



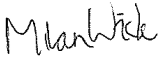
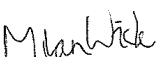
Moir Shire Council Cobram South Growth Area

Drainage Strategy Review

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1. INTRODUCTION

1.1 Background

Engeny has been engaged by Moira Shire Council to undertake a review and analysis of the existing and future development scenarios for the Cobram South Growth Area Drainage Scheme (GADS).

The existing residential development at the Cobram South GADS is serviced by three retarding basins (RB 1, RB 2 and RB 3), with an estimated total catchment area (combined residential and undeveloped land) of approximately 212 hectares to the drainage outlet to Scotts Creek, which leads to the Murray River. Figure 1.1 below shows the existing site conditions, retarding basins and estimated catchment area to the drainage outlet.

Due to the flat terrain within the catchment area, RB 1 and RB 3 have been constructed with pumped outlets, both discharging to 600 mm diameter rising mains. RB 2 has a gravity outlet connection to RB 3. The rising mains and gravity drainage pipe alignments are shown in Figure 1.1.

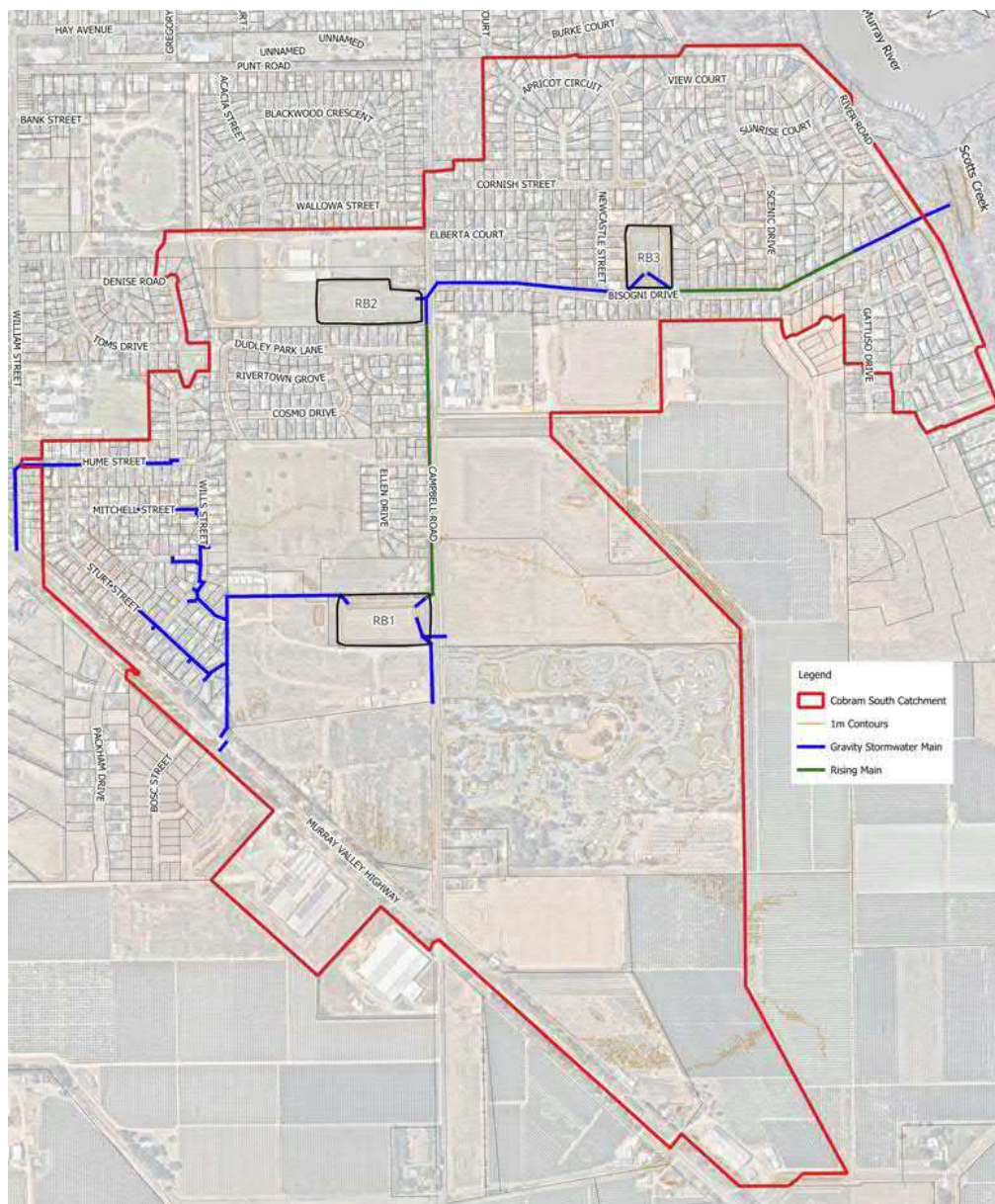


FIGURE 1.1: STUDY AREA

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Future development is proposed within the Cobram South GADS, as shown in Figure 1.2. It is expected that future development will be predominantly standard density residential development. Future development will increase catchment imperviousness and increase runoff rates and volumes in storm events. This report documents Engeny's investigations of additional flood storages (provided through new retarding basins) and other measures (such as an increased pump station capacities at the RB 1 and RB 3 outlets) to cater for the future development and manage the impact of development on flooding in existing development areas.

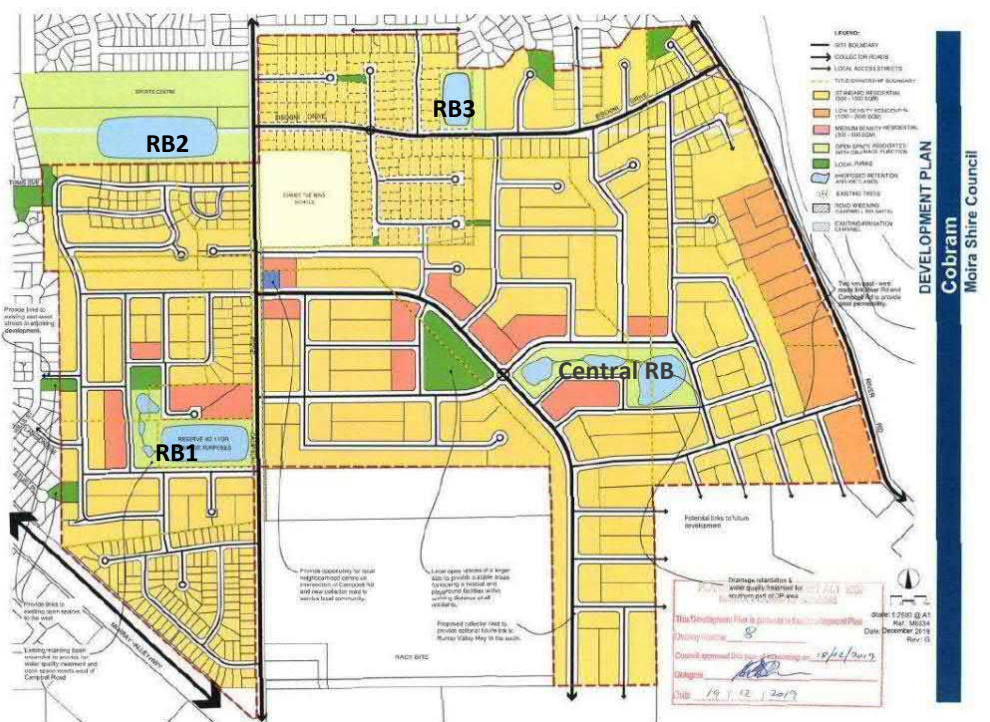


FIGURE 1.2: COBRAM SOUTH FUTURE DEVELOPMENT PLAN

1.2 Objectives

The objectives of this study are to:

- Analyse the performance of the existing drainage system in the Cobram South GADS for the existing development conditions, in the 5% annual exceedance probability (AEP) and 1% AEP flood events.
- Analyse the impact of future development on the performance of the existing drainage system.
- Estimate the requirements for the new distributed retarding basins that are proposed to service future developments.
- Determine potential upgrades to existing drainage infrastructure (pumps, rising mains and gravity drainage systems) that may be required so that the existing drainage system can accommodate increased runoff from future development.
- Determine the stormwater quality treatment asset sizes required to treat the new development areas to Best Practice Environmental Management Guidelines (Best Practice).
- The ultimate purpose of Engeny's study is to recommend a suitable drainage strategy to service the existing and future development at the Cobram South GADS.

1.3 Methodology

The following key tasks were undertaken as part of Engeny's study to inform the project objectives:

- Review of the N.M Craigie & Associates (1989) drainage strategy.
- Development of an existing conditions hydrological (RORB) model in line with Australian Rainfall & Runoff (ARR) 2019 guidance and the Melbourne Water Flood Mapping Project Specifications (August 2023).
- Development of an existing conditions TUFLOW hydraulic model which had been simulated for the 5% AEP and 1% AEP flood events.

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- Development of a post-developed conditions RORB model, to reflect the proposed land use once the Cobram South GADS is fully developed.
- Sizing of stormwater treatment assets (sediment basins and wetlands) for future developments in the Cobram South GADS using Fair & Geyer calculations and MUSIC modelling.
- Development of a post-developed conditions TUFLOW model with inclusion of new distributed retarding basins to service future development areas.
- Determine pump, rising main and underground pipe upgrades required from the outlet of the existing retarding basins so that flooding in the developed conditions is no worse than existing conditions and sufficient freeboard is provided to adjacent lots.

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2. INFORMATION REVIEW

2.1 Existing Drainage System

The following data was provided by Council, which Engeny has either incorporated directly into the flood modelling, or used to inform modelling decisions:

- AutoCAD drawing file *ACAD-10673_C3D_01*. This file provides details of the existing Council trunk drainage at Campbell Road and Bisogni Drive. It was also used to identify the locations of existing pumps at RB1 and RB3, and identify which pipes are rising mains and gravity mains.
- GIS layers for existing pipes and pits within the Cobram South catchment (October 2023). The GIS layers were used for informing catchment delineation in RORB.
- Existing pump information for the RB1 and RB3 outlets. Table 2.1 shows the RB1 and RB3 pumps start levels which have been modelled in the existing conditions TUFLOW modelling.

TABLE 2.1: RB1 AND RB3 PUMP START LEVEL DETAILS

	RB1 (Campbell Road)	RB3 (Bisogni Drive)
Pump 1 Start RL	110.6257	110.664
Pump 2 Start RL	111.0757	111.115
Pump 3 Start RL	111.5257	111.415

- Pump Specifications (*N3201, Flygt Industries*) Data Sheet. The pump curve in these specifications has been represented in the existing conditions TUFLOW model. In the absence of any further information, it has been assumed that the same pump is provided for Pumps 1-3 in both RB1 and RB3. The pump curve discharge head (m) – flow (L/s) relationship is shown in Figure 2.1.

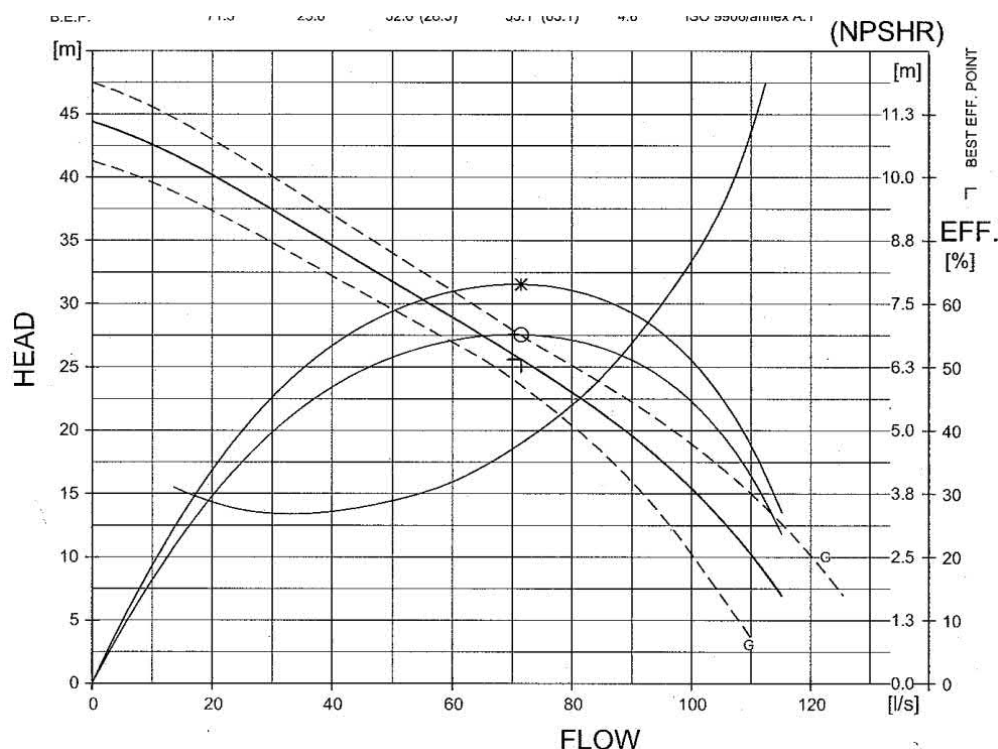


FIGURE 2.1: PUMP CURVE

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2.2 Previous Drainage Strategies

2.2.1 N.M Craigie & Associates (1989)

Engeny has reviewed the drainage strategy prepared by N.M Craigie & Associates (1989), which recommended:

- The existing retarding basins 1-3 to function as one storage system with the same retarding basin base level and design top water level.
- For the discharge from the three retarding basins to be controlled by a pump station at RB 3 only, with a pump station capacity of 0.5 m³/s.

The sizing of the retarding basins was undertaken N.M Craigie & Associates based on the following methodology:

- Retarding basins sized in RORB, for a 1 in 20-year (5% AEP) event capacity.
- Retarding basins sized based Australian Rainfall & Runoff (ARR) 1987 methodology (current at the time of the N.M Craigie & Associates study, but now superseded by ARR 2019).
- An imperviousness of 30% for rural / residential developments and 40% for proposed urban areas within the catchment area discharging to retarding basins 1 – 3.

The layout plan from the N.M Craigie & Associates (1989) strategy is shown in Figure 2.2.

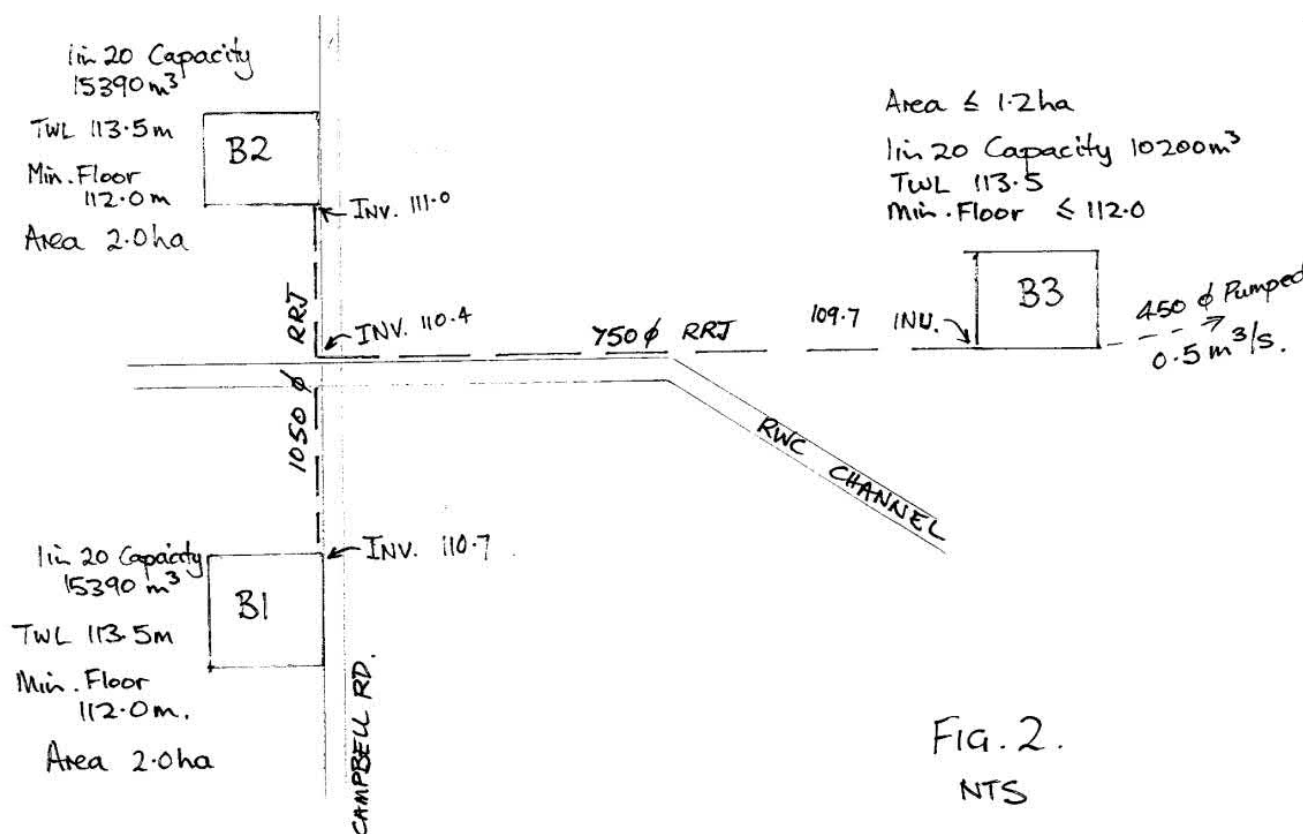


FIG. 2.
NTS

FIGURE 2.2: RETARDING BASINS LAYOUT PLAN (N.M CRAIGIE & ASSOCIATES, 1989)

Engeny's review of this drainage strategy has identified the following key details:

- The actual imperviousness within the existing development in the Cobram South GADS is higher than the imperviousness assumed in the N.M Craigie & Associates (1989) strategy. This has implications on peak flow and runoff volume discharging to the retarding basins. In Engeny's 2024 drainage strategy, the catchment imperviousness has been updated, as detailed in Section 3.2.2.
- The retarding basins have been constructed differently to that proposed in the N.M Craigie & Associates (1989) strategy. Both retarding basins 1 and 3 have been constructed with pumped outlets, whereas the drainage strategy has proposed for just RB 3 to have a pumped outlet.

