Glenorchy CBD Stormwater System Management Plan

Report, November 2018

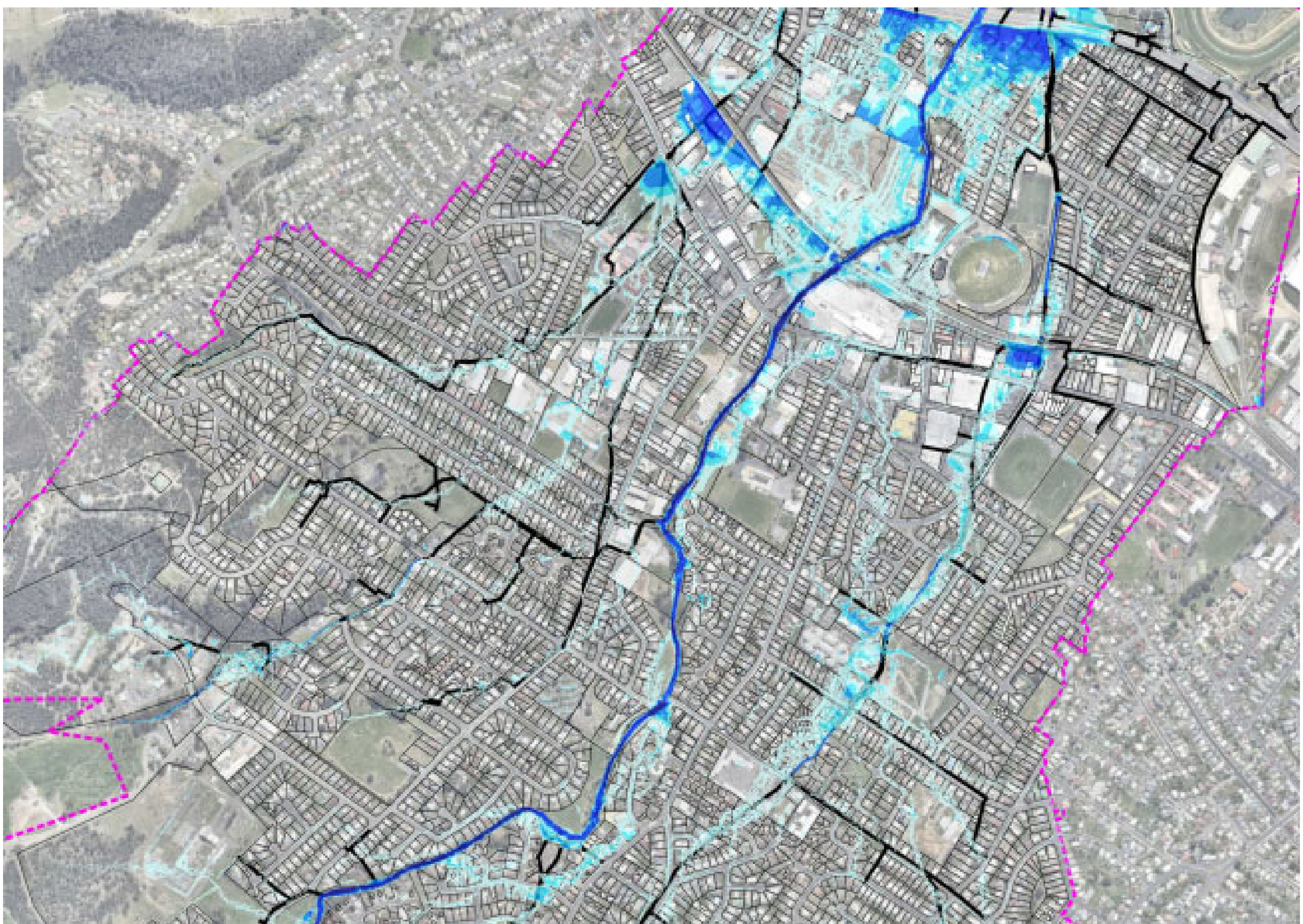

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TABLE OF CONTENTS

TABLE OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

The Glenorchy CBD and its surrounding area lie within three catchments that drain to Elwick Bay: Humphreys Rivulet, Littlejohn Creek and Barossa Creek. Together these three catchments form the Study Area (27 km^2) for this stormwater system management plan.

Glenorchy City Council (GCC) engaged SMEC to undertake a detailed flood study of Glenorchy CBD and surrounding area, situated within one of the northern suburbs of Hobart, Tasmania.

This report documents the work undertaken to develop the plan, including:

- A review of the available data and historic flooding records
- Hydrological modelling
- Hydraulic modelling
- Calibration of the models to the 1996 flood event
- Flood damage assessment

A rainfall-runoff model has been set up to describe the Study Area. The model was then calibrated to water levels measured at various locations within the Study Area after the February 1996 flood event. The model was then used to estimate inundation extents for a range of design flood events including the 1 in 20 AEP, 1 in 100 AEP and the Probable Maximum Flood (PMF) for several development scenarios including the impact of climate change.

The rainfall-runoff model was set up as a hybrid of RORB and Tuflow 'rainfall-on-grid' to assess the breakout flood risk from Humphreys Rivulet, Barossa Creek and Little John Creek and the existing drainage system performance.

A set of inundation extent and hazard maps have been generated for the different flood scenarios modelled to provide GCC with estimates of inundation levels. These will allow for planning controls, floor level setting and a baseline to plan mitigation options.

From the flood damage assessment, the Average Annual Damages (AAD) are currently estimated at $3.4 million per year. The developed scenarios indicate that these annual damages may increase by between $0.6 million and $3 million. The climate change scenario for the year 2090 estimates the Average Annual Damages (AAD) increase to over $19 million per year.

It should be noted that the flood damage assessment is likely to be sensitive to the assumption that floor levels are 300 mm higher than property ground levels. It is recommended that a floor level survey be completed for properties modelled as flooded in the 1 in 100 AEP events and the damage assessment revised.

ABBREVIATIONS AND ACRONYMS

Abbreviation/ Acronym	Description
AEP	Annual Exceedance Probability
ANCOLD	Australian National Committee on Large Dams Inc.
ARF	Areal Reduction Factor reduces the design rainfall as the catchment area increases
AVM	Average Variability Method uses a representative design rainfall temporal pattern per duration
BoM	The Australian Bureau of Meteorology
CFD	Computational Fluid Dynamics
CL	Continuing Loss (mm/hr)
DCF	Dam Crest Flood
DV	Product of depth and velocity (m^2/s)
FSL	Full Supply Level
GCC	Glenorchy City Council
GSAM	Generalised South Australia Method estimates PMP rainfall for durations equal or longer than 24 hours appropriate to the South East of Australia
GSDM	Generalised Short-Duration Method estimates PMP rainfall for durations equal to or shorter than 6 hours
HAT	Highest Astronomical Tide (mAHD)
IFD	Intensity Frequency Duration refers to statistics on design rainfall
IL	Initial Loss (mm)
IWL	Initial Water Level describing the first water level during a stormwater model simulation
k_c	Catchment routing parameter used in the rainfall-runoff model
PMF	Probable Maximum Flood is the theoretical largest discharge combining the most saturated catchment conditions with the largest rainfall (PMP) (m^2/s)
PMP	Probable Maximum Precipitation is the theoretical largest rainfall (mm)
Q	Discharge (m^3/s)
RCP	Representative Concentration Pathways are scenarios of future greenhouse gas trajectories
RFFE	Regional Flood Frequency Estimate
SLR	Sea Level Rise (m)
T_c	Catchment lag time used in the rainfall-runoff model (hr)

1. INTRODUCTION

1.1. General

SMEC Australia has been engaged by Glenorchy City Council (GCC) to undertake detailed flood study of Glenorchy CBD and its surrounding area, situated within one of the northern suburbs of Hobart, Tasmania. The study has been undertaken to advise future land use planning, emergency management, community consultation/education and capital expenditure planning.

The Study Area includes the Humphreys Rivulet, Barossa Creek and Little John Creek catchments with a focus on the breakout flooding from the waterways and the performance of the major stormwater system.

The objectives of the analysis as stated by GCC are as follows:

- Ensure an appropriate level of understanding and management of the flood risk and public stormwater systems within the Study Area.
- Evaluate the hydraulic performance of the major stormwater reticulation system.
- Identify the overland flow paths and associated hazard levels within the Study Area.

GCC wish to use the outcomes of the report to:

- Develop and prioritise future capital works, forecast and prepare budgets, and specify cost apportionment arrangements between GCC, State Government and other stakeholders (e.g. developers).
- Build resilience and consider climate change impacts to address future demands on the urban stormwater system.
- Integrate stormwater management into the urban water cycle to achieve the goals of social, environmental and economic sustainability.
- Enhance community awareness of, and participation in, appropriate management of stormwater.

1.2. Scope of Work

The scope of work includes the following tasks

- Collate data
- Hydrological modelling
    - Update and validate RORB model to Australian Rainfall and Runoff (Ball et al. 2016)
    - Sensitivity of RORB model parameters
    - Modelling event scenarios
- Hydraulic modelling
    - Update existing Tuflow model developed by SMEC for a previous study
    - Calibrate Tuflow model to historic event(s)
    - Complete a sensitivity analysis of selected Tuflow model parameters
    - Modelling events scenarios
- Flood damage assessment
- Reporting
    - Hydrology draft
    - Final study report (this document)

1.3. Peer Review

As recommended by SMEC, WMAwater was engaged by GCC to provide peer review for the flood study. Reviews were completed at the following stages of the project development:

- After completion of the RORB modelling and issue of the Hydrology Report
- After calibration of the Tuflow to measured water levels, as a hold point prior to commencing design runs
- Ongoing, as methodology is discussed and agreed upon.
- Prior to the issuing of the final study report.

2. DATA RECEIVED

A variety of documents were provided by GCC to assist in the analysis as follows:

- Humphreys Rivulet Flood Protection Assessment (Thompson & Brett, 1997)
- Barossa Creek Flooding Analysis (Thompson & Brett, 1999)
- Report on Flooding – Glenorchy Creeks (GCC Engineering Department, 1967)
- Report on Flood Protection, Hobart Rivulet (Hobart Rivulet Flood Protection Authority, 1963)
- Report of the Hobart Rivulet Advisory Committee (1960)
- City of Glenorchy Stormwater System Management Plan – Roseneath Rivulet Catchment Flood Study (GCC 2017)
- Flood Hazard Mapping Study for Humphreys Rivulet (BMTWBM, 2013)
- Drawings of bridge crossings over Humphreys Rivulet at Brent St, Grove Rd, Main Rd, and KGV
- GIS data
    - Pipe network
    - Pit network
    - Building outlines
    - Parcel data
    - Contours 2, 5, 10m
    - Planning Scheme Zoning regions
    - Road layout and names
    - Stormwater catchments
    - Digital elevation model (DEM)
- Rainfall data
    - From event on 21st January 2007

In addition, a range of data was downloaded from the Tasmania Government Department of Primary Industries, Parks, Water and Environment website (http://dpipwe.tas.gov.au/) as follows:

- Lidar with the following characteristics
    - 1 m resolution captured by Photomapping Services (2011),
    - Horizontal spatial accuracy is 0.30 m; Vertical accuracy is 0.15 m.
    - Map projection is GDAS94 MGA55.
    - from www.theLIST.tas.gov.au ©State of Tasmania
- October 2015 Aerial Imagery from www.theLIST.tas.gov.au ©State of Tasmania

3. DESCRIPTION OF CATCHMENTS

3.1. General

The Study Area is 27 km^2 including three catchments that drain to Elwick Bay: Humphreys Rivulet (19.3 km^2), Littlejohn Creek (2.7 km^2) and Barossa Creek (5.0 km^2). There is some interaction between the creeks during major flood events (refer to Figure 3-1). Knights Creek and Islet Rivulet are two tributaries of Humphreys Rivulet.

Most of the Study Area is rural forested catchment on the slopes of Mt Wellington. In the north of the Study Area is Glenorchy, Hobart. This region (8 km^2) consists of low and medium density residential and commercial/industrial businesses. There are also many parks and recreational facilities throughout the urban region.

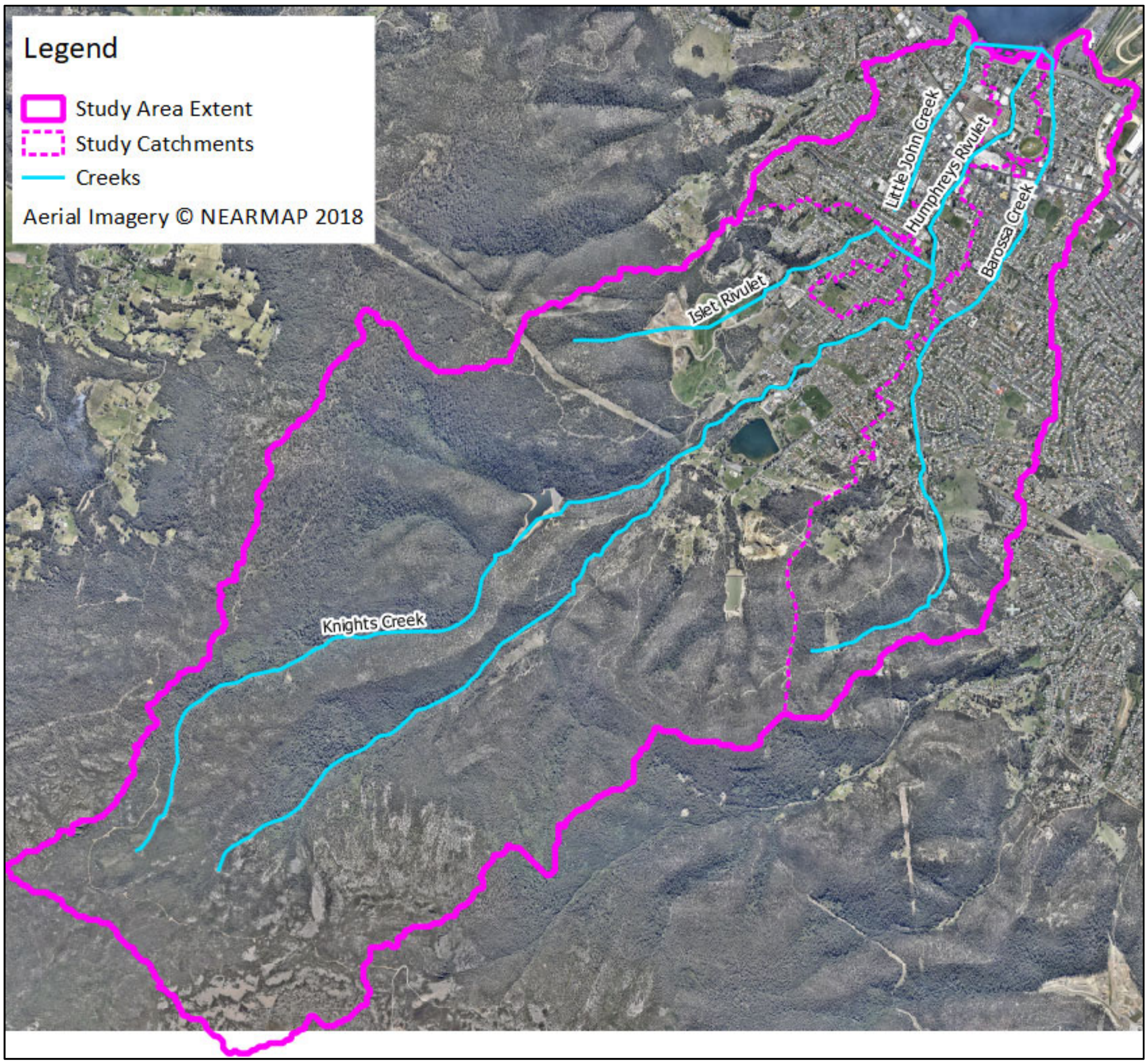


Figure 3-1 – Study Area Locality Plan.

To assess the flood risk of the Glenorchy stormwater system, a hydrology analysis is required to provide design hydrographs to input into a hydraulic model of the urban catchments of the Study Area. It is desirable to have the design hydrographs match the reality of major historic flood events to customise the flood model to match the real catchment conditions. This is done through calibration.

SMEC developed a hydrological model for the investigation of the dams on Knights Creek and Limekiln Gully and Tolosa Reservoir (SMEC 2017). That rainfall-runoff model was developed for the purposes of dam break modelling and consequence assessment. The definition of sub-catchments was fairly coarse through the urban areas. This definition was insufficient to provide direct inputs to the Glenorchy stormwater system.

In this study, the hydrological model previously developed by SMEC was updated and used to determine flows from the upper forested catchments (including the three dams, Knights Creek Dam, Limekiln Gully Dam and Tolosa Reservoir) only. The flows from the urban catchment was developed using the rainfall-on-grid method within a Tuflow model. This method is discussed in more detail in the Tuflow Model section (refer to Section 4.3).

The hydrology is, therefore, a hybrid of the two models, routed through the catchments using different mechanisms and computational methods, but calibrated to the same historic storms.

The hydrological analysis has comprised a range of sub tasks as follows:

- Update the rainfall-runoff model for the Study Area previously developed, and remove the urban sub-catchments
- Analyse regional rainfall data to develop an understanding of catchment runoff processes.
- Derive parameters for the updated rainfall-runoff model.
- Input dam storage and flow relationships
- Selection of model parameters
- Run the model with a range of scenarios
- Undertake sensitivity assessments

3.1.1. Catchment Hydrology

The Study Area encompasses the catchments of Humphreys Rivulet, Littlejohn Creek and Barossa Creek. The rainfall gauges in the region indicate that there is a strong orographic influence on rainfall (Table 3-1).

Table 3-1 – Orographic relationship between rainfall depth and elevation

Location	Elevation (m AHD)	Average Rainfall Depth (mm)
Hobart – Botanical Gardens	27	574
Hobart – Ellerslie Road	51	614
Glenorchy Reservoir	93	764
Mount Wellington – Kunanyi	1260	1,155

There are three dams within the Study Area, namely Knights Creek Dam, Limekiln Gully Dam and Tolosa Reservoir. These were previously water supply storages. It is understood that TasWater is intending to decommission Tolosa Reservoir in 2019.

3.2. Flood Frequency Analysis

Data from adjacent catchments has been analysed to develop an understanding of rainfall-runoff processes in the region and to further assist in developing rainfall-runoff model parameters for the Study Area.

A flood estimate for the 1 in 100 AEP event has been generated for the purposes of defining a target flow to use in RORB model calibration. The flow has been estimated through:

- Flood frequency analyses derived from peak observed flows from nearby catchments.
- A regional flood frequency analysis based on data across southeastern Tasmania.

In addition to the 1 in 100 AEP flood event flow estimation, the hybrid RORB/Tuflow model was calibrated to the flood levels recorded during the February 1996 event as documented by Thompson & Brett (1997).

3.2.1. Regional Analysis

Flow data was obtained for a range of gauging stations in the Hobart region and surrounds. Data for the majority of the catchments was obtained from the Water Information System of Tasmania (WIST) website which is managed by the Department of Primary Industries, Parks, Water and Environment (DPIPWE).

http://wrt.tas.gov.au/wist/ui?command=content&pageSequenceNo=41&click=[0].HomeLink#fopt

The catchment areas contributing to each gauge site were estimated using available contour data. Catchment areas were supplied for the Hobart Rivulet gauges by Hobart City Council. The data downloaded from the WIST website was supplemented in the case of the Hobart Rivulet @ Gore Street gauge by data sourced from HEC (1997). The available data for each gauge is listed in Table 3-2.

Table 3-2 – Streamflow Gauging Station Characteristics

Gauge Location	No.	Period of Record	Source
Hobart Rivulet @ Gore Street	353	1962-1985 (peak annual flow)	HEC (1997)
		1986-2016 (peak daily flow), 1997-06/2006 &1994 missing	HCC
Hobart Rivulet @Argyle Street.	354	1985-1994 (peak daily flow)	WIST
Peak Rivulet @ 3.5km upstream Esperence River	1012	1975-1997 (peak daily flow)	WIST
Jordan River @ Bridgewater	4210	1983-1992 (peak daily flow)	WIST
Browns River @ Summerleas Road	5200	1963-1992 (peak daily flow)	WIST
Mountain River @ Grundys Creek	6200	1968-1996 (peak daily flow)	WIST

Flood frequency analyses were undertaken on those data sets to obtain flow estimates for a range of probability events. The flood frequency analysis was undertaken applying the Tuflow-Flike software package. Flike is an extreme value analysis package that allows users to match a range of probability distributions (Generalised Extreme Value (GEV), Log Pearson three (LP3), Log Normal, Gumbel and Generalised Pareto) with a fitting method (Bayesian Inference Method and higher order (H) linear (L) Moment ratios). The fitting methods are used by Flike to fit the flow data to the probability distribution.

Multiple combinations of fitting method and probability distribution were trialled to select the best fit. Preference was made based on the current understanding of the best performing curve fitting techniques in South Eastern Australia (Rahman et.al. 2009).

In undertaking the analysis, the curve fit adopted was GEV with optimised LH moments. In a couple cases, Bayesian fitting or LH moments of zero provided a better fit, and these became the selected outcome.

The flood quantiles from the fitted distributions are listed in Table 3-3. Some gauge sites have a record length that was too short to give confidence to some flood quantiles and these cells have been greyed out in Table 3-3. Typically, confidence was given to record length (in years) that were roughly half or more than that of the AEP quantile (1 in y).

Graphs of the curve fits and text output are included as Appendix A.

Table 3-3 - Flood Frequency Analysis results for regional catchments

River	Catchment Area (km²)	Record Length (years)	Peak Flow (m³/s)			
			1 in 10 AEP	1 in 20 AEP	1 in 50 AEP	1 in 100 AEP
Hobart Rivulet @ Gore Street	16.3	47	27	35	48	61
Hobart Rivulet @Argyle Street.	19	10	60	75.7		
Peak Rivulet @ 3.5km upstream Esperence River	36.5	22	99	123	159	
Browns River @ Summerleas Road	11.1	29	20	28	42	
Mountain River @ Grundys Creek	40	28	61	75	93	

The outcomes presented in the above table are for a variety of different catchment areas. The flows from Table 3-3 have been modified in Table 3-4 to allow a direct comparison with the Study Area. The flows have been modified based upon catchment area by applying the following equation as described in Grayson et.al. (1996):

$$Q_{(\text{Unregulated Catchment})} = Q_{(\text{Gauged Catchment})} \times (A_{(\text{Unregulated Catchment})} / A_{(\text{Gauged Catchment})})^{0.7}$$

Where:

Q = Discharge (m^3/s)

A = Catchment Area (km^2)

Note: Exponent can vary between 0.5 and 0.85. If data is available, the exponent may be calibrated, otherwise, 0.7 is typically applied (Grayson et.al. 1996).

The exponent of 0.7 has been adopted as there is no flow record within the study area to calibrate the exponent. The Study Area catchment area (27.3 km^2) versus each gauged catchment area ratio (i.e. the $(A_{(U)} / A_{(G)})^{0.7}$ portion of the equation above) has been calculated and listed in the second column of Table 3-4 as 'Multiplier'. These modified flow values are presented in Table 3-4.

Table 3-4 - Flood Frequency Adjusted for the Study Area Catchment Area (27.3 km^2)

River	Grayson et. al 1996 Multiplier	Peak Flow (m³/s)			
		1 in 10 AEP	**1 in 20 AEP**	**1 in 50 AEP**	**1 in 100 AEP**
Hobart Rivulet @ Gore Street	1.43	38.7	49.6	68.6	87.5
Hobart Rivulet @Argyle Street.	1.29	77.3	97.6		
Peak Rivulet @ 3.5km upstream Esperence River	0.82	80.8	100.4	129.7	
Browns River @ Summerleas Rd	1.88	37.6	52.6	78.9	
Mountain River @ Grundys Crk	0.77	46.7	57.4	71.2	

Peak Rivulet is located in the Huon catchment and the flow conditions relating to the weather patterns of this catchment may not be representative of those in the Study Area. The Hobart Rivulet at Argyle Street gauge presents different outcomes to the Gore Street gauge despite being on the same watercourse and having similar catchment areas. It has been reported that following a review of the two gauges by Entura, the Gore Street gauge is considered to be more reliable (Fiona Ling, WMAWater, pers. comm.) After considering these exclusions, of the remaining observed flows above, it is considered that the most appropriate comparison gauges are Hobart Rivulet at Gore Street, Mountain River and Browns River.

Of the nearby gauges, the Gore Street gauge has the longest period of record and is the only gauge for which a 1 in 100 AEP flow estimate may be considered to be reliable. Notwithstanding this observation, it is noted that the rainfall of record for the Hobart Rivulet catchment occurred in 1960 is not captured in the flow data record. The Hobart Rivulet Advisory Committee (1960) estimates the 1960 flow as 2460 ft^3/s or 69.7 m^3/s. Without inclusion of that data point the 1 in 100 AEP flow estimate is 34.3m^3/s. With the 1960 data point included the flow estimate increases to 61m^3/s. There is confidence that this data point is not an outlier as similar depth rainfalls were recorded in pluvio records of 1854, 1954 and 1957.

While no 1 in 100 AEP flow estimates are available for the Mountain River and Browns River gauges, the 1 in 50 AEP estimates for those stations are substantially higher than the Hobart Rivulet gauge (without 1960) by factors of 1.4 and 1.8 respectively, but comparable when the 1960 data point is included. Overall, the available catchment data indicates that a 1 in 100 AEP flow for the Study Area is in the range of 80 m^3/s to 90 m^3/s.

3.2.2. Regional Flood Frequency Estimate (RFFE)

In addition to the outcomes described above, a regional analysis has been undertaken using a newly developed regional procedure called Regional Flood Frequency Estimate (RFFE) described in Ball et al. (2016). RFFE has been computed utilising the relevant website (accessed 7/11/16) as follows: http://rffe.arr-software.org/ The estimates from this analysis are presented in Table 3-5.

Table 3-5 – Regional Flood Frequency Estimate

Catchment Area (km^2)	Peak Flow (m^3/s)			
	1 in 10 AEP	**1 in 20 AEP**	**1 in 50 AEP**	**1 in 100 AEP**
27.3	25.4 (Range 10-64)	34.6 (Range 11-100)	48.3 (Range 12-170)	60.6 (Range 13-250)

The regional analysis draws upon nearby gauging stations from a database of catchments across Tasmania. In the case of this analysis, the outcomes from 15 catchments in central and eastern Tasmania have been drawn from.

The analysis provides a useful comparator, although the flow estimate is not considered to be highly accurate (note the wide confidence limits indicated by the range) for the Study Area given the wide variety of different hydrologic conditions in the 15 regional catchments. There are five nearby gauges used by RFFE that vary in area between 75% and 190% of the Study Area.

3.2.3. Previous Studies

A range of studies have been undertaken by others in the past which have applied various techniques to estimate flows in Humphreys Rivulet and nearby catchments.

Three studies have provided estimates for Humphreys Rivulet and one has provided an estimate for the nearby Browns Rivulet.

The various studies have reported on flows in different portions of the catchment. The flows have been documented in Table 3-6 along with the equivalent flow for the larger Study Area catchment at the downstream outlet to Elwick Bay. The flows have been altered for catchment area using the same formula described above.

Table 3-6 – Flow Comparison (other studies)

Study		1 in 100 AEP flow

	Catchment Area (km²)	Reported	27 km² Equivalent
Thompson & Brett (1997)	19	88	113
Thompson & Brett (2002)	18	108	145
BMTWBM (2012)	14	50	80
GHD (2016) – Browns Rivulet	12	47	84

The above-listed reports each incorporated the development of a rainfall-runoff model. Only GHD (2016) had sufficient data to undertake a calibration. Thompson and Brett (2002) validated modelled outcomes against limited observed flow depths for a few flood events in the mid-1990s. The model described in BMTWBM (2012) was not calibrated or validated. Of the above studies, therefore, the greatest confidence should be placed in the outcomes from GHD (2016). The result from GHD (2016) is within the 60 m^3/s to 100 m^3/s range suggested by the flood frequency analysis.

3.2.4. Summary of Flood Frequency Analysis

The outcomes from the analysis are presented in Table 3-7.

Table 3-7 - Flood Frequency Adjusted for Catchment Area

River	Catchment Area (km²)	Multiplier	Peak Flow (m^3/s)			
			1 in 10 AEP	**1 in 20 AEP**	**1 in 50 AEP**	**1 in 100 AEP**
Hobart Rivulet @ Gore Street	27.3	1.43	38.7	49.6	68.6	87.5
Hobart Rivulet @Argyle Street.	27.3	1.29	77.3	97.6		
Peak Rivulet @ 3.5km upstream Esperence River	27.3	0.82	80.8	100.4	129.7	
Browns River @ Summerleas Road	27.3	1.88	37.6	52.6	78.9	
Mountain River @ Grundys Creek	27.3	0.77	46.7	57.4	71.2	
RFFE	27.3	-	25.4	34.6	48.3	60.6 (Range 13-250)
Thompson & Brett (1997)	27.3	1.29	55.4	70.9	92.8	113
Thompson & Brett (2002)	27.3	1.34			119	145
BMTWBM (2012)	27.3	1.60				80
GHD (2016) – Browns Rivulet	27.3	1.78		55.1	72.9	83.6

Based upon the outcomes of regional flood frequency analyses and studies undertaken by others, it is considered that the 1 in 100 AEP flow at the outlet of Humphreys Rivulet and inclusive of Littlejohn Creek and Barossa Creek is between 60 m^3/s and 100 m^3/s. It is considered that a flow towards the higher end of that range should be adopted based upon both recorded flows and the outcomes from other studies. A target 1 in 100 AEP peak flow of 80 m^3/s -90 m^3/s has been targeted for the purposes of calibrating the rainfall-runoff model.

3.3. Design Rainfall

3.3.1. General

Design rainfalls were developed for the Studay Area and applied to the RORB and Tuflow models.

In following the Generalised Short-Duration Method (GSDM), the Study Area was estimated to consist of 100% rough terrain to determine the PMP depths (BoM 2003).

3.3.2. Design Rainfall Estimation

Design rainfall depths used in the development of the RORB storm files were obtained as follows:

- 1 in 20 and 1 in 100 AEP design rainfalls were estimated using the online Bureau of Meteorology (BoM) website tool located at http://www.bom.gov.au/water/designRainfalls/ifd/index.shtml. It may be noted that currently there are two IFD relationships available on this website, being 1987 and 2016 data sets. The 2016 IFD data set has been applied in this analysis.
- Areal reduction factors (ARF) were applied to rainfalls using the procedure described in Ball et. al. (2016). Book 2, Section 2.4.3 Equation 2.4.1 was used for durations shorter than 24 hours and for durations of 24 hours and longer Equation 2.4.4 was used with the Tasmania region coefficients.
- Probable Maximum Precipitation (PMP) rainfall depths for durations up to 6 hours were developed using BoM (2003).
- Probable Maximum Precipitation (PMP) rainfall depths for durations greater than 6 hours were developed using the Generalised Southeast Australia Method (GSAM) described in BoM (2006).

Rainfall depths (including areal reduction factors (ARF)) for the study are included as Appendix B.

3.3.3. Losses

Ball et al. (2016) provide guidance on loss models to apply and values to adopt when undertaking rainfall-runoff modelling. When undertaking extreme flood analyses, it is preferred that the continuing loss model be used, since there is explicit guidance on how to evaluate continuing losses for extreme flood events. There is no similar guidance available in extreme events for the proportional loss model. In more frequent events, it is open to the practitioner as to which loss model to apply. Ball et al. (2016) recommend the use of continuing loss for ungauged catchments.

This analysis has considered both of the loss models. However, it has adopted the continuing loss model for all scenarios. Laurenson et al. (2010) recommend that the loss values should be determined through a calibration utilising rainfall and runoff data from selected historical storm events. Where there is insufficient data on or near the catchment under investigation, then the approach can be to apply regional values, to review available data from similar catchments or other studies and to undertake a reconciliation against independent flood frequency estimates.

There is insufficient data on the rural catchment in the Study Area to calibrate that model portion in isolation. However, water levels recorded after the February 1996 event allow for calibration of the entire model. See Section 5 for more details.

Regional Values

Ball et al. (2016) documents regional approaches to the estimation of initial and continuing loss. In addition, it documents outcomes from analysis of a select number of catchments, including the Hobart Rivulet Gauge and Argyle Street. The outcomes of the assessment are detailed in Table 3-8.

Table 3-8 – Regional Loss Values

Source	Initial Loss (mm)	Proportional Loss	Continuing Loss (mm/hr)
Regional Data	28	NA	3.8
Hobart Rivulet Data	1-18 (Range) 7.9 (average)	0.05-0.95 (Range) 0.55 (average)	1-5 (Range) 1.4 (average)

Other Studies

Losses on the Study Area and nearby catchments have been assessed as part of flood studies undertaken by others. The outcomes and adopted loss values are reported in Table 3-9.

Table 3-9 – Loss Comparison (other studies)

Study	Initial Loss (mm)		Continuing Loss (mm/hr)	
	Calibration	Adopted	Calibration	Adopted
Thompson & Brett (1997)	0-35	15	0-4	1.5 (calibration) 2.5 (design runs)
Thompson & Brett (2002)	NA	20-0 (varies)	NA	2.5-0 (varies)
BMTWBM (2012)	NA	15	NA	Not reported
Engineers Australia (2015)	0-8	8	0.9-7	2
GHD (2016) – Browns Rivulet	10-40	15	5-11	1.5

The previous studies applied burst temporal patterns, so the initial losses apply to the burst and cannot be directly applied to the complete storm temporal patterns used in this study. A larger storm initial loss is appropriate, to account for losses during pre-burst and burst portions of the storm.

Observed Data

A limited quantum of observed water level data was made available for Knights Creek Dam (TasWater water level telemetry from 2004-2016). This data was used to assess the quantum of runoff entering the reservoir as a proportion of the incident rainfall. Rainfall data from the closest available rainfall gauge was used in the analysis, and it was assumed that there was zero outflow from the dam during the event. The results of the analysis are presented in Table 3-10.

Table 3-10 – Observed Proportional Loss

Event date	Proportional Loss
22/07/2013	0.4
12/08/2010	0.6
15/01/2015	0.6
30/01/2004	0.7

Comparison of losses

The range of loss estimates as detailed above are presented in Table 3-11. 'Calibration' loss estimates are described in detail in Section 5 from the calibration of the hydrologic and hydraulic models to recorded water levels for a historic storm event.

Table 3-11 – Loss Estimates

Source	Initial Loss	Proportional Loss	Continuing Loss (mm/hr)
Regional Values	1-28	0.05-0.95	1-5
Other Studies	0-20	NA	0-11
Observed Data	NA	0.4-0.7	NA
Calibration (refer Section 5)	**28**	**NA**	**1.5**

For comparison, a flood frequency curve of the RFFE estimates, other regional estimates, previous studies estimates, and the final calibration outcomes are plotted in Figure 3-2. All these data sets of catchments different to the Study Area were scaled to the Study Area catchment area of 27.3 km^2 using the Grayson et. al. (1996) equation described in Section 3.2.1.

In sensitivity testing of the model parameters (refer to Section 4.4.2) other parameter sets were trialled, two of which are also plotted in Figure 3-2: continuing loss of 7.5 mm/hr, initial loss of 29 mm and k_c of 7.0; and proportional loss of 38%, initial loss of 29 mm and k_c of 7.0.

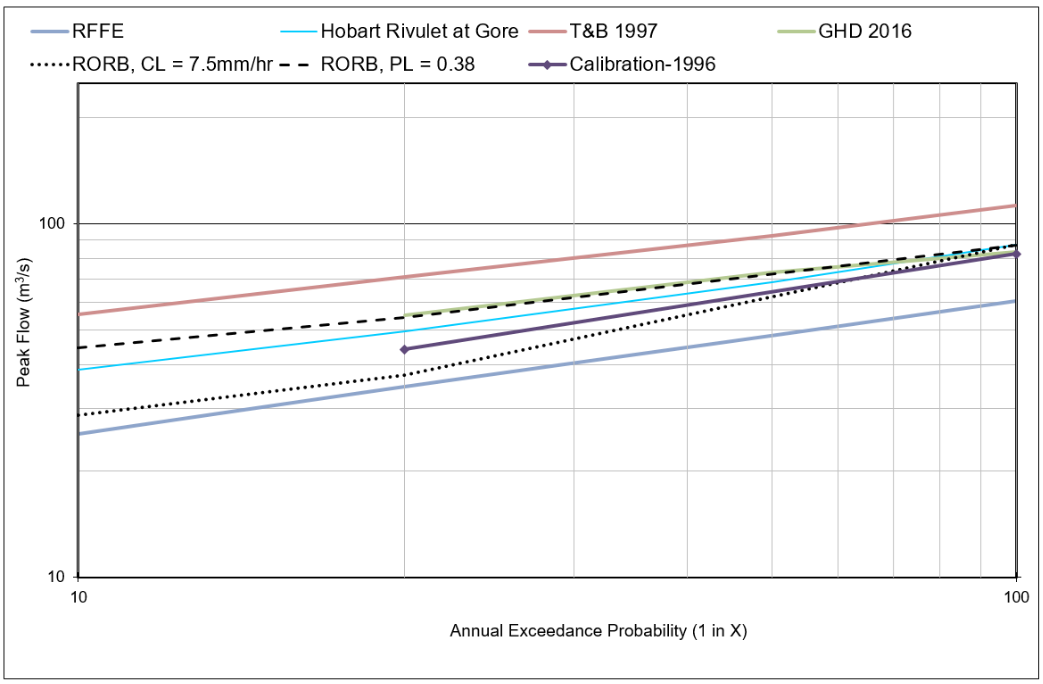


Figure 3-2 – FFC comparison of RORB flows using PL, CL, with other studies/data sets.

It was observed that the sensitivity test using proportional loss of 38% (refer to Section 4.4.2) and the calibration to 1996 event (using continuing loss of 1.5 mm/hr, refer to Section 5) provide the best match to the GHD 2016 and Hobart Rivulet gauge data.

<u>Adopted Losses</u>

After calibration to the 1996 event, these losses have been adopted for this study (Table 3-12).

Table 3-12 - Adopted Losses

Storm AEP	Initial Loss (mm)	Continuing Loss (mm/hr)
Calibration	28.0	1.5
20	28.0	1.5
100	28.0	1.5
PMF	0.0	1.0

3.3.4. Rainfall Temporal Patterns

Ball et al. (2016) recommends that consideration should be given to the impact which a variety of temporal patterns may have on the modelled outcomes from design event rainfall-runoff modelling. It is suggested that this ensemble approach represents an advance on the previous technique of using an averaged representative pattern or the Average Variability (AVM) method.

Ball et al. (2016) further recommends the use of complete storm temporal patterns as representative of actual storm events. The complete storm pattern consists of an initial period of lesser intensity rainfall called the pre-burst providing a pre-wetting of the catchment. The pre-burst is followed by a period of highly intense rainfall called the burst pattern. The duration label applied to any given design storm relates to the duration of the burst period. The complete storm is a much longer duration.

Despite recommending the use of an ensemble of complete storms, the current version of the Ball et al. (2016) datahub (http://data.arr-software.org/), which provides the regional data to apply the improved method, only provides a variety of storm burst patterns.

For this study SMEC has assembled complete storm temporal patterns by combining the variety of temporal patterns for eastern Tasmania (Southern Slopes Tasmania) (Ball et al. 2016) with an extreme AVM pre-burst patterns for short (Jordan et al. 2005) and long durations (BoM 2006). The AVM pre-burst pattern has been scaled to match the Ball et al. (2016) datahub median depths appropriate to each storm event probability and duration.

The result of this combined complete storm temporal pattern ensemble is illustrated in Figure 3-3 for the 1 in 20 AEP, and in Figure 3-4 for the 1 in 100 AEP. The ten (10) largest GSAM source storms (BoM 2006) have been used as an ensemble of PMP temporal patterns in combination with the GSAM AVM pre-burst for use in the PMP event (Figure 3-5).

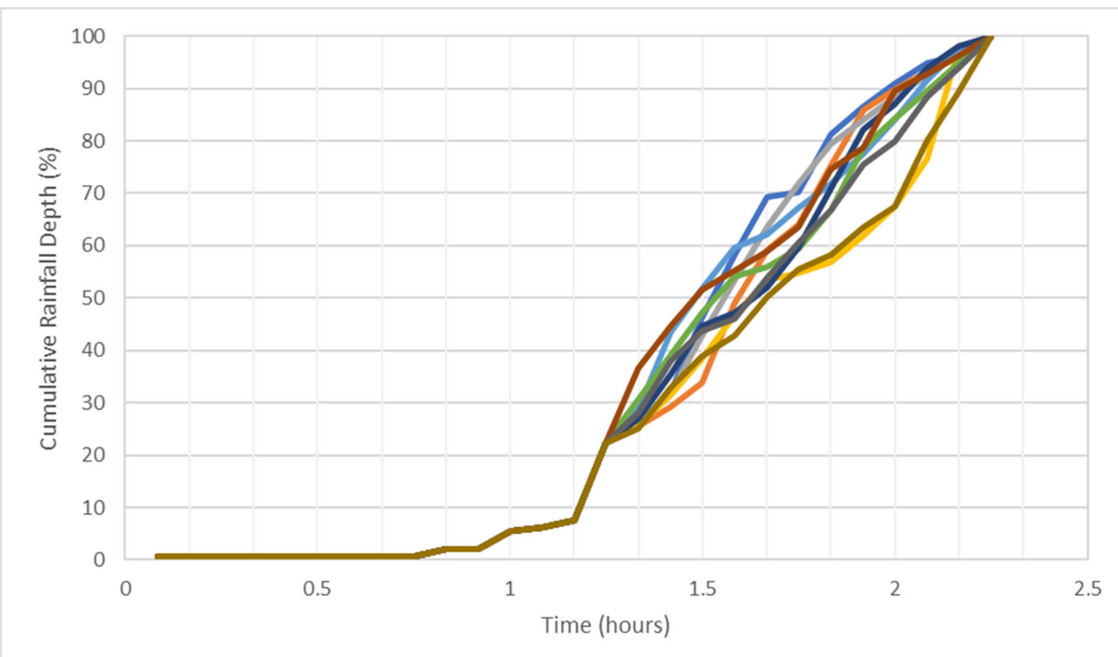


Figure 3-3 – 1 hour duration 1 in 20 AEP ensemble of temporal patterns

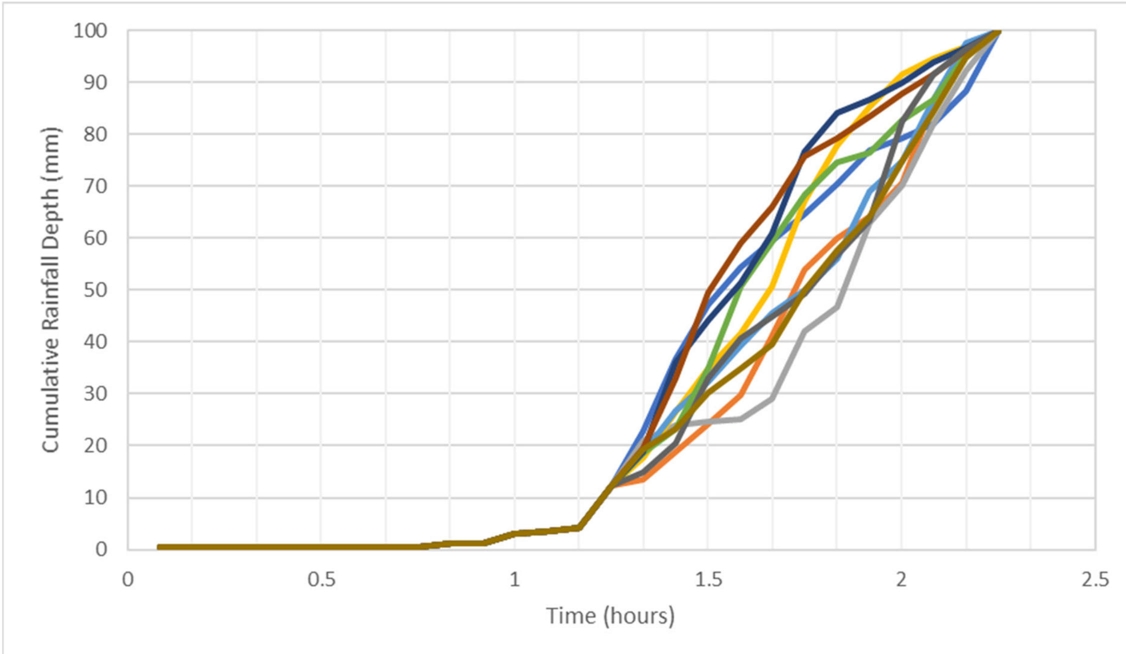


Figure 3-4 – 1 hour duration 1 in 100 AEP ensemble of temporal patterns

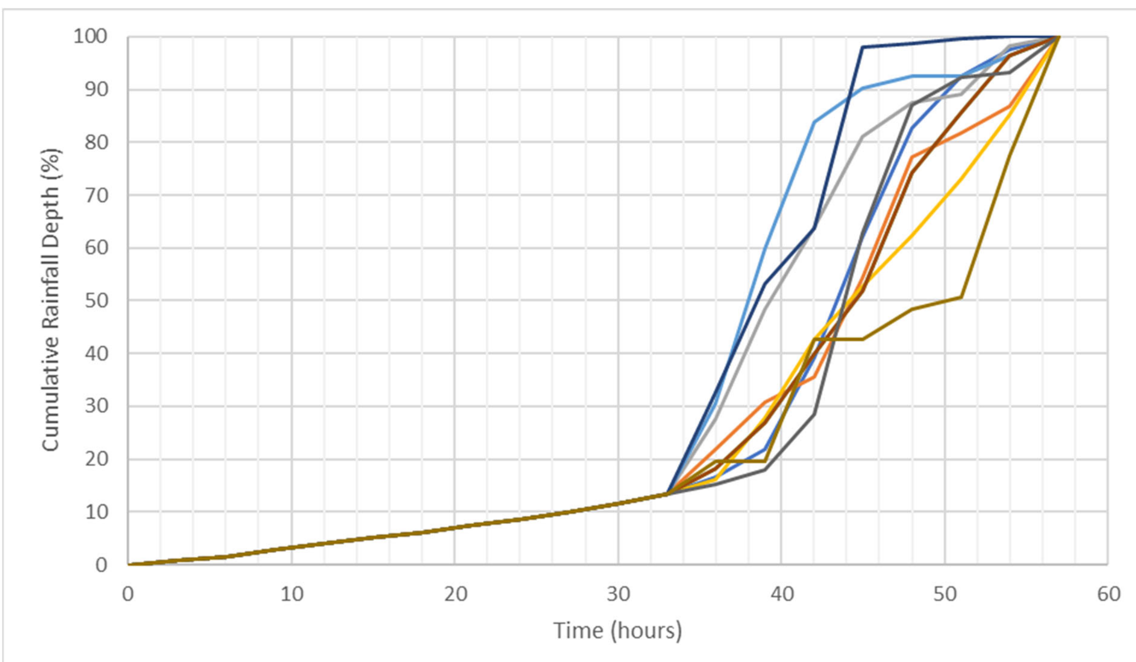


Figure 3-5 – 24 hour duration PMP ensemble of temporal patterns

3.3.5. Rainfall Spatial Patterns

3.3.5.1. General

As discussed in Section 3.1.1, the Study Area has a strong orographic influence on rainfall. The very large range of topographic elevations from Mount Wellington (1,260 mAHD) to sea level affects how much rain falls on different parts of the Study Area.

3.3.5.2. Spatial Pattern from the BoM IFD Design Rainfall Grid

BoM (2006) provides guidance for extreme rainfall using a topographic adjustment factor for long-duration extreme/PMP storms. Ball et al. (2016) recommend that for catchments larger than 20 km^2 spatial patterns be estimated from the design rainfall grids (Book 2, Section 6.3.2) used by BoM to generate the Intensity-Frequency-Duration (IFD) curves.

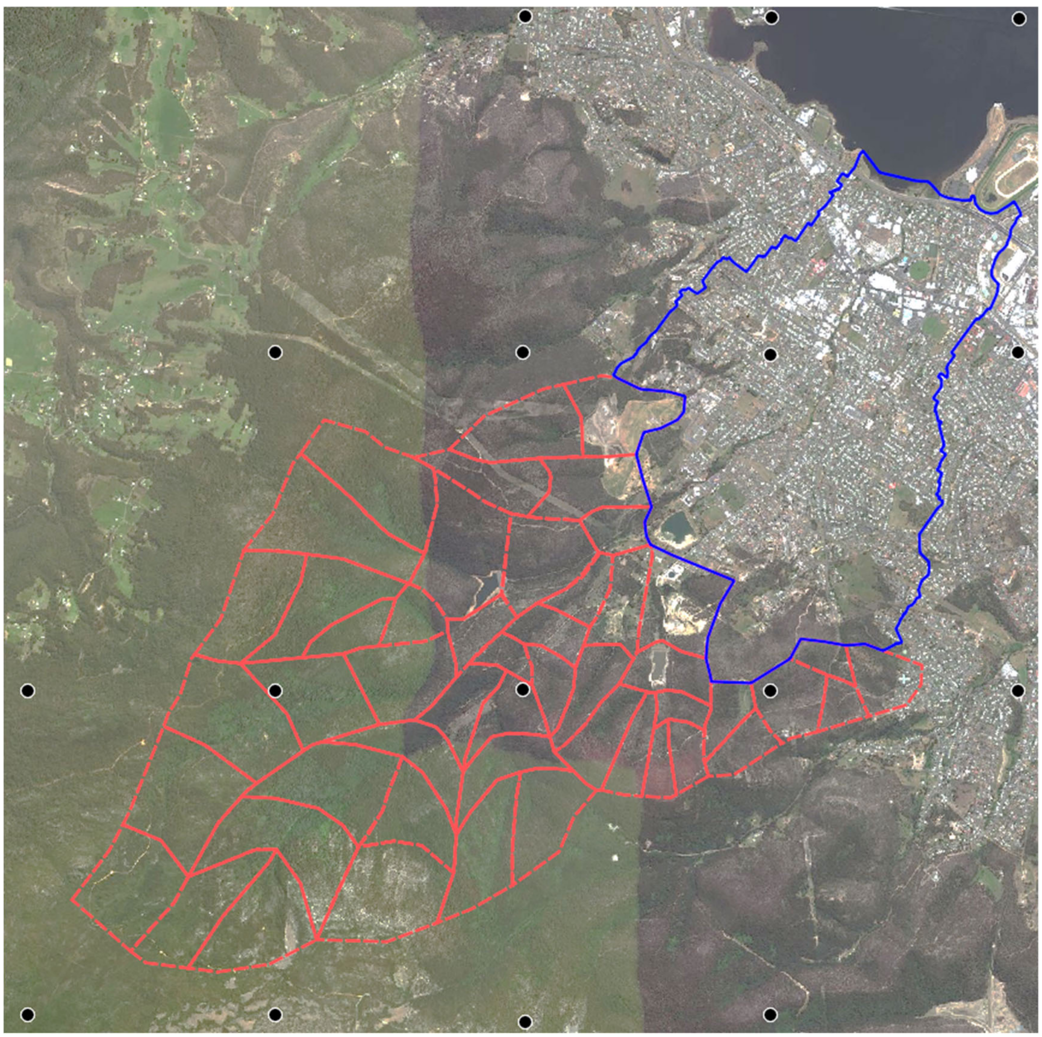


Figure 3-6 – Location of design rainfall grid points.
Black Circles show grid points; RORB subareas in red; Tuflow rainfall-on-grid extent shown in blue.

