Appendix A

WGA - Stormwater Management - Water, Wastewater and Recycled Water

WGA

Walker Corporation

Riverlea Park

STORMWATER MANAGEMENT - WATER, WASTEWATER AND RECYCLED WATER

WGA080163

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April 2023

Revision History

REV	DATE	ISSUE	ORIGINATOR	CHECKER	APPROVER
A	Nov 08	For Client Review	JME	DB	DB
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F	Dec 22	Final Issue	FL	DB	DB
G	April 23	Addressed EPA/DEW comments	GL	DB	DB

CONTENTS

Figures

Tables

Appendices

1 INTRODUCTION

1.1 Introduction

This is an updated version of the Technical Paper prepared in 2009 to take into account the following changes to the development proposal:

- Introduction of a Salt Water Lake scheme within the development to provide amenity but to also provide stormwater detention for a significant component of the site.
- Changes to the Gawler River Flood Model, and updating the 100 year ARI floodplain mapping based on the updated model and introduction of the salt water lakes.
- Updates by SA Water in regards to the Potable Water Supply to the development.
- Updates by SA Water in regards to the provision of irrigation (recycled water) to the site, as a result of the recent construction of the Northern Adelaide Irrigation Scheme (NAIS)

1.2 Background

The Riverlea development by Walker Corporation, which is now currently under construction with Precinct 1 and sections of Precinct 2 under construction. It will comprise approximately 12,000 residential allotments, a number of commercial and industrial precincts, three permanent neighbourhood centres, one district centre, one retail centre and both primary and high schools, local shopping areas and employment opportunities. Figure 1-1 below shows the Masterplan layout of the proposal.

Figure 1-1: Riverlea Park Proposal Masterplan

Construction of the proposal will be staged over a 25 year period. The provision of infrastructure (such as the stormwater, potable water and waste water) will also be staged, and constructed as demand requires it. Therefore, capital costs associated with implementation of infrastructure will be progressive over the 25 year construction period.

Figure 1-2 shows the current Riverlea Park proposal staging plan developed so far. The intention is for development to progressively move from the east to the west as that is the logical path to bring infrastructure to the site.

Figure 1-2: Riverlea Park Proposal Staging Plan

1.3 Planning and Design Code

The Riverlea site is within the City of Playford and is zoned Masterplanned Neighbourhood.

As a result of the area’s horticultural character, the Riverlea Park are currently has no major water or sewer trunk services available, however recycled water is currently supplied to the residents for irrigation and horticultural purposes via the WRSV (Western Reticulation Systems Virginia) pipeline and more recently through the extension of the Northern Adelaide Irrigation System (NAIS), which has a pipeline in Port Wakefield Highway.

1.4 Site Description

The Riverlea Park site covers an approximate area of 1,308 hectares. The site is situated approximately 32km north of the Adelaide CBD, bounded by Gawler River to the north, Buckland Dry Creek salt fields to the south, Port Wakefield Highway to the east (see Figure 1-3 for the locality plan). The Riverlea Park site is approximately 2.7 kilometres inland of the Gulf St Vincent coastline and it is for this reason it is not considered to be a coastal site.

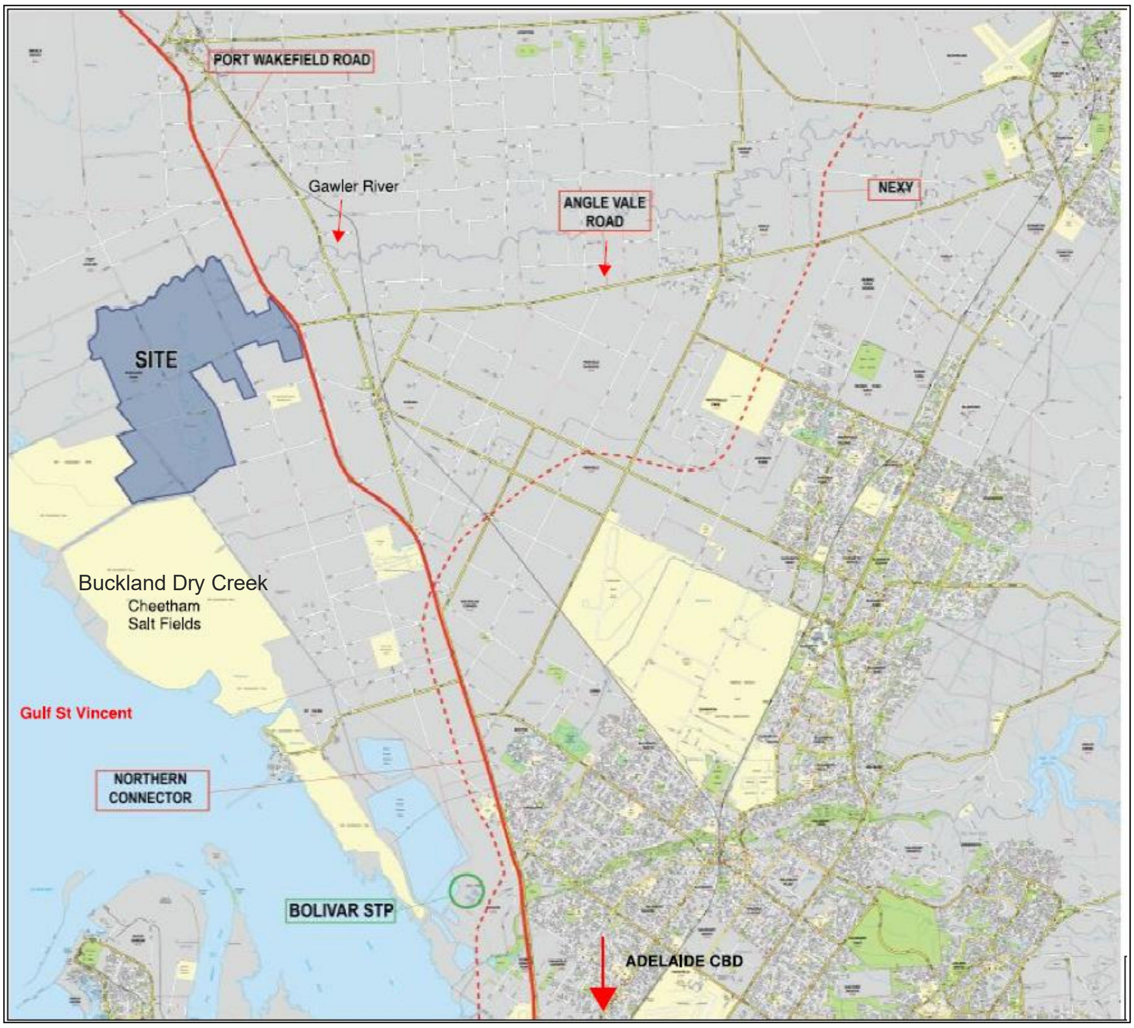

Figure 1-3: Locality Plan

The topography of the site is relatively flat with an approximately fall of 0.2% across the site from east to west. The site also lies within the Gawler River flood plain. Figure 1-4 shows the site location in relation to the surrounding community.

Figure 1-4: Site Boundary in Context of Surroundings

As a part of the initial site investigations ground water mapping was undertaken by Resource and Environmental Management (REM) (Reference 7). This mapping indicated that the depth to ground water within the site ranges from 0.2 metres to 7 metres below the natural surface level. It can be seen in Figure 1-5 that approximately 75% of the site has a depth to ground water of approximately 3 metres below the surface level.

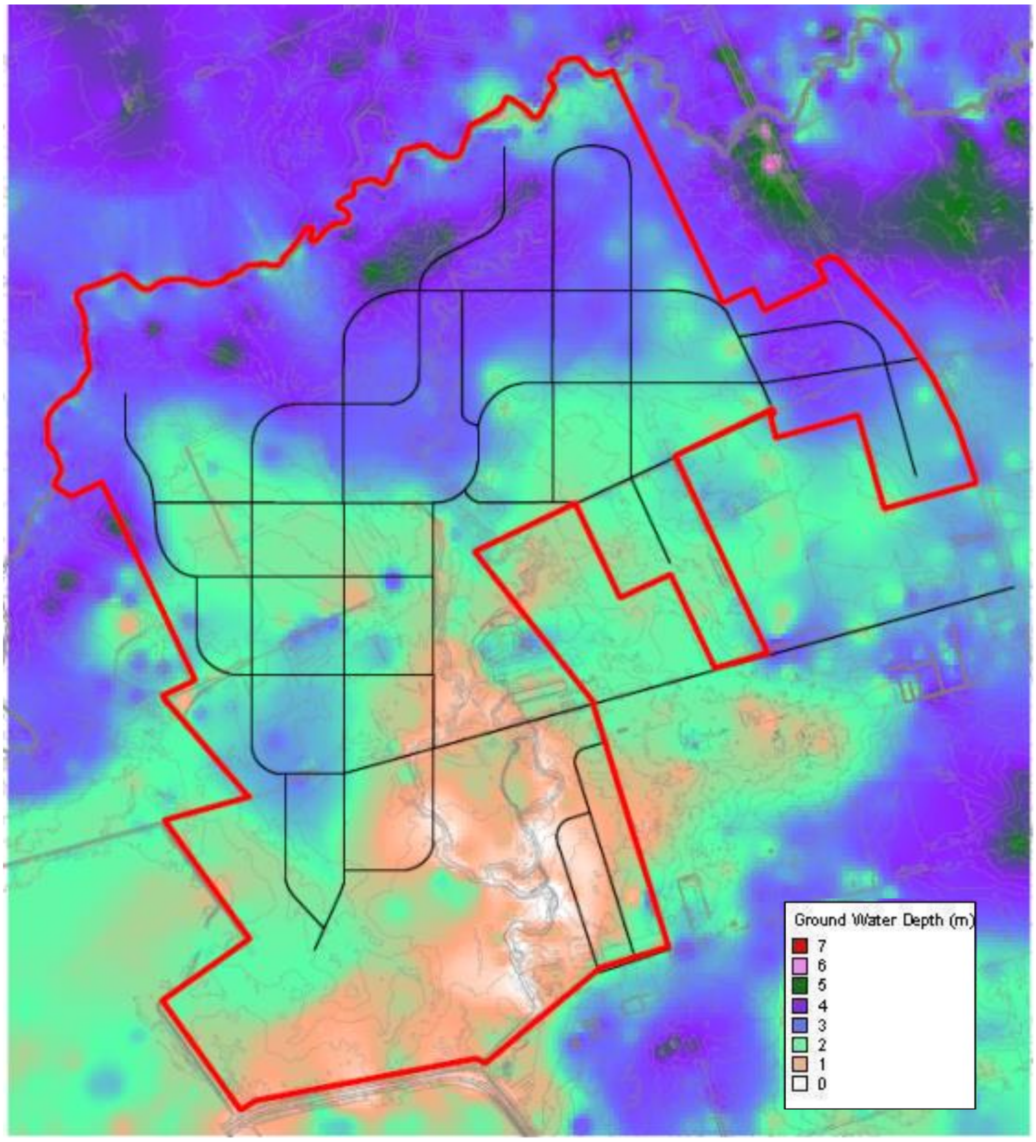

Figure 1-5: Depth to Groundwater

Site investigations by both Golder Associates (Reference 5) and REM (Resource and Environmental Management, reference 7) revealed the ground water in the Riverlea Park area is highly saline, with the salinity ranging from 1000ppm to 5000ppm Total Dissolved Solids (TDS). These investigations also indicated that some portions of the site are affected by Acid Sulphate Soils (ASS).

1.5 Water Management Aims

This technical paper outlines the formulation of the following concepts as they relate to the Riverlea Park proposal:

- Stormwater capture, treatment and reuse (minor flow management)
- Stormwater Management (major flow management)
- Sewerage reticulation systems
- Potable water supply
- Flood protection from Gawler River
- Provision of Recycled Water (NAIS) to the site.

These concepts will be discussed in relation to site conditions and how they influence the recommendations for water infrastructure and the layout of the proposal's Masterplan – particularly the location and configuration of stormwater management facilities.

The EIS Guidelines that will be addressed in this report are outlined in Appendix A.

2 STORMWATER

2.1 Introduction

The current method of stormwater management within the Riverlea Park site relies on a system of natural open creek lines and roadside open drains and culverts to move the stormwater runoff through the catchment and discharge it to the ocean via the Thompson Outfall Channel.

The Riverlea Park site generally drains away from the Gawler River in a south westerly direction towards the Thompson Outfall Channel. The Gawler River is situated within the northern section of the Riverlea Park site and is a perched river system. As the banks of the Gawler River are higher than the adjacent floodplain, stormwater runoff from the Riverlea Park site will not drain to the Gawler River nor to the Buckland Lake System as they are both effectively located upstream of the Riverlea Park proposal site.

Figure 2-1 shows the site levels in metres to Australian Height Datum (AHD) and shows that the site falls away from the Gawler River towards the Thompson Outfall Channel.

Section 2 of this report will focus primarily on minor and major internal stormwater flow management whilst water quality and the management of external flood water flows will be addressed in Sections 3 and 4 respectively.

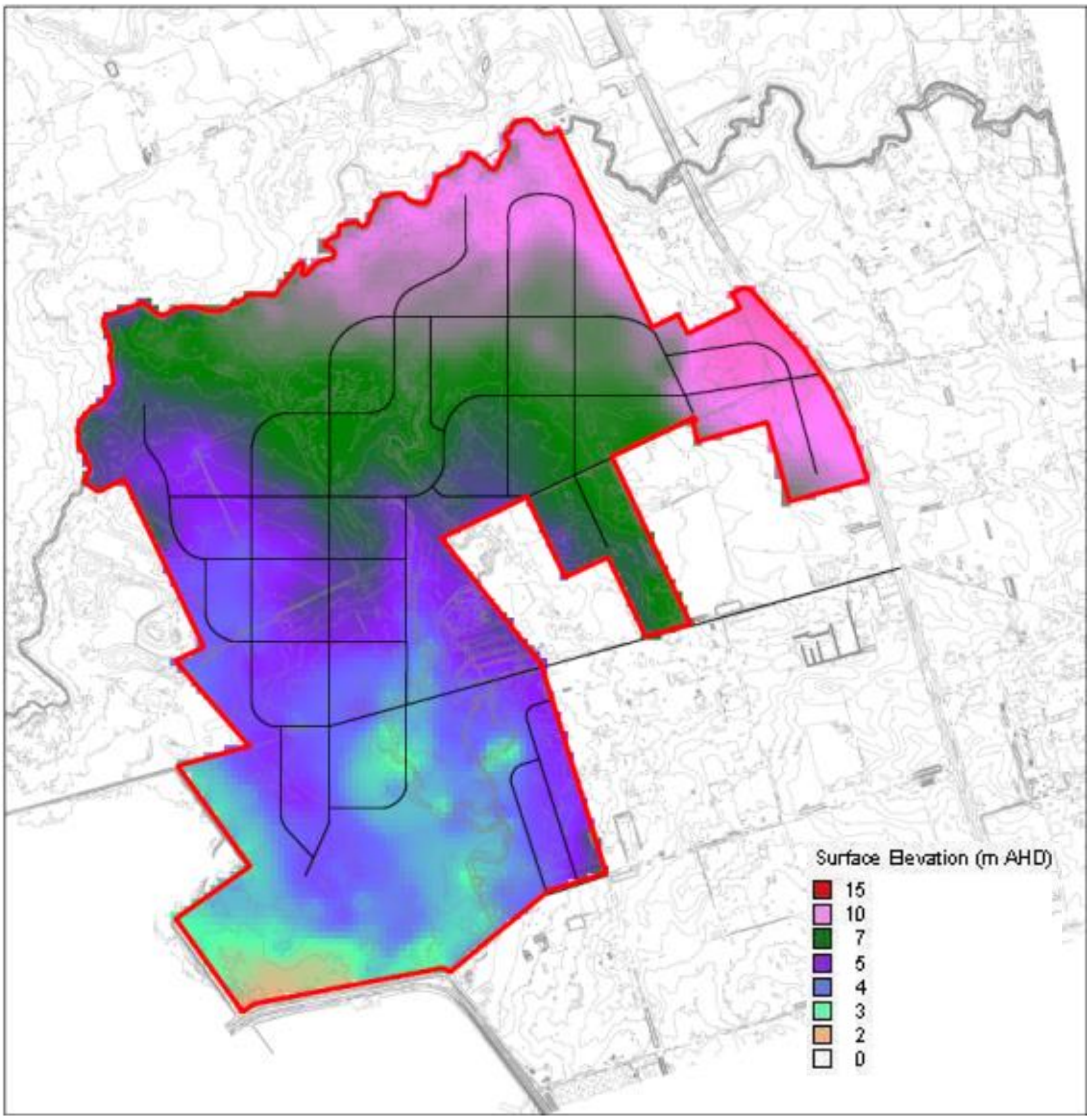

Figure 2-1: Existing Site levels

2.2 Pre-Development Site Conditions

Currently stormwater infrastructure in the Riverlea Park area is limited. The majority of the stormwater flows are carried by a system of natural creek lines, culverts and open drains that run along the road side and discharge to the Thompson Outfall Channel (see Figure 2-2 for stormwater infrastructure layout).

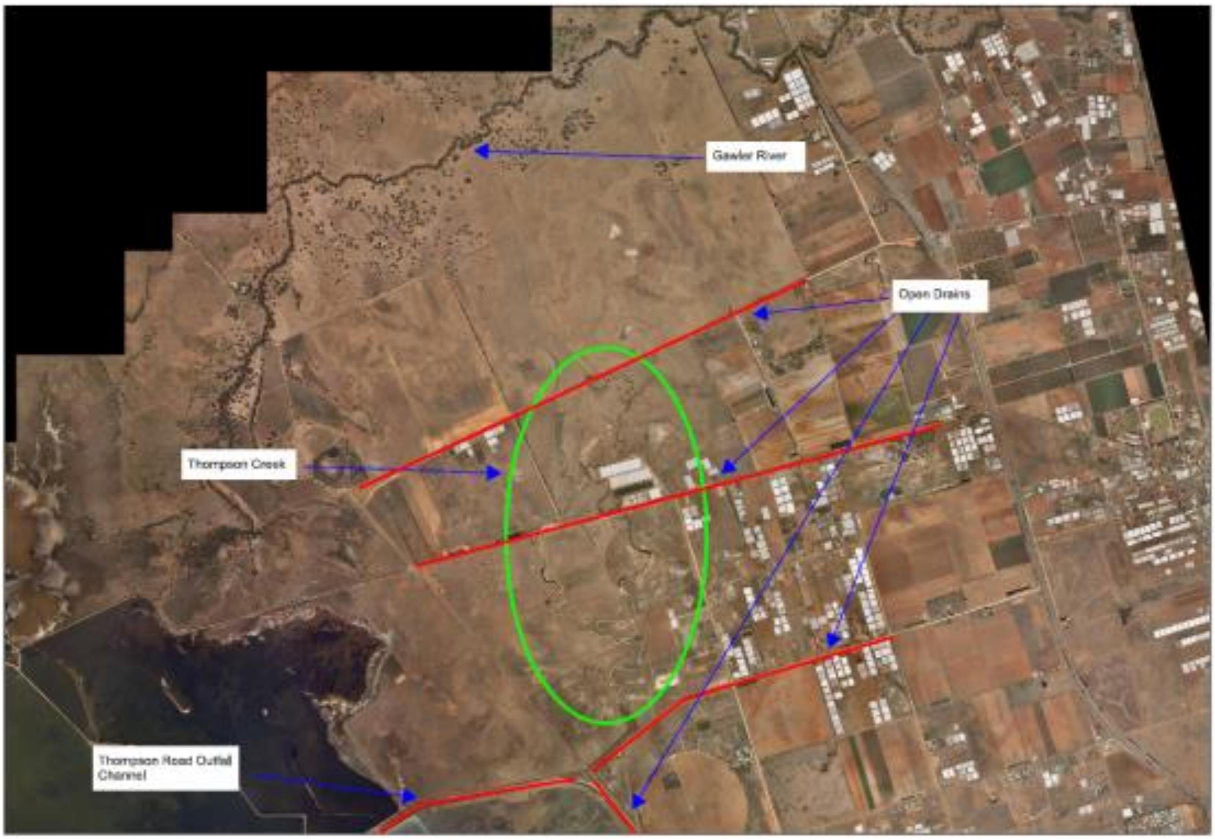

Figure 2-2: Existing Stormwater Infrastructure

The Thompson Outfall Channel is a large earth channel that extends from the western most end of Thompson Road and discharges into Gulf St Vincent.

Thompson Creek is a natural creek which runs through the centre of the Riverlea Park site (see Figure 2-2). The catchment that contributes to Thompson Creek extends west from Port Wakefield Highway, between Thompson Road and the Gawler River.

2.2.1 Thompson Outfall Channel

Thompson Outfall Channel extends from the western most end of Thompson Road in Riverlea Park and runs parallel with the SA Water Bolivar effluent discharge channel (see Figure 2-2 for location). The drain is earth lined with a varying trapezoidal cross section.

Thompson Outfall Channel receives stormwater runoff from a large catchment of approximately 85km2 known as the Western Virginia Catchment. This catchment lies within the bounds of Gawler River to the north, Andrews Road, Munno Para Downs in the east, St Kilda Road to the south and the Salt crystallization pans to the west. The outfall channel discharges directly to Gulf St Vincent and the capacity of the channel will be affected by tide levels.

It is a requirement of the Planning and Design Code that all new projects make an allowance for rises in sea level when designing stormwater outlets that discharge to the sea. The Port Adelaide Seawater Stormwater Flooding study (Reference 11) undertook a detailed assessment of tidal and rainfall records to determine if there was a relationship between tides and storms. The study determined there was no direct correlation and formulated a series of criteria for combined storm and tide events based on likely probability.

Port Adelaide Enfield Council adopts the following when assessing the drainage strategies for projects:

- 1 in 100 year ARI (1% AEP) storm with a corresponding long term Mean High Water Springs (MHWS) tide.
- 1 in 1 year ARI storm (100% AEP), with a long term 1 in 100 year tide event (1%AEP)

Taking into account predicted long term sea level rise at the downstream end of the Thompson Outfall Channel, an outlet tailwater level of 1.95m AHD has been adopted. This level was determined as follows:

- Mean High Water Springs (MHWS) level = 0.95m AHD
- Expected sea level rise (2100) = 1.0m

Mean High Water Springs is a level that is the average of all the twice daily high tides in spring.

In order to determine the capacity of the Thompson Outfall Channel a HEC-RAS computer model was setup. HEC-RAS is a software package that uses one dimensional hydraulic calculations to analyse flows in natural or constructed channels. The parameters used in the analysis include the following:

- Mannings n = 0.04
- Downstream water level = 1.95m AHD (as indicated previously)
- Length 2.6km

From the analysis it was determined that the maximum capacity of the outfall channel is approximately 28 to 30m3/s assuming the existing degraded levee on the northern banks is reinstated to a level similar to the dividing levee to the Bolivar Outfall channel which is set at approximately RL 3m AHD.

2.2.2 Thompson Creek

Thompson Creek is a naturally occurring creek that runs directly through the centre of the site (see Figure 2-2 for location).

The creek currently meanders through the site with a number of branching tributaries and terminates at Thompson Road where it connects into the Thompson Outfall Channel.

2.2.3 Stormwater Drainage Infrastructure

Pre-development, the stormwater infrastructure within the site was limited, with the stormwater runoff from the undeveloped site being carried through the catchment area via a system of road side open drains and culverts (see Figure 2-2 for details) that terminate at the Thompson Road outfall channel.

The exact capacity of the current stormwater drainage system is not known, but is expected to be limited.

2.2.4 Gawler River

The Gawler River is a perched waterway that runs along the northern most boundary of the site.

The river is situated upstream of the site and the banks of the river are raised so they are higher than the surrounding floodplain as shown in Figure 2-1. As such the Gawler River receives no contribution of stormwater runoff from the Riverlea Park site.

The site will however experience flood events from water breaking the banks of the Gawler River. This is discussed in detail in Section 5 of this report.

2.3 Post-Development Stormwater Management

Once the proposal is complete, the Riverlea Park catchment will produce a significantly larger volume of stormwater runoff than it would currently give its undeveloped state. Therefore, to capture and discharge the runoff to Gulf St Vincent, whilst considering and managing the environmental impacts of the increased flows, a more structured stormwater management system will be required.

In order to meet the Council's criteria that peak stormwater flows discharged from the Riverlea Park proposal must not exceed the pre-developed discharge rate and considering the relatively limited capacity of the Thompson Outfall Channel, onsite detention will be required within the proposal's Masterplan.

Stormwater detention will be provided by two means, the salt water lakes will provide stormwater detention above lake water level for those catchments draining to the lakes. For the southern most catchments and parts of Precinct 1, a detention basin/wetland will be constructed at the southern most portion of the site prior to discharge to Thompson's Outfall Channel.

In order to model the estimated peak flows from the developed site a TUFLOW model was created to model the 20% AEP and 1%AEP events. A more detailed Flood Modelling Report is included in Appendix E.

DRAINS was used to estimate the pre-development flows and TUFLOW has been used to model the post development flows and model the impacts of stormwater detention.

TUFLOW is a software package used for designing and analysing urban stormwater drainage systems. TUFLOW uses hydraulic and hydrologic calculations to simulate rainfall events on catchment areas. From this it then calculates the resultant flows, velocities, and hydraulic grade lines that are produced by the rainfall events.

A 1D/2D TUFLOW model has been developed in accordance with AR&R 2019 guidelines. The latest design surface for the development site has been used. The modelling has been undertaken for 1% AEP event.

In order to effectively convey and capture the stormwater runoff created by the proposal a number of different techniques will be used. These techniques include the following:

- A network of concrete pipes to collect local drainage from rooves and roadways
- A network of linear drainage reserves to convey larger flows that will provide a dual use for water quality treatment
- Detention basins and lakes to reduce the peak outflow from the proposal

Detention above the Salt Water Lakes combined with a single large detention basin in the south western corner of the site was considered appropriate. The southern basin was chose as the low lying nature of the land in this area makes it unsuitable for residential purposes also zoned as 'open space'.

2.3.1 Stormwater Modelling

The analysis required the setup of a DRAINS model for pre-development runoff, and a TUFLOW model for post development runoff.

A number of hydrologic parameters need to be established in order to undertake the DRAINS analysis, particularly in regards to estimating runoff from pervious areas. These assumptions were constant for both the undeveloped and developed site and include the following:

- Soil Type = 2 (Moderate infiltration rates and Moderately well drained)
- Antecedent Moisture Content (AMC) = 3
- Grassed initial loss = 40mm
- Paved initial loss = 2mm
- Supplementary paved initial loss = 2mm

Rainfall data is also required to be entered into the model. In this situation rainfall intensities for the Light Region situated slightly north of Riverlea Park was considered to be the closest and most accurate representation of rainfall at Riverlea Park. Recent reports prepared by the CSIRO suggest that in the future Climate Change could increase the intensities of storms experienced in South Australia by up to 4 to 5 % higher by 2050 (Reference 4). In order to take some account for climate change the rainfall intensity from Australian Rainfall and Runoff were increased by a factor of 15% to allow for some further potential increases in predictions through to 2100. This was achieved in the model by specifying a rainfall multiplier factor of 1.15.

Table 2-1 shows a comparison between the undeveloped and developed stormwater peak runoff volumes for both the 100 year ARI and 1 year ARI storm events and also the increased flows attributed to accounting for climate change.

Table 2-1 - Peak Flow Rates for the Developed and Undeveloped Site Conditions

	UNDEVELOPED (M^3/s)	DEVELOPED (M^3/s)	DEVELOPED WITH CLIMATE CHANGE ALLOWANCE (M^3/s)
100% AEP	4	22	25
1%AEP	10	82	92

The runoff from the developed catchment in a 1%AEP storm is approximately 82m3/s greater than the undeveloped peak flow rate. In accordance with Council's requirements this flow will be detained within the site to curtail the peak so that it does not exceed the undeveloped flow rate of 10m3/s.

2.3.2 Pipe Network

A network of concrete pipes will be used to collect the stormwater runoff from the developed catchment area including the commercial and residential areas as well as from the roadways and other impervious surfaces. Following collection, the pipe network will discharge at intermittent locations into a network of Salt Water Lakes and major linear drainage reserves as shown in Figure 2-3.

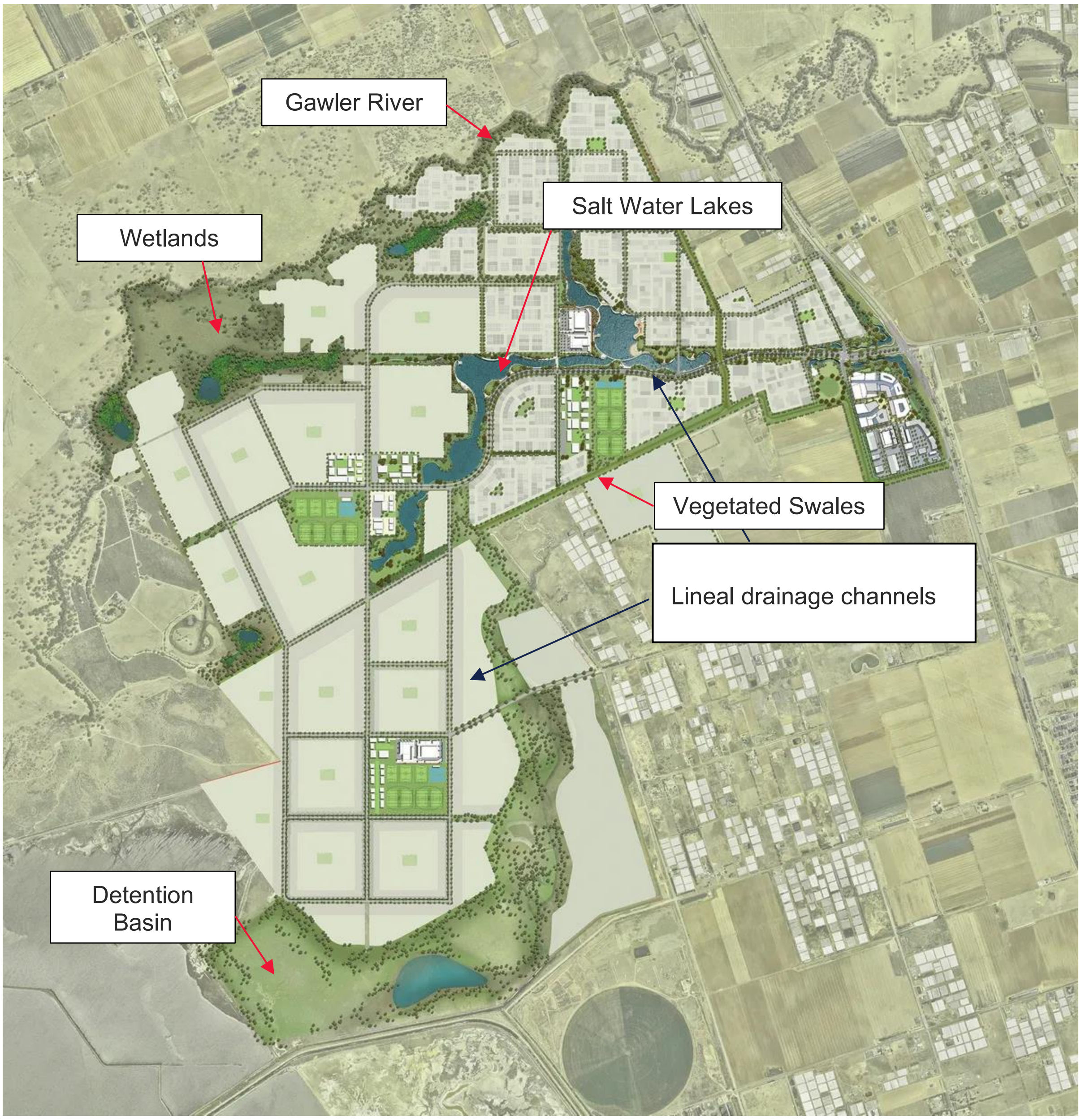

Figure 2-3: Proposed Lake and Lineal Open Drainage System

2.3.3 Linear Drainage Reserves

Linear drainage reserves will be placed within the Masterplan to convey the peak stormwater flows through the site to the Salt Water Lake system to provide stormwater quality treatment and parts of the site will drain through the southern detention basin to the Thompson Outfall Channel. These drains are positioned within the site to take advantage of the natural slope of the land.

The preliminary sizing of these drainage reserves was on the basis that it becomes more practical and cost effective to capture and pass 1%AEP flows within open channels, when these flows begin to exceed the capacity of the combined street and drainage system. This is considered to be when flows reach levels of the order of approximately 5m3/s. From calculations it has been estimated that a catchment area of approximately 50 hectares would be required to produce this magnitude of peak flow in a 100 year storm event. Figure 2-3 shows the proposed locations of the drainage reserves and Salt Water Lakes.

The concept design for the linear channels includes a low flow channel that will accommodate up to a 100% AEP flow and an upper portion that will accommodate a 1%AEP peak flow. The low flow channel aims to collect minor flows and minimise scour across the base of the channel, and will confine the low flows to provide for better water quality treatment.

The assumptions that were made in the design process include the following:

LAND USE	MANNING'S N VALUE
Salt Water Lakes	0.03
Park reserve	0.04
Open space/channel	0.03
Water surface/wetland	0.05
Lots	0.30
Roads	0.02

The channel sizes presented are indicative sizes only. The channels will need to be individually designed during the detailed design process when the catchment area contributing to each drain can be more confidently determined, however it is considered that the extent of the network as shown will be required due to the size of the proposal. The network of drainage channels also provide for flood protection from the Gawler River, which will be discussed further in Section 5.

Due to the length and depth of the proposed drainage channels a significant amount of excavation will need to be undertaken and therefore a significant amount of excavated material will be produced. This excavated material will be used within the site to fill lower areas of the site, to provide shape for road drainage on the flatter areas of the site and also to provide flood protection.

2.3.4 Detention Basins

The pre-development peak flow rate was calculated to be approximately 10m3/s, whereas the post-development peak 1%AEP flow rate was found to be 92m3/s based on the allowance for Climate Change. The proposed detention basin will be located in the south western corner of the site and will reduce the peak flows from the site to a maximum of 1.5 m3/s which is significantly lower than the pre-development flows of 10m3/s. This is primarily due to the significant size of the proposed saltwater lake system which provides for significant stormwater attenuation.

This location for the southern detention basin was chosen for the following reasons:

- Lowest point on the site
- Low possibility of encountering acid sulphate soils (see ASS report)
- Limited development potential of this area as the site elevations are low

Detained outflows from the saltwater lake system are also passed through the large southern detention basin providing double attenuation to the majority of the site.

Figure 2-4 and Figure 2-5 summarise the Peak Flood Depths and Peak Flood Levels (AHD) for the 1% AEP event.

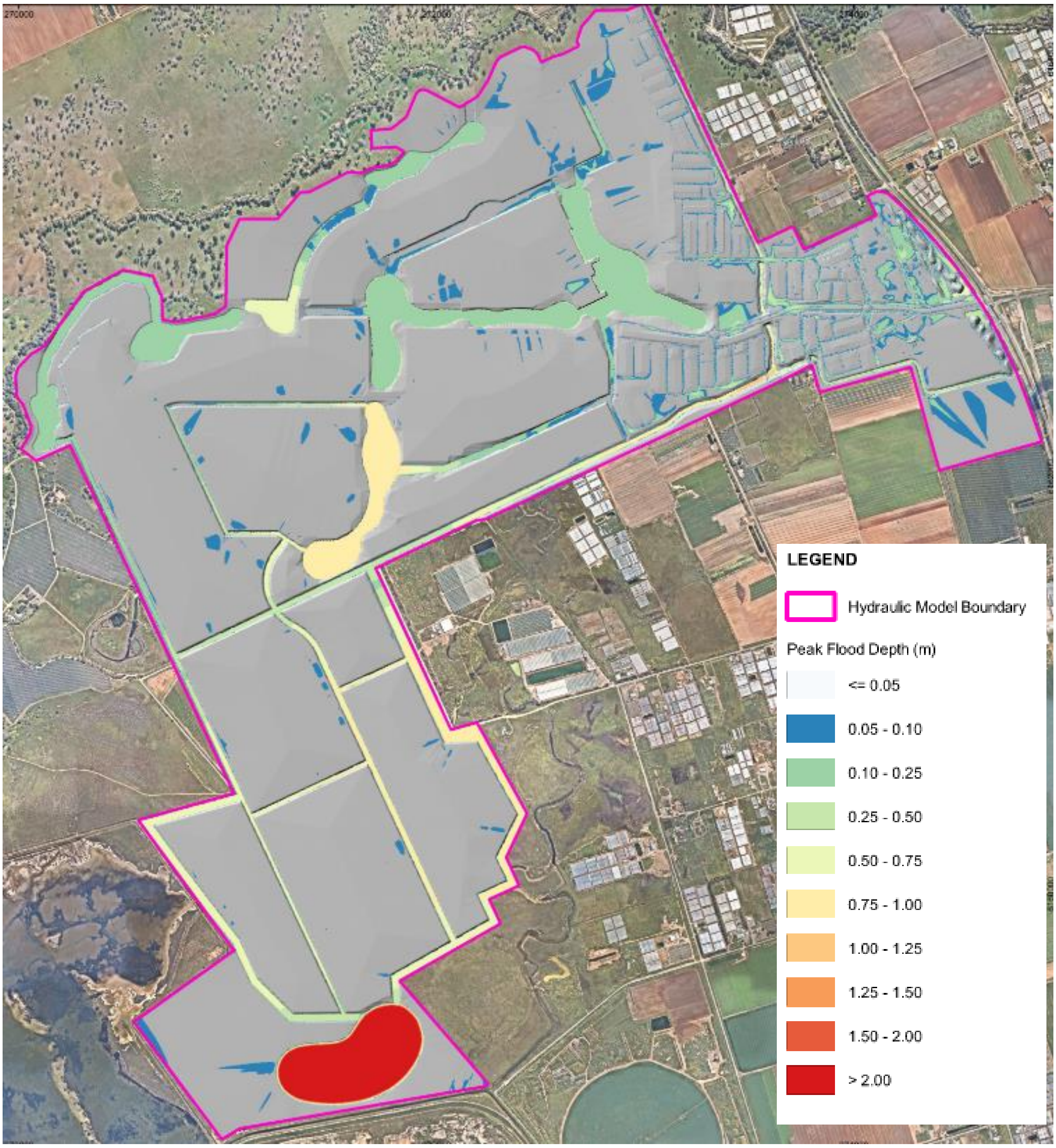

Figure 2-4: Peak 1%AEP Flood Depths

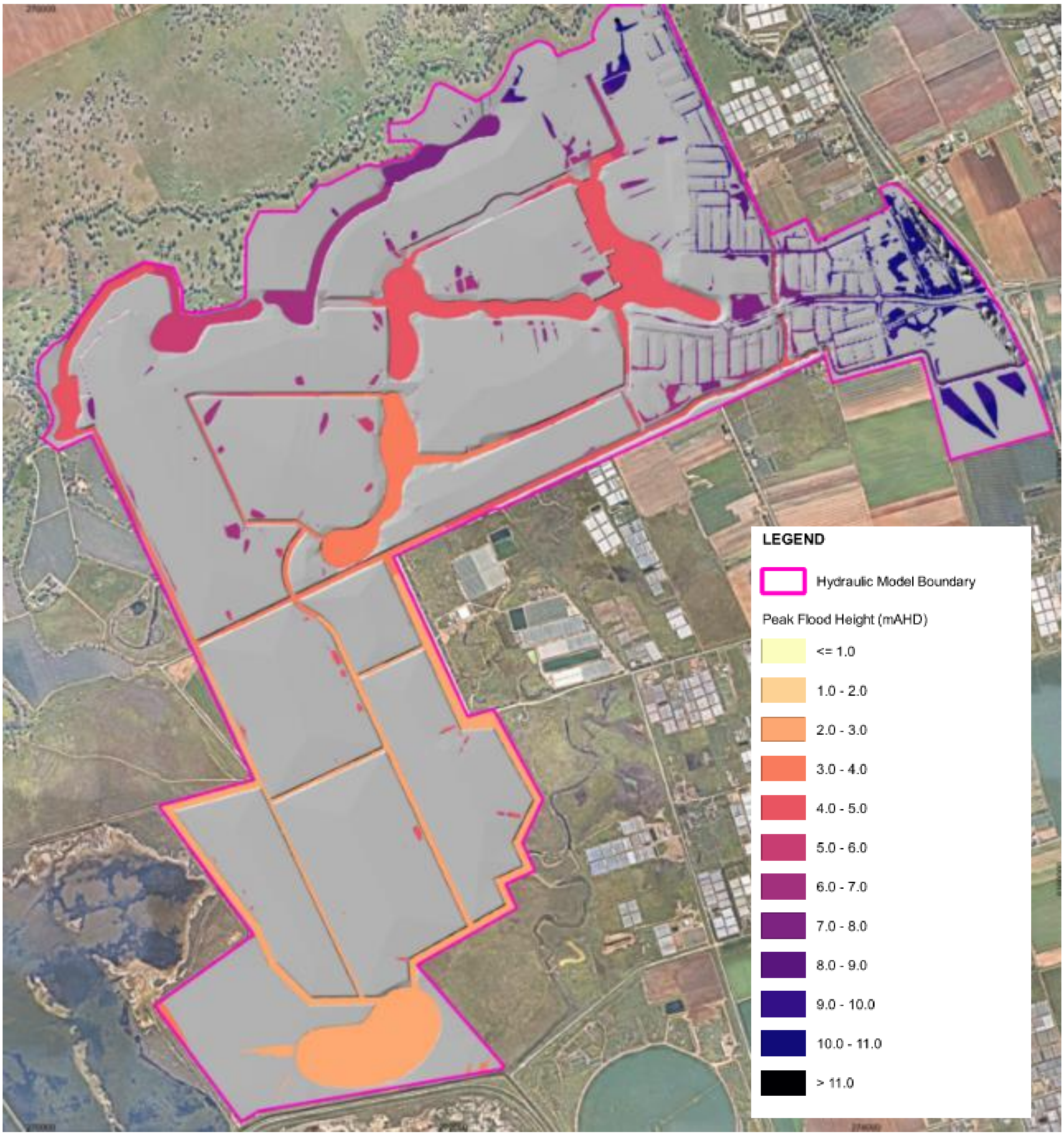

Figure 2-5: Peak 1%AEP Flood Levels in AHD

A copy of the detailed Flood Modelling Report is provided in Appendix E.

3 WATER QUALITY

3.1 Introduction

A Water Sensitive Urban Design (WSUD) approach will be adopted at both a Masterplan and a detailed design level. The basis of the WSUD for the proposal as a whole has been set in the stormwater management system designed for the Masterplan. In terms of stormwater management, this places an emphasis on stormwater treatment, peak flow mitigation, harvesting and reuse, while also ensuring that such practices adopt the multi-objective approach to stormwater management.

The multi-objective approach includes features such as:

- Detain and slow the conveyance of stormwater through the site
- Use vegetation and landscaping to filter and treat stormwater (primarily in the extensive open channel network)
- Integrate the stormwater management into the landscaping
- Water efficient landscaping and the use of local indigenous vegetation species
- Protection of the water related environments and their associated values
- Protection and enhancement of recreational, social, and cultural values
- Improved biodiversity, ecological and habitat outcomes
- Community education and demonstration

Overall, the proposal will incorporate the following stormwater management features:

- Capture and treatment of stormwater runoff at the allotment level, and at the site level
- Treatment of stormwater via wetlands, and vegetated swales in open lineal channels
- Management of the major storm events up to the 1%AEP as discussed in Section 2

This report will focus on the areas of WSUD required at the macro Masterplan level, noting the intention is also to include WSUD features throughout the proposal at the detailed precinct level.

Some examples of typical WSUD features that might be incorporated throughout the proposal are shown in the following images.

Rain Garden/Bio-Filtration Bed

Biofiltration Systems

Infiltration/Wetland Pond

Vegetated Swale

Pre-development stormwater runoff from the site was not treated prior to discharging via the Thompson Outfall Channel.

The stormwater runoff from Riverlea Park will need to be treated to achieve the South Australian Environmental Protection Authority (SAEPA) – Environment Protection (Water Quality) Policy 2015, guidelines, on the basis the water will either be discharged to the marine environment or to the aquifer for storage.

It is recognised by the Institute of Engineers Australia (refer Reference 6) that treatment of up to a 1 in 3 month storm event, is equivalent to treatment of 93% of the annual runoff. It is not considered practical to capture and treat water for events greater than a 100%AEP. For water quality treatment, a design treatment event between a 333%AEP and a 100%AEP event is normally adopted.

A MUSIC (Model for Urban Stormwater Improvement Conceptualisation) model was established to assist in developing the proposed water quality treatment strategy to achieve the SA EPA Water Quality Policy Guidelines.

3.2 Objectives and Water Quality Criteria

The objective of this stormwater quality assessment is to evaluate the treatment performance of the proposed/revised systems within Riverlea estate against the required standards at a master plan level.

The proposed stormwater treatment system was designed to treat the runoff in accordance with the standards as defined by:

- The South Australian EPA water quality policy WSUD targets.
- WSUD pollutant reduction targets as defined in the WSUD Guidelines for the Greater Adelaide Region (2013).

The pollutant treatment performance targets as specified in the above guidelines are:

- 80% retention of typical annual urban load of suspended solids (TSS)
- 60% retention of typical annual urban load of total phosphorus (TP)
- 45% retention of typical annual urban load of total nitrogen (TN)
- 90% reduction of gross pollutants of typical urban load (GP)

In addition to the above targets for the site as a whole, it was also aimed to achieve the treatment performance targets before discharging into the Salt Water Lakes (SWL). The basis of this is that the SWLs can be negatively impacted by the poor quality stormwater inflows from local catchments as described by BMT (2021) in Riverlea Concept Stormwater Quality Management Plan.

Figure 3-1: Proposed Riverlea Master Plan (December 2022) Showing Extensive Open Channel Network and Wetlands

3.3 Stormwater Treatment Strategy

In order to determine the level of water treatment required to meet the SA EPA guidelines a preliminary treatment strategy was prepared. The strategy employs the use of large lineal treatment swales and wetlands to promote natural water treatment processes to occur as the flows move through the catchment area.

A MUSIC model was setup to evaluate the effectiveness of these treatment strategies.

It can be seen in the stormwater layout that gross pollutant traps, swales and 2 wetlands are proposed to treat the stormwater prior to its reuse, or discharge.

3.3.1 Water Quality Criteria

There are a number of guidelines and standards that can be used to assess the outcomes of a water quality strategy.

