Glismann Road Drainage Scheme

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PROJECT DETAILS

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GLOSSARY OF TERMS

Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level. Introduced in 1971 to eventually supersede all earlier datums.
Average Recurrence Interval (ARI)	Refers to the average time interval between a given flood magnitude occurring or being exceeded. A 10 year ARI flood is expected to be exceeded on average once every 10 years. A 100 year ARI flood is expected to be exceeded on average once every 100 years.
Catchment	The area draining to a site. Generally relates to a particular location and may include the catchments of tributary streams as well as the main stream.
Design flood	A significant event to be considered in the design process; various works within the floodplain may have different design standards. A design flood will generally have a nominated AEP or ARI (see above).
Discharge	The rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow, which is a measure of how fast the water is moving rather than how much is moving.
Top Extended Detention Depth (TED)	Maximum depth of water ponding above the permanent pool in the wetland or sedimentation basin, before flow starts to discharge over the outflow weir.
Flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or overland runoff before entering a watercourse and/or coastal inundation resulting from elevated sea levels and/or waves overtopping coastline defences.
Flood hazard	Potential risk to life and limb caused by flooding. Flood hazard combines the flood depth and velocity.
Flood mitigation	A series of works to prevent or reduce the impact of flooding. This includes structural options such as levees and non-structural options such as planning schemes and flood warning systems.
Floodplain	Area of land which is subject to inundation by floods up to the probable maximum flood event, i.e. flood prone land.
Fraction Impervious	The ratio of impermeable surface area to permeable surface area. A fraction impervious of 1 equals a totally impervious surface.
Freeboard	A factor of safety above design flood levels typically used in relation to the setting of floor levels or crest heights of flood levees. It is usually expressed as a height above the level of the design flood event.
Hydraulics	The term given to the study of water flow in a river, channel or pipe, in particular, the evaluation of flow parameters such as stage and velocity.
Hydrograph	A graph that shows how the discharge changes with time at any particular location.
Hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
HY-8	A hydraulic modelling tool used in this study to undertake culvert computations and assess the performance of culverts.

Normal Water Level (NWL) Level at which water should sit within a sediment pond or wetland in most instances.

Permanent Pool Depth Depth of the permanent pool of water in the sediment pond or wetland.

Retarding Basin Holding basin for the temporary storage of floodwaters during the passage of a flood.

Sediment Pond Sediment ponds (also called sediment basins) are WSUD (see below) water bodies designed to remove sediments by providing temporary stormwater detention.

TUFLOW A hydraulic modelling tool used in this study to simulate the flow of flood water through the floodplain. The model uses numerical equations to describe the water movement.

Peak flow The maximum discharge occurring during a flood event.

RORB A hydrological modelling tool used in this study to calculate the runoff generated for design rainfall events.

Runoff The amount of rainfall that actually ends up as stream or pipe flow, also known as rainfall excess.

Water Sensitive Urban Design (WSUD) An approach to the planning and design of urban environments that supports healthy ecosystems, lifestyles and livelihoods through smart management of all our waters.

Wetland Wetlands are shallow, vegetated WSUD assets which treat stormwater by allowing for sedimentation, filtration and biological uptake.

Wetland Detention Time The time a particle of water spends in the wetland.

1D (one dimensional) Refers to hydraulic modelling where the flow is represented in one dimension (i.e. the direction of flow). Typically used where the primary direction of the flow is known.

2D (two dimensional) Refers to hydraulic modelling where the direction of flow is variable and/or complex. Often used where the flow is not confined to a waterway and the direction and velocity is influenced by features of the floodplain. Using a grid of the topography, the model will estimate not only how high and how fast water will flow but also calculate the direction of flow across the grid.

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1. INTRODUCTION

Water Technology has been engaged by Cardinia Shire council to establish a drainage strategy for the proposed future development of the Glismann Road precinct in Beaconsfield. The location of the study area is shown in Figure 1-1. This report covers the hydrology, hydraulics and water quality assessments, and outlines the scheme works required to cater for future developments in the area.

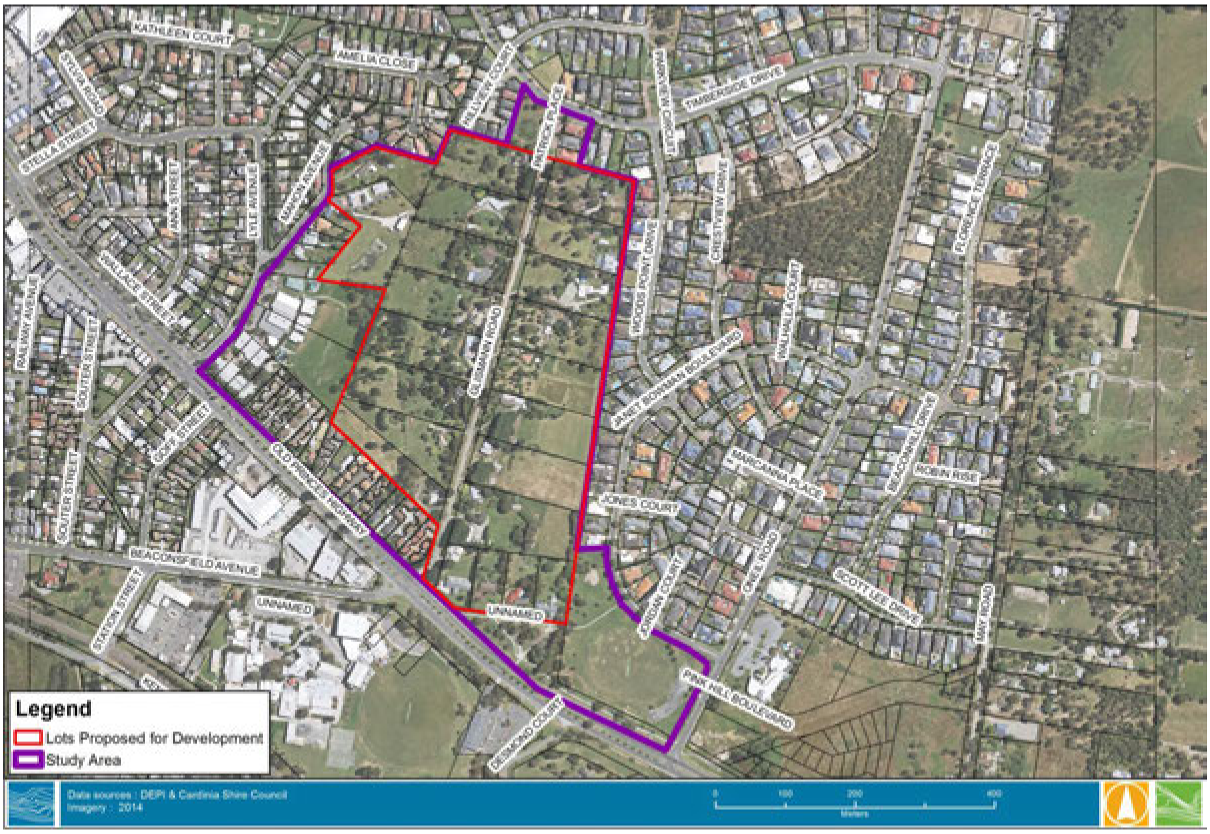

Figure 1-1 Study Area

1.1 Scope of Works

The scope of works for the investigations comprised of the following:

- Review of relevant data and previous studies;
- Review of future development areas;
- Hydrological modelling:
    - Assess impacts of the future developments on flows within the catchment; and
    - Sizing of storages to ensure that flows at the catchment outlet are retarded back to existing conditions.
- Water Quality modelling:
    - Identify and size water quality works for the development areas.
- Drainage Design:
    - Locate and size stormwater pipes for new development areas;
    - Connectivity of recommended drainage to the existing drainage system;
    - Identify overland flow paths; and
    - Reservation of flood prone areas.
- Liaison with Cardinia Shire Council and Melbourne Water.

2. CATCHMENT DESCRIPTION

The 33 hectare ('Ha') study area is located to the east and west of Glismann Road in Beaconsfield. The study area was originally part of Melbourne Water's O'Neill Road Drainage Scheme.

The focus within the study area is on 22 rural living zone parcels (lot sizes range from 0.4 Ha to 1.2 Ha) that are earmarked for possible future development. The rural zoned lots are currently occupied by individual dwellings. Other key features within the study area include:

- Beaconsfield Primary School to the west;
- Existing residential units to the south (along Old Princes Highway);
- Open Space in the south-east corner, potentially vacant Crown land; and
- Existing water body to the north, at the intersection of Timberside Drive and Patrick Place. This is believed to be a retarding basin servicing the external residential catchment to the north.

The study area comprises of steep terrain (slopes of 20%) in the northern portion of the site with relatively flat land (slopes 2-3%) in the southern section of the site. Two distinct catchments (Figure 2-1) exist within the study area, with flows from external residential areas draining into the site.

The trunk drainage system through the catchment is a Melbourne Water asset. The catchments drain (via a pipe network) into a designated waterway south of old Princes Highway. The waterway ultimately discharges into Cardinia Creek.

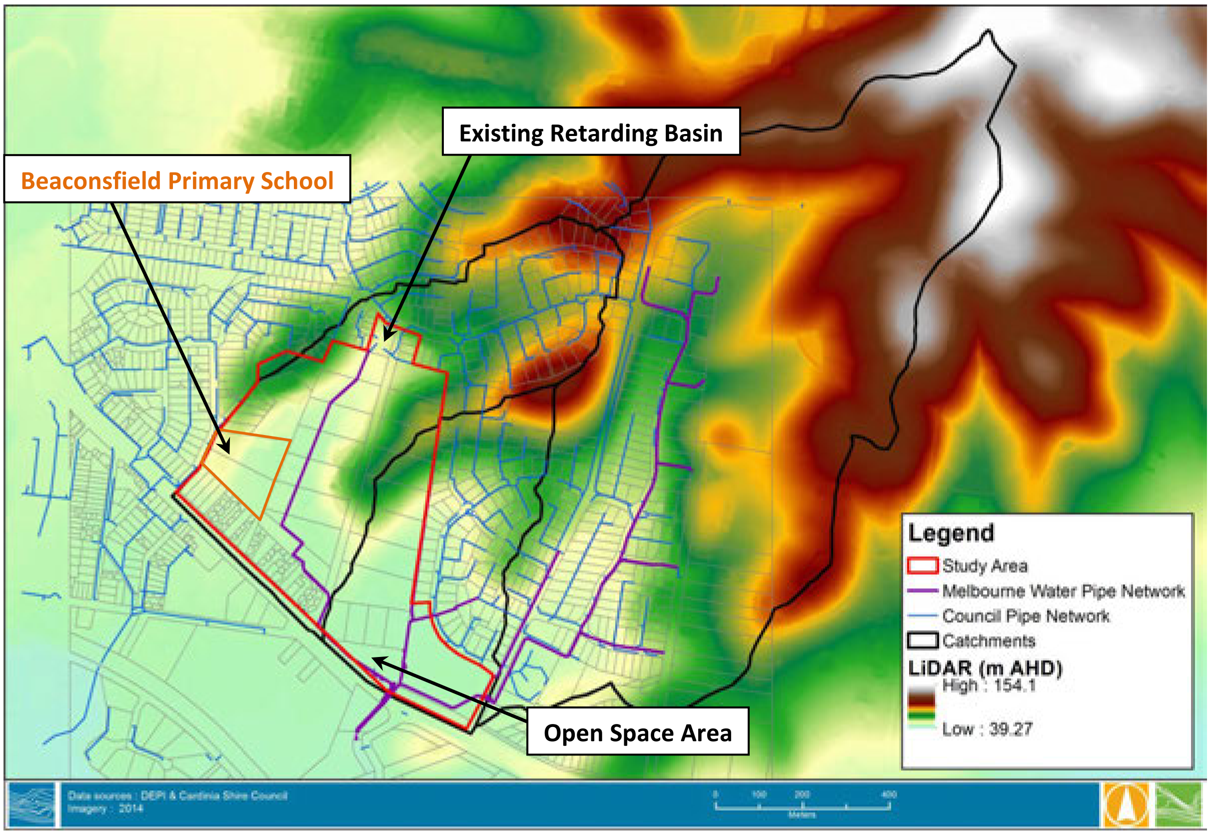

Figure 2-1 Catchment Map and Drainage Network

2.1 Future Development

The Glismann Road precinct has been identified as a potential area for future residential development. Any development will be managed through a development plan overlay (master plan) developed in consultation with the local community.

At the time of this report the preferred development layout was not available. As such, preliminary concept development layout and development densities, based on discussions with council, were used. The concept development plan consists of higher density residential areas (200 - 400 m^2 lots) to the south, conventional density lots (700 - 1000 m^2 lots) in the north-west and larger lots (> 1000 m^2 lots) and an open space area in the north-east.

Fraction impervious (FI) values were calculated for each individual lot based on the adopted FI values shown in Table 2-1. Examples of typical FI values are also provided in Table 2-1 for comparison. Conservative (higher) FI values were applied to the residential land types to allow for any future changes to the FI values of the site such as changes to the development density and the location of connection roads through the development.

Table 2-1 Fraction Impervious Values for the Development Areas

Land Use	Adopted Fraction Impervious Values	Typical Value
High density residential (200 – 400 m^2)	0.85	0.8
Conventional density residential (700 – 1000 m^2)	0.6	0.5
Low density residential (> 1000 m^2)	0.49	0.45
Open space	0.1	0.1

Locations, lot number and a thematic developed FI map of properties proposed for development under the drainage scheme are shown in Figure 2-2 below.

