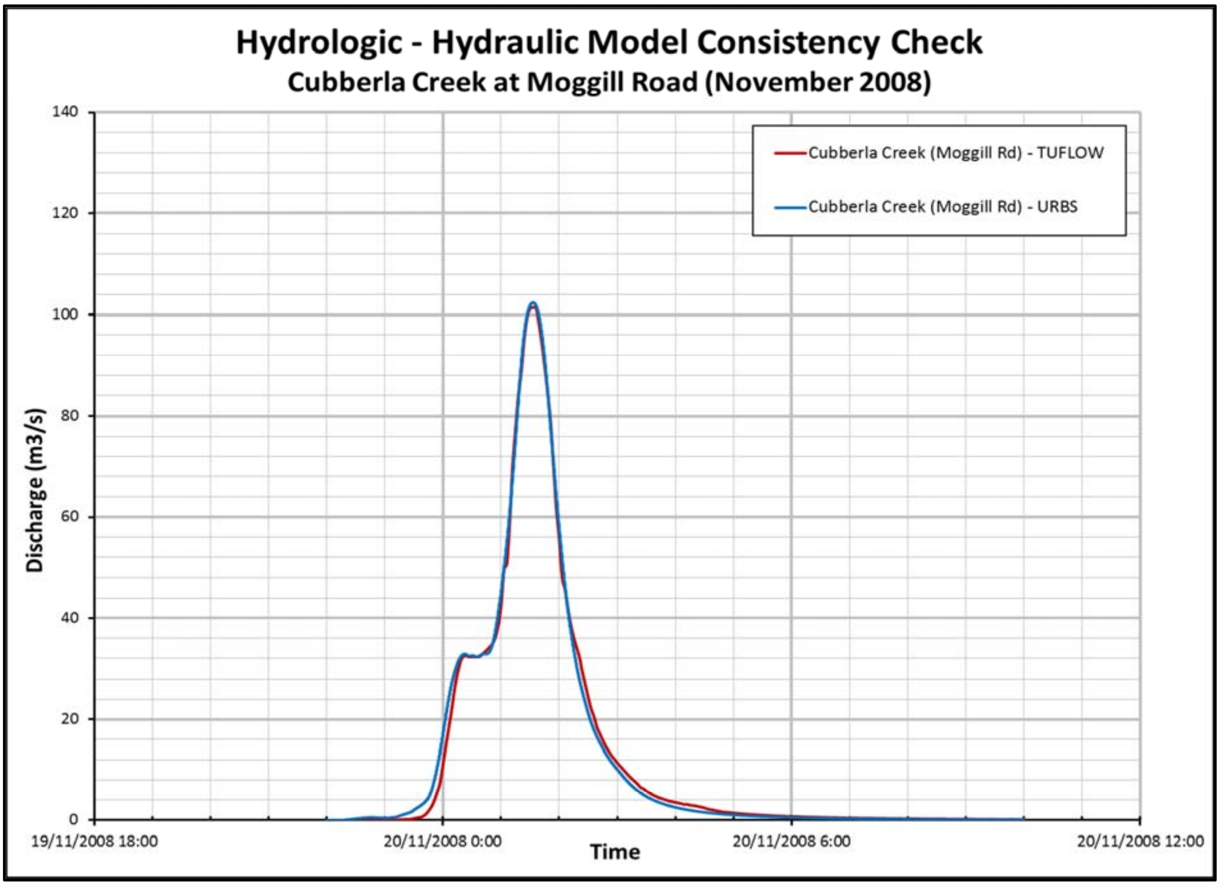


Figure 5.8: Cubberla Creek at Moggill Road (November 2008)

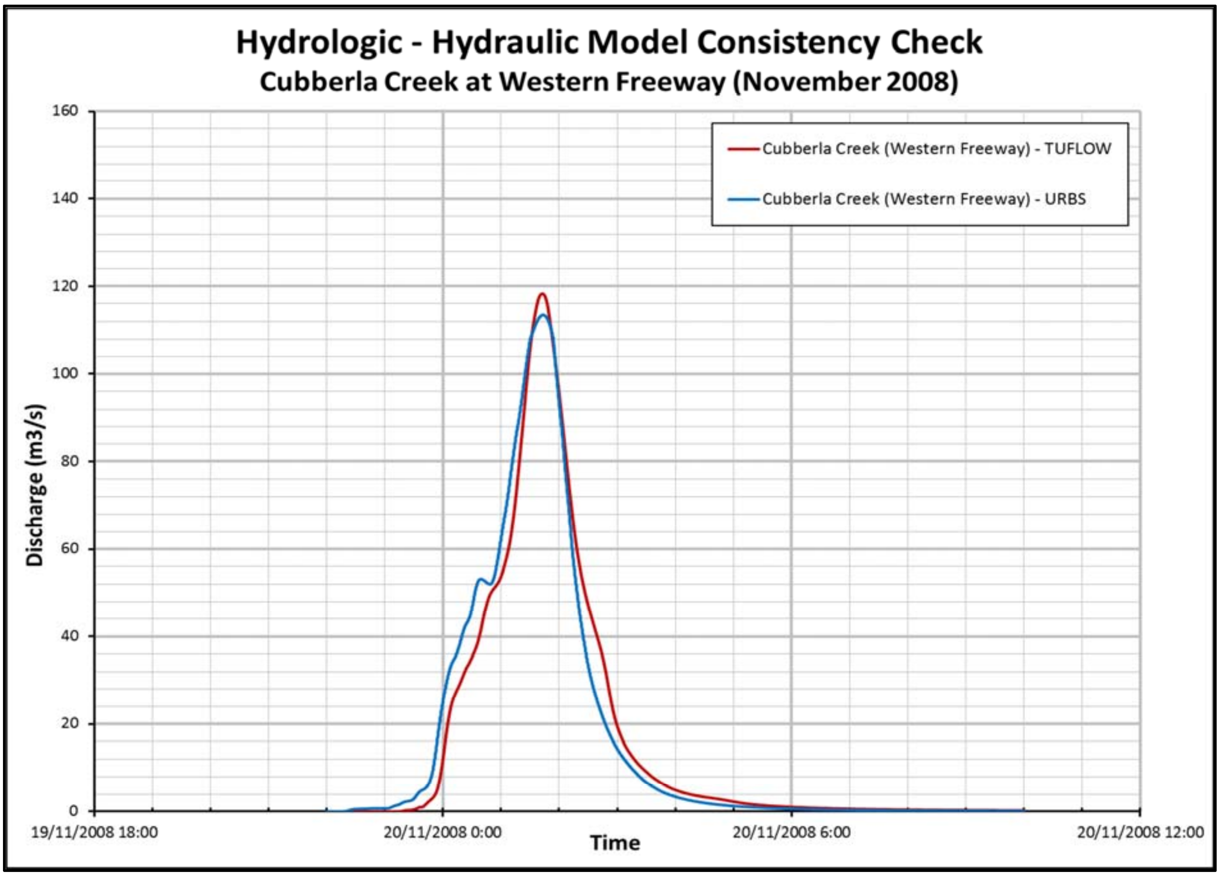


Figure 5.9: Cubberla Creek at Western Freeway (November 2008)

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. 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.

5.9 Discussion on Calibration and Verification

The calibration and verification of the Cubberla Creek hydrologic and hydraulic models has been based purely on the peak flood level comparison at the MHGs. The shape, timing and volume of the flood hydrograph have not been able to be verified against stream gauge records as there are no such gauges within the catchment. However, the calibration and verification of the Moggill Creek URBS model used the same historical events and was fortunate to have three stream gauges from which to better assess the shape and volume of the flood hydrograph. Where possible, the Cubberla Creek URBS model has adopted the same hydrologic parameters as the Moggill Creek URBS model.

The MHG coverage is quite extensive with gauges located in the upper, middle and lower sections of Cubberla Creek as well as on the Boblynne Street Branch and Gubberley Creek. There are no MHGs on the minor tributaries of the Akuna Street Branch and Tributary C.

The calibration and verification of Cubberla Creek and the Boblynne Street Branch was very good with the simulated peak flood levels for all four events being within the ideal tolerance of +/- 0.3 m.

The calibration and verification of Gubberley Creek also resulted in the simulated peak flood levels for all four events being within the ideal tolerance. It was established that flood levels in the Gubberley Creek Detention Basin are dependent on the degree of blockage of the grated low-flow piped outlet. For 3 out of 4 events, it was necessary to apply blockage to adequately simulate the peak flood level in the basin. This is considered conceivable as the grated inlet would appear to be at high risk from blockage by plant / leaf litter originating from within the basin.

The URBS model was able to accurately replicate the TUFLOW model at all locations within the catchment. As noted previously in Section 4.10, there were two areas for which level-pool routing was used in lieu of Muskingum channel routing to better represent the flood storage effects. These areas are as follows:

- Upstream of the Western Freeway incorporating the sporting fields on the left-hand side floodplain.
- Between the Western Freeway and the Brisbane River incorporating the wide expansive floodplain areas (AMTD 2100 to AMTD 0).

Given that the results of the calibration and verification are very good and that the events ranged from frequent (~2-yr to 5-yr ARI) to infrequent (~10-yr to 20-yr ARI), there is some confidence that the hydrologic and hydraulic models would be suitable for producing accurate flood levels for the full range of design event modelling.

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 6.3 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' roughness 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. up to 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.

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 Background

During the course of this flood study, AR&R 2016 was released which incorporated a full revision of the synthetic design storm methodology. As this flood study was nearing completion when AR&R 2016 was released, it was agreed to complete the study on the basis of the AR&R 1987 methodology.

6.2.2 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 common method used to estimate the design flood is the rainfall based synthetic design storm concept from AR&R 1987.

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

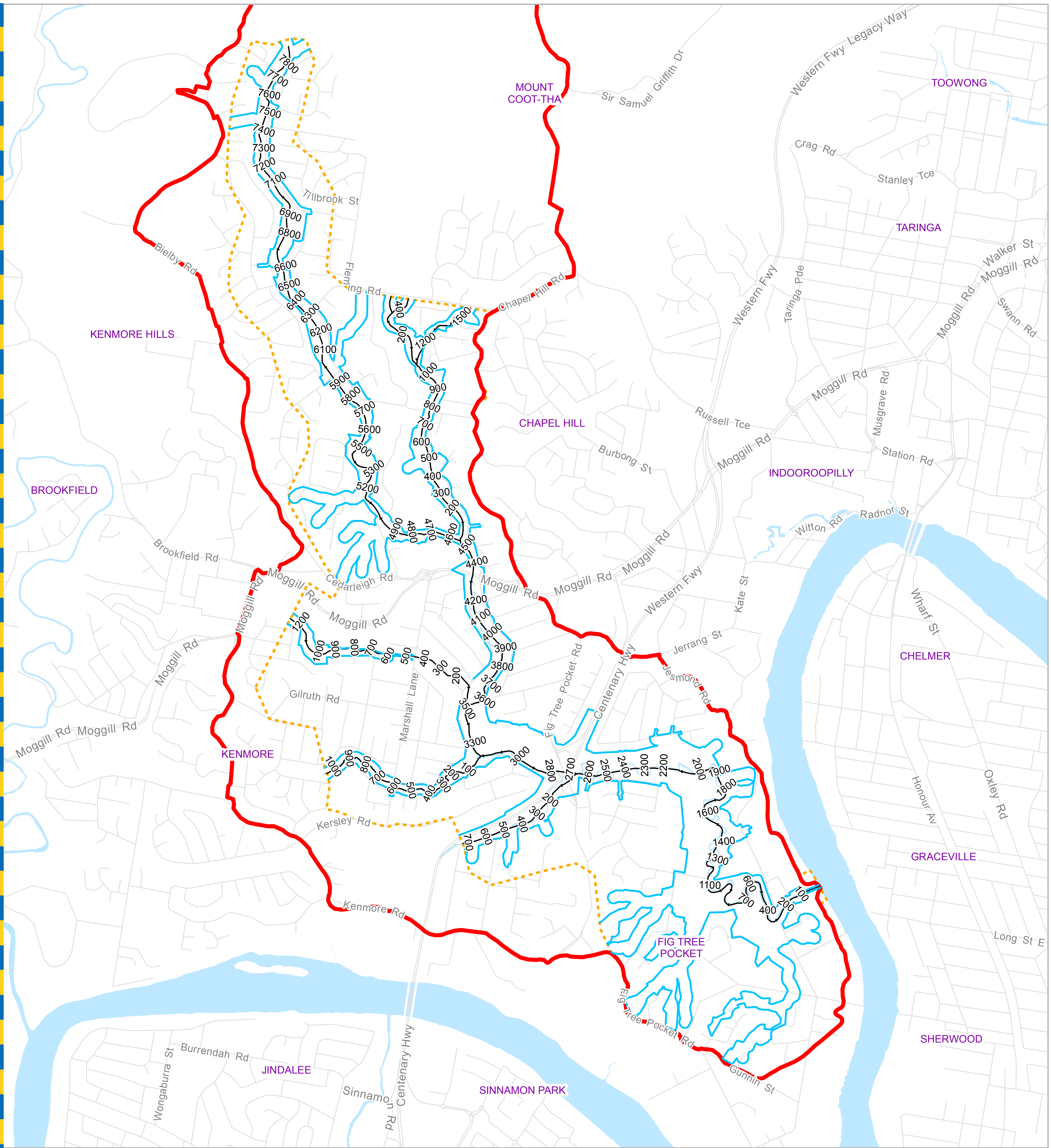
As there are no continuous recording stream gauges within the catchment it is not possible to undertake FFA on the basis of recorded floods within the catchment. The MHG records are not suitable for statistical analysis due to the random nature of the sampling interval, which could range from numerous times a year during a wet year to many years apart during times of drought. Manual reading at each MHG is also discretionary and not dependent on for example exceeding a nominated flood level.

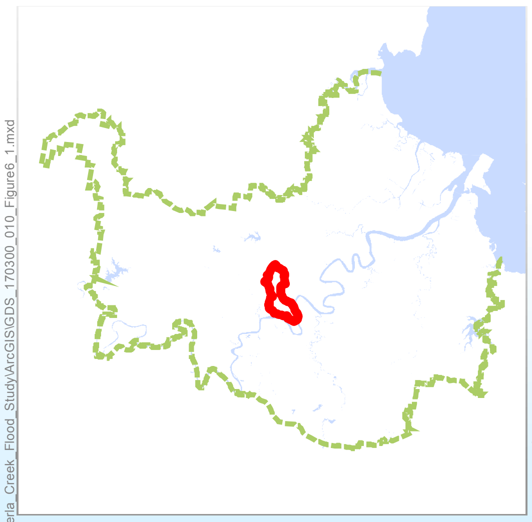
<u>Adopted Methodology for Design Flood Estimation</u>

Based on the review of the suitability of FFA, it was decided that the most appropriate methodology was to utilise the synthetic design storm concept from AR&R 1987.

The methodology is as follows:

- Design Intensity Frequency Duration (IFD) estimates are determined from AR&R 1987 for the full range of storm ARIs (2-yr to 100-yr) and durations (30 minute to 6 hour).
- 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 model 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.





0 125 250 375 500
Metres

N E S W

Legend

- Flood Corridor
- AMTD Line
- Catchment Area
- Model Boundary
- Streets

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DATA INFORMATION

The flood maps must be read in conjunction with the flood study report and interpretedby a qualified professional engineer. The flood maps are based on the best data available to Brisbane City Council ("Council") at the time the maps were developed. Council, and the copyright owners listed below, give no warranty in relation to the data (including accuracy, reliability, completeness, currency or suitability) presented in these maps and the user uses and relies upon the data in the maps at its own sole risk and liability. Council is not liable for errors or omissions in the flood maps. To the full extent that it is able to do so in law, the Council disclaims all liability (including without limitation, liability in negligence) for any loss, damage or costs (including indirect and consequential loss and damage), caused by or arising from anyone using or relying on the data contained in the flood maps for any purpose whatsoever.

Prepared : 081335
Checked : JS
Revision : 1
Publication Date : 15 May 2017
Project Number : 170300

Cubberla Creek Flood Study

Figure 6.1: Modelled Flood Corridor

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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 calibration 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 generally resulted in minor increases in impervious area for the majority of sub-catchments. However, for a number of sub-catchments the increase in impervious area was quite substantial. These included (in order from the highest): Sub-catchments 18, 15, 36, 22 and 13 where the impervious area increased by more than 20 % of the total sub-catchment area.

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 6.2 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 6.2 – 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	68.9	90.6	104	122	146	165
1	46.1	61.2	70.6	82.9	99.8	113
1.5	36.4	48.7	55.7	64.5	79.4	90.1
2	29	38.5	44.5	52.3	63	71.4
3	21.6	28.7	33.1	38.9	46.9	53.2
6	13	17.2	19.8	23.3	28	31.7

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 minute, 1 hour, 1.5 hours, 2 hours, 3 hours and 6 hours.

Gubberley Creek Detention Basin

As there is no permanent water in the Gubberley Detention Basin, the initial storage was assumed to be empty for the purposes of modelling the design events.

As the likelihood of blockage of the low-flow pipe grated inlet is considered high, the low-flow outlet was modelled as fully blocked. This is consistent with the findings from the calibration / verification where it was necessary to apply blockage to better match the MHG level in three out of the four historical events. This approach will typically result in slightly more conservative flood levels downstream of the detention basin.

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 minute to 6 hour.

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 Western Freeway Barrier Blockage

For the purposes of design event modelling, all the barriers as discussed previously in Section 5.3.4 were assumed to be blocked. This included assuming the large noise barriers to be fully blocked and impervious to flow. A comparison of the difference in the 100-yr ARI (1 % AEP) flood level between fully blocked and un-blocked noise barriers is presented in Section 6.4.3.

6.3.5 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 1.21 mAHD.

6.4 Results and Mapping

6.4.1 Critical Durations

A full range of durations (30 minute to 6 hour) were simulated for the 2-yr ARI (50 % AEP) to 100-yr ARI (1 % AEP) events. From the results, the critical durations at key locations within the catchment were extracted and are provided in Table 6.3. For this purpose, the critical duration is the storm duration which produces the peak flood level.

The results indicate that along Cubberla Creek the 60-minute to 120-minute durations produce the peak flood levels. Within the Boblynne Street Branch, the 60-minute duration is critical for the entire modelled length. The 30-minute duration is critical along the modelled length of both the Akuna Street Branch and Tributary C. Within Gubberley Creek, the 30-minute to 90-minute duration produces the peak flood levels.

Table 6.3 – 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)
Cubberla Creek						
Greenford Street (S19)	90	60	60	60	60	60
Goolman Street (S14)	60	60	60	60	60	60
Moggill Road (S7)	60	60	60	60	60	60
Western Freeway (S3)	90	90	90	90	90	90
Confluence with Brisbane River	90	90	90	120	90	90
Boblynne Street Branch						
U/S Model Extent	60	60	60	60	60	60
Confluence with Cubberla Creek	60	60	60	60	60	60
Gubberley Creek						
D/S Detention Basin	90	90	90	60	60	60
Marshall Lane	90	90	90	60	60	60
Akuna Street Branch						
U/S Model Extent	30	30	30	30	30	30
Marshall Lane (S26)	30	30	30	30	30	30
Tributary C						
U/S Model Extent	30	30	30	30	30	30
Western Freeway On Ramp (S29)	60	60	60	60	60	60

6.4.2 Peak Discharge Results

Table 6.4 provides peak flow results at selected major roads for the Scenario 1 conditions. This information is from the URBS hydrologic model.

Table 6.4 – Design Event Peak Discharge at Selected Major Roads (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)
Cubberla Creek						
Greenford Street (S19)	4.4	6.5	7.8	9.4	11.6	13.6
Dillingen Street (S18)	34.6	48.8	57.8	70.0	83.4	96.5
Goolman Street (S14)	43.7	60.8	71.8	86.3	102.7	118.2
Moggill Road (S7)	63.7	87.2	102.0	121.6	145.7	166.7
Western Freeway (S3)	79.4	108.6	120.1	132.7	153.8	172.8
Jesmond Road (S1)	67.5	86.2	97.4	110.3	129.5	148.5
Gubberley Creek						
Marshall Lane	6.7	12.3	15.6	19.5	24.7	30.2
Akuna Street Branch						
Marshall Lane (S26)	14.7	20.3	23.9	28.7	32.5	37.3
Tributary C						
Western Freeway On Ramp (S29)	21.1	29.4	34.6	41.7	47.4	54.5

The results indicate that there is significant flow attenuation from upstream of the Western Freeway to Jesmond Road, which becomes more noticeable as the size of the event increases. The attenuation is primarily due to the wide expansive floodplains and also the confined channel in the vicinity of Jesmond Road / Brisbane River confluence. However, the Western Freeway is also a contributing factor due to the blockage and storage behind the many barriers within the road corridor.

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 E
- Scenario 3: 2-yr ARI (50 % AEP) to 100-yr ARI (1 % AEP) events – Appendix F

The peak flood levels are the maximum flood level when considering the full range of durations from 30-minute to 6 hour. The peak flood levels are extracted along the current AMTD line for all creeks. Where there was no AMTD line, an assumed line was drawn to enable flood levels to be extracted. The lower section of the catchment is dominated by flooding originating from the Brisbane River; as such the reported peak flood levels in this area will be lower than the Brisbane River peak flood levels for each respective ARI (AEP).

Table 6.5 indicates a comparison of the difference in the Scenario 1 100-yr ARI (1 % AEP) flood level between fully blocked and un-blocked noise barriers

Table 6.5 – Comparison of Noise Barrier Impacts 100-yr ARI (1 % AEP)

Creek	Location	Scenario 1 100-yr ARI (1 % AEP) Flood Level (m AHD)		
		Noise Barrier Fully Blocked	Noise Barrier Excluded	Difference (m)
Cubberla	Gubberley Creek Junction	13.94	13.94	0.00
Cubberla	Akuna St. Branch Junction	12.94	12.87	0.07
Cubberla	U/S Western Freeway	12.72	12.50	0.22
Cubberla	D/S Western Freeway	7.50	7.57	-0.07
Cubberla	500m d/s of Western Freeway	10.61	10.89	-0.28
Tributary C	U/S Freeway On Ramp	13.03	13.03	0.00

The results indicate that flood levels upstream of the Western Freeway are up to 0.22 m higher with the noise barrier blockage included. At the Akuna Street Branch Junction, the flood level is 0.07 m higher with the noise barrier blockage included. Downstream of the freeway, the inclusion of the noise barrier results in flood level reductions of up to 0.28 m.

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 6.2 and Figure 6.3.

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

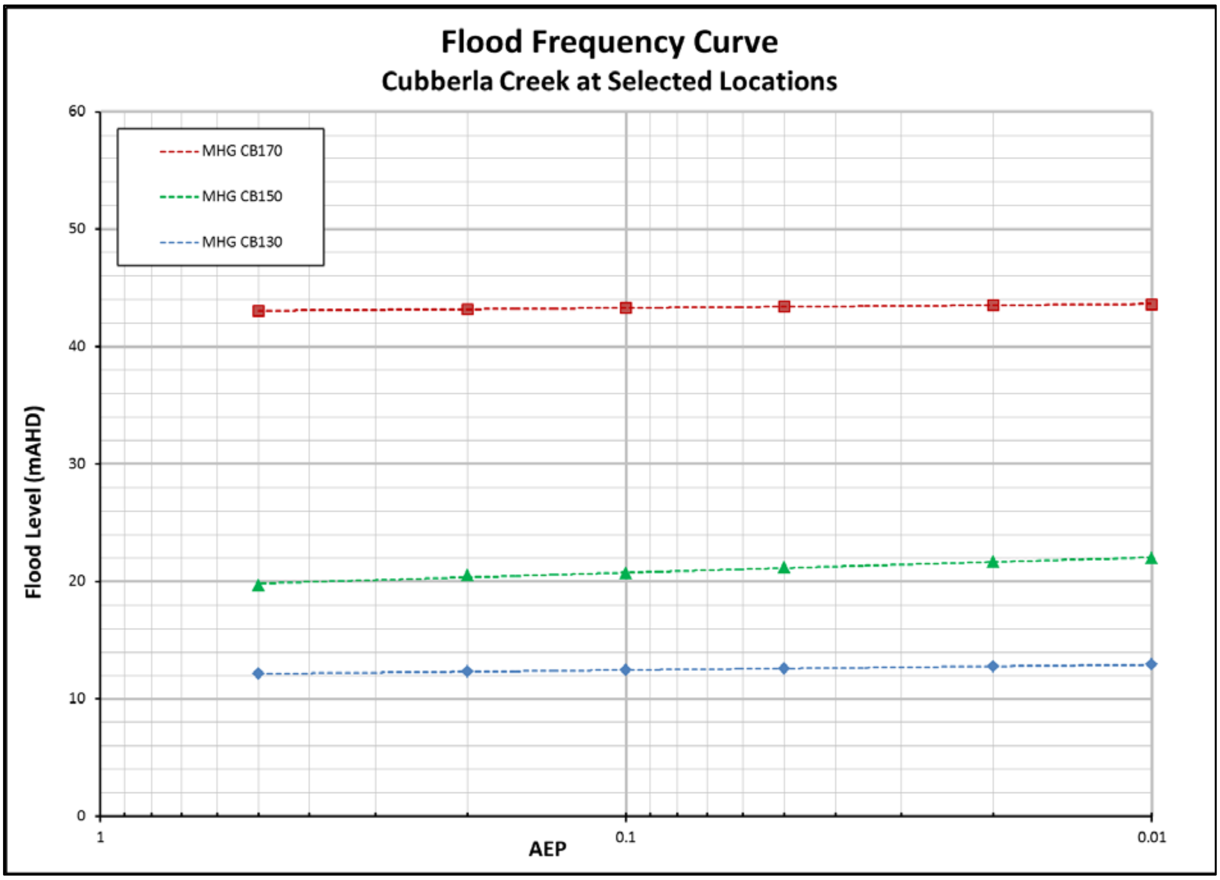

Figure 6.2: Flood Frequency Curve – Cubberla Creek at Selected Locations

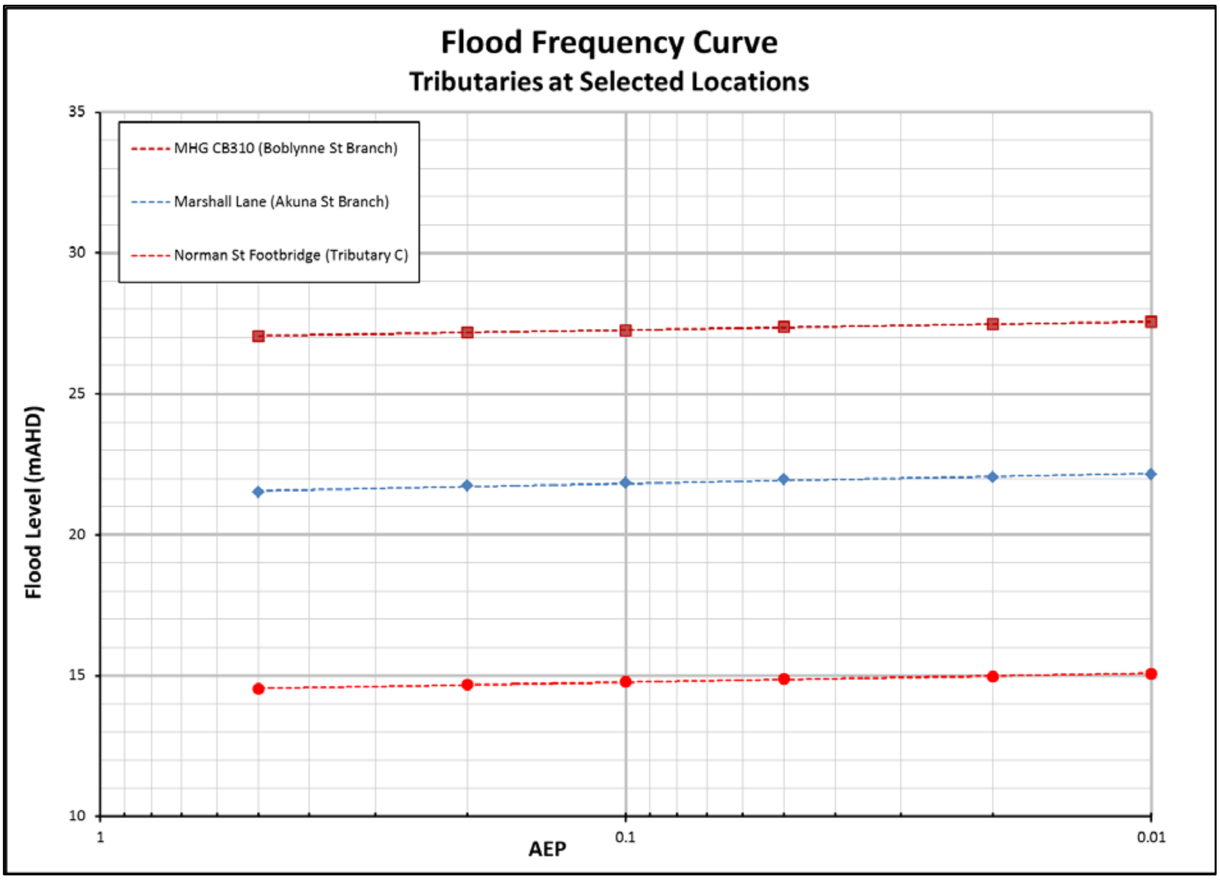

Figure 6.3: Flood Frequency Curve – Tributaries at Selected Locations

Table 6.6 – Estimated Magnitude of Historical Events

Location	Event Magnitude			
	May 2015	Jan 2013	May 2009	Nov 2008
Cubberla Creek				
MHG CB170	5-yr ARI (20 % AEP)	5-yr to 10-yr ARI (20 % to 10 % AEP)	5-yr to 10-yr ARI (20 % to 10 % AEP)	20-yr ARI (5 % AEP)
MHG CB150	< 2-yr ARI (50 % AEP)	2-yr to 5-yr ARI (50 % to 20 % AEP)	2-yr to 5-yr ARI (50 % to 20 % AEP)	10-yr to 20-yr ARI (10 % to 5 % AEP)
MHG CB130	2-yr to 5-yr ARI (50 % to 20 % AEP)	5-yr ARI (20 % AEP)	5-yr to 10-yr ARI (20 % to 10 % AEP)	10-yr to 20-yr ARI (10 % to 5 % AEP)
Boblynne Street Branch				
MHG CB310	< 2-yr ARI (50 % AEP)	< 2-yr ARI (50 % AEP)	2-yr ARI (50 % AEP)	5-yr ARI (20 % AEP)
Akuna Street Branch				
Marshall Lane	< 2-yr ARI (50 % AEP)	< 2-yr ARI (50 % AEP)	2-yr ARI (50 % AEP)	< 2-yr ARI (50 % AEP)
Tributary C				
Norman Street Bridge	< 2-yr ARI (50 % AEP)	< 2-yr ARI (50 % AEP)	2-yr ARI (50 % AEP)	< 2-yr ARI (50 % 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 I. These locations are generally in the vicinity of hydraulic structures and include:

- Greenford Street (S19) – Cubberla Creek
- Dillingen Street (S18) – Cubberla Creek
- Goolman Street (S14) – Cubberla Creek
- Moggill Road (S7) – Cubberla Creek
- Western Freeway (S3) – Cubberla Creek
- Jesmond Road (S1) – Cubberla Creek
- 2 x Bulk Water Mains (S21) – Boblynne Street Branch
- Marshall Lane – Gubberley Creek
- Marshall Lane (S26) – Akuna Street Branch

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 MHWS (1.21 mAHD). In the lower reach of Cubberla Creek, 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 6.7. The flood immunity indicated does not consider flooding originating from the Brisbane River. As a result the waterway crossings located downstream of the Western Freeway (i.e. Jesmond Road) are likely to have a lower flood immunity.

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 6.7 – Flood Immunity at Major Structures

Location	Flood Immunity (ARI)
Cubberla Creek	
Greenford Street (S19)	50-yr ARI (2 % AEP)
Dillingen Street (S18)	20-yr ARI (5 % AEP)
Goolman Street (S14)	2-yr ARI (50 % AEP)
Tristania Road (S10)	< 2-yr ARI (50 % AEP)
Moggill Road (S7)	10-yr ARI (10 % AEP)
Western Freeway (S3)	5-yr ARI (20 % AEP)
Jesmond Road (S1)	20-yr ARI (5 % AEP)
Boblynne Street Branch	
St. James Estate Access (S20)	20-yr ARI (5 % AEP)
Gubberley Creek	
Marshall Lane	2-yr ARI (50 % AEP)
Akuna Street Branch	
Marshall Lane (S26)	< 2-yr ARI (50 % AEP)
Tributary C	
Western Freeway On Ramp (S29)	< 2-yr ARI (50 % AEP)
Western Freeway Off Ramp (S28)	2-yr ARI (50 % AEP)
Fig Tree Pocket Road (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 60-minute duration storm event.

The locations where comparative plots were undertaken are as follows:

(i) Cubberla Creek – Goolman Street
(ii) Cubberla Creek – Moggill Road
(iii) Cubberla Creek – Western Freeway
(iv) Cubberla Creek – Outlet at Brisbane River
(v) Boblynne Street Branch - Confluence with Cubberla Creek
(vi) Gubberley Creek – Detention Basin Outlet

Figure 6.4 to Figure 6.9 provide comparative plots at each of the six locations. Table 6.8 provides a comparison of the peak flows at these six locations plus some additional locations.

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 Cubberla Creek, there is a very good comparison between the URBS and TUFLOW hydrographs for all three events. However, in the lower section of Cubberla Creek (downstream of the Western Freeway) 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 due to the considerable storage effects in this area.

There is a very good comparison between the URBS and TUFLOW hydrographs for all three events at the outlet of the Boblynne Street Branch.

At the downstream extent of the Gubberley Creek Detention Basin, there is a very good comparison between the URBS and TUFLOW hydrographs for all three events. Further downstream at Marshall Lane, the URBS and TUFLOW peak flows are very similar in magnitude.

At Marshall Lane on the Akuna Street Branch, the URBS and TUFLOW peak flows are very similar in magnitude.

Table 6.8 – Peak Flow Comparison (60-minute duration), URBS and TUFLOW

Location	Model	60-minute Duration Peak Flow (m^3/s)		
		5-yr ARI (20 % AEP)	20-yr ARI (5 % AEP)	100-yr ARI (1 % AEP)
Cubberla Creek at Dillingen Street	URBS	48.8	70.0	96.5
	TUFLOW	46.2	65.1	88.7
Cubberla Creek at Goolman Street	URBS	60.8	86.3	118.2
	TUFLOW	57.6	81.8	110.2
Cubberla Creek at Moggill Road	URBS	87.2	121.6	166.7
	TUFLOW	84.5	119.0	162.6
Cubberla Creek at the confluence with Gubberley Creek	URBS	97.0	134.6	184.5
	TUFLOW	98.1	136.4	183.7

Location	Model	60-minute Duration Peak Flow (m^3/s)		
		5-yr ARI (20 % AEP)	20-yr ARI (5 % AEP)	100-yr ARI (1 % AEP)
Cubberla Creek at the confluence with the Akuna Street Branch	URBS	104.9	145.1	199.0
	TUFLOW	109.3	151.6	198.7
Cubberla Creek at Western Freeway	URBS	106.7	131.5	168.8
	TUFLOW	108.4	128.9	162.1
Cubberla Creek at the confluence with the Brisbane River	URBS	80.3	102.8	130.7
	TUFLOW	72.2	98.3	128.0
Boblynne Street Branch at the confluence with Cubberla Creek	URBS	33.4	47.1	63.8
	TUFLOW	31.9	44.9	60.2
Gubberley Creek Detention Basin Outflow	URBS	7.7	13.8	22.4
	TUFLOW	7.9	13.6	21.0
Gubberley Creek at Marshall Lane	URBS	11.0	19.1	30.2
	TUFLOW	10.2	17.8	27.2
Akuna Street Branch at Marshall Lane	URBS	19.3	27.0	35.8
	TUFLOW	18.6	26.3	34.7

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 J.

