APPENDIX 2

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Northern Growth Area - Lakes Entrance Development Concept Plan Hydrology

July 2012

smec urban consulting group

EAST GIPPSLAND SHIRE COUNCIL

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Cover Photo: View across Guillot property, Water Technology site visit October 2011

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1. INTRODUCTION

In late 2011, five separate planning scheme amendment requests were lodged for land at the edge of the existing Lakes Entrance Township. The total area covered by the five requests covered approximately 150 hectares, located on the east side of Ostler Road; Myer Street extending north from Outlook Avenue and the Lakes Entrance Secondary College to a point adjoining the intersection on Blairs Road; and Ostlers Road in Lakes Entrance (refer Figure 1-1). This land is situated away from the low-lying main township region and is to be rezoned for residential use. Collectively the area planed for rezoning and residential development is known by the East Gippsland Shire Council (EGSC) as the Northern Growth Area (NGA).

EGSC initially engaged SMEC Urban to oversee the preparation of an Outline Development Plan (ODP). The ODP is a detailed plan of the development that establishes the vision and objectives for development of the area to provide a structured plan for growth. It is split into three key focus areas: Infrastructure; Environment; and Infrastructure Provision and Maintenance responsibilities.

As part of the ODP process Water Technology was requested to assist with the Environment focus area, specifically the hydrological aspects relevant for the NGA. This report was prepared and submitted in mid November 2011. The analysis at the time was high level with more detailed work slated for completion as the planning processes progressed. Water Technology was asked to undertake specific investigations into the Protection of Waterways, Water Sensitive Design and to a lesser degree Drainage Infrastructure.

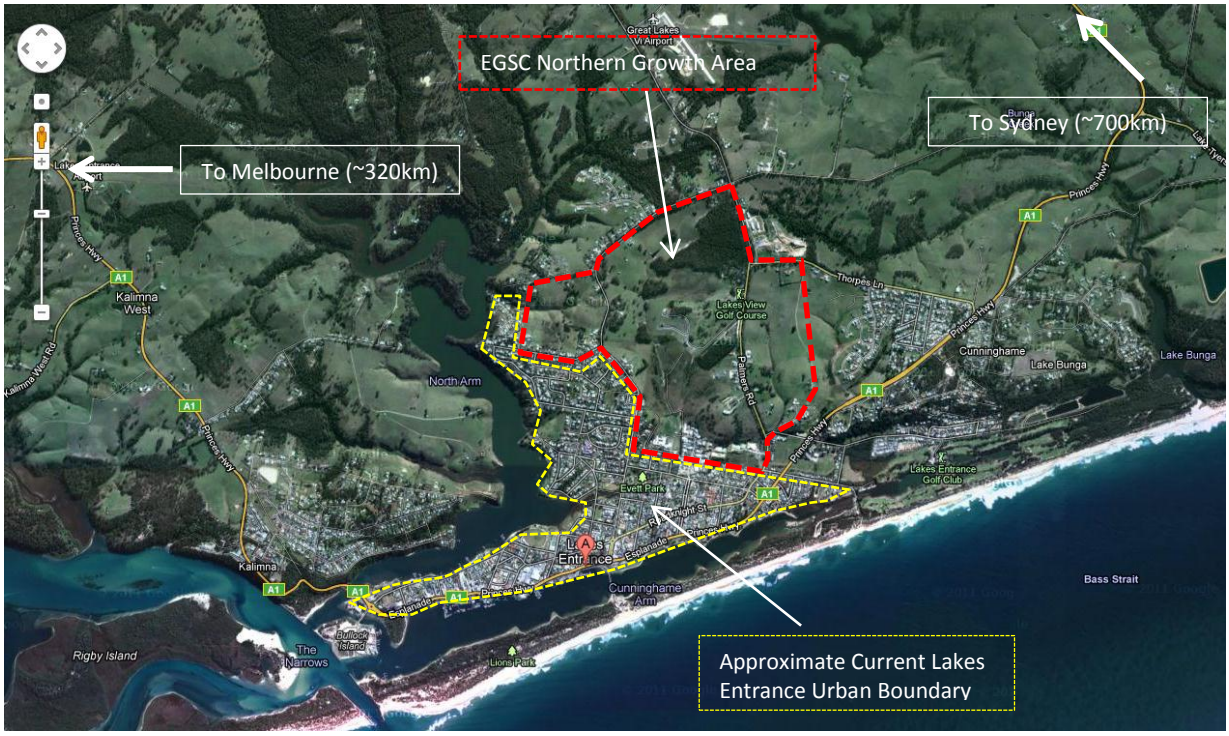


Figure 1-1 EGSC Northern Growth Area (Background image - Google Maps 2011)

The scope for the 2011 investigation comprised:

- A site inspection, including a meeting with SMEC Urban and/or where appropriate the client;
- Review of relevant data (hydrologic/survey etc.) for the ODP area;
- Review of site survey plans and preliminary assessment of drainage issues across the ODP area;
- Preliminary review of proposed development plans (appropriate to their level of detail) for the ODP area;
- Preparation of a report describing the main water quality/drainage issues for the ODP area describing the constraints that these issues may place on development;
- Discussion on the way in which the proposed development may fit into these constraints;
- Determination of the designated waterways and the restrictions placed on development associated with their presence;
- Conceptual sizing and locations of attenuation basins across the ODP;
- Preliminary designated waterway feature offset requirements (where appropriate);
- Commentary on impacts of external catchments contributing flows to the five developments; and
- Recommendations as to what further work should be carried out as part of the detailed assessments and setting out the associated timing and budget. Issues relating to water quantity (drainage) and quality (storm water) management will be assessed.

In further discussions between SMEC Urban and the landowners in 2012, it was determined that a Development Contributions Plan (DCP) should be developed to encompass the entire ODP area. As a result of this, a broader and more detailed assessment of the ODP area is required to determine / refine the storm water management requirements. Additionally, storm water management concept design costs are required to inform joint drainage infrastructure decisions. As detailed in our proposal dated 17th July 2012, tasks undertaken in this updated report included:

- Review of hydrology and creation of an appropriate RORB hydrological model of the study area. This task will provide a solid basis for concept design of storage and water quality features, and is a necessary step for establishing credible cost determinations. The RORB modelling will include basin storage sizing and concept outlet configuration;
- Review and update of the existing MUSIC model to refine the actual expected water quality requirements for the DCP to a level that will permit concept sizing and concept costing of these assets;
- Combine the basins and water quality treatment assets into a conceptual integrated drainage and storm water management design for the DCP. This will include descriptive and map based presentation of the appropriate concept designs;
- Based on the revised storage volume and water quality estimations, develop appropriately detailed costs for the proposed system based on known industry estimates. The cost assessment will include a recommendation on separate land parcel contributions based on the individual impacts of each lot area; and
- Preparation of a report describing the study process undertaken and recommendations / conclusions. Map data sets will be utilised as appropriate in the report.

Further to the above scope, and in response to discussions between SMEC Urban and Water Technology on the 20th July, a supplementary scope was submitted by Water Technology on the 23rd July to conduct 1D-2D flood modelling of a section of the Eastern Creek drainage line. Specifically this scope provided for the following work:

- Construct 1D/2D hydraulic flood model of Eastern Creek between Palmers Road and Carpenter Street/Outlet to North Arm. The creek and surface will be based on the available LiDAR data together with on-site tape measure estimates of structure and channel dimensions recorded by Water Technology;
- Model existing conditions (pre development) and assess flood velocities, depths and Water Surface Elevation;
- Model developed conditions with the GHD works in place and assess flood velocities, depths and Water Surface Elevation;
- Model developed conditions with the Northern Growth Area basins in place and assess flood velocities, depths and Water Surface Elevation; and
- Prepare brief reporting on model set-up and conclusions regarding the impacts and benefits of development and associated works in each case.

This updated report details the work undertaken in July 2012, and is structured as a standalone report that builds upon the November 2011 document. As such, portions of both reports are common (particularly the descriptive sections), but all hydrological modelling and water quality results represent the most recent work completed.

2. DEVELOPMENT PROPOSAL

The area identified for development includes five principal individual parcels which are proposed for residential development. The ODP (2011) documentation showed five development areas in different stages of planning, with the more progressed developments showing detailed plans, while others showed more conceptual layouts. The individual Proponents involved in the ODP had nominated reserve areas that were largely associated with designated waterway features within the developments. A review of available data for land within the Development Concept Plan (DCP) area shows that the proposed developments are impacted to some degree by designated waterways (shown as blue lines in Figure 2-1). Current planning overlays show the areas are not subject to either a Flood Overlay (FO) or Land Subject to Inundation overlay (LSIO).

The available development plans supplied to Water Technology, show the developments will consist of a significant portion of residential development coupled with large open space reserves. A summary of each development has been provided in Table 2-1. Where lot sizes were not available it was assumed that they will consist of standard residential lots. Using this assumption, a basic land assessment was determined for each of the catchment areas within the DCP area. The adopted land characteristics are shown in Table 2-2.

Table 2-1 Catchment Characteristics

Catchment	~ Dev Size (Ha)	Existing Land Zone
1	4.3	FZ3 - Farming
2	9.1	FZ3 - Farming
3	5.5	FZ3 - Farming
4	19.6	FZ3 - Farming
5	19.0	FZ3 - Farming
6	5.2	FZ3 - Farming
7	4.2	FZ3 - Farming
8	5.3	FZ3 - Farming
9	13.0	FZ3 – Farming & R1Z Residential
10	1.0	R1Z Residential
11	3.7	R1Z Residential
12	9.9	FZ3 - Farming
13	12.5	FZ3 - Farming
14	7.0	FZ3 - Farming

Figure 2-1 Available Concept Development Plan (Source: SMEC Urban)

Table 2-2 Catchment Hydrology Characteristics

Catchment Name	Area (Ha)	RORB sub-Catchments*
1	4.3	X
2	9.1	S, T, U, V and W
3	5.5	J
4	19.6	A,B, C, D, E, F and G
5	19.0	L, M, I, H,J, K, M and N
6	5.2	P
7	4.2	A,B, C, D, E, F, G, H, I, J, K, L, M, N, O, P and Q
8	5.3	A,B, C, D, E, F, G, H, I, J, K, L, M, N, O, P, Q and R
9	13.0	AD, AE, AF, AH and AG
10	1.0	AS
11	3.7	AR
12	9.9	Y and Z
13	12.5	Y, Z and AA
14	7.0	Y, Z, AA, AB and AC

* Including external catchments

Under existing catchment conditions, the majority of the NGA region drains south towards the town's major storm water collection channel (Eastern Creek) in the lower lying and flatter portion of Lakes Entrance, and then moves west, discharging into the North Arm (part of the Gippsland Lakes system). The western most development (catchment 1 and 2) flows directly west and also discharges into the North Arm waterway.

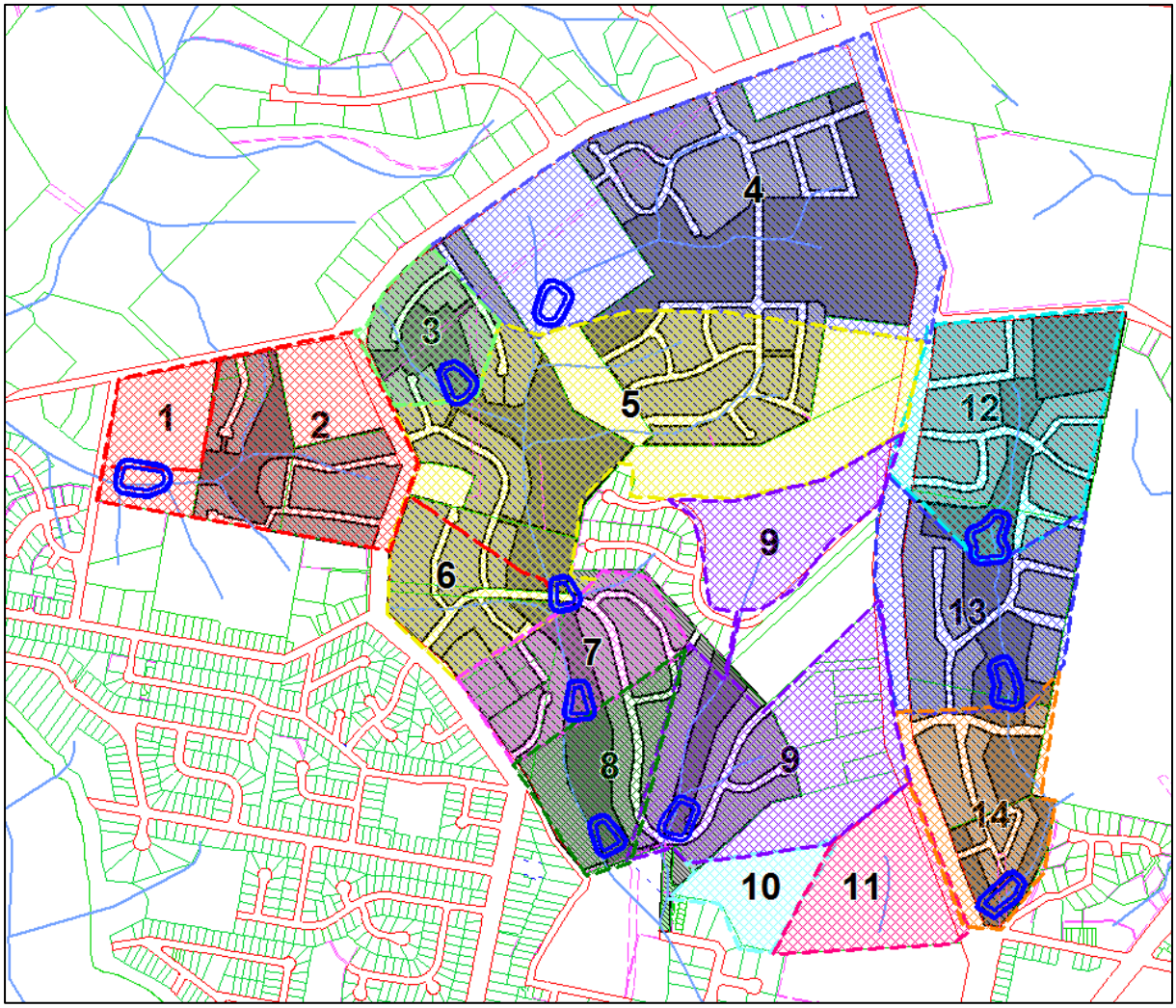


Figure 2-2 **Waterway features, internal catchment boundaries and available lot layouts within DCP (background image Vic. State Govt. Planning Maps online 2011)**

