Witton Creek Flood Study

Volume 1 of 2

Flood Study Report Disclaimer

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Witton Creek Flood Study

Volume 1 of 2

Prepared by Arup Australia Pty Limited
Prepared for Brisbane City Council

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Executive Summary

Introduction

Brisbane City Council (BCC) periodically updates its flood studies to reflect the current conditions of the catchment and best practice flood modelling techniques. The most recent study undertaken for Witton Creek was the Stormwater Management Plan completed by Water & Environment City Design, Brisbane City Council in 2000.

Witton Creek Catchment has a total area of 4.09 km^2 and the catchment centroid is located approximately 7.5km south-west of Brisbane CBD. The inner-city suburbs of Indooroopilly and Chapel Hill are partly contained within the Witton Creek Catchment area. The headwaters of Witton Creek are located within the Mount Coot-Tha and Brisbane Forest Park Bushland reserve areas. Witton Creek ultimately drains into the Brisbane River. The major creeks / tributaries within the catchment are Witton Creek, Witton Creek Tributary A, Witton Creek Tributary B and Witton Creek Tributary C.

Project Objectives

The primary objectives of the project were as follows:

- Update the Witton 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 models are suitable for the purpose of simulating design flood events.
- Estimate design and very rare / extreme flood magnitudes in accordance with Australian Rainfall and Runoff 2019 guidelines (AR&R2019), incorporating increased rainfall intensities due to projected climate variability effects.
- Determine flood levels for the design and very rare /extreme events.
- Quantify the impacts of the Minimum Riparian Corridor (MRC) and filling/development outside the “Modelled Flood Corridor”.
- Produce flood extent mapping for the selected range of design, very rare and extreme events.
- Quantify the impacts 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

Hydrological and hydraulic models of the Witton 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 39 sub-catchments, with an average sub-catchment size of 100 ha. The sub-catchment delineation was based upon BCC 2019 1m grid LIDAR and considered the location of major and minor tributaries, the BCC stormwater network, as well as man-made boundaries such as the Western Freeway.

The hydraulic model uses more sophisticated routing to simulate the movement of flood water through 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 TUFLOW hydraulic model consists of a 1d / 2d linked model schematisation with the 1d domain modelled in ESTRY and the 2d domain in TUFLOW. The hydraulic model incorporated Witton Creek, Witton Creek Tributary A, Witton Creek Tributary B and Witton Creek Tributary C within the 2d domain.

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, February 2020, March 2017 and May 2015 events. Verification of the URBS and TUFLOW models utilised two historical storms namely, June 2016 and January 2013 events.

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 are no continuous recording stream gauges within the catchment against which to confirm timing and volumes of the flood events.

Utilising the adopted parameters from the calibration process, model verification was undertaken. Similar to the calibration, the verification achieved a good correlation between the simulated and historical records for both verification events.

Given the results of the calibration and verification process were good, 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, Very Rare and Extreme Event Modelling

The calibrated hydrologic and hydraulic models were then used to simulate a range of synthetic design flood events. Design, very rare and extreme flood magnitudes were estimated for the full range of events from 50 % AEP to Council's PMF storm. 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.

Two waterway scenarios were considered, as follows:

- Scenario 1 – Existing Waterway Conditions: Based on the current waterway conditions– refer to Section 4.2.3 and Section 6.3.3 for further detail.
- Scenario 3 – Ultimate Conditions: As per Scenario 1, but includes an allowance for the minimum riparian corridor and 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 Flood Planning Areas (FPAs) 1, 2, and 3 and the Waterway Corridor.

The results from the TUFLOW hydraulic model were used to determine and produce the following:

- Design flood discharges (Section 6.4.1)
- Design flood levels at 100m intervals along the AMTD line (Appendices F, G, I and J)
- Scenario 1 design flood extent mapping (Volume 2 of 2)

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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.
2019 ALS Data	This dataset is part of the Brisbane-Ipswich LiDAR 2019 Project, acquired by Aerometrex 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 Data Hub	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 (2019).
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.
Climate Change (CC)	Representative Concentration Pathway (RCP) 4.5 – 2100 horizon
Digital Elevation Model (DEM)	A three-dimensional model of the ground surface elevation.
Design Event, Design Storm	A hypothetical flood / storm representing a likelihood of occurrence
ESTRY	ESTRY is the 1d hydrodynamic solver used by TUFLOW.
Floodplain	Area of land subject to inundation by floods up to and including the Probable Maximum Flood (PMF) event.
Flood Classification (BOM Definition)	Minor - Causes inconvenience. Low-lying areas next to water courses are inundated. Minor roads may be closed and low-level bridges submerged. In urban areas inundation may affect some backyards and buildings below the floor level as well as bicycle and pedestrian paths. In rural areas removal of stock and equipment may be required. Moderate - In addition to the above, the area of inundation is more substantial. Main traffic routes may be affected. Some buildings may be affected above the floor level. Evacuation of flood affected areas may be required. In rural areas removal of stock is required.

Glossary of Terms (cont)

Term	Definition
	Major - In addition to the above, extensive rural areas and/or urban areas are inundated. Many buildings may be affected above the floor level. Properties and towns are likely to be isolated and major rail and traffic routes closed. Evacuation of flood affected areas may be required. Utility services may be impacted.
Flood Frequency Analysis (FFA)	Flood Frequency Analysis (FFA) refers to procedures that use recorded and related flood data to identify underlying probability model of flood peaks, at a particular location in the catchment.
Flood Planning Area (FPA)	Flood Overlay Code development control mechanism that recognises the susceptibility of flooding in terms of frequency, flow velocity and flood depth. There are five FPAs (1 to 5), where FPA1 is subject to the most stringent development assessment requirements.
HEC-RAS	Hydraulic modelling software package developed by USACE
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 developed by DHI
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
TIN	Triangular Irregular Network (TIN) - a series of non-overlapping triangles from which the 3d vertices (x,y,z) are used as an approximation of the 3d surface.
TUFLOW	Hydraulic modelling software package developed by BMT
URBS	Hydrologic modelling software package developed by D.G. Carroll
WBNM	Hydrologic modelling software package developed M.J. Boyd

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 2019	Australian Rainfall and Runoff (2019)
BCC	Brisbane City Council
CBD	Central Business District
CL	Continuing rainfall loss (mm/hr)
CC	Climate Change
DEA AR&R 2019	Design Event Approach Australian Rainfall and Runoff (2019)
DTMR	Department of Transport and Main Roads (Queensland)
FPA	Flood Planning Area
HPC	TUFLOW HPC (Heavily Parallelised Compute) solver
IFD	Intensity Frequency Duration
IL	Initial rainfall loss (mm)
IL_s	Initial loss for the rainfall event (mm)
IL_b	Initial loss for the rainfall burst (mm)
IWL	Initial Water Level (mAHD)
mAHD	metres above AHD
MHG	Maximum Height Gauge
MRC	Minimum Riparian Corridor
POT	Peak Over Threshold
RCBC	Reinforced Concrete Box Culvert
RCP	Reinforced Concrete Pipe
RCP4.5	Representative Concentration Pathway 4.5
SGS	TUFLOW Sub-grid Sampling
WC	Waterway Corridor

1.0 Introduction

1.1 Catchment Overview

Witton Creek Catchment has a total area of 4.09 km^2 and is located approximately 7.5 km south-west of Brisbane CBD. The inner-city suburbs of Indooroopilly and Chapel Hill are partly contained within the Witton Creek Catchment area, with the creek's head waters located within the Mount Coot-Tha and Brisbane Forest Park Bushland reserve areas. The major creeks / tributaries within the catchment are Witton Creek, Witton Creek Tributary A, Witton Creek Tributary B and Witton Creek Tributary C. Figure 1.1 indicates the location of the catchment.

1.2 Study Background

BCC is in the process of updating their 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 Procedure V9.0.

The most recent flood study for Witton Creek catchment was the Witton Creek Stormwater Management Plan (2000). For the purpose of this report, the 2000 Witton Creek Stormwater Management Plan will be termed as 2000 Witton SMP.

1.3 Scope of the Flood Study

The update of the Witton Creek Flood Study is in accordance with the current BCC Flood Study Procedure V9.0 document and incorporates best practice flood modelling techniques.

To meet the project objectives, the scope of the flood study is as follows:

- Develop a new URBS hydrologic model of the catchment, where the URBS model will be compatible with the new extents of the updated hydraulic model; incorporate the latest major development / infrastructure works and make allowance for catchment development based on the current planning scheme (City Plan 2014).
- Develop a new 1d / 2d TUFLOW hydraulic model of the creek system to replace the existing 1d MIKE11 model. The TUFLOW model extents will be significantly larger than the previous MIKE11 model and will incorporate the major and minor tributaries throughout the catchment. The model will also incorporate recent major development / infrastructure works; the latest LiDAR dataset (2019 ALS) and field survey.
- 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 very rare / extreme flood magnitudes in accordance with Australian Rainfall and Runoff 2019 (AR&R 2019). This will include an allowance for increased rainfall intensities due to projected climate variability effects.

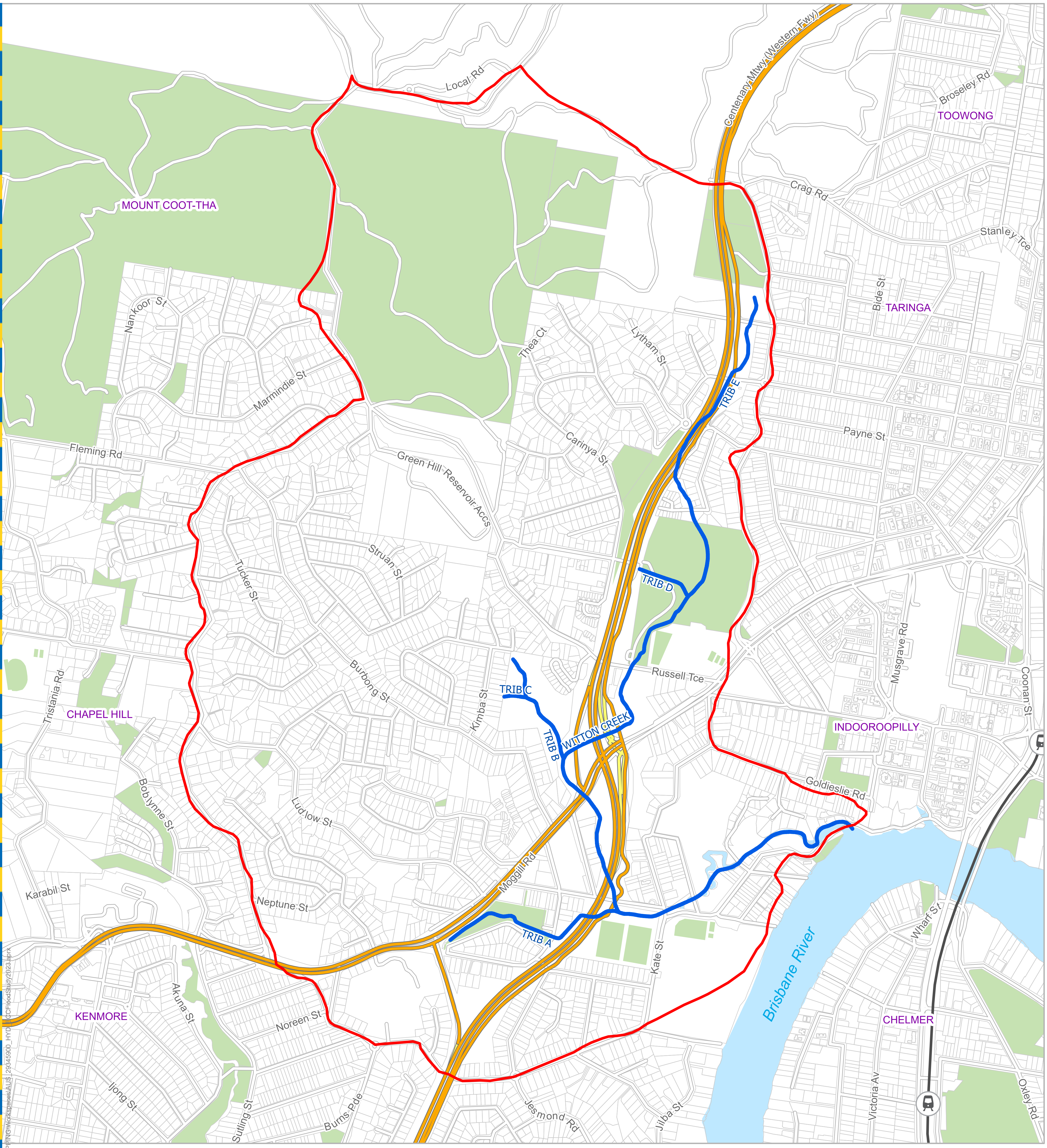
- Estimate the flood levels for the design and very rare / extreme events, accounting for the effects of Minimum Riparian Corridor (MRC) and floodplain development / filling in accordance with current planning policy. This will include an allowance for sea-level rise due to projected climate variability effects.
- Produce flood extent mapping for the selected range of design and very rare / extreme events.
- Produce Hydraulic Structure Reference Sheets (HSRS) to capture the flooding and hydraulic characteristics of the major hydraulic structures.

1.4 Study Limitations

The results from this flood study are largely derived from the hydrologic and hydraulic models developed for this study. It is important to be aware of the limitations of these models, which include (but is not limited to) the following:

- The models have only been calibrated / verified at locations where Maximum Height Gauge (MHG) records exist. This should be considered when reviewing the accuracy of results outside the influence of the gauge locations. Refer to Figure 3.6 for the hydrometric gauge locations.
- The models are catchment scale and have been developed to simulate the flooding characteristics at a broad scale. As a result, smaller more localised flooding and drainage characteristics may not be apparent in the results.
- Durations tested within the model were 30 mins to 6 hours, in keeping with BCC's standard approach to flood studies of this nature. Critical durations identified along the main creek were greater than 30 mins and less than 6 hours. Some upper areas of the tributaries were shown to have critical durations of 30 minutes. These areas may have shorter critical durations (less than 30 minutes) with slightly higher flood levels – however this is not expected to be significant, and likely falls within the general accuracy of the flood modelling.
- The models have been developed to simulate creek flooding characteristics – while tailwater levels have been set for where the model discharges to the Brisbane River (as per the standard BCC modelling approach for studies of this nature), note that Brisbane River flooding (either independently or coincidentally) has not been assessed, nor has it been mapped. This is further outlined in Section 6.3.3, and should be considered when drawing on the flood study outputs.
- The 2019 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 2019 ALS dataset of the waterway was reviewed against field survey (2023) for validation, and was generally found to compare relatively well. Some adjustments were undertaken where required (refer to Section 3.2.1.1 and Section 5.2.4).

- The accuracy of the model results is directly linked to the following:
    - The accuracy limits of the data used to develop the model (i.e. ALS, survey information, structure data, pipe network data, etc.).
    - The accuracy and quality of the hydrometric data used to calibrate / verify the models.
    - The number of observed records, including MHG readings throughout the catchment.



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Metres

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Checked : T WEBB
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Legend

- Catchment Area
- Railway Stations
- Creek Centreline
- Railway Line
- Freeways/Highways
- Major Roads
- Streets
- Property Holding
- Waterway
- Parks

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DATA INFORMATION

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Witton Creek
Figure 1.1 Locality Plan
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2.0 Catchment Description

2.1 Catchment and Waterway Characteristics

2.1.1 General

The confluence of Witton Creek and the Brisbane River is 350 m upstream of Walter Taylor Bridge at Indooroopilly. The total catchment area of Witton Creek Catchment is approximately 4.09 km^2 and comprises the following tributaries:

- Witton Creek: 2.07 km^2
- Witton Creek Tributary A: 1.17 km^2
- Witton Creek Tributary B: 0.14 km^2
- Witton Creek Tributary C: 0.41 km^2
- Witton Creek Tributary D: 0.08 km^2
- Witton Creek Tributary E: 0.22 km^2

Figure 1.1 indicates the major creeks and tributaries within the catchment.

2.1.2 Witton Creek

Witton Creek is the largest waterway within the catchment with a length of approximately 2.45 km from the upstream extent of Chapel Hill and Indooroopilly suburbs to the Brisbane River at Radnor Street. The catchment headwaters are within the Mount Coot-Tha and Brisbane Forest Park Bushland reserve areas, characterised by steep slopes and forested vegetation. The catchment is bounded by Breakfast Creek Catchment (north); Toowong Creek Catchment and Sandy Creek Catchment (east); Cubberla Creek (west); and Brisbane River (south).

Witton Creek is open waterway for the majority of its length. During the urbanisation of the catchment, the natural waterway has been modified in numerous areas, including channelisation / straightening, culverts / bridges; flood plain filling etc. The bed slope of the creek overall is between 3% to 6%.

There are 2 major arterial road crossings of Witton Creek, namely Moggill Road (AMTD 900 m) and the Western Freeway (AMTD 1200 m). These transport corridors have influenced the major drainage path substantially, where:

- Moggill Road runs in an east to west orientation across the southern area of the catchment.
- Western Freeway traverses the entire length of the catchment in the north-south orientation.

2.1.3 Witton Creek Tributary A

Tributary A is one of four western tributaries of Witton Creek. The most upstream and downstream sections of the creek are fully piped, with the lower section consisting of an open waterway with a length of approximately of 0.59 km.

The average slope of the trunk piped section is approximately 2% to 6%, whereas the open channel is less steep with an average slope of 3%. The tributary joins Witton Creek at AMTD 661 m, with an invert level of approximately 2.2 m AHD.

2.1.4 Witton Creek Tributary B

Tributary B is the second of four western tributaries of Witton Creek. The most upstream section of the tributary is fully urban and piped, with the lower section consisting of open creek with an approximate length of 0.35 km.

The average slope of the trunk piped section is approximately 2% to 5%, whereas the open creek has an average slope of approximately 1.5%. The tributary joins Witton Creek at AMTD 1378 m, with an invert level of approximately 8.2 m AHD.

2.1.5 Witton Creek Tributary C

Tributary C is the third of four western tributaries and joins Witton Creek Tributary B at AMTD 230. The most upstream section of the tributary is fully urban and piped, with the lower section consisting of open creek with an approximate length of 0.06 km.

The average slope of the trunk piped section is approximately 5% to 10%, whereas the open creek has an average slope of approximately 10%. The tributary joins Witton Creek Tributary B at AMTD 230 m, with an invert level of approximately 11.1 m AHD.

2.1.6 Witton Creek Tributary D

Tributary D is the last of four western tributaries. The most upstream section of the tributary is fully urban and piped, with the lower section consisting of open creek with an approximate length of 0.17 km.

The average slope of the trunk piped section is approximately 3%, whereas the open creek has an average slope of approximately 6%. The tributary joins Witton Creek at AMTD 2077 m, with an invert level of approximately 15.3 m AHD.

2.1.7 Witton Creek Tributary E

Tributary E is the northern tributary of Witton Creek. It is located adjacent to the Western Freeway and located on Mount Coot-Tha. The most upstream section of the tributary is open creek, draining to a piped network that outfalls to Witton Creek.

The open creek has an average slope of approximately 3% to 10%, with the average slope of the trunk piped section being approximately 3% to 8%. Tributary E has an approximate length of 0.64 km and connects to Witton Creek at AMTD 2455 m at an invert level of approximately 21.8 m AHD.

2.2 Land Use

There is significant development through the Witton Creek Catchment, with the predominant land use zoning being “Low Density Residential” (43.2%). The next largest land use is “Environmental Management and Conservation (21.9%), and then followed by “Road Reserve: (17.2%). Figure 2.1 provides a breakdown of the catchment land use types by percentage and the Appendix C provides a Map indicating the distribution of the land-use through the catchment. The land-use data used within this study and shown in both these figures are based upon the BCC City Plan 2014.

The “Environmental Management and Conservation” zones are primarily located within the head waters within Mount Coot-Tha and Brisbane Forest Park Bushland reserve areas. These areas are characterised by forest on steep slopes.

The “Educational Purposes” and “Open Space” zones are mainly located along Witton Creek and Witton Creek Tributaries, within the downstream reaches where the floodplain widens. Large pervious areas include Moore Park., Kennewell Park, Market Street Park and Jack Bowers Oval.

The “Emerging Community” zones are areas that would become urban developments in the future. This zone is located in the upper reach of Witton Creek Catchment and represents 1.1% of the catchment area.

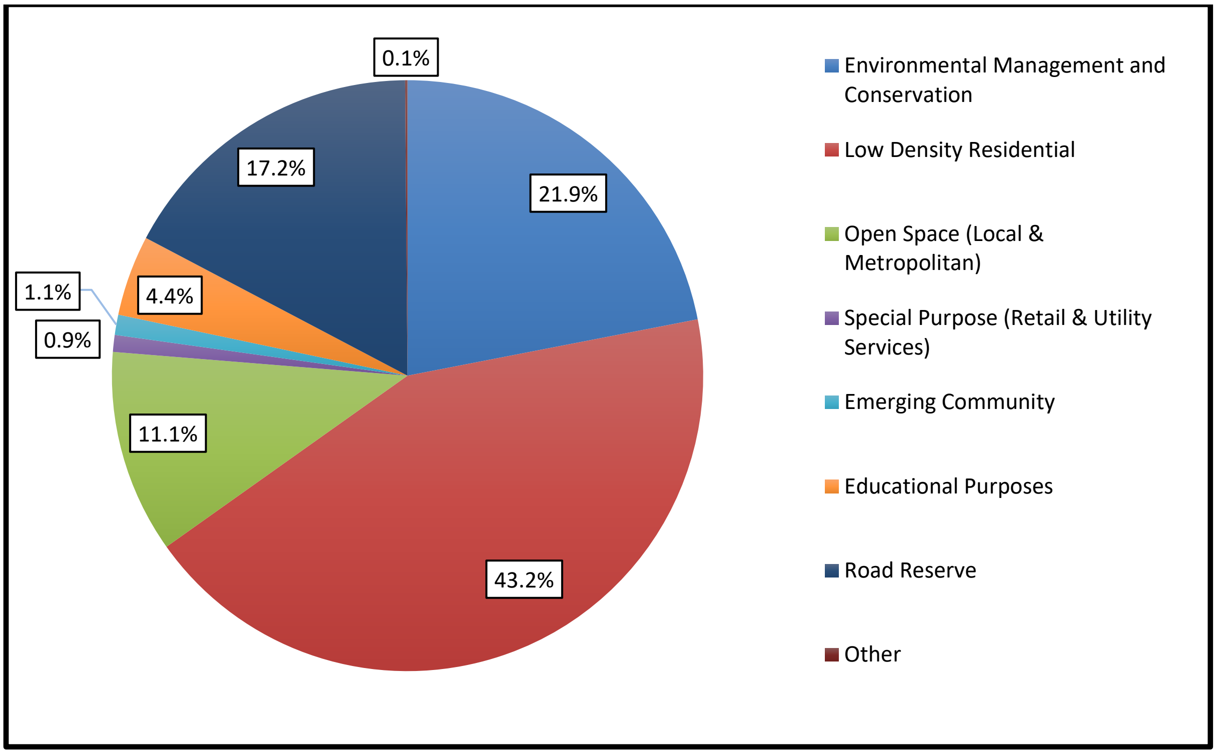


Figure 2.1: Witton Creek Catchment Land Use

3.0 Available Data

3.1 Previous Studies

3.1.1 General

The previous study undertaken for Witton Creek Catchment was the 2000 Witton Creek SMP, completed by BCC. The following section provides an overview of the 2000 Witton Creek SMP.

3.1.2 Witton Creek Storm Water Management Plan (2000)

This stormwater management plan was undertaken by BCC to set an approach to meet the Urban Stormwater Management Strategy (Version 2, 1999 – 2001) and to meet the challenges to the principles of ecological sustainable development. The goal of the SMP was a detailed assessment of the creek flood characteristics, the assessment of ecological/habitat value of the riparian corridor and the assessment of the existing water quality regime.

This study utilised XP- RAFTS for the hydrology and MIKE11 for hydraulic modelling. The structure information from the 2000 SMP was used as a reference for this study.

3.2 Topographic Survey Data

3.2.1 Field Survey

Topographic field survey data was acquired for use in this flood study from a range of sources. The following lists several of the sources whereby the survey was not already part of the existing hydraulic model as indicated in Section 3.3.

3.2.1.1 Witton Creek Survey (2023)

Topographical field survey was undertaken in 2023 for the purpose of this study. This survey was intended to supplement cross-sectional and hydraulic structure information along Witton Creek and included the following:

- 35 creek cross sections
- 8 basic hydraulic structures

3.2.2 LiDAR

3.2.2.1 General

2019 LiDAR Survey (1m resolution DEM) was utilised for this project. This LiDAR data is Airborne Laser Scanning (ALS) from the Queensland Government. Details of these ALS datasets are outlined below and their use within the study is discussed further in Section 5.0.

3.2.2.2 2019 BCC LiDAR (ALS)

The 2019 ALS data was captured as part of the Brisbane-Ipswich LiDAR 2019 Project, undertaken by Aerometrex Pty Ltd on behalf of the Queensland Government. The ALS data was acquired between 11/06/2019 and 16/08/2019 from a fixed wing aircraft at a flying height of 1250 m above sea level.

Brisbane-Ipswich LiDAR 2019 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

3.2.3 Aerial Photography

The following sources of aerial imagery taken during different points in time were available to be used in this study:

- BCC aerial photography –2009, 2013, 2015, 2016, 2017, 2018, 2020 and 2021

3.3 Existing Hydraulic Models

Hydraulic structure data and structure sheets from the existing Witton Creek Model were used in the development of the Witton Creek TUFLOW model. The model is listed below in Table 3.1 and the use of data is discussed further in Section 5.0.

Table 3.1: Hydraulic Models used in Model Development

Model	Waterway	Type	Year	Model Developer
Witton Creek SMP MIKE 11 Model	Witton Creek	1D	2000	BCC

3.4 Hydrometric Data and Storm Selection

3.4.1 Selection of Historical Storm Events

Five significant flood events that have occurred within the Witton Catchment over the last 10 years have been selected for calibration and validation. Table 3.2 indicates the peak flood level at the MHG gauges as well as the approximate size of the events. MHG W120 is located along the channel at Jack Bowers Oval and MHG W110 is located along the channel downstream of Aaron Place Bridge.

Table 3.2: Historical Peak MHG Levels on Witton Creek

Event	Peak Flood Level (m AHD)		Approximate Size of Event* (Based on Rainfall Assessment)	Number of Stream Gauge / MHG Records
	MHG W120	MHG W110		
February 2020	3.56	2.73	< 50% AEP	2
March 2017	4.64	3.67	50% AEP to 10% AEP	2
May 2015	4.3	3.53	50% AEP to 20% AEP	2
June 2016	4.29	3.53	20% AEP to 5% AEP	2
January 2013	4.27	3.74	50% AEP to 5% AEP	2

* Note that :

- The estimates of storm event magnitude as presented in Table 3.2 are based on the IFD assessment of the recorded rainfall for each historical event, across a full range of storm durations (i.e. 1-hour to 72-hour storm durations). The greatest event magnitude is quoted, noting this may not be aligned with the critical storm duration that generates peak flood conditions in the catchment. For further details, refer to refer to Section 3.4.6.
- The rainfall-based estimates of storm event magnitude may differ appreciably from flood-level based estimates of event magnitude associated with the corresponding flood event. Such differences can relate to the responsivity of the catchment, the critical storm duration vs actual event duration, antecedent catchment conditions, the spatial distribution of the rainfall (i.e. gauge-measured rainfall vs actual rainfall extent / intensity), the downstream boundary condition, etc. Accordingly, care must be taken to differentiate between historical event magnitudes when discussing rainfall and flooding to avoid the assumption that they are the same.

The three events selected for calibration were:

- February 2020
- March 2017
- May 2015

The two events selected for verification were:

- June 2016
- January 2013

3.4.2 Availability of Historical Data for Selected Storms

3.4.2.1 Continuous Recording Rainfall Stations

There are eight rainfall stations located around the Witton Creek Catchment. Table 3.3 indicates the location, details and availability of the rainfall station data for each of the selected storm events. Of the eight rainfall stations, one rainfall station (540465) was identified in the Theissen Polygon assessment to be used for calibration and verification event modelling (Section 4.4.1 and Appendix A). Figure 3.6. shows the location of rainfall station (540465) that was utilised for the calibration and verification events modelling.

Table 3.3: Rainfall Station data Availability

Rainfall Gauge ID	Sensor ID	Locations	Data Availability				
			Feb 2020	March 2017	May 2015	June 2016	Jan 2013
540117	E1512	ABQ-2 Mt Coot-Tha	✓	✓	✓	✓	✓
540099	E1515	Chadston Close, Kenmore Hills	✓	✓	✓	✓	✓
540465	E1852	Green Hill Reservoir, Russell Tce Chapel Hill	✓	✓	✓	✓	✓
540281	E1749	Anzac Park, Toowong	✓	✓	✓	✓	✓
540071	E2020	Corinda High School, Corinda	✓	✓	✓	✓	✓
540132	E1554	Caswell St, East Brisbane	✓	✓	✓	✓	✓
540470	E1747	Dulcie St, Salisbury	✓	✓	✓	✓	✓
540134	E1548	Joachim St, Holland Park West	✓	✓	✓	✓	✓

3.4.3 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 Witton Creek Catchment.

3.4.4 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 or overtopped during the event and there is a nearby debris mark, then the recorded water level is typically based on this debris level.

There are two MHGs within the Witton Creek Catchment, both located along the main creek channel. These gauges are MHG W120, located upstream of Jack Bowers Oval, and MHG W110, located downstream of Aaron Place bridge. Table 3.4 indicates the availability of MHG data for each flooding event.

Table 3.4: Maximum Height Gauge data Availability

MHG ID	Locations	Data Availability				
		Feb 2020	March 2017	May 2015	June 2016	Jan 2013
W120	Jack Bowers Oval	✓	✓	✓	✓	✓
W110	DS Aaron Place Bridge	✓	✓	✓	✓	✓

3.4.5 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 3.5 indicates the details of the nearest upstream and downstream gauges to the mouth of Witton Creek utilised in this study. Figure 3.7 indicates the locations of the stream gauges utilised in this study. The Brisbane River AMTD at the confluence with Witton Creek is 41.8km.

Table 3.5: Nearest Brisbane River Stream Gauges

Gauge ID	Sensor ID	Owner	BNE AMTD (km)	Location
540683	E1856	BCC	34.2	St Lucia AL
540192	E6731	Seqwater	52.1	Jindalee AL

Table 3.6 indicates the availability of stream gauge data for the five calibration / verification events. Note that recorded flood levels are available at the St Lucia and Jindalee gauges for all calibration/verification events. Refer to Section 5.3.8 for further details on the adoption of downstream boundary conditions.

Table 3.6: Brisbane River Stream Gauge Data Availability

Gauge ID	Sensor ID	Locations	Data Availability				
			Feb 2020	March 2017	May 2015	June 2016	Jan 2013
540683	E1856	St Lucia AL	✓	✓	✓	✓	✓
540192	E6731	Jindalee AL	✓	✓	✓	✓	✓

3.4.6 Characteristics of Historical Events

The following analysis of the historical events was undertaken for Rainfall Station 540645. This station was selected through the Thiessen polygon assessment discussed further in Section 4.4.1. From this assessment only rainfall station (540465) was utilised for the calibration and verification events for the Witton Creek Catchment. The cumulative rainfall graphs for Rainfall Station 540645 for each historic event are presented in Appendix A.

3.4.6.1 February 2020

The February 2020 event is a relatively small flood event within Witton Creek, which produced a flood level of 3.56 m AHD at MHG W120 on Witton Creek at John Bowers Oval.

This event had a total rainfall of 161 mm recorded in 24 hrs on the 6th of February. The most intense burst occurred over 2 hours between 5:30am and 7:30am on the 6th of February, with approximately 43 mm of rainfall recorded at Rainfall Station 540465. The cumulative rainfall for Rainfall Station 540465 is presented in Appendix A.

Figure 3.1 provides a comparison of the IFD curve for Rainfall Station 540465 against the BCC LIMB IFD curve generated at the catchment centroid. The equivalent design rainfall AEP event at Rainfall Station 540465 would have been as follows:

- 1-hour rainfall: < 63.2% AEP event
- 2-hour rainfall: < 63.2% AEP event
- 3-hour rainfall: < 63.2% AEP event
- 6-hour rainfall: 50% AEP to 20% AEP event
- 12-hour rainfall: 20% AEP event
- 24-hour rainfall: 20% AEP event

3.4.6.2 March 2017

The March 2017 event is a relatively large flood event within Witton Creek, which produced a flood level of 4.64 m AHD at MHG W120 on Witton Creek at John Bowers Oval.

This event had a total rainfall of 272 mm recorded in 24 hrs on the 30th of March. The most intense burst occurred over 3 hours between 8:30am and 11:30am on the 6th of March, with approximately 108 mm of rainfall recorded at Rainfall Station 540465. The cumulative rainfall for Rainfall Station 540465 is presented in Appendix A.

Figure 3.2 provides a comparison of the IFD curve for Rainfall Station 540465 against the BCC LIMB IFD curve generated at the catchment centroid. The equivalent design rainfall AEP event at Rainfall Station 540465 would have been as follows:

- 1-hour rainfall: 20% AEP to 10% AEP event
- 2-hour rainfall: 50% AEP to 20% AEP event
- 3-hour rainfall: 10% AEP to 5% AEP event
- 6-hour rainfall: 5% AEP to 2% AEP event
- 12-hour rainfall: 5% AEP to 2% AEP event
- 24-hour rainfall: 5% AEP to 2% AEP Event

3.4.6.3 May 2015

The May 2015 event is a relatively small to medium flooding event within Witton Creek, which produced a flood level of 4.30 m AHD at MHG W120 on Witton Creek at John Bowers Oval.

This event had a total rainfall of 180 mm recorded in 24 hrs on the 1st of May. The most intense burst occurred over 6 hours between 1:30pm and 7:30 pm on the 1st of May, with approximately 133 mm of rainfall recorded at Rainfall Station 540465. The cumulative rainfall for Rainfall Station 540645 is presented in Appendix A.

Figure 3.3 provides a comparison of the IFD curve for Rainfall Station 540465 against the BCC LIMB IFD curve generated at the catchment centroid. The equivalent design rainfall AEP event at Rainfall Station 540465 would have been as follows:

- 1-hour rainfall: 63.2% AEP event to 50% AEP event
- 2-hour rainfall: <63.2% AEP event
- 3-hour rainfall: 63.2% AEP event to 50% AEP event
- 6-hour rainfall: 10% AEP Event to 5% AEP event
- 12-hour rainfall: 10% AEP event to 5% AEP event
- 24-hour rainfall: 20% AEP event to 10% AEP event

3.4.6.4 June 2016

The June 2016 event is a medium flooding event in Witton Creek, which produced a flood level of 4.29 m AHD at MHG W120 on Witton Creek at John Bowers Oval.

This event had a total rainfall of 164 mm recorded in 24 hrs on the 19th of June. The most intense burst occurred over 5.5 hours between 3:00 pm and 6:30 pm on the 19th of June, with approximately 132 mm of rainfall recorded at Rainfall Station 540465. The cumulative rainfall for Rainfall Station 540645 is presented in Appendix A.

Figure 3.4 provides a comparison of the IFD curve for Rainfall Station 540465 against the BCC LIMB IFD curve generated at the catchment centroid. The equivalent design rainfall AEP event at Rainfall Station 540465 would have been as follows:

- 1-hour rainfall: 10% AEP event to 5% AEP event
- 2-hour rainfall: 5% AEP event
- 3-hour rainfall: 20% AEP event to 50% AEP event
- 6-hour rainfall: 5% AEP event to 2% AEP event
- 12-hour rainfall: 10% AEP event
- 24-hour rainfall: 20% AEP event

Table 3.7: shows the 4-day, 14-day antecedent rainfall and monthly total rainfall records from the Bureau of Meteorology (BoM) prior to the June 2016 event. The monthly rainfall totals for May and April are low (< 50 mm) showing that the catchments had very dry conditions leading up to the June 2016 event. The catchments experienced 4 mm of rainfall in the four days lead up to the event and between 95 to 130 mm in the preceding 14 days. Accordingly, the catchment is unlikely to have been saturated, and would have had elevated infiltration potential.

Table 3.7: BoM Rainfall Prior to June 2016 Event

Station	Location	Rainfall Total (mm)		Monthly Total (mm)	
		4 days prior	14 days prior	Month prior (May)	2 Month prior (April)
40976	Botanic Garden, Mt Coot-Tha Station	4.4	130.4	24.1	11.2
40913	Brisbane City	4.0	95.0	27.6	12.8

3.4.6.5 January 2013

The January 2013 event is a relatively large flooding event in Witton Creek, which produced a flood level of 4.27 m AHD at MHG W120 on Witton Creek at John Bowers Oval.

This event had a total rainfall of 258 mm recorded in 24 hrs on the 27th of January. The most intense burst occurred over 9 hours between 11:00am and 20:00 pm on the 27th of January, with approximately 196 mm of rainfall recorded at Rainfall Station 540465. The cumulative rainfall for Rainfall Station 540645 is presented in Appendix A.

Figure 3.5 provides a comparison of the IFD curve for Rainfall Station 540465 against the BCC LIMB IFD curve generated at the catchment centroid. The equivalent design rainfall AEP event at Rainfall Station 540465 would have been as follows:

- 1-hour rainfall: 50% AEP event to 20% AEP event
- 2-hour rainfall: 20% AEP event
- 3-hour rainfall: 10% AEP Event to 5% AEP event
- 6-hour rainfall: 5% AEP to 2% AEP event
- 12-hour rainfall: 5% AEP to 2% AEP event
- 24-hour rainfall: 5% AEP to 2% AEP event

