Morwell North-West DCP Drainage Report – Final Study Report

April 2016

DOCUMENT STATUS

Version	Doc type	Reviewed by	Approved by	Distributed to	Date issued
V01	Study Report	TJC	SMH	Chas Shinkfield	23/03/2016
V02	Final Study Report	PGA Group	SMH	Chas Shinkfield	01/04/2016

PROJECT DETAILS

Project Name	Morwell North-West DCP Drainage
Client	Paroissien Grant and Associates Pty Ltd
Client Project Manager	Chas Shinkfield
Water Technology Project Manager	Simon Hof
Report Authors	S Hof, C Dotto, A Vendargon & T Cousland
Job Number	3926-01
Report Number	R03
Document Name	3926-01_R03v02.docx

Cover Photo: Site visit image, Existing wetland, Morwell West (2015)

15 Business Park Drive
Notting Hill VIC 3168

Telephone (03) 8526 0800
Fax (03) 9558 9365

ACN No. 093 377 283
ABN No. 60 093 377 283

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

Latrobe City Council is currently overseeing the development for a large greenfield area north west of the Morwell CBD, with drainage within this area to be controlled via a formal Development Contributions Plan (DCP). Water Technology has been commissioned to undertake a detailed drainage investigation of the region covered by the proposed DCP and to investigate flooding issues across the site, with consideration giving to proposed development. The drainage investigation is intended to provide sufficient detail to demonstrate the feasibility of the proposed Water Sensitive Urban Design (WSUD) and stormwater management for the study area.

1.1 Site Location

The study area consists of approximately 134 Ha of irregularly shaped land west of Maryvale Road, south of Old Melbourne Road and east of Latrobe Road (Figure 1-1). Generally the land drains from east to the west with all of the study area draining to a single outlet (designated waterway) found at the northern end of Latrobe Road. This waterway drains west via the Morwell River diversion system through the Yallourn mine before discharging into the Latrobe River.

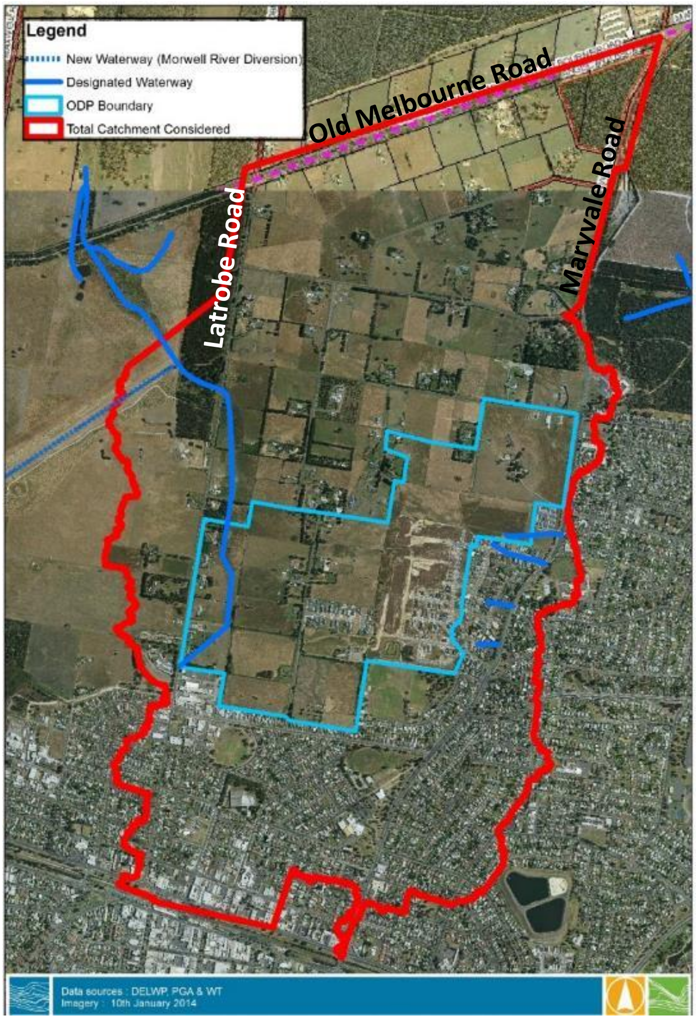


Figure 1-1 Study Area

2. STUDY SCOPE OF WORKS

The following scope of works was undertaken for this investigation:

2.1 Data Collection & Collation

- Site visit by Water Technology staff member; and
- Review of available information.

2.2 Hydrologic Analysis

- Development of an Existing conditions hydrologic model (RORB) to determine both external and internal catchment flows which impact the proposed development;
- Development of a Developed conditions hydrologic model (RORB). This will help describe what the internal development effect is;
- Development of two Mitigated conditions hydrologic models (RORB) to determine attenuation requirements to meet best practice design conditions (1% AEP storm) and requirements to meet the downstream capacity constraints (currently it is assumed the existing stormwater infrastructure will be unable to manage existing conditions peak flow rates);

2.3 WSUD (Water Quality) Analysis

- Development of a MUSIC model to determine the required water quality treatment;

2.4 Concept Design

- Integrate findings of hydrological analysis (attenuation and water quality) into the site constraints of the ODP;
- Complete typical waterway cross sections for the modelled reach. Typical dimensions and batter slopes will be included in the concept design.
- Develop concept design surface for the waterway based on the typical cross sections, also including online storage requirements details.

2.5 Hydraulic Analysis

- Development of an Existing conditions hydraulic model (TUFLOW) to establish existing conditions flood levels and feature (culvert) capacities;
- Development of a Developed conditions hydraulic model (TUFLOW) to establish developed conditions flood levels and feature (culvert) capacities, this will help describe the what the internal development effect is;
- Development of two Mitigated conditions hydraulic models (TUFLOW) to establish mitigated conditions flood levels and prove that the proposed works are functional and fit for purpose;

2.6 Reporting

- Compilation of a brief project report summarising work undertaken by Water Technology, key to this output will be a concise summary suitable for direct inclusion in any PGA reporting outputs;
- Meeting with Mesh Planning and PGA to go over the draft report;
  Post PGA's review of Water Technology's draft report, recommended amendments will be reviewed and incorporated into the final study report.

3. DATA COLLECTION & COLLATION

The current study is not the first drainage investigation undertaken inside the study area. Several past investigations and plans were provided to Water Technology for review. Data provided is detailed in study reference list.

3.1 Reports & Plans

Four key documents were reviewed, the following text summaries key findings and conclusions from these reports and plans:

- *Morwell North West Development Plan (Background Analysis: Final)* by CPG (formally Coomes consulting) - June 2010. The area investigated is shown in Figure 3-1. The study identified that:
    - "Lower Catchment" required a retarding basin with storage of approximately 30,000 m^3 in the drainage reserve immediately south of Gordons Road (Street); and a total wetland treatment area of 1.7 hectares;
    - *"Middle Catchment"* retarding basin discussed but no volume identified, combined with a wetland treatment area of 0.45 hectares;
    - *"The Upper Catchment" required* on-site detention (no volume identified) and no specific wetland treatment sizes nominated. Stormwater quality requirements offset using "oversized" wetlands in the upper and middle catchments.
- *Morwell North West Development Plan FINAL*- CPG October 2010 discussed:
    - The main drainage reserve within the ODP nominating typical cross-section widths of 25-40 m).
    - Four WSUD features proposed, 3 retarding basins and 2 culvert crossings sized (see Figure 3-2)
- *Heritage Boulevard Site Stormwater Management Plan* CPG March 2011- This report looked a small section of the of the "*Lower Catchment*" from the *Morwell North West Development Plan* and nominated that;
    - No storage (attenuation) was required with HEC-RAS modelling used to demonstrate that increases in depths downstream were not significant (0.03 m), with the conclusion stating "the impact on downstream land owners needs further consideration by Council";
    - MUSIC modelling determined sediment traps (674 m^2 & 477 m^2) and raingardens (1,880 m^2 & 970 m^2) would meet best practice requirements;
        - These features were represented in lieu of the WR02/3 wetland discussed in *Morwell North West Development Plan FINAL- CPG October 2010*
- *Amended Endorsed Landscape Plans* by Ian Barker Gardens from May 2013. These plans appear to be detailed designs showing two wetlands in series (the plans appear to be broadly based on the wetland WR02/3 discussed earlier);
    - No further documentation describing the wetlands observed during the site visit were available for review.

Unfortunately no technical design assumptions or hydrological or water quality models were available for review as part of the study. As such all analysis completed by Water Technology cannot be directly compared to that undertaken in the past.

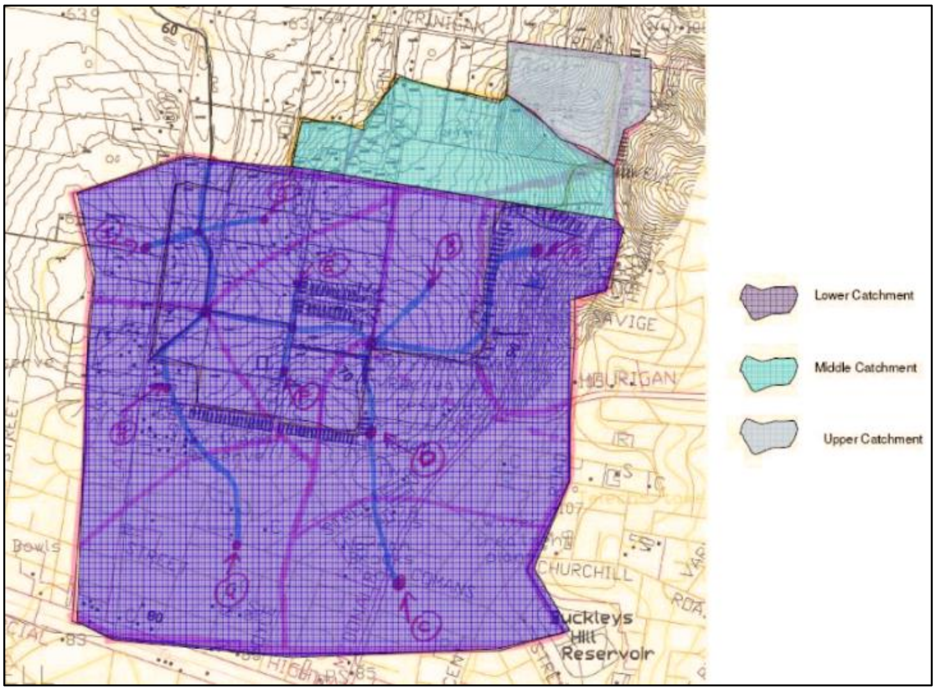


Figure 3-1 Coomes Consulting 2010 – Overall catchment Plan (Source: CPG Morwell North West Development Plan)

Figure 3-2 CPG 2010 – Drainage Infrastructure (Source: Morwell North West Development Plan FINAL)

3.2 Spatial Data & Survey

Serval key data spatial sets were collected for use in this study that was provided by Latrobe City council and DELWP, this information was further enhanced by observation and records collected during site survey and site visits by PGA and Water Technology staff. Figure 3-3 shows the LiDAR topography covering the study area and some of the site survey collected by PGA group.

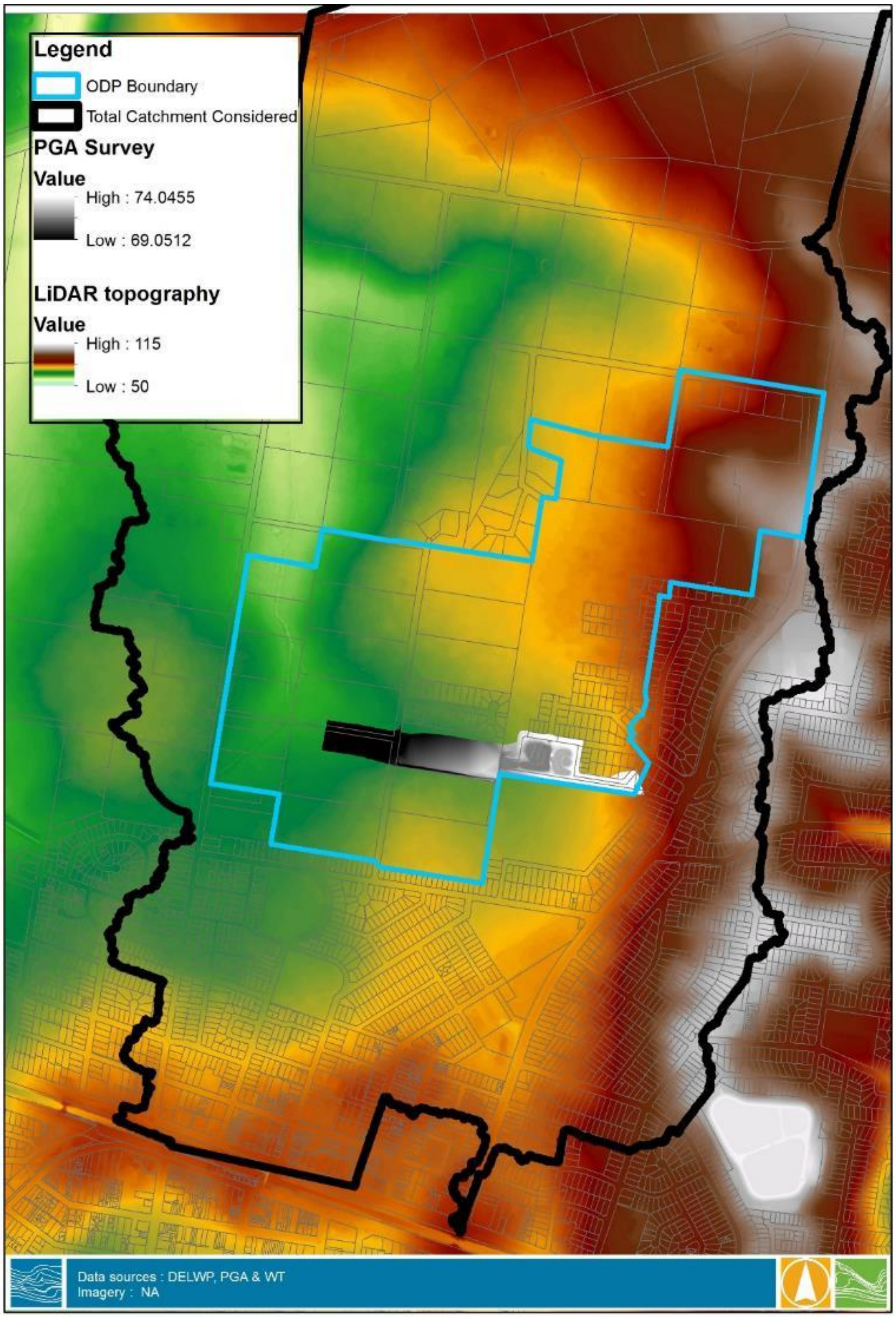


Figure 3-3 Existing Conditions Topography of the Study Site

3.3 The Existing Wetland / Retarding Basin Feature (WR02/3)

Two constructed wetlands in series exist within the study area, the attenuation and treatment performance of these assets was not known at the beginning of the project. Site survey and site visit observations by PGA and Water Technology were used to establish the performance of these features.

The following was noted:

- The eastern wetland is perched above the west by ~500 mm;
- The extended detention (0.5 m) in the basin is the main storage component of the east wetland (1,365 m³), above this level flows are controlled a large (~15 m wide) weir;
- Low flows between the two features is controlled by an orifice plate and weir arrangement (Figure 3-5);
- Western (downstream) wetland is largely drowned out (downstream controlled), with the water level in the downstream open drain higher than the intended normal water level in the wetland (when visited) see Figure 3-6;
- Wetlands appear to be designed/constructed using best practice methodology (at the time of construction).

