BALLARAT WEST PSP REVIEW

Drainage Strategy Update

Prepared on behalf of City of Ballarat

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11 APRIL 2024

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Rev	Date	Description	Author	Reviewer	Project Mgr.	Approver
0	26/09/2023	Client Issue	Harry George	Glenn Ottrey	Glenn Ottrey	Paul Clemson
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5	11/04/2024	Client Issue	Harry George	Glenn Ottrey	Glenn Ottrey	Paul Clemson
Signatures:						

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INTRODUCTION

The City of Ballarat is undertaking a review of the Ballarat West Precinct Structure Plan (PSP) and Development Contributions Plan (DCP).

As part of the PSP and DCP review Engeny was engaged by the City of Ballarat (Council) to undertake an update of the Ballarat West PSP drainage strategy, which comprises Precinct 1, Precinct 2 and Precinct 4. The drainage strategy provides inputs to the PSP in terms of the required drainage and stormwater treatment infrastructure and to the DCP with cost estimates undertaken for the proposed assets. Some changes have been made since the strategy was first developed in 2011 and to date, they were largely to accommodate construction staging and implementation of drainage works. The 2023 update focuses more on the changes that maybe required to the drainage strategy to reflect with the most recent updated guidelines and standards that have been released since 2011. The updated guidelines include Australian Rainfall and Runoff 2019 (ARR 2019), updated design guidelines, updated Urban Stormwater Management Guidelines (EPA Victoria, June 2021) and the Ballarat Integrated Water Management Plan (Feb 2018).

While some changes have been made since the original strategy was developed in 2011, the objectives and location of key infrastructure is still largely in line with the original strategy. This updated strategy will supersede all previous strategy documents and be the working strategy for the implementation of the remaining assets in the drainage strategy.

1.1 Scope of Works

The scope of works for this drainage strategy update includes the following:

1.1.1 Part A – Review of Current Status

A determination of the current status of the drainage strategy and its implementation. This involved the following:

- Review documentation including plans and report regarding changes to the drainage strategy which have occurred since the previous reviews were undertaken or the strategy was setup (as appropriate).
- Determining which assets were already constructed or committed due to the level of progression design or construction work already completed in accordance with the previous strategy.
- Determine which areas still required drainage, treatment or retardation assets to be constructed in order to service those parts of the development.
- Summarising this work in a memo to Council the details of which are included in this report.

1.1.2 Part B – Modelling Updates

- Update the RORB hydrologic model to reflect the following:
  - Current development status, including all changes made to the scheme.
  - The storage available above the extended detention depth level of the wetland where wetlands and retarding basins are co-located (in line with current MW guidance).
- Update the RORB model to be compliant with Australian Rainfall and Runoff 2019
  - Update the land use types to reflect effective impervious areas, indirectly connected areas and pervious areas.
  - Update the intensity, frequency and duration rainfall data from the Bureau of Meteorology.
  - Update the model to an initial and continuing loss model (from runoff coefficient).
  - Update the flow validation of the RORB model based on guidance from the Corangamite CMA or other regional validation methods.
  - Expand the RORB model to include the whole Winter Creek catchment.
  - Rerun the RORB model for the 20% and 1% AEP events and determine if the Retarding Basin (RB) sizing is acceptable to meet the flow targets
  - Rerun the RORB model for the 20% and 1% AEP events for climate change scenario.

- Update the MUSIC water quality model to include the following:
  - To reflect the current development status.
  - Consideration of Gross Pollutant Traps (GPTs) at the entries to wetlands.
  - To reflect the guidance provided by Melbourne Water in their Wetland Design Manual (reducing the extended detention depth to 350 mm from 500 mm and adjusting the sedimentation basin sizing to be based on a Fair and Geyer calculation).
- Consideration of implementation of rainwater tanks on lot scale and / or stormwater harvesting for the oval from the adjacent wetland/retarding basin to try to achieve the goals set out in the Urban Stormwater Best Practice Environmental Guidelines issued by the EPA. These guidelines have strong total flow volume reduction targets, which can be challenging to achieve with traditional wetlands and sedimentation basins alone.
- Consideration of staging and delivery of future assets to guide the priority of the delivery of as yet unconstructed assets
- Noting the assumptions and exclusions used in updated this strategy.

1.1.3 Part C – Final report

- Summary of development completed within the PSP and the drainage infrastructure delivered along with any changes to the drainage strategy
- Overview of the current works completed relative to the updated guidelines
- Details of the proposed changes to make the remaining undeveloped parts of the scheme compliant with the updated guidelines, including justification for why the changes are needed

  High level cost estimates of the proposed wetland, sedimentation basin, retarding basin and pipe assets. We note that we are not quantity surveyors and are not proposing to engage quantity surveyors but will use previous construction rates we are aware of and also information provided by the City of Ballarat relating to local construction costs. The more recent local information that can be provided the better our cost estimates will be. We will also require information from the City of Ballarat to inform likely land acquisition costs based on recent previous acquisitions. Engeny has significant experience in costing drainage schemes for Melbourne Water and undertook a project on behalf of Melbourne Water to review and update the standard rates to cost drainage schemes.
- Details on the proposed staging and development of works including a table showing which infrastructure is required to support each property to develop.
- Staging plan for the next 10 years to help deliver good stormwater management outcomes in the remainder of the drainage scheme.

1.2 Previous Drainage Strategy Reports

The following previous drainage strategy reports have been used to guide this updated drainage strategy as they have materially changed the PSP stormwater management strategy direction. There are other adjustments to the delivery of on ground infrastructure which have been implemented as the designs have progressed from concept design to detailed design but are considered to be generally in accordance with the intent of the scheme design and so are not listed below:

- Ballarat West Growth Area PSP Drainage Report by SMEC Urban / Engeny Management (February 2011)

  Engeny was previously engaged in 2011 by SMEC and the City of Ballarat to inform the Ballarat West Development Contributions Plans (DCP) in relation to drainage infrastructure. Engeny undertook the hydrologic and water quality modelling, developed concept layouts for pipes and retarding basins, and prepared preliminary cost estimates for the drainage assets.
- Updated functional designs of retarding basins 11, 12 and 13 by Neil Craigie (2015)

  The location and designs of retarding basins 11, 12 and 13 were updated to help facilitate development in the north western area of Precinct 1. This included areas of the Delacombe Town Centre and adjacent residential development.
- Review of Main Drain proposals for the Power Park Catchment in Precinct 1 by Neil Craigie (August 2015)

  An update to the proposed drainage layout and layout of RB 28 which is proposed within the Power Park reserve. This review recommended the removal of RB30 and replaced it with an online sedimentation basin.
- Lot32 and 32A Tait Street IWMS by Niel Craigie (September 2015)

Proposed a staged approach to the construction of RB18 to help facilitate development

- RB26 Catchment and Outfall IWMS by Neil Craigie (July 2016)

  A variation to the original stormwater management strategy which amalgamated RB25 and RB26 into a single basin as part of the Ploughmans Arms development.

- Memorandum: Update of Engeny RORB Modelling and Adjustments to the SWMS Across the BWGA by Neil Craigie (April 2019)
- Ballarat West Growth Area PSP by Engeny (November 2021)

  Engeny was engaged by the City of Ballarat to undertake a review of the Ballarat West Precinct Structure Plan (PSP) drainage catchment design. An update was required to reflect changes to the drainage network caused by the need to build new infrastructure to support developments built “out-of-sequence”. This included drainage upgrades needed for the delivery of Webb Road (East) and Ascot Gardens Drive resulting in runoff being directed west of Webb/Cherry Flat Road. This report was prepared to assist Council with:

  - Determining the development contributions needed to facilitate a timeline for implementation of drainage assets (i.e. identifying when and where the infrastructure will be needed).
  - Optimising the sequence of development to ensure timely provision of infrastructure.
  - Budget forecasting using estimated costs associated with the drainage assets.

- Ballarat West Growth Area PSP: Precinct 2 Review by Engeny (April 2022)

  This report update was required to reflect changes to the drainage network caused by the need to build new infrastructure to support developments built “out-of-sequence”. This included drainage upgrades needed for the delivery of Webb Road (East) and Ascot Gardens Drive resulting in runoff being directed west of Webb/Cherry Flat Road.

- Memorandum: Ballarat RB04 and RB05 Review – Initial Drainage Review Findings by Engeny, (September 2022)

  This memo review focuses on the drainage of the southern portion of the Alluvium Estate and the drainage of the adjacent parcels of land in Precinct 2. The key updates included an updated strategy developed by Neil Craigie in 2019 and a review of the strategy in Precinct 2 in 2020 by Engeny. The Engeny review largely adopted the recommendations of the work by Neil Craigie.

2. DRAINAGE ASSETS REVIEW

Council has provided engineering drawings and related documentation for most of drainage infrastructure assets, which includes retarding basins, wetlands and biofiltration systems. Layout plans of the asset locations are shown in the following Figure 2.1 and Figure 2.2 and the drainage assets list and status are provided in Table 2.1. Appendix D displays the pipe layout plans with diameter and pipe ID visible for each precinct.

The retarding basins outside of the PSP area have been added to the hydrology model to ensure that their impact on the timing of peak flows is accounted for in the modelling.

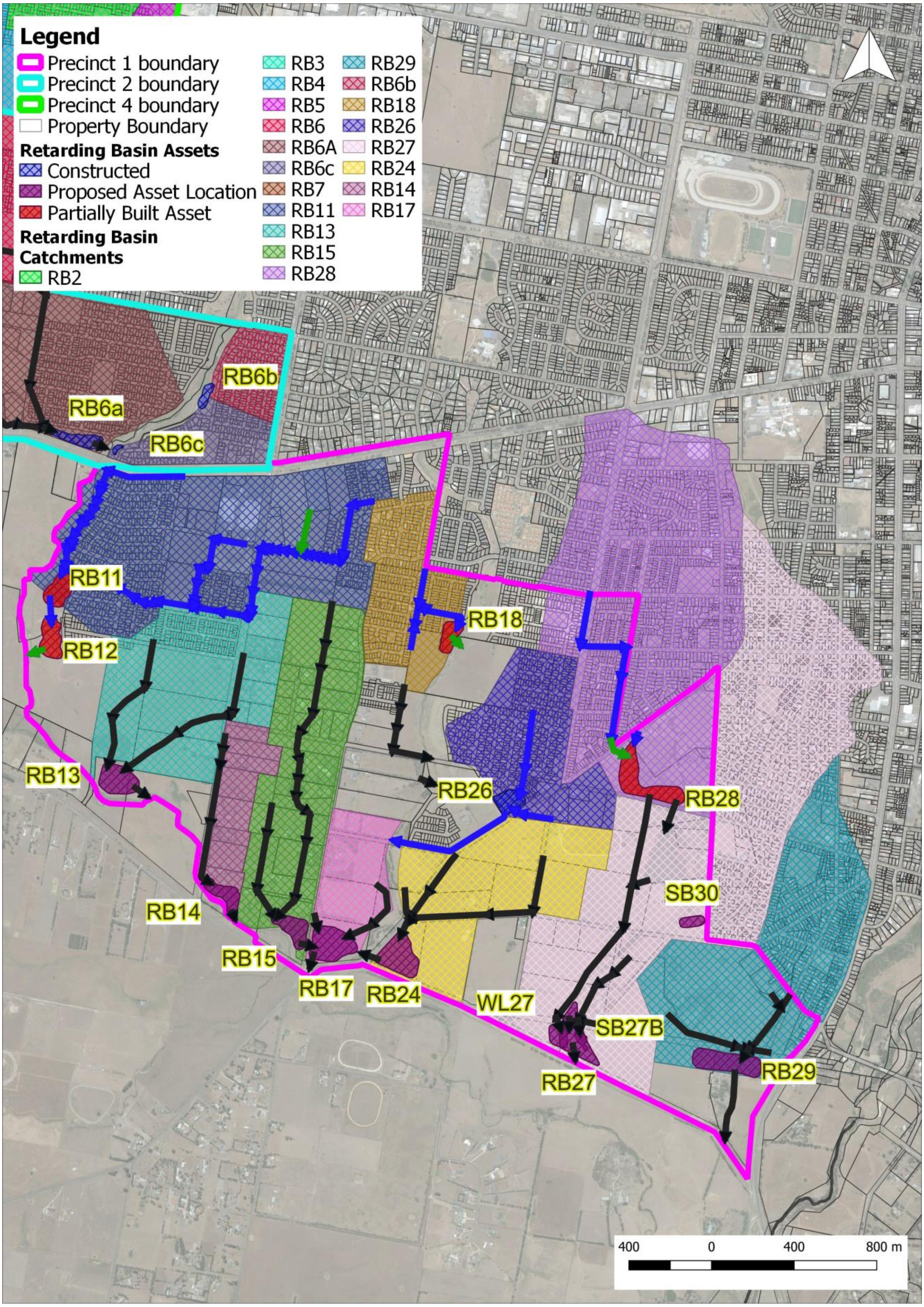

FIGURE 2.1: BALLARAT WEST PSP PRECINCT 1 LAYOUT PLAN

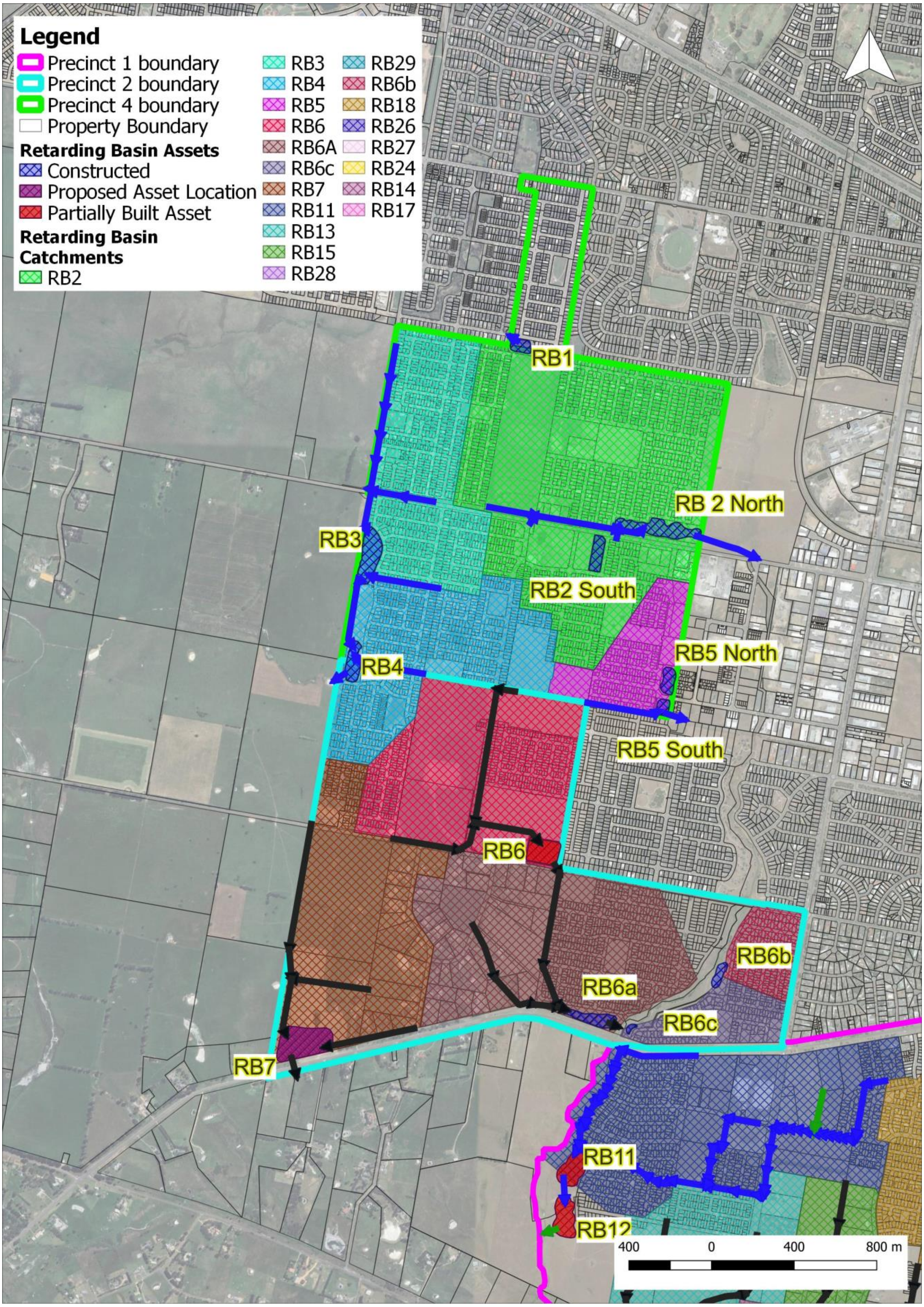

FIGURE 2.2: BALLARAT WEST PSP PRECINCT 2 AND PRECINCT 4 LAYOUT PLAN

TABLE 2.1: DRAINAGE ASSETS LIST

Drainage Asset	Residential Estate	Asset Status	Asset catchment size (km^2)	Available Data	Designer	Notes
RB DZ	The Chase	Completed		Drawings (design): in PDF	TGM	Outside of the Ballarat West PSP
RB EB	Alfredton Park	Completed		Drawings (design): in PDF	City of Ballarat	Outside of the Ballarat West PSP
RB FW	Winter Creek	Completed		Drawings (design): in PDF	City of Ballarat	Outside of the Ballarat West PSP
RB 1 (RB DY)	Winter Valley Rise Estate	Completed		Drawings (as built): in PDF and CAD Memo: Update of RB1 Catchment Main Drainage Proposal (Neil Craigie, June 2018)	Cardno TGM	-
RB 2	Alluvium Estate	Completed	1.4	Drawings (as built): in PDF and CAD	Reeds Consulting	-
RB 3	Winter Valley Rise Estate	Completed	0.6	Drawings (as built): in PDF	Cardno TGM	-
RB 4	Winter Valley Rise Estate	Partially Completed	0.6	Drawings (as built): in PDF	Cardno TGM	RB 4 has been partially completed.
RB 5	Carringum Estate	Completed	0.24	Drawings (design): in PDF and CAD Memo: RB 5 specifications	Beveridge Williams	-
RB 6	Winterfield Estate	Partially Completed	0.75	n/a	n/a	Functional layout plan endorsed and interim sedimentation basin works commenced.
RBs 6A, 6B & 6C (previously Biofilters 8, 9 & 10)	Winterfield Estate	Completed	6A - 0.87 6B - 0.12 6C - 0.16	Drawings (design): in PDF	KLM spatial	-
RB 7	n/a	Not Built/ Committed	0.7	n/a	n/a	-
RB 11	Pinnacle Estate	Partially Completed	1.02	Drawings (design): in PDF Memo: RB 11 & 12 specifications	Spiire	Design has been completed and endorsed. Minor construction of sedimentation has been undertaken to enable some development.

Drainage Asset	Residential Estate	Asset Status	Asset catchment size (km^2)	Available Data	Designer	Notes
RB 12	Pinnacle Estate	Partially Completed	1.13	Drawings (design): in PDF Memo: RB 11 & 12 specifications	Spiire	See comments for RB 11.
RB 13	n/a	Not Built	0.61	n/a	n/a	
RB 14	n/a	Not Built	0.2	n/a	n/a	
RB 15	n/a	Not Built	0.6	n/a	n/a	
RB 17	n/a	Not Built	0.22	n/a	n/a	
RB 18	n/a	Partially Completed	0.33	n/a	n/a	
RB 24	n/a	Not Built	0.53	n/a	n/a	
RB 25 (combined with 26)	Ploughmans Arms Estate	Completed	0.41	Drawings (design): in PDF	Scott Campbell Design & Drafting Pty Ltd	
RB 27	n/a	Not Built	1.68	n/a	n/a	
RB 28	n/a	Partially Completed	1.44	Drawings (design): in PDF and CAD	Axiom Consulting Engineers	Full design has been completed and outfall has been constructed.
RB 29	n/a	Not Built	0.81	n/a	n/a	
SB 30 (RB30 has been replaced with a sedimentation basin in an adjacent location)	n/a	Not Built		n/a	n/a	

3. HYDROLOGY

3.1 Hydrology Model Update

Hydrological modelling for the original 2011 drainage strategy was undertaken using RORB software and based on Australian Rainfall and Runoff (ARR) 1987. Since then, a new version of Australian Rainfall and Runoff (ARR 2019) has been released and the current RORB modelling update for the strategy has been undertaken in accordance with ARR 2019 guidelines. The updated RORB modelling for the existing condition scenario was undertaken to assess the existing peak flow at the model outlet, LK2 (confluence of Winter Creek and Yarrowee River) and Winter Creek, LT1 which is just upstream of the confluence of Winter Creek and Yarrowee River as shown in Figure 3.1.

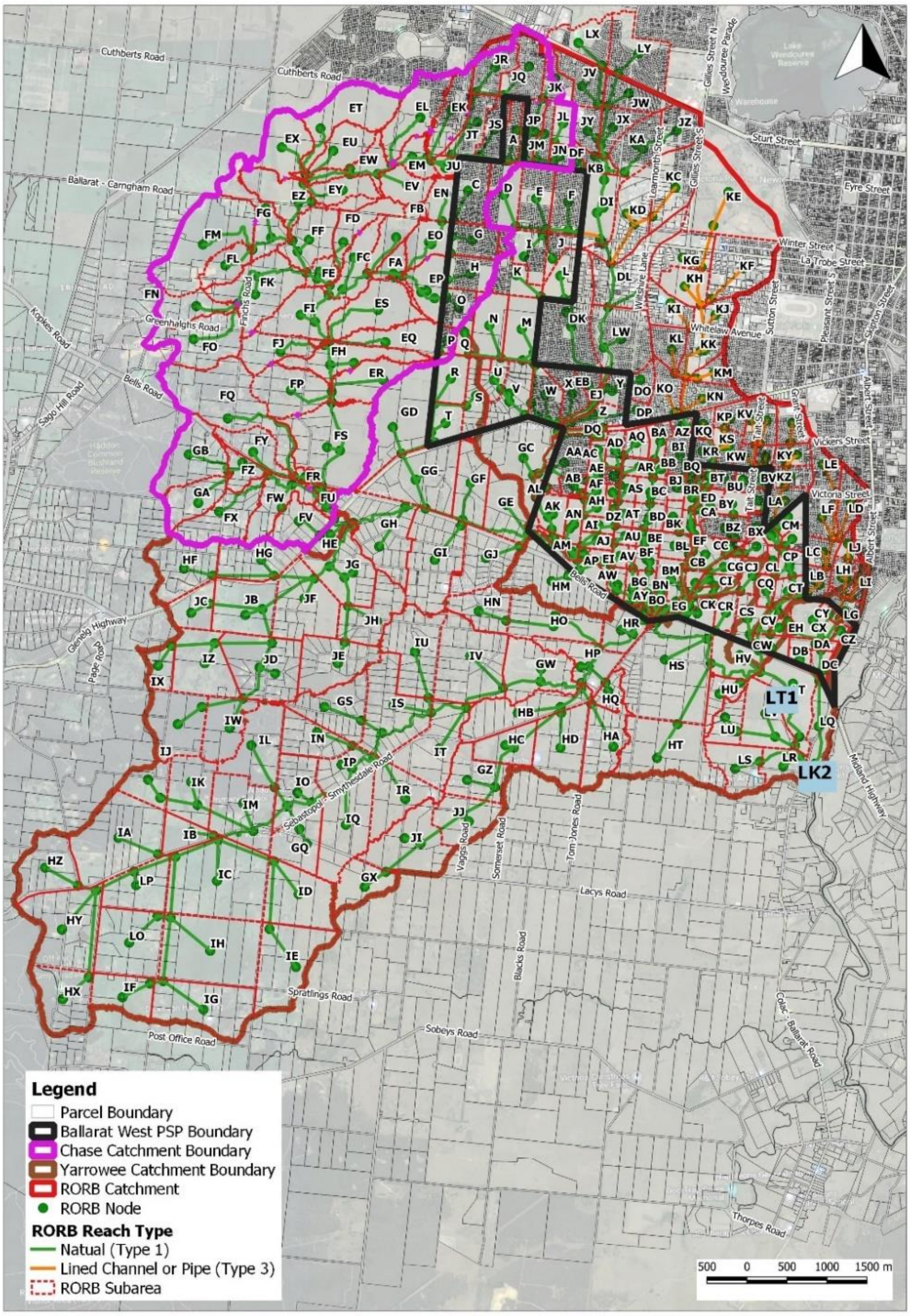

FIGURE 3.1: RORB MODELLING FOR BALLARAT WEST PSP CATCHMENT PLAN

In addition to the ARR update, the RORB modelling catchment for the existing conditions scenario has also been expanded to include the whole Winter Creek catchment area. This expanded catchment is to provide consistency that will be required at a later stage to properly understand the impact that the retarding basins may be having to the peak flows in Winter Creek for the post developed scenario.

The updated RORB model includes data from the "Chase catchment" RORB model and the existing Yarrowee River RORB model, which were previously developed for the Council on previous projects (refer to Figure 3.1 for catchments boundaries). Both Chase and Yarrowee River RORB modellings (ARR 2019) for existing conditions were previously prepared by Water Technology and were provided by Council for Engeny's use in this project. These models have also been used for calibration purposes.

To assess the existing Winter Creek catchment (80 km^2), which includes the Ballarat West PSP area and the impact on the receiving waterway (Winter Creek), the existing RORB model for "the Chase catchment" has been combined with the existing Ballarat West RORB model as shown in Figure 3.1, with additional subareas taken from Yarrowee River RORB model. The subareas from the Yarrowee River RORB model have been split to improve the resolution of the model in the area of interest, the Winter Creek catchment. The delineation of reaches and the fraction impervious in existing conditions have been updated as follows:

- For sub-catchments within the Ballarat West PSP (shown by a thick black line in Figure 3.1) have largely been classified as "Type 1 – Natural" reaches with a total fraction impervious of 0.1 in line with the existing RORB models (this impervious fraction has been modelled as indirectly connected area due to the lack of pit and pipe drainage systems in these areas).
- For sub-catchments within the existing Chase RORB modelling area (shown by a pink line in Figure 3.1 have largely been classified as "Type 1 – Natural" reaches with total fraction impervious of 0.1 in line with the existing RORB models prepared by Water Technology.
- Sub-catchments immediately to the east of Ballarat West PSP in the existing township areas of Ballarat have largely been classified as "Type 3 – Lined Channel or Pipe" reaches with some area classified as "Type 1 – Natural" reaches, with total fraction impervious ranging from 0.1 to 0.75 in line with existing conditions.
- Sub-catchment immediately to the southwest of Ballarat West PSP have largely been classified as "Type 1 – Natural" reaches with total fraction impervious of 0.1, in line with the existing RORB model of the Yarrowee River prepared by Water Technology.
- A detailed breakdown of the subareas size, impervious fraction and location can be found in Appendix A:.

The existing RORB model was run for two scenarios as follows:

- Existing / Baseline Conditions
- Existing / Baseline Conditions with climate change scenario.

3.2 Modelling Parameters and Modelling Input for Retarding Basins

The RORB model parameters adopted are as summarised as follows:

3.2.1 Intensity-Frequency-Duration (IFD) Rainfall Data

- Rainfall data was adopted based on the centroid of the updated extended Ballarat West RORB model as per Table 3.1 (-37.6037°S, 143.76647°E).
- Point rainfall temporal patterns were adopted. It is noted that point temporal patterns are generally recommended for catchment areas that do not exceed 75 km^2. The total catchment area for the extended Ballarat West RORB model is 80 km^2. Engeny has run a sensitivity analysis using the areal and point temporal patterns and found that the peak flows at the model outlet using either pattern were very similar.
- In addition, while the total catchment is 80 km^2, the sub-catchments draining from the Ballarat West PSP are around 1 km^2 in area. Hence point temporal patterns have been used for all durations, which Engeny believes is appropriate for the purposes of this assessment.

TABLE 3.1: BOM IFD TABLE FOR OVERALL SITE CATCHMENT (-37.6037°S, 143.76647°E).

	Annual Exceedance Probability (AEP)					
Duration	50 %	20 %	10 %	5 %	2 %	1 %
10 minutes	7.48	11	13.6	16.4	20.5	23.9
15 minutes	9.07	13.4	16.6	20	25	29.2
30 minutes	12	17.6	21.8	26.2	32.7	38.1
1 hour	15.2	21.9	27	32.4	40	46.3
2 hours	19.2	27.1	33	39.1	47.8	54.9
3 hours	22.2	30.9	37.3	43.9	53.2	60.8
6 hours	28.9	39.3	46.7	54.4	65.4	74.4
12 hours	37.8	50.9	60	69.5	83.1	94.2
18 hours	43.9	59.1	69.8	80.7	96.6	109
24 hours	48.5	65.5	77.6	89.8	107	121

3.2.2 Spatial Variation

- A uniform spatial distribution for rainfall was adopted.
- It is noted that per ARR 2019, it is recommended that non-uniform spatial distributions are considered for catchments exceeding 20 km^2. Engeny has assessed and compared the variation in rainfall depth across the catchment using IFD data based on the centroid of the whole catchment, and centroid of subareas KE, FM, LQ, and HX (refer to Figure 3.1), which represents sub-catchments in the northeast, northwest, southeast and southwest edges of the catchment respectively. As shown in Table 3.2, there is a marginal difference (ranging between 1% to 3%) in IFD rainfall depths of other areas in the catchment compared to the catchment centroid, thus, a uniform spatial variation was deemed appropriate for this study.

TABLE 3.2: COMPARISON OF THE BOM IFD TABLE ACROSS THE RORB MODEL CATCHMENT (20% AEP)

Duration	Subarea KE	Subarea FM	Subarea LQ	Subarea HX	Catchment Centroid
10 minutes	11.2	11.0	10.9	11.2	11.0
15 minutes	13.6	13.4	13.3	13.6	13.4
30 minutes	17.8	17.6	17.4	17.9	17.6
1 hour	22.2	22	21.7	22.3	21.9
2 hours	27.3	27.2	26.6	27.4	27.1
3 hours	31.0	30.9	30.2	31.2	30.9
6 hours	39.3	39.3	38.2	39.5	39.3
12 hours	50.6	50.7	49.3	50.9	50.9
18 hours	58.7	58.8	57.4	58.9	59.1
24 hours	64.9	65	63.7	65.1	65.5

3.2.3 Pre-burst application

- For this study, a complete storm approach has been modelled in RORB to account for pre-burst rainfall. This was achieved by appending pre-burst rainfall depths obtained from the ARR Data Hub to the BoM IFD burst rainfall. Based on the flow results calibration, median pre-bursts (rather than 75th percentile pre-bursts) were adopted.
- The recent Benchmarking ARR 2019 for Victoria study undertaken by HARC (2020) found that the 75^{th} percentile pre-burst rainfall magnitudes provided by ARR Data Hub provided a better fit across catchments in loss region 3 when compared to the median pre-burst rainfall magnitudes. The RORB model catchment falls within this loss region 3. Engeny compared the peak flows at key locations from the RORB model using the 50th and the 75^{th} percentile pre-burst rainfall and found that the flows generated from application of 50th percentile pre burst rainfall compared better to the calibrated Yarrowee River RORB Model. As such the 50th pre-burst rainfall depths have been adopted for this study.

3.2.4 Initial and Continuing Losses

- The model adopts a rural initial loss of 25 mm and a continuing loss of 2.0 mm/h. These losses were determined from the calibrated Yarrowee River and 'The Chase' RORB Models and have been adopted for the current model.
- In addition to utilising the rural initial loss and continuing losses from the ARR Data Hub, ARR 2019 also provides a methodology to calculate the initial loss and continuing loss values for other land uses. Losses in RORB were assigned based on three surface types:
  - Effective Impervious Area (EIA) – comprising areas which are impervious and are directly hydraulically connected to the drainage system (e.g., a roof connected to an underground drain by downpipes)
  - Indirectly Connected Area (ICA) – comprising impervious areas which are not directly connected to the drainage system (e.g., a paved patio or footpath) and pervious areas that interact with impervious areas which are not directly connected (e.g., nature strips and garden areas)
  - Pervious area – comprising large parklands and bushlands reserves but not small pocket parks in urban areas.

Table 3.3 summarises the loss values adopted for each surface type modelled.

TABLE 3.3: SUMMARY OF ADOPTED LOSS VALUES BY SURFACE TYPE

Surface Type	Initial Loss (IL)	Continuing Loss (CL)	Source
Pervious Area (from ARR Datahub)	25.0 mm	2.0	Yarrowee River and The Chase RORB model (calibrated)
Effective Impervious Area (EIA)	1.0 mm	0 mm/h	ARR Data Hub and ARR 2016, Book 5, Chapter 3 - Section 3.5.3.2.1
Indirectly Connected Area (ICA)	16.8 mm	2.0 mm/h	ARR Data Hub and ARR 2016, Book 5, Chapter 3 - Section 3.5.3.2.1

3.2.5 Areal Reduction Factor (ARF).

With regards to areal reduction factors (ARFs), two scenarios have been considered as follows:

- ARF for a catchment size of 360 km^2, which is the area of the Yarrowee River catchment, was adopted to allow for the comparison of flows between the existing Yarrowee River RORB model and the current RORB model at Winter Creek just upstream of the confluence with the Yarrowee Creek.
- ARF for a catchment size of 80 km^2, which is the catchment area of the current RORB model through to the confluence of Winter Creek and Yarrowee River, adopted when analysing the impact of developing the Ballarat West PSP on the receiving waterways (Winter Creek and Yarrowee River).

3.2.6 Routing Parameter

The routing parameter (k_c) was determined using the same k_c divided by Distance average (D_{av}) based on the previous Yarrowee River RORB model. The Yarrowee River RORB model has been calibrated to a flood frequency analysis at the (Mt Mercer - 233215). By utilising the same k_c divided by D_{av} ratio consistency in the flow estimates produced by the models can be achieved. Corangamite Catchment Management Authority have provided in principle support to use a k_c divided by D_{av} estimation for the k_c of the catchment within a larger calibrated RORB model (the existing Yarrowee_Gnarr RORB modelling). The m routing parameter was maintained at the recommended default of 0.8.

Table 3.4 provides a summary of the k_c, d_{av} and k_c/d_{av} ratios from the Yarrowee_River RORB modelling.

TABLE 3.4: CALCULATED K_C/D_{AV} RATIOS FOR THE RORB MODELS

Source RORB Model	k_c	d_{av}	k_c/d_{av} Ratio
Yarrowee_River RORB Model (ARR 2019 Watertech Model)	30	14.76	2.03
Ballarat West PSP RORB (ARR 2019 Engeny Model)	19.56	9.59	2.04

3.3 Modelling Results

3.3.1 Pre-Development Conditions

Engeny has compared the 1 % AEP peak outflows at at the Node LK2 on Winter Creek, just upstream of the confluence of Winter Creek and Yarrowee River (refer to Figure 3.1) to the pre-developed flows from the Yarrowee River RORB model for both existing climate conditions (based on the IFD data available from the Bureau of Meteorology) and the Year 2100 climate conditions (incorporating an 18.5 % rainfall intensity increase, in line with the guidance provided within Melbourne Water's Technical Specifications). Table 3.5 provides a summary of the resultant peak flows.

TABLE 3.5: 1% AEP EXISTING CONDITIONS PEAK FLOWS AT CONFLUENCE OF WINTER CREEK

RORB Model	Existing Condition Peak Flow (m^3/s)
Yarrowee_River RORB Model (ARR 2019 Watertech Model)	72.3
Ballarat West PSP RORB (ARR 2019 Engeny Model)	83.5*

*the Engeny model has been run with an ARF of 360 km^2 to match these flows as the Yarrowee River RORB model was also run with an ARF of 360 km^2. This value is only relevant for this validation comparison, the existing conditions flow for PSP assessment purposes is shown in Table 3.8.

As shown above, the flow result from the updated ARR 2019 RORB model for Ballarat West PSP shows a comparable result (with difference of 14%) from the Yarrowee River RORB model result. The minor difference in the flows is due to the following:

- **Reaches -** Sub-catchments immediately to the east and north of Ballarat West PSP in the existing township areas of Ballarat have largely been classified as "Type 3 – Lined Channel or Pipe" reaches and "Type 2 – Excavated but Unlined" reaches respectively in the current model. These reaches have however been modelled as "Type 1 – Natural "in the Yarrowee River RORB model and thus contribute to the differences in peak flows.
- **Losses –** The losses in the current RORB model were assigned based on three surface types (i.e., pervious Area, EIA, and ICA), while in the Yarrowee River RORB model, the losses only represented on a single value for each sub-catchment instead of assigned to different surface types. This could also account for the difference in peak flow.

In addition to the above results, peak flows results have also been compared with the previous Engeny model. Engeny's original RORB model (2011) had a total of ten discharge locations that capture all flows into the waterways and discharge points for precincts 1, 2 and 4, as shown in Figure 3.2. Engeny has compared 1% AEP peak flows between the existing conditions for the 2011 study and current model as presented in Table 3.6. The results show comparable predicted pre-development flows in most locations. The current RORB modelling update for the strategy has been undertaken in accordance with the ARR 2019 guidelines, which largely account for the differences in flows. In addition, the current model has included the Wensleydale retarding basin, which was not modelled in the 2011 study and thus also accounts for the large difference in peak flows in Location 4 (flows to the Kensington Creek at Glenelg Highway).

