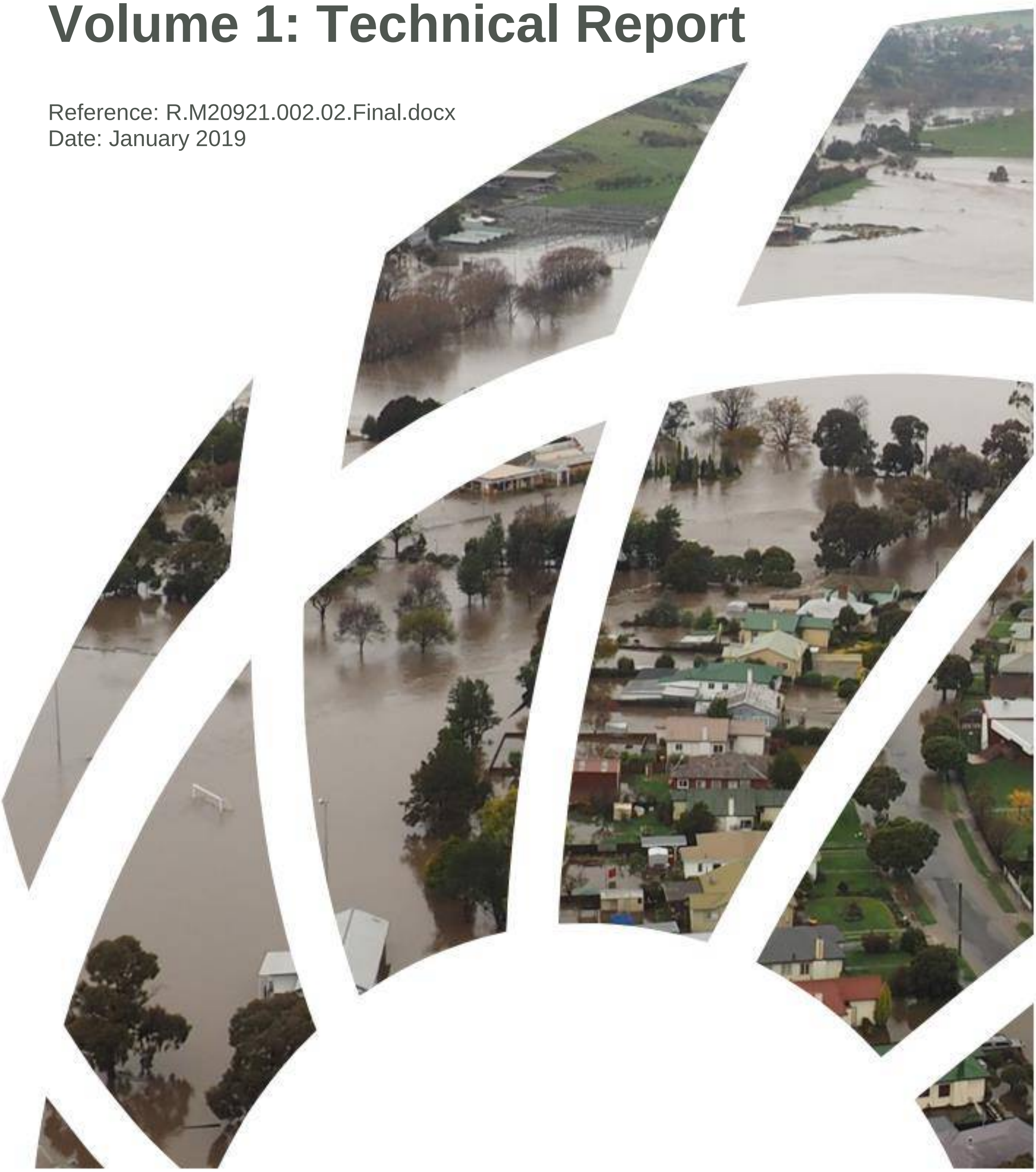
North and South Esk Rivers Flood Modelling and Mapping Update Volume 1: Technical Report

Reference: R.M20921.002.02.Final.docx
Date: January 2019



Document Control Sheet

BMT Eastern Australia Pty Ltd
Level 5, 99 King Street
Melbourne Vic 3000
Australia

Tel: +61 3 8620 6100
Fax: +61 3 8620 6105

ABN 54 010 830 421

www.bmt.org

Document:	R.M20921.002.02.Final.docx
Title:	North and South Esk Rivers Flood Modelling and Mapping Update Volume 1: Technical Report
Project Manager:	Michael South
Author:	Michael South and Daniel Machado
Client:	City of Launceston
Client Contact:	Michael Newby
Client Reference:	

Synopsis: **This technical report presents the methodology and results of the flood modelling undertaken for the North and South Esk Rivers Flood Modelling and Mapping Update.**

REVISION/CHECKING HISTORY

Revision Number	Date	Checked by		Issued by	
0	17 Aug 2018	JL		MS	
1	16 Nov 2018	JL		MS	
2	18 Jan 2019	DM		MS	

DISTRIBUTION

Destination	Revision										
	0	1	2	3	4	5	6	7	8	9	10
City of Launceston	1	1	1								
BMT File	1	1	1								
BMT Library	1	1	1								

Contents

List of Figures

List of Tables

1 Introduction

BMT was commissioned by the City of Launceston (Council) to undertake an update of the North and South Esk Rivers flood modelling and mapping, originally completed by BMT (formerly known as BMT WBM) in 2008 (BMT WBM 2008).

The purpose of the flood mapping update is to:

- Update the existing flood modelling (hydrologic assessment and hydraulic modelling) to current best practice standards in line with the 2016 release of the Australian Rainfall and Runoff Guidelines (ARR 2016) (Ball, et al. 2016)
- Calibrate the flood model to the June 2016 flood event
- Produce a series of flood mapping and intelligence outputs for use in Council's GIS system
- Assess levee breach scenarios (documented separately)
- Develop a Flood Warning and Intelligence Information System ((FloodIntel), documented separately)

Since the completion of the previous flood modelling and mapping project (BMT WBM, 2008), the flood model has undergone minor changes, mainly the inclusion of additional survey data in response to the assessment of various floodplain developments and levee upgrades. However, the fundamentals of the model are the same including; the river inflows (hydrology), tidal boundaries, LiDAR data and bathymetric survey, model grid cell size and TUFLOW model version.

With the June 2016 flood event providing a significant amount of flood data (flood levels and flood extents), the release of ARR 2016, advances in computing power, and improvements to TUFLOW and associated modelling packages, updating the flood modelling and mapping will aid Council's and others, ability to define and manage flood risk in Launceston.

This report presents the methodology and results of the flood model and mapping update.

This technical report documents the methodology and results of the flood model and mapping update. This includes; data collection, hydrologic assessment, hydraulic modelling and mapping methodology. The flood mapping products are presented in Volume 2: Flood Mapping.

1.1 Catchment Description

Launceston is located at the confluence of the North and South Esk Rivers (Figure 1-1) where they form the Tamar River estuary, draining to Bass Straight 70 km to the north at Low Head. The Tamar River is a tidal estuary, and tidal influences are observed up to approximately St Leonards on the North Esk River and past Kings Bridge on the South Esk River.

As shown in Figure 1-2, Launceston is protected from riverine flooding by a levee system that is designed to provide protection up to the 95% confidence level, 0.5% Annual Exceedance Probability (AEP) design flood event (as determined by the 2008 study), an event with a magnitude equal to approximately the 1929 historic flood event.

The North Esk River catchment (Figure 1-1) covers an area of about 1,065 km^2 directly to the east of Launceston before it flows into the Tamar River (DIPW 2009). The two main rivers draining the North Esk River catchment are the North Esk River and the St Patricks River. Both rivers originate on the slopes of Ben Nevis.

The South Esk River catchment (Figure 1-1) encompasses the major catchments of the South Esk, Macquarie and Meander Rivers. Upstream of Lake Trevallyn the rivers merge to form an overall catchment with an approximate area of 9,120 km^2. When flows exit Lake Trevallyn via the spillway, they flow towards Launceston from the west via Cataract Gorge.

1.2 Previous Studies

There have been previous hydrologic and flood studies related studies have been undertaken for Launceston, and the North Esk, South Esk and Tamar Rivers. The key previous studies relating to this flood mapping update include:

- ***River Tamar & North Esk River Flood Study Final Report*** **(BMT WBM 2008)** – This study established the TUFLOW flood model and associated mapping that was used to upgrade Launceston's levee system. This study provided the basis of the flood model that is been updated for the current study
- ***Trevallyn Flood Frequency Review for Launceston City Council*** **(HydroTAS 2008)** - This study determined the design event flows for the South Esk River that were used in BMT WBM (2008)
- ***North Esk Flood Frequency Analysis at Corra Lin*** **(WRL 2006)** – This study determined the design event flows for the North Esk River that were used in BMT WBM (2008)
- ***Report on Flood Mitigation Measures for the City of Launceston*** **(Munro 1959)** – This study was a comprehensive hydrologic and hydraulic study undertaken to define flood risk in Launceston for the establishment of the original levee system

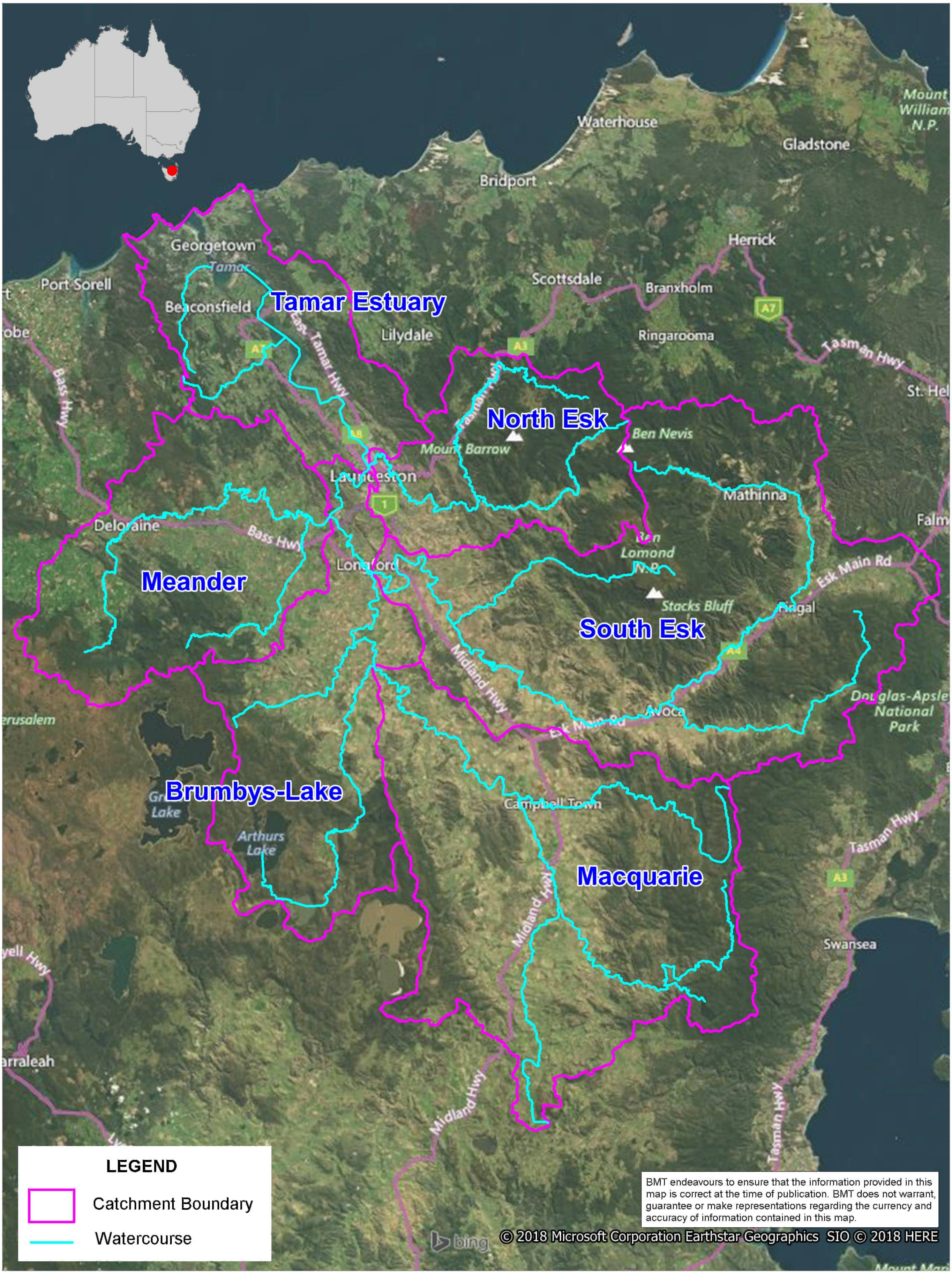


Figure 1-1 North and South Esk River Catchments

T:\M20921.MS.Launceston_Mapping_2017\MapInfo\Drawings\Final\Fig1-1_Location_RevA.WOR

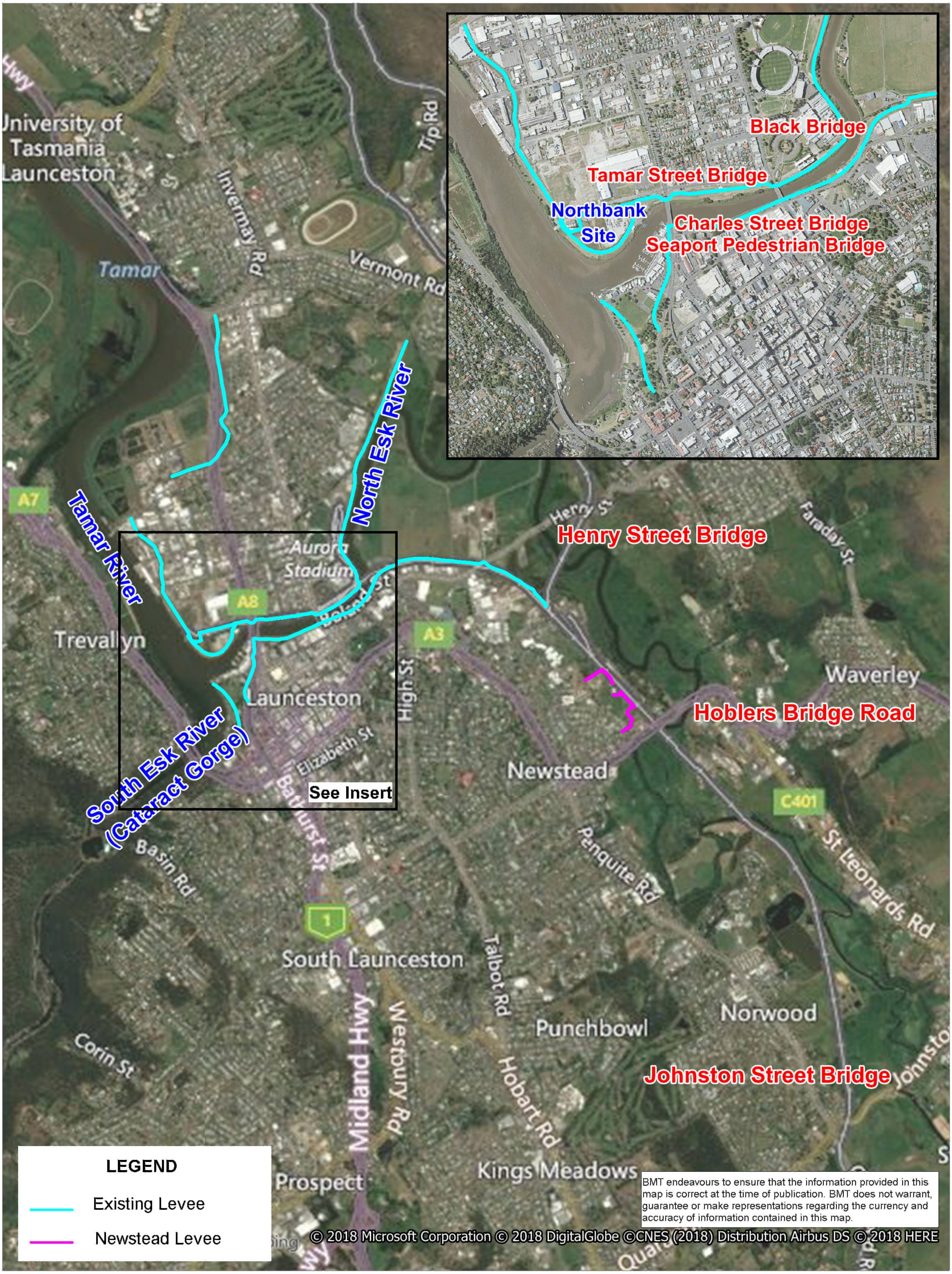


Figure 1-2 Study Area Layout

T:\M20921.MS.Launceston_Mapping_2017\MapInfo\Drawings\Final\Fig1-2_Layout_RevA.WOR

2 Data Collection and Review

This section documents the data that has been collated for the Study. Information has been sourced information from:

- City of Launceston
- Bureau of Meteorology
- Hydro Tasmania
- TasWater
- Department of Primary Industries, Parks Water and Environment (DPIPWE)
- Tasmanian Government List Data website

The data was comprehensively reviewed to identify any significant data gaps and to gain a complete understanding of issues in the study area.

2.1 Topographic Data

The following topographic data sets were provided by Council for this study:

- 1 m gridded LiDAR (captured in 2013), to form the basis of the Digital Elevation Model (DEM) is used to define the hydrologic model’s base topography
- The Geoscience Australia, 1 second Shuttle Radar Topography Mission (SRTM) DEM of the entire catchment
- Bathymetry survey captured in 2017 for the North Esk River and the Tamar River. Bathymetry survey captured on 2 June 2016 and 18 June 2016, either side of the June 2016 flood event, was also provided to assess the impacts of scour on model calibration
- Additional ground and levee survey
- Design surfaces of proposed Newstead Levee and Northbank Site re-development

2.2 Aerial Photography

Aerial photography captured in January 2017, covering the study area was provided by Council. Aerial photography of the catchment is an important tool for verifying study area characteristics such as land use, surface roughness, building footprints and other structures.

2.3 Planning, Cadastral and Floor Level Data

Planning and cadastral data for the catchment was sourced from the Tasmanian Government’s List Data Website in GIS format, including; planning zones, planning overlays, cadastre and roads. This data is used in conjunction with the aerial photography and on-ground photography to define the to define factors such as fraction impervious and Manning's values (roughness).

Council provided floor level survey to be used flood risk assessment and emergency response purposes.

2.4 Hydraulic Structure Data

Council provided bridge details accompanied by photography. This data provided information about the layout of bridges along the North Esk River for inclusion in the hydraulic model. BMT also referred to design drawings provided as part of BMT WBM (2008) to determined details of the bridges along the North Esk River

Penstock details were also supplied by Council for inclusion in the hydraulic model.

2.5 Stream Gauge Data

To review the flood frequency analysis and define the design event model inflow conditions, stream gauge data was obtained for the North Esk River at Corra Linn stream gauge from the Bureau of Meteorology (BoM) and at the Lake Trevallyn Spillway from Hydro Tasmania.

For model calibration, council also provided stream level data for the June 2016 flood event at:

- North Esk River at Johnston Road
- North Esk River at Hoblers Bridge Road
- North Esk River at Henry Street
- North Esk River at Tamar Street

The stream gauges listed above are shown in Figure 2-1.

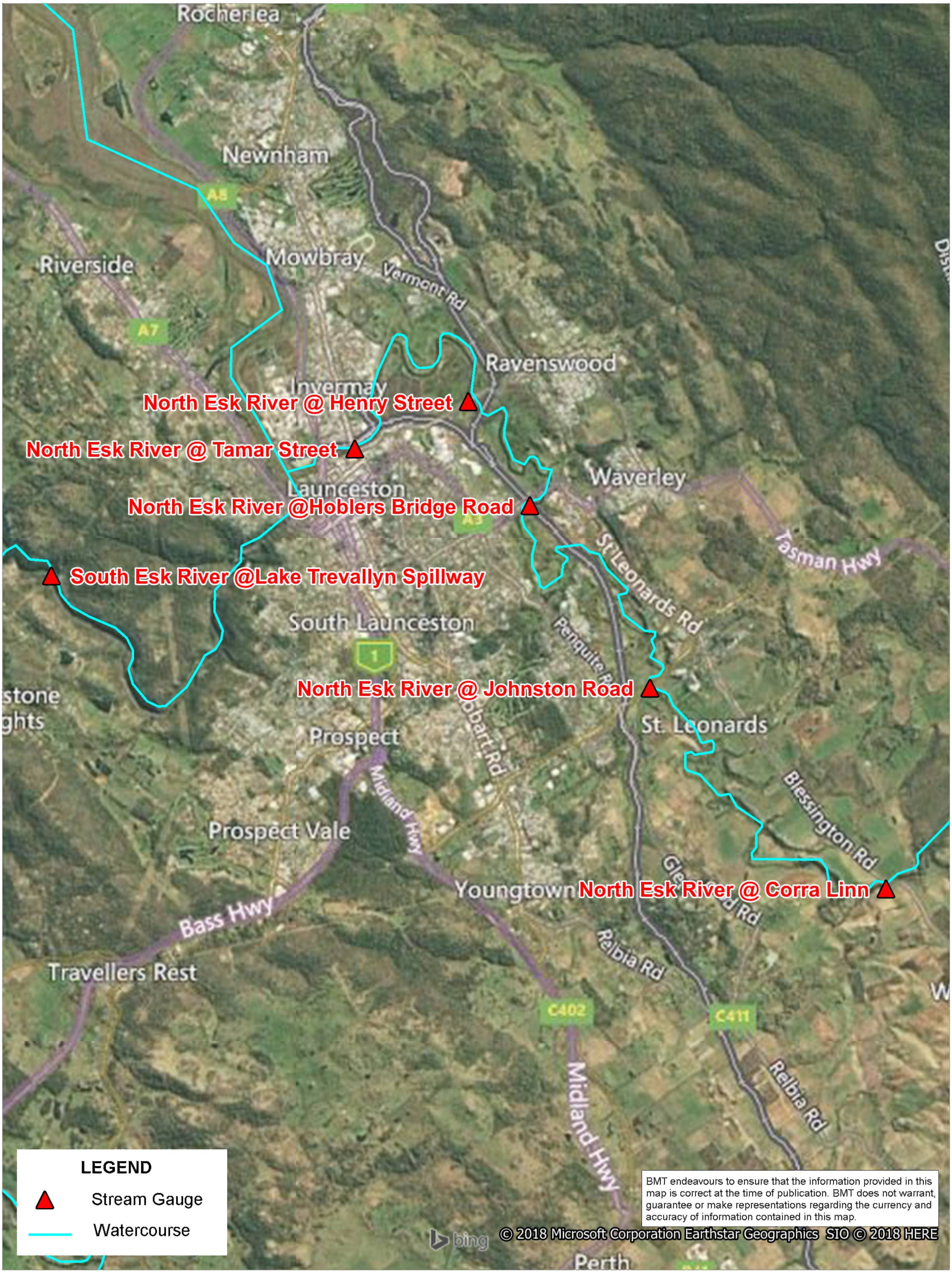


Figure 2-1 Stream Gauge Locations

T:\M20921.MS.Launceston_Mapping_2017\MapInfo\Drawings\Final\Fig2-1_Gauges_RevA.WOR

2.5.1 North Esk River at Corra Linn Stream Gauge Data Review

BoM provided the following stream level data for the North Esk River at Corra Linn stream gauge:

- 1929, 1936, 1958 (peak flood levels only)
- 1962 - 1973 (incomplete series of continuous flows recorded at three hourly intervals)
- 1986 (peak flood level only)
- 1987 – 1989 (incomplete series of continuous flows recorded at three hourly intervals)
- 1990 - 2017 (incomplete series of continuous flows)

During the period from 1990 to 2016, there are significant gaps in stream level data, as presented in Table 2-1. To ensure that annual maximum water levels did not occur during periods of missing gauge data, daily maximum flows from a calibrated Source (formerly WaterCAST) catchment model established for the entire Tamar River catchment was used to identified high water level/flow events. To do this, the Source model was run for an extended period covering the period of continuous gauge recordings (1990 - 2016).

Figure 2-2 shows a comparison of the Source model and recorded flows for the North Esk at Corra Linn gauge for 1994. As shown in Figure 2-2 that during the period of missing data beginning 5 September 1994 there were no significant flow events in the North Esk River and that annual maximum water level was recorded by the stream gauge.

As shown in Figure 2-2, while the Source model provides and adequate representation of when significant flow events occur it does not provide an accurate estimate of the magnitude of flow. For this reason, the Source model results are not able to be used to provide an annual flow maximum for 1997 and 1998 where the annual maximum flow was not recorded.

Table 2-1 North Esk River at Corra Linn Stream Gauge Data Gaps (1990 - 2016)

Year	Comments	Year	Comments
1990	Continuous data starts 22/03/90	2004	
1991		2005	Data missing 23/05/05 - 25/05/05
1992		2006	Data missing 16/10/06 - 8/12/06
1993		2007	Data missing 14/08/07 - 18/10/07
1994	Data missing 5/09/94 - 9/02/95	2008	
1995		2009	
1996		2010	Data missing 23/03/10 - 20/04/10
1997	Data missing 23/01/97 - 1/07/98	2011	
1998	Data missing 31/07/98 - 22/08/99	2012	
1999		2013	
2000	Data missing 30/06/00 - 1/08/00	2014	
2001	Data missing 18/10/01 - 20/12/02	2015	
2002	Data missing 3/02/02 - 2/08/02	2016	
2003	Data missing 14/08/03 - 17/01/02		

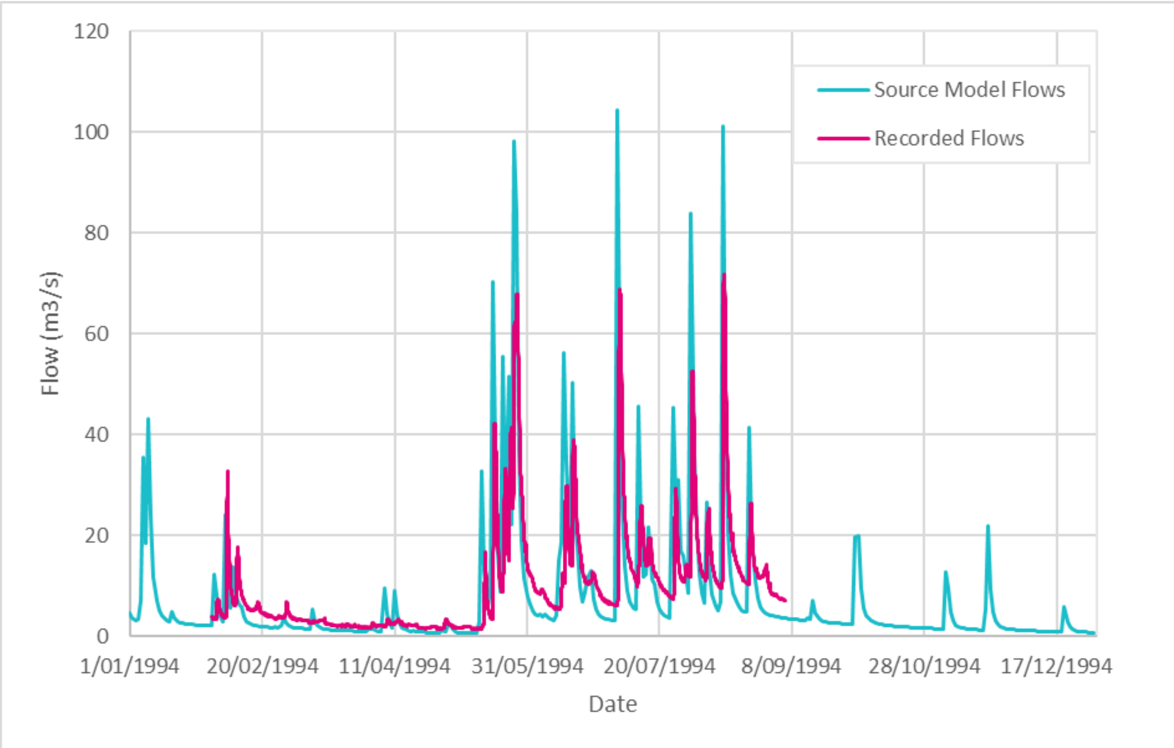


Figure 2-2 1994 Source Model and Recorded Flows for North Esk River at Corra Linn

2.5.1.1 North Esk River at Corra Linn Rating Curve Verification

It is believed that the current rating curve for the North Esk River at Corra Linn (Figure 2-3) was reviewed by Hydro Tasmania Consulting in 2005 (HydroTAS 2005). The current rating table extends to a level of 4.5 m, where for higher water levels, such as the June 2016 flood event (6.44 m), it is believed that the rating curve was extrapolated as shown in Figure 2-3.