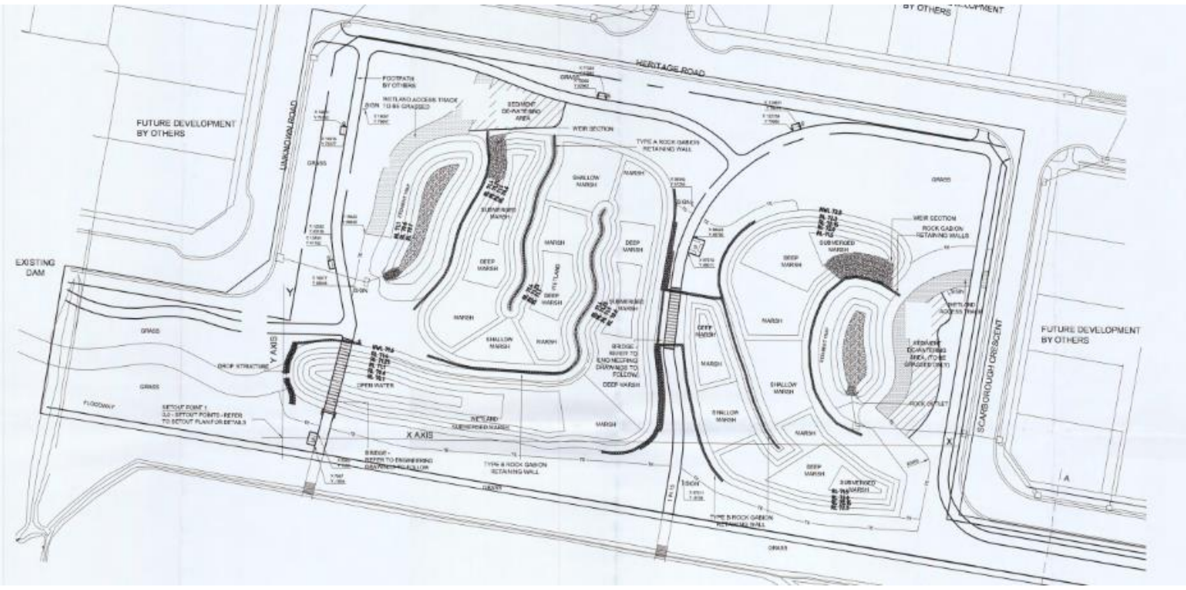


Figure 3-4 Amended Endorsed Landscape Plans by Ian Barker Gardens from May 2013

Figure 3-5 Eastern Wetland Outlet Arrangement (Source: WT site visit September 2015)

Figure 3-6 Western Wetland Outlet Arrangement (Source: WT site visit September 2015)

4. STORMWATER QUANTITY MANAGEMENT

4.1 Overview

Under developed conditions, it is proposed to split the developed area in to four regions (Figure 4-1). Each region would drain to a combined retarding basin wetland feature. Catchment details of each region are shown in Table 4-1. Each one of these systems would eventually drain to the designated outlet on the western side of Latrobe Road. Conveyance around the development would be managed using a conventional pit and pipe network (up to 20% AEP events) and roadways and drainage reserves >20% AEP event.

These conditions and resultant designs, builds on work by CPG (Coomes) and uses the locations for attenuation nominated in that work. Basin / Water Quality features would be online, resulting in external catchment flows being routed through each feature (where applicable).

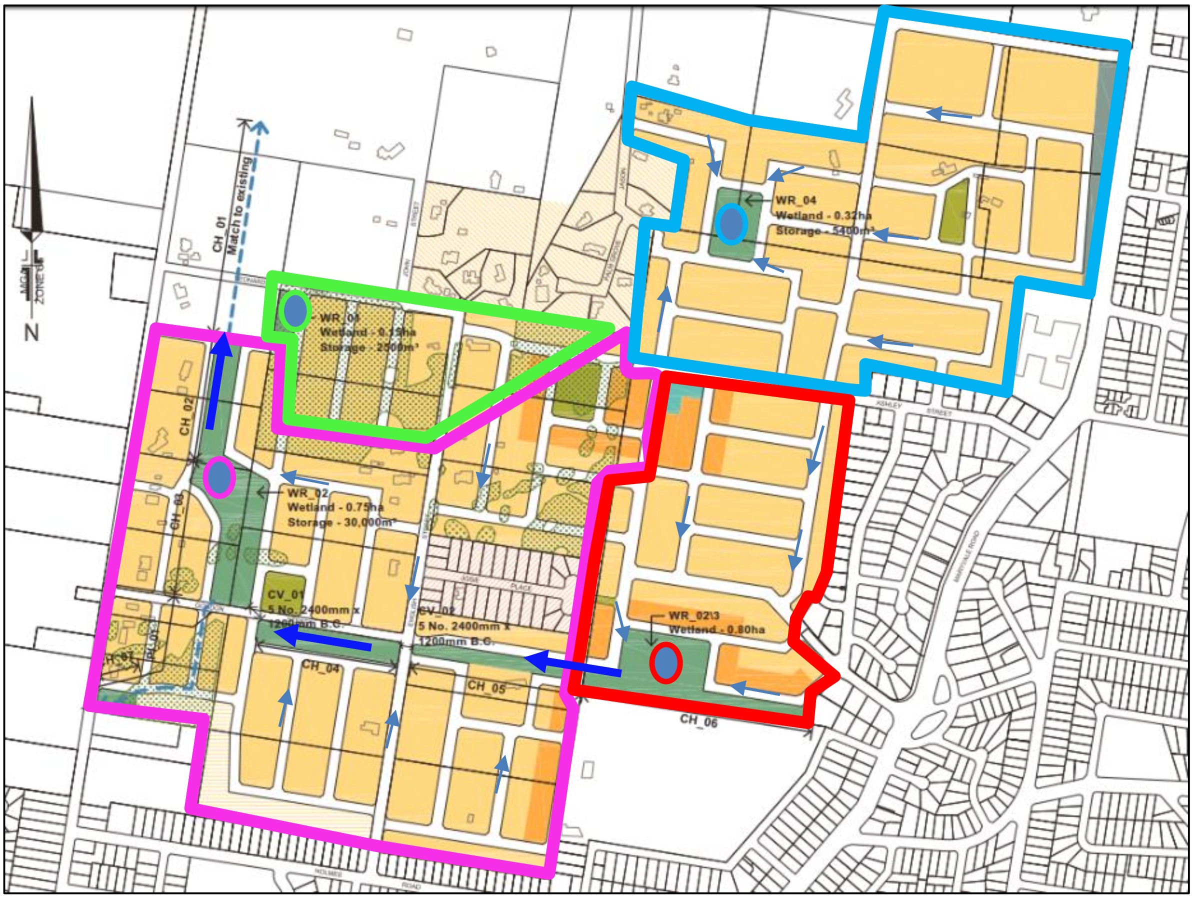


Figure 4-1 Proposed Developed Concept Drainage Layout

Table 4-1 Internal Catchments

Internal Catchment	Basin Name (CPG)	~ Catchment Area (km²)
Red	WR02/3*	0.2
Pink	WR02	0.65
Green	WR03	0.1
Blue+	WR04	0.4

* Existing attenuation & treatment features
+ A portion of this catchment may free drain to the north, water treatment and attenuation will be over compensated for in other wetland features.

5. HYDROLOGIC ANALYSIS

To better understand the hydrological impacts of the proposed works at the subject site, a RORB (rainfall runoff and streamflow routing) model was constructed for existing conditions and reconciled to peak flow estimates using the Rational Method.

Once reconciled the RORB model was modified to reflect the proposed changes to the subject site, namely changes in flow directions and the increase in fraction imperviousness of the sub-catchments within the development.

5.1.1 Rational Method Flow Estimate

A Rational Method analysis was undertaken for 7 independent areas (Figure 5-1) in accordance with the methodology outlined in Book 2 of Australian Rainfall and Runoff (AR&R, 1987). Further detail of rational calculations are provided in Appendix A, Table 5-1 show the peak flow estimates for the 7 areas considered in this investigation.

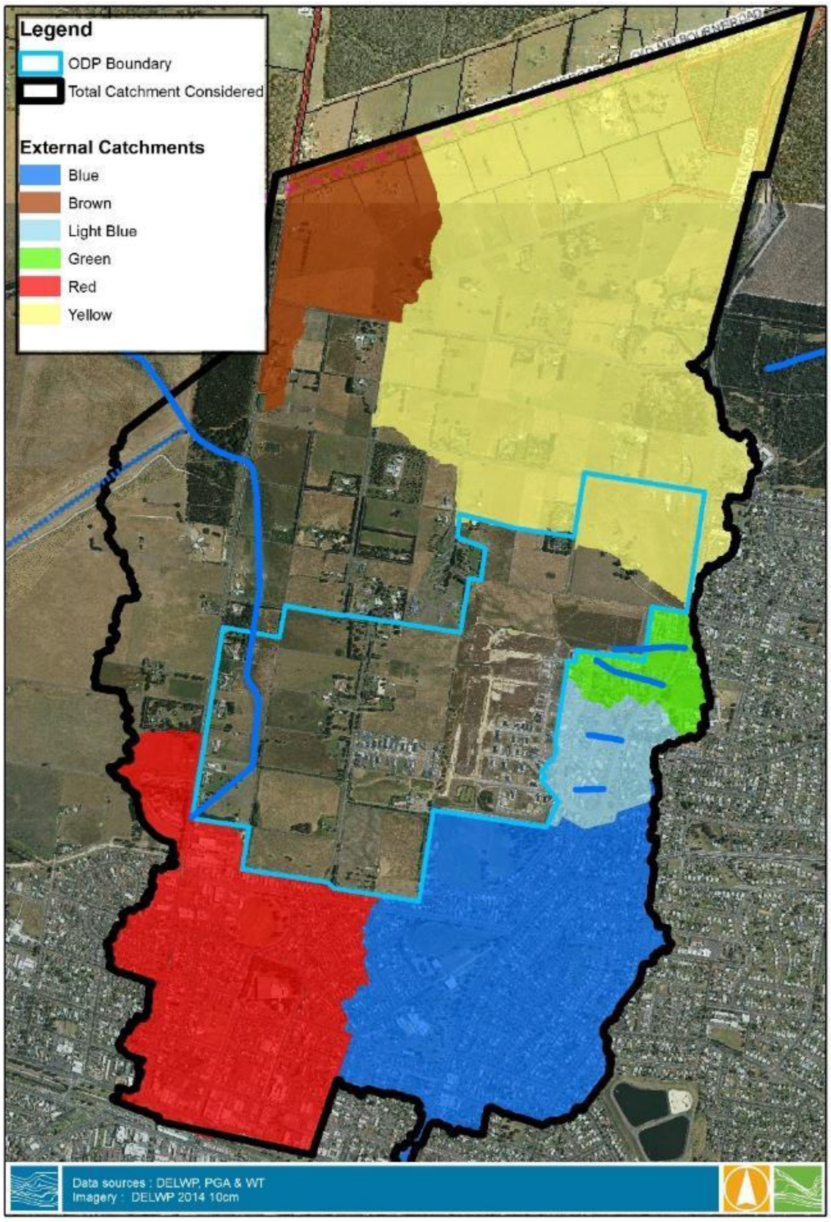


Figure 5-1 Location of Interstation Area's Considered in RORB Modelling

Table 5-1 Rational Method Peak Flow Estimates

Interstation area	Area (km^2)	FI	1% AEP (Rational)
Red	0.814	0.59	23.47
Blue	0.947	0.54	19.39
Light Blue	0.156	0.60	4.80
Green	0.117	0.60	3.76
Yellow	1.886	0.12	4.07*
Brown	0.329	0.10	1.07*
Internal Catchment	2.31	0.14	4.73*

* Rural Rational Method adopted

5.1.2 RORB Model Setup

A schematisation of existing drainage conditions was constructed using GIS software and then integrated into RORB. The site was modelled with 97 sub catchment areas of similar size for existing conditions based on the topography of the study area (1m LiDAR).

The contributing catchment areas adopted were generally consistent with CPG RORB modelling from 2010 (Figure 3-1) however it would appear the current modelling includes a slightly larger area upstream of the DCP area and also includes all contributing catchments to Latrobe Road (not considered in the CGP work).

The RORB model was set up with multiple interstation areas. Taking this approach assisted with maintaining model verification when catchment drainage paths were modified under developed conditions. Sub-area boundaries were delineated, with nodes placed at all junctions and areas of interest. Nodes were then connected by RORB reaches, each representing the length, slope and reach type. RORB sub-catchment delineation for the existing conditions are shown in Appendix A.

Temporal pattern used in the RORB model was taken from the ARR for Zone 1. A uniform areal pattern was considered appropriate given the simple, small scale nature of the model. No areal reduction factors were applied to the model

The existing fraction impervious values were determined based on the current Planning Scheme codes, and verified with current aerial photography. Fraction impervious values were weighted when sub areas contained two (or more) types of Planning Scheme codes within them. Faction impervious estimations were consistent with current best practice approaches and were based on Melbourne Water recommended values.

5.1.3 Model Calibration

The RORB model parameter determination followed a general approach which reconciles the 100 year design peak flows from the Rational Method and RORB model through adjustment of the RORB routing parameter, kc. The existing model had 7 interstation areas which were all individually verified through adjustment of the kc parameter, the results of this process is shown in Table 5-2.

It should be noted that the existing wetland retarding basin feature inside the study area (WR02/3) was excluded from the RORB model during the verification process. The existing wetland was then latter added to establish the current flow regime throughout the study area. The specific RORB modelling parameters adopted are discussed further in the Appendix A.

As expected the urban catchments showed lower (shorter) critical durations (15-25 mins) while the rural catchments showed longer catchment response times (in the order of 2-3 hours). Overall the study area had a critical duration of 6 hours which reflects the greater percentage of the catchment being rural (under exiting conditions).

Table 5-2 Calibrated Flows (1% AEP)

Interstation area	Q (Rational)	Q (RORB)	Diff	Crit. Duration
Internal Study Area	4.73	4.73	0.00	4.5h
RED	23.47	23.57	-0.11	15m
BLUE	19.39	19.42	-0.03	25m
L_BT	4.80	4.77	0.03	15m
GRN	3.76	3.74	0.02	15m
YLW	4.07	4.02	0.05	3h
BRN	1.07	1.07	0.00	2h

5.1.4 Existing Conditions Model Results Including the Effect of WR02/3

To accurately describe the current drainage conditions within the study area the verified RORB model was augmented to include the existing wetland retarding basin feature (WR 02/3) discussed in section 3.3. Given the complex nature of the feature is was decided to analyse it within a hydraulic model to establish an appropriate inflow/outflow relationship. Further detail of how this basin was modelled in RORB is provided in Appendix A. The relationship established in the hydraulic model was then fed back in to the RORB model, with the resultant effect on downstream flows is shown in Table 5-3.

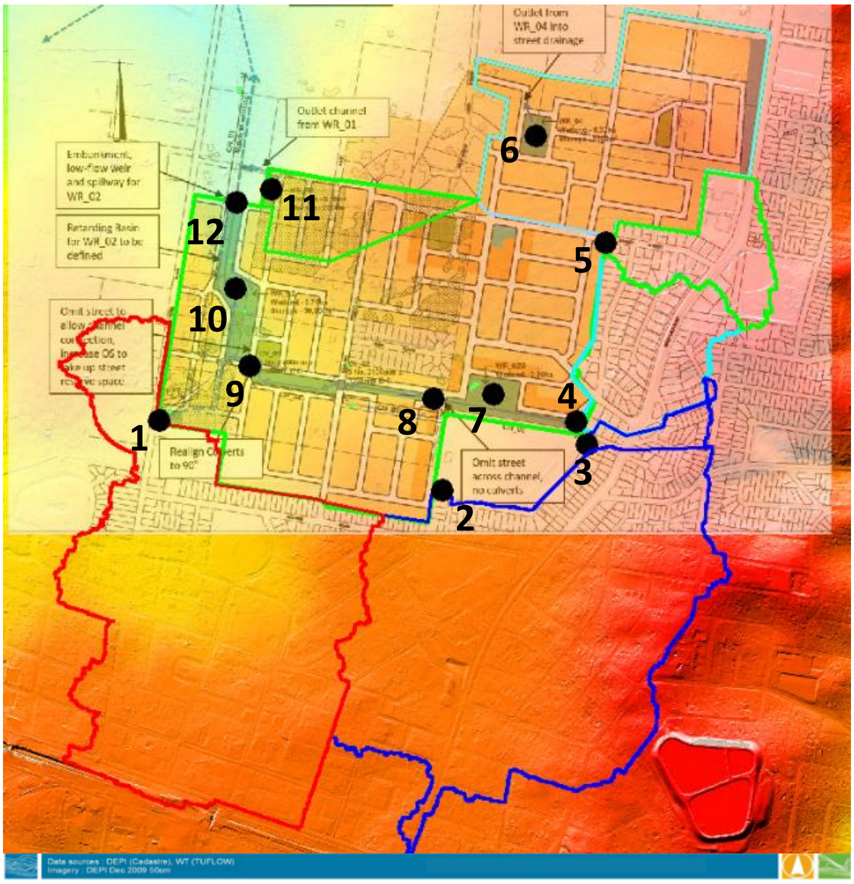


Figure 5-2 Key flows determined from RORB modelling

Table 5-3 Existing Conditions - Key peak flows throughout the study area (1% AEP)

Ref #	Location	Q 1% AEP Flow without WR02/3 (m^3/s)	Q 1% AEP Flow with WR02/3 (m^3/s)
1	External Catchment Inflow SW	16.06	no change
2	External Catchment Inflow S	18.89	no change
3	External Catchment Inflow SE	1.06	no change
4	External Catchment Inflow E1	4.77	no change
5	External Catchment Inflow E2	3.74	no change
6	WR04 Inflow	0.82	no change
7	WR02/3 Inflow	4.62	no change
8	Drainage Reserve flow 1	16.92	14.80
9	Drainage Reserve flow 2	15.02	13.35
10	WR02 Inflow	17.17	16.33
11	WR03 Inflow	0.51	no change
12	DCP Outlet flow	17.45	16.61

Peak flow attenuation from WR02/3 is observed immediately downstream (e.g. at drainage reserve flow 1), however this effect is quickly lost by the influence of external catchment flows in the system downstream of English Street.

