Engineering | Surveying | Planning

321–399 IBBOTSON STREET, ST LEONARDS

EXISTING FLOOD STUDY AND SITE STORMWATER MANAGEMENT PLAN

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Project Name	321-399 Ibbotson Street, St Leonards
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The information provided represents the best estimate based on currently available information as described; this information is subject to change as new information becomes available as further studies or significant alterations to the proposed development are carried out.

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EXECUTIVE SUMMARY

TGM Group has been engaged by the Costa Property Nine Pty Ltd to submit a combined Planning Scheme Amendment Application under Section 96A of the Planning and Environment Act 1987 for the rezoning and staged multi-lot subdivision of land at 321-399 Ibbotson Street, St Leonards.

An application is being made to amend the Greater Geelong Planning Scheme and enable a rezoning of the subject land from the Farming Zone to the General Residential Zone Schedule. The application also seeks approval for a staged multi-lot subdivision which will create approximately 483 conventional residential lots.

The following report consists of a detailed existing flood analysis and Site Stormwater Management Plan (SSMP). The Flood Impact Assessment (FIA) will be detailed in the next revision following completion of flood mitigation designs for the land south of the existing waterway. This report has been prepared to inform and support the application process and incorporates current best practice techniques and the latest industry standards.

Study Objectives

The purpose of this study is to –

- Define the exiting regional flood characteristics of the St Leonards Lake catchment area.
- Develop a site stormwater management plan for the proposed development, and
- Undertake a flood impact assessment. *[not reported in this document]*

The objectives of this study focus on three principle sections –

Existing Flood Study

The study will provide an understanding of flooding associated with the existing waterway, the availability of developable land and identification of stormwater constraints. The overriding objectives of the flood investigation are to –

- Establish the existing flood characteristics for the site, and
- Identify the associated flood risk.

Flood risk will be defined by the following safety and hazard criteria

- 1% AEP flood extent,
- Depth profile,
- Velocity profile, and
- Velocity x Depth profile.

The flood risk assessment will inform the plan of subdivision and staging within the site.

Site Stormwater Management Plan (SSMP)

The objective of the stormwater management plan is to meet the conditions and requirements, set out by the City of Greater Geelong (COGG) Council and the Corangamite Catchment Management Authority (CCMA) in the planning application for stormwater management. Stormwater mitigation systems are designed to ensure that stormwater quality and quantity targets are met. The stormwater targets are:

1. Best Practice reductions for Water Quality
   - 80% reduction in Suspended solids (SS),
   - 45% reduction in total nitrogen (TN),
   - 45% reduction in total phosphorus (TP),
   - 70% reduction in gross pollutants (GP).

2. No-worsening stormwater peak discharges
   - Ensure 1% AEP pre-development flows are maintained.

Flood Impact Assessment (FIA)

It is imperative that development of the Ibbotson Street site does not have an adverse impact on the surrounding areas during the 1% AEP regional flood event. The impact of the development will be assessed against the following flood characteristics:

- Flood extents;
- Flood storage;
- Velocities and flow safety and hazard characteristics;
- Duration; and
- Cumulative flooding impact.

Note: The FIA is yet to be completed and will not be addressed in this report.

Study Methodology

This study was undertaken using detailed hydrologic and hydraulic assessment incorporating an integrated systems approach to analyse the performance of a range of stormwater and water cycle management options. This type of analysis is dependent on detailed inputs including topography, climate, geology, hydrology, stormwater quality and sound urban design principles.

The study methodology focused on the following core elements for a robust hydrological and hydraulic analysis:

- Topography and exiting Infrastructure;
- Climatic Processes;
- Geology;
- Hydrology;
- Design Event Modelling;
- Stormwater Quality; and
- Hydraulics.

Study Results

Existing Flooding

The existing 1% AEP flood for St Leonards Lake catchment was established and approved by COGG as an accurate representation of the predicted flood characteristics.

Figure A: Existing 1% AEP Flood

The flood risk assessment identified that the development site is affected by the 1% AEP flood event. The site is divided into two (2) sectors (north and south) by the existing waterway. Land north of the waterway is not subject to flooding outside the waterway corridor and can be developed without the requirement for regional flood mitigation.

Land south of the waterway is subject to inundation under existing conditions. Flood mitigation and a detailed flood impact assessment will be required to reclaim developable land.

Flood risks, associated with safety hazards, are relatively low as the '*unsafe'* characteristics are contained within the existing waterway.

Site Stormwater Management

The proposed stormwater management strategy and design assumed full site development north and south of the existing waterway. The SSMP was able to demonstrate the ability to achieve the study objectives for:

Stormwater quality

Table A: Stormwater quality treatment efficiency

Treatment Train Efficiency			
Criteria	**Sources**	**Residual Load**	**Reduction (%)**
Flow (ML/yr)	102	87.4	14.2
Total Suspended Solids (kg/yr)	19400	3640	81.2
Total Phosphorus (kg/yr)	41	12	70.8
Total Nitrogen (kg/yr)	293	147	49.7
Gross Pollutants (kg/yr)	4440	929	93.4

Stormwater discharge

Table B: Ibbotson Street stormwater peak discharge objective

Base Case (Existing Conditions)	**Developed Site** (Mitigated)	
2.168 m³/s	**1.906 m³/s**	**1.662 m³/s**
9 hour storm duration	*9 hour storm duration*	*20 minute storm duration*

Study Conclusion

The proposed development site is separated into a northern and southern catchment by the existing waterway. Northern catchment can be developed without the requirement for flood mitigation as the proposed development does not encroach into the 1% AEP flood extent. The southern catchment is subject to flooding during the 1% AEP flood event and this area will require further assessment to quantify developable land area.

The nominated site stormwater management design proves that stormwater runoff generated within the fully developed 321-399 Ibbotson Street site in St Leonards can be mitigated to meet ‘best-practice’ objectives in water quality and a ‘no-worsening’ of peak discharges from the site.

The analysis documented in this report has demonstrated that the proposed development can be constructed and meet the requirements and objectives for stormwater management and supports development of the land north of the existing waterway, however, flood mitigation design and analysis of flood impact and safety for the developed site south of the waterway still needs to be finalised. This analysis will be provided as part of a revised document including a detailed flood impact assessment when at hand.

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1. BACKGROUND

City of Greater Geelong (COGG) has identified two prospective Growth Areas within the Geelong regional town of St Leonards. This report is focused on the site identified as Growth Area 1.

The site is located at 321-399 Ibbotson Street, St Leonards and is a 39 hectare land parcel currently used for agricultural purposes. The site is located approximately 33 km east of the Geelong CBD. The site location is depicted in Figure 1.1, below.

Figure 1.1: Growth Area 1 site location, St. Leonards

TGM Group has been engaged by the Costa Property Nine Pty Ltd to submit a combined Planning Scheme Amendment Application under Section 96A of the Planning and Environment Act 1987 for the rezoning and staged multi-lot subdivision of land at 321-399 Ibbotson Street, St Leonards.

The subject land is currently zoned as Farming Zone under the Greater Geelong Planning Scheme and is recommended for residential development under the 2014 St Leonards Structure Plan.

An application is being made to amend the Greater Geelong Planning Scheme and enable a rezone of the subject land from the Farming Zone to the General Residential Zone Schedule 1. The application also seeks approval for a staged multi-lot subdivision which will create approximately 483 conventional residential lots.

This site stormwater management plan (SSMP) has been prepared to inform and support this process and incorporates current best practice techniques and the latest industry guidelines.

1.1 Site Description

The site is located on a high point overlooking the town of St Leonards. The northern section of the site has an elevation of 10.5 m AHD. The site slopes in a southerly direction, towards a designated waterway at approximately 4 m AHD that traverses the southern extent of the site.

The site contours can be seen in Figure 1.2.

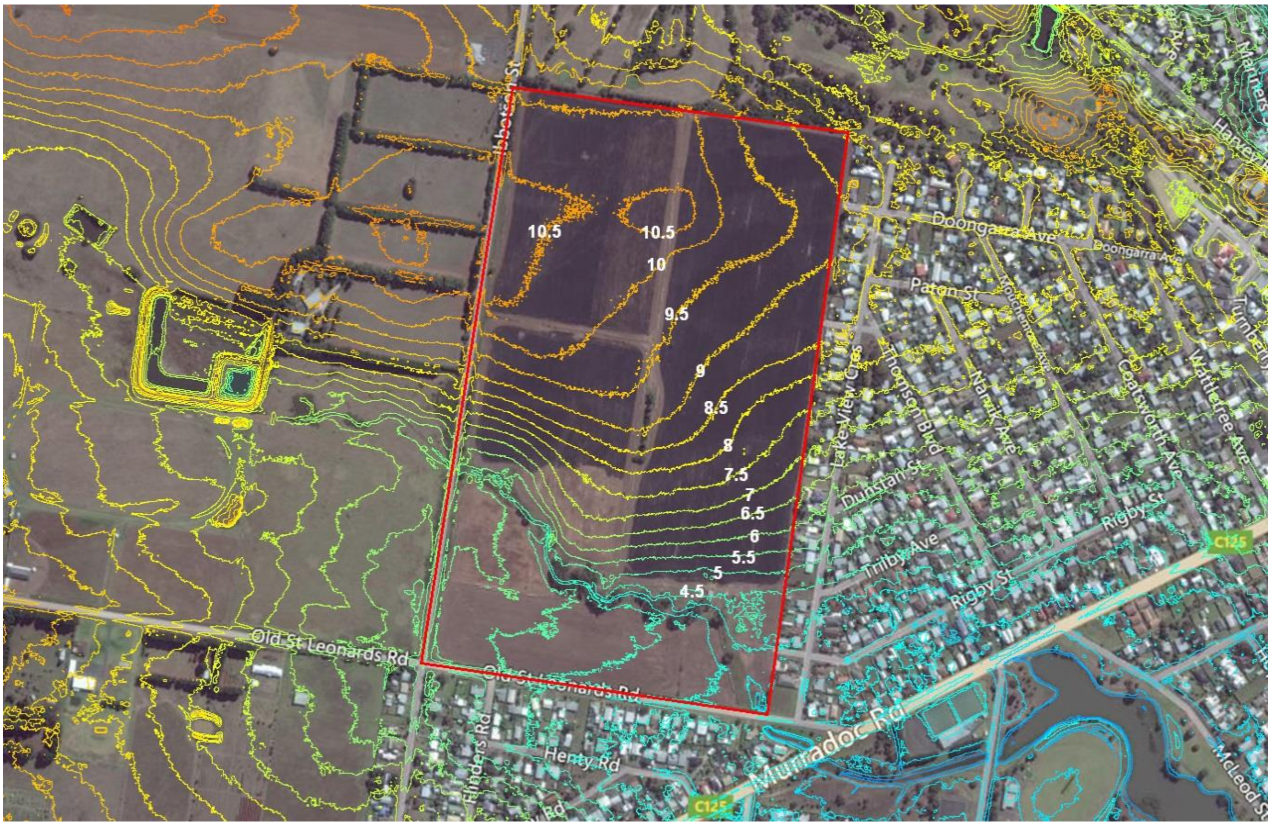


Figure 1.2: Ibbotson Street – site contours

1.2 Site Stormwater Catchments

The site has three distinct catchments, defined by the northern ridge and the southern waterway. Existing topography allows stormwater runoff to discharge from the site via the waterway and across the northern, eastern and southern boundaries as uncontrolled overland flow; this is represented in Figure 1.3.

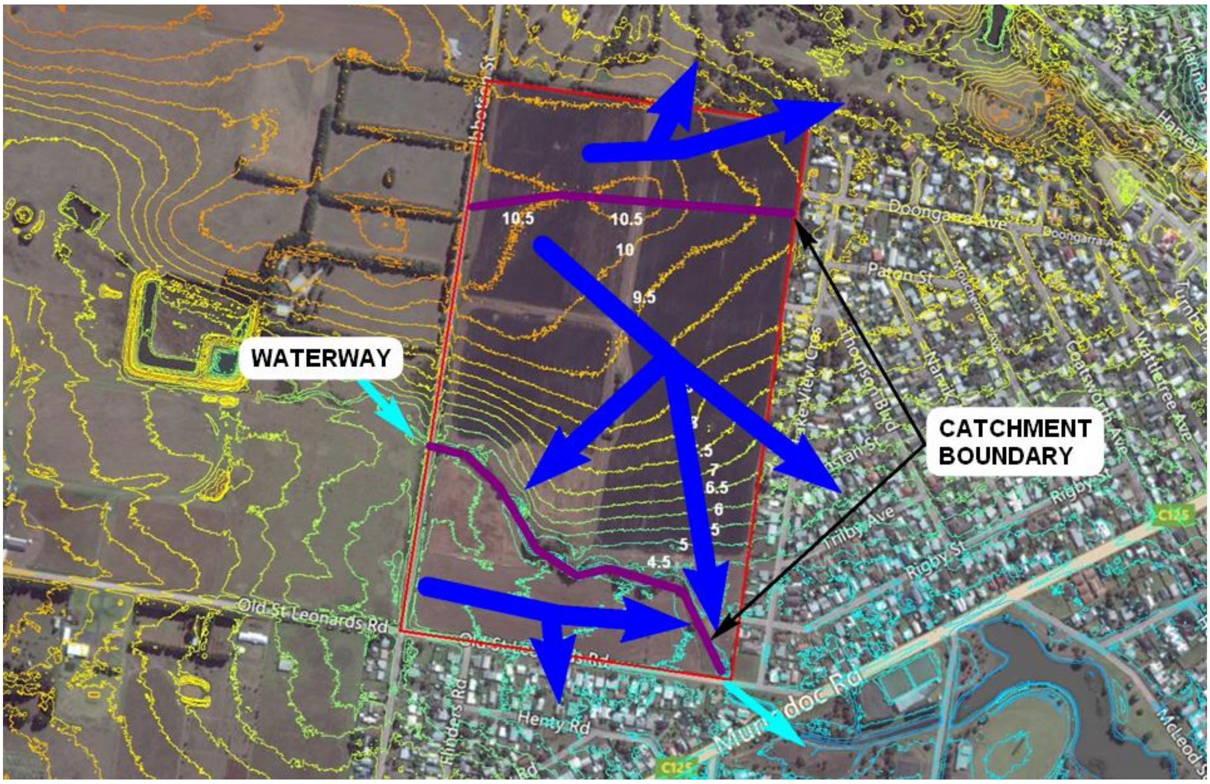


Figure 1.3: Ibbotson Street site stormwater catchments and flowpaths

The site catchment areas are shown in Table 1-1.

Table 1-1: Existing Ibbotson Street site stormwater catchment area

Catchment	Area (ha)
Northern Catchment	6.84
Central Catchment	25.34
Southern Catchment	7.45

These catchments will be broken down into smaller sub-catchments, reflecting the proposed design catchments and their point of discharge from the site to allow finer definition of the local hydrology and quantification of target design flows.

Stormwater runoff generated in catchments upstream of the western boundary, enter the site predominantly via the waterway.

1.3 Regional Stormwater Catchments

To ensure accurate definition of the site's hydrology and understand the volume of stormwater runoff impacting the site, the contributing regional catchment area was identified.

The total regional catchments of the study area contributing to St Leonards Lake were identified using topographical data and DEPI defined HY_Watercourse data and verified against the City of Greater Geelong catchment maps – Catchment 81, 83 & 84; and updated CCMA catchments.

It has been determined that the total hydraulic catchment area for Growth Area 1 (including site) is approximately **1,437 hectares.** The hydraulic catchment consists of sub-catchments that generate local runoff; streamflow and cross-catchment flow from the upstream regional catchment.

A total regional stormwater catchment area of 940 hectares generates stormwater flows that will pass through the Ibbotson Street site via a designated waterway.

Cross-catchment flows generated within a 231 hectare rural and urban catchment area provide a contribution to the total stormwater flow impacting the site during larger storm events.

The catchment delineation is depicted in Figure 1.4.

Figure 1.4: Ibbotson Street site (Growth Area 1) – hydraulic catchments

The upper catchment is a principally cleared agricultural farmland with the downstream catchments medium density residential. Sub-catchment delineation and definition of the contributing catchment area analysed in this study is discussed in more detail in Section 4.

A downstream catchment area of 227 hectare generates stormwater runoff that may have a hydraulic impact on flows discharging the site. These catchments contribute runoff to St Leonards Lake which defines the downstream boundary condition for site discharges.

1.3.1 St Leonards Lake

St Leonards Lake is a man made permanent water body nestled within the St Leonards urban area. The Lakes downstream boundary is controlled by a dam weir with a spillway R.L of 1.792 m AHD (±1mm), see Figure 1.5.

Figure 1.5: St Leonards Lake Weir – Spillway (1.792 m AHD) and Low Flow arrangement

The weir structure enables the Lake to maintain a higher top water level and prevent sea water incursion from Port Phillip Bay, which is situated on the other side of Bluff Road, approximately 120 m to the east.

Stormwater runoff, from the site and upstream catchments, enters St Leonards Lake via the waterway drainage channel passing beneath Old St Leonards, Murradoc Road and Cole Street.

The TWL of St Leonards Lake is driven by the volume and rate of inflows and the controlled outflow weir arrangement. During high rainfall events, it is anticipated that the inflows to St Leonards Lake will exceed the outflow capacity of the weir, causing backflow effects through the system.

To ensure an accurate hydraulic assessment was undertaken for Growth Area 1, TGM extended the hydraulic model to include the catchment area contributing to St Leonards Lake. The entire catchment area analysed in this study is depicted in Figure 1.6.

1.3.2 Existing Drainage

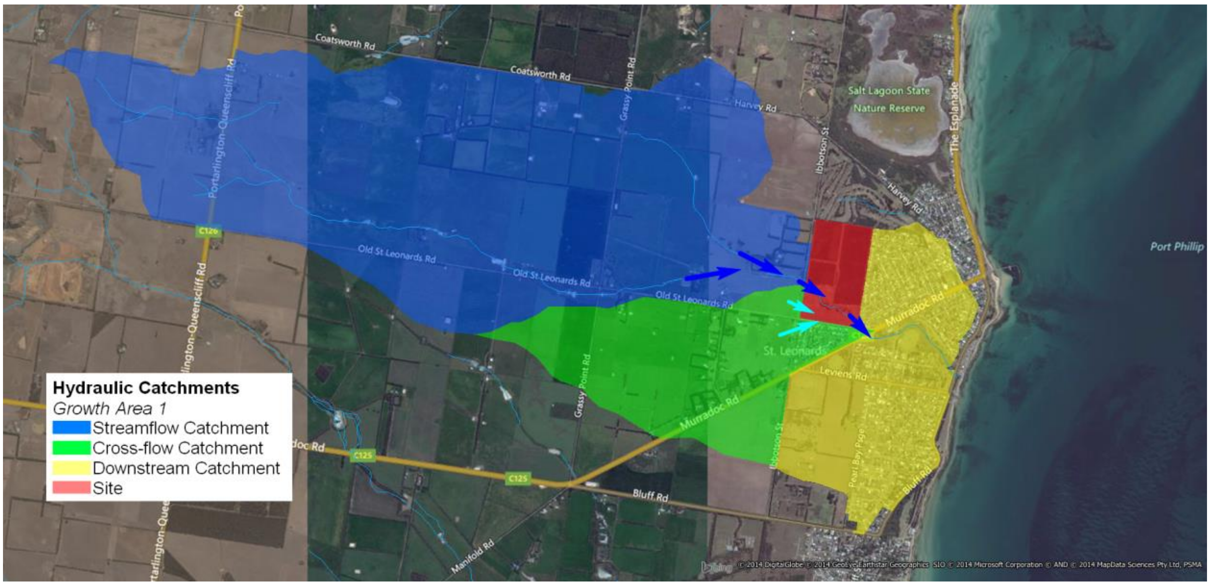


Figure 1.6: St Leonards Lake – contributing catchment plan

St Leonards Lake catchment exhibits a predominantly agricultural land use that constitutes nearly 90% of the total catchment area; the remaining area consists of urban development in the form of traditional detached housing. Land use distribution can be clearly seen in Figure 1.4 and 1.6 above.