The proposed stormwater treatment system is assessed to treat the runoff in accordance with the standards as defined by:

- The South Australian EPA water quality policy WSUD targets.
- WSUD pollutant reduction targets as defined in the WSUD Guidelines for the Greater Adelaide Region (2013).

The pollutant treatment performance targets as specified in the above guidelines are:

- 80% retention of typical annual urban load of suspended solids (TSS)
- 60% retention of typical annual urban load of total phosphorus (TP)
- 45% retention of typical annual urban load of total nitrogen (TN)
- 90% reduction of gross pollutants of typical urban load (GP)

The stormwater treatment strategy also adopts the principles of the Australian and New Zealand Environment and Conservation Council (ANZECC) water quality guidelines as a framework. This relates to providing a sound approach that facilitates an environmental duty to prevent or minimise harm to the downstream environment though a treatment train approach.

3.3.2 MUSIC Modelling

A MUSIC model was prepared for the strategy in accordance with the South Australian MUSIC Guidelines (2021) This includes all modelling parameters, model setup and approach top modelling comply with the Guidelines (2021). This is consistent with the Stormwater Quality Modelling Technical note (2022) provided in Appendix F. The model is available for Auditing by Authorise upon request.

MUSIC is a software model which predicts the performance of stormwater quality improvement systems by simulating the quantity and quality of runoff produced by catchments and assessing the effectiveness of downstream treatment points to reduce pollutant loads. The treatment systems adopted in this strategy include:

- Gross Pollutant Traps (GPT's)/Trash racks
- Swales
- Wetlands
- Ponds

There are a number of pollutants which can be present in stormwater runoff. Within the MUSIC model only the following are analysed:

- Total Nitrogen
- Total Phosphorus
- Total Suspended Solids
- Gross pollutants

Other pollutants are expected to be present in the runoff prior to treatment, it is known however that fine particulate pollutants attach themselves to other particulate pollutants such as Total Phosphorus (TP) and Suspended Solids (SS). MUSIC therefore assumes that by targeting pollutants such as TP and SS it will also be treating other pollutants.

Figure 3-3 shows how the stormwater strategy has been arranged within the MUSIC model. It can be seen that each sub-catchment is connected to a GPT/Trash rack and a swale prior to entering either a wetland or a capture basin.

This layout is not a true representation of how the system will operate, but was an altered version constructed to suit the capacity of the modelling program.

Figure 3-2 also shows the Treatment Catchments for the development.

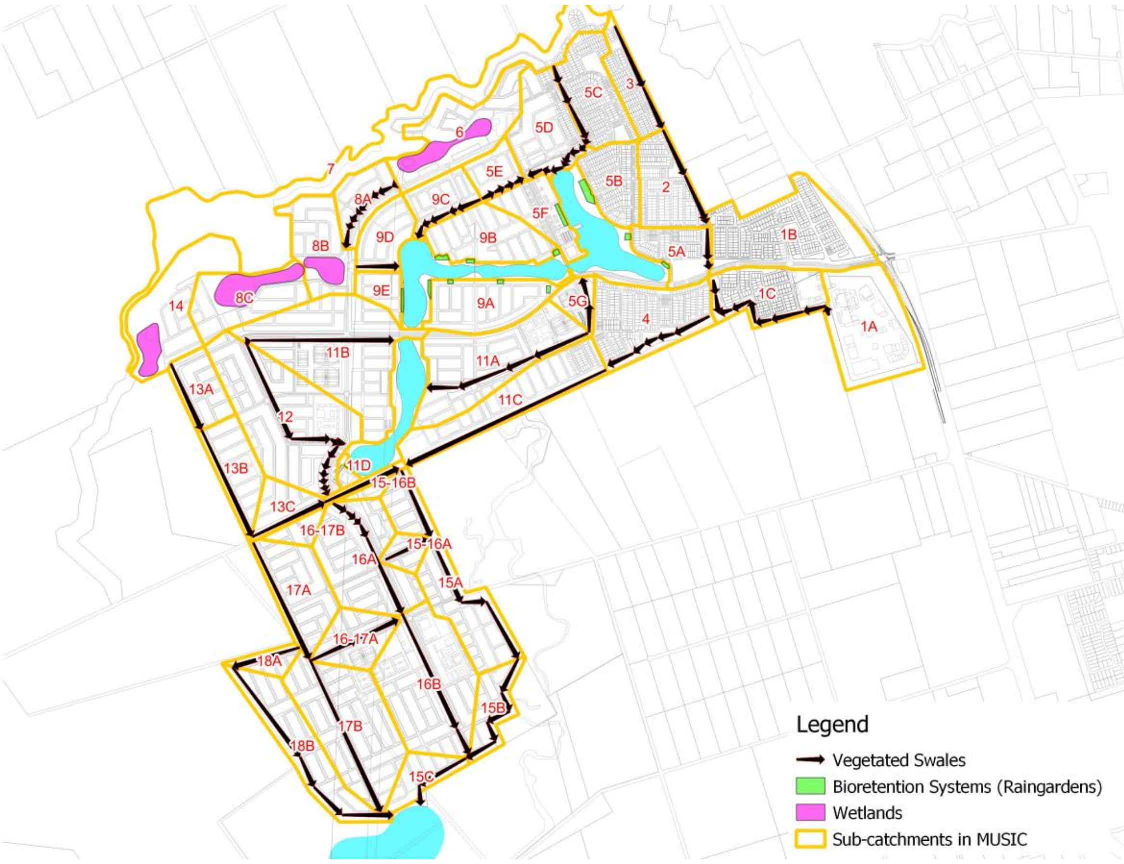

Figure 3-2: MUSIC Model Catchment Plan and WSUD Assets Locations

Figure 3-3 shows the MUSIC Model Schematic and Table 3-1 shows the Water Quality Results. These results are reported prior to discharge into the Saltwater Lakes. Therefore, the strategy has adopted the Saltwater Lakes as being the receiving environment.

Table 3-1: Water Quality Results Compared to Best Practice Standards

POLLUTANT TYPE	TSS	TP	TN	GROSS POLLUTANTS/LITTER
Target percentage reduction (%)	80	60	45	>50 mm and retention in 3-month ARI
Reduction achieved at SWL1 (%)	95.9	73.9	57.4	100% trapped (averaged over the simulated period)
Reduction achieved at SWL2 (%)	96.1	70.5	57.4	100% trapped (averaged over the simulated period)
Reduction achieved at SWL3 (%)	96.8	83.2	66.0	100% trapped (averaged over the simulated period)
Reduction achieved at Northern Outlet (%)	100	100	100	93.4% trapped (averaged over the simulated period)
Reduction achieved at Southern Outlet (%)	97.2	86.8	69.7	100% trapped (averaged over the simulated period)
Reduction achieved at Site Overall (%)	97.3	85.4	70.9	99.2% trapped (averaged over the simulated period)

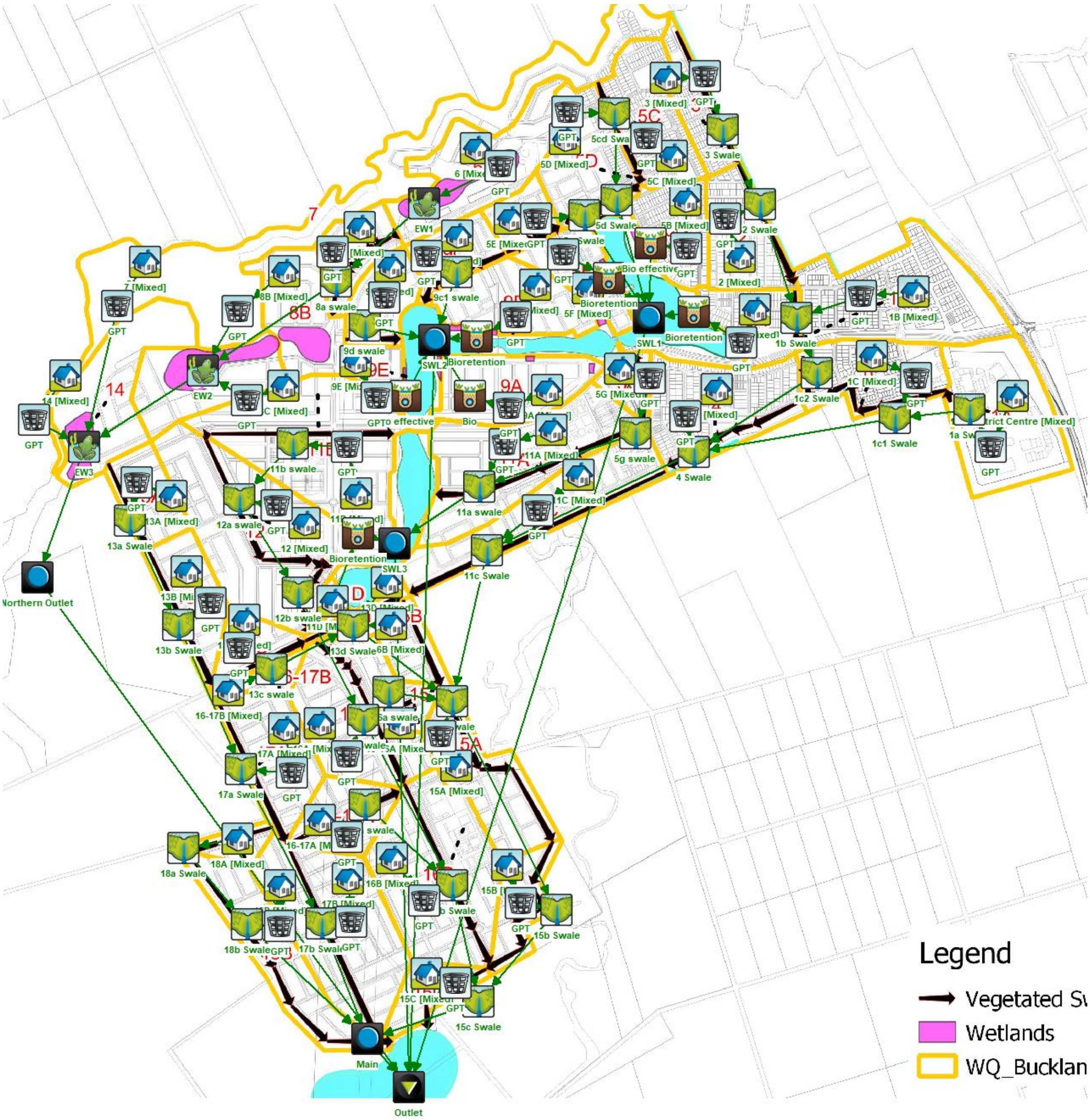

Figure 3-3: MUSIC Model Schematic

3.4 Water Quality Summary

This Master Plan level assessment of the stormwater treatment strategy for Riverlea Park indicates that stormwater quality discharging from the estate (to the Salt Water Lakes) will meet the treatment performance targets as defined in EPA WSUD treatment targets and the Greater Adelaide Region's WSUD pollutant reduction targets. The Strategy has adopted the ANZEC framework with regards to the adoption of a treatment train approach that minimise risk or harm to the receiving waters. Furthermore, the reported treatment targets a based on the point of discharge into the Salt Water Lakes and therefore ensure that stormwater do not impact the water quality within the lakes.

A more detailed Stormwater Quality Modelling Report is provided in Appendix F

3.5 Aquifer Storage and Recovery Potential

The Aquifer Storage and Recovery Potential at Riverlea Park has been assessed by REM in their report Aquifer Storage and Recovery Potential for Riverlea Park, (Reference 7). REM has advised the T2 aquifer has the potential to accept up to 50ML/a of water without pressurising the aquifer. Pressuring the aquifer would potentially result in increased storage potential, however, it would significantly impact on all existing bores connected to the T2 aquifer, requiring the bore heads to be sealed, and pumps changed to suit the new aquifer pressure.

There are a currently 287 recorded local bores that could be affected by pressurising the aquifer and it is therefore concluded planning should exclude this option.

For the purposes of assessing the ASR potential of the site, it has been assumed a maximum of 50ML/a of treated water can be discharged to the local T2 aquifer, compared to the potential to capture up to 2000ML/a of annual runoff.

The ASR potential is therefore very limited in terms of its ability to be a reliable source of secondary water supply, unless above ground storages with floating covers are considered which have proven to be very costly and would add significantly to the cost of water. SA Water advised that sufficient recycled water will be made available from Bolivar for the recycled water supply for the entire proposal. On this basis it is likely that the 50ML/a of ASR potential will be used to provide recycled water for irrigation of some parks, and to top up wetland water bodies.

For the provision of irrigation water to the development, SA Water have advised that connection to the NAIS scheme can be provided which will allow for a relatively cheap source of irrigation water. ASR is no longer being considered for the development.

4 FLOOD PROTECTION FROM GAWLER RIVER

4.1 Introduction

The Riverlea Park site is currently subject to flooding during a 5%AEP event via a breakout from the Gawler River. Refer to the Floodplain Mapping for the Gawler River – Technical Report 2008, prepared by Water Technology and Australian Water Environments (Reference 1). Appendix B contains the Gawler River Flood Plain Maps.

The lower reaches of the Gawler River through Virginia and Riverlea Park is an example of a 'perched' river, as its banks are higher than the surrounding floodplain. When water breaks the banks of the Gawler River in these areas, water flows away from the Gawler River as opposed to being contained in a low lying floodplain. There are a number of breakouts that enter the site as shown in Figure 4-1.

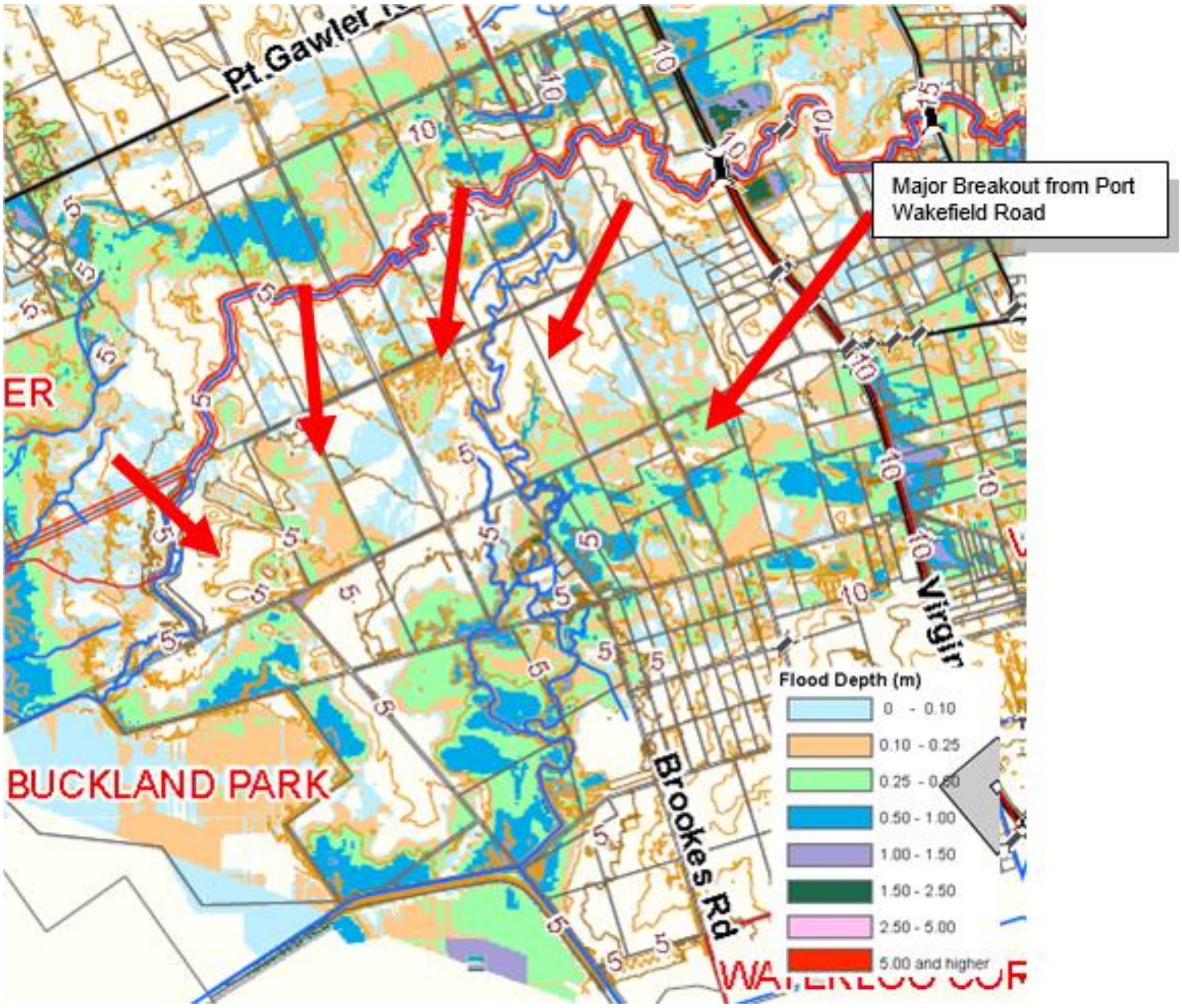

Figure 4-1: Extract from 1%AEP Floodplain Map from AWE/Water Technologies Floodplain Report

The flows are relatively shallow in nature and in terms of Flood Hazard as defined by the Australian Government SCARM 2000, Floodplain Management in Australia, Best Management Practices and Principles, the flood hazards are primarily in the low to medium category as they are relatively shallow and the flow velocities are low.

The largest breakout from the Gawler River approaches the site from the east via Port Wakefield Highway and in the 100 year ARI event, is in excess of 100m3/s. The other breakouts are relatively minor, however, they do pose some risk to the site and need to be managed. Figure 4-2 shows in greater detail the predicted extent of flooding within the site in the 100 year ARI event.

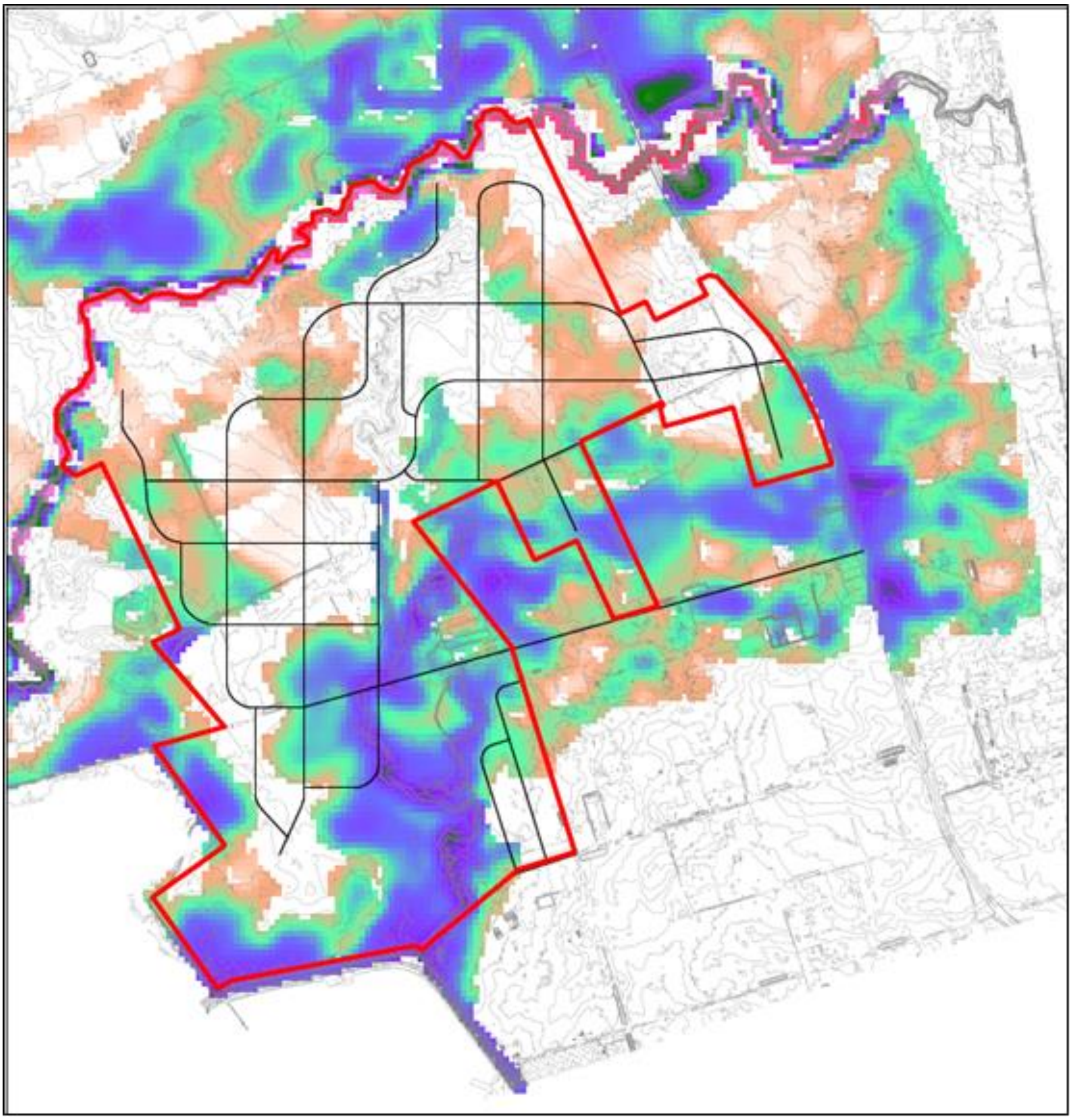

Figure 4-2: 1%AEP Gawler River Floodplain as it Relates to the Riverlea Park Site

4.2 Flood Management Strategy

The flood management strategy proposed for the site involves of a series of flood channels.

The use of levees was initially trialed, particularly against the banks of the Gawler River, however, it was found that introducing levees to control breakouts often forced breakouts in other areas. Similarly, the introduction of a levee system often diverts flood flows to other areas, potentially adversely impacting adjoining properties.

It should be noted that pre-development when the 1%AEP breakout flows leave the southwestern boundary of the site, they overtop the Thompson Outfall Channel into the Cheetham salt crystallisation pans, and into the Bolivar Outfall channel.

As this would occur in a 1%AEP flood event, and to alter this situation would require significant works outside of the site boundaries, the flood mitigation strategy allows this to continue to occur in the future as it would do now, and provides protective works within the site.

The proposed major drainage channel system proposed for Riverlea Park is shown in Figure 4-3. The system consists of a number of major drains through the site to capture the breakout flows from the Gawler River. It should be noted that a flood event that would produce a breakout in the Gawler River is a long duration storm event, peaking after some 20 to 30 hours. Refer Hydrological Study of the Gawler River Catchment (Reference 2).

The critical storm durations for the internal drainage system are of the order of 30 to 60 minutes. Therefore, the drainage system within Riverlea Park would not need to accommodate a coincident peak flood event from the Gawler River and from within the site, hence, significant sections of the proposed major drainage system have been designed to provide a dual purpose.

The drains are relatively flat, particularly the main capture drain which is as flat as 0.05% in some areas. The drains have been kept relatively shallow, up to a maximum of 2.0m, to keep the invert as high as possible to keep the risk of groundwater intrusion to a minimum.

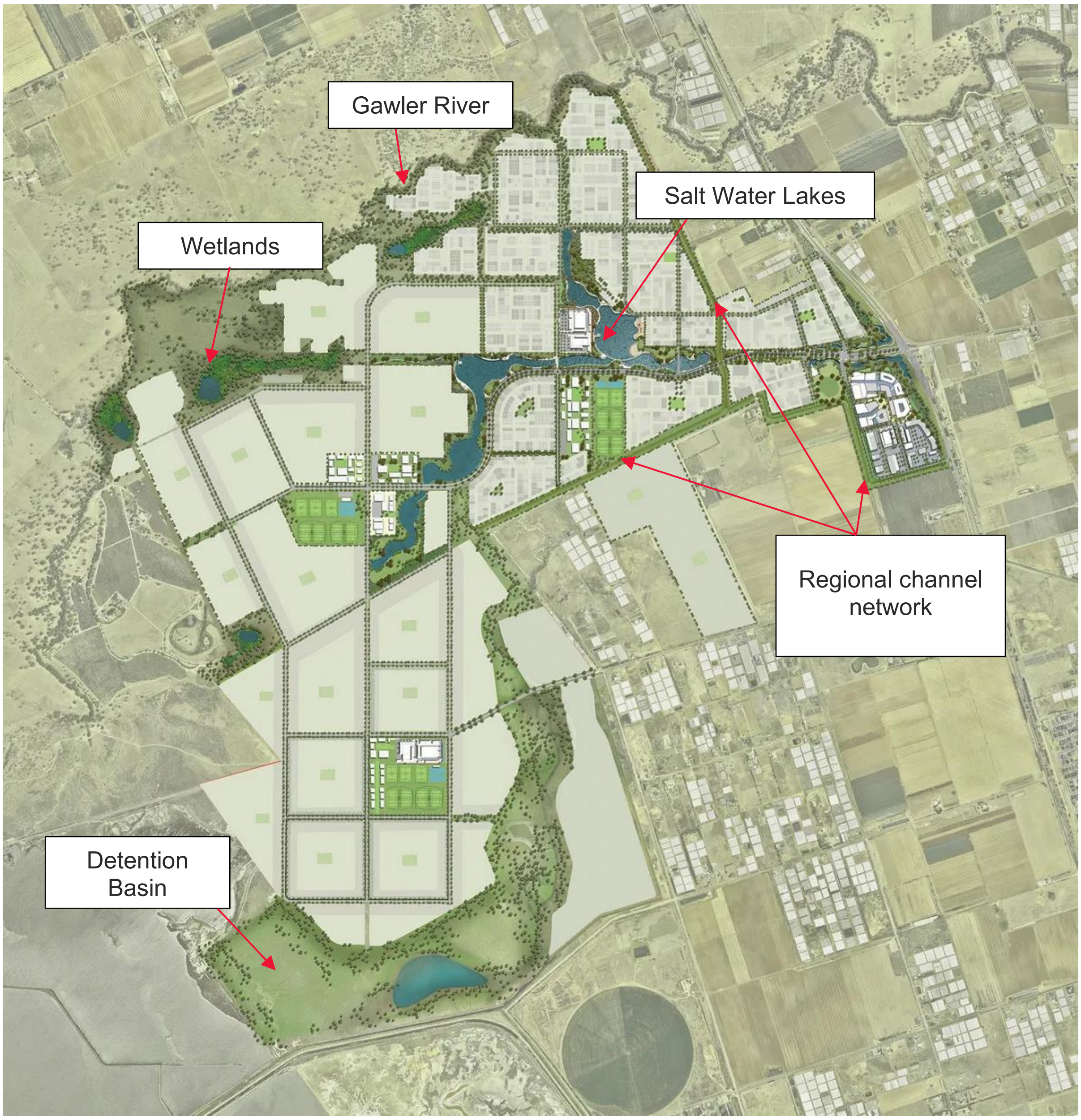

Figure 4-3: Proposed Riverlea Park Major Regional Drainage Channel Network

The major drainage system is the large open channel networked depicted in Figure 4-3.

4.3 Modelling

The modelling of the flood performance from breakouts from the Gawler River has been undertaken by Water Technologies as the consultants for the Gawler River Floodplain Mapping Project.

The modelling has been undertaken using the two dimensional floodplain model MIKE 21, using the modelling assumptions adopted and agreed for that study.

A series of trials have been carried out which have led to the preferred solution for the proposal.

4.4 Results

Figure 4-4 presents the results of a 1%AEP event on the Gawler River.

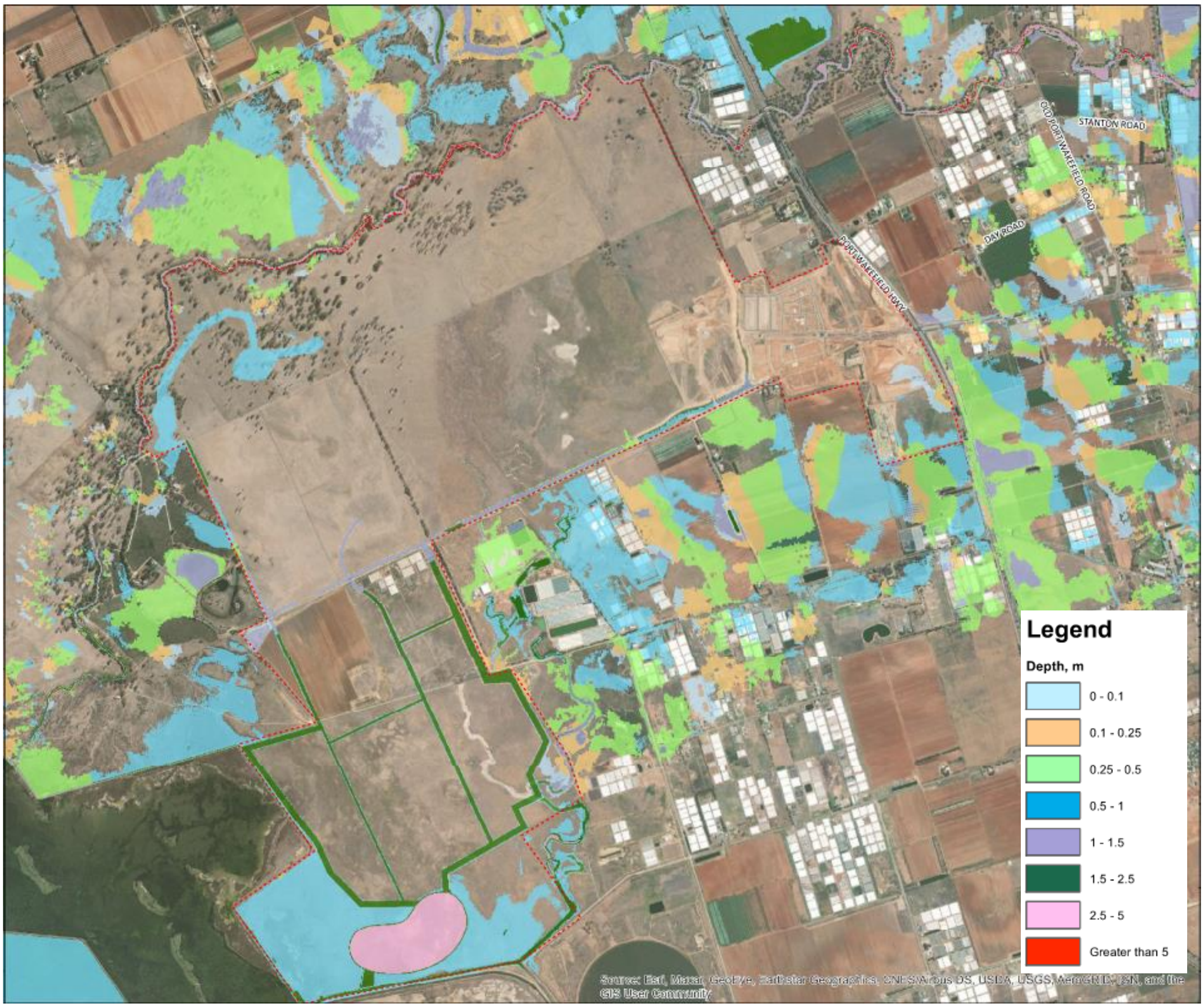

Figure 4-4: 100 Year ARI Event in Gawler River with Proposed Flood Protection Channels

The modelling shows that the proposed open channel system has the capacity to capture and pass the 1%AEP event Gawler River breakouts through the site, the exception is the proposed District Centre and Mixed Use precinct adjacent Port Wakefield Highway which has been highlighted in Figure 4.4.

4.5 Impacts of blockage in the Gawler River

The potential for a blockage to occur on the Gawler River, and the resulting impacts this would have on flooding in Riverlea Park has been considered.

In the 2005 flood event in the Gawler River, a fallen tree contributed significantly to the flooding, primarily by causing a break in a levee on the banks of the River (Personal Communication with AWE, November 2008).

Consideration included the potential flood impacts of an obstruction in the Gawler River, between Port Wakefield Highway and the site's western boundary. A channel blockage factor of 25% was considered a reasonable upper limit. A 25% blockage was trialed at a number of locations, however, no additional breakouts were predicted, as the section of Gawler River downstream of Port Wakefield Highway has greater capacity than sections upstream, and water will break the banks of the Gawler River at locations indicated in AWE mapping, resulting in flows less than the capacity of the Gawler River in the channel downstream of Port Wakefield Highway.

Figure 4.7 and 4.8 show the predicted 100 year ARI floodplain in Riverlea Park created by placing 25% blockages at two locations on the Gawler River, downstream of Port Wakefield Highway.

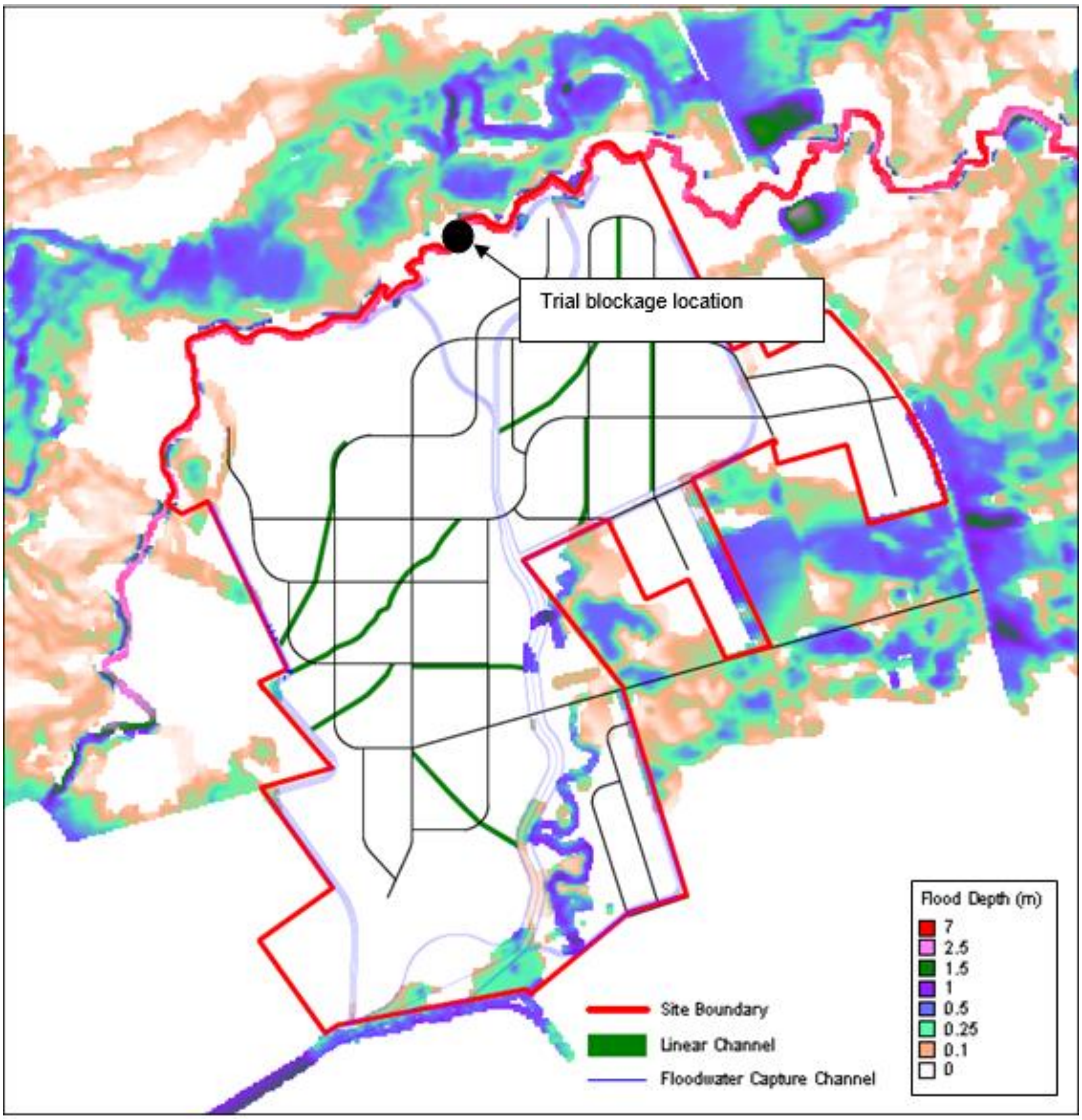

Figure 4-5: 100 Year ARI Floodplain with a 25 Percent Blockage of Gawler River at Location 1

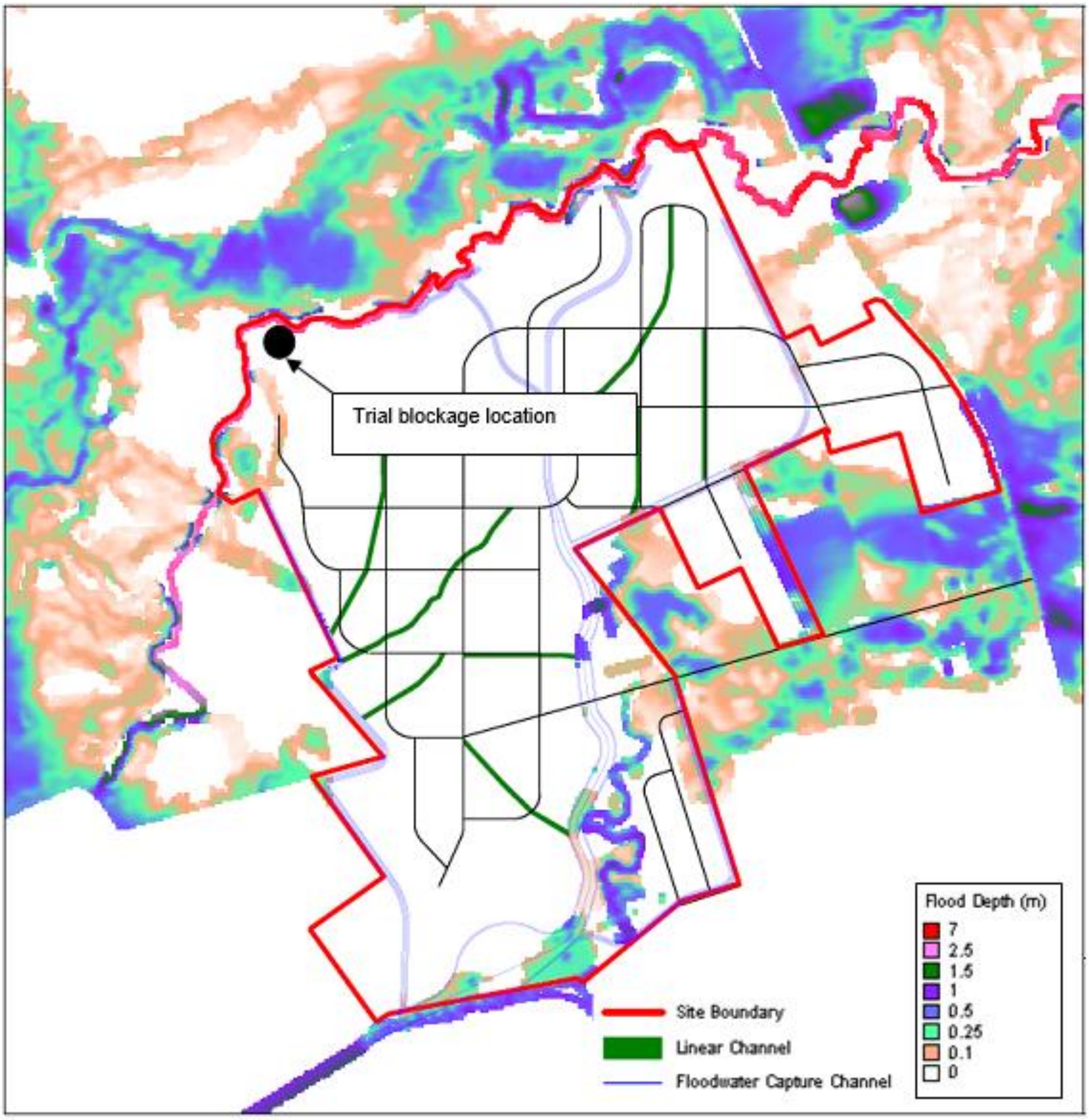

Figure 4-6: 100 Year ARI Floodplain with a 25 Percent Blockage of Gawler River at Location 2

The modelling indicates that the risk of a blockage occurring in the Gawler River downstream of Port Wakefield Highway has little to no impact on an increase in flood risk in the 100 year ARI event.

A more detailed flood assessment report by Water Technologies is included in Appendix B.

5 WASTE WATER

5.1 Introduction

Pre-development within the Riverlea Park area there was no formal system for the collection and disposal of waste water.

New waste water infrastructure will therefore be required to serve the proposal.

SA Water have advised (Reference 9) that a new rising main will be required from the site to deliver sewage directly to the Bolivar Wastewater Treatment Plant, located approximately 14km south of the site.

In order to determine the most efficient method of waste water collection system for this proposal the following network types were considered:

- Vacuum
- Pressure
- Gravity
- Septic Tank Effluent Disposal System (STEDS)
- Full Sewer

These four sewerage schemes were assessed based on their cost effectiveness, and the suitability of their design characteristics for the environmental conditions on site.

The environmental conditions within the Riverlea Park site that could significantly impact on the suitability of the use of a particular sewerage system include the following:

- High ground water level
- Highly saline ground water
- Acid sulphate soils

Based on the preliminary costing and the expected site environmental conditions a vacuum system was recommended for the Riverlea Park proposal see the Network Options Report (W&G, August 2008) in Appendix C.

5.2 Environmental Conditions

Site specific environmental conditions are instrumental in determining the suitability of a sewer system. The selection of an environmentally suitable sewer system could significantly reduce the risk of cost escalations during construction, reduce ongoing running costs and increase constructability.

5.2.1 High Ground Water

The majority of the site has a depth to water table of less than 3 metres. To minimise the length of drain constructed below the groundwater table the maximum drain depth was set to 3 metres. In order to keep the pipes as shallow as possible pump stations would need to be installed at regular intervals.

From analysis it was determined for a gravity system approximately 35 pump stations would be required to keep the pipe invert level within 3 metres of the surface level. Even with this large number of pump stations, as much as 75% of the gravity drains would still be installed within the ground water zone, this is prior to considering the impacts of long term sea level rise on groundwater levels. Figure 5-1 shows a depth to water table plan for the site highlighting all areas where the groundwater is less than 3m below the surface. This map is based on recent site mapping undertaken by REM.

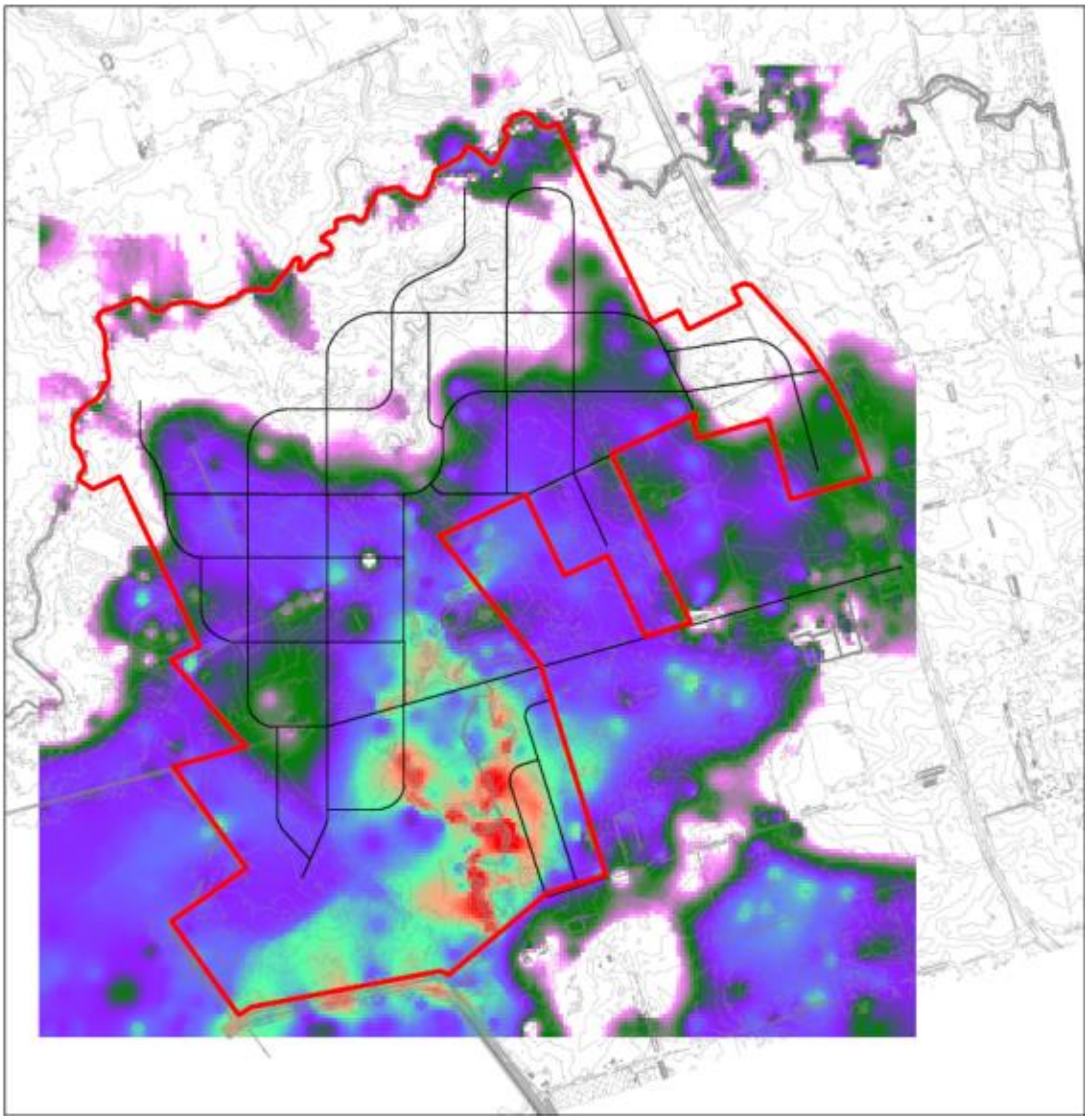

Figure 5-1: Depth to Groundwater within 3m of Existing Surface Level

It should be noted that seasonal fluctuations of up to 1 metre could be experienced based on the advice from the REM report (Reference 10). This could see as much as 95% of the gravity drain being below the standing groundwater level.

Constructing a gravity system within the ground water table could potentially result in water infiltration at manholes, pump stations and any breaks or cracks in the pipe work. STED systems also have potential for ground water ingress at septic tanks.

The drains for a vacuum system are generally installed between a depth of 1.2m and 1.5m. It is estimated that for a vacuum system only 10% of vacuum drains would be installed within the water table.