3. PRELIMINARY ASSESSMENT NOTES

A preliminary assessment was undertaken in 2011 to ascertain any flooding, water quality or quantity issues as a result of the development. While this work has been refined in the RORB modelling undertaken for the current scope, a number of the initial comments and notes remain relevant for this updated report.

3.1 General Drainage Issues

The EGCMA is the referral authority for any drainage issues on site. As there are recognised impacts from works adjacent or on designated waterways the EGCMA will need to be consulted during the planning process. For any new subdivision the EGCMA and EGSCC will typically require the developer to demonstrate the following:

- Maintenance of pre-development peak flows;
- Maintenance of conveyance/storage on site;
- No negative impacts on flood levels for the upstream and downstream properties;
- Consideration of water quality requirements; and
- A 'net gain' for the waterway through the development.

A site visit was conducted on the 26th of October 2011. The purpose of the visit was to develop an understanding of the site including drainage under existing conditions, likely drainage conditions under proposed development conditions and any significant site constraints which could present drainage / stormwater issues for the proposed development. A location referenced photo log of the site visit is included in Appendix A.

3.1.1 Existing Conditions Commentary

Based on site contours (0.5m resolution) supplied by the EGSC, and observations made during the 2011 site visit, the following general drainage notes are presented for context.

Catchments 1 and 2

The topography data supplied by EGSC identifies three clear drainage paths within these catchments. The area consists of generally flatter portions on the areas of higher ground adjacent to quite steep well defined channel features. The topography falls away dramatically along the drainage lines where defined waterways meet. Slopes in the higher portion of the area are typically between 8 to 10% while in the lower sections they are as high as 28%. Under existing conditions, the majority of the catchment areas are under pasture.

The majority of the total site area contributes to flows at the principal discharge location of the site, leading ultimately into Eastern Creek, with the balance discharging to the north onto Blairs Road.

Catchments 3, 5 (western portion), 6 - 11

The topography data supplied by EGSC identifies two clear drainage paths within these areas. The largest to the west has a total regional catchment of approximately 123 hectares. The central and lower portions of the drainage path display quite significant side slopes (>20%) indicating fast catchment response times under existing conditions. Under existing conditions the western drainage path is characterised by open pasture farm land scattered with a few houses on the upper reaches along Ostlers road.

The drainage path to the east has a much smaller regional catchment (~36.5ha), which is bound by Palmers road and the southern boundary of the Lakes View Golf course. It displays similar (steep) side slopes to the western drainage path.

Catchments 4 and 5 (eastern portion)

The topography data supplied by EGSC identifies two main drainage paths within catchment 4 that join up and flow south into catchment 5. Catchments 4 and 5 are heavily vegetated in their eastern parts, and slopes are typically milder than in the steep gully lines associated with the more southern areas.

Catchments 12-14

These catchment areas consist of moderately sloped developable areas draining into a single well defined and relatively steep gully line. The gully line has several existing farm dams associated with it. The topography and dimensions of these catchment areas are very similar in context to the adjoining Merrangbaur Heights development, located to the immediate east. The gully line running through this area exhibits milder slopes than the gully lines to the west.

3.1.2 Developed Conditions

Although the development plan is in preliminary form, and detailed lot layouts for all areas are not available, a concept development layout was supplied to Water Technology for discussion and review. The appropriateness of the location and size of open space reserves was considered against the existing topography and calculations for flood storage. Where the allocated reserve locations were sub optimal, alternate locations have been proposed that take advantage of natural low points and existing drainage paths within the development. Similarly, in areas where no allowance has been made for designated waterways, these areas have been identified as requiring further consideration.

3.1.3 Storage & Drainage Features

Due to the generally steep nature of the development area, little flexibility is available for modification of existing flow paths from storm events to the water treatment and conveyance features within the development areas. The amount (and costs) of earthworks that would be required to achieve significant changes in finished development contours will likely be prohibitive, and developments will largely need to accommodate the existing landforms. Where feasible, existing farm dam features have been proposed for the attenuation storages. Where they are proposed to be retained, engineering modifications will be required for the existing farm dams, including earthworks to increase / decrease storage volume, incorporation of low flow pipes to maintain storage capacity, and construction of scour protected spillways.

Likely storage and drainage feature locations are depicted in Figure 4-3 and Figure 4-4.

3.1.4 Declared waterways

Designated waterways exist across the NGA and in all proposal areas. The presence of declared waterways creates a number of obligations for the individual Proponents. Specifically these requirements will typically include preparation of:

- A Surface Water Management Strategy (SWMS) that should include:
  - Detailed / functional design of stormwater drainage scheme system
  - Detailed / functional design of surface water retardation storages
  - Detailed / functional design of water quality treatment (wetlands, rain gardens etc.)
- A Waterway Management Plan (WMP) for the EGCMA that should include:
  - Design of best practice waterway management options and maintenance requirements to ensure the short and long term health and function of the designated waterways

The EGCMA promotes a minimum buffer distance of 30m either side of the waterway centre line (or waterway top of bank for larger waterways). The CMA's general preference will be for this buffer area to be left undisturbed where feasible. Where modifications (re-alignment / removal / civil works etc.) are proposed for any designated waterways, the following steps are typically required:

- Consultation with the CMA - in this case the EGCMA;
- Calculation of relevant offset requirements as described by the EGCMA - typically 150% offset in waterway areas for any affected buffer zones (assuming a 30m buffer); and
- Completion of a Works on Waterways permit process – submitted to and approved by the EGCMA.

4. DEVELOPMENT CONCEPT PLAN (DCP) AREA RORB MODELLING

4.1 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.

Four new RORB models were constructed for this study for four different scenarios, the four scenarios included:

- Rural Conditions – The Model assumes that all catchments are rural and all reaches are natural
- Existing Conditions – The model assumes fraction imperviousness values depending on existing planning scheme zones. Reaches are input into the model to replicate the current drainage infrastructure
- Developed Conditions – The model assumes fraction imperviousness values depending on existing and future planning scheme zones. Reaches are input into the model to replicate the current and future drainage infrastructure. Design Storages are input into this model
- Developed Conditions with GHD Retarding Basins – This model is the same as the developed conditions model except the design storages are replaced with the previously designed storages by GHD.

4.2 Rational Method Calculations

4.2.1 Overview

A Rational Method estimate was required for reconciliation in five locations in the RORB model. The points for reconciliation are shown in Figure 4-1.

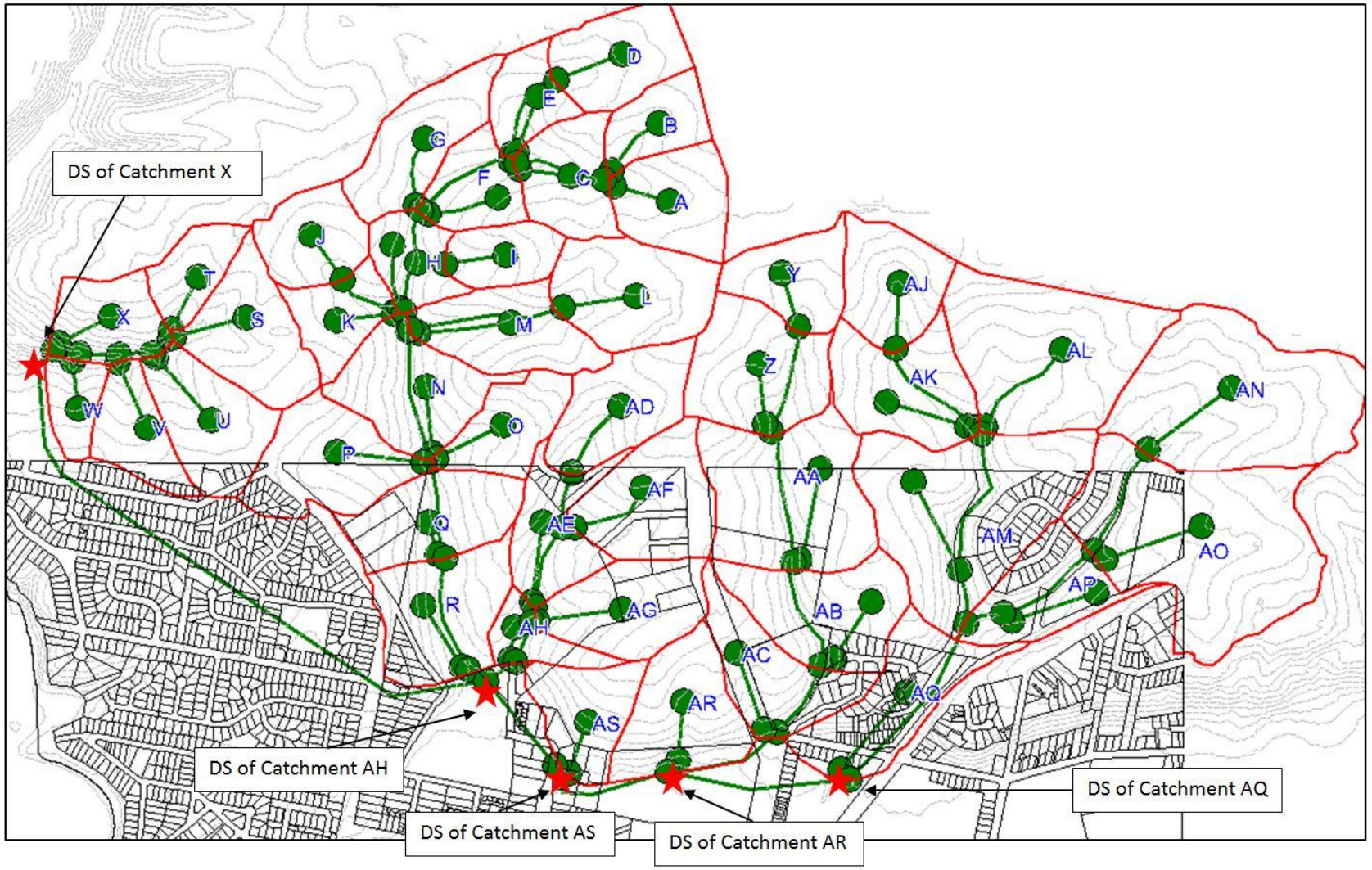


Figure 4-1 Rational Method Reconciliation Locations

4.2.2 Time of Concentration Calculation

The Adams method was used for calculating the time on concentration for each of the catchments outlined above. The Adams method was chosen because the reconciliation of the RORB model to the rational method was done for an undeveloped scenario where every sub area was assumed to be rural. The "natural" reach type was assumed for each RORB reach type under the undeveloped scenario. The formula for the Adams method is:

$$Tc = 0.76 \times Area^{0.38}$$

4.2.3 IFD Parameters and Rainfall Intensities

An IFD chart specific to Lakes Entrance was created based on parameters from the Bureau of Meteorology as shown in Table 4-1.