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- Modelling of the retarding basins in the existing conditions and future development conditions can be more accurately undertaken in a hydraulic modelling package such as TUFLOW. As RB 1 and RB 3 have both been constructed with pumped outlets, the retarding basins do not function as one storage system. Factors such as timing of flow into and out of the retarding basins can be better represented in TUFLOW and provide different results compared to RORB hydrological modelling (as adopted by N.M Craigie & Associates).

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3. HYDROLOGICAL MODELLING

3.1 Model Overview

Engeny has developed a RORB hydrological model for the Cobram South GADS. Figure 3.1 provides a layout plan of the RORB model, showing the catchment and subarea boundaries.

RORB modelling was undertaken for the following scenarios:

- Existing conditions: to represent the catchment's current development conditions.
- Developed conditions: to represent the future developed conditions to account for future residential growth areas within the study area, in accordance with the Cobram South Future Development Plan in Figure 1.2.

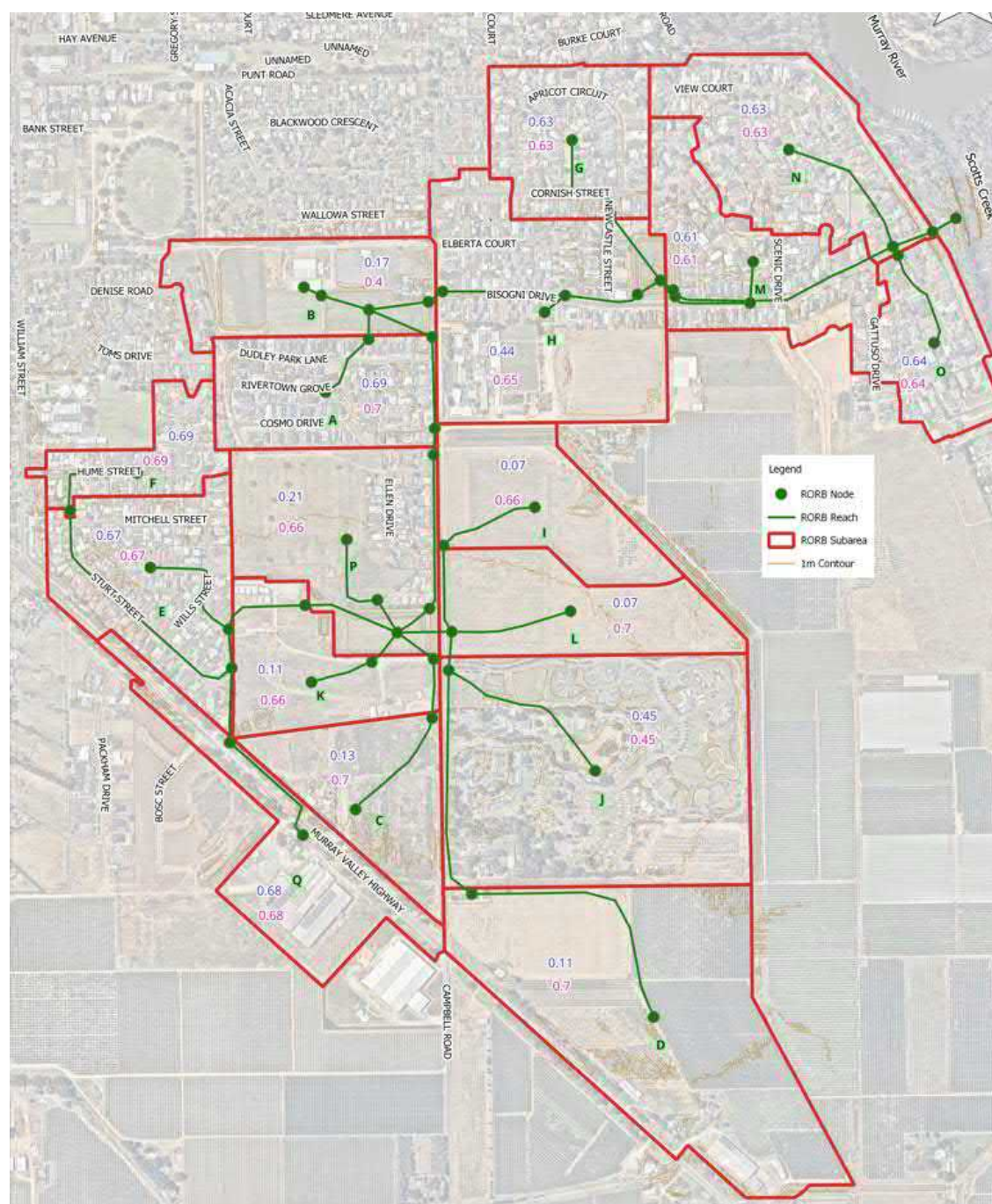


FIGURE 3.1: RORB MODEL LAYOUT PLAN

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3.2 Model Setup

3.2.1 Catchment Delineation

The RORB model catchment delineation was undertaken with consideration of the following:

- Council provided LIDAR (capture date 29 March 2022).
- Existing pipe alignments, from the drainage GIS layers provided by Council.
- Property boundaries.

3.2.2 Fraction Impervious

The RORB modelling was undertaken using the Effective Impervious Areas (EIA) and Indirectly Connected Areas (ICA) approach to model fraction imperviousness. The EIA / ICA modelling approach is in accordance with the recommendations in ARR 2019 Book 5 and guidance in the Melbourne Water Technical Specifications (August 2023). Figure 3.1 shows the RORB existing condition imperviousness in blue text and developed condition imperviousness in magenta text.

3.2.2.1 Existing Condition Fraction Impervious

For the existing conditions RORB modelling, the fraction impervious value adopted for each sub-catchment has been calculated by assigning a fraction impervious based on residential lot sizes or other land use type. These values are based on a combination of the values indicated in the Melbourne Water MUSIC Guidelines (2024) and assessing the imperviousness of typical lots within the Cobram South GADS.

Table 3.1 summarises the fraction imperviousness implemented for each lot size range:

TABLE 3.1: EXISTING CONDITIONS FRACTION IMPERVIOUSNESS BY LAND TYPE

Lot Size Range or Land Type (m ²)	Total Fraction Impervious Value
Residential 300 - 500	0.80
Residential 500 - 600	0.75
Residential 600 - 800	0.70
Residential 800 - 1000	0.60
Residential 1000 - 2000	0.60
Road Reserves	0.70
Open Space Areas	0.05 - 0.10

3.2.2.2 Developed Conditions Fraction Impervious

Table 3.2 shows the fraction impervious values that have been assigned for future development areas shown in the Cobram South development plan. Sub-catchment D is not indicated for future development in the development plan, however, following discussions with Council, it has been modelled with a fraction imperviousness of 0.70 to represent future standard residential development.

In addition, the existing condition imperviousness modelled in the RACV site (sub-catchment J), was retained in the developed conditions modelling, as discussed with Council. If further development of this site is proposed in the future, the additional runoff is required to be managed within the development prior to discharge to Council's drainage system.

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TABLE 3.2: POST-DEVELOPED CONDITIONS FRACTION IMPERVIOUSNESS BY LAND TYPE

Land Type	Fraction Impervious Value
Standard Density Residential (600-800 m ² lots)	0.70
Medium Density Residential (300-500 m ² lots)	0.80
Local Parks	0.05 – 0.10

3.3 Model Validation and Parameters

3.3.1 Initial and Continuous Loss values

Table 3.3 indicates the initial and continuous loss values that have been adopted in the existing and developed conditions RORB modelling.

TABLE 3.3: RORB MODEL INITIAL AND CONTINUOUS LOSS VALUES

RORB Surface Type	Initial Loss (mm)	Continuous Loss (mm/h)	Source
Pervious / Rural Areas	23	2.5	ARR Data Hub
Effective Impervious Areas	1.5	0	ARR 2019 Book 5 Guidance
Indirectly Connected Impervious Areas	16.1	1.75	ARR 2019 Book 5 Guidance (applied as 70% of the pervious area loss values)

3.3.2 RORB Routing Parameter Kc

The RORB model routing parameter – Kc, was selected by identifying the relevant regional equation that produced the best match of the developed conditions RORB model at the RORB model outlet to urban peak flow estimates from the Nikolaou/von't Steen equation, and Rule of Thumb flow estimates.

In this catchment, the applicable regional equations to calculate Kc are shown in Table 3.4.

TABLE 3.4: APPLICABLE REGIONAL EQUATIONS FOR RORB ROUTING PARAMETER KC

No	Regional Equation	Application	Source
1	$k_c = 0.49 \times A^{0.65}$	Areas with annual rainfall < 800 mm	ARR2019 Book 7, Chapter 6.2.1.3
2	$k_c = 2.2 \times A^{0.5}$	General	RORB V6 User Manual Equation 2-5
3	$k_c = 1.25 \times d_{av}$	Victoria	Pearse et al. (2002)

The RORB Equation Kc (Kc = 3.20) produced the best match to the urban flow estimates and was therefore adopted for the existing and developed conditions RORB modelling.

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3.3.3 Pre-Burst Rainfall

Pre-burst rainfall has been incorporated into the RORB model by prepending the pre-burst storm depths to the burst storm, thereby modelling the complete storm, in accordance with the guidance in ARR 2019 Book 5. As the subject site is located within ARR Loss Region 3, the 75th percentile pre-burst rainfall depths were modelled. The default (GSAM and Jordan et al) RORB pre-burst temporal patterns were modelled in RORB.

3.3.4 Aerial Reduction Factors

The Areal Reduction Factors (ARFs) modelled in RORB use the ARFs from the ARR Data Hub text file.

3.4 RORB Sub-Catchment Details

The RORB sub-catchment details are shown in Table 3.5. Sub-catchments C, D, H, I, K, L and P have been modelled with a higher imperviousness in the developed conditions to reflect the proposed future land use in these catchments.

For sub-catchments that are predominantly developed, the EIA has been assumed to be 60% of the total fraction impervious, with the remaining area assigned as ICA. For catchments that are predominantly undeveloped or open space, the EIA has been assumed to be zero, with the total impervious area assigned as ICA and the remaining area assigned as rural.

TABLE 3.5: RORB SUB-CATCHMENT DETAILS

Sub-Catchment	Area (ha)	Existing Condition Total Fraction Impervious	Existing Condition EIA / ICA / Rural Portion	Developed Condition Overall Fraction Impervious	Developed Condition EIA / ICA / Rural Portion
A	9.7	0.69	EIA: 0.42 ICA: 0.58	0.70	EIA: 0.42 ICA: 0.58
B	10	0.17	EIA: 0 ICA: 0.17 Rural: 0.83	0.40	EIA: 0.24 ICA: 0.16 Rural: 0.60
C	8.7	0.13	EIA: 0 ICA: 0.13 Rural: 0.87	0.70	EIA: 0.42 ICA: 0.58 Rural: 0
D	27.4	0.11	EIA: 0 ICA: 0.11 Rural: 0.89	0.70	EIA: 0.42 ICA: 0.58 Rural: 0
E	12	0.67	EIA: 0.40 ICA: 0.60 Rural: 0	0.67	EIA: 0.40 ICA: 0.60 Rural: 0
F	4.9	0.69	EIA: 0.41 ICA: 0.59 Rural: 0	0.69	EIA: 0.41 ICA: 0.59 Rural: 0
G	9.2	0.63	EIA: 0.38 ICA: 0.62 Rural: 0	0.63	EIA: 0.38 ICA: 0.62 Rural: 0
H	19.5	0.44	EIA: 0.27	0.65	EIA: 0.40

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Sub-Catchment	Area (ha)	Existing Condition Total Fraction Impervious	Existing Condition EIA / ICA / Rural Portion	Developed Condition Overall Fraction Impervious	Developed Condition EIA / ICA / Rural Portion
			ICA: 0.73 Rural: 0		ICA: 0.60 Rural: 0
I	9.3	0.07	EIA: 0 ICA: 0.07 Rural: 0.93	0.66	EIA: 0.41 ICA: 0.59 Rural: 0
J	27.6	0.45	EIA: 0.27 ICA: 0.73 Rural: 0	0.45	EIA: 0.27 ICA: 0.73 Rural: 0
K	8.2	0.11	EIA: 0 ICA: 0.11 Rural: 0.89	0.66	EIA: 0.41 ICA: 0.59 Rural: 0
L	9.7	0.07	EIA: 0 ICA: 0.07 Rural: 0.93	0.70	EIA: 0.42 ICA: 0.58 Rural: 0
M	11.3	0.61	EIA: 0.37 ICA: 0.63 Rural: 0	0.61	EIA: 0.37 ICA: 0.63 Rural: 0
N	13.3	0.63	EIA: 0.38 ICA: 0.62 Rural: 0	0.63	EIA: 0.38 ICA: 0.62 Rural: 0
O	6.6	0.64	EIA: 0.38 ICA: 0.62 Rural: 0	0.64	EIA: 0.38 ICA: 0.62 Rural: 0
P	13.9	0.21	EIA: 0.13 ICA: 0.09 Rural: 0.79	0.66	EIA: 0.40 ICA: 0.60 Rural: 0
Q	10.4	0.68	EIA: 0.40 ICA: 0.60 Rural: 0	0.68	EIA: 0.40 ICA: 0.60 Rural: 0

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4. STORMWATER QUALITY MODELLING

4.1 Policy and Reduction Targets

The State Environment Protection Policy (SEPP) defines the required water quality conditions for urban waterways. Clause 56.07-4 of the Victorian Planning Provisions (and the clause 53.18 extension of these requirements to non-residential development) sets the stormwater treatment targets required for development in Victoria to comply with SEPP and the Planning Scheme. The Urban Stormwater – Best Practice Environmental Management (BPEM) Guidelines (Victorian Stormwater Committee, 1999) water quality targets are specified by the Victorian Planning Provisions as achieving compliance with the SEPP. The BPEM targets are presented in Table 4.1

TABLE 4.1: BEST PRACTICE REDUCTION TARGETS

Pollutant	Performance Objective
Total Suspended Solids (TSS)	80% reduction from typical urban load
Total Phosphorus (TP)	45% reduction from typical urban load
Total Nitrogen (TN)	45% reduction from typical urban load
Gross Pollutants (GP)	70% reduction from typical urban load

4.2 Sediment Pond and Wetland Sizing

Engeny's understanding is that there are no existing stormwater quality assets within the Cobram South GADS. Stormwater treatment sediment ponds and wetlands are proposed to treat future development areas to BPEM Guidelines.

Sediment ponds were sized using Fair & Geyer calculations, in accordance with the Melbourne Water Deemed to Comply requirements. The sediment ponds are to:

- Capture 95% of coarse particles $\geq 125 \mu\text{m}$ diameter for the peak three-month ARI.
- Provide adequate sediment storage volume to store five years of sediment, with the top of the sediment accumulation zone being 500 mm below the normal water level (NWL).

Model for Urban Stormwater Improvement Conceptualisation (MUSIC) water quality modelling software was used to assess the performance of the various stormwater quality management systems proposed for the future development areas. The program utilises user-defined inputs to represent the upstream catchment (areas and fraction impervious) draining into the given stormwater quality treatment asset(s) during a representative 10-year rainfall and evapotranspiration data set. As an output, the program reports on the pollutant load reduction and other node water balance values.

