Horsham South Structure Plan

Catchment A Concept Stormwater Strategy

4th April 2024

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Report for: **Horsham Rural City Council**

Document Verification

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Note that for the purposes of this report, Stormy Water Solutions (**SWS**) consists of two business entities being:

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Contents

1 Introduction

Horsham Rural City Council (**Council**) are in the process of preparing a Structure Plan (**SP**) for Horsham South. Stormy Water Solutions (**SWS)** has been engaged by Council to develop concept designs of the major drainage assets which will be required to service Catchment A (as defined in Figure 4) of the SP.

The current SP proposals are generally described in the report '*Horsham South, Issues and Opportunities Background Report, Mesh, October 2019*' (the **Mesh Background Report**). The most current proposals are shown in Figure 1.

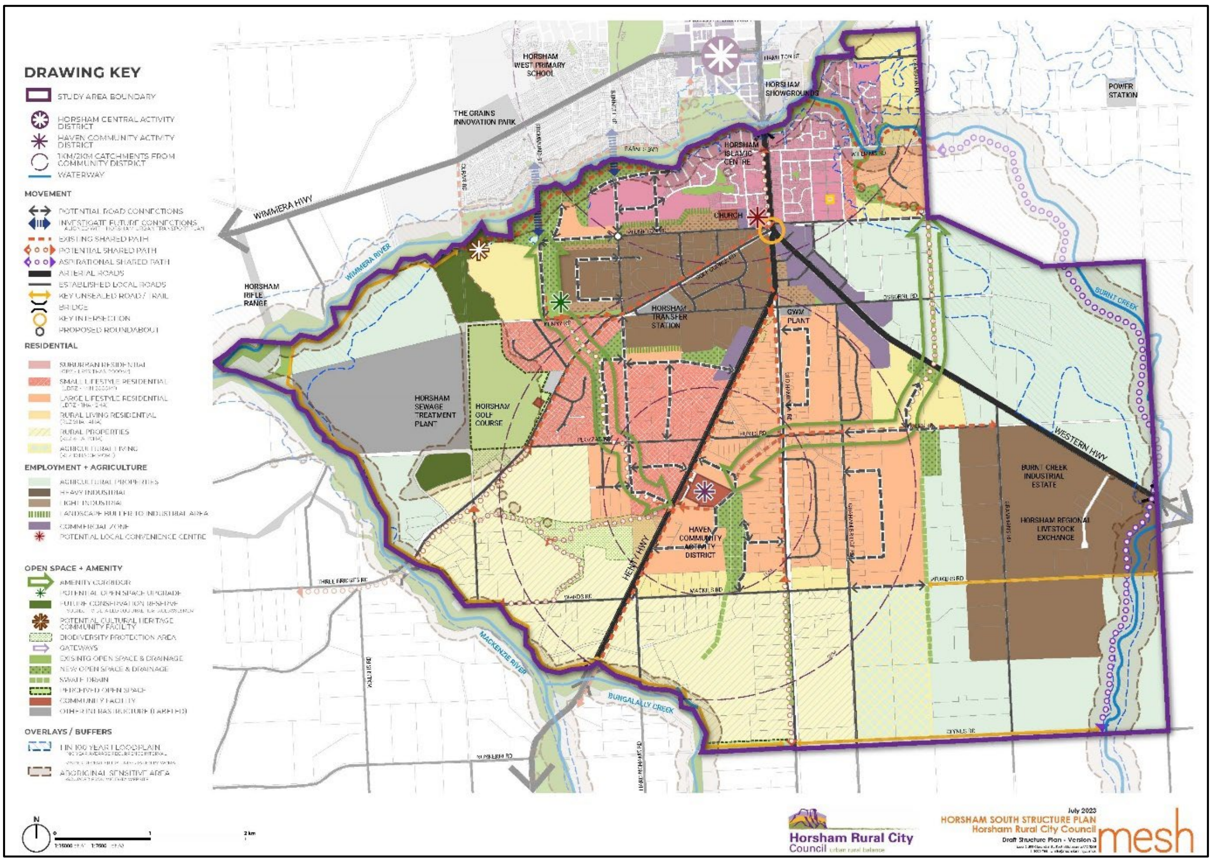


Figure 1 SP Proposals. Source: July 2023 Proposal

Generally the SP Proposal <u>do not</u> intend to change the land zonings within the SP region. Figure 2 shows the current zonings across much of the SP region.

Currently much of the region is currently zoned either:

- Rural Living Zone (**RLZ**);
- Low Density Residential Zone (**LDRZ**); or
- Industrial 1 Zone (**IN1Z**).

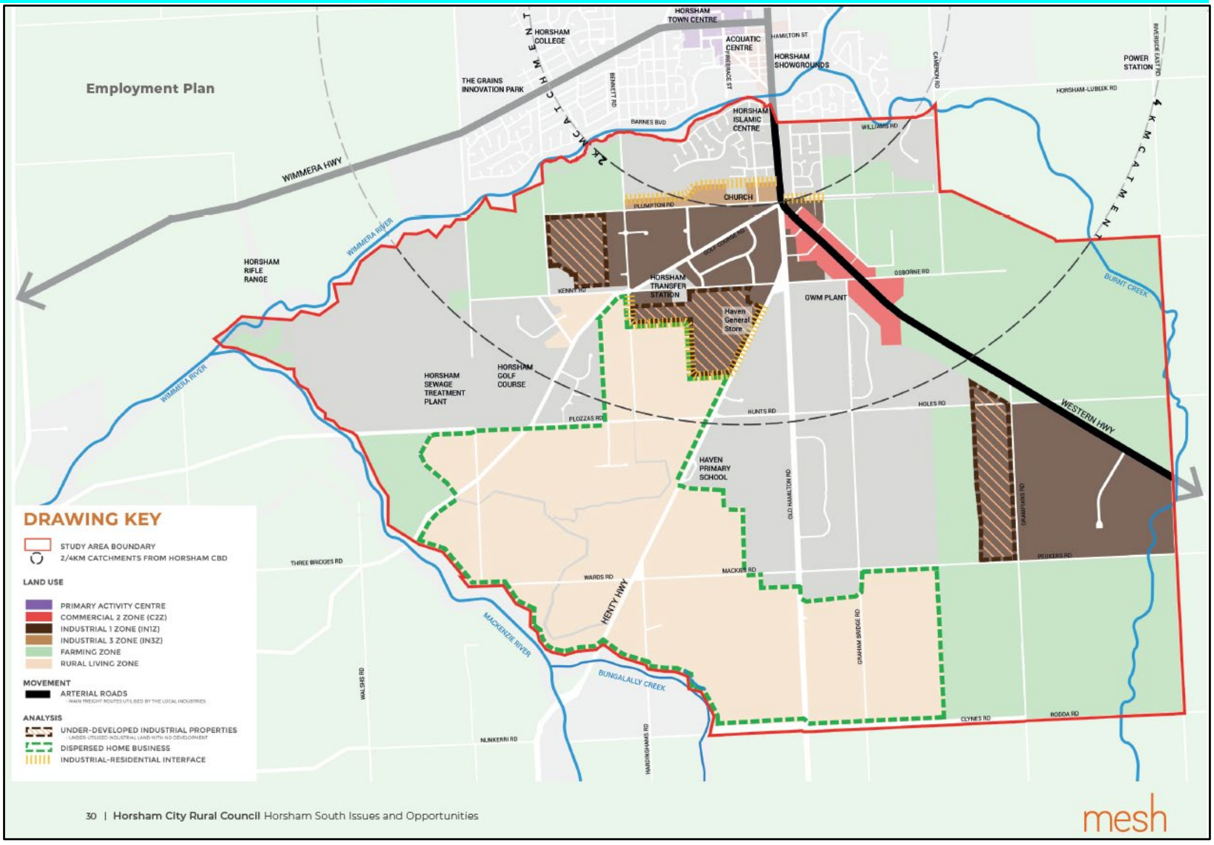


Figure 2 **Current Zonings within the SP Region. Source: Mesh Background Report**

However, the current uses, and lot densities do not necessarily reflect the current zonings of the land.

As such, into the future, when land is developed, or sub-divided, there is the potential for detrimental stormwater impacts. Drainage assets (and reserves) to mitigate these impacts must be allowed for in the SP proposals going forward.

Figure 3 below shows the areas that changes are expected into the future.

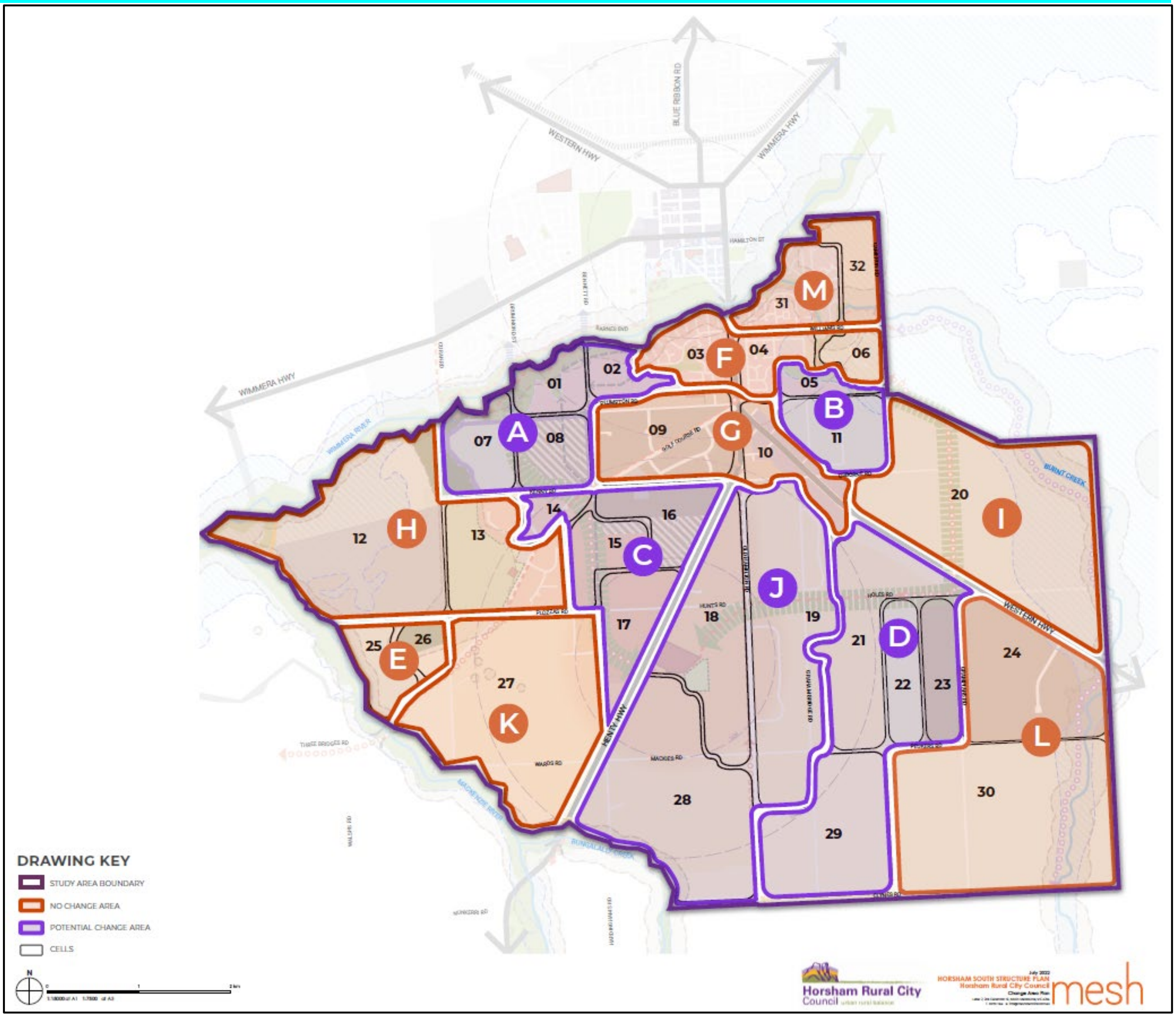


Figure 3 **SP Change Area Plan. Source: Mesh July 2022.**
Purple: Potential "change" areas
Orange: Potential "no change" areas

In late 2022, SWS was engaged by Council and prepared the report "Horsham South Structure Plan, Preliminary Drainage Assessment, 9/03/2023, Stormy Water Solutions" (the **PDA**).

The PDA was a high-level assessment that sought to provide Council a direction as to how stormwater runoff could be managed within the SP moving forward. Figure 4 details the primary (possible) drainage assets required as identified in the PDA.

The PDA identified nine major catchments/outfalls from the SP region as shown in Figure 4.

The PDA identified catchment A as the highest priority catchment as this catchment covers much of the change areas 'A', 'C' and 'J' in Figure 3, including the Haven township and IN1Z land in the northeast of the SP as shown in Figure 5.

Thus, this report has been produced to document further design development of the drainage proposals within Catchment A.

The drainage system designs presented herein are to a Concept design standard. That is that the drainage reserve land takes are expected to be reasonable going forward into the design process. Further work will be required to obtain cost estimates of the assets for implantation into an infrastructure contributions plan (**ICP**) or development contributions plan (**DCP**) as the SP is progressed.

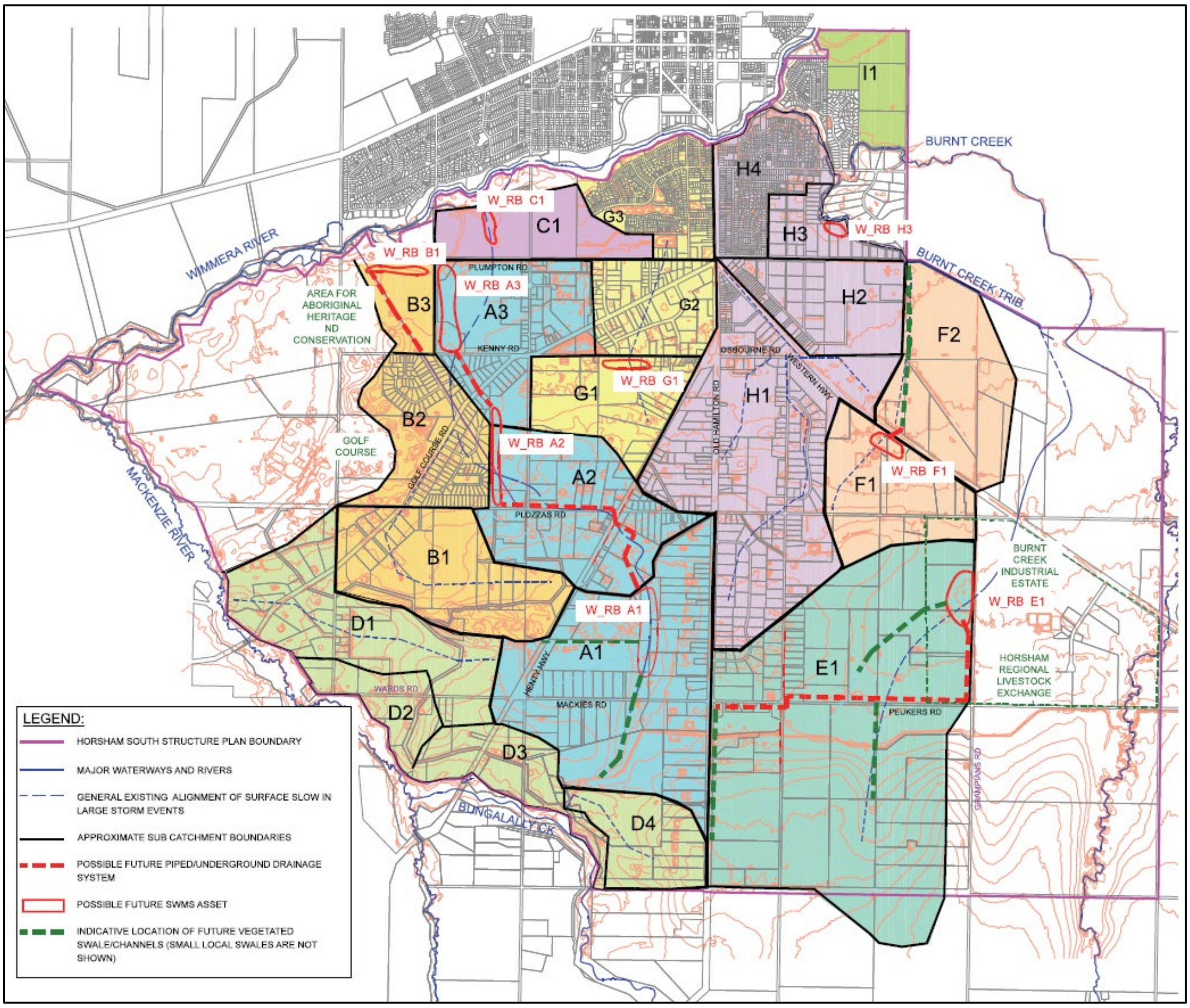


Figure 4 **PDA recommendations.**

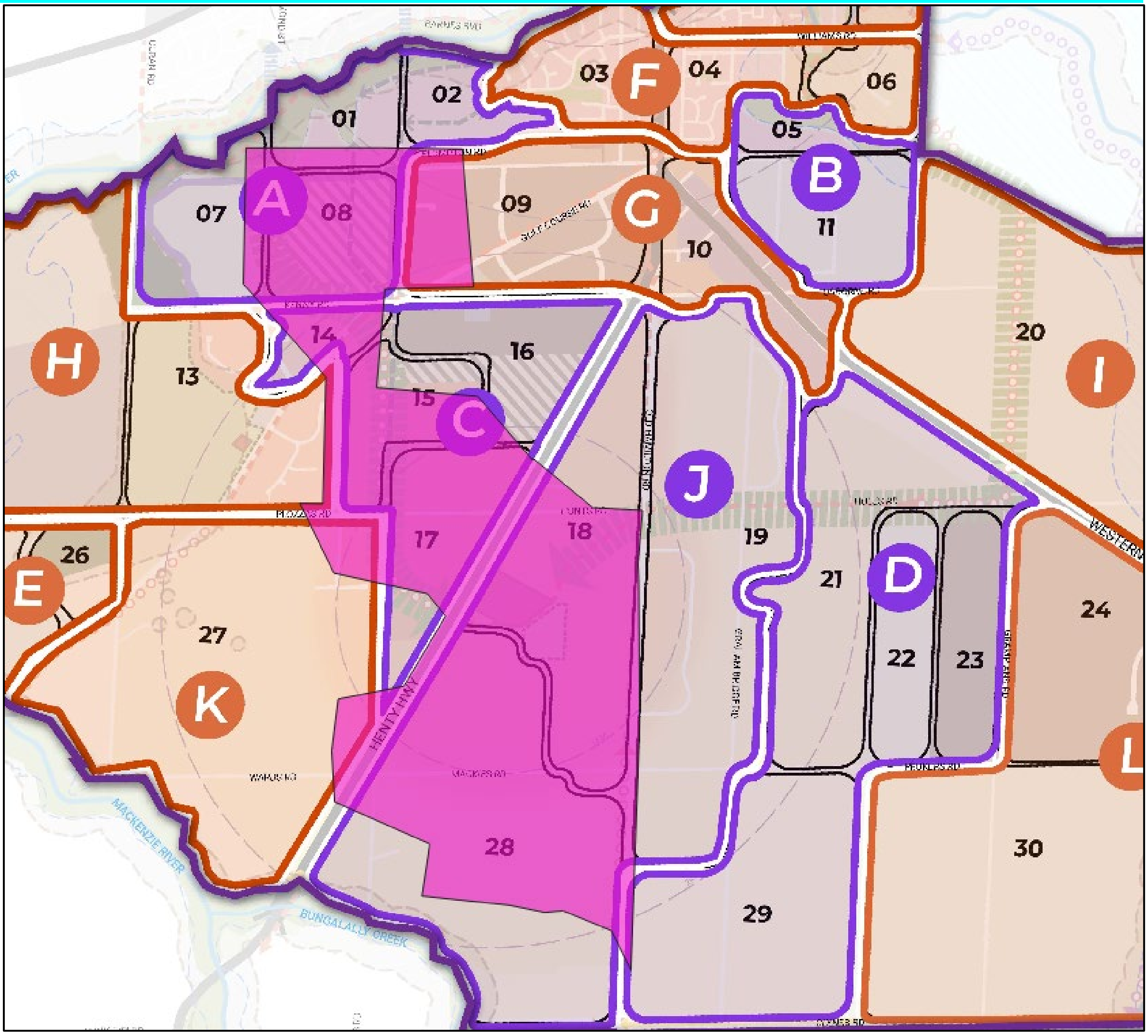


Figure 5 **Approximate overlay of the PDA's Catchment A (magenta) on the Change Plan.**
Note that the Catchment A boundary has changed (from that shown above) given the results of the current work – see Figure 6.