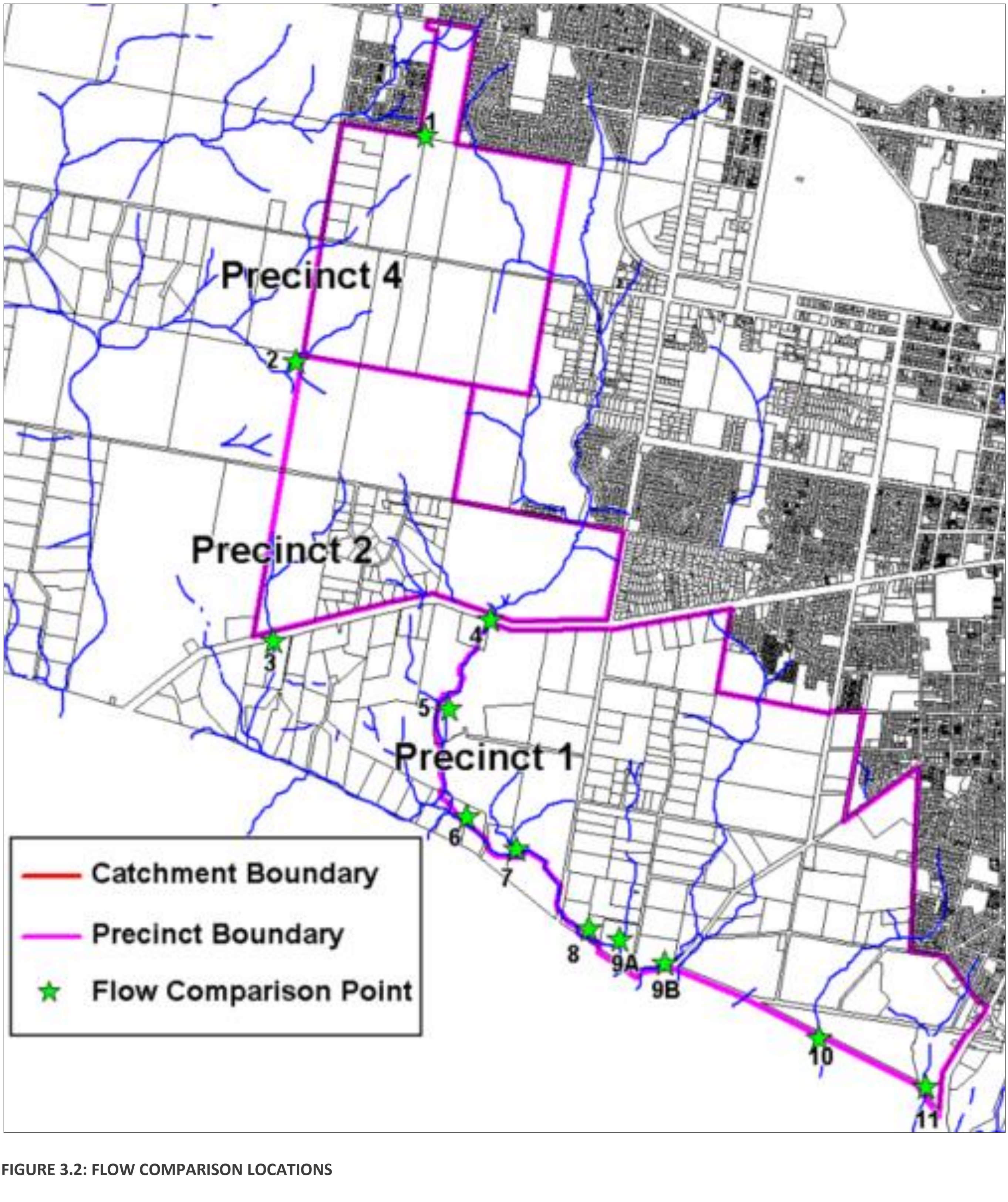

FIGURE 3.2: FLOW COMPARISON LOCATIONS

TABLE 3.6: ENGENY 1% AEP PRE-DEVELOPMENT PEAK FLOW TARGETS COMPARISON FROM BALLARAT DCP STUDY IN 2011

Comparison Locations	2011 Ballarat DCP study in 2011 (ARR 1987) (m^3/s)	Current Study (ARR 2019) (m^3/s)
Location 1	3.40	3.26
Location 2	4.10	3.78
Location 3	3.20	3.14
Location 4	32.20	28.69
Location 5	2.40	4.07
Location 6	1.10	2.18
Location 7	4.30	3.63
Location 8	1.30	1.17
Location 9a	3.40	3.00
Location 9b	23.40	23.68
Location 10	13.20	13.89
Location 11	4.80	4.60

3.3.2 Post Development Condition

Engeny has updated the developed condition RORB model to include details of the already built retarding basins and adjusted the sizing of the retarding basins which have not yet been built to try and achieve the best retardation outcomes possible. Table 3.7 shows the pre and post developed flows at the flow comparison locations where were referenced in Figure 3.2. The table shows that the pre-development flow rate is maintained or reduced at 8 of the locations but increases at 4 locations.

The increases have occurred as the original RORB modelling which informed the design of the retarding basins which have already been built was undertaken in ARR 1987 methodologies in 2011, whereas the current assessment uses ARR 2019 methodologies.

The updated modelling also accounts for an increase in development density that is reflected in the higher yields of 17-18 lots/ha which have been occurring in more recent development within the precinct. Overall, the current Ballarat West precinct average is 16 lots/ha. The modelling also accounts for an expected future increase in development density outlined in the *Precinct Structure Planning Guidelines: New Communities in Victoria, (VPA, October 2021)* that has been introduced by the VPA. These guidelines increased the proposed development density of greenfield development from 15 dwellings per hectare which was assumed for the initial drainage strategy to 20-25 dwellings per hectare under the new guidelines. The increase in density has translated to a total impervious fraction of 0.75 up from the previous assumption of 0.6. The increases to development density have not been considered retrospectively in catchments in which development and assets have already been constructed. There is not considered scope to change those assets, as they were built to the appropriate engineering standard at that time. In areas where the basins have not been constructed the basins sizes and outfalls have been adjusted to try and meet the predevelopment flows. In some parts of the catchment there is a mixture of constructed and not constructed basins. In these areas it may not have been possible to achieve predeveloped flow targets.

Table 3.7 also includes a comparison at the downstream end of Winter Creek just before it enters the Yarrowee River. The table shows that there is a 1.2 m^3/s increase in flows. This increase represents a 1.3% increase on the predevelopment flow rate. There are a few factors which are leading to this increase in flow.

(1) Change in hydrology methodology. The original drainage strategy was setup using ARR 1987 methodology while the current strategy has been reviewed using ARR 2019 methodology. The update to ARR 2019 represents a significant change in hydrologic methods which

would be expected to show some difference in flows. This is support by the comparison shown in Table 3.6 which compares the developed flow targets using ARR 1987 and ARR 2019. The general trend is for lower target flows. Location 4 is a key callout as the target flow has dropped by almost 4 m^3/s.

(2) Partial completion of drainage scheme. Approximately half of the retarding basins in the drainage scheme have already been constructed or committed to construction. The sizing of those basins was based on ARR 1987 methodologies. When the performance of those basins is reassessed using ARR 2019 methodologies they are not always meeting the new current design criteria (however they did meet the design criteria which was current when they were built or approved). This is effectively applying a new design criteria to an already constructed asset. In most cases the performance is similar to what the new design criteria would propose, however it is not fully compliant (this is to be expected). Using the example of location 4 above, under the ARR 1987 methodologies the flow target was 32.2 m^3/s, under the ARR 2019 methodologies it is 28.69 m^3/s. Given that all of the basins upstream of location 4 have already been constructed or committed using ARR 1987 methodology this increase in flow under the updated hydrology design criteria is locked in.

To offset this increase in flows would require a significant oversizing of basins in the as yet undeveloped areas of the scheme. This has equity issues from a development contributions point of view as land owners who have yet to develop are effectively paying to offset the impacts of previous development. The previous development was also compliant with the appropriate standard at the time of design acceptance. Some minor (and the overall increase of 1.3% is minor) change in flow rates should be expected with such a significant change in methodology and should not undermine the integrity of the previous built assets which used the best available information at the time.

(3) Increase in development density. There has also been a gradual increase in development density as the drainage scheme has progressed. It is likely that some of the earlier developments were at or below the design density of 15 lot/ha which was used to inform the modelling. As the density has increased, if the basins have not also increased in size then they may be spilling more flow, as either an increase in peak flow or as an increase in total volume of flow. The total volume of flow can become more important when the overall impact on Winter Creek is assessed as it can impact the timing of peak flows.

As there is an increase in peak flows predicted, hydraulic modelling of Winter Creek and the downstream Yarrowee Creeks has been undertaken to determine the impact of the increased flows on flood depths and extents. This is discussed further in section 8 but the overall impacts are considered negligible in the context of the overall modelled flooding. Some areas record minor increases in peak flood depths and other areas record minor reductions.

TABLE 3.7: ENGENY 1% AEP PRE-DEVELOPMENT AND POST DEVELOPMENT FLOW COMPARISON

Comparison Locations	Predeveloped flow (m^3/s)	Post developed retarded flow (m^3/s)
Location 1	6.12	6.31
Location 2	3.59	3.55
Location 3	2.90	2.57
Location 4	23.66	20.77
Location 5	3.64	5.57
Location 6	1.26	Outfalls at location 7 under developed conditions
Location 7	4.53	3.84
Location 8	1.39	0.83
Location 9a	2.57	1.55
Location 9b	22.13	22.19
Location 10	10.94	10.86
Location 11	4.36	4.2
Winter Creek upstream of Confluence with Yarrowee Creek*	91.5	92.7

* model run with ARF set to 80 km^2 for this flow comparison point only. All others run with ARF set to 1 km^2

The developed conditions assets have been designed to current climate conditions. Consideration of climate change shows that there will be a significant increase in peak flows if there is an 18.5% increase in rainfall intensity as predicted at the year 2100. Without explicitly designing assets for the climate change event, the best approach to managing to risk of large flows as a result of climate change (and also the risk of storms rarer than a 1% AEP under current climate conditions) is to ensure that unimpeded overland flow paths are available along all flow paths and that no areas are designed with trapped low points serviced only by pipe connections. Overland flow paths typically are able to convey larger flows than they are designed for due to the allowance of freeboard (typically 300 mm) before any dwellings are flooded. Underground drainage pipes are typically only able to convey the design flow, with any additional flow above the design flow rate causing flooding or overland flow.

This should be a key consideration in the assessment of development layout plans and plans which propose trapped low points or increased pipe sizes to minimise overland flows should be subject to additional security to ensure that flows larger than the 1% AEP event will not immediately flood private properties or dwellings (i.e. minimum freeboard requirements must still be maintained).

The figures in Appendix D show where the key overland flow paths required in the development areas are. These overland flow paths need to be accounted for in the development layouts and the functional and detailed designs of the developments.

3.3.3 Climate change

Engeny has undertaken climate change modelling to understand the likely impact of climate change in the PSP. The rainfall has been increased by 18.5% for the 2100 climate change modelling scenario, in line with the guidance from ARR 2019. The results from the modelling are shown in Table 3.8. This results in an 34% increase of flow from the existing climate conditions for Ballarat West PSP compared to the 2100 climate change conditions. The increase in flows is notably larger than the increase in rainfall intensity, which is 18.5%. Predicting a larger increase in flows than the increase in rainfall intensity is common for climate change modelling. This also demonstrates that increases in rainfall do not provide a like for like increase in total expected flows.

TABLE 3.8: CLIMATE CHANGE MODELLING RESULTS (RORB MODEL ARF 80 KM²)

Existing Condition Peak Flow (m^3/s)	Developed conditions Peak Flow (m^3/s)	Developed conditions 2100 Climate Condition Peak Flow (m^3/s)		
	1 %	1% Climate Change	2 % Climate Change	5 % Climate Change
91.5	92.7	125.0	100.4	72.1

3.4 Retarding Basins

Table 3.9 shows the key design criteria for the retarding basins that have not been constructed or designed and committed at the time this review was completed. It also shows details of the basins which were constructed with a design that is not considered in accordance with the original PSP. Basins constructed generally in accordance with the original drainage strategy and PSP are not shown. Only the outstanding retarding basins are subject to change as part of this review. The retarding basins have been designed to a detailed concept level only and so additional design work is required prior to the construction of the basins. The table shows the storage volume required in the 1% AEP event, the peak outflow in the 1% AEP event and the estimated cut volume that is needed to achieve this storage volume. It may be possible to reduce the require cut volumes with further design work however future designs must demonstrate that they are generally in accordance with the key design criteria of the basins and meet the minimum performance requirements.

TABLE 3.9: RETARDING BASIN KEY DESIGN CRITERIA

Drainage Asset	1% AEP storage volume (m³)	Assumed outlet pipe Diameter (mm)	Peak 1% AEP outflow (m³/s)	Estimated Cut Volume (m³)	Notes
RB7	19,600	2 x 675	2.57 (Pipe flow)	35,800	
RB 13	17,400	2 x 825	3.84 (Pipe flow)	39,300	RB location slightly adjusted to reduce number of parcels contributing land
RB 14	9,860	525	0.83 (Pipe flow)	14,500	RB location slightly adjusted
RB15	12,000	2 x 650	2.42 (Pipe flow)	26,000	
RB 17	25,200	675	1.56 (Pipe & Spillway)	43,400	
RB 24	25,900	600	3.03 (Pipe & Spillway)	38,600	
RB 27	21,200	1 x 600 1 x 1050	10.86 (Pipe flow)	N/A	Retarding basin is proposed as an embankment across the waterway. Pipe dimensions are sized based on the RB27 design reverting flows back to the pre-development in the 1 % AEP
RB 29	17,200	2 x 750	2.86 (Pipe flow)	36,500	
SB 30 (RB30 has been replaced with a sedimentation basin in an adjacent location)	N/A	N/A	N/A	N/A	RB 30 has been removed and replaced with a sedimentation basin only. No retardation is required at this asset

TABLE 3.10:RETARDING BASIN LAND UPATKE

RB Name	Area of RB (m^2)	# of Parcel 1	Area Parcel 1 (m^2)	# of Parcel 2	Area Parcel 2 (m^2)
RB1	8939	211	8939		
RB2 North	31803	213	31803		
RB2 South	10543	215	10543		
RB3	25020	220	25020		
RB4	15663	220	15663		
RB5 North	10050	214	10050		
RB5 South	6589	214	6589		
RB6	20000	157	20000		
RB6a	15960	158	15960		
RB6b	5697	160	5697		
RB6c	1417	159	1417		
RB7	38616	209	38616		
RB11	20267	1	20267		
RB12	19679	1	19679		
RB13	23695	12	19188	11	4507
RB14	17413	81	17016	82	397
RB15	22516	83	22516		
RB17	35631	96	35631		
RB18	12727	65	6309	67	6418
RB24	35958	101	33990	102	1968
RB26	13970	87	13970		
RB27	44818	134	11270	154	33548
RB28	62042	114	5036	116	57006
RB29	34328	154	10913	153	23415
SB30	5865	128	5865		

3.4.1 RB1

Retarding basin 1 has already been constructed. The design was adjusted to increase the overall footprint. The basin is split into two parts, a wet sediment basin section in the northern half and a "dry creekbed" section in the southern half. The retarding basin was made larger than was originally proposed in the 2011 drainage strategy.

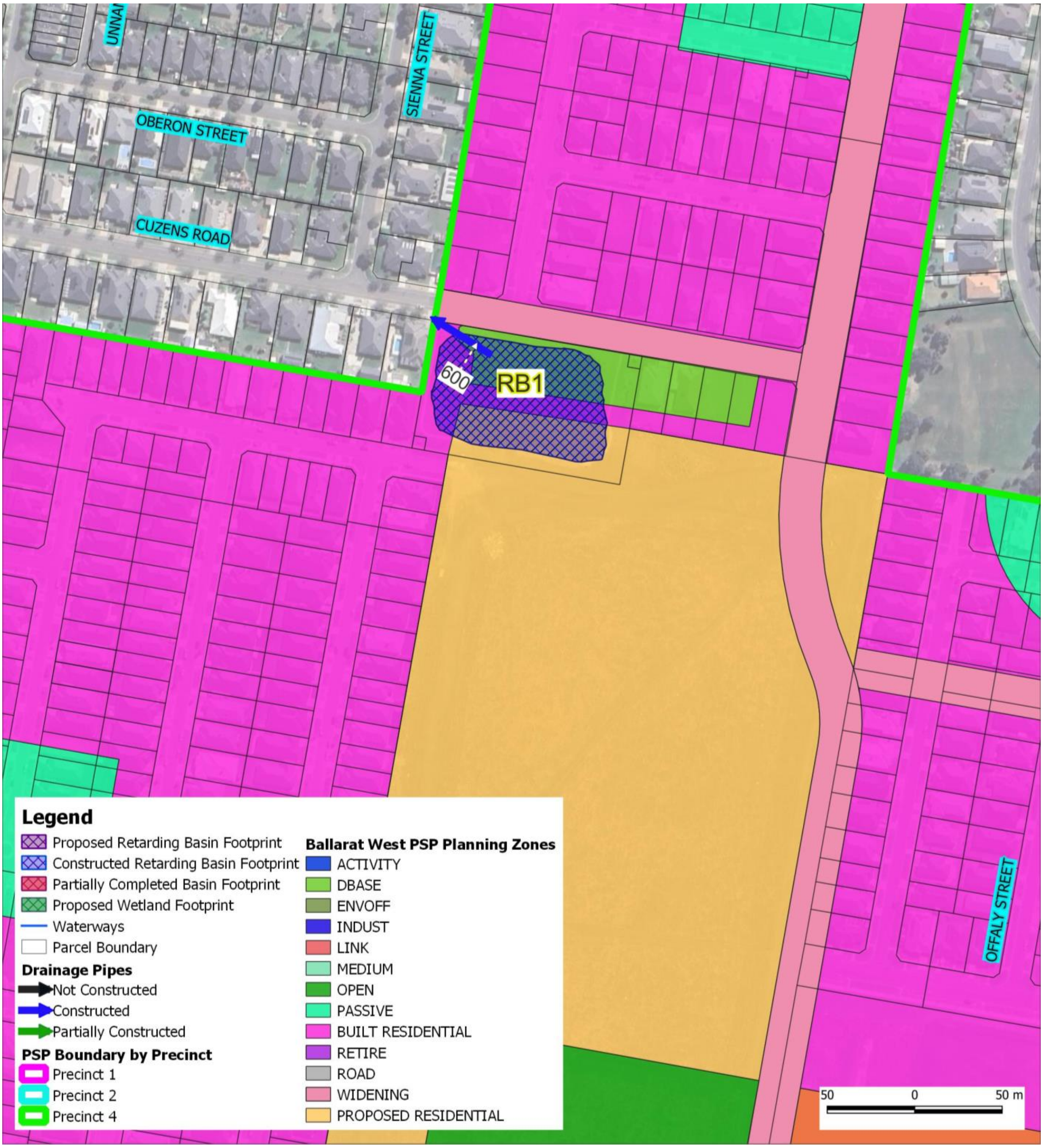

FIGURE 3.3: RETARDING BASIN 1 LAYOUT

3.4.2 RB2

Retarding basin two has already been constructed. The basin has been split into two halves. The northern half was constructed first as it was required by the earlier development stages and was the downstream section. The southern half was constructed second when the adjacent development also occurred. The key reason for the split in the basin and adjusting it to straddle both sides of Ballarat Carngham Road was to help facilitate drainage outfalls in this area. There is very little fall between RB2 south and the outfall to Kensington Creek to the East. By creating long linear wetlands an effectively flat water grade can be created. This can significantly reduce the fill required for the remaining part of the development as the pipes can discharge to a lower level further away from the creek without compromising the required hydraulic conveyance.

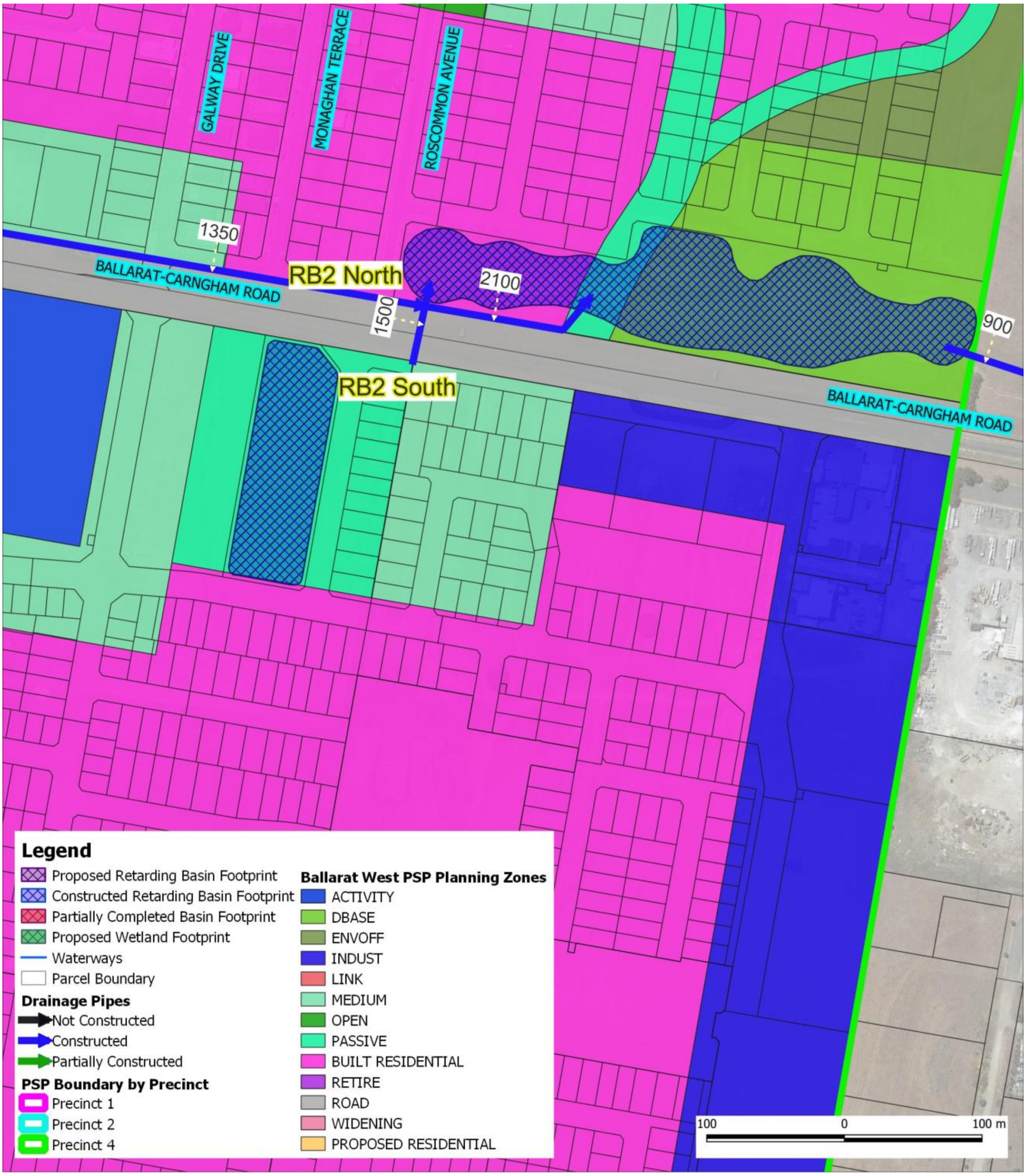

FIGURE 3.4: RETARDING BASIN 2 LAYOUT

3.4.3 RB4

Retarding basin four has been constructed. The retarding basin was moved and constructed in two parts to help facilitate development staging. The basin was moved north from its original position. The northern half, which was a sedimentation basin and retarding basin, was constructed first to facilitate the adjacent development. The southern half, which includes the wetland and additional retardation volume, was constructed a few years later when that estate reached the point at which it needed the drainage asset. Figure 3.5 shows the detailed design playout plan of retarding basin 4.

FIGURE 3.5: RETARDING BASIN 4 LAYOUT

3.4.4 RB5

Retarding basin 5 has been committed and is under construction. The asset has been split into two parts with a road running through the middle. Figure 3.6 shows the detailed design drawing of the basin. The northern part of the basin includes the sedimentation basin and part of the wetland, while the southern part includes the remainder of the wetland. The northern and southern parts combined provide the retardation function of the basin. The basin is generally in the same location as proposed in the 2011 drainage strategy, however the road through the middle has been included to provide a better development outcome, including providing better road links between adjacent estates.

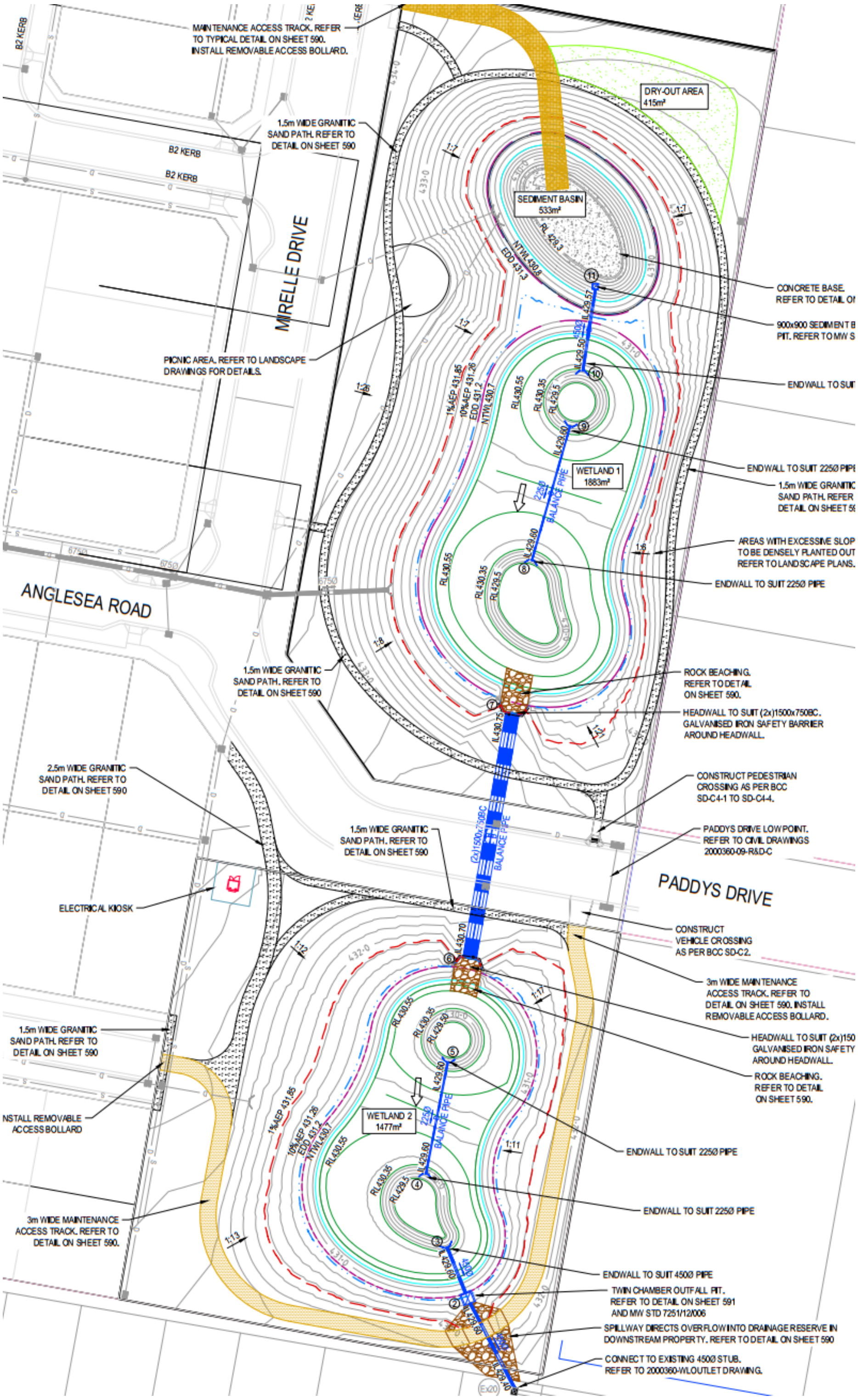

FIGURE 3.6: RETARDING BASIN 5 LAYOUT

3.4.5 RB6

Retarding Basin 6 is currently in the process of being delivered as part of the development of the land on which it is located. Figure 3.7 shows the proposed functional design layout. The asset if being delivered in a location which is broadly in accordance with what was proposed in the 2011 drainage strategy. The size of the wetland asset strategy has been reduced significantly compared to what was proposed in the 2011 drainage strategy due to the introduction of RB6A which is discussed below.

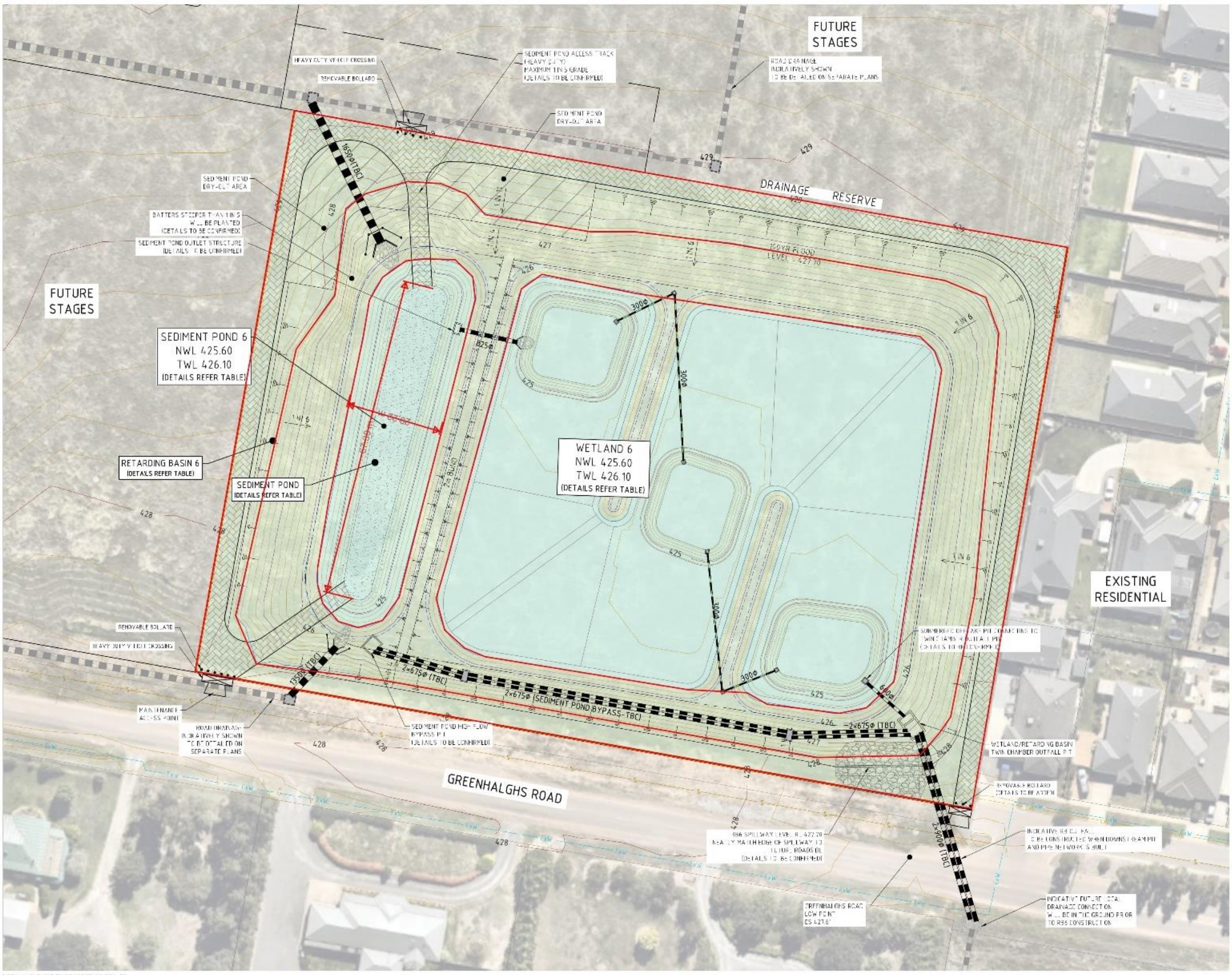

FIGURE 3.7: RETARDING BASIN 6 FUNCTIONAL DESIGN LAYOUT

There was a modification to the drainage strategy proposed by Neil Craigie (Kensington Creek Catchment – Review of drainage proposals between Greenhalghs Road and Glenelg Highway date 21 April 2016) as part of some proposed adjustments to retarding basin 6 and also the raingardens which were proposed adjacent to Kensington Creek. Basins 6A, B and C were developed based on this report and are discussed further in section 3.4.6 below. This proposal suggested that a longer narrower basin for RB6 which extended along Greenhalghs Road. The key benefit this would provide is in reducing the length of incoming pipe runs which could reduce the amount of excavation needed for the basin and wetland. This option was assessed by the developer of the site but not pursued due to commercial reasons for wanting to maximise the developable land fronting onto Greenhalghs Road.

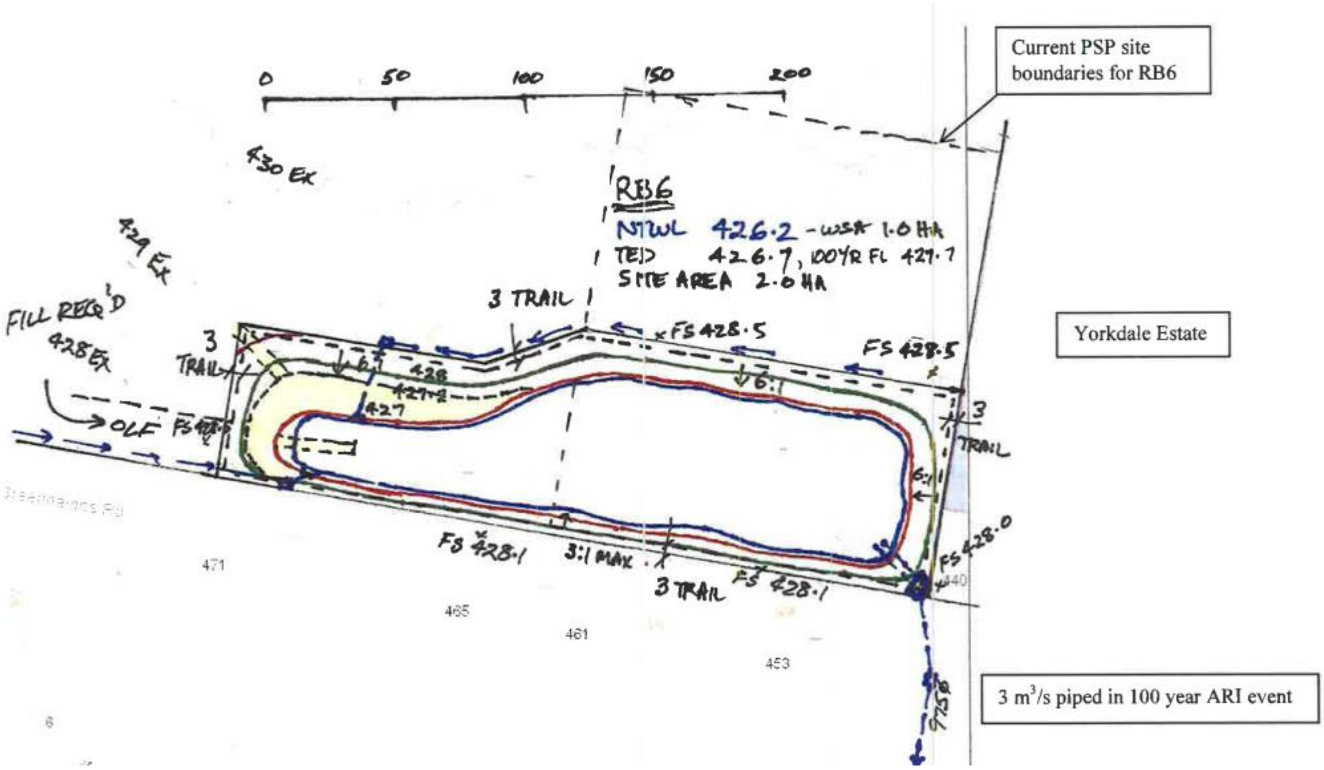

FIGURE 3.8: RETARDING BASIN 6 ALTERNATIVE DESIGN LAYOUT – NOT PURSUED

3.4.6 RB6A, B and C

The integrated sediment ponds/retarding basins RB 6A, 6B and 6C have been proposed to replace a series of biofilters as part of the stormwater treatment measures of Precinct 2. Neil Craigie completed a functional design of RB 6A which is shown in Figure 3.9. This asset has a land area of 1.85 hectares and also incorporates a 5200 m^2 sediment basin and 200 m^2 biofilter. This asset replaces Biofilter 9 that was proposed in the original drainage strategy. The combination of a sedimentation basin and biofilter will be easier for Council to maintain than a biofilter alone which would be subject to high loads of sediment and likely to have issues with surface blockage. In line with Neil Craigie's design, Engeny has also modelled the existing 1800 x 900 diameter box culvert on Glenelg Highway to carry a maximum of 3.8 m^3/s from the retarding basin to the downstream property south of Glenelg Highway, whilst the remainder of the retarding basin's outflows will be piped east to Kensington Creek. The existing box culvert discharges to a property located outside of the PSP development area.

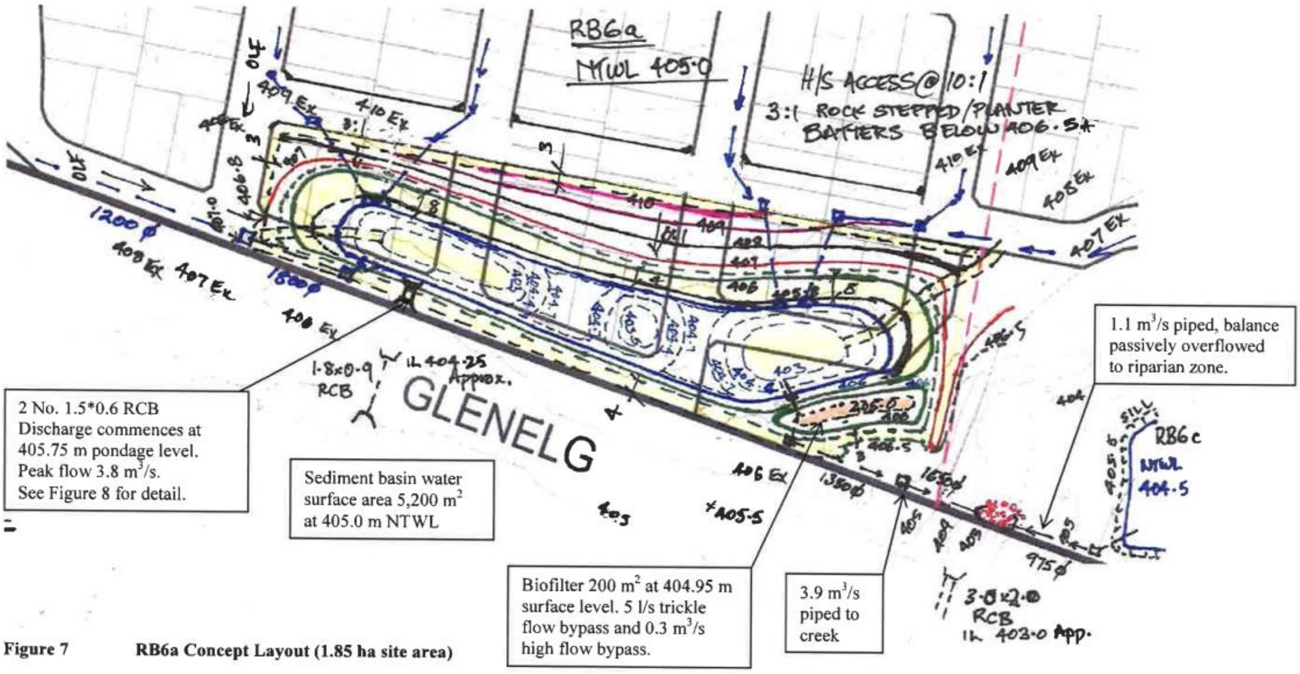

FIGURE 3.9: RETARDING BASIN 6A FUNCTIONAL DESIGN LAYOUT

RB 6B and RB 6C were constructed to replace Biofilters 8 and 10 respectively. Design plans by KLM Spatial which were provided to Engeny by Council were used for used for hydrological and water quality modelling in this study. As proposed by Neil Craigie, who undertook the functional designs, RB6B and RB6C are proposed to be offline RBs, meaning that they are not situated within the main Kensington Creek waterway. Figure 3.10 shows the layout of RB6B and Figure 3.11 shows the layout of RB6C. RB6B caters for the 1 % AEP flows arising from sub-catchment Z1 in addition to a further 2 m^3/s coming from the 21.5 hectares external catchments east of Wiltshire Lane (sub-catchments DO and DP) and sub-catchment Y. As shown in Figure 3.10, it is proposed that by using a flow diversion structure, 2 m^3/s will be piped to RB 6B and the balance will overflow into Kensington Creek. As with RB6A replacing biofilters with sedimentation basins will provide for assets that are easier to maintain. While they do not achieve the same nitrogen removal rates as biofilters, including the treatment of the external catchments has boosted the pollutant removal to a level that it meets the aims of the strategy.