The following summarises the MUSIC model inputs:

- 10-year rainfall data set (1/01/1990 – 31/12/1999) applicable to Cobram (Goulburn Murray Rainfall Station – 080109)
- Monthly aerial potential evapotranspiration (PET)
- 6-minute time steps
- Soil store capacity: 120 mm
- Field capacity: 50 mm

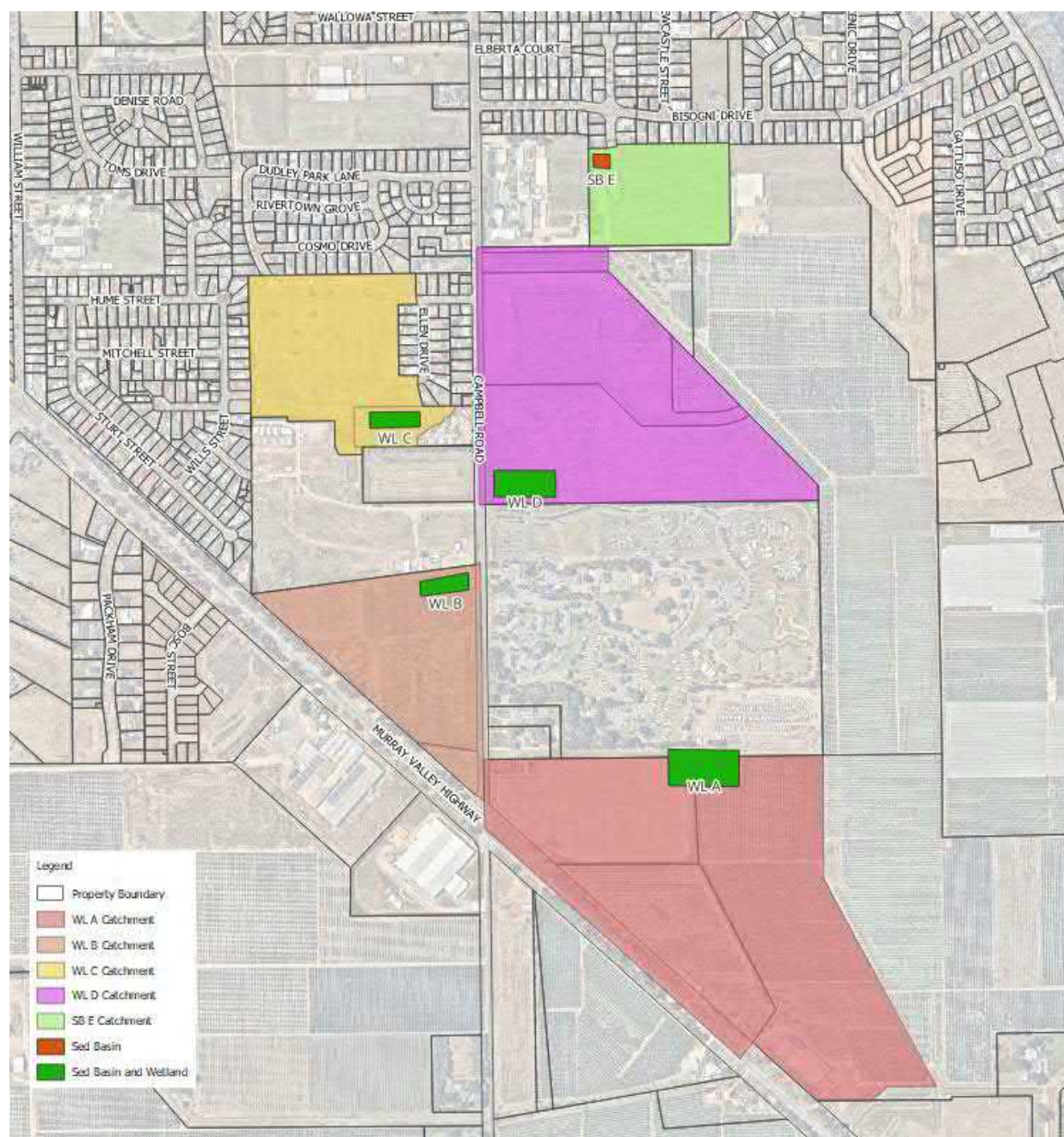
The sediment basin and wetland asset sizes computed from the Fair & Geyer calculations and the MUSIC modelling are shown in Table 4.2. The WSUD layout plan, showing the catchments that discharge to each asset is shown in Figure 4.1.

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TABLE 4.2: SEDIMENT BASIN AND WETLAND ASSET SIZES

Treatment Asset	Sediment Basin NWL Area (m ²)	Wetland NWL Area (m ²)
A	810	5000
B	500	1250
C	500	1250
D	640	3000
E	500	Not Provided


FIGURE 4.1: WSUD LAYOUT PLAN

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4.3 Treatment Results

MUSIC modelling results for the future development areas are shown in Table 4.3, which shows that Best Practice water quality targets are achieved for the future development areas.

TABLE 4.3: MUSIC MODEL RESULTS

	Source Volume (kg/yr)	Volume Removed (kg/yr)	MUSIC Model % Reduction	BPEMG Target (%)
Total Suspended Solids (TSS)	34,400	28670	83.5	80
Total Phosphorus (TP)	69	49	70.8	45
Total Nitrogen (TN)	479	221	46.1	45
Gross Pollutants	7200	6962	96.7	70

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5. EXISTING CONDITIONS HYDRAULIC MODELLING

5.1 Model Overview

The existing conditions hydraulic (TUFLOW) model has been set up in accordance with the following:

- The model has been run utilising TUFLOW HPC (Build: 2023-03-AC-iSP-w64).
- The model's grid orientation was rotated to align parallel to Mitchell Street, to align with key flow paths in the study area.
- A three-metre cell size was adopted to provide suitable resolution of overland flow paths while also providing reasonable model simulation times.
- Rainfall excess hydrographs from RORB were applied to the surface evenly as 2d_SA_ALLs and rainfall hydrographs were applied directly into pits in the western developed areas and in the north of the modelling extent using 1d-BC polygons.
- Rising mains and drainage pipes were modelled using the Council provided drainage data.
- A water level vs time (HT) downstream boundary condition has been applied at the downstream end of the model extent at the outlet to Scotts Creek and at the end of the model extent at Williams Street. In the absence of further information, the HT boundary water level was set at the estimated top of bank of Scotts Creek, at RL 111.2 m AHD, based on the provided LIDAR terrain. Flood modelling of Murray River and Scotts Creek has not been included in this project. Engeny's understanding is that riverine flooding does not impact the development areas, but this should be confirmed with North East CMA.
- Pumps were modelled at Campbell Road (RB1) and Bisogni drive (RB3) with provided pump curves and pump operating levels from council (as discussed in Section 2). Based on the pump rates indicated in the pump curve specifications, the maximum pump outflow rate from retarding basins 1 and 3 when all pumps are operating is approximately 0.35 m³/s. This is less than the 0.5 m³/s pump rate proposed for RB3 in the 1989 Craigie strategy.
- A front, middle and back loaded temporal pattern was run for storm events up to the 24-hour duration. The median temporal pattern of each event was identified, and then the maximum of each of the median events was selected for analysis.

The TUFLOW model setup for the existing conditions is shown in Figure 5.1. The Manning's 'n' roughness values that apply to overland flows are shown in Figure 5.2, which are in accordance with the values indicated in the Melbourne Water Flood Mapping Project Specifications (August 2023).

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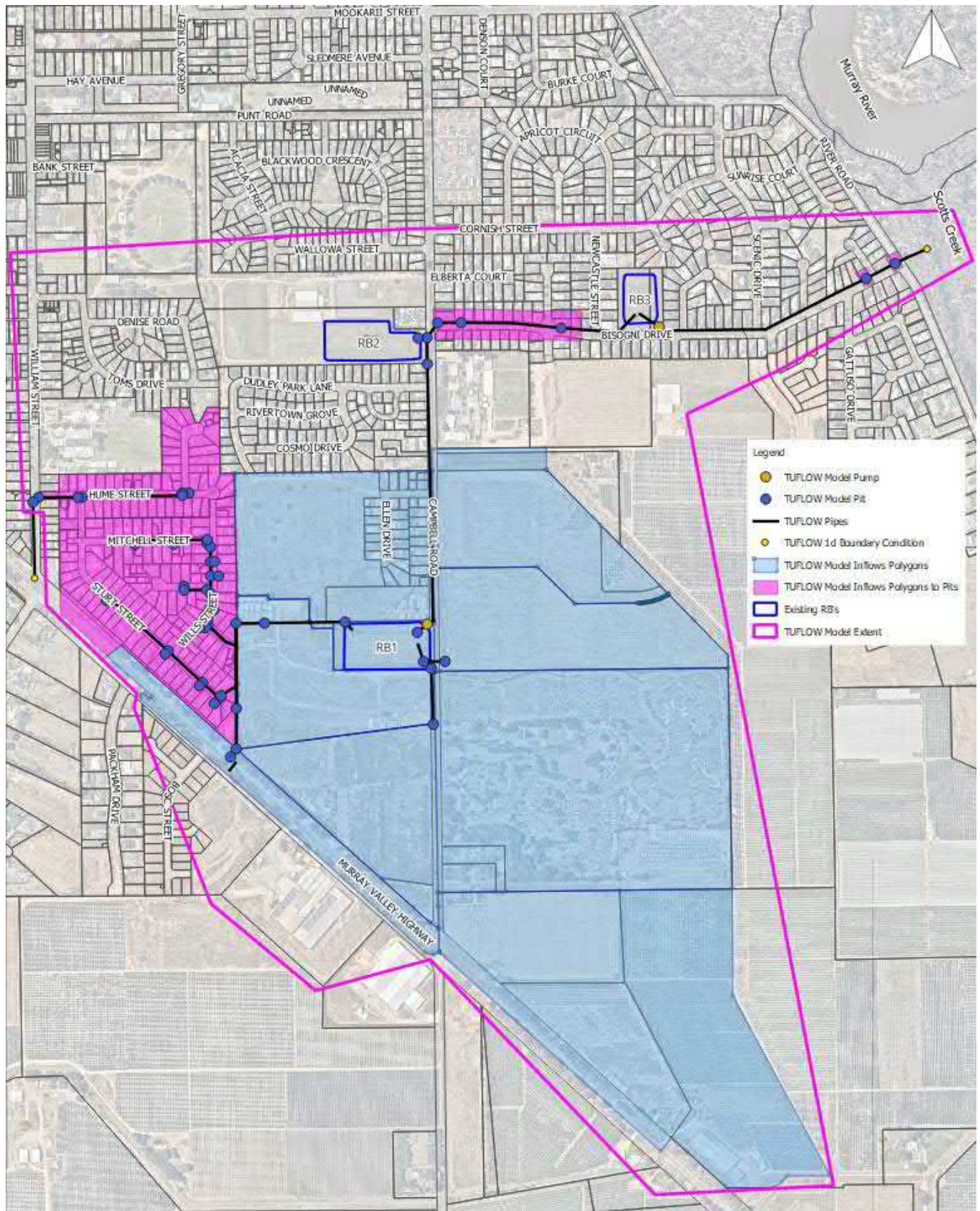


FIGURE 5.1: EXISTING CONDITIONS TUFLOW MODEL SETUP

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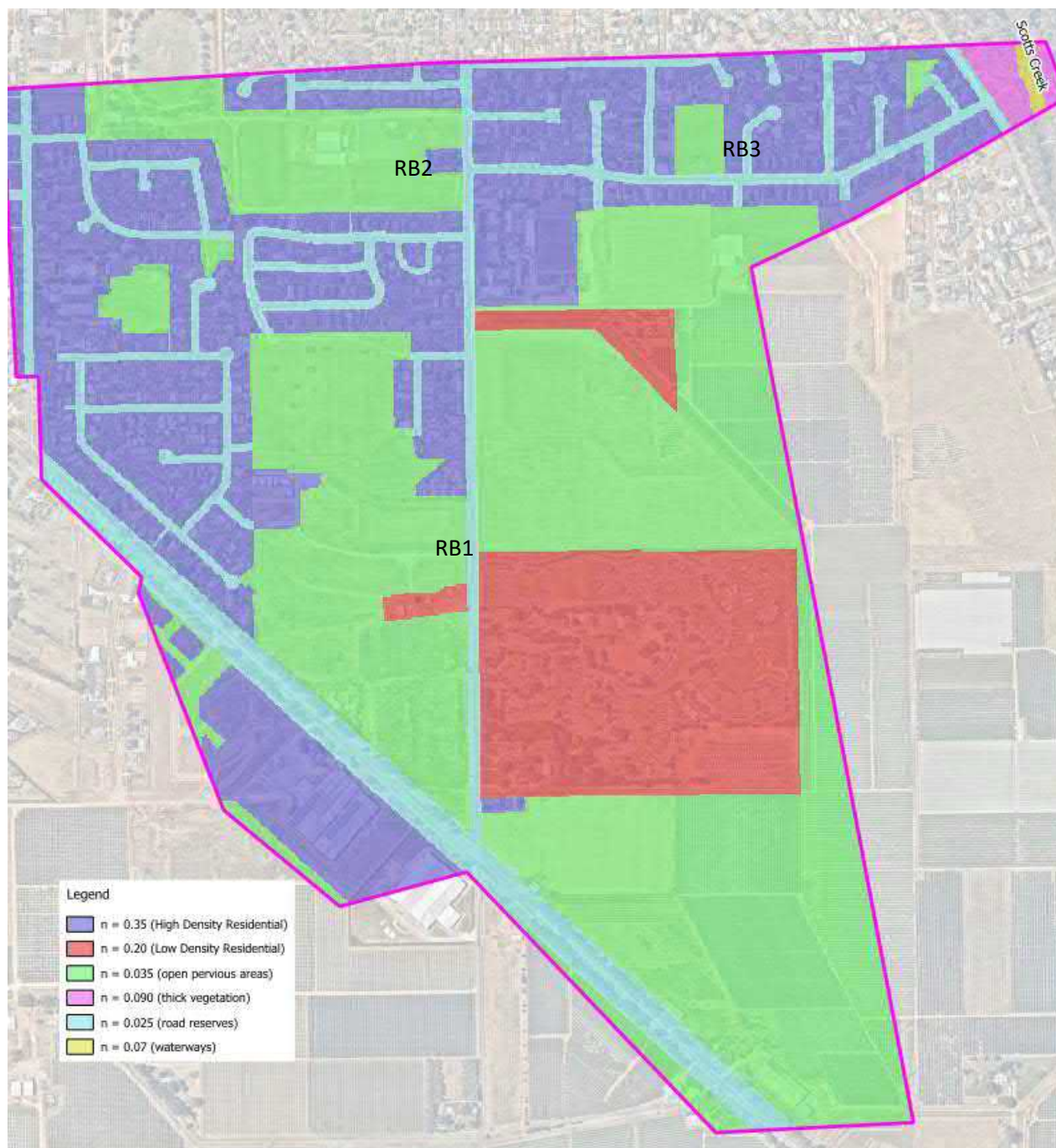


FIGURE 5.2: MANNINGS MATERIALS

5.2 Existing Condition Flood Results

Existing conditions flood modelling was undertaken for the 5% AEP and 1% AEP events, with the model's predicted flood depths shown in Figure 5.3 and Figure 5.4 respectively. The full-scale 5% AEP and 1% AEP flood depths and flood hazard for the existing conditions are shown in Appendix A.

Freeboard for RB 1 and RB 3 has been assessed from the retaining basin top water level (TWL) to the lowest surface level along the perimeter of the retaining basin. For RB 2, freeboard has been assessed from the retaining basin TWL to the lowest surface level that LIDAR has picked up at building footprints in adjacent properties, which is approximately RL 113.45. This is because RB 2 is a shallow retaining basin, and subsequent developed conditions TUFLOW iterations have demonstrated that it is challenging to contain flood depths to the grass area of the retaining basin, without encroaching onto Dudley Park Lane.

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Table 5.1 indicates the existing conditions flood results, which shows:

- In the 5% AEP flood event, freeboard is achieved for RB1 and RB3. RB2 is overtopping to adjacent properties.
- In the 1% AEP flood event, only RB3 has freeboard. RB1 and RB2 are overtopping into adjacent areas.
- 1% AEP flood depths at the low point on Mitchell Street are approximately 500 mm, with a H3 hazard rating, making flood conditions in this area unsuitable for vehicles, children and the elderly.
- 1% AEP flood depths at the low point on Dudley Park Lane are approximately 800 mm, with a H3 hazard rating, making flood conditions in this area unsuitable for vehicles, children and the elderly.

The key outcome of the flood modelling of existing conditions is that the existing drainage system is predicted to have insufficient capacity to cater for 5% AEP or 1% AEP storm events.

TABLE 5.1: EXISTING CONDITIONS FLOOD RESULTS

	5% AEP Top Water Level (m AHD)	5% AEP Freeboard (mm)	1% AEP Top Water Level (m AHD)	1% AEP Freeboard (mm)
Retarding Basin 1	112.94	360	113.41	-110 (retarding basin is overtopping to the south)
Retarding Basin 2	113.66	-210 (retarding basin is overtopping to the south)	113.82	-370 (retarding basin is overtopping to the south)
Retarding Basin 3	113.72	980	114.24	460