1.4 Developed Site

The preferred development plan proposes creation of 483 medium-density housing lots within the 39 hectare site. The preferred development plan is shown in Figure 1.7.

Figure 1.7: Ibbotson Street, Subdivision Master Plan Layout

The preferred development plan shown in Figure 1.7 shows that the majority of the development (85%) is located north of the waterway alignment. Development of the southern section of the site, indicated by the 'greyed' area in Figure 1.7, is dependent upon further detailed hydraulic analysis to identify and mitigate any adverse impact on regional flood characteristics.

The preferred subdivision will increase impervious surfaces within the site by approximately 70%. The increase in impervious surfaces results in a decrease in stormwater infiltration and initial losses, creating an increase in the stormwater runoff volumes, velocities and pollutant loads being generated on the site. Stormwater mitigation measures must be designed to counteract the impact of development upon the downstream catchments and receiving environment.

2. STUDY OBJECTIVES

Establishing the existing 1% AEP flood conditions will allow an understanding of the availability of developable land and identification of regional stormwater constraints associated with the development site. The existing flood characteristics will form the 'base-case' for the analysis.

The objective of this Site Stormwater Management Plan is to demonstrate that the site can be developed using best practice stormwater management principles and techniques. Enabling the subdivision to meet the conditions and requirements, set in a planning permit for stormwater management and ensure that appropriate design and stormwater mitigation is applied to ensure that stormwater quality and quantity targets are achieved and maintained.

Specific objectives are detailed below.

2.1 Existing Flooding Objectives

The overriding objectives of the flood investigation are to -

- Establish the existing flood characteristics for the site, and
- Identify the associated flood risk.

2.2 Site Stormwater Objectives

The site stormwater objectives are:

1. Best Practice reductions for Water Quality
   - 80% reduction in Suspended solids (SS)
   - 45% reduction in total nitrogen (TN)
   - 45% reduction in total phosphorus (TP)
   - 70% reduction in gross pollutants (GP)
2. No-worsening stormwater peak discharges
   - Ensure pre-development flows are maintained

2.3 Regional Stormwater Objectives

Development of the Ibbotson Street site must not have a negative impact on the surrounding areas during the 1% AEP regional flood event. The impact of the development will be assessed in regards to the following:

- Flood extents – No worsening of flood extents;
- Flood storage – No loss of waterway flood storage;
- Velocities and flow characteristics – Velocity-Depth product must not exceed safety limits for people and access;
- Duration – restrict change in flood durations in external properties; and
- Cumulative flooding impact – No worsening of overall flood impacts.

3. STORMWATER MITIGATION OPTION

Stormwater mitigation will be required to achieve the water quality and quantity objectives identified in Section 2. Mitigation facilities will be designed to ensure no-worsening of runoff characteristics being caused by the proposed development.

The proposed mitigation option will consist of wetlands and sedimentation basins to manage stormwater quality and a detention basin system to manage stormwater flows generated within the development site.

The detention basin and wetland system will be constructed adjacent to the designated waterway traversing the site and which discharges into St Leonards Lake west of the site. A concept layout of the proposed stormwater detention and treatment facility is shown in Figure 3.1, below.

Figure 3.1: Concept Plan – Drainage Reserve Detention Basin and Wetlands

It is anticipated that stormwater flows generated within the developed site will be conveyed via a combination of the underground drainage network (minor flows) and overland flow paths along kerb and channel (major flows) to a designated drainage reserve.

The quantity of stormwater runoff generated within the site will be managed through the implementation of detention basins located either side of the existing waterway.

The quality of stormwater runoff generated within the developed site will be mitigated by incorporation of Water Sensitive Urban Design (WSUD) elements within a 'Green Corridor' along the existing waterway alignment. The 'Green Corridor' is an ecological corridor, consisting of encumbered and unencumbered land, which provides a buffer between the urban development and natural waterway, as depicted in Figure 3.2.

Figure 3.2: Concept Plan – 'Green Corridor'

Wetlands, sedimentation basins and grassed open swales will constitute the WSUD elements of the treatment train. The wetlands will be situated within the detention basin footprint, adjacent to the existing waterway. The basin-wetland configurations will form a stormwater interface between the developed site and the waterway.

A typical interface can be seen in Figure 3.3, below.

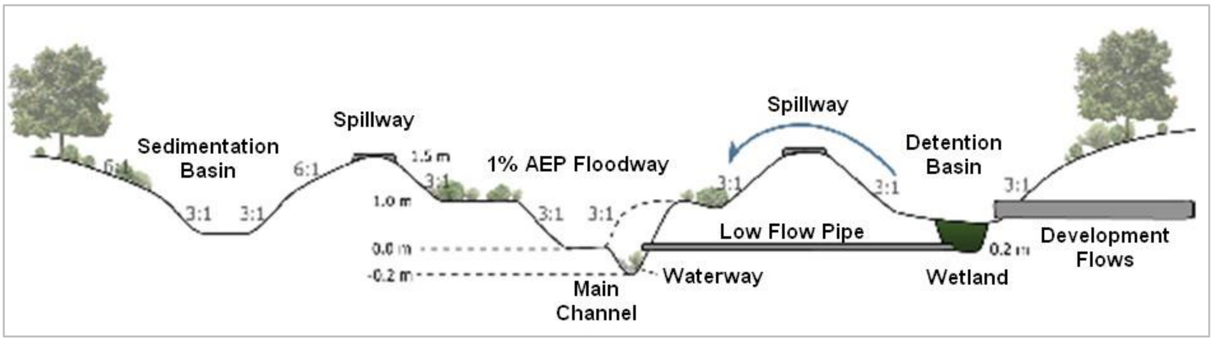


Figure 3.3: Linear Detention – Indicative Waterway Interface

The 'base-case' and proposed mitigation option are discussed in further detail below.

3.1 Base Case – Existing Conditions

To accurately assess the impact of the proposed development, the existing conditions 'base case' must first be established. This will enable the level of mitigation required for the developed site to be determined. Stormwater runoff from the existing site and its associated catchments has been investigated and documented as described in Section 1.

The quality, quantity, flow regimes, and peak stormwater discharges from the catchments in the existing state, generated by the 1% AEP storm event, were evaluated. These values formed the basis for comparison and design of the mitigation option.

3.2 Stormwater Mitigation – Design Conditions

3.2.1 Runoff Quality and Quantity

Stormwater runoff from the site will be mitigated by an end-of-line detention basin and integrated wetland system located within the adjacent drainage reserve.

The northern portion of the site is proposed to discharge to a large detention basin and constructed wetland system with the southern section of the site discharging to smaller wetlands, sedimentation basins, detention basins and (for a small section of the site) vegetated swales.

The constructed wetlands will be designed to mitigate the quality of the stormwater discharging from the developed site to achieve best practice efficiencies.

A constructed wetland consists of two separate zones:

1. **Inlet Zone (sedimentation basin) –** to remove coarse sediments, reducing scour and limiting nutrient build-up prior to entering the wetland proper; and a
2. **Macrophyte zone –** forming the biological wetland system, acting as a shallow vegetated area that allows the removal of fine particulates and the uptake of soluble pollutants. This system is designed primarily to remove stormwater pollutants associated with fine to colloidal particulates and dissolved contaminants.

A typical wetland cross-section is shown in Figure 3.4, below.

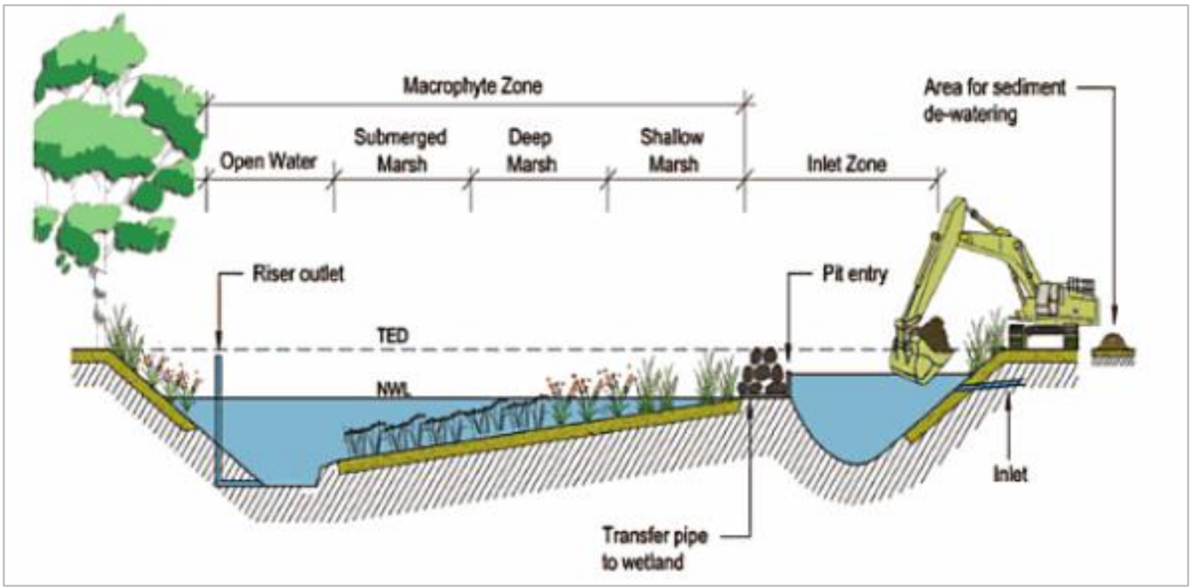


Figure 3.4: Linear Detention Typical wetland cross section[1]

Integration of the constructed wetland with the detention basin will assist in reducing the total footprint or land area required for both facilities. The detention basin is situated above the wetland with the required detention volume provided as additional storage above the total extended detention depth of the wetland; this can be seen in the typical cross-section profile in Figure 3.5.

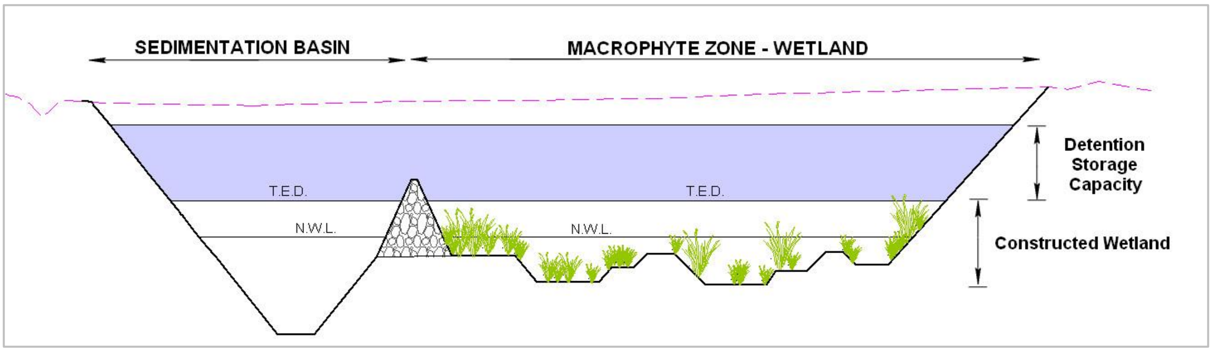


Figure 3.5: Indicative wetland and detention basin relationship cross section

The design will result in a permanent pool within the base of the detention basin and a dedicated planted area (macrophyte zone).

[1] Melbourne Water (2005). WSUD Engineering Procedures: Stormwater, CSIRO Publishing, pp. 156.

This proposal also incorporated a grassed swale to ensure best practice is achieved. A small portion of the southern catchment will discharge into a grassed swale to ensure treatment of the maximum possible number of discharge points prior to discharge from the site.

Vegetated swales are open channel systems which utilise vegetation to aid removal of suspended solids. Vegetated swales can assist in reducing peak flows for a range of events and volumetric reduction through infiltration.

A typical section of a vegetated swale is shown in Figure 3.6 below.

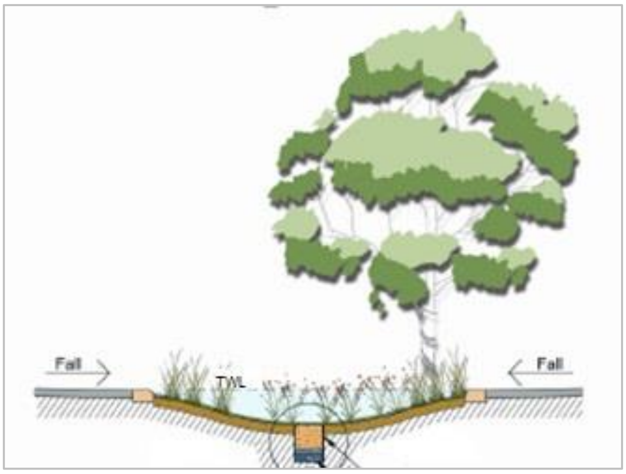


Figure 3.6: Indicative vegetated swale cross section

4. STUDY METHODOLOGY

Definition of the 1% AEP flood and design of a stormwater management plan for the Ibbotson Street development was undertaken using an integrated systems approach to define regional hydrology and analyse the performance of a range of stormwater and water cycle management options. This type of analysis is dependent on detailed inputs including topography, climate, geology, hydrology, stormwater quality and sound urban design principles.

4.1 Topography and existing infrastructure

The topography of the site, St Leonards Lake and contributing catchment area was defined using a 0.5 m grid generated from LiDAR spot points. The site ranges from an elevation of 10.5 m AHD to 2.7 m AHD over a distance of 740 m, the terrain topography is clearly defined using the 1.0 m spot points and 0.5 m contours. The level of detail can be seen the Digital Terrain Model (DTM) shown in Figure 4.1, below.

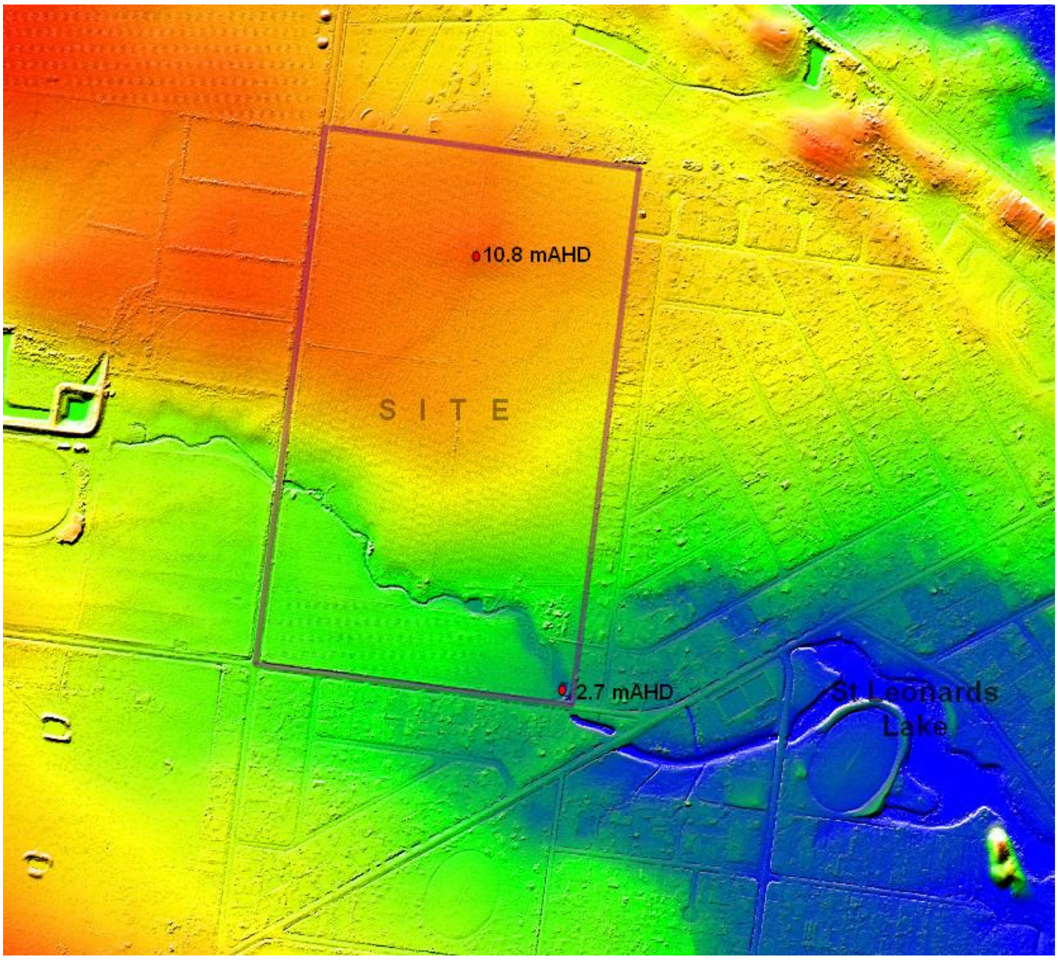


Figure 4.1: Ibbotson Street DTM generated from 0.5 m LiDAR grid

The LiDAR data, flown for the Department of Environment, Land, Water and Planning (DELWP) on 21 April 2007; provided classification of the ground surface to a vertical accuracy of +/- 10 cm enabling robust definition of the subject site. Additionally, half metre contours provided by COGG, DSE contours and other waterway data was used to define the greater catchments.

Ground and feature survey undertaken by TGM Group, around Growth Area 1, was used to refine the LiDAR data ensuring the accurate definition of hydraulic flowpaths and infrastructure. This enables greater accuracy in flood extent prediction to be achieved during the hydraulic simulation.

The feature survey for Growth Area 1 is shown in Figure 4.2, below.

