STORMWATER MANAGEMENT STRATEGY

BRIODY DRIVE WEST DEVELOPMENT PLAN
AUGUST 2022

PREPARED FOR SUMMERSET GROUP HOLDINGS

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Note# : updated to include minor amendments from planning consultant review.

Spiire Job Number: 306395

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1 INTRODUCTION

Spiire Australia has been engaged by Summerset Group Holdings to prepare a Stormwater Management Strategy (SWMS) to inform the drainage requirements for the Development Plan for the residential development of a consortium of properties at Briody Drive, Torquay. The subject area will be referred to as the Briody Drive West Development Plan Area (BDW DP).

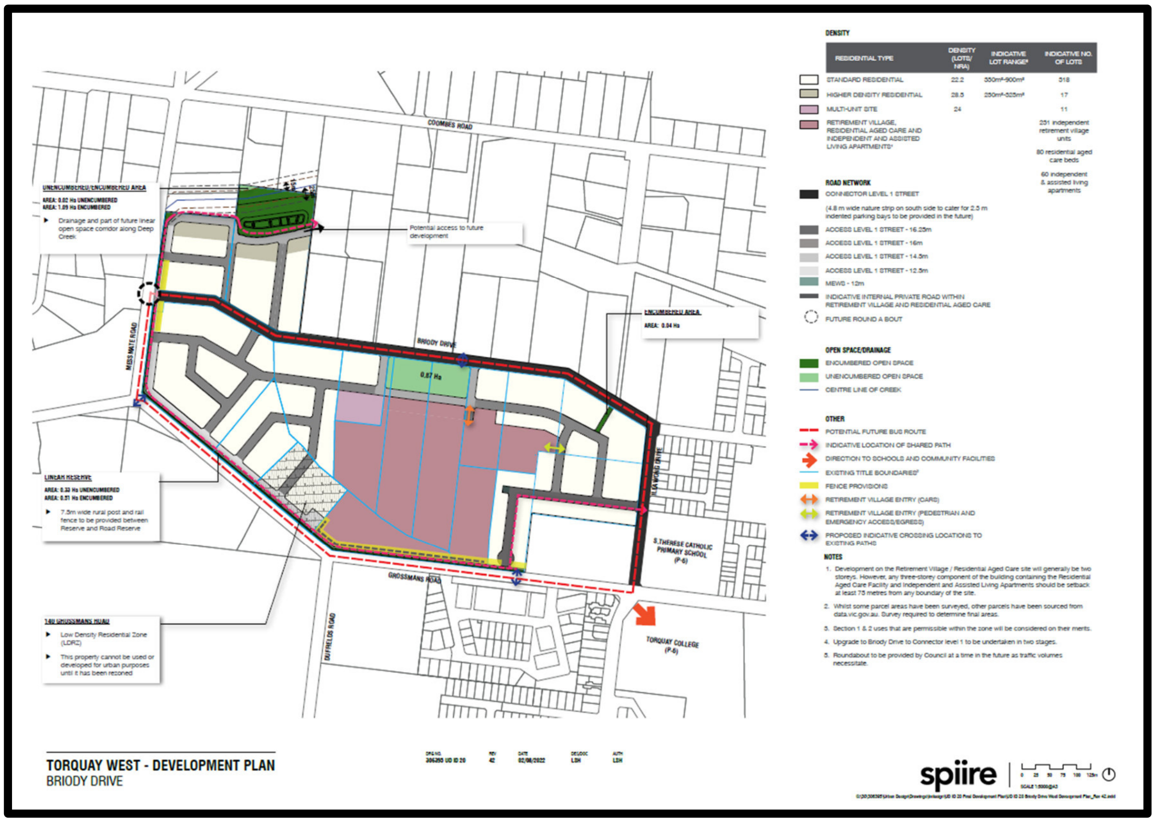

Figure 1: Site Overview

This SWMS is intended to inform and accompany the Urban Design Layout and other supporting documents for the proposed development plan submission associated with the subdivision for the BDW DP and includes:

- Ensuring the relevant local and state planning requirements are addressed through a current assessment of:
  - Validation of stormwater infrastructure management requirements to support development of the subject site in the context of the Development Plan Overlay (DPO10) of the Surf Coast Shire Planning Scheme, and the previously endorsed Development Plan;
  - Confirmation of overland flow paths and critical drainage flow rates to inform earthworks design and major drainage works;
  - Stormwater quality (MUSIC) and hydrologic / hydraulic analysis to support the above;
  - Application of a Works on Waterway Application to the CCMA (consequent to finalisation of this report);

- Allowance for conceptual grading of subdivisional roads and proposed basin and outfall infrastructure.

2 BACKGROUND

2.1 Site and Planning Context

Any residential development within the BDW DP site area must be generally in accordance with the BDW DP (endorsed by the Surf Coast Shire December 2017), or as amended. The previously endorsed development plan specifies an urban layout comprised of substantial residential development integrated with open space, walkways and stormwater treatment areas. The urban layout indicates that the majority of housing product will be of standard residential density (22.2 lots/ha), with some higher density (28.5 lots/ha) housing specified in areas adjacent to the public open space parkland and the drainage reserves. Also of note is the integrated 7.5m vegetation buffers along Grossmans Rd and Messmate Rd. A copy of the currently endorsed BDW DP is provided in Figure 2 below.

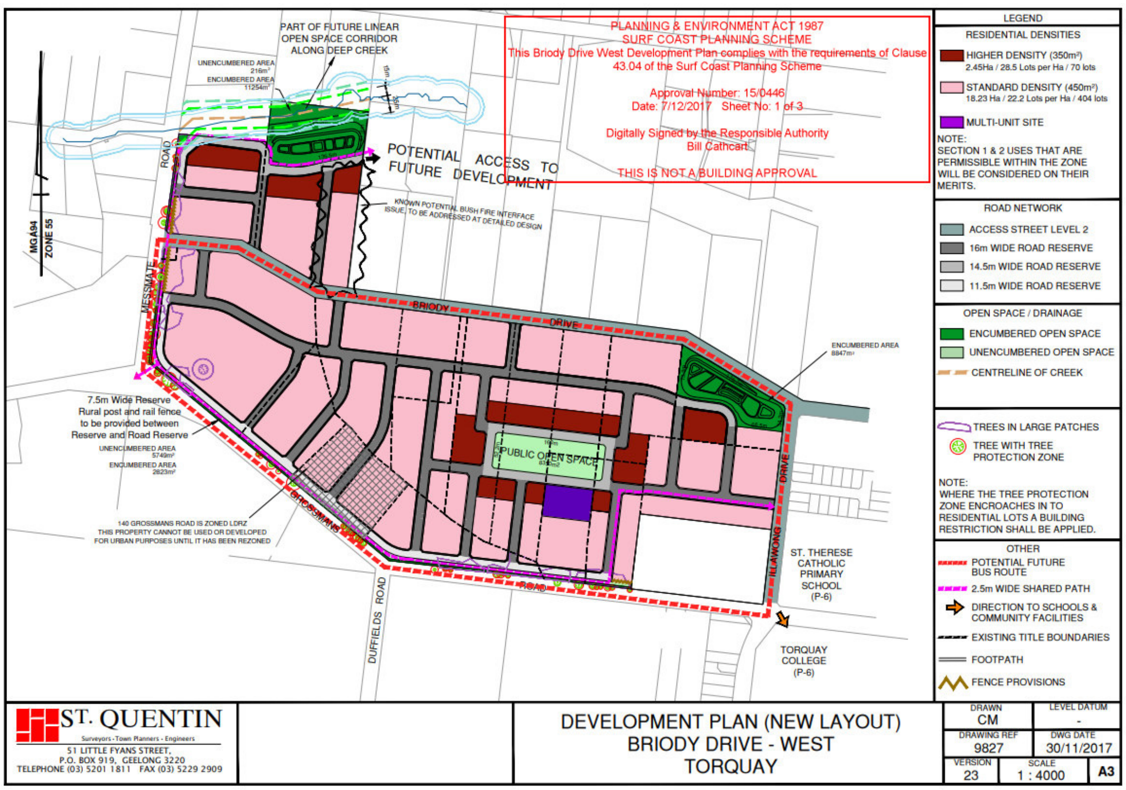

Figure 2: Currently Endorsed Development Plan for Briody Drive West (dated 07/12/2017)

Schedule 10 to Clause 43.04 Development Plan Overlay in the Surf Coast Planning Scheme (referred to as DPO10) specified that the development plan must include:

- A Flooding, Stormwater and Drainage Management Plan that takes an integrated approach to stormwater system management, designed with reference to the two catchments that affect the land and includes
  - An integrated stormwater management system for the properties discharging directly to Deep Creek (170 Grossmans Road and 150 and 170 Briody Drive) that ensures

the peak discharge rate and pollutant load of stormwater leaving the subject land within the area affected by this schedule is no greater than pre-development levels, meets current best practice and is discharged to the existing drainage system.

- An integrated stormwater management system for the remainder of the land that ensures the pollutant load of stormwater leaving the land is no greater than pre-development levels, meets current best practice and the stormwater is discharged to Deep Creek via the Council walkway and designed to cater for the 1 in 100 year (1% AEP) storm to the existing drainage system.
- Any interim stormwater management arrangements that could provide for out of sequence residential development.
- Input from the Corangamite Catchment Management Authority for works in, on or over Deep Creek, which is a designated waterway.
- Where required, a description of the methodology and apportionment of costs for the provision of the integrated stormwater management system including how its costs will be equalised across all landowners. This may be implemented via a condition on a planning permit that approves a residential subdivision, for a Section 173 Agreement that requires a cash contribution to equalise the costs associated with providing land for and the construction of the system or any other mechanism to the satisfaction of the responsible authority.

DPO 10 also states that a permit for subdivision of the land may require a Section 173 Agreement under the Planning and Environment Act 1987 to:

- provide for the development of an integrated stormwater management system and the equalisation of costs associated with the provision of land for and the construction of the system, or
- ***provide for any other approach to the management of stormwater to the satisfaction the responsible authority.***

Standard C25 from Clause 56.07-4 of the Surf Coast Planning Scheme also specifies that the stormwater management system must be:

- Designed and managed in accordance with the requirements and to the satisfaction of the relevant drainage authority.
- Designed and managed in accordance with the requirements and to the satisfaction of the water authority where reuse of stormwater is proposed.
- Designed to meet the current best practice performance objectives for stormwater quality as contained in the Urban Stormwater - Best Practice Environmental Management Guidelines (Victorian Stormwater Committee, 1999).
- Designed to ensure that flows downstream of the subdivision site are restricted to pre development levels ***unless increased flows are approved by the relevant drainage authority and there are no detrimental downstream impacts.***
- Designed to contribute to cooling, improving local habitat and providing attractive and enjoyable spaces.

The BDW DP is made up of predominantly GRZ1 zoned land with one property (140 Grossmans Road) zoned LDRZ, and is subject to the following planning overlays:

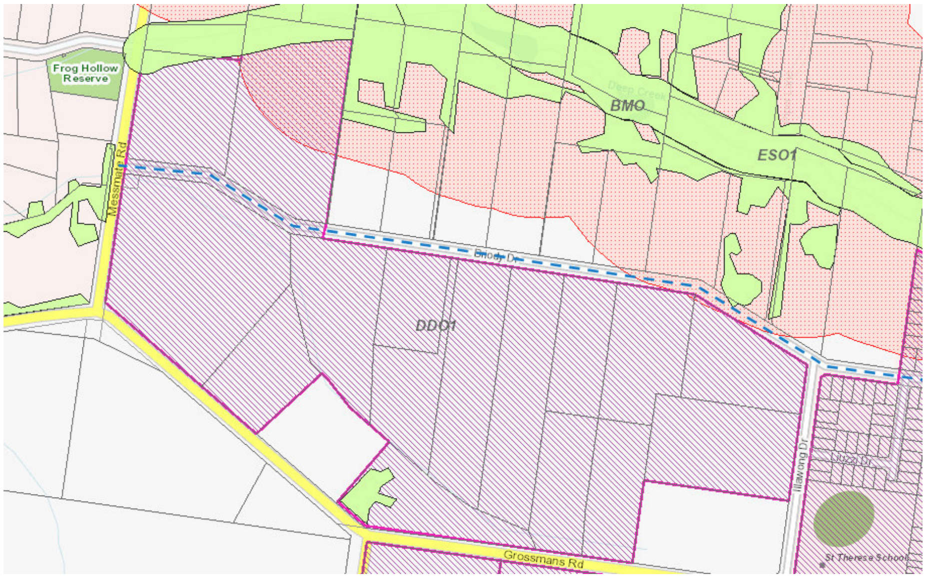

Figure 3: BDW DP with planning overlays: DD0 (Schedule 1), BMO, ESO (Schedules 1 and 4), DPO (Schedule 10). DCP Overlay (Schedule 2) has been removed for clarity.

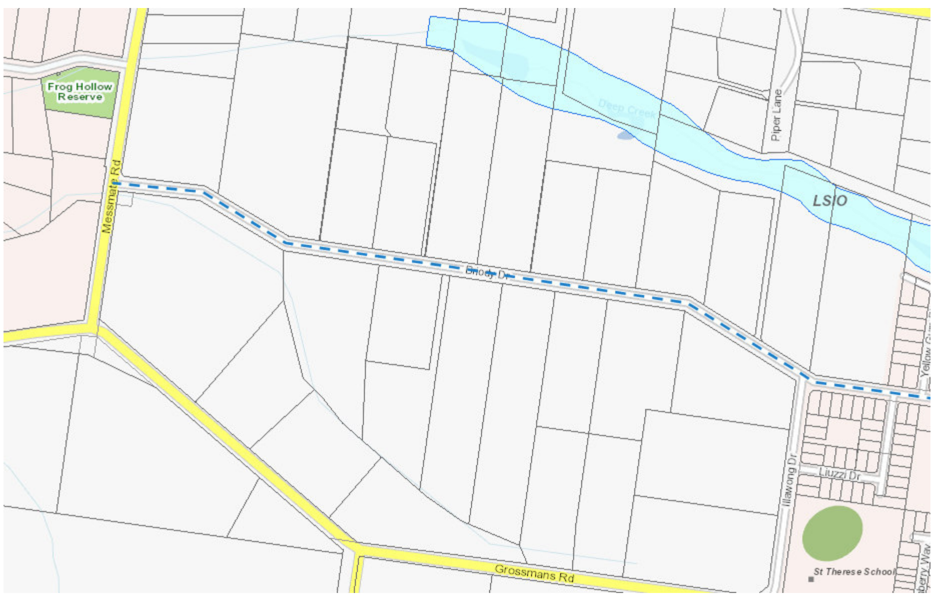

Figure 4: No Flood related overlays (SBO, LSIO, FO) are currently present in the BDW DP. An LSIO does apply to Deep Creek from a section starting downstream of the Western Catchment outlet.

No properties making up the BDW DP parcel are identified as having any flood overlays with regard to data provided on the CCMA flood portal (accessed 01/10/19).

2.2 Existing Stormwater Studies

A Stormwater Management Strategy was prepared by Peter Berry & Associates Pty Ltd (version 5, dated December 5, 2017) to meet the requirement of DPO10.

Key points of the existing SWMS (Berry, 2017) included that:

- The site is comprised of three existing catchments (denoted in Figure 5 as East Catchment - 23.06Ha, South Catchment – 2.80Ha and West Catchment – 7.46Ha).
- The runoff from the West Catchment is conveyed into a new treatment wetland and retarding basin located near the northern interface with Deep Creek. Minor and Major flows are conveyed to the basin and then discharged into Deep Creek at pre-developed flow rates, with provision for a new angled endwall (subject to CCMA application).
- The drainage from the East Catchment has been assumed to be conveyed to Deep Creek directly without detention.
- Within the sub-division the 10-year runoff volumes are to be conveyed using a piped network, with overland flow routes to provide conveyance of the gap flows for events up to and including the 100-year.
- 3-month flows from the East Catchment are to be directed to a treatment wetland to be located in the north-east corner of the BDW DP before being discharged to Deep Creek (larger flows are to bypass beneath this system).
- Based on topography it is not likely to be possible to direct the southern catchment (shown in pink in Figure 5) into the western catchment. Majority of the southern catchment naturally falls towards the eastern outfall.
- Cost estimates were prepared for the proposed drainage works, to inform the Developer Contributions Plan (which would be managed via the use of S173 agreement as a planning permit condition).

