JETTY RD REZONING – STAGE 2 FLOOD STUDY

EXISTING CONDITIONS REPORT

Version 6

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1. BACKGROUND

TGM Group has been engaged by Curlewis Bellarine Pty Ltd to prepare a combined Planning Scheme Amendment and Subdivision Application under Section 96A of the Planning and Environment Act 1987 for the rezoning and subdivision of the land at Coriyule Road, Curlewis. The properties included in this study are 32-70 McDermott Road and 91-125 Coriyule Road, Curlewis, however, this study also includes properties upstream and downstream from the subject site contained within the catchment.

The purpose of this report is to detail the hydrological characteristics of the proposed site and contributing catchment area and outline the methodology applied in the analytical process. The report discusses the methodology employed in the development of the hydrology model used to provide inputs into a 2D hydrodynamic model to predict the existing conditions flood extent.

This phase of the study will focus on generating a suitably detailed, and refined existing conditions flood model used to inform the proposed development layout and Site Stormwater Management Plan (SSMP), and to provide a 'base-case' for the analysis of the Flood Impact Assessment (FIA).

This study employed methods and data from Australian Rainfall and Runoff (ARR2016), including improved Intensity Frequency Duration (IFD) curves and the Regional Flood Frequency Estimator (RFFE) that are based on 30 years of additional rainfall and streamflow data; and ensembles of storm burst rainfall temporal patterns.

2. STUDY AREA

2.1 Site Context

The Coriyule Road study site, herein known as 'the site', consists of a number of existing properties within a Farming Zone (FZ) context. The site boundary and corresponding properties are depicted in Figure 2.1.

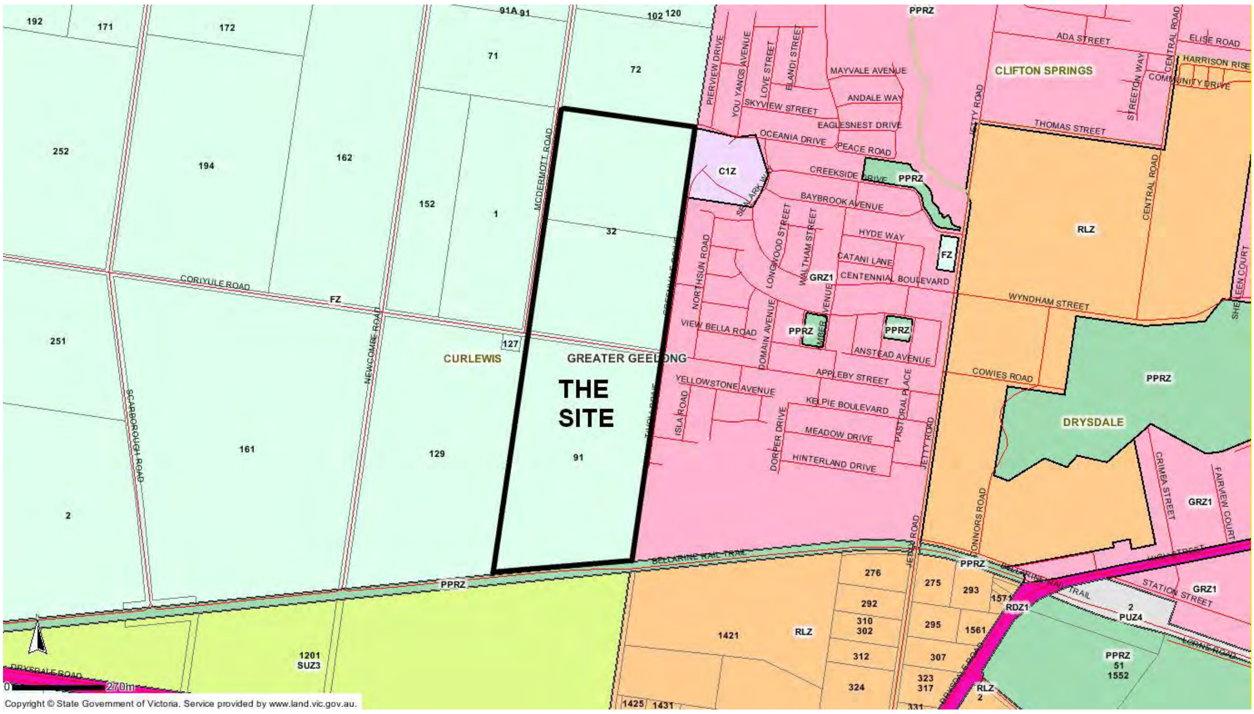

Figure 2.1: Coriyule Rd 'The Site' – Boundary and Existing Property Numbers

The site comprises a significant rural holding of approximately 51.91 hectares. It is located north of the Bellarine Rail Trail, and delimited by McDermott Rd on the west side and by Greenvale Drive/Tivoli Drive on the east side. The site is identified as part of a broader area as Stage 2 of the Jetty Road Urban Growth Area.

The site does not have any flood, land subject to inundation or special building overlays (FO, LSIO, SBO) associated with it under the planning scheme as depicted in Figure 2.2.

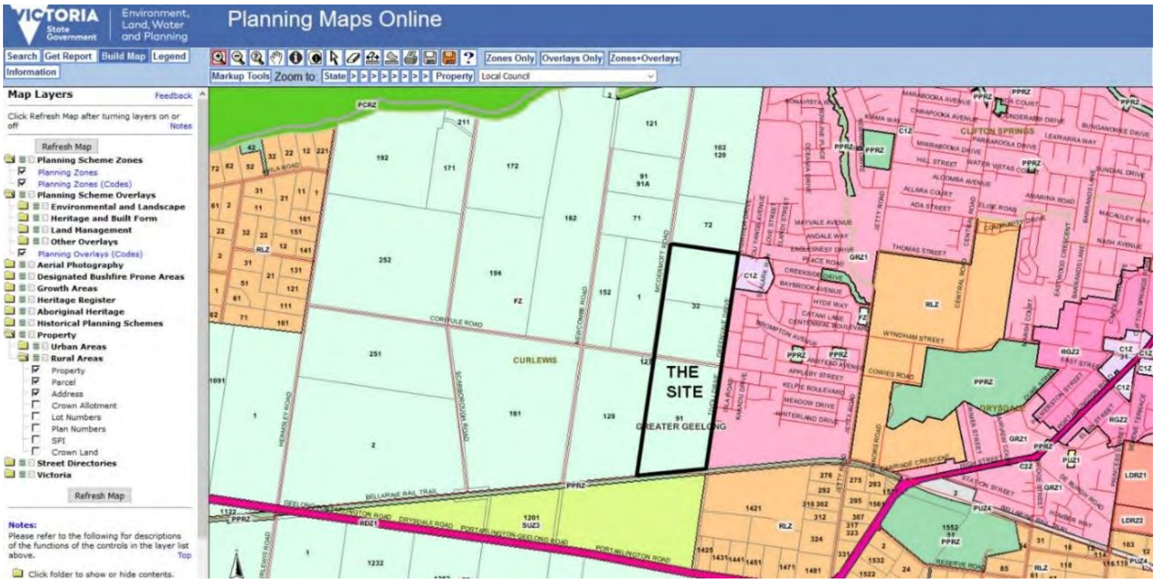

Figure 2.2: Stormwater overlays within proximity of the site[1]

However, it is identified that a designated waterway passes through a southern portion of the property. Historic rail infrastructure and road alignments, and limited existing stormwater drainage systems indicate stormwater drainage issues may occur, resulting in localised flooding during rarer storm events. A conceptual layout of the existing waterways in relation to the site are shown in Figure 2.3 on Page 9

[1] Sourced: Planning Maps Online, 24 July, 2017

Figure 2.3: Existing Waterways within proximity of the site[2]

2.1.1 Coriyule Road Drainage Upgrades

Significant upgrades to the underground drainage network along Coriyule Road have been recently undertaken by City of Greater Geelong[3] (CoGG). The upgrades are designed to convey the urban stormwater runoff from Bellaview Estate and Curlewis Parks (located immediately east of the site) generated during storm events up to and including the 1% AEP, to the Legal Point of Discharge (LPOD), located in the main un-named waterway between Newcombe Rd and unmade Scarborough Rd.

The underground drainage system is shown in Figure 2.4.

The Ø750 mm pipe has been designed to convey the stormwater flows from the detention basin located in the north catchment of the Curlewis Park Estate to the LPOD[4]. The culvert capacity is capable of conveying the 1% AEP stormwater flows, which are characterised by a peak discharge of 0.87 m^3/s according to stormwater management plan prepared for the Curlewis Park Estate[5] by SMEC.

[2] Sourced: VicPlan Online, 27 November, 2019
[3] City of Greater Geelong (June 2015). Proposed Drainage Improvements Coriyule Road, Curlewis. General Layout Plan & Longitudinal Section (CH 0-96).
[4] SMEC (May 2017). Curlewis Parks – Stage 11 _ City of Greater Geelong Roadworks and Drainage – Interim Northern Basin Layout Plan.
[5] SMEC (March 2019). Drainage strategy. Curlewis Park Estate (South) – Stormwater Management.

Similarly, the 1% AEP stormwater flows collected within the detention basin located on the south-west corner of Bellaview Estate are conveyed into the LPOD through a Ø1050 mm pipe located beneath Coriyule Rd embankment.

COGG provided underground drainage plans identifying the two pipes (Ø750 mm from Curlewis Parks and Ø1050 mm from Bellaview Estate) that connect to a Ø1200 mm pipe before the intersection between Coriyule Road and McDermott Road. Finally, the Ø1200 mm pipe splits into three-barrel Ø750 mm pipes immediately before the LPOD.

Figure 2.4: Underground Drainage System - Jetty Rd Growth Area Outfall

3. HYDROLOGY MODEL

The hydrology analysis was performed using the Innovyze software package XP-STORM applying the Laurenson routing technique. XP-STORM 2017 provides features to efficiently interface with the ARR Data Hub and Bureau of Meteorology (BOM) to obtain IFD and rainfall data to generate temporal patterns for a range of event probabilities.

The XP-STORM model applied ensemble rainfall patterns, storm burst loss factors and runoff estimation techniques from Australian Rainfall & Runoff 2016[6] to the study catchment area to generate runoff hydrographs and predict the volume of stormwater generated.

As detailed in ARR2016[7] the majority of hydrograph estimation methods used for flood estimation require a temporal pattern that describes how rainfall falls over time as a design input. Traditionally a single burst temporal pattern has been used for each rainfall event duration. The use of a single pattern has been questioned for some time[8] as the analysis of observed rainfall events from even a single pluviograph shows that a wide variety of temporal patterns is possible.

The hydrology model utilises outputs from the local XP-STORM model created for this study in addition to outputs from the previously undertaken Greater Bellarine Peninsula Model. The chosen outputs are at locations where catchments would have the potential to affect the hydraulics within the waterway downstream of Coriyule Road.

3.1 Sub-Catchment Delineation

TGM previously established a regional hydrological and 2D hydrodynamic model for the Greater Bellarine Peninsula covering a regional catchment area of 52.72 km² (Discussed further in Section 3.1.1 and shown in Figure 3.1). The above-mentioned regional model delineated catchments within the study area to a course level. However, utilisation of the regional model allowed for the assessment of possible cross-catchment flows, facilitating further delineation within the areas of interest.

For this study, a 6.14 km² sub-catchment study area (herein identified as "the Jetty Road sub-catchment") was identified between the Curlewis Golf Course and Port Phillip Bay.

The study area is located within three different COGG designated catchments: (1) the Coriyule Rd Creek catchment (COGG designation C042, 853.11 ha), (2) the Hermsley Rd/Scarborough Rd catchment (COGG designation C038, 603.67 ha) and (3) the Scarborough Rd/Newcomb Rd catchment (COGG designation C137, 5049 ha).

[6] Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors), Cardno.2020, Australian Rainfall and Runoff: A Guide to Flood Estimation, Commonwealth of Australia.
[7] Babister, M, Retallick, M, Loveridge, M, Testoni, I, and Podger, S, 2016. Temporal Patterns, Chapter 5 Book 2 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia
[8] Nathan, R.J. and Weinmann, P.E. (1995). The estimation of extreme floods - the need and scope for revision of our national guidelines. Aus J Water Resources, Volume 1(1), pp.40-50.

Initial delineation of the sub-catchments was carried out by applying a mathematical algorithm called TauDEM[9,10] (Terrain Analysis Using Digital Elevation Models). Delineation was further refined by reviewing the existing development construction plans which informed changes within the developed section of the catchment that have undergone significant construction works since the LIDAR flyover.

3.1.1 Greater Bellarine Peninsula Model.

As part of a previous project, TGM has established hydrologic and hydraulic models for the Greater Bellarine Peninsula. The models were developed using the ARR2016 processes, also discussed and used in this report. The models were generated to provide a high-level definition of flooding throughout the Bellarine Peninsula, from Leopold to Queenscliff, and from the Ocean to the Bay.

The Curlewis to Clifton Springs region (Including Curlewis, Drysdale & Clifton Springs) of the Greater Bellarine Peninsular Model was referenced in this study to define flooding around the study area allowing for assessment of possible cross-catchment flows, confirmation on contributing catchment areas and to evaluate critical storm durations.

The extent of the Curlewis to Clifton Springs Sub Model is shown in Figure 3.1, Note it includes sections of the Mannerim and Wallington/Ocean Grove model catchments. These catchments discharge in a South to South-Easterly direction and do not enter the study site.

[9] http://hydrology.usu.edu/taudem/taudem5/index.html.
[10] Tarboton, D. G., (1997), "A New Method for the Determination of Flow Directions and Contributing Areas in Grid Digital Elevation Models," Water Resources Research, 33(2): 309-319.

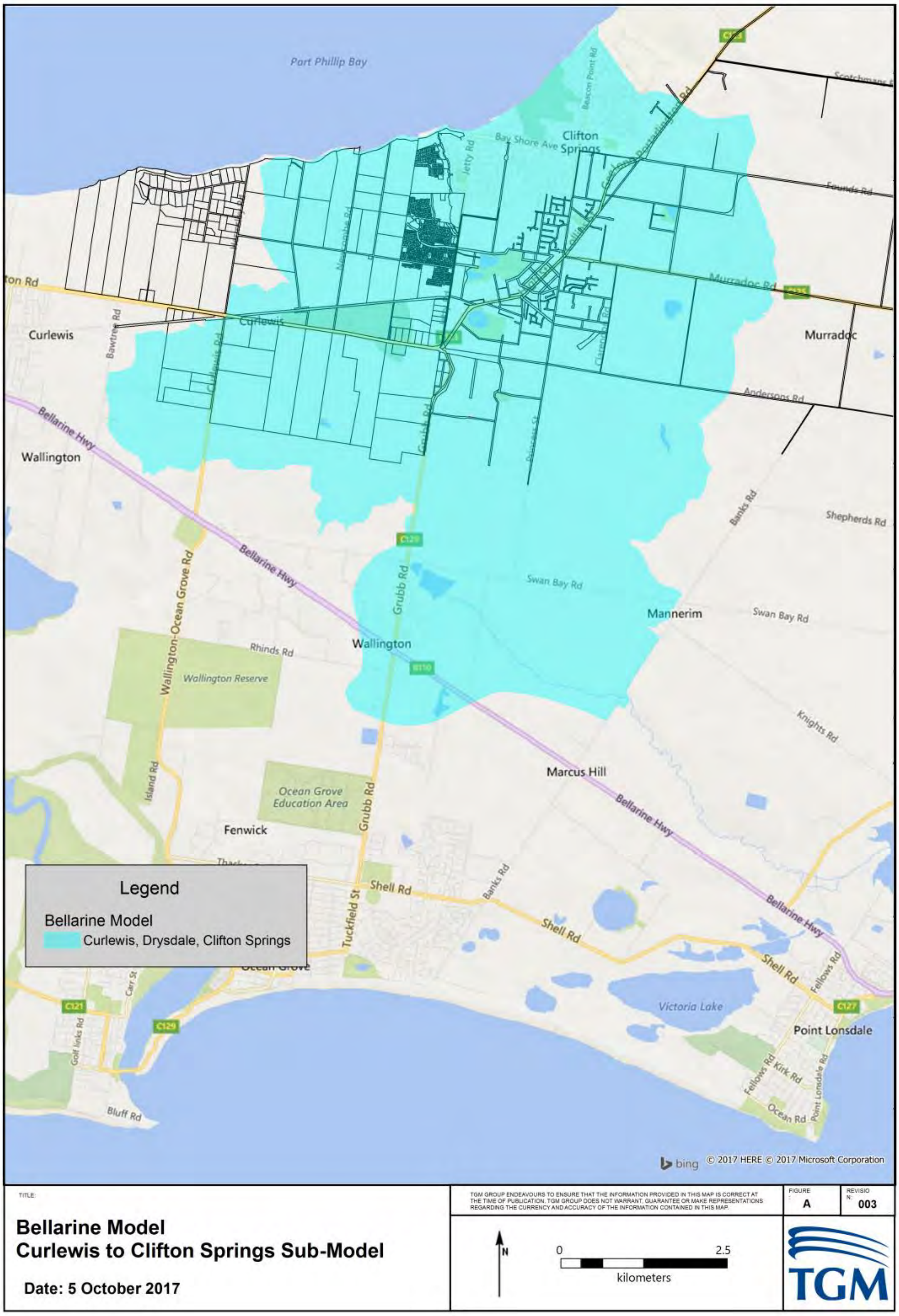

Figure 3.1: Bellarine Peninsula Model (Curlewis - Clifton Springs Sub Model)

3.1.2 TauDem

TauDEM is a suite of Digital Elevation Model (DEM) tools for the extraction and analysis of hydrologic information from topography as represented by a DEM. The algorithm has been developed by the Hydrology Research Group led by Prof. David Tarboton at the Utah State University (USA).

Please refer to Section 4.2 for a detailed description of the DEM used in this study.

TauDEM provides the distinctive advantage of applying an objective technique to calculate the stream flow paths and directions, the contributing areas using both single and multiple flow direction methods, as well as to delineate the watersheds and sub-watersheds draining to each stream segment.

A comparison between the overland flow paths defined using TauDEM and the underground pipe drainage network has been performed in the urban area to better delineate the sub-areas.

The 1% AEP stormwater runoff is conveyed through overland flow paths defined by kerb and channel, swales and local terrain grades which have been calculated using TauDEM in the area of interest.

In addition, the definition of the sub-catchments has been carried out by taking into account the soil cover properties (i.e., urban and rural areas) in order to obtain homogeneous sub-catchments.

The sub-catchment definition performed using TauDEM and subsequent construction documentation review is shown in Figure 3.2.