Figure 5-2 indicates the area of the site that the depth to ground water is less than 1.5m

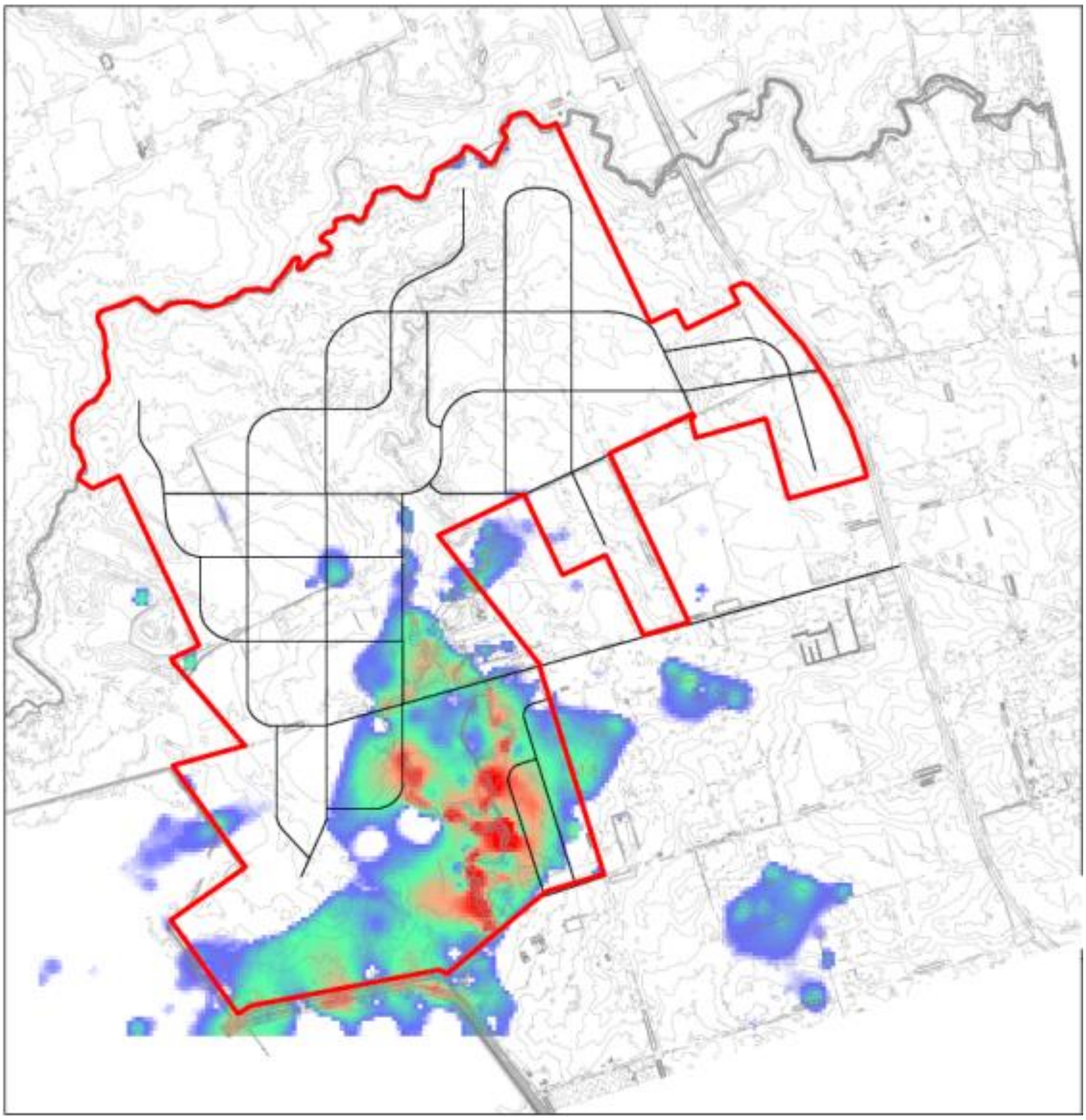

Figure 5-2: Depth to Groundwater within 1.5m of Existing Surface Level

It should be noted some of the areas within the proposed urban areas shown here as being within 1.5 of groundwater, will be filled to provide for adequate protection from long term sea level rise.

5.2.2 Salinity

Ingress of saline ground water into the waste water pipe network could cause the salinity of the waste water to increase and highly saline waste water can impact on the effectiveness of the operation of the WWTP at Bolivar. Increased salinity could also impact on the potential number of reuse applications for the treated effluent.

The ground water within the Riverlea Park site is expected to have salinity in the order of 1000ppm to 5000ppm (TDS).

The salinity of typical treated waste water schemes in South Australia is between 800ppm and 1000ppm (TDS). This would mean relatively small volumes of ingress could significantly impact on producing treated waste water of an acceptable salinity level.

The Riverlea Park proposal places a high priority on the potential to reuse the treated waste water, therefore the potential for ingress of saline groundwater into the waste water management system was a significant factor in selecting the most appropriate method of waste water management.

5.2.3 Acid Sulphate Soils

It has been confirmed by Golders Associates (Reference 5) that sections of the Riverlea Park site have the potential to encounter acid sulphate soils below the ground water level (see Figure 5-3 for potential acid sulphate soil locations)

If Acid Sulphate Soils (ASS) are encountered within trenches, the soil will need to be treated prior to the installation of any infrastructure, therefore causing construction costs to increase.

Precautions will need to be taken to prevent ingress of leachate from ASS getting into the trenches and being transported around the site. Both vacuum and pressure systems will minimise leachate ingress due to the relatively shallow depth of drains. Gravity drains also drain for long distances at a constant downward grade which facilitates the transport of leachate (if encountered). Both the vacuum and pressure sewerage drains are not required to constantly grade downward, this in itself would minimise the spread of ASS leachate should it be encountered.

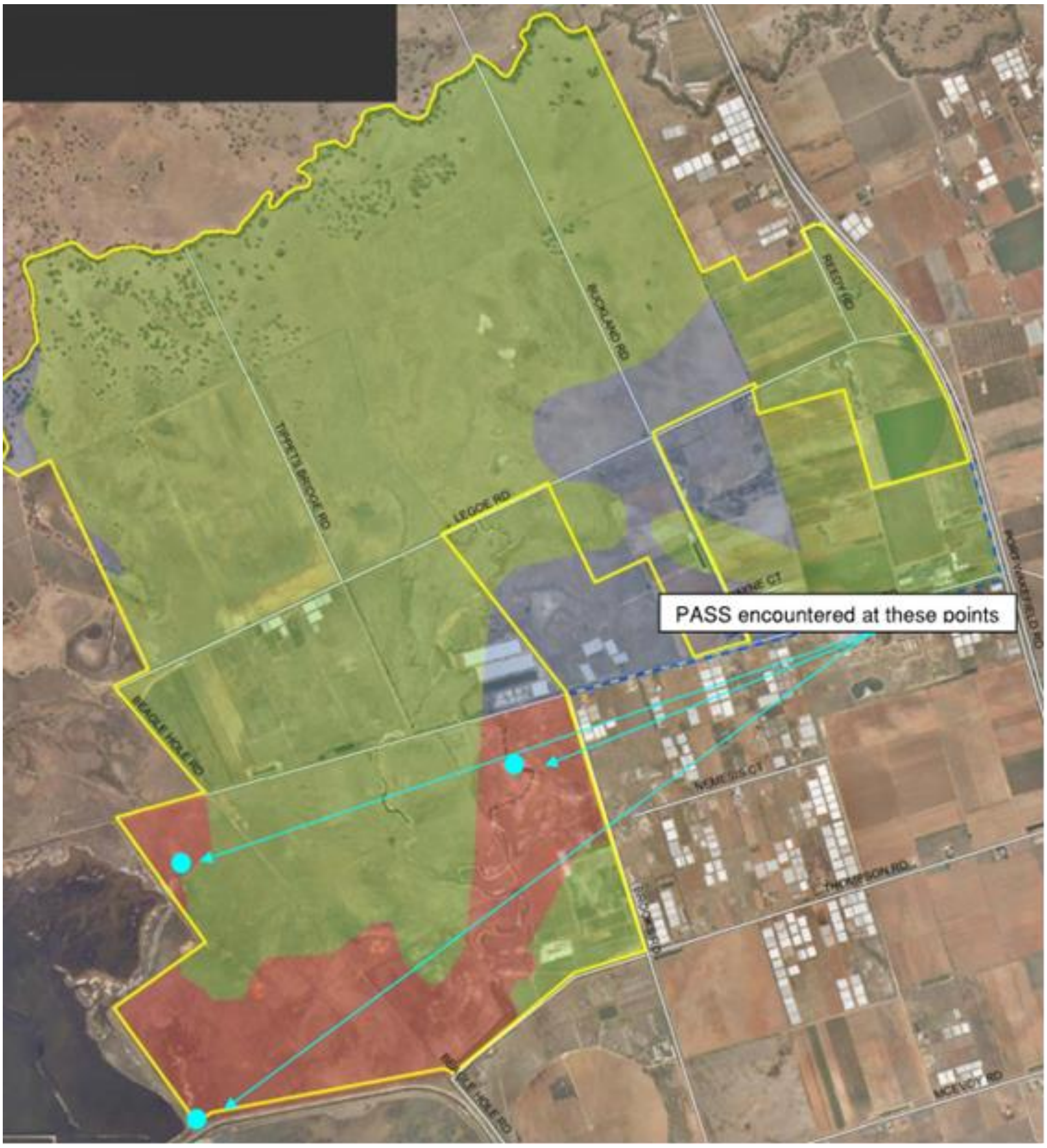

Figure 5-3: Potential Acid Sulphate Soil Locations

5.3 Recommended Waste Water Management System

From WGA's analysis in Appendix C it was determined that the most suitable form of communal waste management system for Riverlea Park is a vacuum system.

The reasons for recommending this option include:

- Lower estimated capital cost and all of life costs
- Reduced potential impacts of salinity on the reuse applications
- Lesser impact of peak wet weather flows on the WWTP and pump stations
- Lesser potential for long term ground water ingress
- Reduced risk of system failure due to groundwater ingress
- Lower pumping costs associated with limited groundwater ingress
- Approximately 75% of drains in a gravity system would be installed below the current ground water levels, even with the installation of 35 pumping stations

5.4 Methods for Disposal of Waste Water

In order to determine the most feasible method for treating and disposing of Riverlea Park's waste water a number of scenarios were considered.

The main scenarios that were considered are shown in the table below:

SCENARIO	INTERIM	ULTIMATE
1	Onsite WWTP with 5000 Person capacity	450mm pipe to pump waste water to Bolivar WWTP
2	225mm pipe to pump waste water to Bolivar WWTP	450mm pipe to pump waste water to Bolivar WWTP
3	150mm pipe to pump waste water to Bolivar WWTP	450mm pipe to pump waste water to Bolivar WWTP
4	33,000 person capacity onsite WWTP	

The above scenarios include opportunities for the disposal method to be staged in order to cater for:

- Initial capital cost reduction
- Waste water flow production

The treatment of effluent in an onsite WWTP has been considered and discounted for the following reasons:

- Buffer areas around the Plant will require a large area within the site, which may be more efficiently used for urban purposes.
- There are environmental constraints associated with areas that do not have urban potential, which preclude a WWTP. For example, significant flora, high ground water and potential acid sulphate soils.
- A new facility may be more costly to construct than augmentation at an existing WWTP facility.

The preferred method for disposal of the effluent generated by the completed Riverlea Park proposal is pumping the effluent via a rising main to the Bolivar WWTP.

The Bolivar WWTP is located approximately 14 kilometres south of the Riverlea Park site. This represents a considerable pumping distance and will result in large friction losses and potentially long travel times.

Refer to Appendix D, for a summary of the proposed pumping and rising main staging options for the development. There are 5 Vacuum Pump Stations proposed and a series of Booster Pump Stations and Rising Mains to take wastewater to the Bolivar WWTP.

6 WATER SUPPLY

6.1 Introduction

There is a limited amount of SA Water infrastructure in the area.

Upon completion, the Riverlea Park proposal will comprise approximately 12,000 allotments. A proposal of this scale will create a large demand for potable water in a previously undeveloped area.

In order to provide a reliable source of potable water, major infrastructure works will be required. SA Water outlined a number of potential potable water supply options can be considered for the proposal (see Appendix D). These options include potential for short term water supply from existing infrastructure during the initial stages of construction and occupation. This will reduce initial capital costs and will also potentially provide the site with a long term backup potable water source.

Water restrictions, and the ever increasing need to conserve water resources, have made recycled water use for applications that do not require drinking quality water a necessity. Recycled water is sourced from waste water treatment systems and stormwater runoff, and is increasingly being used for non potable applications within industry and also in new residential communities. With SA Water having recently completed the NAIS scheme, which now passes by the development in Port Wakefield Highway, Walker Corporation are negotiating with SA Water to bring the NAIS water into the site for the purposes of irrigation water only.

Appendix D contains SA Water's assessment of the water supply options, available to the Riverlea Park proposal, which involve a number of significant pipe upgrades outside of the site, that will need to be funded and constructed over a number of budget periods.

6.2 Recycled Water Supply

To ensure potable water supply sustainability, the use of recycled water for all applications which do not require drinking water quality water is becoming more and more common in residential, industrial and commercial projects.

Typically, the incentive for consumers to use recycled water is its cost. Recycled water is cheaper than potable water as it commonly does not require the same high level of treatment that potable water does.

Recycled water can be used for most applications where humans do not have direct contact with the water, such as:

- Toilet flushing
- Garden watering
- Car washing
- Irrigation

Using recycled water for the above applications would significantly reduce the use of potable water.

6.2.1 Recycled Water Sources

Sources of recycled water available to the Riverlea Park proposal include:

- Treated waste water delivered from the Bolivar Waste Water Treatment Plant via the Western Reticulation Systems Virginia (WRSV) pipeline or a new pipeline direct from the Bolivar WWTP.
- Treated waste water delivered from the Bolivar Waste Water Treatment Plant via the Northern Adelaide Irrigation (NAIS) pipeline.
- Stormwater runoff

Walker Corporation are negotiating with SA Water to provide irrigation water to the site via the NAIS scheme.

7 SEA LEVEL RISE AND MINIMUM SITE LEVELS

7.1 Coastal Protection Board

The current figures advised the required minimum Site Level (SL) and Finished Floor Level (FFL) to prevent coastal flooding for design to 2050 and 2100, as outlined in Table 7-1.

Table 7-1: Minimum Site Levels (Coastal Protection Board SA, 2008)

	2050	2100
Minimum SL (m AHD)	3.30m AHD	3.30m AHD+0.7m = 4.0m AHD
Minimum FFL m AHD)	3.55m AHD	3.55m AHD+0.7m = 4.25m AHD

Figure 7-1 shows the extent of existing land within the site that is less than the recommended Coastal Protection Board 2100 site level of 4.0m AHD.

Areas within the proposed residential and commercial zones identified on the Masterplan that have a ground level below 4.0m AHD will be filled to achieve this minimum requirement. Further fill above this level will be required on site in order to create fall on the land and to achieve drainage and minimum road grades.

Although the proposal is located several kilometres from the Gulf St Vincent, the site is linked to the Gulf via the Thompson Outfall Channel and would therefore be subject to tidal surge.

7.2 Recommendation

The recommended minimum site level is therefore 4.0m AHD with minimum floor levels of 4.25m AHD. It should be noted however, that due to the need to create falls across the site to drain the road system that the majority of properties will have site levels well in excess of the recommended minimum level.

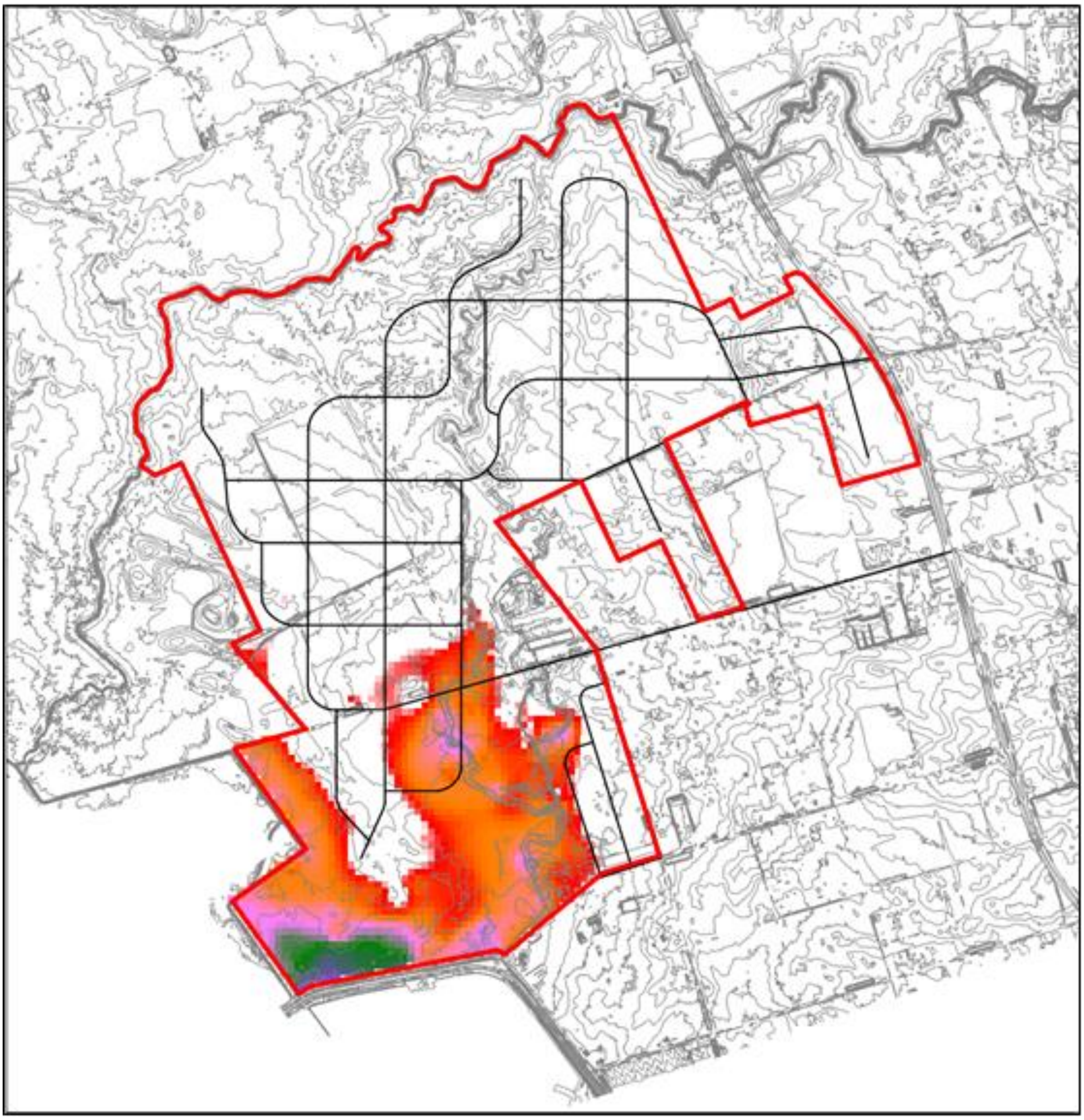

Figure 7-1: Extent of Existing Site Less Than 4.0m AHD

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8 SUMMARY

The following is a brief summary of the outcomes of this study.

8.1 Stormwater Management

- A Water Sensitive Urban Design approach will be applied across the entire site and is incorporated in the Masterplan.
- At least two significant wetlands will be developed on the higher sections of the site.
- Shallow groundwater levels confine the construction of wetland and water bodies to the higher areas of the site.
- A series of lineal stormwater management corridors will be constructed to manage minor stormwater flow water quality treatment and for the passage of major flows from the site prior to the proposed Salt Water Lake systems or the open channel drainage network. These are incorporated in the Masterplan.
- A series of major channels will also act as capture channels to intercept flood water 'breakouts' from the Gawler River and provide protection for the 1%AEP flood. These are incorporated in the Masterplan.
- Site level collection of stormwater for reuse will be adopted where practical.
- An on site detention above the Salt Water Lakes, and at the southern most portion of the site are proposed to control post development flows to less than predevelopment levels.
- ASR potential on the site is limited to 50ML/a which is significantly less than the estimated annual runoff. ASR is no longer being consider for the site.

8.2 Wastewater

- A vacuum sewer scheme is proposed to accommodate the shallow groundwater levels across the site which will include approximately 5 vacuum pump stations.
- All wastewater will be pumped to the Bolivar Wastewater Treatment Plant.
- An interim series of rising mains and pump stations with boosters will be developed to deliver wastewater to Bolivar in a staged manner
- The ultimate rising main to Bolivar is likely to 2 x 300mm rising mains.

8.3 Potable Water

- Potable water supply to the site will come from the Little Para Water Treatment Plant.
- Short term options have been proposed by SA Water that can supply up to 1100 services.
- The ultimate scheme will require a new supply main from the Little Para system that is based on a number of pipeline upgrades and extensions.
- Walker is working with SA Water to ensure short, medium and long-term portable water solutions are in place as each stage is progressed.

8.4 Recycled Water

- A third pipe system will be provided throughout the site for irrigation purposes only through the NAIS scheme developed by SA Water.

9 GLOSSARY OF TERMS

AHD = Above height datum

AMC = Antecedent moisture content

ARI = Average recurrence interval

ASS = Acid sulphate soil

EL = Elevation level

FFL= Finished floor level

GL = Giga litres

GPT = Gross pollutant trap

GRC = Glass reinforced concrete

HEC-RAS = Hydrologic Engineering Centre river analysis system

LGA = Local government association

MHWS = Mean high water springs

ML = Mega litre

ML/a = Megalitres per annum

MUSIC = Model for urban stormwater improvement conceptualisation

PASS = Potential acid sulphate soil

ppm = Parts per million

PRV = Pressure release valve

RO = Reverse osmosis

SCADA = Supervisory control and data acquisition

SL = Surface level

STED = Septic tank effluent disposal

TDS = Total dissolved solids

TP = Total phosphorus

TSS = Total suspended solids

WRSV = Western reticulation scheme Virginia

WTP = Water treatment plant

WWTP = Waste water treatment plant

10 REFERENCES

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4. CSIRO, Climate Change in Australia, prepared for the Department of the Environment and Water Resources, October 2007
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6. IE Aust, T, Wong, Australian Rainfall and Runoff Quality- A guide to water sensitive urban design, 2006
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8. SA Environment Protection (Water Quality) Policy 2003 – Schedule 2
9. SA Water, Memorandum: Ultimate water supply for the Riverlea Park/Waterloo Corner Area, prepared by Paul Feronas for Wallbridge & Gilbert, September 2008
10. Sinclair Knight Merz, Groundwater investigation – Riverlea Park proposal, prepared for Walker Corporation, November 2008
11. Tonkin Consulting, Port Adelaide Seawater Stormwater Flooding Study, prepared for the City of Playford Council, October 2005
12. Tonkin Consulting, Western Catchment - Stormwater Master Plan, prepared for the City of Playford Council, April 2008

APPENDIX A
EIS GUIDELINES

3.1.1 Determine the flood potential for the area, including flood plain mapping for a 1 in 100year ARI storm, as a result of the restriction of the floodplain in the vicinity of the proposed development and taking into account the construction of a dam on the North Para River.

Section 4

3.1.2 Outline the requirements for the likely location of water, sewerage, stormwater management infrastructure.

Section 2, 4, 5, 6

3.1.3 Describe the approach to water sustainability, including ways in which mains water supply use can be minimised or supplemented and opportunities for reducing and recycling water, particularly stormwater and waste water from the Virginia Pipeline through Water Sensitive Urban Design (WSUD).

Section 6

3.1.4 Identify opportunities for the reuse of grey water.

Section 6

3.1.5 Detail measures to minimise impacts and to protect the Gawler River and coastal environments during both the construction phase and on an ongoing basis.

Section 2

3.1.6 Identify the impact of possible erosion, subsidence or inundation as a result of flooding arising from construction on this low lying part of the coast.

Section 1.2

3.1.7 Describe the connection to water supply for the proposed development, the required upgrading or provision of pipelines and the implications for water sources, include information on the quantity of potable water required.

Section 6

3.1.8 Describe the proposed method of dealing with wastewaters.

Section 5 and 6

3.1.9 Describe measures to protect, maintain and monitor suitable water quality in waterways.

Section 3

4.2.11 Outline measures to prevent soil, fertilizers, herbicides and pesticides derived from residential allotments and open space reserves from entering the waterways.

Section 3

4.2.12 Identify the potential effects as a result of stormwater runoff on the St Kilda-Chapman Creek and Barker Inlet-St Kilda Aquatic Reserves (nursery areas) ecosystem and fish breeding grounds.

Section 1.2 and Section 3

4.2.13 Identify the potential effects of the proposal on the adjacent salt operations (intake water quality issues) such as storm water discharge, nutrients management, sewage management, waste management, water pollution from littering and illegal dumping, oil and fuel spill management, wash down and toxic seepage.

Section 3

4.2.19 Describe the proposal of excavated materials for the proposed waterways.

Section 2 and 7

4.2.20 Describe how the proposal will comply with the coastal flooding policy outlined in the Development Plan.

Section 7

4.2.24 Describe any special engineering requirements for infrastructure due to the expected high water table in this area including the costs of developing and maintaining infrastructure for saline and acid sulphate soils, seasonal variations in height and groundwater rise due to sea level rise.

Section 5 and 7

4.3.5 Describe the requirements of the sea level rise policies in the Development Plan and how these would be achieved in undertaking this proposed development.

Section 7

4.3.7 Describe any impacts on the neighbouring Port Gawler Conservation Park, adjacent Crown land and the Riverlea Park Lake System.

Section 3

4.3.8 Outline the potential effects of climate change from a risk management perspective, including adaptive management strategies.

Section 2 and 7

4.3.31 Describe the likely effects on marine organisms and seagrasses, in the context of runoff from the proposed development into the river and out to sea potentially reducing the salinity and increasing nutrients, suspended sediments and pollutants, particularly heavy metals.

4.7.1 Describe the condition and capacity of existing trunk infrastructure and the likely impacts of the development on that capacity.

Section 6

4.11.6 Describe how the proposal would comply with the requirements under the Environment Protection Act, 1993 and the Adelaide Dolphin Sanctuary Act, 2005 and the duty of care under these Acts.

Section 3

APPENDIX B
GAWLER RIVER FLOOD MAPS

MEMORANDUM

To Brent Eddy

From Alison Miller

Date 31 October 2022

Subject Modelling of Riverlea development in the broader Gawler River floodplain model

Riverlea is a proposed housing development at Buckland Park, currently under development by Walker Corporation. Water Technology have been engaged at various stages of the project to provide advice on riverine flood impacts at the development site and adjacent properties.

This memo documents the hydraulic modelling undertaken to assess the performance of the proposed division of floodwaters from the Gawler River along the western side of the development. Modelling was undertaken in the broader Gawler River floodplain model, versions of which are currently being used in the development of the Gawler Stormwater Management Plan and for the Enhanced Flood Hazard Mapping project.

MODEL DETAILS

The existing conditions model, currently being developed for the Enhanced Flood Hazard Mapping project, was adopted as the base case for assessment of the Riverlea development. The model is a coupled MikeFlood model, with the river and floodplain represented in 2D (Mike21), linked to 1D representation of culverts (Mike11).

Topography

The model adopts a flexible mesh representation, which allows higher resolution detail to be incorporated in the model where required (e.g. along the river) without dramatically increasing run times. The model adopts elevations from the two recently captured LiDAR datasets:

- Middle Beach 50cm LiDAR, captured 26 November 2021
- Adelaide Metro LiDAR, captured 21-31 January 2022.

The two datasets overlap along the alignment of the Gawler River. Where this has occurred, the 2022 data has been used in preference.

Note that the only difference between the model adopted for this assessment, and that in development for the Gawler SMP, is the underlying topography. The Gawler SMP model adopts the 2021 LiDAR, but the topography on the south-eastern side of the river alignment is based on a series of earlier topographic datasets.

The model incorporates 344 dike structures, which have been used to control the level at which water can move across various areas. Typically, these are representative of levees, however dikes have also been used to incorporate other key features such as road crests, where the element vertex sampling may have missed this detail. Crest elevations for each dike have been sampled from the 2021 or 2022 LiDAR.

Inflow/outflow boundaries

Inflow boundaries to the model were retained, and include:

- A hydrograph input for the South Para River at South East of Gawler
- A hydrograph input for the North Para River downstream of Turretfield.

Note that the hydrology inputs were derived from the XP-RAFTS hydrology model which incorporates the Bruce Eastick Dam and the upgraded South Para Dam. Hydrographs to the model were extracted at the spatial location of the hydraulic model. This is downstream of the South Para Dam (hence the flood mitigation is incorporated in the hydrology) and upstream of the Bruce Eastick Dam (flood mitigation here is incorporated in the hydraulic model).

A sea level of 1.5 mAHD (equivalent to the Highest Astronomical Tide) was applied as a downstream boundary along the western and (partial) southern model edges. This has been retained form the original study in 2008 which assessed tidal data for Port Adelaide and Outer Harbour.

A second ‘free outflow’ boundary has been incorporated on the southern edge of the model further upstream, on the western side of the Northern Expressway. This was to prevent breakouts from the Gawler River from artificially ponding at the model edge. In reality, this water is anticipated to flow initially south-west and then further west to meet other breakout flows from the Gawler River near Port Wakefield Road.

Infrastructure

All major bridges and culverts, of which there are 89, have been incorporated in the 1D domain. These were adopted from the previous Light River and Smith Creek models. Where these relate to drainage infrastructure for the Northern Expressway, these have been validated against details in the DRAINS model provided by City of Playford.

Where the mesh resolution was coarser than the width of the culvert/bridge outlet, the elevation of the linking cell has generally required altering to represent the invert.

Updates for the current assessment

The underlying mesh was refined across the area of the Riverlea site, to ensure sufficient resolution to capture the proposed development layout of swales. As a result of changes to the mesh, existing conditions have also been updated to ensure the same representation of detail.

The proposed development conditions have been represented by sampling a digital elevation model of the proposed conditions, created from the design drawing provided by Walker Corporation ‘Riverlea_Existing+Sitewide EW_05092022.dwg’.

Further details of the model schematisation will be made available through the Enhanced Flood Hazard Mapping project report for the Gawler River.

Note that the model is currently undergoing validation, and further refinements will be made. This will include re-enforcement of the bank levels on the eastern side of the Gawler River near Windermere. The model version adopted here, is appropriate for comparing like-for-like but may not necessarily be representative of actual flood levels, depending on the outcome of the validation process.

SCENARIOS

Scenarios analysed for this assessment include:

- Current conditions (referred to as 'existing').
- Future development conditions.

The digital elevation model for the proposed developed conditions can be seen in Figure 1. The proposed design includes a concept for diverting breakouts from the Gawler River into a zone along the northern edge of the development, conveying floodwaters along the north and western borders to a discharge point at the south-western corner.

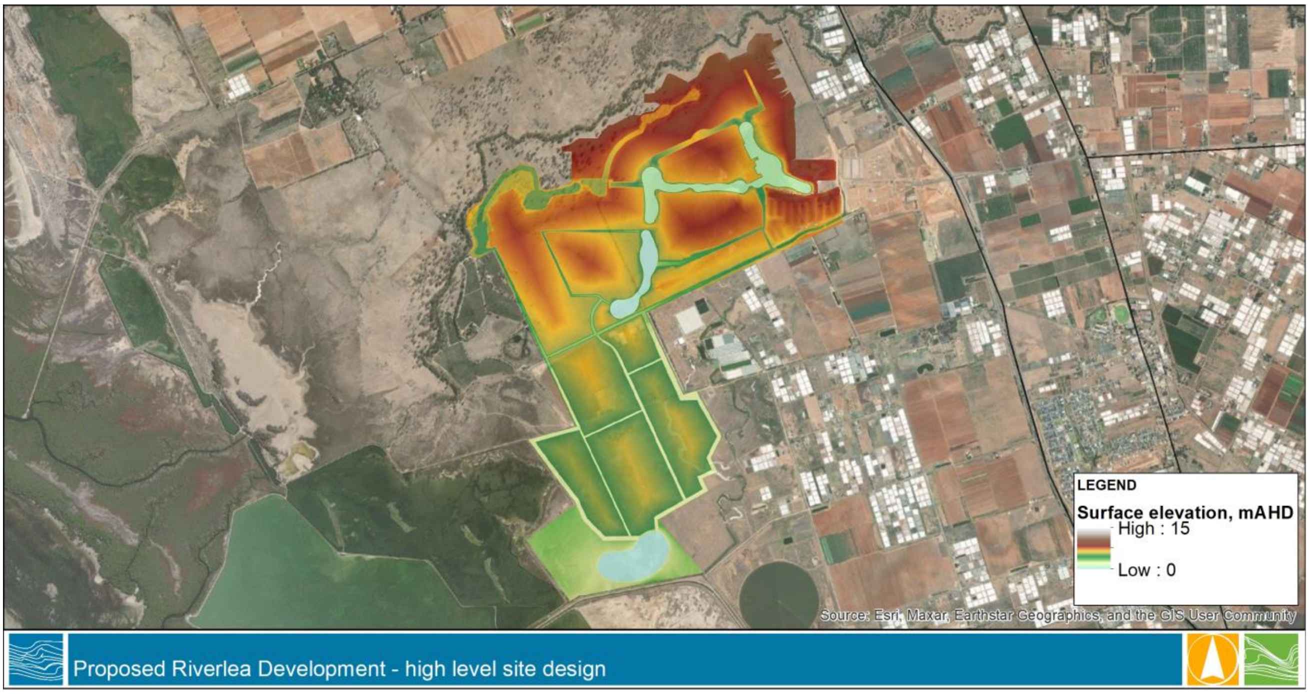


Figure 1 Proposed development surface elevations

RESULTS

The resulting flood depth for the 1% AEP flood event in the Gawler River for the current and future development scenarios is provided in Attachment 1 and 2. The scheme to divert breakouts to the south-western corner works as intended, however it demonstrates that the floodwaters are diverted from the location further west than intended.

The developed conditions (Attachment 2) show an extensive area of flooding surrounding the most southern basin, near the existing salt pans. While the majority of this area is inundated in existing conditions, refinement to the outflow path may need to be considered.

Differences in 1% AEP flood levels between the two scenarios is shown in Figure 2 (and Attachment 3). The results indicate reduced flooding along the western portion of the development (i.e. 'was wet now dry'), and reduced flood levels further west and south of the site.

Note that the existing conditions 1% AEP flood extent differs slightly to that provided previously. Output from the previously adopted TUFLOW site specific model indicated floodwaters breakout out near the intersection with Port Wakefield Road to south of the Gawler River, inundating the existing greenhouses and extending south-west across the Riverlea site. This breakout flow is not observed in the updated modelling adopted here as the bank heights have been more accurately represented through the adoption of recently captured 2022 LiDAR.

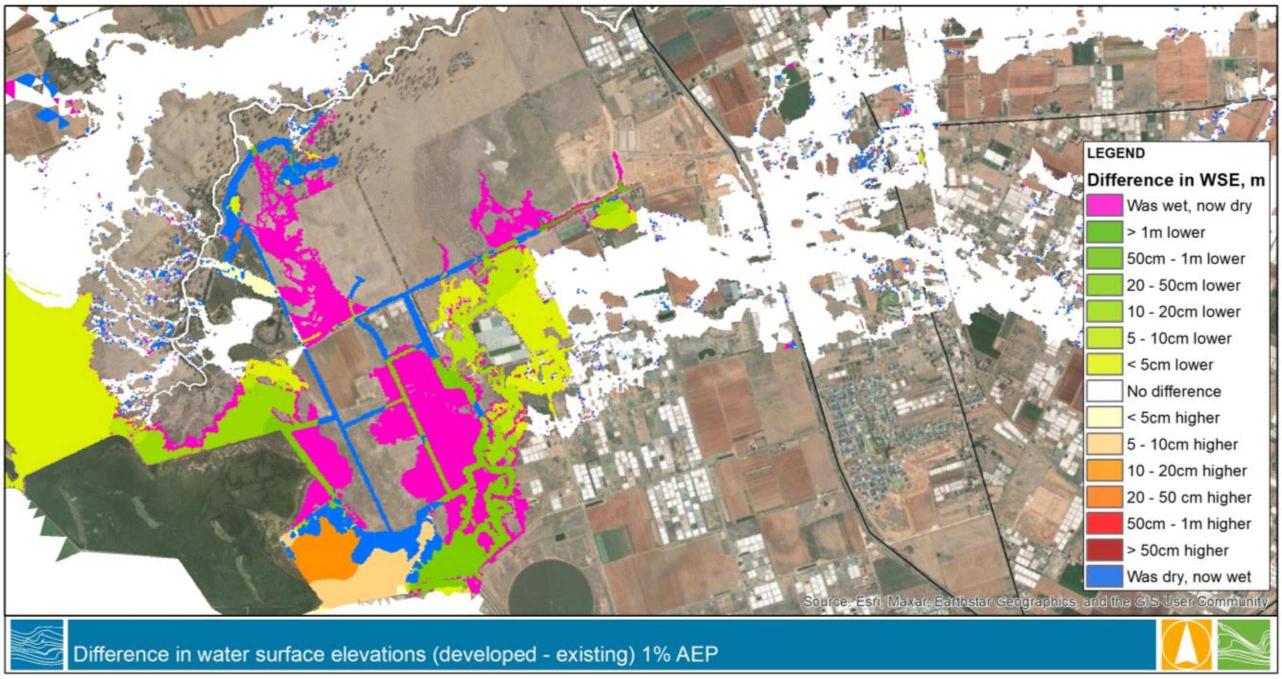


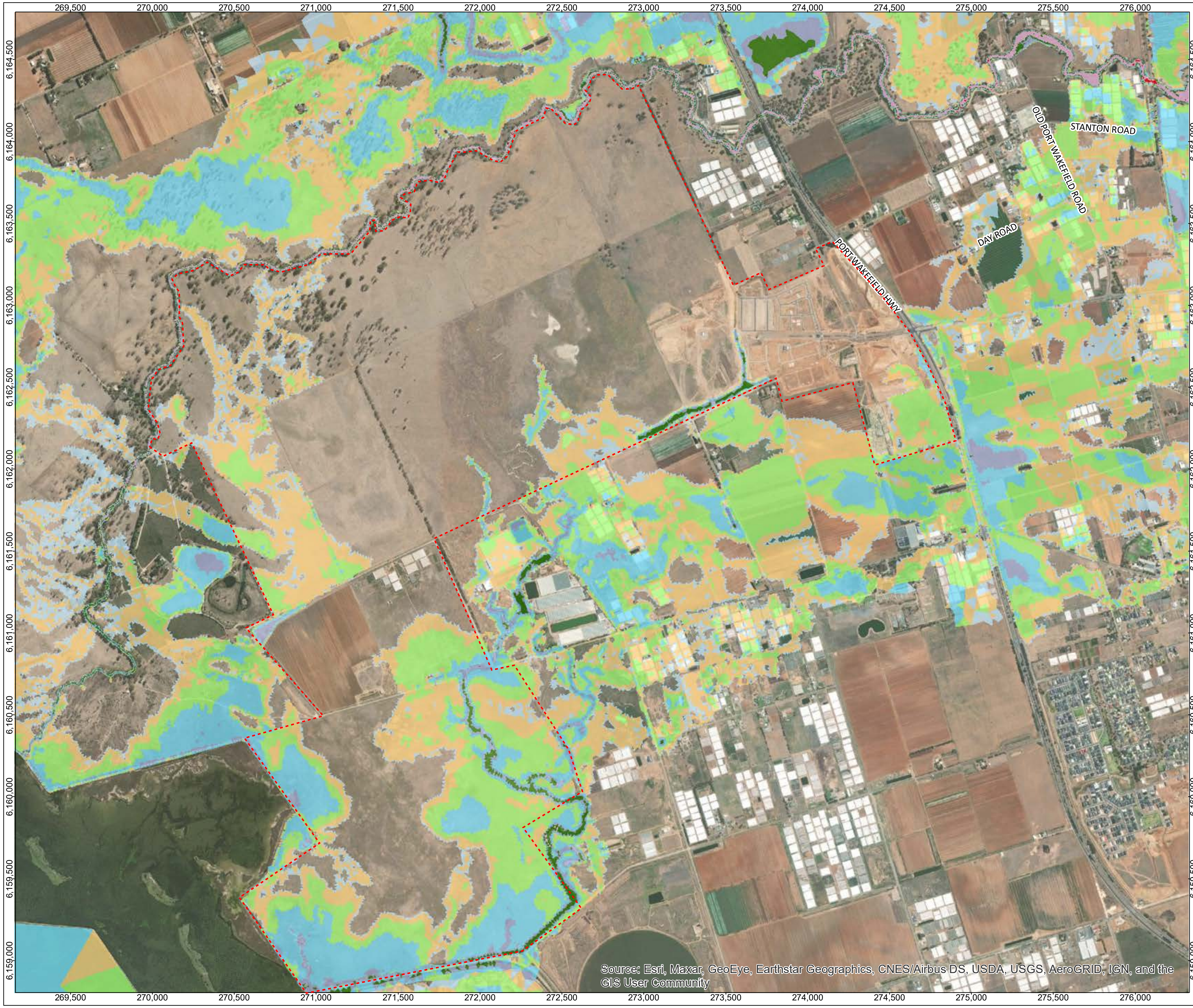
Figure 2 1% AEP flood depth for current development conditions across site

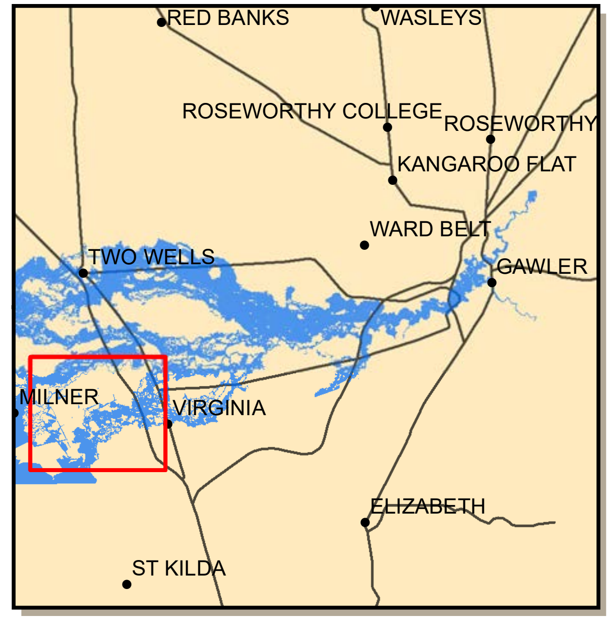
Enclosed:

Attachment 1 – 1% AEP flood depth, existing conditions

Attachment 2 – 1% AEP flood depth, proposed development conditions

Attachment 3 – 1% AEP difference in water surface elevation (developed minus existing)





Legend

Depth, m

- 0 - 0.1
- 0.1 - 0.25
- 0.25 - 0.5
- 0.5 - 1
- 1 - 1.5
- 1.5 - 2.5
- 2.5 - 5
- Greater than 5

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0 250 500 1,000 Meters

Coordinate System: GDA 1994 MGA Zone 54

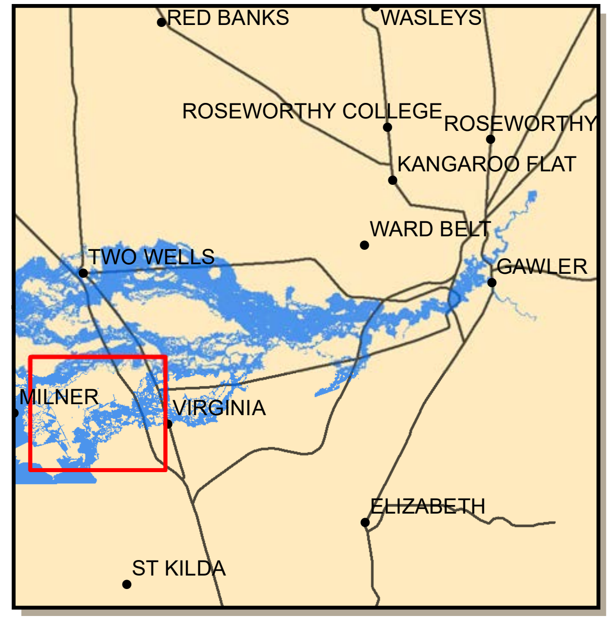
1:25,000 at A3

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Existing Conditions 1% AEP Depth
Riverlea Development Site

REFERENCE: A3L_wInset_Riverlea_Ex_1AEP_DEPTH

DATE: 28/10/2022 | SHEET: 1 of 1 | DRAWING NUMBER: 1



Legend

Depth, m

- 0 - 0.1
- 0.1 - 0.25
- 0.25 - 0.5
- 0.5 - 1
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0 250 500 1,000 Meters