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)

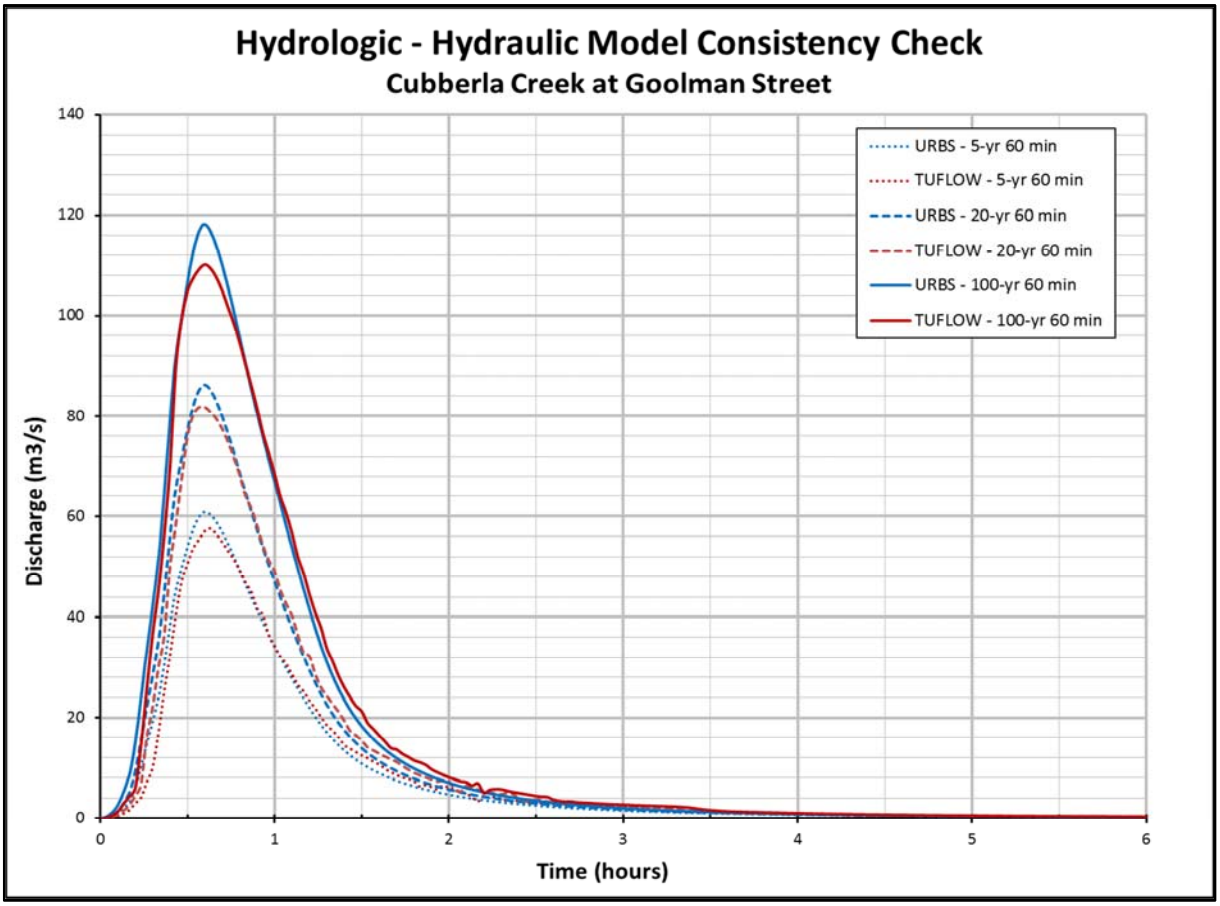

Figure 6.4: Cubberla Creek at Goolman Street

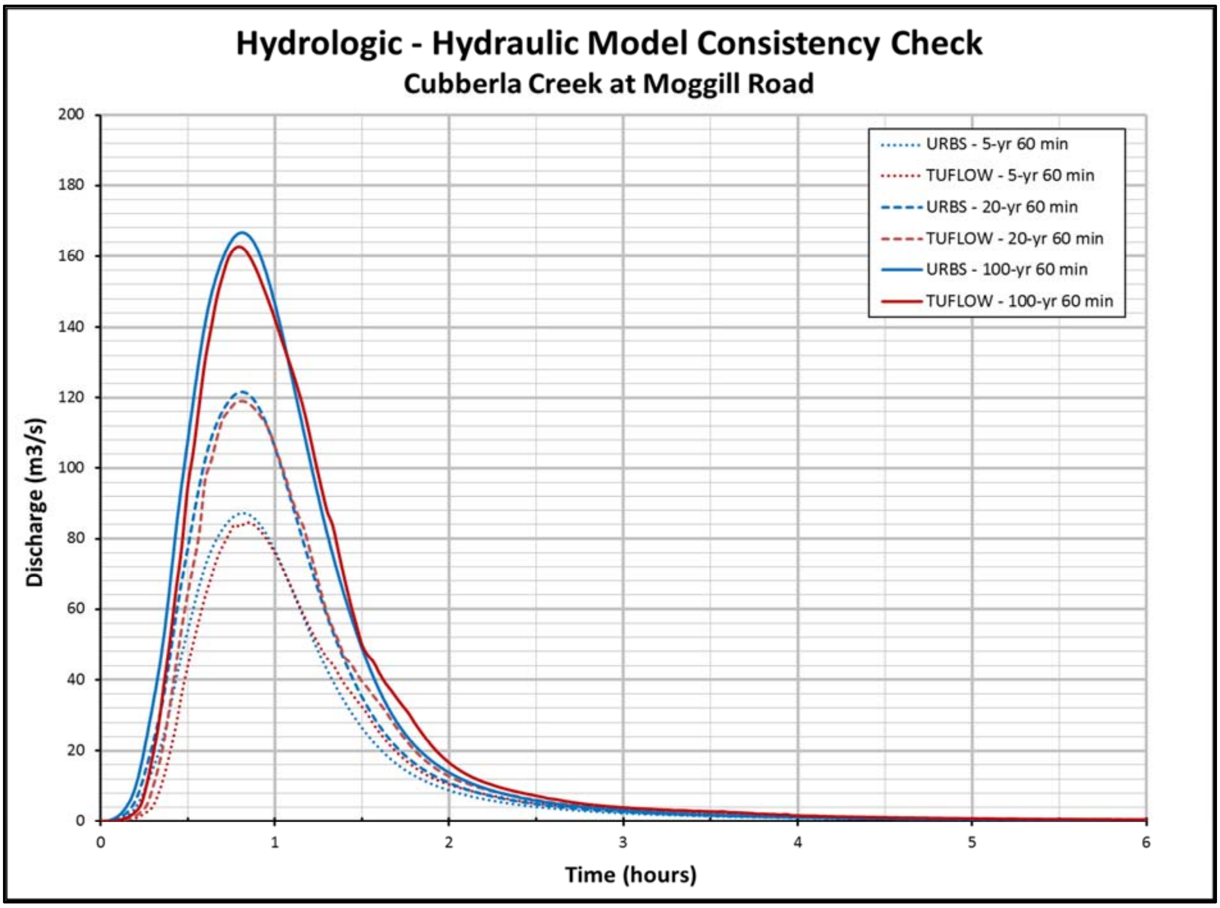

Figure 6.5: Cubberla Creek at Moggill Road

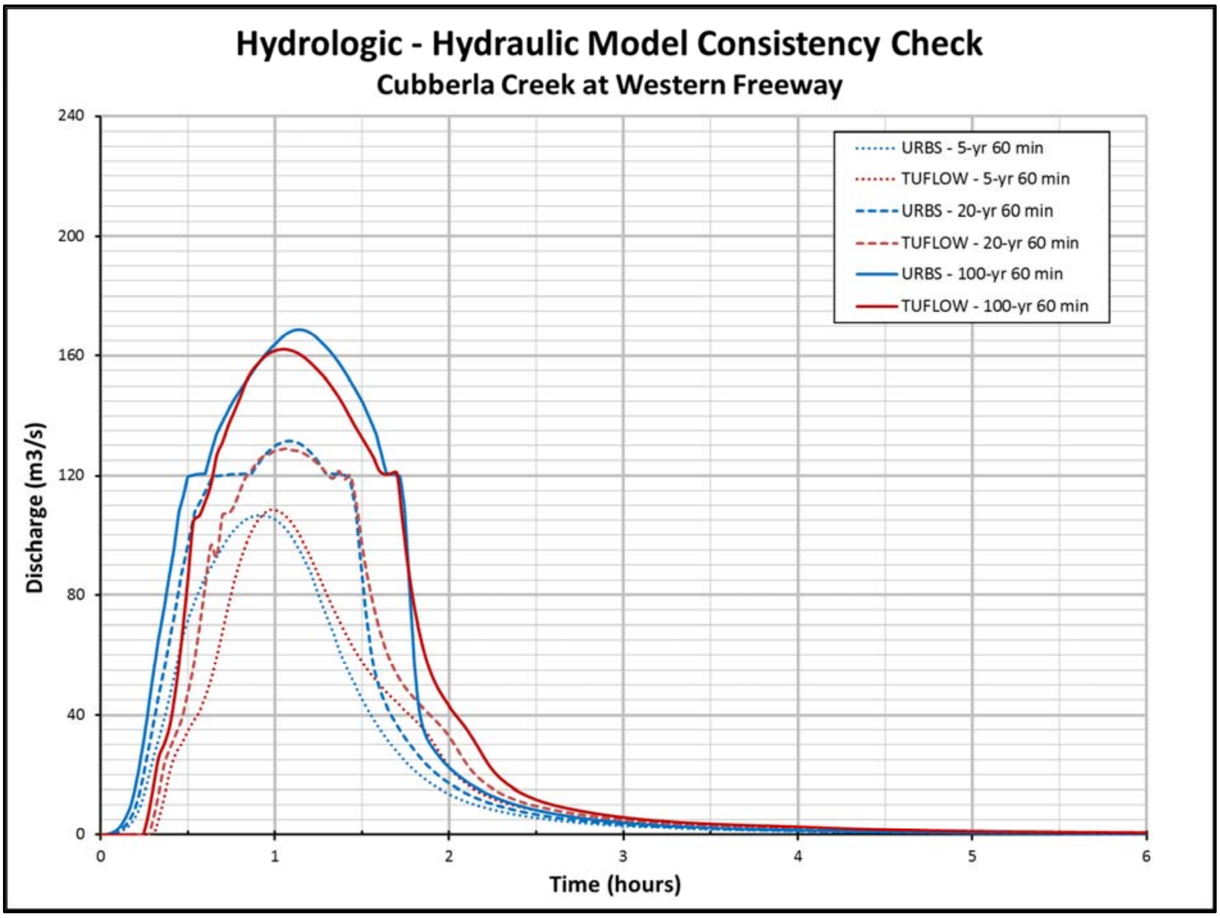

Figure 6.6: Cubberla Creek at Western Freeway

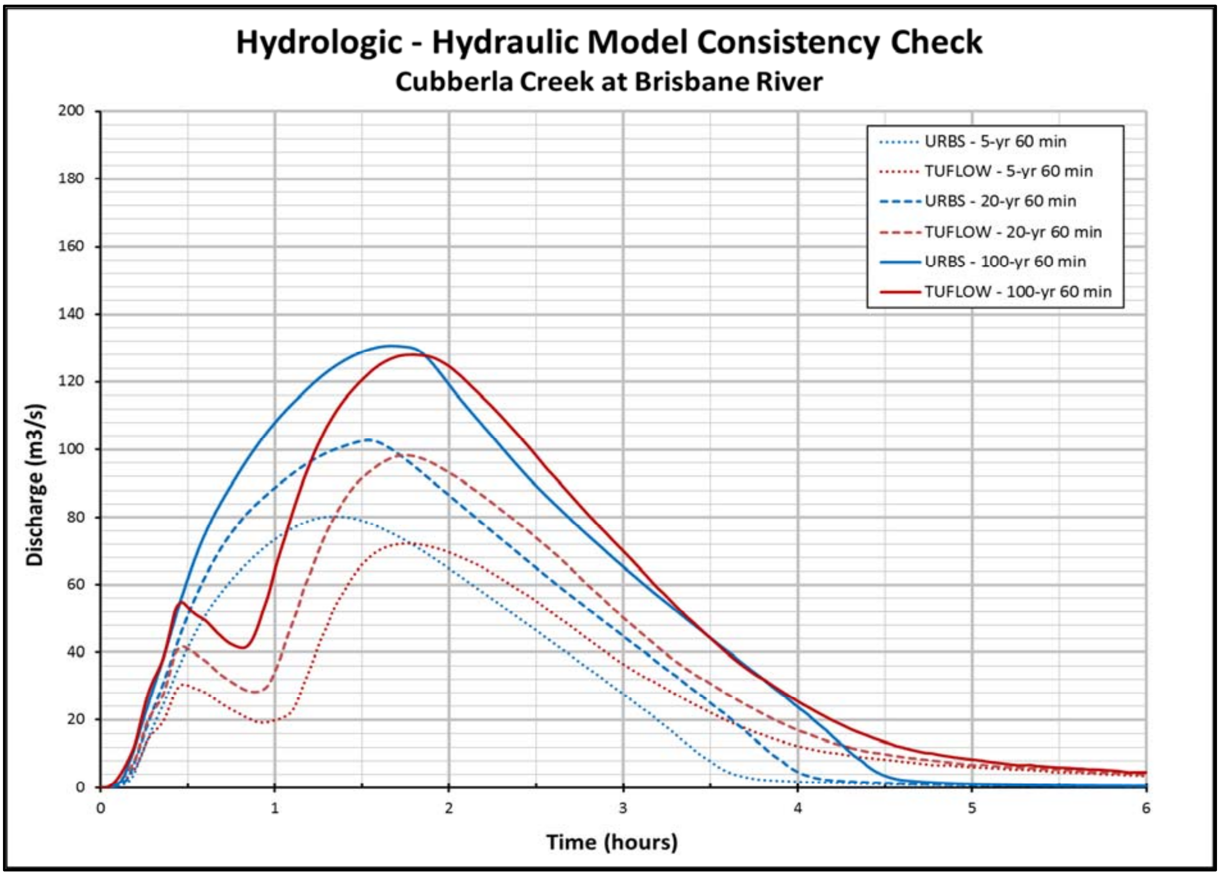

Figure 6.7: Cubberla Creek at Brisbane River

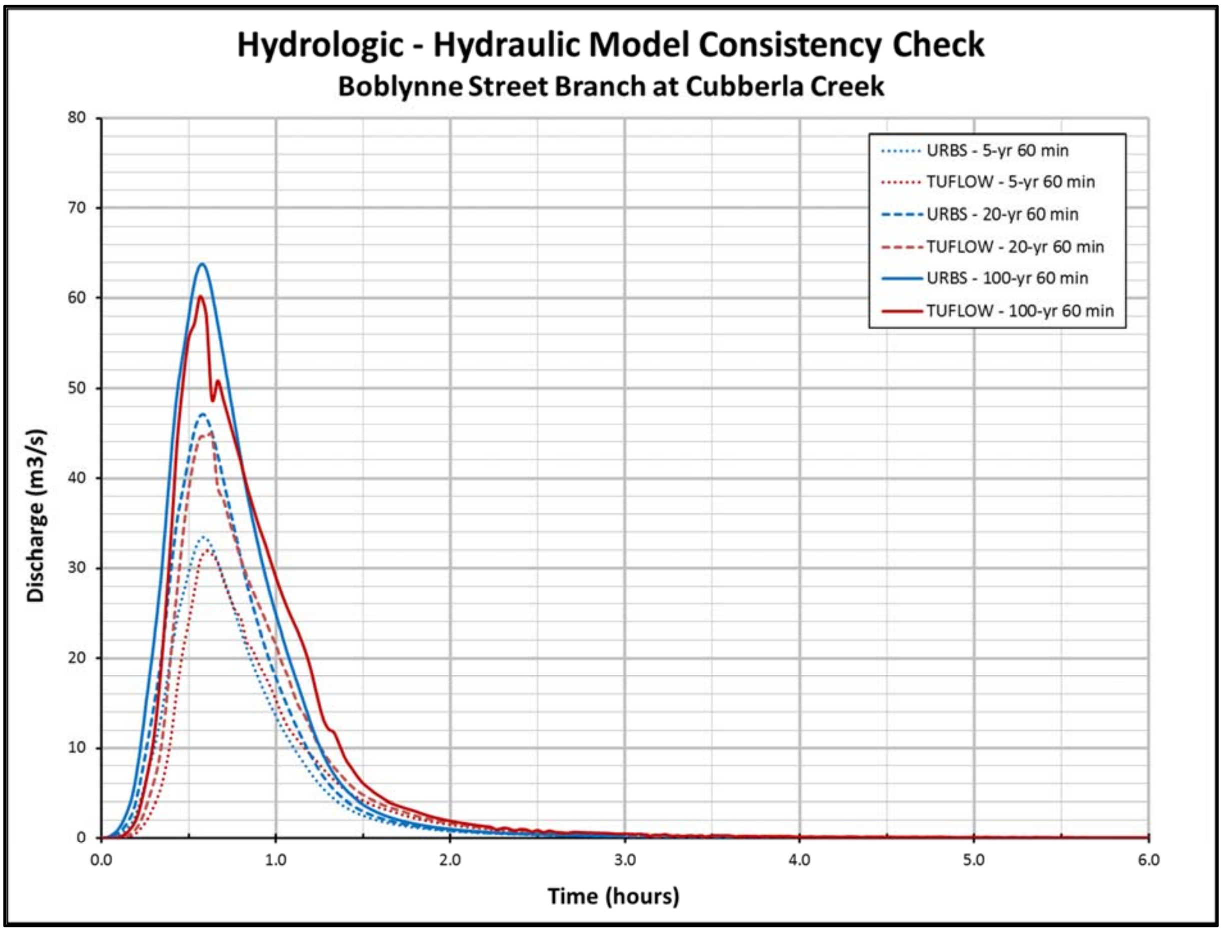

Figure 6.8: Boblynne Branch at Cubberla Creek

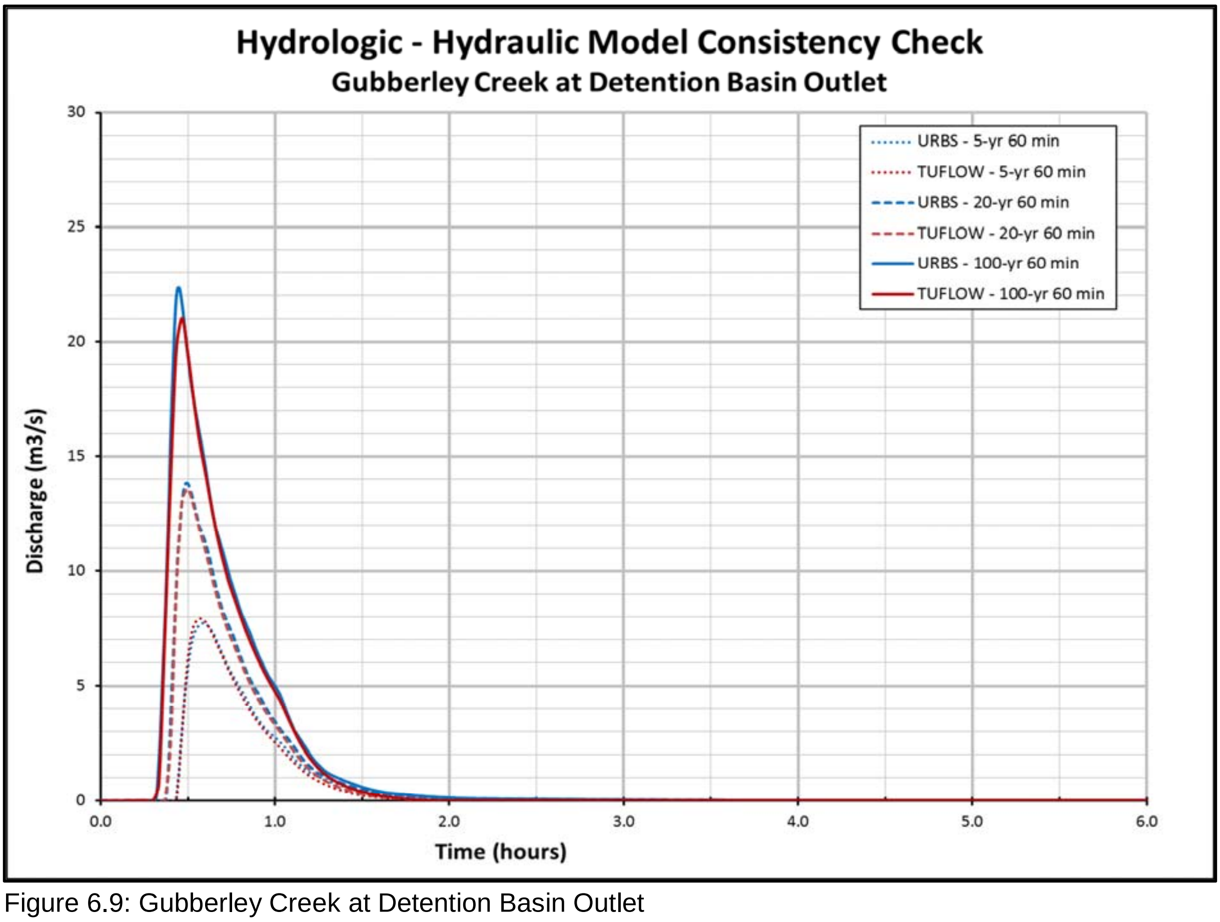

Figure 6.9: Gubberley Creek at Detention Basin Outlet

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 200 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 1.5 and 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 1987 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	165	189.9	222.2
1	113	134	156.8
1.5	90.1	106.1 (1)	124.2 (1)
2	71.4	83.3 (1)	97.5 (1)
3	53.2	61.1	71.5
6	31.7	36.9	43.2

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)	Cumulative Rainfall (%)	Rainfall (mm)		Time (hr)	Cumulative 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 1987 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 Western Freeway Barrier Blockage

For the purposes of rare and extreme event modelling, all the barriers as discussed previously in Section 5.3.4 were assumed to be blocked. This included assuming the large noise barriers to be fully blocked and impervious to flow. A comparison of the difference in the 2000-yr ARI (0.05 % AEP) flood level between fully blocked and un-blocked noise barriers is presented in Section 7.5.2.

7.4.5 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 1.82 mAHD.

7.4.6 Hydraulic Structures

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

To limit issues with model instabilities generated by extreme flows, the TUFLOW model for the PMF event excluded the access bridge at 70 Tristania Road (S12).

7.5 Results and Mapping

7.5.1 2000-yr ARI (0.05 % AEP)

During the course of this flood study it became apparent that for some of the smaller creeks / tributaries the 2000-yr ARI (0.05 % AEP) super-storm methodology was producing peak flows lower than those produced by the 500-yr ARI (0.2 % AEP) AR&R 1987 methodology. In some areas the 200-yr ARI (0.5 % AEP) AR&R 1987 methodology produced higher flows than the 2000-yr ARI (0.05 % AEP) super-storm methodology. Areas where there are anomalies with the 2000-yr ARI (0.05 % AEP) results include the Boblynne Street Branch (Upper to Middle); Gubberley Creek; Akuna Street Branch; Tributary A, Tributary B and Tributary C (Upper). This appears to be a result of a short time to peak (i.e. small catchment) in combination with higher short duration rainfall intensities when compared with the super-storm rainfall intensities.

To remain consistent with the other recently completed BCC flood studies, the 2000-yr ARI (0.05 % AEP) super-storm methodology was not changed.

7.5.2 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 G
- Scenario 3: 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) events – Appendix H

The lower section of the catchment is dominated by flooding originating from the Brisbane River; as such the reported peak flood levels in this area will be lower than the Brisbane River peak flood levels for each respective ARI (AEP).

Table 7.4 indicates a comparison of the difference in the Scenario 1 2000-yr ARI (0.05 % AEP) flood level between fully blocked and un-blocked noise barriers.

Table 7.4 – Comparison of Noise Barrier Impacts 2000-yr ARI (0.05 % AEP)

Creek	Location	Scenario 1 2000-yr ARI (0.05 % AEP) Flood Level (m AHD)		
		Noise Barrier Fully Blocked	Noise Barrier Excluded	Difference (m)
Cubberla	Gubberley Creek Junction	14.43	14.33	0.10
Cubberla	Akuna St. Branch Junction	14.29	13.60	0.69
Cubberla	U/S Western Freeway	14.11	13.09	1.02
Cubberla	D/S Western Freeway	10.82	10.91	-0.09
Cubberla	500m d/s of Western Freeway	7.85	8.06	-0.21
Tributary C	U/S Freeway On Ramp	14.09	13.17	0.92

The results indicate that flood levels upstream of the Western Freeway are up to 1.02 m higher with the noise barrier blockage included. Upstream flood level differences are apparent from the vicinity of the Gubberley Creek Junction to the Western Freeway. Downstream of the freeway, the inclusion of the noise barrier results in flood level reductions of up to 0.21 m.

7.5.3 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)

7.5.4 Discussion of Results

A longitudinal plot of the Scenario 1 100-yr ARI (1 % AEP) to PMF flood profiles for the major creeks is provided in Figure 7.1 to Figure 7.5.

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 most of the creeks, noting the anomalies discussed in Section 7.5.1.

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 four tributaries occur at the confluence with Cubberla Creek, which is primarily due to backwater effects from Cubberla Creek.

The Cubberla Creek flood profile identifies Moggill Road, Western Freeway and the section of channel in the vicinity of Jesmond Road as possible restrictions to flow in the rare and extreme events.

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 7.5. The results indicate the largest differences are in Cubberla Creek and the smallest in Gubberley Creek.

Table 7.5 – Average Increase in Flood Level

Event	Average Increase in Flood Level (m) with reference to the 100-yr ARI (1 % AEP) flood level				
	Cubberla Creek	Boblynne Street Branch	Gubberley Creek	Akuna Street Branch	Tributary C
200-yr ARI (0.5 % AEP)	0.21	0.18	0.12	0.13	0.17
500-yr ARI (0.2 % AEP)	0.40	0.34	0.22	0.29	0.37
2000-yr ARI (0.05 % AEP)	0.65	0.32 (1)	0.13 (1)	0.19 (1)	0.56
PMF	1.85	0.99	0.74	0.98	1.93

(1) In areas with a small upstream catchment, the 2000-yr ARI (0.05 % AEP) super-storm method does not always produce a flow greater than the 500-yr ARI (0.2 % AEP)

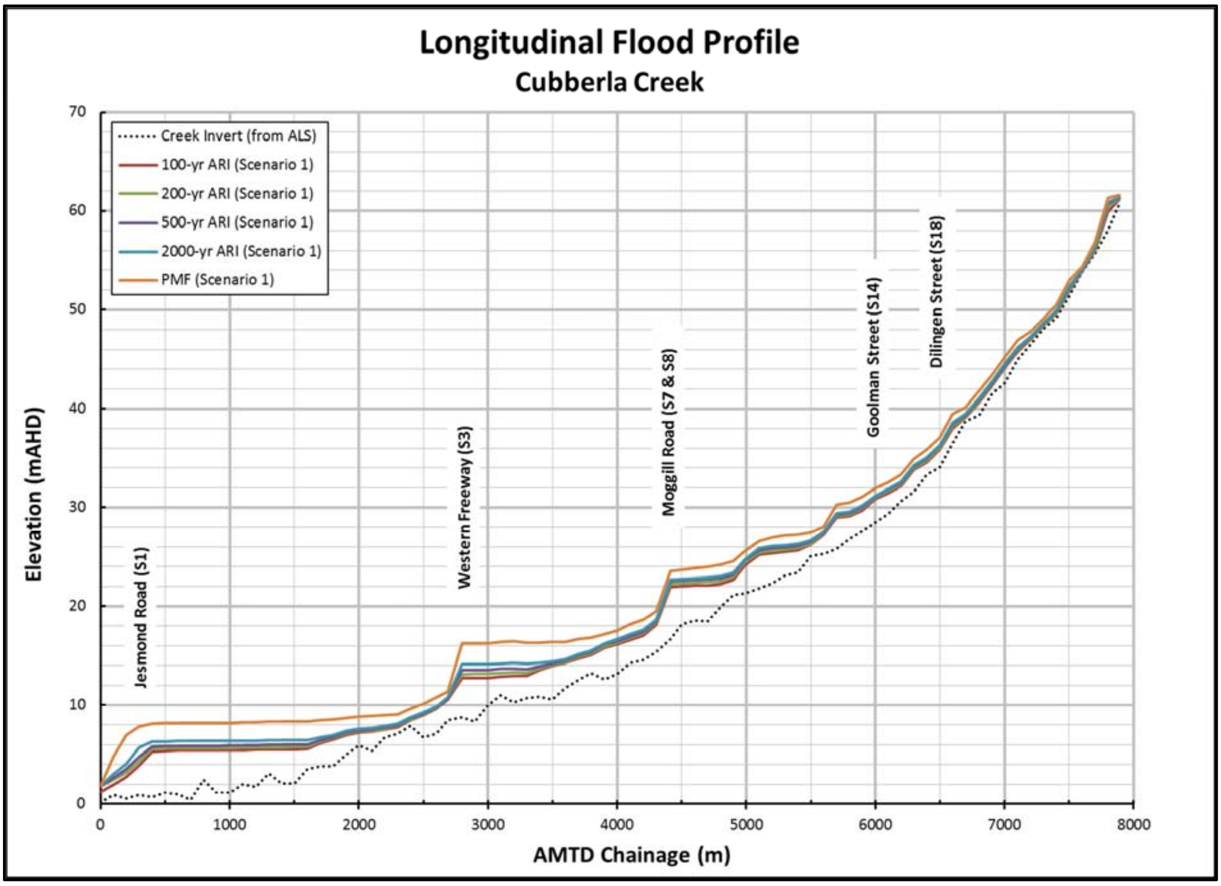

Figure 7.1: Longitudinal Flood Profile – Cubberla Creek

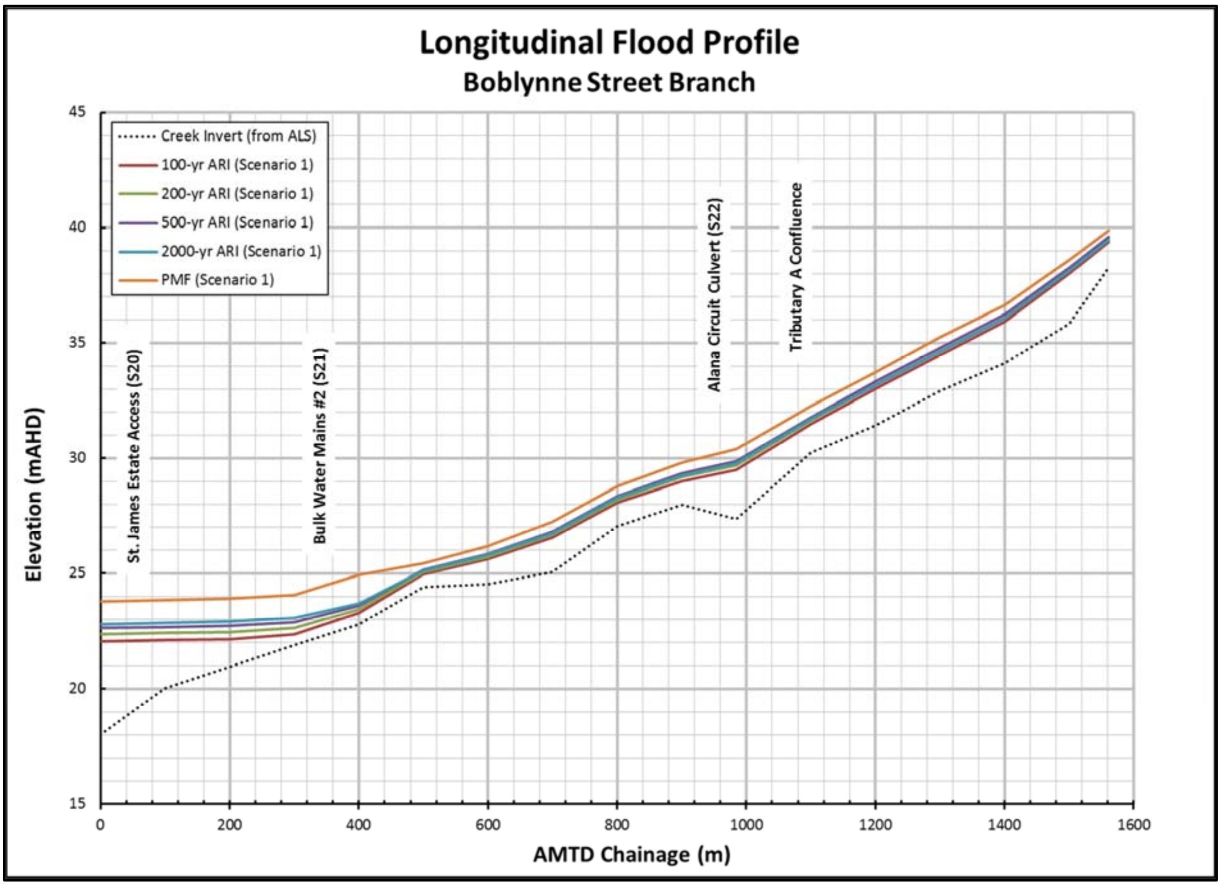

Figure 7.2: Longitudinal Flood Profile – Boblynne Street Branch

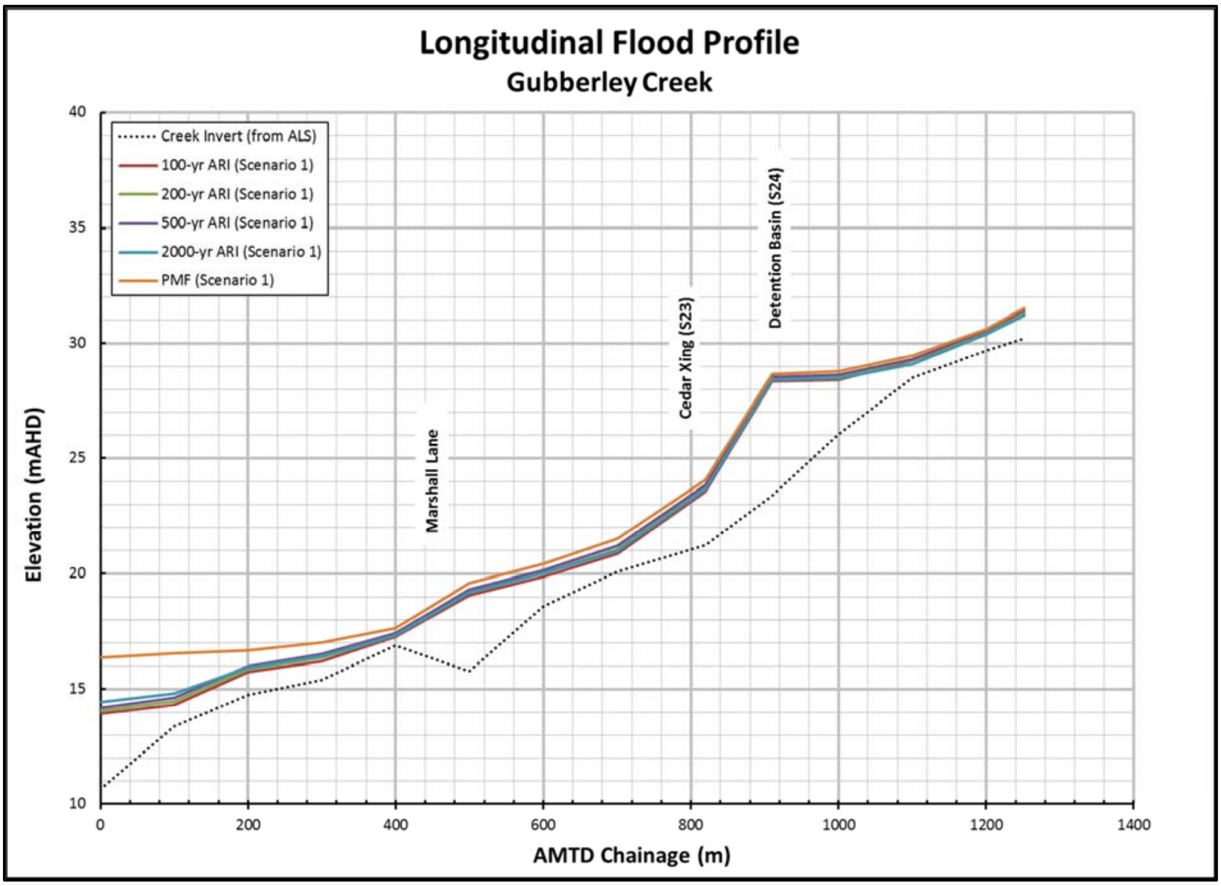

Figure 7.3: Longitudinal Flood Profile – Gubberley Creek

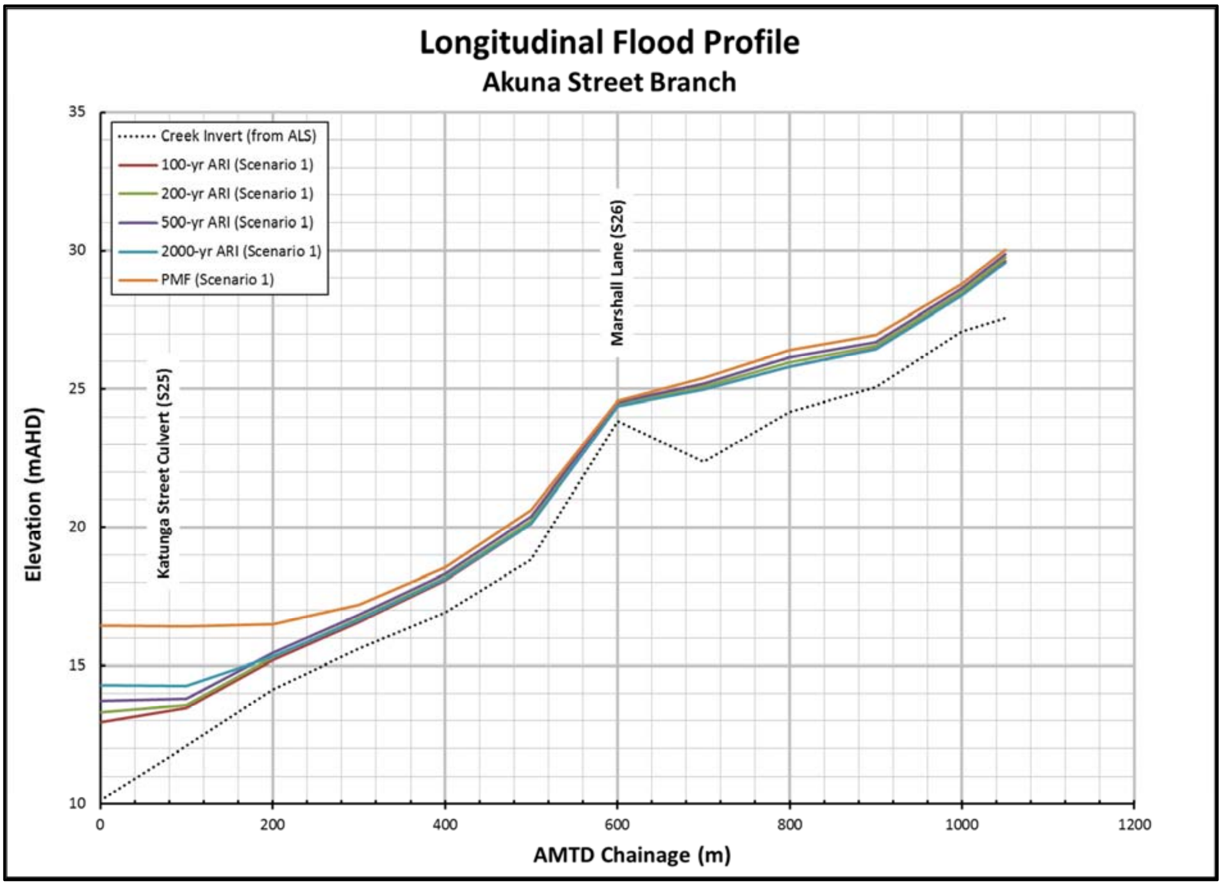

Figure 7.4: Longitudinal Flood Profile – Akuna Street Branch

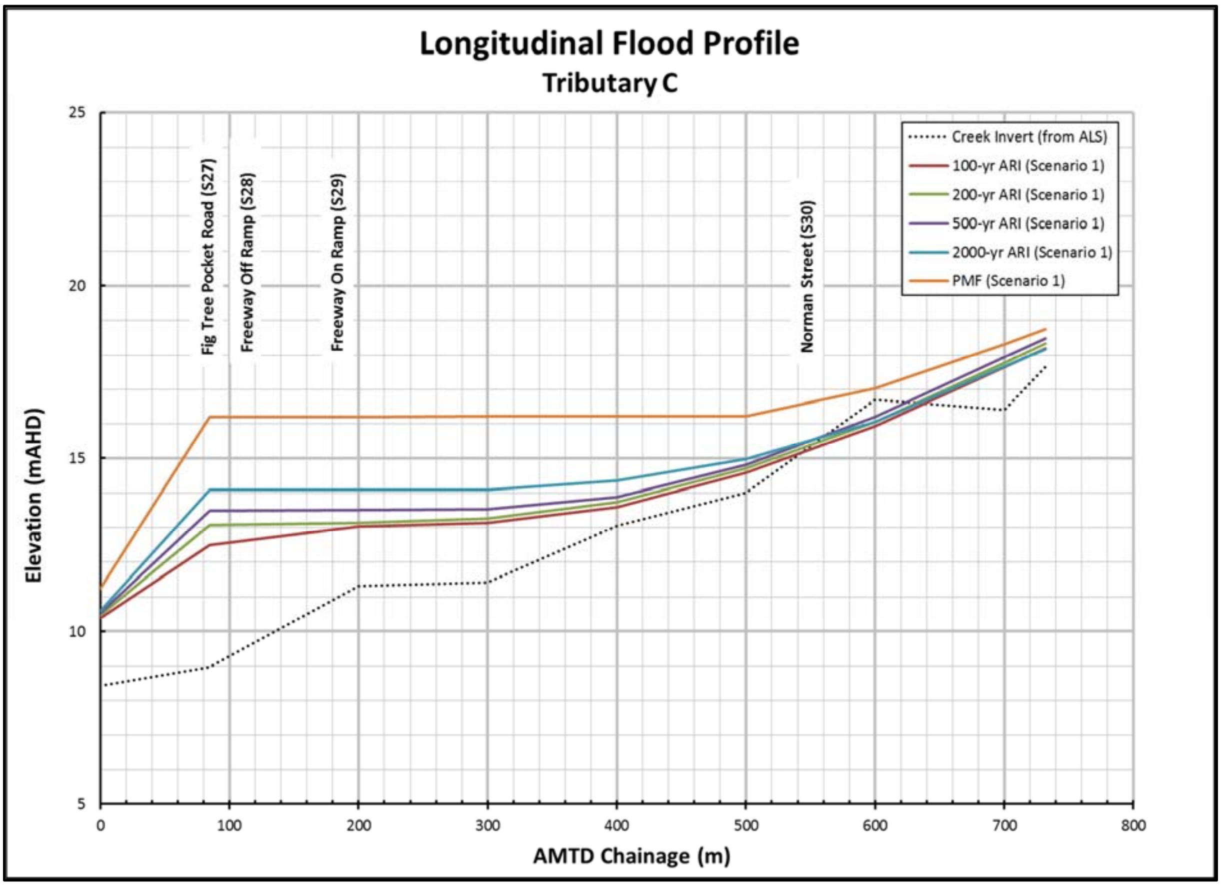

Figure 7.5: Longitudinal Flood Profile – Tributary C

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8.0 Climate Variability

8.1 Overview

There is general consensus that human activities are contributing to observed changes in climate. Human induced climate change has the potential to alter the prevalence and severity of rainfall extremes, storm surge and floods. [8]

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 and climate change adaption, there is a requirement to understand the impacts of climate variability on flooding. As part of this climate variability assessment, two future planning horizons were considered, namely 2050 and 2100.

The latest practitioner guidance on the climate change impacts of rainfall intensity is from AR&R 2016. AR&R 2016 recommends the consideration of two representative concentration pathways; namely RCP4.5 and RCP8.5. RCP8.5 assumes greater greenhouse gas emissions than RCP4.5, resulting in increased rainfall intensity.

The four climate futures included in the modelling are as follows:

- Year 2050 (RCP4.5)
    - 6.7 % increase in rainfall intensity
    - 0.3 m increase in mean sea level
- Year 2050 (RCP8.5)
    - 8.8 % increase in rainfall intensity
    - 0.3 m increase in mean sea level
- Year 2100 (RCP4.5)
    - 9.3 % increase in rainfall intensity
    - 0.8 m increase in mean sea level
- Year 2100 (RCP8.5)
    - 21 % increase in rainfall intensity
    - 0.8 m increase in mean sea level

[8] Bates B, McLuckie D, Westra S, Johnson F, Green J, Mummery J, Abbs D, 2016, Climate Change Considerations, Chapter 6 Book 1 in Australian Rainfall and Runoff – A Guide to Flood Estimation, Commonwealth of Australia

Currently the guidance on rainfall intensity increases due to climate change only extend as far as 2090. The AR&R 2016 Data Hub (Beta) only provides values from 2030 to 2090. In order to obtain a value for 2100 an extrapolation was undertaken based on the values of 2080 and 2090.

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	RCP	Rainfall Intensity	Tailwater Condition	Scenario 1	Scenario 3
100	1	2050	4.5	+ 6.7 %	MHWS + 0.3 m = 1.51mAHD	✓	✓
			8.5	+ 8.8 %		✓	✓
		2100	4.5	+ 9.3 %	MHWS + 0.8 m = 2.01mAHD	✓	✓
			8.5	+ 21 %		✓	✓
200	0.5	2050	4.5	+ 6.7 %	HAT + 0.3 m = 2.12mAHD	✓	×
			8.5	+ 8.8 %		✓	×
		2100	4.5	+ 9.3 %	HAT + 0.8 m = 2.62mAHD	✓	×
			8.5	+ 21 %		✓	×
500	0.2	2100	4.5	+ 9.3 %	HAT + 0.8 m = 2.62mAHD	✓	×
			8.5	+ 21 %		✓	×

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 2050 (RCP4.5); 2050 (RCP8.5); 2100 (RCP4.5) and 2100 (RCP8.5) rainfall intensity scenarios. The inflow boundary locations did not change from the design event modelling.

8.2.4 Impacts of Climate Variability

Table 8.2 to Table 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. Figure 8.1 to Figure 8.4 indicate the differences in the 100-yr ARI (1 % AEP) event at four locations along Cubberla Creek.