Where TauDEM identified additional contributing areas within COGG designated sub catchment (Section 3.1 above), appropriate additional areas were included within the hydrologic model, specifically in catchments 38.24 & 38.23.

Catchments directly contributing to the nominated LPOD (detailed in Section 2.1.1 and Figure 2.4) at the outlet of the Coriyule Road drainage line to the designated waterway are highlighted in pink. Catchments that affect the hydraulics within the waterway downstream of Coriyule Road are highlighted in blue and catchments highlighted in green do not contribute to the flooding in the area of interest. The characteristics of the sub-catchments have been summarised in Table 3.1.

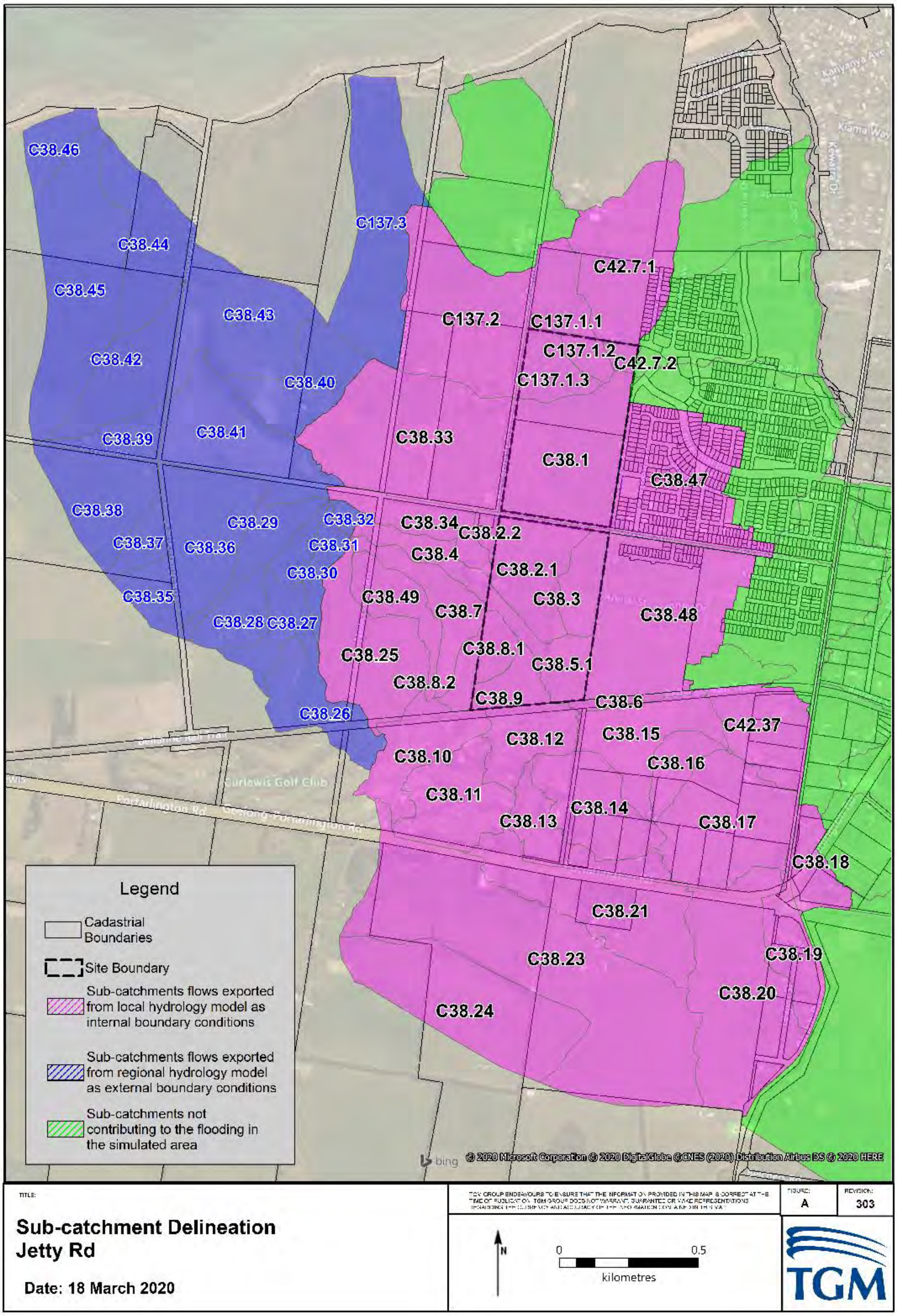

Figure 3.2: Sub-catchment Delineation for the Study Area

Table 3.1: Sub-Catchment ID and catchment area

Sub-Catchment ID	Area (ha)	Area (km²)	Sub-Catchment ID	Area (ha)	Area (km²)
C137.1.1	5.887	0.059	C38.24	22.431	0.224
C137.1.2	2.073	0.021	C38.25	14.866	0.1487
C137.1.3	5.009	0.050	C38.26	8.637	0.0864
C137.2	21.851	0.219	C38.27	5.525	0.0553
C137.3	20.801	0.2080	C38.28	4.980	0.0498
C38.1	19.036	0.190	C38.29	5.300	0.0530
C38.2.1	9.938	0.099	C38.30	2.579	0.0258
C38.2.2	1.127	0.011	C38.31	3.037	0.0304
C38.3	4.794	0.048	C38.32	0.953	0.0095
C38.4	4.107	0.041	C38.33	28.316	0.2832
C38.5.1	5.209	0.052	C38.34	6.539	0.0654
C38.6	1.29	0.0129	C38.35	4.002	0.0400
C38.7	6.979	0.0698	C38.36	13.675	0.1368
C38.8.1	2.620	0.0262	C38.37	3.011	0.0301
C38.8.2	2.165	0.0217	C38.38	11.639	0.1164
C38.9	3.820	0.0382	C38.39	4.259	0.0426
C38.10	9.118	0.091	C38.40	9.794	0.0979
C38.11	10.887	0.109	C38.41	15.920	0.1592
C38.12	7.167	0.072	C38.42	10.546	0.1055
C38.13	7.275	0.073	C38.43	17.092	0.1709
C38.14	11.202	0.112	C38.44	12.156	0.1216
C38.15	10.538	0.105	C38.45	24.534	0.2453
C38.16	4.053	0.041	C38.46	4.180	0.0418
C38.17	17.812	0.178	C38.47*	19.890	0.1989
C38.18	5.143	0.051	C38.48*	24.910	0.2491
C38.19	6.561	0.066	C38.49	5.944	0.0594
C38.20	25.290	0.253	C42.37	9.66	0.0966
C38.21	17.750	0.178	C42.7.1	18.375	0.1838
C38.23	49.474	0.495	C42.7.2	1.016	0.0102

** Catchments further divided / refined due to existing Jetty Road growth area and associated stormwater infrastructure*

3.1.3 Refinement of the Existing Jetty Road Development

Further refinement of the TauDEM catchments within the area of interest (the existing Jetty Road development) were required due to the large-scale earthworks, addition of detention basins and other recent infrastructure constructed between the 2007 LIDAR flyover and present.

The refinement was undertaken using available civil design plans (supplied in part by Council) for the Jetty Road growth area.

The refinement to catchments C38.47 and C38.48 can be seen in Figure 3.2 below. The characteristics of the sub-catchments have been summarised in Table 3.2.

Table 3.2: Summary - Characteristics of the sub-catchments

Sub-Catchment ID	Area (ha)	Area (km^2)
C38.47.1	11.688	0.117
C38.47.2	7.246	0.072
C38.47.3	1.230	0.012
C38.48.1	15.881	0.159
C38.48.2	5.590	0.056
C38.48.3	1.079	0.011
C38.48.4	3.433	0.034

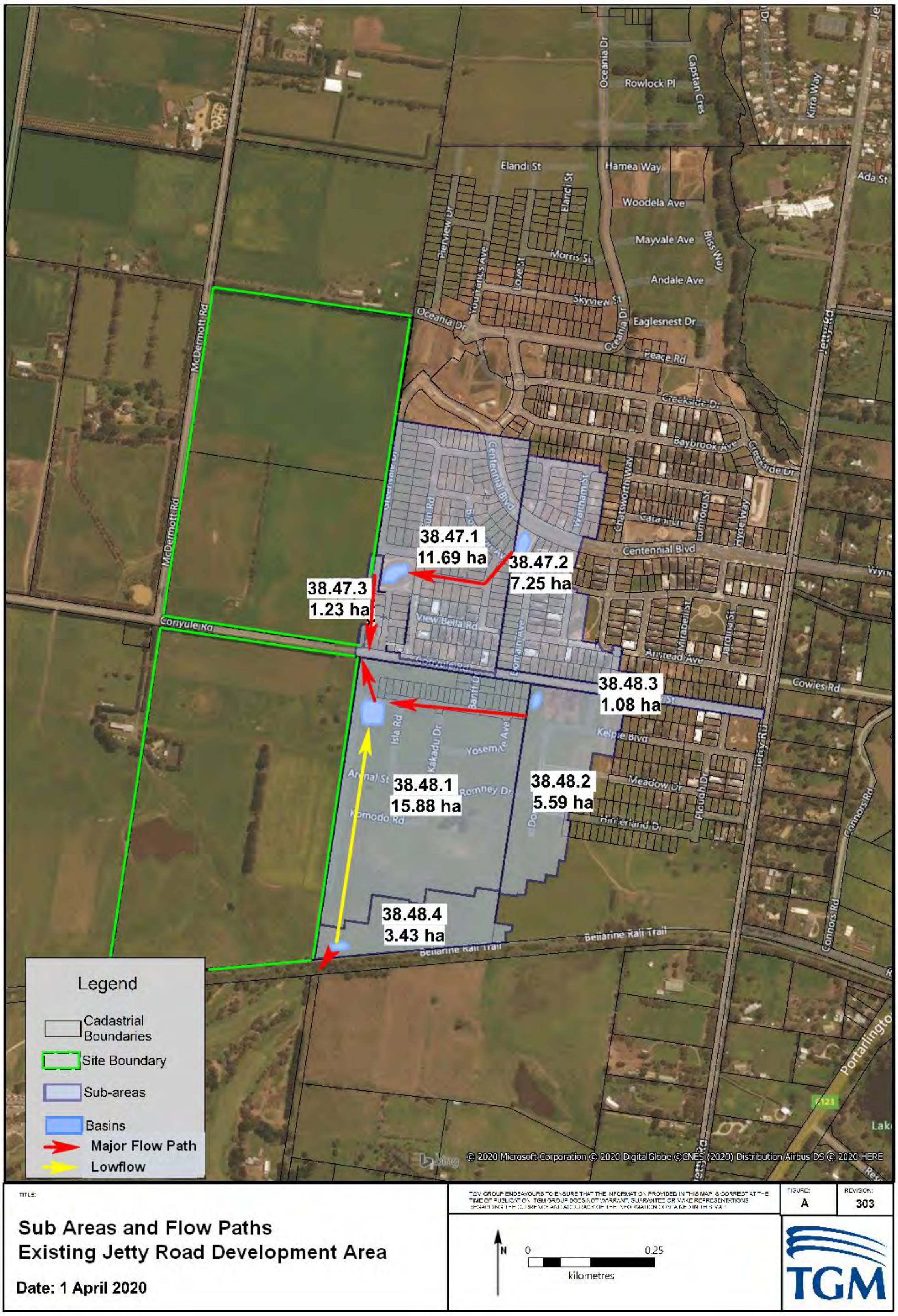

Figure 3.3: Jetty Road Developemnt Catchment Plan

3.2 Set up of the Hydrology Model

The hydrology model was set up using XP-STORM 2017.1.1. A series of nodes and links were defined to represent the 1D hydrological network connecting each sub-catchment, as represented in Figure 3.4.

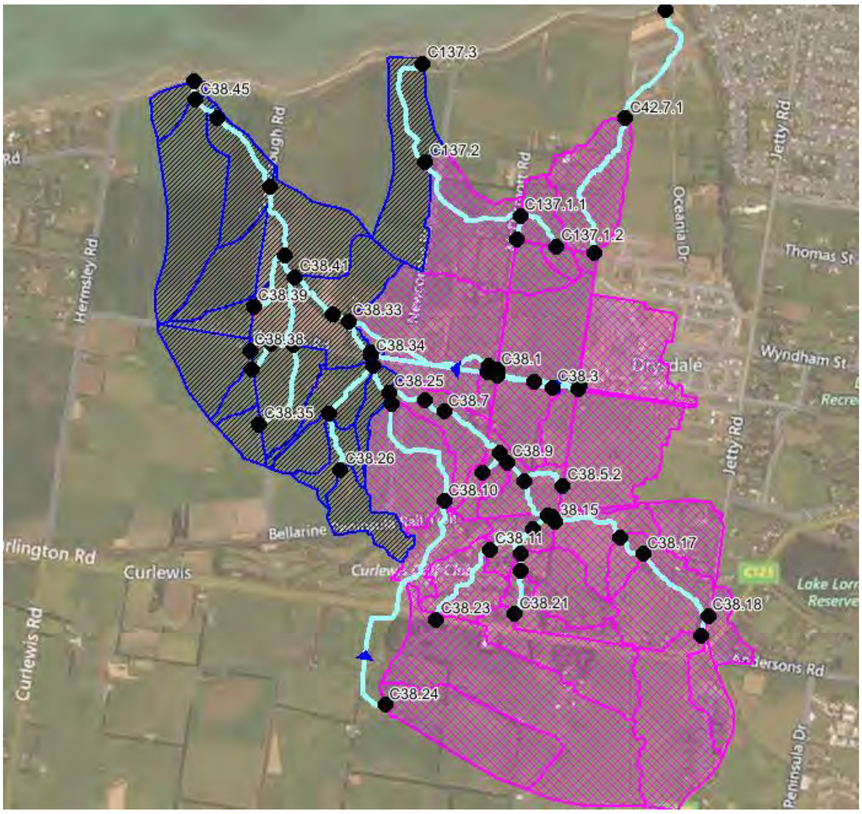

Figure 3.4: Hydrology Model Network

Two catchment node categories were represented in the model – stand-alone Catchment Nodes and combined Catchment/Network Nodes.

Both sub-catchment node classifications contain the sub-catchment hydrologic characteristics (including flow path length) required to define the stormwater runoff at the outlet of each sub-catchment. The principle difference is the Catchment/Network node allows routing of upstream sub-catchments through the node.

Both the Catchment and Network Nodes have been identified by considering the locations where sub-catchment flows enter the hydrology network, and the confluences between flow paths.

Where

3.2.1 Undeveloped Catchment Links

Centrelines of links within the undeveloped catchment have been imported directly into XP-STORM. Natural links were then draped over the DEM to define the geometric profiles of the 1D network. An average slope profile of each reach was selected to represent the longitudinal link parameters. Cross section profiles were also created through draping of cross sections over the DEM with further manual review to select a shape profile that best represented the individual reaches.

Validation of the selected link properties was further carried out through comparison of the 1D and 2D results as discussed in Section 5.2

3.2.2 Existing Jetty Road Development Catchment Links

Due to the multiple detention basins and various other civil infrastructure present in the Jetty Road catchment, additional catchment delineation and a supplementary link methodology was required for use in the existing conditions model.

Links within the developed Jetty Road catchment were included between all detention basins. The parameters of these links were informed by design plans for the underground drainage network and road network, and SSMPs where detailed design plans had not yet been completed. Underground links were also provided for the major underground drainage network discharging to Corryule Road from Bellaview Estate (North) and Curlewis Parks Estate (South).

3.3 Hydrology Model Parameters

3.3.1 Permeability and Fraction Impervious

A fraction impervious percentage was assigned to the sub-catchments to reflect the actual permeability of the local soil. The fraction impervious for each sub-catchment has been determined on the basis of the planning zone classification (refer to Figure 3.5) according to the Melbourne Water guidelines[11] summarised in Table 3.3.

Table 3.3: Typical Impervious Fraction Range for Planning Scheme Zones[11]

Zone	Fraction Impervious Range
Rural Living Zone	0.10 – 0.30
General Residential (601 to 1,000 m²)	0.50 – 0.80
General Residential (300 to 600 m²)	0.70 - 0.80
Low Density Residential (> 1,001 m²)	0.10 – 0.30
Farm Zone	0.00 – 0.05
Special Use Zone	0.50 - 0.80

The fraction impervious range identified in Table 3.3 was only used as a starting guide. Visual inspection of aerial imagery was carried out to determine an appropriate value to adopt for each sub-catchment.

It is worth noting that a great portion of the Jetty Rd sub-catchment is located in the Farming Zone (FZ) area, where minimal impervious surfaces have been detected. The small amount of impervious area that exists within the catchment is generally non-connected in nature resulting in the fraction impervious being set to zero.

Bellaview Estate (C38.47) and Curlewis Parks (C38.48) were allocated a fraction impervious of 80%.

The Curlewis Golf Course situated along the southern boundary is classified as Special Use Zone 3 (SUZ3) which, according to the Melbourne Water Guidelines, should have a weighted fraction impervious range between 50% – 80%. The actual area of the golf course within the Jetty Road sub-catchment is entirely pervious (with the exception of unconnected impervious areas such as club rooms, outbuilding etc) and was therefore attributed a fraction impervious of 0%.

The south-east corner of the Jetty Rd sub-catchment is located in the Rural Living Zone (RLZ), with a fraction impervious range characterised by a lower limit of 0.10 (i.e., 10%) according to Table 3.3. However, from a visual inspection of the Nearmap image dated 28 January 2017, a fraction impervious much smaller than 10% has been observed. Therefore, a fraction impervious of zero has been adopted for the sub-catchments classified as RLZ.

A general overview of the planning zones present within the study area is provided in Figure 3.5.