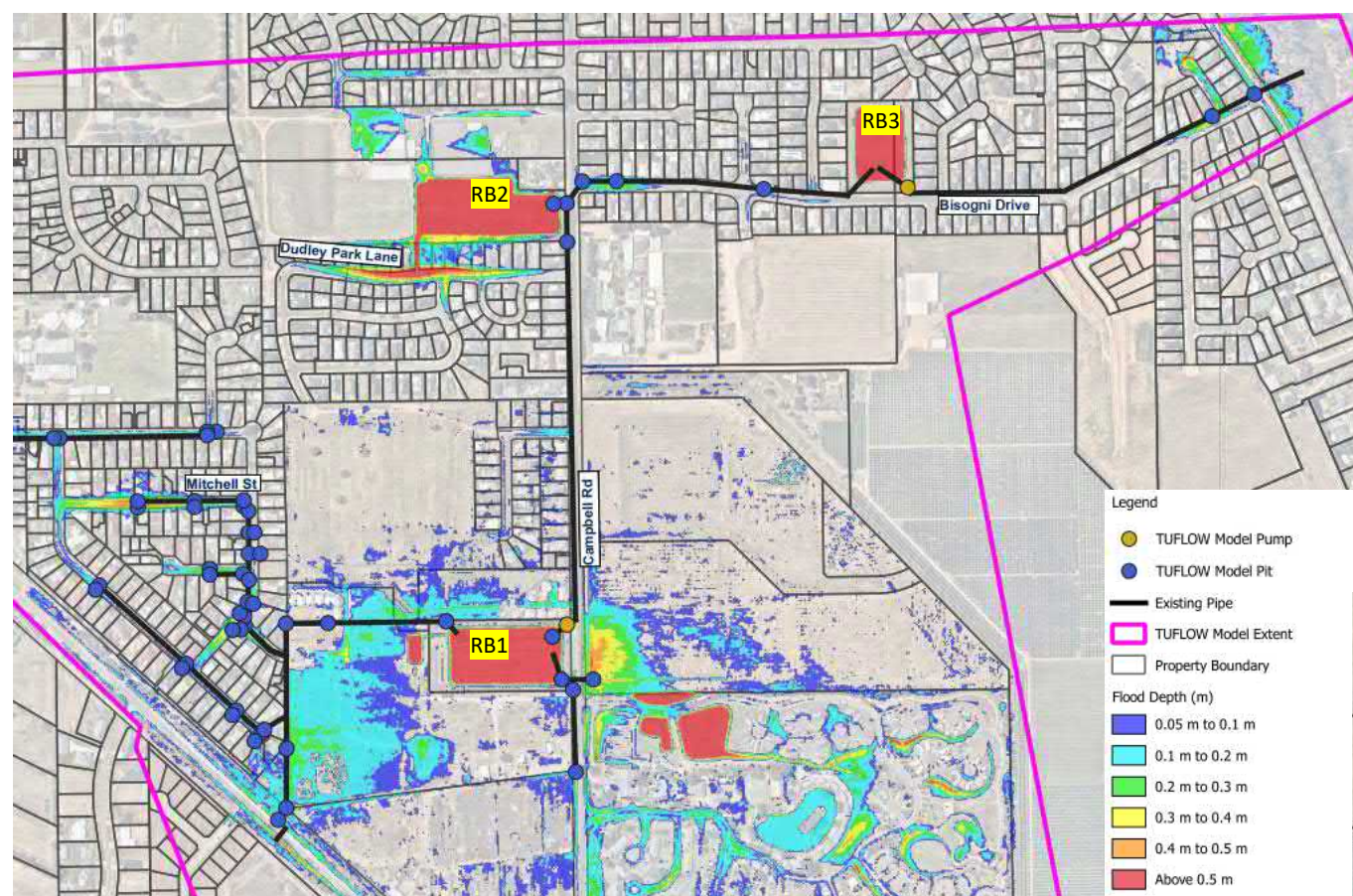


FIGURE 5.3: EXISTING CONDITIONS 5% AEP FLOOD DEPTHS

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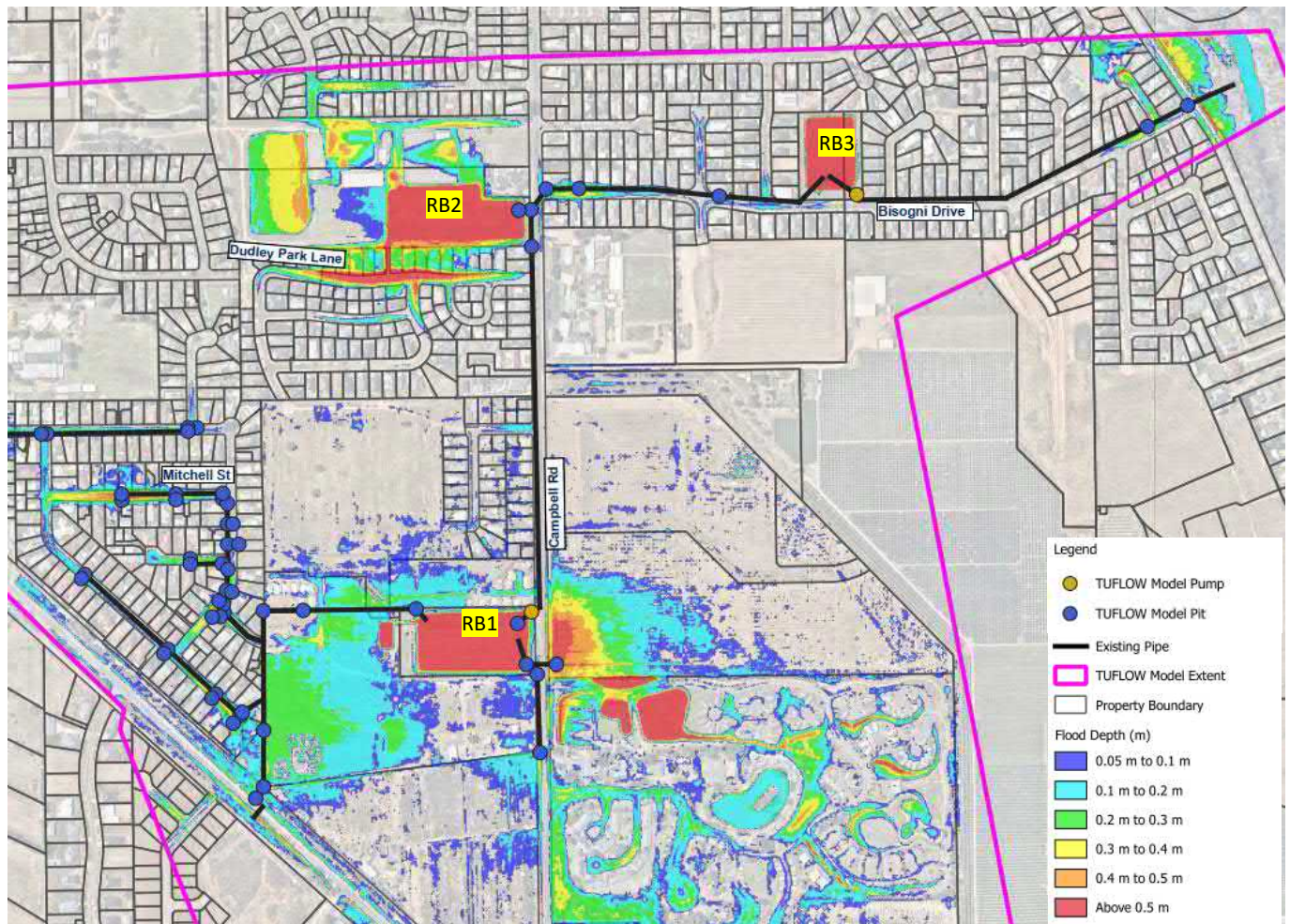


FIGURE 5.4: EXISTING CONDITIONS 1% AEP FLOOD DEPTHS

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6. DEVELOPED CONDITIONS HYDRAULIC MODELLING

6.1 Objectives

The primary objectives of the flood modelling of developed conditions are to:

- Determine flood impacts on the existing retarding basins 1 – 3, when the Cobram South GADS is fully developed and the runoff from the future development drains into the existing drainage system.
- Determine sizing requirements for new distributed Retarding Basins A-D that are proposed to manage runoff from the future development areas prior to runoff discharging into the existing drainage system.
- Determine terrain modifications, pump and outlet upgrades required at the existing retarding basins to alleviate flooding and provide sufficient freeboard.
- Ultimately, based on the optioneering work undertaken, to provide a recommendation to Council for the preferred drainage strategy to be implemented for the Cobram South GADS.

6.2 Overview of Scenarios

Developed conditions TUFLOW modelling has been undertaken for a range of scenarios to investigate potential drainage works to cater for future development and improve the performance of the existing drainage system. Subsequent sections of this report provide details of the investigations undertaken for the various drainage works scenarios. The scenarios that have been investigated are:

- **Developed do-nothing scenario:** in this scenario, the new distributed retarding basins A-D for future development areas discharge into RB1, with no upgrade works modelled on the existing drainage system. This scenario estimates the potential adverse impacts on the existing drainage system if future development occurred without upgrades to the existing drainage system. The base areas of the retarding basins modelled in the 'developed do-nothing' TUFLOW model were sized by applying a 30% increase to the total treatment footprint (sediment pond, wetland and sediment dryout area). A flood storage depth of 1 metre was then provided in the retarding basin. The flood results for the developed do-nothing conditions are shown in Appendix A.
- **Option 1A:** in this scenario, pump upgrade works are proposed at RB 1, with the aim of providing at least 600 mm freeboard in this RB and new distributed retarding basins A-D. This option only benefits RB 1 and the distributed retarding basins upstream of RB1. The flood model results for Option 1A are shown in Appendix B.
- **Option 1B:** in this scenario, in addition to the RB1 pump upgrade works, to provide additional flood storage the RB 1 base has been lowered by 1 m. As per Option 1A, this option only benefits RB 1 and the distributed retarding basins upstream of RB1. The flood model results for Option 1B are shown in Appendix C.
- **Option 2A:** in this scenario, the recommended upgrades identified in the Option 1B scenario have been implemented along with an increase in RB 3 pump rate to account for the increased flow from RB 1 and to reduce top water levels in RB 2. The flood model results for Option 2A are shown in Appendix D.
- **Option 2B:** in this scenario, the same modifications as Option 2A have been implemented, in addition to lowering the sports field next to RB 2 to increase the flood storage volume of RB 2. The flood model results for Option 2B are shown in Appendix E.
- **Option 2C:** in this scenario, RB 1 does not discharge to RB 2. The intention is to provide a new rising main discharging pumped flows to the Central Basin in the Cobram East drainage strategy, instead of utilising the existing rising main to RB 2. The flood model results for Option 2B are shown in Appendix F.
- **Option 3:** in this scenario, the outlet pipes for the new distributed retarding basins A-D for future development areas have been upsized to increase discharge rate from these retarding basins and provide sufficient freeboard in the distributed RBs. An increased RB 1 pump rate has also been proposed in this scenario, to provide sufficient freeboard to RB 1. Modification works at RB 2, including lowering the sports field next to RB 2 and upgrades of the RB 2 outlet have also been provided. The RB 3 pump rate has also been increased in this scenario. The flood model results for Option 3 are shown in Appendix G.
- **Option 4:** in this scenario, the distributed RB footprints have been increased to increase the flood storage volume in these retarding basins, retaining the originally proposed outlet pipe dimensions to provide sufficient freeboard. An increased RB 1 pump rate has also

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been proposed in this scenario, to provide sufficient freeboard to RB 1. Modification works at RB 2, including lowering the sports field next to RB 2 and upgrades of the RB 2 outlet have also been provided. The RB 3 pump rate has also been increased in this scenario. The flood model results for Option 4 are shown in Appendix H.

An additional scenario that was considered, but discounted during discussions with Council, involved building a large diameter gravity outfall from RB3 to Scotts Creek, through the ridge. This option may enable decommissioning of RB3 pump and reduce the dependence of the drainage system of the reliability of pumps. However, Council advised that this option would not be feasible due to constructability issues, including poor geotechnical conditions. Based on the advice from Council, the option was discounted from further consideration. If a geotechnical study has not been undertaken yet, it is recommended that this is to be completed to confirm that this option is not viable.

6.3 Model Overview

The TUFLOW model setup for the developed conditions is shown in Figure 6.1. Key changes between the existing conditions and developed conditions model setup are:

- Incorporating terrain modifications to represent the new distributed retarding basins A – D, with new outlet pipes from the RBs discharging to the existing pipes at Campbell Road.
- Applying RORB routed flow hydrographs from the future development areas to the new distributed retarding basins A-D (the future drainage systems within the developments are not explicitly modelled in TUFLOW).
- Modifications and upgrades at the existing retarding basins 1,2 and 3, as discussed in subsequent sections of the report.

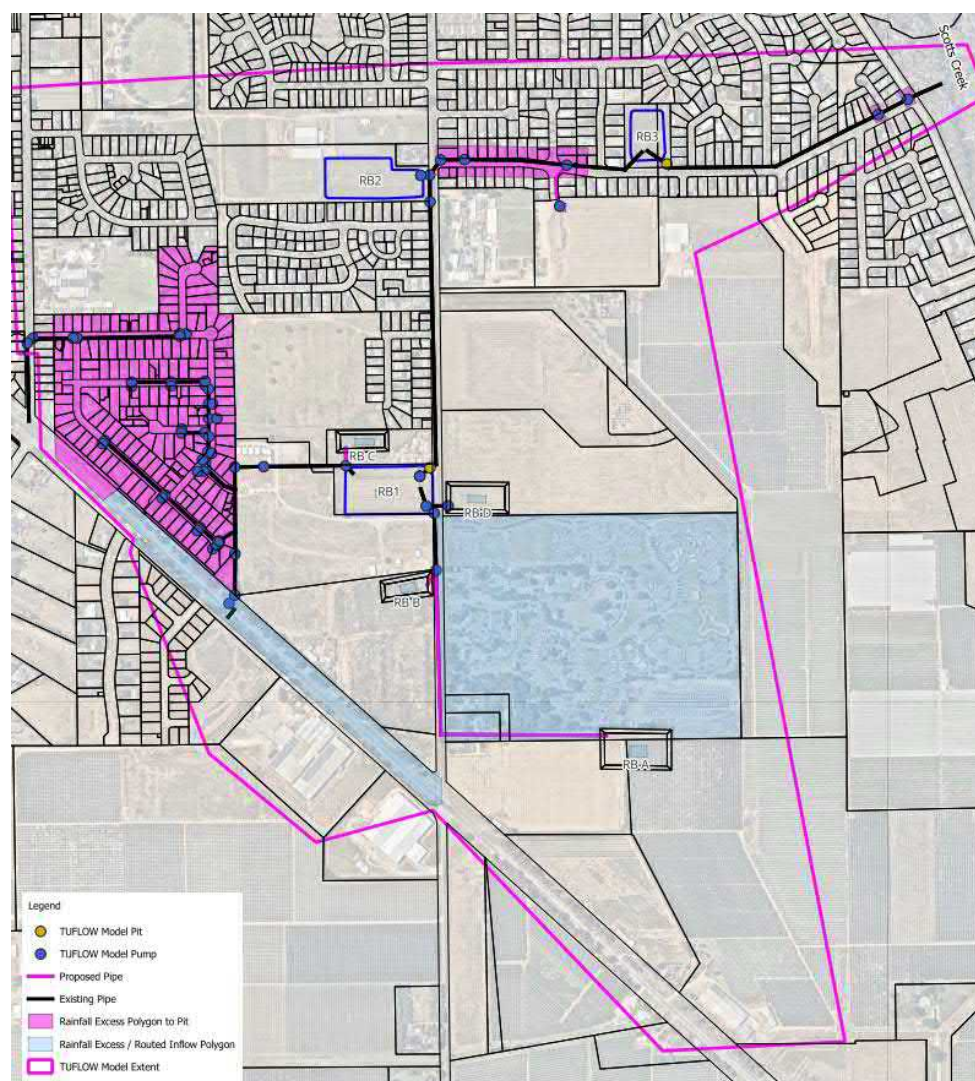


FIGURE 6.1: DEVELOPED CONDITIONS TUFLOW MODEL SETUP

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6.4 Developed Do-Nothing Scenario

In the developed do-nothing scenario, the additional new distributed retarding basins A-D were incorporated in the TUFLOW model, along with the existing drainage infrastructure. In this scenario, retarding Basins A-D discharge to the existing RB 1, with the existing configuration of retarding basins 1, 2 and 3 retained in the model. The layout plan for the developed do-nothing scenario is shown in Figure 6.2.

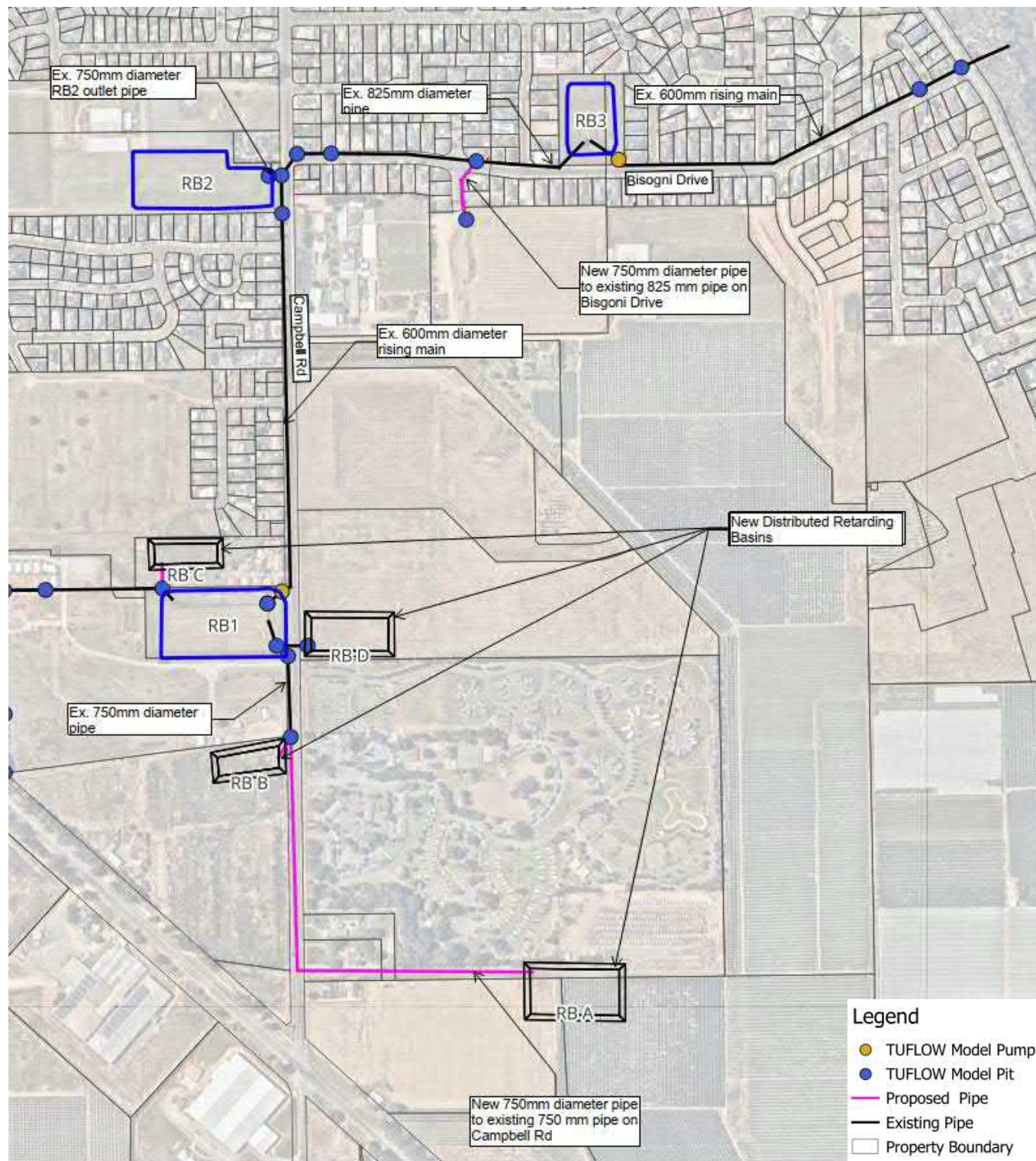


FIGURE 6.2: DEVELOPED DO-NOTHING LAYOUT PLAN

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The new distributed retarding basins were modelled with a base area that is 30% larger than the total treatment footprints discussed in Section 4, to allow for a buffer to fit in additional infrastructure such as access tracks. The distributed RB outlets were connected to the existing drainage network at Campbell Road through new 600 mm or 750 mm diameter pipes.