The gauge is in narrow gorge at a location that experiences extremely high velocities, excessive turbulence and the formation of a back eddy across half the cross section (Munro 1959). This creates great difficulty in physically gauging reliable velocities/flows, and in turn results in uncertainty in the accuracy of the current rating curve. This is highlighted in a comparison of estimated peak flow for the June 2016 event at the Corra Linn gauge and the Johnston Road Pipe Bridge gauge (located downstream). Figure 2-3 shows that at Corra Linn for the peak recorded water level of 6.44 m a flow of 795 m^3/s is estimated. This is significantly less than 935 m^3/s estimated at the Johnston Road Pipe Bridge for the peak recorded water level of 9.26 m. Given the close proximity of these two gauges and the absence of a major inflow between them, a similar peak flow is expected at the two gauges for a given flood event.

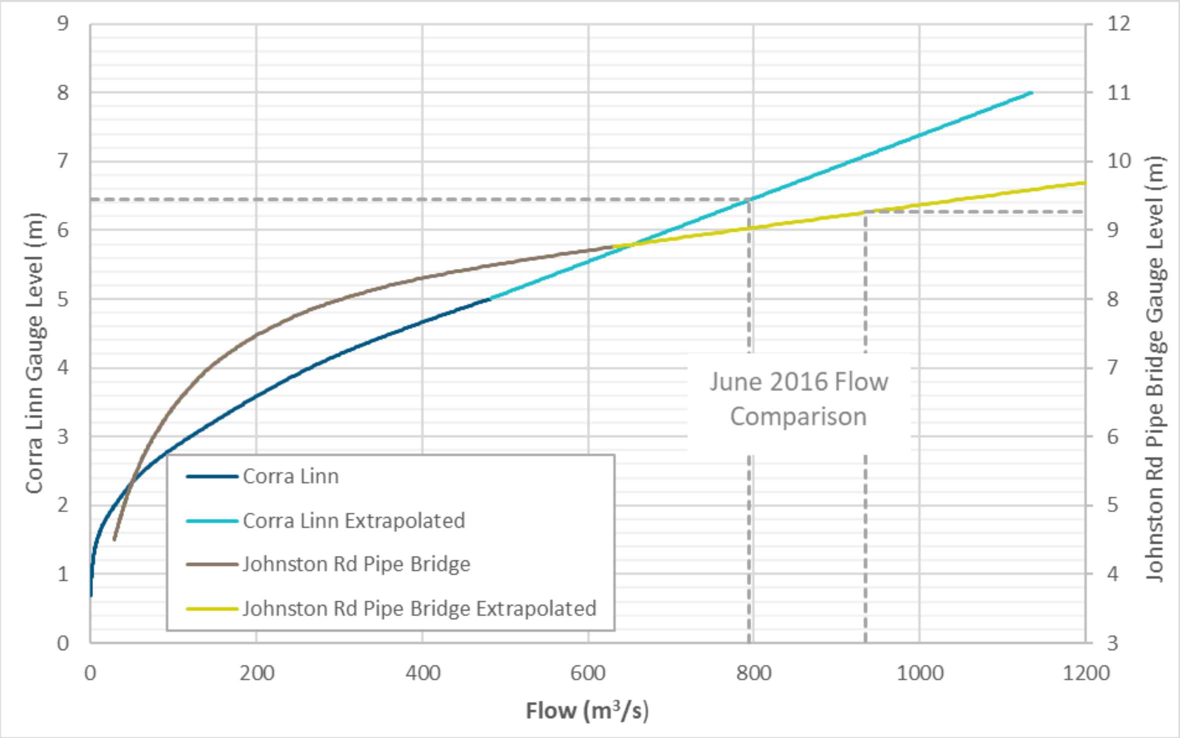


Figure 2-3 Current North Esk River at Corra Linn and Johnston Road Pipe Bridge Rating Curves

Initially to validate the rating curve a 2D hydraulic model was established of the gauge location. However, during the model calibration (Section 4.6) it was found that this method greatly underestimated flows resulting in much lower water levels downstream than recorded. A flow of 555 m^3/s was estimated for the June 2016 event, lower than that current estimation of 795 m^3/s and much lower than the flow estimated at the Johnston Road Pipe Bridge (935 m^3/s), derived from the current rating curve. As with the physical gaugings, an eddy also formed in the 2D model resulting in lower model flow outputs.

Based on the above information, it was decided to use the hydraulic model calibrated to recorded gauge levels and flood marks at the Johnston Road Pipe Bridge and downstream for the June 2016 (Section 4.6) to relate a revised rating curve at Johnston Road Pipe Bridge to Corra Linn. To achieve this the North Esk River at the Corra Linn stream gauge was modelled in 1D to remove the influence of the complex flow conditions from the model outputs. The resulting revised rating curves are shown in Figure 2-4. To verify the accuracy of the revised rating curves a comparison of peak recorded water levels to those related using the revised rating curves at the Johnston Road Pipe Bridge for a high flow event which occurred in July 2016. The results of this verification are presented in Table 2-2.

It should be noted that the revised rating curves relate the hydraulic modelling flow inputs developed for this study. For other uses, particularly for estimating low flows, further verification of the rating curves is recommended.

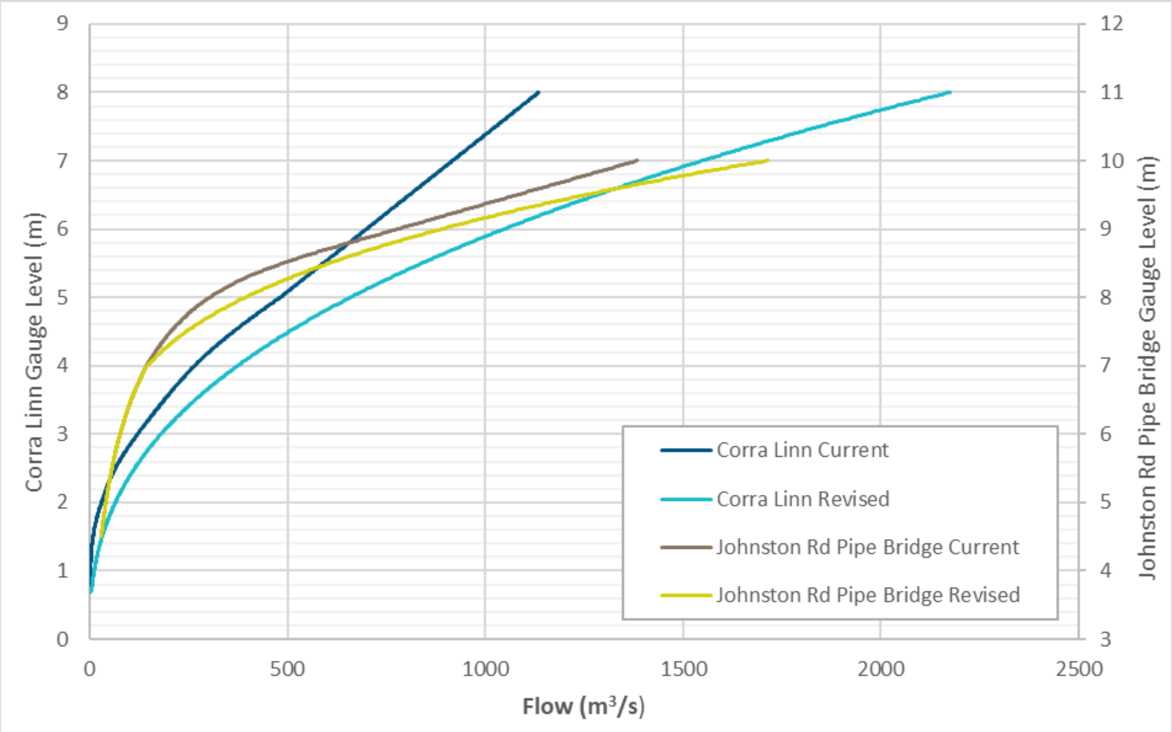


Figure 2-4 Revised North Esk River at Corra Linn and Johnston Road Pipe Bridge Rating Curves

Table 2-2 July 2017 Rating Curve Verification

Corra Linn Recorded Peak Water Level (m)	Revised Peak Flow (m^3/s)	Johnston Rd Pipe Bridge Related Water Level (m)	Johnston Rd Pipe Bridge Recorded Peak Water Level (m)
3.84	337	7.83	7.86

2.5.2 South Esk River at Lake Trevallyn Spillway Data Review

Hydro Tasmania provided spillway half-hour flow data for the Lake Trevallyn Dam spillway from 1955, when the Lake Trevallyn was completed, to 2017.

Annual maximum flows between 1901 and 1955 were sourced from HydroTAS (2008). These were gauged near the retired Duck Reach Power Station located in Cataract Gorge downstream of Lake Trevallyn. It is likely that the methods of estimating flow in the period between 1901 and 1955 are not as accurate as the estimates obtained from a hydraulic model, however, there is insufficient data available to undertake a rating curve review. This is because the exact location of where the stream gaugings were taken and the level of the stream gauge in m AHD (required to relate recorded water levels to hydraulic model results) are unknown.

As published in HydroTAS (2008), Munro (1959) also identified eight significant flow events in the 1800s. Two of these flood events, 1852 and 1863 are estimated to be larger than the largest recorded flood event in 1929.

2.6 Additional June 2016 Flood Event Data

Council provided further data that they captured during and after the June 2016 flood event. This data will be used for model calibration and includes:

- Surveyed flood marks
- Flood photography captured from a helicopter during the event
- A GIS flood extent digitised by Council from observed debris marks

3 Hydrologic Assessment

The hydrologic assessment is used to determine the flow rate and timing of the inflows of the hydraulic model for the North and South Esk Rivers. Flows have been derived for the following design events under existing, 2050 and 2090 climate conditions:

- 20% AEP
- 10% AEP
- 5% AEP
- 2% AEP
- 1% AEP
- 1 in 200 AEP
- 1 in 500 AEP
- 1 in 1000 AEP
- 1 in 2000 AEP
- Probable Maximum Flood (PMF) (existing conditions only)

The hydrologic assessment is used to determine the flow rate and timing of the inflows of the hydraulic model for the North and South Esk rivers. The original study (BMT WBM, 2008) used design flows determined from the flood frequency analyses (FFAs) undertaken by Hydro Tasmania Consulting (HydroTAS 2008) for the South Esk River at Lake Trevallyn and Water Research Laboratory (WRL 2006) for the North Esk River at Corra Linn.

Figure 3-1 taken from ARR 2016 shows the relative efficacy of different approaches to estimation of design floods. As shown in Figure 3-1 for 'rare' and 'very rare' design flood events, at-site flood frequency analysis provides the high level of efficacy. For this reason, the design hydrology will be determined for this study by updating the existing FFAs with additional streamflow data and new analysis techniques.

Regional flood frequency estimates (RFFEs) can also be used to reduce the uncertainty the determination of design flows and hence have also been assessed.

Hydrograph shape and timing has been determined by analysis historic flood events.

To produce flow estimates for the 2050 and 2090 climate conditions, a RORB hydrologic model was developed. The purpose of the RORB hydrologic modelling was to determine a multiplication factor on existing conditions flows as derived from the FFA for future climate change conditions. As such the RORB model was not calibrated, rather validated against the FFA.

If the RORB model is to be used for any future determination of design events, including extreme events, model calibration is required.

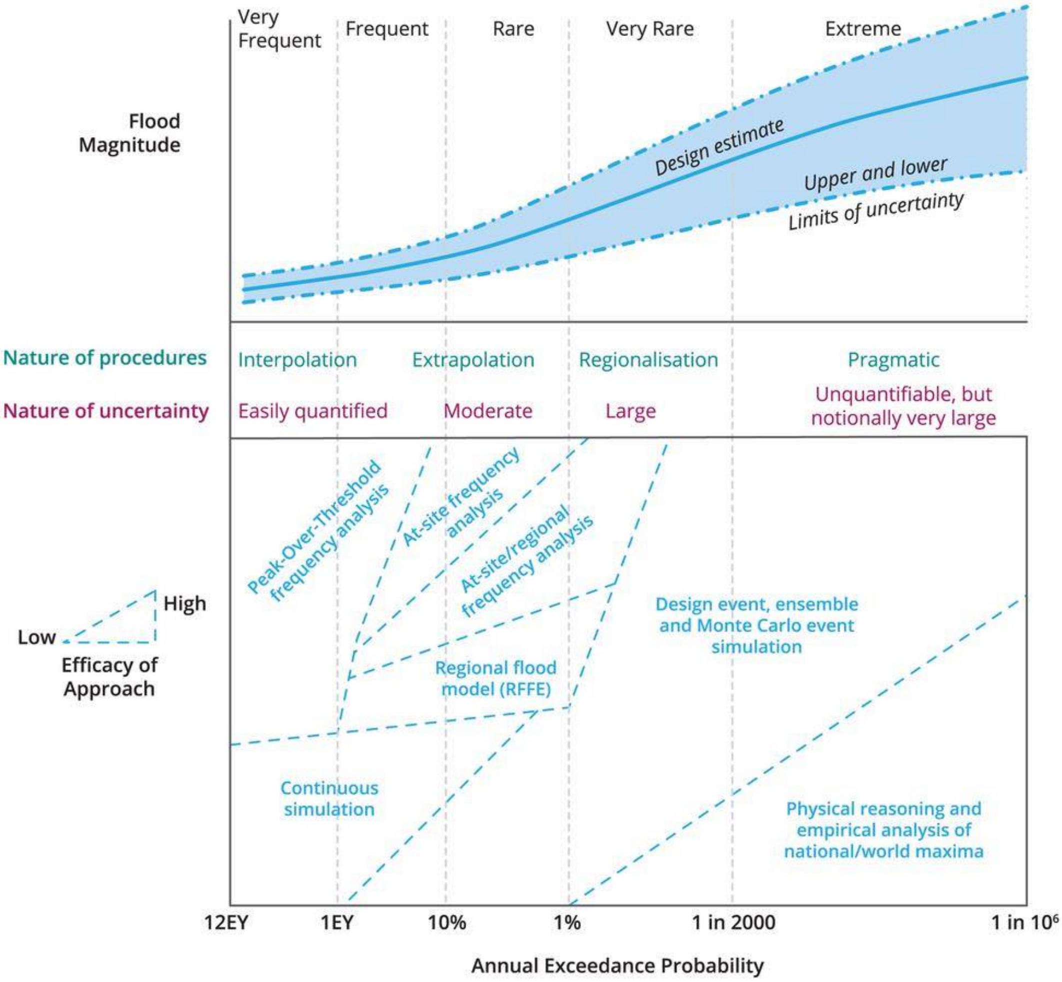


Figure 3-1 Relative Efficacy of Different Approaches for the Estimation of Design Floods (Ball, et al. 2016)

3.1 Regional Flood Frequency Estimates

RFFEs were completed for the North Esk River at Corra Linn and the South Esk River at Lake Trevallyn. This has been completed using the ARR Regional Flood Frequency Estimation Model tool (http://rffe.arr-software.org) and the guidelines provided in Book 3, Chapter 3 of ARR 2016.

RFFEs attempt to transfer flood characteristics from a group of gauged catchments to ungauged locations of interest to determine peak flow estimates of design flood events. For this study, both the North and South Esk catchments are gauged so the purpose of undertaking RFFEs is to reduce the uncertainty the determination of design flows using the at-site FFA method as described in Section 3.2.

The results of the RFFEs for the North Esk River at Corra Linn and the South Esk River at Lake Trevallyn are presented in Table 3-1 and Table 3-2 respectively.

Table 3-1 North Esk River RFFE Results

AEP	Peak Flow (m^3/s)	90% Quantile Probability Limits (m^3/s)	
20%	698	314	1,570
10%	876	338	2,240
5%	1,060	344	3,150
2%	1,320	340	4,770
1%	1,540	335	6,350

Table 3-2 South Esk River RFFE Results

AEP	Peak Flow (m^3/s)[1.]	90% Quantile Probability Limits (m^3/s)	
20%	1,170	536	2,580
10%	1,460	569	3,690
5%	1,760	574	5,200
2%	2,190	560	7,890
1%	2,530	548	10,500

[1.] The South Esk River at Lake Trevallyn has a catchment area of 8,990 km^2. This is outside the recommended catchment size of 0.5 to 1,000 km^2 and results have lower accuracy.

3.2 Flood Frequency Analyses

The at-site FFAs for the North Esk River at Corra Linn and the South Esk River at Lake Trevallyn has been undertaken using the guidelines provided in Book 3, Chapter 2 of ARR 2016. The FFA was undertaken using the Flike software package. Flike provides a Bayesian framework for comprehensive at-site flood frequency estimation that allows the inclusion of ungauged historical events.

The fitting of flood frequency distributions using Flike was undertaken with the following steps:

- Prepare data:
  - Collect gauged streamflow data
  - Collect historic data, including the review of the previous studies listed in Section 1.2
  - Undertake standard data checks on the stream flow data including checking error codes, cataloguing data gaps and undertaking visual inspections
  - Extract the annual maximum series and check peaks for independence
- Using Flike, fit an extreme value distribution to the annual maximum series, including the influence of:
  - Historic data (data that exists beyond the extent of the annual maximum series)
  - Censoring low flows with a multiple Grubbs Beck test from the data to ensure that the distributions are 'aware' of the full length of record as opposed to block censoring the data

- Prior parameters information from the RFFE model were applied to Flike as prior information

3.2.1 North Esk River at Corra Linn Flood Frequency Analysis

The existing design event peak flow estimates for the North Esk River at Corra Linn were undertaken by Water Research Laboratory in 2006 (WRL 2006). The revised FFA, incorporates 11 more years of stream gauge data (including the June 2016 flood event - the largest on record), a review of the rating curve and advances in the methodologies available to undertake FFAs since the completion of the WRL study in 2006.

3.2.1.1 Annual Maximum Flows

The adopted annual maximum flow series for the North Esk at Corra Linn is shown in Table 3-3. As described in Section 2.5.1, the series covers a continuous period of 31 years from 1986 to 2016 and flows were derived using the revised rating curve.

The period between 1986 to 2016 contains two incomplete years of data in 1997 and 1998. The United States Federal Emergency Management Agency (FEMA) has assessed the impact of including years with missing data in a FFA, and found that it may be acceptable if the years with missing data make up less 25% of annual maximum series and that missing data is not as a result of a significant flood event causing the gauge failure (FEMA 2004). As discussed in Section 2.5.1, there were no significant flood events in either 1997 or 1998, therefore including these years of missing data is considered acceptable for the purposes of this study.

Table 3-3 North Esk River at Corra Linn Adopted Annual Maximum Flows

Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)
1986	266	1997	21[1.]	2007	434
1987	212	1998	110[1.]	2008	110
1988	504	1999	202	2009	210
1989	180	2000	154	2010	236
1990	208	2001	457	2011	355
1991	221	2002	495	2012	121
1992	435	2003	515	2013	354
1993	178	2004	208	2014	310
1994	123	2005	635	2015	109
1995	177	2006	128	2016	1,253
1996	282				

[1.] The data set is not complete for 1997 and 1998 and the annual maximum flows are not available.

3.2.1.2 Historic Information

The peak water level data provided by BoM indicates that the June 2016 flood event was the largest gauged event for North Esk River at Corra Linn since water levels were first recorded in 1929. This information was incorporated into Flike by identifying no events were above the threshold flow of 1,253 m3/s (June 2016) in the 57 year period from 1929 to 1986.

3.2.1.3 Removal of Probable Influential Low Flows

During the period of record there were several low flow years. As recommend in ARR 2016, low flows were censored from the dataset to ensure that these low flows did not unduly affect the fit of the flood frequency curve. A discharge censor below 109 m^3/s was determined by using the multiple Beck Grubbs test which resulted in 1 event being censored.

3.2.1.4 Prior Parameters Information

The higher order Log Pearson Type III parameters derived from the RFFE (Table 3-4) were used as prior information to the Bayesian framework in Flike.

Table 3-4 North Esk River at Corra Linn RFFE Parameters

Parameter	Mean	St Dev	Correlation		
Mean ($\log_e$ flow)	5.888	0.540	1.000		
St dev ($\log_e$ flow)	0.510	0.387	-0.330	1.000	
Skew ($\log_e$ flow)	0.135	0.169	0.150	-0.440	1.000

3.2.1.5 Flood Frequency Analysis Results

The results of the FFA for the North Esk River at Corra Linn are shown in Table 3-5 and Figure 3-2. The best fit to the annual maximum data series was achieved using Bayesian inference framework and a Log-Pearson III probability model.

Table 3-5 North Esk River at Corra Linn FFA Results

AEP	Expected Quantile (m^3/s)	90% Quantile Probability Limits	
20%	400	331	491
10%	529	428	674
5%	670	527	889
2%	878	666	1,228
1%	1,056	778	1,534
1 in 200	1,252	895	1,892
1 in 500	1,543	1,059	2,458
1 in 1000	1,791	1,190	2,978
1 in 2000	2,064	1,330	3,559

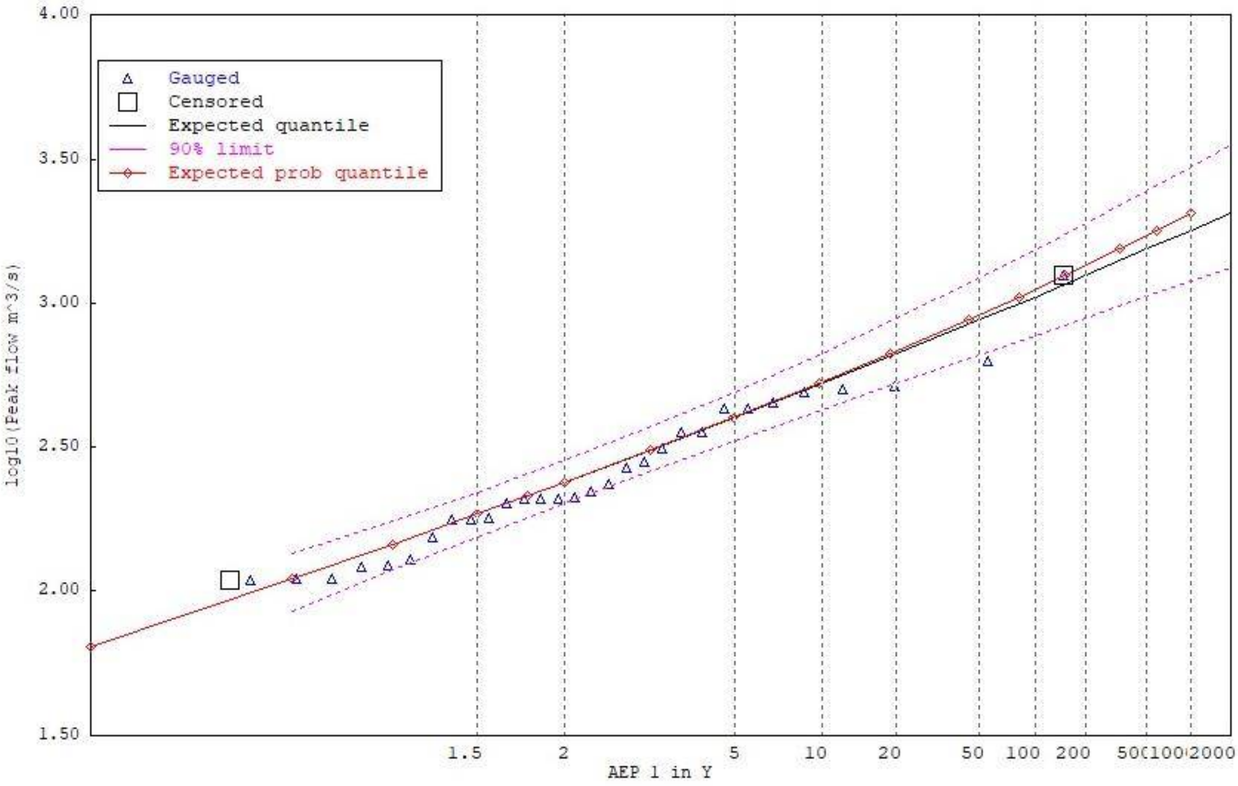


Figure 3-2 North Esk River at Corra Linn FFA Results

3.2.1.6 Flood Frequency Analysis Comparison

A comparison of the WRL (2006) and the revised FFA results is provided in Table 3-6. There is a significant difference in the results, where the revised FFA estimates are far larger than estimated in WRL (2006). However, this is primarily a function of the revised rating curve, increasing flow estimates. For example, the revised 1% AEP event peak flow of 1,056 m^3/s corresponds to gauge level of approximately 6.02 m. Using the current rating curve, a gauge level of 6.02 m relates to a flow of 703 m^3/s, not significantly greater than the WRL (2006) 1% AEP event peak flow estimate of 614 m^3/s.

The difference in the FFA results is due to:

- Revised rating curve and recorded flow estimates
- The revised FFA uses a continuous annual maximum flow series (including the incomplete years of 1997 and 1998) while the WRL (2006) FFA used a broken annual maximum flow series with 29 years of data spanning from 1929 to 2005
- The revised FFA includes the June 2016 event which is the largest gauged flood event on the North Esk River at Corra Linn
- The inclusion of historic flow data (no flood bigger than the 2016 event between 1929 and the commencement of the gauge)
- The use of prior parameters derived from the RFFE

Table 3-6 North Esk River at Corra Linn FFA Comparison

AEP	Adopted Peak Flow (m^3/s)	WRL (2006) Peak Flow (m^3/s)
20%	400	275
10%	529	345
5%	670	419
2%	878	526
1%	1,056	614
1 in 200	1,252	-
1 in 500	1,543	-
1 in 1000	1,791	-
1 in 2000	2,064	-

3.2.1.7 *Comparison to Historic Flood Events*

The results of the flood frequency analysis for the North Esk River (Table 3-5) can be used to estimate the AEPs for the historic flood events that have been recorded on the North Esk River. This analysis (Table 3-7) shows the 2016 flood event is estimated as being a 1 in 200 AEP flood.

Table 3-7 AEP Estimates for Historic Flood Events – North Esk River

Historic Event	Flow (m^3/s)	AEP Estimate
2016	1253	1 in 200
1929	710	4%
2005	635	6%
2003	515	11%

3.2.2 South Esk River at Lake Trevallyn Spillway Flood Frequency Analysis

The existing design event peak flow estimates for the South Esk River at Lake Trevallyn Spillway were completed by Hydro Tasmania Consulting in 2008 (HydroTAS 2008). The revised FFA, incorporates nine more years of stream gauge data and advances in the methodologies available to undertake FFAs since the completion of the Hydro Tasmania Consulting in 2008.

3.2.2.1 Annual Maximum Flows

The adopted annual maximum flow series for the South Esk River at Lake Trevallyn Spillway are shown in Table 2-1. As described in Section 2.5.2, the series covers a continuous period of 116 years from 1901 to 2016.