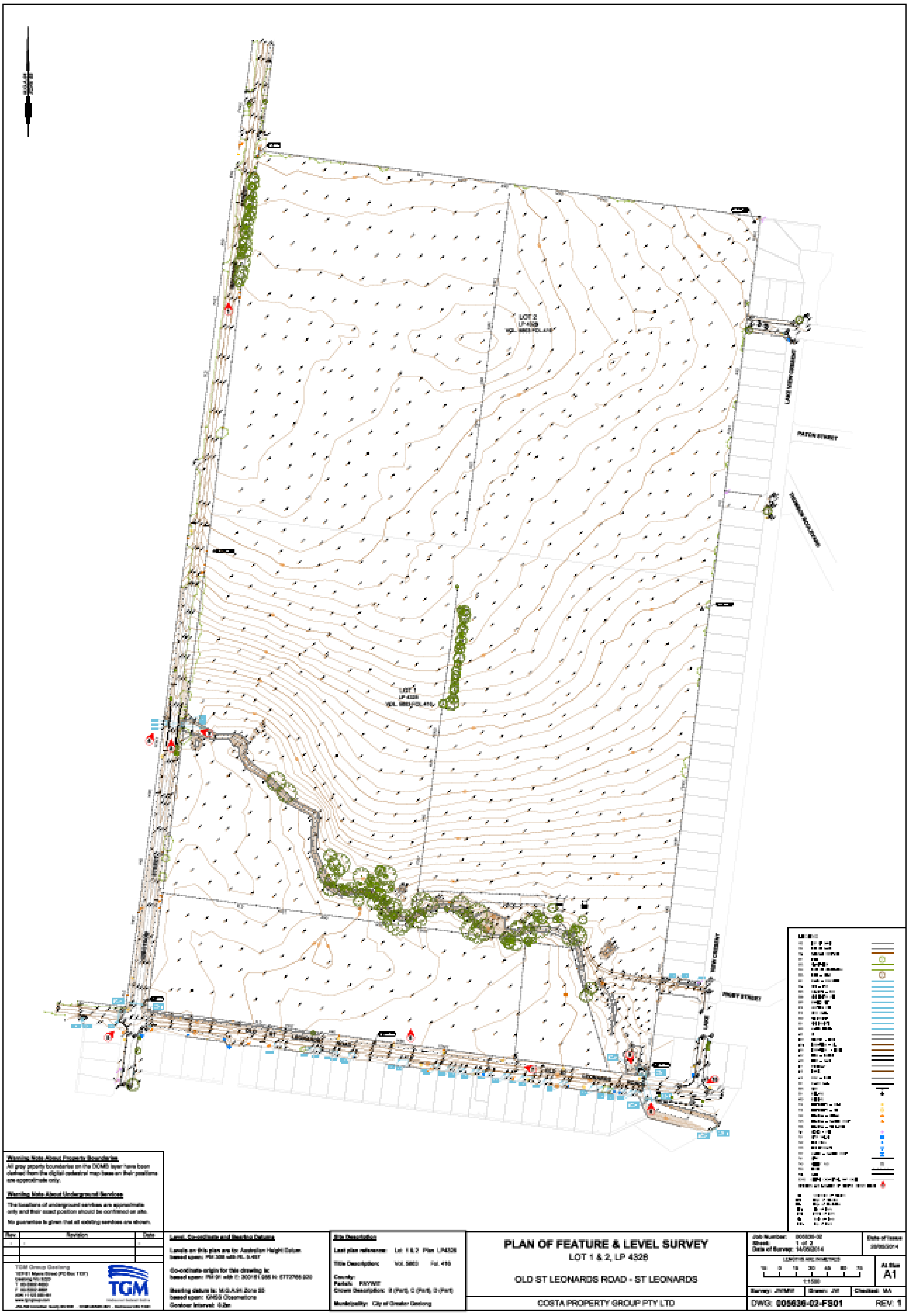


Figure 4.2: St Leonards Feature Survey – Site Waterway Survey

The field inspections and feature survey, undertaken by TGM Group, identified existing relevant stormwater infrastructure and GIS drainage tables provided by City of Greater Geelong Council was used to identify underground drainage systems.

The dimensions and condition of stormwater infrastructure, preferred flowpaths and existing conditions were identified and confirmed during onsite field inspections. There is little in the way of existing stormwater infrastructure within the site, other than the culverts within the waterway.

There is limited existing stormwater infrastructure within the study area. Existing underground drainage systems within the study area are principally sized to accommodate smaller events; however a larger 1125 mm culvert system has been constructed along Old St Leonards Road, between Ibbotson Street and Murradoc Road; to convey the larger event flows and attempt to alleviate existing flooding issues in the urban development south of the site.

Undefined flows paths provide the primary conveyance of stormwater runoff generated, within the developed catchment during large events, as overland flow discharging into the central water-body, St Leonards Lake and the upstream waterway as depicted in Figure 4.3.

Figure 4.3: St Leonards Lake – Discharge locations GA1 and part GA2

A waterway originating in the upper catchments of Growth Area 1 conveys stormwater to St Leonards Lake. The waterway traverses through the southern end of the site (depicted in Figure 4.3) with cells of culverts located beneath the road network to allow the waterway to flow into St Leonards Lake. Culverts along the creek alignment down to Cole Street and at key locations around the site were identified and inspected during the survey process. The existing culvert conditions can be seen in Figure 4.4.

Figure 4.4: Existing culvert locations and condition

The existing site and upper catchment is predominantly agricultural ploughed farm land. The loose soil structure exasperates erosion and contributes to high sediment loading within the stormwater runoff. This has caused many of the existing culvert inlet/outlets to become blocked (by sediment or vegetation), retain water in pools formed by erosion, and to exhibit various degrees of operational effectiveness. For design purposes the culverts have been modelled in hydraulic simulations as operating at design capacity with a sensitivity analysis undertaken to assess the impact of blockages and other hydraulic anomalies.

The St Leonard Lake weir is a significant piece of infrastructure, pertinent to regional hydraulics. The level of the weir spillway will determine the Lakes initial water level (IWL) and dictate the degree of future storm tide impact. TGM Group surveyed the weir spillway, located ~100 meters north of an existing datum benchmark. The confirmed weir level of 1.792 m AHD is accurate to ±1mm. This level was represented in model simulations.

4.1.1 Ibbotson Street Subdivision Site Stormwater Catchments

Identifying and defining the contributing stormwater catchment area for the Ibbotson Street site is crucial to understanding the volume, rate and quality of stormwater being generated.

4.1.1.1 External Catchments

The undeveloped Ibbotson Street site is subject to stormwater runoff from the rural surrounding catchments. Topography data obtained from the spatial services database and urban drainage infrastructure plans provided by COGG and discussions with CCMA, enabled the identification of the current stormwater sub-catchments discharging to the Ibbotson Street subdivision site. The contributing stormwater sub-catchments for Ibbotson Street are shown in Figure 4.5 below.

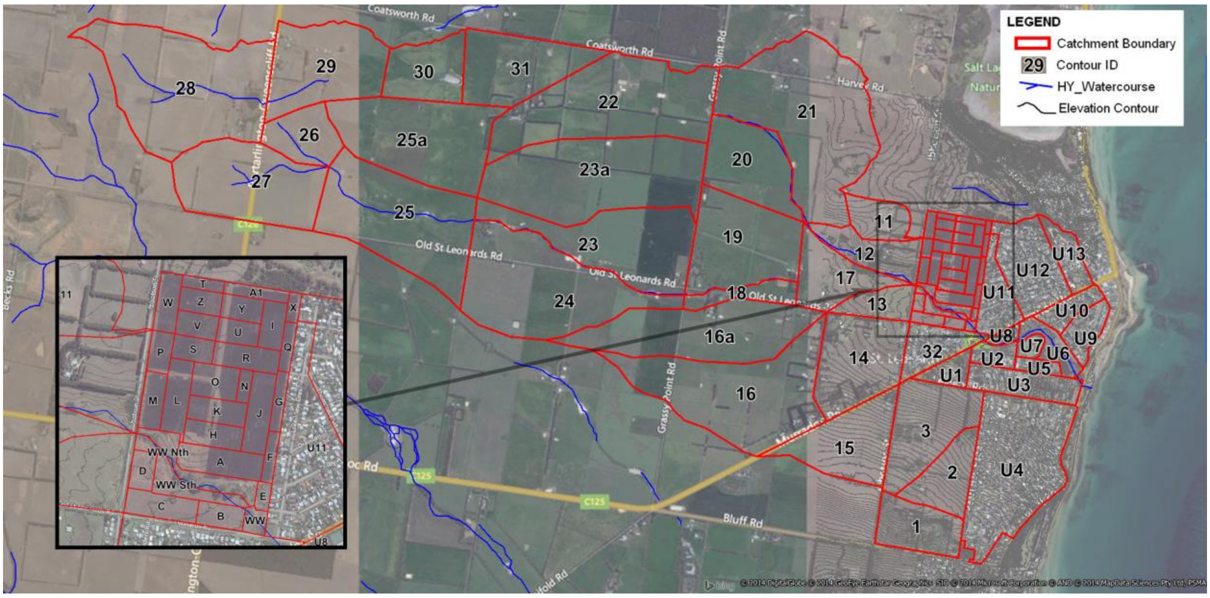


Figure 4.5: Ibbotson Street external stormwater catchment boundaries

The sub-catchment breakdown is shown in Table 4-1.

Table 4-1: Characteristics of Ibbotson Street external stormwater catchments

Catchment	Area (ha)	Land Use	Catchment	Area (ha)	Land Use
1	20.7	Farming – Rural	**26**	18.4	Farming – Rural
2	19.2	Farming – Rural	**27**	54.5	Farming – Rural
3	35.6	Farming – Rural	**28**	91.8	Farming – Rural
11	12.4	Farming – Rural	**29**	47.0	Farming – Rural
12	17.5	Farming – Rural	**30**	24.4	Farming – Rural
13	10.8	Farming – Rural	**31**	37.0	Farming – Rural
14	38.8	Farming – Rural	**32**	14.2	Farming – Rural
15	33.9	Farming – Rural	**U1**	6.3	Farming – Rural
16	75.3	Farming – Rural	**U2**	5.3	Farming – Rural
16a	58.5	Farming – Rural	**U3**	15.6	Residential
17	22.9	Farming – Rural	**U4**	64.6	Residential
18	10.0	Farming – Rural	**U5**	3.3	Residential
19	47.6	Farming – Rural	**U6**	8.3	Residential
20	34.4	Farming – Rural	**U7**	2.7	Residential
21	95.3	Farming – Rural	**U8**	2.1	Residential
22	77.9	Farming – Rural	**U9**	8.7	Residential
23	54.9	Farming – Rural	**U10**	7.2	Residential
23a	81.9	Farming – Rural	**U11**	13.0	Residential
24	76.3	Farming – Rural	**U12**	26.1	Residential
25	72.4	Farming – Rural	**U13**	9.2	Residential
25a	62.2	Farming – Rural			

Internal catchments were delineated reflecting the proposed developed cub-catchment layout; these are identified in the following section.

4.1.1.2 Ibbotson Street Site Developed Catchments

The preliminary development plan for the Ibbotson Street site will see the construction of approximately 483 residential lots, roads reserves, parking lots and public open space. The preliminary development plan is shown in Figure 4.6 below. The developed catchments have been defined using the preliminary design surfaces. Site topography may change during detail design.

It is noted that under the existing conditions there was some overland flow from the north of the site into the adjoining land. Under the developed conditions this catchment will be routed south into the wetland/basin and discharge to the existing waterway.

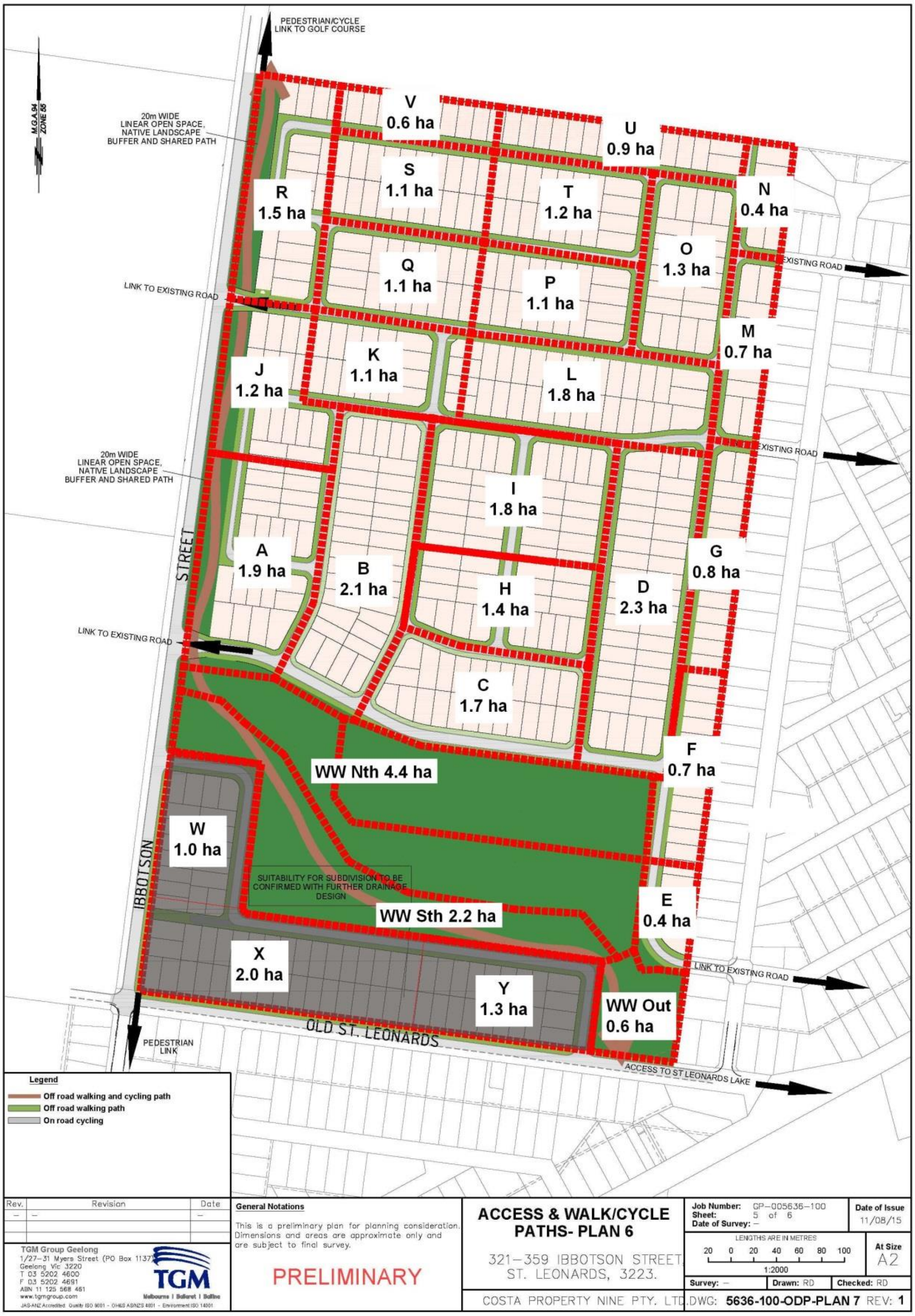


Figure 4.6: Ibbotson Street preliminary development plan and sub-catchments

Potential for full site development will need to be confirmed through detailed design. The 'hatched' area identified in Figure 4.6 depicts area requiring further flood impact assessment. For the purpose of the Site Stormwater Management Plan discussed in this report, the site has been assumed to be fully developed.

Development of the proposed Ibbotson Street Estate will increase the proportion of impervious surfaces within the site and the magnitude of stormwater discharges and contaminant loadings from each of the Sub-Catchments shown in Figure 4.6.

The sub-catchment areas, proportion of impervious surfaces and land use for the developed site shown in Table 4-2, were adopted for this study.

Table 4-2: Characteristics of Ibbotson Street developed catchments

Sub-catchment	Area (ha)	Impervious area (%)	Dominant land use
A	1.9	70	Residential
B	2.1	70	Residential
C	1.7	70	Residential
D	2.3	70	Residential
E	0.4	70	Residential
F	0.7	70	Residential
G	0.8	70	Residential
H	1.4	70	Residential
I	1.8	70	Residential
J	1.2	70	Residential
K	1.1	70	Residential
L	1.8	70	Residential
M	0.7	70	Residential
N	0.4	70	Residential
O	1.3	70	Residential
P	1.1	70	Residential
Q	1.1	70	Residential
R	1.5	70	Residential
S	1.1	70	Residential
T	1.1	70	Residential
U	0.9	70	Residential
V	0.6	70	Residential
W	1.0	70	Residential
X	2.0	70	Residential
Y	1.3	70	Residential
WW North	4.4	0	Waterway Zone & Green Corridor
WW South	2.2	0	Waterway Zone & Green Corridor
WW Out	0.6	0	Waterway Zone & Green Corridor

4.2 Climate Processes

Australia experiences one of the most variable climatic regimes on the planet. The extreme natural variation of the continent's climate includes cyclic patterns of droughts and floods throughout recorded history.

An understanding of the temporal and spatial variation of climate processes is essential for an accurate analysis of hydrology. Further, the need to effectively include climate processes in analysis of water resources and flooding has increased with the onset of climate change. It is now widely understood that the earth's climate system has been subject to significant warming that will increase the variability of climate processes. [2,3]

Four long term rainfall sequences were analysed for the Ibbotson Street site in St Leonards and compared with the council suggested rainfall gauge for urban development of Geelong North (BOM station number 87133, located 32 km from the site). There is no long term sequence rain gauge located within St Leonards itself. Pluviograph rain gauges situated at Portarlington (BOM station number 87053); Queenscliff (BOM station number 87054); Drysdale (BOM station number 87114); and South Channel Island (BOM station number 86344), located 8 km, 11.8 km, 13 km and 12 km from the Ibbotson Street site respectively, were used in the analysis. The long term rainfall sequence for the four sites is shown in Figure 4.7.

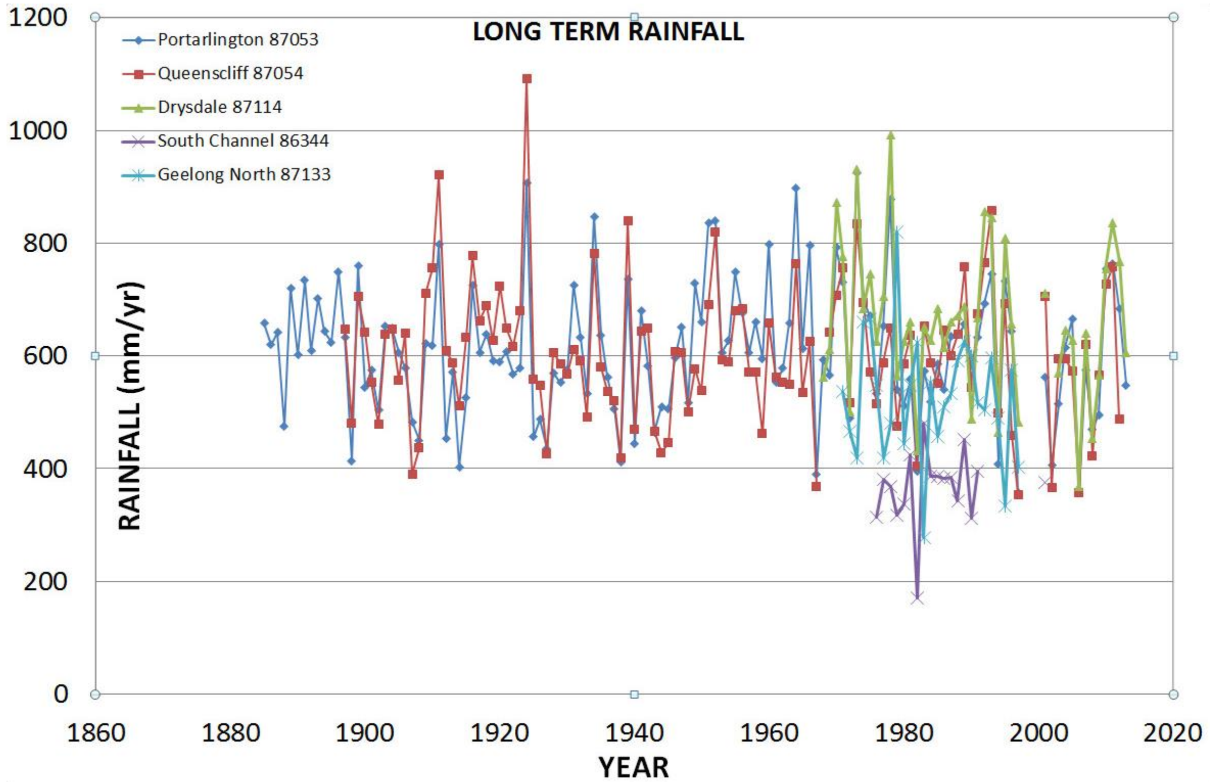


Figure 4.7: Long term annual rainfall time series around the Ibbotson Street site at St Leonards

[2] DSE (2008). Climate change in Victoria: 2008 summary – the Victorian climate change adaptation program.
[3] Coombes P., Colegate M., et. al (2012). Greater Melbourne systems model – modelling in support of the Living Victoria Ministerial Advisory Council.