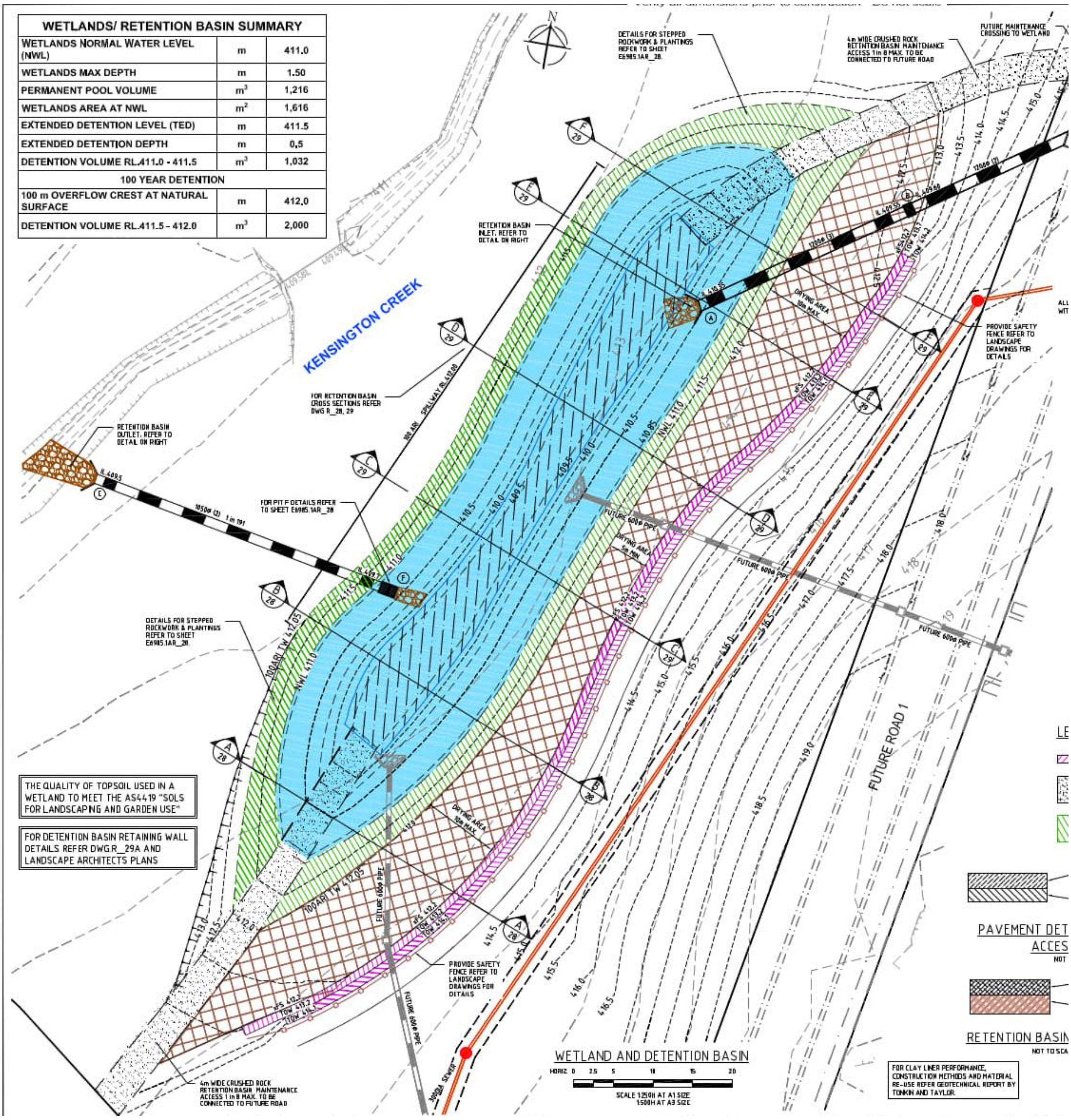

FIGURE 3.10: RETARDING BASIN 6B FUNCTIONAL DESIGN LAYOUT

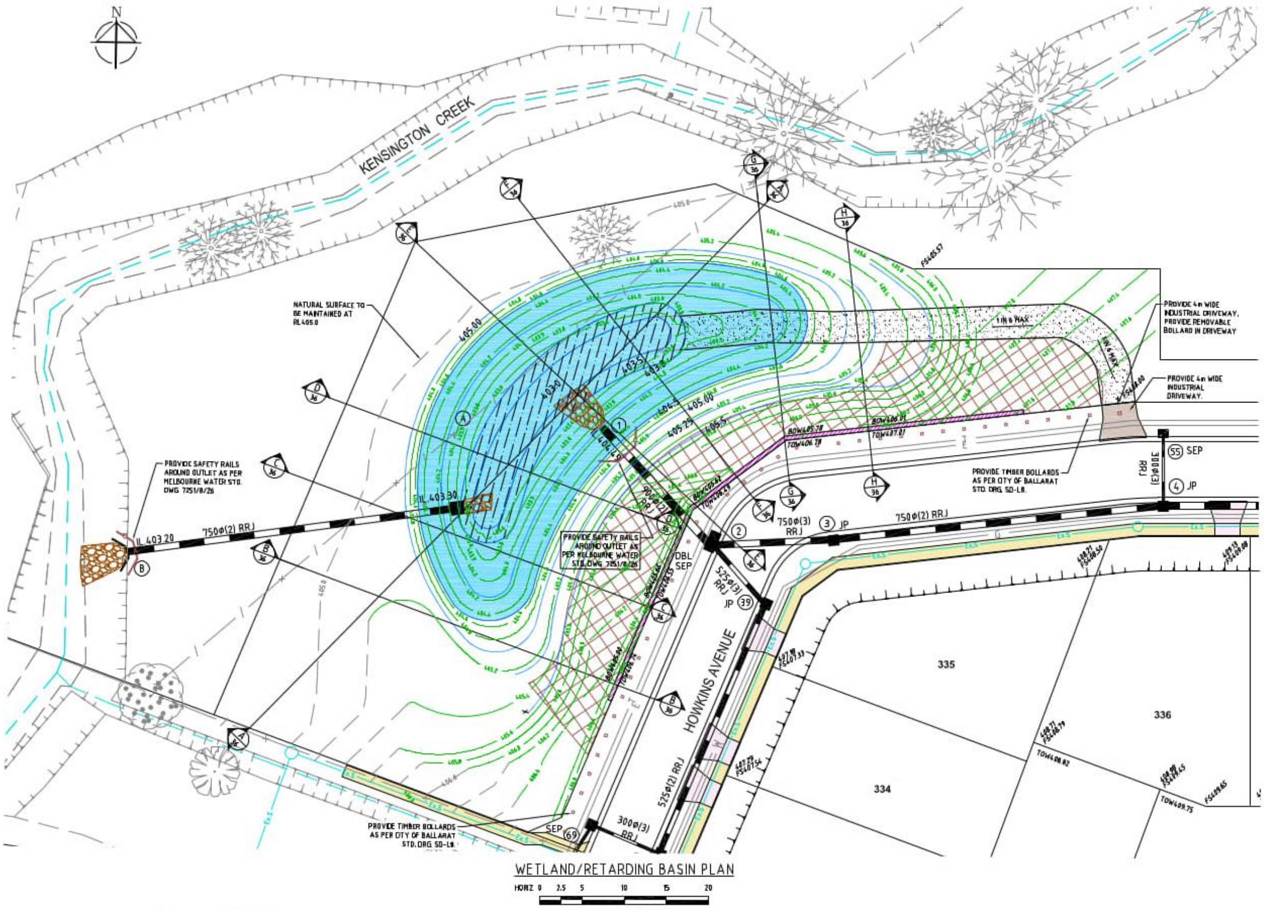

FIGURE 3.11: RETARDING BASIN 6C FUNCTIONAL DESIGN LAYOUT

3.4.7 RB7

Figure 3.12 shows the updated layout of RB7. The wetland and retarding basin are in the same general location as in the previous strategy, however the footprint has been expanded to account for the changes in wetland design standards (such as a reduction in extended detention depth from 0.5 m to 0.35 m, larger, dedicated areas for sedimentation drying, etc) and changes in the hydrology design from ARR 1987 to ARR 2019. Table 3.9 and Table 4.1 contain the key design criteria for the basin and wetland.

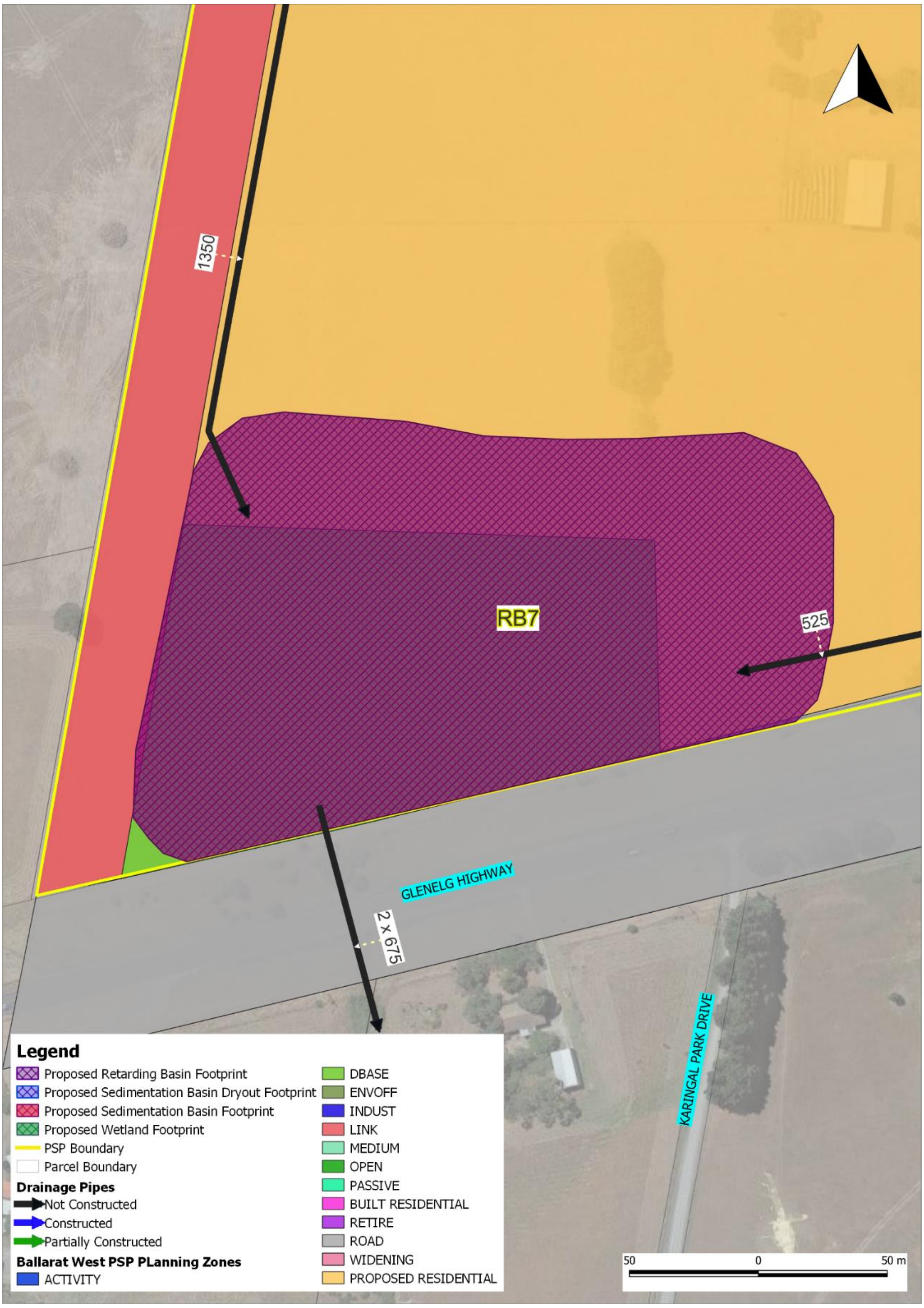

FIGURE 3.12: RETARDING BASIN 7 CONCEPT LAYOUT

3.4.8 RB11 and RB12

Retarding basins 11 and 12 have been moved and resized to help facilitate the staging of development. the catchments draining to RB11, RB12 and RB13 have also been adjusted. The original drainage strategy proposed that runoff from properties along Webb Road be piped south following the pre development fall of the land to Retarding basins 12 and 13 adjacent to Winter Creek. Pipe upgrades along Webb Road captured runoff from subcatchments north of Webb Road and divert piped flows to RB11 via the Cherry Flat Road Outfall Drain. These pipe diversions have been constructed because the areas to the south, where the 2011 strategy directed the pipe drainage, were not yet developing and therefore constructing pipes through these areas would be disruptive and expensive with the infrastructure not required in the short to medium term. The pipe diversions increase the flow to RB11 and RB12, and reduce the flow to RB13.

Figure 3.13 and Figure 3.14 shows the adopted layouts of RB11 and RB12. These figures have been sourced from the "Review of main drainage proposals for the precinct 1 MAC and Abiwood Lands – Version 3" by Neil Craigie dated 22/082016

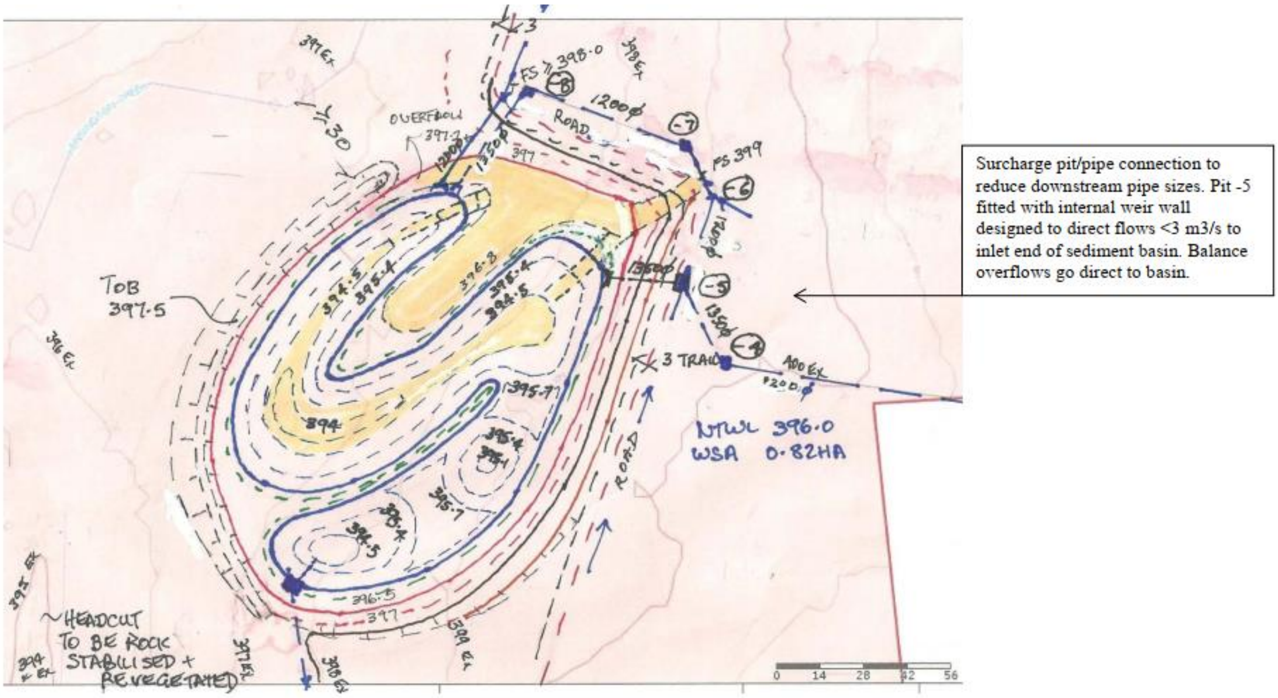

FIGURE 3.13: RETARDING BASIN 11 LAYOUT

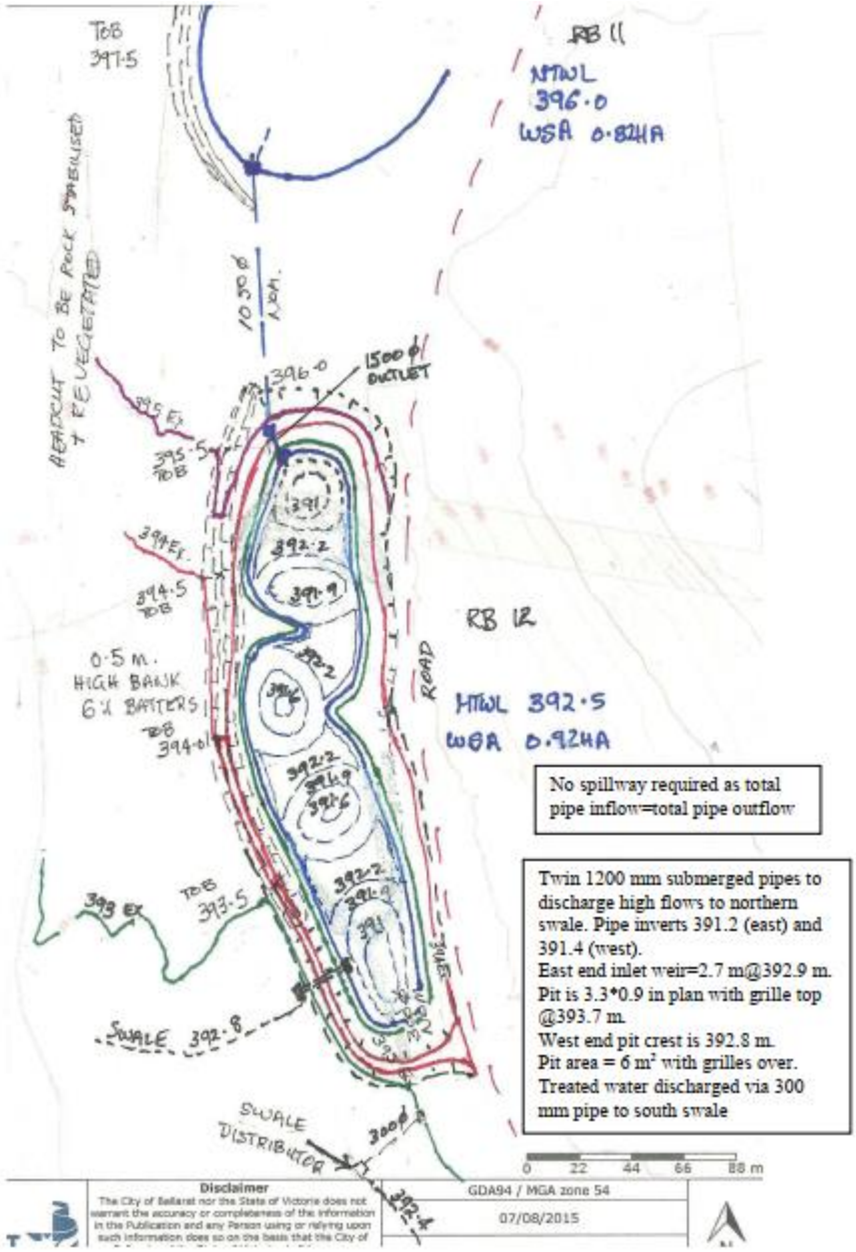

FIGURE 3.14: RETARDING BASIN 12 LAYOUT

3.4.9 RB13

As discussed in section 3.4.8 RB 13 has been resized to help accommodate staging of earlier development in north of the catchment around Webb Road. The catchment flowing to RB 13 has been reduced while the catchment flowing to RBs 11 and 12 was increased. The RB13 design has also been updated to account for the changes in the wetland design guidelines and the updated hydrological modelling. Figure 3.15 shows the updated layout of RB13. Table 3.9 and Table 4.1 contain the key design criteria for the basin and wetland.

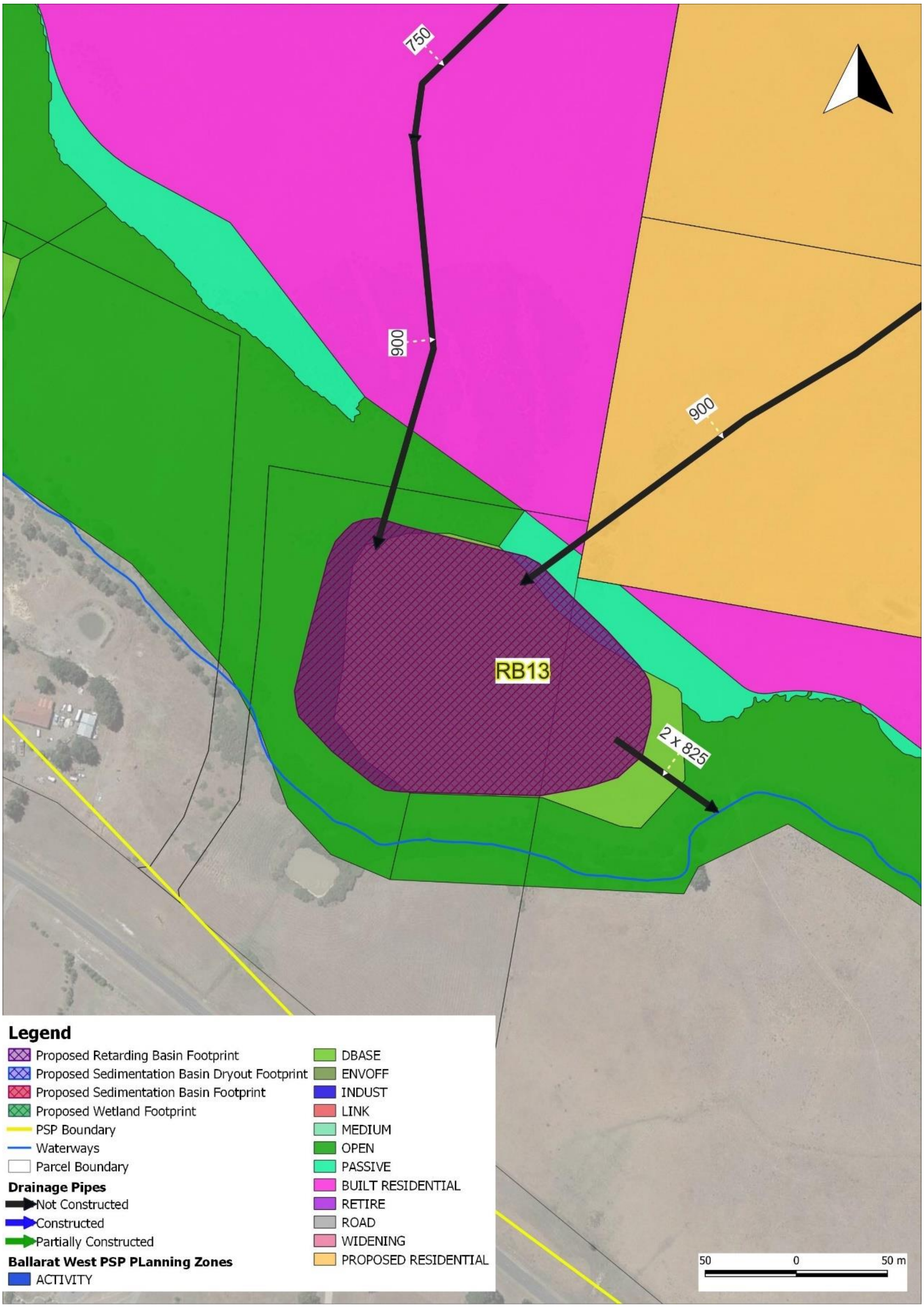

FIGURE 3.15: RETARDING BASIN 13 CONCEPT LAYOUT

3.4.10 RB14

Figure 3.16 shows the updated layout of RB14. RB 14 has been moved further west and is now proposed to be located within a single parcel. This move should assist with the development staging in the area and should help to simply the construction by reducing the need for multiple land owners to be involved. The basin is still located within open space adjacent to Winter Creek so there is no loss of developable area. The RB14 design has also been updated to account for the changes in the wetland design guidelines and the updated hydrological modelling. Table 3.9 and Table 4.1 contain the key design criteria for the basin and wetland.

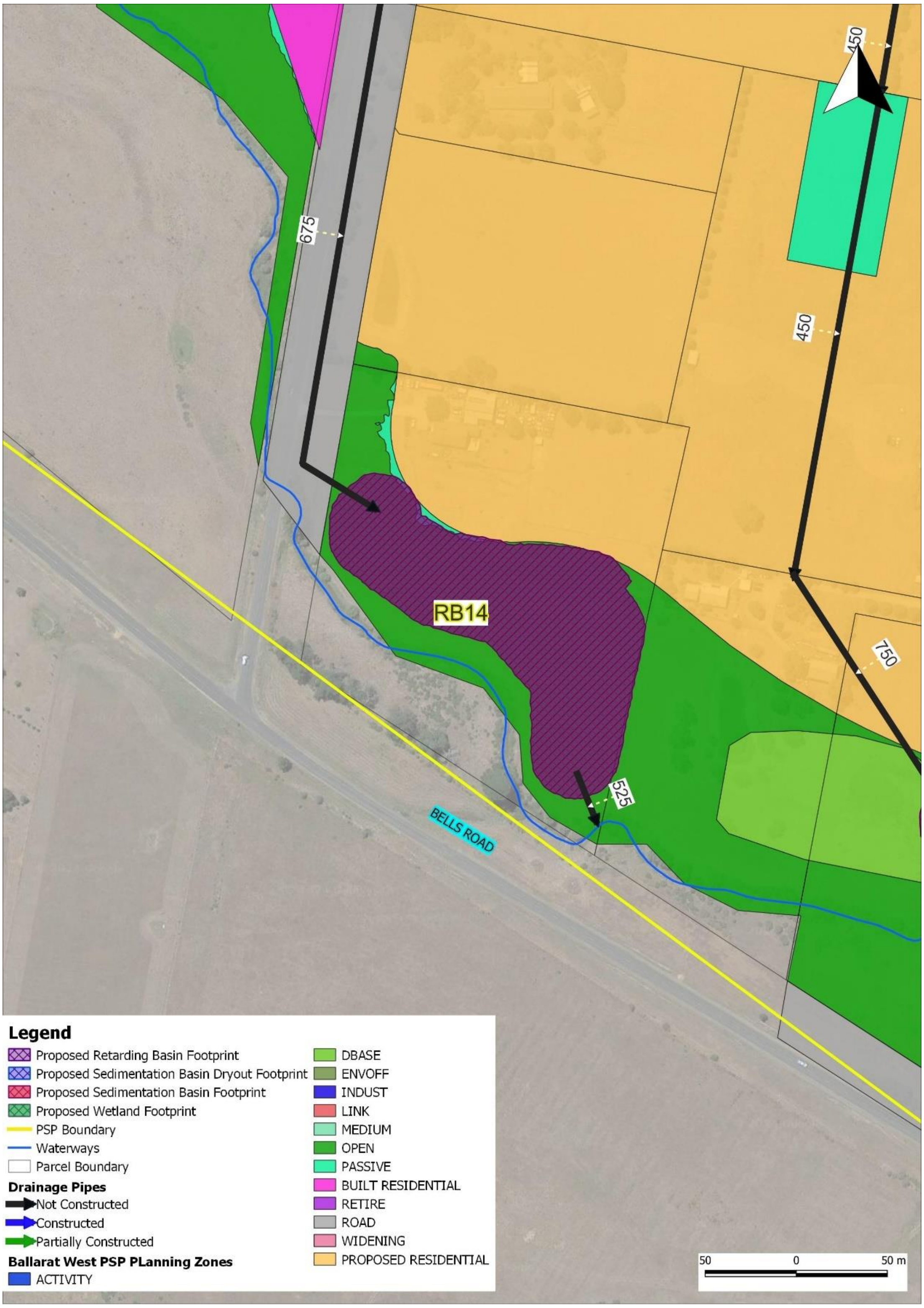

FIGURE 3.16: RETARDING BASIN 14 CONCEPT LAYOUT

3.4.11 RB15 and RB17

Figure 3.17 shows the updated layout of RB 15 and RB17. The proposed location and size of RB15 and RB 17 is very similar to the previous 2011 drainage strategy. The main change is that the footprint has been enlarged to respond to the updated design criteria in particular the lower extended detention depth in the wetland. Offsets from Winter Creek have also been further considered which has also adjusted the shapes slightly. Table 3.9 and Table 4.1 contain the key design criteria for the basin and wetland.

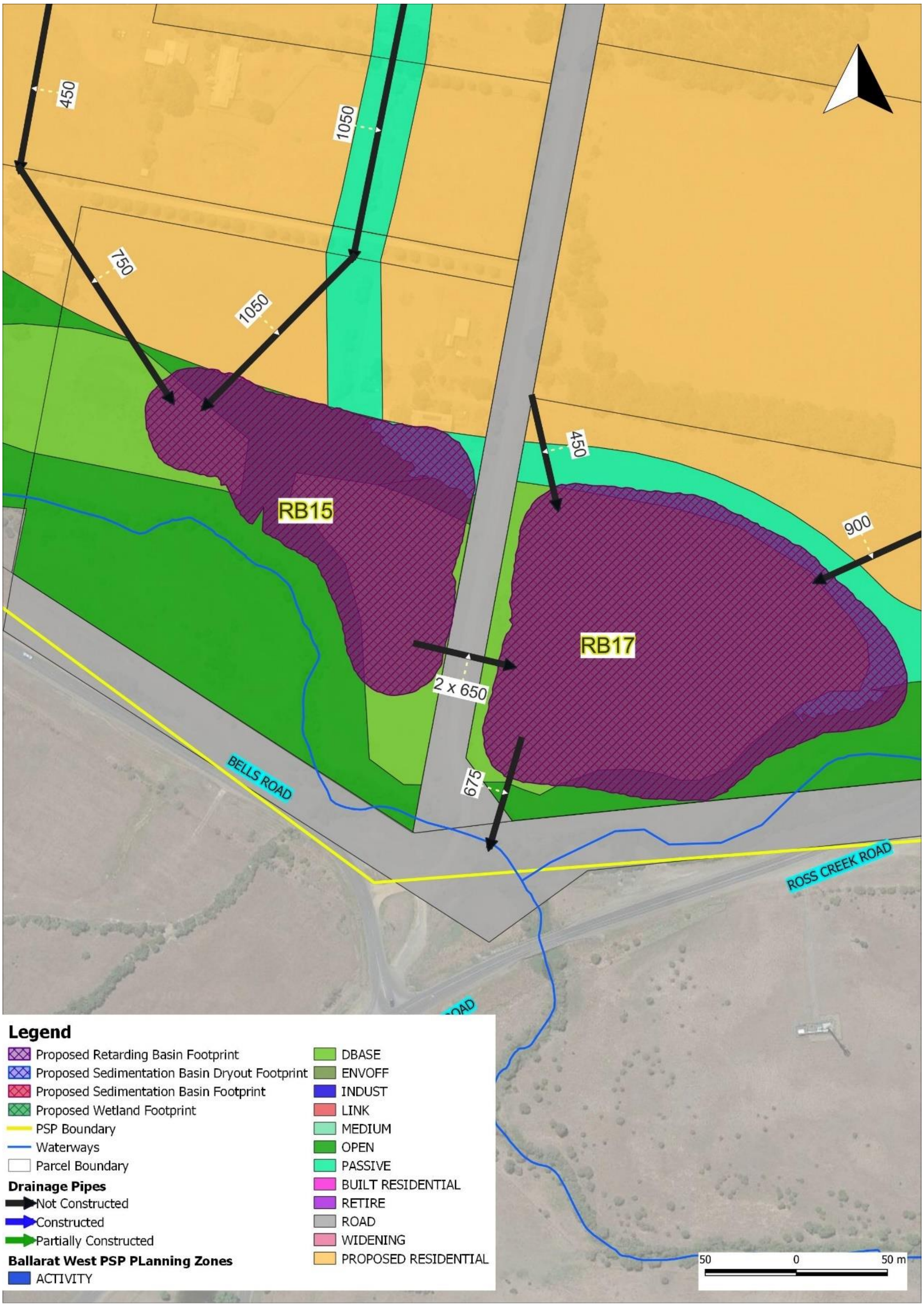

FIGURE 3.17: RETARDING BASIN 17 CONCEPT LAYOUT

3.4.12 RB18

Retarding basin 18 has been moved closer to Bonshaw Creek, enlarged and extended over two parcels. RB18 was moved to increase the catchment which can drain to it, allowing for better flow control and stormwater quality treatment. It's updated location also provides better connectivity between the wetland habitat and the creek habitat and corridor. It also helps to limit the number of drainage outfalls required into Bonshaw Creek and reduces the velocity of the flows discharging to Bonshaw Creek. The asset is currently partially constructed, with the northern section already built. The southern section will be built when the parcel on which it sits is developed. Figure 3.18 shows the layout of the retarding basin.

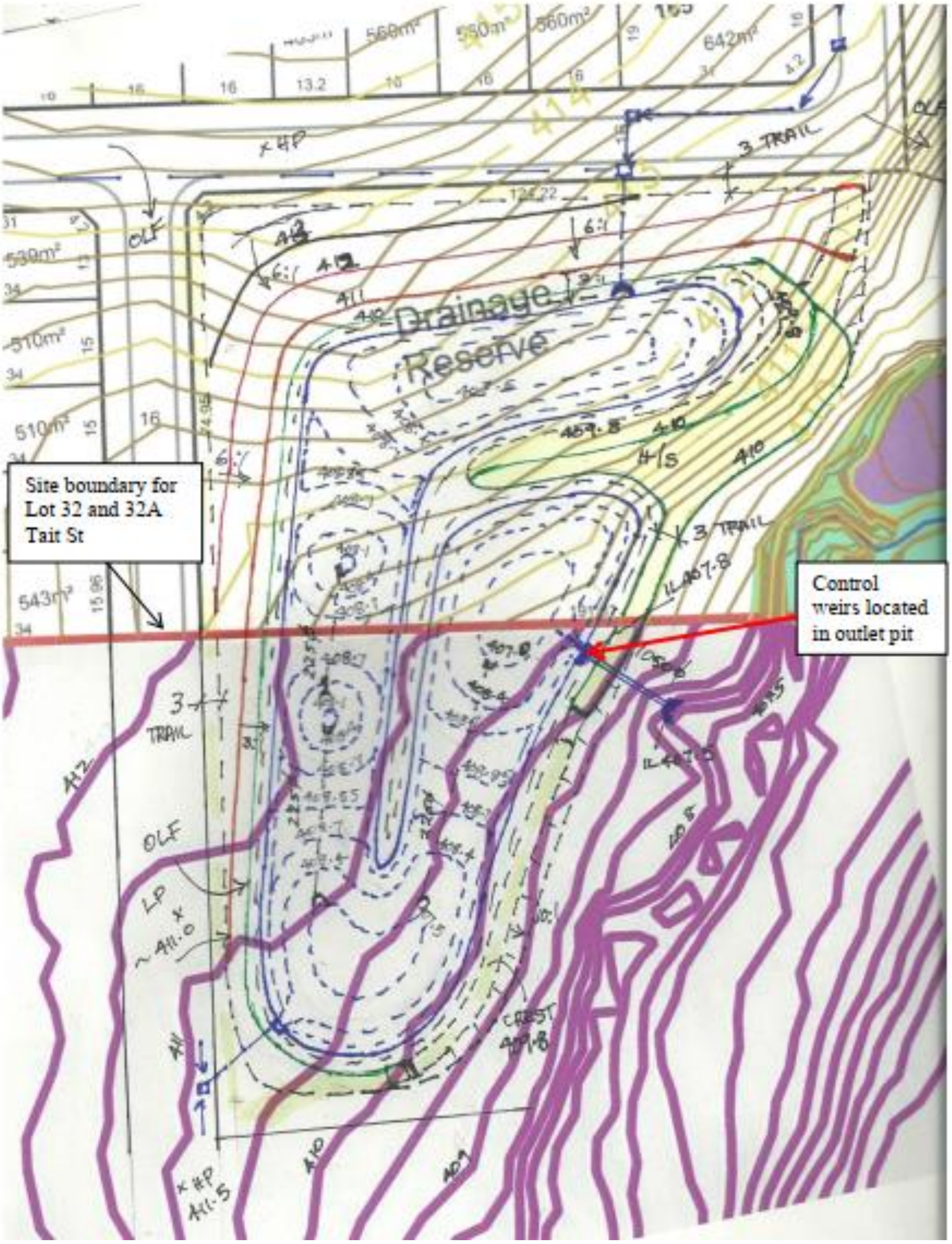

FIGURE 3.18: RETARDING BASIN 18 LAYOUT

Source: Lot 32 and 32A Tait Street, Bonshaw IWMS, Niel Craigie, 29/09/2015

3.4.13 RB24

Figure 3.19 shows the updated layout of RB24. The proposed location and size of RB 24 is very similar to the previous drainage strategy. The main change is that the footprint has been enlarged to respond to the updated design criteria with the key point being the lower extended detention depth in the wetland. Table 3.9 and Table 4.1 contain the key design criteria for the basin and wetland.

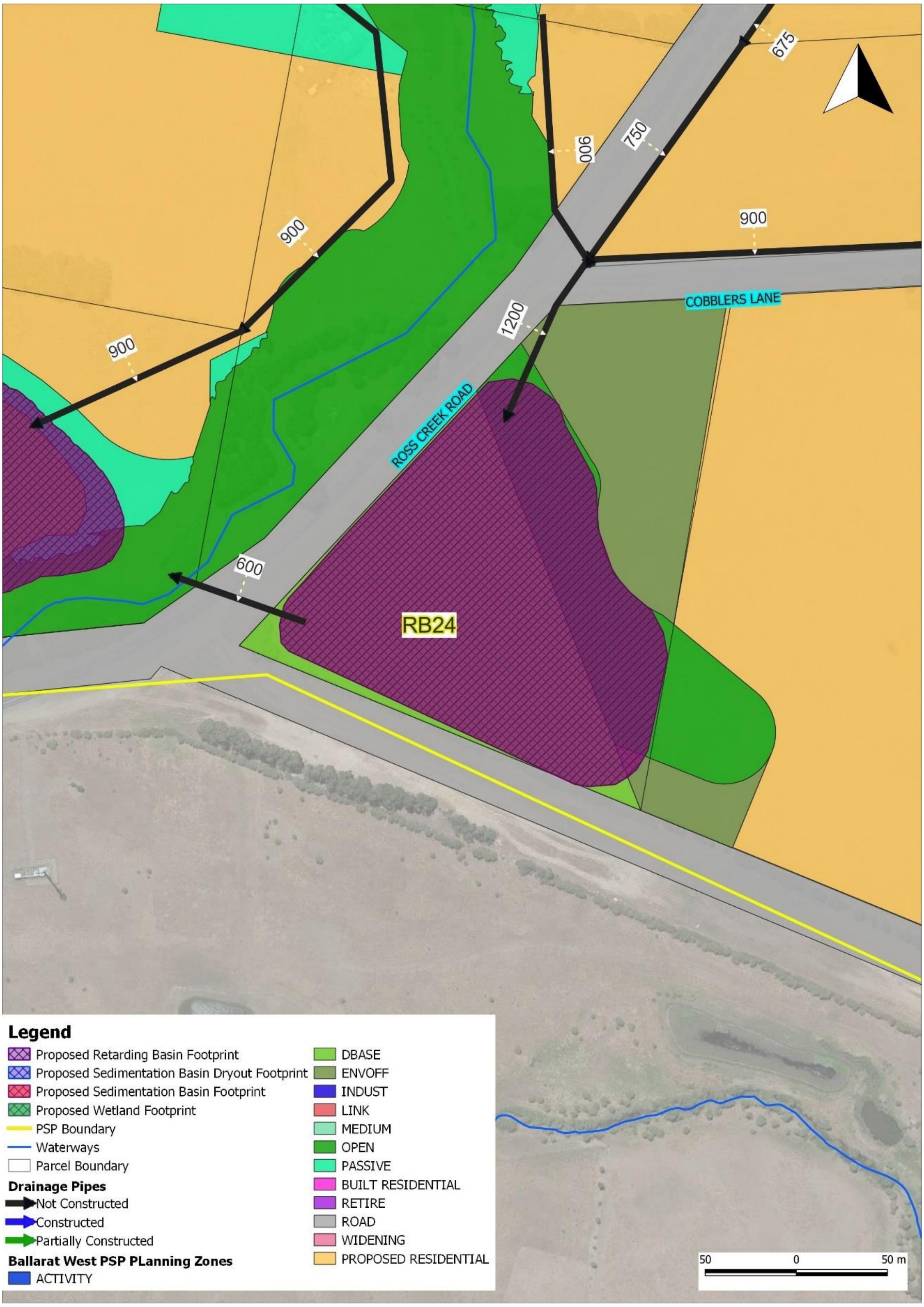

FIGURE 3.19: RETARDING BASIN 24 CONCEPT LAYOUT

3.4.14 RB25 and RB26

Retarding basins 25 and 26 have been consolidated into a single basin at the location where RB26 was proposed in the 2011 strategy. RB26 is larger than was proposed in the 2011 strategy. This change has been undertaken to allow for a reduction in the number of assets that Council will need to maintain and to improve the development layout of the estate in which the two proposed basins were situated. The two basins were relatively close together so this is a fairly minor change from what was proposed in the 2011 drainage strategy. Figure 3.20 shows the location of RB26. RB 26 has already been constructed.