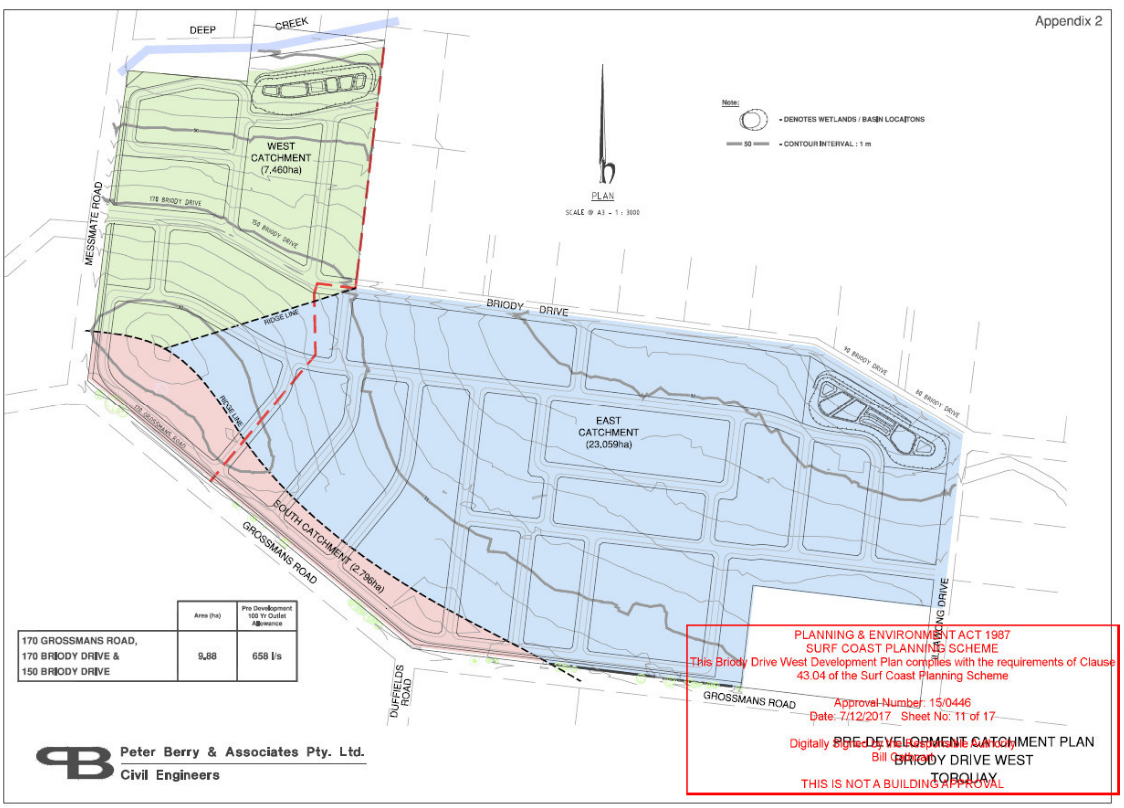

Figure 5: Catchment Plan provided as part of Stormwater Management Strategy by Peter Berry

A Flood Impact Assessment has since been completed by Water Technology (V01, dated 28 August 2019). The purpose of the assessment was to determine inundation for the 1% AEP flood event within Deep Creek, in the vicinity of the BDW DP. The assessment also included a review of the impacts of the proposed BDW DP on a range of additional flood events in Deep Creek. The assessment considered both a mitigated scenario (where all flows from the site are restricted to pre-development flow rates) and a scenario where there is no retardation of flows being provided for either catchment from the development site. Key outcomes from the report include:

- That modelled 1% AEP flood depths within Deep Creek show a minor increase in flood depths downstream of the BDW DP site, for both the developed and mitigated (site retardation) scenarios. This is a result of the increase in flow volume (ie. longer flow durations from the site). **This change is a minor increase of only 6%, which has been considered insignificant by Water Technology during further conversation. Water Technology also provided advice that this change is unlikely to have any detrimental impacts to the established waterway.*
- Minor decreases along the reach of Deep Creek between the proposed outfalls from the Western Catchment and Eastern Catchment. In the developed non-retarded scenario the peak flows from the site can discharge to the creek and pass downstream before the higher Deep Creek flood flows reach the outfalls from the site.
- That the channel within the portion of the creek adjacent to the BDW DP is well defined with an estimated bed level of 5m below bank level. No meaningful changes in flooding were identified in modelled extents for the 10% AEP and 63.2% AEP events.

Generally speaking, the report showed that the section of Deep Creek nearby the subject site has a large 'slow' (primarily rural) catchment, and that adding a small 'fast' (developed) catchment has very little impact on the existing waterway inundation due to the peak flows flushing through before the major flood flow comes down the creek. The results of this analysis has been used to determine the most suitable approach to flow conveyance and retardation at the site.

An extract from the report showing flood levels in Deep Creek at various locations as part of the existing, developed and mitigated (site retardation) scenarios is provided below, with the full report provided in Appendix C.

TABLE 4-9 1% AEP 1.5 HR DURATION - FLOOD LEVEL COMPARISON (M AHD)

Location	1 (m AHD)	2 (m AHD)	3 (m AHD)	4 (m AHD)	5 (mA AHD)	6 (m AHD)
Existing	47.10	42.66	37.82	31.33	27.50	27.47
Developed	47.10	42.66	37.82	31.30	27.56	27.54
Mitigated	47.10	42.67	37.83	31.33	27.54	27.52

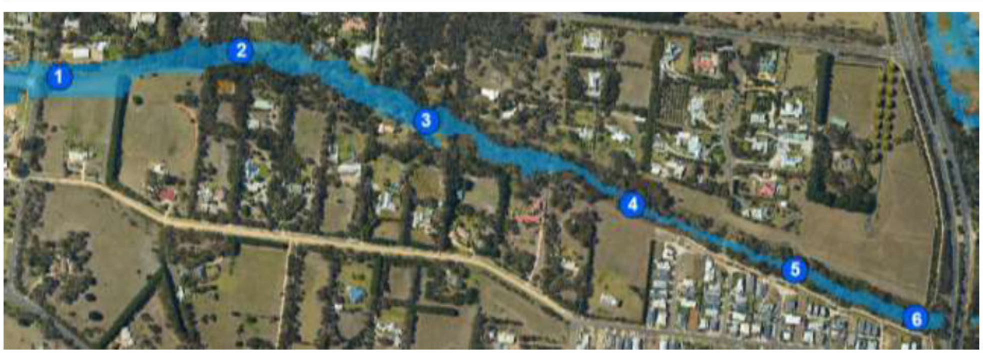

Figure 6: Key Extracts from Flood Impact Assessment (Water Technology)

3 HYDROLOGICAL AND HYDRAULIC ANALYSIS

The objectives of the hydrological and hydraulic analysis are to:

- Determine existing and proposed catchment boundaries and outlet conditions;
- Determine the magnitude of stormwater flows to be conveyed, treated and potentially retarded and thus to inform stormwater asset sizing; and
- Determine the associated flowrates required to be conveyed by major and minor drainage networks.

3.1 Catchments and Outfall Conditions

The topography results in the site being split into two main catchments (East Catchment and West Catchment), each with an outlet into Deep Creek. A copy of the catchment plan is provided in Appendix A.

Table 3.1: Catchments and Outfall Conditions

Catchment	Outfall Description
East Catchment	The east catchment will drain to the north-east corner of the site, with stormwater treatment located within (under) a proposed reserve. All flows for events up to the 1% AEP will be conveyed to a newly formed outlet in Deep Creek via a 1% AEP pipe located within the 8m wide reserve, immediate east of No.90 Briody Drive that connects Briody Drive to the creek. The exact details of installation of this pipe will need to be confirmed during detailed design with regard to any impacts on existing Tree Protection Zones (TPZs) in this location (refer to Arborist Report prepared by AXIOM) and through discussions with the relevant property owners. Exact outlet details will also need to be confirmed as part of the response to the Works on Water Application to the CCMA. Briody Drive will be reconstructed as part of development and will include provision for formalised drainage infrastructure. This formalised drainage infrastructure will be designed to convey flows up to the 1% AEP scenario in an underground piped network. The new road construction works in conjunction with conveyance of all flows up to the 1% AEP within a pipe network will reduce current nuisance flooding occurring at No.90 Briody Drive. The internal drainage network throughout the proposed development is to be designed so that the piped outlet conveys flows up to the 1% AEP scenario. To accommodate this, the piped network throughout the catchment must be sized gradually from the typical 20% AEP flow conveyance in the southern (upstream) portion of the catchment to 1% AEP flow conveyance towards the north-eastern (downstream) portion and the outlet. Flows in extreme events beyond the 1% AEP will be directed such that they are able to utilise the existing overland flow-path located within this 8m reserve.

Catchment	Outfall Description
West Catchment	The west catchment will drain to a reserve near the north-west corner of the site, with stormwater treatment located in this reserve. Flows for events up to the 3-month ARI will be conveyed into the treatment asset. A new piped outlet to provide for all flows up to the 1% AEP will be constructed to convey flows to a newly formed outlet in Deep Creek which adjoins this reserve. Exact outlet details will also need to be confirmed as part of the response to the Works on Water Application to the CCMA.

3.2 Developed Conditions

The developed parcel is intended to be a mix of standard and higher density residential, public open space and include a retirement village and residential aged care facility. The following fraction impervious values (f) and run-off co-efficients (C) were adopted for the run-off analysis, with design rainfall intensities calculated using Australian Rainfall and Runoff (ARR) 2019.

Table 3.2: Developed Catchment Characteristics

Catchment	Area (Ha)	Overall Fraction Impervious (f)	$C_{1\%AEP}$	$C_{20\%AEP}$
East	29.5	0.76	0.83	0.66
West	7.6	0.73	0.80	0.64

Note: 1% Annual exceedance probability (AEP) represents a 100 year average recurrence interval (ARI) rain event (see Section 2.2.3 of ARR2016). Similarly a 20% AEP reflects a 4.48 year ARI rain event.

3.3 Drainage Network

3.3.1 Minor Flows

The proposed development is to be designed so that the piped drainage conveys flows up to the 1% AEP scenario. To accommodate this, the piped network throughout the catchment must be sized gradually the typical 20% AEP flow conveyance in the southern (upstream) portion of the catchment to 1% AEP flow conveyance towards the north-eastern (downstream) portion and the outlet. This will result in pipes sized to convey >20% AEP flows in proximity to the outlet positioned in the north-east corner of the BDW DP. This will reduce the need for large 1% AEP capture pits at the outlet and reduce flooding risk.

The internal drainage network will be designed using a pit and pipe network into stormwater quality treatment assets with appropriate capacity in accordance with the IDM and other relevant Surf Coast Shire design guides (see Standard C25, Cl56.07-4 of the VPP, and Section 16 of the current version of the Infrastructure Design Manual, IDM).

3.3.2 Major Flows

The major drainage system (overland flow paths) carries the ‘gap flow’, which is defined as the difference between the 1% AEP storm event and the piped flows (generally 20% AEP). The drainage network is to be designed so that gap flow will be at bare minimum on Briody Dr and on the roads in close proximity. This is shown clearly in Table 3.4: Developed Catchment Characteristics.

Typical road cross-sections have been modelled to approximate the maximum flow rate able to be safely conveyed at varied grades in accordance with Melbourne Water's velocity-depth criteria i.e. average VxD (velocity–depth product) less than 0.35. A table showing the conveyance capacity of a number of typical cross-sections is provided as follows.

Table 3.3: Road Conveyance Capacity

Road/Drainage Reserve Width (m)	Longitudinal Grade (1 in x)	Design Gapflow capacity of road (m^3/s)	Velocity Depth, v-d Safety Criteria (must be less than 0.35)
15m[1]	150	1.2	0.08
16m[2]	100	1.9	0.09
16m	150	1.6	0.09
16m	200	1.4	0.09
20m mew[3]	150	5.0	0.31

Note 1: 10mm freeboard at road reserve boundary, see Figure 7
Note 2: 10mm freeboard at road reserve boundary, see Figure 8
Note 3: 200mm freeboard at reserve boundary, see Figure 9

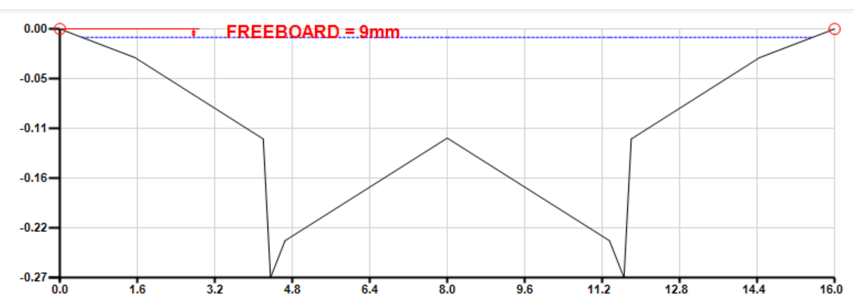

Figure 7**:**

15m Road Reserve

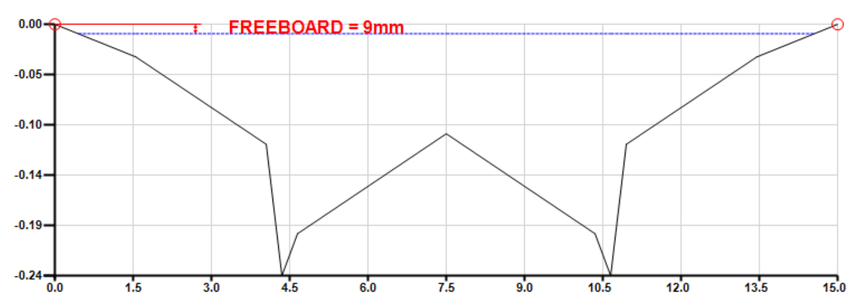

Figure 8:

16m Road Reserve

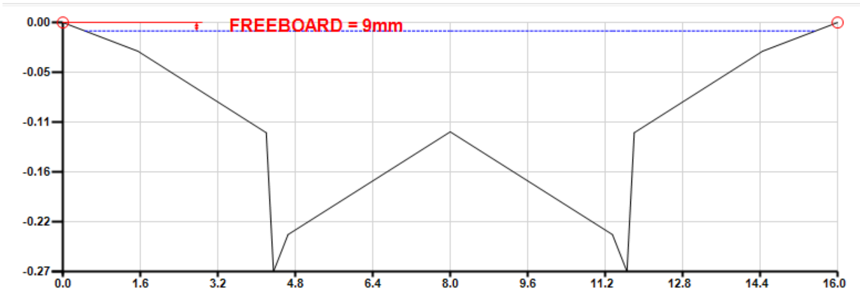

Figure 7:

15m Road Reserve

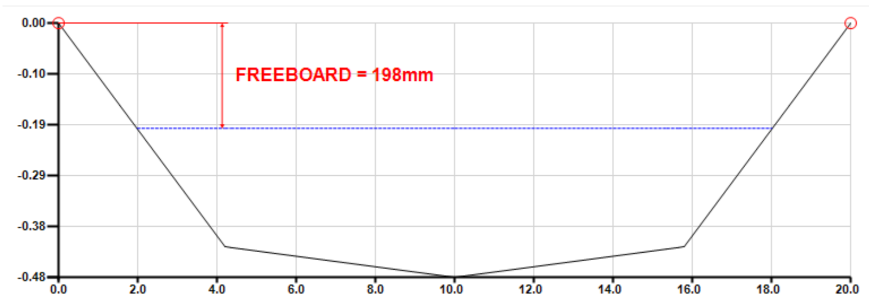

Figure 9:

20m Walkway Reserve

The total flow and maximum gap flows under developed conditions were calculated for a number of key points throughout the road network (using ARR2016) and are provided in Table 3.4 below with these locations shown in Appendix A.

These values can be compared to the design capacity of different standard road widths provided in previous Table 3.3, for confirmation of the road cross-sections and widths required to convey these flows into the stormwater treatment facilities and ultimate outfall pipes. This advice has been used in developing the updated urban design for the proposed development. The gap flow conveyance capacity of the road sections is based on nature strips having positive grades (ie. downward from the title boundary to the back of kerb) on both sides of the road. *Gap flows and piped network sizing are to be confirmed during detail design to satisfy relevant authority requirements*

Table 3.4: Developed Catchment Characteristics

Key Road Location [1]	$Q_{1\%}$	$Q_{pipe\ (1\%-20\%)}$ [2]	Q_{gap} [3]
0	4.1	4.1	0.0
1	3.4	3.4	0.0
2 [4]	1.6	1.6	0.0
3 [4]	2.0	2.0	0.0
4	1.2	0.6	0.7
5	0.8	0.3	0.4
6	1.0	0.4	0.6

Note 1: See Appendix A for Key Road Gap Flow Locations
Note 2: Internal Minor Drainage pipe capacities vary from 1% AEP to 20% AEP to mitigate flood risk along Briody Dr. Key Road Locations 0, 1, 2 & 3 are proposed to have 1% AEP capacity therefore no gap flow required.
Note 3: Q_{gap} is defined by the difference between $Q_{1\%}$ and Q_{pipe}. Where Q_{gap} = 0.0, this means that piped drainage has 1% AEP capacity, and the drainage network will be designed so that flows will enter the piped drainage as efficiently as possible through side entry pits.
Note 4: Key Road Locations 2 and 3 Q_{pipe} and Q_{gap} to be refined during design to satisfy relevant authority requirements.