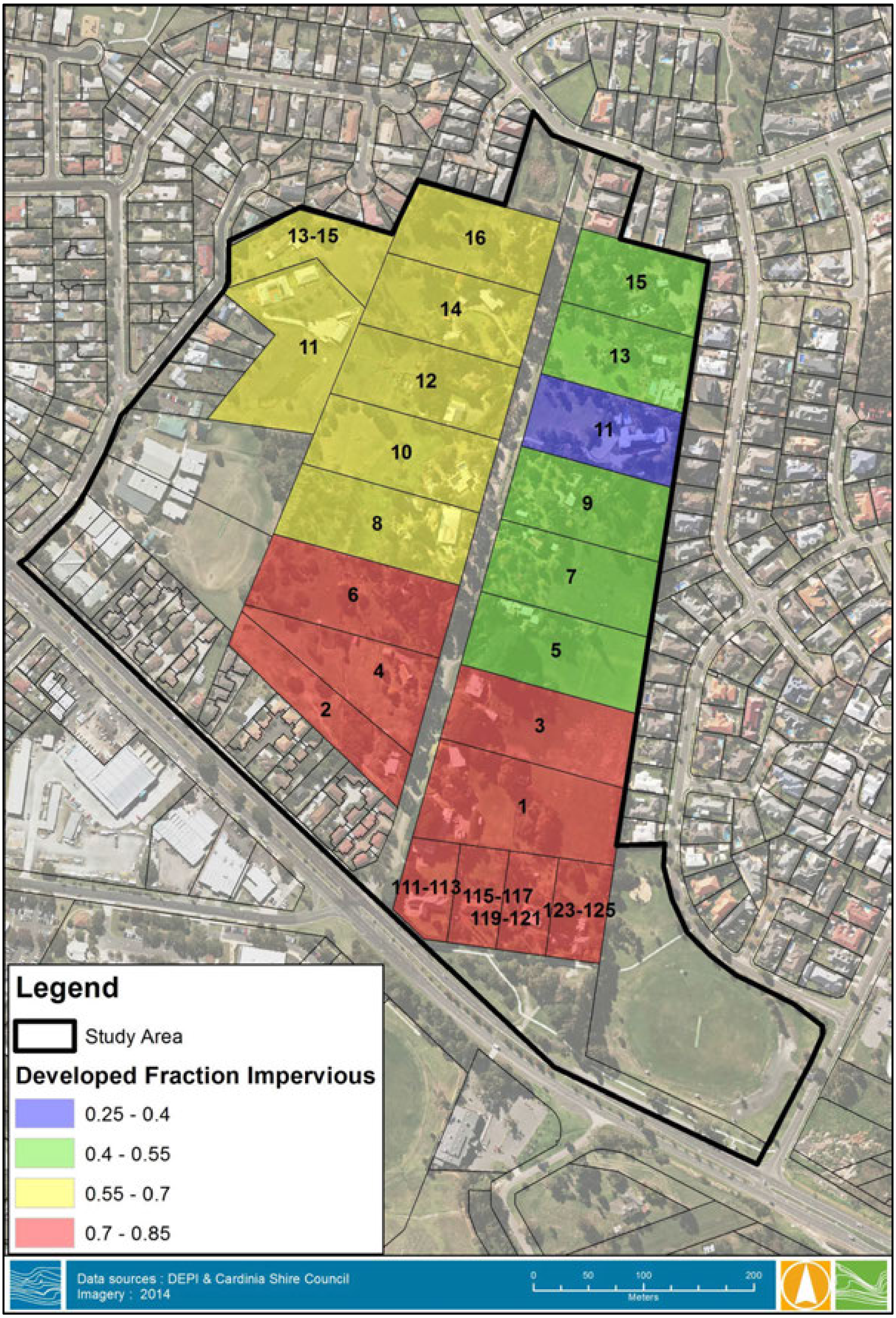

Figure 2-2 Location of Drainage Scheme Properties and Developed FI Map

3. STUDY HYDROLOGY

3.1 Overview

The hydrologic analysis of the site was undertaken using the runoff routing program RORB. Models were run for existing, developed and mitigated conditions to determine flow hydrographs across the site under each condition and to size the flood retention features. Details of the RORB model setup are provided in Appendix A.

3.2 Existing Conditions

Under existing conditions, discharges from the entire site and the external catchments make their way to the culverts under the Old Princes Highway. The discharges are conveyed to this point via the council/Melbourne Water pipe network and overland flow paths.

The flows exiting the study area are controlled by the capacity of the pipes under the highway. The pipe arrangement, on the upstream side of the highway, consists of a 1,500 mm reinforced concrete pipe (RCP) at an invert level of 46.01 m AHD, and a further two 1500 mm RCP culverts at an invert level of 47.22 m AHD. The peak 100 year ARI flow exiting the culverts on the downstream side of the highway is approximately 12.6 m^3/s.

Flows at key locations across the study area as determined from the RORB modelling are shown in Figure 3-1 below.

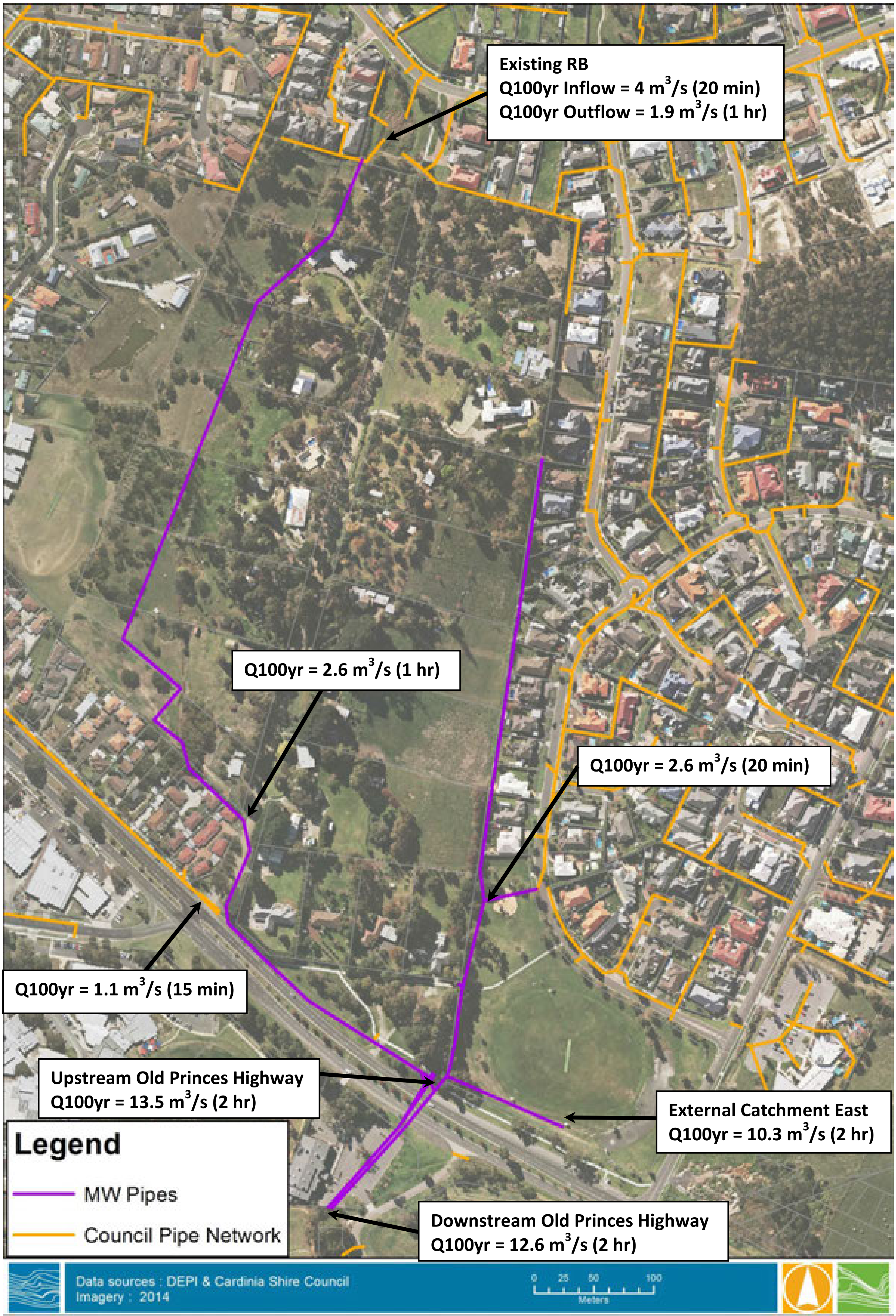

Figure 3-1 Peak 100 Year ARI Flows under Existing Conditions

3.3 Changes in Peak Flows under Future Developed Conditions

Peak flows under existing conditions and future developed conditions are shown in Table 3-1. As expected, the future developments have resulted in an increase in peak flows leaving the study area and also an increase in the peak water level upstream of the Old Princes Highway. Despite the increase in peak water level on the north side of the highway, the highway is not overtopped under developed conditions.

Table 3-1 Existing and Future Developed Conditions Peak Flow

Location	Existing Flow (m^3/s)	Future Developed Flow (m^3/s)
Downstream Old Princes Highway	12.6 (2 hr)	15.2 (2 hr)

3.4 Future Conditions Flow Mitigation

Proposed Retarding Basin Location and Configuration

It is proposed to mitigate future developed peak flows leaving the catchment by utilising the open space area in the triangular parcel of land upstream of the highway. The use of this land for retardation and WSUD features provides the opportunity to retain the more usable land within the site for development. This location is also favourable from a drainage point as it is located at the downstream end of the site and is adjacent to the culverts under the highway.

The location of the proposed basin is shown in Figure 3-2. The proposed basin will receive major inflows from the western catchment across the development site. Flows from the smaller eastern catchment across the development site and the external catchment to the east will bypass the retarding basin.

To achieve the required peak flow retardation, a storage volume of approximately 7,500 m^3 would need to be provided within the reserved area. This volume can be met by excavating an area of approximately 6,400 m^2 (including batters) above 47 m AHD (the wetland normal water level). The basin outlet pipe (1,050 mm diameter) should be set at 46.25 m AHD and be connected to the pipe under the highway. The basin outlet pipe level was set at 46.25 m AHD to allow for maintenance drawdown in the proposed wetland located at the base of the retarding basin. A small 5 m wide spillway along the south east corner of the basin has been set at 48 m AHD to direct overflows into adjacent channel. Details of the basin outlet structure are shown in Figure 3-3.

The hydraulic control for the proposed basin is the basin outlet pipe (which will be drowned out in the 100 year ARI event) and the existing pipes under the Old Princes Highway. The discharge relationship for the basin’s outlet structure was calculated using HY-8, a hydraulic culvert calculator.

The proposed retarding basin successfully mitigates peak development flows leaving the catchment back to existing conditions (12.6 m^3/s), as shown in Table 3-2. The 4.5 hour duration event was found to be critical for flood storage in the proposed basin, with a peak storage volume of 7,510 m^3 and peak water level of 48.77 m AHD.

A 10 m buffer has been provided between the basin and the lots to the north. The buffer can be used for maintenance and access to the existing properties.