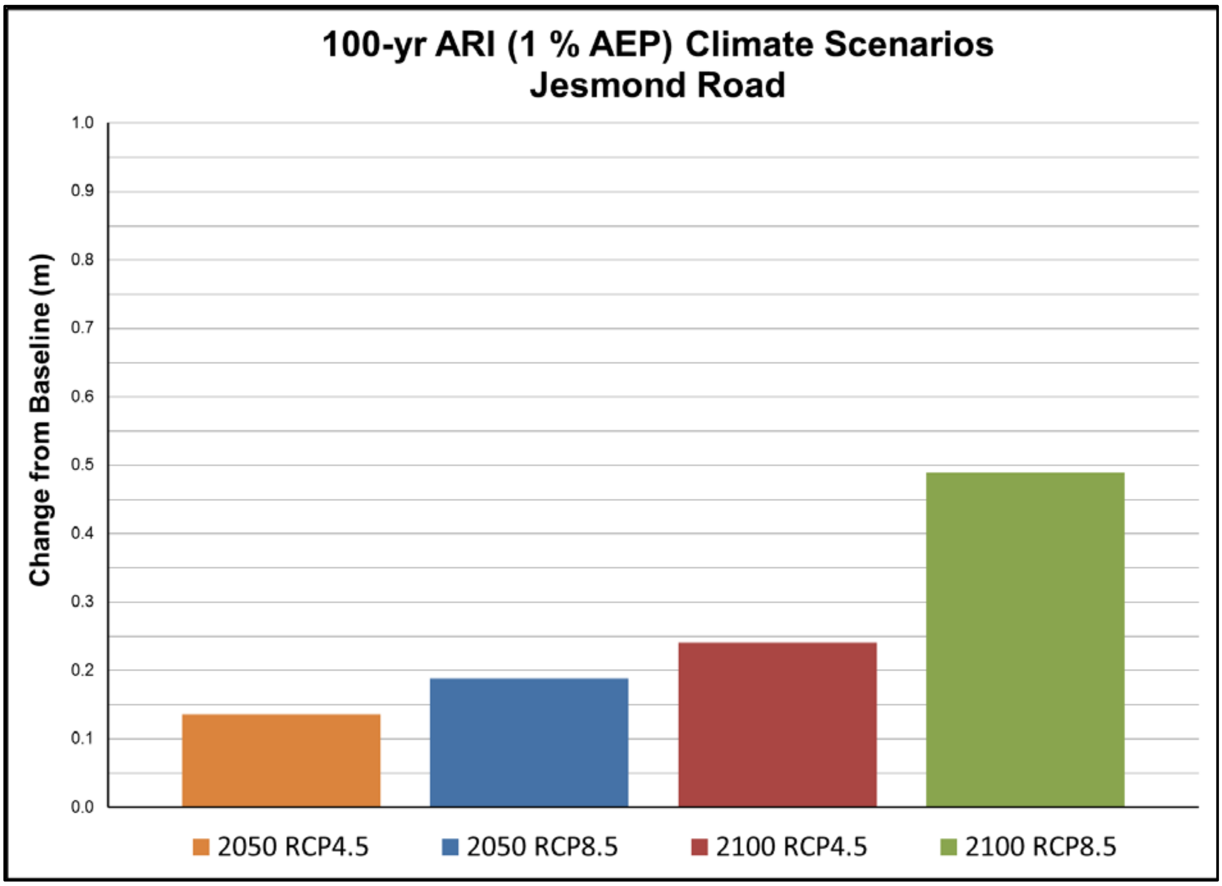

Figure 8.1: 100-yr ARI (1% AEP) Climate Scenario Differences – Jesmond Road

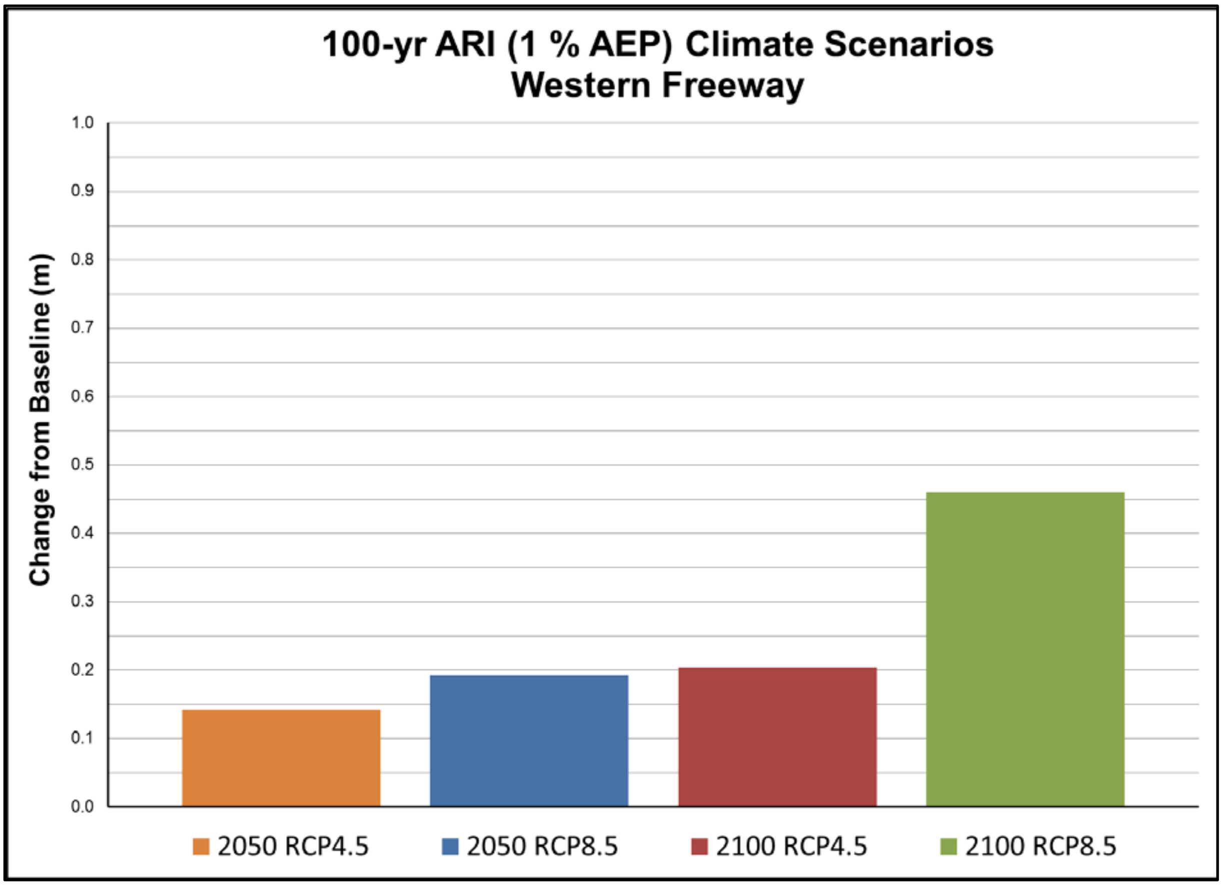

Figure 8.2: 100-yr ARI (1% AEP) Climate Scenario Differences – Western Freeway

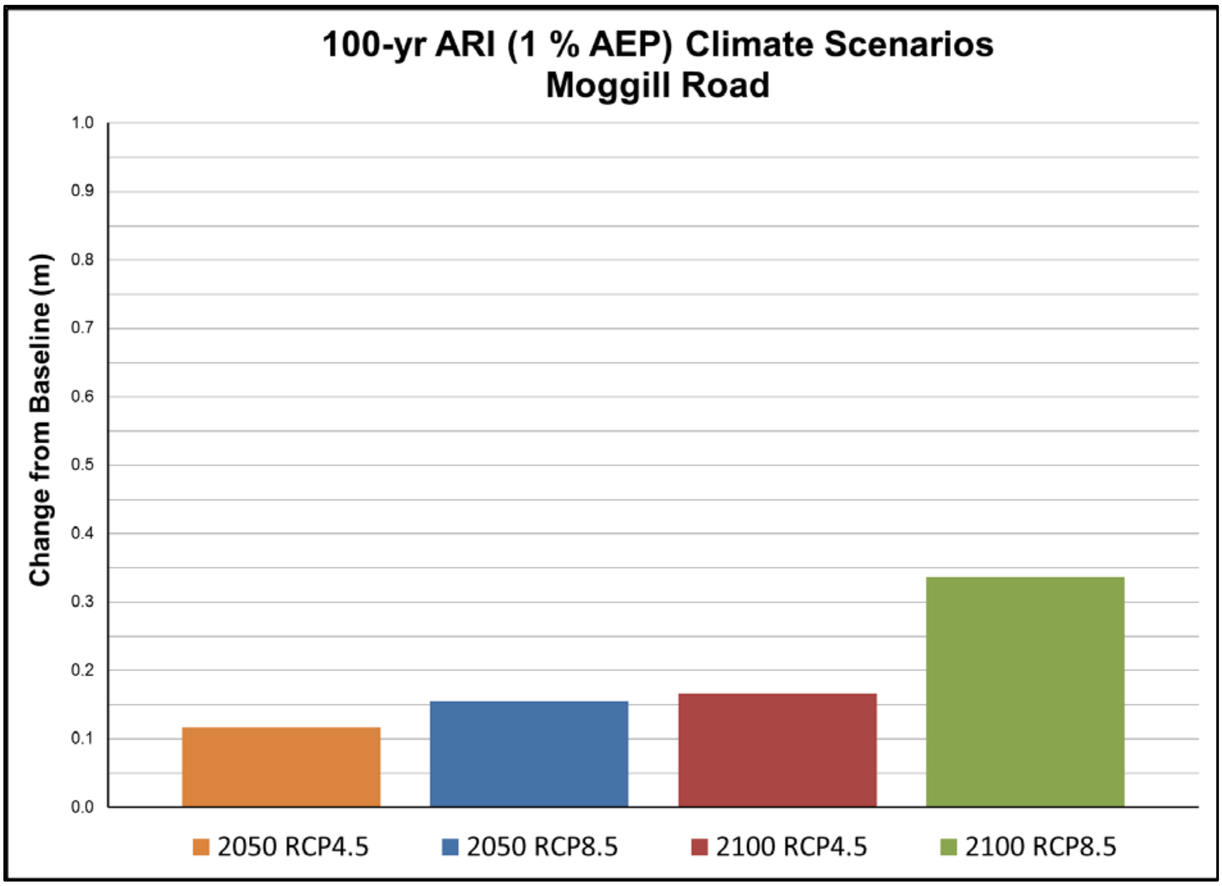

Figure 8.3: 100-yr ARI (1% AEP) Climate Scenario Differences – Moggill Road

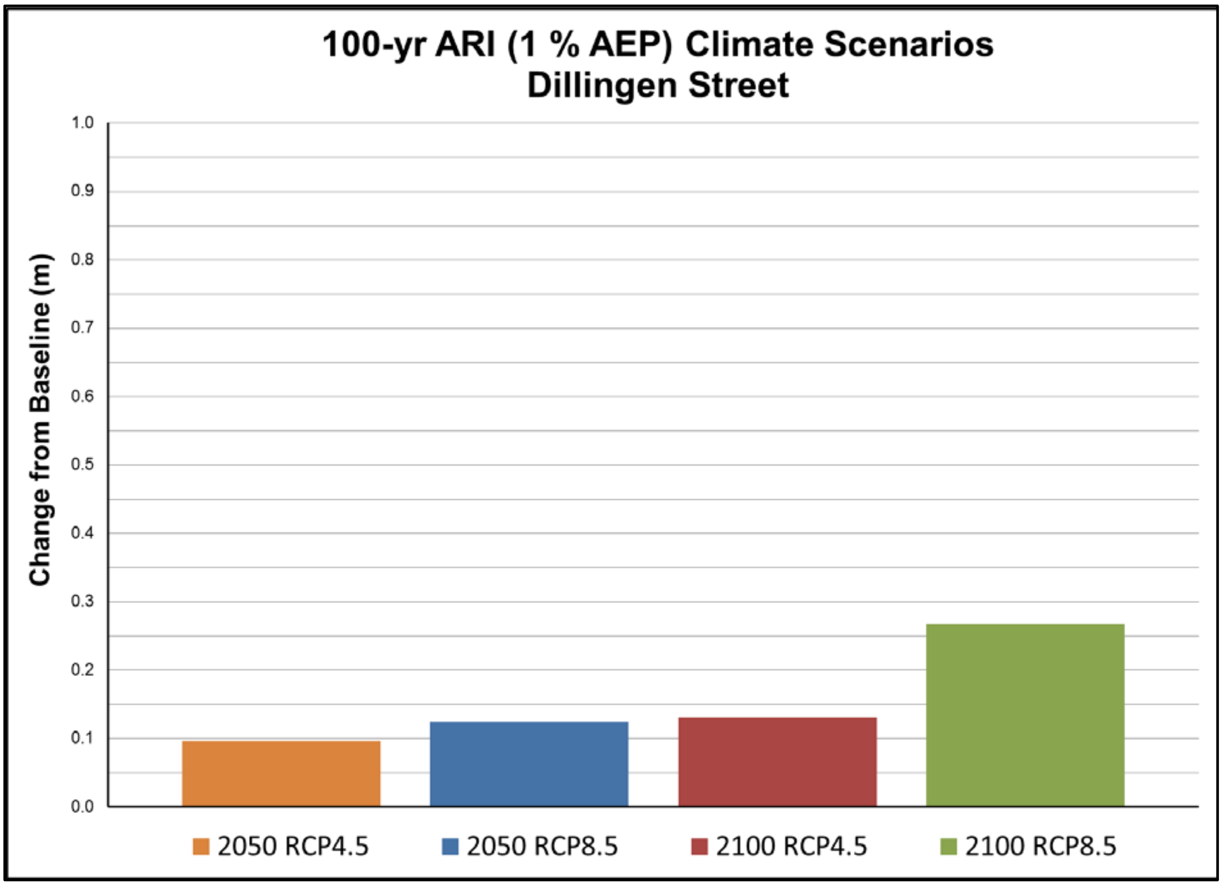

Figure 8.4: 100-yr ARI (1% AEP) Climate Scenario Differences – Dillingen Street

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 RCP4.5**		**2050 RCP8.5**		**2100 RCP4.5**		**2100 RCP8.5**	
		WL (mAHD)	**Afflux (m)**	**WL (mAHD)**	**Afflux (m)**	**WL (mAHD)**	**Afflux (m)**	**WL (mAHD)**	**Afflux (m)**
Cubberla Creek									
Greenford Street (S19)	59.93	60.15	0.23	60.22	0.29	60.23	0.30	60.48	0.55
Dillingen Street (S18)	37.53	37.65	0.12	37.69	0.16	37.70	0.17	37.84	0.31
Goolman Street (S14)	31.02	31.08	0.06	31.10	0.08	31.10	0.08	31.20	0.18
Tristania Road (S10)	25.02	25.07	0.05	25.08	0.07	25.08	0.07	25.21	0.19
Moggill Road (S7)	21.87	21.99	0.12	22.03	0.16	22.04	0.17	22.21	0.34
Western Freeway (S3)	12.66	12.81	0.14	12.86	0.20	12.87	0.21	13.13	0.47
Jesmond Road (S1)	3.93	4.06	0.14	4.11	0.19	4.17	0.24	4.42	0.49
Boblynne Street Branch									
St. James Estate Access (S20)	22.08	22.20	0.12	22.24	0.15	22.25	0.17	22.42	0.34
Gubberley Creek									
Detention Basin	28.33	28.36	0.03	28.37	0.04	28.38	0.05	28.43	0.10
Marshall Lane	19.01	19.05	0.04	19.06	0.05	19.06	0.05	19.13	0.12
Akuna Street Branch									
Marshall Lane (S26)	24.76	24.79	0.03	24.80	0.04	24.81	0.05	24.86	0.10
Tributary C									
Western Freeway On Ramp (S29)	13.03	13.07	0.04	13.08	0.05	13.09	0.06	13.16	0.13

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 RCP4.5		2050 RCP8.5		2100 RCP4.5		2100 RCP8.5	
		WL (mAHD)	Afflux (m)	WL (mAHD)	Afflux (m)	WL (mAHD)	Afflux (m)	WL (mAHD)	Afflux (m)
Cubberla Creek									
Greenford Street (S19)	60.44	60.58	0.14	60.61	0.18	60.62	0.18	60.76	0.32
Dillingen Street (S18)	37.82	37.90	0.08	37.92	0.10	37.92	0.11	38.05	0.23
Goolman Street (S14)	31.18	31.24	0.06	31.26	0.08	31.26	0.08	31.37	0.19
Tristania Road (S10)	25.19	25.22	0.03	25.25	0.05	25.25	0.06	25.36	0.17
Moggill Road (S7)	22.19	22.29	0.10	22.31	0.13	22.32	0.13	22.49	0.30
Western Freeway (S3)	13.09	13.22	0.13	13.27	0.18	13.28	0.19	13.57	0.48
Jesmond Road (S1)	4.36	4.53	0.17	4.58	0.22	4.66	0.30	4.92	0.56
Boblynne Street Branch									
St. James Estate Access (S20)	22.39	22.51	0.11	22.53	0.13	22.54	0.14	22.70	0.31
Gubberley Creek									
Detention Basin	28.43	28.48	0.05	28.49	0.06	28.49	0.06	28.55	0.12
Marshall Lane	19.12	19.15	0.03	19.16	0.04	19.17	0.05	19.23	0.11
Akuna Street Branch									
Marshall Lane (S26)	24.84	24.87	0.03	24.88	0.05	24.89	0.05	24.95	0.11
Tributary C									
Western Freeway On Ramp (S29)	13.14	13.25	0.11	13.30	0.16	13.31	0.17	13.59	0.44

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 RCP4.5		2100 RCP8.5	
		WL (mAHD)	Afflux (m)	WL (mAHD)	Afflux (m)
Cubberla Creek					
Greenford Street (S19)	60.72	60.82	0.10	60.92	0.20
Dillingen Street (S18)	38.01	38.10	0.09	38.21	0.20
Goolman Street (S14)	31.34	31.43	0.08	31.54	0.19
Tristania Road (S10)	25.36	25.43	0.06	25.55	0.19
Moggill Road (S7)	22.46	22.56	0.10	22.70	0.24
Western Freeway (S3)	13.49	13.73	0.24	14.07	0.58
Jesmond Road (S1)	4.77	5.35	0.58	5.61	0.84
Boblynne Street Branch					
St. James Estate Access (S20)	22.66	22.78	0.12	22.92	0.26
Gubberley Creek					
Detention Basin	28.51	28.58	0.06	28.63	0.12
Marshall Lane	19.22	19.26	0.05	19.35	0.13
Akuna Street Branch					
Marshall Lane (S26)	24.93	24.98	0.05	25.05	0.12
Tributary C					
Western Freeway On Ramp (S29)	13.51	13.75	0.24	14.08	0.57

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. The following observations were made from the results:

- Flood level increases are greater under RCP8.5 climate projections when compared with RCP4.5 climate projections.
- 2050 RCP8.5 and 2100 RCP4.5 flood levels are almost identical for those areas not affected by projected sea level increases.
- Based on RCP8.5 climatic projections, by the year 2100, 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 for those areas not affected by projected sea level increases.
- Based on RCP8.5 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 for those areas not affected by projected sea level increases.

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9.0 Summary of Study Findings

This flood study report details the calibration and verification, design event, rare / extreme event and sensitivity modelling for the Cubberla Creek Catchment. This includes Cubberla Creek; Boblynne Street Branch; Gubberley Creek; Akuna Street Branch; Tributary A; Tributary B and Tributary C. 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 rainfall and maximum height gauge records. 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 the synthetic design flood events.

Cross-checks of the TUFLOW hydraulic 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, rare 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. A fixed tidal boundary was used at the downstream model extent to represent the Brisbane River.

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

The lower section of the catchment is dominated by flooding originating from the Brisbane River; as such the reported peak flood levels in this area will be lower than the Brisbane River peak flood levels for each respective ARI (AEP).

As part of the required sensitivity analysis, a climate variability analysis was then undertaken to determine the impacts for four climate futures; namely Year 2050 RCP4.5; Year 2050 RCP8.5; Year 2100 RCP4.5 and Year 2100 RCP8.5. This included making allowances for increased rainfall intensity and increased mean sea level. 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 indicated that climate variability impacts within the catchment will increase the magnitude of flooding. The following observations were made from the results:

- Flood level increases are greater under RCP8.5 climate projections when compared with RCP4.5 climate projections.
- 2050 RCP8.5 and 2100 RCP4.5 flood levels are almost identical for those areas not affected by projected sea level increases.
- Based on RCP8.5 climatic projections, by the year 2100, 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 for those areas not affected by projected sea level increases.
- Based on RCP8.5 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 for those areas not affected by projected sea level increases.

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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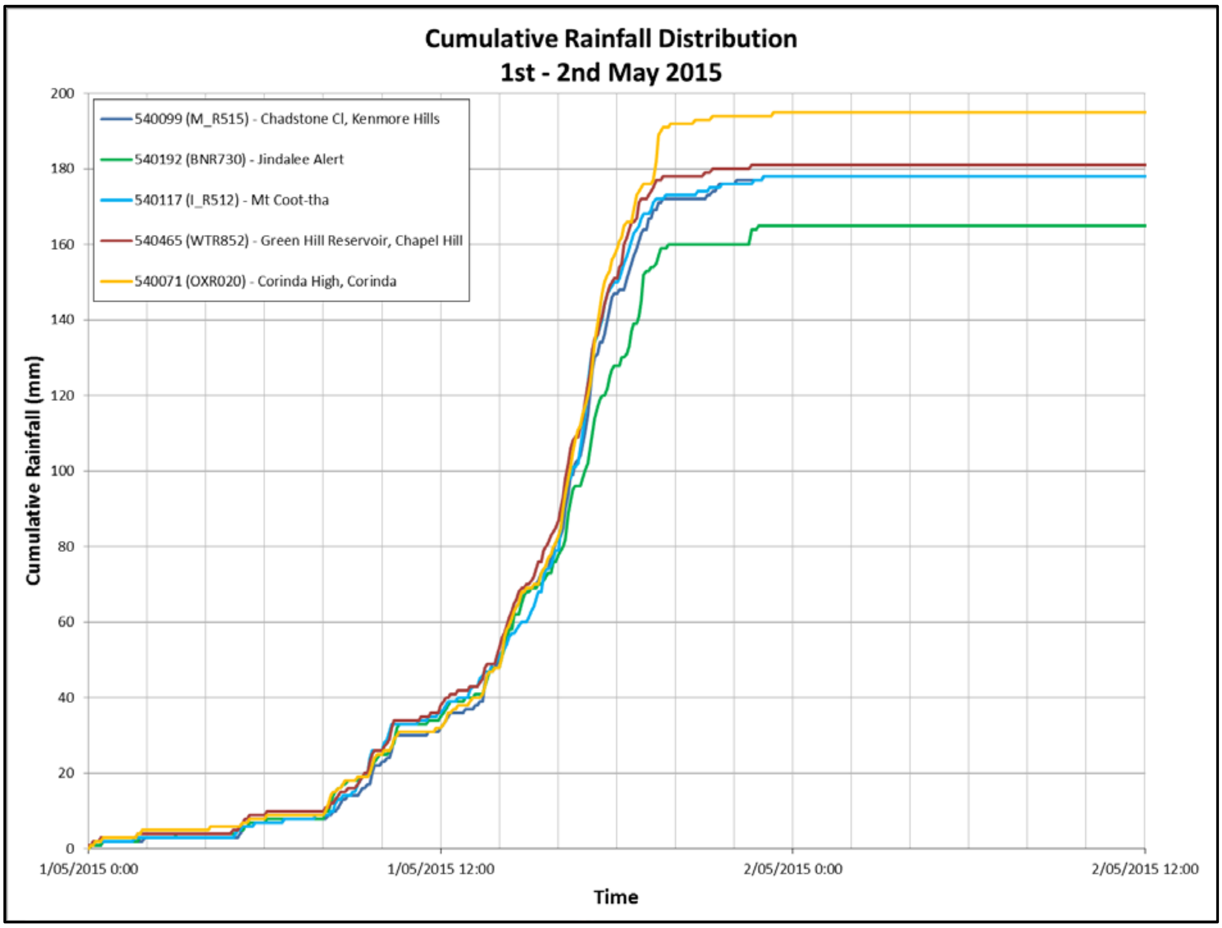
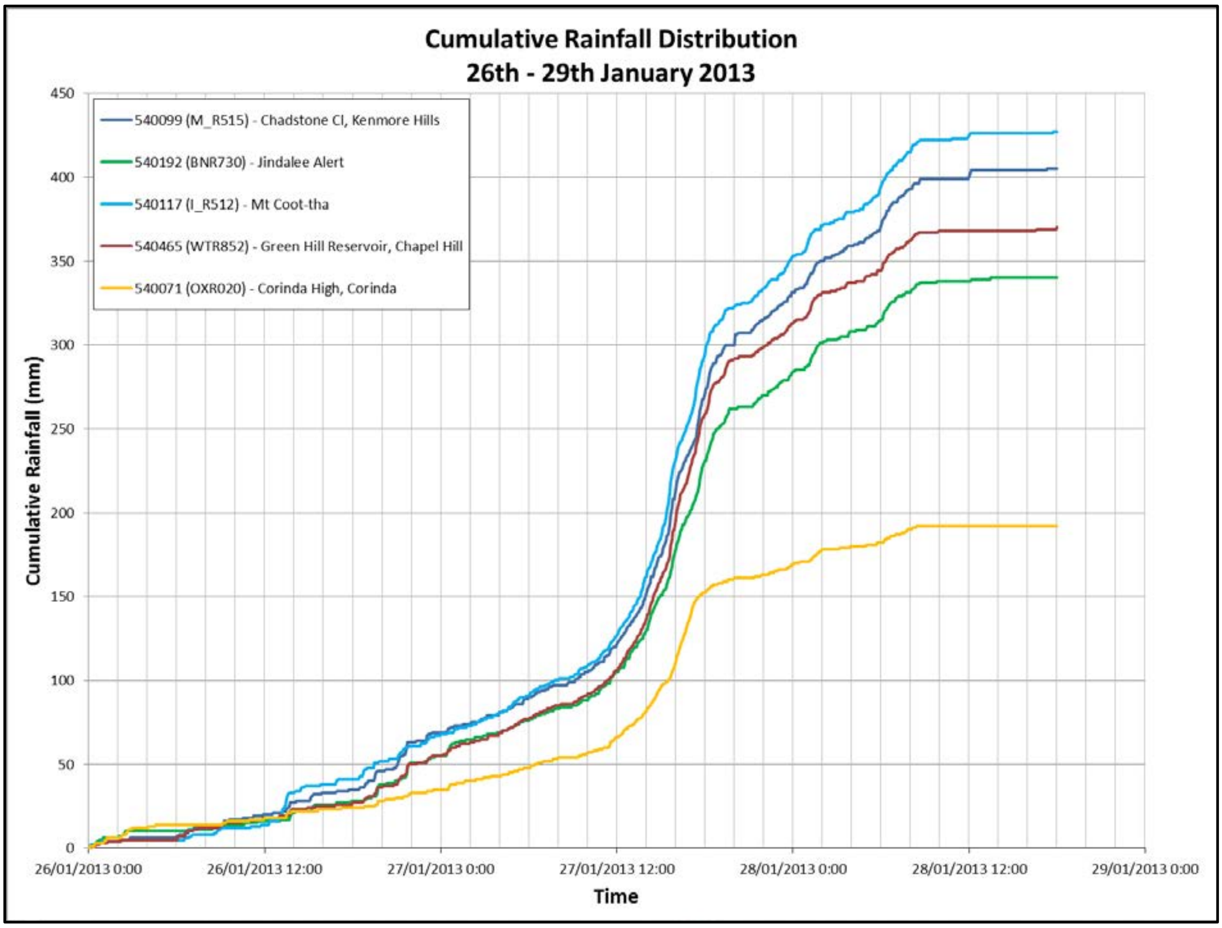
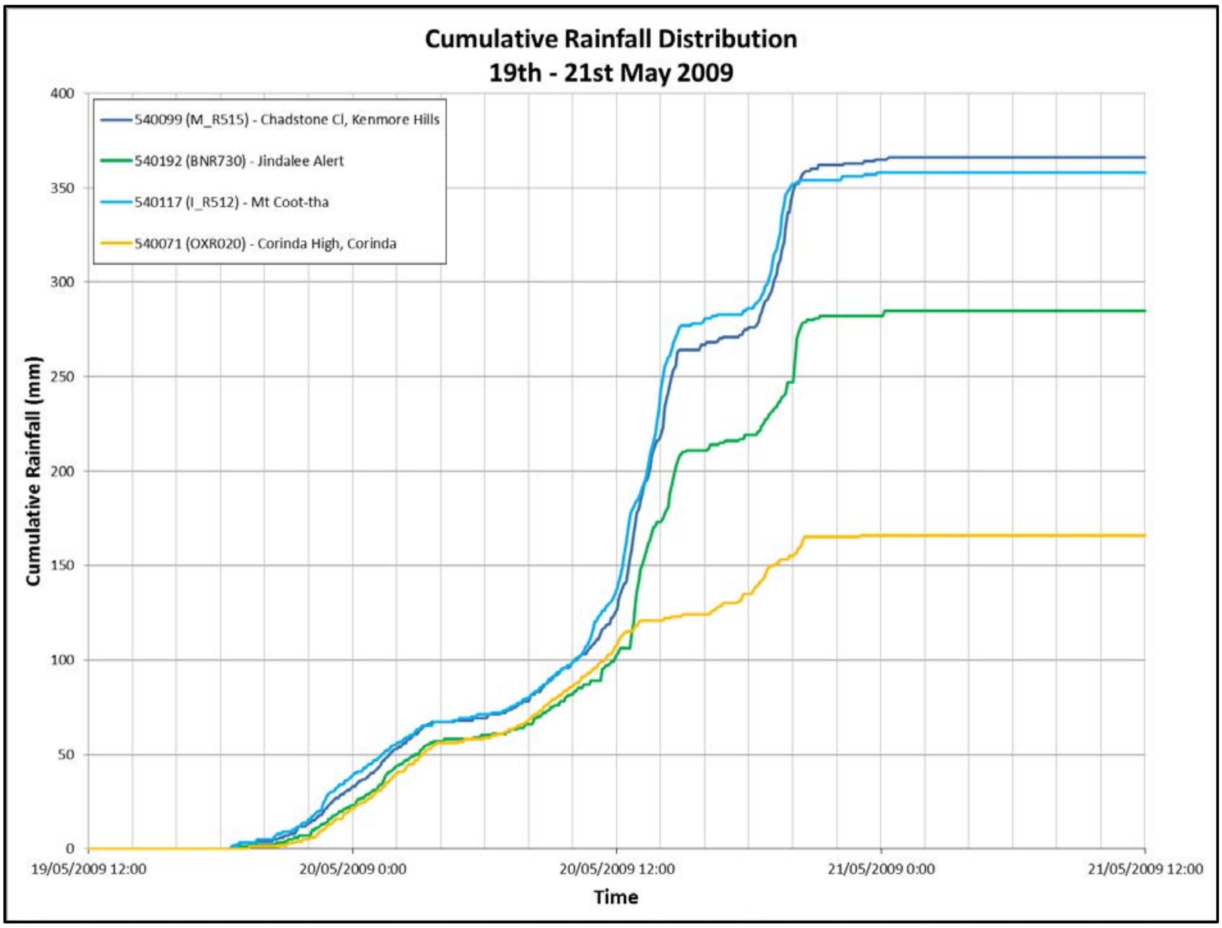


Cumulative Rainfall Distribution
1st - 2nd May 2015
540099 (M_R515) - Chadstone Cl, Kenmore Hills
540192 (BNR730) - Jindalee Alert
540117 (I_R512) - Mt Coot-tha
540465 (WTR852) - Green Hill Reservoir, Chapel Hill
540071 (OXR020) - Corinda High, Corinda
Cumulative Rainfall (mm)
0
20
40
60
80
100
120
140
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
26th - 29th January 2013
540099 (M_R515) - Chadstone Cl, Kenmore Hills
540192 (BNR730) - Jindalee Alert
540117 (I_R512) - Mt Coot-tha
540465 (WTR852) - Green Hill Reservoir, Chapel Hill
540071 (OXR020) - Corinda High, Corinda
Cumulative Rainfall (mm)
0
50
100
150
200
250
300
350
400
450
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
540192 (BNR730) - Jindalee Alert
540117 (I_R512) - Mt Coot-tha
540071 (OXR020) - Corinda High, Corinda
Cumulative Rainfall (mm)
0
50
100
150
200
250
300
350
400
19/05/2009 12:00
20/05/2009 0:00
20/05/2009 12:00
21/05/2009 0:00
21/05/2009 12:00
Time

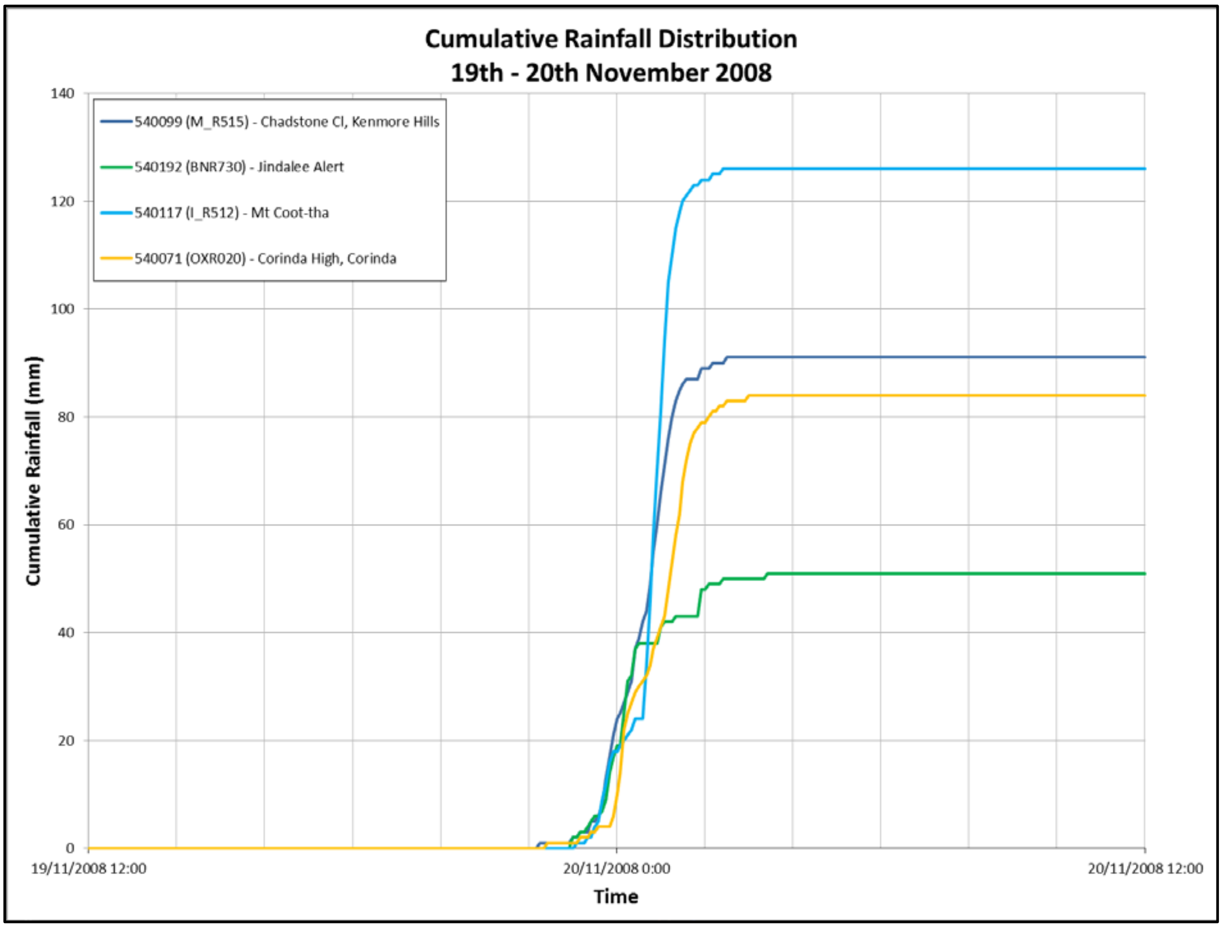

Cumulative Rainfall Distribution
19th - 20th November 2008
540099 (M_R515) - Chadstone Cl, Kenmore Hills
540192 (BNR730) - Jindalee Alert
540117 (I_R512) - Mt Coot-tha
540071 (OXR020) - Corinda High, Corinda
Cumulative Rainfall (mm)
0
20
40
60
80
100
120
140
19/11/2008 12:00
20/11/2008 0:00
20/11/2008 12:00
Time

Figure A-1: Thiessen Polygons for May 2015 and January 2013

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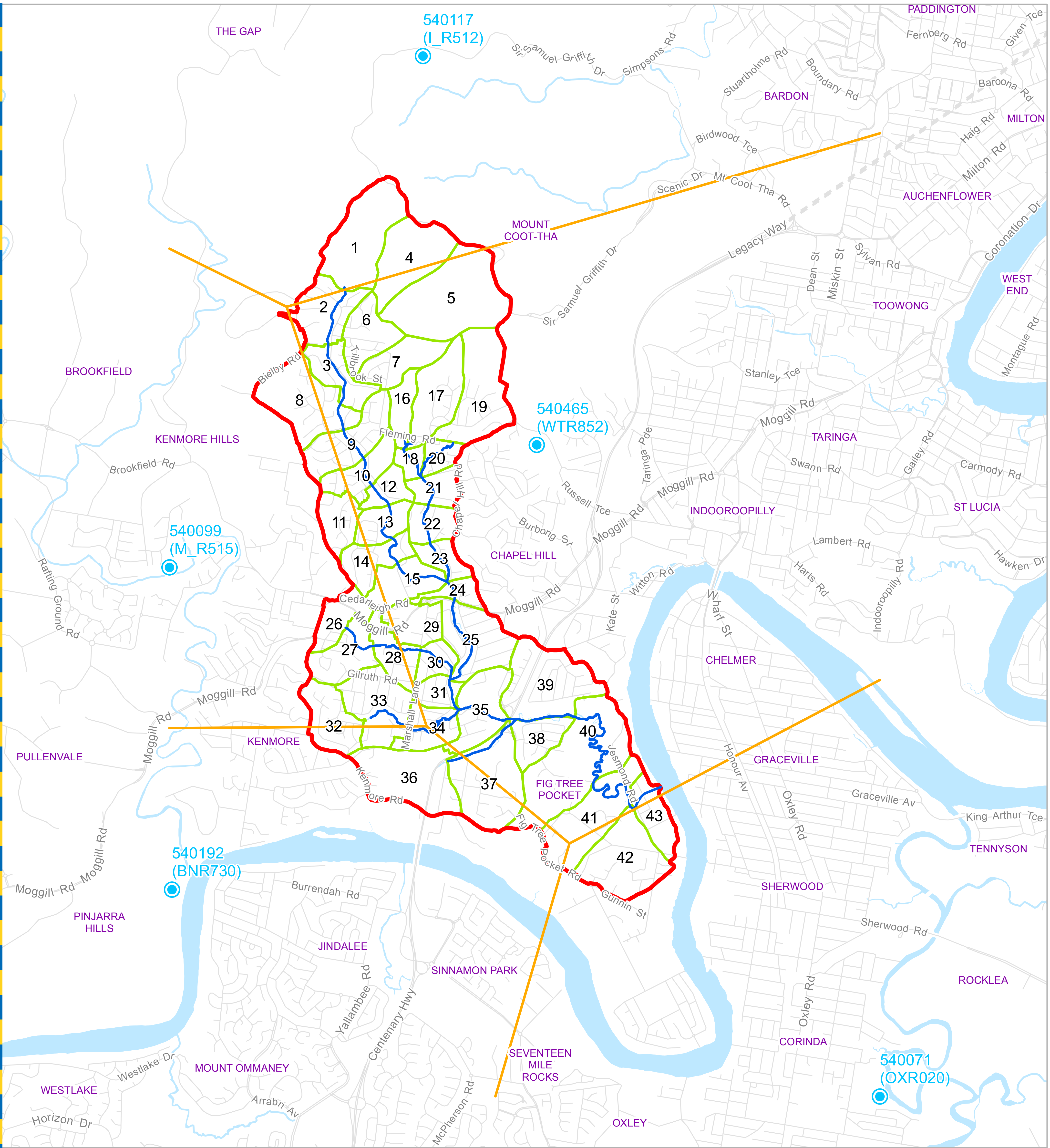

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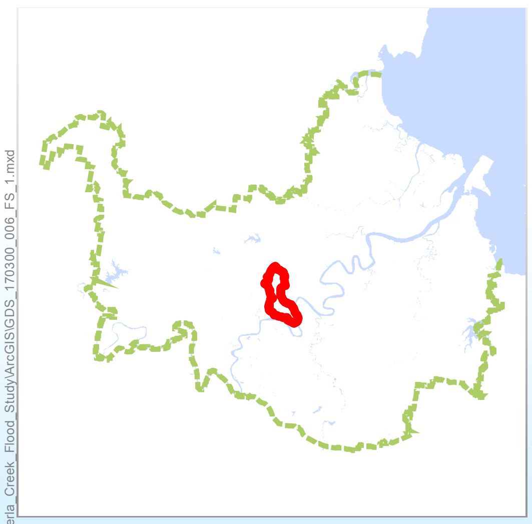

0 375 750
Metres

N
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Prepared : 081335
Checked : JS
Revision : 1
Publication Date : 17 May 2017
Project Number : 170300

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Legend

- Pluviograph Stations
- Rainfall Distribution
- Creek Centreline
- URBS Substations (1 - 43)
- Catchment Area
- Streets

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DATA INFORMATION

Cubberla Creek Flood Study

Figure A.1: Thiessen Polygons for May 2015 and January 2013

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Figure A-2: Thiessen Polygons for May 2009 and November 2008