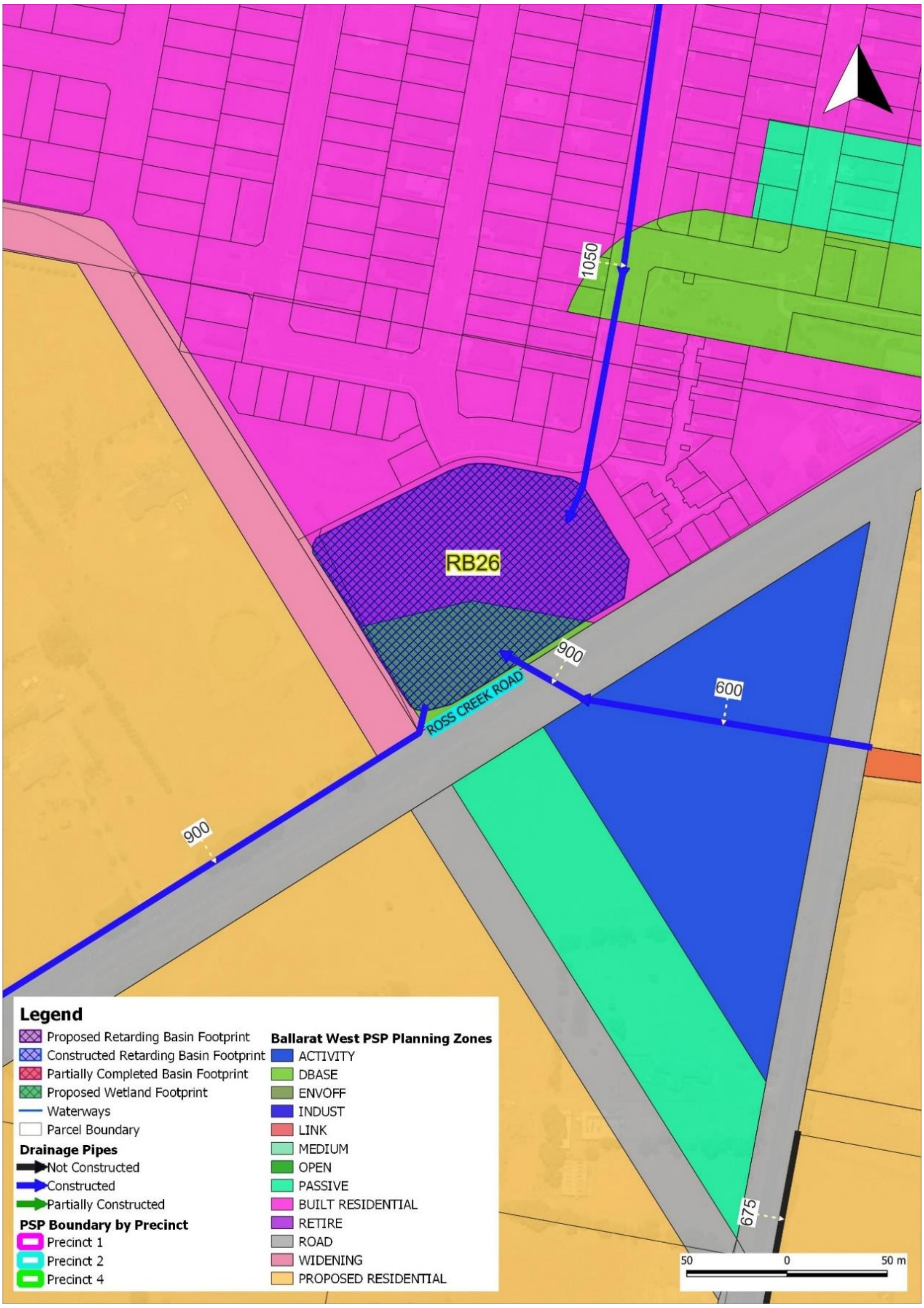

FIGURE 3.20: RETARDING BASIN 26 LAYOUT

3.4.15 RB27

Retarding basin 27 has been significantly reconfigured as part of this review. The wetland associated with the basin will remain as an offline asset on the western side of the waterway. Low flows only from the upstream catchment will need to be directed into the wetland for treatment. A sedimentation basin is also proposed on the eastern side of the waterway to provide primary treatment to the runoff from the catchments on the eastern side of the waterway. Only low flows (up to the 1 exceedance per year) would need to be conveyed to the sedimentation basin. Figure 3.21 shows the updated layout of RB7.

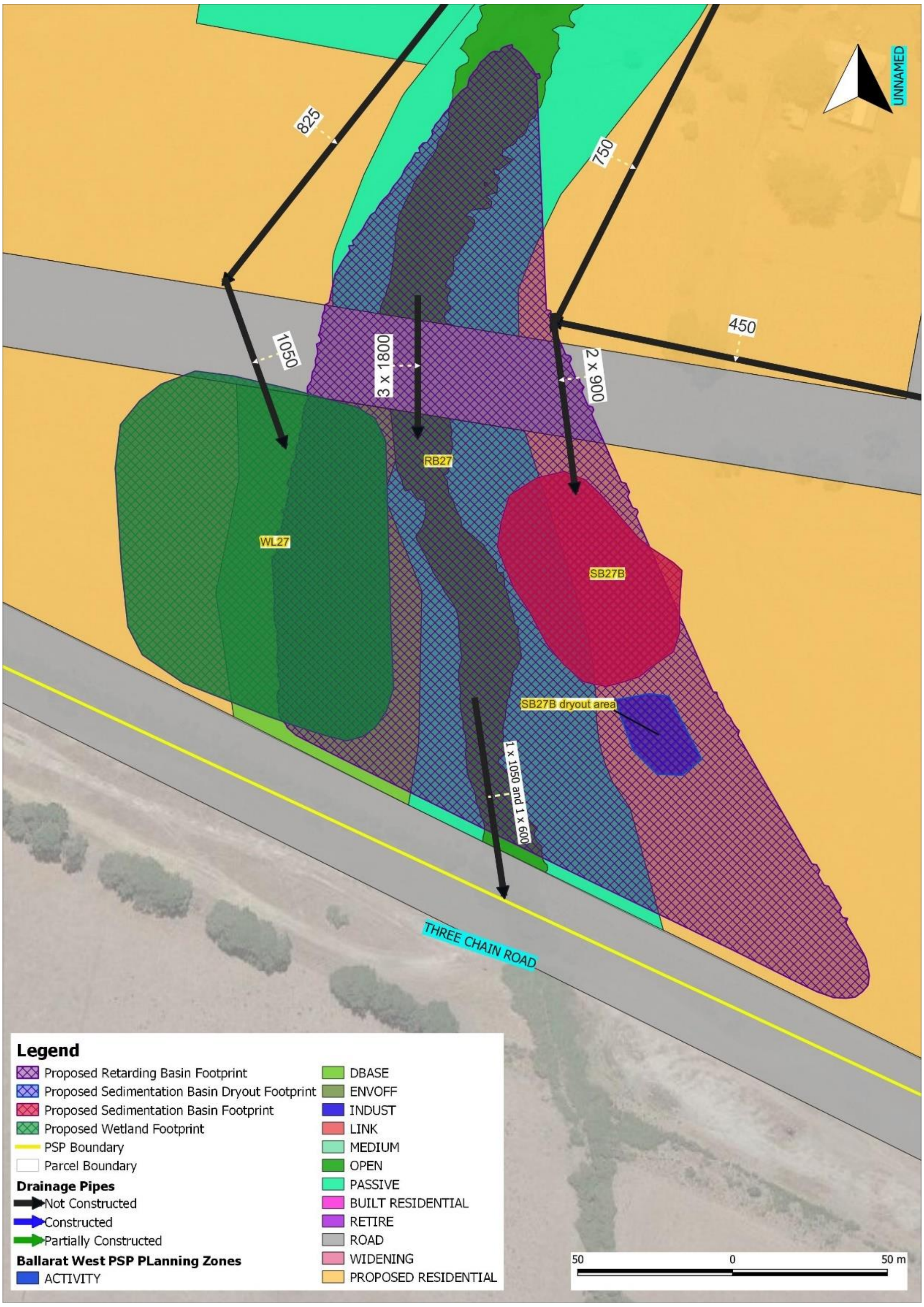

FIGURE 3.21: RETARDING BASIN 27 CONCEPT LAYOUT

For retardation an embankment across of the valley floor is proposed with culverts under the embankment providing the flow rate control. The embankment would need to extend to 388.1 m AHD. The 1% AEP flood level within the basin would extend to 387.8 m AHD. The embankment would be in the order of 5 m tall in the centre. No additional excavation is required behind the embankment wall to achieve the require storage. It has also been assumed that there is no storage available within the future road reserve which is north of Three Chain Road. It is expected that an embankment and culvert (sized to pass the unretarded 1% AEP flow) would be built within this road reserve, reducing the available storage. The retarding basin would also flood the wetland on the western side of the waterway and the proposed sedimentation basin on the eastern side of the waterway. The assets should be protected from flooding in up to a 10% AEP as part of the detailed design.

An embankment of this size creates an elevated level of risk associated with embankment failure (as compared to there being no embankment on the waterway). The land downstream of the embankment is within the Golden Plains Shire and is currently not zoned for urban development. Engeny understand that there is a proposal to undertake urban development in this area. If urban development proceeds in this area it will change the risk profile for the embankment compared with the current land use.

The retarding basin is able to achieve the required flow reduction to redeveloped flows so that there is no increase on the downstream section of waterway. This point is also the boundary between the City of Ballarat and Golden Plains Shire. The waterway flows for a few hundred metres before joining Winter Creek. Further hydrological analysis has revealed that there is no change in the peak flow on Winter Creek either with or without the retarding basin. The critical duration storm on Winter Creek is the 12 hour storm, while the critical duration for the catchment to RB 27 is the 1 hour storm. The peak flow from the local catchment is less than the retarded outflow peak flow rate from the 1 hour storm. This means that the retarding basin is meeting the drainage strategy requirement to not increase flows downstream. If the land directly downstream of the retarding basin is developed to urban housing then the proposed embankment does not represent an ideal outcome from a risk management point of view. It would be a better financial and engineering outcome if the waterway between Three Chain Road and Winter Creek could be protected or modified to convey the unretarded flow directly into Winter Creek. The hydrologic outcome to Winter Creek would be the same. It is recommended that the City of Ballarat explore this option with the proposed developer of the land, Corangamite CMA and Golden Plains Shire to establish if it would be an acceptable outcome to have an increase in flows between Three Chain Road and Winter Creek to avoid the need to construct the expensive embankment associated with Retarding Bains 27. The wetlands and sedimentation basin should be constructed regardless of if the embankment which forms the retarding function is completed or not.

Table 3.9 and Table 4.1 contain the key design criteria for the basin and wetland.

3.4.16 RB28

Retarding basin 28 has been constructed. the design of the basin has evolved from what was proposed in the original concept in the 2011 drainage strategy. Additional consideration has been given to the inverts of the incoming drains from Crown Street and the outgoing culverts and piped outfalls south under Morgan Street. The existing lake at the WorldMark Resort is to be retained (this was uncertain at the time that the 2011 drainage strategy was developed). Retaining this lake means that runoff must be directed to it to provide for suitable turnover to prevent water quality issues. The low flows from the wetland are being directed to the lake so that it received treated runoff to help maintain the water quality in the lake. Higher flows are being bypassed around the lake to help protect the structural integrity of the lake. The updated design of RB 28 also helps to minimise cut volumes and minimise disturbance in the area which contains historical tailings from mining operations. Figure 3.22 shows the location of RB 28.

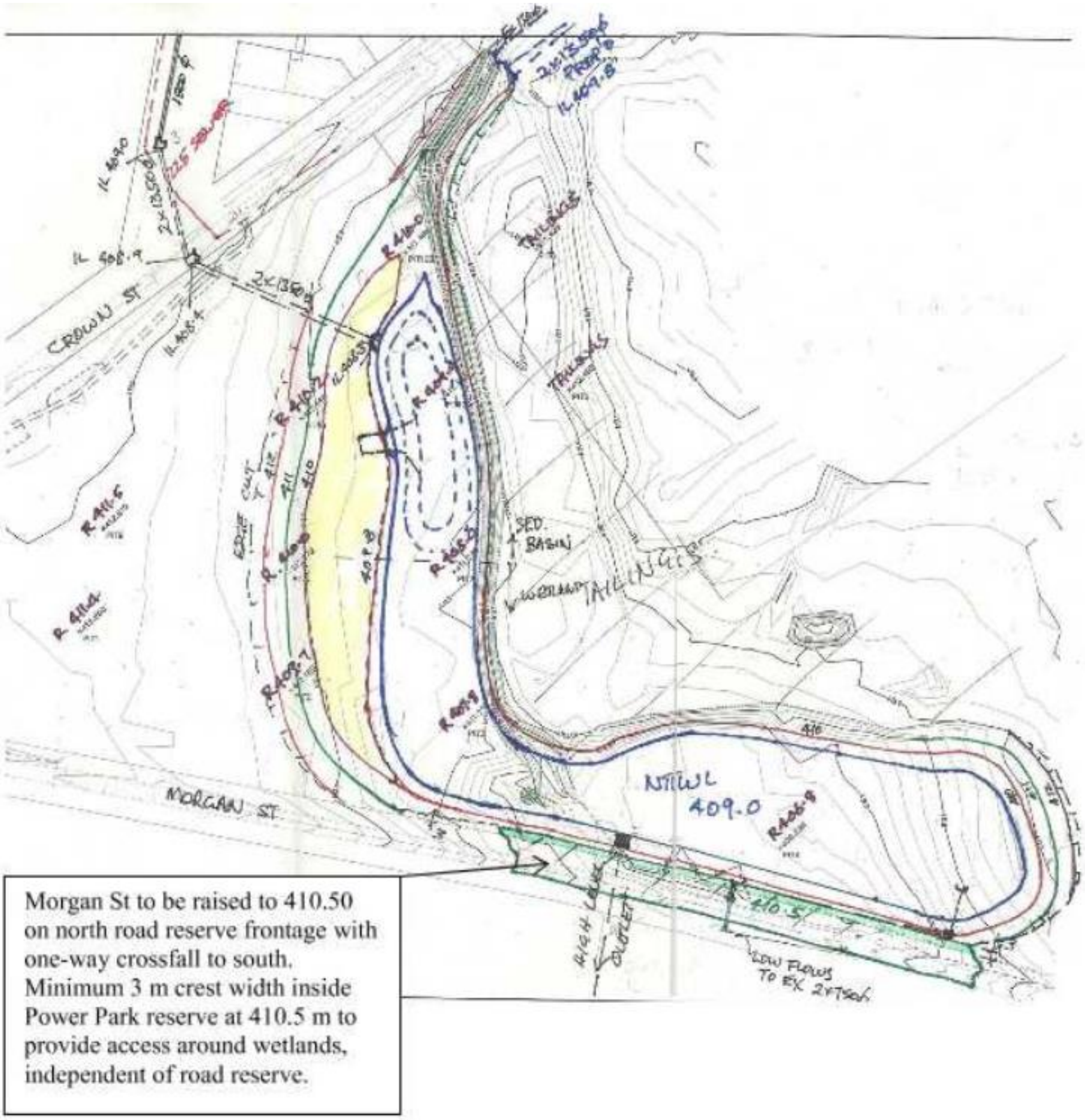

FIGURE 3.22: RETARDING BASIN 28 CONCEPT LAYOUT

Source: Review of Main Drainage Proposals for the Power Park catchment in Precinct 1, Neil Craigie, 25/08/2015

3.4.17 RB29

Figure 3.23 shows the updated layout of RB29. The retarding basin and wetland have been extended west to allow space for the maintenance paths sedimentation drying and the lower extended detention depth. Table 3.9 and Table 4.1 contain the key design criteria for the basin and wetland. RB 29 is larger than was proposed in the 2011 strategy and is taking land which was previous proposed as open space. It is also worth highlighting that since the 2011 strategy was completed this area has been identified as having heritage values (understood to be associated with historical mining) and also has the potential for ground contamination. The current costs estimate does not include an allowance to address these potential issues as they will need to be further investigated to understand the magnitude of the impacts.

FIGURE 3.23: RETARDING BASIN 29 CONCEPT LAYOUT

3.4.18 RB30

Retarding Basin 30 is proposed to be removed and replaced with a sedimentation basin (SB30) nearby and online to the existing unnamed tributary of Winter Creek. This concept was first proposed in work undertaken by Neil Craige in 2015 as part further design work completed on RB 28 in the MR Power Park Reserve. The lake at the WorldMark Resort is proposed to be retained. At the time when the 2011 drainage strategy was developed it was unclear what would happen to this lake. The lake has a large surface area, and while it is not designed specifically to retard flows it does have an attenuating effect on them. Given that it is now being retained and with the reconfiguration of retarding basin 28, the retarding function associated with RB 30 is no longer required. There is still a need for some stormwater treatment as no treatment is being claimed by the lake as it is an existing asset. SB30 treat a large catchment which is external to the development area and has little to no stormwater treatment at the moment. The credits gained from treating this runoff is used to offset pollutants generated within the development area. The net effect is the same or better on the receiving waterways as less untreated runoff is entering Winter Creek. Figure 3.24 shows the proposed layout of sedimentation basin 30. The basin is contained to the waterway reserve.

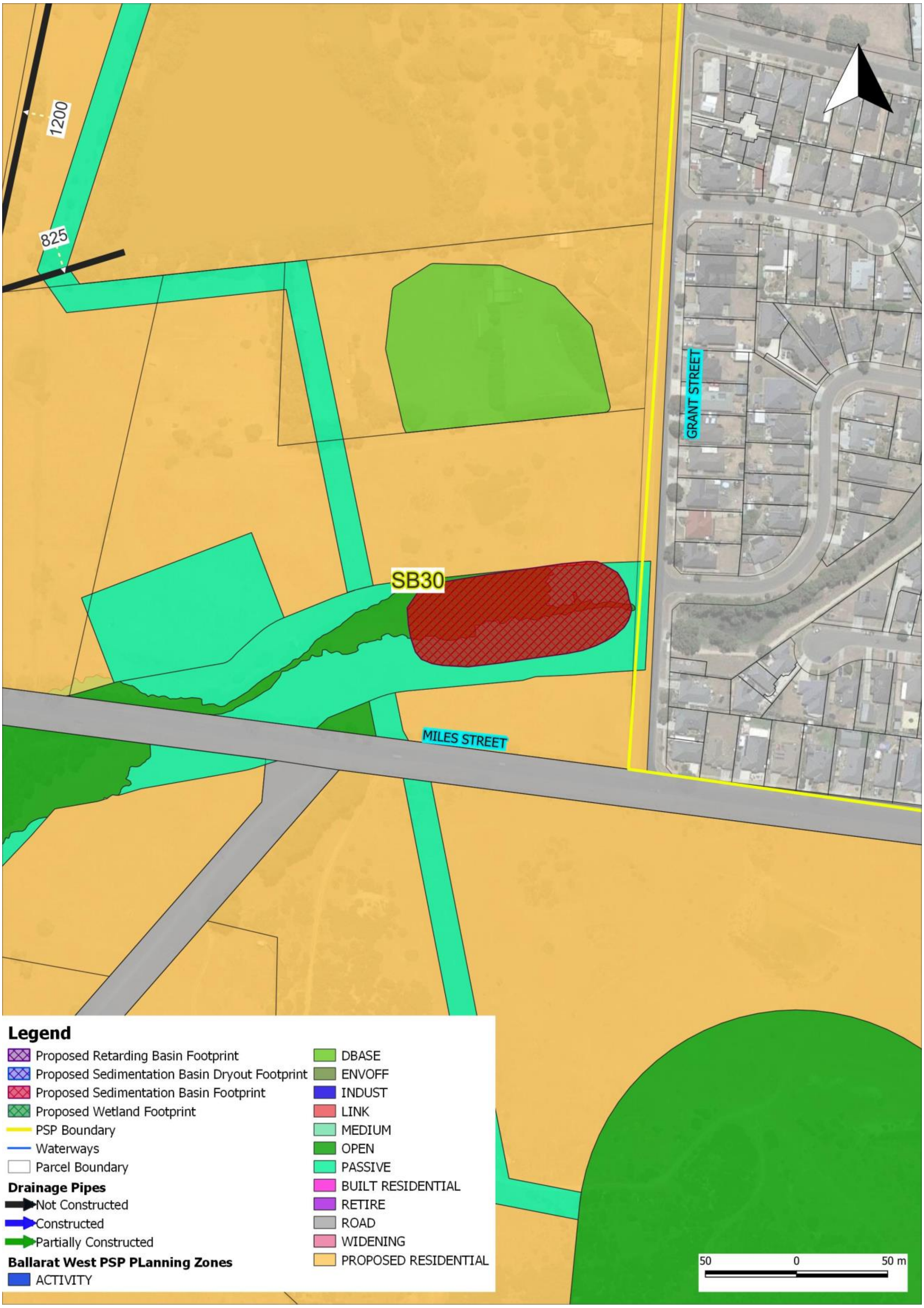

FIGURE 3.24: SEDIMENT BASIN 30 CONCEPT LAYOUT

3.4.19 Constructed or Committed Retarding Basins

Table 3.11 shows the details of the retarding basins which have already been constructed or committed within the PSP area.

TABLE 3.11: CONFIRMED RETARDING BASINS

Asset ID	1% AEP Storage Volume (m^3)	Outlet Configuration Weir (m AHD) or Pipe Diameter (mm)	Peak 1 % AEP Outflow (m^3/s)
RB1	4,680	Weir Outlet Weir 1. (Elevation – 440 Length – 0.3 m) Weir 2. (Elevation 440.5 Length – 1.2 m) Weir 3. (Elevation – 441.55 Length – 10.0 m) Pipe outlet – 1 x 600	6.30 (spillway)
RB2	38,100	Weir Outlet Elevation – 1.5 Length – 100.0 m	6.28 (spillway)
RB3	19,600	Weir Outlet Weir 1. (Elevation – 428.4 Length 0.3 m) Weir 2. (Elevation – 428.9 Length – 0.8 m) Weir 3. (Elevation - 430.3 Length – 50.0 m)	1.92 (spillway flow)
RB4	15,100	3 x 750	3.54 (pipe flow)
RB5	6,950	Weir Outlet Wear 1. (Elevation – 431.85 Length 10 m) 450	6.10 (pipe & spillway)
RB6	20,000	2 x 900	3.18 (pipe flow)
RB6A	7,830	1650	7.71 (pipe flow)
RB6B	1,260	1050	2.74 (pipe flow)
RB6C	184	750	1.31 (pipe flow)
RB7	19,600	2 x 675	2.57 (pipe flow)
RB11	17,900	Weir Outlet Weir 1. Elevation – 396.0 Length – 0.3 m) Weir 2. (Elevation – 396.5 Length – 2.8 m) Weir 3. (Elevation – 397.0 Length – 20.0 m)	5.57 (spillway flow)

Asset ID	1% AEP Storage Volume (m³)	Outlet Configuration Weir (m AHD) or Pipe Diameter (mm)	Peak 1 % AEP Outflow (m³/s)
RB12	23,500	Weir Outlet Weir 1. (Elevation - 392.5 Length – 0.2 m) Weir 2. (Elevation – 392.9 Length – 0.8 m) Weir 3. (Elevation - 394.45 Length – 60.0 m)	3.22 (spillway flow)
RB18	6,930	Weir Outlet Weir 1. (Elevation – 409.8 Length – 10.0 m) Pipe Outlet 1 x 600	3.5 (pipe & spillway flow)
RB26	7,190	1 x 900	2.63 (pipe flow)
RB28	26,300	1 x 1500 2 x 750	6.23 (pipe flow)

4. STORMWATER QUALITY

The Clause 56 of the planning scheme and Corangamite CMA requires the water discharged into existing waterways from urban areas is treated to the Best Practice Environmental Guideline Target for Stormwater Treatment. This requires that 80% of suspended solids, 45% of total phosphorus, 45% of total nitrogen be removed and 70% of gross pollutants be removed. To achieve these targets a range a water sensitive urban design (WSUD) techniques can be used, by incorporating a combination of Wetlands, Sediment Basins and Gross Pollutant Traps (GPTs).

The Ballarat West PSP drainage strategy includes a total of twenty wetlands and two stand-alone sediment basins (SB30 and a secondary basin within RB27) to achieve BPEMP objectives. Thirteen of these wetlands have been constructed or committed to construction and so the designs have not been updated as part of this project however Engeny has confirmed their makeup and contribution to the strategy. All treatment assets have been proposed to be located within the precinct boundary. Consideration has been undertaken to the consolidation of treatment assets by conveying flows to centralised locations, which also facilitates minimising piped outfalls into the waterways.

Inlet ponds for each wetland and the stand-alone sediment basins have been sized using the Fair and Geyer Equation. Typically, a 4 exceedances per year (EY) (3 month ARI) design flow is adopted in these calculations. A copy of the sedimentation basin sizing calculation sheets is included in Appendix B.

It has generally been assumed that each wetland will be constructed in cut. This makes achieving outlets from upstream drainage easier and is a conservative approach in terms of costing. The normal water level has been identified by Engeny based on both upstream and downstream level constraints and considering that at approximately one metre of storage depth is required above the extended detention depth of the treatment assets in order to provide some retardation of flows (peak flow control is discussed in Section3.4).

Engeny has sized the inlet pond area, sediment drying area and wetland treatment area for each asset. The sediment drying area has been estimated based on a sediment stockpile height of 0.5 metres in line with Melbourne Water's Wetland Design Manual. High level 12d modelling has been undertaken of the batter slopes (assumed to be 1 in5) and includes the allowance for a maintenance access track (4 m wide) around the wetlands. Further details such as wetland bathymetry, final wetland shape layout, sedimentation basin access path, high flow bypasses and landscaping have not been considered as part of this work. The total treatment footprint of the asset includes a buffer of an additional 20% of the wetland, sedimentation basin and sedimentation dry out area to allow for details discussed above but not explicitly modelled. It would be expected that the modelled wetland performance will improve when custom stage storages and outfall are added to the model at the functional design phase and that the additional space allowance should be suitable to incorporate the remaining design elements.

Table 4.1 summarises the key parameters for each treatment asset. It also provides a summary of the total footprint area for each asset at normal water level (NWL).

4.1 Wetlands

Table 4.1 shows the key design criteria of the remaining wetlands which have not yet been constructed or committed under the previous strategy work. Each of the wetlands serves a dual treatment and retardation purpose, with RB27 (discussed further in section 0) being the only asset proposing a significant embankment. All of the other assets have been assumed to be constructed in cut. Changes to the footprints may be required through detailed design, however it would be expected that where possible designs are generally in accordance with the concept designs or can be demonstrated to achieve equal or improved treatment performance outcomes. The column titled "Asset footprint (inc. battering and maintenance track)" is estimated total land take required for the asset.

TABLE 4.1: BALLARAT WEST PSP SEDIMENT BASIN AND WETLAND KEY DETAILS

Wetland	Total Catchment (ha)	4EY design flow (m^3/s)	Sed basin permanent volume (m^3)	Sed Basin Area (m^2)	Sed basin drying area (m^2)	Wetland Treatment area (m^2)	Asset footprint (inc battering and maintenance track)	Assumed NWL (m AHD)
RB7	75	0.75	600	800	702	12570	35800	405.2
RB 13	122	0.54	2000	2000	2429	8760	22400	387.5
RB 14	31	0.27	500	700	604	3830	13800	384.5
RB 15	65	0.34	1000	1200	1285	4010	16600	383.9
RB 17	22	0.32	400	600	437	12910	29500	383.9
RB 24	53	0.43	700	900	832	11530	28000	385.9
RB 27	32	0.43	500	700	506	2290	8100	386
SB27b	25	0.56	290	600	386	N/A	3300	385.5
RB 29	79	0.65	1000	1200	1244	9910	29400	390.8
SB 30 (RB30 has been replaced with a sedimentation basin in an adjacent location	100	1.00	1330	1500	1561	N/A	7300	401.0

4.2 Design Standards

It is recommended that as much as is practical, the wetlands and sedimentation basins are designed in accordance with the Melbourne Water Wetland Design Guidelines. If variations from these standards are required they should be considered by Council to determine if they improve the overall social and environmental outcomes of the wetland asset. Gross pollutant traps should be included upstream of all sedimentation basins and wetlands to help reduce the load of litter entering the systems. Council should be consulted as to which units they are able to maintain prior to detailed design of the units being completed.

4.3 Stormwater quality modelling results

The Model for Urban Stormwater Improvement Conceptualisation (MUSIC) computer software was used to model the proposed WSUD features. The model was setup using 6 minute rainfall data from the Ballarat Aerodrome Berea of Meterology station. The average annual rainfall of this station is 694 mm. The MUSIC model was run using 10 years of data between 1980 and 1989.

Engeny has updated the previous MUSIC modelling of precincts 1 and 2 to include the details of the revised concept design terrain modelling. This has resulted in some increases and some decreases in wetlands sizes, however overall there is a similar area of wetland treatment provided.

Table 4.2 shows the stormwater treatment targets which are required by the planning scheme and the EPA general environmental duty.

TABLE 4.2: STORMWATER QUALITY TREATMENT TARGETS

Pollutant	Pollutant Load Reduction Target
Total Suspended Solids	80%
Total Phosphorus	45%
Total Nitrogen	45%
Gross Pollutants	70%

Table 4.3 shows the stormwater quality treatment results for Precinct 1. Table 4.4 shows the results for Precinct 2, while Table 4.5 shows the combined results for Precincts 1 and 2. There are external and non developing sub-catchments which have been included in the Precinct 1 and 2 MUSIC models. There is no requirement for the PSP to treat runoff from those areas to best practice, however runoff from some of those areas does flow through PSP assets. The requirement is for the PSP to remove the amount of pollutants equal to the targets shown in Table 4.2 from the developing areas only. If pollutants are removed from the external developed catchments which have no stormwater treatment then this can be used to offset lower percentage removal from the PSP development area. As such the percentage reduction rate shown in the tables below is in reference to the entire model. The “percentage removed from development area” column in Table 4.3 and Table 4.4 contains the outcomes for the treatment achieved within the development areas in the PSP.

TABLE 4.3: PRECINCT 1 MUSIC RESULTS

	Source	Residual Load	Percentage Reduction Rate	Total from development area	Amount removed	Percentage removed from development area
Mean Annual Flow (ML/yr)	2522	2370	6.02	1896	152	8.0%
Total Suspended Solids (kg/yr)	511873	194000	62.1	385223	317873	82.5%
Total Phosphorus (kg/yr)	1041	503	51.7	784	538	68.7%
Total Nitrogen (kg/yr)	7260	4770	34.3	5459	2490	45.6%
Gross Pollutants (kg/yr)	115663	19200	83.4	86709	96463	111.2%

The following subareas from the precinct 1 model have been considered as external or non developing: KV, KT, KW, KU, KX, KY, KZ, LA, LE, LD, LF, LC, LB, LJ, LH, LI, LG. The pollutants generated from these subareas have been removed from the source totals when determining the percental removal from the development area.

TABLE 4.4: PRECINCT 2 MUSIC RESULTS

	Source	Residual Load	% Reduction Rate	Total from development area	Amount removed	% removed from development area
Mean Annual Flow (ML/yr)	185	175	5.39	132	10	7.6%
Total Suspended Solids (kg/yr)	34189	9060	73.5	23573	25129	106.6%
Total Phosphorus (kg/yr)	62	26	57.4	40	35	88.4%
Total Nitrogen (kg/yr)	429	275	35.9	276	154	55.8%
Gross Pollutants (kg/yr)	18871	1170	93.8	16426	17701	107.8%

The following subarea from the precinct 2 model have been considered as external or non developing subareas for the purposes of this modelling: DP, DO, EJ

TABLE 4.5:COMBINED PRECINCT 1 AND 2 RESULTS

	Source	Residual Load	% Reduction Rate	Total from development area	Amount removed	% removed from development area
Mean Annual Flow (ML/yr)	2707	2545	6.0%	2707	162	8.0%
Total Suspended Solids (kg/yr)	546062	203060	62.8%	546062	343002	83.9%
Total Phosphorus (kg/yr)	1103	529	52.0%	1103	574	69.6%
Total Nitrogen (kg/yr)	7689	5045	34.4%	7689	2644	46.1%
Gross Pollutants (kg/yr)	134534	20370	84.9%	134534	114164	110.7%

A summary of the performance of each individual wetland is included in Appendix C:.

4.4 Ballarat City Integrated Water Management Plan

Council and Central Highlands Water have developed an Integrated Water Management Plan in 2018. This plan commits to the following targets and goals in relation to planning for growth:

- *incorporate the Ballarat City IWM Plan as a reference document within the Ballarat Planning Scheme*
- *utilise preferred IWM strategies (such as stormwater harvesting, recycled water and actively used rainwater tanks) to drive water-wise development in designated areas*
- *consider design stormwater drainage to water street trees in development areas to utilise runoff as passive irrigation*
- *harvest stormwater for open space irrigation*
- *restore and plan to protect creeks in new development areas*
- *investigate partnerships for water-wise developments.*

Figure 4.1 shows the preferred IWM strategies for growth areas within Ballarat. The BWUGZ is the area covered by this drainage strategy. The key action identified in the legend is titled "stormwater to Winter Creek to Lal Lal" which refers to the concept of harvesting excess stormwater runoff from Winter Creek and directing it to the Lal Lal reservoir to be treated and mixed with natural runoff from the catchment. Lal Lal reservoir supplies water to Central Highlands Water and Barwon Water as part of Ballarat and Geelong's potable water systems.

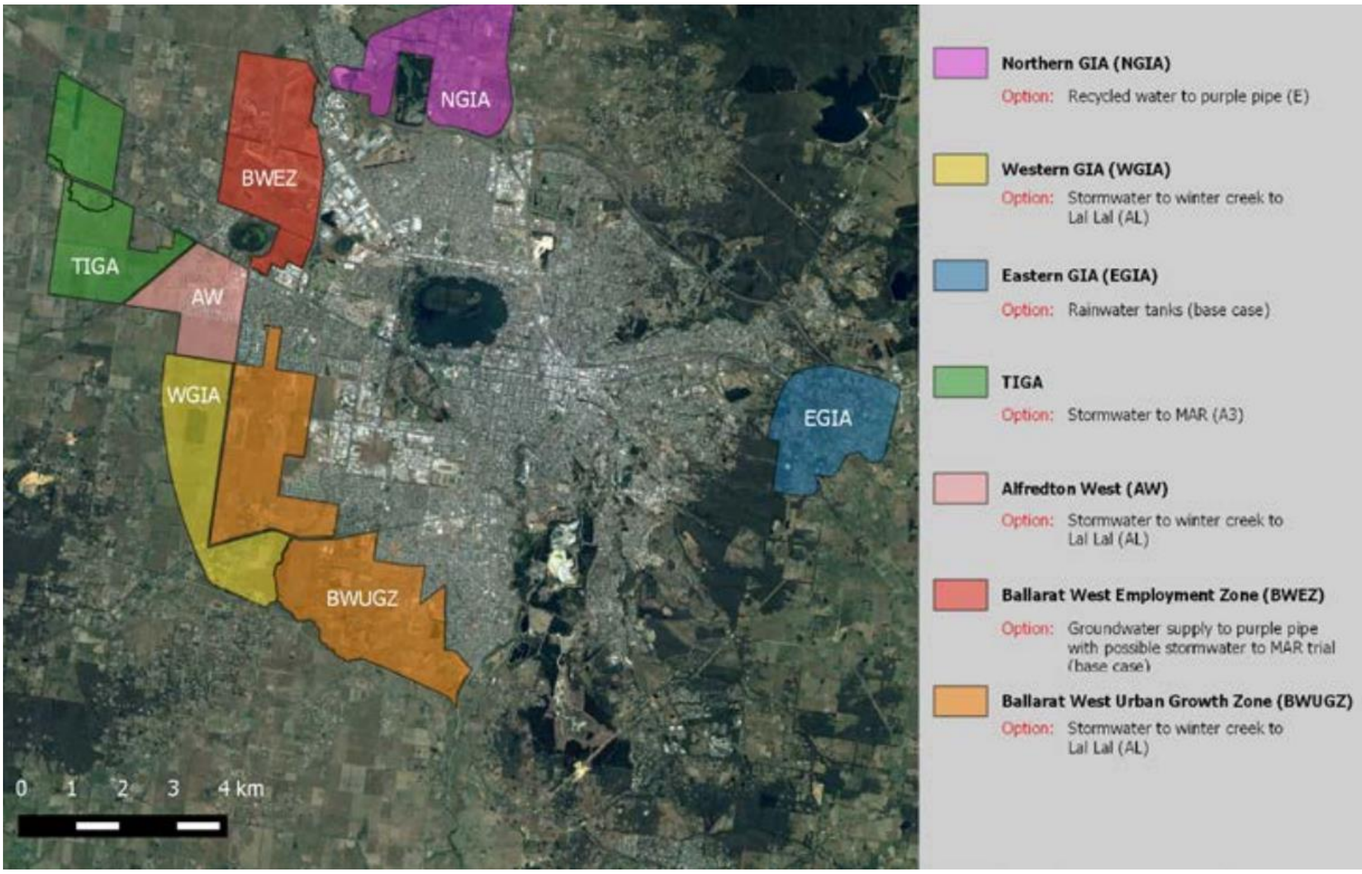

FIGURE 4.1: PREFERRED IWM STRATEGIES FOR GROWTH AREAS (SOURCE: BALLARAT IWMP)

The IWMP key goals that relate to this strategy are use of actively used rainwater tanks and stormwater harvesting for open space irrigation or other uses. Rainwater tanks are discussed further in Section 5 and represent one of the best options for reducing total runoff volume from the Ballarat West Growth area.

Stormwater harvesting is also a potential option that could be explored further within the Ballarat West Growth area. The ideal setup for large scale stormwater harvesting is to have pretreatment in a water sensitive urban design asset such as a wetland or raingarden and then a separate storage pond which can be sized to meet anticipated current or future demand. Having a standalone harvesting pond allows for complete draw down on the pond to empty (or as near to empty as is practically possible given pumping setups). This means that the maximum amount of water can be provided at the driest times of the year when it is most required. While this approach provides the ideal scenario there are significant capital expenditure and potential additional land take costs associated with this setup.

A secondary method for harvesting which can still be effective but may reduce the yield total yield of stormwater is to harvest directly from a wetland. The limitation with this approach is that the effective storage area is typically limited to a few hundred millimetres of depth in the wetland before there is risk of damaging or killing wetland plants by removing too much water. The deep pools in the wetland are usually connected by sub-surface pipe meaning the deep pools stay at same water depth. Drawing from 1 pond equally draws from all of the deep sections. To improve yield this might require a larger tank storage capacity at the sports precinct than typical so that water can be harvested when available (i.e. in wetter months) to avoid detrimental draw down.

