Colliers

Briody Drive West Development Plan

[Summerset]

Stormwater Management Strategy

Revision A

Report Date: October 2023

PLANNING & ENVIRONMENT ACT 1987
SURF COAST PLANNING SCHEME
This Development Plan complies with the requirements of Clause
43.04 of the Surf Coast Planning Scheme

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Date: 18/04/2024 Sheet No: 1 of 76

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Document Control

Revision A

Version	Date	Details	Prepared	Checked	Approved	Signed
A	Oct 2023	Issued for Approval	C. Cosgriff	J. Barker	J. Barker	J. Barker

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EXECUTIVE SUMMARY

Colliers Engineering & Design was engaged to formulate a Stormwater Management Strategy for a parcel of land (referred to within this document as "the Site") located at Briody Drive, Torquay, *south west of Melbourne in the municipality of Surf Coast Shire Council.*

The previous strategy for the region prepared by Spiire (Rev. I dated 15 June 2022) included a Flood Impact Assessment 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, downstream from the Briody Drive West Development.

The assessment highlights that as the section of Deep Creek nearby the development has a large 'slow' rural catchment, adding a small 'fast' developed catchment has very little impact on the existing waterway inundation. By the time the peak flows discharged from the 'slow' upstream catchment, passes through the section of the Creek north of the Site, the peak flows of the development will have flushed through by that point. Therefore, there is no flood attenuation proposed with this development.

The proposed development is designed so that the piped network conveys flows up to the 1% AEP scenario. To accommodate this, the piped network throughout the catchment will be sized gradually for capacity for the typical 20% AEP scenario in the south of the catchment (upstream), up to the 1% AEP scenario in the northern (downstream) sections.

Flows greater than the 20% AEP and up to and including the 1% AEP design event are to be conveyed through the Site utilising the road network as urbanised floodways. The road reserves will be designed in order to convey these flows whilst ensuring DEWLP's floodway safety criteria (Dmax <= 0.3m, Vmax <= 2.0m/s and VDmax <=0.3m²/s). The drainage network is designed so that the overland flow at Briody Dr and roads in close proximity will be minimised.

Stormwater treatment for the site will achieve best practice guidelines. The western catchment will utilise the following assets to reduce the pollutant load.

- *A 324 m² Sediment Pond*
- *A 1400 m² Wetland*

The eastern catchment will utilise more distributed methods of treatment. The retirement village stormwater runoff will be treated by underground proprietary products, combined with water tanks and irrigation storage. The Atlan (SPEL) Stormwater products have only been proposed to treat runoff from within the retirement village, and are to be managed privately. The surrounding residential development discharges into more traditional water treatment assets in the form of dual sediment forebays and Bio-retention system.

It is proposed that a Section 173 agreement be applied to all residential lots to ensure provision of rainwater tanks to promote capture and re-use. Notwithstanding this, a 50% uptake was modelled for the purposes of water quality, as agreed with Surf Coast Shire Council. The eastern catchment is serviced by the following assets.

- *A 230m² Raingarden (Bio-retention system)*
- *A 40m² and 80m² sediment forebay*
- *SPEL Stormceptor (OL.45180)*
- *8 x SPEL Filter Vaults (SF.30-EMC)*
- *2 x SPEL Vortceptor GPT's*
- *Distributed rainwater tanks on lots (2kL)*
- *4x10kL stormwater harvesting tanks.*

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1. Introduction

Colliers Engineering & Design was engaged to formulate a Stormwater Management Strategy for a parcel of land (referred to within this document as "the Site") located at Briody Drive, Torquay, south west of Melbourne in the municipality of Surf Coast Shire Council. The Site relates directly to the Briody Drive West Development Plan Area (BDW DP).

The plan will outline any potential stormwater quantity and quality measures required within the Site to comply with Corangamite CMA and Council requirements. We have developed a stormwater strategy that reflects the demands and needs of the Site and its surrounding parcels that contribute to the same catchment.

1.1. Scope of the Plan

The Site is located in Torquay, immediately north of Grossmans Road and south of Deep Creek, it has an area of 32.9 hectares (ha). Refer to Figure 1.

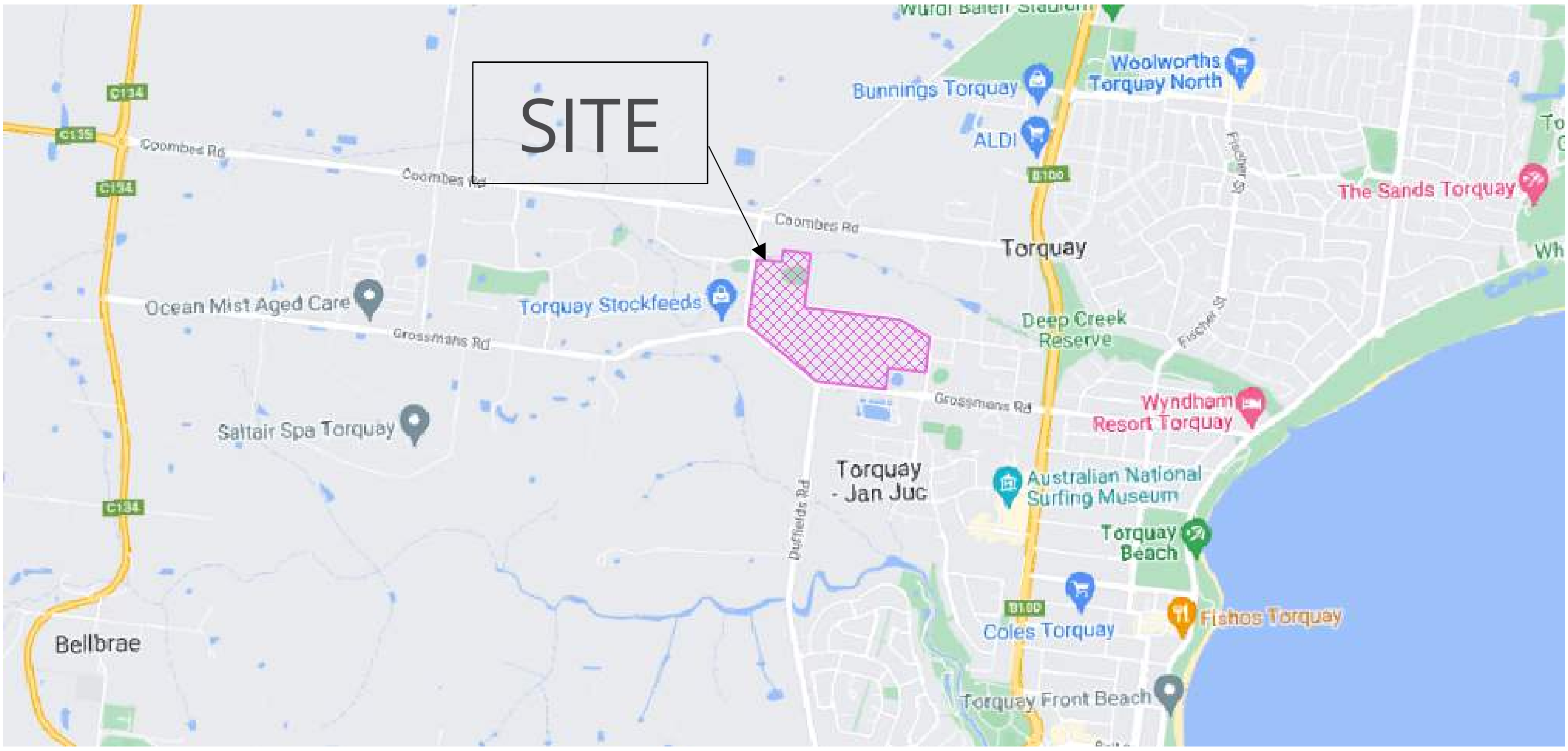

Figure 1. Site Locality

In existing conditions, the Site is substantially bare and is considered highly permeable, with the exception of a number of scattered residential dwellings and small sheds. The majority of Site falls towards the north-east, see Figure 2. Natural topography data indicates approximately 12.1 m of fall across the Site from the south-west, from 57.4 m AHD at the southern boundary to 45.3 m AHD along the north-east facing boundary, see Figure 2.

Figure 2. The Site

The Site does not reside within a Precinct Structure Plan (PSP) or a Melbourne Water Drainage Scheme.

2. Assumptions and Constraints

The Site is to comply with Surf Coast Shire Council and Corangamite Catchment Management Authority requirements. Based on the current draft plan the Site will yield approx. 341 residential allotments along with open space, drainage reserves and a retirement village, see Figure 3.

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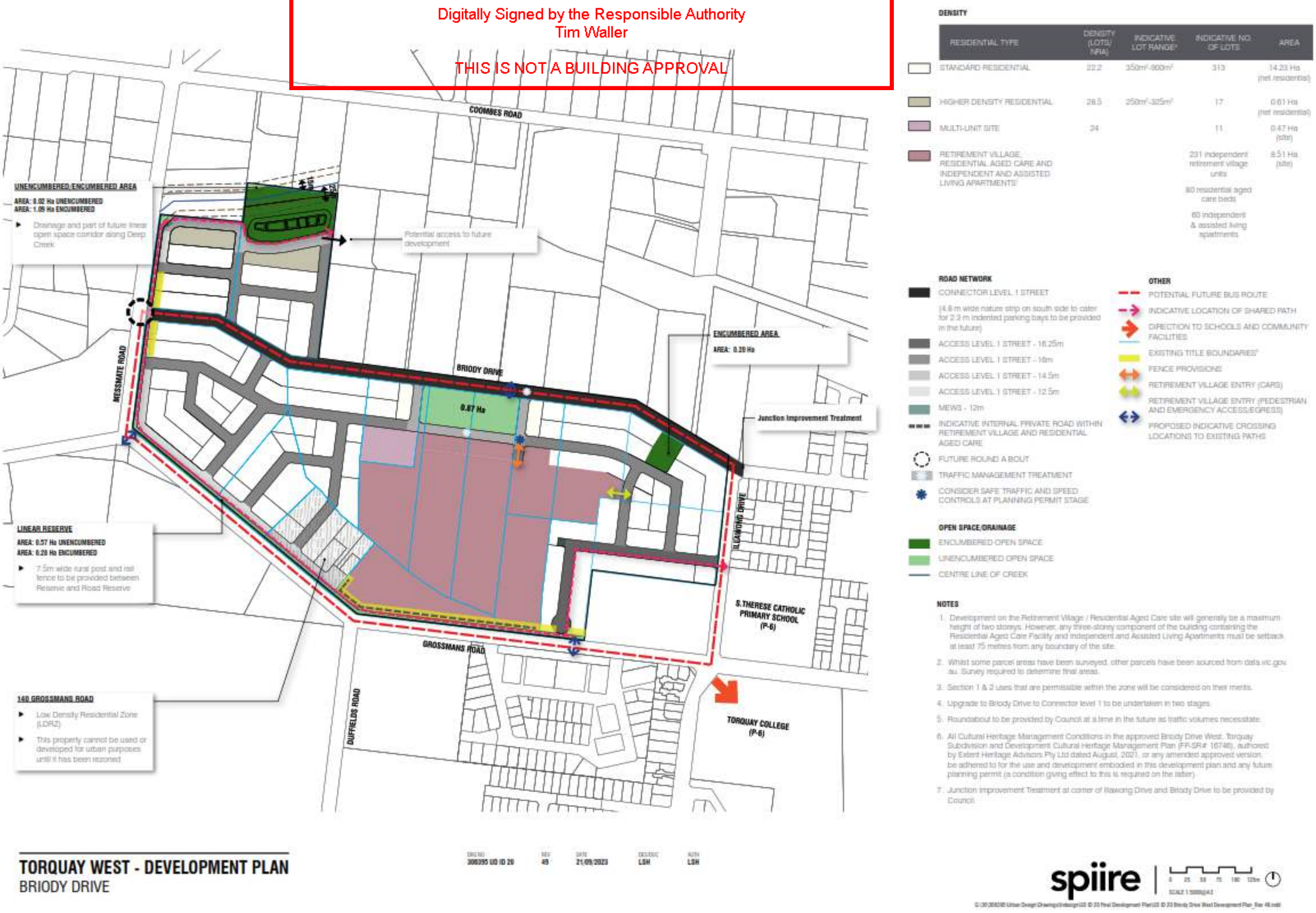

Figure 3. Proposed Development Plan

The SWMS will inform and accompany the above Urban Design Layout to allow the subdivision of the Site, which is subject to the requirements of the Development Plan Overlay (DPO10) of the Surf Coast Shire Planning Scheme.

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 the following.

- 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 170 Briody Dr) that ensures the peak discharge rate and pollutant load of stormwater leaving the Site within the affected area is no greater than the predevelopment levels, meets best practice and is discharged to the existing drainage system.
    - An integrated stormwater management system for the remaining land that ensures the pollutant load of the stormwater leaving the land is no greater than predevelopment 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 100yr (1% AEP) storm to the existing drainage system.

- Any interim stormwater management arrangements that could provide for out of sequence residential development.
- Input from Corangamite Catchment Management Authority (CCMA) 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 costs will be equalised across all landowners. This may be implemented via a condition on a planning permit that approves the residential subdivision, for a Section 173 Agreement that requires a cash contribution to equalise the costs associated providing land for and the construction of the system or any other mechanism to the satisfaction of the responsible authority.

DPO10 also states that a permit for subdivision of the land may require a Section 173 Agreement under the Planning and Environmental 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 of 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 predevelopment levels unless increased flows are approved by the relevant drainage authority and there are no detrimental downstream impacts.
- Designed to contribute to cooling, improving the local habitat and providing enjoyable spaces.

The Site is made up predominantly of GRZ1 zones land with one property (140 Grossmans Road) zones as LDRZ.

2.1. Previous Assessments

2.1.1. Approved Stormwater Management Strategy (Peter Berry & Associates Pty Ltd, December 2017)

A Stormwater Management Strategy was prepared by Peter Berry & Associates Pty Ltd (Version 5 dated December 5, 2017) to meet the requirements of the DPO10. Key points from the approved SWMS included the following.

- The Site comprised of three existing catchments.
- The runoff from the west catchment is conveyed into a new treatment wetland and retarding basin located near the northern interface with Deep Creek. Major and minor flows are discharged into Deep Creek at predevelopment rates, with an end wall dependant on the CCMA application for connection.
- The drainage from the eastern catchment has been proposed to connect to Deep Creek directly without detention.
- Within the subdivision the 10yr runoff volumes are to be conveyed using a pit and pipe network, with overland flow routes to provide conveyance of gap flows for events up to the 100yr ARI.
- 3-month flows from the eastern catchment are directed to a wetland treatment located in the north east corner of the BDW DP before being discharged to Deep Creek (large flows bypassed)

- The southern catchment described in the report is to be directed to the eastern catchment and discharged as per above.
- Cost estimates we prepared for the proposed drainage works to inform the Development Contributions Plan (DCP), which would be managed vcia the use of S173 agreement as part of the planning permit condition.
- The catchment plan presented in the SWMS can be seen below in Figure 4.

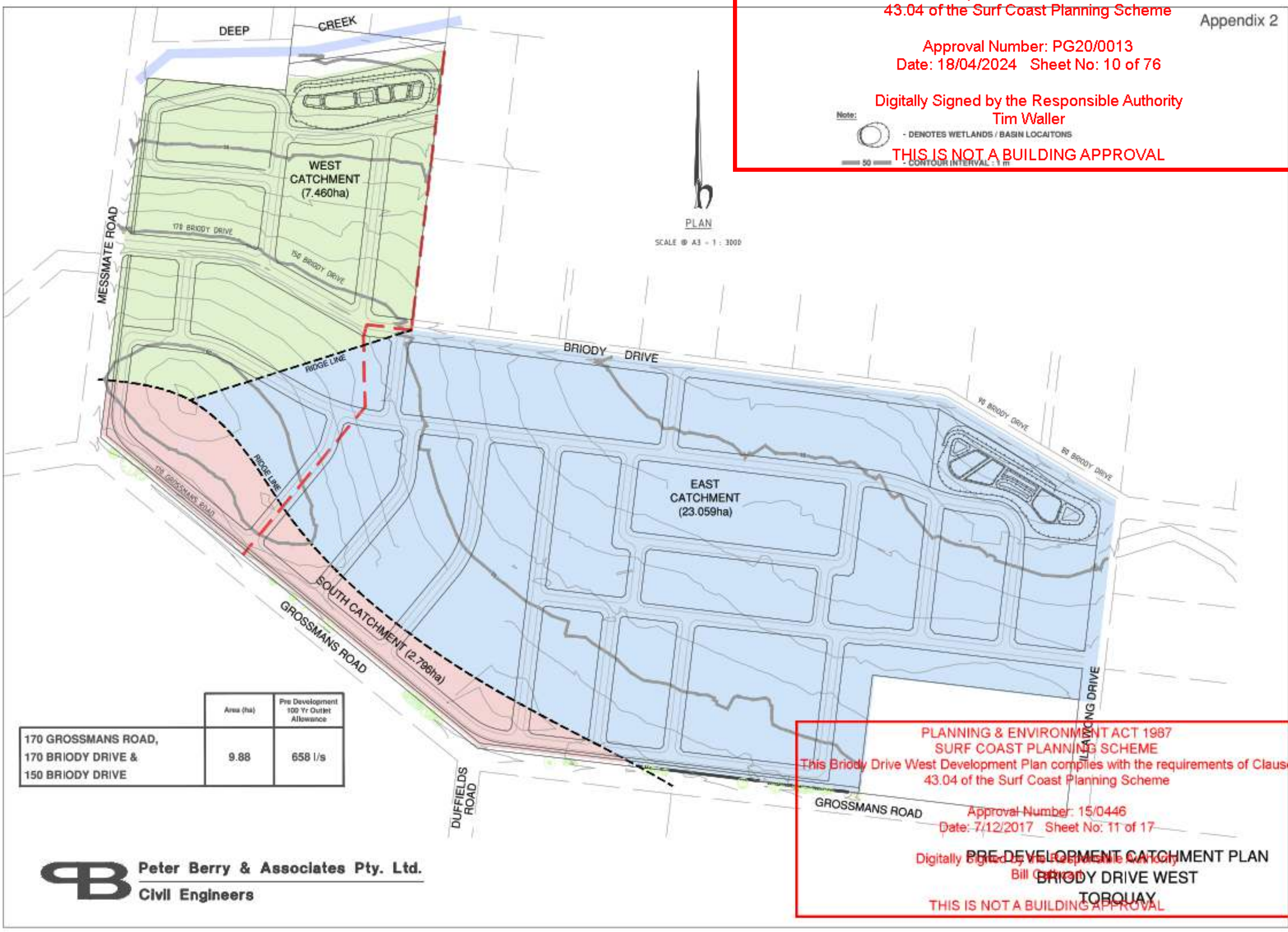

Figure 4 Peter Berry & Associates SWMS Catchments

2.1.2. Stormwater Management Strategy (Spiire, June 2022)

A Stormwater Management Strategy was prepared by Spiire (Rev. I dated 15 June 2022), which was previously submitted to Council. Key components of the existing SWMS included the following:

- The site comprises of two catchments, east and west (refer Figure 5).
- The proposed minor drainage network is to be designed to convey the 20% AEP and up to the 1% AEP flows utilising pit and pipe network. The piped network will be gradually upsized throughout the catchment to avoid large inlet pits at the outlet.
- The major drainage system (overland flow) carries the gap flow, which is defined as the difference between the 20% AEP and 1% AEP storm events. The major drainage has been designed so that the

gap flow is minimised at Briody Dr by accepting a larger portion of the gap within underground drainage.

- Treatment of all stormwater runoff within the development plan area targeted best practice utilising the installation of:
    - SPEL Stormceptor and Hydrosystem units (Eastern Catchment)
    - Constructed wetland and sedimentation basin (Western Catchment)
- Collected stormwater from the site (including Briody Dr and additional small upstream catchment areas outside the Development Plan Area) will be discharged via newly constructed 1% AEP pipelines to Deep Creek.

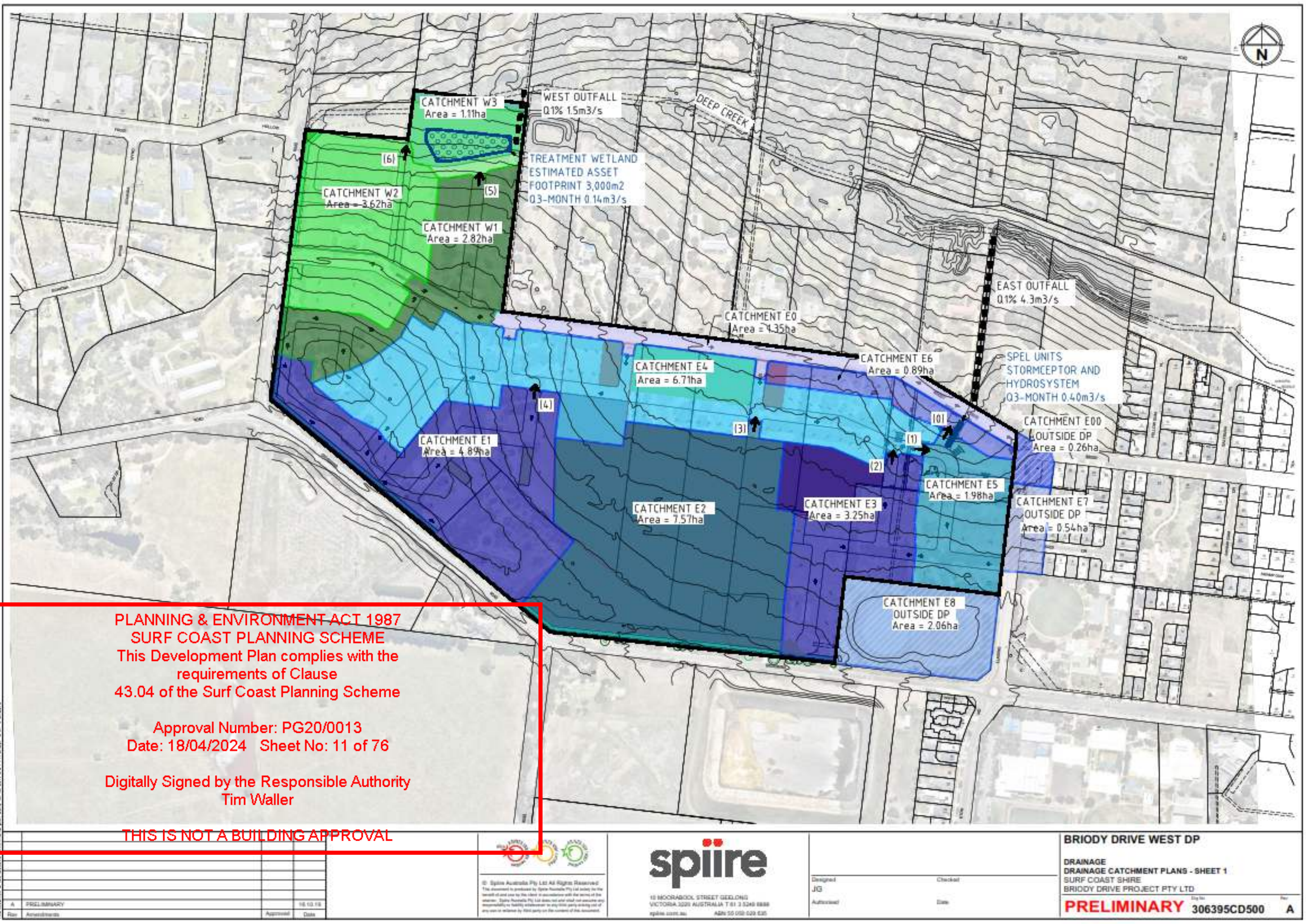

Figure 5 Spiire SWMS Catchment Plan

The catchments presented within this SWMS do not deviate significantly from the Spiire SWMS. A summary of the changes to the strategy is listed below.