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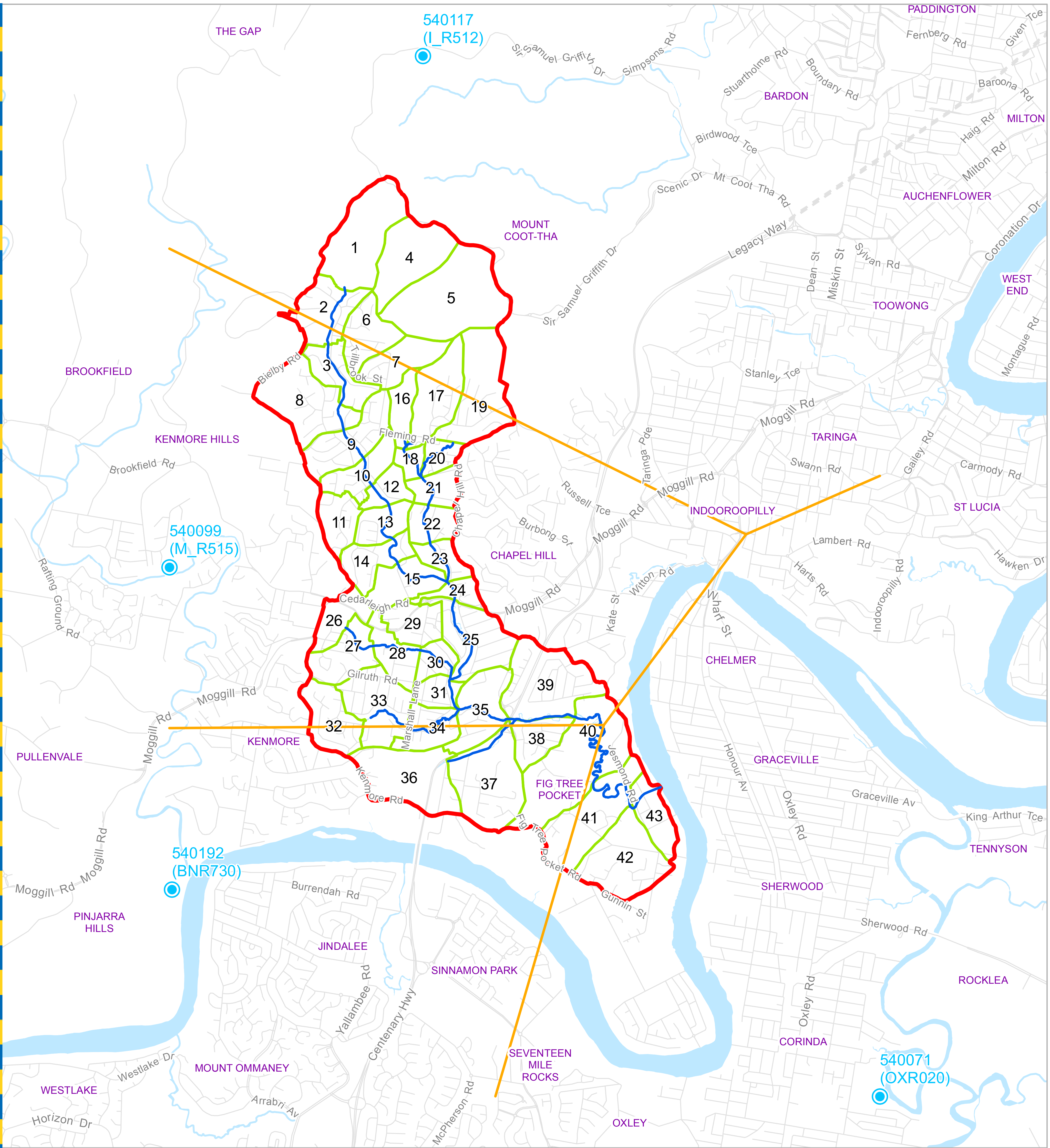

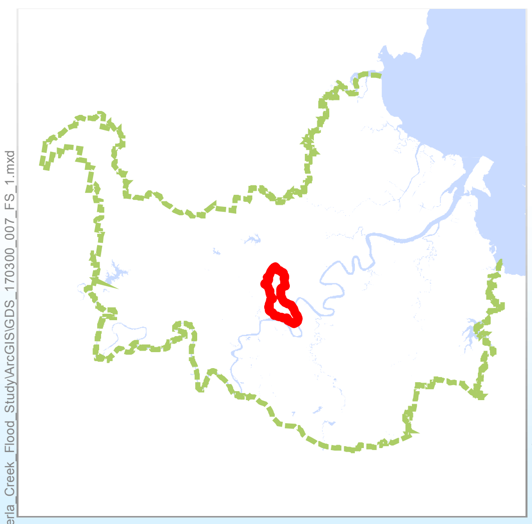

Legend

- Pluviograph Stations
- Rainfall Distribution
- Creek Centreline
- Catchment Area
- URBS Subcatchments (1 - 43)
- Streets

0 250 500 750
Metres

N E S W

Prepared : 081335
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Publication Date : 15 May 2017
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DATA INFORMATION

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Cubberla Creek Flood Study

Figure A.2: Theissen Polygons for May 2009 and November 2008

GDS - 170300 - 007

Appendix B: URBS Model Parameters

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URBS Calibration / Verification Event Sub-catchment Parameters

Sub-catchment	Area (km^2)	UL	UM	UH	UR	I
1	0.437	0.000	0.007	0.012	0.981	0.015
2	0.253	0.000	0.626	0.226	0.148	0.516
3	0.182	0.000	0.621	0.219	0.160	0.508
4	0.339	0.000	0.010	0.019	0.971	0.022
5	0.603	0.000	0.003	0.009	0.987	0.010
6	0.210	0.000	0.583	0.243	0.174	0.510
7	0.206	0.000	0.259	0.123	0.618	0.240
8	0.392	0.000	0.475	0.208	0.317	0.424
9	0.283	0.016	0.626	0.257	0.101	0.547
10	0.178	0.293	0.482	0.177	0.047	0.445
11	0.159	0.006	0.733	0.260	0.001	0.601
12	0.107	0.044	0.795	0.162	0.000	0.549
13	0.141	0.216	0.657	0.069	0.058	0.423
14	0.183	0.268	0.484	0.178	0.071	0.442
15	0.204	0.624	0.120	0.169	0.087	0.306
16	0.120	0.078	0.025	0.030	0.867	0.051
17	0.288	0.021	0.339	0.125	0.515	0.285
18	0.062	0.702	0.135	0.062	0.100	0.229
19	0.406	0.000	0.488	0.155	0.356	0.384
20	0.084	0.224	0.539	0.193	0.044	0.476
21	0.122	0.209	0.602	0.166	0.023	0.482
22	0.134	0.427	0.337	0.106	0.131	0.328
23	0.108	0.212	0.496	0.156	0.136	0.420
24	0.083	0.013	0.410	0.577	0.000	0.726
25	0.327	0.015	0.545	0.268	0.172	0.516
26	0.153	0.000	0.749	0.204	0.047	0.558
27	0.214	0.083	0.686	0.206	0.025	0.541
28	0.165	0.000	0.666	0.287	0.047	0.591
29	0.169	0.000	0.584	0.415	0.000	0.666
30	0.100	0.000	0.600	0.352	0.047	0.617
31	0.099	0.054	0.605	0.235	0.106	0.522
32	0.236	0.000	0.672	0.327	0.000	0.631
33	0.318	0.000	0.693	0.281	0.026	0.599

Sub-catchment	Area (km^2)	UL	UM	UH	UR	I
34	0.175	0.000	0.678	0.262	0.061	0.574
35	0.267	0.065	0.373	0.194	0.368	0.371
36	0.408	0.528	0.277	0.104	0.090	0.312
37	0.431	0.147	0.609	0.214	0.031	0.519
38	0.161	0.003	0.591	0.292	0.114	0.559
39	0.438	0.025	0.537	0.266	0.171	0.512
40	0.632	0.334	0.179	0.110	0.377	0.238
41	0.393	0.410	0.285	0.091	0.214	0.286
42	0.390	0.271	0.504	0.127	0.097	0.407
43	0.167	0.789	0.022	0.077	0.113	0.198

URBS Design Event Sub-catchment Parameters

Sub-catchment	Area (km^2)	UL	UM	UH	UR	I
1	0.437	0.000	0.007	0.012	0.981	0.015
2	0.253	0.000	0.626	0.226	0.148	0.516
3	0.182	0.000	0.621	0.219	0.160	0.508
4	0.339	0.000	0.010	0.019	0.971	0.022
5	0.603	0.000	0.003	0.009	0.987	0.010
6	0.210	0.000	0.583	0.243	0.174	0.510
7	0.206	0.000	0.259	0.123	0.618	0.240
8	0.392	0.000	0.475	0.208	0.317	0.424
9	0.283	0.000	0.642	0.262	0.096	0.557
10	0.178	0.000	0.595	0.359	0.046	0.621
11	0.159	0.000	0.697	0.301	0.002	0.619
12	0.107	0.000	0.630	0.371	0.000	0.648
13	0.141	0.000	0.495	0.446	0.059	0.649
14	0.183	0.000	0.585	0.345	0.070	0.603
15	0.204	0.000	0.353	0.559	0.088	0.680
16	0.120	0.000	0.054	0.079	0.867	0.098
17	0.288	0.000	0.347	0.138	0.514	0.298
18	0.062	0.000	0.401	0.500	0.099	0.650
19	0.406	0.000	0.488	0.155	0.356	0.384
20	0.084	0.000	0.622	0.333	0.045	0.611
21	0.122	0.000	0.678	0.298	0.024	0.607
22	0.134	0.000	0.493	0.375	0.132	0.584
23	0.108	0.000	0.577	0.288	0.135	0.548
24	0.083	0.000	0.408	0.592	0.000	0.737
25	0.327	0.015	0.545	0.268	0.172	0.516
26	0.153	0.000	0.570	0.382	0.048	0.629
27	0.214	0.000	0.730	0.245	0.025	0.586
28	0.165	0.000	0.666	0.287	0.047	0.591
29	0.169	0.000	0.580	0.420	0.000	0.668
30	0.100	0.000	0.600	0.352	0.047	0.617
31	0.099	0.145	0.401	0.348	0.106	0.536
32	0.236	0.000	0.672	0.327	0.000	0.631

Sub-catchment	Area (km^2)	UL	UM	UH	UR	I
33	0.318	0.000	0.598	0.377	0.025	0.638
34	0.175	0.000	0.678	0.262	0.061	0.574
35	0.267	0.358	0.373	0.194	0.075	0.415
36	0.408	0.006	0.471	0.433	0.090	0.626
37	0.431	0.000	0.663	0.306	0.031	0.607
38	0.161	0.000	0.592	0.294	0.114	0.560
39	0.438	0.000	0.547	0.282	0.171	0.527
40	0.632	0.232	0.401	0.184	0.183	0.401
41	0.393	0.160	0.535	0.178	0.127	0.452
42	0.390	0.066	0.539	0.336	0.058	0.582
43	0.167	0.729	0.081	0.175	0.015	0.307

Gubberley Detention Basin: Stage - Storage - Discharge Relationship

Stage (mAHD)	Area (m^2)	Storage (m^3)	Discharge (m^3/s)	
			Low-flow Fully Open	Low-flow Fully Blocked
23.00	0	0	0.00	0.00
23.50	5	2	0.37	0.00
23.75	34	7	0.66	0.00
24.00	65	19	1.07	0.00
24.25	113	42	1.51	0.00
24.50	279	91	1.73	0.00
24.75	511	189	1.93	0.00
25.00	760	348	2.11	0.00
25.25	1127	584	2.27	0.00
25.50	1602	925	2.43	0.00
25.75	2082	1386	2.57	0.00
26.00	2475	1955	2.71	0.00
26.25	2819	2617	2.84	0.00
26.50	3153	3364	2.96	0.00
26.75	3583	4206	3.08	0.00
27.00	4219	5181	3.20	0.00
27.25	4802	6309	3.31	0.00
27.50	5414	7586	3.41	0.00
27.66	5833	8504	3.48	0.00
27.75	6069	9021	4.11	0.59
28.00	6940	10647	9.17	5.55
28.25	7641	12470	18.95	15.03
28.50	8368	14471	39.15	35.13
28.75	9197	16666	67.08	62.98
29.00	10254	19098	101.98	97.79
29.25	11393	21804	142.94	138.74
29.50	12543	24796	189.71	185.51
29.75	13572	28060	242.70	238.50
30.00	14497	31569	301.06	296.86

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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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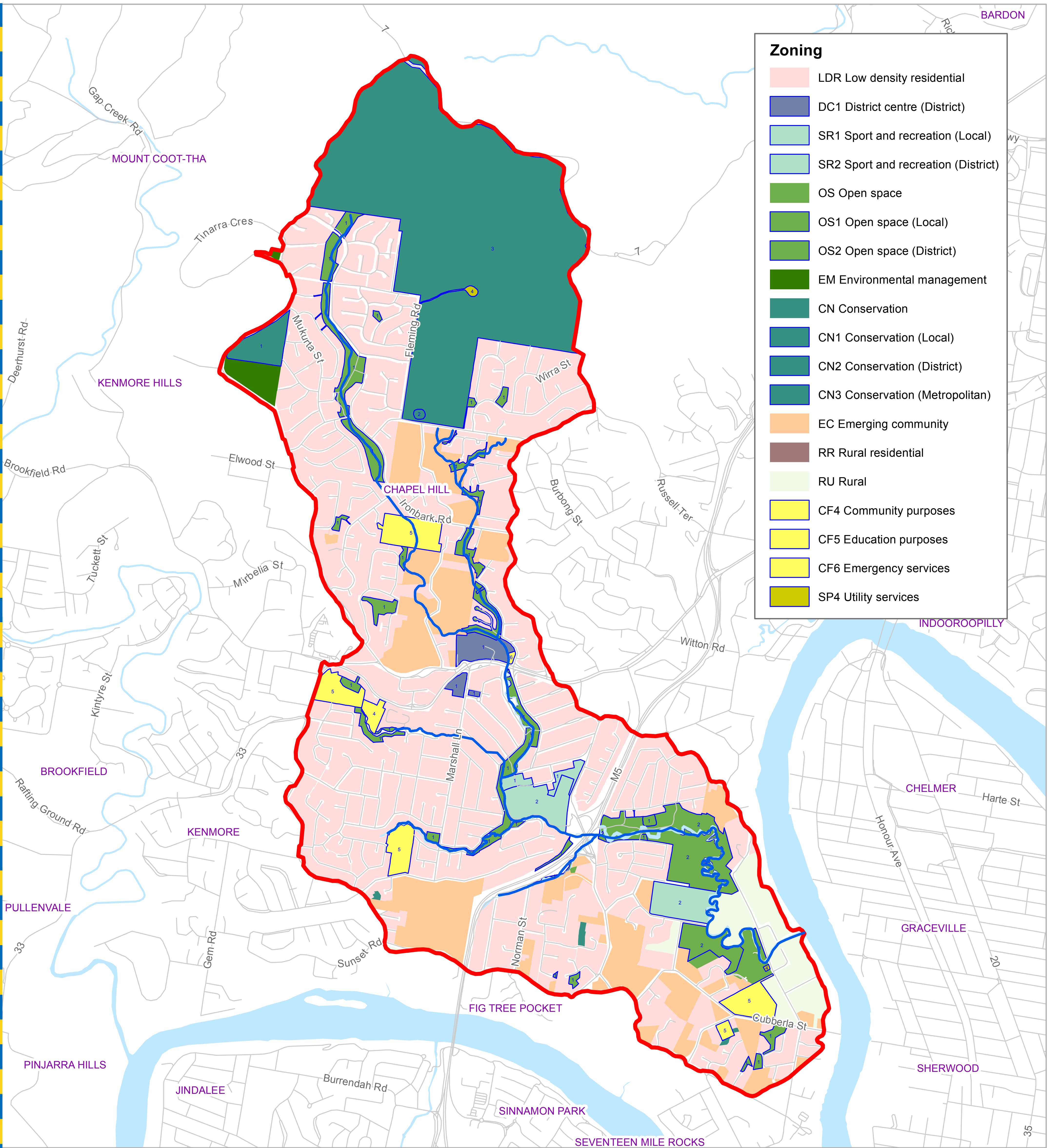

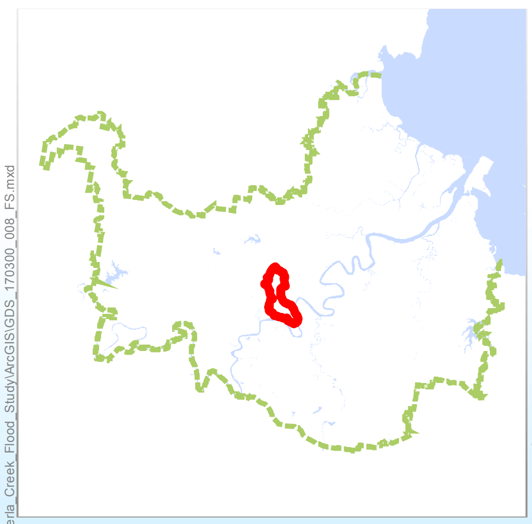

0 125 250 375
Metres

N W E S

Legend

- Creek Centrelines
- Catchment Area
- Streets

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Prepared by (Insert Consultant Name here) for:
Brisbane City Council
City Projects Office
GPO Box 1434
Brisbane Qld 4001

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Prepared : 089958
Checked : NC
Revision : 0
Publication Date : 20 Jan 2017
Project Number : 170300

File : G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\ArcGIS\GDS_170300_008_FS.mxd

Cubberla Creek Flood Study

Figure C - 1: 2014 City Plan Zones

GDS - 170300 - 008

Figure C-2: 2015 Aerial Photo

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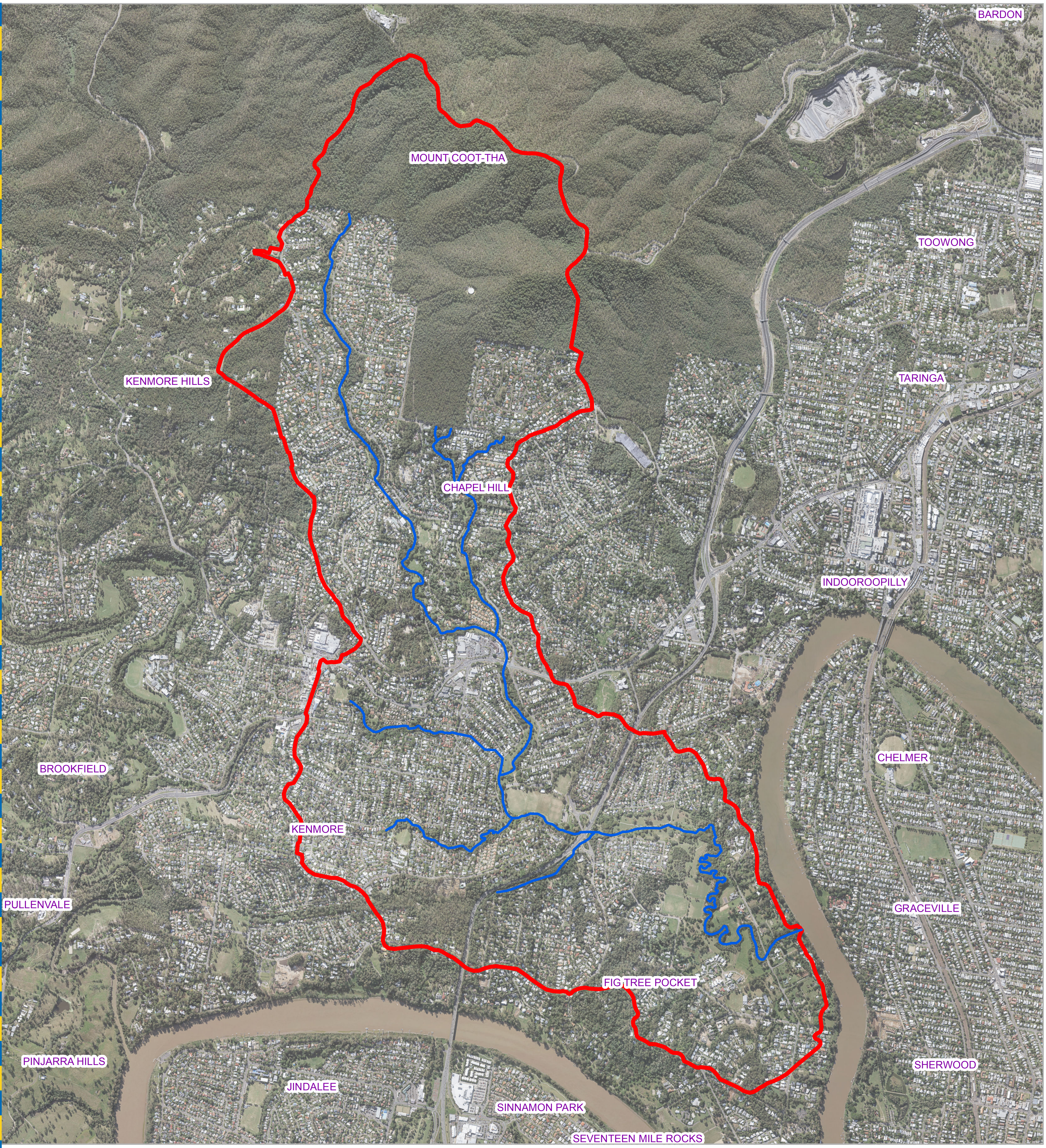

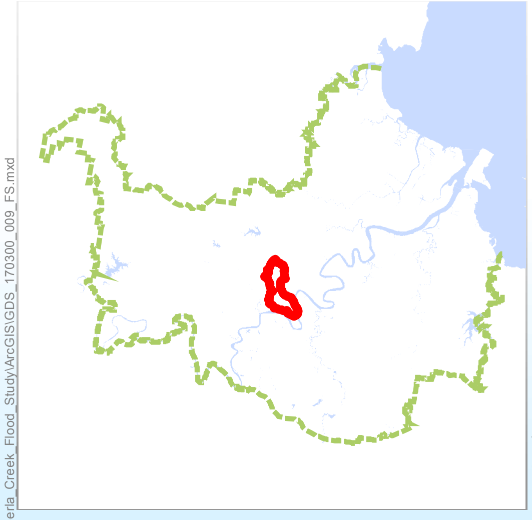

Legend

- Creek Centrelines
- Catchment Area

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Prepared by (Insert Consultant Name here) for:
Brisbane City Council
City Projects Office
GPO Box 1434
Brisbane Qld 4001

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0 125 250 375
Metres

N W E S

Prepared : 089958
Checked : NC
Revision : 0
Publication Date : 27 Jan 2017
Project Number : 170300

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Cubberla Creek Flood Study

Figure C - 2: 2015 Aerial Photo

GDS - 170300 - 009

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: URBS – TUFLOW Comparative Plots

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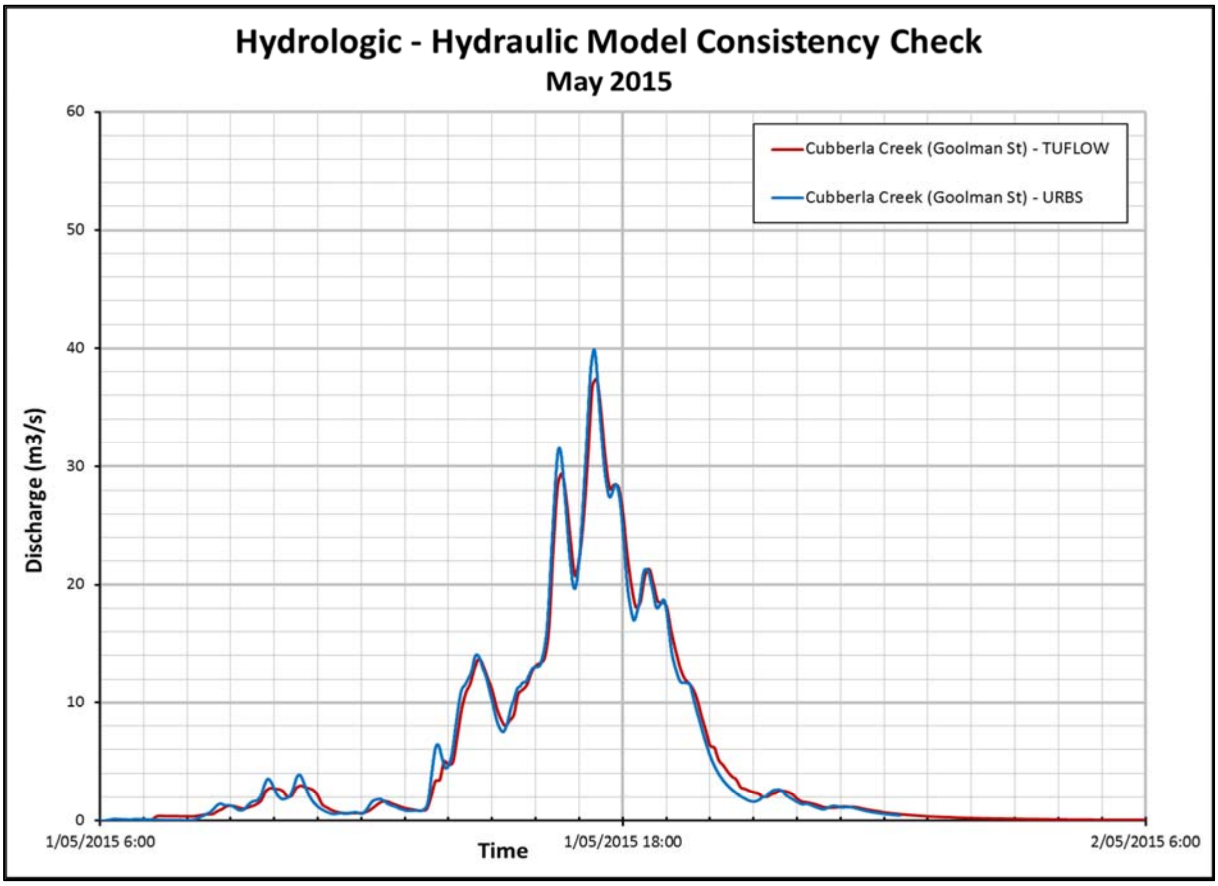
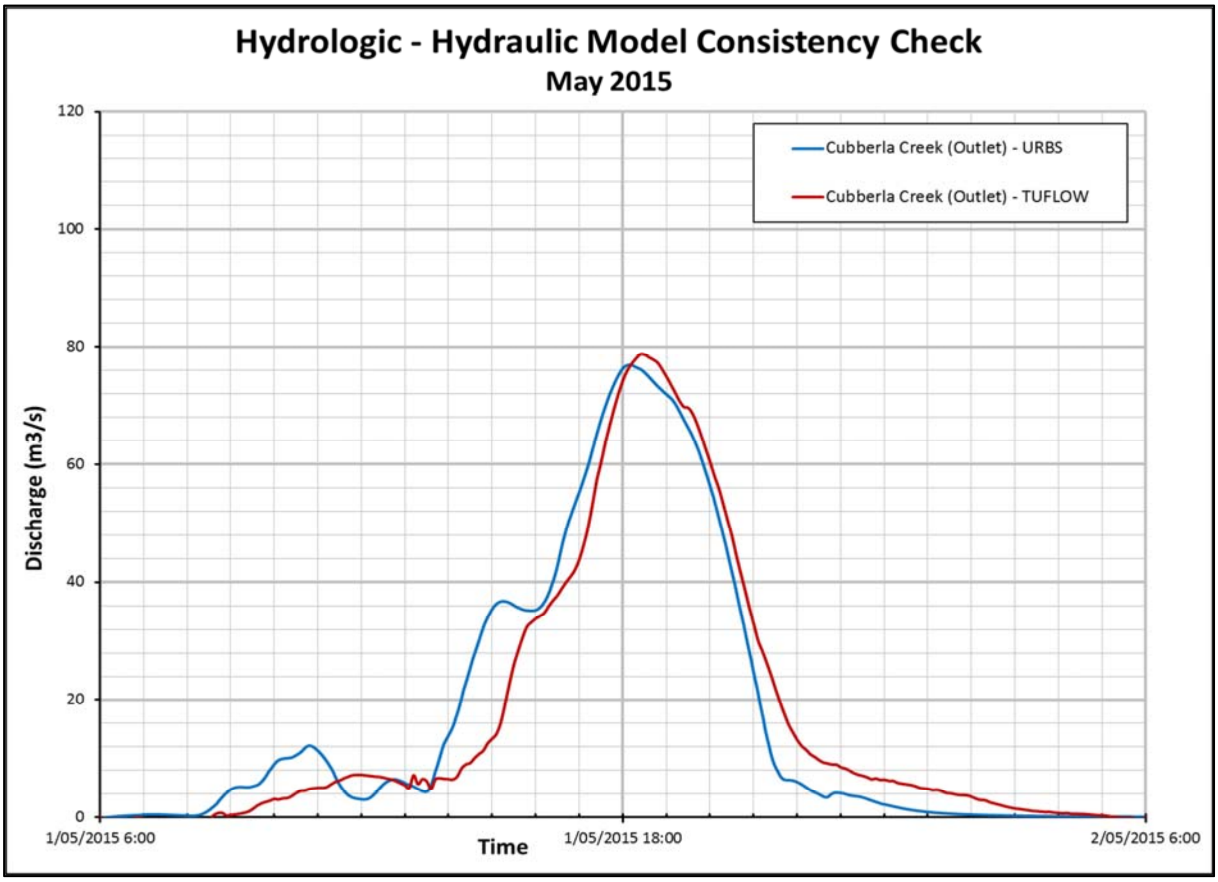
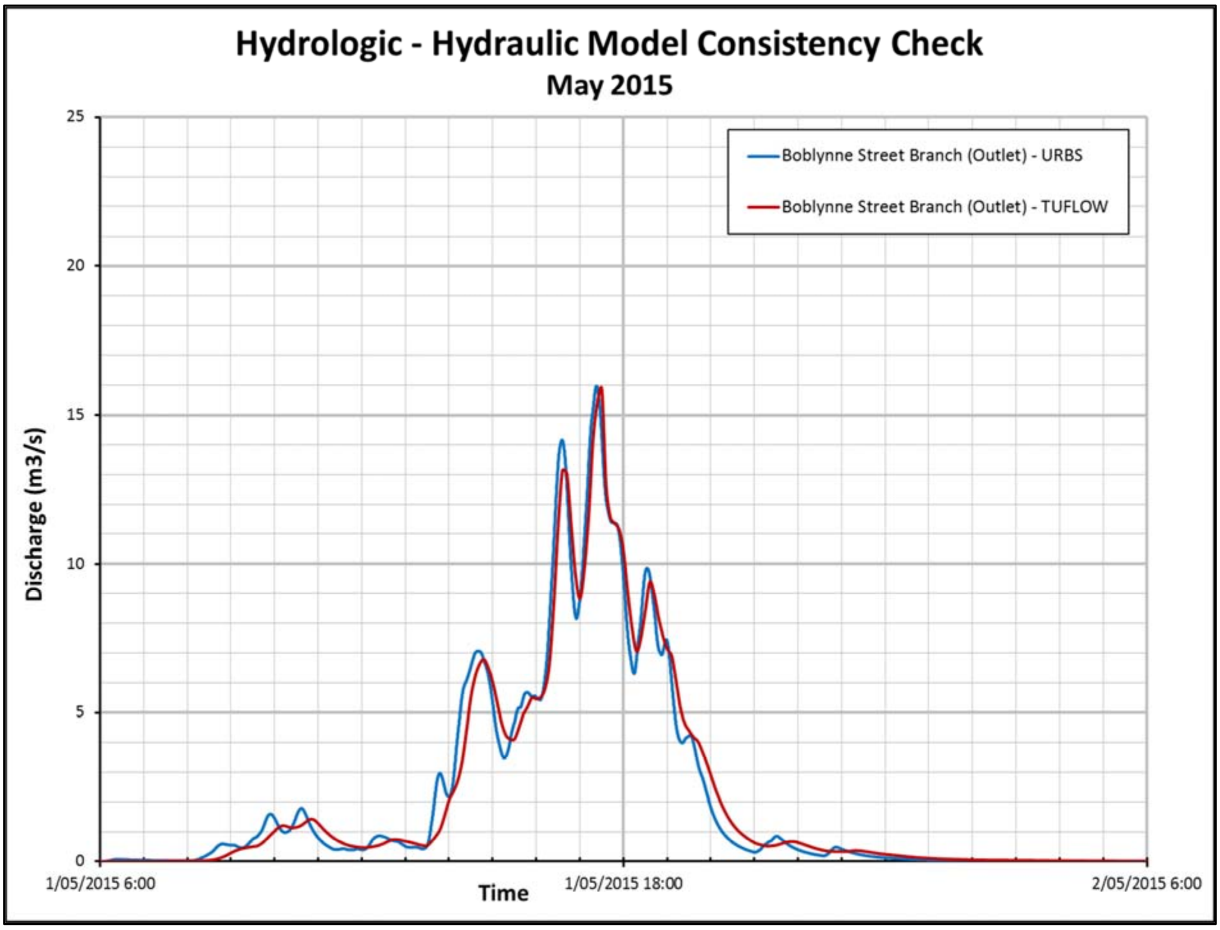
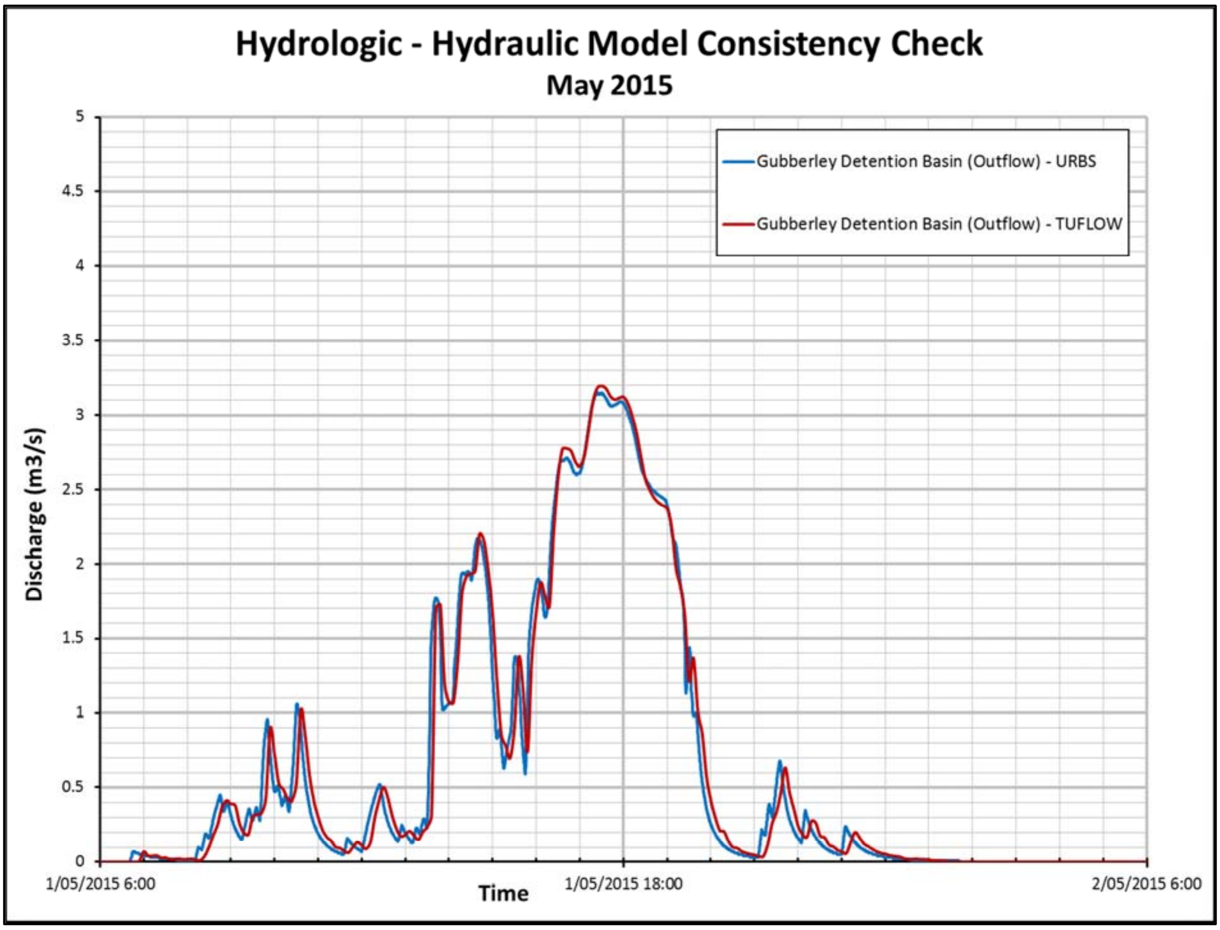
Historical Events



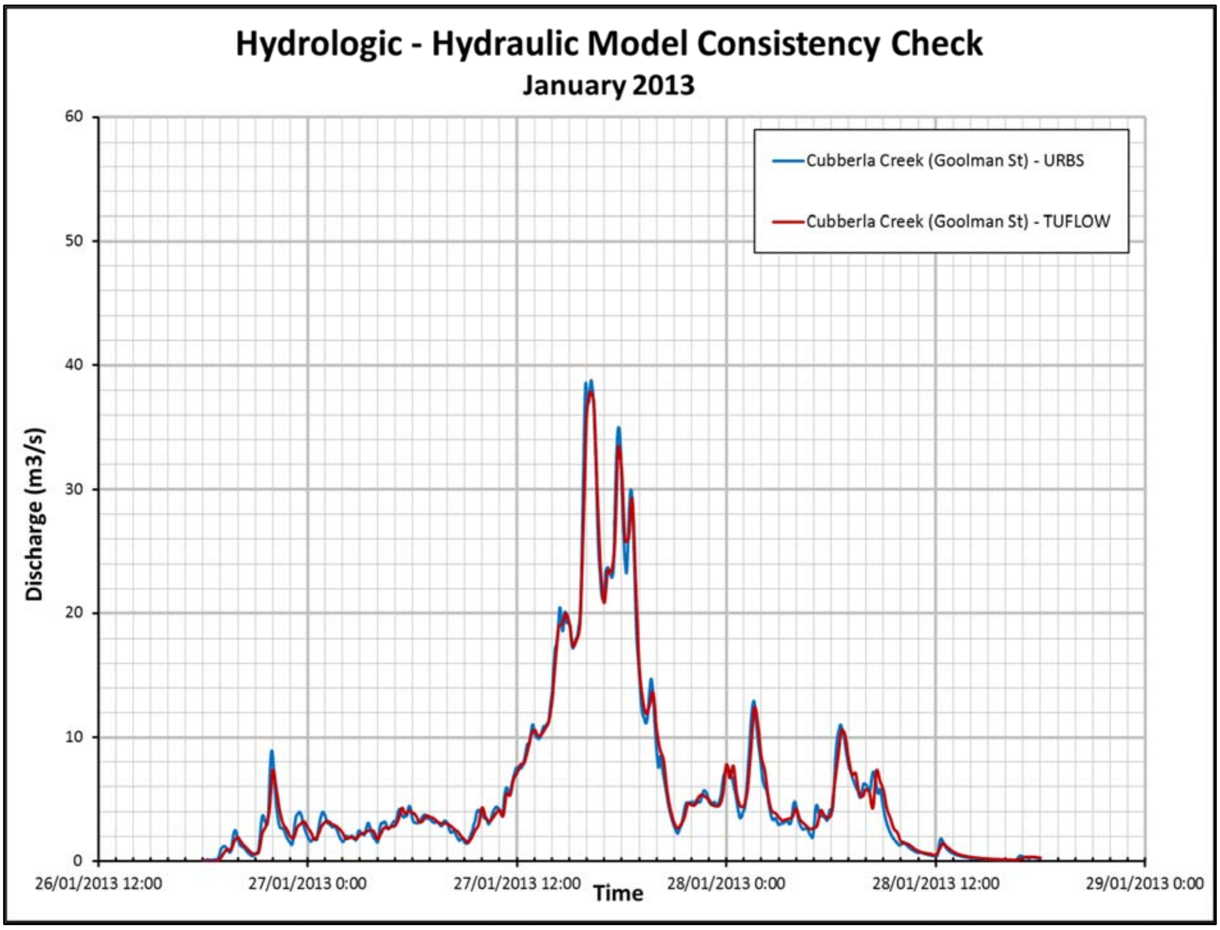




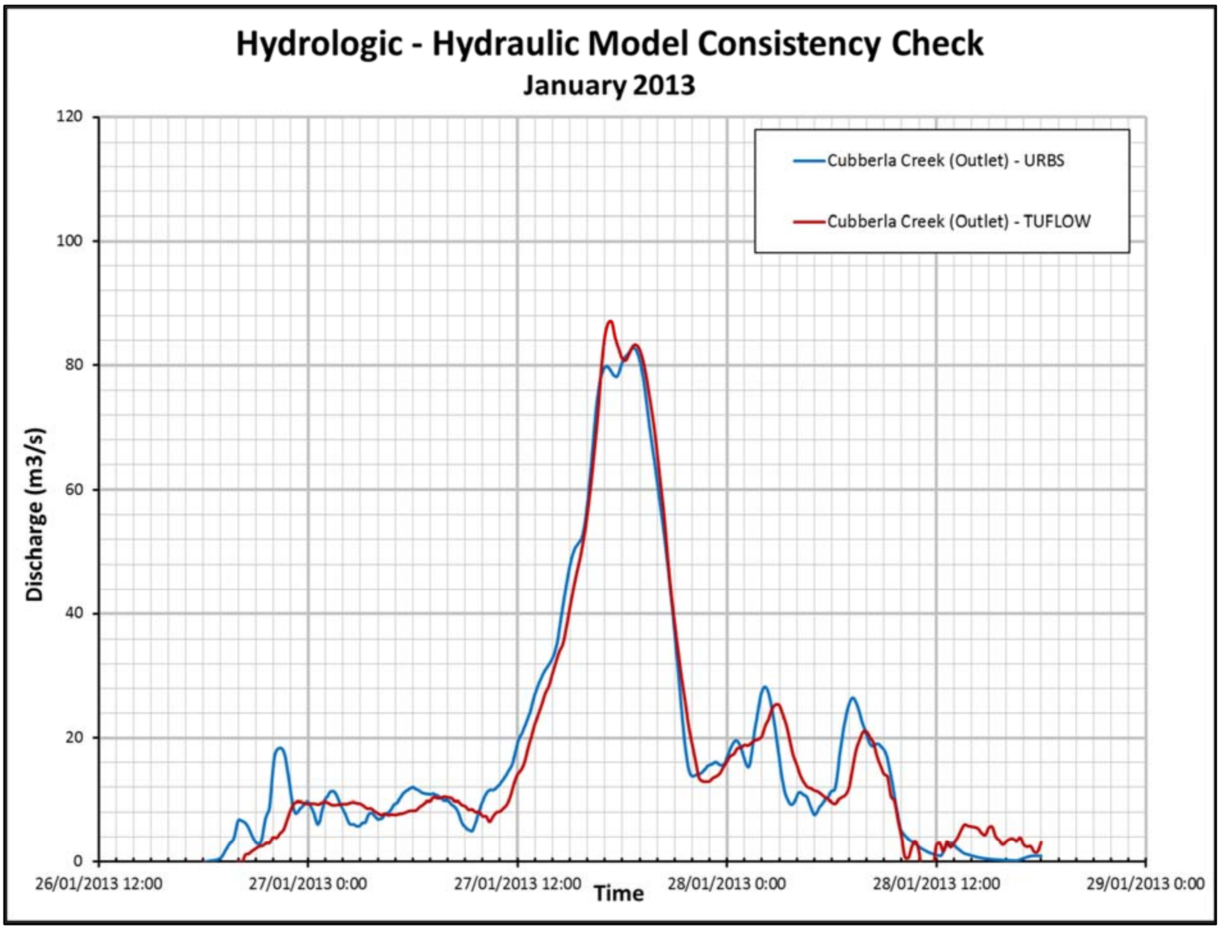
Hydrologic - Hydraulic Model Consistency Check
May 2015
Boblynne Street Branch (Outlet) - URBS
Boblynne Street Branch (Outlet) - TUFLOW
Discharge (m3/s)
25
20
15
10
5
0
1/05/2015 6:00
1/05/2015 18:00
2/05/2015 6:00
Time