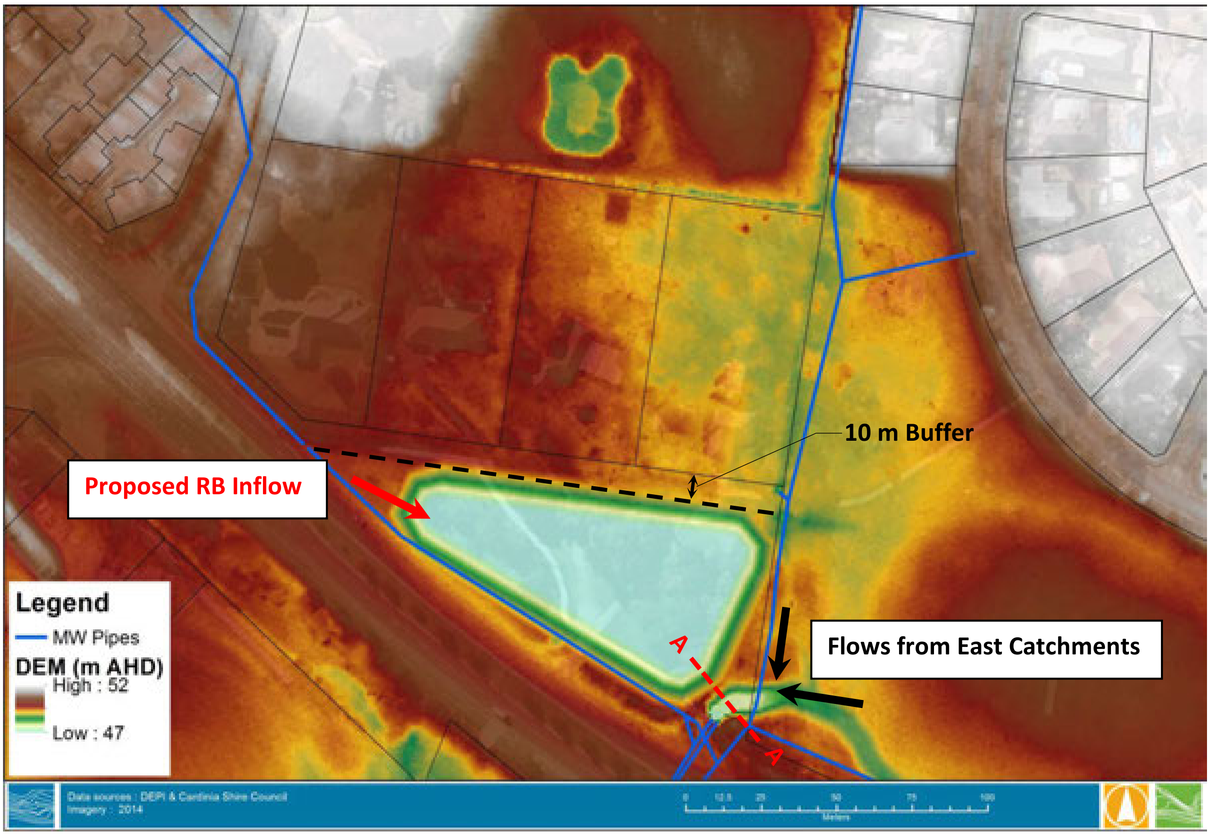

Figure 3-2 Location of the Proposed Retarding Basin

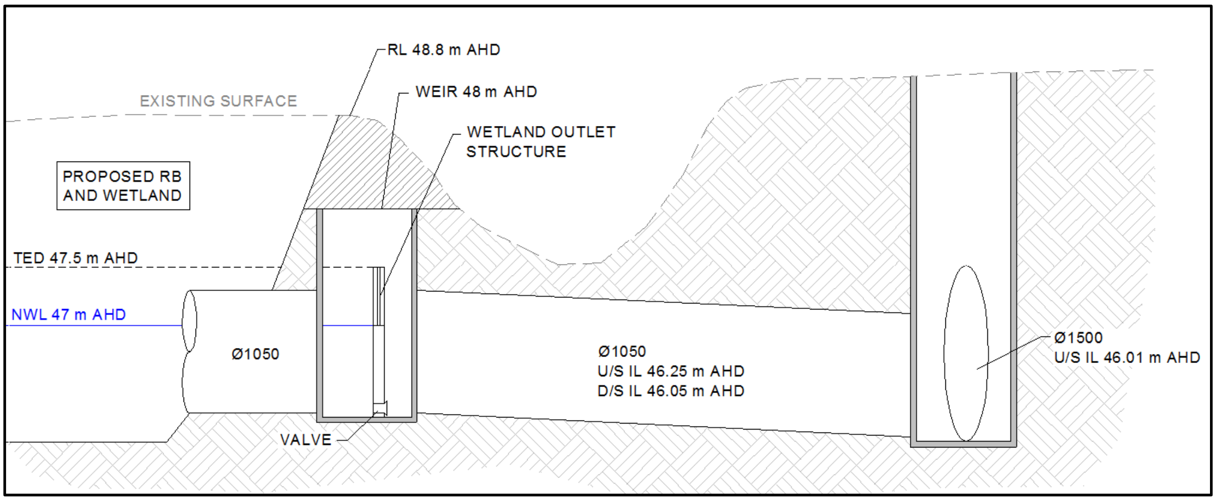

Figure 3-3 Proposed Basin Outlet Structure (Section A-A) – Not to Scale

Table 3-2 Performance of the Proposed Retarding Basin

Duration	Proposed RB Inflow (m^3/s)	Proposed RB Outflow (m^3/s)	Flow from East Catchments (m^3/s)	Flow Upstream Highway (m^3/s)	Flow Downstream Highway (m^3/s)
10m	5.0	0.9	9.8	9.9	9.6
15m	5.7	1.3	11.5	11.5	11.4
20m	6.1	1.5	12.7	12.7	11.6
25m	5.9	1.8	12.2	12.2	12.0
30m	5.6	1.9	11.5	11.5	11.4
45m	5.3	2.2	10.8	10.9	10.8
1h	5.6	2.4	11.7	12.6	12.1
1.5h	5.5	2.4	11.4	12.4	12.0
2h	5.8	2.5	12.6	13.6	**12.6**
3h	4.2	2.4	8.7	10.2	10.0
4.5h	4.1	2.6	9.0	10.8	10.6
6h	3.2	2.4	7.2	9.7	9.6
9h	3.0	2.2	6.8	9.0	9.0
12h	2.7	2.1	6.1	7.9	7.9
18h	1.7	1.5	3.9	5.2	5.2
24h	1.8	1.7	4.3	6.1	6.1
30h	1.5	1.3	3.3	4.5	4.5
36h	1.3	1.2	3.0	4.2	4.2
48h	1.6	1.4	3.7	5.1	5.1
72h	1.0	0.9	2.3	3.2	3.2

Alternative Retarding Basin Locations and Configurations

In response to the uncertainty of siting the retarding basin within the preferred location (over the vacant Crown land parcel) due to non-drainage constraints, two other locations were identified for investigation. The alternative basin locations (Figure 3-4) are sited on:

- Council land to the north-west of the oval; and,
- Private land over 6 and 8 Glismann Road.

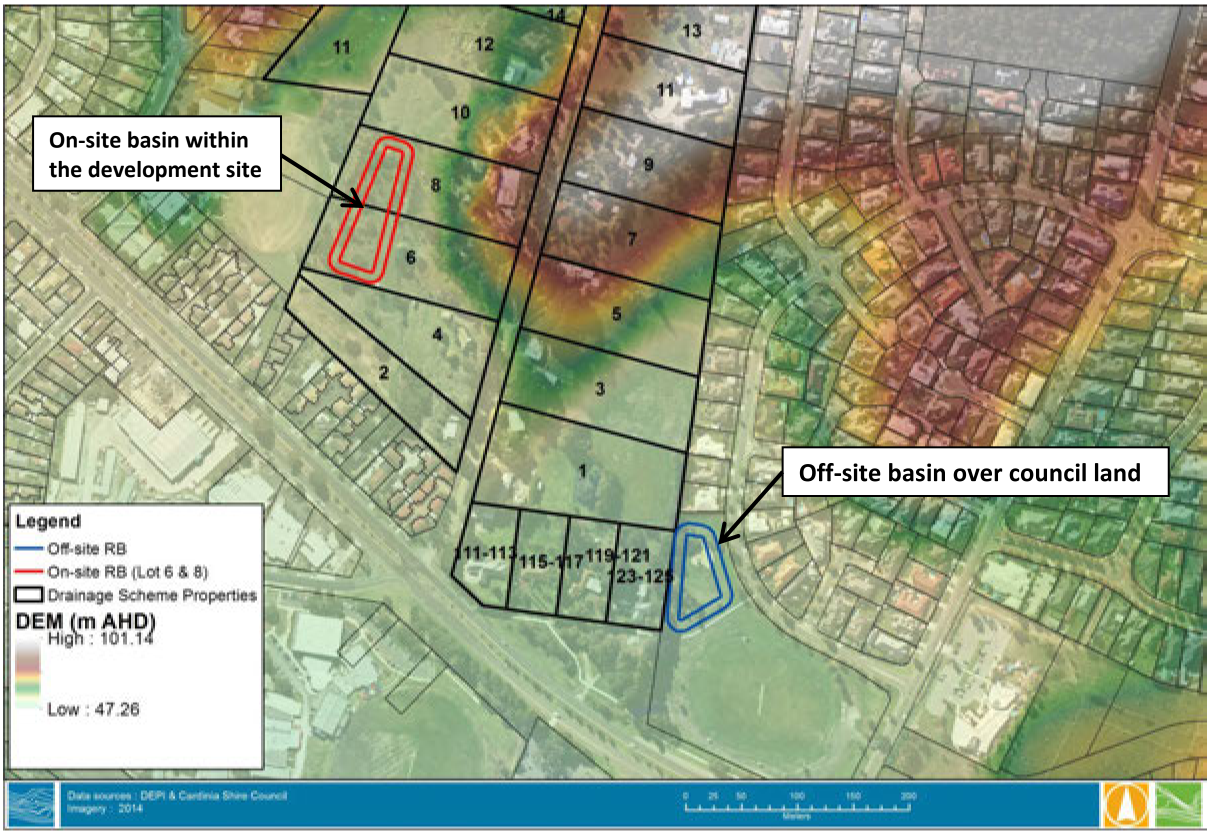

Figure 3-4 Alternative Retarding Basin Sites

The off-site basin proposed within the council land utilises relatively flat land and can be created by excavation with no embankments. The outlet for this basin will connect to the existing culvert under the highway. The outlet will be drowned out in the 100 Year ARI event. This location avoids the use of land set aside for development but would require the removal/redesign (with safety considerations) of the existing playground.

The on-site basin located over 6 and 8 Glismann Road utilises land which is currently subject to some inundation. If no basin was located over these lots, then the low lying areas through 6 and 8 Glismann Road would likely be filled to facilitate development. The basin is proposed to be located along the west side of the two lots, as a linear design, due to grade constraints and surface levels. Retardation of flow at this point will have the benefit of reducing overland flows crossing Glismann Road and the highway.

Initially both basin locations were investigated separately, as independent assets servicing the whole development. It was found that even with the basin size maximised at each location, the individual basins only capture and retard flow from part of the catchment and do not limit the total discharge on the downstream side of the highway to the desired 12.6 m^3/s.

The two basins were then modelled together. To minimise the use of developable land over the site, the size of the off-site basin (located within council land) was maximised and the storage volume required within the on-site basin (6 & 8 Glismann Road) then iteratively sized to mitigate the total development peak flows leaving the site. Concept design details of the two alternative basin options are provided in Table 3-3. The performance of the two alternative basin options in mitigating peak flows is shown in Table 3-4.

Table 3-3 Alternative Retarding Basin Concept Design

Proposed Storage Location	Basin Footprint (m^2)	NWL (m AHD)	Peak 100 Year Water Level (m AHD)	Peak Storage (m^3)	Outlet Structure
Council Land –north west of the oval	4,600	47.0	48.55	4,290	1 x Ø1350 mm pipe
6 and 8 Glismann Road	8,275	48.0	49.33	5,040	1 x Ø600 mm pipe

Table 3-4 Performance of the Alternative Retarding Basins

Duration	On-site RB (Lot 6 & 8) Inflow (m^3/s)	On-site RB (Lot 6 & 8) Outflow (m^3/s)	Off-site RB Inflow (m^3/s)	Off-site RB Outflow (m^3/s)	Flow Upstream Highway (m^3/s)	Flow Downstream Highway (m^3/s)
10m	3.3	0.8	3.4	0.0	9.3	9.1
15m	3.8	1.0	3.8	0.2	11.0	10.9
20m	3.7	1.1	3.5	0.5	12.0	11.2
25m	4.0	1.1	3.7	0.6	12.0	11.7
30m	3.7	1.1	3.5	0.6	11.4	11.2
45m	3.3	1.1	3.0	0.8	10.5	10.5
1h	3.6	1.2	3.2	0.8	12.1	11.7
1.5h	3.5	1.4	3.0	0.8	11.9	11.7
2h	3.7	1.5	3.2	0.9	13.0	**12.2**
3h	2.6	1.3	2.1	0.8	9.7	9.6
4.5h	2.7	1.3	1.9	0.9	10.2	10.1
6h	2.0	1.3	1.3	0.9	9.1	9.1
9h	1.9	1.1	1.3	0.9	8.6	8.6
12h	1.8	1.1	1.3	0.9	7.7	7.7
18h	1.2	1.0	0.8	0.6	5.1	5.1
24h	1.2	1.1	0.8	0.7	6.1	6.1
30h	1.0	0.8	0.6	0.5	4.5	4.5
36h	0.9	0.8	0.6	0.5	4.2	4.2
48h	1.0	0.9	0.7	0.6	5.0	5.0
72h	0.7	0.6	0.4	0.4	3.2	3.2

4. HYDRAULIC MODELLING

4.1 Overview

Hydraulic modelling of the area immediately upstream of the proposed basin (for existing conditions) and the area downstream of the highway was undertaken to:

- Confirm the capacity of the pipes under the Princes Highway;
- Investigate the flooding, properties at risk and overland flow paths along the Princes Highway under existing conditions;
- Investigate the performance of the proposed retarding basin (located in the vacant crown land parcel) i.e. to verify the RORB model results;
- Confirm that downstream flooding will not be worsened with the proposed retarding basin in place; and,
- Investigate the upstream flood risk in the event that the highway culverts are blocked (50% blockage).

There is currently no known flood modelling over the site. As such, a new hydraulic (TUFLOW) model was constructed for the area immediately upstream and downstream of the Old Princes Highway, consistent with the Melbourne Water Technical Specifications and Requirements (MWC, 2012).

4.2 TUFLOW Model Setup

The TUFLOW model setup is shown in Figure 4-1.

The model's terrain was created using LiDAR data. The existing Melbourne Water assets within the models extent were included as 1D components.

Inflows to the hydraulic model were taken from the existing and developed conditions RORB models respectively. The majority of the model inflows were input into the pipe network, allowing the flows to initially run into the pipe network before surcharging onto the 2D domain.

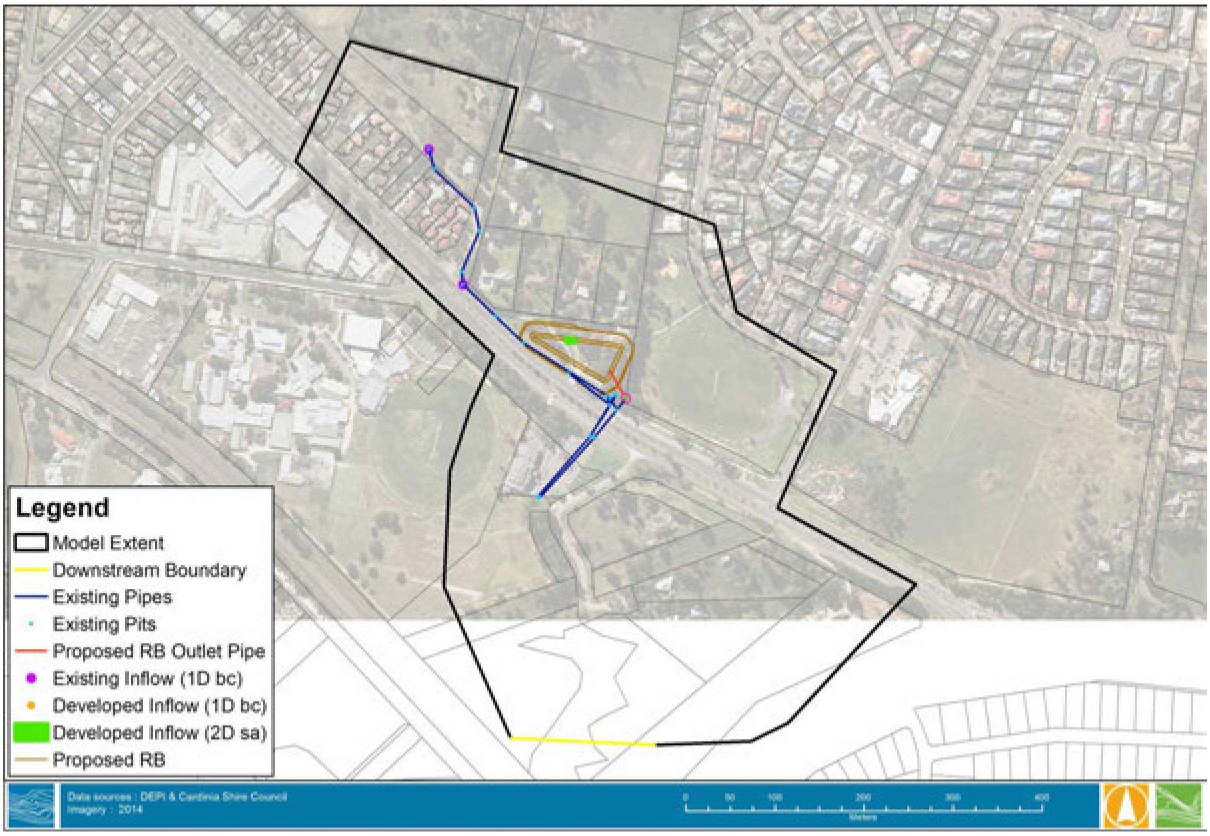

Figure 4-1 TUFLOW Model Setup

4.3 Model Results

The existing conditions 100 year ARI flood depths and heights for the 2 hour duration are shown in Figure 4-2 and Figure 4-3.