Version 2 of this report incorporates minor changes to reserve allocations, given Council request to optimise reserve allocations going forward. No change to required system sizes has occurred from version 1 of this report.

2 Background Reports, Information and Designs

The formulation of this document has had regard to the information from the following sources relating to designs, studies, models and/or current works in the catchments/sites surrounding the SP. Information obtained from each source below is described in more detail in subsequent parts of this report where required.

- The report “Stormwater Drainage, Water and Sewer Infrastructure Assessment Report, TGM Group, May 2019” (**2019 TGM Report**);
- The report “Final Report, Horsham and Wartook Valley Flood Investigation, V02, 16/08/2019, Water Technology” (the **Regional Flood Mapping**);
- Aerial LiDAR flown in September 2019 (the **LiDAR information**);
- The report “Horsham South, Issues and Opportunities Background Report, October 2019, Mesh” (the **Mesh Background Report**);
- The report “Kenny Road, Haven – Stormwater Management Plan, Midbrook Pty Ltd, June 2022, V02, 28/06/2022, Water Technology” (the **Kenny Road SWMS**);
- The Report “Horsham South Structure Plan, Preliminary Drainage Assessment, 9/03/2023, Stormy Water Solutions” (the **PDA**);
- The Permit PA2200512 for 55 Kenny Road, 15/05/2023 (the **Kenny Road Permit**);
- The drawing “Horsham South Structure Plan, Horsham Rural City Council, Draft Structure Plan – Version 3, July 2023, Mesh” (the **July 2023 Proposal**);
- General planning scheme information available on the VicPlan website, accessed in July 2023, <https://mapshare.vic.gov.au/vicplan/>;
- Nearmap aerial imagery (dates in figures as required); and
- Observations made at site visits from SWS staff to the general region on the 25th and 26th of August 2022.

3 Manuals and Guidelines

Where applicable, the designs developed herein will generally be consistent the “Infrastructure Design Manual, Local Government Infrastructure Design Association, V5.40, 1/09/2022” (the **IDM**). However, as the designs proposed also cover assets that are not common within the IDM, the following Manuals or Guidelines are also referenced:

1. CSIRO (1999). “Urban Stormwater Best Practice Environmental Management Guidelines.” CSIRO PUBLISHING, Melbourne (**BPEMG**);
2. Melbourne Water (2005). “*WSUD Engineering Procedures: Stormwater Melbourne”*, CSIRO Publishing (the **WSUD Engineering Procedures**);
3. Melbourne Water (2013), “*Waterway Corridors, Guidelines for greenfield development areas within the Port Phillip and Westernport Region*” (the **Greenfield Waterway Guidelines**);
4. Melbourne Water (2018). “*MUSIC Guidelines - Input parameters and modelling approaches for MUSIC users in Melbourne Water’s service area”,* Melbourne Water (the **MUSIC Tool Guidelines**);
5. Australian Rainfall and Runoff 2019, Geoscience Australia, (**ARR 2019**);
6. Department of Environment, Land, Water and Planning (**DELWP**) (2019). “Guidelines for Development in Flood Affected Areas”, February 2019, DELWP (the **DELWP Flood Guidelines**);
7. Melbourne Water (2020). “*Wetland Design Manual, Part A2: Deemed to Comply Criteria*.”, (the **Wetland Design Manual**); and
8. Environmental Protection Agency Victoria (2021), ‘*Urban Stormwater Management Guidance*’, publication 1739.1, June 2021 (the **EPA Guidance**).

Despite being referenced to assist in the design of assets herein, the above Manuals or Guidelines, specifically the Wetland Design Manual, have not been followed exactly. The catchment characteristics, and the Council maintenance regimes (which are both different to a typical Melbourne Water application) result in (minor) aspects of the Wetland Design Manual not being achieved.

4 Catchment Characteristics

4.1 Size

The PDA utilised 1 m contour information provided by Council to determine its catchment boundaries.

This project has predominantly utilised the LiDAR information. The LiDAR information has allowed a better understanding of the catchment to be obtained. The use of the LiDAR information has increased the Catchment A estimated size from the PDA as shown in Figure 6. The estimated size increased because the catchment is relatively flat and the 1 metre contour information did not adequately define the catchment in the PDA.

The catchment within Figure 6 is reasonable. However, given the many (large) local depressions, and irrigation channels throughout the SP region, having exact catchment delineations is difficult (as they change depending on the severity of the storm event). Generally, the catchments within Figure 6 are what the catchment is expected to be in the 1% Annual Exceedance Probability (**AEP**) event.

This larger Catchment A (compared to the PDA) is what has been assumed herein. The catchment has a size of in the order of 1,047 ha to the existing outfall culverts at the intersect of Plumpton Road and the unnamed road, in the north west of the catchment.

It is also noted that the catchment is relatively flat. Typical grades of 1V:300H to 1V:500H are common across much of Catchment A. This has the potential to make traditional pit-and-pipe servicing of the catchment difficult without large quantities of fill.

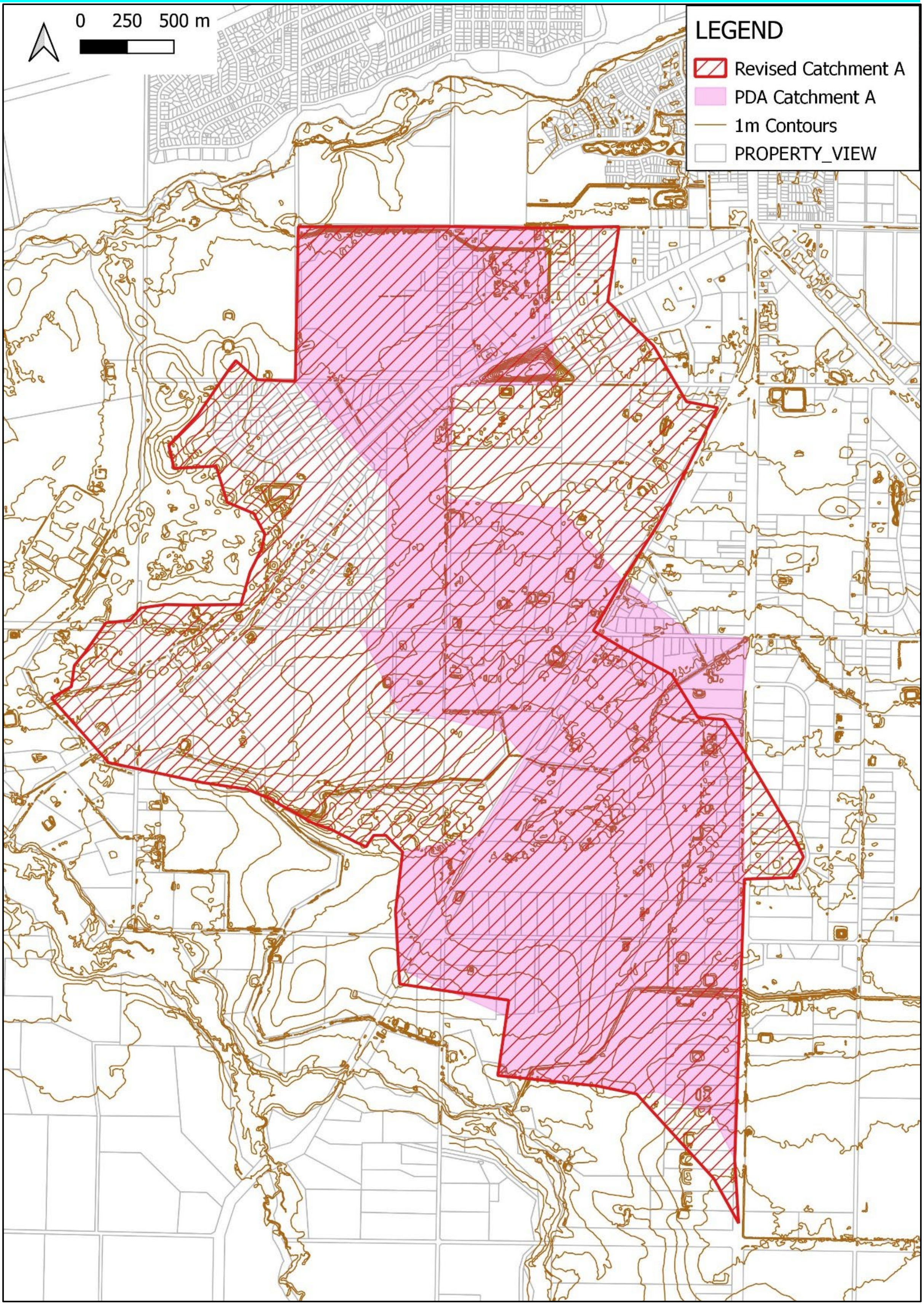


Figure 6 Total Catchment A definition

4.2 Land use assumptions

Within hydrological models, land-use (and its change) are typically simulated with a fraction impervious (**F_{imp}**) value. F_{imp} is the ratio of the impervious areas to the total area of the catchment.

With reference to the MUSIC Tool Guidelines, the current land zoning, Nearmap imagery and site visit observations, Table 1 below summarises the existing and future land use assumptions made herein within catchment A. Figure 7 shows these assumptions spatially across the catchment.

Table 1 F_{imp} assumptions made

PDA Catchment (see Figure 4)	Current Land Zones (approx.)	Typical existing use	Typical existing F_{imp}	Expected change	Expected F_{imp}
A1	RLZ	Generally, as per zoning with large farm lots (4 ha typical)	0.05	North of Mackies Road - 1 to 2 ha rural living South of Mackies Road - 2 ha (min) rural living	North of Mackies Road - 0.20 South of Mackies Road - 0.10
A2	PUZ6 & RLZ	Generally, as per zoning with large farm lots	0.20	Haven west of Henty Hwy - 2,000 m2 lots Haven east of Henty Hwy - 1 ha lots	Haven west of Henty Hwy - 0.30 Haven east of Henty Hwy - 0.30
A3	PPRZ, RLZ & IN1Z	Generally as per zoning with 4,500 m2 typical lots in the RLZ, but the IN1Z land is farmed, not developed	PPRZ = 0.05 RLZ = 0.35 IN1Z = 0.05	No change to PPRZ and RLZ. INZ1 assumed developed	PPRZ = 0.05 RLZ = 0.35 IN1Z = 0.70
B1	RLZ	Generally, as per zoning with some small lots (2ha typical) and some larger lots (4ha typical)	0.10	Zoning remains the same, but all lots are assumed to be 2 ha	0.20
B2	SUZ1 & LDRZ	Generally, as per zoning with 4,000 m^2 typical lots	0.35	No Change	0.35
G1 & G2	IN1Z	G1 IN1Z land is farmed, not developed	0.05	INZ1 assumed developed	0.70

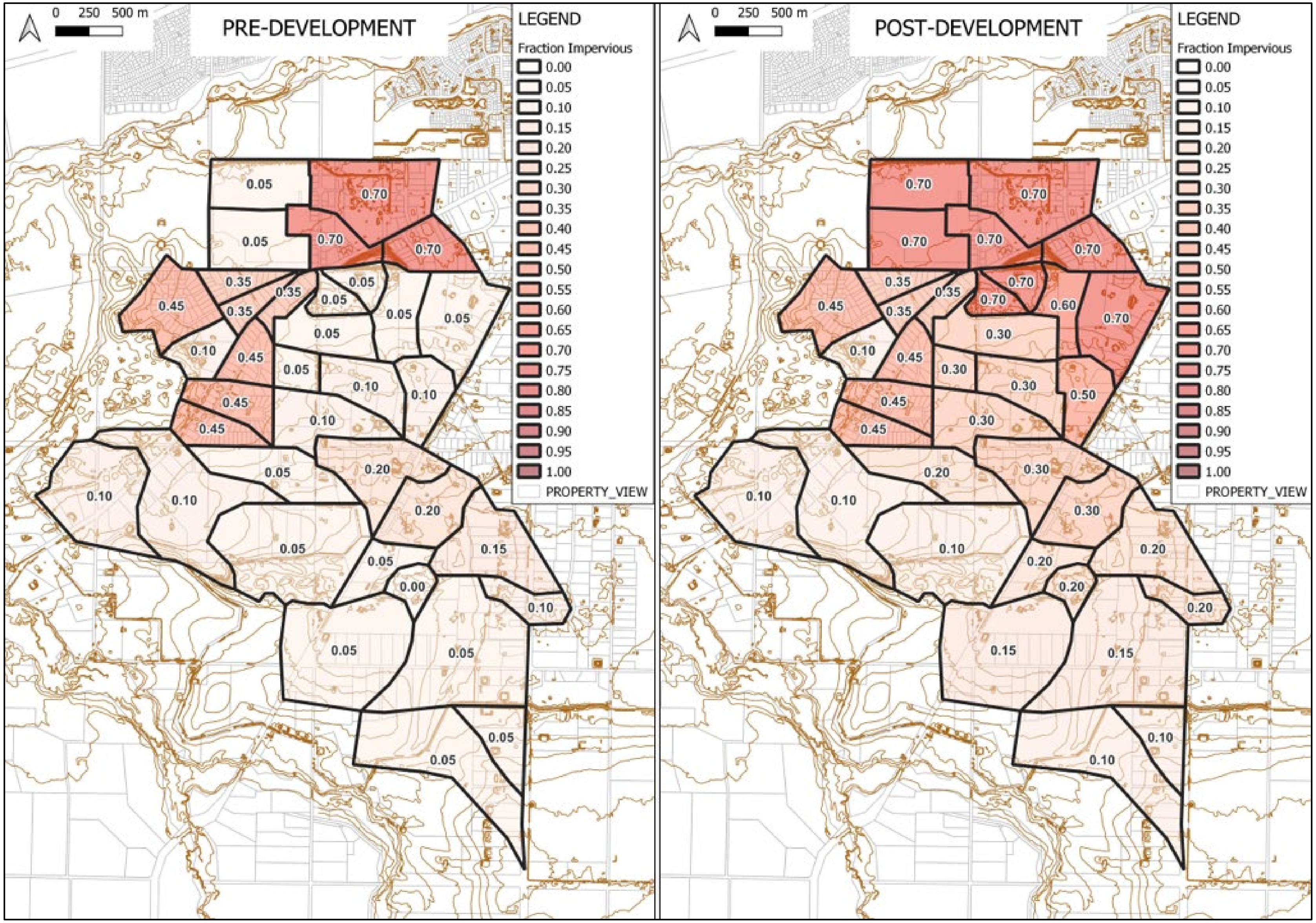


Figure 7 **Comparison of the F_{imp} assumptions made**

5 Development Requirements

It is expected that Clause 53.18-4 of the Horsham Planning Scheme will apply to much of the SP region.

Standard W1 of Horsham Planning Scheme has many objectives, but generally, the requirements can be summarised as below.

5.1 Hydrologic

Standard W1 from Horsham Planning Scheme requires that the stormwater management system be "*designed to ensure that flows downstream of the subdivision site are restricted to pre-development levels unless increased flows are approved by the relevant drainage authority and there are no detrimental downstream impacts*".

When overall strategies are proposed for SP's (as part of a ICP or DCP) the objective is generally met at the outlet from the SP, not at individual points within the SP. Thus herein, this objective has been taken to be:

> "*At the outlet from Catchment A (located at the western extent of Plumpton Road), flows existing Catchment A must not exceed the pre-development flow rates for the 50% and 1% AEP events*".

5.2 Stormwater Treatment

Standard W1 from Horsham Planning Scheme requires that the stormwater management system be "*designed to meet the current best practice performance objectives for stormwater quality as contained in the Urban Stormwater - Best Practice Environmental Management Guidelines*".

The BPEMG design targets as per Table 2. These targets have been adopted at the outlet from the SP, not at individual points within the SP.

Table 2 BPEMG Performance Objectives

Pollutant:	**Objective:**
Total Suspended Solids (**TSS**)	80% retention of the typical urban annual load;
Total Phosphorus (**TP**)	45% retention of the typical urban annual load;
Total Nitrogen (**TN**)	45% retention of the typical urban annual load;
Litter	70% reduction of the typical urban annual load; and
Flows	Maintain discharges for the 1.5-year ARI at pre-development levels

In June 2021 the Environment Protection Authority Victoria (**EPA Vic**) released updated 'urban stormwater management guidance' (EPA Vic 2021) (referred to as the **EPA Guidance** herein). The EPA Guidance is clear that it does not impose compliance obligations. Rather, the EPA Guidance provides quantitative performance objectives for urban stormwater which set an objective that should be aimed to be met as far as 'reasonably practicable'.

The SP is not a priority area under the EPA Guidance. Thus, the EPA Guidance in addition to BPEMG sets the following additional performance objectives to be met as far as reasonably practicable (rainfall band 400mm):

> "*From the impervious runoff from the catchment:*
>
> - *Harvest/evapotranspire 33% of the mean annual runoff (**MAR**); and*
> - *Infiltrate/filter 0% of the MAR.*"

It is likely that these targets will be able to be achieved if lot scale rainwater harvesting and re-use are assumed throughout the SP region (or Golf Club re-use is allowed for, see Section 6.8). However, as is common in the formulation of region drainage strategies, conservatively re-use has not been assumed in the sizing of the regional elements herein. Any future stormwater reuse will enhance the stormwater benefits detailed in this report.

Despite the above, as the SP is developed, rainwater harvesting, and re-use (especially lot scale uses) should be adopted and encouraged. This approach is deemed reasonably practicable in this instance.

5.3 Hydraulic

5.3.1 Minor System

Standard W1 from Horsham Planning Scheme requires for all events up to the 20% AEP standard, "*Stormwater flows should be contained within the drainage system to the requirements of the relevant authority*".

The IDM (Table 9) specifies that the minor system should be designed for the 20% AEP event in urban areas and the 10% AEP in industrial areas. The IDM's minimum level of service adopted herein for the pipe systems.

It is assumed that subdivisional design within the SP into the future will be able to show that this standard can be achieved. That is, this standard is generally not shown to be met at this concept design stage of the project, although some indicative trunk system sizes are specified.

5.3.2 Major System

Standard W1 from Horsham Planning Scheme requires for all events greater than the 20% AEP, and up to and including the 1% AEP standard, "*Provision must be made for the safe and effective passage of stormwater flows*".

Roads:

The DELWP Flood Guidelines set the limits of 'safe' passage of stormwater flows down roads. These are limits reproduced in Table 3.

Table 3 DELWP Flood Guidelines Safety Criteria Safety Limits

Hydraulic Characteristic	Limit
Maximum Depth (D_{max})	≤ 0.30 m
Maximum Velocity (V_{max})	≤ 2.0 m/s
The product of the Maximum Depth and Velocity ($V_{max} \times D_{max}$)	≤ 0.30 m^2/s

It is assumed that subdivisional design within the SP into the future will be able to show that the criteria within Table 3 can be achieved (i.e. it is not shown to be met at this concept design stage of the project).