Coordinate System: GDA 1994 MGA Zone 54

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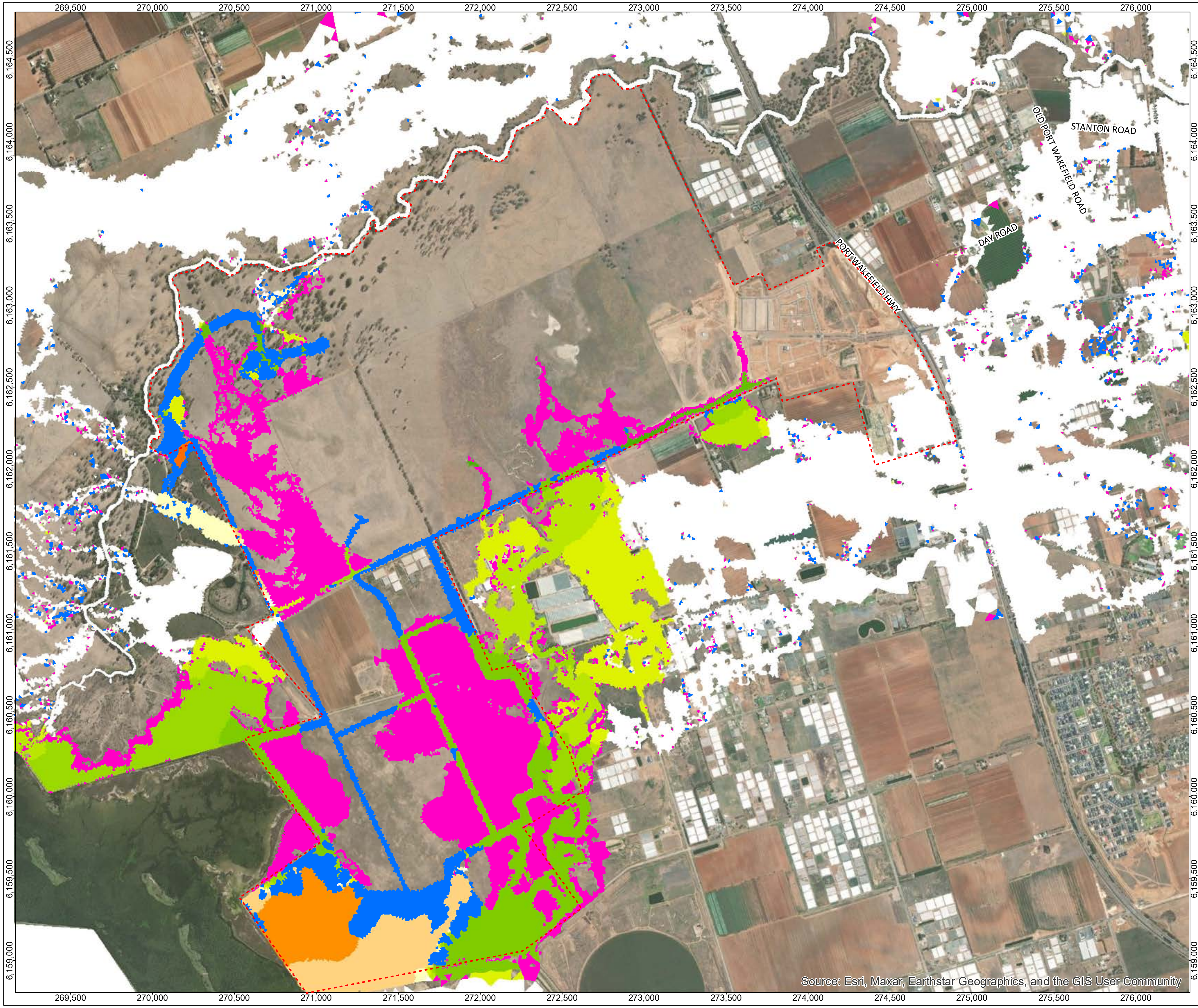
WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

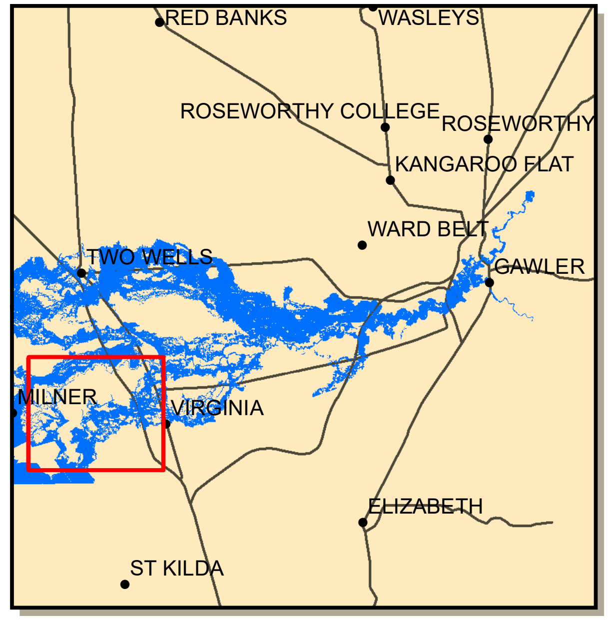
Developed Conditions 1% AEP Depth

Riverlea Development Site

REFERENCE: A3L_wInset_Riverlea_Dev_1AEP_DEPTH_v3

DATE: 28/10/2022 | SHEET: 1 of 1 | DRAWING NUMBER: 1





LEGEND

Difference in WSE, m

- Was wet, now dry
- > 1m lower
- 50cm - 1m lower
- 20 - 50cm lower
- 10 - 20cm lower
- 5 - 10cm lower
- < 5cm lower
- No difference
- < 5cm higher
- 5 - 10cm higher
- 10 - 20cm higher
- 20 - 50 cm higher
- 50cm - 1m higher
- > 50cm higher
- Was dry, now wet

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N E S W

0 250 500 1,000 Meters

Coordinate System: GDA 1994 MGA Zone 54

1:25,000 at A3

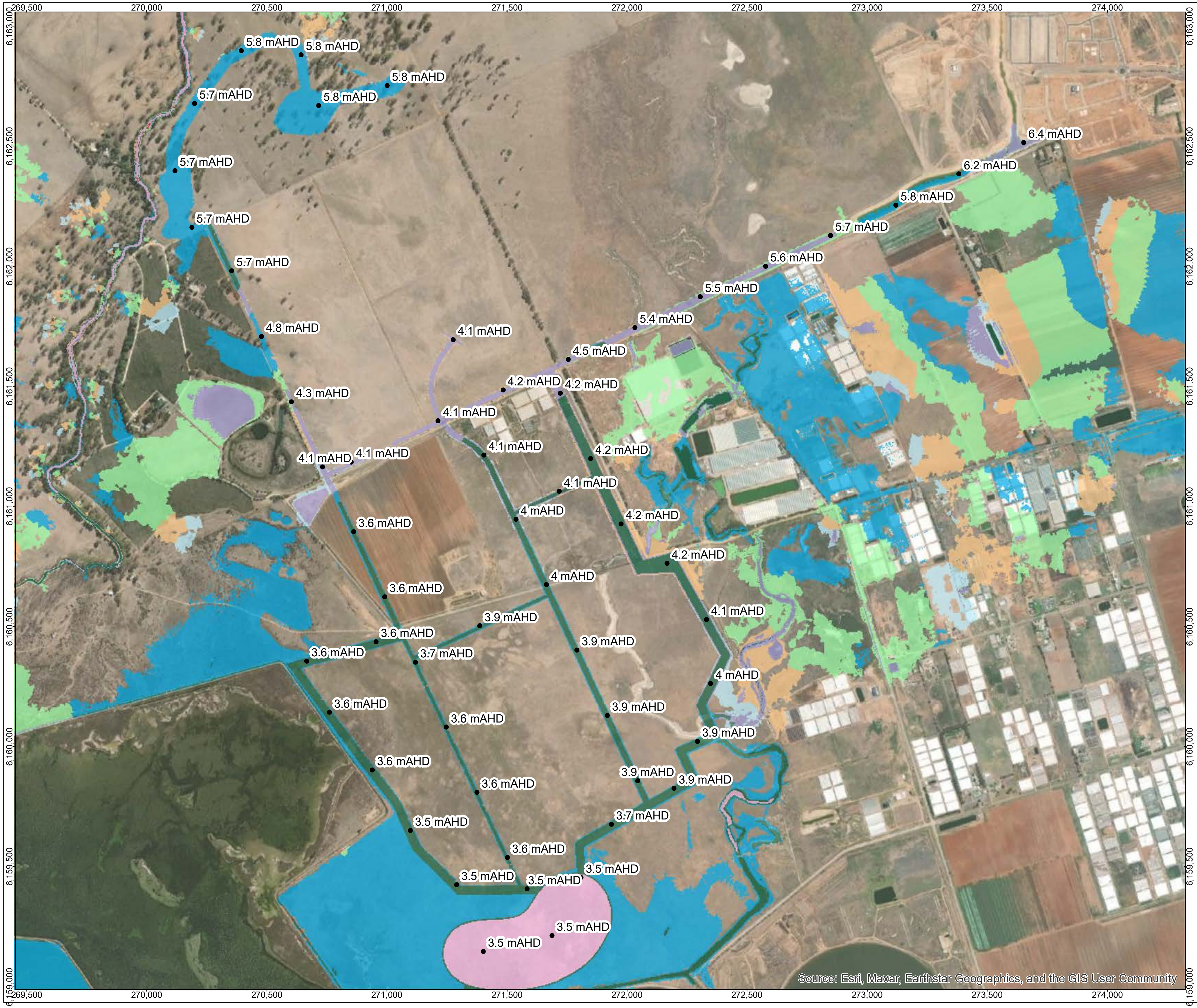
WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Difference in flood levels (Dev-Ex) 1% AEP Depth

Riverlea Development Site

REFERENCE: A3L_wInset_Riverlea_Dev-Ex_1AEP_DIFF_v2

DATE: 31/10/2022 | SHEET: 1 of 1 | DRAWING NUMBER: 1



Legend

- • Water surface elevation at point

Depth, m

- 0 - 0.1
- 0.1 - 0.25
- 0.25 - 0.5
- 0.5 - 1
- 1 - 1.5
- 1.5 - 2.5
- 2.5 - 5
- Greater than 5

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N W E S

0 250 500 1,000 Meters

Coordinate System: GDA 1994 MGA Zone 54 1:17,000at A3

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Developed Conditions 1% AEP Depth
Riverlea Development Site

REFERENCE: A3L_wInset_Riverlea_Dev_1AEP_DEPTH_v3_w_WSEpts
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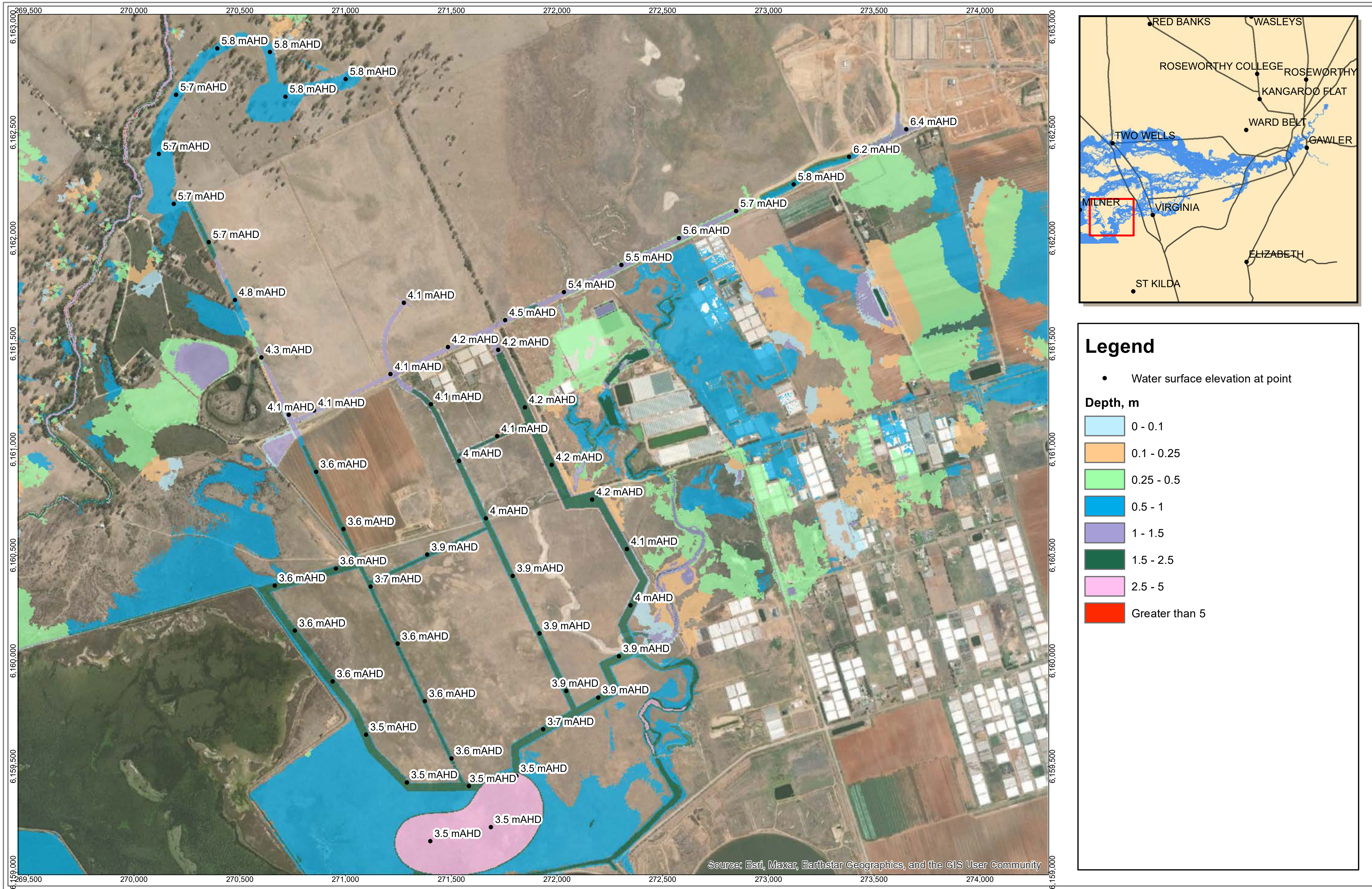

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Coordinate System: GDA 1994 MGA Zone 54

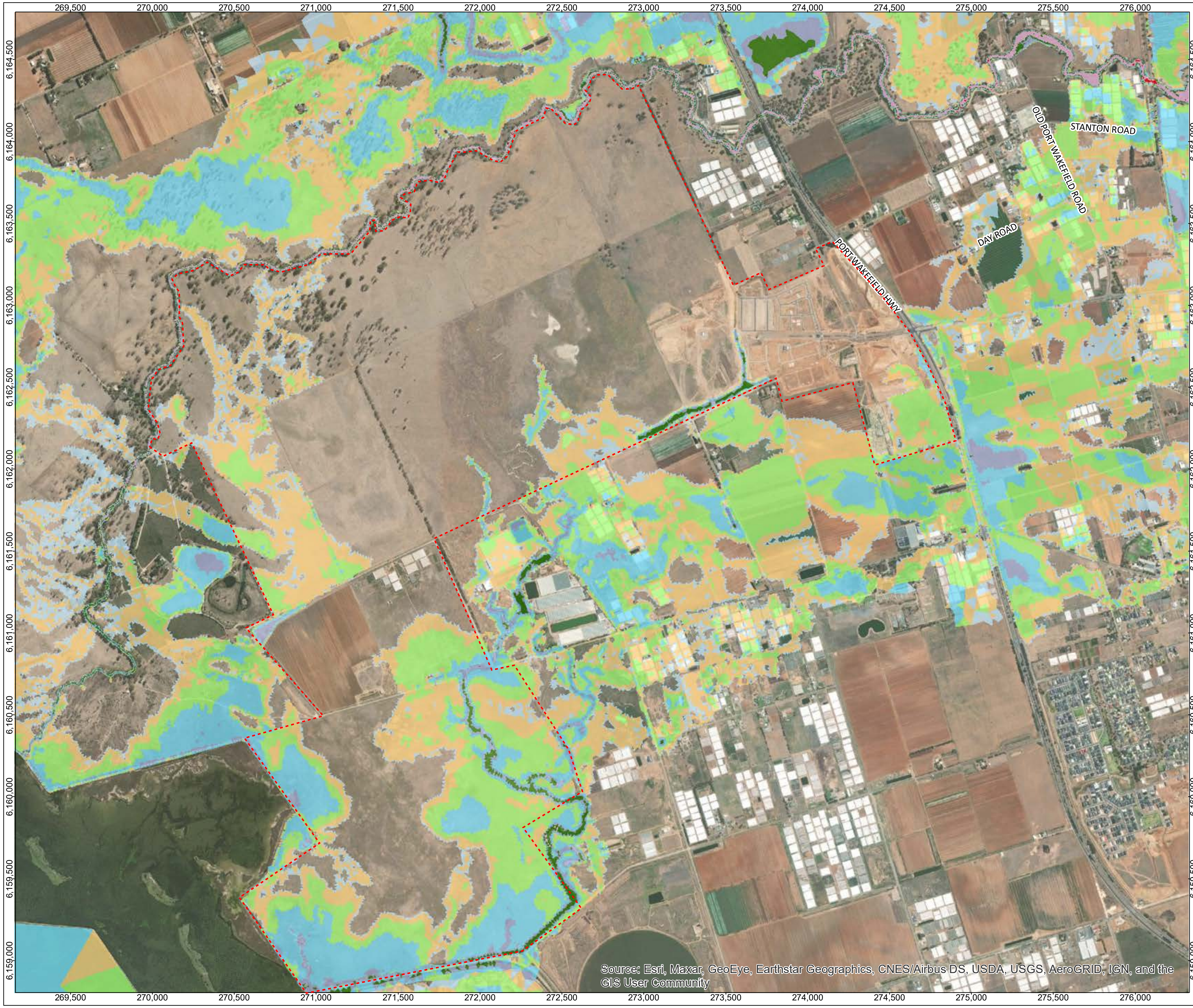
1:17,000at A3

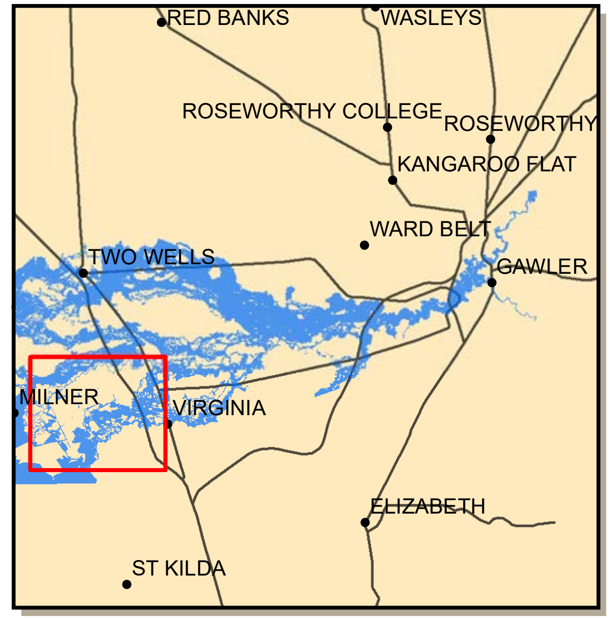
WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Developed Conditions 1% AEP Depth
Riverlea Development Site

REFERENCE: A3L_wInset_Riverlea_Dev_1AEP_DEPTH_v3_w_WSEpts

DATE: 2/11/2022 | SHEET: 1 of 1 | DRAWING NUMBER: 1





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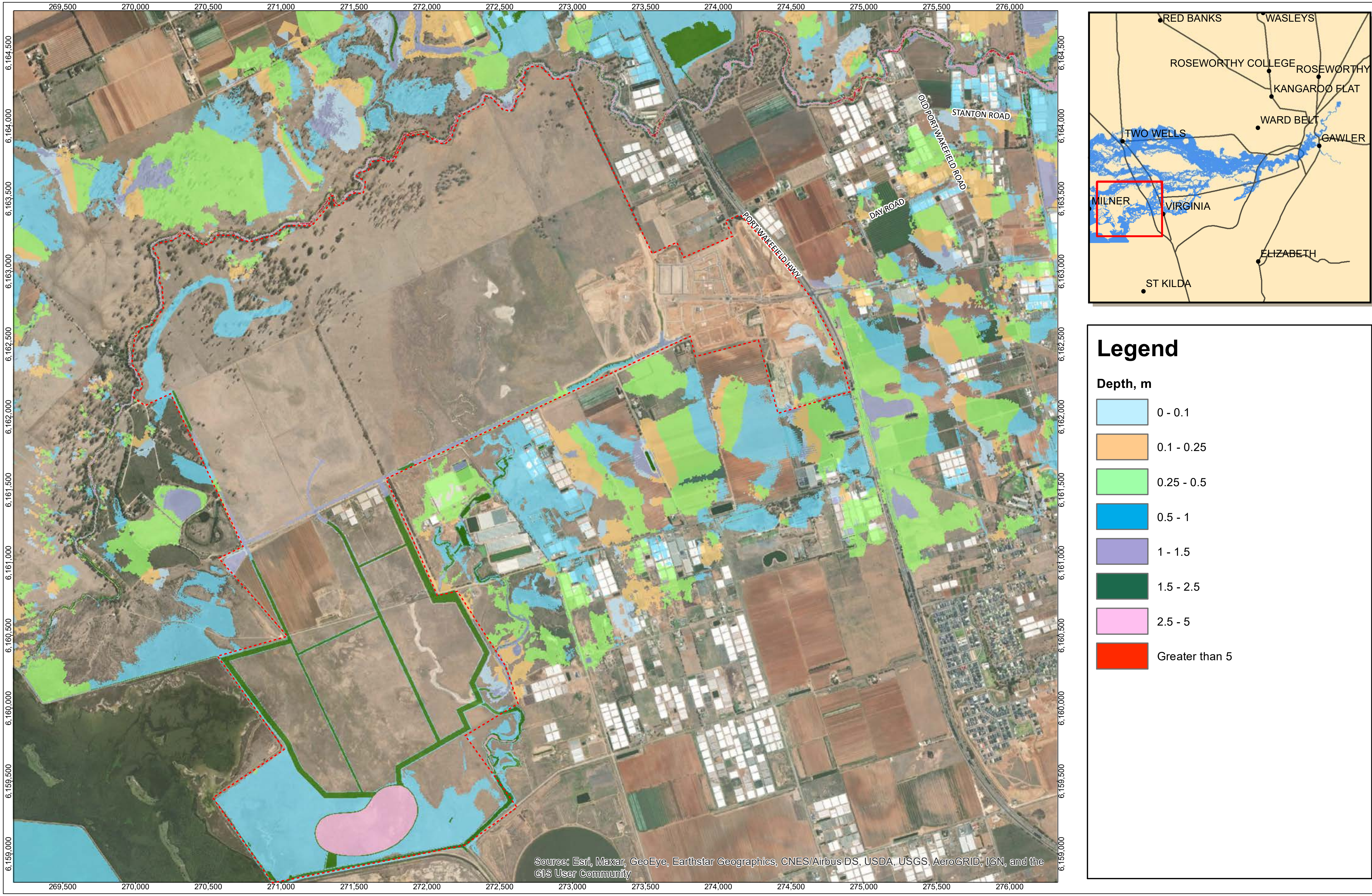
WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Existing Conditions 1% AEP Depth

Riverlea Development Site

REFERENCE: A3L_wInset_Riverlea_Ex_1AEP_DEPTH

DATE: 28/10/2022 | SHEET: 1 of 1 | DRAWING NUMBER: 1



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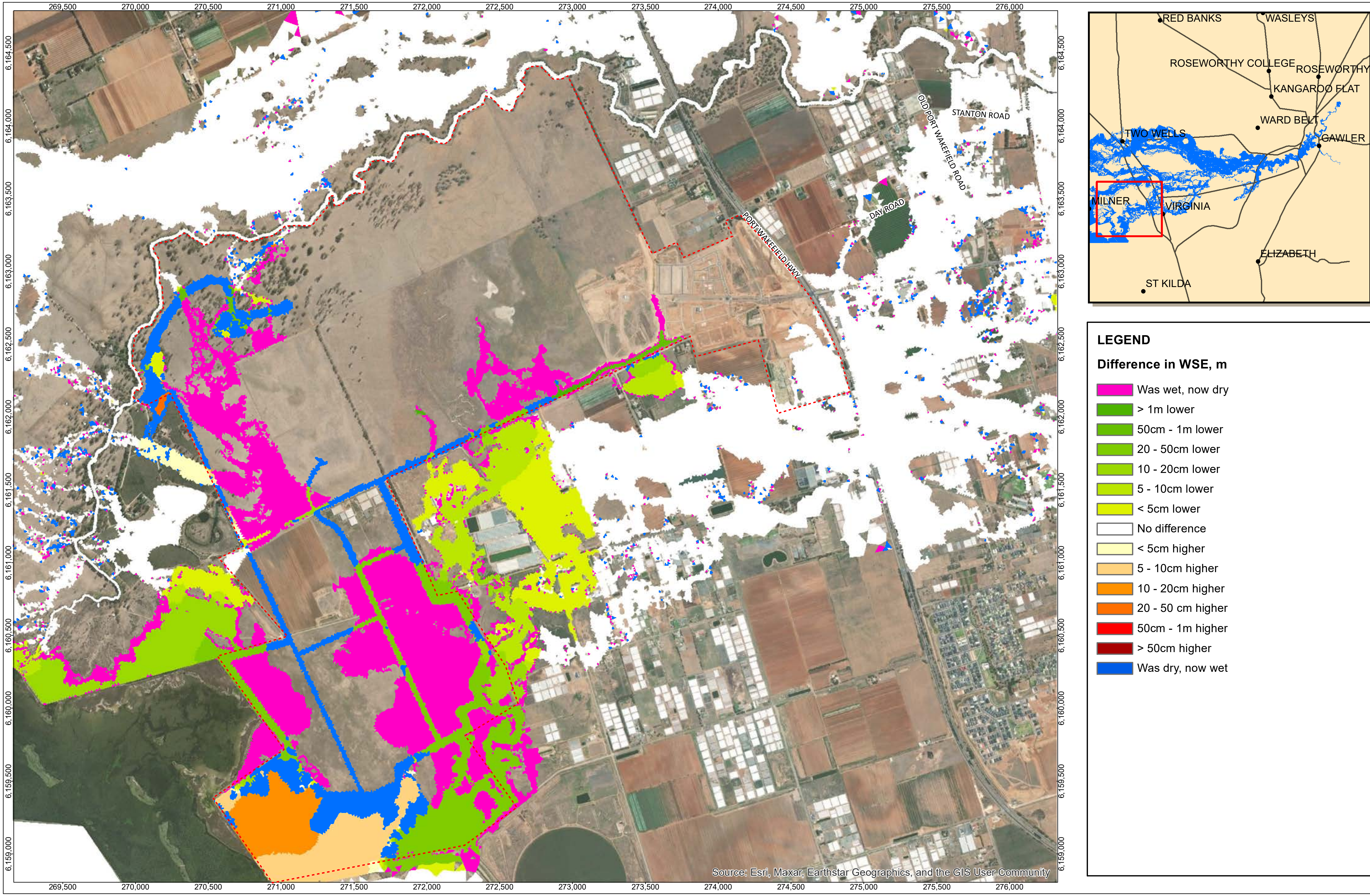
WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Developed Conditions 1% AEP Depth
Riverlea Development Site

REFERENCE: A3L_wInset_Riverlea_Dev_1AEP_DEPTH_v3

DATE: 28/10/2022 | SHEET: 1 of 1 | DRAWING NUMBER: 1



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N E S W

0 250 500 1,000 Meters

Coordinate System: GDA 1994 MGA Zone 54

1:25,000 at A3

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

Difference in flood levels (Dev-Ex) 1% AEP Depth
Riverlea Development Site

REFERENCE: A3L_wInset_Riverlea_Dev-Ex_1AEP_DIFF_v2
DATE: 31/10/2022 | SHEET: 1 of 1 | DRAWING NUMBER: 1

APPENDIX C
WGA ASSESSMENT OF WASTE WATER TREATMENT METHODS

BUCKLAND PARK
WASTEWATER COLLECTION SYSTEMS

NETWORK OPTIONS ASSESSMENT

prepared for

WALKER CORPORATION

by

WALLBRIDGE & GILBERT
Consulting Engineers

NOVEMBER 2008

REV E

Job No: C080163

CONTENTS

EXECUTIVE SUMMARY

Wallbridge & Gilbert (W&G) were engaged by the Walker Corporation to undertake a first order assessment of the most economically and technologically suitable form of communal collection system for domestic wastewater within the Buckland Park proposal.

Gravity drainage systems are commonly thought to be the most economically viable wastewater management systems. However, issues such as a high water table, acid sulphate soils and high salinity levels within the groundwater at Buckland Park mean that the cost to build a gravity system could escalate, and there would be an increased potential for groundwater ingress into the system. These factors prompted the need to investigate the viability of vacuum and pressure systems.

This report assesses the applicability of the following four collection systems:

- Gravity
  - Septic Tank Effluent Disposal System (STEDS)
  - Full Sewer
- Pressure
- Vacuum

A general technical description of the characteristics of each scheme as well as a summary of the advantages and disadvantages is enclosed within this report.

First order cost estimates for each of the systems have been presented with the all of life costs derived using the LGA's CWMS (Community Waste Management System) all of life cost model.

The latest groundwater mapping indicates that the depth to ground water across the site varies from 0.2m to 7m below the current surface level. In the order of 60% of the proposal site has a ground water depth of less than 2m from the surface level. Seasonal water level fluctuations in the order of 0.5m to 1m could be expected.

The key impacts that the high ground water table, acid sulphate soils and saline ground water conditions have on the suitability of a waste management system include:

- Increased construction costs for deeper drains, manholes and pump stations
- Increased risk of cost escalation during the construction phase especially if construction is undertaken in years when the seasonal variation in groundwater is higher than current measurements
- Increased OHS&W risks associated with construction of the drains (this adds to the cost for mitigation but also to the potential of an accident)
- Increased risk of system failure from overflow due to ground water intrusion into the wastewater management network.

- Increased running costs associated with pumping and treatment systems
- Increase in capital costs to cater for emergency storage or increased pump sizes to cater for peak wet weather flows
- Greater potential for a higher salinity within treated effluent, therefore limiting potential reuse applications
- Increases the risk of future settlement of reinstated trenches due to difficulties in achieving compaction.
- Potential to create a greater trench footprint due to collapsing trenches during construction.
- Risk of creating acidic soil conditions due to construction in acid sulphate soils, also creating the potential to transport leachate along the trench spreading the extent of the potential impacts.

Table E1 (on page 3) summarises the capital and all of life cost estimates for the various collection options assessed. These costs are for comparative purposes only and have been based on indicative layouts. Cost estimates of the preferred option would be produced after a preliminary design has been completed.

The following assumptions need to be considered when reading the table:

1) The gravity sewer concept is based on a maximum drain depth of 3m. This results in the order of 35 pump stations being required to service the proposal.
2) Vacuum sewer is based on 3 vacuum stations and an average of 5 connections per valve pit.
3) It has been assumed that the capital costs for scheme installation are expended in year 1. In practice this will not be the case but the costings are for comparative purposes only and are not intended as an absolute measure of the all of life costs.
4) The costs do not include treatment or disposal, they relate to the collection network only.
5) The all of life costs shown in the summary table do not include an allowance for increased operational costs due to ground water ingress, as the impact is difficult to estimate.
6) The costs of installation and maintenance of property pumps has been included in the pressure system. This cost is often excluded from cost estimates for schemes in South Australia as traditionally these costs have been met by the individual land owner, however W&G believe that if a true cost comparison is to be made between the schemes then these costs should be included.

Table E1 – Summary of Comparative costs

Effluent		
Discount Rate	Capital Cost	All of Life Cost
4%	$78,100,000	$100,900,000
Sewer		
Discount Rate	Capital Cost	All of Life Cost
4%	$41,800,000	$58,400,000
Pressure		
Discount Rate	Capital Cost	All of Life Cost
4%	$132,700,000	$248,000,000
Vacuum		
Discount Rate	Capital Cost	All of Life Cost
4%	$37,900,000	$55,300,000

Recommended Collection System for Buckland Park

W&G recommend that design development be based on a vacuum sewerage system.

The reasons for recommending this option include:

- The lower estimated capital cost and all of life costs
- The reduced potential impacts of salinity on the reuse applications
- Lesser impact of peak wet weather flows on the WWTP and pump stations
- Lesser potential for long term ground water ingress
- Reduced potential for system overflow at the pump stations during peak wet weather events or power outages
- Reduced risk of system failure due to groundwater ingress
- Lower pumping costs associated with limited groundwater ingress (which is not captured in Table 5.1.1)
- Reduced operational requirements in a major power failure scenario
- Approximately 75% of drains in a gravity system would be installed below the current ground water levels, even with the installation of 35 pumping stations
- Aeration of the sewage through the collection network will have a positive impact on the WWTP operation.

1. INTRODUCTION

Wallbridge & Gilbert (W&G) were engaged by the Walker Corporation to undertake an assessment to determine the most appropriate wastewater collection system for the Buckland Park proposal.

It is currently envisaged that the Buckland Park proposal will ultimately consist of 12,000 properties with a likely ultimate population of up to 33,000 persons.

This report summarises the general characteristics of the following four collection systems:

- Gravity
  - Septic Tank Effluent Disposal System (STEDS)
  - Full Sewer
- Pressure
- Vacuum

It outlines the suitability of each of the systems as applicable to Buckland Park, as well as comparing estimates of the capital and all of life costs that could be expected for each of the systems.

Section 3 and 4 of this report have been included as background knowledge for those who are not familiar with collection technologies and provide a general description of each of the systems and generic advantages and disadvantages of each.

2. PURPOSE

The purpose of this report is to:

- To enable comparison of the various collection system options available
- Inform the utility owner of the operational implications of each individual system inclusive of the impacts on future reuse applications
- Outline comparative all of life costs for operation of the schemes
- Recommend the most suitable option to adopt for design development.

3. GENERAL DESCRIPTION OF SCHEMES

Gravity

Gravity sewer systems are the oldest and most commonly used form of collection system utilised in South Australia.

There are two basic forms of gravity systems employed in South Australia. Generically these are full sewer (also known as conventional sewerage) and Septic Tank Effluent Disposal Schemes (STEDS).

A gravity system collects wastewater from all properties via gravity and as such the connection point has to be deep enough to drain the site. In steep terrain where land slopes away from the main drains, this can result in deep excavations for individual property owners.

Gravity systems grade downhill from the top of the catchment to the lowest point. A pump station is generally located at this point to pump the wastewater to a treatment facility, either directly or indirectly via other catchments.

The system consists of a network of main drains and individual property connections. An Inspection Point (IP) is located at all property boundaries, with the property owner being responsible for plumbing within the property and the authority for all drains downstream of the connection IP. The main drains may be in public land such as road reserves or within easements through private property.

At all significant changes of direction and regular spacings along straight runs, flushing points are installed. Flushing points take the form of maintenance holes or access chambers for full sewer systems but can be IP's (also known as risers) for STEDS.

Pump stations are used at low points in the catchment to lift the effluent to the treatment plant or to ensure the depth of the gravity drains is minimised. Placement of pumping stations is at the discretion of the designer and is dependent on the local conditions, which may limit the viability of installing deep gravity drains.

The major difference between the two gravity systems is that a full sewerage system transfers all wastewater from the property including solids, whereas the STED schemes utilise a septic tank at each individual residence to capture the solids and only transfer the effluent to the collection system.

This difference has resulted in STED schemes having smaller pipes laid at lesser grades. While the prior removal of solids significantly reduces the number of maintenance holes or access shafts required. This generally results in the collection network for a STED scheme being shallower and having a lower capital cost to install especially in existing communities where all properties have septic tanks operating. STED schemes require a septic tank pump out program to desludge each tank on a four yearly cycle.

Pressure

Pressure sewerage schemes are becoming more widely adopted in South Australia, particularly over the past 8 years. Each property is fitted with a storage tank. In South Australia, this tank is required to provide 600 L of emergency storage for a residential domestic dwelling. The pump chamber is placed directly inline with the house's plumbing and hence receives all wastewater from the dwelling (inclusive of solids). A typical pressure system layout for a residential property is shown in Figure 3.1

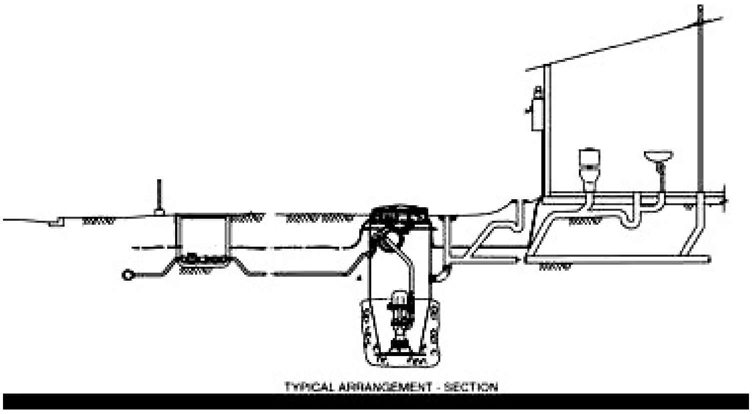

Figure 3.1 Pressure system layout

Diagram obtained from Environmental Systems Limited

For the pressure sewerage systems, a single grinder or cutter pump is installed in the pump sump to pump the wastewater from the property to the network. The network of drains may either deliver directly to a treatment facility or may pump to a main pumping station, which then transfers the wastewater to the treatment facility. Generally, where the treatment site is either elevated or a long distance from the network (i.e. high pumping heads) a transfer pumping station will be required. Over the past few years there have been significant developments in domestic pump units and several are now capable of duties approaching 50m head. A package system has recently been released to the market which is capable of pumping against a 60m head.

Each property connection consists of a 32mm connection line from the pump chamber to the main drain. A valve pit is to be located at each property boundary containing isolation valves and a non return valve.

Pressure systems have traditionally utilised centrifugal submersible pumps, but the introduction of more sophisticated control systems have resulted in positive displacement pumps also now being suitable for this application.

Pressure systems allow for the use of smaller bore drains than gravity systems and can be laid at shallower depths, as they do not require a minimum downhill grade and can be laid to the contour of the land.

Most of the pump supply companies in South Australia now market a package system suitable for installation in domestic situations. The quality and capability of each of the systems varies and needs to be assessed for the particular application.

Pump selection is a critical component of the design of a pressure network. Utilising pumps with performance curves that differ from that of the design can adversely impact on the system performance. Ensuring that the pumps specified in the design are actually installed requires vigilant monitoring and control. Most land owners will substitute the specified pump for cheaper alternatives if the installation is not monitored and strict controls placed on pump installation.

The reticulation network in a pressure system generally remains full of wastewater. Each time an individual property pumps into the system it forces wastewater in at the top end of the catchment and consequently out of the system at the outlet end. In large networks significant volumes of wastewater can be retained within the pipe network for long periods of time.

The period of time the wastewater remains in the network depends on the volume of the pipes within it and the volume of wastewater being pumped into it. The biochemical reaction occurring in the sewage/effluent quickly uses all available oxygen in the process. Once this occurs, anoxic or even anaerobic conditions are established, which causes septicity to occur, a by product of this process is hydrogen sulphide which is highly corrosive, toxic and at low concentrations has an unpleasant odour.

The potential for hydrogen sulphide generation within the systems will impact on the system design. The location of air valves need to be considered carefully so as not to position them in areas likely to be sensitive to odours. Head works at the treatment plant need to be designed to cater for the higher

Hydrogen sulphide load as it is highly corrosive. The gas can also be highly toxic, so safety of operators needs to be considered in the design. In addition to this the treatment process itself needs to account for the septic conditions particularly when calculating oxygen demands.

Vacuum

In a vacuum system, houses gravity feed to a chamber, usually located in the road reserve. Inside this chamber there is a level sensor, which activates the opening of the valve. The pressure in the pipeline is lower than that in the pit and the contents of the pit are effectively “sucked out”. The valve then closes to allow the network to maintain its vacuum. A significant volume of air is “sucked” into the line along with the wastewater. The wastewater slug that results from the valve opening and emptying the chamber soon disintegrates and flows via gravity to a low point in the system, where it reforms. Subsequent flows of air push the wastewater through the system to the vacuum/pump station.

Figure 3.2 shows the generic layout of a vacuum scheme

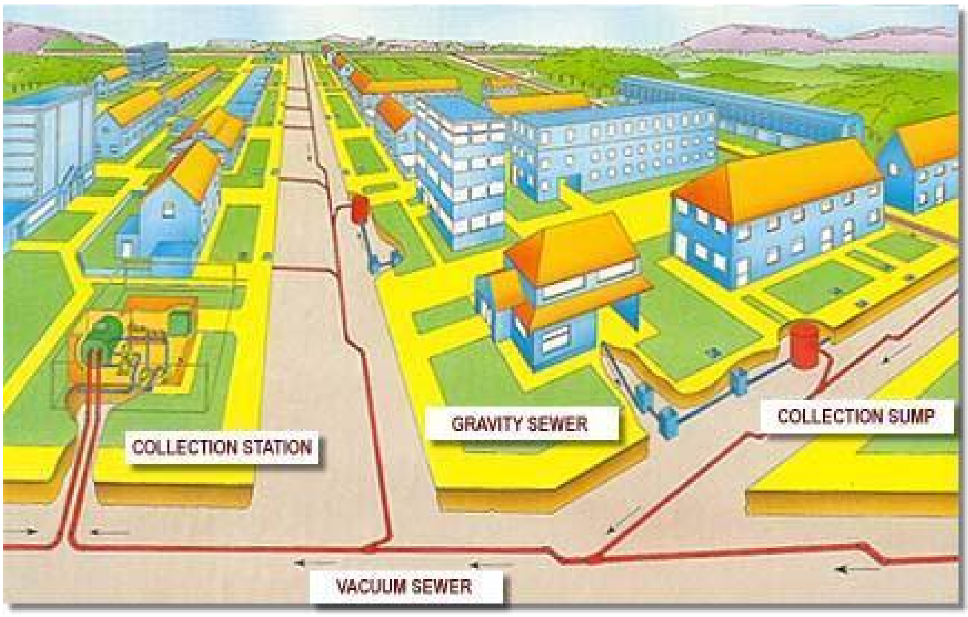

Fig 3.2 Typical Scheme Layout

Diagram provided by Flovac Pty Ltd

At the main pump station there are two types of pumps. One is used to create and maintain the vacuum in the pipe network, and the other is a conventional pump, which transfers the wastewater from the pump station to the treatment facility.

The chambers located outside the residential properties are generally service between 1 and 10 connections. The chambers vent via an 80-100mm vent located within each property. Unlike pressure or STEDS, vacuum systems do not require infrastructure other than the vent and drains on each individual property.

Care needs to be taken when situating the vents if the site is in a flood prone area. They will allow infiltration into the system if inundated.

The vacuum network is designed with a saw-tooth system, which allows a shallow depth to be maintained. Figure 3.3 outlines a typical detail for a property connection. It also outlines the saw-tooth arrangement for the main drains.

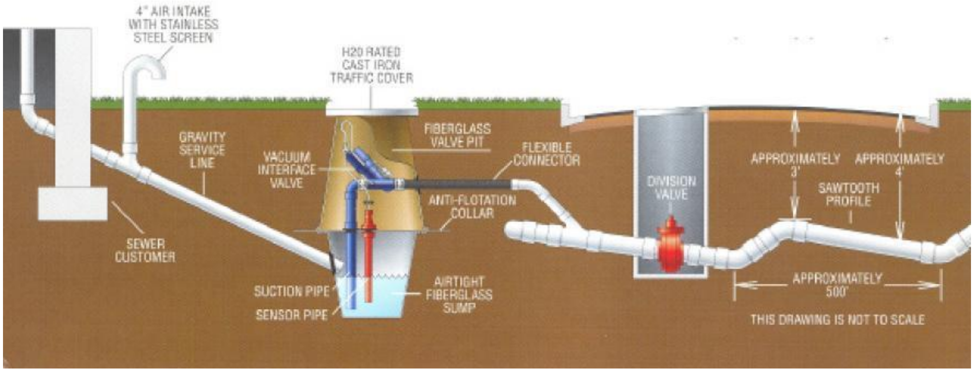

Figure 3.3 Saw-tooth Design

Diagram from Eurobodalla Shire Council Community fact sheet

The introduction of air each time the valve opens and the velocity of the wastewater within the network, acts to aerate the sewage and reduce the potential for odour generation otherwise resulting from the creation of septic conditions.

Vacuum pumps generally operate continuously to maintain the required pressure differential in the system. In times of low flow the pumps turn off and a vacuum vessel maintains the pressure differential in the system.

The vacuum pump stations have a high capital cost, which tends to result in the vacuum systems having a high unit connection cost, where the number of connections is low.

4. GENERAL COMPARISON OF AVAILABLE COLLECTION NETWORKS

4.1 SUITABILITY OF THE COMMUNITY WASTEWATER MANAGEMENT SYSTEMS

The decision to select a pressure or gravity system is dependent on a number of factors but the key issues include:

- The terrain and ground conditions
- Number of connections within the system
- The level of skills within authority's operations personnel and/or contract administrators.

No individual system is generally "better" than the others, as the functionality of each system will vary from site to site. The following guidelines can be used to select the most appropriate and economical option.

Gravity

Suited to;

- Gently sloping terrain towards one side of the site
- Areas with good excavation conditions
- Reasonably dense housing (i.e. not sparsely spaced blocks)
- Remote areas where system response times are likely to be long
- Areas with a high probability of prolonged power failure.

Pressure

Suited to:

- Areas where excavation conditions are difficult
- Areas with high ground water
- Sites that are elongated such as those that follow coastlines or rivers
- Areas with sparsely located houses
- Areas with significantly undulating terrain
- Areas that require large lifts from individual properties
- Hilly areas (vacuum lift is restricted to about 6m)
- Areas where construction impact needs to be minimised
- Where significant land acquisition would be required to install gravity drains

Vacuum

Suited to:

- Areas where excavation is difficult
- Areas of high ground water
- Proposals with over 100 connections
- Gently undulating sites
- Sites that are elongated such as those that follow coastlines or rivers

4.2 GENERIC ADVANTAGES AND DISADVANTAGES

There are a number of advantages and disadvantages of each of the waste collection alternatives. These are outlined below:

Gravity

Advantages

- There are limited maintenance issues for property owners (STEDS do require maintenance of the septic tank)
- Access to individual properties is not necessarily required by the authority (STEDS may require access depending on the pump out arrangements in place for septic tanks)
- The system is simple with very few mechanical parts or valves that may result in choke points.
- Power failure does not result in total system shutdown. Emergency response in such circumstances requires response to only a few key locations such as pump stations. This can be achieved by a trailer mounted diesel pump or a generator.
- Systems have minimal electrical requirements.
- Most civil contractors are able to install a gravity sewerage scheme.

Disadvantages

- Tracking infiltration or illegal stormwater discharges is difficult.
- Network isolation for maintenance purposes is more difficult.
- Drains tend to be deeper, making access for maintenance or replacement difficult. It also increases the construction costs, particularly when adverse ground conditions exist.
- Internal plumbing for individual properties may require deep excavation.
- Ground water ingress potential is higher than for either pressure or vacuum.
- Wastewater egress from the system is almost untraceable.

- System may not be appropriate to accept effluent and full sewer. Full sewer would accept effluent connections but STEDS drains may not accept full sewer.
- Stormwater ingress can significantly increase peak flows, which may cause system capacities to be exceeded (particularly at the pump stations or treatment plants).
- Deep excavation can cause considerable damage to nearby structures, as can the removal of rock by percussion.
- Pump stations have 9m or 12m vent stacks and may also have pump sheds that can have an adverse visual impact.
- Odour may be generated at pump stations in low flow situations as the wastewater may sit in the pump station for some time, which may result in septicity.
- System upgrades can be costly as gravity flow through a system is limited by the capacity of the pipe. Pressure and vacuum systems are a little more flexible as the system pressures can be increased to increase flow capacity.
- A larger working corridor is required for construction of the drains. Where access is required through sensitive areas or private property then gravity systems will require the largest construction corridor and hence cause the greatest damage during construction.
- Construction tolerances on the main drain are relatively small especially in schemes constructed on flat ground (due to being at flat grades 0.15% to 0.4% minimum grade)
- Septic tanks are located on the property for a STEDS which impacts on the space available for building on the allotment. Therefore larger minimum allotment sizes are required, reducing the efficiency of land use.