- The eastern catchment is to be treated by sediment forebays and a bioretention garden prior to discharge at Briody Dr, in addition to SPEL proprietary treatment servicing only the Retirement Village and to be managed privately.
- Updated land use in accordance with the updated layout.

2.1.3. Deep Creek Flood Impact Assessment (Water Technology, August 2019)

The previous strategy for the region prepared by Spiire (Rev. I dated 15 June 2022) included a Flood Impact Assessment by Water Technology (V01, dated 28 August 2019), see Figure 6. The purpose of the assessment was to determine inundation for the 1% AEP flood event within Deep Creek, downstream from the Briody Drive West Development. The existing, developed, and mitigated scenarios were modelled where key differences and findings included:

- The modelled 1% AEP flood depths within Deep Creek showed slight variations in flood depths downstream of the development, for both developed and mitigated scenarios in comparison to existing conditions of the development, see Table 1. This is a result of the increase in flow volume (i.e., longer flow durations from the Site).
- Minor decreases along Deep Creek reach between the proposed outfalls from the Western and Eastern Catchment. In the developed scenario the peak flows from the Site can discharge to the creek and pass downstream before the larger Deep Creek flood flows reach the outfall from the Site.
- The channel within the portion of the creek adjacent to the development is well defined with an estimated bed level of 5 m below bank level. No meaningful changes in flooding were identified in during the modelled storm events.

The assessment highlights that as the section of Deep Creek nearby the development has a large 'slow' rural catchment, adding a small 'fast' developed catchment has very little impact on the existing waterway inundation. By the time the peak flows discharged from the 'slow' upstream catchment, passes through the section of the Creek north of the Site, the peak flows of the development will have flushed through by that point.

Similar to the conclusions reached in the Spiire strategy prepared for the development, Colliers Engineering & Design have confirmed the hydrological aspect of the Flood Impact Assessment and will be proposing no mitigated measures for the development due to the insignificant effects it would provide to inundation within Deep Creek.

Table 1. 1% AEP 1.5 hr duration – Flood Level Comparison (Water Technology – Table 4 - 11)

Location	1 (m AHD)	2 (m AHD)	3 (m AHD)	4 (m AHD)	5 (m 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

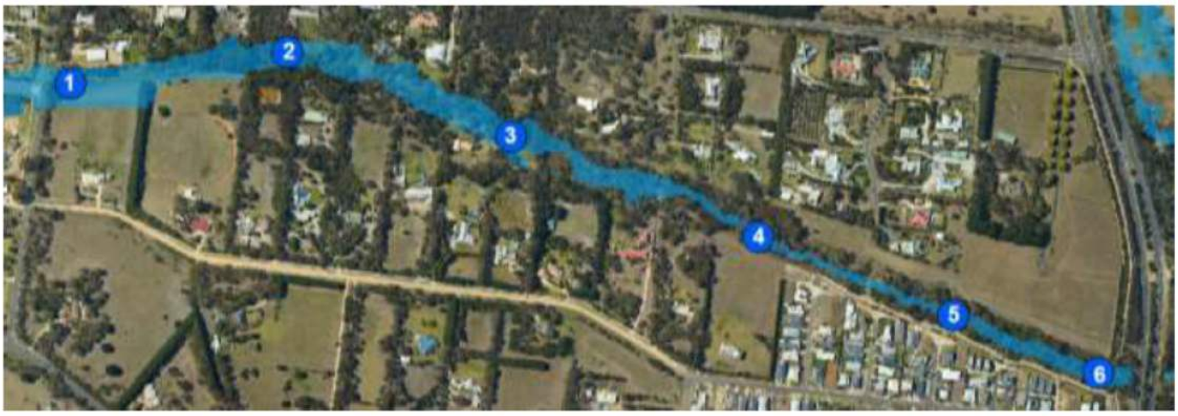

Figure 6. Flood Level Point Locations (Water Technology – Figure 4 - 12)

The previous assessment undertaken by Water Technology was based on an urban design layout with minor differences to the proposal within this SWMS. Additional land-use information allowed for a more detailed assessment of the post development fraction impervious than what was presented within the Spiire SWMS and informed the FIA. The developed land-use fraction impervious calculation, informed by the final development plan, and Melbourne Water MUSIC modelling guidelines for fraction impervious, found that proposed land-use within this document is less impactful than that undertaken by Water Technology. In addition, the short-fast discharge of the eastern catchment does not impact on the larger regional catchment driving the peak flooding within Deep Creek. Therefore, there is no impact to the previously completed hydraulic modelling.

3. CATCHMENT & OUTFALL CONDITIONS

The Site has two internal catchments falling towards the north-east and north-west. Calculations within this report are based on the assumption that all flows will discharge to the Council designated Legal Points of Discharge (LPOD), in the form of the proposed 1% AEP pipes and ultimately into Deep Creek

There is no external catchment earmarked to be conveyed through the site.

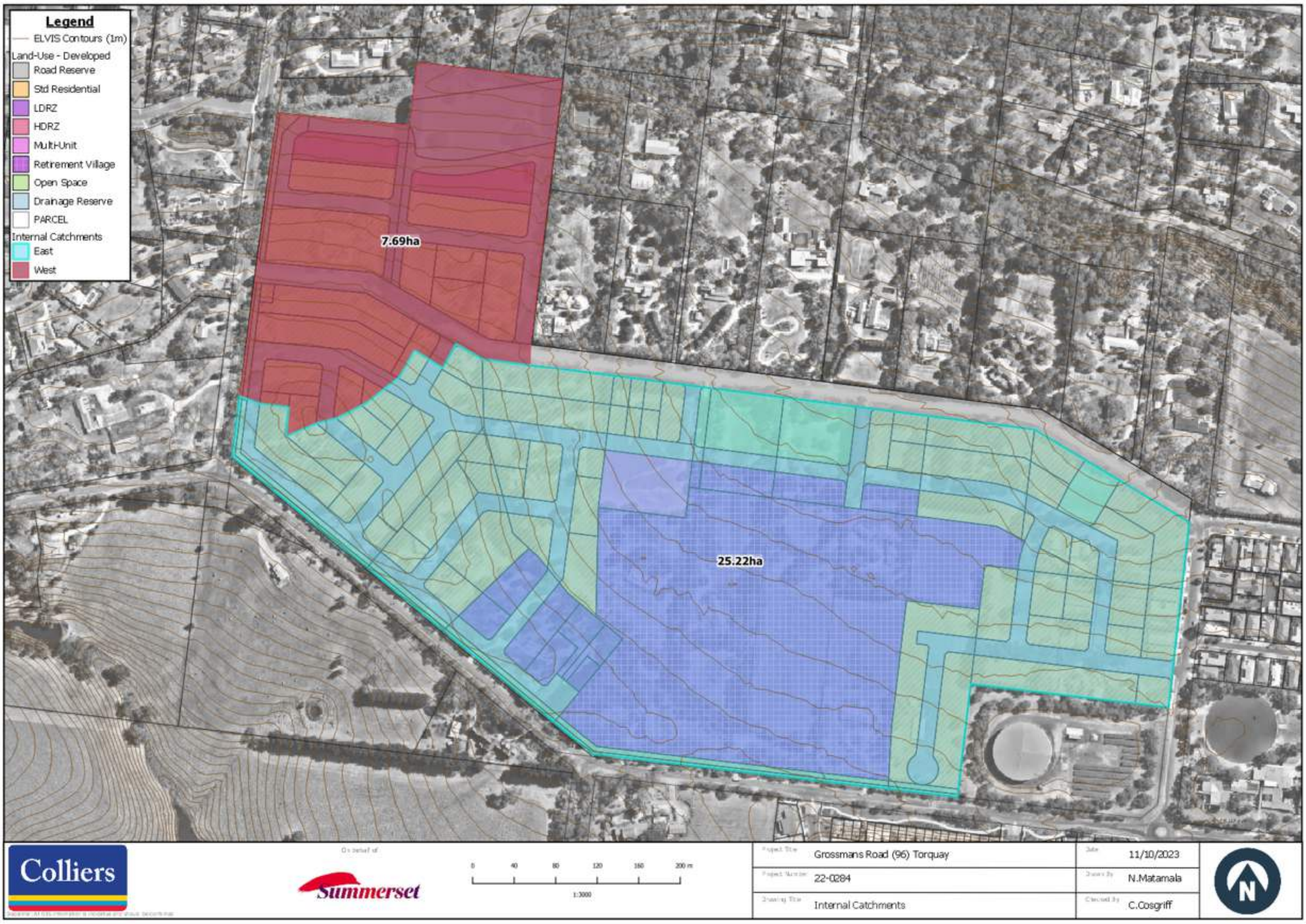

Figure 7. Post Development Catchments

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3.1. Eastern Catchment

The east catchment will drain to the north-eastern corner of the site, with stormwater treatment located within the 0.2 ha drainage reserve, and within the retirement village.

All flows for the events up to the 1% AEP will be conveyed to Deep Creek via a newly formed outlet pipe within the available 8m wide reserve immediately west of 90 Briody Dr. The exact details of the pipe will be confirmed during detailed design through expert advice from Arborists on Tree Protection Zones (TPZ) and discussions with relevant property owners. Exact outlet details will also 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 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.

3.2. Western Catchment

The western catchment will drain to the allocated reserve in the north-west corner of the site, with stormwater treatment in the form of a wetland.

Flows up to the 4EY (3-month ARI) will be conveyed into the treatment asset, with a new piped outlet to provide for flows up to the 1% AEP scenario for discharge into Deep Creek, which borders the drainage reserve. Exact outlet details will also be confirmed as part of the response to the Works on Water Application to the CCMA.

4. HYDROLOGICAL ANALYSIS

4.1. Pre-Developed / Existing Conditions

The existing Site has a substantially cleared landscape with rural roads and dwellings, see Figure 8. The Site is considered highly permeable with the full catchment breakdown in Table 2.

Land-use fraction imperviousness values have been based on Melbourne Water MUSIC Guidelines (2018).

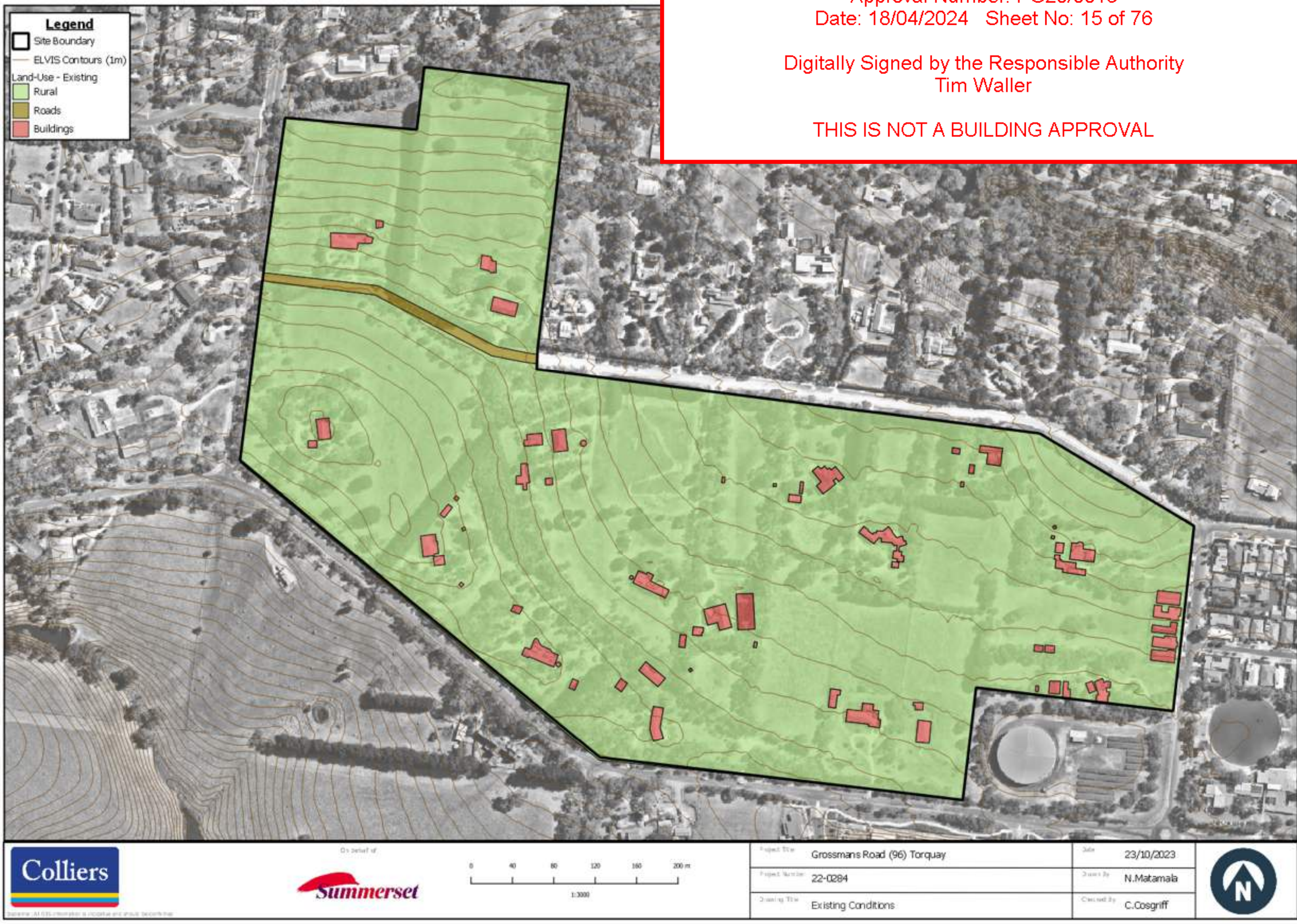

Figure 8. Existing Conditions

Table 2. Pre-Developed Catchment Break-Up – Western Catchment

Land Use	Fraction Impervious	Area (ha)
Rural	0.10	7.319
Roads	1.00	0.257
Buildings	1.00	0.118
Total	0.14	7.69

Table 3. Pre-Developed Catchment Break-Up – Eastern Catchment

Land Use	Fraction Impervious	Area (ha)
Rural	0.10	24.273
Buildings	1.00	0.942
Total	0.14	25.22

4.2. Developed Conditions

The Site is to be developed into a mix of standard, lower and higher density residential, public open space, retirement village and residential aged care facility. The developed conditions Layout Plan is represented in Figure 9. This proposed development will increase the imperviousness of the Site resulting in higher (site derived) peak flow rates and stormwater pollutant loads.

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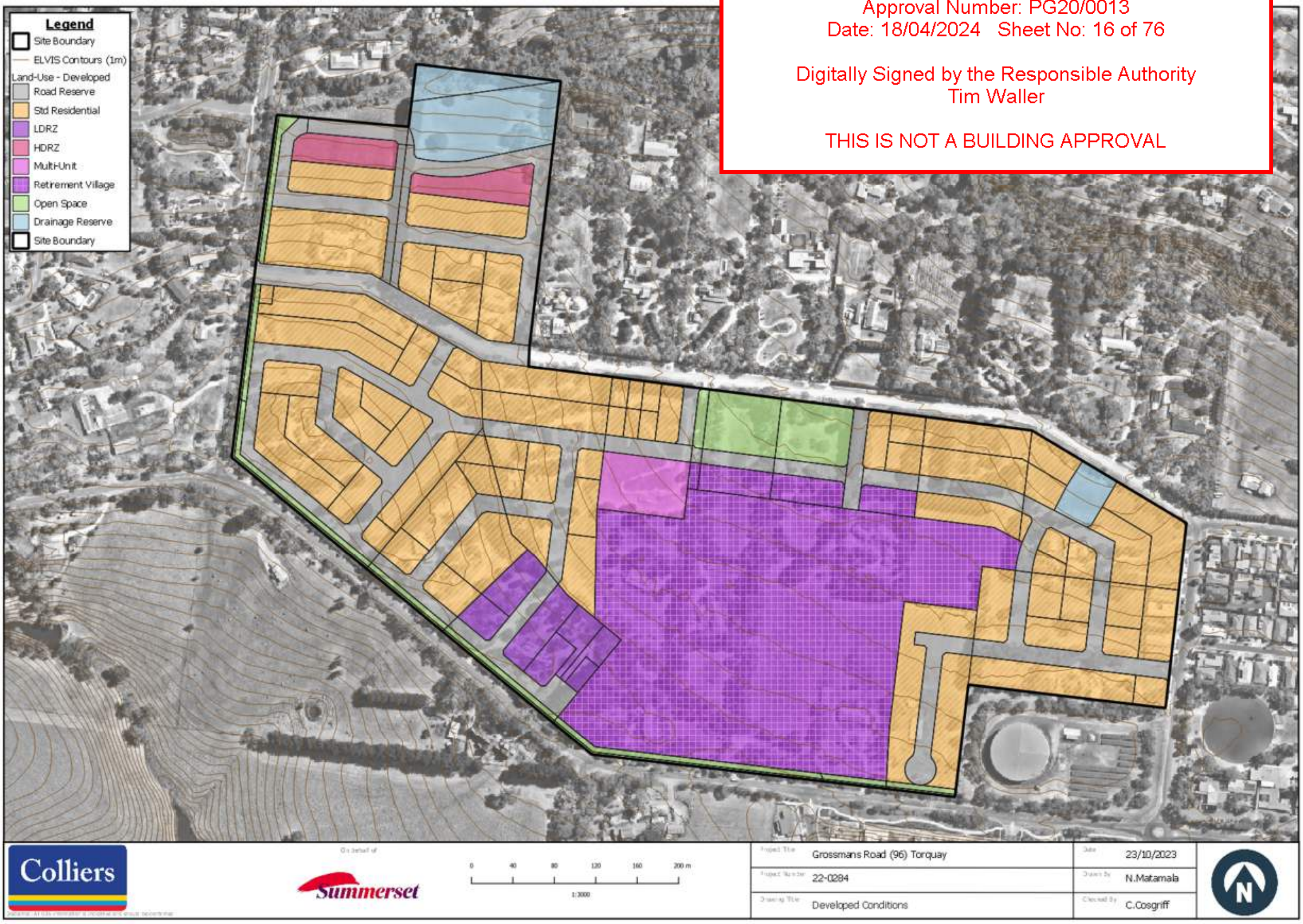

Figure 9. Developed Conditions

Fraction Impervious levels for each land-use type have been determined from Melbourne Water MUSIC Guidelines (2018), with the associated land-use break-up summarized in Table 4.

Table 4. Post-Developed Catchment Break-Up – Western Catchment

Land Use	Fraction Impervious	Area (ha)
Standard Residential	0.75	3.502
High Density Residential	0.90	0.586
Road Reserve	0.60	2.299
Open Space	0.10	0.193
Drainage Reserve	0.10	1.114
Total	0.61	7.69

Table 5. Post-Developed Catchment Break-Up – Eastern Catchment

Land Use	Fraction Impervious	Area (ha)
Standard Residential	0.75	9.507
Low Density Residential	0.60	0.941
Mixed Unit	0.90	0.468
Road Reserve	0.60	3.954
Open Space	0.10	1.506
Retirement Village	0.75	8.642

Drainage Reserve	0.10	0.200
Total	0.70	25.22

4.3. Hydrological Assessment

Water Technology have prepared hydrological and hydraulic modelling for the area, including discharge from the Site into Deep Creek, see Section 2.1 above.

Additional land-use information allowed for a more detailed assessment of the post development fraction impervious than what was presented within the Spiire SWMS and informed the FIO. The develop land-use fraction impervious calculation, informed by the final development plan, and Melbourne Water MUSIC modelling guidelines for fraction impervious, found that proposed land-use within this document is less impactful than that undertaken by Water Technology.

In addition, the short-fast discharge of the eastern catchment does not impact on the larger regional catchment driving the peak flooding within Deep Creek. Therefore, there is no impact to the previously completed hydraulic modelling.

Table 6. Land-Use Fraction Impervious Comparison

	Fraction Imperviousness		
Catchment	Peter Berry	Spiire	Colliers
West	0.70	0.73	0.61
East	Not specified	0.76	0.70

4.4. Minor Flows

The proposed development is to be designed do that the piped network conveys flows up to the 1% AEP scenario. To accommodate this, the piped network throughout the catchment will be sized gradually for capacity for the typical 20% AEP scenario in the south of the catchment (upstream), up to the 1% AEP scenario in the northern (downstream) sections. This will result in upsized pipes close to the outlet which will reduce the need for large 1% AEP capture pits at the outlet, reducing flood risk.

Underground drainage within the Retirement Village will be sized to convey up to the 1% AEP scenario with consideration given to larger events. The minor flow arrangement can be seen below in Figure 10.