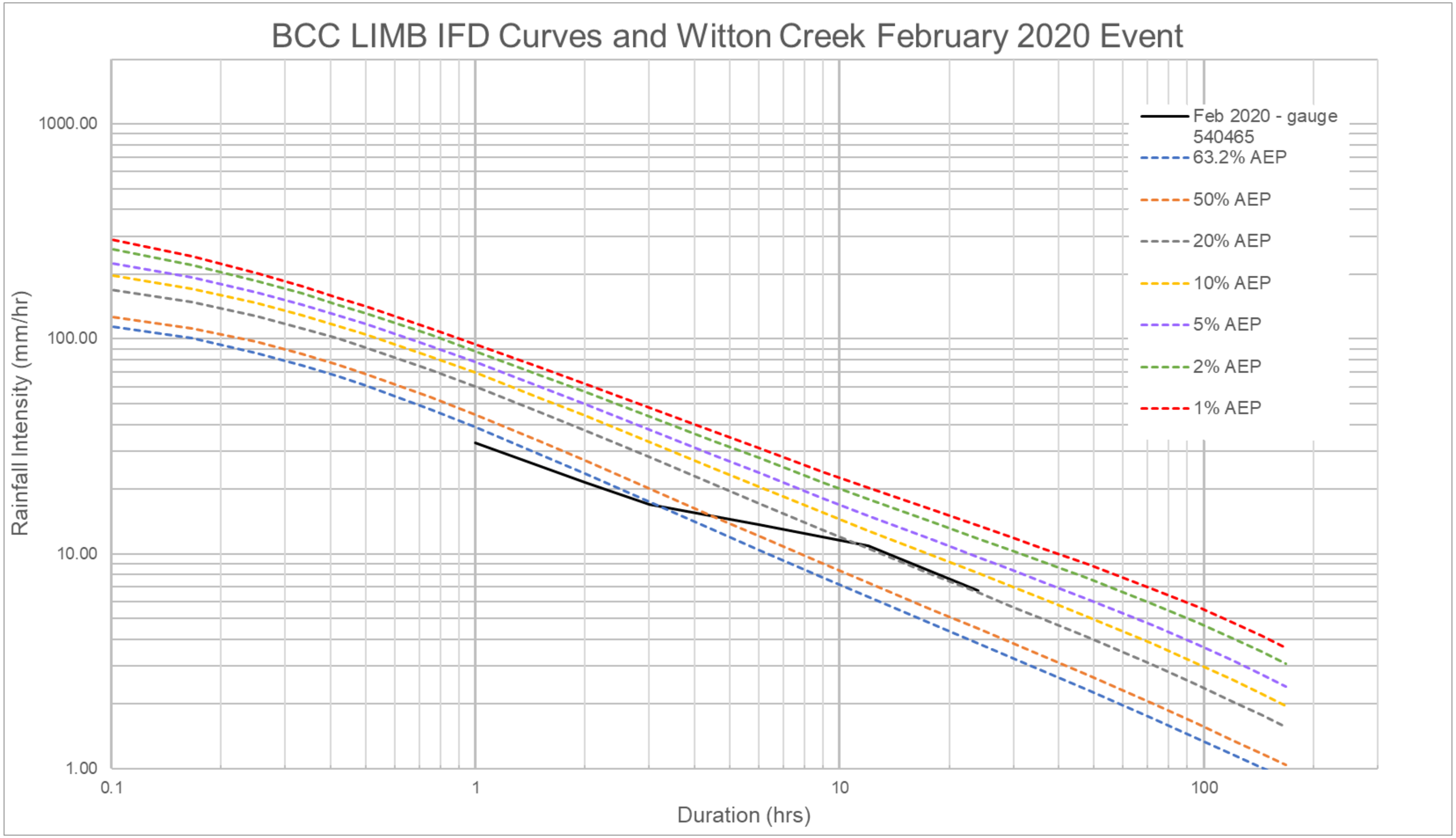


Figure 3.1: IFD Curve for February 2020

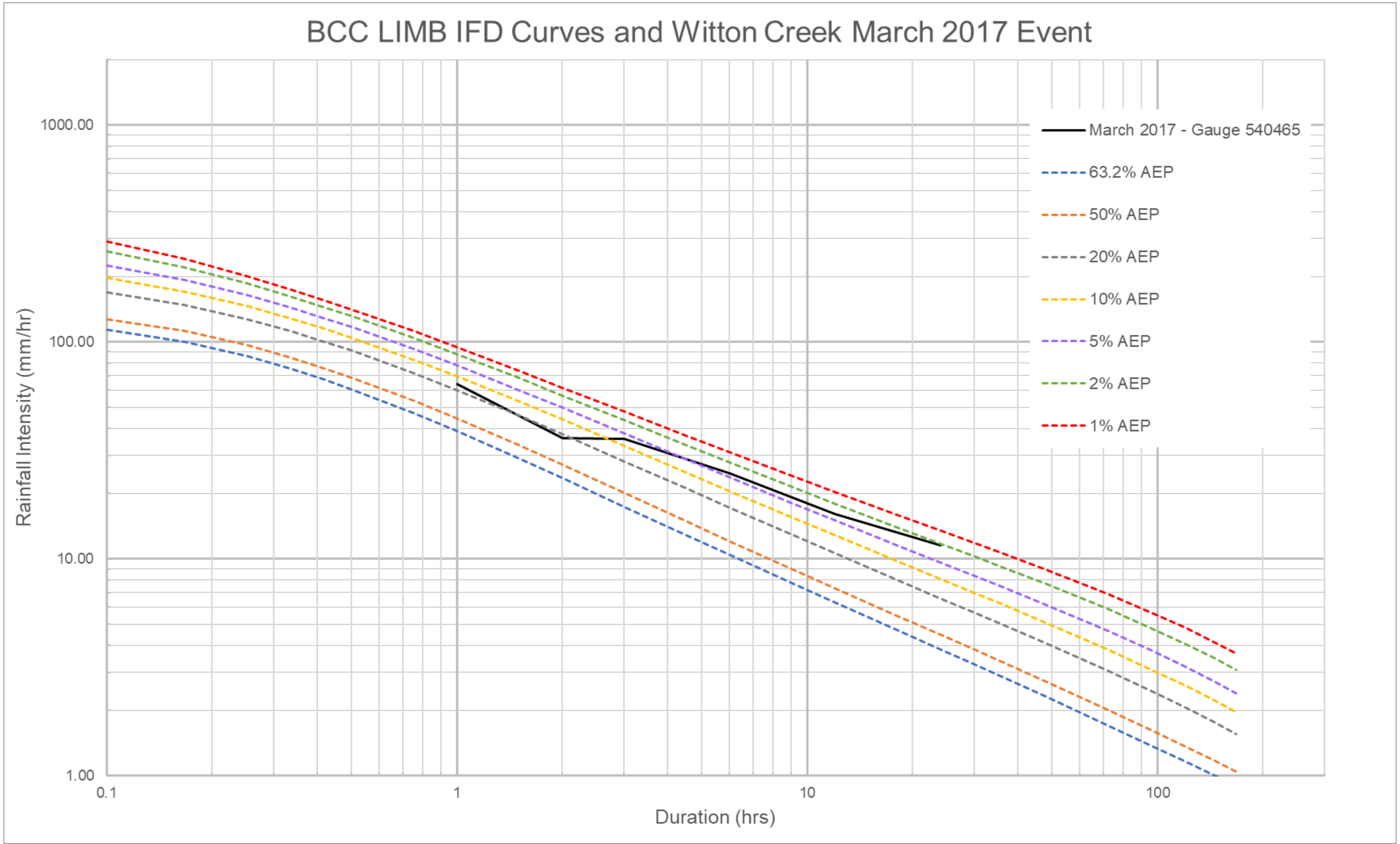


Figure 3.2: IFD Curve for March 2017

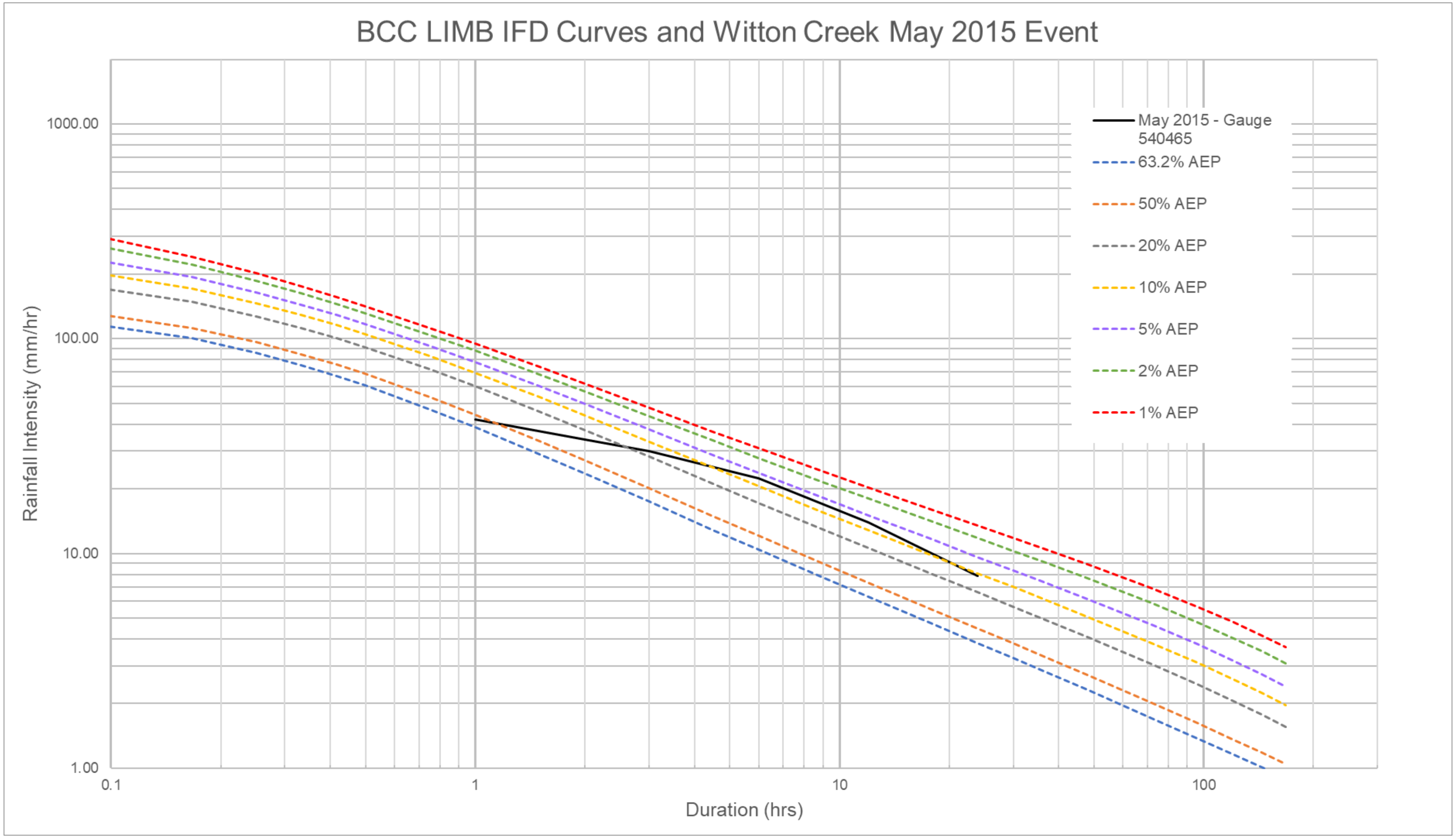


Figure 3.3: IFD Curves for May2015

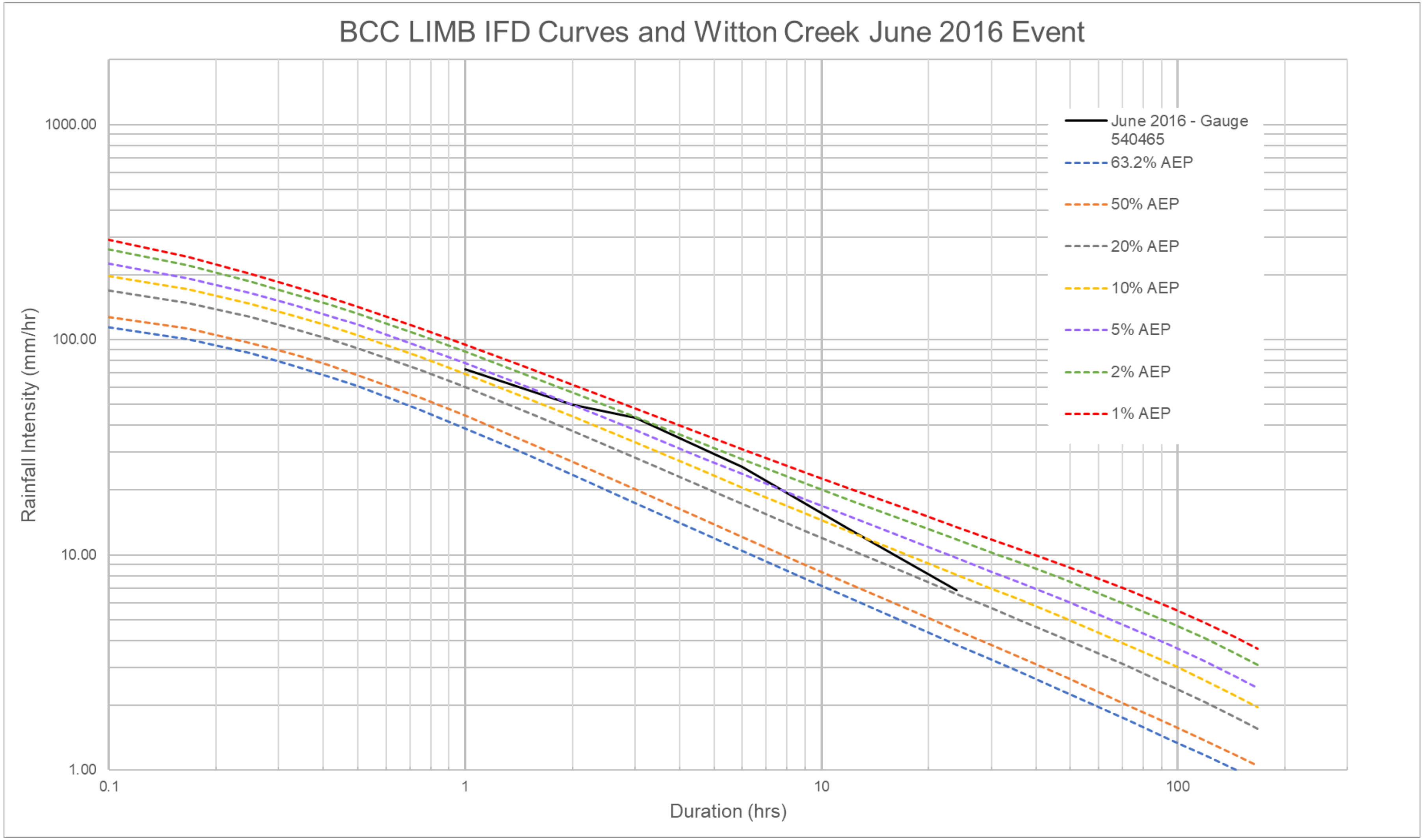


Figure 3.4: IFD Curve for June 2016

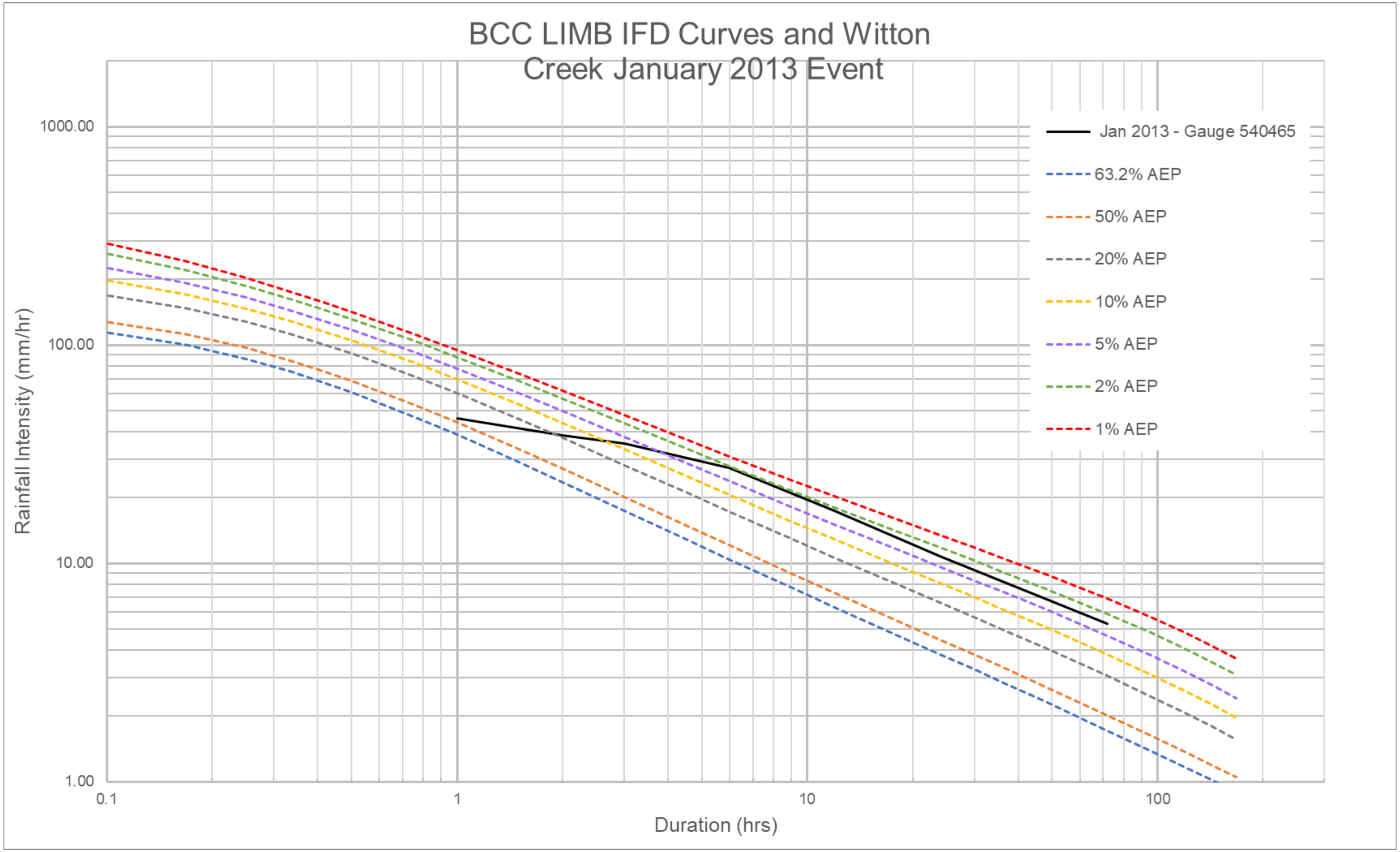
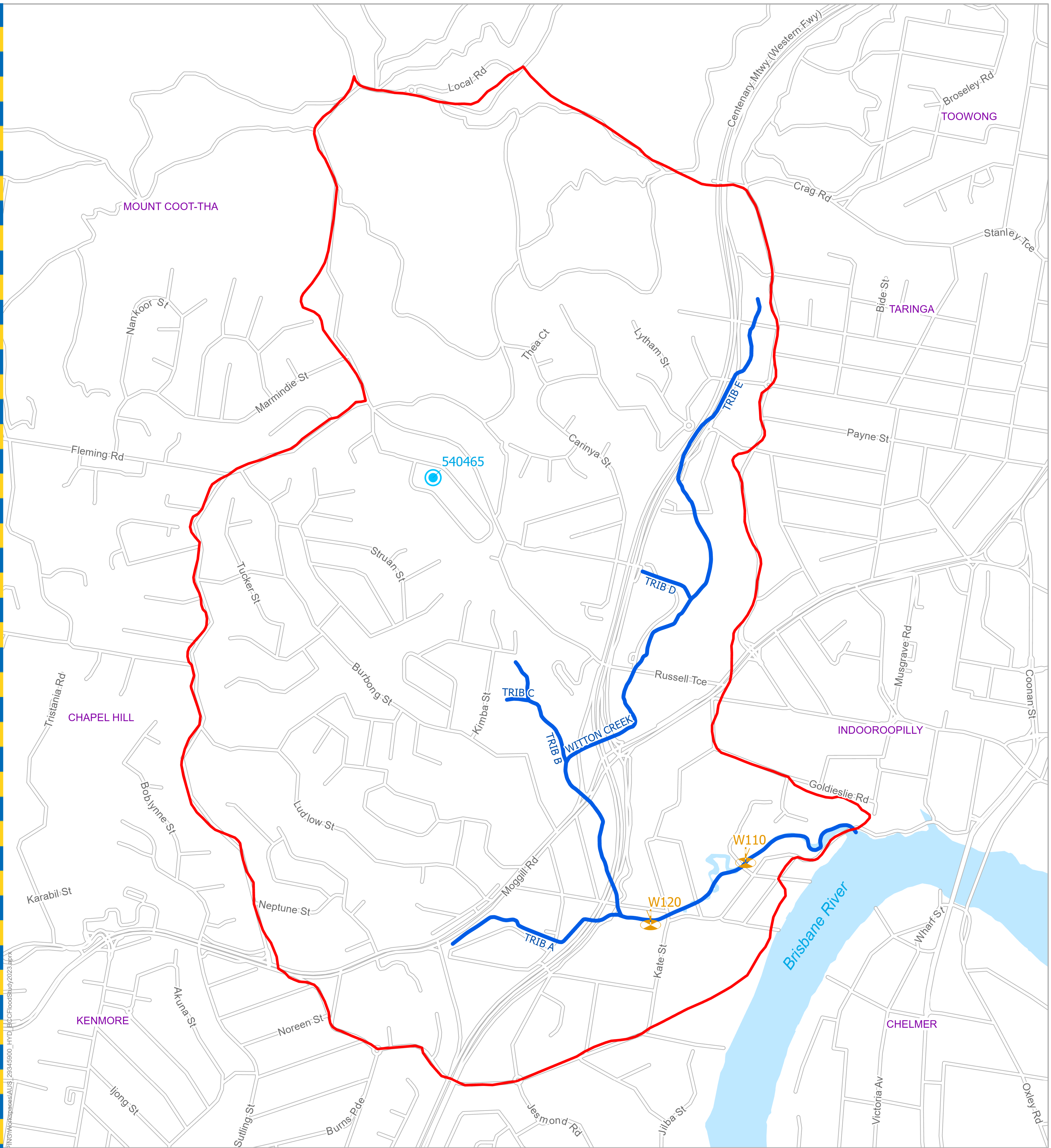
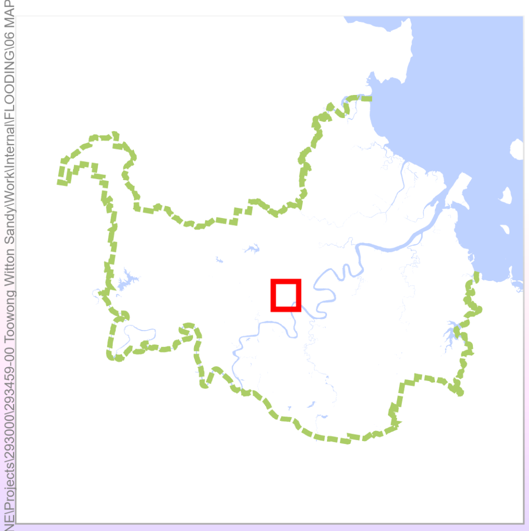


Figure 3.5: IFD Curve for January 2013





Legend

- Pluviograph Station
- Maximum Height Gauge
- Catchment Area
- Creek Centreline
- Streets

0 125 250 375 500
Metres

N W E S

Prepared : O SMITH
Checked : T WEBB
Revision : 1
Publication Date : 09 Nov 2023
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Witton Creek
Figure 3.6 MHG and Rainfall Gauge Locations
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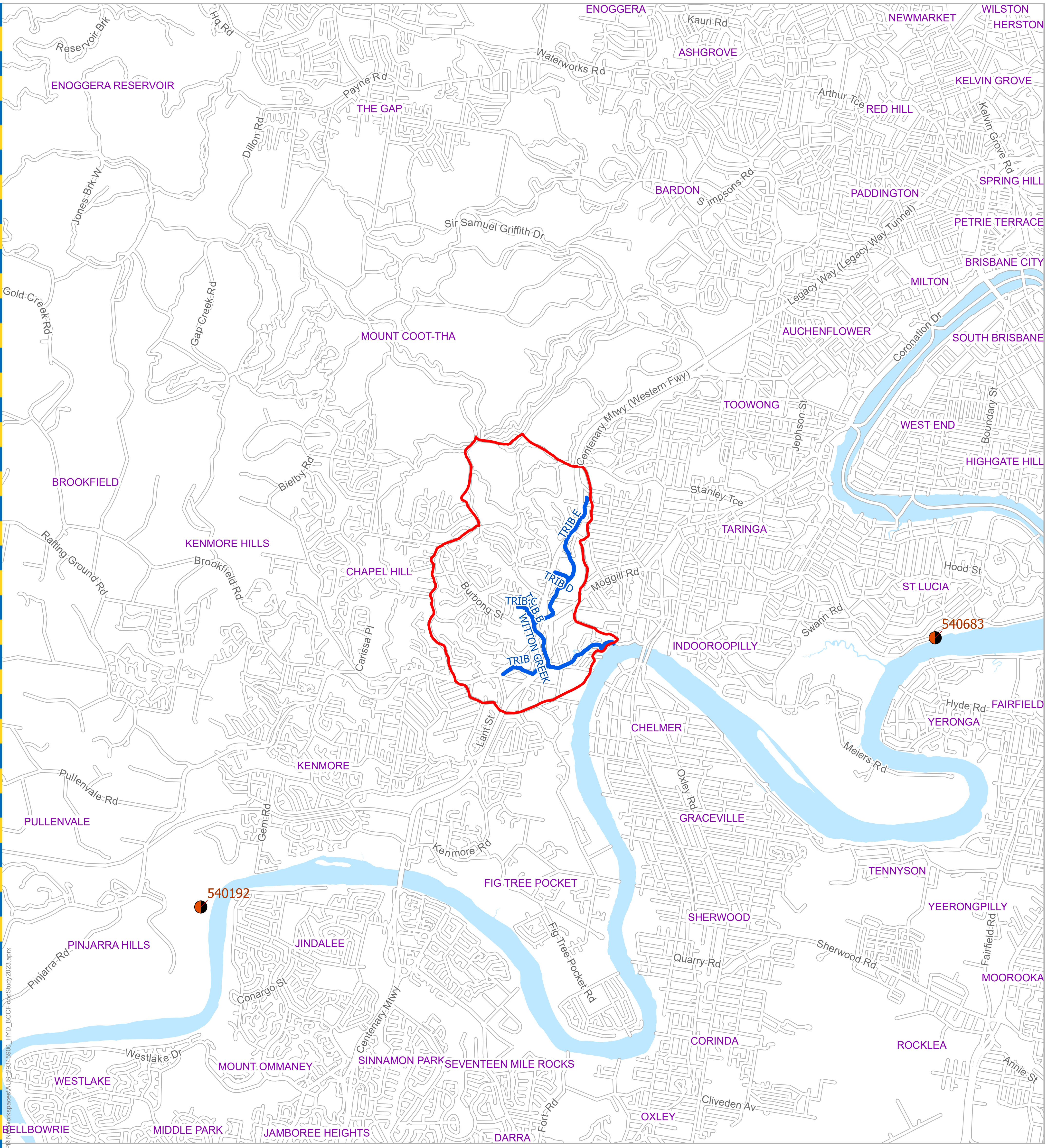

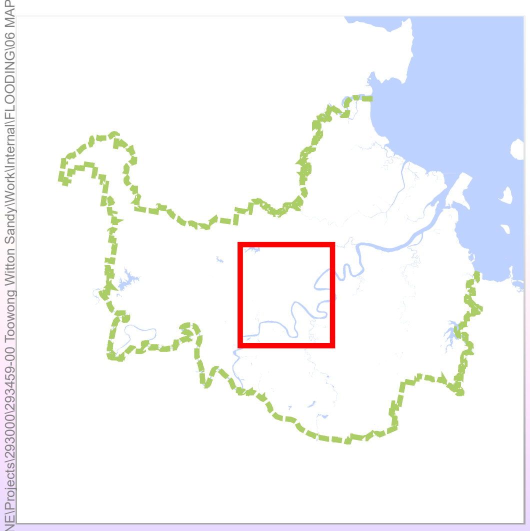

0 500 1,000 1,500 2,000

Metres

N W E S

Legend

- Brisbane River Stream Gauge
- Catchment Area
- Creek Centreline
- Streets

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Witton Creek Figure 3.7 Stream Gauge Locations Page 1 of 1

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4.0 Hydrologic Model Development and Calibration

4.1 Overview

The hydrologic model simulates the rainfall-runoff-routing processes within the catchment. Hydrologic modelling for this study was performed using the URBS (Version 6.62) software. URBS allows for the effects of development / urbanisation to be assessed, which makes it suitable for the large, urbanised catchments like Witton 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.

No URBS model was developed for the Witton Creek Catchment as part of the 2000 Witton SMP. The 2000 Witton SMP models utilised XP RAFTS hydrologic model in conjunction with a MIKE 11 hydraulic model. Accordingly, a new URBS model was developed to cover the Witton Creek Catchment.

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

For further details on this modelling approach refer to URBS User Manual.[1]

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 of the study.

For this study, the following URBS parameters were utilised:

[1] DG Carrol 2021 – URBS *A Rainfall Runoff Routing Model for Flood Forecasting and Design*, Version 6.6

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 Faction

The adopted sub-catchment parameters for calibration and verification events are presented in Appendix B. The same sub-catchment parameters were used for all calibration and verification events due to the recent age of the historical flood events (within the last 10 years) and the minimal changes in catchment and channel topography and development during this period.

4.2.2 Sub-catchment Delineation

The URBS model was divided into 39 sub-catchments and is shown in Figure 4.1. Based on the total catchment area of 4.09 km^2, the average sub-catchment size was 100 ha. The sub-catchments were delineated based on:

- 2019 BCC LiDAR (ALS)
- Location of major and minor tributaries
- BCC stormwater drainage GIS information
- Aerial photography (dated 2009 to 2022)
- Man-made boundaries, such as motorways and railways.
- Consideration for inflow locations for the hydraulic TUFLOW model

Sub-catchment delineation aimed to achieve similarly sized catchments, limit excessively small or large sub-catchments, limit elongated/odd shaped sub-catchments and ensure that there were at least five sub-catchments upstream of any routed total hydrograph inflow reporting locations.

4.2.3 Land-use and Impervious Area

The effect of development / urbanisation is modelled within URBS using the Urbanisation Index (U) and Impervious Fraction (I), where:

- Urbanisation Index (U) is used to determine the decrease in catchment lag due to urbanisation.
- Impervious Fraction (I) is used to determine the increase in runoff volume due to urbanisation.

The Urbanisation Index (U) for each sub-catchment is determined with respect to Urbanisation Indices; UL, UM, UH and UR respectively. The urbanisation indices 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 being occupied by Urban Low Density (UL) urbanisation index.

To determine the value of UL, UM, UH and UR for each sub-catchment, the following approach was taken:

- Adopt impervious fraction for each Urbanisation Indices.
- Determine the total impervious area of each sub-catchment.

Impervious Fractions for Urbanisation Indices

Urbanisation indices (UL, UM, UH and UR) were assigned for this study as per the following: UL (0.15), UM (0.5), UH (0.9) and UR (0 – URBS default).

Total Impervious Area

Total Impervious Area for each sub-catchment was determined based on a review of BCC City Plan 2014 land-use maps, adopted land-use percentage impervious (refer to Appendix C) and aerial photography. The impervious fraction of the road was assigned on a sub-catchment basis to reflect actual conditions. The adopted land use fraction impervious values were selected from review of aerial photography and previous BCC flood studies (BCC Cubberla Creek Flood Study 2017 and BCC Bulimba Creek Flood Study 2022).

With the impervious fractions for each urbanisation index and total impervious area for each sub-catchment defined, the following process was used to assign values:

(i) Each land-use zone from BCC City Plan 2014 was assigned an appropriate urbanisation index (UL, UM, UH, UR).

(ii) The area of each land use zone within a sub-catchment was determined and the total area of each urbanisation index within each sub-catchment was calculated.

(iii) The impervious area for each sub-catchment was calculated using the adopted fraction impervious for each Urbanisation Index.

(iv) The calculated impervious area was compared to the total impervious area for each sub-catchment.

The urbanisation index applied to each land-use is shown in Table 4.1.

Table 4.1: Adopted Urbanisation Indices per Land-Use

Land Use	Urbanisation Index (Ultimate Case)
Character residential (Character)	UM/UL*
Emerging community	UM
Environmental management	UR
Low density residential	UM
Mixed use (Centre frame)	UH
Mixed use (Corridor)	UH
Neighbourhood centre	UH
Open space (Local)	UR
Open space (Metropolitan)	UR
Road Corridors	UH
Special purpose (Utility services)	UH
Specialised centre (Large format retail)	UH

*UL classification altered based on aerial imagery of area

4.3 URBS Channel Data

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

L: Channel length (mandatory parameter)

Sc: Channel Slope

The channel length and average channel slope was determined using GIS software and the 2019 BCC Lidar (ALS).

4.4 Event Rainfall

4.4.1 Observed Rainfall

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

Thiessen Polygons were utilised for each event to enable the gauge rainfall to be apportioned to each of the sub-catchments in the URBS model, where those sub-catchments that fell totally within a polygon were fully assigned to the respective rainfall station. The Theisen Polygon distributions for all 6 events are presented in Appendix A for reference. For all six historical events, the Witton Creek Catchment was completely contained within the Thiessen Polygon of rainfall station 540465.

4.4.2 Rainfall Losses

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

The IL (mm) is the amount of rainfall that occurs before the start of surface runoff. The initial loss comprises factors such as interception storage (i.e. tree leave/canopies); depression storage (i.e. 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 predominately dependant on the underlying soil type and porosity.

4.5 Stream Gauge Rating Curve

There were no stream gauges located within the Witton Creek Catchment. No stream rating curves were used in the calibration and verification of this study.

4.6 Calibration and Verification Procedure

4.6.1 General

The calibration and verification process were 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 were no stream gauges located within the catchments, it was not possible to calibrate and verify the hydrologic model to observed hydrographs. Accordingly, it was not possible to calibrate and verify the volume and shape of the hydrographs, 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 with the TUFLOW model.

4.6.2 Methodology

The following methodology was undertaken for the hydrologic calibration and verification, where the results of the hydraulic calibration are presented in Section 5.4.

The following approach was taken for calibration of the URBS and TUFLOW models:

1. Observed flood event rainfall data was incorporated into the URBS models as outlined in Section 4.2.
2. Rainfall events (February 2020, March 2017, and May 2015) were run through URBS, where:
    a. URBS model parameters (α, β and m) initially selected were based upon URBS recommended ranges, catchment characteristics and previous BCC studies of similar catchments (BCC Cubberla Creek Flood Study 2017).
    b. IL and CL loss values were initially taken from the AR&R 2019 Data Hub values.
3. Inflows from the URBS calibration event runs were applied to the TUFLOW models and were run through the TUFLOW models. The simulated results were compared against the observed flood levels at the Maximum Height Gauges (MHG).

4. Iteratively, the TUFLOW and URBS model parameters were adjusted, where reasonable to do so, to match the MHG data. The following URBS parameters that were adjusted in calibration included catchment lag parameter (β) catchment non-linearity parameter (m) and pervious losses IL (mm) and CL (mm/hr). The adjustments to URBS IL (mm) were undertaken to represent the event specific rainfall at the start of the historic event.
5. The URBS and TUFLOW hydrographs for all events were compared at a number of locations within the model extent. The URBS channel lag parameter (α) was adjusted to replicate the results of the TUFLOW model.
6. Steps 2 to 5 were repeated as necessary.
7. A single set of URBS parameters (IL, CL, α, β and m) and TUFLOW model parameters were adopted based off calibration results.
8. The selected validation events (June 2016 and January 2013) were run through the calibrated URBS and the TUFLOW models with the preferred model parameters. The simulated results were compared against the MHG readings of observed validation events.

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

- Peak flows 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 4.2 indicates the start and finish times of the rainfall events applied in the hydrologic simulations as well as the time step used in the URBs model.

Table 4.2: Hydrologic Simulation Parameters

Event	Start Time	End Time	Duration (hours)	Timestep (min)
February 2020	6/02/2020 3:03 AM	6/02/2020 11:59 PM	20.9	5
May 2015	30/04/2015 4:13 AM	1/05/2015 11:59 PM	43.8	5
March 2017	29/03/2017 2:23 PM	30/03/2017 11:59 PM	33.6	5
June 2016	19/06/2016 2:43 AM	19/06/2016 11:59 PM	21.3	5
January 2013	25/01/2013 12:30 AM	27/01/2013 11:59 PM	71.5	5

4.8 Hydrologic Model Calibration Results

As the URBS model calibration and verification was undertaken in conjunction with the TUFLOW model, the peak flood level results can be found in the hydraulic model calibration and verification sections; Section 5.4 and Section 5.5. The consistency checks between the URBS and TUFLOW models are presented in Section 5.7.

The first calibration run used URBS parameters that were based on the URBS recommended values for parameters. [2]

Using the calibration and validation methodology outlined previously in Section 4.6, the calibration was undertaken until the results were considered satisfactory. During the calibration process, the channel lag parameter (α) was altered to achieve consistency between the URBS and TUFLOW hydrographs.

Table 4.3 indicates the parameters adopted from the hydrologic calibration of the three historical events.

Table 4.3: Adopted URBS Parameters

Parameter	Description	Adopted Value
Imp IL	Impervious Area Initial Loss (mm)	0
Imp CL	Impervious Area Continuing Loss (mm/hr)	0
*Perv IL	Pervious Area Initial Loss (mm)	18
*Perv CL	Pervious Area Continuing Loss (mm/hr)	1.4
α	Channel lag parameters	0.05
β	Catchment Lag Parameter	4
m	Catchment non-linearity parameter	0.8

*In line with AR&R 2019 Data Hub IL and CL recommended values

4.9 Hydrologic Model Verification Results

The adopted URBS parameters as per the calibration outcomes were then used in the two verification events (June 2016 and January 2013) to confirm model performance.

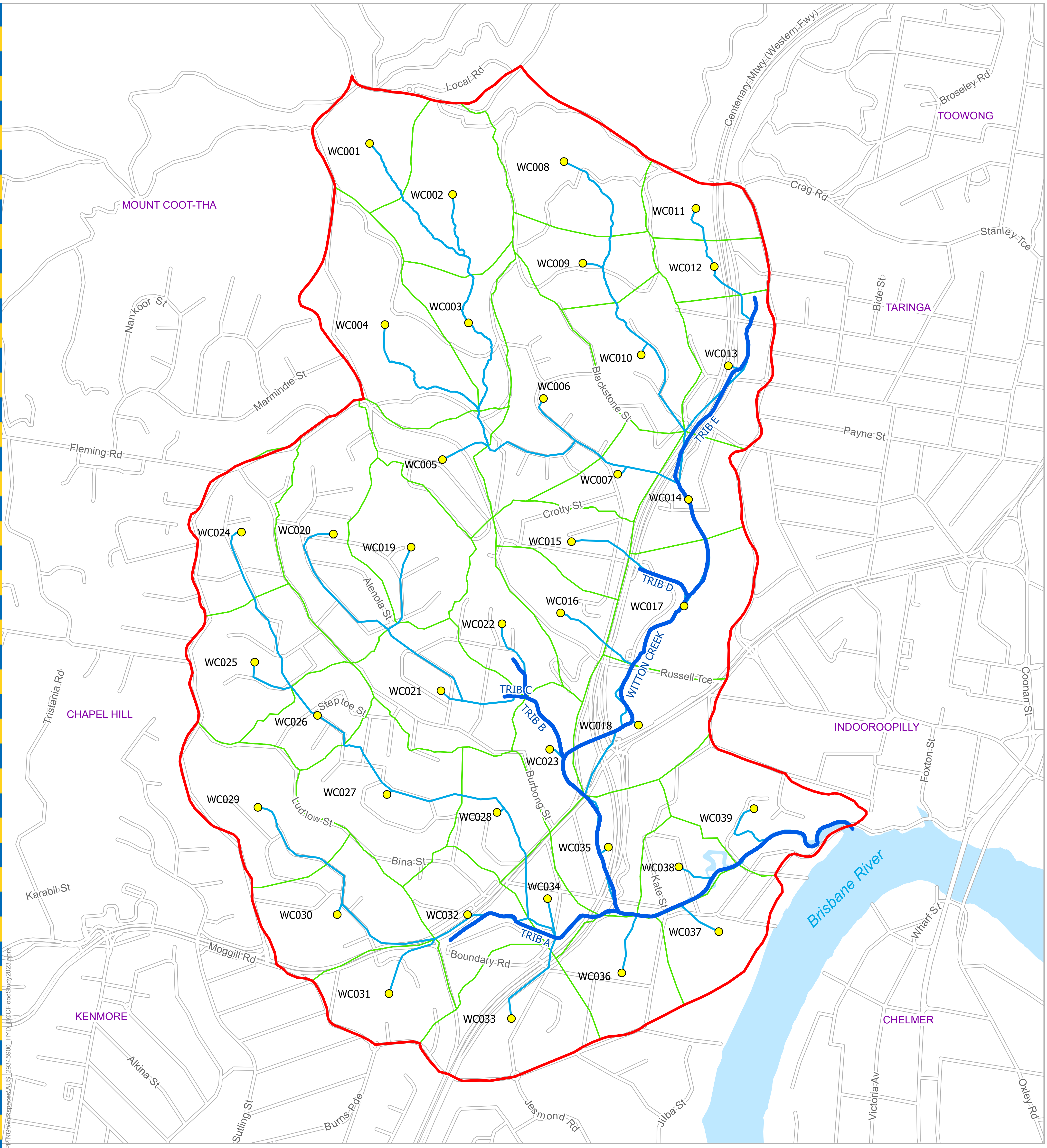
For the June 2016 event, the pervious area initial loss was increased to 60 mm. The initial loss was increased to 60 mm from review of the rainfall record preceding the June 2016 event (refer to Section 3.4.6.4) as the catchment had very dry conditions leading up to the June 2016 event. For the January 2013 event, the pervious area initial loss was kept per the recommended Data Hub values with an IL of 18 mm.

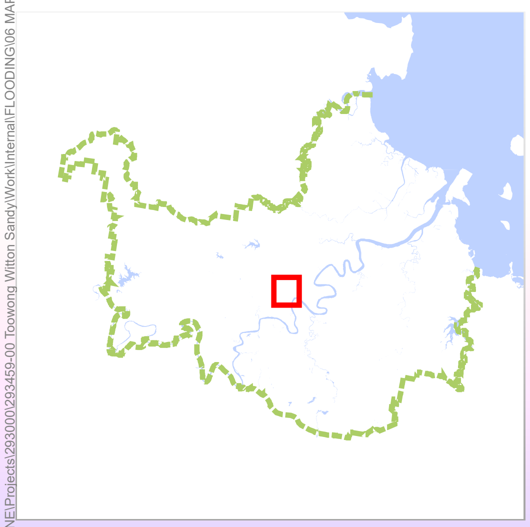
[2] DG Carrol 2021 – URBS *A Rainfall Runoff Routing Model for Flood Forecasting and Design*, Version 6.6

Satisfactory model verification was achieved for both verification events. As the URBS model calibration and verification was undertaken in conjunction with the TUFLOW model, the peak flood levels results can be found in the hydraulic model calibration and verification results sections – see Section 5.4 and Section 5.5.

4.10 URBS Model Consistency Checks (Historical Events)

As noted above, the results of the consistency check between the URBS and TUFLOW models are presented in Section 5.7. As mentioned above, the channel lag parameter (α) was required to be decreased in calibration for better consistency between the URBS and TUFLOW hydrographs. No other URBS parameters were adjusted.





Legend

- URBS Nodes
- Creek Centreline
- URBS Links/Routes
- URBS Subcatchments (1-39)
- Catchment Area
- Streets

0 125 250 375 500
Metres

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Prepared : O SMITH
Checked : T WEBB
Revision : 1
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Witton Creek
Figure 4.1 URBS Model Schematisation
Page 1 of 1

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5.0 Hydraulic Model Development and Calibration

5.1 Overview

The previous hydraulic model of Witton Creek was a one-dimensional MIKE 11 model, developed for the 2000 Witton SMP. To achieve best practice, it was considered to develop a new 1d / 2d model that would provide:

- Better representation of floodplain flooding characteristics
- Better representation of creeks and tributaries
- Better representation of stormwater trunk drainage networks
- A more efficient tool to produce flood mapping products.

The TUFLOW hydrodynamic model was selected for the hydraulic analysis of the Witton Creek Catchment. The TUFLOW modelling was undertaken in version 2020-10-AF.[3]

5.2 Model Development

5.2.1 Model Extents

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

5.2.2 Utilised Hydraulic Model Data

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

- 2019 BCC LiDAR (ALS)
- 2023 Witton Creek Field Survey (35 cross-sections and 8 hydraulic structures)
- Aerial photography – 2009 to 2021
- BCC City Plan 2014
- BCC GIS Stormwater Database (including layers: Pipe, Culvert, Surface Drain, Gully, Junction, Manhole, End Structure, Pipe Survey Waterbody)
- Nearmap 2022 Building Footprint layer
- Hydraulic Structure drawings /reference sheets. Refer to Appendix K for further information.
- 2000 Witton SMP MIKE 11 model structure information as a check for hydraulic structure details

[3] Latest version of TUFLOW at project commencement.

- Drawings / as-cons (where available) as a check for hydraulic structure details

5.2.3 Base Terrain Data

The base 2d terrain consists of a 2 m grid model with sub-grid sampling (SGS) of 1 m. TUFLOW SGS is a method that improves the accuracy of flood and water flow simulations by dividing larger computational grids into smaller sub-grid cells, this technique captures detailed flow patterns and hydraulic interactions at a finer level of detail within larger-scale computational grids.

The base 2d terrain was created from a 1m ASCII grid file (MGA Zone 56) of the 2019 BCC LiDAR (ALS). Details of this dataset are provided previously in Section 3.2.2.

5.2.4 Waterways

The waterways (both major and minor) were modelled within the 2D domain. Review of the base terrain showed that both major and minor waterways were sufficiently captured within the 2D base terrain. Table 5.1 summarised the 2D waterways included within the TUFLOW model as well as the modelled length, the downstream waterway, and the major source(s) of data.

Table 5.1: Waterways included in the TUFLOW model

Water Way	Model Length (km)	Downstream Confluence	Major Data Source for 2d waterways
Witton Creek	2.45	Brisbane River	2019 BCC LiDAR (ALS)
Witton Creek Tributary A	0.59	Witton Creek	2019 BCC LiDAR (ALS)
Witton Creek Tributary B	0.35	Witton Creek	2019 BCC LiDAR (ALS)
Witton Creek Tributary C	0.07	Tributary B	2019 BCC LiDAR (ALS)
Witton Creek Tributary D	0.17	Witton Creek	2019 BCC LiDAR (ALS)
Witton Creek Tributary E	0.64	Witton Creek	2019 BCC LiDAR (ALS)

Cross-sectional field survey of major waterways for each creek catchment was utilised (previously outlined in Section 3.2.1). This field survey was reviewed against the BCC 2019 LIDAR (ALS) and the TUFLOW base terrain for the TUFLOW model. Where major difference between the data sets occurred, the base terrain was modified to appropriately capture the cross-section of the waterways.

Table 5.2 summarises the waterways that were adjusted using the field survey within the Witton Creek model.

Table 5.2: Waterways adjustment in 2D domain for the Witton Creek TUFLOW model

Waterway	Modelled Length (m)	Downstream Confluence	Adjustment
Witton Creek (Aaron Street Bridge to Survey Section 31)	162	Brisbane River	Section lowered and widened to match field survey 1/03/2023 from Aaron Place Bridge Section S12 to Section S31 along the Witton Creek.
Witton Creek (Survey Section 31 to Brisbane River)	346	Brisbane River	Section lowered and widened to match field survey 1/03/2023 from Section S31 to Radnor Street Bridge Section S13 and the Brisbane River.

5.2.5 Land Use and Hydraulic Roughness

The Manning's 'n' roughness values adopted within the 2d domain of the TUFLOW model are shown in Table 5.3. The assignment of suitable roughness values to the land use / topographical features was undertaken using a combination of site visit information, aerial photography, BCC City Plan 2014, BCC Flood Study Procedure V9.0, previous BCC flood studies (BCC Cubberla Creek Flood Study 2017 and BCC Bulimba Creek Flood Study 2022) and relevant hydraulic literature (i.e. BCC Natural Channel Design 2013 Appendix C, Table 6.2.2 of Book 6, Chapter 2 in ARR 2019).

Building footprints were modelled with high Manning's 'n' (set at n=1.0) as per BCC recommendations.

Table 5.3: Adopted TUFLOW roughness parameters

Topographical Feature / Land Use	Adopted Manning's 'n'
Land Use BCC City Plan 2014	
Character Residential	0.15
High Density Residential	0.15
Medium Density Residential	0.15
Low Density Residential	0.12
Low – Medium Density Residential	0.15
Medium Density Residential	0.15
Low Impact Industry	0.10
Mixed use	0.15
Centre (District Major Principle)	0.15
Neighbourhood Centre	0.10
Community Facilities (Cemetery)	0.04
Community Facilities (Community Purposes)	0.10
Community Facilities (Emergency Services)	0.15

Topographical Feature / Land Use	Adopted Manning's 'n'
Land Use BCC City Plan 2014	
Community Facilities (Education Purposes)	0.06
Community Facilities (Health Care Purposes)	0.15
Emerging Communities	0.12
Environmental management and conservation	0.08
Sport and recreation	0.04
Open Space	0.04
Rural	0.04
Special Purpose (Transport Infrastructure)	0.04
Special Purpose (Utility Services)	0.04
Specialised Centre (Large Format Retail)	0.15
Specialised Centre (Major Education and Research)	0.10
Additional Roughness Categories	
Channel – concrete lined	0.02
Vegetation – Light density	0.035
Vegetation – Medium Density	0.06
Vegetation – High Density	0.15
Waterways – Vegetated Channel	0.035
Waterways - Medium Vegetated	0.04
Waterway - Heavily Vegetated	0.07
Road pavement	0.02
Road verge	0.03
Building Footprint	1.00

5.2.6 Hydraulic Structures

The major bridge and culvert structures within the model extents were included in the TUFLOW model. These structures generally consisted of the waterway crossing from motorways, railways, major roads, local roads, pedestrian / bikeway crossings and private access roads.

The hydraulic structures included in the TUFLOW model are presented in Appendix K. Table 5.4 indicates the location and details of these structures within the TUFLOW model and modelling approach used.

Table 5.4: Summary of Hydraulic Structure included within the Witton Creek

Creek	Structure ID	AMTD (m)	Structure Location	Structure Detail	Model Representation
Witton Creek	S1	6	Radnor Street	4 Lane Bridge	2d Lfcsh
	S2	441	Aaron Place	Single Span Bridge	2D Lfcsh
	S3	714	Kate Street	4 x 3m (W) x1.5 m (H) Culvert	1D Culvert
	S5	892	Witton Road	3 x 3m (W) x1.5 m (H) Culvert	1D Culvert
	S6	948	Western Freeway	4x 3m (W) x 3 m (H) Culvert	1D Culvert
	S7	1231	Moggill Road	4 x DN1500 Culvert	1D Culvert
	S8	1423	Western Freeway Onramp	4 x DN1950 Culvert	1D Culvert
	S9	1482	Western Freeway	3 x DN1950 Culvert	1D Culvert
	S10	1785	Russell Terrace	4 x DN1650 Culvert	1D Culvert
	S11	1800	Russell Terrace	Single 1.2 m (W) by 0.6 m (H) Culvert	1D Culvert
Witton Creek Tributary A	S4	24	Western Freeway Bikeway	Single Span Pedestrian Bridge	2D Lfcsh
	S14	428	Kennewell Park	Single DN1700 Culvert	1D Culvert

Bridges

Bridges structures were modelled as 2D layer flow constriction shapes.

METHOD C was selected for the layered form loss coefficient (FLC) approach as recommended by TUFLOW. TUFLOW recommends the use of METHOD C as it is shown to overcome the overestimation of bridge losses of the previous methods (i.e. CUMULATE or METHOD B) and emulates the behaviour from CFD bridge modelling. METHOD C utilised the CUMULATE through to the top of Layer 3 and PORTION above layer 3.

In the absence of practical methodology for calculation of FLCs for METHOD C, all bridges were assigned a FLC of 0.15 for Layer 2 and a FLC of 0.13 for Layer 3. These FLC values were taken from a real-world, calibrated bridge crossing example outlined in TUFLOW release notes 2020-10-AD[4]. The FLC's documented within the aforementioned TUFLOW release notes (i.e. for the real-world example bridge structure) were applied for the purposes of this flood study, as the example bridge structure is relatively similar to the bridges located within the Witton Creek Catchment.

The head-losses across two (2) bridges within the Witton Creek catchment were validated utilising HEC-RAS modelling software, as recommended in the TUFLOW manual. Refer to Section 5.6 for further details.

5.2.7 Piped Drainage

Although this flood study is for the analysis of open channel/creek systems, Witton Creek Catchment is a heavily urbanised catchment. Accordingly, it was necessary to include sections of stormwater trunk drainage network to determine flood levels more accurately. This stormwater trunk drainage network was included within the model as 1d network. Pipes equal to and greater than 0.6m diameter were modelled, with pipes smaller than 0.6m in diameter only modelled where required for continuity of the main trunk drainage lines. The pipe network data used in the modelling was taken from BCC stormwater network information.

The flow interchange between the 2D domain and the 1D pipe network was assumed to occur "freely" at the inlet pits, such that the hydraulic control would be the limiting size of the pipe and not the size of the pit inlet.

Pipe roughness for reinforced concrete pipe was selected as a Manning's n of 0.013. Pipe roughness was selected based off relevant hydraulic literature (i.e. HEC-RAS Manual Version 6.3, 2020 or AR& R2019 Book 6, Chapter 2).

5.2.8 Boundary Conditions

5.2.8.1 Inflow Boundaries

Inflows to the TUFLOW hydraulic model were taken from the URBS hydrologic model, where inflows were applied as either:

- URBS sub-catchment routed rainfall excess hydrographs to the 2d domain along waterways via 2d_sa to defined 'streamlines',

[4] TUFLOW Classic and HPC 2020-01 and 2020-10 Release Notes, 2020, TUFLOW

- URBS sub-catchment routed rainfall excess hydrographs to the bottom of 1d piped network via 1d_bc,
- URBS routed total inflow hydrograph (single location) to the 2d domain along waterways via 2d_sa to defined ‘streamlines’

All inflows were represented as a discharge versus time (Q-T) relationship. The inflow locations are indicated in Figure 5.1. Inflow locations were generally adopted to appropriately fit the URBS model sub-catchment schematisation.

5.2.8.2 Downstream Boundary

A varying water level versus time (H-T) boundary was used to represent the downstream boundary condition at the mouth of Witton Creek for calibration and verification. The H-T boundary was derived based upon the interpolation between the closest upstream and downstream river gauges as there is no stream gauge at the mouth of Witton Creek. The mouth of Witton Creek is located along the Brisbane River at AMTD 41800 m, which resulted in the two closest gauges to the confluence being:

- 540192 – Jindalee AL (E6731) (Upstream)
- 540683 – St Lucia AL (E1856) (Downstream)

5.2.9 Run Parameters

5.2.9.1 TUFLOW Solver

The TUFLOW model was run using TUFLOW’s Heavily Parallelised Compute (HPC) solver to reduce and optimise simulation runtimes of the models.

HPC solver is an alternative 2D solver to TUFLOW Classic solver. HPC provides parallelisation of TUFLOW models, which allows for a single model to be run on a GPU graphics card and/or across multiple CPU cores.

5.2.9.2 Time Step

The following time steps were used:

- 1D ESTRY component was set to 0.5 second.
- 2D TUFLOW component time step was set to 1 second.

The TUFLOW HPC solver uses an adaptive timestep derived from the hydraulic conditions during simulation. With the HPC solver, the time step commands define:

- The first calculation time step in the 2D TUFLOW component and all subsequent calculations are completed using the adaptive time step approach.
- The maximum limiting timestep of the 1D ESTRY solver.

