WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Final Report

Twenty-Four Lane "The Grange" - Drainage Strategy Advice

Development Outcomes Pty Ltd

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Photo of St Anne's Vineyard, Moama
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1 INTRODUCTION

Water Technology were engaged by Development Outcomes (DO) to assist with the development of a drainage strategy for the proposed development at Twenty-Four Lane "The Grange" Moama. Hydrological and water quality modelling was completed to ensure the drainage design meets the intent of the *Moama Mid-West Drainage Strategy* (RPS, 2019) and, ultimately, to help gain Murray River Council's endorsement of the drainage system design.

The objective of this study was to:

- Confirm storage (attenuation) requirements;
- Determine water quality treatment requirements; and
- Provide design advice to Development Outcomes to ensure the intent of the modelling is carried through to the proposed design.

1.1 Subject Site

The subject site is located in Murray River Council Local Government Area, immediately north of Beer Road and east of Twenty Four Lane. It is currently an agricultural site but has been earmarked for development to cater for future residential requirements in Moama. River Golf Club is immediately north and St Anne's Vineyards immediately west of the site. Figure 1-1 shows the subject site location.

FIGURE 1-1 SUBJECT SITE

The site is relatively flat with a significant depression in its centre, as shown in Figure 1-2. Surface levels vary between 92 m AHD and 97 m AHD. Under existing conditions, runoff drains towards this depression, which does not have a free draining outfall point. Water is predominantly lost via infiltration and evapotranspiration.

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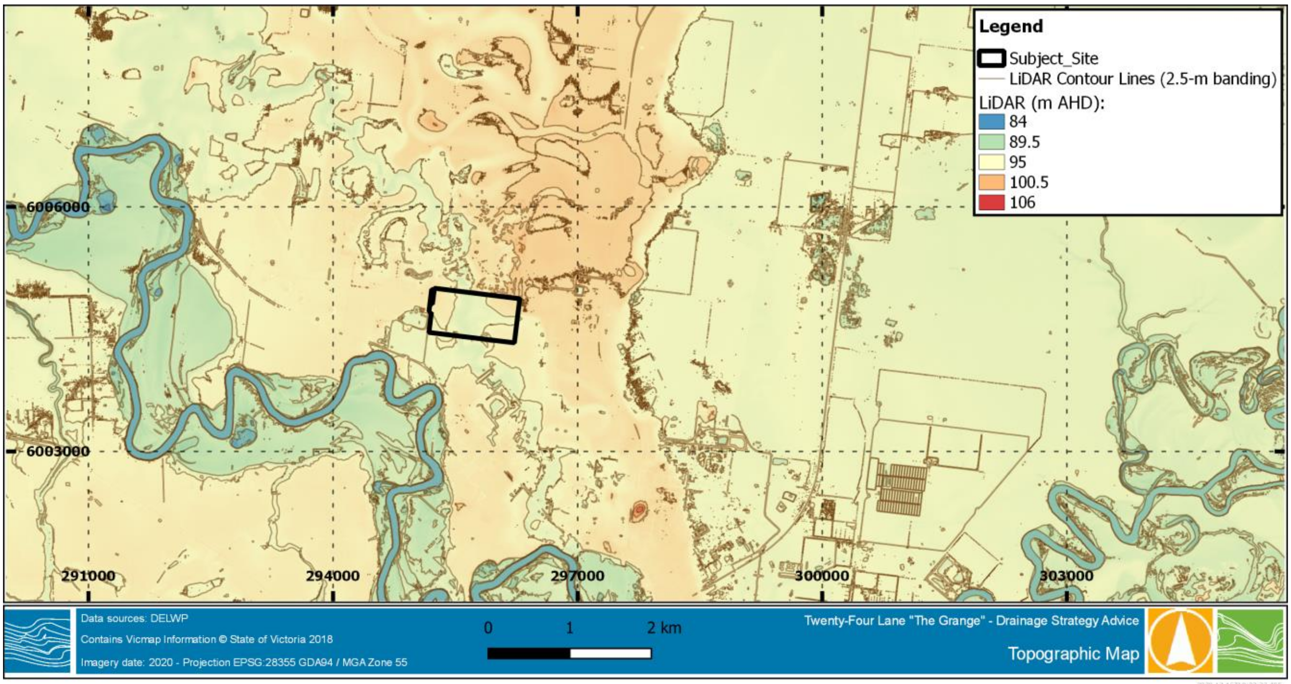

FIGURE 1-2 TOPOGRAPHY

1.2 Proposed Subdivision

The proposed Twenty-Four Lane "The Grange" development is for the subdivision of an agricultural parcel in Moama, to provide approximately 369 new residential lots. The development will be progressed in three stages, indicatively shown in Figure 1-3:

- Phase 1 is for the area shown in yellow in Figure 1-3;
- Phase 2, is for the entire western area; and
- Phase 3 is for the entire eastern area.

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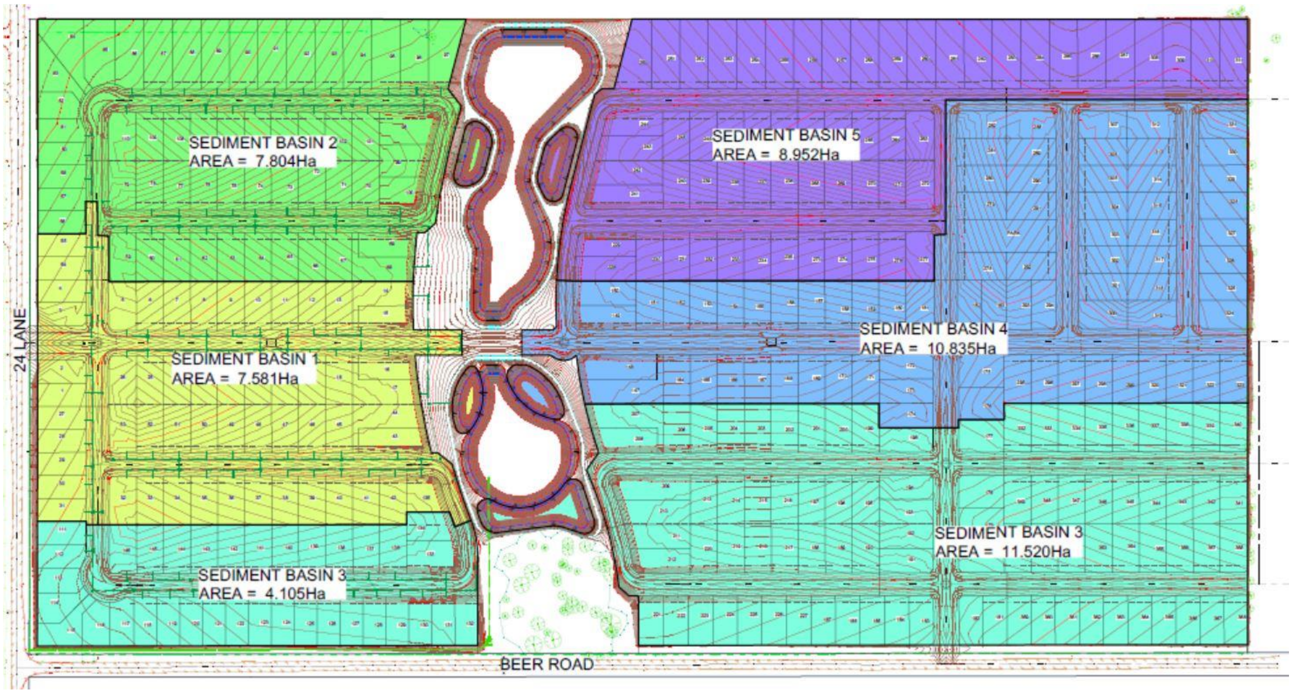

FIGURE 1-3 PROPOSED SUBDIVISION

This new subdivision will be serviced by new road infrastructure, as well as water infrastructure, including "raw" water pipes and stormwater assets. These have been considered in this report, with modelling completed to identify and confirm:

- 1% AEP storage requirements for the subject site, with reference to the *Moama Mid-West Drainage Strategy*;
- Water quality treatment train, with reference to water quality objectives;
- Water yield and water-use potential, to meet bushfire and raw water supply requirements.

1.3 Moama Mid-West Drainage Strategy

RPS were engaged by Murray River Council to develop a strategy for the Moama Mid-West Drainage area, which includes the subject site. The primary objective of this strategic document was to *identify and design (at a concept level) a stormwater management system for the area that aligns with the master plan for the area and Council's stormwater and infrastructure strategies*". This strategic document aims to ensure the implementation and construction of holistic drainage infrastructure to service the planned residential developments in the catchment, in a coordinated manner.

An overview of the proposed drainage infrastructure for this site is shown in Figure 1-4 and this would include:

- Conveyance assets, such as pipe and swales, to convey stormwater runoff to an end-of-line system;
- An end-of-line constructed wetland, to provide water quality treatment, including:
  - Raw water storage (as part of the permanent wetland pool) for local supply to residential estates and public open space.
- Detention basin to capture and retard runoff for the 1% AEP event:
  - 23.5 ML of flood storage (based on the 1 hr duration storage requirements);
  - The detention basin will be controlled by a pump (electric pump in a wet well configuration).