The enveloped (1 hour to 12 hour) developed conditions (with mitigation works) flood heights is shown in Figure 4-4. It should be noted that the developed conditions model and results do not cover the area upstream of the proposed retarding basin.

A flood height difference plot between developed (with mitigation works) and existing conditions is shown in Figure 4-5.

The results show that:

- Low lying land at 4 and 6 Glismann Road is subject to flooding;
- Under existing conditions the existing residential units along the highway are subject to flooding. This occurs once the capacity of the Melbourne Water pipe network that runs to the north of the properties is exceeded and overland flows move across the properties, towards the highway;
- Flows travel to the east, moving along the highway towards the existing culverts under the highway. The highway is not overtopped and flooding is contained to the northern lane of the highway;
- The existing peak 100 year ARI flow leaving the highway culverts (12.7 m^3/s) matches up well with the existing 100 year ARI peak flow (12.6 m^3/s) from the RORB model.
- The highway culverts are flowing full and cannot convey any additional flow without increasing the flood level upstream of the highway;

- The peak 100 year ARI flood level in the proposed basin (48.72 m AHD) matches up closely to peak 100 year ARI flood level derived from the RORB model (48.77 m AHD);
- The peak developed (with mitigation works) conditions 100 year ARI flow leaving the highway culverts is 11.9 m^3/s. This is lower that the peak flow from the RORB model (12.6 m^3/s). The large difference in peak outflow is due to difference in how the programs model the interaction between the basin outlet pipes and the highway culverts. The complexity of the outlet structure makes it difficult to model in RORB and can be modelled more accurately through the hydraulic model;
- The difference plot between developed and existing conditions (Figure 4-5) shows that the flood levels downstream of the highway are unchanged following the development; and
- With the highway culverts partially blocked (50% blockage), the peak flood height on the upstream side of the highway rises by 150 mm, from 48.72 m AHD to 48.87 m AHD. Therefore with 300 mm freeboard above the 100 year ARI level, the lots adjacent to the proposed basin will remain flood free even with the 50% blockage on the highway culverts.

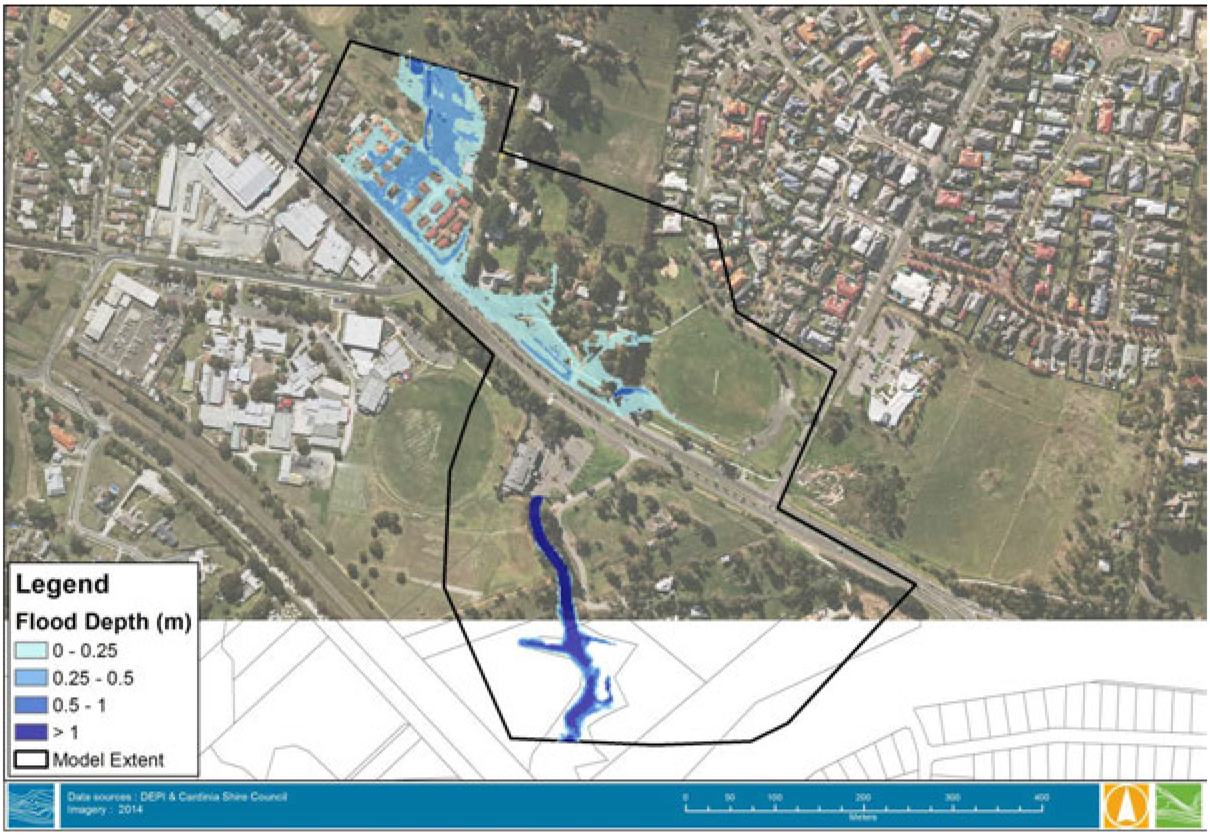

Figure 4-2 Existing 100 Year 2 Hour Flood Depths

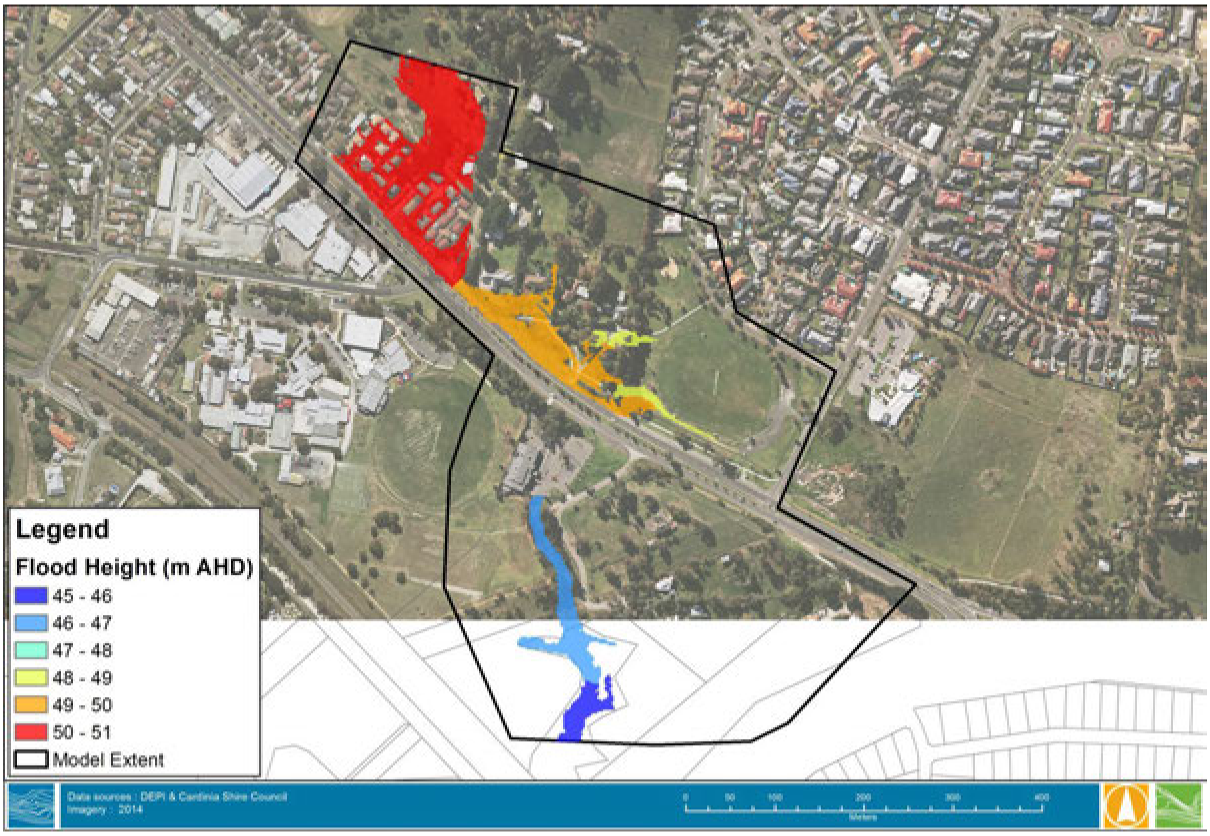

Figure 4-3 Existing 100 Year 2 Hour Flood Heights

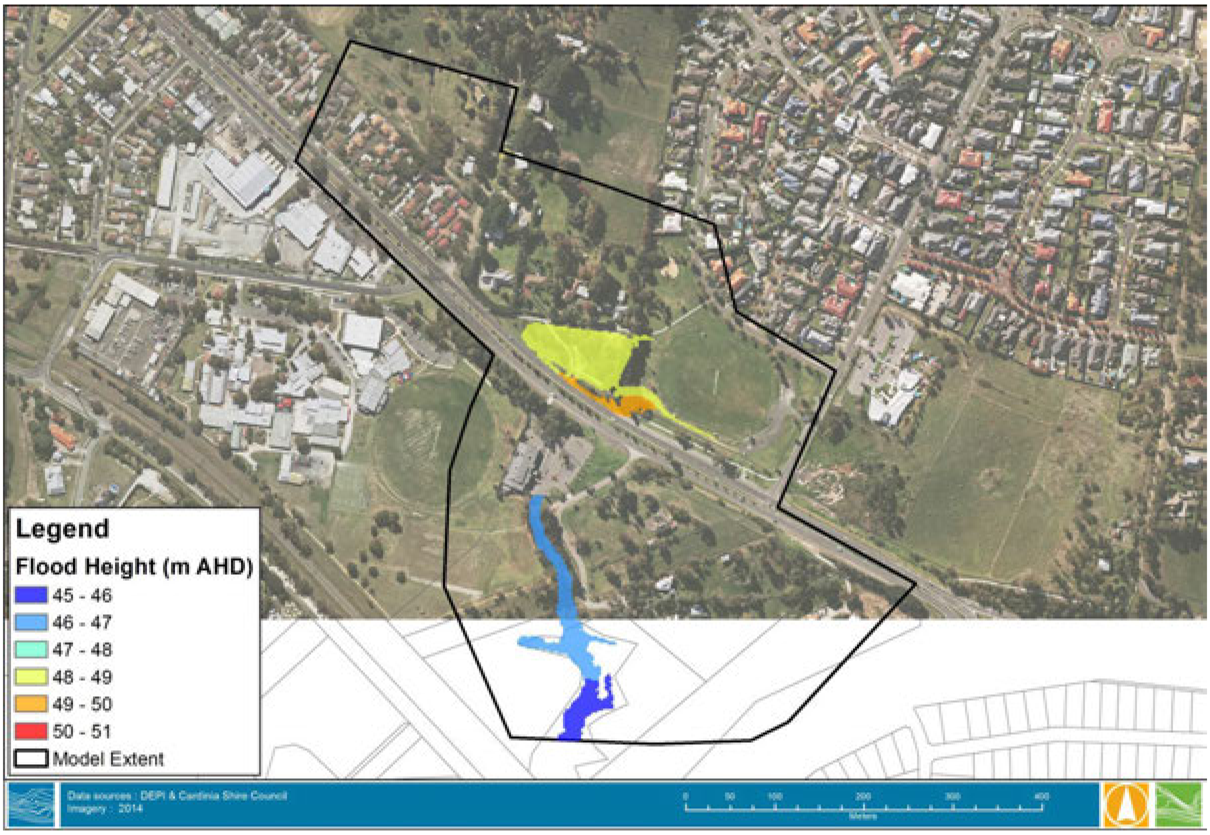

Figure 4-4 Developed 100 Year Enveloped Flood Heights (1 hour to 12 hour)

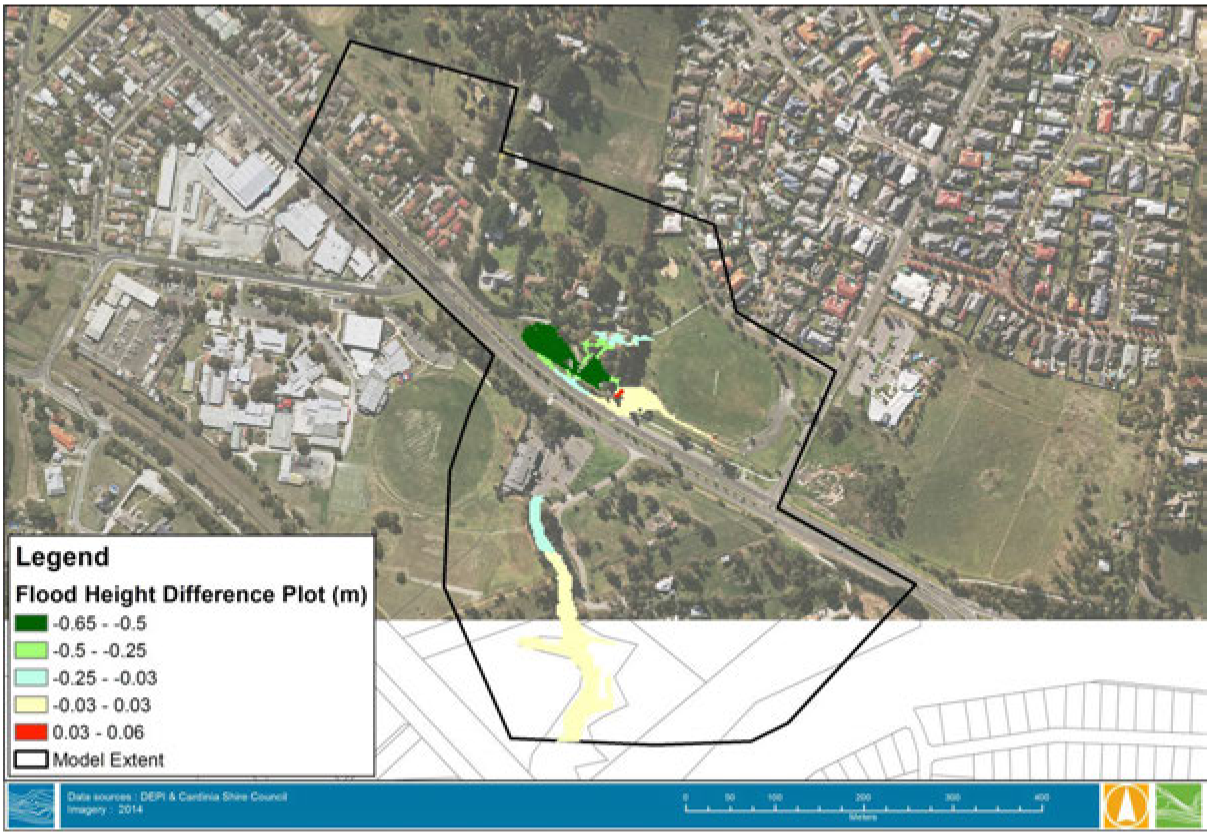

Figure 4-5 Flood Height Difference Plot (Developed With Mitigation Minus Existing Conditions)

5. WATER QUALITY MODELLING

5.1 Proposed WSUD Works

A wetland within the base of the proposed retarding basin is proposed to minimise the footprint area required for drainage works. The overall concept layout of the proposed WSUD works is shown in Figure 5-1.