3.4 Drainage Outfalls

An assessment of outfall pipe sizes and grades to Deep Creek for each catchment are also provided as follows. Pipes have been sized for conveyance of the 1% AEP event, with no retardation beyond what is required for stormwater treatment, based on the findings of the Flood Impact Assessment of Deep Creek by Water Technology (see notes in Section 2.2). Exact requirements of the connection points to Deep Creek will be confirmed via a Works on Waterway Submission to the CCMA upon finalisation of this document (which will provide the background and technical information to support the application)

Table 3.4: Outfall Sizing

Catchment Outfall	$Q_{1\%}$	Indicative Pipe Size	Indicative Pipe Grade
East	**4.3**	1200mm dia	1 in 75 (approx.)
West	**1.5**	825mm dia	1 in 40 (approx.)

4 WATER QUALITY TREATMENT

Conventional approaches to stormwater management within urbanisation impact on the natural hydrology in a variety of ways. This includes increased runoff volumes and frequency, increased flash flooding and reduced infiltration. This leads to erosion of watercourses and the possible damage to riparian and fringing vegetation. Urbanisation is also a primary factor in stormwater runoff quality and the consequent pollution of receiving waters with pollutants such as sediments, hydrocarbons, nutrients and gross pollutants (ACT Government, 2014).

Water sensitive urban design (WSUD) is an approach to integrating the urban water cycle into urban planning and design and mitigating the impacts of urbanisation on waterways. Key principles of WSUD as outlined by Victorian Stormwater Committee (1999) in the Best Practice Environmental Management Guidelines (BPEMG) are to:

- Protect and enhance natural water systems within urban environments;
- Integrate stormwater treatment into the landscape, maximising the visual and recreational amenity of developments;
- Improve the quality of water draining from urban developments into receiving environments;
- Reduce runoff and peak flows from urban developments by increasing local detention times and minimising impervious areas; and
- Minimise drainage infrastructure costs of development due to reduced runoff and peak flows.

4.1 Water Quality objectives

Guidelines for urban stormwater quality management in Victoria are contained in *Urban Stormwater Best Practice Environmental Management Guidelines (BPEMG)*. These guidelines are included in all municipal planning schemes as State Policy. The guidelines seek to minimise the detrimental effects of urbanisation on receiving waterways. Table 4.1 lists the water quality objectives.

Table 4.1: Best Practice Stormwater Management Objectives

Pollutant	Water Quality Objectives
Total Suspended Solids (TSS)	80% retention of the typical urban load
Total Phosphorus (TP)	45% retention of the typical urban load
Total Nitrogen (TN)	45% retention of the typical urban load
Litter[1]/Gross Pollutants (GP)	90% retention of the typical urban load

Note 1 – Litter is defined as anthropogenic material larger than five millimetres (Source: Victorian Stormwater Committee, 1999)

4.2 Water Quality Infrastructure Sizing

Model for Urban Stormwater Improvement Conceptualisation v6.2.1 (MUSIC) has been used to determine the indicative size of treatment infrastructure required to ensure the stormwater runoff from the proposed development is treated to the appropriate standard defined in BPEMG targets.

Geelong North rainfall data for the reference period from 1971-1980 (10 years) with a 6 minute time step has been input into the MUSIC, with other parameters adopted with guidance provided in Melbourne Water's MUSIC Modelling Guidelines (MWC, 2018). The average annual rainfall using this template is 533mm with a mean annual evapotranspiration of 1108mm.

Western Catchment

The proposed water quality treatment system is proposed to consist of a constructed wetland with a sediment pond located at the inlet. Flows beyond the 3-month ARI event will be bypassed from entering the sediment pond and wetland, and directed to the Deep Creek outlet via the new Q100 1% AEP) pipe.

The treatment asset sizing has been calculated with the latest version of MUSIC and the latest proposed catchment extents, and may have reduced in area from the footprint proposed in the earlier Stormwater Management Strategy by Peter Berry and Associates. This smaller footprint may assist with protection of the existing native vegetation to which is located in this future reserve area (the significance of which was indicated as high in the earlier vegetation assessment report).

Table 4.1: Best Practice Stormwater Management Objectives

Catchment	Size (ha)	Q-3month (m^3/s)	Likely required area for Stormwater Treatment (m^2, measured at NWL)	Estimated Total Required Asset Footprint (m^2, total)
West	7.55	0.14	1,900	3,000

A screenshot of the MUSIC model and results showing best practice targets are met is also provided below.

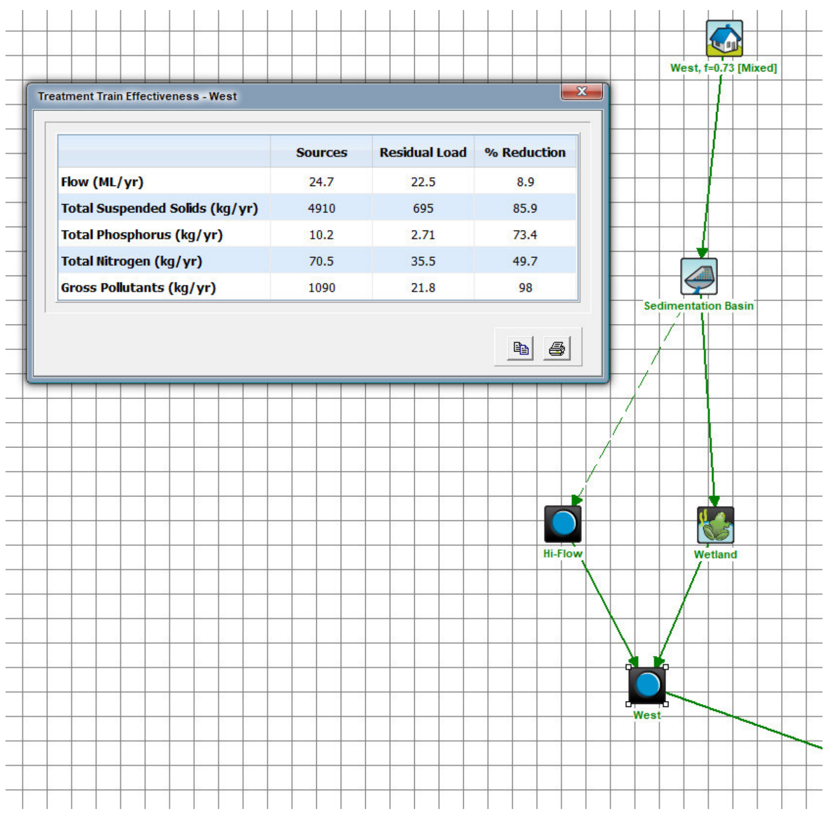

Figure 10: MUSIC model results (Western Catchment)

Eastern Catchment

Treatment for the eastern catchment is proposed to be with the use of SPEL (or approved equivalent) proprietary treatment products located in the reserve (20m mew) at the north-east corner of the BDW DP. This proposal has received in-principal support by Council in discussions between Spiire and Surf Coast Shire in September 2019. Asset siting with regard to maintenance, amenity and clearance to any other services will be confirmed as part of the future functional design to be prepared for this site. Treatment device selection will be completed with regard to the SQID Database (Stormwater Australia) and MUSIC modelling, and is likely to comprise of Stormceptor and Hydrosystem units.

Flows beyond the 3-month ARI event will be bypassed from the treatment devices, and directed to the Deep Creek outlet via the new Q100 (1% AEP) pipe. The treatment will be sized to ensure BPEMG targets are being met for the entire development area as required by the permit area, in accordance with local and state policy discussed earlier.

A screenshot of the MUSIC model and results showing best practice targets are met is also provided below.

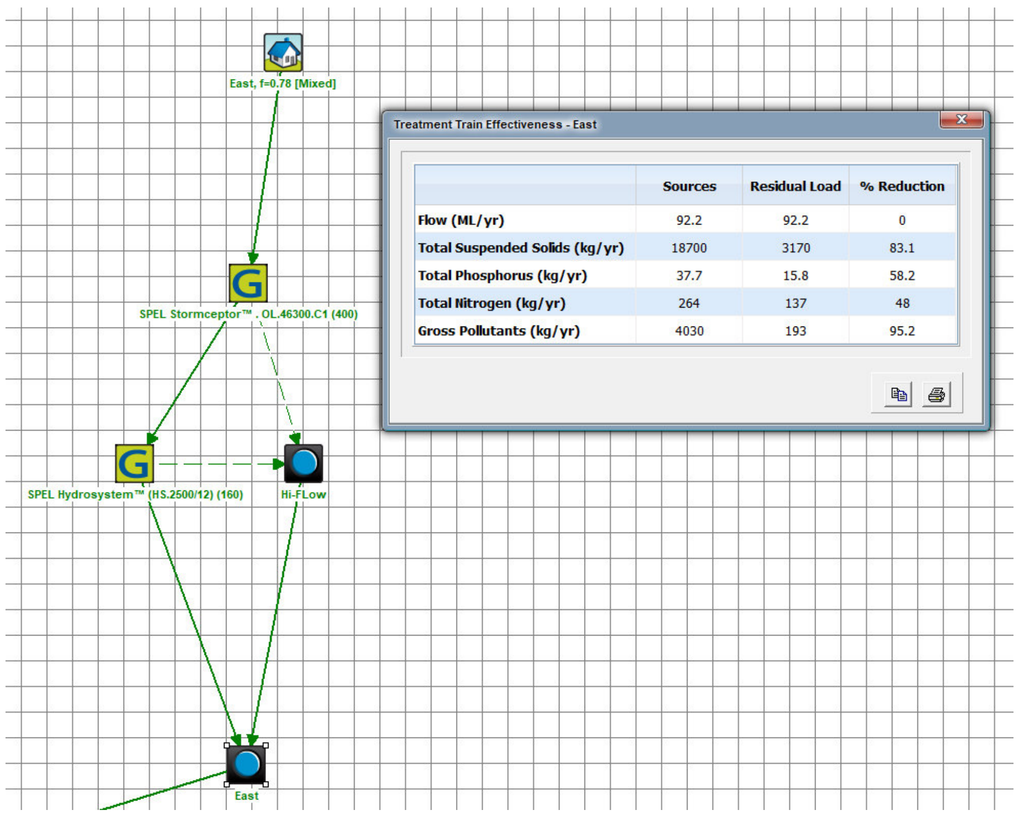

Figure 11: MUSIC model results (Eastern Catchment)

5 COST SHARING AND STAGING

The Shared Infrastructure Funding Plan has been updated to reflect the revised Development Plan as well as update costs for land and infrastructure based on more recent land valuations or refined cost estimates for infrastructure. It is noted that the Staging Plan proposed is generally consistent with that already approved, however additional detail regarding critical infrastructure to be delivered with the first stage of subdivision has been identified.

6 CONCLUSIONS

This report outlines an updated stormwater management strategy, in accordance with objectives of:

- the previously endorsed Development Plan and associated documents;
- Schedule 10 of the Development Plan Overlay (DPO10);
- Local and state planning policies for stormwater management.

The updated stormwater management strategy takes into account additional relevant studies of the subject site and the hydrologic impacts of its development, including a Flood Impact Assessment of Deep Creek completed by Water Technology (see Appendix C). It also includes recent inputs from discussion with Surf Coast staff regarding options for stormwater treatment at the site.

In summary, the development of the proposed site would include:

- Treatment of all stormwater runoff within the development plan area to best practice targets (80/45/45) with the installation of:
  - SPEL Stormceptor and Hydrosystem units (eastern catchment)
  - Constructed wetland and sedimentation basin (western catchment)
- Provision of a major and minor drainage network throughout the development area to safely convey 1% AEP and 20% AEP flows respectively, and confirmation that the provided road widths will be sufficient for these purposes;
- Collection of stormwater run-off from the site (including Briody Drive and additional small upstream catchment areas outside the Development Plan area), for discharge via newly constructed Q100 (1% AEP) pipes to Deep Creek. These outlets will be provided for the eastern and western catchments (to the satisfaction of the CCMA);

This strategy and the inclusion of its associated stormwater management measures is recommended for adoption under an amended Development Plan for the area.

APPENDIX A

CATCHMENT PLAN

file name 306395CD500.dwg layout name CD500 plotted by Jarrod Gaut
file location G:\30\306395\Civil\ACAD plot date 16/10/2019 3:34 PM Sheet 5 of 6 Sheets

Rev	Amendments	Approved	Date
A	PRELIMINARY		16.10.19

Scale

spiire
10 MOORABOOL STREET GEELONG
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spiire.com.au ABN 55 050 029 635

Designed JG
Checked
Authorised
Date

BRIODY DRIVE WEST DP

DRAINAGE
DRAINAGE CATCHMENT PLANS - SHEET 1
SURF COAST SHIRE
BRIODY DRIVE PROJECT PTY LTD

PRELIMINARY
Drg No 306395CD500
Rev A

APPENDIX B

RAINFALL AND RUNOFF COMPUTATIONS

spiire

Project:	**Briody Drive West**	**Designed:**	JG
Reference No:	**306395**	**Checked:**	

Annual Exceedance Probability (%)

AEP Coefficients	63.20%	50%	20%	10%	5%	2%	1%
C0	0.07808984	0.22660825	0.60322022	0.80864781	0.98394233	1.1880969	1.3278757
C1	0.65767115	0.66171199	0.65764624	0.64538169	0.62873858	0.51449609	0.43679208
C2	0.19950064	0.18559358	0.1690993	0.17281321	0.18357731	0.31278628	0.40060654
C3	-0.1446233	-0.1338045	-0.11548713	-0.1114237	-0.1113812	-0.1633779	-0.19868696
C4	0.03368737	0.03049739	0.024454879	0.022439523	0.0214352	0.03092215	0.037356488
C5	-0.00336	-0.0029748	-0.00220834	-0.00191798	-0.0017396	-0.002545765	-0.003091325
C6	0.00012178	0.00010553	7.23E-05	5.90E-05	5.01E-05	7.61E-05	9.36E-05

AEP to ARI Conversion

AEP %	ARI
63.20%	1
50%	1.44
20%	4.48
10%	10
5%	20
2%	50
1%	100

1% AEP URBAN ARI Drainage Calculations

DEVELOPED CATCHMENT

Catchment	Notes	Area (ha)	∑A (ha)	C 1%	C 20%	Ae 1% (ha)	∑Ae 1% (ha)	Ae 20% (ha)	∑Ae 20% (ha)	Flow Length (m)	Velocity 1% (m/s)	Velocity 20% (m/s)	Tc 1% (mins)	Tc 20% (mins)	Int 1% (mm/hr)	Int 20% (mm/hr)	Q 1% m3/s	Qpipe m3/s	Qgap m3/s
E0	**Catchments**	1.35	1.35	0.88	0.69	1.18	1.18	0.94	0.94	640	0.8	1.5	18.33	12.11	79.11	47.20	0.26	0.12	0.14
E00		0.26	0.26	0.67	0.53	0.18	0.18	0.14	0.14	60	0.8	1.5	6.25	5.67	130.80	66.58	0.06	0.03	0.04
E1		4.89	4.89	0.88	0.69	4.29	4.29	3.40	3.40	485	0.8	1.5	15.10	10.39	88.32	51.03	1.05	0.48	0.57
E2		7.57	7.57	0.88	0.69	6.64	6.64	5.26	5.26	300	0.8	1.5	11.25	8.33	102.80	56.67	1.90	0.83	1.07
E3		3.25	3.25	0.88	0.69	2.85	2.85	2.26	2.26	770	0.8	1.5	21.04	13.56	72.83	44.46	0.58	0.28	0.30
E4		6.71	6.71	0.82	0.65	5.48	5.48	4.34	4.34	770	0.8	1.5	21.04	13.56	72.83	44.46	1.11	0.54	0.57
E5		1.98	1.98	0.88	0.69	1.74	1.74	1.38	1.38	400	0.8	1.5	13.33	9.44	94.41	53.46	0.46	0.20	0.25
E6		0.89	0.89	0.88	0.69	0.78	0.78	0.62	0.62	250	0.8	1.5	10.21	7.78	107.59	58.45	0.23	0.10	0.13
E7		0.54	0.54	0.67	0.53	0.36	0.36	0.29	0.29	120	0.8	1.5	7.50	6.33	122.47	63.75	0.12	0.05	0.07
E8		2.06	2.06	0.52	0.41	1.08	1.08	0.85	0.85	120	0.8	1.5	7.50	6.33	122.47	63.75	0.37	0.15	0.22
W1		2.82	2.82	0.88	0.69	2.47	2.47	1.96	1.96	480	0.8	1.5	15.00	10.33	88.66	51.17	0.61	0.28	0.33
W2		3.62	3.62	0.88	0.69	3.18	3.18	2.51	2.51	335	0.8	1.5	11.98	8.72	99.69	55.50	0.88	0.39	0.49
W3		1.11	1.11	0.37	0.29	0.41	0.41	0.33	0.33	80	0.8	1.5	6.67	5.89	127.90	65.61	0.15	0.06	0.09
0	**Road**	27.89	27.89	0.83	0.66	23.23	23.23	18.39	18.39	1025	0.8	1.5	26.35	16.39	63.17	40.02	4.08	2.04	2.03
1	**Capacity**	25.29	25.29	0.75	0.60	19.06	19.06	15.09	15.09	995	0.8	1.5	25.73	16.06	64.16	40.49	3.40	1.70	1.70
2	**Checks**	20.94	20.94	0.29	0.23	6.04	6.04	4.78	4.78	405	0.8	1.5	13.44	9.50	94.03	53.31	1.58	0.71	0.87
3		15.43	15.43	0.46	0.37	7.17	7.17	5.68	5.68	300	0.8	1.5	11.25	8.33	102.80	56.67	2.05	0.89	1.15
4		15	15	0.33	0.26	4.97	4.97	3.94	3.94	485	0.8	1.5	15.10	10.39	88.32	51.03	1.22	0.56	0.66
5		11.91	11.91	0.26	0.21	3.09	3.09	2.44	2.44	480	0.8	1.5	15.00	10.33	88.66	51.17	0.76	0.35	0.41
6		11.04	11.04	0.33	0.26	3.68	3.68	2.91	2.91	335	0.8	1.5	11.98	8.72	99.69	55.50	1.02	0.45	0.57
Outfall East	**Outfalls**	29.5	29.5	0.83	0.66	24.59	24.59	19.47	19.47	1025	0.8	1.5	26.35	16.39	63.17	40.02	4.31	2.16	2.15
Outfall West		7.55	7.55	0.80	0.64	6.06	6.06	4.80	4.80	480	0.8	1.5	15.00	10.33	88.66	51.17	1.49	0.68	0.81

APPENDIX C

DEEP CREEK FIA (WATER TECHNOLOGY)

Deep Creek Flood Impact Assessment

Deep Creek

Briody Drive Pty Ltd

August 2019

Document Status

Version	Doc type	Reviewed by	Approved by	Date issued
V01	Draft	Ben Hughes	Ben Hughes	28/08/2019

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Quality ISO 9001 SAI GLOBAL | Environment ISO 14001 SAI GLOBAL | Health & Safety AS 4801 SAI GLOBAL

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1 INTRODUCTION

1.1 Overview

Watch Technology has been engaged to undertake an assessment of the existing and developed conditions inundation for the 1% AEP flood event at a proposed future development location. The subject site is located within the township of Torquay, within the Grossmans Road, Messmate Road and Coombes Road area. The location of the subject site is shown in Figure 1-1. The flood assessment was undertaken to define flood risk and inform potential development layouts within the property. The assessment included the development of catchment hydrology using RORB. Flows developed as part of the RORB model were used as inflow boundaries to a TUFLOW 1D-2D hydraulic model to define flood depth, extent and velocity during 1% and 10% AEP flood events at the subject site.