The annual maximum flows presented in Table 2-1 include the Trevallyn Hydroelectric Power Station offtake flows for the period between 1996 and 2016 when power station flows were recorded. The average power station flow, taken from the Lake Trevallyn and discharged directly into Tamar River, of 83.5 m^3/s was also included in the period between 1956 and 1996. This provides consistency with the pre-1956 flow series and provides a conservative FFA estimate if the power station is not in use during future flood events.

Table 3-8 South Esk River at Lake Trevallyn Spillway Adopted Annual Maximum Flows

Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)
1852[1.]	4,190	1929	3,964	1959	508	1989	773
1863[1.]	4,625	1930	386	1960	1,452	1990	299
1901	610	1931	1,997	1961	305	1991	348
1902	249	1932	811	1962	557	1992	744
1903	1,002	1933	366	1963	324	1993	966
1904	520	1934	485	1964	830	1994	131
1905	1,509	1935	1,002	1965	274	1995	482
1906	743	1936	904	1966	925	1996	891
1907	356	1937	520	1967	1,195	1997	411
1908	283	1938	1,157	1968	553	1998	963
1909	558	1939	1,105	1969	2,636	1999	305
1910	953	1940	417	1970	1,696	2000	556
1911	1,553	1941	194	1971	685	2001	438
1912	417	1942	636	1972	498	2002	212
1913	678	1943	558	1973	612	2003	1,103
1914	164	1944	1,435	1974	1,668	2004	847
1915	1,211	1945	417	1975	1,034	2005	1,056
1916	1,211	1946	1,157	1976	373	2006	132
1917	1,053	1947	628	1977	526	2007	399
1918	450	1948	211	1978	1,289	2008	135
1919	300	1949	410	1979	494	2009	1,155
1920	485	1950	442	1980	374	2010	893
1921	417	1951	549	1981	869	2011	1,629
1922	1,157	1952	992	1982	634	2012	265
1923	1,435	1953	612	1983	501	2013	682
1924	558	1954	682	1984	1,198	2014	509
1925	564	1955	590	1985	432	2015	30
1926	2,064	1956	1,307	1986	1265	2016	2,398
1927	386	1957	353	1987	234		
1928	826	1958	1,295	1988	744		

[1.] Historic flood events.

3.2.2.2 Historic Information

The 1852 and 1863 flood events, were estimated by Fuller, et al. (1990) to have a peak flow of 4,190 m^3/s and 4,625 m^3/s respectively, greater than the highest recorded annual maximum of 3,964 m^3/s

(1929) in the record period (1901 – 2016). The period of historic flood event records dates back to 1828, the earliest record of a historic flood events in Launceston (Munro 1959). This information was incorporated into Flike by identifying two events above the threshold flow of 3,964 m^3/s (1929) in the 73 year period from 1828 to 1901.

3.2.2.3 Removal of Probable Influential Low Flows

During the period of record there were several low flow years. As recommend in ARR 2016, low flows were censored from the dataset to ensure that these low flows did not unduly affect the fit of the flood frequency curve. A discharge censor below 131 m^3/s was determined by using the multiple Beck Grubbs test which resulted in 1 event being censored.

3.2.2.4 Prior Parameters Information

The South Esk River at Lake Trevallyn has a catchment area of 8,990 km2. This is outside the recommended catchment size of 0.5 to 1,000 km2 and RFFE results will have a lower accuracy. For this reason, the higher order Log Pearson III parameters derived from the RFFE were not used as prior information to the Bayesian framework in Flike.

3.2.2.5 Flood Frequency Analysis Results

The results of the FFA for the South Esk River at Lake Trevallyn Spillway are shown in Table 3-9 and Figure 3-3. The best fit to the annual maximum data series was achieved using Bayesian inference framework and a generalised extreme value (GEV) probability model.

The GEV probability model was adopted over the Log Pearson III as it provided a slightly better fit to the rarer historic events, with only marginally wider error bounds. The GEV model also provided the conservative flow estimate as detailed in Table 3-10.

Table 3-9 South Esk River at Lake Trevallyn Spillway FFA Results

AEP	Expected Quantile (m^3/s)	90% Quantile Probability Limits	
20%	1,147	1,017	1,302
10%	1,594	1,381	1,878
5%	2,132	1,782	2,643
2%	3,034	2,389	4,086
1%	3,902	2,916	5,631
1 in 200	4,975	3,517	7,751
1 in 500	6,796	4,440	11,710
1 in 1000	8,559	5,245	16,009
1 in 2000	10,744	6,180	21,802

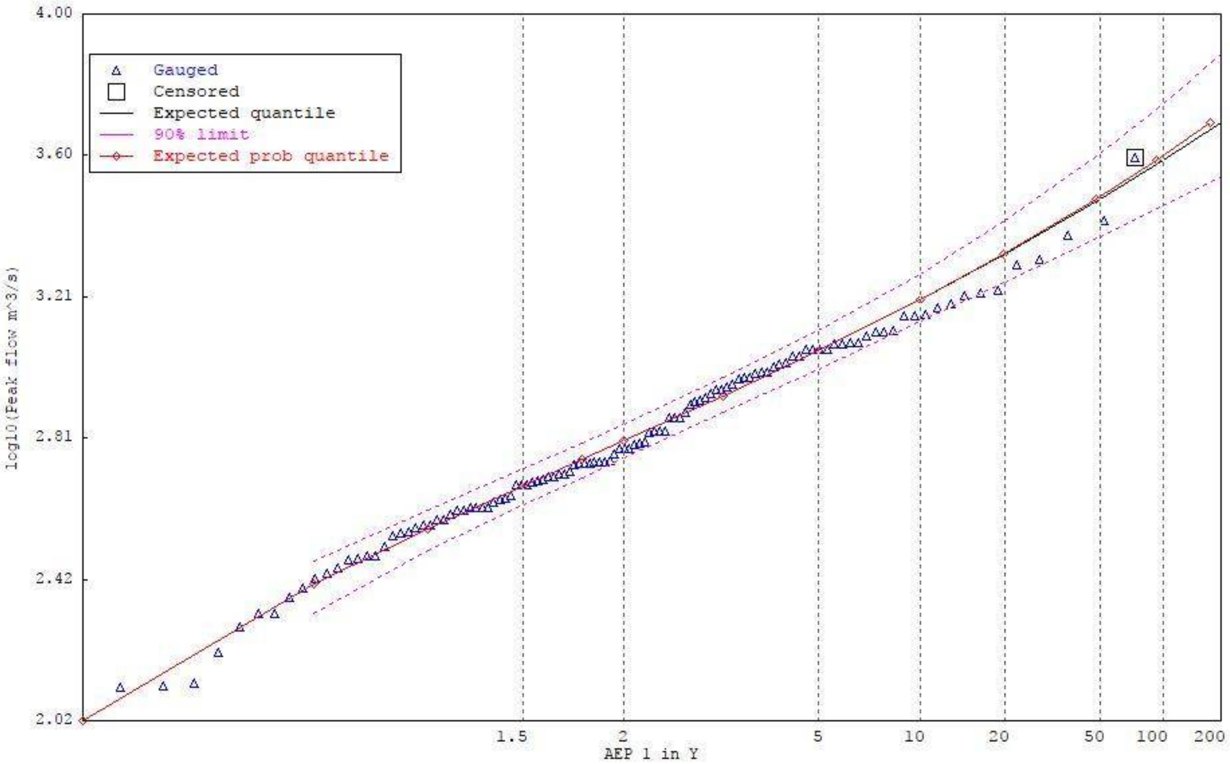


Figure 3-3 South Esk River at Lake Trevallyn Spillway Results – GEV

3.2.2.6 Flood Frequency Analysis Comparison

A comparison of the HydroTAS (2008) and the revised FFA results is provided in Table 3-10. As shown in Table 3-10, the revised FFA results are significantly higher than those from HydroTAS (2008), particularly for the rarer flood events (2%, 1% and 0.5% AEP events).

As shown in Table 3-10 the difference in the FFA results is mainly due to the inclusion of the 1852 and 1863 historic events. For HydroTAS (2008) it was considered that the inclusion of the historic flood events (1800s) skewed the results. However, now historic flood events can be included as censored data in Flike. Further, Munro (1959) stated that '*the weight of evidence now available seems to point to the 1863 flood being the first in an array of flood magnitudes, with the 1852 slightly bigger than 1929*'.

The adoption of the GEV probability model also increased the flow estimates.

Table 3-10 South Esk River at Lake Trevallyn Spillway FFA Comparison

AEP	Adopted Peak Flow (m^3/s)	HydroTAS (2008) Peak Flow (m3/s)	Peak Flow Estimate without 1800's Historic Events (LP3) (m^3/s)	Peak Flow Estimate LP3 (m^3/s)
20%	1,147	1,090	1,118	1,174
10%	1,594	1,430	1,192	1,623
5%	2,132	1,810	1,895	2,137
2%	3,034	2,330	2,480	2,933
1%	3,902	2,910	2,968	3,639
1 in 200	4,975	3,430	3,499	4,448

3.2.2.7 Comparison to Historic Flood Events

The results of the flood frequency analysis for the South Esk River (Table 3-9) can be used to estimate the AEPs for the historic flood events that have been recorded on the South Esk River. This analysis (Table 3-11) shows the 1929 flood event is estimated as being the 1% AEP flood event, whilst the largest event on record (1863) is estimated as being a 1 in 140 AEP flood event.

Table 3-11 AEP Estimates for Historic Flood Events – South Esk River

Historic Event	Flow (m^3/s)	AEP Estimate
1863	4,625	1 in 140
1852	4,190	1 in 110
1929	3,964	1%
1969	2,636	3%
2016	2,398	4%
1926	2,064	5%
1931	1,997	6%
1970	1,696	9%
1974	1,668	9%
2011	1,629	10%

3.3 Design Event Inflow Hydrographs

To determine the design event inflow hydrograph shapes and timing for the hydraulic modelling, historic hydrographs were analysed to determine average hydrograph shapes and timing.

Downstream of the Corra Linn stream gauge, the Rose Rivulet flows into the North Esk River. To determine the influence of inflows from Rose Rivulet on flood levels in the North Esk River, recorded flood levels at the Johnston Road bridge were assessed to determine if there was a significant increase in water level prior to the North Esk River flows arriving. No significant increase in recorded water level were observed. Therefore, an inflow boundary for the Rose Rivulet has not been established (this approach is consistent with BMT WBM (2008).

3.3.1 North Esk River at Corra Linn Hydrograph

To determine the design hydrograph shape for the North Esk River at Corra Linn inflow boundary, the largest recorded historic flood event hydrographs (appropriate data was available for six events) were time corrected and factored to the 1% AEP peak as determined by the FFA (Section 3.2.1.5). As shown in Figure 3-4, the average was then taken to develop the design hydrograph for the 1% AEP design event.

The 1% AEP design event hydrograph was factored for the other AEP design events assessed.

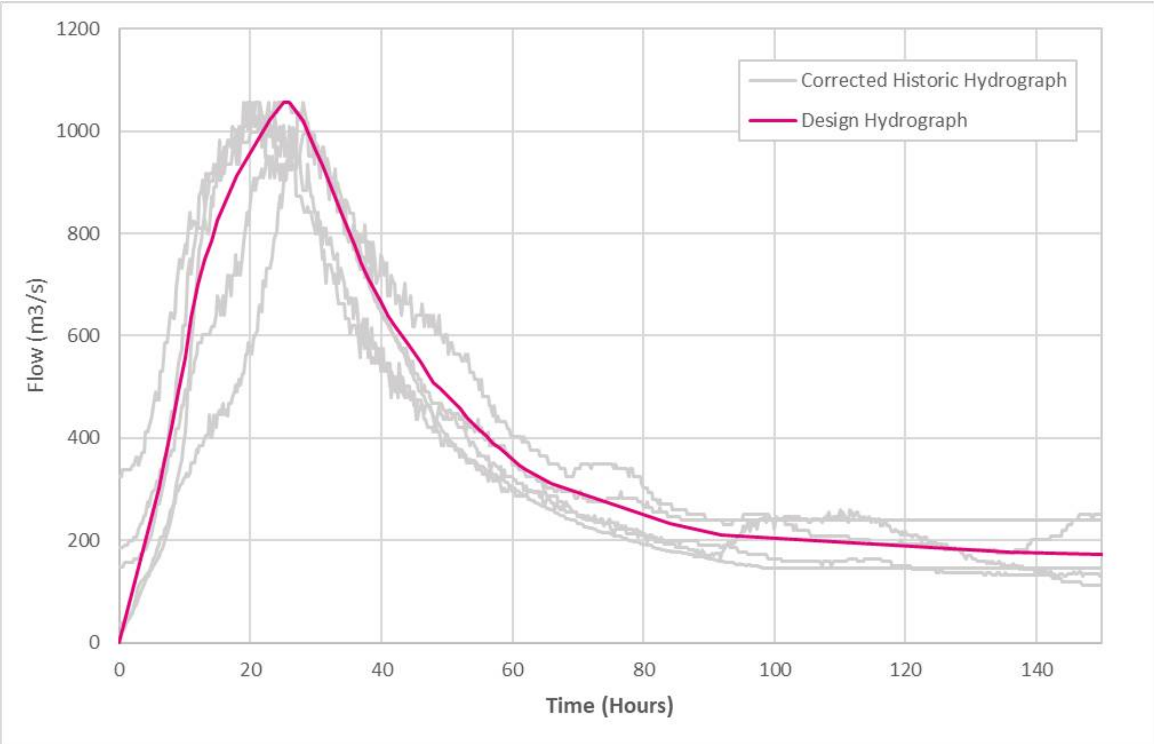


Figure 3-4 North Esk River at Corra Linn 1% AEP Design Hydrograph

3.3.2 South Esk River at Lake Trevallyn Spillway Hydrograph

To determine the design hydrograph shape for the South Esk River at Lake Trevallyn Spillway inflow boundary, the largest recorded historic flood event hydrographs (10 events) were time corrected and factored to the 1% AEP peak as determined by the FFA (Section 3.2). As shown in Figure 3-5, the average was then taken to develop the design hydrograph for the 1% AEP design event.

The 1% AEP design event hydrograph was factored for the other AEP design events assessed.

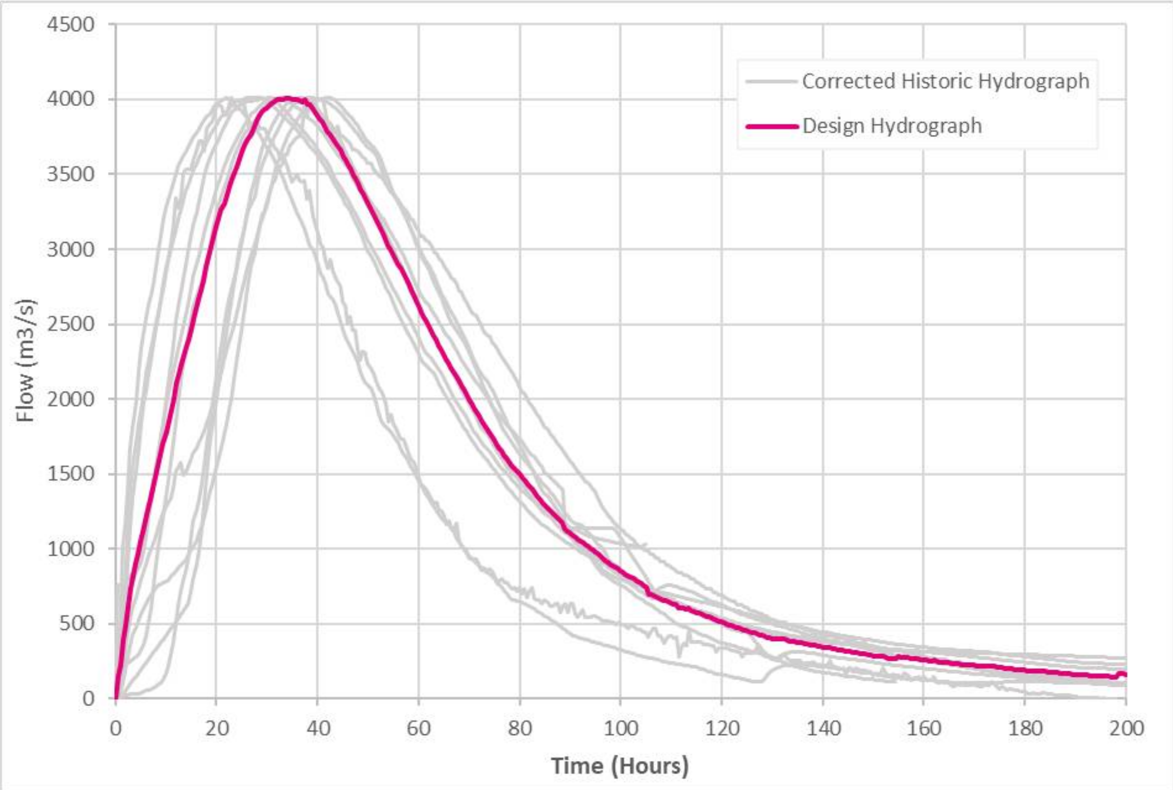


Figure 3-5 South Esk River at Lake Trevallyn Spillway 1% AEP Design Hydrograph

3.3.3 Hydrograph Timing

To determine the hydrograph timing, i.e. the lag between North Esk River flows and South Esk River flows, an assessment of the average lag time for historic flood events was undertaken. This assessment showed that for events where data was available for comparison for catchment wide flood events from 1990 – 2016, that the average lag time is approximately 20 hours. However, given that the only rare flood event in this period occurred in 2016 had a lag of approximately 30 hours which matches Council's previous recommendation (BMT WBM 2008) a hydrograph timing difference of 30 hours has been adopted for this study.

3.4 Increased Rainfall Intensity Modelling

To determine flow estimates for the PMF and, 2050 and 2090 climate conditions design flood events a basic RORB hydrological model was developed.

In consultation with Council, it was determined that due to time constraints it was not necessary to include dams (including flow release operating procedures), natural storages or flow diversions in the RORB for estimating PMF flows and assessing increases in rainfall intensity. As a result, the RORB model was not calibrated and validated against historic flood events, rather it was validated against FFA design event flow estimates (Section 3.2).

3.4.1 Model Setup

A RORB model was developed for both the North and South Esk River catchments, covering an area of approximately 850 km^2 and 8,990 km^2 respectively (Figure 3-6).

The catchment and sub-catchment boundaries were initially determined using the software package Encom Discover, based on the SRTM dataset. The initial catchment and sub-catchment boundaries determined using Encom Discover were then refined to ensure consistency in sub-catchment size. The sub-catchment breakup is shown in Figure 3-6. As predominately rural and conservation area catchments, a fraction impervious of 0 was adopted.

RORB Reach Type 1, natural channels, was used in the RORB model setup.

3.4.2 Design Rainfall

Design event rainfall was used to validate the RORB models to the FFA design event flow estimates.

To define the design rainfall for AEP events, rainfall depths for the North Esk and South Esk catchment were generated using the Bureau of Meteorology's 2016 Rainfall IFD Data System (http://www.bom.gov.au/hydro/has/cdirswebx/cdirswebx.shtml), which is recommended for use by ARR 2016.

3.4.2.1 Temporal Patterns

The 10 temporal patterns for the Southern Slopes (Tasmania) region, based on the methodology developed in Chapter 3 of ARR 2016 Book 2 were adopted. Each of the 10 temporal patterns was used to determine the median temporal pattern for each event duration.

3.4.2.2 Areal Reduction Factors

The Areal Reduction Factors (ARFs) for the Southern Temperate ARF region, based on the methodology developed in Chapter 4 of ARR 2016 Book 2 were adopted.

3.4.2.3 Spatial Rainfall Patterns

Chapter 6 of ARR 2016 Book 2 states that as a minimum it is recommended that a single non-uniform spatial pattern is applied to catchments with an area greater than 20 km^2. Given that the intention is to validate the design event hydrographs to the FFA results it was determined unnecessary to develop spatial patterns.

3.4.3 Validation and Parameters

As it was beyond the scope of the modelling update to calibrate the RORB model to historic flood events, the model was validated against the FFA results (Section 3.2). RORB can be validated by varying the FI, initial loss, continuing loss, reach type, kc and m.

FI and reach type were defined to represent the catchment characteristics as described in Section 3.4.1 and are considered "fixed" parameters. Regional initial losses, including an allowance for pre-burst depths, were adopted from the ARR Data Hub tool (http://data.arr-software.org). A value of 0.8 was used for the m parameter as recommended by the RORB manual for ungauged catchments such as these.

The RORB models were validated to the 1% AEP FFA flow estimates by varying the continuing loss and kc parameters for all events and temporal patterns. It was found that the 24 hour rainfall event is critical in the North Esk River catchment to Corra Linn and the 72 hour rainfall event is critical in the South Esk River catchment to Lake Trevallyn. Continuing losses were then adjusted to validate peak flows against the FFA for all other AEP events.

The resulting validated parameters are presented in Table 3-12.

Table 3-12 RORB Model Parameters

Parameter		North Esk River	South Esk River
k_c		40	156
m		0.8	0.8
Initial Loss (excluding allowance for pre-burst depth)		18 mm	23 mm
Continuing Loss	20%	2.8	1.6
	10%	2.9	1.9
	5%	3.0	1.9
	2%	3.1	1.9
	1%	3.2	1.9
	1 in 200	3.6	1.9
	1 in 500	3.7	1.5
	1 in 1000	3.8	1.2
	1 in 2000	3.8	0.9

3.4.4 Increased Rainfall Intensity

Climate change or increased rainfall intensity design rainfall was estimated using the ARR 2016 Data Hub BETA tool (http://data.arr-software.org). Increased rainfall intensity factors of 7.2% and 16.1% for the RCP 8.5 climate change scenarios for 2050 and 2090 climate conditions have been adopted.

The resulting increase in peak design event flows for 2050 and 2090 climate conditions is presented in Table 3-14. It should be noted that climate change conditions peak flows are applied to the hydraulic model using the hydrographs presented in Section 3.3.

Table 3-13 Climate Change Conditions Peak Flows

AEP	North Esk River Flow (m^3/s)			South Esk River Flow (m^3/s)		
	Existing Conditions	2050 Climate Conditions	2090 Climate Conditions	Existing Conditions	2050 Climate Conditions	2090 Climate Conditions
20%	400	478	577	1,147	1,429	1,767
10%	529	619	733	1,594	1,877	2,240
5%	670	774	902	2,132	2,463	2,895
2%	878	1,001	1,158	3,034	3,551	4,261
1%	1,056	1,207	1,383	3,902	4,548	5,300
1 in 200	1,252	1,414	1,614	4,975	5,656	6,506
1 in 500	1,543	1,727	1,959	6,796	7,610	8,626
1 in 1000	1,791	2,003	2,266	8,559	9,482	10,633
1 in 2000	2,064	2,309	2,610	10,744	11,776	13,054

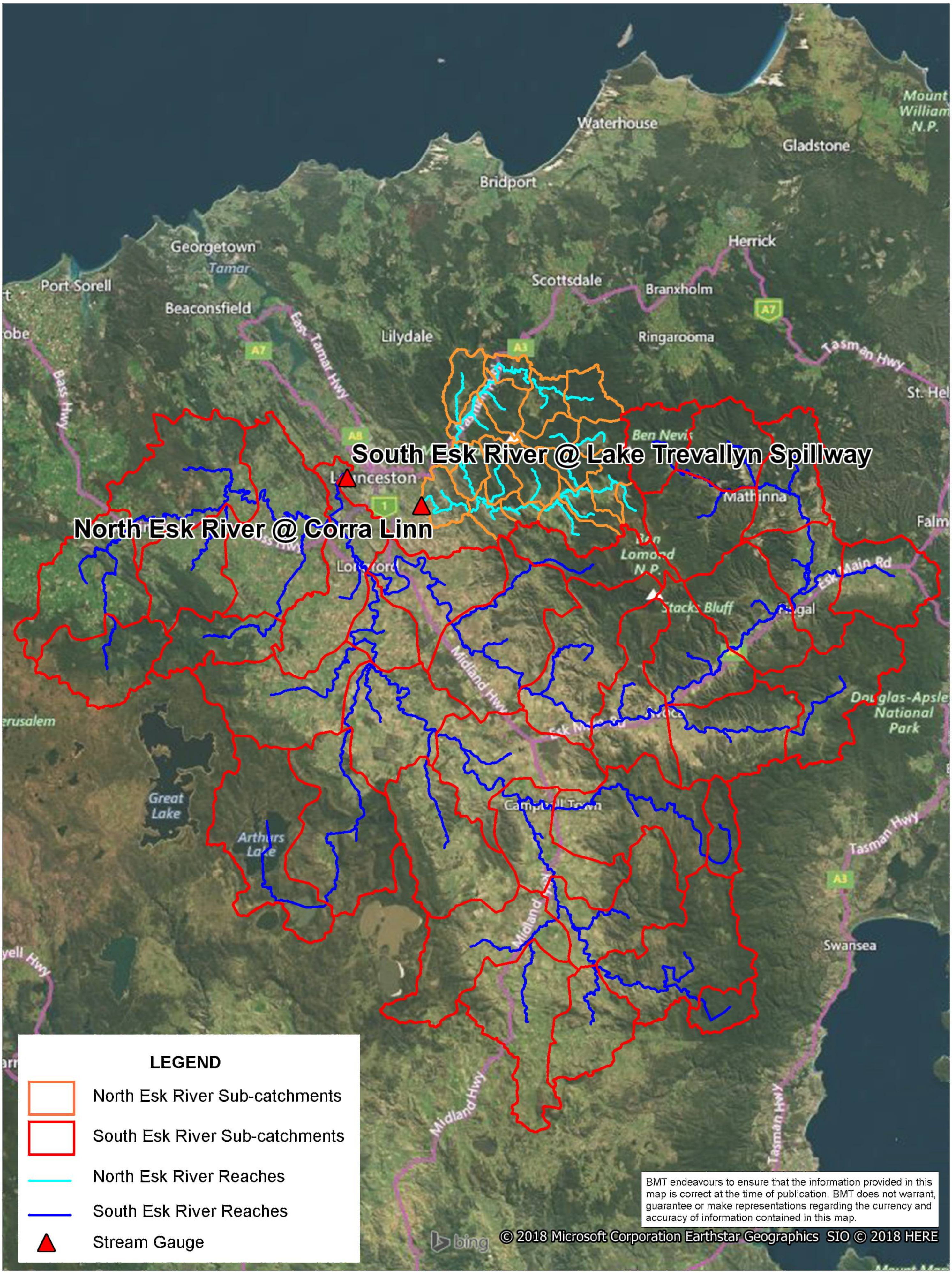


Figure 3-6 RORB Model Layout

T:\M20921.MS.Launceston_Mapping_2017\MapInfo\Drawings\Final\Fig3-6_RORB_RevA.WOR

3.5 Estimation of Probable Maximum Flood

Without a calibrated hydrologic model available, three separate methods were used to ascertain the most appropriate PMF flow estimates for the North and South Esk Rivers:

(1) **Generalised Southeast Australia Method (GSAM)** (BoM 2006). Probable Maximum Precipitation (PMP) rainfall events were derived, and aerially and temporally distributed, using the GSAM and applied to the validated RORB model (Section 3.4) to produce flow estimates.

(2) **A Quick Method for Estimating the PMF in South Eastern Australia** (Nathan et al 1994). Peak flow estimates based on the regression equations (Equation 1) derived from analysis of PMF estimates and can be applied to catchments with an area between 1 and 1,000 km^2.

$$Peak\ Flow = 129.1 Area^{0.616} \qquad \text{Equation 1}$$

(3) **Extrapolation of FFA peak flow estimates**. Peak flow was estimated by assigning an AEP to the PMF (assumed to be equal to the PMP) of 9.0e-^{7} for the North Esk River and 9.0e-^{6} for the South Esk River using Figure 8.3.2, Book 8, Chapter 3.4 of ARR 2016 and extrapolating the FFA results using a Log$_{10}$ series.