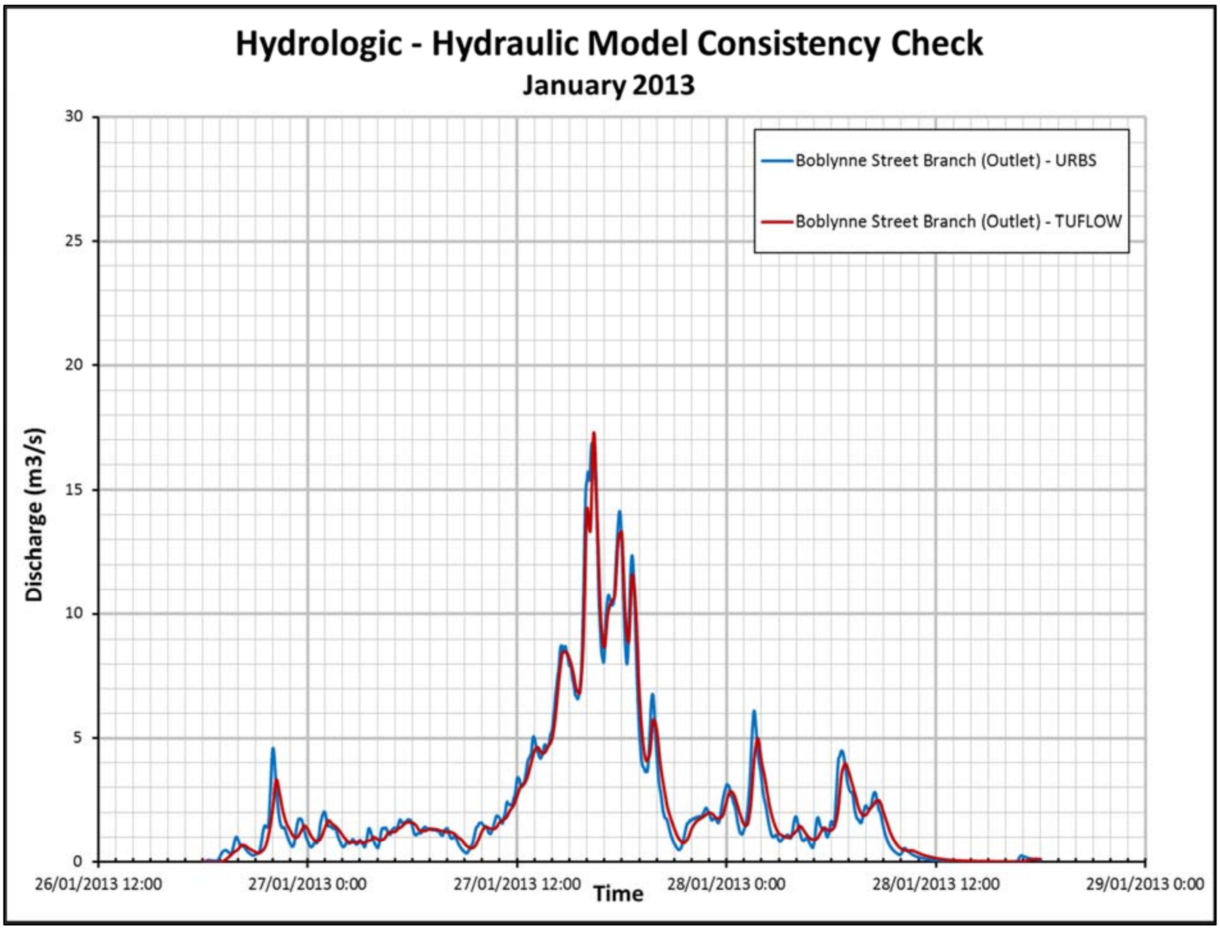
Hydrologic - Hydraulic Model Consistency Check
May 2015
Gubberley Detention Basin (Outflow) - URBS
Gubberley Detention Basin (Outflow) - TUFLOW
Discharge (m3/s)
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
1/05/2015 6:00
1/05/2015 18:00
2/05/2015 6:00
Time

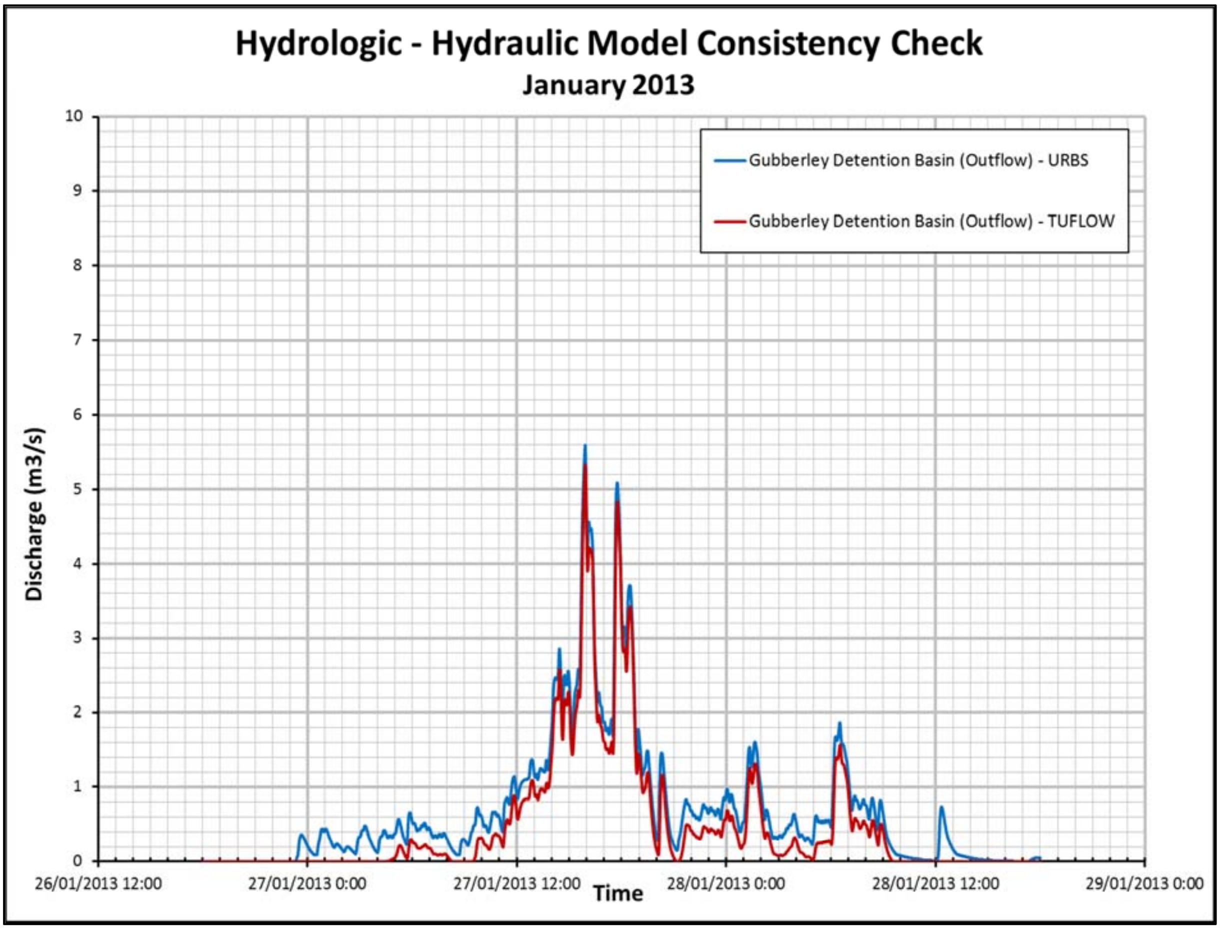


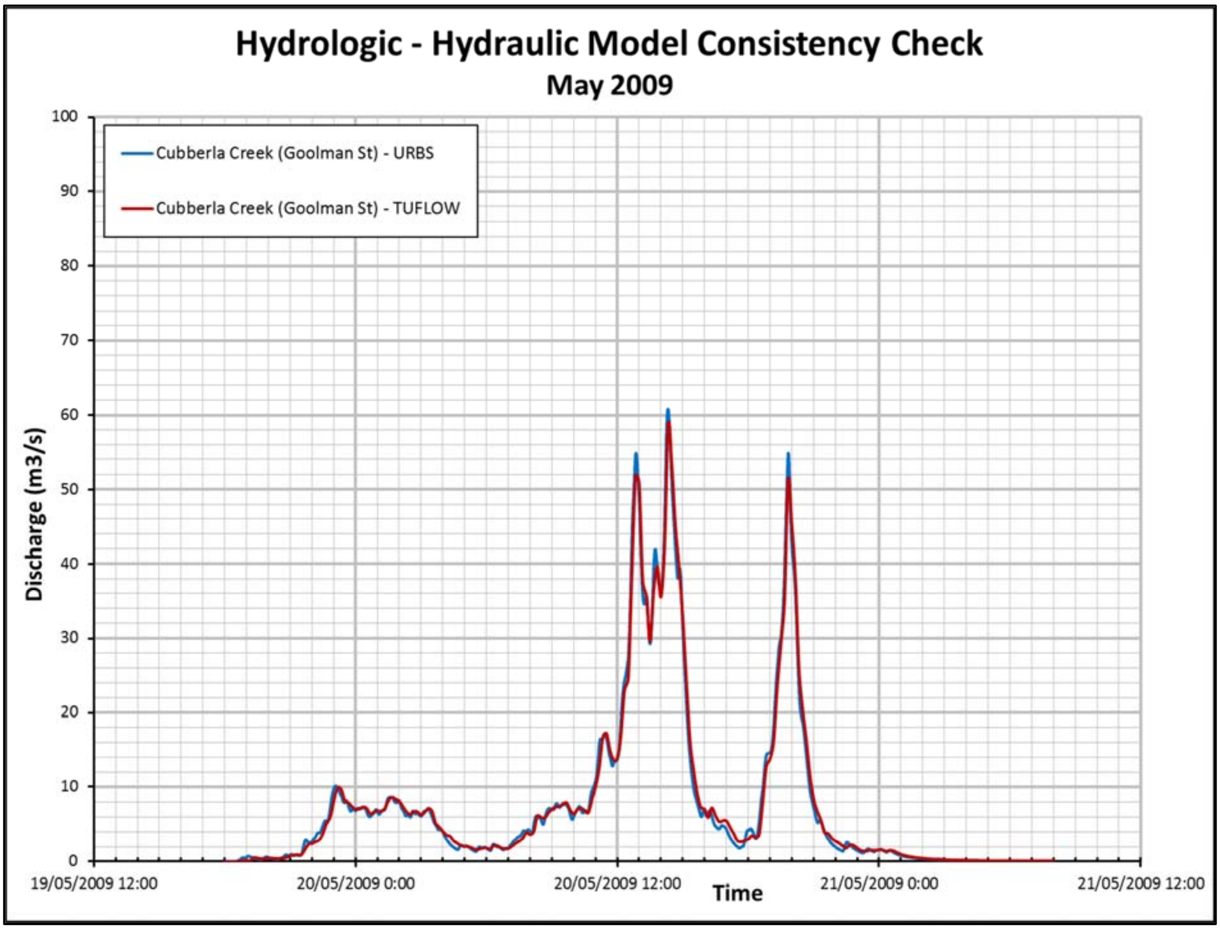
Hydrologic - Hydraulic Model Consistency Check
January 2013
Cubberla Creek (Goolman St) - URBS
Cubberla Creek (Goolman St) - TUFLOW
Discharge (m3/s)
0
10
20
30
40
50
60
26/01/2013 12:00
27/01/2013 0:00
27/01/2013 12:00
28/01/2013 0:00
28/01/2013 12:00
29/01/2013 0:00
Time



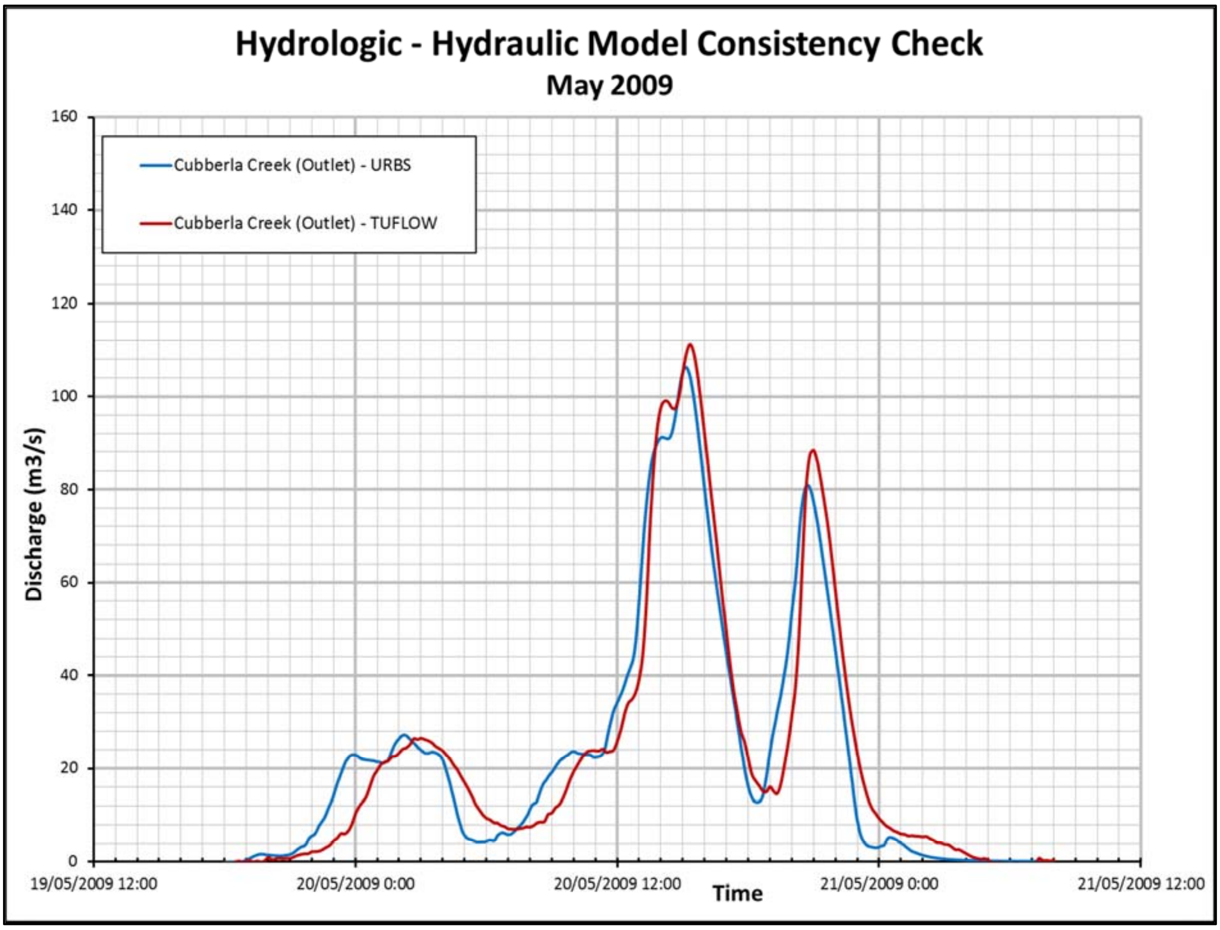
Hydrologic - Hydraulic Model Consistency Check
January 2013
Cubberla Creek (Outlet) - URBS
Cubberla Creek (Outlet) - TUFLOW
Discharge (m3/s)
0
20
40
60
80
100
120
26/01/2013 12:00
27/01/2013 0:00
27/01/2013 12:00
28/01/2013 0:00
28/01/2013 12:00
29/01/2013 0:00
Time


Hydrologic - Hydraulic Model Consistency Check
January 2013
Boblynne Street Branch (Outlet) - URBS
Boblynne Street Branch (Outlet) - TUFLOW
Discharge (m3/s)
30
25
20
15
10
5
0
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

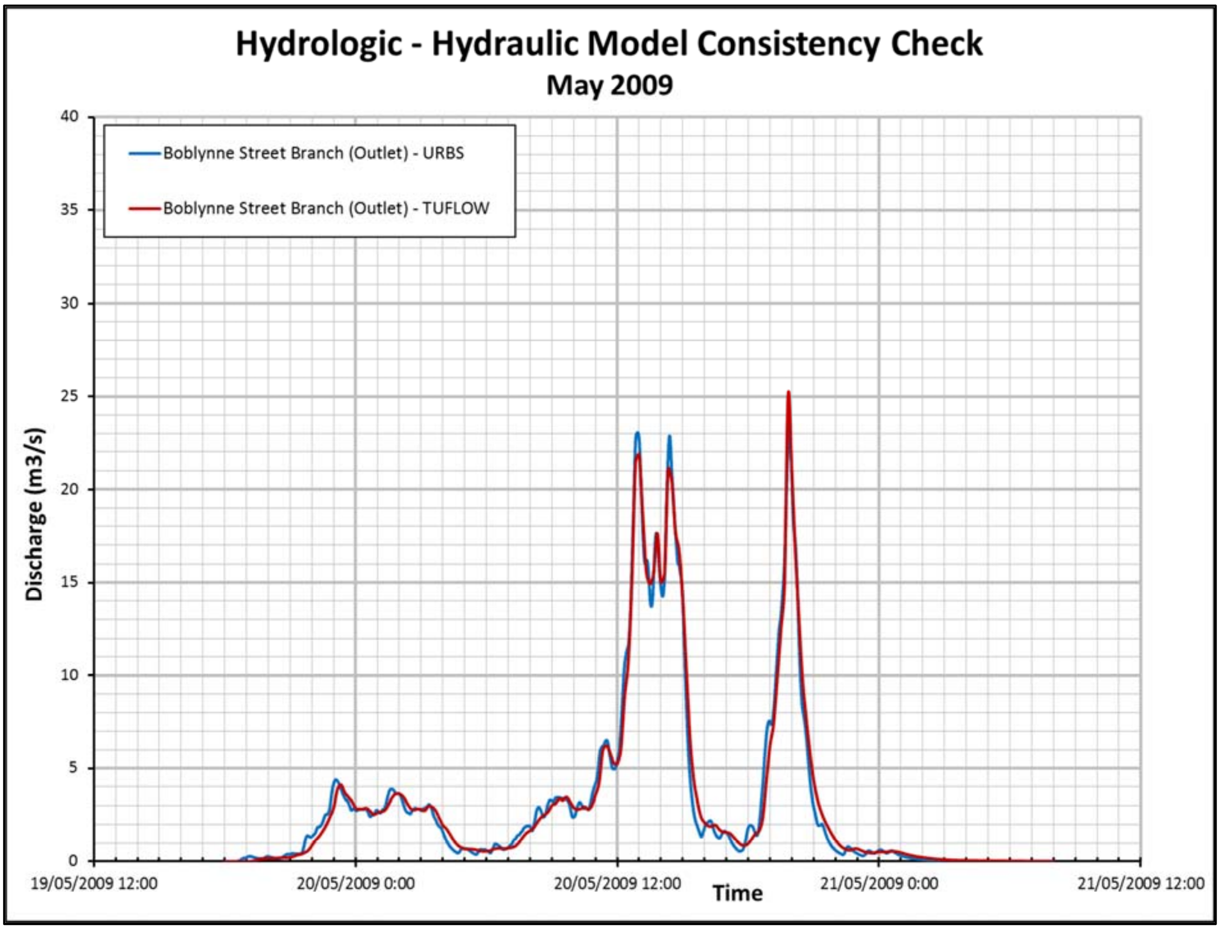

Hydrologic - Hydraulic Model Consistency Check
January 2013
Gubberley Detention Basin (Outflow) - URBS
Gubberley Detention Basin (Outflow) - TUFLOW
Discharge (m3/s)
10
9
8
7
6
5
4
3
2
1
0
26/01/2013 12:00
27/01/2013 0:00
27/01/2013 12:00
28/01/2013 0:00
28/01/2013 12:00
29/01/2013 0:00
Time

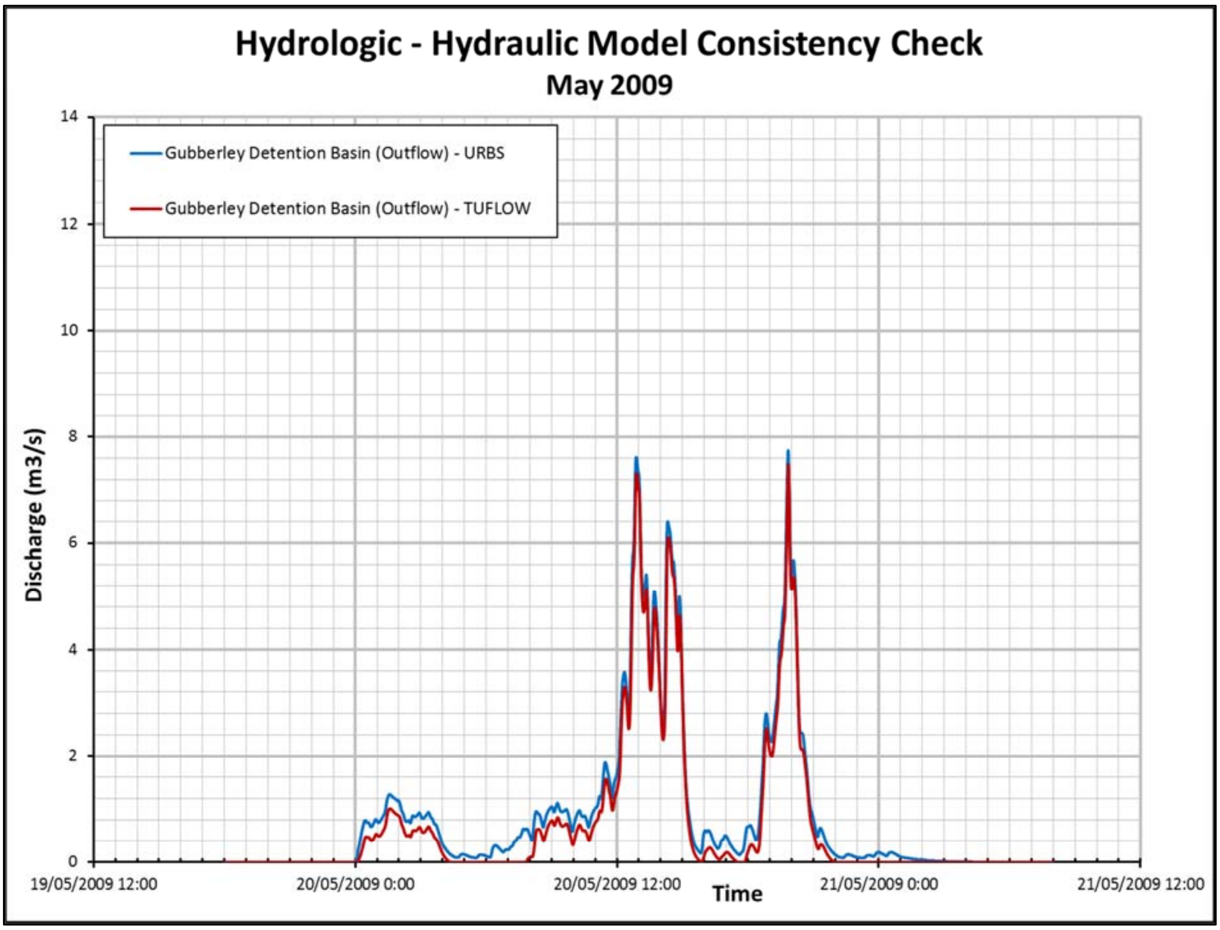


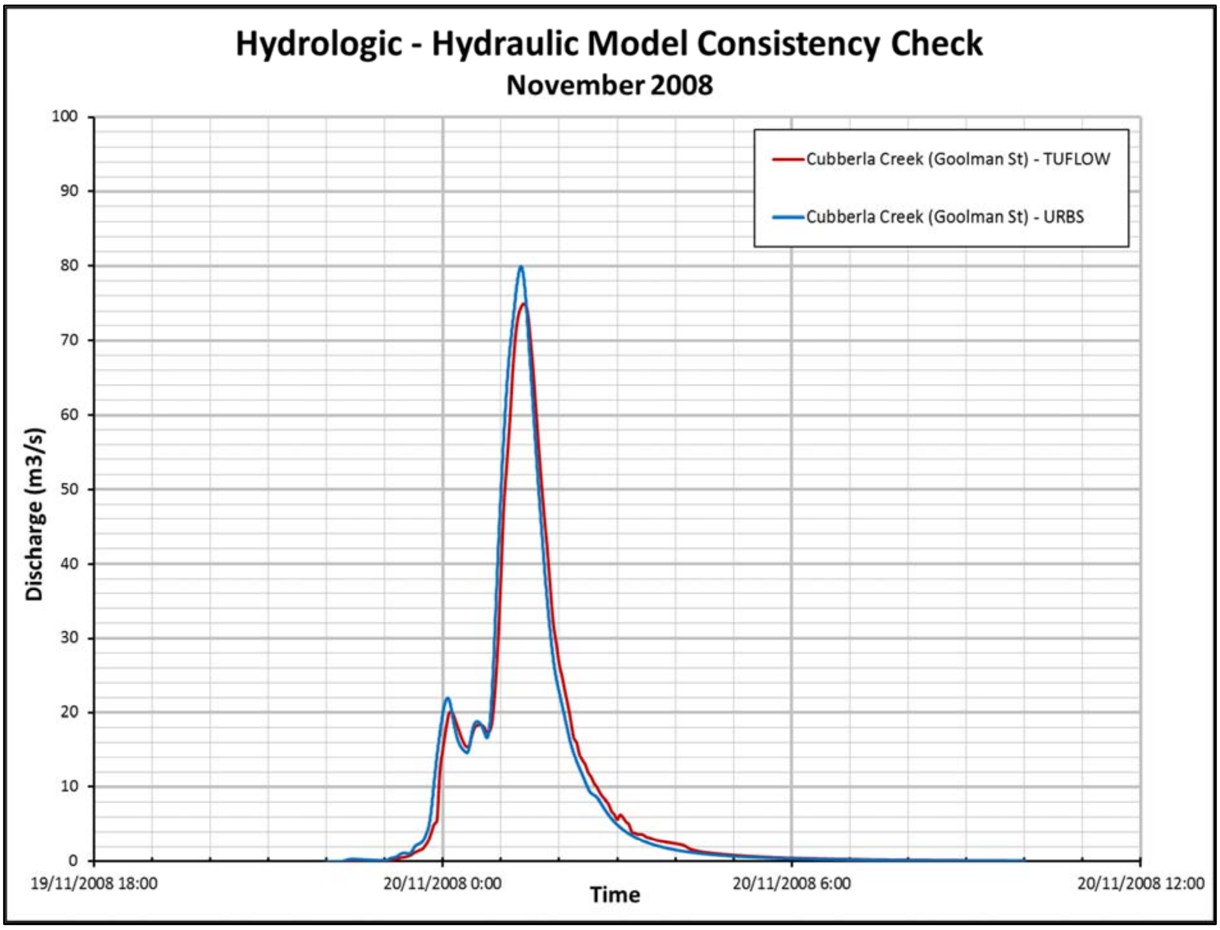
Hydrologic - Hydraulic Model Consistency Check
May 2009
Cubberla Creek (Goolman St) - URBS
Cubberla Creek (Goolman St) - TUFLOW
Discharge (m3/s)
Time
100
90
80
70
60
50
40
30
20
10
0
19/05/2009 12:00
20/05/2009 0:00
20/05/2009 12:00
21/05/2009 0:00
21/05/2009 12:00

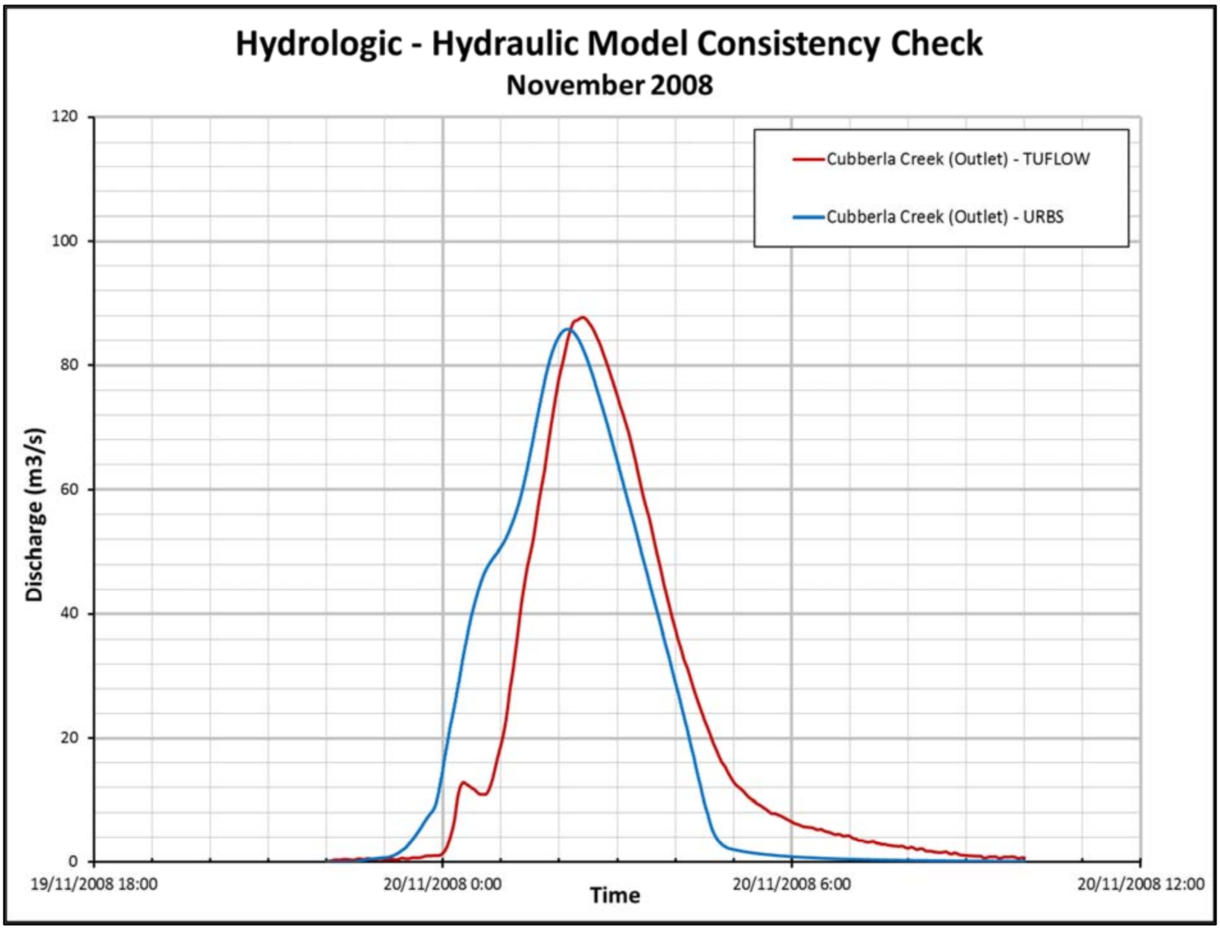


Hydrologic - Hydraulic Model Consistency Check
May 2009
Cubberla Creek (Outlet) - URBS
Cubberla Creek (Outlet) - TUFLOW
Discharge (m3/s)
Time
160
140
120
100
80
60
40
20
0
19/05/2009 12:00
20/05/2009 0:00
20/05/2009 12:00
21/05/2009 0:00
21/05/2009 12:00


Hydrologic - Hydraulic Model Consistency Check
May 2009
Boblynne Street Branch (Outlet) - URBS
Boblynne Street Branch (Outlet) - TUFLOW
Discharge (m3/s)
Time
0
5
10
15
20
25
30
35
40
19/05/2009 12:00
20/05/2009 0:00
20/05/2009 12:00
21/05/2009 0:00
21/05/2009 12:00


Hydrologic - Hydraulic Model Consistency Check
May 2009
Gubberley Detention Basin (Outflow) - URBS
Gubberley Detention Basin (Outflow) - TUFLOW
Discharge (m3/s)
Time
0
2
4
6
8
10
12
14
19/05/2009 12:00
20/05/2009 0:00
20/05/2009 12:00
21/05/2009 0:00
21/05/2009 12:00


Hydrologic - Hydraulic Model Consistency Check
November 2008
Cubberla Creek (Goolman St) - TUFLOW
Cubberla Creek (Goolman St) - URBS
Discharge (m3/s)
Time
19/11/2008 18:00
20/11/2008 0:00
20/11/2008 6:00
20/11/2008 12:00


Hydrologic - Hydraulic Model Consistency Check
November 2008
Cubberla Creek (Outlet) - TUFLOW
Cubberla Creek (Outlet) - URBS
Discharge (m3/s)
Time
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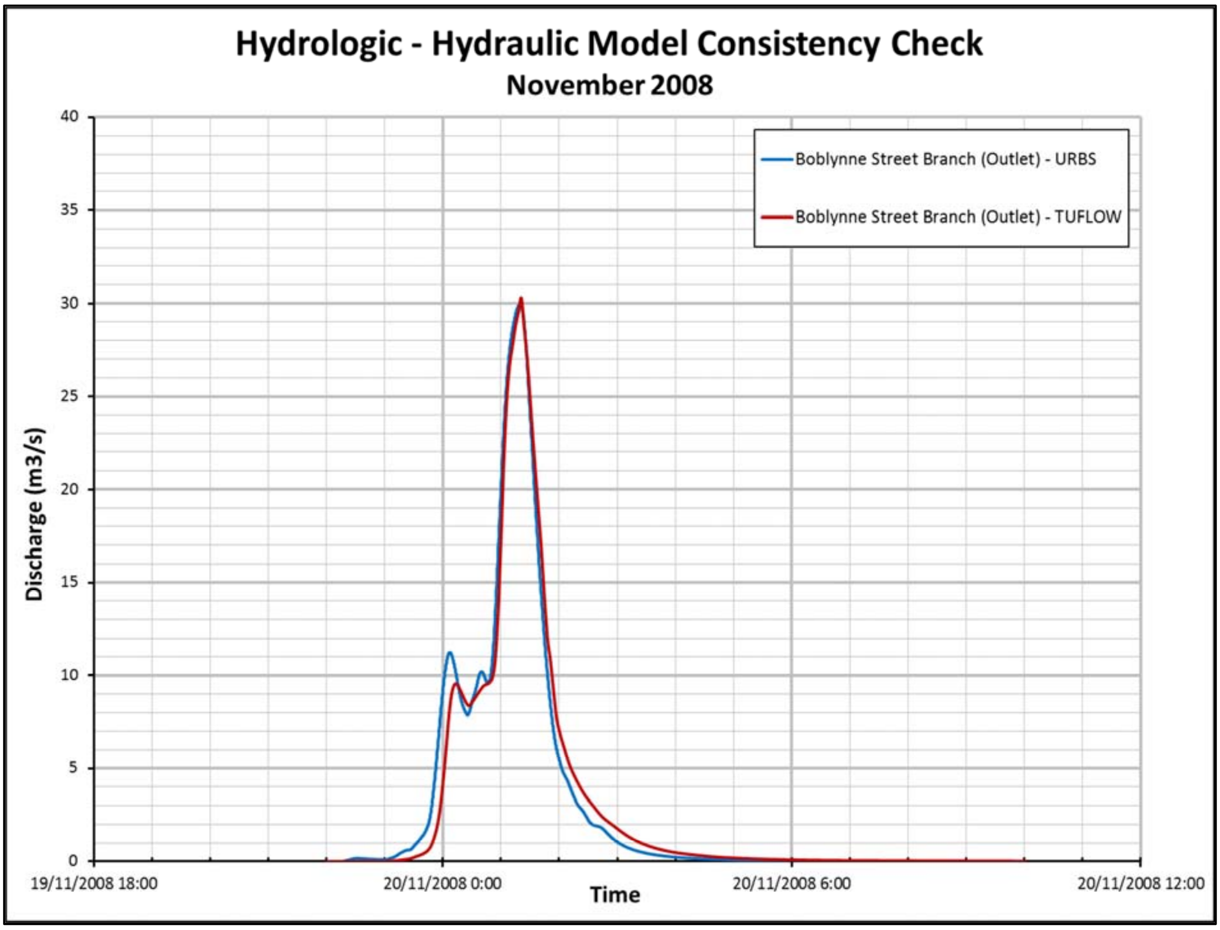
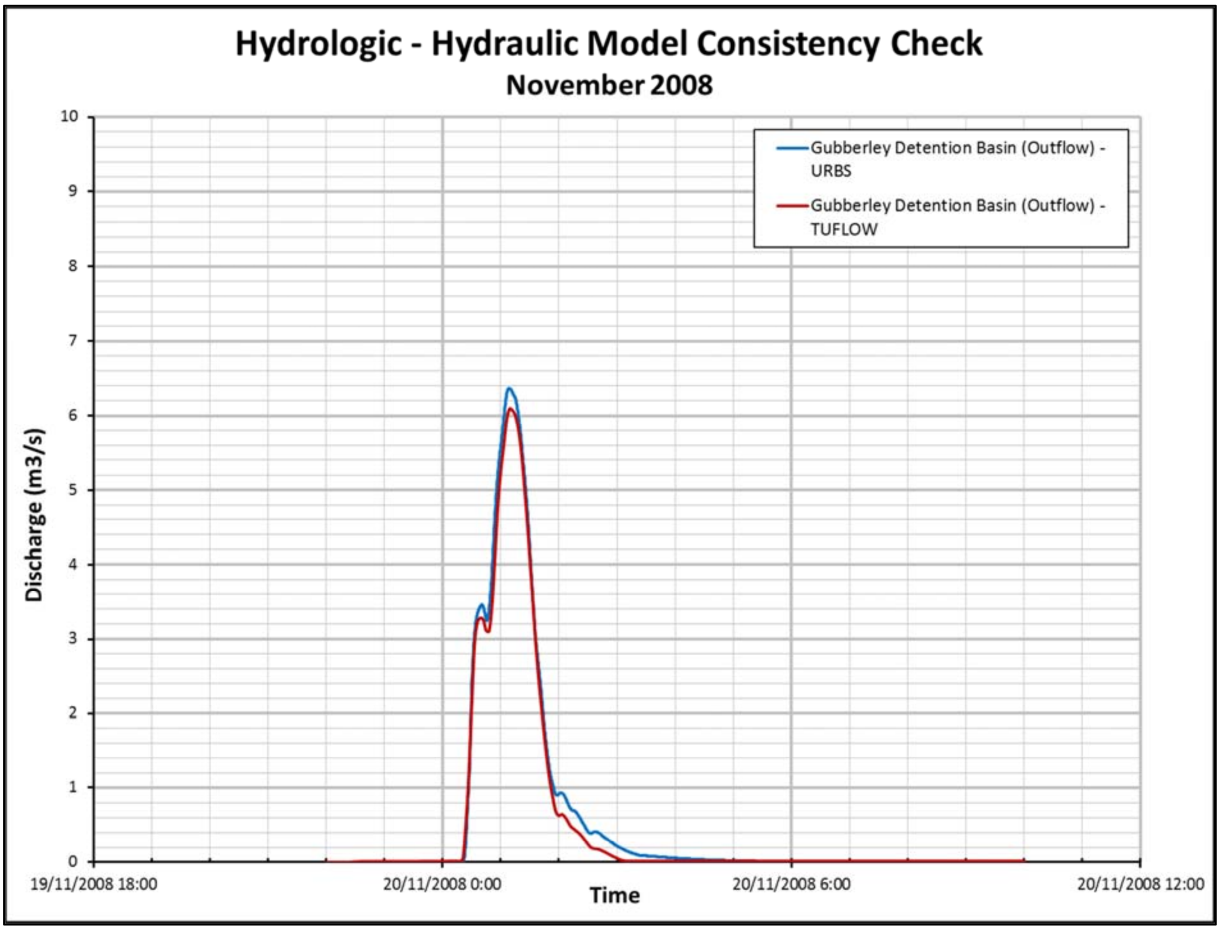
20/11/2008 0:00
20/11/2008 6:00
20/11/2008 12:00


Hydrologic - Hydraulic Model Consistency Check
November 2008
Boblynne Street Branch (Outlet) - URBS
Boblynne Street Branch (Outlet) - TUFLOW
Discharge (m3/s)
Time
19/11/2008 18:00
20/11/2008 0:00
20/11/2008 6:00
20/11/2008 12:00


Hydrologic - Hydraulic Model Consistency Check
November 2008
Gubberley Detention Basin (Outflow) - URBS
Gubberley Detention Basin (Outflow) - TUFLOW
Discharge (m3/s)
Time
19/11/2008 18:00
20/11/2008 0:00
20/11/2008 6:00
20/11/2008 12:00