Analysis of these rainfall grid depths show a 33% difference between higher rainfall at Mt Wellington peak (higher elevation) and lower rainfall over Glenorchy CBD (lower elevation) for the critical duration. The design rainfall grids would produce a coarse spatial pattern relative to the Study Area.

Instead, a custom spatial pattern was adopted based on the largest storm rainfalls recorded by nearby rainfall gauges.

3.3.5.3. Creation of Custom Spatial Pattern

A procedure was developed to create a spatial pattern from the analysis of the rainfall gauges around the Study Area. The BoM rainfall gauges used in this procedure are illustrated in Figure 3-7.

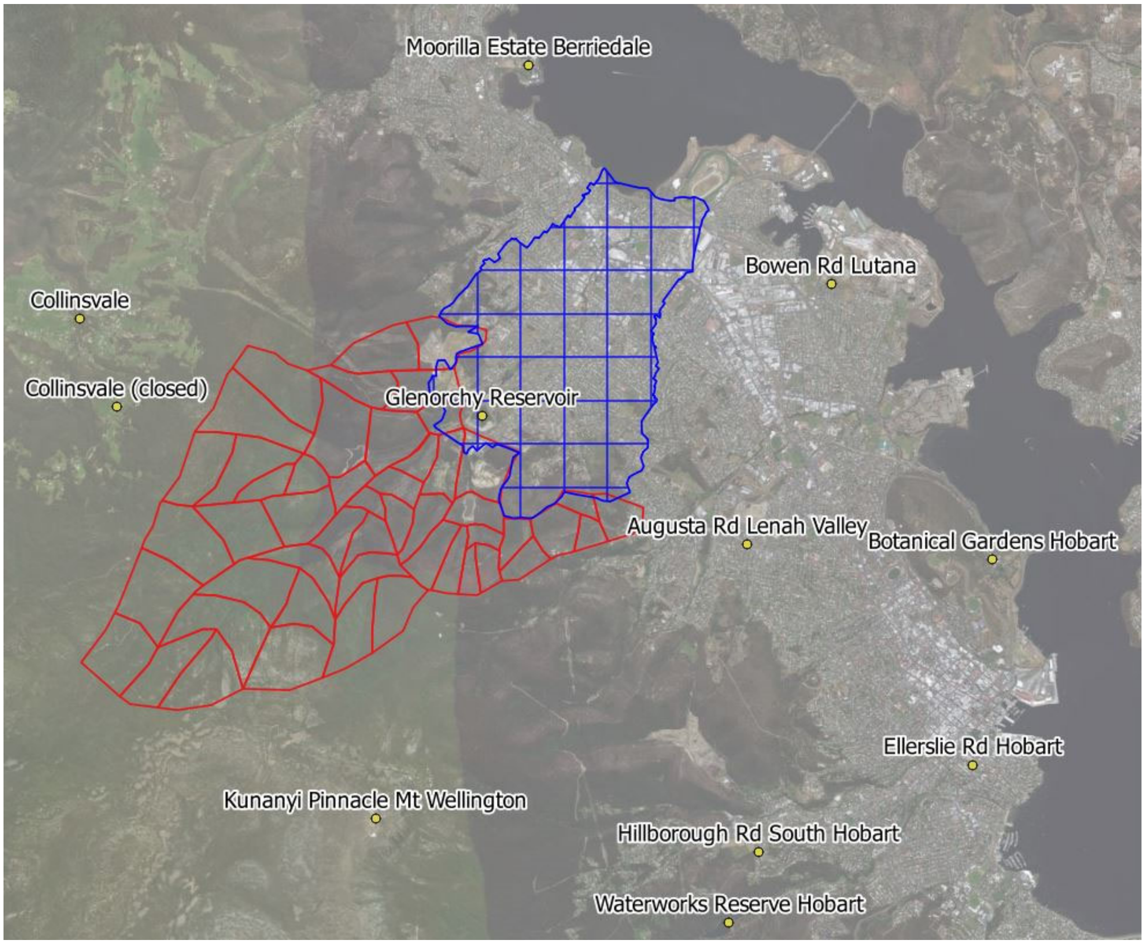


Figure 3-7 – Location of rainfall gauges relative to the Study Area

The procedure applied to create the spatial pattern was:

- Analyse the BoM daily rainfall gauge data to compile a list of the biggest storm events across the region
- Plot each storm event showing the rainfall values for each gauge spatially (see Appendix D)
- Calculate an average percentage relative to the catchment-representative gauge (i.e. Tolosa Reservoir)
  - Isolate the rainfall events that show orographic related spatial variability (refer to the seven events in Appendix D)
  - For each gauge calculate the ratio of rainfall depth to the Tolosa Reservoir gauge rainfall depth as a percentage
  - Calculate the (mean) average of the ratios of the previous step for each gauge. These are illustrated in Figure 3-8 by the yellow points.
- Connect the gauge points with 'contour' lines of equal percentage roughly following the alignment of the actual elevation contours. Refer to green lines of Figure 3-8)
- Apply a percentage value from the 'contours' to each sub-catchment

The mean value is adjusted to achieve an areal-weighted pattern by the formula:

$$\frac{\sum(SubArea\times PatternFactor)}{TotalArea}=100$$

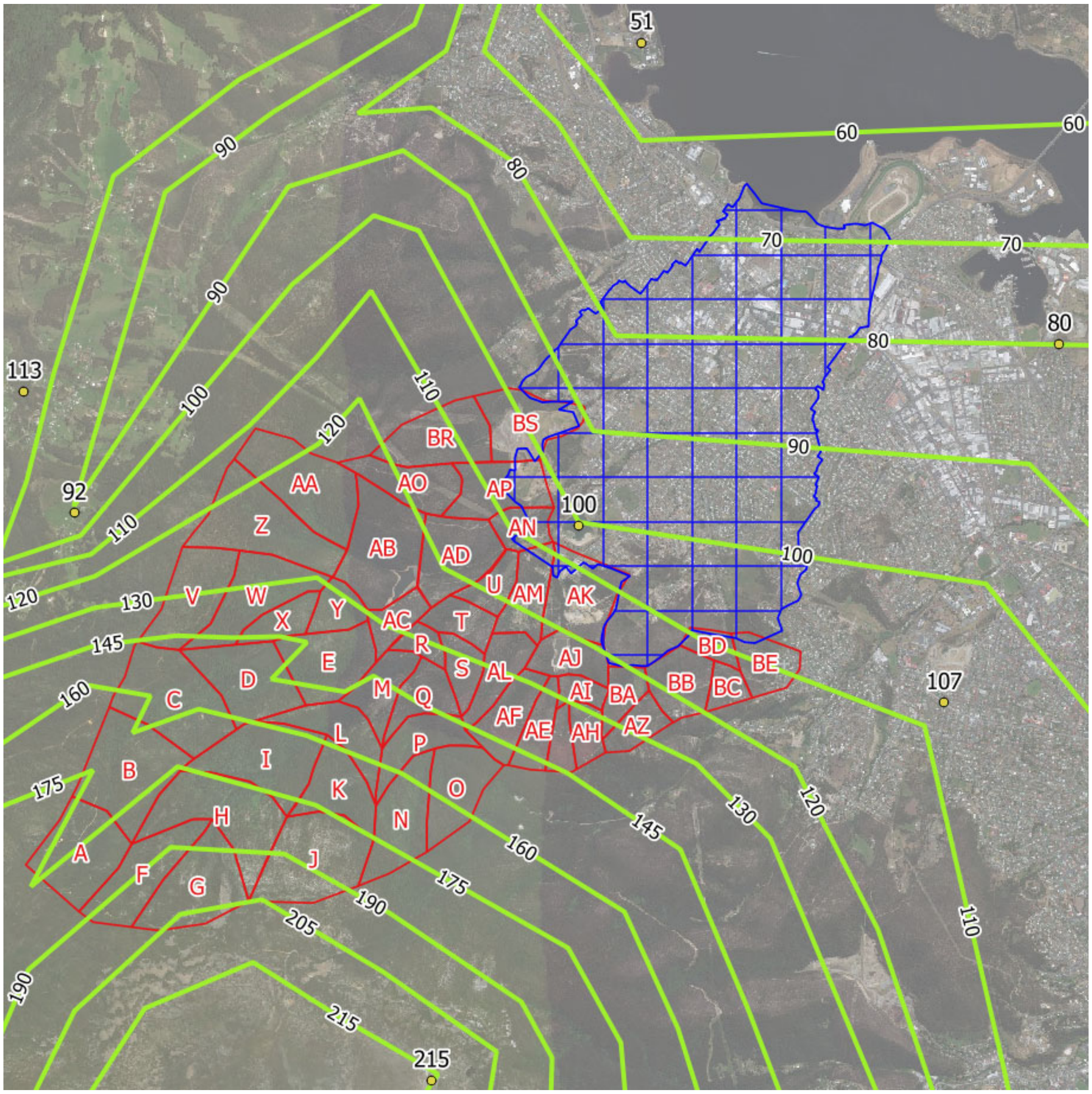


Figure 3-8 – Intermediate/Rare spatial pattern.
Green contours show percentages of average rainfall depth. Red regions are RORB sub-areas. The blue hatching shows the extent of the Tuflow model (slightly larger than where rainfall is applied to improve hydrograph inflows from the forested RORB model. Refer to Section 4).

In compiling the biggest storms, there was no strong indication that the orographic feature of the catchment strongly influenced most major storms. Half of the dozen events examined showed a strong orographic influence, and the other half showed no influence or a slight reverse trend. It is thought that the approach direction of the storms influences the spatial distribution of rainfall for the Study Area via a varied influence from the varied ground elevations.

Section 4.4.3 describes the calibration of the model with and without this spatial pattern. The outcome was very sensitive to the pattern. The calibration demonstrated that applying more rain over rural catchment, using a custom spatial pattern, lead to greater runoffs. These greater runoffs better represented the flow conditions and levels observed during the 1996 Calibration flood event.

3.3.5.4. Adopted Spatial Patterns

The Study Area was modelled in RORB and Tuflow using a variety of spatial patterns depending upon the application of the model. When running the model for:

- The 1 in 20 AEP and 1 in 100 AEP events, the custom spatial pattern (Section 3.3.5.3) was applied.
- For short duration PMF events, the GSDM spatial pattern was applied over the Study Area. In these cases, the storm was centred over the Glenorchy CBD area. (Refer to Figure 3-9).
- For long duration PMP events, the GSAM spatial pattern was applied over the Study Area. This pattern applies an orographic influence on rainfall. See the pattern illustrated in Figure 3-10.

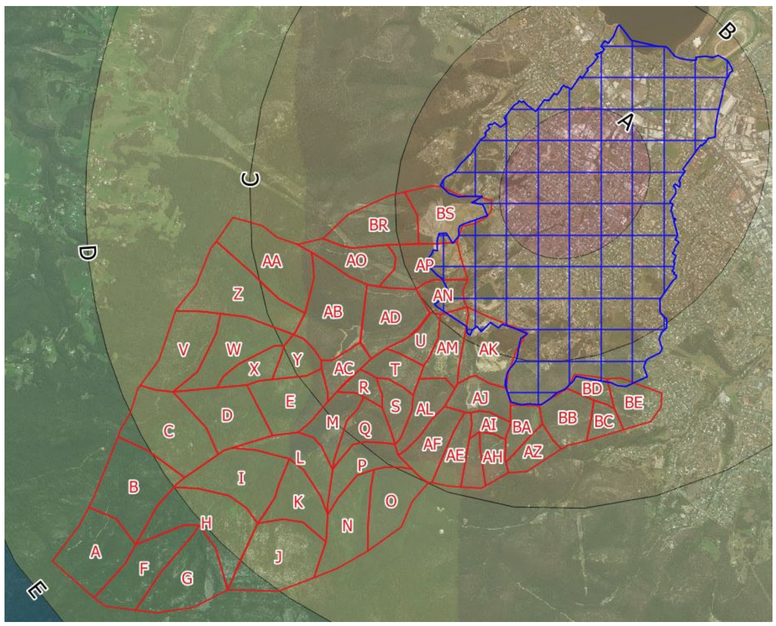


Figure 3-9 – GSDM spatial pattern centred on Glenorchy CBD.
Red labels are sub-areas. Black labels are Ellipse IDs. The blue hatching shows the extent of the Tuflow model (slightly larger than where rainfall is applied to improve hydrograph inflows from the forested RORB model. Refer to Section 4).

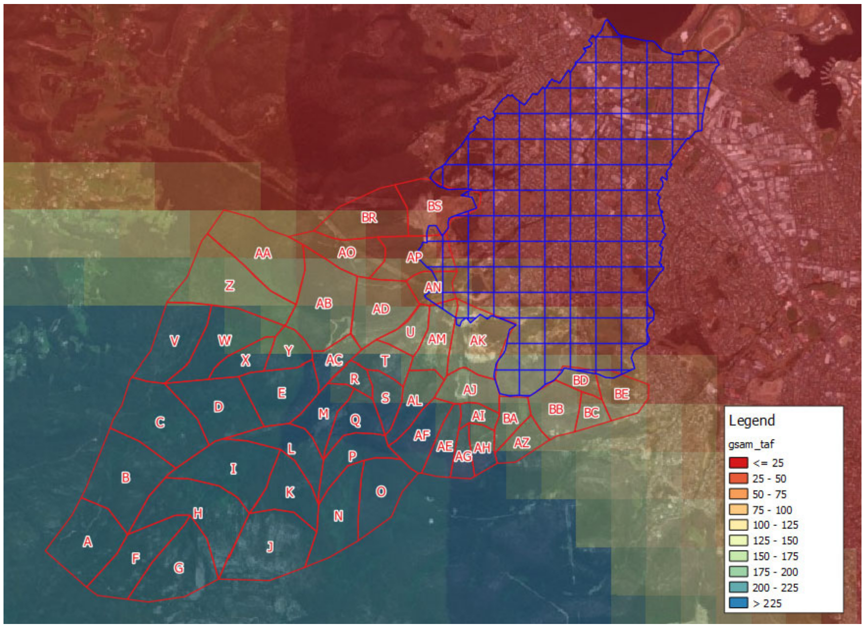


Figure 3-10 – GSAM spatial pattern based on the ratio of sub-area TAF to catchment TAF

4. MODEL SETUP

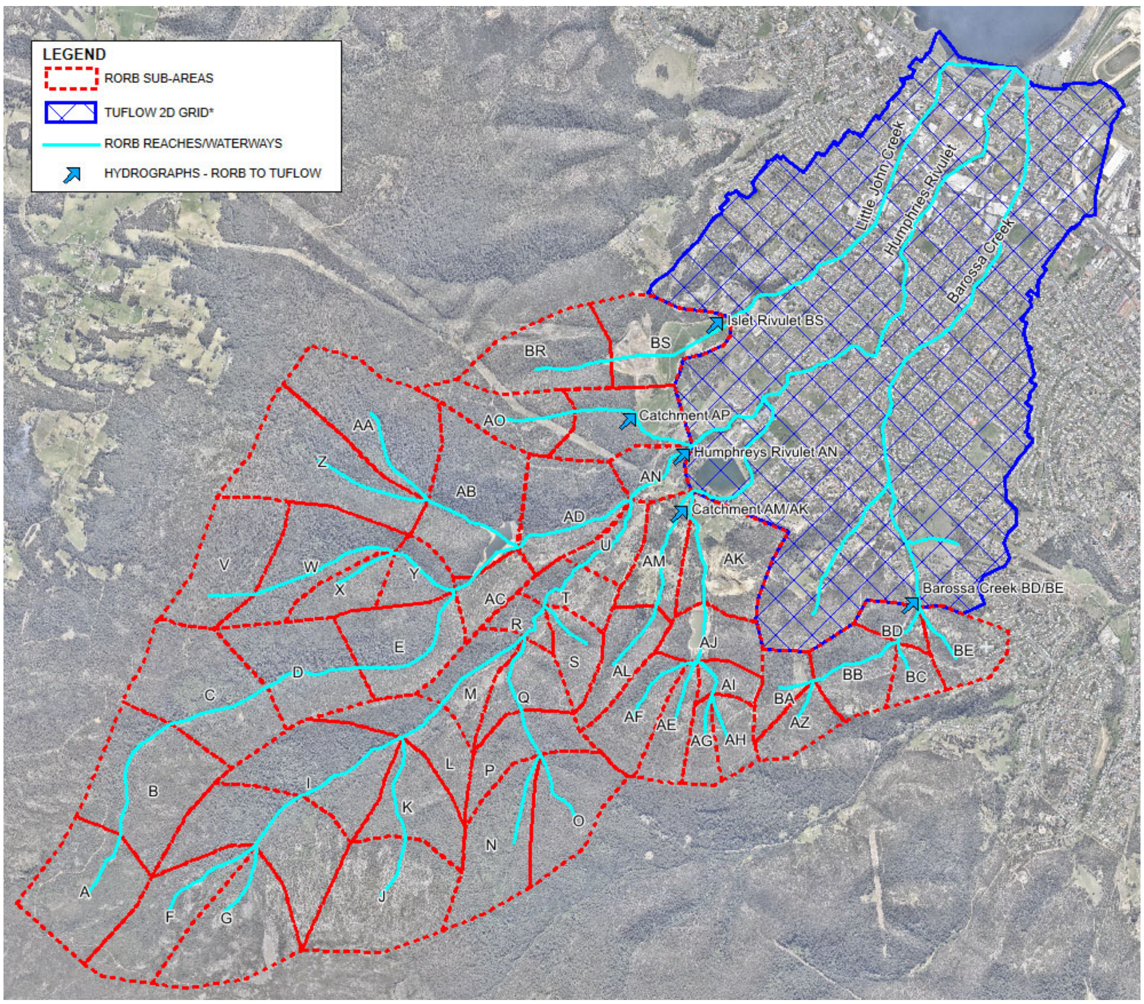
4.1. Hybrid Model

4.1.1. General

A rainfall-runoff model has been set up to describe the Study Area, and the layout diagram is shown as Figure 4-1.

The Study Area has been divided into two types, namely 'rural' and 'urban'.

- The rural catchment has been modelled utilising RORB – a 1D, non-linear, runoff routing model.
- The urban catchment was modelled using rainfall-on-grid with Tuflow HPC (Heavily Parallelised Compute), a dynamic hydraulic model which combines 1D calculation for pit and pipe flow with 2D overland flow calculations.



*square hatching is cosmetic only and does not represent 2D grid size or orientation

Figure 4-1 – Hybrid model layout - RORB sub area break up and Tuflow domain.

4.1.2. Model Scenarios

The hybrid model was run for a set of scenarios required by GCC to assess the breakout flood risk from the three watercourses and the drainage system performance (see Table 4-1). As noted in Ball et al. (2016), rainfall-on-grid is "a relatively recent development in 2D hydraulic modelling" and "where possible models should be calibrated to measurements". Section 5 describes the calibration to water levels measured after the February 1996 flood event (described in T&B 1997).

Table 4-1 – Model Scenarios

Scenario	Events Modelled	Catchment Condition	Rainfall	Dams Water Level
Calibration Scenario	1996 Event (Refer Section 0)	existing	recorded rainfall	Full Supply Level
Scenario 1	1 in 20 AEP, 1 in 100 AEP and PMF	existing	design rainfall	reduced operating level (drawn down)
Scenario 2	1 in 20 AEP, 1 in 100 AEP and PMF	developed	design rainfall	Full Supply Level Tolosa decommissioned
Scenario 3	1 in 20 AEP, 1 in 100 AEP and PMF	developed	design rainfall	reduced operating level (drawn down) Tolosa decommissioned
Scenario 4	1 in 20 AEP and 1 in 100 AEP (refer Section 4.1.5)	developed	design rainfall increased by Climate Change factor	existing dam draw down water levels Tolosa decommissioned

For each event duration, the ensemble method of 10 complete storms produced 10 flow outcomes. The closest flow to the mean of these 10 was selected as the event duration flow estimate.

4.1.2.1. Dams Initial Water Level

There are three dams in the Study Area: Limekiln Gully Dam, Knights Creek Dam and Tolosa Reservoir. The starting water level in each of the three dams varied by scenario (refer Table 4-1).

These starting conditions were based on TasWater's planned operating regimes for the dams (refer to Table 4-2 for specific water levels). SMEC's understanding of these expected operating regimes was based on recent studies completed by SMEC for TasWater (SMEC 2017).

Table 4-2 – Model Scenarios

Dam	Full Supply Level (mAHD)	Reduced Operating Level (mAHD)
Knights Creek Dam	189.43	184.13
Limekiln Gully Dam	166.42	161.92
Tolosa Reservoir	107.02	102.02

4.1.3. Existing Scenario

For Scenario 1 the rural and urban environments were modelled as they are at present. The fraction impervious value was selected based upon the degree of development. The impervious value was set in accordance with the values presented in Figure 4-2 and Table 4-3.

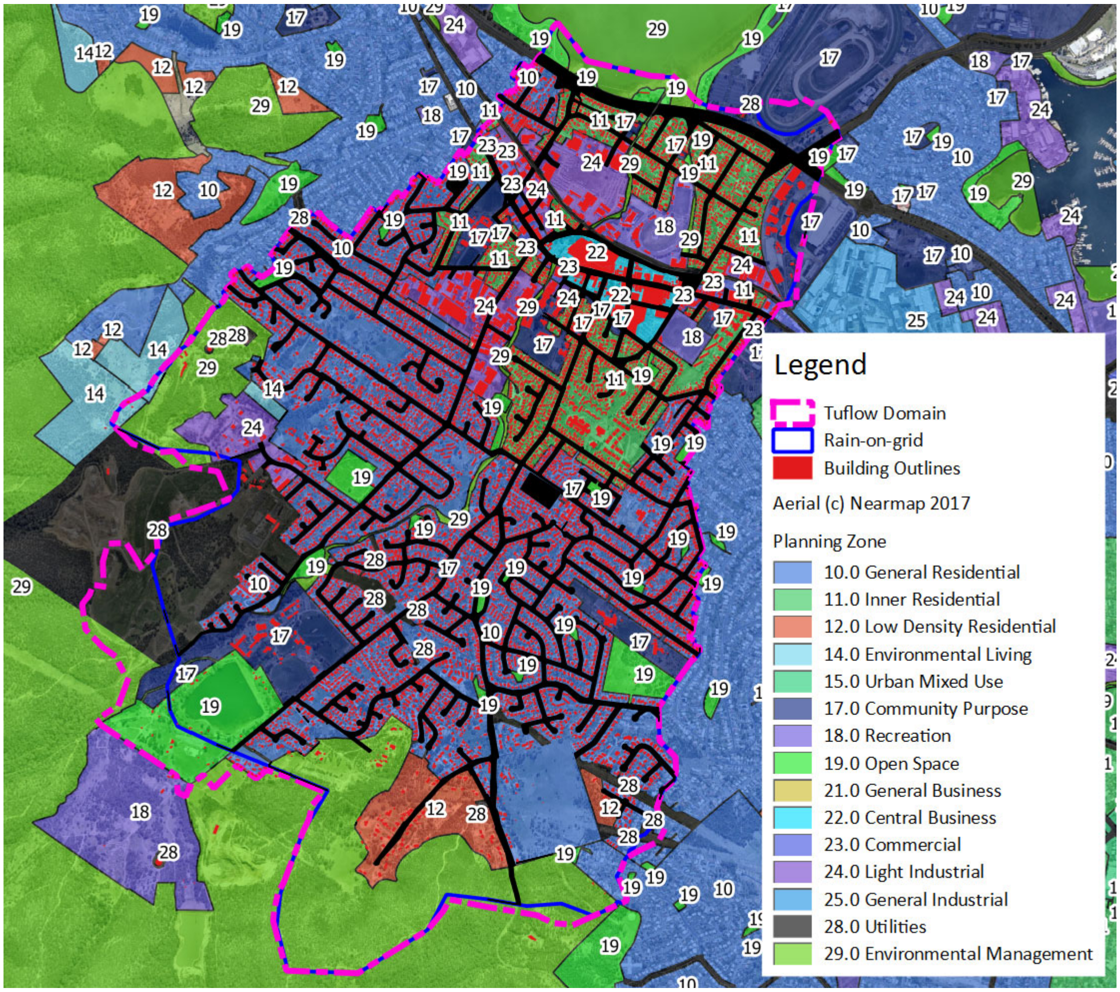


Figure 4-2 – Study Area Planning Scheme Zoning.

The land type/planning zone for each location was determined through interrogation of information on the interactive planning scheme maps on following website (accessed January 2018): http://iplan.tas.gov.au/pages/plan/book.aspx?exhibit=maps&hid=225916

All subareas of the RORB hydrological model fall within the environmental management zone. A fraction impervious value of 0.05 was used for all RORB subareas in the existing scenario.

Table 4-3 – Adopted Zoning Imperviousness

Land Type/Planning Zone	Typical Fraction Impervious Range	Adopted Fraction Impervious
General residential	0.5 - 0.8	0.6
Inner Residential	0.5 – 0.9	0.8
Low density / Environmental Living	0.1 – 0.3	0.2
Community Purpose	0.0 – 0.2	0.1
Recreation	0.0 – 0.2	0.1
Open Space	0.0 – 0.2	0.1
Local Business / Central Business / Commercial / Light Industrial	0.7 – 0.95	0.9
Utilities*	0.0 – 0.2	0.1
Environmental Management	0.0 – 0.05	0.05
Roads	0.5 – 0.8	0.8
Waterbodies/Rivulets	1.0	1.0

*Main roads (e.g. Brooker Hwy) are listed: 'Utilities'. Fraction impervious of roads supersede other land uses

4.1.4. Developed Scenario

For scenarios 2 to 4, the urban environment was modelled as 'developed'. As the urban landscape undertakes infill development, the impervious portion of the Study Area increases leading to greater runoff rates and volumes. It was understood that changes to land use zones are not expected for the developed conditions. Changes to the imperviousness were limited to the potential for infill development.

The detail of the future changes to the urban landscape was unknown. To model this scenario, existing model roughness values were maintained, whilst losses were reduced relative to the potential maximum increase in the fraction impervious.

Table 4-4 compares the existing and developed fraction impervious values (based on Table 4-3) and lists the developed losses calculated from the developed fraction impervious.

Table 4-4 – Developed Losses by Land Use

Land Type/Planning Zone	Tuflow Material ID	Existing Fraction Impervious	Developed Fraction Impervious	Initial Loss (mm)	Continuing Loss (mm/hr)
10 General residential	1	0.6	0.8	5.6	0.30
11 Inner Residential	7	0.8	0.8	1.4	0.30
12 Low Density Residential	11	0.2	0.3	19.6	1.05
17 Community Purpose	14	0.1	0.2	22.4	1.20
18 Recreation	3	0.1	0.2	22.4	1.20
19 Open Space	4	0.1	0.2	22.4	1.20
21 General Business	8	0.9	0.9	2.8	0.15
28 Utilities	4	0.1	0.2	22.4	1.20
29 Env. Management	10	0.05	0.05	26.6	1.425
Roads	2	0.8	0.8	5.6	0.30
Waterbodies/Rivulets	13	1.0	1.0	0.0	0.00

All subareas of the RORB hydrological fall within the environmental management zone. A fraction impervious value of 0.05 has been used for all RORB subareas in the developed scenarios.

4.1.5. Climate Change Scenarios

The climate change scenario for this study was based on:

- Southern Slopes Tasmania Natural Resource Management Cluster
- Interest in 1 in 100 AEP places planning horizon out to the year 2090; and
- Practitioner assumption: high emissions (RCP8.5) scenario (IPCC 2013).

Ball et al. (2016) provides guidance for climate change impact on rainfall intensities at a regional level (allocating Tasmania to a region with Southern Victoria and NSW).

It is worth noting that the flood mitigation infrastructure resulting from this study will have design lives out to 100 years, and therefore adequate justification for the long-term planning horizon needs to be considered and adopted.

The Climate Futures for Tasmania (CFT) study used a downscaling approach to create climate projections from the IPCC Special Report on Emissions Scenarios (SRES) (Nakićenović N & Swart R (IPCC) 2000) at a finer grid scale over Tasmania (ACE CRC 2010). ACE CRC (2010) reports the temperatures slightly lower than the Ball et al. (2016) values. ACE CRC (2010) reports that in the high emissions scenario the 2090 temperature rise for Tasmania is 2.6 to 3.3 °C, and rainfall depth increases 12-30% seasonally and 24% average increase annually.

Ball et al. (2016) uses the (more recent) IPCC (2013) Representative Concentration Pathways (RCPs) compared to ACE CRC (2010) use of SRES, and its climate change chapter is based on coarser scale regional climate modelling by CSIRO and BoM (2015).

Ball et al. (2016) allows practitioner judgement of choice between Representative Concentration Pathways (RCPs) (IPCC 2013) of RCP4.5 and RCP8.5. RCP8.5 has been selected based on the most current CO_2 trajectories, and USA withdrawal from Paris COP21 2015 Agreement.

Following the Ball et al. (2016) procedure on the basis of these inputs, the CSIRO and BoM (2015) estimates that on average the Tasmanian region will be more than 3°C hotter and a median temperature of 3.6°C hotter in 2090. From this temperature, the Intensity factor (F_{CC}) calculation gives a multiplicative factor of 1.19, or a 19.2% increase in rainfall intensity (Ball et al. 2016).

The results (and emissions pathways selected) between the two studies are reasonably comparable. Table 4-5 summarises the climate change scenario parameters adopted for this study.

Table 4-5 – Climate Change Scenarios

Climate Scenario	AEP	Rainfall Intensity (mm/hr)	Sea Level (mAHD)	Storm Surge (m)	Water Level Adopted (mAHD)
CC1	1 in 20	1 in 20 Intensity x F_{CC}*	2010 HAT + SLR 0.8 + 0.82 = 1.62	0.0	1.62
CC2	1 in 100	1 in 100 Intensity x F_{CC}*	2010 HAT = 0.8	0.0	0.80
CC3	1 in 100	1 in 100 Intensity x F_{CC}*	2010 HAT + SLR = 1.62	0.0	1.62
CC4	1 in 100	1 in 100 Intensity x F_{CC}*	2010 HAT + SLR = 1.62	0.4	2.02

*F_{CC} adopted is 1.24.

This study proposed an approach of adopting the local climate change model study, the Climate Futures report (ACE CRC 2010). Therefore, a rain depth increase of 24% (F_{CC} = 1.24) was applied.

It is noted that this is comparable but slightly more than the estimate of 20% increase used in the Roseneath Rivulet flood study (GCC 2017).

The base tide level adopted was the 2010 Highest Astronomical Tide (HAT) of 0.8mAHD (GCC 2017). The adopted sea level rise (SLR) is 1.6mAHD, and 2.0mAHD for SLR plus storm surge (GCC 2017).

These values match with the range of 2090 values for the Hobart 20year SLR and storm surge presented in McInnes et al. (2012).

4.2. RORB Model

4.2.1. General

A RORB rainfall-runoff routing model has been used to simulate the hydrologic performance of the rural catchment of the Study Area. The model has been used to provide inflow hydrographs from the forested upper catchment into the Tuflow hydraulic model to assess flood hazard within the Glenorchy CBD and surrounding area.

The RORB (Laurenson et. al. 2010) model simulates the catchment routing characteristics utilising a representation of the stream network and the parameters k_c and m. The effect of the catchment in delaying the runoff response from rainfall is represented by k_c and the non-linearity in the storage discharge relationship for the catchment is represented by m. The RORB model also incorporates a loss model to account for rainfall lost to groundwater stores, evaporation and various other sinks.

A RORB model was previously developed for the Study Area as part of the SMEC (2017) study of Knights, Limekiln Gully, and Tolosa Dams. The sub-catchment layout of the developed region of Glenorchy in that model was too coarse for this assessment. Those sub-catchments have been ignored in the model. Instead, the developed region was modelled using Tuflow rainfall-on-grid (Section 4.3). The hybrid model configuration is shown in Figure 4-1.

4.2.2. Sub-catchment layout

The Study Area model (including Humphreys Rivulet, Barossa Creek and Little John Creek) consists of an area of 27 km^2, which was previously delineated into 78 sub-catchment areas ranging in size from 0.1 km^2 to 1.3 km^2. The stream network was established based upon the overland flow paths as indicated by surface contour information.

The layout of sub areas for the model have a number of competing influences, including:

- A preference for between 3 and 5 sub-areas upstream of any point where flow measurements are required.
- A preference to keep sub areas across the catchment to as similar a size as possible.
- A preference to reduce the impact of large point source inflows to the downstream inundation area when modelling inundation consequences.

The 27 sub-catchments across the developed lower catchment region in the previous RORB model have been removed from the updated RORB model in this study. Instead, the Tuflow model was developed to cover these sub-catchments, as shown in Figure 4-1.

4.2.3. Dams

4.2.3.1. General

As part of the RORB modelling, it was necessary to include the relevant dam characteristics for Knights Creek, Limekiln Gully and Tolosa Dams. The elements of relevance to the hydrologic modelling are as follows:

- Elevation-Discharge Relationship
- Elevation-Storage Relationship

These details were included in the original model (SMEC 2017) and were not modified for this study, except to set appropriate initial water levels (IWL). See Section 4.1.2.1 for details of IWL. The methodology applied for modelling the three dams is included in Appendix C.

4.2.4. Selection of RORB Model Parameters

4.2.4.1. General

In the absence of rainfall and flow data across the Study Area for calibration, parameters have been determined by considering past studies and data from adjacent catchments. In determining the appropriate parameter set for the RORB model, an early iteration of the model was used, where all dams were removed from the model and coarse sub-catchments out to Elwick Bay were included.

The model parameters were validated using the design storm rainfall to achieve an appropriate match to 1 in 100 AEP flow estimates as determined using a range of regional estimation techniques described above (refer to Section 3.3). After development of the Tuflow model, the RORB model parameters were updated via calibration to the 1996 event water levels (refer to Section 5).

4.2.4.2. K_C Value

Laurenson et al. (2010) recommended the approach for selecting RORB model runoff routing parameters k_c and m is to calibrate the catchment file utilising rainfall and runoff data from selected historical storm events. Where there is insufficient data within or nearby the Study Area, then the approach can be to apply regional equations, to review available data from similar catchments, and also to review outcomes from other studies.

Regional Equations

A range of regional equations can determine the catchment delay which typically take the form of:

$k_c = b \times A^d$

Where: A = area in (km²)
b = Coefficient
d = Coefficient

It is common practice to apply relationships derived in Victoria for Tasmanian conditions due to the broad hydrologic similarity in the two states and also because there are few similar studies of catchment delay for Tasmanian conditions. A number of different k_C estimates were considered during this study, which are outlined Table 4-6.

Table 4-6 - Estimate k_C parameter equations

Estimate	Equation	Kc
Dandenong Creek and Westernport Catchments	$k_c = 1.53 * A^{0.55}$	9.4
Yarra and Maribyrnong Catchments	$k_c = 1.19 * A^{0.56}$	7.5
Victoria (MAR>800mm)	$k_c = 2.57 * A^{0.45}$	11.3
Victoria (MAR<800mm)	$k_c = 0.49 * A^{0.65}$	4.2
Recommended for Tasmania (Ball et.al. 2016)	$k_c = 0.86 * A^{0.57}$ – (m = 0.75)	5.6
	$Q_{100} = 80\ m^3/s$, (m = 0.8)	4.7

The k_c value recommended for Tasmania has been developed with an m value of 0.75. An adjustment can be applied to determine an equivalent value with an m value of 0.8, but it varies with the model peak discharge (Laurenson et.al. 2010). The k_c value can be adjusted by a factor:

$$\left(\frac{Q_p}{2}\right)^{m-m'}$$

Where Q_p = peak discharge (m^3/s)
m = old value of the m parameter
m' = new value of the m parameter

An equivalent k_c value has been presented above for a 1 in 100 AEP flow estimate of 80 m^3/s using the determined factor of 0.83.

The derivation of the Victorian equations (referred to in Table 4-6) are described in Hansen (1986). The equations for Dandenong Creek and the Yarra River (major Melbourne River catchments) have been derived internally by Melbourne Water and are unpublished.

The results from the analysis indicated the range of k_c values that would be expected for the catchment were between 4.2 and 11.3.

Adjacent Catchments

The Hobart Rivulet catchment is located within close proximity to the Study Area and it has a reletively reliable and long rainfall and runoff record. The data from Hobart Rivulet has been used to assist in understanding the potential hydrologic characteristics of the Study Area.

Hobart Rivulet has a catchment area of 16.3 km^2 at the comparison streamflow gauge and its catchment centroid is located a distance of around 6 km from the Study Area.

The delay parameter for the catchment as applied in RORB is a quantification of storage delay throughout the catchment. This delay was discussed in Laurenson et al. (2010) and it was noted that the storage delay and the peak runoff delay can be assumed to be equivalent for the catchment.

The runoff delay can be expressed in the form:

$T_c = k_c \times Q^P$

Where T_c = lag (hrs)
Q = mean outlet discharge (m^3/s)
P & k_c = constant (p = -0.25)

It may be noted that the lag is defined as the delay between the centroid of the rainfall excess and the centroid of the resulting surface runoff. The peak runoff delay can be estimated from observed hydrograph and rainfall data and can, in turn, be used to estimate the mean and variability in catchment delay. This in turn can be used to infer the appropriate storage delay parameter to be applied to the RORB model.

The available flow and rainfall data from the Hobart Rivulet catchment were collated and the largest hydrographs and rainfall hyetographs were extracted from the data set. The delay was estimated for a range of events. An example extracted hydrograph is presented in Figure 4-3 and the plot of all events is included as Figure 4-4.

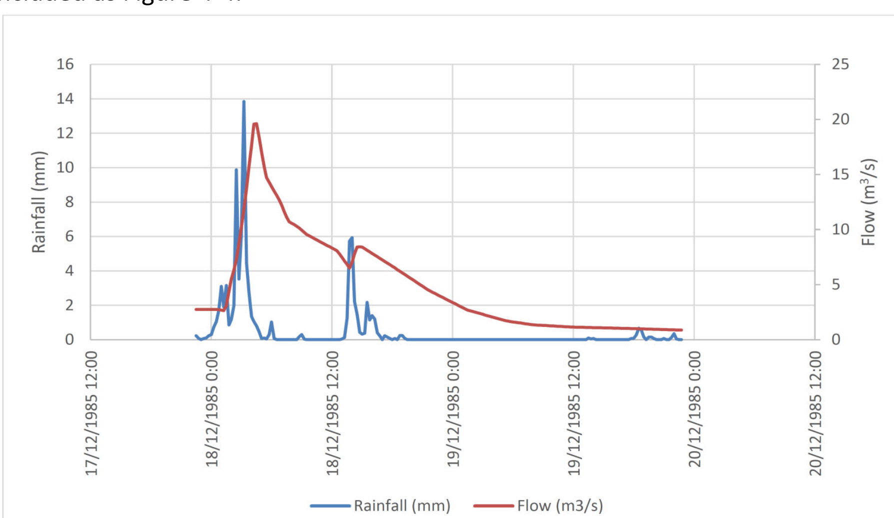


Figure 4-3 – Observed Rainfall-Runoff Event

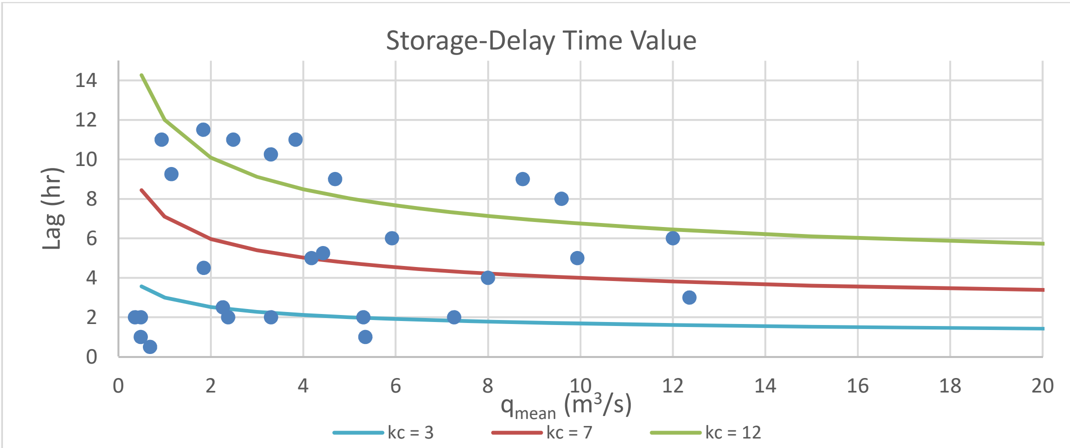


Figure 4-4 – Hobart Rivulet Observed Storm Delay

In determining the mean flow certain portions of the events were censored. Any flow at the start of the event (baseflow) was ignored. Additional inflow due to secondary rainfall peaks during the event were subtracted from the subsequent hydrograph tail. The impact of rainfall loss on the data points was ignored, since it is judged that it would not have a significant impact on the delay estimate in most cases. The data from some events was too noisy to allow the delay for a single event to be determined and in such cases the data was not included in the analysis. Events that were 'too noisy' were multi-peak events where it was unclear which rainfall peak corresponded to which runoff peak.

It is recognised that the process of deriving the delay incorporates some subjective judgement on the part of the analyst, however, the variability in the outcomes is consistent with a similar analysis described in Kjeldsen et.al. (2016) with a more rigorous methodology.

Figure 4-4 includes a range of lines representing the range within which the average delay could be expected to fall. The Hobart Rivulet catchment area associated with the above delay times is 16.3 km^2. The Study Area has a catchment area of 27 km^2 which is larger and the delay may be expected to be proportionately larger. The various equations included in Table 4-6 suggest that the delay time for the Study Area catchment should be around 30% larger than the smaller Hobart Rivulet catchment. This equates to a catchment delay kc value in the range of 4 - 16.

Other Studies

Engineers Australia (2015) in part considered delay parameters for the adjacent Hobart Rivulet. The analysis includes a calibration for a number of large rainfall events for Hobart Rivulet. The rainfall-runoff model was different to RORB, although similar in concept to RORB. That rainfall-runoff model uses a calibration parameter 'alpha' which is the RORB equivalent of the ratio of two variables: i.e. $\alpha \approx k_c/D_{av}$. The alpha parameter adopted for use in Engineers Australia (2015) for Hobart Rivulet is 1.3. The Study Area RORB model has a D_{av} value of 5.54. An equivalent alpha value of 1.3 for the Study Area would result in a k_c value of 7.2 (i.e 1.3x5.54=7.2).

Adopted Parameter

The range of delay parameters which may be considered applicable to Humphreys Rivulet along with the parameter adopted for this study are detailed in Table 4-7.

Table 4-7 – Streamflow Gauging Station Characteristics

Source	Delay Parameter (k_c)
Regional Equations	4.2-11.3 (Range) 7.8 (average)
Hobart Rivulet Observed Flow Data	4-16 (Range) 10 (average)
Hobart Rivulet Study	7.2
Adopted from Calibration	**4.0**

It is of interest to note that both the range of delay parameters from regional equations and the observed data are similar. The observed data indicates that the delay can vary substantially between events and that no single delay parameter with the rainfall-runoff model is likely to represent the range of conditions that may occur over the catchment.

Current accepted practice is, nevertheless, to apply a single parameter for analyses. The adopted delay parameter was determined through calibration of the 1996 event. This indicates that during that event the catchment was very responsive, indicative of a saturated catchment.

4.2.4.3. m Value

Although, the m parameter can vary in the range of 0.6 to 1.0, it is recommended in Ball et al. (2016) that a value of 0.8 be used for ungauged catchments in the absence of evidence supporting an alternative value. Therefore, the value of 0.8 has been adopted for the purposes of developing design storm hydrographs.

4.2.4.4. Losses

This analysis has considered both continuing and proportional loss models, however, has adopted the continuing loss model for all scenarios. See Section 3.3.3 for a detailed assessment of losses.

After calibration to the 1996 event, these losses have been adopted for this study (Table 4-8).

Table 4-8 - Adopted Losses

Storm AEP	Initial Loss (mm)	Continuing Loss (mm/hr)
Calibration	28.0	1.5
20	28.0	1.5
100	28.0	1.5
PMF	0.0	1.0

4.2.4.5. Adopted Model Parameters

The adopted parameters for the RORB model are outlined in Table 4-9.

Table 4-9 - Adopted RORB parameters

Parameter	Value
m	0.8
K_C	7.0
Initial Loss (mm)	28 (0.0 in PMF)
Continuing Loss (mm/hr)	1.5 (1.0 in PMF)

Note that the runoff coefficient is computed as being 1 minus the proportional loss.

4.3. Tuflow Model

4.3.1. General

The urban catchment of the Study Area has been modelled using Tuflow HPC hydraulic model. The Tuflow model utilises input hydrographs developed from the rural model at the five locations shown in Figure 4-5.

The Tuflow model represents the urban catchment using 2D surface terrain, surface roughness, and a 1D pit and pipe network (no less than 450mm diameter or equvilent). Tuflow version 2018-03-AB single precision has been used with HPC GPU settings.

HPC, Heavily Parallelised Compute, allows very large models with a fine grid size to be run in shorter timeframes. The model domain covers an area of 8.8 km^2, which includes a small overlap with the rural catchment. The rainfall is applied 'rainfall-on-grid' to only the urban catchment of 8.3 km^2, with no 'rainfall-on-grid' layer applied to the small overlap (illustrated by the magenta and blue lines in Figure 4-5).

To balance runtime and model definition a grid size of 2x2 m was used, specifically to enhance the detail of some narrow rivulet channels modelled using the 2D grid surface. A grid size this fine for an area this large has recently become possible through the HPC version of the model.

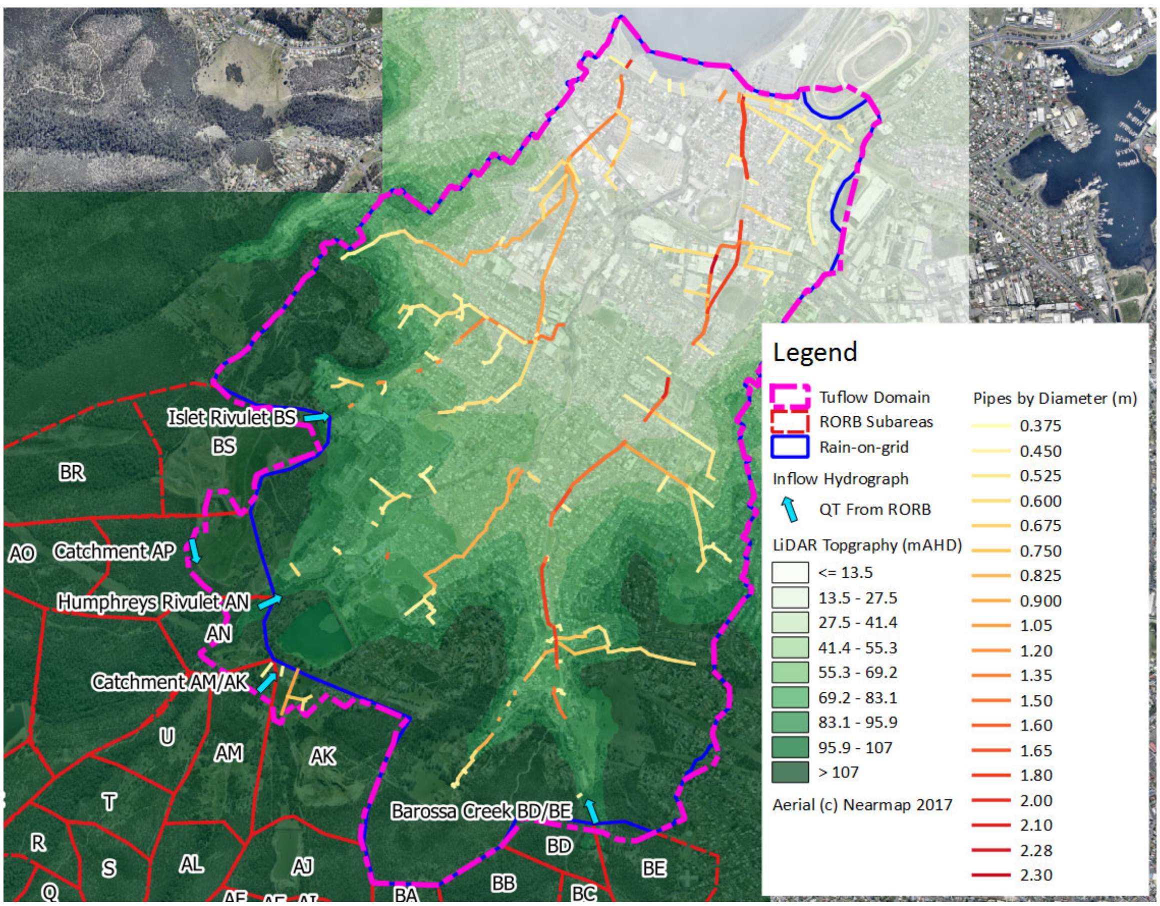


Figure 4-5 –Tuflow model Layout

4.3.1.1. Surface Elevation - LiDAR

LiDAR DEM was supplied (Figure 4-5) by GCC and was used as the basis for the representation of the catchment surface terrain within the Tuflow model. Modifications to the LiDAR-based surface were made to represent elements in the catchment as follows:

- Tolosa Reservoir bathymetry has been estimated and applied
- in some instances, additional elevation geometry has been used to reduce the riverbed elevation to the invert level recorded (in GIS) for the culvert outlet.
- Additional geometry adjusted the channel beds in some locations connecting low elevations in channel beds that may have become 'blocked' by the elevation sampling of the grid points.
- Elevations were reduced (as channels) at the bed levels for culvert outlets to the bay

4.3.1.2. Mannings 'n' Roughness

The land type/planning zone for each sub-area was determined through interrogation of information on the interactive planning scheme maps on the following website (accessed September 2016): http://iplan.tas.gov.au/pages/plan/book.aspx?exhibit=maps&hid=225916
The impervious value was set (Table 4-10) in accordance with the figures presented in Figure 4-6.