A summary of the rainfall gauges is shown in Table 4-3 below.

Table 4-3: Rainfall gauges reviewed in this analysis

Record	Start Date	End Date	Average Annual Rainfall (mm/yr)	Length (years)	Distance from site (km)
Geelong North rainfall gauge (87133)	1969	17/02/2003	534.2	34.5	32
Portarlington rainfall gauge (87053)	1884	Open	609.5	126	8
Queenscliff rainfall gauge (87054)	1896	2012	612.4	117	11.8
Drysdale rainfall gauge (87114)	1968	Open	673.8	47	13
South Channel Island rainfall gauge (86344)	1975	2002	386.3	21	12

The average annual rainfall for the locations in close proximity to the development site sharing similar topographical and meteorological attributes vary by approximately 1%, the rainfall for gauges further from the site and being subject to differing site characteristics have a greater variation of 25% with south channel island being up to 40%.

The sample group provides pluviographs that display comparable Average Annual Rainfall within a relatively close spatial proximity to the subject site. The Portarlington gauge displays a longer record and what would appear to be a more consistent record and would normally be adopted for the analysis, however, COGG standards dictate the use of the North Geelong rainfall record for these types of analyses.

To conduct a robust hydrological analysis, evapotranspiration must also be analysed in the assessment. Evapotranspiration is the term used for the transfer of water from the land surface to the atmosphere as water vapour. It is based on different climate zones, which vary throughout the Australian continent due to the availability of water and vegetation.

Data presented by the Bureau of Meteorology[4] enabled the extraction of average monthly areal potential evapotranspiration (PET) that correlated to our site. This data was employed for the hydrologic and water quality analyses. A map of the average annual areal PET is shown below in Figure 4.8.

[4] Bureau of Meteorology (http://www.bom.gov.au/jsp/ncc/climate_averages/evapotranspiration/index.jsp?maptype=3&period=an)

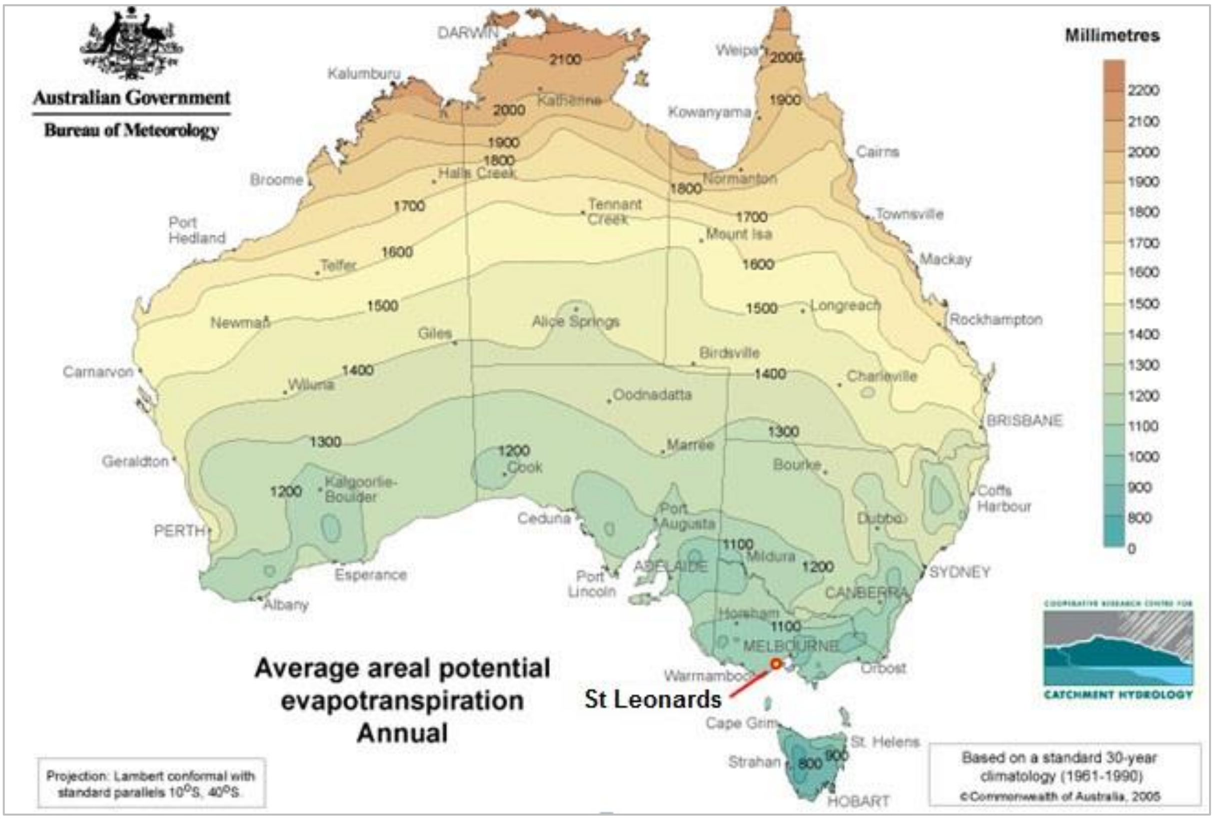


Figure 4.8: Average annual areal potential evapotranspiration

Although the local rainfall gauge at Portarlington has a long continuous data set and a more representative rainfall pattern COGG have advised within the IDM design notes that 110 years of Geelong data have been analysed as well as best practise formulisation of evapotranspiration data. The COGG recommendation has subsequently been adopted.

4.3 Geology

Geological maps of St Leonards depict a geology composed of two distinct layers. The top layer is representative of Victorian coastal towns and has been formed by deposits of sand, silt, clay and gravel that were compacted and solidified to form durable clay. This layer is geologically ‘young’ at only one to two million years old.

The lower layer is part of the Moorabool Viaduct formation, a larger, older and more resistant geological unit which formed between four and five million years ago. It includes compressed gravels, sands, silts and sandy clays containing an iron component.

Council has confirmed that the upper contributing catchment is comprised of 80% fine sandy loam that exhibits low permeability and 20% light clay, fine sandy loam with a very low permeability potential.

Specific soil characteristics need to be detailed to guide calibration of rainfall runoff within the MUSIC water quality modelling process. Ideally, the relevant Council specifies soil parameters that are considered to reflect typical characteristics for the region. City of Greater Geelong MUSIC Modelling Guidelines[5] have been adopted for our site.

The soil parameters utilised are shown below in Table 4-4.

Table 4-4: Soil characteristics for St Leonards

Parameter	Urban Residential
Rainfall Threshold (mm)	1
Soil Capacity (mm)	30
Initial Storage (%)	30
Field Capacity	20
Infiltration Capacity coefficient a	200
Infiltration Capacity coefficient b	1
Initial Depth (mm)	10
Daily Recharge Rate (%)	25
Daily Base flow Rate (%)	5
Daily Deep Seepage Rate (%)	0

4.4 Hydrology

It was an objective for this study that the development should not detrimentally affect stormwater runoff volume or quality characteristics. The assessment of the stormwater runoff characteristics of the site in the existing and developed states was undertaken using the hydrological model XP-RAFTS developed by XP-Solutions.

4.4.1 Model Description

The XP-RAFTS program is a non-linear runoff routing model using the Laurenson routing procedure to produce stormwater runoff hydrographs from design storm events derived from Intensity-Frequency Duration data (based on the AR&R 1987 methodology) or actual recorded events.

The more simplistic Rational Method calculations were not employed in this study (other than for purposes of validation for ungauged catchments) because this type of method does not account for the volumes of rainfall in storm events or the range of variable initial conditions that impact on stormwater runoff in defined catchments.

4.4.2 Intensity Frequency Duration Data

The hydrology model applied design storm parameters from Australian Rainfall and Runoff[6], to define the hydrology of the stormwater catchments in the study area. Design storms were generated for average

[5] City of Greater Geelong (2012). MUSIC Modelling Guidelines, No. 3
[6] IEAust. (2001). Australian rainfall and runoff: a guide to flood estimation. Vols. 1 and 2. The Institution of Engineers, Australia.

exceedance probabilities (AEP's) using a regional skewness of 0.4 and temporal pattern region 1. The intensity frequency duration (IFD) climatic data used in the hydrology model to simulate performance is shown in Table 4-5.

Table 4-5: IFD data inputs for the St Leonards

ARI (years)	Rainfall intensity (mm/hour) for a given duration (hours)		
	1	12	72
2	17.95	3.61	0.93
50	34.04	6.63	1.92

The generated IFD Table for this region is shown in Table 4-6 below.

Table 4-6: IFD Table (St Leonards)

ARR storm location: St Leonards
Latitude: 38.18 Longitude: 144.70
Zone: 1 Skew: 0.40
ARR standard intensities:

ARI	6-min	1 hour	12 hour	72 hour
2	57.49	17.95	3.61	0.93
50	123.51	34.04	6.63	1.92

Average intensity (mm/hr)

Duration	years 1	2	5	10	20	50	100	200	500
5 min	44.92	59.8	81.58	96.6	116.69	145.82	170.21	196.94	236.36
10 min	34.2	45.29	61	71.71	86.08	106.78	123.99	142.77	170.31
15 min	28.45	37.55	50.13	58.64	70.1	86.51	100.11	114.89	136.49
20 min	24.7	32.51	43.12	50.25	59.87	73.61	84.95	97.25	115.17
25 min	22.01	28.91	38.14	44.31	52.65	64.52	74.3	84.88	100.26
30 min	19.96	26.17	34.37	39.83	47.22	57.71	66.34	75.65	89.16
45 min	15.91	20.77	26.99	31.09	36.67	44.54	50.98	57.9	67.9
1.00 hr	13.46	17.51	22.58	25.89	30.41	36.77	41.96	47.51	55.51
1.50 hr	10.44	13.58	17.46	20	23.47	28.34	32.31	36.55	42.66
2.00 hr	8.69	11.29	14.5	16.59	19.45	23.47	26.73	30.23	35.25
3.00 hr	6.68	8.68	11.12	12.71	14.88	17.93	20.41	23.06	26.87
4.50 hr	5.14	6.66	8.52	9.72	11.38	13.69	15.57	17.57	20.45
6.00 hr	4.26	5.53	7.05	8.04	9.4	11.3	12.84	14.49	16.85
9.00 hr	3.28	4.25	5.41	6.16	7.19	8.64	9.81	11.05	12.84
12.00 hr	2.72	3.53	4.48	5.1	5.95	7.14	8.1	9.12	10.59
18.00 hr	2.03	2.64	3.39	3.88	4.56	5.5	6.27	7.09	8.27
24.00 hr	1.65	2.15	2.78	3.19	3.76	4.56	5.21	5.91	6.92
30.00 hr	1.39	1.82	2.37	2.74	3.23	3.93	4.5	5.12	6.01
36.00 hr	1.21	1.59	2.08	2.4	2.85	3.47	3.99	4.54	5.35
48.00 hr	0.97	1.27	1.68	1.95	2.31	2.84	3.27	3.74	4.42
72.00 hr	0.69	0.9	1.21	1.42	1.69	2.09	2.42	2.78	3.31

4.4.3 Loss Parameters

TGM Group has adopted the COGG initial and continual loss parameters used to define the catchment hydrology. These parameters are shown in Table 4-7.

Table 4-7: Hydrology Runoff parameters (COGG)

Surface Type	Initial Loss (IL)	Continuing Loss (CL)
Pervious	20 mm	3 mm/hr
Impervious	2.5 mm	0 mm/hr

XP-RAFTS allows each catchment to be split into two sub-areas that can be routed separately. For this analysis separate sub-areas were used to model the pervious and impervious area to enable application of the different loss parameters. The fraction impervious was calculated for each catchment.

4.4.4 Fraction Impervious

The fraction of impervious surfaces within a catchment is an integral input in hydrological modelling. The area impervious was ascertained from aerial and satellite images, field inspections and proposed development plans. The impervious fractions adopted are shown in Table 4-8.

Table 4-8: Impervious fractions relative to land use

Land Use	Impervious Fraction
Residential (Existing)	0.4
Residential (Developed)	
< 300 m²	0.95
300 to 400 m²	0.6
400 to 600 m²	0.5
Gravel Roads	0.7
Older Asphalt Roads	0.8
New/Resurfaced Roads	1
Ponds and Water Bodies	1
Public Open Space	0.1

4.4.5 Validation of Hydrology Model

An analysis of flooding in any region is subject to a range of variable influences that impact on hydrological assessment. Calibration of hydrological models to actual conditions is crucial to ensuring the accuracy of analysis. A key element of the calibration process is identification of the stormwater catchments that impact on the study area, the characteristics of those catchments and the configuration of waterways. Factors such as availability of observed rainfall data, soil type, soil conditions, land use and local knowledge were considered in this investigation.

Due to the lack of historical rainfall runoff or stream flow data in catchments in and around the study area, calibration of the model to recorded data was not possible; therefore a model Validation process was undertaken.

Validation of results from the hydrological model was undertaken using empirical methods. Peak flows generated by the XP-RAFTS model selected catchments were compared to a peak discharge derived by the application of some form of the Probabilistic Rational Method (PRM).

It is noted that City of Greater Geelong's stormwater detention storage design notes [COGG 2012] state that for estimation of rural or pre-development discharge the runoff-routing model (XP-RAFTS) must be validated against the VicRoads Method for rural discharge and may be adjusted ±30% to reflect catchment conditions. The Rational method produced a 1% AEP (Q_{100}) peak discharge which was used as a 'ball-park' figure for the XP-RAFTS validation process. The validation process was undertaken for every sub-catchment and at key routing locations within the hydrological model.

The Probabilistic Rational Method (PRM) assumes runoff is generated within undeveloped rural catchments, therefore the study catchments were analysed as undeveloped for the calibration process.

The various forms of the PRM adopted in this study are shown Table 4-9.

Table 4-9: Calculation methods used for validation of hydrology model

Method	Description	Catchment Size (ha)
Adam's Method	PRM calculations fitted for Victorian conditions	Adopted for catchments < 100 ha
VicRoads Method	Victorian Method with an Area factor applied for larger Catchments	Adopted for catchments > 100 ha

The Initial and Continual Loss factors, nominated in Table 4-7, were not capable of matching the predicted peak discharge generated by the various PRM calculations for each of the undeveloped contributing sub-catchments. The global storage multiplication factor was adjusted, using B modification factor in XP-RAFTS, until peak stormwater discharges generated by the hydrology model were comparable (± 30%) to peak discharges derived using Probabilistic Rational Method calculations. The storage multiplication factor is discussed below.

Storage Coefficient Multiplication Factor

During calibration of a gauged catchment the Storage Coefficient Multiplication Factor (B_X) may be used to modify the calculated storage time delay coefficient (B). The Storage Coefficient Multiplication Factor uniformly modifies all sub catchment Storage Time Delay Coefficient values previously computed or determined from the default equation. This process has been applied to enable validation of the hydrology model.

The B_X modification factor selection process is discussed in Section 4.6.1, below.

Catchments that were not analysed in the 2D hydraulic model were detailed in the hydrology model. Routed hydrographs were extracted at the interface between the two models. Validation of these points was undertaken. Node BC16, C45 and C51 (see Figure 4.9) depict the validation process applied in this analysis.

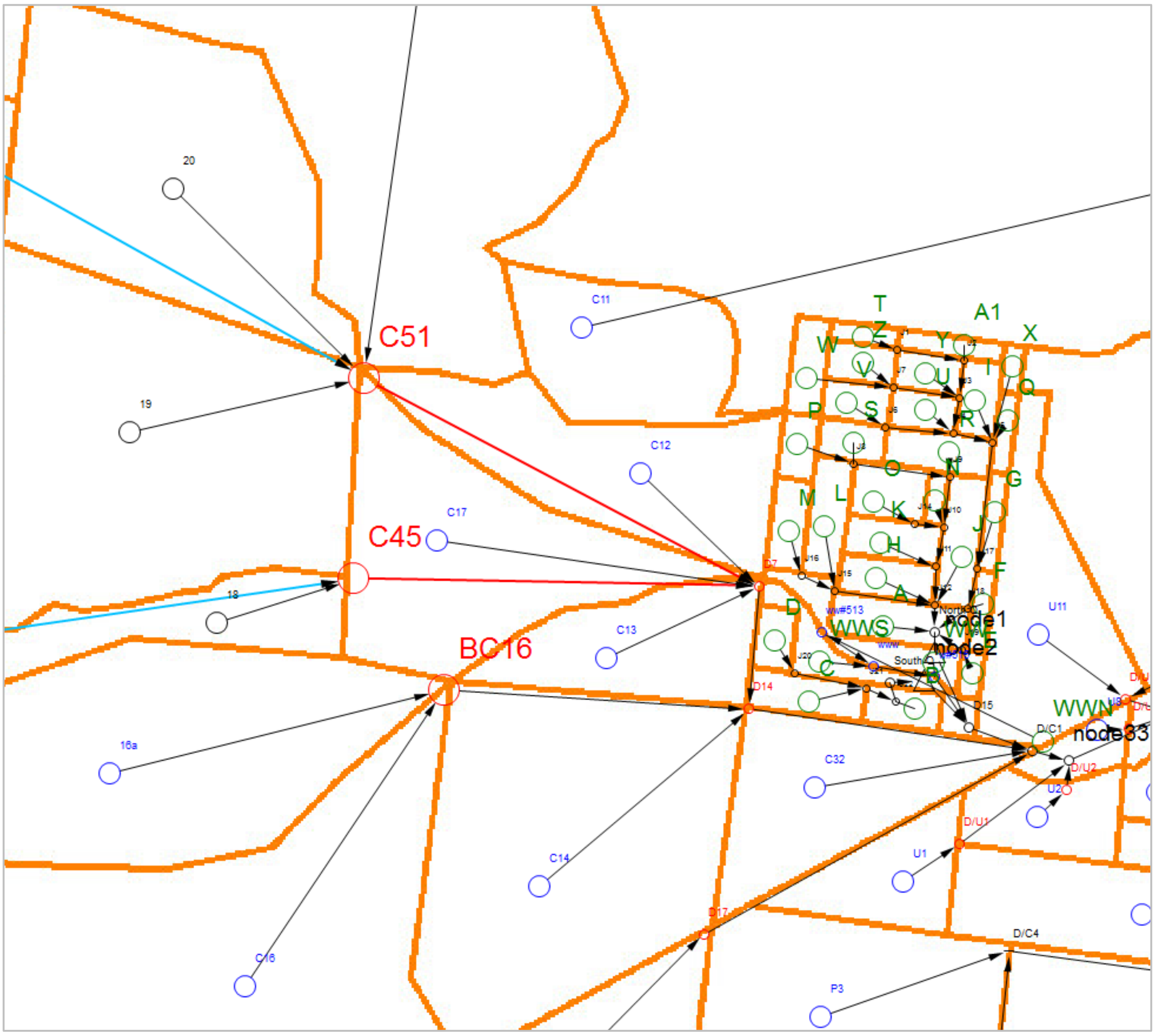


Figure 4.9: Node BC16, C45 and C51 validation locations

The nodes selected for validation reflect the external inflow boundary conditions for the hydraulic model and constitute routed runoff from a cluster of sub-catchments. ***Note:*** All internal catchments were validated using the same process.