Pressure

Advantages

- Pipelines are shallower than for gravity and can follow the terrain.
- Pipelines are a smaller bore than for gravity.
- Drainage network is cheaper to install.
- Greater tolerance in levels and alignment can be accepted than for the other two systems.
- Being a pressure system, groundwater ingress into the system is highly unlikely (due to the pressure in the pipe being higher than the external water pressure) other than at main pump stations or on the individuals' property.
- Tracking of illegal connections can be facilitated by requiring hour run meters or flow meters and checking volumes.
- System is easily adaptable from effluent to full sewer, as long as the treatment facility has the sludge handling facilities.

- Most pump manufacturers in South Australia produce a pressure pump unit, so there is a good choice of supply.
- Avoiding services during construction is straight forward.
- Due to using a smaller bore pipe and being at a minimal depth the construction corridor is smaller than for other systems and the damage caused by installation is minimised.

Disadvantages

- Pumps are required for each individual property.
- The question of who owns and maintains the pumps needs to be addressed. If the owner maintains the pump then the likelihood of malfunction due to poor maintenance is increased. If the authority maintains the pump then the issue of access to infrastructure arises.
- If pump unit installation is not controlled (types of pumps) can impact negatively on the operation of the system.
- The system can not operate in the event of a power failure. The scheme then relies on individual on-site storage. Hence extended power failure is difficult to mitigate, since every allotment would need to be pumped out by portable pumps and disposal units.
- Leakage of effluent from the system can be difficult to trace or detect.
- The area for buildings on each allotment is restricted by mandatory setback distances.
- Design is significantly more complex than for gravity.
- Air valves are required throughout the scheme. This can result in odour within residential areas. Air valves also tend to be prone to leakage which results in small releases of effluent to the environment.
- The onus is on the land owner to detect faults and either fix or report the fault depending on what ownership model is adopted.
- Pump sumps are located on each allotment which limits the area available for buildings. Therefore larger allotments are required, reducing the efficiency of land use.
- There are more mechanical and electrical components within the system which will result in a more rigorous maintenance regime being required.
- Each individual property owner is paying the power bill for the pumps. This is a hidden community cost which artificially deflates the comparable cost of this system.

Vacuum

Advantages

- Pipeline construction can be kept at a minimum depth, saving excavation costs. Generally these mains will be deeper and larger than for pressure systems but shallower than for gravity. This helps reduce water ingress.
- Generally there will be fewer pump stations than for a gravity system.
- Eliminates the need for maintenance holes, reducing costs compared to conventional sewerage and reducing ground water ingress.
- The system is easily adaptable from effluent to full sewer as long as the treatment facility has the sludge handling facilities.
- Property owners do not need to maintain infrastructure, as is the case with conventional sewerage. With pressure schemes they have a pump and sump, STEDS they have a septic tank.
- The risk of egress of effluent to the environment is less than any other scheme due to the low pressures in the mains.
- The potential for ingress of stormwater is reduced. Suppliers have indicated that the system can tell due to the loss of vacuum if water is getting into the system. The system has the potential to track illegal stormwater discharges depending on the level of monitoring equipment installed.
- The mixture of air and wastewater in the system maintains wastewater in an aerobic state, reducing the potential for odour and providing a small level of pre-treatment before it is delivered to the WWTP. This reduction in septicity also reduces the potential for corrosion.

Disadvantages

- Stormwater ingress can occur upstream of the vacuum chambers.
- A vent is required on the individual property so there is a potential for odour in the event that effluent remains in the chamber for some time.
- Should the vacuum pumps fail then the whole system will become inactive. There is some limited storage at the vacuum chambers.
- Suppliers have indicated that it is possible to track leaks in the system. However, this is done via an elimination process and could be time consuming.
- Because individual connections are via gravity then deep connections within each property may be required. This is the same as for gravity. Pressure systems do not present this difficulty.
- Design costs are significant as design is significantly more complex than for a gravity system.
- Adherence to tolerances is very important, making construction standards and supervision very important.

- There are limited suppliers of vacuum systems.
- There are more mechanical components within the system (compared to gravity) which will result in a more rigorous maintenance regime being required.
- System requires more vigorous monitoring than a gravity system, to ensure vacuum pressures are maintained. This is likely to require a SCADA system for this monitoring to be effective.

5. SITE SPECIFIC CONDITIONS AND ISSUES AT BUCKLAND PARK

The following conditions and issues at the Buckland Park site will influence the selection of an appropriate collection network.

High Ground Water Levels

The majority of the site has a depth to water table of less than 3 metres. To minimise the length of drain constructed below the groundwater table level the maximum drain depth was set to 3m by installing pump stations. To achieve this, approximately 35 pump stations would be required to service the proposal. Even with this number of pump stations up to 75% of the gravity drains would be installed within the water table. Appendix A shows a depth to water table plan for the site highlighting all areas where the groundwater is less than 3m below the surface. This map is based on recent site mapping undertaken by Golder and Associates. It should be noted that seasonal fluctuations of up to 1m could be experienced. This would result in the majority of gravity drain being below the standing groundwater level.

Constructing a gravity system within the ground water table could potentially result in water infiltration at manholes, pump stations and any breaks or cracks in the pipe work. STED systems also have potential for ground water ingress at septic tanks.

Sewer systems generally have more manholes in the system than STEDS and the drain depths are greater due to the larger minimum grade required for sewer systems so the risk of infiltration is increased.

The drains for vacuum systems are generally installed between a depth of 1.2m and 1.5m. Appendix B indicates the area of the site that the depth to ground water is less than 1.5m. It is estimated that for a vacuum system only 10% of vacuum drains would be installed within the water table.

The maximum number of houses connected to each valve pit should be set to minimise the depth of the vacuum pits. The cost estimate in this report has assumed an average of 5 connections per pit, however, when detailed design is undertaken up to 8 houses may be able to connect and still keep the pits above the standing water table level. It is likely that some of these pits will need to be installed below the current groundwater levels.

With a pressure system almost all the drains will be above the ground water level which will minimise construction costs. Since the drains are pressurised it is unlikely that ingress would occur in any case as the pressure in the pipe network is likely to prevent infiltration.

It is likely however that a fair percentage of the domestic pumping units will be installed within the ground water table, which does introduce the potential for ground water ingress. If GRC pumping units are used then precautions will be required to prevent flotation of the pump chambers.

Salinity

Ingress of saline ground water into the waste management network causes the salinity of the waste water to increase and highly saline waste water can impact on the effectiveness of the WWTP operation. It will also impact on the potential number of reuse applications that the treated effluent may be used for.

The ground water within the Buckland Park site has salinity in the order of 3000ppm to 5000ppm (TDS).

This would mean that relatively small volumes of ingress could have a significant impact on the salinity of the waste water.

The salinity of typical treated waste water schemes in South Australia is between 800ppm and 1000ppm (TDS). Anecdotal information from the Virginia region indicates that soil salinity in the area is of concern to the local growers. As such if salinity of the treated water increases much above the typical values then the applications for reuse may be limited.

Within the Buckland Park proposal a high priority is placed on the potential to reuse the treated waste water, therefore the potential for ingress of saline groundwater into the waste water management system is likely to be a significant factor in selecting the most appropriate method of waste water management.

Salinity can be managed in a number of ways:

- Reduce the potential for it entering the system (by implementing a vacuum or pressure system)
- Shandy the treated water with mains or harvested stormwater.
- Install a desalinisation (RO) plant. This is likely to increase capital cost by $300,000 to $400,000 and running costs by $40,000 to $50,000 per annum.
- Do not reuse the water and dispose via evaporation (not a desirable option).

Acid Sulphate Soils

It has been confirmed within a report prepared by Golders Associates (November 2008) that sections of the Buckland Park site have the potential to encounter acid sulphate soils below the ground water level.

Construction within these zones is likely to occur if installation of a gravity waste management system is to be implemented.

If Acid Sulphate Soils (ASS) are encountered the soil will need to be treated prior to the installation of any infrastructure, therefore causing the construction cost to increase.

Precautions will need to be taken to prevent ingress of leachate from ASS getting into the trenches and being transported around the site. Both vacuum and pressure systems will minimise this due to the relatively shallow depth of drains. Gravity drains also drain for long distances at a constant downward grade which facilitates the transport of leachate (if encountered). Both the vacuum and pressure sewerage drains are not required to constantly grade downward, this in itself would minimise the spread of ASS leachate should it be encountered.

Resource Availability

When selecting a scheme the resource availability and skill levels within the region need to be considered.

The gravity options will have the lowest site maintenance requirements and also require the lowest level of system familiarisation.

This needs to be carefully considered when selecting the most appropriate system for the Buckland Park proposal.

Technology

The gravity options are the oldest form of collection system and their operation is generally understood.

Pressure technology uses conventional pumps and as such there is a wide variety of suppliers and a considerable availability of skilled labour to service the pumps, however with the increased number of mechanical and electrical components the potential for faults is increased.

Vacuum is not a commonly utilised technology in SA with only 3 schemes currently known to W&G being:

- Hindmarsh Island Marina
- Waterfall Gully
- A marina project within the Murray Bridge Council area.

The Alexandrina Council are about to install a significant scheme to expand the area serviced by its STEDS network at Goolwa. There is also a vacuum system currently being constructed at Port Wakefield.

This technology however is widely used in Western Australia particularly and also in some of the eastern states.

From all reports and the research undertaken by W&G these systems are proving to be reliable if designed and operated appropriately.

6. COST COMPARISON

Table 5.1.1 provides a summary of the capital and all of life costs that are associated with each of the different wastewater management systems. These costs have been calculated taking into account a discount rate of 4 percent. The cost comparison has been based on estimates completed on capital and running costs produced by W&G through experience and industry knowledge using rates based on similar recently completed projects. These estimates have then been entered into the LGA's all of life cost model to obtain the all of life cost per connection, for each of the options.

These cost comparisons have been based on servicing the projected total population of 33,000 as outlined previously.

Table 6.1.1 Summary of costs from LGA cost evaluation spreadsheet

Effluent		
Discount Rate	Capital Cost	All of Life Cost
4%	$78,100,000	$100,900,000*
Sewer		
Discount Rate	Capital Cost	All of Life Cost
4%	$41,800,000	$58,700,000*
Pressure		
Discount Rate	Capital Cost	All of Life Cost
4%	$132,700,000	$248,000,000
Vacuum		
Discount Rate	Capital Cost	All of Life Cost
4%	$37,900,000	$55,300,000

- *This does note take into account additional pumping costs to cater for groundwater ingress*
- *It also does not account for additional costs to manage salinity for any reuse applications*
- *The impact of staging the proposal has not been taken into account in the all of life cost comparison.*

The gravity sewer concept has been based on a maximum drain depth of 3m, resulting in the need for 35 pumping stations and 75% of the drains laid in the water table.

The vacuum sewer cost has been based on an average of 5 houses being serviced by each vacuum pit and the scheme requiring 3 vacuum pumping stations.

From the above summary it can be seen that a vacuum system will require the lowest all of life cost and capital cost out of all four of the options considered.

It should be noted that the two gravity options will have the greatest risk/potential for cost escalations during construction due to unfavourable ground conditions.

Given these cost estimates have been produced at the proposal's concept design stage a number of assumptions have been made.

The accuracy limits of the cost model would suggest that the all of life costs for the options outlined above with less than a 10% differential could be considered to be of comparable value and should not totally influence the decision for the selection of the most appropriate scheme. In this instance for the purpose of comparison it can be assumed that the gravity sewer and the vacuum systems are of the same order of cost and as such other factors should determine which system is adopted.

7. RECOMMENDATION

W&G recommend that design development be based on a vacuum sewerage system.

The reasons for recommending this option include:

- The lower estimated capital cost and all of life costs
- The reduced potential impacts of salinity on the reuse applications
- Lesser impact of peak wet weather flows on the WWTP and pump stations
- Lesser potential for long term ground water ingress
- Reduced potential for system overflow from the pump stations during peak wet weather events or power outages
- Reduced risk of system failure due to groundwater ingress
- Lower pumping costs associated with groundwater ingress (which is not captured in Table 5.1.1)
- Reduced operational requirements in a major power failure scenario
- It is estimated that approximately 75% of drains in a gravity system would be installed below the current ground water levels, even with the installation of 35 pumping stations
- Aeration of sewage through the collection network will have a positive impact on the WWTP operation.

We recognise vacuum sewer systems are a new technology in South Australia, and they are likely to require additional resources for maintenance, than a gravity scheme. We also recognise that specialist skills are required to operate the system.

We believe that these issues can be mitigated by:

- Ensuring the construction contract allows for significant training and support after the scheme is installed.
- The economies of scale offered by a proposal of this scale justify creation of a maintenance team with specialist training.
- Ensuring spare parts are provided as part of the supply contract, and keeping the valves on hand so they can simply be swapped in the field, and the valves later repaired in the workshop.

Salinity is critical for future reuse applications within the proposal and therefore all practical measures should be taken to prevent, or at least minimise, the potential for groundwater ingress.

8. REFERENCES

Golder and Associates Geotechnical Investigations Report (2008)

Interview with Michael Frost, Mid Murray Council, 21/2/06

Interview with Peter Adams, SA Water 27/2/06

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South East Water – Pressure v Gravity Comparison, no source reference

Yara Valley Water – Pressure v Gravity Comparison – Community fact sheet

Eurobodalla Shire Council – Conventional Gravity Sewerage System Comparison – Community Fact Sheet

Eurobodalla Shire Council – Vacuum Sewerage System – Community Fact Sheet

Eurobodalla Shire Council – Low Pressure Sewerage System – Community Fact Sheet
www.eurocoast.nsw.gov.au

Wagga Wagga Protocol Draft Pressure Sewerage Guidelines – March 202

EPA# 625477011 Alternatives for Small Wastewater Treatment Systems: Volume 1 - On-Site Disposal/Septage Treatment and Disposal

EPA# 625477011 Alternatives for Small Wastewater Treatment Systems: Volume 2 - Pressure Sewers/Vacuum Sewers

EPA# 625477011 Alternatives for Small Wastewater Treatment Systems: Volume 3 - Cost/Effectiveness Analysis

EPA# 600275072 Economical Residential Pressure Sewer System with No Effluent

EPA# 625191024 Manual: Alternative Wastewater Collection Systems

EPA# 600979010 National Conference on Less Costly Wastewater Treatment Systems for Small Communities

EPA# 600278166 Pressure and Vacuum Sewer Demonstration Project: Bend, Oregon

EPA# 832F02006 Wastewater Technology Fact Sheet: Sewers, Pressure

FloVac – product Catalogue

Auqutec Fluid Systems – Product Catalogue

Australian Standards and Codes

SA Water Sewer construction manual and technical standards

SA Water Technical Standard TS130 – Pressure Sewer Systems, 8/12/05

SA Water Pressure Sewerage Design Manual – available on SA Water Website

WSAA – Pressure Sewerage Code of Australia – Interim Edition 2005

WSAA – Sewerage Code of Australia 2002 version 2.3

WSA – Vacuum Sewerage Code of Australia 2004 Version 1.1

Local Government Association – STEDS Design Guildelines, 1997

Draft Standard – Connection to a Communal Waste Control System, Department of Health South Australia

APPENDIX D
SA WATER CONSIDERATION OF WATER SUPPLY OPTIONS

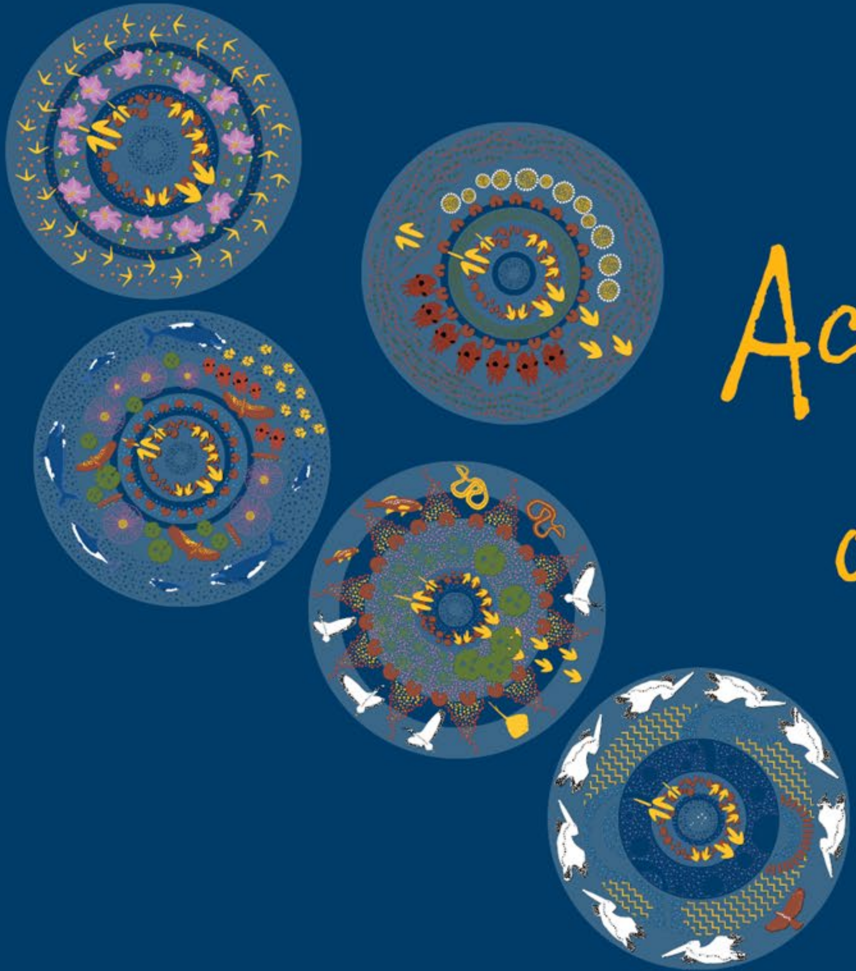
SA Water

Riverlea Masterplan presentation

27th October 2022

SA Water House

Government of South Australia



Acknowledgement of Country

Agenda

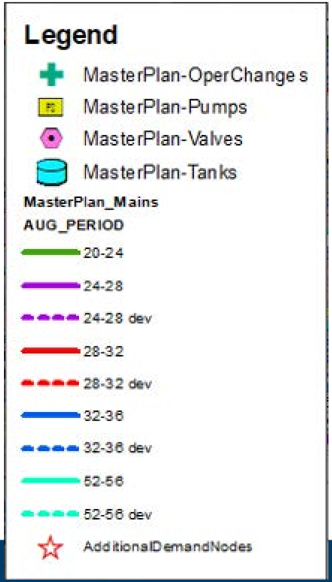
- Master planning – process and context
- Water Servicing
- Wastewater Servicing
- Open Space Irrigation Servicing
- Next steps

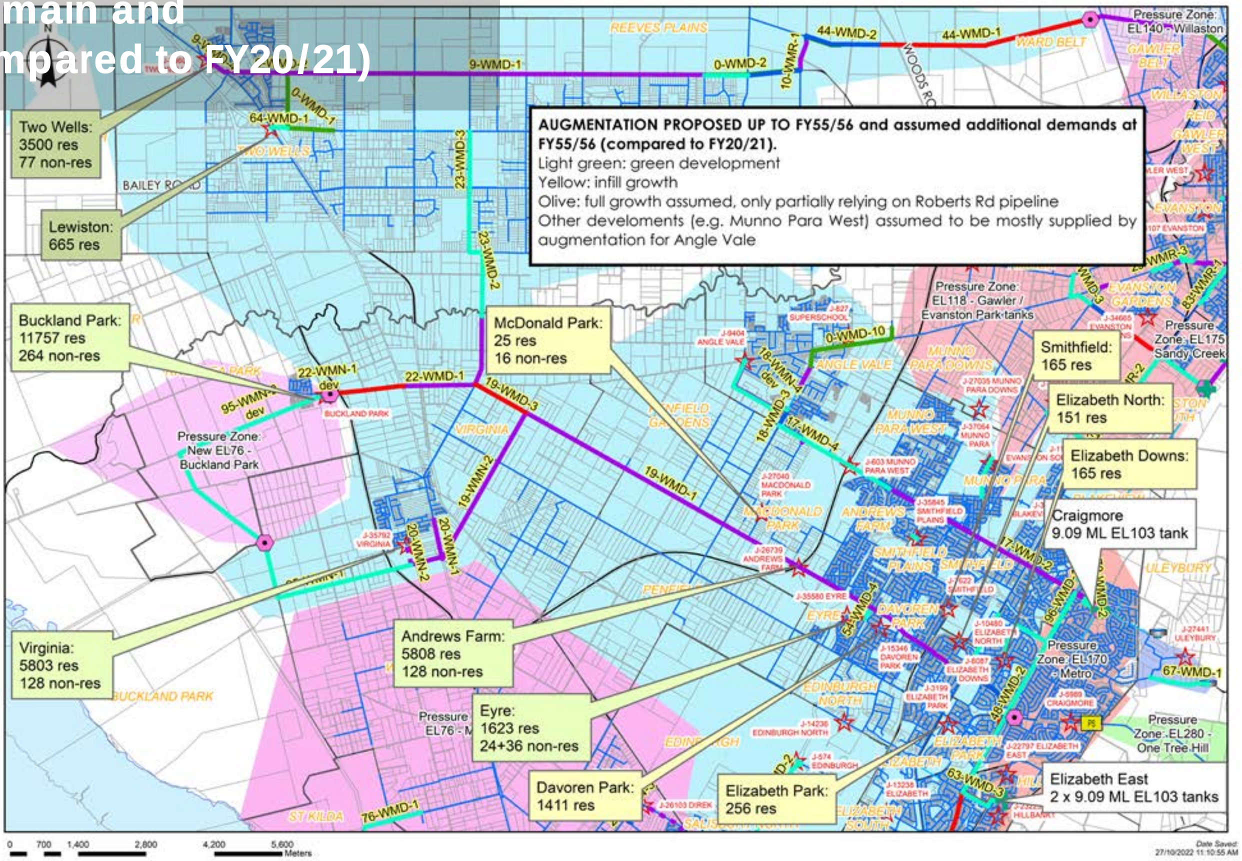
Master planning Process

- Whole of system review with the purpose of meeting Riverlea requirements
- Completed to show the ultimate servicing solution for the Riverlea (based on information to hand)
- Reliant on assumptions
  - Growth numbers
  - Timing of the growth
- High level and may change over time (20+yrs life of the development)
  - Responding to:
    - Actual growth levels
    - Actual development delivery (more/less, commercial, golf course?)
    - Changes in technology.... The list goes on

Water servicing

master plan 2022 supply main and additional demands (compared to FY20/21)

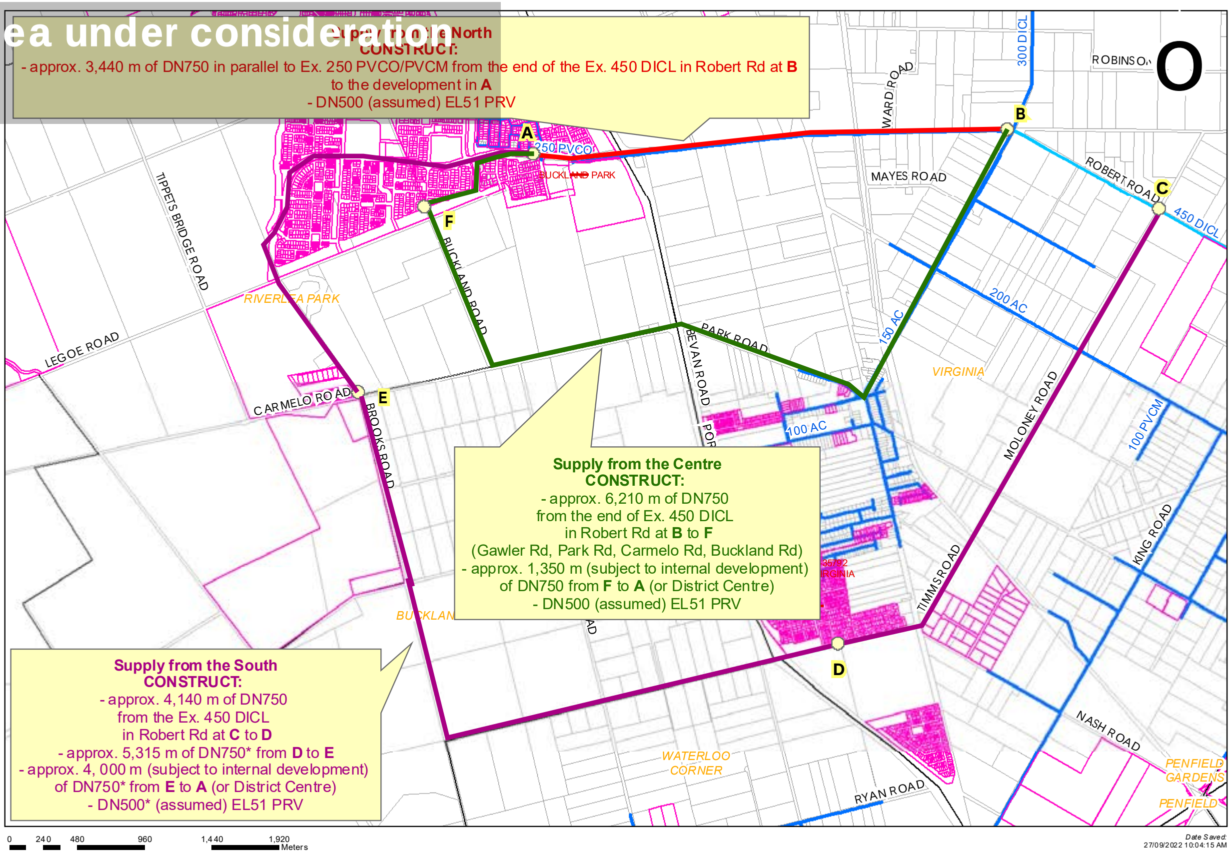




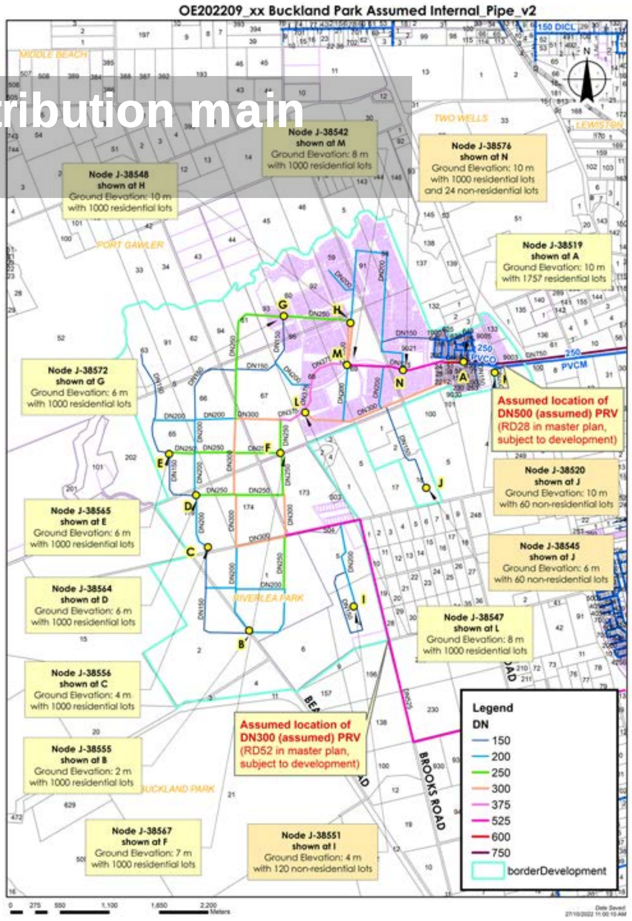
Summary of the augmentation 'directly' used by Riverlea development (downstream of storage tanks)

AUG ODE	AUG DN	AUG PERIOD	AUG REASON	LENGTH (m)
19-WMD-1	1000	24-28	DN1000 duplication along Robert Rd (from FP 7617680 to Moloney Rd) to improve supply to Virginia and Buckland Park	6459
22-WMD-1	750	24-28	DN750 main duplication in Angle Vale Rd (from Old Pt Wakefield Rd to crossing with Baker Rd, Virginia) to improve supply to Buckland Park	1477
19-WMD-4	1200	24-28	DN1200 duplication main along Petherton Rd (from FP7617680 to Main North Rd) to supply Virginia and Buckland Park	4119
38-WMD-1	750	28-32	Duplicate 250 PVCM with DN750 along Angle Vale Rd from Old Pt Wakefield Rd in Virginia to supply Buckland Park	1651
19-WMD-3	1000	28-32	DN1000 duplication in Robert Rd from 19-WMN-1 in Moloney Rd (Virginia) to Gawler Rd to supply Buckland Park	1198
95-WMN-1	525	52-56	New DN525 main in McEvoy Rd (Buckland Park) from end of 21-WMN-1 supplying Virginia, along Brooks Rd until new (southern) EL76 PRV for the development	3979
95-WMN-2 dev	525	52-56	New DN525 pipe modelled to simplify the pipes internal to the Buckland Park development (i.e. between the northern and southern new EL76 PRVs)	5824

Supply options into Riverlea under consideration

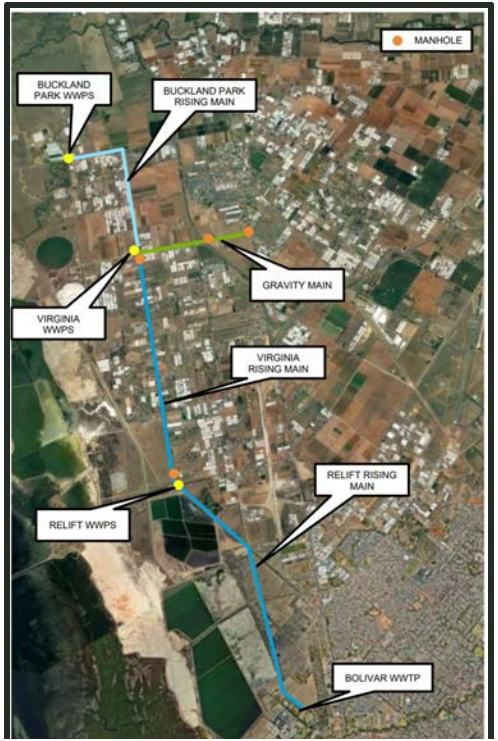


Assumed layout and distribution main



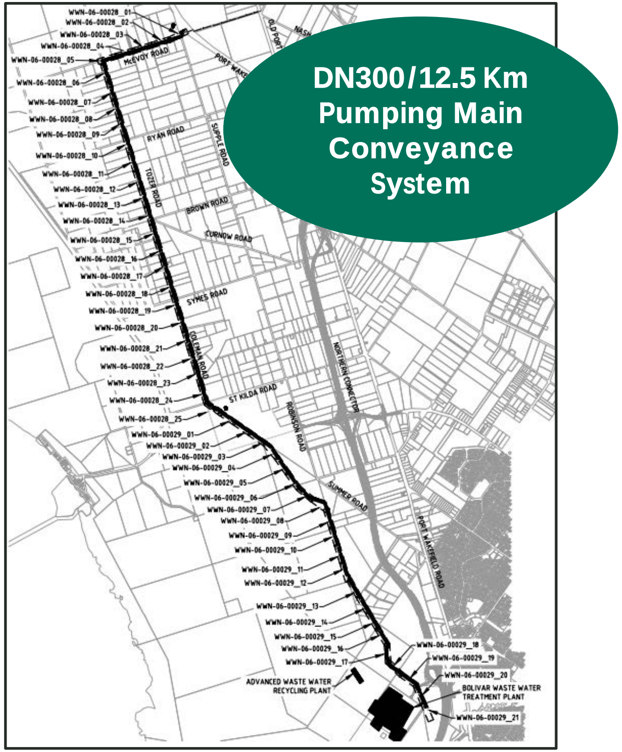
Sewer servicing

SA Water Network Growth Program – BP-V Augmentation Project

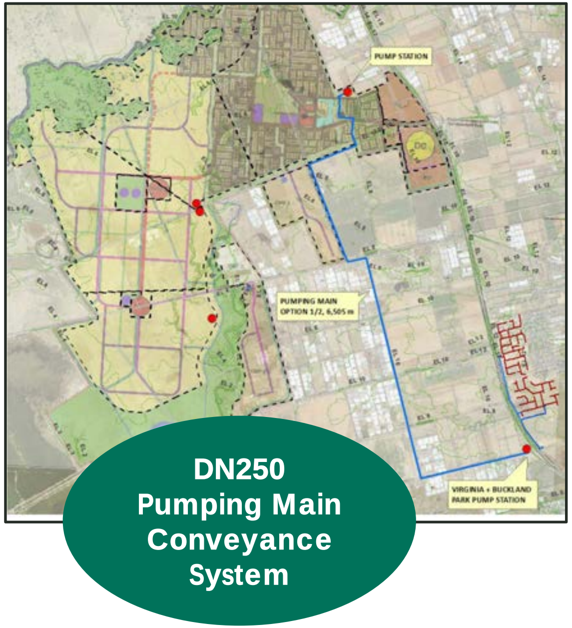


CONCEPT ONLY – FINAL ALIGNMENTS HAVE BEEN SUBJECT TO DETAILED DESIGN PHASE

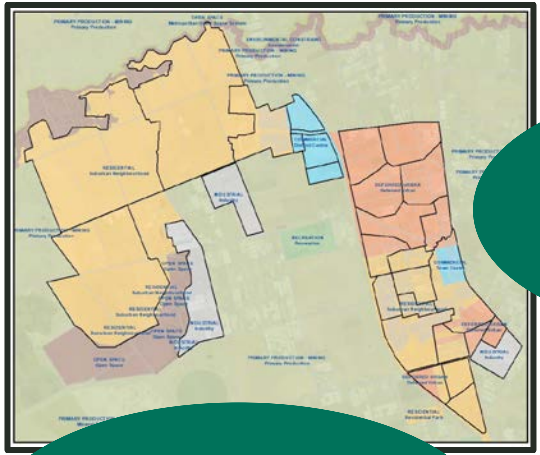
12 km transfer pumping system from Virginia to Bolivar WWTP (in delivery)

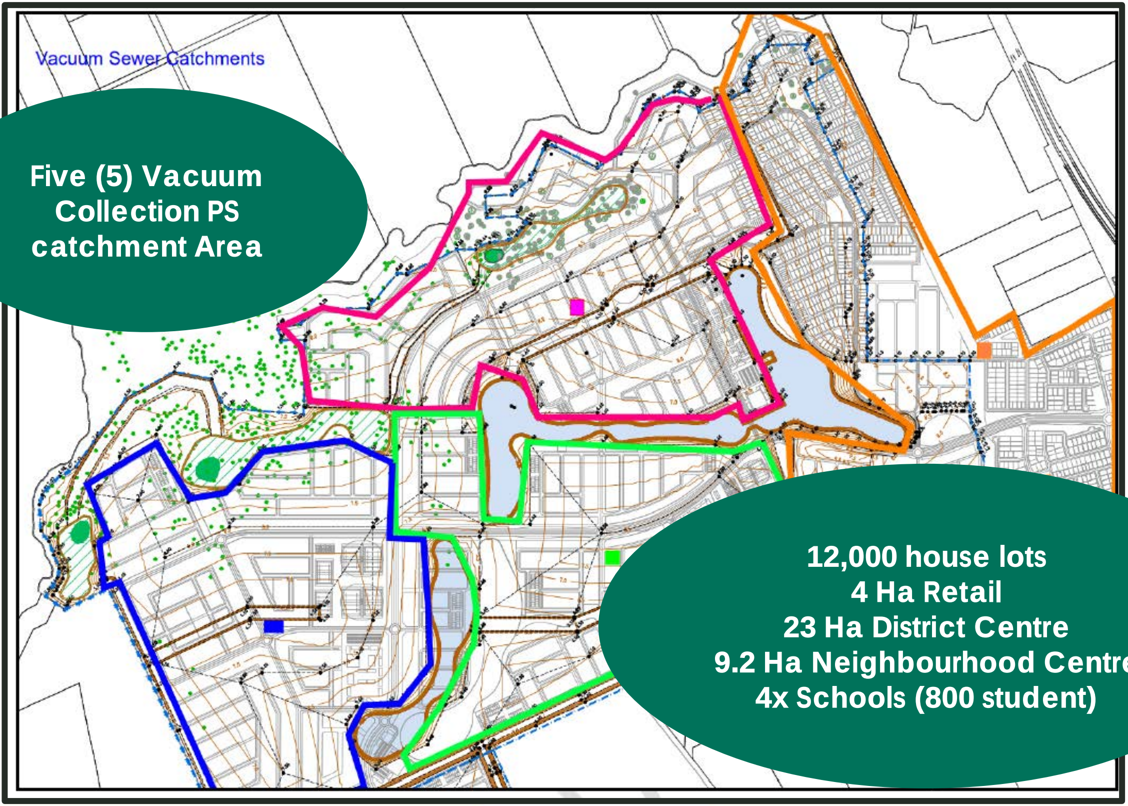


6 km transfer pumping system from Buckland Park to Virginia (in design)



Vacuum catchment areas



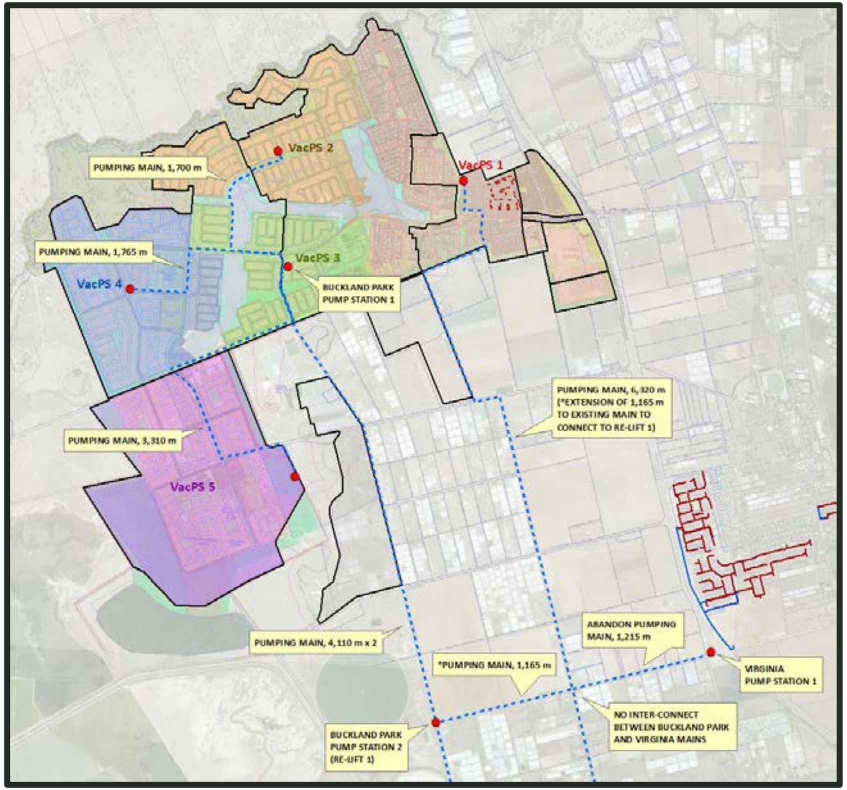


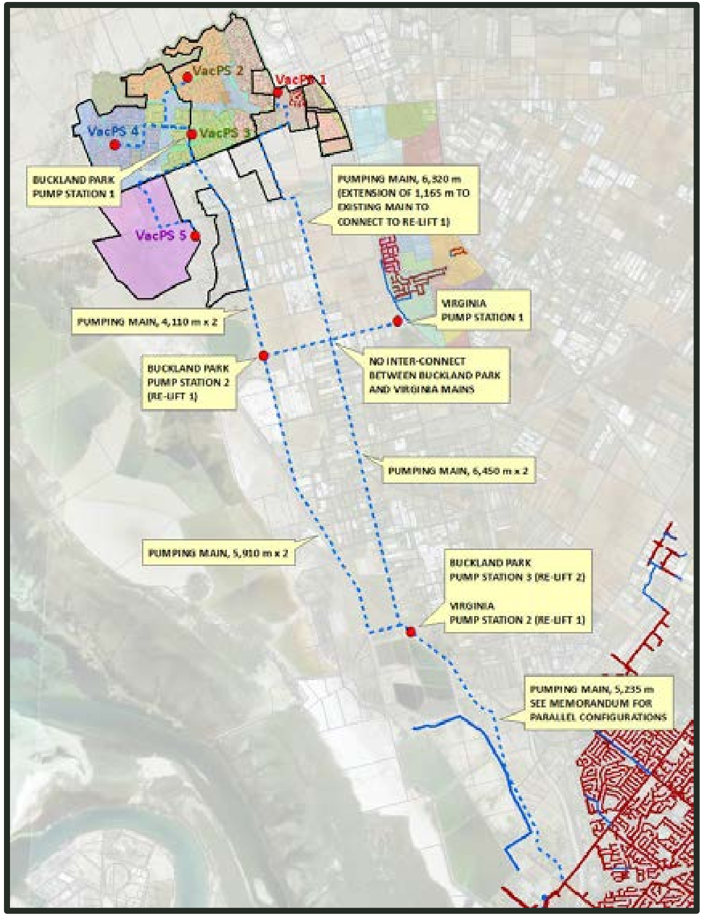
Five (5) Vacuum
Collection PS
catchment Area

EXCLUDED:
2x Employment Lands
(Total 115 Ha)

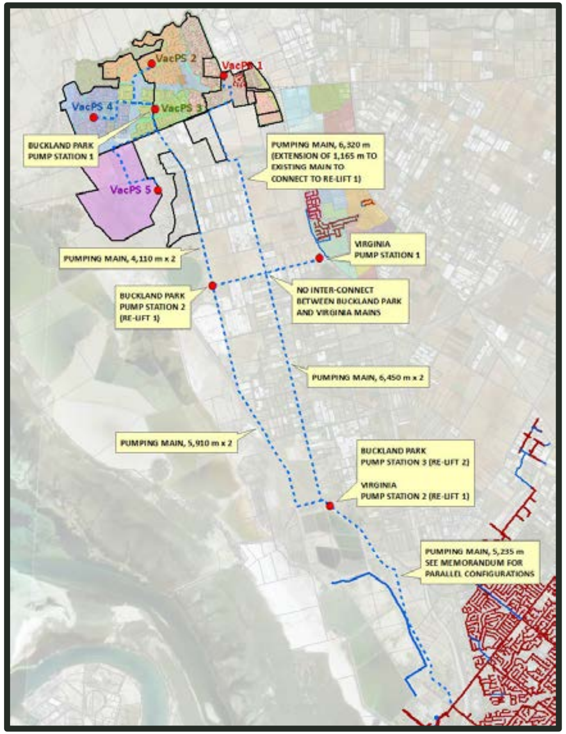
12,000 house lots
4 Ha Retail
23 Ha District Centre
9.2 Ha Neighbourhood Centre
4x Schools (800 student)

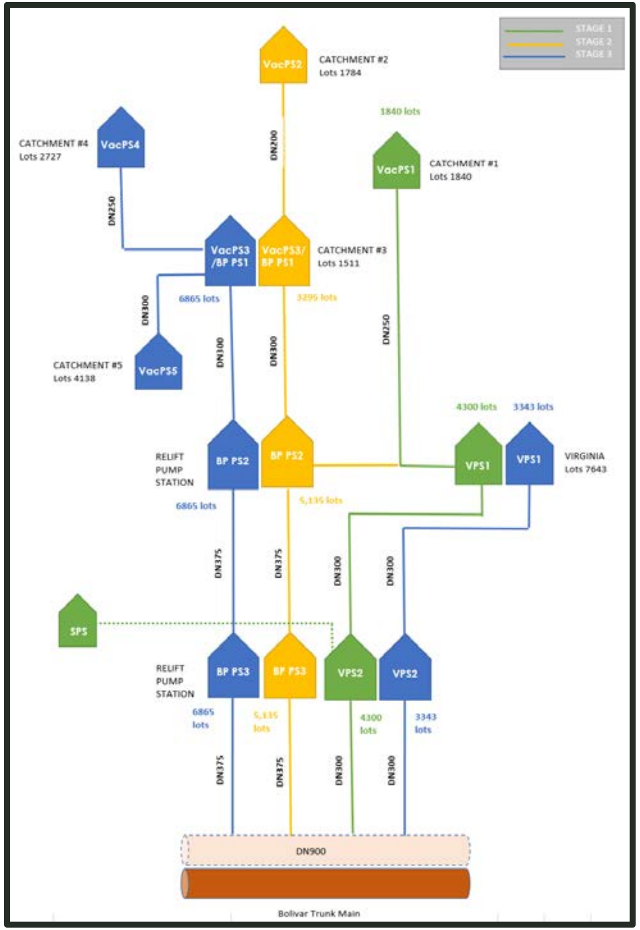
2022 Scheme Layout





Infrastructure stages





STAGES	BUCKLAND PARK	VIRGINIA
1	1,840	1,700
2	5,135 (+3,295)	4,300 (+2,600)
3	12,000 (+6,865)	7,643 (+3,343)

Infrastructure requirements

STAGES	BUCKLAND PARK	VIRGINIA
1	1,840	1,700
2	5,135 (+3,295)	4,300 (+2,600)
3	12,000 (+6,865)	7,643 (+3,343)

CONVEYANCE INFRASTRUCTURE - STAGE 1

FROM	TO	DISTANCE (m)	PUMP DETAILS		PUMPING MAIN DIAMETER	COMMENTS
			FLOW (L/s)	HEAD (mH2O)		
VacPS1	VPS1	6505	55*	36*	DN250	*Under design
VPS1	VPS2	6450	92	38	DN300	In construction
VPS2	Ex. Bolivar TM	5235	94	29	DN300	In construction, includes allowance from Defence at St Kilda

CONVEYANCE INFRASTRUCTURE - STAGE 2

FROM	TO	DISTANCE (m)	PUMP DETAILS		PUMPING MAIN DIAMETER	COMMENTS
			FLOW (L/s)	HEAD (mH2O)		
VacPS1	~~VPS1~~ BP PS1	extend by 1165m (from Cnr. McEvoy Rd/Tozer Rd)	55	37	DN250	Virginia PS is now dedicated for Virginia (Buckland Park de-coupled)
VacPS2	BP PS1	1700	45	25	DN200	New
BP PS1	BP PS2	4110	99	33	DN300	New, includes VacPS2 & VacPS3 catchment
BP PS2	BP PS3	5910	154	36	DN375	New
BP PS3	Ex. Bolivar TM	5235	154	35	DN375	New