The proposed treatment measures include the following:

- 2,300 m^2 wetland within the proposed retarding basin; and
- 600 m^2 and 250 m^2 sediment ponds constructed at the west and north-east wetland inlet zones respectively.

Figure 5-1 Plan View of Proposed WSUD Works

The sediment ponds were designed to treat the 1 Year ARI flow from the development site. The sediment pond size was also checked to ensure it was adequately sized to allow cleanout frequencies (every 5 years) for sediment loads from the external catchments.

The proposed treatment measures were designed with the following parameters:

- Sediment pond extended detention depth = 0.5 m;
- Sediment pond permanent pool depth = 1.0 m;
- Wetland extended detention depth = 0.5 m;
- Average wetland permanent pool depth = 0.5 m;
- Normal Water Level (NWL) = 47 m AHD; and
- Wetland detention time = 72 hrs.

5.2 MUSIC Modelling

MUSIC has been used to size the proposed WSUD works. The layout of the MUSIC model is shown in Figure 5-2. MUSIC requires the determination of various hydrologic parameters to represent conditions on the site. The following inputs were used:

- Six minute rainfall data from from Koo Wee Rup for the reference year (2004);
- Source Nodes – The model's source node parameters (area and FI) and source node breakup as per the values used in the RORB modelling.

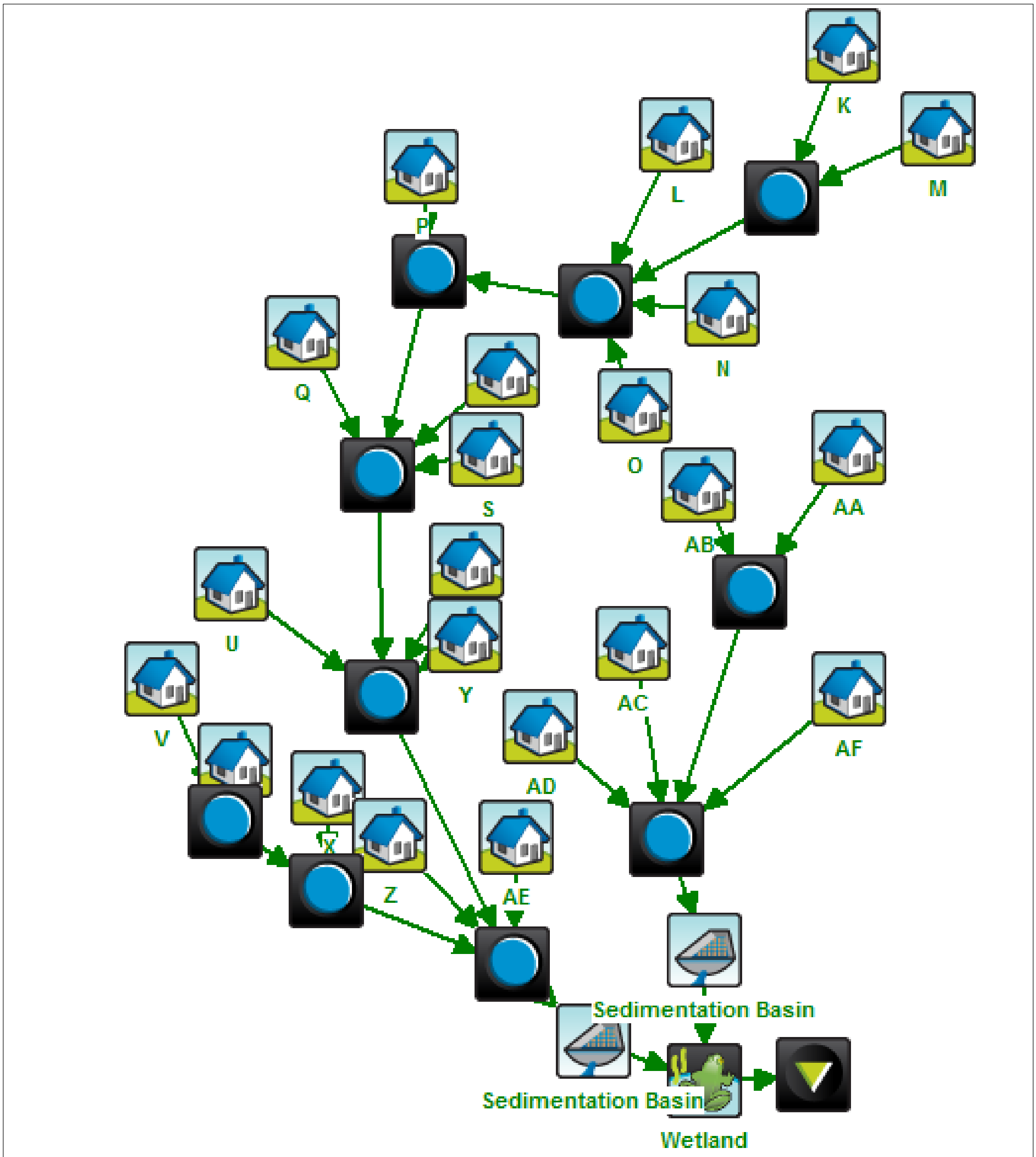

Figure 5-2 MUSIC Model Layout

The MUSIC results (Table 5-1) show that the proposed WSUD features meet water quality objectives for the future development in Glismann Road. The results show that the proposed treatment asset will remove greater Total Suspended Solids, Total Phosphorous and gross pollutant loads than those generated within the development site, thus exceeding best practise requirements.

Table 5-1 MUSIC Model Results

Parameters	Total source loads	Residual load after treatment	Load removed in proposed WSUD assets	Development source loads	% Removal of development source loads
Total Suspended Solids (kg/yr)	54,800	23,400	31,400	22,500	>100
Total Phosphorous (kg/yr)	113	64	49	45	>100
Total Nitrogen (kg/yr)	805	659	146	318	46
Gross Pollutants (kg/yr)	9,800	0	9,800	3,990	>100

5.3 Alternative WSUD Asset Locations

The feasibility and sizing of WSUD assets within the two alternative basin sites were also investigated. MUSIC modelling of the alternative WSUD asset configuration showed that WSUD assets are required in both basin locations. The basin footprints at both sites are sufficient to meet the treatment area requirements for the WSUD assets. The MUSIC modelling results for the alternative option is shown in Table 5-2.

The sizes of the WSUD assets are outlined in Table 5-3. With two assets in the catchment there will be an increased operation and maintenance cost.

The NWL and downstream connection levels in the off-site wetland (council land) allow for a maintenance drawdown capacity in this wetland. To allow for some maintenance drawdown capacity in the on-site wetland (lots 6 and 8) the NWL may need to be raised marginally.

Table 5-2 MUSIC Model Results

Parameters	Total source loads	Residual load after treatment	Load removed in proposed WSUD assets	Development source loads	% Removal of development source loads
Total Suspended Solids (kg/yr)	54,800	28,800	26,000	22,500	>100
Total Phosphorous (kg/yr)	113	72	42	45	>100
Total Nitrogen (kg/yr)	805	663	142	318	45
Gross Pollutants (kg/yr)	9,800	1,960	7,840	3,990	>100

Table 5-3 Sizing of WSUD Assets at the Alternative Basin Location

Proposed Storage Location	Sediment Pond Size (m^2)	Wetland Size (m^2)	NWL (m AHD)
Council Land –north west of the oval	250	1,550	47.0
6 and 8 Glismann Road	400	1,200	48.0

5.4 Stormwater Harvesting Opportunities

There are opportunities to consider using treated stormwater from the wetland for irrigation of the adjacent oval or for other non-potable demands (e.g. for demands at Beaconsfield Community Centre). A pumped connection from the wetland outlet can potentially be connected to storage tanks and used to irrigate the oval. Harvesting water for re-use will further reduce stormwater volumes, runoff frequencies and pollutant loads leaving the catchment.

Based on harvesting 50% of the treated flows, the proposed wetland system could potentially yield 140 ML/yr. Water Technology has not undertaken any calculations on the viability of harvesting stormwater from the proposed wetland. This will need to be investigated in further detail to determine the reliability (yield vs demand) and constructability of a water harvesting system.

6. STORMWATER DRAINAGE

6.1 Minor Drainage Network

The minor drainage for the site was designed to cater for the 5 year ARI flow. It is proposed to utilise the existing Melbourne Water drain through the site together with the new subdivision pipes.

A concept drainage design plan was prepared using Melbourne Water's drainage scheme spreadsheet. The calculation details are provided in Appendix C. Pipe grades and levels used for the analysis were based on the existing surface levels across the site. Drainage outfalls were designed for all developable lots greater than 0.4 Ha.

The external catchment upstream of the site has a different time of concentration to that of the site, but for this analysis the peak external and peak site flows were conservatively combined.

The analysis shows that the existing Melbourne Water drain through the site has sufficient capacity to cater for minor flows from the external catchment and the site. The lower end of the Melbourne Water drain will need to be regraded and extended to allow for a connection into the proposed retarding basin and wetland.

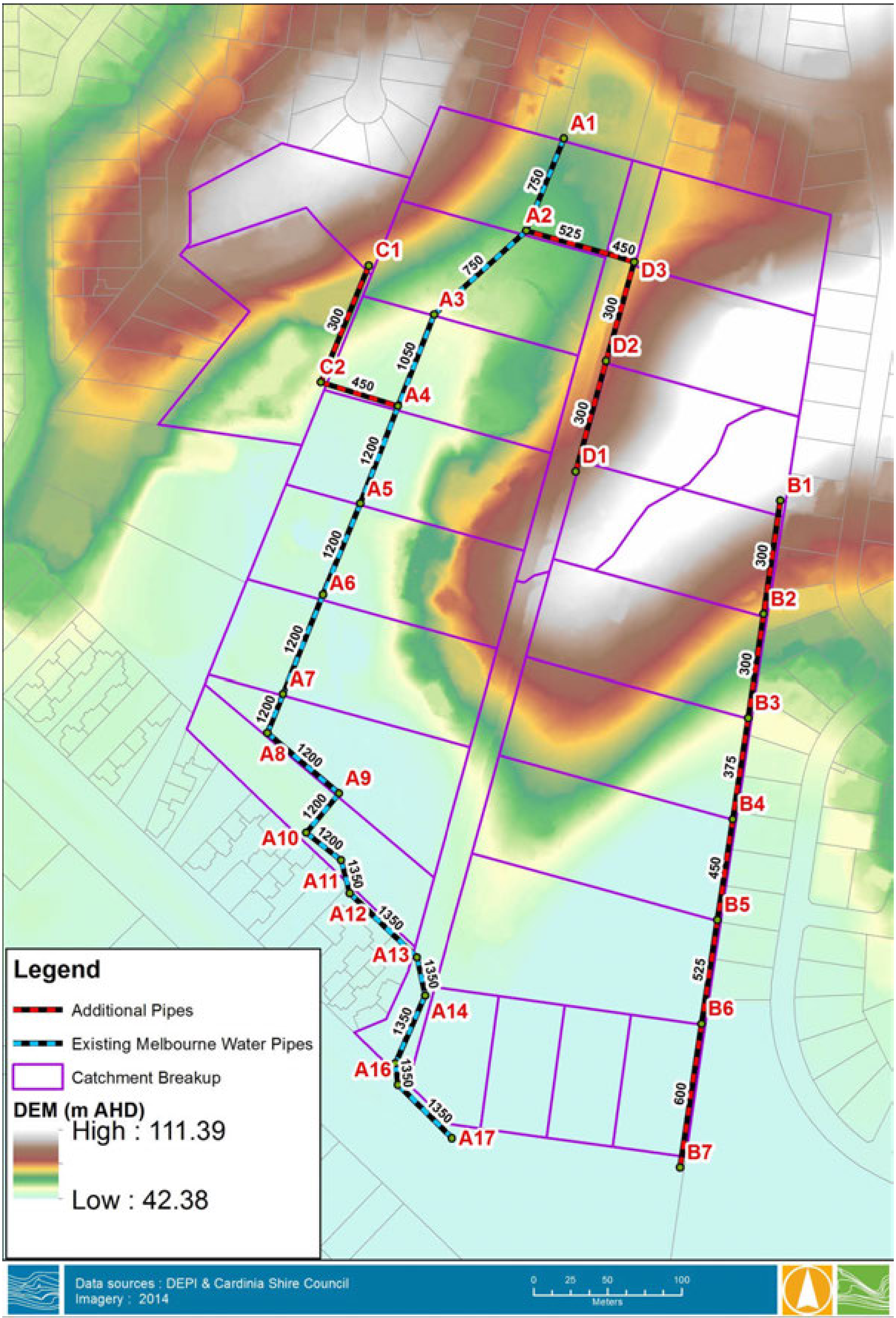

Figure 6-1 Proposed Drainage Network through the Site

Table 6-1 Details of the Site Drainage Network

Pipe Reference	Length	U/S IL	D/S IL	Pipe Size (mm)
A1-A2	68	56.30	54.05	750
D1-D2	77	56.26	55.49	300
D2-D3	70	55.49	54.79	300
D3-D4	18	54.79	54.61	450
D4-A2	56	54.61	54.05	525
A2-A3	84	54.05	51.35	750
A3-A4	67	51.35	49.05	1050
C1-C2	85	50.44	49.59	300
C2-A4	54	49.59	49.05	450
A4-A5	70	49.05	48.82	1200
A5-A6	67	48.82	48.60	1200
A6-A7	72	48.60	48.35	1200
A7-A8	28	48.35	48.25	1200
A8-A9	63	48.25	48.05	1200
A9-A10	35	48.05	47.95	1200
A10-A11	30	47.95	47.65	1200
A11-A12	23	47.65	47.60	1350
A12-A13	63	47.60	47.41	1350
A13-A14	26	47.41	47.35	1350
A14-A15	50	47.35	47.18	1350
A15-A16	15	47.18	47.14	1350
A16-A17	51	47.14	47.00	1350
B1-B2	77	71.00	59.00	300
B2-B3	71	59.00	52.00	300
B3-B4	69	52.00	49.10	375
B4-B5	69	49.10	48.50	450
B5-B6	71	48.50	47.90	525
B6-B7	98	47.90	47.00	600

6.2 Major Drainage Network

A concept design of the major overland flow paths through the site is shown in Figure 6-1 below, with flow paths along the pipe easements.