The catchment area contributing to each node is shown in Figure 4.10.

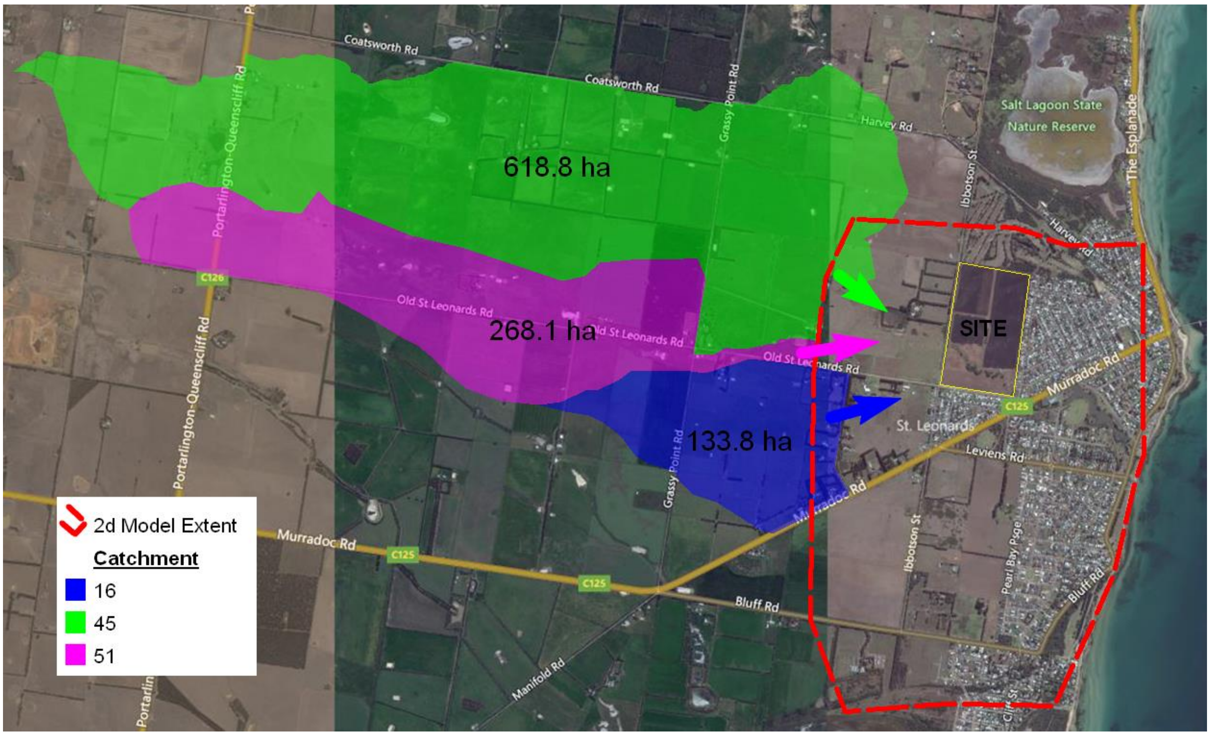


Figure 4.10: External catchment locality plan

Catchment calculation inputs are shown in Table 4-10, below.

Table 4-10: Catchment Detail – Rational Method Input

Catchment	BC16		C45		C51	
Adopted Rational Method	VicRoads		VicRoads		VicRoads	
Area (km²)	1.338		2.681		6.188	
Time of Concentration (t_C) – minutes	50.9		66.3		91.1	
Runoff Coefficient	P_{10}	P_{100}	P_{10}	P_{100}	P_{10}	P_{100}
	0.10	0.26	0.10	0.256	0.10	0.233
Rainfall Intensity (mm/hr) – 1% AEP	47.41		39.95		32.10	
Area Size Factor (F_A)	2.0		1.97		1.79	
1% AEP Peak Q (m³/s)	4.6		7.6		12.8	

Table 4-10 indicates the 1% AEP inputs only. Calculations were undertaken for the full range of design storm events (1% - 100% AEP). Validation was undertaken for each sub-catchment for the full range of storm events.

4.4.6 B_X modification factor selection

Selection of a suitable global storage modification factor *(B_X)* was achieved through a 'trial and error' process to match the rural RAFTS catchments to within ±30% of the calculated rational method discharges.

The validation process was carried out for all sub-catchments, however, it was the primary inflow boundary locations C45, C51 and BC16 that determined the final B_X factor.

The results of the B_X factor selection process are shown in Table 4-11.

Table 4-11: B_X factor selection

Node	Rational Method Peak (m³/s)		*Bx* Factor & Discharge (m³/s)			
	VicRoads Method	± 30% Range (COGG)	2.1	2.0	1.9	1.8
C45 (268.1 ha)	7.7	5.4 - 10	8.8	9.1	9.5	9.9
C51 (618.8 ha)	12.8	9 - 16.7	16.1	16.3	16.6	17
BC16 (133.8 ha)	4.6	3.2 - 6	3.2	3.3	3.5	3.6

The validated XP-RAFTS model was then adapted to current conditions by applying the effect of urbanisation in the form of impervious surfaces. Urbanisation of a catchment can have a dramatic effect on peak discharges and catchment hydrology.

The hydrological model was used to define the current climate existing conditions, proposed design conditions and quantify the level of mitigation required to produce a 'no-worsening' of stormwater peak discharges for 1% AEP storm events to protect waterways from erosion and sedimentation and reduces risks to people and property up to this event.

4.4.7 Existing Catchment Conditions

The St Leonards Lake catchment (Figure 1.6) delineation allowed for the definition of runoff hydrographs that will constitute key inflow points into the hydraulic model. Delineation of the site sub-catchments was undertaken to reflect the proposed developed sun-catchment arrangement to ensure an accurate comparison could be made.

The XP-RAFTS stormwater network utilised in this analysis is depicted in Figure 4.11 below.

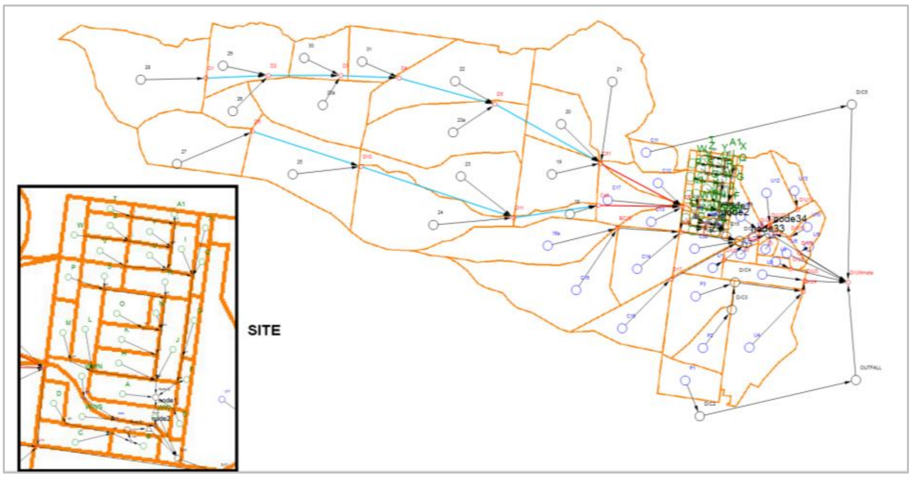


Figure 4.11: XP-RAFTS stormwater network

Key catchment parameters adopted in the XP-RAFTS model include catchment area, vectored slope, land use and PERN (roughness) values, were estimated from available topographic data, aerial and satellite photography, field inspections and local knowledge.

The sub-catchment parameters adopted in the XP-RAFTS model are summarised, in Table 4-12. The Global B_X storage modification factor was used to enable validation of hydrology models.

Table 4-12: XP-RAFTS External Sub-Catchment Properties

Catchment	Area (ha)	Slope (%)	PERN (Per/Imp)	Catchment	Area (ha)	Slope (%)	PERN (Per/Imp)
1	20.7	2.5	0.035	**26**	18.6	3.2	0.035
2	19.2	1.5	0.035	**27**	54.5	3.2	0.035
3	35.6	1.5	0.035	**28**	91.8	2.2	0.035
11	12.4	0.9	0.035	**29**	47	3.2	0.035
12	17.1	2.5	0.035	**30**	24.4	3	0.035
13	11.2	2.5	0.035	**31**	37.9	2.1	0.035
14	38.2	2.4	0.035	**32**	14.2	2.4	0.035
15	33.9	1.3	0.035	**U1**	6.3	0.75	0.035
16	74.3	3.2	0.035	**U2**	5.3	1.4	0.035
16a	58	3.2	0.035	**U3**	15.6	0.9	0.035/0.016
17	24.7	2.3	0.035	**U4**	64.6	2.1	0.035/0.016
18	10	1.8	0.035	**U5**	3.25	0.2	0.035/0.016
19	47.6	1.2	0.035	**U6**	2.7	0.05	0.035/0.016
20	34.4	2.6	0.035	**U7**	2.7	0.35	0.035/0.016
21	95.3	2.6	0.035	**U8**	2.13	0.7	0.035/0.016
22	77.9	2.6	0.035	**U9**	8.66	0.8	0.035/0.016
23	54.8	1.5	0.035	**U10**	7.22	0.7	0.035/0.016
23a	81.11	1.5	0.035	**U11**	13.1	1.4	0.035/0.016
24	76.3	1.5	0.035	**U12**	26.06	1.5	0.035/0.016
25	71.9	3	0.035	**U13**	9.24	0.9	0.035/0.016
25a	62.7	3	0.035	**Global Modification Factor B_x**			1.9

The PERN, or Manning's roughness coefficient applied to each catchment reflect the undeveloped (pervious) area of the sub-catchments. For sub-catchments that contain regions of urban development (impervious area), lower PERN values have been adopted to reflect the increased responsiveness of these land uses types to rainfall run-off generation. The impervious areas were modelled using the second sub-catchment approach in XP-RAFTS. The PERN value for the impervious areas has been set to 0.016.

The existing conditions discharge rates for the 'validation' locations can be seen in Table 4-13.

Table 4-13: 1% AEP design storm peak flows - validation process

Node	Rational Method Peak (m³/s)		XP-RAFTS Peak Discharge (m³/s)	
	VicRoads Method	**± 30% Range**	Validated Conditions	Current Conditions
C45 (268.1 ha)	7.7	5.4 - 10	9.5	9.5
C51 (618.8 ha)	12.8	9 - 16.7	16.6	16.7 (9hr) 16.8 (12hr)*
BC16 (133.8 ha)	4.6	3.2 - 6	3.5	6.04

** Under current conditions the 720 minute storm produces the highest peak for catchments contributing flow to node C51.*

The model simulated for a number of storm event durations in order to identify the critical duration for each catchment. The event hydrographs can be seen in the following figures for catchment nodes BC16, C45 and C51, respectively.

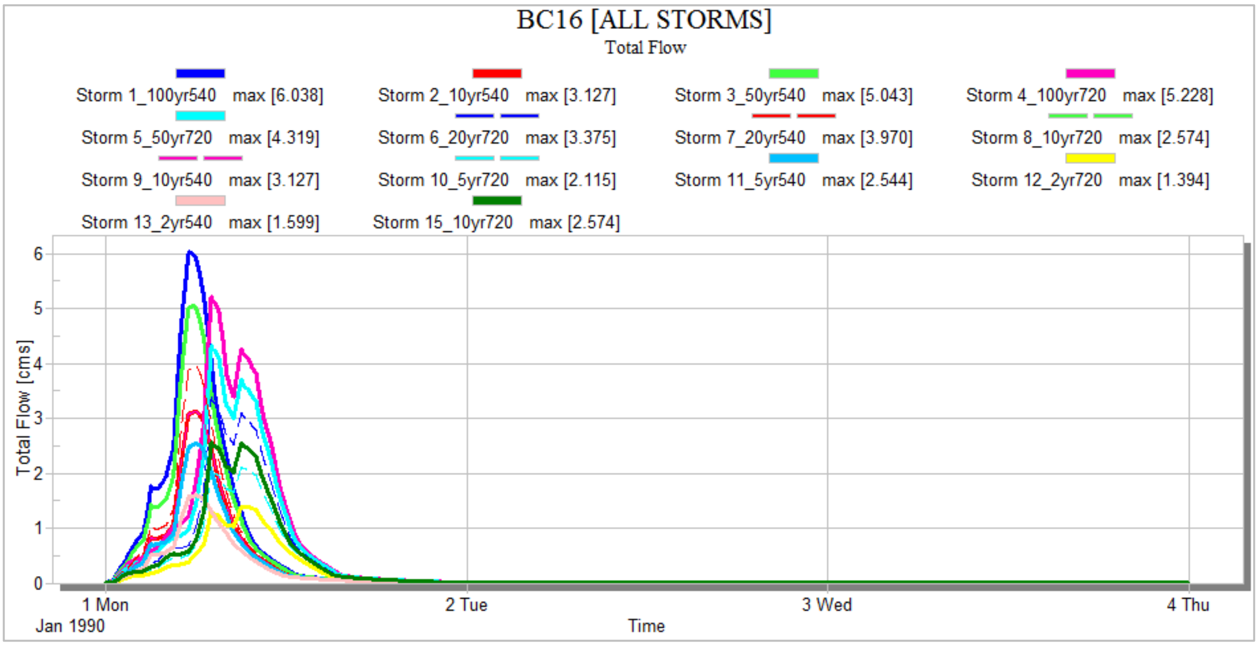


Figure 4.12: Catchment Node BC16

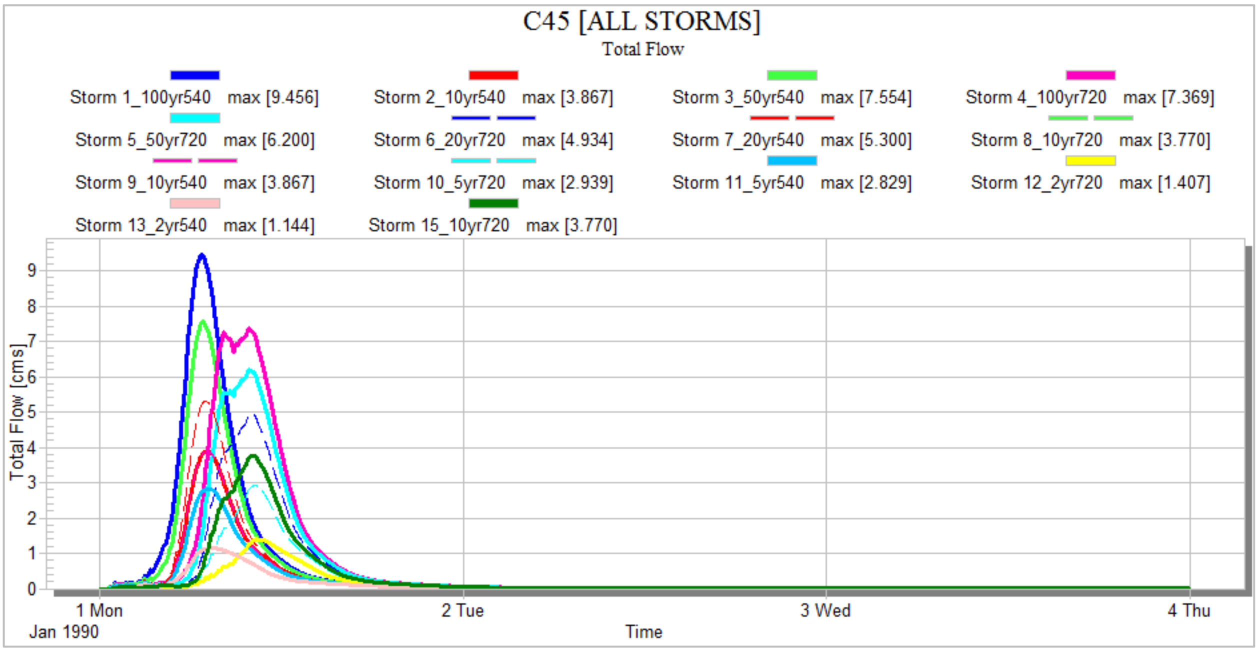


Figure 4.13: Catchment Node C45

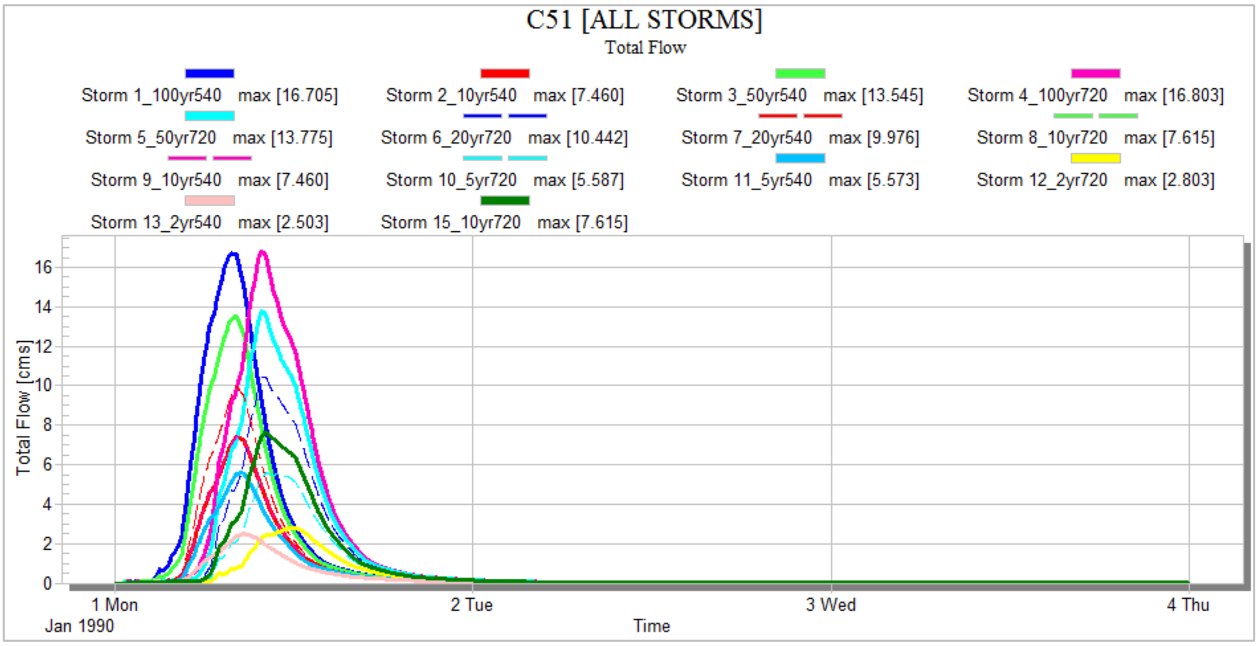


Figure 4.14: Catchment Node C51

Note: *Above hydrographs are indicative of a sample selection of storm events and durations simulated.*

It is noted that the critical event duration for the 1% AEP storm event is 540 minutes (9 hours), other than for catchment node C51 which has a 0.1 m^3/s greater peak discharge for the 720 minute (12 hour) duration. The following storm events were analysed during this process.