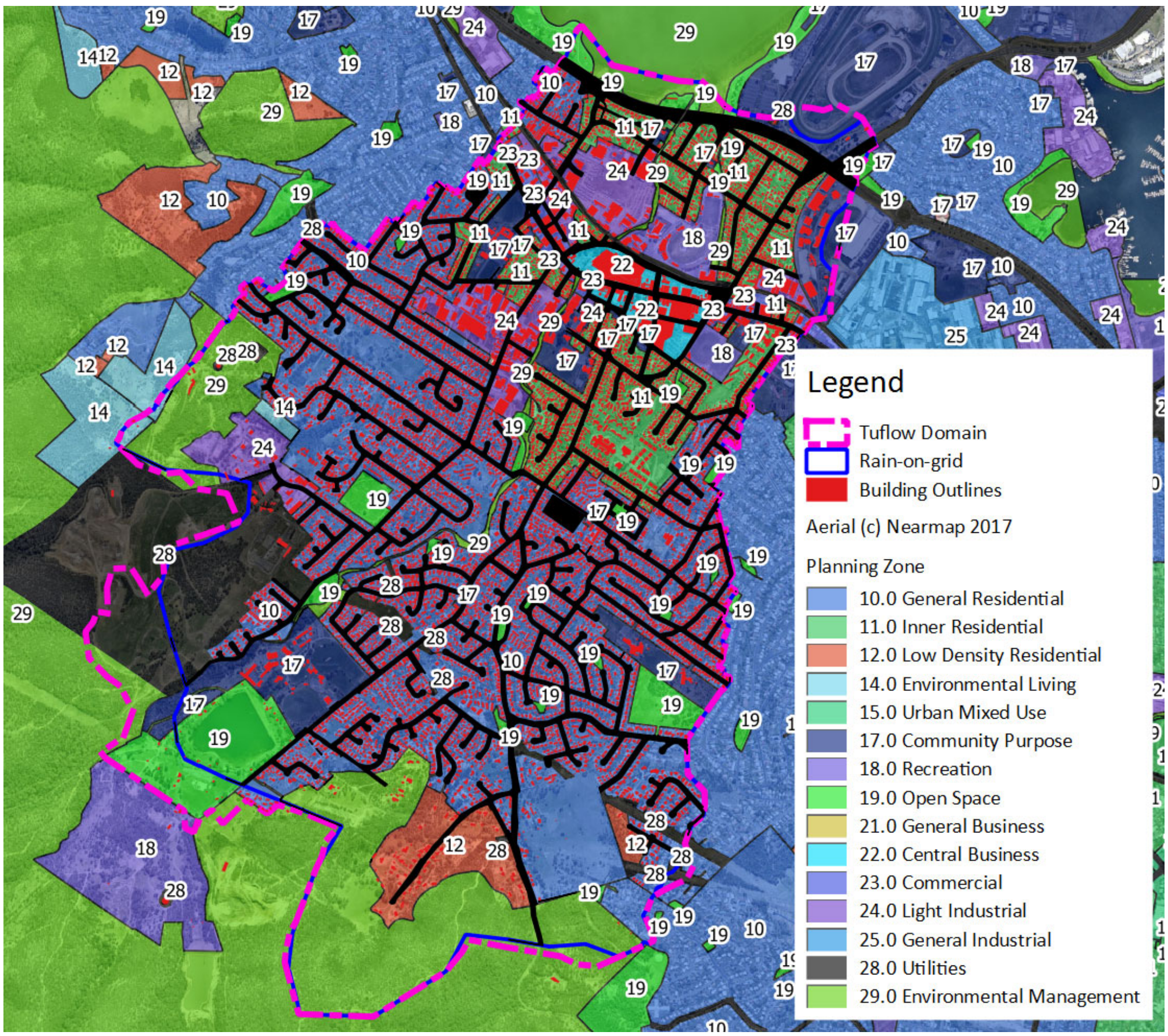


Figure 4-6 – Study Area Planning Scheme Zoning.

A Mannings roughness was selected for each land use and for the building outlines (provided as a GIS cadastral layer). In most cases, a single roughness value was selected (Table 4-10).

Rainfall-on-grid models have large amount of time where the water is very shallow and standard Mannings roughness values would be inappropriate. Hence, in some cases, a variable (varies with water depth) roughness was applied in line with Tuflow's industry guidance for rainfall-on-grid models (*https://wiki.tuflow.com/index.php?title=Tutorial_Module08*).

For residential land use (of all densities) a depth-varying Mannings has been applied in three partitions:

- Less than 100 mm constitutes shallow water depth. This partition assesses the water that is highly impeded by garden beds, fences, etc. and is therefore highly attenuated. For this partition, a higher Mannings value is applied.
- Between 100 mm depth and 500 mm depth, the Manning's roughness transitions via a linear interpolation from a higher value to a lower value (simulating the increasingly destructive nature of the water as it begins to move obstructions out of its way).
- Above 500 mm depth the floodwater a lower Mannings value is selected to represent the greater destructive power of the floodwaters as it can remove obstructions in its way, for example, by knocking down fences and trees, and generally making the landscape smoother for it to pass over.

For buildings, depth varying roughness has been applied in three partitions:

- Less than 40 mm constitutes very shallow water depth. This partition assesses rooftop runoff from a roof. Rooftops typically have a greater slope than the topography. For this partition, a very low Mannings value is applied.
- Between 40 mm depth and 500 mm depth, the Manning's roughness transitions via a linear interpolation from a very low value (simulating rooftop runoff) to a high value (simulating the obstruction that the building gives to floodwaters).
- Above 500 mm depth, a very high Mannings value is selected to represent the attenuation of deeper water attempting to pass through (or under) the walls, doors, or windows.

Table 4-10 – Mannings n Roughness by Land Use

Land Type/Planning Zone	Fraction Impervious	Tuflow Material ID	Manning's n n value (depth m)
10 General residential	0.6	1	0.08 - 0.045 (0.1-0.5)
11 Inner Residential	0.6	1	0.08 - 0.045 (0.1-0.5)
12 Low Density Residential	0.2	11	0.15-0.045 (0.5-1.0)
14 Environmental Living	0.1	11	0.15-0.045 (0.5-1.0)
17 Community Purpose	0.1	14	0.030
18 Recreation	0.1	3	0.035
19 Open Space	0.1	4	0.045
21 General Business	0.9	8	0.045
22 Central Business	0.9	8	0.045
23 Commercial	0.9	8	0.045
24 Light Industrial	0.9	8	0.045
28 Utilities*	0.05	4	0.045
29 Environmental Management	0.05	10	0.150

Land Type/Planning Zone	Fraction Impervious	Tuflow Material ID	Manning's n n value (depth m)
Roads	0.8	2	0.020
Buildings	N/A	12, 15-21	0.02-0.5 (0.04-0.2)
Waterbodies – waterways/bay	1.0	13	0.040

*Some roads (e.g. Brooker Hwy) are listed: 'Utilities'. Fraction impervious of roads supersede other land uses

4.3.1.3. Losses

The losses selected (Table 4-9) were applied as parameters within Tuflow to the full rainfall hyetograph. Setup of the model this way means that the calibration model is identical to the design storm model, with only a change of the rainfall, inflow and tidal inputs.

For impervious surfaces, the storm initial loss was zero (to be consistent with the built-in RORB calculation, noting that 1 mm is more typical in Tuflow models). For pervious surfaces, 28 mm initial loss was applied (Table 4-9).

For each land use, a fraction impervious was selected (Table 4-11). That fraction impervious set what percentage of that area is impervious (and thus impervious losses are applied) and what percentage is pervious (where the Table 4-9 losses are applied).

For example, General Residential planning zone was given a fraction impervious of 0.6. For all catchment areas designated by this zone, 60% is impervious (0mm initial loss), and 40% is pervious (28 mm initial loss). Averaging these two fractions together gives a total initial loss of that land type of 11.2 mm. Table 4-11 summarises this calculation for all initial loss and continuing loss values.

Table 4-11 – Losses by Land Use

Land Type/Planning Zone	Tuflow Material ID	Fraction Impervious	Initial Loss (mm)	Continuing Loss (mm/hr)
10 General residential	1	0.6	11.2	0.60
11 Inner Residential	7	0.6	11.2	0.30
12 Low Density Residential	11	0.2	22.4	1.20
14 Environmental Living	11	0.2	22.4	1.20
17 Community Purpose	14	0.1	25.2	1.35
18 Recreation	3	0.1	25.2	1.35
19 Open Space	4	0.1	25.2	1.35
21 General Business	8	0.9	2.8	0.15
22 Central Business	8	0.9	2.8	0.15
23 Commercial	8	0.9	2.8	0.15
24 Light Industrial	8	0.9	2.8	0.15
28 Utilities	4	0.1	25.2	1.35
29 Environmental Management	10	0.05	26.6	1.425
Roads	2	0.8	5.6	0.30
Waterbodies/Rivulets	13	1.0	0.00	0.00

4.3.1.4. Bridges

The 2D layered flow constriction bridge method has been applied for all bridges within the model based on available drawings and site photos. Layered Flow Constrictions include four layers (Figure 4-7):

- waterway below bridge deck (below red line);
- bridge deck (between red and blue lines);
- bridge railings (between blue and purple lines);
- and above railings (above purple line).

Additional geometry layers have been used to smooth the road surface (blue line) over the bridge and river bed surface (green line) under the bridge removing transition instabilities due to missing lidar data beneath the bridge (Figure 4-7).

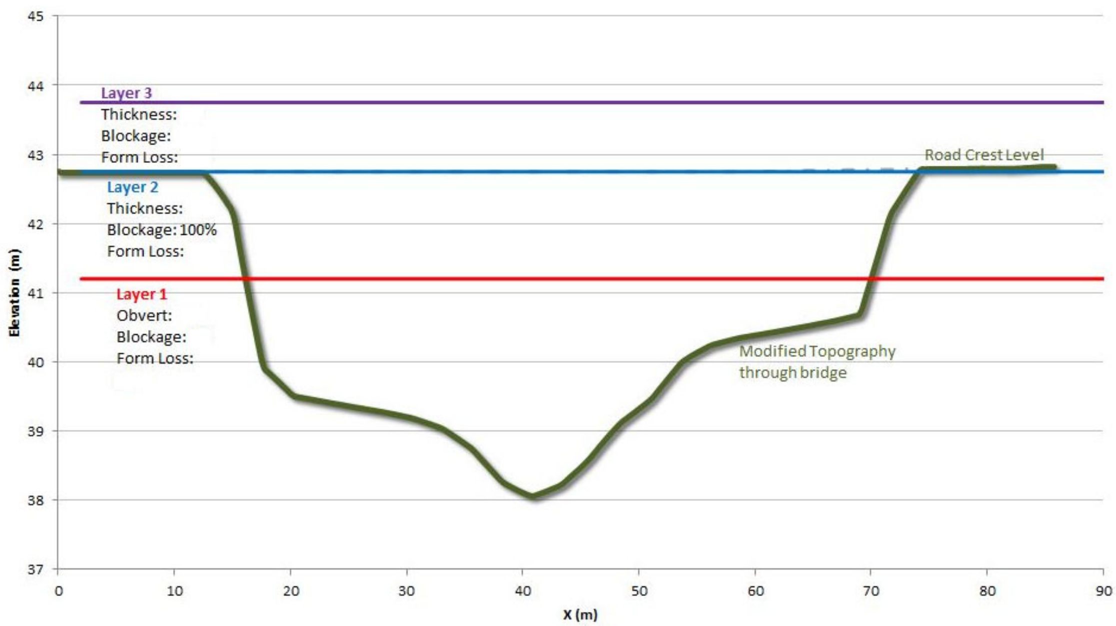


Figure 4-7 – Typical Bridge Cross Section showing 2D Layered Flow Constriction Layers.

4.3.1.5. 1D Network – Pipes and Pits

The GIS pit/pipe information provided by GCC has been used to set up the 1d pipe network. The trunk drainage system of pipes 450 mm diameter and larger has been included. All pits were modelled as rectangular opening 'R' type as 1.5 m wide by 0.2 m opening height. All headwalls were modelled as 'Node' type. Some pits have been shifted slightly to ensure maximum connection to the closest 2D drainage paths. Where parallel pipes of equal depth and size were observed in the GIS information, they have been replaced by a single object with its attribute describing the number of parallel pipes.

4.3.2. Boundary Conditions

4.3.2.1. *Tidal Boundary*

A tidal boundary condition (elevation versus time) has been applied where the rivulets discharge to Elwick Bay. A historical relation has been used for the calibration model, whilst a fixed water level is applied to the design model runs and varied for each scenario.

Table 4-12 – Tidal Boundary Condition Levels

Model Scenario	Tidal Boundary Condition (mAHD)
Calibration	Varies with time Hobart Tide Gauge, 1996
Existing	0.16
Developed with Full Dams	0.16
Developed with Dams at Existing Water Levels	0.16
Climate Change	Varies with each sub-scenario See Section 4.1.5 for more details.

It is considered that selecting the average conditions for the Elwick Bay water level is more appropriate than the worst case. Any given design storm event has an independent probability to the tide level in Elwick Bay at the moment of maximum flow. Without conducting a joint probability assessment, the average conditions are considered to be most likely during a storm event.

Tidal gauges around Tasmania were assessed to augment understanding of tidal conditions in the Derwent River. A comparison of Hobart tidal data with Spring Bay over the same time series suggested that they share the same amplitude but differ slightly in mean (Spring Bay is higher by 0.2 m). Both gauges are somewhat sheltered from the open ocean with minimum water levels around 0.0 mAHD compared with, for example, the Burnie tidal gauge with typical -1.0 mAHD minimum tide levels.

The selected tidal boundary level of 0.16 mAHD is based on the average level of ~30 years of continuous recordings at Spring Bay of 0.36mAHD (mean and median are the same for the 2 gauges; adjusted down by 0.2 m to 0.16mAHDfor Hobart).

4.3.2.2. *Inflow Boundary*

Flood inflow hydrographs have been applied to the Tuflow model at the locations illustrated in Figure 4-5 (labelled as 'QT from RORB'; 'QT' is flow (Q) versus time (T), also known as a hydrograph). These locations provide the connection points between the RORB model and the Tuflow model and are the points where the hybrid model switches from 1D to 2D rainfall-runoff routing.

4.3.2.3. *Rainfall Boundary*

The rainfall is applied to every model grid-cell with the rainfall-on-grid region shown in Figure 4-5. The boundary of the model extends slightly further than this region (indicated by 'Tuflow Domain' on the same figure). A single temporal pattern is selected for each model run and is applied consistently across the entire model.

The rainfall depth is reduced differently, for different portions of the Study Area as discussed below. The rainfall depth is reduced by a combination of losses (initial and continuing) and by the custom spatial pattern (Figure 3-8). The losses are varied using fraction impervious which is selected based on the planning zone (see Table 4-11 for all loss rates applied).

4.3.3. Model Convergence/Adaptive Timestep

Healthy models are those that demonstrate model convergence. Traditionally model convergence of Tuflow models has been examined through interrogation of the Mass Balance Error. This is still appropriate when using Tuflow Classic calculation scheme. However, Tuflow HPC calculation scheme has a 2D mass error of 0% as momentum and volume are conserved between cells.

Model convergence of HPC models is examined through interrogation of the timestep length. The HPC calculation scheme reduces and repeats timesteps as required to maintain stability. The model was checked for repeating timesteps and found that they occurred at a regular interval to synchronise the model time with the output time. These were not due to stability concerns and therefore the model has been confirmed to be stable for all model runs.

4.3.4. Depression Storage

A concern relating to rainfall-on-grid models is "that the topographic information included in the model means that the model can include relatively large depression storage areas which interact with losses" (Ball et. al. 2016).

With this model grid size (2m x 2m), 100mm of depression storage within a single grid cell is 400 litres. If not accounted for these depressions can cause double-counting of losses and under-estimate the magnitude of flood impacts.

An analysis of depression storage with the LiDAR topography was conducted to assess model losses that are not controlled by the modeller. With the full Tuflow model, a 6-hour model run was used to assess the extent of depression storage.

Within the first minute of the model, 500 mm of rainfall depth was applied across the whole model (without the losses applied), and then no further water added for the rest of the run. The total water volume in the model over time (i.e. total rainfall volume minus model outflows) is presented in Figure 4-8).

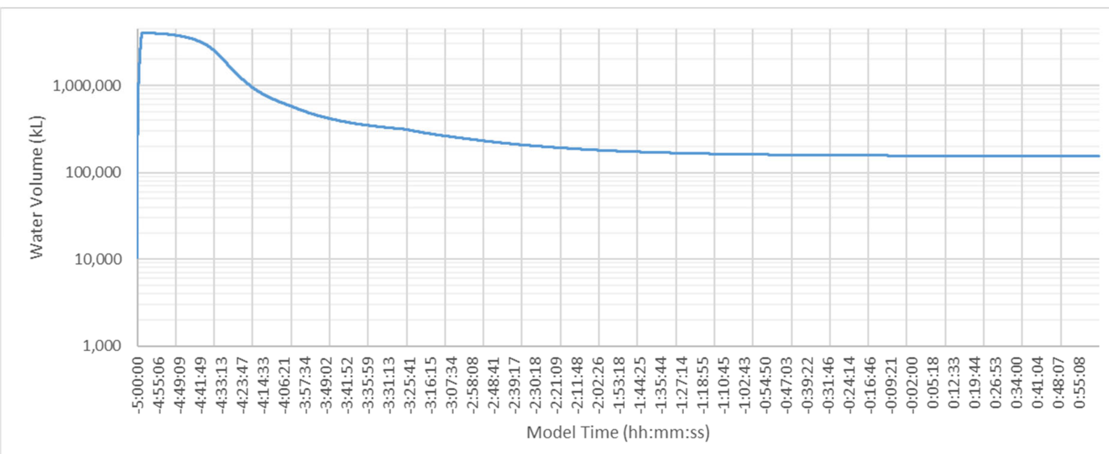


Figure 4-8 – Analysis of LiDAR depression storage volume

The water volume remaining within the model at the end of its run is considered to be equivalent to the depression storage in the model.

The outcomes of the depression storage analysis are:

- 6 hours is enough to estimate resulting volume no longer draining from the model (asymptote of volume line at the end of model runtime (i.e at 1hr).
- The total water volume captured by the model (154 ML) of ~18.5mm uniform depth across the urban catchment. However, 115 ML relates to water volume stored in Tolosa Reservoir,

30 ML within Elwick Bay at 0 mAHD, and captured by depression storage is 9 ML (~1mm uniform depth across urban catchment)

- This volume of depression storage is equivalent to 0.7% of calibration event rainfall depth before applied losses.
- Volumes will vary depending on the tidal level selected in the model run (due to water volume in Elwick Bay).

It was considered that this volume (as 0.7% of calibration event rainfall depth before applied losses) is negligible and unlikely to affect outcomes or objectives.

An approach to filtering out the effects of depression storage could be to run the model with depressions filled as an initial condition, however, given the minor nature of this issue and the effort required, this approach was not adopted.

Instead, volumes were tracked in the calibration model run (Figure 5-4) and in design runs to confirm that peak inundation outcomes were not affected.

4.4. Sensitivity Analysis

4.4.1. General

As part of the model parameter selection process, the sensitivity of the modelled outcomes to a range of different parameters was trialled. The parameters were tested either in isolation within the RORB model or Tuflow, or in combination across the two models.

The elements which were reviewed for sensitivity in RORB included proportional loss, delay and spatial pattern. Note that the sensitivity of parameters was conducted on the model with the dams relationships removed to be consistent with the calibration.

In Tuflow, the parameter sensitivity assessment was conducted in the context of achieving calibration water levels. The conventional parameters originally applied did not provide sufficiently high water levels. The commentary on the Tuflow sensitivity was whether the model responded by changing the water levels at the observed and recorded locations (See Section 5 for more detailed discussion on the model calibration).

4.4.2. RORB Sensitivity

4.4.2.1. Delay

It is recognised that there are a variety of different parameter combinations which can produce similar modelled outcomes. The impact of reducing or increasing the delay value was considered, since the proportional loss adopted falls in the middle of the range of expected values. The parameters required to achieve validation of model parameters to the target 1 in 100 AEP flow were determined with a fixed initial loss. The outcomes are presented in Table 4-13.

Table 4-13 – Delay and Loss Relative Sensitivity

Delay (kc)	Initial Loss (mm)	Proportional Loss	Peak flow (m^3/s)	Critical Duration (hr)
5	8	0.63	81.7	6
7	8	0.56	80.5	6
9	8	0.48	80.2	3
11	8	0.42	81.7	3

The outcomes demonstrate that an increased delay results in a reduction in proportional loss and likewise, a reduced delay results in an increase in proportional loss. The variety of delays trialled represent most of the plausible range. The proportional loss values associated with each of the modelled delays lie within the observed range and as such, each of the parameter sets are considered viable. Table 4-13 offers a variety of parameter sets which may be trialled.

The outcomes of Table 4-13 indicates the relative uncertainty between variables. The impact of varying just one parameter provides an indication of the absolute uncertainty of that parameter on the model outcomes. The outcomes of varying the delay parameter are presented in Table 4-14.

Table 4-14 – Delay Sensitivity

Delay (kc)	Initial Loss (mm)	Continuing Loss (m^3/s)	Peak flow (m^3/s)	Critical Duration (hr)
4	28	1.5	160	3
7	28	1.5	138	3
11	28	1.5	116	3

The peak 1 in 100 AEP flow varies by 16% within the plausible range of delay values indicating the level of uncertainty.

4.4.2.2. Continuing Loss

The outcomes of varying the continuing loss parameter are presented in Table 4-15

Table 4-15 – Continuing Loss Sensitivity

Continuing Loss (m^3/s)	Delay (kc)	Initial Loss (mm)	Peak flow (m^3/s)	Critical Duration (hr)
1.5	7	28	138	3
2.5	7	28	132	3
3.8	7	28	124	3
7.5	7	28	88	3

The peak flow reduces by 4% - 10 % within the plausible range of continuing loss values indicating the level of uncertainty.

4.4.2.3. Spatial Pattern

Section 3.3.5 details creation of a custom spatial pattern to model the orographic impact of the great variability of ground elevation through the Study Area. In that assessment, there was no clear indication in the rainfall gauge history that the orographic impact affected every storm, with only half exhibiting the trend. The other half of the events showed no orographic trend, or even a slight reversed trend.

The model was calibrated, in Section 4.2.4.4, with both the custom spatial pattern and a uniform pattern. The outcome was that the model was not sensitive to that orographic spatial pattern for the loss/delay set adopted. The model was tested to see if it was sensitive for a set of longer delay/smaller loss. The outcomes of the assessment are presented in Table 4-16.

*Table 4-16 – Spatial Pattern Sensitivity (Initial loss fixed at 8mm except for *)*

Spatial Pattern	Delay (kc)	Proportional Loss	Peak flow (m^3/s)	Critical Duration (hr)
Rare Storms	7	0.59	81.7	6
Uniform	7	0.56	80.5	6
Rare Storms	7	0.42* (IL = 28mm)	87.5	6
Uniform	7	0.38* (IL = 28mm)	87.0	6
Rare Storms	11	0.46	81.0	6
Uniform	11	0.42	81.7	3

The outcomes demonstrate that the model is not sensitive to that spatial pattern, independent of loss and delay. This is discussed further, in the context of the hybrid model, in Section 5.2.3.

4.4.3. Tuflow/Hybrid Model Sensitivity

Through the course of achieving the calibration, different parameter variations were trialled to attempt to match water levels observed in 1996, without resorting to unreasonable values. The following is a summary of the parameters assessed and the resulting model sensitivity to variation in that parameter (Table 4-17).

Table 4-17 – Parameter Sensitivity from Calibration of Hybrid model

Model Parameter	Comments	Sensitivity	Adopted Model Parameter
Rainfall temporal pattern Trialled historic measurements from Hobart RO & Mt Wellington	Better calibration with Mt Wellington and with consistent pattern between rural forested and urban catchments	Sensitive	Mt Wellington
Inflow hydrographs from rural forested catchment into the upstream end of the urban catchment	Calibration only able to be achieved with large inflows from the forested catchment	Very sensitive	N/A
Spatial pattern Uniform or custom	The custom pattern provided more inflows from the forested catchment. NB this is discussed in Section 5.2.3 in relation to RORB (no) sensitivity.	Very sensitive	Custom Spatial Pattern
Losses	The catchment needed to be saturated in order to respond to the water levels observed. A very small loss was required to achieve the calibration water levels. The very small loss values required are the equivalent rainfall excess of using standard antecedent catchment conditions (from the RORB validation) and increasing the rainfall depth by 50%	Very sensitive	IL_S = 28mm; CL = 1.5mm/hr
RORB routing parameter K_c – 7.0 or 4.0	The K_c was reduced to 4.0, speeding up the forested catchment response and increasing the flow rate at the upstream end of the urban catchment. The reduction improved the calibration	Sensitive	4.0
Pit losses Constant K of 0.5 for all pits, or varied according to pipe inlet/outlet configuration	Very little calibration improvement from changes to the runoff routing speed from an urban catchment	Not sensitive	Varied K according to pipe inlet/outlet configuration
Grid size 4m or 2m	The 2m grid size better represented the Rivulet channel capacities – increasing the channel capacities and reducing the modelled water levels	Sensitive	2m
Mannings n Constant vs variable with depth	Constant Mannings n provided worse calibration by a negligible amount. Very little calibration change from changes to the runoff routing speed from an urban catchment	Not sensitive	Depth varying
Mannings n Waterway roughness (rivulets and Elwick Bay) trialled with 0.022, 0.025, 0.03, and 0.04	Mannings n of 0.04 provided the best calibration outcome by a negligible amount. Very little calibration change from changes to the runoff routing speed from an urban catchment	Not sensitive	Waterway roughness = 0.04

Model Parameter	Comments	Sensitivity	Adopted Model Parameter
Mannings n Building roughness with water depth less than 40mm trialled with 0.005, 0.01, and 0.02	Mannings n of 0.02 provided the best calibration outcome by a negligible amount. Very little calibration change from changes to the runoff routing speed from an urban catchment	Not sensitive	Building roughness = 0.02
Mannings n Residential roughness (i.e front/back yards) with water depth less than 100mm trialled with 0.06, 0.07, 0.08, 0.10, 0.15, and 0.20	Mannings n of 0.08 provided the best calibration outcome by a negligible amount. Very little calibration change from changes to the runoff routing speed from an urban catchment	Not sensitive	Residential roughness = 0.08
Mannings n Different combinations of waterway, buildings, and residential roughness values from above trialled together	Very little calibration change from changes to the runoff routing speed from an urban catchment	Not sensitive	N/A
Bridge definition Different arrangements of the layered flow constriction trialled to attempt to keep the bridge deck and waterway channel smooth and reduce instability	Bridges smoothed to reduce constriction of flows, which cause significant breakout flooding from Humphreys Rivulet	Sensitive	N/A
1D pipe network 700x pipes of 450mm diameter or larger compared to no 1D network (zero pipes)	Removing the entire 1D pipe network made the calibration worse, but by a negligible amount. Very little calibration change from changes to the runoff routing speed from an urban catchment	Not sensitive	700x pipes of 450mm diameter or larger
1D pipe network 700x pipes of 450mm diameter or larger; 1000x pipes 300mm diameter or larger; and 3000x pipes of 100mm or larger	Keeping all pipes in the GIS database connected to the model caused widespread instability and significantly increased runtimes. Needed to cut back to just trunk network to remove most of instability within the 1D network	N/A	700x pipes of 450mm diameter or larger
Tidal boundary condition – trialled fixed 0 mAHD level and historic recorded levels	All calibration levels lie higher than the tidal levels	Not sensitive	Historic recorded levels
Z shapes to smooth channel beds or add levees along rivulet banks	These changes changed the capacity of the channels and in some places prevented large portions of flood volume from spilling from the waterway into the urban landscape	Sensitive	N/A

5. CALIBRATION OF HYBRID MODEL

5.1. General

As part of the T&B (1997) study, a flooding survey allowed members of the community to provide inputs to the study. GCC received 57 submissions. From that survey information on property inundation from three events was obtained. After the survey follow-up visits were made to 13 landowners, who said that they had recorded flood marks. These observed flood locations are indicated in Figure 5-1.

Figure 5-1 – Location of observed water levels during Feb 1996 event (T&B 1997). Blue line marks Tuflow model extent.

5.2. 1996 Event Rainfall

5.2.1. General

A rainfall hyetograph at Glenorchy was not available for the calibration event for this study. The February 1996 event was recorded over Hobart (at Ellerslie Rd) and Mt Wellington (T&B 1997) and as daily totals at Tolosa Reservoir in Glenorchy (Table 5-1).

5.2.2. 1996 Event Rainfall Depth

The (daily totals) rainfall depth recorded at Tolosa Reservoir, Glenorchy provided a rainfall depth of 119 mm used in the calibration process. Other rainfall depths were recorded at three other rainfall gauges (Table 5-1), ranging from 83.1mm to 256mm.

Calibration to the 1996 event was achieved using a custom spatial pattern (refer to Sections 3.3.5.3 and 5.2.3) which is an areal weighted average with a factor applied over Tolosa Reservoir that reduces the rainfall depth.

Applying an average rainfall depth of 157 mm, with the custom spatial pattern, gives a rainfall depth over Tolosa Reservoir of 119 mm (spatial pattern factor over Tolosa Reservoir of 0.77).

Table 5-1 – Rainfall Depth 08-09/02/1996 (T&B 1997)

Location	Rainfall Depth (mm)
Glenorchy – Tolosa Reservoir	119
Hobart R O (Ellerslie Rd)	83.1
The Springs	201
Mt Wellington	256
Custom Spatial Pattern	157

Figure 5-2 illustrates the calibrated rainfall depth applied to both RORB and Tuflow using the Mt Wellington temporal pattern (NB the first 0.19 mm (0.1%) of the event was skipped to optimise runtime (it would require a further 5 hours of model time for negligible benefit)).

A sensitivity run was conducted using a uniform spatial pattern. For that model-run, the rainfall depth of 119 mm was applied.

5.2.3. Rainfall Spatial Pattern

Section 3.3.5 details the procedure to determine the spatial pattern in Figure 3-8. That section details how the RORB model was not sensitive to the selection of any temporal pattern. However, noting the particularly strong orographic influence on the three temporal patterns recorded for the 1996 calibration event (T&B 1997), this spatial pattern was applied during the calibration run.

To apply the spatial pattern within Tuflow the areal weighted spatial factor (as applied in RORB) was applied to the rainfall regions as a multiplicative factor 'f2' (Table 5-2).

Table 5-2 – Spatial Pattern Factor

Mean Rainfall Depth (Figure 3-8) (%)	Areal Weighted Factor (%)
120	91.9
110	84.2
100	76.5
90	68.9
80	61.2
70	53.6

The February 1996 event was one of the events analysed to determine the custom spatial pattern as it showed strong orographic spatial variation. The probability of the rainfall for the 1996 event can be estimated (e.g. T&B 1997 as 30yr ARI). However, the calibration of this event required that catchment conditions for the peak runoff from the event were near fully saturated (1.5 mm/hr CL, PMF events assume full saturation using 1 mm/hr CL). This model loss parameter corresponds to the real observations of the 24 to 36 hours of significant rainfall prior to the burst period from 10am to 7pm on 9/2/1996 (using the Mt Wellington Pattern; or 40mm rainfall (of 120mm) in the day previous, using the daily data at Glenorchy Reservoir Gauge). Using the probability neutral perspective of design AEPs, despite not having flow gauge records for this Study Area to verify, the '30yr ARI rainfall' produced less frequent than 1 in 30 AEP runoff.

Whilst the RORB model was not sensitive to the selection of any temporal pattern (Section 4.4.2.3), sensitivity of the hybrid model (refer to Section 4.4.3) indicated that it was highly sensitive to the spatial pattern. Given that the purpose of model calibration is to obtain model parameters and outcomes that best match the real catchment, the custom spatial pattern was applied to 1 in 20 AEP and 1 in 100 AEP design runs, and the GSDM (BoM 2003) spatial pattern centred on Glenorchy CBD was applied for PMF design runs.

5.2.4. Rainfall Temporal Pattern

Temporal patterns were available from Hobart Regional Office and Mt Wellington rainfall gauges (refer to Figure 5-2). The Mt Wellington temporal pattern was selected for the entire Study Area (refer to the grey lines: 'Calibration' in Figure 5-2). The daily totals at Glenorchy (Tolosa Reservoir) showed two-thirds of the rain occurred on the second day. Mt Wellington pattern was consistent with this proportion. However, the Hobart pattern had most of the rain occurring on the first day.

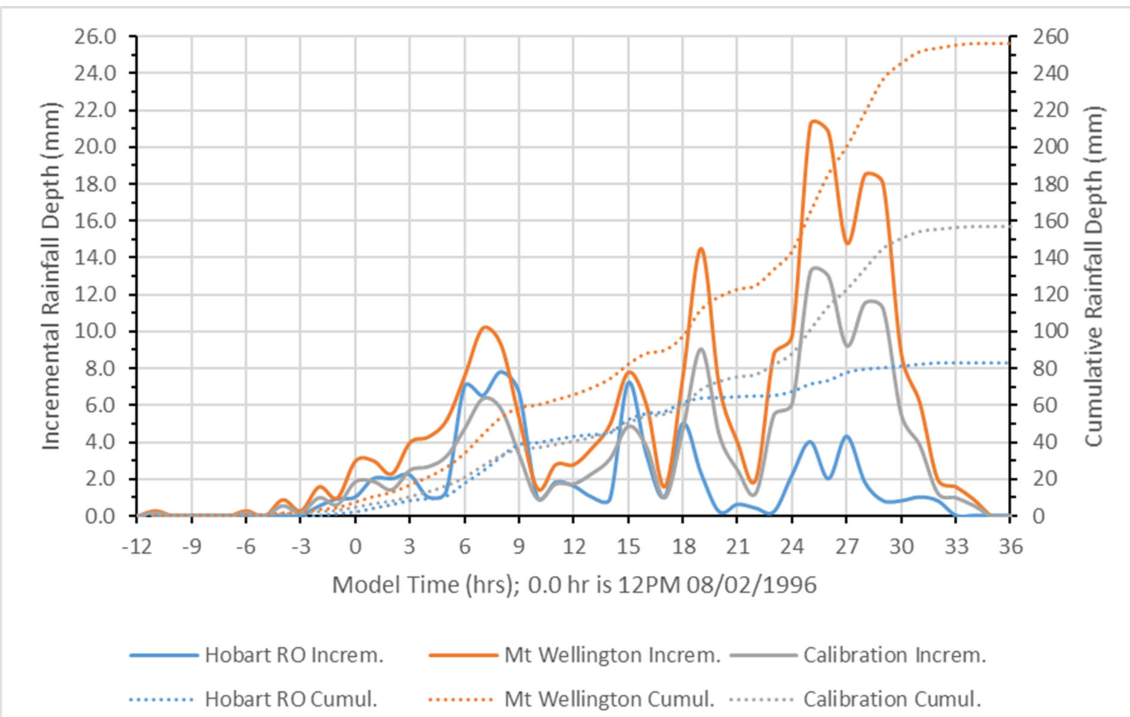


Figure 5-2 - Rainfall (hyetograph) depth time-series 08-09/02/1996 applied in calibration (T&B 1997). Incremental rainfall depths (solid lines) are plotted on the primary (left) axis. Cumulative rainfall depths (dotted lines of same colour) are plotted on the secondary (right) axis.

5.2.5. Rainfall Losses

The storm initial loss has been applied to Tuflow through the materials files. For impervious surfaces, the initial loss was zero (to be consistent with the built-in RORB calculation, noting that 1 mm is more typical in Tuflow models). For pervious surfaces, 28 mm initial loss was applied. For each land use, a fraction impervious was selected, and the initial loss was calculated as the proportion of the two values (i.e. 0 and 28 mm).

Table 5-3 – Losses by Land Use

Land Type/Planning Zone	Tuflow Material ID	Fraction Impervious	Initial Loss (mm)	Continuing Loss (mm/hr)
10 General residential	1	0.6	11.2	0.60
11 Inner Residential	7	0.8	11.2	0.30
12 Low Density Residential	11	0.2	22.4	1.20
14 Environmental Living	11	0.2	22.4	1.20
17 Community Purpose	14	0.1	25.2	1.35
18 Recreation	3	0.1	25.2	1.35
19 Open Space	4	0.1	25.2	1.35
21 General Business	8	0.9	2.8	0.15
22 Central Business	8	0.9	2.8	0.15
23 Commercial	8	0.9	2.8	0.15
24 Light Industrial	8	0.9	2.8	0.15
28 Utilities	4	0.1	25.2	1.35
29 Environmental Management	10	0.05	26.6	1.425
Roads	2	0.8	5.6	0.30
Waterbodies/Rivulets	13	1.0	0.00	0.00

The on-going losses were applied within Tuflow to the complete storm rainfall hyetograph. Setup of the model this way means that after calibration the design rainfalls were applied by changing the rainfall file alone.

The RORB/Tuflow hybrid model was found mostly insensitive to any parameter change except the increase of rainfall volume through reduction in losses (see Section 4.4.3). The 1.5 mm/hr CL was trialled (as per the T&B 1997 calibration), and the model found to be sensitive with improved calibration levels. For the Tuflow model, 0.0mm/hr CL was adopted for impervious surfaces (Ball et al. 2016).

It is noted that T&B 1997 increased the CL to 2.5 mm/hr for their design runs. It is thought that the small loss required for the calibration is due to the event being multiple peaked with a large rainfall depth prior to the final peak at 28hrs, 4pm on 9/2/1996 (largest intensity and volume). Study of the largest events with hyetographs (at Hobart) between 1854 and 2018 show that more than half are multiple peaked events and/or with a large volume of rainfall prior to the largest event peak. It is suggested that the meteorology of Hobart, including the orographic influence of Mt Wellington, would cause multiple peak events to occur with reasonably high probability.

It is intended that CL of 1.5mm/hr be applied to the initial design runs of 1 in 20 AEP and 1 in 100 AEP and the PMF CL of 1.0mm/hr be applied in the PMF runs. A decision on what losses to actually apply was made once the impacts of these very small losses were observed on the 1 in 100 AEP inundation extent. The design runs will be assessed prior to a recommendation being made as to what losses be applied in the design runs.

5.3. Water Levels within Reservoirs

According to T&B (1997) the water levels within Limekiln Gully Dam, Knights Creek Dam and Tolosa Reservoir were not measured before, during or after the February 1996 event. It is reported (anecdotally) that of these three, only Tolosa did not spill. Therefore, Knights Creek and Limekiln Gully Dams were modelled with initial water levels at FSL (within RORB, to ensure spilling) and Tolosa Reservoir at Reduced Operating Level (refer to Table 4-2 in Section 4.1.2 for these water levels).

5.4. Boundary Conditions

T&B (1997) present a tidal relationship in Figure A.4 for historical tide levels for the Hobart Tide Gauge (supplied by Marine Board of Hobart). The tidal relationship is only provided for the 09/02/1996. The historical tide level data for the previous day was not available for this study.

Given that the peak flow was recorded at about 4pm on the 09/02/1996, it is considered unlikely that the outcomes will not be sensitive to the tidal levels on the 08/02/1996. A water level has been applied by taking the first tide level in the record (a high tide level) and projecting the level through the previous day (Figure 5-3). This level may cause higher levels in Elwick Bay and the lower drainage channel at the beginning of the model run, however, there is 14 hours for that water to flow out to Elwick Bay prior to the peak Study Area water levels.

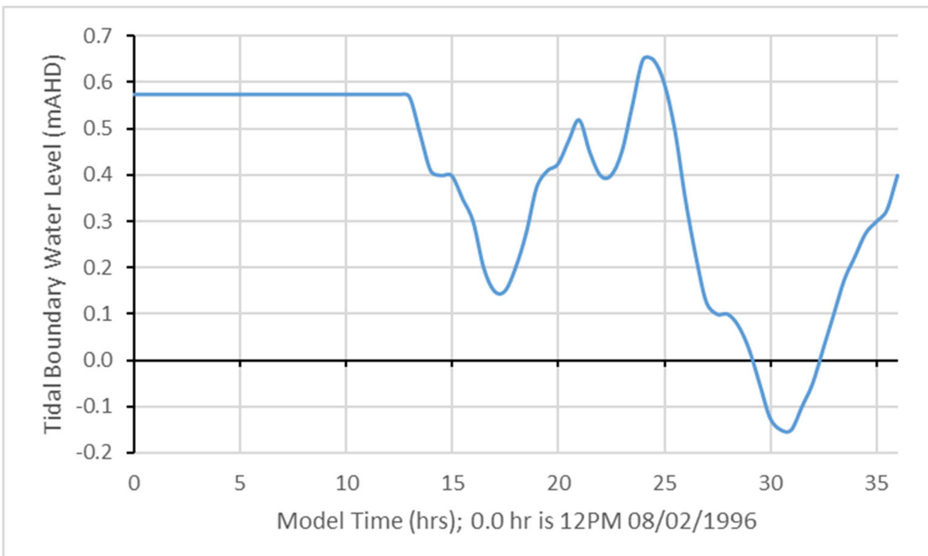


Figure 5-3 - Tidal relationship in Derwent River on 09/02/1996 (Figure A.4 T&B 1997).

Flood hydrographs have been applied to the Tuflow model at the locations illustrated in Figure 4-5.

5.5. Depression Storage Check

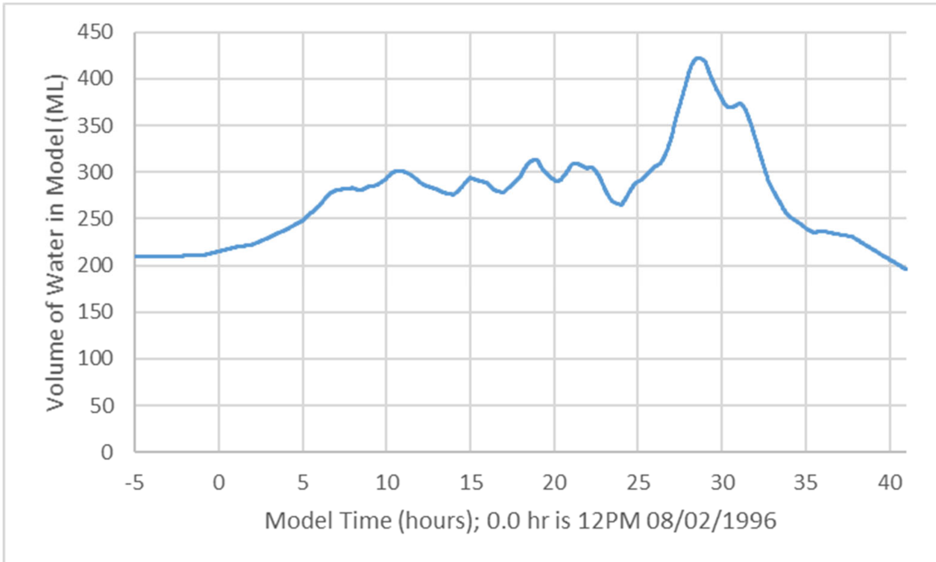


Figure 5-4 – Water volume in Tuflow during calibration run (start = 208 ML; end = 196 ML)

The final water volume in the Calibration model is smaller than the starting volume due to a different water level at the tidal boundary condition. The entire inflow volume of the calibration storm event is accounted for.

5.6. Modelling the Calibration Locations

Plot output (PO) lines have been digitised to provide time-series model output (level and flow) close to the locations (Figure 5-1) where water levels were observed in 1996 (T&B 1997). The maximum of the time series provided a first-pass estimate of the water level.

For many locations the precise measurement location is unclear, so the region where the location was possible was interrogated. For each region interrogated, a range of water levels from the model were extracted from the maximum gridded water level outputs. The water level ranges estimated from the model are listed in Table 5-4 compared to the levels measured and the levels that T&B (1997) achieved from their model. Comments are provided to give indication on where the water levels were taken from the model (refer to Table 5-4).

5.7. Calibration Outcomes

As recommended by SMEC (and mentioned in Section 1.3), WMA Water were engaged by GCC to provide peer review for this flood study. Two reviews were completed after calibration of the Tuflow to measured water levels. On 31/07/2018, WMA Water approved the calibration and adopted parameters (later summarised in this report) for use in the design runs.

The water levels estimated from the model are listed in Table 5-4 compared to the levels measured and the levels that T&B (1997) achieved from their model.

Table 5-4 – Calibration Water Levels

Address	Description of Level Location (T&B 1997)	Flood Level Observed (mAHD)	T&B '97 Calibration		This Study Calibration		Comment
			Level (mAHD)	Difference (m)	Model Water Level (mAHD)	Difference b/w Model and Observed (m)	
40 Anfield Street	mark on door frame of garage door	2.844	2.904	0.060	(2.834)	(-0.010)	Garage is visible on aerial. Model water level is constant for the entire length. Most clear point for calibration.
55a Grove Road	bottom rail on side fence	4.335	4.245	-0.090	4.190 to 4.415	-0.145 to 0.080	Unclear what location along the side fence was measured. Water level range output from model taken from fence portion visible from house.
55 Grove Road	mark on shed wall	4.262	4.212	-0.050	4.205 to 4.283	-0.057 to 0.021	Current aerial does not show a shed, but instead shows this property has been subdivided into 3 units. Older 2008 Google Streetview shows the location of the garage when the property was a single dwelling. Assumed this garage is the referenced 'shed'. Model water levels taken over the region of the previous garage.
1 Young Street	mark on cupboard in garage	6.853	6.833	-0.020	6.295 to 6.478	-0.558 to -0.375	T&B 1997 report on a 'breach' of the levee near Olympic Pool. The breach (or low point overtopping) has since been repaired. The dimensions of the breach are unclear. A breach has been modelled and will be removed (filled in) for the design runs. The modelled breach is underestimating flow for 1 Young Street but is providing reasonable water levels at 2 Young Street and 40 Anfield Street.
2 Young Street	mark on dog kennel	7.948	8.088	0.140	7.560 to 8.110	-0.388 to 0.162	
21 Balmain Street	mark on top of bank	31.462	31.502	0.040	(31.429)	(-0.034)	During site inspection with Mathew Brockman, Drainage Works Officer GCC (06/12/2017) anecdotal information was shared that the Humphreys Rivulet waterway downstream of Brent St was realigned following the 1996 event when properties to the north were threatened. T&B 1997 likely had access to the channel bed topography that was current at the time of the event. The LiDAR of this model has the current waterway alignment. As such, it is not considered possible to get a match to the 4 locations downstream of Brent St bridge: 21 Balmain St, 5 Chelmsford Pl, 28 Barrett St and 11 Farnell St. T&B (1997) provided the measured water level at 11 Farnell St but did not record model calibration at it (Table C.2 includes the 'Flood level observed' with blank space for 'Predicted' and 'Diff' columns). No comment made by T&B in explanation.
5 Chelmsford Place	top stone wall RHS	37.404	37.364	-0.040	(36.704)	(-0.700)	
28 Barrett Street	100 mm above GL next to tree	38.755	38.755	0.000	(38.646)	(-0.110)	
11 Farnell Street	200 mm above wall at kennel	34.934			(34.700)	(-0.234)	

Address	Description of Level Location (T&B 1997)	Flood Level Observed (mAHD)	T&B '97 Calibration		This Study Calibration		Comment
			Level (mAHD)	Difference (m)	Model Water Level (mAHD)	Difference b/w Model and Observed (m)	
Brent Street Bridge	left side abutment upstream	42.235	41.535	-0.700	42.210 to 42.420	-0.025 to 0.185	During site inspection with Mathew Brockman, Drainage Works Officer GCC (06/12/2017) anecdotal information was shared that Brent St bridge overtopped in Feb 1996 and that the gabion U/S of Brent St has been installed since the 1996 flood. Model channel shape is likely different to 1996 so a good match is unlikely, however, model shows bridge slightly (<0.2m) overtops.
1/12 Brent Street	mark on side fence	42.644			42.345 to 42.400	-0.299 to -0.244	During site inspection with Mathew Brockman, Drainage Works Officer GCC (06/12/2017) anecdotal information was shared that Brent St bridge overtopped in Feb 1996 and that the gabion U/S of Brent St has been installed since the 1996 flood. Model channel shape is likely different to 1996 so a good match is unlikely, however, model shows bridge slightly (<0.2m) overtops. T&B (1997) provided the measured water level at 1/12 Brent St but did not record model calibration at it (Table C.2). No comment is made by T&B in explanation.
171a Chapel Street	top bank of Rivulet	65.615			65.630 to 66.900	0.015 to 1.285	It is unclear from the aerial or lidar where along the top of the bank is referenced. A calibration to this point is not considered precise. T&B (1997) provided the measured water level at 171a Chapel St but did not record model calibration at it (Table C.2). No comment is made by T&B in explanation.
2/16 Whitbread Crescent	top Telstra conduit back unit	70.052			69.460 to 70.460	-0.592 to 0.408	It is unclear from the aerial or Google Streetview where the Telstra conduit is or was in 1997. Property varies in level by 1.0 m from front to back, so calibration is not precise at this property. T&B (1997) provided the measured water level at 2/16 Whitbread Cr but did not record model calibration at it (Table C.2). No comment is made by T&B in explanation.

6. MODEL RUNS OF DESIGN SCENARIOS

6.1. General

Once calibrated, the hybrid model was run for a set of scenarios required by GCC to assess the breakout flood risk from Humphreys Rivulet and the system performance. The four key scenarios modelled include:

- Scenario 1 – existing
- Scenario 2 – developed with the dams at Full Supply Level (i.e. completely full without spilling)
- Scenario 3 – developed with the existing dam draw down water levels:
- Scenario 4 – developed with climate change impacting rainfall intensity and ocean water levels

6.2. Model Outcomes

6.2.1. Critical durations

Each of the probability storm events was run for nine storm durations ranging from 30 minutes to 24 hours. Each duration was run in RORB ten times, each with a different temporal pattern. The pattern that produced a peak flow at the RORB/Tuflow boundary closest to the mean average of the ten was selected to be run within Tuflow. Table 6-1 and Table 6-2 summarise the critical duration and flow at each of the boundary locations for design and climate change rainfall intensities.