Regional Conveyance

In regional conveyance systems (waterways, wetlands) etc, safe passage of flows is achieved by setting minimum lot levels above the 1% AEP flood level estimate. The DELWP Flood Guidelines specify that the freeboard is typically between 300 mm to 600 mm above the 1% AEP flood level estimate.

Herein, 600 mm of freeboard from the 1% AEP flood level estimate to lot levels is assumed from any regional conveyance systems.

1% AEP flood level estimates are provided for the retarding basins proposed in this strategy. These flood levels may vary if designs change significantly as the project proceeds. 1% AEP flood levels along the “trunk” drainage system should be confirmed at the functional design stage of the project.

6 Other Constraints and Assumptions

6.1 Regional Flooding

Figure 8 shows the current flood overlays in the vicinity of the SP. There are no regional flooding overlays that impact on Catchment A.

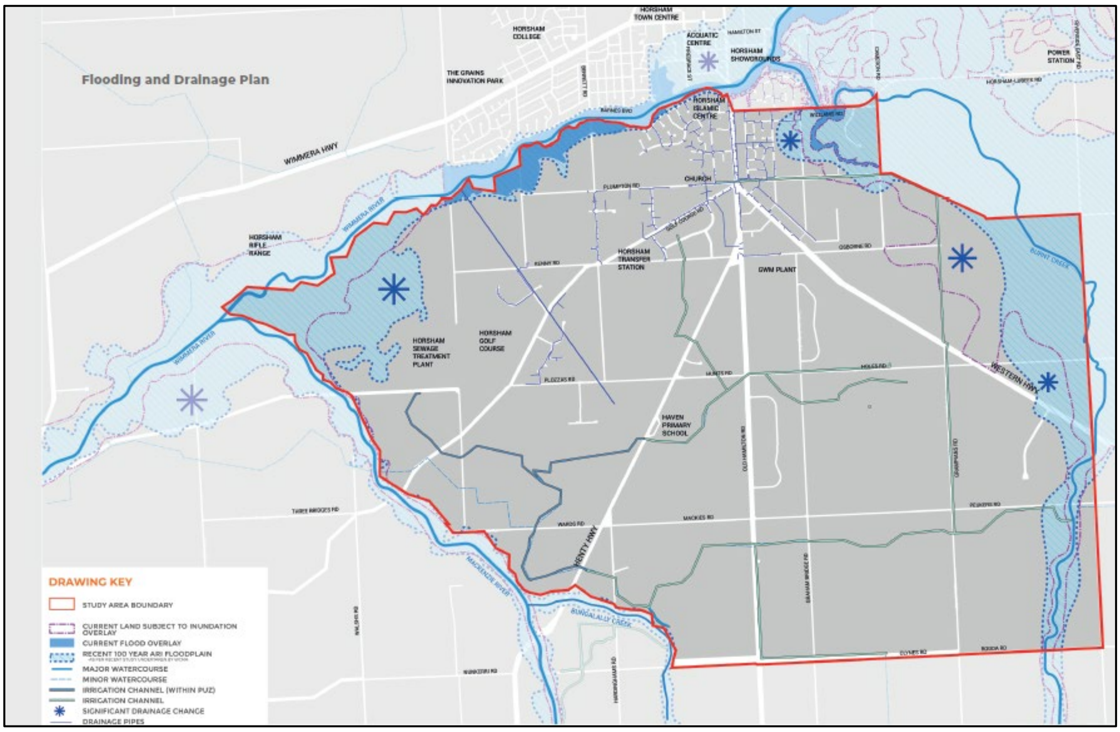


Figure 8 Current Flood overlays. Source: Mesh Background Report

6.2 Local Flooding

As previously described in the PDA and the Regional Food Mapping, much of the SP area is subject to shallow flooding in the 1% AEP event due to poor surface gradients and ill-defined drainage paths.

The proposals in this report aim to provide a mechanism to aid in allowing future drainage systems (constructed as part of future development) to “drain” effectively, and thus decrease local flood effects in the SP area.

6.3 Sewer and Water

West of the golf course is the Horsham Sewage treatment plant.

Currently there are a trunk sewer pipe and a water supply pipe that follow the Kenny Road alignment. The level of these pipes are not known at the current time. Whether or not these pipes bisects any drainage proposals should be confirmed into the future.

6.4 Aboriginal Cultural Heritage and Post Contact Heritage

It is assumed that no heritage constraints will impact any of the proposals herein. This is to be confirmed into the future.

6.5 Flora and Fauna

It is assumed that no flora or fauna constraints will impact any of the proposals herein. This is to be confirmed into the future.

6.6 Groundwater

It is assumed herein that there will be minimal interaction with any of the proposed excavation with the groundwater table (the depth of which is currently unknown). If there is interaction, the base of the wetland/retarding basin systems should be clay lined to reduce the interaction.

6.7 Downstream Outfall

The most crucial design level within this strategy is the invert level of the existing culverts at the intersect of Plumpton Road and the unnamed road, in the northwest of the catchment as shown in Figure 9.

Figure 9 Existing outfall culverts at approx. 36.734S, 142.178E. Source: Site Visit

At this stage, the culverts (based on Site Visit estimates and the LiDAR information) are assumed to be 2No. x 1200 mm (wide) x 750 mm (high) box culverts at an invert of 124.00 m AHD.

These sizes and invert levels should be confirmed with survey into the future.

6.8 Existing Council Drainage System

Council have advised that the existing LDRZ development around Mackenzie Court is serviced by an old irrigation pipe (as shown in Figure 10) that runs from the southeast to the northwest.

At the Site Visit this pipe was full of standing water. Council staff advised that it does not provide the required level of service to this existing development.

It is assumed that the standing water within this pipe may be due to the water level within the existing pondage at location 'A' (see Figure 10). In addition, as this was an old irrigation pipeline, invert level irregularities may also be causing standing water issues in the pipe.

The design herein will aim to lower the water level at 'A' and hence (hopefully) provide more capacity to this existing system and the existing Colonial Drive development.

Figure 10 Council Pipe System. Source: Mesh Background Report

6.9 Golf Club Reuse

At the site visit it was observed that at location ‘A’ (Figure 10) there was a pump station labelled the ‘*Horsham Golf Club Storm and Reclaimed Water Treatment Wetland Project*’.

SWS have not been provided any details of this system.

However, retrofitting (or modifying) the proposals herein to account for golf course use (if required) should be easily accommodated into the future, and will only supplement the benefits of the drainage proposals detailed in this report.

6.10 55 Kenny Road

The stormwater management of 55 Kenny Road, and Catchment G1 from the PDA, is of relevance to this study as proposals within this site and catchment directly impact the sizing of the future waterway on the south of Plumpton Road in the IN1Z land.

Council have advised that a permit, PA2200512, has been granted for 55 Kenny Road on the 15/05/2023 (the **Kenny Road Permit**).

The PDA identified 55 Kenny Road as the site which is to have the majority (if not all) of the future asset W_RG_G1, likely fronting Kenny Road.

Given the Kenny Road Permit endorses the plan of subdivision shown in Figure 11, it is unlikely that W_RG_G1 will be able to be delivered as the PDA assumed. It is noted that the Kenny Road Permit conditions 16 and 17 do potentially allow for some form of regional asset to be delivered.

However, the current Kenny Road SWMS does <u>not</u> propose a regional asset. The Kenny Road SWMS only proposes an asset that accommodates for the development of 55 Kenny Road. It is noted that the Kenny Road SWMS has <u>not</u> yet been endorsed under the condition 16 of the Kenny Road Permit so there still may be scope to provide a regional asset at this location.

Given the above, at this stage, this strategy assumes that lots within the industrial land south of Kenny Road (but not in Catchment A) will provide their own lot scale drainage solutions if/when they develop.

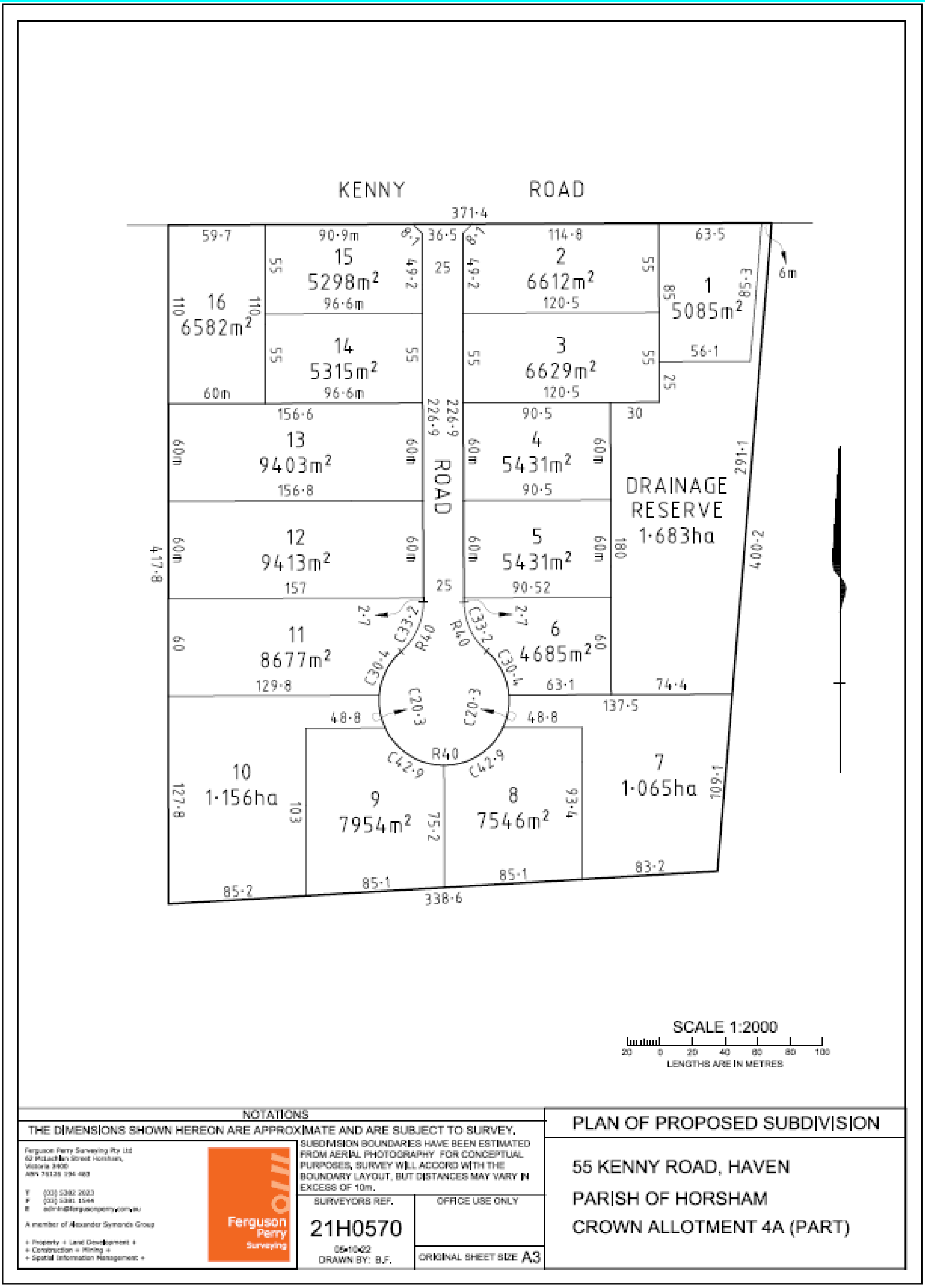


Figure 11 Endorsed Plans under condition 1 of the Kenny Road Permit.

7 Catchment A Drainage Concept

Drawings 2350/CONC/1 to 5 (see Appendix A) detail the concept designs for the major Catchment A drainage assets. Details of the overall drainage system configuration and specific assets are discussed in more detail below.

7.1 Overall Drainage Configuration

Drawing 2350/CONC/1 details the overview of the primary asset configuration in Catchment A.

Specifically, a series of three wetland/retarding basin systems (connected by pipelines) are proposed to traverse the centre of the catchment from south to north. This is referred to as the main 'trunk' drainage system.

Once this system is constructed, surrounding sites should then be able to discharge (via pit and pipe and/or grassed swale connections) into this system.

Pipe outfalls to the 'trunk' drainage system can be achieved by:

- Connecting straight into the trunk drainage system (for land parcels located directly adjacent to the 'trunk' drainage system (although these parcels may also be required to construct pipelines along their boundaries (as they develop) to ensure upstream land parcels are also afforded an outfall once they develop in the future); and
- Connecting into, or constructing a "connection pipeline" (via agreement with downstream landowners or the road authority) if a parcel is not located directly adjacent to the 'trunk' drainage system.

For clarity the key drainage assets are:

- The existing natural wetland/retarding basin W_RB_A1, which will retain its natural formation and existing vegetation (although this may be supplemented in the future), with the only "construction" being provision of a small outlet pipe system connecting this system to the proposed downstream 'trunk' pipe system;
- Constructed wetland/retarding basin W_RB_A2;
- Constructed wetland/retarding basin W_RB_A3;
- A 1% AEP pipeline connection between W_RB_A2 and W_RB_A3; and
- A 20% AEP pipeline connection between W_RB_A1 and W_RB_A2.

Note that Appendix B.2, Appendix C and Appendix D describe the following in more detail:

- The three wetland/retarding basin assets;
- The preliminary design of the pipelines joining the wetland/retarding basin assets;
- Gross pollutant trap (**GPT**) application;
- Grassed swale application;
- The application of tanks for stormwater reuse; and
- The preliminary design of the waterway proposed as the outfall from Catchment G.

Key implications of the strategy are discussed below.

7.2 Longitudinal Grade

Drawing 2350/CONC/2 shows a conceptual longitudinal section of the main 'trunk' drainage proposed within Catchment A. Generally, once implemented, the 'trunk' drainage will provide a drainage outfall throughout Catchment A that is at least 2 metres deep below the surrounding natural surface levels (**NSL**'s).

For a flat region such as this catchment, having a pipe outfall in the order of 2 metres deep below the NSL will allow for simple subdivisional drainage designs, that simply connect into the 'trunk' drainage system (as described above).

At this stage, no services checks have been undertaken along the 'trunk' drainage systems alignment. However, if there are found to be clashes into the future, the same concept likely could be used along different alignments (located generally north to south along the centre region of Catchment A).

The trunk drainage pipes generally have an assumed grade of 1V:500H (with pits assumed every 80 m, and a 0.02 m fall in every pit).

As shown Drawing 2350/CONC/2, the online wetlands W_RB_A2 and W_RB_A3 are crucial to the proposed concept as they allow the 'trunk' drainage system's invert to be flat over approximately 1,700 metres. If the wetlands were offline (and hence required to be bypassed with a pipe), the system invert would be required to be graded (assume 1V:500H), and the entire benefit of having 'deep' trunk drainage outfalls throughout Catchment A would be lost.

7.3 Fill

Significant fill in flat rural areas can quickly cause development to become not economically viable. Fill will not only be required directly adjacent to the drainage reserves. If not managed correctly, fill requirements propagates up through the whole development due to the required grade that is needed on the subdivisional drainage pipe systems (and the associated "pipe cover" requirements).

The key secondary benefit of the 'trunk' drainage system as proposed within 2350/CONC/2 is that most of the freeboard and cover requirements are accommodated in cut below the NSL. That is, there should be minimal need to fill across Catchment A to enable development if the above strategy is implemented.

7.4 Implementation and Development Timing

Construction of the 'trunk' drainage system can be staged, but ideally should be constructed from downstream to upstream (i.e. south to north).

The preferred sequence of implementation for this concept is:

- IN1Z land develops and provides W_RB_A3;
- Council (or a developer) provide the pipe connection between W_RB_A3 and W_RB_A2 (note, as shown in 2350/CONC/3 and discussed within Section 7.4, this connection probably will require land acquisition through a private property);

- Development of 41 Watsons Lane provides W_RB_A2;
- Council (or developer(s)) provide the pipes upstream of W_RB_A2 within Plozzas Road and the Henty Hwy to service Haven; and then
- Council constructs the pipeline from Henty Highway to the new outlet pipe from W_RB_A1.

There is little scope within this proposal for development to occur out of the above sequence (unless significant fill is utilised) given the flat nature of the catchment.

It is also noted, at this stage, other than the connection discussed within Section 7.5, all 'trunk' pipes are proposed either within existing Council reserves, or within road reservations.

7.5 Land Acquisition

For the development timing within Section 7.4, there are clear development triggers to deliver almost all assets proposed.

However, as shown in 2350/CONC/3, a drainage connection is required between the outlet of W_RB_A2 and Golf Course Road. Delivering this connection will be dependent on land acquisition through private properties, that have no clear development triggers.

This connection is expected to be 3 x 900 mmØ pipes and an extreme flow provision.

The connection could either be with an easement or a reserve. Council will have to decide as to which option is beneficial in this instance.

If an easement is chosen, as per the IDM, the minimum easement width should be 5 metres, however a wider easement would be beneficial.

If a reserve is chosen, this will provide Council more control, and also allow for the Haven to the Wimmera River walking and cycling connection proposed within Figure 1.

Within 2350/CONC/3, the connection is shown through 231 Golf Course Road, Haven, 3401. This is the ideal property. However, as shown in 2350/CONC/3, the drainage concept could relatively easily be 'tweaked' if the connection had to be through any of:

- 219 Golf Course Road, Haven, 3401;
- 229 Golf Course Road, Haven, 3401; or
- 237 Golf Course Road, Haven, 3401.

7.6 Cost Sharing

The concept herein shows how the overall ultimate Catchment A solution could work.

Clearly, there are two parcels, 41 Watsons Lane (W_RB_A2) and the IN1Z land on Plumpton Road (W_RB_A3) which bear more proposed drainage assets than the other parcels.

The assets, W_RB_A2 and W_RB_A3, provide benefits to the entire catchment. They both provide the drainage outfall provisions for all parcels and they combine to meet Clause 56.04 requirements (see Section 7.6.1).

Given this, the cost of W_RB_A2 and W_RB_A3, all 'trunk' drainage pipes, and the connection discussed within Section 7.5, should be paid for by development throughout the entire catchment.

Herein, cost-estimates have not been developed. However, as the design is progressed, cost estimates should be generated.

Having cost estimates will enable the development of an infrastructure Contributions Plan (**ICP**) or a Development Contributions Plan (**DCP**) for the SP region into the future. The ICP or DCP provide a formal mechanism for the cost-sharing of the drainage assets.

7.7 Development Requirements

7.7.1 Ultimate

Appendix B details the hydrologic modelling and design which shows that by providing the systems proposed within 2350/CONC/1 to 5, that the pre-development flow targets specified in Section 5.1 are not exceeded as shown in Table 4 at the Catchment A outfall.

Table 4 Flow estimate comparison at the Catchment A outfall

AEP	Pre		Post	
	Q (m^3/s)	Duration	Q (m^3/s)	Duration
50%	1.30	3-hour	1.25	4.5-hour
1%	19.65	6-hour	19.60	6-hour

Notes: All flow estimates rounded to the nearest 0.05 m^3/s and are reported as the peak average for the critical duration.

Appendix C details the stormwater treatment modelling and design which shows that by providing the systems proposed within 2350/CONC/1 to 5, that the BPEMG treatment targets specified in Section 5.2 are achieved as shown in Table 5 by the Catchment A outfall.

Table 5 Overall stormwater treatment performance within Catchment A of the SP

Pollutant	Total catchment inflow load (kg/yr)	Total catchment outflow load (kg/yr)	Load retained (kg/yr)	% retention of the SP area	BPEMG Target	Target Met
Total Suspended Solids	152,000	27,900	124,100	81.6%	80.0%	Yes
Total Phosphorus	335	111	224	66.9%	45.0%	Yes
Total Nitrogen	2,530	1,390	1,140	45.1%	45.0%	Yes
Gross Pollutants (Litter)	36,400	0	36,400	100.0%	70.0%	Yes

Appendix B and D detail the estimations of 1% AEP flood levels associated with each of the assets. Table 6 summarises these. As shown within drawing 2350/CONC/2, the flood levels are mostly below the NSL's and can be used to inform subdivisional design into the future.