The PMF estimates for each of the three methods are presented in Table 3-14. With a calibrated hydrologic model available, the GSAM method is considered the most robust and is recommended in ARR 2016. However, the flow estimates derived by applying GSAM rainfall to the un-calibrated South Esk River RORB model are significantly higher than the other estimation methods. Therefore, it was decided to adopt the PMF estimate derived from the extrapolation of FFA (incorporates approximately 200 years of historic flood data) peak flow estimates for the South Esk River.

For the North Esk River, the GSAM method was adopted. It is considered that the validated RORB model is more accurate in the North Esk River catchment due to the lack of major storages.

Table 3-14 Comparison of PMF Peak Flow Estimate Methods

	GSAM (m^3/s)	Quick Estimate Method (m^3/s)	Extrapolation of FFA (m^3/s)
North Esk	**11,405**	8,230	5,810
South Esk	65,512	35,190	**38,520**

4 Hydraulic Modelling

TUFLOW, a fully 2D hydraulic modelling package with the ability to dynamically next 1D elements was adopted for this study. The model covers approximately 73 km^2 of the North and South Esk Rivers, the Tamar River and the adjacent floodplain. As shown in Figure 4-1, the model extends from the Corra Linn stream gauge on the North Esk River (Blessington Road) and Kings Bridge on the South Esk River to approximately 4.5 km downstream of Tamar Island on the Tamar River. To balance the model run times whilst still providing an accurate representation of the river system and floodplain, the model is based on a 10 m grid. Each square grid element contains information on ground topography sampled from the DEM at 5m spacing, surface resistance to flow (Manning's n value) and initial water level.

1D elements were embedded in the 2D domain to represent the North Esk River gorge at the Corra Linn (upstream boundary) and key culvert structures.

4.1 TUFLOW Model Version

Model runs were undertaken using the 2017-09-AC-iSP-w64 build of TUFLOW. The HPC ('Heavily Parallelised Computing') TUFLOW modelling engine has been used for this study as it uses a second-order solution scheme that provides high numerical accuracy, similar to the standard TUFLOW Classic model whilst utilising a computers graphics processing unit (GPU) to deliver speed increases of 10 to 100 times.

4.2 Topography

Council provided a 1 m gridded DEM of LiDAR captured in 2013. The 2013 LiDAR data provided the base topography for the 2D model domain but was supplemented with the following topography modifications:

- Bathymetry survey captured in March 2017 of the Tamar River from River Street, extending up the North Esk River to Scotch Oakburn Park
- Bathymetry used in BMT WBM (2008) of the Tamar River from River Street to the downstream model boundary, and in the North Esk River from Scotch Oakburn Park to Old Mac's Farm and Fishery
- Ground survey of the Landsborough Avenue development
- Proposed design ground levels of the Northbank Re-Development Site

Council provided levee crest survey of the existing Launceston levee system that was included in the model using 3D breaklines. To ensure that the flood mapping does not become outdated in the known future, the proposed Northbank Levee was lowered to a level of 4.2 m AHD and the proposed Newstead levee was included in the model.

Ground survey was used to reinforce significant hydraulic features in such as major roads, railway lines and the silt pond in the hydraulic using 3D breaklines.

4.3 Manning's n Coefficients

The roughness layer, or Manning's 'n' layer, is based on areas of different land-use type, determined from, planning maps, aerial photography and site inspections. The adopted values are summarised in Table 4-1 and the Manning's layer is shown in Figure 4-2. The values used are based on the previous calibrated TUFLOW model (BMT WBM 2008) and standard texts such as Chow (1959) and were verified as part of the calibration process (Section 4.6).

Table 4-1 2D Domain Manning's 'n' Values

Land Use	Manning's n
Roads	0.020
Residential	0.200
Commercial and Industrial	0.400
Schools, Hospitals, Other Pubic Buildings	0.300
Grass/Pasture	0.040
Low Density Vegetation	0.050
Medium Density Vegetation	0.070
Clean Straight Stream	0.030
Straight Stream with Stones and Weeds	0.035
Tamar Estuary Channel	0.015-0.016
Tamar Estuary Edge	0.025
Home Reach Channel	0.028
Home Reach Edge	0.027
Lower North and South Esk	0.022

4.4 Hydraulic Structures

There are 12 existing bridges across the North Esk River (Figure 4-1). These bridges have been included in the hydraulic model using TUFLOW's 2D layered flow constriction approach with varying deck heights elevations as required. Form loss co-efficient for bridge piers were determined using the methodology outlined in AustRoads (1994). The proposed Inveresk to Lawrence St pedestrian bridge was not included as the available concept preliminary design drawings are insufficient to adequately represent the bridge structure in the hydraulic model.

Key culverts and smaller railway bridges throughout the North Esk River floodplain were also included in the model. Culverts are included in the model as embedded 1D elements. The flow through the structures has been assumed to be unimpeded by the presence of flood debris (there are no blockage factors applied to any structures).

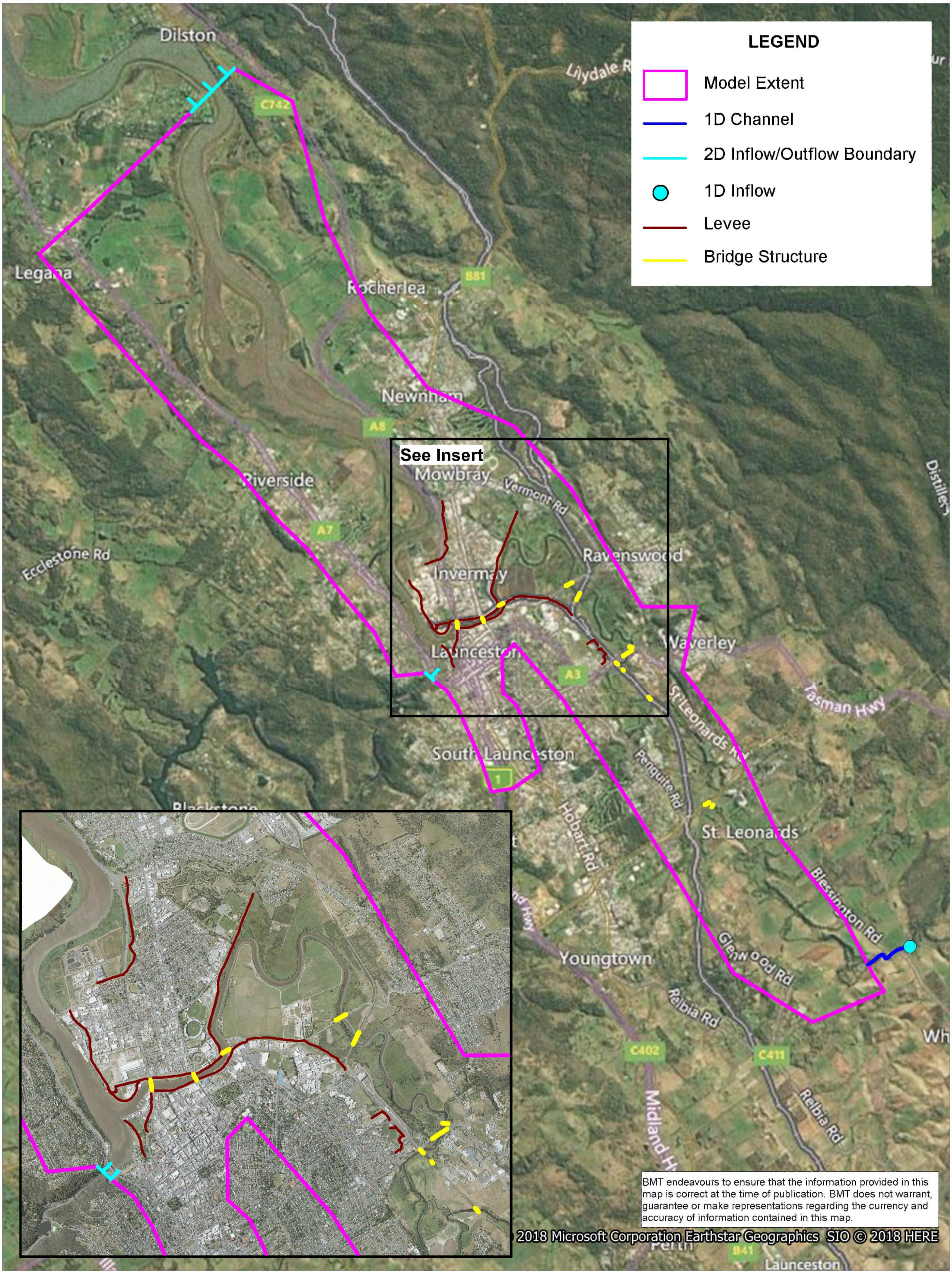


Figure 4-1 TUFLOW Hydraulic Model Layout

T:\M20921.MS.Launceston_Mapping_2017\MapInfo\Drawings\Final\Fig4-1_TUFLOW_RevA.WOR

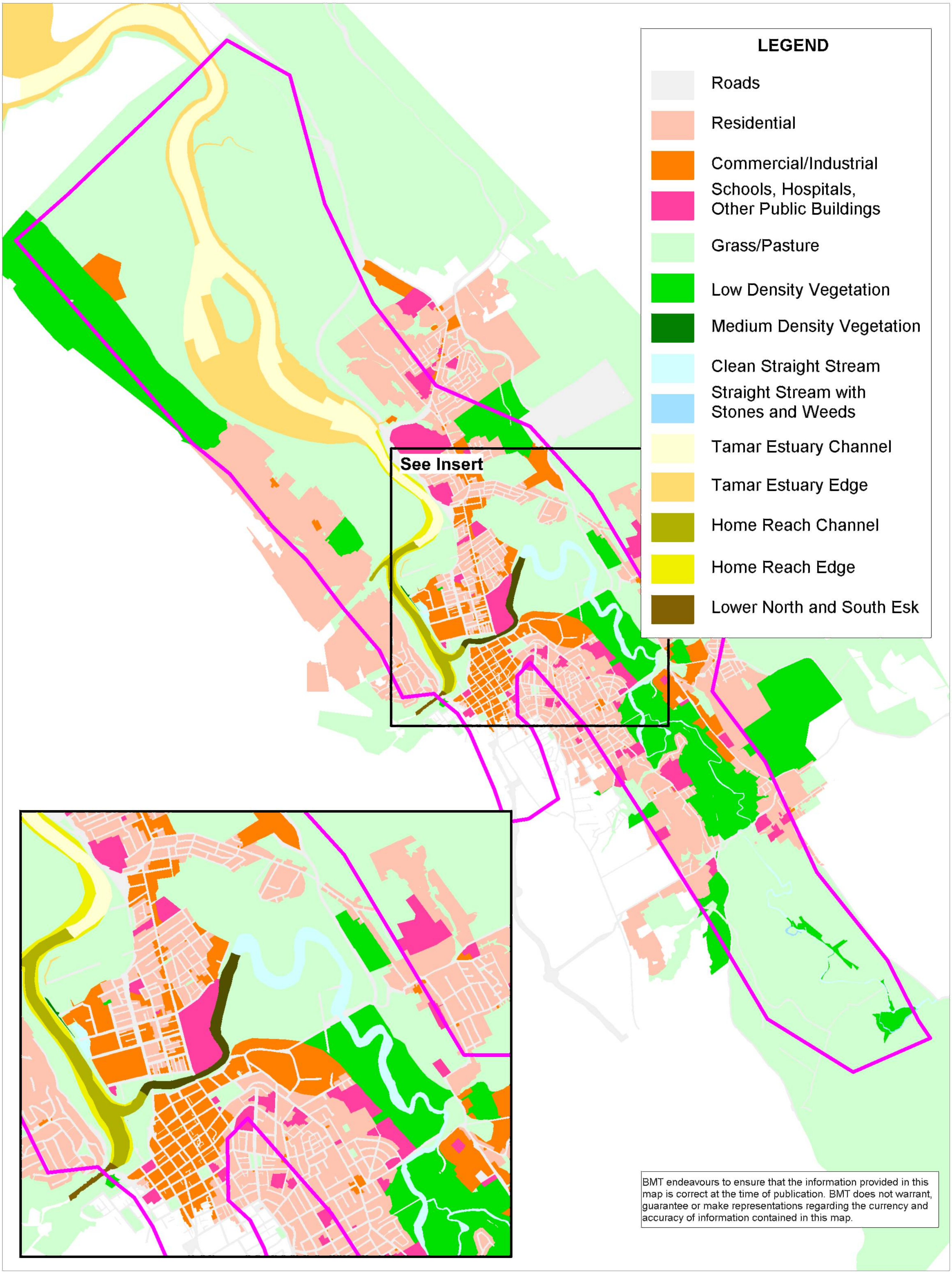


Figure 4-1 TUFLOW Hydraulic Model Layout

T:\M20921.MS.Launceston_Mapping_2017\MapInfo\Drawings\Final\Fig4-2_Mannings_RevA.WOR

4.5 Boundary Conditions

The hydraulic model has two inflow boundaries, one on the North Esk River at the Corra Linn stream gauge and the other on the South Esk River at Kings Bridge, and a downstream tidal boundary on the Tamar River approximately 4.5 km downstream of Tamar Island (Figure 4-1).

4.5.1 River Inflow Boundaries

As detailed in Section 3, peak design event flows for the North and South Esk Rivers have been defined using FFAs with hydrograph timing and shape synthesised using historic flood events. The boundaries are applied to the hydraulic model as QT (flow - time) boundaries. As described in Section 2.5.1.1, the North Esk River flow boundary was applied to a 1D model domain as it was found to better represent flow through the gorge at the Corra Linn gauge. The South Esk River boundary was applied directly to the 2D domain.

As part of the June 2016 event model calibration process (Section 4.6) it was found the flows from tributaries of the North Esk River that outflow downstream of Corra Linn such as Rose Rivulet and Distillery Creek do not impact on peak flood levels of catchment wide (North and/or South Esk River flood events). Therefore, inflows for these tributaries were not determined or applied to the hydraulic model.

4.5.2 Tide Boundary

As discussed in more detail in Section 5.1, there is a tidal influence on flood levels in the Tamar River and lower North Esk River. Tide levels at Low Head (George Town) for the 10%, 5%, 2% and 1% AEP storm surge events were developed for *The Climate Futures for Tasmania: extreme tide and sea-level events* study (McInnes, et al 2012). Linear interpolation of the Log_{10} series of these levels was used to provide tide levels at Low Head for the full suite of design events presented in Table 4-2.

Consistent with BMT WBM (2008), the tidal boundary was synthesised for Low Head using a simple harmonic spring tide that was then corrected to match the design event peak storm surge levels (Table 4-2). A review of the June 2016 flood event storm surge showed that surge lasted for a period of approximately 5 days (10 tide cycles). For this reason, a constant tide harmonic was adopted for design event modelling. The design tide cycle of for the 1% AEP storm surge event at low head is shown in Figure 4-3.

As described in BMT WBM (2008), the tide cycle amplifies with passage along the estuary (from Low Head to Launceston) and there is a progressive lag and change in shape of the predominantly sinusoidal oceanic tide. To model the tide passage from Low Head to the downstream boundary of the flood model, the TUFLOW FV estuary model developed by BMT (BMT WBM 2008) was used. The resulting peak storm surge tide levels at the flood model downstream boundary and Launceston (Home Reach) are shown in Table 4-2. Figure 4-3 shows a comparison of the 1% AEP storm surge event design tide cycle at Low Head, the flood model downstream boundary and Launceston.

For the design event modelling, the timing of the tide was adjusted such that the peak tide level at Launceston coincided with the peak flood flow at Launceston.

Table 4-2 Design Event Storm Surge Levels

	AEP Peak Tide Level (m AHD)								
	20%	10%	5%	2%	1%	1 in 200	1 in 500	1 in 1000	1 in 2000
Low Head	1.87	1.89	1.91	1.94	1.96	1.98	2.01	2.03	2.05
DS Boundary	2.03	2.04	2.06	2.08	2.09	2.11	2.13	2.14	2.16
Launceston	2.10	2.12	2.13	2.15	2.17	2.18	2.20	2.22	2.23

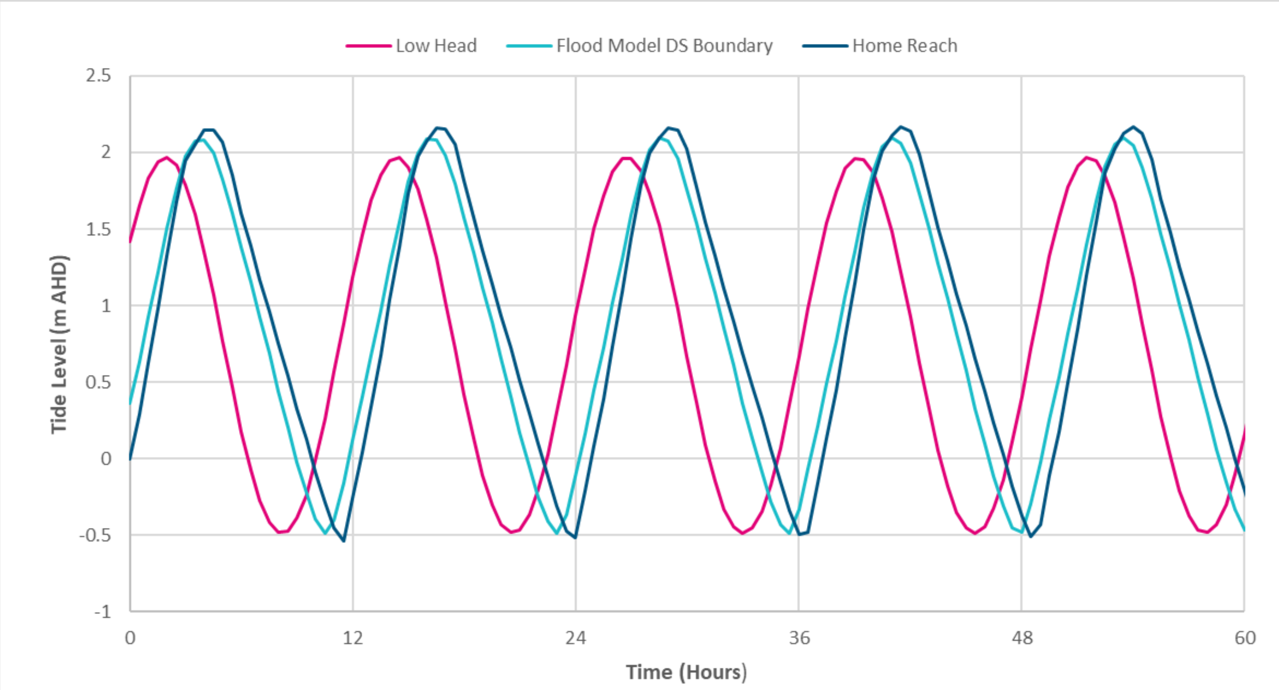


Figure 4-3 1% AEP Tide Comparison

4.5.2.1 Sea Level Rise

The Tasmanian Government has provided sea level rise allowances for each coastal municipality based on the Intergovernmental Panel on Climate Change Fifth Assessment Report (IPCC AR5) RCP 8.5 emissions scenario. For Launceston, sea level rise allowance of 0.22 m and 0.72 m for 2050 and 2090 climate conditions are prescribed (McInnes, et al 2016). The resulting tide levels at Launceston are presented in Table 4-3.

Table 4-3 Sea Level Rise Tide Levels at Launceston

	AEP Peak Tide Level (m AHD)								
	20%	10%	5%	2%	1%	1 in 200	1 in 500	1 in 1000	1 in 2000
2050	2.32	2.34	2.35	2.37	2.39	2.4	2.42	2.44	2.45
2090	2.82	2.84	2.85	2.87	2.89	2.9	2.92	2.94	2.95

4.5.3 Initial Water Levels

Initial water levels can be set within a TUFLOW model to reduce the risk of initial model instability caused by large differences between water level in the downstream boundary and adjacent ground levels. Therefore, initial water levels equal to the starting level of the design tide cycles are applied to the model.

4.6 June 2016 Model Calibration

Model calibration provides an overall check of the reliability of a model in representing the flow conditions of the physical system by comparing model results against measured flood levels and extents and adjusting model parameters to obtain a “best-fit” (Ball et al. 2016).

Following the June 2016 flood event, Council was able compile set of historic flood event data for model calibration including:

- 44 surveyed flood marks
- River level data at the Johnston Road Pipe Bridge (including flow), Hoblers Bridge Road, Henry Street and Tamar Street on the North Esk River, along with level and flow data at model inflow boundaries at Corra Linn and Lake Trevallyn Spillway.
- A GIS flood extent digitised by Council from observed debris marks
- Aerial flood photography captured from a helicopter during the event

Following initial model calibration runs it became evident that the flows estimated for the North Esk River at Corra Linn were far too low. As a result, the North Esk River gorge was converted from the 2D model domain to a 1D domain in the hydraulic model and the rating curve at Corra Linn was revised resulting in larger flow estimate for the June 2016 flood event. Further detail of this process is provided in Section 2.5.1.1.

Following the revision of the North Esk River flow estimates, a trend was observed that resulted in higher than recorded flood levels along the North Esk River upstream of the railway bridge adjacent to Sandown Road and lower than recorded downstream of the bridge. This location is a significant constriction on the North Esk River floodplain with an active flow width of only 80 m until the railway embankment is overtopped. It was therefore determined that the channel bathymetry required review. This location is upstream of the extent of 2017 bathymetry survey, so the bathymetry used in BMT WBM (2008) was used to define the North Esk River channel from Scotch Oakburn Park to Old Mac’s Farm and Fishery. The resulting bathymetry was then lowered by a further 1 m in the vicinity of the bridge to represent the localised ‘hole’ in the river channel resulting from high flow velocities through the constriction.

After making the above two adjustments to the hydraulic model, the resulting modelled flood levels were in close enough agreement to those recorded, that model parameters such as surface roughness and bridge losses could be calibrated/validated.

The calibration results for recorded water levels at the Johnston Road Pipe Bridge, Hoblers Bridge, Henry Street and Tamar Street stream gauges are shown in Figure 4-4 to Figure 4-7 respectively. These figures show good agreement between the recorded and modelled flood levels for both flood level and timing.

Figure 4-8 shows a comparison of 37 surveyed flood marks (7 of the 44 surveyed flood marks were beyond the model extent) to peak modelled flood levels adopting a tolerance of ± 0.2 m. As per the legend, a positive number (coloured orange/red) indicates that the modelled level is higher than the recorded level, while a negative number (coloured green) indicates that the modelled level is lower than the recorded level. The yellow colour indicates where the model is within the ± 0.2 m tolerance. The results of the flood mark comparison are also presented in Table 4-4 with a histogram provided in Figure 4-9.

The flood mark comparison shows that at 59% of the flood marks the model is within ± 0.2 m tolerance with an absolute average difference of 0.21 m and a standard deviation of 0.26 m. As shown in Figure 4-9 the flood model tends to over-represent flood levels resulting in an average flood mark difference of + 0.10 m. In general, the modelled flood levels in the North Esk River upstream of Black Bridge are higher than those recorded while those in the Tamar River and the North Esk River downstream of Black Bridge are lower.

Figure 4-8 also shows a comparison of the flood extent derived from debris marks and the modelled flood extent. The model shows a very close agreement between the recorded and modelled flood extent except for one area on the North Esk River floodplain immediately downstream of Corra Linn where the recorded flood extent appears to end abruptly. A check of the underlying topography in this area was undertaken and it appears that is most likely due to limitation in the data available in digitising the recorded flood extent.

Overall the hydraulic model shows good agreement to the recorded flood stream gauge levels, flood marks and flood extent and is considered calibrated for the purpose of design event flood modelling and mapping.