Table 6-1 – RORB model results at a range of locations connecting the RORB and Tuflow

Location	1 in 20 AEP		1 in 100 AEP		PMF	
	Peak FLow	Critical Duration	Peak FLow	Critical Duration	Peak FLow	Critical Duration
Islet Rivulet at BS	2.69	6 hour	4.66	3 hour	107	1 hour
Catchment AP	2.52	6 hour	4.13	3 hour	84.8	1 hour
Humphreys Rivulet at AN	44.8	2 hour	76.0	6 hour	884	1.5 hour
Catchment AK/AM	4.08	6 hour	6.34	3 hour	126	1 hour
Barossa Creek at BD/BE	4.59	6 hour	7.08	3 hour	147	30 minute

Table 6-2- RORB results at locations connecting RORB and Tuflow with climate change rain intensity

Location	1 in 20 AEP		1 in 100 AEP		PMF	
	Peak FLow	Critical Duration	Peak FLow	Critical Duration	Peak FLow	Critical Duration
Islet Rivulet at BS	4.00	6 hour	6.17	3 hour	107	1 hour
Catchment AP	3.92	2 hour	5.56	3 hour	84.8	1 hour
Humphreys Rivulet at AN	69.5	1 hour	105	6 hour	884	1.5 hour
Catchment AK/AM	6.23	2 hour	8.98	2 hour	126	1 hour
Barossa Creek at BD/BE	7.09	2 hour	10.2	2 hour	147	30 minute

These tables above summarise the critical flows at these locations. However, nine different hydrographs were fed into the Tuflow model, one for each duration (the average peak flow of the ensemble of temporal patterns) at each location matching to the rainfall-on-grid hyetographs duration.

To identify the critical storm duration across the urban region of the Study Area the maximum inundation extents for each storm duration were compared and presented in the Critical Event Map (Figure 6-1 presents the Existing Scenario 1 in 20 AEP event; Figure 6-2 presents the Existing Scenario 1 in 100 AEP event).

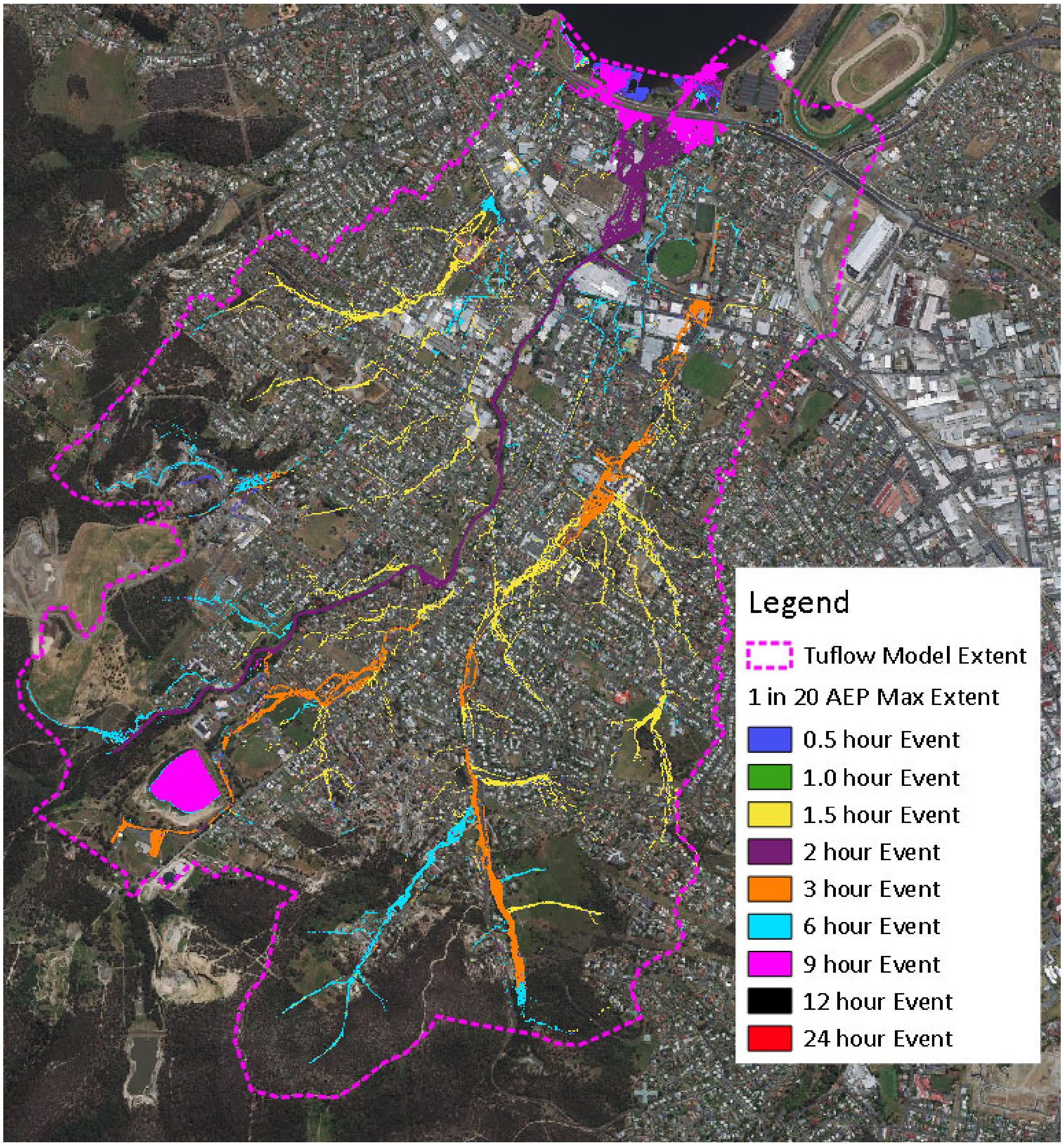


Figure 6-1 – Critical Event Map for Existing Scenario 1 in 20 AEP Inundation Event

Figure 6-1 indicates that rainfall duration has an impact on the maximum inundation extents across the Study Area.

In the 1 in 20 AEP event, Humphreys Rivulet reaches maximum inundation depth in the 2-hour event, whilst the Barossa and Little John Creeks reach maximum depth in the 90 minute and 3-hour events respectively. For the upper parts of the Study Area and central CBD, the 6-hour event causes the maximum flood depth.

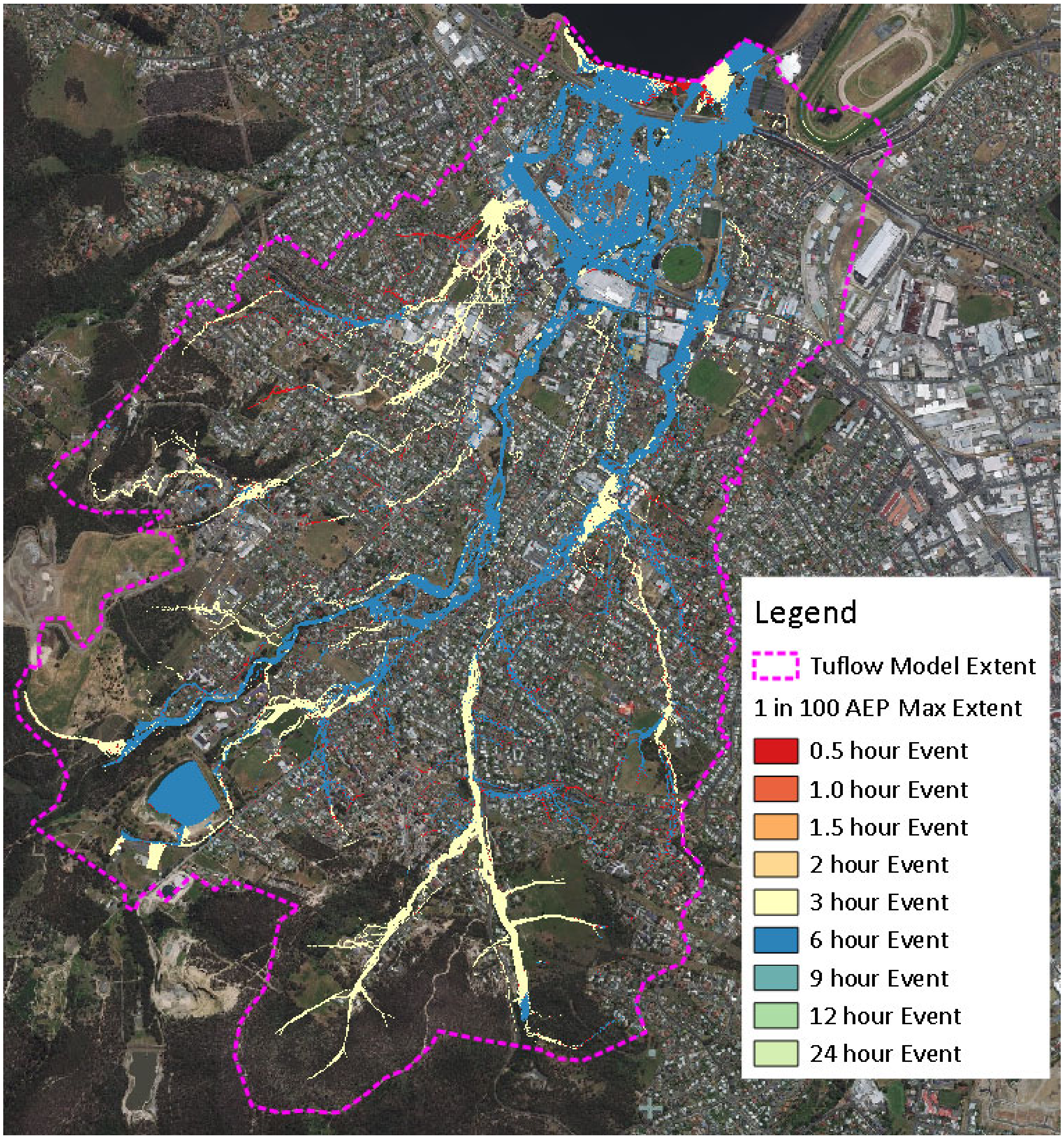


Figure 6-2 – Critical Event Map for Existing Scenario 1 in 100 AEP Inundation Event

The 6 hour 1 in 100 AEP causes maximum inundation depths throughout much of the Study Area (Figure 6-2), especially along Humphreys Rivulet. The upper catchment of Barossa Creek and Islet Rivulet reach maximum depth in the 3-hour event.

For some parts of the Study Area flash flooding occurs in the 30-minute event causing maximum inundation depth along local streets (red).

6.2.2. Filtering of Results

The rainfall-on-grid rainfall-runoff process applies the rainfall in a distributed manner across the entire catchment and then leaves the routing to hydraulic processes across the grid surface. This can leave behind small clusters of flooding up to a dozen grid cells within localised depressions in the model grid that are not necessarily representative of the real topography. These small water clusters, or 'puddles', produce a speckled effect on the inundation maps that distract from the information being presented and so require removal.

Melbourne Water (2012) guidelines on minimum requirements for Flood Mapping Projects provide guidance on the inundation map filtering parameters expected for projects within their jurisdiction. "The filtering parameters were that all points with a depth greater than or equal to 50mm AND a velocity times depth product greater than 0.008 would be used for the flood extent determination."

Similar filtering criteria were applied to this study. To account for Glenorchy's on average steeper topography, a depth criterion of 30mm was applied in addition to the product of depth and velocity (DV) of 0.008 m^2/s.

The adopted filtering parameters are:

- Remove all inundated area with water depth less than 30mm and with DV (depth times velocity) less than 0.008 m^2/s
- Remove all separate 'puddles' with an area of 5 grid cells (i.e. 21 m^2) or smaller

6.3. Flood Mapping Outcomes

Inundation depth, flood hazard (DV) and hazard category (low to extreme) maps for all scenarios are included in Appendix E.

Flood hazard category maps are presented using Ball et. al. (2016) Book 6, Section 7.2.3 Flood Hazard for People Stability, using Table 6.7.1 hazard regimes for adults. Flood hazard categories include Low, Moderate, Significant and Extreme. Moderate and Significant categories do not include depths above 1.2 m or velocity above 3.0 m/s, which were categorised as Extreme as per Table. 6.7.1.

It is noted that the flood hazard categories of the maps is for adults (not children, vehicles, buildings, et. cetera which have different categories). For example, flood hazard categories Moderate, Significant and Extreme are all considered to be categorised as 'Extreme' for Children between 25 to 50 m.kg (height x mass).

Some key results of the flood mapping outputs are:

- Significant property flooding in the Study Area with the following number of properties modelled as flooded:
  - 1 in 20 AEP: 40 properties
  - 1 in 100 AEP: 84 properties
  - PMF: 1,630 properties
- Significant flooding of key community infrastructure in 1 in 100 AEP:
  - Northgate Shopping Centre
  - Glenorchy Pool
  - KGV Oval
  - Glenorchy Plaza
  - Barossa Park Lodge aged care service
  - Tiny Tackers Children Centre
  - Dominic College

It should be noted that the inundation extent is likely to be sensitive to the assumption that floor levels are 300 mm higher than property ground levels. Many property buildings on the inundation maps show flooding depths of less than 300 mm, and so lower than floor levels. It is recommended that floor level survey should be completed for properties modelled as flooded in the 1 in 100 AEP event prior to use of these maps for other purposes (e.g. flooding overlays on planning maps.

6.4. Future Development Scenarios

Three scenarios have been developed to assess the outcomes to different future development impacts within the Study Area. The two key future impacts on the Study Area tested in this study are the impact of the water level in Knights Creek and Limekiln Gully Dams, and of climate change.

6.4.1. Storage Level in Dams

For the 1 in 20 AEP and 1 in 100 AEP events, there is very little difference between the Existing Scenario and the Developed with Dams at Drawdown. This outcome is related to the very low continuing loss required to match calibration to the 1996 event.

There is a marked difference between the two Developed Scenarios with different dams storage levels. The 1 in 20 AEP for Dams Full (Scenario 2) flood impact is very similar to the 1 in 100 AEP with Dams at Drawdown Levels (Scenario 3). This outcome demonstrates how effective the two dams are at attenuating the impact of intermediate and rare storm events on the Study Area urban catchment. However, any future proposal with regards to utilising these two dams for flood attenuation purpose needs to be carefully considered from not only a hydraulic perspective but also from a dam safety and asset management perspective.

6.4.2. Climate Change

The climate change scenarios show substantially more impact than the Existing or other Developed Scenarios, especially along the coastline of Elwick Bay.

The 1 in 20 AEP flood impact is greater than the Existing or Developed 1 in 100 AEP event. However, it is difficult to divide the coastal flooding impact between the Sea Level Rise (SLR) and the storm-related water level due to catchment flooding, storm surge and high tide.

Part of the inundation extent along the coast should be considered the ‘new coastline’ and the remainder, impact from the flood event. This is especially applicable in the flood damage assessment as the Climate Change damage costs will include inundated property damage costs from prior to the flood event (from the SLR).

6.5. Flood Damage Assessment

6.5.1. General

A flood damage assessment was undertaken to estimate the monetary costs of flooding impact. The flood damages assessment was conducted following the industry standard method to establish the relative damage costs experienced within the Study Area for all flood events modelled under existing and developed scenarios. Flood damages at properties were estimated using the averaging approach method presented in the Disaster Loss Assessment Guidelines (2002) and Floodplain Management in Australia (SCARM 2000).

The damage costs estimated in this study represent a potential approximation only, determined following the standard methodology. The damages are not intended to be an exhaustive assessment of the full economic impact of a flood event. Nor does this assessment account for situations where people may attempt to protect their property from damage during the event and reduce the monetary impact.

Building damages have been based on standard recommended ‘damage curves’ rather than historical or real-time insurance data. Nevertheless, this methodology is considered appropriate for the intended purpose of providing relative cost comparisons between scenarios and providing a benchmark for comparing mitigation options against.

Damages from a disaster can be classified as direct (i.e. damages resulting from the action of floodwaters and flow) or indirect (i.e. disruption to daily activities due to the disaster or relief aid and clean-up costs). Damages can also be sub-classified as tangible (i.e. can be assigned a monetary value) or intangible (e.g. loss of life or injury). This study will limit the scope of assessment to tangible (monetary) costs of flooding.

The comparative indicator between scenarios that is derived from this assessment is the Average Annual Damage (AAD). AAD is the total damage caused by floods over a long period of time divided by the number of years in that period (SCARM 2000). It is calculated by plotting loss estimates for the flood hazard at a range of magnitudes (i.e. inundation depth), against the probability of occurrence of the flood event (i.e. the AEP). AAD represents the area under this curve, an estimated monetary impact of the flood damage sustained every year on average (mean) over an extended period.

All monetary values have been adjusted to 2018 dollars using information published by the Australian Bureau of Statistics.

6.5.2. Procedure

The flood damage assessment required the following input data:

- Property boundaries (supplied by GCC as GIS layers)
- Design flood depths for a range of probability events
- Floor levels at each property (approximated by adding 300 mm to ground level data at each property from LiDAR (2011) supplied by GCC).

The key steps involved in the flood damage assessment are:

1. Create a database of residential, commercial, industrial, and community buildings and their respective floor levels.
2. For each zone type of property, determine a depth-cost relationship for flooding based on accepted methods/resources.
3. For each property in the Study Area, assign a damage cost based on the modelled inundation depth and the appropriate depth-cost curve.
4. Repeat step 3 for each scenario and flood event.
5. Calculate the Average Annual Damages (AAD).

6.5.3. Damage Assessment Outcomes

The flood damage assessment results are summarised in Table 6-3 for all scenarios and flood events modelled. The final Average Annual Damages (AAD) for each scenario is also included. The damage cost curves are plotted in Figure 6-3.

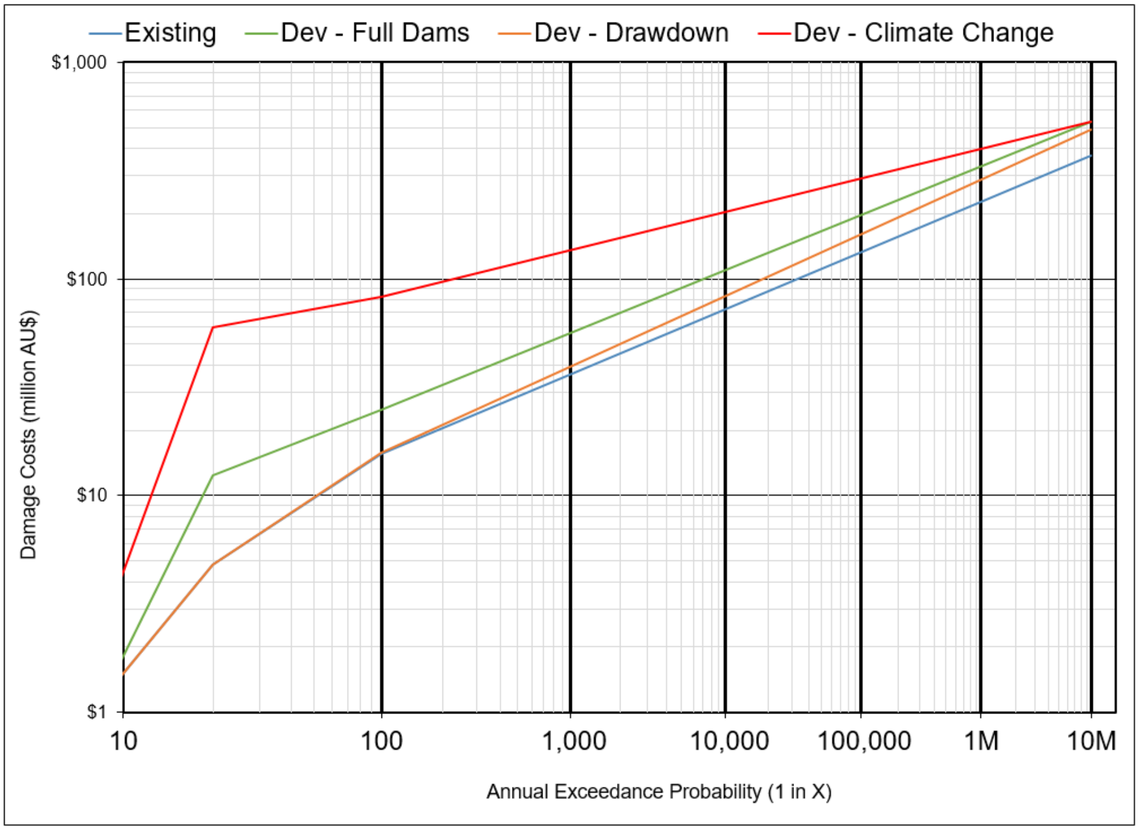


Figure 6-3 –Damage Costs Against Flood Probability

It should be noted that the flood damage assessment is likely to be sensitive to the assumption that floor levels are 300 mm higher than property ground levels, and it is recommended that a floor level survey should be completed for properties modelled as flooded in the 1 in 100 AEP event and the damage assessment revised.

Reviewing the outcomes, it is expected to observe that there is substantially more damage in the developed scenarios where there is increased rainfall (climate change), and reduced flood protection from the three dams (Tolosa decommissioned and 'Full Dams').

The outcome showing Existing and Developed (dams at drawdown levels) as similar for the more frequent events suggests that the Glenorchy community does not face substantial increased flood risk from additional infill development.

The Climate Change outcomes show an increase of $16 million average annual damages. Of this increase, an unknown portion (related to roughly half of the water level increase at Elwick Bay) comes from the sea level rise impact prior to the flood event. These are displacement damages rather than flood damages, but are difficult to remove from the outcome.

Table 6-3 – Summary of Flood Damage Costs

Scenario	AEP (1 in Y)	Residential		Commercial		Community		Structural Damages	Total	AAD Total
		Number of Dwellings	Damages	Number of Dwellings	Damages	Number of Dwellings	Damages			
Existing	20	22	$1,519,000	10	$976,000	8	$1,192,000	$0	$4,776,000	$3,411,000
	100	48	$2,931,000	27	$5,894,000	9	$1,769,000	$122,000	$15,566,000	
	PMF	1185	$115,529,000	265	$103,929,000	180	$47,511,000	$5,246,000	$370,207,000	
Developed - Full Dams	20	41	$2,656,000	26	$4,808,000	8	$1,011,000	$0	$12,390,000	$6,317,000
	100	87	$4,956,000	45	$10,064,000	9	$1,578,000	$122,000	$24,763,000	
	PMF	1377	$153,614,000	304	$160,087,000	180	$57,758,000	$14,762,000	$532,202,000	
Developed - Dams at Draw Down	20	22	$1,523,000	10	$1,030,000	8	$1,192,000	$0	$4,789,000	$4,025,000
	100	50	$2,958,000	27	$6,097,000	9	$1,578,000	$122,000	$15,721,000	
	PMF	1341	$145,363,000	292	$142,576,000	191	$57,758,000	$13,420,000	$491,400,000	
Developed - Climate Change	20 (CC1)	157	$21,406,000	26	$5,075,000	23	$8,237,000	$14,518,000	$59,412,000	$19,289,000
	100 (CC3)	224	$28,371,000	51	$10,102,000	34	$9,918,000	$18,788,000	$82,812,000	

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APPENDIX A FLOOD FREQUENCY CURVES

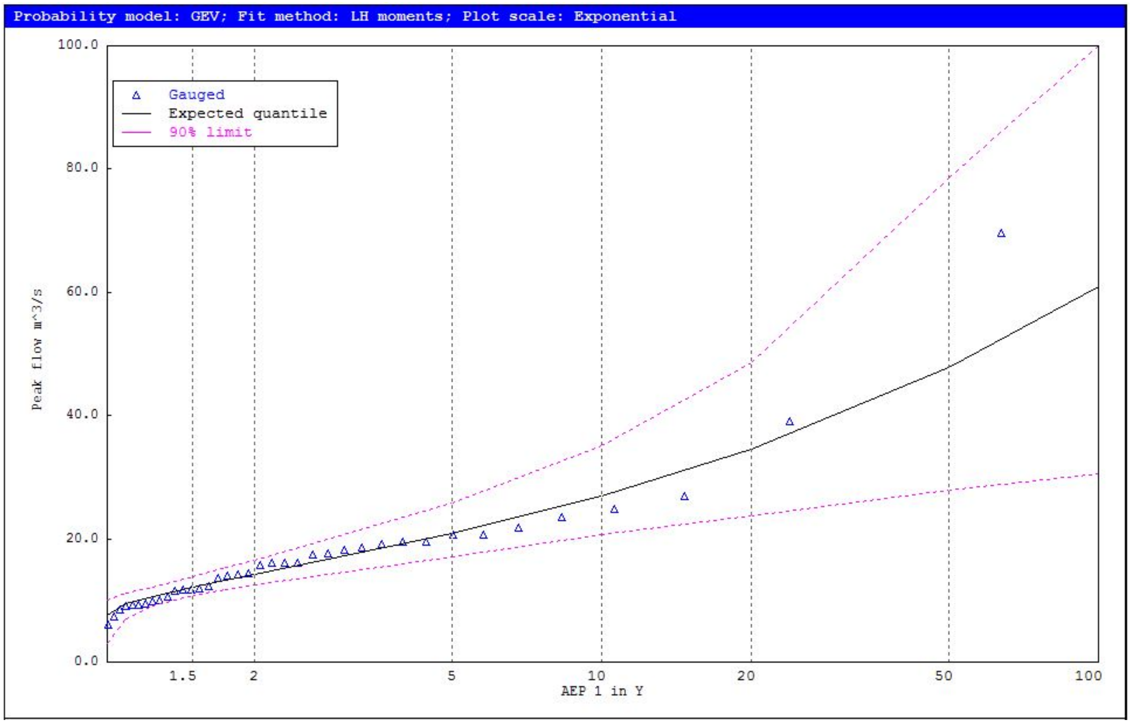


Figure A-1: #353-Hobart Rivulet, Gore Street Flood Frequency Curve

Title: #353-Hobart Rivulet at Gore Street
Optimized L moment shift = 1

GEV Fit Results

Parameter	LH	Mean	Std dev	Correlation
tau	12.581	12.619	0.799	1.000
a	4.181	4.401	1.150	0.405 1.000
k	-0.354	-0.288	0.190	0.219 0.501 1.000

AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	7.6	3.0	10.1	1.53
1.10	9.4	7.1	11.2	0.87
1.25	10.7	9.2	12.2	0.48
1.50	12.2	10.8	13.8	0.09
1.75	13.3	11.8	15.3	-0.17
2.00	14.2	12.5	16.6	-0.37
5.00	20.9	17.1	25.9	-1.50
10.00	27.0	20.6	35.1	-2.25
20.00	34.6	23.8	48.6	-2.97
50.00	47.8	27.8	78.6	-3.90

100.00	61.0	30.5	117.0	-4.60

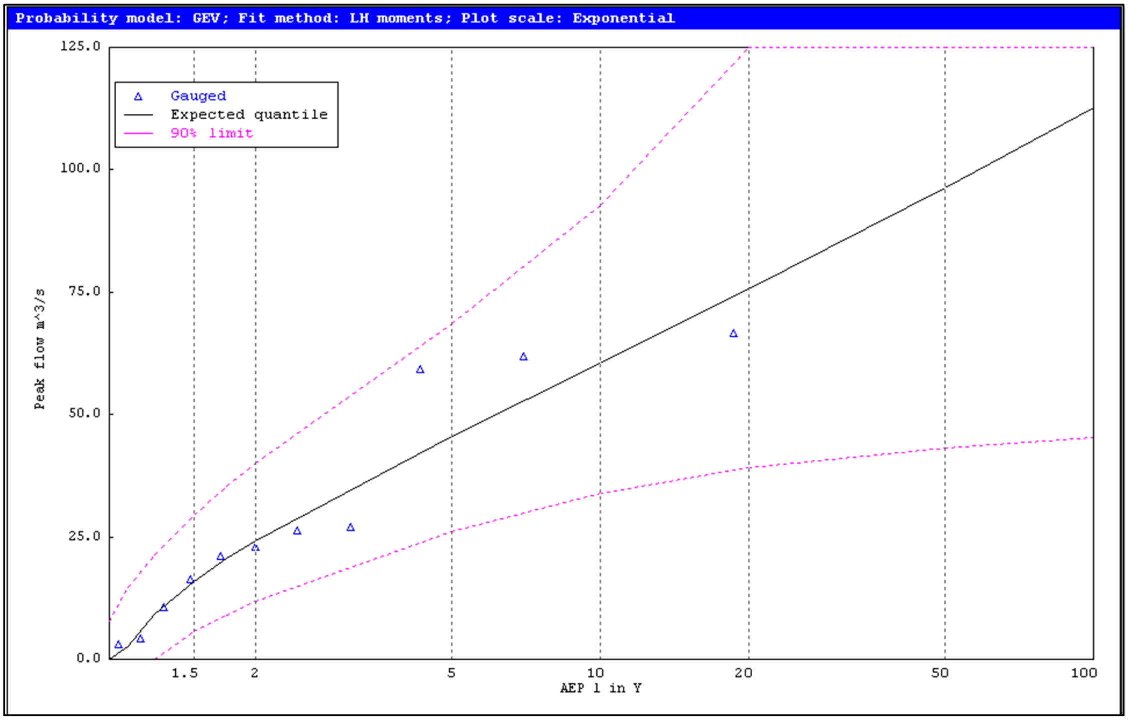


Figure A-2: #354- Hobart Rivulet, Argyle Street Flood Frequency Curve

Title: #354- Hobart Rivulet at Argyle Street
Optimized L moment shift = 0

GEV Fit Results

Parameter	LH	Mean	Std dev	Correlation		
tau	17.617	18.031	6.295	1.000		
a	17.710	17.507	5.204	0.441	1.000	
k	-0.065	-0.013	0.272	0.392	0.351	1.000

AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-8.2	-31.8	7.9	1.53
1.10	2.6	-8.4	14.8	0.87
1.25	9.3	-0.2	21.4	0.48
1.50	16.0	5.6	29.5	0.09
1.75	20.6	9.1	35.3	-0.17
2.00	24.2	11.8	40.0	-0.37
5.00	45.5	26.2	68.5	-1.50
10.00	60.5	33.9	92.8	-2.25
20.00	75.7	39.1	124.9	-2.97

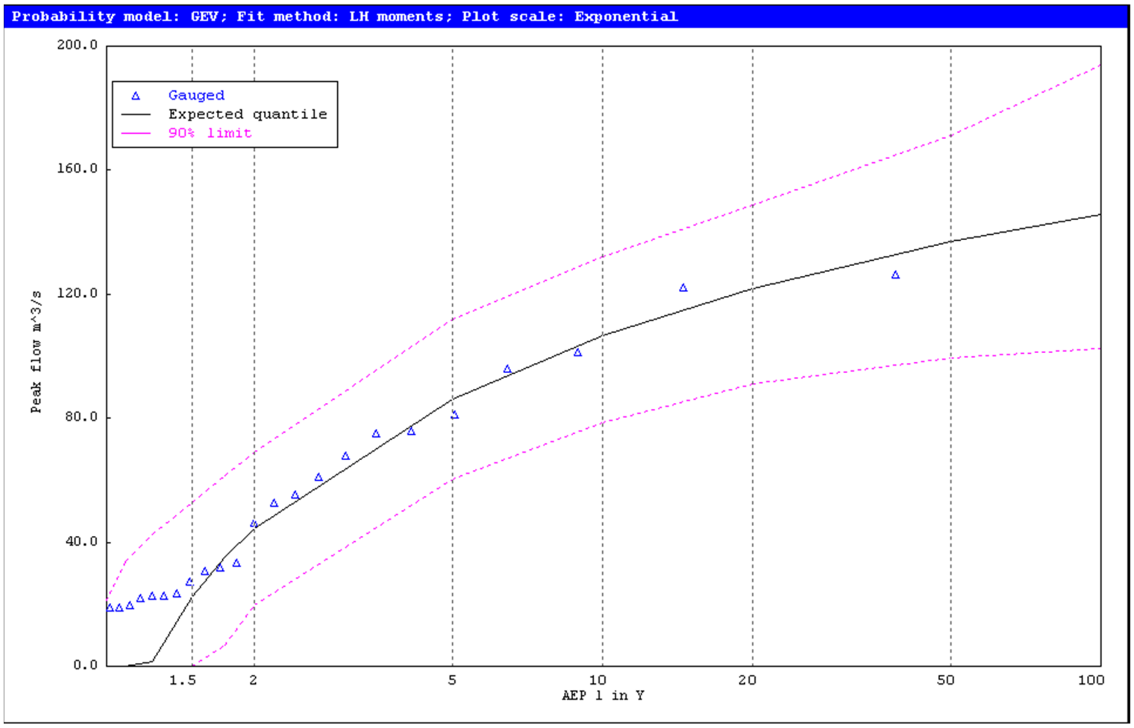


Figure A-3: #1012 Peak Rivulet, 3.5km Upstream Esperence River Flood Frequency Curve

Title: #1012 Peak Rivulet 3.5km Upstream Esperence River
Optimized L moment shift = 4

L moment	Value
1	59.300
2	8.046
3	-1.888
4	-3.523

GEV Fit Results

Parameter	LH	Mean	Std dev	Correlation		
tau	27.295	25.831	16.095	1.000		
a	49.727	55.463	26.285	-0.611	1.000	
k	0.326	0.367	0.308	-0.346	0.863	1.000

AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-71.3	-408.6	21.3	1.53
1.10	-23.0	-157.8	33.9	0.87
1.25	1.7	-70.1	42.6	0.48
1.50	22.5	-17.0	52.8	0.09
1.75	35.3	6.8	61.3	-0.17
2.00	44.5	19.6	68.9	-0.37
5.00	86.3	60.5	112.0	-1.50
10.00	106.6	78.5	132.1	-2.25
20.00	121.9	90.9	148.6	-2.97
50.00	137.1	99.6	171.3	-3.90

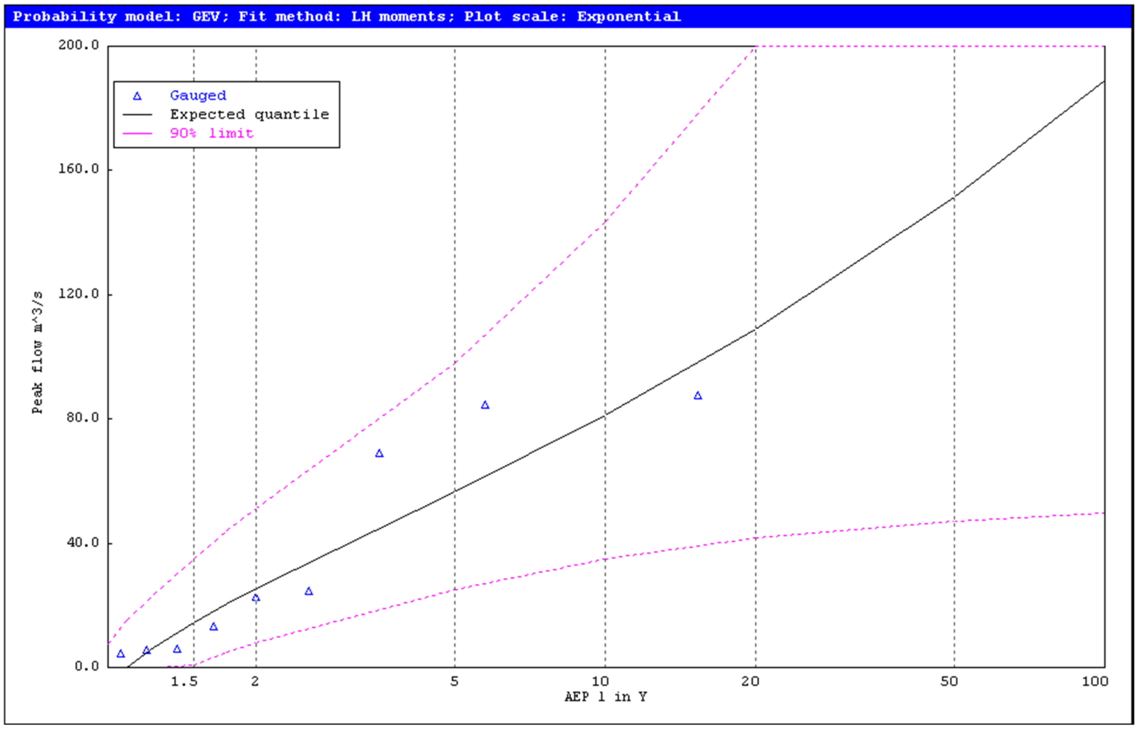


Figure A-4: #4210 Jordan River, Bridgewater Flood Frequency Curve

Title: #4210 Jordan River Bridgewater
Nominated L moment shift = 0

L moment	Value
1	35.404
2	19.657
3	6.002
4	-2.453

GEV Fit Results

Parameter	LH	Mean	Std dev	Correlation		
tau	16.723	17.678	9.216	1.000		
a	22.608	22.531	8.387	0.552	1.000	
k	-0.203	-0.107	0.317	0.383	0.302	1.000

AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-13.0	-46.7	7.5	1.53
1.10	-1.4	-15.5	15.3	0.87
1.25	6.5	-5.7	23.7	0.48
1.50	14.6	0.9	35.1	0.09
1.75	20.5	5.0	44.1	-0.17
2.00	25.3	8.0	51.2	-0.37
5.00	56.4	25.1	98.1	-1.50
10.00	81.2	34.8	143.5	-2.25
20.00	108.9	41.7	208.7	-2.97

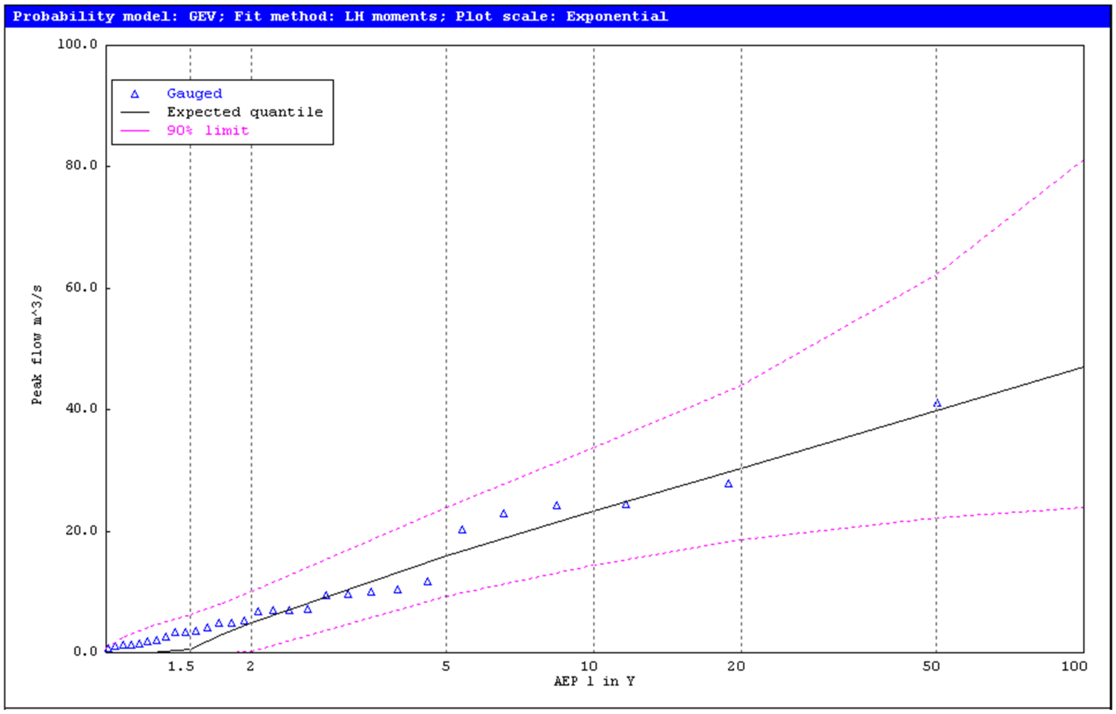


Figure A-5: #5200 Browns River, Summerlease Rd Bridge Flood Frequency Curve

Title: 5200 Browns River at Summerleas Road Bridge
Optimized L moment shift = 4

L moment	Value
1	9.443
2	4.973
3	2.028
4	0.949

GEV Fit Results

Parameter	LH	Mean	Std dev	Correlation		
tau	1.420	0.969	3.102	1.000		
a	9.505	10.828	4.973	-0.585	1.000	
k	-0.019	0.041	0.268	-0.407	0.811	1.000

AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-12.9	-57.0	0.8	1.53
1.10	-6.8	-28.7	2.7	0.87
1.25	-3.1	-16.2	4.3	0.48
1.50	0.5	-7.1	6.3	0.09
1.75	3.0	-2.5	8.2	-0.17
2.00	4.9	0.2	10.0	-0.37
5.00	15.9	9.3	24.0	-1.50
10.00	23.3	14.4	33.7	-2.25
20.00	30.4	18.6	44.1	-2.97
50.00	39.9	22.2	62.2	-3.90

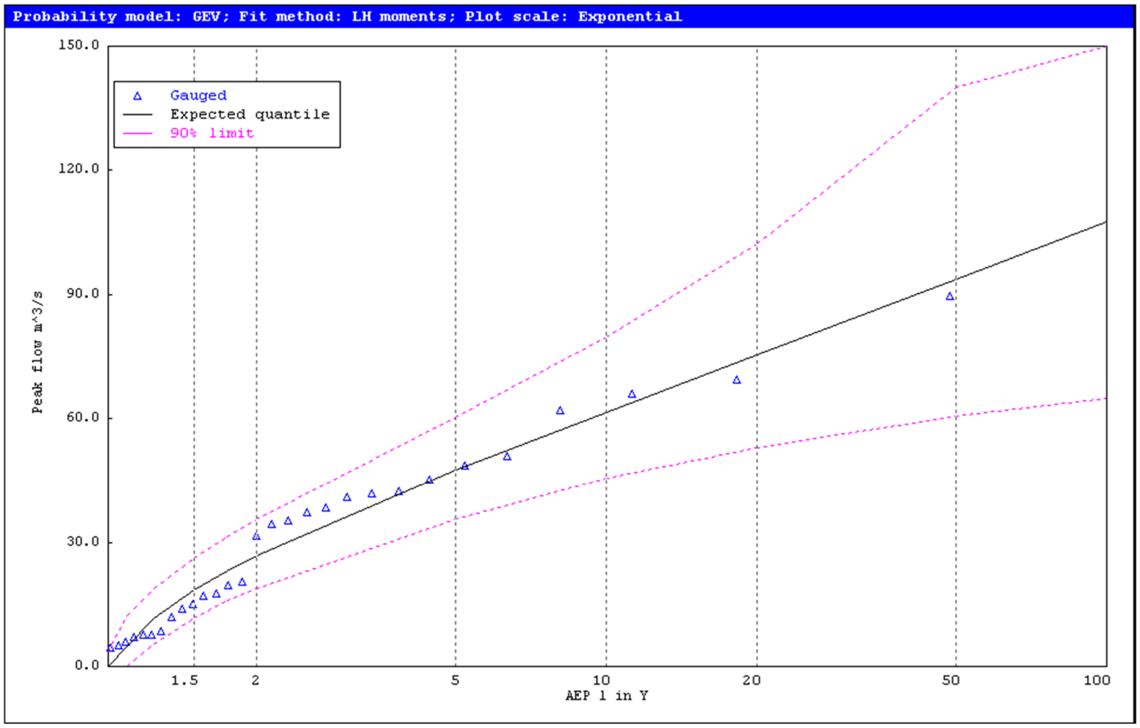


Figure A-6: #6200 Mountain River, Downstream of Grundys Creek Flood Frequency Curve

Title: 6200 Mountain River Ds Grundys Creek
Nominated L moment shift = 0

L moment	Value
1	30.919
2	12.679
3	2.369
4	0.630

GEV Fit Results

Parameter	LH	Mean	Std dev	Correlation		
tau	20.123	20.191	3.784	1.000		
a	17.799	17.686	2.972	0.409	1.000	
k	-0.029	-0.014	0.150	0.364	0.346	1.000

AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-6.5	-18.4	3.7	1.53
1.10	4.8	-2.0	12.1	0.87
1.25	11.7	5.5	18.8	0.48
1.50	18.5	11.8	26.1	0.09
1.75	23.1	15.8	31.4	-0.17
2.00	26.7	18.8	35.6	-0.37
5.00	47.4	35.6	60.4	-1.50
10.00	61.5	45.5	79.6	-2.25
20.00	75.3	53.0	102.2	-2.97
50.00	93.6	60.5	140.1	-3.90

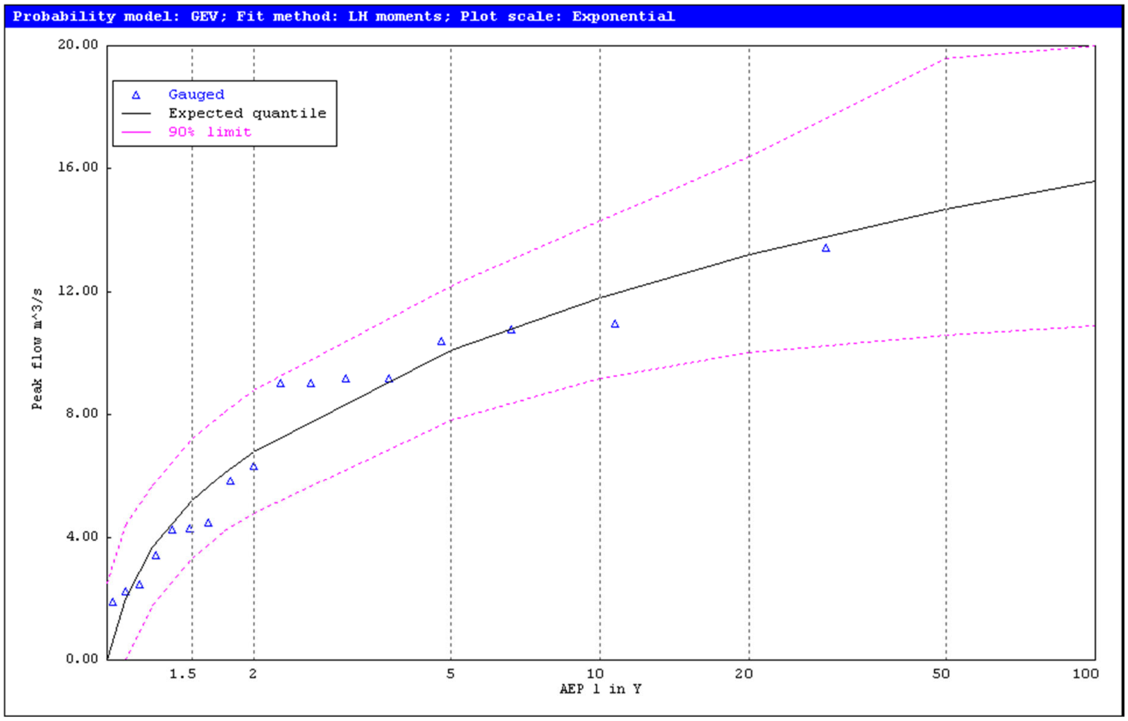


Figure A-7: #6202- Rileys Creek, Upstream Dam Flood Frequency Curve

Title: 6202- Rileys Creek Upstream Dam
Nominated L moment shift = 0

L moment	Value
1	6.897
2	2.093
3	0.059
4	-0.068

GEV Fit Results

Parameter	LH	Mean	Std dev	Correlation		
tau	5.512	5.539	0.993	1.000		
a	3.596	3.552	0.727	0.097	1.000	
k	0.234	0.245	0.205	0.415	0.488	1.000

AEP 1 in Y	Quantile	5%	95%	Gumbel reduced variate
1.01	-1.1	-5.8	2.5	1.53
1.10	2.0	-0.3	4.4	0.87
1.25	3.7	1.8	5.7	0.48
1.50	5.2	3.3	7.2	0.09
1.75	6.1	4.2	8.1	-0.17
2.00	6.8	4.8	8.8	-0.37
5.00	10.1	7.8	12.2	-1.50
10.00	11.8	9.2	14.3	-2.25
20.00	13.2	10.0	16.4	-2.97
50.00	14.7	10.6	19.6	-3.90

APPENDIX B RAINFALL DEPTHS

Table B.1 - Rainfall Depths for Humphreys Rivulet and Barossa Creek for selected exceedance probabilities and durations

Areal rainfall depth (mm)

Duration			Annual Exceedance Probability (1 in X)			
(hrs)	(mins)		20		100	
	Burst	Complete	Burst	Complete	Burst	Complete
0.5	30	105	17.4	24.0	24.4	28.7
1.0	60	135	22.9	29.5	30.9	35.2
1.5	90	165	27.0	32.0	35.8	41.3
2.0	120	195	30.5	37.6	40.0	45.5
3.0	180	405	36.7	45.0	47.7	68.6
6.0	360	585	51.3	65.3	66.7	94.7
9.0	540	990	62.6	76.7	82.2	104
12	720	1,170	71.7	85.9	95.2	111
24	1,440	2,340	95.7	108.9	130	141

Table B.2 - Rainfall Depths for Humphreys Rivulet and Barossa Creek for PMF and selected durations

Areal rainfall depth (mm)

Duration			Annual Exceedance Probability (1 in X)	
(hrs)	(mins)		10,000,000	
	Burst	Complete	Burst	Complete
0.5	30	75	145.0	149.5
1.0	60	150	220.0	226.8
1.5	90	225	270.0	278.4
2.0	120	300	310.0	319.6
3.0	180	315	370.0	381.5
6.0	360	630	485.0	500.0
9.0	540	945	583.8	753.2
12	720	1,260	682.5	880.6
24	1,440	3,420	880.0	1017

Table B.3 - Rainfall Depths for Humphreys Rivulet and Barossa Creek for selected complete storm exceedance probabilities and durations with an intensity climate change factor of 1.24 applied

Areal rainfall depth (mm)

Duration			Annual Exceedance Probability (1 in X)	
(hrs)	(mins)		20	100
	Burst	Complete	Complete	Complete
0.5	30	105	29.8	35.6
1.0	60	135	36.6	43.6
1.5	90	165	39.7	51.2
2.0	120	195	46.6	56.4
3.0	180	405	55.8	85.1
6.0	360	585	81.0	117
9.0	540	990	95.1	129
12	720	1,170	107	138
24	1,440	2,340	135	175