Preliminary sizing of the flow paths were undertaken using a Manning's calculation to provide the magnitude of the overland flows and an indication of the widths of these flows. Overland flows were calculated as the gap flow between the 100 year ARI flow and the pipe full flow. The calculation details are provided in Appendix D. Figure 6-2 and Figure 6-3 show concept designs of the longitudinal profiles for the two drainage lines.

The analysis shows that a flow width of 4.5 m is sufficient to accommodate the overland flow through the steeper north section through lots 12, 14 and 16, and along the eastern boundary of the site. Fill levels should not be excessive and the appropriate flood safety criteria can be met for the overland flow paths (velocity x depth < 0.35). The concept fill requirements also account for a minimum pipe cover of 750 mm across the site.

There is a significant opportunity to reduce flooding at the existing residential units along the highway by providing a flow path (within lot 2) that directs flow towards Glismann Road, rather than through the existing units. This can be achieved by lowering the existing surface levels through this point to lower the flood levels and direct flows to the east.

The southern end of Glismann Road should be cambered to efficiently direct site (overland) flows to the east, into the swale drain on the northern side of the highway. This will prevent any excessive ponding of water at the intersection of the highway and Glismann Road. The existing swale drain along the highway should be formalised (proposed channel depth of 250 mm, base width of 2 m, and 1 in 4 side slopes) to direct overland flows into the proposed basin. Once the capacity of the swale drain is exceeded, flows will engage the northern lane of the highway and continue to the east towards the proposed basin.

6.3 Alternative Flow Path Alignment

An alternative overland flow path arrangement is also shown in Figure 6-1. This alignment follows the potential future road network along the school boundary more closely. The alternative arrangement can maximise the development opportunity provided that the Melbourne Water pipe is realigned to follow the proposed road. It will also require appropriate fill to grade overland flows towards the proposed road adjacent to the site's western boundary.

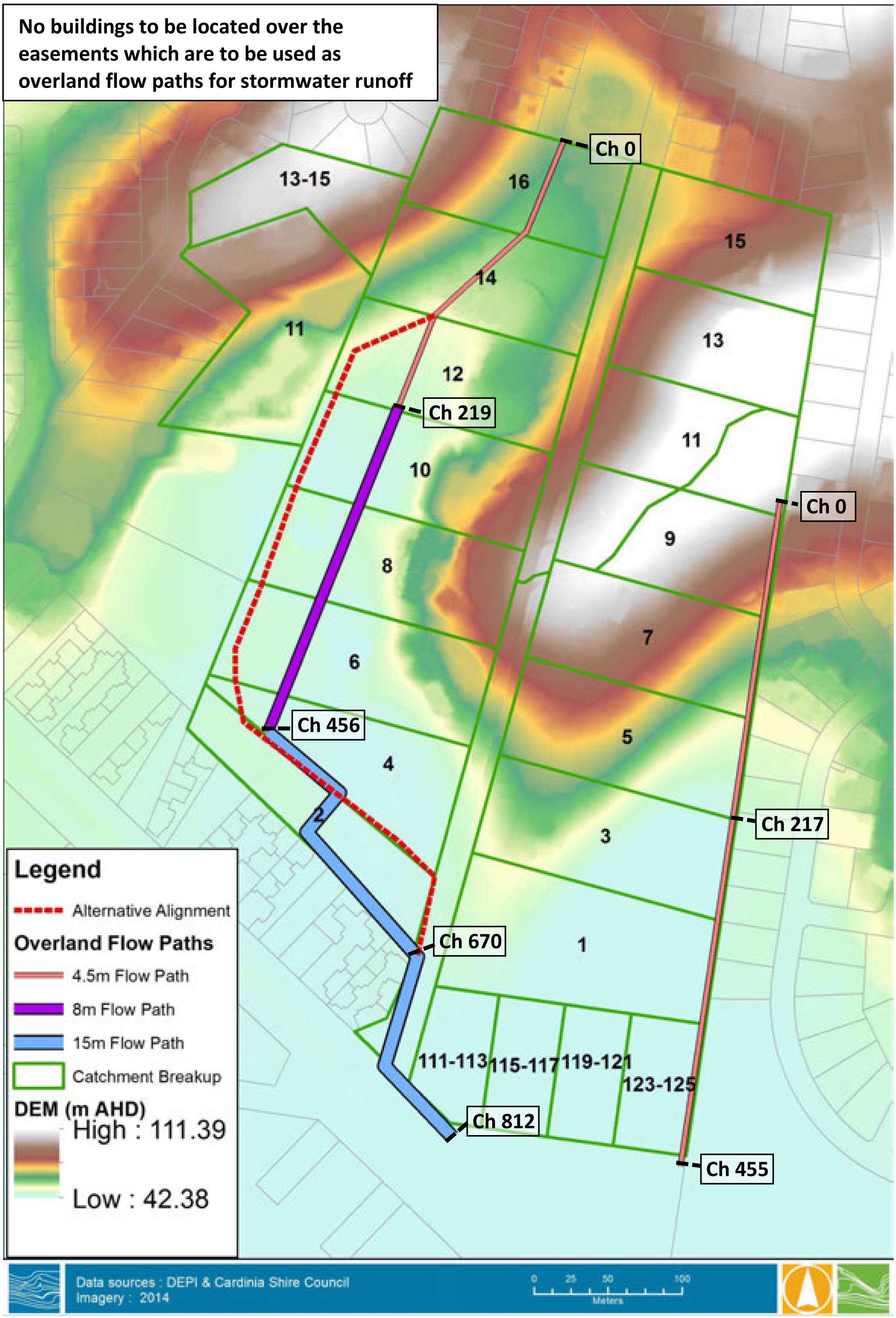

Figure 6-1 Proposed Overland Flow Path

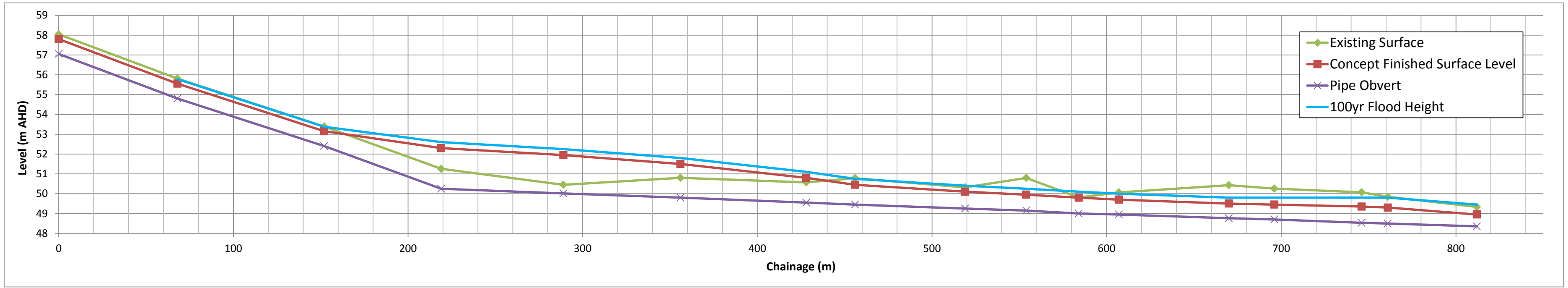

Figure 6-2 West Overland Flow Profile

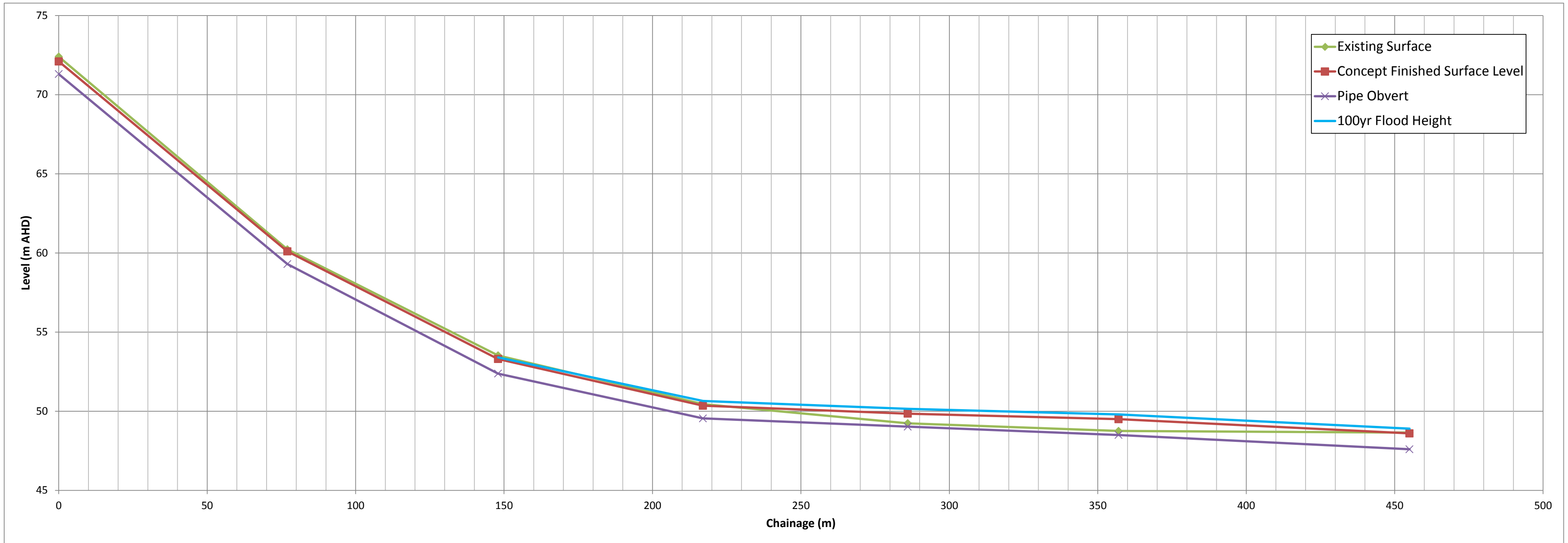

Figure 6-3 East Overland Flow Profile

7. FREEBOARD REQUIREMENTS

The building floor level for any new developments should be at least 300 mm above the 100 year flood level.

Under existing conditions, the southern end of lots 111-113 and 115-117 are subject to flooding from breakout overland flows. With the future proposed overland flow path, the existing flood level (~49.8 m AHD) at this point has been maintained. To reduce the flood risk it is important that the final surface levels for future developments over lots 111-113 and 115-117 have at least 300 mm of freeboard above this flood level.

The floor levels for lots adjacent to the proposed retarding basin (lots 111 – 125) should be set with at least 300 mm of freeboard above the 100 year flood level (48.72 m AHD) in the basin. This is achievable with only minimal fill (~300 mm) required for lot 123-125.

8. CONCLUSIONS

A retarding basin and wetland is proposed within the open space reserve (crown land) in the south east corner of the site. Locating the basin/wetland structure in this area will maximise the development opportunities within the site. An alternative option was also investigated, should the preferred location be unavailable for siting the basin/wetland. The alternative option requires the construction of two basin/wetland assets in the catchment; one located off-site in a council owned open space recreational area and a second asset located over private land (lots 6 and 8 Glismann Road).

Minor flows will be piped to the proposed retarding basin while major flows will run along the proposed overland flow paths (easements and road network) through the site, towards the retarding basin.

There are two alternative drainage line options to consider. The first option is to retain the existing Melbourne Water pipe through the site, with a designated easement over the pipe to convey overland flows. The second option is to realign the Melbourne Water pipe to follow the future road network proposed along the site's western boundary. The first option does not require a realignment of the existing Melbourne Water pipe and also reduces fill requirements for the development as it follows the natural depression through the site. The second option will require additional earthworks and pipe works, but does improve the opportunity to develop over the site.

The proposed drainage works for the Glismann Road development will ensure that:

- Offsite discharges are retarded back to existing conditions and there is no adverse flooding impact on downstream properties;
- Appropriate flow paths are provided to ensure that the new development areas and surrounding properties are protected from flooding and can be appropriately drained; and
- Stormwater pollutants from the development are treated to meet best practise requirements.

9. REFERENCES

Cardinia Shire Council 2013, *Beaconsfield Structure Plan*.

Cardinia Shire Council 2013, *Beaconsfield Structure Plan Background Paper*.

Department of Natural Resources and Environment 2002, *Identification and Assessment of Salinity risk in the Growth Corridor Area of Cardinia Shire*.

Heritage Advisors 2010, *Cultural Heritage Management Plan No 11452*.

Ecology Partners 2010, *Biodiversity Assessment for Area 1, 'Beaconsfield'*.

