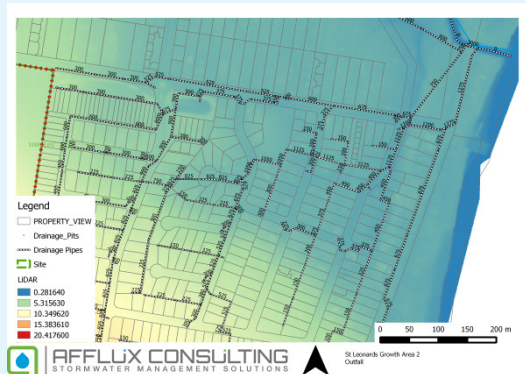




Growth Area 2, St Leonards Stormwater Management Plan



September 2014



AFFLUX CONSULTING
STORMWATER MANAGEMENT SOLUTIONS

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Project Name	Growth Area 2, Leviens Rd
Client	St Quintans
Reference Number	J138
Report Version	V02
Document Name	GA2 St Leonards
Author	CMB

1. Introduction

This report forms a stormwater management strategy for the development of a parcel of land known as Growth Area 2 and abutting Levens Rd, Ibbotson St and Bluff Rd, St Leonards. The site is approximately 70 hectares and is undergoing rezoning to residential land. The site can be considered as an in-fill development, with existing residential subdivisions to the North, South and East of the development.

This report will cover the major drainage, flooding and water quality associated with the development. The intention of this report is to meet and in some cases exceed the standard requirements of:

- Retention of post development flows to pre-development levels
- Meet the EPA best practice water quality requirements as defined under Clause 56.07-4 of the Geelong Planning Scheme
- Ensure flooding of the site, or potential off-site impacts are reduced or eliminated

To meet these requirements a range of hydrological, hydraulic and water quality modelling has been undertaken.

Information Sources

A number of information sources have been used in the formation of this strategy, these include:

- Site inspection undertaken Thursday 11th August
- DEPI LiDAR information October 2008
- DEPI planning scheme and cadastral information as accessed online July 2014
- Discussions with City of Greater Geelong
- Discussions and information as provided by Corangamite Catchment Management Authority

2.Existing Catchment

The proposed development, relative LiDAR levels, and Existing pipe networks are shown in Figure 1 below.

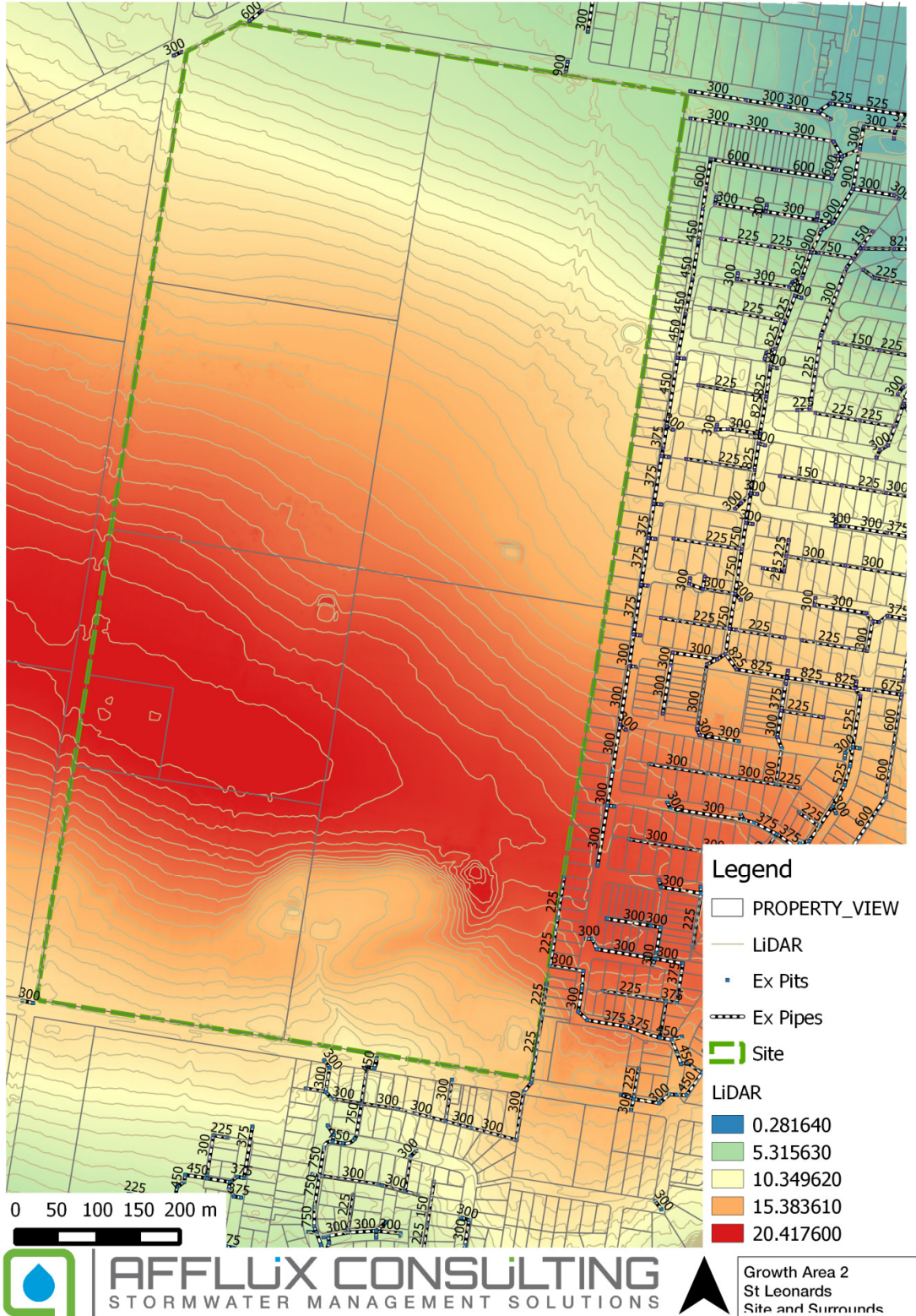


Figure 1 Site layout and terrain

Existing Catchments

The existing site is covered by grasslands with a number of access tracks, defined drainage paths and stormwater capture systems. Existing subdivisions exist to the north, east and south of the property with a range of property sizes. In general the northern interfaces are large, acre (4000m²) or greater properties. The southern and eastern interface houses range from 600m² to larger 1000m² blocks.

The site generally sits at the top of the catchment, with a majority of the site draining towards the north. Two catchments drain to the south, with the smaller portion draining to the south west. Catchment delineation can be seen in Figure 2 with the development site and aerial photography depicting the low imperviousness of the surrounds.

A site inspection was undertaken to assess the current drainage paths and site coverage. It was clear that the existing catchment has been modified with a number of cut-off drains to direct water away from the existing subdivisions and along the southern boundary to two main discharge points. These are described further in the section below.



Figure 2 Existing Catchments

Major Existing Discharge locations

Three main concentrated discharge locations currently exist for the site. A main northern discharge through the subdivision and along Leviens Road to the north east; a main southern discharge under Bluff Road and through the subdivision to the south; and a small catchment to the south west that discharges to existing table drains along Bluff road. These locations were checked for capacity and elevation for both existing and developed conditions. It is noted that the St Leonards Growth Area structure plan highlights the potential for the areas downstream of the site to flood. It is therefore inherent in any drainage solution for the site to meet an obligation to minimise any potential for additional downstream flooding.

Northern Discharge

The majority of the site drains to the northern boundary along Leviens Road, before running east towards Port Phillip Bay. No direct outfalls to the Bay exist within St Leonards with all drainage either discharging directly to St Leonards Lake, or as is the case with this area, into the Lake outfall channel. Two 300mm pipes service the site currently, however no direction connection is made with either. The northern 300mm pipe is part of the existing road drainage system, and runs into a main line of increasing pipe diameters before eventually discharging into the Lake outfall with a 750mm pipe. The second 300mm outfall currently services the apartments along Leviens drive within the Seachange estate. This line discharges into a small retarding basin, before entering the main outfall as described above. This system is depicted in Figure 3 and it is noted that the major overland flow path would be directly down Leviens Drive (Figure 4).