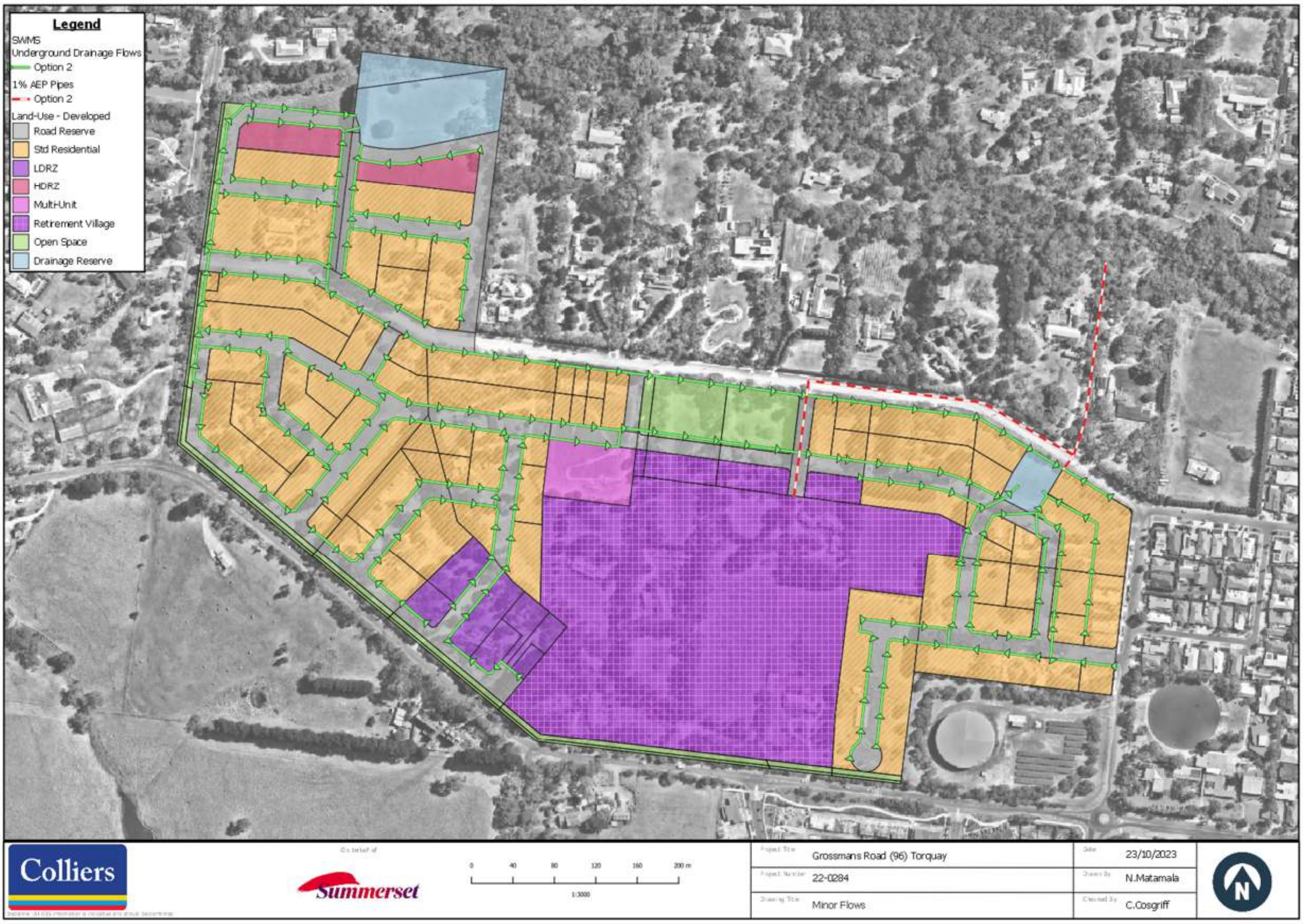

Figure 10. Minor Flows

4.5. Major Flows

Flows greater than the 20% AEP and up to and including the 1% AEP design event are to be conveyed through the Site utilising the road network as urbanised floodways. The road reserves will be designed in order to convey these flows whilst ensuring DEWLP's floodway safety criteria (Dmax <= 0.3m, Vmax <= 2.0m/s and VDmax <=0.3m²/s).

The drainage network is to be designed so that the overland flow at Briody Dr and roads in close proximity will be a bare minimum. Figure 11 highlights the proposed overland flows that will impact on the Site.

PC Convey at critical locations was completed to determine the road reserve capacity. Figure 11 highlights the critical overland flow section and contributing catchment. Table 7 summarizes the overland flow calculations undertaken.

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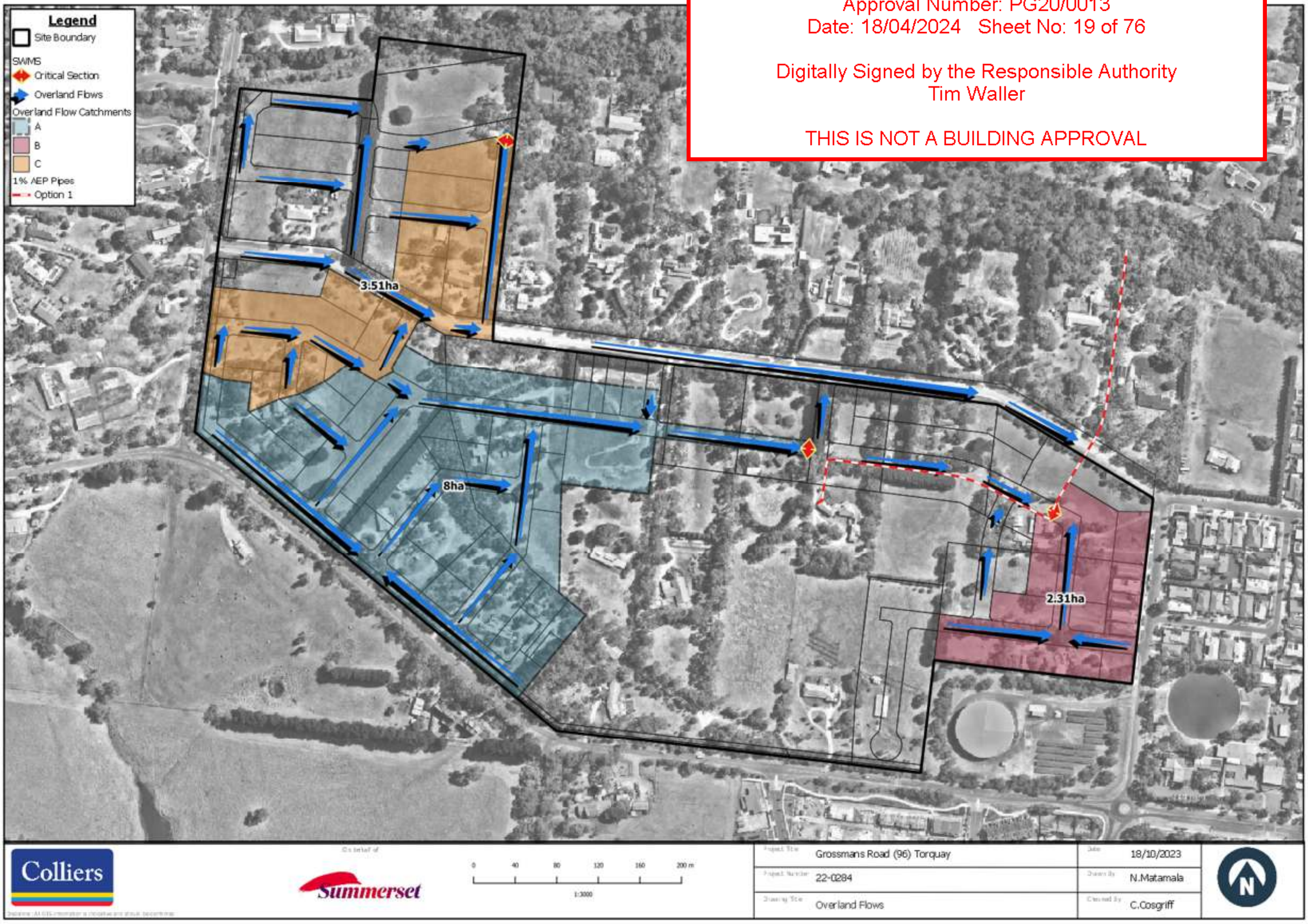

Figure 11. Overland Flows

Table 7. Overland Flow Calculation

Catchment	Area (ha)	Tc (mins)	C1%	I1% (mm/hr)	Q1% (m³/s)	C20%	I20% (mm/hr)	Q20% (m³/s)	$Q_{overland}$
A	8.000	18.1	0.759	79.83	1.35	0.601	37.85	0.51	0.84
B	2.308	7.6	0.793	122.12	0.62	0.628	59.20	0.24	0.38
C	3.513	10.4	0.776	106.60	0.81	0.614	50.96	0.31	0.50

Table 8. Oversized Pipe – Overland Flow

Key Road Location	$Q_{1\%}$	$Q_{1\%-20\%}$	Q_{gap}	Road Width	Grade Long
A	1.35	0.51	0.84	14.5m	1 in 200
B	0.62	0.24	0.38	14.5m	1 in 500
C	0.81	0.31	0.50	16.0m	1 in 50

Figure 12 highlights that the water surface elevation within the critical sections of the Site provides sufficient freeboard. HEC-RAS modelling of all road reserves within the development will be completed as part of the detailed design to satisfy this condition.

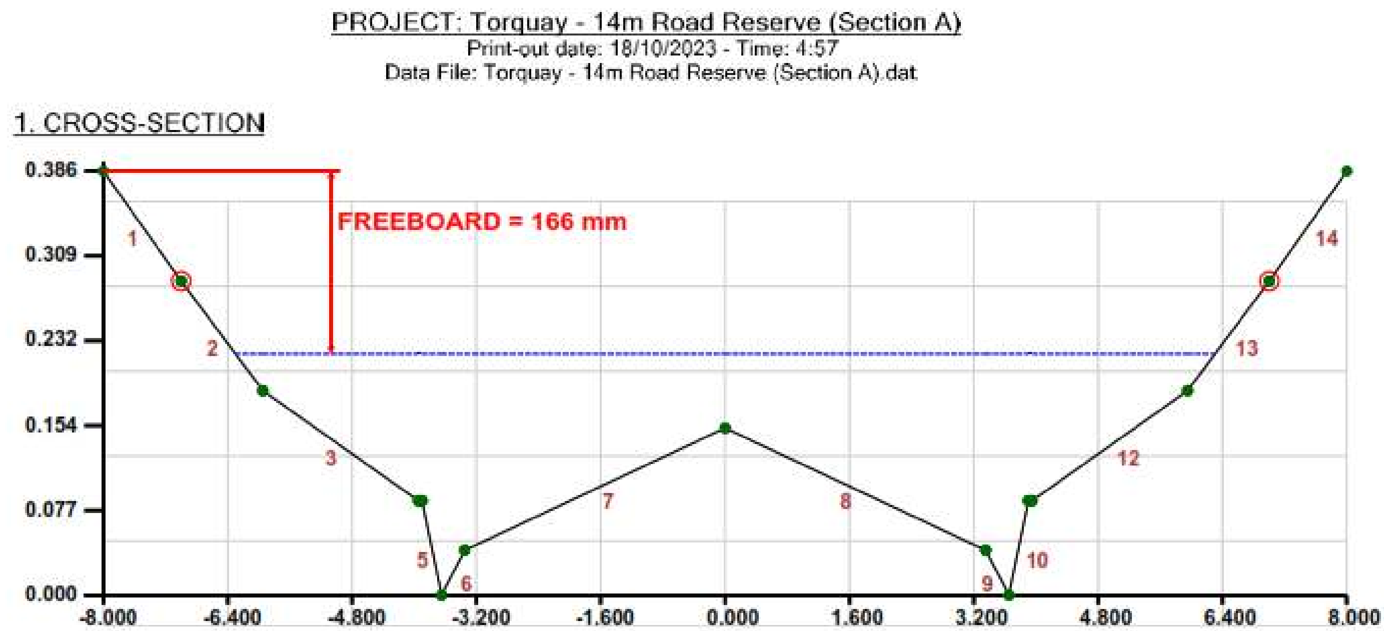

Figure 12. Critical Section PC Convey Calculation – Catchment A

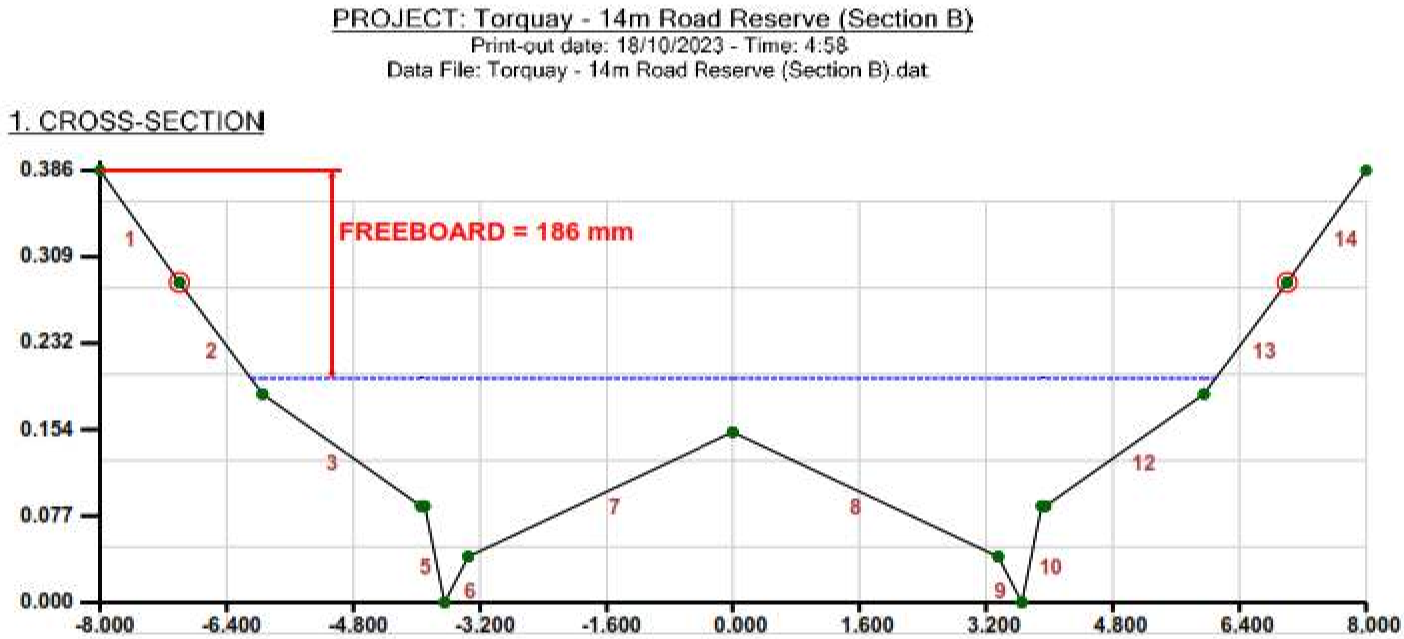

Figure 13. Critical Section PC Convey Calculation – Catchment B

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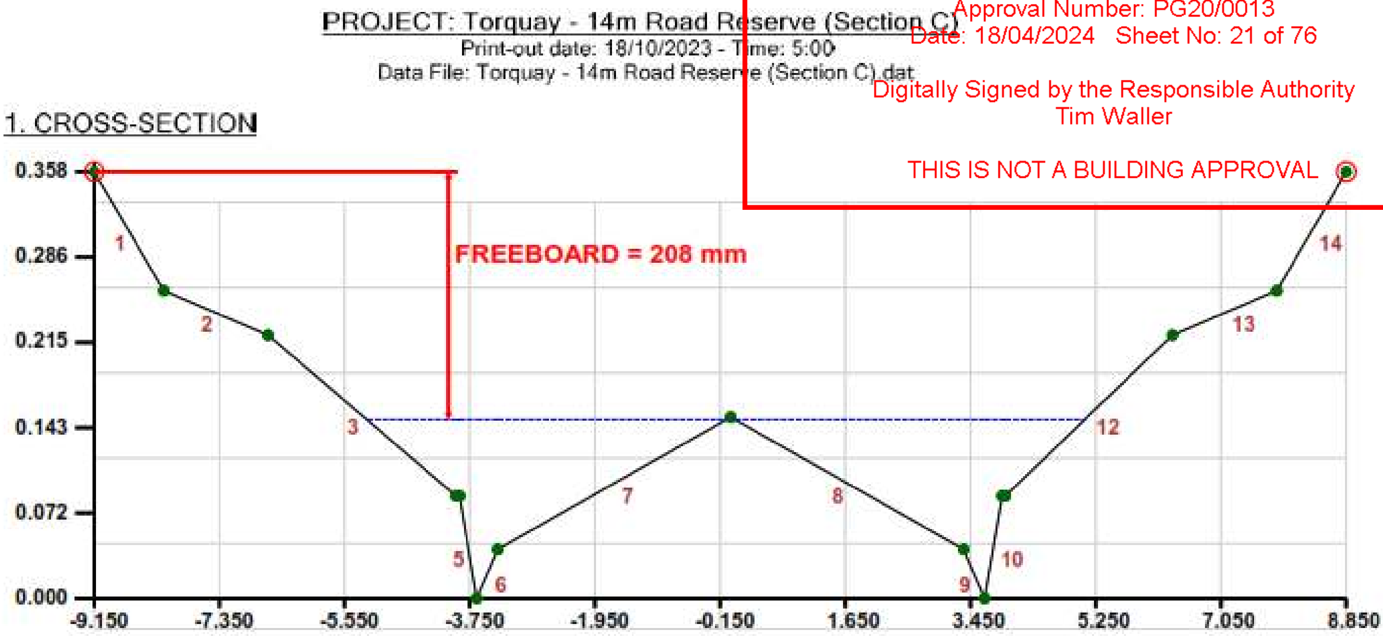


Figure 14. Critical Section PC Convey Calculation – Catchment C

4.6. Ultimate Outfalls

An assessment of the outfall pipe sizes and grades to Deep Creek for each catchment are provided below in Table 9. Pipes are to cater for the 1% AEP event with no flood attenuation required beyond the extended detention within the water treatment assets, due to the findings of the Flood Impact Assessment of Deep Creek by Water Technology. Exact details of the connection to the Creek will be confirmed with the Works on Waterway Submission to the CCMA upon the finalisation of the document.

Table 9. Outfall Sizing

Catchment	$Q_{1\%}$	Indicative Grade	Indicative Pipe size
East	4.04 m^3/s	1 in 75	1200mm dia
West	1.55 m^3/s	1 in 40	750mm dia

The above sizing is based on a manning calculation of the estimated pipe sizes at the expected grade. The exact required pipe size will be investigated further during detailed design of the outlets.

5. STORMWATER QUALITY

To satisfy the environmental values expected, a series of treatments is to be provided throughout the wider catchment area. These assets will be designed to ensure they satisfy best practice targets set out in the Best Practice Environmental Management Guidelines (BPEMG), which are:

- 45% reduction in Total Nitrogen (TN) from typical urban loads.
- 45% reduction in Total Phosphorus (TP) from typical urban loads.
- 80% reduction in Total Suspended Solids (TSS) from typical urban loads; and
- 70% reduction in Litter from typical urban loads.

In order to determine the required levels of water quality treatment for the Site a MUSIC model was created. The “Geelong North” Rainfall template was used due to its proximity to the Site.

Table 10. MUSIC Model Parameters

Rainfall Parameters		Runoff Parameters	
Rainfall Station	087133 - Geelong North	Soil Storage Capacity	120mm
Date	1971 - 1980	Initial Storage	25mm
Time Step	6 Minutes	Field Capacity	50mm

The Site has been broken down in various catchments to represent the various land-uses, as represented in Figure 15 and Table 11. The land use fraction impervious is in accordance with Melbourne Waters MUSIC guidelines and shown below in Table 12.