Appendix E: 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)
Cubberla Creek						
0	1.21	1.21	1.21	1.21	1.21	1.21
100	1.41	1.54	1.62	1.73	1.84	1.95
200	1.72	1.98	2.13	2.31	2.52	2.72
Structure S1 – Jesmond Road Bridge						
300	2.50	2.94	3.16	3.41	3.69	3.93
400	3.52	4.12	4.43	4.75	5.08	5.30
500	3.75	4.27	4.56	4.88	5.19	5.39
600	4.03	4.42	4.66	4.93	5.22	5.42
700	4.07	4.44	4.67	4.93	5.23	5.42
800	4.07	4.45	4.67	4.94	5.23	5.42
900	4.07	4.45	4.67	4.94	5.23	5.42
1000	4.11	4.47	4.69	4.95	5.25	5.44
1100	4.16	4.51	4.73	4.98	5.27	5.46
1200	4.20	4.54	4.76	5.00	5.29	5.48
1300	4.23	4.56	4.77	5.02	5.30	5.49
1400	4.32	4.60	4.79	5.03	5.32	5.50
1500	4.58	4.73	4.86	5.06	5.33	5.51
1600	4.89	5.02	5.07	5.18	5.41	5.57
1700	5.72	5.92	5.96	6.02	6.11	6.18
1800	6.06	6.27	6.31	6.38	6.47	6.53
1900	6.48	6.70	6.75	6.82	6.92	6.98
2000	6.73	6.94	6.99	7.07	7.17	7.24
2100	6.78	7.00	7.05	7.13	7.23	7.30
2200	7.03	7.23	7.27	7.34	7.43	7.50
2300	7.45	7.56	7.59	7.64	7.71	7.77
Structure S2 – Dobell Street Footbridge						
2400	8.17	8.27	8.30	8.34	8.41	8.47
2500	8.71	8.81	8.83	8.88	8.94	8.99
2600	9.32	9.46	9.47	9.52	9.58	9.63
2690	10.02	10.32	10.32	10.41	10.53	10.61
Structure S3 – Western Freeway Bridge						
2800	10.68	11.17	11.60	11.96	12.41	12.72

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)
2900	10.99	11.33	11.70	12.02	12.45	12.75
3000	11.31	11.51	11.75	12.03	12.44	12.74
Structure S4 – Garaboo Street Footbridge						
3100	11.67	11.87	12.08	12.27	12.61	12.89
3200	12.04	12.22	12.35	12.48	12.72	12.94
Structure S5 – Akuna Street Footbridge						
3300	12.51	12.61	12.67	12.74	12.83	12.99
3400	13.06	13.20	13.27	13.35	13.45	13.54
3500	13.48	13.62	13.70	13.78	13.87	13.95
3600	13.81	13.95	14.01	14.09	14.17	14.25
3700	14.18	14.36	14.45	14.54	14.65	14.74
3800	14.52	14.69	14.79	14.89	15.00	15.09
Structure S6 – Henry Street Footbridge						
3900	15.15	15.33	15.44	15.55	15.68	15.79
4000	15.50	15.68	15.78	15.90	16.03	16.14
4100	15.91	16.10	16.21	16.34	16.48	16.60
4200	16.28	16.50	16.62	16.75	16.89	17.01
4300	17.34	17.63	17.73	17.87	18.01	18.13
Structures S7 and S8 – Moggill Road Culvert						
4415	19.65	20.50	20.72	21.19	21.66	21.96
4500	19.82	20.58	20.80	21.26	21.72	22.01
4600	20.04	20.75	20.92	21.35	21.79	22.07
4700	20.20	20.88	21.01	21.42	21.84	22.13
4800	20.92	21.23	21.35	21.64	21.99	22.24
4900	21.88	22.10	22.20	22.36	22.54	22.71
Structure S9 – Bulk Water Mains #1						
4990	22.45	22.81	23.05	23.45	23.93	24.15
Structure S10 – Tristania Road Culvert						
5100	24.32	24.64	24.78	24.93	25.10	25.24
5200	24.38	24.73	24.87	25.05	25.23	25.38
Structure S11 – 56 Tristania Road Access Bridge						
5300	24.53	24.84	24.99	25.16	25.33	25.48
Structure S12 – 70 Tristania Road Access Bridge						

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)
5400	24.96	25.15	25.26	25.39	25.54	25.67
5500	25.69	25.82	25.90	26.01	26.11	26.20
5600	26.69	26.86	26.97	27.10	27.21	27.27
Structure S13 – Chapel Hill State School Culvert						
5700	27.70	28.11	28.40	28.66	28.81	28.98
5800	28.00	28.37	28.59	28.81	28.95	29.09
5900	28.69	29.01	29.21	29.38	29.57	29.70
Structure S14 – Goolman Street Culvert						
6000	29.62	30.32	30.52	30.68	30.80	30.88
6100	30.80	31.03	31.14	31.27	31.38	31.48
6200	31.50	31.69	31.80	31.92	32.05	32.16
6300	33.17	33.37	33.48	33.59	33.72	33.82
6400	33.77	34.00	34.12	34.27	34.42	34.54
6500	35.16	35.36	35.47	35.61	35.75	35.86
Structure S18 – Dillingen Street Culvert						
6600	36.84	37.15	37.32	37.54	37.82	38.02
6700	38.70	38.84	38.91	38.98	39.06	39.12
6800	40.13	40.28	40.37	40.47	40.57	40.65
6900	41.68	41.86	41.97	42.09	42.20	42.29
7000	43.51	43.70	43.81	43.93	44.02	44.10
7100	45.29	45.45	45.55	45.65	45.74	45.81
7200	46.43	46.63	46.74	46.84	46.93	46.99
7300	48.20	48.25	48.27	48.30	48.36	48.40
7400	49.23	49.37	49.44	49.52	49.59	49.64
7500	50.92	51.16	51.30	51.66	51.83	51.90
7600	53.28	53.56	53.68	53.85	53.88	53.90
7700	55.86	55.92	55.96	55.97	56.01	56.05
Structure S19 – Greenford Street Culvert						
7800	58.26	58.37	58.45	58.82	59.39	59.93
7887	60.86	60.98	61.02	61.09	61.14	61.19
Tributary C						
0	9.85	10.12	10.12	10.21	10.32	10.39
Structure S27 – Fig Tree Pocket Road Culvert						

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)
85	11.34	11.56	11.68	11.74	11.80	12.49
Structures S28 and 29 – Western Freeway On and Off Ramp Culverts						
200	12.29	12.52	12.66	12.83	12.94	13.03
300	12.37	12.61	12.73	12.91	13.03	13.13
400	12.84	13.09	13.20	13.37	13.49	13.60
500	13.92	14.13	14.23	14.39	14.51	14.59
Structure S30 – Norman Street Footbridge						
600	15.37	15.53	15.63	15.75	15.83	15.94
700	17.03	17.23	17.34	17.46	17.55	17.65
732	17.58	17.77	17.88	18.00	18.09	18.19
Akuna Street Branch						
0	12.08	12.26	12.38	12.50	12.73	12.95
Structure S25 – Katunga Street Culvert						
100	13.07	13.21	13.27	13.35	13.41	13.48
200	14.50	14.76	14.87	14.99	15.09	15.19
300	15.98	16.16	16.26	16.38	16.48	16.58
400	17.49	17.68	17.77	17.89	17.97	18.07
500	19.42	19.65	19.77	19.90	19.99	20.11
600	23.90	24.10	24.19	24.24	24.32	24.39
Structure S26 – Marshall Lane Culvert						
700	24.21	24.50	24.62	24.77	24.87	24.97
800	25.26	25.38	25.45	25.59	25.70	25.82
900	25.97	26.12	26.19	26.29	26.36	26.45
1000	27.89	28.06	28.15	28.25	28.32	28.41
1050	28.99	29.18	29.29	29.42	29.51	29.61
Gubberley Creek						
0	13.47	13.61	13.69	13.77	13.86	13.94
100	13.72	13.91	14.00	14.11	14.22	14.32
200	15.10	15.27	15.39	15.50	15.64	15.73
300	15.42	15.64	15.78	15.92	16.08	16.20
400	N/R	16.94	17.03	17.12	17.20	17.26
Marshall Lane Piped Drainage						
500	17.33	18.58	18.72	18.84	18.96	19.08

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)
600	18.76	19.28	19.42	19.56	19.73	19.87
700	20.12	20.37	20.48	20.60	20.75	20.88
Structure S23 – Cedar Xing Culvert						
820	22.11	22.57	22.79	23.16	23.44	23.58
Structure S24 – Gubberley Creek Detention Basin						
910	27.90	28.07	28.12	28.20	28.28	28.35
1000	27.98	28.13	28.19	28.26	28.35	28.43
1100	29.10	29.13	29.18	29.21	29.23	29.26
1200	30.09	30.19	30.24	30.30	30.36	30.41
1252	30.96	31.06	31.12	31.19	31.23	31.28
Boblynne Street Branch						
0	19.94	20.64	20.86	21.30	21.76	22.04
Structure S20 – St. James Estate Access Culvert						
100	20.12	20.70	20.94	21.37	21.82	22.10
200	20.62	20.90	21.10	21.45	21.87	22.14
300	21.66	21.84	21.94	22.06	22.19	22.36
Structure S21 – Bulk Water Mains #2						
400	22.67	22.86	22.97	23.09	23.19	23.29
500	24.62	24.72	24.77	24.84	24.91	24.97
600	25.14	25.28	25.35	25.45	25.54	25.61
700	26.07	26.20	26.27	26.37	26.46	26.55
800	27.53	27.67	27.76	27.87	27.98	28.07
900	28.38	28.56	28.66	28.79	28.91	29.02
985	28.75	28.97	29.09	29.25	29.38	29.51
Structure S22 – Alana Circuit Culvert						
1100	30.81	31.04	31.15	31.27	31.39	31.49
1200	32.39	32.57	32.67	32.79	32.91	33.01
1300	33.85	34.02	34.11	34.24	34.36	34.47
1400	35.41	35.55	35.64	35.75	35.85	35.95
1500	37.53	37.67	37.75	37.84	37.93	38.01
1561	38.93	39.06	39.13	39.22	39.30	39.37
Tributary A						
0	30.80	31.04	31.14	31.26	31.37	31.48

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)
100	32.36	32.39	32.40	32.40	32.41	32.47
200	33.38	33.48	33.54	33.62	33.69	33.76
300	34.75	34.83	34.88	34.96	35.04	35.11
400	36.24	36.33	36.38	36.44	36.49	36.55
479	37.69	37.81	37.88	37.97	38.04	38.11
Tributary B						
0	35.41	35.51	35.58	35.67	35.75	35.83
90	37.80	37.94	37.99	38.05	38.12	38.18

N/R = no overland flooding

Appendix F: 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)
Cubberla Creek						
0	1.21	1.21	1.21	1.21	1.21	1.21
100	1.41	1.54	1.61	1.71	1.83	1.95
200	1.71	1.97	2.10	2.28	2.50	2.70
Structure S1 – Jesmond Road Bridge						
300	2.48	2.92	3.11	3.36	3.65	3.89
400	3.49	4.12	4.39	4.73	5.07	5.30
500	3.72	4.26	4.53	4.85	5.18	5.38
600	4.01	4.41	4.63	4.91	5.21	5.41
700	4.06	4.44	4.64	4.91	5.21	5.41
800	4.08	4.45	4.66	4.93	5.23	5.43
900	4.08	4.45	4.66	4.93	5.23	5.43
1000	4.12	4.48	4.68	4.95	5.25	5.45
1100	4.18	4.53	4.73	4.98	5.27	5.47
1200	4.23	4.57	4.76	5.00	5.30	5.49
1300	4.27	4.60	4.78	5.02	5.32	5.51
1400	4.36	4.64	4.81	5.04	5.33	5.52
1500	4.60	4.77	4.88	5.07	5.35	5.53
1600	4.88	5.00	5.08	5.18	5.42	5.59
1700	5.69	5.86	5.95	6.01	6.10	6.17
1800	6.02	6.21	6.29	6.35	6.45	6.51
1900	6.43	6.63	6.72	6.79	6.89	6.96
2000	6.69	6.88	6.97	7.04	7.15	7.22
2100	6.77	6.97	7.06	7.13	7.23	7.31
2200	6.99	7.16	7.25	7.31	7.41	7.48
2300	7.40	7.51	7.58	7.62	7.70	7.76
Structure S2 – Dobell Street Footbridge						
2400	8.23	8.33	8.38	8.42	8.49	8.55
2500	8.74	8.84	8.90	8.94	9.01	9.06
2600	9.33	9.48	9.54	9.59	9.66	9.71
2690	10.04	10.32	10.44	10.54	10.67	10.77
Structure S3 – Western Freeway Bridge						
2800	10.63	11.34	11.62	11.95	12.41	12.72

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)
2900	11.01	11.48	11.72	12.02	12.45	12.76
3000	11.35	11.64	11.82	12.06	12.47	12.77
Structure S4 – Garaboo Street Footbridge						
3100	11.70	11.91	12.10	12.27	12.60	12.87
3200	12.04	12.22	12.35	12.47	12.70	12.92
Structure S5 – Akuna Street Footbridge						
3300	12.54	12.65	12.71	12.77	12.88	13.03
3400	13.07	13.23	13.31	13.39	13.49	13.59
3500	13.52	13.70	13.78	13.87	13.97	14.06
3600	13.92	14.09	14.17	14.25	14.36	14.45
3700	14.25	14.44	14.54	14.64	14.75	14.85
3800	14.55	14.74	14.83	14.93	15.04	15.14
Structure S6 – Henry Street Footbridge						
3900	15.12	15.34	15.46	15.59	15.74	15.86
4000	15.54	15.73	15.84	15.96	16.11	16.22
4100	15.96	16.16	16.27	16.40	16.55	16.67
4200	16.29	16.53	16.65	16.80	16.95	17.08
4300	17.33	17.63	17.76	17.89	18.05	18.17
Structures S7 and S8 – Moggill Road Culvert						
4415	19.53	20.30	20.57	21.02	21.54	21.85
4500	19.76	20.45	20.71	21.14	21.64	21.95
4600	20.01	20.60	20.85	21.25	21.73	22.03
4700	20.22	20.73	20.96	21.37	21.82	22.12
4800	21.08	21.31	21.43	21.76	22.07	22.32
4900	21.98	22.22	22.35	22.51	22.69	22.87
Structure S9 – Bulk Water Mains #1						
4990	22.56	22.92	23.11	23.43	23.83	24.10
Structure S10 – Tristania Road Culvert						
5100	24.32	24.69	24.83	25.01	25.20	25.36
5200	24.41	24.82	24.98	25.19	25.41	25.58
Structure S11 – 56 Tristania Road Access Bridge						
5300	24.60	24.97	25.11	25.32	25.53	25.70
Structure S12 – 70 Tristania Road Access Bridge						

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)
5400	24.99	25.23	25.35	25.52	25.70	25.86
5500	25.73	25.88	25.98	26.11	26.23	26.34
5600	26.67	26.85	26.95	27.10	27.22	27.29
Structure S13 – Chapel Hill State School Culvert						
5700	27.56	28.02	28.33	28.62	28.78	28.95
5800	27.97	28.37	28.47	28.79	28.93	29.08
5900	28.67	28.99	29.17	29.35	29.54	29.66
Structure S14 – Goolman Street Culvert						
6000	29.59	30.29	30.49	30.63	30.78	30.87
6100	31.21	31.48	31.60	31.75	31.90	32.01
6200	31.79	32.06	32.19	32.32	32.40	32.48
6300	33.37	33.61	33.72	33.85	33.98	34.09
6400	34.13	34.43	34.58	34.78	34.97	35.12
6500	35.25	35.48	35.60	35.77	35.95	36.11
Structure S18 – Dillingen Street Culvert						
6600	36.87	37.19	37.36	37.59	37.87	38.09
6700	38.78	38.93	39.01	39.10	39.19	39.27
6800	40.19	40.34	40.44	40.55	40.66	40.75
6900	41.71	41.89	42.00	42.13	42.24	42.33
7000	43.51	43.70	43.81	43.93	44.03	44.12
7100	45.29	45.45	45.56	45.67	45.76	45.84
7200	46.44	46.64	46.76	46.86	46.95	47.02
7300	48.22	48.27	48.29	48.34	48.40	48.45
7400	49.25	49.40	49.48	49.55	49.63	49.68
7500	50.92	51.16	51.30	51.66	51.83	51.90
7600	53.28	53.56	53.68	53.84	53.88	53.90
7700	55.86	55.92	55.96	55.97	56.02	56.05
Structure S19 – Greenford Street Culvert						
7800	58.26	58.37	58.45	58.82	59.39	59.93
7887	60.86	60.98	61.02	61.09	61.14	61.19
Tributary C						
0	9.88	10.14	10.25	10.35	10.48	10.56
Structure S27 – Fig Tree Pocket Road Culvert						

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)
85	11.32	11.55	11.69	11.74	11.80	12.49
Structures S28 and 29 – Western Freeway On and Off Ramp Culverts						
200	12.27	12.52	12.66	12.83	12.94	13.02
300	12.36	12.61	12.73	12.91	13.03	13.13
400	12.83	13.09	13.20	13.38	13.51	13.61
500	13.91	14.14	14.24	14.40	14.53	14.61
Structure S30 – Norman Street Footbridge						
600	15.39	15.55	15.65	15.78	15.88	15.97
700	17.10	17.33	17.45	17.58	17.68	17.80
732	17.67	17.89	18.01	18.16	18.26	18.38
Akuna Street Branch						
0	12.08	12.26	12.38	12.49	12.71	12.93
Structure S25 – Katunga Street Culvert						
100	13.13	13.28	13.35	13.43	13.50	13.57
200	14.52	14.78	14.89	15.02	15.12	15.23
300	16.01	16.24	16.35	16.48	16.58	16.70
400	17.52	17.73	17.83	17.95	18.04	18.15
500	19.47	19.73	19.86	20.00	20.10	20.23
600	23.90	24.09	24.19	24.23	24.32	24.39
Structure S26 – Marshall Lane Culvert						
700	24.21	24.50	24.63	24.78	24.88	24.99
800	25.28	25.41	25.51	25.67	25.78	25.92
900	26.00	26.16	26.25	26.37	26.46	26.56
1000	27.90	28.08	28.17	28.28	28.36	28.46
1050	29.00	29.20	29.31	29.45	29.54	29.65
Gubberley Creek						
0	13.51	13.68	13.77	13.86	13.96	14.05
100	13.85	14.08	14.19	14.30	14.43	14.54
200	15.10	15.28	15.40	15.51	15.64	15.74
300	15.42	15.65	15.79	15.93	16.09	16.21
400	N/R	16.95	17.03	17.12	17.20	17.26
Marshall Lane Piped Drainage						
500	17.33	18.59	18.75	18.89	19.02	19.12

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)
600	18.76	19.29	19.44	19.59	19.77	19.92
700	20.14	20.40	20.51	20.64	20.79	20.93
Structure S23 – Cedar Xing Culvert						
820	22.11	22.57	22.79	23.16	23.43	23.57
Structure S24 – Gubberley Creek Detention Basin						
910	27.90	28.07	28.13	28.20	28.28	28.35
1000	27.98	28.13	28.19	28.27	28.36	28.43
1100	29.15	29.18	29.23	29.29	29.31	29.34
1200	30.10	30.21	30.26	30.33	30.39	30.45
1252	30.94	31.05	31.12	31.19	31.23	31.29
Boblynne Street Branch						
0	19.91	20.53	20.79	21.20	21.69	22.00
Structure S20 – St. James Estate Access Culvert						
100	20.14	20.63	20.90	21.27	21.76	22.06
200	20.66	20.90	21.11	21.40	21.83	22.12
300	21.68	21.88	21.99	22.12	22.26	22.39
Structure S21 – Bulk Water Mains #2						
400	22.66	22.86	22.98	23.11	23.22	23.31
500	24.65	24.75	24.81	24.89	24.96	25.03
600	25.18	25.33	25.42	25.52	25.61	25.69
700	26.11	26.26	26.35	26.45	26.56	26.65
800	27.73	27.93	28.05	28.19	28.33	28.45
900	28.60	28.84	28.97	29.13	29.28	29.41
985	28.89	29.16	29.31	29.47	29.64	29.78
Structure S22 – Alana Circuit Culvert						
1100	30.80	31.04	31.15	31.27	31.39	31.49
1200	32.39	32.58	32.68	32.81	32.92	33.03
1300	33.86	34.03	34.14	34.27	34.40	34.51
1400	35.42	35.57	35.66	35.78	35.89	35.99
1500	37.54	37.67	37.75	37.85	37.94	38.03
1561	38.93	39.06	39.14	39.22	39.30	39.38
Tributary A						
0	30.80	31.03	31.14	31.26	31.37	31.48

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)
100	32.36	32.39	32.40	32.40	32.45	32.52
200	33.38	33.48	33.55	33.62	33.70	33.77
300	34.75	34.84	34.90	34.98	35.06	35.13
400	36.26	36.36	36.41	36.48	36.54	36.60
479	37.72	37.85	37.92	38.01	38.09	38.16
Tributary B						
0	35.42	35.54	35.62	35.72	35.80	35.88
90	37.81	37.93	37.98	38.05	38.11	38.17

Appendix G: 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 (1) (0.05 % AEP)
Cubberla Creek			
0	1.82	1.82	1.82
100	2.49	2.69	3.00
200	3.19	3.54	4.07
Structure S1 – Jesmond Road Bridge			
300	4.37	4.78	5.75
400	5.55	5.78	6.27
500	5.63	5.86	6.32
600	5.66	5.89	6.34
700	5.66	5.89	6.34
800	5.67	5.90	6.34
900	5.67	5.89	6.34
1000	5.68	5.91	6.35
1100	5.71	5.94	6.38
1200	5.73	5.97	6.40
1300	5.74	5.98	6.42
1400	5.75	5.99	6.42
1500	5.76	6.00	6.43
1600	5.80	6.04	6.46
1700	6.29	6.44	6.72
1800	6.63	6.75	6.95
1900	7.09	7.21	7.36
2000	7.35	7.48	7.61
2100	7.42	7.55	7.68
2200	7.61	7.73	7.85
2300	7.86	7.96	8.08
Structure S2 – Dobell Street Footbridge			
2400	8.55	8.66	8.77
2500	9.07	9.16	9.26
2600	9.70	9.77	9.86
2690	10.69	10.75	10.82
Structure S3 – Western Freeway Bridge			
2800	13.13	13.52	14.11

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 (1) (0.05 % AEP)
2900	13.15	13.54	14.12
3000	13.15	13.53	14.11
Structure S4 – Garaboo Street Footbridge			
3100	13.29	13.67	14.23
3200	13.32	13.71	14.29
Structure S5 – Akuna Street Footbridge			
3300	13.25	13.59	14.20
3400	13.70	13.89	14.27
3500	14.07	14.20	14.43
3600	14.38	14.51	14.65
3700	14.88	15.03	15.17
3800	15.24	15.38	15.52
Structure S6 – Henry Street Footbridge			
3900	15.96	16.12	16.26
4000	16.31	16.48	16.63
4100	16.78	16.97	17.15
4200	17.21	17.39	17.57
4300	18.30	18.46	18.60
Structures S7 and S8 – Moggill Road Culvert			
4415	22.27	22.52	22.67
4500	22.33	22.60	22.76
4600	22.39	22.66	22.84
4700	22.44	22.72	22.90
4800	22.55	22.83	23.02
4900	22.96	23.22	23.39
Structure S9 – Bulk Water Mains #1			
4990	24.40	24.61	24.74
Structure S10 – Tristania Road Culvert			
5100	25.44	25.66	25.84
5200	25.61	25.85	26.05
Structure S11 – 56 Tristania Road Access Bridge			
5300	25.70	25.94	26.13
Structure S12 – 70 Tristania Road Access Bridge			

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 [(1)] (0.05 % AEP)
5400	25.87	26.08	26.27
5500	26.33	26.49	26.63
5600	27.36	27.46	27.53
Structure S13 – Chapel Hill State School Culvert			
5700	29.16	29.30	29.44
5800	29.29	29.45	29.58
5900	29.90	30.07	30.21
Structure S14 – Goolman Street Culvert			
6000	30.98	31.08	31.18
6100	31.63	31.78	31.90
6200	32.32	32.46	32.58
6300	33.97	34.12	34.24
6400	34.72	34.89	35.05
6500	36.03	36.19	36.32
Structure S18 – Dillingen Street Culvert			
6600	38.27	38.47	38.66
6700	39.22	39.33	39.47
6800	40.78	40.91	41.07
6900	42.42	42.56	42.74
7000	44.22	44.35	44.55
7100	45.93	46.06	46.24
7200	47.10	47.22	47.35
7300	48.48	48.59	48.71
7400	49.73	49.85	49.96
7500	52.05	52.24	52.40
7600	53.96	54.02	54.08
7700	56.12	56.28	56.42
Structure S19 – Greenford Street Culvert			
7800	60.44	60.72	60.89
7887	61.25	61.31	61.35
Tributary C			
0	10.46	10.52	10.60
Structure S27 – Fig Tree Pocket Road Culvert			

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 (1) (0.05 % AEP)
85	13.07	13.49	14.08
Structures S28 and 29 – Western Freeway On and Off Ramp Culverts			
200	13.14	13.51	14.09
300	13.26	13.52	14.10
400	13.73	13.88	14.35
500	14.70	14.82	14.97
Structure S30 – Norman Street Footbridge			
600	16.06	16.21	16.05
700	17.78	17.93	17.66
732	18.33	18.48	18.17
Akuna Street Branch			
0	13.32	13.71	14.29
Structure S25 – Katunga Street Culvert			
100	13.58	13.79	14.28
200	15.31	15.45	15.33
300	16.69	16.83	16.64
400	18.18	18.30	18.12
500	20.23	20.37	20.12
600	24.45	24.52	24.39
Structure S26 – Marshall Lane Culvert			
700	25.08	25.20	24.99
800	25.97	26.15	25.82
900	26.56	26.70	26.43
1000	28.51	28.64	28.37
1050	29.74	29.88	29.57
Gubberley Creek			
0	14.06	14.19	14.43
100	14.48	14.64	14.81
200	15.86	15.98	15.94
300	16.37	16.51	16.44
400	17.34	17.41	17.34
Marshall Lane Piped Drainage			
500	19.20	19.32	19.20

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 (1) (0.05 % AEP)
600	20.04	20.16	20.03
700	21.06	21.21	21.00
Structure S23 – Cedar Xing Culvert			
820	23.72	23.84	23.67
Structure S24 – Gubberley Creek Detention Basin			
910	28.44	28.53	28.40
1000	28.53	28.62	28.48
1100	29.29	29.33	29.09
1200	30.47	30.55	30.39
1252	31.35	31.42	31.22
Boblynne Street Branch			
0	22.36	22.63	22.79
Structure S20 – St. James Estate Access Culvert			
100	22.41	22.68	22.86
200	22.46	22.73	22.92
300	22.64	22.89	23.08
Structure S21 – Bulk Water Mains #2			
400	23.45	23.61	23.69
500	25.06	25.15	25.11
600	25.72	25.84	25.80
700	26.68	26.81	26.74
800	28.21	28.35	28.28
900	29.18	29.33	29.25
985	29.70	29.86	29.77
Structure S22 – Alana Circuit Culvert			
1100	31.65	31.77	31.70
1200	33.16	33.32	33.17
1300	34.63	34.79	34.62
1400	36.09	36.23	36.09
1500	38.13	38.25	38.13
1561	39.48	39.59	39.48
Tributary A			
0	31.63	31.76	31.68

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 (1) (0.05 % AEP)
100	32.58	32.69	32.64
200	33.86	33.96	33.90
300	35.21	35.31	35.24
400	36.63	36.71	36.63
479	38.21	38.31	38.21
Tributary B			
0	35.93	36.03	35.96
90	38.26	38.33	38.36

(1) In areas with a small upstream catchment, the 2000-yr ARI (0.05 % AEP) super-storm methodology does not always produce a peak flood level greater than the 200-yr ARI (0.5 % AEP) and / or 500-yr ARI (0.2 % AEP) peak flood level using AR&R 1987 methodology.

Appendix H: 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)		
	100-yr ARI (1 % AEP)	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)
Cubberla Creek			
0	1.21	1.82	1.82
100	1.95	2.48	2.68
200	2.70	3.17	3.52
Structure S1 – Jesmond Road Bridge			
300	3.89	4.32	4.73
400	5.30	5.56	5.80
500	5.38	5.65	5.88
600	5.41	5.67	5.91
700	5.41	5.67	5.91
800	5.43	5.69	5.93
900	5.43	5.69	5.92
1000	5.45	5.71	5.94
1100	5.47	5.73	5.97
1200	5.49	5.76	6.00
1300	5.51	5.77	6.02
1400	5.52	5.78	6.03
1500	5.53	5.79	6.04
1600	5.59	5.84	6.08
1700	6.17	6.29	6.45
1800	6.51	6.62	6.75
1900	6.96	7.07	7.19
2000	7.22	7.33	7.46
2100	7.31	7.42	7.55
2200	7.48	7.59	7.71
2300	7.76	7.86	7.97
Structure S2 – Dobell Street Footbridge			
2400	8.55	8.64	8.75
2500	9.06	9.14	9.23
2600	9.71	9.77	9.85
2690	10.77	10.85	10.91
Structure S3 – Western Freeway Bridge			
2800	12.72	13.14	13.52

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)		
	100-yr ARI (1 % AEP)	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)
2900	12.76	13.16	13.54
3000	12.77	13.17	13.55
Structure S4 – Garaboo Street Footbridge			
3100	12.87	13.25	13.61
3200	12.92	13.27	13.61
Structure S5 – Akuna Street Footbridge			
3300	13.03	13.30	13.63
3400	13.59	13.77	13.97
3500	14.06	14.21	14.37
3600	14.45	14.61	14.77
3700	14.85	15.01	15.18
3800	15.14	15.29	15.47
Structure S6 – Henry Street Footbridge			
3900	15.86	16.06	16.25
4000	16.22	16.42	16.61
4100	16.67	16.88	17.09
4200	17.08	17.30	17.52
4300	18.17	18.36	18.52
Structures S7 and S8 – Moggill Road Culvert			
4415	21.85	22.21	22.45
4500	21.95	22.31	22.55
4600	22.03	22.39	22.64
4700	22.12	22.48	22.74
4800	22.32	22.67	22.94
4900	22.87	23.15	23.38
Structure S9 – Bulk Water Mains #1			
4990	24.10	24.38	24.58
Structure S10 – Tristania Road Culvert			
5100	25.36	25.60	25.84
5200	25.58	25.85	26.12
Structure S11 – 56 Tristania Road Access Bridge			
5300	25.70	25.97	26.23
Structure S12 – 70 Tristania Road Access Bridge			

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)		
	100-yr ARI (1 % AEP)	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)
5400	25.86	26.10	26.35
5500	26.34	26.52	26.70
5600	27.29	27.42	27.53
Structure S13 – Chapel Hill State School Culvert			
5700	28.95	29.16	29.29
5800	29.08	29.29	29.45
5900	29.66	29.88	30.05
Structure S14 – Goolman Street Culvert			
6000	30.87	31.01	31.13
6100	32.01	32.17	32.30
6200	32.48	32.61	32.76
6300	34.09	34.26	34.40
6400	35.12	35.36	35.56
6500	36.11	36.35	36.55
Structure S18 – Dillingen Street Culvert			
6600	38.09	38.35	38.57
6700	39.27	39.39	39.52
6800	40.75	40.89	41.03
6900	42.33	42.47	42.61
7000	44.12	44.24	44.37
7100	45.84	45.97	46.10
7200	47.02	47.13	47.25
7300	48.45	48.55	48.67
7400	49.68	49.78	49.90
7500	51.90	52.05	52.24
7600	53.90	53.96	54.02
7700	56.05	56.18	56.35
Structure S19 – Greenford Street Culvert			
7800	59.93	60.44	60.72
7887	61.19	61.25	61.31
Tributary C			
0	10.56	10.64	10.71
Structure S27 – Fig Tree Pocket Road Culvert			

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)		
	100-yr ARI (1 % AEP)	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)
85	12.49	13.07	13.48
Structures S28 and 29 – Western Freeway On and Off Ramp Culverts			
200	13.02	13.13	13.50
300	13.13	13.26	13.51
400	13.61	13.74	13.88
500	14.61	14.73	14.84
Structure S30 – Norman Street Footbridge			
600	15.97	16.10	16.24
700	17.80	17.95	18.12
732	18.38	18.53	18.71
Akuna Street Branch			
0	12.93	13.27	13.62
Structure S25 – Katunga Street Culvert			
100	13.57	13.66	13.82
200	15.23	15.36	15.50
300	16.70	16.83	16.98
400	18.15	18.27	18.41
500	20.23	20.36	20.53
600	24.39	24.46	24.53
Structure S26 – Marshall Lane Culvert			
700	24.99	25.11	25.23
800	25.92	26.08	26.27
900	26.56	26.70	26.87
1000	28.46	28.58	28.71
1050	29.65	29.76	29.90
Gubberley Creek			
0	14.05	14.20	14.36
100	14.54	14.72	14.92
200	15.74	15.86	15.98
300	16.21	16.37	16.50
400	17.26	17.33	17.39
Marshall Lane Piped Drainage			
500	19.12	19.26	19.48

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)		
	100-yr ARI (1 % AEP)	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)
600	19.92	20.11	20.26
700	20.93	21.13	21.30
Structure S23 – Cedar Xing Culvert			
820	23.57	23.71	23.87
Structure S24 – Gubberley Creek Detention Basin			
910	28.35	28.44	28.53
1000	28.43	28.54	28.64
1100	29.34	29.38	29.42
1200	30.45	30.52	30.61
1252	31.29	31.36	31.44
Boblynne Street Branch			
0	22.00	22.35	22.60
Structure S20 – St. James Estate Access Culvert			
100	22.06	22.41	22.67
200	22.12	22.47	22.74
300	22.39	22.68	22.95
Structure S21 – Bulk Water Mains #2			
400	23.31	23.48	23.67
500	25.03	25.12	25.22
600	25.69	25.83	25.92
700	26.65	26.79	26.93
800	28.45	28.63	28.81
900	29.41	29.60	29.79
985	29.78	29.99	30.19
Structure S22 – Alana Circuit Culvert			
1100	31.49	31.65	31.78
1200	33.03	33.20	33.36
1300	34.51	34.69	34.86
1400	35.99	36.14	36.30
1500	38.03	38.16	38.28
1561	39.38	39.49	39.60
Tributary A			
0	31.48	31.64	31.77

AMTD (m)	Rare Events – Scenario 3 (Ultimate Waterway Conditions) Peak Water Levels (mAHD)		
	100-yr ARI (1 % AEP)	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)
100	32.52	32.64	32.76
200	33.77	33.87	33.98
300	35.13	35.24	35.35
400	36.60	36.69	36.79
479	38.16	38.27	38.37
Tributary B			
0	35.88	35.99	36.10
90	38.17	38.25	38.34