Another option that is possible is to install a vertically adjustable weir in addition to the typical penstock slider to allow for variation in the normal water level or extended detention depth of the wetland depending on the demand for stormwater harvesting. An emerging space is the application of Smart Cities technology to achieve “Process Automation” and potentially water quality monitoring to minimise risk and enhance operational ease – the ingredients for proactive use. This might be applied to multiple wetlands in series to improve yield. For example – Wetland A holds back 5 cm of water above NTWL for harvesting purposes. When that is depleted the upstream Wetland B releases it’s held 5 cm down to Wetland A for harvesting purposes. This is an applied example of the “linked storage concept” in the IWM Plan 2018.

Planting species should be very carefully considered if this approach is taken, with a preference given to taller emergent macrophytes which can survive long periods of deeper inundation than the base design case for the wetland. It is also worth considering discussing with a Wetland Ecologist the need for a greater mix of species that recruit from rhizome, rather than reproducing from seed only to improve vegetation resilience. This may limit the plant species available for use in the wetland, however the potential trade off in terms of available water for harvesting could be significant. More attention to ecological monitoring and evaluation will also be required to ensure no negative impacts from unseasonal inundation.

Within the Ballarat West PSP the following wetlands present the best opportunity for stormwater harvesting due to the proposed land uses adjacent to the wetlands:

- RB 29 is directly adjacent to two proposed sporting ovals. This is an ideal situation for stormwater harvesting and this location should be prioritised as it has the source and demand centres for reusing water right next to each other minimising distribution costs.
- RB 4 which is currently under construction, close to completion, is also relatively close to proposed sporting fields which presents an opportunity for stormwater harvesting.
- Wetlands 15, 17 and 24 are all quite close together and are served by a large total catchment. There are no ovals or likely areas to irrigate directly adjacent to these assets, however given they are close together it could be possible to collect water from all of these wetlands and provide a single rising main to a demand source at one or multiple locations where sporting fields are proposed. It may be possible to gravity drain the low flows from wetlands 15 and 17 to Wetland 24 (or nearby to wetland 24) and then pump from a single location. This could be tied into the option of harvesting stormwater and pumping to Lal Lal reservoir should that proceed.
- An alternative option which Council could consider would be the use of floating wetlands, which can provide a higher level of stormwater treatment per square metre than a traditional wetland. This would free up land from a traditional wetland to allow for a harvesting pond. Floating wetlands also have higher maintenance costs and maintenance risks compared to a traditional wetland due to a need to undertake more activities near deeper water. By using a floating wetland the remaining land within the footprint of a proposed wetland could be converted to a harvesting storage pond. This could be especially effective in the area near wetlands 15, 17 and 24 as three large wetlands are proposed in close proximity and it may be possible to divert low flows from more than one wetland into a harvesting pond adjacent to a floating wetland. There is no open space directly adjacent to these assets which means that water would likely need to be pumped to a reuse location.

There is also a role in the PSP more broadly around the protection and enhancement of existing waterways. Wherever possible Council should look to work with the developers of properties adjacent waterways to ensure that:

- Appropriate setbacks to waterways are maintained to allow for a riparian habitat zone to be established and protected.
- Development that is “fronted on” to a waterway has a road between proposed dwellings and the waterway. This significantly improves access to and passive surveillance of the waterway, reducing the likely of illegal dumping and promoting community interaction and ownership of the waterway. This also creates the opportunity for shared use paths along side the waterway corridors to help improve opportunities for passive recreation, liveability and connectivity between public assets like schools and social services.
- Planting or revegetation of the riparian habitat is undertaken as part of the development, or that existing riparian habitat is protected. This vegetation provides crucial links for wildlife and can also help protect the waterway from erosion, reducing the future maintenance burden to Council.

5. GENERAL ENVIRONMENTAL DUTY

In 2017 the Victoria Environmental Protection Act was updated. A key part of the change to the Act was the introduction of the General Environmental Duty (GED). Under the GED all businesses have a responsibility to reduce the risk that they will cause harm to people or the environment. For the context of this report the key focus under the GED is how stormwater runoff is managed. This includes at all stages of development, including construction and post construction when the development work has been completed and greenfield areas become a functioning residential or commercial area. This report only focuses on the post construction goals, however compliance with the GED during construction is also very important.

Victorian Environmental Protection Agency (EPA) publication 1739.1 “Urban Stormwater Management Guidelines” provides advice on how to manage the risk of pollution from stormwater runoff. Table 1 of the document also sets out the quantitative performance objectives for urban stormwater. A reproduction of the table and notes is included below in Figure 5.1

Indicator	Performance objective				
Suspended solids	80% reduction in mean annual load (Note:1)				
Total phosphorus	45% reduction in mean annual load (Note:1)				
Total nitrogen	45% reduction in mean annual load (Note:1)				
Litter	70% reduction of mean annual load				
Flow (water volume)		Priority areas (Notes 2, 4, 5, 6)		Other areas (Notes 3, 4, 5, 6)	
	rainfall band (ml)	Harvest/evapotranspire (% mean annual impervious run-off)	Infiltrate/filter (% mean annual impervious run-off)	Harvest/evapotranspire (% mean annual impervious run-off)	Infiltrate/filter (% mean annual impervious run-off)
	200	93	0	37	0
	300	88	0	35	0
	400	83	0	33	0
	500	77	5	31	4
	600	72	9	29	7
	700	68	11	27	9
	800	64	14	26	11
	900	60	16	24	13
	1000	56	18	22	14
	1100	53	19	21	15
	1200	50	21	20	17
	1300	48	22	19	18
	1400	46	23	18	18
	1500	44	25	18	20
	1600	42	26	17	21
	1700	40	27	16	22
	1800	38	28	15	22

FIGURE 5.1: QUANTITATIVE PERFORMANCE OBJECTIVES FOR URBAN STORMWATER (VIC EPA 1739.1)

Notes to Figure 5.1 (source Vic EPA 1739.1):

(1) *'Reduction in mean annual load' refers to the reduction in load discharged from the development with management. This is compared to the load that would be discharged without management. Load (or pollutant load) means the mass per unit time of an indicator/pollutant.*

(2) *These areas are priority areas for enhanced stormwater management. They have high ecological value waterways. The Melbourne Water Healthy Waterways Strategy identifies these areas. A map of them can be found here: https://data-melbournewater.opendata.arcgis.com/datasets/hws2018-stormwater-priorityareas. Note the map needs to be downloaded to distinguish the urban areas.*

A transparent process is required to identify priority areas for enhanced stormwater management outside the greater Melbourne area. Urban stormwater management guidance 9

(3) *These objectives are to help arrest further degradation in these areas. To restore a waterway to pre-urban conditions, in an already degraded environment (highly modified waterway), it is likely that the priority objective or better would need to be applied.*

(4) *Mean annual impervious run-off volume refers to the percentage of run-off from the impervious surface.*

(5) *Note, council or other authorities may have specific requirements that will apply, for example, on-site detention requirements.*

The infiltration performance objective may be inapplicable if the site is subject to requirements in an EPA permission directing that stormwater infiltration be minimised or is subject to an environmental audit statement that restricts stormwater infiltration. Victoria's planning framework includes requirements to identify potentially contaminated land at the planning scheme preparation/amendment stage and to manage any potential risks, including via EPA's environmental audit system. More information is available on DELWP and EPA websites.

(6) *For further understanding about how to model objectives, see Healthy Waterways Strategy Stormwater Targets: Practitioners Note (https://www.melbournewater.com.au/building-and-works/developer-guidesand-resources/guidelines-drawings-and-checklists/guidelines)*

The table includes the same pollutant reduction targets that have existed in the Victorian Planning Scheme for many years, with the focus being on the reduction of suspended solids, nitrogen and phosphorus from runoff before it enters the receiving waterway. The new addition to these targets is the flow (volume) reduction targets. The mean annual rainfall in Ballarat is 687 mm per year (Ballarat aerodrome station number 089002). It is understood that there are currently no priority waterways set within the Corangamite Catchment Management Authority's (CCMA) catchments, which includes the Winter Creek catchment which Ballarat West development area drains to. This means that the flow reduction targets for the Ballarat West PSP area are a 29% reduction via harvesting or evapotranspiration and 7% infiltration for a total of 36% reduction in flows discharged to the waterway from the developed catchment.

The Ballarat West PSP area has already been developed for many years prior to this review. This means that a large amount of the infrastructure has already been constructed. In these areas it is not seen as reasonable or practical to try and achieve the new targets. Equally some catchments are currently partially developed, which also makes the achievement of these targets unlikely.

Engeny's understanding is that the requirement is to achieve the flow reduction targets under a framework considering what is reasonably practicable. This means that there may be cases where the targets are not achieved and the GED is considered to be being met, however it would need to be demonstrated that everything reasonably practicable has been done to achieve the targets.

Engeny also notes that current engineering practice is still being updated with guidance on how to construct stormwater treatment assets which focus on flow reduction rather than just on stormwater treatment, however many existing practices are available and should be used to demonstrate compliance with the GED. In the context of this PSP, there are also limitations around previously proposed asset sizes and a desire to avoid significant changes to the PSP at this late stage in its development.

The Urban Stormwater Management Guidelines (Vic EPA, 2021) highlights that a range of measures will be required to meet the flow reduction targets set under the GED. This means that in addition to the works proposed under the drainage strategy, additional measures are likely to be required at a lot level scale in order to mee the GED. The simplest additional measure to implement is including rainwater tanks on each dwelling which are plumbed to flush toilets and potentially also possibly to some laundry uses, in addition to garden watering.

5.1 Rainwater tank modelling

Engeny has modelled 4 different rainwater tank size and reuse combinations to provide some guidance on the likely reduction in flow volumes that can be achieved by using rainwater tanks.

The scenarios modelled were:

- 2 kilolitre tank plumbed to toilet flushing only
- 2 kilolitre tank plumbed to toilet, laundry and used for irrigation
- 4 kilolitre tank plumbed to toilet only
- 4 kilolitre tank plumbed to toilet, laundry and used for irrigation

The following assumptions were made in the modelling. Adjustments to these assumptions would change the effectiveness of the harvesting.

- 20 houses per hectare
- 100 m^2 of roof area for each property connected to each individual rainwater tank
- Total impervious fraction of the development 75%
- Toilet flushing uses 20 litres per person per day
- Laundry usage is 15 litres per person per day
- Irrigation use is a fixed 60 litres per day
- 2.7 people are assumed to live in each house

Using these assumptions, the reductions in total runoff volume shown in Table 5.1 can be achieved from 1 hectare of urban development.

The goal for new development in Ballarat is to achieve a 29% reduction by harvesting or evapotranspiration and a 7% reduction by infiltration. Table 4.5 shows that the precinct scale infrastructure is able to achieve an 8% reduction in volume (Mean Annual Flow), largely via evapotranspiration from the proposed wetlands. Additional reductions would be possible if stormwater harvesting projects are implemented using the wetlands as a source of water. The exact reduction achieved will depend on the scale and setup of the harvesting project and could be determined as part of the design process. If the proposed infrastructure (without any stormwater harvesting) is combined with the removal rates from using rainwater tanks a total reduction in flows of up to 38% may be possible. Table 5.1 shows the reductions in mean annual flow that can be achieved in areas which are not yet developed if rainwater tanks are plumbed to internal reuse demands. It is not proposed that rainwater tank harvesting be applied retroactively to the areas of the PSP that have already developed in the same way that it is not proposed to increase or adjust the size of retarding basin and wetland assets which have already been constructed as it was not a requirement at the time that the dwellings or assets where constructed. Meeting these targets should be considered and address in areas which have not yet been developed.

The GED applies to all Victorians, including developers and the City of Ballarat. It is not up to Council on its own to demonstrate that these targets can be met (or why they cannot be met) the requirement also falls to the developers who are undertaking the change, which will have the impact, to demonstrate how they can meet the GED or why it cannot be reasonably met.

TABLE 5.1: RAINWATER TANK FLOW REDUCTIONS TABLE

Rainwater tanks	Percentage reduction in flows	Percentage of reuse demand met
Only Toilets 2 kL tank	10.9	98.6
Only Toilets 4 kL tank	11.1	100.0
Toilets, Laundry and Irrigation 2 kL tank	26.8	83.9
Toilets, Laundry and Irrigation 4 kL tank	29.8	93.5

6. COST ESTIMATES

Engeny has updated the designs of the wetlands, retarding basin and pipe assets which have not yet been constructed or committed in the Ballarat West PSP. The costs of the associated assets have also been updated to reflect any changes in asset footprints or length / size. Costs have been based on original base costing rates and methodology. Costs have been increased by 37.4% in line with the change in the road and bridge construction price index published by the Australian Bureau of Statistics. This increase is to March 2023.

In addition to increasing the base costs by the road and bridge CPI additional cost factors have been included to cover the delivery items shown in Table 6.1. The rates used have been taken from the VPA Benchmark Infrastructure Costing Report and are the applicable rates for culverts (the only drainage item listed in the VPA Benchmark cost report).

TABLE 6.1:DELIVERY ITEMS COSTS (% OF BASE COST)

Delivery item	Percentage of base cost
Council Fees	3.25
Authority Fees	1
Traffic Management	5
Environmental Management	0.5
Surveying and Design	5
Supervision and Project Management	9
Site Establishment	2.5
Contingency	15
Total of Delivery items	**41.25**

The 2011 drainage strategy applied delivery fees which totalled 38.25% (3.25% Council fees, 15% Design/consultancy fees, 20% contingency) to wetland and retarding basins and fees of 28.25% (3.25% Council fees, 15% Design/consultancy fees, 10% contingency) to the drainage pipes The updated delivery fees are a similar overall percentage and are now aligned to the fees in the VPA Benchmark Infrastructure Costing Report.

Table 6.2 shows the pipe costs and that status for each drainage pipe within the PSP. Each asset is given one of the following the statuses.

Altered – asset size has been altered from the 2011 strategy.

No change – asset size has been maintained from the 2011 strategy.

Built – asset has been built in line with the 2011 strategy.

Review Pipe Built – asset built although altered from 2011 strategy.

Removed – asset has been removed from strategy.

Table 6.3 shows the wetland/retarding basin costs. The plans in Appendix D: show the location of each of the assets.

TABLE 6.2: PIPE COSTS

Asset ID	Diameter (mm)	Length (m)	Status	Cost in 2011 dollars	Cost in 2011 delivery costs (+28.25%)	Cost in 2023 dollars (2011 cost + CPI of 37.4%)	Cost in 2023 dollars inc delivery costs (2011 cost +CPI of 37.4% + delivery costs of 41.25%)
Pipe_1			Removed				
Pipe_2			Removed				
Pipe_3	525	205.14	Altered	$54,772.4	$70,245.6	$75,257.3	$106,300.9
Pipe_4	1050	120.95	Altered	$87,205.0	$111,840.3	$119,819.6	$169,245.2
Pipe_5	1050	219.08	Altered	$157,956.7	$202,579.4	$217,032.5	$306,558.4
Pipe_6	1050	111.79	Altered	$80,600.6	$103,370.3	$110,745.2	$156,427.6
Pipe_7	1050	133.89	Altered	$96,534.7	$123,805.7	$132,638.7	$187,352.1
Pipe_8	1050	96.19	Altered	$69,353.0	$88,945.2	$95,291.0	$134,598.5
Pipe_9	1050	85.01	Altered	$61,292.2	$78,607.3	$84,215.5	$118,954.4
Pipe_10	1050	99.4	Altered	$71,667.4	$91,913.4	$98,471.0	$139,090.3
Pipe_11	1050	151.05	Altered	$108,907.1	$139,673.3	$149,638.3	$211,364.1
Pipe_12	1050	282.06	Altered	$203,365.3	$260,815.9	$279,423.9	$394,686.2
Pipe_13	1050	115.68	Altered	$83,405.3	$106,967.3	$114,598.9	$161,870.9
Pipe_14	2 x 675	53.18	Altered	$37,651.4	$48,288.0	$51,733.1	$73,073.0
Pipe_15	900	247.44	No Change	$141,535.7	$181,519.5	$194,470.0	$274,688.9
Pipe_16	900	124.68	Altered	$71,317.0	$91,464.0	$97,989.5	$138,410.2
Pipe_17	675	60.31	Altered	$21,349.7	$27,381.0	$29,334.5	$41,435.0
Pipe_18	450	60.98	Altered	$14,086.4	$18,065.8	$19,354.7	$27,338.5
Pipe_19	900	163.72	Review Pipe Built	$93,647.8	$120,103.4	$128,672.1	$181,749.4
Pipe_20	600	102.53	Review Pipe Built	$31,681.8	$40,631.9	$43,530.8	$61,487.2
Pipe_21	825	84.38	Review Pipe Built	$42,021.2	$53,892.2	$57,737.2	$81,553.8
Pipe_22	675	108.85	No Change	$38,532.9	$49,418.4	$52,944.2	$74,783.7
Pipe_23	750	101.79	No Change	$41,428.5	$53,132.1	$56,922.8	$80,403.5
Pipe_24	825	101.36	No Change	$50,477.3	$64,737.1	$69,355.8	$97,965.0
Pipe_25	825	176.02	Altered	$87,658.0	$112,421.3	$120,442.0	$170,124.4
Pipe_26	600	58.3	Altered	$18,014.7	$23,103.9	$24,752.2	$34,962.5
Pipe_27	1050	278.05	Review Pipe Built	$200,474.1	$257,108.0	$275,451.3	$389,075.0

Asset ID	Diameter (mm)	Length (m)	Status	Cost in 2011 dollars	Cost in 2011 delivery costs (+28.25%)	Cost in 2023 dollars (2011 cost + CPI of 37.4%)	Cost in 2023 dollars inc delivery costs (2011 cost +CPI of 37.4% + delivery costs of 41.25%)
Pipe_28	600	144.35	Built	$44,604.2	$57,204.8	$61,286.1	$86,566.6
Pipe_29	900	45.36	Built	$25,945.9	$33,275.6	$35,649.7	$50,355.2
Pipe_30	1050	200.14	Review Pipe Built	$144,300.9	$185,066.0	$198,269.5	$280,055.7
Pipe_31	900	594.36	Built	$339,973.9	$436,016.6	$467,124.2	$659,812.9
Pipe_32	675	223.41	Altered	$79,087.1	$101,429.3	$108,665.7	$153,490.3
Pipe_33	750	145.29	Altered	$59,133.0	$75,838.1	$81,248.8	$114,763.9
Pipe_34	1200	97.82	Altered	$89,407.5	$114,665.1	$122,845.9	$173,519.8
Pipe_35	675	263.82	Altered	$93,392.3	$119,775.6	$128,321.0	$181,253.4
Pipe_36	750	222.17	Altered	$90,423.2	$115,967.7	$124,241.5	$175,491.1
Pipe_37	900	374.28	Altered	$214,088.2	$274,568.1	$294,157.1	$415,496.9
Pipe_38	900	147.5	Altered	$84,370.0	$108,204.5	$115,924.4	$163,743.2
Pipe_39	600	74.8	Altered	$23,113.2	$29,642.7	$31,757.5	$44,857.5
Pipe_40	900	222.62	Review Pipe Built	$127,338.6	$163,311.8	$174,963.3	$247,135.6
Pipe_41	1200	154.2	Review Pipe Built	$140,938.8	$180,754.0	$193,649.9	$273,530.5
Pipe_42	900	251.94	Review Pipe Built	$144,109.7	$184,820.7	$198,006.7	$279,684.5
Pipe_43	1800	305.24	Review Pipe Built	$622,689.6	$798,599.4	$855,575.5	$1,208,500.4
Pipe_44	2 x 1350	113.02	Altered	$255,877.3	$328,162.6	$351,575.4	$496,600.2
Pipe_45	2 x 1350	36.09	Review Pipe Built	$81,707.8	$104,790.2	$112,266.5	$158,576.4
Pipe_46	2 x 1350	135	Altered	$305,640.0	$391,983.3	$419,949.4	$593,178.5
Pipe_47			Removed				
Pipe_48	450	136.39	Altered	$31,506.1	$40,406.6	$43,289.4	$61,146.2
Pipe_49	825	541.63	Altered	$269,731.7	$345,931.0	$370,611.4	$523,488.6
Pipe_50	1050	55.75	No Change	$40,195.8	$51,551.0	$55,229.0	$78,010.9
Pipe_51	1 x 600 and 1 x 1050	62.78	Altered	$64,663.4	$82,930.8	$88,847.5	$125,497.1
Pipe_52			Removed				
Pipe_53			Removed				
Pipe_54			Removed				

Asset ID	Diameter (mm)	Length (m)	Status	Cost in 2011 dollars	Cost in 2011 delivery costs (+28.25%)	Cost in 2023 dollars (2011 cost + CPI of 37.4%)	Cost in 2023 dollars inc delivery costs (2011 cost +CPI of 37.4% + delivery costs of 41.25%)
Pipe_55			Removed				
Pipe_56			Removed				
Pipe_57			Removed				
Pipe_58			removed				
Pipe_59	900	286.03	Altered	$163,609.2	$209,828.7	$224,799.0	$317,528.6
Pipe_60	900	42.31	Altered	$24,201.3	$31,038.2	$33,252.6	$46,969.3
Pipe_61	900	258.21	Altered	$147,696.1	$189,420.3	$202,934.5	$286,644.9
Pipe_62	900	297.21	Altered	$170,004.1	$218,030.3	$233,585.7	$329,939.7
Pipe_63			Removed				
Pipe_64	525	221.28	Altered	$59,081.8	$75,772.4	$81,178.3	$114,664.4
Pipe_65	750	231.53	No Change	$94,232.7	$120,853.5	$129,475.7	$182,884.5
Pipe_66	900	225.84	Altered	$129,180.5	$165,674.0	$177,494.0	$250,710.2
Pipe_67	2 x 825	64.52	Altered	$64,261.9	$82,415.9	$88,295.9	$124,717.9
Pipe_68	600	288.34	No Change	$89,097.1	$114,267.0	$122,419.4	$172,917.3
Pipe_69	525	72.54	No Change	$19,368.2	$24,839.7	$26,611.9	$37,589.3
Pipe_70	600	72.51	No Change	$22,405.6	$28,735.2	$30,785.3	$43,484.2
Pipe_71	675	305.84	Altered	$108,267.4	$138,852.9	$148,759.4	$210,122.6
Pipe_72	525	27.94	Altered	$7,460.0	$9,567.4	$10,250.0	$14,478.1
Pipe_73			Removed				
Pipe_74	450	145.01	No Change	$33,497.3	$42,960.3	$46,025.3	$65,010.7
Pipe_75	450	269.26	No Change	$62,199.1	$79,770.3	$85,461.5	$120,714.4
Pipe_76	750	151.93	Altered	$61,835.5	$79,304.0	$84,962.0	$120,008.8
Pipe_77	600	374.33	No Change	$115,668.0	$148,344.2	$158,927.8	$224,485.5
Pipe_78	825	319.75	Altered	$159,235.5	$204,219.5	$218,789.6	$309,040.3
Pipe_79	600	97.04	Altered	$29,985.4	$38,456.2	$41,199.9	$58,194.8
Pipe_80	2 x 750	323.8	Altered	$263,573.2	$338,032.6	$362,149.6	$511,536.3
Pipe_81	1200	50.86	Altered	$46,486.0	$59,618.3	$63,871.8	$90,218.9
Pipe_82	1200	52.82	Altered	$48,277.5	$61,915.9	$66,333.3	$93,695.7

Asset ID	Diameter (mm)	Length (m)	Status	Cost in 2011 dollars	Cost in 2011 delivery costs (+28.25%)	Cost in 2023 dollars (2011 cost + CPI of 37.4%)	Cost in 2023 dollars inc delivery costs (2011 cost +CPI of 37.4% + delivery costs of 41.25%)
Pipe_83	2 x 1200	60	Altered	$109,680.0	$140,664.6	$150,700.3	$212,864.2
Pipe_84	450	366.95	Built	$84,765.5	$108,711.7	$116,467.7	$164,510.7
Pipe_85			Removed				
Pipe_86			Removed				
Pipe_87			Removed				
Pipe_88	1200	268.32	Review Pipe Built	$245,244.5	$314,526.0	$336,965.9	$475,964.4
Pipe_89	525	180.14	Altered	$48,097.4	$61,684.9	$66,085.8	$93,346.2
Pipe_90	525	97.63	Built	$26,067.2	$33,431.2	$35,816.3	$50,590.6
Pipe_91	525	252.35	Built	$67,377.5	$86,411.6	$92,576.6	$130,764.5
Pipe_92			Removed				
Pipe_93			Removed				
Pipe_94	825	77.5	Altered	$38,595.0	$49,498.1	$53,029.5	$74,904.2
Pipe_95	1200	647.14	Altered	$591,486.0	$758,580.7	$812,701.7	$1,147,941.2
Pipe_96	450	71.91	No Change	$16,611.2	$21,303.9	$22,823.8	$32,238.6
Pipe_97	1050	320	Altered	$230,720.0	$295,898.4	$317,009.3	$447,775.6
Pipe_98	1200	165	Altered	$150,810.0	$193,413.8	$207,212.9	$292,688.3
Pipe_99	2 x 900	45	No Change	$51,480.0	$66,023.1	$70,733.5	$99,911.1
Pipe_100	1350	38	Altered	$43,016.0	$55,168.0	$59,104.0	$83,484.4
Pipe_101	825	279.34	No Change	$139,111.3	$178,410.3	$191,139.0	$269,983.8
Pipe_102	1350	250.85	No Change	$283,962.2	$364,181.5	$390,164.1	$551,106.7
Pipe_103	1200	118	No Change	$107,852.0	$138,320.2	$148,188.6	$209,316.5
Pipe_104	600	616.99	No Change	$190,649.9	$244,508.5	$261,953.0	$370,008.6
Pipe_105	825	373.27	Altered	$185,888.5	$238,401.9	$255,410.7	$360,767.7
Pipe_106	1200	141.47	Altered	$129,303.6	$165,831.8	$177,663.1	$250,949.2
Pipe_107	1350	276	Altered	$312,432.0	$400,694.0	$429,281.6	$606,360.2
Pipe_108	2 x 675	87.36	Altered	$61,850.9	$79,323.8	$84,983.1	$120,038.6
Pipe_109	525	438	Altered	$116,946.0	$149,983.2	$160,683.8	$226,965.9
Pipe_110	750	460	Altered	$187,220.0	$240,109.7	$257,240.3	$363,351.9

Asset ID	Diameter (mm)	Length (m)	Status	Cost in 2011 dollars	Cost in 2011 delivery costs (+28.25%)	Cost in 2023 dollars (2011 cost + CPI of 37.4%)	Cost in 2023 dollars inc delivery costs (2011 cost +CPI of 37.4% + delivery costs of 41.25%)
Pipe_111	750	211	Altered	$85,877.0	$110,137.3	$117,995.0	$166,667.9
Pipe_112	1350	228.86	Built	$259,069.5	$332,256.7	$355,961.5	$502,795.6
Pipe_113	1350	404.63	Built	$458,041.2	$587,437.8	$629,348.6	$888,954.8
Pipe_114	2100	116.87	Built	$337,170.0	$432,420.5	$463,271.5	$654,371.0
Pipe_115	1500	40.81	Built	$56,766.7	$72,803.3	$77,997.5	$110,171.4
Pipe_116	750	43.71	Built	$17,790.0	$22,815.6	$24,443.4	$34,526.3
Pipe_117	900	300.14	Built	$171,680.1	$220,179.7	$235,888.4	$333,192.4
Pipe_119	1200	311.87	Built	$285,049.2	$365,575.6	$391,657.6	$553,216.3
Pipe_120	900	90.72	Built	$51,891.8	$66,551.3	$71,299.4	$100,710.4
Pipe_121	1200	238.36	Built	$217,861.0	$279,406.8	$299,341.1	$422,819.3
Pipe_122	675	167.39	Built	$59,256.1	$75,995.9	$81,417.8	$115,002.7
Pipe_123	675	140.21	Built	$49,634.3	$63,656.0	$68,197.6	$96,329.1
Pipe_124	750	139.38	Built	$56,727.7	$72,753.2	$77,943.8	$110,095.6
Pipe_125	1050	122.25	Built	$88,142.3	$113,042.4	$121,107.5	$171,064.3
Pipe_126	1050	140.76	Built	$101,488.0	$130,158.3	$139,444.5	$196,965.3
Pipe_127	675	154.15	Built	$54,569.1	$69,984.9	$74,977.9	$105,906.3
Pipe_128	825	149.23	Built	$74,316.5	$95,311.0	$102,110.9	$144,231.7
Pipe_129	2 x 900	50.87	Built	$58,195.3	$74,635.4	$79,960.3	$112,943.9
Pipe_130	825	447.64	Built	$222,924.7	$285,901.0	$306,298.6	$432,646.7
Pipe_131	750	392.13	Built	$159,596.9	$204,683.0	$219,286.2	$309,741.7
Pipe_132	600	35.39	Built	$10,935.5	$14,024.8	$15,025.4	$21,223.4
Pipe_133	1200	447.38	Built	$408,905.3	$524,421.1	$561,835.9	$793,593.2
Pipe_134	3 x 750	45.06	Built	$55,018.3	$70,560.9	$75,595.1	$106,778.1
Pipe_201	1050	114.67	Review Pipe Built	$82,677.1	$106,033.3	$113,598.3	$160,457.6
Pipe_202	1050	105.07	Review Pipe Built	$75,755.5	$97,156.4	$104,088.0	$147,024.3
Pipe_204	1800	30.92	Review Pipe Built	$63,076.8	$80,896.0	$86,667.5	$122,417.9
Pipe_205	1800	174.8	Review Pipe Built	$356,592.0	$457,329.2	$489,957.4	$692,064.8
Pipe_206	1650	129.95	Review Pipe Built	$221,174.9	$283,656.8	$303,894.3	$429,250.7

Asset ID	Diameter (mm)	Length (m)	Status	Cost in 2011 dollars	Cost in 2011 delivery costs (+28.25%)	Cost in 2023 dollars (2011 cost + CPI of 37.4%)	Cost in 2023 dollars inc delivery costs (2011 cost +CPI of 37.4% + delivery costs of 41.25%)
Pipe_207	1350	114.96	Review Pipe Built	$130,134.7	$166,897.8	$178,805.1	$252,562.2
Pipe_208	1350	37.13	Review Pipe Built	$42,031.2	$53,905.0	$57,750.8	$81,573.0
Pipe_209	1200	24.08	Review Pipe Built	$22,009.1	$28,226.7	$30,240.5	$42,714.7
Pipe_210	1200	90.53	Review Pipe Built	$82,744.4	$106,119.7	$113,690.8	$160,588.3
Pipe_211	1200	43.22	Review Pipe Built	$39,503.1	$50,662.7	$54,277.2	$76,666.6
Pipe_212	1200	19.09	Review Pipe Built	$17,448.3	$22,377.4	$23,973.9	$33,863.1
Pipe_213	1200	69.99	Review Pipe Built	$63,970.9	$82,042.6	$87,896.0	$124,153.0
Pipe_214	1350	79.97	Review Pipe Built	$90,526.0	$116,099.6	$124,382.8	$175,690.7
Pipe_215	1350	23.24	Review Pipe Built	$26,307.7	$33,739.6	$36,146.8	$51,057.3
Pipe_216	1350	2.95	Review Pipe Built	$3,339.4	$4,282.8	$4,588.3	$6,481.0
Pipe_217	1200	6.52	Review Pipe Built	$5,959.3	$7,642.8	$8,188.1	$11,565.6
Pipe_218	1050	5.83	Review Pipe Built	$4,203.4	$5,390.9	$5,775.5	$8,157.9
Pipe_219	1050	21.71	Review Pipe Built	$15,652.9	$20,074.9	$21,507.1	$30,378.8
Pipe_220	1050	37.98	Review Pipe Built	$27,383.6	$35,119.4	$37,625.0	$53,145.4
Pipe_221	1050	39.03	Review Pipe Built	$28,140.6	$36,090.4	$38,665.2	$54,614.6
Pipe_222	1050	43.69	Review Pipe Built	$31,500.5	$40,399.4	$43,281.7	$61,135.4
Pipe_223	1050	43.69	Review Pipe Built	$31,500.5	$40,399.4	$43,281.7	$61,135.4
Pipe_224	525	16.49	Review Pipe Built	$4,402.8	$5,646.6	$6,049.5	$8,544.9
Pipe_225	525	5.34	Review Pipe Built	$1,425.8	$1,828.6	$1,959.0	$2,767.1
Pipe_226	900	33.58	Review Pipe Built	$19,207.8	$24,634.0	$26,391.5	$37,277.9
Pipe_227	900	33.58	Review Pipe Built	$19,207.8	$24,634.0	$26,391.5	$37,277.9
Pipe_228	900	33.59	Review Pipe Built	$19,213.5	$24,641.3	$26,399.3	$37,289.0
Pipe_229	900	33.59	Review Pipe Built	$19,213.5	$24,641.3	$26,399.3	$37,289.0
Pipe_230	900	33.59	Review Pipe Built	$19,213.5	$24,641.3	$26,399.3	$37,289.0
Pipe_231	900	33.59	Review Pipe Built	$19,213.5	$24,641.3	$26,399.3	$37,289.0
Pipe_232	525	16.34	Review Pipe Built	$4,362.8	$5,595.3	$5,994.5	$8,467.2
Pipe_233	525	5.33	Review Pipe Built	$1,423.1	$1,825.1	$1,955.4	$2,761.9
Pipe_234	1350	51.69	Review Pipe Built	$58,513.1	$75,043.0	$80,397.0	$113,560.7

Asset ID	Diameter (mm)	Length (m)	Status	Cost in 2011 dollars	Cost in 2011 delivery costs (+28.25%)	Cost in 2023 dollars (2011 cost + CPI of 37.4%)	Cost in 2023 dollars inc delivery costs (2011 cost +CPI of 37.4% + delivery costs of 41.25%)
Pipe_235	1350	51.44	Review Pipe Built	$58,230.1	$74,680.1	$80,008.1	$113,011.5
Pipe_236	1350	85.08	Review Pipe Built	$96,310.6	$123,518.3	$132,330.7	$186,917.1
Pipe_237	1350	112.61	Review Pipe Built	$127,474.5	$163,486.1	$175,150.0	$247,399.4
Pipe_238	1350	89.74	Review Pipe Built	$101,585.7	$130,283.6	$139,578.7	$197,154.9
Pipe_239	1350	67.08	Review Pipe Built	$75,934.6	$97,386.1	$104,334.1	$147,371.9
Pipe_240	1050	113.1	Review Pipe Built	$81,545.1	$104,581.6	$112,043.0	$158,260.7
Pipe_242	1050	44.27	Review Pipe Built	$31,918.7	$40,935.7	$43,856.3	$61,947.0
Pipe_245	1200	147.51	Review Pipe Built	$134,824.1	$172,912.0	$185,248.4	$261,663.3
Pipe_246	1200	147.63	Review Pipe Built	$134,933.8	$173,052.6	$185,399.1	$261,876.2
Pipe_301	750	36.45	Altered	$14,835.2	$19,026.1	$20,383.5	$28,791.7
Pipe_302	750	38.6	Altered	$15,710.2	$20,148.3	$21,585.8	$30,490.0
Pipe_303	750	94.76	Altered	$38,567.3	$49,462.6	$52,991.5	$74,850.5
Pipe_304	750	22.39	Altered	$9,112.7	$11,687.1	$12,520.9	$17,685.8
Pipe_305	750	53.32	Altered	$21,701.2	$27,831.8	$29,817.5	$42,117.2
Pipe_306	750	43.94	Altered	$17,883.6	$22,935.7	$24,572.0	$34,708.0
Pipe_307	900	42.91	Altered	$24,544.5	$31,478.3	$33,724.2	$47,635.4
Pipe_308	900	40.8	Altered	$23,337.6	$29,930.5	$32,065.9	$45,293.0
Pipe_309	900	66.34	Altered	$37,946.5	$48,666.4	$52,138.5	$73,645.6
Pipe_310	1050	41.93	Altered	$30,231.5	$38,771.9	$41,538.1	$58,672.6
Pipe_311	1050	36.75	Altered	$26,496.8	$33,982.1	$36,406.5	$51,424.2
Pipe_312	1050	81.87	Altered	$59,028.3	$75,703.8	$81,104.8	$114,560.6
Pipe_313	1350	33.55	Altered	$37,978.6	$48,707.6	$52,182.6	$73,707.9
Pipe_314	1650	45	Altered	$76,590.0	$98,226.7	$105,234.7	$148,644.0
Pipe_315	750	111	Altered	$45,177.0	$57,939.5	$62,073.2	$87,678.4
Pipe_316	750	94	Altered	$38,258.0	$49,065.9	$52,566.5	$74,250.2
Pipe_317	750	192	Altered	$78,144.0	$100,219.7	$107,369.9	$151,659.9
Pipe_318	2 x 900	56	Altered	$64,064.0	$82,162.1	$88,023.9	$124,333.8
Pipe_319	1050	657	Altered	$473,697.0	$607,516.4	$650,859.7	$919,339.3

Asset ID	Diameter (mm)	Length (m)	Status	Cost in 2011 dollars	Cost in 2011 delivery costs (+28.25%)	Cost in 2023 dollars (2011 cost + CPI of 37.4%)	Cost in 2023 dollars inc delivery costs (2011 cost +CPI of 37.4% + delivery costs of 41.25%)
Pipe_320	1500	336	Altered	$467,376.0	$599,409.7	$642,174.6	$907,071.7
Pipe_321	600	87	Altered	$26,883.0	$34,477.4	$36,937.2	$52,173.9
Pipe_322	1200	32.98	Built - Altered	$30,143.7	$38,659.3	$41,417.5	$58,502.2
Culvert_1	2 x 1800	44	Altered	$179,520.0	$230,234.4	$246,660.5	$348,407.9
Pipe_118			Removed				
Total				$18,343,882.8	$23,526,029.7	$25,204,494.9	$35,601,349.1

Table 6.3 shows the updated wetland cost estimates for the wetlands which were updated as part of this 2023 strategy update. Costs are shown in 2011 and 2023 prices to allow for comparison between original PSP DCP cost estimates and the updated PSP cost estimates. The 2011 costs shown are based on the updated concept designs and not the original concept designs. An allowance has also been added to the cost estimates for the supply of a gross pollutant trap to be installed upstream of each sediment basin and wetland. The cost estimates range from $80,000 to $155,000 in 2023 dollars for each GPT (depending on estimated treatment flow). The costs for the GPTs are based on information provided by propriety systems providers and are an estimate only.

The exception to the above is for retarding basin 27. This basin is proposed as an embankment across the waterway to retard flow. There are more unknowns and risk in this design and so a 50% contingency is proposed for the cost estimate instead of the standard 15% used for the remaining assets. This should be narrowed down following the completion of a functional design and ANCOLD risk of failure assessment. To provide a cost estimate at this stage it has been assumed that the ANCOLD risk ranking of the embankment would be a High C (on the basis that there will be a future arterial road directly downstream of the embankment and that residential development is also possible downstream of the embankment) and that this would require rock armouring of the entire downstream face of the embankment which would also act as the spillway in rare events. It has been assumed that a d_{50} of 500 mm (d_{50} meaning 50% of the rock placed has a diameter equal to 500 mm) would be suitable and would be required at a depth of 1 m, it is assumed to cost $150/m³ to import and place. The quality and type of the material to be excavated as part of the WL27 works is not known and so it has been assumed that all material for the embankment will need to be imported. A rate of $100 per m³ has been assumed as an average rate, noting that a sand filter is likely, with rates for filter material being up to $200 per m³ to import and place, however rates for the clay core and bulk backfill are likely to be significantly less. Further design work is recommended to improve the accuracy of the cost rating.