Table 6 Estimates of the 1% AEP flood levels at each of the assets

Asset	1% AEP Flood Level Estimate (m AHD)
W_RB_A1	133.15
W_RB_A2	126.95
W_RB_A3	126.15

Note: Flood level estimates are rounded up to the nearest 50 mm.
Flood levels may vary if design development significantly changes the designed detailed in this report

In summary, if the total system is delivered generally in accordance with 2350/CONC/1 to 5, the key statutory requirements for the development of Catchment A can be met at a regional scale.

7.7.2 Interim

Section 7.4 details the preferred timing of development within Catchment A. Development timing may not follow this sequence.

If development occurs out of sequence, the development will be required to show how:

- In isolation it can achieve all of the requirements within Section 5; while not
- Compromising the proposals within 2350/CONC/1 to 5.

This will likely require either (or both):

- Downstream cleanout; and/or
- Interim (temporary) assets.

However, it is noted that if development occurs after a DCP or ICP is implemented, the stormwater treatment requirements (Section 5.2) may be met via payment of the DCP or ICP rate, in-lieu of interim tertiary stormwater treatment assets.

7.8 IN1Z Waterway

Drawing 2350/CONC/4 and Appendix D show a 40 metre wide waterway reserve abutting the south of Plumpton Road. The size of this waterway is directly impacted by the future stormwater management within catchment G1 of the PDA, including 55 Kenny Road.

At this stage, the sizing of the waterway is likely conservative (as it assumed no retardation of new industrial development in Catchment G). The waterway size should be reviewed once more certainty is known regarding the 55 Kenny Road proposals and W_RB_G1.

7.9 Landscape and Ecology

The proposals and reporting herein focus heavily on the drainage requirements and functions of the proposed assets.

However, once constructed the assets have the potential to be high-value landscape and ecological assets to Council.

It is envisaged that into the future landscape designs and ecological management plans be developed for the assets to ensure that they provide additional benefits for Council. i.e. W_RB_A2 and W_RB_A3 should:

- Be incorporated into the Wimmera River walking and cycling connection proposed within Figure 1;
- Have local loop paths (running/walking) around the assets, including boardwalk crossovers; and
- Potentially incorporate playgrounds and barbeque areas within their reserves.

8 Further Work Required

To further develop the concepts, the following further work is required (at a minimum):

1. Develop a plan regarding how to deliver the drainage connection through 231 Golf Course Road, Haven, 3401 (or other surrounding properties as detailed in Section 7.4);

2. Complete feature survey (including service proving) of:
   a. All drainage reserves proposed;
   b. Along all pipelines proposed (especially of the existing services within Kenny Road); and
   c. Of (and downstream of) the existing outfall system at the western end of Plumpton Road;

3. Development of background ‘existing conditions’ reports relating to the designs proposed including (at least):
   a. Flora and fauna;
   b. Groundwater;
   c. Cultural heritage (aboriginal and post-contact); and
   d. Stormwater harvesting (if proposed);

4. Functional designs of the key trunk assets proposed herein (W_RB_A2, W_RB_A3, all pipes and the IN1Z waterway);

5. Cost estimates of the functional designs; and

6. Preparation of a formal cost-sharing mechanism (i.e. a ICP or DCP).

Separately, it is also advised that Council investigate a catchment scale solution to meet the relevant requirements for catchment G1 within Figure 4 (i.e. W_RB_G1). If implemented, W_RB_G1 has the potential to reduce the size of the IN1Z waterway shown in 2350/CONC/4.

9 Concluding Remarks

The designs herein detail a concept as to how the drainage within Catchment A of the SP could be managed.

As shown in drawing 2350/CONC/1, this is proposed via the use of 'trunk' drainage assets which allow catchment A to:

- Be serviced by conventional pit and pipe drainage without the need for excess fill; and
- Meet its relevant statutory requirements.

As shown within Section 8, there is still a large amount of further work to be undertaken.

However, it is likely that as the final designs are developed further that they will be generally in accordance with the proposals herein.

10 References

Allison, R. A., T. Walker, F. H. S. Chiew, I. C. O'Neill and T. A. McMahon (1998). *From roads to rivers: gross pollutant removal from urban waterways (Technical Report No. 98/6)*. Melbourne, Cooperative Research Centre for Catchment Hydrology.

Nearmap (2023). *Areal imagery for locations and dates shown on Figures where applicable*.

Willing and Partners Pty Ltd (1992), *Design Guidelines for Gross Pollutant Traps*, prepared for ACT Planning Authority, Department of Environment, Land and Planning, Project No. 3015.

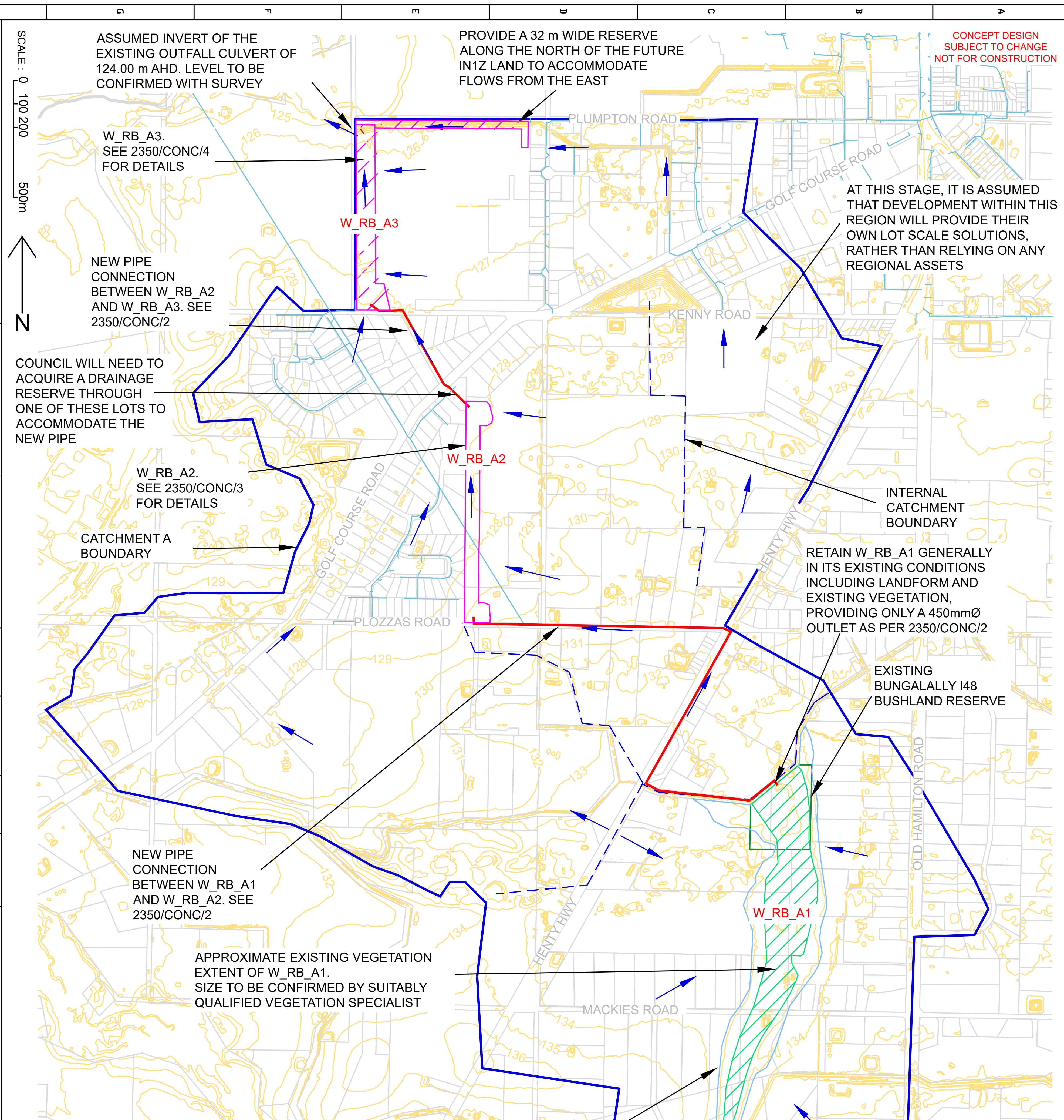
Note: Also see Sections 2 and 3 of this report.

11 Abbreviations, Descriptions and Definitions

The following table lists some common abbreviations and drainage system descriptions and their definitions which may be referred to in this report.

Abbreviation / Descriptions	**Definition**
AHD - Australian Height Datum	Common base for all survey levels in Australia. Height in metres above mean sea level.
ARI - Average Recurrence Interval.	The average length of time in years between two floods of a given size or larger. A 100 Year ARI event has a 1 in 100 chances of occurring in any one year.
AEP – Annual Exceedance Probability	The chance of a storm (flow) of that magnitude (or larger) occurring in a given year. $\boldsymbol{AEP} = \mathbf{1} - \boldsymbol{e}^{\left(\frac{-1}{ARI}\right)}$. i.e. 1% AEP = 100 Year ARI
BPEMG	Best Practice Environmental Management Guidelines. See CSIRO (1999)
DSS or DS	Development Services Scheme (DSS) or Drainage Scheme (DS) is a master plan developed my MWC for drainage within a catchment area.
EY – Exceedances per year	The amount of times a storm (flow) of that magnitude is expected to be exceeded per year. i.e. 4 EY = 3 Month ARI
HECRAS	A hydraulic software package that enables the calculations of flood levels and velocities along a waterway given a specified flow.
m^3/s -cubic metre/second	Unit of discharge usually referring to a design flood flow along a stormwater conveyance system
MUSIC	Hydrologic computer program used to calculate stormwater pollutant generation in a catchment and the amount of treatment which can be attributed to the WSUD elements placed in that catchment
MWC / MW	Melbourne Water Corporation
Retarding basin	A flood storage dam which is normally empty. May contain a lake or wetland in its base
NWL - Normal Water Level	Water level of a wetland or pond defined by the lowest invert level of the outlet structure
NSL – Natural Surface Level	The surface level of the natural (existing) surface before works.
RORB	Hydrologic computer program used to calculate the design flood flow (in m^3/s) along a stormwater conveyance system (e.g. waterway)
Sedimentation basin (Sediment pond)	A pond that is used to remove coarse sediments from inflowing water mainly by settlement processes.
Swale	A small shallow drainage line designed to convey stormwater discharge. A complementary function to the flood conveyance task is its WSUD role (where the vegetation in the base acts as a treatment swale).
TED	The top level of water stored for treatment within a wetland before bypass occurs
TSS	Total Suspended Solids – a term for a particular stormwater pollutant parameter
TP	Total Phosphorus – a term for a particular stormwater pollutant parameter
TN	Total Nitrogen – a term for a particular stormwater pollutant parameter
WSUD - Water Sensitive Urban Design	Term used to describe the design of drainage systems used to: o Convey stormwater safely o Retain stormwater pollutants o Enhance local ecology o Enhance the local landscape and social amenity of built areas
Wetland	WSUD element which is used to collect TSS, TP and TN. Usually incorporated at normal water level (NWL) below which the system is designed as shallow marsh, marsh, deep marsh and open water areas.

Appendix A Concept Drawings



1% AEP FLOOD LEVEL ESTIMATES:

W_RB_A1	= 133.15 m AHD
W_RB_A2	= 126.95 m AHD
W_RB_A3	= 126.15 m AHD

APPROXIMATE 1% AEP FLOOD LEVEL ESTIMATE OF 133.15 m AHD AT W_RB_A1.
NOTE: THE 1% AEP FLOOD EXTENT WITHIN W_RB_A1 JUST ENSURES NO INCREASE IN FLOOD EFFECT DUE TO THE DEVELOPMENT. NO ADDITIONAL COUNCIL LAND ACQUISITION IS REQUIRED

SHEET CONTROL:

2350/CONC/1	OVERVIEW
2350/CONC/2	TRUNK LONGITUDINAL SECTION
2350/CONC/3	W_RB_A2 CONCEPT PLAN
2350/CONC/4	W_RB_A3 CONCEPT PLAN
2350/CONC/5	GENERAL DETAILS

NOTES:

1. THIS DRAWING SET SHOULD BE READ IN CONJUNCTION WITH THE ASSOCIATED STORMY WATER SOLUTIONS 'CATCHMENT A DRAINAGE CONCEPT' REPORT, AUGUST 2023.
2. INLET PIPE LOCATIONS SUBJECT TO CHANGE AS DESIGNED BY OTHERS.
3. THE LOCATION AND LEVELS OF ALL EXISTING SERVICES TO BE CONFIRMED.
4. DETAILS OF LOT AND SUB DIVISIONAL SCALE DRAINAGE SOLUTIONS ARE NOT SHOWN WITHIN THIS DRAWING SET. THESE SOLUTIONS COULD INCLUDE:
   - TANKS FOR STORMWATER REUSE;
   - SWALE OUTFALLS TO THE MAIN TRUNK DRAINAGE SYSTEM;
   - PIPE OUTFALLS TO THE MAIN TRUNK DRAINAGE SYSTEM; AND/OR
   - GROSS POLLUTANT TRAPS.
5. ALL BATTERS ABOVE THE NORMAL WATER LEVEL'S (NWL) ARE AT 1V:5H (MAX).
6. THE DESIGN CONTOURS DETAILED ARE INDICATIVE ONLY AND SHOULD BE CONFIRMED UTILISING A 12D MODEL (OR SIMILAR).
7. THE GENERAL OUTLET ARRANGEMENTS FOR W_RB_A2 AND W_RB_A3 ARE BASED ON MWC SD 7251/12/4003.
8. ADDITIONAL INSPECTION PITS MAY BE REQUIRED ALONG THE ALIGNMENTS OF ANY PIPES PROPOSED ON THIS DRAWING. INSPECTION PIT LOCATIONS ARE TO BE DETERMINED BY OTHERS AT THE DETAILED DESIGN STAGE.
9. THE SIZE AND SPECIFICATION OF ALL HEADWALLS TO BE DETERMINED BY OTHERS AT THE DETAILED DESIGN STAGE.
10. ALL WATER BODY EDGE TREATMENTS AS PER MWC SD 7251/12/010.
11. SUITABLE GROSS POLLUTANT TRAPS ARE TO BE PROVIDED ON ALL PIPE OUTFALLS DIRECTLY INTO A SEDIMENT BASIN OR WETLAND SYSTEM.
12. AN ANCOLD ASSESSMENT MAY BE REQUIRED ON THE PROPOSED EMBANKMENT(S).

LEGEND:

- CATCHMENT A BOUNDARY
- INTERNAL CATCHMENT BOUNDARY
- INDICATIVE FLOW DIRECTIONS
- PROPOSED DRAINAGE RESERVES
- PROPOSED 'TRUNK' DRAINAGE PIPES
- W_RB_A1 APPROXIMATE EXISTING VEGETATION EXTENT
- W_RB_A1 APPROXIMATE 1% AEP FLOOD LEVEL ESTIMATE
- W_RB_A1 EXISTING BUSHLAND RESERVE
- EXISTING COUNCIL PIPES
- EXISTING PARCELS
- 1m COUNCIL CONTOURS

REVISION		DATE
V1	INITIAL ISSUE	08/08/2023
V2	RESERVE SIZES OPTIMISED	03/04/2024

Stormy Water Solutions

DESIGNED	Michael Mag
CHECKED	Valerie Mag
AUTHORISED BY	

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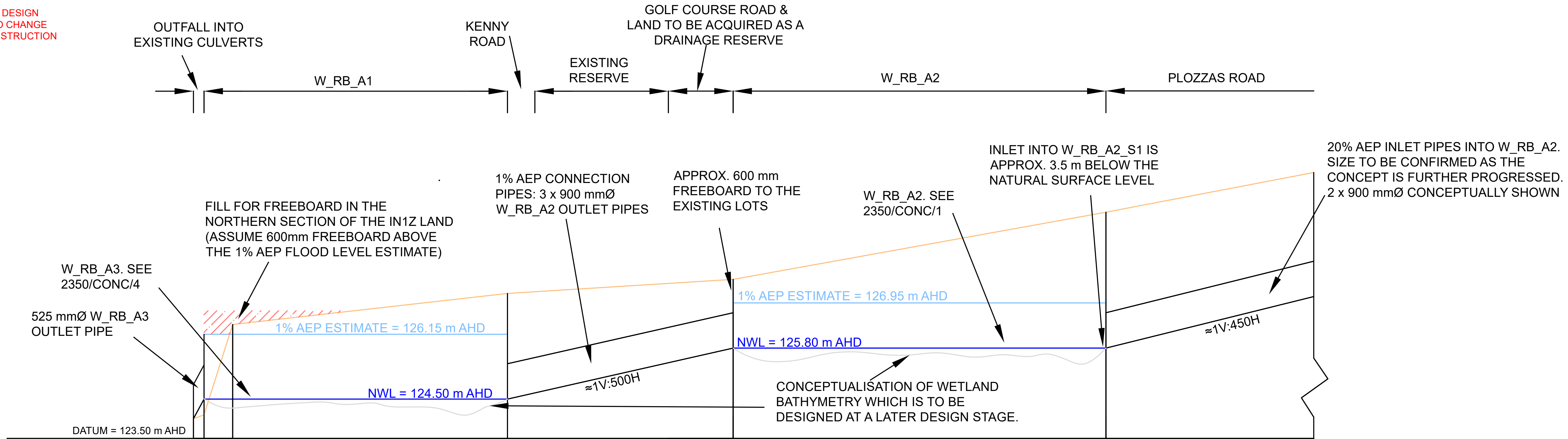
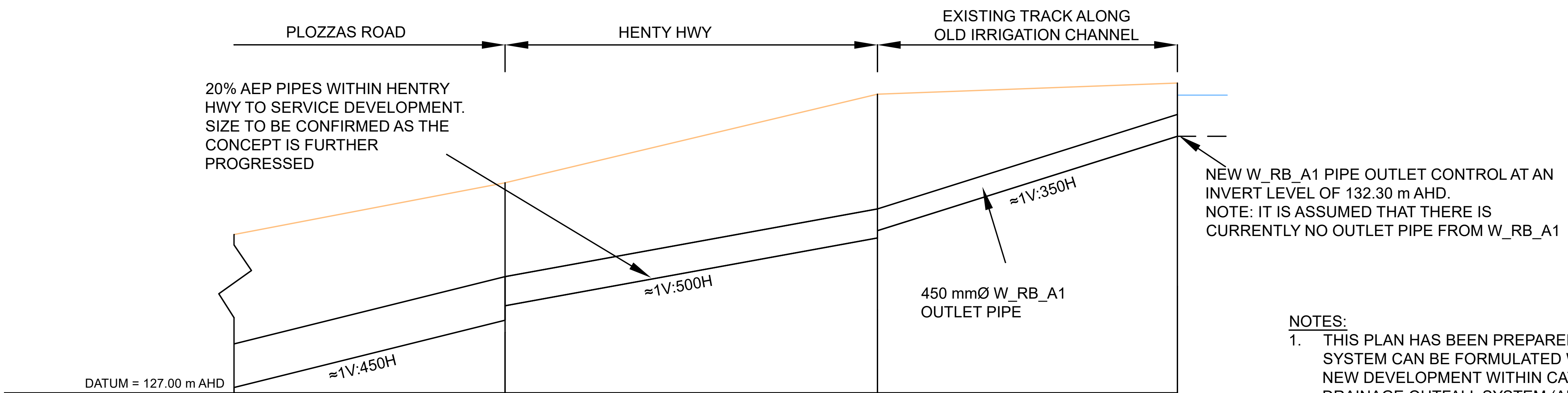
HORSHAM SOUTH
STRUCTURE PLAN
CATCHMENT A DRAINAGE CONCEPT
OVERVIEW

SCALE: AS SHOWN
SHEET 1 OF 5
DRAWING No. 2350/CONC/1

CONCEPT DESIGN
SUBJECT TO CHANGE
NOT FOR CONSTRUCTION



APPROX. CHAINAGE (m):	0	27	800	1375	2325	
APPROX. NATURAL SURFACE LEVEL (m AHD):	124.00	124.10	127.20	127.60	129.25	
APPROX. SYSTEM INVERT LEVEL (m AHD):	124.00	124.50	124.50	125.80	125.80	



APPROX. CHAINAGE (m):		3415	4185	4805
APPROX. NATURAL SURFACE LEVEL (m AHD):		131.60	133.15	133.40
APPROX. SYSTEM INVERT LEVEL (m AHD):	PIPE INVERTS WITHIN THIS REGION WILL BE SUBJECT TO DETAILED DESIGN			132.30

NOTES:
1. THIS PLAN HAS BEEN PREPARED TO SHOW HOW A 'TRUNK' DRAINAGE SYSTEM CAN BE FORMULATED WITHIN CATCHMENT A TO ENSURE THAT ALL NEW DEVELOPMENT WITHIN CATCHMENT A CAN INCORPORATE A 'DEEP' DRAINAGE OUTFALL SYSTEM (AND TO ALLOW DEVELOPMENT WITHOUT DOWNSTREAM IMPACTS OR SIGNIFICANT FILL).
2. W_RB_A1 IS TO BE RETAINED AS PER THE EXISTING CONDITIONS APART FROM THE PIPE CONNECTION UPSTREAM OF CH4805.
3. THE LOCATION AND LEVELS OF ALL EXISTING SERVICES TO BE CONFIRMED.
4. ALL PIPE SIZES ARE SUBJECT TO DETAILED DESIGN.
5. PIPE GRADES ARE EXPECTED TO BE BETWEEN 1V:400H AND 1V:500H DEPENDING ON DETAILED DESIGN.
6. THE DESIGN LEVELS HEREIN ASSUME PIT SPACING EVERY 80 m AND A 0.02 m DROP AT EVERY PIT.
7. THE NATURAL SURFACE LINE IS APPROXIMATE ONLY.