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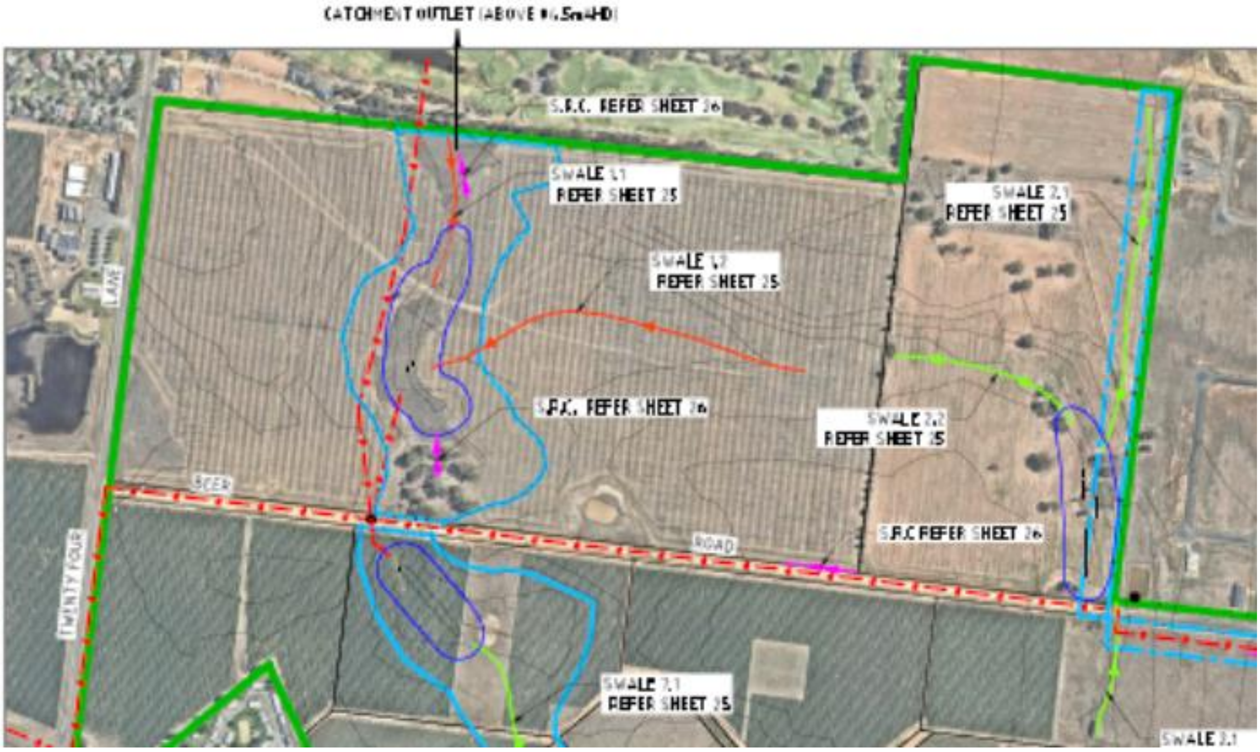

FIGURE 1-4 CATCHMENT 1 (SUBJECT SITE) – STORMWATER CONCEPT DESIGN

The following sections detail the further modelling undertaken to confirm that the drainage infrastructure identified in the *Moama Mid-West Drainage Strategy* (and to be funded by owners, and/or by Council and recovered via s94 Developer's Contributions) is sufficient to meet water quantity and quality objectives.

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2 WATER QUALITY

Stormwater modelling is carried out with regards to best practice industry methods. WSUD Book 2: Planning and Management (Landcom, 2009) outlines the following water quality treatment targets:

- 85% of Total Suspended Sediments (TSS);
- 45% of Total Nitrogen (TN);
- 65% of Total Phosphorous (TP).

As discussed in the *Moama Mid-West Drainage Strategy*, "Total Suspended Solids are the primary risk to the receiving waterway quality, with water quality sampling suggesting nutrient pollution is a lower risk".

The Model for Urban Stormwater Improvement Conceptualisation (MUSIC) was used to determine minimum sizes for the proposed Water Sensitive urban design assets required to meet the above targets. It is appropriate for the design of these asset to be finalised at the functional design stage.

2.1 Modelling Methodology

A schematic of the model is shown in Figure 2-1. The key inputs and assumptions adopted in the water balance modelling were as follows:

- Pluviographic rainfall data from 1995 to 2005 (10 years) from a composite rainfall dataset:
  - Rainfall data from Cobram (about 100 km east):
    - Average (398 mm/year) & seasonal pattern was considered representative of Moama/Echuca conditions (425 mm/year)[1];
  - Local average monthly evaporation values (from a Gridded Areal Potential Dataset (BoM));
- Fraction Imperviousness was modelled as 60%:
  - This is representative of the typical lot size (generally <1,000 m^2);
- Default parameters recommended in NSW MUSIC Modelling Guidelines (2015) were adopted, where appropriate:
  - It was assumed the soils were medium-heavy clay (see Table 2-2).

TABLE 2-2 PERVIOUS AREA RAINFALL-RUNOFF PARAMETERS (MACLEOD, 2008)

Soil texture	SSC (mm)	FC (mm)	Inf "a" (mm/d)	Inf "b"	DRR (%)	DBR (%)	DDSR (%)
Medium-heavy	94	70	135	4.0	10%	10%	0%

- Sedimentation basins incorporated, allowing for:
  - A maximum detention time of less than eight hours (generally between four and five hours);
- Exfiltration rate set to zero, consistent with lined wetland cells:

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[1] We note this is more representative than the Echuca Aerodrome dataset used in the RPS report, which significantly under-estimates annual average rainfall.

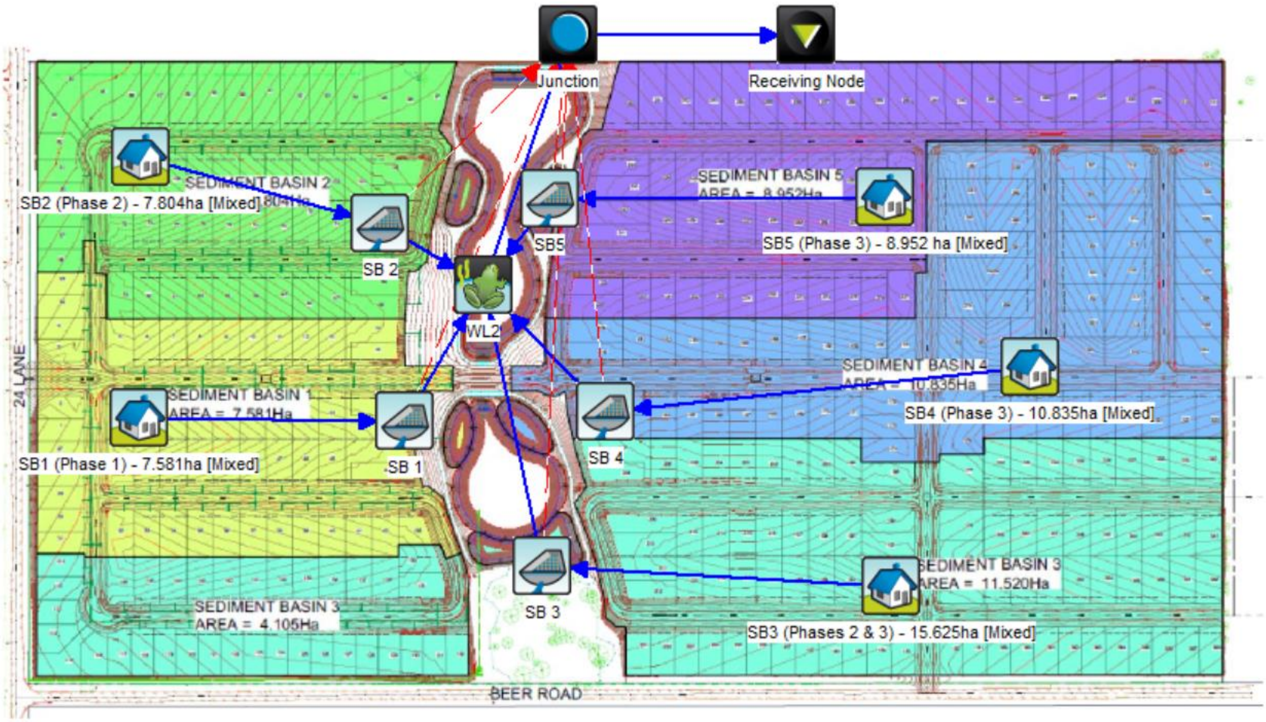

FIGURE 2-1 MUSIC MODEL SCHEMATIC – PHASE 3

2.2 Phase 1

2.2.1 Treatment Train

The treatment train incorporates one sediment basin and one end-of-line wetland:

- **Sediment Basin 1**: a 325 m^2 sediment basin immediately upstream of the stormwater wetland:
  - Calculations indicate that the sedimentation basin will capture 95% of coarse particles (≥ 125 µm diameter) for the peak 3-month ARI flow[2]. This basin would have/require a ~5 year clean-out frequency.
- **Wetland**: A 1,600 m^2 wetland, to provide tertiary treatment.