5.1.5 Developed RORB Model

Fraction impervious conditions, overland flow routing and reach characteristics were modified from the existing conditions RORB model to reflect proposed changes within the subject site. Changes in fraction impervious conditions were restricted to the DCP area and based on the proposed developed concept drainage layout discussed in section 4.1 and shown in Figure 4-1. Further detail on the developed conditions modelling is discussed in the Appendix A of this report.

Modelling of developed (unmitigated conditions) showed that internal flows within the DCP area would effectively double without attenuation to reduce the increased runoff. Table 5-4 documents the changes in peak flows resulting from the increased impervious area. Flows quoted can be referenced to Figure 5-2. Figure 5-3 shows resultant hydrographs extracted from RORB at the DCP outlet.

Table 5-4 Change in peak flow resulting from the proposed development (1% AEP)

	Location	Developed 1% AEP Flow inc. WR02/3 (m^3/s)	Existing 1% AEP Flow with WR02/3 (m^3/s)
6	WR04 Inflow	7.5	0.4
10	WR02 Inflow	31.1	7.8
11	WR03 Inflow	2.1	0.2
12	DCP Outlet flow	31.5	16.61

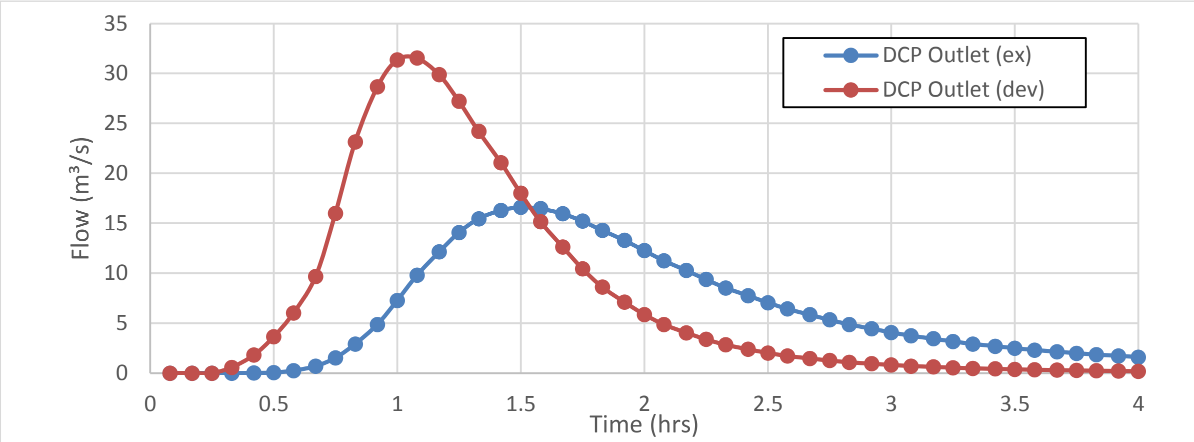


Figure 5-3 Peak 1% AEP Hydrographs at the DCP Outlet – Existing & Developed Conditions

5.1.6 Mitigated RORB Model

Without attenuation, flows from the proposed development have detrimental effects on the land and stakeholders downstream of the work as such attenuation is required to manage flows. Developed conditions RORB models were augmented to include 3 new retarding basin features inside the study area area. Locations of these basins were broadly consistent with those identified by CPG in past strategies and discussed in the proposed developed concept drainage layout (section 4.1 and shown in Figure 4-1). Initial hydrologic mitigated modelling (the topic of this report) was focused on meeting best practice requirements for the DCP. Given the .available land within the DCP drainage reserves it was not considered viable to pick up additional storage to manage downstream capacity constraints (i.e. Latrobe road crossing capacity). Similarly initial comments from Council suggested this process should be undertaken in consultation with VicRoads, consequently it was not investigated further in this report.

Conceptual basins were modelled where possible as shallow basins (~1-1.5 m deep, and 1 in 6 batters) to ensure they can be integrated into the open space within the ODP with minimal impact on the spaces functionality and liveability. The exception to this is WR02 were the entire open space is required to be modified to achieve the required storage volume. Significant excavation at this location will be required to achieve the desired storage (~2 m deep basin). Flood storage requirements are shown in Table 5-5. Outlet arrangements used in these scenarios are shown in Table 5-6.

Table 5-5 Flood Storage Sizing and Storage Performance in the 1% AEP Event

Basin name (CPG)	Flood Storage (m^3)	RORB Flow 1% AEP event (m^3/s)			Difference (m^3/s)
		Existing Conditions	Developed Conditions	Mitigated Conditions	
WR02	40,100	17.17	31.14	16.27	-0.90
WR03	1,730	0.51	2.15	0.49	-0.02
WR04	11,000	0.82	7.49	0.79	-0.03

Table 5-6 Proposed Outlet Arrangements

Basin name (CPG)	Basin Depth (m)	Outlet pipe Size (m) RCP	Number of pipes	Invert of pipe(s) (m AHD)	Weir width (m)	Weir Invert (m AHD)
WR02	1.97	1.05	6	62	80	63.98
WR03	1	0.375	2	65	20	65.98
WR04	1.5	0.375	1	78.5	40	79.98
		0.450	1	78.5		

5.1.7 Discussion – Recommended Basin Volumes

It is noted that basin volumes determined in this study (particularly for WR02 and WR04) are significantly larger than those recommended in the CPG investigation 5 years prior. Without having the actual RORB models to review it is very difficult to ascertain why there is a significant difference in required volumes.

Checking Water Technology's basin volumes against basin "rule of thumb" checks (such as Boyd's method - 1989) it would appear that the storage volumes are appropriate.

Water Technology's retarding basins have been designed "on-line" which means that significant external catchment flows will be routed through the basins. This arrangement is a design constraint arising from the natural topography of the study area and the drainage reserve area available. This may be one factor which has influenced the differences in storage volume estimates.

6. WATER SENSITIVE URBAN DESIGN (WSUD) OPPORTUNITIES

A number of WSUD options are available for treating urban stormwater from the developed site. Constructed wetlands are large, man-made, significantly vegetated ponds that provide a natural way to reduce velocities, treat stormwater and remove sediment and contaminants before discharging stormwater downstream. Typically used for large developments, they can be incorporated into the base of floodplain storage areas. Wetlands usually follow sedimentation ponds in the treatment train. The sediment ponds are detention systems which slow stormwater runoff and allow sediments to settle and deposit. These sediments can then be removed from the system on a periodic basis.

Treatment trains consisting of sediment ponds followed by wetlands are proposed for the Morwell North-West DCP area. These treatments are proposed to be coupled with grassed swales to convey and pre-treat stormwater (particularly for the "pink catchment" identified in Figure 4-1).

The treatment train components were modelled using the MUSIC (Model for Urban Stormwater Improvement Conceptualisation) modelling program. The predicted performance of the treatment train has been assessed against the targets described in the Urban Stormwater Best Practice Guidelines (CSIRO). These specify the removal of key pollutants as follows:

- 80% of total suspended sediments;
- 45% of total nitrogen;
- 45% total phosphorous; and,
- 70% gross pollutants.

6.1 MUSIC Modelling and Treatment Train Sizing

MUSIC model was established in line with the current Melbourne Water MUSIC Guidelines with the proposed WSUD features for the site. The model was run using local 6 minute rainfall data from 1961-1977.

In this instance the MUSIC model served two functions:

- To establish the performance of the existing wetland feature(s) found in the "Red Catchment" (with reference to Figure 4-1); and
- Size all other treatment features inside the DCP.

The catchment breakup for MUSIC was based on the proposed development layout (by CPG) as well as the current conditions within the study area, as shown in Figure 4-1. The layout of the MUSIC model is presented in Figure 6-1. Details for the existing wetland were established using GIS data (aerial imagery), survey by PGA group and site visit records. Further detail of the existing wetland configuration and the overall catchment physical characteristics.

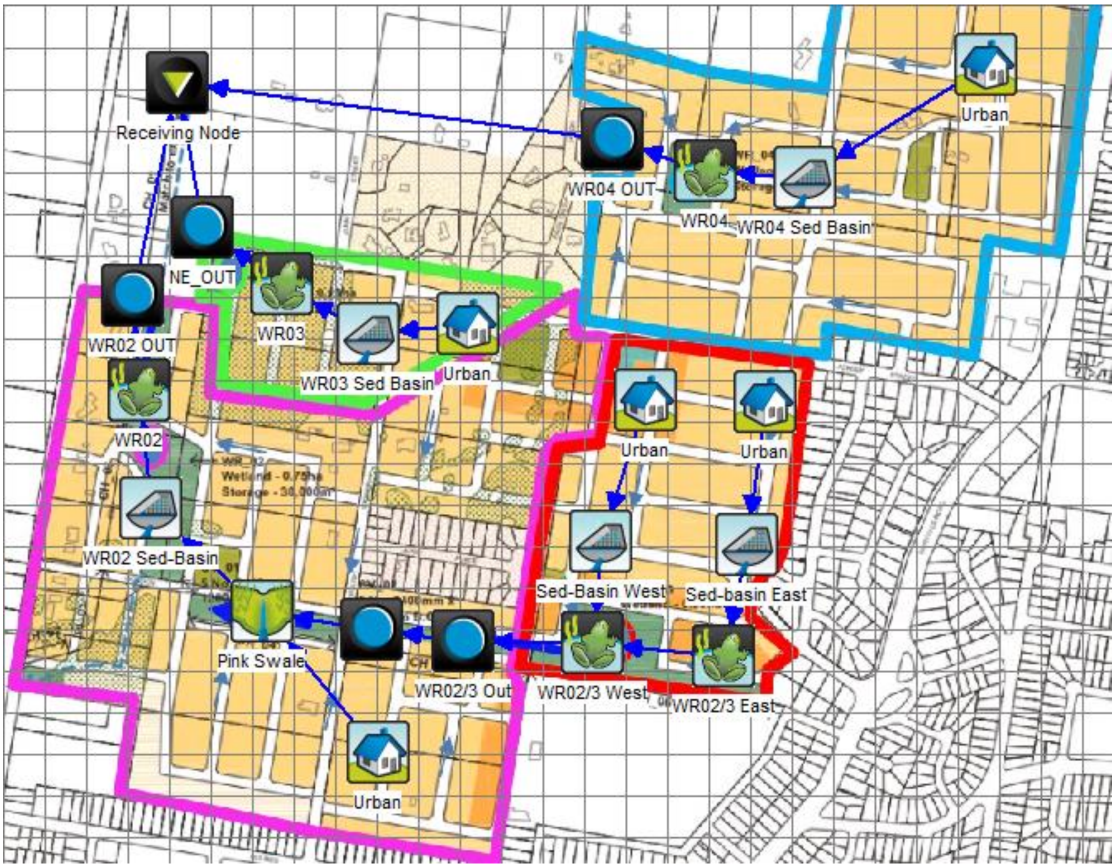


Figure 6-1 MUSIC Model Layout for Developed Conditions

Table 6-1 Catchment Details

Catchment	Total Area (Ha)	Fraction Impervious	Comments
Red (total catchment)	21.45	0.52	Based on CPG ODP
Red (East catchment)	13.94 (65%)		Contributing Catchments scaled on wetland surface area
Red (West catchment)	7.51 (35%)		Contributing Catchments scaled on wetland surface area
Pink	62.54	0.55	
Green	11.10	0.58	
Blue	38.54	0.58	

6.2 Performance of the Existing Wetland (WR02/3)

As discussed in section 3.3, the existing treatment system consists of two wetlands in series. No data on contributing catchments to each of the wetlands was available so it was assumed the contributing catchments were proportional to the surface area of each wetland, 34% of the total catchment was assumed to drain to the eastern wetland with the balance draining to the western wetland.

Sediment pond sizing was checked for the two major inflow points of WR02/3 wetland. Results of this analysis (shown in Table 6-2) suggest the sedimentation basins were appropriately sized for the contributing catchments.

Figure 6-2 Key Features in WR02/3 Wetland

Table 6-2 WR02/3 Sedimentation Basin Details

Details	East Sed-Basin	West Sed-Basin
Surface Area	272 m^2	531 m^2
Extended Detention Depth*	0.5 m	0.5 m
Permanent Pool Depth*	1 m	1 m
Permanent Pool Volume*	136 m^3	265 m^3
Percentage of Suspended Solids Removal	98%	98%
Contributing Urban Catchment Area	7.5 Ha	13.9 Ha
Clean Out Frequency	9.1 years	9.5 years

*assumed conditions not based on survey or as constructed drawings

Wetland treatment train effectiveness was tested in the MUSIC model (results shown in Table 6-4). It was found that the existing wetlands treated the local catchments (recently/or currently being developed) to above best practice requirements. This result reduced some of the land take pressure on the downstream feature sizes (i.e. WR02).

Table 6-3 WR02/3 Wetland Details

Design Element	East Wetland	West Wetland
Area	1870 m^2	3506 m^2
Extended detention depth*	0.5 m	0.5 m
Permanent pool depth*	Varies (Average depth = 0.5 m)	Varies (Average depth = 0.5 m)
Permanent pool volume*	935 m^3	1735 m^3

*assumed conditions not based on survey or as constructed drawings

Table 6-4 Existing Wetland (WR02/3) Treatment Train MUSIC Model Results

Pollutant	% Reduction East Wetland	% Reduction West Wetland	% Reduction Target
Total Suspended Solids (kg/yr)	88	87	80
Total Phosphorus (kg/yr)	76	75	45
Total Nitrogen (kg/yr)	55	53	45
Gross Pollutants (kg/yr)	98	99	70

6.3 Proposed Concept Design

Treatment trains consisting of a sediment pond followed by a wetland were proposed for each of the 4 catchments. The sediment ponds were designed to achieve a 95% removal rate of 125 µm diameter particles in the peak 1 year ARI design flow. In order to protect the integrity of the macrophyte zone, the wetlands were designed for the 3 month ARI design flows. In addition (were possible), grassed swales were considered to convey and pre-treat stormwater. A graphical image of the DCP concept design is shown in Figure 6-3.

6.3.1 WSUD for the Pink Catchment (WR02)

Two swales features combined with an end of the line sedimentation basin / wetland system is proposed to treat stormwater generated in the Pink catchment. Details of the swale conditions are shown in Table 6-5. It is noted that further design work will be required at the functional design stage to demonstrate velocity requirements for stable vegetation can be achieved in these features.

Table 6-5 Grassed Swale Details

Details	Swale
Cumulative Length	580 m
Representative Bed Slope	1 %
Base Width	2 m
Top Width	12 m
Vegetation Height (grass)	0.25 m

Details of the designed sedimentation basin are shown in Table 6-6 and further calculations are provided in Appendix B.

Table 6-6 WR02 Sedimentation Basin Details

Details	Sedimentation Basin
Surface Area	2,050 m^2
Extended Detention Depth	0.5 m
Permanent Pool Depth	1.0 m
Permanent Pool Volume	1,640 m^3
Percentage of Suspended Solids Removal	95%
Contributing Urban Catchment Area	62.5 Ha
Clean Out Frequency	8.2 years

A wetland of 5,000 m^2 following the sediment pond was designed to treat the runoff generated within the new development to best practice levels. The treatment performance is shown in Table 6-7.

Table 6-7 WR02 Wetland Details

Design Element	Details
Area	5,000 m^2
Extended detention depth	0.5 m
Permanent pool depth	Varies (Average depth = 0.5 m)
Permanent pool volume	2,500 m^3

Table 6-8 shows the MUSIC treatment results achieved by the system consisting of a swale 580 m long, a 2,050 m^2 sediment pond and a 5,000 m^2 wetland. It is noted that this system is designed downstream of WR02/3 which over treated its local catchment this has led to a smaller wetland surface area requirement for the pink catchment.

Table 6-8 Pink Catchment Treatment Train MUSIC Model Results

Pollutant	% Reduction Pink Catchment	% Reduction Target
Total Suspended Solids (kg/yr)	91	80
Total Phosphorus (kg/yr)	75	45
Total Nitrogen (kg/yr)	45	45
Gross Pollutants (kg/yr)	100	70

6.3.2 WSUD for the Green Catchment (WR03)

An end of the line system consisting of a sedimentation pond and wetland is proposed to treat stormwater generated in the green catchment.