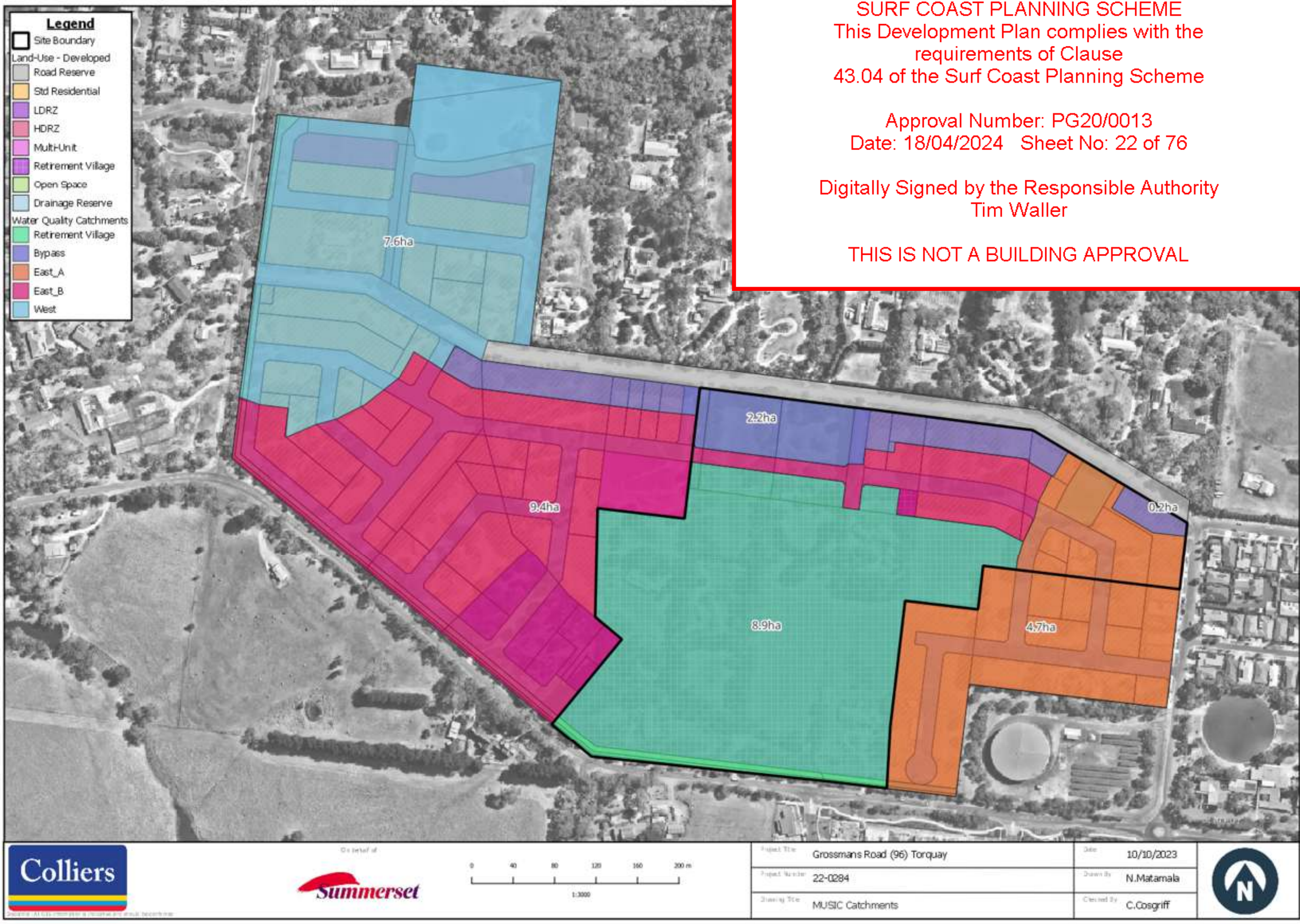


Figure 15 MUSIC Catchments

Table 11. MUSIC Model Catchment Node Details

Land Use	Fraction Impervious	Area (ha)
West	0.63	7.605
East_A [Roof to Tank]	1.00	0.640
East_A [Remainder]	0.66	4.065
East_B [Roof to Tank]	1.00	1.149
East_B [Remainder]	0.63	8.227

Bypass [Roof to Tank]	1.00	0.225
Bypass [Remainder]	0.45	2.123
Retirement Village [Roof to Tank]	1.00	0.720
Retirement Village [Remainder]	0.71	8.167

Table 12. Land Use Fraction Impervious

Land Use	Fraction Impervious
Multiple Unit	0.90
Road Reserve	0.60
Open Space	0.10
Standard Residential	0.75
Low Density Residential	0.60
High Density Residential	0.90
Retirement Village	0.75
Drainage Reserve	0.10

The strategy for each catchment outlet is described below.

5.1. Western Catchment

The proposed water quality treatment system is proposed as a constructed wetland with a sediment pond located upstream as the primary treatment. Flows exceeding the peak 4EY event flow will be bypassed upstream of the sediment pond and directed to deep creek via the proposed 1% AEP pipe.

The treatment asset sizing was done by first sizing the sediment pond using the Fair and Geyer equation, then upsizing the wetland marsh area until best practice water pollutant reduction targets were met.

Fair and Geyer Equation – Equ 10.3 WSUD Stormwater Technical Manual (2004)

$$R = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \cdot \frac{(d_e + d_p)}{(d_e + d^*)}\right]^{-n} \qquad \lambda = 1 - 1/n; \qquad n = \frac{1}{1-\lambda}$$

R = fraction of Initial Solids Removed = 80 - 90 % typ.
d_p = Depth of permanent pool
d_e = Extended detention depth above permanent pool
d* = depth below permanent pool sufficient to retain particles (lower of 1.0m or d_p)
Q = design flow (Typically 3 month, 6 month or 1 year flow)
A = Basin Surface Area
n = turbulence parameter (see above) = 1 for significant short circuiting and turbulence
= 5 for insignificant short circuiting and turbulence
v_s = setting velocity for particles

Figure 16. Fair and Geyer Equation for Sedimentation Basin Sizing

Table 13. Interim Sedimentation Basin Parameters

Particle Size Target	125µm
Particle Settling Velocity	0.011m/s
Capture Rate	95%
Basin Surface Area	324 m^2
Extended Detention Depth	0.35m
Permanent Pool Depth	1.50m
Design Flow	0.130m^3/s (3-month)
Turbulence Parameter	1.12
Clean Out Frequency	5-year
Sediment Loading	25m^3/ha/yr
Gross Pollutant Loading	0.4m^3/ha/yr
Sediment Storage Volume Required	75m^3

Dry Out Area Required (500mm depth)	150m²

The MUSIC model and treatment nodes, showing the treatment train arrangement and performance can be seen below in Figure 17 and Figure 18.

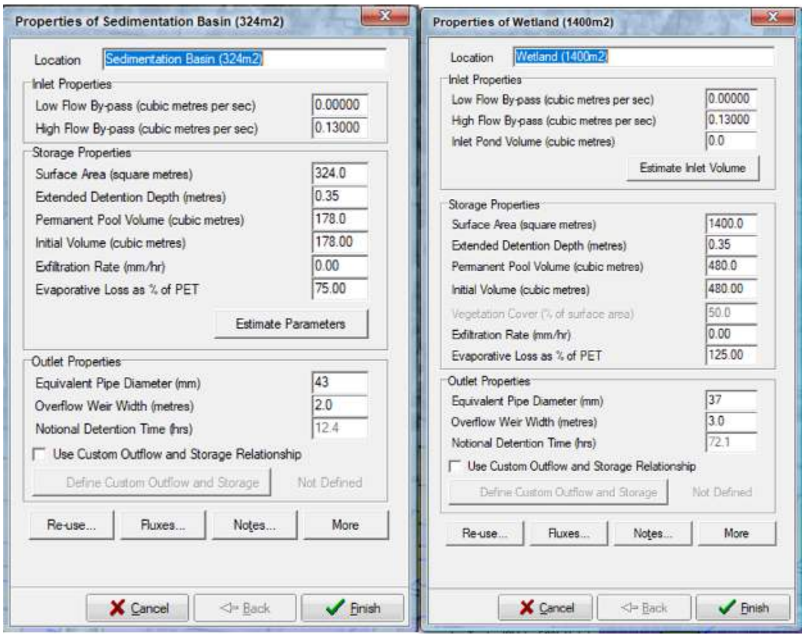
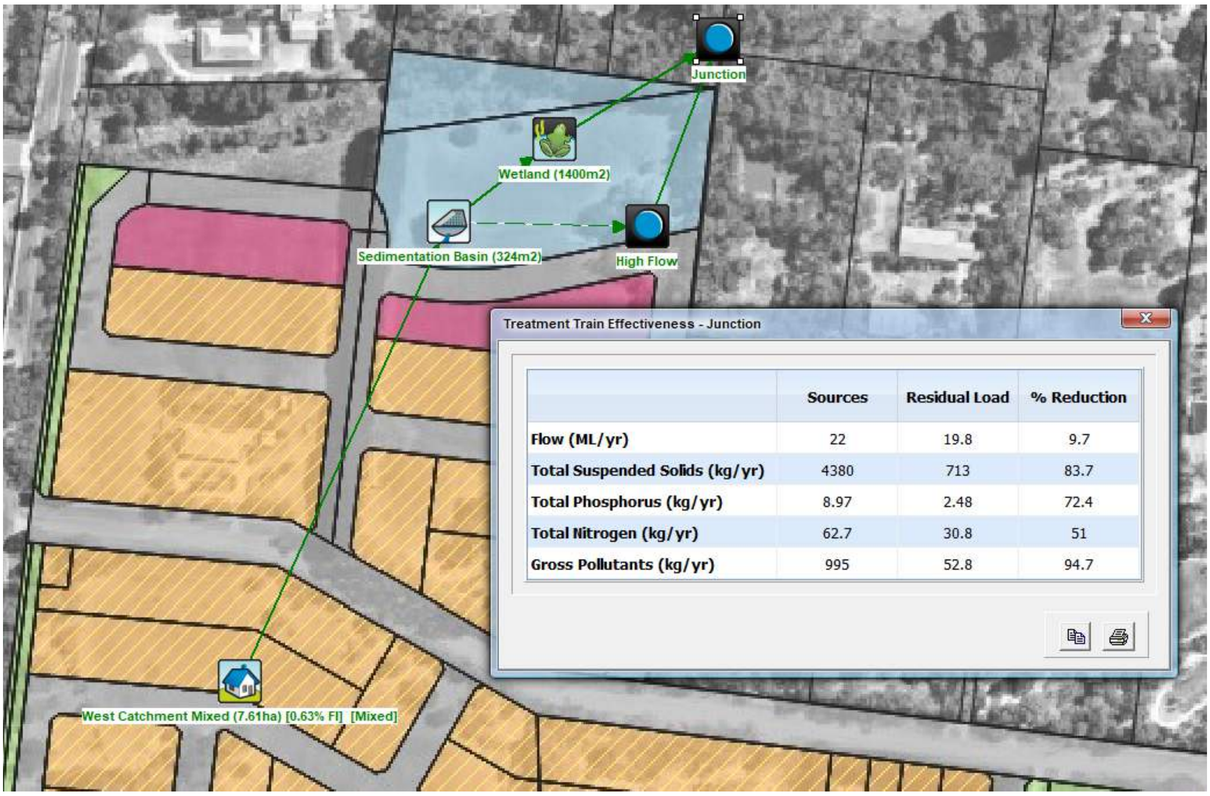


Figure 17 Western Treatment Nodes



	Sources	Residual Load	% Reduction
Flow (ML/yr)	22	19.8	9.7
Total Suspended Solids (kg/yr)	4380	713	83.7
Total Phosphorus (kg/yr)	8.97	2.48	72.4
Total Nitrogen (kg/yr)	62.7	30.8	51
Gross Pollutants (kg/yr)	995	52.8	94.7

Figure 18 Western MUSIC Model

The western catchment meets the best practice treatment targets using a 324m² sediment pond and a 1400m² wetland, with an expected footprint of approximately 0.3ha.

5.2. Eastern Catchment

Water quality objectives will be achieved on Site using an integrated approach to stormwater management, incorporating a variety treatments and water saving measures.

Rainwater tanks are to be utilised solely within the eastern residential development and main residential aged care facility. It is proposed that a Section 173 agreement be applied to all residential lots to ensure provision of rainwater tanks. The Surf Coast Shire Council has allowed the modelling of 50% uptake for water quality purposes. This is considered a conservative approach and provides confidence that water quality objectives for the site will be exceeded.

The end of line treatment is to be separated between the retirement village and remaining residential development. The retirement village will be treated using privately owned SQIDEP approved proprietary products, a SPEL Stormceptor, and tertiary treatment units in the form of 8xSPEL Filter Vault. The surrounding residential development will be treated by a bio-retention system with upstream primary treatment in the form of sediment forebays.

In order to determine residential rainwater usage demand a 20L/person/day has been used. Census data has informed the standard 2.7 persons/house. An average lot size of 450m² has been assumed. Rainwater tank catchments have been conservatively estimated as 70% of roof areas. Roof areas have been estimated at 70% of lot area.

Residential rainwater reuse:

- Residents = 2.7 persons/house
- Re-use demand = 20L/person/day
- Average lot size = 450m²
- Assumed roof catchment = 70% of lot size
- Assumed tank catchment = 70% of roof catchment
- Tank size = 2kL
- Tank uptake = 50% lots

Rainwater tank usage within the retirement village is proposed via catchment from the main building roof. Reuse is proposed for internal use (20L/person/day) and irrigation of open space within the retirement village.

Retirement Village rainwater reuse:

- Residents = 417
- Internal re-use demand = 20L/person/day
- Internal re-use demand total = 8.34kL/
- Irrigation re-use demand
    - Irrigation area = 0.285ha
    - Irrigation demand = 3.2ML/yr/ha
- Tank catchment size = 7200m²
- Tank size = 4x 10kL

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Using the stormwater harvesting and reuse demands for irrigation detailed by City of Geelong's MUSIC guideline document (The City of Greater Geelong, 2019). An approximation of the amount of reuse required for irrigation purposes for the main retirement village facility was input into the MUSIC modelling. Refer to Table 14 and Table 15 for the irrigation demand and the breakdown of the monthly proportion for the Site, respectively.

Table 14. Typical Reuse Demand

Type	Typical Reuse Demand (ML/Ha/yr)
Warm Season Turf	3.2
Cool Season Turf	4.5

Table 15. Monthly Distribution of Reuse Demand

Month	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
% of annual demand	22	17	16	4	0	0	0	0	2	8	12	19

Table 16. Annual Irrigation Demand for Main Building

Total Open Space Area (Main Building)	2850 m²
Annual Application Rate for Irrigation	3.2 ML/Ha/yr
Annual Volume Requirement	0.912 ML/yr

The full estate was modelled in MUSIC to ensure Best Practise is being achieved within the development where the model configuration and overall performance of the system is shown in Figure 19.

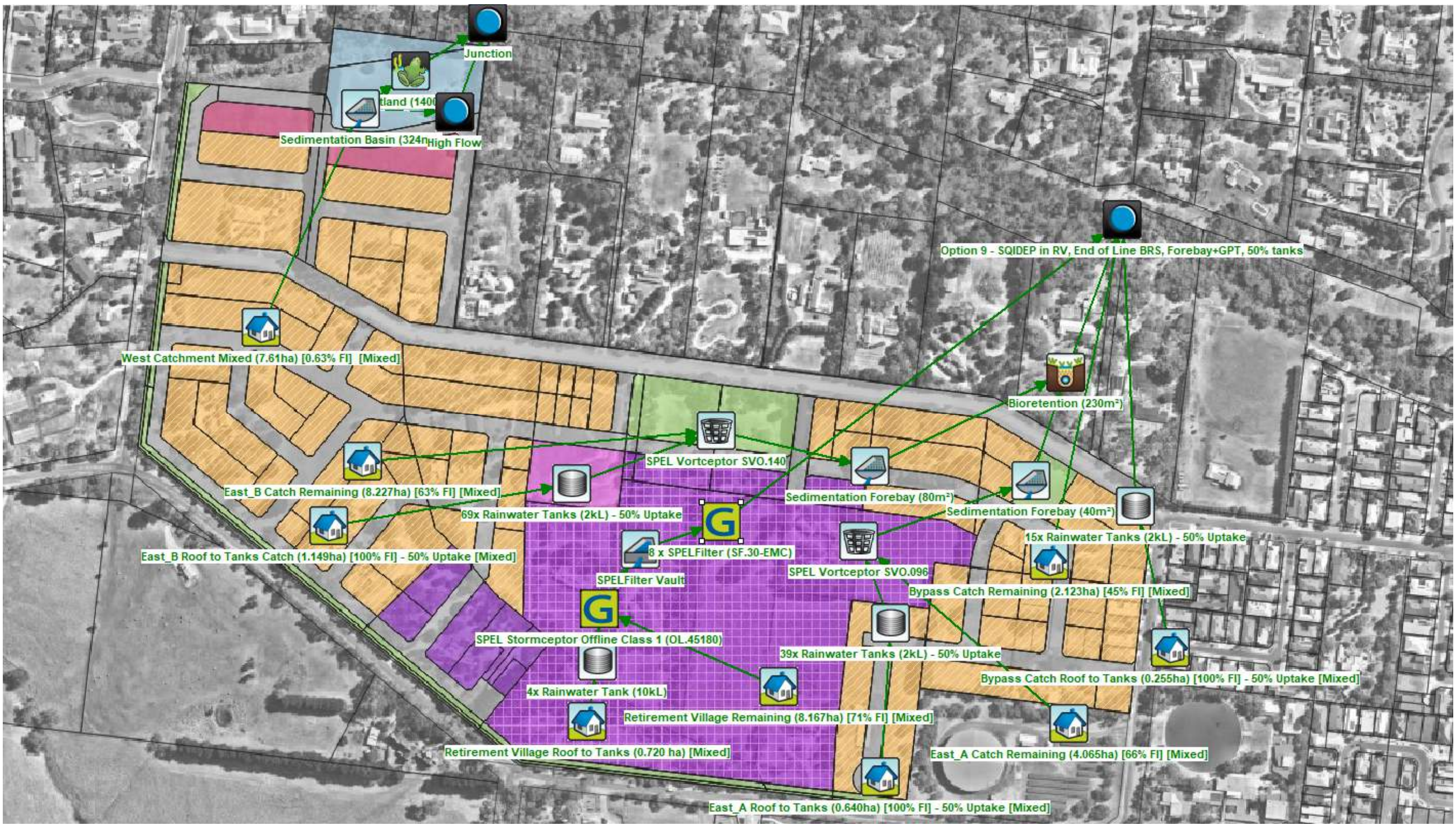

Figure 19. MUSIC Model Configuration

The MUSIC nodes for the SPEL Stormceptor (OL.45180.C1), SPEL Vortceptor (SVO.140) and SPEL Filters (SF.30-EMC) used in the modelling have been provided by the manufacturer and haven't been altered. The treatment properties of these nodes have been detailed in Table 17. The node properties and monthly distribution of the rainwater tanks are represented in Figure 20.

Table 17 Treatment Properties of SPEL MUSIC Nodes

	SPEL Stormceptor (OL.45180.C1)	SPEL Vortceptor (SVO.096 or SVO.140)	SPELFilter (SF.30-EMC)
High Flow By-pass	0.3 m³/s	0.096 or 0.136 m³/s	0.024 m³/s
Flow	0%	0%	0%
Gross Pollutants	100%	99%	100%

Total Phosphorus	10%	70%	26%
Total Nitrogen	23%	0%	41%
Total Suspended Solids	87%	30%	98.5%

Section

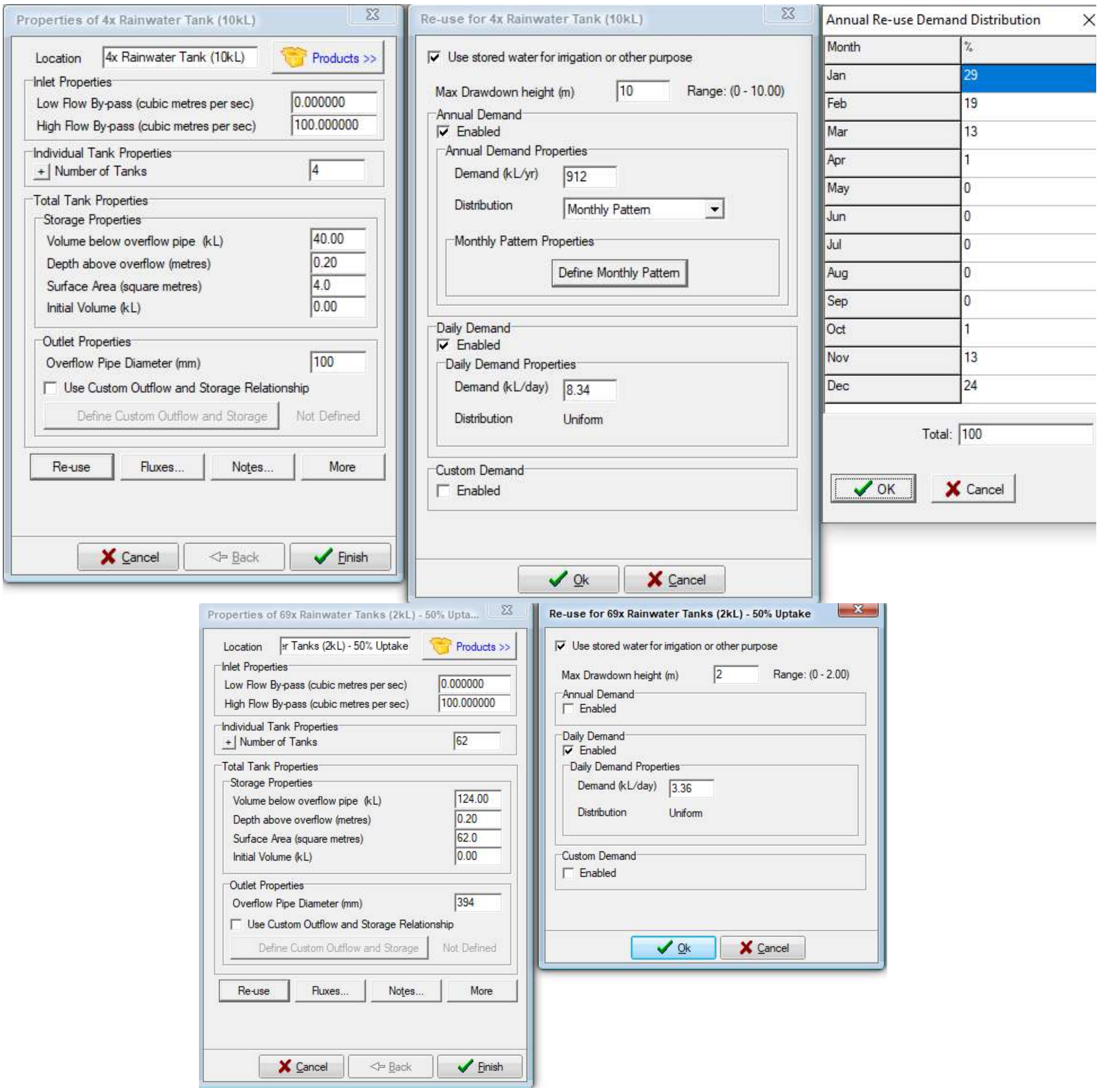

Figure 20. Rainwater Tanks MUSIC Node Properties

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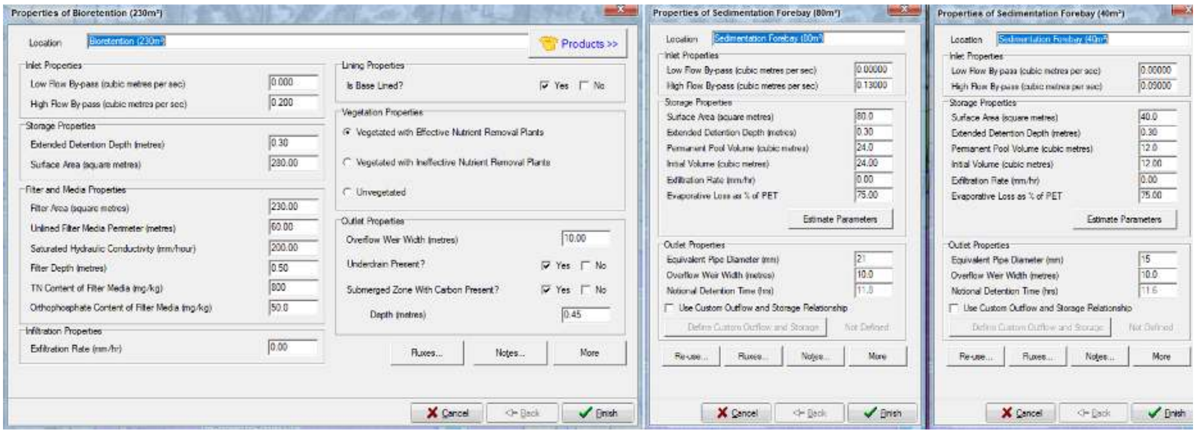