Table B.4 - Rainfall Depths for Humphreys Rivulet and Barossa Creek for PMF and selected durations with an intensity climate change factor of 1.24 applied

Areal rainfall depth (mm)

Duration			Annual Exceedance Probability (1 in X)
(hrs)	(mins)		10,000,000
	Burst	Complete	Complete
0.5	30	75	185.4
1.0	60	150	281.2
1.5	90	225	345.2
2.0	120	300	396.3
3.0	180	315	473.1
6.0	360	630	620.0
9.0	540	945	934.0
12	720	1,260	1092
24	1,440	3,420	1261

APPENDIX C HYDROLOGY PROCEDURE FOR MODELLING THE DAMS

C.1. Knights Creek Modelling

C.1.1. Elevation Discharge Relationship

Table C-7-1 – Knights Creek Spillway Elevation Discharge Relationship (SMEC 2017)

Elevation (mAHD)	Discharge (m^3/s)		
	Spillway	Embankment Overtopping	Total
189.60	0	0	0
189.78	3	0	3
189.99	10	0	10
190.19	20	0	20
190.39	33	0	33
190.57	47	0	47
190.85	63	0	63
191.47	80	0	80
192.14	100	0	100
192.66	120	0	120
193.02	135	0	132
193.52	148	0	148
193.61	161	9	170
193.80	170	47	217
194.31	195	225	420
194.63	211	372	583
194.89	225	510	735
195.33	250	777	1,027
195.82	280	1,120	1,400
196.10	310	1,323	1,633
196.48	345	1,626	1,971
196.82	380	1,920	2,300
197.15	420	2,216	2,636
197.45	460	2,495	2,955

Table C-7-2 – Knights Creek Low-Level Outlet Elevation Discharge Relationship (SMEC 2017)

Elevation (mAHD)	Discharge (m^3/s)
167	0
168	0.083
169	0.118
170	0.145
171	0.167
172	0.187
173	0.204
174	0.221
175	0.236
176	0.250
177	0.264
178	0.277
179	0.289
180	0.470
181	0.488
182	0.505
183	0.522
184	0.538
185	0.553
186	0.568
187	0.583
188	0.596
189	0.610
190	0.619

C.1.2. Elevation Storage Relationship

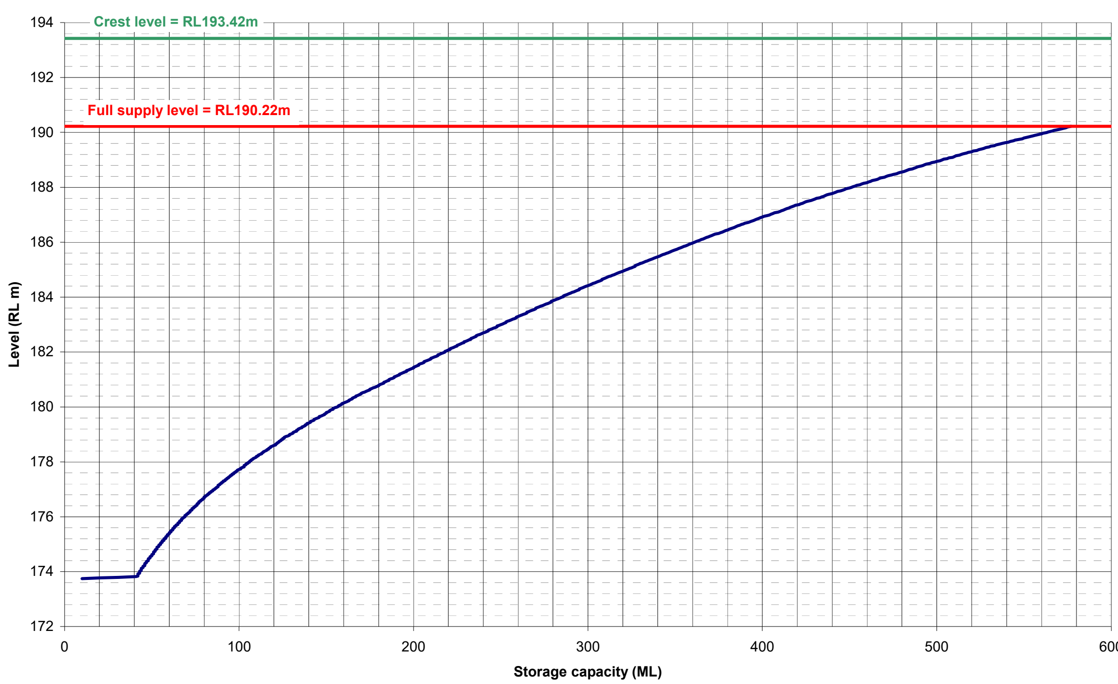


Figure C-7-1: Knights Creek Elevation Storage Relationship (SMEC 2017)

C.1.3. Flood Frequency Curve

Table C-7-3 - Results for Knights Creek (SMEC 2017)

AEP (1 IN X)	Peak Rain Depth (mm)	Peak Storage Elevation (m)	Inflow (m^3/s)	Peak Outflow (m^3/s)	Critical Duration (hrs)
2	31.6	189.9	9.9	8.4	9
5	43.9	190.0	14.2	12.6	9
10	52.0	190.1	17.0	15.2	9
20	59.8	190.1	19.8	17.7	9
50	69.7	190.2	23.8	20.8	9
100	77.2	190.2	26.9	23.4	9
1,000	160	190.7	67.3	55.9	12
10,000	239	191.7	120.7	88.3	12
50,000	314	192.8	177.2	126.9	12
100,000	352	193.3	207.0	146.8	12
200,000	102	193.4	241.3	171.7	1
500,000	122	193.7	295.1	249.4	1
1,000,000	139	193.8	342.8	304.8	1
10,000,000	185	194.3	554.3	540.2	0.75

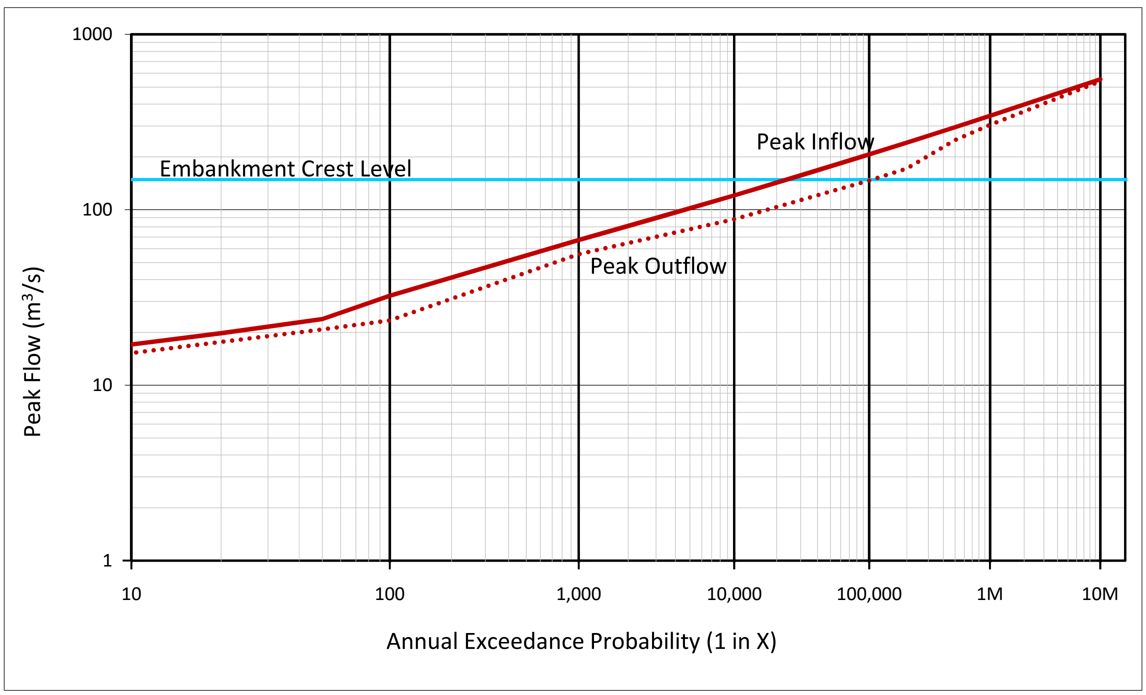


Figure C-7-2: Knights Creek Dam Inflow and Outflow flood frequency Curves (SMEC 2017)

C.2. Limekiln Gully Modelling

C.2.2. Elevation Discharge Relationship

Figure C-7-3: Limekiln Gully Spillway Rating Curve, (SMEC 2017)

Table C-7-4 – Limekiln Gully Low-Level Outlet Elevation Discharge Relationship (SMEC 2017)

Elevation (mAHD)	Discharge (m^3/s)
144.1	0
145	0.178
146	0.259
147	0.320
148	0.371
149	0.416
150	0.456
151	0.493
152	0.528
153	0.560
154	0.591
155	0.620
156	0.648
157	0.674
158	0.700
159	0.725
160	0.749
161	0.772
162	0.794
163	0.816
164	0.838
165	0.856
166	0.877
166.4	0.885

C.2.3. Elevation Storage Relationship

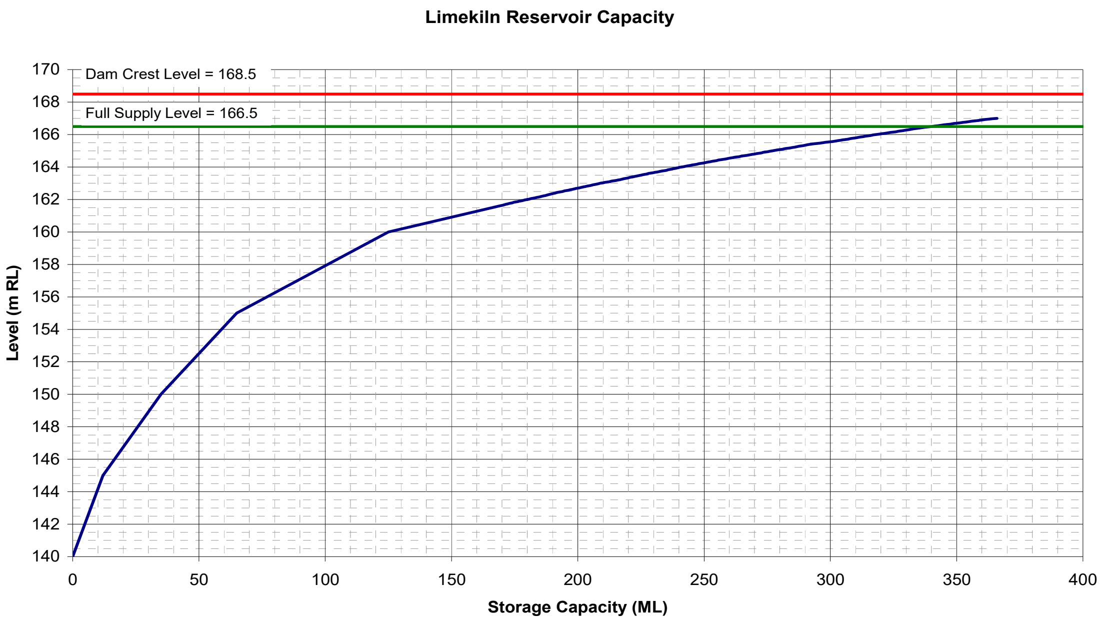


Figure C-7-4: Limekiln Gully Elevation Storage Relationship (SMEC 2017)

C.2.4. Flood Frequency Curve

Table C-7-5 - Results for Limekiln Gully (SMEC 2017)

AEP (1 IN X)	Peak Rain Depth (mm)	Peak Storage Elevation (m)	Inflow (m^3/s)	Peak Outflow (m^3/s)	Critical Duration (hrs)
2	35.8	166.6	1.8	0.7	12
5	43.9	166.7	2.8	1.0	9
10	52.0	166.8	3.8	1.3	9
20	59.8	166.8	5.0	1.5	9
50	69.7	166.8	6.4	1.9	9
100	77.2	166.9	9.7	2.1	9
1,000	233	167.3	18.6	6.6	12
10,000	239	167.7	32.3	11.8	12
50,000	314	168.1	46.1	17.0	12
100,000	352	168.2	53.1	19.6	12
200,000	392	168.4	61.6	22.4	12
500,000	451	168.6	74.7	26.3	12
1,000,000	499	168.8	86.0	29.5	12
10,000,000	270	169.7	132.7	47.5	1.5

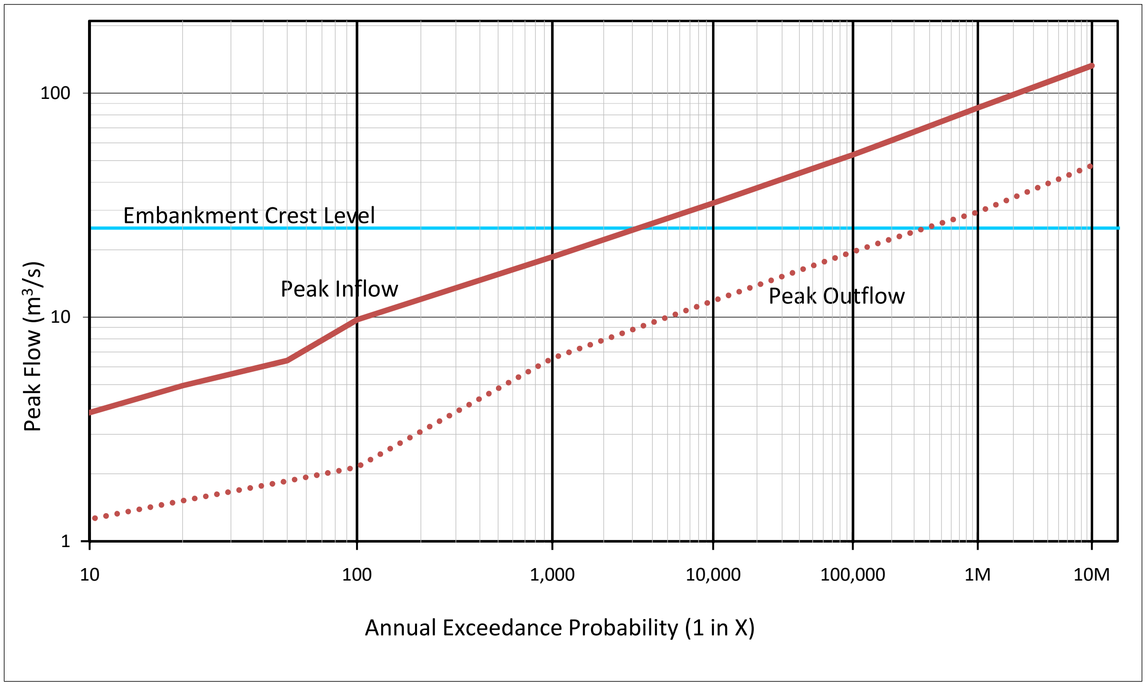


Figure C-7-5: Limekiln Gully Inflow and Outflow flood frequency Curves (SMEC 2017)

C.3. Tolosa Modelling

C.3.2. Elevation Discharge Relationship

Table C-7-6 – Tolosa Reservoir Elevation Discharge Relationship (SMEC 2017)

Elevation (mAHD)	Discharge (m^3/s)
107.00	0.0
107.87	1.4
107.92	2.88
107.97	6.69
108.02	12.89
108.07	22.2
108.12	35.06
108.17	51.88
108.21	72.32
108.26	96.51
108.31	123.48
108.36	153.14
108.40	185.3
108.45	219.88
108.50	257.29

C.3.3. Elevation Storage Relationship

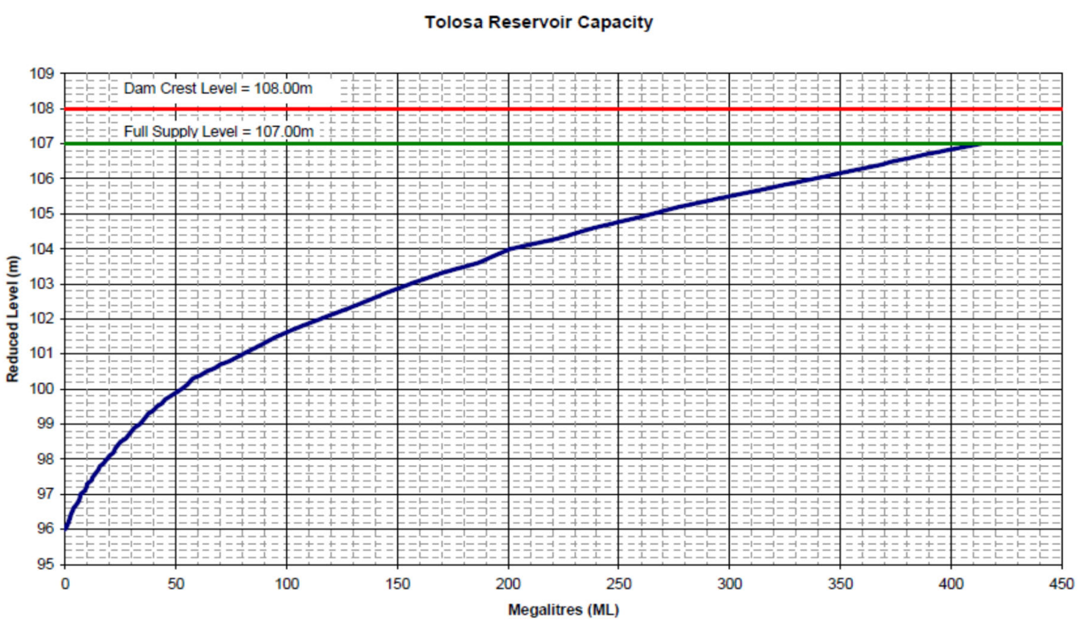


Figure C-7-6: Tolosa Reservoir Elevation Storage Relationship (SMEC 2017)

C.3.4. Flood Frequency Curve

Table C-7-7 - Tolosa flood frequency relationship (SMEC 2017)

AEP (1 IN X)	Peak Rain Depth (mm)	Peak Storage Elevation (m)	Inflow (m^3/s)	Peak Outflow (m^3/s)	Outflow Critical Duration (hrs)
2	52.1	107.03	0.46	0.6	36
5	74.3	107.05	0.68	0.8	36
10	89.8	107.06	0.83	0.10	36
20	105	107.07	0.96	0.11	36
50	127	107.09	1.14	0.14	36
100	143	107.10	1.28	0.15	36

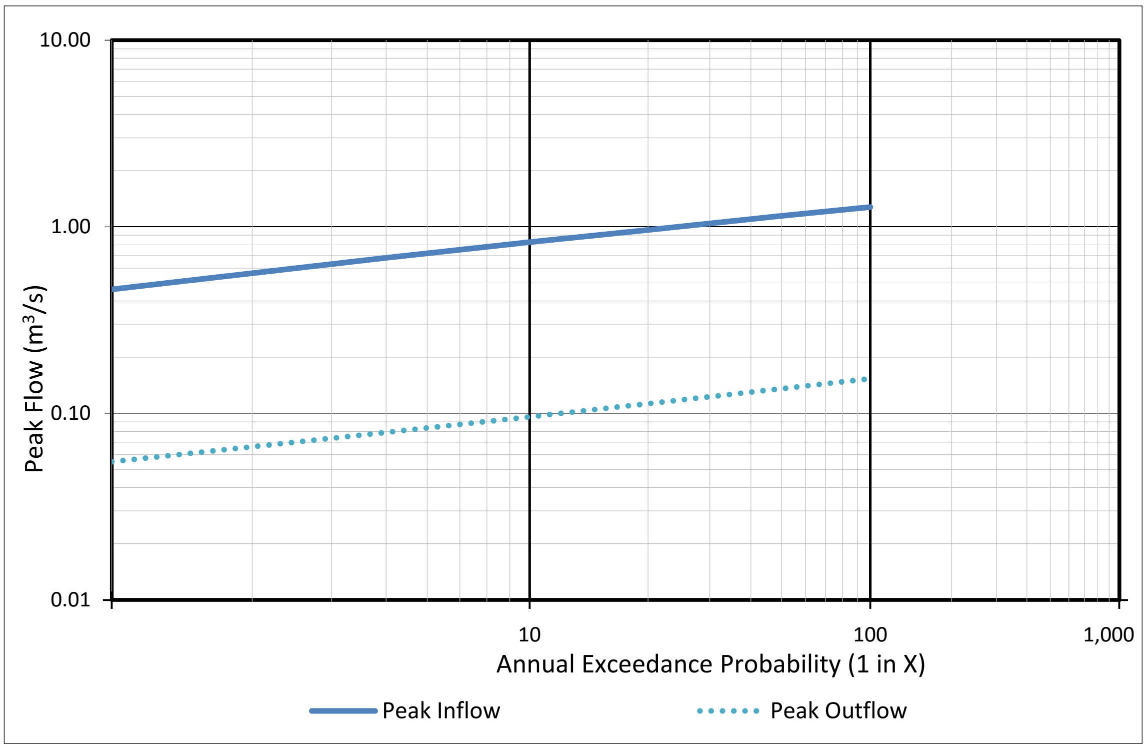
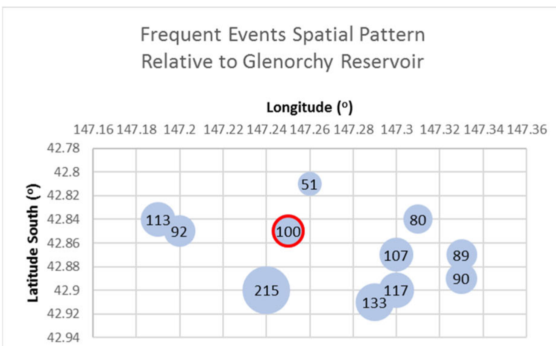
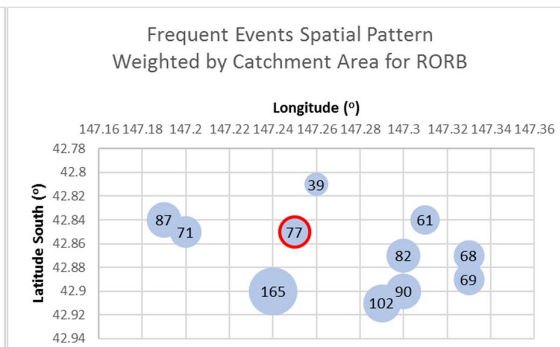


Figure C-7-7: Tolosa Inflow and Outflow flood frequency Curves (SMEC 2017)

APPENDIX D SPATIAL PATTERN ASSESSMENT

Gauge Name	Longitude (°)	Latitude South (°)
Glenorchy Reservoir	147.25	42.85
Glenorchy - Murrayfield (closed)	147.27	42.84
Moonah East (closed)	147.30	42.85
Hobart - Ellerslie Rd	147.33	42.89
Hobart Botanical Gardens	147.33	42.87
Lenah Valley - Augusta Rd (closed)	147.30	42.87
Hobart - Waterworks Res	147.29	42.91
Kunanyi Mt Wellington Pinnacle	147.24	42.90
Collinsvale	147.19	42.84
Berriedale - Moorilla Estate	147.26	42.81
South Hobart - Hillborough Rd	147.30	42.90
Rosetta (closed)	147.25	42.83
Lutana - Bowen Rd (closed)	147.31	42.84
Collinsvale (closed)	147.20	42.85





See Figure 3-7 for the names of the gauges.

The gauge circled red is the reference Glenorchy Reservoir gauge

The black text (and size of the blue circle) is the spatial pattern (on the left) relative to Glenorchy Reservoir, the percentage of the Glenorchy Reservoir rainfall to apply at that gauge; (on the right) the same percentage weighted by sub-catchment area summing to 100 for RORB.

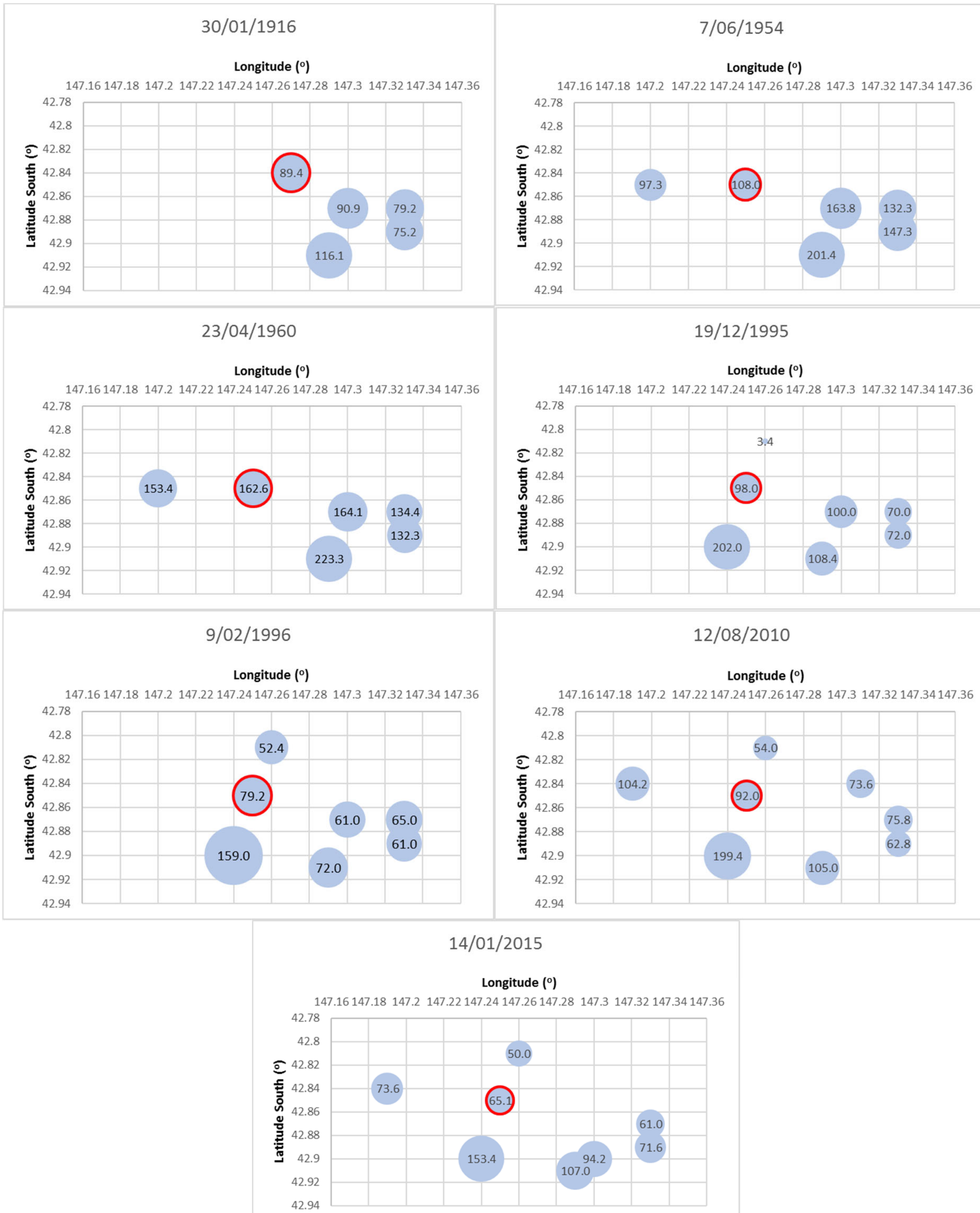
This (left image) value is calculated by:

6. Isolate the rainfall events that show orographic related spatial variability (the seven events on the next page)
7. For each gauge calculate the ratio of rainfall depth to the Glenorchy gauge rainfall depth as a percentage (shown as the size of blue circles on following pages)
8. Calculate the (mean) average of those step 2 ratios for each gauge.

For the (right image) value:

9. The value from step 3 is adjusted by the formula: sum(SubArea x pattern)/TotalArea=100

Events with obvious orographic spatial gradient:



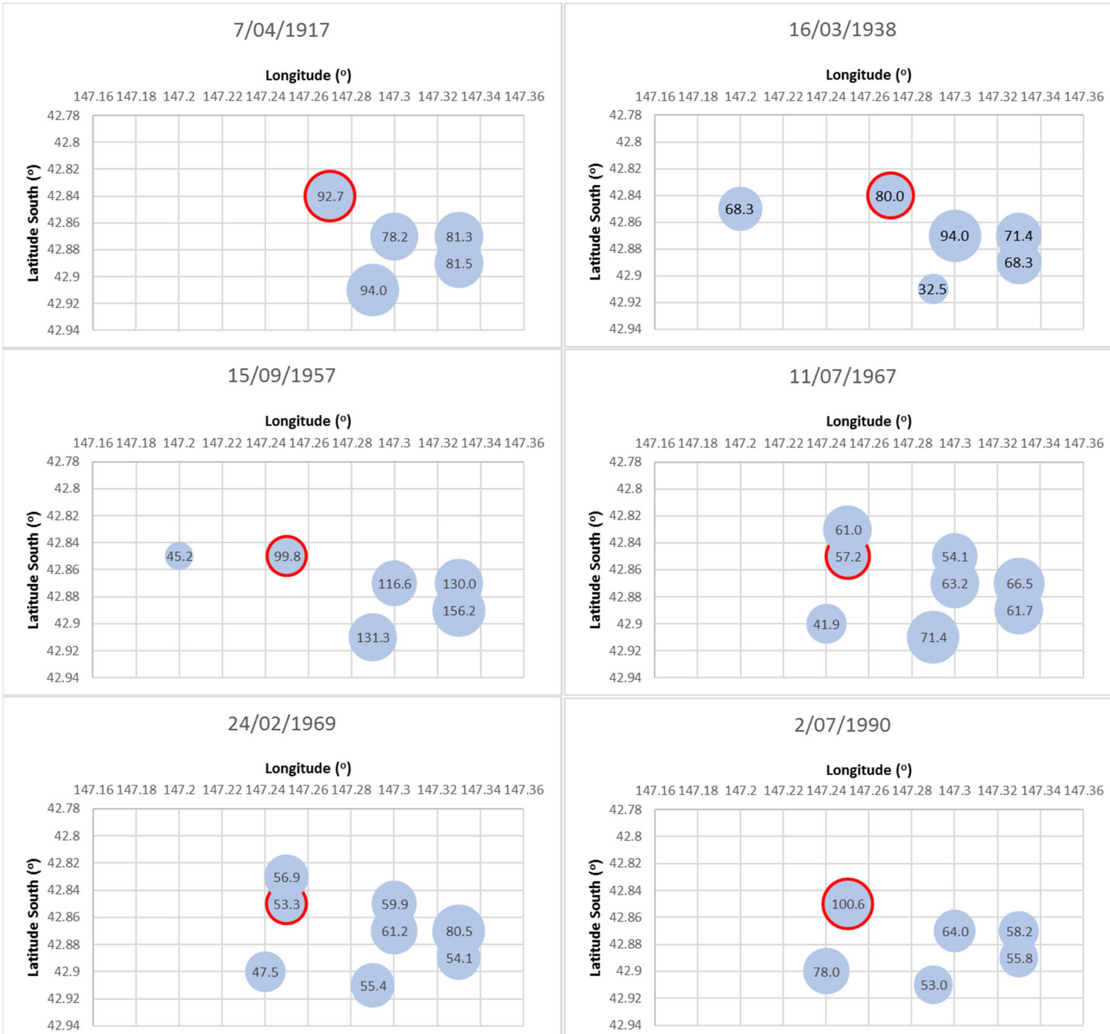
The gauge circled red is the reference Glenorchy gauge (at Reservoir/Murrayfield depending on year)

The black text is the daily total rainfall depth at that gauge.

The size of the blue circle shows the ratio of that gauge rainfall depth to the Glenorchy reference gauge.

See Figure 3-7 for the names of the gauges.

Events with neutral or reverse orographic gradient:

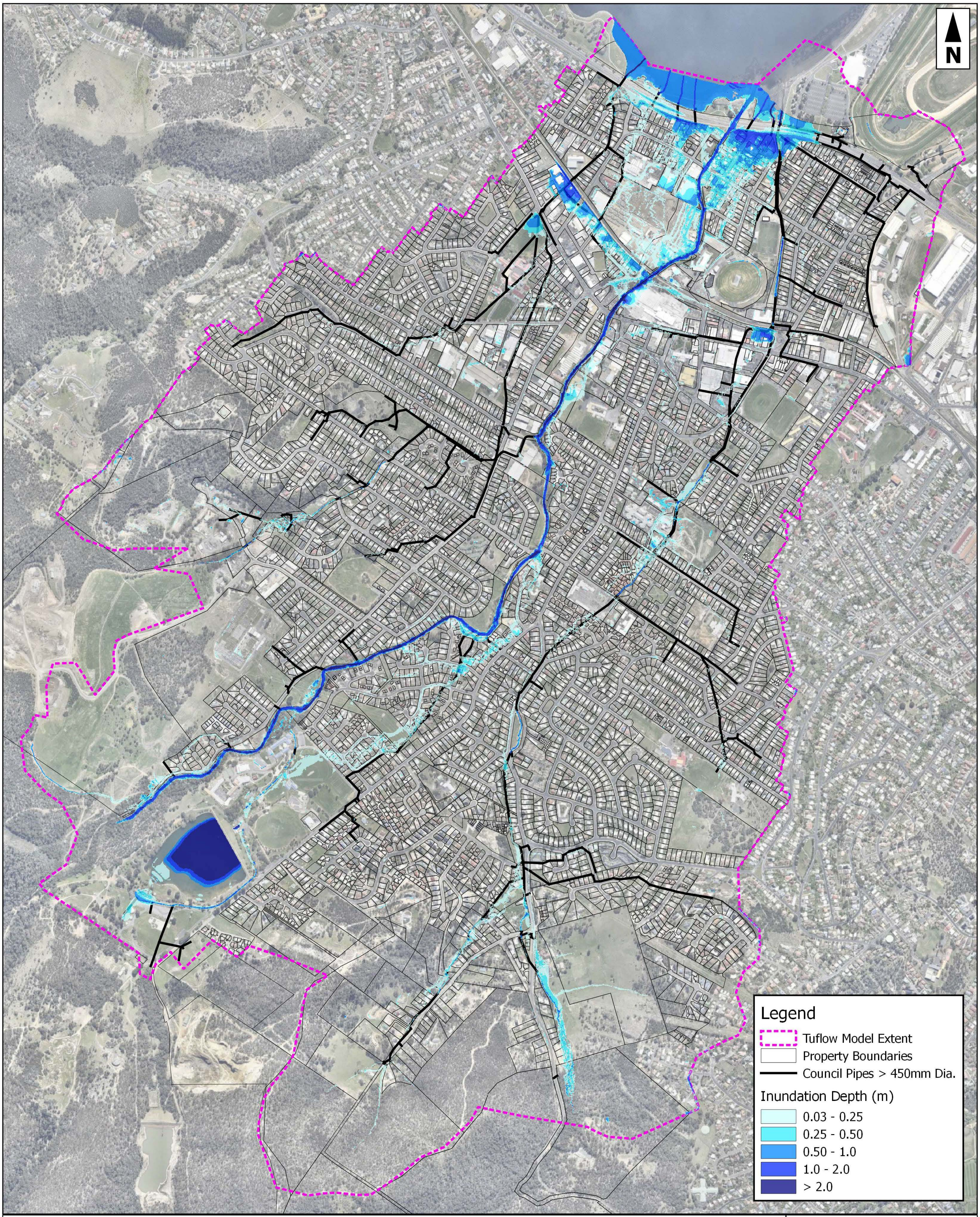


The gauge circled red is the reference Glenorchy gauge (at Reservoir/Murrayfield depending on year)

The black text is the daily total rainfall depth at that gauge.

The size of the blue circle shows the ratio of that gauge rainfall depth to the Glenorchy reference gauge.
See Figure 3-7 for the names of the gauges.

APPENDIX E INUNDATION MAPS (DEPTH AND DV)



REVISION: A
STATUS: FINAL
SOURCES: © NEARMAP 2018
AUTHOR: H. Nguyen-Mallen
DATE: 7/09/2018
FIGURE: 30041618-E-cal1

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1996 CALIBRATION EVENT - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
PAGE SIZE: A3
SCALE: 1:13,000

0 200 400 600 m

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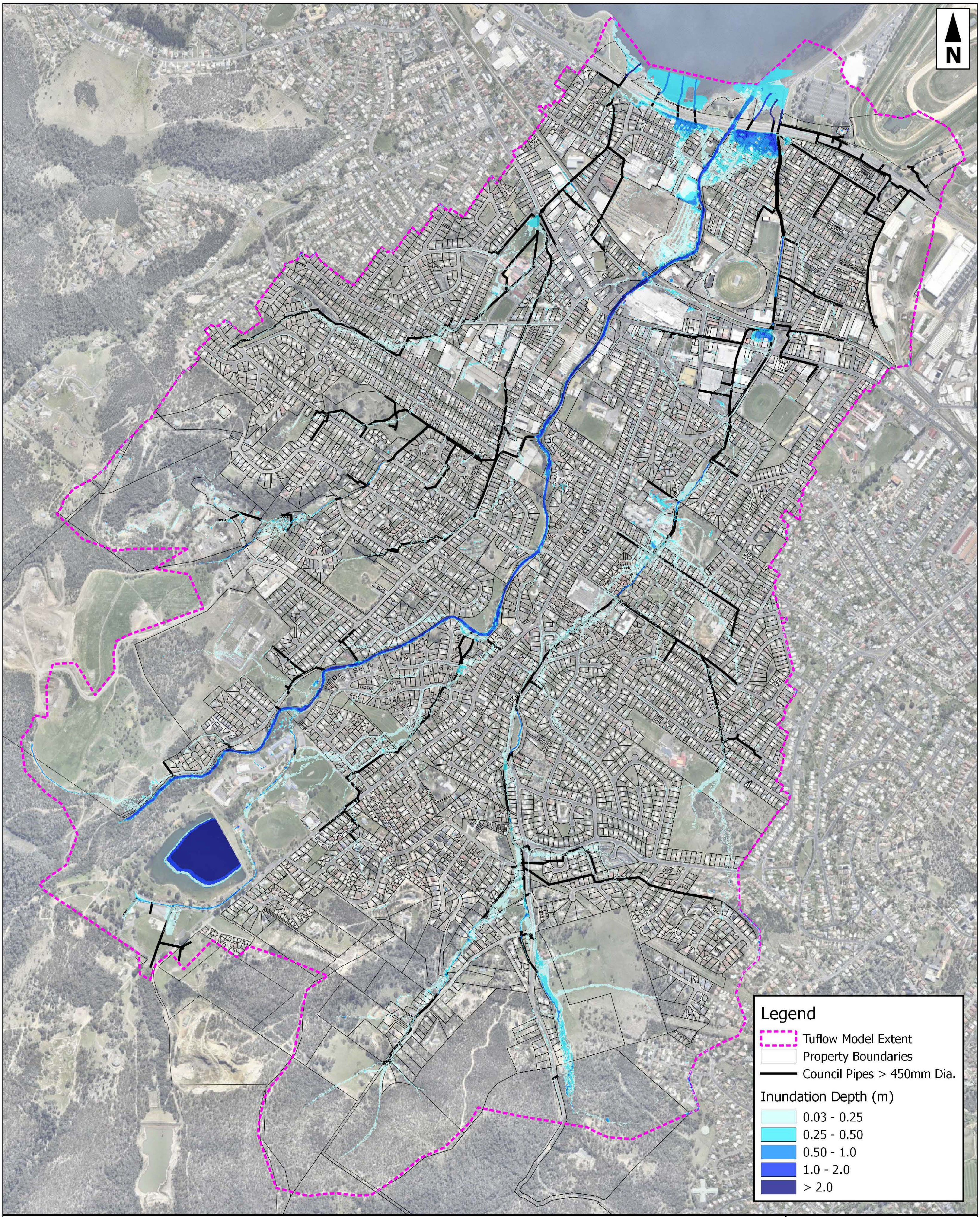

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AUTHOR:	H. Nguyen-Mallen	DATE:	7/09/2018	FIGURE:	30041618-E-ex1

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 20 AEP EXISTING - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

0 200 400 600 m

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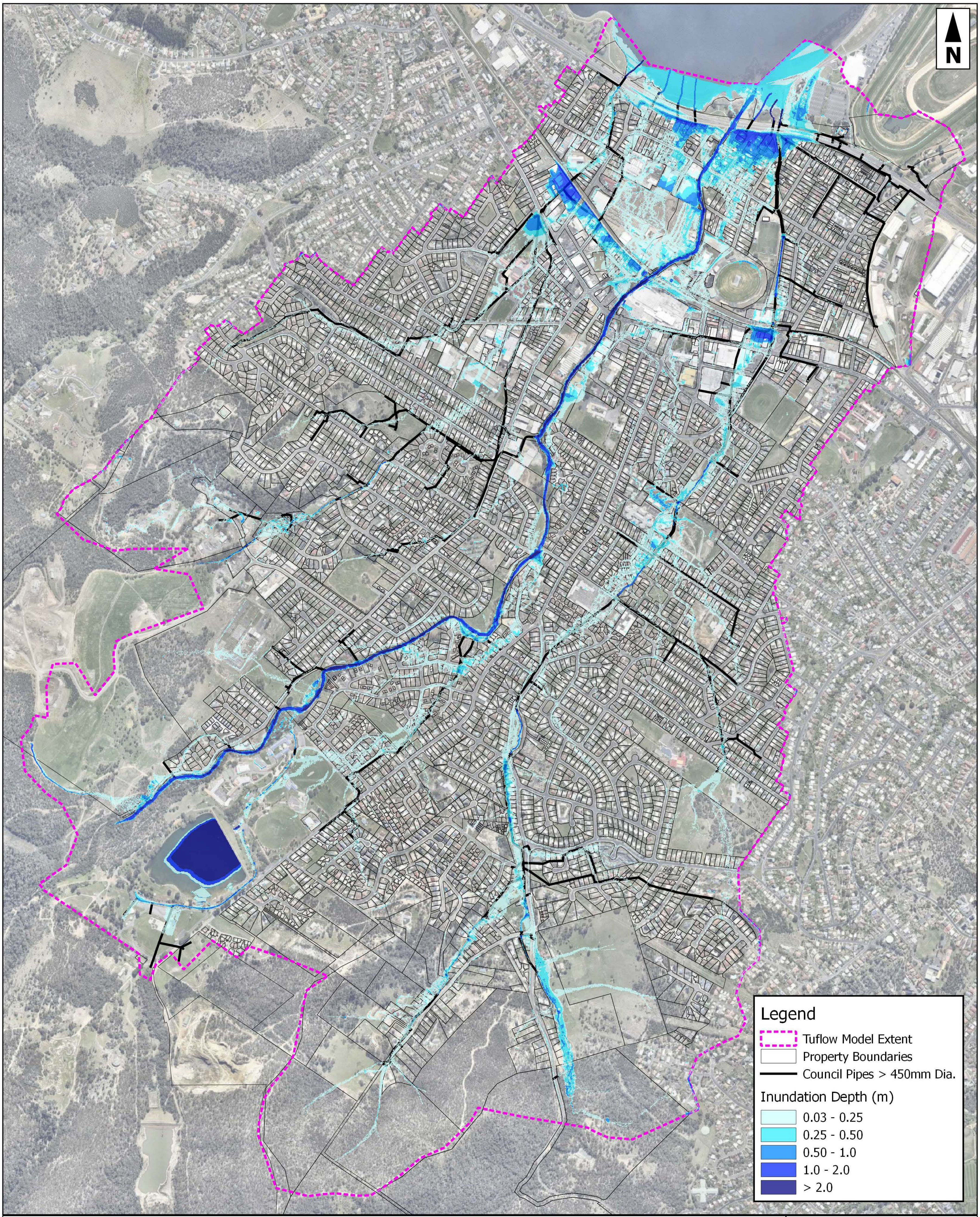

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AUTHOR:	H. Nguyen-Mallen	DATE:	7/09/2018	FIGURE:	30041618-E-ex3

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP EXISTING - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

0 200 400 600 m

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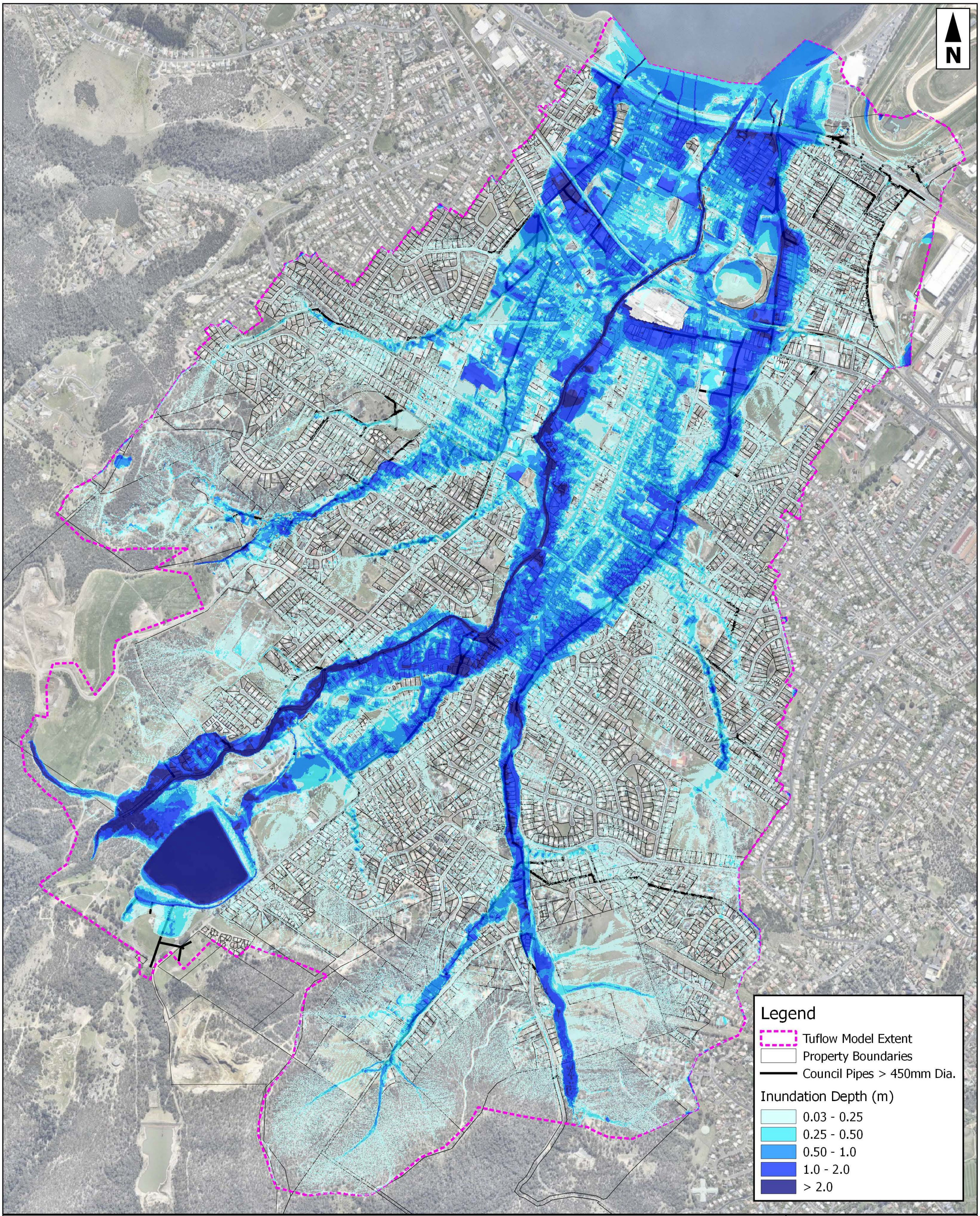

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AUTHOR: H. Nguyen-Mallen	DATE: 7/09/2018	FIGURE: 30041618-E-ex5

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: PMF EXISTING - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

0 200 400 600 m

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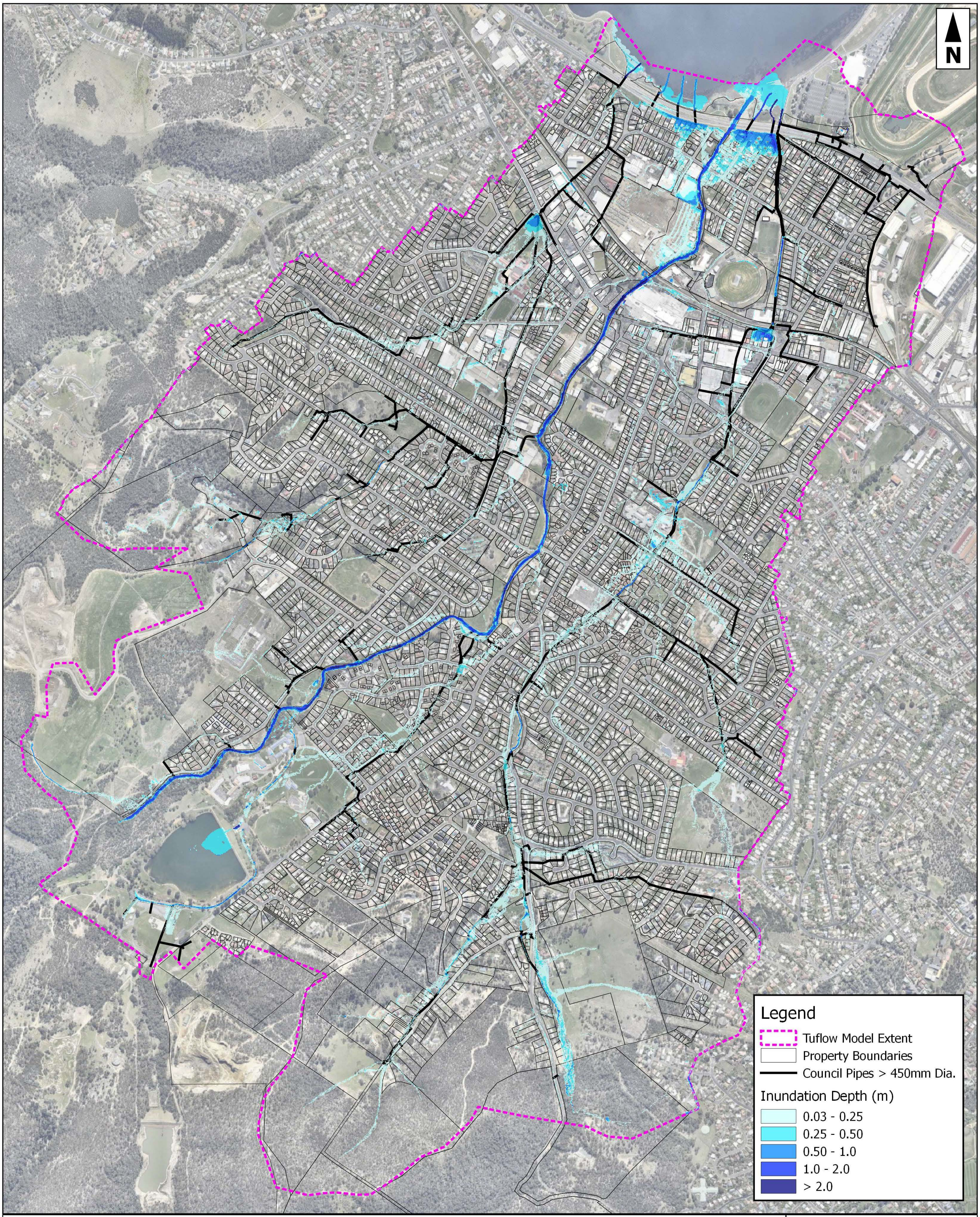