REVISION	DATE
V1 – INITIAL ISSUE	08/08/2023
V2 – RESERVE SIZES OPTIMISED	03/04/2024

Stormy Water Solutions

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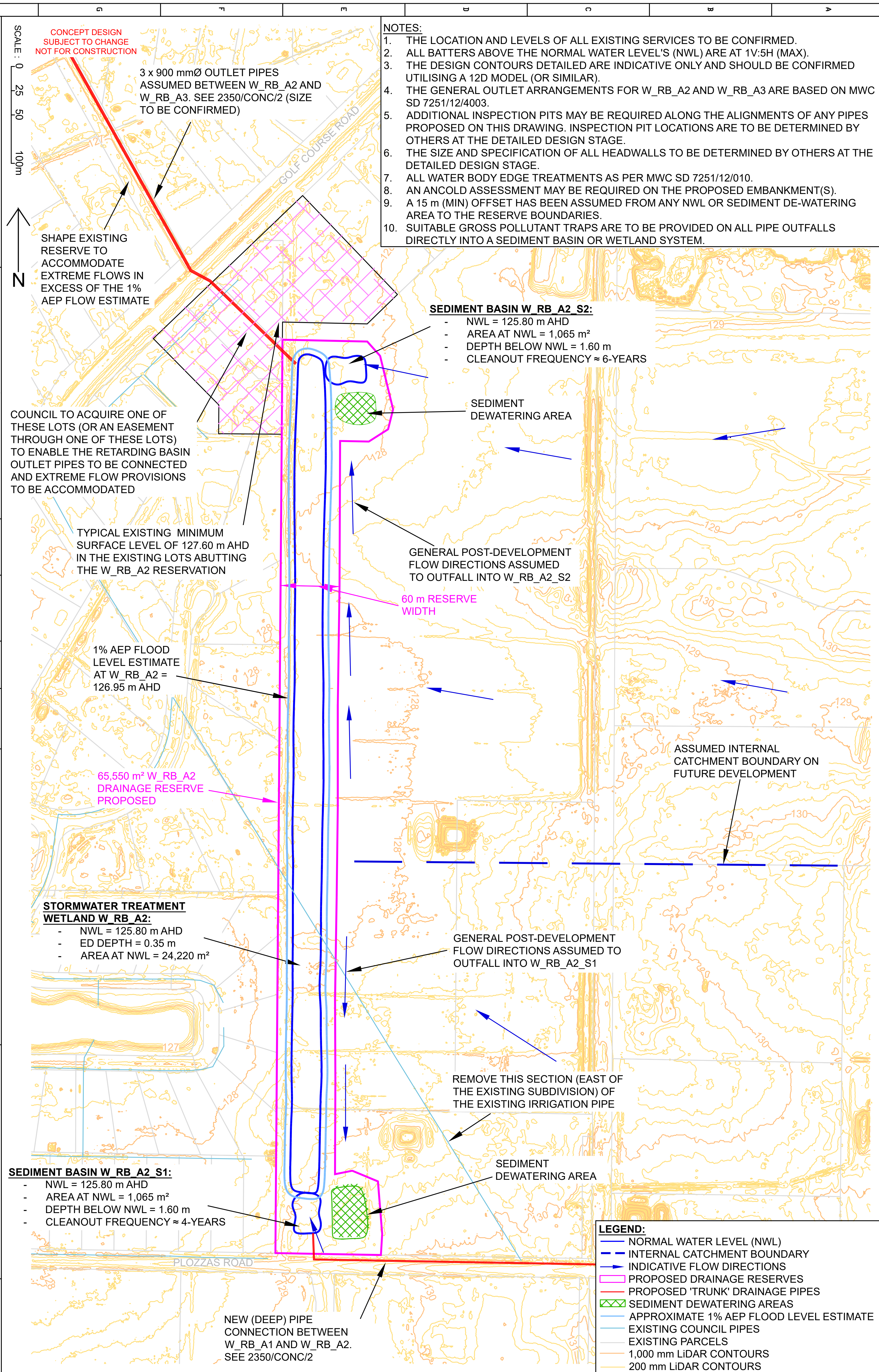
HORSHAM SOUTH
STRUCTURE PLAN
CATCHMENT A DRAINAGE CONCEPT
TRUNK LONGITUDINAL SECTION

SCALE: AS SHOWN
SHEET 2 OF 5
DRAWING No. 2350/CONC/2



NOTES:

1. THE LOCATION AND LEVELS OF ALL EXISTING SERVICES TO BE CONFIRMED.
2. ALL BATTERS ABOVE THE NORMAL WATER LEVEL'S (NWL) ARE AT 1V:5H (MAX).
3. THE DESIGN CONTOURS DETAILED ARE INDICATIVE ONLY AND SHOULD BE CONFIRMED UTILISING A 12D MODEL (OR SIMILAR).
4. THE GENERAL OUTLET ARRANGEMENTS FOR W_RB_A2 AND W_RB_A3 ARE BASED ON MWC SD 7251/12/4003.
5. ADDITIONAL INSPECTION PITS MAY BE REQUIRED ALONG THE ALIGNMENTS OF ANY PIPES PROPOSED ON THIS DRAWING. INSPECTION PIT LOCATIONS ARE TO BE DETERMINED BY OTHERS AT THE DETAILED DESIGN STAGE.
6. THE SIZE AND SPECIFICATION OF ALL HEADWALLS TO BE DETERMINED BY OTHERS AT THE DETAILED DESIGN STAGE.
7. ALL WATER BODY EDGE TREATMENTS AS PER MWC SD 7251/12/010.
8. AN ANCOLD ASSESSMENT MAY BE REQUIRED ON THE PROPOSED EMBANKMENT(S).
9. A 15 m (MIN) OFFSET HAS BEEN ASSUMED FROM ANY NWL OR SEDIMENT DE-WATERING AREA TO THE RESERVE BOUNDARIES.
10. SUITABLE GROSS POLLUTANT TRAPS ARE TO BE PROVIDED ON ALL PIPE OUTFALLS DIRECTLY INTO A SEDIMENT BASIN OR WETLAND SYSTEM.

LEGEND:

- NORMAL WATER LEVEL (NWL)
- INTERNAL CATCHMENT BOUNDARY
- INDICATIVE FLOW DIRECTIONS
- PROPOSED DRAINAGE RESERVES
- PROPOSED 'TRUNK' DRAINAGE PIPES
- SEDIMENT DEWATERING AREAS
- APPROXIMATE 1% AEP FLOOD LEVEL ESTIMATE
- EXISTING COUNCIL PIPES
- EXISTING PARCELS
- 1,000 mm LiDAR CONTOURS
- 200 mm LiDAR CONTOURS

REVISION	DATE
V1 - INITIAL ISSUE	08/08/2023
V2 - RESERVE SIZES OPTIMISED	03/04/2024

DESIGNED	Michael Mag
CHECKED	Valerie Mag
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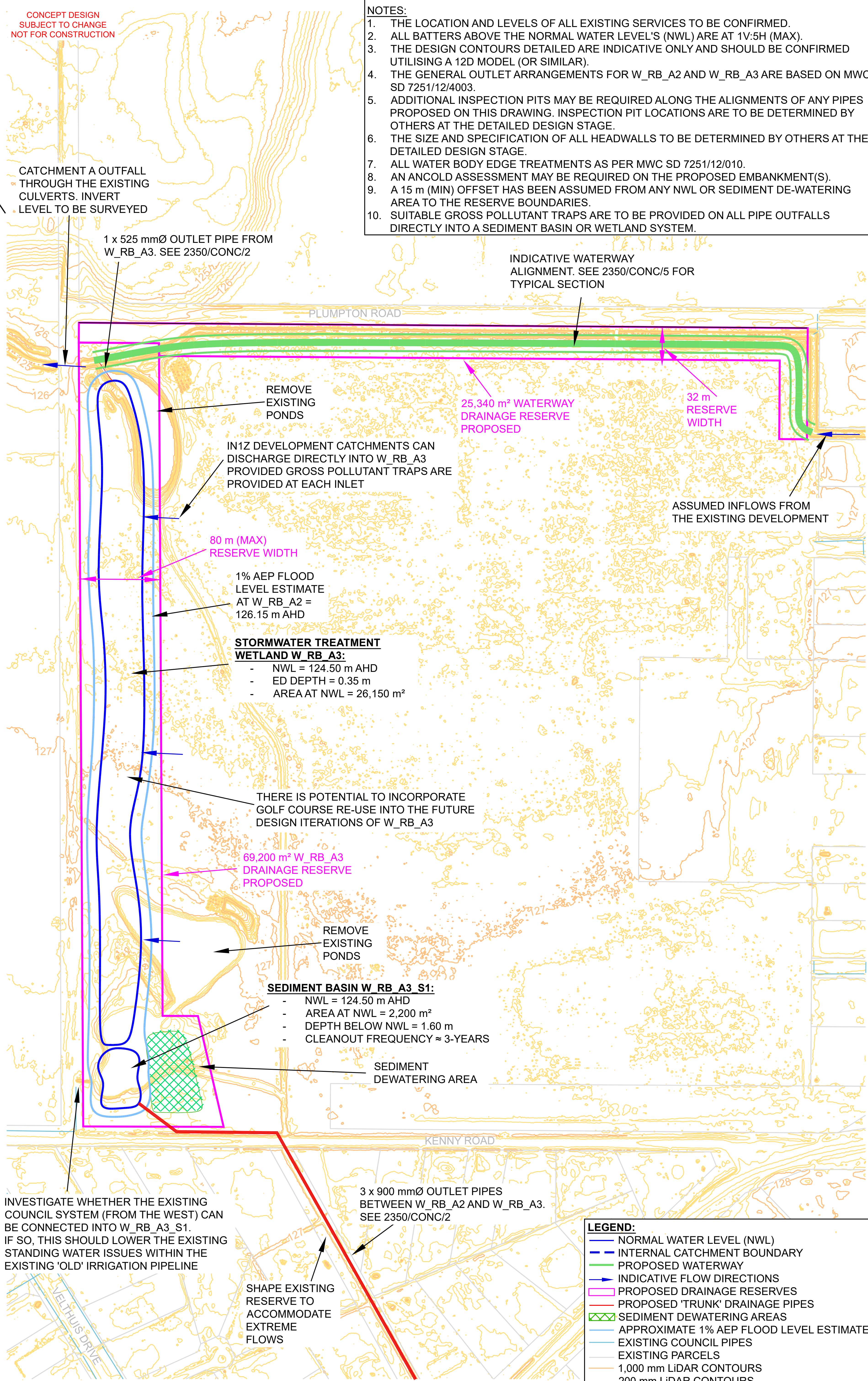
HORSHAM SOUTH
STRUCTURE PLAN
CATCHMENT A DRAINAGE CONCEPT
W_RB_A2 CONCEPT PLAN

SCALE: AS SHOWN
SHEET 3 OF 5
DRAWING No.
2350/CONC/3

CONCEPT DESIGN
SUBJECT TO CHANGE
NOT FOR CONSTRUCTION

NOTES:
1. THE LOCATION AND LEVELS OF ALL EXISTING SERVICES TO BE CONFIRMED.
2. ALL BATTERS ABOVE THE NORMAL WATER LEVEL'S (NWL) ARE AT 1V:5H (MAX).
3. THE DESIGN CONTOURS DETAILED ARE INDICATIVE ONLY AND SHOULD BE CONFIRMED UTILISING A 12D MODEL (OR SIMILAR).
4. THE GENERAL OUTLET ARRANGEMENTS FOR W_RB_A2 AND W_RB_A3 ARE BASED ON MWC SD 7251/12/4003.
5. ADDITIONAL INSPECTION PITS MAY BE REQUIRED ALONG THE ALIGNMENTS OF ANY PIPES PROPOSED ON THIS DRAWING. INSPECTION PIT LOCATIONS ARE TO BE DETERMINED BY OTHERS AT THE DETAILED DESIGN STAGE.
6. THE SIZE AND SPECIFICATION OF ALL HEADWALLS TO BE DETERMINED BY OTHERS AT THE DETAILED DESIGN STAGE.
7. ALL WATER BODY EDGE TREATMENTS AS PER MWC SD 7251/12/010.
8. AN ANCOLD ASSESSMENT MAY BE REQUIRED ON THE PROPOSED EMBANKMENT(S).
9. A 15 m (MIN) OFFSET HAS BEEN ASSUMED FROM ANY NWL OR SEDIMENT DE-WATERING AREA TO THE RESERVE BOUNDARIES.
10. SUITABLE GROSS POLLUTANT TRAPS ARE TO BE PROVIDED ON ALL PIPE OUTFALLS DIRECTLY INTO A SEDIMENT BASIN OR WETLAND SYSTEM.



LEGEND:
- NORMAL WATER LEVEL (NWL)
- INTERNAL CATCHMENT BOUNDARY
- PROPOSED WATERWAY
- INDICATIVE FLOW DIRECTIONS
- PROPOSED DRAINAGE RESERVES
- PROPOSED 'TRUNK' DRAINAGE PIPES
- SEDIMENT DEWATERING AREAS
- APPROXIMATE 1% AEP FLOOD LEVEL ESTIMATE
- EXISTING COUNCIL PIPES
- EXISTING PARCELS
- 1,000 mm LiDAR CONTOURS
- 200 mm LiDAR CONTOURS

REVISION	DATE
V1 – INITIAL ISSUE	08/08/2023
V2 – RESERVE SIZES OPTIMISED	03/04/2024

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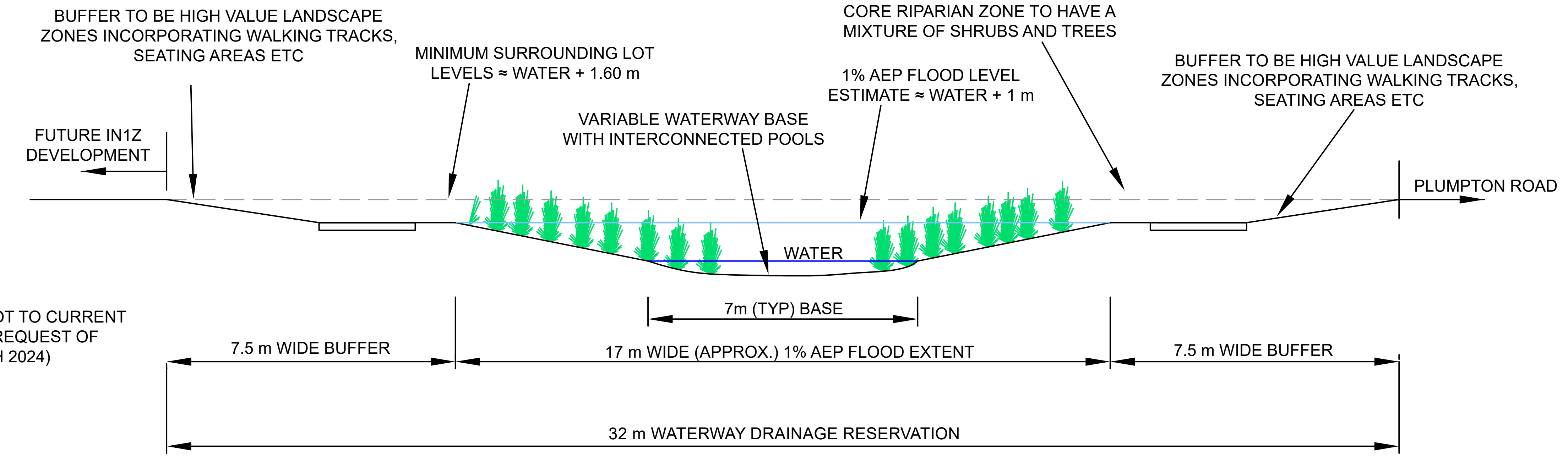
HORSHAM SOUTH
STRUCTURE PLAN
CATCHMENT A DRAINAGE CONCEPT
W_RB_A3 CONCEPT PLAN

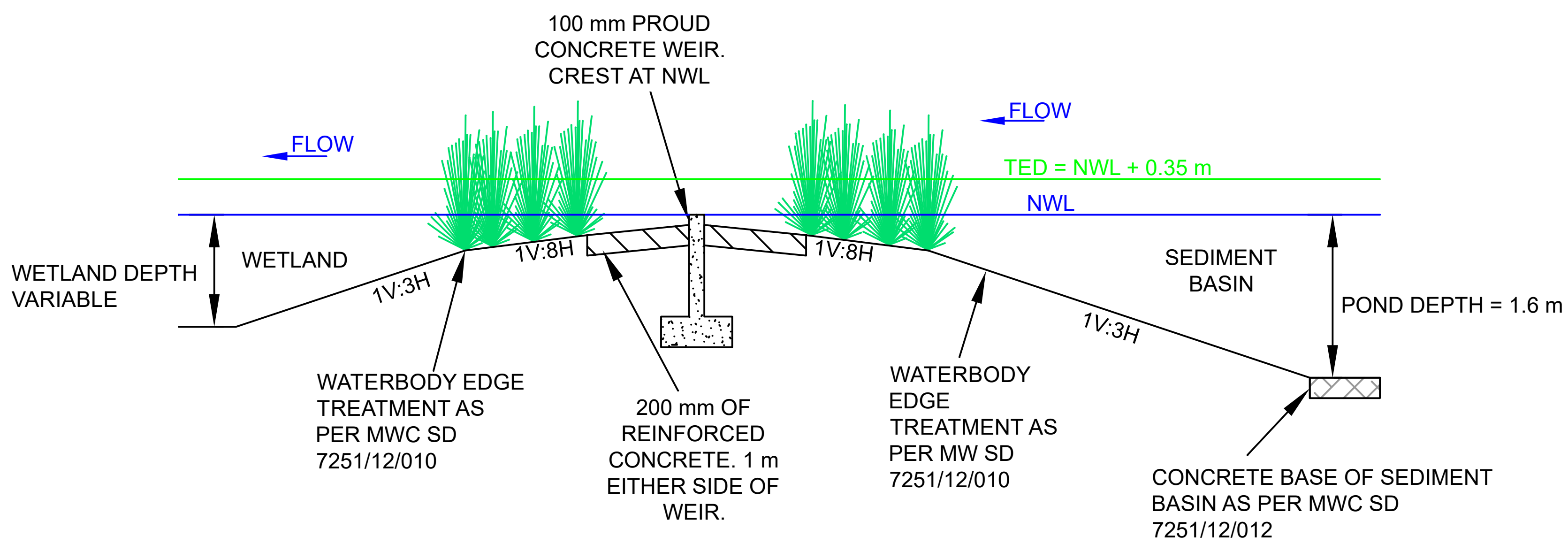
SCALE: AS SHOWN
SHEET 4 OF 5
DRAWING No. 2350/CONC/4