Table 4-1 IFD Parameters for Lakes Entrance

2y 1h (mm/h)	2y 12h (mm/h)	2y 72h (mm/h)	50y 1h (mm/h)	50y 12h (mm/h)	50y 72h (mm/h)	Skew	F2 Value	F50 Value
19.51	4.61	1.37	36.72	8.61	2.88	0.3	4.21	15.43

4.2.4 Rational Method 100 Year ARI Peak Flow Estimates

A Rational Method analysis was undertaken for each reconciliation location in accordance with the methodology outlined in Book 2 of Australian Rainfall and Runoff (*IEAust, 1997*). The equation is as follows:

Q_{100} = C.I.A/360

Where:

- Q_{100} is the flow in m^3/s for the 100 year ARI design event;
- C is the runoff coefficient;
- I is the rainfall intensity specific to the area, corresponding to the t_c (time of concentration of the catchment); and,
- A is the area of the catchment in hectares.

Using the travel time and design rainfall estimates, rainfall intensities for the time of concentration were interpolated via methods prescribed in Australian Rainfall and Runoff and a Rational Method Estimate (Q) was calculated as shown in Table 4-2.

Table 4-2 Rational Method Calculations

Location	FI	I (mm/hr)	tc (mins)	Fy	C_{10}	Total Area (ha)	Q_{100} (m^3/s)
DS Catchment X	0.1	68.65	30.76	1.2	0.139	35.5	**1.75**
DS Catchment AH	0.1	46.92	54.62	1.2	0.139	160.8	**5.41**
DS Catchment AS	0.1	101.01	16.23	1.2	0.139	6.6	**0.48**
DS Catchment AR	0.1	59.58	38.28	1.2	0.139	63.1	**2.70**
DS Catchment AQ	0.1	50.96	48.39	1.2	0.139	116.9	**4.27**

4.3 RORB Model Reconciliation

The pre-developed conditions RORB model was reconciled to match the 100 year peak flow estimates from the Rational Method Calculation shown above at each reconciled location (interstation area). This was achieved by varying the Kc parameter. All run parameters are shown in Table 4-3.

Table 4-3 RORB model reconciliation and parameters

Reconciliation Location	Kc	IL	RoC	m	Critical storm duration	Q RORB m^3/s	Q Rational Method m^3/s	Difference in Q m^3/s
DS Catchment X	0.7	20	0.6	0.8	4.5hr	1.76	1.75	-0.01
DS Catchment AH	2.15	20	0.6	0.8	9hr	5.47	5.41	-0.06
DS Catchment AS	0.2	20	0.6	0.8	2hr	0.53	0.48	-0.05
DS Catchment AR	1.15	20	0.6	0.8	9hr	2.67	2.7	0.03
DS Catchment AQ	2.1	20	0.6	0.8	9hr	4.31	4.27	0.04

4.4 Existing Conditions RORB Model Development

The RORB model was created in RORBV6.15 using the MiRORB tool. This allows the model to be viewed spatially in the RORB GUI (Graphical User Interface).

4.4.1 Subareas

The catchment was broken into 44 sub-areas. The delineation of the sub-areas followed the drainage lines and drainage infrastructure in the 100 year ARI.

4.4.2 Reach Types

Reach types were assigned based on the overland flow conditions along that reach. Flow along roads and piped systems were assigned to be 'type 3', flows across open grassed areas or natural waterways assigned 'type 1' and reaches that were deemed to be a mixture of underground drainage and natural drainage alignment were assigned 'type 2'. 'Type 5' dummy reaches were used throughout the model also to create hydrographs in desirable locations. No 'type 4' drowned reaches were used in the model.

4.4.3 Nodes

Nodes have been included at the downstream end of each sub catchment so that a flow from each sub catchment can be generated.

4.4.4 Interstation Areas

The RORB model has been adapted to run with 'interstation areas' to enable the Rational Method to be reconciled at each of the reconciliation locations. Figure 4-2 displays the RORB structure.

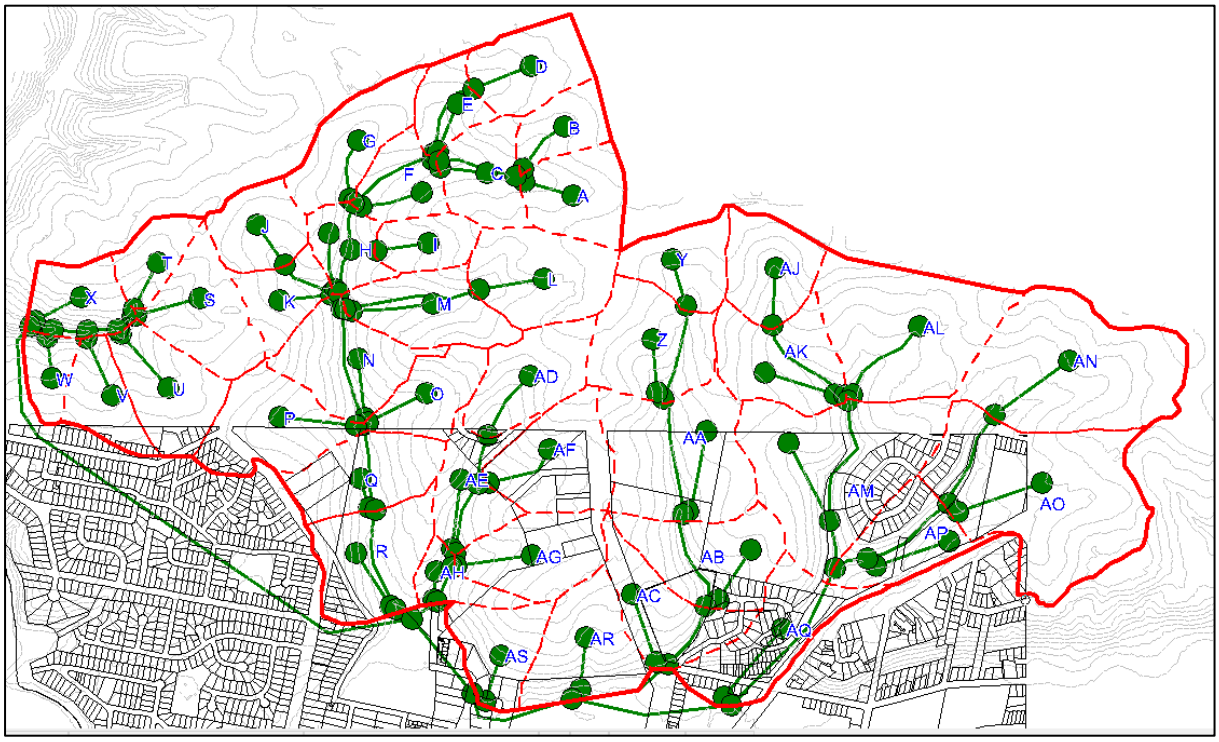


Figure 4-2 RORB Model Structure

4.4.5 Fraction Impervious Data

Fraction impervious values were calculated for each individual RORB sub-catchment. Fraction impervious values were assigned based on the existing and future land uses, with the typical fraction impervious value for each different land use applied as detailed below in Table 4-4.

Table 4-4 Design impervious area coefficients

Land Use	Typical FI
Future Drainage Reserve	0.10
Conservation Area	0.10
Golf Course	0.10
Cemetery	0.70
Open Space	0.10
Existing R1Z	0.45
Existing LDRZ	0.20
Rural area	0.10
RDZ1	0.70
PPRZ	0.10
INZ1	0.90
PUZ2	0.20
Commercial Mix Use	0.90
Future Low Density Residential	0.40
Future Standard Density Residential	0.60
Future Medium Density Residential	0.75

Table 4-5 Existing Conditions Fraction Impervious Values

RORB Sub Area	Area (km2)	FI (Existing Conditions)
A	0.072	0.183
B	0.046	0.172
C	0.066	0.1
D	0.073	0.145
E	0.043	0.104
F	0.051	0.1
G	0.081	0.109
H	0.045	0.1
I	0.043	0.1
J	0.06	0.104
K	0.05	0.1
L	0.103	0.134
M	0.084	0.1
N	0.094	0.155
O	0.058	0.247
P	0.096	0.213
Q	0.085	0.11
R	0.101	0.194
S	0.088	0.101
T	0.041	0.124
U	0.083	0.335
V	0.052	0.405
W	0.032	0.358
X	0.059	0.1
Y	0.062	0.119
Z	0.105	0.199
AA	0.152	0.11
AB	0.102	0.14
AC	0.116	0.368
AD	0.082	0.145
AF	0.096	0.297
AE	0.064	0.135
AG	0.086	0.129
AH	0.031	0.1
AJ	0.074	0.161
AK	0.095	0.104
AL	0.177	0.295
AM	0.203	0.217
AN	0.206	0.45
AO	0.239	0.45
AP	0.076	0.45
AQ	0.099	0.385
AS	0.066	0.304
AR	0.094	0.105

4.4.6 RORB Parameters

The RORB model was run with AR&R 1987 method with an aerial reduction factor area of 0.0km^2. The Initial Loss applied was 10mm (urban catchments) and 20mm (rural catchments). Temporal patterns were fully filtered. Runoff coefficients used are shown in Table 4-6.

Table 4-6 Runoff Coefficient for ARI events

ARI Event	Runoff Coefficient
5 Year	0.25
10 Year	0.35
20 Year	0.45
50 Year	0.55
100 Year	0.60

4.5 Existing Conditions RORB Model Results

The existing conditions scenario RORB modelling has been run with the parameters outlined above, Table 4-7 shows the 100 year peak flows for the existing conditions modelling scenario.

Storage Basin locations outlined later in this report have had print nodes added to that location in the RORB model. This allows comparison of peak flows to be performed when sizing the storages.

Table 4-7 Existing Conditions Peak Flow Results

Hydrograph ID	Hydrograph Location:	100 Year ARI, Existing FI	
		Peak Flow (m3/s)	Critical Duration
1	RB3 Location	1.86	9hr
2	RB2 Location	0.39	4.5hr
3	RB4 Location	4.00	9hr
4	RB5 Location	4.08	9hr
5	RB6 Location	4.19	9hr
6	RB7 Location	1.64	9hr
7	DS Catchment AH	5.63	9hr
8	DS Catchment AQ	4.72	9hr
9	RB8 Location	0.96	2hr
10	RB9 Location	1.32	9hr
11	RB10 Location	2.06	9hr
12	DS Catchment AR	2.15	9hr
13	DS Catchment AS	0.68	2hr
14	DS Catchment X (RB1)	2.09	4.5hr

4.6 Developed Conditions RORB Model Results

The RORB model developed for the existing conditions scenario was modified to represent the Lakes Entrance Northern Growth Area ODP supplied by SMEC Urban. Modifications were made to the RORB reaches, subarea Fraction Imperviousness were altered, and storage nodes were added.

Storage Basins are proposed to be located throughout the proposed new development. The location of the proposed basins can be seen below in Figure 4-3 and Figure 4-4.

Table 4-8 shows a summary of the storages which have been designed through an iterative process in RORB.