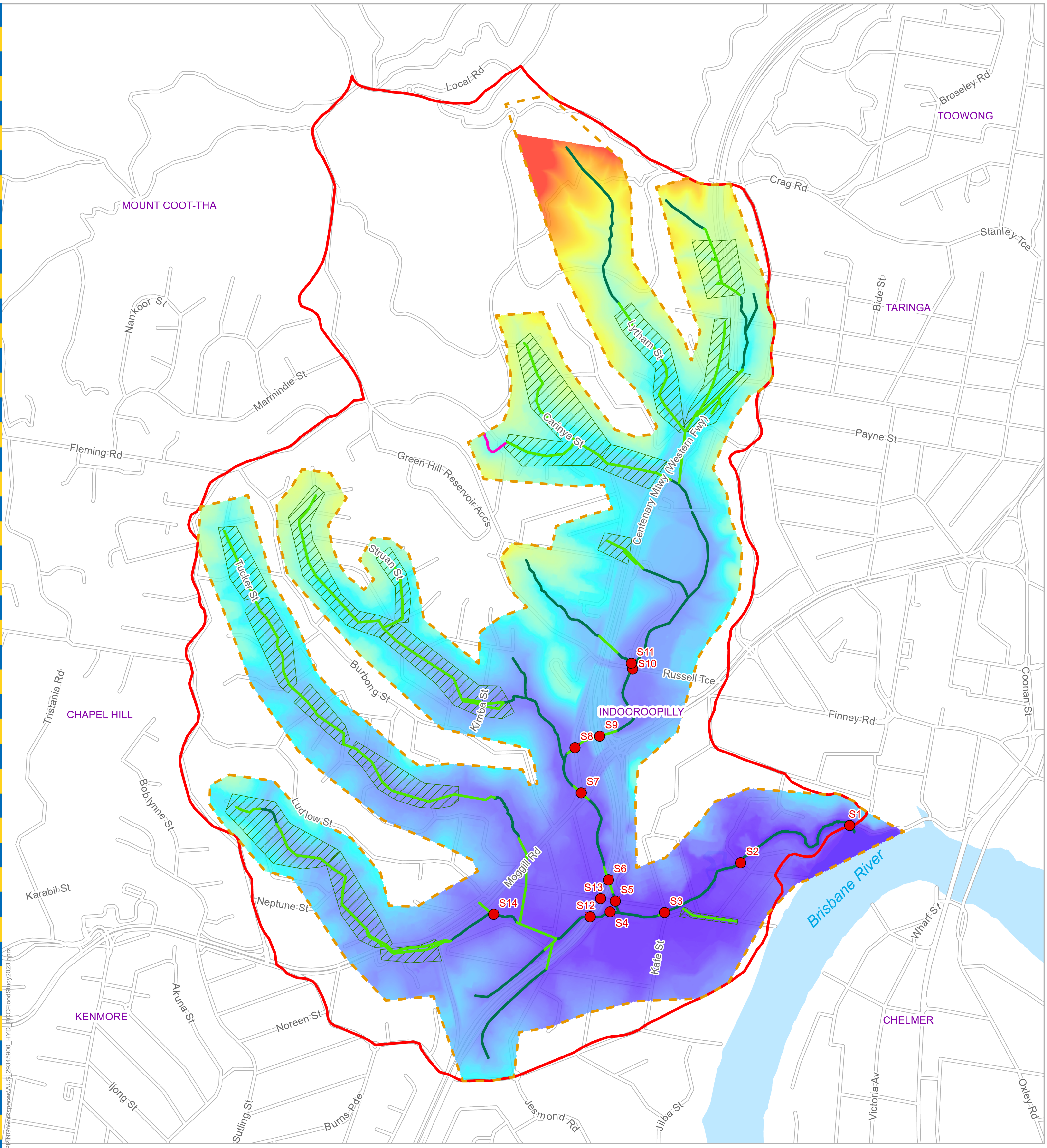
5.2.9.3 Eddy Viscosity

With the adoption of the TUFLOW HPC solver, the HPC solver defaults to a new eddy viscosity (turbulence) model that combines both 2D and 3D turbulence effects. The model is a slightly adapted version of that described by Wu et. al. 2005 [5]. Unlike the Smagorinsky model, where the turbulent length scale is related to cell size, the length scales used in the Wu model are related to water depth, and hence the computed eddy viscosity is not related to or dependent on cell size. This has been shown to significantly improve the cell-size convergence of model results compared to the Smagorinsky model.

The TUFLOW HPC default values of Wu Coefficient 3D (C_{3D}) of 7 and 2D (C_{2D}) of 0 were applied per TUFLOW recommendations in release notes 2020-10-AD. Default coefficients have been found to be agreeable through benchmarking by TUFLOW[6].

[5] A depth-averaged two-dimensional model for flow, sediment transport, and bed topography in curved channels with riparian vegetation, Weiming Wu, F. Douglas Shields Jr., Sean J. Bennett, and Sam S. Y. Wang, WATER RESOURCES RESEARCH, VOL. 41

[6] TUFLOW Classic and HPC 2020-01 and 2020-10 Release Notes, 2020, TUFLOW



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Prepared : O SMITH
Checked : T WEBB
Revision : 1
Publication Date : 05 Sep 2023
Project Number : 293459-00

Legend

- Catchment Area
- Model Boundary
- Hydraulic Structure
- 1d Piped Drainage
- 1d Inflow Location
- 2d Inflow Location – Rainfall Excess
- 2d Inflow Location – Total Inflow
- Streets

TUFLOW Model
158.227
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Witton Creek
Figure 5.1 TUFLOW Model Schematisation
Page 1 of 1

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5.3 Calibration Procedure

5.3.1 Tolerances

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

- Continuous recording stream gauges - within ± 0.15 m of the peak flood level.
- MHGs - within ± 0.3 m of the peak flood level.
- Debris marks - within ± 0.4 m of the peak flood level.
- Good replication of the timing of peaks and troughs.

As no stream gauges are located within the Witton Creek Catchment, calibration and verification was undertaken to MHGs, debris marks, and in comparing consistency between URBS and TUFLOW hydrographs.

5.3.2 Methodology

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

1. Using the flow inputs from URBS model, run the calibration events (February 2020, March 2017, May 2015) through the TUFLOW model and compare the simulated results against the observed flood levels at the MHGs.
2. Iteratively adjust the URBS and TUFLOW model parameters and re-run the model with the aim of achieving a good fit with the observed data. The predominant model parameter adjusted was bed resistance/hydraulic roughness (Manning's 'n'), water course bathymetry, and structure losses.
3. Adopt model parameters based on the calibration results.
4. Using the flow inputs from the URBS model, run the two verification events (June 2016 and January 2013) through the calibrated TUFLOW model and compare the simulated results against the observed levels at the MHGs.

As the creek conditions for all historical events were generally similar, the same model schematisation and parameters have been used for all five historical events; with the hydrologic flow inputs and downstream boundary at the Brisbane River updated for each historical event. This methodology ensures that the TUFLOW model is sufficiently robust to be utilised for the design and extreme event modelling.

5.4 Hydraulic Model Calibration Results

5.4.1 Calibration Overview

The following sections outline the outcomes of the calibration and validation for the Witton Creek hydraulic model. The flood level differences between modelled results and MHG records have been classified into 3 categories as illustrated in Table 5.5.

Table 5.5: Legend for MHG Comparison (tolerance thresholds)

Difference Ranges (m)	Description
± 300	Within Tolerance
+300 – +400	Slightly Above BCC Tolerance
> +400	Above MHG Tolerance Requirement

To achieve calibration across all calibration events, the following parameters were adjusted:

- Exit loss coefficient of 0.5 was applied for culverts in line with flow discharging to open channels. The adjustment was taken from review of velocities through the waterways, and accordingly, to account for the approach and departure velocities for the culvert structures. This also has drawn upon separate research work that Arup has conducted, which has shown that standard culvert loses (particularly the outlet loss noting it's default value of 1.0) can generate conservative / increased head loss. An outlet culvert loss of 0.5 was adopted at culverts discharging in line with the departing flowpath within the watercourse, and where the departing flow was channelised and non-stationary. This generates a total head loss factor of 1.0 across nested 1D/2D culverts, as opposed to the value of 1.5 that is often applied as a default.
- Delineation of hydraulic roughness (Manning's n) for the creek banks and creek bed in the waterway section from Aaron Bridge (AMTD 441 m) to the Brisbane River (AMTD 0 m). This section of Witton Creek is a constrained waterway and controls water levels at MHG gauge W110. The waterways hydraulic roughness was split into banks and creek bed, to appropriately capture the vegetation conditions along the bank and the smooth/muddy creek bed.
- Adjustment of the topography of the waterway section from Aaron Bridge (AMTD 441 m) to Brisbane River to remove erroneous triangulation features in the 2019 BCC LiDAR (ALS) and reinforce the creek channel size. The topography of the creek was adjusted using field survey levels from Aaron Place bridge through to the Brisbane River.

5.4.2 February 2020

The February 2020 event was simulated in TUFLOW for the peak of the flood for 6.5 hours from 6/02/2020 5:21 pm. Table 5.6 provides a comparison between the TUFLOW water level results and the MHG recorded peak flood levels. This table shows that at the operational MHGs the simulated flood levels were within the desired flood level tolerance at W110 and W120.

Table 5.6: Calibration to Peak Flood Level Data (February 2020)

Gauge ID	Location	Record Peak WL (m AHD)	Simulated Peak Water Level (m AHD)	Difference (m)
W110	DS Aaron Street Bridge	2.73	2.77	0.04
W120	US Kate Street/Witton Road Culvert	3.56	3.83	0.27

5.4.3 March 2017

The March 2017 event was simulated in TUFLOW for the peak of the flood for 8.5 hours from 30/03/2017 5:04 am.

Table 5.7: provides a comparison between the TUFLOW water level results and the MHG recorded peak flood levels. This table shows that at the operational MHGs the simulated flood levels were within the desired flood level tolerance at W120 and W110.

Table 5.7: Calibration to Peak Flood Level Data (March 2017)

Gauge ID	Location	Record Peak WL (m AHD)	Simulated Peak Water Level (m AHD)	Difference (m)
W110	DS Aaron Street Bridge	3.67	3.96	0.29
W120	US Kate Street/Witton Road Culvert	4.64	4.64	0.00

5.4.4 May 2015

The May 2015 event was simulated in TUFLOW for the peak of the flood for 12 hours from 1/05/2015 1:13 pm.

Table 5.8: provides a comparison between the TUFLOW water level results and the MHG recorded peak flood levels. This table shows that at the operational MHGs the simulated flood levels were within the desired flood level tolerance.

Table 5.8: Calibration to Peak Flood Level Data (May 2015)

Gauge ID	Location	Record Peak WL (m AHD)	Simulated Peak Water Level (m AHD)	Difference (m)
W110	DS Aaron Street Bridge	3.53	3.48	-0.05
W120	US Kate Street/Witton Road Culvert	4.30	4.30	0.00

5.5 Hydraulic Model Verification Results

5.5.1 Witton Creek validation event results

5.5.1.1 June 2016

The June 2016 event was simulated in TUFLOW for the peak of the flood for 5.5 hours from 19/06/2016 3:13 pm.

Table 5.9: provides a comparison between the TUFLOW water level results and the MHG recorded peak flood levels. This table shows that at the operational MHGs the simulated flood levels were within the desired flood level tolerance.

Table 5.9: Validation to Peak Flood Level Data (June 2016)

Gauge ID	Location	Record Peak WL (m AHD)	Simulated Peak Water Level (m AHD)	Difference (m)
W110	DS Aaron Street Bridge	3.53	3.66	0.13
W120	US Kate Street/Witton Road Culvert	4.29	4.30	0.01

5.5.1.2 January 2013

The January 2013 event was simulated in TUFLOW for the peak of the flood for 15 hours from 27/01/2013 8:59 am.

Table 5.10: provides a comparison between the TUFLOW water level results and the MHG recorded peak flood levels. This table shows that at the operational MHGs the simulated flood levels were within the desired flood level tolerance.

Table 5.10: Validation to Peak Flood Level Data (January 2013)

Gauge ID	Location	Record Peak WL (m AHD)	Simulated Peak Water Level (m AHD)	Difference (m)
W110	DS Aaron Street Bridge	3.74	3.60	-0.04
W120	US Kate Street/Witton Road Culvert	4.27	4.38	0.18

5.6 Hydraulic Structure Verification

5.6.1 Bridge Head-loss Checks

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 (i.e. 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:

- Aaron Place Bridge (S2)
- Radnor Street Bridge (S1)

Table 5.11 provides a comparison of the head-loss across the structure between TUFLOW and the HEC-RAS model, with the last row of each table corresponding with the 0.05% AEP peak discharge. Generally, the TUFLOW head-losses for the bridge structures checked were close to the HEC-RAS values, and of similar / consistent magnitude. This is considered reasonable and gives confidence in the TUFLOW results.

Table 5.11: HEC-RAS Bridge Head-Loss Checks

Flows (m3/s)	HEC-RAS Head-Loss	TUFLOW Head Loss	Difference (m) TUFLOW vs HEC
Structure S1 – Radnor Street Bridge (Witton Creek)			
22.4	0.07	0.05	-0.02
36.7	0.38	0.47	0.09
39.3	0.11	0.13	0.02
40.9	0.14	0.11	-0.03
48.6	0.18	0.21	0.03
55.7	0.08	0.12	0.04
60.7	0.1	0.15	0.05
65.4	0.12	0.19	0.07
70.6	0.15	0.22	0.07
75.3	0.2	0.26	0.06
80.1	0.25	0.30	0.05
85.4	0.24	0.18	-0.06
90	0.26	0.13	-0.13
Structure S2 – Aaron Place Bridge (Witton Creek)			
22.0	0.36	0.35	-0.01
35.7	0.28	0.30	0.02
38.5	0.37	0.31	-0.06
39.9	0.40	0.33	-0.07
47.9	0.60	0.47	-0.13
55.4	0.99	0.85	-0.14
60.2	0.69	0.83	0.14
65.6	1.04	0.80	-0.24
70.4	0.94	0.80	-0.14
75.1	1.06	0.80	-0.26
80.8	1.12	0.93	-0.19
85.1	1.15	0.85	-0.30
87.5	0.99	0.80	-0.19

5.7 Hydrologic-Hydraulic Model Consistency Checks (Historical Events)

Comparison checks were undertaken between the URBS and TUFLOW model to understand how closely the hydrologic and hydraulic models match and as a means to confirming whether the URBS model was adequately calibrated. Accordingly, comparative plots were undertaken at four locations across the catchment. The locations where the comparative plots were undertaken are as follows:

i) Moore Park (Witton Creek AMTD 2272 m)

ii) Russell Terrace, downstream of culvert (S10) (Witton Creek AMTD 1785 m)

iii) Aragon Street (Witton Creek Tributary A AMTD 585m)

iv) Radnor Street, upstream of bridge (S1) (Witton Creek AMTD 6 m)

Figure 5.2 to Figure 5.6 provide comparative plots at Russell Terrace (ii) on Witton Creek. The remainder of the comparative plots for the other listed locations are provided in Appendix D. Table 5.12 provides a comparison of the peak flows at the 4 locations across the calibration and verification events.

The results of the comparison indicate that the URBS and TUFLOW models show a good correlation with relation to peak flow and hydrograph timing and shape across the model.

In the upper sections of the catchment and at Tributary A, there is good comparison between the TUFLOW and URBS hydrographs as the URBS model and TUFLOW can both appropriately capture the linear routing of these steeper, conveyance-dominated upper catchment reaches. However, further downstream in the mid and lower part of the catchment, the floodplain storage within the catchment takes effect and is better represented in the TUFLOW model (as storages were not explicitly modelled within the URBS model, and the URBS model struggles with non-linear routing). Accordingly, the URBS peak flow typically exceeds the TUFLOW peak flow.

At the outlet of the catchment (Radnor Street), the difference between the TUFLOW and URBS hydrographs tends to increase slightly. This difference is owing to the significant storage effects due to the wide expanse of floodplain areas in the mid and lower catchment areas that is captured within the TUFLOW model. Accordingly, the URBS peak flow typically exceeds the TUFLOW peak flow slightly.

Table 5.12: Peak Flow comparision between URBS and TUFLOW*

Location	Model	Peak Flow (m^3/s)				
		Feb 2020	Mar 2017	May 2015	Jun 2016	Jan 2013
Moore Park	URBS	7.5	21.9	13.6	14.8	15
	TUFLOW	7.8	21.5	14	15.3	15.1
Russell Terrace (S10)	URBS	8.7	25.7	16.2	17.5	17.9
	TUFLOW	9.7	26.1	16.8	18.6	18.3
Aragon Street (Witton Creek Tributary A)	URBS	2.5	6.5	3.8	5.2	4.2
	TUFLOW	2.8	6.6	4.1	5.4	4.3
Radnor Street (S1)	URBS	20.5	55.7	37.3	43.9	40.8
	TUFLOW	22.3	48.8	36.7	41	39.4

** Note that the peak flows reported in the table above show the TUFLOW model having slightly higher peak flows than the URBS model, with the exception of Radnor St. This is likely due to the setup of the reporting locations in the TUFLOW model being slightly downstream from the URBS catchment outlet. Accordingly, the TUFLOW peak flows show a slight increase in peak flow as the reporting location captures a small portion of the downstream sub-catchment flows. Radnor St URBS discharges are generally higher than those of TUFLOW expectedly owing to the mid- to lower-catchment storage effects, which cannot be accurately reflected in URBS.*

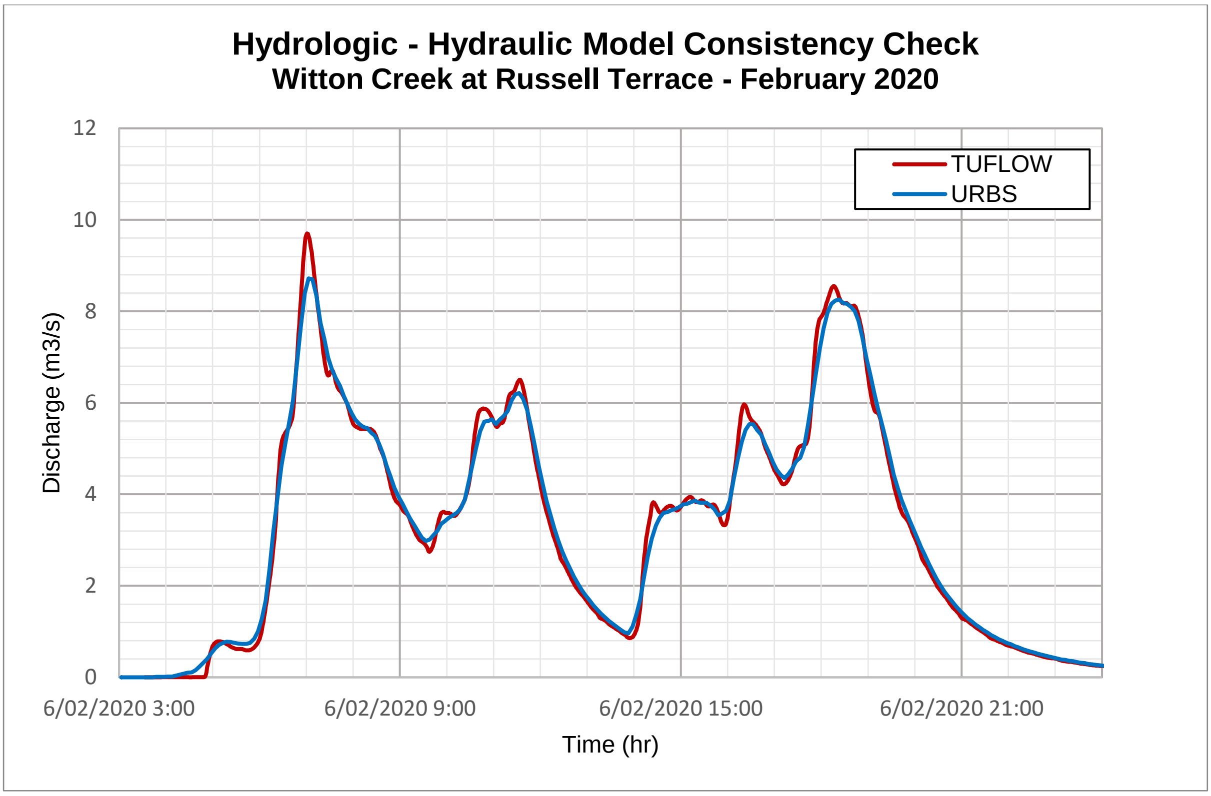


Figure 5.2: Witton Creek at Russell Terrace URBS vs TUFLOW comparision (February 2020)

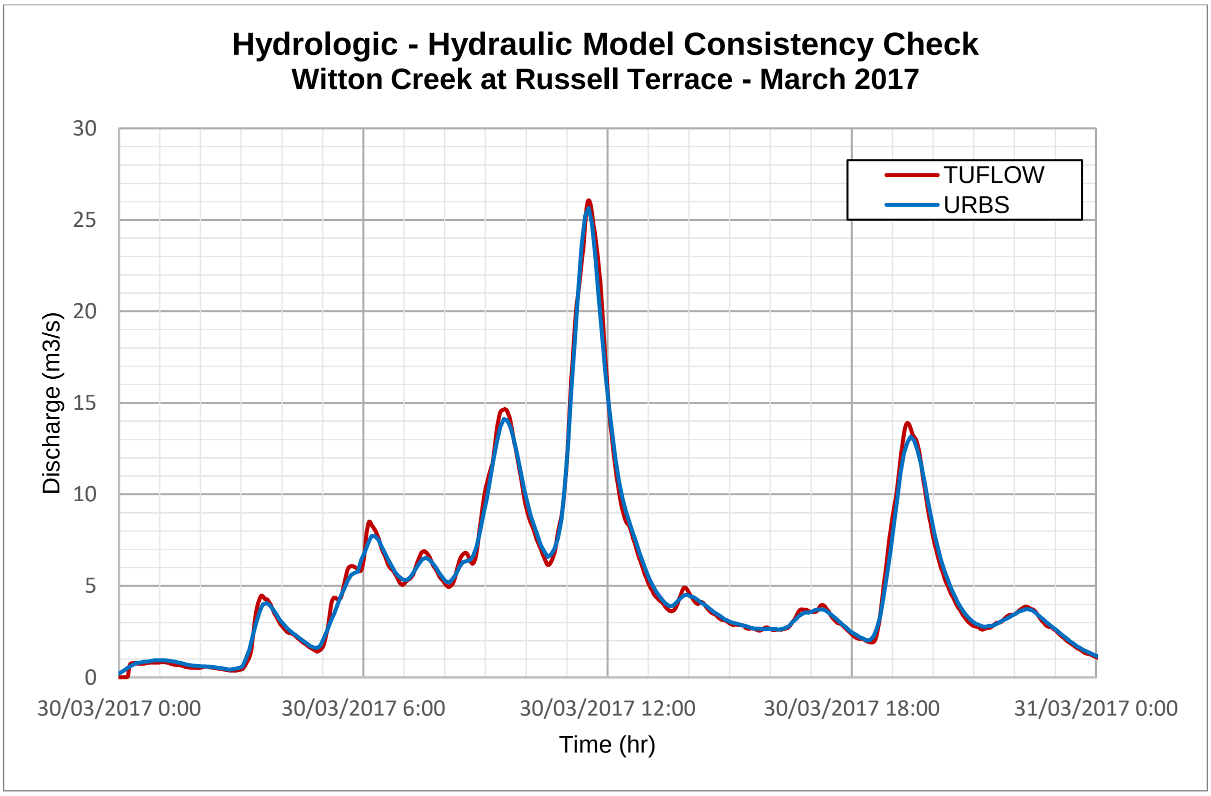


Figure 5.3: Witton Creek at Russell Terrace URBS vs TUFLOW comparision (March 2017)

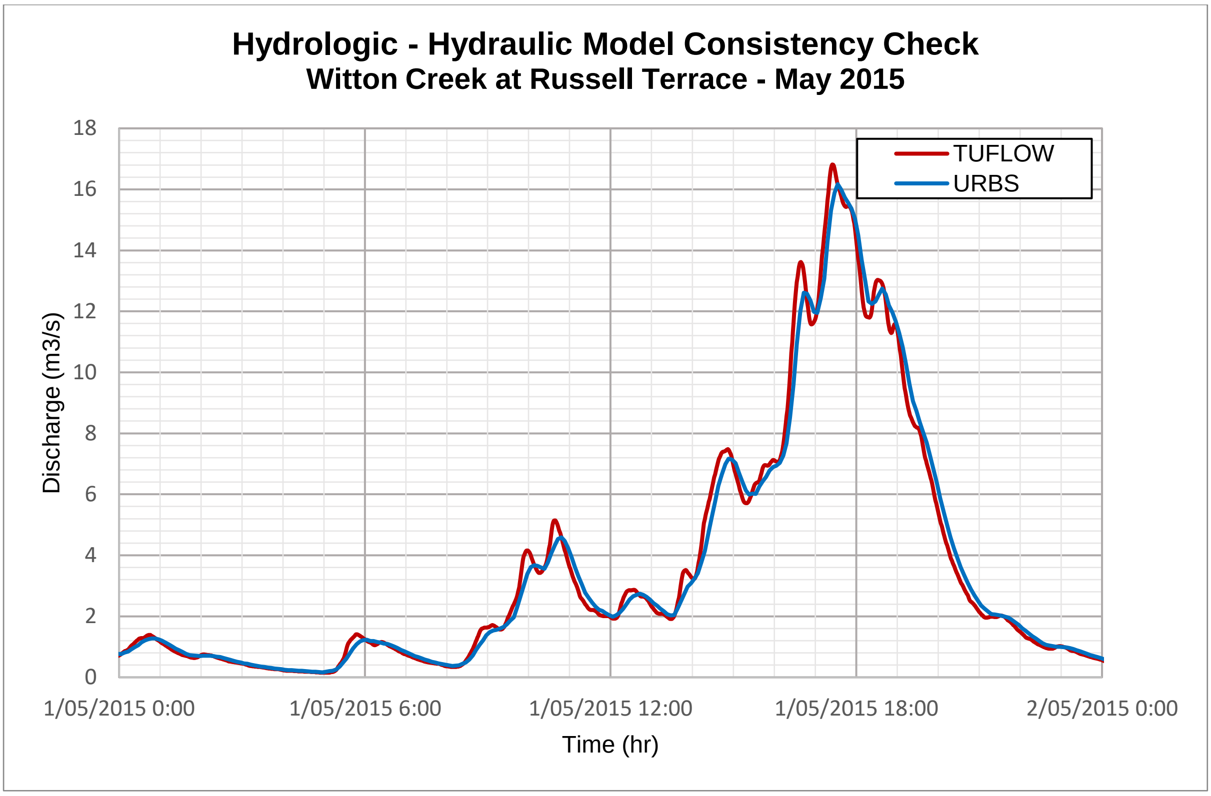


Figure 5.4: Witton Creek at Russell Terrace URBS vs TUFLOW comparision (May 2015)

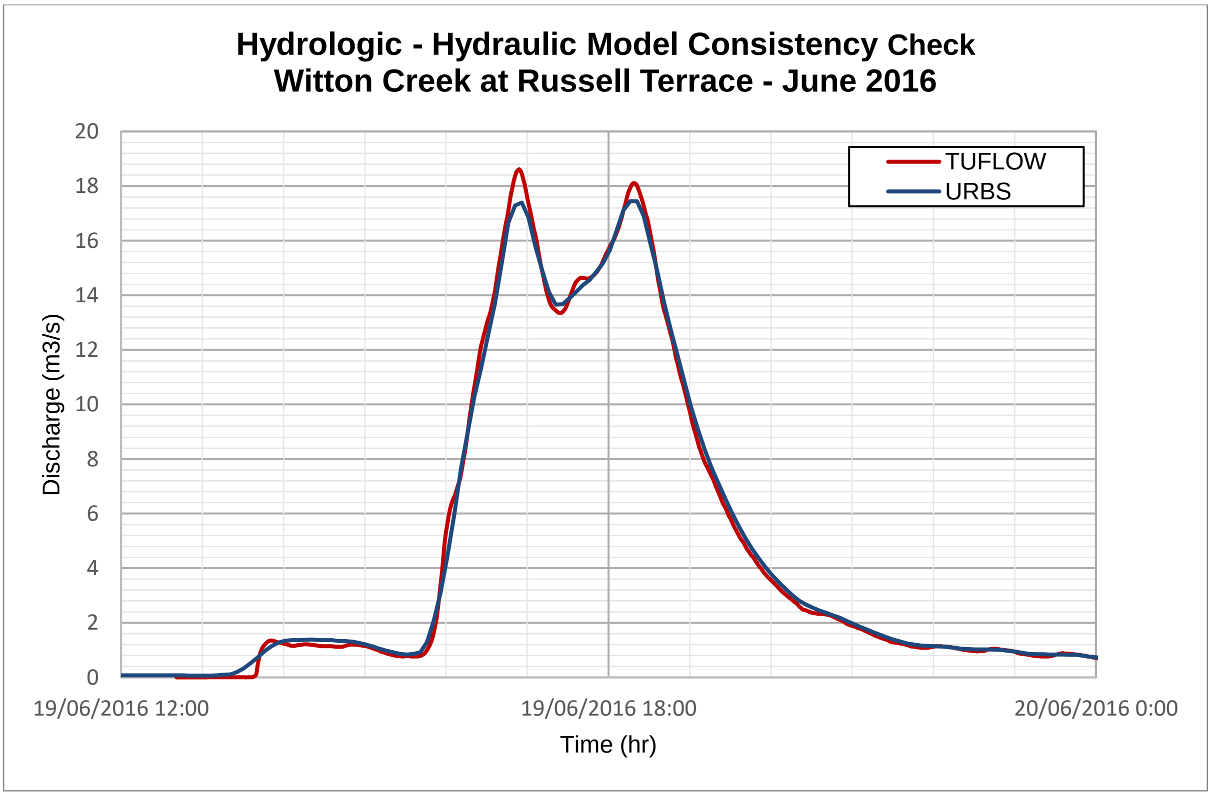


Figure 5.5: Witton Creek at Russell Terrace URBS vs TUFLOW comparision (June 2016)

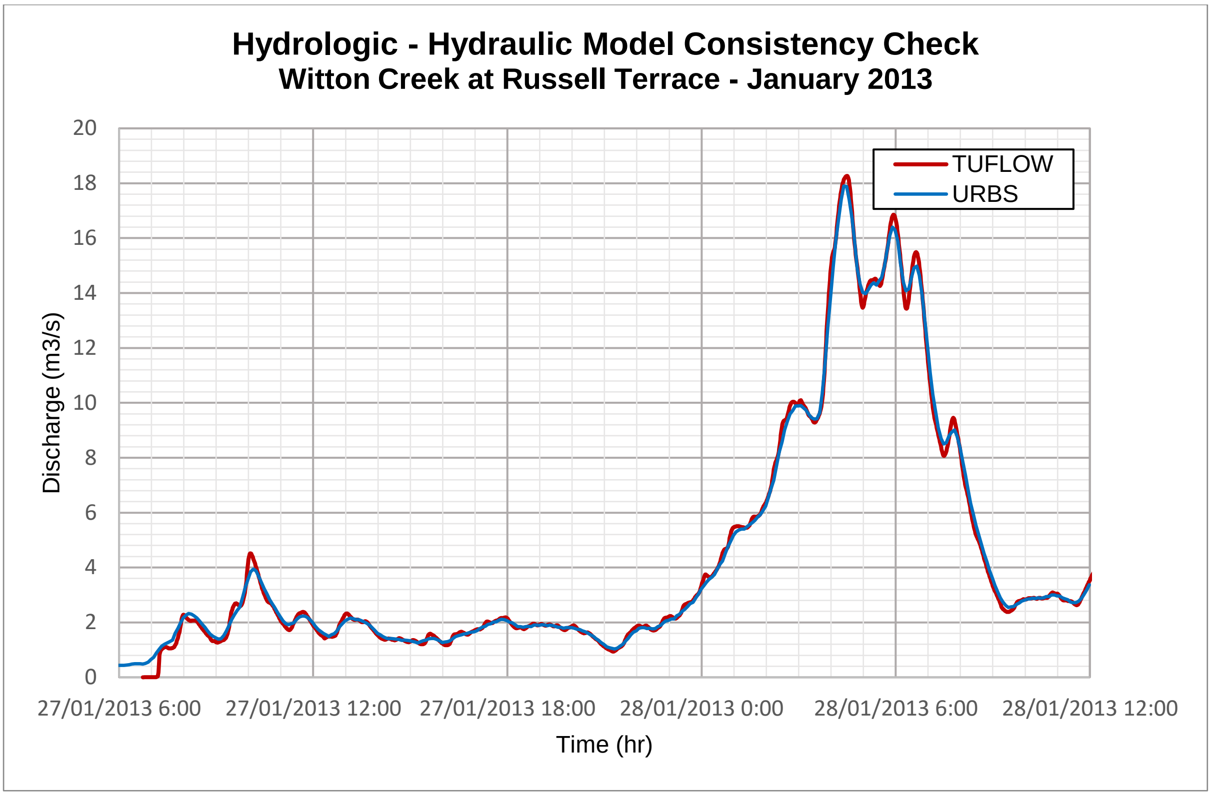


Figure 5.6: Witton Creek at Russell Terrace URBS vs TUFLOW comparision (January 2013)

5.8 Discussion on Calibration and Verification

The calibration and verification of the Witton Creek hydrologic and hydraulic models have 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.

Additionally, the MHG coverage is not extensive with only two gauges located within the main creek channel; W110 at Aaron Place (AMTD 424 m) and W120 at Kate Street Culvert (AMTD 778 m). There are no gauges on the tributaries of Witton Creek.

However, the calibration and verification of Witton Creek URBS and TUFLOW models have shown that:

- The TUFLOW model has simulated peak flood levels within the ideal tolerances for all the historical events modelled as part of the calibration and verification exercise.
- The URBS model can appropriately replicate the TUFLOW model hydrographs at a range of locations within the catchment for all historical events.

Given that the results of the calibration and verification are good, and the historical events used range from relatively frequent to infrequent (50% to 5% AEP events – refer to Section 6.4.3), there is confidence that the hydrologic and hydraulic models are suitable for producing flood levels for a full range of design event modelling.

6.0 Design Event Analysis

6.1 Design Event Scenarios

Table 6.1 indicates the scenarios utilised in the modelling of the design events, noting that all design event scenarios were modelled using ultimate catchment hydrological conditions.

For the purpose of this report, the term “design events” refers to the following events:

- Frequent: 50 % AEP and 20 % AEP, and
- Intermediate: 10 % AEP and 5 % AEP, and
- Rare: 2 % AEP and 1 % AEP

Table 6.1: Design Event Scenarios

Event	Scenario 1	Scenario 1 + CC	Scenario 3 + CC
50% AEP	✓	✓	✓
20% AEP	✓	✓	✓
10% AEP	✓	✓	✓
5% AEP	✓	✓	✓
2% AEP	✓	✓	✓
1% AEP	✓	✓	✓

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 3: Filling to the Modelled Flood Corridor + Minimum Riparian Corridor (MRC)
Scenario 3 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 (15 m wide each side from the low flow channel) was defined by changing the Manning’s ‘n’ roughness 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.”

The “Modelled Flood Corridor” is the greater extent of the Waterway Corridor (WC) and Flood Planning Areas (FPAs) 1, 2 and 3, including (where appropriate) adjacent parks and roadway areas. Figure 6.1 indicates the “Modelled Flood Corridor” for all creeks and tributaries within the catchment. Scenario 3 assumes filling to the “Modelled Flood Corridor” boundary to represent potential future development. In the design events, 50 % AEP to 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 under BCC City Plan.

6.2 Design Event Hydrology

This study utilises the AR&R 2019 approaches for design flood estimation, as detailed in the following sections.

6.2.1 Flood Frequency Analysis

As there are no stream gauges located within Witton Creek Catchment, no flood frequency assessment was undertaken within this study.

6.2.2 Adopted Methodology for the Design Event Approach (DEA) AR&R 2019

AR&R 2019 recommends the following for the estimation of flood events:

- The use of a simple average (or median value) to represent the flood magnitude at any locations within the catchment (Book 1, Table 1.3.2)
- The use of an ensemble (10) temporal patterns for each design storm be utilised to be representative of variability of actual historical events. (Book 2, Chapter 5)

Accordingly, the selection of design temporal pattern was undertaken using the TUFLOW model and is discussed further in Section 6.3.2.

Eight (8) storm durations (30 minutes, 45min, 1hour, 1.5 hours, 2 hours, 3 hours, 4.5 hours, and 6 hours) were used for the DEA AR&R 2019 hydrologic modelling. The hydrologic methodology used for this study is as follows:

- Updating the calibrated URBS model using data from the AR&R 2019 Data Hub at the catchment centroid.
- Populate the URBS model with the information from the AR&R 2019 Data Hub information and based on parameters adopted in model calibration. This is an automated process undertaken within URBS. Refer to Section 6.2.3 for details on URBS data hub parameter use.
- Run the ensemble of 10 temporal patterns through the URBS for the 8 storm durations outlined above (30 minutes to 6 hours) for 50% AEP to 1% AEP events to create inflow hydrographs for the TUFLOW model. This is a total of 80 simulations per AEP event.
- Use the URBS design hydrographs (sub-catchment routed rainfall excess and routed hydrographs) as input for the TUFLOW design event modelling.

6.2.3 URBS Model Set-up

The calibrated URBS model was used to simulate the design storm event rainfall-runoff and sub-catchment routing process. The following section describes the parameters used within the design event hydrologic modelling and the adjustments made to the calibration model to simulate the design events.

Catchment development

The design events were modelled using the 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 (BCC City Plan 2014). Accordingly, an increase in development typically results in an increase in impervious land use factors.

Appendix B presents the URBS catchment parameters that were adopted for the design event modelling scenarios. 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.

Design IFD Data

A suite of new localised IFD data has been commissioned for South-east Queensland local government areas of Lockyer Valley, Ipswich, Moreton Bay and Brisbane LGAs and termed as LIMB 2020 IFDs.

The LIMB 2020 IFDs were developed to reflect the localised nuances in rainfall distribution and severity not reflected by the AR&R 2016 IFD data, and to reduce the local biases across all AEPs, durations and areas. The design IFDs used within this study were LIMB 2020 IFD high resolution gridded data extracted at the centroid of the catchment. Table 6.2 indicates the adopted design IFDs. These values represent current climate and do not account for climate change.

Table 6.2: Adopted Design Event IFD Data (LIMB 2020)

Duration (Hrs)	Rainfall Intensity (mm/hr)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
0.5	68	91	105	117	132	142
0.75	54	72	83	93	104	112
1	44	60	70	73	88	95
1.5	33	46	53	60	68	74
2	27	38	44	50	57	62
3	20	28	33	38	44	48
4.5	15	21	25	29	34	37
6	12	17	21	24	28	31

Climate Change

An increase in rainfall intensity was included in the design event modelling. This increase in rainfall intensity was estimated using the climate projection models and guidance provided in AR&R 2019 Data Hub.

A 9.8% increase in rainfall intensity due to projected climate change variability effects was applied to the IFDs outlined in Table 6.2. This 9.8% increase in rainfall intensity is representative of RCP 4.5 to the Climate Future Year 2100. The increase in rainfall intensity was obtain through linear extrapolation based on the AR&R 2019 Climate Change values of the Year 2080 and Year 2090, as shown in Table 6.3.

Table 6.3: RCP 4.5 Climate Change Factors

Year	RCP 4.5 Climate Change Factor
2080	9.2%
2090	9.5%
2100*	9.8%

* Linearly extrapolated from 2080/2090 values

Burst Initial Loss

The Burst Initial Loss (IL_b) is the portion of the Storm Initial Loss (IL_s) that occurs within the burst, where the IL_s is assumed to be the depth of rainfall prior to the commencement of surface runoff:

The Burst Initial Loss (IL_b) = Storm Initial Loss (ILs) – pre-burst rainfall

- IL_b (impervious area) – a value of 0 was adopted for the impervious areas within the catchment, which is the URBS default value.
- IL_b (pervious area) – IL_b is the sum of IL_S minus the pre-burst rainfall. An IL_S value of 18 mm was adopted for pervious areas within the catchment. The IL_S value was adopted from AR&R 2019 Data Hub information, and from calibration and verification of the URBS model (refer to Section 4.8).

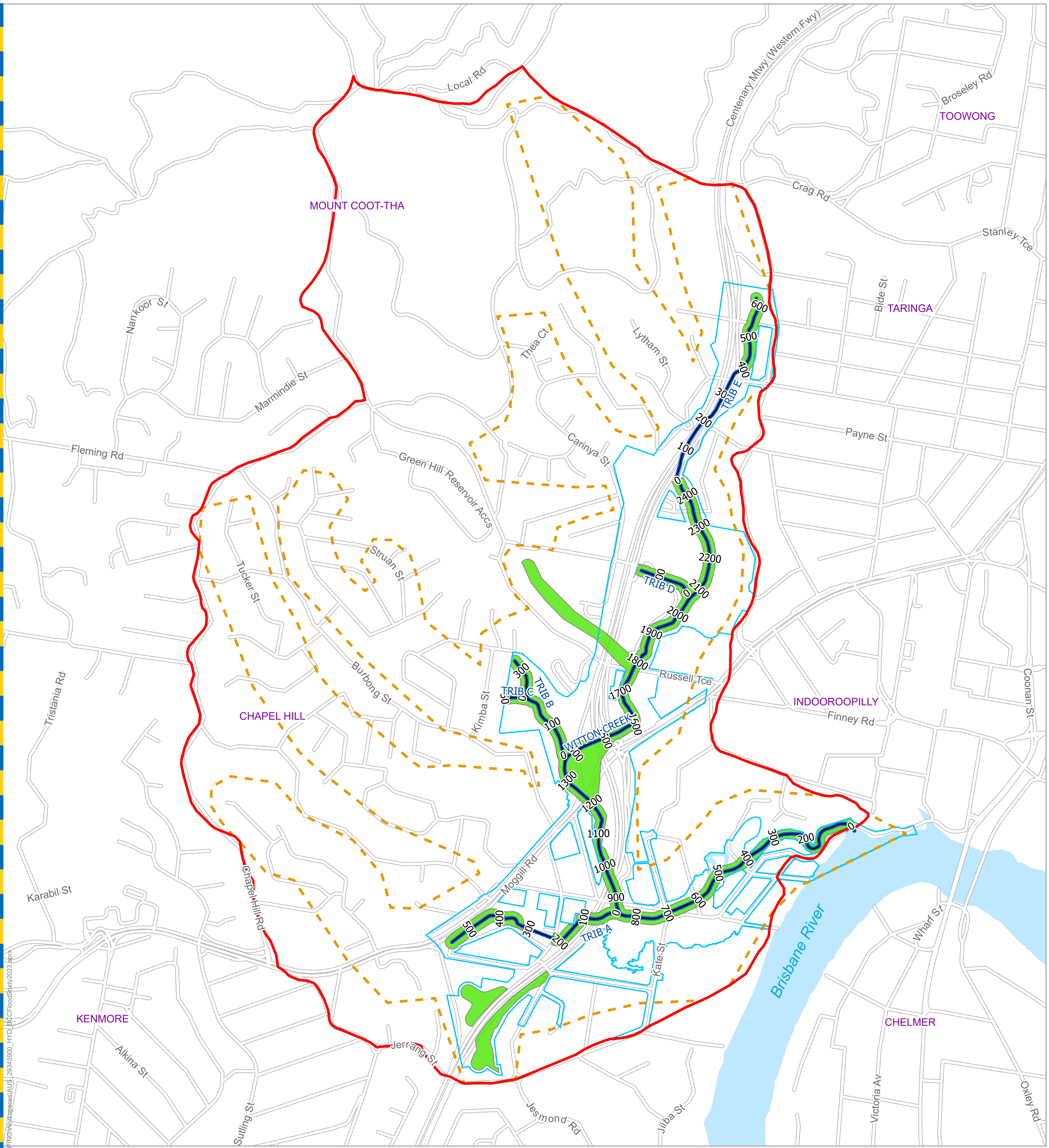
Continuing loss

The following values were adopted for the Continuing Loss (CL)

- CL (impervious area) – a value of 0 mm/hr was adopted for the impervious areas within the catchment, which is URBS default value,
- CL (pervious area) – a value of 1.4 mm/hr was adopted for the pervious areas within the catchment. This CL was provided by the AR&R 2019 Data Hub as being representative for the geographical regions in which Witton Creek Catchment is located. A CL of 1.4 mm/hr was also confirmed from the results of the calibration and verification process (refer to Section 4.8).

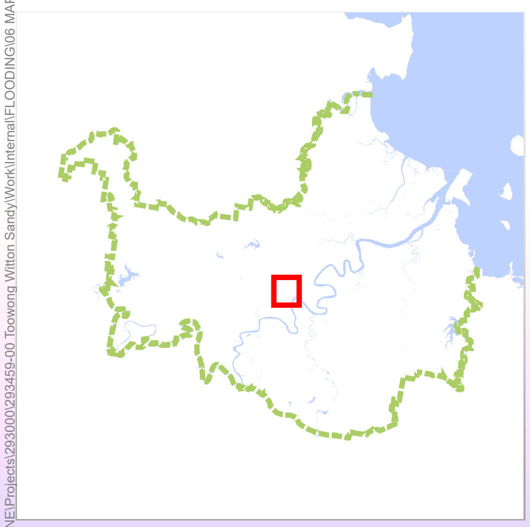
Aerial Reduction Factor

The determination of ARFs is a primary function of catchment area, storm event duration and to a lesser extent, AEP event. The application of ARFs to whole-of-catchment flood studies is not straightforward, due to innumerable potential points of interest throughout the catchment. Accordingly, an aerial reduction factor of 1 was adopted for the Witton Creek Catchment, as documented in the current version of the BCC Flood Procedure Document (Version 9, 2023). It is noted that this is considered a somewhat conservative approach due to the aforementioned technical difficulties associated with ARF application.



Legend

- Catchment Area
- Model Boundary
- Creek Centreline
- AMTD Line
- Flood Corridor
- Minimum Riparian Corridor
- Streets



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Prepared : O SMITH
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Publication Date : 06 Sep 2023
Project Number : 293459-00

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Witton Creek Figure 6.1 Modelled Flood Corridor Page 1 of 1

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6.3 Design Event Hydraulic Modelling

6.3.1 Overview

The TUFLOW model was used to determine design flows and flood levels for the scenarios outlined in Section 6.1, for the 50% AEP to the 1% AEP events. These AEP events were simulated for durations from 30 minutes to 6 hours, using the DEA AR&R 2019 approach outlined in Section 6.2.2.

6.3.2 Methodology

Each storm duration from 30 minutes to 6 hours was modelled with the 10 ensembles (E0 to E9), which resulted in a total of 80 simulations per AEP. The total number of TUFLOW simulations required to complete the design event modelling was 1440, comprising:

- Scenario 1 (with and without Climate Change) – 960 simulations
- Scenario 3 (inclusive of Climate Change) – 480 simulations

To select the median design temporal pattern (ensemble method), critical duration and design flood levels, the following approach was undertaken:

- For each AEP, the median flood level across the TUFLOW model extent was determined for each of the 8 storm durations (30 minutes to 6 hours). This was undertaken using TUFLOW post processing tools which produces a design flood level surface (GRID) of the median flood level for each duration, where:
    - A total of eight median flood level surfaces are produced, along with a separate grid of the median design temporal pattern (source) grid for each duration. The median temporal pattern source grid at any location within the model can be determined through GIS inspection of the median design temporal pattern source grid.
    - The TUFLOW post-processing tool 'median' function will then select the median as the 6th ranked result, noting there to be an even number of grids to choose from.
- A single design flood level is then produced for each AEP using TUFLOW post-processing tools, by extracting the peak flood level of the eight median flood level surfaces (one for each duration). A separate grid of the critical duration (source grid) is produced through the post-processing, from which the critical duration at any location within the model can be determined from GIS inspection.
- The design flow for each AEP at any location in the model can be determined from review of the TUFLOW time varying results with respect to the critical duration and the median ensemble.

6.3.3 TUFLOW Model Set-up

TUFLOW model extents

The model extent for Scenario 1 and Scenario 3 was the same as the TUFLOW model extent developed for the calibration and verification events (refer to Figure 5.1).

TUFLOW Roughness

The hydraulic roughness for the design event modelling assumed the ultimate catchment development conditions in accordance with BCC City Plan 2014. Based on BCC City Plan 2014 and historical aerial photography, Witton Creek is a highly developed and urban catchment, and has been for an extended period covering all calibration events modelled in this study. Accordingly, the hydraulic roughness in the TUFLOW design event model set-up remained consistent with that of the calibration model for Scenario 1.

The hydraulic roughness for the minimum riparian corridor was updated for Scenario 3, as outlined in Section 6.1.

TUFLOW Boundaries

Design Inflows

The design inflow (Q-T) boundaries in the TUFLOW model were taken from the URBS model for each AEP, duration and temporal pattern. The inflow locations remain the same as the TUFLOW model inflow locations developed for the calibration and verification events (refer to Figure 5.1).

Design Tailwater Conditions

The design event TUFLOW model adopted a fixed water level (H-T) boundary as the downstream model boundary as follows:

- Current Climate: Mean High Water Spring (MHWS) 1.04 m AHD.
- Future Climate Change RCP4.5 Year 2100: MHWS + Sea Level Rise of 0.8m = 1.84 m AHD.

The MHWS for Witton Creek was interpolated from MHWS reported at Port Office and Indooroopilly.

6.4 Results and Mapping

6.4.1 Design Discharge Results

A full range of 8 durations (30 minutes to 6 hours) were simulated for 50% AEP to 1% AEP events. Table 6.4 outlines the design flow results at major waterway crossings for Scenario 1 (with Climate Change), taken from the TUFLOW model.