Table 4-14: Analysed storm events

Duration	Average Recurrence Interval (ARI)	Annual Exceedance Probability (AEP)
	100 year	1%
	50 year	2%
	20 year	5%
	10 year	10%
	5 year	20%
	2 year	50%

Each catchment was analysed using a range of storm durations, from 15 minutes to 72 hours, ensuring identification of the critical event for each catchment. The critical duration for the various storm events range from 9 to 12 hours for the rural catchments and 15 to 20 minutes for the urban catchments.

4.4.7.1 Existing Site Conditions

The site catchments are a uniform cleared agricultural pasture land use and do not comprise any impervious surfaces under existing conditions.

The run-off hydrographs generated within each contributing catchment and key confluence points were extracted from the XP-RAFTS model and used as inputs for the hydraulic analysis.

4.4.8 Developed Conditions

A developed RAFTS model was created reflecting the changed sub-catchment delineation (refer Figure 4.6) and runoff characteristics within the developed site. The RAFTS inputs are detailed in Table 4-15.

Table 4-15: XP-RAFTS Site Sub-Catchment Properties – Developed

Catchment	Area (ha)			Slope (%)	PERN (Per/Imp)
	Total	Imp	Per		
A	2.54	1.9	0.64	2	0.035 / 0.016
B	1.27	0.95	0.32		
C	1.97	1.48	0.49		
D	1.02	0.76	0.26		
E	0.44	0.33	0.11		
F	0.67	0.5	0.17		
G	0.79	0.59	0.2		
H	1.59	1.19	0.4		
I	1.27	0.95	0.32		
J	2.2	1.65	0.55		
K	1.58	1.19	0.39		
L	2.02	1.51	0.51		
M	1.41	1.06	0.35		
N	0.48	0.35	0.13		
O	1.73	0.17	1.56		

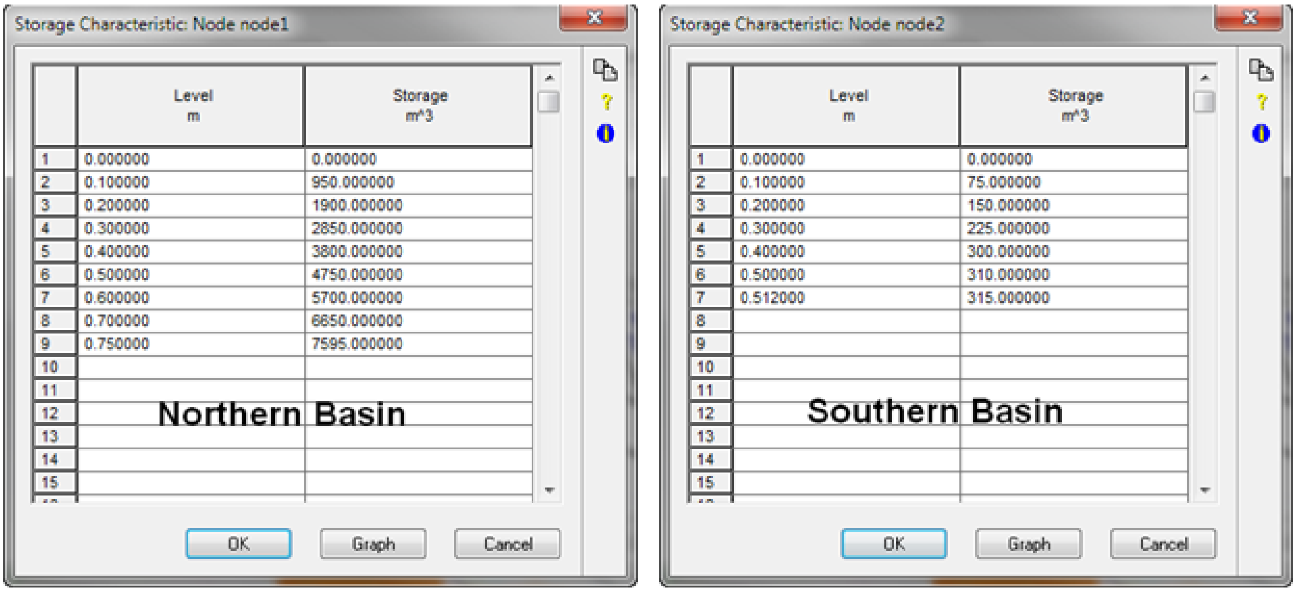
Catchment	Area (ha)			Slope (%)	PERN (Per/Imp)
	Total	Imp	Per		
P	1.03	0.78	0.25	2	0.035 / 0.016
Q	0.67	0.52	0.15		
R	1.77	1.37	0.4		
S	1.11	0.83	0.28		
T	0.56	0.42	0.14		
U	1.11	0.83	0.28		
V	1.12	0.84	0.28		
W	1.47	1.17	0.3		
X	0.41	0.31	0.1		
Y	1.15	0.86	0.29		
Z	1.13	0.85	0.28		
A1	0.87	0.65	0.22		
WW North	2.4	0	2.4		
WW South	2.2	0	2.2		

Detention basin designed to mitigate the stormwater runoff from the developed site were sized in the 1D hydrology model and analysed in the 2D hydraulic model.

4.4.8.1 Detention Basin Design

A detention basin stage-storage relationship was calculated to ensure a 'no worsening' of discharges from the developed site was achieved. The basin parameters were input into the XP-RAFTS program using the Retarding Basin feature.

The XP-RAFTS model was used to optimise detention basin performance and design. The stage-storage relationship of the required detention basins are shown in Figure 4.15.


Storage Characteristic: Node node1

	Level m	Storage m^3
1	0.000000	0.000000
2	0.100000	950.000000
3	0.200000	1900.000000
4	0.300000	2850.000000
5	0.400000	3800.000000
6	0.500000	4750.000000
7	0.600000	5700.000000
8	0.700000	6650.000000
9	0.750000	7595.000000

Northern Basin

Storage Characteristic: Node node2

	Level m	Storage m^3
1	0.000000	0.000000
2	0.100000	75.000000
3	0.200000	150.000000
4	0.300000	225.000000
5	0.400000	300.000000
6	0.500000	310.000000
7	0.512000	315.000000

Southern Basin

Figure 4.15: Detention Basin Stage Storage Relationship – XP-RAFTS model

4.4.9 Design Event Modelling

ARR design storm events were simulated in the XP-RAFTS hydrological model. Hydrological analysis was undertaken for the 1%, 2%, 5%, 10% and 20% and 50% AEP design storm events and resulting hydrographs were used to define the internal and external boundary conditions to the TUFLOW hydraulic model – refer Section 4.12 for more detail.

4.4.9.1 Critical Storm Duration

The critical duration for each design event probability and each sub-catchment may vary depending on a number of conditions. Therefore, consideration of a number of storm durations is important to ascertain the critical storm duration impacting the site and study area. The majority of the upstream contributing catchments is agricultural land and will typically be driven by the larger event durations; whereas the urban developed area in the lower catchments generate its peak discharge in the smaller duration events.

The critical storm duration for the upper catchment, the middle catchments (site) and lower urban catchments are depicted in the XP-RAFTS hydrographs outputs below.

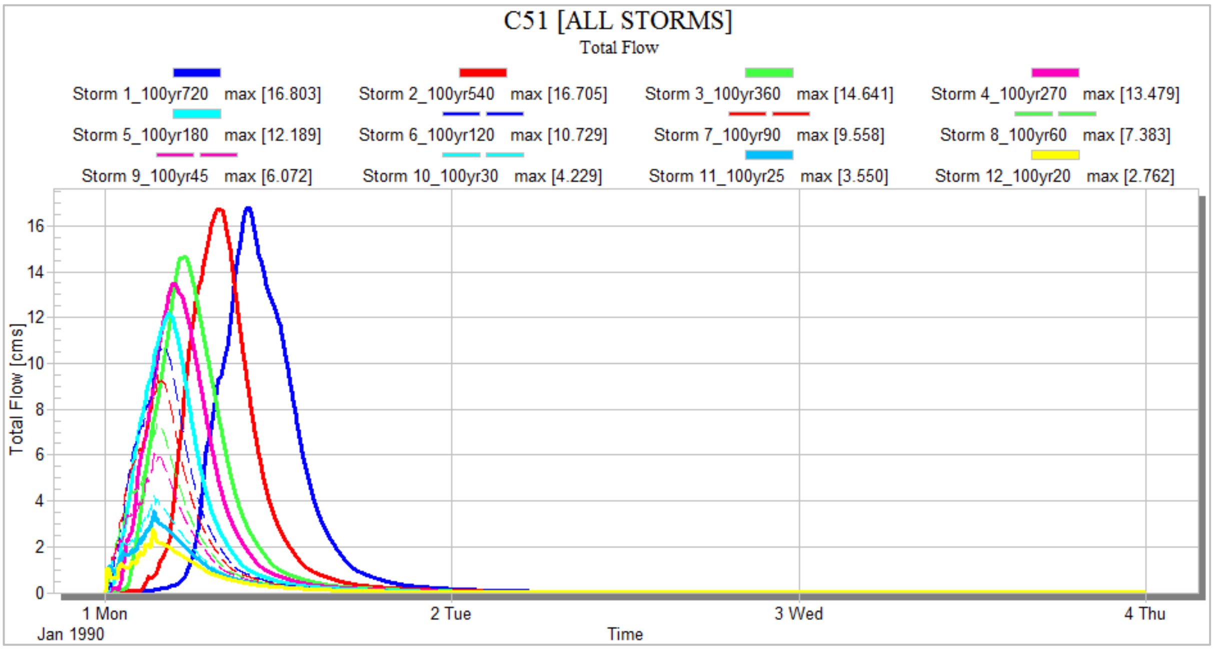


Figure 4.16: Design Storm Event hydrographs – Upper Catchment

The critical storm duration for runoff generated within the upper catchment (node C51) and conveyed within the waterway is the 720 minute (12 hour) event for the 1% AEP. It is noted that the critical duration of catchment clusters C45 and BC16 is 540 minutes (9 hours).

The critical duration for the undeveloped Ibbotson Street development site is 540 minutes (9 hour) and can be seen in Figure 4.17.

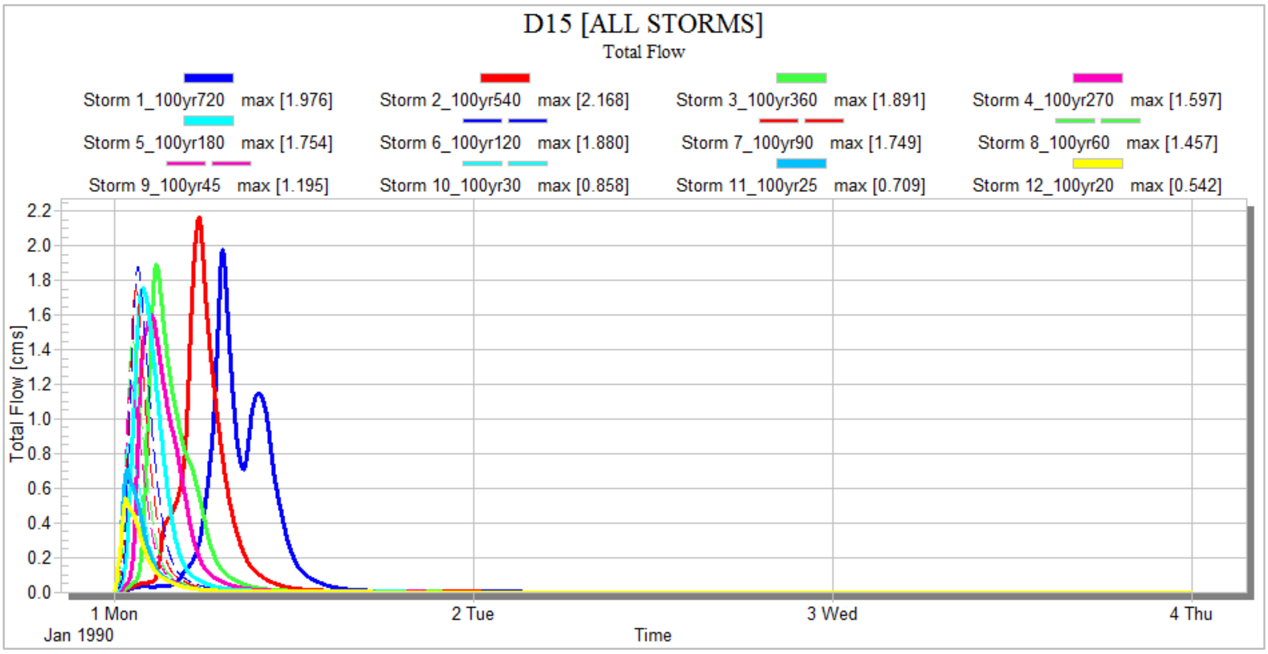


Figure 4.17: Design Storm Event hydrographs – Middle Catchment (Site)

The lower catchments are largely urbanised, resulting in lower critical storm duration, this is observed in Figure 4.18 below.

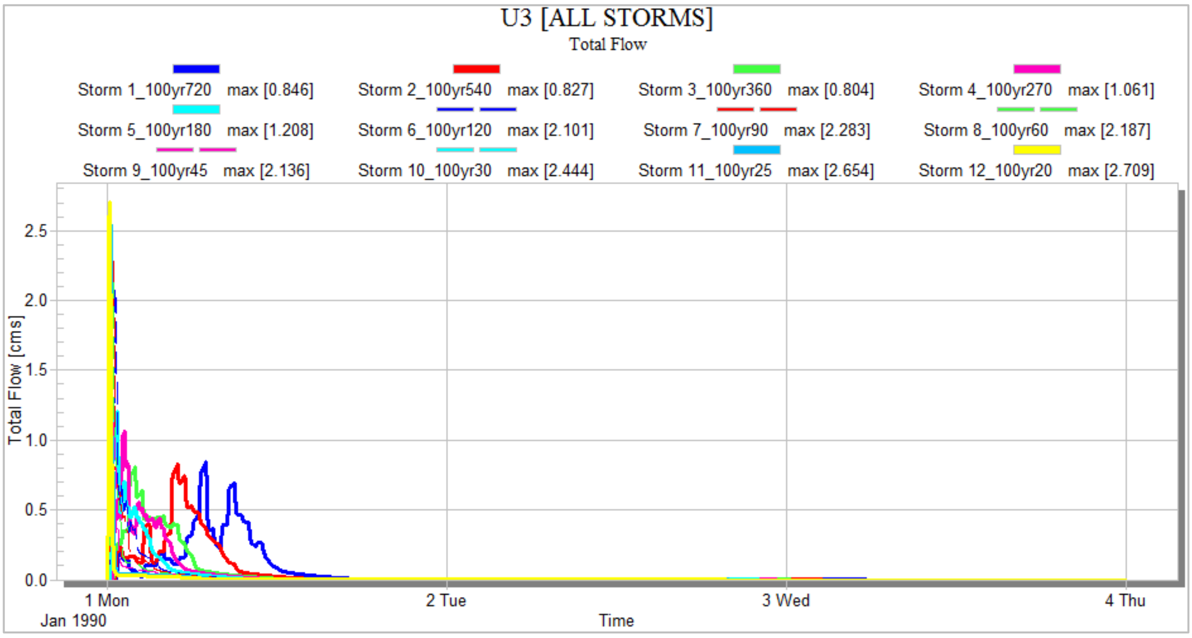


Figure 4.18: Design Storm Event hydrographs – Lower Urban Catchment

The critical storm duration for the urban catchments is the 20 minute design storm event. However, the volume of runoff during the 20 minute event, compared to the larger 540 and 720 minute events, is much smaller. The regional flood extents in the St Leonards Lake catchments are driven by volume and not peaks, therefore TGM has adopted the larger event durations generating the greater volumes for this analysis.

4.4.10 Extreme Design Storm Duration Sensitivity

As part of the sensitivity testing of the larger events, TGM Group also modelled a range of durations between 12 hours and 72 hours, the resulting hydrographs are shown in Figure 4.19.

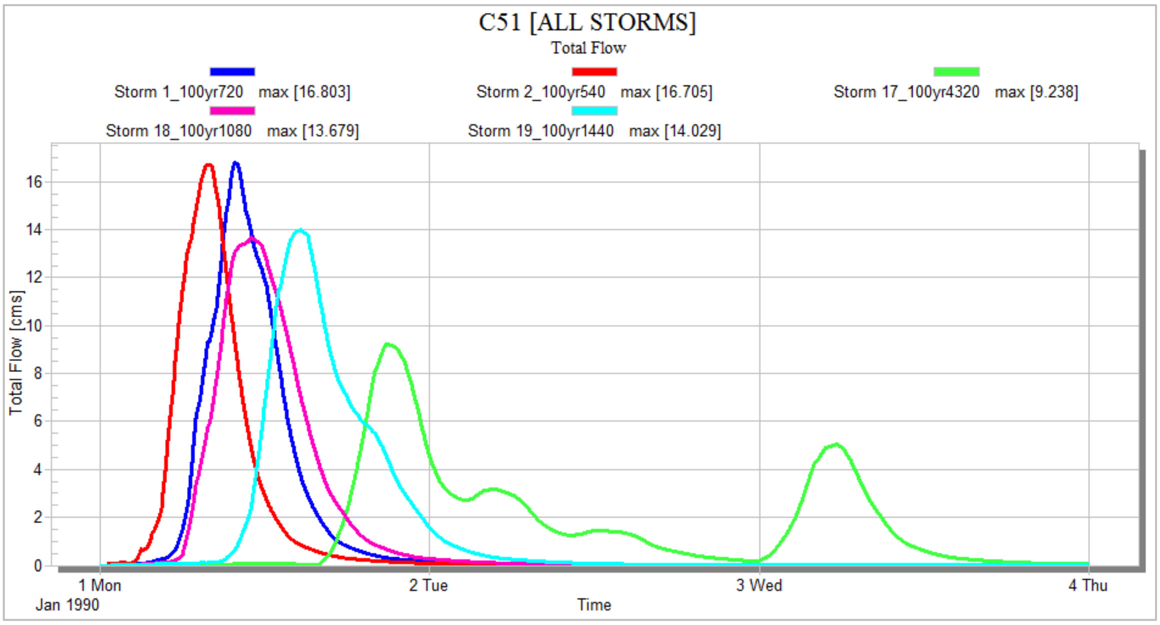


Figure 4.19: Sensitivity testing of extreme event durations

The critical storm duration for the upper catchments, as represented in Figure 4.19, is the 12 hour (720 minute) event.