Table 6.1 summarises the predicted flood levels in the existing retarding basins 1, 2 and 3 and future retarding basins A, B, C and D.

In comparison to the existing conditions 1% AEP flood event, the developed do-nothing conditions 1% AEP flood modelling predicts the following:

- An increase in flood level in RB1 of approximately 10 – 15 mm.
- An increase in flood level in RB2 of approximately 35 mm.
- An increase in flood level in RB3 of approximately 55 mm.

The developed do-nothing scenario also indicates that there is insufficient flood storage in the new distributed retarding basin A-D, as there is overtopping or insufficient freeboard in retarding basins in the 1% AEP flood event. The overtopping of the retarding basins is due a combination of insufficient flood storage and tailwater constraints due to the high flood level in RB 1 and shortfall in capacity in the existing drainage system in Cambell Street.

TABLE 6.1: DEVELOPED DO NOTHING SCENARIO FLOOD RESULTS AT RETARDING BASINS 1-3

	5% AEP Top Water Level (m AHD)	5% AEP Freeboard (mm)	1% AEP Top Water Level (m AHD)	1% AEP Freeboard (mm)
Retarding Basin 1	113.15	150	113.42	-120 (retarding basin is overtopping to the southern land)
Retarding Basin 2	113.70	-250 (retarding basin is overtopping to the southern land)	113.85	-400 (retarding basin is overtopping to the southern land)
Retarding Basin 3	113.82	880	114.29	410
Retarding Basin A	113.96	540	114.38	120
Retarding Basin B	113.44	110	113.79	-240 (retarding basin is overtopping)
Retarding Basin C	113.15	50	113.42	-220 (retarding basin is overtopping)
Retarding Basin D	113.32	-120 (retarding basin is overtopping)	113.7	-500 (retarding basin is overtopping)

6.5 Options 1A & 1B

Options 1A and 1B have been investigated to reduce flooding at RB 1. The flood modelling predicts that RB 1 overtops in the 1% AEP event in existing conditions, with flood levels predicted to increase by 15 mm in the developed do-nothing scenario.

In the modelling of Options 1A and 1B, modifications have been made at RB 1 to achieve approximately 600 mm freeboard in this RB in the 1% AEP flood event. Reducing the flood level in RB1 may also reduce the flood levels in retarding basins A-D as the distributed RBs are hydraulically linked to RB 1.

In these scenarios, modifications are not provided in retarding basins 2 and 3. The Option 1 work alone is not suitable for implementation since this option exacerbates the existing flooding at and around RB2 which is currently overtopping onto adjacent properties. The intention of the Option 1 work is to identify upgrades required at RB 1, to understand how the works at RB 1 would impact flooding in the downstream drainage system (RB 2 and RB 3) and to then continue modelling these upgrades in the Option 2 work. In Option 2, modifications are proposed at RB 2 and RB 3 as well, with the intention of providing sufficient freeboard to RB2.

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The drainage modifications included in Option 1A and Option 1B are:

- Option 1A: the RB 1 pump station capacity was increased from approximately 0.35 m³/s to 1.3 m³/s to provide the required freeboard at RB 1, following multiple iterations of different pump rates. Increasing the RB 1 pump capacity may also require upgrading the rising main from the pump station. The Option 1A layout plan is shown in Figure 6.3.
- Option 1B: The RB 1 base was lowered by 1 m to increase the available flood storage and the RB 1 pump station capacity was increased from approximately 0.35 m³/s to 1.0 m³/s to provide the required freeboard at RB 1, following multiple iterations of different pump rates. Increasing the RB 1 pump capacity may also require upgrading the rising main from the pump station. The Option 1B layout plan is shown in Figure 6.4.

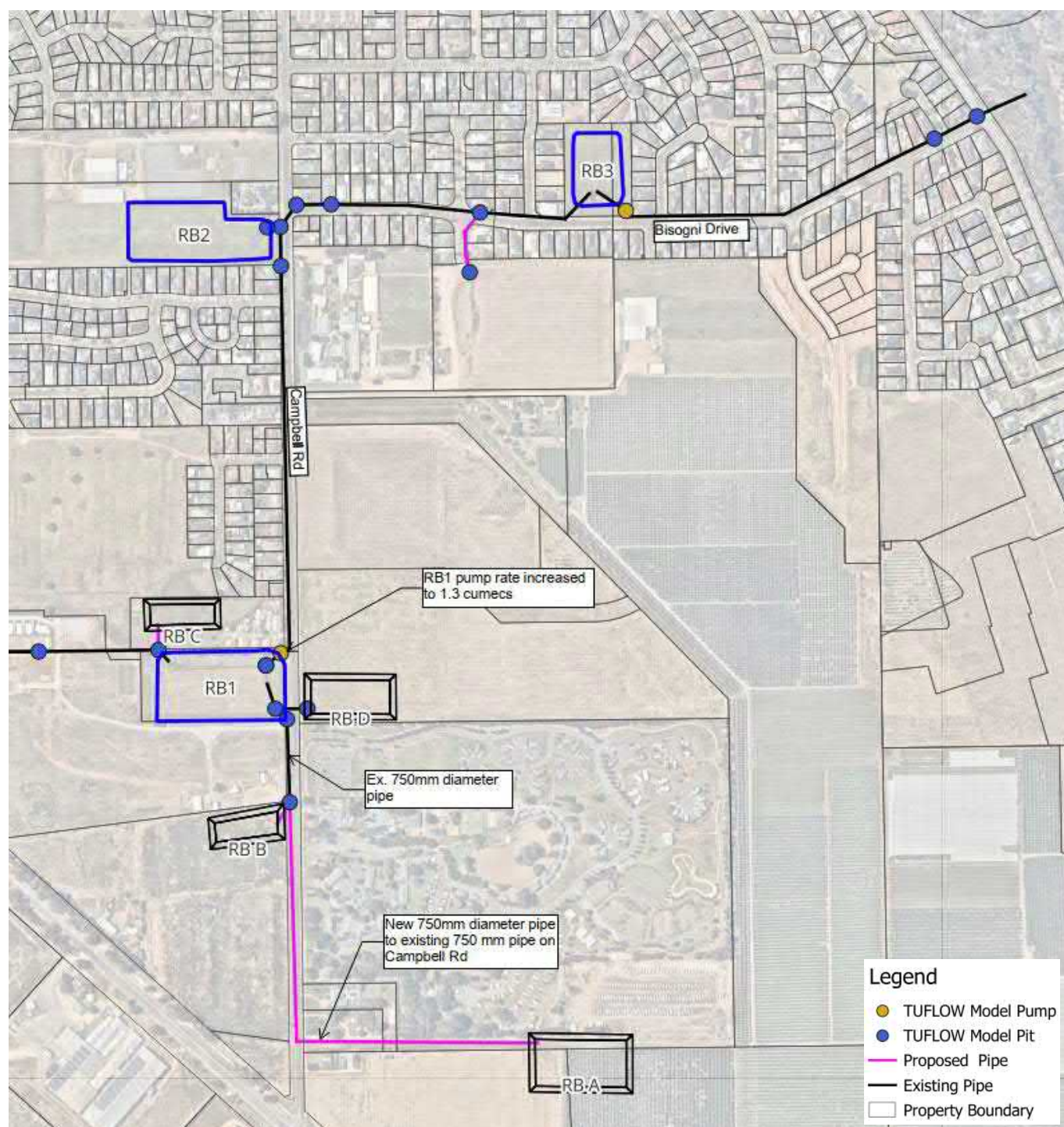


FIGURE 6.3: OPTION 1A LAYOUT PLAN

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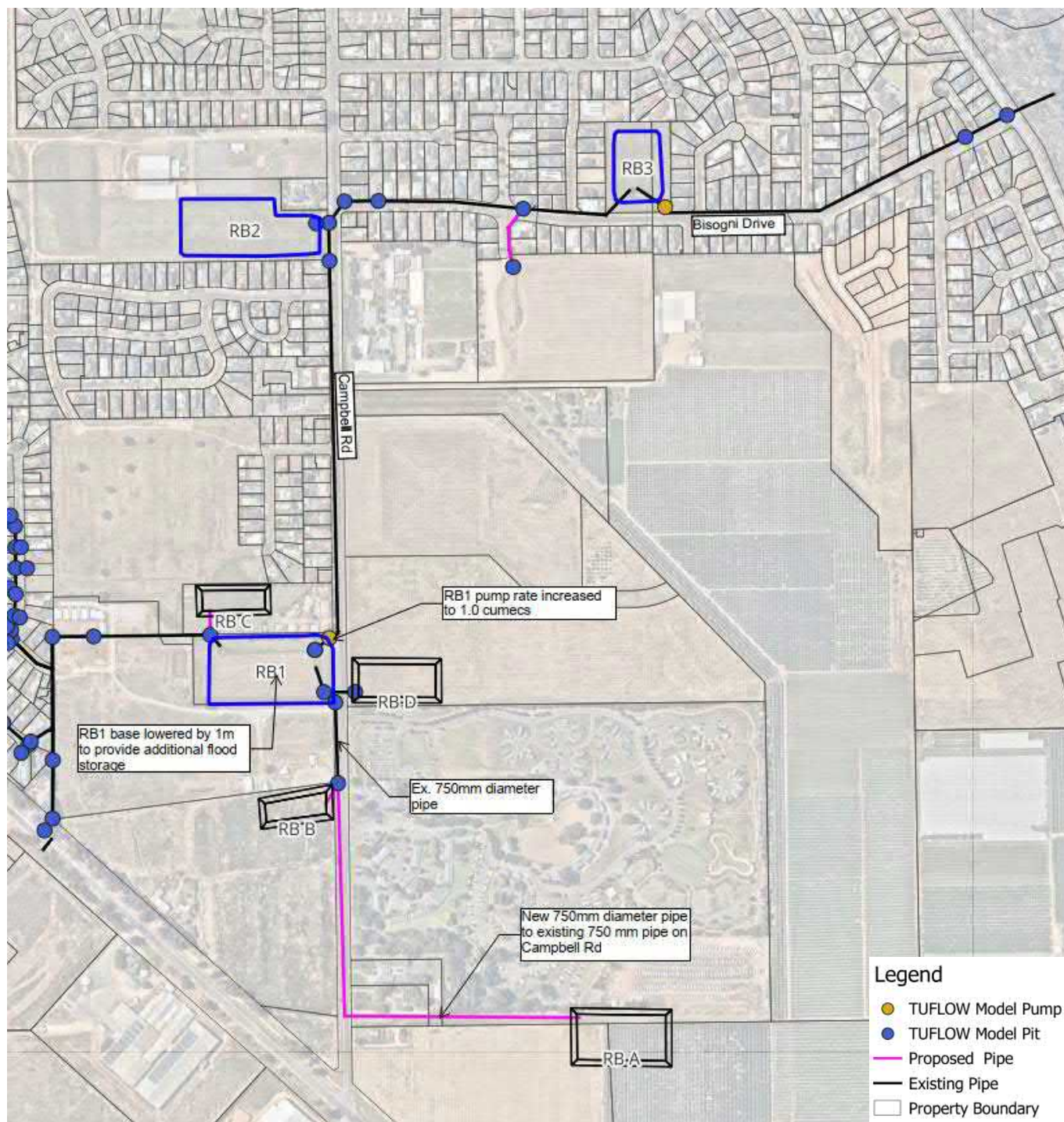


FIGURE 6.4: OPTION 1B LAYOUT PLAN

Of the two options that have been modelled, Option 1B was determined to be preferable to Option 1A as the pump upgrades needed in Option 1B are less substantial than Option 1A.

Compared to existing conditions, the flood modelling of Option 1B predicts:

- A reduction in the 1% AEP flood level of 720 mm at RB1. This flood level reduction is due to the additional flood storage and increased pump outflow rate from RB1.
- Increases in the 1% AEP flood level within RB2 and RB3 of 330 mm and 190 mm respectively, due to the increased flow being conveyed downstream from RB 1.
- No change to existing flood depths at Mitchell Street, which indicates that flood depths in this area are driven by the capacity of the existing drainage system rather than backflow from RB 1.

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Compared to the developed do-nothing scenario, the flood modelling of Option 1B predicts:

- Negligible change in RB A, with only a 30 mm reduction in flood level.
- 90 mm reduction in flood level at RB B, 490 mm reduction in flood level at RB C, and a 400 mm reduction in flood level at RB D

The Option 1B flood modelling results for retarding basins 1-3 and distributed RBs A-D are shown in Table 6.2.

TABLE 6.2: OPTION 1B FLOOD RESULTS

Location	5% AEP Top Water Level (m AHD)	5% AEP Freeboard (mm)	Change in 5% AEP Flood Level ¹ (mm)	1% AEP Top Water Level (m AHD)	1% AEP Freeboard (mm)	Change in 1% AEP Flood Level ¹ (mm)
Retarding Basin 1	112.21	1090	-730	112.69	610	-720
Retarding Basin 2	113.97	-520 (retarding basin is overtopping)	310	114.15	-700 (retarding basin is overtopping)	330
Retarding Basin 3	114.13	570	410	114.43	270	190
Retarding Basin A	113.95	550	-10	114.35	150	-30
Retarding Basin B	113.41	140	-30	113.70	-150 (retarding basin is overtopping)	-90
Retarding Basin C	112.72	480	-430	112.93	270	-490
Retarding Basin D	112.77	430	-550	113.30	-100 (retarding basin is overtopping)	-400

¹ Results compared to existing conditions for RB 1-3 or developed do-nothing for RBs A-D)

6.6 Options 2A & 2B

Options 2A and 2B involve the works included in Option 1B as well as modifications at RB 2 and RB 3, to reduce the top water level in RB 2 and recued the flooding that is predicted around RB 2. The Option 2A and 2B layout plans are provided in Figure 6.5 and Figure 6.6 respectively.

The drainage modifications included in Option 2A and Option 2B are:

- Option 2A:
 - As per Option 1B, the RB 1 base is lowered by 1 metre to increase the flood storage volume in the RB and the RB 1 pump station capacity has been increased from approximately 0.35 m³/s to a rate of 1 m³/s.
 - The RB 3 pump station capacity has been increased from approximately 0.35 m³/s to a rate of 1.5 m³/s, with the intention of reducing RB 2 flood levels as RB 2 and RB 3 are hydraulically linked.
- Option 2B:
 - Option 2B has been modelled with the same modifications as Option 2A detailed above, but also includes lowering the sports field adjacent to RB 2 by 0.5 metres to provide additional flood storage in RB 2. Two 900 mm pipes are provided between RB 2 and the sports field to link these two areas for flood storage.

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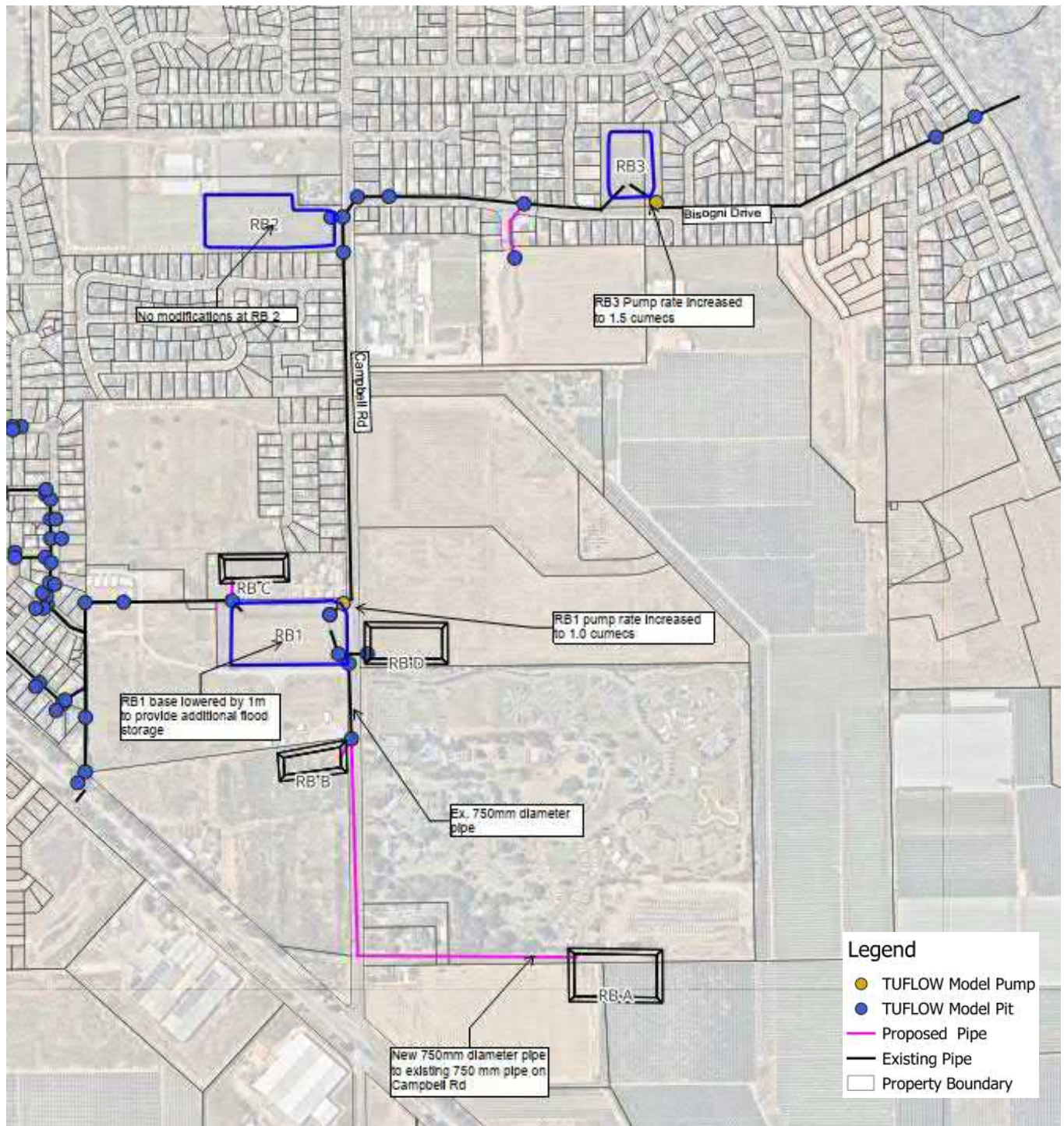