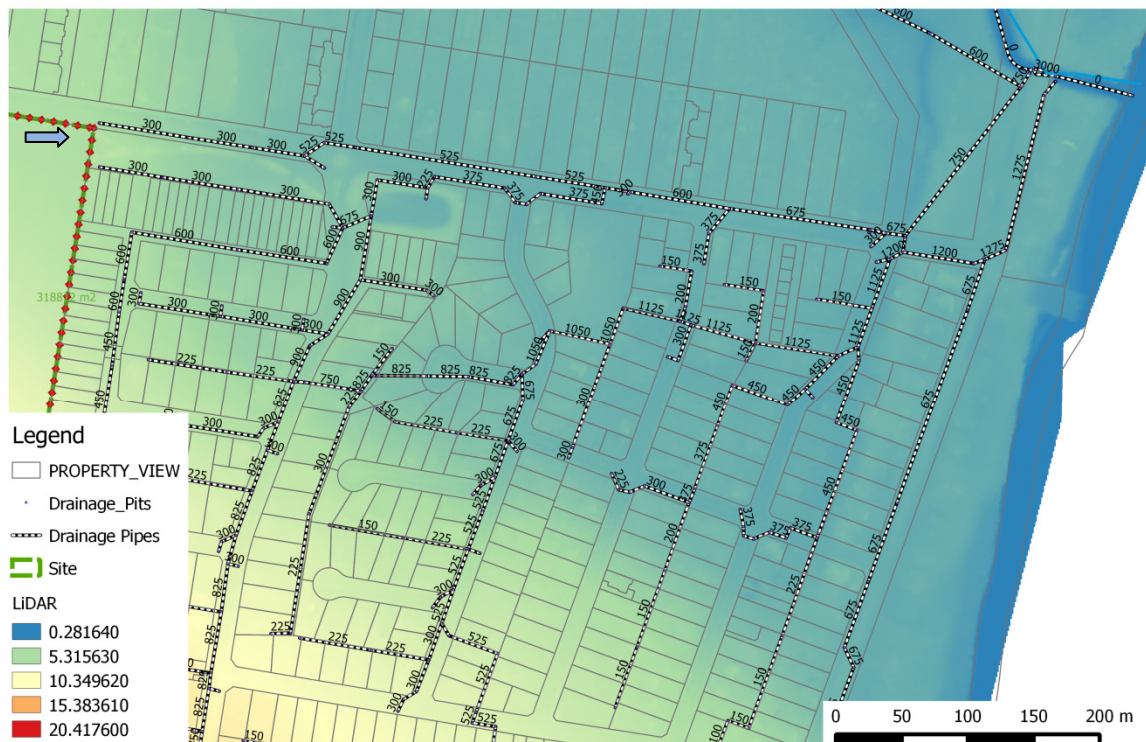


Figure 3 Northern Outfall locations and drainage system



Figure 4 Leviens Drive – Discharge Overland flow path (Looking east from site)

South Eastern Discharge Location

Approximately 20% of the site drains south of the site under Bluff road and through the existing subdivision to the south. A significant depression exists within this southern catchment, with all surface waters draining towards this depression. An existing set of culverts are located under Bluff road with two 450mm pipes discharging into the subdivision to the south. A depiction of this crossing point can be seen in Figure 5. It is believed that these existing culverts currently cater for the majority of the 100y ARI storm event. However, the overland flow path downstream of the site is unclear.



Figure 5 Southern Discharge location – Upstream and Downstream of Bluff Rd

South Western Discharge Location

A small catchment currently discharges to the south west corner of the site and flows across into the existing table drains. This drainage system is reasonably informal, and infrequently maintained. There is however outfall for this system approximately 500 meters west of the site with a formal crossing of Bluff Road via 2 x 450 x 150 culverts and a swale system down to Swan Bay. A depiction of this site outfall and location can be seen in Figure 6.

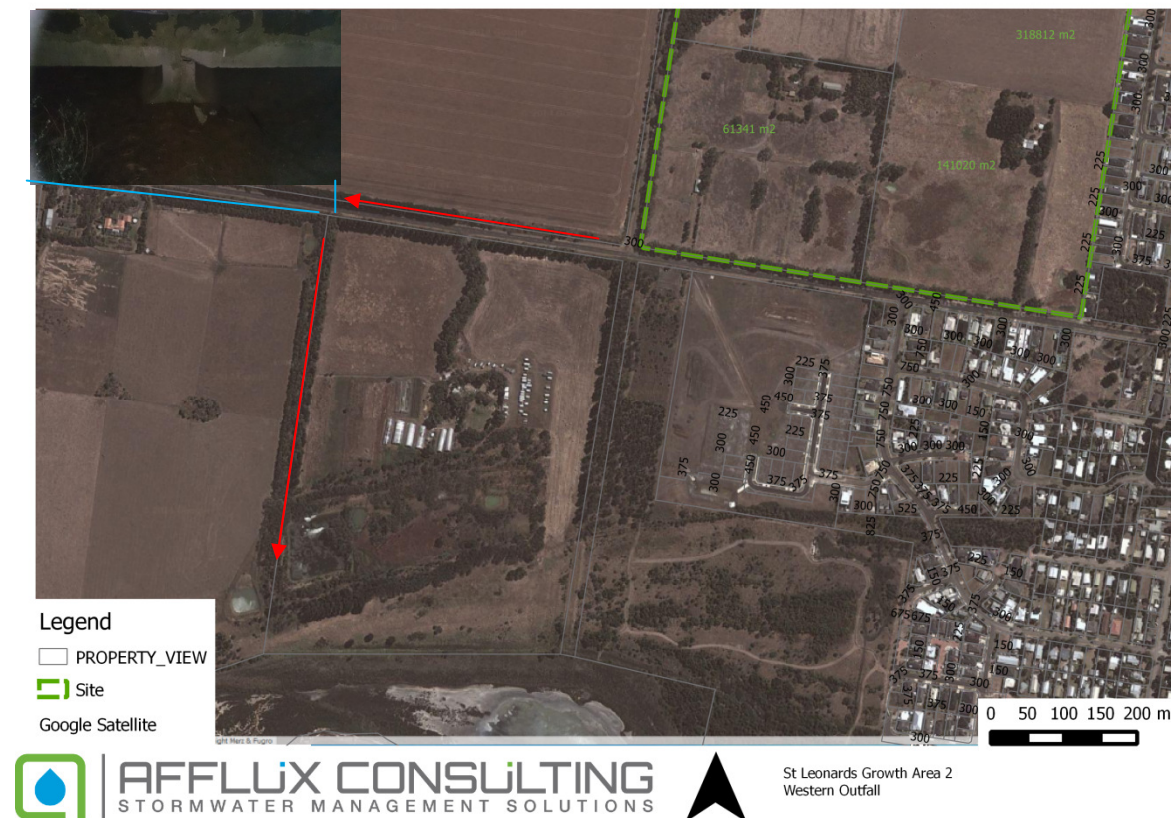


Figure 6 Western Outfall

3. Catchment Design Objectives

All development has the potential to adversely affect downstream environments through the effects of stormwater runoff. Increased impervious areas resulting in increased volumetric and peak flows have been extensively researched and linked to downstream environmental degradation. Contaminants contained in the runoff have also been linked with adverse changes to both water quality and stream ecology. Finally the contribution of increased runoff can be linked to downstream flooding and capacity constraints.

To combat these affects a range hydrological and water quality mitigation measures have been researched, and legislated in Victorian planning schemes. The design objectives for this catchment are considered below.

General Considerations

The Victorian State Planning Policy Framework includes provisions incorporating the provisions for stormwater management in its integrated water management clauses. The City of Greater Geelong Council has incorporated Clause 56.07-4 into its planning scheme setting out objectives for stormwater runoff. This site must meet these objectives as defined below.

Water Quality Requirements

Current water quality requirements as listed by the Victorian EPA Best Practice Environmental Management (BPEM) Guidelines are:

- 80% Total Suspended Solids (TSS) reduction
- 45% Total Nitrogen reduction
- 45% Total Phosphorus reduction
- 70% Gross Pollutant capture

Flood Storage Requirements

The site shall be designed to ensure that flows are not to increase above the pre-development levels. This will be applied to the 100 year Average Recurrence Interval (ARI) storm and checked at each of the site discharge points. The St Leonards Structure plan also specifically refers to the requirement of a Flood Impact Plan as part of the Stormwater strategy, detailing how changes in hydrology may affect the existing residential areas.

Flood Protection Requirements

All lots within the development will be provided at least 150mm freeboard above any predicted 100 year ARI flood level (with floors a further 150mm higher). All retarding basins will be designed to be cut into the natural surface avoiding any potential dam wall construction issues.

Ecological Objectives

Given the short distance to the Port Phillip Bay outfall, and largely piped system connection, no specific Ecological objectives can be outlined for the area other than the reduction of nutrients to the bay. As nitrogen is the limiting nutrient for saltwater systems this is of particular relevance.

Specific Challenges

A number of catchment specific challenges have presented themselves in the review of the site. These include:

- Lack of defined outfall for the majority of the site – whilst two 300mm outfalls exist for the northern discharge, these are significantly undersized for the existing rural catchment, yet alone an urbanised catchment. This provides a significant challenge for the site and will be explored in detail later in this report
- Minimising hydrological effects on downstream environments – associated with the lack of outfall's for the site is the acknowledgement that the downstream system is constrained, and any additional water may lead to localised flooding of the existing system.
- The southern discharge point should be limited to a 100 year ARI level that does not overtop Bluff road, and limits the possibility of overland flow to the south of the site

4. Hydrological Investigations

To evaluate the hydrology of the proposed development a number of hydrological models have been formed and compared. This section explores these models and the flows from the proposed development.

RORB Model

The primary model for flow evaluations for the site is Monash Universities RORB model. RORB was produced by Laurenson and Mein as a runoff routing model for the production of flood hydrographs. It is considered the industry standard model for Victorian Flood studies.