TABLE 6.3: WETLAND COSTS

Asset ID	Cost in 2011 dollars	Cost in 2011 dollars inc delivery fees	Cost in 2023 dollars	Cost in 2023 dollars inc delivery fees	Comments
RB7	$4,137,492	$5,720,083	$5,684,914	$8,029,942	
RB12	$1,984,173	$2,743,119	$2,726,254	$3,850,834	
RB13	$2,576,596	$3,562,144	$3,540,243	$5,000,593	
RB14	$1,632,855	$2,257,422	$2,243,543	$3,169,005	
RB15	$1,969,234	$2,722,466	$2,705,727	$3,821,840	
RB17	$3,324,885	$4,596,654	$4,568,392	$6,452,854	
RB18	$1,458,723	$2,016,685	$2,004,286	$2,831,053	
RB24	$3,198,484	$4,421,904	$4,394,717	$6,207,537	
WL27	$1,080,279	$1,493,486	$1,484,304	$2,096,579	This cost is only for the offline wetland asset on the western side of the waterway. A wetland was proposed at this location in the 2011 strategy
RB27	$1,873,900	$2,590,667	$2,574,739	$4,537,977	Costs are largely associated with the embankment and costing methodology is described above, includes a 50% contingency. A RB was proposed at this general location in the 2011 strategy
SB27B	$422,178 (New Asset)	$583,661 (New Asset)	$580,073	$819,353	New asset added to PSP as part of review on the eastern side of the waterway
RB29	$3,402,006	$4,703,274	$4,674,357	$6,602,529	

Asset ID	Cost in 2011 dollars	Cost in 2011 dollars inc delivery fees	Cost in 2023 dollars	Cost in 2023 dollars inc delivery fees	Comments
SB30	$810,249 (New Asset)	$1,120,170 (New Asset)	$1,113,283	$1,572,512	Asset changed form a retarding basin/wetland to a sedimentation basin
Total	**$27,871,056**	**$38,531,734**	**$38,294,831**	**$54,992,607**	

It is understood that stand alone wetlands and sedimentation basins were not included in the original DCP, however combined retarding basin wetlands were. It is not the intention of this strategy to decide what assets are included in the DCP, however the costs are provided so that if particular asset types are included the information is available.

Table 6.4 shows the costs of the previously constructed or committed wetland retarding basins. Please note that the 2011 report applied total contingency, council fees and consulting costs of 41.9% on top of the base fee estimate, whereas the updated costs apply a 30% contingency on top of the base fee estimate. Where there have been significant design changes the updated design has been re-costed at the 2011 rates. This means for RBs 6, 6a, 6b, 6c, 11, 12, 18 the 2011 costs will not match the 2011 report costs and the updated design has been costed and noted as the 2011 cost.

TABLE 6.4: CONSTRUCTED OR COMMITTED WETLAND COSTS

Asset ID	Cost in 2011 dollars	Cost in 2011 dollars inc delivery fees	Cost in 2023 dollars	Cost in 2023 dollars inc delivery fees	Comments
RB1	$567,840	$805,765	$780,212	$1,014,276	
RB2	$4,025,400	$5,712,043	$5,530,900	$7,190,169	
RB3	$1,564,860	$2,220,536	$2,150,118	$2,795,153	
RB4	$1,438,224	$2,040,840	$1,976,120	$2,568,956	
RB5	$1,713,810	$2,431,896	$2,354,775	$3,061,207	
RB6	$2,312,580	$3,281,551	$3,177,485	$4,130,731	Updated design costed
RB6A	$2,551,941	$3,621,205	$3,506,367	$4,558,277	New asset not in 2011 strategy
RB6B	$629,922	$893,860	$865,513	$1,125,167	New asset not in 2011 strategy
RB6C	$492,957	$699,506	$677,323	$880,520	New asset not in 2011 strategy
RB11	$2,092,329	$2,969,015	$2,874,860	$3,737,319	Updated design costed
RB25 and 26	$1,465,797	$2,079,966	$2,014,005	$2,618,207	RB 25 and 26 have been consolidated into one asset
RB28	$3,673,380	$5,212,526	$5,047,224	$6,561,391	
Total	**$22,529,041**	**$31,968,710**	**$30,954,903**	**$40,241,374**	

A number of bioretention or rain garden assets were proposed in the 2011 strategy. All of those assets have been removed from the strategy, with the original IDs and costs (2011 dollars) shown in Table 6.5. the bioretention basins have been removed as they can be challenging assets to maintain and without pretreatment of stormwater are prone to surface clogging from sediments. The role that they were playing in the stormwater treatment has been replaced by the sedimentation basins and wetlands. This results in fewer overall assets for Council to maintain and also provides better community assets as wetlands typically provide better overall amenity.

ENGENY

TABLE 6.5: BIORENTION AREAS

Asset ID	Filter Area (m^2)	Cost Estimate	Status
AZ	50	$16,260	Removed
BT	50	$16,260	Removed
BR	50	$16,260	Removed
CA	50	$16,260	Removed
BL	50	$16,260	Removed
CB	50	$16,260	Removed
CT	50	$16,260	Removed
CU	50	$16,260	Removed
CV	50	$16,260	Removed
DB	50	$16,260	Removed
DC	50	$16,260	Removed
CR	50	$16,260	Removed
CW	50	$16,260	Removed
Y	300	$97,557	Removed
EB	150	$48,778	Removed
W & X	2000	$773,725	Removed
Z	400	$130,075	Removed
RB1	500	$162,594	Removed

7. STAGING

Council has provided a plan showing the current status of development applications within the Ballarat West PSP area. Areas where development applications have been received and approved now make up a significant portion of the total area. A challenge that Council faces for managing stormwater is that most of the remaining wetlands and retarding basins are along the southern boundary of the development area adjacent to Winter Creek. This is the most downstream location in the catchments and so allows for most of the upstream catchment areas to be captured, maximising the treatment and retardation potential of the assets. As the development is generally being undertaken from north (existing areas of Delacombe) to south it means that the wetlands are potentially located on properties likely to be the last to develop. There are also some properties where the wetlands cover a significant portion of the property, reducing the remaining land available for development and the potential interest or viability of development on those properties. In some of these areas Council may need to take a proactive role in acquiring some land and potentially building some trunk drainage infrastructure to facilitate upstream development.

Engeny has assessed the remaining retarding basin and pipe infrastructure as being required in either the short, medium or long term. Short term requirements for infrastructure have been assigned to assets which will be required to service properties either currently under construction or with issued planning permits. (as per Figure 7.1) Properties which have infrastructure requirements downstream and are expected to lodge planning permits soon has been assessed as medium priority. The remaining areas where there are no lodged permits and none or only a single property likely to lodge soon has been assessed and long term priority. The definitions for short medium and long term and not intended to link to a particular time frame as even developments with issues planning permits can years to commence construction. Instead, they are intended to guide the focus of the general order in which assets will need to be delivered across the precinct. It is worth noting that most of the remaining retarding basins and wetlands are identified as short or medium term needs. The plans in Appendix E show the proposed staging term for each of the remaining assets.

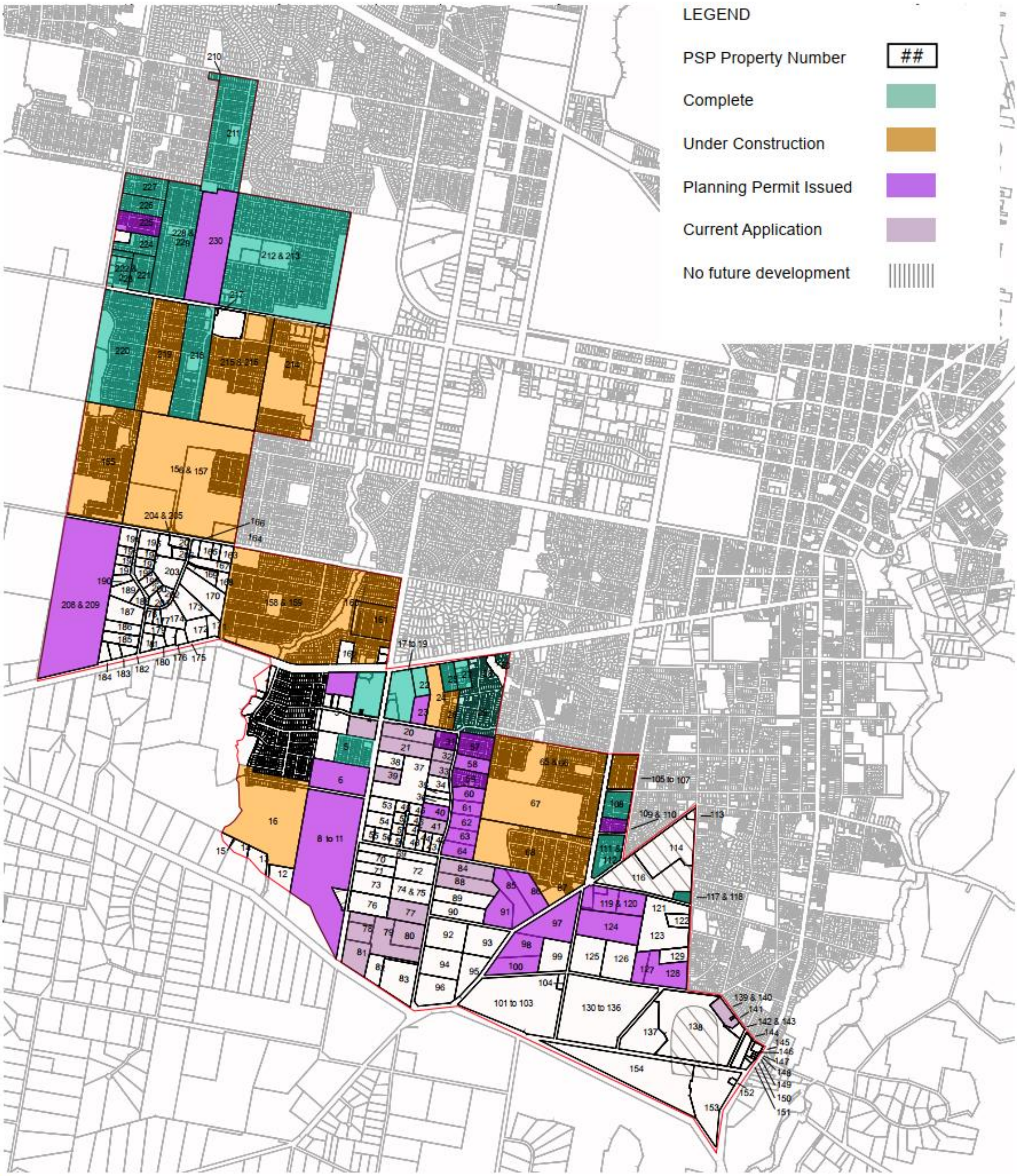

FIGURE 7.1: CURRENT PERMIT STATUS AND PROPERTY IDS

7.1 Highest priority (short term)

The highest priority for Council should be to consider areas where construction is already underway on the property or where permits have already been lodged and where the ultimate drainage infrastructure is not yet built and will not be built as part of the development. Temporary solutions may be required by some developers, however where possible these should be minimised.

Current examples of where some Council intervention may be necessary includes property 12. The read of this property has almost no saleable development potential with nearly the entire part of property within the PSP boundary proposed for either open space or a wetland and retarding basin asset (RB13). Council should consider purchasing this property and either managing the construction of the wetland and retarding basin asset itself or engaging with the developer of property 16 to deliver this asset. The development of property 16 will be limited or require temporary assets without the construction of WLRB 13 which is located in property 11 and 12. Figure 7.2 shows the property IDs and the locations of the basins discussed above.

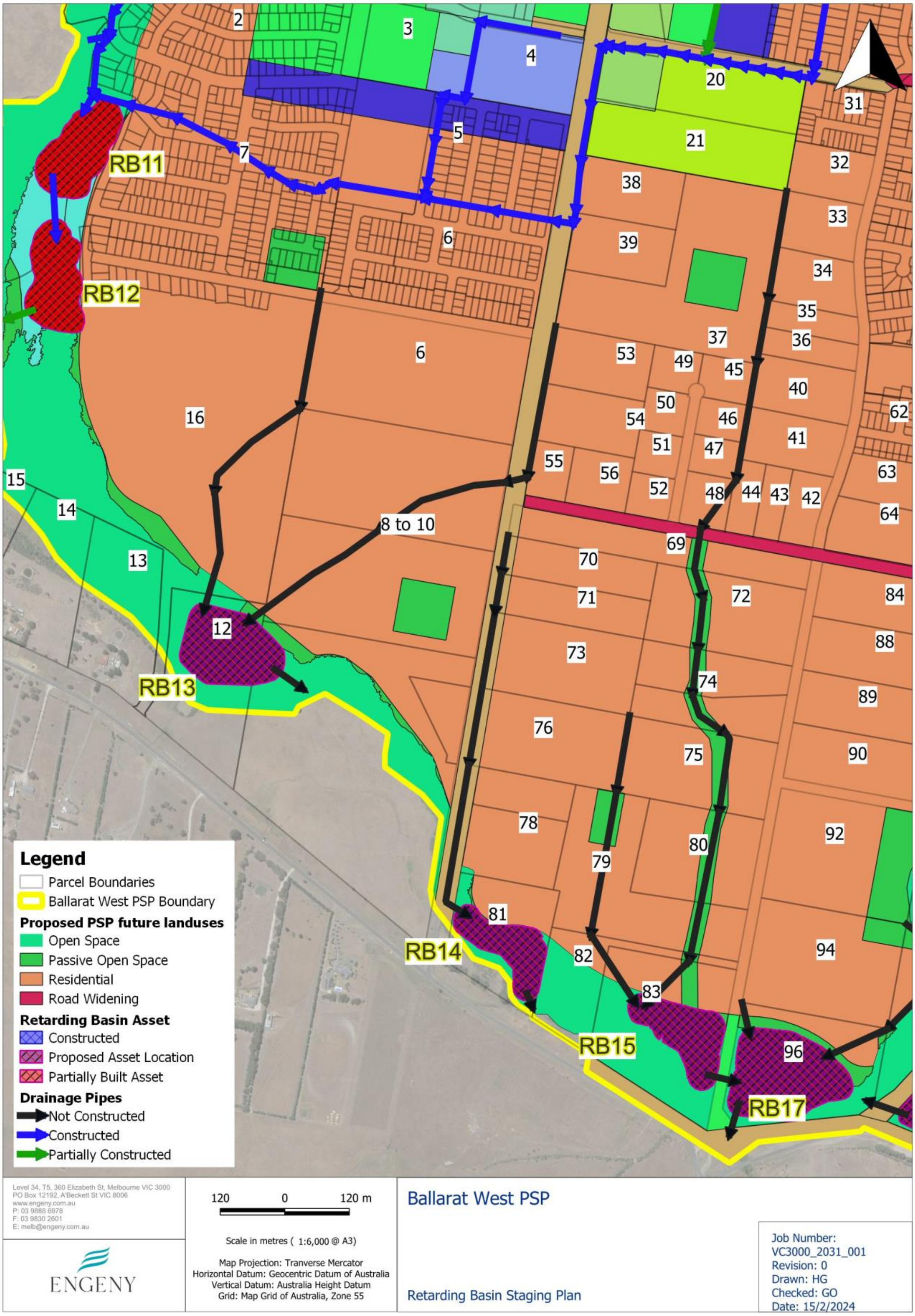

FIGURE 7.2: RETARDING BASINS AND PROPERTY NUMBERS

7.2 Secondary priority (short-medium term)

The next highest priority for Council should be to consider which properties are close to lodging development applications and consider undertaking strategic projects to help facilitate the orderly development of these properties.

Facilitating the delivery of RB7 on property 209 will provide the final retarding basin and wetland asset in precinct 2. This should help to facilitate the remaining development within the precinct as all end of line treatment assets will be constructed.

The area shown in Figure 7.3 which is bounded by Schreenans Road / Webb Road and Cherry Flat Road and also includes Olivemay Court poses potential challenges. The development of properties 78, 79, 80, 81, 82, and 83 should be encouraged and facilitated where possible as this has the potential to deliver WLRB 14 and 15, which will help facilitate the upstream development. Properties 33, 34, 35, 36, 37, 38 and 39 (northern cluster) are somewhat stranded from a drainage point of view. The existing natural waterways or overland flow paths flow from the north to the south and pass through the smaller existing smaller properties which front Olivemay Court, Schreenans Road or Webb Road (40-52) (Olivemay cluster). The development incentive for these properties may be less than for the larger properties upstream and downstream due to their smaller size. To help facilitate the development of the northern cluster of property Council could consider undertaking or assisting in the implementation of one of the following options. The options are shown below in Figure 7.3.

- Constructing the underground drainage through the Olivemay Court cluster to Schreenans Road or through to property 80, to connect to the drain which the developer of that cluster of properties should be able to deliver in the near term. If the underground drain is only constructed to Schreenans Road it may be possible connect it to some of the dams which are online to the waterway downstream of Schreenans Road. Some interim retardation may be required to ensure that flows through these properties are not increased to a point that it has an unacceptable impact on those properties.
- Option 1 is an alternative to using the existing easements requires the creation of a new easement along the rear of properties 40 and 41 and down the western side of property 44. An easement along the western side of property 44 may be challenging as the existing dwelling is situated fairly close to the property boundary.
- Option 2 would be to utilise the existing easement through the western side of property 45 and then construct the rest of the pipeline along Olivemay Court within the existing road reserve. This option involves the least disruption to private property, however is also further away from the low point and so while facilitating the drainage of the northern cluster it does not assist with the development of the eastern properties in the Olivemay Court cluster which will occur at some point in the future. Properties 40 to 44 could not connect to this asset and properties 46 to 48 may also be unable to drain the entire property to this drain. If the main drain was constructed along this alignment then a secondary drain would likely be needed along the currently proposed alignment, however it could be smaller than is currently proposed as it is only draining the properties 40-44 and 46-48. If this option was to be pursued Engeny would recommend that the cost of the new smaller pipeline be determined and this amount reserved from the reimbursement available for the construction of pipes 5 and 6. The balance of funds could be provided to fund the main drain through property 45 and along Olivemay Court with the developer/s of the northern cluster picking up the shortfall as the works are being adjusted to facilitate quicker development and reduce the costs of onsite detention.
- Option 3: It is understood that there is an existing drainage easement at the rear of PSP properties 45-48 in the Olivemay Court Cluster. It is understood that there are a number of large trees in or adjacent to this easement which would need to be removed if this easement was used for the construction of this drain. It is understood that Council legally has the power to undertake the tree removal if they are in Council’s drainage easement, however this may not to be well received by existing land owners. This option does provide drainage outfalls to properties 40 and 41, however they would be connecting to a pipe within an easement on an adjoining property.
- An overland flow path, likely in the form of a road, will be required along a similar alignment to option 1 in the future to allow for the conveyance of gap flow from the upstream development to the future drainage reserve south of Schreenans Road regardless of which option is pursued.

Engeny recommends engagement with all of the property owners in the Olivemay Court cluster to determine what the most practical solution to providing a drainage outfall for the upstream northern cluster is. From a purely engineering perspective the best alignment for the pipe is option 1. It provides outlets to properties 40 and 41 which meets the strategy’s intended aim. As these properties are the ones to benefit by being provided with an outfall, the pipe also located on their land.

Construction of the main outfall drain along Cherry Flat Road or Schreenans Road (the north south running section), is not considered viable due to the height above the valley floor and low points which require drainage.

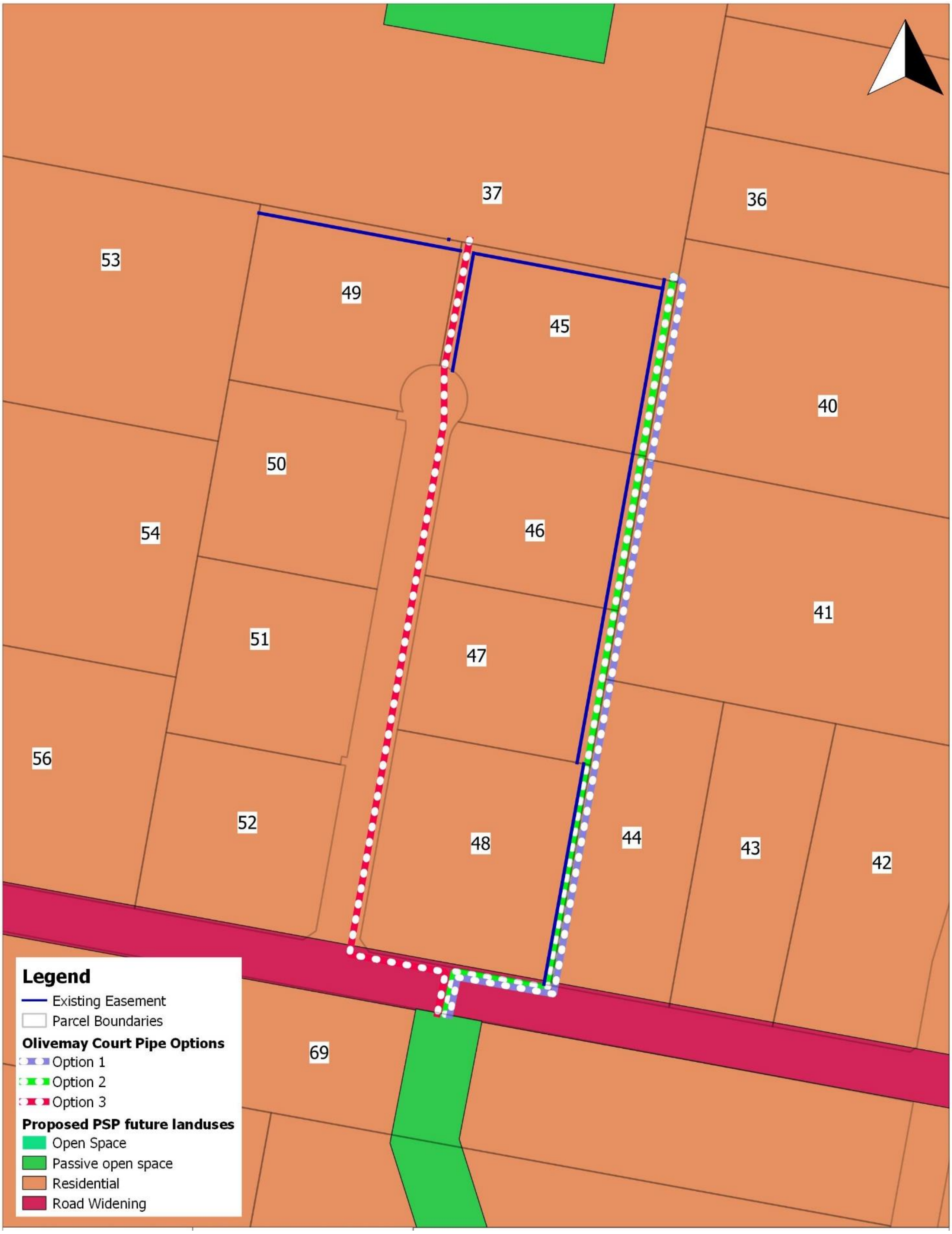

FIGURE 7.3: SCHREENANS ROAD PRECINCT

8. HYDRUALIC MODELLING

8.1 Purpose

Hydraulic TUFLOW modelling has been undertaken to help quantify the impact of the proposed development within the Ballarat West PSP on flooding downstream. In a meeting to discuss the development precinct the Corangamite CMA have stated that that up to 20 mm of flooding increase may be an acceptable level of increase.

8.2 Approach

A combined 1D/2D dynamic hydraulic modelling of the study area was undertaken using TUFLOW to estimate flood water levels, extents, flows and other hydraulic variables for the 1 % Annual Exceedance Probability (AEP) Storm Event. The model was run using the latest version of TUFLOW HPC with Subgrid Sampling (2023-03-AA) at the commencement of the modelling.

8.2.1 Methodology Overview

The following steps outline the tasks undertaken to develop the TUFLOW model for the study catchment and to obtain the results and outputs which were used for flood mapping.

- Generate a digital elevation model (DEM) based on latest available LiDAR, obtained from the Elvis portal maintained by Geoscience Australia. Simulate RORB hydrology models and compile hydrographs to determine critical storms for the study area. Refer to section 8.2.3 for details on ARF and critical duration.
- Apply rainfall excess hydrographs to flood model. Where appropriate 2D_streamlines have been utilised to improve model simulation runs times and reduce the impact of artificial depressions storage (compared to 2D_sa_all approach). Flows that had been routed in the hydrology RORB model has been applied through 2d_bc lines or sa_all polygons within waterways.
- Develop a Manning's surface roughness (materials layer)
- Input, review and verify drainage asset data (provided by Watertech).
- Represent the 3 major bridge crossings structures (Colac-Ballarat Road, Sebastopol-Smythesdale Rd, Bells Rd) (provided by Watertech)
- Apply z-shapes break lines to the road crest to ensure overland flow does not artificially travel through model cells due to the SGS modelling approach.
- Set 1D and 2D boundary conditions.
- Run the model in TUFLOW HPC with a 3-metre grid with sub-grid sampling at 0.75 metres.
- Produce and prepare flood mapping outputs.

8.2.2 Development Scenarios

As discussed in section 3.4.6 the proposed design of RB27 is able to achieve the required flow reduction to redeveloped flows so there is limited increase on the downstream section of the waterway. This proposed design will require an embankment 5 meters tall in the centre. An embankment of this size will create an elevated risk associated with possible embankment failure. Opportunities to limit the associated risk have been identified and trialled. Three variations of RB 27 were modelled to assess the downstream impacts, these include the following.

Scenario 1 (SO1) - RB27 sized to restrict flows back to pre-development within the 1 % AEP (current proposed design)

Scenario 1 aims to assess the performance of the proposed RB27 when designed to restrict flows back to predevelopment within the 1 % AEP event. Key considerations for scenario 1 include:

- Peak flow discharge from RB27 is 11.03 m^3/s (slightly higher than pre-development conditions)
- Embankment height would extend to 388.1 m AHD

Scenario 2 (SO2) - RB27 sized to restrict flows back to pre-development within the 10 % AEP

Scenario 2 aims to assess the performance of the proposed RB27 when designed the restrict flows back to predevelopment within the 10 % AEP.

Key considerations for scenario 2 include:

- Peak flow discharge from RB27 is 15.3 m^3/s
- Embankment height would extend to 387.43 m AHD

Scenario 3 (SO3) - No RB27

Scenario 3 aims to assess the downstream impacts of having no flow retardation on the waterway at the proposed location for RB27. The wetlands would still be required for stormwater treatment.

Key considerations for scenario 3 include:

- Peak flow discharge from RB27 is 19.6 m^3/s
- No embankment required

8.2.3 Areal Reduction Factors and Critical Storms

The IFD data provided by the BoM is applicable for rainfall in small catchments. As catchment size increases the chance of that average intensity of rainfall occurring over the entire catchment decreases. To address this issue an Areal Reduction Factor (ARF) can be applied to the IFD data to account for the larger catchment area. The critical storms have been identified through compiling and analysing outputs from the hydrology RORB model. Figure 8.1 identifies the key locations to determine the significant critical storm duration and temporal pattern for the 1 % AEP event.

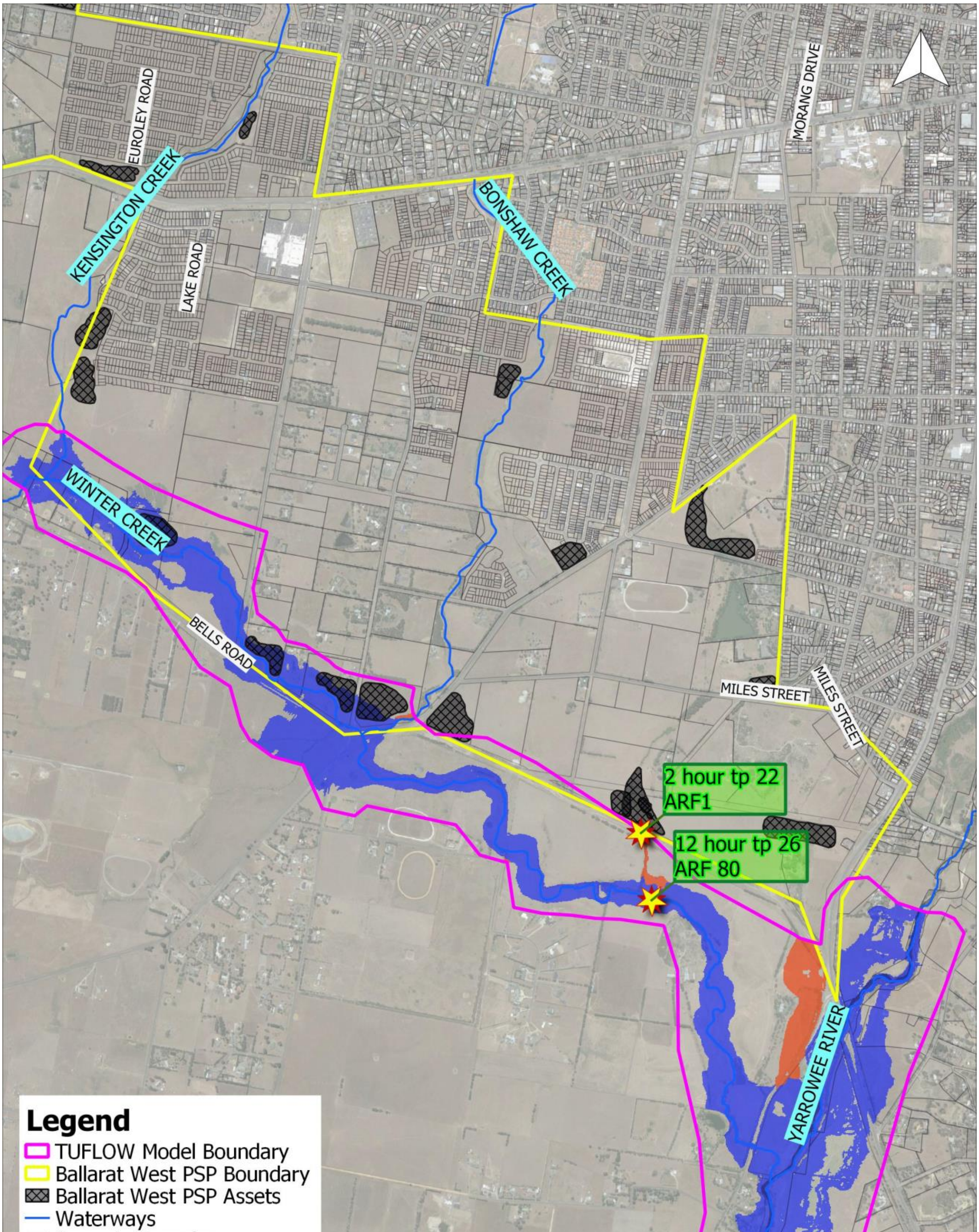

FIGURE 8.1: KEY LOCATION IDENTIFIED FOR CRITICAL DURATION AND TEMPORAL PATTERNS FOR THE 1 % AEP EVENT

8.3 Results

Appendix F shows the flood depth and flood level difference plots for the 1 % AEP event for all four scenarios including the existing conditions results.

Appendix G focuses in on the ~200 m waterway stretch between the outlet of RB27 and Winter Creek (purple box in Figure 8.2) and provides the depths and flood level difference plots for the 1 % AEP event for all four scenarios including the existing conditions results.

Figure 8.2 shows the flood level difference for scenario 3 which has no flow constraints on the waterway at the location of the proposed RB27, this scenario provides the highest peak flow discharge out of the PSP. It should be noted that flood level increase for all scenarios when compared to existing conditions outside of the ~200 m waterway stretch between the outlet of RB27 and Winter Creek (purple box in Figure 8.2) is less than 20 mm.

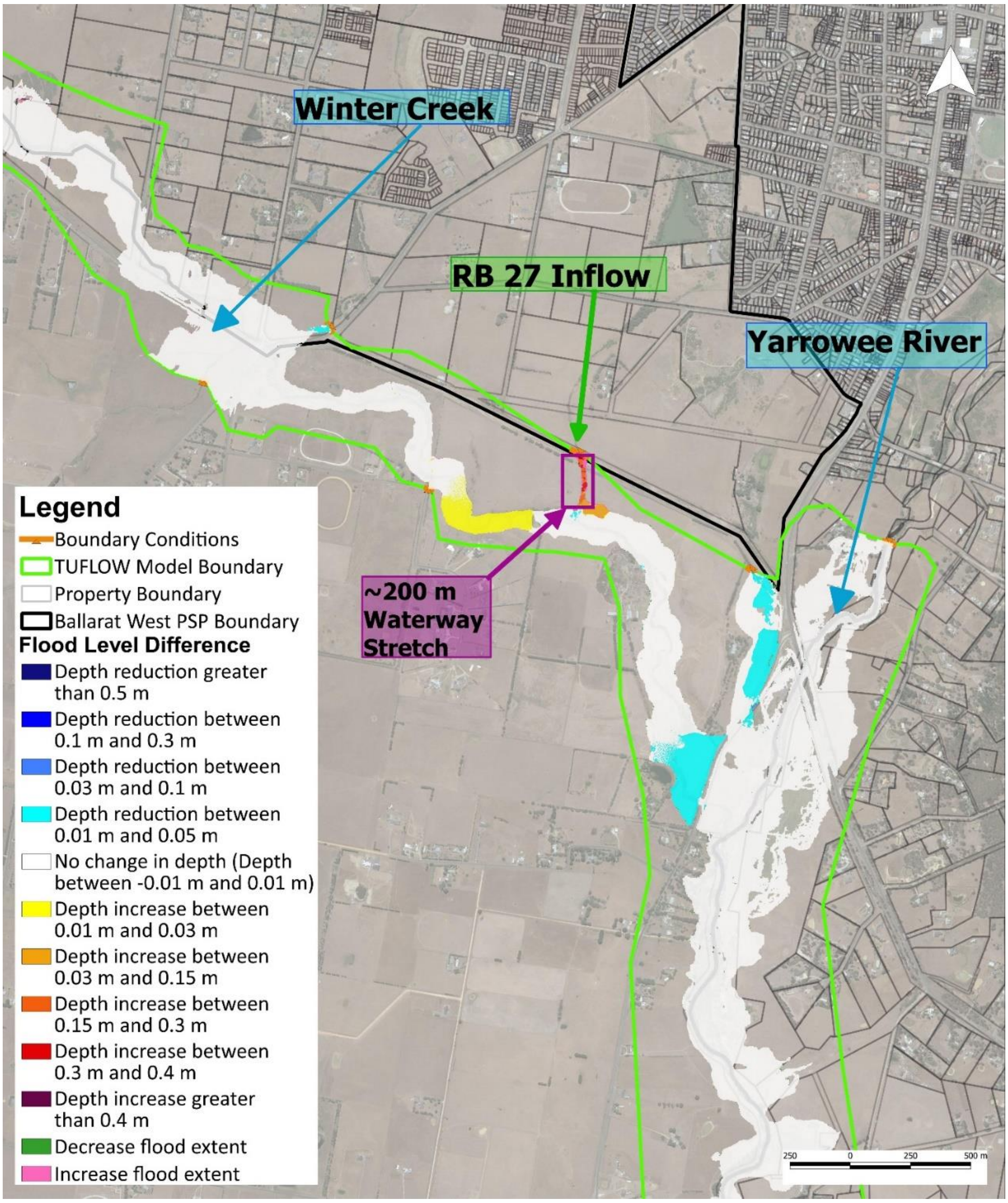

FIGURE 8.2: 1 % AEP FLOOD LEVEL DIFFERENCE FOR SCENARIO 3

Figure 8.3 is zoomed into the purple box seen in Figure 8.2. It highlights that the significant flood level increases are mainly contained to within 30 metres of the waterway centreline. The current land use in this area appears to be rural farming. The additional increase in flood depth in the 1% AEP event would have a minimal impact on the current land use. Should the area be developed in the future (noting that the property is within Golden Plains Shire Council and not currently zoned for development the waterway corridor setback requirement for each side of the waterway set by the Victorian Government under clause 14.02-1S in the Victorian Planning Scheme is 30 m and so there would not be a significant impact on the properties development potential.

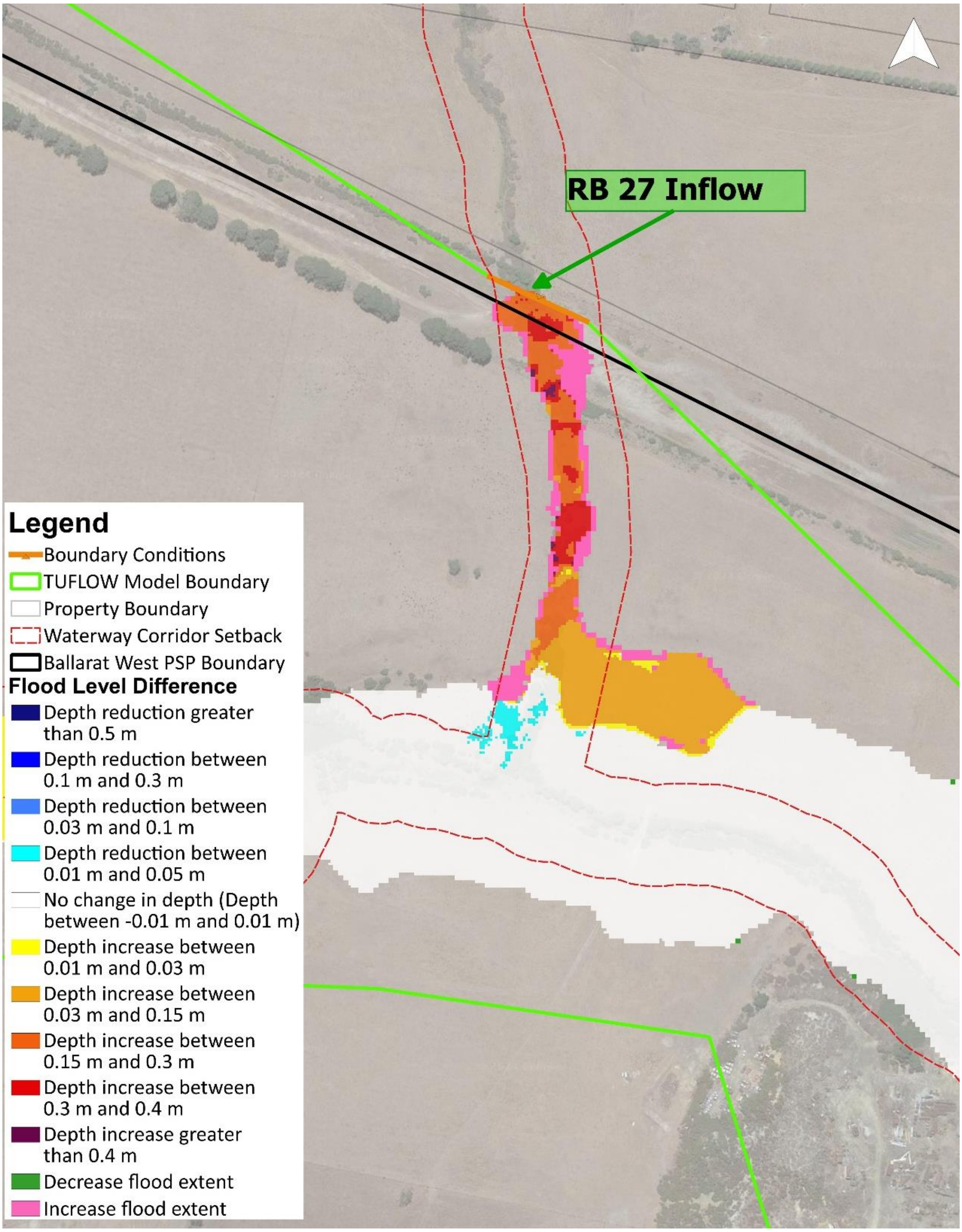

FIGURE 8.3: 1 % AEP FLOOD LEVEL DIFFERENCE FOR SCENARIO 3 ZOOMED TO ~200 M WATERWAY STRETCH

Table 8.1 summarises the peak flows and peak flood level differences for each of the scenario immediately downstream of RB27 proposed locations.