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[11] Melbourne Water *MUSIC Tool Guidelines*. https://www.melbournewater.com.au/. Accessed 30 March 2017

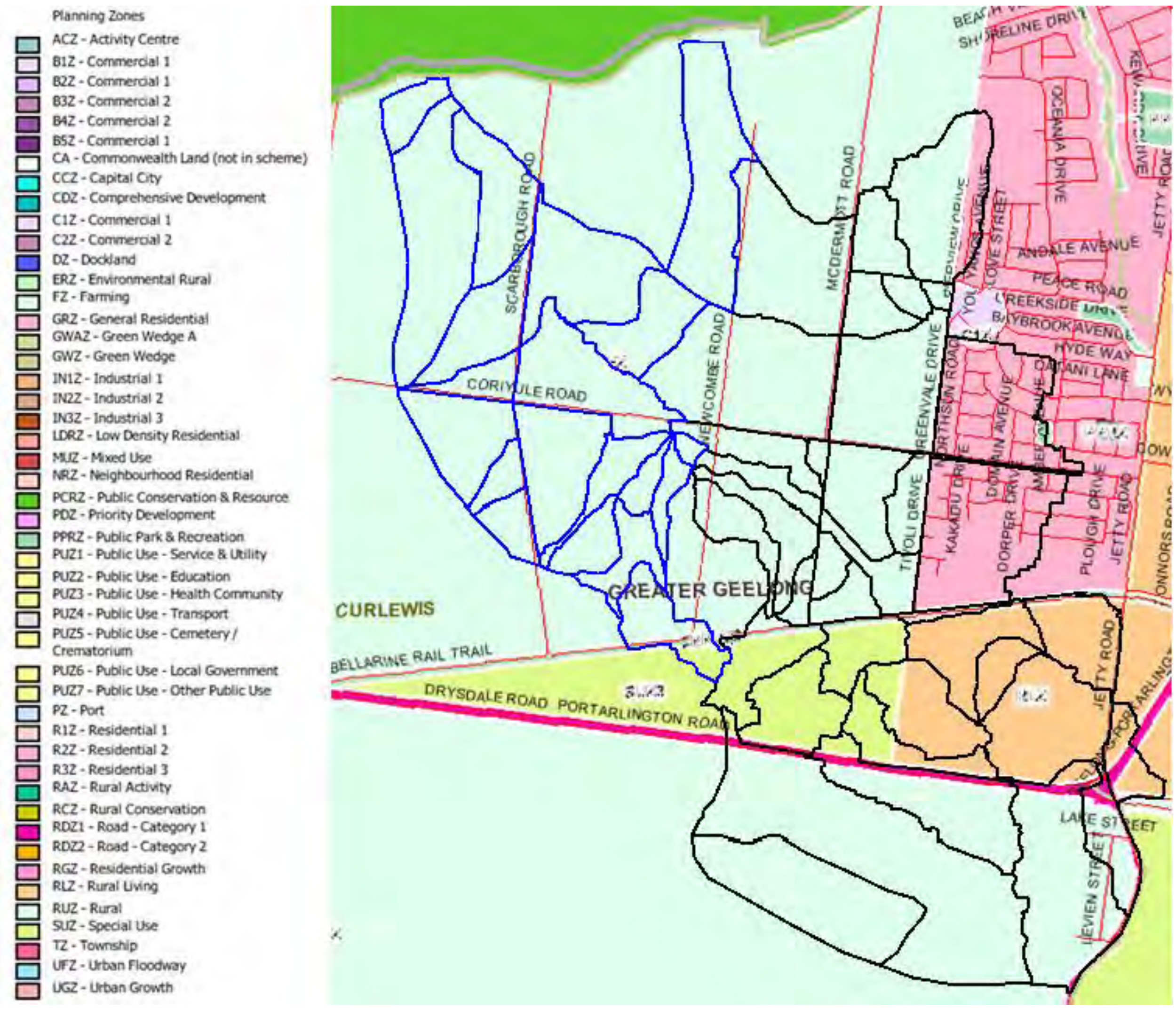

Figure 3.5: Planning Zones - Hydrology Model Study Area

3.3.2 Loss Parameters

XP-STORM was run as an Initial Loss and Continuing Loss (IL/CL) model using parameters provided from the ARR Data Hub[12].

The ARR Data Hub is a tool which utilises all the research of the updated ARR2016 methodology to provide design inputs for modelling. The ARR Data Hub uses prediction equations to define the IL/CL parameters for all of Australia.

The Prediction equations used to develop the recommended loss values utilised attributes from the Australian Water Resource Assessment – Landscape (AWRA-L) model system which was developed by CSIRO and the Bureau of Meteorology[13].

The full storm IL/CL parameters from the ARR Data Hub are shown in Table 3.4 for pervious surfaces.

Table 3.4: Hydrological Pervious Surface Loss Parameters

Source	Initial Loss (mm)	Continuing Loss (mm/hr)
ARR Data Hub	19	3

The ARR2016 parameters derived using the loss prediction equations and the AWRA-L model were adopted for this analysis in conformance with the ARR2016 flood estimation methodology and processes applied in this study.

It is noted that the identified ARR initial losses reflect the full storm IL values and should only be applied to hydrology models that are running full storm patterns.

The following study analysed storm burst pattern ensembles, therefore, the storm initial loss (IL_s) nominated in Table 3.4 was adjusted to account for the impact of pre-burst rainfall to create a burst initial loss (IL_b) using the following simple equation –

$$IL_s - \text{Pre-Burst} = IL_b \qquad \text{[equation 1]}$$

ARR2016 states[14] that in locations and for durations that do not have significant pre-burst, the pre-burst depth can be ignored when applying temporal patterns. Therefore, the Burst IL (IL_b) can be taken as the Storm IL (IL_s).

The Pre-Burst depth for the Jetty Road study catchment in Curlewis can be read off the thematic map shown in Figure 3.6.

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[12] http://data.arr-software.org/

[13] Ball, J, and Weinmann, E, 2016, Flood Hydrograph Estimation, Chapter 3 Book 5 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia

[14] Babister, M, Retallick, M, Loveridge, M, Testoni, I, and Podger, S, 2016. Temporal Patterns, Chapter 5 Book 2 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia

The study catchment is located within the Bellarine Peninsula rain shadow and as such has a pre-burst depth within the range 5-10 mm. It can be deduced that the study catchment does not experience 'significant' pre-burst rainfall.

It has been assumed, for this study, that pre-burst rainfall depths within the study catchment are 5 mm, resulting in an IL_b of 14 mm. Selection of the lower limit pre-burst rainfall from the ARR recommendations ensures a conservative adopted storm initial loss within the hydrology model.

The hydrologic losses adopted in this study are summarised in Table 3.5.

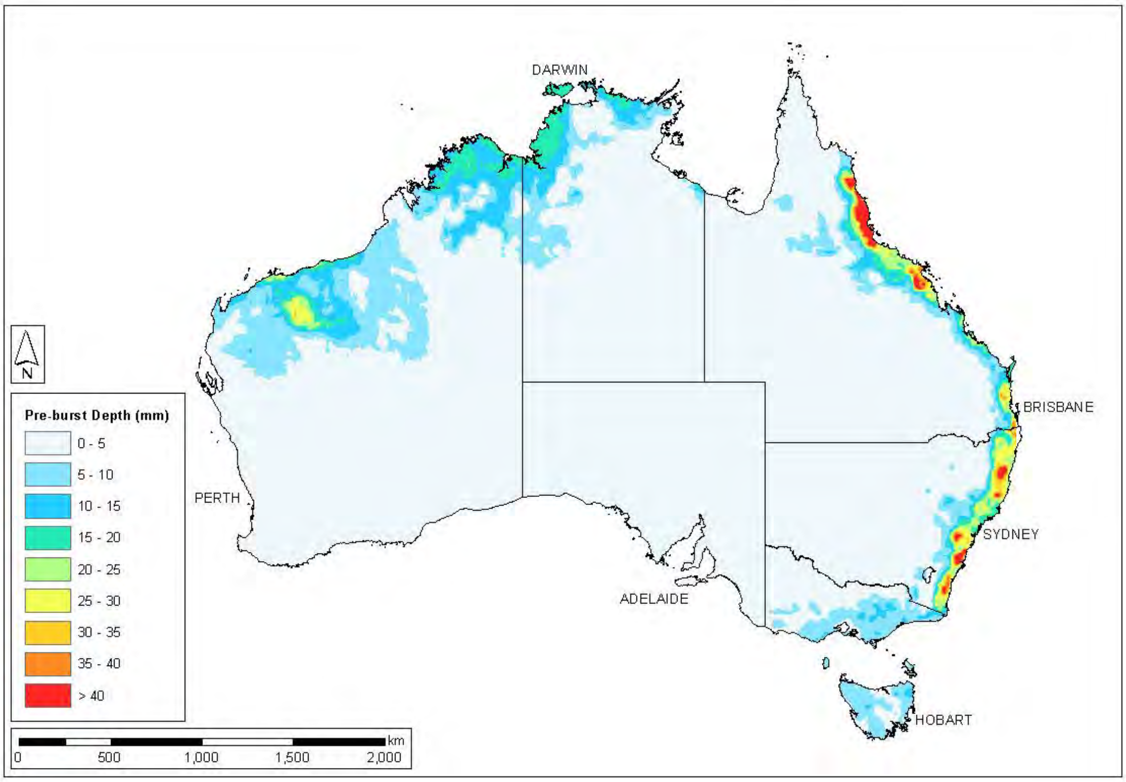


Figure 3.6: Pre-burst Rainfall Depth (mm) - [*ARR2019 figure 2.5.10*]

Table 3.5: Adopted Hydrological Loss Parameters

Surface	Storm Initial Loss (mm)	Pre-burst Depth (mm)	Adopted Losses	
			Burst Initial Loss (mm)	Continuing Loss (mm/hr)
Pervious	19	5	14	3
Impervious	0		0	0

It should also be noted that loss parameters provided by the ARR data hub have recently been the subject of a review in New South Wales and are currently under review in Victoria, however at the time of commencing this study, and throughout the study revision process, the ARR2016 data hub values are considered the best available information. Validation of the adopted modelling parameters are discussed further in Section 3.7

3.3.3 Manning's Roughness Coefficients

In the hydrology model, all sub-catchments are also characterised by Manning's 'n' coefficients, which describe the hydraulic roughness properties of the soil surfaces.

The Manning's coefficients adopted in this study are summarised in Table 3.6.

Table 3.6: Manning's Coefficients 'n' adopted in the Hydrology Model

Surface	Manning's Coefficients 'n'
Pervious	0.03 – 0.05
Impervious	0.018

The Manning's coefficients have been determined through the model calibration process (please refer to Section 3.7 for a detailed description of the adopted calibration process). Note: Mannings roughness coefficients are also applied in the hydrodynamic model (please refer Section 4.4).

3.4 Temporal Patterns

This study uses fixed temporal patterns over the entire catchment for design flood estimation and does not include spatial variation.

For this study, the Bureau of Meteorology's 2016 IFD data and ARR2016 temporal patterns were used to produce an ensemble of storm burst patterns which were analysed for a whole catchment response.

3.5 IFD Data

The 2016 rainfall intensity frequency duration (IFD) climatic data used in the hydrology model was extracted from the Bureau of Meteorology (BOM) website[15] 18 May 2017.

The IFD curves are shown in Figure 3.7, below.

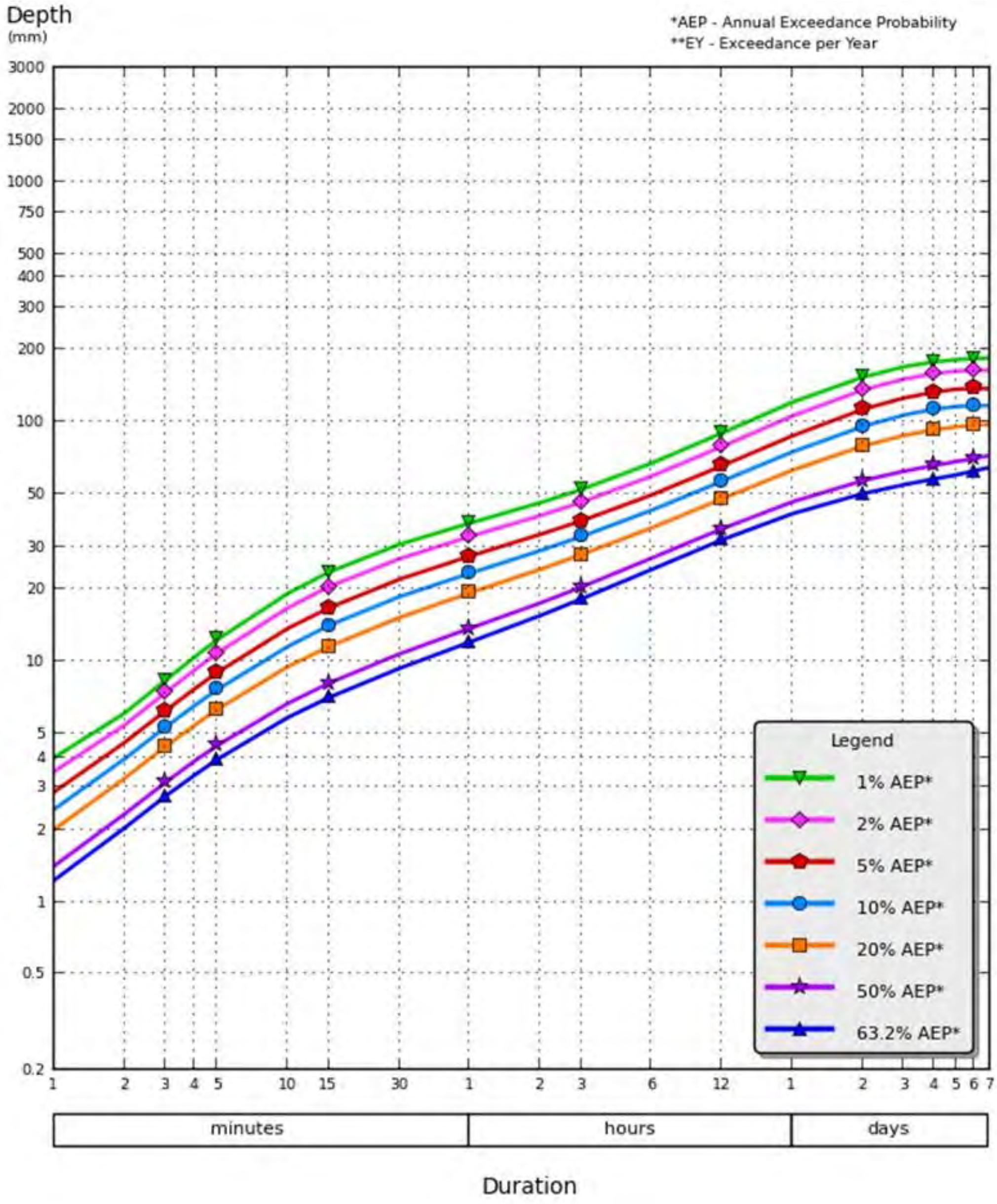


Figure 3.7: 2016 IFD Curves – Bureau of Meteorology 18 May 2017

15 http://www.bom.gov.au/water/designRainfalls/

Note:

- *The 50% AEP IFD does not correspond to the 2-year Average Recurrence Interval (ARI) IFD. Rather it corresponds to the 1.44 ARI.*
- *The 20% AEP IFD does not correspond to the 5-year Average Recurrence Interval (ARI) IFD. Rather it corresponds to the 4.48 ARI.*

3.6 Ensemble Storm Burst Patterns

The historical process of using peak flows derived from a single critical storm burst does not account for the hydrology processes generated by the reality of complete (full volume) storms.

For this analysis, 10 storm burst temporal patterns were extracted for 24 duration periods, for the 1%, 10% and 50% AEP events.

A total of 240 storm burst patterns were analysed for the above mentioned AEPs. The analysed events and durations are shown in Table 3.7.

The median value of the peak discharges generated for 10 temporal patterns has been calculated. The critical temporal pattern has been selected by identifying the temporal pattern characterised by the peak discharge closest to the median for each of the 240 durations. The procedure has been then repeated for each event probability.

The procedure described in this Section has been applied to both the calibration process and the existing conditions simulations.

Table 3.7: Analysed Rainfall Patterns, Durations and Events

Number of Storm Burst Patterns in Ensemble (per event duration)	Storm Durations Analysed (minutes)			Event Probability Range Analysed (AEP)	
				(%)	(1 in *x*)
10	10	120	1800	1	100
	15	180	2160	10	10
	20	270	2880	20	5
	25	360	4320	50	2
	30	540	5760		
	45	720	7200		
	60	1080	8640		
	90	1440	10080		

3.7 Model Calibration and Validation

The parameters of hydrological models are usually determined through a calibration procedure to optimise the model performances in relation to a specific site.

The most used calibration procedure in rainfall-runoff hydrological models involves the comparison between observed and computed data. In the calibration phase, hydrology model parameters are adjusted to attain an output that matches the observed data.

Model validation is defined as the process of demonstrating that a given site-specific model is capable of making accurate predictions[16]. In hydrology, the validation process is carried out by applying the calibrated hydrological model to a different real event without changing the model parameters, and compare the computed and observed data.

In the absence of gauged data, both calibration and validation are not possible. However, ARR2016 provide the Regional Flood Frequency Estimation (RFFE) tool, which allows practitioners to calibrate their hydrological models when working with ungauged catchments.

A key element of the calibration process is the identification of the stormwater catchments that impact on the study area, the characteristics of those catchments and the configuration of waterways.

Factors such as availability of observed rainfall data, soil type, soil conditions, land use and local knowledge were considered in this investigation.

The calibration has been undertaken to the 10% AEP event identified using the ARR2016 Regional Flood Frequency Estimation (RFFE) tool for the Jetty Rd Sub-catchment.

The model calibration was setup to reflect rural or pre-development catchments to allow the comparison with flood estimation techniques. More in detail, for calibration purposes only, all sub-catchments have been considered as rural, and the pervious fraction has been set at 100%.

The calibration procedure has been performed by changing the Manning coefficient 'n' for the pervious surfaces to better represent the energy losses of the site at hand, as summarised in Table 3.8.

Table 3.8: Calibration Conditions - Surface Characteristics

Catchment	Area (ha)	Pervious Area (ha)	Impervious Area (ha)	Pervious **Manning's *'n'***
Jetty Rd Sub-catchment	621.2	621.2	0.00	0.03-0.05

A proper validation process was not possible for this study as no observed, historic data or locally adopted loss parameters are available for comparable catchments in the vicinity of the subject site. Therefore, a verification of the model predictability has been carried out by comparing the calibrated XP-STORM model to the RFFE predicted peak discharges for the 1%, 2%, 5%, 10%, 20% and 50% AEP events.

16 Refsgaard, J.C. 1996, Parametrisation, calibration and validation of distributed hydrological models – Journal of Hydrology. Danish Hydraulic Institute.

3.7.1 ARR2016 Regional Flood Frequency (RFFE) Model

The three catchments which define the area of interest (i.e., the Coriyule Rd Creek catchment, the Hermsley Rd/Scarborough Rd catchment and the Scarborough Rd/Newcomb Rd catchment) are ungauged catchments, therefore an at-site Flood Frequency Analysis (FFA) was not possible for the study catchments. Regional Flood Frequency Estimation (RFFE) techniques were required for the study catchments, applying a data-driven approach, in order to transfer flood characteristics from a group of gauged catchments to the study catchments.

The ARR2016 RFFE model[17,18] available online at http://arr.ga.gov.au/; was used to provide peak flow estimates for the study catchment. These were then used to calibrate the peak flow whole catchment response hydrographs generated by the ensemble rainfall patterns within the XP-STORM model.