The catchment was broken into a number of sub-catchments and reaches as can be seen in Figure 7. Catchment areas, and reach lengths are displayed to give the reader a sense of relative for the site. At least 3 sub-catchments occur before all reporting locations with all sub-catchments within individual reaches approximately the same area. The three major discharge locations as described above have been specifically modelled.

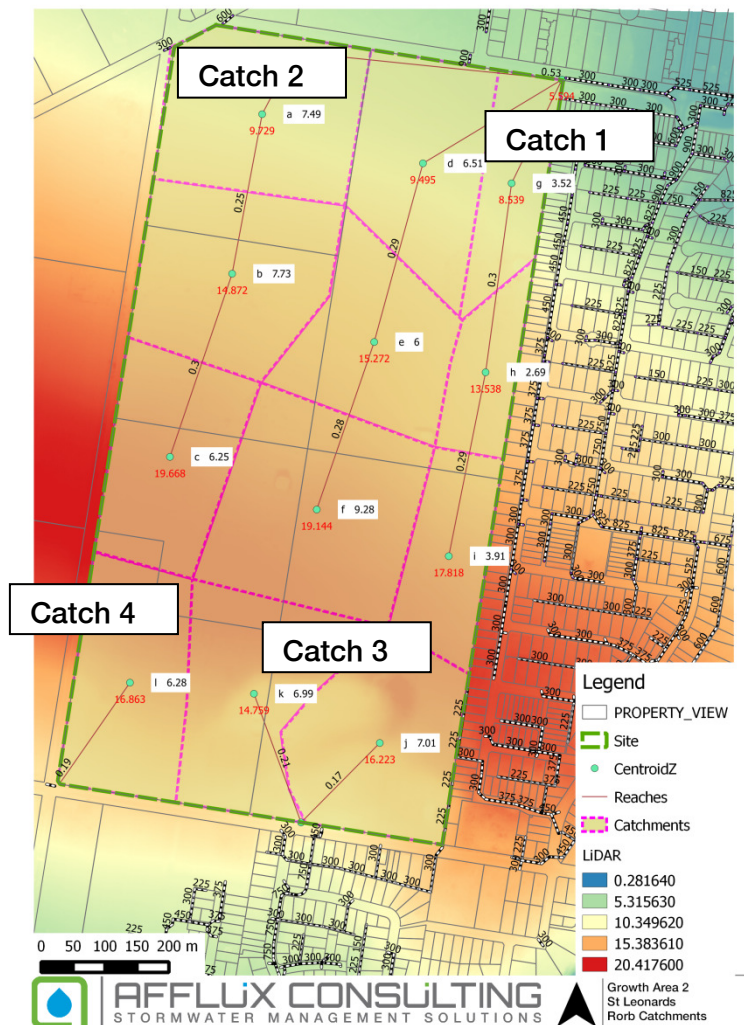


Figure 7 Rorb model catchments and reaches

Each of the sub-catchments were then given a Fraction Impervious Value for both existing and developed conditions. The Developed conditions FI's were based on an estimated lot size of between 600-800m². Lot size FI values were based on the Melbourne Water recommended FI values. Reach lengths, and slopes were calculated based on the existing terrain, with developed conditions reach's converted to concrete lined channels. All existing channels were modelled as unlined channels as per Melbourne Water guidelines.

These parameters have been summarised in Table 1 below.

Table 1 Rorb Model Parameters

Catchment	Area	FI (ex)	FI (Dev)	Centroid z	Reach	length	slope	Reach Type (ex)	Reach Type (dev)
c	0.062	0.1	0.6	19.668	1cb	0.3	1.6%	Unlined	lined
b	0.077	0.1	0.6	14.872	1ba	0.25	2.1%	Unlined	lined
a	0.075	0.1	0.6	9.729	1ao	0.53	0.8%	Unlined	lined
o				5.594					
f	0.093	0.1	0.6	19.144	2fe	0.28	1.4%	Unlined	lined
e	0.06	0.1	0.6	15.272	2ed	0.29	2.0%	Unlined	lined
d	0.065	0.1	0.6	9.495	2do	0.25	1.6%	Unlined	lined
o				5.594					
i	0.039	0.1	0.6	17.818	2ih	0.29	1.5%	Unlined	lined
h	0.027	0.1	0.6	13.538	2hg	0.3	1.7%	Unlined	lined
g	0.035	0.1	0.6	8.539	g2o	0.18	1.6%	Unlined	lined
o				5.594					
l	0.063	0.1	0.6	16.863	4lq	0.19	2.2%	Unlined	lined
q				12.639					
k	0.07	0.1	0.6	14.759	3kp	0.21	1.8%	Unlined	lined
p				10.998					
j	0.07	0.1	0.6	16.223	4jp	0.17	3.1%	Unlined	lined
p				10.998					

Model Reconciliation

The flow routing in RORB is based on the following equation relating storage (S) and discharge (Q) of a watercourse.

$$S = kQ^m \text{ where } k = KcKr$$

The values Kc and m are parameters that can be obtained by calibration of the model using corresponding sets of data on rainfall for selected historical flows. If historical flows are unknown, values can be estimated from regional analysis or by values suggested by Australian Rainfall and Runoff (AR&R). In this case, flow gauging information was not available.

As such the Rorb model was calibrated to a number of other flow derivation methods including the Rational Method, Regional regression curves and Regional Kc formulations. These are detailed below:

Rational Method Formulation (Adams method)

Table 2 Rational Method Calibration Flows

ARI (years)	Q (m3/s)	I (mm/hr)	tc	Fy	C'10	C10	Cy	Total Area (ha)
1	0.526527	16.88161	40.5862	0.8	0.111884	0.190696	0.152557	73.6
2	0.731218	22.06537	40.5862	0.85	0.111884	0.190696	0.162092	73.6
5	1.064839	28.75037	40.5862	0.95	0.111884	0.190696	0.181161	73.6
10	1.293061	33.16669	40.5862	1	0.111884	0.190696	0.190696	73.6
20	1.603162	39.16258	40.5862	1.05	0.111884	0.190696	0.200231	73.6
50	2.136362	47.64969	40.5862	1.15	0.111884	0.190696	0.2193	73.6
100	2.553479	54.58009	40.5862	1.2	0.111884	0.190696	0.228835	73.6

Equivalent Kc = 2.2

DSE Regional Regression Method

Rural Catchments $4.67A^{0.763}$

Flow output = 3.69

Kc = 1.3

Based on the above testing the more conservative estimate of a Kc of 2.2 was adopted for the model. This resulted in the model outlet flows as seen in Figure 8.

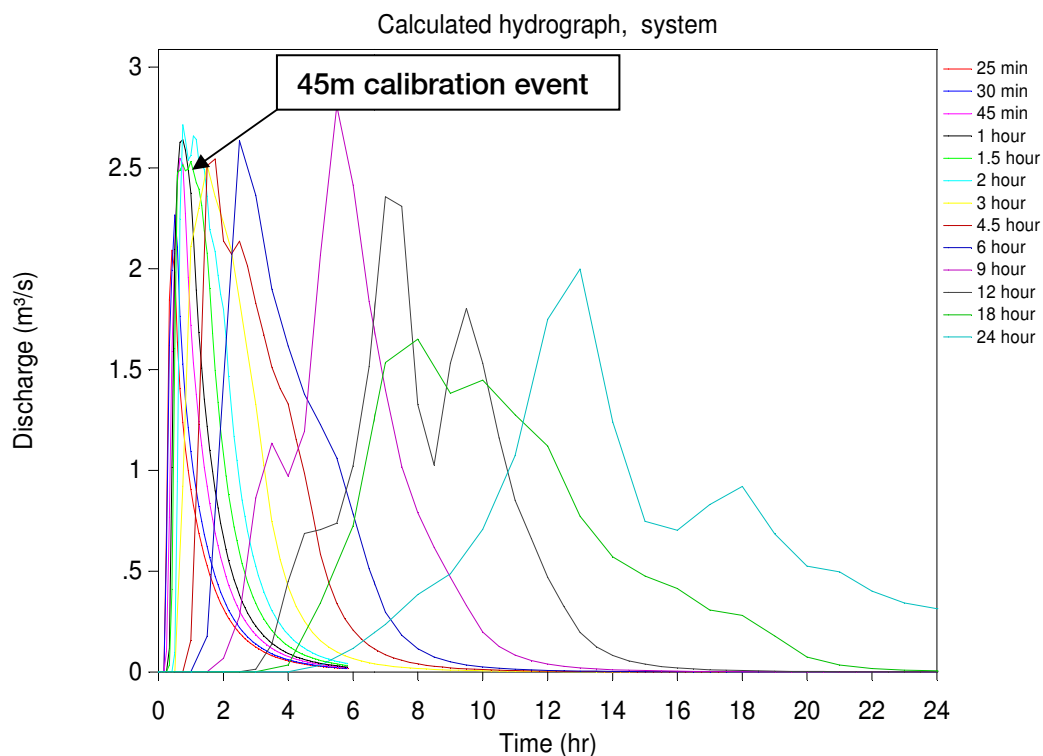


Figure 8 Site Outlet Calibration

Following the reconciliation of the model, each of the catchments was re-run with the developed conditions model including increased FI's and reach type changes. The results of this are discussed for each catchment in the below sections.