TABLE 8.1: SUMMARY OF RESULTS FOR WATERWAY STRETCH BETWEEN OUTLET OF RB27 AND WINTER CREEK

Scenario	Peak 1% AEP event flows (m^3/s)	Peak flood level difference (m)(Compared to existing conditions)
Existing conditions	10.46	-
Scenario 1	11.03	0.037
Scenario 2	15.21	0.326
Scenario 3	19.51	0.44

8.4 Discussion

Table 8.2 provides a summary of the positives and negatives for each of the design scenario modelled.

TABLE 8.2:SUMMARY OF POSITIVES AND NEGATIVES FOR THE DIFFERENT SCENARIOS

Scenario	Positives	Negatives
Scenario 1 (1% AEP RB)	• Very minor increase in flood level in private property downstream of the PSP, likely will meet the CMA flood level increase regulations. • Small decrease in flood levels (10 mm to 50 mm) downstream at Colac-Ballarat Road	• Building an embankment will increase the risk to future downstream development and will need to meet ANCOLD consequence of failure guidelines • The ANCOLD consequence of failure guidelines will likely require ongoing monitoring of the proposed retarding basin embankment. Changes to downstream land uses, including within the Three Chain Road reserve or the downstream farmland could significantly increase the risk category of the retarding basin and should be considered during design. • Expensive option that will require extensive design and complexing construction
Scenario 2 (Smaller RB)	• Flows discharging from RB27 are returned to pre-development in the 10 % AEP, protecting the waterways and the downstream properties in the more frequent events • Downstream flood increases are mostly contained to within 30 m of the waterway centreline	• Scenario 2 RB27 design will also require an embankment and therefore will increase the risk to future downstream development and will need to meet ACOLD guidelines • Expensive options that will require extensive design and complexing construction • Causing an increase in flood levels (10 mm – 30 mm) at Colac-Ballarat Road (the other two options are resulting in a decrease at this location)
Scenario (No RB)	• Increases in flood levels on waterway between Three Chain Road and Winter Creek • Downstream flood increases are mostly contained to the waterway corridor setback zone • Small decrease in flood levels (10 mm to 50 mm) downstream at Colac-Ballarat Road	• Waterway erosion protection works would be beneficial to protect the waterway from erosion .

9. CONCLUSION

The Ballarat West PSP Drainage Strategy has been updated to consider:

- The past 12 years of development within the precinct which has resulted in the completion of more than half of the proposed stormwater treatment and retardation assets
- Updated technical guidelines, including Australian Rainfall and Runoff 2019, Melbourne Water's Constructed Wetland Design Guidelines and update Environmental Protection Agency guidance on urban stormwater management and the general environmental duty
- Updated stormwater quality modelling in MUSIC and updated stormwater flow management in RORB compliant with the new guidelines.
- Changes to the drainage scheme to respond to the staging of development.

A result of these updates is that the asset sizing and costing has been updated. Generally the proposed footprints for wetland assets has increased, pipe sizes have typically stayed similar or slightly decreased and retarding basin volumes have increased, with the key drivers being the updated ARR 2019 methodologies and the increase in development density.

The plans in Appendix D: show the updated infrastructure layout.

The cost estimates have also been revised but costed using the original methodology. Costs have been increased by 37.4% in line with the change in the road and bridge construction price index (Victoria) from the original stormwater management strategy and this report as published by the Australian Bureau of Statistics.

This strategy document should be used to inform all drainage strategy implementation decisions moving forward. It is also acknowledged that while this update has considered the information available at the time, design considerations have only been undertaken to a concept level. There may be good practical reasons why the designs proposed may need to be adjusted as the design process progresses. This should be considered as an opportunity to improve the proposed designs and ensure that at a minimum the same levels of treatment and retardation are achieved by drainage strategy assets.

10. QUALIFICATIONS

(a) In preparing this document, including all relevant calculation and modelling, Engeny Australia Pty Ltd (Engeny) has exercised the degree of skill, care and diligence normally exercised by members of the engineering profession and has acted in accordance with accepted practices of engineering principles.

(b) Engeny has used reasonable endeavours to inform itself of the parameters and requirements of the project and has taken reasonable steps to ensure that the works and document is as accurate and comprehensive as possible given the information upon which it has been based including information that may have been provided or obtained by any third party or external sources which has not been independently verified.

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   (ii) Engeny considers it prudent to revise any aspect of the works in light of any information which becomes known to it after the date of submission.

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(g) This Report does not provide legal advice.

APPENDIX A: RORB MODEL DETAILS

Table A.1 shows the RORB catchment areas and the breakdown of the directly connected (or effectively connected area (EIA)), indirectly connected (ICA)and rural pervious areas.

TABLE A.1: RORB CATCHMENT AREA AND BREAKDOWN

Subarea	Area (km^2)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
A	0.138	0.39	0.61	0.00
C	0.326	0.43	0.57	0.00
D	0.328	0.43	0.57	0.00
E	0.329	0.38	0.62	0.00
F	0.326	0.32	0.68	0.00
G	0.244	0.41	0.59	0.00
I	0.289	0.39	0.61	0.00
J	0.126	0.47	0.53	0.00
M	0.332	0.39	0.61	0.00
N	0.328	0.31	0.69	0.00
O	0.171	0.22	0.78	0.00
P	0.071	0.43	0.57	0.00
Q	0.087	0.38	0.62	0.00
R	0.249	0.47	0.53	0.00
S	0.229	0.52	0.48	0.00
T	0.196	0.45	0.55	0.00
U	0.133	0.52	0.48	0.00
V	0.307	0.52	0.48	0.00
W	0.232	0.41	0.59	0.00
X	0.194	0.37	0.63	0.00
Y	0.125	0.32	0.69	0.00
Z2	0.076	0.39	0.62	0.00
AA	0.317	0.27	0.73	0.00
AB	0.075	0.40	0.60	0.00
AC	0.066	0.31	0.69	0.00

Subarea	Area (km^2)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
AD	0.103	0.45	0.55	0.00
AE	0.046	0.42	0.58	0.00
AF	0.051	0.53	0.48	0.00
AG	0.007	0.00	0.19	0.81
AH	0.072	0.52	0.48	0.00
AI	0.083	0.52	0.48	0.00
AJ	0.083	0.47	0.53	0.00
AK13	0.114	0.27	0.73	0.00
AL	0.049	0.00	0.18	0.82
AM	0.037	0.00	0.10	0.90
AN	0.123	0.39	0.61	0.00
AO	0.033	0.00	0.11	0.90
AP	0.021	0.00	0.18	0.82
AQ	0.112	0.52	0.48	0.00
AR	0.091	0.51	0.49	0.00
AS	0.069	0.52	0.48	0.00
AT	0.067	0.52	0.48	0.00
AU	0.059	0.52	0.48	0.00
AV	0.057	0.52	0.48	0.00
AW	0.079	0.40	0.60	0.00
AX	0.026	0.53	0.48	0.00
AY	0.084	0.37	0.63	0.00
AZ	0.055	0.38	0.62	0.00
BA	0.112	0.41	0.59	0.00
BB	0.044	0.52	0.48	0.00
BC	0.119	0.48	0.52	0.00
BD	0.130	0.52	0.48	0.00

Subarea	Area (km^2)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
BE	0.072	0.48	0.52	0.00
BF	0.085	0.47	0.53	0.00
BG	0.085	0.38	0.62	0.00
BH	0.031	0.24	0.76	0.00
BI	0.143	0.44	0.56	0.00
BJ	0.075	0.49	0.51	0.00
BK	0.085	0.53	0.48	0.00
BL	0.123	0.51	0.49	0.00
BM	0.140	0.43	0.57	0.00
BN	0.031	0.52	0.48	0.00
BO	0.022	0.00	0.25	0.75
BP	0.029	0.00	0.27	0.73
BQ	0.036	0.34	0.66	0.00
BR	0.049	0.38	0.62	0.00
BS	0.026	0.31	0.69	0.00
BT	0.080	0.40	0.60	0.00
BU	0.061	0.36	0.64	0.00
BV	0.062	0.43	0.57	0.00
BW	0.070	0.46	0.54	0.00
BX1	0.026	0.43	0.57	0.00
BY	0.109	0.42	0.58	0.00
BZ	0.163	0.39	0.61	0.00
CA	0.090	0.37	0.63	0.00
CB	0.121	0.49	0.51	0.00
CC	0.051	0.40	0.60	0.00
CD	0.051	0.42	0.58	0.00
CE	0.071	0.39	0.62	0.00

Subarea	Area (km^2)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
CF	0.015	0.45	0.55	0.00
CG	0.081	0.43	0.57	0.00
CH	0.044	0.52	0.48	0.00
CI	0.090	0.52	0.48	0.00
CJ	0.117	0.52	0.48	0.00
CK	0.144	0.37	0.63	0.00
CL	0.051	0.48	0.52	0.00
CM	0.103	0.00	0.10	0.90
CN	0.047	0.00	0.13	0.87
CO	0.073	0.00	0.22	0.78
CP	0.117	0.50	0.50	0.00
CQ	0.085	0.52	0.48	0.00
CR	0.125	0.52	0.48	0.00
CS	0.186	0.47	0.53	0.00
CT	0.096	0.37	0.63	0.00
CU	0.035	0.53	0.48	0.00
CV	0.100	0.39	0.61	0.00
CW	0.114	0.47	0.53	0.00
CX	0.224	0.31	0.69	0.00
CY	0.027	0.53	0.48	0.00
CZ	0.036	0.52	0.48	0.00
DA	0.081	0.33	0.67	0.00
DB	0.066	0.52	0.48	0.00
DC	0.091	0.41	0.59	0.00
DF	0.044	0.42	0.58	0.00
DI	0.364	0.14	0.86	0.00
DK	0.713	0.41	0.59	0.00

Subarea	Area (km^2)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
DL	0.579	0.25	0.75	0.00
DO	0.124	0.42	0.58	0.00
DP	0.078	0.31	0.69	0.00
DQ	0.062	0.49	0.51	0.00
DX	0.038	0.43	0.57	0.00
DY	0.032	0.52	0.48	0.00
DZ	0.021	0.52	0.48	0.00
EA	0.021	0.42	0.58	0.00
EB	0.082	0.41	0.59	0.00
EC	0.042	0.19	0.81	0.00
ED	0.020	0.08	0.92	0.00
EE	0.063	0.00	0.32	0.68
EF	0.033	0.00	0.30	0.70
EG	0.057	0.00	0.29	0.71
EH	0.036	0.53	0.48	0.00
EI	0.057	0.00	0.43	0.57
EJ	0.062	0.00	0.13	0.87
EK	0.341	0.51	0.49	0.00
EL	0.486	0.51	0.49	0.00
EM	0.175	0.29	0.71	0.00
EN	0.183	0.00	0.11	0.89
EO	0.258	0.00	0.03	0.97
EP	0.299	0.00	0.05	0.95
EQ	0.342	0.00	0.02	0.98
ER	0.376	0.00	0.02	0.98
ES	0.533	0.00	0.02	0.98
ET	0.581	0.28	0.72	0.00

Subarea	Area (km^2)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
EU	0.309	0.00	0.01	0.99
EV	0.228	0.00	0.04	0.96
EW	0.231	0.00	0.04	0.96
EX	0.423	0.00	0.02	0.98
EY	0.228	0.00	0.02	0.98
EZ	0.447	0.00	0.03	0.97
FA	0.143	0.00	0.06	0.94
FB	0.258	0.00	0.05	0.95
FC	0.327	0.00	0.04	0.96
FD	0.282	0.00	0.03	0.97
FE	0.119	0.00	0.16	0.84
FF	0.384	0.00	0.02	0.98
FG	0.361	0.00	0.08	0.92
FH	0.421	0.00	0.02	0.98
FI	0.453	0.00	0.04	0.96
FJ	0.311	0.00	0.05	0.95
FK	0.626	0.00	0.04	0.96
FL	0.222	0.00	0.01	0.99
FM	0.877	0.00	0.03	0.97
FN	0.277	0.00	0.00	1.00
FO	0.564	0.00	0.00	1.00
FP	0.485	0.00	0.02	0.98
FQ	0.962	0.00	0.01	0.99
FR	0.047	0.00	0.17	0.83
FS	0.924	0.00	0.01	0.99
FT	0.032	0.00	0.15	0.85
FU	0.065	0.00	0.00	1.00

Subarea	Area (km^2)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
FV	0.314	0.00	0.00	1.00
FW	0.341	0.00	0.09	0.91
FX	0.478	0.00	0.05	0.95
FY	0.276	0.00	0.04	0.96
FZ	0.167	0.00	0.07	0.93
GA	0.314	0.00	0.11	0.89
GB	0.530	0.00	0.06	0.94
GC	0.684	0.00	0.05	0.95
GD	0.770	0.00	0.03	0.97
GE	0.383	0.00	0.03	0.97
GF	0.379	0.00	0.04	0.96
GG	0.712	0.00	0.02	0.98
GH	0.712	0.00	0.01	0.99
GI	0.755	0.00	0.02	0.98
GJ	0.477	0.00	0.03	0.97
GQ	0.378	0.00	0.04	0.96
GS	0.497	0.00	0.02	0.98
GW	0.538	0.00	0.03	0.97
GX	0.327	0.00	0.01	0.99
GZ	0.397	0.00	0.01	0.99
HA	0.444	0.00	0.03	0.97
HB	0.533	0.00	0.06	0.94
HC	0.308	0.00	0.03	0.97
HD	0.553	0.00	0.03	0.97
HE	0.130	0.00	0.08	0.92
HF	0.517	0.00	0.01	0.99
HG	0.436	0.00	0.02	0.98

Subarea	Area (km^2)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
HM	0.862	0.00	0.04	0.96
HN	0.330	0.00	0.07	0.93
HO	0.519	0.00	0.01	0.99
HP	0.350	0.00	0.03	0.97
HQ	0.125	0.00	0.12	0.88
HR	0.245	0.00	0.10	0.90
HS	1.248	0.00	0.01	0.99
HT	0.794	0.00	0.04	0.96
HU	0.180	0.00	0.04	0.96
HV	0.295	0.00	0.11	0.89
HX	0.518	0.00	0.04	0.96
HY	0.806	0.00	0.03	0.97
HZ	0.476	0.00	0.02	0.98
IA	0.955	0.00	0.02	0.98
IB	0.209	0.00	0.15	0.85
IC	1.108	0.00	0.01	0.99
ID	0.609	0.00	0.03	0.97
IE	0.701	0.00	0.01	0.99
IF	0.353	0.00	0.05	0.95
IG	0.705	0.00	0.02	0.98
IH	1.020	0.00	0.01	0.99
IJ	0.258	0.00	0.03	0.97
IK	0.441	0.00	0.05	0.95
IL	0.540	0.00	0.03	0.97
IM	0.628	0.00	0.03	0.97
IN	0.344	0.00	0.05	0.95
IO	0.409	0.00	0.05	0.95

Subarea	Area (km²)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
IP	0.267	0.00	0.09	0.91
IQ	0.670	0.00	0.00	1.00
IR	0.467	0.00	0.01	0.99
IS	0.710	0.00	0.05	0.95
IT	0.592	0.00	0.02	0.98
IU	0.629	0.00	0.03	0.97
IV	0.809	0.00	0.02	0.98
IW	0.395	0.00	0.07	0.93
IX	0.542	0.00	0.03	0.97
IZ	0.552	0.00	0.05	0.95
JA	0.177	0.00	0.09	0.91
JB	0.524	0.00	0.02	0.98
JC	0.256	0.00	0.07	0.93
JD	0.703	0.00	0.02	0.98
JE	0.521	0.00	0.02	0.98
JF	0.626	0.00	0.00	1.00
JG	0.510	0.00	0.06	0.94
JH	0.429	0.00	0.05	0.95
JI	0.631	0.00	0.02	0.98
JJ	0.399	0.00	0.03	0.97
JK	0.173	0.41	0.59	0.00
JL	0.132	0.07	0.93	0.00
JM	0.131	0.42	0.58	0.00
JN	0.078	0.42	0.58	0.00
JO	0.067	0.43	0.57	0.00
JP	0.129	0.42	0.58	0.00
JQ	0.337	0.42	0.58	0.00

Subarea	Area (km^2)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
JR	0.287	0.42	0.58	0.00
JS	0.222	0.43	0.57	0.00
JT	0.242	0.46	0.54	0.00
JU	0.075	0.00	0.14	0.86
JV	0.283	0.41	0.59	0.00
JW	0.263	0.42	0.58	0.00
JX	0.200	0.38	0.62	0.00
JY	0.177	0.42	0.58	0.00
JZ	0.279	0.40	0.60	0.00
KA	0.327	0.39	0.61	0.00
KB	0.098	0.26	0.74	0.00
KC	0.443	0.17	0.83	0.00
KD	0.498	0.23	0.77	0.00
KE	0.806	0.00	0.02	0.98
KF	0.552	0.22	0.78	0.00
KG	0.333	0.20	0.80	0.00
KH	0.238	0.00	0.04	0.96
KI	0.235	0.19	0.81	0.00
KJ	0.183	0.22	0.78	0.00
KK	0.232	0.28	0.72	0.00
KL	0.201	0.42	0.58	0.00
KM	0.122	0.40	0.60	0.00
KN	0.234	0.42	0.58	0.00
KO	0.255	0.42	0.58	0.00
KP	0.136	0.42	0.58	0.00
KQ	0.096	0.42	0.58	0.00
KR	0.097	0.42	0.58	0.00

Subarea	Area (km^2)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
KS	0.138	0.48	0.52	0.00
KT	0.123	0.46	0.54	0.00
KU	0.064	0.52	0.48	0.00
KV	0.104	0.42	0.58	0.00
KW	0.067	0.51	0.49	0.00
KX	0.184	0.35	0.65	0.00
KY	0.129	0.41	0.59	0.00
KZ	0.139	0.42	0.58	0.00
LA	0.144	0.42	0.58	0.00
LB	0.127	0.42	0.58	0.00
LC	0.143	0.40	0.60	0.00
LD	0.198	0.42	0.58	0.00
LE	0.206	0.40	0.60	0.00
LF	0.224	0.38	0.62	0.00
LG	0.107	0.42	0.58	0.00
LH	0.131	0.34	0.66	0.00
LI	0.077	0.42	0.58	0.00
LJ	0.071	0.42	0.58	0.00
LO	0.667	0.00	0.01	0.99
LP	0.430	0.00	0.03	0.97
LQ	0.265	0.00	0.00	1.00
LR	0.202	0.00	0.04	0.96
LS	0.350	0.00	0.02	0.98
LT	0.465	0.00	0.13	0.87
LU	0.203	0.00	0.08	0.92
LV	0.413	0.00	0.00	1.00
LW	0.570	0.39	0.61	0.00

Subarea	Area (km²)	Fraction Directly Connected	Fraction indirectly Connected	Fraction Rural pervious Area
LX	0.327	0.33	0.67	0.00
LY	0.501	0.44	0.56	0.00
Z1	0.079	0.41	0.59	0.00
AK12	0.056	0.22	0.78	0.00
Le	0.206	0.40	0.60	0.00
LLa	0.118	0.50	0.50	0.00
LLb	0.030	0.42	0.58	0.00
KKe	0.012	0.42	0.58	0.00
LLc	0.026	0.41	0.59	0.00
LLd	0.012	0.41	0.59	0.00
KKf	0.019	0.24	0.76	0.00
HHa	0.145	0.31	0.69	0.00
HHe	0.017	0.31	0.69	0.00
HHd	0.015	0.42	0.58	0.00
HHb	0.067	0.42	0.58	0.00
HHc	0.007	0.42	0.58	0.00
KKc	0.017	0.42	0.58	0.00
KKa	0.073	0.38	0.62	0.00
KKb	0.062	0.42	0.58	0.00
KKd	0.035	0.42	0.58	0.00
BX2	0.039	0.44	0.56	0.00

Figure A.1 shows the layout of the existing conditions RORB model. The figure also shows the PSP boundary in black and the location of a previous model for “The Chase” development which was used in the development of the existing conditions RORB model

Figure A.2 shows the impervious fractions assumed in the developed RORB model. The values in the figure match the values in Table A.1.

Figure A.3 and Figure A.4 show the developed RORB model layout in Precincts 1 and 2.

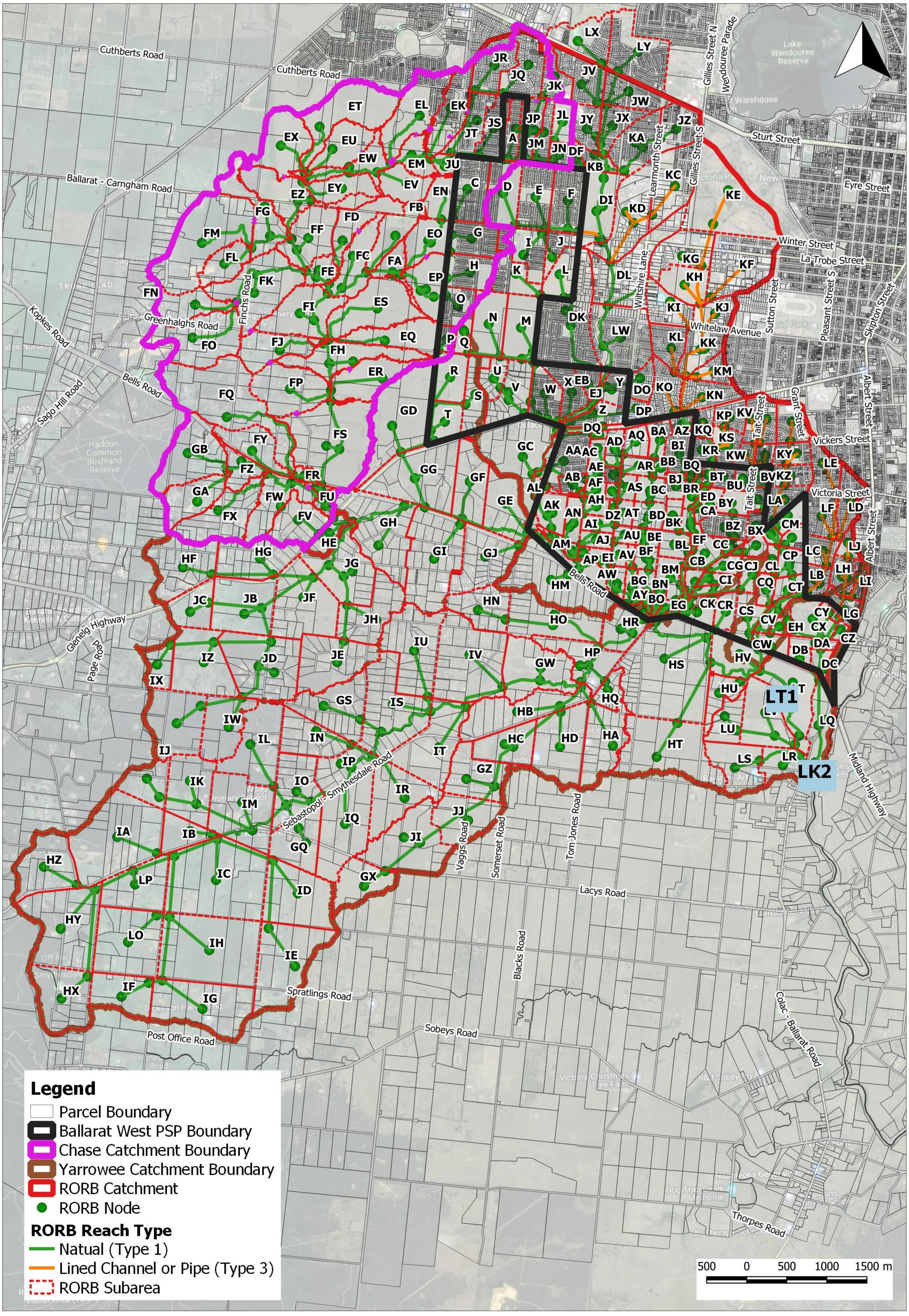


FIGURE A.1: EXISTING CONDITIONS RORB MODEL

ENGENY

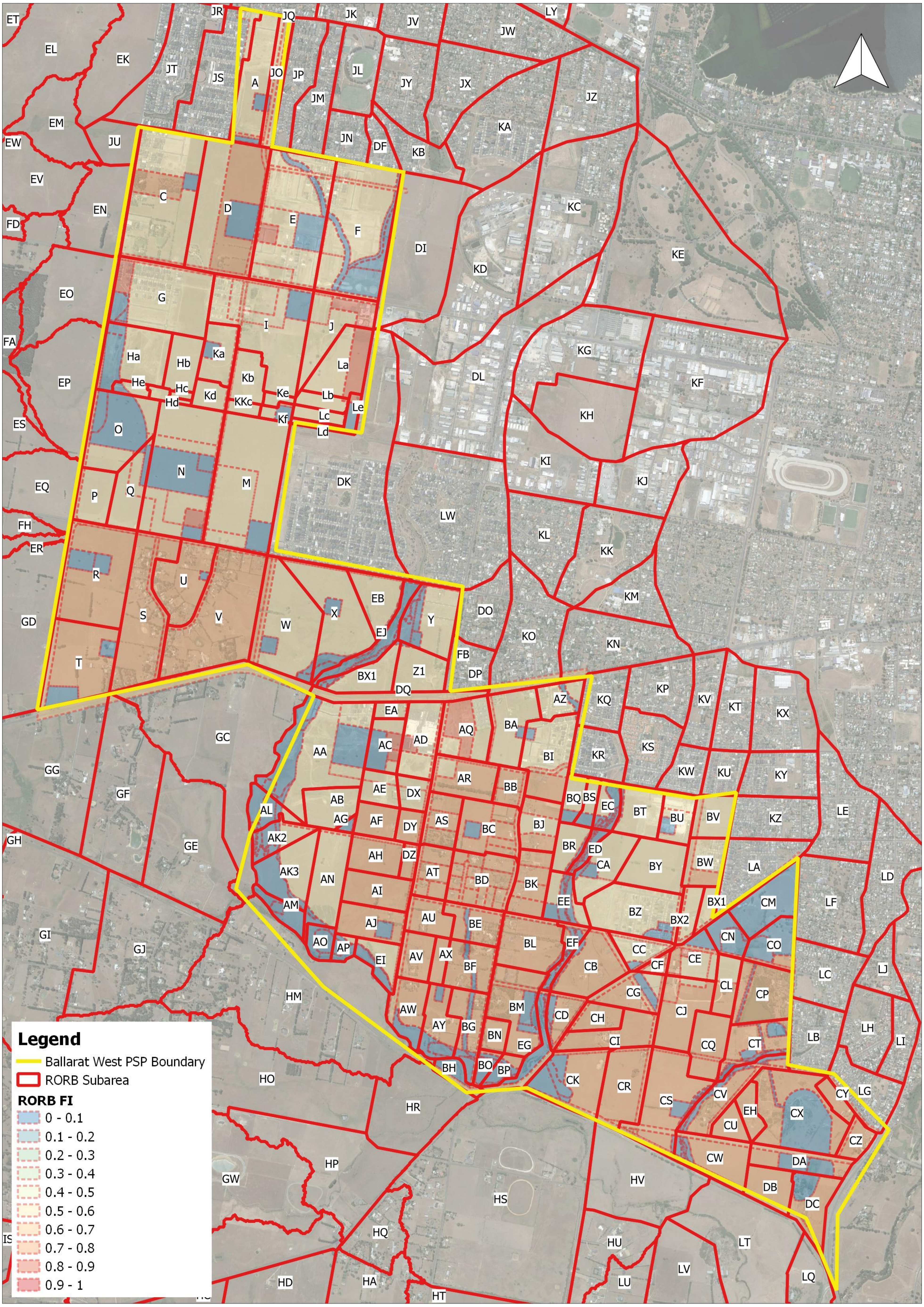


FIGURE A.2: RORB IMPERVIOUS FRACTIONS

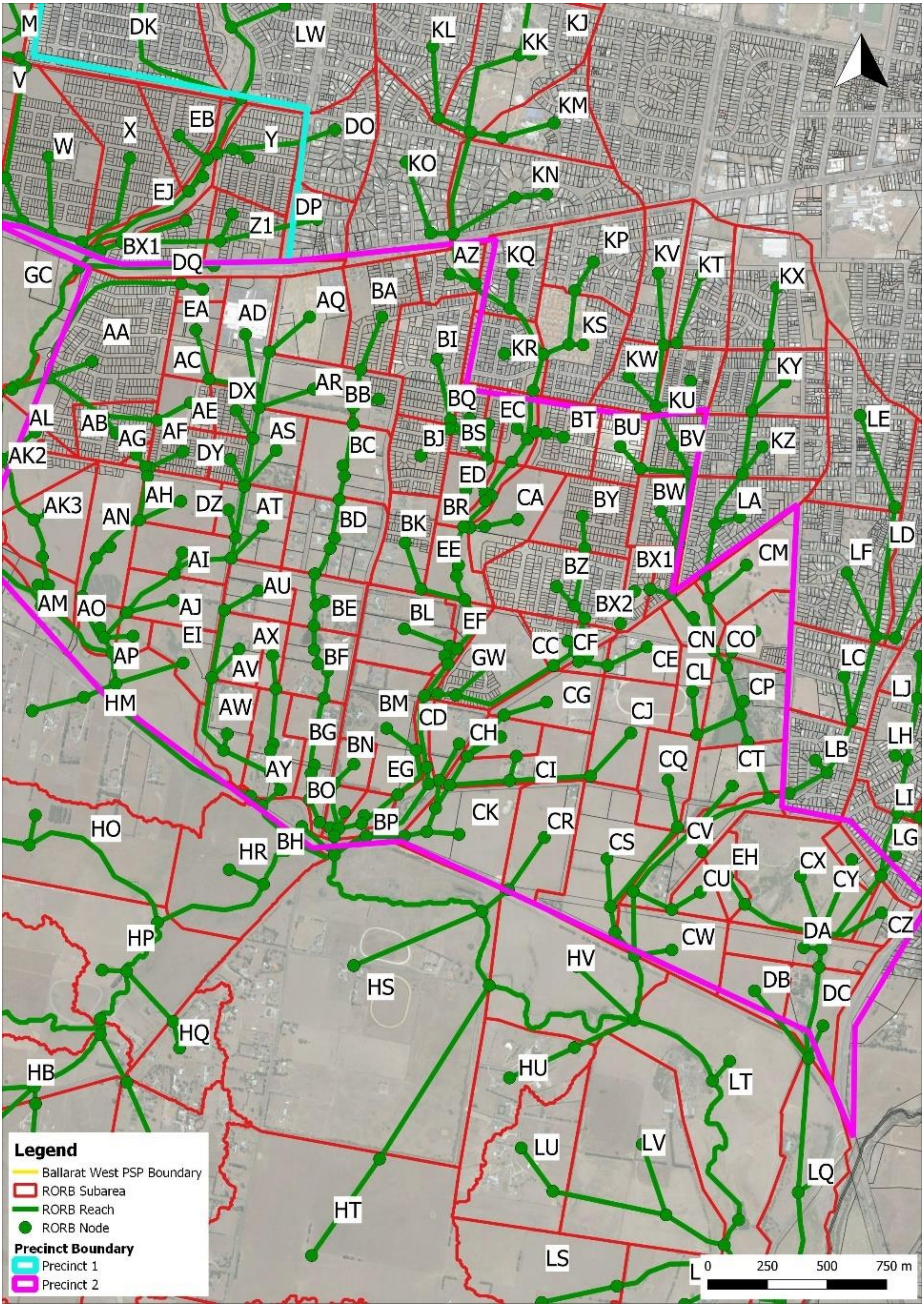


FIGURE A.3: DEVELOPED CONDITIONS PRECINCT 1 RORB LAYOUT

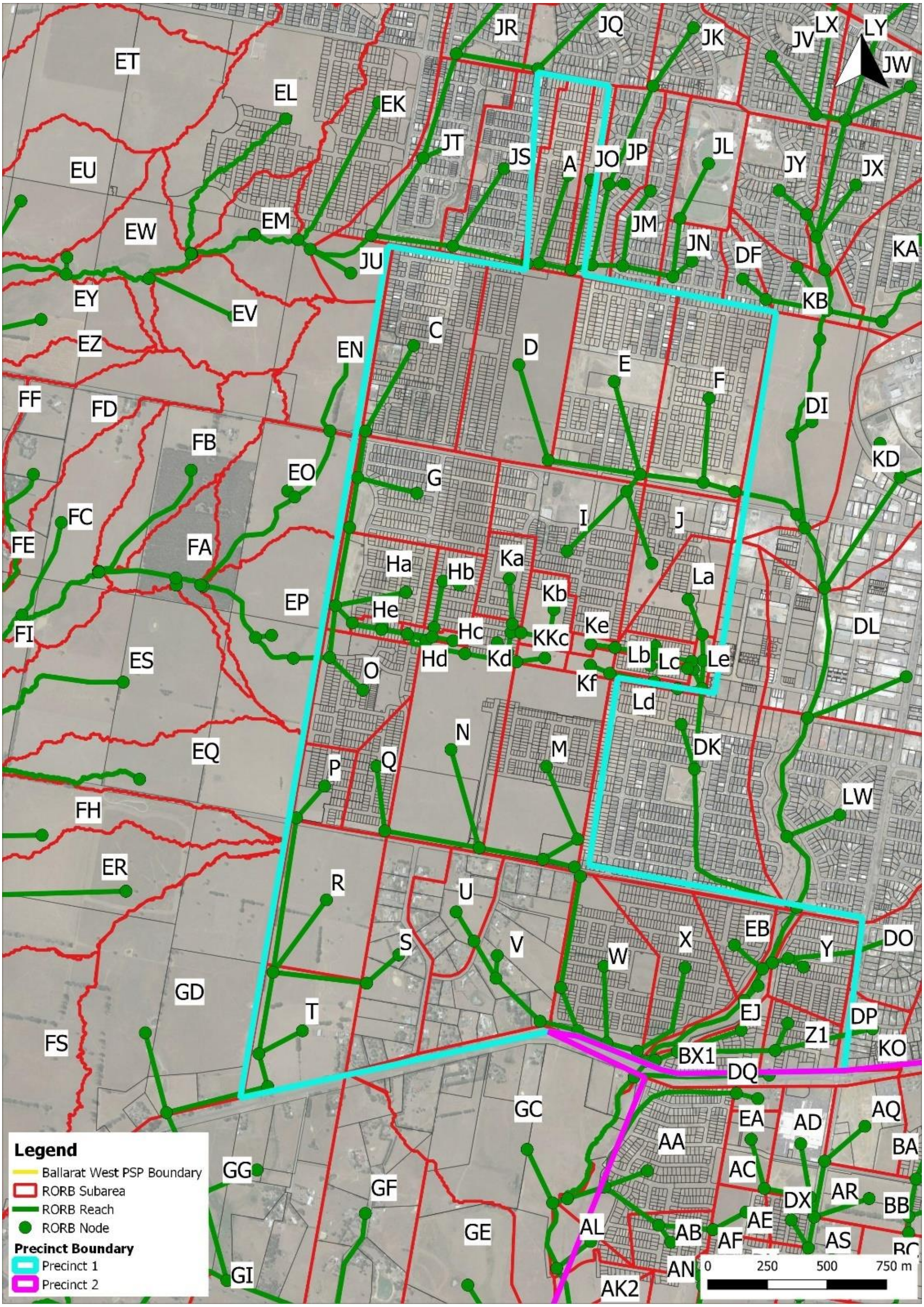


FIGURE A.4: DEVELOPED CONDITIONS PRECINCT 2 RORB LAYOUT

APPENDIX B: SEDIMENTATION BASIN CALCULATIONS

Ballarat West PSP Sediment Basin 7

Surface Area:		calculated using Equation 4.3 of WSUD Engineering Procedures
R (removal fraction)	0.98	change A below to achieve 0.95
hydraulic efficiency	0.26	see Fig 4.3 of WSUD Engineering Procedures, design objective is this value should be 0.5 or higher where possible
n (number of CSTRs)	1.4	calculated using Equation 4.2 of WSUD Engineering Procedures
v_s (m/s)	0.011	settling velocity for 125 micrometre particle size, otherwise see Tabl 4.1 of WSUD Engineering Procedures
d_e (m)	0.35	extended detention depth
d_p (m)	1.5	depth of the permanent pool volume
d* (m)	1.0	sediment can accummulate up to 0.5m below normal water level
A (m^2)	1400	SA of the sediment pond
Side lenth:width ratio 1:	3	
Q (m^3/s)	0.75	use 4EY flow
Required volume:		
S	587	
C (ha)	75	catchment Area
R	0.98	capture efficiency from above equation (not less than 0.95)
L (m^3/ha)	1.6	sediment loading rate (1.6m3/ha is typical loading rate for developed catchments)
Fr (years)	5	desired clean out frequency, should be 3 years or greater
Permanent Pool Volume (PPV)		
PPV Req:	880	accumulated sediment not to exceed 2/3 of available storage volume within 5 years (MW Constructed Wetlands Guidelines)
Estimated minimum PPV	1218	Assumes rectanglular shape with ratio specified above and saefty bench specified in dam capacity calcs
	OK	

Ballarat West PSP Sediment Basin 13

Surface Area:	calculated using Equation 4.3 of WSUD Engineering Procedures	
R (removal fraction)	0.99	change A below to achieve 0.95
hydraulic efficiency	0.26	see Fig 4.3 of WSUD Engineering Procedures, design objective is this value should be 0.5 or higher where possible
n (number of CSTRs)	1.4	calculated using Equation 4.2 of WSUD Engineering Procedures
v_s (m/s)	0.011	settling velocity for 125 micrometre particle size, otherwise see Tabl 4.1 of WSUD Engineering Procedures
d_e (m)	0.35	extended detention depth
d_p (m)	1.5	depth of the permanent pool volume
d* (m)	1.0	sediment can accummulate up to 0.5m below normal water level
A (m^2)	2000	SA of the sediment pond
Side lenth:width ratio 1:	3	
Q (m^3/s)	0.54	use 4EY flow
Required volume:		
S	1214	
C (ha)	122.2	catchment Area
R	0.99	capture efficiency from above equation (not less than 0.95)
L (m^3/ha)	2	sediment loading rate (1.6m3/ha is typical loading rate for developed catchments)
Fr (years)	5	desired clean out frequency, should be 3 years or greater
Permanent Pool Volume (PPV)		
PPV Req:	1821	accumulated sediment not to exceed 2/3 of available storage volume within 5 years (MW Constructed Wetlands Guidelines)
Estimated minimum PPV	1926	Assumes rectanglular shape with ratio specified above and saefty bench specified in dam capacity calcs
	OK	

Ballarat West PSP Sediment Basin 14

Surface Area:	calculated using Equation 4.3 of WSUD Engineering Procedures	
R (removal fraction)	0.99	change A below to achieve 0.95
hydraulic efficiency	0.26	see Fig 4.3 of WSUD Engineering Procedures, design objective is this value should be 0.5 or higher where possible
n (number of CSTRs)	1.4	calculated using Equation 4.2 of WSUD Engineering Procedures
v_s (m/s)	0.011	settling velocity for 125 micrometre particle size, otherwise see Tabl 4.1 of WSUD Engineering Procedures
d_e (m)	0.35	extended detention depth
d_p (m)	1.5	depth of the permanent pool volume
d* (m)	1.0	sediment can accummulate up to 0.5m below normal water level
A (m^2)	700	SA of the sediment pond
Side lenth:width ratio 1:	3	
Q (m^3/s)	0.27	use 1EY flow
Required volume:		
S	302	
C (ha)	30.5	catchment Area
R	0.99	capture efficiency from above equation (not less than 0.95)
L (m^3/ha)	2	sediment loading rate (1.6m3/ha is typical loading rate for developed catchments)
Fr (years)	5	desired clean out frequency, should be 3 years or greater
Permanent Pool Volume (PPV)		
PPV Req:	453	accumulated sediment not to exceed 2/3 of available storage volume within 5 years (MW Constructed Wetlands Guidelines)
Estimated minimum PPV	457	Assumes rectanglular shape with ratio specified above and saefty bench specified in dam capacity calcs
	OK	