Figure 21. Bioretention MUSIC Node Properties

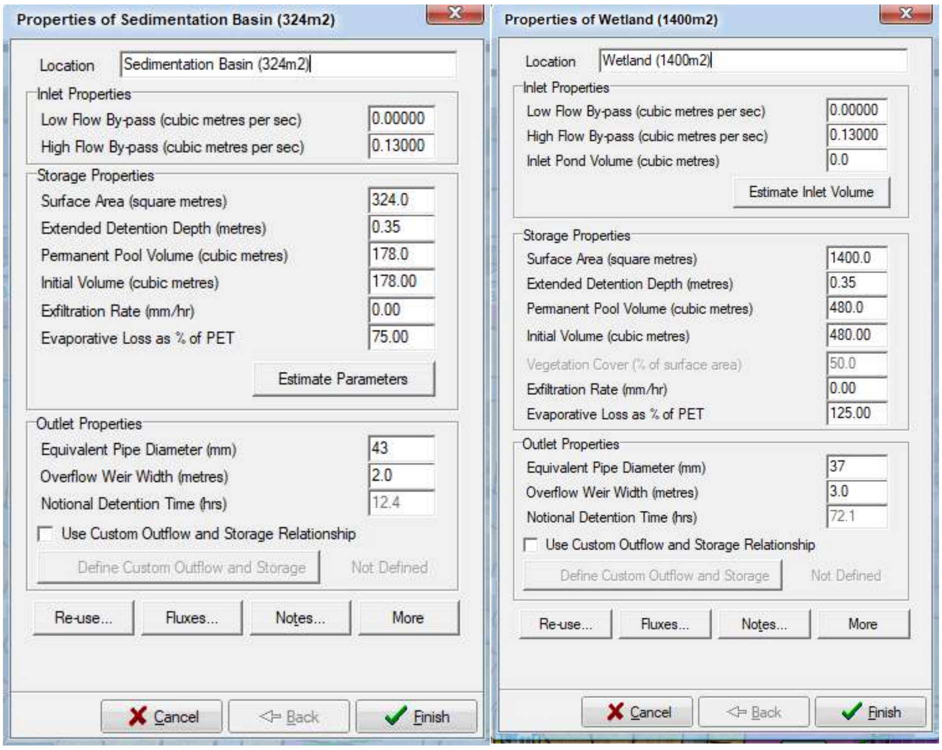
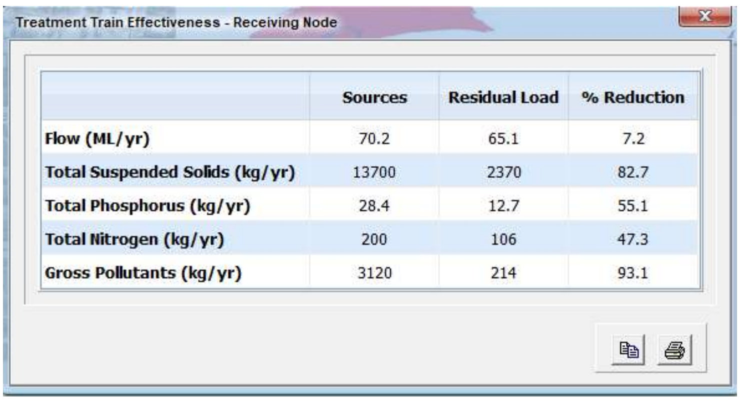


Figure 22. Wetland MUSIC Node Properties


Treatment Train Effectiveness - Receiving Node

	Sources	Residual Load	% Reduction
Flow (ML/yr)	70.2	65.1	7.2
Total Suspended Solids (kg/yr)	13700	2370	82.7
Total Phosphorus (kg/yr)	28.4	12.7	55.1
Total Nitrogen (kg/yr)	200	106	47.3
Gross Pollutants (kg/yr)	3120	214	93.1

Figure 23. MUSIC Model Treatment Train Effectiveness

Through the above-mentioned stormwater quality treatment train, Best Practise Environmental Management Guidelines have been achieved for pollutant targets, GP TSS, TP, and TN.

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5.3. Water Quality Summary

Western Catchment – Council Maintained

- Sediment Basin
    - Treatment size: 324m²
    - Council maintained
- Wetland
    - Treatment size: 1400m²
    - Council maintained

Eastern Catchment

- 2x Gross Pollutant Traps
    - Treatment flow: 3-month
    - Council maintained
- 2x Sediment Forebays
    - Treatment size: 80m² and 40m²
    - Council maintained
- Bioretention Basin
    - Treatment size: 230m²
    - Council maintained
- Residential rainwater tanks
    - Size: 2kL
    - Privately owned

Retirement Village

- Proprietary Treatment (SPEL/Atlan)
    - Treatment system:
        - Stormceptor OL.45180
        - 8x Filter SF.30
    - Privately maintained
- Main building rainwater tanks
    - Size: 4x 10kL
    - Privately owned

6. Cost Estimate

The Shared Infrastructure Funding Plans (SIFP) has been updated to reflect the revised Development Plan as well as updated 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.

Refer to the Shared Infrastructure Funding Plan (SIFP) for a full breakdown of all drainage assets in both catchments.

7. SUMMARY

Colliers Engineering & Design was engaged to undertake a Stormwater Management Strategy for Briody Drive, Torquay, ensuring that the estate adheres to the expectations of Surf Coast Shire Council and other regulatory guidelines from a more holistic perspective.

The existing Site has a total area of 32.91 ha and is substantially bare and is considered highly permeable, with the exception of several scattered residential dwellings, rural roads and small sheds. The Site is to be developed into varying density residential lots with the inclusion of open space areas and a retirement village.

This strategy takes into consideration separate studies of the area, such as Water Technology's Flood Impact Assessment on Deep Creek where the site ultimately discharges. Flows generated on Site will not require detention prior to being discharged into Deep Creek. The flood assessment indicated that discharging at the post development flow rate without mitigation had an negligible impact on the flood levels within Deep Creek and the wider region. The land uses and fraction impervious proposed within this report do not exceed the assumptions from the Flood Impact Assessment.

Stormwater quality objectives will be achieved within the east by residential rainwater tanks and an end of line bioretention basin. Treatment within the retirement village will be via stormwater harvesting and the installation of a SPEL Stormceptor and SPELFilter units, all of which will be privately owned and maintained. These systems have been designed to achieve Best Practice guidelines for stormwater quality. The western catchment meets its stormwater quality objectives via a sediment pond and a wetland, sized to satisfy the sediment storage and particle settlement requirements and the wetland iterated in size until best practice was met.

Within the retirement village all flows up to the 1% AEP will be collected and conveyed underground to the Site's legal point of discharge, avoiding overland flows within the development for major events. As such, the road network won't be utilised as an urbanised channel to convey the gap flows, simplifying its design in order to mitigate the tripping hazards to the residents of the retirement village.

A standard drainage network will be utilised for all parts of the development outside of the retirement village, with flows up to the 20% AEP being captured and conveyed by the underground drainage network. Flow exceeding the 20% AEP events and up to the 1% AEP event (gap flows) will utilise the road network as urbanised channels. The pipes will be gradually upsized as the catchment moved towards Broidy Dr, to avoid excessive overland flow and subsequent capture pits at this location, which would cause an unnecessary flood risk.

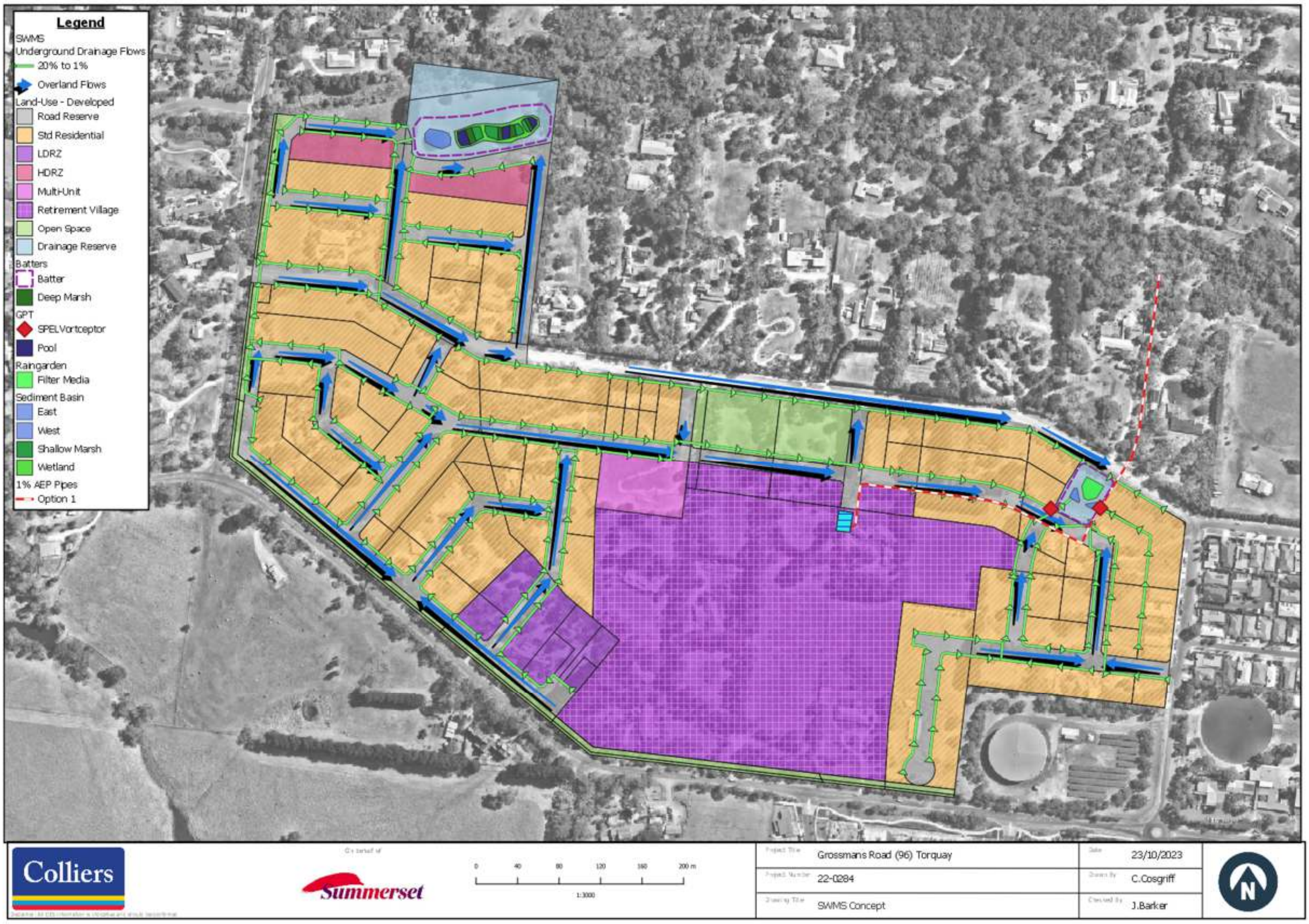

Figure 24. SWMS Concept

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8. REFERENCES

- *Australian Rainfall and Runoff 2016*
- Melbourne Water (2018), MUSIC Guidelines: *Recommended Input Parameters and Modelling Approaches for MUSIC Users*
- Victorian Stormwater Committee (1999), *Urban Stormwater Best Practice Environmental Management Guidelines*
- Growth Areas Authority (2011), *Engineering Design and Construction Manual for Subdivision in Growth Areas*
- Melbourne Water (2013), Waterway Corridors – *Guidelines for greenfield development areas within Port Phillip and Westernport Region*
- DEWLP (2019), *Guidelines for Development in Flood Affected Areas*

Appendix A Development Plan

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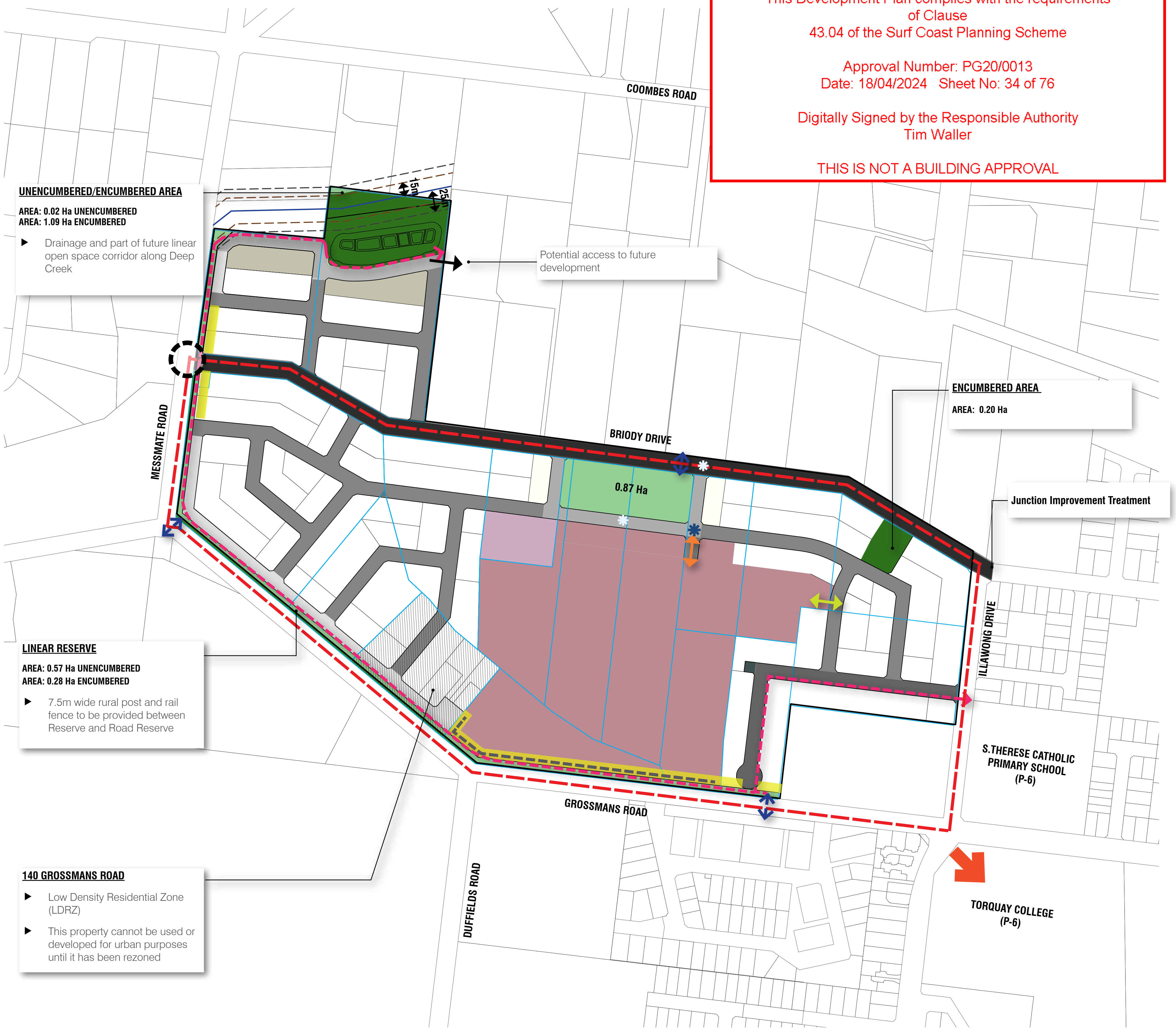

DENSITY

RESIDENTIAL TYPE	DENSITY (LOTS/ NRA)	INDICATIVE LOT RANGE[6]	INDICATIVE NO. OF LOTS	AREA
STANDARD RESIDENTIAL	22.2	350m²-900m²	313	14.23 Ha (net residential)
HIGHER DENSITY RESIDENTIAL	28.5	250m²-325m²	17	0.61 Ha (net residential)
MULTI-UNIT SITE	24		11	0.47 Ha (site)
RETIREMENT VILLAGE, RESIDENTIAL AGED CARE AND INDEPENDENT AND ASSISTED LIVING APARTMENTS[1]			231 independent retirement village units 80 residential aged care beds 60 independent & assisted living apartments	8.51 Ha (site)

ROAD NETWORK

- CONNECTOR LEVEL 1 STREET
  (4.8 m wide nature strip on south side to cater for 2.3 m indented parking bays to be provided in the future)
- ACCESS LEVEL 1 STREET - 16.25m
- ACCESS LEVEL 1 STREET - 16m
- ACCESS LEVEL 1 STREET - 14.5m
- ACCESS LEVEL 1 STREET - 12.5m
- MEWS - 12m
- INDICATIVE INTERNAL PRIVATE ROAD WITHIN RETIREMENT VILLAGE AND RESIDENTIAL AGED CARE
- FUTURE ROUND A BOUT
- TRAFFIC MANAGEMENT TREATMENT
- CONSIDER SAFE TRAFFIC AND SPEED CONTROLS AT PLANNING PERMIT STAGE

OTHER

- POTENTIAL FUTURE BUS ROUTE
- INDICATIVE LOCATION OF SHARED PATH
- DIRECTION TO SCHOOLS AND COMMUNITY FACILITIES
- EXISTING TITLE BOUNDARIES[2]
- FENCE PROVISIONS
- RETIREMENT VILLAGE ENTRY (CARS)
- RETIREMENT VILLAGE ENTRY (PEDESTRIAN AND EMERGENCY ACCESS/EGRESS)
- PROPOSED INDICATIVE CROSSING LOCATIONS TO EXISTING PATHS

OPEN SPACE/DRAINAGE

- ENCUMBERED OPEN SPACE
- UNENCUMBERED OPEN SPACE
- CENTRE LINE OF CREEK

NOTES

1. Development on the Retirement Village / Residential Aged Care site will generally be a maximum height of two storeys. However, any three-storey component of the building containing the Residential Aged Care Facility and Independent and Assisted Living Apartments must be setback at least 75 metres from any boundary of the site.
2. Whilst some parcel areas have been surveyed, other parcels have been sourced from data.vic.gov.au. Survey required to determine final areas.
3. Section 1 & 2 uses that are permissible within the zone will be considered on their merits.
4. Upgrade to Briody Drive to Connector level 1 to be undertaken in two stages.
5. Roundabout to be provided by Council at a time in the future as traffic volumes necessitate.
6. All Cultural Heritage Management Conditions in the approved Briody Drive West, Torquay Subdivision and Development Cultural Heritage Management Plan (FP-SR# 16746), authored by Extent Heritage Advisors Pty Ltd dated August, 2021, or any amended approved version, be adhered to for the use and development embodied in this development plan and any future planning permit (a condition giving effect to this is required on the latter).
7. Junction Improvement Treatment at corner of Illawong Drive and Briody Drive to be provided by Council.