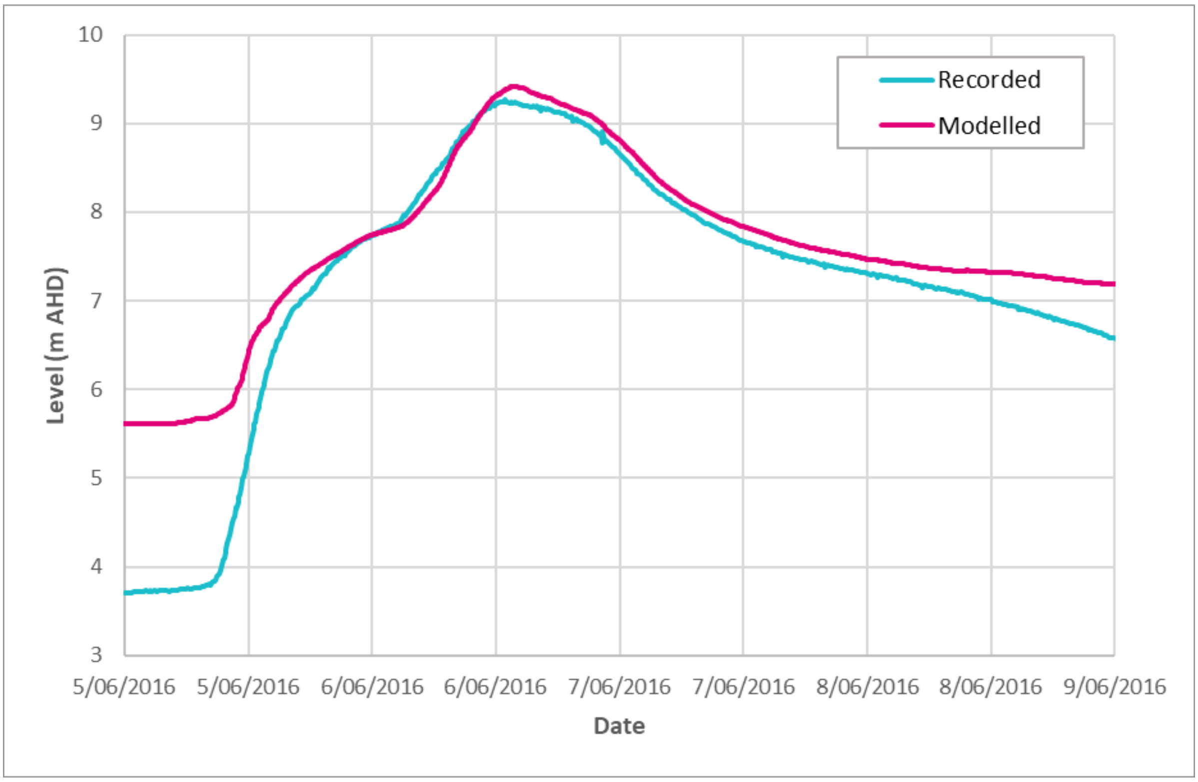


Figure 4-4 June 2016 Recorded and Modelled Levels – Johnston Road Pipe Bridge

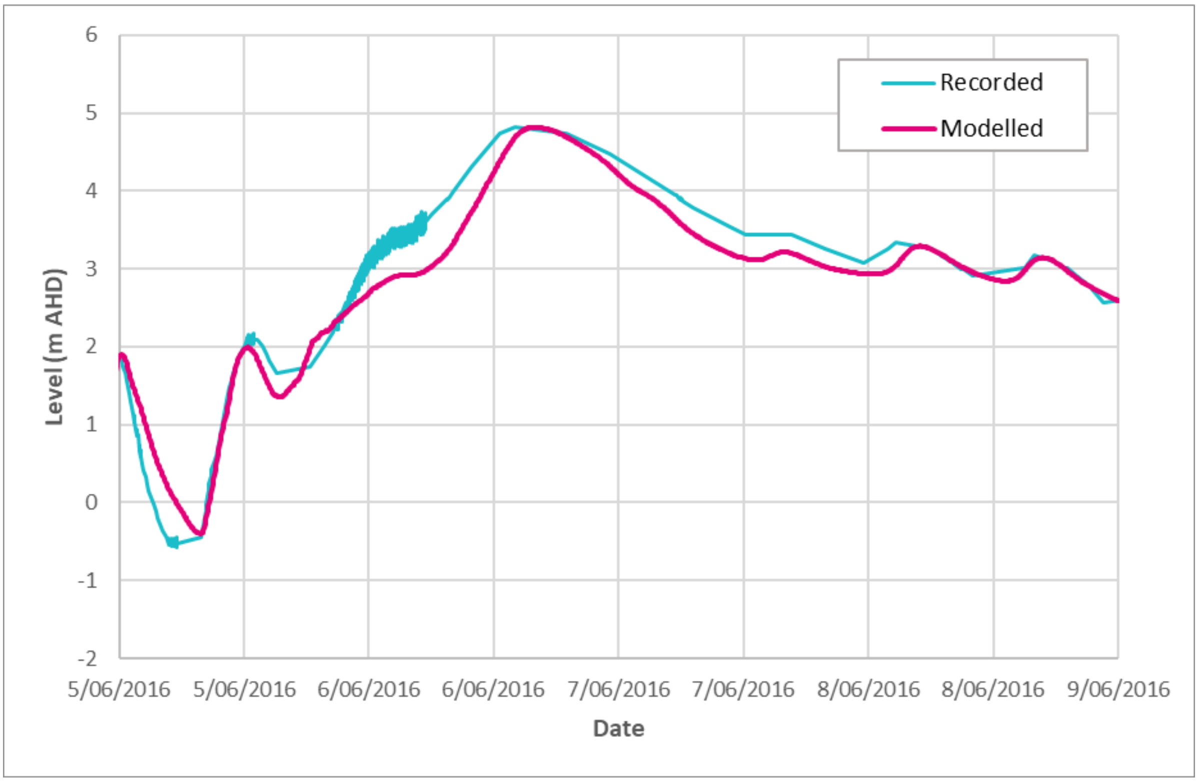


Figure 4-5 June 2016 Recorded and Modelled Levels – Hoblers Bridge

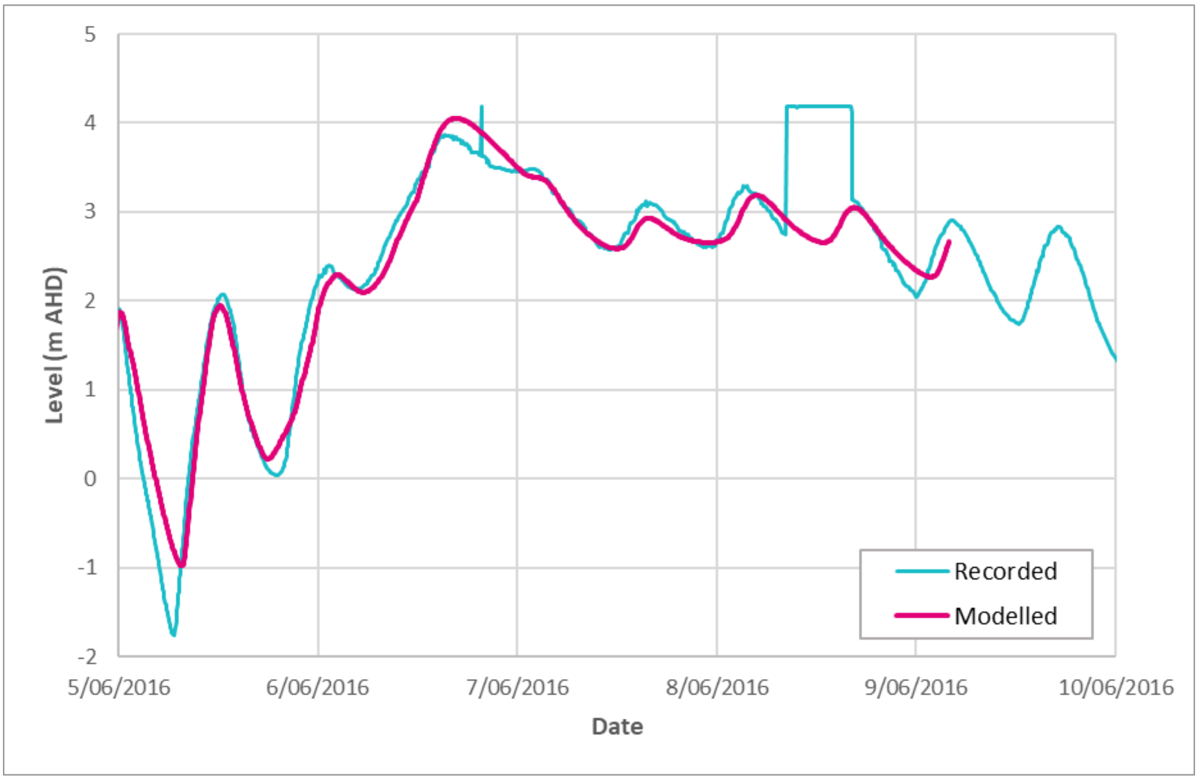


Figure 4-6 June 2016 Recorded and Modelled Levels – Henry Street

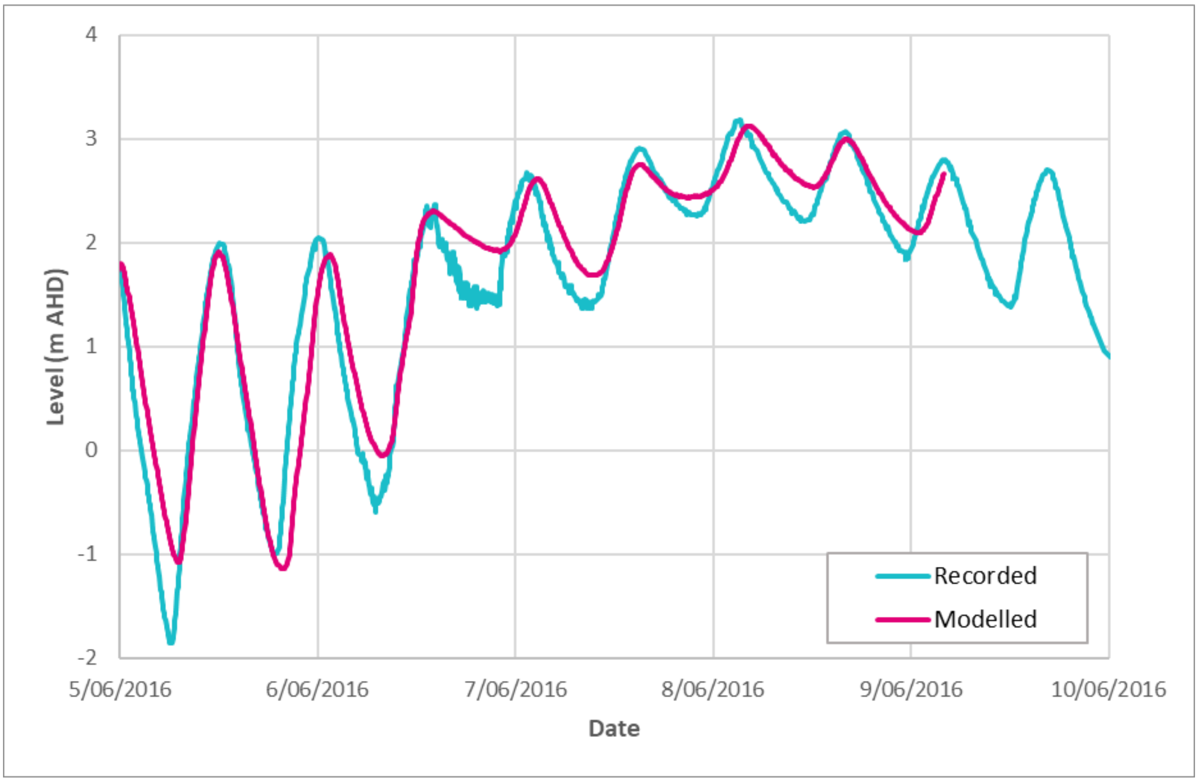


Figure 4-7 June 2016 Recorded and Modelled Levels – Tamar Street

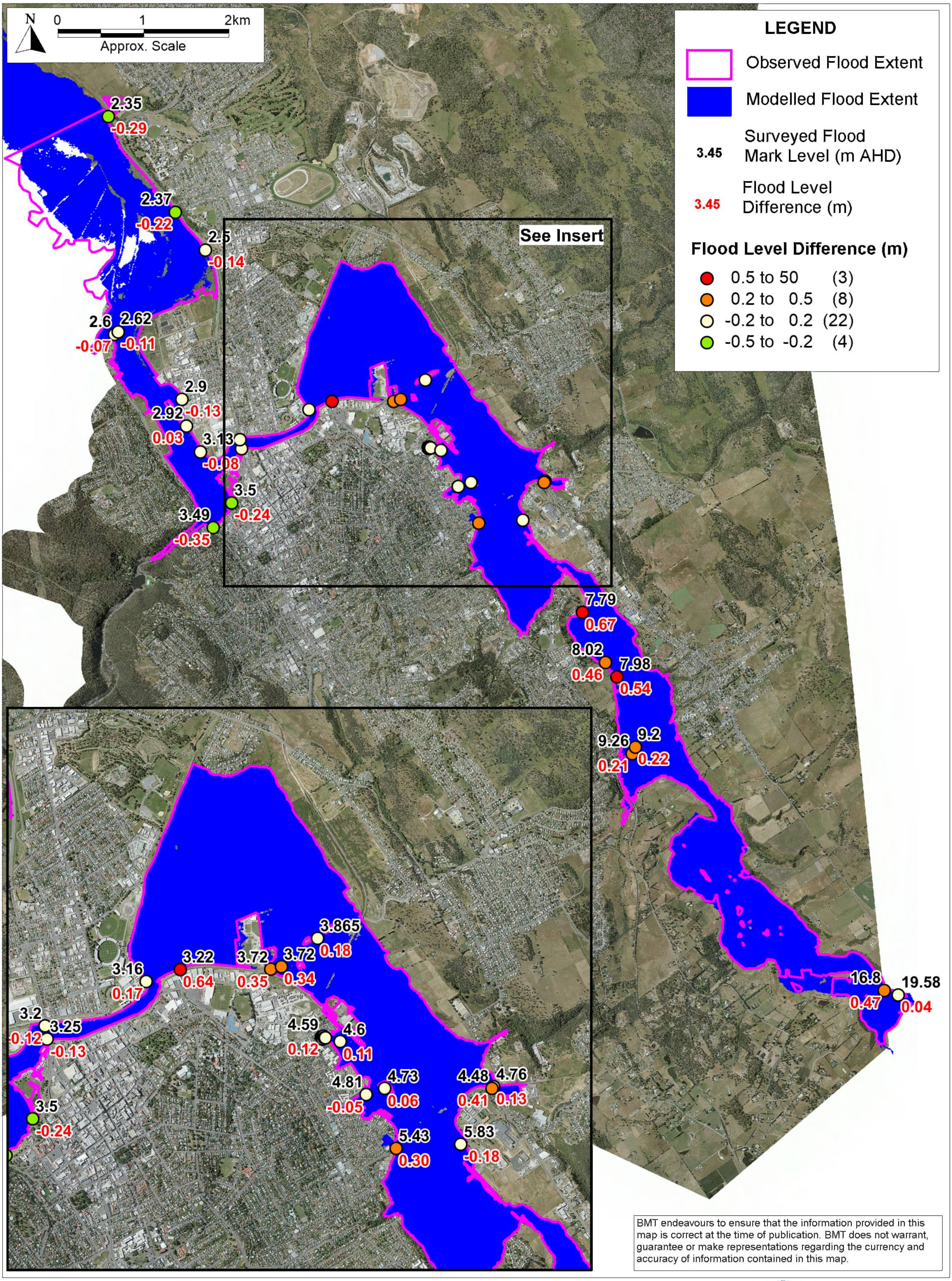


Figure 4-8 June 2016 Calibration Results

T:\M20921.MS.Launceston_Mapping_2017\MapInfo\Drawings\Final\Fig4-8_June2016_RevA.WOR

Table 4-4 June 2016 Flood Mark Comparison Results

Range	Number of Flood Marks within Range	% of Flood Marks within Range
-0.8 m to -0.6 m	0	0%
-0.6 m to -0.4 m	0	0%
-0.4 m to -0.2 m	4	11%
-0.2 m to 0.2 m	21	59%
0.2 m to 0.4 m	5	14%
0.4 m to 0.6 m	4	11%
0.6 m to 0.8 m	2	5%
Average Difference		**+ 0.10 m**
Absolute Average Difference		**0.21 m**
Standard Deviation		**0.26 m**

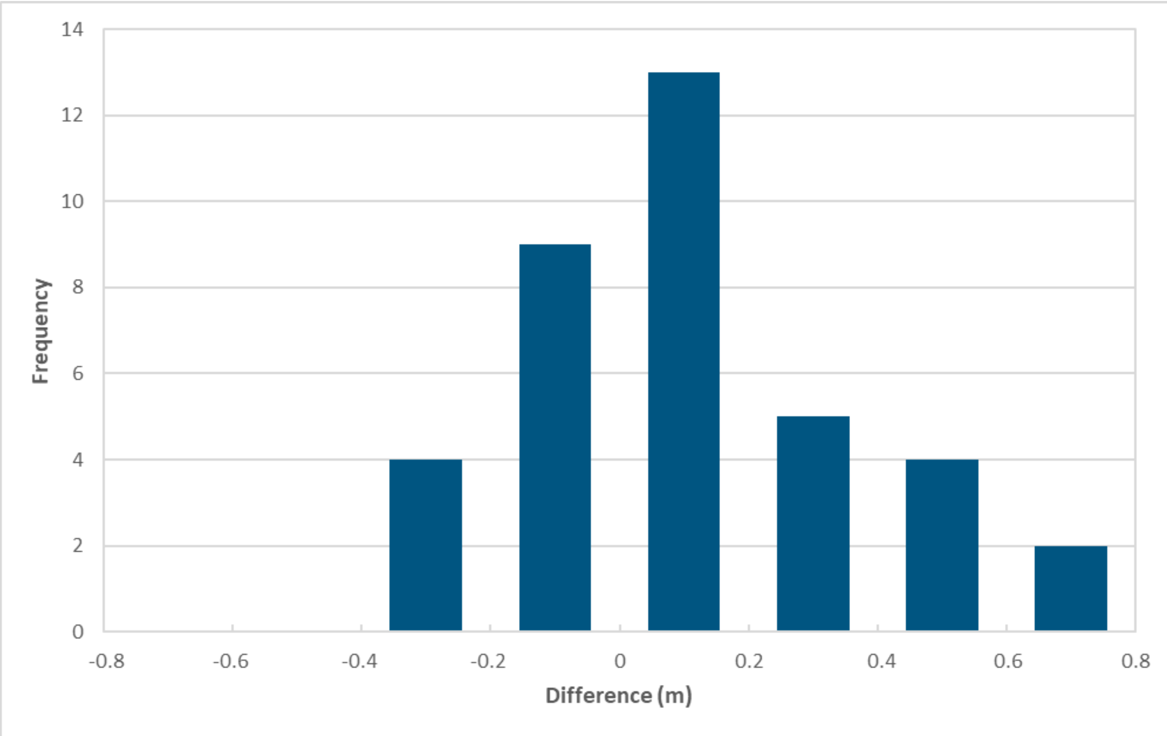


Figure 4-9 June 2016 Flood Mark Comparison Histogram

5 Design Event Mapping

This section provides a brief overview of the floodplain mapping process used for this study. Flood mapping has been produced for each of the events modelled for existing, 2050 and 2090 climate conditions. Flood mapping has been produced for the following design events:

- 20% AEP
- 10% AEP
- 5% AEP
- 2% AEP
- 1% AEP
- 1 in 200 AEP
- 1 in 500 AEP
- 1 in 1000 AEP
- 1 in 2000 AEP
- PMF (Flood Planning Constraint Mapping only)

5.1 Treatment of Joint Probability

Flood risk in Launceston results from a combination fluvial flooding from the North and South Esk Rivers, as well as elevated water levels due to propagation of storm tides upstream along the Tamar River estuary. The joint probability (or dependence) of these three events occurring separately or simultaneously needs to be assessed. Technically, this would be a trivariate probability problem, as it involves three potentially co-dependent events. However, this problem can be tackled using bivariate methods (described in ARR 2016) as further explained in this section.

Book 6, Chapter 5 of ARR 2016 provides practical methodologies, including the Design Variable Method, for assessing the interaction of coastal and catchment flooding in estuarine regions. The first step recommended prior to application of this method consists of a Pre-Screening Analysis to determine whether completion of a more complex Joint Probability Analysis (JPA) is warranted. For the *Pre-Screening Analysis*, it is required to identify the Joint Probability Zone (JPZ); defined as being a '*region in which the dependence between riverine and ocean processes has the potential to influence the design flood level*'.

The concept of the JPZ is illustrated in Figure 5-1, replicated from Figure 6.5.1, Book 6, Chapter 5 of ARR 2016.

It is worth noting that whilst the above definition of the JPZ refers to the influence of ocean processes on flood levels in estuarine regions, the JPZ concept and the Design Variable Method can be also applied to riverine systems where design flood levels around a confluence may be influenced by tributary flows due to storm events in a main river or its tributaries.

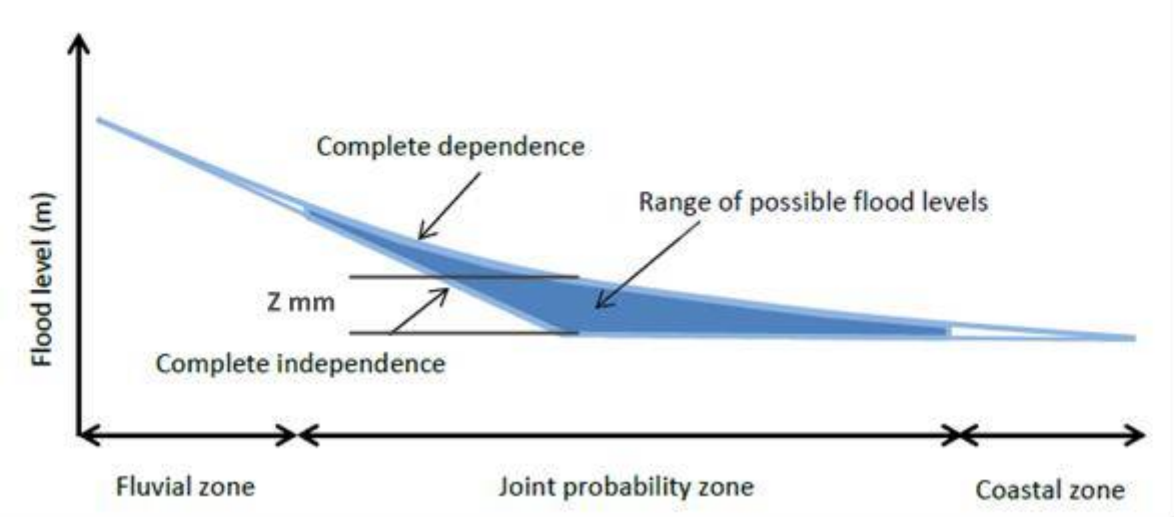


Figure 5-1 Illustration of Joint Probability Zone (Ball, et al. 2016)

5.1.1 Approach Used

According to ARR 2016, if the JPZ is of limited spatial extent and/or the maximum difference in flood levels then a full treatment of joint probability using the methods described in Book 4, Chapter 4 (fluvial) and Book 6, Chapter 5 (tidal) of ARR 2016 is not required. Therefore, a two stage process was applied in this study as follows:

- Stage 1: A Pre-Screening Analysis in which the maximum peak level difference within the JPZ is determined and assessed against an adopted tolerance. If the difference is less than the tolerance, then full dependence (equivalent AEPs) on contributing processes can be assumed and further assessment is not warranted
- Stage 2: If peak level differences in Stage 1 are greater than the adopted tolerance, then proceed to complete a detailed JPA using the Design Variable Method as detailed in ARR 2016

Details of the analyses applied to both the fluvial (North and South Esk River flows) and tidal (storm surge) joint probability considerations is provided in the following sections.

5.1.2 Stage 1 - Pre-Screening Analysis

A pre-screening analysis can be undertaken to assess the influence of joint probability on flood risk in Launceston and whether a full JPA is required, as described in ARR 2016. The following criteria are key in this analysis:

- The extent of the JPZ
- The maximum difference in flood levels (*Z mm*) within the JPZ

A *Z mm* value of 300 mm was adopted on the basis that this value is approximately half of a typical design freeboard of 600 mm used for the levee design in Launceston.

In the case of Launceston, the pre-screening has been applied separately for two combinations; firstly, to determine the JPZ resulting from interaction of storm events in the North Esk and South Esk Rivers (i.e. tributary flows); and secondly, to determine the JPZ resulting from interaction of coastal and fluvial events (i.e. estuarine region).

A graphical representation of the results of the existing conditions 1% AEP pre-screening analysis are presented in Figure 5-2, with the location of the reporting long-section shown in Figure 5-3. Figure 5-2 shows that in the 1% AEP event, the fluvial joint probability zone extends up the North Esk River from downstream of Charles Street (Tamar River confluence) to upstream of Henry Street, while the tidal joint probability zone is downstream of Forster Street.

Whilst Figure 5-2 provides a good visual representation of the pre-screening analysis results, the results differ for each climate scenario and AEP. Table 5-1, Table 5-2 and Table 5-3 show the results of the pre-screening analysis for all AEP events for existing conditions, 2050 and 2090 climate conditions respectively.

As shown in Table 5-1, Table 5-2 and Table 5-3, the joint probability zones vary for each AEP along the North Esk River. Fluvial flows have a greater influence on events greater than 5% AEP while tidal levels have a greater influence in the 20% and 10% AEP events.

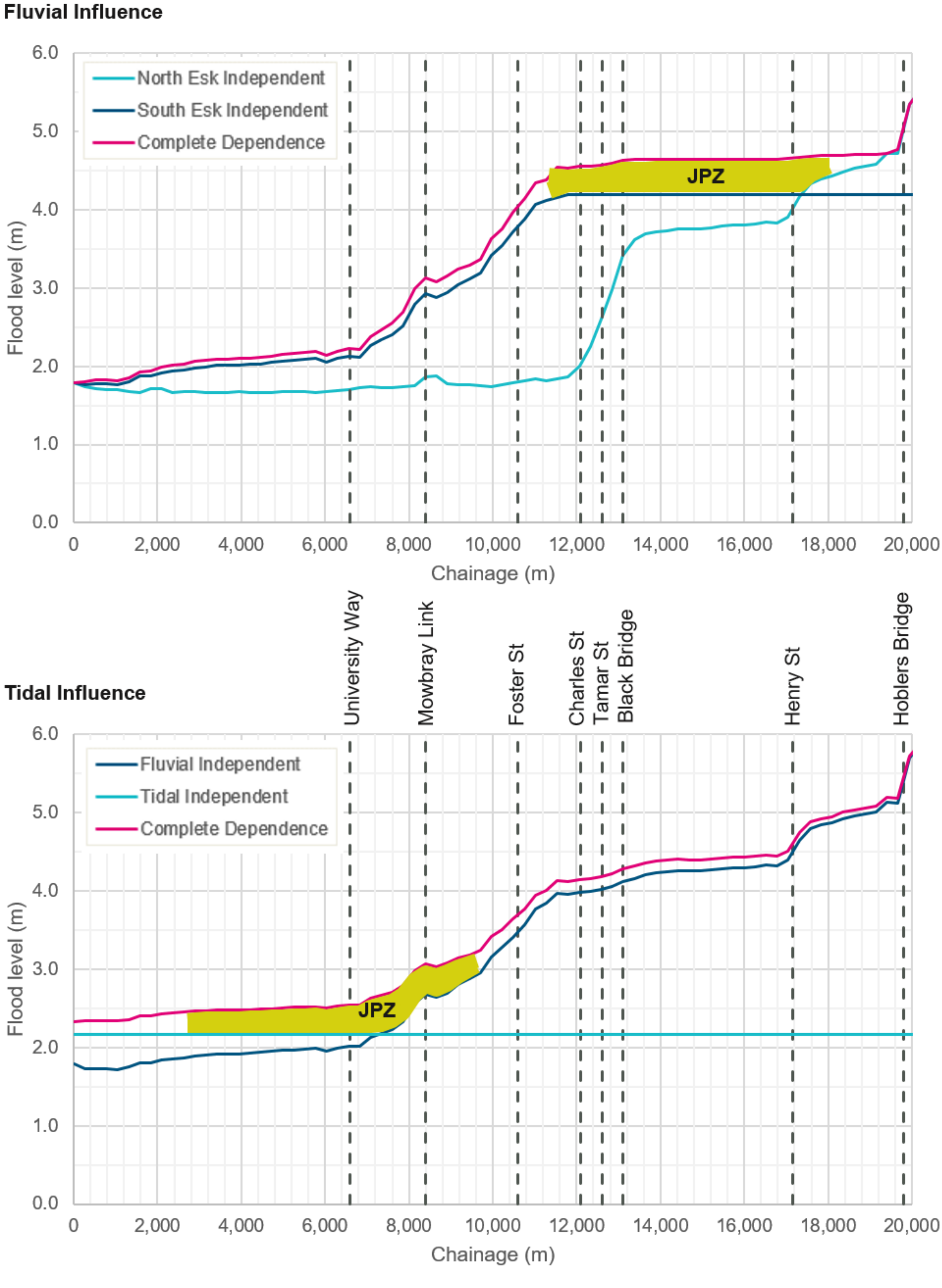


Figure 5-2 Existing Conditions 1% AEP Joint Probability Pre-Screening Analysis Results

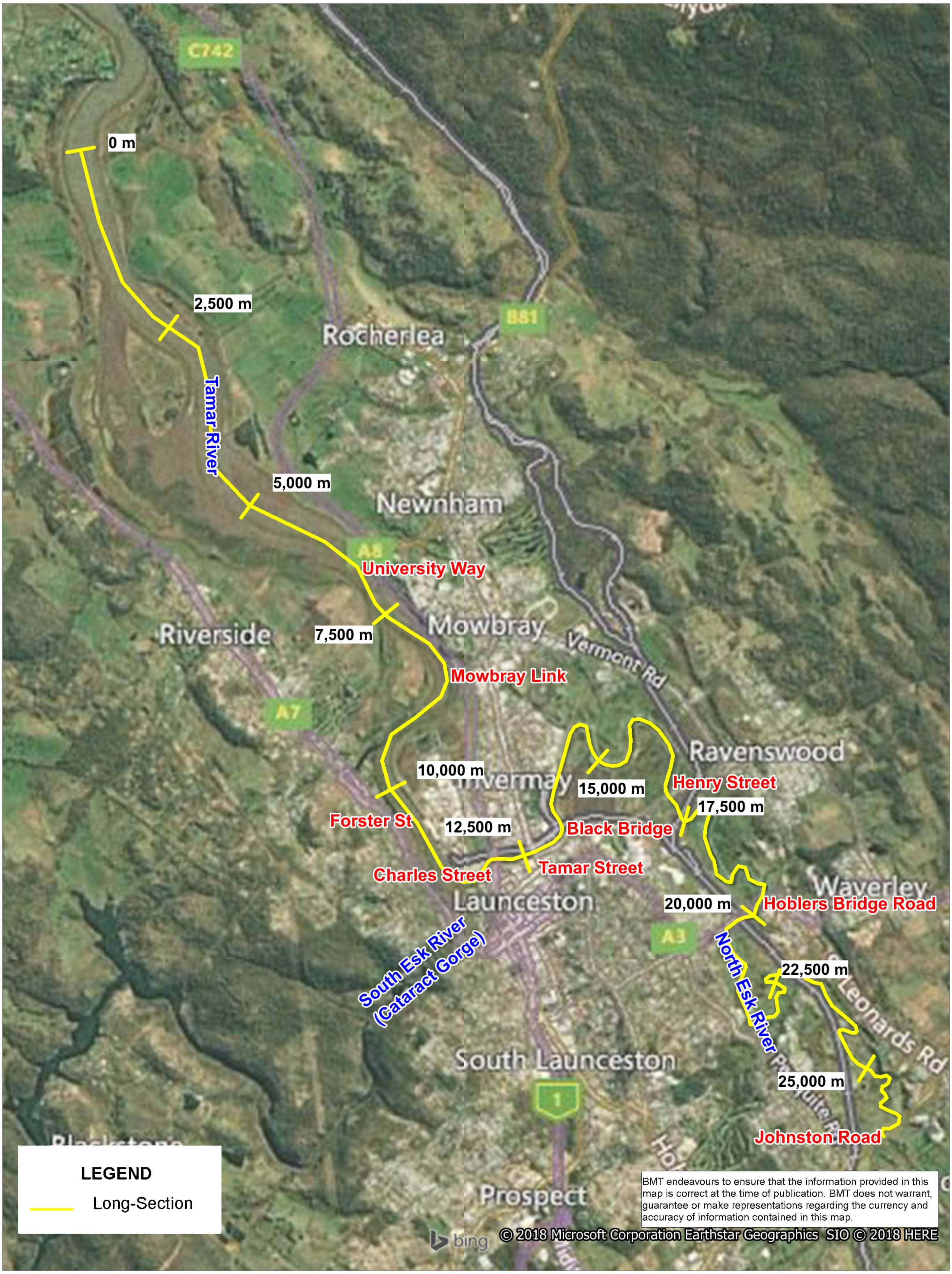


Figure 5-3 Reporting Location Long-Section

BMT

T:\M20921.MS.Launceston_Mapping_2017\MapInfo\Drawings\Final\Fig5-3_JPA_LongSect_RevA.WOR