There are 15 gauged regional catchments, surrounding the site, which made up the sample group for the statistical analysis used in the Jetty Road RFFE model.

The RFFE model interface, input parameters and statistical outputs can be seen in Appendix A. The RFFE model provides peak flood estimates for rural catchments, therefore, for the validation process the study catchment was considered to be undeveloped (pre-development).

Table 3.9: RFFE Model - Estimated Peak Discharge Targets for the Jetty Rd Sub-catchment

Event AEP (%)	Catchment	Area (ha)	Area (km²)	RFFE Discharge (m³/s)
63	Jetty Rd Sub-catchment (pre-development)	614.09	6.14	-
50				2.51
20				4.48
10				6.10
5				7.91
2				10.6
1				13.0

[17] Rahman. A, et al (2013). New Regional Flood Frequency Estimation (RFFE) Method for the whole of Australia: Overview of progress. Paper. Flood plain conference 2013.
[18] Rahman, A, Haddad, K, Kuczera. G and Weinmann, E, 2016, Peak Flow Estimation, Chapter 3 Book 3 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia.

3.7.2 Ensemble Storm Burst Pattern Selection

Using the XP-STORM model an ensemble of 240 storms was analysed for each probability storm event within the Jetty Rd Sub-catchment. For the calibration process, the study catchment was set up as a rural catchment with no impervious surfaces.

Using the burst initial loss and continuing loss identified in Table 3.5, and known catchment characteristics, i.e. area, slope, overland flow path profiles, etc. the model was run using all 240 storm burst patterns for each AEP.

The Median, Average, Maximum and Minimum peak flow output hydrographs were identified for each storm duration using a statistical spreadsheet. The results are presented in a box and whiskers plot in Figure 3.8.

ARR2016 states that the temporal pattern that represents the worst (or best) case should not be used by itself for design. Testing has demonstrated that on most catchments large number of events in the ensemble patterns are clustered around the mean and median[19].

Evaluation of the box and whisker plots shown in Figure 3.8 indicates that peak flows within Jetty Road sub-catchment occur during events with critical durations of less than 30 hours.

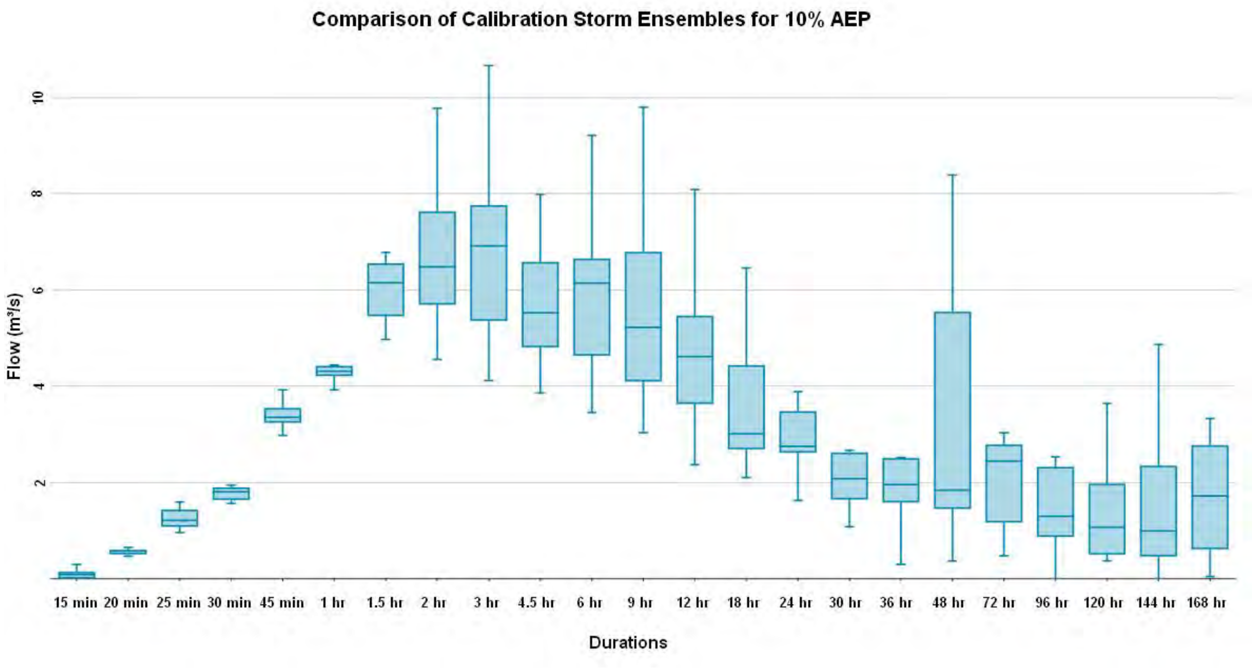


Figure 3.8: Calibration Conditions - Temporal Pattern Box and Whiskers Plot – 10% AEP Event

[19] Babister, M, Retallick, M, Loveridge, M, Testoni, I, and Podger, S, 2019. Temporal Patterns, Chapter 5 Book 2 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia

3.7.3 Calibration Summary

The comparison between the peak discharges generated with the calibrated XP-STORM model and the estimated RFFE model for the Jetty Rd sub-catchment are summarised in Table 3.10 and shown in Figure 3.9.

The hydrological parameters defined by the catchment characteristics, were capable of generating discharges within an acceptable range of the predicted RFFE discharge targets for all event probabilities. Calibration using the calibration factor (Bx) in XP-STORM was not required for this study.

Table 3.10: Jetty Rd Sub-catchment – Peak Discharges

Event AEP (%)	Catchment	Area (km²)	RFFE Discharge (m³/s)	XP-STORM 6 hr duration Discharge (m³/s)
50	Jetty Rd Sub-catchment (pre-development)	6.14	2.51	1.30
20			4.48	3.28
10			6.10	6.14
5			7.91	8.39
2			10.60	11.09
1			13.00	14.03

Initial calibration/validation for the site was centred around the 10% AEP 6 Hr duration event. Comparison to this event was informed by regional modelling discussed in Section 3.1.3, in which it was identified that peak flows with an exceedance probability of 10% AEP were exhibited during events of around 6 hours duration. Outputs from this study discussed in Section 5 and Figure 5.2 confirms the suitability of calibration to this event or events of similar duration.

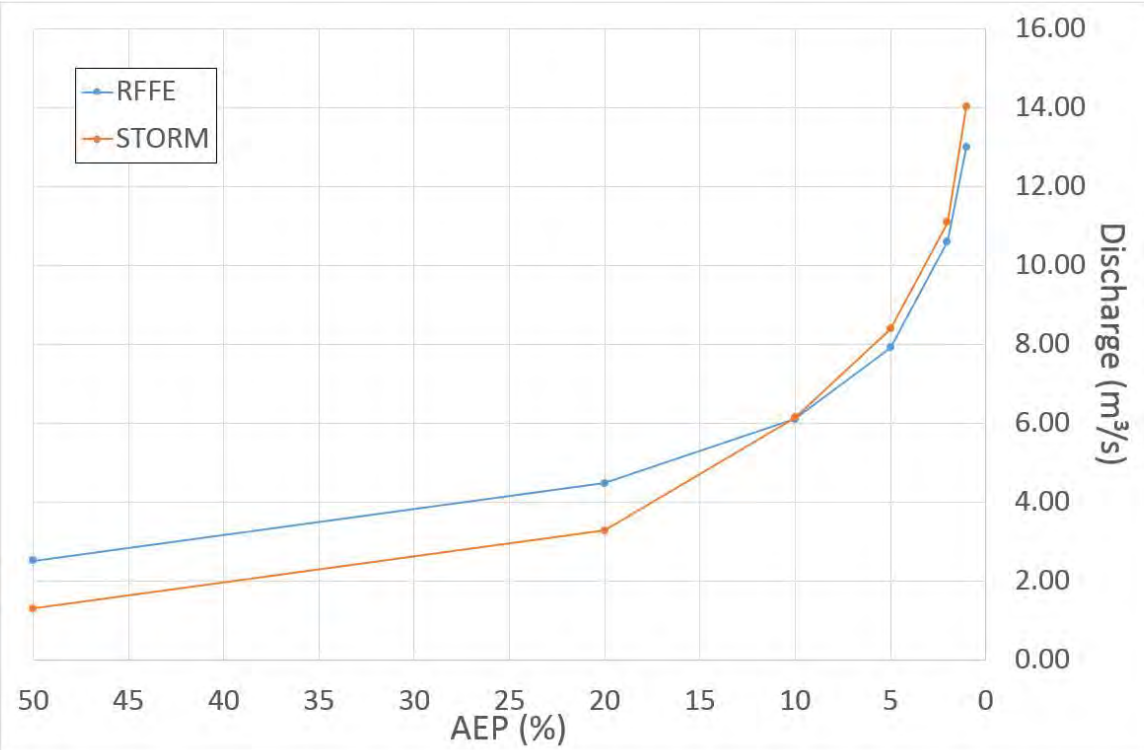


Figure 3.9: Estimated Peak Discharge - XP-STORM vs RFFE

3.7.4 RFFE Accuracy Considerations

A RFFE technique essentially represents a 'transfer function' that converts predictor variables to a flood quantile estimate. It is assumed that the use of a limited number of predictor variables (e.g. catchment area and design rainfall intensity) combined with an optimised transfer function captures the general nature of the rainfall-runoff relationship for flood events and hence provides flood quantile estimates of 'acceptable' accuracy.

ARR2016 identified ongoing concerns about estimation of parameter values (such as runoff co-efficient and time of concentration) that are the basis of using the Probabilistic Rational Method[20].

The use or application of the Probabilistic Rational Method, including the VicRoads variant, is no longer supported or recognised in ARR2016 as being a suitable RFFE technique[21,22].

All RFFE techniques are subject to uncertainty, which generally, is likely to be greater than for at-site Flood Frequency Analysis when a good quality and long record of streamflow data set is available at the location of interest.

The RFFE model estimates of regional flood frequency included substantial error bounds and are considered to be a best estimate of rarer events that cannot be described in the ungauged catchment.

It should be noted that loss parameters provided by the ARR data hub have recently been the subject of a review in New South Wales and are currently under review in Victoria, however at the time of this study and throughout the study revision process the ARR2016 data hub values are still considered the best available information.

3.8 Hydrology Model Simulations

3.8.1 Existing Conditions

The calibrated XP-STORM model was updated to reflect the Existing Conditions, by integrating the impervious surfaces and urban characteristics. It is noted that the majority of impervious surface area is attributed to sub-catchments – C38.47 and C38.48 (and their subsequent further division) representing the Bellaview Estate and Curlewis Park development areas, respectively.

The impervious area applied to the 6.14 km² overall catchment in this study is shown in Table 3.11

[20] Coombes P.J., Babister M., and McAlister A., (2015), *Is the Science and Data underpinning the Rational Method Robust for use in Evolving Urban Catchments.* 36th Hydrology and Water Resources Symposium, Engineers Australia, Hobart.
[21] Rahman, A, Haddad, K, Kuczera. G and Weinmann, E, 2019, Peak Flow Estimation, Chapter 3 Book 3 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia
[22] Coombes, P, Babister, M, McAlister, T, 2015, Is the Science and Data underpinning the Rational Method Robust for use in Evolving Urban Catchments, Conference Paper. Hydrologic Water Resource Symposium.

Table 3.11: Existing Jetty Rd Sub-catchment - Surface Characteristics

Sub-catchment	Area (ha)	Pervious Area (ha)	Impervious Area (ha)	Pervious **Manning's 'n'**	Impervious **Manning's 'n'**
Jetty Road	614.093	575.336	38.758	0.03 – 0.05	0.018

3.8.1.1 Existing Stormwater Infrastructure

Bellaview and Curlewis Park Estates have functional stormwater mitigation facilities designed to manage the impact of the existing development on stormwater discharges.

The stormwater detention facilities (5 basins) were modelled in the 1D XP-Storm Model as per design specifications for each Estate, with the outlet structure details updated to represent the built condition as per data supplied by COGG. Discharge hydrographs were extracted from the 1D model and integrated into the 2D hydrodynamic model as inflow boundary conditions, where the downstream most facilities discharge to the existing drainage network in Coriyule Rd. Refer to Figure 3.10.

The Curlewis Park drainage strategy by SMEC (North and South Basin, 7 March 2016)[23], TGM Group civil design drawings (005148-203, 006158-02 Revision D, 006178-208)[24] and stage-storage computations for the Bellaview Estate design were used to define the stormwater drainage systems and operational characteristics within the XP-Storm model.

The basins input into the 1D model and their data source are described below;

Table 3.12: Jetty Road Stage 1 – Existing Basin Infrastructure

Basin Figure Reference	Basin Name	Estate Name	Storage Data Source	Outflow Structure Data Source
005148-203	Brampton Avenue Basin	Curlewis Park Estate – Stage 3	TGM Drawings 005148-203 Rev02	Council Survey Information
006158-02	Greenvale Basin	Bellaview Estate – Stage 1	TGM Drawings 006158-02 RevA	Council Survey Information
006179-208	Appleby Basin	Curlewis Park Estate – Stage 8	TGM Drawings 006179-208 RevB	TGM Drawings 006179-208 RevB
SMEC North		Curlewis Park Estate – Stage 13	SMEC Drawings 1938E-13-70 RevA	SMEC - Curlewis Park Estate (South) – Stormwater Management 7/3/2016
SMEC South		Curlewis Park Estate – Stage 16	SMEC Drawings 1938E-16-70 RevC	SMEC - Curlewis Park Estate (South) – Stormwater Management 7/3/2016

[23] DRAINAGE STRATEGY Curlewis Park Estate (South) – Stormwater Management (2019). SMEC
[24] Bellaview Estate - STAGE 1. Civil Design Drawings 006158-02 Revision D. TGM Group 2011

The basins peak discharge at the Corryule Road interface is detailed in Table 3.13, below.

Table 3.13: Existing Estate detention basin discharges

Event (AEP)	Discharge (m^3/s)	
	North of Corryule Rd - Bellaview Estate	South of Corryule Rd - Curlewis Parks Estate
1%	1.386	0.84
20%	0.513	0.41

The location of the 5 basins can be seen in Figure 3.10.

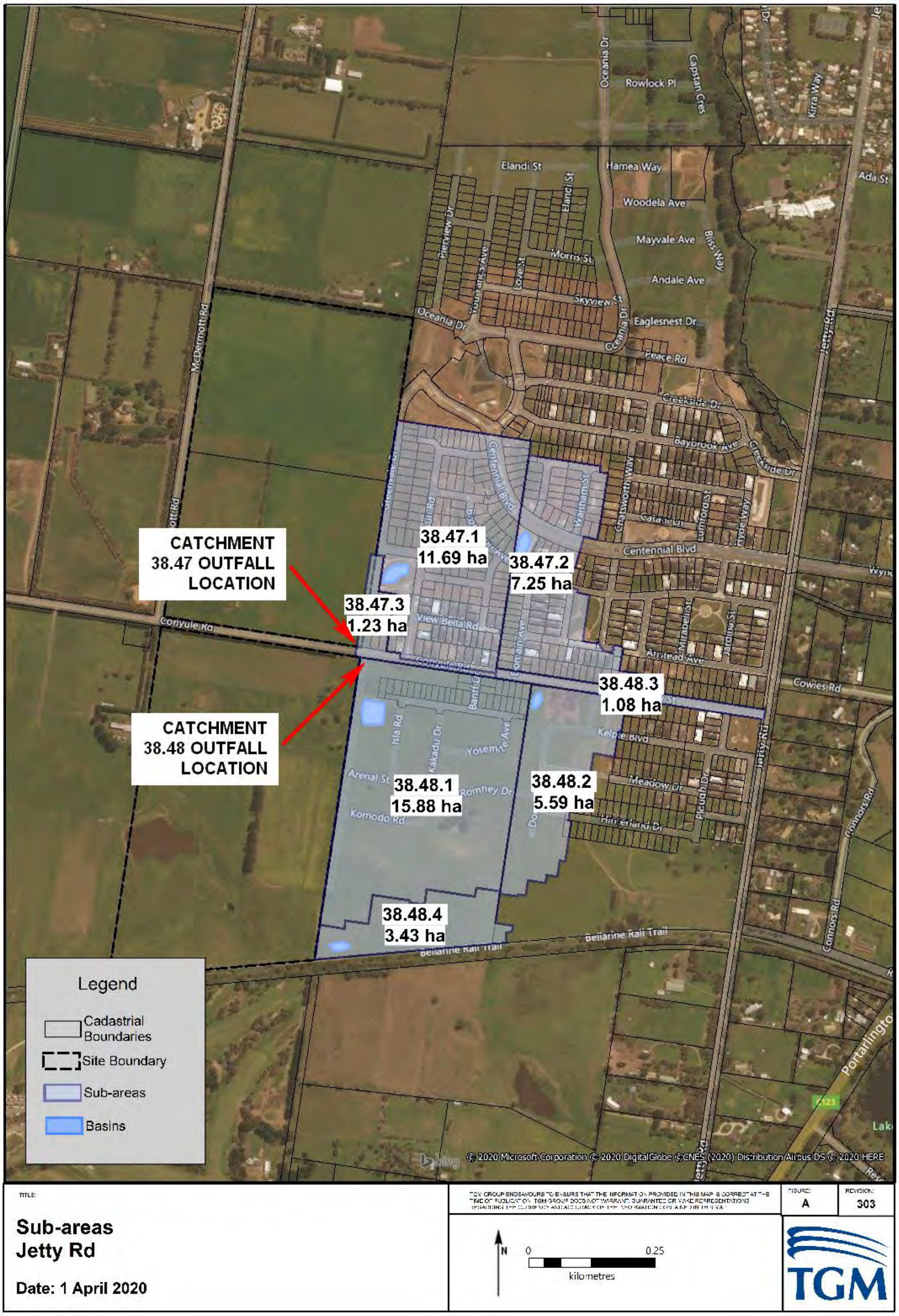


Figure 3.10: Basin Locations

3.8.1.2 Temporal Pattern Selection

The full 240 storm burst pattern ensemble was simulated within the Existing Conditions XP-STORM model for each AEP. The median storm burst patterns were identified for each of the 24 durations analysed for each event probability within the XP-STORM model as described in Section 3.6. The XP-STORM model was setup as a network model to account for hydraulic (1D) routing throughout the catchment. The XP-STORM existing model can be seen in Figure 3.11.