Ballarat West PSP Sediment Basin 15

Surface Area:		calculated using Equation 4.3 of WSUD Engineering Procedures
R (removal fraction)	0.99	change A below to achieve 0.95
hydraulic efficiency	0.26	see Fig 4.3 of WSUD Engineering Procedures, design objective is this value should be 0.5 or higher where possible
n (number of CSTRs)	1.4	calculated using Equation 4.2 of WSUD Engineering Procedures
v_s (m/s)	0.011	settling velocity for 125 micrometre particle size, otherwise see Tabl 4.1 of WSUD Engineering Procedures
d_e (m)	0.35	extended detention depth
d_p (m)	1.5	depth of the permanent pool volume
d* (m)	1.0	sediment can accummulate up to 0.5m below normal water level
A (m^2)	1200	SA of the sediment pond
Side lenth:width ratio 1:	3	
Q (m^3/s)	0.34	use 1EY flow
Required volume:		
S	643	
C (ha)	64.7	catchment Area
R	0.99	capture efficiency from above equation (not less than 0.95)
L (m^3/ha)	2	sediment loading rate (1.6m3/ha is typical loading rate for developed catchments)
Fr (years)	5	desired clean out frequency, should be 3 years or greater
Permanent Pool Volume (PPV)		
PPV Req:	964	accumulated sediment not to exceed 2/3 of available storage volume within 5 years (MW Constructed Wetlands Guidelines)
Estimated minimum PPV	991	Assumes rectanglular shape with ratio specified above and saefty bench specified in dam capacity calcs
	OK	

Ballarat West PSP Sediment Basin 17

Surface Area:	calculated using Equation 4.3 of WSUD Engineering Procedures	
R (removal fraction)	0.98	change A below to achieve 0.95
hydraulic efficiency	0.26	see Fig 4.3 of WSUD Engineering Procedures, design objective is this value should be 0.5 or higher where possible
n (number of CSTRs)	1.4	calculated using Equation 4.2 of WSUD Engineering Procedures
v_s (m/s)	0.011	settling velocity for 125 micrometre particle size, otherwise see Tabl 4.1 of WSUD Engineering Procedures
d_e (m)	0.35	extended detention depth
d_p (m)	1.5	depth of the permanent pool volume
d* (m)	1.0	sediment can accummulate up to 0.5m below normal water level
A (m^2)	600	SA of the sediment pond
Side lenth:width ratio 1:	3	
Q (m^3/s)	0.32	use 1EY flow
Required volume:		
S	219	
C (ha)	22.2	catchment Area
R	0.98	capture efficiency from above equation (not less than 0.95)
L (m^3/ha)	2	sediment loading rate (1.6m3/ha is typical loading rate for developed catchments)
Fr (years)	5	desired clean out frequency, should be 3 years or greater
Permanent Pool Volume (PPV)		
PPV Req:	328	accumulated sediment not to exceed 2/3 of available storage volume within 5 years (MW Constructed Wetlands Guidelines)
Estimated minimum PPV	358	Assumes rectanglular shape with ratio specified above and saefty bench specified in dam capacity calcs
	OK	

Ballarat West PSP Sediment Basin 24

Surface Area:	calculated using Equation 4.3 of WSUD Engineering Procedures	
R (removal fraction)	0.99	change A below to achieve 0.95
hydraulic efficiency	0.26	see Fig 4.3 of WSUD Engineering Procedures, design objective is this value should be 0.5 or higher where possible
n (number of CSTRs)	1.4	calculated using Equation 4.2 of WSUD Engineering Procedures
v_s (m/s)	0.011	settling velocity for 125 micrometre particle size, otherwise see Tabl 4.1 of WSUD Engineering Procedures
d_e (m)	0.35	extended detention depth
d_p (m)	1.5	depth of the permanent pool volume
d* (m)	1.0	sediment can accummulate up to 0.5m below normal water level
A (m^2)	900	SA of the sediment pond
Side lenth:width ratio 1:	3	
Q (m^3/s)	0.43	use 4EY flow
Required volume:		
S	416	
C (ha)	53	catchment Area
R	0.99	capture efficiency from above equation (not less than 0.95)
L (m^3/ha)	1.6	sediment loading rate (1.6m3/ha is typical loading rate for developed catchments)
Fr (years)	5	desired clean out frequency, should be 3 years or greater
Permanent Pool Volume (PPV)		
PPV Req:	624	accumulated sediment not to exceed 2/3 of available storage volume within 5 years (MW Constructed Wetlands Guidelines)
Estimated minimum PPV	663	Assumes rectanglular shape with ratio specified above and saefty bench specified in dam capacity calcs
	OK	

Ballarat West PSP Sediment Basin 27

Surface Area:		calculated using Equation 4.3 of WSUD Engineering Procedures
R (removal fraction)	0.98	change A below to achieve 0.95
hydraulic efficiency	0.26	see Fig 4.3 of WSUD Engineering Procedures, design objective is this value should be 0.5 or higher where possible
n (number of CSTRs)	1.4	calculated using Equation 4.2 of WSUD Engineering Procedures
v_s (m/s)	0.011	settling velocity for 125 micrometre particle size, otherwise see Tabl 4.1 of WSUD Engineering Procedures
d_e (m)	0.35	extended detention depth
d_p (m)	1.5	depth of the permanent pool volume
d* (m)	1.0	sediment can accummulate up to 0.5m below normal water level
A (m^2)	700	SA of the sediment pond
Side lenth:width ratio 1:	3	
Q (m^3/s)	0.43	use 4EY flow
Required volume:		
S	253	
C (ha)	32	catchment Area
R	0.98	capture efficiency from above equation (not less than 0.95)
L (m^3/ha)	1.6	sediment loading rate (1.6m3/ha is typical loading rate for developed catchments)
Fr (years)	5	desired clean out frequency, should be 3 years or greater
Permanent Pool Volume (PPV)		
PPV Req:	379	accumulated sediment not to exceed 2/3 of available storage volume within 5 years (MW Constructed Wetlands Guidelines)
Estimated minimum PPV	457	Assumes rectanglular shape with ratio specified above and saefty bench specified in dam capacity calcs
	OK	

Ballarat West PSP Sediment Basin 27B

Surface Area:	calculated using Equation 4.3 of WSUD Engineering Procedures	
R (removal fraction)	0.97	change A below to achieve 0.95
hydraulic efficiency	0.26	see Fig 4.3 of WSUD Engineering Procedures, design objective is this value should be 0.5 or higher where possible
n (number of CSTRs)	1.4	calculated using Equation 4.2 of WSUD Engineering Procedures
v_s (m/s)	0.011	settling velocity for 125 micrometre particle size, otherwise see Tabl 4.1 of WSUD Engineering Procedures
d_e (m)	0.35	extended detention depth
d_p (m)	1.5	depth of the permanent pool volume
d* (m)	1.0	sediment can accummulate up to 0.5m below normal water level
A (m^2)	600	SA of the sediment pond
Side lenth:width ratio 1:	3	
Q (m^3/s)	0.56	use 4EY flow
Required volume:		
S	193	
C (ha)	25	catchment Area
R	0.97	capture efficiency from above equation (not less than 0.95)
L (m^3/ha)	1.6	sediment loading rate (1.6m3/ha is typical loading rate for developed catchments)
Fr (years)	5	desired clean out frequency, should be 3 years or greater
Permanent Pool Volume (PPV)		
PPV Req:	290	accumulated sediment not to exceed 2/3 of available storage volume within 5 years (MW Constructed Wetlands Guidelines)
Estimated minimum PPV	358	Assumes rectanglular shape with ratio specified above and saefty bench specified in dam capacity calcs
	OK	

Ballarat West PSP Sediment Basin 29

Surface Area:	calculated using Equation 4.3 of WSUD Engineering Procedures	
R (removal fraction)	0.98	change A below to achieve 0.95
hydraulic efficiency	0.26	see Fig 4.3 of WSUD Engineering Procedures, design objective is this value should be 0.5 or higher where possible
n (number of CSTRs)	1.4	calculated using Equation 4.2 of WSUD Engineering Procedures
v_s (m/s)	0.011	settling velocity for 125 micrometre particle size, otherwise see Tabl 4.1 of WSUD Engineering Procedures
d_e (m)	0.35	extended detention depth
d_p (m)	1.5	depth of the permanent pool volume
d* (m)	1.0	sediment can accummulate up to 0.5m below normal water level
A (m^2)	1200	SA of the sediment pond
Side lenth:width ratio 1:	3	
Q (m^3/s)	0.65	use 4EY flow
Required volume:		
S	622	
C (ha)	79	catchment Area
R	0.98	capture efficiency from above equation (not less than 0.95)
L (m^3/ha)	1.6	sediment loading rate (1.6m3/ha is typical loading rate for developed catchments)
Fr (years)	5	desired clean out frequency, should be 3 years or greater
Permanent Pool Volume (PPV)		
PPV Req:	933	accumulated sediment not to exceed 2/3 of available storage volume within 5 years (MW Constructed Wetlands Guidelines)
Estimated minimum PPV	991	Assumes rectanglular shape with ratio specified above and saefty bench specified in dam capacity calcs
	OK	

Ballarat West PSP Sediment Basin 30

Surface Area:	calculated using Equation 4.3 of WSUD Engineering Procedures	
R (removal fraction)	0.98	change A below to achieve 0.95
hydraulic efficiency	0.26	see Fig 4.3 of WSUD Engineering Procedures, design objective is this value should be 0.5 or higher where possible
n (number of CSTRs)	1.4	calculated using Equation 4.2 of WSUD Engineering Procedures
v_s (m/s)	0.011	settling velocity for 125 micrometre particle size, otherwise see Tabl 4.1 of WSUD Engineering Procedures
d_e (m)	0.35	extended detention depth
d_p (m)	1.5	depth of the permanent pool volume
d* (m)	1.0	sediment can accummulate up to 0.5m below normal water level
A (m^2)	1500	SA of the sediment pond
Side lenth:width ratio 1:	3	
Q (m^3/s)	1.00	use 4EY flow
Required volume:		
S	780	
C (ha)	100	catchment Area
R	0.98	capture efficiency from above equation (not less than 0.95)
L (m^3/ha)	1.6	sediment loading rate (1.6m3/ha is typical loading rate for developed catchments)
Fr (years)	5	desired clean out frequency, should be 3 years or greater
Permanent Pool Volume (PPV)		
PPV Req:	1171	accumulated sediment not to exceed 2/3 of available storage volume within 5 years (MW Constructed Wetlands Guidelines)
Estimated minimum PPV	1334	Assumes rectanglular shape with ratio specified above and saefty bench specified in dam capacity calcs
	OK	

APPENDIX C: MUSIC MODEL SETUP AND INDIVIDUAL ASSET RESULTS

Table C.1 to Table C.10 shows the treatment performance of each individual wetland and sedimentation basin asset in the drainage strategy which is not yet constructed or committed. The Precinct 1 MUSIC model contains a number of low flow diversions so the results presented are node balances and not total treatment train effectiveness (which includes all upstream assets as well). Adjustments to the low flow diversions may impact on the pollutant removal achieved by each asset and so care should be taken when adjusting low flow diversions to consider the impact on the treatment achieved in all assets.

TABLE C.1: WLRB7

	Inflow	Outflow	Reduction
Flow (ML/yr)	243	225	7.3
Total Suspended Solids (kg/yr)	49900	15100	69.8
Total Phosphorus (kg/yr)	101	39.9	60.5
Total Nitrogen (kg/yr)	700	404	42.4
Gross Pollutants (kg/yr)	10800	1400	87.1

TABLE C.2: WLRB13

	Inflow	Outflow	Reduction
Flow (ML/yr)	229	216	5.7
Total Suspended Solids (kg/yr)	46500	15900	65.8
Total Phosphorus (kg/yr)	94.2	41.3	56.1
Total Nitrogen (kg/yr)	657	409	37.8
Gross Pollutants (kg/yr)	10200	1850	81.7

TABLE C.3: WLRB14

	Inflow	Outflow	Reduction
Flow (ML/yr)	92.4	86.7	6.2
Total Suspended Solids (kg/yr)	19000	6610	65.2
Total Phosphorus (kg/yr)	38.3	17	55.7
Total Nitrogen (kg/yr)	266	163	38.5
Gross Pollutants (kg/yr)	4190	586	86

TABLE C.4: WLRB15

	Inflow	Outflow	Reduction
Flow (ML/yr)	158	152	3.8
Total Suspended Solids (kg/yr)	32200	14000	56.6
Total Phosphorus (kg/yr)	65.2	34.7	46.8
Total Nitrogen (kg/yr)	456	322	29.3
Gross Pollutants (kg/yr)	7060	1410	80.1

TABLE C.5: WLRB17

	Inflow	Outflow	Reduction
Flow (ML/yr)	158	152	3.8
Total Suspended Solids (kg/yr)	32200	14000	56.6
Total Phosphorus (kg/yr)	65.2	34.7	46.8
Total Nitrogen (kg/yr)	456	322	29.3
Gross Pollutants (kg/yr)	7060	1410	80.1

TABLE C.6: WLRB24

	Inflow	Outflow	Reduction
Flow (ML/yr)	300	283	5.8
Total Suspended Solids (kg/yr)	47000	19900	57.8
Total Phosphorus (kg/yr)	99.3	51.7	48
Total Nitrogen (kg/yr)	746	503	32.6
Gross Pollutants (kg/yr)	9240	2030	78

TABLE C.7: WLRB27

	Inflow	Outflow	Reduction
Flow (ML/yr)	458	454	0.8
Total Suspended Solids (kg/yr)	50800	40100	21.1
Total Phosphorus (kg/yr)	115	104	9.6
Total Nitrogen (kg/yr)	971	928	4.4
Gross Pollutants (kg/yr)	6460	1190	81.6

TABLE C.8: SB27B

	Inflow	Outflow	Reduction
Flow (ML/yr)	74.8	74.3	0.7
Total Suspended Solids (kg/yr)	15300	5710	62.6
Total Phosphorus (kg/yr)	31.2	17	45.5
Total Nitrogen (kg/yr)	215	174	19.1
Gross Pollutants (kg/yr)	3400	0	100

TABLE C.9: WLRB29

	Inflow	Outflow	Reduction
Flow (ML/yr)	205	190	7.3
Total Suspended Solids (kg/yr)	41100	12400	69.8
Total Phosphorus (kg/yr)	84.2	33.5	60.2
Total Nitrogen (kg/yr)	591	340	42.4
Gross Pollutants (kg/yr)	9550	1240	87

TABLE C.10: SB30

	Inflow	Outflow	Reduction
Flow (ML/yr)	205	190	7.3
Total Suspended Solids (kg/yr)	41100	12400	69.8
Total Phosphorus (kg/yr)	84.2	33.5	60.2
Total Nitrogen (kg/yr)	591	340	42.4
Gross Pollutants (kg/yr)	9550	1240	87

Figure C.2 and Figure C.1 show the MUSIC model layouts.

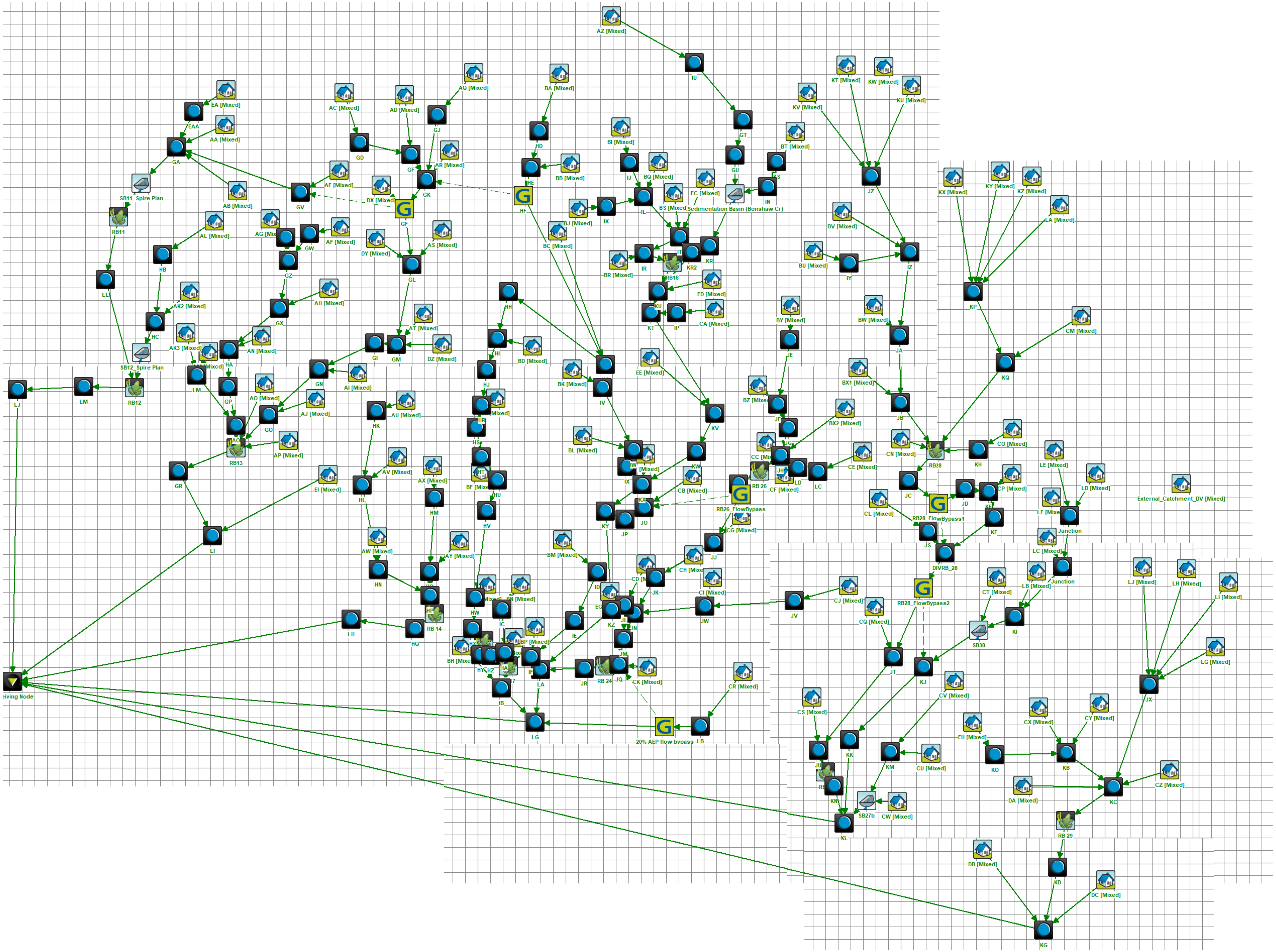


FIGURE C.1: PRECINCT 1 MUSIC MODEL LAYOUT

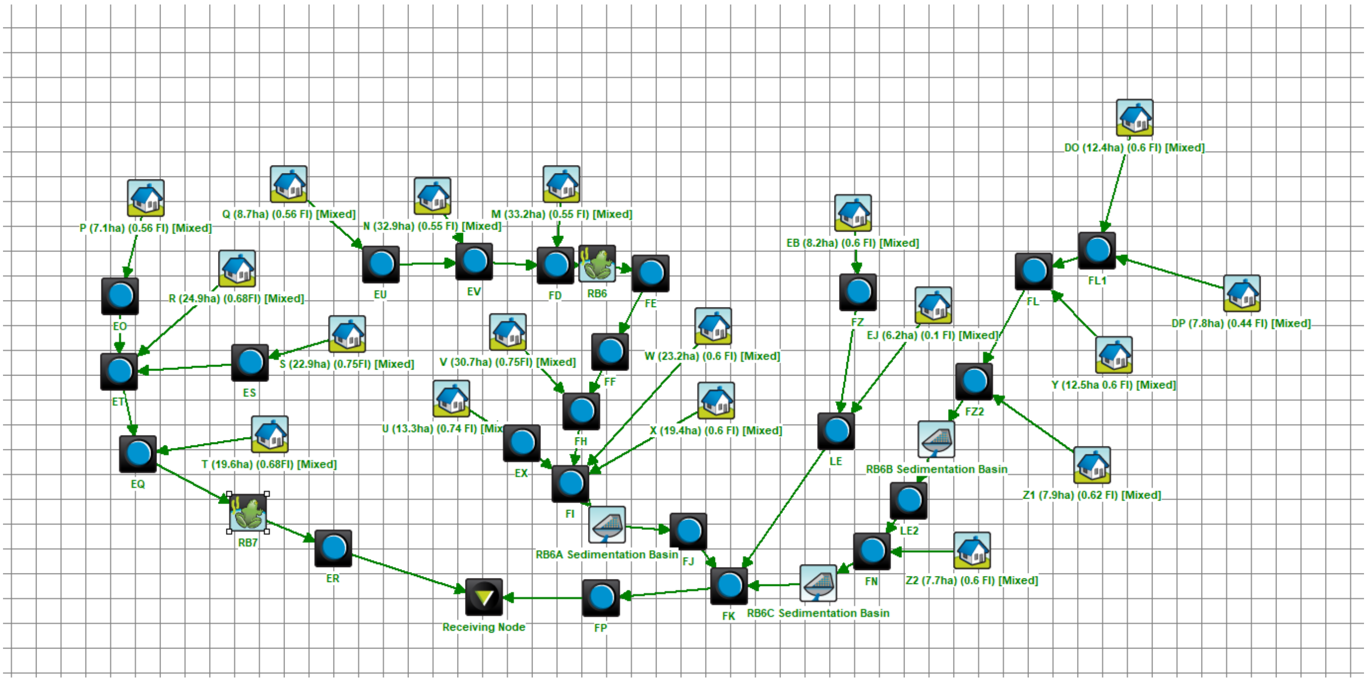
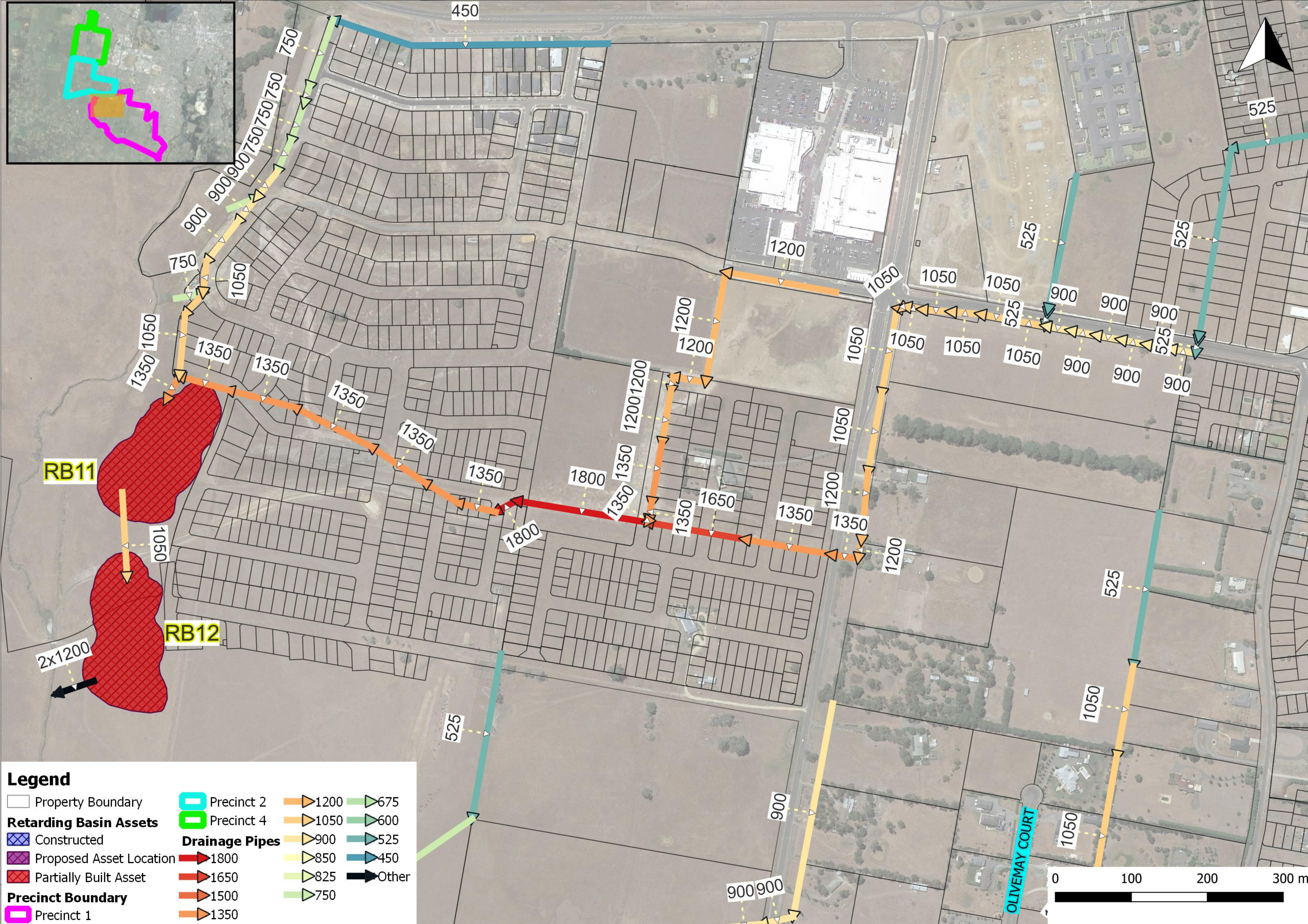
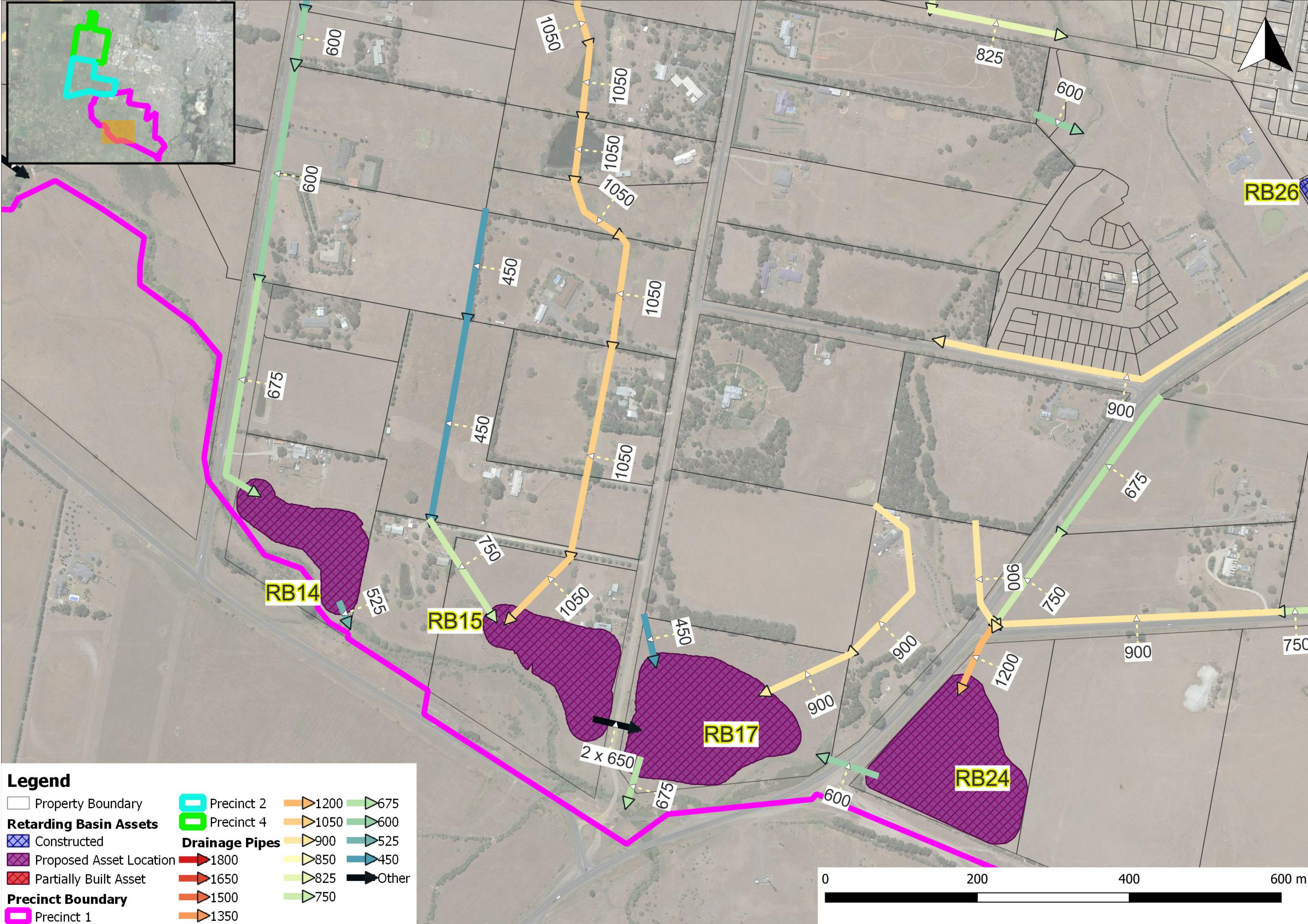


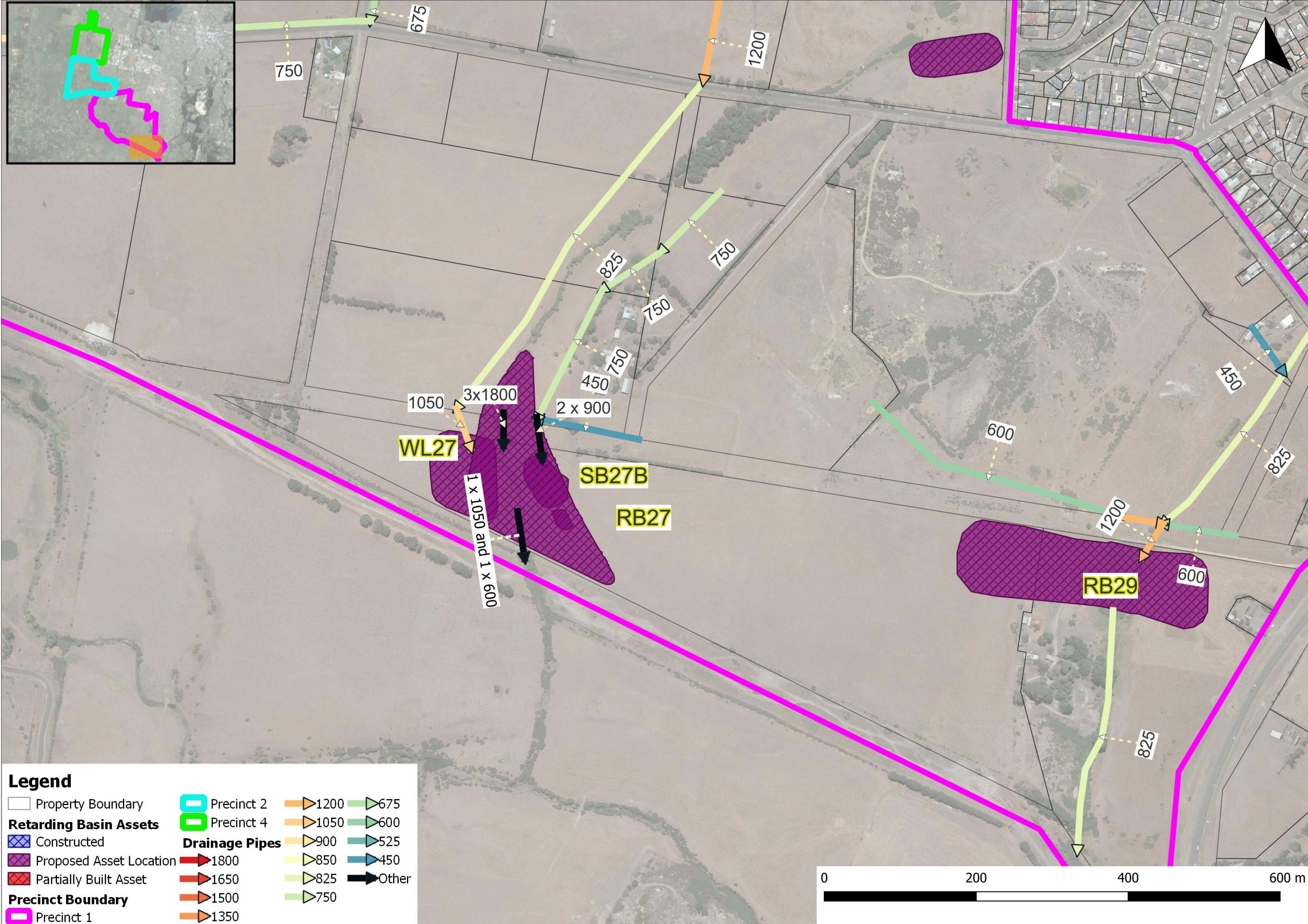
FIGURE C.2: PRECINCT 2 MUSIC MODEL LAYOUT

APPENDIX D: UPDATED DRAINAGE STRATEGY LAYOUT

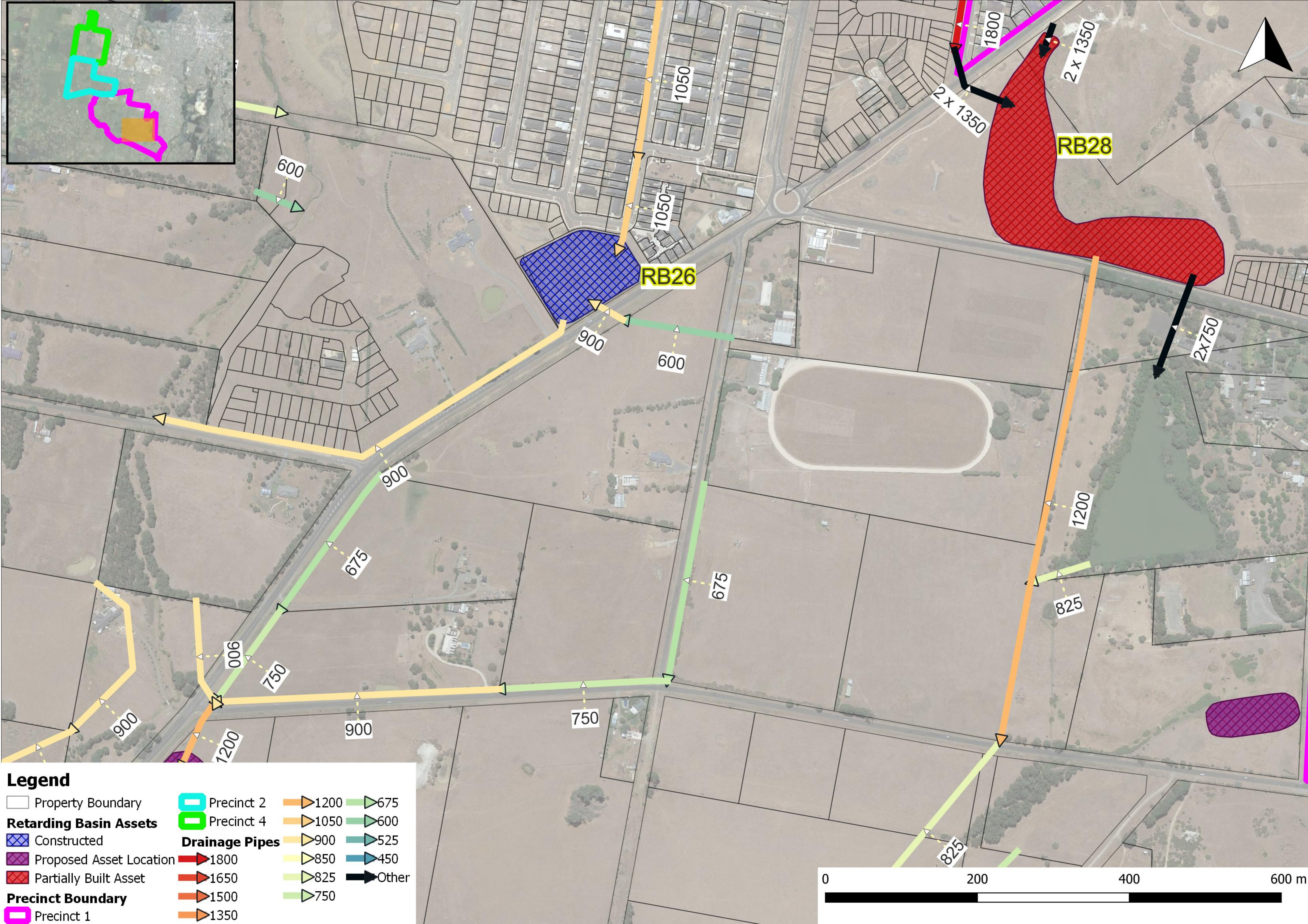

RB11
RB12
2x1200
OLIVEMAY COURT
Legend
Property Boundary
Retarding Basin Assets
Constructed
Proposed Asset Location
Partially Built Asset
Precinct Boundary
Precinct 1
Precinct 2
Precinct 4
Drainage Pipes
1800
1650
1500
1350
1200
1050
900
850
825
750
675
600
525
450
Other
0
100
200
300 m

1350
525
1050
825
600
525
900
750
900
900
1050
675
SCHREENANS ROAD
1050
750
900
525
1050
825
825
900
600
1050
RB13
600
1050
2 x 825
1050
600
450
1050
900
675
450
Legend
Property Boundary
Retarding Basin Assets
Constructed
Proposed Asset Location
Partially Built Asset
Precinct Boundary
Precinct 1
Precinct 2
Precinct 4
Drainage Pipes
1800
1650
1500
1350
1200
1050
900
850
825
750
675
600
525
450
Other
0
200
400
600 m


RB14
RB15
RB17
RB24
RB26
600
675
450
750
1050
1200
900
825
525
2 x 650
Legend
Property Boundary
Retarding Basin Assets
Constructed
Proposed Asset Location
Partially Built Asset
Precinct Boundary
Precinct 1
Precinct 2
Precinct 4
Drainage Pipes
1800
1650
1500
1350
1200
1050
900
850
825
750
675
600
525
450
Other
0
200
400
600 m



675
750
1200
825
750
750
750
450
1050
3x1800
2 x 900
WL27
SB27B
RB27
1 x 1050 and 1 x 600
600
450
825
1200
600
RB29
825
Legend
Property Boundary
Retarding Basin Assets
Constructed
Proposed Asset Location
Partially Built Asset
Precinct Boundary
Precinct 1
Precinct 2
Precinct 4
Drainage Pipes
1800
1650
1500
1350
1200
1050
900
850
825
750
675
600
525
450
Other
0
200
400
600 m



RB26
RB28
1800
2 x 1350
2 x 1350
1050
1050
600
600
900
900
900
900
900
900
2x750
1200
1200
825
825
675
675
750
750
Legend
Property Boundary
Retarding Basin Assets
Constructed
Proposed Asset Location
Partially Built Asset
Precinct Boundary
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Precinct 2
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450
Other
0
200
400
600 m