CONCEPT DESIGN
SUBJECT TO CHANGE
NOT FOR CONSTRUCTION

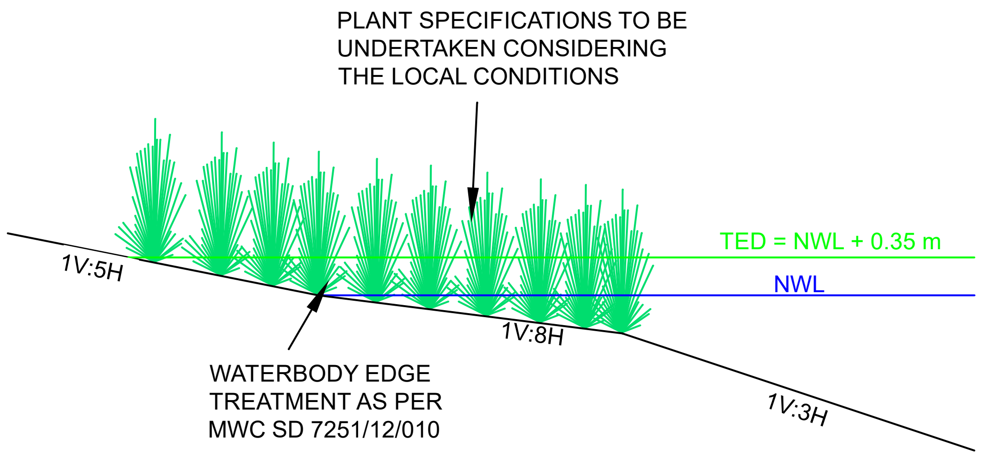


NOTE: RESERVE WIDTH NOT TO CURRENT MWC STANDARDS ON REQUEST OF COUNCIL (MARCH 2024)

TYPICAL IN1Z WATERWAY DETAIL
(NOT TO SCALE)



TYPICAL SEDIMENT BASIN TO WETLAND CONNECTION
(NOT TO SCALE)



TYPICAL WATERBODY EDGE TREATMENT
(NOT TO SCALE)

NOTES:
1. THIS DRAWING HAS BEEN PREPARED TO AID IN THE DESIGN DEVELOPMENT OF THE SEDIMENT BASINS, WETLANDS AND WATERWAY PROPOSALS GOING FORWARD.
2. THE WATERWAY PROPOSAL WILL REQUIRE HECRAS MODELLING TO CONFIRM ITS SIZE INTO THE FUTURE.
3. MWC = MELBOURNE WATER CORPORATION, SD = STANDARD DRAWING

REVISION	DATE
V1 – INITIAL ISSUE	08/08/2023
V2 – WATERWAY RESERVE DECREASED	08/08/2023

Stormy Water Solutions

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HORSHAM SOUTH
STRUCTURE PLAN
CATCHMENT A DRAINAGE CONCEPT
GENERAL DETAILS

SCALE: NOT TO SCALE
SHEET 5 OF 5
DRAWING No. 2350/CONC/5

Appendix B Hydrological Design and Modelling

The RORB Runoff Routing Program – Version 6.45, developed at Monash University by E. M. Laurenson and R. G. Mein, was used to determine the pre and post development scenario design flow estimates originating from Catchment A. RORB is a general runoff and stream flow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall excess and routes this through catchment storage to produce the hydrograph.

RORB is an industry standard software currently used for the formulation of drainage system designs. It was the software utilised within the Regional Flood Mapping.

B.1 Pre-development

B.1.1 Model Description

The pre-development conditions model is based on the LiDAR information and 1 m contour information from Council. The model has been formulated for the expected catchments in the 1% AEP event and hence largely neglects the existing primed system that services the Colonial Drive development (which has limited capacity).

Figure B.1 details the RORB model for the pre-development conditions and Tables B.1 and B.2 detail the tabulation of the RORB model setup (i.e. catchment area, F_{imp}, reach lengths, etc).

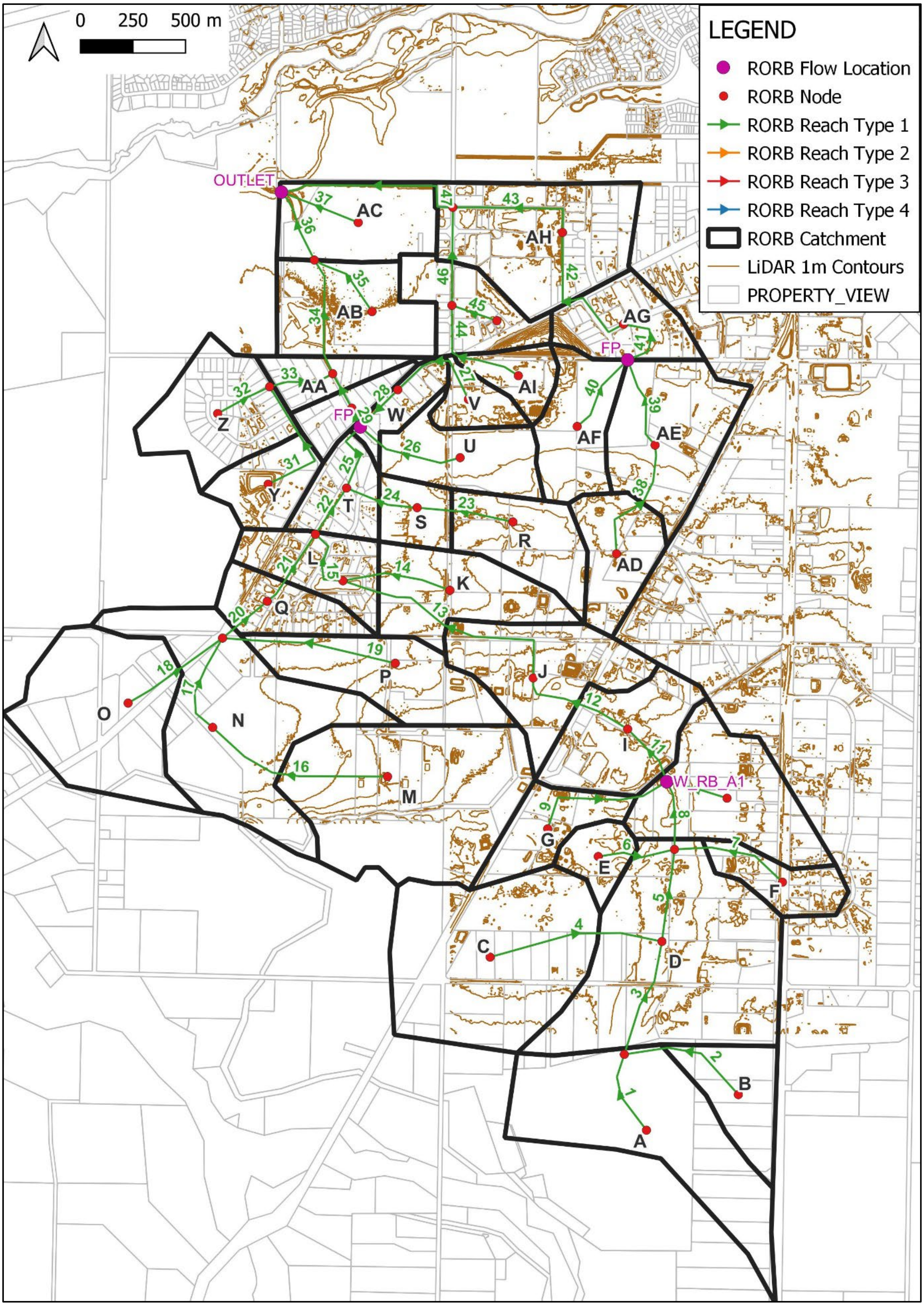

Figure B.1 Pre-development RORB model schematic

Table B.1 Pre-development RORB Catchments

Sub Area	Area (Ha)	Area (km^2)	F_{imp}
A	59.2	0.592	0.05
B	18.7	0.187	0.05
C	70.0	0.700	0.05
D	81.4	0.814	0.05
E	9.4	0.094	0.00
F	13.3	0.133	0.10
G	16.8	0.168	0.05
H	39.3	0.393	0.15
I	29.8	0.298	0.20
J	35.2	0.352	0.20
K	29.3	0.293	0.10
L	73.2	0.732	0.05
M	65.1	0.651	0.10
N	50.3	0.503	0.10
O	32.1	0.321	0.05
P	15.6	0.156	0.45
Q	19.2	0.192	0.45
R	23.3	0.233	0.10
S	10.6	0.106	0.05
T	14.6	0.146	0.45
U	22.9	0.229	0.05
V	8.9	0.089	0.05
W	8.6	0.086	0.35
X	9.3	0.093	0.35
Y	15.0	0.150	0.10
Z	30.1	0.301	0.45
AA	10.7	0.107	0.35
AB	31.1	0.311	0.05
AC	27.5	0.275	0.05
AD	20.0	0.200	0.10
AE	36.7	0.367	0.05
AF	22.4	0.224	0.05
AG	20.2	0.202	0.70
AH	47.1	0.471	0.70
AI	9.9	0.099	0.05
AJ	20.5	0.205	0.70
Total	**1047.4**	**10.474**	**0.17**

Note: No impervious area splitting has been undertaken due to the utilisation of the Regional Flood Mapping's parameter sets. Modelling without impervious area splitting will likely produce conservative results (i.e. higher flows and volumes) (Chapter 5.3.4.1.2, Book 5, ARR 2019).

Table B.2 Pre-development RORB Reaches

Reach	Reach Type	Length (km)	Slope (%)
1	1	0.408	
2	1	0.634	
3	1	0.571	
4	1	0.836	
5	1	0.442	
6	1	0.372	
7	1	0.568	
8	1	0.329	
9	1	0.695	
10	1	0.297	
11	1	0.318	
12	1	0.549	
13	1	1.158	
14	1	0.524	
15	1	0.340	
16	1	0.907	
17	1	0.495	
18	1	0.546	
19	1	0.833	
20	1	0.275	
21	1	0.395	
22	1	0.265	
23	1	0.461	
24	1	0.352	
25	1	0.362	
26	1	0.525	
27	1	0.499	
28	1	0.251	
29	1	0.098	
30	1	0.187	
31	1	0.663	
32	1	0.278	
33	1	0.313	
34	1	0.563	
35	1	0.383	
36	1	0.362	
37	1	0.393	
38	1	0.606	
39	1	0.443	
40	1	0.407	
41	1	0.339	
42	1	0.718	
43	1	0.637	
44	1	0.561	
45	1	0.226	
46	1	0.466	
47	1	0.898	

B.1.2 Model Inputs, Parameters and Validation

ARR 2019 datahub inputs (Location: 36.748 S, 142.185 E, Accessed 10/07/23) have been utilised within the modelling herein (i.e. IFD's, temporal patterns, Areal Reduction Factors). However, given the adoption of the Regional Flood Mapping's sets, no pre-burst from the ARR 2019 datahub has been adopted in the modelling.

Three potential parameter sets have been identified based on the Regional Flood Mapping and ARR 2019 as shown in Table B.3.

Table B.3 Potential RORB Parameter Sets

Set ID	Description	k_c	d_{av} (km)	k_c/d_{av}	m	IL (mm)	Loss
1	Horsham and Wartook Valley Flood Investigation - Rivers	4.10	3.28	1.250	0.8	34	CL = 3 mm/hr
2	Horsham and Wartook Valley Flood Investigation - Rain on Grid	4.10	3.28	1.250	0.8	4	CL = 1.5 mm/hr
3	ARR Datahub Losses	4.10	3.28	1.250	0.8	39	CL = 1.5 mm/hr

The RORB model has been simulated with each of the three potential parameter sets (without any local storages being modelled). The flow estimates at both location "W_RB_A1" and "OUTLET" (see Figure B.1 for the locations) from each of the three sets have been compared to other high level flow estimation methods as shown in Tables B.4 and B.5.

Table B.4 Flow estimates generated at "W_RB_A1" for various methods

Method	1% AEP Flow Estimate (m^3/s)	50% AEP Flow Estimate (m^3/s)
RORB - Set 1	11.40	0.30
RORB - Set 2	18.30	4.15
RORB - Set 3	11.25	0.40
Rational	7.95	0.90
DSE Curve	11.00	-
RFFE	10.50	1.30

Notes: *1. All flow estimates rounded to the nearest 0.05 m^3/s.*

2. Regression Curve = Rural Nikoloau/Vont Steen Equation ($Q = 4.67A^{0.763}$) from the MWC Flood Mapping Guideline. This model does not account for variations in development and reach types within a catchment and should be utilised with caution.

3. Rational Method assumptions:

- *No partial area effects simulated with the rational method.*
- *$C_{1\%AEP}$ = 0.30, $C_{50\%AEP}$ = 0.10.*
- *t_c = determined utilising a L/V method to give a t_c of 98 min.*
- *IFD 2016 IFD at (36.7375 S, 142.1875 E) which is consistent with RORB model.*

4. RFFE to be used with caution as the catchment characteristics match those in which the software sates the RFFE model cannot be applied (https://rffe.arr-software.org/limits.html).

Table B.5 Flow estimates generated at "OUTLET" for various methods

Method	1% AEP Flow Estimate (m^3/s)	50% AEP Flow Estimate (m^3/s)
RORB - Set 1	22.40	1.30
RORB - Set 2	34.75	7.80
RORB - Set 3	24.40	1.55
Rational	20.10	2.25
DSE Curve	28.05	-
RFFE	29.60	3.70

Notes: [1.] *All flow estimates rounded to the nearest 0.05 m^3/s.*

[2.] *Regression Curve = Rural Nikoloau/Vont Steen Equation ($Q = 4.67A^{0.763}$) from the MWC Flood Mapping Guideline. This model does not account for variations in development and reach types within a catchment and should be utilised with caution.*

[3.] *Rational Method assumptions:*

- *No partial area effects simulated with the rational method.*
- *$C_{1\%AEP}$ = 0.30, $C_{50\%AEP}$ = 0.10.*
- *t_c = determined utilising a L/V method to give a t_c of 153 min.*
- *IFD 2016 IFD at (36.7375 S, 142.1875 E) which is consistent with RORB model.*

[4.] *RFFE to be used with caution as the catchment characteristics match those in which the software sates the RFFE model cannot be applied (https://rffe.arr-software.org/limits.html).*

Based on Tables B.4 and B.5, parameter set 1 has been adopted within the study herein as:

- It is producing 1% AEP flow estimates within the expected orders of magnitude; and
- Was the set utilised within the Regional Flood Mapping.

B.1.3 Local Storages

There are expected to be numerous local storages within the catchment. Conservatively, these have not been modelled.

However, the large existing storage at W_RB_A1 has been modelled in the pre-development conditions. Council have supplied Figure B.2 which shows the flood impact at W_RB_A1 in the November 2022 event.

Figure B.2 November 2022 Flood Extent at W_RB_A1. Source: Council – looking south

Currently, it is SWS's understanding that there are no formal (structural) outlets from W_RB_A1. Stormwater ponds behind (south) of the walking track in a flood event and 'sits' for days or weeks until it is infiltrated or evaporated.

Given this, in most small flood events there are no flood outflows in the existing conditions from W_RB_A1.

To reflect this, W_RB_A1 has been simulated in the pre-development model as a retarding basin with characteristics as shown in Table B.6 (which have been estimated based on the LiDAR information).

Table B.6 W_RB_A1 RORB simulation details in the pre-development conditions

Spillway Details				
Crest Elevation (m)	Length (m)			
133.00	10			
Pipe Details				
Length (m)	Slope (%)	Invert (m)	No.	Size (mØ)
N/A	N/A	N/A	N/A	N/A
Storage Details				
Initial Drawdown (m^3)	85780			
Level (m)	Storage (m^3)	Notes:		
133.00	0			
133.50	129485			

B.1.4 Model Results

The model has been simulated for a range of AEP storm events using the full ensembles of 240 temporal patterns as required in ARR 2019. The resultant estimates of the flood flows throughout the catchment are provided in Table B.7.

Table B.7 Pre-development Flow Estimates

Location	1% AEP		50% AEP	
	Q (m^3/s)	Duration	Q (m^3/s)	Duration
W_RB_A1 - Outflow	0.80	48-hour	0.00	168-hour
W_RB_A1 - Inflow	11.40	6-hour	0.30	1.5-hour
Golf Course Road	14.25	6-hour	0.50	36-hour
Outfall	19.65	6-hour	1.30	3-hour

Notes: All flow estimates rounded to the nearest 0.05 m^3/s and are reported as the peak average for the critical duration.

It is also noted that in the pre-development scenario, the 1% AEP flood level estimate within W_RB_A1 is expected to be 133.15 m AHD (when rounded up to the nearest 0.05 m).

B.2 Post-development

B.2.1 Model Description

The post-development conditions model is generally based on the LiDAR information and the proposed servicing within Appendix A. Figure B.3 details the RORB model for the post-development conditions and Tables B.8 and B.9 detail the tabulation of the RORB model setup (i.e. catchment area, F_{imp}, reach lengths, etc).