TORQUAY WEST - DEVELOPMENT PLAN
BRIODY DRIVE

DRG NO.	REV	DATE	DES/DOC	AUTH
306395 UD ID 20	49	21/09/2023	LSH	LSH

spiire

0 25 50 75 100 125m
SCALE 1:5000@A3

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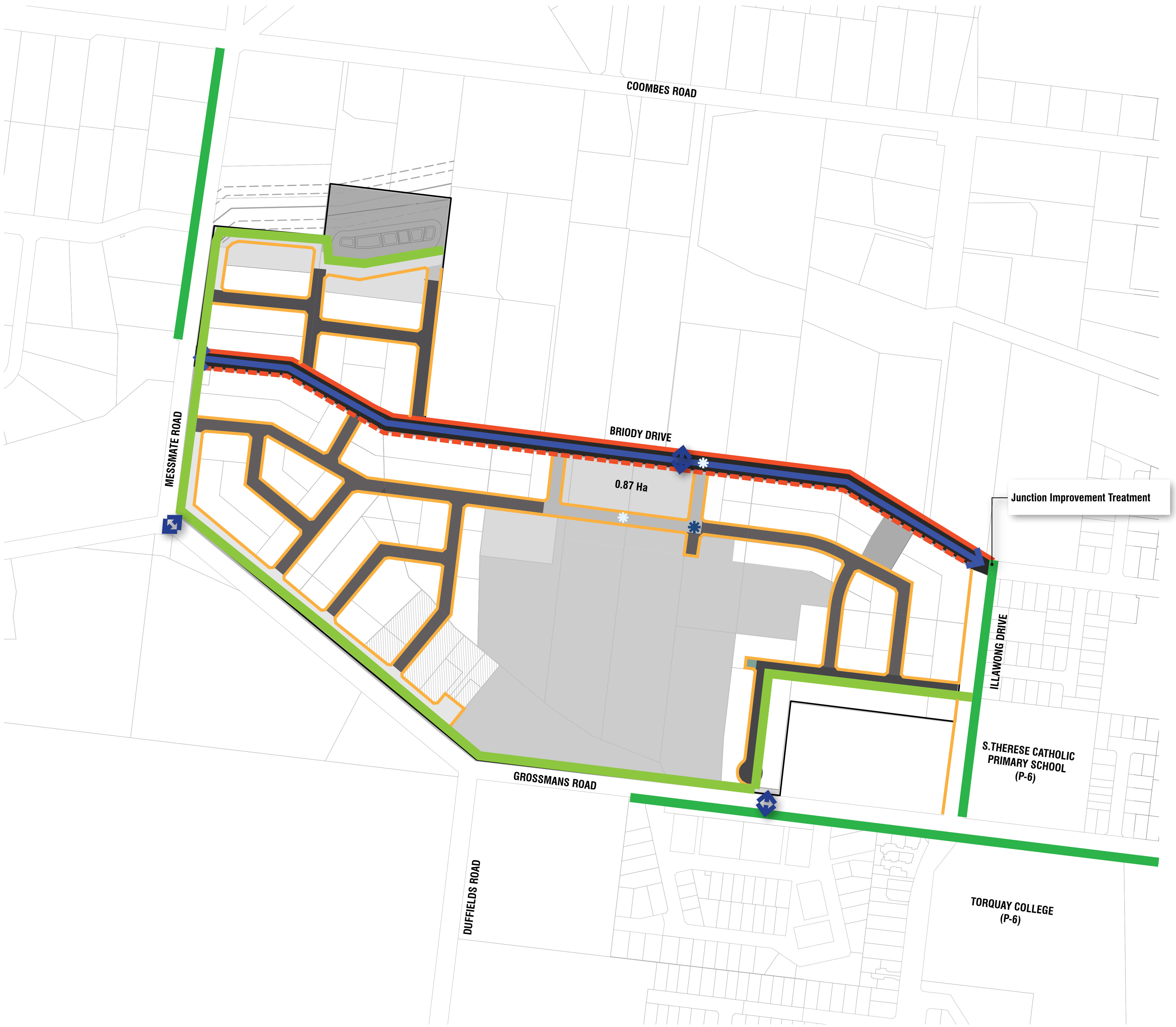

ROAD NETWORK

- BRIODY DRIVE CONNECTOR LEVEL 1 – 20m
  (4.8 m wide nature strip on south side to cater for 2.3 m indented parking bays to be provided in the future)
- ACCESS LEVEL 1 STREET - 16.25m
- ACCESS LEVEL 1 STREET - 16m
- ACCESS LEVEL 1 STREET - 14.5m
- ACCESS LEVEL 1 STREET - 12.5m
- MEWS - 12m
- TRAFFIC MANAGEMENT TREATMENT
- CONSIDER SAFE TRAFFIC AND SPEED CONTROLS AT PLANNING PERMIT STAGE

PATH NETWORK

- EXISTING SHARED PATH
- 2.5m SHARED PATH PROPOSED
- 2m FOOTPATH
- 2m FOOTPATH PROPOSED
- 1.5m FOOTPATH PROPOSED
- PROPOSED INDICATIVE CROSSING LOCATIONS TO EXISTING PATHS

BIKE NETWORK

- BIKE PATH IN EACH DIRECTION ON BRIODY DRIVE

NOTES

1. Briody Drive interim upgrade to include capacity for future indented parking bays on south side.

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TORQUAY WEST - DEVELOPMENT PLAN
BRIODY DRIVE
ROAD NETWORK PLAN - INTERIM

DRG NO.	REV	DATE	DES/DOC	AUTH
306395 UD ID 20	49	21/09/2023	LSH	LSH

spiire

0 25 50 75 100 125m
SCALE 1:5000@A3

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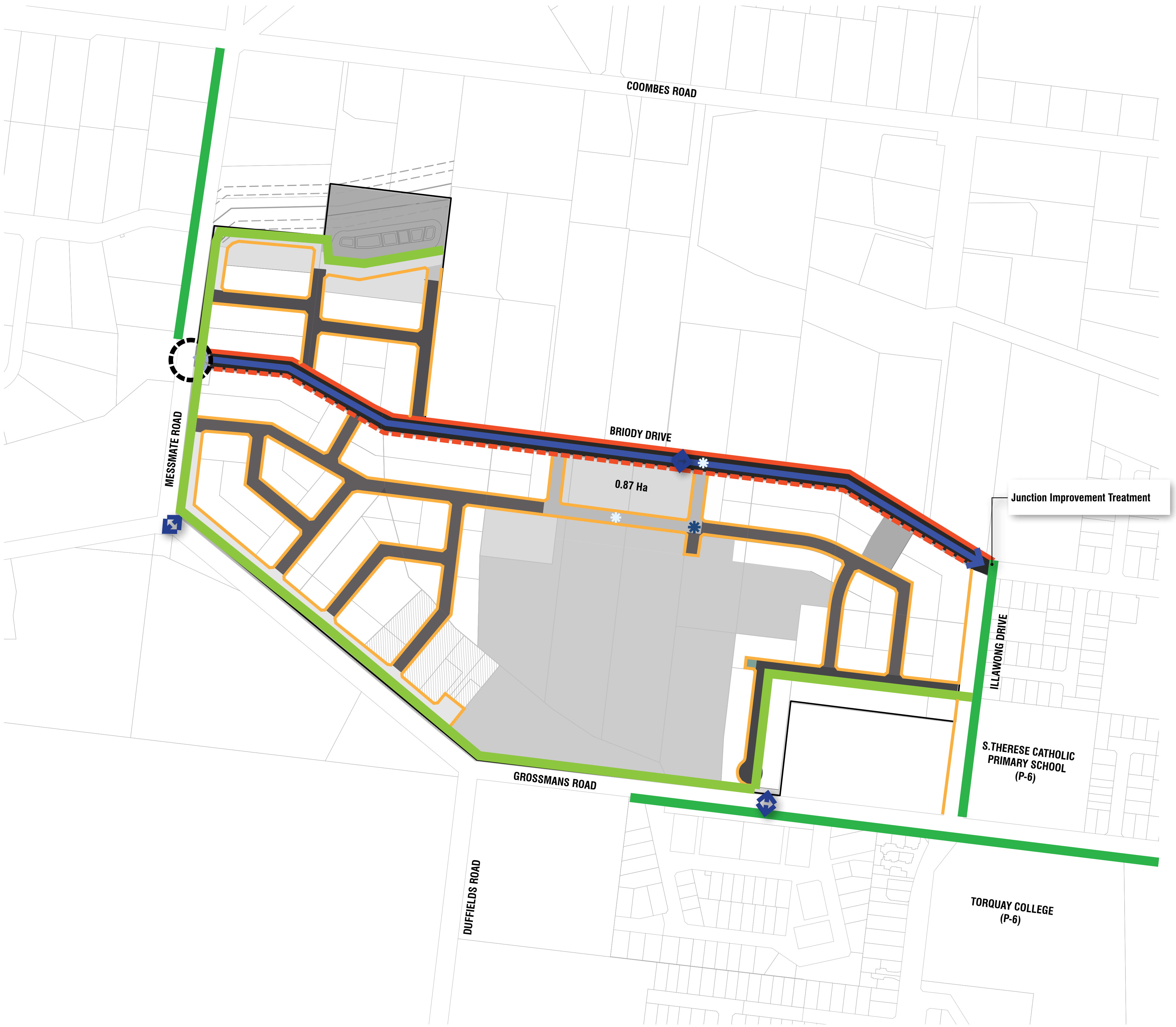

ROAD NETWORK

CONNECTOR LEVEL 1 STREET - 20m

(2.3 m indented parking bays to be incorporated on south side)

ACCESS LEVEL 1 STREET - 16.25m

ACCESS LEVEL 1 STREET - 16m

ACCESS LEVEL 1 STREET - 14.5m

ACCESS LEVEL 1 STREET - 12.5m

MEWS - 12m

TRAFFIC MANAGEMENT TREATMENT

CONSIDER SAFE TRAFFIC AND SPEED CONTROLS AT PLANNING PERMIT STAGE

PATH NETWORK

EXISTING SHARED PATH

2.5m SHARED PATH PROPOSED

2m FOOTPATH

2m FOOTPATH PROPOSED

1.5m FOOTPATH PROPOSED

PROPOSED INDICATIVE CROSSING LOCATIONS TO EXISTING PATHS

BIKE NETWORK

BIKE PATH IN EACH DIRECTION ON BRIODY DRIVE

INTERSECTION

ROUND A BOUT

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NOTES

1. Roundabout treatment to intersection at Briody Drive and Messmate Road to be funded and delivered by Council in the future when traffic volumes necessitate, including land acquisition and vegetation removal.
2. Briody Drive upgrade to Connector Level 1 to be completed by council as traffic volumes necessitate through the provision of indented parking bays on south side.

Refer to the Landscape Master Plan prepared by Tract for cross-sections of roads.

TORQUAY WEST - DEVELOPMENT PLAN

BRIODY DRIVE

ROAD NETWORK PLAN - ULTIMATE

DRG NO.	REV	DATE	DES/DOC	AUTH
306395 UD ID 20	49	21/09/2023	LSH	LSH

spiire

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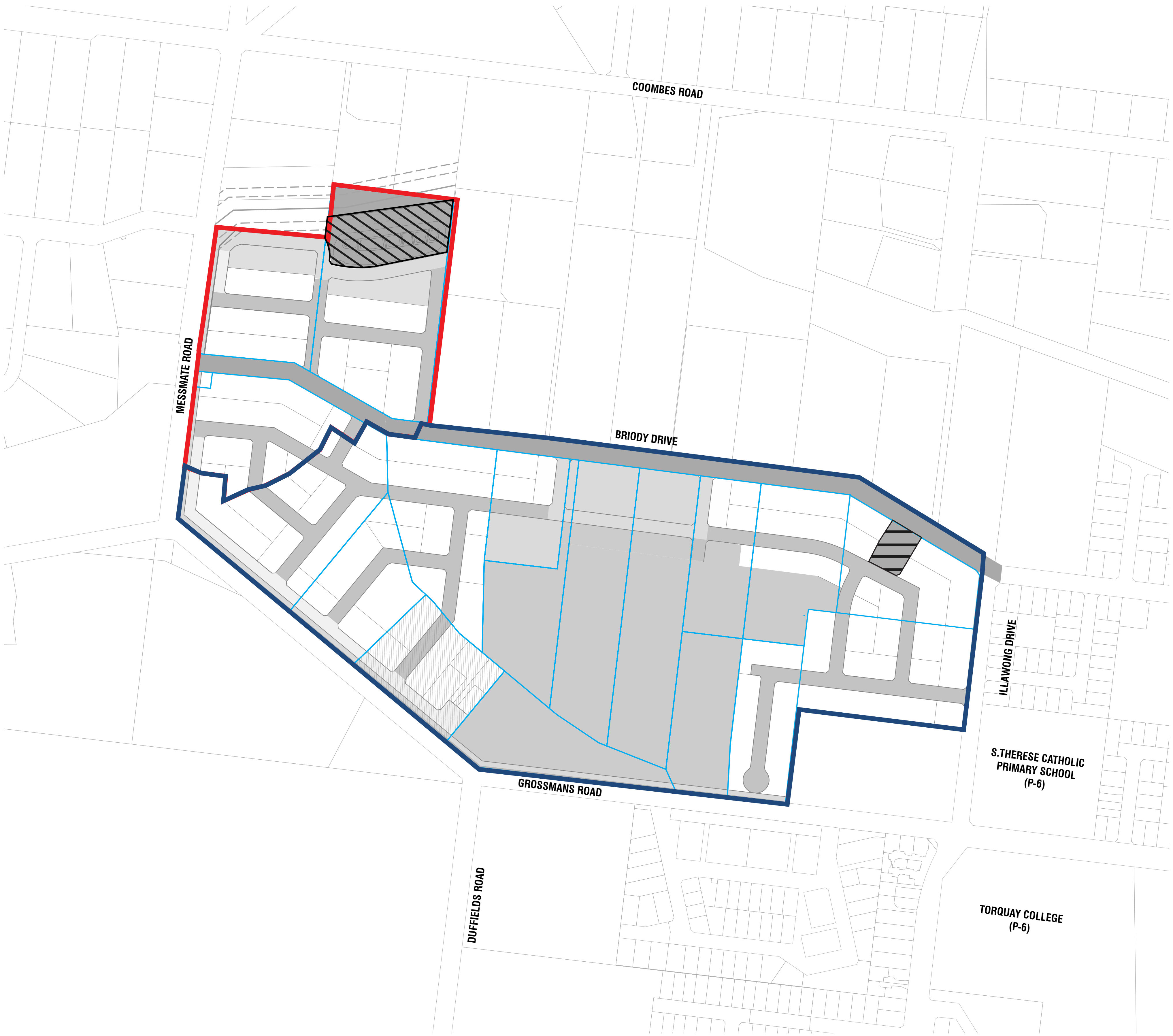

DRAINAGE

- PROPERTY BOUNDARY
- EASTERN PRECINCT
- WESTERN PRECINCT
- BASIN AND/OR WATER QUALITY TREATMENT DEVICE (INCLUDING THE OUTFALL TO DEEP CREEK) PRIOR TO THE FIRST STAGE OF DEVELOPMENT IN EACH CATCHMENT

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NOTES

1. Any future subdivision permit for the first stage of development in each catchment be subject to a condition requiring a preparation of a Waterway Management Plan for Deep Creek (adjacent to the relevant discharge points) which must include details of the existing environmental values, any initial stabilisation and vegetation works, a maintenance regime and the long-term management and maintenance actions that will be required. This plan should be developed by a suitably qualified and experienced professional and also show:
   a) A landscape plan showing the re vegetation of the riparian zone including a species list and proposed density of the plantings. The plantings should be representative of the Ecological Vegetation Class for the site; and
   b) A maintenance plan detailing the establishment, short, medium and long term actions and agencies/developers responsible for each stage.

TORQUAY WEST - DEVELOPMENT PLAN
BRIODY DRIVE
STAGING PLAN

DRG NO.	REV	DATE	DES/DOC	AUTH
306395 UD ID 20	49	21/09/2023	LSH	LSH

spiire

0 25 50 75 100 125m
SCALE 1:5000@A3

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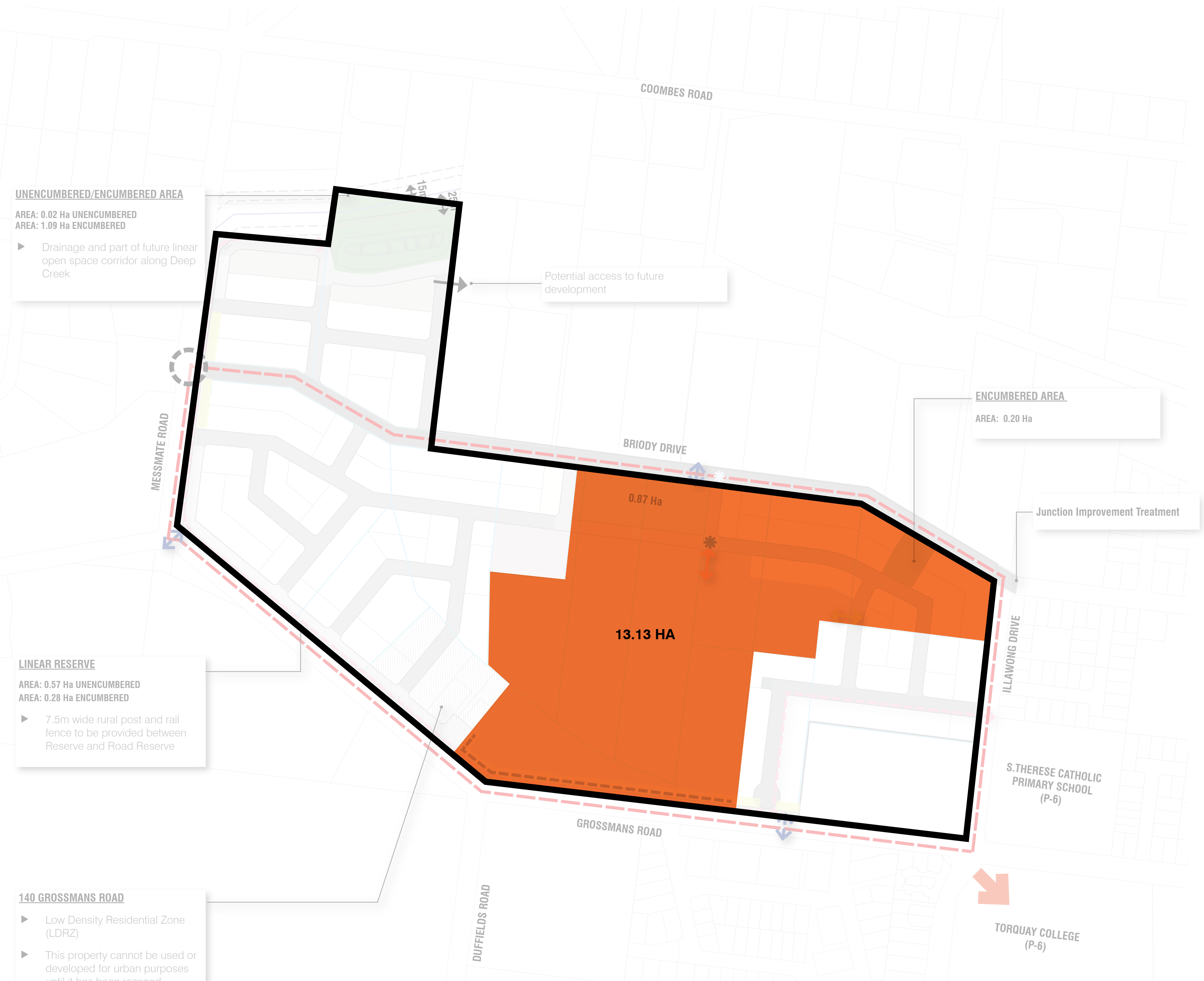

DENSITY

RESIDENTIAL TYPE	DENSITY (LOTS/ NRA)	INDICATIVE LOT RANGE[3]	INDICATIVE NO. OF LOTS	AREA
STANDARD RESIDENTIAL	22.2	350m²-900m²	313	14.23 Ha (net residential)
HIGHER DENSITY RESIDENTIAL	28.5	250m²-325m²	17	0.61 Ha (net residential)
MULTI-UNIT SITE	24		11	0.47 Ha (site)
RETIREMENT VILLAGE, RESIDENTIAL AGED CARE AND INDEPENDENT AND ASSISTED LIVING APARTMENTS[1]			231 independent retirement village units 80 residential aged care beds 60 independent & assisted living apartments	8.51 Ha (site)

ROAD NETWORK

- CONNECTOR LEVEL 1 STREET
  (4.8 m wide nature strip on south side to cater for 2.3 m indented parking bays to be provided in the future)
- ACCESS LEVEL 1 STREET - 16.25m
- ACCESS LEVEL 1 STREET - 16m
- ACCESS LEVEL 1 STREET - 14.5m
- ACCESS LEVEL 1 STREET - 12.5m
- MEWS - 12m
- INDICATIVE INTERNAL PRIVATE ROAD WITHIN RETIREMENT VILLAGE AND RESIDENTIAL AGED CARE
- FUTURE ROUND A BOUT
- TRAFFIC MANAGEMENT TREATMENT
- CONSIDER SAFE TRAFFIC AND SPEED CONTROLS AT PLANNING PERMIT STAGE

OPEN SPACE/DRAINAGE

- ENCUMBERED OPEN SPACE
- UNENCUMBERED OPEN SPACE
- CENTRE LINE OF CREEK

OTHER

- POTENTIAL FUTURE BUS ROUTE
- INDICATIVE LOCATION OF SHARED PATH
- DIRECTION TO SCHOOLS AND COMMUNITY FACILITIES
- EXISTING TITLE BOUNDARIES[2]
- FENCE PROVISIONS
- RETIREMENT VILLAGE ENTRY (CARS)
- RETIREMENT VILLAGE ENTRY (PEDESTRIAN AND EMERGENCY ACCESS/EGRESS)
- PROPOSED INDICATIVE CROSSING LOCATIONS TO EXISTING PATHS

NOTES

1. Development on the Retirement Village / Residential Aged Care site will generally be a maximum height of two storeys. However, any three-storey component of the building containing the Residential Aged Care Facility and Independent and Assisted Living Apartments must be setback at least 75 metres from any boundary of the site.
2. Whilst some parcel areas have been surveyed, other parcels have been sourced from data.vic.gov.au. Survey required to determine final areas.
3. Section 1 & 2 uses that are permissible within the zone will be considered on their merits.
4. Upgrade to Briody Drive to Connector level 1 to be undertaken in two stages.
5. Roundabout to be provided by Council at a time in the future as traffic volumes necessitate.
6. All Cultural Heritage Management Conditions in the approved Briody Drive West, Torquay Subdivision and Development Cultural Heritage Management Plan (FP-SR# 16746), authored by Extent Heritage Advisors Pty Ltd dated August, 2021, or any amended approved version, be adhered to for the use and development embodied in this development plan and any future planning permit (a condition giving effect to this is required on the latter).
7. Junction Improvement Treatment at corner of Illawong Drive and Briody Drive to be provided by Council.