CONVEYANCE INFRASTRUCTURE - STAGE 3

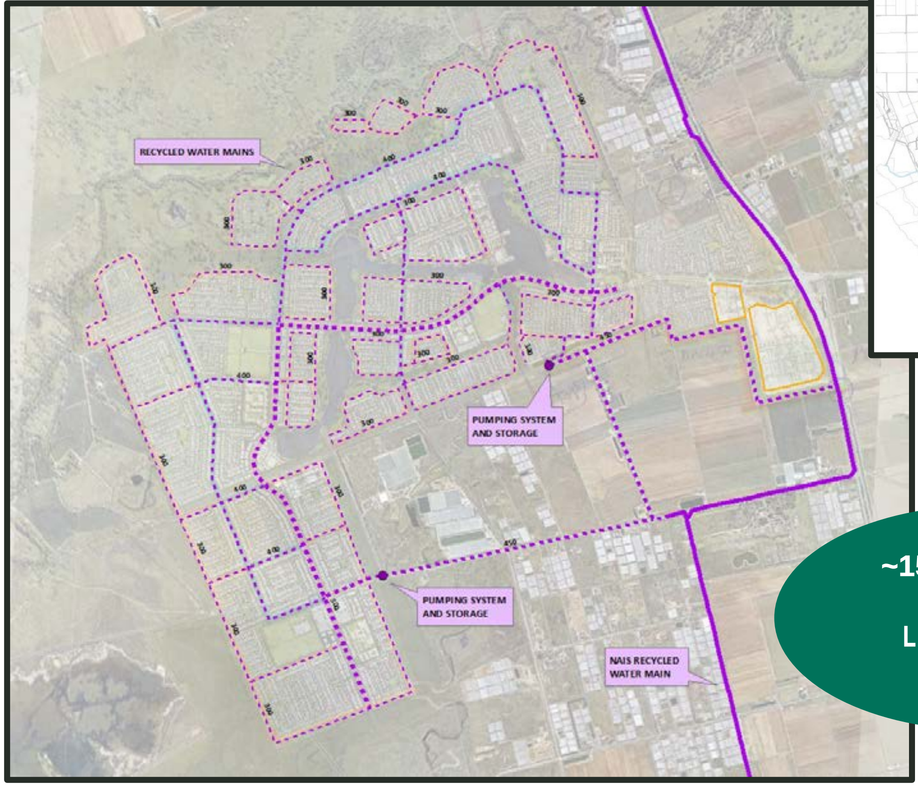
FROM	TO	DISTANCE (m)	PUMP DETAILS		PUMPING MAIN DIAMETER	COMMENTS
			FLOW (L/s)	HEAD (mH2O)		
VPS1	VPS2	6450	92	38	DN300	New, duplicated pumping system
VPS2	Ex. Bolivar TM	5235	94	29	DN300	New, duplicated pumping system
VacPS4	BP PS1	1765	68	22	DN250	New
VacPS5	BP PS1	3310	103	32	DN300	New
BP PS1	BP PS2	existing	127 (upgrade)	50 (upgrade)	DN300	Upgraded pumping capacity
BP PS1	BP PS2	4110	127	50	DN300	New, duplicated pumping system
BP PS2	BP PS3	5910	154	36	DN375	New, duplicated pumping system
BP PS3	Ex. Bolivar TM	5235	154	35	DN375	New, duplicated pumping system

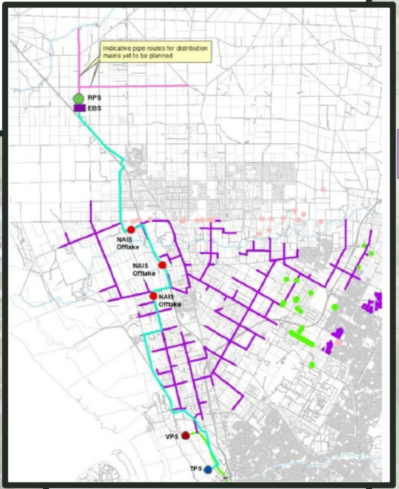
GRAVITY MAINS (FULL BUILD OUT)

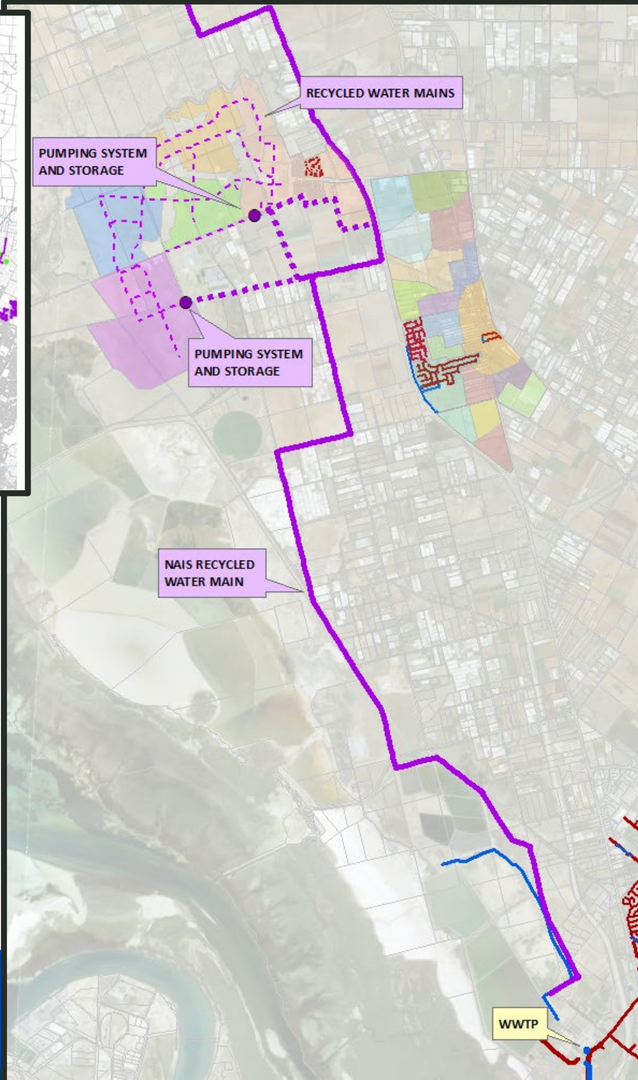
FROM	TO	DISTANCE (m)	DIAMETER	GRADE	COMMENTS
New BP-V Connection	Bolivar WWTP	425	DN900	0.13%	High level, PWWF

Open space landscape irrigation servicing

2022 Scheme Layout







~150 Ha Open Space Landscape Irrigation

Next steps

Information Request from Walker Corp

Current and forecasted

- Timing
- Staging
- Dwelling & commercial tenancy construction commencement
- Dwelling & commercial tenancy completion dates
- Commercial/School forecasting (nature of development, timing meter size and connection size)
- Reserves and meter sizing
- Finished survey information

making life flow

Government of South Australia

SA Water

APPENDIX E
FLOOD MODELLING REPORT

Walker Corporation

Riverlea Park

2009 TECHNICAL PAPER UPDATE – FLOOD ASSESSMENT

WGA080163

WGA080163-RP-CV-0013_A

Rev A

6 December 2022

Revision History

REV	DATE	ISSUE	ORIGINATOR	CHECKER	APPROVER
A	17/11/022	Update to 2009 technical paper	FL	MM	DB

CONTENTS

Figures

Tables

Attachments

1 BACKGROUND

WGA has been engaged Walker Corporation to assess the ability of the proposed integrated saltwater lake and stormwater channels system for the Riverlea Park - Riverlea development to manage a 1% AEP flood event. This includes undertaking a flood modelling assessment for the proposed development and checking the freeboard for saltwater lakes, detention basin, and channels/drains.

The Riverlea development is located approximately 32 km north of the Adelaide CBD, in the City of Playford, bounded by Gawler River to the north. Figure 1 shows the project site locality. The surrounding catchment area is relatively flat and has a gentle slope from east to west.

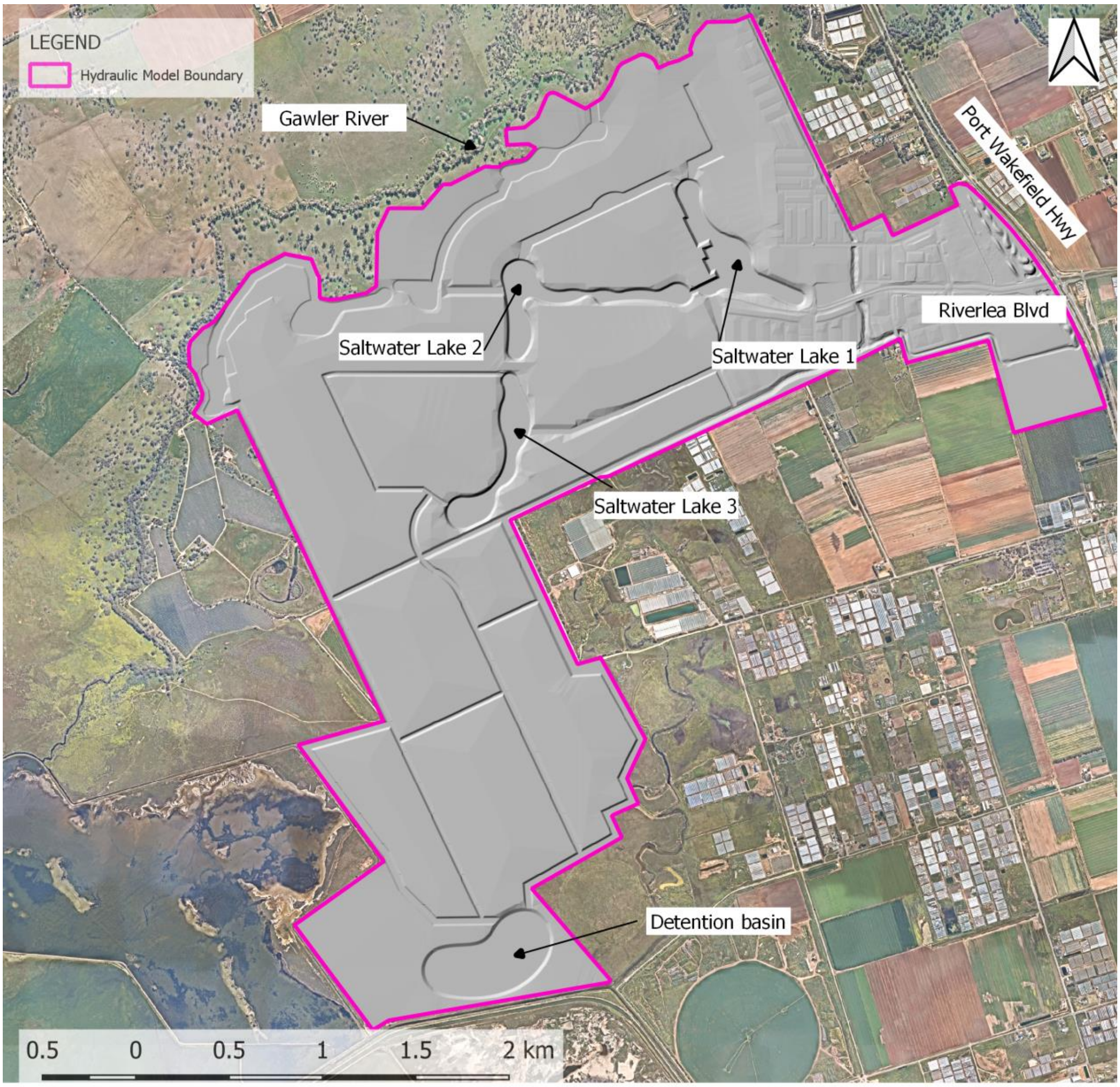


Figure 1: Project Site Locality

The development is proposed to contain three saltwater lakes, several channel networks and a detention basin located at the southern side of the development site. Saltwater lakes 2 and 3 are connected by an overflow weir and excess water from lake 3 is proposed to be released through a 750mm diameter gravity pipe to the open channel system.   Salt water will be pumped into the lake system from the sea. Flood water from the development site is collected through the channel system and transferred to the detention basin, before discharging flows when required out through a button pipe outlet to Thompson's Outfall Channel

In 2009 a technical paper was prepared based on an older design for the development site that included a flood modelling assessment. This current flood modelling assessment for the site is based on the most recent development design which includes three saltwater lakes, channels, and a detention basin.

The saltwater lakes contain saline water and any spills from the channel system may cause potential risk to the environment. The operational functional design capacity of the saltwater lakes, detention basin and channel system to not spill during a 1% AEP event has been assessed in this report.

The following information was used in this flood modelling assessment:

- Riverlea Park proposal, Stormwater management water, wastewater and recycled water, technical paper 2009.
- Riverlea Saltwater Lakes, Second Phase of Preliminary Investigations January 2022.
- Riverlea Development Flood Assessment Addendum 2022.
- Design development area drawings.

For this updated site flood risk assessment Rain on Grid (RoG) 1D/2D TUFLOW modelling has been undertaken to simulate the inflow from the catchments and the flood levels in the channels, salt lakes and the detention basin. The modelling has been performed for a 1% AEP event. This report explains the modelling process and summarises the findings.

2 SCOPE

The key activities undertaken for this report include:

- Reviewing the design information for the development
- Developing a flood model based on:
  - Undertaking 1D/2D TUFLOW modelling
  - Using the Rain on Grid (RoG) approach
  - Modelling 1% AEP as the major storm event
  - Simulating a range of rainfall durations and ten temporal patterns per duration
  - Using AR&R 2019 guidelines for the modelling
  - Modelling the proposed development site design surface
  - Using HPC solver for modelling
- Running the TUFLOW model for the proposed development site
- Processing the results and extracting median results for temporal patterns and peak results for the durations
- Checking the freeboard for the saltwater lakes, channels/drains, and detention basin
- Preparing the flood maps and summarising the findings.

The next sections of the report explain the details and assumptions for the flood modelling and the results.

3 FLOOD MODELLING

3.1 Methodology

A 1D/2D TUFLOW model has been developed in accordance with AR&R 2019 guidelines. The latest design surface for the development site has been used. The modelling has been undertaken for 1% AEP event.

The model boundary is shown in Figure 1 and covers about 10.2 km^2. The flooding from Gawler River was assessed in "Riverlea Development Flood Assessment Addendum - 2002" report prepared by Water Technology. In this assessment only the flooding from the development site area was modelled. The flooding from Gawler River has not been assessed, therefore its catchment has not been included in the model.

A range of storm durations was selected and for each duration 10 temporal patterns were modelled. The median of all 10 temporal patterns for each duration was processed and the maximum of the medians were then extracted to form the critical results. This approach ensures only the critical results are presented for each modelling cell. The results have been checked for all the modelled durations to ensure the peak results have been captured.

Hydrological data including rainfall and losses has been entered directly into the model using the Rain on Grid (RoG) approach, which directly applies rainfall to the modelling area. By using this approach, both hydrologic and hydraulic modelling can be simulated together in TUFLOW rather than separately.

3.2 Digital Elevation Model (DEM)

The latest development site design DEM has been used. Minor modifications have been undertaken to correct identified DEM generated anomalies.

3.3 Durations and Temporal Patterns

A wide range of short and long rainfall durations were modelled to ensure peak flood elevations for the development site were captured. Durations modelled included 15 min, 30 min, 60 min, 120 min, 180 min, 360 min, 540 min, 720 min, 1,080 min, 1,440 min, 1,800 min, 2,160 min and 2,880 min. For each duration 10 temporal patterns were modelled.

3.4 Rainfall Data

Rainfall depths and temporal patterns have been sourced from the AR&R 2019 data hub and the Bureau of Meteorology (BOM). The design rainfall inputs adopted, used the coordinates below, which is the centroid of the modelling area:

- Latitude : -34.663200
- Longitude : 138.507350

3.5 Surface Materials and Manning's n Value

The development site has several different surfaces and terrains to account for with the flood modelling. The surfaces have different loss and roughness coefficients (manning's n value). To model this, the modelling area was classified based on the different land use that will be present with completion of the development site. The surface material classification assigned for the site are shown in Figure 2. The following surface material categories were used in the model:

- Saltwater lakes (standing water)
- Open channel, straight banks, and well-maintained channel
- Roads

- Park reserves, containing light shrub and tree planting and grass lands
- Lots, block of lands containing high density of impervious area such as roofs, concretes and it was assumed 70% of the area was impervious
- Water surface/ wetland, which covers tall shrubs and average depth of flow

The Manning's n value used for the modelled land uses are presented in Table 1.

Table 1: Manning's n Value

LAND USE	MANNING'S N VALUE
Saltwater lakes	0.03
Park reserve	0.04
Open space/channel	0.03
Water surface/wetland	0.05
Lots	0.30
Roads	0.02

3.6 Water Loss Estimation

The initial and continuing loss method has been used for the modelling. The losses have been sourced from the AR&R 2019 data hub. The initial and continuing loss adopted was 29 mm and 4 mm/hr respectively. The initial loss has been adjusted to model the pre-burst rainfall. The pre-burst rainfall depths have been deducted from the initial losses.

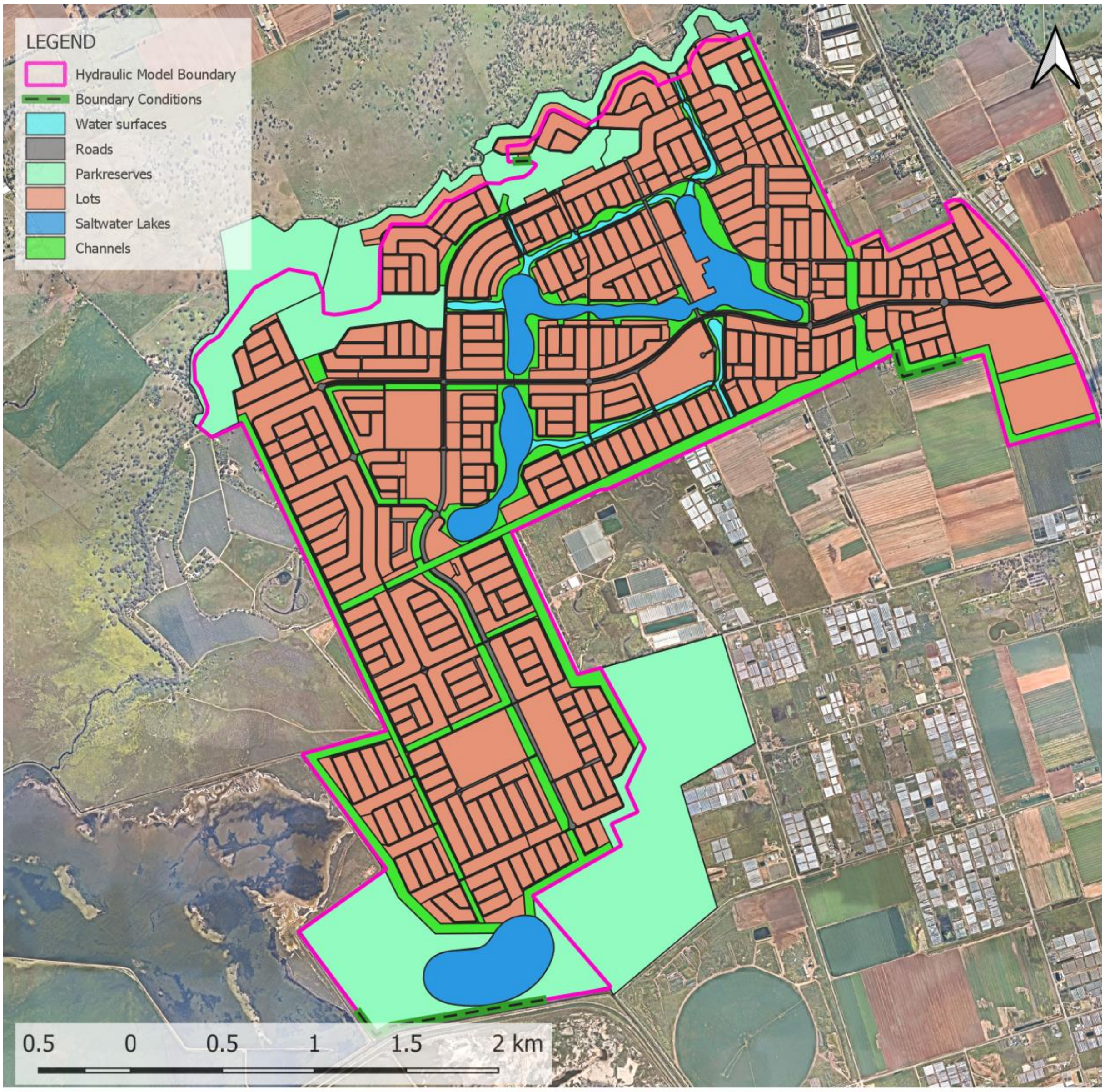


Figure 2: Materials/Land Use Classifications for Losses and Manning's n Value Assignment

3.7 Boundary Condition

The flow boundary conditions have been used for the locations where water flows out from the modelling area. HQ (head-discharge curve) type boundaries were modelled with 0.004, 0.003, 0.01 and 0.38 slopes for four locations. The hydraulic boundary and flow boundary conditions are shown in Figure 3.

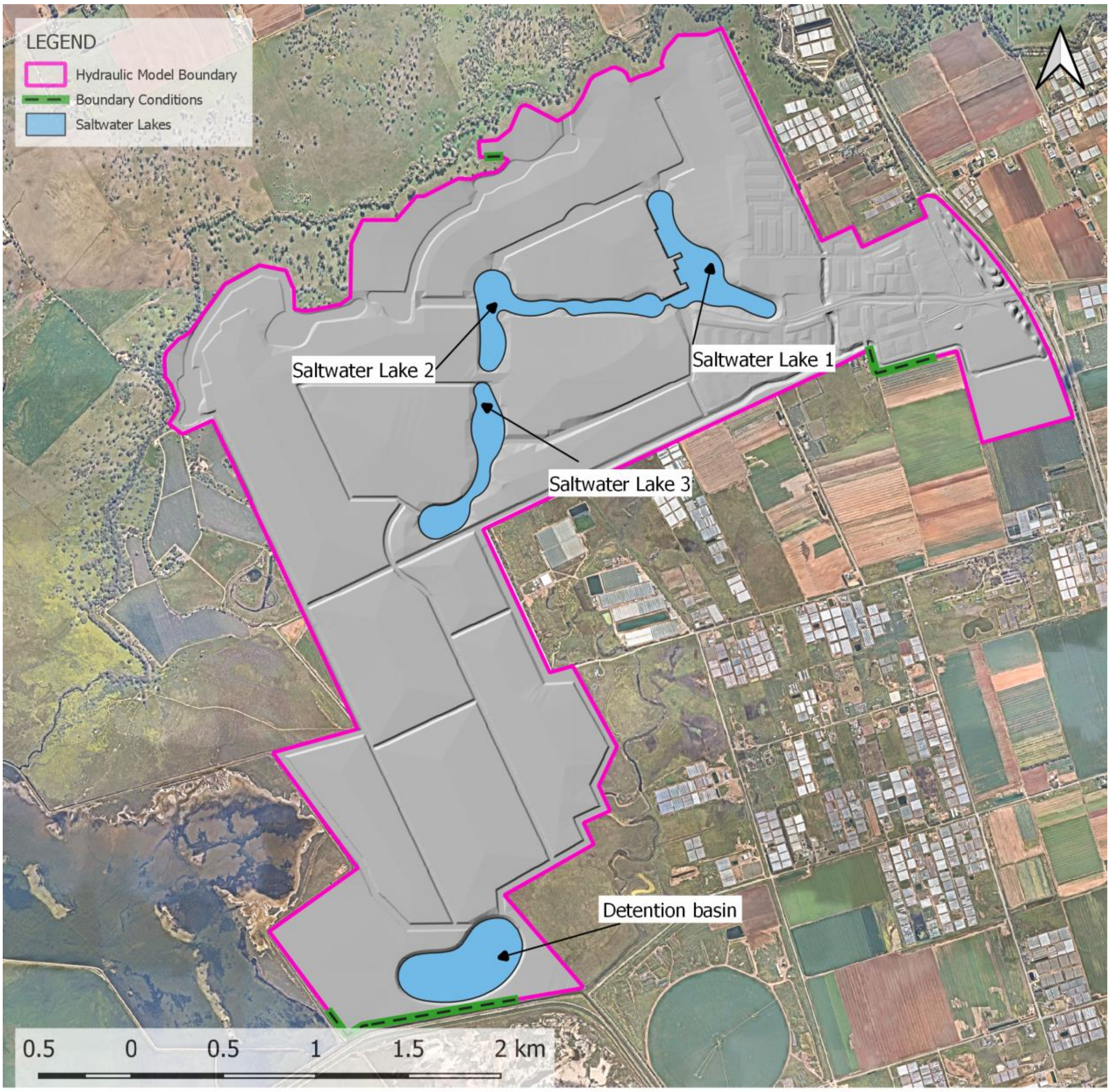


Figure 3: Model Boundary Conditions and Saltwater Lakes

3.8 Initial Water Level

For modelling the initial water level of the saltwater lakes and detention basin have been set to that which they will be normally maintained. The initial conditions applied were 4.0m AHD for Lakes 1 and 2, 3.0m AHD for Lake 3, and -0.5m AHD (no water/empty) for detention basin. The locations of the saltwater lakes and detention basin where the initial condition has been applied are shown in Figure 3.

3.9 1D System

The 1D system modelled for the site included the following water control and transfer elements:

- A wide weir connecting saltwater lakes 1 and 2 to saltwater lake 3
- Saltwater lake 3 outlet
- Detention basin outlet pipe
- Outlet pipe from the western side from the wetlands
- Wetlands connection
- Culverts in the Kapinka Parade, Riverlea BLVD and District Centre-Legoe Road

The location of the 1D system in the model is shown in **Error! Reference source not found.**

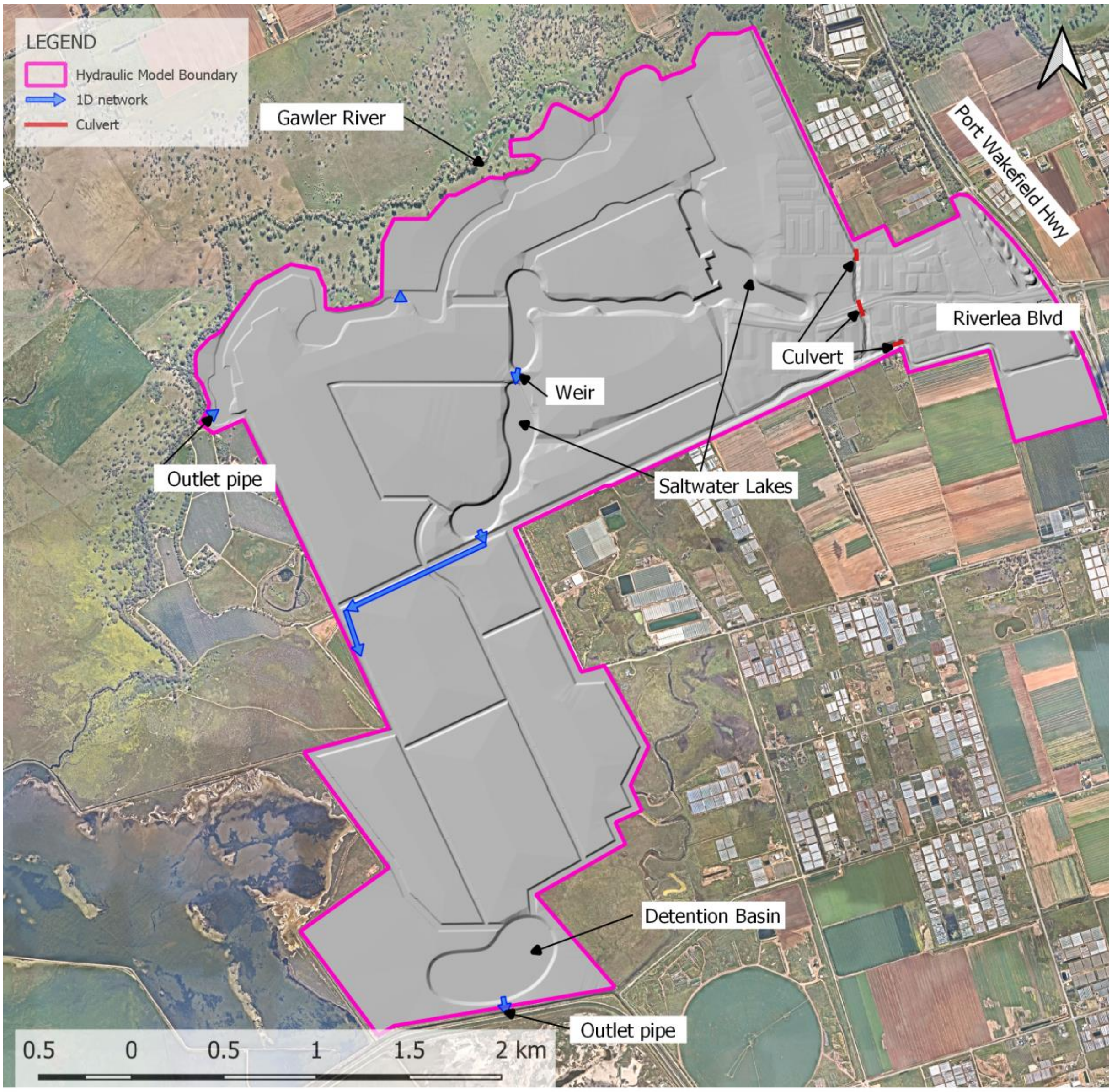


Figure 4: 1D System

3.10 Modelling Results

The modelling results were processed to extract median results from the temporal patterns and the maximum from durations. The flood depth and level maps for these median results were prepared, and are presented in Attachment A. Several cross-sections were prepared to show the peak water levels at key locations including the lakes, basin and channels. The locations of the cross sections are shown in the Figure 5. The cross sections are provided in Attachment B.

The critical duration was identified for the key locations. Table 1 shows the critical duration for the lakes, channel, and basins.

Table 2: Critical Durations

CATCHMENTS	CRITICAL DURATION
Saltwater lake 1	6 hr
Saltwater lake 2	6 hr
Saltwater lake 3	9 hr
Detention basin	24 hr
Channels	Varies from 1 to 12 hr

The details of the results are discussed in the next sections:

3.10.1 Saltwater lake

The saltwater lakes 1 and 2 had a 4.0m AHD water elevation set as their normal condition (beginning of the modelling time) and reached a maximum 4.185 m AHD with the flooding scenarios modelled. The freeboard for lakes 1 and 2 were 1.4 m and 2.4 m respectively.

The saltwater lake 3 had a 3.0m AHD water elevation set as the normal condition, and it reached maximum 3.87m AHD with the flooding scenarios modelled. It provided 1.7 m freeboard to its crest elevation.

3.10.2 Basin

Several outlet pipe diameter sizes were checked for the basin to achieve the desired maximum water elevation (about 2.0m AHD) – resulting in the 750mm diameter pipe being adopted. The maximum water elevation in the detention basin with this pipe size with the flooding scenarios modelled was 2.24m AHD with a water depth of 2.74m AHD. This provides 260 mm freeboard to its crest elevation. The water elevation peaks after 24 hours and is then expected be fully emptied after several hours.

The peak outflow from the basin to Thompson's Outfall Channel was 1.4m^3/s which is a result of the significant amount of stormwater attenuation provided by the lake and wetland system.

3.10.3 Channel

The flood levels for the channels located at the northern side of the development site (upstream side) reached their peak with the shortest events. The channels at the southern side (downstream) reached their peak levels with the longer modelled storm duration events. The events vary from 1 to 12 hours. Freeboard for each of the channel cross sections are shown in Table 3. The freeboard varies from 0.49m to 2.30m. No spill has been modelled to occur from any channel.

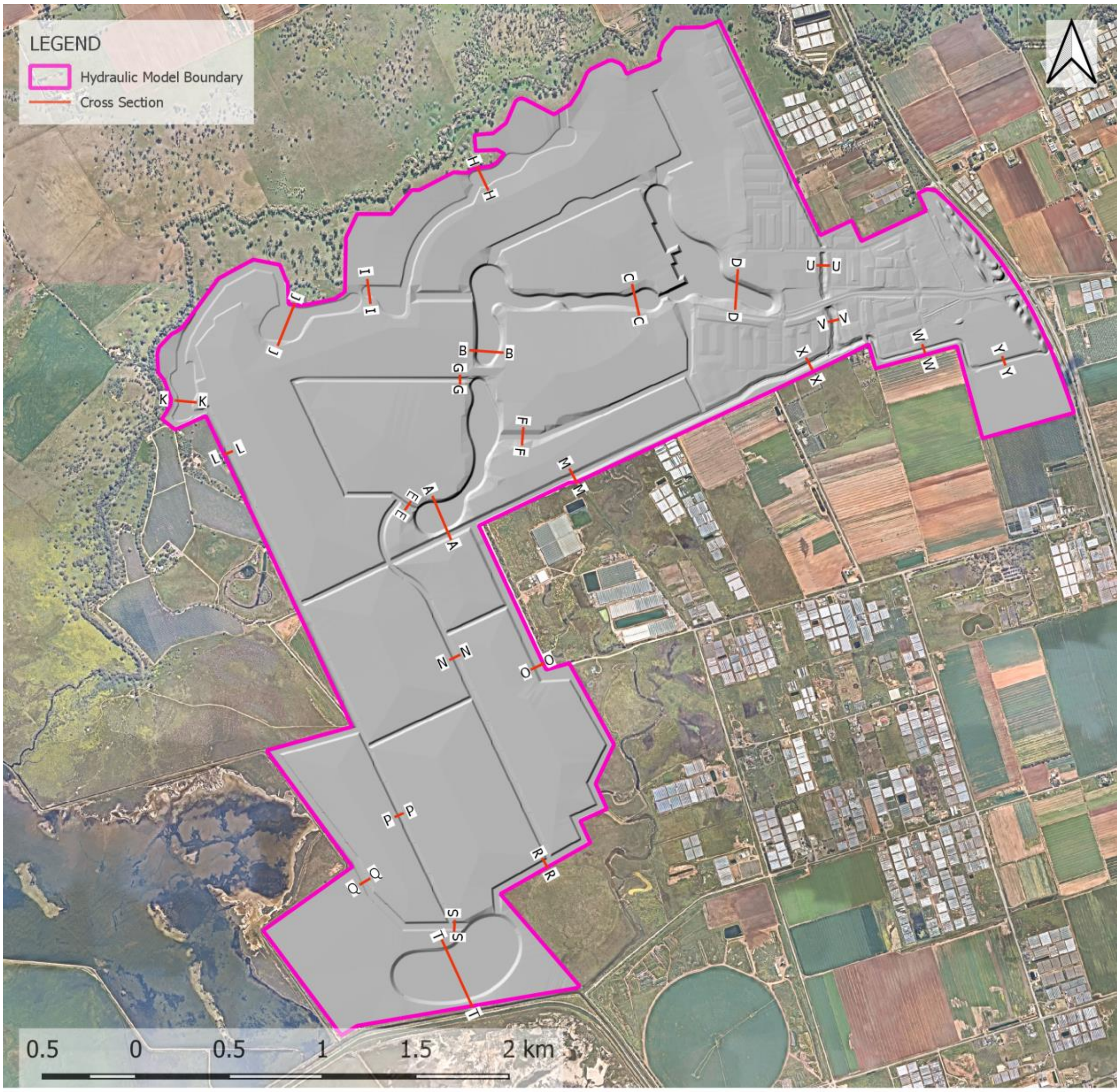


Figure 5: Cross Section Locations

Table 3: Freeboard for Each Cross Section

CROSS SECTION	FREE BOARD (m)
A-A	1.49
B-B	2.37
C-C	2.39
D-D	3.93
E-E	1.70
F-F	1.73
G-G	1.75
H-H	0.90
I-I	1.09
J-J	1.76
K-K	0.76
L-L	2.30
M-M	1.61
N-N	1.68
O-O	0.82
P-P	1.79
Q-Q	0.49
R-R	0.70
S-S	0.50
T-T	0.26
U-U	2.12
V-V	2.20
W-W	1.12
X-X	2.04
Y-Y	1.18

4 CONCLUSIONS

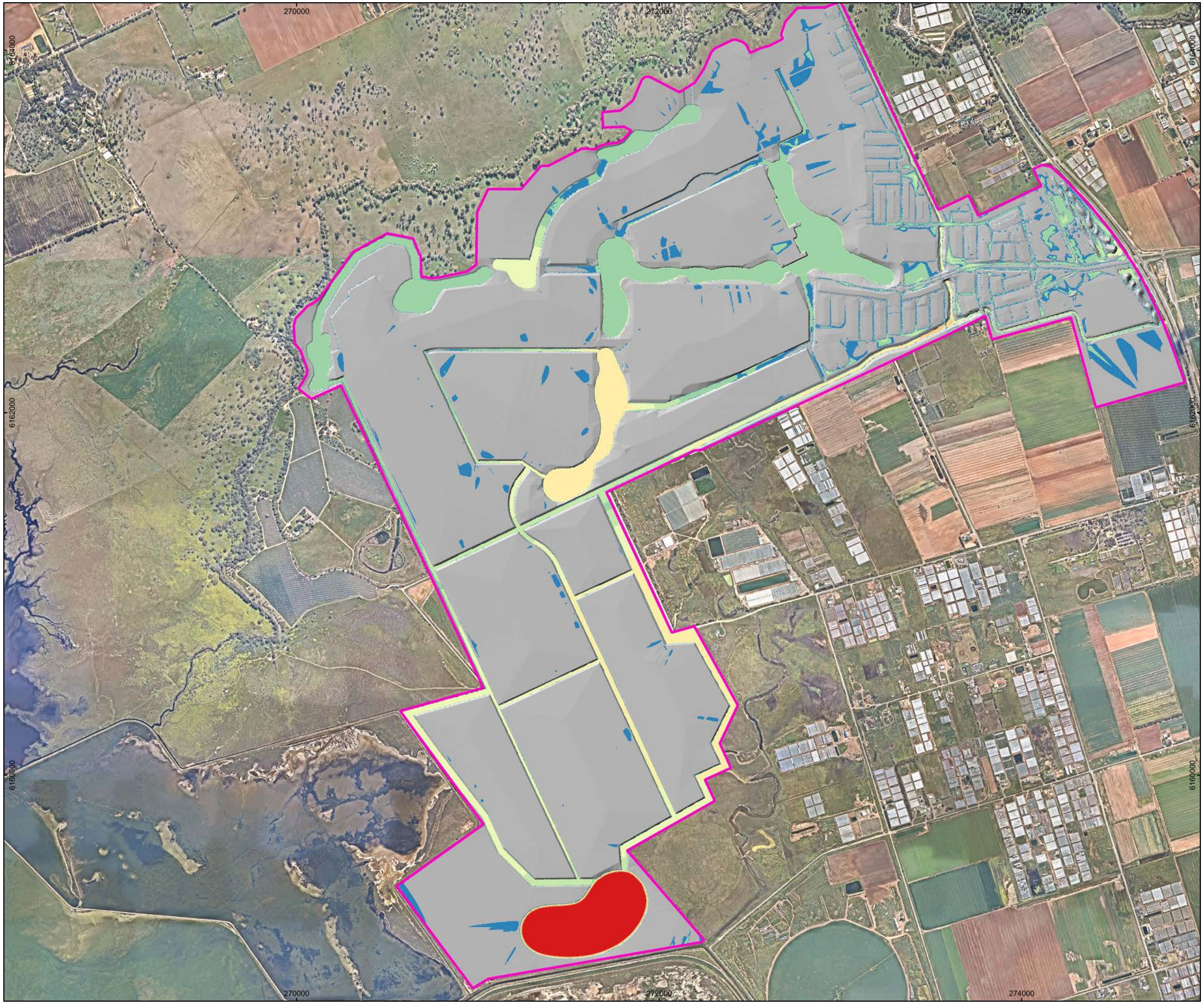
As part of the Riverlea development, a revised flood modelling assessment has been undertaken to account for modifications to the original development site configuration assessed in 2009. The current plans for the development site now include three saltwater lakes, channels, and a detention basin. The saltwater lakes contain saline water and any spills from the lakes may cause potential risks to the environment and adjacent infrastructure. The capacity of the saltwater lakes, detention basin and channel system were assessed in this report to ensure the system can contain 1% AEP storm event.

A 1D/2D TUFLOW model was developed in accordance with AR&R 2019 guideline for undertaking the flood assessment. The latest development site design surface has been used. A range of short to long rainfall durations have been modelled to ensure the peak flood levels were captured in the results.

The flood modelling results demonstrated:

- For the saltwater lakes, 6 and 9 hours storm durations were the critical event. The freeboard for lakes 1, 2 and 3 were 1.4m, 2.4m and 1.7m AHD respectively. Therefore, they have sufficient capacity to contain the 1% AEP storm event.
- The detention basin reaches its peak elevation in a 24-hour event. If a 750 mm diameter outlet pipe is used, the detention basins maximum water elevation is 2.24 m AHD. For these conditions the basin will have 260 mm of freeboard to its spillway elevation.
- For the channels, the critical storm event durations vary from 1 to 12 hours depending on the location of the channel. The channel freeboards vary from 0.26m to 2.30m. No spill event has been modelled to occur from any channel.
- The peak outflow to Thompson’s Outfall Channel is approximately 1.4m^3/s compared to an estimated pre-development flow rate of 10m^3/s.

ATTACHMENT A
FLOOD MAPS



LEGEND

Hydraulic Model Boundary

Peak Flood Depth (m)

- <= 0.05
- 0.05 - 0.10
- 0.10 - 0.25
- 0.25 - 0.50
- 0.50 - 0.75
- 0.75 - 1.00
- 1.00 - 1.25
- 1.25 - 1.50
- 1.50 - 2.00
- > 2.00

200 0 200 400 600 800 m

Scale 1:20,000 on A3

Coordinate System: GDA 1994 MGA Zone 54

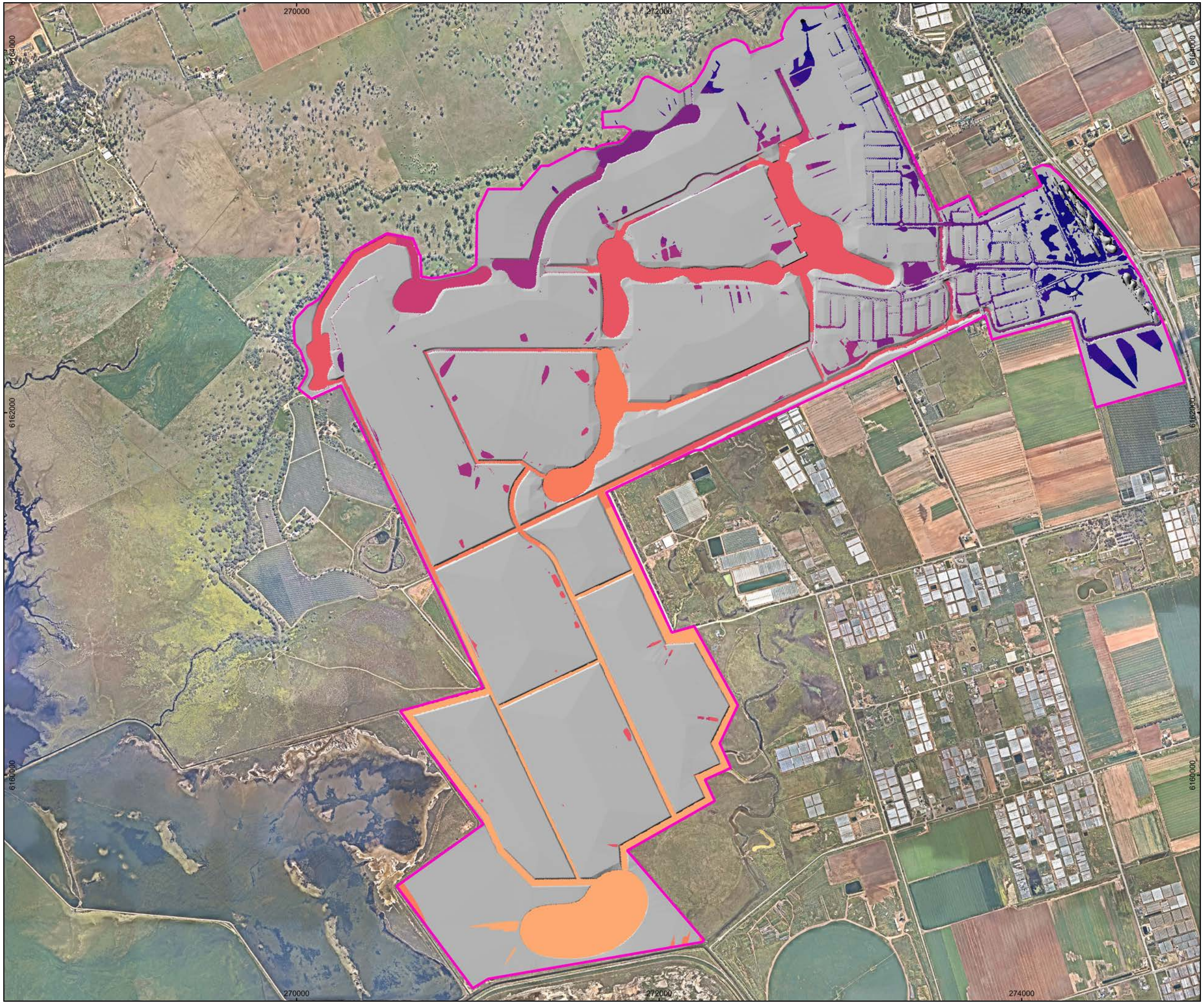
WGA

Map 01

Riverlea Development

Peak Flood Depth
1% AEP

Disclaimer: While all reasonable care has been taken to ensure the information contained on this map is up to date and accurate, no guarantee is given that the information portrayed is free from error or omission. Any relevance placed on such information shall be at the risk of the user.