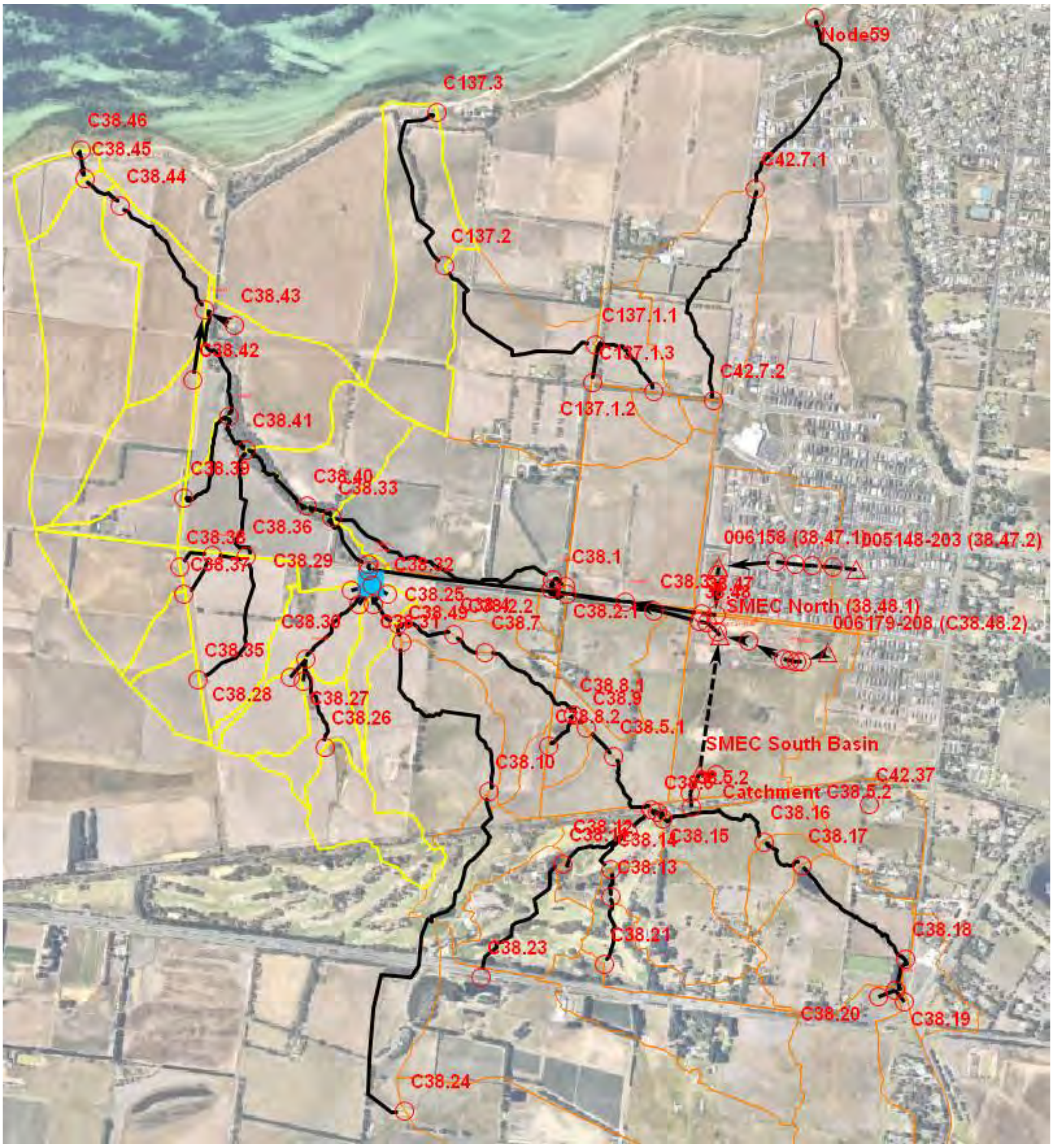


Figure 3.11: XP-STORM Model

The storm patterns closely resembling the median were selected for each duration. A box and whiskers plot can be seen in Figure 3.12 and Figure 3.13 for the 10% and 1% events, respectively.

Note: catchment discharges were taken at the outfall of the Jetty Road catchment (C38) to Port Phillip Bay, representing the 'whole catchment' response. The adopted 1% AEP storm burst patterns are shown in Table 3.14.

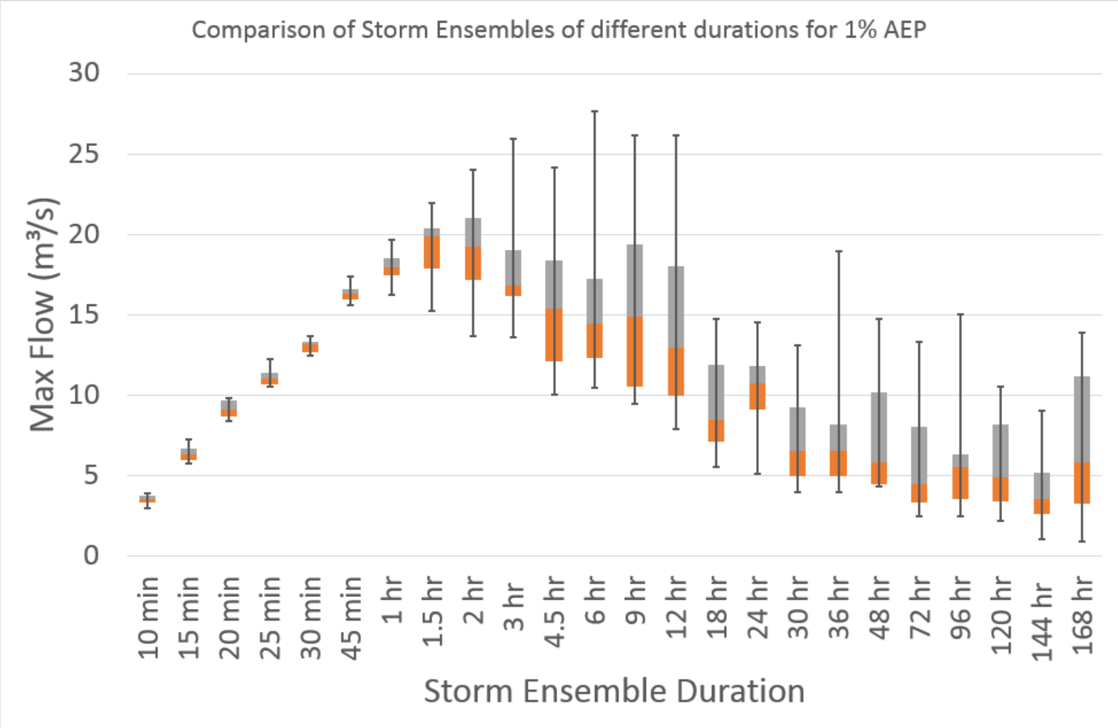


Figure 3.12: Existing Conditions **-** Temporal Pattern Box and Whiskers Plot – 1% AEP event

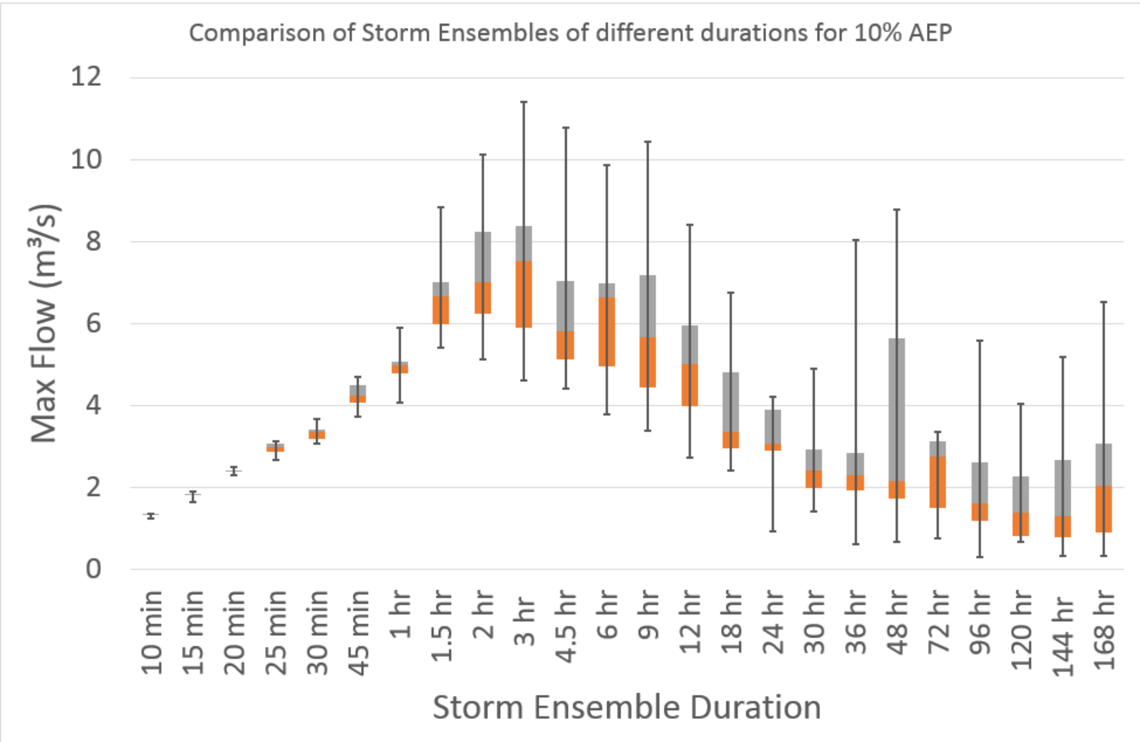


Figure 3.13: Existing Conditions - Temporal Pattern Box and Whiskers Plot - 10% AEP event

Table 3.14: Adopted 1% AEP Storm Burst Patterns – Jetty Rd Catchment

Duration	Duration (min)	Discharge (m^3/s)		Storm Burst Pattern No.
		Median	Adopted	
10 min	10	3.5255	3.585	5
15 min	15	6.303	6.609	6
20 min	20	9.129	9.224	4
25 min	25	11.0115	10.939	3
30 min	30	13.1845	13.173	3
45 min	45	16.308	16.194	2
1 hour	60	17.9885	18.044	7
1.5 hours	90	19.882	19.649	8
2 hours	120	19.26	19.635	2
3 hours	180	16.835	16.827	5
4.5 hours	270	15.392	15.205	4
6 hours	360	14.443	15.464	2
9 hours	540	14.881	15.565	7
12 hours	720	12.967	11.885	7
18 hours	1080	8.4405	8.556	3
24 hours	1440	10.773	10.726	4
30 hours	1800	6.568	6.183	1
36 hours	2160	6.5265	7.273	5
48 hours	2880	5.8335	6.594	4
72 hours	4320	4.494	4.064	7
96 hours	5760	5.5295	5.943	4
120 hours	7200	4.8815	4.093	8
144 hours	8640	3.558	3.265	4
168 hours	10080	5.816	6.451	5

4. HYDRODYNAMIC MODEL

The extent of flooding within the study area was evaluated by employing outputs from the hydrological model in the two-dimensional hydrodynamic model TUFLOW HPC.

TUFLOW HPC solves the two-dimensional (2D) Saint Venant equations, which describe the 2D motion of open channel flows in the hypothesis of non-erodible bed, hydrostatic distribution of pressures and water velocities uniformly distributed along the water depth.

The equations are solved using a finite difference numerical technique on the central nodes of a regular grid of square elements, which schematises the study area.

The model also allows the use of one-dimensional (1D) elements, which are dynamically linked to the 2D elements to better schematise all the morphological features of the territory that can potentially affect the flood evolution.

The 1D elements are usually adopted to represent small tributaries, urban channels and pipe network systems, where the use of pure 2D schemes can create problems related to the stability and accuracy of the numerical solution.

In addition, 1D elements are employed to represent the hydraulic devices that control the flood dynamics within the study area, such as culverts, weirs, sluice gates, pumps, etc.

4.1 Schematisation of the Study Area

The TUFLOW model was simulated using a Highly Paralleled Computing (HPC) solution on a Graphical Processing Unit (GPU). This allowed for significantly quicker runtimes to be achieved in large domains with a fine analytical grid size.

A fine grid (3 metre) model of the Greater Bellarine Peninsula, incorporating the Jetty Road sub-catchment study area, was used to generate the predicted flood extents to be employed as the existing conditions 'base case' for this stage of the project.

The extension and geometry of the hydrodynamic model boundary for the fine grid model has been defined considering the distinctive morphological features of the territory that can have possible effects in the containment and control of floods, such as terrain slopes, road embankments, natural and artificial levees, thus avoiding any possible effects of the model boundary definition on the maximum flood extent.

Furthermore, the hydrodynamic model boundary has been defined taking into account the potential flood affected area in order to optimise simulation runtimes.

4.1.1 2D Model Boundary

The hydrodynamic assessment adopted in this study analysed a 52.72 km² catchment area. The 2d-hydrodynamic model extent can be seen in Figure 4.1.

Figure 4.1: Hydrodynamic Model Extent

4.1.2 2D Grid Size

The mathematical derivation of the shallow water equations solved by TUFLOW requires that the adopted 2D cell size is greater than the local water depth. For this reason, a cell size of 3 m has been adopted to adequately represent the 2D domain from a numerical modelling perspective.

Moreover, the 3-metre grid enabled detailed definition of the variable terrain topography, road network, overland flow paths and various hydraulic obstructions. This grid size was also determined to be of suitable size to allow accurate definition of the terrain without adversely affecting the simulation run times.

4.2 Digital Elevation Model

A bottom elevation value was assigned to each cell of the hydrodynamic model using the most updated DEM available for the 2D model boundary.

The DEM was generated using LIDAR data from the *Port Phillip & Western Port LIDAR Project – 10m Elevation West (Bellarine-Geelong)* dataset flown between 26 April 2007 and 29 July 2007. At the time of project conception, and discussions with stakeholders, the 2007 LIDAR dataset was the best data available. Due to the largely rural condition of the study catchment and availability of detailed design plans where development has taken place, the 2007 LIDAR surface is considered fit for purpose.

The LIDAR digital elevation model (DEM) has a resolution of 1.0 m with a horizontal accuracy of ± 35 cm and a vertical accuracy of ± 10 cm. The DEM was rendered using ESRI ArcGIS at a sampling resolution on 1 metre.

The DEM generated for the analysis is shown in Figure 4.2.

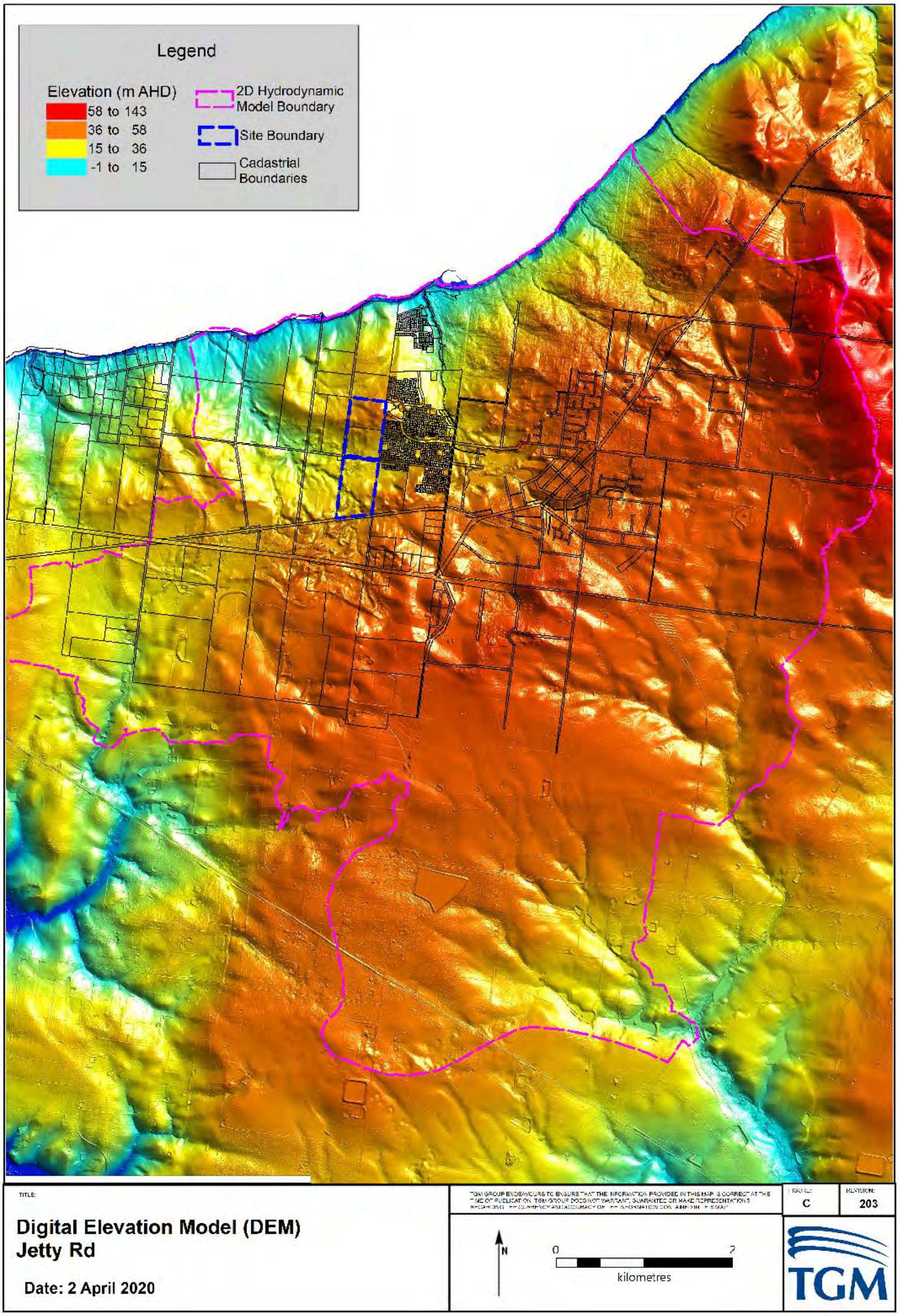


Figure 4.2: Coriyule Rd, Curlewis - Digital Elevation Model (DEM)

4.3 Drainage System

City of Greater Geelong Geographic Information Systems (GIS) drainage tables were downloaded from the Australian Governments National Map website and used to identify the location and characteristics of the underground drainage systems.

Key features of the drainage system in the area of interest, such as bridge culverts, were selected and represented as 1D elements in the TUFLOW hydrodynamic model.

Field inspections and level survey were also carried out at key locations to identify the presence of culverts not detected in the Australian Governments National Map.

For this study, only major 1% AEP drainage lines were included within the 2D model in addition to all known culverts that allow flow conveyance from the southern side of the Bellarine Rail Trail to the north.