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STATUS: FINAL
SOURCES: © NEARMAP 2018
AUTHOR: H. Nguyen-Mallen
DATE: 7/09/2018
FIGURE: 30041618-E-dr1

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 20 AEP DEVELOPED - EXIST DAMS LEVEL - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
PAGE SIZE: A3
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0 200 400 600 m

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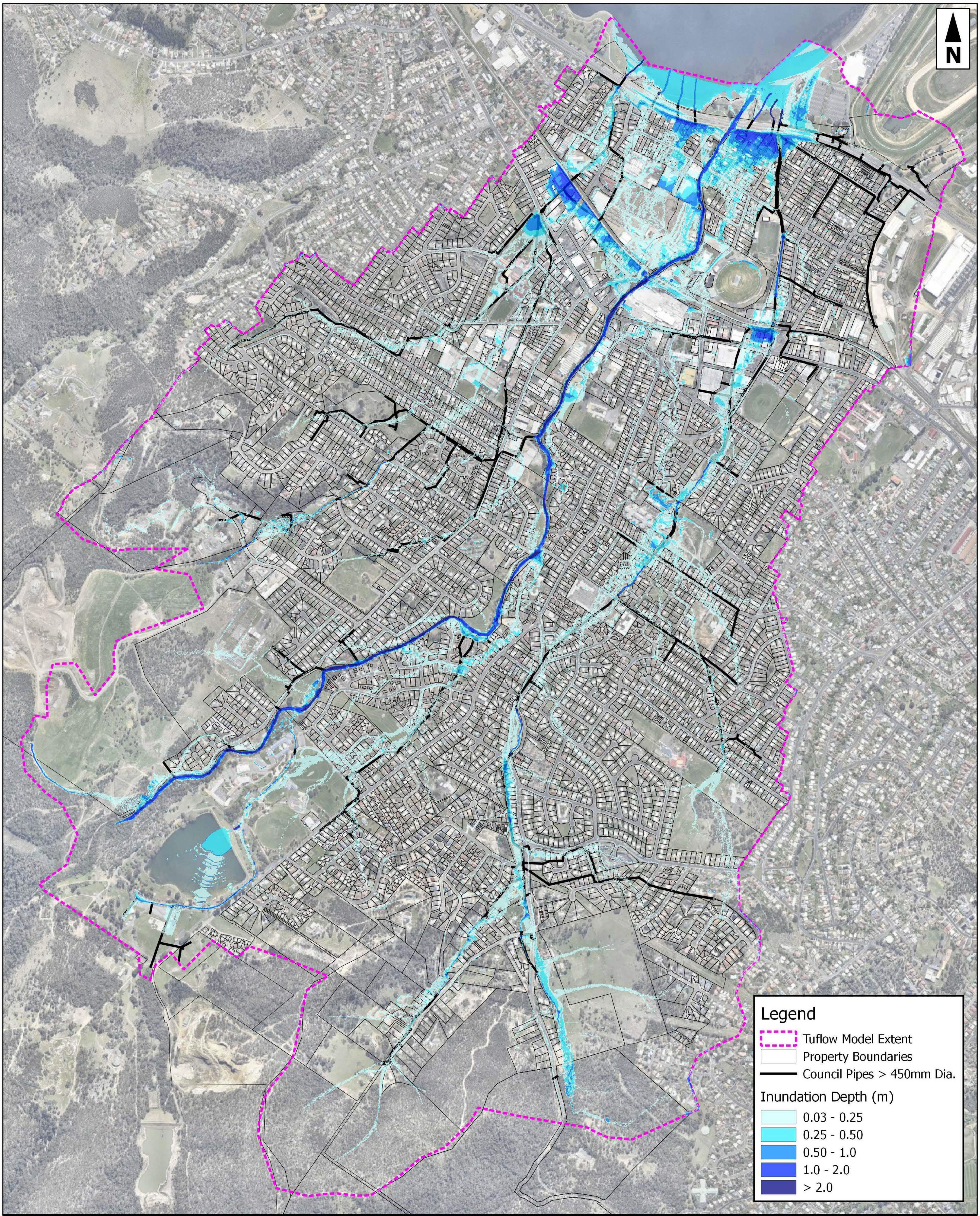

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AUTHOR: H. Nguyen-Mallen
STATUS: FINAL
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FIGURE: 30041618-E-dr3
CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP DEVELOPED - EXIST DAMS LEVEL - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
PAGE SIZE: A3
SCALE: 1:13,000
0 200 400 600 m

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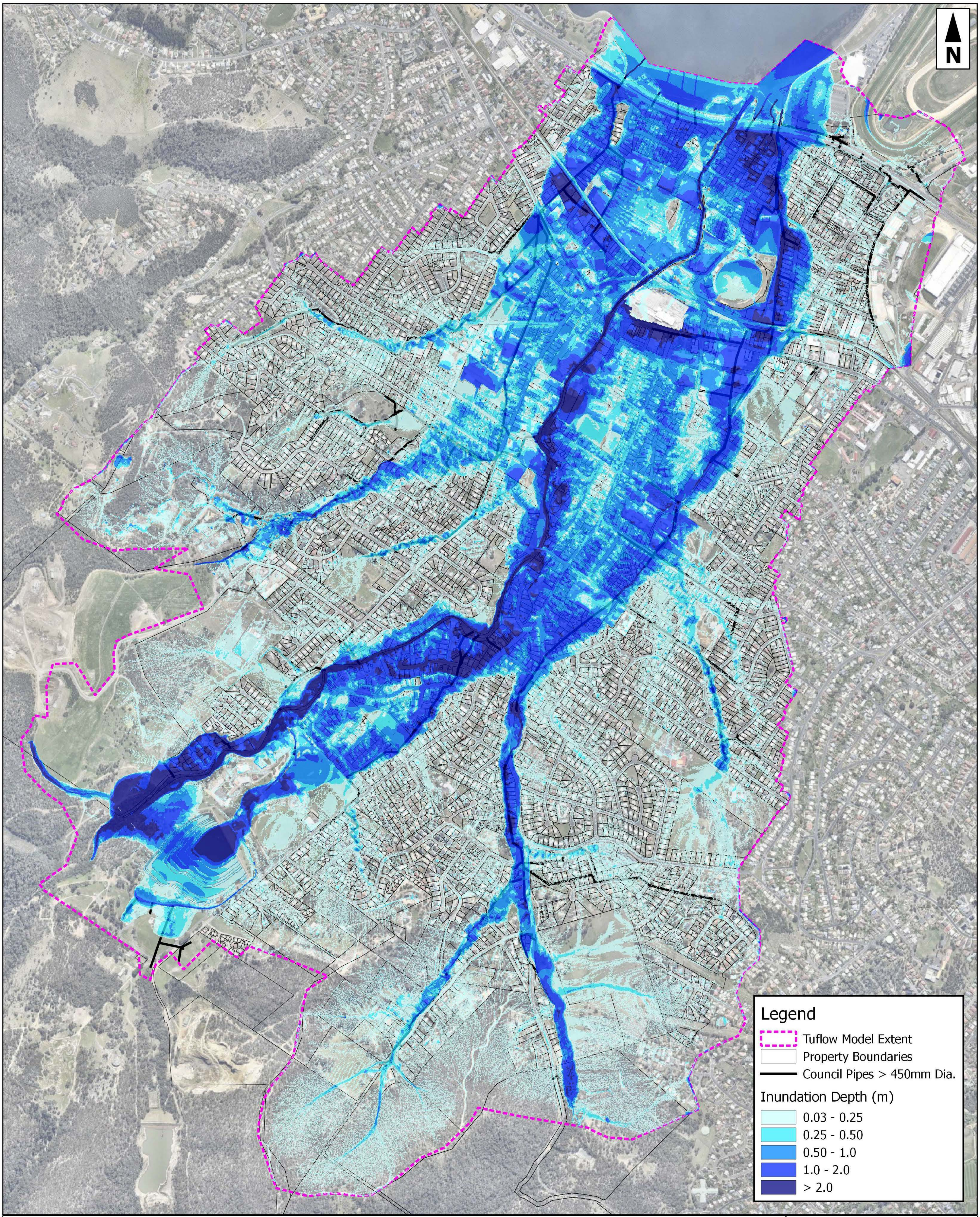
PROJECT NO: 30041618
PROJECT TITLE: GLENORCHY CBD SSMP



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AUTHOR:	H. Nguyen-Mallen	DATE:	7/09/2018	FIGURE:	30041618-E-dr5

FIGURE TITLE: PMF DEVELOPED - EXIST DAMS LEVEL - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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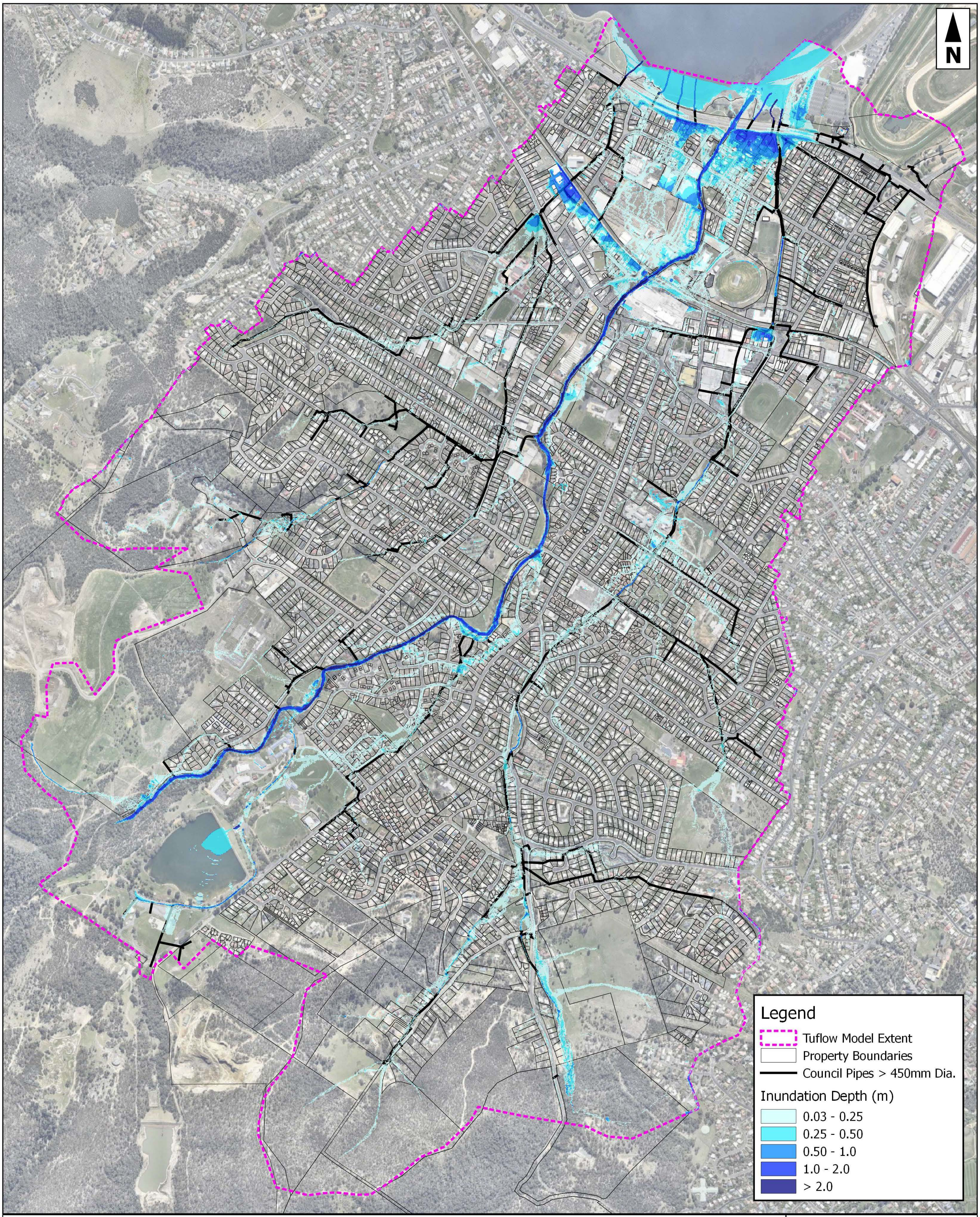

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FIGURE TITLE: 1 in 20 AEP DEVELOPED - DAMS FULL - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

0 200 400 600 m

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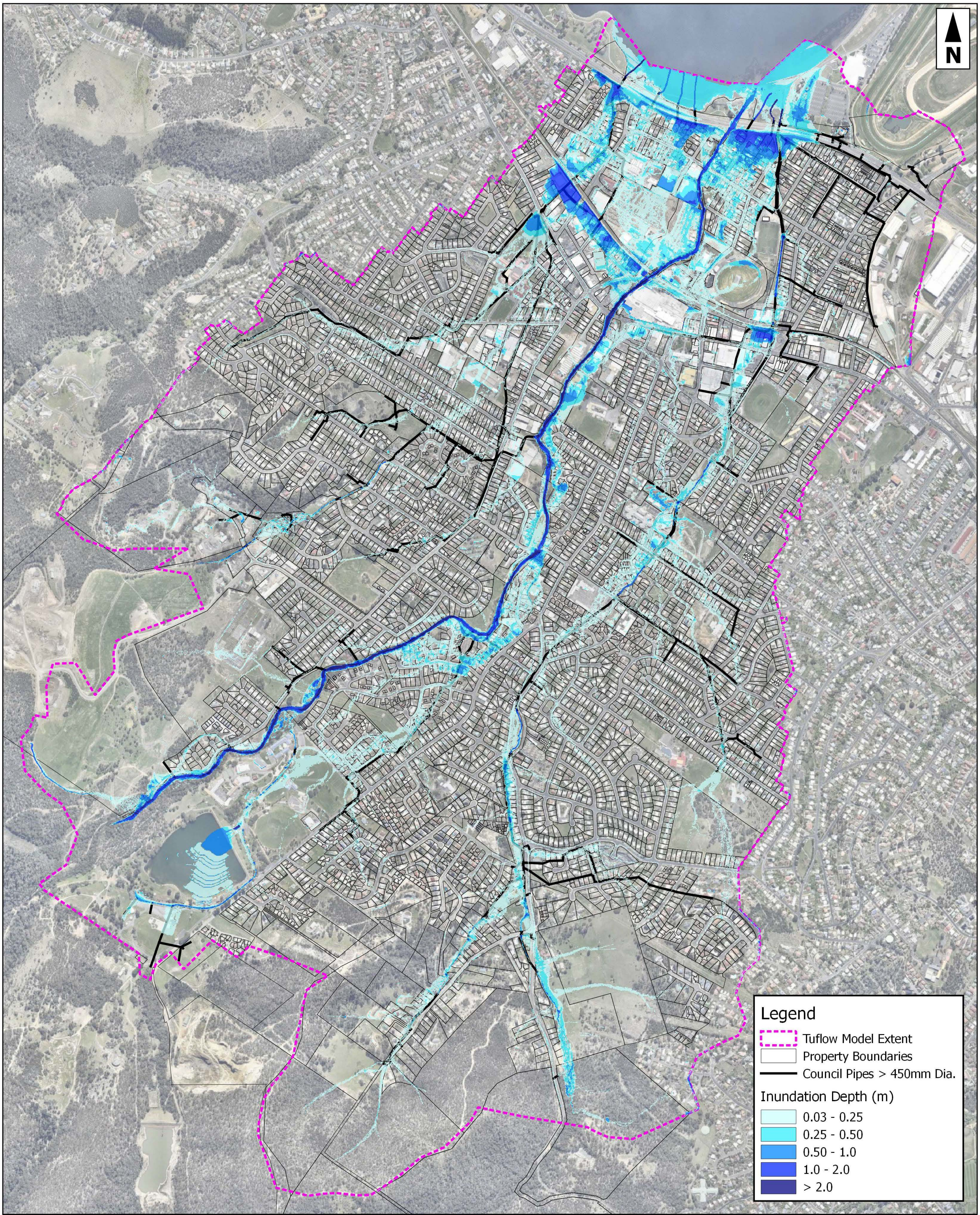

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AUTHOR: H. Nguyen-Mallen	DATE: 7/09/2018	FIGURE: 30041618-E-de3

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP DEVELOPED - DAMS FULL - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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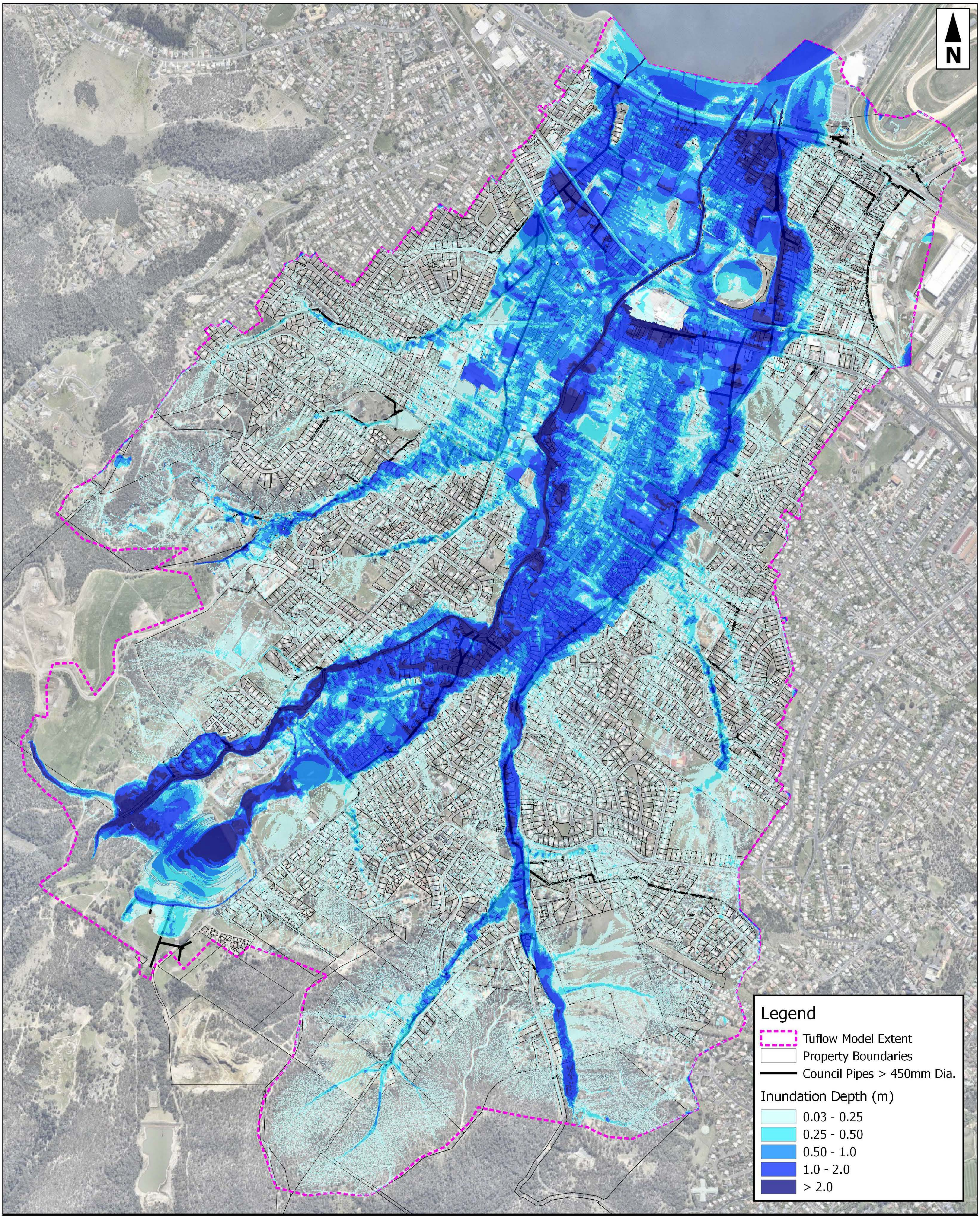

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AUTHOR: H. Nguyen-Mallen	DATE: 7/09/2018	FIGURE: 30041618-E-de5

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: PMF DEVELOPED - DAMS FULL - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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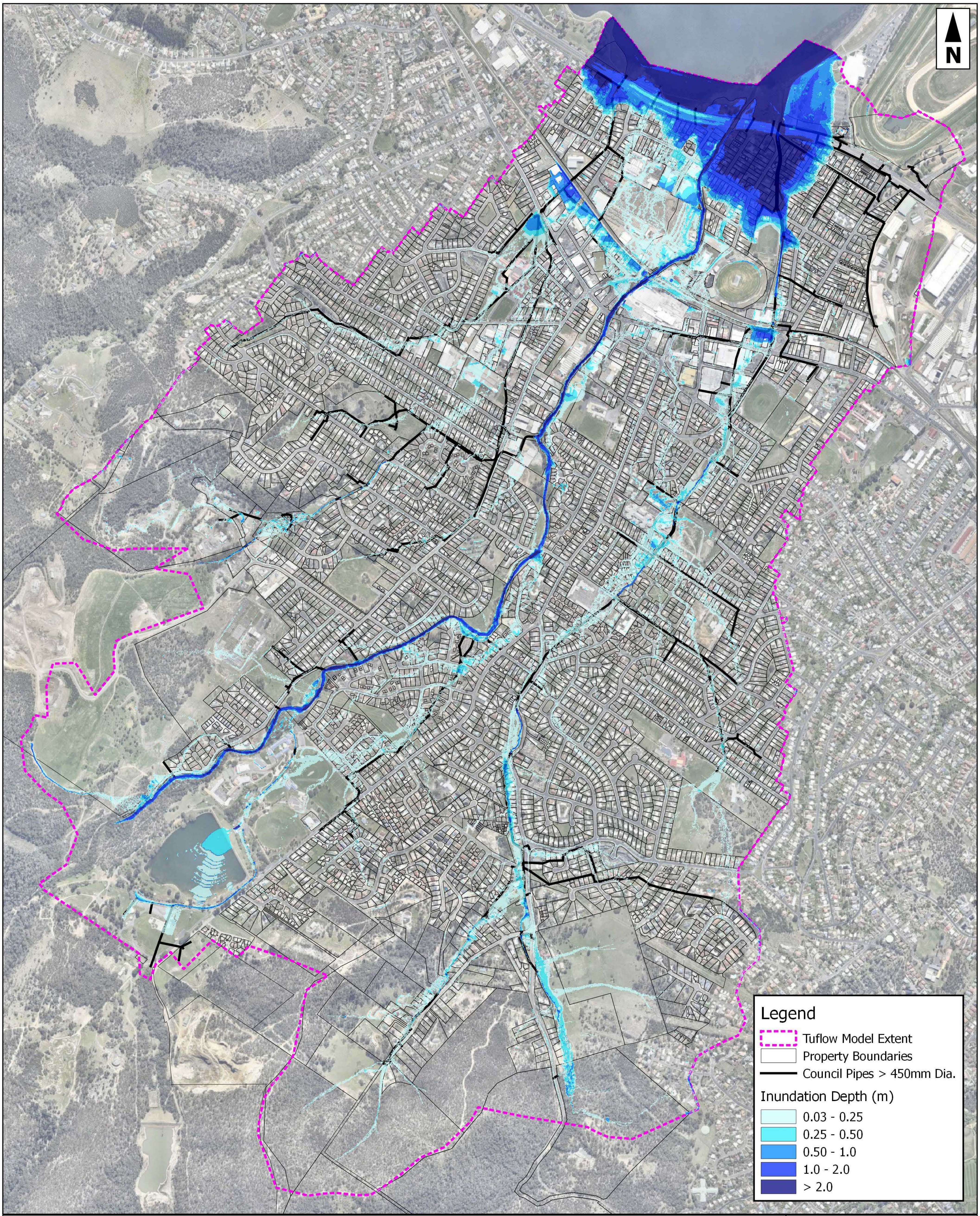

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DATE: 7/09/2018
FIGURE: 30041618-E-cc1

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FIGURE TITLE: 1 in 20 AEP DEVELOPED - CLIMATE CHANGE 1 - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
PAGE SIZE: A3
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0 200 400 600 m

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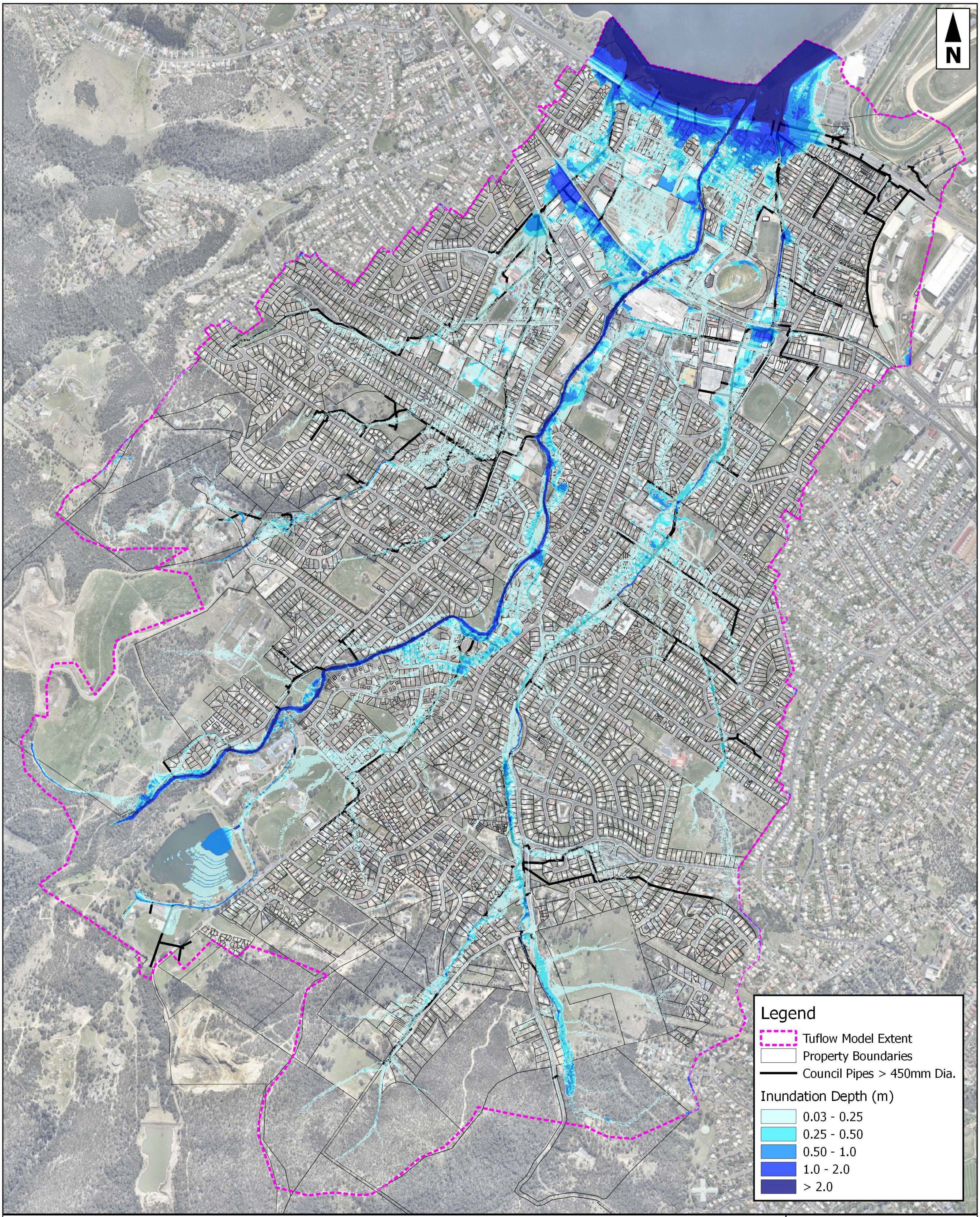

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AUTHOR:	H. Nguyen-Mallen	DATE:	7/09/2018	FIGURE:	30041618-E-cc3

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP DEVELOPED - CLIMATE CHANGE 2 - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

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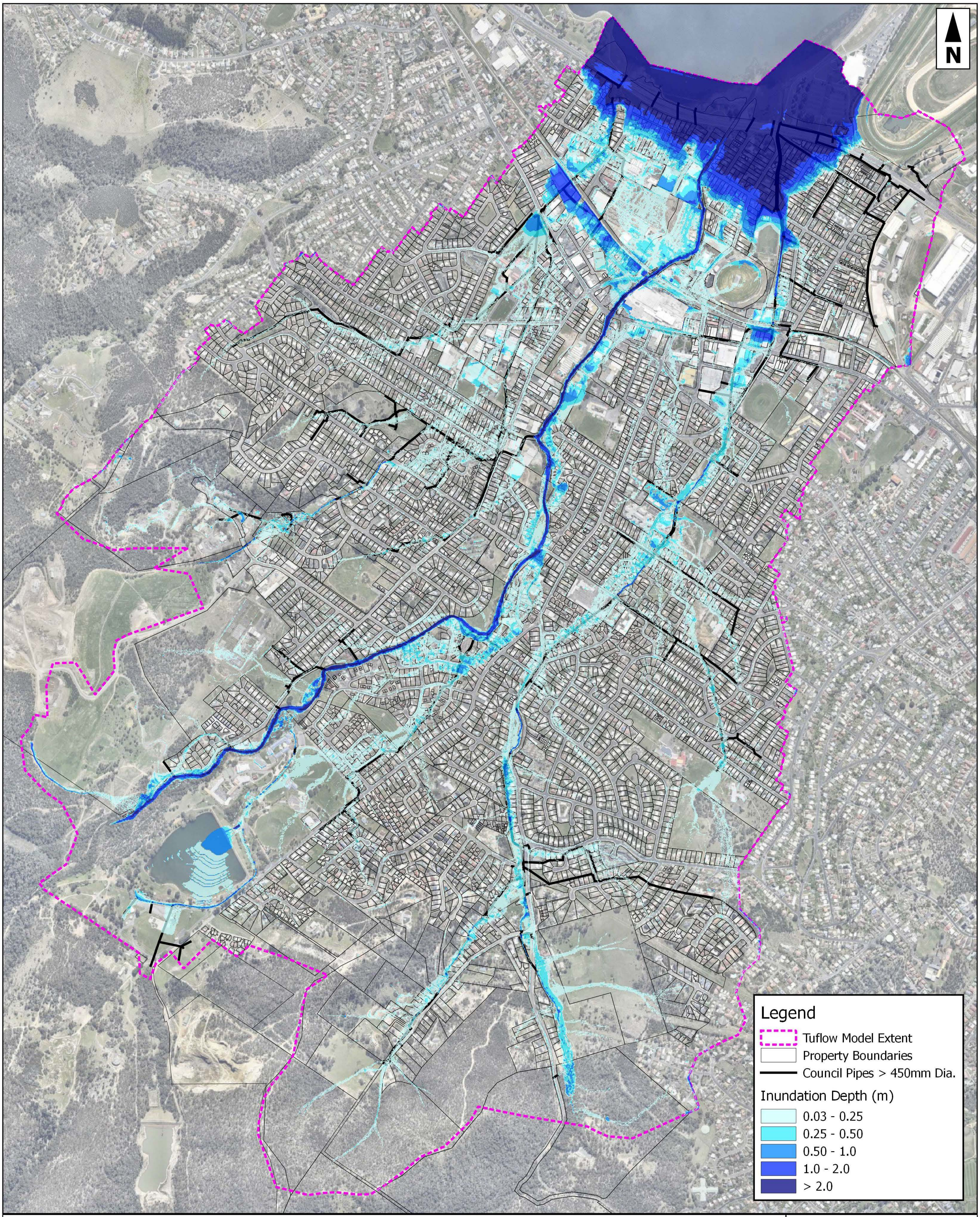

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AUTHOR:	H. Nguyen-Mallen	DATE:	7/09/2018	FIGURE:	30041618-E-cc5	

FIGURE TITLE: 1 in 100 AEP DEVELOPED - CLIMATE CHANGE 3 - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

0 200 400 600 m

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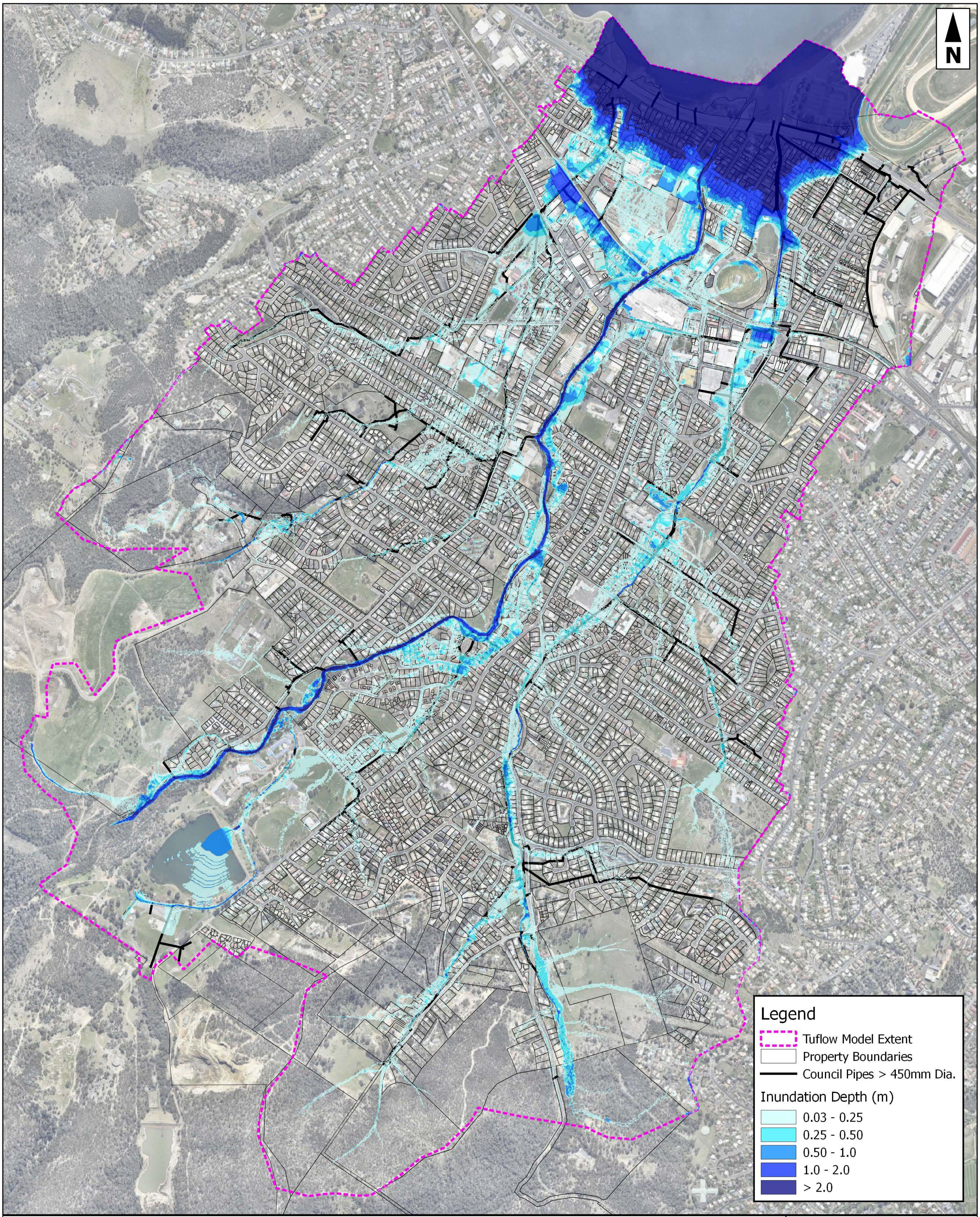

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AUTHOR: H. Nguyen-Mallen	DATE: 7/09/2018	FIGURE: 30041618-E-cc7

FIGURE TITLE: 1 in 100 AEP DEVELOPED - CLIMATE CHANGE 4 - Innundation Depth Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

0 200 400 600 m

PROJECT NO: 30041618 PROJECT TITLE: GLENORCHY CBD SSMP

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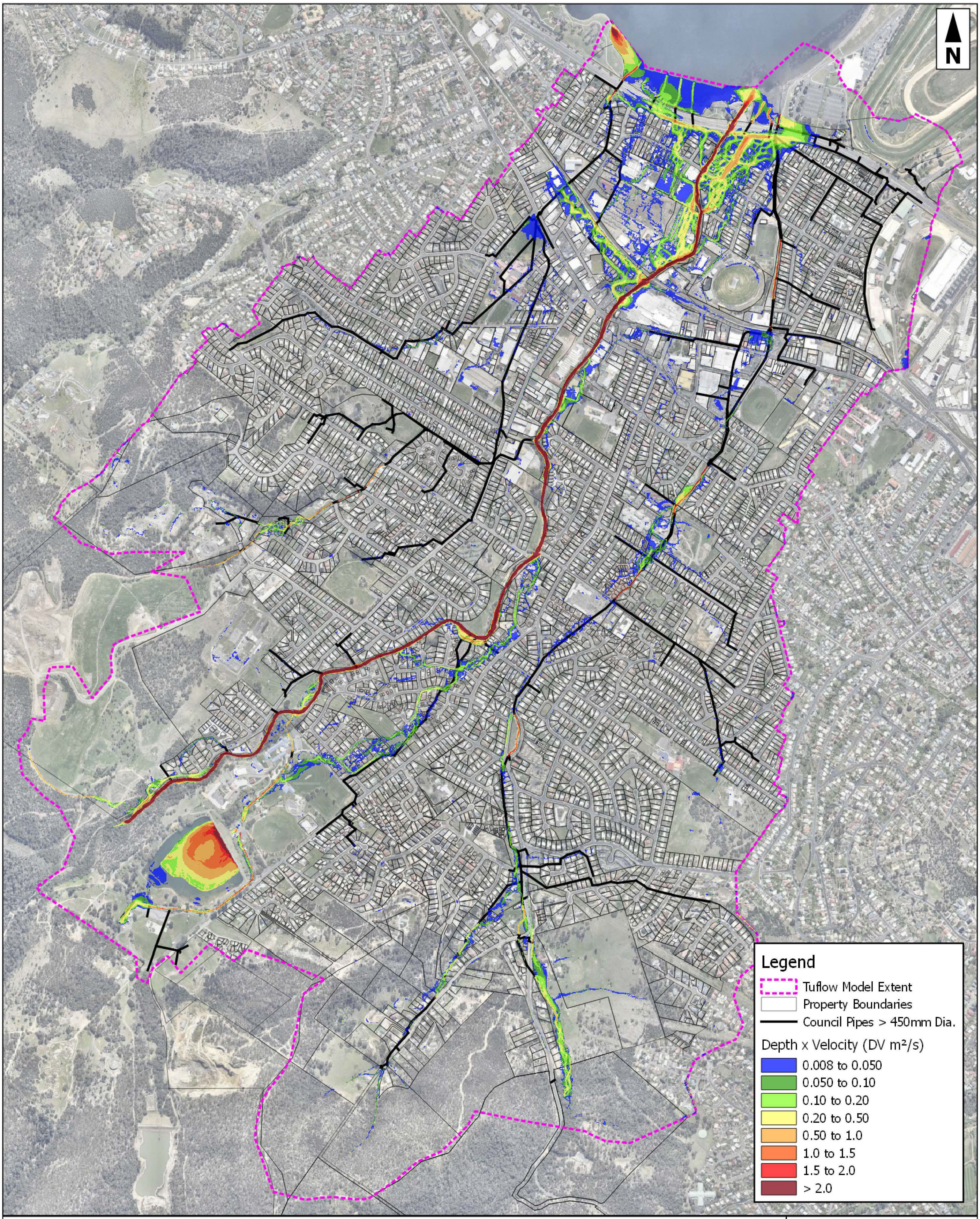

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AUTHOR:	H. Nguyen-Mallen	DATE:	7/09/2018	FIGURE:	30041618-E-cal2

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1996 CALIBRATION EVENT - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

0 200 400 600 m

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PROJECT NO: 30041618 PROJECT TITLE: GLENORCHY CBD SSMP

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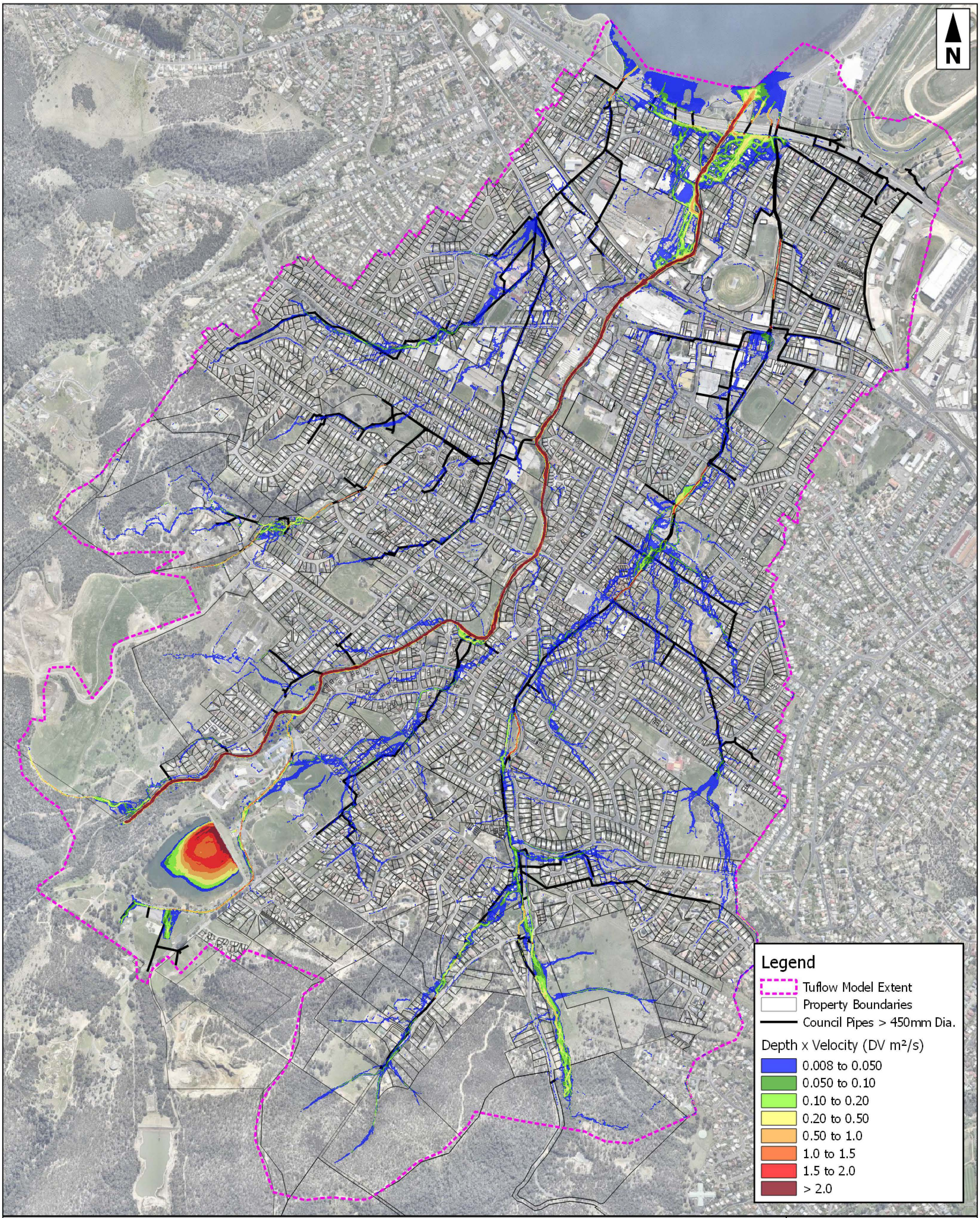

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AUTHOR: H. Nguyen-Mallen
STATUS: FINAL
DATE: 7/09/2018
SOURCES: © NEARMAP 2018
FIGURE: 30041618-E-ex2
CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 20 AEP EXISTING - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
PAGE SIZE: A3
SCALE: 1:13,000
0 200 400 600 m

PROJECT NO: 30041618
PROJECT TITLE: GLENORCHY CBD SSMP

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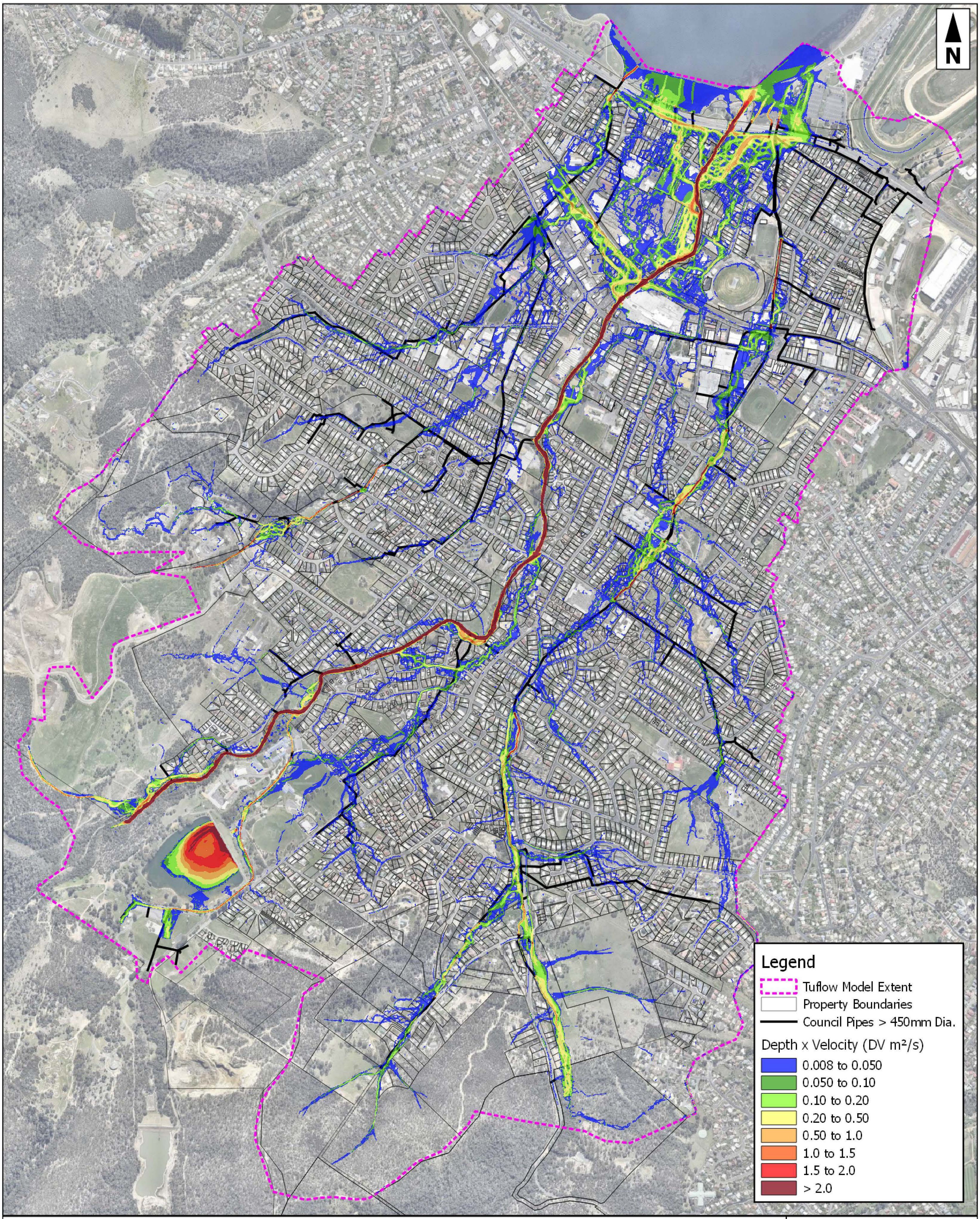

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STATUS: FINAL
DATE: 7/09/2018
SOURCES: © NEARMAP 2018
FIGURE: 30041618-E-ex4
CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP EXISTING - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
PAGE SIZE: A3
SCALE: 1:13,000
0 200 400 600 m