Details of the designed sedimentation basin are shown in Table 6-9 and further calculations are provided in Appendix B.

Table 6-9 WR03 Sedimentation Basin Details

Details	Sedimentation Basin
Surface Area	430 m^2
Extended Detention Depth	0.5 m
Permanent Pool Depth	1.0 m
Permanent Pool Volume	215 m^3
Percentage of Suspended Solids Removal	95%
Contributing Urban Catchment Area	11.1 Ha
Clean Out Frequency	9.7 years

A wetland of 1,700 m^2 (see Table 6-10) following the sediment pond was designed to treat the runoff generated within the new development to best practice levels. The treatment performance is shown in Table 6-11.

Table 6-10 WR03 Wetland Details

Design Element	Details
Area	1,700 m^2
Extended detention depth	0.5 m
Permanent pool depth	Varies (Average depth = 0.5 m)
Permanent pool volume	1,350m^3

Table 6-11 shows the MUSIC treatment results achieved by the system consisting of a 430 m^2 sediment pond and a wetland with a surface area of 1,700 m^2.

Table 6-11 Green Catchment Treatment Train MUSIC Model Results

	% Reduction Green Catchment	% Reduction Target
Total Suspended Solids (kg/yr)	84	80
Total Phosphorus (kg/yr)	72	45
Total Nitrogen (kg/yr)	46	45
Gross Pollutants (kg/yr)	98	70

6.3.3 WSUD for the Blue Catchment (WR04)

An end of the line system consisting of a sedimentation pond and wetland is proposed to treat stormwater generated in the blue catchment.

Details of the designed sedimentation basin are shown in Table 6-12 and further calculations are provided in Appendix B.

Table 6-12 WR04 Sedimentation Basin Details

Details	Sedimentation Basin
Surface Area	1,300 m^2
Extended Detention Depth	0.5 m
Permanent Pool Depth	1.0 m
Permanent Pool Volume	650 m^3
Percentage of Suspended Solids Removal	95%
Contributing Urban Catchment Area	38.54 Ha
Clean Out Frequency	8.4 years

A wetland of 4,500 m^2 (see Table 6-13) following the sediment pond was designed to treat the runoff generated within the new development to best practice levels. The treatment performance is shown in Table 6-14.

Table 6-13 WR03 Wetland Details

Design Element	Details
Area	4,500 m^2
Extended detention depth	0.5 m
Permanent pool depth	Varies (Average depth = 0.5 m)
Permanent pool volume	2,250 m^3

Table 6-14 shows the MUSIC treatment results achieved by the system consisting of a 1,300 m^2 sediment pond and a wetland with a surface area of 4,500 m^2 are adopted. It is noted that while the blue catchments contributing area is significantly less than the pink catchment, the wetland sizes are quite similar. This is because the pink catchment had some additional treatment from the swale features and also had a wetland upstream which was over treating its local catchment flows.

Table 6-14 Blue Catchment Treatment Train MUSIC Model Results

	% Reduction Blue Catchment	% Reduction Target
Total Suspended Solids (kg/yr)	86	80
Total Phosphorus (kg/yr)	72	45
Total Nitrogen (kg/yr)	45	45
Gross Pollutants (kg/yr)	98	70

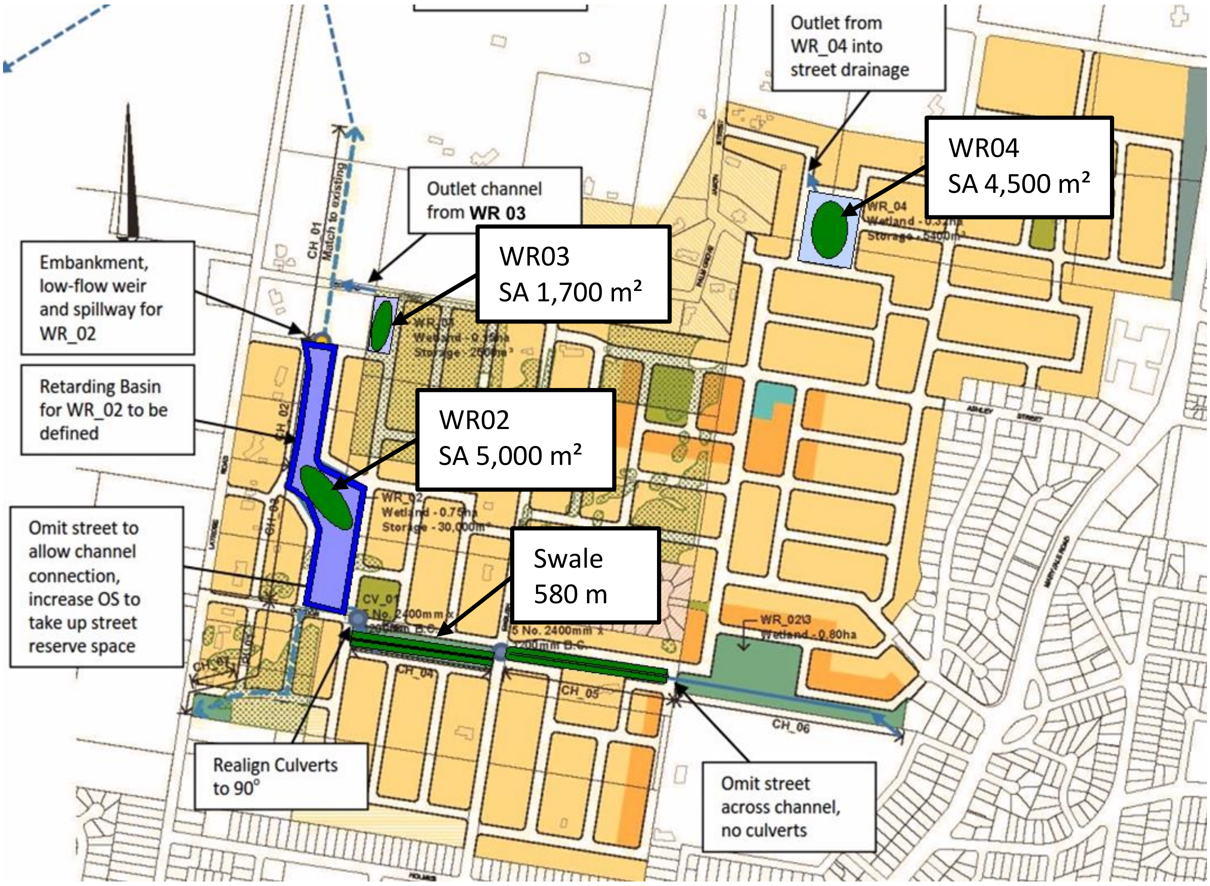


Figure 6-3 Proposed Concept Design for the Development

6.3.4 Options to Treat External Catchments

Available land within the current OPD leaves little space for additional water quality treatment of external catchments. Significant land is available immediately downstream of WR02 and WR03. If this land was converted into a large regional wetland treatment requirements for the upstream features could be moved downstream and incorporated into a larger system. As this land is already marginal in its current and potential uses (from both agricultural and development perspectives) there is some perceived benefit in pursuing this option further.

6.3.5 Maintenance Access

It is noted that the systems will need to be designed with room to facilitate future maintenance access tracks between the WSUD assets. The entry/access points to the tracks, access ramps into the sediment pond and vehicle turning circles should be considered in more detail during detailed design.

6.3.6 Considerations for Function Design

During functional design of the sediment pond and wetland systems, the following components should be considered:

- Total area including batters and freeboard;
- Access for maintenance;
- Flow velocity checks in the sediment ponds and macrophyte zones
- Sediment drying areas within the reserve;
- Sediment pond and wetland outlet structures;
- Potential for wetland bypass systems;
- Wetland macrophyte zone bathymetry.

7. HYDRAULIC MODELLING

7.1 Flood Model Setup

A TUFLOW model was created using LiDAR and survey data of the existing wetlands and drainage reserve area collected by PGA group. Details of the flood model setup are provided in Appendix C.

7.2 Existing Flood Conditions

The existing conditions flood model (including the WR02/3 wetland and surrounding development) was run for several design 1% AEP event durations (1h, 3h and 6h) to see the system responses to different rainfall conditions. From a design perspective, it was considered important to combine learnings from the hydrologic analysis with the first pass hydraulic modelling prior to recommending the preliminary concept design to PGA group.

Existing conditions flood modelling shows that the land set aside for the drainage reserve in the DCP currently conveys a significant amount of stormwater away from the township. Maintaining this function post development will be key design requirement.

Under existing conditions the flood modelling showed peak flood levels inside WR02/3 basin are equal to ~73.2 m AHD within the east (upstream) basin and 72.6 m AHD within the west (downstream) basin, which equate to peak levels between 0.8-1 m above the wetland normal water levels. All stormwater was retained in the features (no over topping) suggesting they are appropriately sized. The outlet on the east (upstream) basin (wetland) does not have sufficient capacity to convey the 1% AEP flow with some stormwater flowing over the bridge approaches on the embankment between the two wetlands.

Downstream from WR02/3 flow is split into two distinct flow paths towards English Street. Some flooding within the new subdivision on English Street is observed however no dwellings are expected to experience above floor flooding. After flows weir over English Street the flow is again split into two paths, before joining with external urban catchment flows from the CBD and flowing northwards. The main flow path north (following Latrobe Road) ranges in width from 50 m to approximately 150 m. Flows in this region are generally shallow (~300 mm).

Once flows reach Latrobe Road the flood depths are quite deep (~600 mm) with water ponding up behind Latrobe Road. West of Latrobe Road floodwater was exits the TUFLOW model using a Height (H) verse Flow (Q) boundary, the slope of this boundary matches the land form in the LiDAR. It is noted that the LiDAR was flown prior to the Morwell River diversion being completed so these conditions are not in the current model set up.

Figure 7-1 shows the preliminary flood depth results from the 1% AEP 3 hour flood within the study area. Figure 7-2 shows the overall flood extent and peak flows extracted from the flood modelling (for the 1% AEP 3 hour event).

7.3 Observed Downstream Capacity Constraints and Implications for this DCP

Approximately 6.3 km² of both urban and rural land drains to the Latrobe Street crossing. Currently overland flow from this catchment is conveyed west under Latrobe Road (towards the Morwell River diversion system) via 3 x Φ1.2 m diameter reinforced concrete pipes (RCP). Not surprisingly even when these pipes are working effectively (as modelled in this work, but not currently the case) they do not have sufficient capacity to convey the 1% AEP flow.

Currently Latrobe Road acts as a pseudo retarding basin with stormwater ponding up behind the road. In a 1% AEP event approximately 6 m^3/s is conveyed through the RCP's with the balance (~14 m^3/s) overtopping the road just south of the culvert crossing.

Under developed conditions significantly more flood volume will be conveyed to this location, it is anticipated that insufficient space exists within the current DCP drainage reserve (without significant modification) to attenuate flow back to a state were Latrobe Road is not compromised by flood waters in a 1% AEP event. However sufficient land does exist downstream of the DCP to meet this requirement (should council wish to pursue this option).

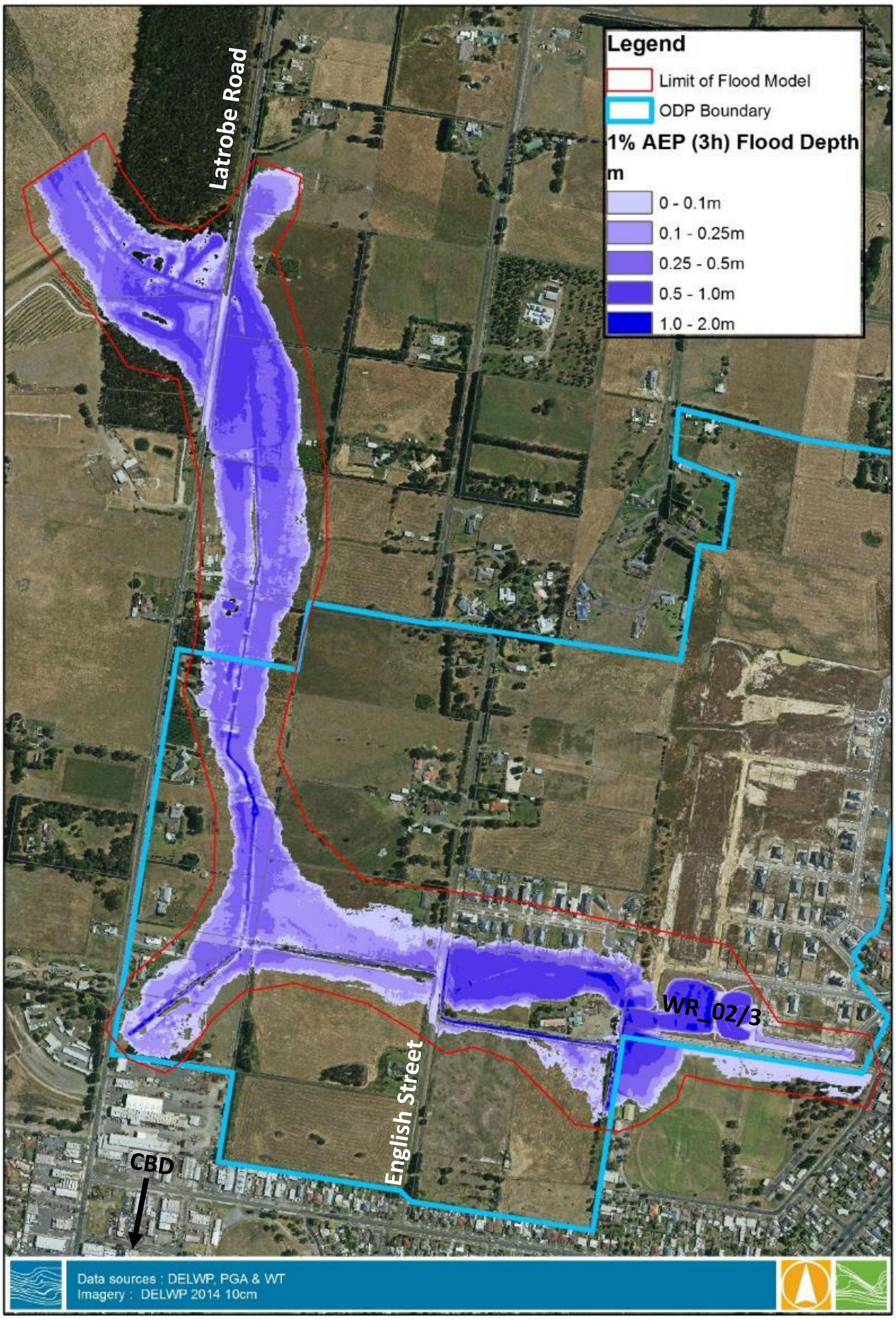
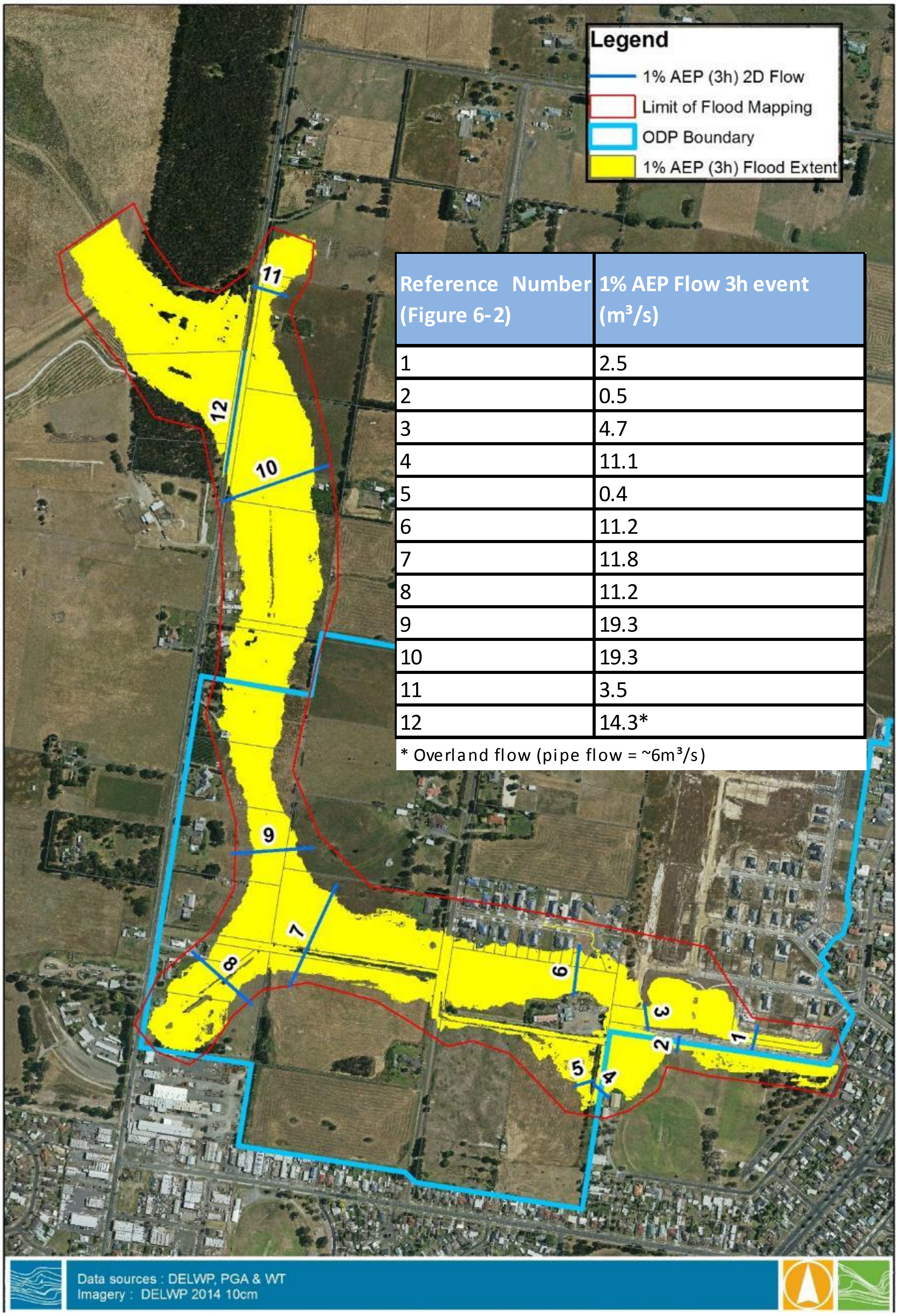