The main model assumptions are summarised in Table 2-1.

TABLE 2-1 ASSET SIZES – PHASE 1

Catchment (ha)	EDD[3] (m)	SB Label/ID	SB area (m^2)	SB volume (m^3)	WL area (m^2)	WL volume (m^3)
7.58	0.5	SB1	325	180	1,600	640

2.2.2 Treatment Performance

The proposed treatment train ensures the above stormwater management targets are met and exceeded for the proposed development, as shown in Table 2-2. These WSUD assets would also capture more than 90% of gross pollutants generated on site.

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[2] Determined using the Rational Method
[3] Extended Detention Depth

TABLE 2-2 MUSIC MODELLING RESULTS – PHASE 1

Component	Total Load	Residual Load	Reduction (%)
Total Suspended Solids (kg/yr)	3,230	478	**85.2%**
Total Phosphorus (kg/yr)	6.6	1.6	**76.4%**
Total Nitrogen (kg/yr)	45.7	19.9	**56.5%**

2.3 Phase 2

2.3.1 Treatment Train

The treatment train incorporates three sediment basins and one end-of-line wetland:

- **Sediment Basin 1**: a 325 m^2 sediment basin immediately upstream of the stormwater wetland;
- **Sediment Basin 2**: a 325 m^2 sediment basin immediately upstream of the stormwater wetland;
- **Sediment Basin 3**: a 475 m^2 sediment basin immediately upstream of the stormwater wetland;
- **Wetland**: A 3,000 m^2 wetland, to provide tertiary treatment.

The main model assumptions are summarised in Table 2-3.

TABLE 2-3 ASSET SIZES – PHASE 2

Catchment (ha)	EDD (m)	SB Label/ID	SB area (m^2)	SB volume (m^3)	WL area (m^2)	WL volume (m^3)
7.58	0.5	SB1	325	180	3,000	1,200
7.80	0.5	SB2	325	180		
4.11	0.5	SB3	475	320		

2.3.2 Treatment Performance

The proposed treatment train ensures the above stormwater management targets are met and exceeded for the proposed development, as shown in Table 2-4. These WSUD assets would also capture more than 90% of gross pollutants generated on site.

TABLE 2-4 MUSIC MODELLING RESULTS – PHASE 2

Component	Total Load	Residual Load	Reduction (%)
Total Suspended Solids (kg/yr)	8,300	1,190	**85.7%**
Total Phosphorus (kg/yr)	16.6	4.1	**75.2%**
Total Nitrogen (kg/yr)	116.0	55.0	**52.6%**

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2.4 Phase 3

2.4.1 Treatment Train

The treatment train would incorporate five sediment basins and one end-of-line wetland:

- **Sediment Basin 1**: a 325 m^2 sediment basin immediately upstream of the stormwater wetland;
- **Sediment Basin 2**: a 325 m^2 sediment basin immediately upstream of the stormwater wetland;
- **Sediment Basin 3**: a 475 m^2 sediment basin immediately upstream of the stormwater wetland;
- **Sediment Basin 4**: a 390 m^2 sediment basin immediately upstream of the stormwater wetland;
- **Sediment Basin 5**: a 350 m^2 sediment basin immediately upstream of the stormwater wetland;
- **Wetland**: A 9,000 m^2 wetland, to provide tertiary treatment.

The main model assumptions are summarised in Table 2-5.

TABLE 2-5 ASSET SIZES – PHASE 3

Catchment (ha)	EDD (m)	SB Label/ID	SB area (m^2)	SB volume (m^3)	WL area (m^2)	WL volume (m^3)
7.58	0.5	SB1	325	180	9,000	3,600
7.80	0.5	SB2	325	180		
15.6	0.5	SB3	475	320		
10.8	0.5	SB4	390	239		
8.95	0.5	SB5	350	202		

2.4.2 Treatment Performance

The proposed treatment train ensures the above stormwater management targets are met and exceeded for the proposed development, as shown in Table 2-6. These WSUD assets would also capture more than 90% of gross pollutants generated on site.

TABLE 2-6 MUSIC MODELLING RESULTS – PHASE 3

Component	Total Load	Residual Load	Reduction (%)
Total Suspended Solids (kg/yr)	21,600	3,200	**85.0%**
Total Phosphorus (kg/yr)	43.3	10.8	**75.1%**
Total Nitrogen (kg/yr)	304.0	142.0	**53.4%**

2.5 Footprint Assessment

A scaling factor of 2.2 (applied to the sized WSUD asset areas) was adopted to estimate the drainage reserve areas required for the water quality assets. The overall WSUD asset footprint following completion of Phase 3 is ~11,000 m^2, resulting in an overall drainage reserve of ~24,200 m^2.

A preliminary concept terrain design provided by Development Outcomes (shown in Figure 2-2 and Table 2-7) shows the area set aside for the water quality assets. The surface area set aside for the water quality assets is 24,230 m^2, noting that the drainage reserve area at 94.0 m AHD[4] is ~61,300 m^2. This will ensure that there is ample space to design and construct these water quality assets.

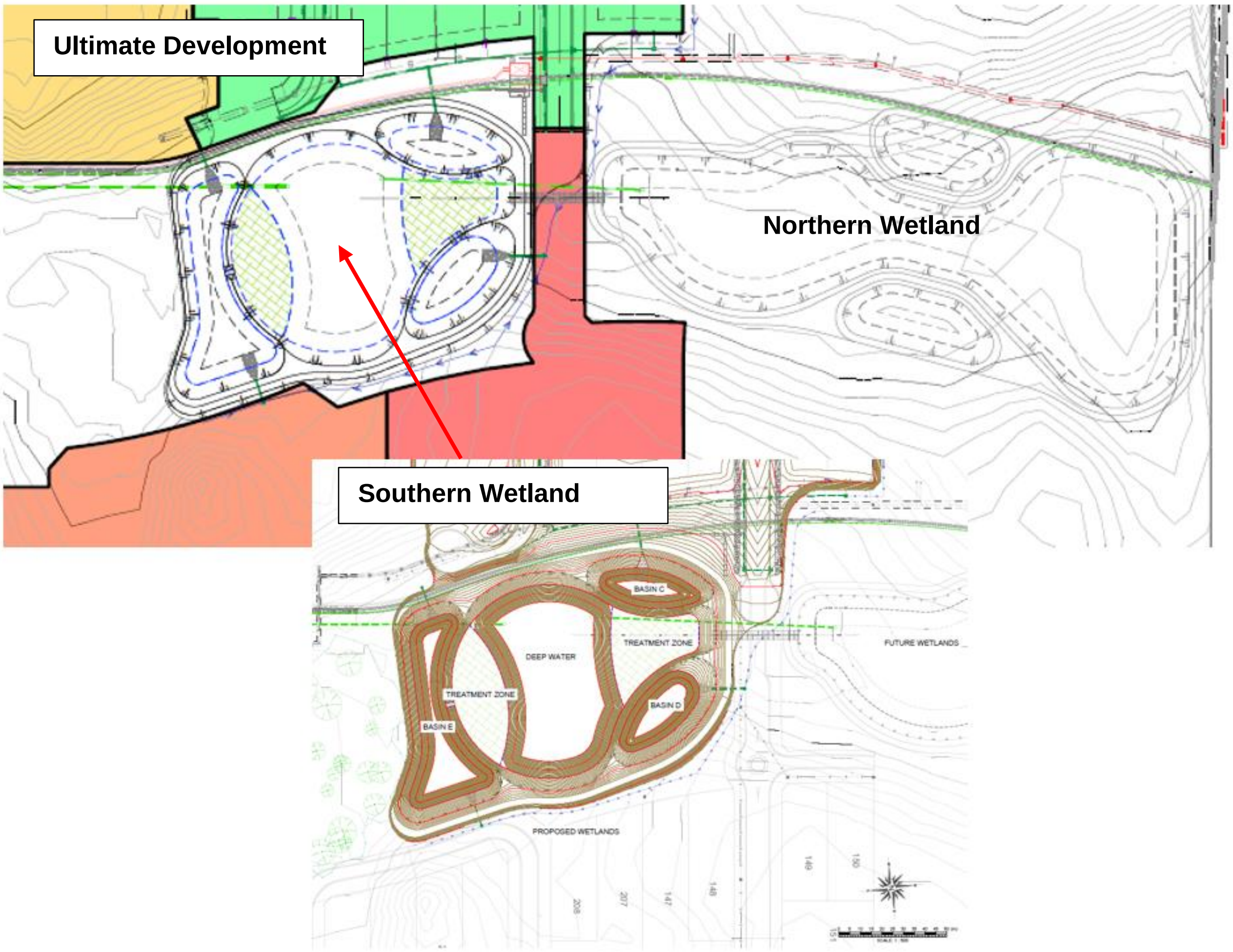

Figure 2-2 WSUD Asset – Indicative Layout (source: Development Outcomes)

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[4] Allowing a minimum of 500 mm freeboard to 1% AEP flood level

Table 2-7 WSUD Asset – Asset Footprint

Asset	Phases	Required Asset area	Reserve Footprint	Over Minimum Asset Size (%)
Southern Wetland	Phases 1, 2 & 3	3,000 m^2	6,769 m^2	126%
Northern Wetland	Phases 2 & 3	6,000 m^2	12,865 m^2	114%
SB1	Phase 1	325 m^2	647 m^2	99%
SB2	Phase 2	325 m^2	682 m^2	110%
SB3	Phases 2 & 3	475 m^2	1,725 m^2	263%
SB4	Phase 3	390 m^2	811 m^2	108%
SB5	Phase 3	350 m^2	732 m^2	109%

2.6 Alternative Design – Floating Wetlands

Floating treatment Wetlands (FWT) - also known as Waterclean Technologies™ - are proprietary treatment systems that rely on a floating treatment media that "hydroponically grows emergent macrophyte plant species in a buoyant matrix floating on the water surface" (Drapper Environmental Consultants, 2014). A schematic of the system is shown in Figure 2-3.