Drainage infrastructure within the existing Jetty Road development was included where required in the 1D model. Inflow to the 2D domain at the extent of the Jetty Road development occurs at Corryule Road, as discussed in Section 4.5.

The location of the culverts and the underground pipe drainage network are shown in Figure 4.3.

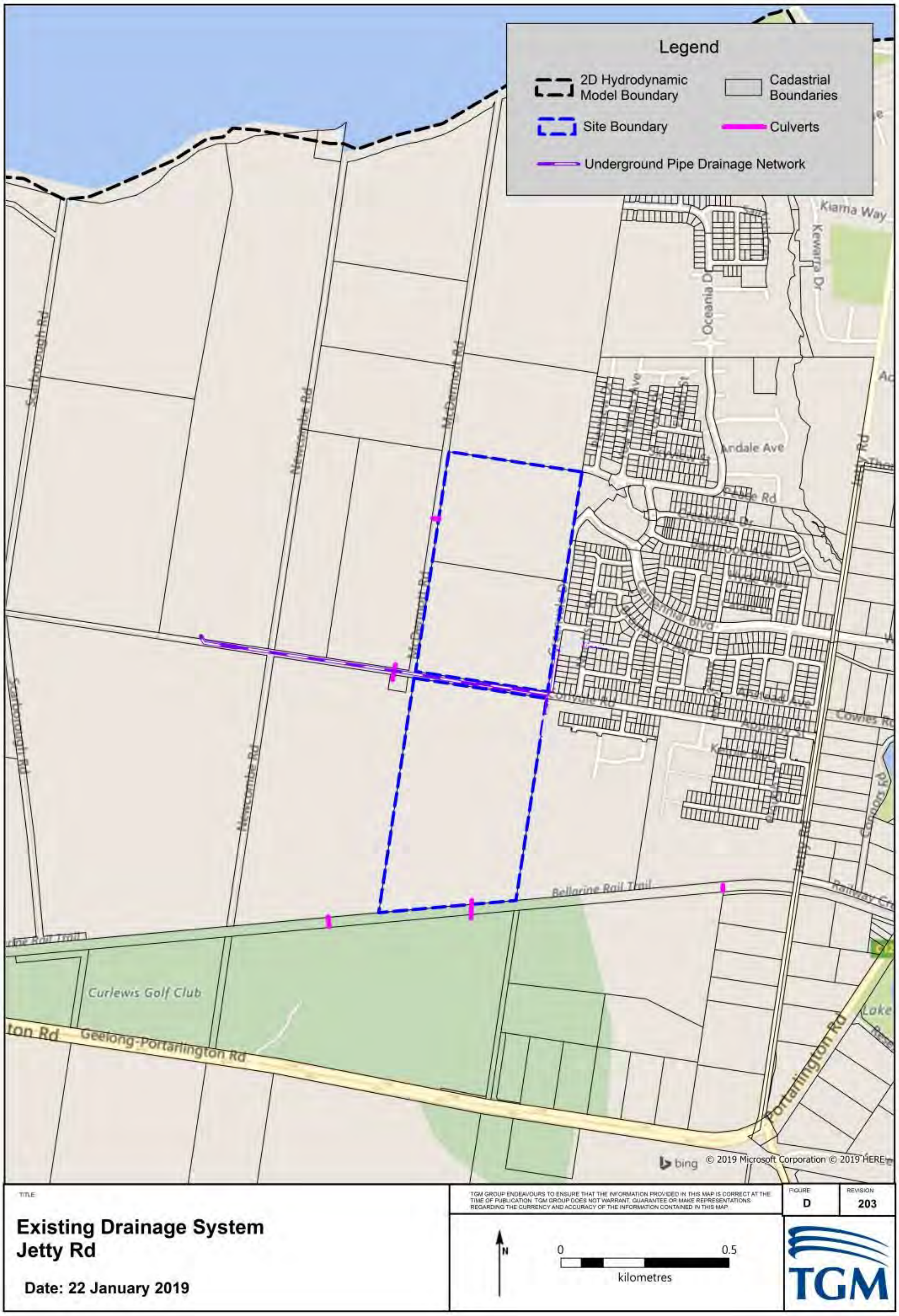


Figure 4.3: 1D Drainage Features - Underground Drainage Network (Fine Grid Model)

4.4 Manning's Roughness Coefficients

Manning's roughness coefficients were adopted for the TUFLOW hydrodynamic model with variation in roughness to reflect the vegetated waterway, water bodies, urban development and sealed/asphalt road networks. The Manning's roughness coefficients applied in this study are shown in Table 4.1.

Table 4.1: Manning's Roughness Coefficients *'n'*

Land Use Zoning	Manning's n	
	Recommended Range	Adopted Value
Residential – urban (higher density) – when building footprints and remainder of parcel are modelled together (with one roughness value)	0.20 – 0.50	0.20*
Residential – rural (lower density) – when building footprints and remainder of parcel are modelled together (with one roughness value)	0.1 – 0.2	N/A
Residential – urban (higher density) - when building footprints are modelled separately to remainder of parcel	0.2 – 0.5 (footprints) 0.08 – 0.12 (remainder)	N/A
Residential – rural (lower density) - when building footprints are modelled separately to remainder of parcel	0.2 – 0.5 (footprints) 0.04 – 0.06 (remainder)	N/A
Industrial/Commercial or large buildings on site	0.1 – 0.5	N/A
Significant Drainage Easement*	0.02 – 0.08	N/A
Open Space or waterway - minimal vegetation	0.03 – 0.05	0.03
Open Space or waterway - moderate vegetation	0.05 – 0.07	0.05
Open Space or waterway - heavy vegetation	0.07 – 0.12	N/A
Open water (with reedy vegetation)	0.05 – 0.08	N/A
Open water (with submerged vegetation)	0.01 – 0.035	N/A
Car park / pavement / wide driveways / roads	0.018 – 0.04	N/A
Railway line	0.05 – 0.2	N/A
Sport pitch (Curlewis Golf Course)	0.03 – 0.05	0.05

* A value of 0.2 has been adopted in this study due to the vast majority of the urban residential area being modelled within the XP Storm 1D model.

4.5 Inflow Boundary Conditions

Based on the review of the Curlewis Drysdale and Clifton Springs region of the Greater Bellarine Peninsular model and the analysis of the model flows (discussed in Section 3.7 and Section 0), it was determined that durations above the 30-hour event were not deemed to be critical.

Output hydrographs from the XP-STORM hydrological model for durations up to and including the 30-hour duration event were applied as inflow boundary conditions into the TUFLOW hydrodynamic model.

Two inflow conditions were input into the 2d-hydrodynamic model, namely External Catchment Flow Boundaries (2d_bc) and Internal Flow Boundaries (2d_sa)

The boundary condition locations are shown in Figure 4.4.

4.5.1 Internal Flow Boundaries (2d_sa)

Catchments directly contributing to the nominated LPOD (Section 2.1.1 and Figure 2.4) at the outlet of the Coriyule Road drainage line and flows generated within or directly adjacent the subject site were extracted from the local Jetty Road sub catchment model. Flow hydrographs were input into the 2D hydrodynamic model (using 2d_sa) at the lowest practical point within the catchment.

4.5.2 External Catchment Flow Boundaries (2d_bc)

Downstream of Coriyule Road, external catchment flows which would have the potential to affect the hydraulics within the waterway were extracted from the regional Greater Bellarine Peninsula model, as where external catchment flows that do not contribute to the flooding in the area of interest. Flow hydrographs were input into the 2D hydrodynamic model (using 2d_bc) into the most practical location within the catchment (i.e. existing depressions, waterways, catchment low points etc).

4.6 Outflow Boundary Conditions

A constant water level of 2m AHD has been assigned as outflow boundary condition along the coast.

4.7 Initial Water Levels

It is common, throughout the rural context of the study area, to observe dam structures constructed along principle flow paths. Dam structures defined within the LIDAR surface as depressions provide storage volume during a regional flood event and possibly mitigate flood impacts.

It is best practice to assume dam structures as being full prior to the modelled event to remove storage effects. Therefore, an initial water level was applied to each identified structure to ensure the dam is represented as 'full' prior to the analysed flood event.

The applied boundary conditions are shown in Figure 4.4.

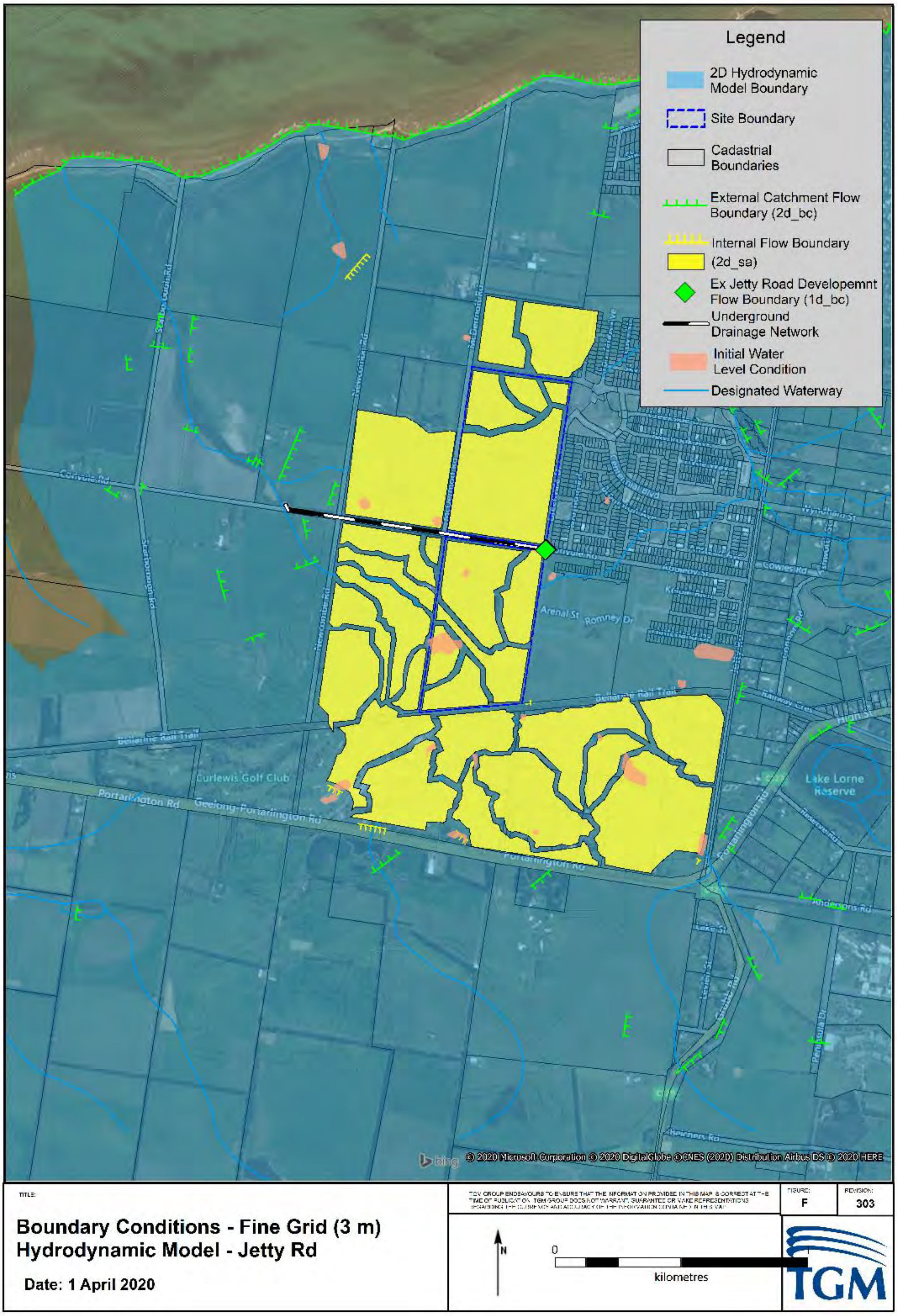


Figure 4.4: 2D Hydrodynamic Model (Fine Grid Model) – Boundary Conditions

5. MODELLING RESULTS

5.1 Critical Duration

The Greater Bellarine Peninsula Model discussed in 3.1.1 utilised hydrographs from the XP-Strom model for each of the 24 event durations as inflow boundary conditions within a coarse grid (10m) 2D hydrodynamic TUFLOW model. The purpose of this model was to identify the critical duration(s) impacting the site for each event probability, identifying potential cross catchment flow and informing critical duration selection.

Analysis of the Greater Bellarine Peninsula Model indicated that the critical duration events throughout the catchments were less than the 30-hour duration event. This assessment informed the inputs to the 3m fine grid hydrodynamic model to include durations ranging from the 10 minutes – 30-hour rainfall event for the 1%, 10% and 50% AEP storm events.

The decision to model multiple exceedance probabilities within this study was made to ensure an evaluation of flooding for major flood events (1% AEP), minor flood events (10% AEP) and frequent events (50% AEP) more commonly associated with water quality treatment infrastructure were achieved.

The critical 1%, 10% and 50% AEP event durations can be seen in Figure 5.1, Figure 5.2 and Figure 5.3.