Table 6.4: Design Discharge at Selected Major Waterway Crossing (Scenario 1 + CC)

Location	Design Discharge (m^3/s)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Witton Creek						
Radnor Street (s1)	31.7	42.3	49.3	44.1	61.9	67.3
Aaron Place (S2)	31.2	41.7	48.4	52.9	55.6	56.6
Kate Street (S3)	30.0	33.8	33.9	34.1	34.1	34.2
Witton Road (S5)	20.6	27.7	28.6	29.4	30.3	30.9
Western Freeway (S6)	20.6	29.0	30.6	31.7	33.2	34.6
Moggill Road (S7)	20.0	28.1	29.7	30.4	30.4	31.5
Western Freeway Onramp (S8)	14.6	18.9	24.5	27.7	28.0	34.8
Western Freeway (S9)	14.5	18.8	24.5	27.7	27.9	34.8
Russell Terrace (S10)	13.8	20.4	23.7	24.4	24.5	24.6
Witton Creek Tributary A						
Western Freeway Bikeway (S4)	13.2	15.5	16.3	16.3	16.4	17.4
Kennewell Park (S14)	5.6	6.2	6.2	6.2	6.2	6.3

6.4.2 Design Flood Levels

Tabulated design flood level results for the 50% AEP to 1% AEP events are provided along the modelled waterway in Witton Creek, located in the following appendices:

- Scenario 1 (including Climate Change): 50% AEP to 1% AEP event – Appendix E
- Scenario 3 (including Climate Change): 50% AEP to 1% AEP event – Appendix F

The design flood levels were extracted along the current AMTD line for all creeks and tributaries using the methodology outlined in Section 6.3.2. At some locations, the AMTD line did not intersect the flood surface, which results in a null value. The critical duration and median ensemble for each AMTD location is provided in Appendix I.

6.4.3 Return Periods of Historic Events

To estimate the return period of the historical events modelled, a simple flood frequency curve (based on flood level) was developed at the approximate location of the MHG locations within the catchment. This was based on the Scenario 1 with current climate design rainfall / flows. The data is shown across Figure 6.2 and Table 6.5 indicates the estimated magnitude of the calibration / verification events expressed as AEP.

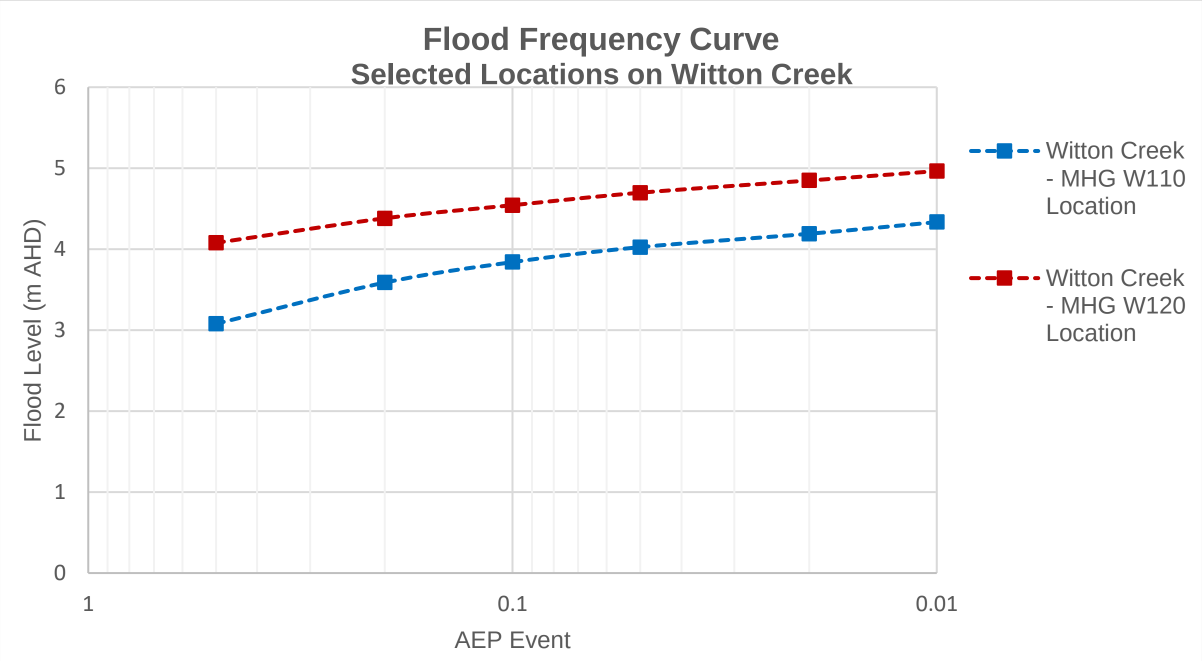

Figure 6.2: Flood Level Frequency Curve on Witton Creek

Table 6.5: Estimated Magnitude of Historical Events Based on Flood Level Comparison

Location	Event Magnitude (AEP)				
	Feb 2020	Mar 2017	May 2015	Jun 2016	Jan 2013
Witton Creek at MHG W110	< 50% AEP	20% to 10% AEP	Approx. 20% AEP	Approx 20% AEP	20% to 10% AEP
Witton Creek at MHG W120	< 50% AEP	10% to 5% AEP	Approx. 20% AEP	Approx 20% AEP	50% to 20% AEP

6.4.4 Rating Curves

As no stream gauges are located within the catchment, no rating curves were developed as part of this study.

6.4.5 Hydrologic-Hydraulic Model Consistency Check (Design Events)

Comparision checks on flows were undertaken between the URBS and the TUFLOW model for the 20% AEP, 5% AEP and 1% AEP with current climate for Scenario 1 at selected locations to understand how closely the hydrologic and hydraulic models were matching. The comparisons were undertaken for the 90 minute storm using Ensemble 1 (URBS TP0). The 90 minute storm duration was chosen as it is considered a mid-range storm event for comparative checks at multiple locations across the catchment, but is not necessarily the critical duration at each of these locations – hence the magnitudes of the peak discharge across events should not be taken as the actual peak discharge. The exercise is only for comparative purposes to check model consistency.

The locations where the comparative plots were undertaken are as follows:

i) Moore Park (Witton Creek AMTD 2272 m)

ii) Russell Terrace, downstream of culvert (S10) (Witton Creek AMTD 1785 m)

iii) Aragon Street (Witton Creek Tributary A AMTD 585m)

iv) Radnor Street, upstream of bridge (S1) (Witton Creek AMTD 6 m)

Figure 6.3 to Figure 6.6 provides the comparative plots at these four locations. Table 6.6 provides a comparison of the peak flows at these four locations.

Table 6.6: Peak Flow Comparision for URBS and TUFLOW for Scenario 1 (current climate)

Location	Model	Peak Flow (m^3/s)		
		50% AEP	5% AEP	1% AEP
Moore Park	URBS	8.2	21.5	22.2
	TUFLOW	8.6	21.3	21.3
Russell Terrace (S10)	URBS	9.4	25.5	26.7
	TUFLOW	10.5	25.8	26.7
Aragon Street (Witton Creek Tributary A)	URBS	2.8	7.7	7.6
	TUFLOW	3.1	7.5	7.4
Radnor Street (S1)	URBS	22.8	60.3	66.1
	TUFLOW	23.7	52.5	59.5

In the upper sections of the catchment and at Tributary A, there is good comparison between the TUFLOW and URBS hydrographs. However, further downstream at the outlet of the catchment (see Figure 6.6) the difference between the TUFLOW and URBS hydrographs tends to increase, as the floodplain storage within the mid and lower catchment takes effect. The floodplain storage is better represented in the TUFLOW model (as storages were not explicitly modelled within the URBS model, and the URBS model struggles with non-linear routing). Accordingly, the URBS peak flow typically exceeds the TUFLOW peak flow in these location

Overall, there is deemed to be a sufficient comparison between the URBS and TUFLOW models. It should also be noted that hydrologic models generally cannot perfectly replicate the complex hydraulics of an entire catchment system.

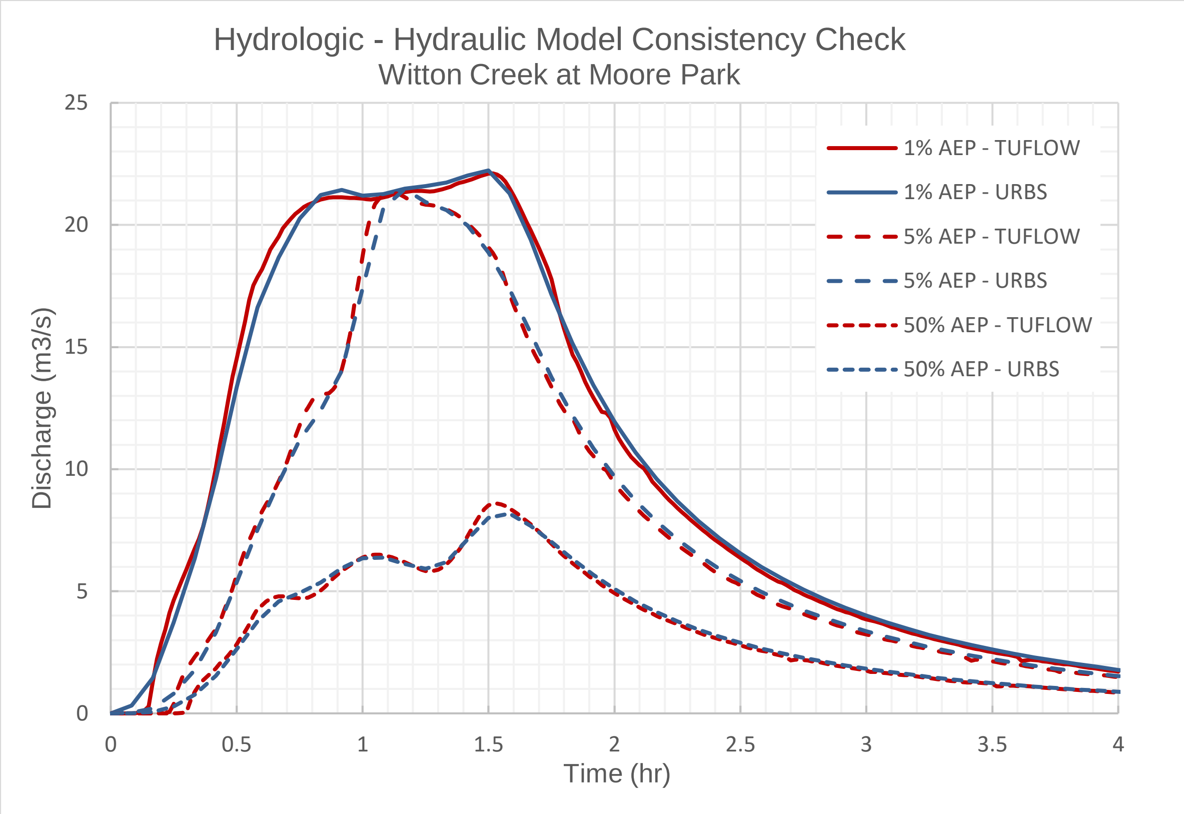

Figure 6.3: Witton Creek at Moore Park URBS vs TUFLOW Comparison.

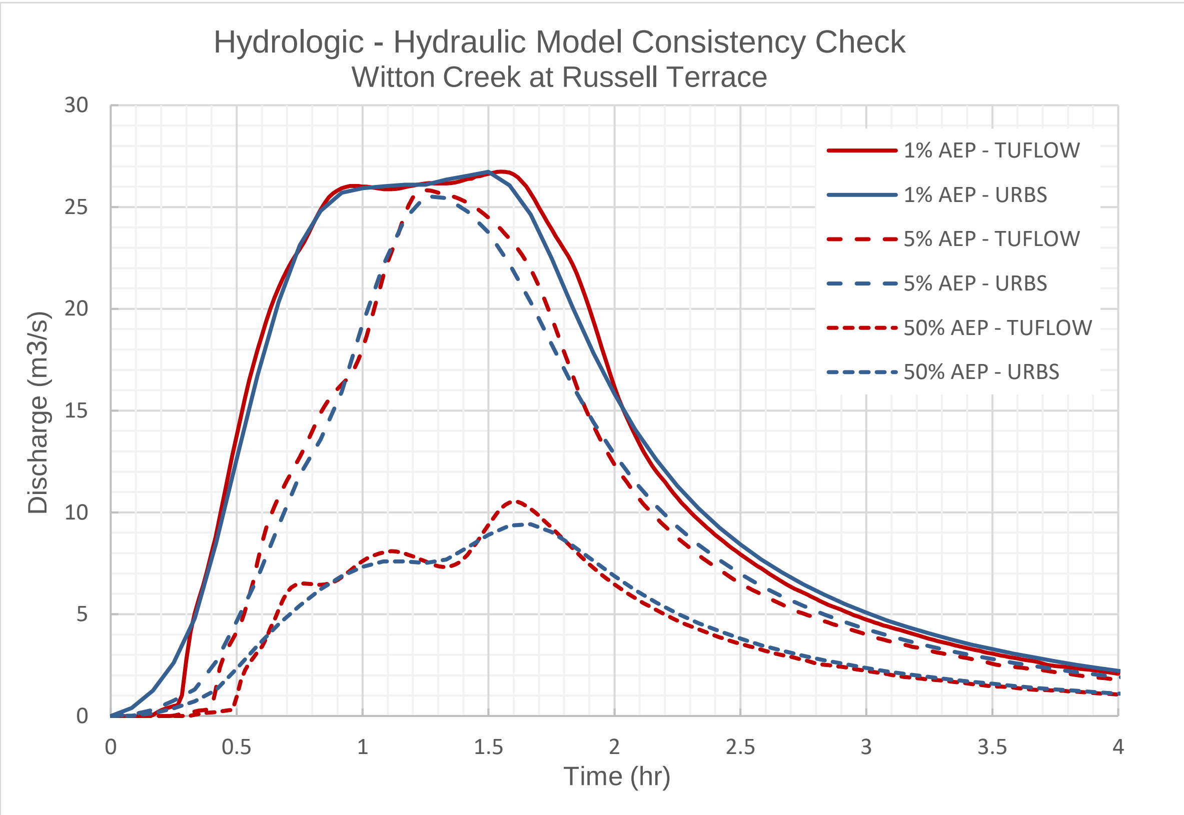

Figure 6.4: Witton Creek Russell Terrace UBRS vs TUFLOW Comparison

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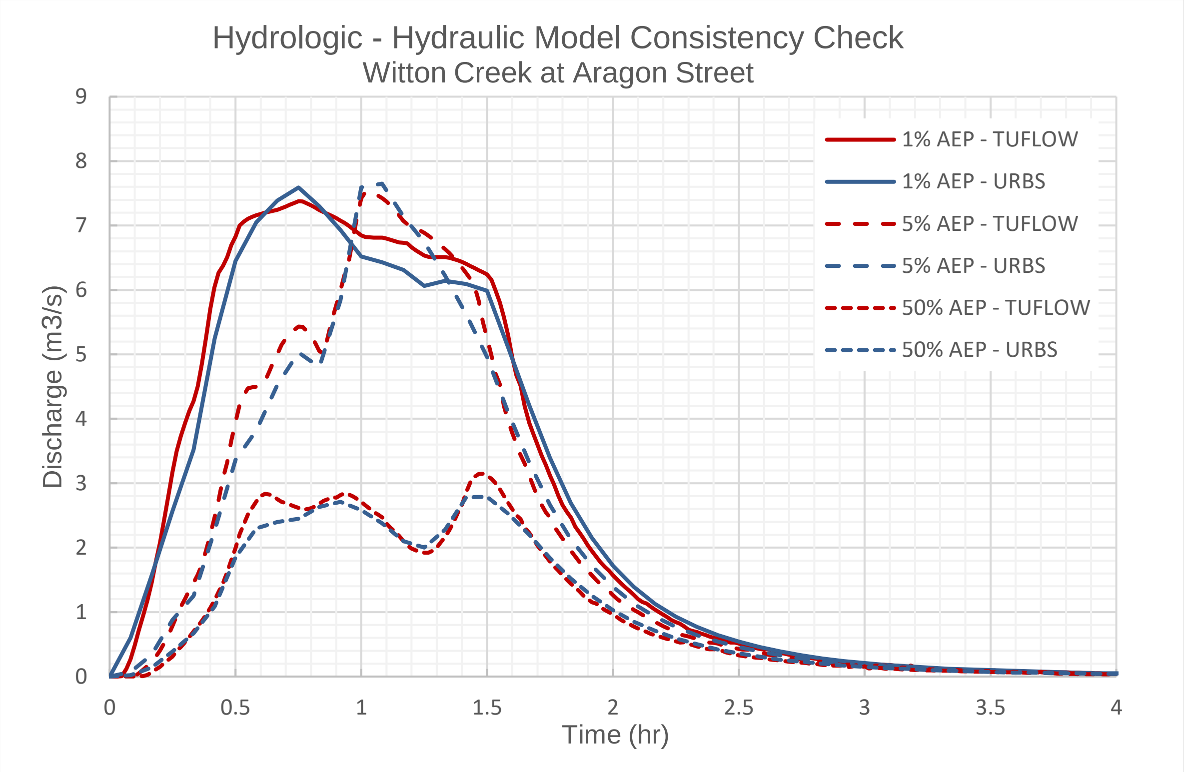

Figure 6.5: Witton Creek at Aragon Street URBS vs TUFLOW Comparision

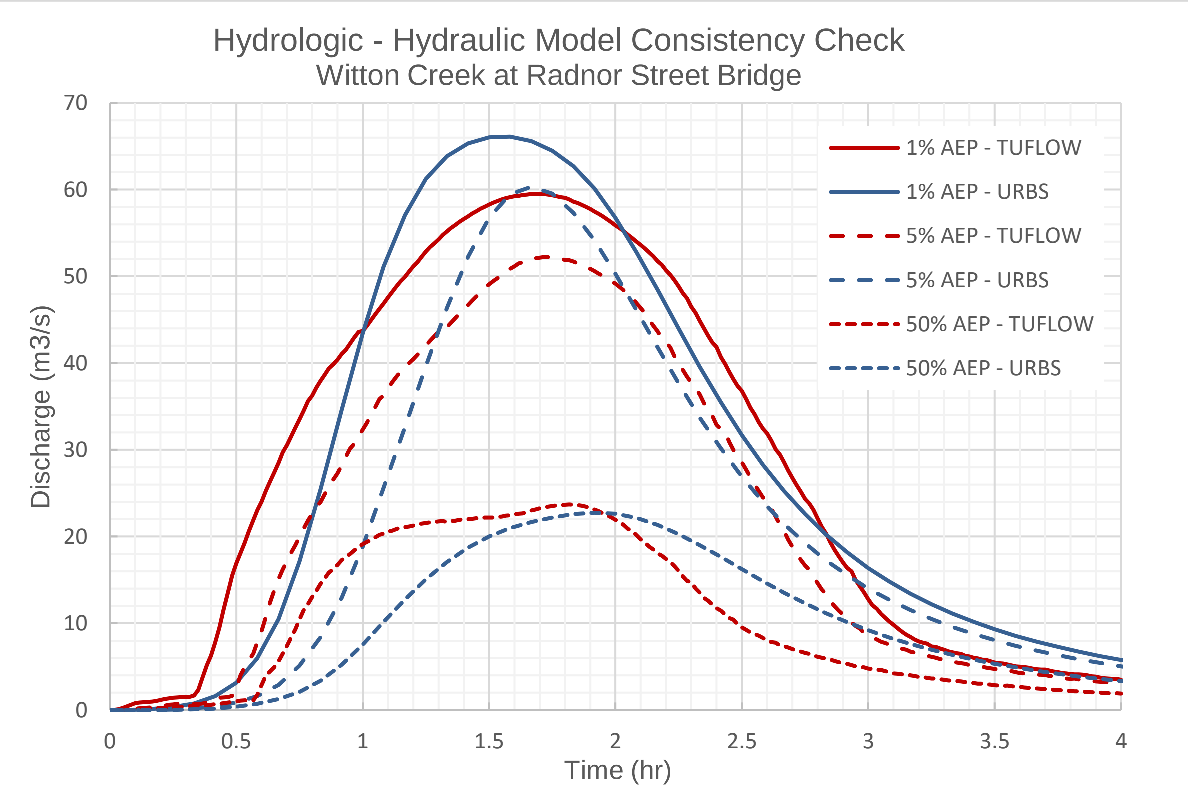

Figure 6.6: Witton Creek at Radnor Street URBS vs TUFLOW Comparison

6.4.6 Hydraulic Structure Reference Sheets

Details of the flood level and flow data derived for the hydraulic structure crossings modelled are summarised in the Hydraulic Structure Reference Sheets. These sheets are located in Appendix K, where the flood levels and flow values are representative of present day conditions and as such do not include increases in rainfall intensity and sea-level rise due to projected climate variability effects.

6.4.7 Flood Mapping

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

- Scenario 1 Flood Extent Mapping 50% AEP to 1% AEP (including Climate Change)

7.0 Very Rare and Extreme Event Analysis

7.1 Very Rare and Extreme Event Scenarios

Table 7.1 indicates the events and scenarios modelled as part of the Very Rare and Extreme event analysis. These scenarios have been previously described in Section 6.1. All Very Rare and Extreme event modelling was undertaken using the ultimate hydrological conditions (for detail refer to Section 6.2.3).

Table 7.1: Very Rare and Extreme Event Scenarios

Event	Scenario 1	Scenario 1 + CC	Scenario 3 + CC
0.5 % AEP	✓	✓	✓
0.2% AEP	✓	✓	✓
0.05 % AEP	✓	✓	×
PMF	✓	×	×

For modelling of the Scenario 3 events, the fill height outside of the "Modelled Flood Corridor" was set to the Scenario 3 – 1% AEP flood level plus an additional freeboard allowance of 0.3 m.

The 1% AEP plus 0.3 m flood surface is stretched to represent a developed floodplain consistent with City Plan requirements. The development of the stretched floodplain surface for this study was undertaken by BCC and provided for very rare event simulations.

7.2 Extreme Event Terminology

For the purpose of the Extreme Event analysis, the term Probable Maximum Flood (PMF) has been used to define the flood event which is produced through the modelling of the 6 hour 'superstorm' Probable Maximum Precipitation (PMP) hyetograph prepared by BCC based on the BoM Generalised Short Duration Method (GSDM).

7.3 Flood Extent Stretching Process

The flood extent stretching process first involves the generation of a new flood surface, exactly 0.3m above the Scenario 3 – 1% AEP + CC modelled flood surface. Following this, a lateral extension of the new flood surface is undertaken, 'stretching' the surface until it intersects the existing terrain.

7.4 Very Rare Event Hydrology

The DEA AR&R 2019 was used for the 0.5% AEP, 0.2% AEP and 0.05% AEP events, with the same approach outlined in Section 6.2.

Design IFD data

The LIMB 2020 IFDs (detailed in 6.2.3) were used for the 0.5% AEP, 0.2% AEP and 0.05% AEP events. The design IFDs used within this study were LIMB 2020 IFD high resolution gridded data extracted at the centroid of the catchment. Table 7.2 indicates the adopted design IFDs. These values represent current climate and do not consider climate change.

Table 7.2: Adopted Very Rare AEP Design Events IFD Data (LIMB 2020)

Duration (Hrs)	Rainfall Intensity (mm/hr)		
	0.5% AEP	0.2% AEP	0.05% AEP
0.5	160	188	234
0.75	127	149	186
1	108	126	157
1.5	83.8	98.1	123
2	69.7	81.8	102
3	54.0	63.2	78.7
4.5	41.8	48.7	60.4
6	34.9	40.7	50.4

7.5 Extreme Event Hydrology

7.5.1 General

The Probable Maximum Flood (PMF) has been used to define the flood event which is produced through the modelling of the PMP hyetograph prepared by BCC based on the BoM Generalised Short Duration Method (GSDM).

7.5.2 Design Hydrograph

Table 7.3 indicates the adopted superstorm pattern and hyetograph for the PMF. The total rainfall depth is 816 mm.

Table 7.3: Adopted Superstorm Hyetogragh for PMF

Time (hr)	Rainfall (mm)	Cumulative Rainfall (%)
0.00	0	0
0.17	9.9	1
0.33	9.9	2
0.50	9.9	4
0.67	9.9	5
0.83	9.9	6
1.00	9.9	7
1.17	13.5	9
1.33	13.5	11
1.50	13.5	12
1.67	18.4	14
1.83	18.4	17
2.00	18.4	19
2.17	27.6	22
2.33	27.6	26
2.50	27.6	29
2.67	38.3	34
2.83	38.3	39
3.00	75.2	48
3.17	75.1	57
3.33	75.1	66
3.50	38.1	71
3.67	27.6	74
3.83	27.6	78
4.00	27.6	81
4.17	18.4	83
4.33	18.4	86
4.50	18.4	88
4.67	13.5	89
4.83	13.5	91
5.00	13.5	93
5.17	9.9	94
5.33	9.9	95
5.50	9.9	96
5.67	9.9	98
5.83	9.9	99
6.00	9.9	100
Total	816	

7.5.3 Rainfall Losses for PMF

The rainfall losses used for the design event hydrology were adopted as per the following for the URBS modelling of the PMF:

- Storm initial loss: 0mm
- Storm continuing loss: 1.4mm/hr

Note that the continuing loss adopted will have negligible impact on the PMF superstorm.

7.6 Very Rare and Extreme Event Hydraulic Modelling

7.6.1 General

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

7.6.2 Methodology

The methodology used for the very rare and extreme events was the same approach described in Section 6.3.2 and Section 6.3.3. The total number of TUFLOW simulations required to complete the very rare event modelling (excluding the single PMF simulation) was 640 and comprised of the following:

- Scenario 1 (with and without Climate Change) – 480 simulations
- Scenario 3 (inclusive of Climate Change) – 160 simulations

7.6.3 TUFLOW Model Set-up

TUFLOW model extents

With regards to very rare and extreme event hydraulic modelling extent, no changes were made to the model extent of the design event (50% to 1%) TUFLOW models for both Scenario 1 and Scenario 3 (refer to Figure 5.1).

TUFLOW model roughness

With regards to very rare and extreme event hydraulic modelling roughness, no changes were made to the roughness of the design event (50% to 1%) TUFLOW models for both Scenario 1 and Scenario 3 (refer to Section 6.3.3).

TUFLOW Boundaries

Design Inflows

The design (Q-T) boundaries to the TUFLOW model were taken from the URBS model for each AEP, duration and temporal pattern. The inflow locations did not change from the design event TUFLOW models for Scenario 1 and Scenario 3 (refer to Figure 5.1).

Design Tailwater Conditions

The design event TUFLOW model adopted a fixed water level (H-T) boundary as the downstream model boundary as follows:

- Current Climate: Highest Astronomical Tide HAT (HAT) 1.64 m AHD.
- Future Climate Change RCP4.5 Year 2100: HAT + Sea Level Rise of 0.8m = 2.44 m AHD.

The HAT for Witton Creek was extrapolated from the HAT level reported at Indooroopilly, noting the mouth of Witton Creek to be located slightly upstream.

7.6.4 Hydraulic Structures

The very rare and extreme event TUFLOW model utilised the same hydraulic structures as the design event TUFLOW models for Scenario 1 and Scenario 3.

7.7 Results and Mapping

7.7.1 Design Discharge Results

A full range of eight durations (30 minutes to 6 hours) were simulated for 0.5% AEP, 0.2% AEP and the 0.05% AEP Event. The PMF 6-hour superstorm for current climate was simulated. Table 7.4 outlines the design flows at major waterway crossings from Scenario 1 conditions (including Climate Change), taken from the TUFLOW model.

While there appears to be some anomalies with PMF structure discharge being less than that of the 1 in 2000 AEP event, note that this appears to be a function of (i) the flood gradient on the PMF for these longer events, and its larger flood volume, can reduce peak discharge through the actual structures (even though flood levels are higher in the PMF than the 1 in 2000 AEP for example, and (ii) larger components of the PMF total flow are overtopping or outflanking the structure.

Table 7.4: Design Discharge at Selected Major Waterway Crossing (Scenario 1 + CC)

Location	Design Discharge (m^3/s)			
	0.5 % AEP	0.2% AEP	0.05% AEP	PMF (current climate)
Witton Creek				
Radnor Street (S1)	75.6	86.6	104.1	218.7
Aaron Place (S2)	57.1	58.2	60.6	69.6
Kate Street (S3)	34.5	35.1	35.7	31.9
Witton Road (S5)	30.7	31.7	32.7	29.5
Western Freeway (S6)	37.1	42.0	49.6	66.1
Moggill Road (S7)	31.8	31.8	31.9	31.8
Western Freeway Onramp (S8)	39.5	44.7	51.3	77.8
Western Freeway (S9)	39.1	45.1	52.7	79.8
Russell Terrace (S10)	24.7	24.8	24.9	24.5
Witton Creek Tributary A				
Western Freeway Bikeway (S4)	18.4	18.8	19.2	35.6
Kennewell Park (S14)	6.3	6.6	6.8	8.8

7.7.2 Design Flood Levels

Tabulated design flood level results for the very rare events are provided for all the modelled waterways within the Witton Creek Catchment and are located in the following appendices:

- Scenario 1 (including Climate Change): 0.5% AEP, 0.2% AEP and 0.05% AEP events – Appendix G
- Scenario 3 (including Climate Change): 0.5% AEP and 0.2% AEP events – Appendix H

The critical storm duration and median ensemble for each tabulated location for Scenario 1 is provided in Appendix J.

7.7.3 Flood Mapping

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

- Scenario 1 Flood Extent Mapping of 0.5% AEP, 0.2% AEP and 0.05% AEP (including Climate Change)

8.0 Summary of Study Findings

This flood study report details the calibration and verification, design and very rare / extreme events modelling for the Witton Creek Catchment. 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, stream and maximum height gauge records. Calibration of the URBS and TUFLOW models was undertaken for the February 2020, March 2017 and May 2015 historic events. Verification of the URBS and TUFLOW models was undertaken for the June 2016 and January 2013 historic 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.

The results of the hydraulic calibration and verification indicated that the URBS and TUFLOW models were able to adequately replicate the historical flooding events to within the specified tolerances for all events, at all locations. 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.

Flood magnitudes were estimated for the full range of events from 50% AEP to PMF. These analyses estimate the design flows based on the ultimate catchment development conditions in accordance with BCC City Plan 2014 and utilised AR&R 2019 methodologies. The design rainfall intensities included an allowance for increased rainfall intensity due to projected climate variability effects, with an increase in rainfall intensity of 9.8% for RCP 4.5 to year 2100. A fixed tidal boundary was used at the downstream model extent with an allowance of 0.8 m for projected climate variability effects.

Two waterway scenarios were considered as follows:

- Scenario 1 is based on the current waterway conditions.
- Scenario 3 includes an allowance for the riparian corridor and also assumes filling to the "Modelled Flood Corridor" boundary to simulate potential development in accordance with City Plan 2014.

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

- Peak flood discharges at selected locations
- Peak flood levels at 100 m intervals along the AMTD line
- Peak flood extent mapping (only for Scenario 1 including Climate Change)
- Hydraulic structure reference sheets for all major crossings located within the extent of mapping.

APPENDICES

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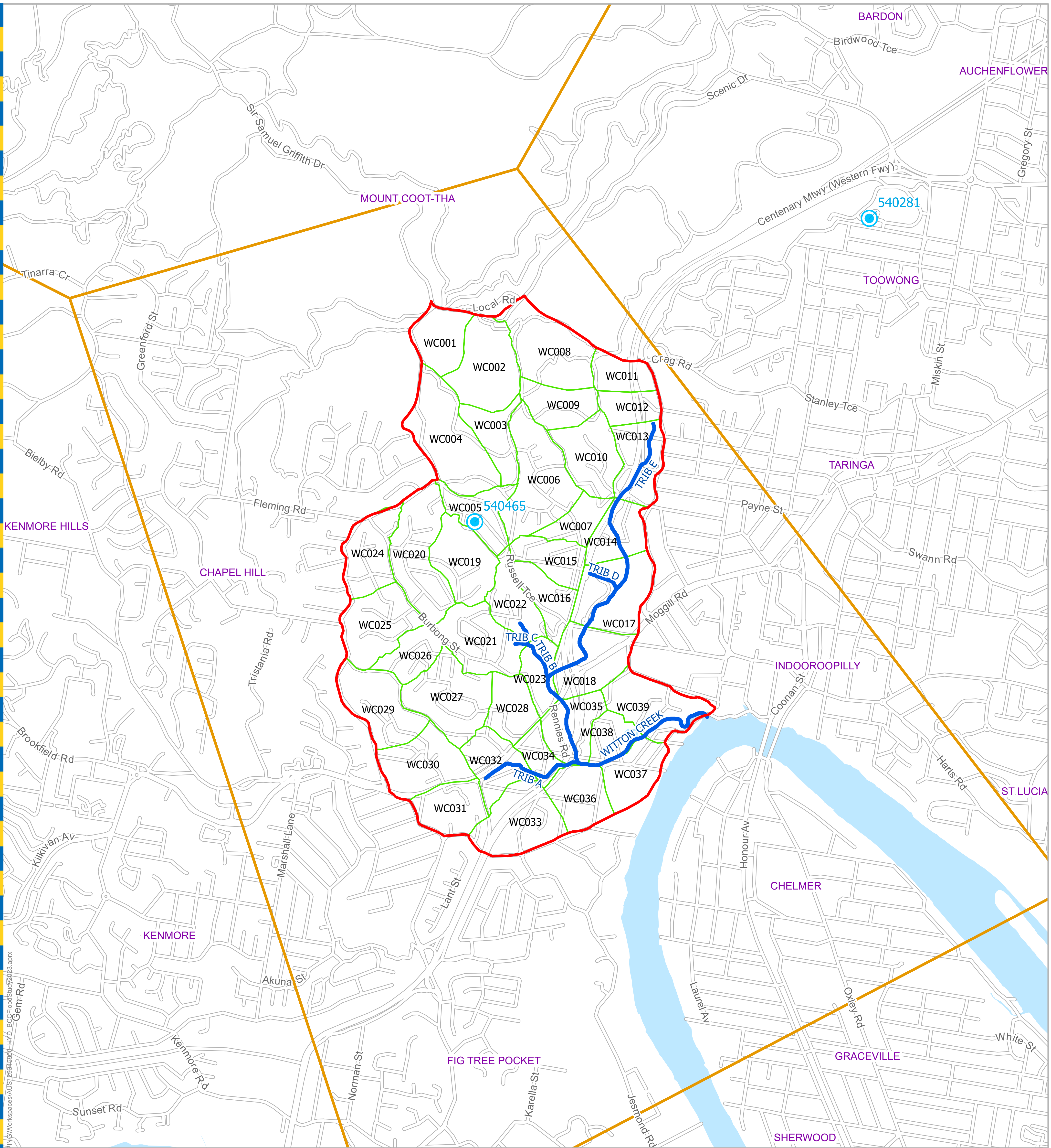
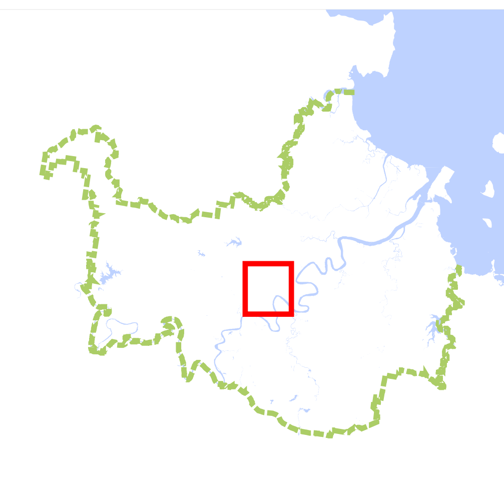
Appendix A: Rainfall Distribution





Legend

- Catchment Area
- Pluviograph Station
- Creek Centreline
- Rainfall Distribution
- URBS Subcatchments (1-39)
- Streets

0 125 250 375 500

Metres

N W E S

Prepared : O SMITH
Checked : T WEBB
Revision : 1
Publication Date : 29 Jun 2023
Project Number : 293459-00

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DATA INFORMATION

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Witton Creek
Figure A.1 Thiessen Polygons
Page 1 of 1

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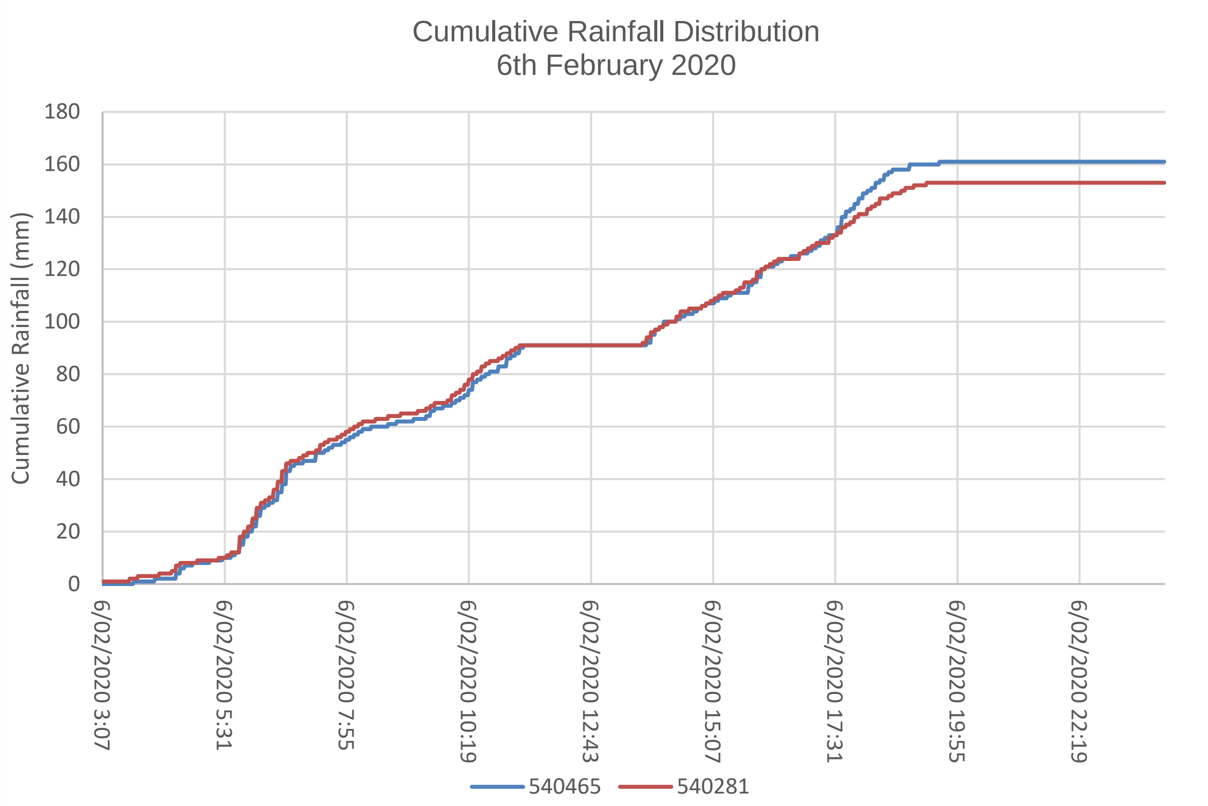
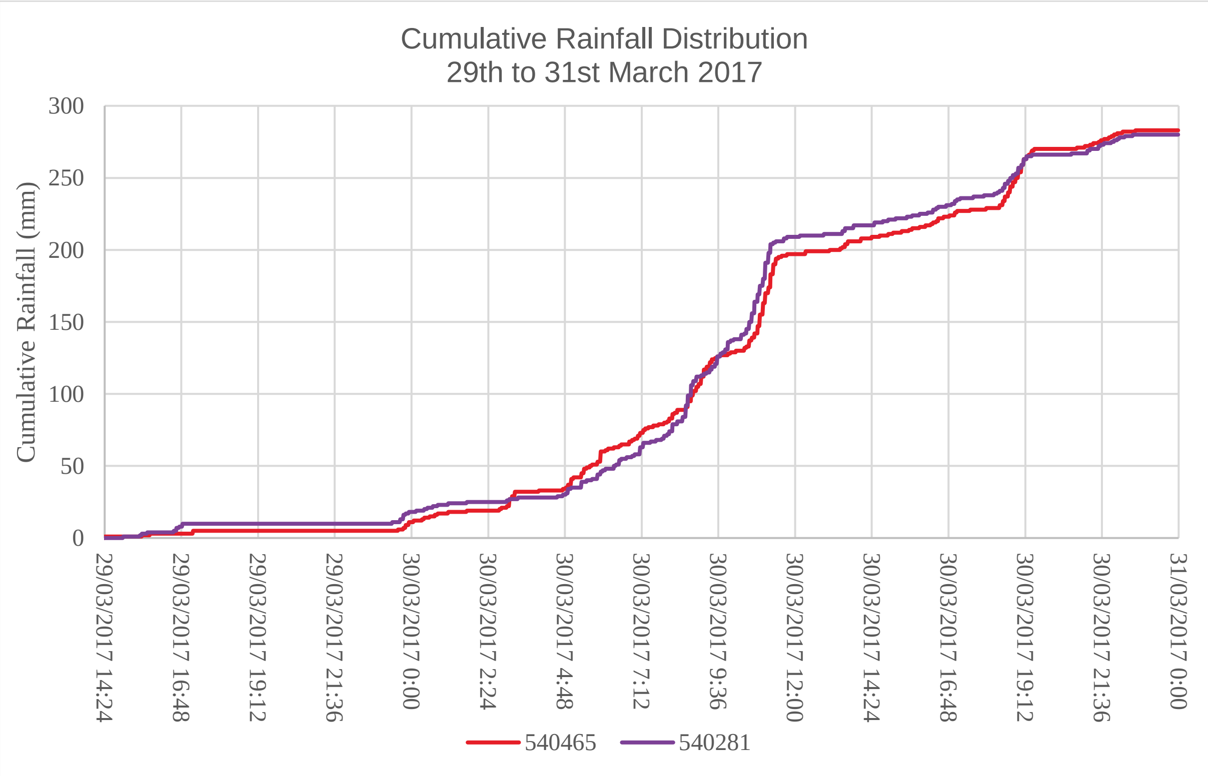

Cumulative Rainfall Distribution
6th February 2020
Cumulative Rainfall (mm)
180
160
140
120
100
80
60
40
20
0
6/02/2020 3:07
6/02/2020 5:31
6/02/2020 7:55
6/02/2020 10:19
6/02/2020 12:43
6/02/2020 15:07
6/02/2020 17:31
6/02/2020 19:55
6/02/2020 22:19
540465
540281


Cumulative Rainfall Distribution
29th to 31st March 2017
Cumulative Rainfall (mm)
300
250
200
150
100
50
0
29/03/2017 14:24
29/03/2017 16:48
29/03/2017 19:12
29/03/2017 21:36
30/03/2017 0:00
30/03/2017 2:24
30/03/2017 4:48
30/03/2017 7:12
30/03/2017 9:36
30/03/2017 12:00
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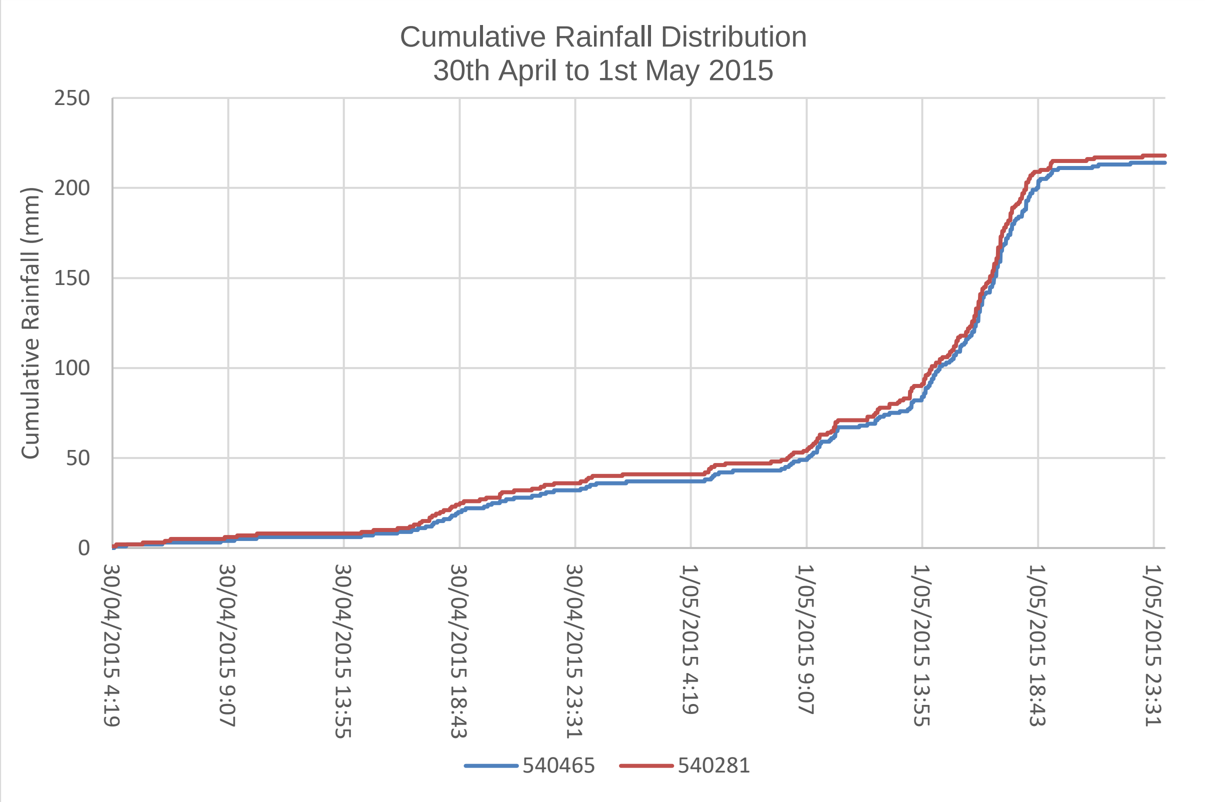
31/03/2017 0:00
540465
540281