LEGEND

Hydraulic Model Boundary

Peak Flood Height (mAHD)

- <= 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 4.0
- 4.0 - 5.0
- 5.0 - 6.0
- 6.0 - 7.0
- 7.0 - 8.0
- 8.0 - 9.0
- 9.0 - 10.0
- 10.0 - 11.0
- > 11.0

200 0 200 400 600 800 m

Scale 1:20,000 on A3

Coordinate System: GDA 1994 MGA Zone 54

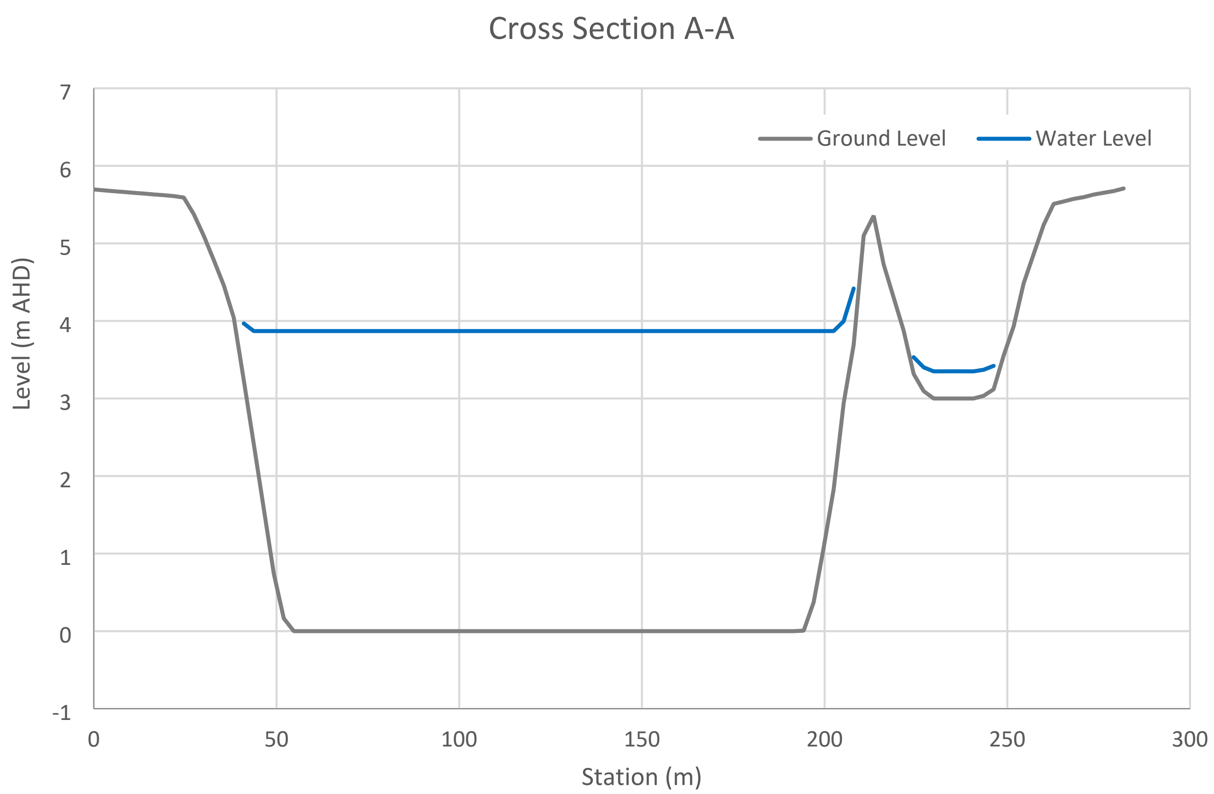
WGA

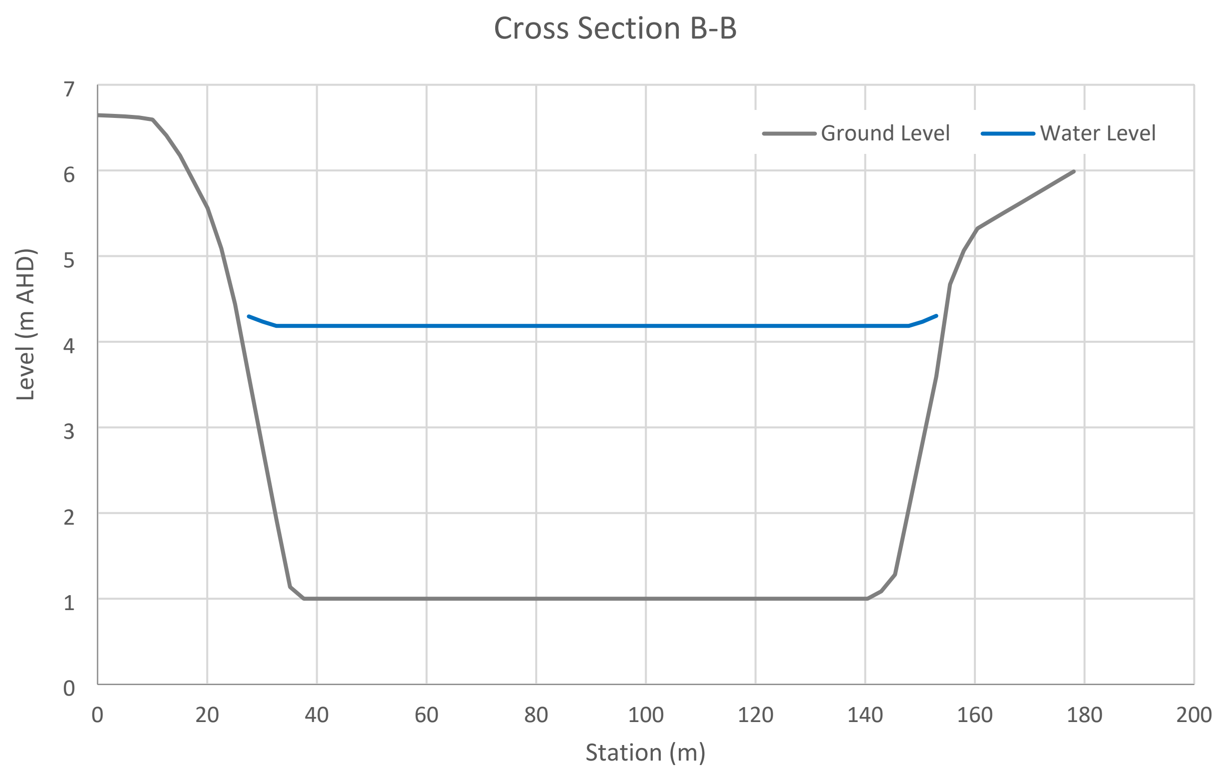
Map 02

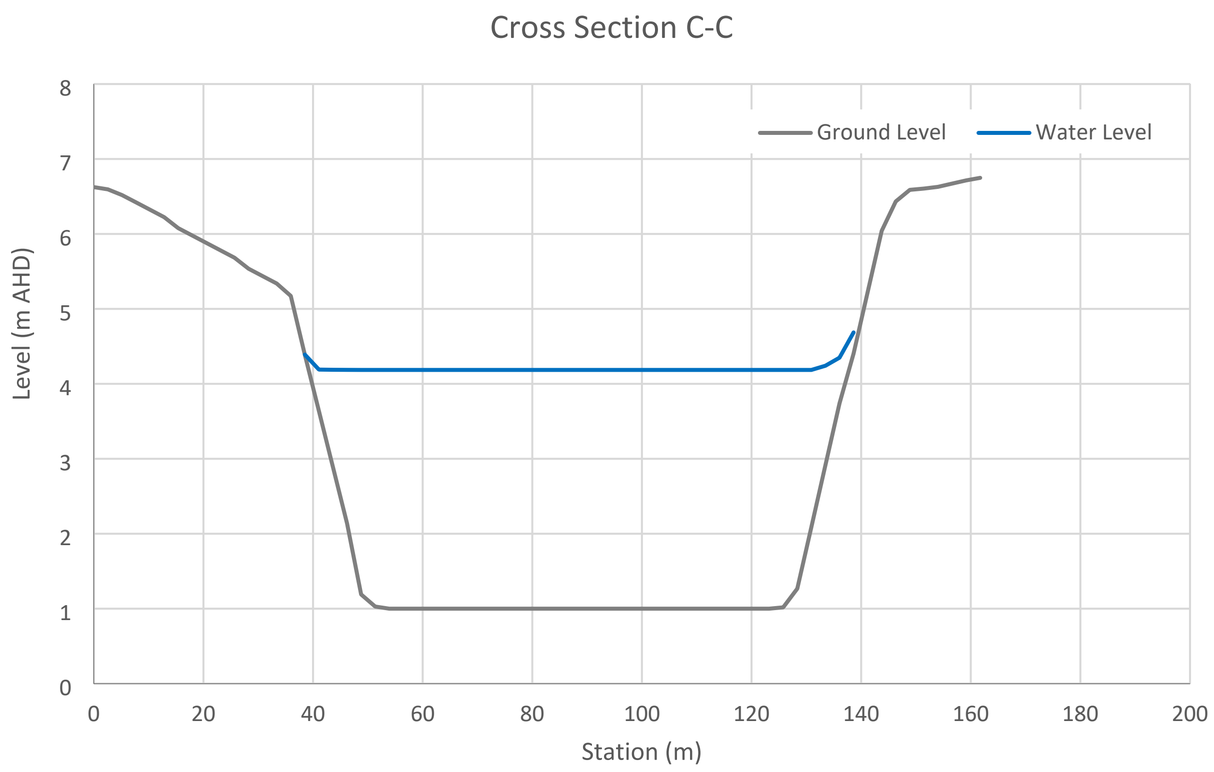
Riverlea Development

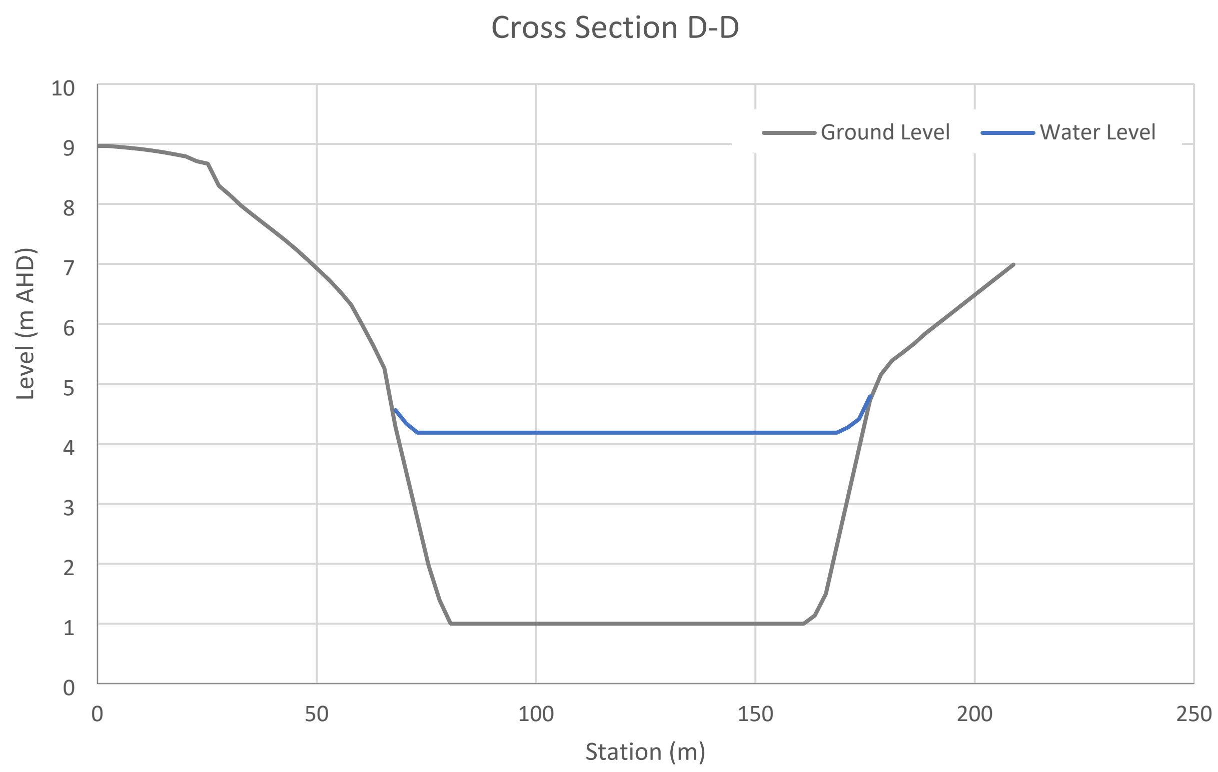
Peak Flood Height
1% AEP

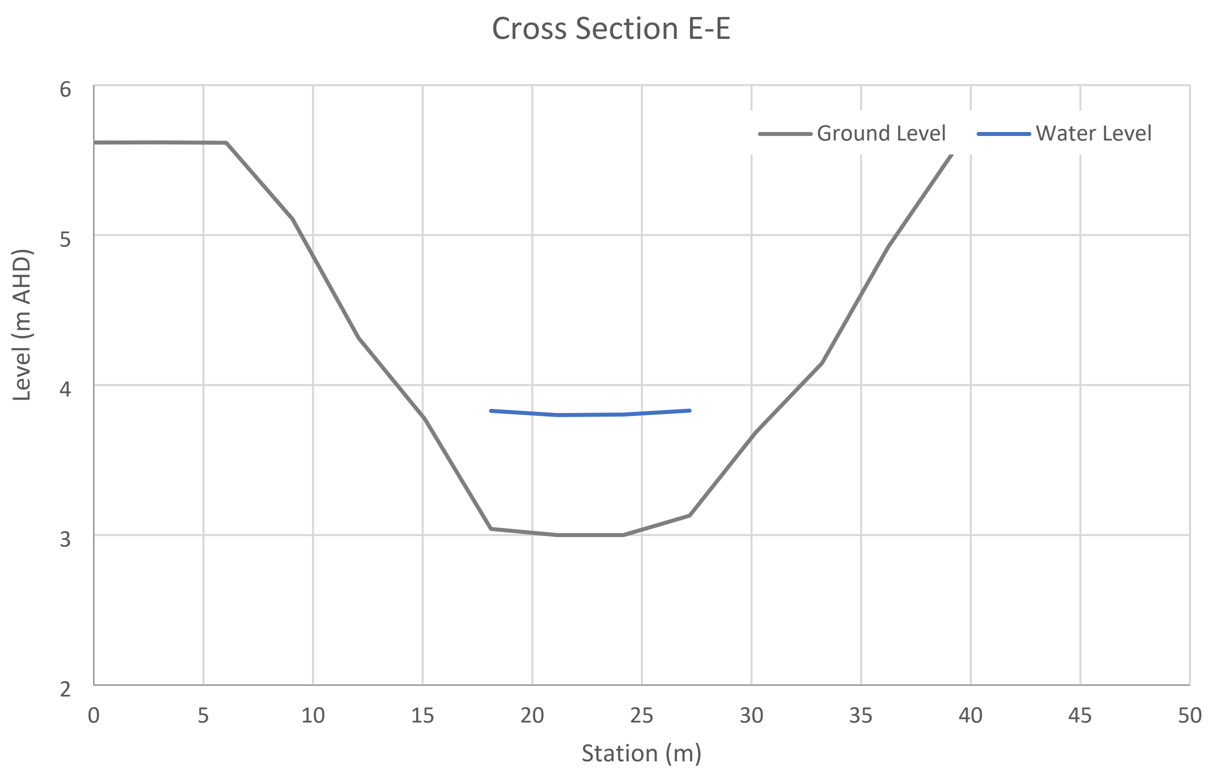
ATTACHMENT B
CROSS SECTIONS FOR PEAK FLOOD WATER LEVELS


Cross Section A-A
Ground Level
Water Level
Level (m AHD)
7
6
5
4
3
2
1
0
-1
0
50
100
150
200
250
300
Station (m)


Cross Section B-B
Ground Level
Water Level
Level (m AHD)
7
6
5
4
3
2
1
0
0
20
40
60
80
100
120
140
160
180
200
Station (m)


Cross Section C-C
Ground Level
Water Level
Level (m AHD)
8
7
6
5
4
3
2
1
0
0
20
40
60
80
100
120
140
160
180
200
Station (m)


Cross Section D-D
Ground Level
Water Level
Level (m AHD)
10
9
8
7
6
5
4
3
2
1
0
0
50
100
150
200
250
Station (m)


Cross Section E-E
Ground Level
Water Level
Level (m AHD)
Station (m)
6
5
4
3
2
0
5
10
15
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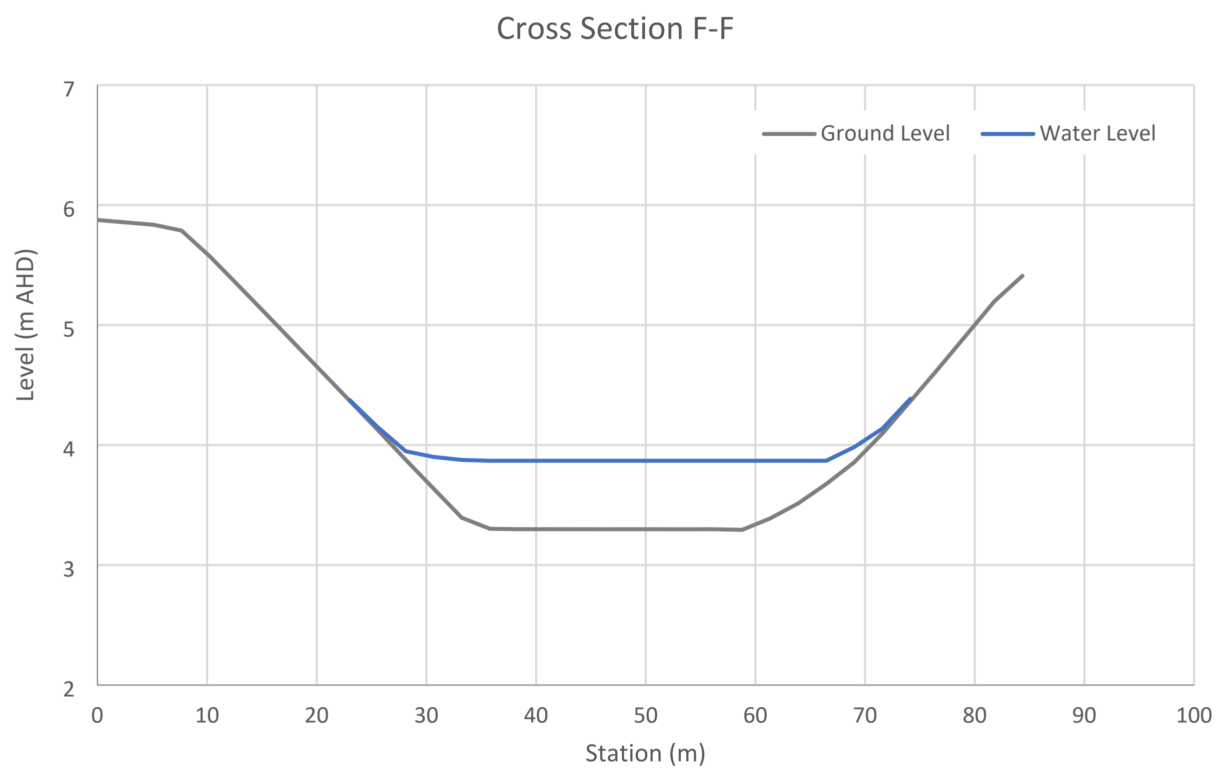
30
35
40
45
50


Cross Section F-F
Ground Level
Water Level
Level (m AHD)
Station (m)
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6
5
4
3
2
0
10
20
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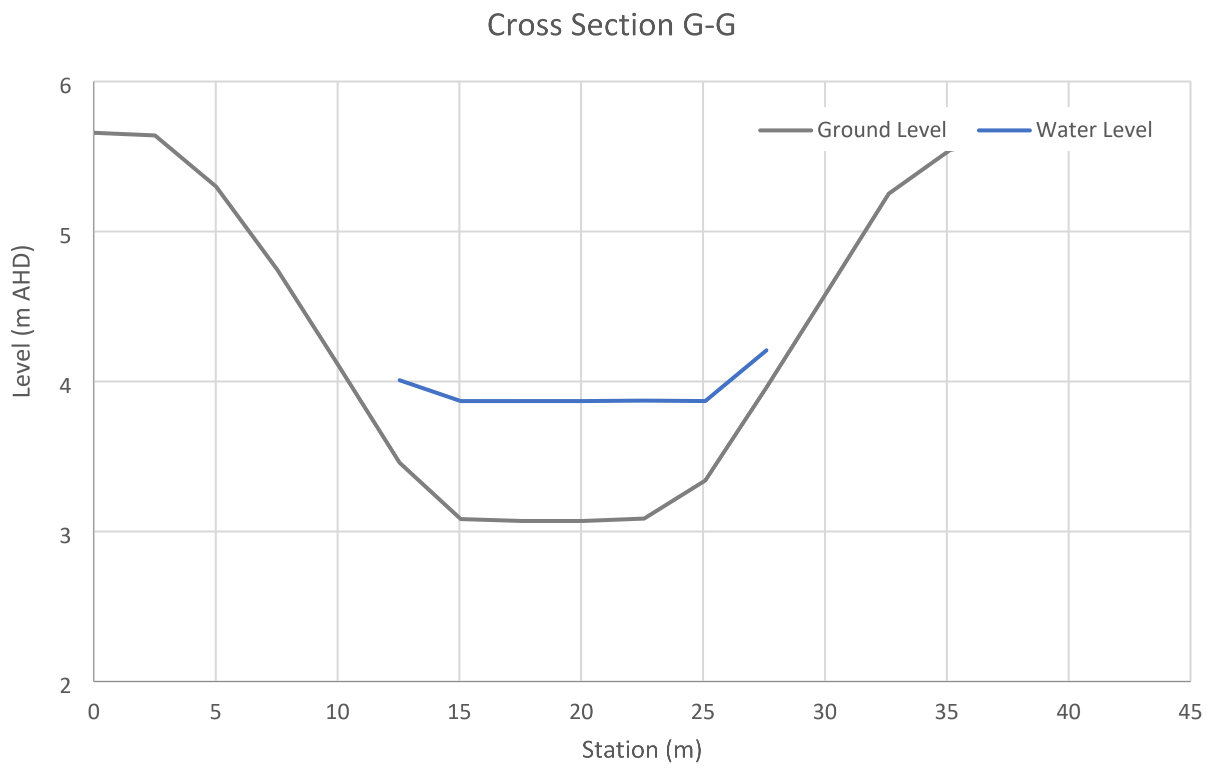
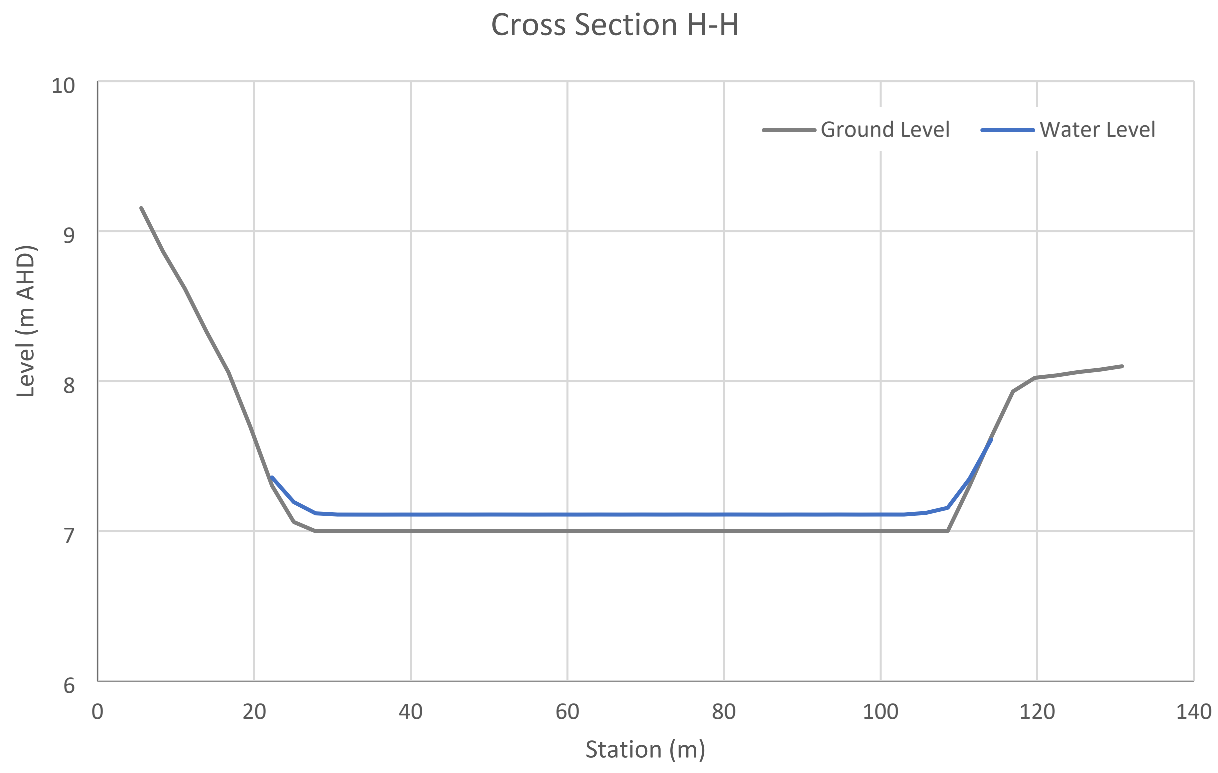
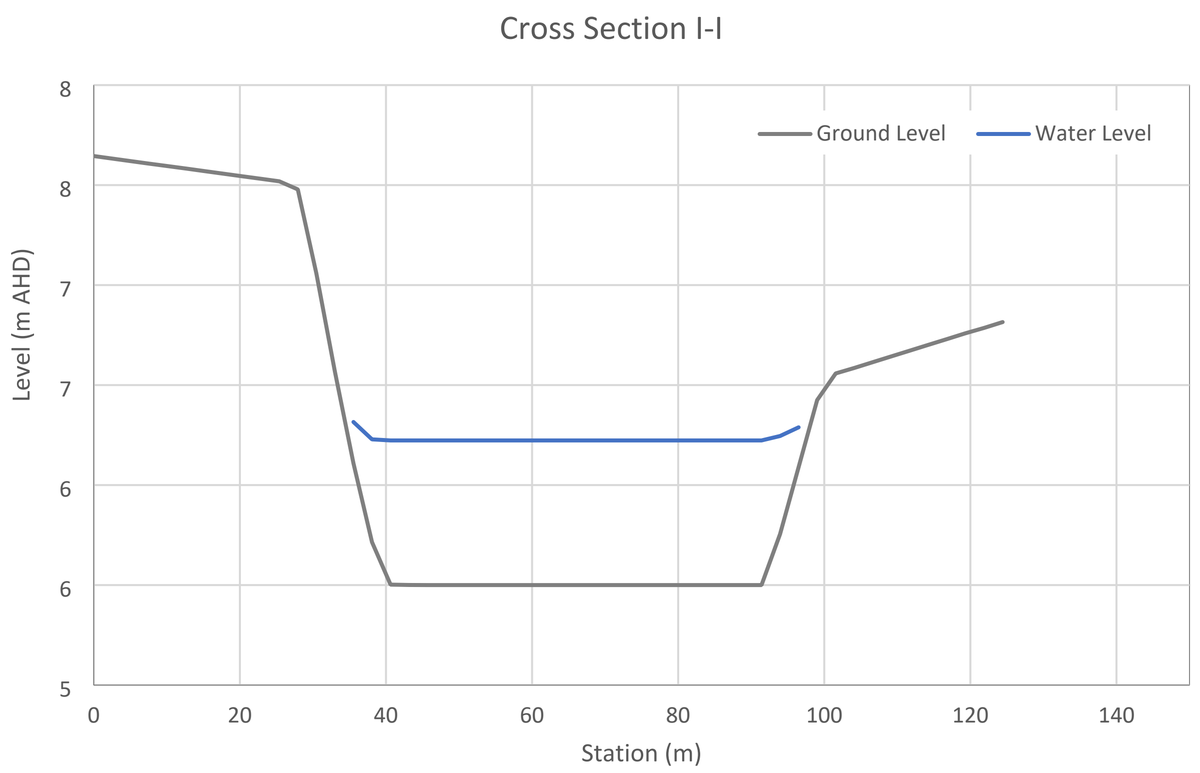
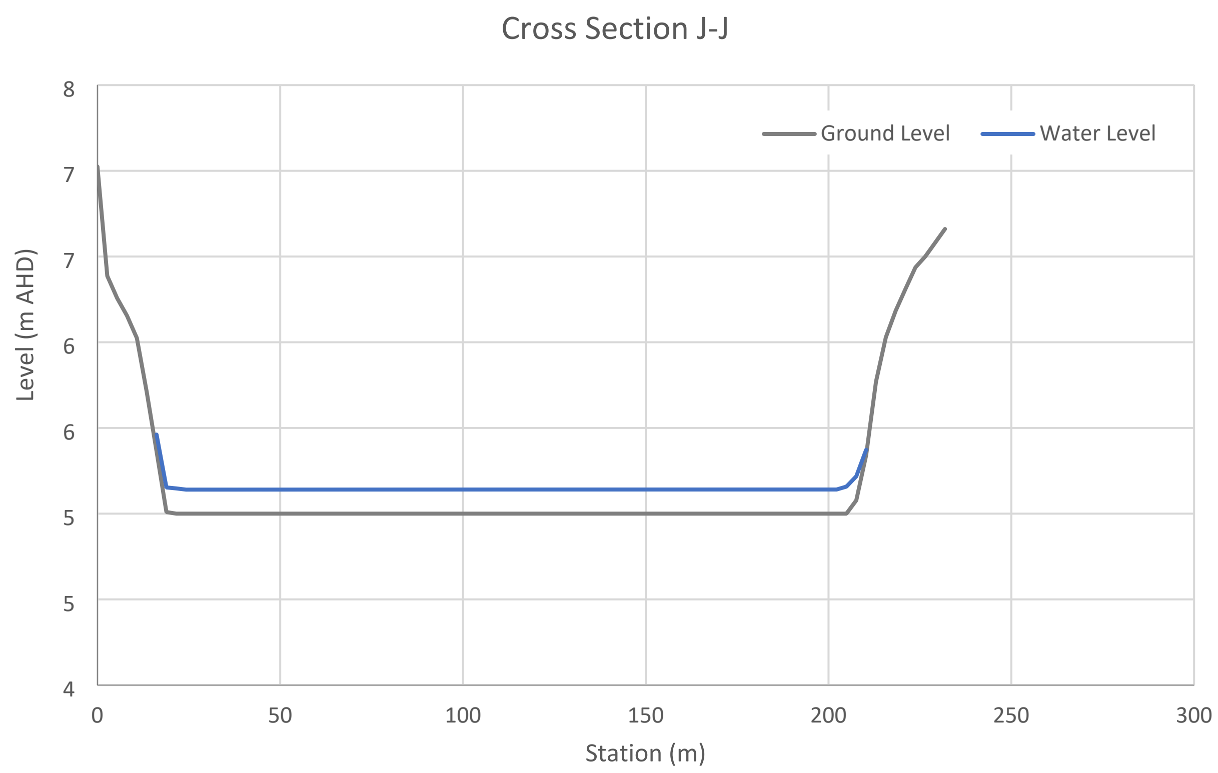
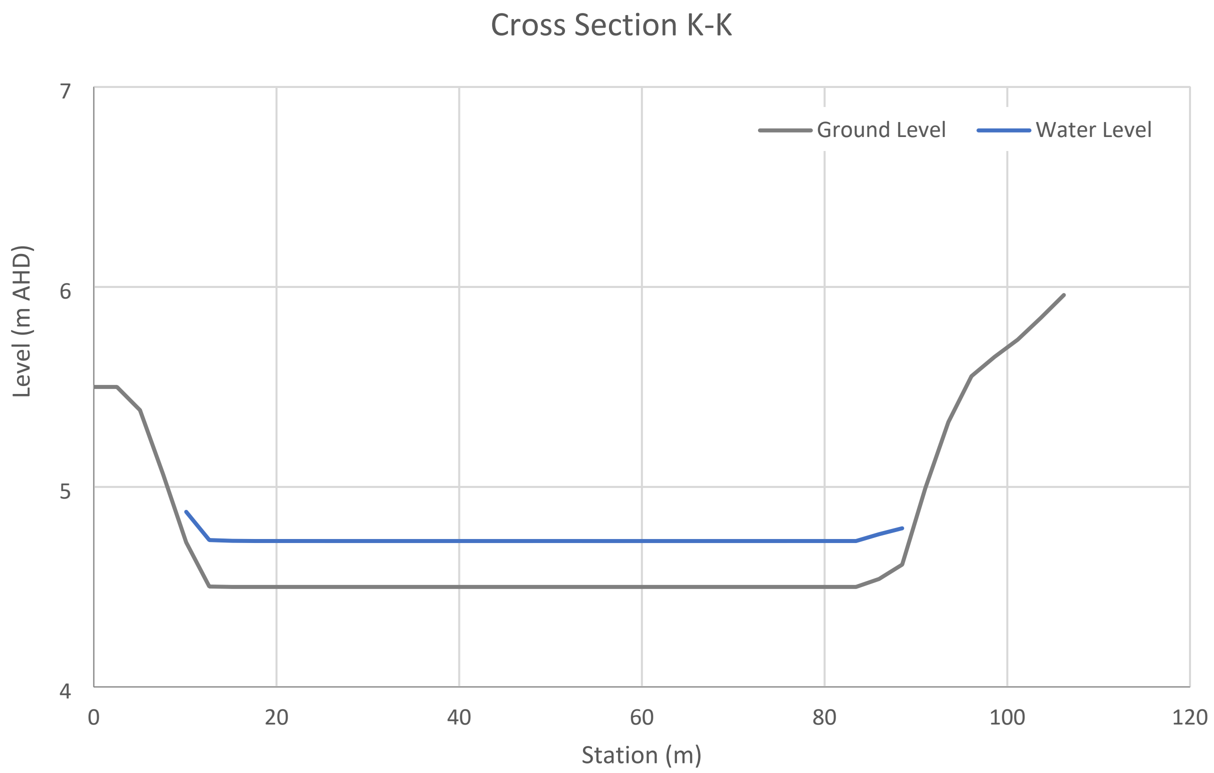
60
70
80
90
100

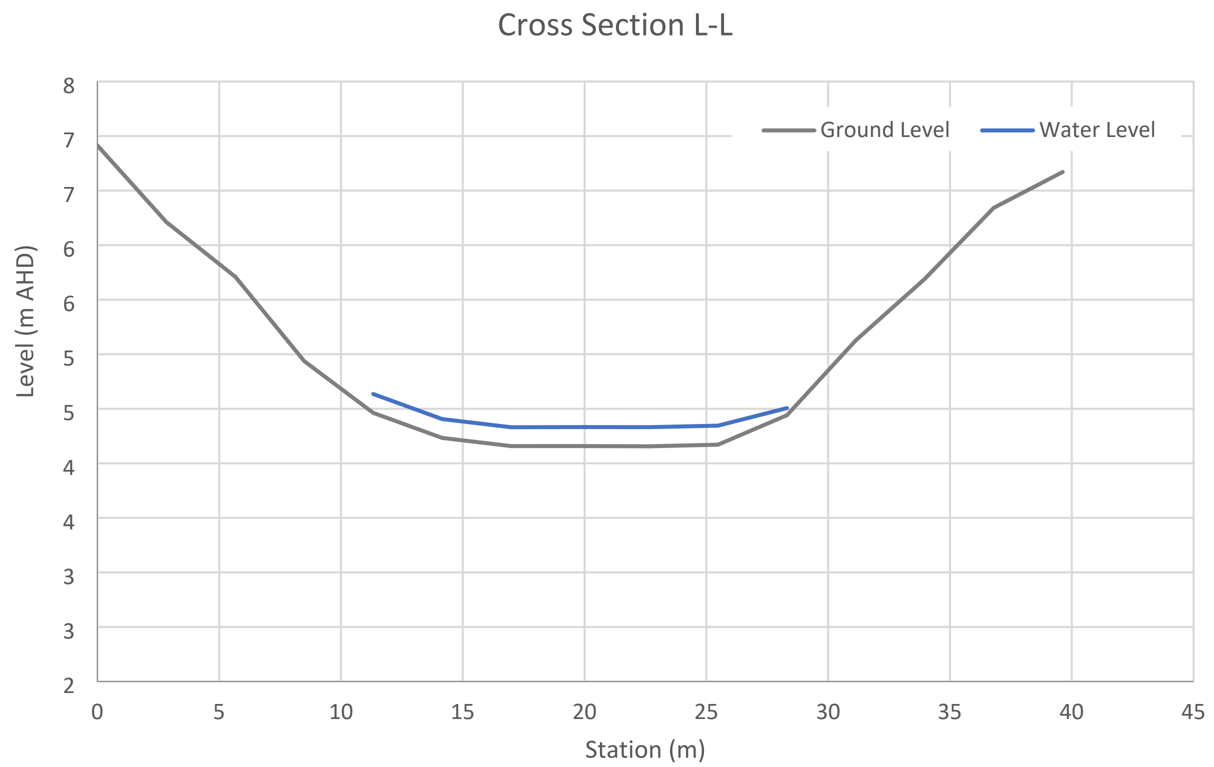

Cross Section G-G
Ground Level
Water Level
Level (m AHD)
Station (m)
6
5
4
3
2
0
5
10
15
20
25
30
35
40
45


Cross Section H-H
Ground Level
Water Level
Level (m AHD)
Station (m)
10
9
8
7
6
0
20
40
60
80
100
120
140


Cross Section I-I
Ground Level
Water Level
Level (m AHD)
Station (m)
8
8
7
7
6
6
5
0
20
40
60
80
100
120
140


Cross Section J-J
Ground Level
Water Level
Level (m AHD)
Station (m)
8
7
7
6
6
5
5
4
0
50
100
150
200
250
300