PROJECT NO: 30041618
PROJECT TITLE: GLENORCHY CBD SSMP

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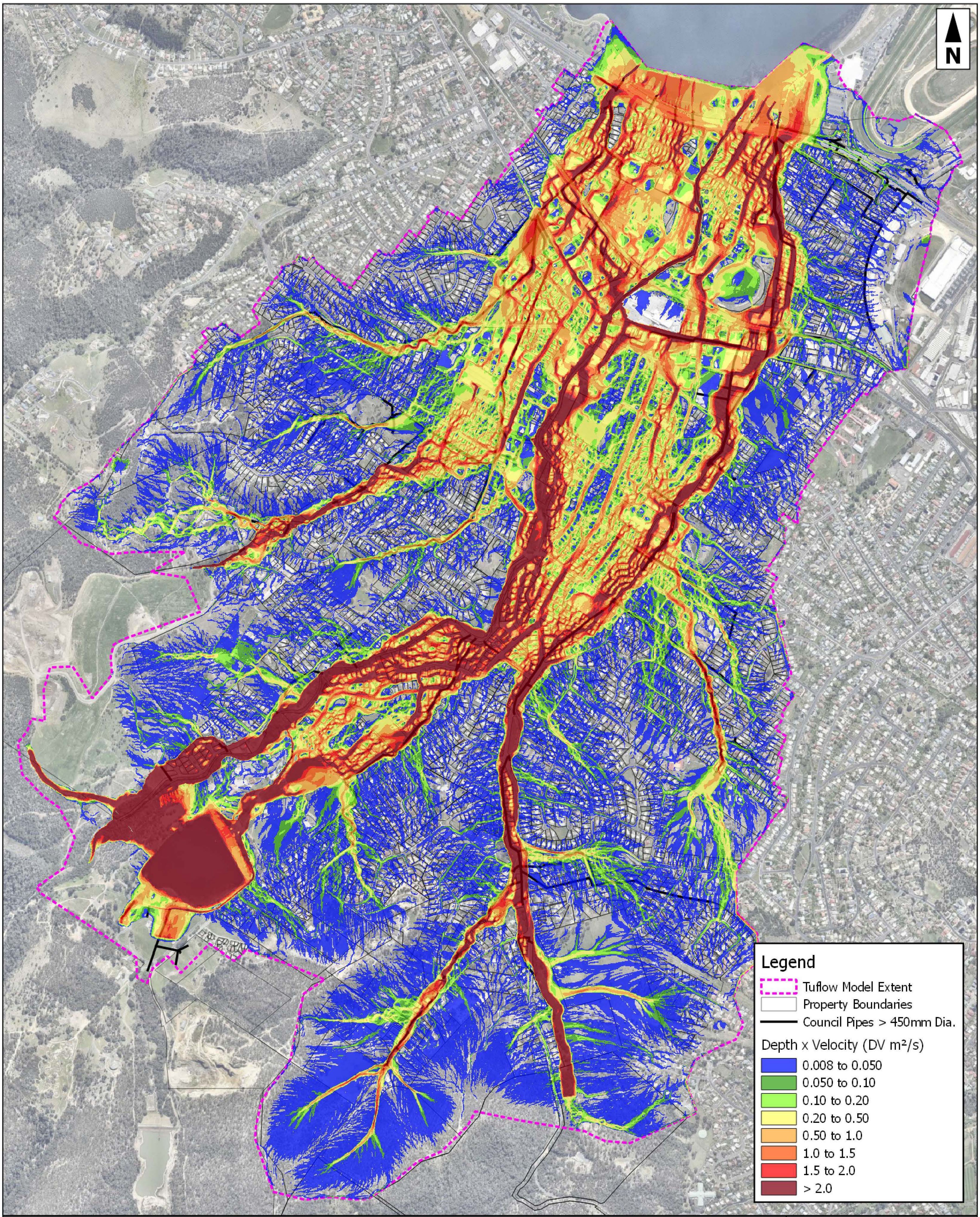

REVISION: A
AUTHOR: H. Nguyen-Mallen
STATUS: FINAL
DATE: 7/09/2018
SOURCES: © NEARMAP 2018
FIGURE: 30041618-E-ex6
CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: PMF EXISTING - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
PAGE SIZE: A3
SCALE: 1:13,000
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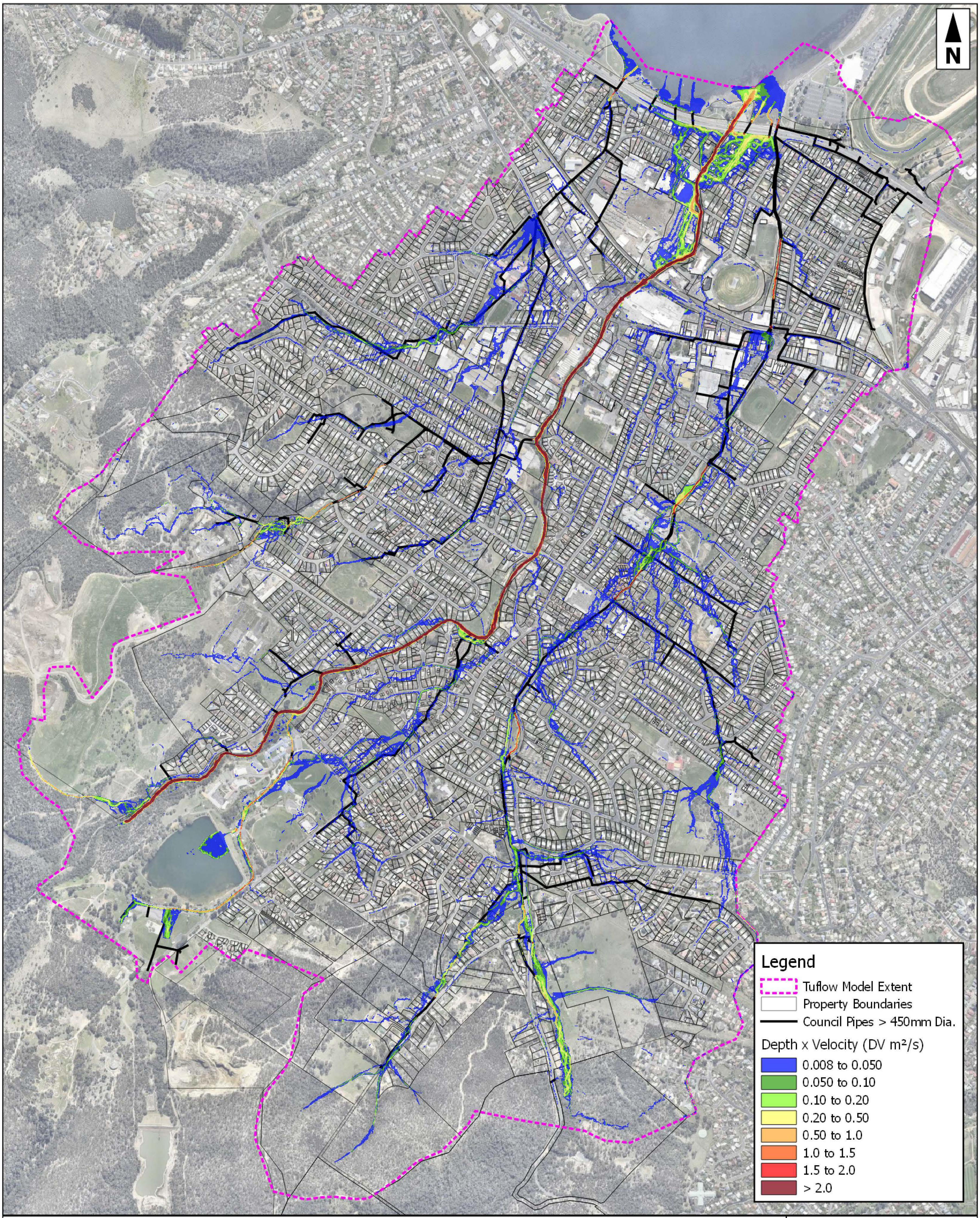
PROJECT NO: 30041618
PROJECT TITLE: GLENORCHY CBD SSMP



REVISION: A | STATUS: FINAL | SOURCES: © NEARMAP 2018

AUTHOR: H. Nguyen-Mallen | DATE: 7/09/2018 | FIGURE: 30041618-E-dr2

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 20 AEP DEVELOPED - EXIST DAMS LEVEL - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

0 200 400 600 m

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PROJECT NO: 30041618 PROJECT TITLE: GLENORCHY CBD SSMP

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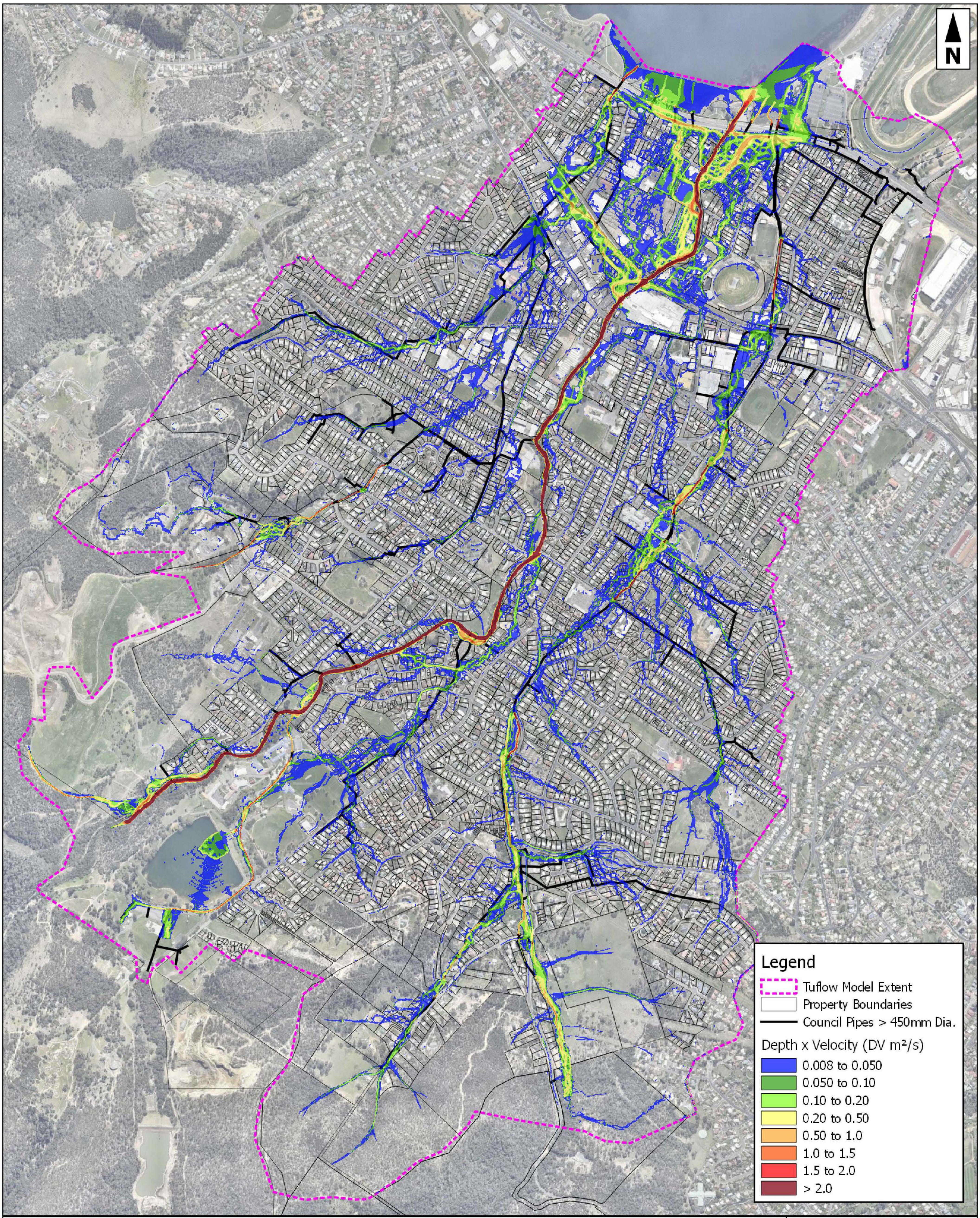

REVISION: A	STATUS: FINAL	SOURCES: © NEARMAP 2018		CLIENT: GLENORCHY CITY COUNCIL
AUTHOR: H. Nguyen-Mallen	DATE: 7/09/2018	FIGURE: 30041618-E-dr4		
FIGURE TITLE: 1 in 100 AEP DEVELOPED - EXIST DAMS LEVEL - Innundation Hazard (DV) Map				
COORDINATE SYSTEM: GDA 94 / MGA ZONE 55		0 200 400 600 m		SMEC
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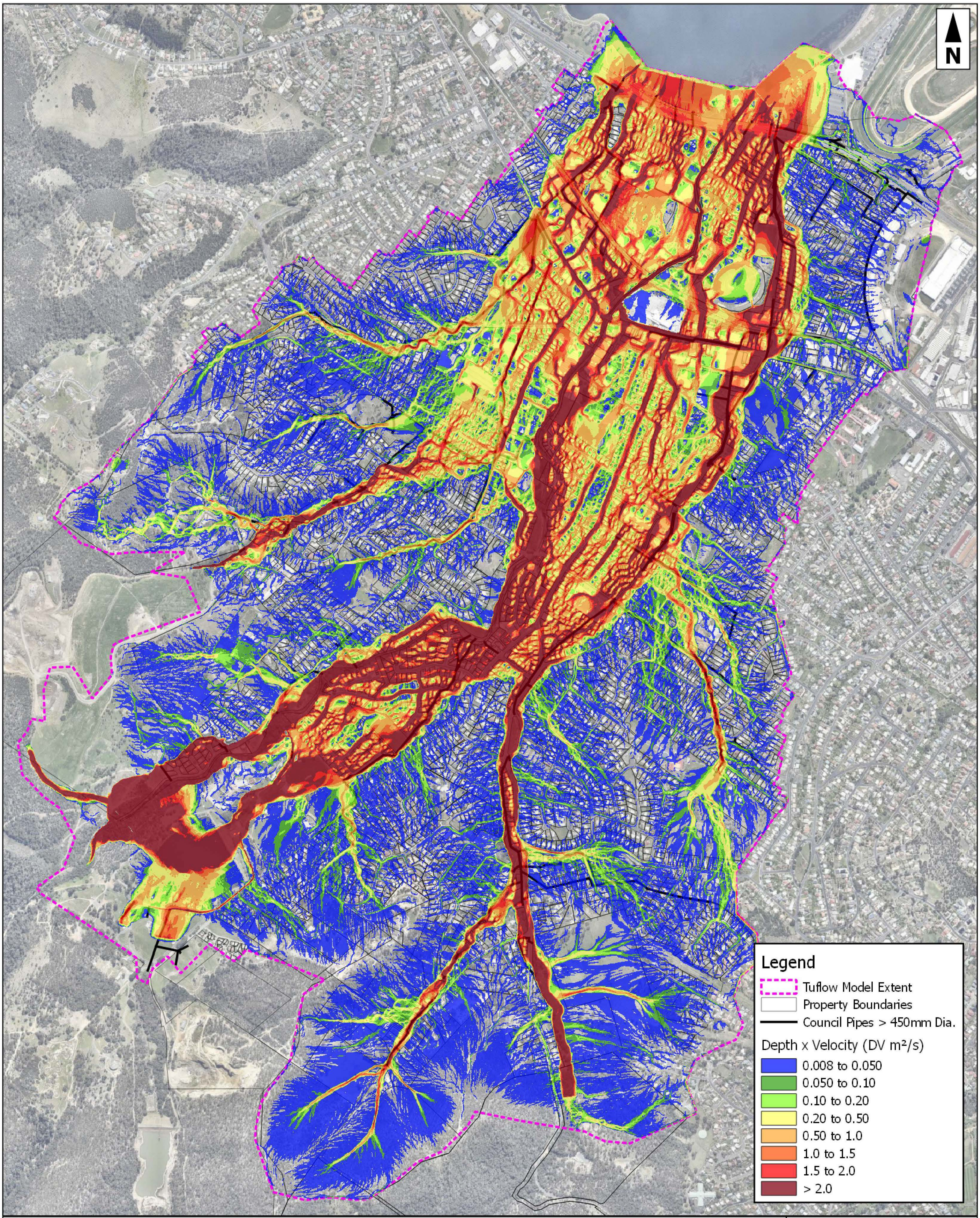

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AUTHOR: H. Nguyen-Mallen	DATE: 7/09/2018	FIGURE: 30041618-E-dr6

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: PMF DEVELOPED - EXIST DAMS LEVEL - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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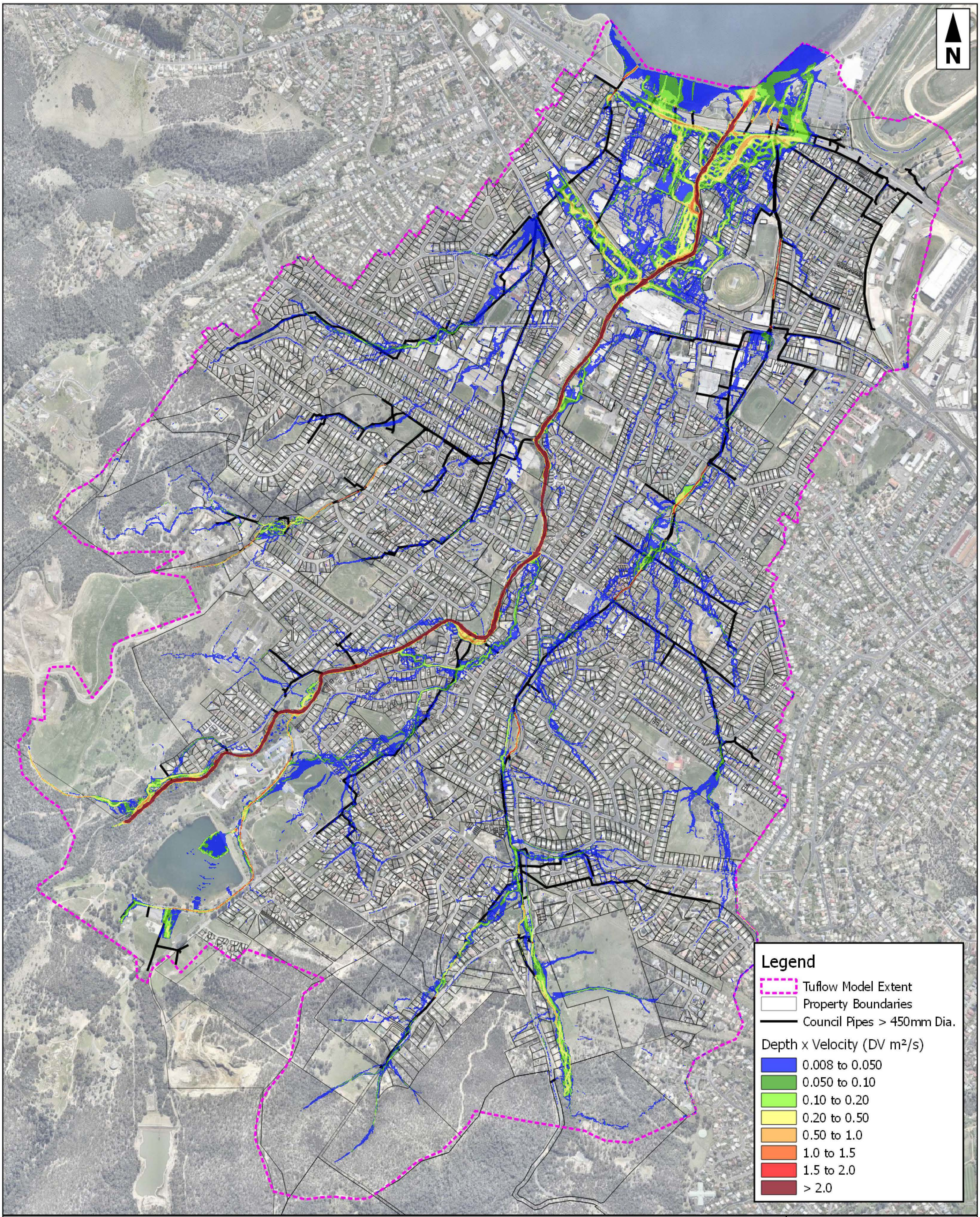

REVISION: A	STATUS: FINAL	SOURCES: © NEARMAP 2018
AUTHOR: H. Nguyen-Mallen	DATE: 7/09/2018	FIGURE: 30041618-E-de2

FIGURE TITLE: 1 in 20 AEP DEVELOPED - DAMS FULL - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

0 200 400 600 m

PROJECT NO: 30041618 PROJECT TITLE: GLENORCHY CBD SSMP

CLIENT: GLENORCHY CITY COUNCIL

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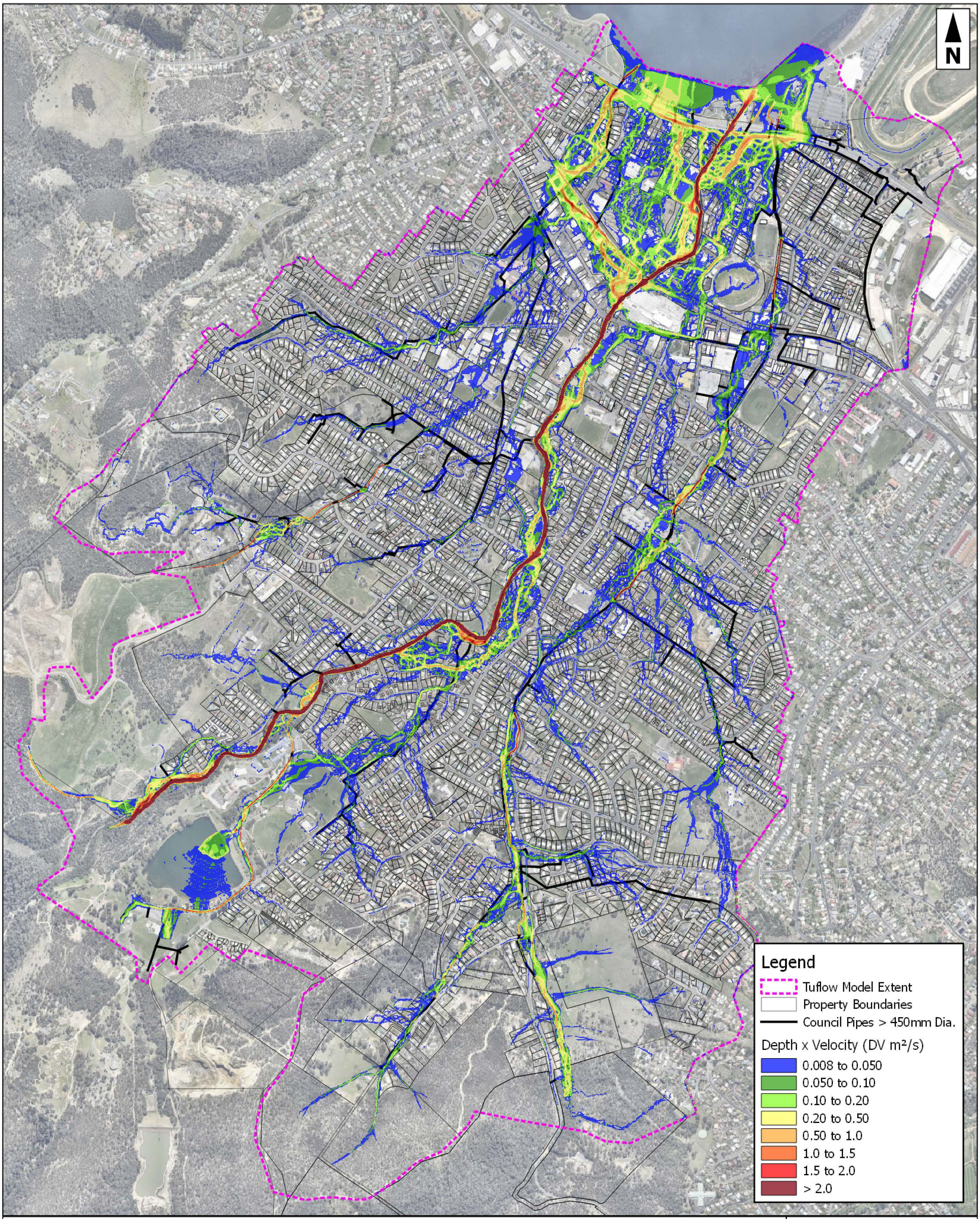

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AUTHOR: H. Nguyen-Mallen	DATE: 7/09/2018	FIGURE: 30041618-E-de4

FIGURE TITLE: 1 in 100 AEP DEVELOPED - DAMS FULL - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

0 200 400 600 m

PROJECT NO: 30041618 PROJECT TITLE: GLENORCHY CBD SSMP

CLIENT: GLENORCHY CITY COUNCIL

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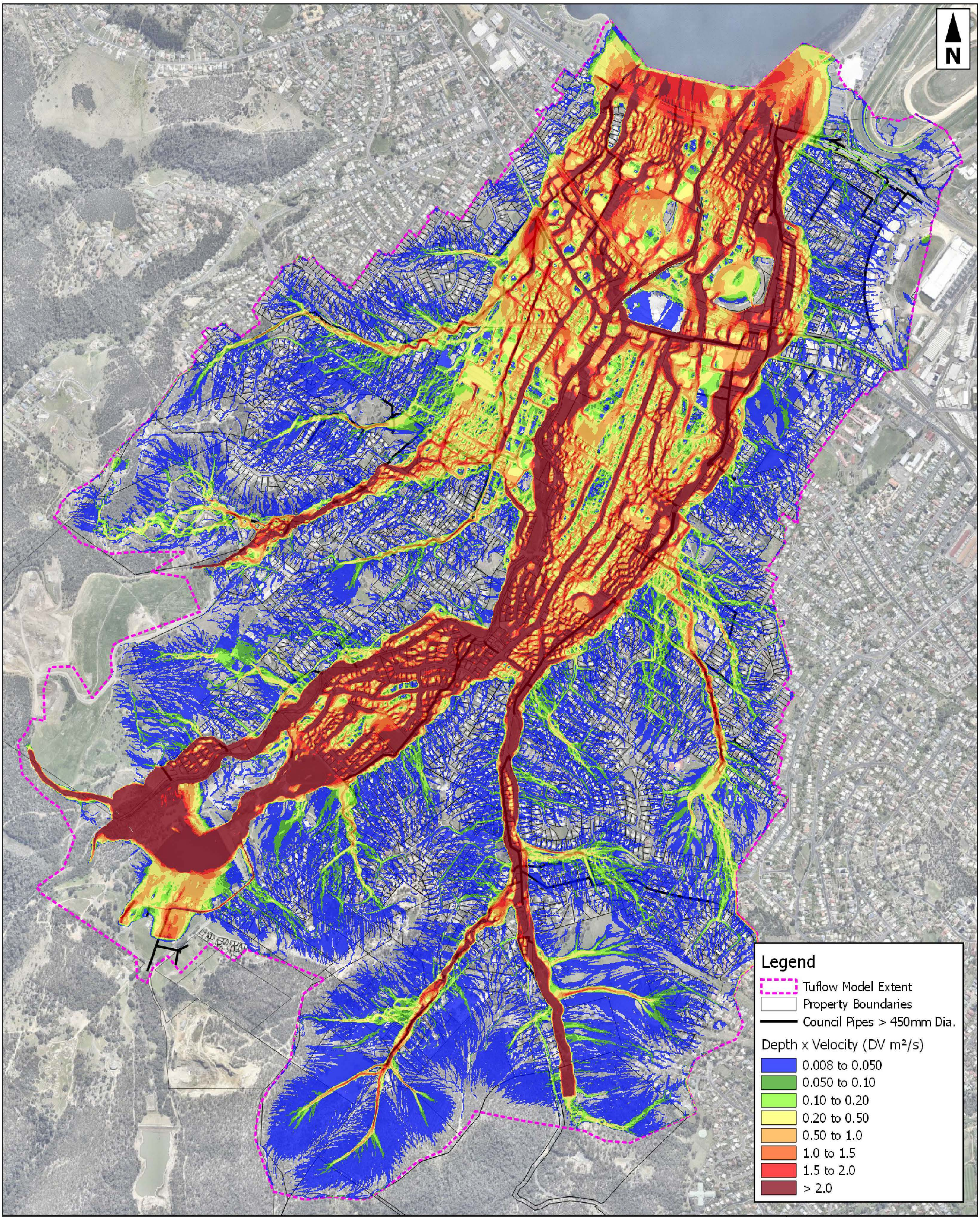

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AUTHOR: H. Nguyen-Mallen	DATE: 7/09/2018	FIGURE: 30041618-E-de6

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: PMF DEVELOPED - DAMS FULL - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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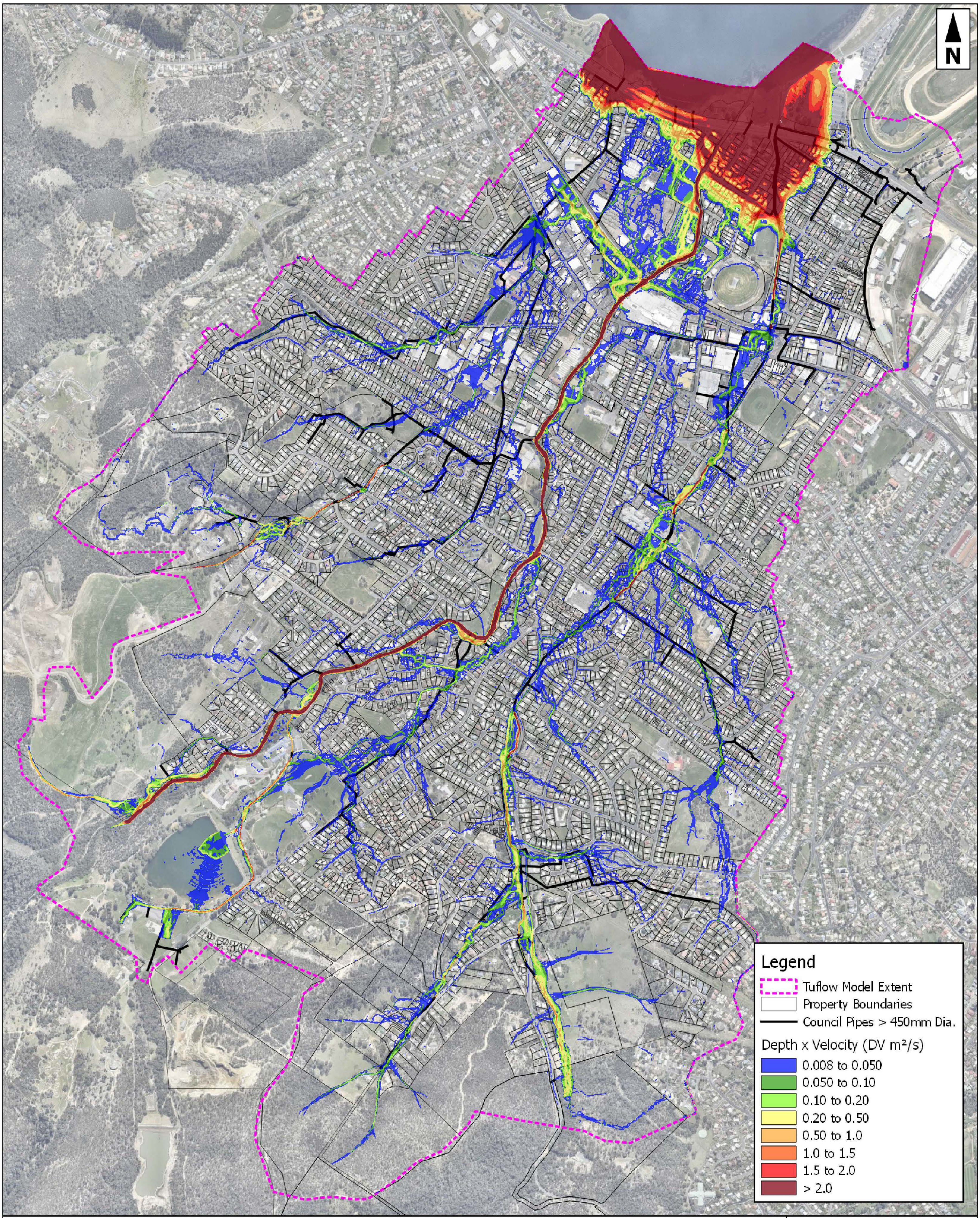

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AUTHOR: H. Nguyen-Mallen	DATE: 7/09/2018	FIGURE: 30041618-E-cc2

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 20 AEP DEVELOPED - CLIMATE CHANGE 1 - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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PROJECT NO: 30041618 PROJECT TITLE: GLENORCHY CBD SSMP

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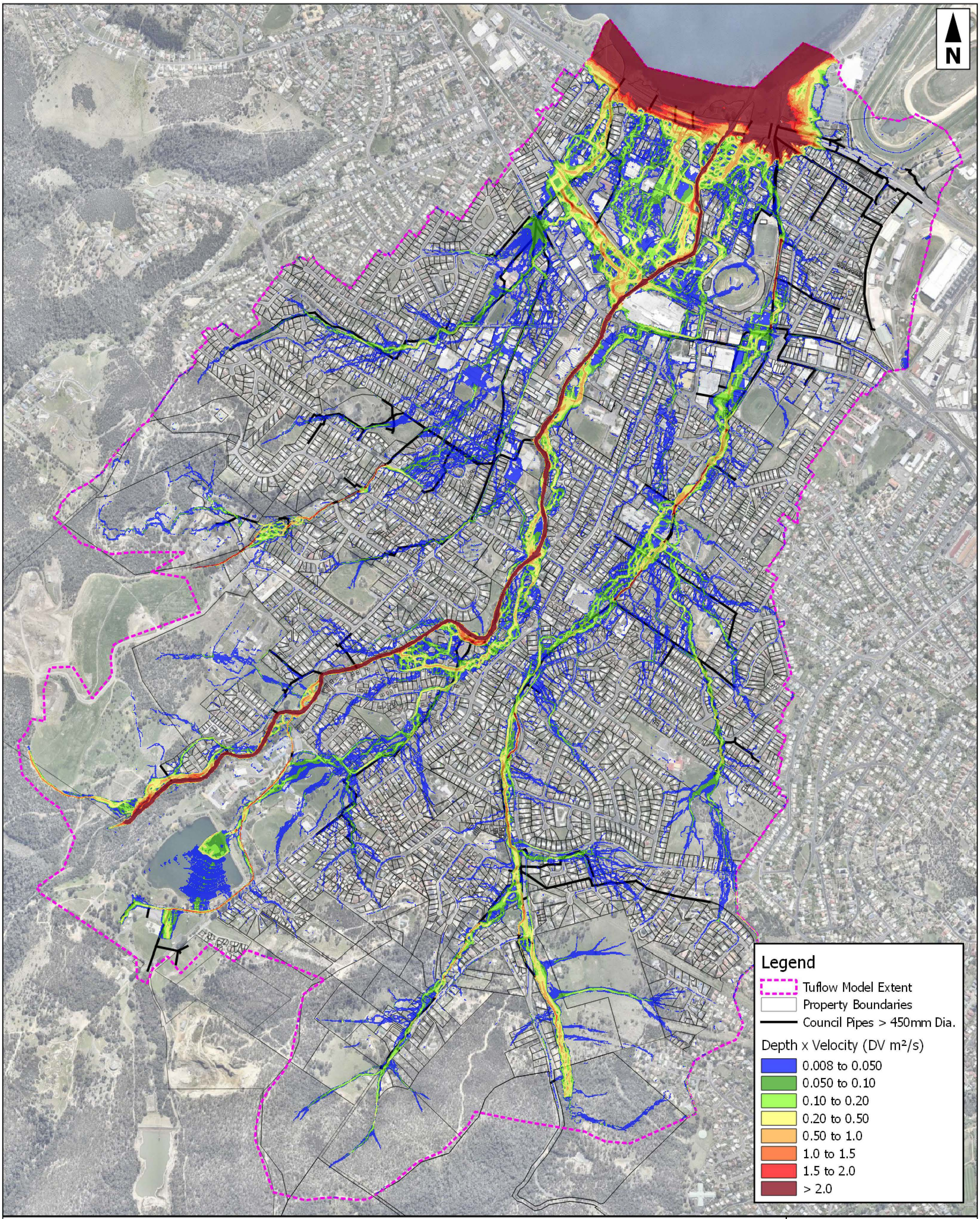

REVISION: A
AUTHOR: H. Nguyen-Mallen
STATUS: FINAL
DATE: 7/09/2018
SOURCES: © NEARMAP 2018
FIGURE: 30041618-E-cc4

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP DEVELOPED - CLIMATE CHANGE 2 - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
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SCALE: 1:13,000

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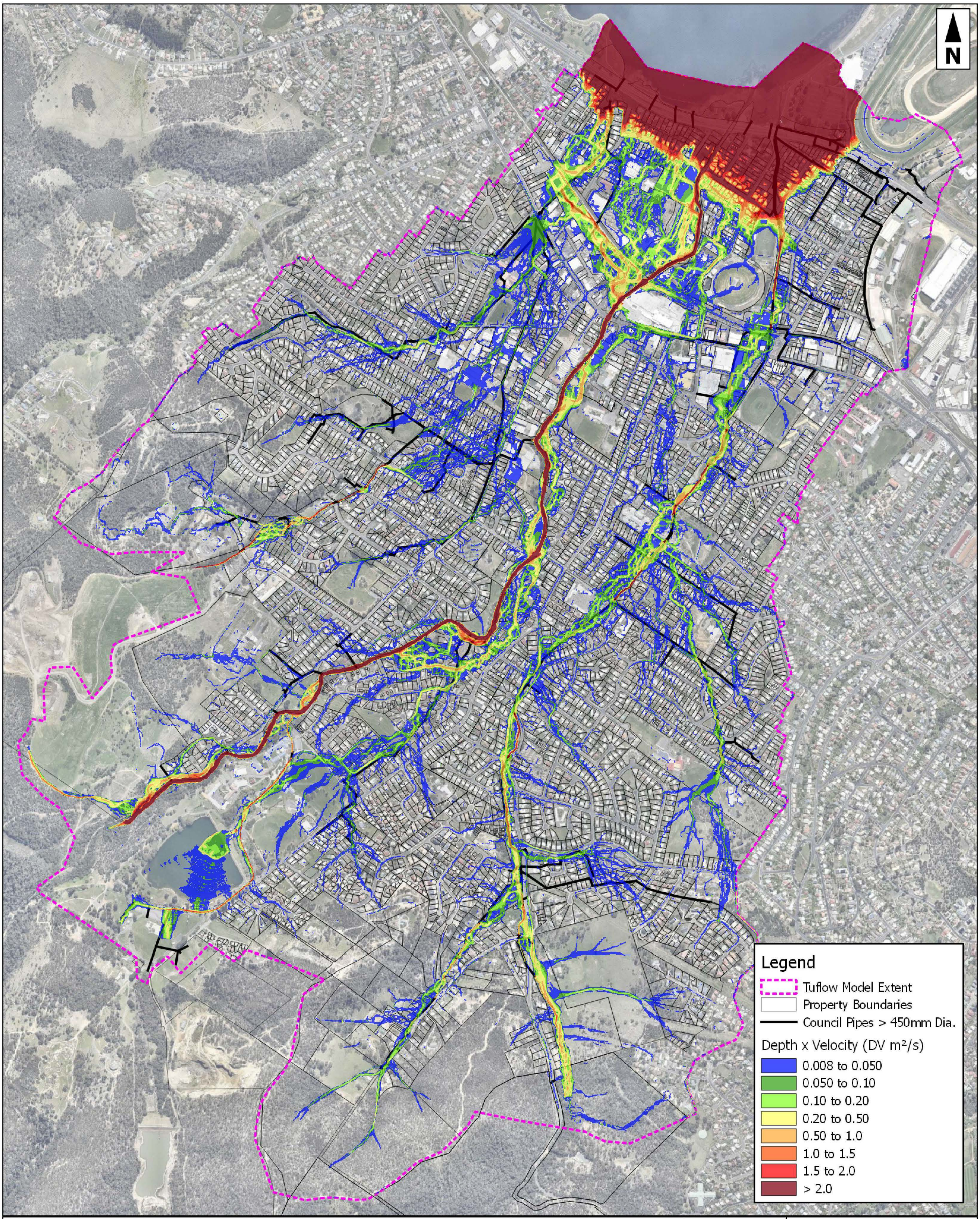

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AUTHOR: H. Nguyen-Mallen	DATE: 7/09/2018	FIGURE: 30041618-E-cc6

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP DEVELOPED - CLIMATE CHANGE 3 - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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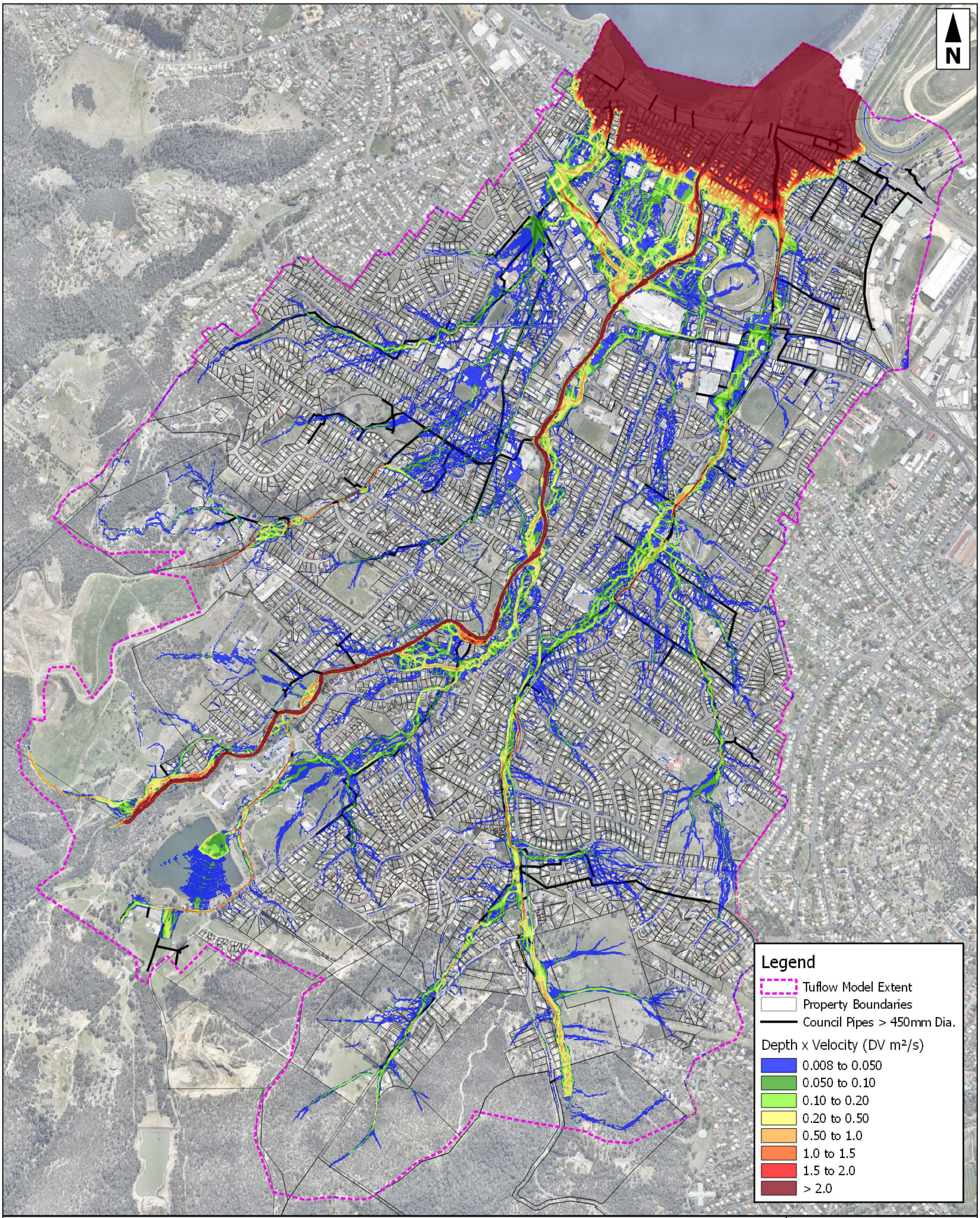

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AUTHOR:	H. Nguyen-Mallen	DATE:	7/09/2018	FIGURE:	30041618-E-cc8

FIGURE TITLE: 1 in 100 AEP DEVELOPED - CLIMATE CHANGE 4 - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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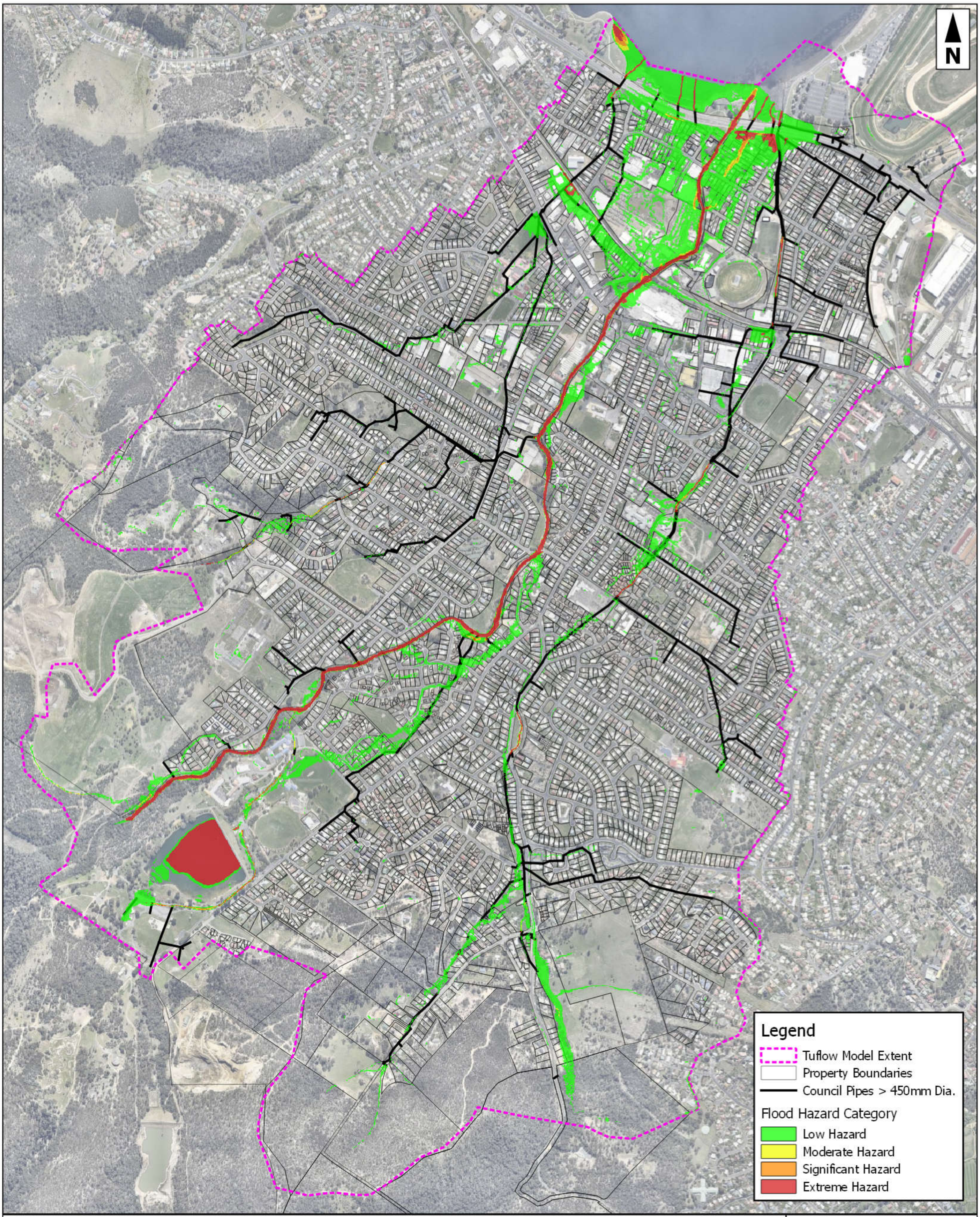

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FIGURE TITLE: 1996 CALIBRATION EVENT - Innundation Hazard (DV) Map

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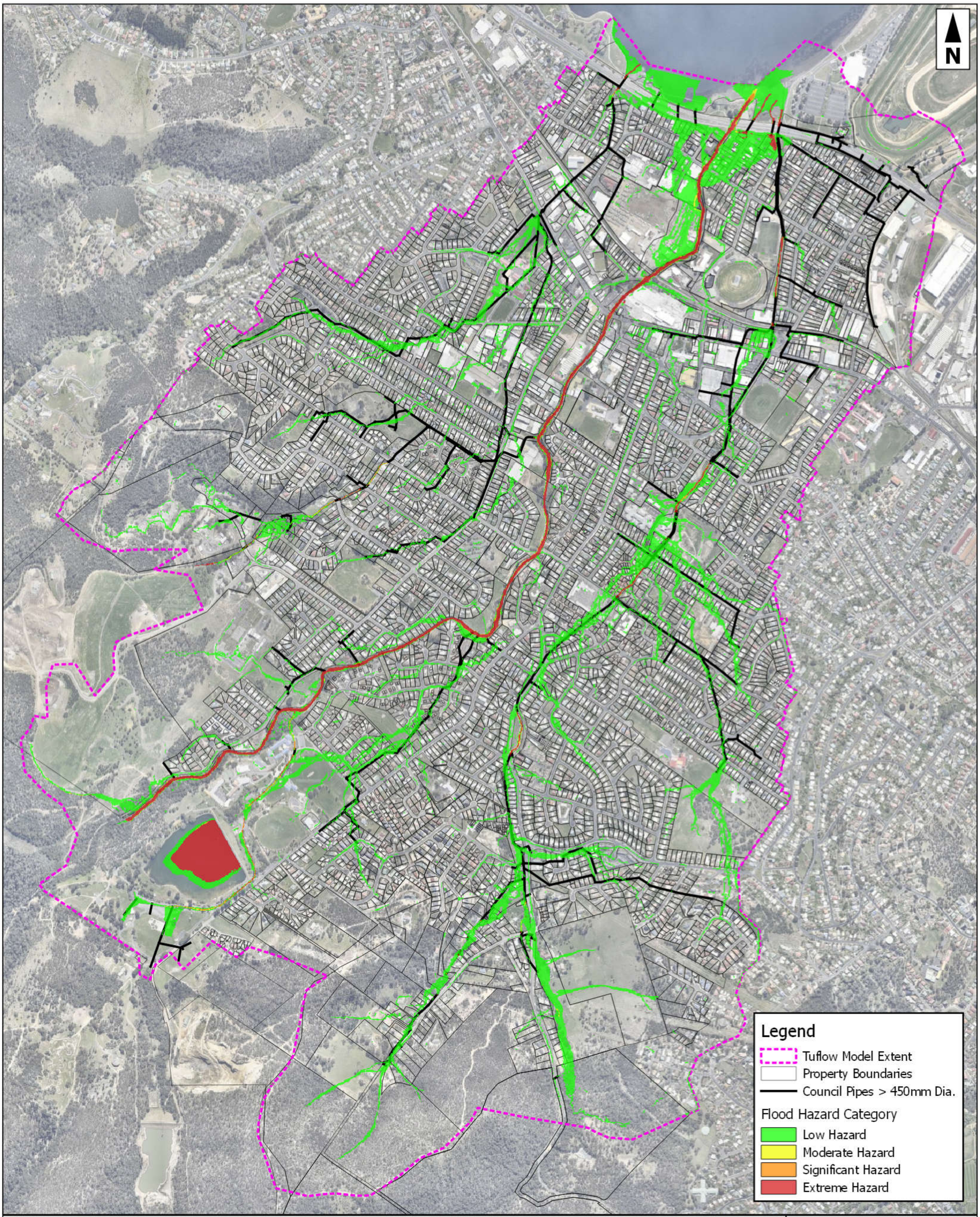