Cumulative Rainfall Distribution
30th April to 1st May 2015
Cumulative Rainfall (mm)
250
200
150
100
50
0
30/04/2015 4:19
30/04/2015 9:07
30/04/2015 13:55
30/04/2015 18:43
30/04/2015 23:31
1/05/2015 4:19
1/05/2015 9:07
1/05/2015 13:55
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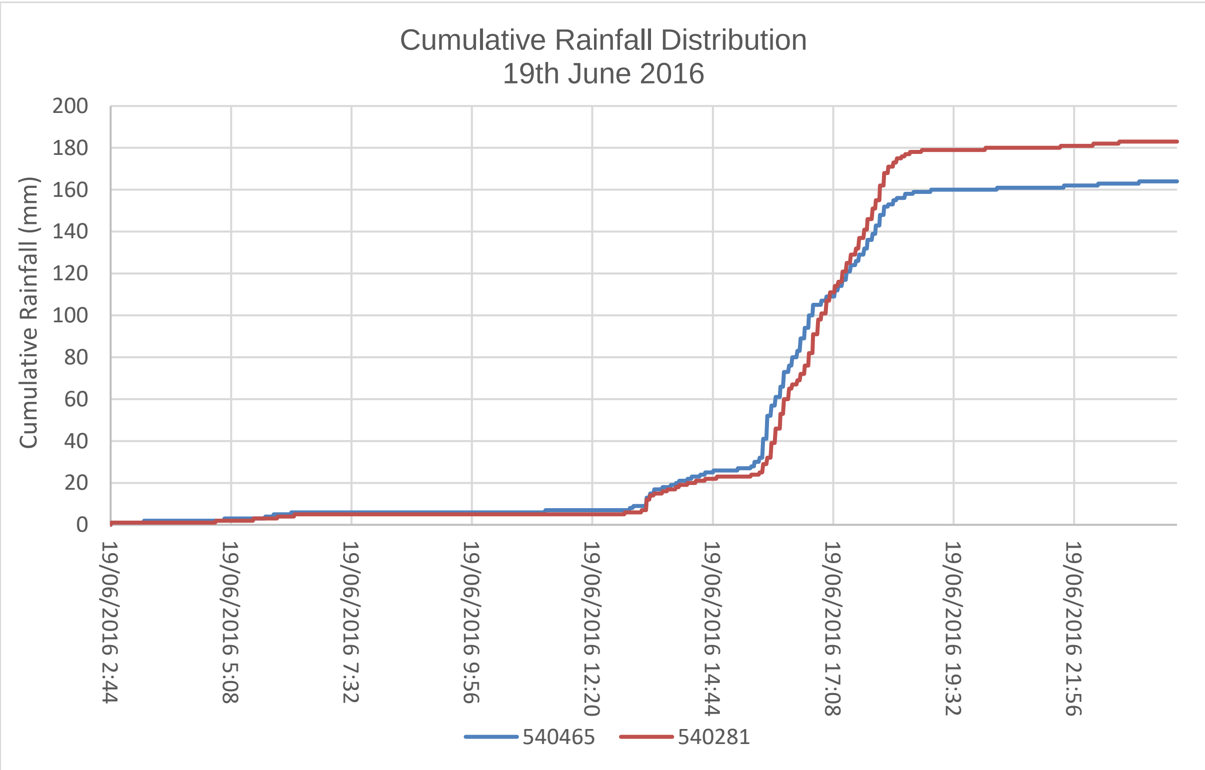
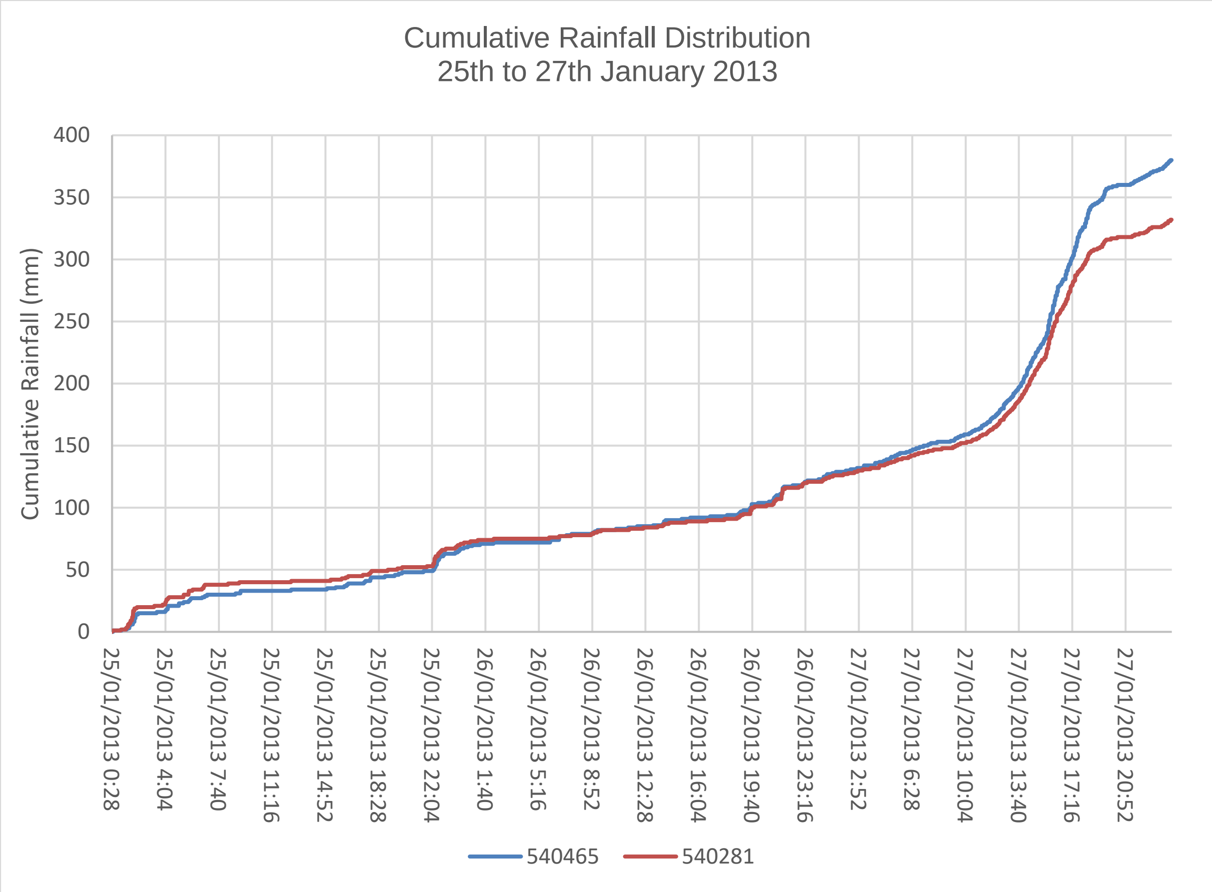
1/05/2015 23:31
540465
540281


Cumulative Rainfall Distribution
19th June 2016
Cumulative Rainfall (mm)
200
180
160
140
120
100
80
60
40
20
0
19/06/2016 2:44
19/06/2016 5:08
19/06/2016 7:32
19/06/2016 9:56
19/06/2016 12:20
19/06/2016 14:44
19/06/2016 17:08
19/06/2016 19:32
19/06/2016 21:56
540465
540281



Cumulative Rainfall Distribution
25th to 27th January 2013
Cumulative Rainfall (mm)
400
350
300
250
200
150
100
50
0
25/01/2013 0:28
25/01/2013 4:04
25/01/2013 7:40
25/01/2013 11:16
25/01/2013 14:52
25/01/2013 18:28
25/01/2013 22:04
26/01/2013 1:40
26/01/2013 5:16
26/01/2013 8:52
26/01/2013 12:28
26/01/2013 16:04
26/01/2013 19:40
26/01/2013 23:16
27/01/2013 2:52
27/01/2013 6:28
27/01/2013 10:04
27/01/2013 13:40
27/01/2013 17:16
27/01/2013 20:52
540465
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Appendix B: URBS Model Parameters

URBS Calibration (Historical Event Modelling Only) – Sub-catchment Parameters						
S/C	**Area (km2)**	**UL**	**UM**	**UH**	**UR**	**I**
1	0.087	0.000	0.000	0.044	0.956	0.040
2	0.147	0.000	0.000	0.022	0.978	0.020
3	0.076	0.000	0.000	0.000	1.000	0.000
4	0.187	0.000	0.000	0.090	0.910	0.081
5	0.082	0.358	0.000	0.437	0.205	0.447
6	0.180	0.000	0.000	0.764	0.236	0.688
7	0.059	0.190	0.000	0.618	0.193	0.584
8	0.160	0.000	0.000	0.025	0.975	0.022
9	0.081	0.000	0.000	0.000	1.000	0.000
10	0.114	0.000	0.000	0.764	0.236	0.688
11	0.060	0.000	0.000	0.150	0.850	0.135
12	0.055	0.000	0.000	0.150	0.850	0.135
13	0.102	0.000	0.313	0.337	0.350	0.460
14	0.082	0.000	0.491	0.271	0.238	0.489
15	0.076	0.000	0.004	0.935	0.061	0.843
16	0.075	0.000	0.002	0.918	0.081	0.827
17	0.147	0.213	0.000	0.097	0.690	0.120
18	0.129	0.215	0.070	0.440	0.276	0.463
19	0.117	0.089	0.000	0.488	0.423	0.453
20	0.164	0.000	0.779	0.221	0.000	0.588
21	0.130	0.000	0.731	0.212	0.057	0.557
22	0.082	0.053	0.664	0.142	0.141	0.468
23	0.062	0.000	0.296	0.139	0.565	0.273
24	0.108	0.000	0.565	0.436	0.000	0.674
25	0.132	0.001	0.000	1.000	0.000	0.900
26	0.067	0.000	0.000	1.000	0.000	0.900
27	0.155	0.289	0.513	0.189	0.009	0.470
28	0.078	0.695	0.086	0.219	0.000	0.345
29	0.132	0.000	0.778	0.210	0.013	0.578
30	0.128	0.684	0.114	0.202	0.000	0.341
31	0.092	0.773	0.000	0.227	0.000	0.320
32	0.093	0.000	0.261	0.739	0.000	0.796
33	0.141	0.185	0.424	0.363	0.028	0.566
34	0.044	0.096	0.136	0.659	0.109	0.675
35	0.095	0.060	0.000	0.736	0.205	0.671

URBS Calibration (Historical Event Modelling Only) – Sub-catchment Parameters						
S/C	**Area (km2)**	**UL**	**UM**	**UH**	**UR**	**I**
36	0.094	0.451	0.000	0.516	0.033	0.532
37	0.090	0.606	0.000	0.394	0.001	0.445
38	0.057	0.416	0.000	0.567	0.017	0.573
39	0.131	0.288	0.253	0.389	0.071	0.519

URBS Design Event and Very Rare and Extreme Events Sub-catchment Parameters						
S/C	**Area (km2)**	**UL**	**UM**	**UH**	**UR**	**I**
1	0.087	0.000	0.000	0.044	0.956	0.040
2	0.147	0.000	0.000	0.022	0.978	0.020
3	0.076	0.000	0.000	0.000	1.000	0.000
4	0.187	0.000	0.000	0.090	0.910	0.081
5	0.082	0.358	0.000	0.437	0.205	0.447
6	0.180	0.000	0.000	0.764	0.236	0.688
7	0.059	0.190	0.000	0.618	0.193	0.584
8	0.160	0.000	0.000	0.025	0.975	0.022
9	0.081	0.000	0.000	0.000	1.000	0.000
10	0.114	0.000	0.000	0.764	0.236	0.688
11	0.060	0.000	0.000	0.150	0.850	0.135
12	0.055	0.000	0.000	0.150	0.850	0.135
13	0.102	0.000	0.311	0.340	0.350	0.461
14	0.082	0.000	0.491	0.271	0.238	0.489
15	0.076	0.000	0.004	0.935	0.061	0.843
16	0.075	0.000	0.002	0.918	0.081	0.827
17	0.147	0.213	0.000	0.097	0.690	0.120
18	0.129	0.000	0.215	0.510	0.276	0.566
19	0.117	0.000	0.089	0.488	0.423	0.484
20	0.164	0.000	0.779	0.221	0.000	0.588
21	0.130	0.000	0.731	0.212	0.057	0.557
22	0.082	0.053	0.664	0.142	0.141	0.468
23	0.062	0.000	0.296	0.139	0.565	0.273
24	0.108	0.000	0.565	0.436	0.000	0.674
25	0.132	0.000	0.000	1.000	0.000	0.900
26	0.067	0.000	0.000	1.000	0.000	0.900
27	0.155	0.256	0.546	0.189	0.009	0.482
28	0.078	0.695	0.086	0.219	0.000	0.345
29	0.132	0.000	0.778	0.210	0.013	0.578
30	0.128	0.346	0.374	0.280	0.000	0.491
31	0.092	0.000	0.773	0.227	0.000	0.591
32	0.093	0.000	0.261	0.739	0.000	0.796
33	0.141	0.185	0.424	0.363	0.028	0.566
34	0.044	0.096	0.136	0.659	0.109	0.675
35	0.095	0.023	0.037	0.736	0.205	0.684

URBS Design Event and Very Rare and Extreme Events Sub-catchment Parameters						
S/C	**Area (km2)**	**UL**	**UM**	**UH**	**UR**	**I**
36	0.094	0.000	0.451	0.516	0.033	0.690
37	0.090	0.023	0.583	0.394	0.000	0.649
38	0.057	0.078	0.338	0.567	0.017	0.691
39	0.131	0.046	0.491	0.392	0.071	0.605

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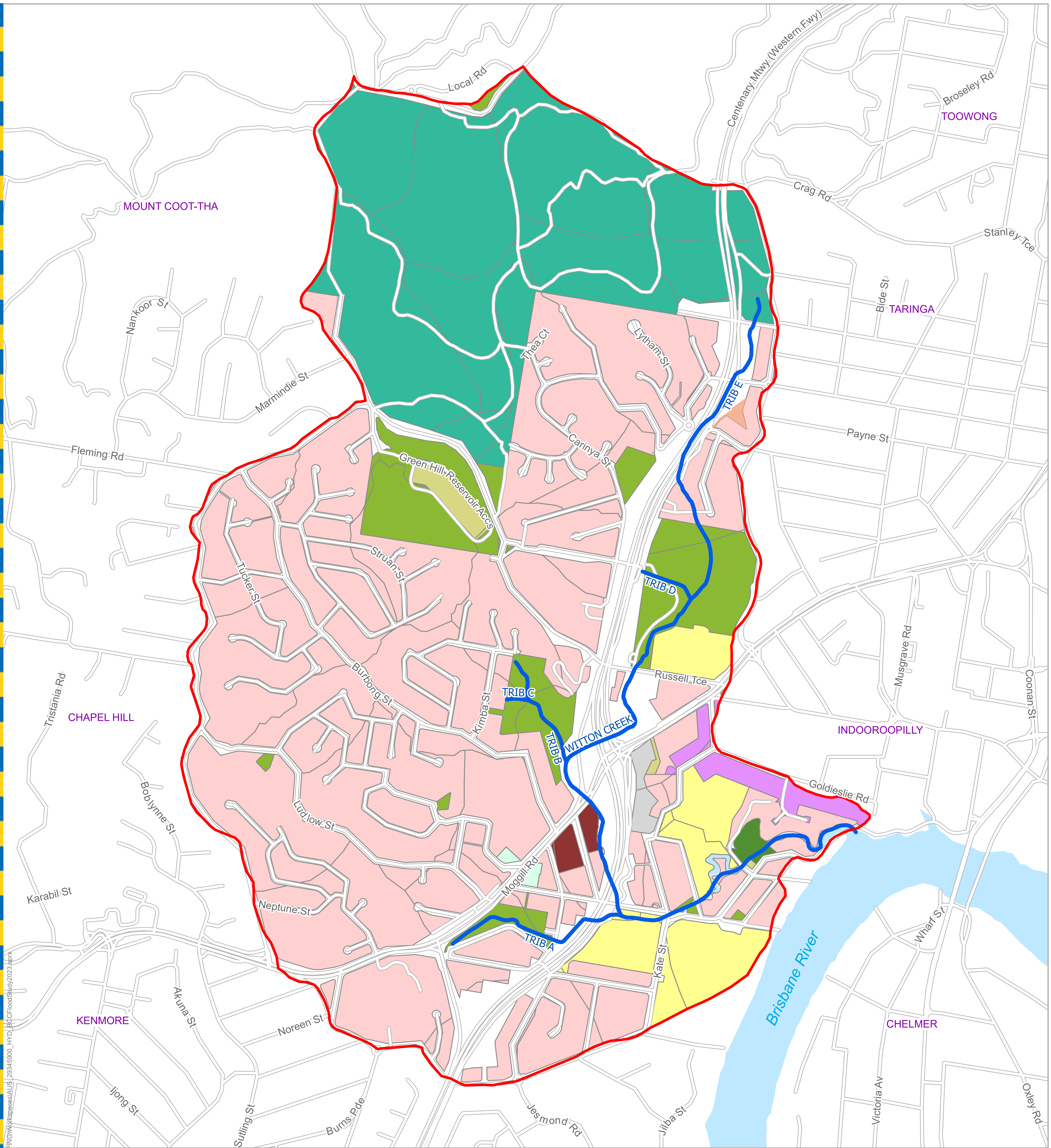
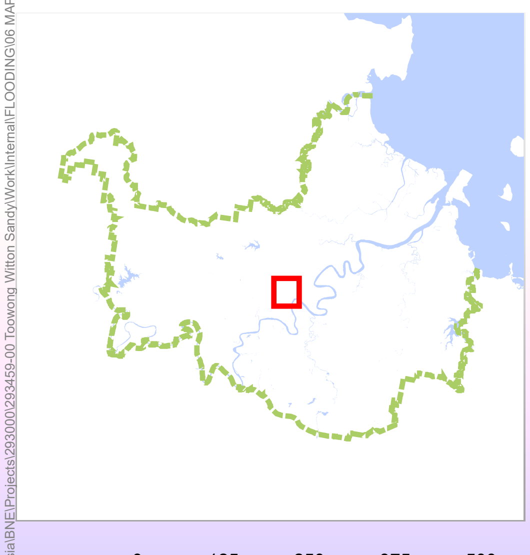
Appendix C: Adopted Land Use





0 125 250 375 500

Metres

N E S W

Legend

- Catchment Area
- Creek Centreline
- Streets

Zoning

- CF5 - Education purpose
- CN3 - Conservation (Metropolitan)
- CR1 - Character
- EC - Emerging community
- EM - Environmental management
- LDR - Low density residential
- MU3 - Corridor
- NC - Neighbourhood centre
- OS1 - Open space (Local)
- OS3 - Open space (Metropolitan)
- SC4 - Specialised centre (Large format retail)
- SP4 - Special purpose (Utility services)

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City Projects Office
GPO Box 1434
Brisbane Qld 4001

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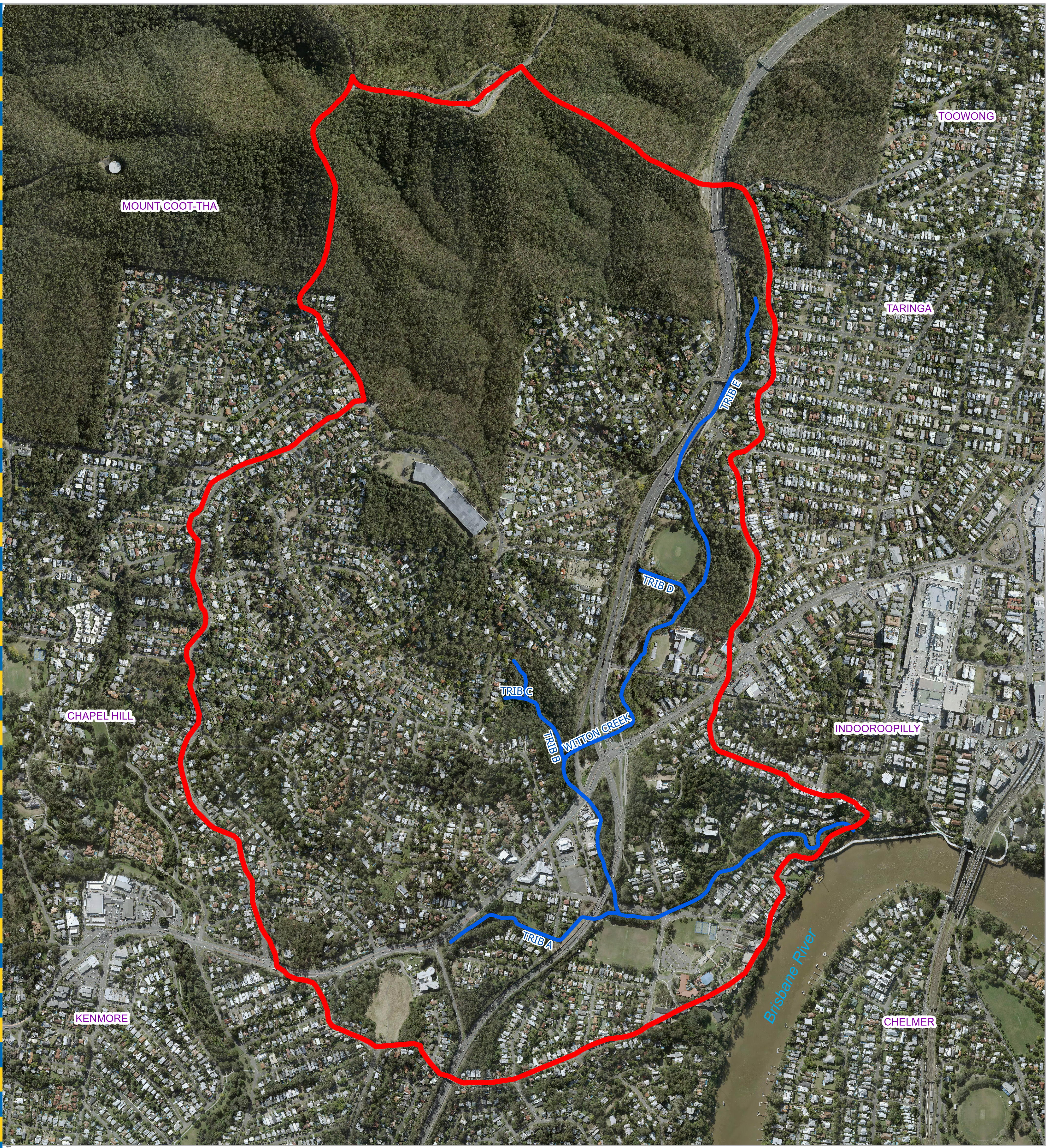
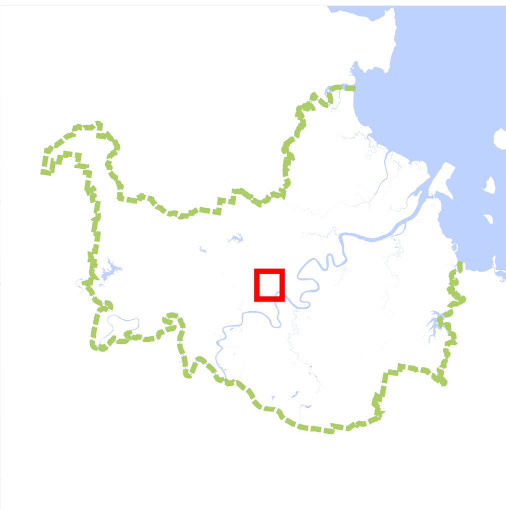
Prepared : O SMITH
Checked : T WEBB
Revision : 1
Publication Date : 29 Jun 2023
Project Number : 293459-00

Witton Creek
Figure C.1 City Plan Zones
Page 1 of 1

ARP - 29345900 - 008





Legend

- Catchment Area
- Creek Centreline

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0 125 250 375 500
Metres

N W E S

Prepared : O SMITH
Checked : T WEBB
Revision : 1
Publication Date : 29 Jun 2023
Project Number : 293459-00

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Witton Creek
Figure C.2 2021 Aerial Photo
Page 1 of 1

ARP - 29345900 - 009

File : \\global.arup.com\australasia\BNE\Projects\293000\293459-00 Toowong Witton Sandy\Work\Internal\FLOODING\06 MAPPI

Land Use	Impervious Fraction (%)
Low density residential	60
Character residential (Character)	70
Low-medium density residential (2 storey mix)	70
Low-medium density residential (2 or 3 storey	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
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
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

Land Use	Impervious Fraction (%)
Environmental management	5
Conservation	0
Conservation (Local)	0
Conservation (District)	0
Conservation (Metropolitan)	0
Emerging community	70
Mixed use (Inner city)	90
Mixed use (Centre frame)	90
Mixed use (Corridor)	90
Rural	5
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 (Large format retail)	90
Specialised centre (Mixed industry and business)	90
Special purpose (Transport infrastructure)	75
Special purpose (Utility services)	75

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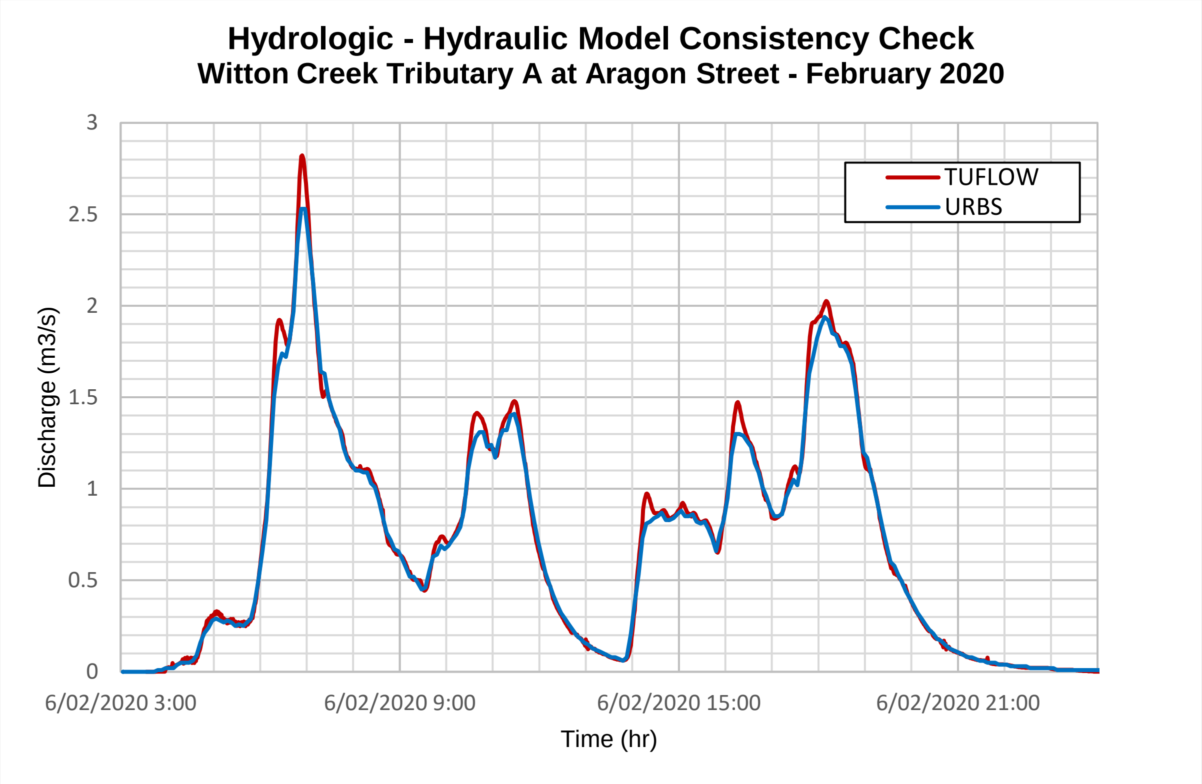
Appendix D: URBS – TUFLOW Comparative Plots

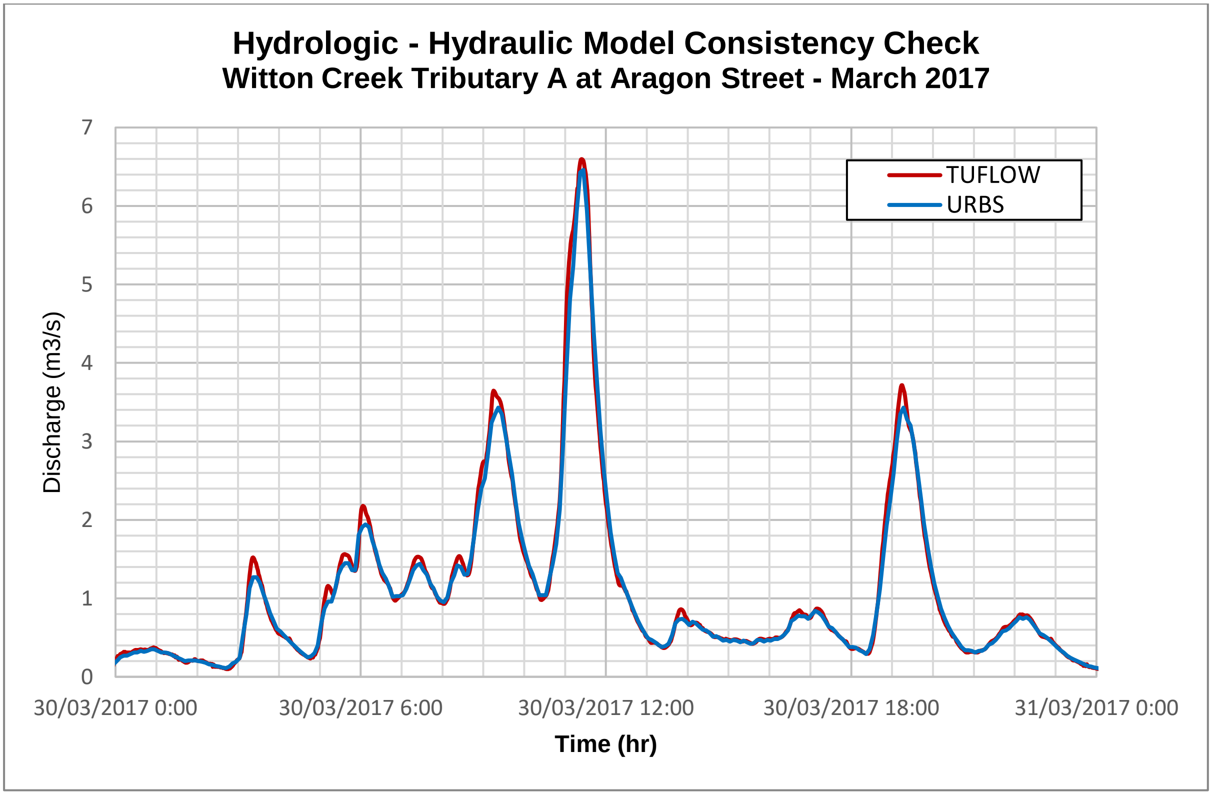
For further information on the hydrologic – hydraulic model consistency checks refer to the following sections:

Calibration Events – Section 5.7

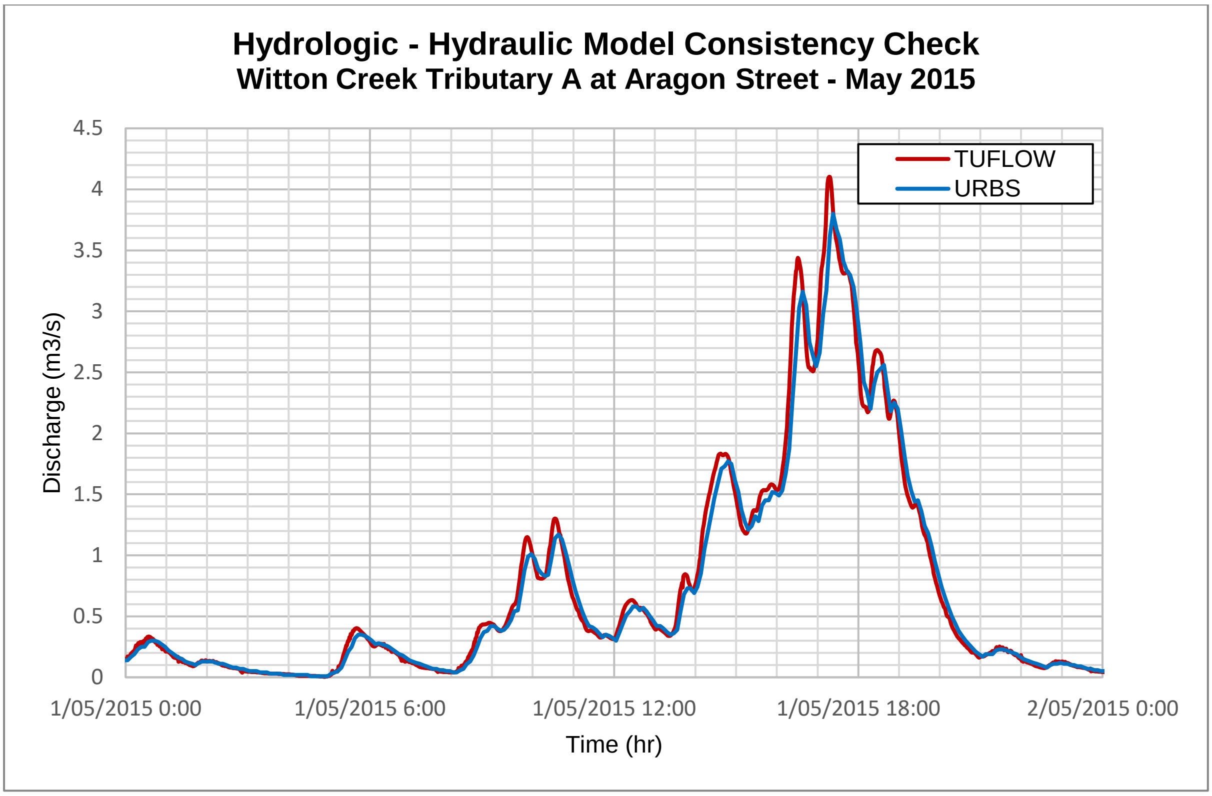
Design Events –Section 6.4.5

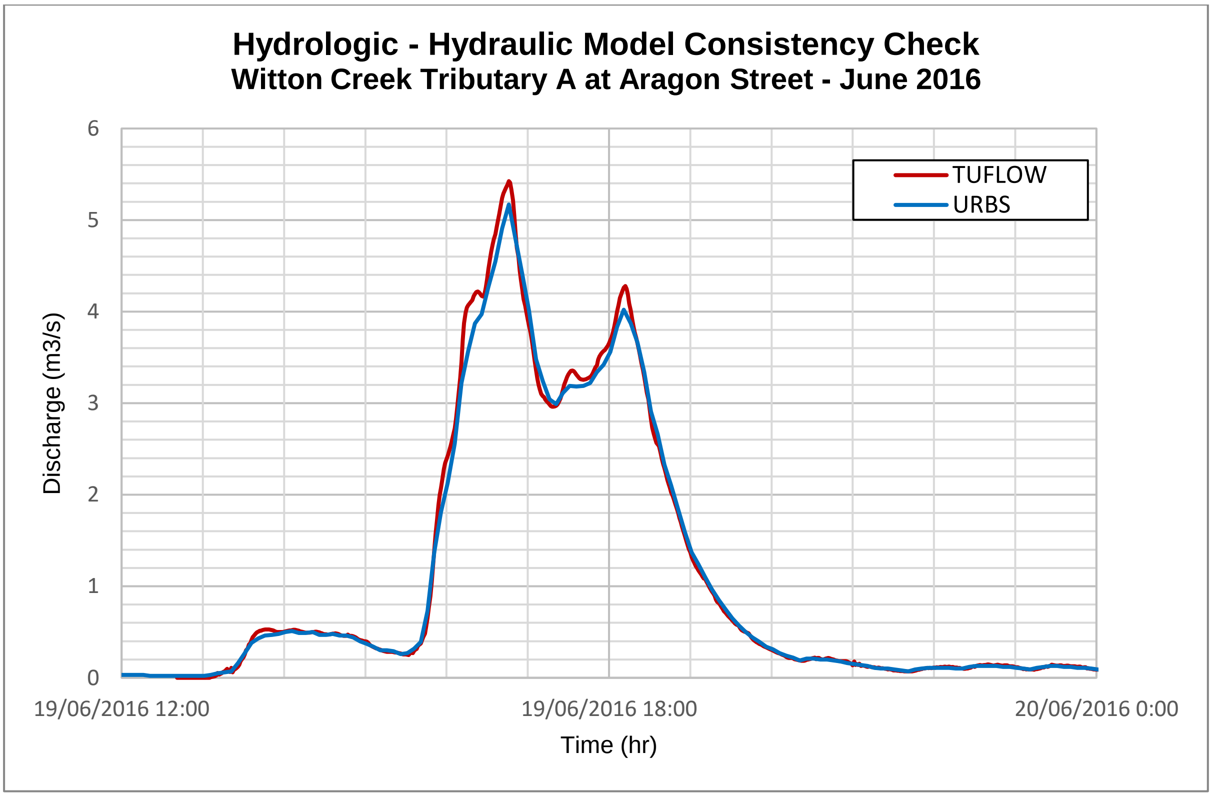


Hydrologic - Hydraulic Model Consistency Check
Witton Creek Tributary A at Aragon Street - February 2020
TUFLOW
URBS
Discharge (m3/s)
0
0.5
1
1.5
2
2.5
3
6/02/2020 3:00
6/02/2020 9:00
6/02/2020 15:00
6/02/2020 21:00
Time (hr)



Hydrologic - Hydraulic Model Consistency Check
Witton Creek Tributary A at Aragon Street - March 2017
TUFLOW
URBS
Discharge (m3/s)
0
1
2
3
4
5
6
7
30/03/2017 0:00
30/03/2017 6:00
30/03/2017 12:00
30/03/2017 18:00
31/03/2017 0:00
Time (hr)





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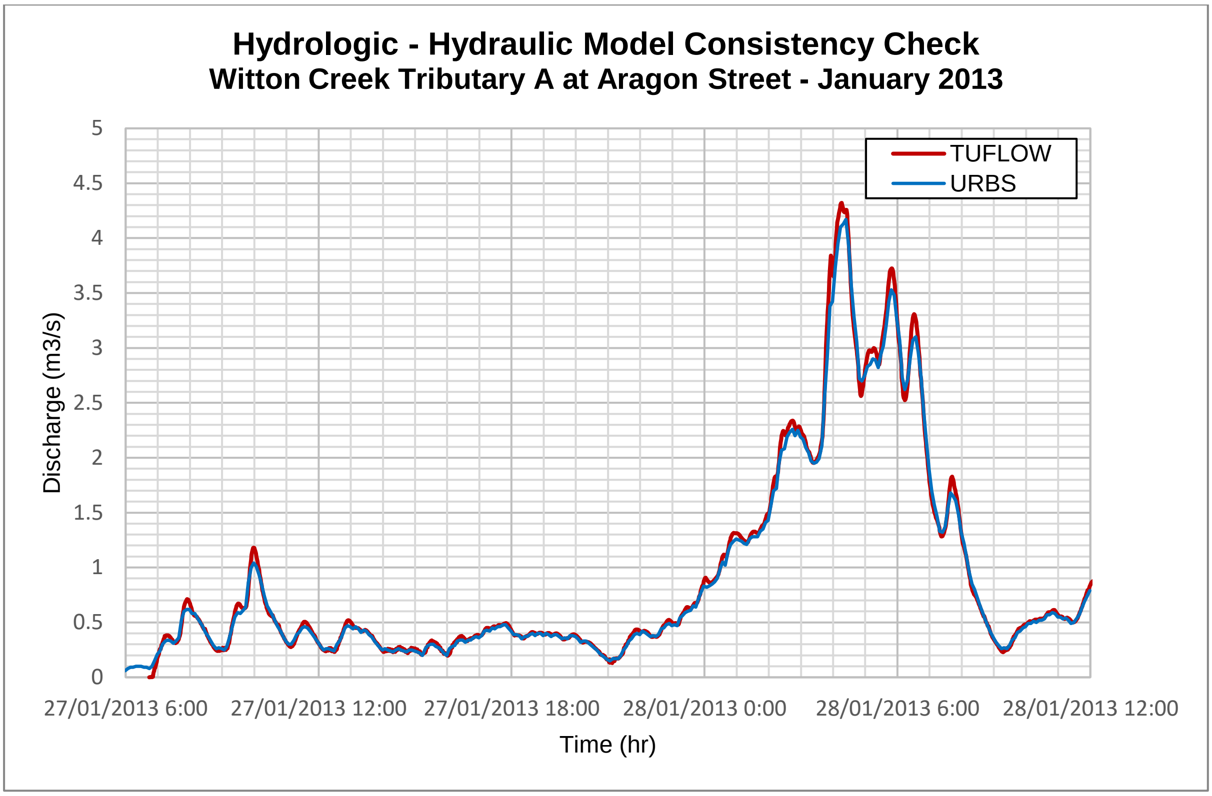
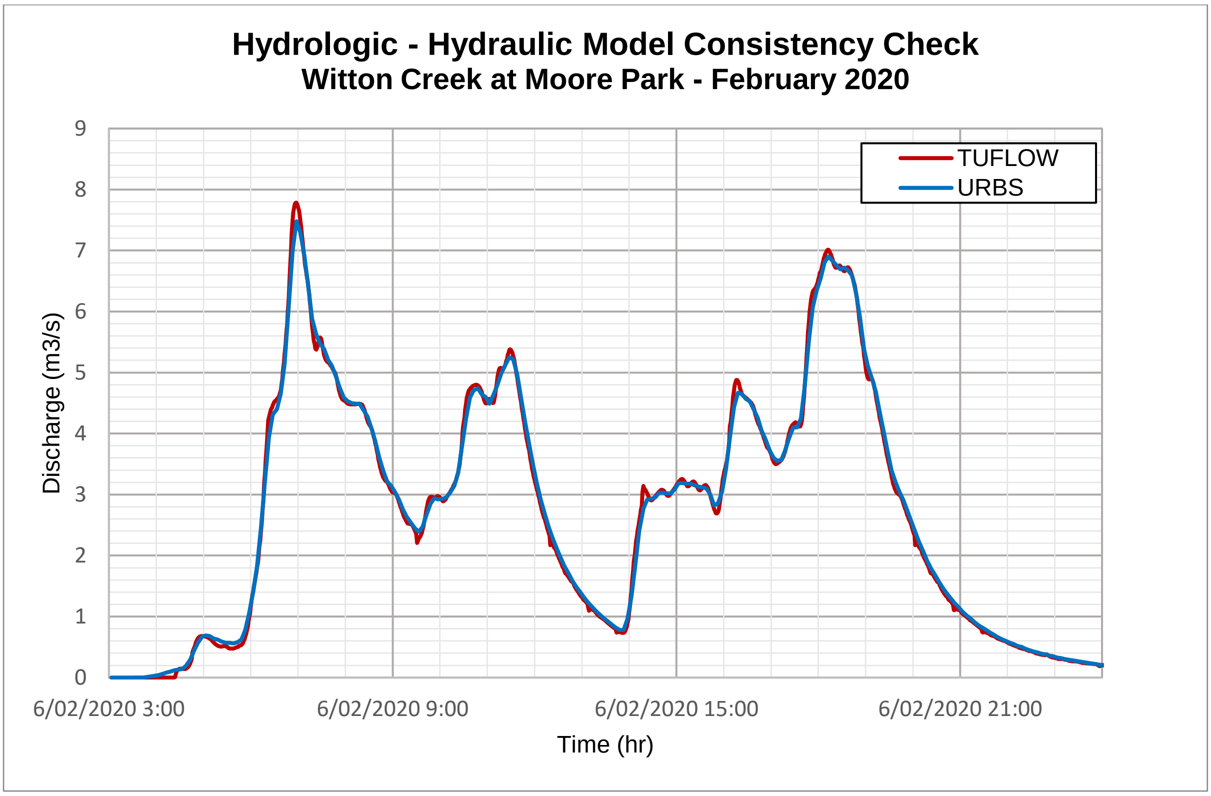
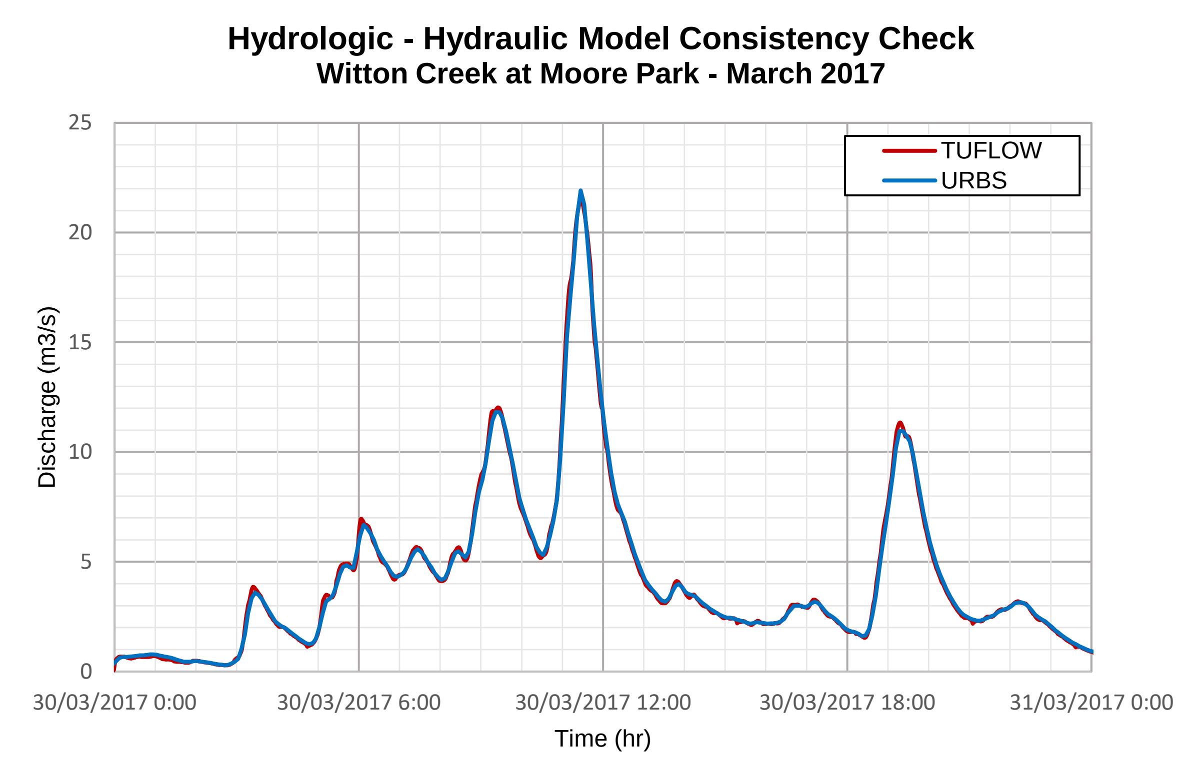
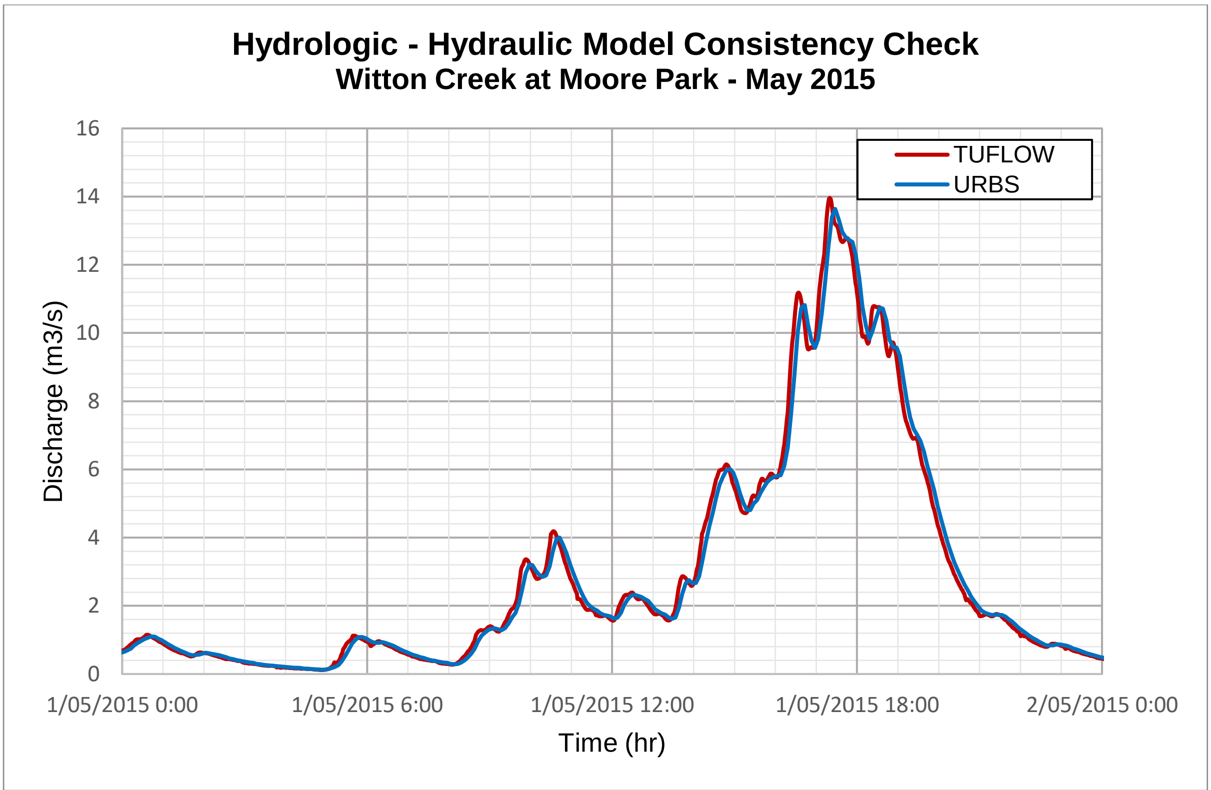
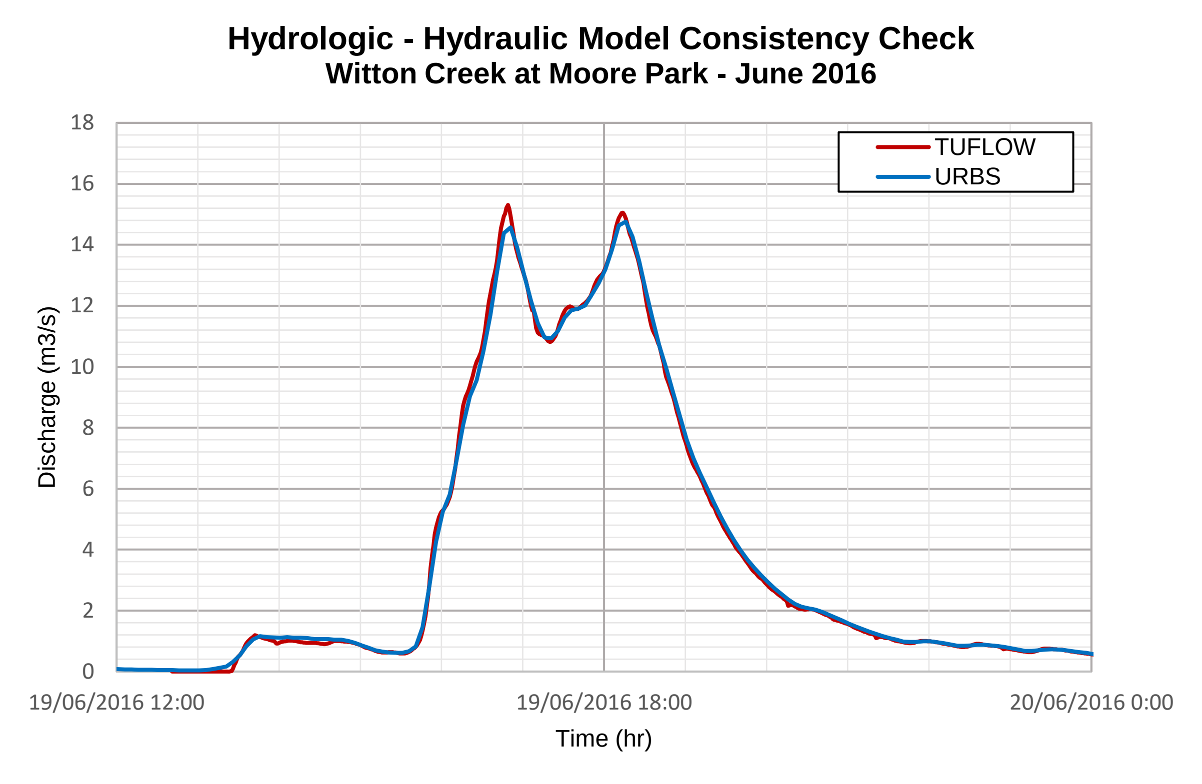
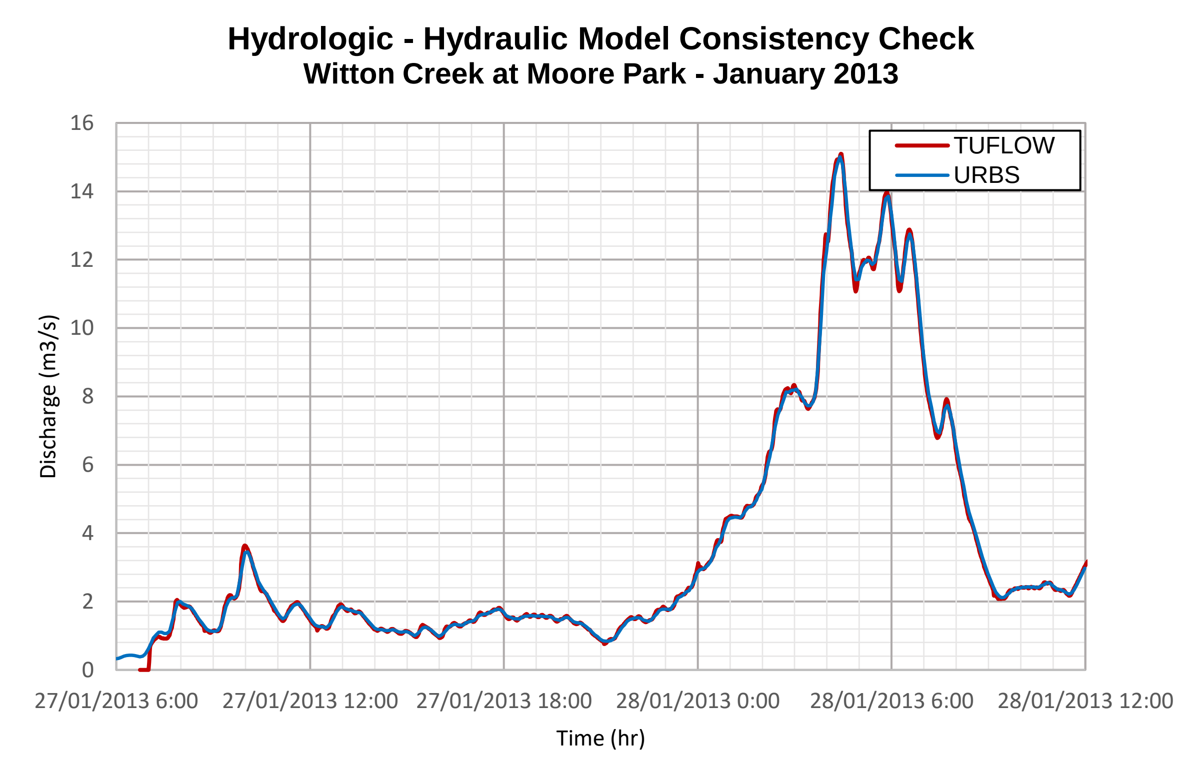