Melbourne Water 2011, *2D Modelling Guidelines for Melbourne Water.*

Melbourne Water 2010, *MUSIC Guidelines.*

APPENDIX A RORB MODELLING

RORB Overview

RORB (Laurenson et al 2005) is a non-linear rainfall runoff and stream flow routing model for calculation of flow hydrographs in drainage and stream networks. The model requires catchments to be subdivided into subareas, connected by conceptual flow reaches. Design storm rainfall is input to the centroid of each pre-defined subarea. Loss parameters are applied to the model depending on the ARI event being studied and are then deducted by RORB with the excess runoff being routed through the conceptual reach network.

Fraction Impervious Data

The FI values for each sub catchment were applied as detailed in Melbourne Water's MUSIC Guidelines.

Model Reconciliation

An undiverted 'existing conditions' RORB model (Figure A - 1) was constructed using MiRORB (MapInfo RORB). The existing conditions RORB catchment boundary was delineated from terrain contours which were created from the LiDAR. Sub area boundaries were then delineated and nodes placed at all areas of interest.

The undiverted 'existing conditions' RORB model was built to allow for reconciliation with the Rational Method. The undiverted 'existing conditions' RORB model was reconciled at the three discharge points shown in Figure A - 1, through adjustment of the models k_c coefficient. The Rational Method calculations for the three discharge points are shown in Table A - 2, Table A - 3 and Table A - 4.

A uniform k_c value of 1.9 was adopted for the entire model, which provided 100 year ARI RORB peak flows that matched up well to Rational Method estimates (Table A - 1).

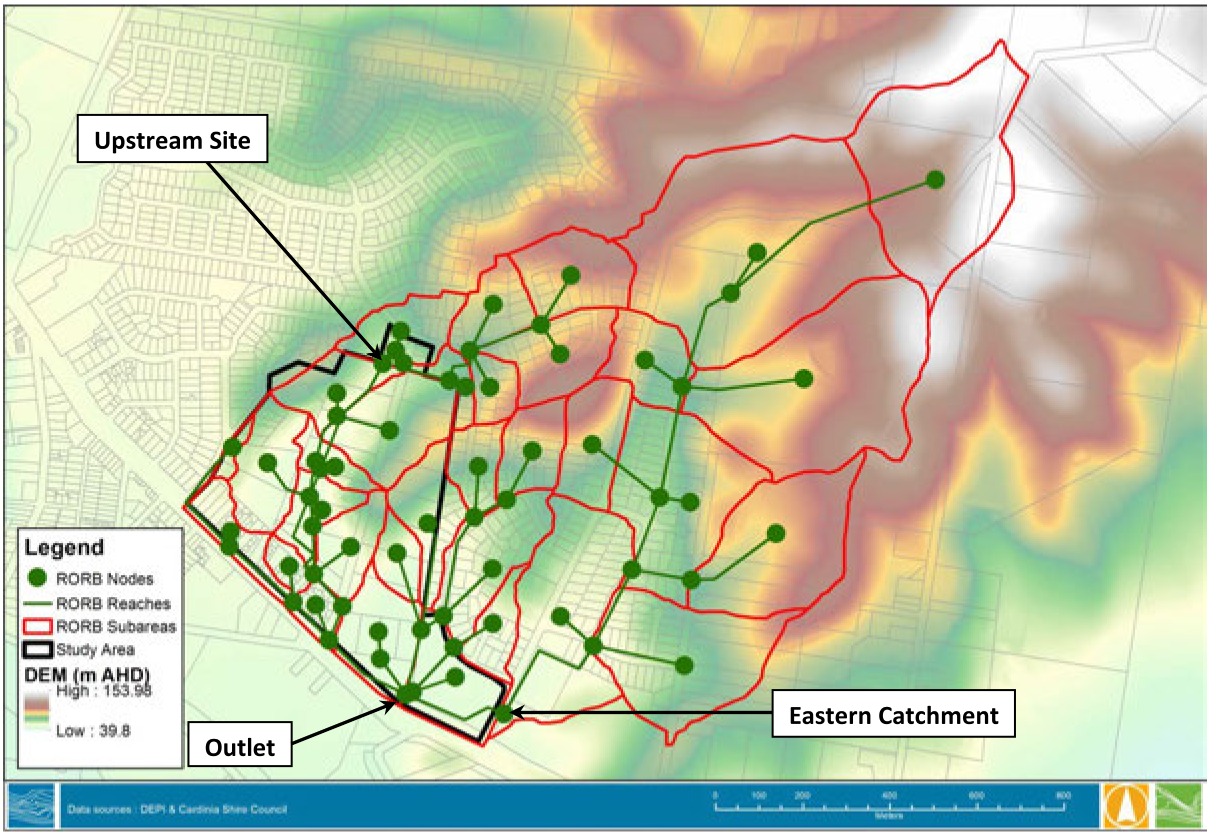

Figure A - 1 Undiverted Existing Conditions RORB Model & Flow Reconciliation Locations

Table A - 1 RORB (Undiverted) Model Reconciliation

Location	Q100 Rational (m^3/s)	Q100 RORB Undiverted (m^3/s)
Upstream Site	3.5	3.4 (25 min)
Eastern Catchment	9.6	9.8 (25 min)
Outlet	13.8	13.8 (2 hr)

Table A - 2 Rational Method – Upstream Site

Catchment Characteristics		
Area	16.92	Hectares
fi	0.50	
$^{1}I_{10}$	25.8	mm/hr
Mode of Tc Calculation	Manual Input	
Initiation time (if rqd)	7	minutes
tc (manual input)	10.1	minutes (7+0.8+2.3)
→ tc	**10.1**	**minutes**

Full Pipe Velocity Calculation	
L	250 m
Upstream Elevation	100.6 m
Downstream Elevation	79.8 m
Slope	0.08 m/m
n*	0.013
Pipe Diameter	0.450 m
R	0.1125
→ V	**5.2** m/s
tc	0.8 min

Full Pipe Velocity Calculation	
L	500 m
Upstream Elevation	79.8 m
Downstream Elevation	59.0 m
Slope	0.04 m/m
n*	0.013
Pipe Diameter	0.450 m
R	0.1125
→ V	**3.7** m/s
tc	2.3 min

ARI (years)	Q (m^3/s)	I (mm/hr)	tc	Fy	C'10	C10	Cy	Total Area (ha)
1	**0.64**	33.6	10.1	0.80	0.111	0.505	0.404	16.92
2	**0.90**	44.6	10.1	0.85	0.111	0.505	0.430	16.92
5	**1.37**	60.6	10.1	0.95	0.111	0.505	0.480	16.92
10	**1.70**	71.5	10.1	1	0.111	0.505	0.505	16.92
20	**2.14**	86.0	10.1	1.05	0.111	0.505	0.531	16.92
50	**2.92**	106.8	10.1	1.15	0.111	0.505	0.581	16.92
100	**3.54**	124.2	10.1	1.2	0.111	0.505	0.606	16.92

Table A - 3 Rational Method – Eastern Catchment

Catchment Characteristics		
Area	98.92	Hectares
fi	0.24	
¹I_{10}	25.8	mm/hr
Mode of Tc Calculation	Manual Input	
Initiation time (if rqd)	7	minutes
tc (manual input)	15.9	minutes (7+8.9)
→ tc	**15.9**	**minutes**

Full Pipe Velocity Calculation		
L	1350	m
Upstream Elevation	76.1	m
Downstream Elevation	49.5	m
Slope	0.020	m/m
n*	0.013	
Pipe Diameter	0.450	m
R	0.1125	
→ V	**2.5**	m/s
tc	8.9	min

ARI (years)	Q (m^3/s)	I (mm/hr)	tc	Fy	C'10	C10	Cy	Total Area (ha)
1	**1.80**	27.2	15.9	0.80	0.111	0.300	0.240	98.92
2	**2.53**	36.0	15.9	0.85	0.111	0.300	0.255	98.92
5	**3.79**	48.4	15.9	0.95	0.111	0.300	0.285	98.92
10	**4.68**	56.8	15.9	1	0.111	0.300	0.300	98.92
20	**5.89**	68.0	15.9	1.05	0.111	0.300	0.315	98.92
50	**7.97**	84.0	15.9	1.15	0.111	0.300	0.345	98.92
100	**9.62**	97.2	15.9	1.2	0.111	0.300	0.360	98.92

Table A - 4 Rational Method - Outlet

Catchment Characteristics		
Area	162.90	Hectares
fi	0 305	
¹I_{10}	25.8	mm/hr
Mode of Tc Calculation	Manual Input	
Initiation time (if rqd)	0	minutes
tc (manual input)	26.4	minutes (10.1+1.7+14.7)
→ tc	**26.4**	**minutes**

Full Pipe Velocity Calculation		
L	300	m
Upstream Elevation	59.0	m
Downstream Elevation	50.5	m
Slope	0.03	m/m
n*	0.013	
Pipe Diameter	0.450	m
R	0.1125	
→ V	**3.0**	m/s
tc	1.7	min

Full Pipe Velocity Calculation		
L	720	m
Upstream Elevation	50 5	m
Downstream Elevation	49 0	m
Slope	0 002	m/m
n*	0 013	
Pipe Diameter	0.450	m
R	0.1125	
→ V	**0.8**	m/s
tc	14.7	min

ARI (years)	Q (m^3/s)	I (mm/hr)	tc	Fy	C'10	C10	Cy	Total Area (ha)
1	**2.68**	21.1	26.4	0.80	0.111	0 352	0.281	162 9
2	**3.75**	27.7	26.4	0.85	0.111	0 352	0.299	162 9
5	**5.56**	36.8	26.4	0.95	0.111	0 352	0.334	162 9
10	**6.82**	42.9	26.4	1	0.111	0 352	0.352	162 9
20	**8.52**	51.0	26.4	1.05	0.111	0 352	0.369	162 9
50	**11.44**	62.6	26.4	1.15	0.111	0 352	0.404	162 9
100	**13.75**	72.1	26.4	1.2	0.111	0.352	0.422	162 9

Existing Conditions

The undiverted 'existing conditions' model was then modified to include the existing retarding basin at the intersection of Timberside Drive and Patrick Place, and the defacto retarding basin upstream of the Old Princes Highway. The diverted existing conditions model was also calibrated by varying the k_c parameter as required to keep the k_c/d_{av} ratio constant.

Developed Conditions

The existing conditions model was then modified to create the future developed conditions RORB model. The model setup for developed conditions is shown in Figure A - 2. The fraction impervious values and reach types were changed to reflect the future developments. The models catchment boundary was also revised slightly to include the additional area on 13-15 Mahon Avenue, which will drain into the study catchment following development. The k_c parameter for the developed conditions model was also varied as required to keep the k_c/d_{av} ratio constant.

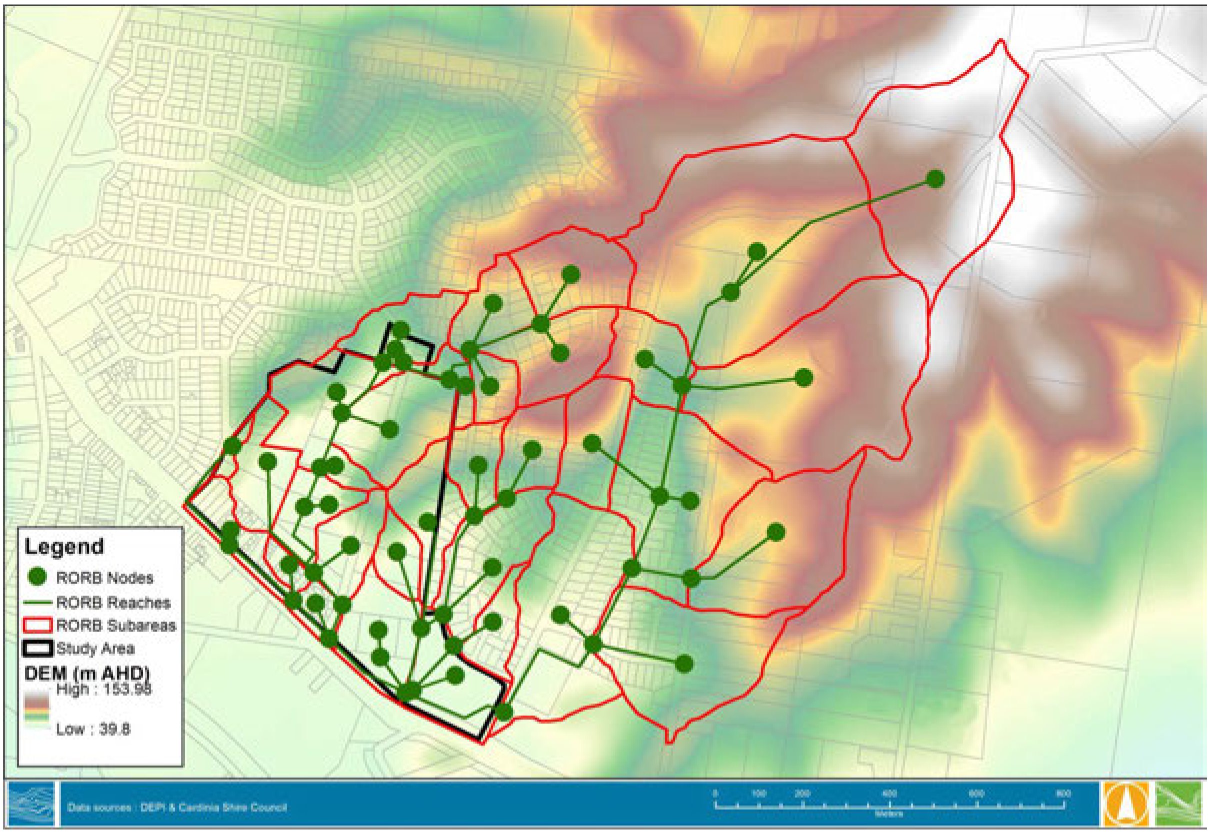

Figure A - 2 Developed Conditions RORB Model

Run Parameters

The models were run for the 100 year ARI for all durations between 10 minutes and 72 hours with the following parameters.

- m = 0.8;
- 100 Year ARI runoff coefficient = 0.6;
- No areal reduction factors;
- Filtered temporal pattern; and
- Uniform areal pattern; and
- Initial loss = 10 mm, as most of the catchment is urbanised.