Figure 7-1 1% AEP (3hour) Flood Depth results – Study Area



Reference Number (Figure 6-2)	1% AEP Flow 3h event (m^3/s)
1	2.5
2	0.5
3	4.7
4	11.1
5	0.4
6	11.2
7	11.8
8	11.2
9	19.3
10	19.3
11	3.5
12	14.3*

* Overland flow (pipe flow = ~$6m^3/s$)

Figure 7-2 1% AEP (3hour) Flood Extent & 2D flow results – Study Area

7.4 Future (developed) Conditions

The existing conditions flood model (including the WR02/3 wetland and surrounding development) was modified to include the proposed developed conditions within the DCP. They included:

- Modification of the land form in the north west corner of Maryvale Recreation Reserve to more efficiently connect existing township flows to the new drainage reserve;
- Introduction of a new drainage reserve immediately downstream of WR02/3 this consisted of:
    - An open channel (swale) from downstream of WR02/3 to English Street;
    - A culvert crossing under English Street;
    - An open channel (swale) from English Street to were the natural drainage line turns north (inflow into WR02 basin)
    - A culvert crossing connecting this channel to the main retarding basin in the system (WR02) and
    - A new retarding basin (WR02) stretching to the downstream end of the DCP study area.
- Increases in catchment flows from the developed land surrounding the drainage reserve (catchments draining to WR02);

Developed conditions modelling was focused on the critical duration (3h) 1% AEP event. A preliminary WR02 basin design was provided by PGA group for testing in the hydraulic model. Using this basin design the upstream swales and crossings were sized. Channel conditions were designed in 12d civil design software using 1 in 6 side slopes on the channel batters.

Channel depths were based on the drainage reserve area available and the inverts of the proposed basin (WR02) and the existing upstream basin (WR02/3). It is noted that currently the upstream basin (WR02/3) is not functional, with the most downstream outlet (weir) drowned out. The developed conditions system has considered this issue and looked to rectify it through creating more fall.

To make the system functional (meeting existing downstream peak flow requirement) additional storage had to be picked up in the channel upstream of WR02 (stage storage relationship shown in Table 7-1). A peak flood depth plot is show in Figure 7-3.

Table 7-1 Recommended Stage / Storage for swale between English St & WR02

Stage (mAHD)	Storage (m^3)	Stage (mAHD)	Storage (m^3)
67.00	13801	65.4	2836
66.95*	13254	65.2	2131
66.8	11741	65	1550
66.6	9923	64.8	1081
66.4	8309	64.6	714
66.2	6902	64.4	438
66	5700	64.2	240
65.8	4630	64	109
65.6	3674	63.8	34

* Approximate peak water level in the swale feature

Implementing the basin and channel system showed net benefits for the system, with flood levels reduced downstream of the DCP boundary (shown in Figure 7-4), it also made WR02/3 basin more functional removing the drowned out downstream weir problem. Flooding of the existing properties on the eastern side of English street was also removed.

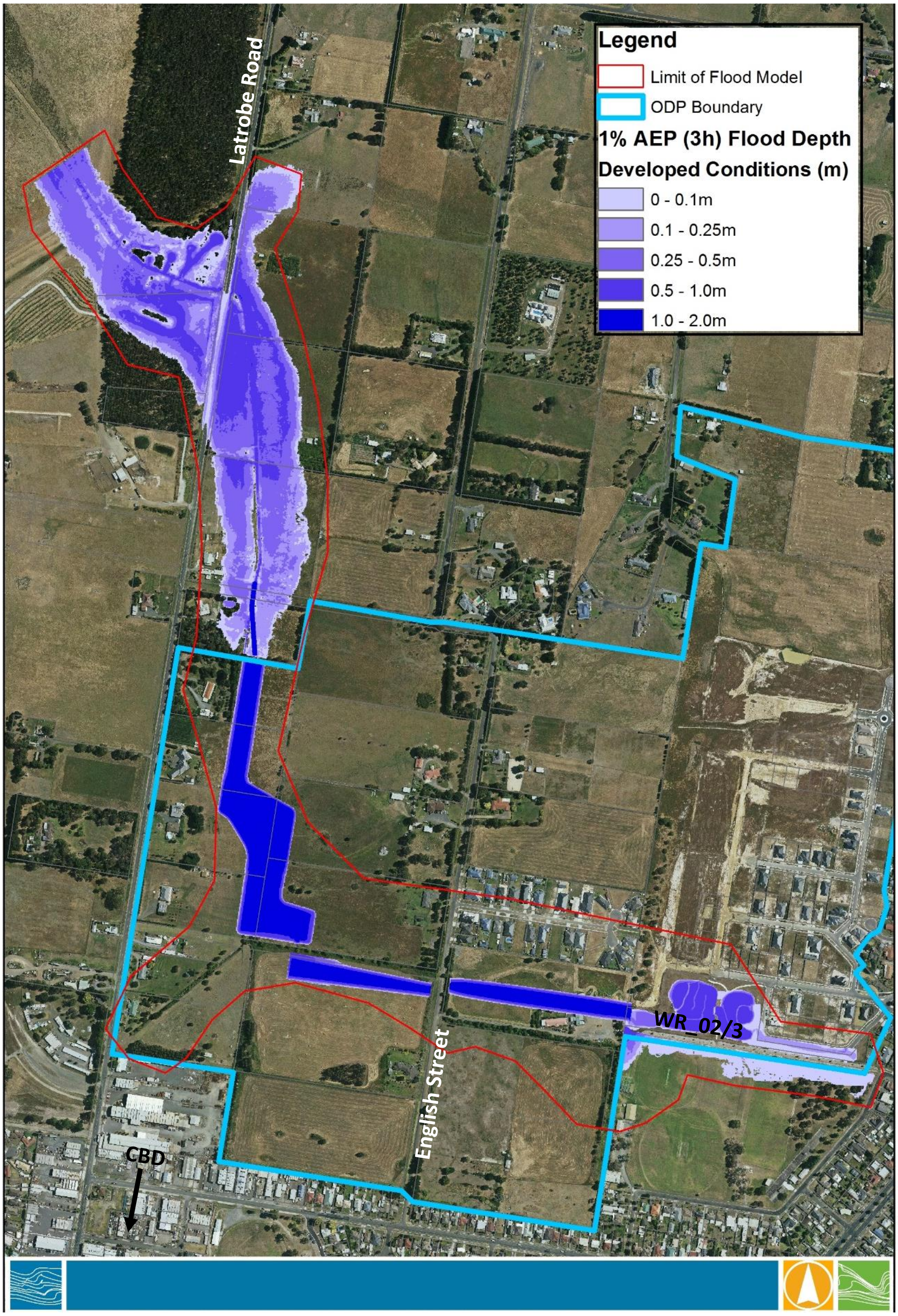


Figure 7-3 1% AEP (3hour) Flood Depth Results – Study Area

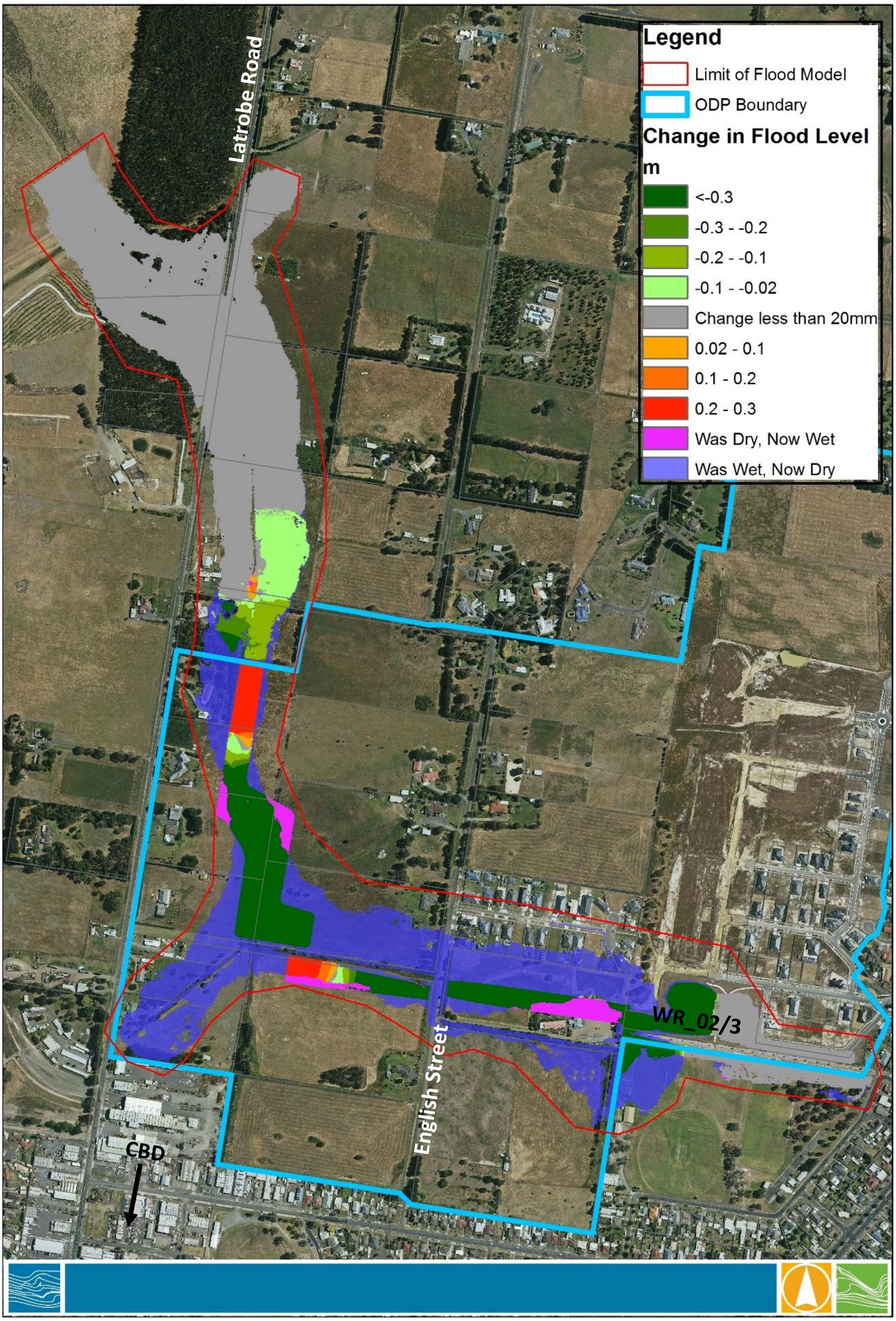


Figure 7-4 1% AEP (3hour) Flood Afflux Results – Study Area

8. CONCEPT DESIGN

Hydrological analysis in this project has established existing and developed flows through the DCP region and greater study area. It has also determined storage requirements (attenuation) to meet best practice targets (Table 8-1). Analysis of site flows from both the hydrology and hydraulic modelling were used to iteratively design a system that met best practice design criteria. The concept design focused on the WR02 drainage system.

Table 8-1 Flood Storage Sizing and Outlet Arrangements

Basin name (CPG)	Flood Storage (m^3)	Basin Depth (m)	Outlet pipe Size (m) RCP	Number of pipes	Invert of pipe(s) (m AHD)	Weir width (m)	Weir Invert (m AHD)
WR02	40100	2.0	1.05	6	62.0	80	63.98

Table 8-2 Preliminary Channelised Conveyance Recommendations (with reference to Figure 8-1)

Section	Depth (m)	Top width (m)	Bottom width (m)	Batter (1 in X)	Assumed Longitudinal slope (m/m)	Approximate Design flow (m^3/s)
1	2	30	6	6	0.009	17.2
2	2.5-2.7	40-50	6	6	0.007	17.7
3*	0.8	8	0.5	6	0.005	2 - 5 (50%-20% AEP flow)

* Low channel feature would sit within the retarding basin routing stormwater to the wetland

Table 8-3 Preliminary Crossing Recommendations (with reference to Figure 8-1).

Crossing	Type	Width (m)	Height (m)	Barrels	Design flow (m^3/s)
1	Box Culvert	2.4	1.2	5	17.2
2	Box Culvert	2.4	1.2	1	15.9
	Box Culvert	1.2	1.2	1	

Figure 8-1to Figure 8-5 show key details of the recommended concept designs. It is anticipated that at the functional design phase of the study some minor design changes will be required to ensure the drainage features fit within the allocated drainage reserve area. This is most evident in the swale between English Street and WR02 basin (section 2) were it is anticipated that some filling (up to 67.25 m AHD – as shown in Figure 8-5) will be required to contain the design flows to the drainage reserve. In addition to this swale batter slopes of 1 in 5 may also offer some reduction in land take for these features.