Hydrologic - Hydraulic Model Consistency Check
Witton Creek Tributary A at Aragon Street - January 2013
TUFLOW
URBS
Discharge (m3/s)
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
27/01/2013 6:00
27/01/2013 12:00
27/01/2013 18:00
28/01/2013 0:00
28/01/2013 6:00
28/01/2013 12:00
Time (hr)

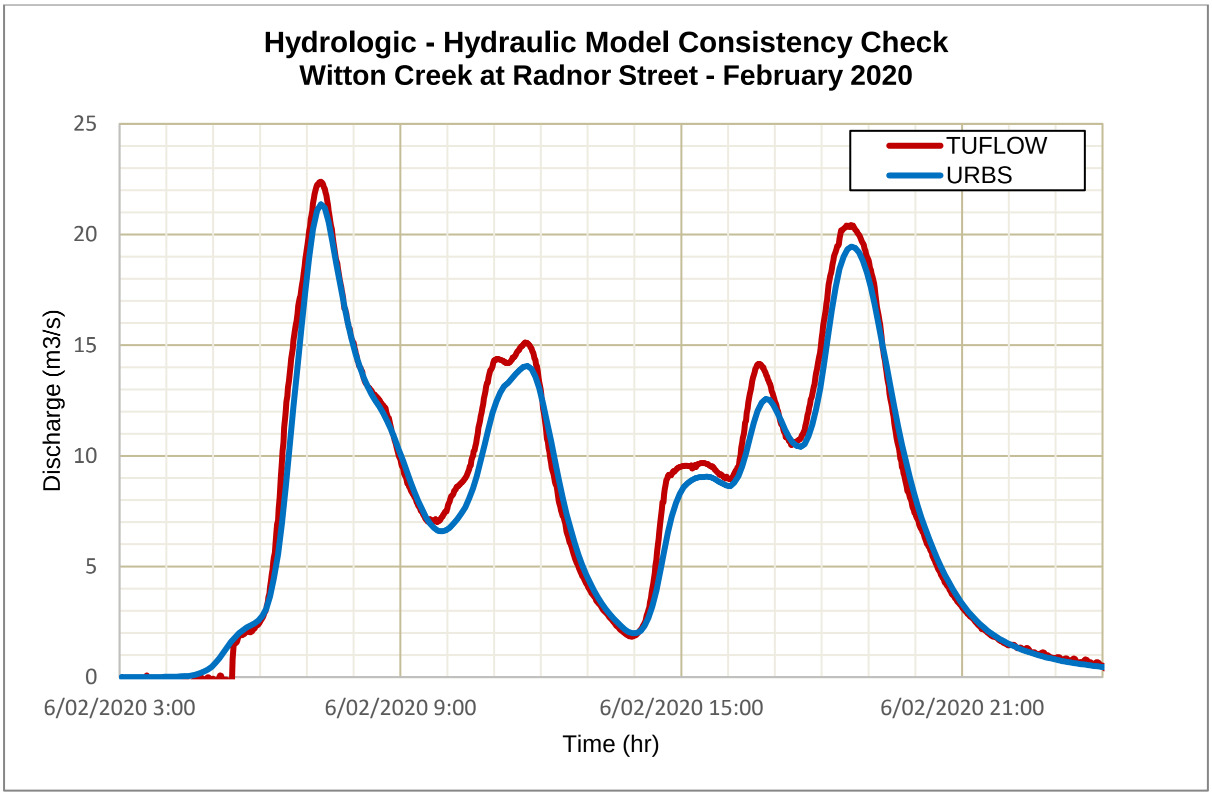

Hydrologic - Hydraulic Model Consistency Check
Witton Creek at Moore Park - February 2020
TUFLOW
URBS
Discharge (m3/s)
9
8
7
6
5
4
3
2
1
0
6/02/2020 3:00
6/02/2020 9:00
6/02/2020 15:00
6/02/2020 21:00
Time (hr)

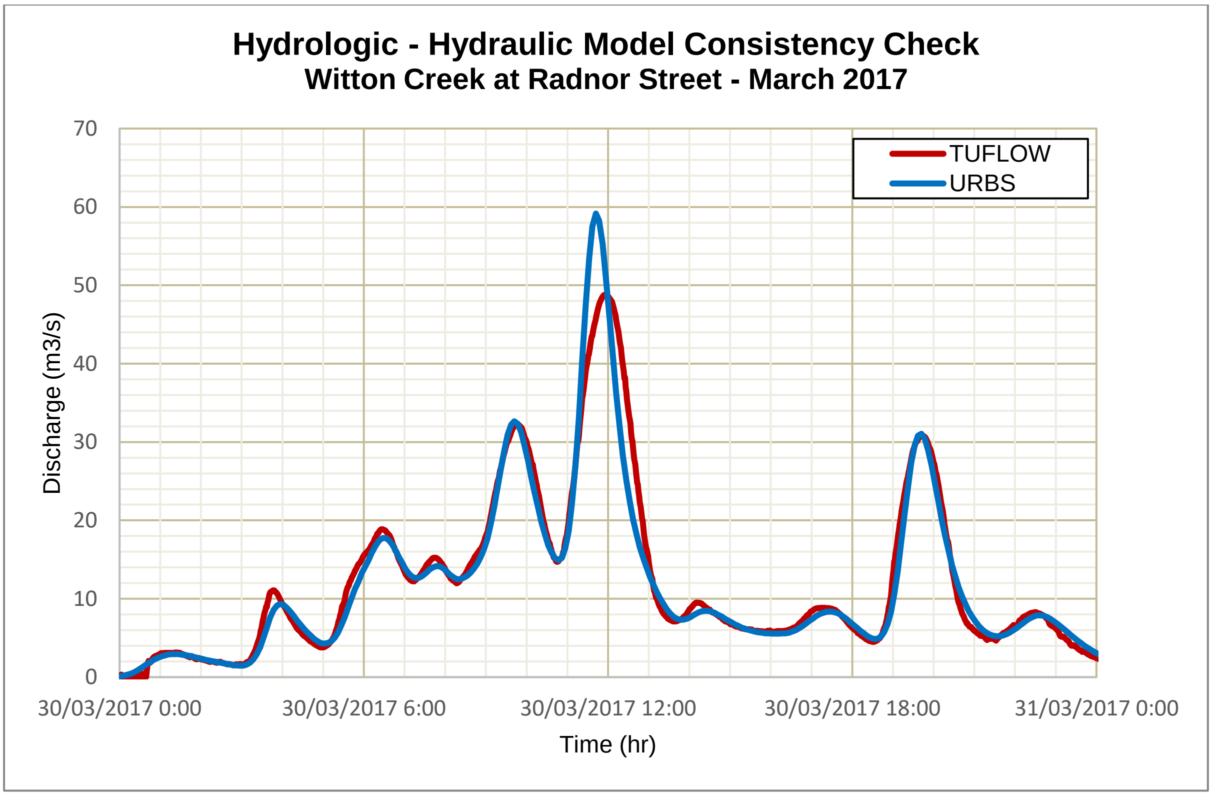

Hydrologic - Hydraulic Model Consistency Check
Witton Creek at Moore Park - March 2017
TUFLOW
URBS
Discharge (m3/s)
0
5
10
15
20
25
30/03/2017 0:00
30/03/2017 6:00
30/03/2017 12:00
30/03/2017 18:00
31/03/2017 0:00
Time (hr)

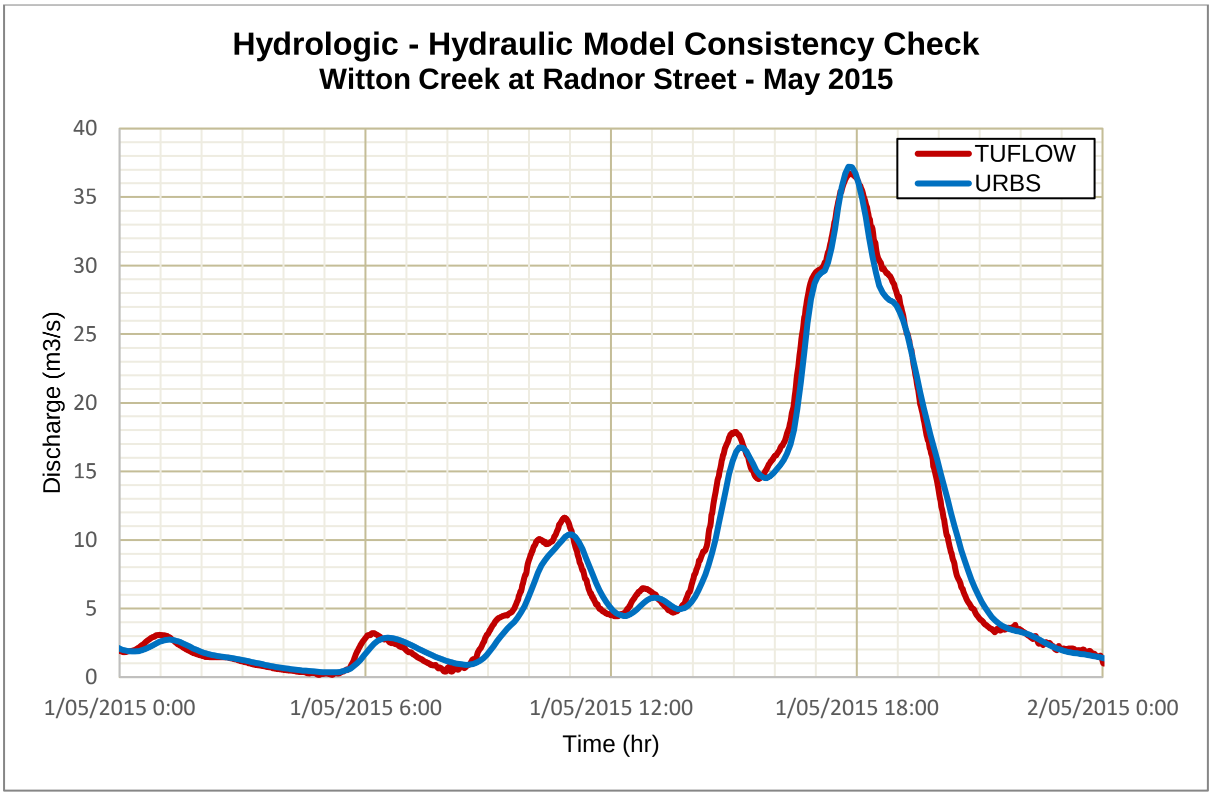

Hydrologic - Hydraulic Model Consistency Check
Witton Creek at Moore Park - May 2015
TUFLOW
URBS
Discharge (m3/s)
0
2
4
6
8
10
12
14
16
1/05/2015 0:00
1/05/2015 6:00
1/05/2015 12:00
1/05/2015 18:00
2/05/2015 0:00
Time (hr)

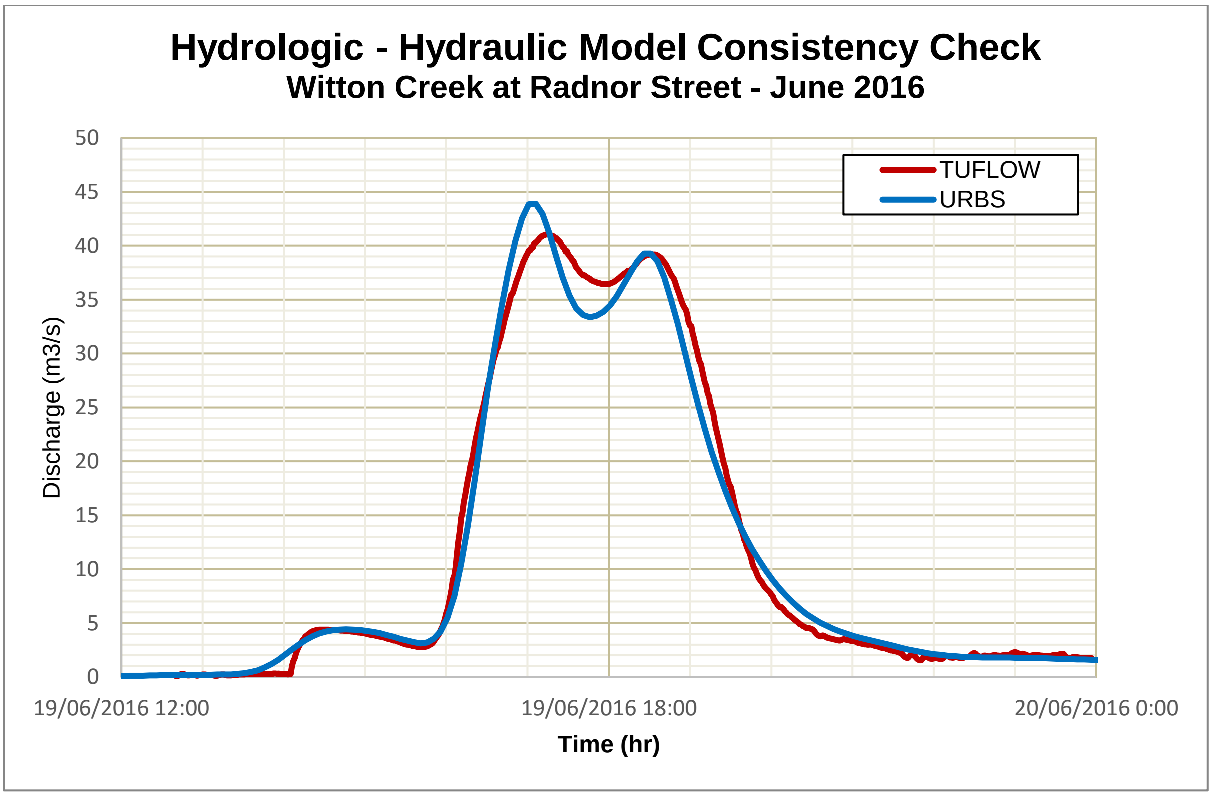

Hydrologic - Hydraulic Model Consistency Check
Witton Creek at Moore Park - June 2016
TUFLOW
URBS
Discharge (m3/s)
0
2
4
6
8
10
12
14
16
18
19/06/2016 12:00
19/06/2016 18:00
20/06/2016 0:00
Time (hr)

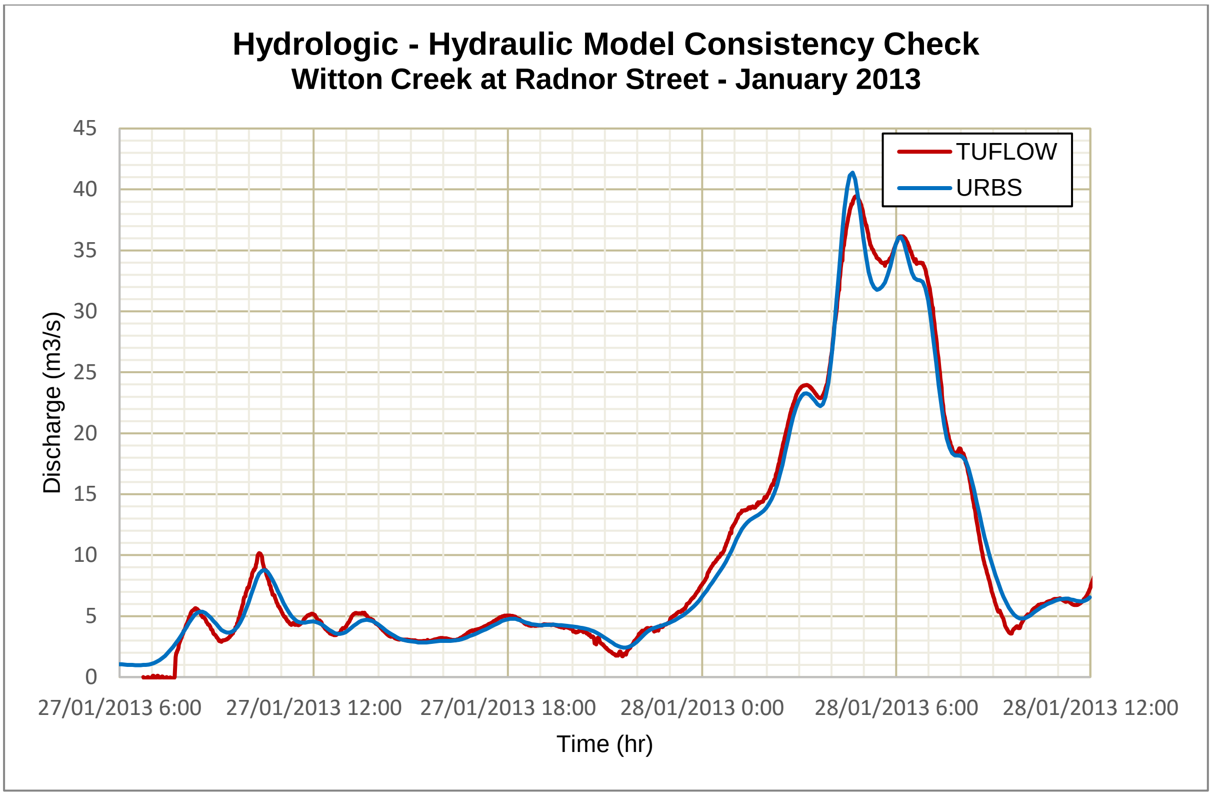

Hydrologic - Hydraulic Model Consistency Check
Witton Creek at Moore Park - January 2013
TUFLOW
URBS
Discharge (m3/s)
0
2
4
6
8
10
12
14
16
27/01/2013 6:00
27/01/2013 12:00
27/01/2013 18:00
28/01/2013 0:00
28/01/2013 6:00
28/01/2013 12:00
Time (hr)


Hydrologic - Hydraulic Model Consistency Check
Witton Creek at Radnor Street - February 2020
TUFLOW
URBS
Discharge (m3/s)
25
20
15
10
5
0
6/02/2020 3:00
6/02/2020 9:00
6/02/2020 15:00
6/02/2020 21:00
Time (hr)


Hydrologic - Hydraulic Model Consistency Check
Witton Creek at Radnor Street - March 2017
TUFLOW
URBS
Discharge (m3/s)
70
60
50
40
30
20
10
0
30/03/2017 0:00
30/03/2017 6:00
30/03/2017 12:00
30/03/2017 18:00
31/03/2017 0:00
Time (hr)


Hydrologic - Hydraulic Model Consistency Check
Witton Creek at Radnor Street - May 2015
TUFLOW
URBS
Discharge (m3/s)
40
35
30
25
20
15
10
5
0
1/05/2015 0:00
1/05/2015 6:00
1/05/2015 12:00
1/05/2015 18:00
2/05/2015 0:00
Time (hr)


Hydrologic - Hydraulic Model Consistency Check
Witton Creek at Radnor Street - June 2016
TUFLOW
URBS
Discharge (m3/s)
50
45
40
35
30
25
20
15
10
5
0
19/06/2016 12:00
19/06/2016 18:00
20/06/2016 0:00
Time (hr)



Hydrologic - Hydraulic Model Consistency Check
Witton Creek at Radnor Street - January 2013
Discharge (m3/s)
45
40
35
30
25
20
15
10
5
0
TUFLOW
URBS
27/01/2013 6:00
27/01/2013 12:00
27/01/2013 18:00
28/01/2013 0:00
28/01/2013 6:00
28/01/2013 12:00
Time (hr)

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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.

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) + CC Peak Water Levels (mAHD) [(2)]					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Witton Creek						
0	1.84	1.84	1.84	1.84	1.84	1.84
100	2.15	2.37	2.53	2.65	2.79	2.93
200	2.60	2.93	3.13	3.30	3.48	3.63
300	2.83	3.19	3.41	3.60	3.79	3.94
400	3.06	3.46	3.72	3.87	4.05	4.21
500	3.72	4.12	4.39	4.57	4.72	4.85
600	3.95	4.33	4.58	4.77	4.94	5.07
700	4.02	4.38	4.62	4.80	4.96	5.10
800	4.30	4.58	4.73	4.86	5.00	5.12
900	4.87	5.27	5.37	5.47	5.58	5.64
1000	4.99	5.47	5.59	5.69	5.80	5.87
1100	6.22	6.60	6.67	6.72	6.80	6.85
1200	6.99	7.38	7.44	7.49	7.56	7.60
1300	8.62	9.28	9.50	9.65	9.80	9.88
1400	9.66	9.93	10.05	10.15	10.26	10.33
1500	N/R	N/R	N/R	N/R	N/R	N/R
1600	12.41	12.77	12.95	13.14	13.34	13.50
1700	12.77	12.93	13.06	13.21	13.40	13.56
1800	N/R	14.49	14.85	14.99	15.07	15.14
1900	14.43	14.74	14.93	15.05	15.13	15.20
2000	16.09	16.32	16.43	16.50	16.57	16.64
2100	17.13	17.36	17.47	17.55	17.63	17.71
2200	18.79	18.94	19.02	19.08	19.15	19.21
2300	20.44	20.64	20.73	20.79	20.87	20.94
2400	21.61	21.92	22.07	22.17	22.31	22.43
Witton Creek Tributary A						
0	4.59	4.85	4.95	5.03	5.12	5.20
100	4.75	4.99	5.07	5.14	5.20	5.26
200	4.87	5.12	5.20	5.28	5.36	5.41
300	N/R	N/R	N/R	5.49	5.51	5.53
400	6.20	6.31	6.35	6.38	6.40	6.41
500	8.12	8.29	8.34	8.37	8.41	8.43
Witton Creek Tributary B						
0	9.37	9.62	9.75	9.86	9.99	10.07
100	10.45	10.58	10.61	10.63	10.66	10.70
200	11.56	11.72	11.78	11.80	11.86	11.93
300	15.66	15.71	15.73	15.76	15.78	15.80

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) + CC Peak Water Levels (mAHD) (2)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Witton Creek Tributary C						
0	11.87	12.06	12.12	12.16	12.20	12.27
50	N/R	12.85	12.89	12.90	13.01	13.08
Witton Creek Tributary D						
0	16.89	17.11	17.22	17.30	17.38	17.45
100	20.15	20.20	20.21	20.23	20.26	20.27
Witton Creek Tributary E						
0	22.64	22.83	22.91	22.97	23.05	23.12
100	N/R	N/R	N/R	N/R	N/R	N/R
200	N/R	N/R	N/R	N/R	N/R	N/R
300	N/R	N/R	N/R	N/R	N/R	N/R
400	36.67	36.84	36.98	37.17	37.33	37.71
500	37.32	37.37	37.40	37.42	37.46	37.72
600	43.11	43.14	43.15	43.16	43.17	43.18

(1) N/R = no result, typically because the AMTD line does not intersect the flood surface.

(2) Flood levels are inclusive of a 9.8% increase in rainfall intensity and a 0.8m increase above MHWS, due to projected climate variability effects.

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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.

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) + CC					
	Peak Water Levels (mAHD) (2)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Witton Creek						
0	1.84	1.84	1.84	1.84	1.84	1.84
100	2.45	2.88	3.10	3.30	3.47	3.62
200	2.99	3.57	3.86	4.12	4.34	4.53
300	3.34	3.98	4.28	4.54	4.78	4.97
400	3.77	4.39	4.70	4.97	5.20	5.40
500	4.26	4.85	5.11	5.34	5.55	5.73
600	4.45	5.03	5.29	5.50	5.70	5.87
700	4.48	5.05	5.31	5.52	5.71	5.88
800	4.67	5.09	5.33	5.54	5.73	5.89
900	5.21	5.46	5.58	5.72	5.88	6.01
1000	5.32	5.63	5.76	5.90	6.08	6.20
1100	6.63	7.01	7.08	7.15	7.24	7.30
1200	7.56	7.96	8.02	8.09	8.17	8.22
1300	8.74	9.53	9.70	9.85	10.02	10.11
1400	9.84	10.17	10.29	10.42	10.57	10.67
1500	N/R	N/R	N/R	N/R	N/R	N/R
1600	12.55	12.91	13.09	13.28	13.48	13.63
1700	12.91	13.11	13.24	13.39	13.57	13.71
1800	N/R	14.67	14.98	15.11	15.20	15.27
1900	14.68	15.07	15.27	15.39	15.49	15.57
2000	16.28	16.56	16.69	16.77	16.85	16.93
2100	17.34	17.63	17.77	17.87	17.97	18.07
2200	18.93	19.16	19.28	19.36	19.46	19.56
2300	20.62	20.88	21.00	21.08	21.18	21.27
2400	21.68	21.98	22.13	22.22	22.35	22.47
Witton Creek Tributary A						
0	5.00	5.23	5.41	5.59	5.76	5.92
100	5.18	5.36	5.49	5.64	5.81	5.96
200	5.43	5.67	5.77	5.87	5.99	6.12
300	5.48	5.72	5.81	5.90	6.02	6.14
400	6.44	6.60	6.67	6.72	6.76	6.79
500	8.51	8.74	8.81	8.86	8.91	8.93
Witton Creek Tributary B						
0	9.61	9.95	10.08	10.21	10.37	10.47
100	10.57	10.72	10.77	10.79	10.85	10.91
200	11.66	11.87	11.95	11.99	12.05	12.14
300	15.66	15.71	15.73	15.76	15.78	15.80

AMTD (m)	Design Events – Scenario 3 (Ultimate Waterway Conditions) + CC					
	Peak Water Levels (mAHD) (2)					
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
Witton Creek Tributary C						
0	11.92	12.16	12.24	12.28	12.34	12.43
50	12.96	13.15	13.20	13.22	13.32	13.39
Witton Creek Tributary D						
0	17.10	17.39	17.53	17.63	17.72	17.81
100	20.20	20.26	20.28	20.31	20.34	20.36
Witton Creek Tributary E						
0	22.73	22.96	23.07	23.15	23.26	23.35
100	N/R	N/R	N/R	N/R	N/R	N/R
200	N/R	N/R	N/R	N/R	N/R	N/R
300	N/R	N/R	N/R	N/R	N/R	N/R
400	36.71	36.88	37.02	37.19	37.36	37.74
500	37.36	37.43	37.47	37.50	37.54	37.77
600	43.12	43.14	43.15	43.17	43.18	43.18

(1) N/R = no result, typically because the AMTD line does not intersect the flood surface.

(2) Flood levels are inclusive of a 9.8% increase in rainfall intensity and a 0.8m increase above MHWS, due to projected climate variability effects.

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Appendix G: Very 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.

AMTD (m)	Scenario 1 (Existing Waterway Conditions) + CC Peak Water Levels (mAHD) [(2)]		
	0.5% AEP	0.2% AEP	0.05% AEP
Witton Creek			
0	2.44	2.44	2.44
100	3.13	3.34	3.68
200	3.85	4.12	4.52
300	4.18	4.47	4.91
400	4.45	4.77	5.21
500	5.04	5.29	5.66
600	5.27	5.51	5.86
700	5.29	5.53	5.87
800	5.31	5.54	5.88
900	5.72	5.87	6.10
1000	5.96	6.14	6.45
1100	6.94	7.08	7.28
1200	7.67	7.78	7.92
1300	9.99	10.12	10.25
1400	10.43	10.56	10.71
1500	N/R	N/R	N/R
1600	13.78	14.27	15.50
1700	13.82	14.29	15.51
1800	15.24	15.36	15.60
1900	15.29	15.41	15.61
2000	16.72	16.85	17.06
2100	17.80	17.96	18.18
2200	19.30	19.44	19.65
2300	21.02	21.16	21.36
2400	22.51	22.88	23.26
Witton Creek Tributary A			
0	5.35	5.57	5.90
100	5.38	5.56	5.89
200	5.50	5.67	5.98
300	5.59	5.70	5.99
400	6.44	6.48	6.53
500	8.46	8.51	8.57
Witton Creek Tributary B			
0	10.17	10.31	10.45
100	10.76	10.84	10.95
200	12.04	12.18	12.38
300	15.83	15.86	15.92

AMTD (m)	Scenario 1 (Existing Waterway Conditions) + CC Peak Water Levels (mAHD) (2)		
	0.5% AEP	0.2% AEP	0.05% AEP
Witton Creek Tributary C			
0	12.38	12.51	12.68
50	13.17	13.29	13.45
Witton Creek Tributary D			
0	17.55	17.69	17.90
100	20.30	20.34	20.40
Witton Creek Tributary E			
0	23.20	23.41	23.71
100	N/R	N/R	N/R
200	N/R	N/R	N/R
300	N/R	N/R	N/R
400	38.24	38.69	39.33
500	38.25	38.69	39.33
600	43.19	43.21	43.23

(1) N/R = no result, typically because the AMTD line does not intersect the flood surface

(2) Flood levels are inclusive of a 9.8% increase in rainfall intensity and a 0.8m increase above HAT, due to projected climate variability effects.

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Appendix H: Very 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.

AMTD (m)	Scenario 3 (Ultimate Waterway Conditions) + CC Peak Water Levels (mAHD) [(2)]		
	1% AEP	0.5% AEP	0.2% AEP
Witton Creek			
0	1.84	2.44	2.44
100	3.62	3.95	4.20
200	4.53	4.90	5.21
300	4.97	5.34	5.66
400	5.40	5.70	6.00
500	5.73	5.95	6.21
600	5.87	6.09	6.34
700	5.88	6.10	6.36
800	5.89	6.11	6.37
900	6.01	6.21	6.45
1000	6.20	6.41	6.65
1100	7.30	7.40	7.52
1200	8.22	8.30	8.40
1300	10.11	10.22	10.38
1400	10.67	10.80	10.95
1500	N/R	N/R	N/R
1600	13.63	13.87	14.35
1700	13.71	13.93	14.39
1800	15.27	15.37	15.50
1900	15.57	15.68	15.82
2000	16.93	17.03	17.19
2100	18.07	18.19	18.36
2200	19.56	19.67	19.85
2300	21.27	21.38	21.56
2400	22.47	22.61	22.94
Witton Creek Tributary A			
0	5.92	6.14	6.39
100	5.96	6.18	6.42
200	6.12	6.32	6.53
300	6.14	6.33	6.54
400	6.79	6.85	6.92
500	8.93	8.97	9.01
Witton Creek Tributary B			
0	10.47	10.59	10.75
100	10.91	11.02	11.17
200	12.14	12.28	12.46
300	15.80	15.83	15.88

AMTD (m)	Scenario 3 (Ultimate Waterway Conditions) + CC Peak Water Levels (mAHD) (2)		
	1% AEP	0.5% AEP	0.2% AEP
Witton Creek Tributary C			
0	12.43	12.57	12.73
50	13.39	13.50	13.62
Witton Creek Tributary D			
0	17.81	17.92	18.09
100	20.36	20.39	20.44
Witton Creek Tributary E			
0	23.35	23.46	23.69
100	N/R	N/R	N/R
200	N/R	N/R	N/R
300	N/R	N/R	N/R
400	37.74	38.27	38.71
500	37.77	38.28	38.71
600	43.18	43.20	43.21

(1) N/R = no result, typically because the AMTD line does not intersect the flood surface.

(2) Flood levels are inclusive of a 9.8% increase in rainfall intensity and a 0.8m increase above HAT, due to projected climate variability effects.

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Appendix I: Design Events (Scenario 1) – Critical Duration and Median Ensemble

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) + CC [(2)]					
	50% AEP		20% AEP		10% AEP	
	Critical Duration	Median Ensemble [(3)]	Critical Duration	Median Ensemble [(3)]	Critical Duration	Median Ensemble [(3)]
Witton Creek						
0	0.5	6	0.5	3	0.5	0.5
100	1	4	3	1	1.5	1
200	1	4	3	1	1.5	1
300	1	4	3	1	1.5	1
400	1	4	3	1	1.5	1
500	1	4	3	1	1.5	1
600	1	4	3	1	1.5	1
700	1	4	3	1	1.5	1
800	1	0	3	1	1.5	1
900	1	4	1	4	1.5	1
1000	1	4	1	4	1.5	1
1100	1	3	1	4	1	1
1200	1	3	1	4	0.75	1
1300	1	3	1	4	0.75	1
1400	1	3	1	0	0.75	1
1500	N/R	N/R	N/R	N/R	N/R	N/R
1600	1	3	1	3	0.75	1
1700	1	3	1	3	0.75	1
1800	N/R	N/R	1	3	0.75	N/R
1900	1	3	1	3	0.75	1
2000	1	3	1	3	0.75	1
2100	1	3	1	3	0.75	1
2200	1	3	1	3	0.75	1
2300	1	3	1	3	0.75	1
2400	1	3	1	3	0.75	1
Witton Creek Tributary A						
0	1	6	3	1	1.5	1
100	1	6	1	3	1.5	1
200	1	6	1	3	0.75	1
300	N/R	N/R	N/R	N/R	N/R	N/R
400	1	3	1	6	0.75	1
500	0.5	4	0.5	7	0.75	0.5
Witton Creek Tributary B						
0	1	3	1	4	0.75	1
100	0.5	4	0.5	4	0.75	0.5
200	0.5	4	0.5	4	0.5	0.5
300	0.5	4	0.5	7	0.5	0.5

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) + CC [(2)]					
	50% AEP		20% AEP		10% AEP	
	Critical Duration	Median Ensemble [(3)]	Critical Duration	Median Ensemble [(3)]	Critical Duration	Median Ensemble [(3)]
Witton Creek Tributary C						
0	0.5	4	0.5	4	0.5	0.5
50	N/R	N/R	0.5	4	0.5	N/R
Witton Creek Tributary D						
0	1	3	1	3	0.75	1
100	0.5	4	0.5	8	0.5	0.5
Witton Creek Tributary E						
0	1	3	1	3	0.75	1
100	N/R	N/R	N/R	N/R	N/R	N/R
200	N/R	N/R	N/R	N/R	N/R	N/R
300	N/R	N/R	N/R	N/R	N/R	N/R
400	1	3	3	4	2	1
500	1	3	1	3	2	1
600	1	3	1	3	0.75	1

(1) N/R = no result, typically because the AMTD line does not intersect the flood surface.
(2) Inclusive of a 9.8% increase in rainfall intensity and a 0.8m increase above MHWS, due to projected climate variability effects.
(3) Reported as URBs temporal pattern notation (TP0, TP1, TP2, TP3, TP4, TP5, TP6, TP7, TP8, TP9)
(4) All durations in hours

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) + CC [2]					
	5% AEP		2% AEP		1% AEP	
	Critical Duration	Median Ensemble [3]	Critical Duration	Median Ensemble [3]	Critical Duration	Median Ensemble [3]
Witton Creek						
0	0.5	7	0.5	2	1.5	4
100	1.5	5	1.5	4	1.5	4
200	1.5	5	1.5	4	1.5	4
300	1.5	5	1.5	4	1.5	4
400	1.5	5	1.5	4	1.5	4
500	1.5	5	1.5	4	1.5	4
600	1.5	5	1.5	4	1.5	4
700	1.5	5	1.5	4	1.5	4
800	1.5	5	1.5	4	1.5	4
900	1.5	5	1.5	2	1.5	2
1000	1.5	5	1.5	2	1.5	2
1100	1.5	5	1.5	2	1.5	2
1200	1.5	5	1.5	2	1.5	2
1300	0.75	4	1.5	2	1.5	2
1400	0.75	4	1.5	2	1.5	2
1500	N/R	N/R	N/R	N/R	N/R	N/R
1600	2	7	1.5	2	1.5	2
1700	2	7	1.5	2	1.5	2
1800	0.75	5	0.75	2	0.75	2
1900	0.75	5	0.75	2	0.75	2
2000	0.75	5	0.75	2	0.75	2
2100	0.75	5	0.75	2	0.75	2
2200	0.75	5	0.75	2	0.75	2
2300	0.75	5	0.75	2	0.75	2
2400	0.75	5	0.75	2	0.75	2
Witton Creek Tributary A						
0	1.5	5	1.5	8	1.5	4
100	1.5	5	1.5	8	1.5	4
200	1	7	1.5	2	1.5	2
300	1	7	1	3	1	8
400	0.75	5	0.5	7	0.5	3
500	0.5	2	0.5	1	0.5	1
Witton Creek Tributary B						
0	0.75	4	1.5	2	1.5	2
100	0.5	7	0.75	7	0.75	7
200	0.5	7	0.75	7	0.75	7
300	0.5	7	0.5	1	0.5	1

AMTD (m)	Design Events – Scenario 1 (Existing Waterway Conditions) + CC (2)					
	5% AEP		2% AEP		1% AEP	
	Critical Duration	Median Ensemble (3)	Critical Duration	Median Ensemble (3)	Critical Duration	Median Ensemble (3)
Witton Creek Tributary C						
0	0.5	7	0.75	7	0.75	7
50	0.5	7	0.75	7	0.75	7
Witton Creek Tributary D						
0	0.75	5	0.75	2	0.75	2
100	0.5	7	0.5	1	0.5	1
Witton Creek Tributary E						
0	2	7	0.75	2	0.75	2
100	N/R	N/R	N/R	N/R	N/R	N/R
200	N/R	N/R	N/R	N/R	N/R	N/R
300	N/R	N/R	N/R	N/R	N/R	N/R
400	2	9	1.5	4	1.5	2
500	2	9	1.5	4	1.5	2
600	0.75	6	0.75	2	0.75	2

(1) N/R = no result, typically because the AMTD line does not intersect the flood surface.
(2) Inclusive of a 9.8% increase in rainfall intensity and a 0.8m increase above MHWS, due to projected climate variability effects.
(3) Reported as URBs temporal pattern notation (TP0, TP1, TP2, TP3, TP4, TP5, TP6, TP7, TP8, TP9)
(4) All durations in hours

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Appendix J: Very Rare Events (Scenario 1) – Critical Duration and Median Ensemble

AMTD (m)	Very Rare Events – Scenario 1 (Existing Waterway Conditions) + CC (2)					
	0.5% AEP		0.2% AEP		0.05% AEP	
	Critical Duration	Median Ensemble	Critical Duration	Median Ensemble	Critical Duration	Median Ensemble
Witton Creek						
0	0.75	0	0.5	8	1	9
100	1.5	4	1.5	4	1.5	4
200	1.5	4	1.5	4	1.5	4
300	1.5	4	1.5	4	1.5	4
400	1.5	4	1.5	4	1.5	4
500	1.5	4	1.5	4	1.5	4
600	1.5	4	1.5	4	1.5	4
700	1.5	4	1.5	4	1.5	4
800	1.5	4	1.5	4	1.5	4
900	1.5	8	1.5	2	1.5	8
1000	1.5	2	1.5	2	1.5	2
1100	0.75	5	1.5	2	1.5	2
1200	0.75	5	0.75	7	1.5	2
1300	0.75	1	0.75	5	0.75	5
1400	0.75	1	0.75	5	0.75	5
1500	N/R	N/R	N/R	N/R	N/R	N/R
1600	1.5	2	1.5	2	1.5	2
1700	1.5	2	1.5	2	1.5	2
1800	1.5	2	0.75	2	0.75	2
1900	1.5	2	0.75	2	0.75	2
2000	0.75	2	0.75	2	0.75	2
2100	0.75	2	0.75	2	0.75	2
2200	0.75	2	0.75	2	0.75	2
2300	0.75	2	0.75	2	0.75	2
2400	0.75	2	0.75	2	0.75	2
Witton Creek Tributary A						
0	1.5	4	1.5	4	1.5	4
100	1.5	4	1.5	4	1.5	4
200	1.5	4	1.5	4	1.5	4
300	1.5	4	1.5	4	1.5	2
400	0.5	1	0.5	1	0.5	1
500	0.5	7	0.5	1	0.5	1
Witton Creek Tributary B						
0	0.75	1	0.75	5	0.75	5
100	0.75	1	0.75	7	0.75	7
200	0.75	1	0.75	1	0.75	7
300	0.5	1	0.5	1	0.5	1

AMTD (m)	Very Rare Events – Scenario 1 (Existing Waterway Conditions) + CC (2)					
	0.5% AEP		0.2% AEP		0.05% AEP	
	Critical Duration	Median Ensemble	Critical Duration	Median Ensemble	Critical Duration	Median Ensemble
Witton Creek Tributary C						
0	0.75	1	0.75	1	0.75	7
50	0.5	6	0.75	1	0.75	7
Witton Creek Tributary D						
0	0.75	2	0.75	2	0.75	2
100	0.5	1	0.5	1	0.5	1
Witton Creek Tributary E						
0	0.75	2	0.75	2	0.75	2
100	N/R	N/R	N/R	N/R	N/R	N/R
200	N/R	N/R	N/R	N/R	N/R	N/R
300	N/R	N/R	N/R	N/R	N/R	N/R
400	1.5	4	1.5	4	1.5	8
500	1.5	4	1.5	4	1.5	8
600	0.75	2	0.75	2	0.75	2

(1) N/R = no result, typically because the AMTD line does not intersect the flood surface.
(2) Inclusive of a 9.8% increase in rainfall intensity and a 0.8m increase above HAT, due to projected climate variability effects.
(3) Reported as URBs temporal pattern notation (TP0, TP1, TP2, TP3, TP4, TP5, TP6, TP7, TP8, TP9)
(4) All durations in hours

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Appendix K: Hydraulic Structure Reference Sheets

The hydraulic structure reference sheets provide an overview of the hydraulic characteristics and performance of the waterway structure for the current catchment and climate conditions. They have been compiled from the best available data for the waterway structure.

Peak flood levels and structure flood immunity have typically been extracted from the design flood surface grids at the structure location, while the overtopping level of the weir / road have been derived from the existing ground surface at the low point of the road alignment in the vicinity of the structure (and not necessarily at the structure).

Flooding characteristics at waterway structures can be complex and it is recommended that the hydraulic structure reference sheets be read in conjunction with the results of the TUFLOW model.