4.5 Stormwater Quality

A comparative analysis of stormwater quality at the point of discharge from the Ibbotson Street subdivision site was integrated with the hydrological simulations. Discussions with COGG have identified the existing waterway discharging into Lake St Leonards as the legal point of discharge for stormwater runoff from the proposed development, prior to entering Port Phillip Bay; therefore the existing waterway was identified as the key reference location for assessment of stormwater quality targets.

Analysis of the impacts of urban development on receiving waterway ecosystem health was conducted using the continuous simulation model MUSIC from eWater CRC and the hydrological model. The MUSIC model was used to analyse the efficiency of the proposed stormwater system:

- Stormwater quality:
  - Total Suspended Solids;
  - Total Phosphorus;
  - Total Nitrogen;
  - Gross Pollutants;
  - Average annual runoff volumes; and
  - Frequency of stormwater runoff as indicated by average annual runoff days.

Stormwater quality measures were designed using MUSIC to meet "best practice" targets as described in Section 2. These design parameters serve the dual purpose of protecting waterway health and improving the amenity of waterways.

4.6 Design Surface

The proposed Design Surface (DS) reflecting the plan of subdivision was created using the 12D civil design package and civil design standards to input the finished grades, road alignments and stormwater flowpaths into the existing digital terrain model (DTM). The design surface DTM for the site is shown in Figure 4.20.

Please note that the design surface depicted throughout this report may be subject to some alterations to better meet those requirements of the community, developer and council as part of the detail design works.

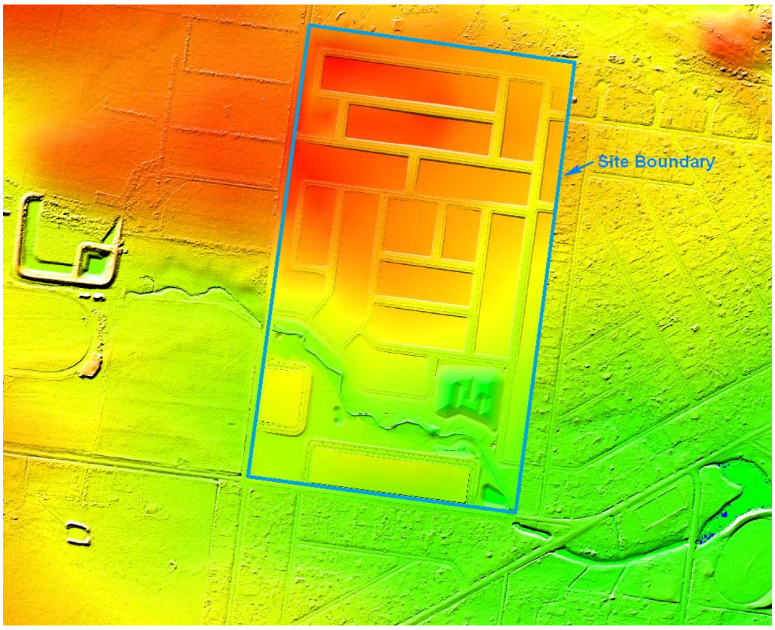


Figure 4.20: St Leonards Rezoning Proposed Design Surface

4.7 Hydraulic Modelling

The hydraulic analysis was undertaken using the fully 2D, TUFLOW (Build 2012-05-AE-iDP-w64) hydraulic modelling package. TUFLOW is a fully 2D hydraulic modelling package with the ability to dynamically integrate 1D elements.

Overland flow paths, obstructions and storages were modelled in the 2D domain, whilst underground drainage systems were represented as 1D elements linked to the 2D domain. The TUFLOW model was run in unsteady state.

4.7.1 Underground Drainage Network

The model incorporated part of the existing underground drainage network within St Leonards township. These components of the model were established using drainage network information provided by COGG.

The drainage network provided by COGG can be seen in Figure 4.21.

Figure 4.21: Existing St Leonards underground drainage network [GIS provided by COGG]

The extent of the drainage network incorporated into the analysis was directly related to its proximity and influence on flows impacting the site and within critical hydraulic locations with known flooding issues.

The drainage network modelled is shown in the below Figure 4.22.

Figure 4.22: Existing underground drainage network analysed

4.7.2 2D Domain Analytical Grid

The 2D domain was analysed using a 2 metre grid mesh. This enabled accurate definition of the variable terrain topography, road obstructions and defined waterways. This grid size was determined to be of optimum size so as to accurately define the terrain and not adversely affect the simulation run times.

Due to the flat, narrow and relatively undefined flow paths, 2d_zsh were embedded into the model to identify key features such as roads, bunds and swales, as identified in Figure 4.23.

Figure 4.23: 2d_zsh break lines (Ibbotson Street & Old St Leonards Road)

The 2d_zsh lines applied in Figure 4.22 define the crest, kerb and swales around Ibbotson Street and Old St Leonards Road intersection, allowing definition of an important flow path within the 2D model.

4.7.3 Manning's Coefficients

The TUFLOW hydraulic model assumed a Manning's roughness coefficient of 0.035 (pasture) for the majority of the catchment, with variation in roughness to reflect the vegetated waterway, urban development, gravel and bitumen road networks, existing water bodies and so forth, as depicted in Figure 4.23. The relevant roughness coefficients adopted are shown in Table 4-16.

Table 4-16: Materials layer – Manning's 'n' value

Surface Description	Manning's '*n*' value
Pasture	0.035
Asphalt	0.016
Gravel	0.029
Urban Residential (buildings)	0.15
Ponds and Water bodies	0.030
Light brush/scrub	0.050
Densely Vegetated Creek	0.10
Dense Riparian vegetation	0.20

Figure 4.24: Variation in Landuse Materials Layer – Study Catchment

4.7.4 Boundary conditions

The inflow hydrographs extracted from the hydrology model were distributed within the hydraulic model using the 2d_bc and 2d_sa functions in TUFLOW as inflow boundary conditions. The 2d_bc (QT) inputs were used for flows external to the model area and the 2d_sa and 2d_sa_pit inputs were used for internal catchments.

Model outlet conditions adopted a head vs flow (HQ) boundary conditions and head vs time (HT) boundary conditions where a tail water effect may occur. The HT boundary representing Port Phillip Bay was set to 1.4 m AHD[7] for 1% AEP storm tide under current climate conditions and 1.73 m AHD[8] for predicted 10% AEP 2100 storm tide conditions – this is discussed in Section 4.13.2.

The boundary condition locations are shown in Figure 4.24.

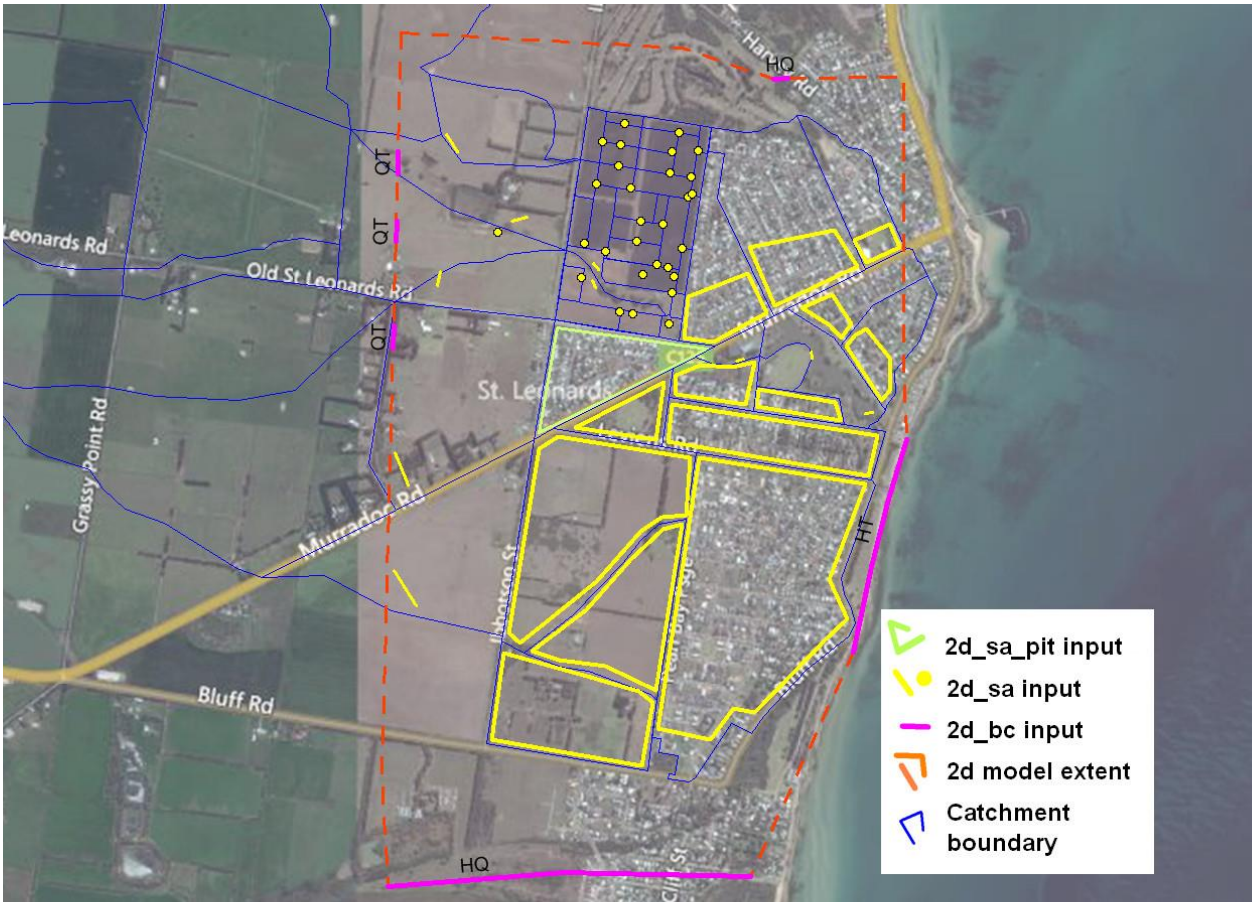


Figure 4.25: Boundary conditions

[7] AR4 A1FI - McInnes. KL, Macadam. I, O'Grady. J. *The Effect of Climate Change on Extreme Sea Level along Victoria's Coast*. Commonwealth Scientific Institute of Research Organisation. 2009

[8] Recommended level as provided by COGG/CCMA

4.8 Hydraulic Sensitivity

A sensitivity analysis was undertaken to assess how the variation in the hydrologic and hydraulic inputs and boundary conditions impact the output of the model. Reasons for undertaking a sensitivity analysis on this study are:

- To identify the factors which have the most influence on model output;
- To identify factors that are insignificant to model output and can be eliminated from further analysis;
- To identify which, if any, factors or groups of factors interact with each other;
- To establish whether model predictions are robust to plausible variations in input factors or, on the other hand, are strongly dependent on fragile assumptions.

The sensitivity analysis will assess physical hydraulic variations in the form of:

- Blockage to stormwater infrastructure; and
- Predicted storm tide influence from the Bay.

Each scenario is described below.

4.8.1 Blockage

Blockage factors have been applied to culverts and pits throughout the study area as part of the sensitivity analysis. The inclusion of blockage in the analysis of hydraulic structures is an important part of understanding the impact and severity of a flood event should the drainage system fail to operate as designed. It is important, however, to ensure the blockage factor estimations applied are not over-estimated or under-estimated as this can dramatically influence the performance of the total system.

For this study, TGM adopted current best practice procedures referenced in the AR&R Project 11 guidelines 'Blockage of Hydraulic Structures', for the selection of design blockage factors.

Design blockage is the blockage condition most likely to occur during a given design storm. It is an “average” of conditions to ensure that the calculated design flood levels should be appropriate for the defined probability.

This is necessary as the assumption of a higher level of blockage than appropriate would result in the calculated design flood level upstream of the structure being higher than would be appropriate for the defined probability. Downstream flood levels would be lower because of the additional flood storage upstream of the structure. On the other hand, an assumed lower level of blockage would result in lower flood levels upstream and higher flood levels downstream.

It is also noted that actual blockage levels vary greatly from event to event with a potential spread from “all clear” to “fully blocked” even in floods of comparable magnitude.

The blockage factors for this study were derived by assessing the following mechanisms of blockage in the contributing catchment:

- Debris availability;
- Debris mobility; and
- Debris transportability.

The contributing catchment consists predominantly of agricultural farmland and is generally devoid of sources of large debris. However, the cleared land and soil type results in high sediment loadings in stormwater runoff. The sediment collects in the drainage culverts and pits. This has resulted in extensive plant growth and blockages in several locations (Ibbotson Street culverts) see Figure 4.25.

Figure 4.26: Ibbtoson Street Culvert Inlet

Assessment of the contributing catchment area has led to the adoption of the following blockage factors:

Table 4-17: Blockage Factors

Blockage Conditions	
Design Blockage	**Severe Blockage**
20%	50%

4.8.2 Storm Tide

The term 'storm tide' refers to the combination of astronomical and meteorological processes forcing the mean sea level to increase. The meteorological processes are commonly referred as 'storm surge' and describe water level fluctuations associated with atmospheric pressure and wind setup.

Estimates of extreme coastal storm tides, including the impact of the projected sea level rise, have been developed (CSIRO 2008/09) for different planning and sea level rise scenarios. City of Greater Geelong and Corangamite Catchment Management Authority have nominated the following scenario to be assessed as part of this study.

Table 4-18: Storm tide Level Port Phillip Bay – St Leonards

Scenario (yr)	AEP (%)	Water Level (Heights relative to sea level)
2100	10	1.73

The level of 1.73 m AHD was used to set the downstream boundary condition (tail water) for these analytical scenarios.

5. RESULTS

The results of the stormwater hydrology and water quality analysis are shown in this section. The design has been undertaken to meet stormwater quality 'best practice' standards and to ensure that peak discharges from the 1% AEP storm event do not exceed existing conditions and/or create a negative impact to neighbouring properties and receiving ecosystems.

5.1 Stormwater Quality

The ability of development to meet stormwater quality 'best practice' standards and the performance of the treatment system was continuously simulated using MUSIC.

5.1.1 Wetland and Detention Basin

An end-of-line treatment train system in the form of a wetland, sedimentation and swale system to manage stormwater quality generated within the developed site is proposed to be utilised.

Wetland dimensions, required to meet best practice reductions, dictates the size and footprint of the regional detention basin. The required dimensions of the wetland component are tabulated below.

Table 5-1: Wetland Parameters – Northern

Parameter	Dimension
Area	5,350 m²
NTWL	3.3 m AHD
EDWL	3.7 m AHD

Table 5-2: Wetland Parameters – Southern

Parameter	Dimension
Area	610 m²
NTWL	3.55 m AHD
EDWL	3.75 m AHD

The MUSIC network for the RD-W Options is shown in Figure 5.1.

Figure 5.1: MUSIC network – Ibbotson Street

The treatment efficiency of the wetland system at the legal point of discharge prior to inlet into Lake St Leonards is shown in Table 5-3.

Table 5-3: Stormwater quality treatment efficiency

Treatment Train Efficiency			
Criteria	**Sources**	**Residual Load**	**Reduction (%)**
Flow (ML/yr)	102	87.4	14.2
Total Suspended Solids (kg/yr)	19400	3640	81.2
Total Phosphorus (kg/yr)	41	12	70.8
Total Nitrogen (kg/yr)	293	147	49.7
Gross Pollutants (kg/yr)	4440	929	93.4

5.2 Hydrology

Hydrology analysis is used to compare peak flow rates and runoff volumes.

Design storm events were used to evaluate stormwater peak discharges from the developed catchments in the Ibbotson Street site and to determine a detention basin capacity required to mitigate the stormwater flows.

Analysis of the undeveloped (Base Case – Existing Conditions) proposed site at Ibbotson Street highlights that peak discharge occurs at 9 hour storm duration. Developed conditions indicate a peak discharge at 20 minute storm duration for the proposed development.

The peak discharge generated for Base Case (Existing) will set the target objective for the developed scenario and is shown in Table 5-4.

Table 5-4: Ibbotson Street stormwater peak discharge objective

Base Case (Existing Conditions)
2.168 m³/s
9 hour storm duration

The existing conditions runoff hydrographs is shown in Figure 5.2.

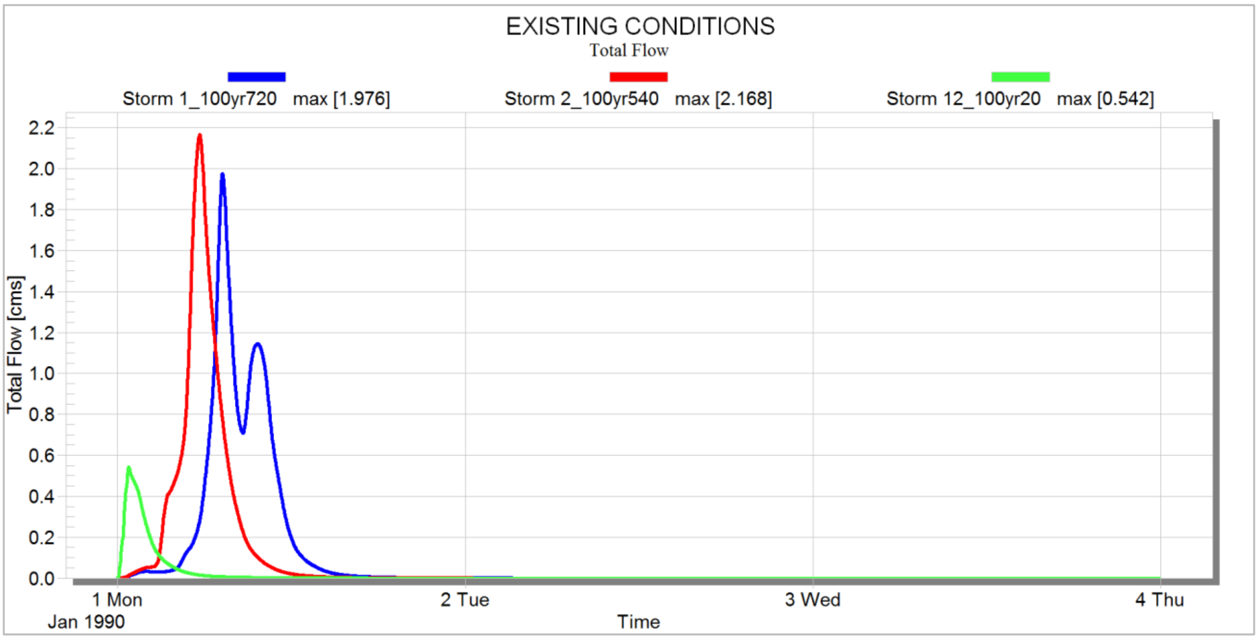


Figure 5.2: Existing Conditions runoff hydrograph – Site @ Ibbotson Street

The developed site peak discharges are shown in Table 5-5 for both the unmitigated and mitigated scenarios. The discharge hydrographs for the mitigated solution is shown in Figure 5.3.