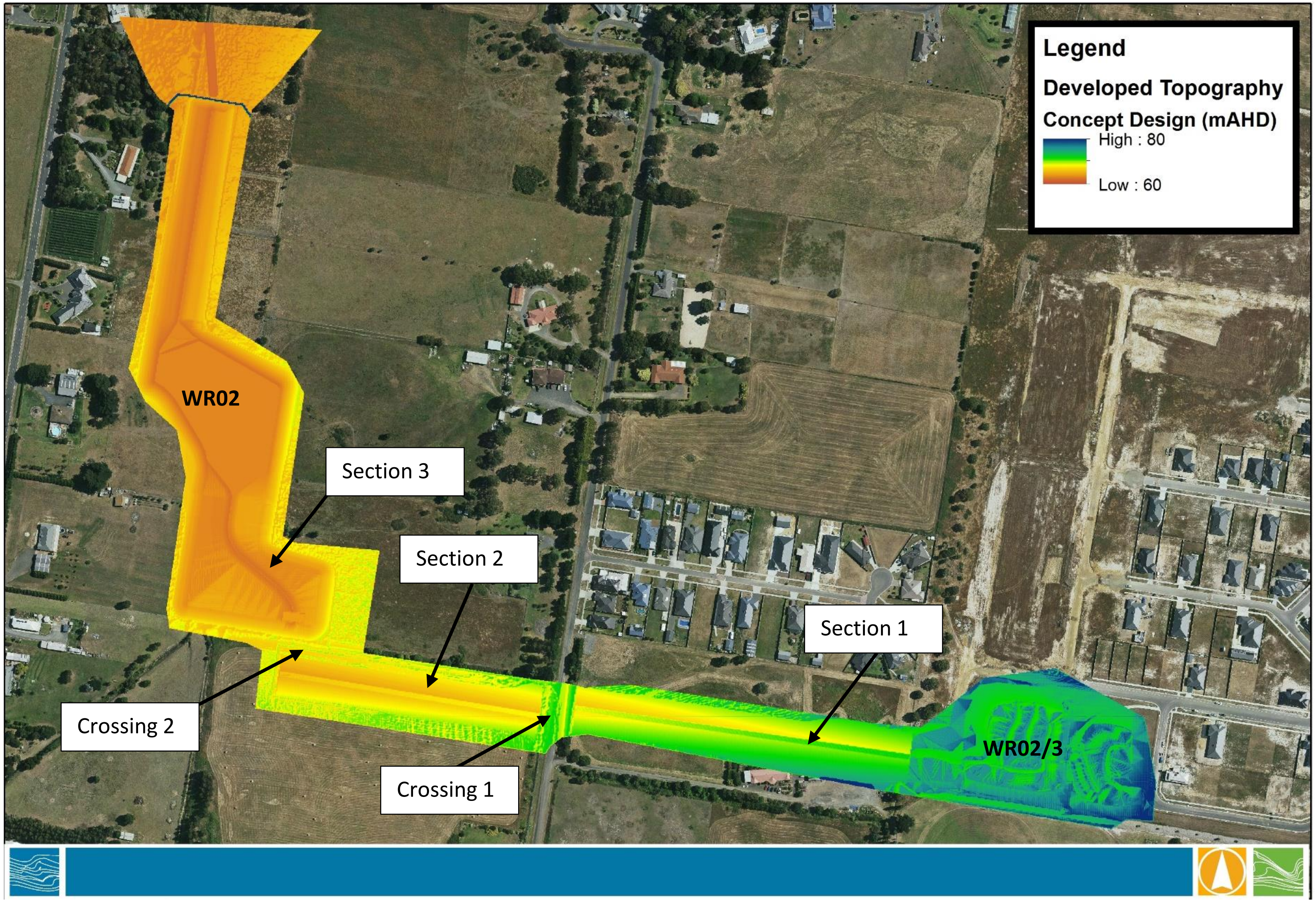


Figure 8-1 Key locations in preliminary concept design

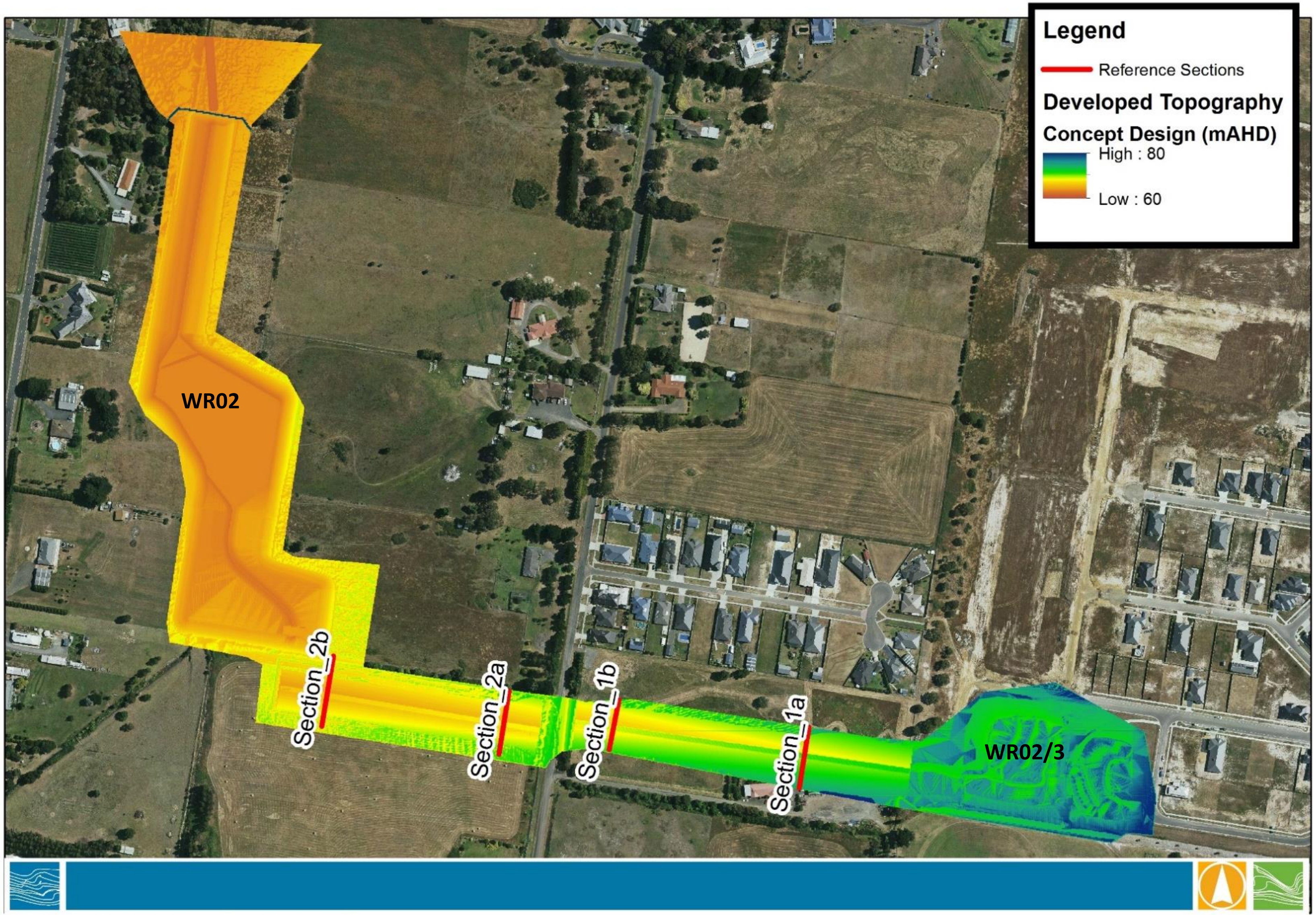


Figure 8-2 Section Location

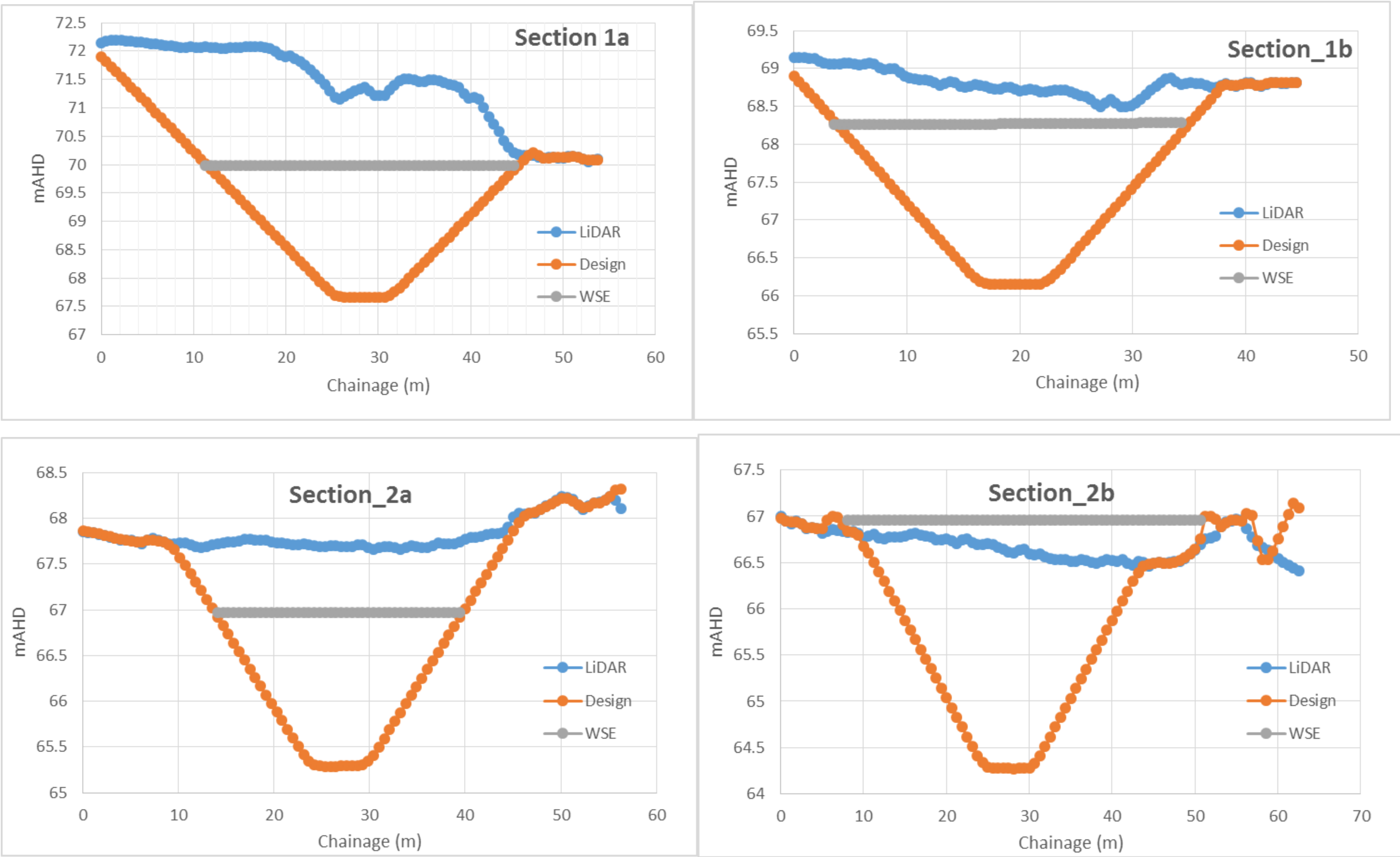
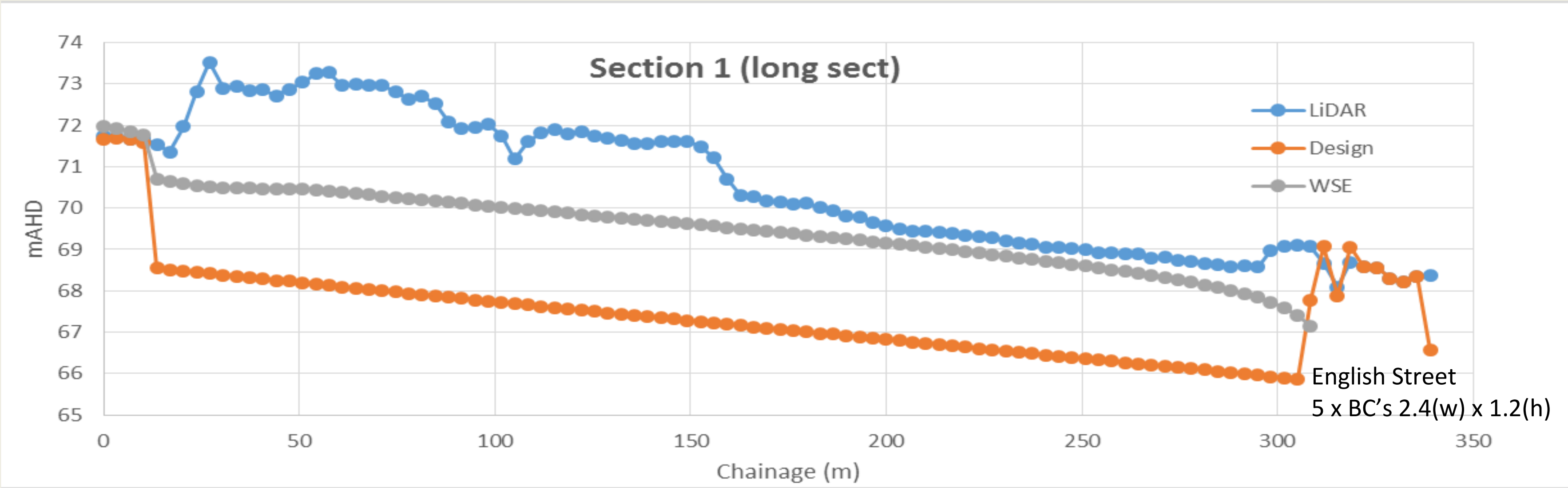


Figure 8-3 Indicative Cross Sections from Concept Design (with reference to Figure 8-2) Error! Reference source not found.



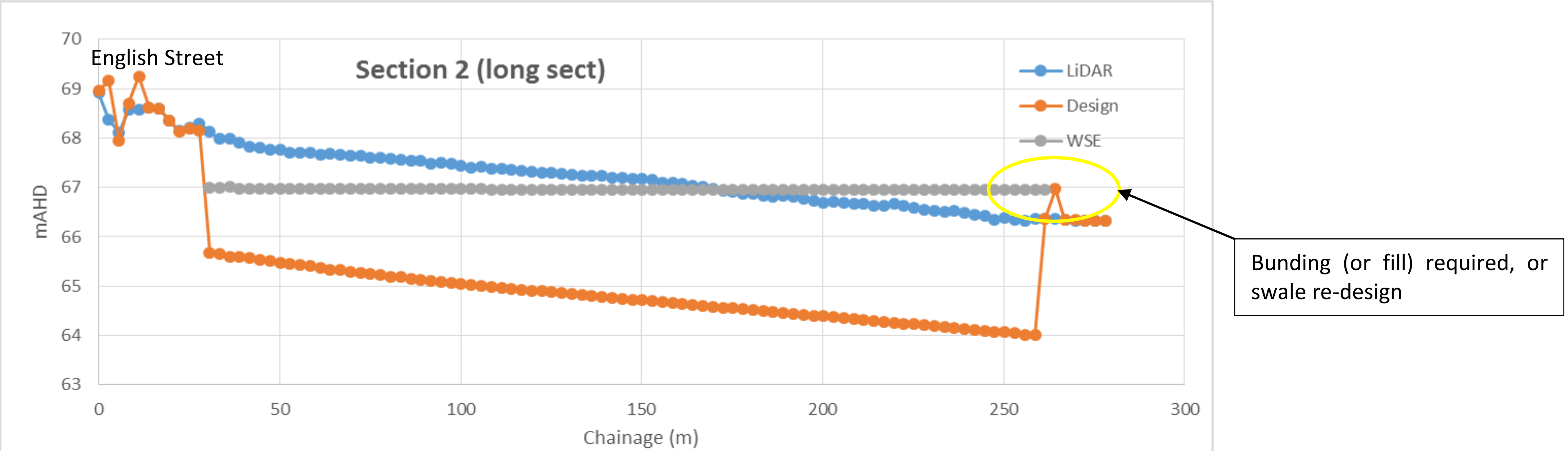


Figure 8-4 **Indicative Long Sections from Concept Design**

Figure 8-5 Swale Concept Design Vs Latrobe City Council Drainage Reserve Area

9. RESPONSE TO LATROBE CITY COUNCIL COMMENTS

Several comments were provided by Latrobe City Council Engineering staff. This feedback has been incorporated into the analysis and concept design. Direct responses to each comment have been provided below. Water Technology welcomes the opportunity to discuss these matters further with council staff if deemed appropriate.

Ray Bright - Team Leader Development – Latrobe City Council – November 2015

'There is an understanding from the original development plan that the stormwater discharge downstream of the development area would not increase as a result of the MNWDP as there would be sufficient retardation constructed within the development plan area to provide for the increased discharge arising from the increased level of development, ie. no net increase downstream of the development area.

Water Technology Response:

Hydrological modelling (RORB), has been used to establish existing and developed flows throughout the study area. This analysis has included establishing the current peak flow at the DCP outlet 16.2m^3/s. A retarding basin has been designed using the hydrologic modelling software. This volume and outlet conditions have been checked using hydraulic modelling. The hydraulic modelling showed additional storage is required throughout the system to manage peak flows.

Afflux analysis shows that flood levels are reduced downstream of the DCP outlet.

Les Hilton - Coordinator Infrastructure Design- Latrobe City Council – November 2015.

It is recommended that PGA review Water Technology's design assumptions and the study findings The report suggests (table 4-1- Data Review and hydrology report) that part of catchment WR04 may "free drain" to the north. The effects of this need to be assessed further as we already have localised flooding in the Palm Grove – Jason St area.

Water Technology Response:

Project clarification provided by PGA group suggested that further design of WR04 is outside the scope of the current investigation with DCP works focused on the system consisting of WR02/3 and WR02. If further analysis of this catchment was undertaken this comment would be considered in the drainage strategy.

The Latrobe Road culvert currently does not meet Vicroads requirements for flood frequency protection. This needs to be discussed with Vicroads for possible augmentation irrespective of precinct development.

Water Technology Response:

Existing crossing capacity of Latrobe road is discussed within the body or the report. Water Technology would be happy to work with VicRoads and Latrobe City Council to identify what changes would be required to increase the level of service of this crossing.

Why are the "existing 1% AEP peak flows with WR02/3" not the same in table 5-3 and 5-4 and comparable with fig 6-2?

Water Technology Response:

Hydrologic modelling routes flow using general relationships between catchment area, fraction imperviousness and reach type, this does not consider (in detail) the physical form of the land. Hydraulic modelling uses flows from hydrologic analysis and estimates how the flow moves over the land. Often when flow is applied to actual topographic data the peak flow rates are different. Flood volumes, will be consistent across both approaches. Concept designs have been validated in the hydraulic modelling to ensure they will "stack up" in the real word.