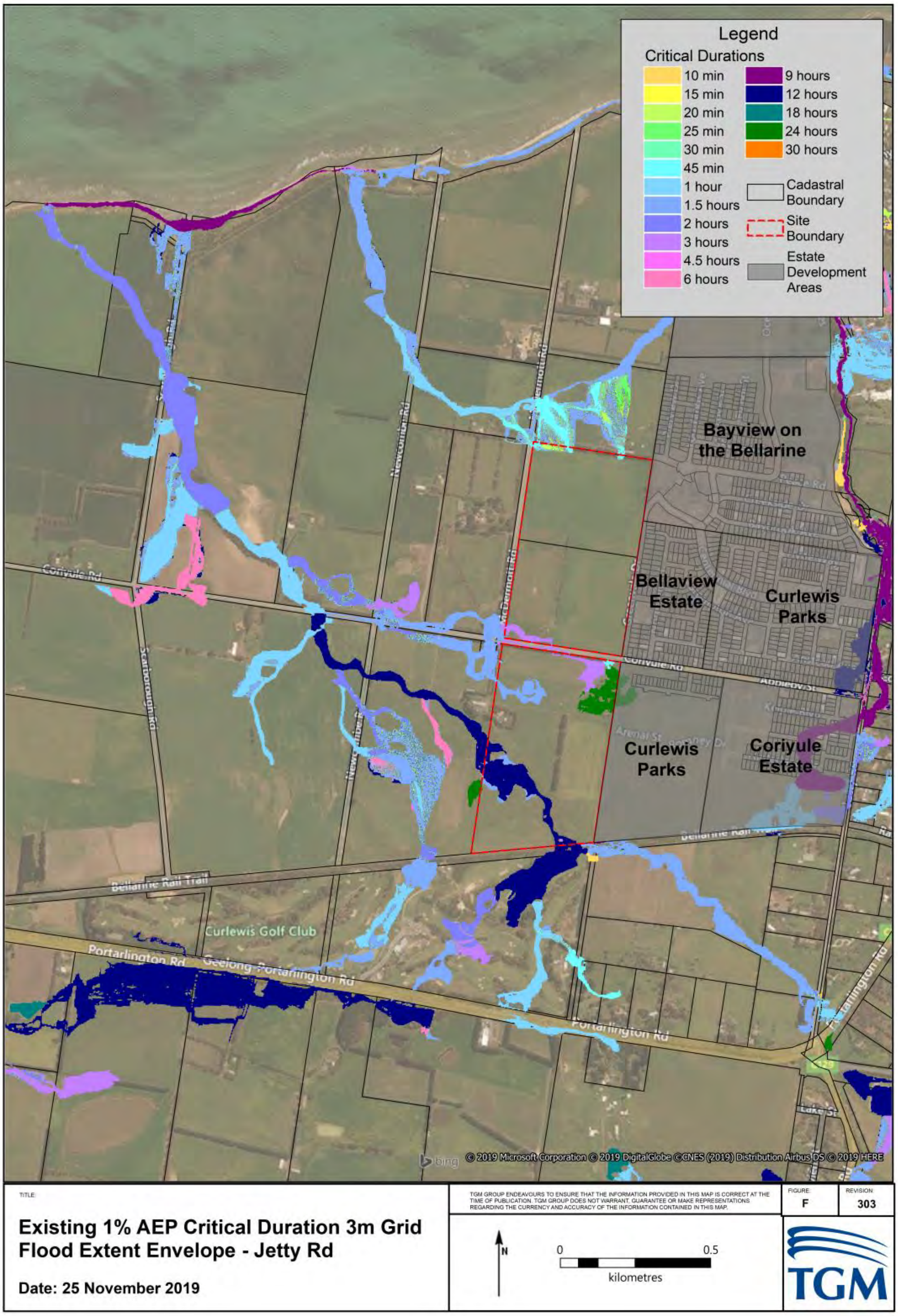


Figure 5.1: 1% AEP Critical Storm Durations

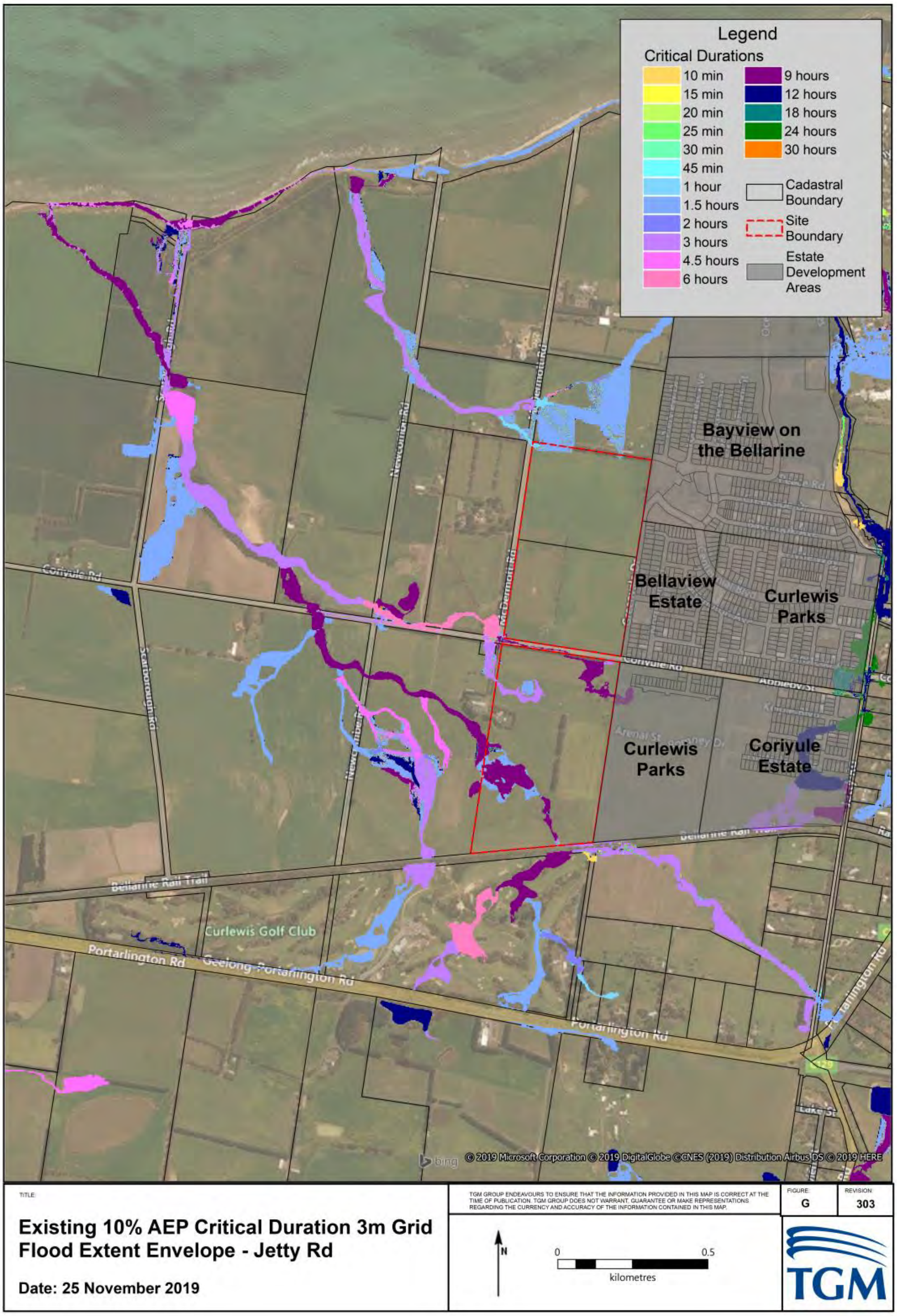


Figure 5.2: 10% AEP Critical Storm Durations

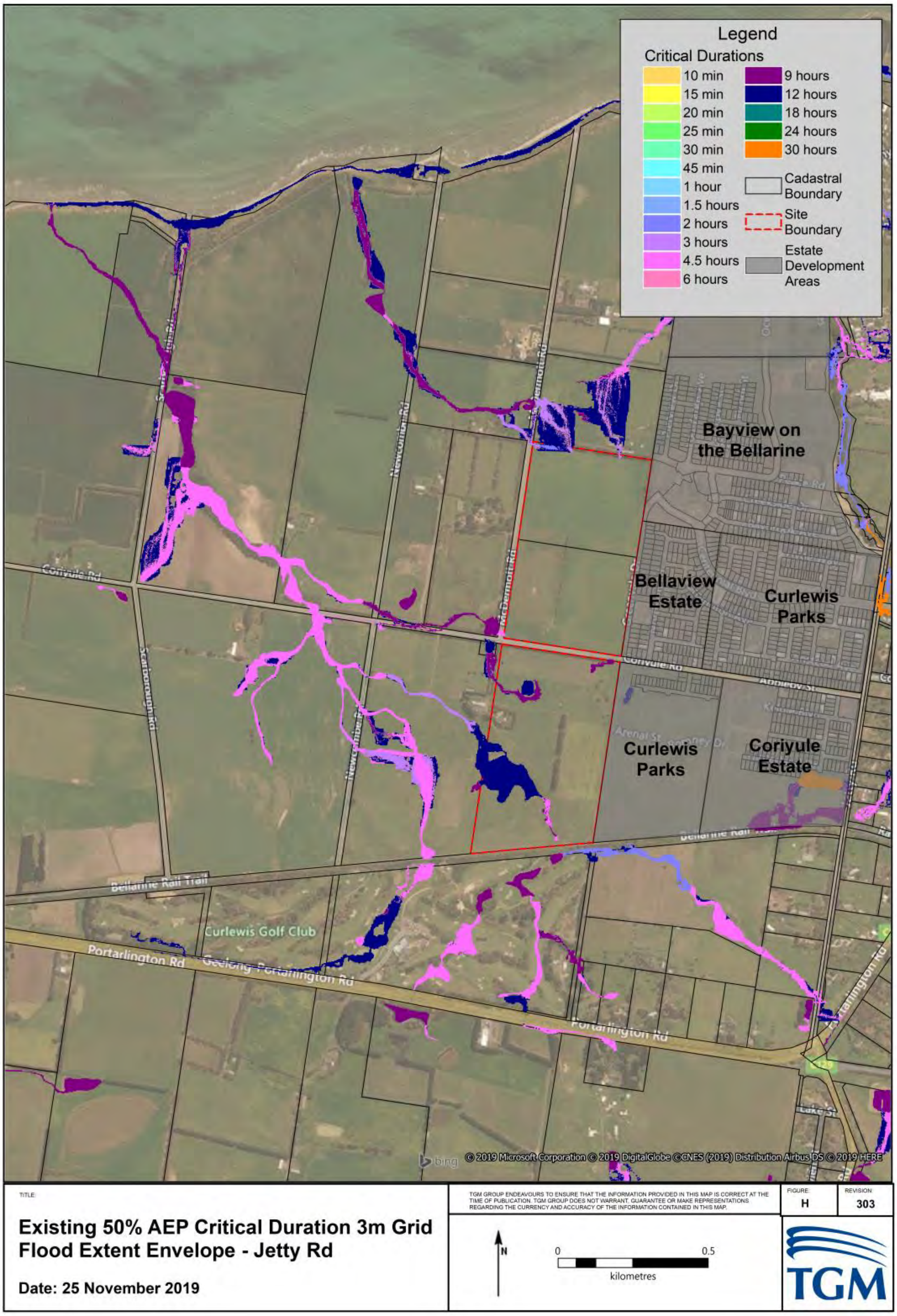


Figure 5.3: 50% AEP Critical Storm Durations

5.2 Critical Discharges and Flow Comparison (1% AEP)

Validation of the hydrodynamic model outputs was undertaken by comparing stormwater discharges at key locations within the Jetty Road catchment area to those from the XP-STORM model.

Reporting locations were nominated in the 2D hydrodynamic model correlating to key locations in the 1D hydraulic model. The reporting locations can be seen in Figure 5.4. The 1% AEP 2-hour duration volumetric flow rates and time of peak (Δt) are detailed in Table 5.1.

The 2-hour duration, 1% AEP event has been used in the below comparison as the indicated locations exhibit critical durations of 2 hours or comparable duration.

Table 5.1: Discharge Comparison

Location	2D Hydraulic Model		1D Hydraulic Model	
	Discharge (m^3/s)	Δt (hr)	Discharge (m^3/s)	Δt (hr)
xs_17	17.35	2.15	*1D model combines catchment C38 outfall*	
xs_18	0.57	2.10		
xs_17/xs_18 (combined outfall)	17.92	2.15	19.37	1.67
xs_20	14.15	1.77	13.41	1.41

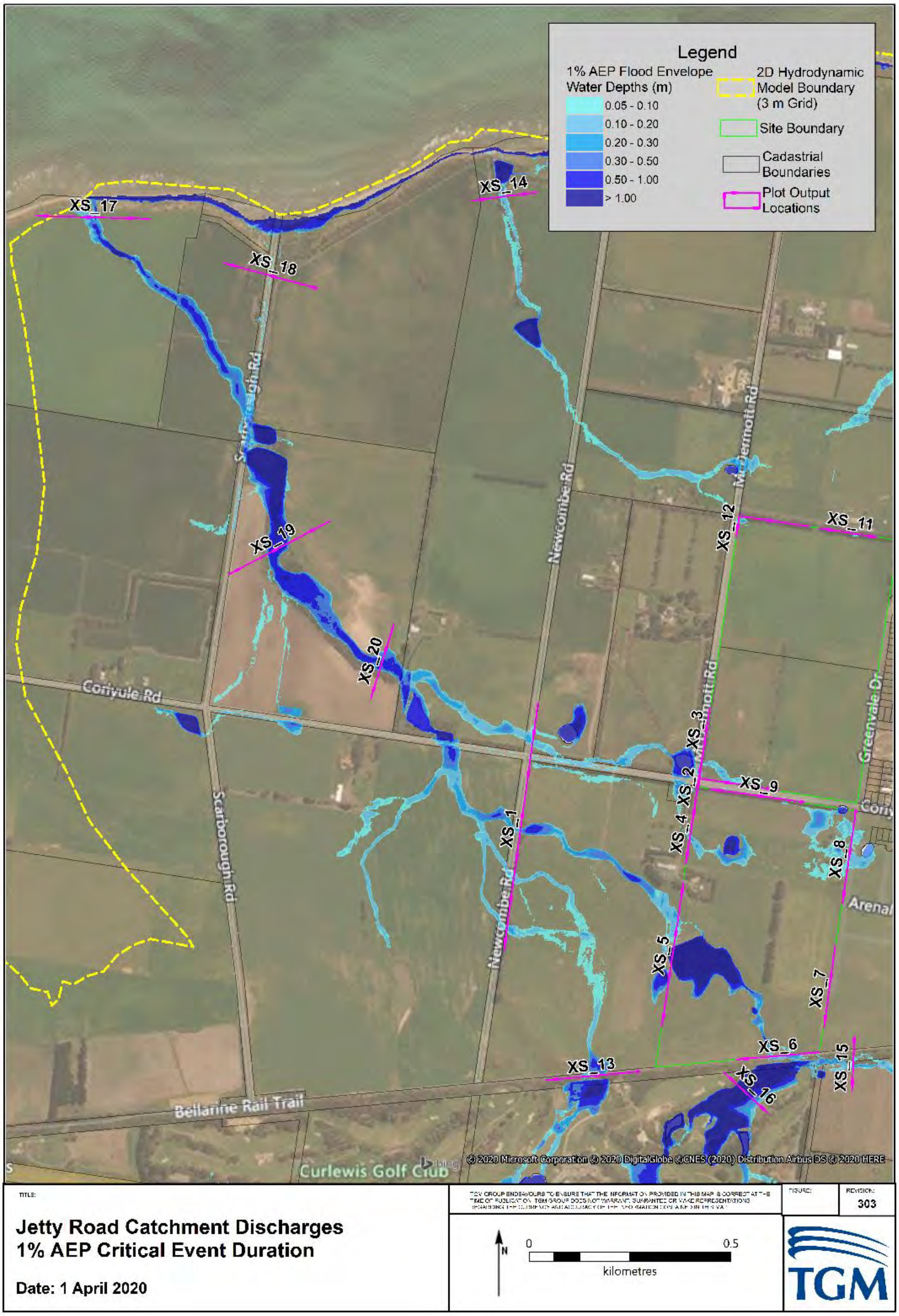


Figure 5.4: Plot output reporting locations

The modelled plot output (PO) reporting location peak discharges are detailed in Table 5.2 for the 1% AEP (2hr), 10% AEP (9hr) and 50% AEP (9hr) events.

Table 5.2: Stormwater discharges

Location	Peak Discharge (m^3/s)		
	1% AEP	10% AEP	50% AEP
xs_1	5.17	2.83	0.28
xs_2	1.8	0.46	0.11
xs_3	1.25	0.35	0.11
xs_4	0.6	0.16	0.02
xs_6	0.024	-	-
Culvert_6	1.43	1.26	0.28
xs_9	0.05	-	-
xs_11	0.10	0.03	0.01
xs_12	0.69	0.19	0.07
xs_13	1.73	-	-
xs_14	2.34	0.58	0.1
xs_15	3.29	1.02	0.26
xs_16	3.63	1.56	0.02
xs_17	17.35	4.95	0.81
xs_18	0.57	0.32	0.13
xs_19	15.80	4.96	1.15
xs_20	14.14	4.58	1.03

5.3 Flood Depth Mapping

The full extent 1% AEP envelope in addition to the 1%, 10% and 50% AEP flood depths (50mm filtering) generated with the fine grid model are shown in Figure 5.5 to Figure 5.8.