Table 5-1 Existing Conditions Joint Probability Pre-Screening Analysis Results

AEP	University Way	Mowbray Link	Forster St	Charles Street	Tamar Street	Black Bridge	Henry Street	Hoblers Bridge Road	Johnston Road
Fluvial Influence Z mm (mm) (300 mm tolerance)									
20%	0	0	80	120	130	150	0	0	0
10%	20	50	120	190	190	220	50	0	0
5%	40	100	170	250	260	300	90	0	0
2%	70	160	220	310	310	360	410	20	0
1%	100	200	250	350	350	420	450	40	0
1 in 200	130	200	230	320	340	380	400	200	0
1 in 500	180	200	200	290	320	340	330	370	0
1 in 1000	210	200	190	310	330	350	350	390	0
1 in 2000	240	190	190	330	350	370	380	400	0
Tidal Influence Z mm (mm) (300 mm tolerance)									
20%	0	0	90	240	280	320	180	80	0
10%	20	130	370	400	400	400	250	60	0
5%	40	260	330	270	270	270	250	60	0
2%	250	470	320	230	230	230	220	140	0
1%	410	320	170	120	120	120	120	110	0
1 in 200	560	300	120	80	100	100	90	80	0
1 in 500	380	120	50	40	40	40	40	40	0
1 in 1000	380	120	50	40	40	40	40	40	0
1 in 2000	240	100	50	0	30	40	40	40	0

Table 5-2 2050 Climate Conditions Joint Probability Pre-Screening Analysis Results

AEP	University Way	Mowbray Link	Forster St	Charles Street	Tamar Street	Black Bridge	Henry Street	Hoblers Bridge Road	Johnston Road
Fluvial Influence Z mm (mm) (300 mm tolerance)									
20%	0	10	90	150	160	180	0	0	0
10%	20	60	140	220	220	240	100	0	0
5%	50	120	180	270	280	320	190	0	0
2%	80	170	230	320	320	350	510	140	0
1%	100	180	240	330	320	400	430	100	0
1 in 200	150	210	240	330	330	360	340	250	0
1 in 500	210	200	200	320	350	370	360	400	0
1 in 1000	230	200	190	310	350	360	340	310	0
1 in 2000	230	200	200	300	340	370	370	400	0
Tidal Influence Z mm (mm) (300 mm tolerance)									
20%	0	30	230	380	380	380	190	80	0
10%	20	160	390	330	330	330	210	80	0
5%	80	370	300	240	240	240	230	90	0
2%	260	340	190	140	140	130	130	70	0
1%	450	250	120	80	70	60	50	40	0
1 in 200	410	180	80	50	40	30	30	30	0
1 in 500	330	120	50	40	40	40	40	30	0
1 in 1000	280	100	40	40	30	30	30	30	0
1 in 2000	230	100	50	80	50	40	40	40	0

Table 5-3 2090 Climate Conditions Joint Probability Pre-Screening Analysis Results

AEP	University Way	Mowbray Link	Forster St	Charles Street	Tamar Street	Black Bridge	Henry Street	Hoblers Bridge Road	Johnston Road
Fluvial Influence Z mm (mm) (300 mm tolerance)									
20%	0	50	100	160	160	190	50	0	0
10%	20	90	150	230	230	240	200	-50	0
5%	60	130	200	280	280	340	270	0	0
2%	120	220	270	290	290	310	440	90	0
1%	160	270	300	280	280	290	290	10	0
1 in 200	190	260	270	300	310	310	270	230	0
1 in 500	210	200	210	350	380	410	410	450	0
1 in 1000	240	200	210	390	390	410	410	370	0
1 in 2000	250	210	220	400	360	390	390	410	0
Tidal Influence Z mm (mm) (300 mm tolerance)									
20%	0	0	130	370	370	370	350	160	0
10%	0	110	340	320	320	320	310	140	0
5%	40	330	320	250	250	240	240	140	0
2%	250	330	210	150	150	150	140	110	0
1%	460	240	120	90	90	80	80	90	0
1 in 200	410	190	90	60	60	60	60	70	0
1 in 500	330	140	70	50	50	50	50	50	0
1 in 1000	280	120	60	50	50	40	40	40	0
1 in 2000	240	130	70	110	60	50	40	40	0

5.1.3 Stage 2 – Joint Probability Analysis

The results of the Pre-Screening Analysis (Section 5.1.2) show the JPZ extending along the study area for a series of AEPs of interest. In turn, a comprehensive Joint Probability Analysis (JPA) is warranted for the combinations of location and AEP highlighted in Table 5-1, Table 5-2 and Table 5-3. For each climate scenario, the Design Variable Method, documented in Book 6, Chapter 5 of ARR 2016 was applied, again separately two times; first in the JPZ under fluvial (tributary flows) influence, and second in the JPZ under tidal (estuarine region) influence.

It is worth noting that the JPA in the JPZ of the tributary flows could be alternatively addressed using the methodologies described in Book 4, Chapter 4 of ARR 2016, which uses a Monte Carlo Simulation framework. However due to the following considerations, the Design Variable Method was adopted as a practical and consistent method for addressing the joint influence of both, tributary flows and estuarine region:

- Limitations encountered when attempted to apply the methodologies of Book 4, Chapter 4 of ARR 2016 (further described in Appendix A)
- The probabilistic trivariate nature of flood risks in the study area
- The limited register of continuous coincident flow rate data of the tributary flows of the South Esk and North Esk Rivers
- The range and number of AEPs being assessed in this study
- The accessibility of a software tool available online from and endorsed by ARR 2016, which makes practical the application of the *Design Variable Method*

The Design Variable Method assumes that statistical dependence between the extreme values of the dependent variates considered can be represented through a bivariate logistic extreme value dependence model. This allows representing the dependence between the relevant variates using a dependence parameter, which values range between zero (0) and one (1) for strong and weak dependence, respectively.

Implementation of the Design Variable Method, following the ARR 2016 recommended approach, was conducted in four steps:

(1) Conduct a *Pre-Screening Analysis*, this is a preliminary step to determining if further analysis is relevant, by estimating the existence and extent of the JPZ (outcomes described in Section 5.1.2).

(2) Select the dependence parameter (α), for the relevant variates and considering the study location and characteristics of the storms of interest.

The relevant dependent variates and dependence parameter (α) used in the JPA in this study are:

(a) For the tributary flows, coincident storm fluvial flows in the main and tributary (resulting from storm rainfall in the sub-catchments). With α = 0.592 determined from the joint dependence analysis of the tributary flows, following the approach outlined by Pedruco et al. (2013). This approach involves: transform the marginal distributions of the inputs to Unit Frèchet series; convert the Unit Frèchet series to radial and angular components; and fit to a point process likelihood model (in this case an asymmetric bilogistic model). We applied this approach using as input the coincident maximum storm flows of the South Esk and North Esk Rivers derived from Flood Frequency Analysis as described in Section 3.

(b) For the estuarine region, coincident storm tide water levels and storm fluvial flow (resulting from rainfall in the whole catchment). With α = 0.950 selected from a map of values for the Australian Coastline provided in ARR 2016 (Book 6, Chapter 5, Figure 6.5.13). This map of values of α was derived by Zheng et al. (2014) who conducted a joint dependence analysis of the relevant variates Australia wide.

(3) Calculate flood levels within the JPZ for a matrix of different combinations of events of the relevant variates, over the AEPs of interest, using hydrological and hydraulic models.

For each climate scenario, the hydraulic model described in Section 4 was run with boundary conditions representative of different combinations of storm events covering AEP between 20% (1 in 5) and 0.05% (1 in 2000), as well as including “no event” cases for defining lower bounds. Model runs were completed separately as follows:

(a) For the tributary flows a first set of 100 model runs completed, for a 10 x 10 matrix of different combinations of events representing the rainfall storm fluvial flow AEPs, of the South Esk and North Esk tributaries. The “no event” fluvial flow was defined to be a “no rain” event, i.e. no river flows. For the this runs, the tail water level (tidal) condition was set equal to the local Mean High Water-level Spring (MHWS).

(b) For the estuarine region a second set of 100 model runs completed, for a 10 x 10 matrix of different combinations of events representing storm tide and rainfall storm fluvial flow AEPs. The “no event” storm tide was defined as a tail water level (tidal) condition equal to the local Mean High Water-level Spring (MHWS) derived from astronomical tide predictions, i.e. with no storm surge component added. Whereas, the “no event” fluvial flow was defined to be a “no rain” event, i.e. no river flows.

All these 200 model runs assumed static (constant) ocean water levels, as tidal dynamics are not considered by the method.

Results of flood level predicted by each model run at the locations within the JPZ, as determined during the Pre-Screening Analysis, were then summarised in matrices of flood levels, separately for the tributary flows and estuarine region.

(4) Estimate the flood levels for exceedance of the joint probability for the AEPs of interest; by integrating the joint probability density function for the co-occurring extremes for specified flood level exceedances (AEPs) using a bivariate logistic extreme distribution.

A software tool developed by researchers of The University of Adelaide in ARR Project 18 and available online (from ARR 2016 website) can be used for completing this step.

The online software tool takes as input, for each location of interest, the relevant dependence parameter and a matrix of flood levels obtained from the hydrologic and hydraulic modelling runs for the different combinations of AEPs.

In this study, the online software tool was used on a per location of interest basis, and separately for assessing the influence of the tributary flows and the estuarine region. That is, the tool was used for each of the locations that had been found to be within the relevant JPZ (as highlighted in Table 5-1), using the dependence parameter (α) values described in Step 2 above and setting up matrices of flood level for each location obtained from the hydraulic model runs completed in Step 3 above, for tributary flows and estuarine region, as appropriate.

Example outcomes of the JPA resulting from application of the Design Variable Method for existing conditions, at locations within the JPZ, are presented in Figure 5-4 for the tributary flows region and Figure 5-5 for the estuarine region.

Each graph in these figures shows three curves of estimated flood levels against probability for: complete dependence, incomplete dependence and joint probability outcomes. Each of these curves correspond to different dependence assumptions on the co-occurrence of the variate events (i.e. storm fluvial flows and storm tides):

- The complete dependence curve assumes that for each AEP storm events always co-occur, and thus these results are considered conservative flood level estimates
- The complete independence curve assumes that for each AEP storm events never co-occur, and thus these results are considered unconservative flood level estimates
- The dependence curve represents estimated flood levels, for each AEP, as a joint probability outcome of storm events co-occurring, based on the dependence analysis of the relevant variates, following the Design Variable Method

In general, the JPA results of the tributary flows show relatively small difference of flood levels between the complete dependence curve and dependence curve, sometimes almost overlapping (Figure 5-4). Consistent with Pedruco et al (2013), this indicates a strong association between the main and tributary fluvial storm flow variates, in other words, catchment rainfall tends to result in South and North Esk River flows.

In contrast, the JPA results of the estuarine region show relatively larger difference of flood levels between the complete dependence curve and dependence curve, with the dependence results tending to be closer to the complete independence ones (Figure 5-5). This indicates a weaker association between the fluvial storm flow and storm tide level variates, in other words, rainfall storms and ocean storm surges of similar AEP not necessarily co-occur in the catchment and the Tamar River estuary. Another general observation from Figure 5-5 is; the influence of co-occurrence of ocean and fluvial storms on flood levels tends to decrease along the estuary on the upstream direction, i.e. larger difference between dependent and independent results are estimated in the downstream in the estuary; which is generally expected considering that the effect of ocean processes on estuary water level is limited by tidal and surge reach.

It is worth noting that in some cases the online software tool produced unexpected behaviour with some sections of the complete dependence and independence curves “crossing”, e.g. at Hoblers Bridge for the tributary flows JPA (Figure 5-4). This type of behaviour has been previously reported in other studies completed by BMT to the software developers at The

University of Adelaide, who have advised that possible reasons for this have to do with issues with interpolation and extrapolation algorithms applied by the method, particularly when difference between the complete dependence and independence flood levels are relatively small and when rarer AEPs are involved. The suggested approach to deal with this issue is to assume complete dependence results, as these represent conservative estimates of flood levels, in any case.

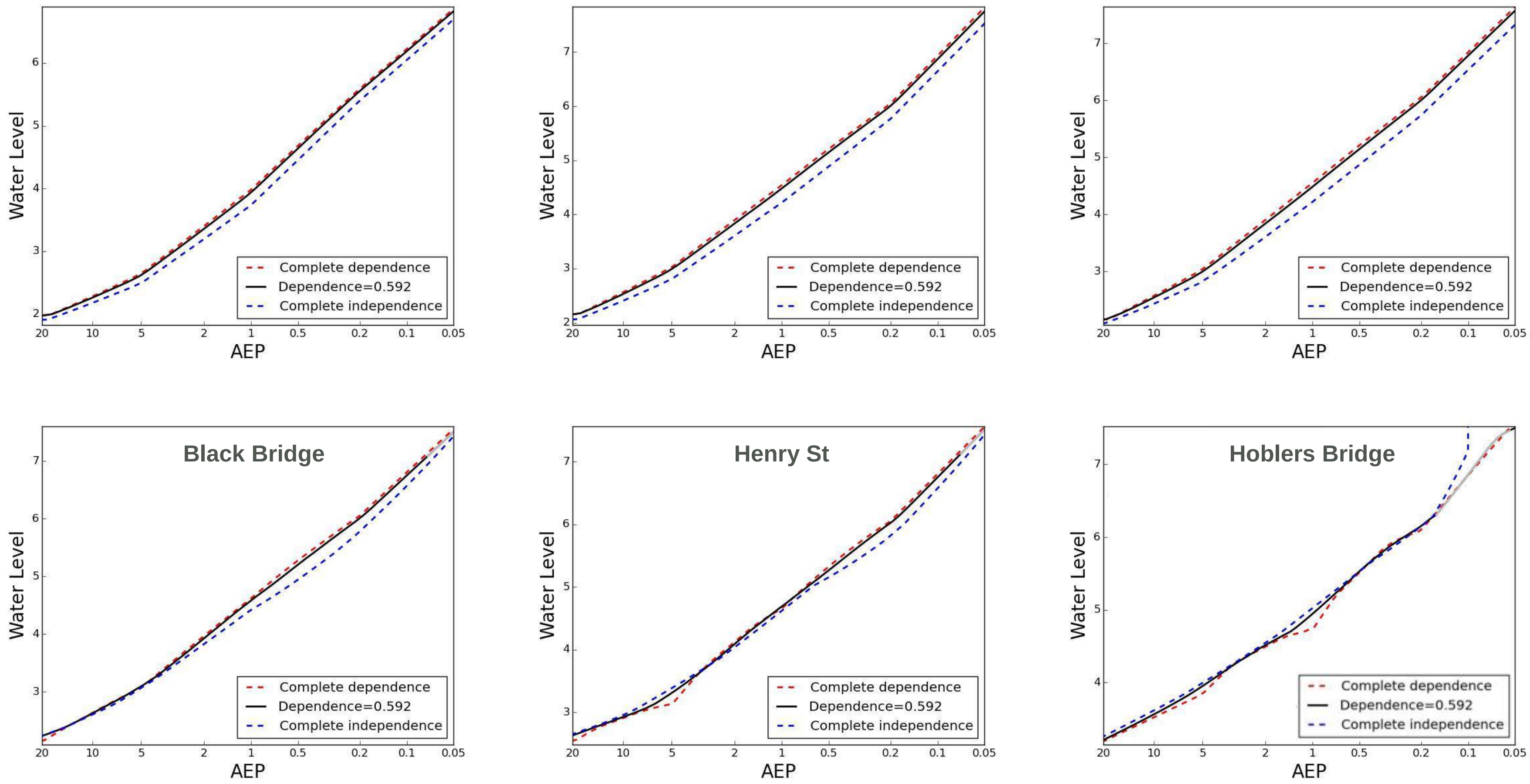


Figure 5-4 Flood Level Outcomes of Joint Probability Analysis at Key Locations along the Joint Probability Zone in the Tributary Flows Region

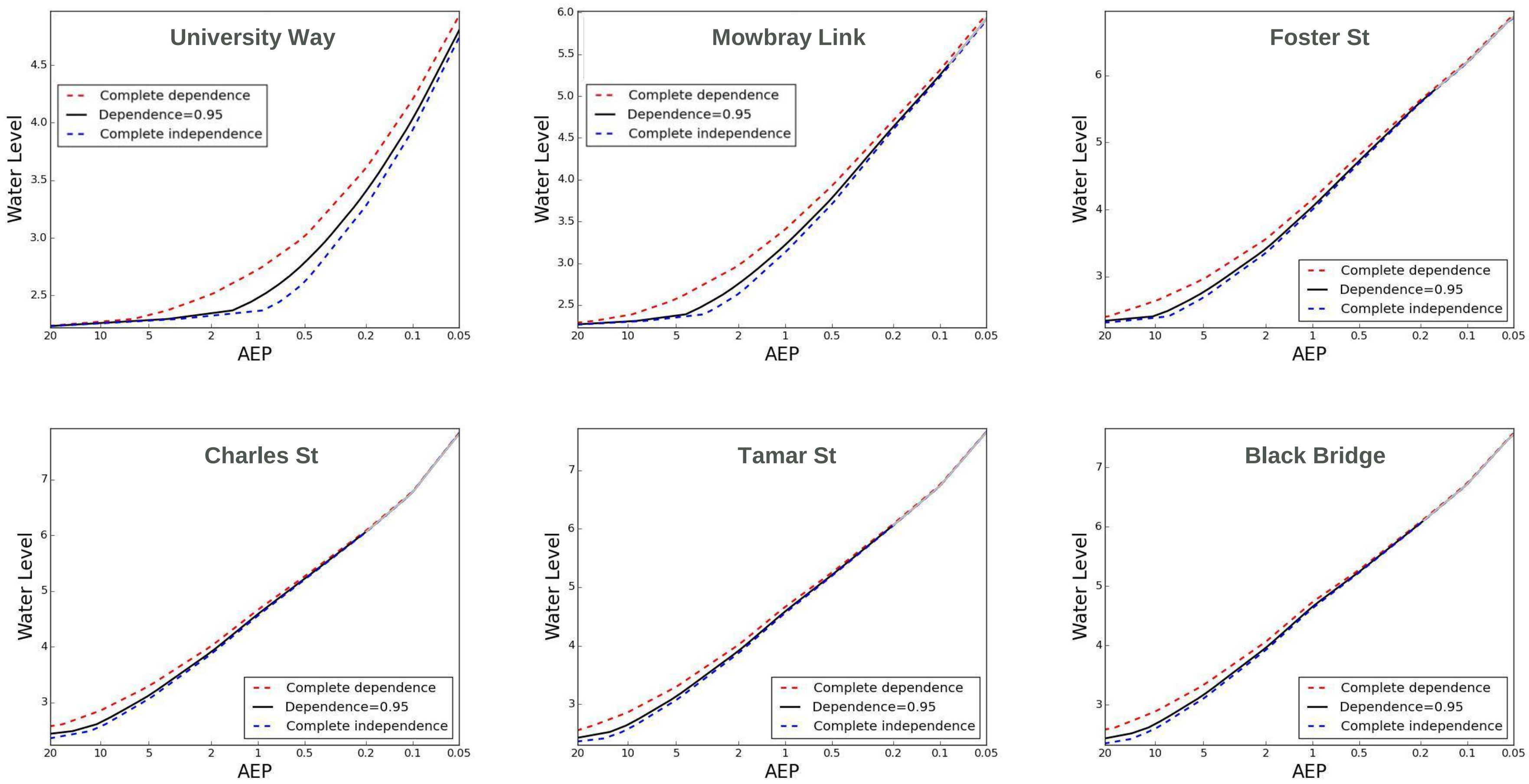


Figure 5-5 Flood Level Outcomes of Joint Probability Analysis at Key Locations along the Joint Probability Zone in the Estuarine Region

5.1.4 Design Event Flood Levels

The final AEP design event flood levels for the key locations in the Joint Probability Zone are presented in Table 5-4, Table 5-5 and Table 5-6 for the existing, 2050 and 2090 climate conditions respectively. Long sections of the design event flood levels are presented in Figure 5-6, Figure 5-7 and Figure 5-8.

Table 5-4 Existing Conditions Design Event Flood Levels

AEP	University Way	Mowbray Link	Forster St	Charles Street	Tamar Street	Black Bridge	Henry Street	Hoblers Bridge Road	Johnston Road
20%	2.25	2.28	2.35	2.45	2.45	2.45	2.57	3.19	7.75
10%	2.28	2.31	2.42	2.65	2.65	2.68	2.81	3.52	7.97
5%	2.29	2.38	2.77	3.00	3.00	3.16	3.20	3.85	8.21
2%	2.35	2.76	3.42	3.83	3.84	3.92	3.96	4.29	8.60
1%	2.48	3.22	4.04	4.48	4.48	4.58	4.65	4.71	8.87
1 in 200	2.79	3.78	4.73	5.15	5.15	5.20	5.24	5.29	9.20
1 in 500	3.41	4.64	5.61	6.01	6.01	6.01	6.02	6.08	9.69
1 in 1000	4.03	5.25	6.20	6.87	6.78	6.75	6.75	6.73	10.12
1 in 2000	4.80	5.92	6.87	7.75	7.57	7.50	7.51	7.57	10.58

Table 5-5 2050 Climate Conditions Design Event Flood Levels

AEP	University Way	Mowbray Link	Forster St	Charles Street	Tamar Street	Black Bridge	Henry Street	Hoblers Bridge Road	Johnston Road
20%	2.46	2.51	2.61	2.76	2.76	2.79	2.85	3.48	7.90
10%	2.51	2.60	2.75	3.04	3.04	3.07	3.12	3.85	8.17
5%	2.54	2.66	3.13	3.40	3.40	3.56	3.60	4.20	8.44
2%	2.60	3.16	3.88	4.29	4.29	4.37	4.45	4.72	8.88
1%	2.80	3.64	4.51	4.97	4.97	5.04	5.13	5.18	9.20
1 in 200	3.16	4.19	5.11	5.54	5.54	5.58	5.59	5.63	9.57
1 in 500	3.82	4.98	5.92	6.37	6.35	6.34	6.35	6.43	10.15
1 in 1000	4.47	5.61	6.57	7.23	7.14	7.08	7.07	7.08	10.58
1 in 2000	5.37	6.28	7.25	8.28	8.06	7.94	7.94	8.01	10.94

Table 5-6 2090 Climate Conditions Design Event Flood Levels

AEP	University Way	Mowbray Link	Forster St	Charles Street	Tamar Street	Black Bridge	Henry Street	Hoblers Bridge Road	Johnston Road
20%	3.04	3.08	3.21	3.34	3.32	3.34	3.38	3.84	8.06
10%	3.08	3.14	3.31	3.60	3.61	3.65	3.69	4.21	8.36
5%	3.10	3.23	3.67	3.94	3.94	4.10	4.14	4.56	8.69
2%	3.17	3.74	4.44	4.98	4.98	5.04	5.07	5.12	9.13
1%	3.39	4.23	5.07	5.39	5.39	5.45	5.46	5.54	9.49
1 in 200	3.77	4.72	5.59	5.94	5.94	5.93	5.92	6.05	9.92
1 in 500	4.42	5.42	6.30	6.81	6.76	6.74	6.77	6.85	10.55
1 in 1000	5.06	6.07	6.94	7.76	7.62	7.54	7.53	7.53	11.01
1 in 2000	5.85	6.71	7.68	8.93	8.63	8.45	8.45	8.46	11.39