Table 5-5: Ibbotson Street stormwater peak discharge - Developed

Developed Site (Unmitigated)	**Developed Site** (Mitigated)
9.162 m³/s	**2.165 m³/s**
20 minute storm duration	*12 hour storm duration*

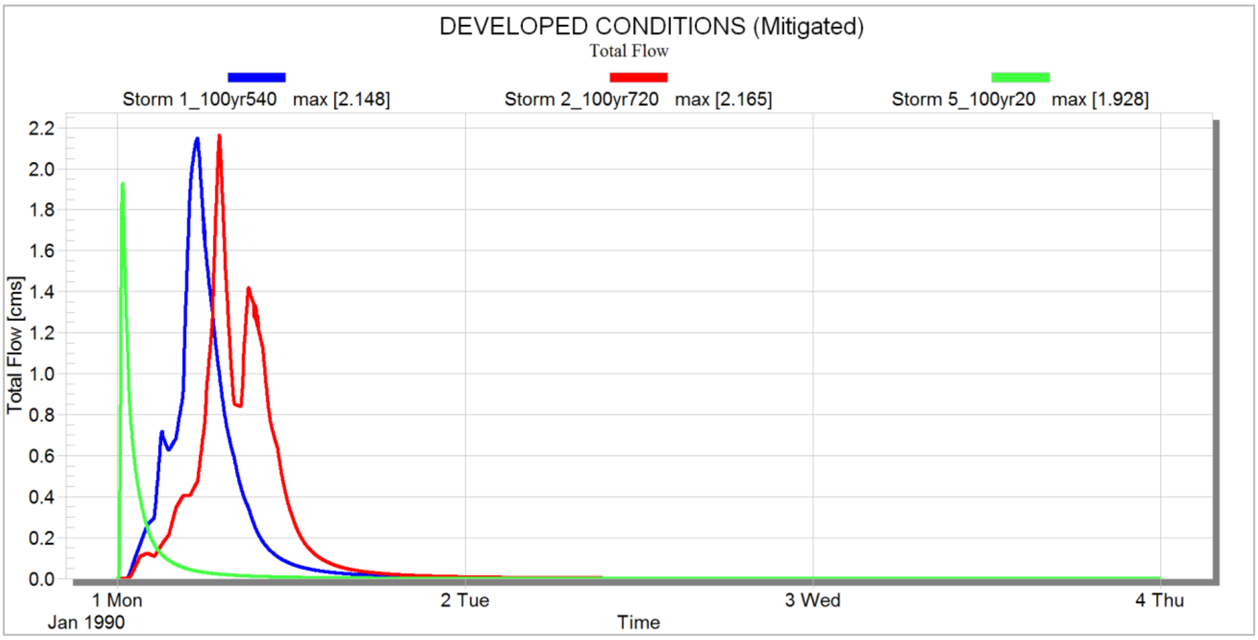


Figure 5.3: Developed Conditions runoff hydrograph – Site @ Ibbotson Street

There is an increase in stormwater runoff volumes and peak discharges as a result of the increase in impervious surfaces, resulting in a shorter duration and more intense runoff. The proposed development at Ibbotson Street will generate an increase in the peak stormwater discharges, unmitigated, of 6.99 m³/s and changes the critical storm duration from 9 hours to 20 minutes. The significant increase is largely due to the hydrological and topographical nature of the existing site and the characteristic change to developed/residential land use.

The stormwater peak discharge was mitigated using a detention basin to ensure no-worsening compared to existing conditions. The required detention volume is shown in Table 5-6.

Table 5-6: Detention basin capacity

Detention Basin	Required Volume
Northern	6,785 m³
Southern	600 m³

The wetland and detention facilities will be located either side of the waterway alignment, within the Green Corridor as shown in Figure 5.4 below.

Figure 5.4: Wetland and detention basin location within the ecological 'Green Corridor'

5.3 Hydraulics

The ability for the development to mitigate the impact of development on the regional flood extents is depicted in this section.

5.3.1 Hydraulic Simulation

The simulated existing 'base case' model returned a mass error (ME) of 0.18% - recommended mass balance error range for a healthy model is between +1% and -1%. The model experienced a maximum mass error of -1.68% at time 0, however, this had stabilized to an acceptable level within a few timesteps.

To maintain an acceptable courant number the model was run at a 1 second timestep which is half the grid cell size. TUFLOW manual recommends a 2D model timestep ½ to ⅕ of the grid cell size.

The TUFLOW simulation summary is shown in Figure 5.5.

```
-------------------------------
--    Simulation FINISHED    --
-------------------------------

                          PRIOR TO SIMULATION
  Number of WARNINGS:     _messages layer =      0     Total =       0
  Number of CHECKS:       _messages layer =      9     Total =       9

                           DURING SIMULATION
  Number of WARNINGS:     _messages layer =      0     Total =       0
  Number of CHECKS:       _messages layer =      0     Total =       0

SIMULATION SUMMARY

Input File: D:\005636 (St Leonards)\model\StLeonards_2m_303_EXIST.tcf
Log File: D:\005636 (St Leonards)\model\log\StLeonards_2m_303_EXIST.tlf

Start Time (h): 0.
End Time (h):   17.
Computational Steps (based on largest 2D timestep): 61200

CPU Time (hh:mm:ss):       15:58:39 or 15.98 hours
Clock Time (hh:mm:ss):     15:59:25 or 15.99 hours

Simulation FINISHED

Total 1D Negative Depths: 0
Total 2D Negative Depths: 0

WARNINGS prior to simulation:     0  [0 not in _messages layer]
WARNINGS during simulation:       0  [0 not in _messages layer]
CHECKS prior to simulation:       9  [0 not in _messages layer]
CHECKS during simulation:         0  [0 not in _messages layer]

Peak Flow In (m3/s):   44.8 at Time 6.33
Peak Flow Out (m3/s):  38.9 at Time 8.06
Volume at Start (m3):  15
Volume at End (m3):    56180
Total Volume In (m3):  913300
Total Volume Out (m3): 860357
Volume Error (m3):     3222 or 0.2% of Volume In + Out
Final Cumulative ME:   0.18%

                                    Whole Simulation          Qi+Qo > 5%
Peak +ve dV (m3):                    43.3 at    0.24h       43.3 at    0.24h
Peak -ve dV (m3):                   -17.0 at    0.33h      -17.0 at    0.33h
Peak ddV over one timestep:          -3.1 at    0.28h        2.4 at    0.40h
Peak ddV as a % of peak dV:           7.1%                   5.5%
Peak Cumulative ME:                 -1.68% at    0.00h      0.56% at    0.26h
```

Figure 5.5: TUFLOW simulation summary (.tlf)

5.3.2 Base Case – Existing Conditions

The existing conditions 1% AEP full flood extent is depicted in Figure 5.6 below.

Figure 5.6: Existing 1% AEP Flood (Full Extent)

Representation of the full extent is useful for identifying flow paths and cross catchment flows, however these should not be used for planning purposes. The existing flood extents, with depths greater or equal to 50 mm, are depicted in Figure 5.7 and constitute the 1% AEP flood extent appropriate to define the planning flood boundary.

Figure 5.7: Existing 1% AEP Flood (≥ 50 mm)

5.3.3 Developed Conditions

The impact of the development on regional flooding needs to be assessed following confirmation of the ultimate development masterplan.

5.3.4 Hazard Mapping – Velocity x Depth

The hazard mapping was undertaken using the methodology in ARR Project 10 – '*Appropriate Safety Criteria for People'* and Melbourne Water's '*Guidelines for Development in Flood-Prone Areas'*. The hazard analysis is used to determine whether it is safe for people to move during a flood event. 'Safety' is defined by the product of Velocity and Depth of flood waters. The hazard regime is identified in Table 5-7.

Table 5-7: Flow hazard regimes for infants, children and adults[9]

DV (m^2s^{-1})	Infants, small children (H.M ≤ 25) and frail/older persons	Children (H.M = 25 to 50)	Adults (H.M > 50)
0	Safe	Safe	Safe
0 – 0.4	Extreme Hazard; Dangerous to all	Low Hazard[1]	Low Hazard[1]
0.4 – 0.6		Significant Hazard; Dangerous to most	
0.6 – 0.8		Extreme Hazard; Dangerous to all	Moderate Hazard; Dangerous to some[2]
0.8 – 1.2			Significant Hazard; Dangerous to most[3]
> 1.2			Extreme Hazard; Dangerous to all

[1] Stability uncompromised for persons within laboratory testing program at these flows (to maximum flow depth of 0.5 m for children and 1.2 m for adults and a maximum velocity of 3.0 ms^{-1} at shallow depths).
[2] Working limit for trained safety workers or experienced and well equipped persons (D.V < 0.8 m^2s^{-1})
[3] Upper limit of stability observed during most investigations (D.V > 1.2 m^2s^{-1})

The colour scheme observed in Table 5-7, is represented in the following Hazards maps, for Existing conditions.

[9] Cox, R.J., Shand, T.D and Blacka, M.J. (2010), Appropriate Safety Criteria for People in Floods. Report for Institution of Engineers Australia, Australian Rainfall and Runoff Guidelines: Project 10.

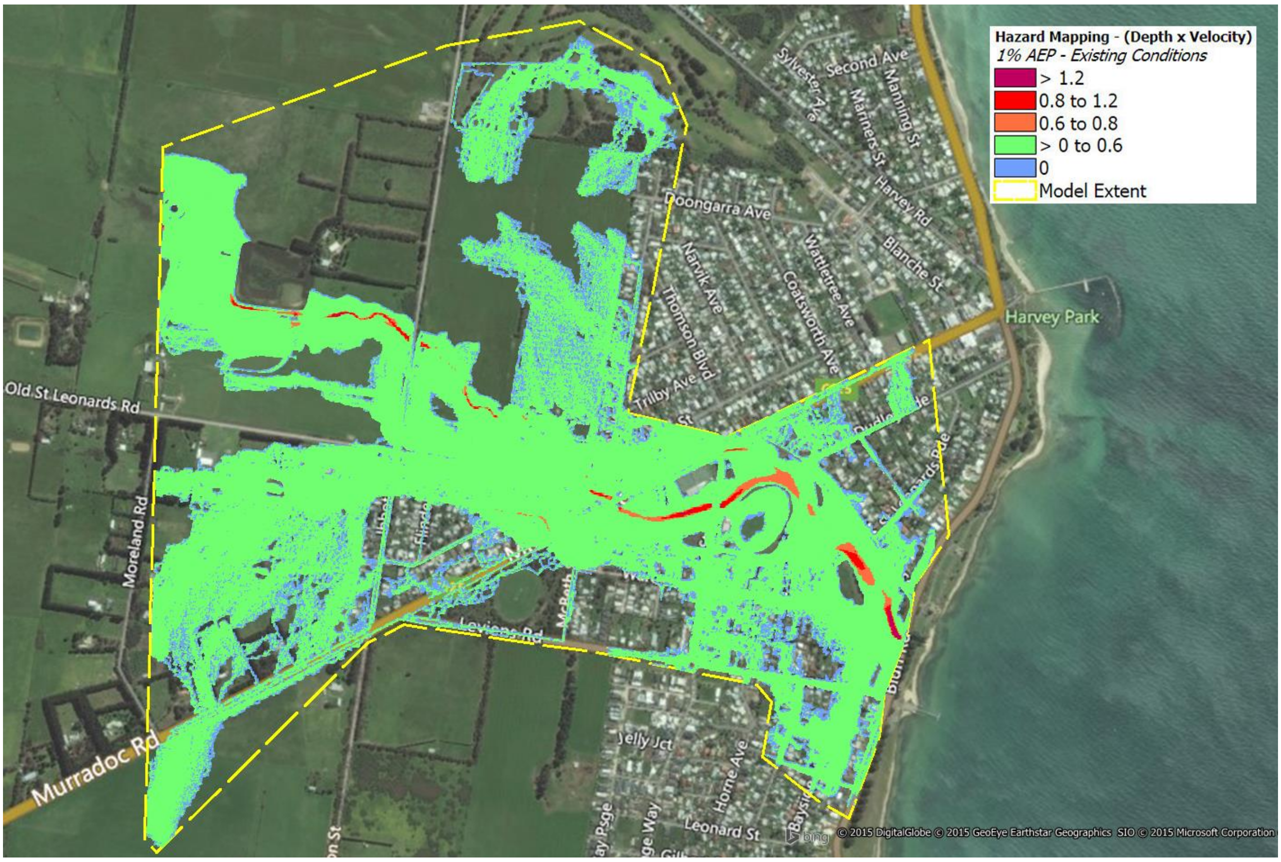


Figure 5.8: Hazard Map – Existing Conditions

Figure 5.8 indicates that, under existing conditions, flood waters within the subject area meet the safety criteria for the movement of people. The locations exhibiting a 'Moderate' to 'Extreme' hazard are contained within the waterways and Lake.

As per ARR Project 10, Developments should not occur where depth and flow of flood water on a property will be hazardous. Safety is defined in terms of the depth, velocity and the product of the depth multiplied by velocity.

Safety criteria during a 1% AEP flood event are:

1. For People
   - Depth must be no greater than or equal to 0.5 metres;
   - Velocity must be no greater than or equal to 3.0 m/s; and
   - The product of depth by velocity must be no greater than or equal to 0.4 m^2/s.
2. For Vehicles
   - Depth must be no greater than or equal to 0.3 metres;
   - Velocity must be no greater than or equal to 3.0 m/s; and
   - The product of depth by velocity must be no greater than or equal to 0.3 m^2/s.

The following flood depth safety map (Figure 5.9) identifies the hazard areas.

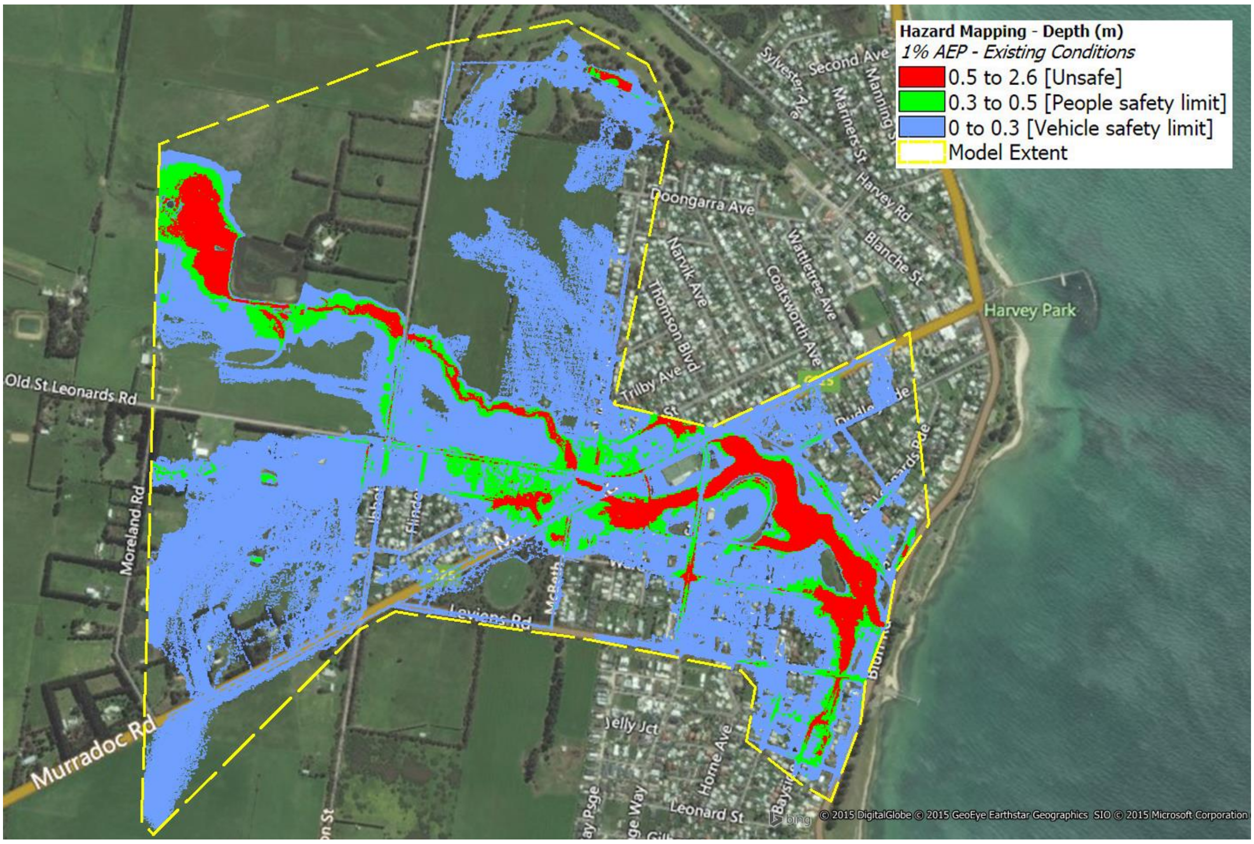


Figure 5.9: Depth Safety Map – Existing Conditions

Figure 5.7 indicates that the 'unsafe' areas within the site are contained within the waterway alignment. Poor drainage systems within the existing St Leonards residential area results in depths considered 'unsafe', future development will have to ensure these problems area aren't exasperated. The Hazard maps will form the 'base case' to assess the suitability of the design development.

5.4 Hydraulic Sensitivity Results

To be undertaken as part of 'Final Report'.

6. CONCLUSION

The stormwater management section of the report proves that stormwater runoff generated within the fully developed 321-399 Ibbotson Street site in St Leonards can be mitigated to meet 'best-practice' objectives in water quality and a 'no-worsening' of peak discharges from the site.

The continuous simulation model MUSIC was used to design and confirm the mitigation required to meet water quality discharge objectives. Water quantity mitigation was designed using XP-RAFTS.

A full and proper hydrological and hydraulic flood investigation was undertaken applying the latest best practice analytical techniques, research and science to define the 1% AEP flood for St Leonards Lake. The existing 1% AEP flood identified in this study will form the 'base case' for future flood impact assessments of the proposed Ibbotson Street development.

The fully two-dimensional TUFLOW model was developed, applying hydrological inputs from XP-RAFTS to define the existing 1% AEP flooding, simulate the design mitigation options and ensure that the proposed residential development at Ibbotson Street, St Leonards has no significant impacts on flood characteristics external to the site and to ensure that the development meets specific objectives for safety and egress during floods.

The flood risk assessment identified that the development site is affected by the 1% AEP flood event. The site is divided into two (2) sectors (north and south) by the existing waterway. Land north of the waterway is not subject to flooding outside the waterway corridor and can be developed without the requirement for regional flood mitigation.

Land south of the waterway is subject to inundation under existing conditions. Flood mitigation and a detailed flood impact assessment will be required to reclaim developable land.

Flood risks, associated with safety hazards, are relatively low as the '*unsafe'* characteristics are contained with the existing waterway.

The analysis undertaken in this study has demonstrated that the proposed development north of the waterway can be constructed and meet the requirements and objectives for site stormwater management, although further detailed analysis is required to assess potential to develop south of the waterway and the flood impact and safety performance of the development during regional flood events.

7. REFERENCES

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