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TORQUAY WEST - DEVELOPMENT PLAN
BRIODY DRIVE

DWG NO 306395 UD ID 20 REV 49 DATE /09/2023 DES/DOC LSH LSH

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0 25 50 75 100 125m
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Appendix B SWMS Concept

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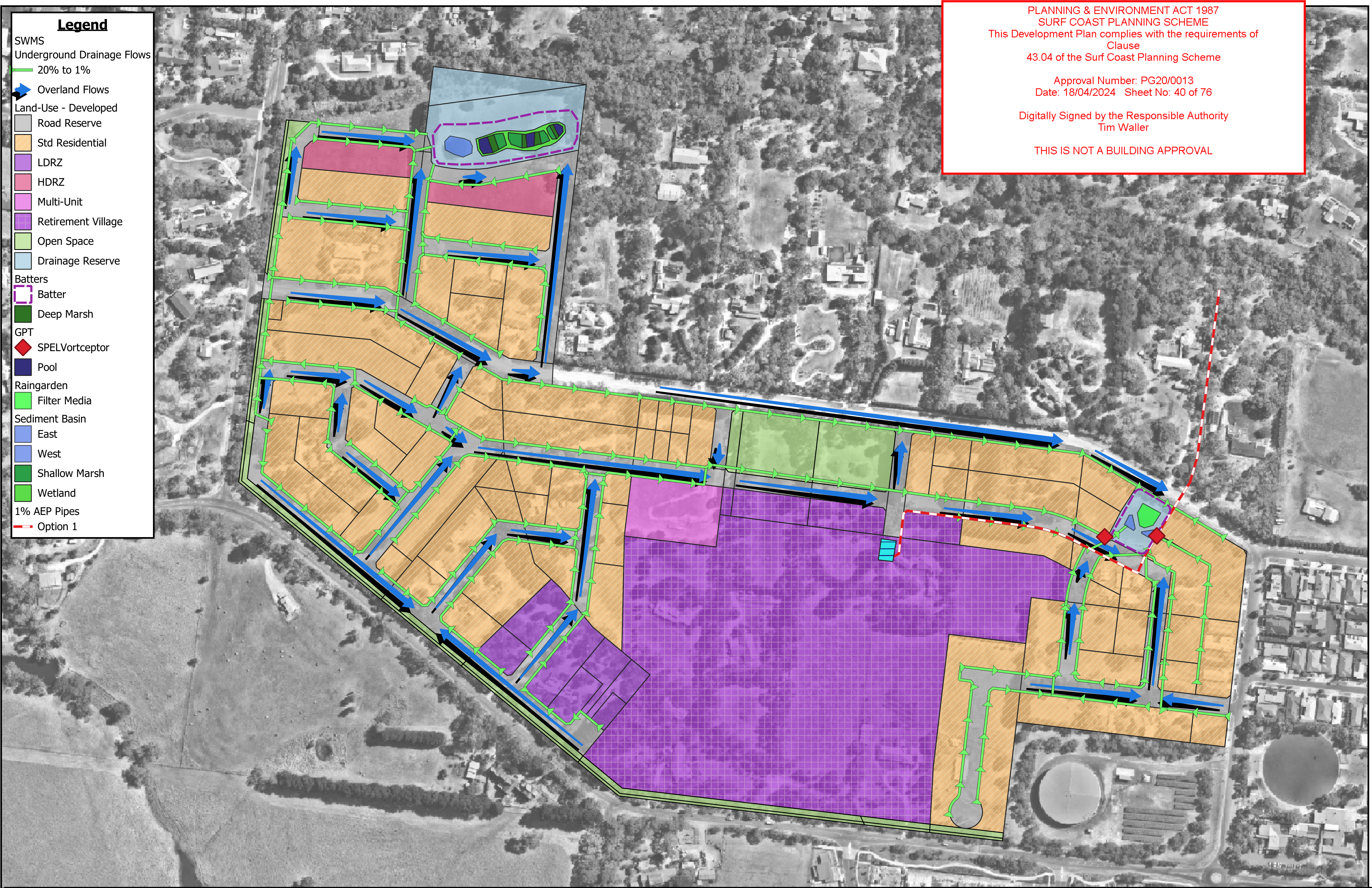

Colliers

On behalf of

Summerset

Project Title	Grossmans Road (96) Torquay	Date	23/10/2023
Project Number	22-0284	Drawn By	C.Cosgriff
Drawing Title	SWMS Concept	Checked By	J.Barker

Appendix C Flood Impact Assessment of Deep Creek (Water Technology)

Deep Creek Flood Impact Assessment

Deep Creek

Briody Drive Pty Ltd

August 2019

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Document Status

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

Project Details

Project Name	Grossmans Road Flood Impact Assessment
Client	Briody Drive Pty Ltd
Client Project Manager	Alexander Brown (Spiire)
Water Technology Project Manager	Johanna Theilemann
Water Technology Project Director	Ben Hughes
Authors	Johanna Theilemann
Document Number	20010016_DeepCree-Briody Development Torquay

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ABN 60 093 377 283

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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.

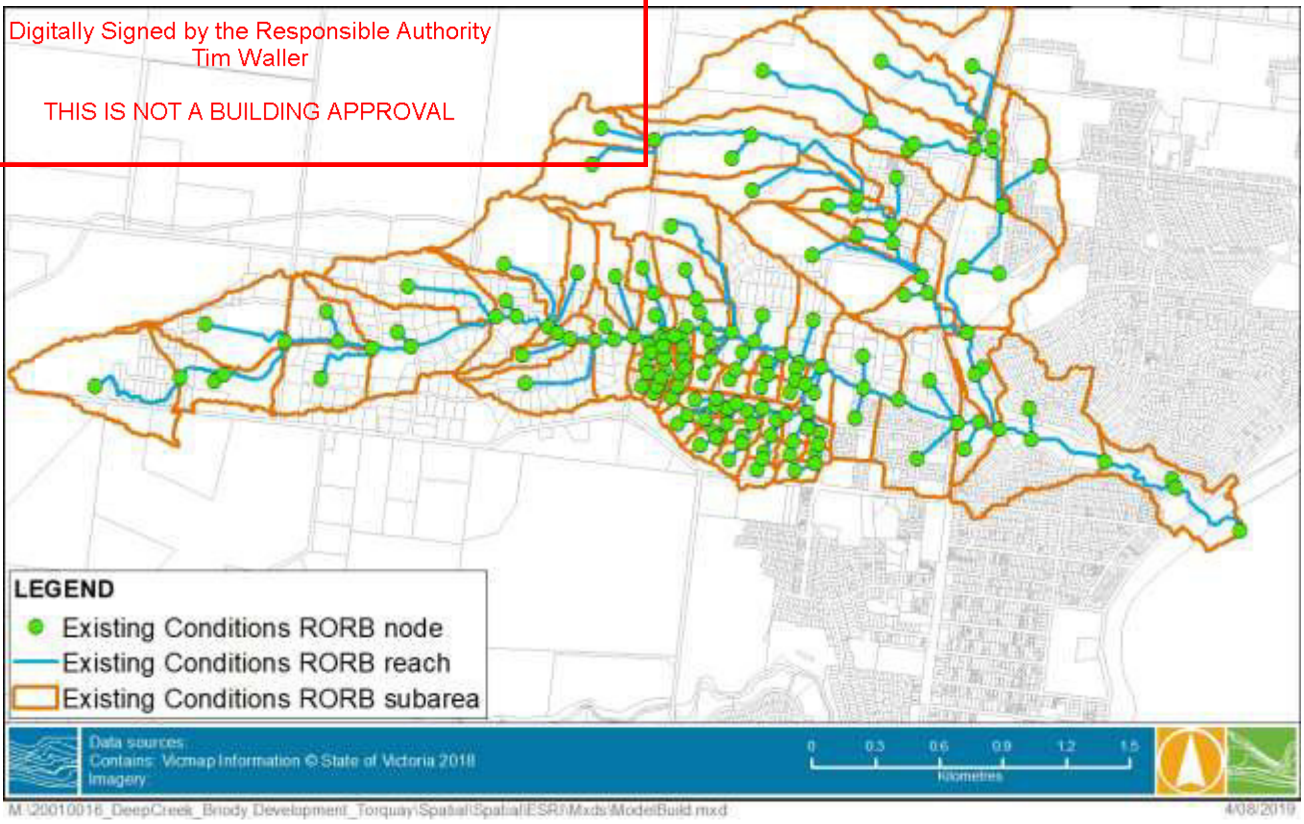

FIGURE 2-1 RORB MODEL SCHEMATISATION

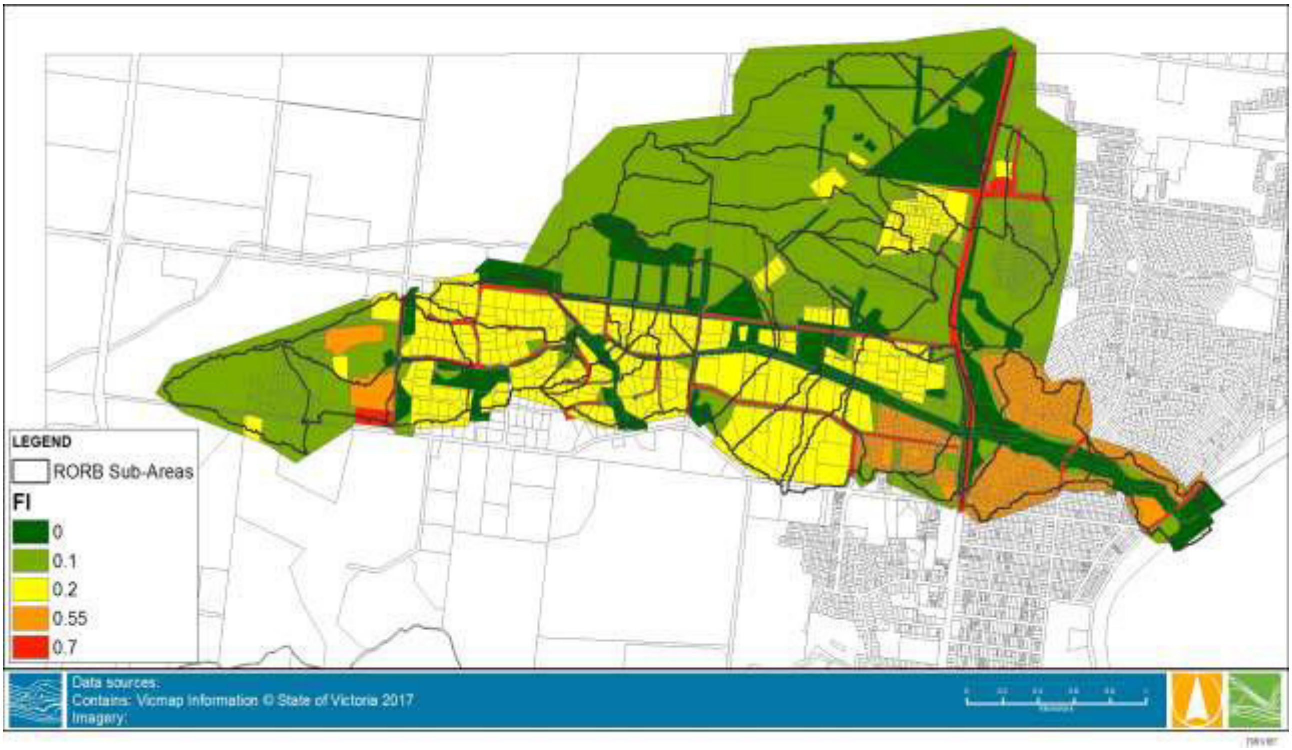

FIGURE 2-2 FRACTION IMPERVIOUS DISTRIBUTION IN THE DEEP CREEK CATCHMENT

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

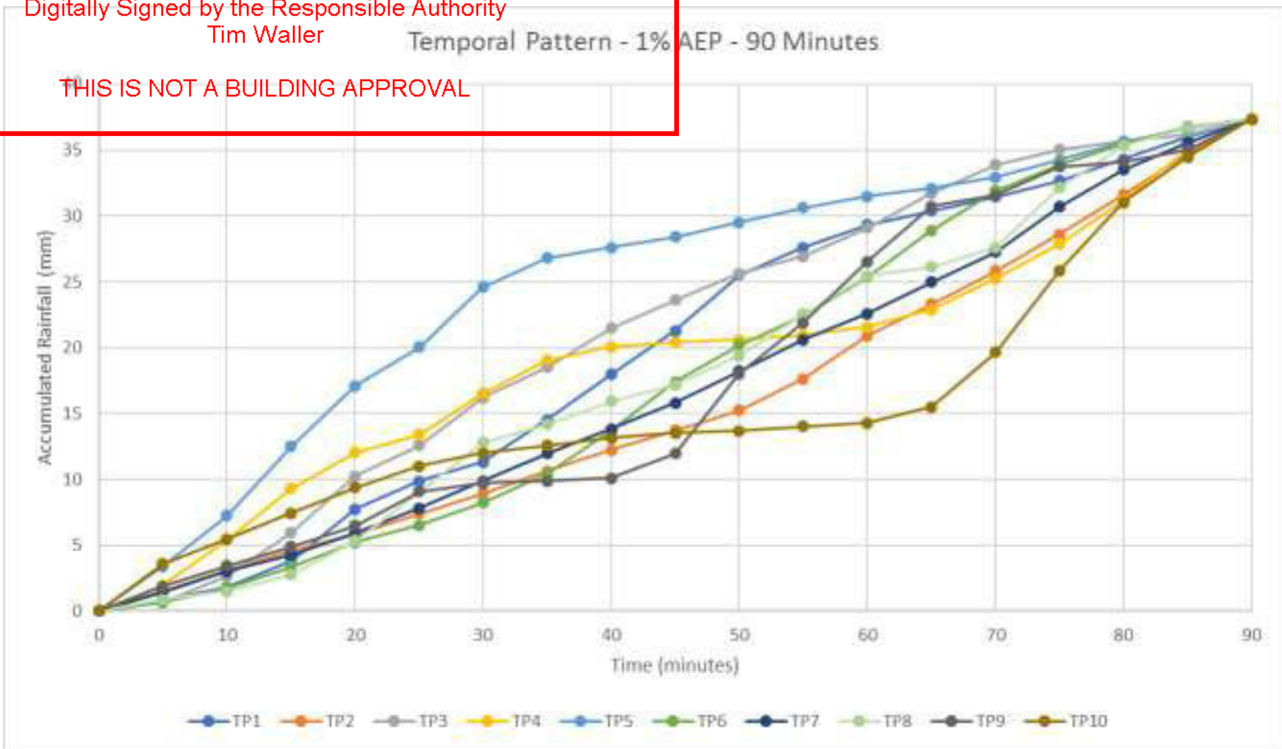

FIGURE 2-3 TEMPORAL PATTERN VARIATION

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

---

[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	Kc
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)

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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.

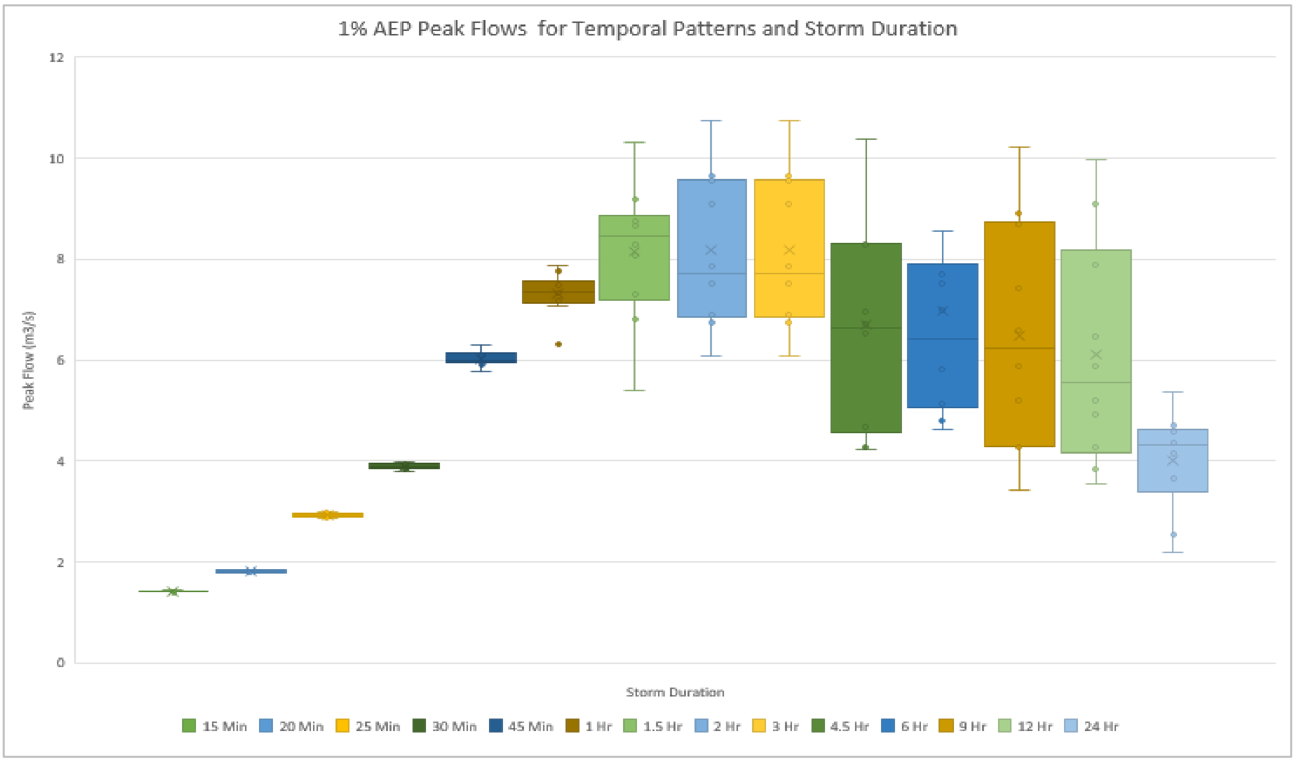

FIGURE 2-4 TEMPORAL PATTERN AND PEAK FLOWS

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.

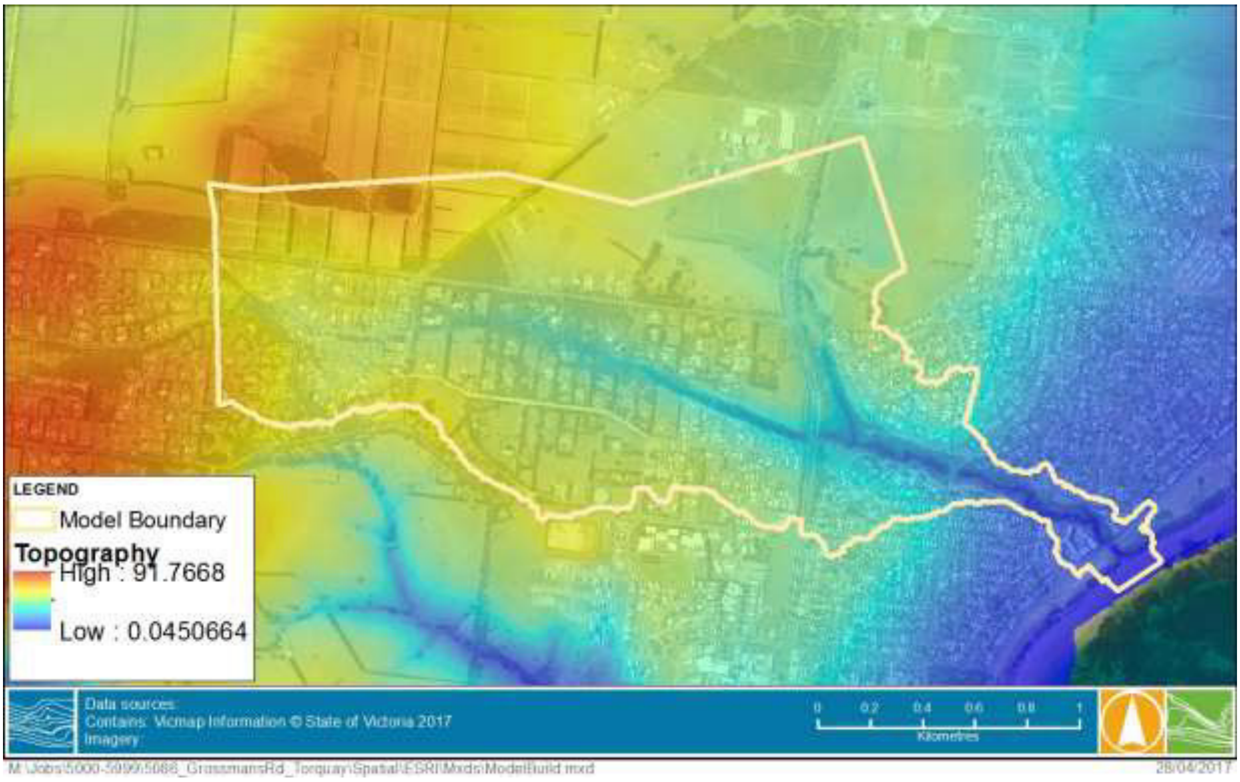

FIGURE 3-1 DEEP CREEK– TOPOGRAPHY

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

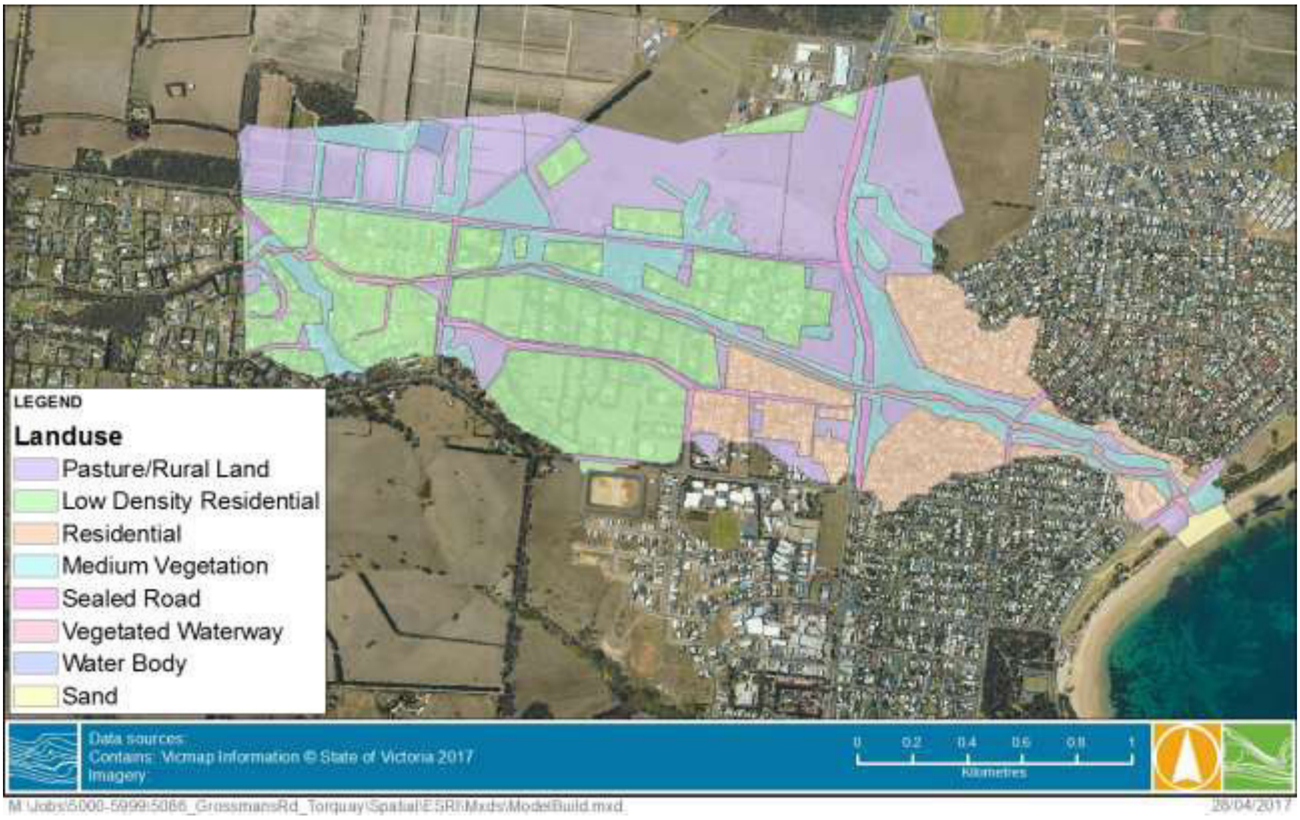

FIGURE 3-2 DEEP CREEK TUFLOW MODEL MANNING'S ROUGHNESS

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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.