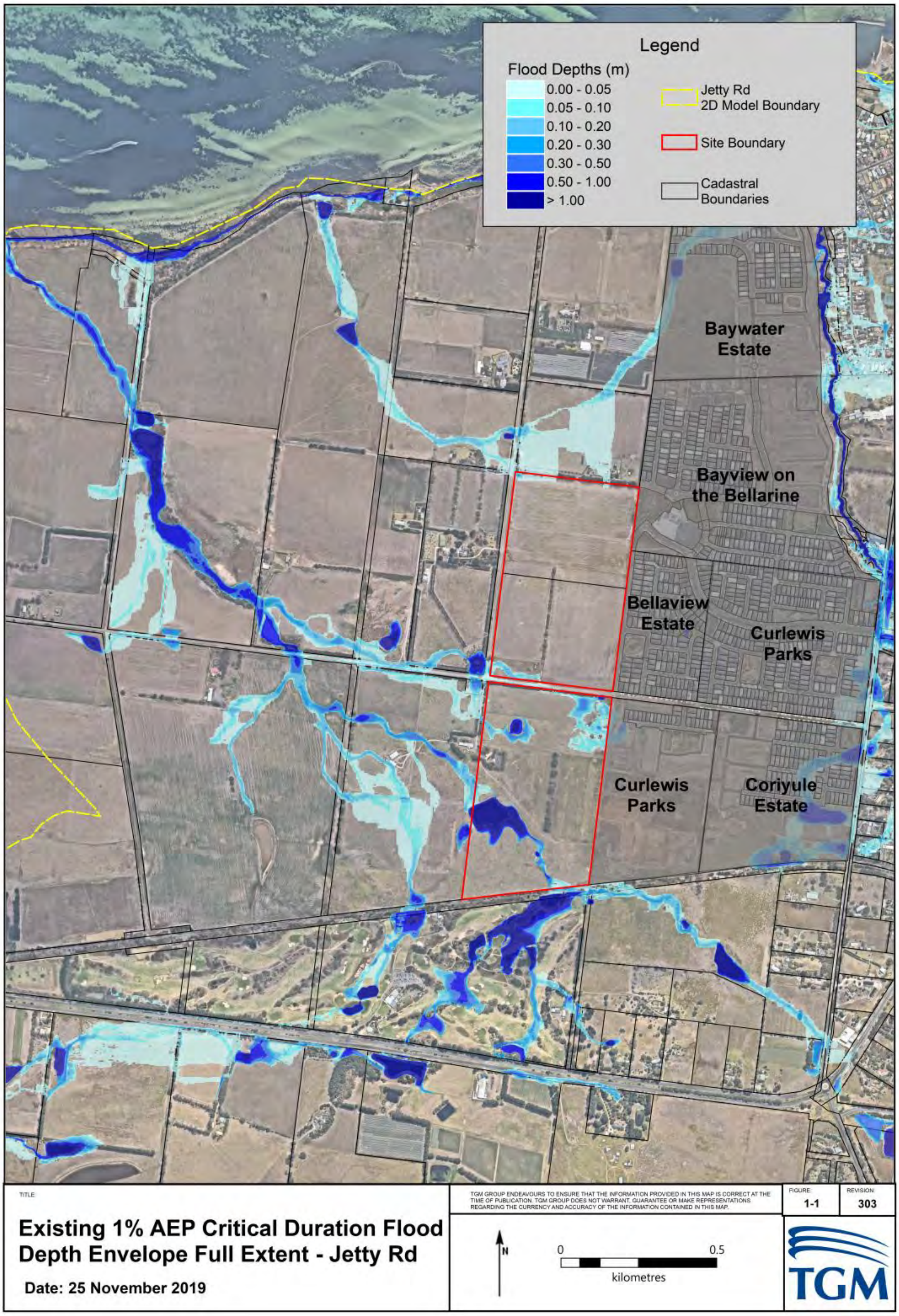


Figure 5.5: 1% AEP Flood Extent - 3m Grid – Existing Conditions

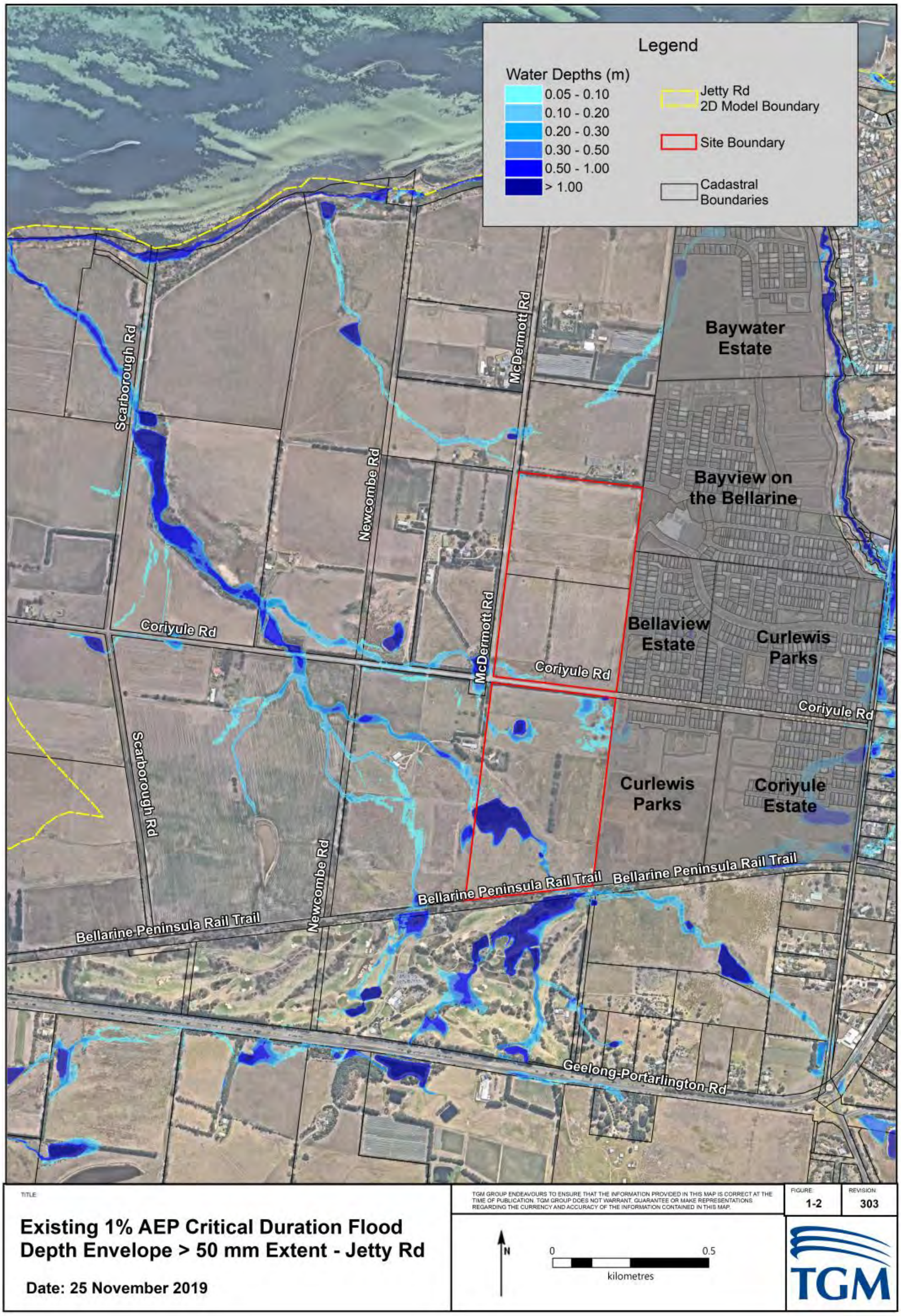


Figure 5.6: 1% AEP Flood Extent (≥ 50 mm depths) - 3m Grid – Existing Conditions

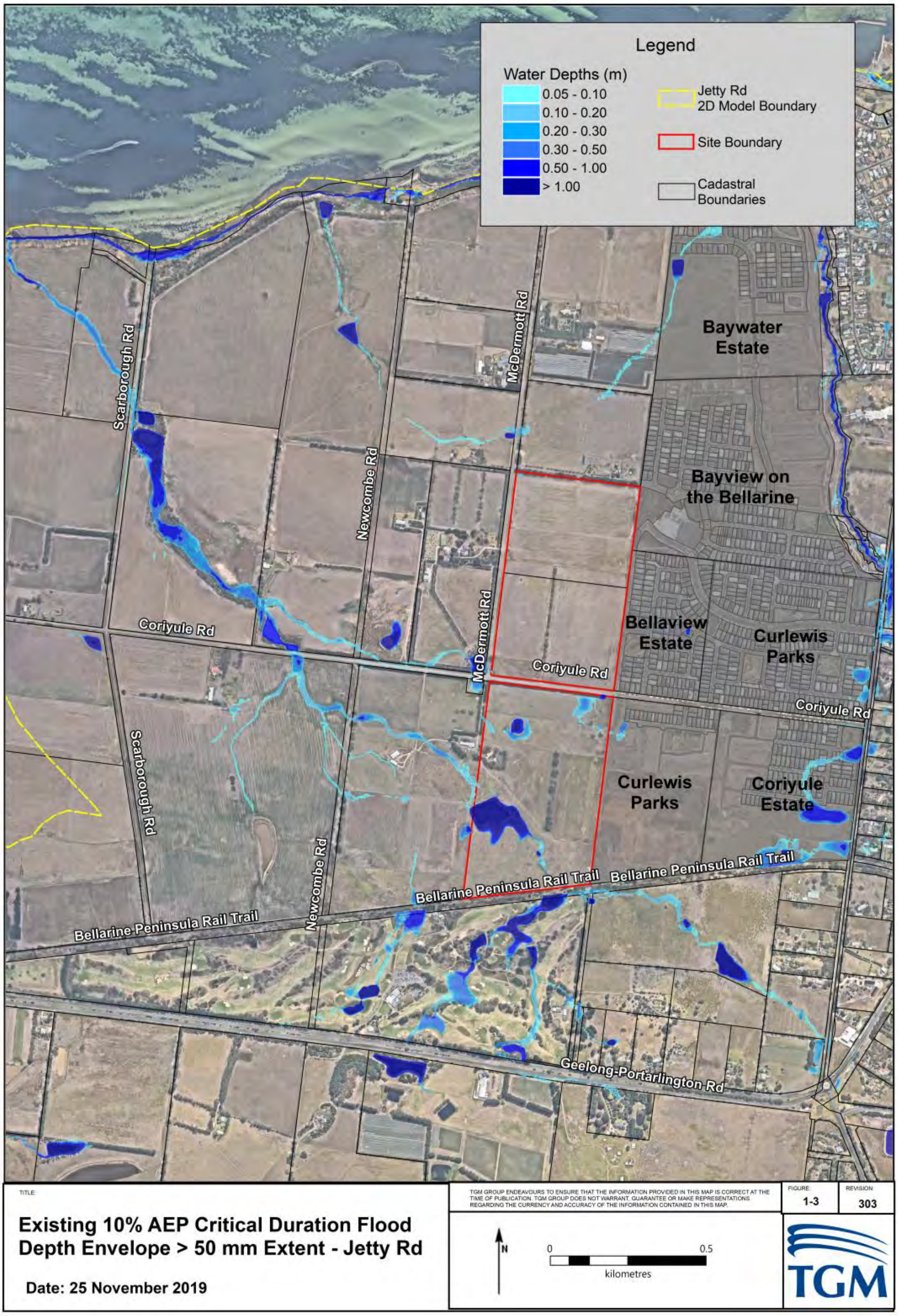


Figure 5.7: 10% AEP Flood Extent (≥ 50 mm depths) - 3m Grid – Existing Conditions

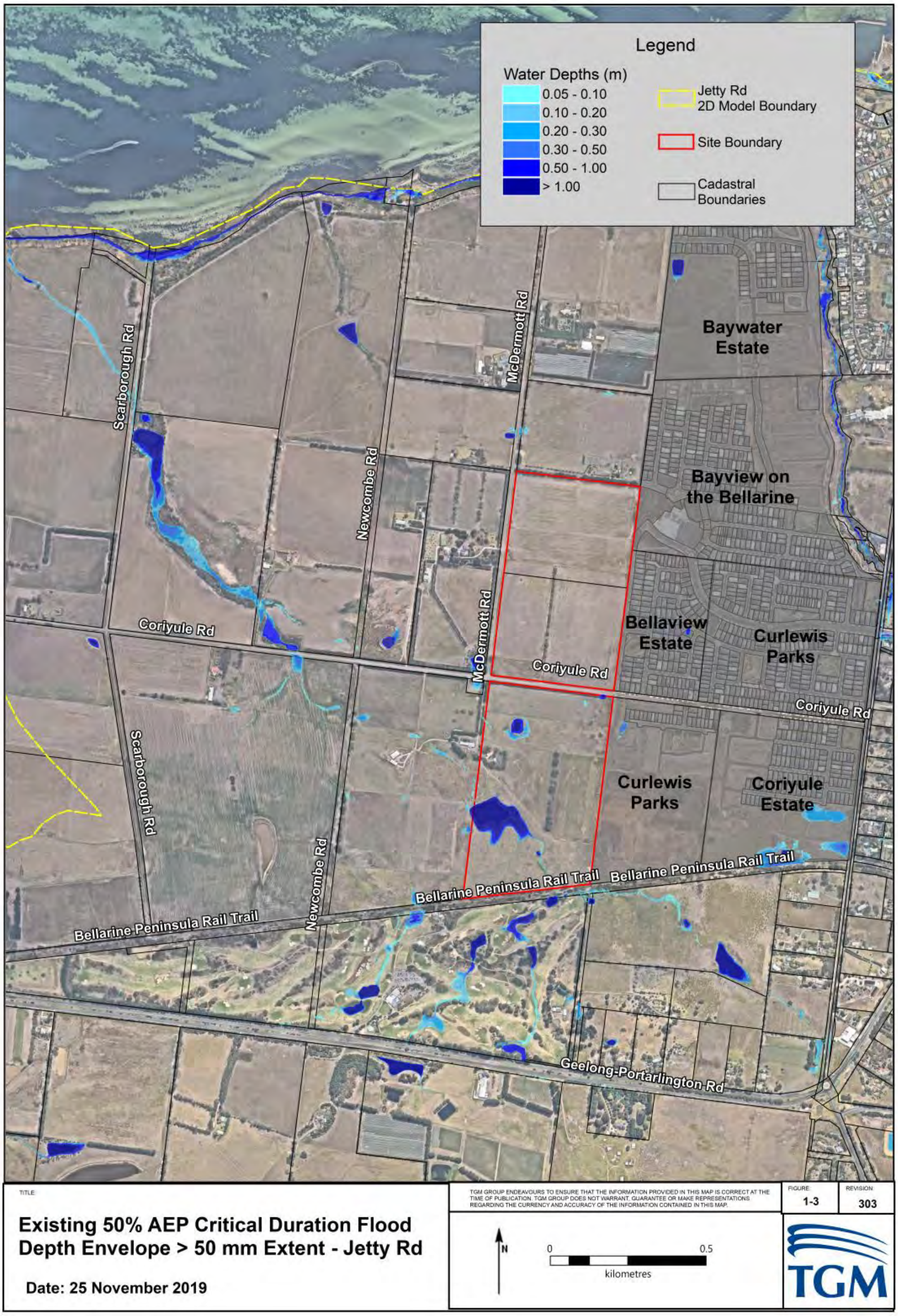


Figure 5.8: 50% AEP Flood Extent (≥ 50 mm depths) - 3m Grid – Existing Conditions

5.4 Flood Hazard Mapping

Safety is defined in terms of the depth, velocity and the product of the depth multiplied by velocity. ARR2016[25] defines the safety criteria for flood waters in relation to movement of people and vehicles.

Hazards related to stormwater runoff generated during a predicted 1% AEP critical duration storm event is presented in Figure 5.9.

The hazard maps have been created accordingly to the Safety and Hazard Criteria defined by ARR Project 10, which state that flow velocity, depths and the product of velocity and depth must not exceed safety limits for people and vehicle access (egress) to (from) the site. The criteria are as follows:

- ➢ Site Safety (People)
    - Depth must be no greater than or equal to 0.5 metres;
    - Velocity must be no greater than or equal to 3.0 m/s; and
    - The product of depth multiplied by velocity must be no greater than or equal to 0.4 m^2/s.
- ➢ Access Safety (Vehicles)
    - Depth must be no greater than or equal to 0.3 metres;
    - Velocity must be no greater than or equal to 3.0 m/s; and
    - The product of depth multiplied by velocity must be no greater than or equal to 0.3 m^2/s

As safety for vehicles sets the lower threshold on acceptable hazard limits it will be adopted in this study as the safety criteria.

[25] Smith, G, Cox, R, 2019. Safety Design Criteria. Chapter 7 Book 6 in Australian Rainfall and Runoff - A Guide to Flood Estimation, Commonwealth of Australia

TITLE:

Existing 1% AEP Critical Duration - Jetty Rd Overall Hazard Envelope (>50mm depth)

Date: 1 April 2020

FIGURE: 2-1 REVISION: 303

N 0 0.5 kilometres

Figure 5.9: 1% AEP Overall Hazard Envelope - Existing Conditions

6. CONCLUSION

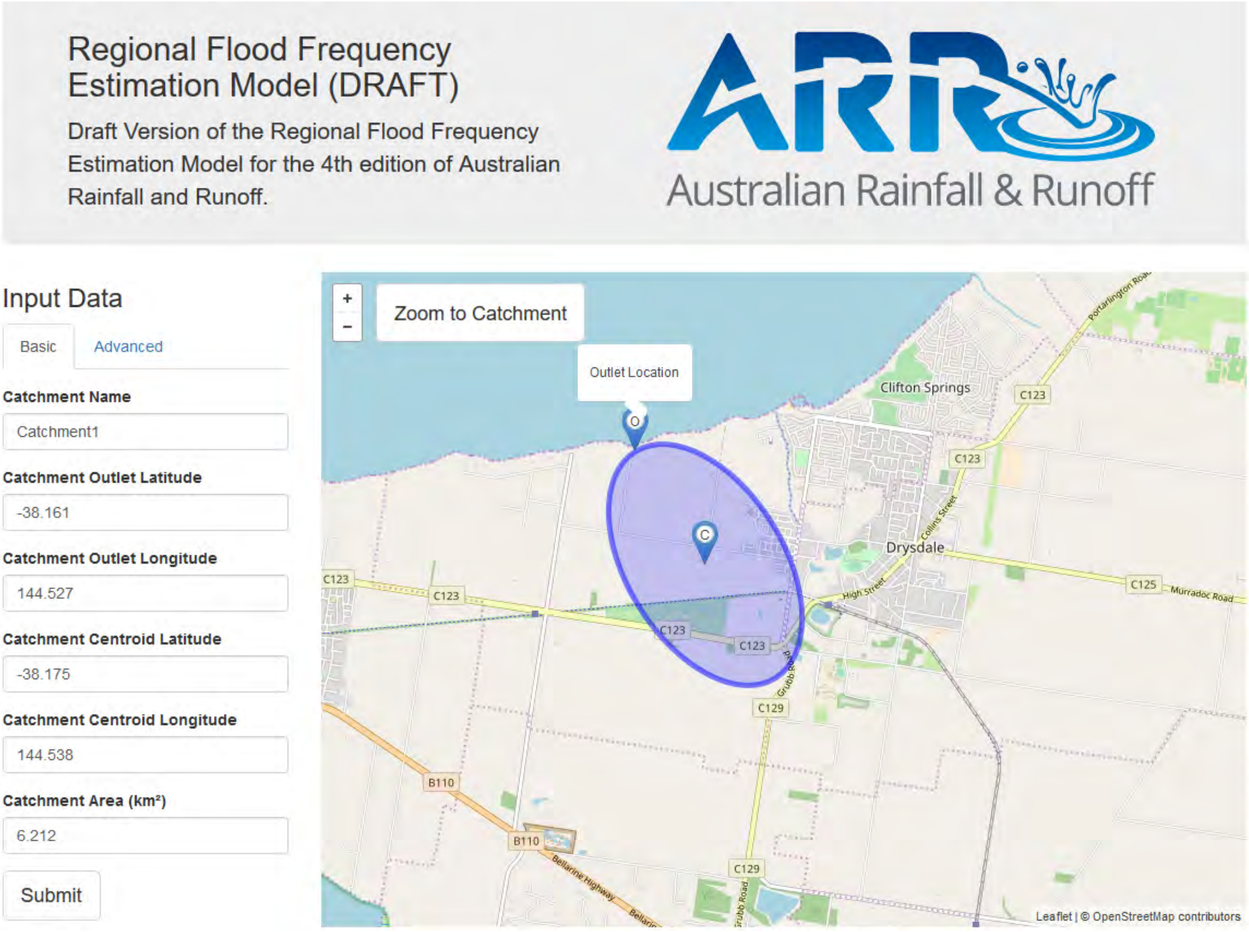
TGM conclude that the rainfall and flood estimation techniques employed in this study are consistent with ARR2016 current industry best practice. The mathematical modelling output is an accurate representation, based on best available data, of the estimated regional flood extent impacting the site during the 1% AEP design storm event with a range of critical storm durations.

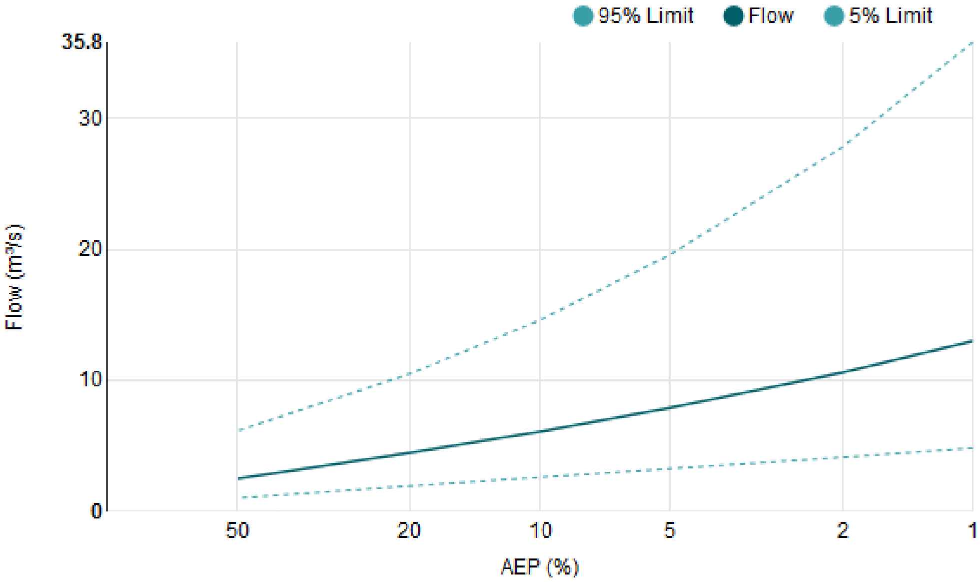
TGM proposes that the hydrodynamic simulation for a 3-metre grid be adopted as the existing conditions 'Base Case' scenario flood extent for the area in the vicinity of Coriyule Rd, Curlewis.

It is intended that this scenario will be used to undertake a feasibility assessment on the development site. It is expected that the 'Base Case' will be refined throughout the project as new data and detail is made available.

TGM seeks the City of Greater Geelong Council and Corangamite Catchment Management Authority approval that this Flood Study is fit for the purpose of providing the predicted 1% AEP existing conditions 'Base Case' flood extent to be used to undertake a feasibility assessment including a Stormwater Management Plan (SMP) and Flood Impact Assessment (FIA) for the proposed development of the site.

7. APPENDIX A: REGIONAL FLOOD FREQUENCY ESTIMATION MODEL





AEP (%)	Discharge (m^3/s)	Lower Confidence Limit (5%) (m^3/s)	Upper Confidence Limit (95%) (m^3/s)
50	2.51	1.05	6.16
20	4.48	1.96	10.5
10	6.10	2.62	14.6
5	7.91	3.29	19.6
2	10.6	4.15	27.8
1	13.0	4.85	35.8

Input Data

Date/Time	2017-09-27 09:03
Catchment Name	Catchment1
Latitude (Outlet)	-38.161
Longitude (Outlet)	144.527
Latitude (Centroid)	-38.175
Longitude (Centroid)	144.538
Catchment Area (km^2)	6.212
Distance to Nearest Gauged Catchment (km)	28.29
50% AEP 6 Hour Rainfall Intensity (mm/h)	4.443677
2% AEP 6 Hour Rainfall Intensity (mm/h)	9.752582
Rainfall Intensity Source (User/Auto)	Auto
Region	East Coast
Region Version	RFFE Model 2016 v1
Region Source (User/Auto)	Auto
Shape Factor	0.73
Interpolation Method	Natural Neighbour
Bias Correction Value	0.358

Statistics

Variable	Value	Standard Dev
Mean	0.911	0.654
Standard Dev	0.664	0.217
Skew	0.141	0.030

Note: These statistics come from the nearest gauged catchment. Details.

Correlation		
1.000		
-0.330	1.000	
0.170	-0.280	1.000

Note: These statistics are common to each region. Details.

1% AEP Flow vs Catchment Area