Mitigation Sizing

To mitigate the effects of residential development on the site each of the four catchments have been re-run with a proposed retarding basin. Basins have been sized through an iterative process of terrain manipulation and hydrological evaluation. A number of basin setups have been trialled, however only the successful runs are shown here.

Combined Northern System

To retard the flows from the northern portion of the site two retarding basins have been proposed in series forming a single asset. The 300mm outfalls have a capacity of around 0.4m³/s and their influence can be clearly seen in the mitigated hydrographs. The basins reduce the proposed development flows to below the existing discharge, and place the majority of the volume of the storm back into the existing stormwater network. The design hydrographs can be seen in Figure 9, with the conceptual basins shown in Figure 10. These basins have been conservatively modelled with the peak water level of the downstream basin forming the base outlet level of the upstream basin. The basin details can be seen in Table 3.

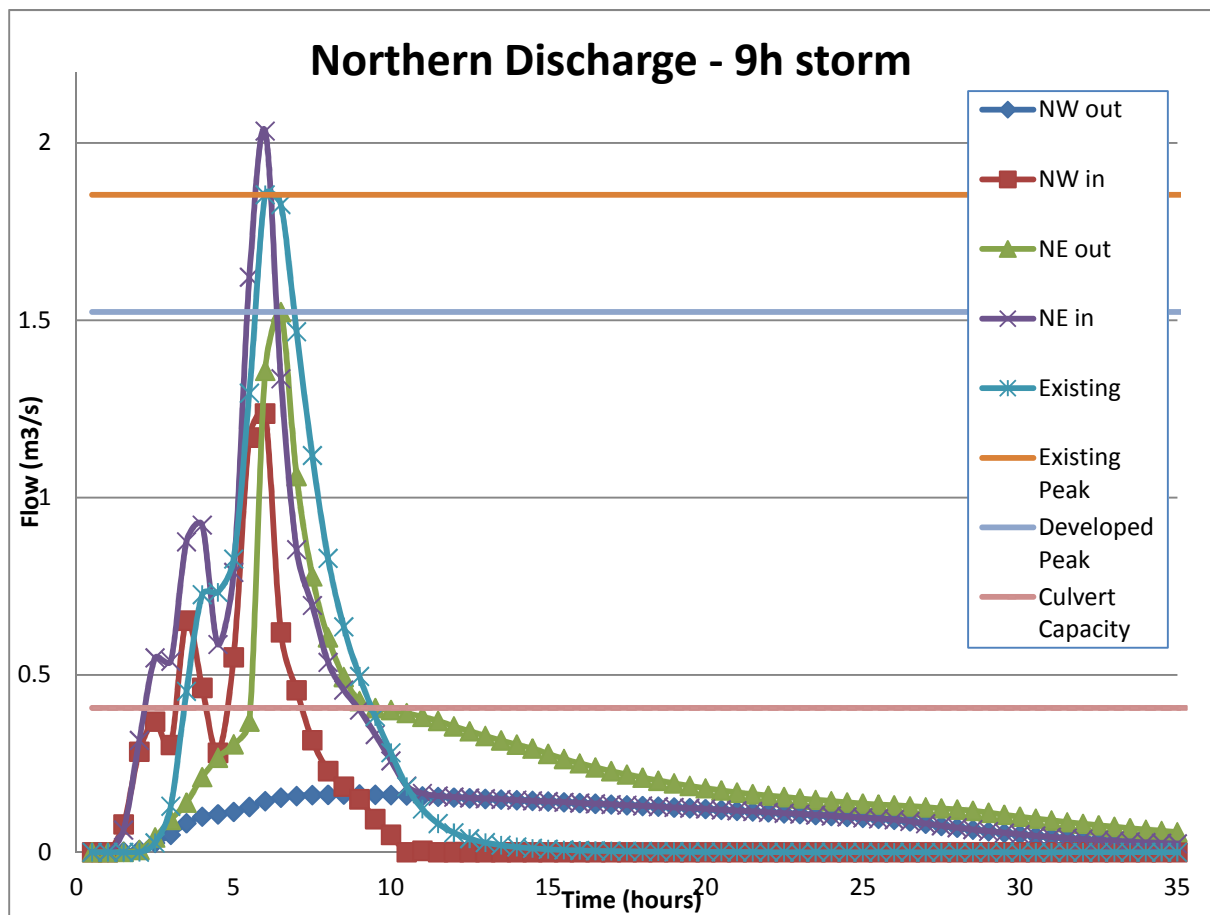


Figure 9 Proposed Northern Retardation

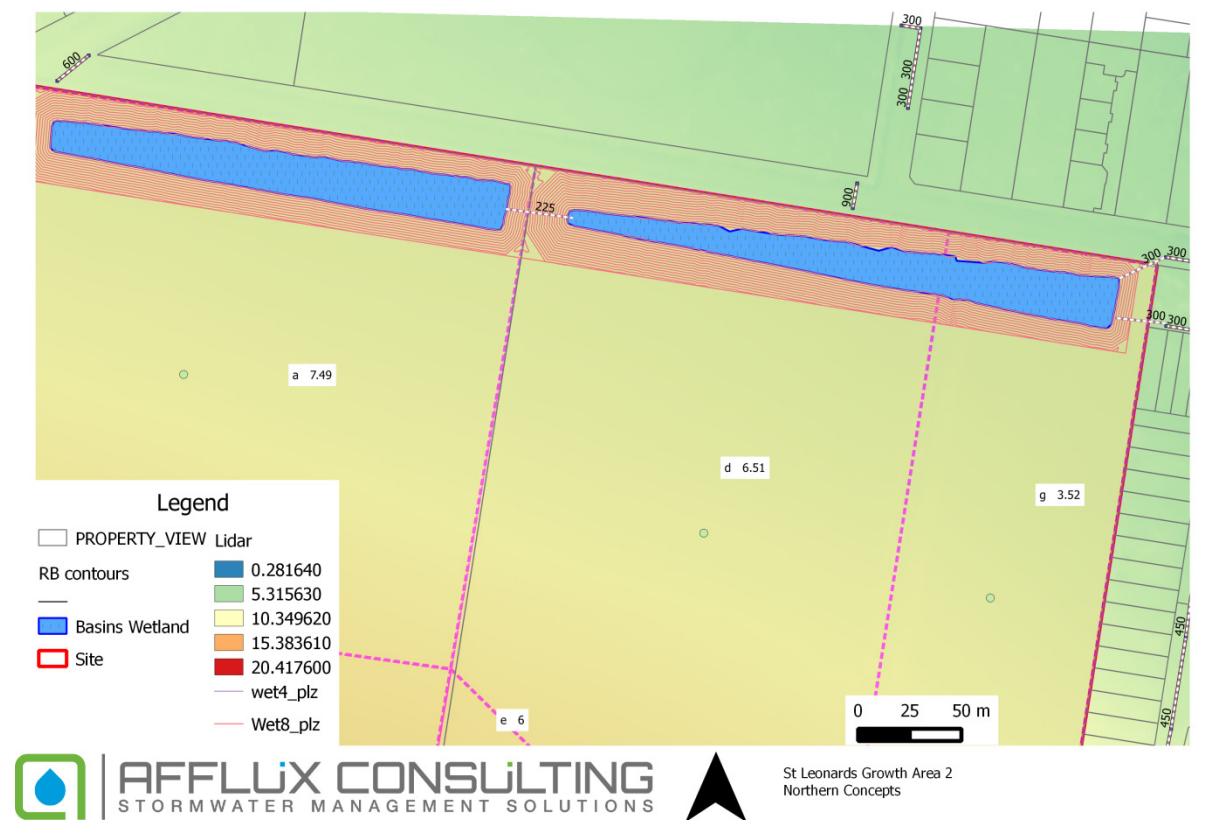


Figure 10 Northern Concept Designs

Table 3 Northern Basin Details

Basin	Basin Reserve Area (m ²)	Basin Peak Volume (m ³)	Basin Peak Water Level (mAHD)	Basin Outlet (m)	Basin Weir (mAHD)	Base Area (m ²)
North East	13,100	9,840	5.55	2 x 0.3	5.45	4,486
North West	11,000	10,100	7.06	0.225	7.3	4,450
Total	23,100	19,900				8,936

Given this northern basins interaction with downstream developed catchments, care has been taken to ensure that all 100 year ARI flows are retarded below the existing rates. The reductions found with this basin setup can be seen in Table 4, with the peak 9 hour duration highlighted. This will result in significant reductions in downstream flows, and hence flooding reductions. It should also be noted that all 100 year storms up to the 2 hour event will remain in pipe. This is particularly relevant given the summer thunderstorm events that this region is known for. It is noted that the current concept has been conservatively designed at this preliminary stage of design. This allows for both changes in lot size and density and should be revisited at the detailed design phase.