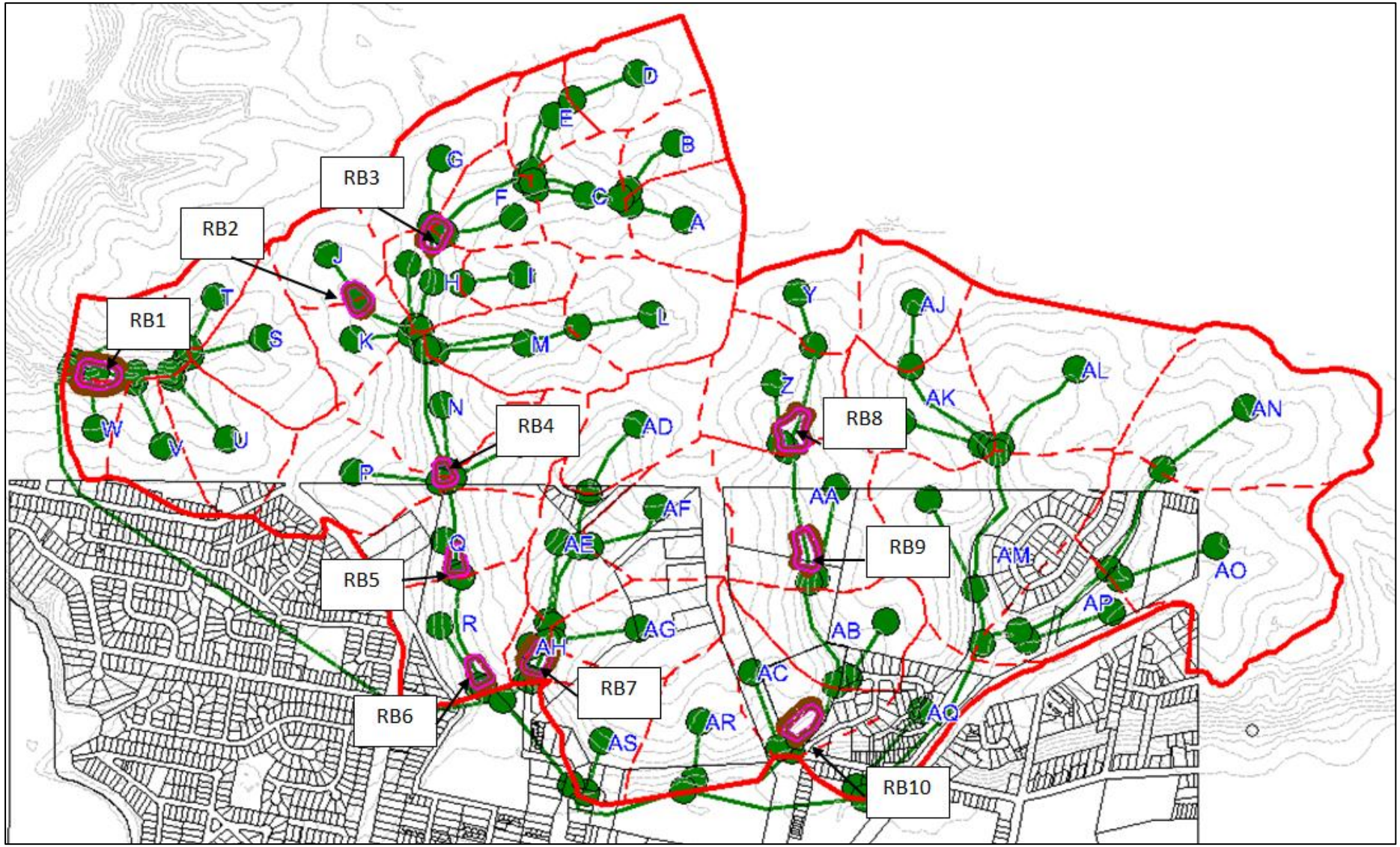


Figure 4-3 Proposed Storage Basin Locations (RORB)

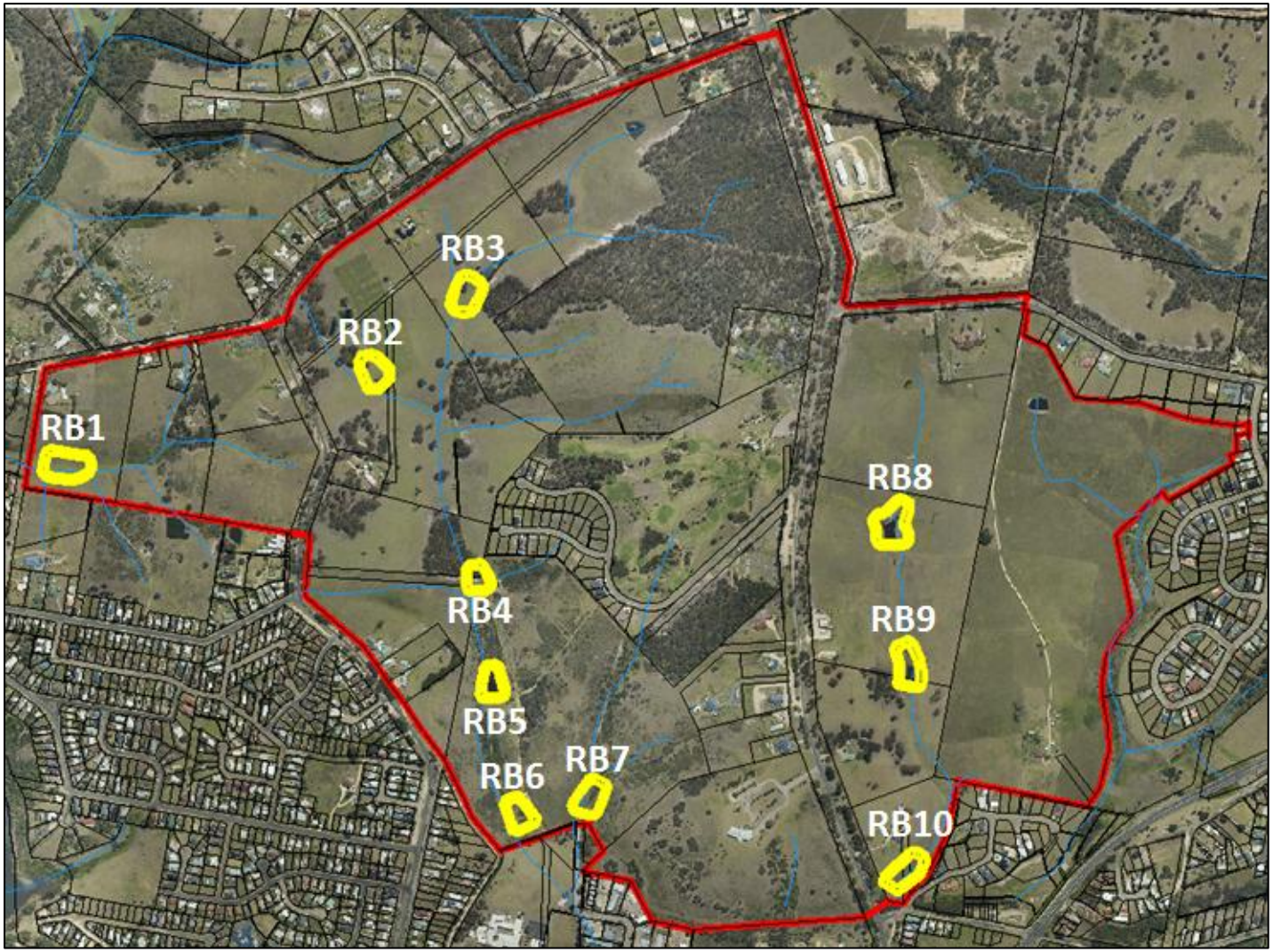


Figure 4-4 Proposed Storage Basin Locations (aerial imagery)

Table 4-8 Summary of Storages

RB ID	RB1	RB2	RB3	RB4	RB5	RB6	RB7	RB8	RB9	RB10
US Invert Level (mAHD)	16.0	43.0	39.0	25.0	22.0	16.0	18.0	45.0	34.0	18.0
No of pipes	2	2	1	2	2	2	1	3	1	1
Diameter (mm)	900	450	1050	900	1200	900	900	525	750	900
Length (m)	15	15	15	15	15	15	15	15	15	15
Slope (%)	1	1	1	1	1	1	1	1	1	1
INFLOW (m^3/s)	3.38	1.39	2.61	5.11	4.04	4.18	2.48	3.51	4.67	2.55
OUTFLOW (m^3/s)	2.01	0.34	1.77	3.95	4.04	4.04	1.64	0.96	1.27	1.96
Outflow duration	2hr	4.5hr	9hr	9hr	9hr	9hr	9hr	4.5hr	9hr	9hr
EXISTING condition flow	2.09	0.39	1.86	4.00	4.08	4.19	1.64	0.96	1.32	2.06
Difference in flow	-0.08	-0.06	-0.09	-0.05	-0.04	-0.15	0.01	0.00	-0.05	-0.10
100yr level (mAHD)	17.06	43.47	40.26	26.6	23.33	17.64	19.34	45.62	35.38	19.6
Storage (m^3)	4,840	1,150	4,150	3,290	2,960	4,300	4,810	2,570	4,800	5,200
Area (including 300mm freeboard) (m^2)	5,635	3,071	4,563	3,282	3,392	4,064	5,005	5,136	5,048	4,942

4.7 GHD Storage Basins Scenario RORB Results

Previous work undertaken by GHD has proposed three storage basins at the bottom of catchments R and AH (similar locations as Water Technology's RB6 & RB7). The effect that these storage basins have on the developed flows has been analysed separately through adjusting the developed conditions scenario RORB model. This adjustment replaced the proposed Water Technology storages (RB 1-10) with the three GHD basins.

Location of the GHD retarding basins and results of the RORB Routing through the GHD Storages is shown in Figure 4-5 and Table 4-9.

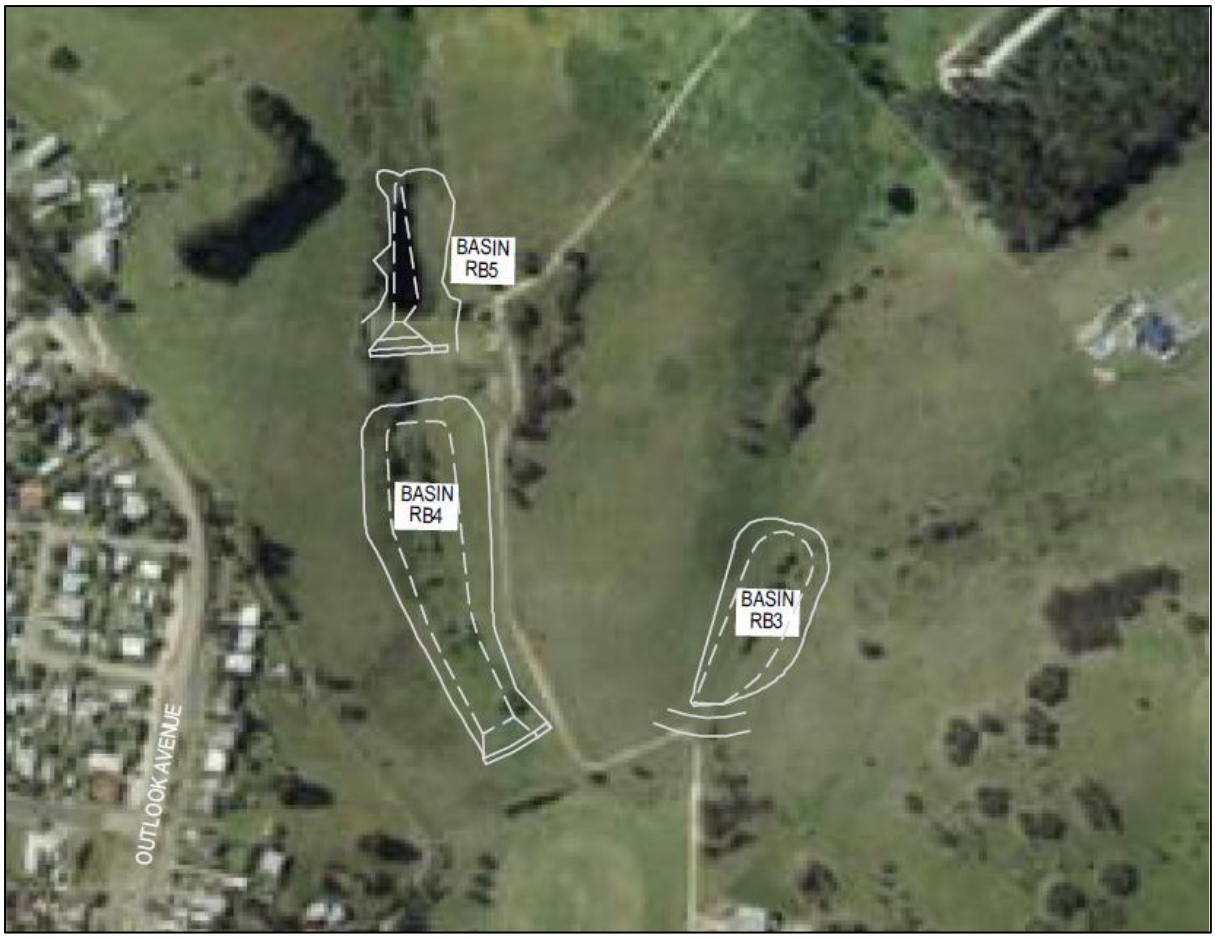


Figure 4-5 GHD Storage Basin Locations

Table 4-9 GHD Storages RORB Results

RB ID	GHD RB3	GHD RB4	GHD RB5
Spillway Height (m AHD)	18	15.6	21.5
Spillway Length (m)	60	15	13
OUTLET			
length (m)	50	50	50
slope (%)	7.4	2.3	2.5
US Invert Level (m AHD)	15.75	12.7	18.4
no of pipes	1	1	1
dia. (mm)	525	1050	1050
INFLOW (m^3/s)	2.4753	4.2887	4.7033
OUTFLOW (m^3/s)	1.1487	3.6777	4.2887
100yr level (mAHD)	17.48	14.76	20.86
storage (m^3)	6,920	19,200	7,930

4.8 Comparison of RORB Scenarios

A comparison of the three RORB scenarios at the outlets of the catchments is shown below in Table 4-10

Table 4-10 Comparison of RORB results

	DS Catchment AH				DS Catchment AQ		
	EXISTING	DEVELOPED	GHD		EXISTING	DEVELOPED	GHD
PEAK flow	5.63	5.44	4.78	PEAK flow	3.98	4.04	4.04
Duration	9hr	9hr	9hr	Duration	9hr	9hr	9hr

	DS Catchment AR				DS Catchment AS		
	EXISTING	DEVELOPED	GHD		EXISTING	DEVELOPED	GHD
PEAK flow	2.78	2.21	3.67	PEAK flow	0.68	0.91	0.91
Duration	9hr	9hr	4.5hr	Duration	2hr	25min	25min

	DS Catchment X		
	EXISTING	DEVELOPED	GHD
PEAK Flow	2.09	2.01	3.38
Duration	4.5hr	2hr	1hr

5. FLOODING ISSUES

5.1 Land Subject to Inundation

The site is not impacted by any currently defined flood ways. Given the elevated location of the NGA, regional flooding is not anticipated to be of concern to the NGA. The current FO / LSIO boundaries are depicted in Figure 5-1.