Appendix I: Rating Curves

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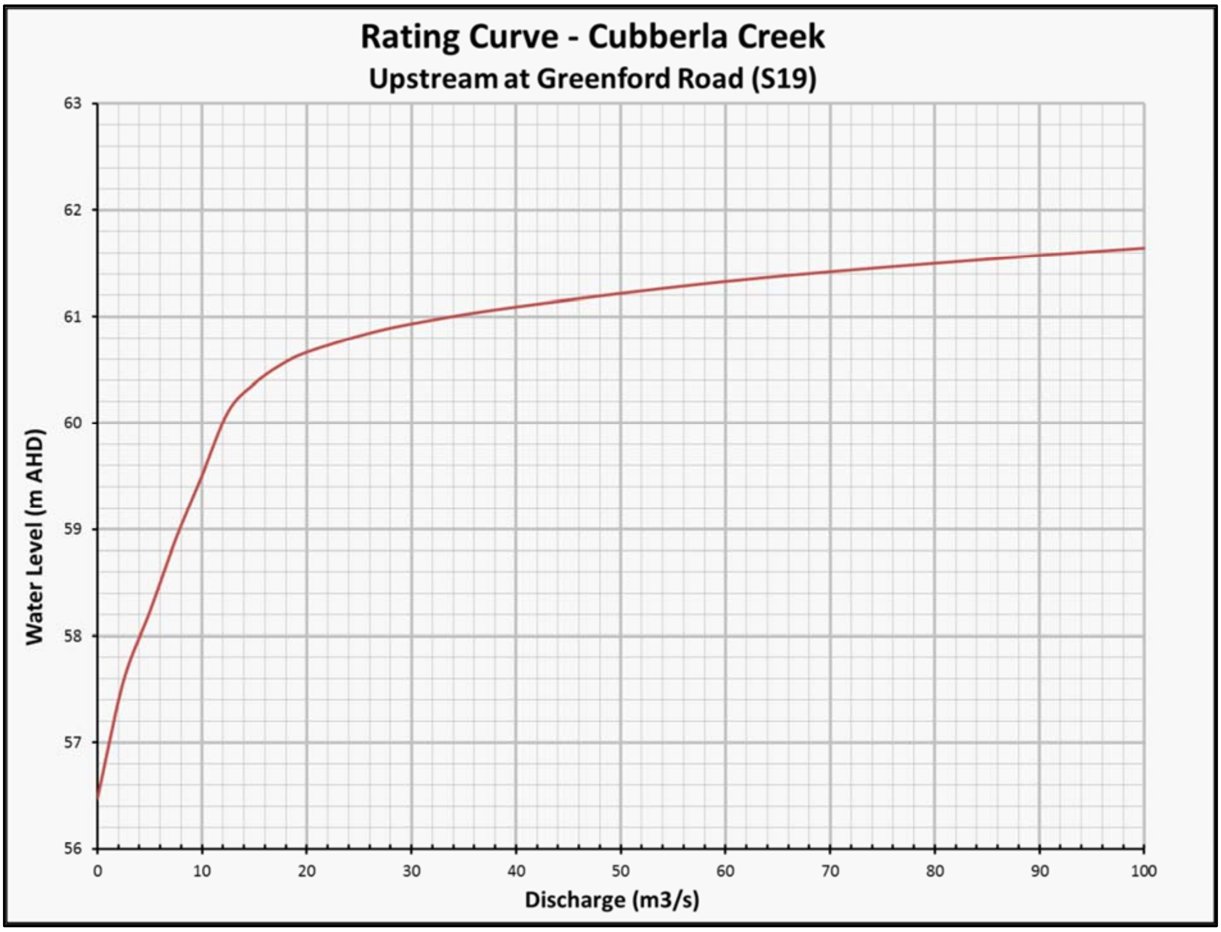
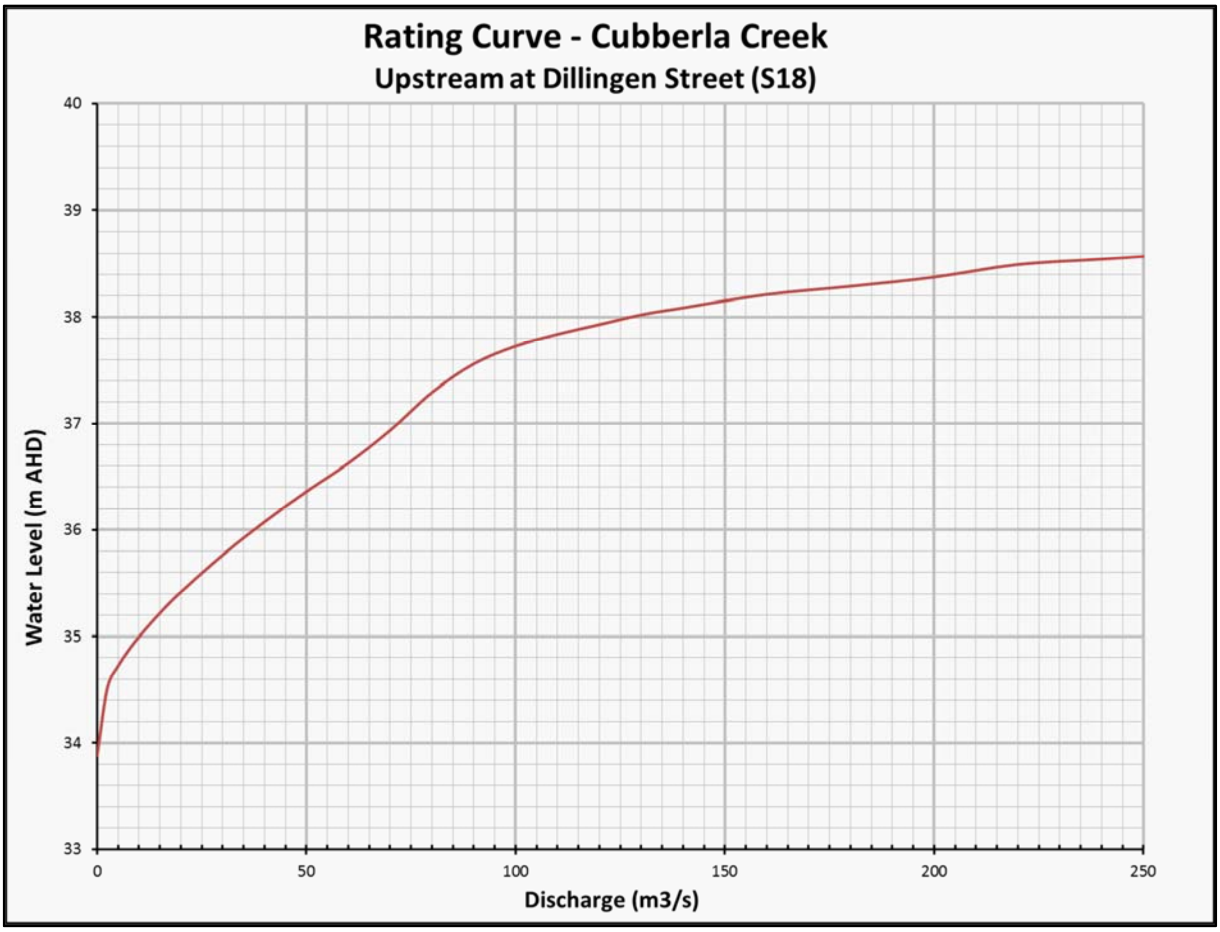
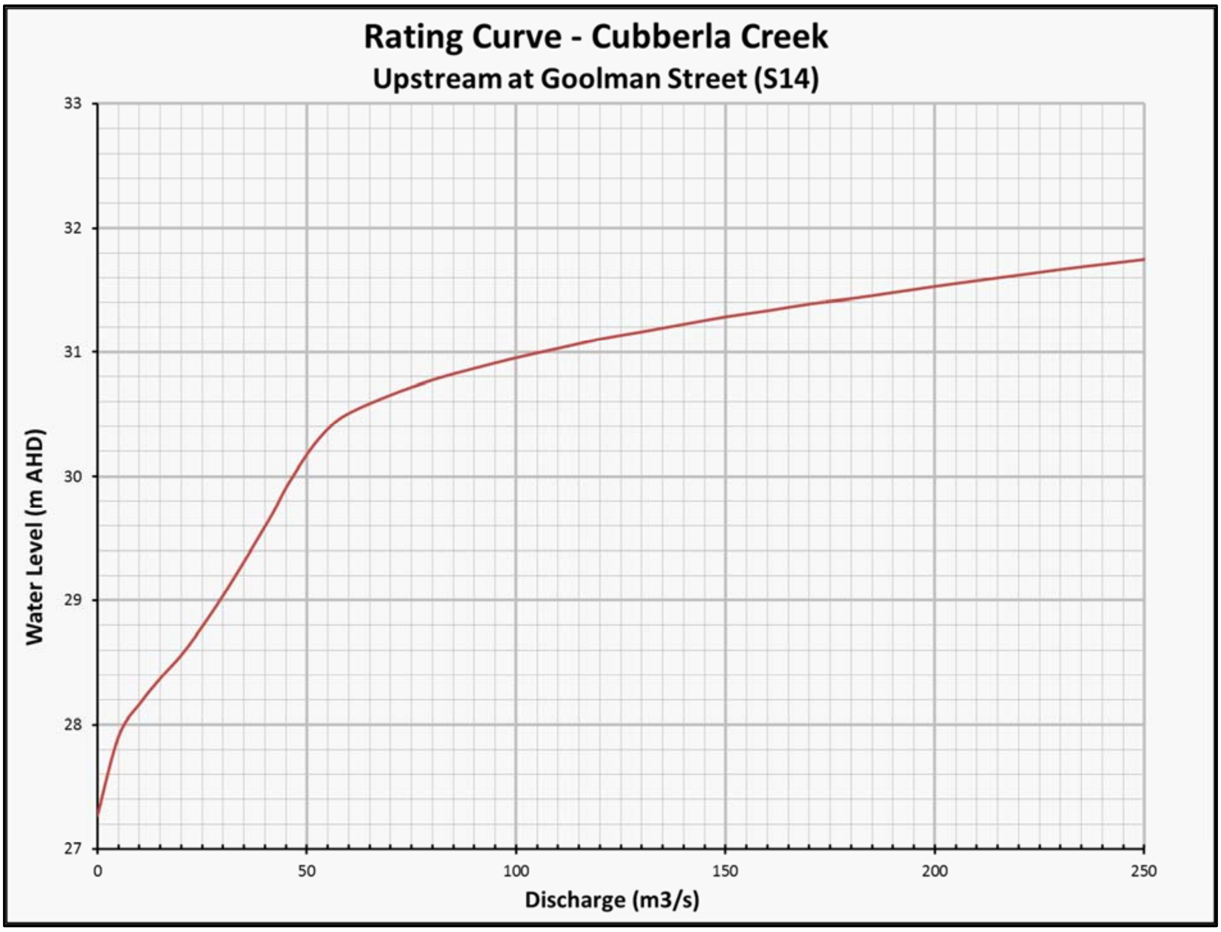
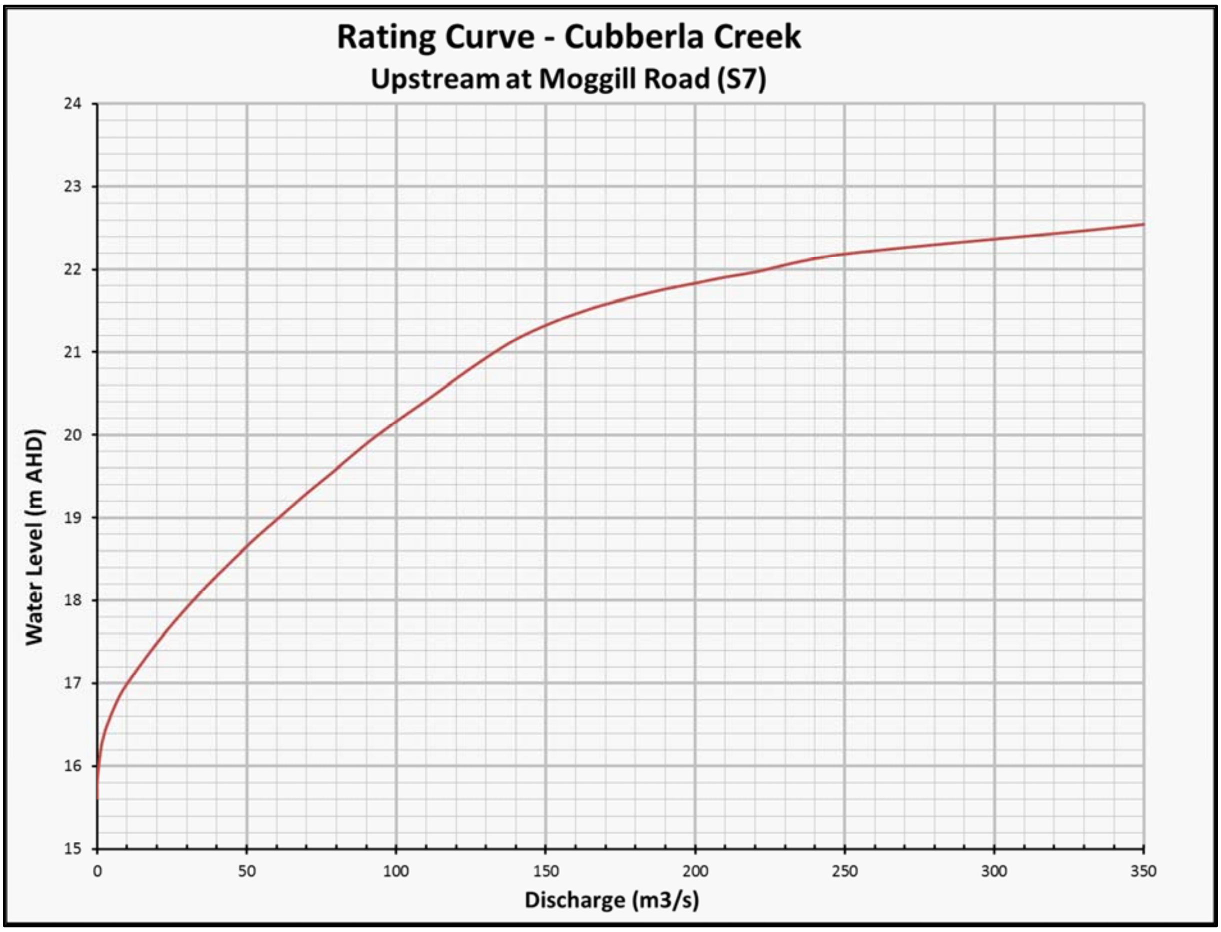
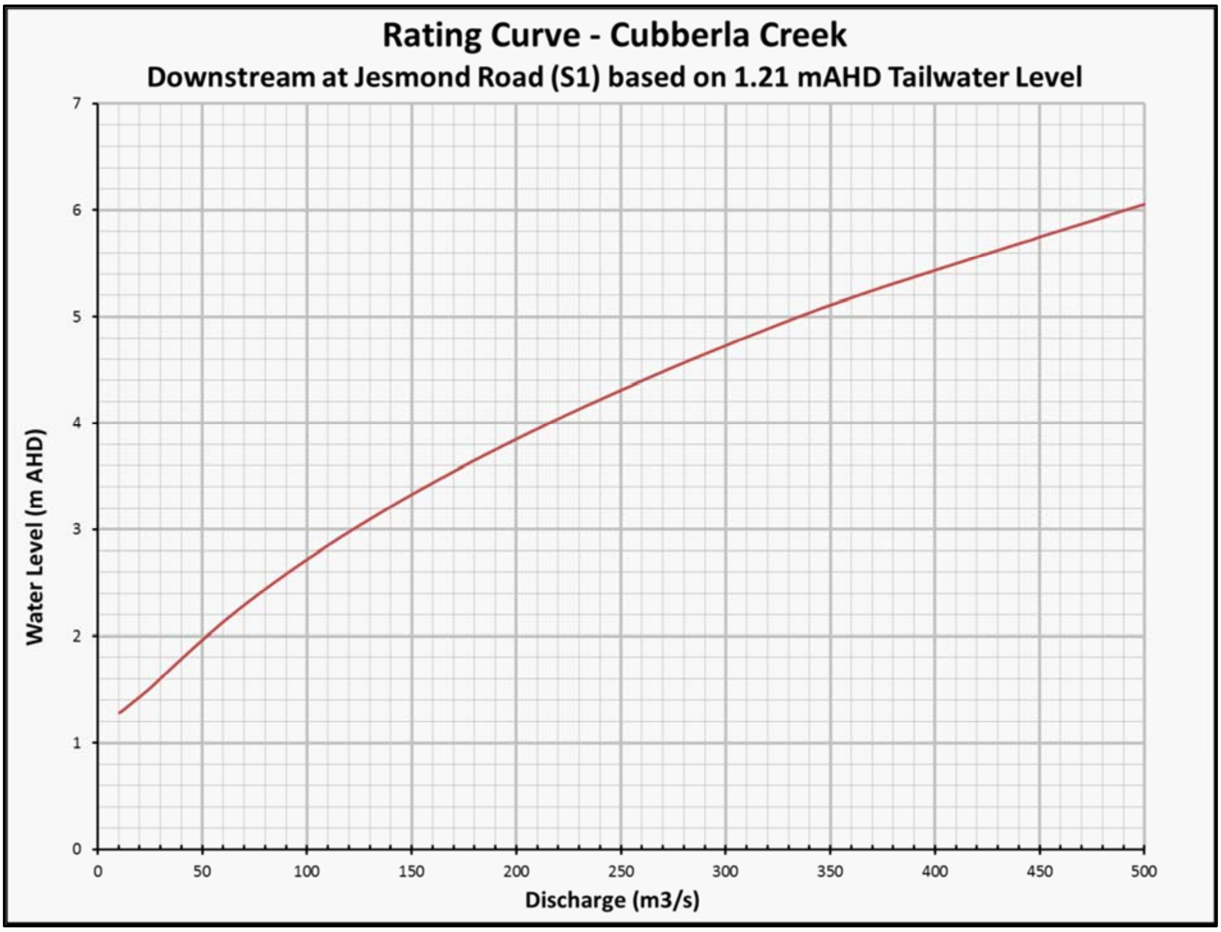
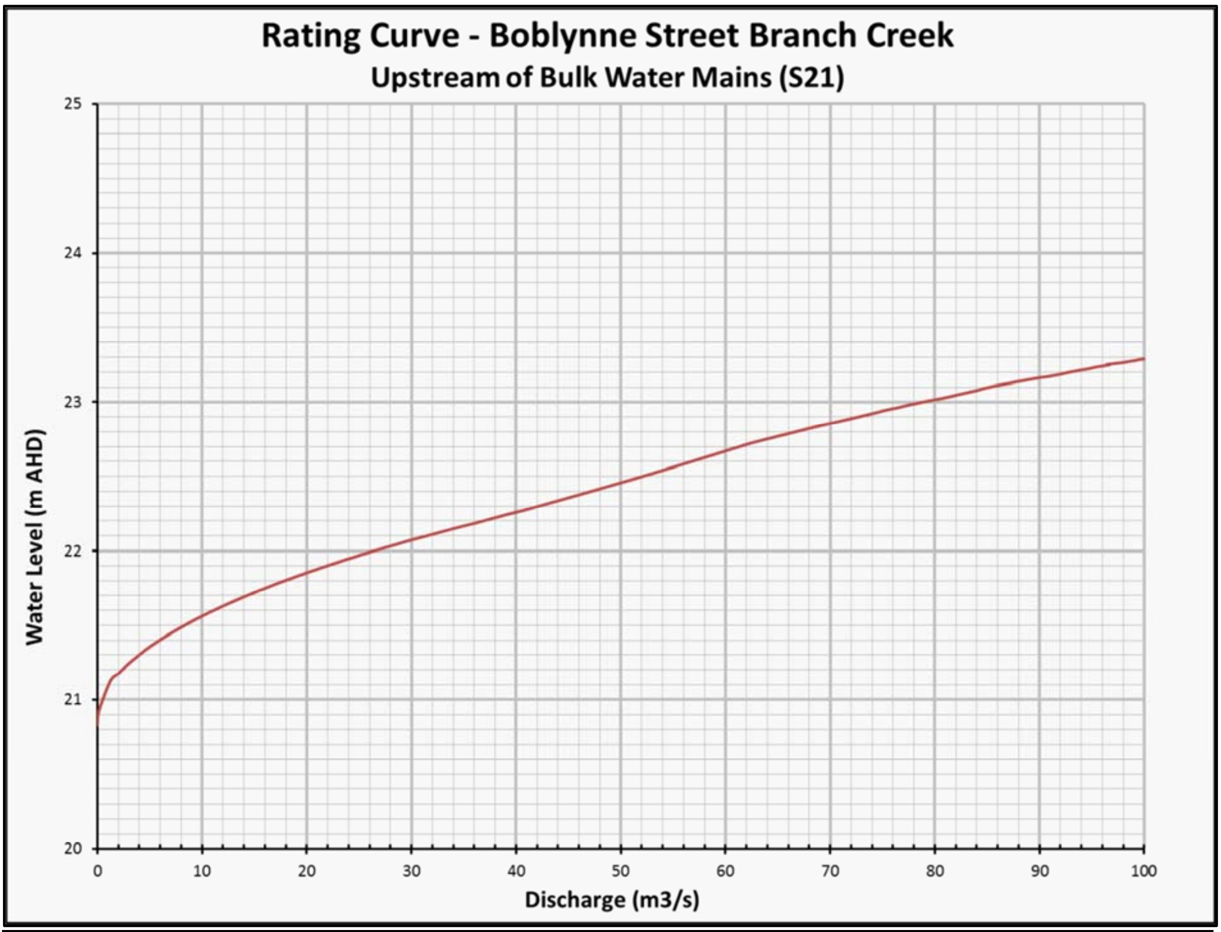

Rating Curve - Cubberla Creek
Upstream at Greenford Road (S19)
Water Level (m AHD)
63
62
61
60
59
58
57
56
0
10
20
30
40
50
60
70
80
90
100
Discharge (m3/s)

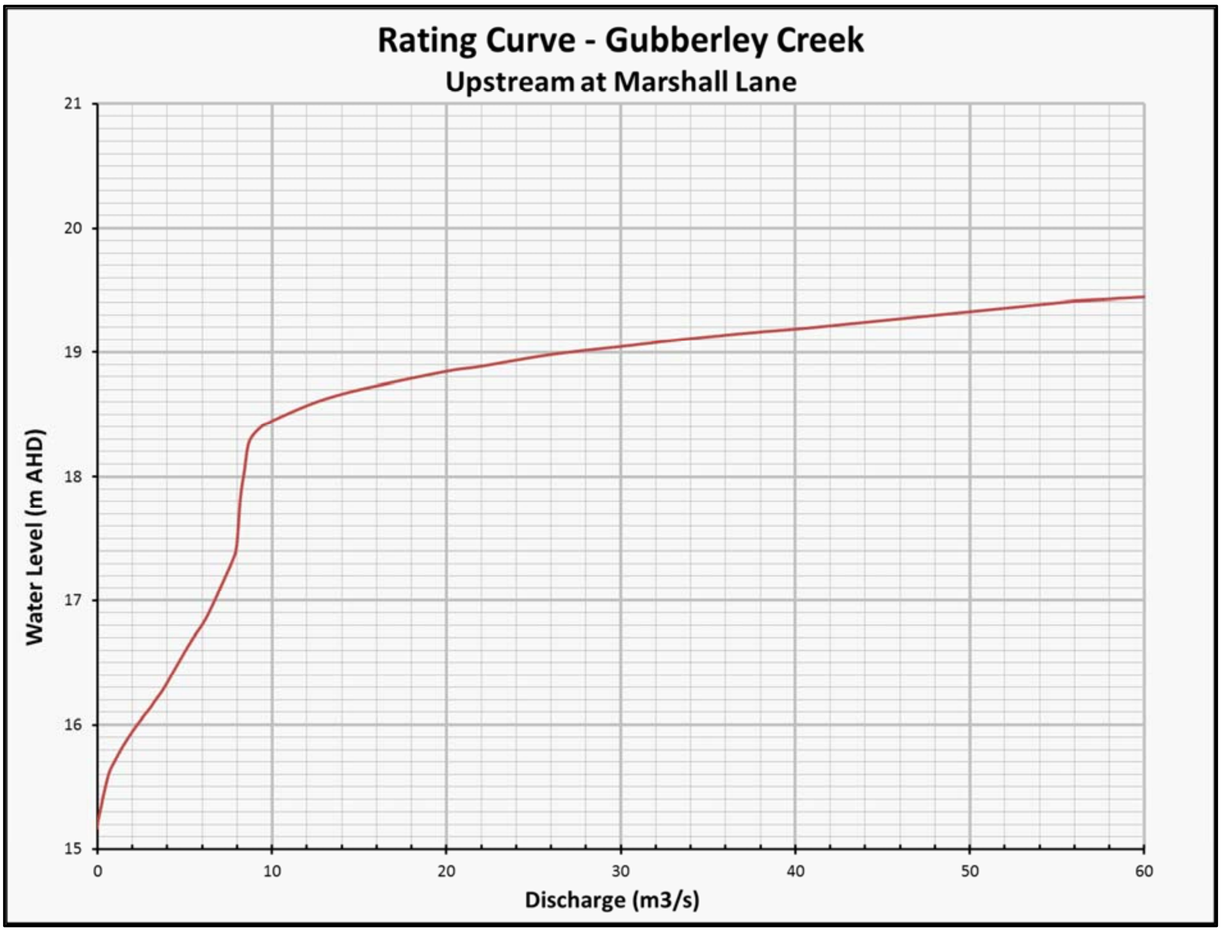

Rating Curve - Cubberla Creek
Upstream at Dillingen Street (S18)
Water Level (m AHD)
40
39
38
37
36
35
34
33
0
50
100
150
200
250
Discharge (m3/s)

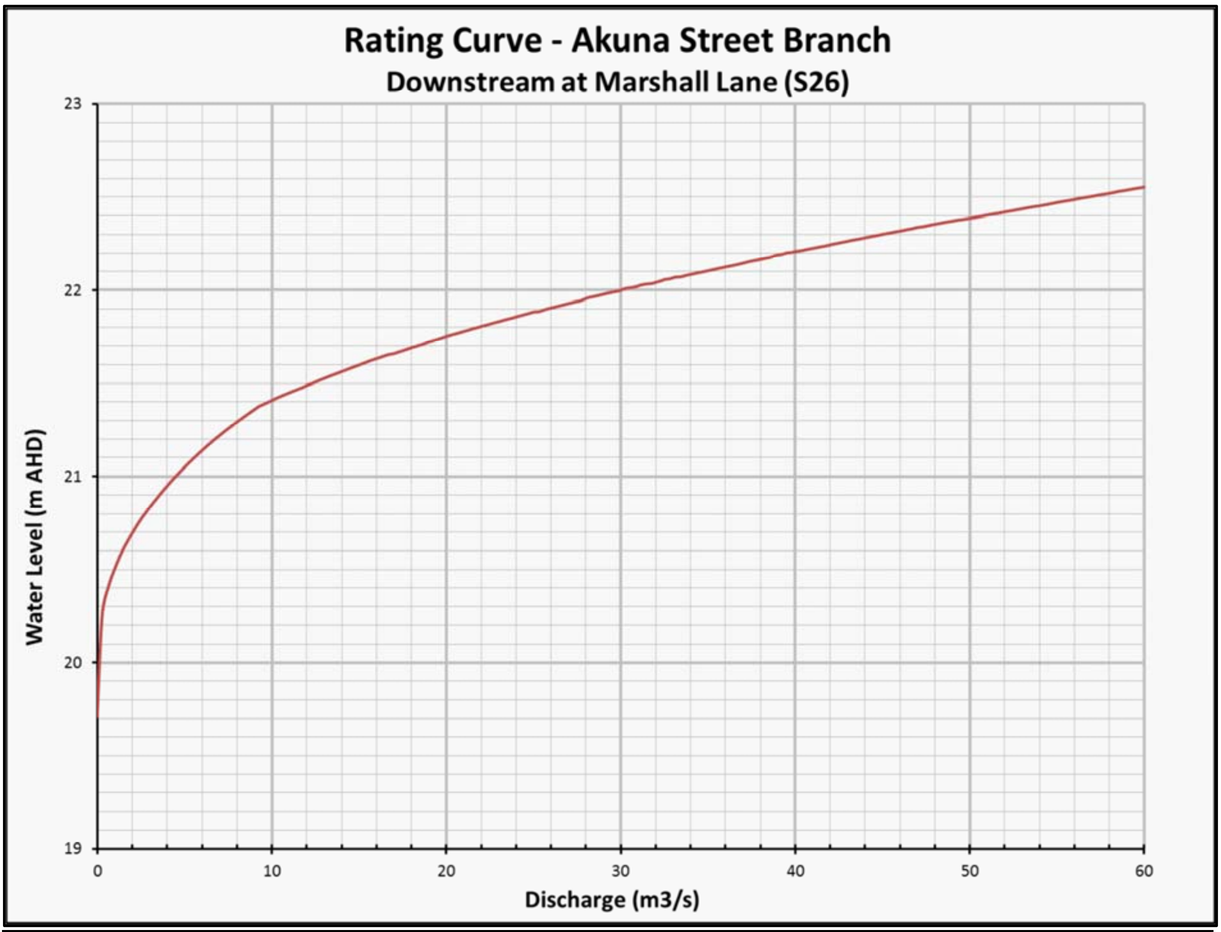

Rating Curve - Cubberla Creek
Upstream at Goolman Street (S14)
Water Level (m AHD)
33
32
31
30
29
28
27
0
50
100
150
200
250
Discharge (m3/s)


Rating Curve - Cubberla Creek
Upstream at Moggill Road (S7)
Water Level (m AHD)
24
23
22
21
20
19
18
17
16
15
0
50
100
150
200
250
300
350
Discharge (m3/s)


Rating Curve - Cubberla Creek
Downstream at Jesmond Road (S1) based on 1.21 mAHD Tailwater Level
Water Level (m AHD)
Discharge (m3/s)
0
1
2
3
4
5
6
7
0
50
100
150
200
250
300
350
400
450
500


Rating Curve - Boblynne Street Branch Creek
Upstream of Bulk Water Mains (S21)
Water Level (m AHD)
Discharge (m3/s)
20
21
22
23
24
25
0
10
20
30
40
50
60
70
80
90
100


Rating Curve - Gubberley Creek
Upstream at Marshall Lane
Water Level (m AHD)
Discharge (m3/s)
21
20
19
18
17
16
15
0
10
20
30
40
50
60


Rating Curve - Akuna Street Branch
Downstream at Marshall Lane (S26)
Water Level (m AHD)
Discharge (m3/s)
23
22
21
20
19
0
10
20
30
40
50
60

Appendix J: Hydraulic Structure Reference Sheets

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Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Jesmond Road Bridge

BCC Asset ID	B1070	Tributary Name	Cubberla Creek
Owner	BCC	AMTD	283
Year of Construction	1976	Coordinates (GDA94)	E 496701, N 6955550
Year of Significant Modification	1976 – former bridge collapsed in 1974 flood	Hydraulic Model ID	S1
Source of Structure Information	As-constructed drawings + creek survey (1995)	Flood Model Representation	1d bridge / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\1_Cubberla\AMTD 290 - Jesmond Road		

Structure Description		3 span concrete bridge	
Bridges		**Culverts**	
Number of Spans	3	Number of Barrels	N/A
Number of Piers in Waterway	2	Dimensions (m)	N/A
Pier shape and Width (m)	0.45 Octagonal	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	-0.08	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		~ 10	
Span Length (m)		7.73	
Lowest Level of Deck Soffit (m AHD)		4.26	
Lowest Level of Weir/Road (m AHD) (not including handrail)		4.94	
Average Handrail Height (m)		~ 1.24	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Downstream
Date	July 2014
Source	BCC Asset Management Records

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				20-yr ARI (5 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	302.8	282.6	5.76	4.73	1.04	4.4	N/A	N/A
0.2	226.3	223.0	4.62	4.12	0.49	3.8	N/A	90
1	155.6	143.9	3.68	3.26	0.42	3.5	N/A	90
2	132.3	127.6	3.46	3.07	0.39	3.4	N/A	90
5	107.7	107.7	3.17	2.82	0.36	3.3	N/A	90
10	93.4	93.4	2.96	2.63	0.33	3.1	N/A	90
20	81.0	81.0	2.75	2.45	0.30	3.0	N/A	90
50	59.2	59.2	2.36	2.11	0.25	2.7	N/A	90

[1] Flow underneath the road and only for 1D structures

[2] Measured at centre-span of bridge or at centre of culvert

[3] This is afflux at peak water level

[4] (i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening

[5] (i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model

[6] Velocities provided here are approximate only and the model should be interrogated for design purposes.

[7] Based on peak water level

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Dobell Street Footbridge

BCC Asset ID	B9722	Tributary Name	Cubberla Creek
Owner	BCC	AMTD	2376
Year of Construction	2009	Coordinates (GDA94)	E 495918, N 6956336
Year of Significant Modification	Former bridge replaced in 2009	Hydraulic Model ID	S2
Source of Structure Information	Design drawings + creek survey (circa 2009)	Flood Model Representation	1d bridge / 1d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\1_Cubberla\AMTD 2380 - Dobell St Foot Bridge		

Structure Description		Single span steel footbridge	
Bridges		**Culverts**	
Number of Spans	1	Number of Barrels	N/A
Number of Piers in Waterway	N/A	Dimensions (m)	N/A
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	5.69	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		~ 3.3	
Span Length (m)		11.15	
Lowest Level of Deck Soffit (m AHD)		7.07	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 7.56 (at structure)	
Average Handrail Height (m)		~ 1.4	

Image Description	Looking Downstream
Date	28th October 2015
Source	BCC Asset Management Records

Image Description	Looking Upstream
Date	28th October 2015
Source	BCC Asset Management Records

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				< 2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	309.0	21.5	8.72	8.30	0.43	3.3	1.8	N/A
0.2	260.4	21.0	8.61	8.20	0.41	3.2	1.7	90
1	185.9	20.0	8.42	8.05	0.37	3.1	1.5	90
2	166.4	19.8	8.37	8.00	0.36	3.1	1.5	90
5	141.8	19.8	8.29	7.95	0.35	3.0	1.4	90
10	126.2	19.7	8.24	7.91	0.34	3.0	1.4	90
20	117.3	19.5	8.22	7.88	0.33	3.0	1.3	90
50	84.4	19.2	8.11	7.78	0.32	2.9	1.2	90

[1] *Flow underneath the road and only for 1D structures*

[2] *Measured at centre-span of bridge or at centre of culvert*

[3] *This is afflux at peak water level*

[4] *(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5] *(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6] *Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7] *Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Western Freeway

BCC Asset ID	N/A	Tributary Name	Cubberla Creek
Owner	DTMR	AMTD	2718
Year of Construction	Circa 1981	Coordinates (GDA94)	E 495588, N 6956316
Year of Significant Modification	Circa 1999 – bikeway bridge added	Hydraulic Model ID	S3
Source of Structure Information	DTMR design drawings + creek survey (1995 & 2016) + 2014 ALS	Flood Model Representation	1d bridge / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\1_Cubberla\AMTD 2720 - Western Freeway		

Structure Description		Single span concrete bridge	
Bridges		**Culverts**	
Number of Spans	1	Number of Barrels	N/A
Number of Piers in Waterway	N/A	Dimensions (m)	N/A
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	7.75	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		~ 42.8	
Span Length (m)		13.56 with allowance for 30 degree skew	
Lowest Level of Deck Soffit (m AHD)		10.29	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 11 (on road at structure)	
Average Handrail Height (m)		N/A – numerous barriers	

Image Description	Looking Upstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				5-yr ARI (20 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	316.6	192.8	14.09	10.84	3.25	6.4	N/A	N/A
0.2	265.7	176.9	13.49	10.76	2.74	5.9	N/A	90
1	189.8	152.9	12.66	10.62	2.05	5.1	N/A	90
2	170.0	143.7	12.34	10.54	1.81	4.8	N/A	90
5	129.0	129.0	11.88	10.42	1.45	4.3	N/A	90
10	118.5	118.5	11.47	10.33	1.14	3.9	N/A	90
20	110.0	110.0	10.75	10.30	0.46	3.9	N/A	90
50	77.8	77.8	10.24	10.03	0.22	2.9	N/A	90

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Garaboo Street Footbridge

BCC Asset ID	B0810	Tributary Name	Cubberla Creek
Owner	BCC	AMTD	3075
Year of Construction	Circa 1981	Coordinates (GDA94)	E 495270, N 6956427
Year of Significant Modification	N/A	Hydraulic Model ID	S4
Source of Structure Information	1996 HEC2 + onsite measurements + creek survey (1995)	Flood Model Representation	1d bridge / 1d weir
Link to Data Source	N/A		

Structure Description		Single span concrete bridge	
Bridges		**Culverts**	
Number of Spans	1	Number of Barrels	N/A
Number of Piers in Waterway	N/A	Dimensions (m)	N/A
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	8.31	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		~ 2.1	
Span Length (m)		13.68	
Lowest Level of Deck Soffit (m AHD)		11.18	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 11.35 (on floodplain)	
Average Handrail Height (m)		~ 1.3	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				Bridge: 2-yr ARI (50 % AEP) Floodplain: < 2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	332.7	73.3	14.22	14.11	0.11	2.7	2.3	N/A
0.2	276.4	77.1	13.66	13.53	0.13	2.8	2.2	90
1	195.8	76.4	12.88	12.74	0.14	2.8	1.8	90
2	173.8	77.0	12.59	12.44	0.14	2.8	1.6	90
5	149.4	77.7	12.22	12.04	0.18	2.8	1.3	90
10	129.5	76.3	12.02	11.80	0.22	2.8	1.0	90
20	111.7	73.2	11.79	11.65	0.14	2.7	0.6	90
50	79.6	61.7	11.59	11.55	0.04	2.3	0	90

[1]Flow underneath the road and only for 1D structures

[2]Measured at centre-span of bridge or at centre of culvert

[3]This is afflux at peak water level

[4](i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening

[5](i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model

[6]Velocities provided here are approximate only and the model should be interrogated for design purposes.

[7]Based on peak water level

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Akuna Street Footbridge

BCC Asset ID	B1250	Tributary Name	Cubberla Creek
Owner	BCC	AMTD	3297
Year of Construction	Circa 1979	Coordinates (GDA94)	E 495086, N 6956468
Year of Significant Modification	N/A	Hydraulic Model ID	S5
Source of Structure Information	Design drawings + onsite measurements + creek survey (circa 2011)	Flood Model Representation	1d bridge / 1d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\1_Cubberla\AMTD 3300 - Akuna St		

Structure Description		Single span concrete bridge	
Bridges		**Culverts**	
Number of Spans	1	Number of Barrels	N/A
Number of Piers in Waterway	N/A	Dimensions (m)	N/A
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	10.08	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		~ 2.1	
Span Length (m)		11.90	
Lowest Level of Deck Soffit (m AHD)		12.35	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 12.00 (on floodplain)	
Average Handrail Height (m)		~ 1.0	

Image Description	Looking Downstream
Date	13th August 2013
Source	BCC Asset Management Records

Image Description	Looking Upstream
Date	9th February 2016
Source	BCC Asset Management Records

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				Bridge: 2-yr ARI (50 % AEP) Floodplain: < 2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	327.3	36.9	14.29	14.20	0.09	2.4	2.0	N/A
0.2	263.9	34.9	13.70	13.59	0.11	2.3	1.7	90
1	182.3	35.9	12.96	12.98	-0.02	2.4	1.0	90
2	159.9	35.7	12.79	12.83	-0.04	2.4	0.9	90
5	138.4	37.1	12.68	12.73	-0.04	2.4	0.7	60
10	117.0	37.3	12.61	12.66	-0.05	2.4	0.6	90
20	100.2	37.1	12.55	12.60	-0.05	2.4	0.5	90
50	71.1	35.1	12.46	12.50	-0.04	2.4	0	90

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Henry Street Footbridge

BCC Asset ID	B0960	Tributary Name	Cubberla Creek
Owner	BCC	AMTD	3888
Year of Construction	Unknown	Coordinates (GDA94)	E 495251, N 6956958
Year of Significant Modification	N/A	Hydraulic Model ID	S6
Source of Structure Information	Detailed survey (circa 2011) + onsite measurements	Flood Model Representation	1d bridge / 1d weir
Link to Data Source	N/A		

Structure Description		Single span wooden bridge	
Bridges		**Culverts**	
Number of Spans	1	Number of Barrels	N/A
Number of Piers in Waterway	N/A	Dimensions (m)	N/A
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	12.11	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		~ 1.8	
Span Length (m)		11.7	
Lowest Level of Deck Soffit (m AHD)		14.75	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 14.47 (adjacent bridge)	
Average Handrail Height (m)		~ 1.1	

Image Description	Looking Downstream
Date	20th April 2016
Source	BCC Asset Management Records

Image Description	Looking Upstream
Date	20th April 2016
Source	BCC Asset Management Records

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				< 2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	286.7	55.9	16.23	15.76	0.47	3.6	2.1	N/A
0.2	245.4	54.9	16.10	15.63	0.47	3.5	2.1	60
1	166.5	51.0	15.76	15.34	0.42	3.3	1.8	60
2	145.2	49.6	15.65	15.26	0.40	3.2	1.7	60
5	122.5	47.9	15.52	15.15	0.37	3.1	1.6	60
10	104.2	46.8	15.41	15.05	0.36	3.0	1.5	60
20	88.0	45.3	15.30	14.96	0.34	2.9	1.4	90
50	63.4	40.2	15.12	14.81	0.31	2.7	1.2	60

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Moggill Road Culvert (downstream)

BCC Asset ID	N/A	Tributary Name	Cubberla Creek
Owner	DTMR	AMTD	4336
Year of Construction	1969	Coordinates (GDA94)	E 495091, N 6957354
Year of Significant Modification	N/A	Hydraulic Model ID	S7
Source of Structure Information	Design drawings + onsite measurements + 2014 ALS	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\1_Cubberla\AMTD 4330 - Moggill Road Culvert		

Structure Description		Concrete box culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	1
Number of Piers in Waterway	N/A	Dimensions (m)	~ 7.92w x 5.38h
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	15.61
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	15.49
Structure Length (m) (in direction of flow)		25.9	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 21.55 (Moggill Road)	
Average Handrail Height (m)		~ 1.36	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking downstream towards culvert junction / entrance
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				10-yr ARI (10 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	279.7	181.6	22.32	19.02	3.30	8.3	N/A	N/A
0.2	239.7	178.2	22.18	18.88	3.30	7.7	N/A	60
1	162.7	153.3	21.49	18.56	2.93	6.2	N/A	60
2	141.3	140.2	21.18	18.42	2.76	6.0	N/A	60
5	119.1	119.1	20.66	18.25	2.41	5.7	N/A	60
10	101.0	101.0	20.19	18.09	2.10	5.5	N/A	60
20	87.7	87.7	19.81	17.96	1.86	5.2	N/A	60
50	61.0	61.0	19.01	17.65	1.36	4.7	N/A	60

[1] *Flow underneath the road and only for 1D structures*

[2] *Measured at centre-span of bridge or at centre of culvert*

[3] *This is afflux at peak water level*

[4] *(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5] *(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6] *Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7] *Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Moggill Road Culvert (upstream)

BCC Asset ID	N/A	Tributary Name	Cubberla Creek
Owner	-	AMTD	4376
Year of Construction	1983	Coordinates (GDA94)	E 495100, N 6957396
Year of Significant Modification	N/A	Hydraulic Model ID	S8
Source of Structure Information	1996 HEC2 & HSRS	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Old Models\HEC2		

Structure Description		Concrete box culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	2
Number of Piers in Waterway	N/A	Dimensions (m)	~ 3.66w x 3.34h
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	16.60
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	16.26
Structure Length (m) (in direction of flow)		56.2	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 21.55 (Moggill Road)	
Average Handrail Height (m)		-	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	279.7	102.6	22.67	22.32	0.35	6.8	N/A	N/A
0.2	239.7	102.2	22.52	22.18	0.34	6.8	N/A	60
1	162.7	102.6	21.96	21.49	0.46	6.7	N/A	60
2	141.3	102.3	21.66	21.18	0.48	6.8	N/A	60
5	119.1	99.2	21.19	20.66	0.54	6.7	N/A	60
10	101.0	96.5	20.72	20.19	0.53	6.4	N/A	60
20	87.7	86.2	20.50	19.81	0.68	4.9	N/A	60
50	61.0	61.0	19.66	19.01	0.64	4.4	N/A	60

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Bulk Water Mains #1

BCC Asset ID	N/A	Tributary Name	Cubberla Creek
Owner	Seqwater	AMTD	4968
Year of Construction	1947	Coordinates (GDA94)	E 494631, N 6957622
Year of Significant Modification	1964 second pipe installed	Hydraulic Model ID	S9
Source of Structure Information	BCC records + 1996 HSRS + creek survey (1995)	Flood Model Representation	1d bridge / 1d weir
Link to Data Source	N/A		

Structure Description		2 x bulk water mains	
Bridges		**Culverts**	
Number of Spans	Multiple	Number of Barrels	N/A
Number of Piers in Waterway	Multiple	Dimensions (m)	N/A
Pier shape and Width (m)	Rectangular	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	20.3	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		~ 6.4	
Span Length (m)		Unknown	
Lowest Level of Deck Soffit (m AHD)		22.24	
Lowest Level of Weir/Road (m AHD) (not including handrail)		24.01	
Average Handrail Height (m)		N/A	

Image Description	Looking from west to east
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				50-yr ARI (2 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	171.7	93.3	24.73	23.57	1.16	4.8	1.4	N/A
0.2	147.6	95.1	24.60	23.42	1.18	4.8	1.3	60
1	103.0	92.9	24.13	22.99	1.14	4.7	0.6	60
2	93.0	89.8	23.90	22.88	1.02	4.6	0	60
5	79.7	78.5	23.41	22.74	0.68	4.0	0	60
10	66.1	65.8	23.00	22.59	0.40	3.4	0	60
20	57.5	57.5	22.75	22.49	0.27	2.9	0	60
50	41.5	41.5	22.39	22.27	0.11	2.1	0	60

[1] *Flow underneath the road and only for 1D structures*

[2] *Measured at centre-span of bridge or at centre of culvert*

[3] *This is afflux at peak water level*

[4] *(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5] *(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6] *Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7] *Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Tristania Road Culvert

BCC Asset ID	C0259B	Tributary Name	Cubberla Creek
Owner	BCC	AMTD	5006
Year of Construction	1968	Coordinates (GDA94)	E 494610, N 6957652
Year of Significant Modification	N/A	Hydraulic Model ID	S10
Source of Structure Information	1996 HEC2 & HSRS + onsite measurements	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Old Models\HEC2		

Structure Description		Concrete box culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	1
Number of Piers in Waterway	N/A	Dimensions (m)	~ 3.05w x 3.01h
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	20.22
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	19.94
Structure Length (m) (in direction of flow)		9.15	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 23.47	
Average Handrail Height (m)		1.2	

Image Description	Looking Downstream
Date	9th December 2013
Source	BCC Asset Management Records

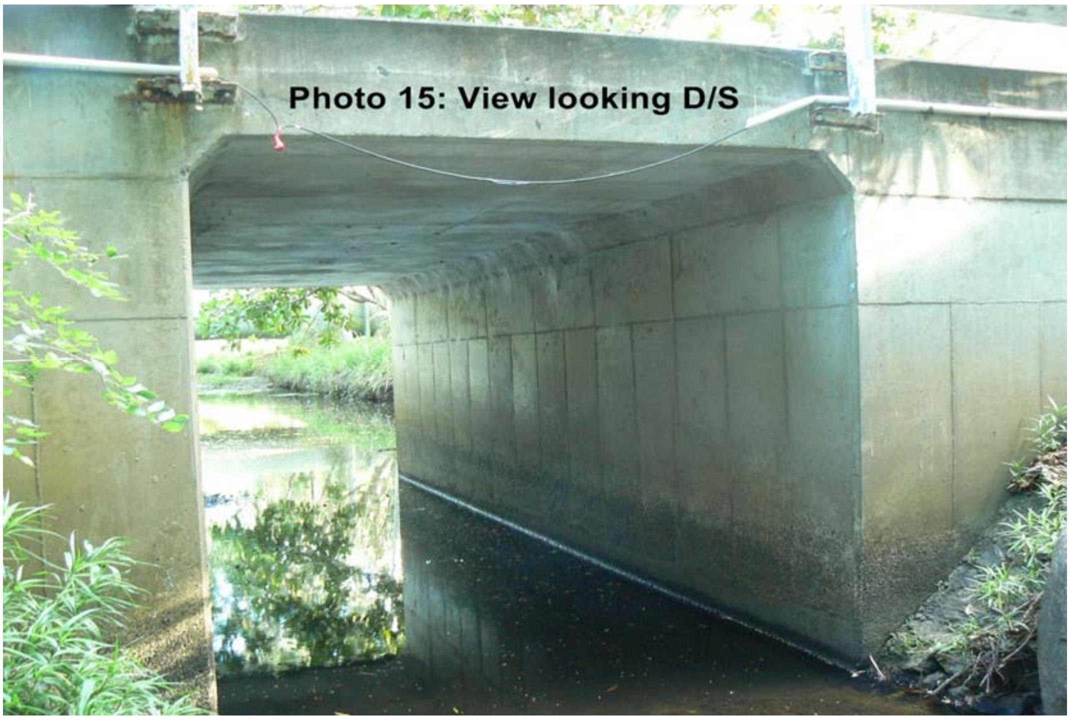

Image Description	Looking Upstream
Date	9th December 2013
Source	BCC Asset Management Records

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				< 2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	187.8	41.4	25.50	24.75	0.75	7.5	2.4	N/A
0.2	165.2	41.8	25.36	24.62	0.75	7.6	2.3	60
1	117.1	41.5	25.02	24.16	0.85	7.5	2.1	60
2	103.7	41.4	24.91	23.94	0.96	7.5	2.0	60
5	86.3	40.5	24.78	23.48	1.30	7.3	2.0	60
10	70.6	39.5	24.65	23.08	1.57	7.2	1.9	60
20	59.9	38.7	24.54	22.85	1.69	7.0	1.8	60
50	41.6	36.3	24.24	22.48	1.75	6.6	1.5	60