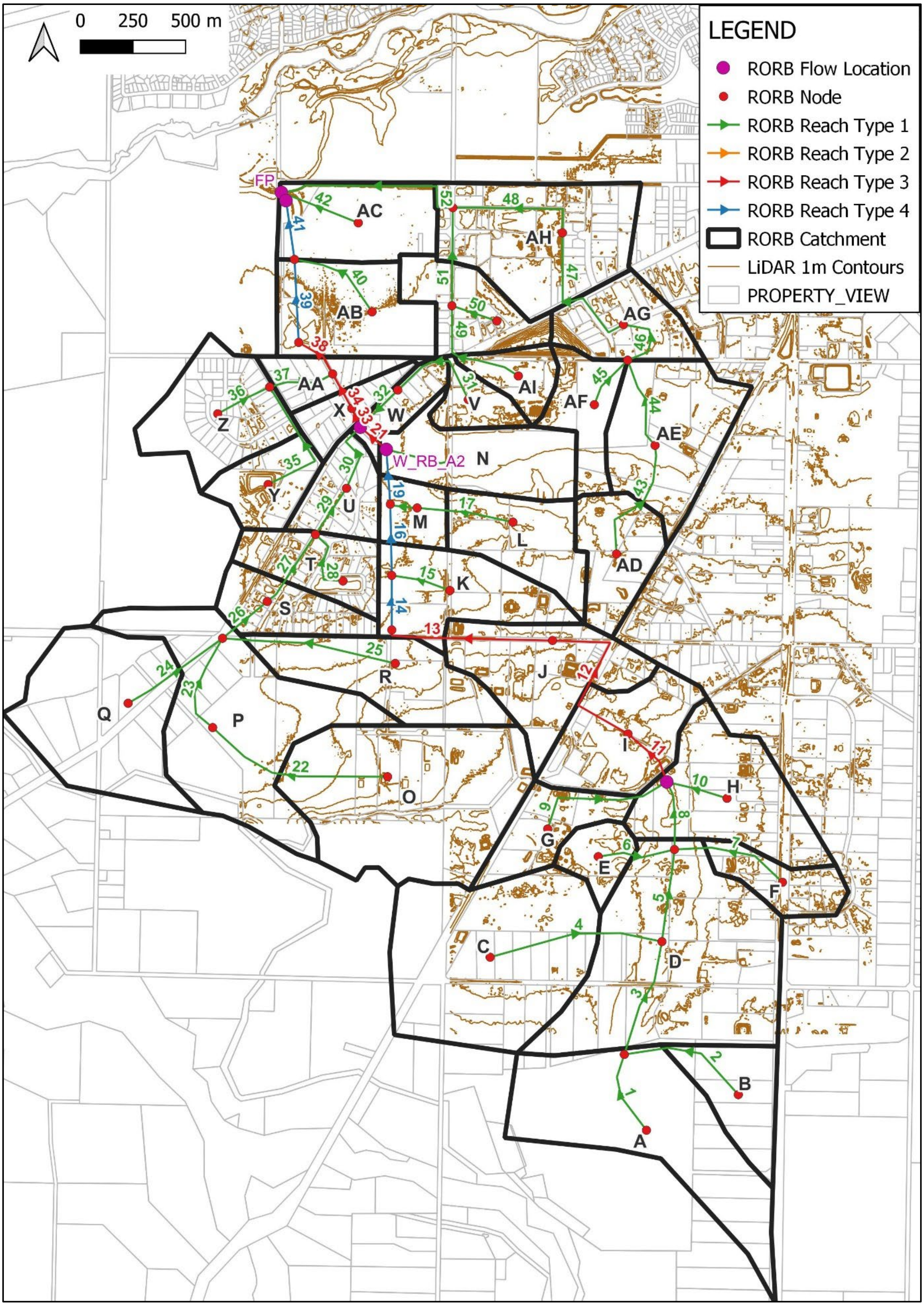

Figure B.3 Post-development Conditions RORB model Schematic

Table B.8 Post-development RORB Catchments

Sub Area	Area (Ha)	Area (km^2)	F_{imp}
A	59.2	0.592	0.10
B	18.7	0.187	0.10
C	70.0	0.700	0.15
D	81.4	0.814	0.15
E	9.4	0.094	0.20
F	13.3	0.133	0.20
G	16.8	0.168	0.20
H	39.3	0.393	0.20
I	29.8	0.298	0.30
J	35.2	0.352	0.30
K	29.4	0.294	0.30
L	23.8	0.238	0.30
M	9.9	0.099	0.30
N	29.1	0.291	0.30
O	73.2	0.732	0.10
P	65.1	0.651	0.10
Q	50.3	0.503	0.10
R	32.1	0.321	0.20
S	16.3	0.163	0.45
T	18.5	0.185	0.45
U	14.5	0.145	0.45
V	8.9	0.089	0.70
W	8.6	0.086	0.35
X	9.3	0.093	0.35
Y	15.0	0.150	0.10
Z	30.1	0.301	0.45
AA	10.7	0.107	0.35
AB	31.1	0.311	0.70
AC	27.5	0.275	0.70
AD	20.2	0.202	0.50
AE	36.7	0.367	0.70
AF	16.2	0.162	0.60
AG	20.2	0.202	0.70
AH	47.1	0.471	0.70
AI	9.9	0.099	0.70
AJ	20.5	0.205	0.70
Total	**1047.4**	**10.474**	**0.31**

Table B.9 Post-development RORB Reaches

Reach	Reach Type	Length (km)	Slope (%)
1	1	0.408	
2	1	0.634	
3	1	0.571	
4	1	0.836	
5	1	0.442	
6	1	0.372	
7	1	0.568	
8	1	0.329	
9	1	0.695	
10	1	0.297	
11	3	0.302	0.20%
12	3	0.883	0.20%
13	3	0.792	0.20%
14	4	0.259	
15	1	0.287	
16	4	0.339	
17	1	0.461	
18	1	0.129	
19	4	0.261	
20	1	0.363	
21	3	0.162	0.20%
22	1	0.907	
23	1	0.495	
24	1	0.546	
25	1	0.833	
26	1	0.275	
27	1	0.395	
28	1	0.340	
29	1	0.265	
30	1	0.362	
31	1	0.499	
32	1	0.251	
33	3	0.098	0.20%
34	3	0.187	0.20%
35	1	0.663	
36	1	0.278	
37	1	0.313	
38	3	0.229	0.20%
39	4	0.395	
40	1	0.474	
41	4	0.327	

Reach	Reach Type	Length (km)	Slope (%)
42	1	0.393	
43	1	0.606	
44	1	0.443	
45	1	0.270	
46	1	0.339	
47	1	0.718	
48	1	0.637	
49	1	0.561	
50	1	0.226	
51	1	0.466	
52	1	0.898	

Note: Lengths and slopes subject to change.

B.2.2 Model Inputs, Parameters and Validation

Generally, the same parameter set and inputs as the pre-development model have been utilised. The only change being the K_c has now been set to 4.16 to reflect the new d_{av} of the model (3.33 km).

B.2.3 Retardation Basin High Level Concepts

Iteratively with the MUSIC modelling (Appendix C), three RB's (which contain three stormwater treatment wetlands) are proposed throughout Catchment A within the SP.

At this high-level concept stage, the general outlet arrangement of W_RB_A2 and W_RB_A3 is expected to be generally as per the MWC standard drawing 7251/12/4003 (reproduced as Figure B.4 below).

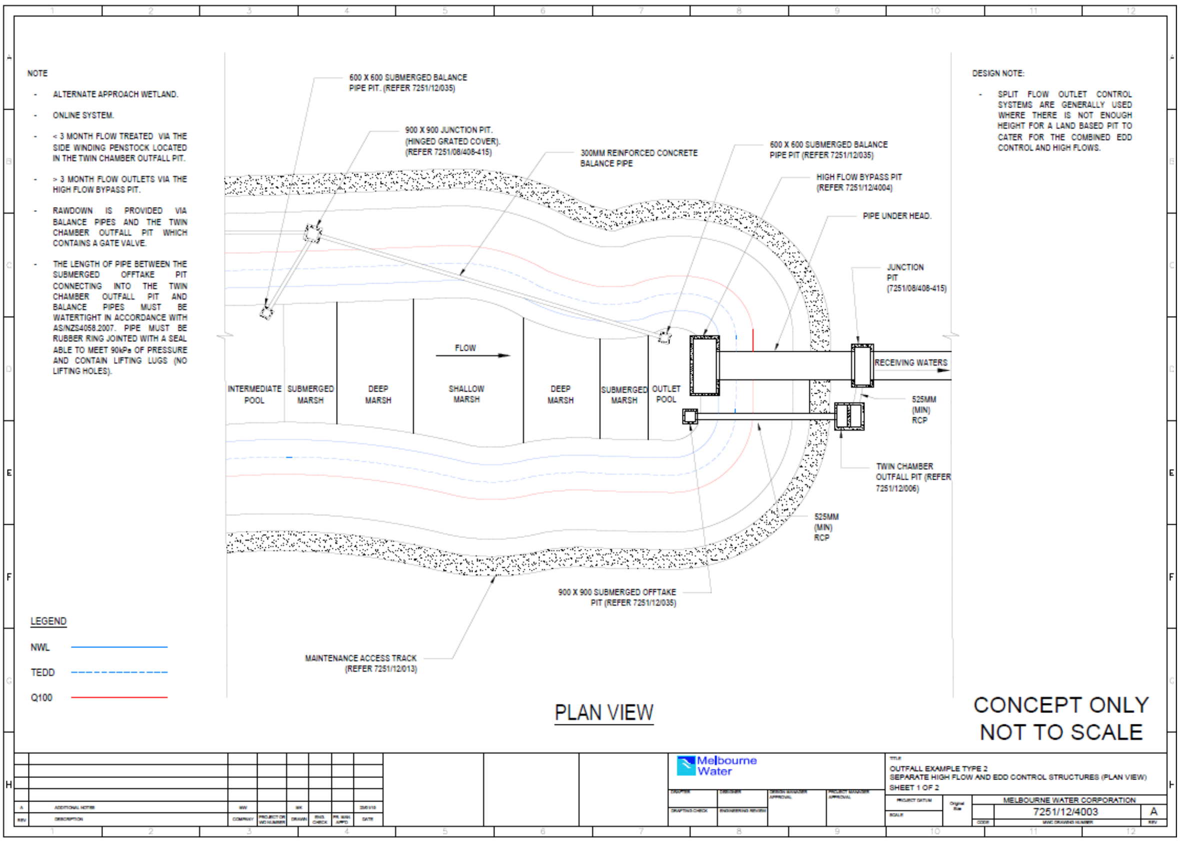

Figure B.4 MWC standard drawing 7251/12/4003 – the expected general form all RB outlets.

It is noted that, the designs proposed for W_RB_A1, W_RB_A2 and W_RB_A3 are conceptual only. The designs may change as the catchment A strategy is further developed.

W_RB_A1

W_RB_A1 is proposed in the approximate current footprint of the existing depression that floods (see Figure B.2).

However, it is recognised that having floodwaters 'sitting' for days or weeks after a storm is not an ideal design outcome. Thus, it is proposed to provide a 450 mmØ outfall pipe, at an invert level of 300 mm above the existing base of the depression.

This arrangement:

- Is assumed to allow for the current ecology to be maintained, being that in frequent events, the base (lowest 300 mm) of the depression ponds extended periods of time; but
- Allows for the depression to drain more quickly in an extreme flood event.

Drawing 2350/CONC/2 details the arrangement proposed. Table B.10 provides details into how the proposed asset has been simulated within the RORB model.

Table B.10 W_RB_A1 RORB post development conceptual modelling

Spillway Details				
Crest Elevation (m)	Length (m)			
133.00	10			
Pipe Details				
Length (m)	Slope (%)	Invert (m)	No.	Size (mØ)
600	0.22	132.30	1	0.450
Storage Details				
Initial Drawdown (m^3)	3447			
Level (m)	Storage (m^3)	Notes:		
132.30	0	Pipe invert		
132.60	8957			
132.80	35082			
133.00	82333	Spillway level		
133.50	211818			

W_RB_A2

W_RB_A2 is proposed as per drawing 2350/CONC/3. This retarding basin is proposed in the void space above the stormwater treatment wetland. The advantage of this asset (combined with W_RB_A3) is that it allows for a 'deep' pipe outfall at Plozzas Road which can be utilised to service the expected densification of Haven without the need for excess fill (for both pipe outfalls and 1% AEP freeboard). In addition, the 1% AEP outflow is contained to the outlet pipe system, and therefore does not impact the existing downstream development.

Table B.11 provides details into how the proposed asset has been simulated within the RORB model.

Table B.11 W_RB_A2 RORB post development conceptual modelling

Spillway Details				
Crest Elevation (m)	Length (m)			
127.60	10			
Pipe Details				
Length (m)	Slope (%)	Invert (m)	No.	Size (mØ)
600	0.22	125.80	3	0.900
Storage Details				
Initial Drawdown (m^3)	10060			
Level (m)	Storage (m^3)	Notes:		
126.15	0	Wetland TED		
126.50	11617			
127.00	30731			
127.50	52830			
127.60	57550	Spillway level		
128.00	76428			

W_RB_A3

W_RB_A3 is proposed as per drawing 2350/CONC/4. This retarding basin is proposed in the void space above the stormwater treatment wetland. The advantage of this asset (combined with W_RB_A3) is that it allows for a 'deep' pipe outfall at Plozzas Road which can be utilised to service the expected densification of Haven without the need for excess fill (for both pipe outfalls and 1% AEP freeboard).

W_RB_A3's outlet (together with the upstream RB's) has also been sized to ensure that the 50% and 1% AEP pre-development estimates at the Catchment A outfall are not exceeded.

Table B.12 provides details into how the proposed asset has been simulated within the RORB model.

Table B.12 W_RB_A2 RORB post development conceptual modelling

Spillway Details				
Crest Elevation (m)	Length (m)			
125.40	12			
Pipe Details				
Length (m)	Slope (%)	Invert (m)	No.	Size (mØ)
30	0.22	124.85	1	0.525
Storage Details				
Initial Drawdown (m^3)	10675			
Level (m)	Storage (m^3)	Notes:		
124.85	0	Wetland TED		
125.00	5002			
125.50	23182	Approx. Upper Spillway		
126.00	43697			
126.50	65387			

B.2.4 Model Results

Pre to Post Flow Comparison

Tables B.13 below compares the pre to post-development flow estimates at the catchment A outfall (approx. 36.734 S, 142.178 E) and shows that for the 1% AEP and 50% AEP events, the pre-development flow estimates are not exceeded if the strategy and assets proposed in Appendix A are implemented.

Table B.13 Flow estimate comparison at the Catchment A outfall

AEP	Pre		Post	
	Q (m^3/s)	Duration	Q (m^3/s)	Duration
50%	1.30	3-hour	1.25	4.5-hour
1%	19.65	6-hour	19.60	6-hour

Notes: *All flow estimates rounded to the nearest 0.05 m^3/s and are reported as the peak average for the critical duration.*

Retardation Basin Flood Function

The RORB model has also been used to produce flood level and storage estimates required for the various retarding basin assets proposed across Catchment A as shown in Tables B.14.

Crucially, the 1% AEP flood level estimates within Table B.14 provide:

- For W_RB_A1, a 1% AEP flood level estimate as per the pre-development level estimate (when rounded up to the nearest 0.05 m);
- For W_RB_A2, around 600 mm of freeboard to the (assumed) minimum lot levels of the existing lots in the vicinity of 229 Golf Course Road, Haven (approx. 127.60 m AHD); and
- For W_RB_A3, a 1% AEP flood level estimate roughly equal to the natural surface level of the IN1Z lot (approx. 126.20 m AHD), meaning that minimal fill will be required on this lot for flood protection.

Table B.14 Estimates of the 1% AEP flood function for the proposed RB's

Asset	1% AEP Inflow Estimate		1% AEP Outflow Estimate		Representative Outflow Temporal Pattern	1% AEP Flood Level Estimate (m AHD)	1% AEP Flood Storage Estimate (m^3)
	Q (m^3/s)	Duration	Q (m^3/s)	Duration			
W_RB_A1	12.05	6-hour	1.10	36-hour	30	133.15	119,000
W_RB_A2	11.15	2-hour	3.10	6-hour	22	126.95	27,300
W_RB_A3	15.65	6-hour	14.00	6-hour	29	126.15	50,200

Note: *All flow estimates are rounded to the nearest 0.05 m^3/s and are taken as the peak average for the critical duration. Flood storage estimates are rounded up to the nearest 100 m^3 and flood level estimates are rounded up to the nearest 50 mm.*

Stormwater Treatment Asset Design Flows

The RORB model has also been used to produce design flow estimates required for the various stormwater treatment assets proposed across Catchment A as shown in Tables B.15. It is noted that the very frequent flow estimates within Table B.15 are estimates and will have to be refined/revised as the designs, and development proposals, are further progressed into the future.

Table B.15 Flow estimates into the proposed treatment elements.

Treatment Element	1% AEP		63% AEP		4 EY
	Q (m^3/s)	Duration	Q (m^3/s)	Duration	Q (m^3/s)
W_RB_A2_S1	5.05	2-hour	0.85	1.5-hour	0.35
W_RB_A2_S2	6.10	2-hour	0.90	1.5-hour	0.35
W_RB_A3_S1	12.85	6-hour	1.00	2-hour	0.40

Notes: All flow estimates rounded to the nearest 0.05 m^3/s and are reported as the peak average for the critical duration.

B.2.5 Design Checks

Climate Change

ARR2019 requires that designers assess climate change risks. Conservatively, the RCP8.5, year 2090 IFD rainfall increases of 20.2% have been simulated. Table B.16 shows the difference between the 1% AEP flood level estimates at each of the retarding basins under the current and potential future climate scenario.

Provided adequate freeboard (600 mm from the estimates within Table B.14) is provided, and suitable extreme flow provisions are allowed for, there should not be any adverse 1% AEP flood impacts in the potential future climate scenario on the proposed assets as the freeboard allowance suitability contains the increased 1% AEP climate change flood level estimates.

Table B.16 Climate change 1% AEP flood level estimates

Asset	Current climate 1% AEP flood level estimate (m AHD)	Potential future climate 1% AEP flood level estimate (m AHD)	Potential Change (m)
W_RB_A1	133.15	133.30	0.15
W_RB_A2	126.95	127.25	0.30
W_RB_A3	126.15	126.35	0.20

Note: All flood level estimates are rounded up to the nearest 50 mm.

Blockage

A blockage analysis as determined from ARR 2019, Book 6, Chapter 6 has been applied to the design of the three retarding basin outlet systems as per Table B.17 below.

Table B.17 ARR Blockage Factor Determination

Aspect	ARR Reference	W_RB_A1	W_RB_A2	W_RB_A3
L_{10} (m)	Section 6.4.4.1	1.5	1.5	1.5
Debris Availability	Table 6.6.1	M	M	M
Debris Mobility	Table 6.6.2	H	L	L
Debris Transportability	Table 6.6.3	L	L	L
Debris Potential	Table 6.6.4	M	L	L
1% AEP adjusted Debris Potential	Table 6.6.5	M	L	L
Control Dimension Inlet Clear Width, W (assume 200 mm bar spacings) (m)		0.2	0.2	0.2
Design Inlet Blockage	Table 6.6.6	50%	25%	25%
Likelihood of Sediment being deposited	Table 6.6.7	L	L	L
Design Depositional Blockage	Table 6.6.8	25%	0%	0%
Blockage Factor Applied		**50%**	**25%**	**25%**

Using the factors from Table B.17, the RORB model was re-simulated and new 1% AEP blockage flood level estimates have been generated as per Table B.18 below.

Table B.18 Blockage 1% AEP flood level estimates

Asset	Normal operation 1% AEP flood level estimate (m AHD)	1% AEP flood level estimate with blockage (m AHD)	Change (m)
W_RB_A1	133.15	133.20	0.05
W_RB_A2	126.95	127.05	0.10
W_RB_A3	126.15	126.20	0.05

Note: All flood level estimates are rounded up to the nearest 50 mm.

Provided adequate freeboard (600 mm from the estimates within Table B.14) is provided, and suitable extreme flow provisions are allowed for, there should not be any adverse 1% AEP flood impacts in the potential blockage scenario.

Appendix C WSUD Design and Modelling

C.1 Primary/Sediment Treatment Proposals

C.1.1 Gross Pollutant Traps

Gross pollutant traps (**GPT**'s) are proposed upstream of all pipe outfalls (either into a wetland or a sediment basin) throughout the Catchment A region within the SP.

Exact specification of the GPT's has not been undertaken at this stage. However, within the MUSIC modelling (see Appendix C.3), "standard" performance of the GPT's have been simulated being:

- Litter 98% capture assumed;
- TSS 70% capture assumed;
- TP 30% capture assumed; and
- TN 0% capture assumed.

C.1.2 Sediment Basins

Three sediment basins (IDM terminology **Sedimentation Basins**) are proposed at key development inlets to the wetlands as shown in drawing 2350/CONC/3 AND 4. These basins have been sized as per Table C.1.

It is noted that the Wetland Design Manual requires online sediment basins (as is proposed herein) be set with a NWL 100 mm higher than the downstream wetland system. Given the flat, constrained nature of the catchment, at this stage this 100 mm difference has not been assumed (as per the typical "sediment pond to wetland" detail shown within drawing 2350/CONC/5). As the designs are progressed and further refined, it may be possible to incorporate this level difference.

It is also noted that the IDM does not specify a design cleanout frequency for sediment basins. The Wetland Design Manual recommends 5-years. However, given the size of the catchments this cannot be achieved within 'typical' basin sizes or depths. As such, more frequent cleanout frequencies are proposed herein.