Table A - 5 shows the RORB Model initial loss and kc values used for each scenario.

Table A - 5 RORB Model IL and kc Parameters

Modelling Scenario	IL (mm)	d_{av}	kc/d_{av}	kc
Existing Conditions (Undiverted)	10	1.10	1.73	1.90
Existing Conditions (Diverted)	10	1.10	1.73	1.90
Future Developed Conditions	10	1.10	1.73	1.90
Future Developed Conditions with Proposed Storage	10	1.12	1.73	1.93

APPENDIX B SEDIMENT POND SIZING

Sediment Pond Sizing – West (Main) Inflow Point

Vs =	0.011	m/s
d_e =	0.5	m
d_p =	1.0	m
d* =	1.0	m
$\frac{(d_e+d_p)}{(d_e+d^*)}$ =	1.0	
Q =	0.46	m^3/s
A =	600	m^2
$\frac{V_s}{Q/A}$ =	14.35	
λ =	0.26	pond shape assumption
n =	1.35	
Fraction of Initial Solids Removed		
R =	96%	

Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle

Cleanout Frequency

Catchment Area =	45.4	ha	
Sediment load =	1.60	m^3/ha/yr	(Willing and Partners 1992)
Gross Pollutant Load =	0.40	m^3/ha/yr	(Alison et al 1998)
Actual basin depth =	1.5	m	
Actual Basin area =	600	m^2	

Therefore, cleanout frequency required = $\frac{(1.6+0.4)A_{catchment}}{0.5d_{basin}*A_{basin}}$ = 0.20 per year | Clean out every 5.0 years

Assumes cleanout when basin 50% full

Try to minimise cleanouts - ideally, once every 5 years OK

Sediment Pond Sizing – East Inflow Point

Vs =	0.011	m/s
d_e =	0.5	m
d_p =	1.0	m
d* =	1.0	m
(d_e+d_p) = / (d_e+d^*)	1.0	
Q =	0.25	m^3/s
A =	250	m^2
V_s = / Q/A	11.00	
λ =	0.26	pond shape assumption
n =	1.35	
Fraction of Initial Solids Removed		
R =	95%	

Requirement: Melbourne Water Requires R = 95% for a 125 micrometer particle

Cleanout Frequency

Catchment Area =	18.7	ha	
Sediment load =	1.60	m^3/ha/yr	(Willing and Partners 1992)
Gross Pollutant Load =	0.40	m^3/ha/yr	(Alison et al 1998)
Actual basin depth =	1.5	m	
Actual Basin area =	250	m^2	

Therefore, cleanout frequency required = $\frac{(1.6+0.4)A_{catchment}}{0.5d_{basin}*A_{basin}}$ = 0.20 per year | Clean out every 5.0 years

Assumes cleanout when basin 50% full

Try to minimise cleanouts - ideally, once every 5 years OK

APPENDIX C PIPE DRAINAGE CALCULATIONS

Pipe Ref.	Upstream Area A (ha)	Cumulative Upstream Area ΣA (ha)	5 Year ARI Runoff Coefficient C_5	ARI (y)	Effective Area A_e (ha)	Cumulative Effective Area ΣA_e (ha)	Time of Concentration t_c (min)	Rainfall Intensity I_y (mm/hr)	Design Flow Q_y (m^3/s)	Length L (m)	NS Elev u/s (m)	NS Elev d/s (m)	Slope S (1 in ...)	Pipe Diameter (mm)	Pipe Type/Backfill	Full Flow Q_{full} (m^3/s)	Full Velocity v_{full} (m/s)	Time in Pipe t_{pipe} (min)
A1-A2				5	-			-	0.87	68	56.30	54.05	30	750	RRJ-20% FCR	2.03	4.58	0.25
					-			-	-				-			-	-	-
D1-D2	0.3214	0.32	0.47	5	0.15	0.15	8.0	67.41	0.03	77	56.26	55.49	100	300	RRJ-20% FCR	0.10	1.37	0.94
D2-D3	0.5952	0.92	0.47	5	0.28	0.43	8.9	64.19	0.08	70	55.49	54.79	100	300	RRJ-20% FCR	0.10	1.37	0.85
D3-D4	1.7288	2.65	0.56	5	0.97	1.40	9.8	61.56	0.24	18	54.79	54.61	100	450	RRJ-20% FCR	0.29	1.79	0.17
D4-A2	0.5858	3.23	0.70	5	0.41	1.81	10.0	61.08	0.31	56	54.61	54.05	100	525	RRJ-20% FCR	0.43	1.99	0.47
					-			-	-				-			-	-	-
A2-A3	0.9484	4.18	0.56	5	0.53	2.34	10.4	59.77	1.26	84	54.05	51.35	31	750	RRJ-20% FCR	2.00	4.52	0.31
A3-A4	1.0100	5.19	0.56	5	0.57	2.91	10.7	58.94	1.35	67	51.35	49.05	29	1050	RRJ-20% FCR	5.06	5.84	0.19
					-			-	-				-			-	-	-
C1-C2	0.6846	0.68	0.56	5	0.38	0.38	8.0	67.41	0.07	85	50.44	49.59	100	300	RRJ-20% FCR	0.10	1.37	1.04
C2-A4	1.3120	2.00	0.56	5	0.73	1.12	9.0	63.87	0.20	54	49.59	49.05	100	450	RRJ-20% FCR	0.29	1.79	0.50
					-			-	-				-			-	-	-
A4-A5	1.0590	8.25	0.56	5	0.59	4.62	10.9	58.45	1.62	70	49.05	48.82	298	1200	RRJ-20% FCR	2.26	2.00	0.58
A5-A6	1.1150	9.36	0.56	5	0.62	5.24	11.5	56.99	1.70	67	48.82	48.60	312	1200	RRJ-20% FCR	2.21	1.95	0.57
A6-A7	1.1700	10.53	0.56	5	0.66	5.90	12.1	55.65	1.78	72	48.60	48.35	288	1200	RRJ-20% FCR	2.30	2.03	0.59
A7-A8	1.2290	11.76	0.74	5	0.91	6.81	12.7	54.35	1.90	28	48.35	48.25	280	1200	RRJ-20% FCR	2.33	2.06	0.23
A8-A9	0.9002	12.66	0.74	5	0.67	7.47	12.9	53.87	1.99	63	48.25	48.05	315	1200	RRJ-20% FCR	2.20	1.94	0.54
A9-A10	0.0000	12.66	0.74	5	-	7.47	13.4	52.77	1.97	35	48.05	47.95	350	1200	RRJ-20% FCR	2.08	1.84	0.32
A10-A11	0.7713	13.43	0.74	5	0.57	8.04	13.8	52.15	2.04	30	47.95	47.65	100	1200	RRJ-20% FCR	3.90	3.45	0.15
A11-A12	0.0000	13.43	0.74	5	-	8.04	13.9	51.87	2.03	23	47.65	47.60	460	1350	RRJ-20% FCR	2.49	1.74	0.22
A12-A13	0.0000	13.43	0.74	5	-	8.04	14.1	51.45	2.02	63	47.60	47.41	332	1350	RRJ-20% FCR	2.93	2.05	0.51
A13-A14	0.7474	14.18	0.70	5	0.52	8.57	14.6	50.52	2.07	26	47.41	47.35	433	1350	RRJ-20% FCR	2.56	1.79	0.24
A14-A15	0.0000	14.18	0.74	5	-	8.57	14.9	50.09	2.06	50	47.35	47.18	294	1350	RRJ-20% FCR	3.11	2.17	0.38
A15-A16	0.0000	14.18	0.74	5	-	8.57	15.3	49.43	2.05	15	47.18	47.14	375	1350	RRJ-20% FCR	2.76	1.93	0.13
A16-A17	7.5000	21.68	0.56	5	4.20	12.77	15.4	49.21	2.62	51	47.14	47.00	364	1350	RRJ-20% FCR	2.80	1.95	0.44
					-			-	-				-			-	-	-
B1-B2	0.3448	0.34	0.18	5	0.06	0.06	8.0	67.41	0.01	77	51.95	51.18	100	300	RRJ-20% FCR	0.10	1.37	0.94
B2-B3	0.6770	1.02	0.47	5	0.32	0.38	8.9	64.19	0.07	71	51.18	50.47	100	300	RRJ-20% FCR	0.10	1.37	0.86
B3-B4	1.0490	2.07	0.47	5	0.49	0.87	9.8	61.53	0.15	69	50.47	49.78	100	375	RRJ-20% FCR	0.18	1.59	0.72
B4-B5	1.1060	3.18	0.47	5	0.52	1.39	10.5	59.50	0.23	69	49.78	49.09	100	450	RRJ-20% FCR	0.29	1.79	0.64
B5-B6	1.1660	4.34	0.74	5	0.86	2.26	11.2	57.84	0.36	71	49.09	48.38	100	525	RRJ-20% FCR	0.43	1.99	0.60
B6-B7	1.4650	5.81	0.74	5	1.08	3.34	11.8	56.39	0.52	98	48.38	47.40	100	600	RRJ-20% FCR	0.61	2.17	0.75

APPENDIX D OVERLAND FLOW PATH CALCULATIONS

Reference	Q_{100}	Pipe Full Flow Q_{full}	Design Flow Q_{gap}	Length L	NS Elev u/s	NS Elev d/s	Grade (1 in …)	Manning's 'n'	Side Slope	Base Width (B_1)	Channel Depth (d_1)	Channel x-sect Area	Velocity v_p	V * D	Channel Top Width	Channel Q_{full}
	m^3/s	m^3/s	m^3/s	m	m	m				m	m	m^2	m/s		m	m^3/s
A1-A2	1.89	2.03	-	68	57.80	55.55	30	0.035	3	2.7	0.00	-	-	-	2.70	-
A2-A3	3.15	2.00	1.15	84	55.55	53.15	35	0.035	3	2.7	0.23	0.8	1.58	0.36	4.08	1.23
A3-A4	3.43	5.06	-	67	53.15	52.30	79	0.035	3	2.7	0.23	0.8	1.05	0.24	4.08	0.82
A4-A5	4.31	2.26	2.05	70	52.30	51.95	200	0.030	3	6.4	0.30	2.2	0.97	0.29	8.20	2.12
A5-A6	4.56	2.21	2.36	67	51.95	51.50	149	0.030	3	6.4	0.30	2.2	1.12	0.34	8.20	2.46
A6-A7	4.82	2.30	2.52	72	51.50	50.80	103	0.035	3	6.4	0.30	2.2	1.16	0.35	8.20	2.54
A7-A8	5.19	2.33	2.86	28	50.80	50.45	80	0.035	3	6.4	0.30	2.2	1.31	0.39	8.20	2.88
A8-A9	5.48	2.20	3.29	63	50.45	50.10	180	0.035	3	13.0	0.30	4.2	0.91	0.27	14.80	3.80
A9-A10	5.40	2.08	3.32	35	50.10	49.95	233	0.035	3	13.0	0.30	4.2	0.80	0.24	14.80	3.34
A10-A11	5.62	3.90	1.73	30	49.95	49.80	200	0.035	3	13.0	0.30	4.2	0.86	0.26	14.80	3.60
A11-A12	5.60	2.49	3.11	23	49.80	49.70	230	0.035	3	13.0	0.30	4.2	0.81	0.24	14.80	3.36
A12-A13	5.57	2.93	2.64	63	49.70	49.50	315	0.035	3	13.0	0.30	4.2	0.69	0.21	14.80	2.87
A13-A14	5.75	2.56	3.18	26	49.50	49.45	520	0.020	3	13.0	0.30	4.2	0.94	0.28	14.80	3.91
A14-A15	5.71	3.11	2.60	50	49.45	49.35	500	0.020	3	13.0	0.30	4.2	0.96	0.29	14.80	3.99
A15-A16	5.65	2.76	2.90	15	49.35	49.30	300	0.020	3	13.0	0.30	4.2	1.24	0.37	14.80	5.15

Reference	Q_{100}	Pipe Full Flow Q_{full}	Design Flow Q_{gap}	Length L	NS Elev u/s	NS Elev d/s	Grade (1 in …)	Manning's 'n'	Side Slope	Base Width (B_1)	Channel Depth (d_1)	Channel x-sect Area	Velocity v_p	V * D	Channel Top Width	Channel Q_{full}
	m^3/s	m^3/s	m^3/s	m	m	m				m	m	m^2	m/s		m	m^3/s
A16-A17	7.46	2.80	4.66	51	49.30	48.95	146	0.035	4	2.0	0.25	0.8	0.77	0.19	4.00	0.58

Reference	Princes Highway Flow Q_{100} -Channel Q_{full}	Flow Depth (d_2)	Base Width (B_2)	Manning's 'n'	Side Slope	Area A	Velocity v	Freeboard	Top Width with freeboard	Channel Capacity Q_{full}
	m^3/s	m	m			m^2	m/s	m	m	m^3/s
A16-A17	4.08	0.25	9.90	0.02	2	2.6	1.58	0.00	10.90	4.11

Reference	Q_{100}	Pipe Full Flow Q_{full}	Design Flow Q_{gap}	Length L	NS Elev u/s	NS Elev d/s	Grade (1 in …)	Manning's 'n'	Side Slope	Base Width (B_1)	Channel Depth (d_1)	Channel x-sect Area	Velocity v_p	V * D	Channel Top Width	Channel Q_{full}
	m^3/s	m^3/s	m^3/s	m	m	m				m	m	m^2	m/s		m	m^3/s
B1-B2	0.04	0.38	-	77	72.10	60.10	6	0.035	3	1.5	0.00	-	-	-	1.50	-
B2-B3	0.23	0.30	-	71	60.10	53.30	10	0.035	3	1.5	0.00	-	-	-	1.50	-
B3-B4	0.50	0.36	0.14	69	53.30	50.35	23	0.035	3	1.5	0.10	0.2	1.14	0.11	2.10	0.20
B4-B5	0.76	0.27	0.50	69	50.35	49.85	138	0.035	3	1.5	0.30	0.7	0.86	0.26	3.30	0.62
B5-B6	1.19	0.40	0.80	71	49.85	49.50	203	0.035	3	2.7	0.30	1.1	0.76	0.23	4.50	0.82
B6-B7	1.72	0.59	1.13	98	49.50	48.60	109	0.035	3	2.7	0.30	1.1	1.04	0.31	4.50	1.13