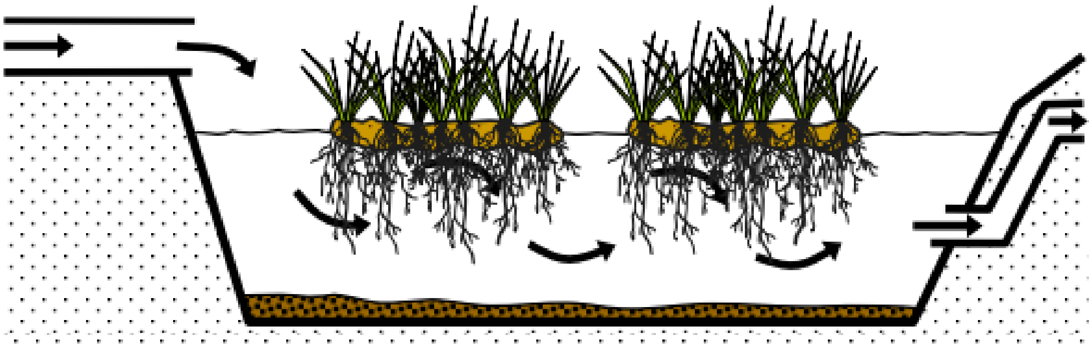

Figure 2-3 Schematic Longitudinal Cross-Section of FTW (source: Headley et al.*, 2008)*

An example of a system is shown in Photo 2-1. Preliminary water quality modelling by SPEL (proprietary supplier of this product) suggests that the overall size of the water quality assets would be reduced compared to a traditional constructed wetland asset (sans its ancillary sedimentation basins). Benefits of these systems include:

- Smaller footprint area compared to conventional wetland with comparable water quality treatment (i.e., system can be designed to meet water quality objectives).
- Larger permanent water volume, potentially increasing raw water storage for local supply to residential estates and public open space;
  - Floating wetland can be incorporated in deeper waterbodies whereas constructed wetlands require 80% of the area of the macrophyte zone to be ≤ 350 mm deep to support shallow and deep marsh vegetation.

It is important to note that the modelling documented in this report show that the end-of-line constructed wetland system is sufficient to meet water quality objectives for the catchment. FWT is only discussed here as an alternative option, that may/can be explored during the detailed design stage. Either option will ensure that there is ample space to design and construct these water quality assets.

Photo 2-1 Example of a Floating Wetland[5]

[5] Source: https://frogenvironmental.co.uk/research-paper/floating-treatment-wetlands-innovative-solution-enhance-removal-fine-particulates-copper-zinc/

3 FLOOD STORAGE

A hydrological RORB model was built to determine the peak 1% AEP flows and to assess the flood storage requirements following the proposed subdivision development detailed in section 1.2. The modelling was undertaken using the latest Australian Rainfall and Runoff rainfall datasets and standards (ARR2019).

Figure 3-1 depicts the proposed layout within the subject site.

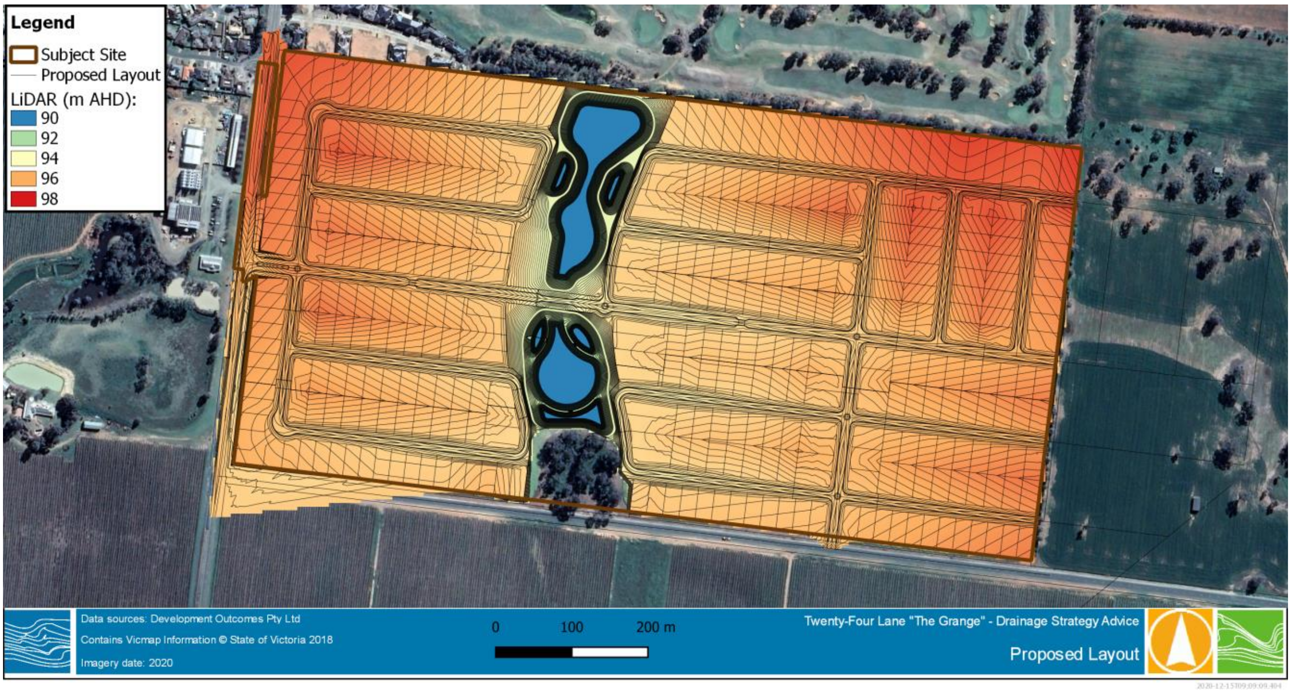

FIGURE 3-1 PROPOSED LAYOUT

The contributing catchment to the subject site was delineated based on the existing terrain surface. The catchment assessment revealed that the study area's catchment encompasses the entire site with negligible contributions from external catchments. Consequently, and for modelling purposes, the subject site was used to outline the catchment boundaries as shown in Figure 3-2. The total catchment area, including the stormwater assets, is 58.3 ha. This is comparable to the catchment area of 59.6 ha adopted in the *Moama Mid-West Drainage Strategy* (RPS, 2019).

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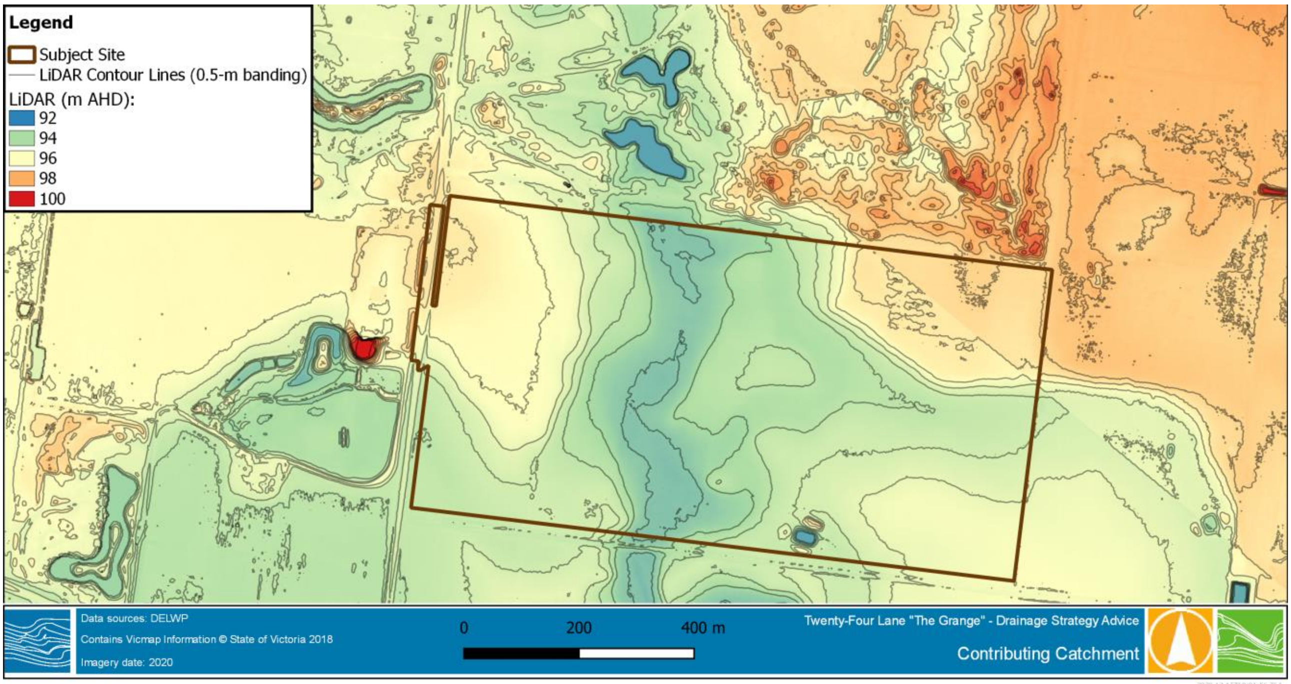