10. CONCLUSIONS

The original project brief from PGA group identified several key objectives, they have been reproduced below with information showing how this study has achieved them:

10.1 Whether Existing Internal Construction to the East Includes a Storage Component to Wetlands;

Table 10-1 Attenuation Effect of WR02/3 (1% AEP)

Location	Q 1% AEP Flow without WR02/3 (m^3/s)	Q 1% AEP Flow with WR02/3 (m^3/s)
Drainage Reserve Upstream of English Street	16.92	14.80
Drainage Reserve Downstream of English Street	15.02	13.35
WR02 Inflow	17.17	16.33
DCP Outlet flow	17.45	16.61

Peak flow attenuation from WR02/3 is observed immediately downstream (e.g. at drainage reserve upstream of English Street), however this effect is quickly lost by the influence of external catchment flows in the system downstream of English Street.

10.2 The Required Channel Size of Each Channel Reach

Table 10-2 Preliminary Channelised Conveyance Recommendations

Section	Depth (m)	Top width (m)	Bottom width (m)	Batter (1 in X)	Assumed Longitudinal slope (m/m)	Approximate Design flow (m^3/s)
Drainage Reserve upstream of English Street	2	Up to 33	6	6	0.009	17.2
Drainage Reserve downstream of English Street	2.5-2.7	40-50	6	6	0.007	17.7
Low Flow channel inside WR02	0.8	8	0.5	6	0.005	2 - 5 (50%-20% AEP flow)

* Low channel feature would sit within the retarding basin routing stormwater to the wetland

10.3 Flood Levels along the Channel System;

Table 10-3 Flood Levels Within the Channel System

Section	Upstream Peak WSE (mAHD)	Downstream Peak WSE (mAHD)
Drainage Reserve upstream of English Street	70.8	67.3
Drainage Reserve downstream of English Street	66.95	
Low Flow channel inside WR02	64.33	

10.4 The Required Culvert Sizes;

Table 10-4 Flood Levels Within the Channel System

Crossing	Type	Height (m)	Width (m)	Barrels	Design flow (m^3/s)
English Street crossing	Box Culvert	1.2	2.4	5	17.2
Gordon Street crossing	Box Culvert	1.2	2.4	1	15.9
	Box Culvert	1.2	1.2	1	

It is noted that two additional crossing were originally scoped to be sized (Leonard Street and Latrobe Street upgrades, as these locations are outside the DCP they have not been included in the analysis.

10.5 The Required Storage Volume in the Lowest Channel Reach

The recommended drainage system includes storage in both the WR02 basin (lowest channel reach) of approximately 40,000 m^3 combined with an additional 13,000 m^3 in the reach between WR02 AND English Street.

10.6 Water Quality Sites and Target Areas

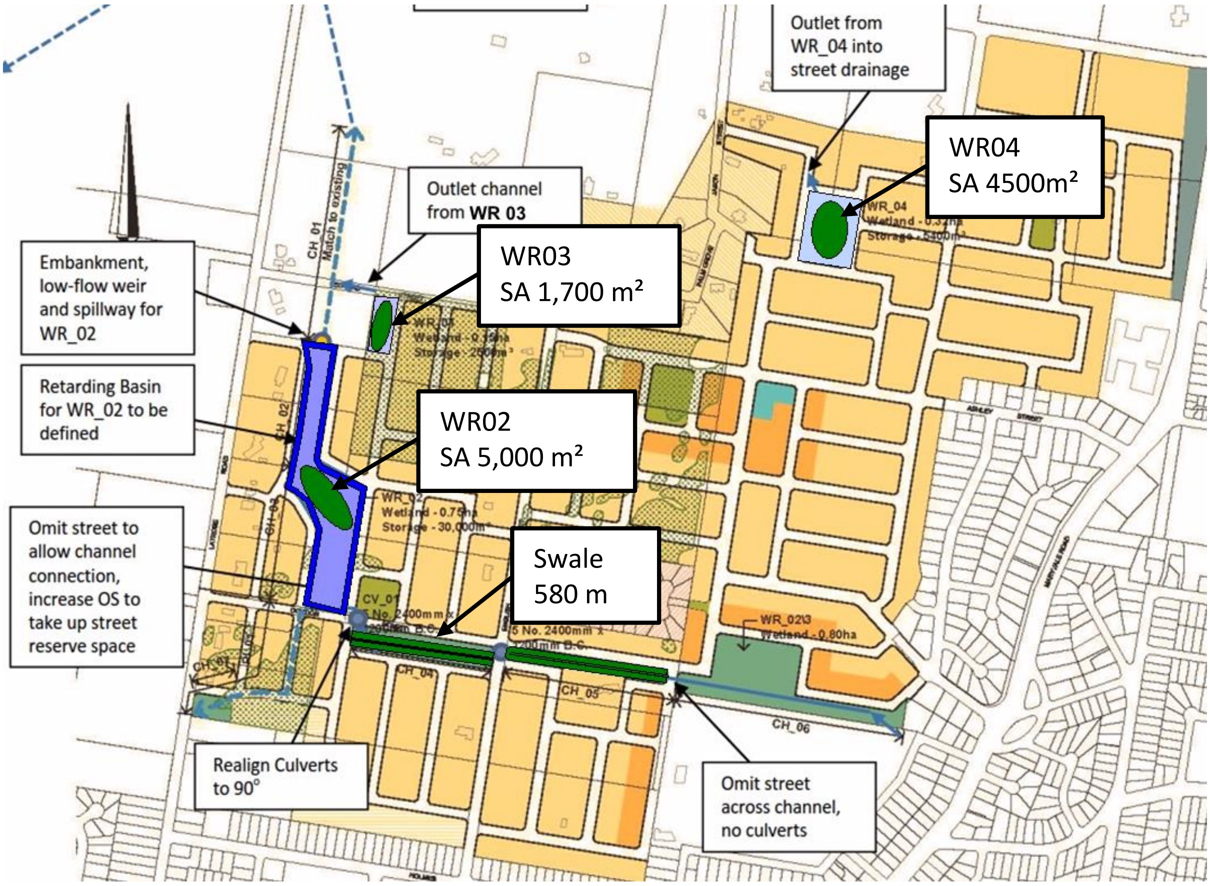


Figure 10-1 Proposed Wetland Sizing for the Development

10.7 Weir Sizing / Outfall Culvert Arrangements at the End of the Lower Storage

Table 10-5 Flood Storage Sizing and Outlet Arrangements

Basin name (CPG)	Flood Storage (m^3)	Basin Depth (m)	Outlet pipe Size (m) RCP	Number of pipes	Invert of pipe(s) (m AHD)	Weir width (m)	Weir Invert (m AHD)
WR02	40100	2.0	1.05	6	62.0	80	63.98

11. REFERENCES

CPG (via Coomes consulting) - June 2010. *Morwell North West Development Plan (Background Analysis: Final)*

CPG (October 2010) *Morwell North West Development Plan FINAL-*

CPG (March 2010) *Heritage Boulevard Site Stormwater Management Plan*

CGP (via Latrobe City Council) (2015), *Heritage Boulevard Landscape Master Plan*

Gold Coast City Council (2007). Water Sensitive Urban Design Guidelines. June. www.goldcoast.qld.gov.au/gcplanningscheme_policies/policy_11.html#guidelines.

Ian Barker Gardens from (May 2013), *Amended Endorsed Landscape Plans*.

Latrobe City Council (2015), *Land ownership map*

Latrobe City Council (2015), *Planning Permit 2011/116/B*

Melbourne Water (2011), *2D Modelling Guidelines for Melbourne Water*

Melbourne Water (2005). WSUD Engineering Procedures: Stormwater. Collingwood, Victoria: CSIRO Publishing.

Melbourne Water (2010). MUSIC Guidelines. Recommended and modelling approaches for MUSIC users.

Melbourne Water (2013). WSUD maintenance guidelines. A guide for asset managers. http://www.melbournewater.com.au/Planning-and-building/Forms-guidelines-and-standard-drawings/Documents/WSUD-Maintenance-manager-guidelines.pdf

Melbourne Water (2015). Design, Construction and Establishment of Constructed Wetlands: Design Manual, Draft, 2015.

PGA (2015) "*Modelling Brief" Morwell West Drainage Investigation*

Water Technology (October 2015) *Morwell North-West DCP Drainage Report – Interim Report – Hydrology.*

Water Technology (November 2015) *Morwell North-West DCP Drainage Report – Interim Report – Water Quality.*

APPENDIX A HYDROLOGY DESIGN CALCULATIONS / PARAMETERS

Rational Method Equation details:

The basic 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.

Rainfall parameters used in this study were derived from the program AusIFD. IFD parameters are specific to Morwell, as shown in **Table A-11-9**.

The calculation details are outlined in below in Table A-11-10.

Table A-1 Design Rainfall Input Parameters

IFD Parameter	$2I_1$ (mm/hr)	$2I_{12}$ (mm/hr)	$2I_{72}$ (mm/hr)	$50I_1$ (mm/hr)	$50I_{12}$ (mm/hr)	$50I_{72}$ (mm/hr)	G	F2	F50
Morwell	18.29	3.85	0.99	41.46	6.89	2.06	0.37	4.23	15.14

Table A-2 Rational Calculation Detail

Interstation area	Area (km^2)	FI	Q_{100} (Rational)	I (mm/hr)	tc	Fy	C'10	C10	Cy
RED	0.814	0.59	23.47	144.9	2	1.2	0.161	0.597	0.716
BLUE	0.947	0.54	19.39	109.7	17.93	1.2	0.161	0.560	0.672
L_BT	0.156	0.60	4.80	152.6	9.32	1.2	0.161	0.604	0.752
GRN	0.117	0.60	3.76	159.7	8.46	1.2	0.161	0.604	0.725
YLW	1.886	0.12	4.07*					11	
BRN	0.329	0.10	1.07*					11	
Internal Catchment	2.31	0.14	4.73*					11	

* Rural rational method adopted

RORB Modelling details:

The following RORB loss parameters were applied in this investigation (Table A-3). They are consistent with best practice approaches and are broadly consistent the current Melbourne Water modelling guidelines.

Table A-3 RORB model Loss Parameters

Loss reduction factors	
Initial Loss (IL) – Existing	20 mm
Initial Loss (IL) –Developed	10 mm
Runoff Coefficient (ROC)	0.6

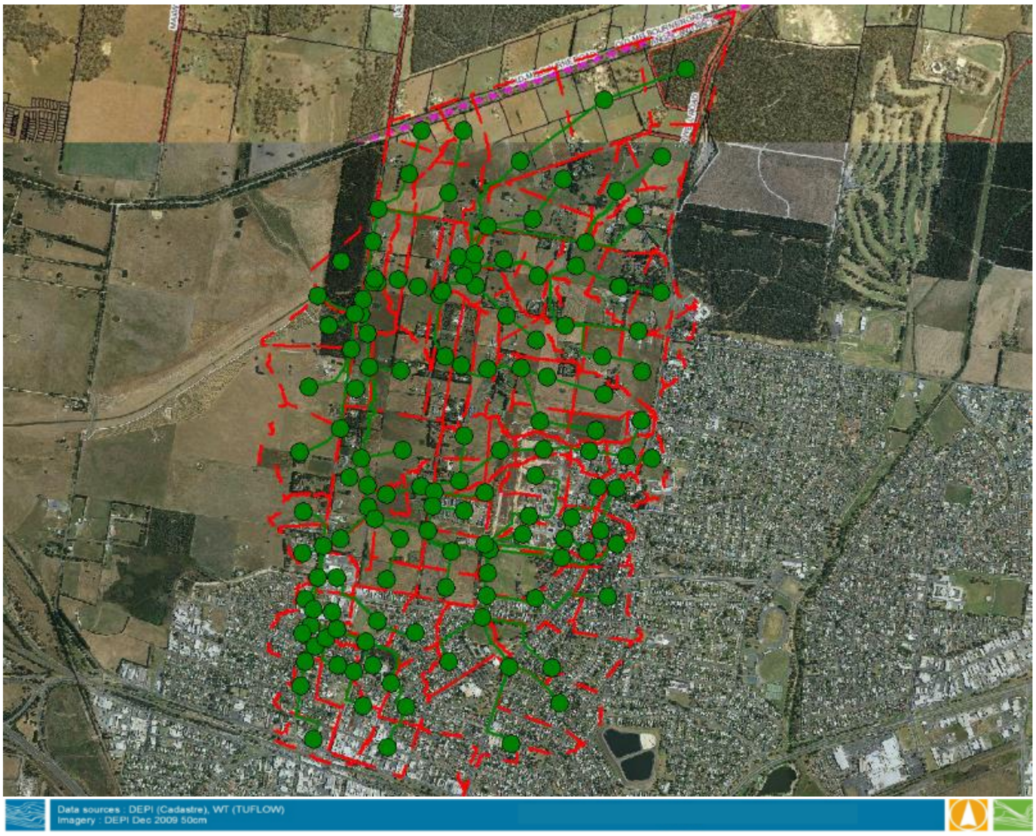


Figure A-1 RORB model setup for existing conditions

Figure A-1 shows the setup of the existing conditions RORB model, a copy of the RORB model catchment file is available upon request, it details specific fraction impervious rates applied to each sub-area in the model. Fraction impervious rates adopted by land use type are show in Table A-4. Figure A-2 shows fraction impervious rates adopted throughout the study area.

Table A-4 Fraction Impervious Designation for Planning Zones

Zone Code	Zone Description	Fraction Impervious (FI)
C1Z – C2Z	Commercial (1 – 2) Zone	0.9
FZ	Farming Zone	0.1
FZ	Farming Zone - Forestry*	0.1
IN1Z – IN3Z	Industrial (1 – 3) Zone	0.9
LDRZ	Low Density Residential Zone	0.2
MUZ	Mixed Use Zone	0.7
PCRZ	Public Conservation and Resource Zone	0
PPRZ	Public Park and Recreation Zone	0.1
PUZ1	Public Use Zone - Service and Utility	0.05
PUZ2	Public Use Zone - Education	0.7
PUZ3	Public Use Zone - Health and Community	0.7
PUZ4	Public Use Zone - Transport	0.7
PUZ6	Public Use Zone - Local Government	0.7
GRZ1	General Residential Zone 1	0.45
NRZ1	Neighbourhood Residential Zone	0.45
RGZ1	Residential Growth Zone 1	0.45
RGZ2	Residential Growth Zone 2	0.6
RDZ1	Road Zone - Category 1	0.7
RDZ2	Road Zone - Category 2	0.6
RLZ2, RLZ3, RLZ4, RLZ6	Rural Living Zone	0.2
SUZ1	Special Use Zone 1	0.6

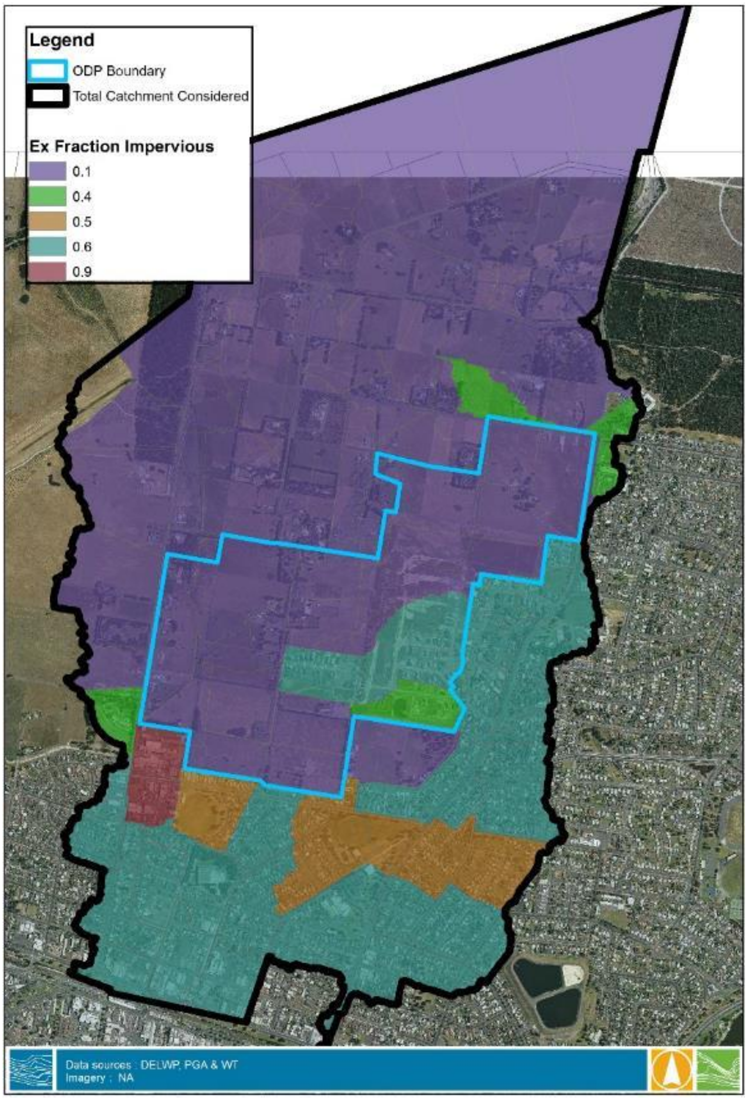


Figure A-2 Existing Conditions Fraction Impervious conditions

11.1.1 Existing Model Calibration

Table A-5 details the general RORB modelling parameters adopted in this study, they are consistent with best practices approaches are broadly consistent the current Melbourne Water modelling guidelines. Table A-6 shows the interstation area specific modelling parameters generated from the calibration process.