Table 4 Northern Basin Flow Reductions

Duration	Developed Flow	Existing Flow	Reduction
20m	0.2331	0.7763	-0.5432
25m	0.2667	0.9599	-0.6932
30m	0.294	1.1096	-0.8156
45m	0.3379	1.4287	-1.0908
1h	0.3693	1.6002	-1.2309
1.5h	0.4047	1.6977	-1.293
2h	0.7764	1.7802	-1.0038
3h	0.9856	1.6994	-0.7138
4.5h	1.0038	1.5855	-0.5817
6h	0.9674	1.7598	-0.7924
9h	1.5232	1.854	-0.3308
12h	1.305	1.6684	-0.3634
18h	0.9336	1.2062	-0.2726
24h	1.2388	1.4347	-0.1959

Southern Discharge

The southern discharge system has a natural depression that leads to the existing set of culverts under Bluff Road. This forms an obvious location for both retardation and water quality features by using the landform in the most efficient way. Care has been taken to ensure that this basin does not overtop Bluff road and operates using the existing pipes only.

The existing, developed and proposed mitigation flows from this catchment can be seen below in Figure 11, with the contouring and location in Figure 12.

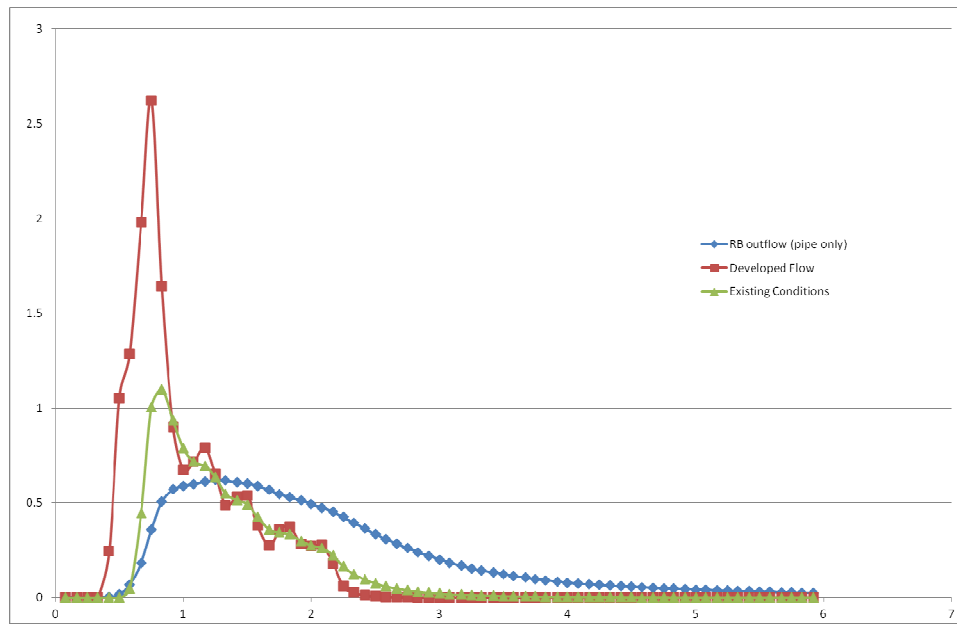


Figure 11 South East Catchment Hydrology

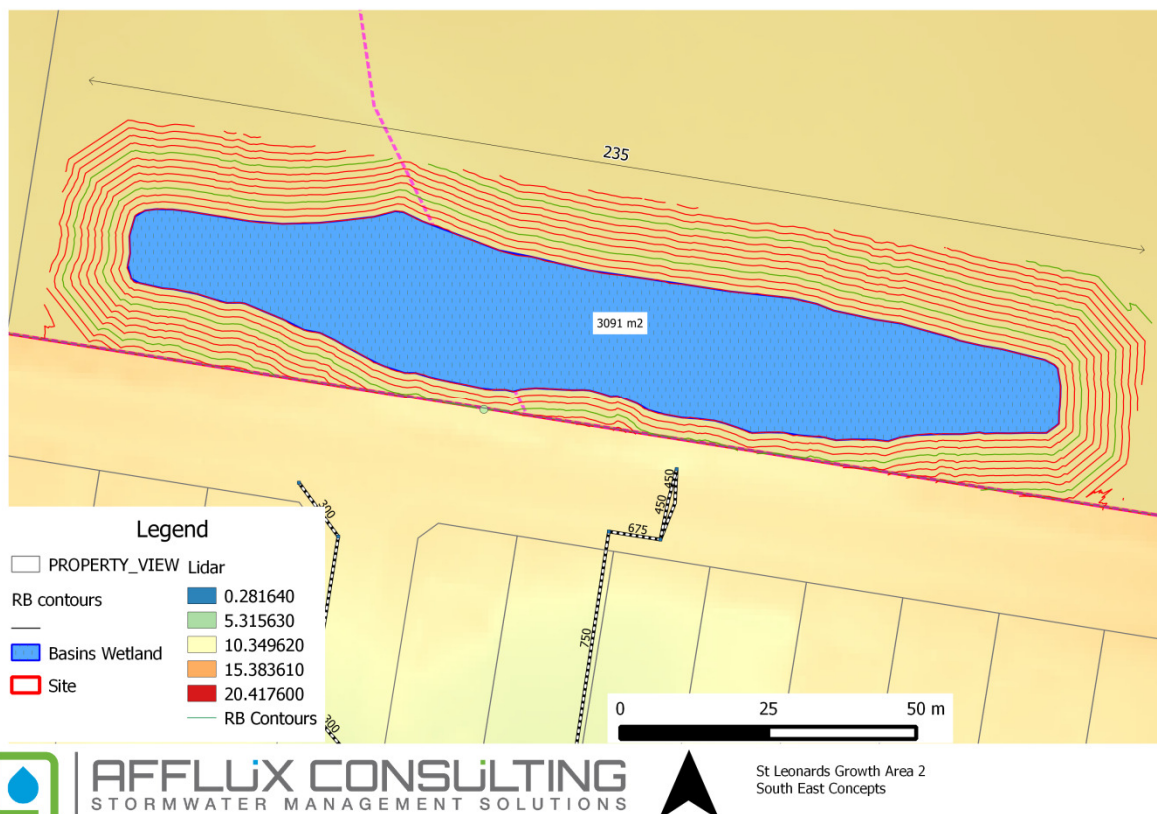


Figure 12 Southern Basin Concept

Table 5 South East Catchment Basin Details

Basin	Basin Reserve Area (m ²)	Basin Peak Volume (m ³)	Basin Peak Water Level (mAHD)	Basin Outlet (m)	Basin Weir (mAHD)	Base Area (m ²)
South East	7,303	2,380	10.94	2 x 450 (exist)	10.99	3,090

South West Catchment

A small basin is required in the south west corner to reduce the flows back to the exiting discharges. This basin pulls the flows back significantly as can be seen in Figure 13 and conceptually in Figure 14. It is noted that it would be technically possible to divert most of this catchments flows back to the South East basin although this would be in contravention of the existing landforms. To achieve the required storage to include this catchment in the South East basin an increase of approximately 25% would be required.

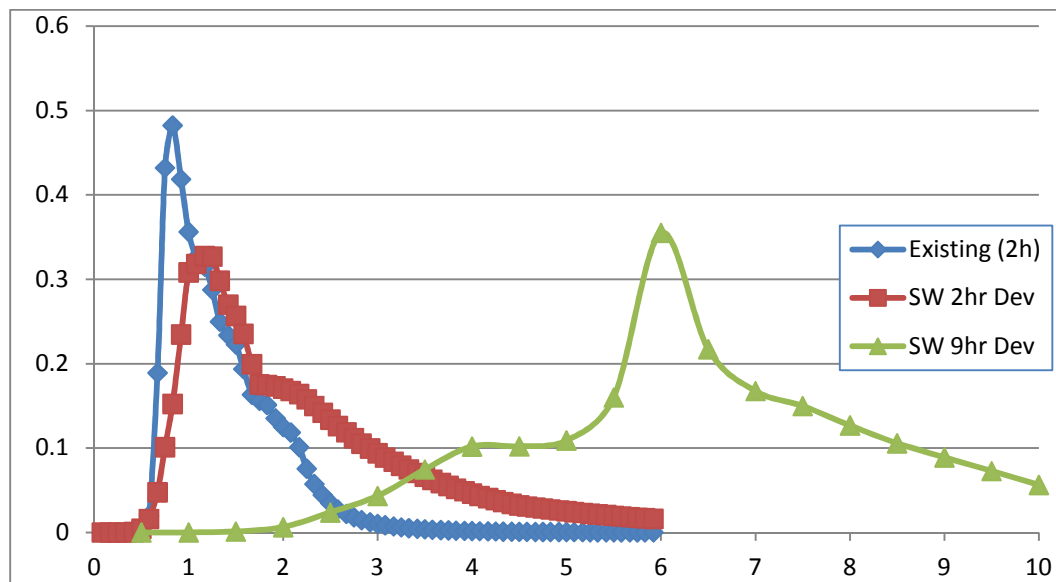


Figure 13 South West Basin hydrographs

Table 6 South West Basin Details

Basin	Basin Reserve Area (m ²)	Basin Peak Volume (m ³)	Basin Peak Water Level (mAHD)	Basin Outlet (m)	Basin Weir (mAHD)	Base Area (m ²)
South West	2,550	1,140	12.59	1 x 0.375	12.55	1,614