FIGURE 6.5: OPTION 2A LAYOUT PLAN

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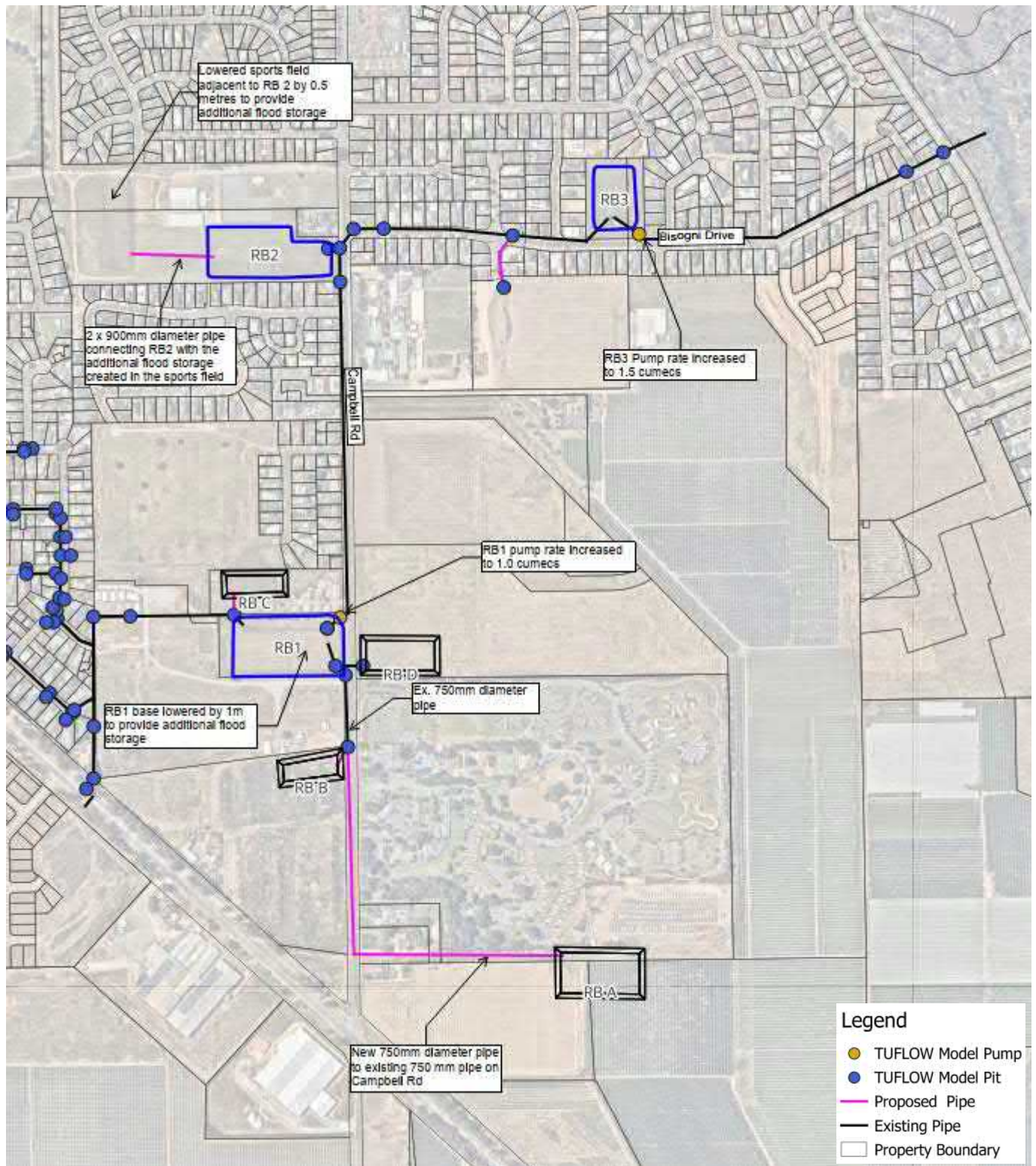


FIGURE 6.6: OPTION 2B LAYOUT PLAN

Analysis of the Options 2A and 2B flood modelling results indicate a reduction in flood level at RB 2 of approximately 150 mm in the 5% AEP event and 60 mm in the 1% AEP event in Option 2B, compared to Option 2A. Therefore, Option 2B was deemed to be the more preferred of the two options, although in this option there is still overtopping to properties adjacent to RB 2. Additional works to further reduce flood levels at and around RB 2 have been investigated in the Option 4 work.

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A summary of the key Option 2B flood modelling results is shown in Table 6.3. The top water levels in retarding basins 1 and A-D remain the same as Option 1B, because the flood levels within these retarding basins are influenced by the works at RB 1 (increased flood storage and pump capacity upgrade), which remains the same as Option 1B.

At RB 2, comparison of the Option 2B flood modelling results with the existing conditions results indicates:

- Reduction in the 5% AEP top water level at RB 2 of approximately 120 mm.
- Reduction in the 1% AEP top water level at RB 2 of approximately 80 mm.
- No change to existing flood depths at Mitchell Street, which indicates that flood depths in this area are driven by undersized drainage rather than backflow from RB 1.

TABLE 6.3: OPTION 2B FLOOD RESULTS

Location	5% AEP Top Water Level (m AHD)	5% AEP Freeboard (mm)	Change in 5% AEP Flood Level ¹ (mm)	1% AEP Top Water Level (m AHD)	1% AEP Freeboard (mm)	Change in 1% AEP Flood Level ¹ (mm)
Retarding Basin 1	112.21	1090	-730	112.69	610	-720
Retarding Basin 2	113.54	-90 (retarding basin is overtopping)	-120	113.74	-290	-80
Retarding Basin 3	112.94	1760	-780	113.24	1460	-1000
Retarding Basin A	113.95	550	-10	114.35	150	-30
Retarding Basin B	113.41	140	-30	113.70	-150 (retarding basin is overtopping)	-90
Retarding Basin C	112.72	480	-430	112.93	270	-490
Retarding Basin D	112.77	430	-550	113.30	-100 (retarding basin is overtopping)	-400

¹ Results compared to existing conditions for RB 1-3 or developed do-nothing for RBs A-D

6.7 Option 2C

Option 2C involves diverting RB 1 pumped outflows to the future central retarding basin in the Cobram East GADS to investigate the benefits for RB 2 if the RB 1 outflows do not enter this retarding basin.

The modelling of Option 2C includes the proposed works in Option 2B at RB 2 and RB 3, which are:

- Lowered sports field adjacent to RB 2 to provide additional flood storage in RB 2, and new pipes to link the existing flood storage area in RB 2 and the new flood storage in the adjacent sports field.
- Increased RB 3 pump station capacity to 1.5 m³/s.

The Option 2C layout plan is shown in Figure 6.7.

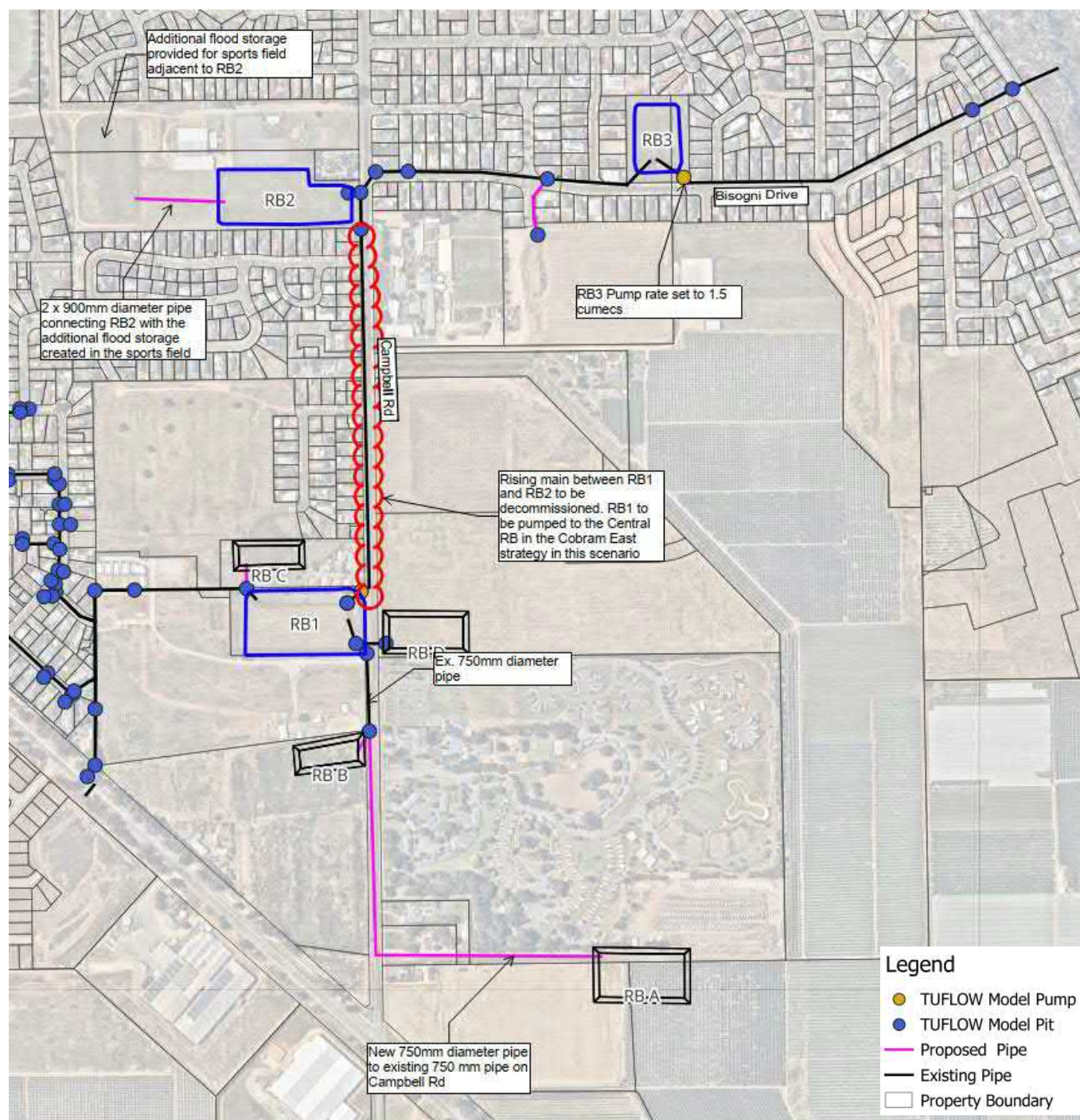


FIGURE 6.7: OPTION 2C LAYOUT PLAN

Comparison of the Option 2B and Option 2C flood modelling results indicate that by diverting RB 1 away from RB 2, a 300 mm reduction in the RB 2 top water level is achieved for both the 5% AEP and 1% AEP events. The Option 2C flood modelling results also indicate that in the 5% AEP flood event, approximately 230 mm of freeboard can be provided from the RB 2 top water level to adjacent properties.

In agreement with Council, Option 2C has been discounted from further consideration. While diverting flows from RB 1 away from RB 2 and RB 3 provides a reduction in the RB2 flood levels, modification works would still be required at RB2 and RB 3 to reduce flood levels and the design of the Cobram East central retarding basin would be more complex.

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TABLE 6.4: OPTION 2C FLOOD RESULTS

Location	5% AEP Top Water Level (m AHD)	5% AEP Freeboard (mm)	Change in 5% AEP Flood Level ¹ (mm)	1% AEP Top Water Level (m AHD)	1% AEP Freeboard (mm)	Change in 1% AEP Flood Level ¹ (mm)
Retarding Basin 2	113.22	230	-440	113.43	20	-390
Retarding Basin 3	112.86	1840	-860	113.20	1500	-1040

¹ Results compared to existing conditions for RB 2 & 3

6.8 Option 3 and Option 4

Option 3 and Option 4 have both been investigated to provide additional freeboard to the distributed retarding basins A-D. The works included in these scenarios are outlined below:

- In the Option 3 modelling, outlet pipes from retarding basins A - D have been upsized from the 600 mm and 750 mm diameter pipes provided in the previous scenarios. The intention is to increase the retarding basin outflows and therefore reduce the top water level. To convey a higher peak flow within Campbell Road to RB 1, the existing 750 mm and 900 mm pipes on Campbell Road have also been upsized to twin 1050 mm and 1200 mm pipes. The RB 1 inlet pipe has also been upsized to twin 1200 mm pipes. Due to the increased peak flows from retarding basins A – D discharging to RB 1, the pump rate from RB 1 has been increased to 1.5 m³/s to provide freeboard to this retarding basin. To reduce top water levels at RB 2, new twin 900 mm retarding basin outlet pipes have been provided and the RB 3 pump rate has also been increased to 1.8 m³/s. The Option 3 layout plan is shown in Figure 6.8.
- In the Option 4 modelling, the RB A – D footprints have been extended instead of upgrading the retarding basin outlet pipes and existing pipes on Campbell Road. The intention is to provide additional flood storage within the distributed retarding basins to reduce top water levels and provide sufficient freeboard. The RB A-D footprints in Option 4 are approximately 30 -35 % larger than the footprints in the previous developed conditions models for RBs A - C. RB D is approximately 60% larger. The RB 3 increased pump station capacity of 1.5 m³/s identified in the previous modelling has been retained. The existing outlet from RB 2 to RB 3 has been retained, however, an additional 1500 mm outlet pipe from RB 2 to RB 3 has also been provided. This will increase RB 2 outflows and reduce top water levels. The Option 4 layout plan is shown in Figure 6.9.

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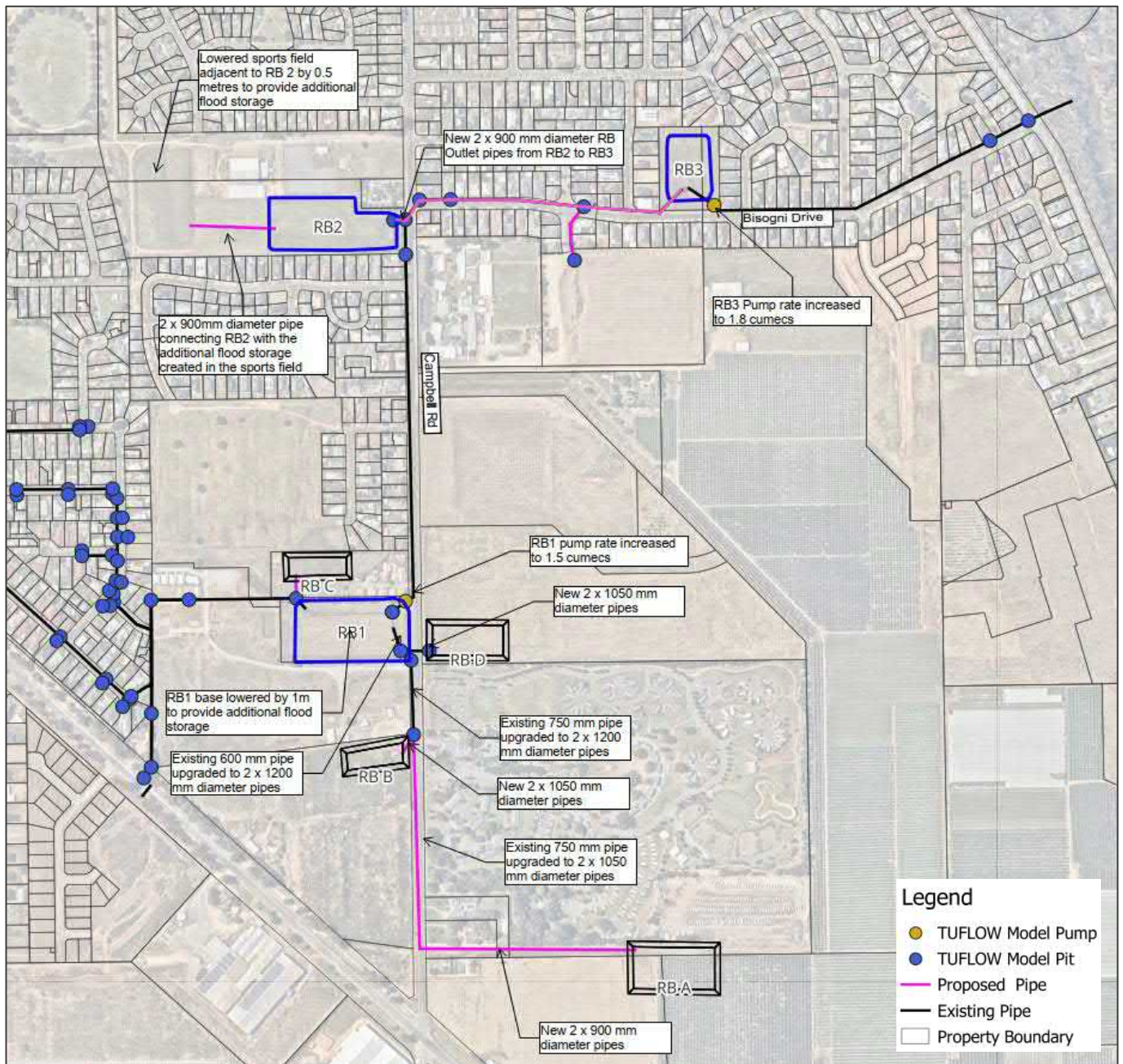