Figure 5-1 Current Flood & Land Subject to Inundation Overlays

6. WATER QUALITY ISSUES

The North Arm and Eastern Creek are the respective receiving water bodies for all sub catchments within the development. As part of the greater Gippsland Lakes system, both waterways are considered to have high environmental, amenity, cultural, stormwater and economic values to the local community.

The NGA development will need to be designed to protect the values of waterways it discharges into. The main water quality issues that will need to be addressed for the proposed development areas include:

- Likely development runoff water quality characteristics; and
- Stormwater management for construction and operational phases of the development.

6.1 Developed Conditions Pollutant Loads

A MUSIC model was utilised to determine the nutrient load off the catchment in developed conditions and sizing of WSUD features to meet Best Practice Stormwater Treatment. Best practice stormwater management requirements include:

- 80% reduction in Total Suspended Solid loads;
- 45% reduction in Total Phosphorus loads;
- 45% reduction in Total Nitrogen loads; and
- 70% reduction in Gross Pollutant loads.

As such, Water Sensitive Urban Design (WSUD) features will be required for the development to meet these objectives. Due to the size and topography of the site, features will be required across the site to treat flows from each drainage line within the development. Final selection and design of these features will need to be undertaken during future detailed design phases across separate developments.

The layout of the MUSIC model is shown in Figure 6-1, with catchment and treatment details shown in Table 6-1 and Table 6-2. WSUD features have been sized to meet best practice for the system and wetlands are to be located within the base of the retarding basins.

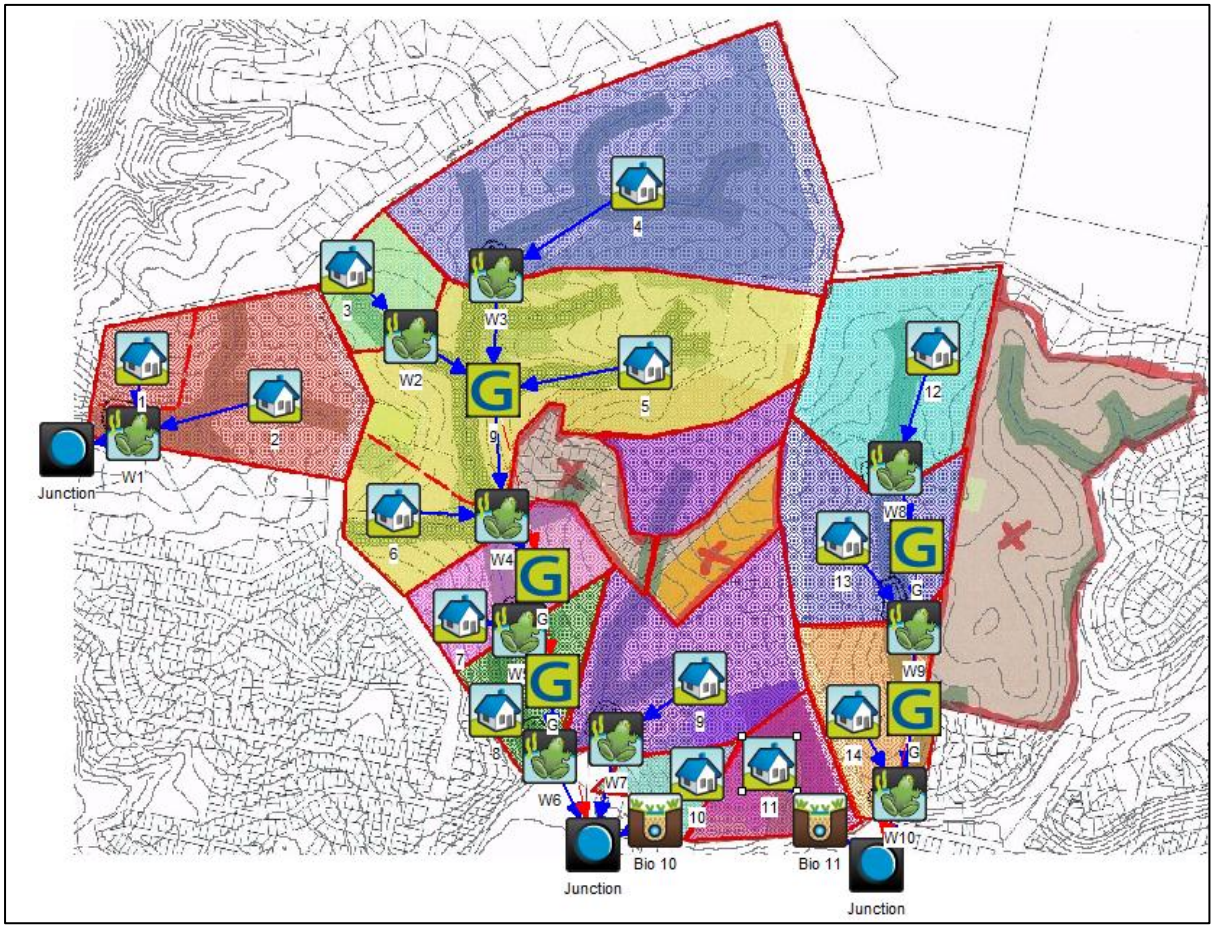


Figure 6-1 MUSIC model layout (background image SMEC Urban, 2012)

Table 6-1 Catchment details

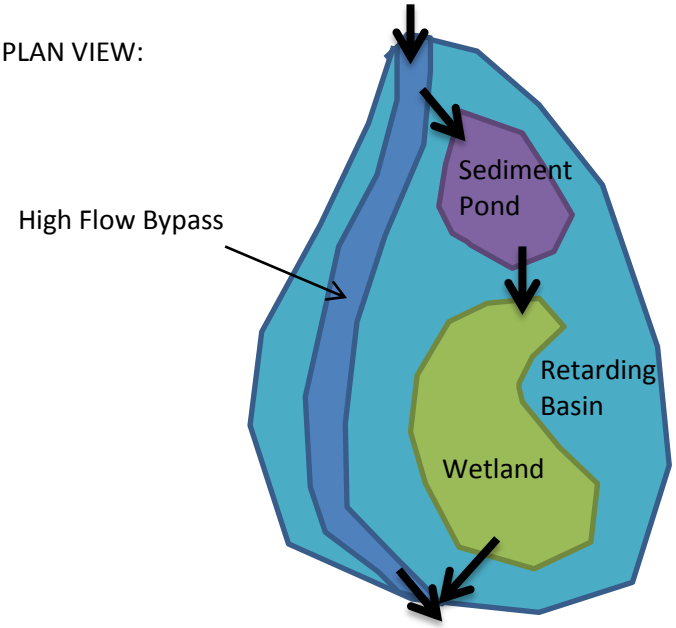
Catchment Name	Effective Impervious Area (hectares)	Fraction Impervious (%)
1	4.3	60%
2	9.1	60%
3	5.5	60%
4	19.6	48%
5	19.0	60%
6	5.2	40%
7	4.2	60%
8	5.3	60%
9	13.0	65%
10	1.0	80%
11	3.7	80%
12	9.9	55%
13	12.5	55%
14	7.0	60%

Table 6-2 WSUD details

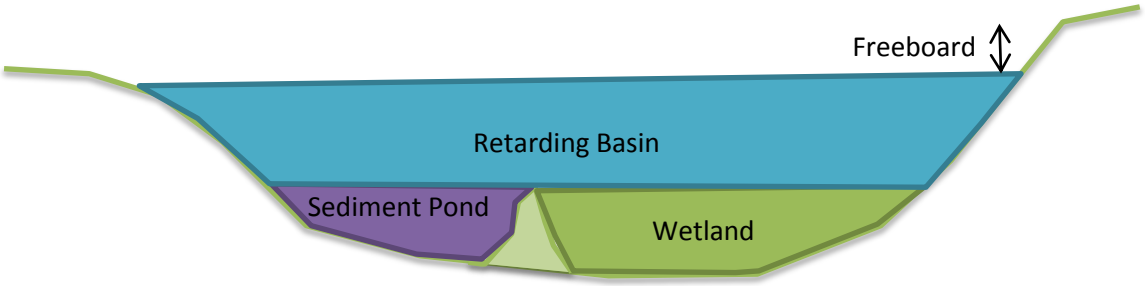
Feature Location	Sediment Pond Area (m^2)	Wetland Area (m^2)	Total Area with batter slopes / maintenance access (m^2)
W1	600	3,000	5,400
W2	500	1,500	3,000
W3	500	2,000	3,750
W4	300	1,000	1,950
W5	300	1,000	1,950
W6	400	1,200	2,400
W7	550	2,000	3,825
W8	600	3,000	5,400
W9	500	2,000	3,750
W10	500	1,500	3,000
Neighbourhood Activity Centre (Catchments 10 & 11)		500 (2* 250 bioretention)	600

For each of the wetlands the following layout was assumed:

PLAN VIEW:



CROSS SECTION VIEW:



Wetland Extended Detention Depth	0.5m
Permanent Pool	Average 0.5m
Sediment Pond Detention Depth	0.5m
Sediment Pond Pool	Average 1.0m
Detention Time	72 hours

6.2 Stormwater Reuse

Opportunities exist throughout the development to store and reuse stormwater for irrigation and/or toilet flushing purposes. This has benefits not only for water conservation, but also in terms of meeting some of the water storage and water quality improvement requirements.

Potential applications of the reused water on the site include:

- Irrigation of landscaped areas within the development;
- Toilet flushing within communal facilities; and
- Rainwater tanks on individual properties for toilet flushing and/or garden watering.

Any reuse strategy would be subject to the developer's requirements for the site and whether rainwater / storage tanks would be on a development scale or on individual lots. Compliance with any reuse strategy would need to be incorporated into a Section 173 agreement for the development.

7. FLOOD MODELLING OF RECEIVING WATERWAY – EASTERN CREEK

7.1 Overview

The hydraulic model routes the design flood hydrographs, obtained from the RORB modelling, along the drainage infrastructure and any associated overland flow path. The hydraulic model, TUFLOW, was employed in this investigation.

TUFLOW is a widely used hydraulic model that is suitable for the analysis of overland flows in urban areas. TUFLOW has three main inputs:

- Topography and drainage infrastructure data;
- Roughness; and,
- Boundary conditions.

A new TUFLOW model was constructed for this study. The TUFLOW model was used to route flows along the path of the Eastern Creek. Flow was routed along one-dimensional (1D) elements as channels. Where the capacity of the 1D elements was exceeded, the excess flows are routed overland in a two dimensional (2D) domain. The TUFLOW model outputs flood depths and velocities.

7.2 Hydraulic model construction and parameters

The TUFLOW model was constructed in MapInfo V11.5. This section details key elements and parameters of the TUFLOW model. Figure 7-1 displays the TUFLOW model configuration and elements for the existing conditions.

Model Version

- The single precision version of the latest TUFLOW release was used for all simulations (TUFLOW Version: 2012-05-AA-iDP).

2D Grid Size and Topography

- A single 2D domain was used with a grid size of 1m. The 1 meter LiDAR data provided by DSE was used to define the Digital Elevated Model (DEM) file for the 2D domain.

1d Network

- All pits, pipes, culverts and other structures were modelled in 1D using the MapInfo Tables compatible with TUFLOW.

Roughness

- For the 2D domain, 2d_mat files were produced for each land use zoning. The Manning's values are specified in the .tmf TUFLOW model file.
- For residential, commercial and industrial land, the roughness values specified are applied to the land including the buildings. Building footprints and remainder of the parcel were modelled together (with one roughness value).
- For the 1D domain, Manning's values are defined for the pipes in the 1d_nwk file and for the bridges in the 1d_nwke file.
- Proposed Manning's 'n' roughness coefficients are listed in Table 7-1 below and visible in Figure 7-2.