FIGURE 1-1 DEVELOPMENT SITE

1.2 Study Area

The study area is within the Deep Creek catchment, which includes several small tributaries upstream of the proposed development area. The Deep Creek catchment is shown by the red outline in Figure 1-2 and covers an area of 6.26 km^2.

Deep Creek is a small ungauged waterway within the Torquay area. The creek begins within rural land west of the Surf Coast Highway and passes through low density residential land, before passing under the Surf Coast Highway through a more densely populated urban area, finally discharging to Zeally Bay.

FIGURE 1-2 DEEP CREEK CATCHMENT

1.3 Available Data

The investigation utilised several existing datasets available from the Department of Environment, Land, Water and Planning (DELWP) and Corangamite CMA including:

- Topography – Light Detection and Ranging (LiDAR), 5m resolution, flown 2008 (DELWP)
- Digital Aerial Photography – Flown Feb 2006 (DELWP)
- Spatial Data – VicMap – 2016 (DELWP)

2 HYDROLOGY

2.1 Overview

A hydrologic model of the Deep Creek catchment was developed to determine design flow hydrographs at several locations within the Deep Creek catchment to be used as inflow boundary conditions in the hydraulic model.

RORB is a non-linear rainfall runoff and streamflow routing model for calculation of flow hydrographs in drainage and stream networks. The model requires catchments to be divided into subareas, connected by a series of conceptual reaches and storage areas. Observed or design storm rainfall is input to the centroid of each subarea. Specific initial and continuing losses are then deducted, and the excess runoff is routed through the reach network.

The adopted methodology described below is based on current guidelines described in the 2019 revision of Australian Rainfall and Runoff (ARR2019). An Ensemble approach was used in this assessment. The Ensemble approach modelled 10 available temporal patterns for each duration recommended in ARR2019 with the temporal pattern which determined the median peak flow for each duration adopted.

2.2 RORB Modelling

2.2.1 Model Setup

2.2.1.1 Sub-area and Reach Delineation

Sub-area boundaries and reaches were delineated using ArcHydro and revised as necessary. Delineation was based on the available LiDAR data. Nodes were placed at areas of interest (to extract flow hydrographs), the centroid of each sub-area and the junction of any two reaches. Nodes were then connected by RORB reaches, each representing the length, slope and reach type. The RORB model had 42 sub-areas ranging in area from 0.08 – 0.4 km^2. The sub-catchment delineation and reach network is shown in Figure 2-1. Smaller sub-catchment and 2 interstation areas were established for the eastern and western portions of the development catchment.

The RORB model was constructed using MiRORB (MapInfo RORB tools), RORB GUI and RORBWIN V6.45.

2.2.1.2 Fraction Impervious

Fraction Impervious (FI) values were calculated using MiRORB. Default sub-area FI values were based on an assessment of current Surf Coast Shire Planning Scheme Zones (current January 2019) and aerial imagery. The spatial distribution of the fraction impervious data is shown in Figure 2-2. It can be seen there is a considerable difference in fraction impervious between the urban areas of the catchment and the upper, agricultural areas.

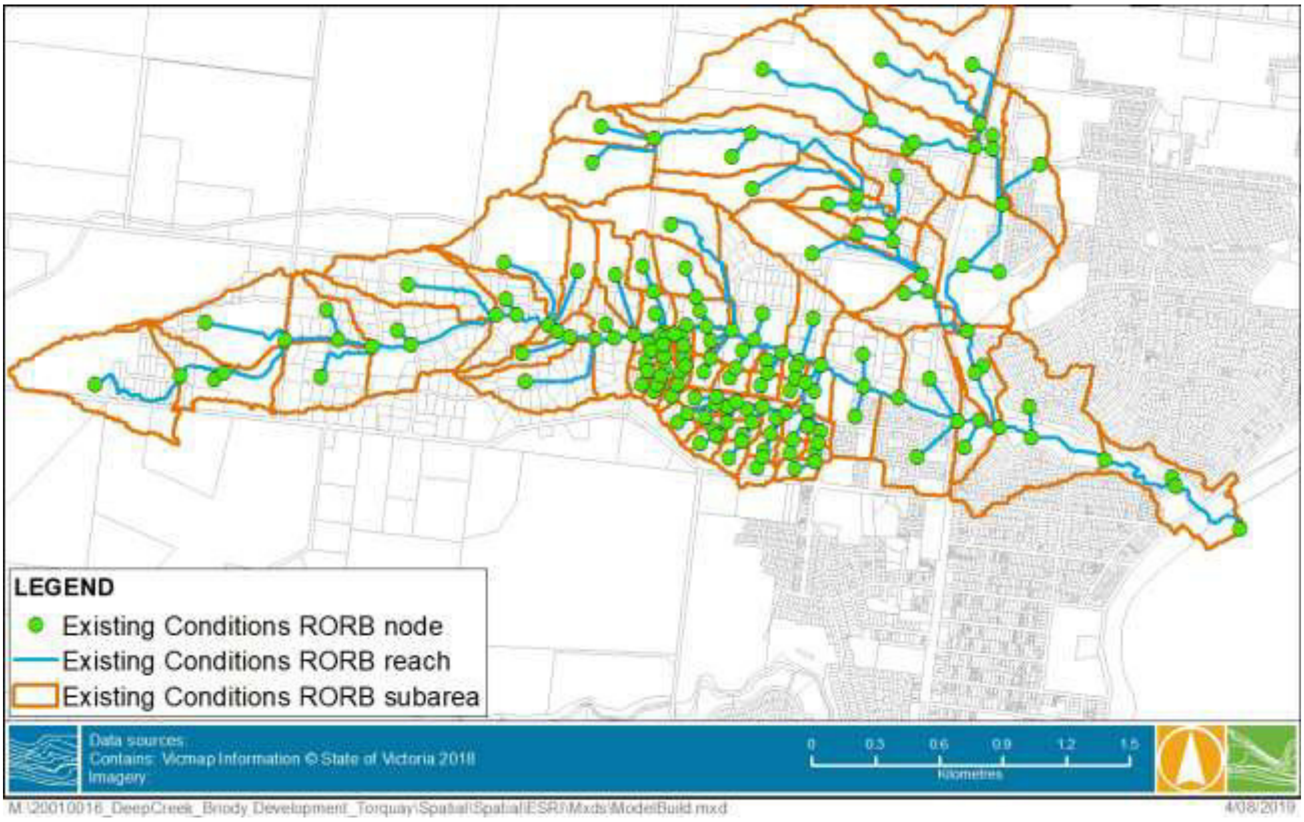

FIGURE 2-1 RORB MODEL SCHEMATISATION

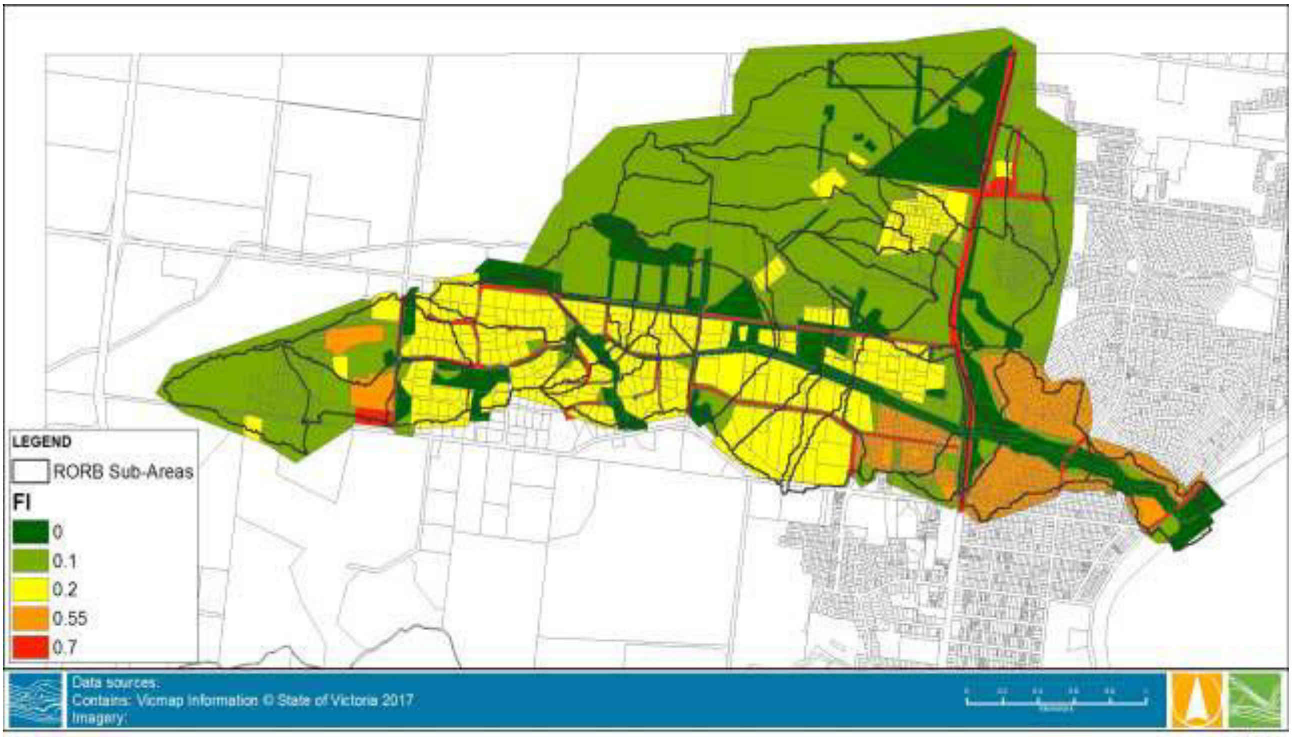

FIGURE 2-2 FRACTION IMPERVIOUS DISTRIBUTION IN THE DEEP CREEK CATCHMENT

Design rainfall depths were determined using the Bureau of Meteorology online IFD tool[1]. The rainfall Intensity Frequency Duration (IFD) parameters were generated for a location in the approximate centre of the Deep Creek catchment (38.31S, 144.27E) and are shown in Table 2-1 below.

TABLE 2-1 DESIGN RAINFALL DEPTH (MM) FOR STORM FREQUENCY AND DURATION

	EY	Annual Exceedance Probability (AEP)					
Duration	**1EY**	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**
1 hour	10.7	12.3	17.6	21.4	25.4	30.9	35.3
2 hour	14	16	22.4	27	31.7	38	43.1
3 hour	16.7	18.9	26.1	31.2	36.4	43.4	49.1
6 hour	22.9	25.6	34.5	40.8	47.2	56.3	63.6
12 hour	31	34.6	46.3	54.6	63	75.6	85.7
24 hour	40.2	45.1	61.2	72.7	84.4	102	116
48 hour	48.5	55.1	76.9	92.6	109	131	149
72 hour	52.4	59.9	84.8	103	122	146	166
96 hour	55.2	63.1	89.4	109	129	154	174
120 hour	57.7	65.7	92.3	112	132	158	178
144 hour	60.2	68.1	94.4	114	133	160	180
168 hour	63	70.6	96	114	133	160	180

2.2.1.3 Temporal Patterns

Temporal patterns from ARR2019 were utilised in the analysis and extracted from the AR&R data hub. As previously described and Ensemble approach was undertaken. The range of temporal patterns modelled are included in Appendix A, with relevant ID numbers assigned as referred to in the RORB model output. The Southern Slopes (Vic/NSW) Zone of temporal patterns was utilised. The ARR2019 temporal patterns are based on historical storms using the extensive network of pluviograph data collected by the Bureau of Meteorology (BoM).

The ARR2019 design temporal patterns were broken into several AEP groupings, these included:

- Very Rare – Rarest 10 within region
- Rare – Suitable AEP range 3.2% AEP and rarer
- Intermediate – Suitable for AEP range 3.2% - 14.4%
- Frequent – Suitable for AEP range more frequent than 14.4%

Previous assessment would have used a single temporal pattern across all design events. The ARR2019 approach recommends that at least 10 temporal patterns be used for each event. These 10 temporal patterns change depending on the duration and the event considered.

[1] Bureau of Meteorology Web Tool, http://www.bom.gov.au/water/designRainfalls/revised-ifd/?year=2016

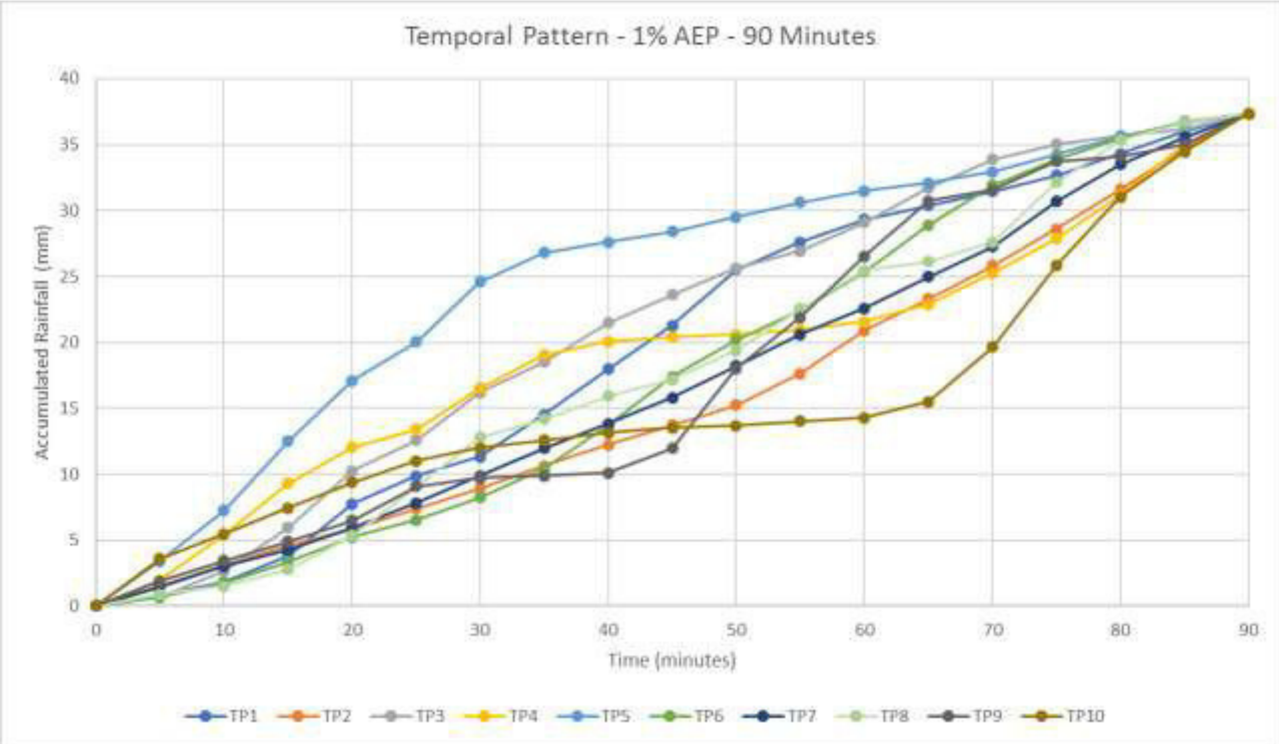

FIGURE 2-3 TEMPORAL PATTERN VARIATION

2.2.1.4 Areal Reduction Factors

Areal reduction factors were used to convert point rainfall to areal estimates and are used to account for the variation of rainfall intensities over a large catchment. AR&R2019 areal reduction factors were applied to the catchment area and extracted from the AR&R data hub[2]. The catchment lies within the Southern Temperate Zone of aerial reduction factors and these were applied for all design modelling.