Table A-5 General RORB Parameters

Parameter	Value
m	0.8
Temporal Pattern	Filtered
Areal Reduction Factor	ARR87 Bk 11 (Figs 1.6 and 1.7)

Table A-6 Calibrated flows (100 year ARI)

	Interstation rea	Q (Rational)	KC	dav	KC/dav	IL	Q (RORB)	Diff	Crit. Duration
1	RED	23.47	0.47	0.79	0.59	10	23.57	-0.10	15m
2	BLUE	19.39	0.35	0.88	0.40	10	19.42	-0.03	25m
3	L_BT	4.80	0.95	0.33	2.88	10	4.77	0.03	15m
4	GRN	3.76	0.11	0.48	0.23	10	3.74	0.02	15m
5	YLW	4.07	3	1.27	2.36	20	4.02	0.05	3h
6	BRN	1.07	1.4	0.78	1.79	20	1.07	0.00	2h
7	MGT	1.16	1.9	0.58	3.28	20	1.17	-0.01	2h
8	Internal Catchment	4.73	3.8	1.43	2.66	20	4.73	0.00	4.5h

11.1.2 Existing Conditions Model Results Including the Effect of WR02/3

As discussed in the body of the report, a hydraulic model was used to describe the relationship between inflow into WR02/3 and its attenuation performance. Design inflows from the 20%, 5%, 2% and 1% events (3hr and 1hr durations) were routed through the hydraulic model to establish the two storage discharge tables show in Table A-7 and Table A-8. These tables were then applied to the RORB model.

Table A-7 Eastern Wetland Stage and flow relationship

Event	Elevation (H) m AHD	~Approximate Storage (S) m^3	Flow (Q) m^3/s
NA	73.6	5256.0	5.00
100y1h	73.19	3269.3	2.64
100y3h	73.17	3180.8	2.05
50y3h	73.14	3049.7	1.50
20y3h	73.11	2920.8	0.89
20y1h	73.10	2878.3	0.74
5y3h	73.04	2628.9	0.25
5y1h	72.97	2348.2	0.13
NA	72.70	1365.6	0.01*
NA	NA	423.9	0.00

* Orifice flow from Wetland outlet

Table A-8 Western Wetland Stage and Flow Relationship

Event	Elevation (H) m AHD	~Approximate Storage (S) m^3	Flow (Q) m^3/s
NA	73.1	9847.5	12
100yr1h	72.66	6309.1	7.7
100yr3h	72.63	6088.5	5.4
50y3h	72.57	5654.2	5.1
20y3h	72.49	5089.3	4.5
5y3h	72.31	3885.5	3.0
NA	7.15	423.9	0

11.1.3 Developed RORB Model

A copy of the developed conditions RORB model catchment file is available upon request, it details the changes in flow paths and specific fraction impervious rates applied to each sub-area in the model. Fraction impervious rates adopted by land use type are show in Table A-4. Figure A-2 shows fraction impervious rates adopted throughout the study area.

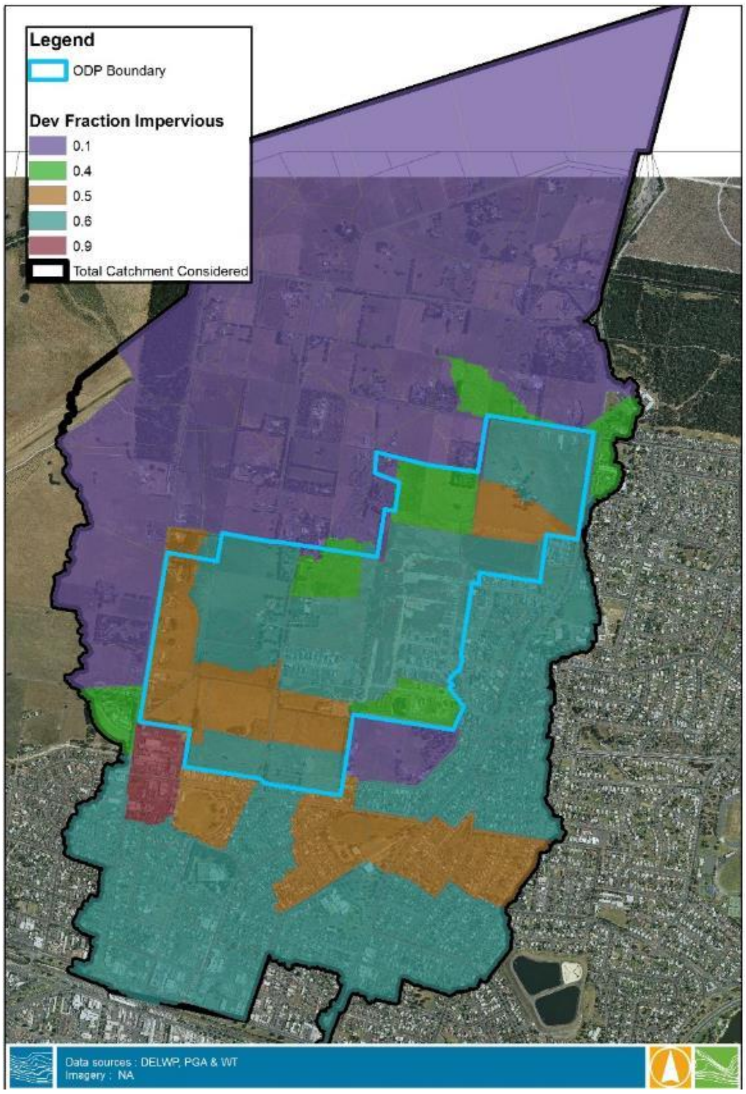


Figure A-3 Developed Conditions Fraction Impervious conditions

APPENDIX B WATER QUALITY CALCULATIONS

Rational Estimates:

The basic 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.

Rainfall parameters used in this study were derived from the program AusIFD. IFD parameters are specific to Morwell, as shown in **Table A-11-9**.

The calculation details are outlined in below in Table A-11-10.

Table A-11-9 Design Rainfall Input Parameters

IFD Parameter	$2I_1$ (mm/hr)	$2I_{12}$ (mm/hr)	$2I_{72}$ (mm/hr)	$50I_1$ (mm/hr)	$50I_{12}$ (mm/hr)	$50I_{72}$ (mm/hr)	G	F2	F50
Morwell	18.29	3.85	0.99	41.46	6.89	2.06	0.37	4.23	15.14

Table A-11-10 Rational Calculation Detail

			Rational Peak flow (m^3/s)			
Catchment	**Area**	**FI**	**3m**	**1yr**	**2yr**	**5yr**
Blue	38.54	0.58	0.62	1.35	1.94	3.08
Green	11.1	0.58	0.21	0.46	0.66	1.06
Pink	62.54	0.55	1.00	2.18	3.14	4.98
Red	21.45	0.52	0.37	0.82	1.18	1.87
Red (east)	7.51	0.52	0.13	0.29	0.41	0.66
Red (west)	13.94	0.52	0.24	0.53	0.76	1.21

SEDIMENT POND SIZING

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

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

$\lambda = 1 - 1/n$; $n = \frac{1}{1-\lambda}$

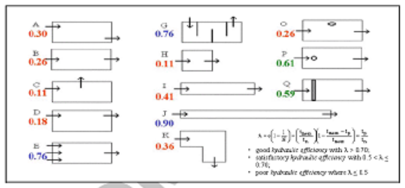


Figure 10.5 Hydraulic Efficiency – A measure of Flow Hydrodynamic Conditions in Constructed Wetlands and Ponds, Range is from 0 to 1, with 1 representing the best hydrodynamic conditions for stormwater treatment [Source: Australian Runoff Quality, 2003]

R = fraction of Initial Solids Removed = 80 - 90 % typ.

Source: WSUD Engineering Procedures:

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

WR02

Calculations			
Target = very fine sand			
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 =	2.18	m^3/s	use rational method to obtain 1 Year ARI flow for sub catchment
A =	2050	m^2	Area of retarding basin
V_s = / Q/A	10.34		25.2
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 =	62.54	ha	Just urban catchment concidered
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	m	
Actual Basin area =	2050	m^2	

Therefore, cleanout frequency required = $\frac{(1.6+0.4)A_{catchment}}{0.5d_{basin} \cdot A_{basin}}$ = 0.12 per year — Clean out every 8.2 years

Assumes cleanout when basin 50% full

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

clean out when sediment level is 500mm below NWL

WR04

Calculations

Target = very fine sand

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 =	1.35	m^3/s	use rational method to obtain 1 Year ARI flow for sub catchment
A =	1300	m^2	Area of retarding basin
$\frac{V_s}{Q/A}$ =	10.59		25.2
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 =	38.54	ha	Just urban catchment concidered
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	m	
Actual Basin area =	1300	m^2	

Therefore, cleanout frequency required = $\frac{(1.6+0.4)A_{catchment}}{0.5d_{basin}*A_{basin}}$ = 0.12 per year

Clean out every 8.4 years

Assumes cleanout when basin 50% full

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

clean out when sediment level is 500mm below NWL

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

WR03

Calculations												
Target = very fine sand												
Vs =	0.011	m/s										
d_e =	0.5	m										
d_p =	1.0	m										
d* =	1.0	m										
(d_e+d_p) =	1.0											
(d_e+d^*)												
Q =	0.46	m^3/s	use rational method to obtain 1 Year ARI flow for sub catchment									
A =	430	m^2	Area of retarding basin									
V_s =	10.28											
Q/A										25.2		
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 =			11.1	ha	Just urban catchment concidered							
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	m								
Actual Basin area =			430	m^2								
Therefore, cleanout frequency required =				$(1.6+0.4)A_{catchment}$ =		0.10	per year		Clean out every		9.7	years
				$0.5d_{basin}*A_{basin}$								
Assumes cleanout when basin 50% full												
Try to minimise cleanouts - ideally, once every 5 years						OK						
			clean out when sediment level is 500mm below NWL									

APPENDIX C TUFLOW MODEL PARAMETERS

The single precision version of the latest TUFLOW release was used for all simulations.

The hydraulic model has four main inputs:

- Topography data;
- Boundary conditions;
- Hydraulic structures; and
- Surface Roughness.

Topography Data

The model extent and setup is shown in Figure C - 1. LiDAR along with survey of the WR02/3 wetlands and surrounds were used to create the digital terrain model.

A grid size of 1 m was adopted to ensure adequate detail of the waterways and floodplain features while maintaining reasonable model run times. Where required 2d zsh layers were used to accurately define key floodplain features.

.

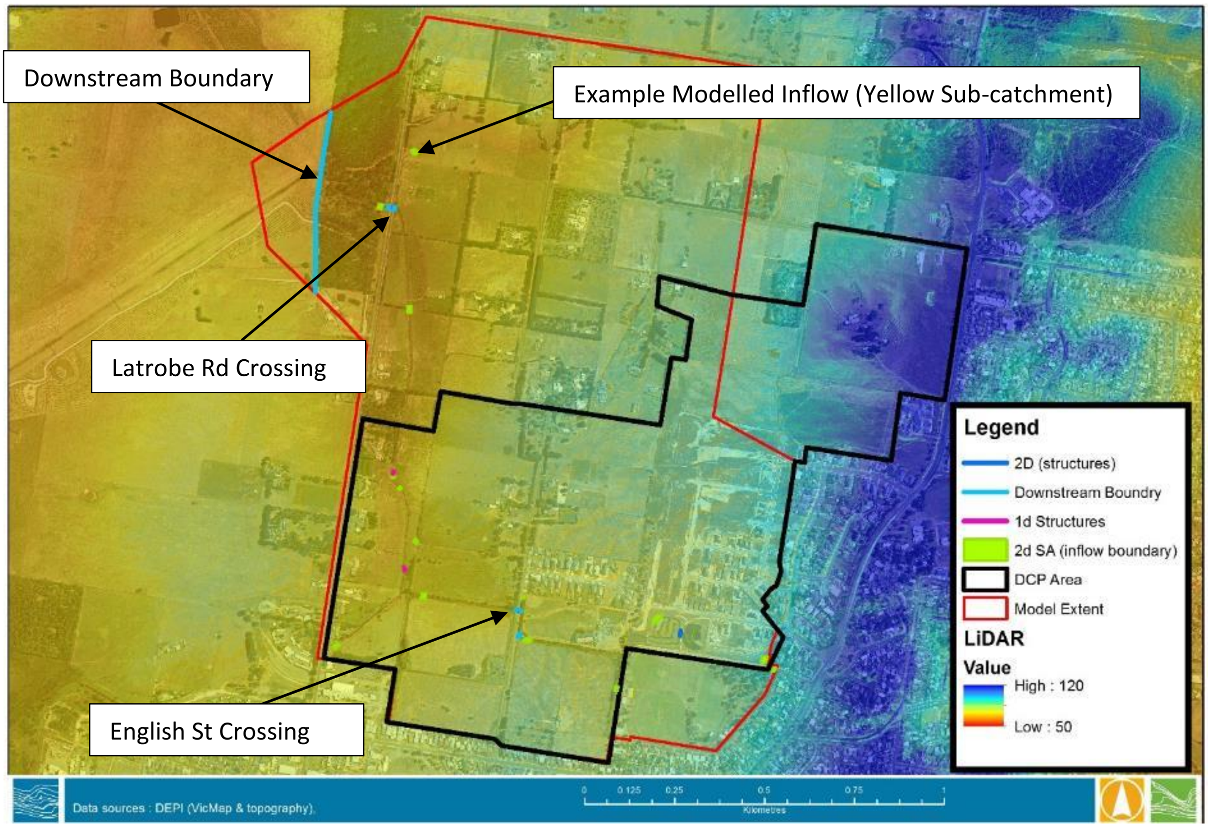


Figure C - 1 Flood Model Setup

Boundary Conditions

14 inflow boundaries were applied throughout the model. Where broad sheet overland flow paths (no defined inflow channel) occurred inflows were placed in the centre of the drainage reserve.

Design hydrographs were adopted from the RORB modelling work.

The major downstream boundary in the model (west of Latrobe street) was modelled with a Level (H) verses Flow (Q) boundary. This boundary type let water out of the model at a steady rate which match the localised channel slope.

Key Hydraulic Structures

The following key structures were included in the model:

- The weir connecting the two wetlands which make up WR02/3. This feature was modelled using a layered flow constriction element, this restricted flow in the 2D domain;
- English Street crossings (x2) these crossings were modelled as 1D elements, sizes and levels were based on PGA survey ; and
- Latrobe Road culvert crossing were modelled as 1D elements, sizes and levels were based on PGA survey.

Initial Water Level

The preliminary modelling included no initial water inside the study area

Surface Roughness

Areas with different roughness types were identified from aerial photos. The roughness parameters used in the study are shown in Table C - 1.

Table C - 1 Hydraulic Model Roughness Parameters

Land Use	Manning's 'n' value
Residential - Urban (higher density) - when building footprints and remainder of parcel are modelled together	0.35
Industrial/Commercial or large building	0.30
Open Space or Waterway - minimal vegetation	0.035
Open Space or Waterway - moderate vegetation	0.06
Open Space or Waterway - heavy vegetation	0.09
Paved Surface/Roads	0.02
Drainage Easement	0.05
Open Water	0.02
Open Water with reedy vegetation	0.065

TUFLOW Model Checks

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

- 2D grid size: Given the size of the study area, a grid size of 1 m was used get the best accuracy from the data available. Where required, zsh layers were used to represent key 2d elements;
- 2D time step: The 2D time step is 0.5 second for the 1 m grid, $^{1}/_{2}$ of the grid size;
- 1D time step: The 1D time step is 60 seconds;
- Model mass errors: The mass errors for all models were no greater than 1 %; and
- Warning messages: Checked and found to be suitable for the system conditions.
- Errors messages: None.

Based on the above checks, we consider the TUFLOW model to meet the requirements as outlined in the Melbourne Water's 2D Modelling Guidelines (2012).