FIGURE 3-2 CONTRIBUTING CATCHMENT FOR MODELLING PURPOSES

3.1 Modelling Methodology

The methodology followed a tiered modelling approach to account for the proposed staging. For each construction phase, a distinct RORB model was built. Inputs were obtained from the existing conditions and proposed layout for pre-developed and developed areas, respectively. Outputs of interest were the 1% AEP design flow and its corresponding critical duration, as well as the maximum and average flood volume for the 1 hour and 72 hours 1% AEP storm durations.

Below is a summary of the key inputs and assumptions adopted in the hydrological modelling:

- The adopted K_c parameter for all scenarios was 0.92:
  - Determined using Equation 7.6.13 of ARR2019, Book 7, Chapter 6.2.1.2, recommended for NSW catchments:
    - $K_c = 1.18A^{0.46}$
- Fraction Imperviousness (FI or TIA for this purpose) was conservatively modelled as 70%, thereby increasing runoff volumes relative to the assumptions outlined in section 2. The catchment fractions were determined for the following three urban surface types:
  - Effective Impervious Area (EIA) – EIA also known as directly connected impervious surfaces, corresponds to the area of an urban catchment that contributes to a rapid runoff response to rainfall events. It is largely made up of impervious areas directly connected to piped or channelised drainage systems. Its value is commonly calculated using the following formula:
    - $EIA = 0.6 * TIA$ (ARR2016, Book 6, Chapter 3.4.2.2.2) when TIA ≤ 80%.
    - $EIA = 0.6\ to\ 1 * TIA$ when TIA > 80% as per EIA/TIA ratio relationship presented in Figure 3-3.

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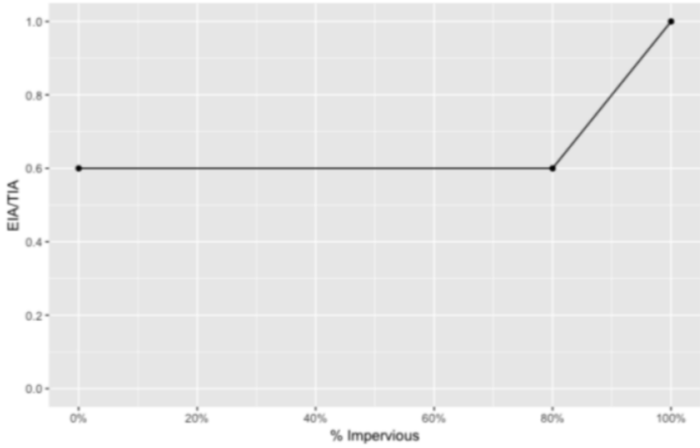

FIGURE 3-3 EIA/TIA RATIO INCREASE FOR HIGHLY IMPERVIOUS CATCHMENT

- Pervious Area (PA) – PA consist of parks and bushland areas which have been independently assessed by aerial photography or site inspections. Its value is determined with the following formula:
  - $PA = 1 - TIA$
- Indirectly connected Area (ICA) – ICA includes impervious surfaces that are not directly connected to drainage systems and pervious areas whose runoff does not enter the drainage system. Its value is computed using the following formula:
  - $ICA = 1 - PA - EIA$

- Developed regions were assigned the FI indicated above whereas undeveloped zones were assigned a FI of 0. By contrast, the FI of areas occupied by the stormwater assets (SB and wetland) was set to 1.
- Initial and continuing losses were determined based on the ARR2019, Book 5, Chapter 3.5.3. Pervious area losses for rural catchments were extracted from the AR2019 Data Hub.
  - EIA
    - The initial and continuing losses were obtained from ARR2019 (Book 5, Chapter 3.5.3.1.2), where it was recommended to adopt an EIA initial loss between 1 to 2 mm (**1.5 mm** was chosen for this study) and continuing loss of **zero**.
  - ICA
    - The ARR2019, Book 5, chapter 3.5.3.2.1 recommends the initial loss for ICA to range between 60 and 80% of the rural initial loss (22 mm). Therefore, for this study, ICA initial loss was assumed to be 70% of the aforementioned loss ($22 * 0.7 = \mathbf{15.4\ mm}$).
    - **2.5 mm/hr** is the typical ICA continuing loss according to ARR2019, Book 5, Chapter 3.5.3.2.2.
  - PA
    - The rural IL and CL are 22 mm and 2.1 mm/hr, respectively (ARR2019 Data Hub). Nevertheless, IL from the data hub is for complete storms rather than burst. Given the fact that the rainfall modelled in RORB are bursts, the rural initial losses must be reduced by the pre-burst rainfall corresponding to the median pre-burst rainfall depth (mm) from the data hub.
    - The median pre-burst rainfall depth (mm) varies across the storm durations. 0.6 mm burst rainfall depth was selected and subtracted from the rural initial loss (22 mm) to derive the burst PA initial loss ($22 - 0.6 = \mathbf{21.4\ mm}$).

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- The rural continuing loss (2.1 mm/hr) is based on a 1-hour timestep. For RORB simulations between 10 minutes and 12-hour, the timestep ranges from 5 to 30 minutes; thus, less than 1 hour. As a result, the continuing loss was increased by a scaling factor of 1.5 to derive the PA continuing loss ($2.1 * 1.5 = \mathbf{3.15\ mm/hr}$).

- Table 3-1 summarises the IL and CL used in the modelling.

TABLE 3-1 INITIAL AND CONTINUING LOSSES ASSUMPTIONS

Surface type	Initial Loss (mm)	Continuing Loss (mm/hr)
EIA	1.5	0
ICA	15.4	2.50
PA	21.4	3.15

- The reach types in the RORB model were set as followed:
  - "Natural" for runoff generated and conveyed in undeveloped areas.
  - "Lined channel or pipe" for runoff being conveyed along developed areas.
  - "Dummy" for linking the sediment basins to the wetland.
  - "Drowned" to account for routing across key end-of- line stormwater assets.
- RORB models were run using the ARR2019 IFD data for the 1% AEP event across the ensemble of temporal patterns and a range of durations (from 10 min to 72-hour duration).

The following sections provide a comprehensive summary for each phase.

3.2 Phase 1

This phase accounts for 26% of the urban development. Figure 3-4 depicts the Phase 1 RORB model.

The 1% AEP design flows at the site outfall and the flood volumes for a range of storms are presented in Figure 3-5.

The 1% AEP critical duration was found to be the 20-minutes storm with a **peak mean flow of 4.23 m^3/s** (mean of the maximum peak flows which does not correspond to any specific temporal pattern). The 1% AEP **peak flow of 4.29 m^3/s** corresponds to the closest peak flow for the 1% AEP 20-minute temporal pattern (Temporal Pattern 27). The average and maximum flood volumes for the 1-hour storm are 16.3 and 16.6 ML respectively, increasing to 24.5 and 37.5 ML for the 72-hour storm.