2.2.1.5 Regional *KC*

KC is the primary routing parameter in RORB. As Deep Creek is an ungauged catchment with no streamflow record, it is not possible to calibrate the RORB model against known catchment flows and rainfall records. As such, a comparison between empirical regional equation estimates was made and a reasonable value within this range adopted. The Pearse et. al. kc prediction equation method is based on Victorian data and has been shown to provide an accurate match to Flood Frequency Analysis (FFA) across several Victorian flood investigations[3] and was used in this project, adopting a *KC* value of 4.59.

TABLE 2-2 CALCULATED KC PARAMETERS

KC Equations	K_c
Default RORB Eqn.	5.45
Victoria data (Pearse et al, 2002)	<u>4.59</u>
Aust Wide Dyer (1994) (Pearce et al)	4.17

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[2] AR&R 2016 Data Hub, http://data.arr-software.org/
[3] Natimuk Flood Investigation (Water Technology, 2014), Hydrology and Hydraulics Assessment, Western Highway Duplication Section 3 (Water Technology, 2017).

KC Equations	K_c
Victoria Mean Annual Rainfall > 800mm	5.81

This is further validated in later sections of this report when comparing adopted and previous design flows. The RORB model was separated into three interstation areas, adopting a varying *KC* value for each.

- Whole of Catchment – *KC* = 4.59
- Site – West Catchment – *KC* = 0.19
- Site – East Catchment – *KC* = 0.45

2.2.1.6 Routing Parameter – m

The RORB 'm' value is typically set at 0.8 as recommended in the RORB User Manual. This value remains unchanged and is an acceptable value for the degree of non-linearity of catchment response (Australian Rainfall and Runoff, 1987). It is rare to vary the 'm' value and there are were no reasons to do so in this study, particularly given the lack of calibration data.

2.2.1.7 Design Losses

ARRR2019, Book 5 Chapter 5 (Hill and Thomson, 2015) contains new recommended initial and continuing losses, as shown below. A web tool has also been developed to derive initial and continuing loss values[4], which was used to extract loss values for this project. The information generated from this web tool in shown in Table 2-3 for the Deep Creek catchment.

TABLE 2-3 DESIGN LOSS PARAMETER ESTIMATES

Source	IL (mm)	CL (mm/h)
ARR 2016 (VIC)	24	4.4

Where - BFI (Baseflow Index) = 0.38, MAR (Mean Annual Rainfall) = 729 mm, PET (Mean Annual Potential Evaporation) is 1275 mm.

Pre-burst loses identified by the ARR databub indicate median pre-burst losses ranging form 0.9 – 3.3 mm. A uniform pre-burst loss of 2mm was adopted for this catchment with the resulting adopted initial loss reducing to 22mm.

In line with recent academic papers (NSW Department of Environment and Heritage[6]) continuing losses as shown by the datahub are likely to be overestimated. This has been verified in several recent studies undertaken by Water Technology[7]. In consideration of this and a comparison of calibrated local flood models a reduced continuing loss of 2.5 mm/hr has been adopted.

Spatial Patterns

The ARR2019 guidelines recommend for non-uniform spatial patterns for catchment areas of more than 20 km^2. The Deep Creek catchment and the upstream catchment of the area of interest are well below this threshold and as such a uniform rainfall pattern for the design modelling was adopted.

[4] ARR2019 - http://data.arr-software.org
[6] NSW Office of Environment and Heritage, Review of ARR design Inputs, 2019
[7] Lara Flood Study, Gnarr Creek and Yarrowee River Flood Mapping Update (Water Technology, 2019)

2.2.2 Design Flows – Existing Conditions

2.2.2.1 RORB – Ensemble

Peak flows for the 1%, 10% and 63.2% Annual Exceedance Probability (AEP) flood events were calculated within the RORB model for durations between the 15 minutes and 48 hour duration events. An ensemble of the 10 available temporal patterns applicable to the 1%, 10% and 63.2% AEP events were run and the event with the median peak flow for each of the modelled durations was adopted.

The whisker plot below shows the upper and lower limits of the calculated peak flows for each of the 10 temporal patters for each duration, along with the corresponding median for each storm duration.

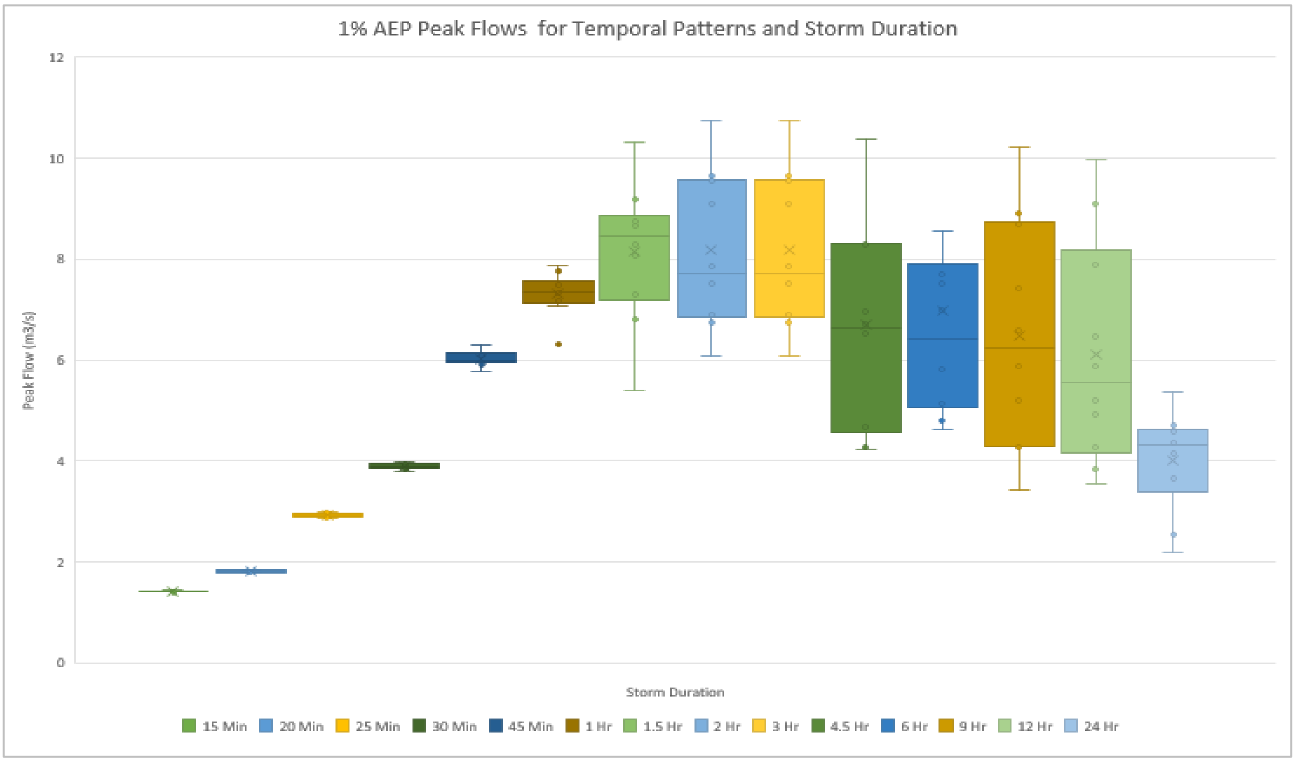

FIGURE 2-4 TEMPORAL PATTERN AND PEAK FLOWS

The event duration which yielded the highest median peak flow was 1.5 hrs. Within the ensemble of temporal patterns, the temporal pattern which gives the peak flow closest (above) the median was TP28. The ensemble outputs for the 1% AEP event in existing conditions are shown in Table 2-4. The highest median results from the ensemble modelling is circled and forms the hydrologic input for the modelled 1% AEP peak flows.

This process of modelling the ensemble of temporal patterns, identifying the maximum of the median ensemble results and selecting the best fit single storm duration and temporal pattern was also undertaken for the 10% AEP and 63.2% AEP events. The adopted peak flows, temporal patterns and critical durations for each of the modelled durations is shown in Table 2-5.

TABLE 2-4 1% AEP RORB ENSEMBLE OUTPUT

Duration	Upstream of Site (Deep Creek)	Downstream of Site (Deep Creek)	Site Western Catchment m^3/s	Site Eastern Catchment m^3/s
15min	1.42	1.63	0.11	0.26
20min	1.83	2.12	0.16	0.31
30min	3.88	4.78	0.38	0.49
1hr	7.35	9.70	0.68	0.90
1.5hr	8.47	12.13	0.70	1.13
2hr	7.71	12.12	0.64	1.08
3hr	7.20	10.83	0.54	1.10
4.5hr	6.64	10.48	0.52	0.98
6hr	6.41	10.42	0.48	1.00
9hr	6.24	10.08	0.40	0.95
12hr	5.55	8.89	0.37	0.83
24hr	3.63	5.75	0.24	0.55

TABLE 2-5 ADOPTED FLOWS AND TEMPORAL PATTERNS

	Upstream of Site (Peak Flow, TP)	Site – West (Peak Flow, TP)	Site -Easy (Peak Flow, TP)
1% AEP (Crit durn 1.5Hr)	8.71 m^3/s, TP28	0.76 m^3/s, TP28	1.15 m^3/s, TP28
10% AEP (Crit durn 3Hr)	3.58 m^3/s, TP15	0.28 m^3/s, TP15	0.46 m^3/s, TP15
63.2% AEP (Peak Flow/TP)	1.00 m^3/s, TP4	0.07 m^3/s, TP4	0.114 m^3/s, TP4

2.2.2.2 Flow Verification

The Deep Creek catchment is ungauged, in the place of observed data the adopted design flows were compared against a range of other flow estimate methods including Rational Method, Regional Flood Frequency Estimation and the Grayson Method, as shown in Table 2-6. The estimation methods (VicRoads and Grayson) produced similar peak outflows to the RORB model for the catchment area immediately upstream of the development site. Whilst these estimation methods are considered to have high uncertainty, they demonstrate that based on the adopted catchment RORB parameters, reasonable flows based on catchment area and IFD parameters have been produced. It is important to note that whilst the RORB flows are higher than the verification methods presented the existing catchment is not considered to be typical rural or undeveloped catchment.

TABLE 2-6 DESIGN FLOW COMPARISON

	Flow (m^3/s)	
	1% AEP (m^3/s)	10% AEP (m^3/s)
Rational (Adams)	2.87	1.34
Rational (VicRoads)	5.73	2.69
RFFE (Rural)	3.26	1.42
Grayson (Rural)	7.31	NA
1% AEP RORB Median Ensemble Results (Upstream of Development)		
30 Minute	3.88	1.02
1 Hour	7.35	1.36
1-5 Hour	8.47	2.08
2 Hour	7.71	2.69
3 Hour	7.2	3.51
6 Hour	6.41	3.23
9 Hour	6.24	2.83
12 Hour	5.55	2.50

2.2.3 Adopted Design Flood Hydrographs

Flows on the Deep Creek were extracted at 3 locations within the catchment boundary. Most critical to the subject site is the model boundary immediately upstream of the development area. Flows for both the 1%, 10% and 63.2% AEP flood events were extracted for several durations including that which produced the maximum peak flow. The respective durations and peak flows for each of the modelled events are shown in Table 2-7 below.

TABLE 2-7 DESIGN FLOWS

AEP	Critical Duration/ Temporal Pattern	Peak Flow Upstream	Peak Flow Site -West	Peak Flow Site - East
1%	1.5hr / TP28	8.71 m^3/s	0.76 m^3/s	1.15 m^3/s
10%	3hr / TP15	3.58 m^3/s	0.28 m^3/s	0.46 m^3/s
63.2%	9hr / TP4	1.00 m^3/s	0.07 m^3/s	0.11 m^3/s

3 HYDRAULIC MODEL DEVELOPMENT

3.1 Model Extent and Topographic Resolution

TUFLOW was used to develop the hydraulic model, with the model extending from west of the subject site to the ocean, including a small tributary entering at the north of the site. Topography of the Deep Creek catchment was available from the 2008 Victorian State Wide LiDAR Project and was used as the basis for a 2 m resolution topography, covering approximately 1.3 km^2. At this grid resolution the width of the creek was appropriately represented. Features such as waterway banks, roads and general floodplain features were well represented by the model. The selected grid size allowed accurate modelling of the site and creek while maintaining manageable model run times.

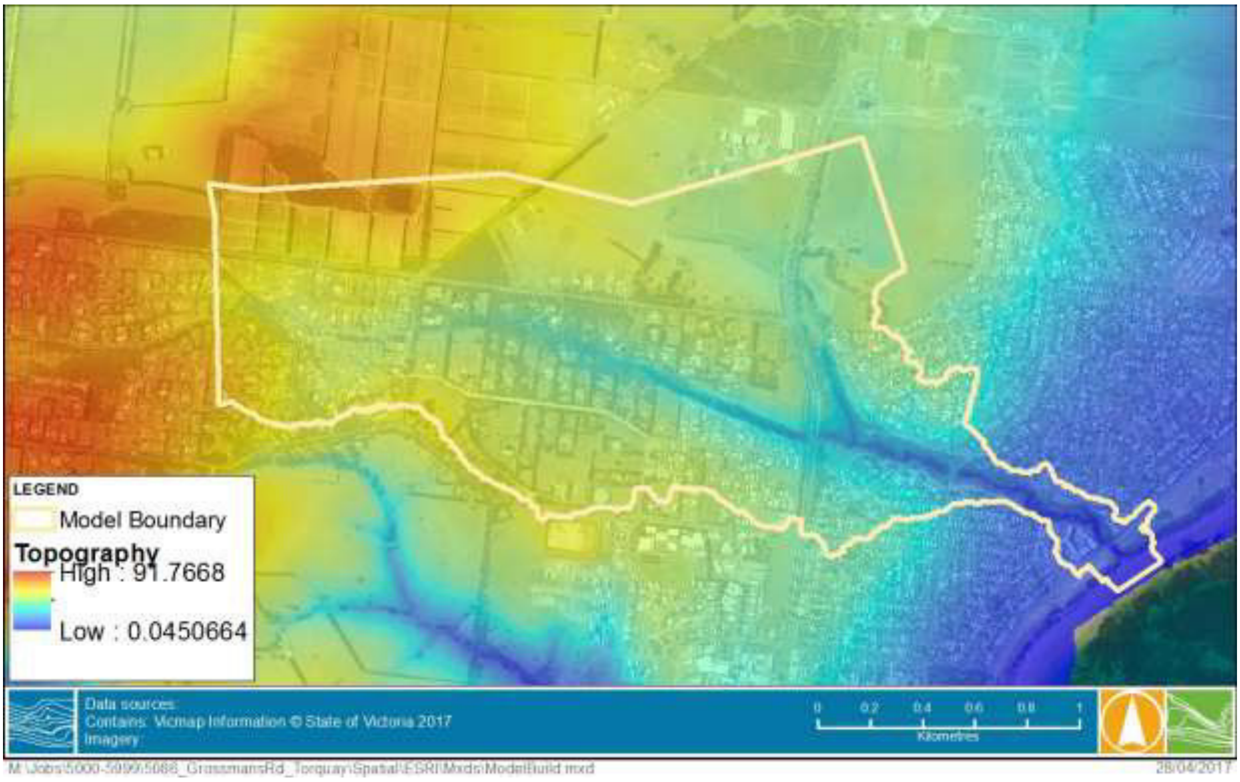

FIGURE 3-1 DEEP CREEK– TOPOGRAPHY

3.1.1 Manning's Roughness

Manning's 'n' was adopted as a representation of floodplain roughness, and has an important impact on flood velocities, flow paths, flood depths and extents. Manning's 'n' roughness values were derived from photographs from the site visit, aerial photography and appropriate industry standard literature (Australian Rainfall and Runoff, Chow (1959), etc).

TUFLOW '*2d_mat'* files were produced based on land use zones, with further refinement through the use of high-resolution aerial photographs and findings from the site visit. The Manning's values were specified in the .tmf (TUFLOW model file). The final layout of Manning's roughness is provided as a model check file and is shown in Table 3-1. They are listed in Table 3-1.