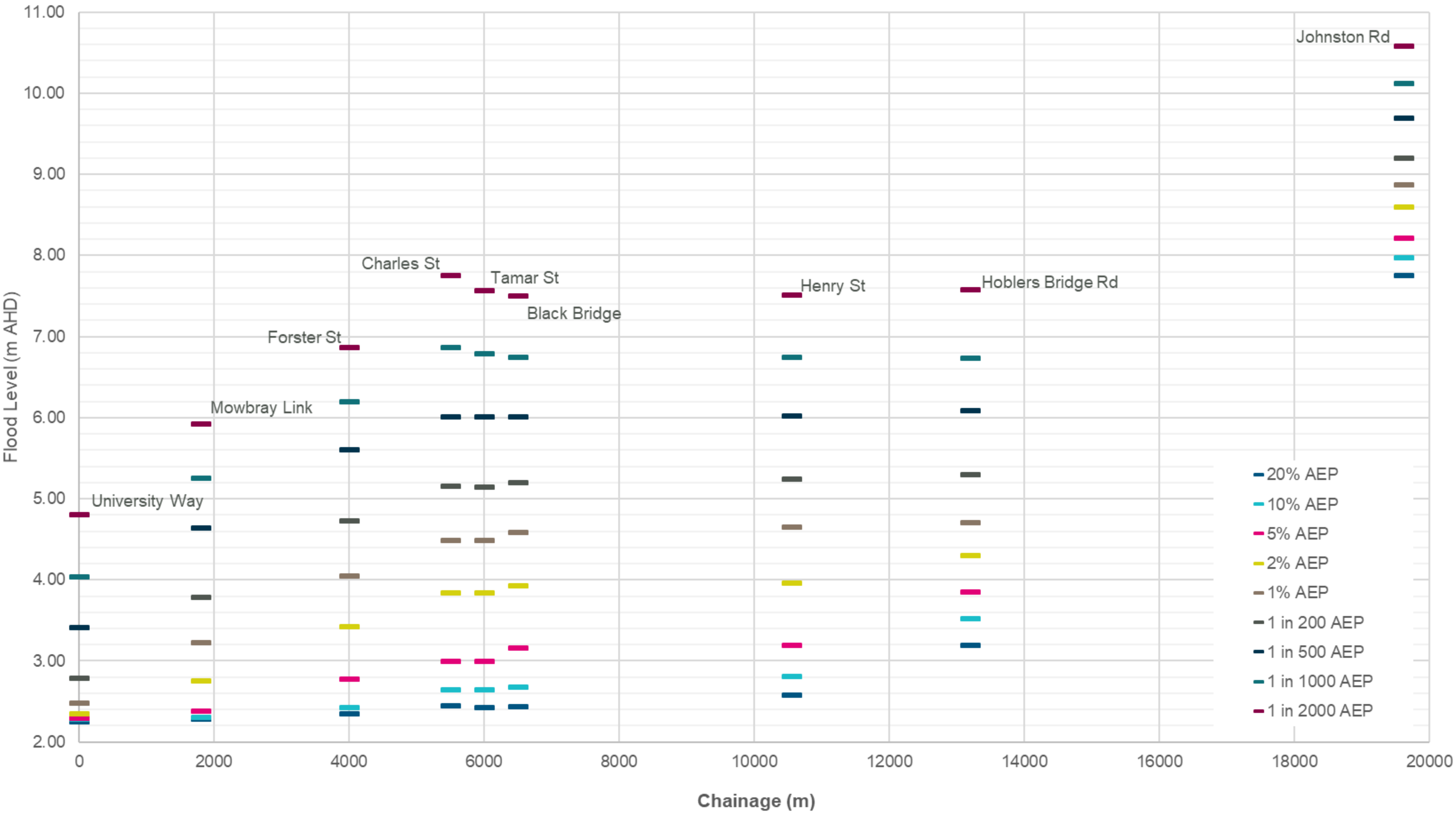


Figure 5-6 Existing Conditions Design Event Flood Levels

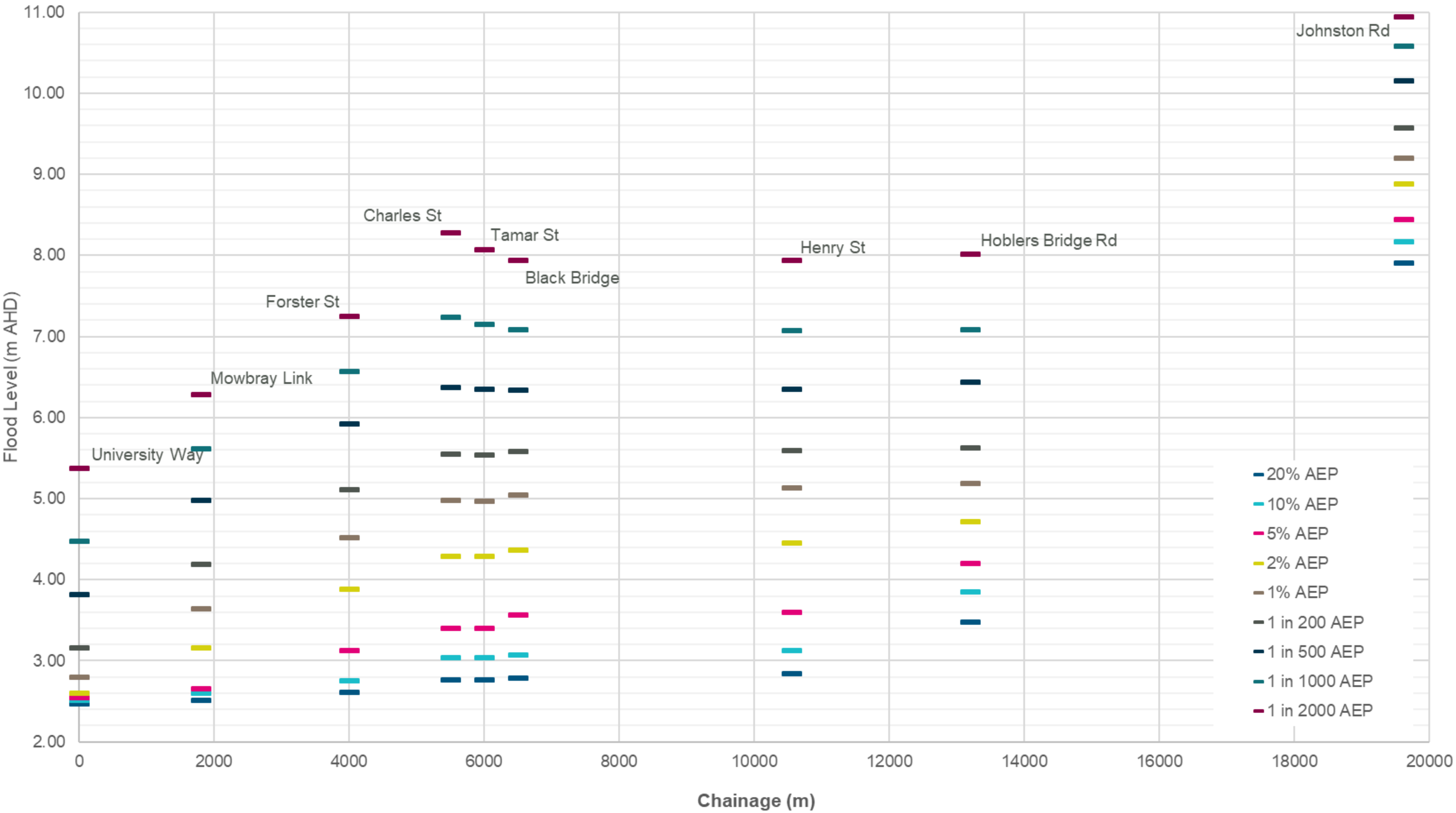


Figure 5-7 2050 Climate Conditions Design Event Flood Levels

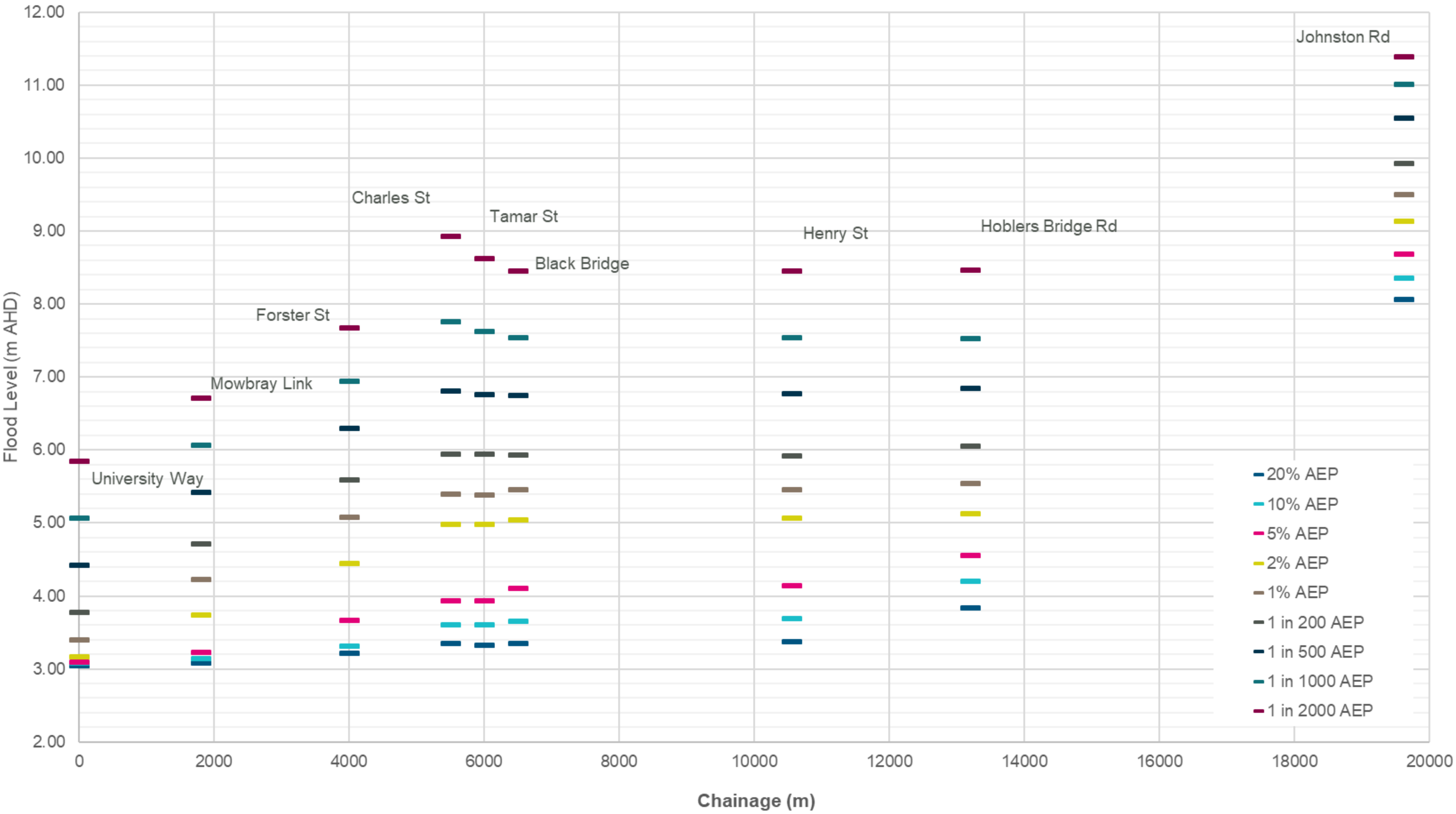


Figure 5-8 2090 Climate Conditions Design Event Flood Levels

5.2 Mapping Outputs

This section provides a brief overview of the floodplain mapping process. The mapping outputs are presented in Volume 2: Flood Mapping.

TUFLOW produces a geo-referenced data set defining peak water levels throughout the model domain at the corners of its computational cells. For a given AEP flood event, the peak flood level from each of the storm durations was selected for each computational cell to generate an envelope of peak flood levels. These data were imported into GIS to generate a DEM of the flood surface.

The flood mapping products presented in the following sections have also been supplied to council in GIS format.

5.2.1 Flood Level Mapping

Flood level mapping for all modelled design events is presented in Volume 2: Flood Mapping, including 1 m and 0.1 m flood level contours.

5.2.1.1 Considerations for Flood Mapping in the Joint Probability Zone

For flood mapping purposes a practical approach has been adopted whereby, for each AEP under consideration one flood surface has been selected which best represents the resulting design flood levels from the JPA (Section 5.1.4) which are location dependent. This approach has been adopted, as opposed to developing composite mapping product that represents multiple flood surfaces matching design flood levels at each location along the river system. This allows for consistency to be maintained between peak flood level and velocity vector mapping and simplifies the use of the mapping for flood planning, response and assessment uses.

To develop the design event flood mapping the hydraulic flood modelling runs for the design events used a time-varying tide boundary, with a simple harmonic idealised spring tide (as described in Section 4.5.2) that was adjusted to match the design event peak flood levels, of the AEP under consideration (Section 5.1.4).

Table 5-7 presents the difference in peak flood levels between the flood level mapping products (Section 5.2.1.1) and the design flood levels from the JPA (Section 5.1.4). Table 5-7 shows that except for the 20% and 10% AEP events at University Way and Mowbray Link in the Tamar River estuary there is a close fit between the flood level mapping and the JPA.

Table 5-7 Comparison of Existing Conditions Flood Mapping and JPA Flood Levels (m)

AEP	University Way	Mowbray Link	Forster St	Charles Street	Tamar Street	Black Bridge	Henry Street	Hoblers Bridge Road	Johnston Road
20%	-0.23	-0.2	-0.13	-0.1	-0.07	-0.07	0	0	0
10%	-0.23	-0.12	0.03	0.02	0.02	0.01	0	0	0
5%	-0.14	0.01	0.03	0.13	0.13	0	0	0	0
2%	-0.05	0.02	-0.01	0.05	0.05	0.01	0	0	0
1%	0.02	0.01	-0.02	0.08	0.08	0.04	0	0	0
1 in 200	-0.03	-0.07	-0.08	0.04	0.03	0.02	0	0	0
1 in 500	-0.02	-0.01	-0.01	0.03	0.04	0.04	0.02	0	0
1 in 1000	-0.05	-0.03	-0.02	-0.13	-0.07	-0.05	-0.05	0	0
1 in 2000	-0.06	-0.03	-0.01	0.03	0.06	0.04	0.04	0	0

5.2.1.2 Comparison to Current Design Event Flood Levels

Table 5-8 shows a comparison of the current adopted design flood levels (BMT WBM 2008) and the updated flood levels. The increases in existing conditions flood level are primarily due the increased North and South Esk River design flows (Section 3.2). As per ARR 2016 the JPA has been undertaken using constant high tide levels, this has also increased design event flood levels.

Resulting from the increased design event flood levels the levee system (Inveresk levee height of approximately 5.1 m AHD) is now overtopped in a 1 in 200 AEP event under existing conditions, with an estimated immunity of approximately 1% AEP without freeboard by 2050.

Table 5-8 Comparison of Current and Updated Peak Flood Levels at the North Esk River Confluence

AEP	Peak Flood Level (m AHD)				
	Pre-2008 Study	2008 Study	Existing Conditions	2050 Conditions	2090 Conditions
5%	2.8	2.8	3.1	3.5	4.1
2%	3.2	3.4	3.9	4.4	5.0
1%	3.4	3.8	4.6	5.1	5.5
1 in 200 (95% Confidence)	3.9	4.2 (4.5)	5.2	5.6	6.1
1 in 500	4.3	5.0	6.1	6.5	6.9

5.2.2 Flood Depth Mapping

Flood depth mapping for all modelled design events is presented in Volume 2: Flood Mapping.

5.2.3 Flood Velocity Mapping

Flood velocity mapping for all modelled design events is presented in Volume 2: Flood Mapping.

The flood velocity mapping is designed to depict both the magnitude (grid format) and direction of the flow (direction vectors) velocities. Flow direction vectors could not be presented in this mapping for clarity reasons but have been produced.

5.2.4 Flood Hazard Mapping

Flood hazard mapping for all modelled design events is presented in Volume 2: Flood Mapping.

Hazard mapping was undertaken using the combined flood hazard criteria presented in Book 6, Chapter 7 of ARR 2016. As shown in Figure 5-9, hazard is defined in terms of the depth and velocity-depth product as follows:

(1) **Generally safe for vehicles, people and buildings** - velocity x depth less than 0.3 m^2/s if depth is less than 0.3 m and velocity is less than 2 m/s

(2) **Unsafe for small vehicles** - velocity x depth less than 0.6 m^2/s if depth is less than 0.5 m and velocity is less than 2 m/s

(3) **Unsafe for vehicles, children and the elderly** - velocity x depth less than 0.6 m^2/s if depth is less than 1.2 m and velocity is less than 2 m/s

(4) **Unsafe for vehicles and people** - velocity x depth less than 1 m^2/s if depth is less than 2 m and velocity is less than 2 m/s

(5) **Unsafe for vehicles and people. All buildings vulnerable to structural damage. Some less robust buildings subject to failure** - velocity x depth less than 4 m^2/s if depth is less than 4 m and velocity is less than 4 m/s

(6) **Unsafe for vehicles and people. All building types considered vulnerable to failure** - velocity x depth greater than 4 m^2/s

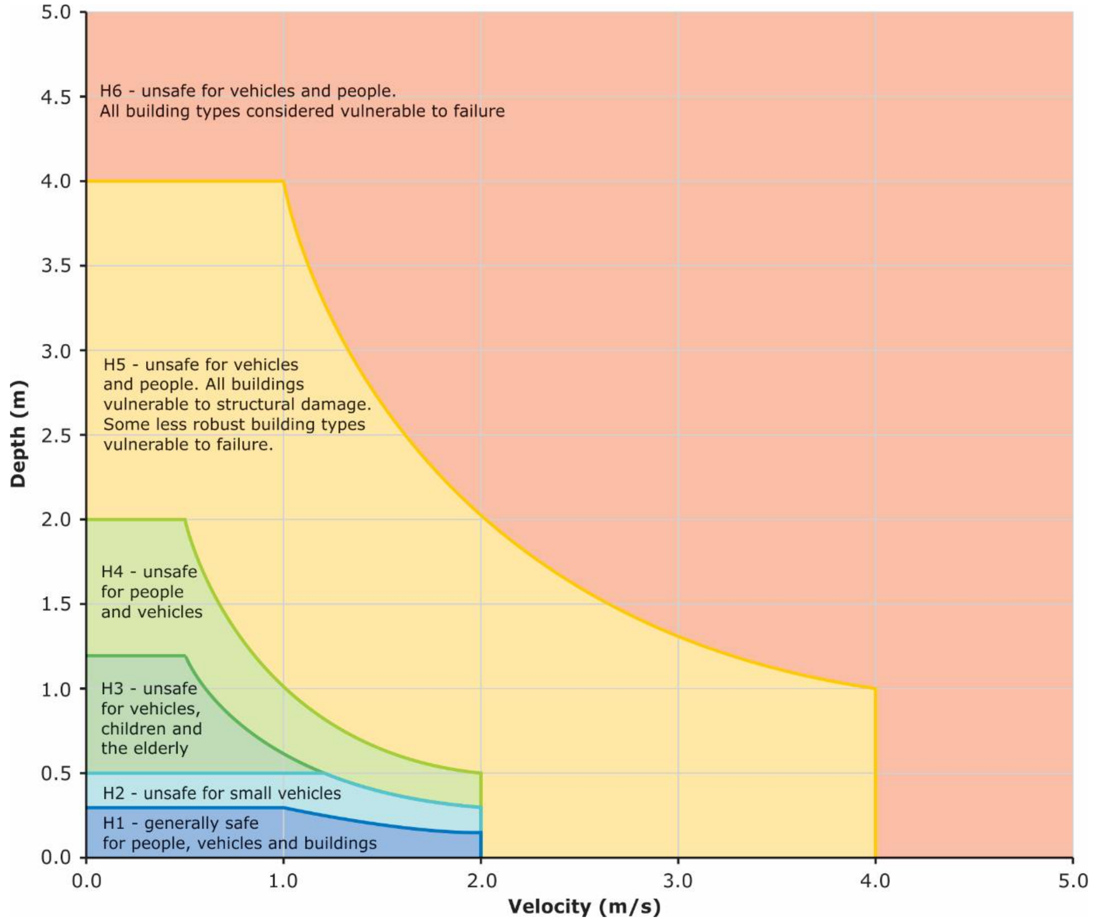


Figure 5-9 Combine Flood Hazard Curves (Ball et al. 2016)

5.2.5 Flooded Properties and Floor Levels

The number of properties with flooding within the property boundary and buildings with above floor flooding are presented in Table 5-9 and Table 5-10 respectively.

The number of properties flooded is based on the cadastre excluding as accessed via the Land Information System Tasmania. The floor level survey provided by Council was used.

Table 5-9 Flooded Properties

Climate Conditions	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 200 AEP	1 in 500 AEP	1 in 1000 AEP	1 in 2000 AEP
Existing	313	339	410	537	633	3,008	3,816	4,412	4,696
2050	344	399	500	619	2,178	3,369	4,266	4,695	5,009
2090	448	536	610	1,786	3,268	3,760	4,590	4,881	5,305

Table 5-10 Above Floor Flooding

Climate Conditions	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 200 AEP	1 in 500 AEP	1 in 1000 AEP	1 in 2000 AEP
Existing	21	27	48	89	106	1,819	1,972	2,086	2,095
2050	21	36	51	85	1,141	1,831	2,060	2,123	2,135
2090	38	53	83	358	1,803	1,930	2,118	2,133	2,138

5.2.6 Flooded Roads

The peak road inundation depth for the road low points identified by Council are presented in Table 5-11, Table 5-12 and Table 5-13 for existing, 2050 and 2090 climate conditions respectively.

The road inundations depths have been assigned a hazard for vehicles as per the combined flood hazard criteria (Section 5.2.4) based on the depth criteria only.

Table 5-11 Existing Conditions Road Inundation Depths (m)

Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 200 AEP	1 in 500 AEP	1 in 1000 AEP	1 in 2000 AEP
East Tamar Hwy (North)	-	-	-	-	0.36	0.84	1.66	2.25	2.92
East Tamar Hwy (South)	-	-	-	-	0.36	0.84	1.68	2.26	2.92
West Tamar Rd (South)	-	0.29	0.72	1.44	2.06	2.61	3.42	4.01	4.83
West Tamar Rd (North)	-	-	-	0.79	1.41	1.96	2.77	3.36	4.19
Home Point Pde	-	-	-	1.18	1.86	2.49	3.34	4.08	5.26
Henry St (South)	0.09	0.30	0.70	1.47	2.15	2.75	3.55	4.20	5.05
Henry St (North)	0.19	0.41	0.80	1.56	2.25	2.84	3.65	4.30	5.15
Ravenswood Rd	-	-	0.08	0.54	0.92	1.50	2.29	2.93	3.77
Waverley Rd	-	-	-	-	1.11	1.70	2.49	3.13	3.97
Hoblers Bridge Rd	-	0.46	0.74	1.11	1.41	1.73	2.21	2.85	3.68
Hart St	-	-	0.56	1.00	1.34	1.67	2.11	2.57	3.40
Killafaddy Rd	-	-	0.30	0.69	1.02	1.37	2.05	2.69	3.53
Johnson Rd	-	-	0.06	0.30	0.44	0.64	1.10	1.55	2.03
Station Rd	0.76	1.08	1.42	1.77	1.97	2.18	2.52	2.89	3.32
Glenwood Rd	-	-	-	-	-	0.14	0.49	0.85	1.27

(yellow)	Depth greater than 0.3 m, unsafe for small vehicles
(pink)	Depth greater than 0.5 m, unsafe for small all vehicles

Table 5-12 2050 Climate Conditions Road Inundation Depths (m)

Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 200 AEP	1 in 500 AEP	1 in 1000 AEP	1 in 2000 AEP
East Tamar Hwy (North)	-	-	-	0.28	0.78	1.28	2	2.57	3.26
East Tamar Hwy (South)	-	-	-	0.21	0.78	1.29	2.01	2.57	3.27
West Tamar Rd (South)	0.35	0.64	1.1	1.88	2.49	3	3.72	4.31	5.26
West Tamar Rd (North)	-	-	0.45	1.23	1.84	2.35	3.07	3.66	4.58
Home Point Pde	-	-	0.67	1.65	2.35	2.87	3.71	4.47	5.82
Henry St (South)	0.33	0.61	1.1	1.95	2.63	3.08	3.89	4.55	5.49
Henry St (North)	0.44	0.71	1.2	2.05	2.73	3.17	3.99	4.65	5.59
Ravenswood Rd	-	0.06	0.41	0.94	1.39	1.83	2.64	3.28	4.21
Waverley Rd	-	-	-	1.13	1.59	2.03	2.83	3.48	4.41
Hoblers Bridge Rd	0.35	0.71	0.98	1.41	1.73	2.09	2.55	3.2	4.12
Hart St	-	0.48	0.86	1.34	1.67	2.03	2.44	2.92	3.83
Killafaddy Rd	-	0.26	0.57	1.03	1.38	1.77	2.4	3.04	3.96
Johnson Rd	-	0.03	0.22	0.43	0.62	0.96	1.58	2.03	2.4
Station Rd	0.96	1.36	1.65	1.96	2.16	2.42	2.91	3.32	3.66
Glenwood Rd	-	-	-	-	0.13	0.39	0.87	1.27	1.62

Depth greater than 0.3 m, unsafe for small vehicles

Depth greater than 0.5 m, unsafe for small all vehicles

Table 5-13 2090 Climate Conditions Road Inundation Depths (m)

Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1 in 200 AEP	1 in 500 AEP	1 in 1000 AEP	1 in 2000 AEP
East Tamar Hwy (North)		0.09	0.31	0.88	1.29	1.74	2.43	3.02	3.70
East Tamar Hwy (South)			0.30	0.88	1.30	1.75	2.44	3.03	3.70
West Tamar Rd (South)	0.95	1.21	1.63	2.45	2.90	3.39	4.11	4.78	5.63
West Tamar Rd (North)	0.30	0.56	0.98	1.80	2.25	2.74	3.46	4.13	4.99
Home Point Pde		0.79	1.36	2.28	2.76	3.30	4.18	5.12	6.64
Henry St (South)	0.89	1.19	1.64	2.57	2.96	3.51	4.30	5.08	6.00
Henry St (North)	0.99	1.29	1.74	2.67	3.06	3.61	4.40	5.18	6.10
Ravenswood Rd	0.06	0.44	0.80	1.36	1.77	2.26	3.04	3.81	4.73
Waverley Rd			0.99	1.55	1.96	2.46	3.24	4.01	4.93
Hoblers Bridge Rd	0.63	0.95	1.26	1.71	2.05	2.38	2.96	3.72	4.63
Hart St		0.82	1.15	1.63	1.97	2.31	2.69	3.44	4.34
Killafaddy Rd	0.14	0.55	0.89	1.37	1.73	2.13	2.80	3.56	4.47
Johnson Rd		0.18	0.34	0.59	0.91	1.35	2.00	2.47	2.86
Station Rd	1.21	1.60	1.84	2.13	2.36	2.72	3.29	3.73	4.10
Glenwood Rd				0.10	0.33	0.68	1.25	1.69	2.06

(yellow)	Depth greater than 0.3 m, unsafe for small vehicles
(pink)	Depth greater than 0.5 m, unsafe for small all vehicles

5.2.7 Flood Planning Constraint Mapping

To assist Council with land use planning activities, a flood planning constraint map has been developed for 2050 climate conditions. *Guideline 7-5: Flood Information to Support Land-use Planning* of the Australian Disaster Resilience Handbook Collection (AIDR 2017) identifies four flood planning constraint categories (FPPCs) across a floodplain. For Launceston the FPCCs mapping has been produced using the following categorisation:

- **FPCC 1 -** Areas of flood hazard class 6 (Section 5.2.4) in the defined flood event (DFE) which is the 1% AEP event
- **FPCC 2 -** Areas of flood hazard class 5 in the DFE or of flood hazard class 6 in the 1 in 2000 AEP event

- **FPCC 3** – Areas within the DFE extent
- **FPCC 4** – Areas within the PMF extent

The resulting FPCC Map is presented in Figure 5-10. Please note, the extent of FPCC 3 does not include an allowance for freeboard. However, it is recommended that an allowance freeboard be considered when developing flood planning controls.

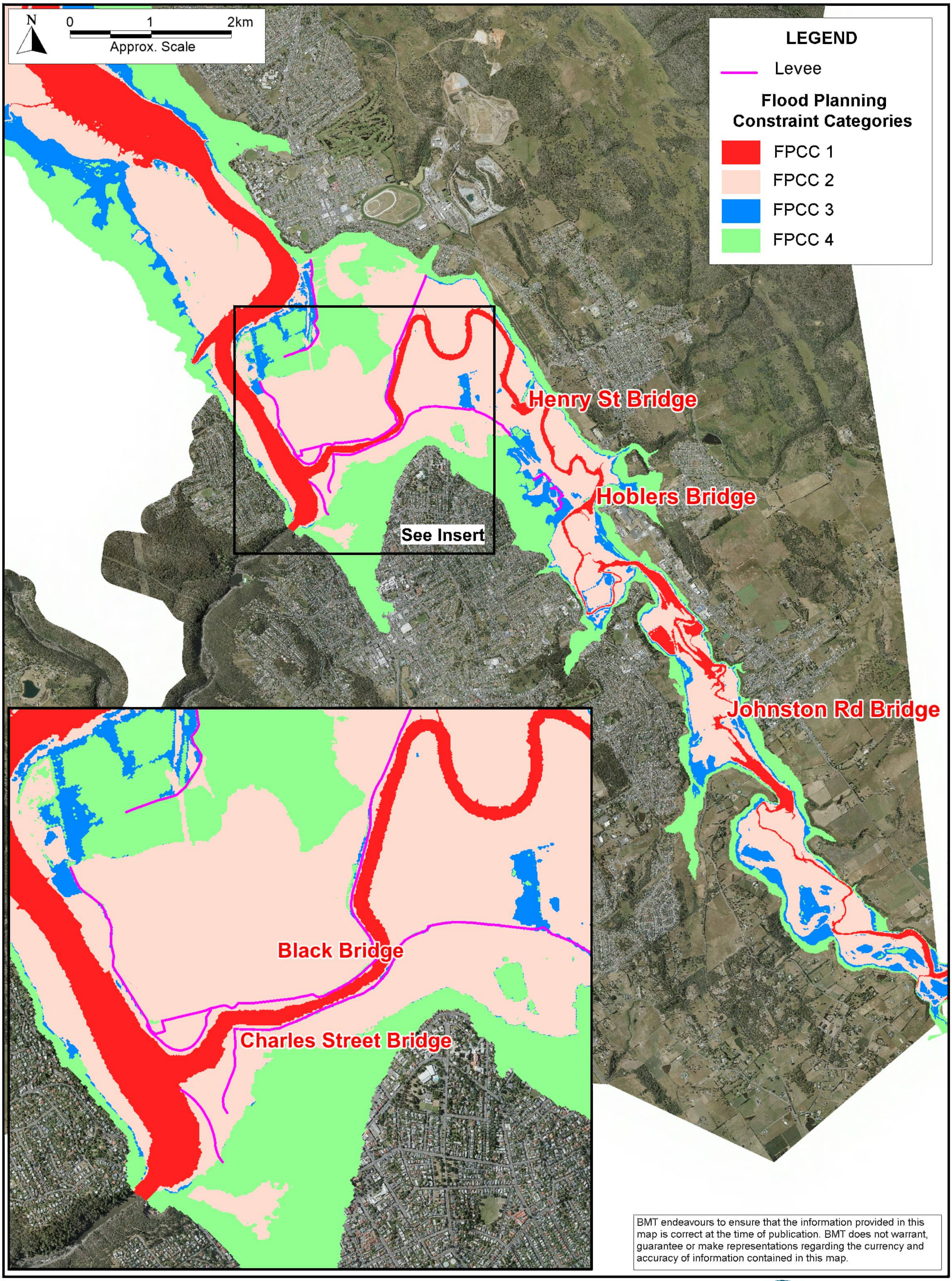


Figure 5-10 2050 Climate Conditions Flood Planning Constraint Map

T:\M20921.MS.Launceston_Mapping_2017\MapInfo\Drawings\Final\FIG5-10_LAU_CC2050_FPCC_RevA.WOR

6 Summary and Recommendations

This report documents the hydrologic and hydraulic modelling undertaken to update the flood mapping for the North and South Esk Rivers at Launceston. The updated flood mapping represents the following major improvements to the previous flood mapping:

- The flood modelling methodology has been updated to current best practice standards in line with ARR 2016
- The TUFLOW hydraulic model incorporates new LiDAR and ground survey topographic data and advances in computing have allowed for the model definition to be improved and extended up the North Esk River to the Corra Linn stream gauge
- The TUFLOW hydraulic has been calibrated to the June 2016 flood event for which a large amount of recent historic flood event data was available
- Additional streamflow data has been used to revise the FFAs defining the North and South Esk Rivers design event inflows
- A joint probability analysis has been undertaken to better define design flood levels
- An estimation of flood risk under 2050 and 2090 climate conditions based on the IPCC AR5, RCP 8.5 emissions estimates

It is recommended that the current riverine design event flood mapping be updated with the flood mapping presented in this report.