Waterway	Structure ID	AMTD	Structure location	Structure details	Modelled structure representation	Origin of Structure Coding	HSRS
Witton Creek	S1	6	Radnor Street	4 Lane Bridge	2D Lfcsh	BCC 2023 Field Survey	YES
	S2	441	Aaron Place	Single Span Bridge	2D Lfcsh	BCC 2023 Field Survey	YES
	S3	714	Kate Street	4 x 3m (W) x1.5 m (H) Culvert	1D Culvert	BCC GIS Data	YES
	S5	892	Witton Road	3 x 3m (W) x1.5 m (H) Culvert	1D Culvert	BCC GIS Data	YES
	S6	948	Western Freeway	4x 3m (W) x 3 m (H) Culvert	1D Culvert	BCC GIS Data	YES
	S7	1231	Moggill Road	4 x DN1500 Culvert	1D Culvert	BCC GIS Data	YES
	S8	1423	Western Freeway Onramp	4 x DN1950 Culvert	1D Culvert	BCC GIS Data	YES
	S9	1482	Western Freeway	3 x DN1950 Culvert	1D Culvert	BCC GIS Data	YES
	S10	1785	Russell Terrace	4 x DN1650 Culvert	1D Culvert	BCC GIS Data	YES
	S11	1800	Russell Terrace	Single 1.2 m (W) by 0.6 m (H) Culvert	1D Culvert	BCC GIS Data	YES
Witton Creek Tributary A	S4	24	Western Freeway Bikeway	Single Span Pedestrian Bridge	2D Lfcsh	BCC 2023 Field Survey	YES
	S12	0	Western Freeway Road Bridge	2 Span Bridge	*Not explicitly represented (no piers single span, soffit well above flood levels) – defined via model topography	BCC 2019 ALS Lidar	YES
	S13	0	Western Freeway Drain Bridge	2 Span Bridge	*Not explicitly represented (no piers single span, soffit well above flood levels) – defined via model topography	BCC 2019 ALS Lidar	YES
	S14	428	Kennewell Park	Single DN1700 Culvert	1D Culvert	BCC 2023 Field Survey	YES

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Radnor Street Bridge (S1)

BCC Asset ID	B1650	Tributary Name	Witton Creek
Owner	Brisbane City Council	AMTD (m)	13.73
Year of Construction	1985	Coordinates (GDA94)	497075.734, 6957684.380
Year of Significant Modification	Unknown	Hydraulic Model ID	RadnorSt_Bridge_WT20
Source of Structure Information	1998 Field Survey 2023 BCC Field Survey	Flood Model Representation	2d lfcsh
Link to Data Source	..\Flood Management\Data\Structures\S1 – Radnor Street Bridge		

Structure Description		2 span concrete bridge	
Bridges		**Culverts**	
Number of Spans	2	Number of Barrels	N/A
Number of Piers in Waterway	1	Dimensions (m)	N/A
Pier shape and Width (m)	Headstock on Piles (circular), 0.62m width	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	-0.16	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		12.2	
Span Length (m)		14.86	
Lowest Level of Deck Soffit (m AHD)		5.55	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		6.51	
Average Handrail Height (m)		0.98	

Image Description	Looking Downstream
Date	30th January 2023
Source	Site inspection undertaken for flood study

Image Description	Looking Downstream
Date	30th January 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				5% AEP to 2% AEP event for Brisbane River Flooding*				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	96.46	96.46	2.27	1.64	0.63	N/A	N/A	1.5 (E4)
0.2	79.53	79.53	1.98	1.64	0.34	N/A	N/A	1.5 (E4)
1	61.55	61.55	1.67	1.04	0.62	N/A	N/A	1.5 (E4)
2	56.78	56.78	1.58	1.04	0.54	N/A	N/A	1.5 (E4)
5	50.96	50.95	1.44	1.04	0.40	N/A	N/A	1.5 (E5)
10	46.93	46.93	1.36	1.04	0.32	N/A	N/A	1.5 (E5)
20	38.98	38.98	1.19	1.04	0.14	N/A	N/A	3 (E1)
50	28.54	28.54	1.04	1.04	0.00	N/A	N/A	1 (E0)

[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*

[(b)]*Backwater affected value*

** Structure is affected by riverine flooding of the Brisbane River*

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Aaron Place Bridge (S2)

BCC Asset ID	Aaron Place Bridge	Tributary Name	Witton Creek
Owner	Brisbane City Council	AMTD (m)	443.34
Year of Construction	Unknown	Coordinates (GDA94)	496751.558, 6957573.096
Year of Significant Modification	Unknown	Hydraulic Model ID	AaronPL_Bridge
Source of Structure Information	1998 Field Survey 2023 BCC Field Survey	Flood Model Representation	2d lfcsh
Link to Data Source	..\Flood Management\Data\Structures\S2 – Aaron Place Bridge		

Structure Description		Single span concrete bridge	
Bridges		**Culverts**	
Number of Spans	1	Number of Barrels	N/A
Number of Piers in Waterway	0	Dimensions (m)	N/A
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	1.02	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		9.1	
Span Length (m)		23.38	
Lowest Level of Deck Soffit (m AHD)		3.6	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		4.4	
Average Handrail Height (m)		1.2	

Image Description	Looking Upstream
Date	30th January 2023
Source	Site inspection undertaken for flood study

Image Description	Looking downstream
Date	30th January 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				< 5% AEP Event for Brisbane River Flooding*				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	95.29	59.30	5.44	5.34	0.10	N/A	N/A	1.5 (E4)
0.2	78.58	57.59	5.03	4.89	0.14	N/A	N/A	1.5 (E4)
1	60.5	55.47	4.52	4.29	0.23	N/A	N/A	1.5 (E4)
2	56.17	53.73	4.36	4.13	0.23	N/A	N/A	1.5 (E8)
5	50.09	49.67	4.17	3.96	0.21	N/A	N/A	1.5 (E5)
10	46.38	46.38	4.03	3.84	0.19	N/A	N/A	1.5 (E5)
20	38.45	38.40	3.67	3.52	0.15	N/A	N/A	3 (E1)
50	28.05	28.05	3.23	3.00	0.22	N/A	N/A	1 (E0)

[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

[(b)]Backwater affected value

** Structure is affected by riverine flooding of the Brisbane River.*

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Kate Street Culvert (S3)

BCC Asset ID	C0195B and C5484B	Tributary Name	Witton Creek
Owner	Brisbane City Council	AMTD (m)	728
Year of Construction	1975	Coordinates (GDA94)	496519.17, 6957422.28
Year of Significant Modification	Unknown	Hydraulic Model ID	C0195B
Source of Structure Information	BCC GIS Dataset: STORMWATER_CULVERT	Flood Model Representation	1d culvert
Link to Data Source	..\Flood Management\Data\Structures\S3 – Kate Street Road Culvert		

Structure Description		Concrete box culverts	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	4
Number of Piers in Waterway	N/A	Dimensions (m)	3x1.5
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	1.825
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	1.685
Structure Length (m) (in direction of flow)		27.6	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		4.16	
Average Handrail Height (m)		~1	

Image Description	Looking downstream
Date	30th January 2023
Source	Site inspection undertaken for flood study

Image Description	Looking downstream
Date	30th January 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				50% AEP Event				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	100.21	35.69	5.73	5.73	0.01	1.98	N/A	1.5 (E4)
0.2	82.39	34.98	5.39	5.38	0.01	1.94	N/A	1.5 (E4)
1	64.47	34.14	4.97	4.96	0.02	1.90	N/A	1.5 (E4)
2	62.33	30.28	4.86	4.84	0.02	1.68	N/A	1.5 (E8)
5	53.02	33.95	4.71	4.67	0.03	1.89	N/A	1.5 (E5)
10	46.38	33.56	4.55	4.49	0.06	1.87	N/A	1.5 (E5)
20	42.32	33.20	4.39	4.28	0.11	1.85	N/A	3 (E1)
50	30.03	28.57	4.06	3.93	0.13	1.59	N/A	1 (E0)

[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

[(b)]Backwater affected value

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Western Freeway Pedestrian/Bike Bridge (S4)

BCC Asset ID	N/A	Tributary Name	Witton Creek Tributary A
Owner	Department of Transport & Main Roads	AMTD (m)	30
Year of Construction	1999	Coordinates (GDA94)	496363.16, 6957429.68
Year of Significant Modification	Unknown	Hydraulic Model ID	WittonRd_Pedestrian_Bridge
Source of Structure Information	Plan No. 274506	Flood Model Representation	2d lfcsh
Link to Data Source	..\Flood Management\Data\Structures\S4 – Pedestrian Bridge		

Structure Description		Single span concrete bridge	
Bridges		**Culverts**	
Number of Spans	1	Number of Barrels	N/A
Number of Piers in Waterway	0	Dimensions (m)	N/A
Pier shape and Width (m)	0	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	1.02	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		4	
Span Length (m)		24	
Lowest Level of Deck Soffit (m AHD)		5.128	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		5.878	
Average Handrail Height (m)		1.201	

Image Description	Looking upstream
Date	30th January 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				< 5% AEP in Brisbane River Flooding*				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	27.25	19.10	5.76	5.76	0.00	N/A	N/A	1.5 (E4)
0.2	24.06	18.67	5.46	5.45	0.01	N/A	N/A	1.5 (E4)
1	20.29	16.83	5.20	5.19	0.01	N/A	N/A	1.5 (E4)
2	20.35	16.10	5.16	5.15	0.01	N/A	N/A	1.5 (E8)
5	18.08	16.31	5.08	5.07	0.01	N/A	N/A	1.5 (E5)
10	16.56	15.58	5.05	5.04	0.01	N/A	N/A	1.5 (E5)
20	15.09	14.96	4.90	4.90	0.01	N/A	N/A	1 (E4)
50	11.87	11.87	4.56	4.55	0.01	N/A	N/A	1 (E3)

[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

[(b)]Backwater affected value

** Structure is affected by riverine flooding of the Brisbane River*

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Witton Road Culvert (S5)

BCC Asset ID	C0196B	Tributary Name	Witton Creek
Owner	Department of Transport & Main Roads	AMTD (m)	900
Year of Construction	Unknown	Coordinates (GDA94)	496377.72, 6957458.46
Year of Significant Modification	Unknown	Hydraulic Model ID	C0196B
Source of Structure Information	BCC GIS Dataset: STORMWATER_CULVERT	Flood Model Representation	1d culvert
Link to Data Source	..\Flood Management\Data\Structures\S5 – Witton Road Culvert		

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)	3 x 1.5
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	2.664
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	2.635
Structure Length (m) (in direction of flow)		20.6	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		3.45	
Average Handrail Height (m)		~1	

Image Description	Downstream side of culvert
Date	30th January 2023
Source	Site inspection undertaken for flood study

Image Description	Looking downstream
Date	30th January 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				50% AEP Event				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	40.95	32.49	5.96	5.79	0.17	2.41	N/A	1.5 (E4)
0.2	36.54	32.19	5.72	5.48	0.24	2.31	N/A	1.5 (E4)
1	32.94	30.27	5.52	5.23	0.28	2.24	N/A	1.5 (E2)
2	32.04	29.74	5.45	5.17	0.28	2.20	N/A	1.5 (E2)
5	30.74	28.81	5.35	5.08	0.27	2.05	N/A	1.5 (E5)
10	29.16	27.73	5.25	5.00	0.25	2.05	N/A	1.5 (E5)
20	26.68	26.41	5.13	4.91	0.23	1.96	N/A	1 (E4)
50	18.33	18.33	4.65	4.54	0.11	1.36	N/A	1 (E3)

[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

[(b)]Backwater affected value

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Western Freeway Culvert, near Witton Road (S6)

BCC Asset ID	C3022B	Tributary Name	Witton Creek
Owner	Department of Transport & Main Roads	AMTD (m)	972
Year of Construction	Unknown	Coordinates (GDA94)	496357.06, 6957520.21
Year of Significant Modification	Unknown	Hydraulic Model ID	C3022B
Source of Structure Information	BCC GIS Dataset: STORMWATER_CULVERT	Flood Model Representation	1d culvert
Link to Data Source	..\Flood Management\Data\Structures\S6 – Western Freeway Culvert		

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)	3 x 3
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	3.59
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	2.728
Structure Length (m) (in direction of flow)		79.5	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		12.7	
Average Handrail Height (m)		N/A	

Image Description	Looking upstream
Date	30th January 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				0.05% AEP event				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	46.32	46.32	6.18	6.01	0.16	2.78	N/A	1.5 (E2)
0.2	37.39	38.65	5.89	5.77	0.12	2.84	N/A	1.5 (E2)
1	33.16	33.16	5.66	5.54	0.11	2.94	N/A	1.5 (E2)
2	32.25	32.25	5.59	5.48	0.11	2.95	N/A	1.5 (E2)
5	30.92	30.92	5.49	5.38	0.11	2.93	N/A	1.5 (E5)
10	29.32	29.32	5.39	5.27	0.11	2.92	N/A	1.5 (E5)
20	26.82	26.82	5.26	5.16	0.10	2.92	N/A	1 (E4)
50	18.27	18.27	4.80	4.67	0.13	2.81	N/A	1 (E3)

[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*

[(b)] *Backwater affected value*

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Moggill Road Culvert (S7)

BCC Asset ID	C3142P	Tributary Name	Witton Creek
Owner	Department of Transport & Main Roads	AMTD (m)	1256
Year of Construction	1963	Coordinates (GDA94)	496274.89, 6957783.53
Year of Significant Modification	Unknown	Hydraulic Model ID	C3142P
Source of Structure Information	BCC GIS Dataset STORMWATER_CULVERT	Flood Model Representation	1d culvert
Link to Data Source	..\Flood Management\Data\Structures\S7 – Moggill Road Culvert		

Structure Description		Concrete pipe culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	4
Number of Piers in Waterway	N/A	Dimensions (m)	1.5
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	6.572
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	5.138
Structure Length (m) (in direction of flow)		24	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		8.87	
Average Handrail Height (m)		N/A	

Image Description	Looking upstream
Date	30th January 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				10% AEP event				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	69.80	31.85	9.92	8.17	1.75	4.51	N/A	0.75 (E2)
0.2	62.00	31.79	9.78	8.01	1.77	4.50	N/A	0.75 (E2)
1	43.53	31.19	9.56	7.86	1.71	4.42	N/A	1.5 (E2)
2	39.45	30.79	9.47	7.81	1.66	4.36	N/A	1.5 (E2)
5	34.32	29.96	9.33	7.75	1.57	4.24	N/A	0.75 (E5)
10	29.84	28.74	9.14	7.69	1.45	4.07	N/A	0.75 (E4)
20	26.00	25.98	8.77	7.55	1.22	3.68	N/A	1 (E4)
50	17.69	17.70	8.27	7.08	1.20	2.56	N/A	1 (E3)

[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

[(b)]Backwater affected value

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Western Freeway On-Ramp, off Moggill Road (S8)

BCC Asset ID	C3024P	Tributary Name	Witton Creek
Owner	Department of Transport & Main Roads	AMTD (m)	1428
Year of Construction	Unknown	Coordinates (GDA94)	496259.61, 6957929.24
Year of Significant Modification	Unknown	Hydraulic Model ID	C3024P
Source of Structure Information	BCC GIS Dataset: STORMWATER_CULVERT	Flood Model Representation	1d culvert
Link to Data Source	..\Flood Management\Data\Structures\S8 – Western Freeway On-Ramp Culvert		

Structure Description		Concrete circular culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	4
Number of Piers in Waterway	N/A	Dimensions (m)	1.95
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	8.38
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	7.94
Structure Length (m) (in direction of flow)		30.175	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		14.9	
Average Handrail Height (m)		N/A	

Image Description	Western Freeway Culvert
Date	Unknown
Source	Department of Transport & Main Roads, Hydraulics, Design and Spatial Engineering & Technology Branch

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)			0.05% AEP event					
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	49.02	48.60	11.94	10.80	1.14	4.06	N/A	0.75 (E2)
0.2	40.86	40.86	11.42	10.60	0.82	3.42	N/A	0.75 (E2)
1	31.67	31.67	10.84	10.35	0.50	2.65	N/A	1.5 (E2)
2	28.74	28.74	10.67	10.26	0.41	2.41	N/A	1.5 (E2)
5	25.25	25.25	10.46	10.15	0.31	2.11	N/A	0.75 (E5)
10	22.28	22.28	10.27	10.03	0.24	1.87	N/A	0.75 (E4)
20	18.94	18.94	10.06	9.89	0.17	1.71	N/A	1 (E4)
50	12.79	12.79	9.69	9.60	0.09	1.62	N/A	1 (E3)

[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

[(b)]Backwater affected value

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Western Freeway Culvert, near Moggill Road (S9)

BCC Asset ID	C3020P	Tributary Name	Witton Creek
Owner	Department of Transport & Main Roads	AMTD (m)	1500
Year of Construction	Unknown	Coordinates (GDA94)	496330.09, 6957953.04
Year of Significant Modification	Unknown	Hydraulic Model ID	C3020P
Source of Structure Information	BCC GIS Dataset: STORMWATER_CULVERT	Flood Model Representation	1d culvert
Link to Data Source	..\Flood Management\Data\Structures\S9 – Western Freeway Culvert		

Structure Description		Steel circular culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	3
Number of Piers in Waterway	N/A	Dimensions (m)	1.95
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	10.54
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	9.5
Structure Length (m) (in direction of flow)		39.864	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		20.59	
Average Handrail Height (m)		N/A	

Image Description	Western Freeway Culvert
Date	Unknown
Source	Department of Transport & Main Roads, Hydraulics, Design and Spatial Engineering & Technology Branch

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				0.05% AEP event				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	49.51	49.51	14.77	11.98	2.79	5.53	N/A	1.5 (E2)
0.2	40.89	40.89	13.77	11.43	2.34	4.61	N/A	1.5 (E2)
1	31.66	31.66	13.09	10.84	2.24	3.76	N/A	1.5 (E2)
2	28.73	28.73	12.93	10.67	2.26	3.76	N/A	1.5 (E2)
5	25.44	25.44	12.74	10.45	2.29	3.77	N/A	2 (E7)
10	22.33	22.33	12.56	10.27	2.29	3.77	N/A	2(E7)
20	19.00	19.00	12.35	10.06	2.29	3.77	N/A	1 (E3)
50	12.77	12.77	11.98	9.69	2.29	3.57	N/A	1 (E3)

[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*

[(b)]*Backwater affected value*

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Russell Terrace Culvert (S10)

BCC Asset ID	C0184P	Tributary Name	Witton Creek
Owner	Department of Transport & Main Roads	AMTD (m)	1790
Year of Construction	1962	Coordinates (GDA94)	496428.908, 6958152.638
Year of Significant Modification	Unknown	Hydraulic Model ID	C0184P
Source of Structure Information	BCC GIS Dataset: STORMWATER_CULVERT	Flood Model Representation	1d culvert
Link to Data Source	..\Flood Management\Data\Structures\S10 – Russel Terrace Culvert		

Structure Description		Concrete pipe culverts	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	3
Number of Piers in Waterway	N/A	Dimensions (m)	1.65
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	11.69
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	11.681
Structure Length (m) (in direction of flow)		23	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		14.65	
Average Handrail Height (m)		N/A	

Image Description	Upstream side of culvert
Date	30th January 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				10% AEP event				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	51.97	24.88	15.45	14.89	0.56	3.88	N/A	0.75 (E2)
0.2	40.18	24.78	15.25	14.31	0.94	3.87	N/A	0.75 (E2)
1	29.09	24.54	15.02	14.03	0.99	3.83	N/A	0.75 (E2)
2	26.38	24.41	14.95	13.97	0.99	3.81	N/A	0.75 (E2)
5	24.43	23.39	14.77	13.86	0.90	3.65	N/A	0.75 (E2)
10	21.57	21.57	14.56	13.79	0.77	3.36	N/A	0.75 (E5)
20	19.39	19.39	14.33	13.71	0.62	3.03	N/A	1 (E5)
50	11.78	11.78	13.61	13.38	0.23	1.84	N/A	1 (E5)

[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*

[(b)]*Backwater affected value*

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Moore Park Parking Lot Culvert, near Russell Terrace (S11)

BCC Asset ID	C0378B	Tributary Name	Witton Creek
Owner	Brisbane City Council	AMTD (m)	1805
Year of Construction	Unknown	Coordinates (GDA94)	496424.533, 6958170.050
Year of Significant Modification	Unknown	Hydraulic Model ID	C0378B
Source of Structure Information	BCC GIS Dataset: STORMWATER_CULVERT	Flood Model Representation	1d culvert
Link to Data Source	..\Flood Management\Data\Structures\S11 – Moore Park Parking Lot 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)	1.2 x 0.6
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	13.94
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	13.51
Structure Length (m) (in direction of flow)		9	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		14.5	
Average Handrail Height (m)		N/A	

Image Description	Upstream side of culvert
Date	4th October 2023
Source	Site inspection undertaken for flood study

Image Description	Downstream side of culvert
Date	4th October 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				5% AEP event				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	4.24	0.76	15.41	15.40	0.01	1.91	N/A	0.5 (E1)
0.2	3.69	0.76	15.30	15.29	0.01	1.91	N/A	0.5 (E1)
1	2.67	0.70	15.04	15.03	0.01	1.85	N/A	0.5 (E1)
2	2.36	0.68	14.96	14.95	0.01	1.83	N/A	0.5 (E1)
5	2.15	0.64	14.69	14.68	0.02	1.80	N/A	0.5 (E7)
10	1.90	0.60	14.42	14.37	0.05	1.76	N/A	0.5 (E7)
20	1.62	0.56	14.39	14.01	0.39	1.72	N/A	0.5 (E4)
50	1.16	0.46	14.34	13.71	0.63	1.61	N/A	0.5 (E4)

[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

[(b)]Backwater affected value

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Western Freeway Bridge over Witton Road (S12)

BCC Asset ID	N/A	Tributary Name	Witton Tributary A
Owner	Department of Transport & Main Roads	AMTD (m)	100
Year of Construction	Unknown	Coordinates (GDA94)	496332.984, 6957466.375
Year of Significant Modification	Unknown	Hydraulic Model ID	N/A – DEM
Source of Structure Information	As con. Plan no. 274515	Flood Model Representation	N/A – DEM
Link to Data Source	..\Flood Management\Data\Structures\S12 – Western Freeway Bridge over Road		

Structure Description		Single span concrete bridge	
Bridges		**Culverts**	
Number of Spans	1	Number of Barrels	N/A
Number of Piers in Waterway	0	Dimensions (m)	N/A
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	2.9	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		19.5	
Span Length (m)		23	
Lowest Level of Deck Soffit (m AHD)		8.986	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		10.52	
Average Handrail Height (m)		1.3	

Image Description	Upstream side of Western Freeway Bridge
Date	4th October 2023
Source	Site inspection undertaken for flood study

Image Description	Downstream side of Western Freeway Bridge
Date	4th October 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				0.05% AEP event				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	32.72	32.72	6.59	5.81	0.78	N/A	N/A	1.5 (E4)
0.2	24.06	24.06	6.413	5.702	0.71	N/A	N/A	1.5 (E4)
1	12.17	12.17	6.11	5.53	0.58	N/A	N/A	1.5 (E4)
2	11.65	11.65	6.00	5.47	0.53	N/A	N/A	1.5 (E8)
5	4.24	4.24	5.83	5.37	0.46	N/A	N/A	1.5 (E5)
10	0.91	0.91	5.63	5.28	0.35	N/A	N/A	1.5 (E5)
20	0.38	0.38	5.51	5.23	0.27	N/A	N/A	1 (E4)
50	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1 (E3)

[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

[(b)]Backwater affected value

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Western Freeway Bridge over Tributary A, near Witton Road (S13)

BCC Asset ID	N/A	Tributary Name	Witton Tributary A
Owner	Department of Transport & Main Roads	AMTD (m)	100
Year of Construction	1999	Coordinates (GDA94)	496302.58, 6957412.12
Year of Significant Modification	Unknown	Hydraulic Model ID	N/A – not modelled
Source of Structure Information	Plan no. 274515	Flood Model Representation	N/A – not modelled
Link to Data Source	..\Flood Management\Data\Structures\S13 – Western Freeway Bridge Over Trib A		

Structure Description		Single span concrete bridge	
Bridges		**Culverts**	
Number of Spans	1	Number of Barrels	N/A
Number of Piers in Waterway	0	Dimensions (m)	N/A
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	N/A
Bridge Invert Level (m AHD)	2.9	Downstream Invert (m AHD)	N/A
Structure Length (m) (in direction of flow)		19.5	
Span Length (m)		23	
Lowest Level of Deck Soffit (m AHD)		8.986	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		10.52	
Average Handrail Height (m)		1.3	

Image Description	Upstream side of Western Freeway Bridge
Date	30th January 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				0.05% AEP event				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	28.30	28.30	5.75	5.74	0.00	N/A	N/A	0.5 (E1)
0.2	24.68	24.68	5.45	5.44	0.01	N/A	N/A	0.5 (E1)
1	21.23	21.23	5.20	5.20	0.00	N/A	N/A	1.5 (E4)
2	20.20	20.20	5.16	5.15	0.00	N/A	N/A	1.5 (E8)
5	17.94	17.94	5.09	5.08	0.01	N/A	N/A	1.5 (E5)
10	16.46	16.46	5.02	5.01	0.01	N/A	N/A	1.5 (E5)
20	15.01	15.01	4.95	4.93	0.02	N/A	N/A	1 (E4)
50	11.66	11.66	4.63	4.61	0.03	N/A	N/A	1 (E3)

[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

[(b)]Backwater affected value

Hydraulic Structure Reference Sheet

Witton Creek Flood Study

Kennewell Park Culvert (S14)

BCC Asset ID	Unknown	Tributary Name	Witton Tributary A
Owner	Brisbane City Council	AMTD (m)	431
Year of Construction	Unknown	Coordinates (GDA94)	496013.5739, 6957419.5700
Year of Significant Modification	Unknown	Hydraulic Model ID	CW8_01
Source of Structure Information	2023 BCC Field Survey	Flood Model Representation	1d culvert
Link to Data Source	..\Flood Management\Data\Structures\S14 – Kennewell Park Culvert		

Structure Description		Circular culvert	
Bridges		**Culverts**	
Number of Spans	N/A	Number of Barrels	1
Number of Piers in Waterway	N/A	Dimensions (m)	1.7
Pier shape and Width (m)	N/A	Upstream Invert (m AHD)	5.87
Bridge Invert Level (m AHD)	N/A	Downstream Invert (m AHD)	5.73
Structure Length (m) (in direction of flow)		7.48	
Span Length (m)		N/A	
Lowest Level of Deck Soffit (m AHD)		N/A	
Overtopping Level of Weir/Road (m AHD) (not including handrail)		7.8	
Average Handrail Height (m)		~1	

Image Description	Upstream of culvert
Date	4th October 2023
Source	Site inspection undertaken for flood study

Image Description	Downstream side of culvert
Date	4th October 2023
Source	Site inspection undertaken for flood study

Link to Flood Model Results	..\Flood Management\Tuflow\results\D006\ ..\Flood Management\Tuflow\results\DX02\
Model Version Number	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf
Model Scenario	Scenario 1 (D006) /Scenario 1 Extreme Events (DX02)

Structure Flood Immunity (immunity of lowest point of weir above structure)				10% AEP event				
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 (m)[3]	Structure Velocity (m/s)[4&6]	Weir Velocity (m/s)[5&6]	Critical Storm Duration (hrs)[7] & Ensemble
0.05	13.34	6.71	8.13	6.58	1.55	3.41	N/A	0.5 (E6)
0.2	11.52	6.51	8.09	6.57	1.53	3.39	N/A	0.5 (E1)
1	9.73	6.23	8.04	6.57	1.47	3.41	N/A	0.5 (E1)
2	9.20	6.23	8.02	6.56	1.46	3.41	N/A	0.5 (E1)
5	8.51	6.22	7.98	6.56	1.42	3.42	N/A	0.75 (E2)
10	7.85	6.22	7.97	6.56	1.41	3.42	N/A	0.75 (E4)
20	6.76	6.11	7.88	6.57	1.31	3.40	N/A	0.5 (E4)
50	5.08	5.08	7.64	6.56	1.09	3.16	N/A	0.5 (E4)

[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*

[(b)]*Backwater affected value*

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Appendix L: External Peer Review Documentation

Member of the Surbana Jurong Group

Memorandum

To	Hanieh Zolfaghari	Date	17 November 2023
From	Atena Roshani, Project Manager	No. of pages	7
Authors	Monte Azmi, Evan O'Brien, Atena Roshani		
Reference	30032794 – TM01- Rev B		
Subject	Witton Creek Flood Study Review		

1. Introduction

SMEC has been commissioned by Brisbane City Council (BCC) to carry out a review of the flood study undertaken by Arup for Witton Creek catchment.

To reflect the current conditions of the catchment and best practice flood modelling techniques, BCC has engaged Arup to update the existing flood study for Witton Creek. The most recent study undertaken for the Witton Creek was the Stormwater Management Plan completed by Water & Environment City Design, Brisbane City Council in 2000.

Witton Creek Catchment has a total area of 4.09 km^2 and the catchment centroid is located approximately 7.5km south-west of Brisbane CBD. The headwaters of Witton Creek are located within the Mount Cooth-tha and Brisbane Forest Park Bushland reserve areas. Witton Creek ultimately drains into the Brisbane River. The major creeks and tributaries within the catchment are Witton Creek, Witton Creek Tributary A, Witton Creek Tributary B and Witton Creek Tributary C.

This memorandum summarises the SMEC review of the model build approach and the calibration of the hydrologic and hydraulic models of the catchment, and the estimated design and rare and extreme flood events in accordance with Australian Rainfall and Runoff 2019 guidelines (ARR2019).

1.1 Review Guidelines

The following guidelines were used for this assessment:

- Australian Rainfall & Runoff (ARR) 2019
- URBS User Manual (2019)
- TUFLOW User Manual (2018)
- Brisbane City Council - Guide for Flood Studies and Mapping in Queensland (2017)
- Queensland Urban Development Manual (QUDM) (2019)

2. Calibration Performance

As part of the Witton Creek Flood study (2023), the followings stages were completed:

- The flood models were calibrated and verified by Arup against historical storm events. The results were presented by Arup in a meeting held on 17 April 2023 including a technical memorandum.
- The provided models (hydrologic and hydraulic) and technical memorandum have been reviewed by SMEC.

2.1 Calibration Methodology

The approach was based on event-based calibration (using rainfall station 540465) of four historical events (February 2020, May 2015, March 2017, June 2016 and January 2013). Details of events' characteristics are elaborated in Sections 3.4.7 to 3.4.11.

Briefly, the hydrological model (URBS) was calibrated, and then the hydraulic model (TUFLOW) was verified to ascertain the consistency between hydrological and hydraulic models. The details of the methodology steps are available in Section 4.6.2 of "Witton Creek Flood Study" (ARUP 2023).

2.2 URBS Model Calibration

Hydrologic modelling for the catchment was undertaken in URBS (version 6.62) software for the purpose of providing inflows into the hydraulic TUFLOW models guided by Australian Rainfall and Runoff 2019 guidelines (ARR2019).

Table 1: URBS model setup

Model Component	Witton Creek
Sub-Catchment delineation	39 sub-catchments
Runoff Routing Model	"Split" model
Land use and Impervious Fraction	Spatially-varying, based on aerial imagery
Streamlines	Mapped from LiDAR
Losses (initial, continuing)	18 mm, 1.4 mm/h (pervious areas) 0 mm, 0 mm/h (impervious areas)

2.2.1 URBS - Calibration Review by SMEC

Comment 1: Hydrologic Model Build

The model build process is clearly described. The authors have followed relevant local and industry guidelines in constructing the URBS models.

Comment 2: Recorded Rainfall Spatial Distribution

Whilst an inverse distance-weighted average approach would be desirable, the use of two rainfall gauges limits the methodology to the use of Thiessen polygons. Perhaps consideration could have been given to filling out the rainfall network with some daily gauge data (eg. Brisbane Botanic Gardens 040976, or even some of the recently closed gauges such as Toowong Bowls Club 040245), which would have enabled the use of the URBS SUBRAIN routine.

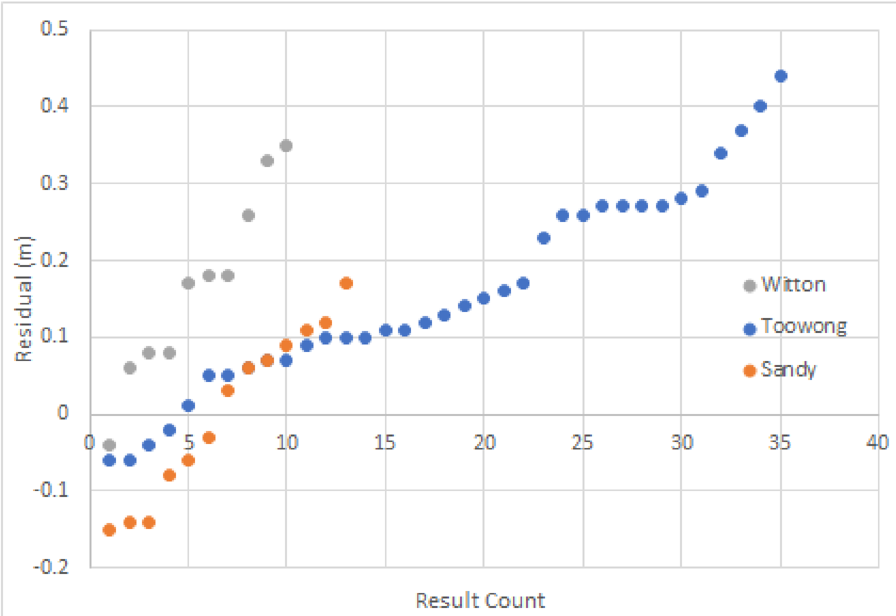
In any case, the effects of spatial variability may not be as important in these small catchments as they are in large river basin models, depending on the duration and type of rainfall event being analysed.

Comment 3: Hydrologic Model Calibration Results

The adopted URBS parameters fall within the expected range and the adoption of a single set of parameters for alpha, beta, m and CL across all events and models should be considered a good result.

Comment 4: Hydraulic Calibration Results

This section provides a detailed and well-considered discussion of the calibration and validation results. Modelled results generally match quite well to their observed counterparts, having regard for the suspected inconsistencies in the data, which are noted in the report, and the limitation of being able to match to maximum heights only. One item of interest was that the residual error (ie. modelled minus observed) tended to skew strongly positive for both the Witton Creek model:



Tailwater sensitivity results notwithstanding, was any likely reason found for this outcome? Could it be that the sparse pluviograph network has over-represented total excess rainfall for these storms (implying greater spatial variability than the gauges suggest)? Could a slightly higher continuing loss value be justified? Overall, though, the results err on the side of conservatism, which is probably warranted from the perspective of urban flood risk management.

2.2.2 Digital Model Files

A random selection of model files (catchment data, vector, rainfall input) were briefly reviewed and found to match the scenarios and results reported in the technical note.

2.2.3 Concluding Remarks

The report is thorough and well-written, and it is evident that significant effort was spent (eg. ground-truthing overland flow paths, removing erroneous terrain data etc) to ensure that the models represent the catchment dynamics as realistically as possible. It is the reviewer's opinion that the models are suitable for use in developing design flood estimates, and any associated planning and design tasks that may arise therefrom.

2.3 TUFLOW Model Calibration

Hydraulic modelling for the catchment was undertaken in TUFLOW (version 2020-10-AF1). The TUFLOW model consist of 1D/2D linked schematisation with the 1D domain modelled in ESTRY and the 2D domain in TUFLOW.

Table 2: TUFLOW model setup

Model Component	Witton Creek
Model Extent	Covering the subcatchments and major waterways. The area is extended enough to avoid any glass wall.
Terrain Data	1m 2019 BCC LiDAR (ALS), 2023 Witton Creek Filed Survey (18 cross-section)
Cell size	a 2 m grid model with sub-grid sampling (SGS) of 1 m
Land use and Manning's number	Shapefiles were created based on "BCC City Plan 2014" and "Aerial photography – 2009 to 2021". Nearmap 2022 Building Footprint layer provided by BCC. Please refer to Table 5.3 of the "Witton Creek Flood Study" for the landuse categories and assigned.
Hydraulic structure	BCC drawings were mainly used for building hydraulic structures. Please refer to "Refer to Appendix J" of the "Witton Creek Flood Study" (ARUP 2023). As a sanity check, 2000 Witton SMP MIKE 11 model structure information was also considered. Please refer to Table 5.4 of the "Witton Creek Flood Study" for more details.

Model Component	Witton Creek
Boundary conditions	Inflow: QT type adopted from URBS model. 2d_sa and 1d_bc were used for 2d domains and drainage respectively. For waterways, “streamlines” option was activated to distribute the flow throughout the waterways. Downstream: A varying water level versus time (H-T) boundary was used to represent the downstream boundary condition at the mouth of Witton Creek for calibration and verification. For design conditions, a fixed tidal boundary was used at the downstream model extent with an allowance of 0.8 m for projected climate variability effects.
Bridge Form Loss Coefficient	Method C. More details in Section 5.2.6
Run parameters	Heavily Parallelised Compute (HPC) was adopted as TUFLOW solver. 0.5 second (> 1/10 of cell size) was set for 1D ESTRY. As a default in 2020 version, Wu method has been adopted.

2.3.1 TUFLOW - Calibration Review by SMEC

In the initial version of model, below major issues were identified:

- SGS was not applied,
- cell alignments needed adjustments
- Loss applied to bridges were not in accordance with TUFLOW best practice
- Underground drainage had issues (invert levels, connections, slopes)
- Remaining “Warnings” to be addressed

At the end of this stage, ARUP addressed all major issues in TUFLOW; and model/outcomes were satisfactory.

3. Design Events

As part of the Arup 2023 study, the following stages are completed for the Witton Creek Flood Study (2023):

- The design and rare / extreme flood magnitudes were estimated by Arup in accordance with Australian Rainfall and Runoff 2019 guidelines (AR&R2019). SMEC reviewed the estimated design events in accordance with the AR&R 2019 guidelines
- Flood levels for the design and rare /extreme events were estimated by Arup and reviewed by SMEC.
- The impacts of the Minimum Riparian Corridor (MRC) and filling/development outside the “Modelled Flood Corridor” were quantified by Arup and reviewed by SMEC.
- SMEC review has been limited to Scenario 3, page 45-46 of “Witton Creek Flood Study” by ARUP. The methodology and assumptions for hydraulic modelling (Manning and topography adjustments) were reasonable. Checking water levels showed increases in compared to base case (Existing) which was reasonable and expectable.
- The Climate Change sensitivity analysis was undertaken by Arup and was reviewed by SMEC in accordance with the ARR2019 guidelines.

3.1 URBS Model - Design Events review by SMEC

A comprehensive cross-check is considered beyond the scope of this review, however a random selection of model files (catchment data, vector, rainfall) was briefly reviewed and found to match the catchment details and events described in the report.

3.2 TUFLOW Model - Design Events review by SMEC

SMEC has reviewed the TUFLOW model for hydraulic components, exactly similar to calibration stage to ensure they are in accordance with guidelines. To be specific, the review of Tuflow modelling for design case was focused on:

1. Review of responses to comments in calibration stage

2. Inflow for design conditions (based on design rainfall from URSB) are properly applied in the model, and
3. post processing of simulations for all AEPs from frequent to PMF based on envelope approach
4. Scenario 3: the methodology and assumptions for hydraulic modelling (Manning and topography adjustments) were reasonable. Checking water levels showed increases in compared to base case (Existing) which was reasonable and expectable.

Details of the review, comments and responses are attached to this report.

4. Technical Report – Review

4.1 Hydrology Sections

Comment1: Section 3.4.6 Characteristics of Historical Events

Plotting observed rainfall accumulations against the design IFD data is a useful way to add context to historic rainfall events.

Comment 2: Section 4.0 Hydrologic Model Development and Calibration

The model has been built in accordance with generally accepted principles and guidelines.

Comment 3: Section 5.7 Hydrologic-Hydraulic Model Consistency Checks", "6.4.5 Hydrologic Hydraulic Model Consistency Check (Design Events)

The comparative plots shown in these sections are an indication that the joint hydrologic-hydraulic calibration process has been successful. The close agreement between URBS and TUFLOW results also shows that there is no double-routing, and a sound explanation is offered for the Radnor Street location where divergences were noted.

Comment 4: Section 6.1 Design Event Scenarios"

Most likely a minor typo. The first paragraph of this section makes reference to the design scenarios; however, results are only presented for Scenario 1 and Scenario 3 (ie. 2 scenarios).

4.1.1 Concluding Remarks

The report is well-conceived, and it is the reviewer's opinion that it meets the requirements of the scope as outlined in report Section 1.3.

4.2 Hydraulic Sections

Comment 1

Page 2 – As a result of TUFLOW model review, multiple anomalies in drainage system was identified, not fully addressed due to the lack a comprehensive feature survey. Therefore, it is required to clearly mention the limitation with the accuracy of the underground drainage system database, and its potential impact on flood outcomes. A recommendation is required to review the database of the council for future studies.

Comment 2

Scenarios 2 and 3 have been combined to see the impact of ultimate changes along with development on the edge of the creek, while it has been labelling as Scenario 3. Jumping from Scenario 1 to 3 without enough explanation is confusing for readers.

Comment 3

Page 32 – As per previous agreement, bridge losses were calculated and applied in TUFLOW based on METHOD C. Having said that, in the report, the HEC-RAS model has been introduced for the verification. *It critical to note that SMEC has not reviewed HEC-RAS model therefore the model and its outcomes were excluded from this review process.*

Comment 4

Appendix I – In many instances, critical durations have been unchanged or even become longer when events moved from frequent to rare. The usual sense is from frequent to rare/extreme; the critical duration should be shorter (flash flood).

There is no clear elaboration regarding the initial threshold of 30min to 6hrs duration for extracting critical durations. Simulations must cover the full spectrum from 10min to long durations (72hrs). Currently there are instances with 30min critical durations (extreme events), which could be shorter (e.g. 20min) if we had started the durations threshold from 10min.

4.3 Review Summary

SMEC undertook a Peer Review of the hydrologic and hydraulic models associated with the Witton Creek Flood Study in line with BCC project brief requirements. Overall, the models and methodology were found to be sound and in line with current best industry practices. After a few rounds of comments and revisions, all comments are addressed and closed.

In preparing this technical note, SMEC has relied upon and presumed accurate, information provided by BCC and Arup. Except as otherwise stated in this technical note, SMEC has not attempted to verify the accuracy or completeness of any such information. If the information is subsequently determined to be false, inaccurate, or incomplete then it is possible that our observations and conclusions as expressed in this report may change.