FIGURE 6.8: OPTION 3 LAYOUT PLAN

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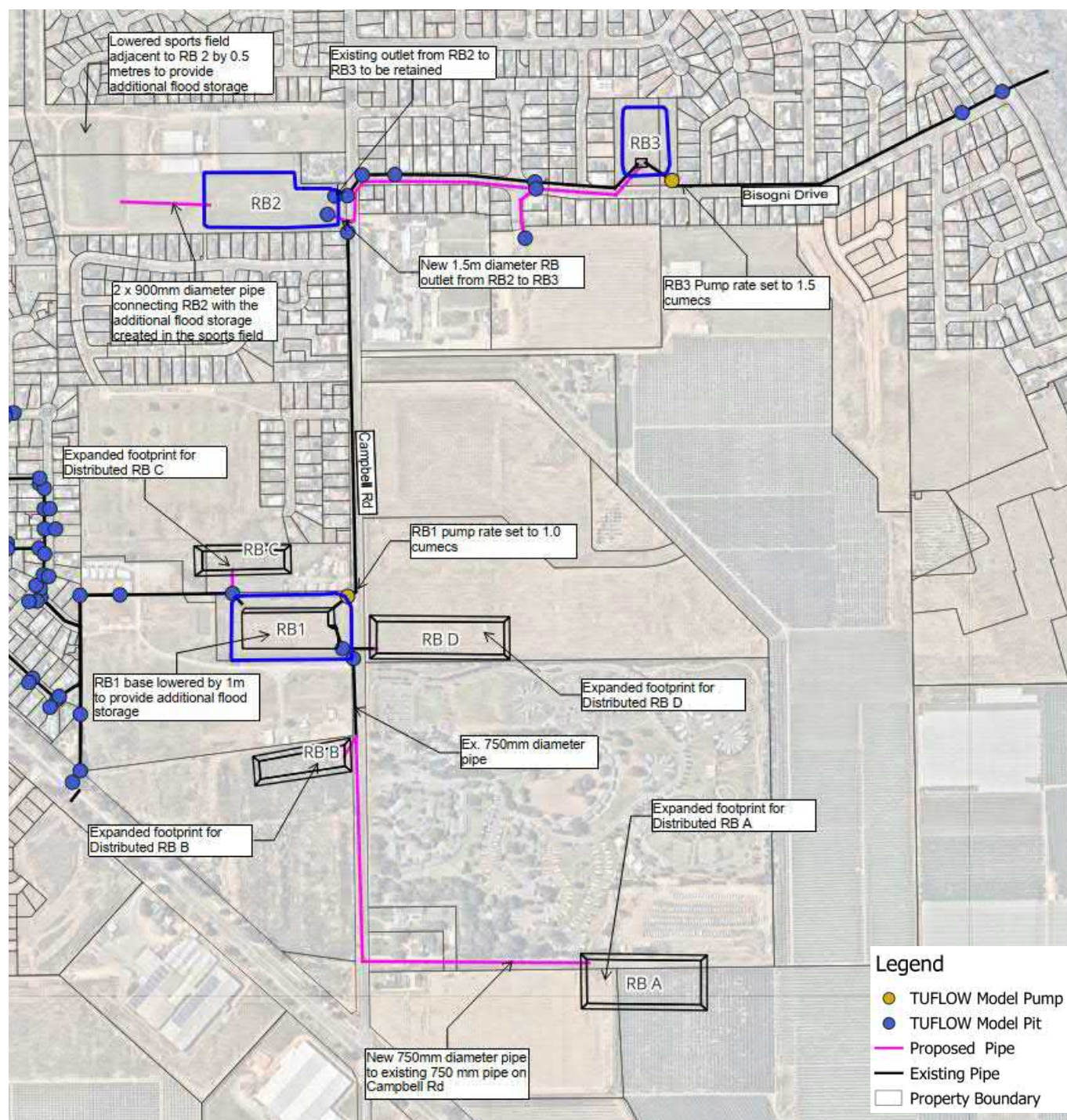


FIGURE 6.9: OPTION 4 LAYOUT PLAN

The Option 3 flood modelling predicts that:

- An increased RB 1 pump station capacity to 1.5 m³/s (from the 1 m³/s previously identified) is required to provide at least 300 mm freeboard at RB 1. This is due to the higher inflow rate from the distributed retarding basins to RB 1 that is achieved by the upsized outlet pipes from the distributed retarding basins.
- A minimum of approximately 300 mm of freeboard can be provided for the new distributed retarding basins A-D, with the upsized retarding basin outlet sizes.

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The Option 4 flood modelling predicts that:

- With the additional increased flood storage at the new distributed retarding basins A-D, the pump rate of 1 m³/s previously determined in Option 1B can be retained, as this pump rate allows for greater than 600 mm freeboard at RB 1.
- A minimum of approximately 300 mm of freeboard can be provided for the new distributed retarding basins A-D, with the expanded retarding basin footprints.
- With the RB 3 pump upgrade and RB 2 outlet works, a 5% level of service (150 mm freeboard) is provided to RB 2.

Comparison of the Option 4 flood modelling results with the existing conditions results indicates:

- A reduction in the 1% AEP top water level at RB 1 of approximately 880 mm. Greater than 600 mm of freeboard is provided at RB 1 in the 1% AEP event.
- Reduction in the 1% AEP top water level at RB 2 of approximately 250 mm, and a reduction in the 5% AEP top water level of 360 mm. This provides approximately 150 mm of freeboard to adjacent developments in the 5% AEP event.

Comparison of the Option 4 results with the developed do-nothing scenario results indicate:

- Reduction in the 1% AEP top water levels at distributed retarding basins A – D from 300 mm at RB A to approximately 600 – 700 mm reduction in top water levels at RB B – D.

Following discussions with Council, Option 4 was preferred over Option 3 as Option 4 does not require replacing existing pipes on Campbell Road and Option 4 requires a less substantial upgrades of the capacity of the RB1 pump station. Option 3 would likely require larger scale upgrades at RB 2 and RB 3 to accommodate the increase pump rate from RB 1.

Table 6.5 summarises key flood modelling results for Option 4.

TABLE 6.5: OPTION 4 FLOOD RESULTS

Location	5% AEP Top Water Level (m AHD)	5% AEP Freeboard (mm)	Change in 5% AEP Flood Level ¹ (mm)	1% AEP Top Water Level (m AHD)	1% AEP Freeboard (mm)	Change in 1% AEP Flood Level ¹ (mm)
Retarding Basin 1	112.12	1180	-820	112.53	770	-880
Retarding Basin 2	113.30	150	-360	113.57	-120 (retarding basin is overtopping)	-250
Retarding Basin 3	113.19	1510	-530	113.62	1080	-620
Retarding Basin A	113.73	770	-230	114.06	440	-320
Retarding Basin B	112.82	730	-620	113.24	310	-550
Retarding Basin C	112.58	620	-570	112.75	450	-670
Retarding Basin D	112.56	640	-760	113.0	200	-700

¹ Results compared to existing conditions for RB 1-3 or developed do-nothing for RBs A-D

7. RECOMMENDED DRAINAGE STRATEGY

Following the flood modelling of multiple options, Option 4 was identified as the preferred scenario for the Cobram South GADS. This involves the implementation of the following infrastructure and upgrades:

- New distributed retarding basins A, B, C and D, each having a sediment pond and wetland within the base of the retarding basin.
- An additional stand-alone sediment pond provided for the future development south of Young Street (within sub-catchment H).
- Increase capacity of the RB 1 pump station to 1 m³/s and lowering the base of RB 1 by 1 metre.
- A new upsized rising main from the RB 1 pump station to the existing 825 mm pipe at Campbell Road, upstream of Retarding Basin 2. A 900 mm rising main is suitable to limit pipe velocities to 1.5 m/s, to convey a flow of 1 m³/s. This is subject to review in future design stages of the pump upgrade.
- Lowering the sports field adjacent to RB 2 by 0.5 metres and provide 2 x 900 mm pipes to connect the sports field and RB 2.
- Additional 1500 mm outlet pipe from RB 2 to RB 3, however the existing RB 2 outlet pipe is also to be retained.
- Increase capacity of the RB 3 pump station capacity to 1.5 m³/s.
- A new upsized rising main from the RB 3 pump station to the gravity stormwater drain on Bisogni Drive. A 1200 mm rising main is suitable to limit pipe velocities to 1.5 m/s, to convey a flow of 1.5 m³/s. This is subject to review in future design stages of the pump upgrade.
- Upgrade of the drainage system on Bisogni Drive, downstream of the RB 3 rising main are recommended, as the flood modelling of Option 4 predicts surcharge through pits in this area and an increase in flood level compared to the existing conditions.

The indicative footprint areas of the new distributed retarding basins are summarised in Table 7.1. These areas are indicative and based on conceptual hydraulic modelling, and are subject to refinement in the future design stages of development.

TABLE 7.1: DISTRIBUTED RETARDING BASIN FOOTPRINTS

Distributed Retarding Basin	Indicative Retarding Basin Footprint Area (ha)
A	1.5 ha
B	0.65 ha
C	0.62 ha
D	1.3 ha

The 5% and 1% AEP change in flood levels (afflux) between Option 4 and existing conditions are shown in Figure 7.1 and Figure 7.2 respectively.

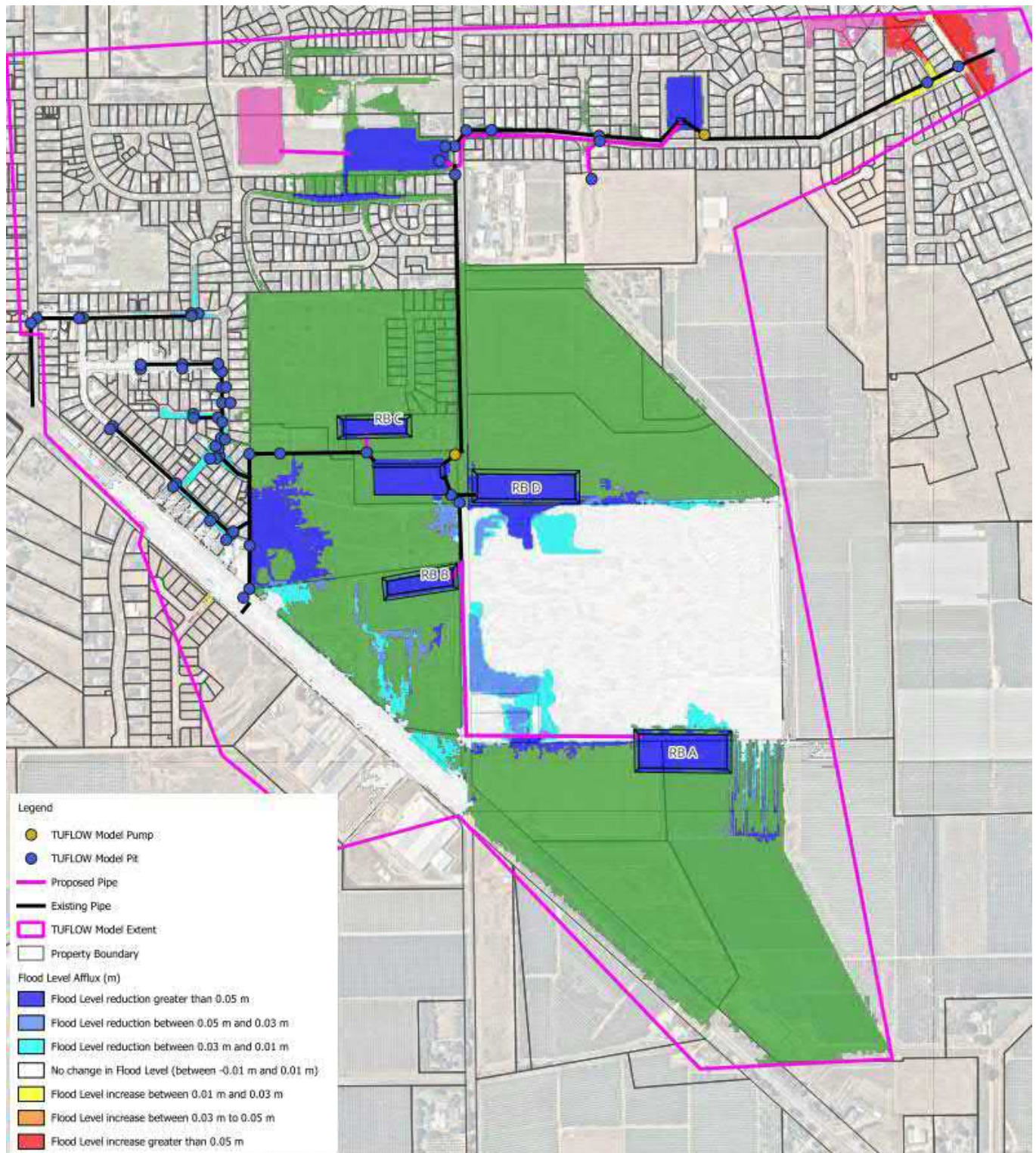


FIGURE 7.1: OPTION 4 5% AEP CHANGE IN FLOOD LEVEL (COMPARED TO EXISTING CONDITIONS)

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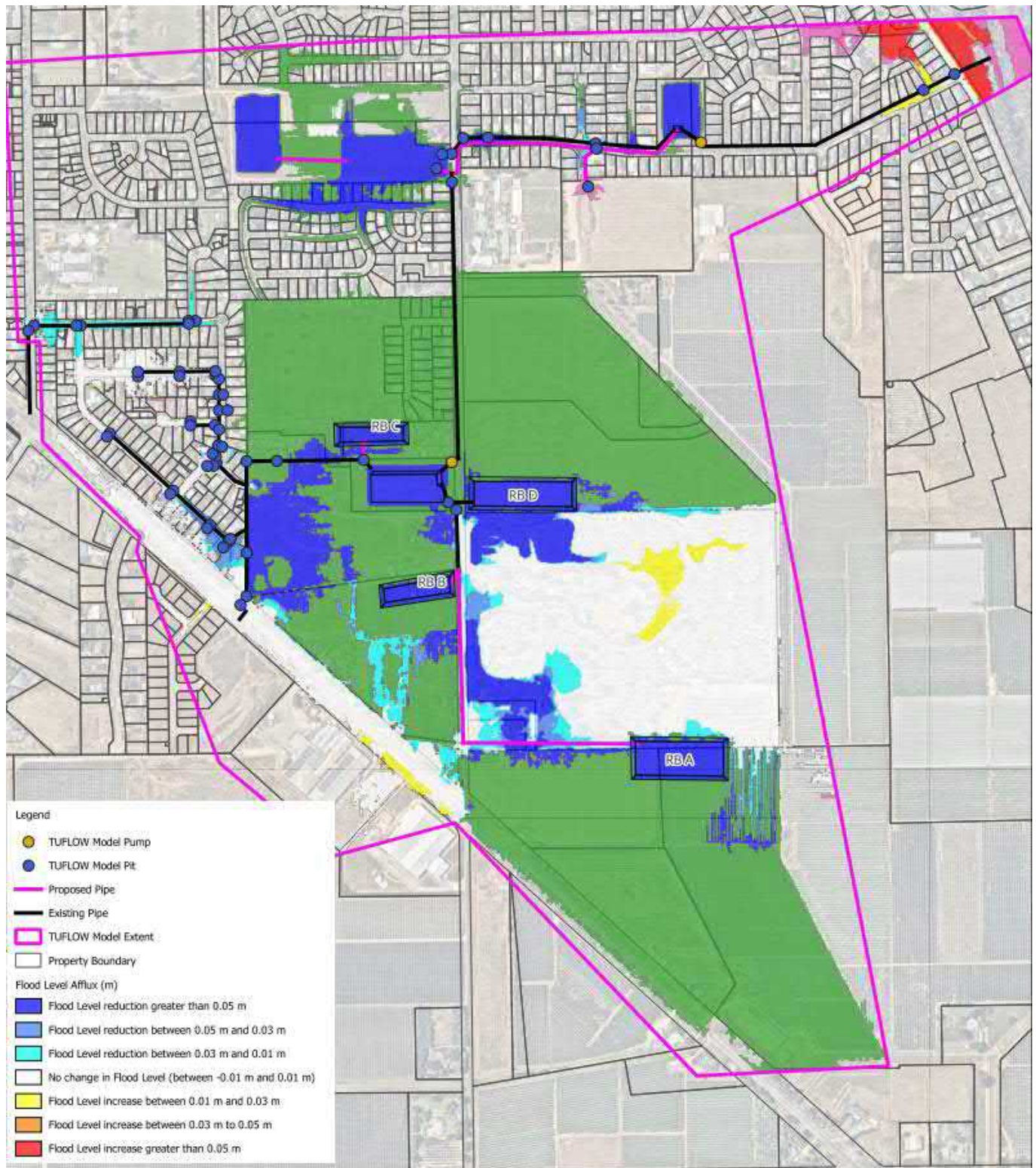


FIGURE 7.2: OPTION 4 1% AEP CHANGE IN FLOOD LEVEL (COMPARED TO EXISTING CONDITIONS)

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8. MITIGATION OF EXISTING FLOODING

An investigation has also been undertaken to investigate the drainage upgrades required for the existing drainage system (including retarding basins 1-3) for the mitigation of flooding in the existing conditions, prior to the planned future development. This allows for a comparison in the scale of upgrades required for the existing drainage system with and without the additional development planned in the Cobram South GADS.

The following drainage upgrades are proposed in this scenario, as shown in Figure 8.1.