TABLE 3-1 LAND USE MANNING'S 'N' ROUGHNESS VALUES

Material	Manning's n Roughness
Pasture/Cleared farmland	0.04
Medium density vegetation	0.075
Dense vegetation	0.100
Caravans, Semi Permanent structures	0.300
Waterway, cobbled and rocky (upstream)	0.050
Waterway, sandy (Lower reaches)	0.040
Sealed roads	0.020
Tanks	1.000
Buildings	0.300
Rock flats on beach	0.040
Sand/estuary/ocean	0.030

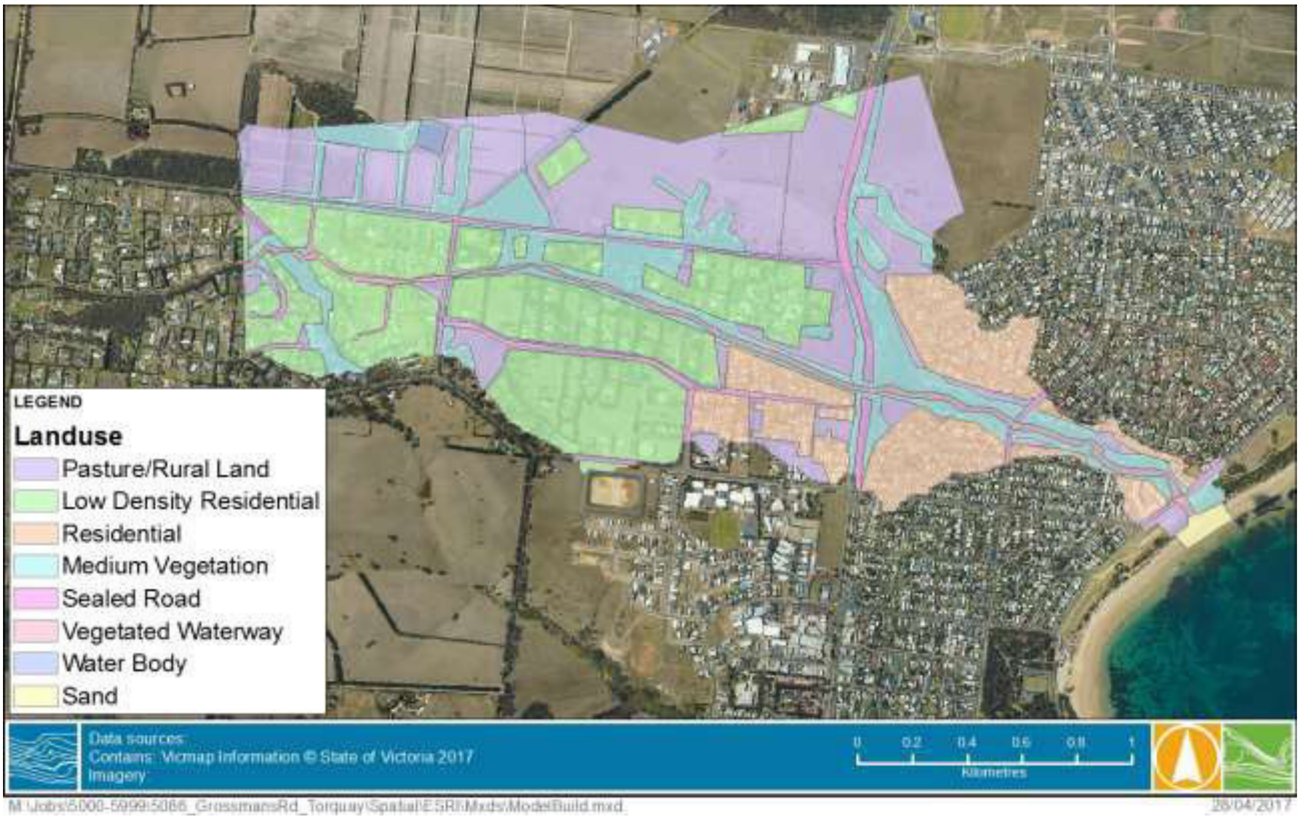

FIGURE 3-2 DEEP CREEK TUFLOW MODEL MANNING'S ROUGHNESS

3.1.2 Key Hydraulic Structures

There are several key hydraulic structures within the model area. Structural information was unavailable within the model extent. To ensure waterway crossings were represented reasonably within the model, culvert sizes were estimated and included. Large bridge structures had the bridge decks removed from the LiDAR. Any back up of water will not affect the site as these structures are far enough downstream. Sensitivity testing was undertaken to ensure the assumptions regarding these structures did not impact on flood extents through the site. Plans to verify the size of these structures were obtained from VicRoads. The estimated structures included:

- Surf Coast Highway – single 1200 mm culverts under the Surf Coast Highway and 900mm under the northern reach of the deep creek tributaries which enters the creek downstream of the Surf Coast Highway.
- Fischer Street cut out of LiDAR as on major flow path and structural information not available.

3.1.3 Boundary Conditions

3.1.3.1 Inflow Boundaries

Hydrographs from the RORB model were used as major inflow boundaries including Deep Creek, upstream of the development, and two secondary inflows identified to the east of the site based on the local drainage lines. Source Area (SA) boundaries were applied to accurately represent the inflows. Two additional inflows to represent the site discharges for both developed and mitigated flooding conditions were included in the model. Under both the developed and mitigated developed conditions inflows were included directly within the waterway corridor to mimic what would be a form drainage system and outlet structure into the creek. Under existing conditions, the inflow boundary for the eastern catchment of the site was input at Briody Drive. Figure 3-3 displays the boundaries applied to the Deep Creek model.

3.1.3.2 Downstream Boundary

The downstream end of the model, located at the outfall to Zeally Bay, utilised a Height/Time (HT) boundary to model the flow of water from the waterway to the ocean. The boundary location is shown in purple in Figure 3-3. A Storm Tide Height of 1.69 m AHD at Lorne, and LiDAR showing the downstream boundary around 1.4 m AHD was used to determine an initial water level of 1.5 m AHD. This was considered a conservative estimate. The development site is considered far enough upstream that the ocean boundary conditions would not cause any impact at the subject site in a large flood event.

FIGURE 3-3 DEEP CREEK MODEL –BOUNDARIES

3.2 Existing Conditions Model Results

Hydraulic modelling of Deep Creek has produced flood depth, height and velocity data for the 1% AEP, 10% and 63.2% AEP flood events. Flood depths during the 1% AEP flood event are shown in Figure 3-4. The flood extent of Deep Creek is largely confined to the channel, with small areas of shallow depths along the banks. Deep Creek passes through part of the subject site (north eastern extent) where inundation depths during 1% AEP flood event range between 0.1- 0.55 metres. Flow velocities within this portion of the property are also likely to reach 1.5 m/s.

Comparatively, inundation extents during minor flooding events, including the 10% AEP and 63.2% AEP events are not greatly different to the 1% AEP. This is likely due to the local sloping topography and defined bed and banks of Deep Creek along the reach of Deep Creek between Messmate Road and the Surf Coast Highway.

M:\20010016_DeepCreek_Briody Development_Torquay\Spatial\Spatial\ESRI\Mxds\ModelBuild.mxd 4/08/2019

FIGURE 3-4 DEEP CREEK 1% AEP FLOOD DEPTH

M:\20010016_DeepCreek_Briody Development_Torquay\Spatial\Spatial\ESRI\Mxds\ModelBuild.mxd 4/08/2019

FIGURE 3-5 DEEP CREEK 1% AEP FLOOD VELOCITY

FIGURE 3-6 10% AEP FLOOD DEPTH

FIGURE 3-7 63.2% AEP FLOOD DEPTH

4 DEVELOPED CONDITIONS

The subject site has been identified for future development which included mixed residential uses. An indicative layout of the proposed development is provided in Figure 4-1. For the purposes of modelling the developed flooding conditions within Deep Creek respective to impact of the development on flood depths and levels within the waterway the following assumptions were made:

- Fraction Impervious for the development site has been set at 0.75 based on an estimated lot size of 300-350m^2.
- Site catchment boundaries remain consistent with existing topographic features and slope draining to the north east and north west.

Modelling of developed conditions included an assessment of the available datasets including depth, water surface elevation (flood level) and velocity. Modelling of the critical durations for peak flows at three locations consistent with the existing conditions modelling was undertaken.

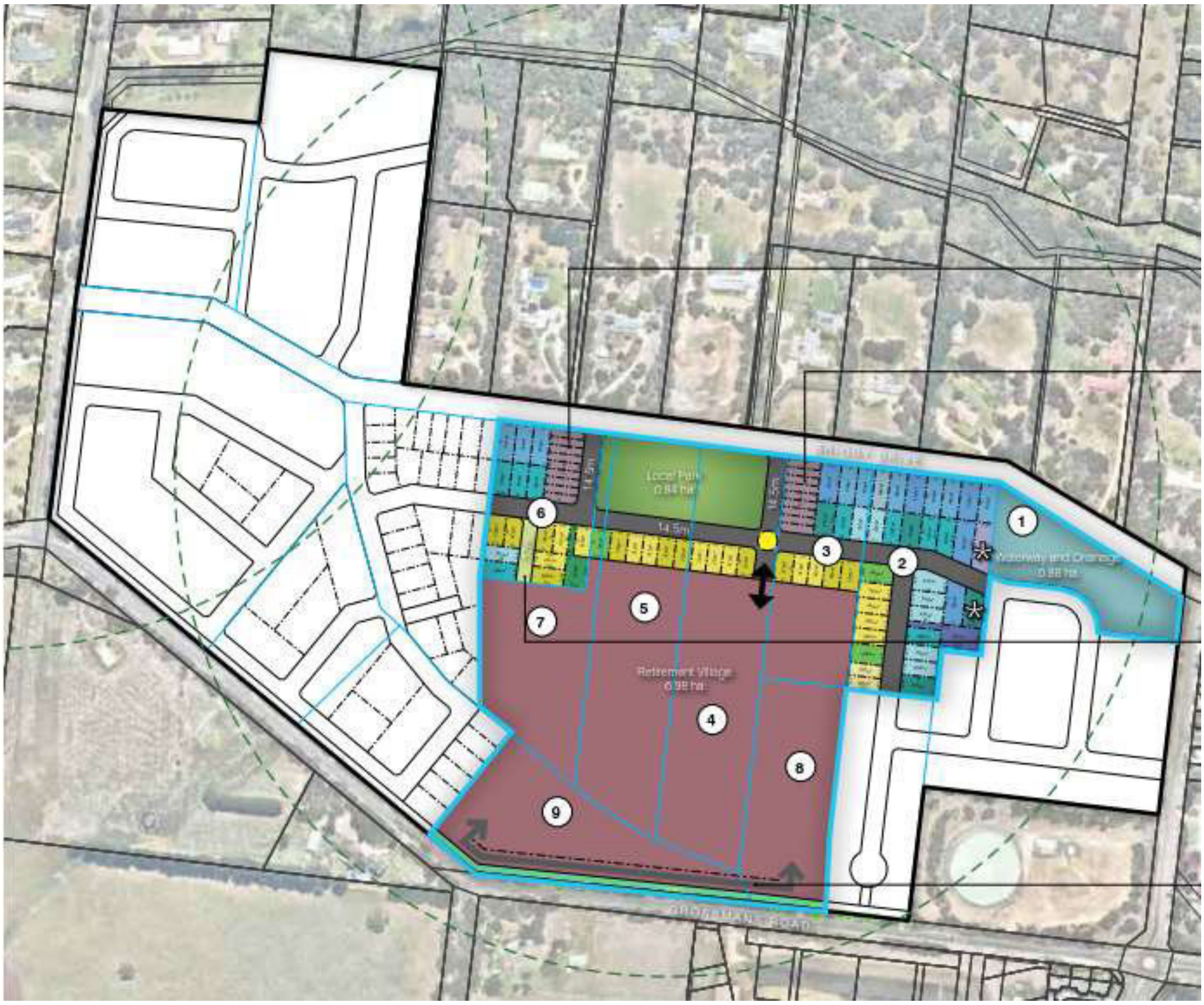

FIGURE 4-1 1% AEP FLOOD EXTENT SHOWING PROPOSED DEVELOPMENT LAYOUT

4.1 Developed Conditions Model Hydrology

A revised RORB model catchment was developed which updated the fraction impervious values within the development site, altering the breakdown of sub-catchment areas consistent with likely drainage layout and road alignment and changes reach types within the catchment from natural to excavated/unlined consistent with current practice.

The updated RORB layout for the developed conditions is shown Figure 4-2. Interstation areas consistent with the existing conditions model were included to provide consistent flow comparison and input with the existing conditions modelling. Minor changes to the catchment layout respective to existing topography and proposed layout were used as the basis for determining the developed catchment layout. The developed catchment is broken down into the western and eastern catchment (as per existing conditions modelling).

It was assumed that each of these catchment areas will have a direct connection discharging to Deep Creek. The developed conditions catchment delineation and estimated outlets locations are shown in Figure 4-3.

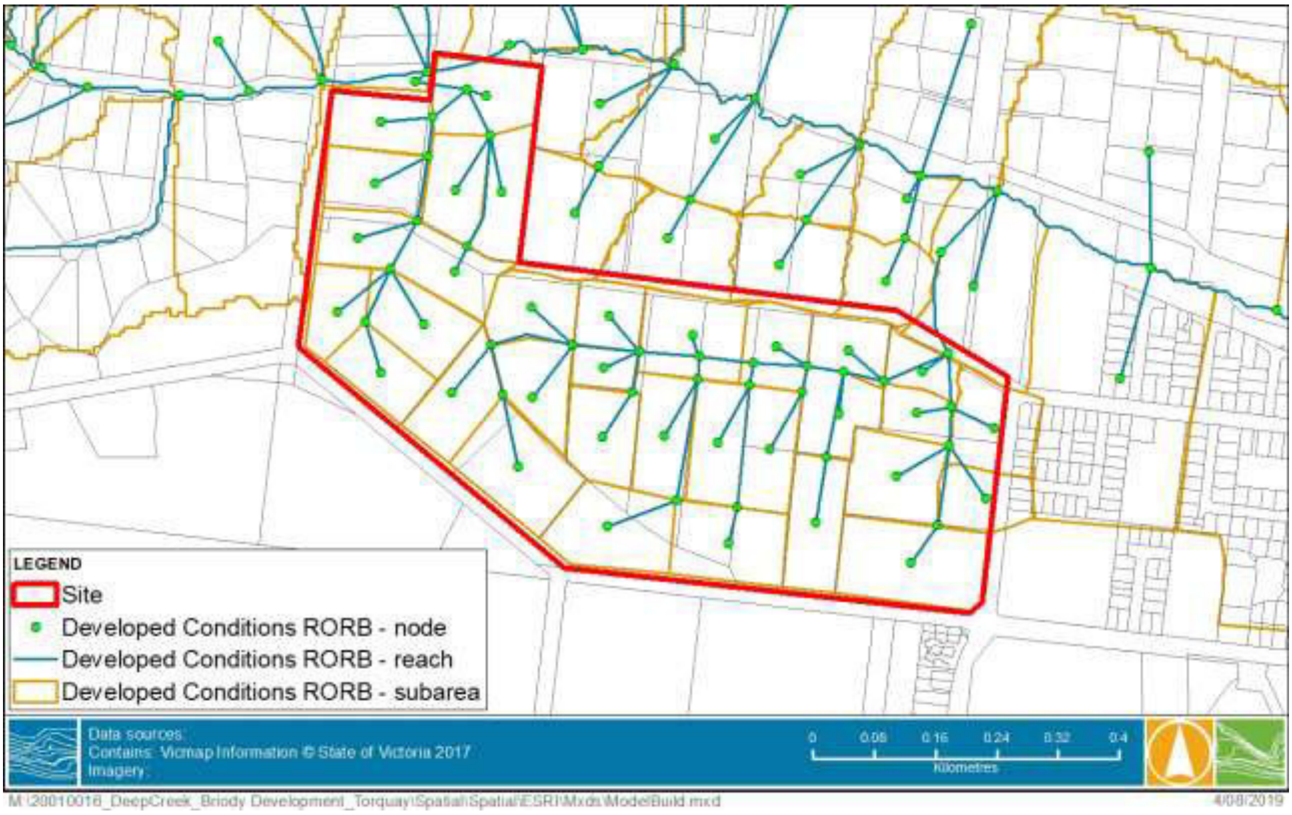

FIGURE 4-2 RORB- DEVELOPED CATCHMENT LAYOUT

For the purposes of this assessment, two developed scenarios were assessed. The first being where runoff from the development is assumed to be directly discharged into Deep Creek with two outlets (west and east). The second includes staged outlet retarding basins to mitigate the flows to pre development peak flow rates for the 1% AEP, 10% AEP and 63.2% AEP flood events.

The storage basins were added into the RORB model and assumptions were made based on identified land area and existing topography to determine area, stage and storage volume relationships. Details on each of the retarding basins is outlined in **TABLE 4-1** and

TABLE 4-2 for the western basin and **TABLE 4-3** and **TABLE 4-4** for the eastern basin.