FIGURE 3-4 DEEP CREEK 1% AEP FLOOD DEPTH

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.

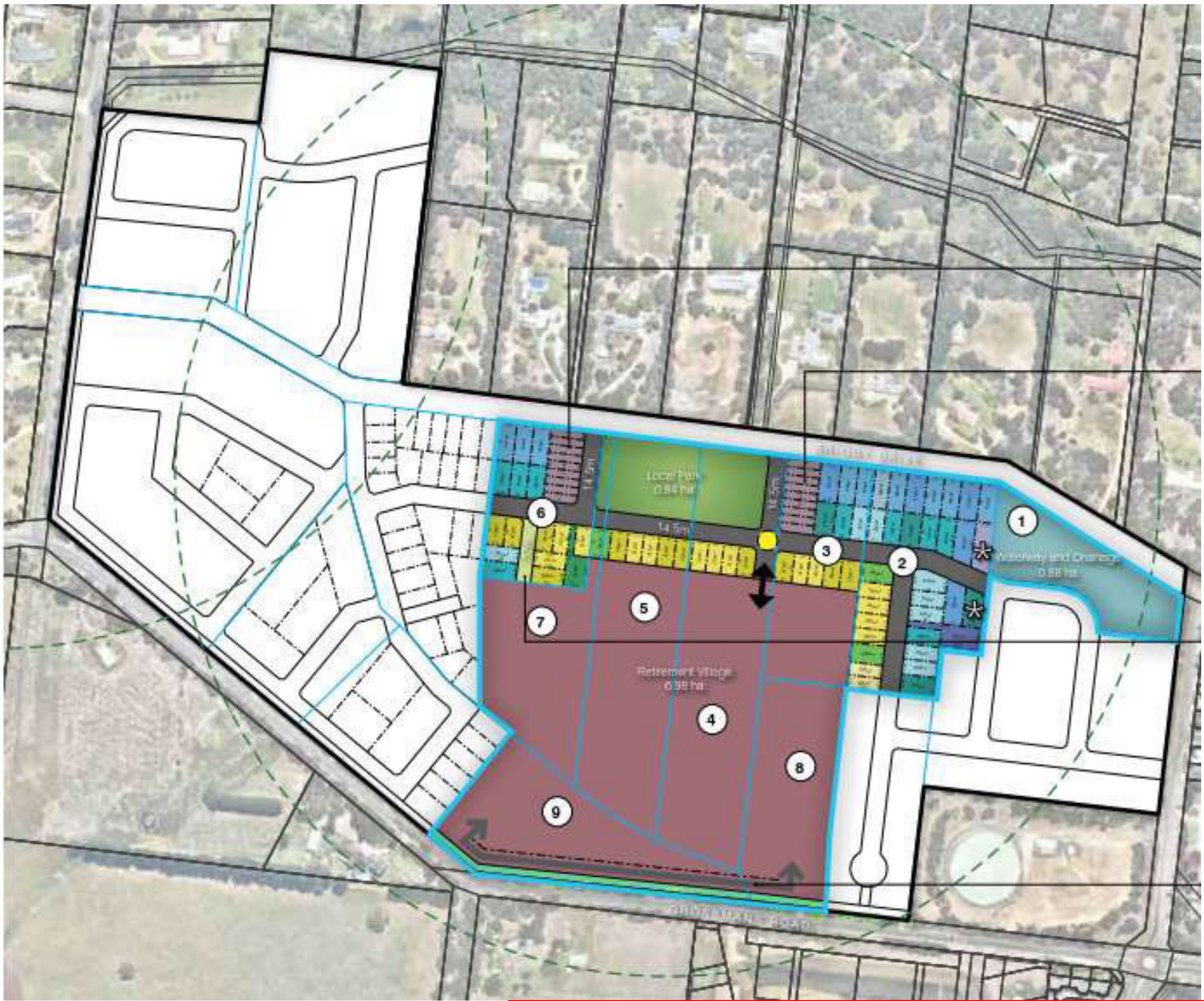

FIGURE 4-1 1% AEP FLOOD EXTENT SHOWING PROPOSED DEVELOPMENT LAYOUT

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.

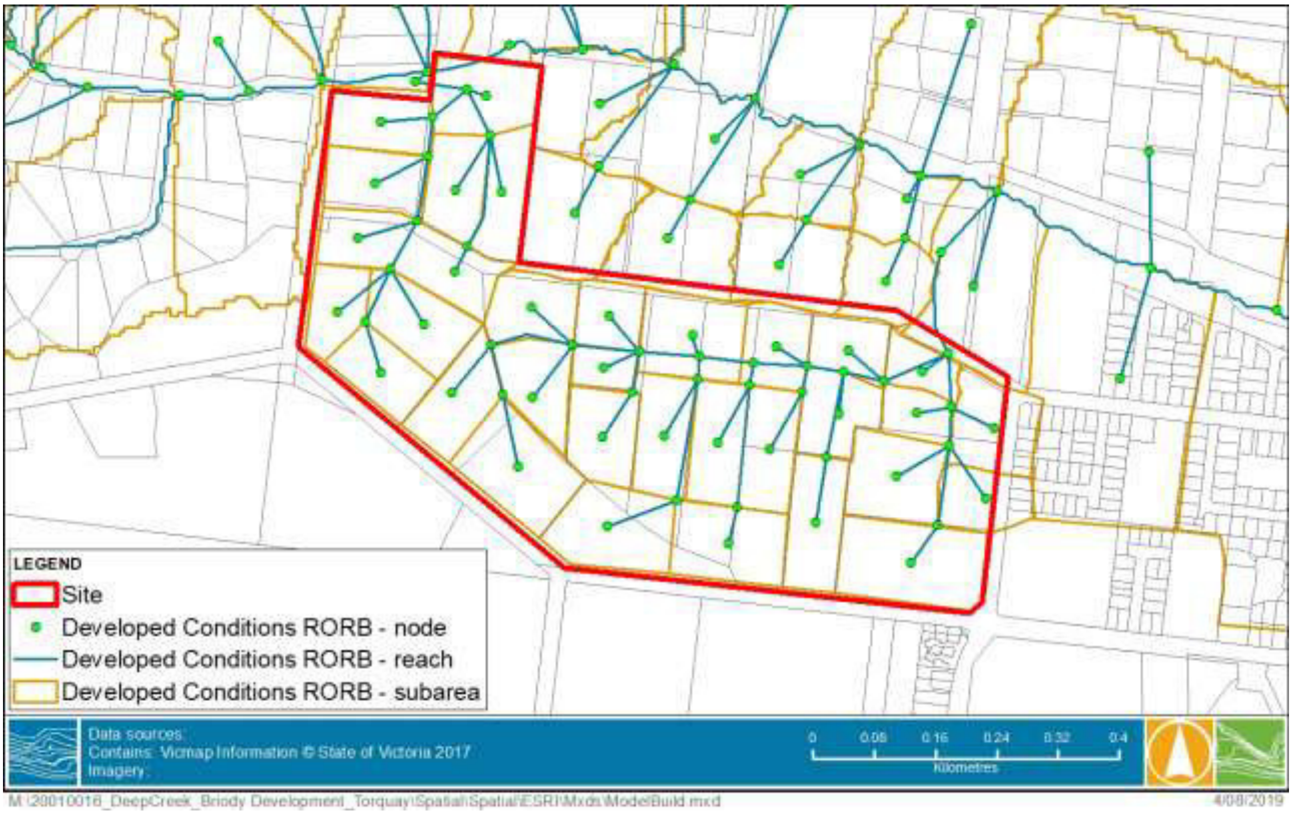

FIGURE 4-2 RORB- DEVELOPED CATCHMENT LAYOUT

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.

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

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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

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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).

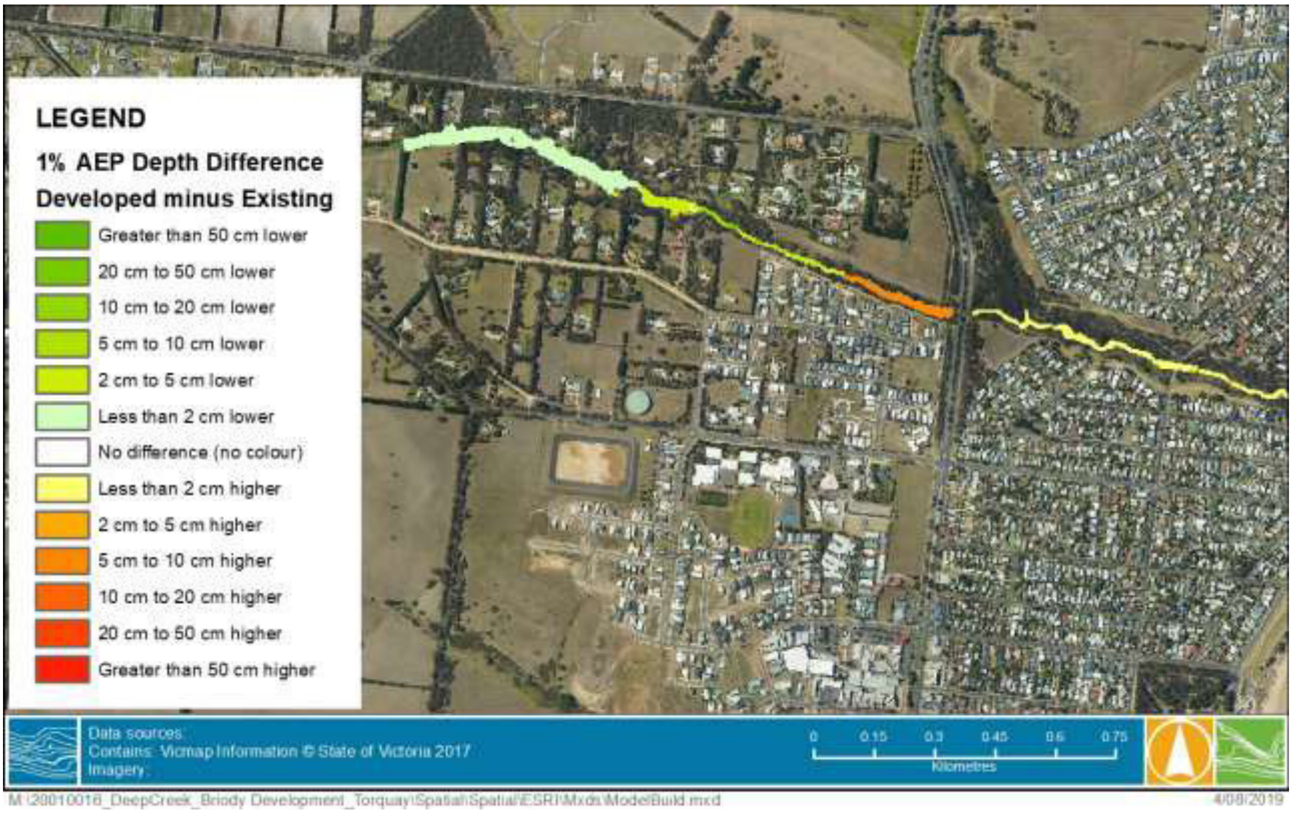

FIGURE 4-10 1% AEP FLOOD DEPTH DIFFERENCE DEVELOPED MINUS EXISTING

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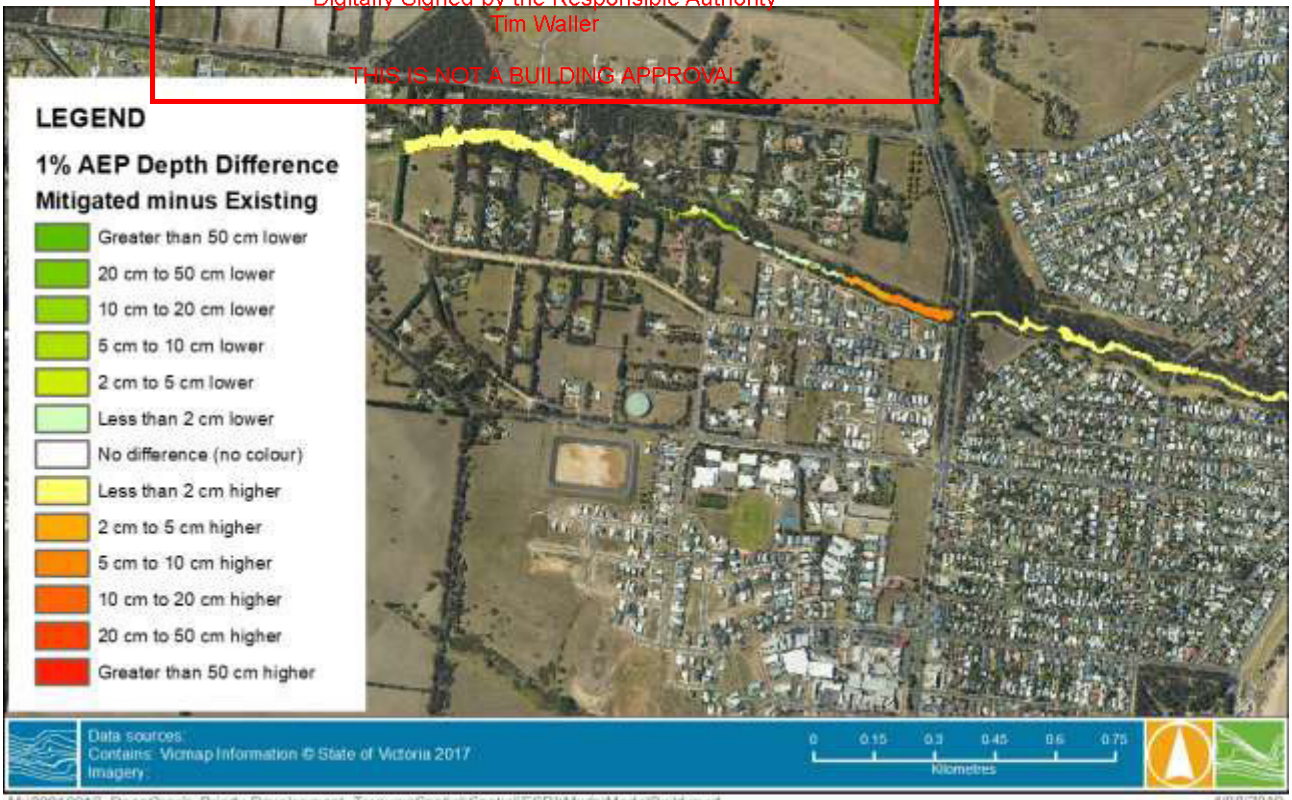

FIGURE 4-11 1% AEP FLOOD DEPTH DIFFERENCE MITIGATED MINUS DEVELOPED

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

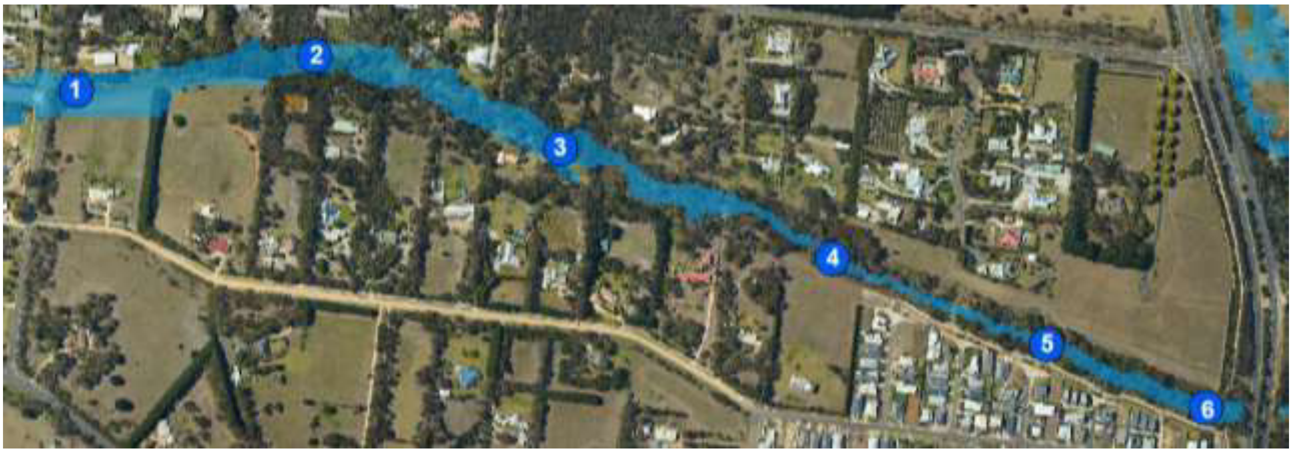

FIGURE 4-12 FLOOD LEVEL POINT LOCATIONS

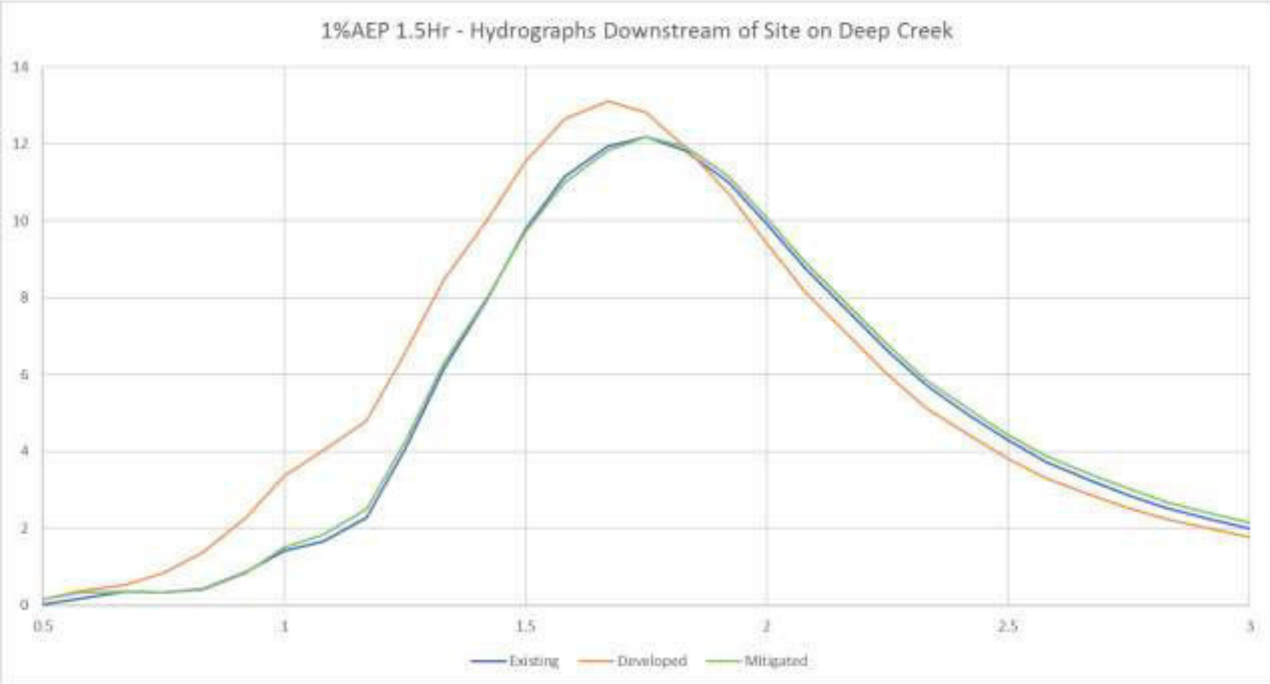

FIGURE 4-13 1%AEP HYDROGRAPH DOWNSTREAM OF SITE ON DEEP CREEK

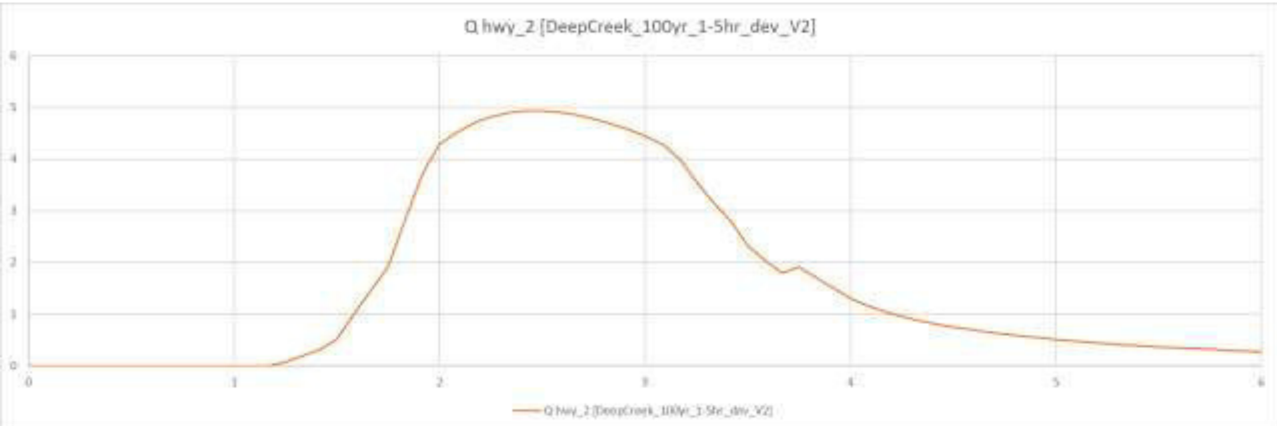

FIGURE 4-14 SURFCOAST HIGHWAY FLOW HYDROGRAPH – 1% AEP 1.5HR

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