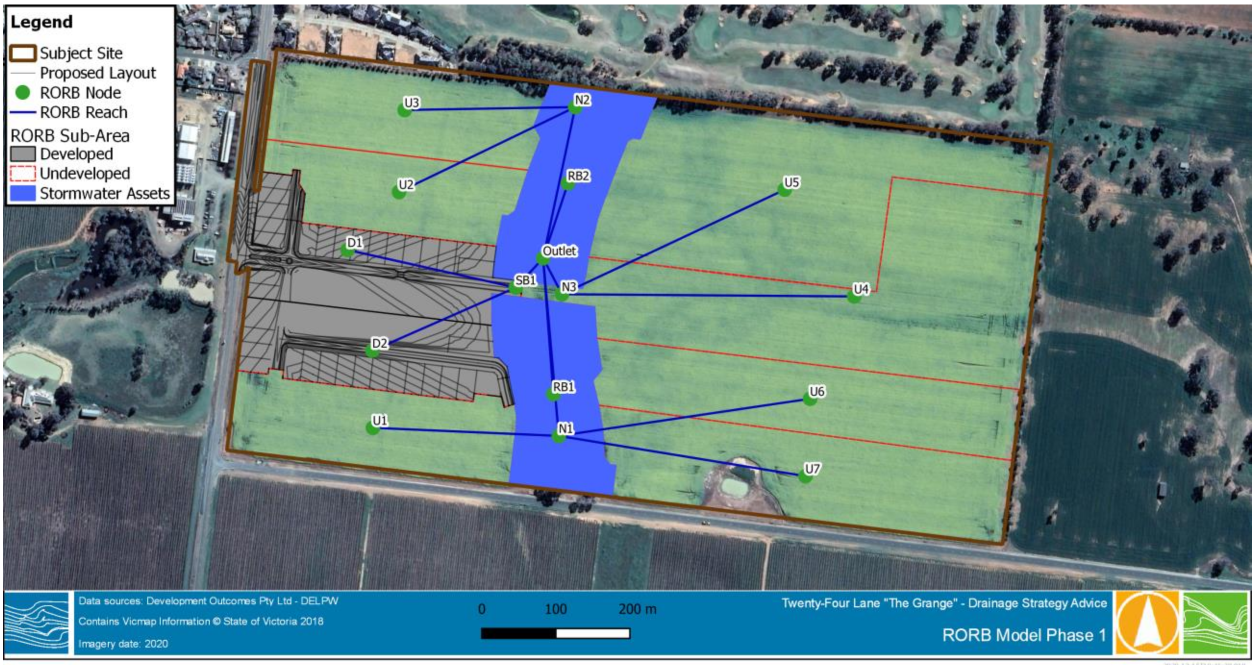
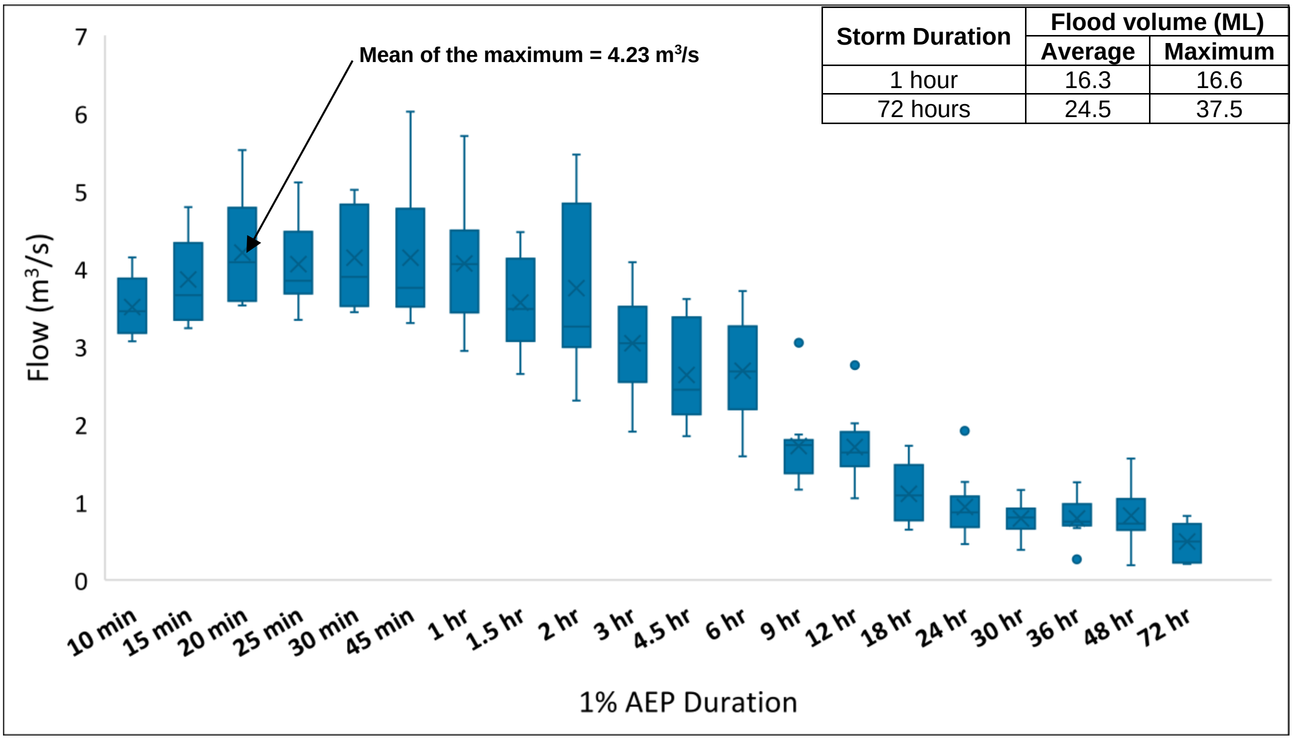


FIGURE 3-4 RORB MODEL FOR PHASE 1



Storm Duration	Flood volume (ML)	
	Average	Maximum
1 hour	16.3	16.6
72 hours	24.5	37.5

FIGURE 3-5 1% AEP PEAK FLOW AND FLOOD VOLUMES FOR PHASE 1

3.3 Phase 2

This phase accounts for 46% of the urban development. Figure 3-6 depicts the Phase 2 RORB model.

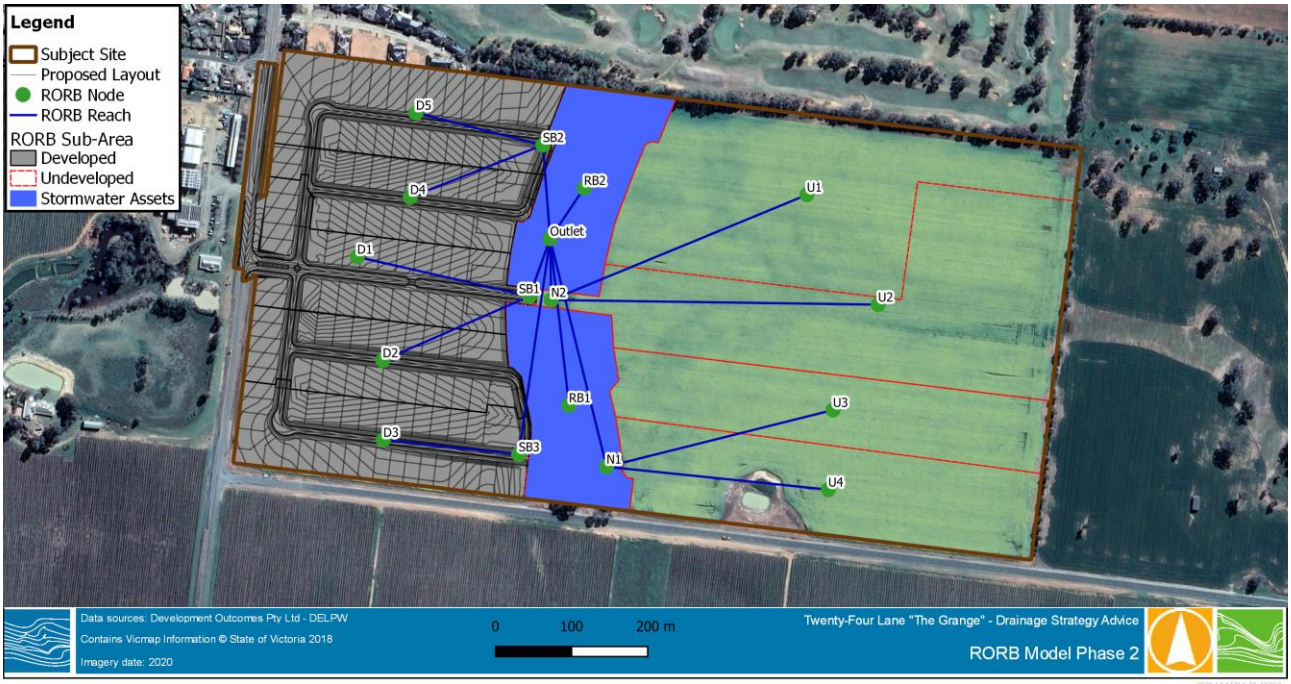


FIGURE 3-6 RORB MODEL FOR PHASE 2

The critical 1% AEP design flow at the site outfall and the flood volumes for the 1-hour and 72-hour storms are presented in Figure 3-7. The 1% AEP critical duration was found to be the 20-minutes storm with a **peak mean flow of 6.30 m^3/s** (mean of the maximum peak flows which does not correspond to any specific temporal pattern). The 1% AEP **peak flow of 6.34 m^3/s** corresponds to the closest peak flow for the 1% AEP 20-minute temporal pattern (Temporal Pattern 21). The average and maximum flood volumes for the 1-hour storm are 17.6 and 17.8 ML respectively, increasing to 28.8 and 42.0 ML for the 72-hour storm.