Cleanout maintenance will require:

- Temporary pump out of water within the sediment pond to the downstream wetland, and
- Temporary low flow pump bypass or upstream pipe flows around the sediment pond to the wetland system downstream.

The drainage reserve allocations allow for sediment dewatering areas. These are typically grassed mown landscaped areas which are used every 3 to 5 years or so to temporarily store excavated sediment for drying before removal from site.

In addition, the reserve allocations allow for future provision of maintenance access paths etc.

Table C.1 Sediment Basin Sizing Calculations

Asset Properties				
Asset ID	W_RB_A2_S1	W_RB_A2_S2	W_RB_A3_S1	
Normal Water Level = NWL =	125.80	125.80	124.50	m AHD
NWL Area = (A_{asset}) =	1,065	1,065	2,200	m^2
Pond Depth = (d_p) =	1.60	1.60	1.60	m
Extended Detention Depth = (d_e) =	0.35	0.35	0.35	m
Volume = (Vol_{TOT}) =	1,010	1,010	2,485	m^3
Sump Volume = (Vol_S) =	695	695	1,800	m^3
1EY Inflow = (Q_{1EY}) =	0.85	0.90	1.00	m^3/s
λ =	0.26	0.26	0.11	
Upstream Catchment Area = (A_{Catch}) =	95.0	63.0	353.0	ha
Target Particle Settling Velocity = (V_s) =	0.011	0.011	0.011	m/s
Removal Efficiency				
d* =	1.6	1.6	1.6	m
$\frac{d_e + d_p}{d_e + d^*} =$	1.0	1.0	1.0	
$\frac{V_s \times A_{asset}}{Q_{4EY}} =$	13.8	13.0	24.2	
$n = \frac{1}{1-\lambda} =$	1.35	1.35	1.12	
$Removal\ efficiency^{5.} = R = 1 - \left[1 + \frac{1}{n} \times \frac{V_s \times A_{asset}}{Q_{4EY}} \times \frac{d_e + d_p}{d_e + d^*}\right]^{-n} =$	**96.2%**	**95.9%**	**97.0%**	
Cleanout Frequency				
Sediment Load = (L_s) =	1.6	1.6	1.6	$m^3/ha/yr$
Gross Pollutant Load = (L_{GP}) =	0.4	0.4	0.4	$m^3/ha/yr$
$\text{Cleanout Frequency} = \frac{R \times (L_S + L_{GP}) \times A_{Catch}}{Vol_S} =$	3.8	5.8	2.6	years
Dry Out Area (at 500 mm deep) =	1,390	1,390	3,600	m^2

Notes: *1. Sump volume taken as the volume below 350mm deep (i.e. below the safety bench).*

2. 1EY Sizing as per the IDM

3. Hydraulic efficiency estimated from Figure 4.3 of the WSUD Engineering Procedures.

4 Target particle size taken as 125 µm (as per IDM) with a settling velocity sourced from Table 4.1 of the WSUD Engineering Procedures.

5. Methodology taken from Chapter 4.3.2 of the WSUD Engineering Procedures.

6. Load estimate sourced from Willing and Partners 1992.

7. Load estimate sourced from Allison et. al. 1998.

C.2 Stormwater Treatment Proposals

C.2.1 Swales

Given the flat catchments, and the proposed redevelopment within the SP, it is likely that grassed swales will be used to service some development (especially in “upper” catchment areas). Grassed swales may typically be used as a lot or subdivisional scale solution, or to convey/treat road flows.

Grassed swales will almost certainly be used as the conveyance mechanism for 20% AEP flows in Catchment A1 (Figure 4).

It is not yet known where grassed swales will be applied in the catchments as drainage outfalls (as opposed to pit and pipe outfalls). As such, conservatively (in regard to stormwater treatment benefits) they have not been included in the strategy modelling herein. If grassed swales are included within the SP into the future, it will only improve the WSUD outcomes of the SP.

C.2.2 Stormwater Re-use

Tanks

It is likely that tanks will be installed on each building within the SP for re-use. In residential areas this would typically be 2,000 litre tanks for toilet and laundry use on each lot. However, as the exact densities across the SP are not known, conservatively tanks have not been included in the strategy modelling herein. If tanks are included within the SP into the future, it will only improve the WSUD outcomes of the SP.

Golf Course

At the site visit it was observed that at location 'A' (Figure 10) there was a pump station labelled the '*Horsham Golf Club Storm and Reclaimed Water Treatment Wetland Project*'.

SWS have not been provided any details of this system. As such, it has not been included in the strategy herein.

However, retrofitting (or modifying) the proposals herein to account for golf course stormwater reuse (if required) should be easily accommodated into the future and will only improve the WSUD outcomes of the SP.

C.2.3 Wetlands

Two constructed, and one informal (existing) stormwater treatment wetlands are proposed as shown within the catchment as described within drawing 2350/CONC/1 and Table C.2.

Assets W_RB_A2 and W_RB_A3 are proposed as constructed stormwater treatment wetland (IDM terminology **Constructed Wetland**).

Though the IDM proposes a 'high-flow bypass' to convey flows in excess of the 1EY design event, this has not been accommodated for within the design of the systems (i.e. the systems are online). Though providing stormwater treatment benefits, the primary purpose of the two assets is to provide a 'deep' drainage outfall for the SP (to reduce fill requirements and allow the SP to be serviced by a convention pit and pipe system). If a bypass is provided, it requires a slope (say 1V:500H). Table C.2 shows that the total length of the combined W_RB_A2 and W_RB_A3 is around 1,700 m. Thus, the invert of the system would have to be approximately 3.4 metres higher than that shown in drawing 2350/CONC/2. This would compromise the main benefit of the system.

Table C.2 Design details for the proposed constructed wetlands

Asset	W_RB_A2	W_RB_A3	Unit
NWL	125.80	124.50	m AHD
ED Depth	0.35	0.35	m
TED	126.15	124.85	m AHD
NWL Area	24,220	26,150	m^2
Volume below NWL	7,275	7,850	m^3
Assumed Detention Time	72	72	hrs
Approx. Length	850	650	m
Minimum width	20	25	m
L:W Ratio	1:43	1:26	

Note: 1. Volume below NWL is estimated assuming the wetlands are on average 0.30 m deep.

A 4EY velocity check (as is required for online wetland systems within the Wetland Design Manual) has been completed for the proposed constructed wetlands as per Table C.3.

Table C.3 4EY wetland velocity checks

Description	Label	W_RB_A2	W_RB_A3	Unit
4EY flow through macrophyte zone	Q_{4EY}	0.35	0.40	m^3/s
1% AEP flow through macrophyte zone	$Q_{1\%AEP}$	5.05	12.85	m^3/s
NWL	NWL	125.80	124.50	m AHD
TED	TED	126.15	124.85	m AHD
10% AEP Level Estimation	FL	126.80	125.50	m AHD
Narrowest Width at NWL	W_{NWL}	20.0	25.0	m
Narrowest Width at TED	W_{TED}	24.2	29.2	m
Narrowest Width at 10% AEP Level	$W_{10\%AEP}$	32	37	m
Flow Area 4EY =	A_{4EY}	7.7	9.5	m^2
1% AEP Flow Area =	$A_{1\%AEP}$	26.0	31.0	m^2
4 EY Flow Velocity = Q/A =	V_{4EY}	0.045	0.042	m/s
Requirement, V_{4EY} <		0.050	0.050	m/s
Is Width Suitable		YES	YES	
1%AEP Flow Velocity = Q/A =	$V_{1\%AEP}$	0.19	0.41	m/s
Requirement, $V_{1\%AEP}$ <		0.50	0.50	m/s
Is Width Suitable		YES	YES	

Note: 1. At this concept stage, the 10% AEP depth is assumed to be 1 m above the NWL.

2 1V:6H batters above NWL assumed.

As per drawing 2350/CONC/1, W_RB_A1 is proposed to be retained in its current natural form (apart from the 450 mm dia connection as per Table B.10). Given this asset is large, and is to be retained, though not a formal constructed wetland, some stormwater treatment benefits have been attributed to it within this strategy.

At a high-level, the 'design' properties of W_RB_A1 are as per Table C.4. These properties have been determined from the existing site characteristics (i.e. the LiDAR Information). The properties in Table

C.4 for W_RB_A1 are indicative only. No changes to the base or form of W_RB_A1 are proposed as part of this strategy. Note that the NWL area just accounts for the area of the existing wetland within the bushland reserve. The area which ponds south of this land has not been included in the modelling.

Table C.4 Assumed properties of W_RB_A1 which is to be retained as is.

Asset	W_RB_A1	Unit
NWL	132.00	m AHD
ED Depth	0.30	m
TED	132.30	m AHD
NWL Area	12,000	m^2
Volume below NWL	3,445	m^3
Assumed Detention Time	24	hrs

C.3 Continuous Simulation Modelling

A Model for Urban Stormwater Improvement Conceptualisation (**MUSIC**), v6.3.0, has been developed to simulate the proposals.

C.3.1 Model Description

Catchments

Subareas and fraction imperviousness used in the MUSIC modelling are generally as per the post-development RORB the model. For quicker simulation times, groups of catchments have been consolidated within MUSIC compared to the RORB model as detailed in Table C.5.

To SWS's knowledge there are not local MUSIC guidelines, or adopted parameter sets. In the absence of any regional parameter sets, the MUSIC Tool Guidelines' sets have been adopted being:

- "Mixed" source node typing has been used to model the pollutants generated from the catchment; and
- Rainfall-Runoff parameters as per the guidelines have been adopted.

Table C.5 MUSIC Catchments

Node ID	Area	Fimp
A-H	308.07	0.15
I-J	64.97	0.30
K	29.42	0.30
L-M	33.66	0.30
N	29.14	0.30
O-R	220.65	0.11
S-U	49.42	0.45
V-W	17.52	0.53
X-AA	65.12	0.34
AB	31.14	0.70
AC	27.49	0.70

It is noted that the majority of the IN1Z land within the catchment has not been included within the model (i.e. catchment G1 from the PDA in Figure 4). Though flows from this catchment are conveyed to the outfall shown in Figure 9, they are largely independent of the majority of the catchment A flows, as they are conveyed in a separate channel to this point. Also, it is understood that separate planning applications, the Kenny Road Permit, has already progressed for the major outstanding development within this catchment.

Climate Data

MUSIC requires climate data (rainfall and evapotranspiration). To SWS's knowledge there are no 'standard' MUSIC climate data sets currently used within or around Horsham. As such, a new set was generated for this project.

The closest rainfall gauge to the study area is the Horsham gauge (079082). This gauge has a long term mean annual rainfall of 426 mm/yr.

The Horsham gauge (079082) has 6-minute rainfall data, but it not always of a usable quality. A visual check was undertaken, and it was found that the 15-year period between the 1/08/1976 and the 1/08/1992 had a reasonable amount of (or more so lack of) missing data and a reasonable amount of accumulated data. As such, this 15-year period has been selected. The mean annual rainfall over this 15-year period is 427 mm/yr.

Evaporation data has been sourced from the Bureau of Meteorology (**BoM**). BoM provides maps of the monthly evaporation across Australia. Using the BoM maps, the seasonal evaporation distribution for Horsham is estimated as per Table C.6 below within the MUSIC modelling undertaken.

Table C.6 Estimated monthly evaporation at Horsham

Month	1	2	3	4	5	6	7	8	9	10	11	12	Annual
Evap (mm)	250	210	175	100	55	35	40	55	80	125	190	210	**1,525**

Treatment Elements

The various treatment elements detailed within Appendix A, and Appendices C.1 and C.2 have been simulated within the MUSIC model.

Hydrologic Routing

No routing has been utilised within the MUSIC modelling undertaken.

Model Schematic

Figure C.2 details the model schematic.

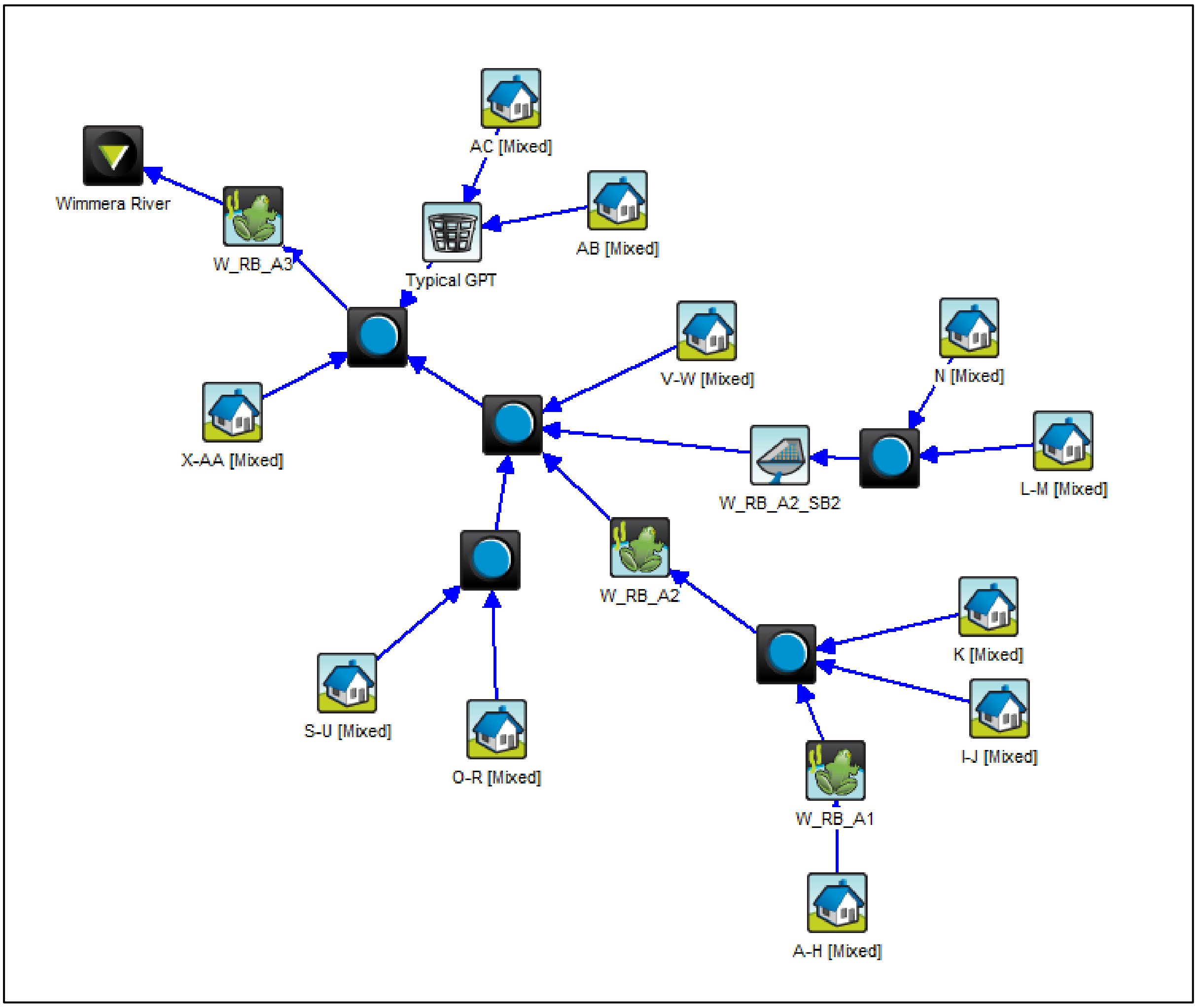

Figure C.2 MUSIC Model Schematic

C.3.2 Model Results – Stormwater Treatment

Table C.7 summarises the overall stormwater treatment performance expected for the catchment and shows that the proposals can achieve the BPEMG targets.

Table C.7 Overall stormwater treatment performance within Catchment A of the SP

Pollutant	Total catchment inflow load (kg/yr)	Total catchment outflow load (kg/yr)	Load retained (kg/yr)	% retention of the SP area	BPEMG Target	Target Met
Total Suspended Solids	152,000	27,900	124,100	81.6%	80.0%	Yes
Total Phosphorus	335	111	224	66.9%	45.0%	Yes
Total Nitrogen	2,530	1,390	1,140	45.1%	45.0%	Yes
Gross Pollutants (Litter)	36,400	0	36,400	100.0%	70.0%	Yes

It should be noted that this treatment performance is for the whole of Catchment A, not just the areas subject to densification under the SP.

Appendix D Hydraulic Design and Modelling

The following Appendix details the key hydraulic calculations utilised in the formulation of the Catchment A concept.

D.1 Concept Trunk Pipe Sizing

The RORB model has been used to estimate that the 20% AEP flows into the W_RB_A2_S1 as 1.65 m^3/s.

This design flow estimate has been utilised to obtain a conceptual sizing of the pipe proposed along Plozzas Road, and parts of the Henty Hwy as shown in 2350/CONC/2 and is conceptually sized in Table D.1.

It is noted that the pipe size will change (likely be reduced due to hydraulic grade line considerations) as the design is further developed with a 12D model or similar. It is also noted that the design herein has not considered any existing services which may impact the design sizing and alignment.

Table D.1 Conceptual Sizing of the Pipe System along Plozzas Road

Capacity Estimate - Manning's			
Number of Pipes	n_o =	2	pipe
Longitudinal Slope =	s =	0.0022	m/m
Mannings n =	n =	0.013	
Pipe Size =	D =	0.90	m
Pipe Radius =	r = D/2 =	0.45	m
Area =	$A = \pi r^2$ =	0.64	m^2
Wetted Perimeter =	$WP = \pi D$ =	2.83	m
Hydraulic Radius =	R = A / WP =	0.23	m
Velocity per pipe=	$V = (R^{2/3} \times s^{0.5}) / n$ =	1.34	m/s
Capacity per pipe=	$Q_{pipe} = A \times V$ =	0.85	m^3/s
Total System Capacity =	$Q_{total} = n_o \times Q_{pipe}$ =	1.71	m^3/s

D.2 Future IN1Z Waterway

As shown in 2350/CONC/1, the IN1Z land south of Plumpton Road which is currently farmland receives inflows from the already developed IN1Z land to the east and the W_RB_G1 catchment.

The RORB model has been used to estimate that the 1% AEP flows along this alignment is 7.5 m^3/s (assuming no regional retardation assets on (or around) 55 Kenny Road).

Currently there is an informal drain along this alignment which does not have 1% AEP capacity. Given this, it is proposed to upgrade the drain to provide it with a 1% AEP capacity. The typical cross section assumed is as per Table D.2 and 2350/CONC/5 (assuming the existing natural surface grade of approx. 1V:625H)

It is noted that the design of this waterway will be required to be refined going forward to incorporate linear pools which will assist in 'steepening' the grade. At this stage, assuming a 1% AEP depth of 1 m results in the flood level estimate being below the surrounding natural surface level. However, prior to development of the IN1Z land south of Plumpton Road, the 1% AEP estimates should be confirmed to ensure appropriate fill and freeboard provisions can be achieved.

Table D.2 Conceptual Sizing of the Waterway along the north of the IN1Z land south of Plumpton Road

Capacity Estimate - Manning's			
Longitudinal Slope =	s =	0.0016	m/m
Mannings n	n =	0.05	
Base Width =	W =	7.00	m
Water Depth =	D =	1	m
Side Slopes =	SS = 1V: x H =	5	
Top Width =	TW = W + SS × 2 × D =	17.00	m
Area =	A = D × (SS × D + W) =	12.00	m^2
Wetted Perimeter =	$WP = W + 2\times((SS\times D)^2 + D^2)^{0.5}$ =	17.20	m
Hydraulic Radius =	R = A / WP =	0.70	m
Velocity =	$V = (R^{2/3} \times s^{0.5}) / n$ =	0.63	m/s
Capacity =	Q = A × V =	7.55	m^3/s

Using the sizing from Table D.2, a 32 m wide drainage reserve is proposed as shown in 2350/CONC/5.

It is noted that the 40m wide reservation is a MWC standard. Council at their discretion Hve requested a smaller reserve given their local setting.