7 References

(AIDR) Australian Institute of Disaster Resilience (2017), ‘Guideline 7-5: Flood Information to Support Land-use Planning’, *Australian Disaster Resilience Handbook Collection*, Australian Institute for Disaster Resilience.

AustRoads (1994), *Waterway Design: A Guide to the Hydraulic Design of Bridges, Culverts and Floodways*, AustRoads.

Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I (Ed) (2016), *Australian Rainfall and Runoff: A Guide to Flood Estimation*, Commonwealth of Australia (Geoscience Australia).

BMT WBM (2008), *River Tamar & North Esk River Flood Study Final Report*, BMT WBM.

BoM (2006), Bureau of Meteorology, *The Estimation of Probable Maximum Precipitation in Australia: Generalised Southeast Australia Method*, Bureau of Meteorology.

Chow V (1959), *Open Channel Hydraulics*, McGraw-Hill Book Company, New York.

(DPIW) Department of Primary Industries and Water (2009), *Annual Waterways Report: North Esk Catchment*, Department of Primary Industries and Water (Water Assessment Branch).

(FEMA) Federal Emergency Management Agency (2004), *Guidelines and Specifications for Flood Hazard Mapping Partners [November 2004]*, Federal Emergency Management Agency, United States.

Fuller D, Norton K, Upreti R (1990), *Review of the Spillway Design Flood for Trevallyn Dam*, Hydro-Electric Commission.

HydroTAS (2008), *Trevallyn Flood Frequency Review for Launceston City Council*, Hydro Tasmania Consulting (Hydro-electric Corporation).

HydroTAS (2005), *North Esk Catchment Gauging and Rating Review*, Hydro Tasmania Consulting (Hydro-electric Corporation).

(IEA) Pilgrim D (Ed) (1987), *Australian Rainfall and Runoff: A Guide to Flood Estimation*, Vol 1, Institute of Engineers Australia.

McInnes K, Monselesan D, O’Grady J, Church J, Zhang X (2016), *Sea-Level Rise and Allownaces for Tasmania based on IPCC AR5*, Commonwealth Scientific and Industrial Research Organisation.

McInnes K, O’Grady J, Hemer M, Macadam I, Abbs D, White C, Corney S, Grose M, Holz G, Gaynor S, Bindoff N (2012), *Climate Futures for Tasmania: extreme tide and sea-level events*, The Antarctic Climate and Ecosystems Cooperative Research Centre.

Munro C (1959), Report on Flood Mitigation Measures for the City of Launceston, The University of New South Wales (Water Research Laboratory).

Nathan R, Weinmann P, Gato S (1994), ‘A Quick Method For Estimating the Probable Maximum Flood in South Eastern Australia’, *Water Down Under 94. Hydrology and Water Resources Symposium, Adelaide, 21-25 November 1994*, Institute of Engineers Australia, pp. 229-234.

Pedruco P, Westra S, Jones J (2013), 'Using joint probability methods to quantify the joint dependence of extreme events on the North and South Esk', Paper presented at 8th Victorian Flood Conference, Melbourne, 15 February 2013.

(WRL) Miller B, Badenhop A, Nittim R (2006), *North Esk Flood Frequency Analysis at Corra Linn*, The University of New South Wales (Water Research Laboratory).

Appendix A Detailed Assessment of Frameworks for Joint Probability Analysis

An assessment of joint probability was required for the flood mapping update due to the position of Launceston in the Joint Probability Zone (JPZ) of the confluence of the North Esk and South Esk Rivers, and the Tamar River estuary, resulting in a flood risk from river flows as well as ocean storm tides.

ARR 2016 provides guidance for carrying out Joint Probability Analysis (JPA) of flood levels where dependent (co-occurring) events may interact. Practical frameworks are provided in ARR 2016:

- In Book 4, Chapter 4, for JPA around the confluence of tributary river flows
- In Book 6, Chapter 5, for JPA in estuarine region, with interaction of coastal and catchment floods

The general framework in Book 4, Chapter 4 of ARR 2016 involves using Monte Carlo (stochastic) simulation and is depicted in Figure A - 1. This appendix documents how this framework was firstly applied for the JPA for North Esk and South Esk River fluvial flow flood events in Launceston, the limitations encountered when applying this framework, and the subsequent approach followed.

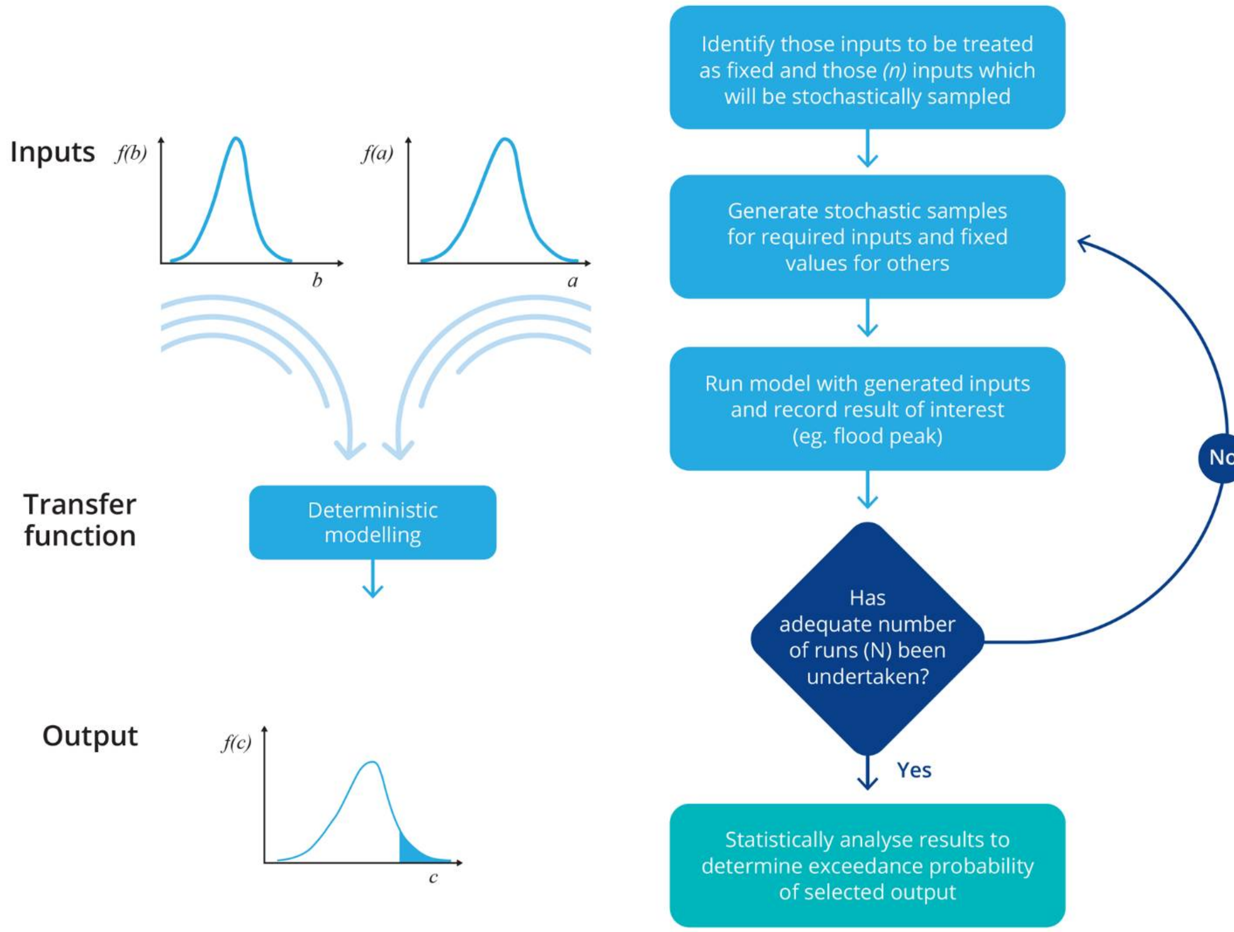


Figure A - 1 General framework for joint probability analysis of stochastic, deterministic problems using Monte Carlo Simulation (Nathan and Weinmann, 2016)

Historic flow data provided a series of historic coincident maximum (peak) fluvial flows of the North Esk (Tributary) and South Esk (Main) Rivers. Analyses of these flow historic maximum indicated mean ± standard variations of 292.9 ± 206.6 m^3/s and 776.9 ± 725.6 m^3/s for the North Esk and South Esk, respectively; and further, a correlation between maximum flows of 0.63.

A stochastic sample (of size *N*) of dependent variates was generated based on the correlation found between tributary flows, following the procedure in Book 4, Chaper4, Section 4.3.2.4, using a Normal distribution and scaling the normal variates by the relevant mean and standard deviation of the log-normal distribution. The historic maximum observed flows and the stochastic maximum samples generated are shown in Figure A - 2.

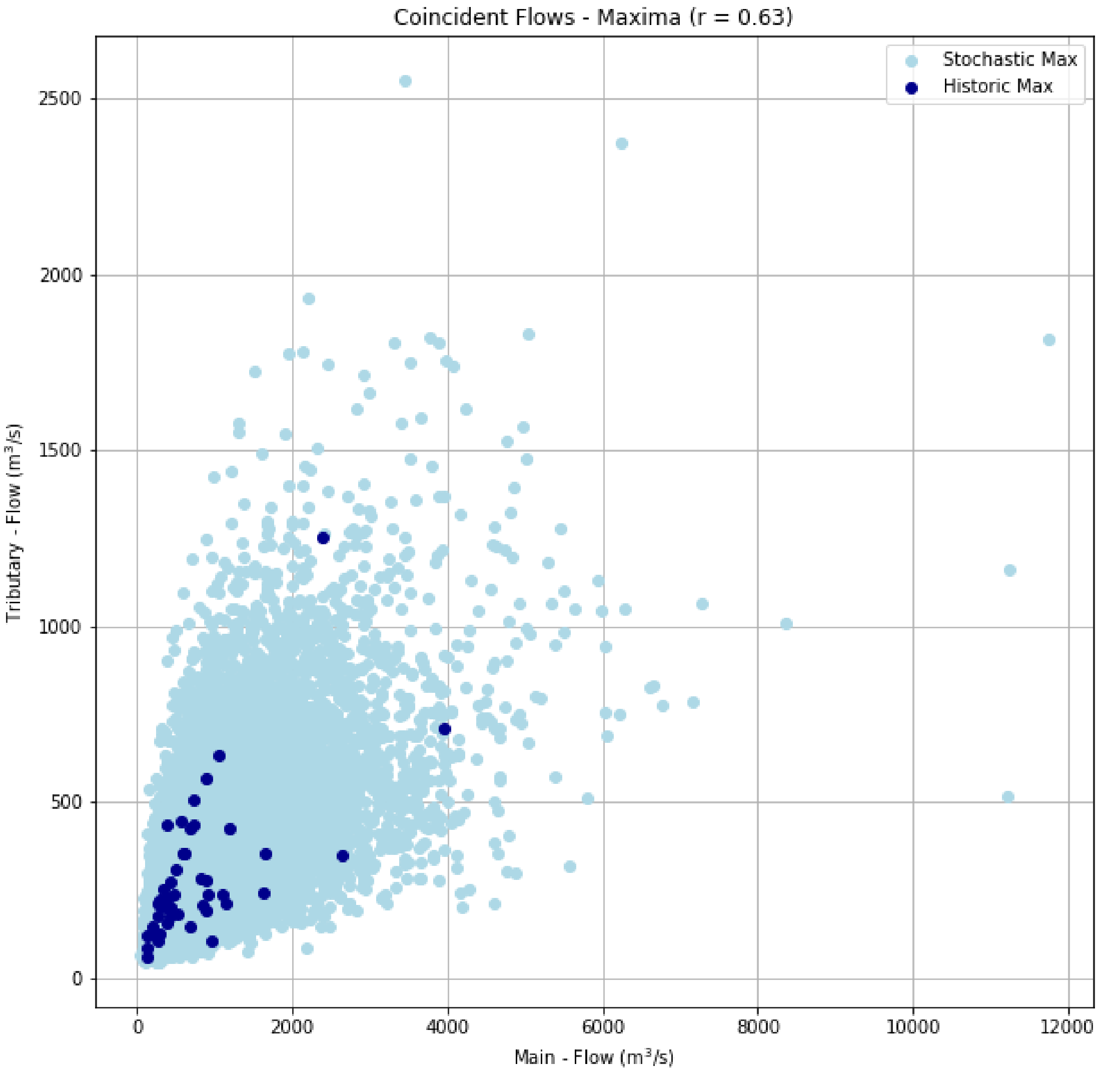


Figure A - 2 Correlation between maximum (peak) flows of South Esk (Main) and North Esk (Tributary) Rivers, for Historic events sample and Stochastically generated series

The next step was to derive a deterministic model (transfer function) of the relationship between tributary flows and flood levels downstream the confluence of the two Rivers. For this, representative fluvial storm flows of the Main and Tributary (based on the AEPs under consideration) were used as input to the hydraulic model to determine resulting flood levels at the points of interest in the fluvial JPZ (Charles St, Tamar St, Black Bridge, Henry St and Hoblers Bridge Rd). For each of these locations, a matrix of flood levels as a (transfer) function of the two tributary flows was produced. For this, a deterministic model for each location was derived by fitting a multiple regression model. An example of goodness of fit for Black Bridge is shown in Figure A - 3.

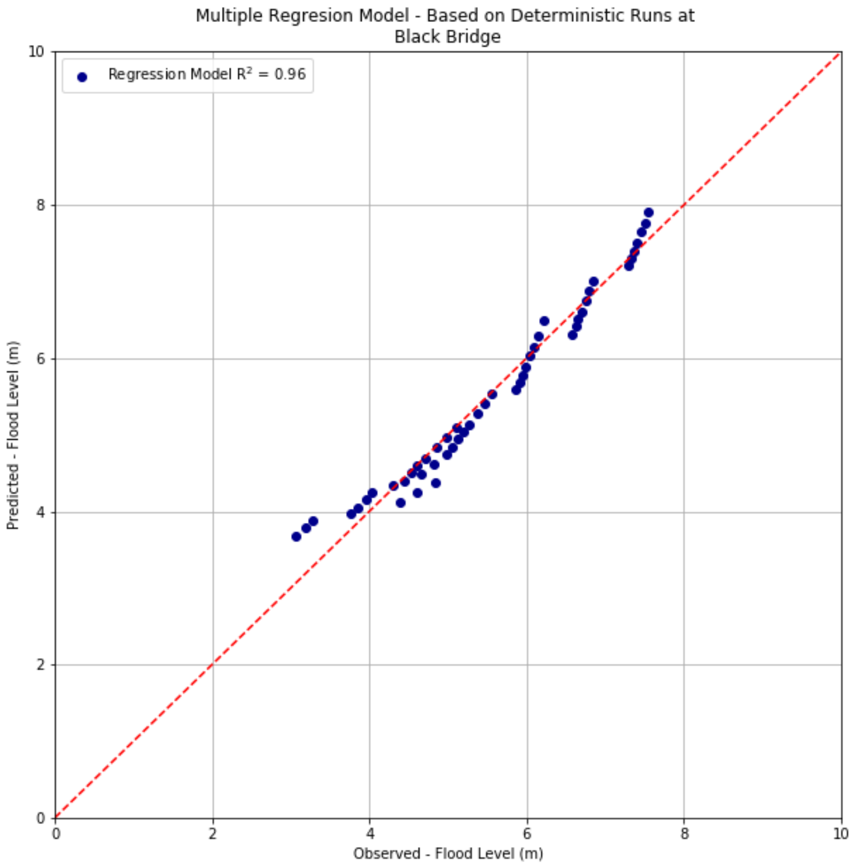


Figure A - 3 Derivation of Deterministic model relating upstream tributary flows of the South Esk and North Esk Rivers to downstream flood levels, showing goodness of fit at the Black Bridge location

Once the deterministic model was derived for each location, these models were evaluated for the stochastically generated sample of *N* tributary flows, producing *N* stochastic estimates of flood levels. The *N* stochastic flood levels were then ranked, using the Weibull plotting position formula. The joint probability flood levels for each AEP under consideration was finally determined by simple linear interpolation of the ranked results. Sensitivity testing of the stochastic sample size was conducted, finding *N* = 20,000 to be adequate as it yielded stable estimates of the AEPs (quantiles) under consideration. Example results of this analysis for Black Bridge are presented in Figure A - 4. A reasonable JPA result was obtained for the 1% AEP, falling within the complete dependent and complete independent estimates. However, inspection of the JPA results for other AEPs made evident that flood levels were not within the complete dependent and complete independent estimates for any other AEP, as the frequency curve obtained had a flat gradient.

Given that these results were not satisfactory, the framework was reapplied but using other distributions for generation of stochastic samples, instead of the Normal distribution. A Frèchet distribution for stochastic sample generation of tributary flows was tested; example of the frequency curve result for Black Bridge, obtained are shown in Figure A - 5. The resulting frequency curve had a reasonable slope; however, it had a substantial offset above the complete dependence estimations, making it again unreasonable, as it overpredicted the flood level for all the AEPs.

As none of the distributions tested on the tributary flows produced reasonable results, this suggested a non-linear dependence relationship between the tributary flows; potentially due to the influence of the Lake Trevallyn Dam in the South Esk River. Subsequently, a conditional (empirical) sampling approach was attempted to generate the stochastic samples, following the procedure described in Section 4.3.2.5 of Book4, Chapter 4 of ARR 2016. The resulting frequency curve obtained for Black Bridge using this approach is shown in **Figure A - 6**, depicting a more reasonable combination of slope and intercept, yet not fitting for all of the AEPs under consideration, as it underpredicted for some and overpredicted for other AEPs.

After conducting these various attempts at using this framework, it was considered that the results obtained were not satisfactory. Given these limitations, the alternative framework of the Design Variable Method of Book 6, Chapter 5 of ARR 2016 was opted for, and subsequently applied separately for the JPA of both tributary flows and estuarine regions.

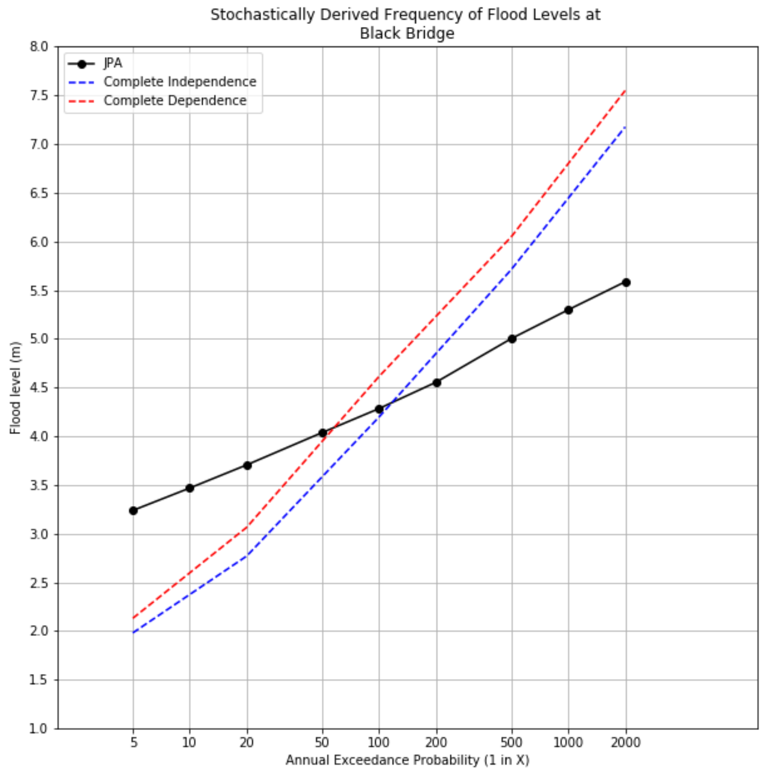


Figure A - 4 Frequency curve of flood levels at Black Bridge using Stochastic samples generated based on a Normal distribution

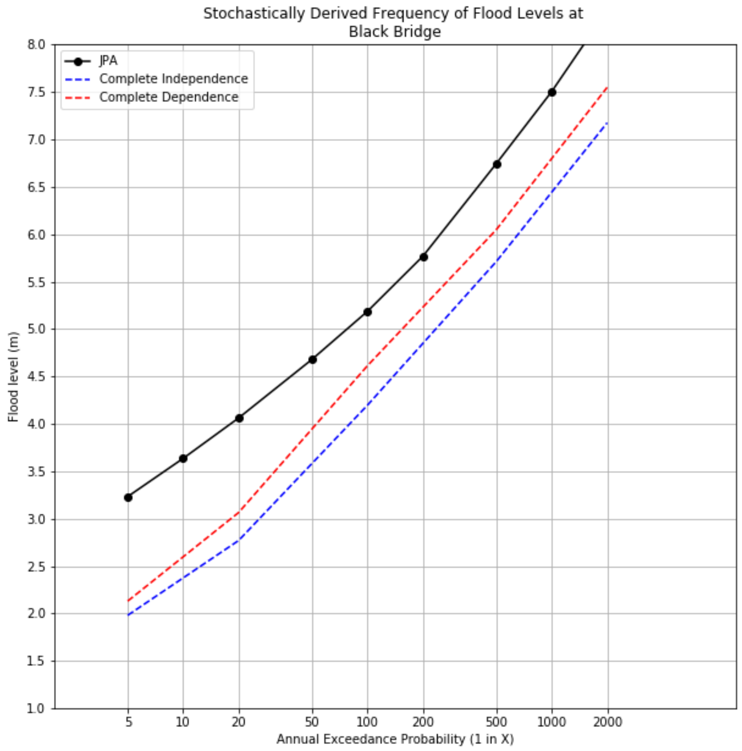


Figure A - 5 Frequency curve of flood levels at Black Bridge using Stochastic samples generated based on a Frèchet distribution.

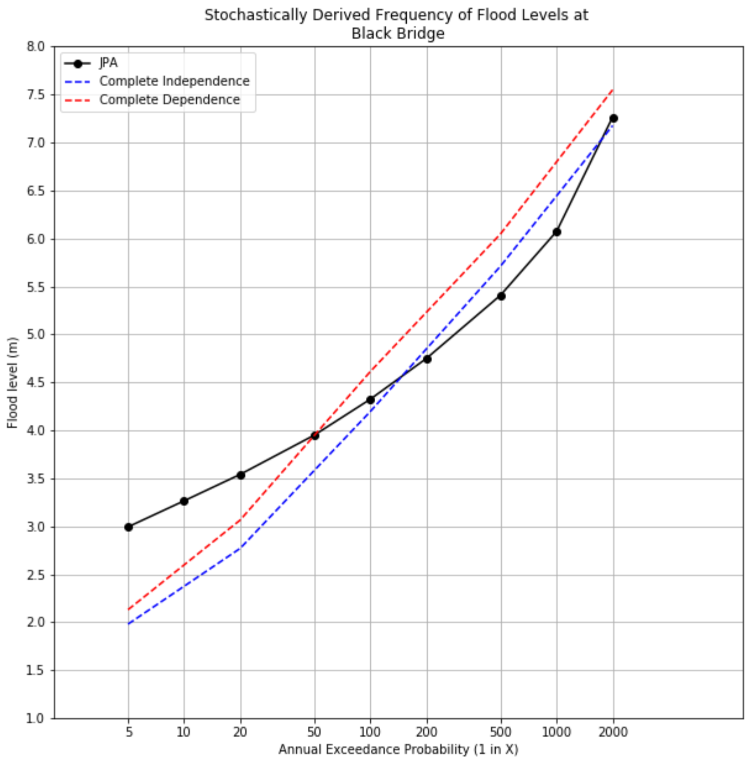


Figure A - 6 Frequency curve of flood levels at Black Bridge using Stochastic samples generated based on an Empirical distribution

Brisbane	Level 8, 200 Creek Street, Brisbane QLD 4000 PO Box 203, Spring Hill QLD 4004 Tel +61 7 3831 6744 Fax +61 7 3832 3627 Email brisbane@bmtglobal.com Web www.bmt.org
Denver	8200 S. Akron Street, #B120 Centennial, Denver Colorado 80112 USA Tel +1 303 792 9814 Fax +1 303 792 9742 Email denver@bmtglobal.com Web www.bmt.org
London	International House, 1st Floor St Katharine's Way, London E1W 1UN Tel +44 20 8090 1566 Fax +44 20 8943 5347 Email london@bmtglobal.com Web www.bmt.org
Melbourne	Level 5, 99 King Street, Melbourne 3000 Tel +61 3 8620 6100 Fax +61 3 8620 6105 Email melbourne@bmtglobal.com Web www.bmt.org
Newcastle	126 Belford Street, Broadmeadow 2292 PO Box 266, Broadmeadow NSW 2292 Tel +61 2 4940 8882 Fax +61 2 4940 8887 Email newcastle@bmtglobal.com Web www.bmt.org
Northern Rivers	6/20 Byron Street, Bangalow 2479 Tel +61 2 6687 0466 Fax +61 2 66870422 Email northernrivers@bmtglobal.com Web www.bmt.org
Perth	Level 4, 20 Parkland Road, Osborne, WA 6017 PO Box 2305, Churchlands, WA 6918 Tel +61 8 6163 4900 Email perth@bmtglobal.com Web www.bmt.org
Sydney	Suite G2, 13-15 Smail Street, Ultimo, Sydney, NSW, 2007 PO Box 1181, Broadway NSW 2007 Tel +61 2 8987 2900 Fax +61 2 8987 2999 Email sydney@bmtglobal.com Web www.bmt.org
Vancouver	Suite 401, 611 Alexander Street Vancouver, British Columbia V6A 1E1 Canada Tel +1 604 683 5777 Fax +1 604 608 3232 Email vancouver@bmtglobal.com Web www.bmt.org