Table 7-1 Manning's n roughness coefficients

TUFLOW .tmf Code	Land Use	Manning's n roughness coefficient
1	Urban residential	0.035
7	Industrial/Commercial	0.300
9	Open Space or Waterway - minimal vegetation	0.040
11	Open Space or Waterway - heavy vegetation	0.090
14	Car park/pavement/wide driveways/roads	0.020

Pit Configuration

- Pits along the 1D pipe section were connected to the 2D using the "SXL" option for the 1d_nwk pit Conn_2D attribute. This option automatically lowered the 2D cells connected to the pits by 0.1-0.2m, specified in the 1d_nwk file, so that more water is forced into the pits and pipes.
- Pits were uniformly given a width of 2m to ensure more water is forced into the pits and pipes due to the steep nature catchment. This resulted in the pipe characteristics being the principal limiting factor to water entering the 1D Network.

Boundary Conditions

- Upstream inflow boundary
  - The Eastern Creek section modelled in TUFLOW has 4 major inflows located at the upstream end of the creek near the intersection between Eastern Beach Road and Princes Highway, along the creek section running north of the Lakes Main Caravan Park, near Whiters Street and along the east side of the school via an open grassed swale as observed during our inspection of the site.
  - This was modelled in TUFLOW with a 1D QT boundary connected to a 2D HX boundary for flows in excess of the 1D network capacity. Inflows were generated from the RORB modelling discussed earlier in this report.

- Outlet boundary
  - Under existing conditions, the outlet of the TUFLOW model was set as a constant water level at the Dargo Street North Arm outfall. The tail water level was set to 0.5m corresponding to the normal high tide water level. This was achieved through the use of a 'HT' boundary polyline set to this level.
  - Under GHD proposed conditions including the Eastern creek re-alignment, the creek diversion pipeline and the retention basins, the outlets of the TUFLOW model were set as constant water levels at the Dargo Street North Arm outfall and Whiters Street Cunninghame Arm outfall. The tail water level was set to 0.5m corresponding to the normal high tide water level. This was achieved through the use of a 'HT' boundary polylines set to these levels.

Figure 7-1 shows the TUFLOW model structure adopted for the Lakes Entrance Eastern Creek existing conditions TUFLOW model with Figure 7-1 showing the roughness (Manning's n) layer constructed for the model.

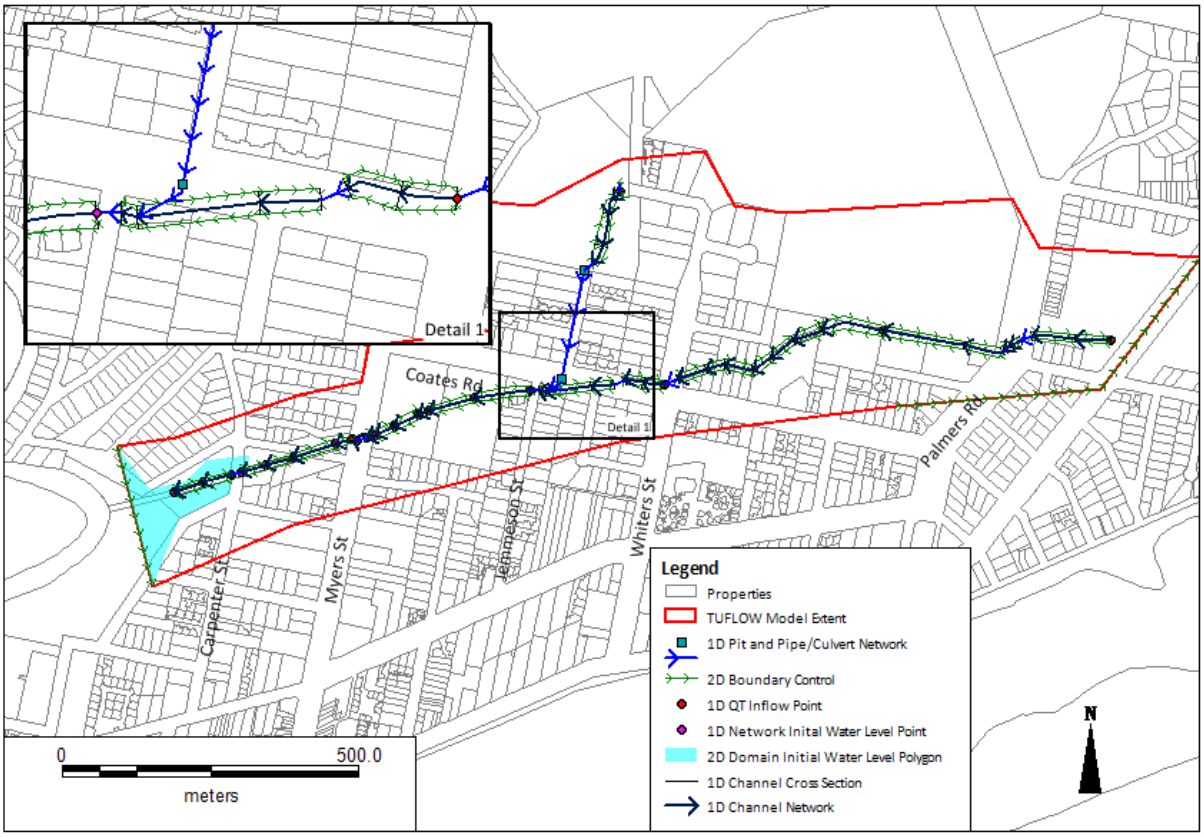


Figure 7-1 TUFLOW existing conditions model structure

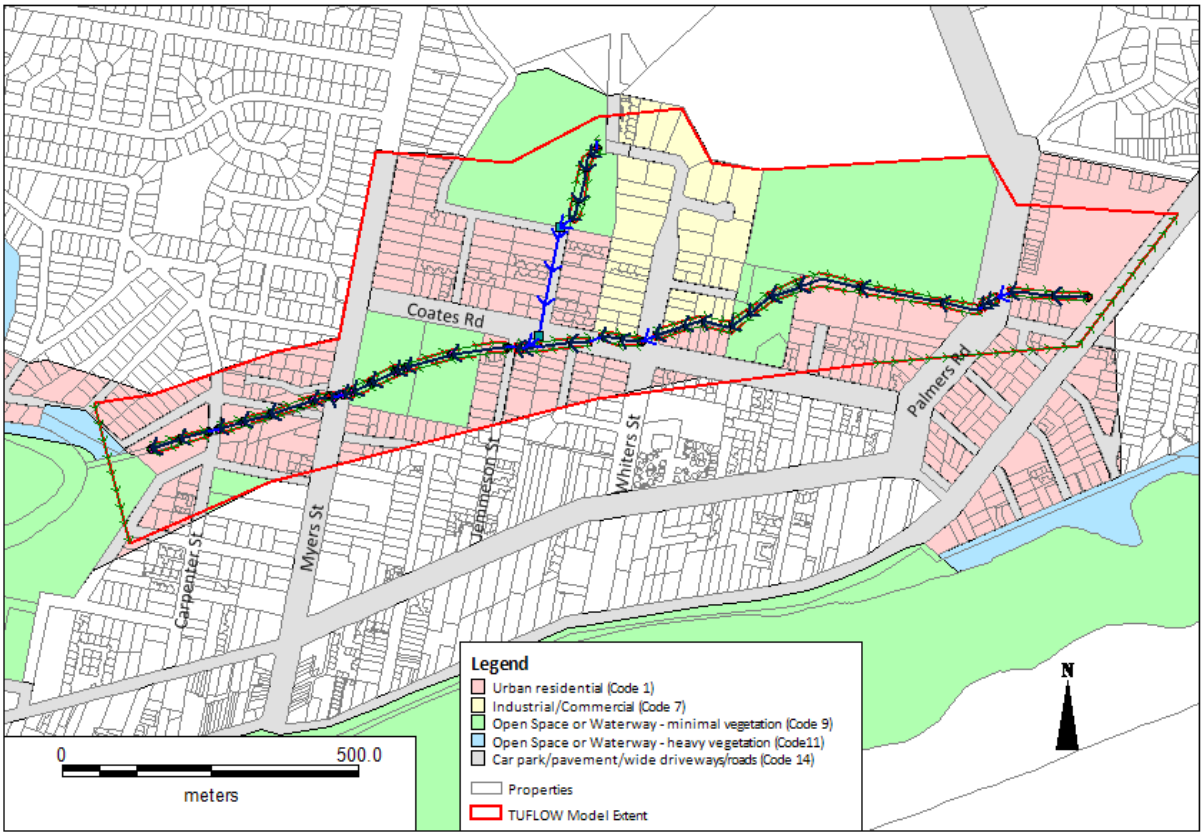


Figure 7-2 TUFLOW Manning's n value map

7.3 Hydraulic model application

The TUFLOW model was run for the critical storm duration determined by the RORB model in the Existing Conditions, GHD Proposed Conditions and Developed Conditions scenarios as shown in Table 7-2.

Table 7-2 TUFLOW scenarios

Scenarios	Description	100 year ARI Critical Storm
Existing	Creek exiting conditions Existing level of development	9 hr
Developed (NGA source mitigation work)	Creek existing conditions Retarding basins/Wetland Treatment features Developed level of development	9 hr
Developed (NGA GHD 2010 mitigation work)	Creek re-aligned (GHD) Creek diversion pipeline (GHD) Proposed Retention basins near the school (GHD) Developed level of development	9 hr

All TUFLOW model runs were controlled through a TUFLOW Event File (.tef) and a series of batch files constructed for use in this project. The use of the .tef file and batch files ensures that the base .tcf (TUFLOW Control File) does not change between runs, with all event specific parameters specified in the .tef file. This reduces the potential for error and also assists in reducing model run and processing times. The use of 'If' statement allowed us to run the three scenarios using a single TUFLOW file for each scenario type.

7.3.1 TUFLOW model checks

The following checks were undertaken on TUFLOW model parameters and outputs:

- 2D grid size: The 2D grid size is 2m, within the range of 2-3m for urban catchments
- 2D time step: The 2D time step is 0.5 seconds, ¼ of the grid size
- 1D time step: The 1D time step is equal to the 2D time step of 0.5 seconds
- Model mass errors: The Mass Errors for all models were no greater than 0.9%
- 2D Model extent: All flood extents are within the model extent.

7.3.2 TUFLOW model outputs

Figure 7-3, Figure 7-4 and Figure 7-6 respectively presents the maximum water depth plots under existing conditions, developed (NGA source mitigation work in place) conditions and developed (NGA, GHD 2010 mitigation work in place) conditions for a 100 year ARI 9 hour event.

Under existing conditions, flooding would occur in the Lakes Main Caravan Park, in properties located between Bills Street and Whiters Street and a few properties located along the Eastern Creek, for a 100 year ARI critical storm (9 hour) event.