In sizing the outfalls for the retarding basins an iterative approach was undertaken. The approach determined the storage volumes required to ensure pre-development peak flows were not exceeded along with outlet sizes and invert levels. The pipe sizes and slopes were then varied at the outfall to ensure no spillway flow in the retentions basin's during the critical design storms. Targets for the retarding basin outflow were determined from existing conditions.

M:\20010016_DeepCreek_Briody Development_Torquay\Spatial\Spatial\ESRI\Mxds\ModelBuild.mxd 4/08/2019

FIGURE 4-3 SITE DRAINAGE CATCHMENTS AND OUTLETS

The RORB model was run for all temporal patterns and storm duration ranging from 15min – 48 hours for each of the three AEPs as outlined for the existing conditions. Peak flows for the 1%, 10% and 63.2% AEPs were calculated. The peak flows and critical durations from the developed conditions were then selected based on the highest median peak at each of the critical inflow locations.

Table 4-5 shows the ensemble outputs for the 1% AEP event in developed conditions. The developed conditions results shown in this table do not include proposed retardation of stormwater from the site. The results indicate that development of the subject site shortens the critical duration for peak flows from the development area. The 1% AEP existing conditions peak flow from the site occurred during the 1.5hr storm duration, while the 1% AEP event under developed conditions was shortened within the western catchment to 20 minutes and within the eastern catchment to 30 minutes. Peak flows at the outlet from the site were increased during all modelled AEP events. During a 1% AEP events peak flow for the western catchment was increased from 0.7m^3/s to 1.94m^3/s. Peak flow from the eastern catchment was increased from 1.13m^3/s to 3.62m^3/s.

Table 4-7 shows the ensemble output for the 1% AEP flood event in mitigated conditions. The results demonstrate that with the inclusion of the proposed retarding basins, peak flows exiting the site are able to be retarded back to predevelopment levels for the 1%, 10% and 63.2% AEP flood events.

TABLE 4-1 WESTERN CATCHMENT RETARDATION BASIN

Western Retardation Basin	
Bottom Length	60m
Bottom Width	30m
Bottom Area	1800m^2
Side Slopes	1 in 5
Outflow Pipe Diameter (all pipes have assumed 1% slope, 2 m length)	3 x 0.14m dia pipe (invert stage 0.00m) 8 x 0.18m dia pipe (invert stage 0.35m) 6 x 0.225 dia pipe (invert stage 0.49m)
Spillway Height (max storage height)	At or above the maximum stage height 1.56m
Max Storage (maximum median adopted)	1680 m^3

TABLE 4-2 WESTERN BASIN STAGE STORAGE

Stage (m)	Storage (m^3)	Area (m^2)
0	0	1800
0.1	185	1891
0.2	378	1984
0.3	581	2079
0.4	794	2176
0.5	1017	2275
0.6	1249	2376
0.7	1492	2479
0.8	1745	2584
0.9	2009	2691
1	2283	2800

TABLE 4-3 EASTERN CATCHMENT RETARDING BASIN

Eastern Catchment Basin	
Bottom Length	110m
Bottom Width	20m
Bottom Area	220m^2
Side Slopes	1 in 5
Outflow Pipe Diameter	2 x 0.18m dia pipe (invert 0.00m) 4 x 0.25m dia pipe (invert 0.75m) 7 x 0.225m dia pipe (invert 1.14m)
Spillway Height (max storage height)	At or above the maximum stage height 1.56m
Max Storage (maximum median adopted)	5130 m^3

TABLE 4-4 EASTERN BASIN STAGE STORAGE

Stage (m)	Storage (m^3)	Area (m^2)
0	0	2200
0.1	227	2331
0.2	466	2464
0.3	719	2599
0.4	986	2736
0.5	1267	2875
0.6	1561	3016
0.7	1870	3159
0.8	2193	3304
0.9	2531	3451
1	2883	3600
1.1	3251	3751
1.2	3633	3904
1.3	4031	4059
1.4	4445	4216
1.5	4875	4375
1.6	5320	4536
1.7	5782	4699
1.8	6260	4864

TABLE 4-5 1% AEP RORB ENSEMBLE OUTPUT – DEVELOPED CONDITIONS

Duration	Upstream of Site (Deep Creek)	Site Western Catchment m^3/s	Site Eastern Catchment m^3/s
15min	1.42	1.89	3.25
20min	1.83	1.94	3.51
30min	3.88	1.80	3.62
1hr	7.35	1.37	2.82
1.5hr	8.47	1.18	2.55
2hr	7.72	1.16	2.68
3hr	7.20	0.79	1.80
4.5hr	6.65	0.75	1.66
6hr	6.41	0.69	1.55
9hr	6.24	0.46	1.06
12hr	5.55	0.47	1.07
24hr	4.30	0.36	0.87

TABLE 4-6 ADOPTED FLOWS AND TEMPORAL PATTERNS- DEVELOPED CONDITIONS

	Upstream of Site (Peak Flow, TP)	Site – West (Peak Flow, TP)	Site -Easy (Peak Flow, TP)
1% AEP (1.5 Hour)	8.71 m^3/s, TP28	1.26 m^3/s, TP28	2.34 m^3/s, TP28
1% AEP (20 Minute)	1.85 m3/s, TP25	2.02 m^3/s, TP25	3.5 m^3/s, TP25
1% AEP 30 Minute)	3.86 m3/s, TP28	1.88 m3/s, TP28	3.64 m^3/s, TP28
10% AEP (15 Minute)	0.71 m^3/s, TP18	1.11 m^3/s, TP18	1.81 m^3/s, TP18
10% AEP (25 Minute)	0.93 m^3/s, TP17	0.93 m^3/s, TP17	1.96 m^3/s, TP17
10% AEP (3 Hour)	3.58 m^3/s, TP15	0.41 m^3/s, TP15	0.97 m^3/s, TP15
63.2% AEP (9 Hour)	1.00 m^3/s, TP4	0.17 m^3/s, TP4	0.38 m^3/s, TP4
63.2% AEP (20 Minute)	0.31 m^3/s, TP7	0.53 m^3/s, TP7	0.85 m^3/s, TP7

TABLE 4-7 1% AEP RORB ENSEMBLE OUTPUT – MITIGATED CONDITIONS

Duration	Upstream of Site (Deep Creek)	Site Western Catchment m^3/s	Site Eastern Catchment m^3/s
15min	1.42	0.29	0.27
20min	1.83	0.38	0.40
30min	3.89	0.50	0.64
1hr	7.36	0.67	1.01
1.5hr	8.48	0.69	1.13
2hr	7.72	0.62	1.08
3hr	7.20	0.51	1.07
4.5hr	6.65	0.48	0.92
6hr	6.42	0.45	0.93
9hr	6.24	0.40	0.91
12hr	**6.24**	0.37	0.79
24hr	5.55	0.29	0.27

TABLE 4-8 ADOPTED FLOWS AND TEMPORAL PATTERNS- DEVELOPED MITIGATED CONDITIONS

	Upstream of Site (Peak Flow, TP)	Site – West (Peak Flow, TP)	Site -Easy (Peak Flow, TP)
1% AEP (1.5 Hour)	8.71 m^3/s, TP28	0.7m^3/s, TP28	1.14 m^3/s, TP28
10% AEP (3 Hour)	3.58 m^3/s, TP15	0.28m3/s, TP15	0.5 m3/s, TP15
63.2% AEP (9 Hour)	1.00 m^3/s, TP4	0.075 m^3/s, TP4	0.13 m^3/s, TP4

4.2 Hydraulic Modelling Results

4.2.1 Developed Conditions

Modelled scenarios of Deep Creek under developed conditions with and without the staged flood retarding basins were modelled for the 1%, 10% and 63.2%AEP flood events. Each of the modelled scenarios assumed that the development provides infrastructure directly connecting to the stormwater network with outlet structures into Deep Creek.

Figure 4-4, Figure 4-7 and Figure 4-6 show the resulting flood depths from the combined maximum envelope of the 1% AEP, 10% AEP and 63.2% AEP flood events for developed (unmitigated) conditions respectively.

Figure 4-7, Figure 4-8 and Figure 4-9 show the resulting flooding depths from the combined maximum envelope 1% AEP, 10% AEP and 63.2% AEP flood event for mitigated (developed with retarding basins) conditions respectively.

FIGURE 4-4 1% AEP FLOOD DEPTH – DEVELOPED CONDITIONS

FIGURE 4-5 10% AEP FLOOD DEPTH – DEVELOPED CONDITIONS

FIGURE 4-6 63.2% AEP FLOOD DEPTH – DEVELOPED CONDITIONS

FIGURE 4-7 1% AEP FLOOD DEPTH – MITIGATED CONDITIONS

FIGURE 4-8 10% AEP FLOOD DEPTH – MITIGATED CONDITIONS

FIGURE 4-9 63.2% AEP FLOOD DEPTH – MITIGATED CONDITIONS

4.2.2 Discussion – Result Comparison

A comparison of the flood depth results from the modelled 1% AEP flood events indicates a minor increase in flood depths within Deep Creek for both the developed and mitigated scenarios. Figure 4-10 shows the difference in flood depths between the existing and developed 1%AEP flood events. Noting the most significant increase in depths are immediately upstream of the Surf Coast Highway culvert, where depths have increased by up to 7 cm.

Minor decreases are shown along the reach of Deep Creek between the western and eastern site outfalls. Decreases along these reaches are likely attributed to the change in timing for the localised development catchment in comparison with the greater upstream catchment. Development of the site significantly increases the impervious area and rate of runoff from the site, and as such flows do peak quickly. In the developed unretarded scenario this peak flow can discharge to the creek and pass downstream before the higher Deep Creek flood flows reach the eastern parts of the site.

Under mitigated conditions, where the proposed two basins would retard peak flows back to predevelopment conditions discharging into Deep Creek, minor increases were also observed. Figure 4-11 shows increases of up to 5cm immediately upstream of the Surf Coast Highway. With minor increases extending further up to where the outlets of the two sites discharge into the creek.

A comparison of the 1% AEP 1.5Hr flood levels between Messmate Road and the Surf Coast Highway is provided in Table 4-9. The comparison shows minor variation in levels along the creek with the greatest variation around the Surf Coast Highway. This indicates that flows are accumulating on the upstream of the Surf Coast highway with the only passing structure a 1200mm pipe culvert.

The channel within this portion of the creek is well defined with a bed level estimated to be at least 5 metres below bank level. This means the modelled increase in both flow and volume from the developed and mitigated 1%, 10% and 63.2% AEP events only provide for a minor increase in depth with no identified changes to modelled extent between the existing and developed scenarios.

This is also evident in the extracted hydrographs from the RORB model downstream of the subject site on Deep Creek. The Hydrographs show minor variation in peak flow and volume of the hydrograph for the 1% AEP 1.5 Hr critical storm duration, refer to Figure 4-13. The flow hydrograph from the culvert under the Surfcoast Highway also indicates sustained high flows of around 5m^3/s for some time, resulting in attenuation of flooding on the upstream side of the highway (Figure 4-14).

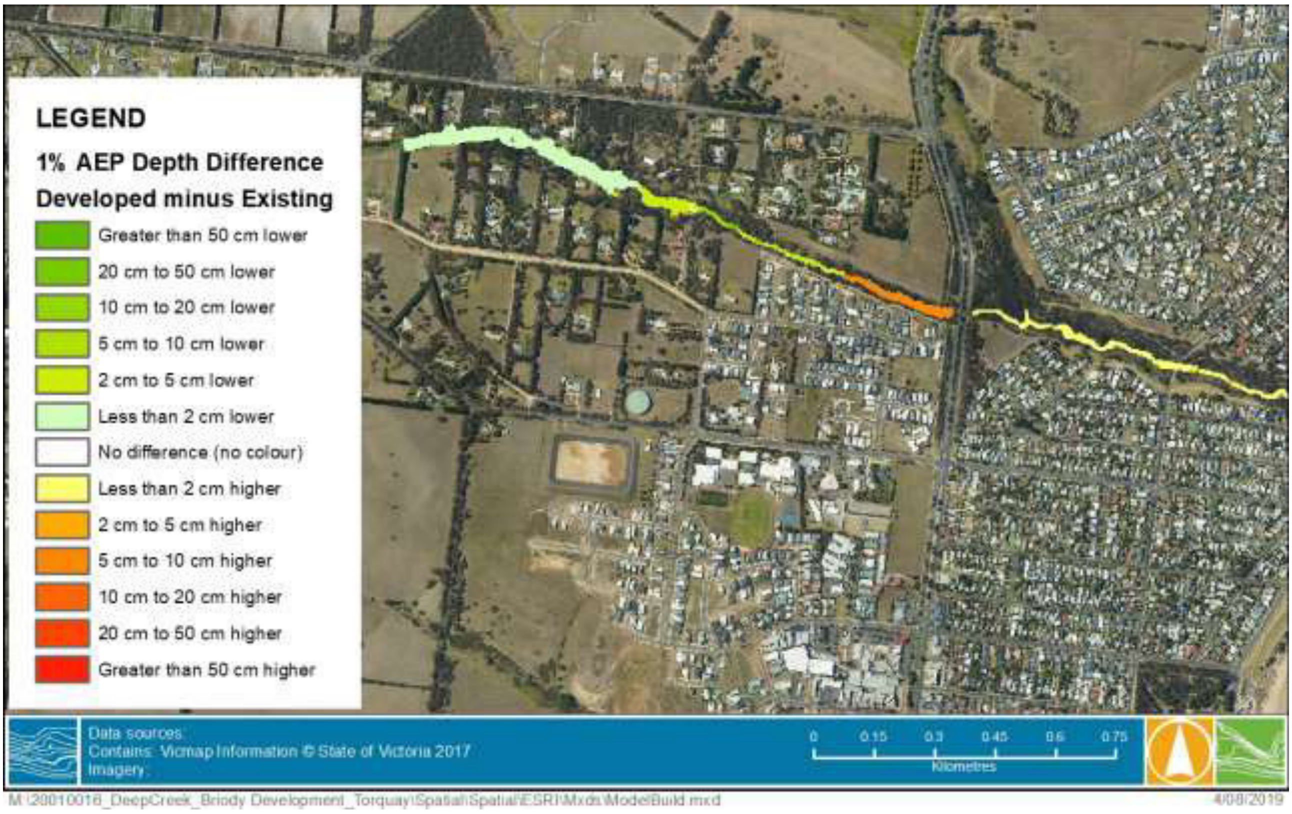

FIGURE 4-10 1% AEP FLOOD DEPTH DIFFERENCE DEVELOPED MINUS EXISTING

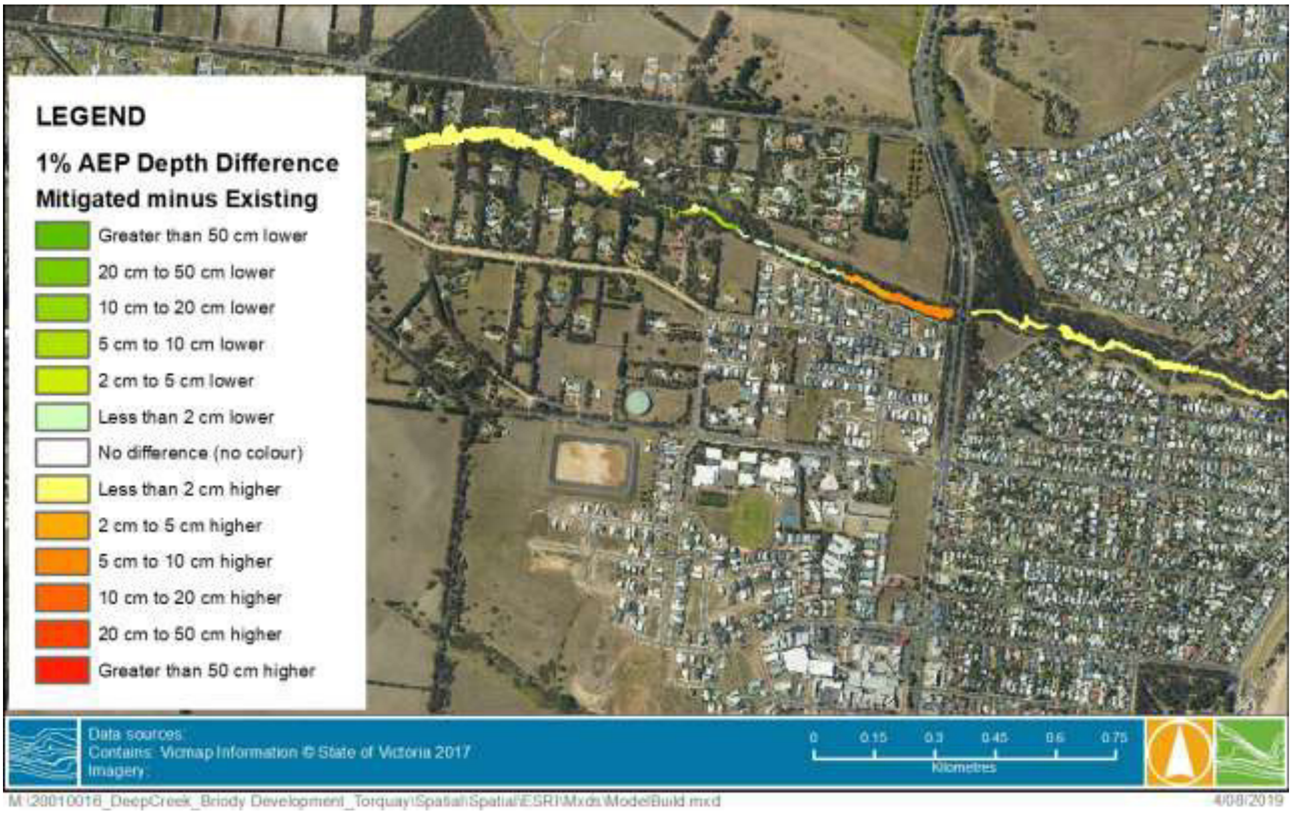

FIGURE 4-11 1% AEP FLOOD DEPTH DIFFERENCE MITIGATED MINUS DEVELOPED

TABLE 4-9 1% AEP 1.5 HR DURATION - FLOOD LEVEL COMPARISON (M AHD)