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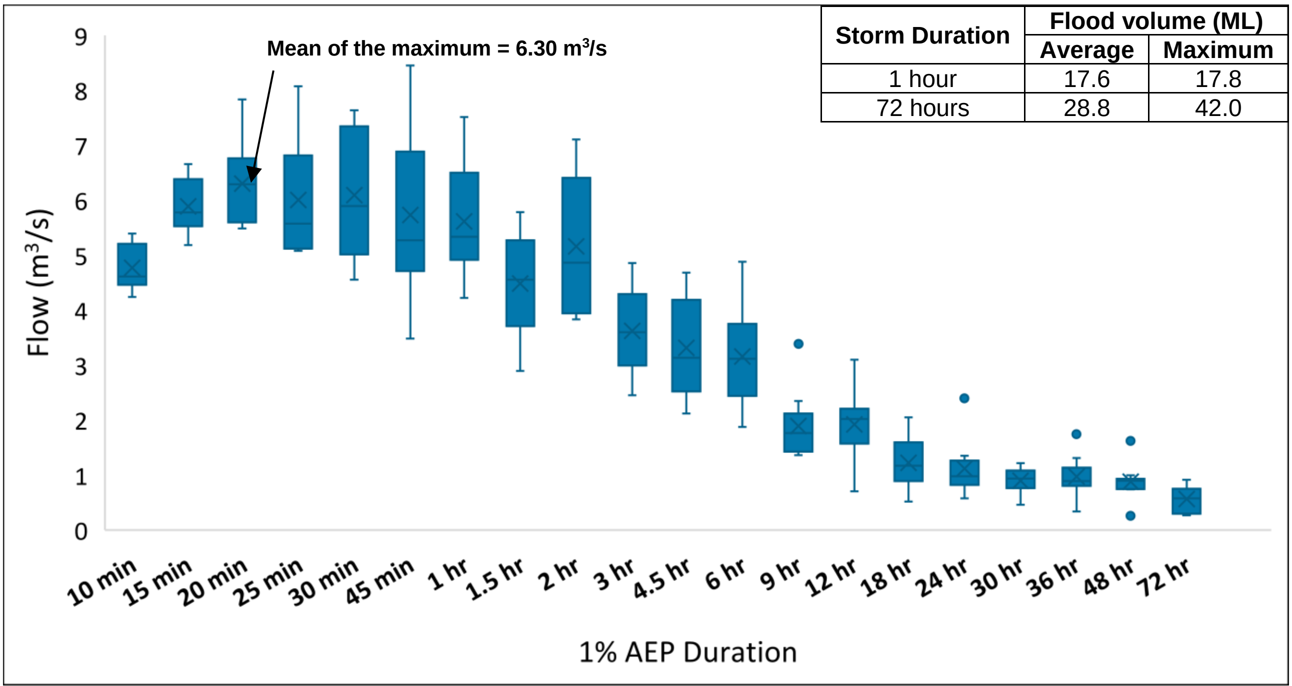

Storm Duration	Flood volume (ML)	
	Average	Maximum
1 hour	17.6	17.8
72 hours	28.8	42.0

FIGURE 3-7 1% AEP PEAK FLOW AND FLOOD VOLUMES FOR PHASE 2

3.4 Phase 3

This phase accounts for 100% of the urban development. Figure 3-8 depicts the Phase 3 RORB model.

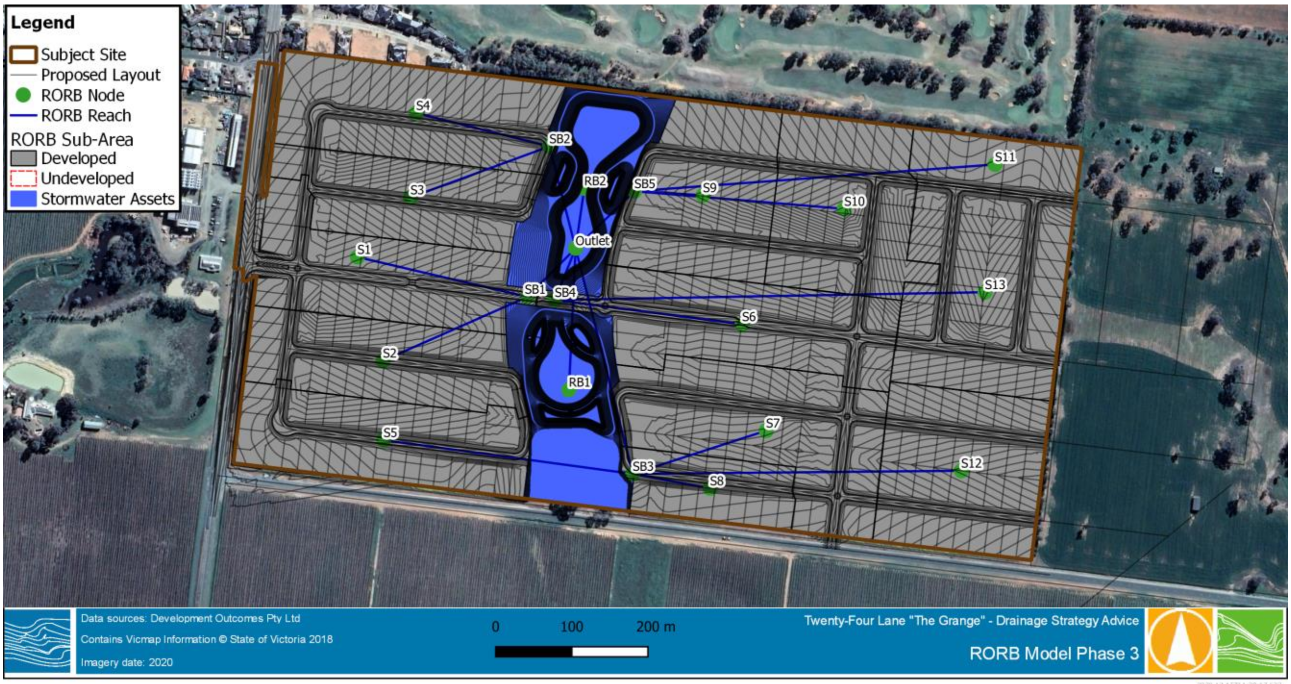

FIGURE 3-8 RORB MODEL FOR PHASE 3

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The critical 1% AEP design flow at the site outfall and the flood volumes for the 1-hour and 72-hour storms are presented in Figure 3-9. The 1% AEP critical duration was found to be the 20-minutes storm with a **peak mean flow of 10.57 m³/s** (mean of the maximum peak flows which does not correspond to any specific temporal pattern). The 1% AEP **peak flow of 10.73 m³/s** corresponds to the closest peak flow for the 1% AEP 20-minute temporal pattern (Temporal Pattern 27). The average and maximum flood volumes for the 1-hour storm are 20.9 and 21.1 ML, respectively. This is comparable but lower than then 1-hour flood storage reported in the *Moama Mid-West Drainage Strategy* (RPS, 2019). The average and maximum flood volumes increase to 40.1 and 53.9 ML for the 72-hour storm.

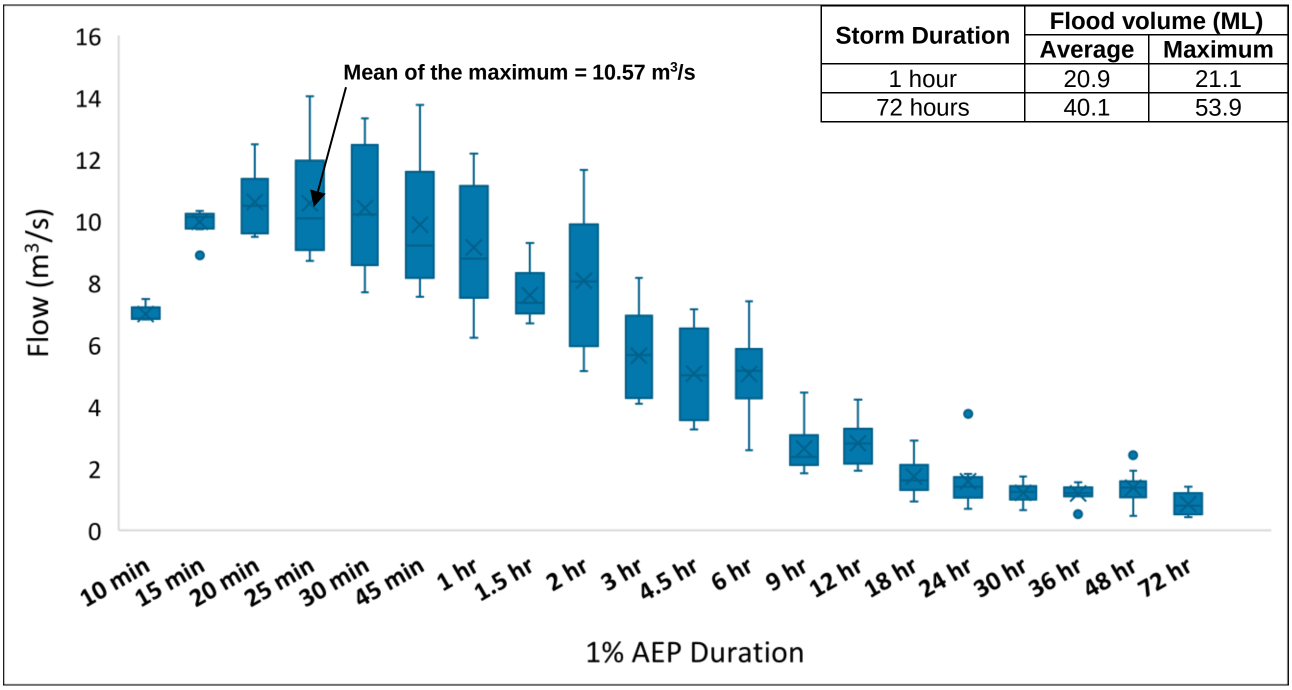

FIGURE 3-9 1% AEP PEAK FLOW AND FLOOD VOLUMES FOR PHASE 3

3.5 Volume Assessment

Preliminary terrain (12D) analysis, shown in Table 3-2, shows the available flood storage within the central drainage reserve. There is ample volume to cater for the anticipated 1% AEP flood volume:

- There is over 30,000 m³ of volume between 92.5 m AHD and 93.5 m AHD:
  - This exceeds the detention volume specified in RPS' *Moama Mid-West Drainage Strategy* (23,500 m³) and outlined in the above sections (~21,000 m³) for the 1% AEP 1 hour event;
  - This assessment ignores any Extended Detention Volume (~13,000 m³) above the Normal Water Level of the water quality assets (i.e., between 92.0 and 92.5 m AHD);
  - This assumes 1 m freeboard to the 1% AEP flood level (i.e., 94.5 m AHD);
- There is over 56,000 m³ of volume between 92.5 m AHD and 94.0 m AHD:
  - This exceeds the detention volume outlined in the above sections (~54,000 m³) for the 1% AEP 72 hour event;
  - This would still allow 0.5 m freeboard to the 1% AEP flood level, with freeboard volume estimated at about 32,000 m³.

TABLE 3-2 VOLUMETRIC ANALYSIS – ULTIMATE RETARDING BASIN (SOURCE: DEVELOPMENT OUTCOMES)

Elevation, m, AHD	94.5	94	93.5	93	92.5	92	91	90	COMMENT
Surface area, closed contour, m2	66654	57340	37015	32525	28230	24233	18599	13472	
Volume, m3, between 94.5m - 94.0m AHD		30998.5							STORAGE BETWEEN 94.5m AHD AND 93.5m AHD = 54,600 m3. STORAGE BELOW 94.5m AHD INCLUDING FEE AIRSPACE FROM 100 YEAR LEVEL TO 93.5m AHD = 54,600 m3 + 22,700 m3 = 77,300 m3.
Volume, m3, between 94.0m - 93.5m AHD			23588.75						
Volume, m3, between 93.5m - 93.0m AHD				17385					100 YEAR STORAGE BETWEEN 92.0m AHD AND 93.5m AHD = 45,700 m3, MINIMUM VOLUME = 23,000 m3 AS REQUIRED IN MOAMA MID WEST DRAINAGE STRATEGY. BALANCE OF 22,700 m3 CONTRIBUTES TO AVAILABLE AIR SPACE FOR FLOWS EXCEEDING 100 YEAR STORM.
Volume, m3, between 93.0m - 92.5m AHD					15188.75				
Volume, m3, between 92.5m - 92.0m AHD						13115.75			
Volume, m3, between 92.0m - 91.0m AHD							21416		EPHEMERAL STORAGE BETWEEN 92.0m AHD AND 90.0m AHD, VOLUME TO 92.0m AHD = 37,500 m3
Volume, m3, between 91.0m - 90.0m AHD								16035.5	

The additional hydrological modelling undertaken as part of this study confirms that there is ample volume (airspace storage) to cater for 1% AEP event runoff, even in the event of prolonged pump failure.

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4 WATER SECURITY

4.1 Bushfire

In line with the NSW Rural Fire Service, and as highlighted in clause 12 of the DC conditions, the proposed development must comply with the following water supply requirement for non-reticulated developments (or where reticulated water supply cannot be guaranteed), as shown in Table 4-1.

TABLE 4-1 WATER SUPPLY REQUIREMENT (SOURCE: NSW RURAL FIRE SERVICE)

DEVELOPMENT TYPE	WATER REQUIREMENTS
Residential lots (<1,000m²)	5,000L/lot
Rural-residential lots (1,000-10,000m²)	10,000L/lot
Large rural/lifestyle lots (>10,000m²)	20,000L/lot
Multi-dwelling housing (including dual occupancies)	5,000L/dwelling

Table 4-2 summarises water requirements to meet NSW Rural Fire Service requirements.

TABLE 4-2 NSW RURAL FIRE SERVICE VOLUME REQUIREMENTS

Phase	Stages	Lots	RFS Required Vol
Phase 1	Stages 1 and 2	43 Lots (<1,000m²) 10 Lots (1,001-1,500m²)	265 kL
Phase 2	Stages 3, 4 and 5	107 Lots (<1,000m²) 25 Lots (1,001-1,500m²) 14 Lots (1,501-2,000m²)	800 kL
Phase 3	Stages 6 to 13	257 Lots (<1,000m²) 90 Lots (1,001-1,500m²) 21 Lots (1,501-2,000m²)	2,020 kL

These volumes are largely met by the proposed southern waterbody (as a standalone asset) as shown in Table 4-3. Importantly, this asset (see Figure 2-2) will be constructed to service Phase 1 and form part of the overall treatment for Phases 2 & 3. The Hydrant suction pump will be designed to extract water from this first pond, noting that the asset servicing Phase 3 would provide additional water volume. Given the fact that the proposed wetland volume largely exceeds the RFS water supply, the bushfire water supply requirement is achieved on-site for each phase.

TABLE 4-3 VOLUMETRIC ANALYSIS – SOUTHERN POND (SOURCE: DEVELOPMENT OUTCOMES)

SOUTH								
Elevation, m, AHD	94.5	94	93.5	93	92.5	92	91	90
Surface area, closed contour, m2	34047	28954	15126	13334	11575	9953	7426	5185
Volume, m3, between 94.5m - 94.0m AHD		15750.25						
Volume, m3, between 94.0m - 93.5m AHD			11020					
Volume, m3, between 93.5m - 93.0m AHD				7115				
Volume, m3, between 93.0m - 92.5m AHD					6227.25			
Volume, m3, between 92.5m - 92.0m AHD						5382		
Volume, m3, between 92.0m - 91.0m AHD							8689.5	
Volume, m3, between 91.0m - 90.0m AHD								6305.5

4.2 “Raw” Water Supply

In addition to the NSW Rural Fire Service requirements, and as per clause 38 of the DC conditions, the proposed development must provide filtered and raw water supply to each allotment in accordance with the approved design. As such, using an average daily demand of 1.89 KL/lot (as defined by Council), the annual usage per phase is calculated and contrasted against the catchment’s stormwater annual yield (Table 4-4).

TABLE 4-4 RAW WATER ANNUAL USAGE AND CATCHMENT ANNUAL YIELD

Phase	Lots	Average daily demand (KL/day/lot)	Annual usage (ML/year)	Catchment annual yield (ML/year)
1	53	1.89	36.6	13.7
2	146	1.89	100.7	35.9
3	368	1.89	253.9	92.6

It can be seen from the table that the annual stormwater yield from the development is insufficient to meet raw water demand. The water balance analysis shows that it would only be possible to meet 40% of the latter (ignoring any impact on wetland health).

The Council pump station on the Murray River has a maximum pumping capacity of 35ML/week (maximum 5ML per day) and the current demand on this pump station is 25ML/week (in peak). The pump capacity therefore exceeds demand by about 10 ML/week. Such spare capacity is more than double what is required for raw water demand in Phase 3 (4.8ML/week). Consequently, this deficit could be addressed by pumping water from Murray River and, if required, storing it in the wetland, at least in the immediately short term until the new private estate subdivision infrastructure is active in Phase 2.

5 SUMMARY

Water Technology were engaged by Development Outcomes (DO) to assist with the development of the drainage strategy for the proposed development at Twenty-Four Lane "The Grange" Moama.

Water Technology provided drainage advice for the proposed subdivision; hydrological and water quality modelling was completed to ensure the drainage design meets the intent of *Moama Mid-West Drainage Strategy* (RPS, 2019) and, ultimately, to help gain Murray River Council's endorsement of the drainage system design.

The modelling, both for stormwater quality and quantity management, showed that there is ample space within the proposed central reserve to cater for the required drainage assets. These assets will ensure that the objectives of the *Moama Mid-West Drainage Strategy* are met, if not exceeded, at the subject site.

The water security analysis proved that the development meets water security for both NSW Rural Fire Service requirements and raw water supply. Regarding the latter, however, the spare capacity of the existing pump station must be used to complement the catchment's stormwater annual yield, in the immediately short term until the new private estate subdivision infrastructure is active in Phase 2..

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