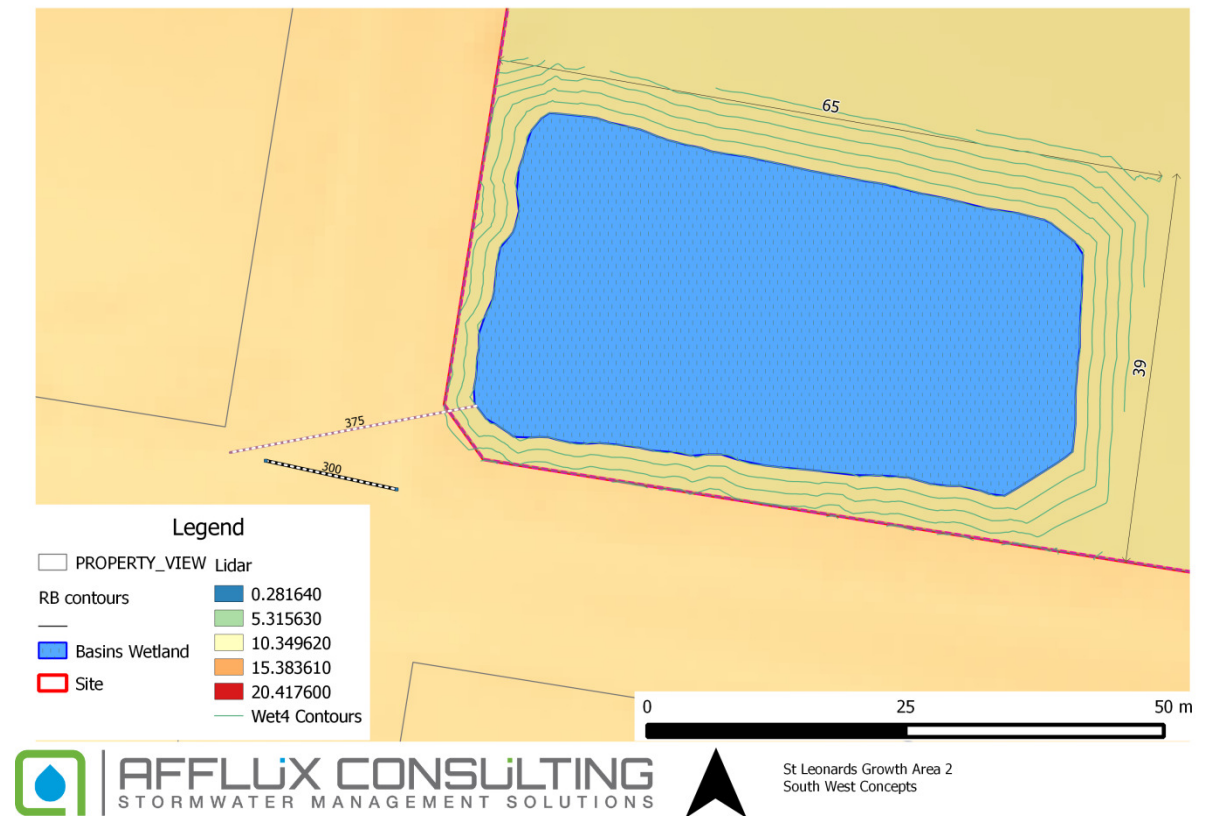


Figure 14 South West Basin Concept

5. Water Quality Modelling

To ensure that the development meets the BPEM requirements of Clause 56-7.04 a MUSIC model (v5) has been created for the catchment. The 'Greater Geelong Music Parameters Design Notes' has been used as the major reference for modelling in this catchment. As such the Geelong North Rainfall station with reference year 1985 has been used along with evaporation data as listed in the guidelines. Rainfall was run at a 6 minute interval to match the lowest Time of Concentration of the catchment. All other parameters were set as per the Greater Geelong Guidelines.

The philosophy has been to generally create wetlands in preference to other water quality measures due to their robustness in long term survival, reduced maintenance, and ability to store greater amounts of water above the Normal Water Level (NWL) in a retarding basin situation.

The development catchment areas and FI's have been calculated as per the hydrologic model and can be seen in Table 7. As the subdivision is in a preliminary planning stage catchments have been assumed to uniformly be of average residential zone (500-800 m² blocks). This results in an effective fraction impervious of 0.6 as per the Greater Geelong guidelines.

Table 7 MUSIC Parameters

Catchment	Ha	FI
1a (NE)	31.9	0.6
1b (NW)	21.4	0.6
1c (SE)	14	0.6
1d (SW)	6.3	0.6

Wetlands

Two sets of wetlands have been proposed, Wetland 1 in the base of the Northern Retarding basin and Wetland 2 in the base of South Eastern retarding basin. Wetlands have been sized in MUSIC and will contain 80% macrophyte coverage. All wetlands should retain at least 20 meters from the wet edge to residential housing, as can be seen in the concept for the northern wetlands. Concept designs of the systems are depicted in Section 0.

Sediment Basins

Both wetlands have been sized to contain sedimentation basins capable of reducing the sediment load into the wetlands themselves, hence increasing life expectancy. Sedimentation basins were sized using the Fair and Geyer equations, with the results summarised below.

Raingardens

One raingarden is proposed to deal with the small south western catchment. Given the limited depth in this area this garden has a shallow filter depth, and shallow extended detention.

Table 8 Sedimentation Basin Sizing

Sediment Pond	Sed Pond Size (m ²)	Target Size	Fraction Removal	Clean out Frequency
Wetland 1	2000	125 micrometer	98%	14.1 years
Wetland 2	800	125 micrometer	96%	14.8 years

Music Model

The MUSIC model setup can be seen in Figure 15 below. The treatment efficiencies for each of the treatment measures, and an overall site water quality balance can be seen in Figure 20 below. As can be seen, the catchment total meets the best practice requirements. Concept designs with the characteristics of each treatment can be seen in Section 7 of this report.

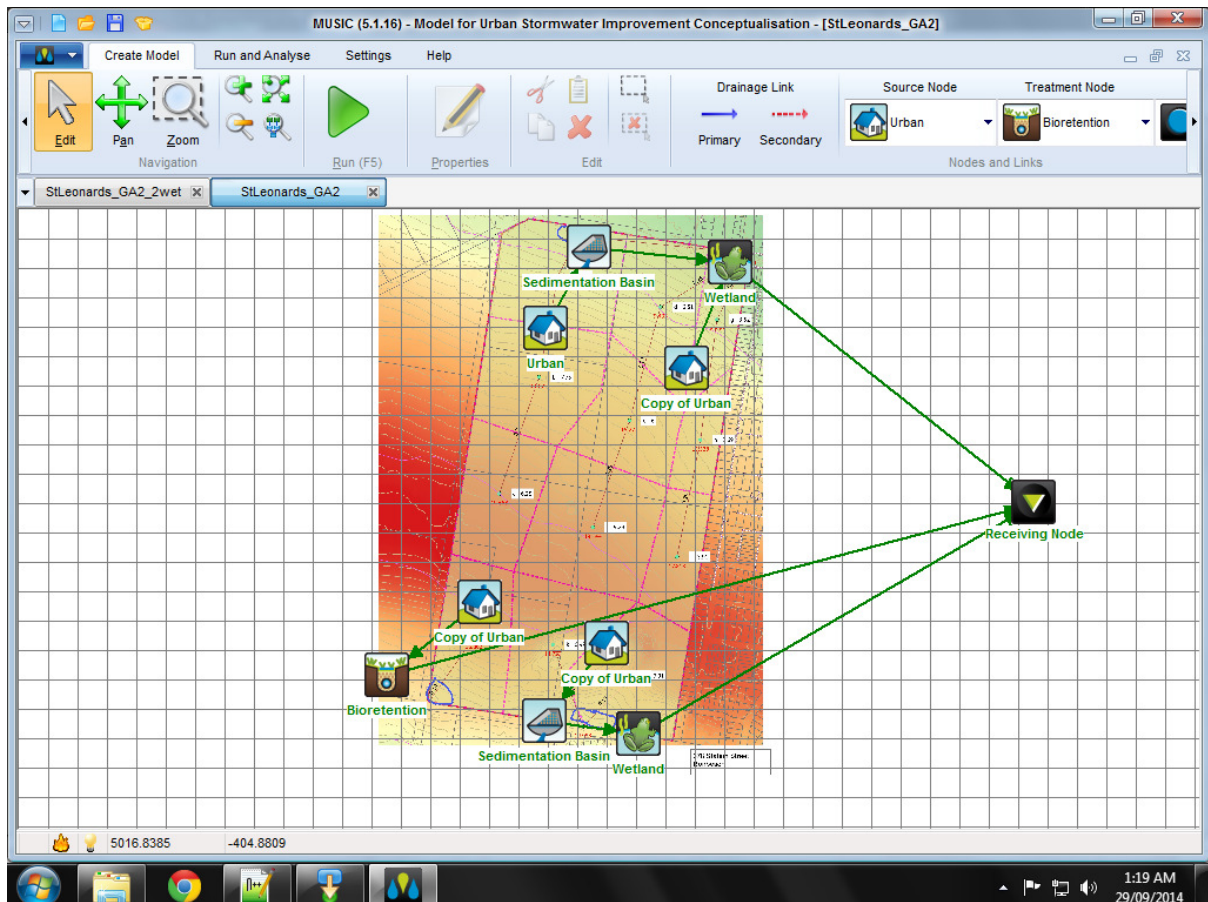


Figure 15 Catchment MUSIC model layout

Treatment Efficiencies

	Sources	Residual Load	% Reduction
Flow (ML/yr)	146	138	5.6
Peak Flow (m3/s)	4.18	0.425	89.8
Total Suspended Solids (kg/yr)	27.3E3	3.72E3	86.4
Total Phosphorus (kg/yr)	57.8	14.9	74.3
Total Nitrogen (kg/yr)	413	229	44.4
Gross Pollutants (kg/yr)	6.36E3	0.00	100.0