- The Retarding Basin 1 base lowered by 1 metre to provide for additional flood storage.
- The Retarding Basin 1 pump station capacity increased to 1 m³/s.
- A new upsized rising main from the Retarding Basin 1 pump station to the existing 825 mm pipe at Campbell Road, upstream of Retarding Basin 2. A 900 mm rising main is suitable to limit pipe velocities to 1.5 m/s, to convey a flow of 1 m³/s. This is subject to review in future design stages of the pump upgrade.
- The sports field adjacent to Retarding Basin 2 lowered by 500 mm to increase flood storage at this retarding basin.
- New twin 900 mm diameter pipes from the RB 2 outlet to RB 3.
- The Retarding Basin 3 pump station capacity increased to 1.2 m³/s.
- A new upsized rising main from the Retarding Basin 3 pump station to the gravity stormwater mains on Bisogni Drive. A 1050 mm rising main is suitable to limit pipe velocities to 1.5 m/s, to convey a flow of 1.2 m³/s. This is subject to review in future design stages of the pump upgrade.
- Upgrades of the drainage system on Bisogni Drive, downstream of the RB 3 rising main are recommended, as the developed conditions modelling predicts surcharge through pits in this area and an increase in flood level compared to the existing conditions.

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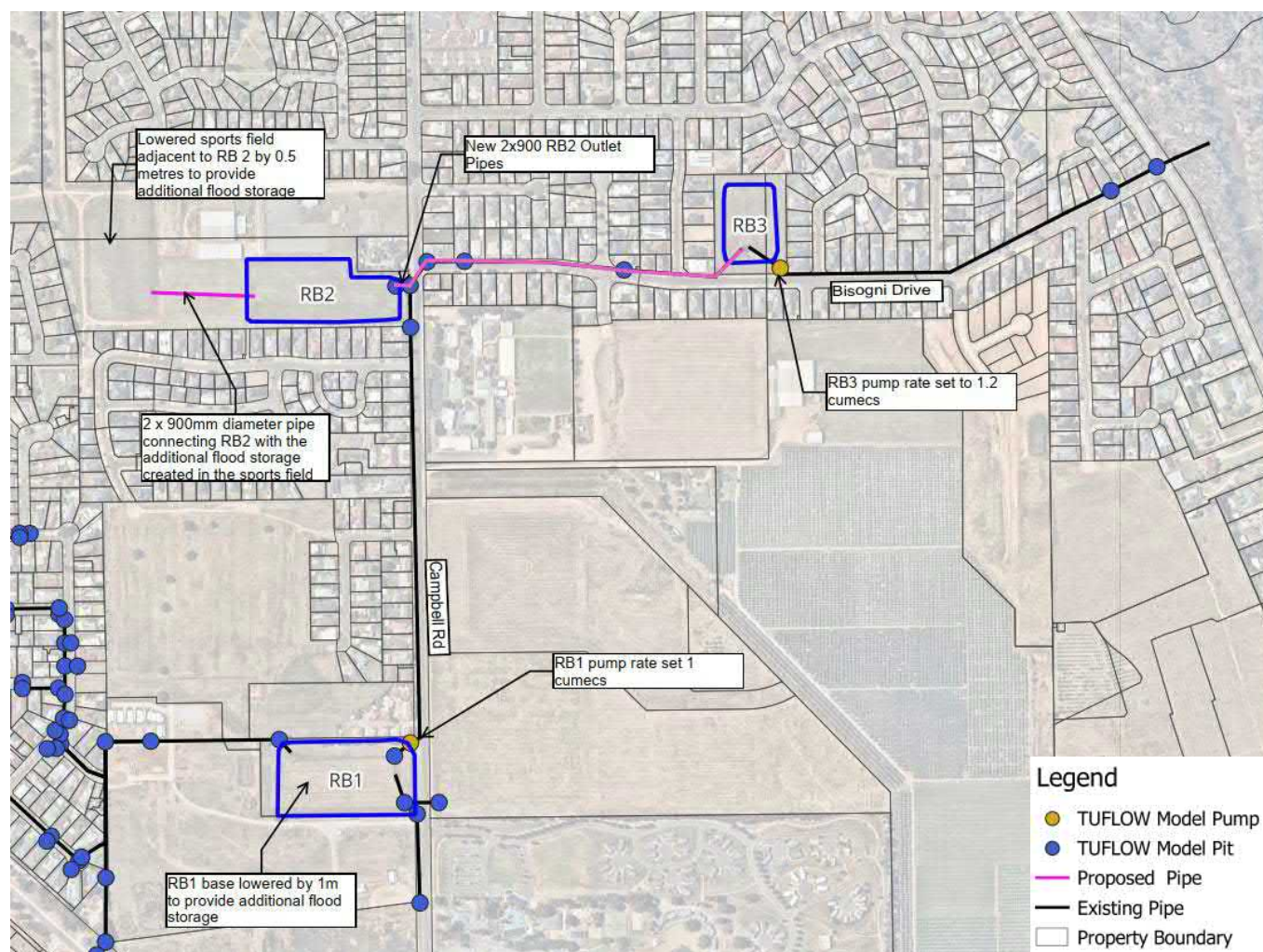


FIGURE 8.1: EXISTING CONDITION DRAINAGE UPGRADES LAYOUT PLAN

A comparison of the upgrades recommended for the future development strategy and the existing development strategy is shown in Table 8.1.

TABLE 8.1: COMPARISON OF FUTURE DEVELOPMENT AND EXISTING DEVELOPMENT STRATEGY

	Future Development Strategy Proposed Upgrades	Existing Development Strategy Proposed Upgrades
Retarding Basin 1 (RB 1)	- RB Base lowered by 1 m - RB pump rate increased to 1 m³/s	- RB Base lowered by 1 m - RB pump rate increased to 1 m³/s
Retarding Basin 2 (RB 2)	- Lowered sports field next to RB 2 by 0.5 m with 2 x 900 mm pipe connecting the lowered sports field with RB 2 - New 1500 m diameter pipe outlet from RB 2 to RB 3 and the existing outlet is retained	- Lowered sports field next to RB 2 by 0.5 m with 2 x 900 mm pipe connecting the lowered sports field with RB 2 - New 2 x 900 mm diameter pipe outlet from RB 2 to RB 3
Retarding Basin 3 (RB 3)	RB 3 pump rate increased to 1.5 m³/s	RB 3 pump rate increased to 1.2 m³/s

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Flood modelling results for the mitigation of existing flooding analysis is shown in Appendix J. Table 8.2 and Table 8.3 provides a comparison of the 5% AEP existing condition flood results with the mitigation interventions in place, which indicates:

- Retarding Basin 1 has a 1% AEP level of service, providing approximately 600 mm freeboard in this flood event.
- Retarding Basin 2 has a 5% AEP level of service, providing approximately 120 mm freeboard in this flood event. In the 1% AEP flood event, Retarding Basin 2 is overtopping to adjacent properties.
- Retarding Basin 3 has a 1% AEP level of service.

TABLE 8.2: 5% AEP MITIGATION FOR EXISTING CONDITIONS FLOOD MODELLING RESULTS

	Existing Conditions 5% AEP Top Water Level (m AHD)	Post- Mitigation 5% AEP Top Water Level (m AHD)	5% AEP Freeboard in Post Mitigation Scenario (mm)
Retarding Basin 1	112.94	111.65	1650
Retarding Basin 2	113.66	113.33	120
Retarding Basin 3	113.72	113.15	1550

TABLE 8.3: 1% AEP MITIGATION FOR EXISTING CONDITIONS FLOOD MODELLING RESULTS

	Existing Conditions 1% AEP Top Water Level (m AHD)	Post- Mitigation 1% AEP Top Water Level (m AHD)	1% AEP Freeboard in Post Mitigation Scenario (mm)
Retarding Basin 1	113.41	112.71	590
Retarding Basin 2	113.82	113.58	-130 (retarding basin is overtopping)
Retarding Basin 3	114.24	113.56	1140

Figure 8.2 and Figure 8.3 respectively show the 5% AEP and 1% AEP change in flood level of the mitigation of existing conditions compared to the existing conditions

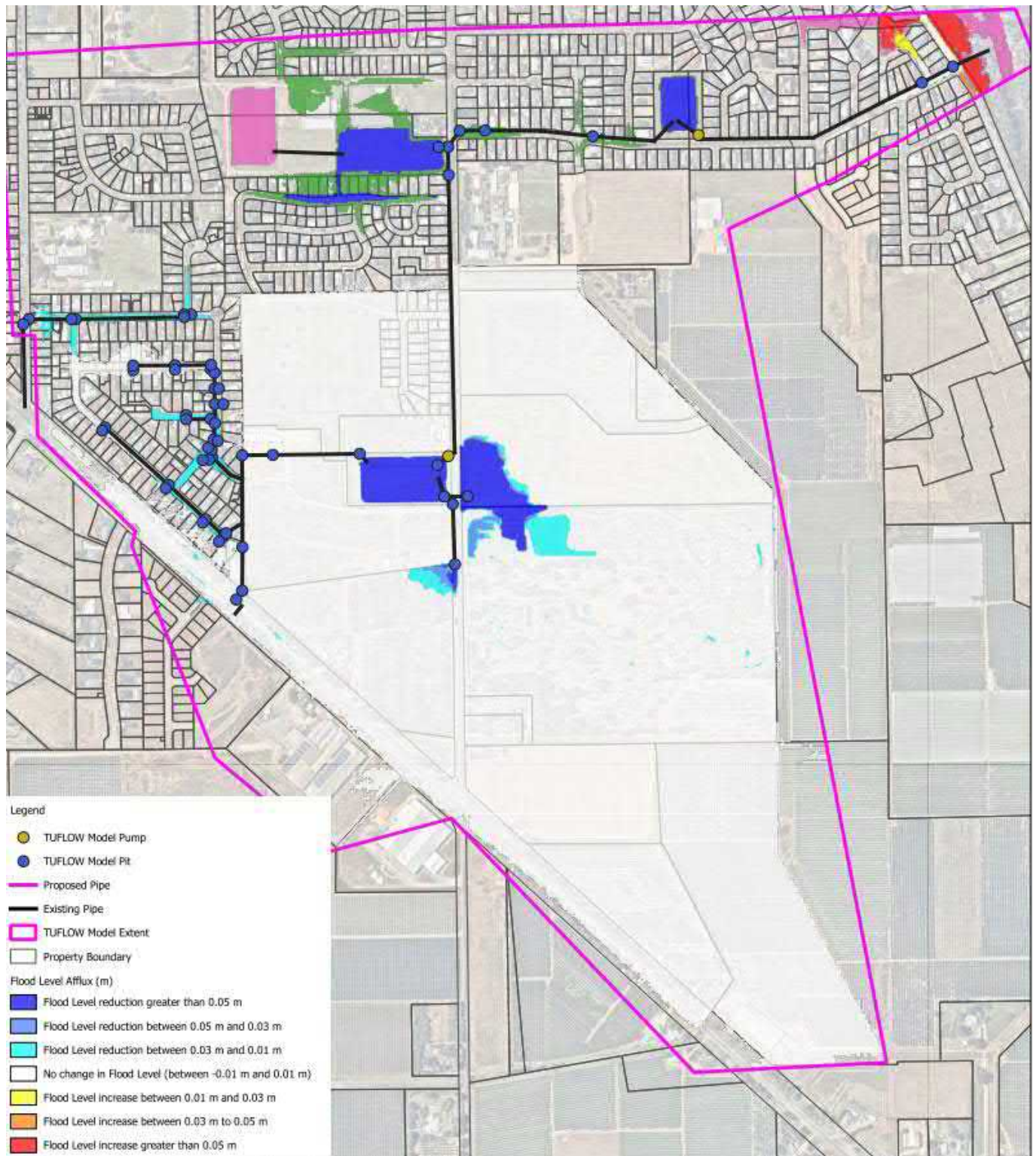


FIGURE 8.2: MITIGATION OF EXISTING CONDITIONS - 5% AEP CHANGE IN FLOOD LEVEL (COMPARED TO EXISTING CONDITIONS)

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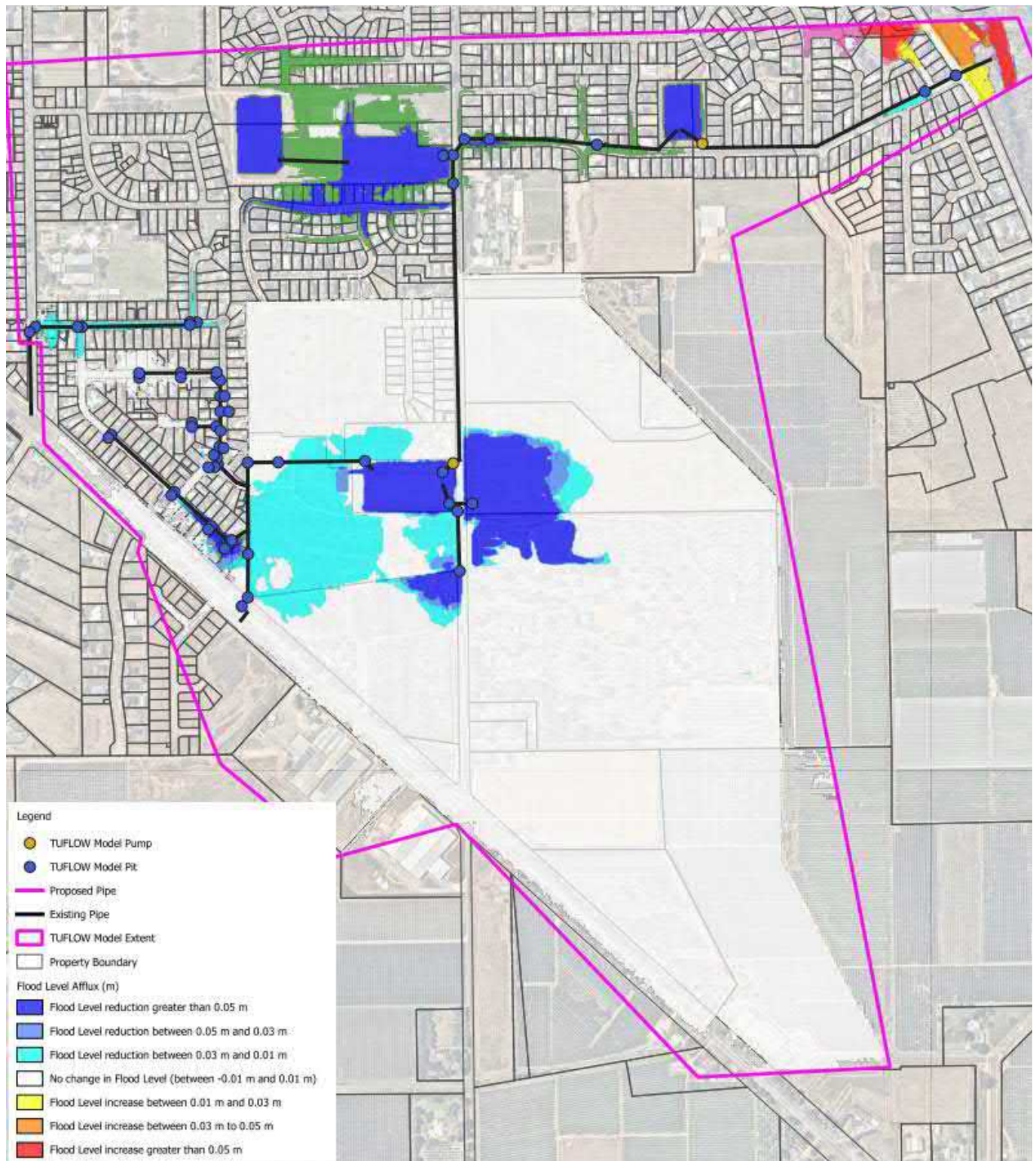


FIGURE 8.3: MITIGATION OF EXISTING CONDITIONS - 1% AEP CHANGE IN FLOOD LEVEL (COMPARED TO EXISTING CONDITIONS)

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9. RECOMMENDATIONS AND NEXT STEPS

Engeny's strategy for the Cobram South Growth Area Drainage Scheme (GADS) has identified:

- Conceptual level WSUD assets (sediment basins and wetlands) sizing within new retarding basins in future development areas, based on sediment basin Fair & Geyer computations and MUSIC modelling.
- Conceptual level future retarding basins sized using TUFLOW, to achieve sufficient freeboard to the top of the retarding basins.
- Modifications and upgrades required at the existing retarding basins, which are:
 - At RB 1, lowering the base by 1 metre to increase the flood storage volume and increasing the pump station capacity to 1 m³/s is proposed, to achieve freeboard to the top of the retarding basin.
 - At RB 2, the sports field adjacent to the retarding basin is recommended to be lower by approximately 0.5 metres to increase flood storage volume with new 2 x 900 mm pipe connections between the sports field and the retarding basin to connect the existing flood storage at RB 2 with the new flood storage at the sports field.
 - At RB 3, the pump station capacity is increased to 1.5 m³/s to reduce top water levels in RB 2.

Further analysis and refinement of the drainage strategy will be required as planning progresses, including consideration of:

- Sizing of the rising mains and gravity stormwater mains downstream of RB 1, RB 2 and RB 3.
- Design of the upgraded pump stations at RB 1 and RB 3, including consideration of wet well size, power supply and duty / standby pump arrangements to achieve the overall pump station capacity.
- Consideration of the impact of pump failure on the flood levels and how future development levels may be designed to provide redundancy in the event of pump failure.
- Sizing of the new distributed retarding basins A-D in future development areas.

An additional scenario that was considered, but discounted during discussions with Council, involved building a large diameter gravity outfall from RB3 to Scotts Creek, through the ridge. Council advised that this option would not be feasible due to constructability issues, including poor geotechnical conditions. If a geotechnical study has not been undertaken yet, it is recommended that this is to be completed to confirm that this option is not viable.

Council may also want to consider further investigation of the predicted flooding issue at Mitchell Street. The flood modelling predicts 1% AEP flood depths at the low point on Mitchell Street of approximately 500 mm, with a H3 hazard rating, making flood conditions in this area unsuitable for vehicles, children and the elderly. This flooding issue is likely due to insufficient drainage capacity from the low point of Mitchell Street to RB1.