SUMMARY OF TUFLOW MODEL CHECKS

FILE LOCATIONS

Design report:	<Insert link to consultant's study report associated with modelling>	**QA filled by:**		Monte Azmi
Model files:	<Insert link to raw model data supplied by consultant> <Insert link to modified or copied model data used as part of review>	**QA checked by (Senior Engineer):**		
		Third Party Consultant, checked by: (if applicable)	SMEC	28/04/2023
Review workspace:	<Insert link to GIS based review workspace>	**MW Project Manager:**		

Item ID	Item	TUFLOW Solver	Ref: AM STA 6200	Is current setup appropriate? (Yes, No, N/A)	Severity of Issue (From Intro tab)	Feature updated for current modelling application (Yes or No)	Reviewer Comments	Originator Response BCC ARUP: 8/05/2023	Reviewer Comments	Reviewer Status	Reviewer Comments on Design Events	Originator Response	Reviewer Status
Model													
1.1	If utilising an existing model, though using more recent release, check change in tuflow version does not alter result. If it does revert back to prior release default setting via the command: Set defaults == pre [year]. Refer to Tuflow Wiki for further guidance	ALL		N/A	1	N/A, no prior model available							
1.2	Tuflow executable version:	ALL		Yes	1	N/A, no prior model available	2020-10-AF\TUFLOW_iSP_w64.exe						
1.3	Model simulation version (e.g. MB_EXG_02hr_01AEP_**045**.tcf):	ALL		Yes	1	N/A, no prior model available	WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_001.tcf						
1.4	Are event (~e~) and or scenario (~s~) logic commands used? If yes, list available options.	ALL		Yes	1	N/A, no prior model available							
1.5	Does model simulation run to completion?	ALL		Yes	1	N/A, no prior model available	from provided tlf files yes.						
Terrain Representation (2D Domain)													
2.1	Is the cell (grid) size appropriate for the intended purpose of the modelling?	ALL		No	3	N/A, no prior model available	2m, usually fine for urban area. However, when 1m LiDAR is available and model does not have significant extent, it is not reasonable to apply cell size courser than DEM resolution. Or if modeller applies courser cell size, instead SGS or quadtree must be applied to keep the accuracy	ARUP: 8/05/2023 2m grid size is appropriate for urban flood modelling and was selected based on a review of model run times, calibration iterations required and the pickup of feature definition across the catchment. Due to the number of calibration iterations and long run times, 1m grid size and SGS was not selected for calibration phase. Testing has been undertaken where SGS with 2m grid with 1m sampling is switch on for all Calibration and Validation events. This was shown to help fine-tune calibration results. It is proposed to used SGS with 1m sampling to be used in design runs for Witton model.	2m grid with 1m SGS is acceptable	Closed			Closed
2.2	Is the cell size is smaller than water depth in main channel/flow path of the subject study? If yes, Wu sub-grid turbulence scheme (i.e. 2020 TUFLOW HPC or newer) must be used? Under these hydraulic conditions Smagorinsky or Constant sub-grid turbulence schemes are not appropriate.	TUFLOW HPC		No	2	N/A, no prior model available	Refere to above. Even if 1m cell size is applied, for desing scenarios (e.g 1%AEP+CC) watre depth inside the main flowpath will likely be more than 1m therefore again SGS will be required.	ARUP: 8/05/2023 2m grid size appropriate, model is using WU scheme per HPC default. See above response for selection of 2m grid and use of SGS.	2m grid with 1m SGS is acceptable	Closed			Closed
2.3	Is variable cell size is used (Quadtree)? If yes, Wu sub-grid turbulence scheme (i.e. 2020 TUFLOW HPC or newer) must be used? Wu is a cell size independent scheme. Smagorinsky or Constant sub-grid turbulence schemes are not cell size independent.	TUFLOW HPC		N/A	1	N/A, no prior model available							
2.4	Is the model grid orientation appropriate?	ALL		No	3	N/A, no prior model available	line is neither pararalel to main upstreamstreams or downstream streamline	ARUP: 8/05/2023 Parallel to the main Witton creek open channel parallel to Witton road (see Tab 2). There is no ideal orientation across the Witton creek model due to the meandering nature of the creek.		Closed			Closed
2.5	Are applied terrain modification layers working as intended?	ALL		Yes	1	N/A, no prior model available							
2.6	Are all important hydraulic controls being represented using topography modifiers?	ALL		Yes	1	N/A, no prior model available							
2.7	Is sub-grid sampling (SGS) used as the topography sampling method	TUFLOW HPC		No	1	N/A, no prior model available	please refer to comments 2.1 and 2.2	ARUP: 8/05/2023 Refer to Comment 2.1.		Closed			Closed
2.8	If SGS is used, are additional ridge/max breaklines required to enforce raised hydraulic controls	TUFLOW HPC		N/A	1	N/A, no prior model available							
2.9	If SGS is used, are unnecessary gully/min breaklines removed (the features are already implicitly represented by the SGS scheme)?	TUFLOW HPC		N/A	1	N/A, no prior model available							
2.10	Are the manning's values appropriate?	ALL		Yes	1	N/A, no prior model available							
2.11	Is the default Material value correct?	ALL		Yes	1	N/A, no prior model available							
2.12	Is / are the Materials Layer(s) delineation reasonable relative to the model cell size?	ALL		Yes	1	N/A, no prior model available							
2.13	If multiple material input layers are used, is data layering of the Materials Layer(s) correct (i.e. The order of the files with the TUFLOW Geometry Control File)? Note, bottom most layer takes precedence where datasets overlap.	ALL		Yes	1	N/A, no prior model available							
Drainage Network Representation (1D Domain)								Comments BCC					
3.01	Are the pipe/channel alignments correct?	ALL		No	3	N/A, no prior model available	There are single pit connected with two outlet pipe. Unusual construction. Please check entire network for similar cases. And confirm if this has been received from council and they are happy with this information. Please refer to TAB1	TAB1 CHECKED ARUP: 8/05/2023 Pipe configuration has been developed using BCC network information (provided by BCC as either GIS, drawings, field survey etc). Review of the model network has been completed. For the location identified, BCC GIS information was used. Additional snippets of BCC council information provided in Tab 1.	Since ARUP confirms that network review has been completed with a satisfaction level with provided information from BCC, this comment can be closed	Closed			Closed
3.02	Are pipes connected throughout system (any snapping issues)?	ALL		Yes	1	N/A, no prior model available							
3.03	Is network free of grade or cover issues?	ALL		Yes	2	N/A, no prior model available	N11847280 has reverse slope	ARUP: 8/05/2023 Noted and has been updated fixed for design runs. Located 600m upstream outlet to creek and will have minimal impact to levels downstream.		Closed			Closed
3.04	Do drainage network asset sizes logical (i.e. increase as move down system)?	ALL		No	3	N/A, no prior model available	Below locations with sizes decreased: N11069850 - According to BCC data the DS pipe is 'UNKOWN' N11166490 - This is according to BCC data N11847237 - Not according to BCC data N11847247 - Not according to BCC data N11847285 - Not according to BCC data N11847298 - Not according to BCC data O11000248 - This is according to BCC data	Checked ARUP: 8/05/2023 Pipe configuration has been developed using BCC network information (provided by BCC as either GIS, drawings, field survey etc). Review of the model network has been completed. Locations noted have been reviewed and are consistent with BCC council information provided. Additional snippets of BCC council information for locations are provided in Tab 3.	Since ARUP confirms that network review has been completed with a satisfaction level with provided information from BCC, this comment can be closed	Closed			Closed
3.05	Are pipe lengths defined properly?	ALL		Yes	1	N/A, no prior model available							
3.06	Are pipe manning's n appropriate?	ALL		No	2	N/A, no prior model available	currently n=0.013. For old pipes 0.015 is a typical number. For existing 0.015 is much more appropriate. Curent number overestimating the pipe netwrok capacity. For ARC bridge, how manning 0.024 calculated? Based on combined manning equation? Details to be provided	ARUP: 8/05/2023 Manning's n of 0.013 is appropriate for older RCP pipes, and was selected for the model from a review of a range of literature and industry guidelines: - From ARR2019 - Table 6.2.3 (Book 6, Chapter 2) for concrete centrifugally spun for straight clean pipes concentrically joined n = 0.009 to 0.012 - From Austroads Guide to Road Design Part 5A (2013a) - Road Surface, Networks, Basin and Subsurface - Table 6.4 recommends Concrete pipes manning's n = 0.011 to 0.013 - From Humes/Holcim (Concrete pipe reference manual, Issue 1, 2015) - recommend n= 0.011 for new concrete pipes - From Melbourne Water Hydrologic and Hydraulic Design (Land development ref 5.3.2) - concrete pipes n =0.013 - From Hec Ras Manual (version 6.3, 2020, Table 6.1) - concrete with bends, connections and some debris - n = 0.011 to 0.014, with a normal value of n = 0.013 (Chow 1959) The pipe network within the catchment is not significantly old (<=50yrs) for an increased manning's of n =0.015 Arc Bridge is a corrugated lined channel - see photo in Tab 4. Manning's and was selected from Hec Ras Manual (Version 6.3 2020, Table 3-1)		Closed			Closed
3.07	Is the manhole loss approach appropriate?	ALL		Yes	1	N/A, no prior model available							
3.08	Is the pipe geometry orientation appropriate for Engelund losses?	ALL		Yes	1	N/A, no prior model available							
3.09	Are additional form loss pipe losses set correctly where required?	ALL		Yes	1	N/A, no prior model available							
3.10	Are contraction coefficients appropriate?	ALL		Yes	1	N/A, no prior model available							
3.11	Is pit modelling approach appropriate?	ALL		No	3	N/A, no prior model available	no spreadsheet for pit rating was found in db_database. I assumed that based on the similar naing you have used the one for Sandy. Confirm please. HEC22 ratings are inside the dataabse however all pit are based on "UNLIMTED" which is in fact a constant rate of 0.18cms. Please elaborate the reason and if this cah already been discussed and confirm by council	?? ARUP: 8/05/2023 Stormwater trunk drainage was included within the model as 1D network for all three (3) TUFLOW models. The flow interchange between the 2D domain and the 1D pipe network was assumed to occur "freely" at the inlet pits, such that the hydraulic control would be the limiting size of the pipe and not the size of the pit inlet. Accordingly 'unlimited' type pit exported from HEC 22 was assumed. Further testing has been undertaken to on pit inlet curves for the "unlimited" type pits, ramping up to 2m3/s. No appreciable difference although marginal improvement to calibration levels generally. It is proposed that the new curve (up to 2m3/s) to be used for design runs to		Closed			Closed
3.12	Are pit loses set appropriately?	ALL		Yes	1	N/A, no prior model available							
3.13	Is the model 1D network either free from Additional Nodal Area (ANA) values, of if these have been used are the values appropriate?	ALL		Yes	1	N/A, no prior model available							
3.14	Are entry/exit losses set or are they automatically defined for pipes that have SX outlets?	ALL		Yes	1	N/A, no prior model available							
2D Bridges (Note, 2D bridges should not be used for Culvert representation unless the culvert size is greater than the cell size)													
4.01	Is the approach used to define the additional hydraulic losses associated with bridges appropriate?			No	2	N/A, no prior model available	There is blockage for Level 3 but no loss associated. The calcualtion spreadhseet was not provided to be reviewed		untill receiving calcualtions and spreadhseet it remains open	Open		as per email from ARUP including additional information, it is acceptable.	Closed
4.02	What bridge form loss calculation method has been used (Method A (cumulative), Method B(Portion), Method C, Method D). Note, Method C and D are only available from release version 2020-10-AA or newer.			Yes	1	N/A, no prior model available	Method B (Portion)	ARUP: 8/05/2023 Noted. It was assumed in Calibration that Method C was default in TUFLOW version 2020-10-AF and was accordingly kept as Default. However, further review release notes show Method B is still used as Default. Testing with Method C was done for all Calibration Events in Witton model and showed minor improvement in water levels at MHG, though no major changes to flood extents. Method C will be used going forward in design runs for Witton Model.		Closed			Closed
4.03	Provide spreadsheets outlining how form loss values are derived with reference to publications including page, chapter, section table etc.	ALL		No	3	N/A, no prior model available	calcualtion spreadhseet was not provided to be reviewed		untill receiving calcualtions and spreadhseet it remains open	Open		as per email from ARUP including additional information, it is acceptable.	Closed
Boundary Condition Representation													
5.01	Are tailwater level(s) or slope parameters associated with HQ downstream boundaries correct? Note, the 2020-10-AA version of HPC and newer uses a consistent approach with Classic for HQ boundaries.	ALL		No	2	N/A, no prior model available	To elaborate: how Brisbane river tidal and flood event has been syncronised? In model, tidal timeseris are from time zero which I supposed is 00:01am. Are the clibration/validation events are also based on time 00:01am. please confrim and explain.	ARUP: 8/05/2023 Tidal timeseries and flood event rainfall were synchronised and started at the same point in time (varies based on flood event).		Closed			Closed
5.02	Are the model upstream and downstream boundaries a sufficient distance away from the study area?	ALL		No	2	N/A, no prior model available	Glass wall must be checked later when rare events are to be applied	ARUP: 8/05/2023 Noted and will be monitored in design runs.	will be re-reviewed in design	Closed		no issue	Closed
5.03	Are model inflows correct?	ALL		No	3	N/A, no prior model available	From notes inside the ecf, it seems that 1d_bc approach werre used based on excess ranifall of catchments. Are they routed flow from URB or purely excess rainfall? If they are routed, it is fine; however if they are purely excess rainfall (without routing), following this approach has skipped the both hydrology and hydraulic routing mechanism of catchment especialy for upstream catchments, potentially significantly impacting the results.	ARUP: 8/05/2023 Noted and can confirm that routed subarea rainfall excess are applied from URBS modelling, as noted in URBs manual Section 6.5.		Closed			Closed
5.04	Is the flow distribution acceptable?	ALL		No	3	N/A, no prior model available	Please refer to above response	ARUP: 8/05/2023 See above response in comment 5.03.		Closed			Closed
5.05	Are the 1D-2D linkages defined correctly?	ALL		Yes	1	N/A, no prior model available							
5.06	Are there terrain adjustments at 1D-2D linkages? If yes, are they appropriate?	ALL		Yes	1	N/A, no prior model available							
5.07	Are IWL conditions applied correctly?	ALL		Yes	1	N/A, no prior model available							
5.08	Is direct rainfall modelling being used as the inflow? If yes, use SGS topography sampling must be used to avoid artificial depression storage artefacts. TUFLOW HPC 2020 or newer is required	ALL		N/A	1	N/A, no prior model available							
5.09	Is direct rainfall modelling being used as the inflow? If yes, the hydraulic model extent should cover the entire upstream catchment area. If this is not the case, RORB hydrology inflows must be used as a source of external inflow to fully accommodate for the upstream contributing catchment inflows. Report the configuration of this model.	ALL		N/A	1	N/A, no prior model available							
TUFLOW Run Files													
6.01	Is 1D time-step equally divisible of the 2D timestep. Is the timestep value appropriate relative to the minimum 1D network channel length and associated flow velocity?	ALL		No	2	N/A, no prior model available	dt* is around 0.15 and 1d time step is set to 0.5, so they arr not divisible. For HBC it is recommended to use "HPC 1D Synchronisation" command to adjust automatically to avoid instabilities of 1d network	ARUP: 8/05/2023 Noted. 1D time step defined in ECF is the maximum/limiting timestep the 1D solver can use. HPC 1d Synchronisation command is used within Witton Model. The command uses a default setting of MAXMISE 1D TIME STEP | 10 (see TLFs). Default settings automatically adjust timestep to syn 1D and 2D based on the size of the "maximum 1D timestep" {MAXMISE 1D TIME STEP } and the" target 2D timestep" of {10}. No adjustment to default settings is proposed.		Closed			Closed
6.02	Is 2D time-step within 1/5 and 1/2 of the grid cell size for TUFLOW Classic simulations?	TUFLOW Classic		N/A	1	N/A, no prior model available	HPC						
6.03	Is the 2D dtStar value reported in the *<<simulation>>.hpc.dt.csv* file greater than recommended minimum relative to the grid cell size for TUFLOW HPC simulations based on the dominant control number (courant, celerity or diffusion number)?	TUFLOW HPC		Yes	1	N/A, no prior model available	dt* min of 0.15 (between 1/10 to 1/40 of ell size) Nu and Nc are fine. Nd was mostly 0.3, showing instabilities in boundaries which can be mostly due to impact of tidal flow	ARUP: 8/05/2023 Noted. Nc, Nu and Nd to be monitored in design runs.	will be re-reviewed in design	Closed		no issue	Closed
6.04	Are TUFLOW default parameters used? If non-default values are used, list them and justify their use?	ALL		Yes	1	N/A, no prior model available							
Log File													
7.01	Does MI Projection Check == Error (not Warning)?	ALL		No	2	N/A, no prior model available	when there are set to warning, a potential for offset of layers due to different coordinations are accepted. Please change to Error	ARUP: 8/05/2023 Noted. Default setting of GIS Projection Check is ERROR and has not been adjusted. Hard coded into TCF for design event runs.		Closed			Closed
7.02	Does MI Save Date == Error (not Warning or Off)	ALL		No	2	N/A, no prior model available	please apply	ARUP: 8/05/2023 Noted. Model is a SHP model, no MapInfo files are used. Default for Check MI Save Date is Error and has not be adjusted (see TFLs). Accordingly model will should flag error in MI GIS layers if MapInfo files are used in the future.		Closed			Closed
7.03	Is the snap tolerance not adjusted from its default value?	ALL		Yes	1	N/A, no prior model available							
7.04	Is the Mass Error (Qi+Qo > 5%) less than < 1.0%?	ALL		N/A	1	N/A, no prior model available	HPC						
7.05	Is the in / out volume changes representative of the expected inflows/outflow (i.e. Not oscillating due to poor stability)?	ALL		Yes	1	N/A, no prior model available							
7.06	Are there no repeat timestep, if there are, are they acceptable?	TUFLOW HPC		Yes	1	N/A, no prior model available							
7.07	Are there no Negative Depth Warnings, if there are, are they acceptable?	TUFLOW Classic		Yes	1	N/A, no prior model available							
Messages Layer													
8.01	Are there no ERRORs in the messages layer?	ALL		Yes	1	N/A, no prior model available							
8.02	CHECK 2118 and WARNING 2118: Are ZC values lowered by a reasonable amount and do the lowered cells match the neighbouring terrain?	ALL		Yes	1	N/A, no prior model available							
8.03	WARNING 1100: Are the invert mismatches acceptable?	ALL		Yes	1	N/A, no prior model available							
8.04	CHECK 1401 and CHECK 1402: Are these failures in automatic manholes creation ok?	ALL		Yes	1	N/A, no prior model available							
8.05	CHECK 1111: Are these overwrites mistakes or by design?	ALL		Yes	1	N/A, no prior model available							
8.06	Are the other Checks and Warnings in the messages layer acceptable?	ALL		No	3	N/A, no prior model available	Warning 2073, Check 2210, Warning 2118 (for more than 0.5m please check the location whethere the lowering is intentional and reasonable)	ARUP: 8/03/2023 "WARNING 2218 - Manning's n value of 1. for Material 32 is unusually low or high." - set to 1 for building footprint per BCC recommendation. "WARNING 2073 - Object ignored. Only Points, Lines, Polylines & Regions used." - NULL objects are in Material Layer (2d_mat_WCFS_LandUse_001_R) and does not affect the way the material layer is read into the model (see DEM M check file). NULL geometries identified and removed for design runs. "CHECK 2210 - Top of first FC Layer is below ground level." - expected where they occur in banks of waterway where obvert of each bridge "WARNING 2118" - 4 point show dropping up to 0.64m at a culvert outlet into open channel. From review of Lidar, Culvert ILs and channel survey, this drop is representative of on ground conditions. Lidar appears to capture higher levels due to vegetation. Other instances less than <0.5m. Review of Lidar, network IL and survey	will be re-reviewed in design	Closed		acceptable	Closed
Results													
9.01	If depth varying Manning n is used, is *Map Output Data Types == n* specified for review of time varying Manning n value	ALL		N/A	1	N/A, no prior model available							
9.02	Is *Map Output Data Types == dt* specified for review of the location that defines the minimum timestep for the simulation?	TUFLOW HPC		Yes	1	N/A, no prior model available							
9.03	Are there any topographic or boundary condition input definition errors which correlate to the location of minimum timestep?	TUFLOW HPC		Yes	1	N/A, no prior model available							
9.04	Check results stability at Culvert SX inlet/outlet; where instability existing consider using SX polygon with A Factor set to 5	TUFLOW HPC		Yes	1	N/A, no prior model available							
9.05	Does reducing the *Control Number Factor* from 1.0 to 0.8 cause a significant change in the maximum water level result for the **designated flood** event? If the answer is yes, the Control Number Factor should be reduced. This should be the final check after all other input and parameter reviews have been completed. Lower CFL is typically only justified in situations of extremely high velocity and/or situations where the cell size is significantly smaller than the flow depth for a given location.	TUFLOW HPC		No	2	N/A, no prior model available	This will be checked for design events	ARUP: 8/05/2023 Noted re CNF sensitivity testing, if deemed required/necessary by BCC.	will be re-reviewed in design	Closed		h and v are in a reasonable range with dt*>1/40 gid cell	Closed
9.06	Are there PO lines at all key locations?	ALL		Yes	1	N/A, no prior model available							
9.07	Are pipes flowing full where expected (refer to _CCA.mif)?	ALL		No	2	N/A, no prior model available	This will be checked for design events	ARUP: 8/05/2023 Noted will be monitored during design runs	will be re-reviewed in design	Closed		no issue	Closed
9.08	Are maximum water surface levels (h) realistic?	ALL		No	2	N/A, no prior model available	This will be checked for design events	ARUP: 8/05/2023 Noted will be monitored during design runs	will be re-reviewed in design	Closed		no issue	Closed
9.09	Are maximum velocities (v) realistic?	ALL		Yes	1	N/A, no prior model available	except for a local area under the bridge (very downstream) with V=4.3m/s others are mostly up to 2m/s for main waterways	ARUP: 8/05/2023 Noted will be monitored during design runs	will be re-reviewed in design	Closed		no issue	Closed
9.10	Are flows in pipes and channels realistic?	ALL		Yes	1	N/A, no prior model available							
9.11	Is the model extent sufficient, such that the area of inundation does not abut against the model extent code boundary for the largest modelled flood event?	ALL		Yes	1	N/A, no prior model available	This will be re-checked for design events. Potential changes in 2d code	ARUP: 8/05/2023 Noted.	will be re-reviewed in design	Closed		no issue	Closed
9.12	Do flood extents for the range of modelled event magnitude follow a logical order of progression (1% > 2% > 5% AEP etc.)	ALL		Yes	1	N/A, no prior model available	This will be checked for design events	ARUP: 8/05/2023 Noted.	will be re-reviewed in design	Closed		no issue	Closed

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Appendix M: Modelling User Guide

Witton Creek Flood Study

Model User Guide

Prepared by Arup PTY
Prepared for Brisbane City Council

November 2023

Table of Contents

Intoduction

1.1 Witton Creek Flood Study (2023)

This document is to be read in conjunction with the Witton Creek Flood Study - Volume 1 (2023).

The Witton Creek Flood Study (2023) incorporates the calibration and verification of the hydrologic and hydraulic models as well as design event and very rare / extreme event 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 February 2020, March 2017, and May 2015. Verification of the URBS and TUFLOW models utilised the January 2013 and June 2016 historical storm events.

Design and very rare / extreme flood magnitudes were estimated for the full range of events from 50 % AEP to PMF. These analyses assumed hydrologic ultimate catchment development conditions in accordance with the current version of BCC City Plan.

Two 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. This scenario was run for both (i) current and (ii) projected future climate conditions.
- 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. This scenario was run for only projected future climate conditions.

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.

Hydrologic Model and Hydraulic Models

1.3 Hydologic Models

1.3.1 General

The URBS modelling has been undertaken using Version 6.34 (beta), with simulations performed using the URBS Control Centre Version 4.3.4.

The URBS modelling has been separated into:

- Calibration / Verification, and
- Design and Very Rare / Extreme

The following sections discuss each, respectively.

1.3.2 Calibration and Verification 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 below.

Event 1 – February 2020

The name and location of the URBS Control Centre project for the February 2020 historical event is:

..\URBS \Model\calib02\WittonCk_Feb2020_5min.prj

The name and location of the February 2020 event folder is indicated below, where the URBS Control Centre Settings are outlined in Figure 1.3.1.

..\URBS\Model\calib02\Feb2020_5min\data

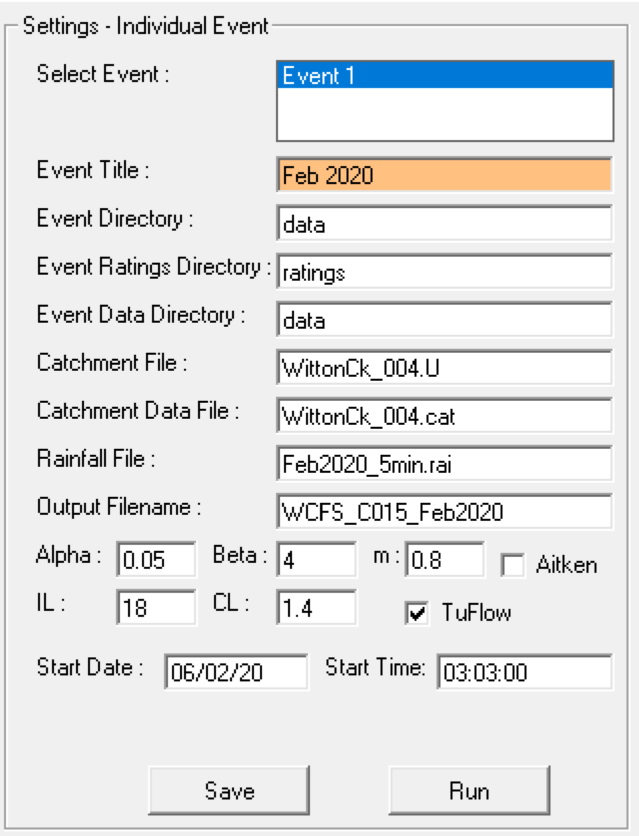

Figure 1.3.1: Febuary 2020 event URBS Settings

Event 2 – March 2017

The name and location of the URBS Control Centre project for the March 2017 historical event is:

..\URBS\ Model\calib02\WittonCk_Mar2017_5min.prj

The name and location of the March 2017 event folder is indicated below, where the URBS Control Centre Settings are outlined in Figure 1.3.2.

..\URBS\ Model\calib02\Mar2017_5min\data

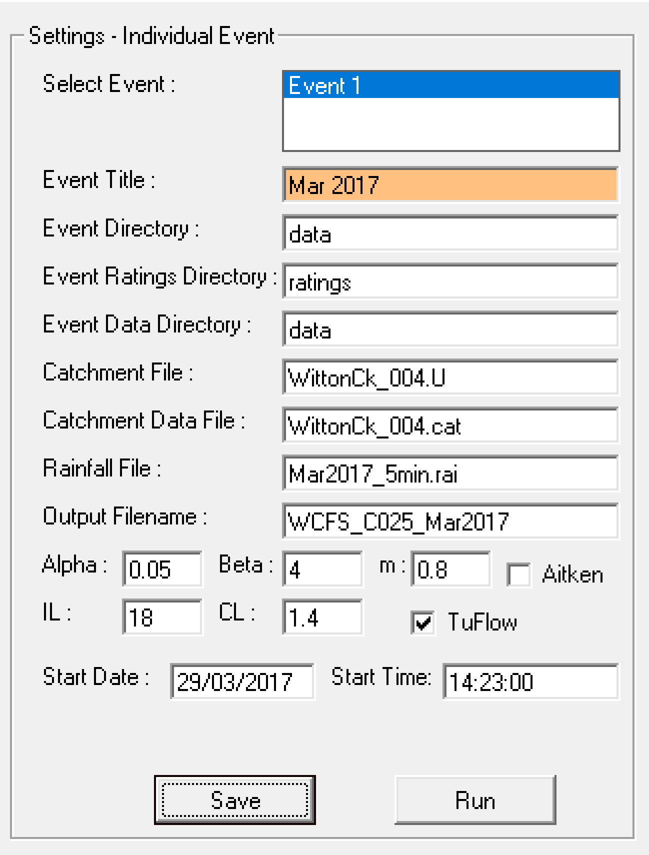

Figure 1.3.2: March 2017 event URBS Settings

Event 3 – May 2015

The name and location of the URBS Control Centre project for the May 2015 historical event is:

..\URBS\Model\calib02\WittonCk_May2015_5min.prj

The name and location of the May 2015 event folder is indicated below, where the URBS Control Centre Settings are outlined in Figure 1.3.3.

..\URBS\Model\calib02\May2015_5min\data

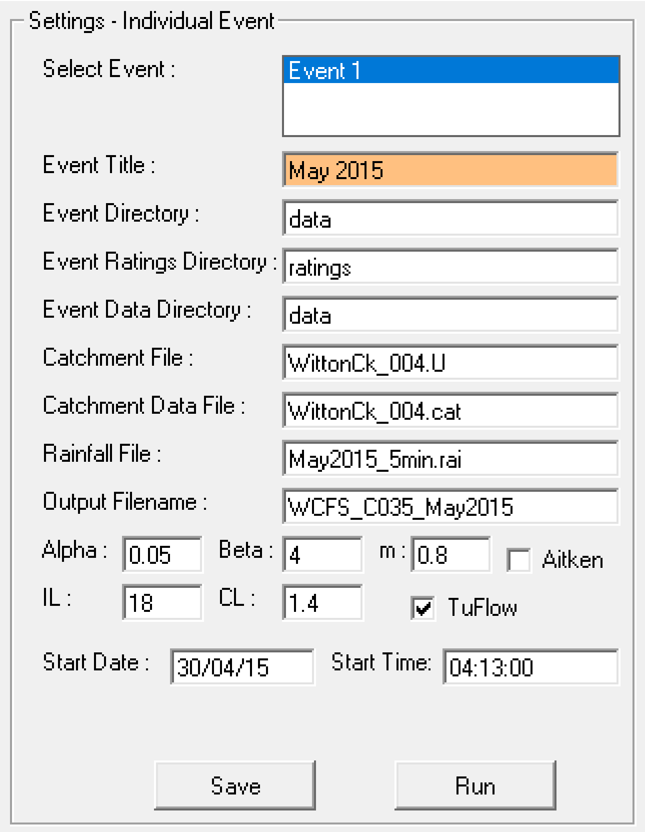

Figure 1.3.3: May 2015 event URBS Settings

Event 4 – January 2013

The name and location of the URBS Control Centre project for the January 2013 historical event is:

..\URBS\Model\calib02\WittonCk_Jan2013_5min.prj

The name and location of the January 2013 event folder is indicated below, where the URBS Control Centre Settings are outlined in Figure 1.3.4.

..\URBS\Model\calib02\Jan2013_5min\data

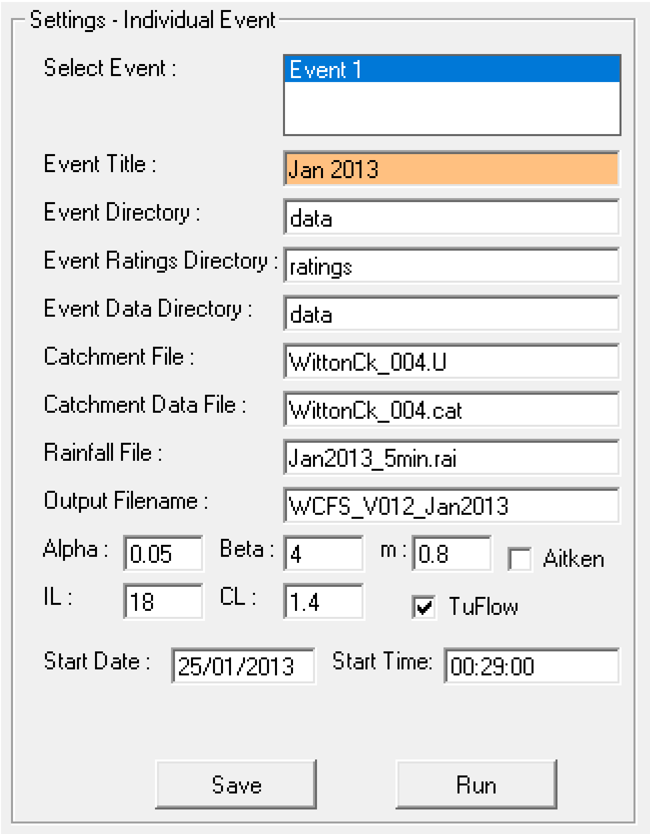

Figure 1.3.4: January 2013 event URBS Settings

Event 5 – June 2016

The name and location of the URBS Control Centre project for the June 2016 historical event is:

..\URBS\Model\calib02\WittonCk_Jun2016_5min.prj

The name and location of the June 2016 event folder is indicated below, where the URBS Control Centre Settings are outlined in Figure 1.3.5.

..\URBS\Model\calib02\Jun2016_5min\data

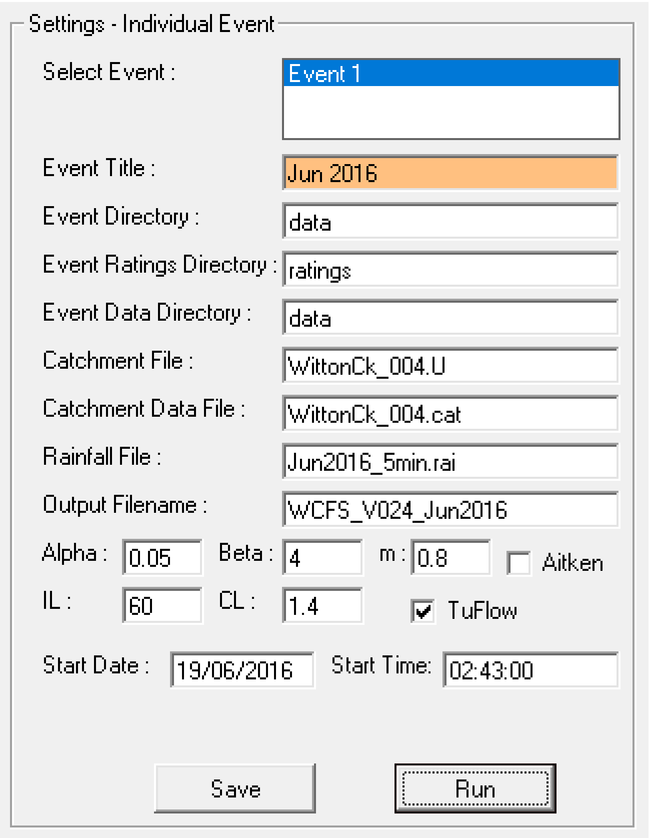

Figure 1.3.5: June 2016 event URBS Settings

1.3.3 Design and Very Rare / Extreme Models

For the Design and Very Rare, and Extreme Event runs, three models were developed. The models developed were:

- Design and Very Rare Event Models – Existing Climate
- Design and Very Rare Event Models – Climate Change for RCP4.5 to Year 2100
- Extreme Model – PMF Existing Climate

These URBS models are discussed individually in the following sections below:

Design and Very Rare Model – Existing Climate

The name of the project fand location of the URBS Control Centre project for the Design Event and Very Rare event model is:

..\URBS \Model\des2023_LIMB\ WittonCk_DES001.prj

The name and location of the Design and Very Rare Model folder is indicated below, where the URBS Control Centre Settings are outlined in Where the URBS Control Centre Settings are outlined in Figure 1.3.6

..\URBS \Model\des2023_LIMB\ run01

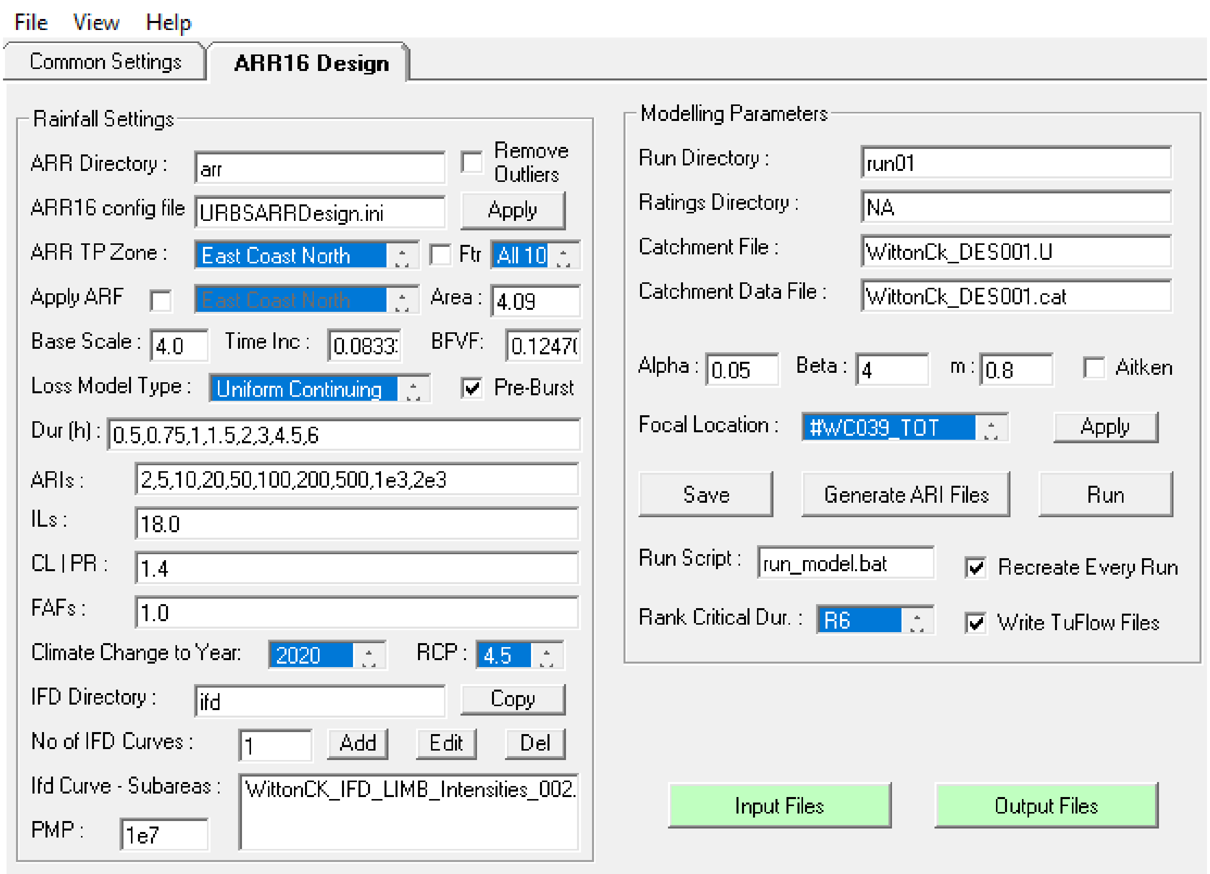

Figure 1.3.6 Design Even Run Settings - 50% AEP to 0.05% AEP Events

Design and Very Rare Model (With Climate Change)

The name and location of the URBS Control Centre project for the Design Event and Very Rare event model with climate change adjustment to RCP4.5 to Year 2100 is:

..\URBS\Model\des2023_LIMB\ WittonCk_DES001_CC.prj

Where the URBS Control Centre Settings are outlined in Figure 1.3.7.

..\URBS\Model\des2023_LIMB\ run02

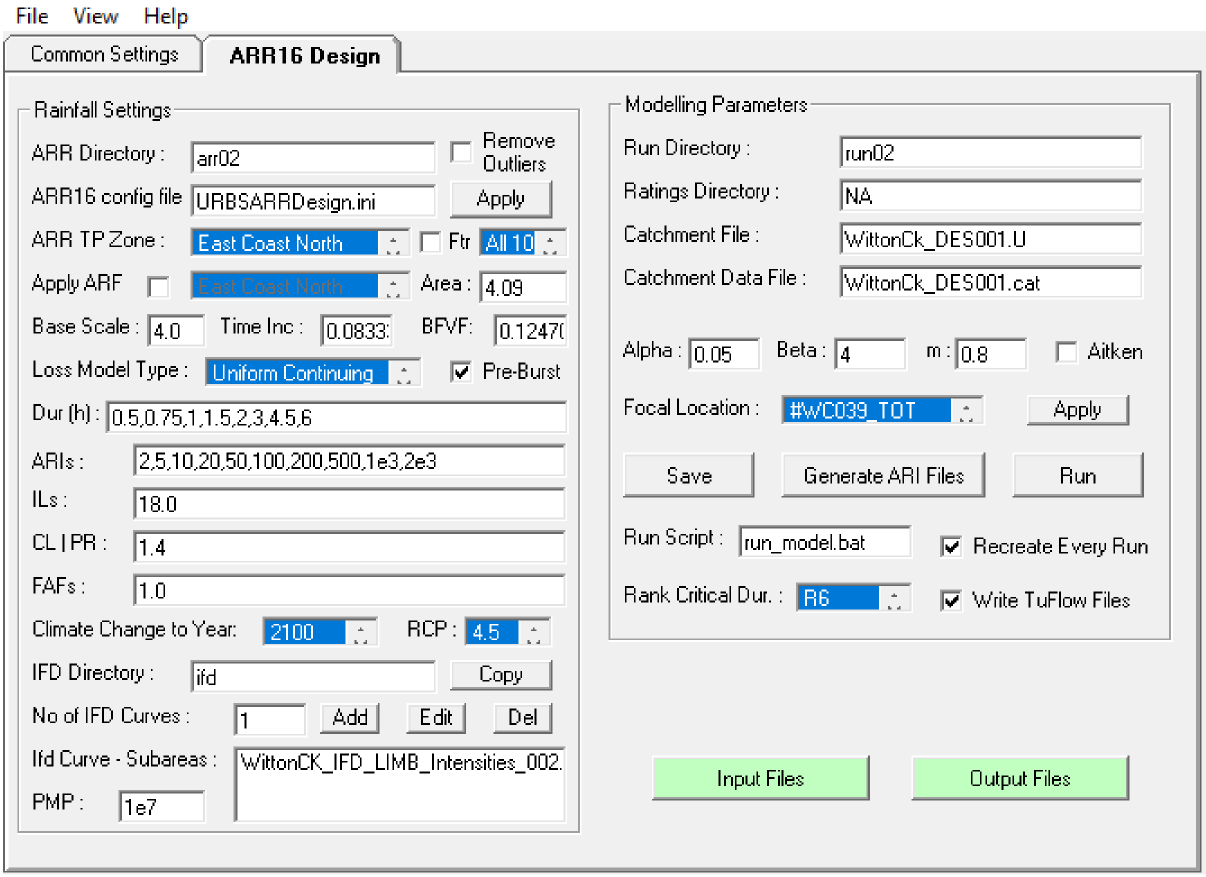

Figure 1.3.7 Design and Very Rare Events Run Settings

Extreme Event (PM) Model

The name and location of the URBS Control Centre project for the Design Event and Very Rare event model is:

..\URBS\Model\des2023_LIMB\ WittonCk_PMF01.prj

Where the URBS Control Centre Settings are outlined in Figure 1.3.8

..\URBS\Model\des2023_LIMB\ run03

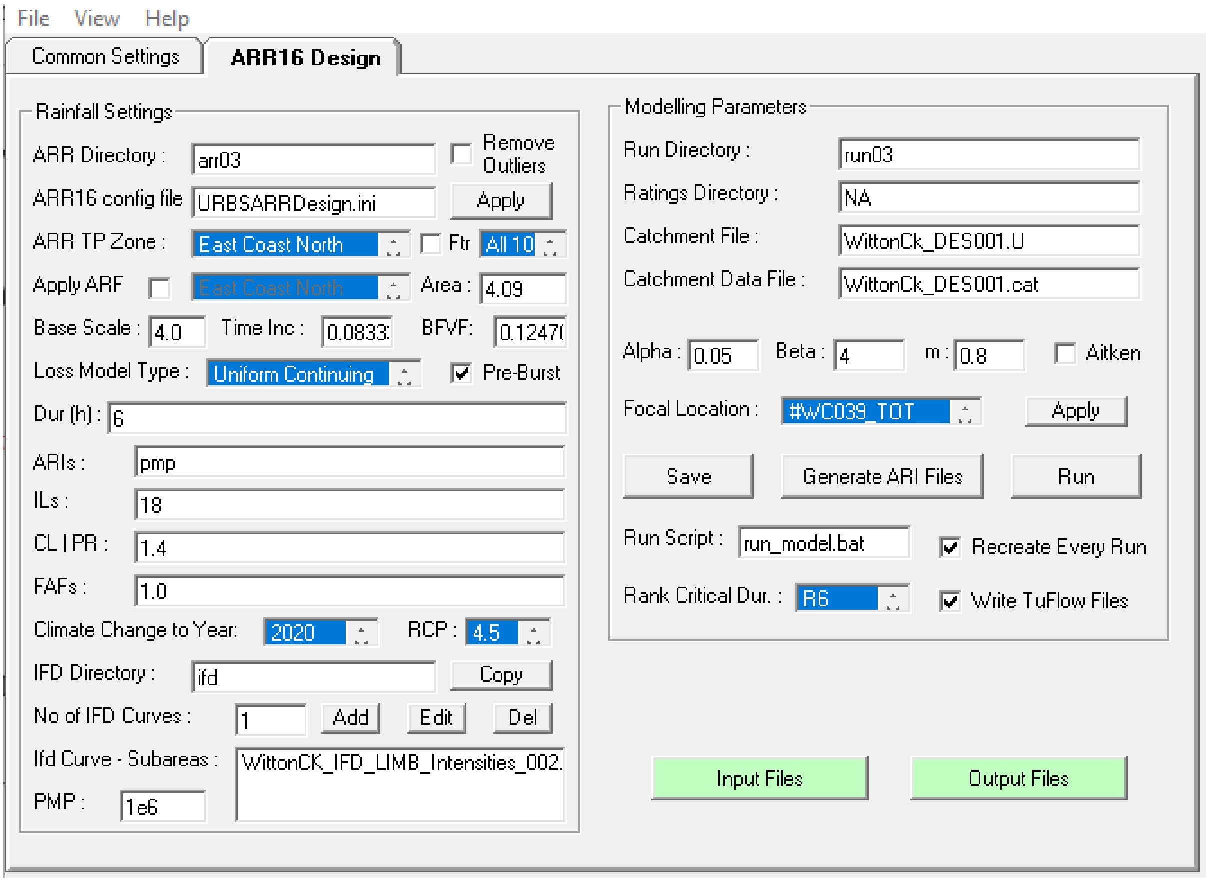

Figure 1.3.8 Extreme Event (PMF) Run Settings

1.4 Hydraulic Models

1.4.1 General

TUFLOW modelling was undertaken in TUFLOW HPC using build: 2020-10-AC-iSP-w64.

The TUFLOW modelling was undertaken using a single TUFLOW Control Files (TCF), which was named:

WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.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.

1.4.1 Calibration and Verification Models

TUFLOW simulations were undertaken for all five historical events. The February 2020, March 2017 May 2015, January 2023, and June 2019 model setup is essentially the same for each historical rainfall event, apart from the boundary condition.

For the calibration and verification runs, the scenario and event codes in the TCF represent the following conditions:

- ~s1~ is the TUFLOW Calibration/Verification Scenario
- ~s2~ is the TUFLOW Solver (HPC or CLA)
- ~e1~ is the Urbs Calibration Run Database
- ~e2~ is the Calibration Event
- ~e3~ is the Historical Event Flag
- ~e4~ is the Tail Water Condition (Stream Gauge Record)

Table 1.4.1indicates the scenario and event codes to be used inside the TUFLOW batch file.

Table 1.4.1 – TUFLOW Calibration and Verification Batch Codes

Model Simulation	Scenario 1 ~s1~	Scenario 2 ~s2~	Event 1 ~e1~	Event 2 ~e1~	Event 3 ~e1~	Event 4 ~e1~
February 2020	C126	HPC	C015	Feb2020	H	Feb20
March 2017	C229	HPC	C025	Mar2017	H	Mar17
May 2015	C326	HPC	C035	May2015	H	May15
January 2013	V110	HPC	V012	Jan2013	H	Jan13
June 2016	V210	HPC	V024	Jun2016	H	Jun16

An example batch file command for February 2020 simulation would be as follows:

%RUN% -s1 C126 -s2 HPC -e1 C015 -e2 Feb2020 -e3 H -e4 Feb20 WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf

1.4.2 Design and Very Rare / Extreme Models

For the Design and Very Rare / Extreme Model simulations, the scenario and event codes in the TCF represent the following conditions:

- ~s1~ is the TUFLOW BCC Scenario (Scenario 1 or Scenario 3)
- ~s2~ is the TUFLOW Solver (HPC or CLA)
- ~e1~ is the AEP Event, where the following codes represent:
  - 50PC - 50% AEP + Existing Climate
  - 20PC - 20% AEP + Existing Climate
  - 10PC - 10% AEP + Existing Climate
  - 05PC - 5% AEP + Existing Climate
  - 02PC - 2% AEP + Existing Climate
  - 01PC - 1% AEP+ Existing Climate
  - 0P50 - 0.5% AEP+ Existing Climate
  - 0P20 - 0.2% AEP+ Existing Climate
  - 0P05 - 0.05% AEP+ Existing Climate
  - 50PC_CC - 50% AEP + Climate Change RCP4.5 to Year 2100
  - 20PC_CC - 20% AEP + Climate Change RCP4.5 to Year 2100
  - 10PC_CC - 10% AEP + Climate Change RCP4.5 to Year 2100
  - 05PC_CC - 5% AEP + Climate Change RCP4.5 to Year 2100
  - 02PC_CC - 2% AEP + Climate Change RCP4.5 to Year 2100
  - 01PC_CC - 1% AEP + Climate Change RCP4.5 to Year 2100
  - 0P50_CC - 0.5% AEP + Climate Change RCP4.5 to Year 2100
  - 0P20_CC - 0.2% AEP+ Climate Change RCP4.5 to Year 2100
- ~e2~ is the Storm Duration, where the following codes represent:
  - M00030 - 30min
  - M00045 - 45min
  - M00060 - 60min
  - M00090 - 90min
  - M00120 - 120min
  - M00180 - 180min
  - M00270 - 270mim
  - M00360 - 360min
- ~e3~ is the Temporal Pattern (TP0 – TP9 URBS naming convention)
- ~e4~ is the Tail Water Condition, where the following codes represent:
  - MHWS - mean high water springs
  - MHWS_2100 - mean high water springs with climate change
  - HAT - highest astronomical tide
  - HAT_2100 - highest astronomical tide with climate change

Design Events

TUFLOW simulations were undertaken for all Scenario 1, and Scenario 3 design events up to and including the 1 % AEP event.

Table 1.4.2 indicates the scenario and event codes to be used inside the TUFLOW batch file for Scenario 1 and Scenario 3 for Design Events.

Table 1.4.2: TUFLOW Scenario 1 and Scenario 3 Design Event Batch Codes

Model Simulation	**Scenario 1 ~s1~**	**Scenario 2 ~s2~**	**Event 1 ~e1~**	**Event 2 ~e1~**	**Event 3 ~e1~**	**Event 4 ~e1~**
Design Events – Scenario 1 (Existing Climate)	D006	HPC	50PC 20PC 10PC 05PC 02PC 01PC	M00030 M00045 M00060 M00090 M00120 M00180 M00270 M00360	TP0 TP1 TP2 TP3 TP4 TP5 TP6 TP7 TP8 TP9	MHWS
Design Events – Scenario 1 (including climate Change)	DC02	HPC	50PC_CC 20PC_CC 10PC_CC 05PC_CC 02PC_CC 01PC_CC	M00030 M00045 M00060 M00090 M00120 M00180 M00270 M00360	TP0 TP1 TP2 TP3 TP4 TP5 TP6 TP7 TP8 TP9	MHWS_2100
Design Events – Scenario 3 (including Climate Change)	DC03	HPC	50PC_CC 20PC_CC 10PC_CC 05PC_CC 02PC_CC 01PC_CC	M00030 M00045 M00060 M00090 M00120 M00180 M00270 M00360	TP0 TP1 TP2 TP3 TP4 TP5 TP6 TP7 TP8 TP9	MHWS_2100

An example batch file command for Scenario 1, 1% AEP, Temporal Pattern 4 , 45 minute duration event (inclusive of climate change) would be as follows:

%RUN% -s1 DC02 -s2 HPC -e1 01PC_CC -e2 M00045 -e3 TP4 -e4 MHWS_2100 WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf

Very Rare and Extreme Events
TUFLOW simulations were undertaken for all Scenario 1, and Scenario 3 for very rare and extreme events up to the PMF event.

Table 1.4.3 indicates the scenario and event codes to be used inside the TUFLOW batch file for Scenario 1 and Scenario 3 for very rare and extreme events.

Table 1.4.3: TUFLOW Scenario 1 and Scenario 3 Design Event Batch Codes for upto 1% AEP event

Model Simulation	**Scenario 1 ~s1~**	**Scenario 2 ~s2~**	**Event 1 ~e1~**	**Event 2 ~e1~**	**Event 3 ~e1~**	**Event 4 ~e1~**
Very Rare Events – Scenario 1 (Existing Climate)	DX02	HPC	0P50 0P20 0P05	M00030 M00045 M00060 M00090 M00120 M00180 M00270 M00360	TP0 TP1 TP2 TP3 TP4 TP5 TP6 TP7 TP8 TP9	HAT
Extreme Events – Scenario 1 (existing Climate)	PMF01	HPC	PMF	M00360	TP0	HAT
Very Rare Events - Scenario 1 (including climate Change)	DXC02	HPC	0P50_CC 0P20_CC	M00030 M00045 M00060 M00090 M00120 M00180 M00270 M00360	TP0 TP1 TP2 TP3 TP4 TP5 TP6 TP7 TP8 TP9	HAT_2100
Very Rare Events Scenario 3 (including Climate Change)	DXC03	HPC	0P50_CC 0P20_CC	M00030 M00045 M00060 M00090 M00120 M00180 M00270 M00360	TP0 TP1 TP2 TP3 TP4 TP5 TP6 TP7 TP8 TP9	HAT_2100

An example batch file command for Scenario 1, 0.5% AEP (inclusive of climate change), Temporal Pattern 4 , 45 minute duration event would be as follows:

%RUN% -s1 DXC02 -s2 HPC -e1 0P50_CC -e2 M00045 -e3 TP4 -e4 HAT_2100 WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf

An example batch file command for PMF event would be as follows:

%RUN% -s1 PMF01 -s2 HPC -e1 PMF -e2 M00360 -e3 TP0-e4 HAT WCFS_~s1~_~s2~_~e1~_~e2~_~e3~_~e4~_002.tcf