Wetland 1

	Sources	Residual Load	% Reduction
Flow (ML/yr)	38.4	34.9	9.2
Peak Flow (m3/s)	1.83	0.101	94.5
Total Suspended Solids (kg/yr)	7.30E3	492	93.3
Total Phosphorus (kg/yr)	15.6	2.92	81.3
Total Nitrogen (kg/yr)	108	46.2	57.3
Gross Pollutants (kg/yr)	1.67E3	0.00	100.0

Wetland 2

	Sources	Residual Load	% Reduction
Flow (ML/yr)	17.3	15.5	10.4
Peak Flow (m3/s)	0.825	0.543	34.3
Total Suspended Solids (kg/yr)	3.33E3	688	79.3
Total Phosphorus (kg/yr)	6.83	3.90	42.9
Total Nitrogen (kg/yr)	48.4	24.4	49.6
Gross Pollutants (kg/yr)	751	0.00	100.0

Raingarden

	Sources	Residual Load	% Reduction
Flow (ML/yr)	202	188	6.7
Peak Flow (m3/s)	4.18	0.577	86.2
Total Suspended Solids (kg/yr)	37.9E3	4.90E3	87.1
Total Phosphorus (kg/yr)	80.3	21.7	73.0
Total Nitrogen (kg/yr)	570	300	47.3
Gross Pollutants (kg/yr)	8.78E3	0.00	100.0