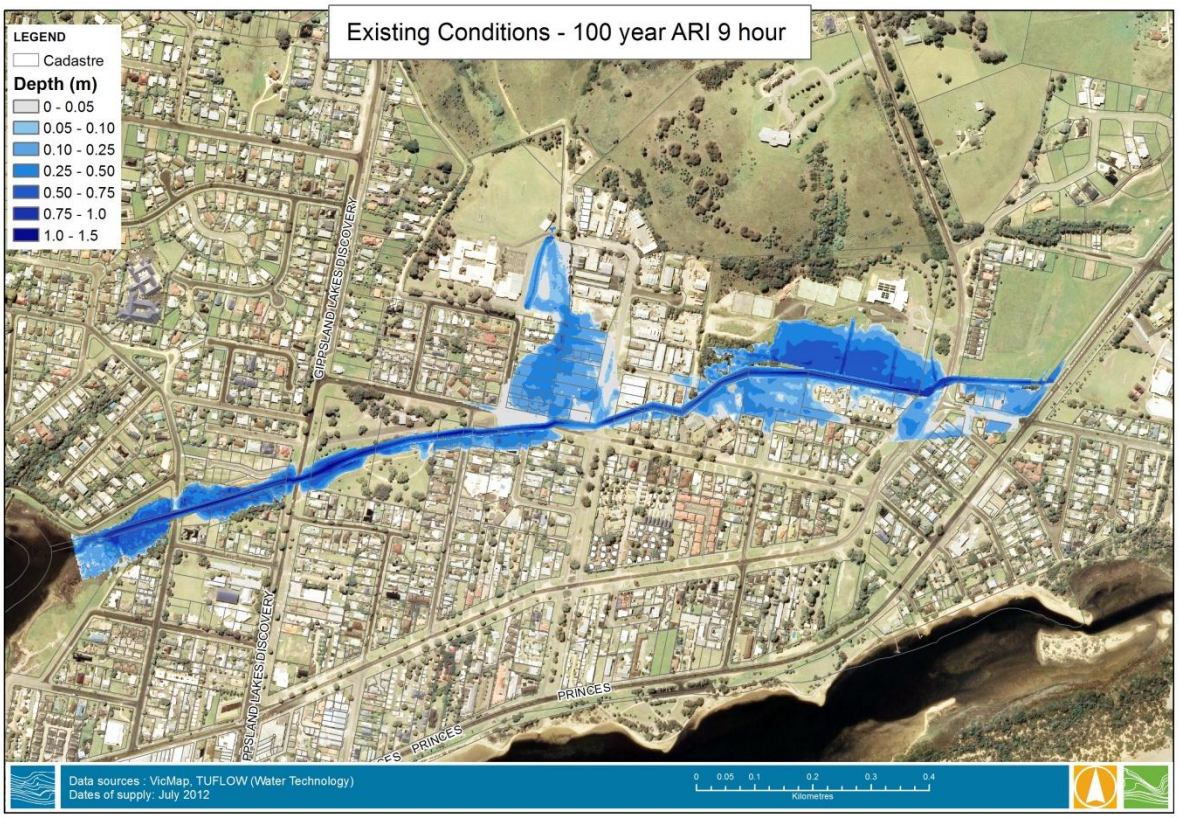


Figure 7-3 Existing Conditions maximum water depth plot

Figure 7-4 Developed Conditions (NGA source mitigation work) maximum water depth plot

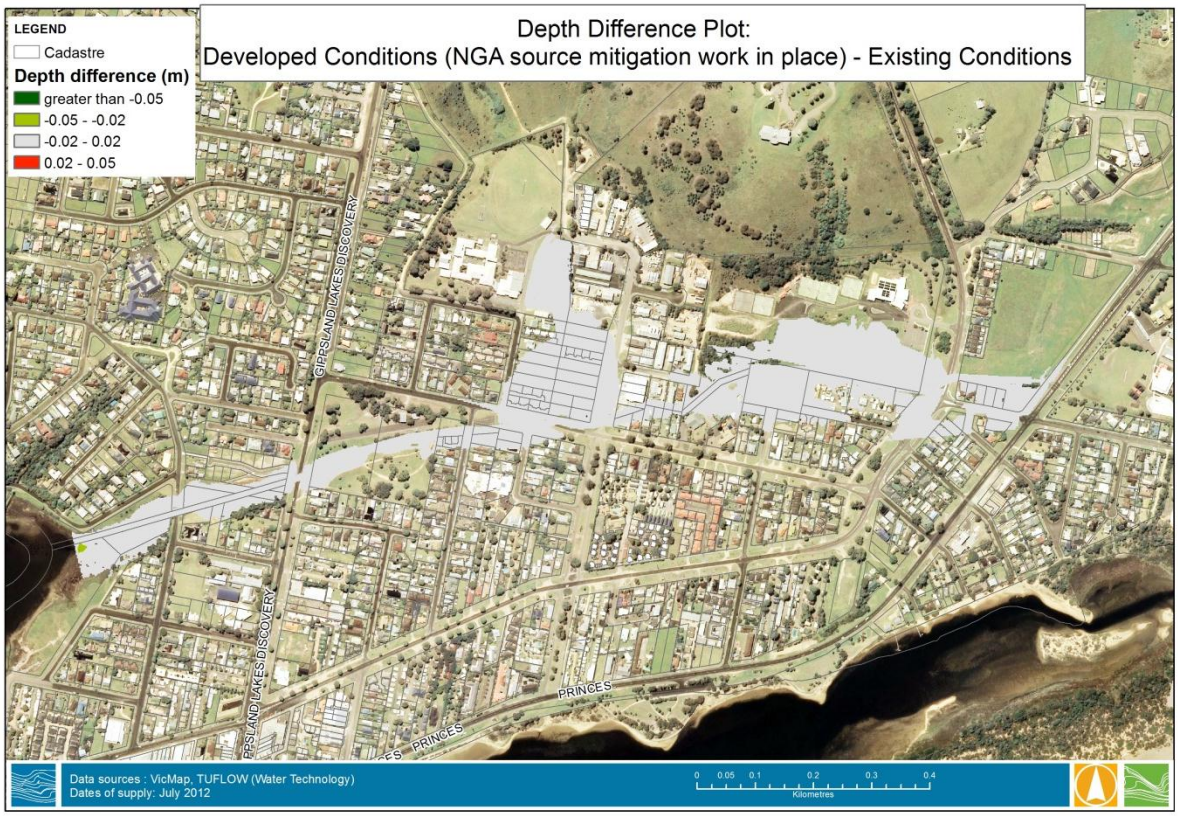


Figure 7-5 Depth difference plot (Developed Conditions NGA source mitigation work minus Existing Conditions)

The 100 year ARI flood extent under the developed (NGA source mitigation work) conditions presented in Figure 7-4 is similar to the 100 year ARI flood under existing conditions (see Figure 7-3). Therefore, the proposed mitigation works upstream using the wetland/retarding basin features would offset any flooding impacts on the existing properties located downstream as shown in the comparison of water levels in Figure 7-5 showing changes limited to between -0.02m and 0.02m.

Figure 7-6 Developed Conditions (NGA GHD 2010 mitigation work) maximum water depth plot

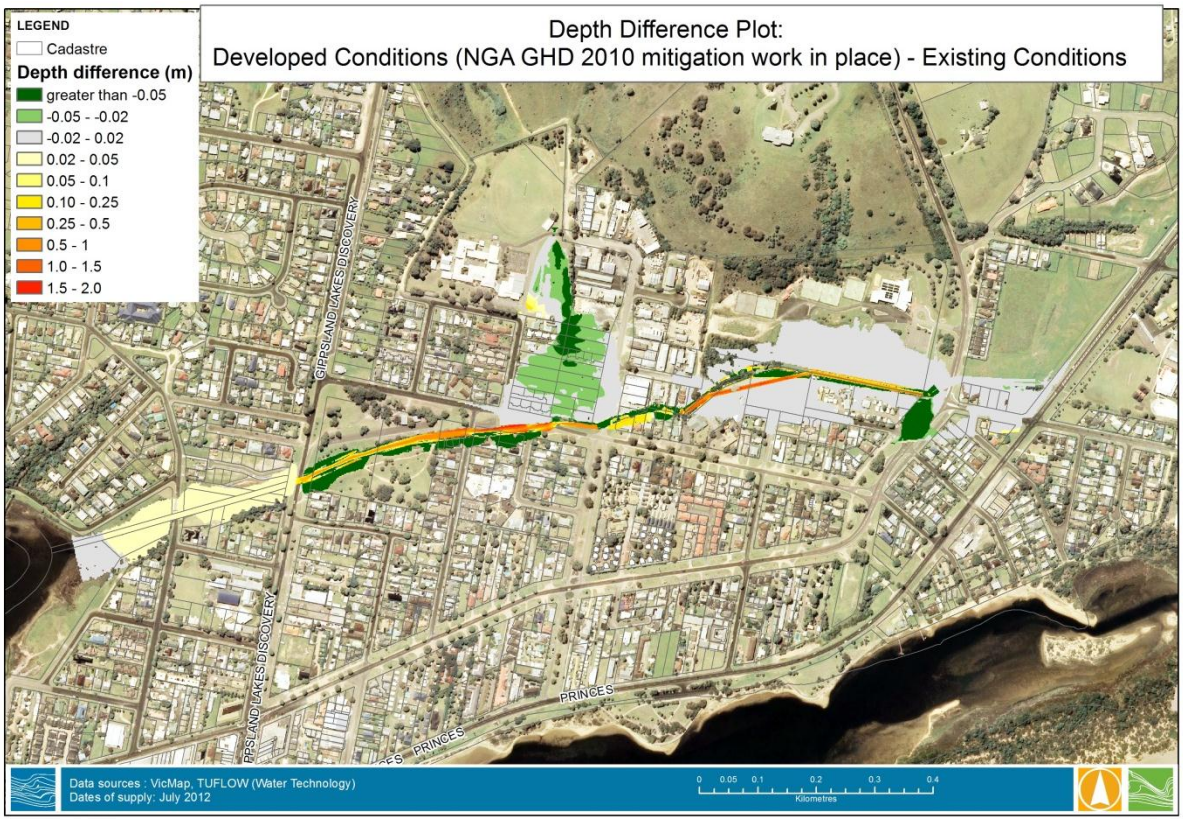


Figure 7-7 Depth difference plot (Developed Conditions NGA GHD 2010 mitigation work minus Existing Conditions)

The 100 year ARI flood extent under the developed (NGA source mitigation work) conditions presented in Figure 7-6 is also similar to the 100 year ARI flood under existing conditions (see Figure 7-3). The proposed mitigation work upstream using retention basins proposed by GHD would offset some flooding impacts in area coloured in grey area, improve the flooding in areas coloured in green and increase flooding impacts in the areas coloured in yellow and red as shown in Figure 7-5.

The above results generally show that with the NGA development in place, the works detailed for source control (i.e. proposed retarding basins 1 to 10) are effective at ensuring that the flooding downstream does not increase due to development. With the GHD works in place (retarding basins 1 – 3 and stream works on Eastern Creek) and the NGA development in place without source mitigation (i.e. proposed basins 1-10 removed), further works would still be required to ensure that flooding does not increase along Eastern Creek.

8. WORKS COSTING

The proposed wetland, bio-retention swales and retarding basin works have been costed in accordance with Melbourne Water's costing spread sheet used for Drainage Schemes. The 'South-Eastern' regional costs were used.

A number of assumptions were made in the costing of works, these include:

- No land acquisition costs are applicable;
- All outlet pipes from retarding basins are rubber ring jointed with 100% fine crushed rock backfill;
- No grassing is required in retarding basins due to the wetlands being located within each basin;
- Retarding basin excavation cost is based on the entire storage volume 0.25m above the normal water level of the wetland. This is a conservative estimate as the areas selected for basin locations aim to minimise the amount of required cut;
- 50% of cut soil (based on the entire storage volume) requires disposal off site;
- Each basin spillway has been estimated to cost $15,000 before contingencies;
- Each retarding basin outlet structure has been estimated to cost $5,000 before contingencies;
- The volume of the required basin embankment wall is estimated to be the product of the embankment length x embankment width x (0.5 x embankment height);
- $25/m^2 for filter media for bio retention swales;
- No GPT's or litter traps;
- Wetlands include pipes, vegetation and excavation; and
- No geotextile for bio-retention.

Estimated costings for retarding basins, wetlands and bioretention swales are shown in Table 8-1 through Table 8-3.

All costings provided are based on regional estimates and should be used as a guide only. Water Technology does not guarantee the absolute accuracy of costing data. Detailed costing should be completed during a functional or detailed design stage.