Location	1 (m AHD)	2 (m AHD)	3 (m AHD)	4 (m AHD)	5 (mA AHD)	6 (m AHD)
Existing	47.10	42.66	37.82	31.33	27.50	27.47
Developed	47.10	42.66	37.82	31.30	27.56	27.54
Mitigated	47.10	42.67	37.83	31.33	27.54	27.52

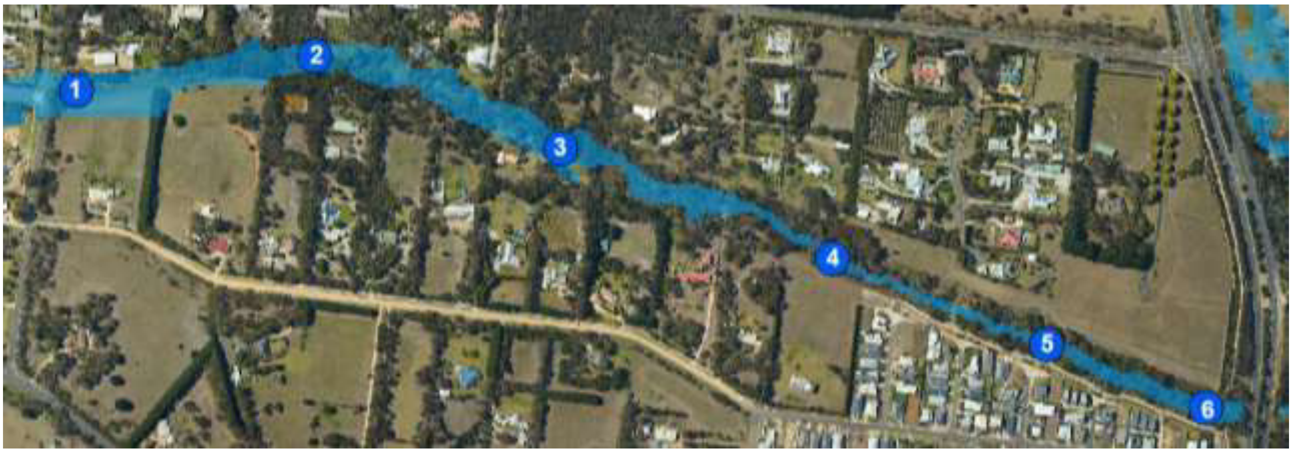

FIGURE 4-12 FLOOD LEVEL POINT LOCATIONS

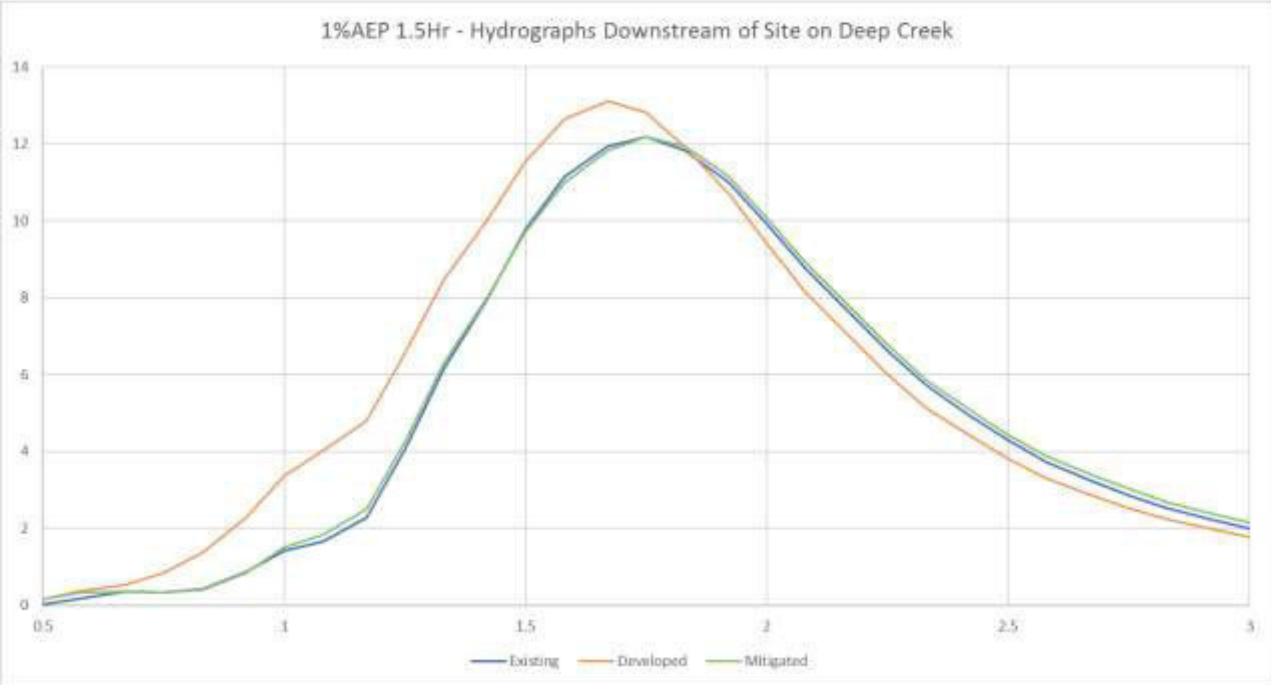

FIGURE 4-13 1%AEP HYDROGRAPH DOWNSTREAM OF SITE ON DEEP CREEK

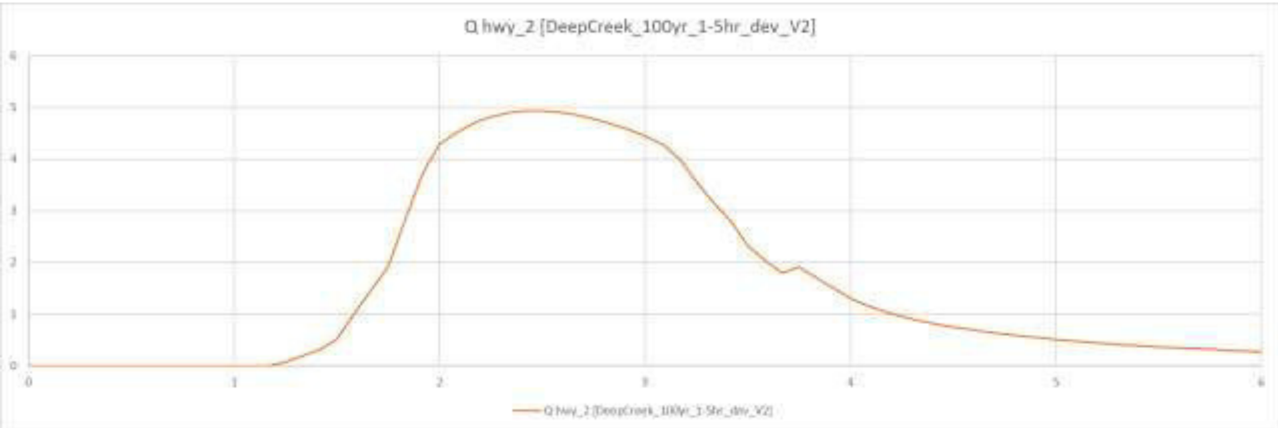

FIGURE 4-14 SURFCOAST HIGHWAY FLOW HYDROGRAPH – 1% AEP 1.5HR

APPENDIX A
AR&R DATA HUB OUTPUT

APPENDIX B RORB MODEL INPUTS

ults - ARR Data Hub

[LONGARF]

Zone,Southern Temperate

a,1.58E-01

b,2.76E-01

c,3.72E-01

d,3.15E-01

e,1.41E-04

f,4.10E-01

g,1.50E-01

h,1.00E-02

i,-2.70E-03

[LONGARF_META]

Time Accessed,19 April 2017 11:31AM

Version,2016_v1

[END_LONGARF]

Storm Losses

[LOSSES]

Initial Losses (mm),24.0

Continuing Losses (mm/h),4.4

[LOSSES_META]

Time Accessed,19 April 2017 11:31AM

Version,2016_v1

[END_LOSSES]

Temporal Patterns

[TP]

CODE,SSmainland

LABEL,Southern Slopes (Vic/NSW)

[TP_META]

Time Accessed,19 April 2017 11:31AM

Version,2016_v1

[END_TP]

#10% Preburst Depths

[PREBURST10]

min (h)\AEP(%),50,20,10,5,2,1,

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90 (1.5),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
120 (2.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
180 (3.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
360 (6.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
720 (12.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
1080 (18.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
1440 (24.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
2160 (36.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
2880 (48.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
4320 (72.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
[PREBURST10_META]
Time Accessed,19 April 2017 11:31AM
Version,2016_v1
[END_PREBURST10]

#25% Preburst Depths
[PREBURST25]
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90 (1.5),0.0 (0.001),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
120 (2.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
180 (3.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
360 (6.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
720 (12.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
1080 (18.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
1440 (24.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
2160 (36.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
2880 (48.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
4320 (72.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),
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Time Accessed,19 April 2017 11:31AM
Version,2016_v1
[END_PREBURST25]

#75% Preburst Depths
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90 (1.5),6.9 (0.481),10.2 (0.504),12.4 (0.506),14.5 (0.502),13.9 (0.4),13.5 (0.342),

120 (2.0),9.1 (0.566),10.5 (0.468),11.4 (0.423),12.3 (0.389),12.5 (0.328),12.6 (0.292),

180 (3.0),10.5 (0.553),12.8 (0.492),14.4 (0.462),15.9 (0.437),12.9 (0.296),10.6 (0.216),

360 (6.0),4.8 (0.189),8.3 (0.24),10.5 (0.258),12.7 (0.27),16.4 (0.291),19.1 (0.3),

720 (12.0),1.7 (0.048),4.5 (0.097),6.4 (0.117),8.2 (0.13),12.9 (0.171),16.5 (0.192),

1080 (18.0),0.3 (0.008),3.9 (0.072),6.4 (0.098),8.7 (0.116),13.1 (0.145),16.4 (0.16),

1440 (24.0),0.2 (0.005),3.1 (0.051),5.0 (0.069),6.8 (0.081),7.8 (0.077),8.6 (0.074),

2160 (36.0),0.0 (0.0),0.7 (0.01),1.2 (0.014),1.6 (0.016),2.8 (0.023),3.6 (0.027),

2880 (48.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.5 (0.004),0.9 (0.006),

4320 (72.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.0 (0.0),0.1 (0.001),0.2 (0.001),

[PREBURST75_META]

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[END_PREBURST75]

#90% Preburst Depths

[PREBURST90]

min (h)\AEP(%),50,20,10,5,2,1,

60 (1.0),19.0 (1.552),20.9 (1.191),22.2 (1.035),23.4 (0.92),26.4 (0.855),28.6 (0.812),

90 (1.5),23.2 (1.622),24.9 (1.233),26.1 (1.065),27.2 (0.942),24.4 (0.701),22.3 (0.564),

120 (2.0),22.9 (1.431),25.1 (1.121),26.6 (0.985),28.0 (0.884),27.2 (0.714),26.5 (0.615),

180 (3.0),19.9 (1.053),26.7 (1.025),31.2 (1.001),35.5 (0.977),34.3 (0.79),33.4 (0.681),

360 (6.0),18.1 (0.705),28.3 (0.819),35.0 (0.859),41.5 (0.88),41.0 (0.728),40.6 (0.639),

720 (12.0),12.6 (0.363),16.8 (0.363),19.6 (0.359),22.3 (0.354),30.2 (0.4),36.2 (0.422),

1080 (18.0),11.1 (0.272),13.7 (0.25),15.4 (0.238),17.1 (0.228),22.7 (0.251),26.8 (0.262),

1440 (24.0),7.9 (0.175),12.2 (0.199),15.0 (0.206),17.7 (0.21),19.7 (0.193),21.1 (0.182),

2160 (36.0),7.9 (0.154),9.7 (0.138),11.0 (0.13),12.2 (0.123),19.2 (0.161),24.5 (0.18),

2880 (48.0),5.9 (0.106),5.6 (0.073),5.4 (0.058),5.2 (0.048),7.0 (0.053),8.3 (0.055),

4320 (72.0),0.2 (0.003),0.7 (0.008),1.0 (0.009),1.3 (0.01),15.1 (0.103),25.4 (0.153),

[PREBURST90_META]

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[END_PREBURST90]

Interim Climate Change Factors

[CCF]

2030,0.719 (3.6%),0.739 (3.7%),0.822 (4.1%),

2040,0.925 (4.6%),0.915 (4.6%),1.119 (5.6%), 2050,1.123 (5.6%),1.085 (5.4%),1.449 (7.2%),

2060,1.271 (6.4%),1.294 (6.5%),1.865 (9.3%),

2070,1.394 (7.0%),1.526 (7.6%),2.333 (11.7%),

2080,1.477 (7.4%),1.778 (8.9%),2.776 (13.9%),

2090,1.527 (7.6%),2.009 (10.0%),3.21 (16.1%),

[CCF_META]

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Note,ARR recommends the use of RCP4.5 and RCP 8.5 values

[END_CCF]

Baseflow Factors

[BASEFLOW]

DOWNSTREAM,0.0

AREA_SQKM,908.982

CATCH_NO,11245.0

R3RUNOFF,0.212

R1RUNOFF,0.041

[BASEFLOW_META]

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[END_BASEFLOW]

[ENDTXT]

APPENDIX C
AR&R – REGIONAL FLOOD FREQUENCY ESTIMATION TOOL

AR&R (2016) has developed a new Regional Flood Frequency Estimate (RFFE) (Rahman, et al, 2015[9]). This method was used to compare Deep Creek flows to other regional methods. The online tool uses the catchment centroid, catchment outlet and size to estimate peak flow outputs for a range of flood magnitudes. The tool was developed utilising data based on gauged catchments to form region based flood relationships.

The RFFE tool has several limitations to its application and should be avoided where:

- The catchment includes greater than 10% urban,
- Catchment storage significantly altered the natural rainfall runoff behaviour,
- Catchment where large scale clearing has taken place,
- Catchments which are greatly affected by irrigation activity and or drainage.

The reliability of the tool is also considered less accurate for catchment less than 0.5 km^2 and or greater than 1,000 km^2 or where a catchment exhibit atypical characteristics.

RESULTS FROM ARR RFFE 2015 MODEL
Datetime: 2017-04-28 11:08
Region name: East Coast
Region code: 1
Site name: Deep Creek
Latitude at catchment outlet (degree) = -38.316
Longitude at catchment outlet (degree) = 144.3
Latitude at catchment centroid (degree) = -38.315
Longitude at catchment centroid (degree) = 144.287
Distance of the nearest gauged catchment in the database (km) = 24.42
Catchment area (sq km) = 1.8
Design rainfall intensity, 1 in 2 AEP and 6 hr duration (mm/h): 4.295056
Design rainfall intensity, 1 in 50 AEP and 6 hr duration (mm/h): 9.401045
Shape factor of the ungauged catchment: 0.85

ESTIMATED FLOOD QUANTILES:

AEP (%)	Expected quantiles (m^3/s)	5% CL m^3/s	95% CL m^3/s
50	0.540	0.200	1.45
20	1.01	0.400	2.59
10	1.42	0.550	3.70
5	1.89	0.710	5.08
2	2.62	0.940	7.38
1	3.26	1.12	9.58

DATA FOR FITTING MULTI-NORMAL DISTRIBUTION FOR BUILDING CONFIDENCE LIMITS:
1 Mean (loge flow) = -1.064
2 St dev (loge flow) = 0.722
3 Skew (loge flow) = 0.136

Moments and correlations: No	Most probable	Std dev	Correlation		
1	-1.064	0.520	1.000		
2	0.722	0.235	-0.330	1.000	
3	0.136	0.030	0.170	-0.280	1.000

FIGURE 4-15 DEEP CREEK - RFFE

[9] AR&R (2016) - http://data.arr-software.org

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