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AUTHOR: H. Nguyen-Mallen
STATUS: FINAL
DATE: 16/11/2018
SOURCES: © NEARMAP 2018
FIGURE: 30041618-E-ex7
CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 20 AEP EXISTING - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
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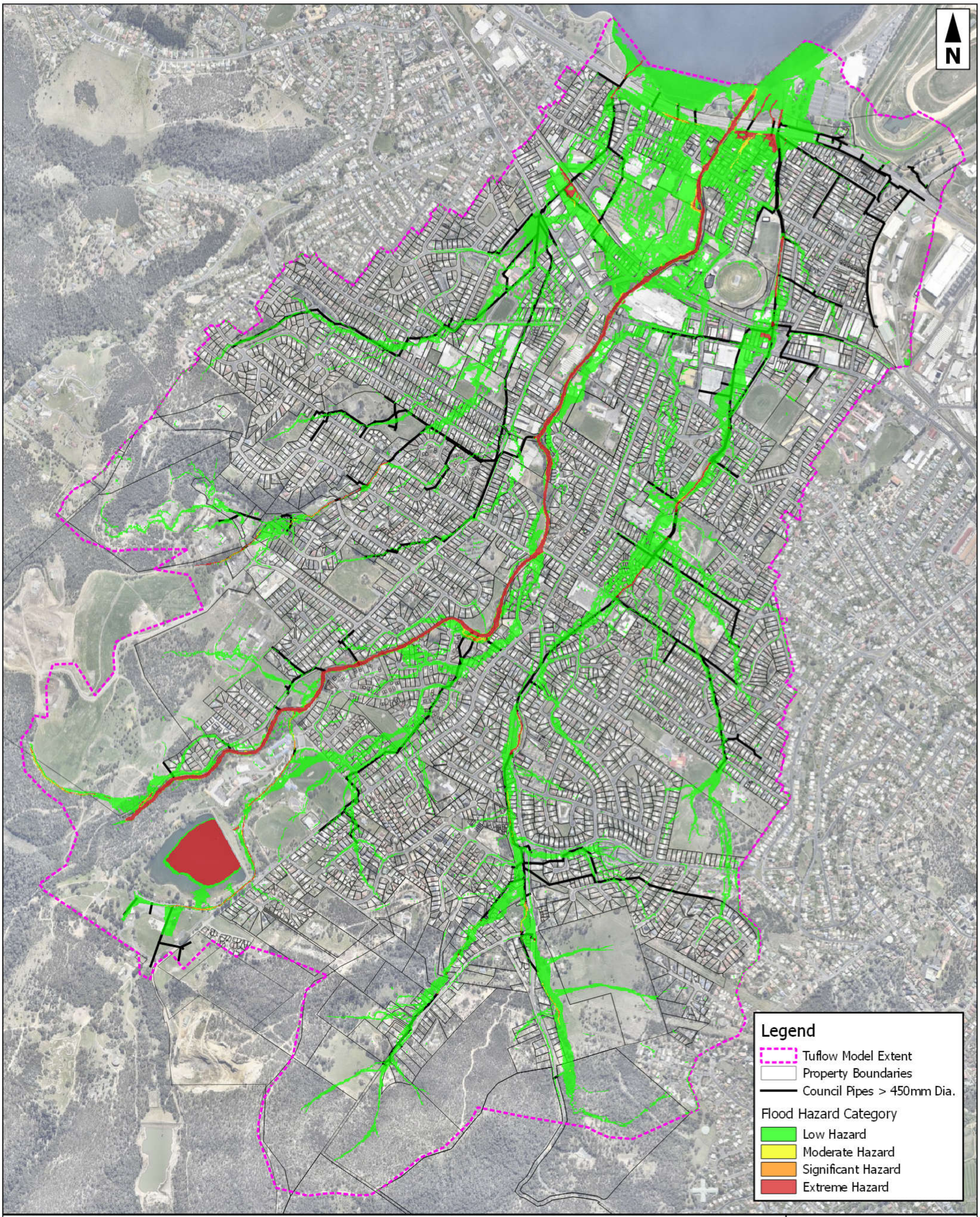

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AUTHOR:	H. Nguyen-Mallen	DATE:	16/11/2018	FIGURE:	30041618-E-ex8

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP EXISTING - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

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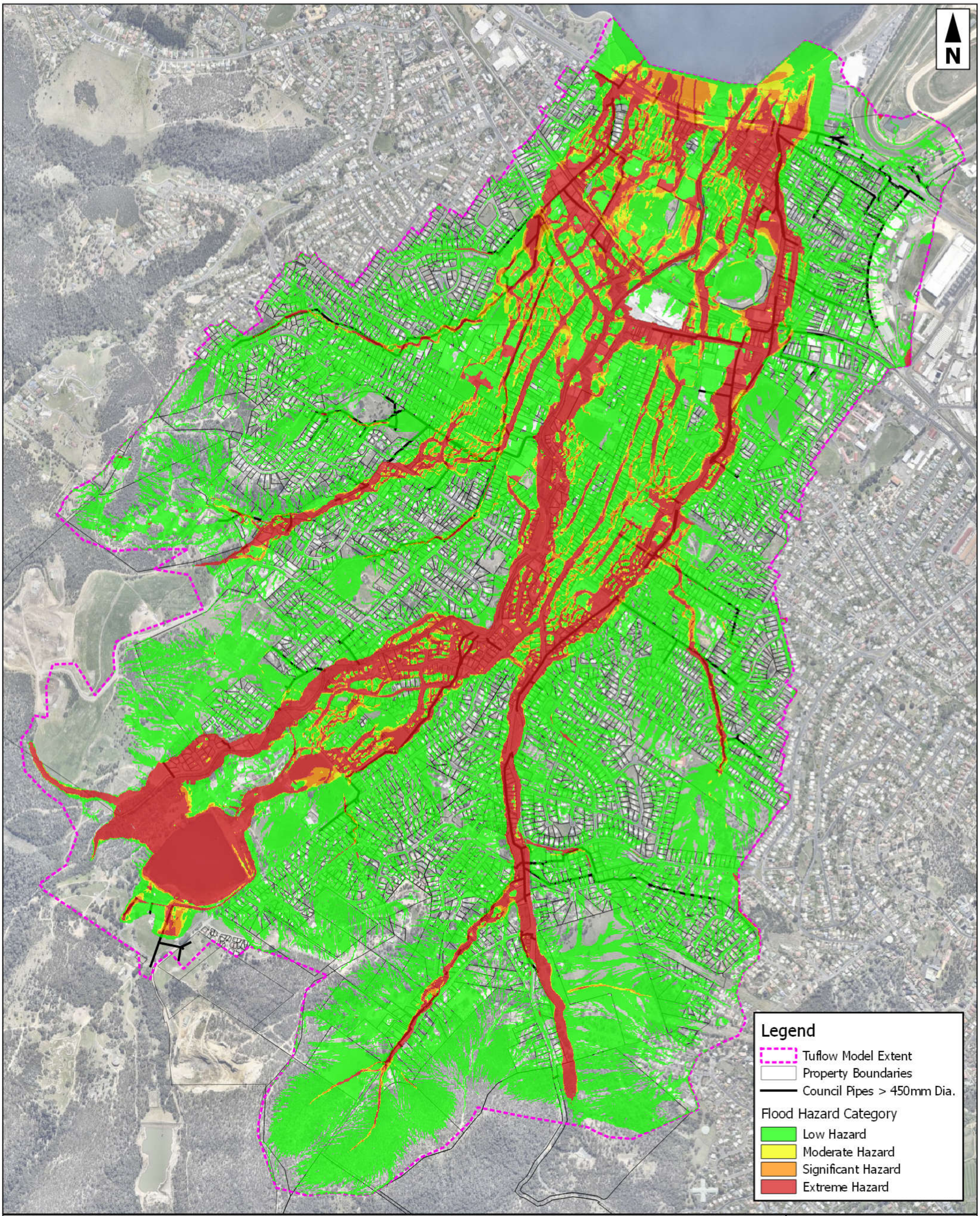

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SOURCES: © NEARMAP 2018
AUTHOR: H. Nguyen-Mallen
DATE: 16/11/2018
FIGURE: 30041618-E-ex9

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: PMF EXISTING - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
PAGE SIZE: A3
SCALE: 1:13,000

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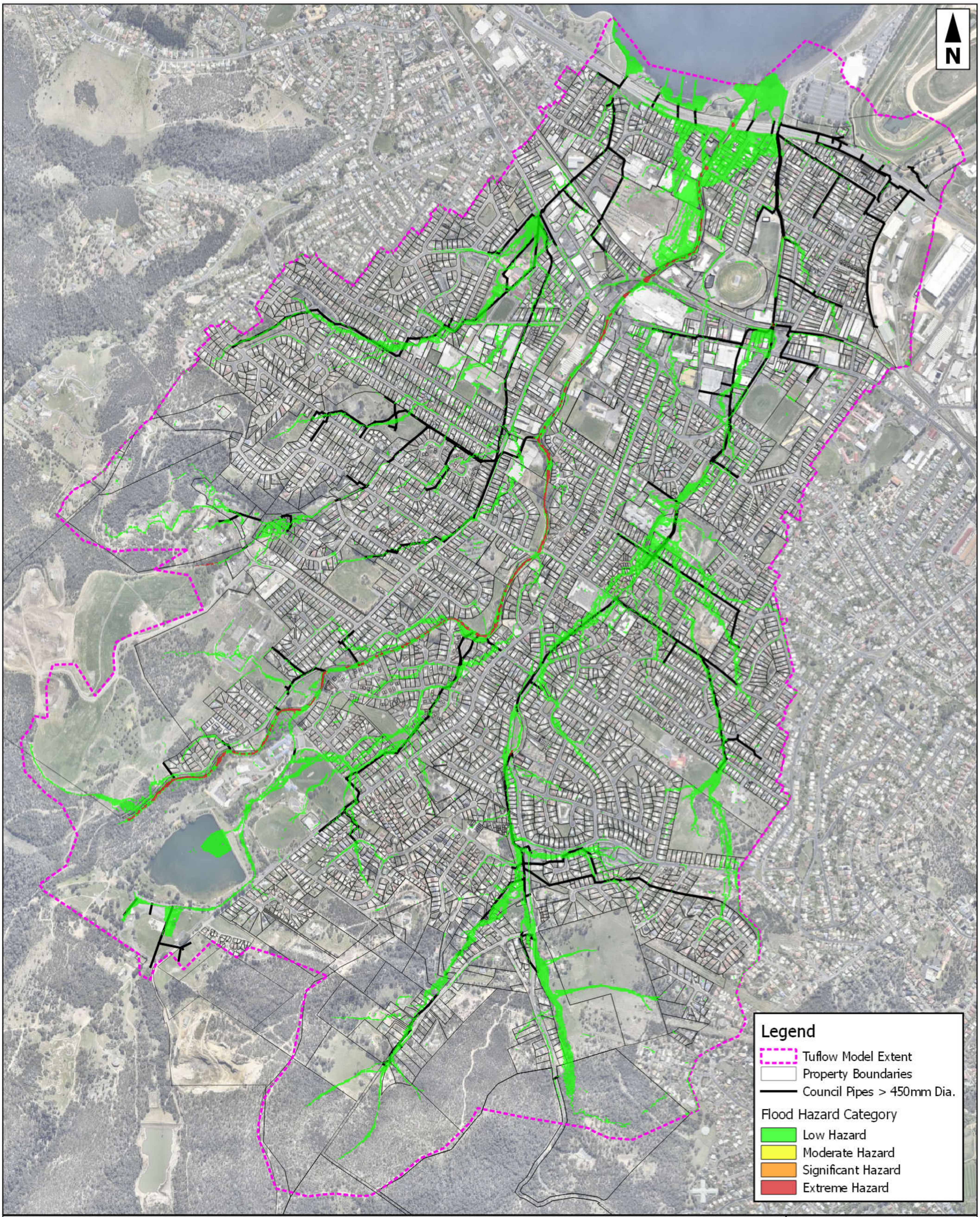

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AUTHOR: H. Nguyen-Mallen	DATE: 16/11/2018	FIGURE: 30041618-E-dr7		

FIGURE TITLE: 1 in 20 AEP DEVELOPED - EXIST DAMS LEVEL - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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PROJECT NO: 30041618 PROJECT TITLE: GLENORCHY CBD SSMP

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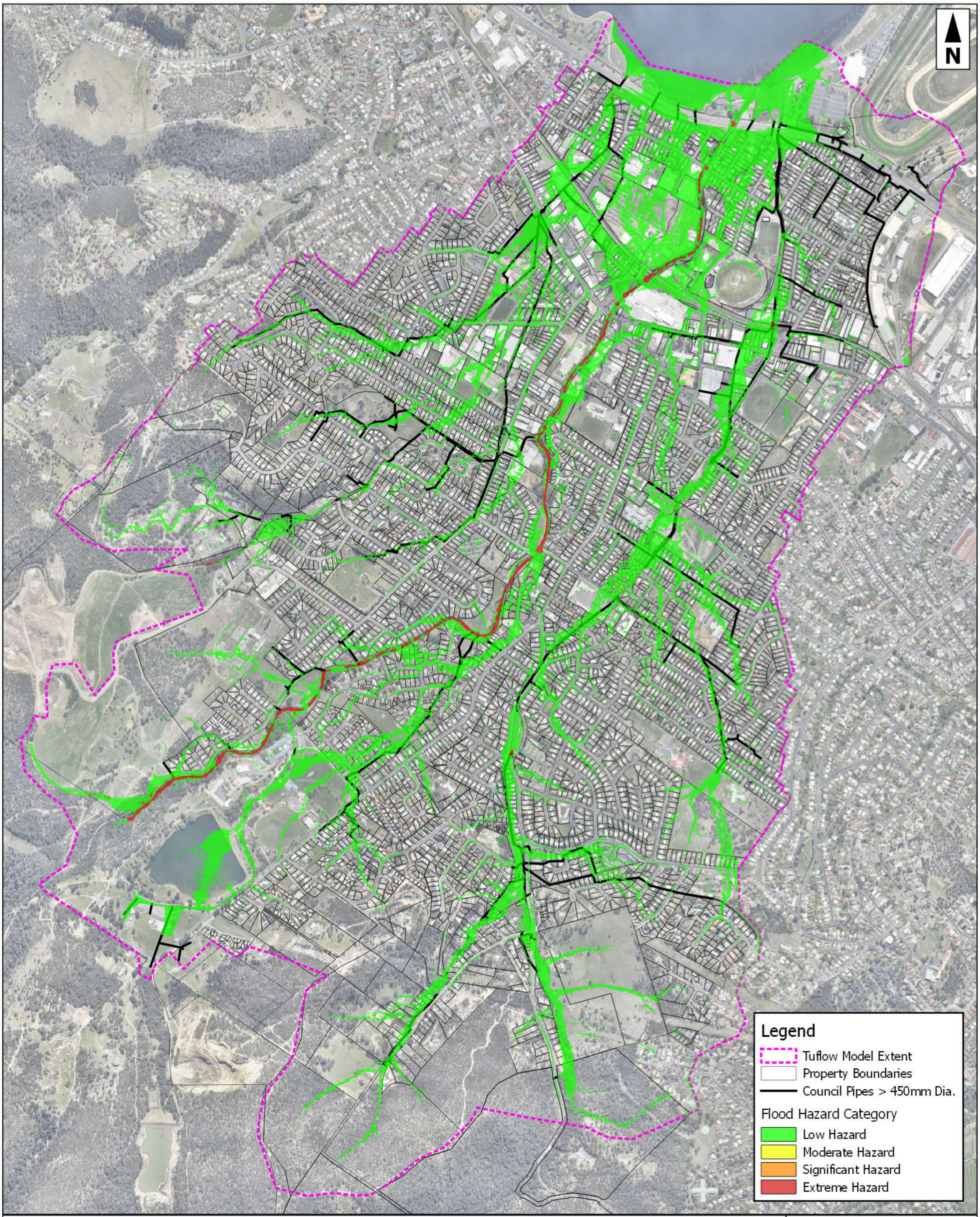

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AUTHOR: H. Nguyen-Mallen	DATE: 16/11/2018	FIGURE: 30041618-E-dr8

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP DEVELOPED - EXIST DAMS LEVEL - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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PROJECT NO: 30041618 PROJECT TITLE: GLENORCHY CBD SSMP

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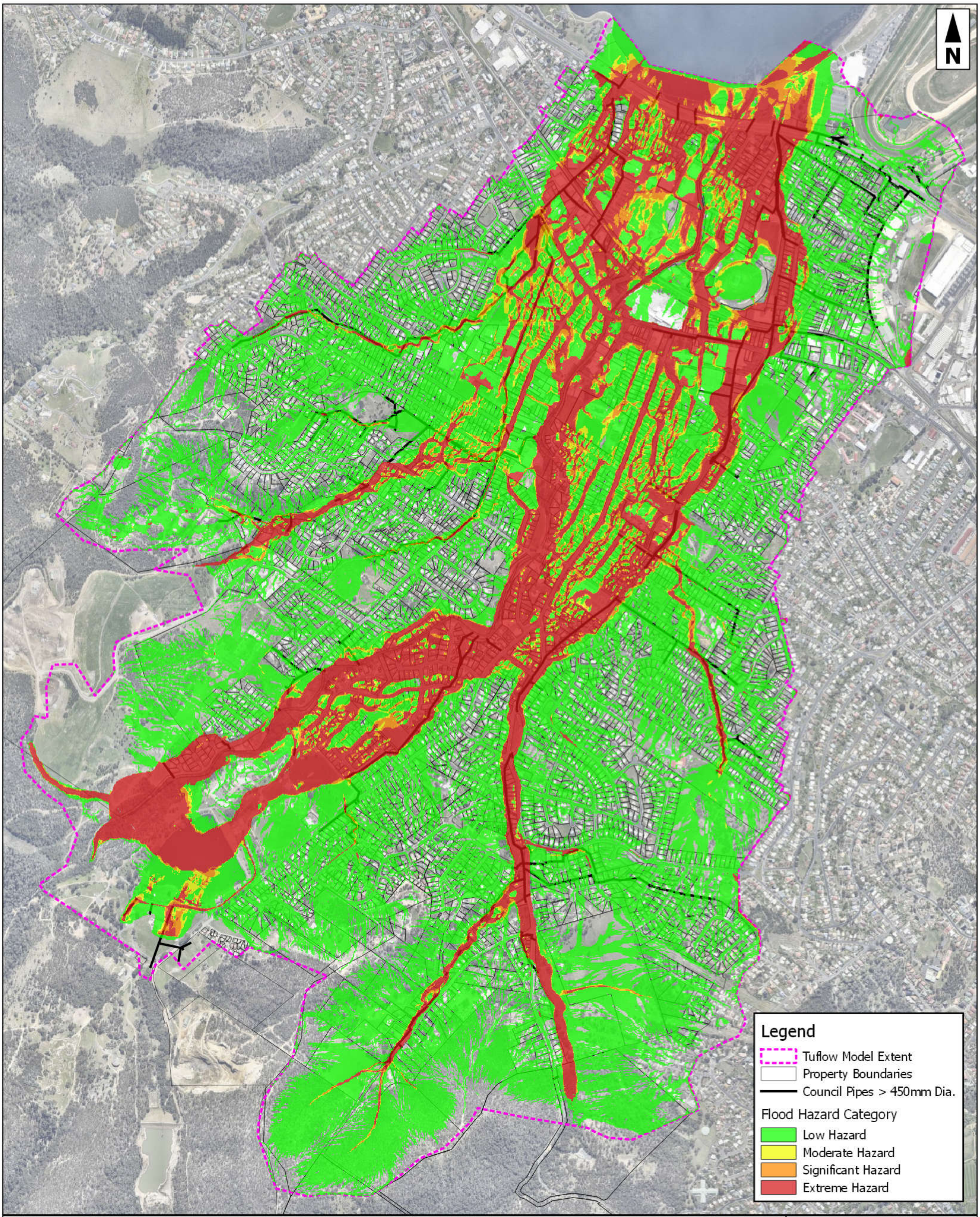

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AUTHOR: H. Nguyen-Mallen	DATE: 16/11/2018	FIGURE: 30041618-E-dr9

FIGURE TITLE: PMF DEVELOPED - EXIST DAMS LEVEL - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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PROJECT NO: 30041618 PROJECT TITLE: GLENORCHY CBD SSMP

CLIENT: GLENORCHY CITY COUNCIL

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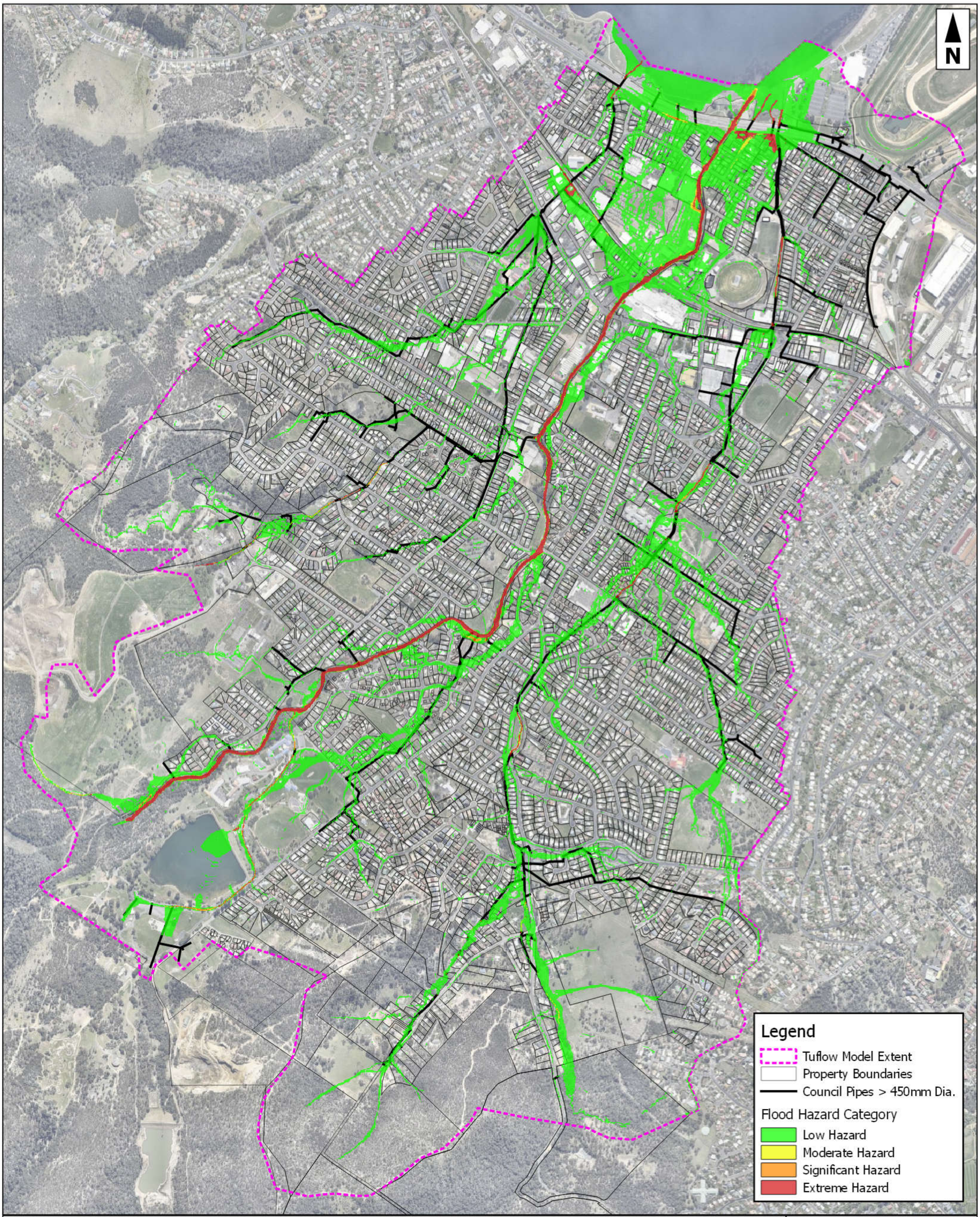

REVISION: A
AUTHOR: H. Nguyen-Mallen
STATUS: FINAL
DATE: 16/11/2018
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FIGURE: 30041618-E-de7
CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 20 AEP DEVELOPED - DAMS FULL - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
PAGE SIZE: A3
SCALE: 1:13,000

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PROJECT NO: 30041618
PROJECT TITLE: GLENORCHY CBD SSMP

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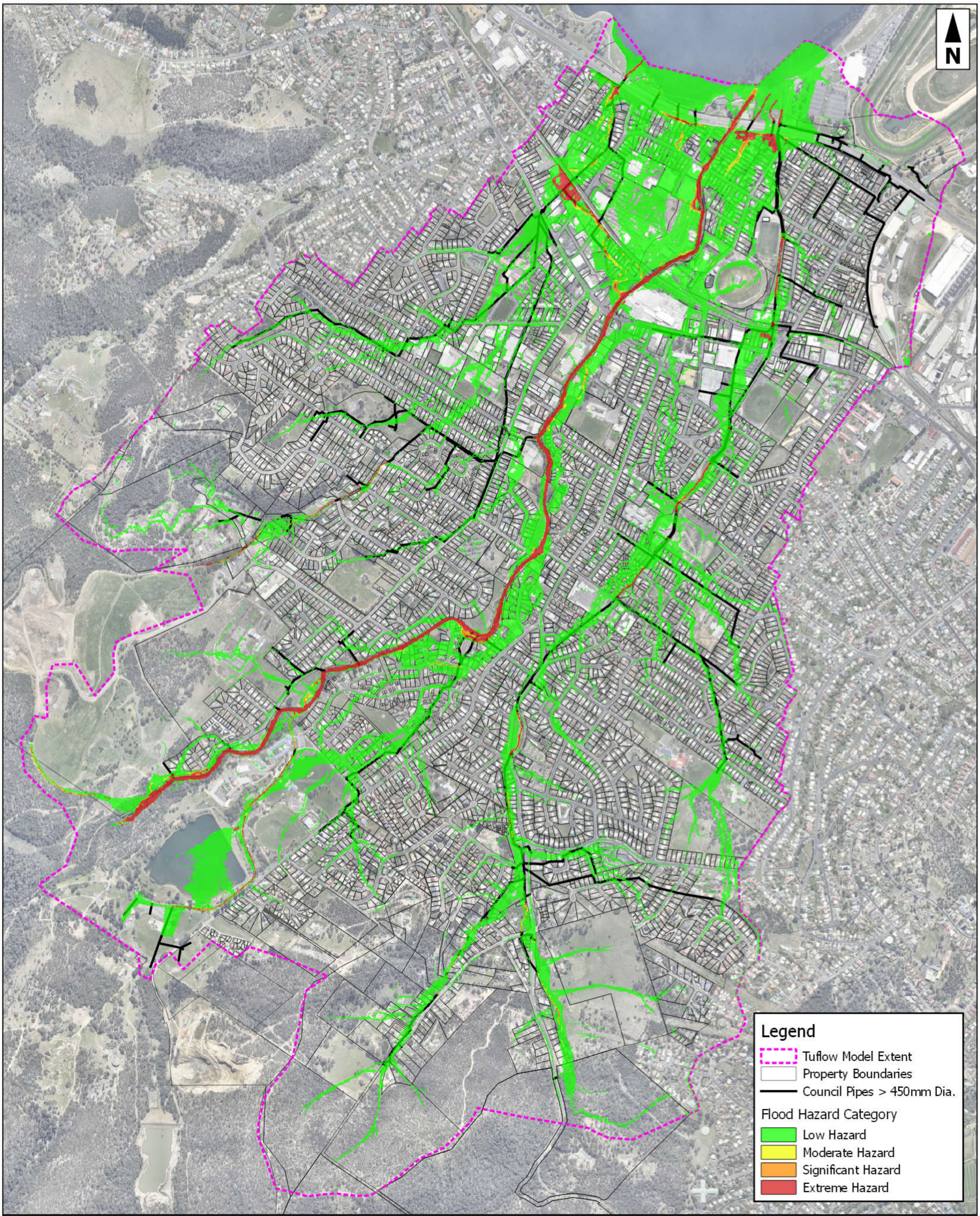

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AUTHOR:	H. Nguyen-Mallen	DATE:	16/11/2018	FIGURE:	30041618-E-de8	

FIGURE TITLE: 1 in 100 AEP DEVELOPED - DAMS FULL - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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PROJECT NO: 30041618 PROJECT TITLE: GLENORCHY CBD SSMP

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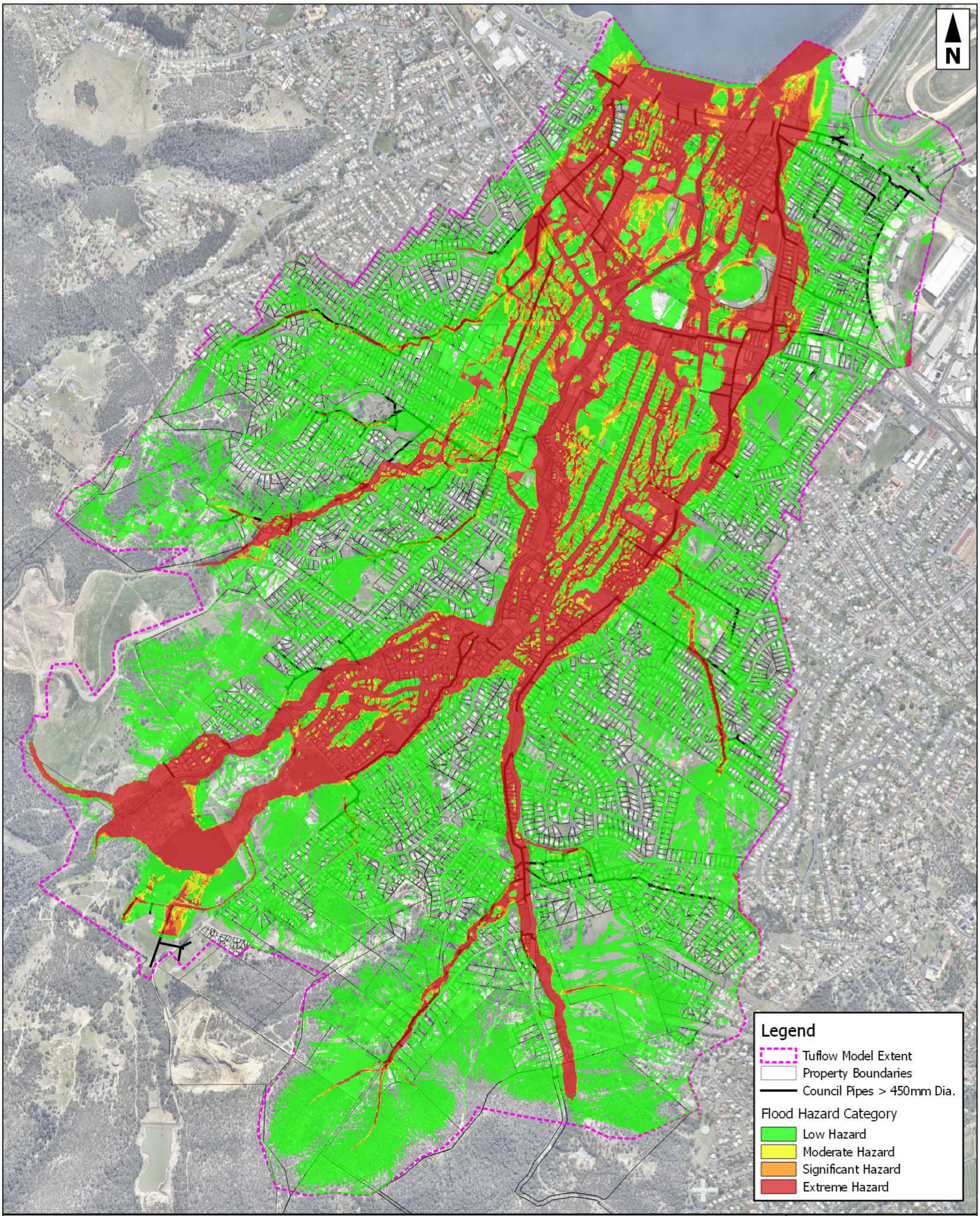

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AUTHOR:	H. Nguyen-Mallen	DATE:	16/11/2018	FIGURE:	30041618-E-de9

FIGURE TITLE: PMF DEVELOPED - DAMS FULL - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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PROJECT NO: 30041618 PROJECT TITLE: GLENORCHY CBD SSMP

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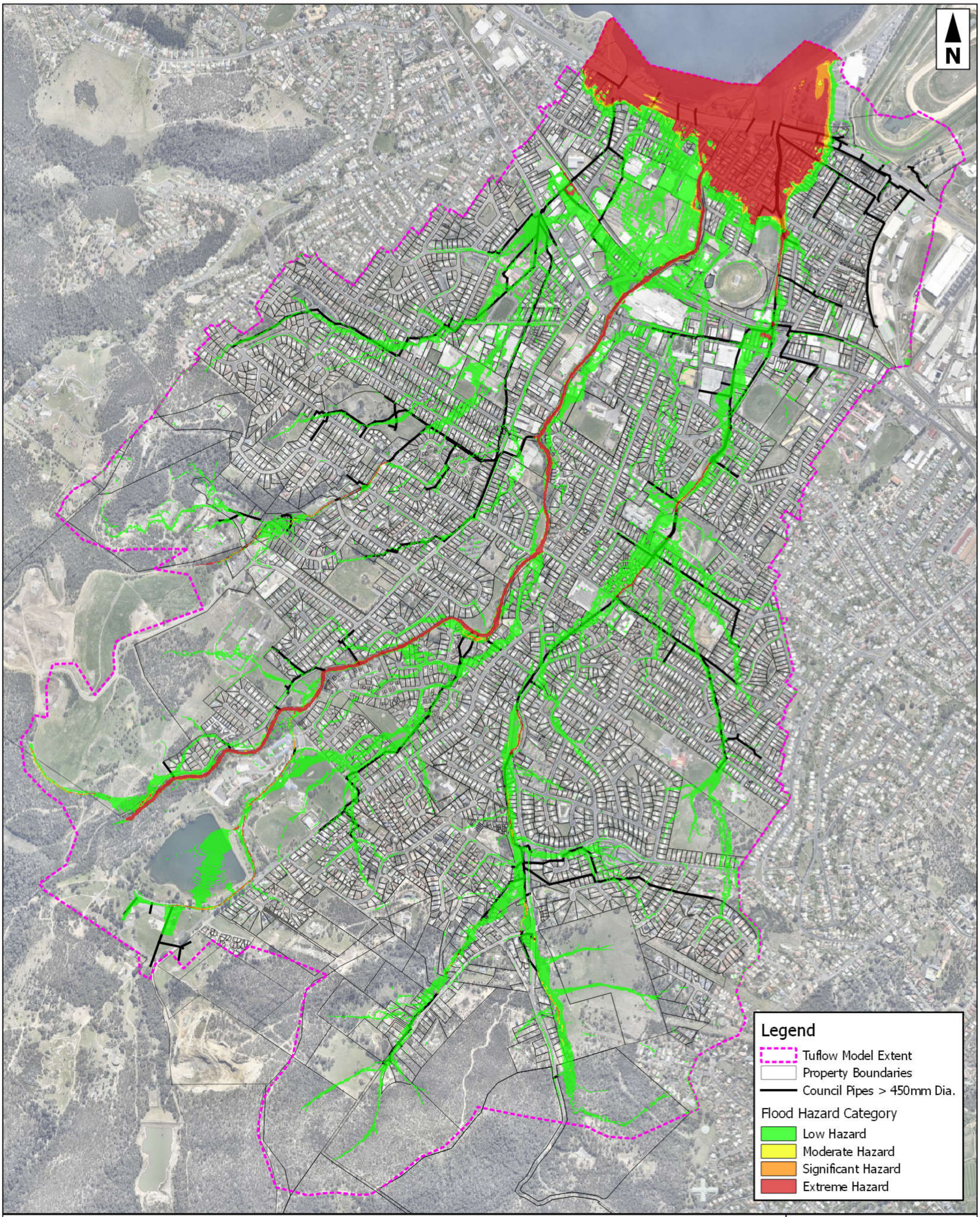

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CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 20 AEP DEVELOPED - CLIMATE CHANGE 1 - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55

PAGE SIZE: A3 SCALE: 1:13,000

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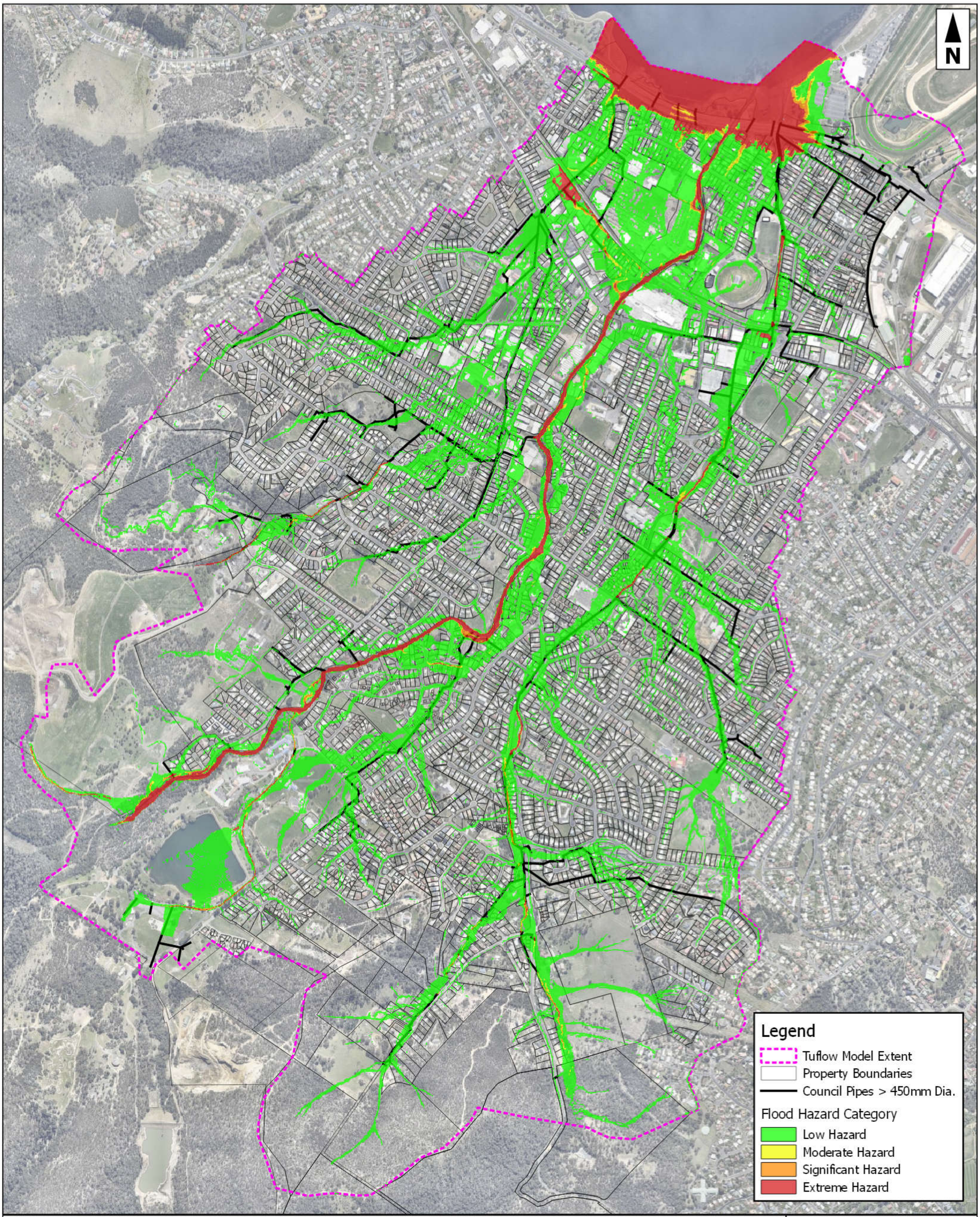

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AUTHOR: H. Nguyen-Mallen
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DATE: 16/11/2018
SOURCES: © NEARMAP 2018
FIGURE: 30041618-E-cc12
CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP DEVELOPED - CLIMATE CHANGE 2 - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
PAGE SIZE: A3
SCALE: 1:13,000

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PROJECT NO: 30041618
PROJECT TITLE: GLENORCHY CBD SSMP

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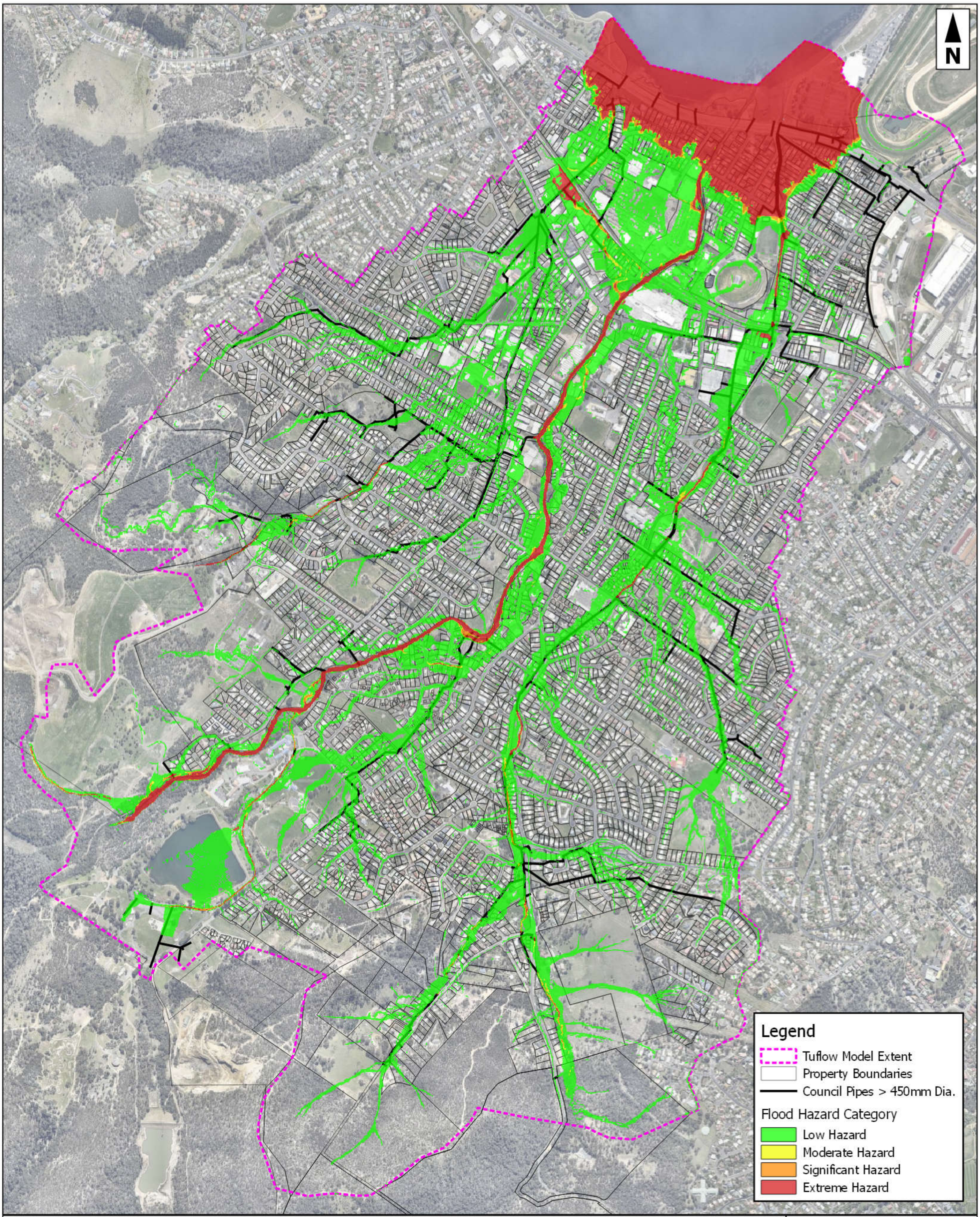

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AUTHOR: H. Nguyen-Mallen
DATE: 16/11/2018
FIGURE: 30041618-E-cc13

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP DEVELOPED - CLIMATE CHANGE 3 - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
PAGE SIZE: A3
SCALE: 1:13,000

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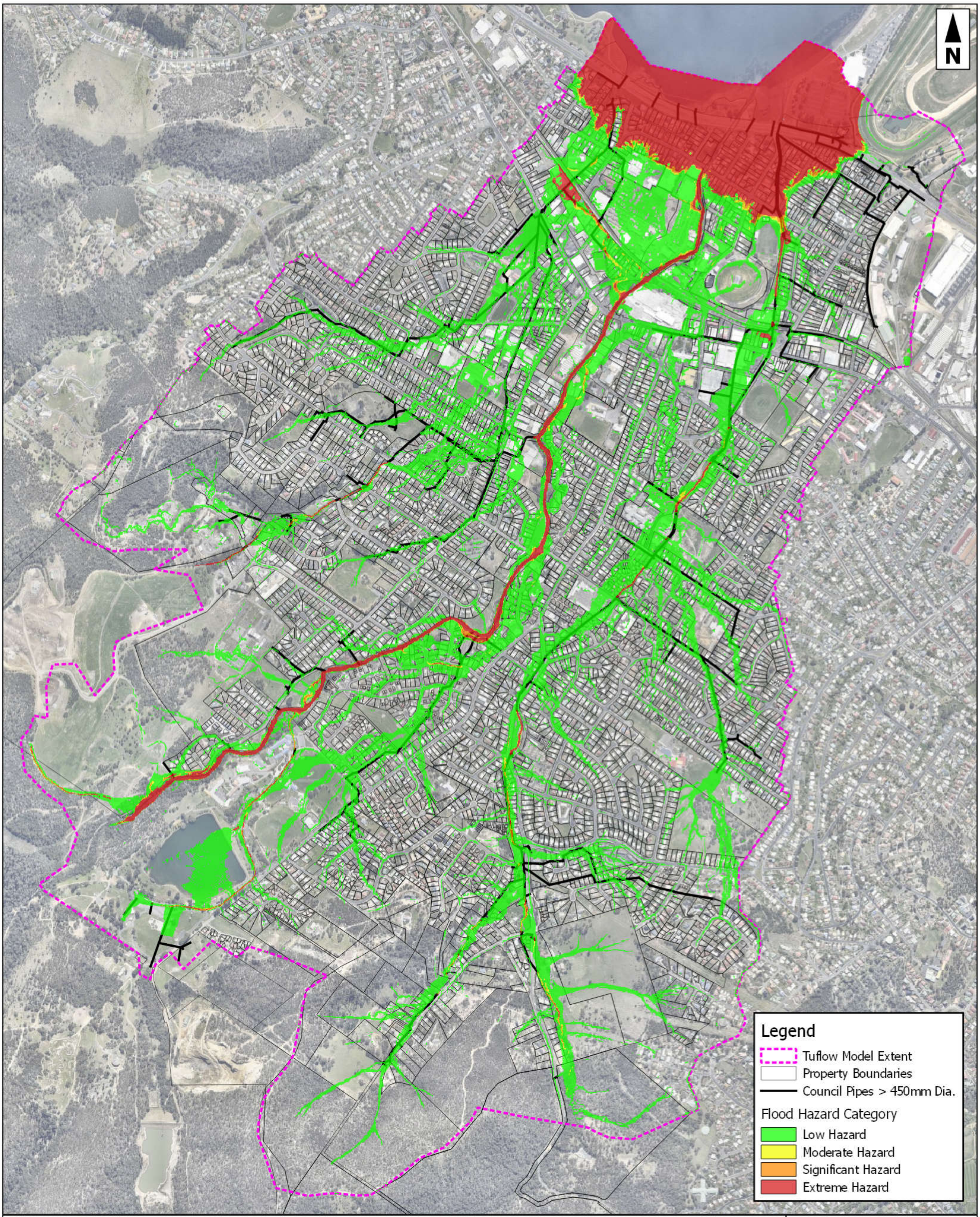

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AUTHOR: H. Nguyen-Mallen
STATUS: FINAL
DATE: 16/11/2018
SOURCES: © NEARMAP 2018
FIGURE: 30041618-E-cc14

CLIENT: GLENORCHY CITY COUNCIL

FIGURE TITLE: 1 in 100 AEP DEVELOPED - CLIMATE CHANGE 4 - Innundation Hazard (DV) Map

COORDINATE SYSTEM: GDA 94 / MGA ZONE 55
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File Location Name:	I:\Projects\30041618 - Glenorchy Stormwater Management Plan\03 Outputs\Reports\3 Final\Glenorchy CBD SSMP Report.docx
Project Name:	Glenorchy CBD Stormwater System Management Plan;
Project Number:	30041618p
Revision Number:	Final Rev. 1 Report

Revision History

Revision	Date	Prepared by	Reviewed by	Approved for Issue by
Final	31-10-2018	H. Nguyen-Mallen	T. Rhodes & S. van Hall	S. van Hall
Final Rev. 1	30-11-2018	H. Nguyen-Mallen	T. Rhodes	S. van Hall

Reviewer Register

Revision	Date	Approved for Issue by	Reviewed by
Final Rev. 1	30-11-2018	SMEC Australia S. van Hall	WMAwater Fiona Ling
Issuer Signature:	Sander van Hall 		Date: 30-11-2018
Reviewer Signature:			Date:

Issue Register

Distribution List	Date Issued	Number of Copies
Glenorchy City Council	31-10-2018	1 Electronic
SMEC Project File	31-10-2018	1 Electronic

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