Table 8-1 Retarding Basin Costing

	RB1		RB2		RB3		RB4		RB5	
Construction and Compaction (m3)	4,706	$ 65,882	5,000	$ 70,000	3,529	$ 49,412	3,529	$ 49,412	3,529	$ 49,412
Topsoil (100mm) (m2)	40	$ 280	30	$ 210	30	$ 210	30	$ 210	30	$ 210
Grassing (m2)	400	$ 600	300	$ 450	300	$ 450	300	$ 450	300	$ 450
Pipes RRJ 100% FCR (#)	40m x 750mm	$ 16,280	30m x 450mm	$ 6,930	15m x 1050mm	$ 10,815	30m x 900mm	$ 17,160	30m x 1200mm	$ 27,420
Inlet/Outlet (#)	1	$ 5,000	1	$ 5,000	1	$ 5,000	1	$ 5,000	1	$ 5,000
Spillway (#)	1	$ 15,000	1	$ 15,000	1	$ 15,000	1	$ 15,000	1	$ 15,000
Excavation (m3)	4840	$ 29,040	1150	$ 6,900	4150	$ 24,900	3290	$ 19,740	2960	$ 17,760
Disposal of cut (% of excavation)	2420	$ 24,200	575	$ 5,750	2075	$ 20,750	1645	$ 16,450	1480	$ 14,800
Total Cost		**$ 156,282**		**$ 110,240**		**$ 126,537**		**$ 123,422**		**$ 130,052**
Contingency (20%)		$ 31,256.47		$ 22,048.00		$ 25,307.35		$ 24,684.35		$ 26,010.35
Total Cost including Contingency		$ 187,538.82		$ 132,288.00		$ 151,844.12		$ 148,106.12		$ 156,062.12

	RB6		RB7		RB8		RB9		RB10	
Construction and Compaction (m3)	4,706	$ 65,882	4,706	$ 65,882	4,706	$ 65,882	4,706	$ 65,882	4,706	$ 65,882
Topsoil (100mm) (m2)	40	$ 280	40	$ 280	40	$ 280	40	$ 280	40	$ 280
Grassing (m2)	400	$ 600	400	$ 600	400	$ 600	400	$ 600	400	$ 600
Pipes RRJ 100% FCR (#)	40m x 750mm	$ 16,280	20m x 900mm	$ 11,440	60m x 525mm	$ 16,020	20m x 750mm	$ -	20m x 900mm	$ 11,440
Inlet/Outlet (#)	1	$ 5,000	1	$ 5,000	1	$ 5,000	1	$ 8,140	1	$ 5,000
Spillway (#)	1	$ 15,000	1	$ 15,000	1	$ 15,000	1	$ 5,000	1	$ 15,000
Excavation (m3)	4300	$ 25,800	4810	$ 28,860	2570	$ 15,420	4800	$ 15,000	5200	$ 31,200
Disposal of cut (% of excavation)	2150	$ 21,500	2405	$ 24,050	1285	$ 12,850	2400	$ 28,800	2600	$ 26,000
Total Cost		**$ 150,342**		**$ 151,112**		**$ 131,052**		**$ 123,702**		**$ 155,402**
Contingency (20%)		$ 30,068.47		$ 30,222.47		$ 26,210.47		$ 24,740.47		$ 31,080.47
Total Cost including Contingency		$ 180,410.82		$ 181,334.82		$ 157,262.82		$ 148,442.82		$ 186,482.82

Table 8-2 Wetland Costing

	Wetland 1	Wetland 2	Wetland 3	Wetland 4	Wetland 5	Wetland 6	Wetland 7	Wetland 8	Wetland 9	Wetland 10
Wetland Cost	**$364,560**	**$314,417**	**$287,197**	**$241,135**	**$241,135**	**$262,333**	**$316,954**	**$364,560**	**$314,417**	**$287,197**
Contingency (20%)	$72,912	$62,883	$57,439	$48,227	$48,227	$52,467	$63,391	$72,912	$62,883	$57,439
Total Cost including Contingency	$437,472	$377,300	$344,636	$289,362	$289,362	$314,800	$380,345	$437,472	$377,300	$344,636

Table 8-3 Bio retention costing

	Bioretention 1	Bioretention 2
Bioretention Cost	**$15,000**	**$15,000**
Contingency (20%)	$3,000	$3,000
Total Cost including Contingency	$18,000	$18,000

8.1 Cost Apportionment per Catchment

Estimated construction costs have also been divided into catchment areas as shown in Table 8-4. With reference to Figure 6-1 and Figure 8-1 for context, the following works are proposed in each catchment.

The total cost of works is the total sum of the works required within each catchment including contingencies as described above. The cost per effective impervious hectare is the total cost of works for that catchment divided by the total effective impervious area in that catchment. It is important to note that Catchments 1 and 2 share the costs of Wetland and Retarding Basin 1 and Catchments 5 and 6 share the costs of Wetland and Retarding Basin 4 hence the cost per effective impervious hectare on these catchments are calculated by the total cost of works divided by the sum of the effective impervious area for the sharing catchments. The final cost per catchment is the cost per effective hectare multiplied by the effective impervious area in the catchment.

Table 8-4 Works costing per catchment

Catchment Name	Effective Impervious Area (hectares)	Required Works	Total Cost of works (including contingency)	Cost per Effective Impervious hectare (including contingency)	Cost per Catchment (including contingency)
1	4.3	W1, RB1	$ 625,011	$ 46,643	$ 200,563
2	9.1			$ 46,643	$ 424,448
3	5.5	W2, RB2	$ 509,588	$ 92,652	$ 509,588
4	19.6	W3, RB3	$ 496,481	$ 25,331	$ 496,481
5	19	W4, RB4	$ 437,468	$ 18,077	$ 343,467
6	5.2			$ 18,077	$ 94,001
7	4.2	W5, RB5	$ 445,424	$ 106,053	$ 445,424
8	5.3	W6, RB6	$ 495,210	$ 93,436	$ 495,210
9	13	W7, RB7	$ 561,680	$ 43,206	$ 561,680
10	1	Bio1	$ 18,000	$ 18,000	$ 18,000
11	3.7	Bio2	$ 18,000	$ 4,865	$ 18,000
12	9.9	W8, RB8	$ 594,735	$ 60,074	$ 594,735
13	12.5	W9, RB9	$ 525,743	$ 42,059	$ 525,743
14	7	W10, RB10	$ 531,119	$ 75,874	$ 531,119

8.2 Land Required for Works

Urban Enterprise have requested the area (or land take) required for each basin and bio retention swale. The approximate surface area of each basin at the top water level plus a 300mm freeboard is given in Table 8-5. Each bio retention swale requires a surface area of 350m^2.

Table 8-5 Land take for Retarding Basins

RB Number	Area (including freeboard) (m^2)
1	5,650
2	3,100
3	4,600
4	3,300
5	3,400
6	4,100
7	5,000
8	5,150
9	5,050
10	4,950

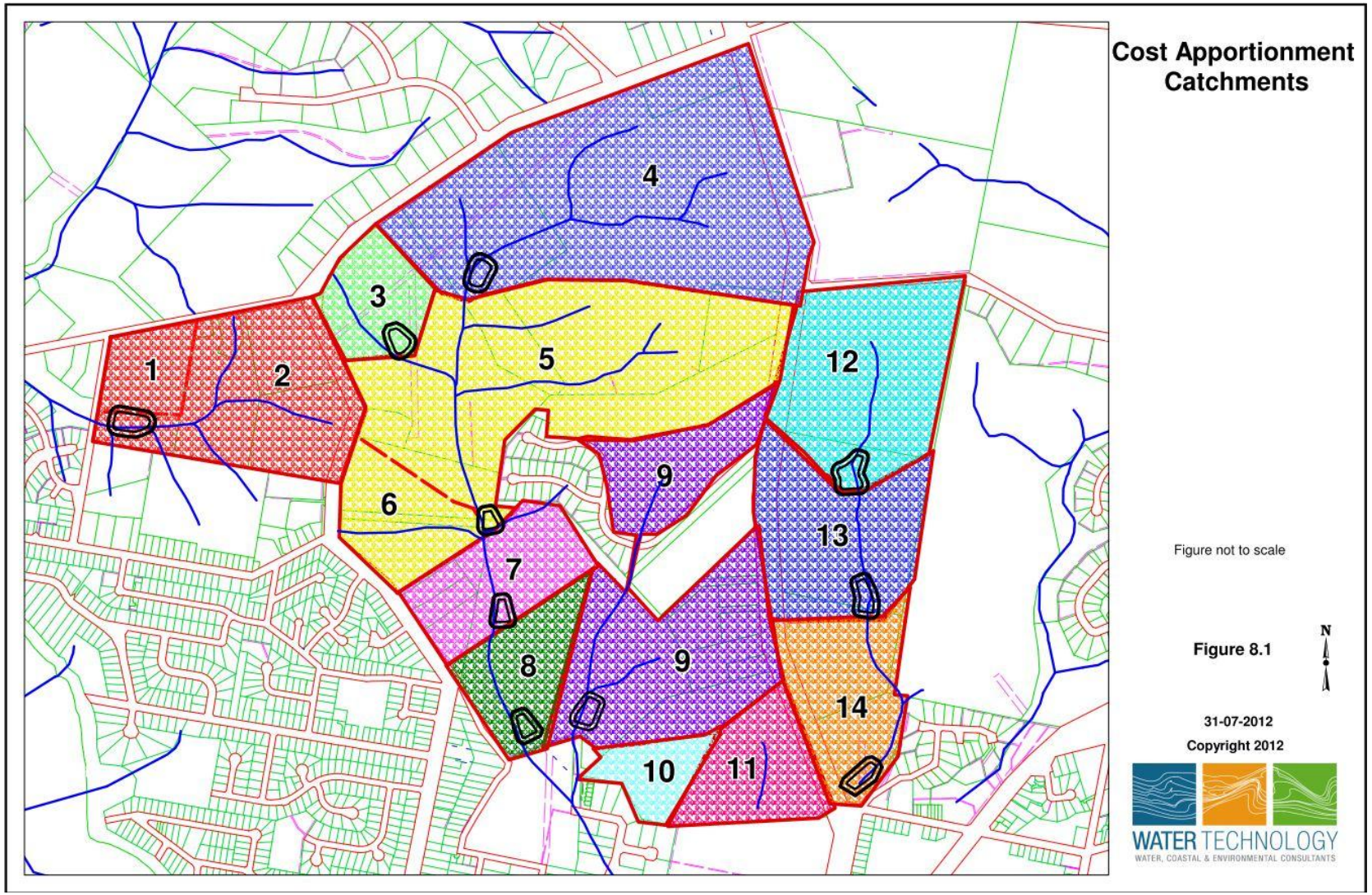


Figure 8-1 Cost apportionment – catchment boundaries

9. FURTHER ASSESSMENTS REQUIRED

Based on this investigation Water Technology advises that a number of further studies will ultimately be required to meet EGSC and EGCMA requirements. This is likely to include detailed flow path modelling (pre development & post development) at the detailed subdivision design scale, detailed surface water drainage design (including final engineering basin sizing / culvert sizing), a Waterway Management Plan and a Stormwater management plan. These studies should address all of the points addressed in this study. Indicative costs provided in this report should be viewed as a guide only. The following hydrological / water quality / stormwater management related components are likely to be required:

9.1 Additional Site Survey

It is considered that the currently available site survey will be sufficiently accurate to undertake the majority of any additional hydrological work required for individual developments. LiDAR information may be available for some sections (generally the lower elevation areas) and this data should be sourced and utilised where possible. Additional detailed survey is likely to be required at all proposed basin locations and existing farm dams to determine the detailed storage characteristics, overflow configurations and general hydraulic behaviour of these features.

It is expected that individual sites will need to be feature surveyed for development plans, and this work could be tied in with the above requirements. Individual site costs will vary but an additional 2-4 days of field work would not be un-reasonable ($4,000 – $8,000 per site).

9.2 Surface Water Management Strategy (~$7,500 - $15,000 per site)

Given the designated waterways present in all sites, a Surface Water Management Strategy (SWMS) will be required for individual development areas that detail stormwater quantity and quality management features for the development. This study should refer to a detailed concept development plan that provides an accurate assessment of lot, POS and road layouts. Individual sub-catchment storage and WSUD features will be detailed and benchmarked with EGSC and EGCMA requirements. Key elements of this study would include:

9.2.1 Hydrologic Assessment

A detailed hydrologic analysis of the site flows is required. The RORB model presented in this report should be used as the primary catchment modelling tool. Depending on detailed design requirements, adjustment or refinement of the RORB sub catchment properties presented in this report may be appropriate.

9.2.2 Water Quality

An assessment of storm water management requirements including design of appropriate water quality treatment measures will need to be completed. This is to ensure Best Practice in meeting any receiving water requirements. This report has presented suitable conceptual treatment components, with detailed design to occur at a later stage in liaison with civil designers.

In general, individual SWMS for separate developments will involve the following:

- Review of the surface water management issues for the site as a whole which may impact on function and sustainability of water bodies and drainage paths including consideration of Water Sensitive Urban Design (WSUD) principles in the proposed development;
- Design / confirmation of proposed stormwater water management system options and assessment of their suitability and effectiveness using Best Practice Management tools (stormwater treatment modelling tool MUSIC). Depending on the final suite of WSUD assets employed, sub versions of the MUSIC model developed for this report may be appropriate to provide additional detail within specific developments;
- Preparation of a report detailing the final design and analysis of the WSUD components of the development.

Based on our work conducted for this investigation, we believe there is ample scope within the development to achieve an attractive and functional stormwater system, with multiple water quality and aesthetic values associated with the designs.

9.3 Reuse Strategy (~$5,500 per site)

If reuse of stormwater is considered a viable and attractive option for the development, Water Technology could develop a reuse strategy which encompasses the following:

- Demand analysis for various potential users in the development;
- Water storage design – whether this is within reservoirs, underground storages or rainwater tanks;
- Consideration of any water quality requirements associated with reuse; and
- Water balance analysis.

9.4 Consultation with relevant authorities (~$3,500 per site)

As the EGSC and EGCMA are important referral authorities for the development, additional meetings will be required to discuss drainage, flooding and water quality related issues.

9.5 Waterway Management Plan (~$3,500 – $5,500 per site)

Given the designated waterways present in all sites, a Waterway Management Plan (WMP) will be required for individual development areas. The WMP will detail the design and implementation of best practice waterway management options and maintenance requirements to ensure the short and long term health and function of the designated waterways.

APPENDIX A SITE VISIT PHOTOS