Total Site

6. Treatment Concepts

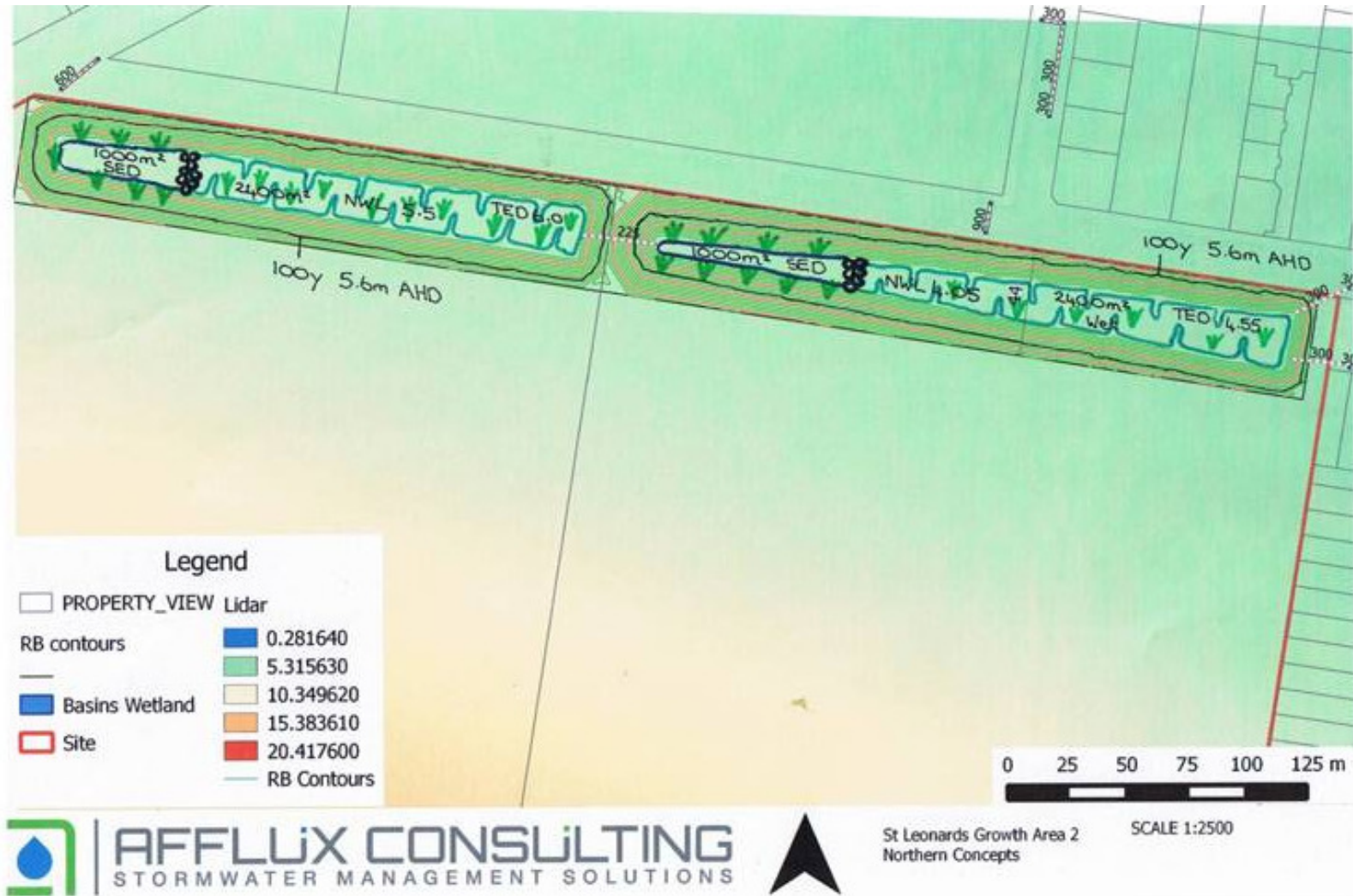


Figure 16

Northern Basin – Concept Design

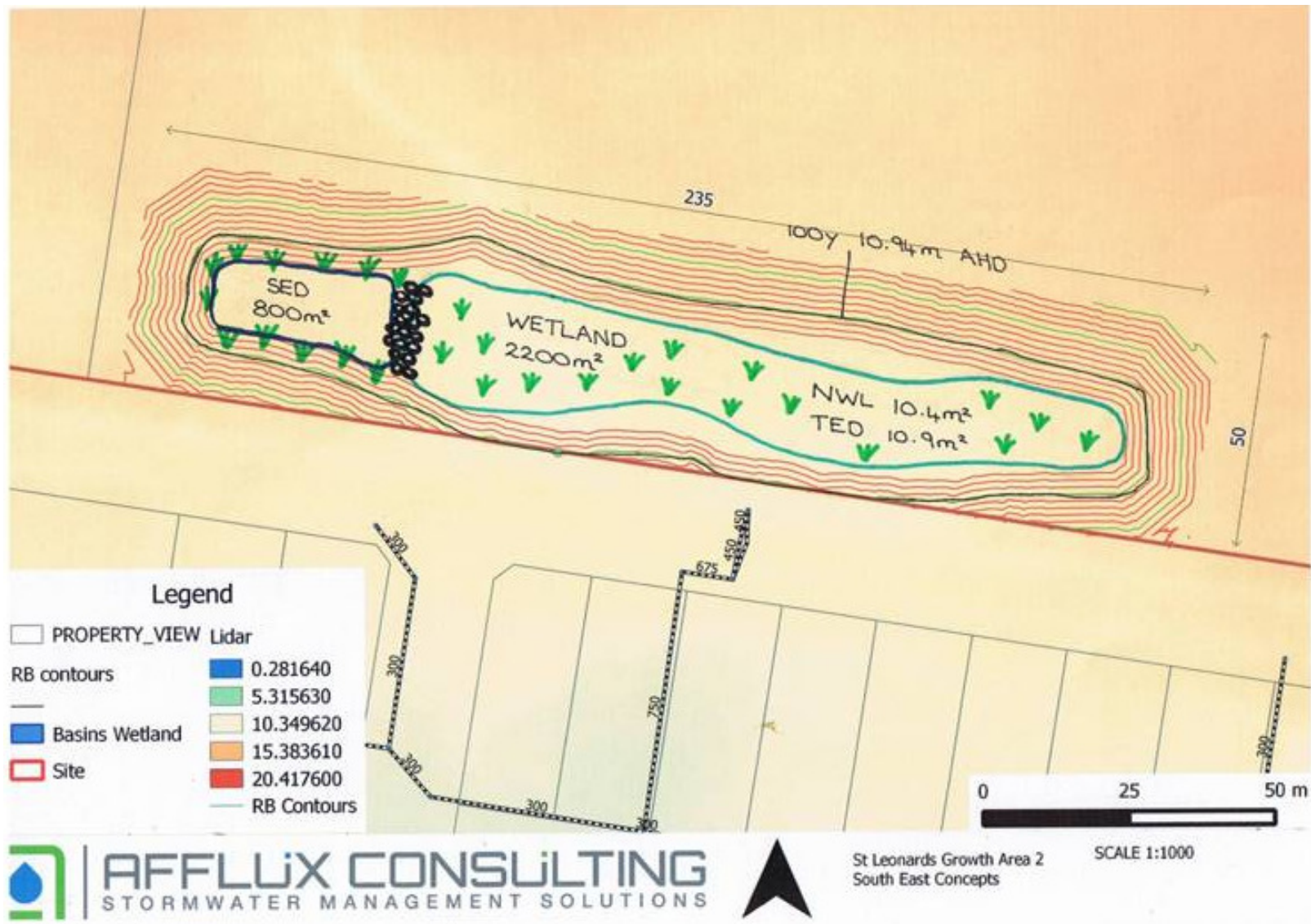


Figure 17 South Eastern Basin – Concept Design

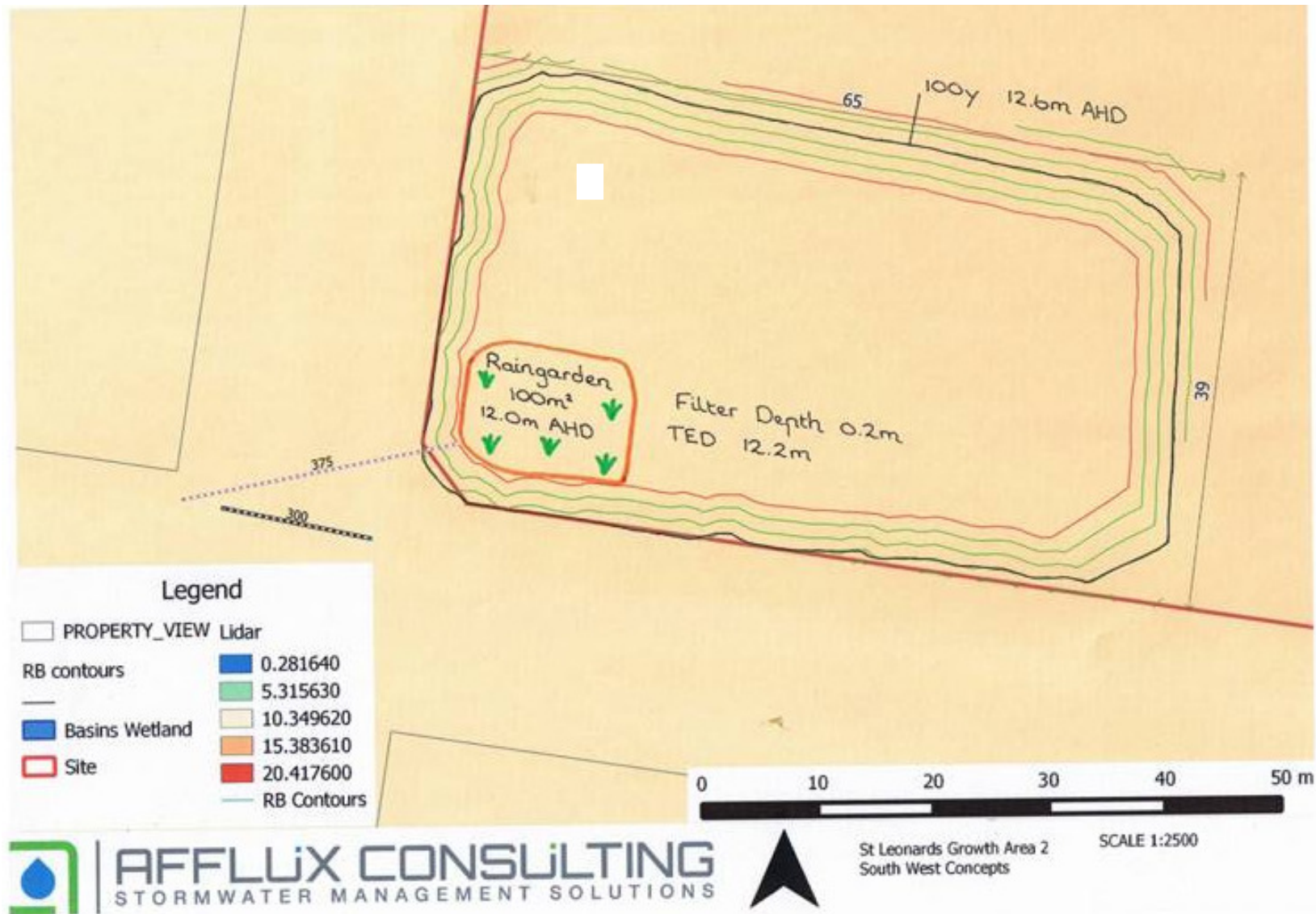


Figure 18 South Western Basin – Concept Design

7. Conclusions

This report has investigated the water quality and quantity requirements for the Growth Area 2 development in St Leonards. The report has shown that all of the provisions of the Greater City of Geelong Planning scheme can be met by the subdivision. This includes:

- All development flows to be below pre-development flows
- All lots to have 300mm freeboard to 100 year ARI flood levels in each of the retarding basins
- Offsite flooding will be reduced via the significant flow reductions associated with these concepts.
- Water quality treatments for the site meet Best Practice requirements of Planning Scheme Clause 56-7.04

As such it can be proved that this development will have no detrimental effect on the surrounding subdivisions or downstream receiving environments.

Appendix A

Theoretical Catchment 1

The north east corner of the site is the discharge point for a significant area of the total development. This contributing area is much greater than the 20Ha property ownership boundary, and as such it can be seen that an additional drainage burden is placed on this property.

To investigate the magnitude of this burden, a model was formed with the two basins disconnected and treating only the property ownership catchment area above them. Whilst this will not meet the flow requirement of the downstream outfall, it will distinguish the required storage volume for Catchment 1 without the addition of flow from Catchment 2.

The results of this model can be seen below (Figure 19), with the mitigated flow matching the existing theoretical 20Ha flow. To achieve this result a storage of 6,220 m³ is required. This basin is 47% smaller than that as proposed to mitigate the entire catchment.

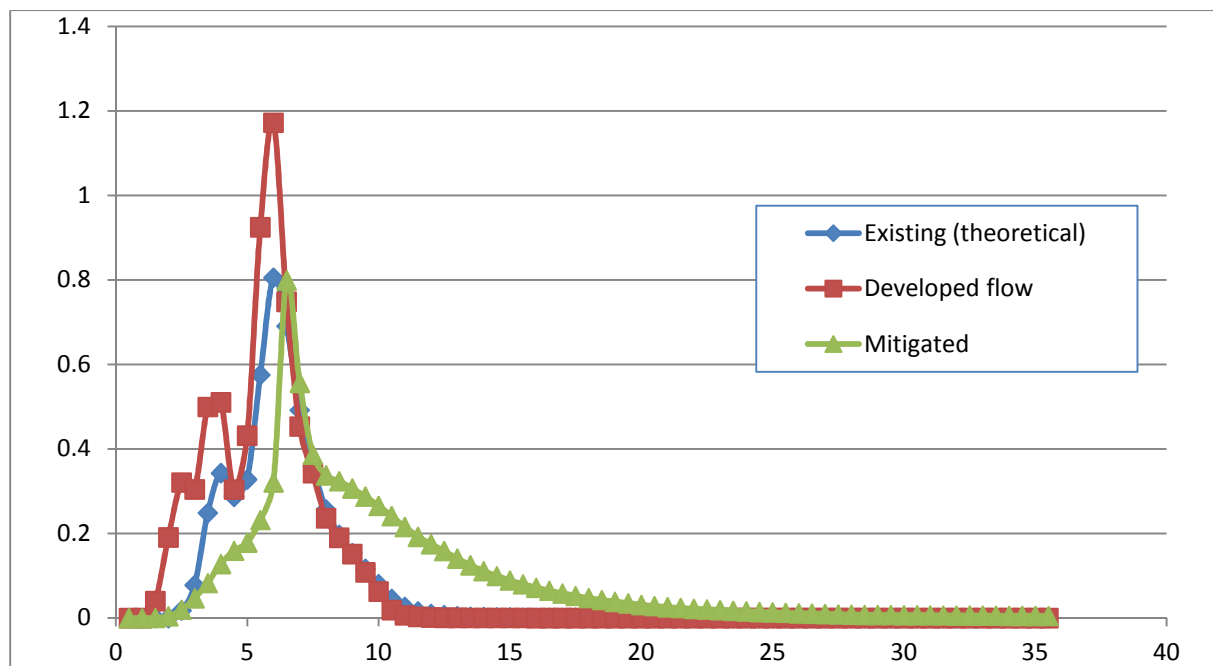


Figure 19 Theoretical Basin (20 Ha North East catchment)

Peak elevation= 5.12 m

Peak outflow = 0.80 m/s (pipe & spillway flow)

Peak storage = 6,220 m³

Peak Storage system = 9,840 m³

Additional storage = 3620 m³