[1] *Flow underneath the road and only for 1D structures*

[2] *Measured at centre-span of bridge or at centre of culvert*

[3] *This is afflux at peak water level*

[4] *(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5] *(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6] *Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7] *Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Chapel Hill State School Culvert

BCC Asset ID	N/A	Tributary Name	Cubberla Creek
Owner	QLD State Government	AMTD	5692
Year of Construction	Unknown	Coordinates (GDA94)	E 494515, N 6958207
Year of Significant Modification	N/A	Hydraulic Model ID	S13
Source of Structure Information	1996 HEC2 & HSRS + 2014 ALS	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Old Models\HEC2		

Structure Description		Concrete box culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	4
Number of Piers in Waterway	N/A	Dimensions (m)	2.4w x 1.8h
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	25.46
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	25.39
Structure Length (m) (in direction of flow)		13.45	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 28.03	
Average Handrail Height (m)		~ 1.2	

Image Description	Looking Downstream
Date	5th January 2017
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	5th January 2017
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	184.9	78.3	29.45	27.91	1.54	7.55	N/A	N/A
0.2	163.5	76.4	29.31	27.85	1.46	7.37	N/A	60
1	112.7	71.8	28.98	27.66	1.33	6.92	N/A	60
2	99.2	69.2	28.81	27.59	1.22	6.68	N/A	60
5	83.2	66.9	28.66	27.51	1.15	6.45	N/A	60
10	68.0	62.7	28.40	27.38	1.02	6.04	N/A	60
20	57.9	57.6	28.11	27.26	0.85	5.55	N/A	60
50	43.8	43.8	27.70	27.05	0.65	3.39	N/A	60

[1] *Flow underneath the road and only for 1D structures*

[2] *Measured at centre-span of bridge or at centre of culvert*

[3] *This is afflux at peak water level*

[4] *(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5] *(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6] *Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7] *Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Goolman Street Culvert

BCC Asset ID	C0699B	Tributary Name	Cubberla Creek
Owner	BCC	AMTD	5937
Year of Construction	1976	Coordinates (GDA94)	E 494355, N 6958389
Year of Significant Modification	N/A	Hydraulic Model ID	S14
Source of Structure Information	Design drawings	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\1_Cubberla\AMTD 5930 - Goolman Street Piped Drainage		

Structure Description		Concrete box culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	4
Number of Piers in Waterway	N/A	Dimensions (m)	3 / 3.05w x 1.22h 1 / 1.83w x 1.22h (U/S) 1 / 3.05w x 1.22h (D/S)
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	27.27
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	27.24
Structure Length (m) (in direction of flow)		20.9	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 29.72	
Average Handrail Height (m)		~ 0.75 (Armco barrier)	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	16th April 2010
Source	BCC Asset Management Records

Link to Flood Model Results	[G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES](G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES)
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	171.4	51.4	31.39	30.55	0.84	3.5	N/A	N/A
0.2	149.8	53.9	31.28	30.37	0.91	4.2	N/A	60
1	101.3	52.3	30.97	30.00	0.97	3.7	N/A	60
2	88.4	51.7	30.87	29.89	0.99	3.5	N/A	60
5	73.4	50.2	30.75	29.73	1.01	3.5	N/A	60
10	60.6	47.9	30.57	29.57	1.00	3.4	N/A	60
20	49.6	45.4	30.35	29.43	0.93	3.3	N/A	60
50	34.9	34.9	29.58	29.01	0.57	2.6	N/A	60

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Dillingen Street Culvert

BCC Asset ID	C0004B	Tributary Name	Cubberla Creek
Owner	BCC	AMTD	6512
Year of Construction	1989	Coordinates (GDA94)	E 494114, N 6958884
Year of Significant Modification	N/A	Hydraulic Model ID	S18
Source of Structure Information	Design drawings + 2014 ALS	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\1_Cubberla\AMTD 6510 - Dillingen Street Culvert		

Structure Description		Concrete box culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	4
Number of Piers in Waterway	N/A	Dimensions (m)	3 / 2.7w x 1.8h 1 / 3.0w x 2.64h
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	34.48 (3 cells) 33.88 (1 cell)
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	34.14 (3 cells) 33.54 (1 cell)
Structure Length (m) (in direction of flow)		23.18	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 37.05	
Average Handrail Height (m)		~ 1.2	

Image Description	Looking Downstream
Date	9th December 2013
Source	BCC Asset Management Records

Image Description	Looking Upstream
Date	9th December 2013
Source	BCC Asset Management Records

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				20-yr ARI (5 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	149.9	96.9	38.17	36.34	1.83	7.3	N/A	N/A
0.2	130.3	94.2	38.01	36.20	1.81	7.1	N/A	60
1	88.7	84.6	37.53	35.87	1.66	6.4	N/A	60
2	77.6	77.6	37.21	35.75	1.46	5.9	N/A	60
5	65.1	65.1	36.77	35.61	1.16	4.9	N/A	60
10	54.1	54.1	36.47	35.48	1.00	4.1	N/A	60
20	46.2	46.2	36.26	35.37	0.89	3.9	N/A	60
50	33.2	33.2	35.87	35.17	0.70	3.6	N/A	60

[1] *Flow underneath the road and only for 1D structures*

[2] *Measured at centre-span of bridge or at centre of culvert*

[3] *This is afflux at peak water level*

[4] *(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5] *(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6] *Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7] *Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Greenford Street Culvert

BCC Asset ID	C0405P	Tributary Name	Cubberla Creek
Owner	BCC	AMTD	Outside current extents
Year of Construction	1988	Coordinates (GDA94)	E 494082, N 6959996
Year of Significant Modification	N/A	Hydraulic Model ID	S19
Source of Structure Information	Design drawings + onsite measurements + 2014 ALS	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\1_Cubberla\Greenford Street Culvert and Piped Drainage		

Structure Description		Concrete Pipe Culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	1
Number of Piers in Waterway	N/A	Dimensions (m)	1.8 diameter
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	56.47
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	56.34
Structure Length (m) (in direction of flow)		~ 24.4	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 59.7	
Average Handrail Height (m)		~ 0.75 (Armco barrier)	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	[G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES](G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES)
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				50-yr ARI (2 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	27.7	14.2	60.88	57.19	3.70	5.6	N/A	N/A
0.2	21.6	13.7	60.72	57.17	3.55	5.4	N/A	60
1	11.5	11.4	59.92	57.10	2.82	4.5	N/A	60
2	9.5	9.5	59.38	57.03	2.35	3.7	N/A	60
5	6.9	6.9	58.79	56.95	1.84	3.4	N/A	60
10	4.7	4.7	58.15	56.86	1.29	2.9	N/A	60
20	3.2	3.2	57.73	56.78	0.95	2.1	N/A	60
50	1.5	1.5	57.33	56.65	0.67	2.8	N/A	90

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

St. James Estate Access Culvert

BCC Asset ID	N/A	Tributary Name	Boblynne St. Branch
Owner	Private	AMTD	20
Year of Construction	Unknown	Coordinates (GDA94)	E 495036, N 6957548
Year of Significant Modification	N/A	Hydraulic Model ID	S20
Source of Structure Information	1996 HEC2 & HSRS + onsite measurements + 2014 ALS	Flood Model Representation	1d culvert / 1d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Old Models\HEC2		

Structure Description		Concrete box culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	2
Number of Piers in Waterway	N/A	Dimensions (m)	3.34w x 3.05h
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	17.93
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	17.82
Structure Length (m) (in direction of flow)		~ 12	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 21.37	
Average Handrail Height (m)		Wall height varies	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking along access road from east to west
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	[G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES]()
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				20-yr ARI (5 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05	86.7	48.6	22.83	22.79	0.04	2.4	1.4	N/A
0.2	83.1	68.7	22.66	22.63	0.04	4.4	1.5	60
1	60.2	58.3	22.08	22.04	0.04	4.1	1.1	60
2	52.4	51.5	21.80	21.76	0.05	3.9	0.9	60
5	44.9	44.9	21.34	21.31	0.04	3.8	0	60
10	37.3	37.3	20.89	20.86	0.02	3.6	0	60
20	31.2	31.2	20.65	20.64	0.02	3.5	0	60
50	22.5	22.5	19.95	19.94	0.01	2.7	0	60

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Bulk Water Mains #2

BCC Asset ID	N/A	Tributary Name	Boblynne St. Branch
Owner	Seqwater	AMTD	330
Year of Construction	1947	Coordinates (GDA94)	E 494920, N 6957813
Year of Significant Modification	1964 second pipe installed	Hydraulic Model ID	S21
Source of Structure Information	BCC records + 1996 HSRS + creek survey (1995)	Flood Model Representation	1d bridge / 1d weir
Link to Data Source	N/A		

Structure Description		2 x bulk water mains	
Bridges		**Culverts**	
Number of Spans	Multiple	Number of Barrels	N/A
Number of Piers in Waterway	Multiple	Dimensions (m)	N/A
Pier shape and Width (m)	Rectangular	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	20.83	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		~ 6.4	
Span Length (m)		Unknown	
Lowest Level of Deck Soffit (m AHD)		22.74	
Lowest Level of Weir/Road (m AHD) (not including handrail)		24.50	
Average Handrail Height (m)		N/A	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking from east to west
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				> 100-yr ARI (1 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05[8]	81.4	81.4	23.29	23.14	0.15	2.4	0	N/A
0.2	86.1	86.1	23.09	22.98	0.11	2.4	0	60
1	60.8	65.8	22.52	22.51	0.01	2.6	0	60
2	52.5	53.3	22.42	22.41	0.01	2.5	0	60
5	44.3	44.3	22.30	22.29	0.01	2.2	0	60
10	36.1	36.1	22.16	22.15	0.01	2.1	0	60
20	30.7	30.7	22.07	22.06	0.01	2.0	0	60
50	22.2	22.2	21.89	21.88	0.01	1.7	0	60

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

[8]*In areas with a small upstream catchment, the 2000-yr ARI (0.05 % AEP) super-storm method does not always produce a flow greater than the 500-yr ARI (0.2 % AEP)*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Alana Circuit Culvert

BCC Asset ID	C0107P	Tributary Name	Boblynne St. Branch
Owner	BCC	AMTD	Outside current extents
Year of Construction	1985	Coordinates (GDA94)	E 494838, N 6958412
Year of Significant Modification	N/A	Hydraulic Model ID	S22
Source of Structure Information	Design drawings + 2014 ALS	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\5_Boblynne\Alana Ct Culvert		

Structure Description		Concrete pipe culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	2
Number of Piers in Waterway	N/A	Dimensions (m)	1.65 diameter
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	28.32
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	27.66
Structure Length (m) (in direction of flow)		~ 44.3	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 30.4	
Average Handrail Height (m)		None	

Image Description	Looking Downstream
Date	6th September 2016
Source	Creek Survey

Image Description	Looking Upstream
Date	6th September 2016
Source	Creek Survey

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				< 2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05[8]	63.8	20.6	31.54	29.78	1.76	4.8	N/A	N/A
0.2	71.7	20.8	31.61	29.88	1.72	4.9	N/A	60
1	49.3	20.1	31.36	29.54	1.83	4.7	N/A	60
2	42.6	19.8	31.27	29.41	1.87	4.6	N/A	60
5	35.9	19.3	31.18	29.27	1.91	4.5	N/A	60
10	29.5	18.7	31.07	29.11	1.96	4.4	N/A	60
20	25.2	18.2	30.98	28.99	1.99	4.3	N/A	60
50	18.8	17.0	30.78	28.79	1.99	4.0	N/A	60

[1] *Flow underneath the road and only for 1D structures*

[2] *Measured at centre-span of bridge or at centre of culvert*

[3] *This is afflux at peak water level*

[4] *(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5] *(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6] *Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7] *Based on peak water level*

[8] *In areas with a small upstream catchment, the 2000-yr ARI (0.05 % AEP) super-storm method does not always produce a flow greater than the 500-yr ARI (0.2 % AEP)*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Cedar Xing Culvert

BCC Asset ID	C0231P	Tributary Name	Gubberley Creek
Owner	BCC	AMTD	Outside current extents
Year of Construction	Unknown	Coordinates (GDA94)	E 494465, N 6956970
Year of Significant Modification	N/A	Hydraulic Model ID	S23
Source of Structure Information	BCC records + creek survey (2016) + 2014 ALS	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\4_Gubberley Ck\Cedar Xing		

Structure Description		Concrete pipe culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	2
Number of Piers in Waterway	N/A	Dimensions (m)	1.65 diameter
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	20.94
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	20.90
Structure Length (m) (in direction of flow)		~ 16	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 23.3	
Average Handrail Height (m)		None	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				20-yr ARI (5 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05[8]	24.0	17.7	23.66	22.49	1.17	4.2	N/A	N/A
0.2	31.8	19.5	23.83	22.68	1.15	4.6	N/A	60
1	20.9	17.1	23.56	22.42	1.14	4.0	N/A	60
2	17.1	16.0	23.43	22.31	1.12	3.7	N/A	60
5	13.4	13.4	23.14	22.16	0.98	3.1	N/A	60
10	10.8	10.8	22.76	22.03	0.73	2.6	N/A	90
20	8.9	8.9	22.54	21.93	0.61	2.4	N/A	90
50	4.8	4.8	22.06	21.68	0.38	1.9	N/A	90

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

[8]*In areas with a small upstream catchment, the 2000-yr ARI (0.05 % AEP) super-storm method does not always produce a flow greater than the 500-yr ARI (0.2 % AEP)*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Katunga Street Culvert

BCC Asset ID	C2503P	Tributary Name	Akuna Street Branch
Owner	BCC	AMTD	62
Year of Construction	Unknown	Coordinates (GDA94)	E 495101, N 6956352
Year of Significant Modification	N/A	Hydraulic Model ID	S25
Source of Structure Information	1996 HEC2 & HSRS + 2014 ALS	Flood Model Representation	1d culvert / 1d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Old Models\HEC2		

Structure Description		Concrete pipe culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	2
Number of Piers in Waterway	N/A	Dimensions (m)	1.5 diameter
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	11.05
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	10.85
Structure Length (m) (in direction of flow)		~ 5	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 12.55 (at structure)	
Average Handrail Height (m)		~ 1.2	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				< 2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05[8]	48.4	12.3	14.30	14.30	0.00	3.5	1.4	N/A
0.2	63.7	13.8	13.34	13.12	0.21	3.9	1.5	30
1	44.6	12.3	13.17	12.66	0.50	3.5	1.3	30
2	38.3	11.8	13.10	12.61	0.49	3.3	1.2	30
5	32.9	11.3	13.05	12.57	0.48	3.2	1.2	30
10	27.0	10.8	12.99	12.46	0.53	3.1	1.1	60
20	22.8	10.4	12.94	12.36	0.58	2.9	1.0	60
50	15.3	9.5	12.83	12.25	0.58	2.7	0.9	60

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

[8]*In areas with a small upstream catchment, the 2000-yr ARI (0.05 % AEP) super-storm method does not always produce a flow greater than the 500-yr ARI (0.2 % AEP)*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Marshall Lane Culvert

BCC Asset ID	C0294P	Tributary Name	Akuna Street Branch
Owner	BCC	AMTD	Outside current extents
Year of Construction	Unknown	Coordinates (GDA94)	E 494678, N 6956245
Year of Significant Modification	N/A	Hydraulic Model ID	S26
Source of Structure Information	BCC Records + 2014 ALS	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\3_Akuna Trib\Marshall Lane Culvert		

Structure Description		Concrete pipe culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	1
Number of Piers in Waterway	N/A	Dimensions (m)	1.5 diameter
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	19.86
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	19.71
Structure Length (m) (in direction of flow)		~ 22.8	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 23.7 (at structure)	
Average Handrail Height (m)		~ 0.75 (Armco barrier)	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Looking Upstream	Looking Upstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				< 2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05[8]	37.5	10.5	24.77	22.17	2.59	6.0	N/A	N/A
0.2	51.9	10.7	24.93	22.44	2.49	6.1	N/A	30
1	36.4	10.5	24.76	22.16	2.60	6.0	N/A	30
2	30.8	10.4	24.69	22.06	2.63	5.9	N/A	30
5	27.2	10.3	24.60	21.96	2.64	5.9	N/A	30
10	22.7	10.2	24.47	21.84	2.63	5.8	N/A	30
20	18.9	10.1	24.37	21.73	2.64	5.7	N/A	30
50	12.6	9.8	24.10	21.52	2.58	5.5	N/A	30

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

[8]*In areas with a small upstream catchment, the 2000-yr ARI (0.05 % AEP) super-storm method does not always produce a flow greater than the 500-yr ARI (0.2 % AEP)*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Fig Tree Pocket Road Culvert

BCC Asset ID	C2123P	Tributary Name	Tributary C
Owner	BCC	AMTD	Outside current extents
Year of Construction	circa 1982	Coordinates (GDA94)	E 495594, N 6956266
Year of Significant Modification	N/A	Hydraulic Model ID	S27
Source of Structure Information	DTMR design drawings + 2014 ALS	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\2_Trib C - Centenary Hwy\Culverts 1 to 3		

Structure Description		Concrete pipe culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	2
Number of Piers in Waterway	N/A	Dimensions (m)	1.8 diameter
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	8.90
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	8.82
Structure Length (m) (in direction of flow)		~ 38.4	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 16.9 (at intersection with Off ramp)	
Average Handrail Height (m)		N/A	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				> 100-yr ARI (1 % AEP)				
AEP (%)	Total Discharge (m^3/s)[8]	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05[9]	56.7	30.1	14.08	10.74	3.34	5.9	0	N/A
0.2	71.6	27.6	13.46	10.66	2.80	5.4	0	90
1	48.1	21.9	12.39	10.51	1.88	4.3	0	90
2	41.4	17.8	11.79	10.41	1.37	3.5	0	60
5	36.0	17.4	11.73	10.34	1.39	3.4	0	60
10	29.8	16.9	11.67	10.25	1.42	3.3	0	60
20	25.0	15.9	11.55	10.21	1.35	3.1	0	60
50	19.7	13.7	11.31	9.94	1.37	3.5	0	60

[1]Flow underneath the road and only for 1D structures

[2]Measured at centre-span of bridge or at centre of culvert

[3]This is afflux at peak water level

[4](i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening

[5](i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model

[6]Velocities provided here are approximate only and the model should be interrogated for design purposes.

[7]Based on peak water level

[8]Based on total discharge upstream of "On Ramp"

[9]In areas with a small upstream catchment, the 2000-yr ARI (0.05 % AEP) super-storm method does not always produce a flow greater than the 500-yr ARI (0.2 % AEP)

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Western Freeway Off Ramp

BCC Asset ID	C3043P	Tributary Name	Tributary C
Owner	BCC	AMTD	Outside current extents
Year of Construction	circa 1982	Coordinates (GDA94)	E 495560, N 6956246
Year of Significant Modification	1999 (culvert extended)	Hydraulic Model ID	S28
Source of Structure Information	DTMR design drawings + 2014 ALS	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\2_Trib C - Centenary Hwy\Culverts 1 to 3		

Structure Description		Concrete pipe culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	2
Number of Piers in Waterway	N/A	Dimensions (m)	1.8 diameter
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	9.08
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	8.99
Structure Length (m) (in direction of flow)		~ 26.4	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 12.1	
Average Handrail Height (m)		~ 0.75 (Armco barrier)	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	9th December 2013
Source	BCC Asset Management Records

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)[8]	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05[9]	56.7	15.9	14.10	14.09	0.01	3.1	N/A	N/A
0.2	71.6	16.2	13.49	13.47	0.02	3.2	N/A	90
1	48.1	15.8	12.69	12.40	0.29	3.1	N/A	60
2	41.4	15.7	12.64	11.81	0.83	3.1	N/A	60
5	36.0	15.5	12.57	11.76	0.81	3.1	N/A	60
10	29.8	15.3	12.48	11.70	0.77	3.0	N/A	60
20	25.0	14.7	12.27	11.57	0.71	2.9	N/A	60
50	19.7	13.5	11.90	11.29	0.61	2.7	N/A	60

[1]Flow underneath the road and only for 1D structures

[2]Measured at centre-span of bridge or at centre of culvert

[3]This is afflux at peak water level

[4](i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening

[5](i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model

[6]Velocities provided here are approximate only and the model should be interrogated for design purposes.

[7]Based on peak water level

[8]Based on total discharge upstream of "On Ramp"

[9]In areas with a small upstream catchment, the 2000-yr ARI (0.05 % AEP) super-storm method does not always produce a flow greater than the 500-yr ARI (0.2 % AEP)

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Western Freeway On Ramp

BCC Asset ID	C0137P	Tributary Name	Tributary C
Owner	BCC	AMTD	Outside current extents
Year of Construction	circa 1982	Coordinates (GDA94)	E 495512, N 6956192
Year of Significant Modification	circa 1991 (relocated) circa 2005 (extended)	Hydraulic Model ID	S29
Source of Structure Information	DTMR design drawings + 2014 ALS	Flood Model Representation	1d culvert / 2d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\2_Trib C - Centenary Hwy\Culverts 1 to 3		

Structure Description		Concrete box culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	3
Number of Piers in Waterway	N/A	Dimensions (m)	1.5w x 1.2h
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	9.44
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	9.35
Structure Length (m) (in direction of flow)		~ 18	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Lowest Level of Weir/Road (m AHD) (not including handrail)		~ 12.05	
Average Handrail Height (m)		~ 0.75 (Armco barrier)	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				< 2-yr ARI (50 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05[8]	56.7	13.3	14.10	14.10	0.00	2.5	N/A	N/A
0.2	71.6	15.3	13.49	13.49	0.00	2.8	N/A	60
1	48.1	14.8	13.03	12.67	0.36	2.7	N/A	60
2	41.4	14.5	12.94	12.63	0.31	2.7	N/A	60
5	36.0	14.1	12.83	12.57	0.27	2.6	N/A	60
10	29.8	13.7	12.66	12.49	0.17	2.5	N/A	60
20	25.0	13.7	12.52	12.29	0.23	2.5	N/A	60
50	19.7	13.4	12.28	11.92	0.36	2.5	N/A	60

[1] *Flow underneath the road and only for 1D structures*

[2] *Measured at centre-span of bridge or at centre of culvert*

[3] *This is afflux at peak water level*

[4] *(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5] *(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6] *Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7] *Based on peak water level*

[8] *In areas with a small upstream catchment, the 2000-yr ARI (0.05 % AEP) super-storm method does not always produce a flow greater than the 500-yr ARI (0.2 % AEP)*

Hydraulic Structure Reference Sheet

Cubberla Creek Flood Study

Norman Street Footbridge

BCC Asset ID	N/A	Tributary Name	Tributary C
Owner	DTMR	AMTD	Outside current extents
Year of Construction	circa 2003	Coordinates (GDA94)	E 495217, N 6955995
Year of Significant Modification	N/A	Hydraulic Model ID	S30
Source of Structure Information	DTMR design drawings + creek survey (2016)	Flood Model Representation	1d bridge / 1d weir
Link to Data Source	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Data\Structures\2_Trib C - Centenary Hwy\Norman St & Musgrave St Bikeway Bridges		

Structure Description		Single span concrete footbridge	
Bridges		**Culverts**	
Number of Spans	1	Number of Barrels	N/A
Number of Piers in Waterway	N/A	Dimensions (m)	N/A
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	13.48	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		3.7	
Span Length (m)		12.1	
Lowest Level of Deck Soffit (m AHD)		15.6	
Lowest Level of Weir/Road (m AHD) (not including handrail)		16.09	
Average Handrail Height (m)		1.2	

Image Description	Looking Downstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Image Description	Looking Upstream
Date	26th October 2016
Source	Site inspection undertaken for flood study

Link to Flood Model Results	G:\BI\CD\Proj17\170300_Cubberla_Creek_Flood_Study\Flood Management\Calculations\Flood Management\Tuflow\results\S1_DES
Model Version Number	CCFS_~s~_~e1~_~e2~_025.tcf
Model Scenario	Scenario 1 Design (S1_DES)

Structure Flood Immunity (immunity of lowest point of weir above structure)				> 100-yr ARI (1 % AEP)				
AEP (%)	Total Discharge (m^3/s)	Discharge through Structure (m^3/s)[1]	U/S Peak Water Level (m AHD)[2]	D/S Peak Water Level (m AHD)[2]	Afflux (mm)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7]
0.05[8]	29.4	29.4	15.25	15.25	0.00	3.3	0	N/A
0.2	37.7	37.7	15.35	15.20	0.15	3.8	0	30
1	26.9	26.9	15.09	15.00	0.09	3.7	0	30
2	23.4	23.4	14.99	14.89	0.09	3.6	0	30
5	20.6	20.6	14.91	14.84	0.07	3.5	0	30
10	16.9	16.9	14.78	14.71	0.07	3.5	0	30
20	14.4	14.4	14.69	14.59	0.10	3.5	0	30
50	10.4	10.4	14.53	14.36	0.16	3.4	0	30

[1]*Flow underneath the road and only for 1D structures*

[2]*Measured at centre-span of bridge or at centre of culvert*

[3]*This is afflux at peak water level*

[4]*(i) Only for 1D structures. (ii) This is the peak of the depth/width averaged velocity within the structure opening*

[5]*(i) Only for 1D structures (ii) This is the peak of the depth/width averaged velocity across the 1D weir section of the model*

[6]*Velocities provided here are approximate only and the model should be interrogated for design purposes.*

[7]*Based on peak water level*

[8]*In areas with a small upstream catchment, the 2000-yr ARI (0.05 % AEP) super-storm method does not always produce a flow greater than the 500-yr ARI (0.2 % AEP)*

Appendix K: External Peer Review Documentation

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Our Ref: L.B20679.008.Cubberla_Creek.docx

2 June 2017

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

Attention: Scott Glover

Dear Scott

RE: CUBBERLA CREEK FLOOD MODELLING PEER REVIEW

Background

BMT WBM was commissioned by Council to undertake a peer review of the Cubberla Creek flood modelling prepared as part of the Cubberla 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:

- Hydrologic models (URBS);
- Hydraulic models including model output files (TUFLOW);
- GIS data; and
- Preliminary flood study reporting.

Generally, no concerns with the models were 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. It is noted that ARR1987 was used to compute the design storm events. This is justified by the fact that the study was well underway by the time ARR2016 was fully released.

ARR2016 climate change guidance has been adopted. This guidance recommends increases in rainfall intensity based on Representative Concentration Pathways (RCPs) with relative forcing values of 4.5 and 8.5. Projections are provided up to the year 2090. Therefore, Council has estimated the 2100 rainfall intensity increases by extrapolation from the years 2080 and 2090. The following rainfall intensity increases were adopted:

- Year 2050 RCP4.5 – 6.7%
- Year 2050 RCP8.5 – 8.8%
- Year 2100 RCP4.5 – 9.3%
- Year 2100 RCP8.5 – 21%

Hydraulic models of the creeks in the study area were developed using TUFLOW. A 4m 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 Cubberla Creek, from Fig Tree Pocket Park, 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. Generally, Council's acceptable tolerance for calibration is 0.15m variance for peak flood levels at stream gauges (there are no stream gauge records available for this study) and 0.3m variance for peak flood levels at maximum height gauges. Council has achieved this tolerance for the MHG gauge records that were available for this study.

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 Cubberla 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.

Appendix L: Modelling User Guide

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Cubberla Creek Flood Study

Model User Guide

Prepared by Brisbane City Council's, City Projects Office

June 2017

BRISBANE CITY

Dedicated to a better Brisbane

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Table of Contents

List of Tables

1.0 Introduction

1.1 Cubberla Creek Flood Study (2017)

This document is to be read in conjunction with the Cubberla Creek Flood Study - Volume 1 (2017).

The Cubberla Creek Flood Study (2017) 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 using both RCP4.5 and RCP8.5.

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\Cubberla\2016\Cubberla.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\Cubberla\2016\Calibration\May_2015

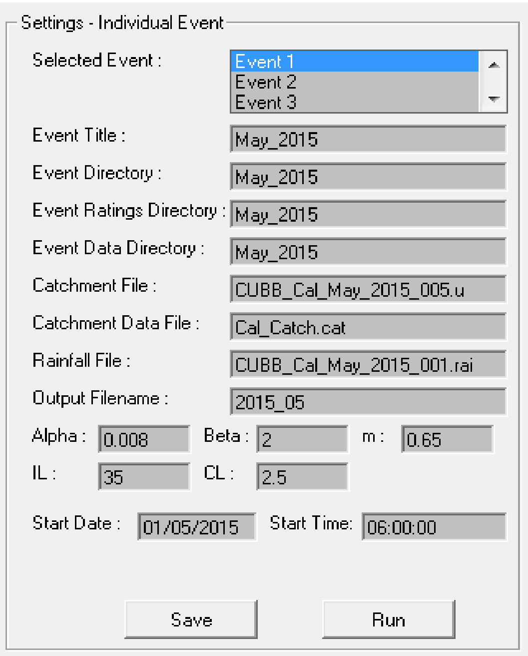
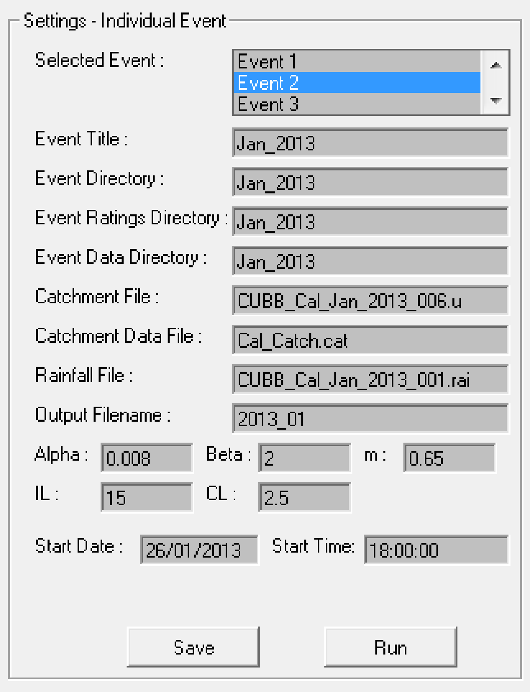


Figure 2.1: Event 1 (May 2015)

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\Cubberla\2016\Calibration\Jan_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\Cubberla\2016\Calibration\May_2009

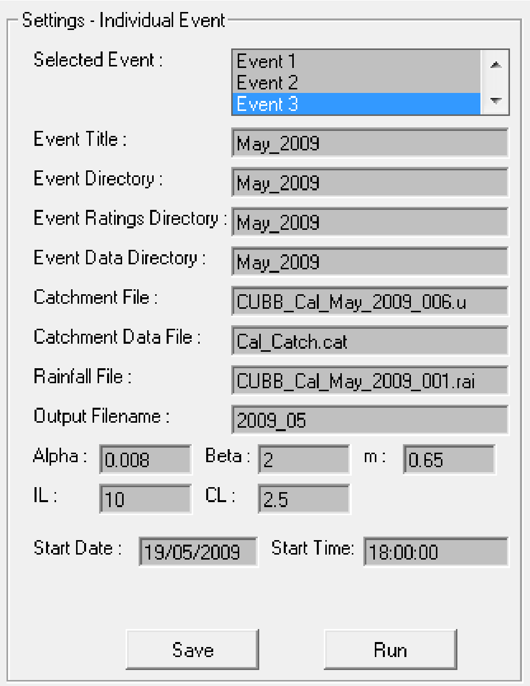


Figure 2.3: Event 3 (May 2009)

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\Cubberla\2016\Calibration\Nov_2008

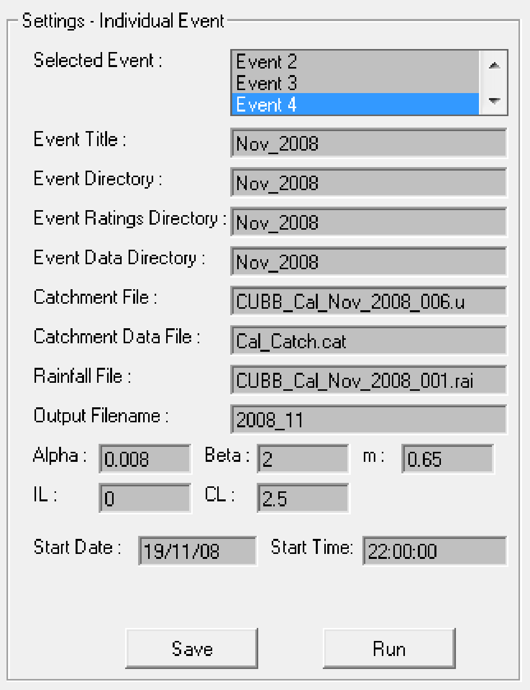


Figure 2.4: Event 4 (November 2008)

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\Cubberla\2016\Design

For the Climate Variability runs, replace "IFD1987.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) RCP4.5: IFD_1987_CC1_RCP4.5_6.7%_Centroid.ifd
- Climate Scenario 1 (2050) RCP8.5: IFD_1987_CC1_RCP8.5_8.8%_Centroid.ifd
- Climate Scenario 2 (2100) RCP4.5: IFD_1987_CC2_RCP4.5_9.2%_Centroid.ifd
- Climate Scenario 2 (2100) RCP8.5: IFD_1987_CC2_RCP8.5_21%_Centroid.ifd

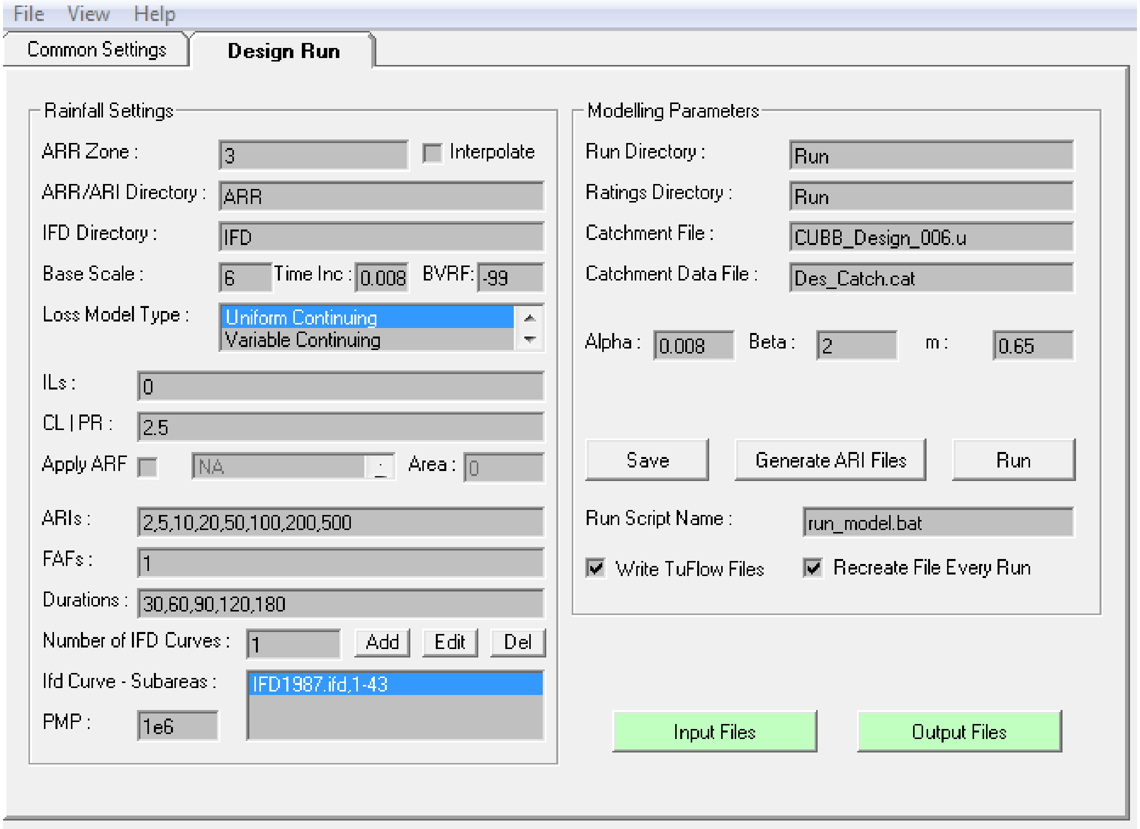


Figure 2.5: Design Run Settings – 2-yr to 500-yr ARI

In order to run the 2000-yr ARI and PMF events, the URBS Control Centre settings are as per Figure 2.6.

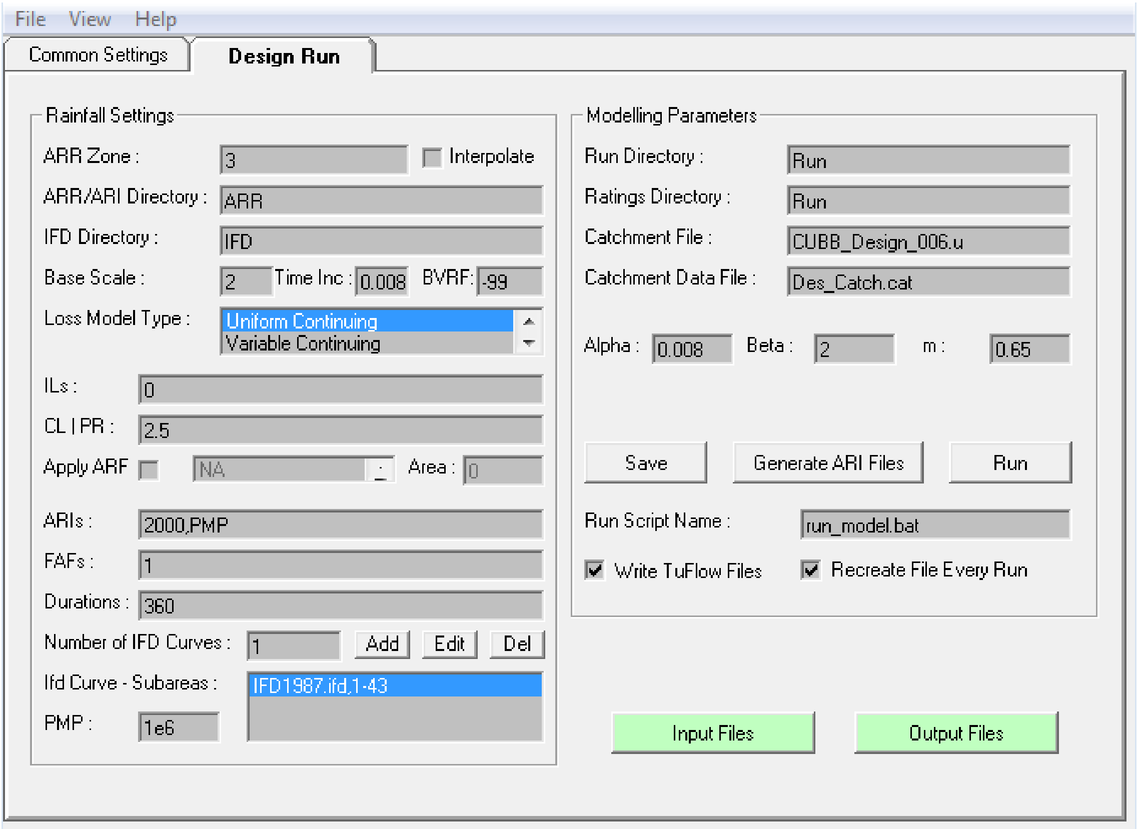


Figure 2.6: Design Run Settings – 2000-yr and PMF

2.2 Hydraulic Models

2.2.1 General

TUFLOW modelling was undertaken using build: 2016-03-AC-iSP-w64.

The TUFLOW modelling was undertaken using a single TUFLOW Control File (TCF), which was named: CCFS_~s~_~e1~_~e2~_025.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 CCFS_~s~_~e1~_~e2~_025.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 090m 120m 180m

Model Simulation	Scenario (~s~)	Event 1 (~e1~)	Event 2 (~e2~)
Design Events (Scenario 2)	S2_DES	100y	030m 060m 090m 120m 180m
Design Events (Scenario 3)	S3_DES	002y 005y 010y 020y 050y 100y	030m 060m 090m 120m 180m

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 CCFS_~s~_~e1~_~e2~_025.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 090m 120m 180m
	S1_EXT	2000y PMF	360m
Extreme Events (Scenario 3)	S3_EXT	200y 500y	030m 060m 090m 120m 180m

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 CCFS_~s~_~e1~_~e2~_025.tcf

2.2.5 TUFLOW Sensitivity Analysis Models

TUFLOW sensitivity simulations were undertaken for climate variability. 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 RCP4.5	S1_CC	100yCC1a 200yCC1a	030m 060m 090m 120m 180m
Climate Variability (Scenario 1) Planning horizon 2050 RCP8.5	S1_CC	100yCC1b 200yCC1b	030m 060m 090m 120m 180m
Climate Variability (Scenario 1) Planning horizon 2100 RCP4.5	S1_CC	100yCC2a 200yCC2a 500yCC2a	030m 060m 090m 120m 180m
Climate Variability (Scenario 1) Planning horizon 2100 RCP8.5	S1_CC	100yCC2b 200yCC2b 500yCC2b	030m 060m 090m 120m 180m
Climate Variability (Scenario 3) Planning horizon 2050 RCP4.5	S3_CC	100yCC1a	030m 060m 090m 120m 180m
Climate Variability (Scenario 3) Planning horizon 2050 RCP8.5	S3_CC	100yCC1b	030m 060m 090m 120m 180m
Climate Variability (Scenario 3) Planning horizon 2100 RCP4.5	S3_CC	100yCC2a	030m 060m 090m 120m 180m
Climate Variability (Scenario 3) Planning horizon 2100 RCP8.5	S3_CC	100yCC2b	030m 060m 090m 120m 180m

As an example, the batch file command for Scenario 1 (2100) RCP4.5 100-yr 60-minute simulation would be as follows:

tuflow_iSP_w64.exe -b -s S1_CC -e1 100yCC2a -e2 060m CCFS_~s~_~e1~_~e2~_025.tcf