Cross Section K-K
Ground Level
Water Level
Level (m AHD)
Station (m)
7
6
5
4
0
20
40
60
80
100
120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Water Level
Level (m AHD)
Station (m)
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7
7
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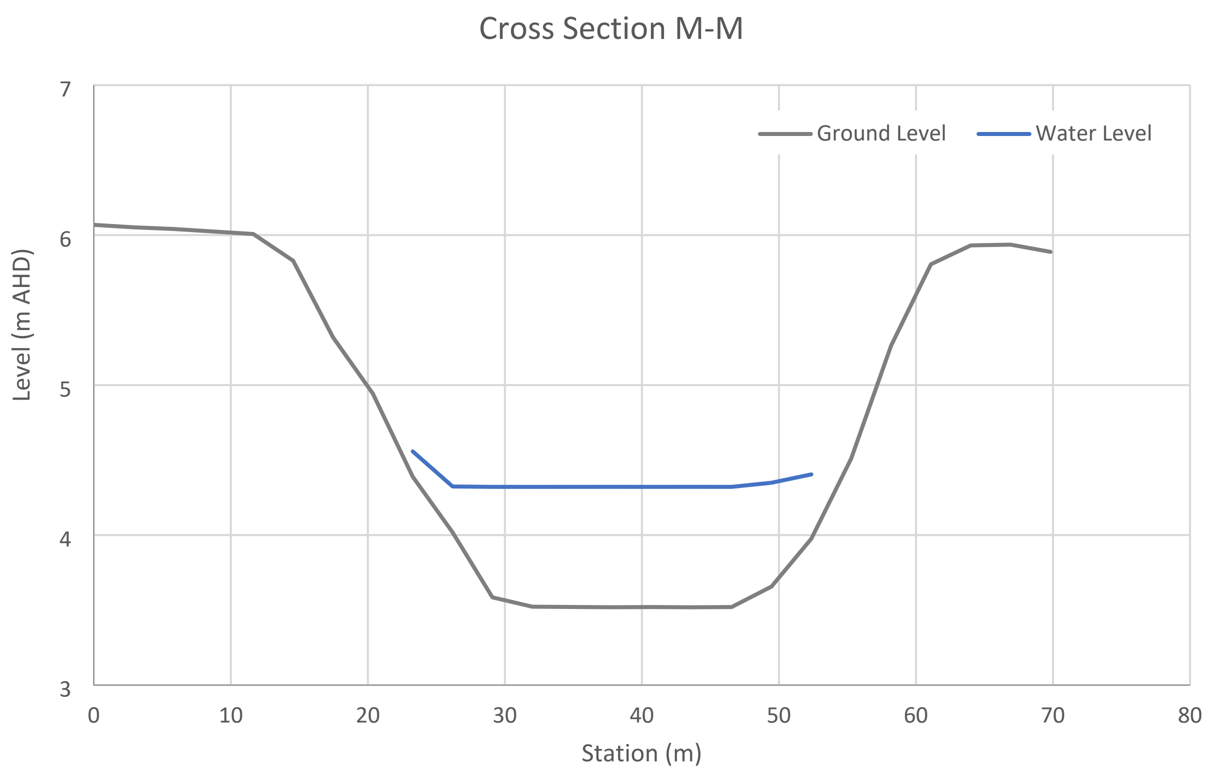
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Station (m)
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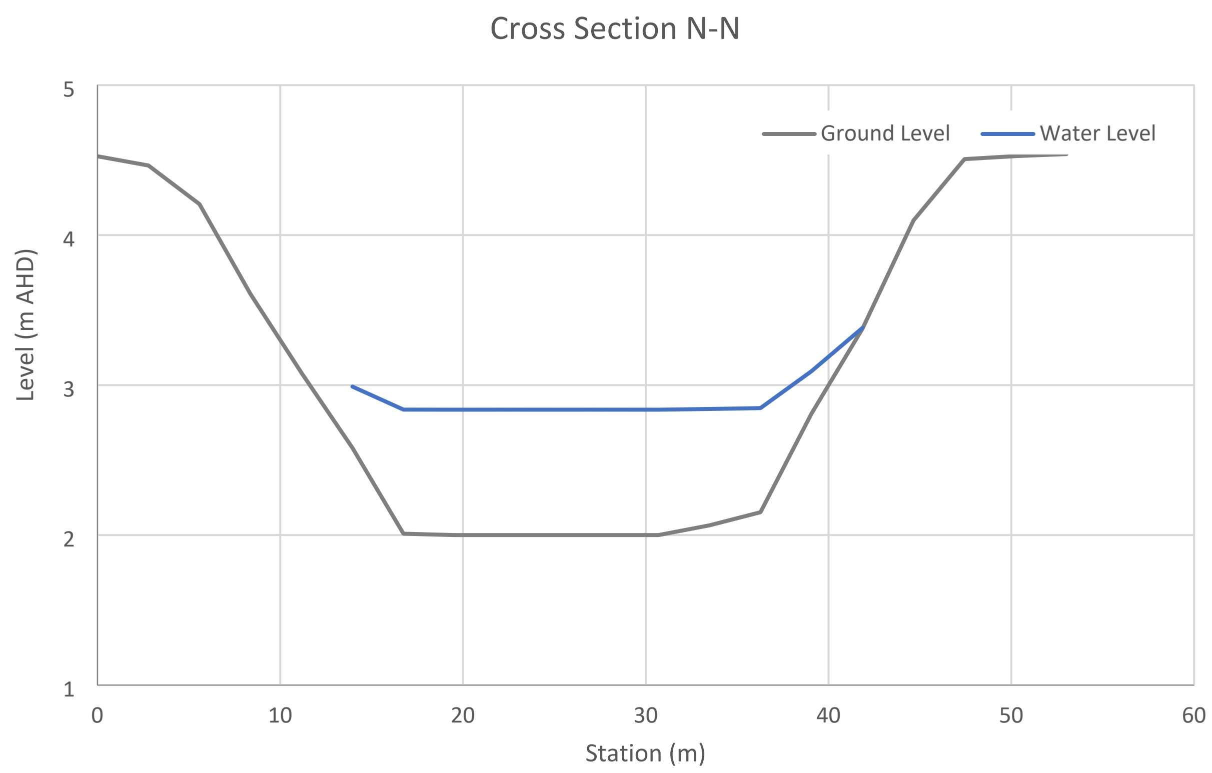
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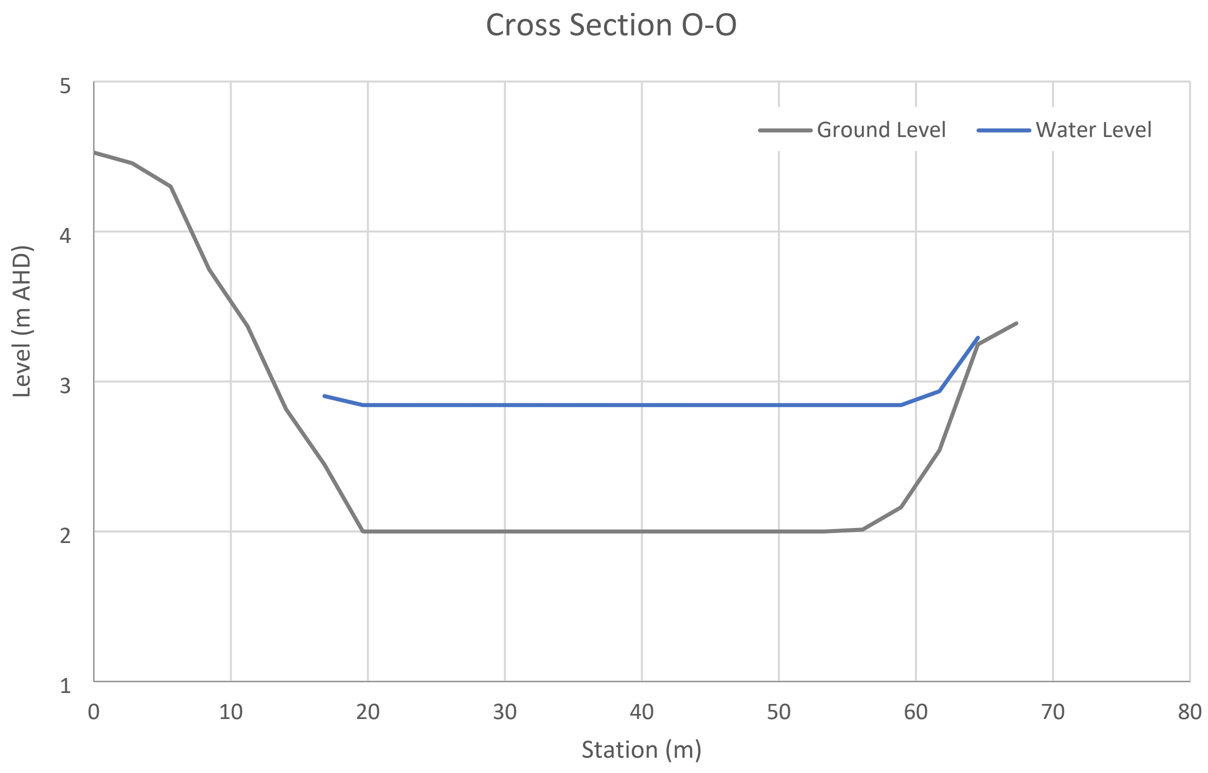
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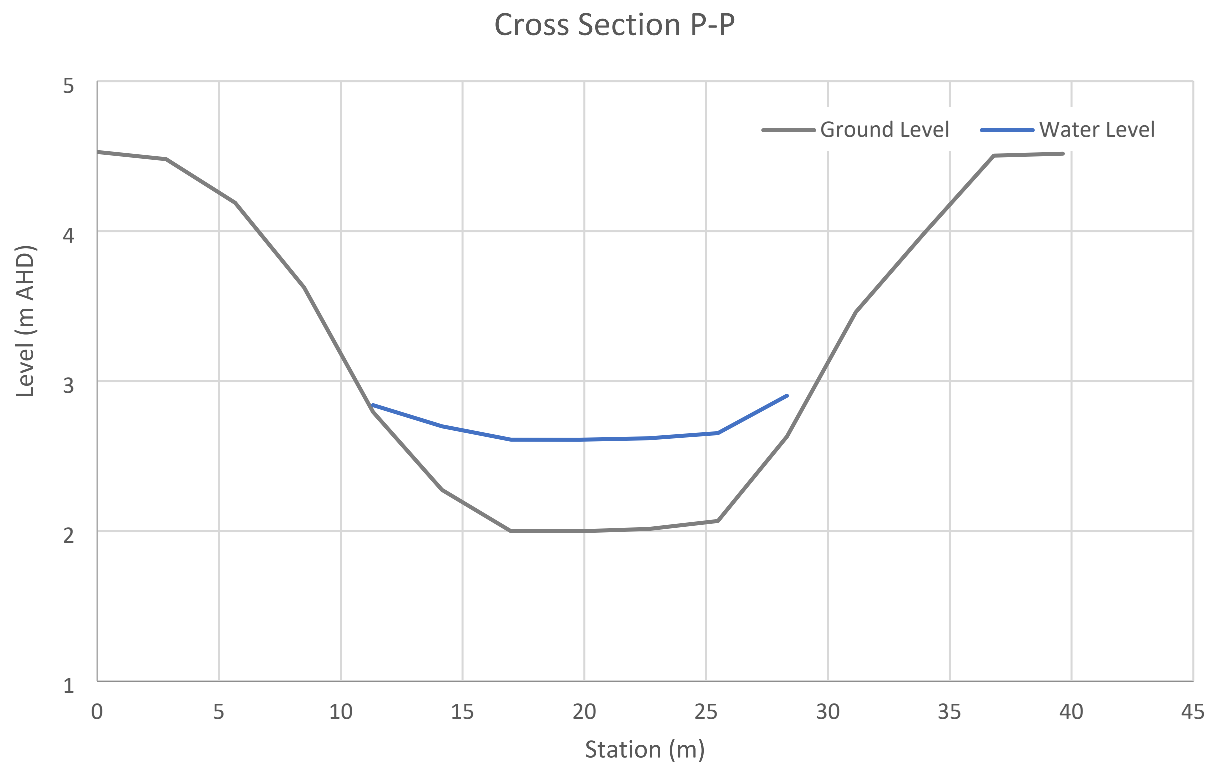
80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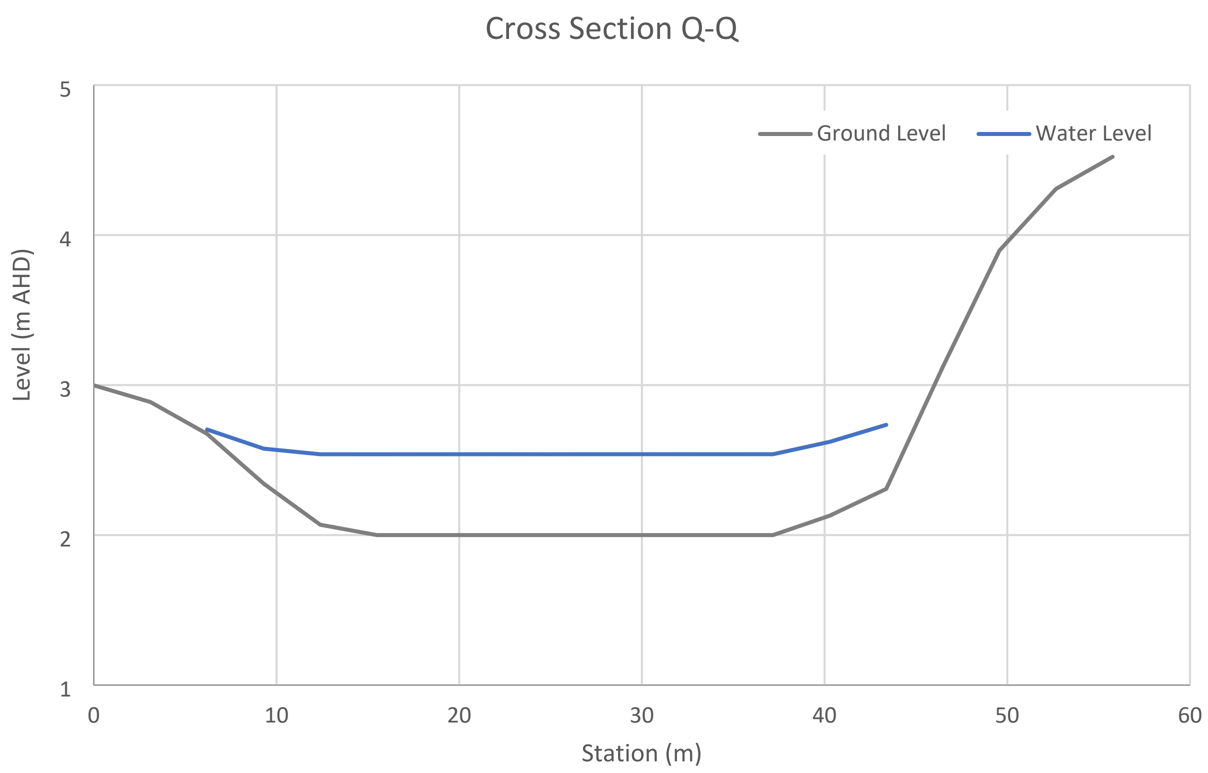
45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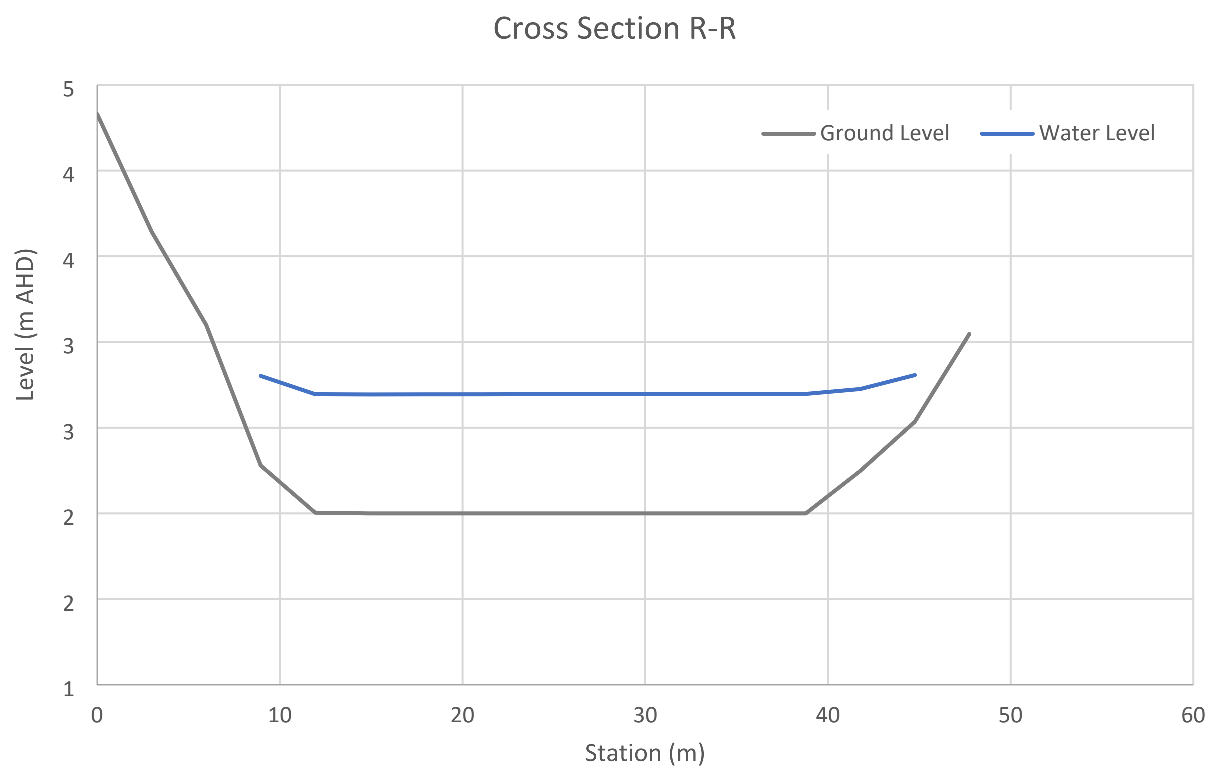
Station (m)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2
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10
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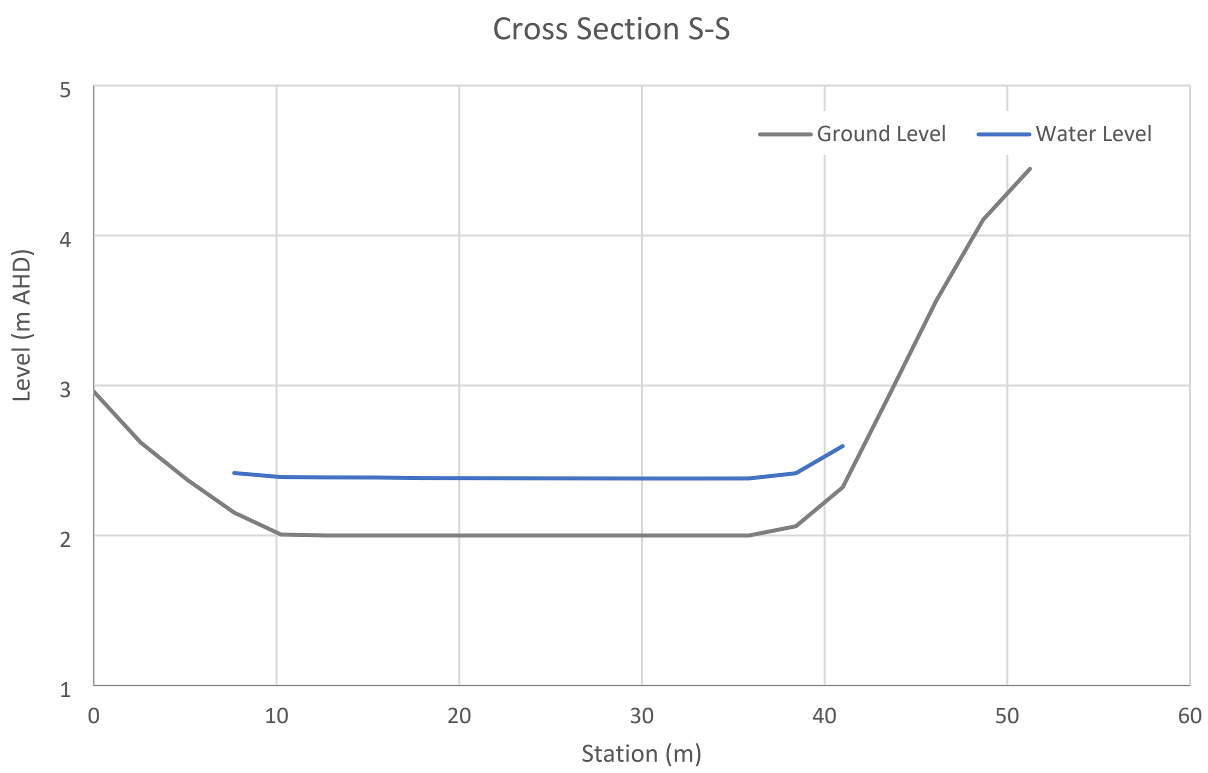
Station (m)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10
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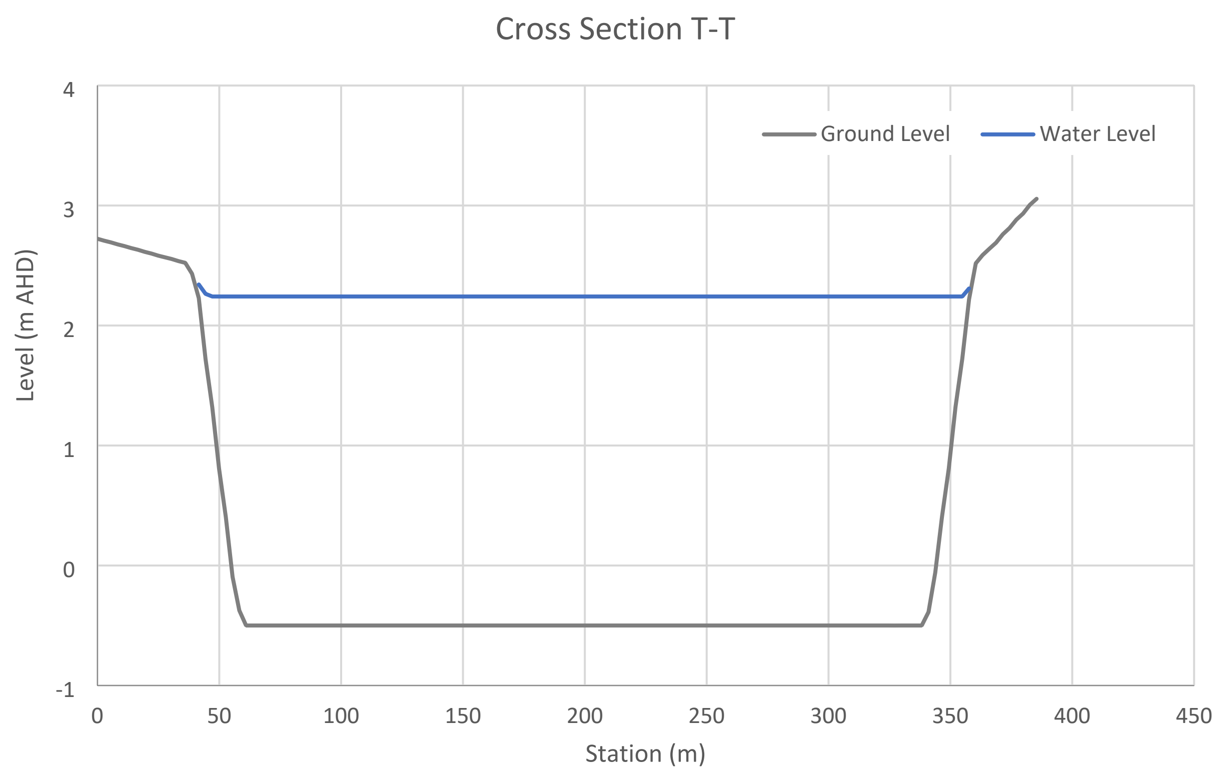
40
50
60
Station (m)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Ground Level
Water Level
Level (m AHD)
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0
-1
0
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100
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250
300
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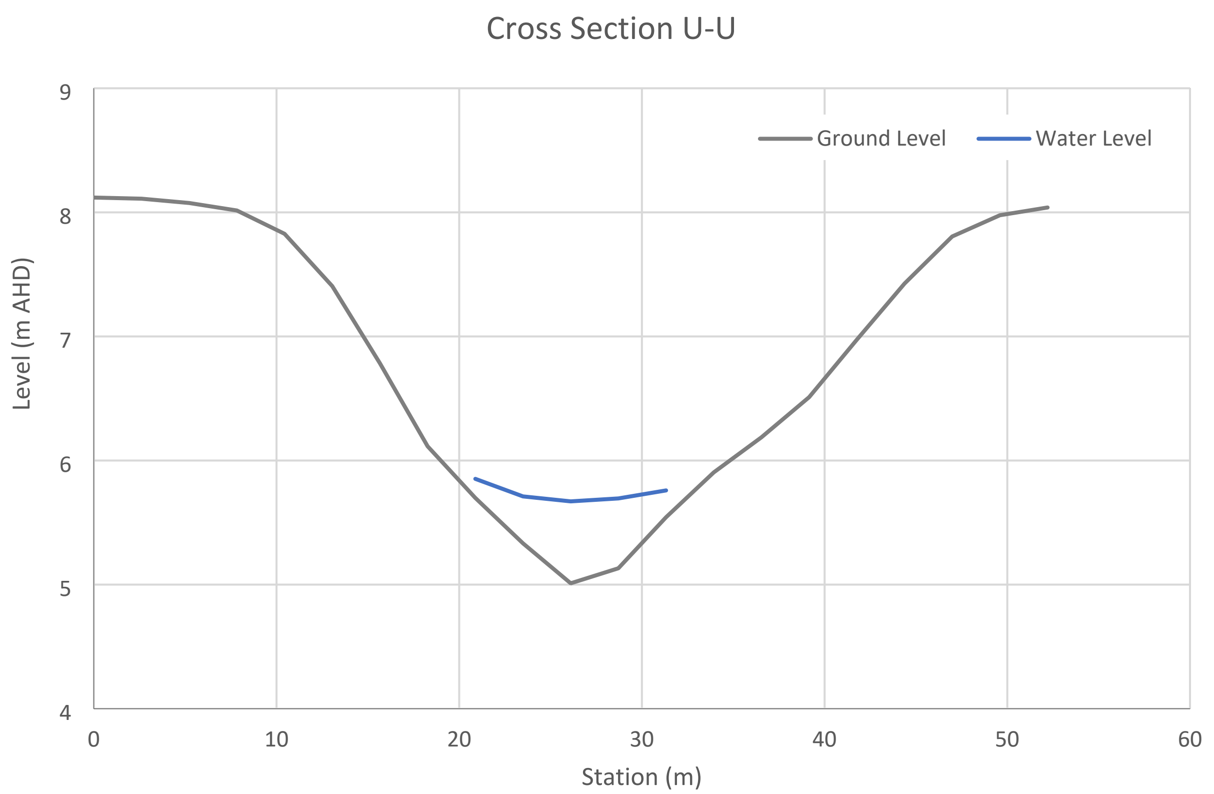
Station (m)


Cross Section U-U
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Water Level
Level (m AHD)
Station (m)
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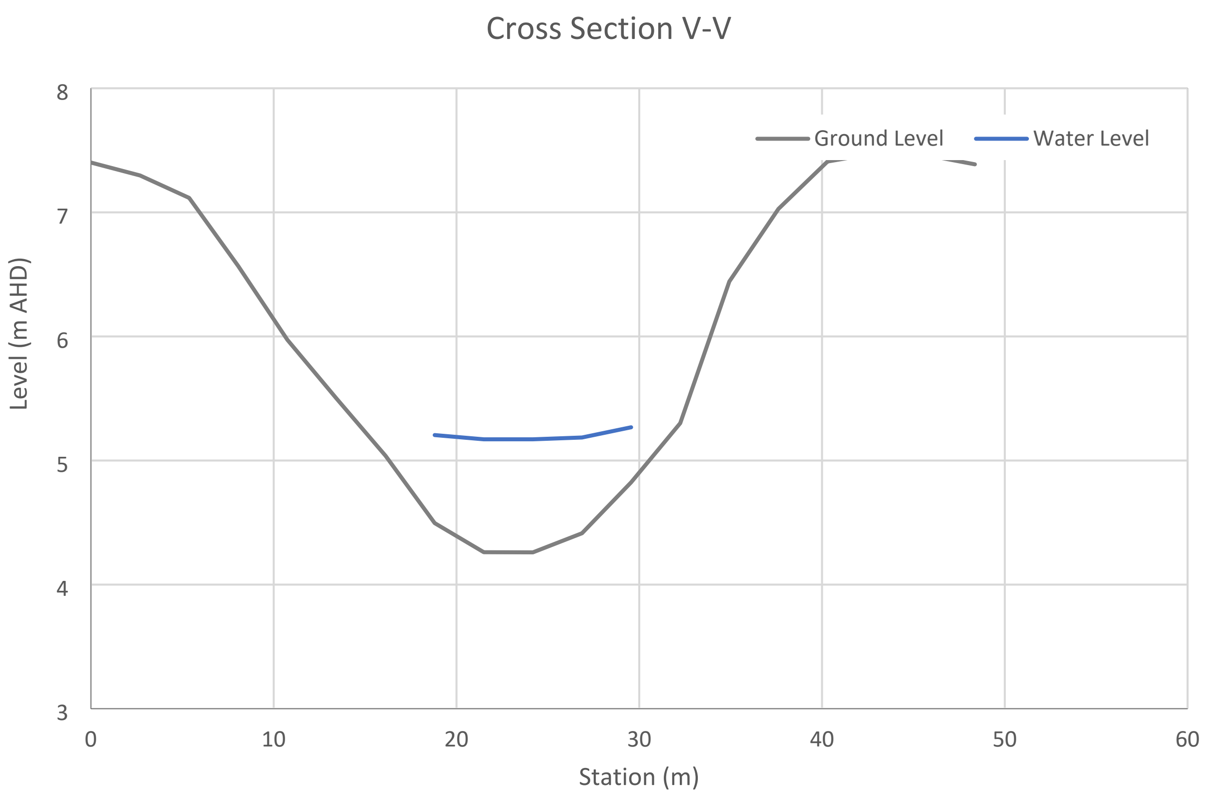
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Station (m)
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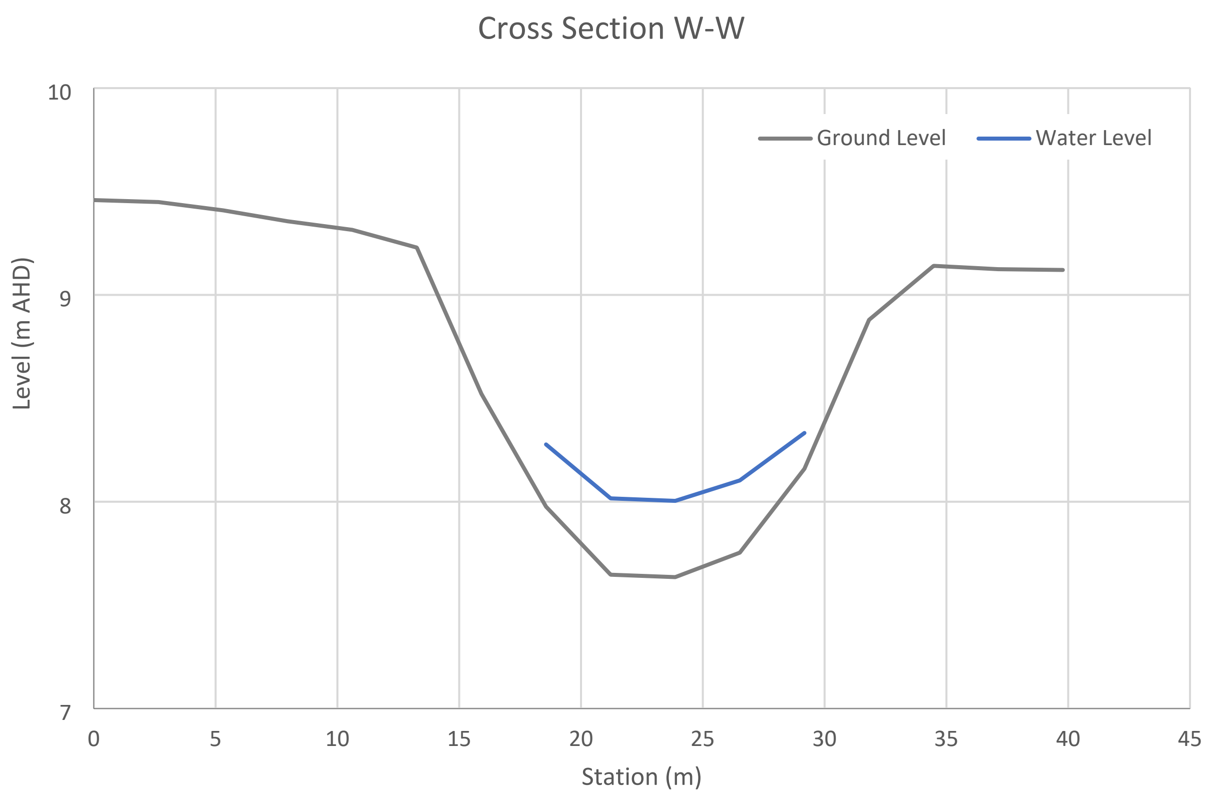
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Water Level
Level (m AHD)
Station (m)
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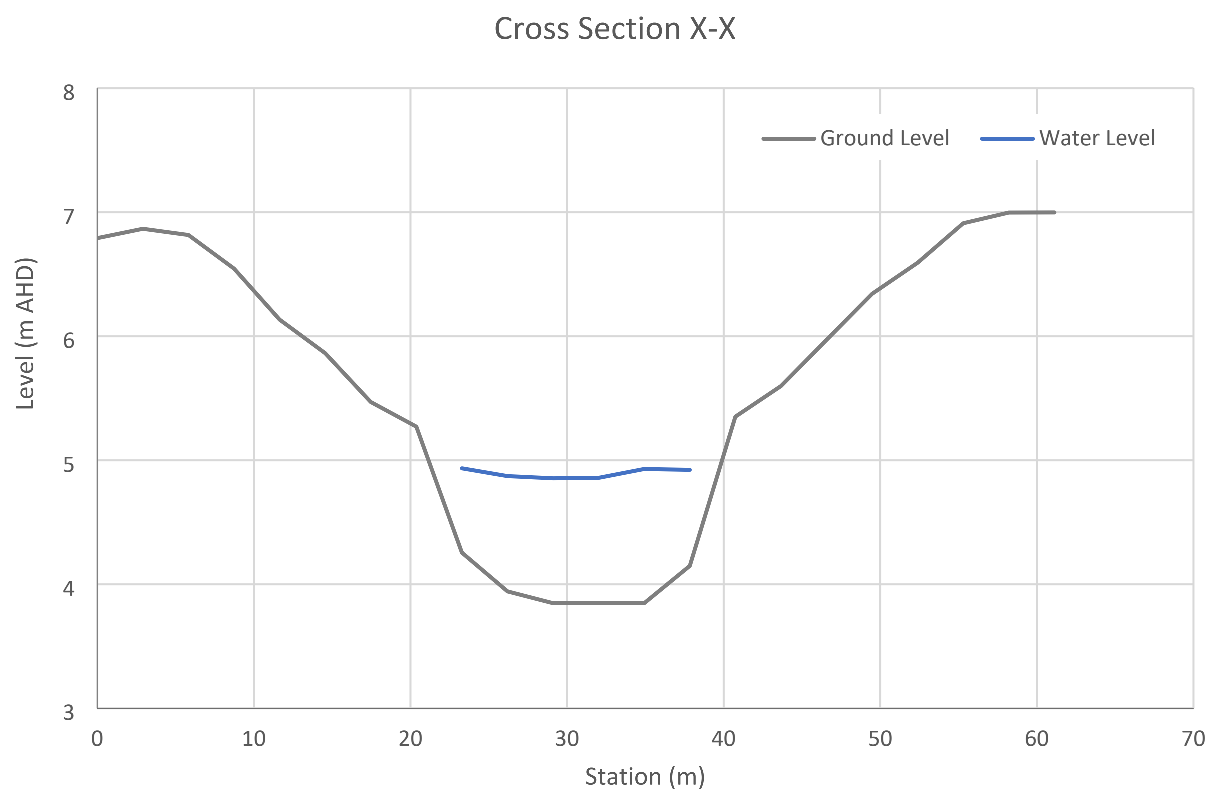
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Ground Level
Water Level
Level (m AHD)
Station (m)
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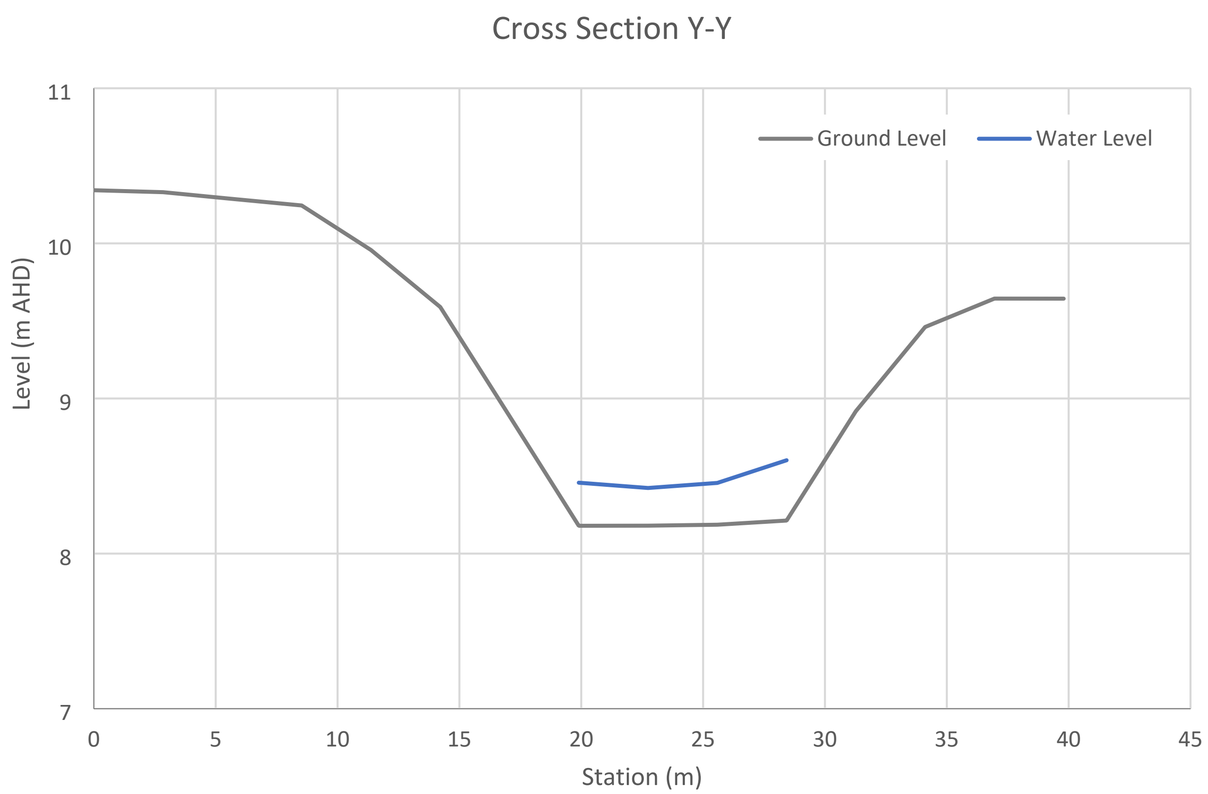

Cross Section Y-Y
Ground Level
Water Level
Level (m AHD)
11
10
9
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0
5
10
15
20
25
30
35
40
45
Station (m)

APPENDIX F
STORMWATER QUALITY MODELLING

Walker Corporation

Riverlea Park

2009 TECHNICAL PAPER UPDATE - STORMWATER QUALITY MODELLING

WGA080163

WGA080163-RP-CV-0012_B .

14 April 2023

Revision History

REV	DATE	ISSUE	ORIGINATOR	CHECKER	APPROVER
A	17.11.2022	Update to 2009 technical paper	SA	JS	DB
B	14.04.2023	Amendments based on EPA and DEW feedback	JL	JL	DB

CONTENTS

Figures

Tables

1 INTRODUCTION AND BACKGROUND

This stormwater treatment quality assessment has been undertaken to update the master plan level stormwater quality treatment analysis performed in the technical paper titled Stormwater Management Water, Wastewater and Recycled Water prepared by WGA (2009) (then W&G).

Since the 2009 Technical Paper, the Proposed Revised Riverlea Master plan (December 2021) now includes internal salt water lakes system (SWL) which integrate with the local trunk stormwater drainage channels in place of the original open drain system. The revised landform proposal now includes 40.4 ha of linked saline lakes centrally located within the development. This proposed SWL also provides an alternative to manage the breakout of the regional Gawler River floodwaters through the site. The concept plan is shown in Figure 1.

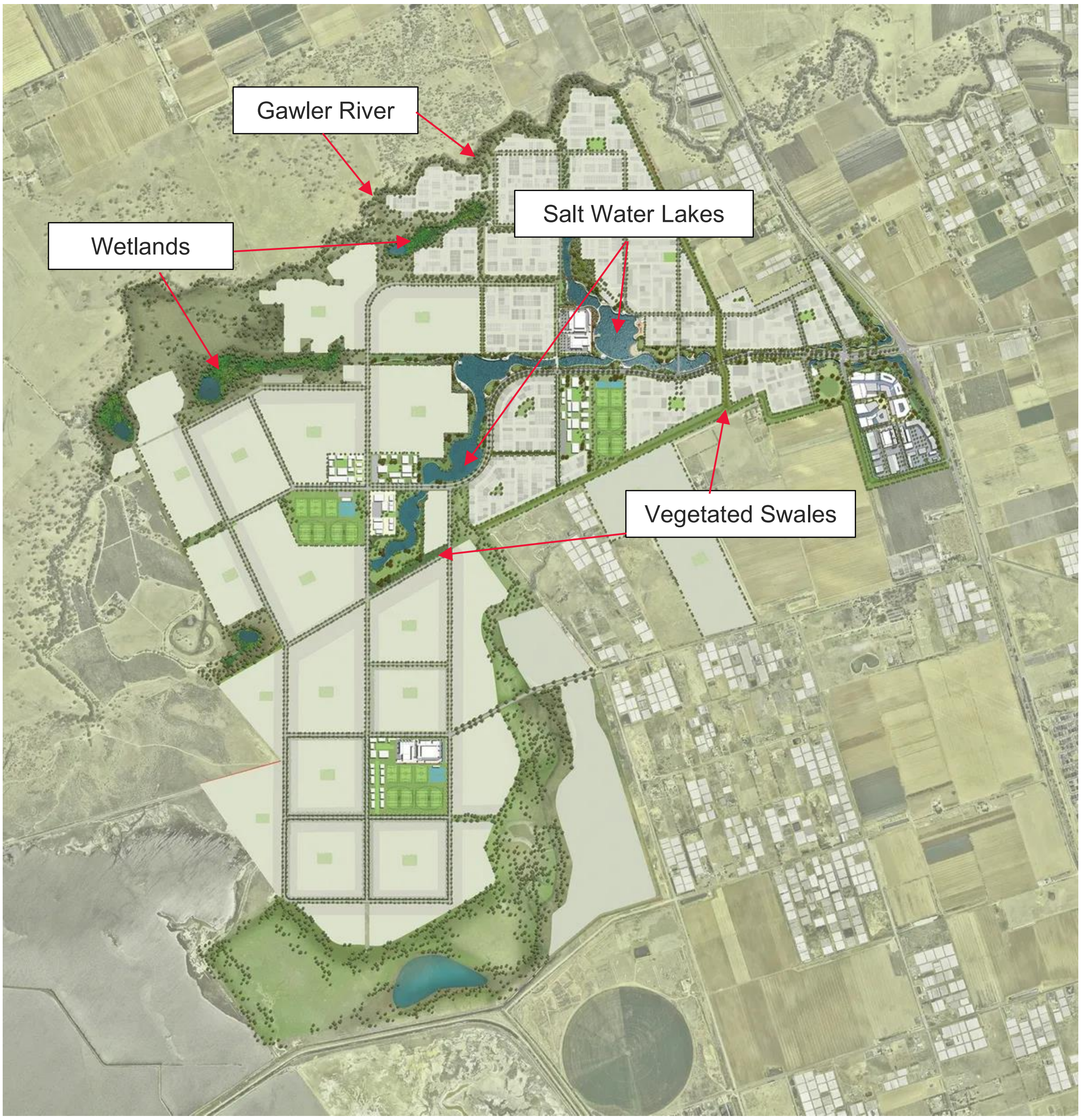

Figure 1 : Proposed Riverlea Master Plan (December 2021)

1.1 Objectives and Water Quality Criteria

The objective of this stormwater quality assessment is to evaluate the treatment performance of the proposed/revised systems within Riverlea Estate against the required standards at a master plan level.

The proposed stormwater treatment system was designed to treat the runoff in accordance with the standards as defined by:

- The South Australian EPA water quality policy WSUD targets.
- WSUD pollutant reduction targets as defined in the WSUD Guidelines for the Greater Adelaide Region (2013).
- Adopts to the framework principles of the ANZEC guidelines with regards to adopting a treatment train approach to minimise harm to downstream waters.

The pollutant treatment performance targets as specified in the above guidelines are:

- 80% retention of typical annual urban load of suspended solids (TSS)
- 60% retention of typical annual urban load of total phosphorus (TP)
- 45% retention of typical annual urban load of total nitrogen (TN)
- 90% reduction of gross pollutants of typical urban load (GP)

In addition to the above targets for the site as a whole, it was also aimed to achieve the treatment performance targets before discharging into the Salt Water Lakes (SWL). The basis of this is that the SWLs can be negatively impacted by the poor quality stormwater inflows from local catchments as described by BMT (2021) in Riverlea Concept Stormwater Quality Management Plan.

1.2 Treatment Catchment Plan

The treatment catchment plan described below was developed for a master plan level assessment. Therefore, the sizes and placement of proposed Water Sensitive Urban Design (WSUD) assets are not at a detailed design accuracy, and the details of these assets are to be further assessed in the detailed precinct level.

The internal catchments and flow directions used in the catchment plan were based on the concept earthwork model for the Master Plan. The locations and treatment catchments of the WSUD assets were also based on the proposed master plan and the concept earthwork model. The catchments and the WSUD assets as used in the MUSIC model are shown in Figure 2.

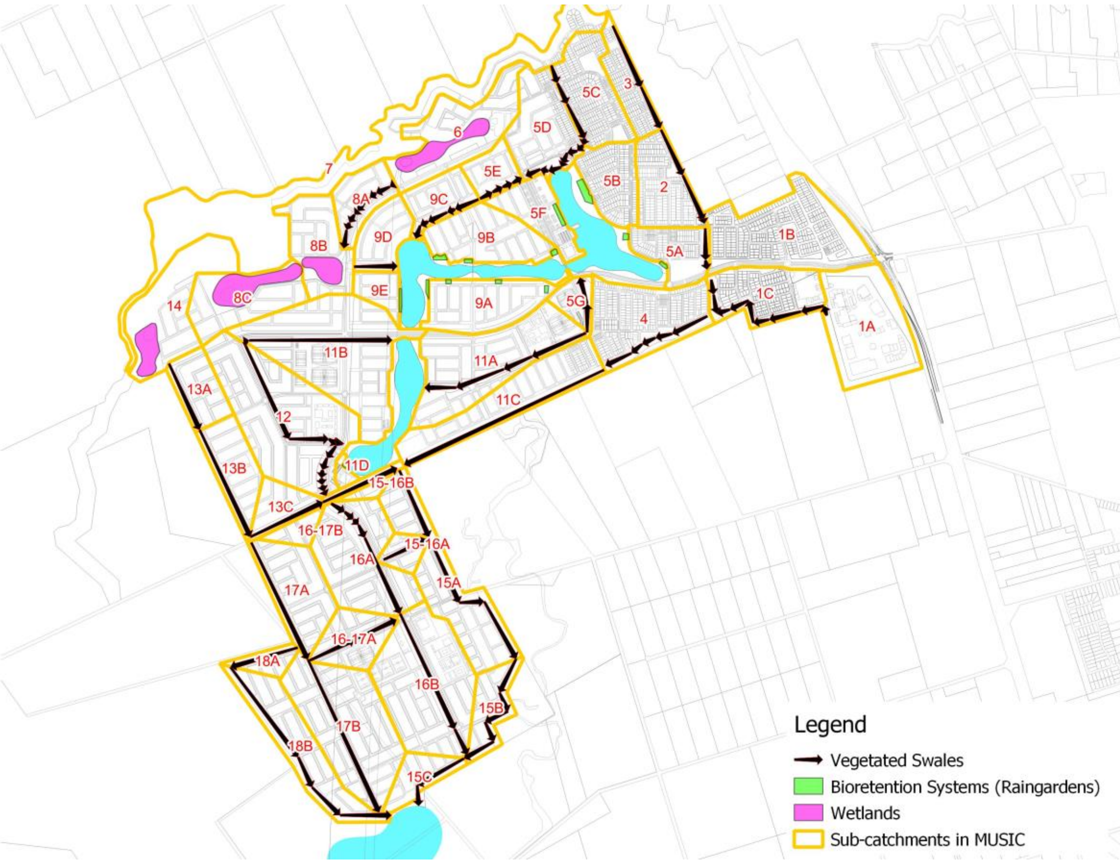

Figure 2 : MUSIC Model Catchment Plan and WSUD Assets Locations

1.2.1 Treatment Assets

The following stormwater treatment assets are considered in the revised master plan based on the site layout, constraints, and opportunities.

Gross Pollutant Traps (GPTs)

It is proposed to incorporate a Gross Pollutant Trap (GPTs) at each major outlet into the vegetated swale or the regional channels These GPTs are to provide an effective means of removing debris and coarse sediments before discharging into the downstream system. GPT's form the firm line of defence to intercept primary gross pollutants. A high performing GPT using CDS technology has been adopted throughout the development that achieves a high level of pollutant trapping performance.

Vegetated Swales/Regional Channels

A system of regional channels has been proposed throughout the Riverlea Park Development in order to manage and convey breakout flows from Gawler River for long duration flooding events in addition to managing stormwater outflows from the development during short duration events. The regional channel network will protect the development from flooding both regional and localised flood events. The basis on which the channels were designed and are based on the flood modelling undertaken by Water Technology (formerly Australian Water Environments).

The basis of this strategy follows those that have been approved and implemented in the Precinct 1's Stormwater Management Plan by WGA (2022). Therefore, this has been adopted as the base design for the entire regional channels across the development. The proposed regional drainage channels include a series of online ephemeral wetland pools integrated into the low flow channels. These pools are densely vegetated shallow water bodies with 200 to 300 mm depth that provide treatment of urban stormwater from the development. Their treatment function provides enhanced sedimentation, fine filtration, adhesion and biological uptake, and chemical processes to remove pollutants from urban stormwater. Given that the channel network is quite long in length, this provides extensive opportunity for stormwater to be treated. The details of these ephemeral wetland treatment pools are described in "Riverlea Development – Stages 1 to 12: Stormwater Management Plan (WGA, 2022)".

Wetlands

The three wetland systems (EW1, EW2 and EW3) are proposed at the northern section of the Development along Gawler River where greater open space areas are available. These wetlands are to treat stormwater from their local internal catchments before discharging into Gawler River at the northwest corner.

The wetland areas were set out to be the same as the indicative areas in the master plan. The permanent pool volumes of the wetlands were estimated based on an average depth of 0.3m under the NWL of the wetlands, considering the varying depths across the macrophyte zones and the open water zones.

These wetlands also act as flood zones for breakout flood flow from the Gawler River to contain the flood within the open space along the northern boundary. Therefore, the areas of the wetlands were not optimised just to meet the treatment targets.

Bioretention

Bioretention systems are proposed for the local catchments which drains into the SWL without the treatment from the vegetated swales. The densely planted bioretention systems at the downstream of the catchments will treat the stormwater runoffs from the local catchments before discharging into the SWLs.

In this Master Plan level assessment, the filter areas of the bioretention systems are sized for 2% of their contributing catchments. These bioretention systems are typically full depth with entire system perimeter fully lined with an impermeable material.

For the vegetation types in the bioretention, it was tested to model with both "Vegetated with effective nutrient removal plants" and "Vegetated with ineffective nutrient removal plants". It was found that some catchments will require effective nutrient removal plants to meet the required treatment criteria. If this cannot be met, larger areas of bioretention will be required to treat the stormwater to the treatment criteria.

1.3 MUSIC Model Setup

The assessment of the water quality uses performed using the industry accepted modelling software MUSIC (Version 6.3) to demonstrate compliance with pollutant reduction targets in accordance with South Australian MUSIC Guidelines (2021).

The parameters entered into MUSIC model for the source and treatment nodes are summarised in Table 1.Table 1 The table provides a general overview of the typical parameters used for the source and treatment nodes. In this case, it is noted that some parameters are stated as being "varied", this is due to the viable dimensional associated with the different areas within the development. The MUSIC model therefore adopts the actual dimension. The source nodes are represented by "urban nodes", and the treatment nodes are represented by GPTs, vegetated swales, wetlands and bioretention. Figure 3 shows the MUSIC model schematic developed based on the treatment catchment plan and the parameters.

Table 1: MUSIC Parameters

MUSIC INPUT PARAMETER	UNITS	VALUE	NOTES	REFERENCE
Rainfall Time Step	Minutes	6	–	South Australian MUSIC Guidelines 2021
Rainfall Template	31 Year Period	Edinburgh RAAF		
Catchment Characteristics (Source Nodes)				
Source Node Type - Urban (Mixed)	%	–	Fraction impervious values vary from nodes to nodes	
Soil Parameters (Residential areas)				
Soil Storage Capacity	mm	40		South Australian MUSIC Guidelines 2021
Initial Storage (% of capacity)	%	25	–	MUSIC Default value
Field Store Capacity	mm	30		South Australian MUSIC Guidelines 2021
Pollutant Concentration Data (Residential areas)				
TSS Mean Storm Flow Concentration	log mg/L	1	Lumped Catchments	South Australian MUSIC Guidelines 2021, Table 4.10
TSS SD Storm Flow Concentration	log mg/L	0.34	Lumped Catchments	South Australian MUSIC Guidelines 2021, Table 4.10
TP Mean Storm Flow Concentration	log mg/L	-0.97	Lumped Catchments	South Australian MUSIC Guidelines 2021, Table 4.10
TP SD Storm Flow Concentration	log mg/L	0.31	Lumped Catchments	South Australian MUSIC Guidelines 2021, Table 4.10
Pollutant Concentration Data				
TN Mean Storm Flow Concentration	log mg/L	0.2	–	South Australian MUSIC Guidelines 2021, Table 4.10
TN SD Storm Flow Concentration	log mg/L	0.2	–	South Australian MUSIC Guidelines 2021, Table 4.10
Serial Correlation For TSS, TP, TN	R Squared	0	–	South Australian MUSIC Guidelines 2021, Table 4.10
Estimation Method	–	Stochastically Generated	–	South Australian MUSIC Guidelines 2021, Table 4.10
Bioretention Design Inputs				
High Flow By-pass	m^3/s	100	–	
Extended Detention Depth	m	0.2	–	
Surface Area	m^2	Varied	–	
Filter Area	m^2	Varied	(Sized up to 2% of catchment)	
Unlined Filter media Perimeter	m	0	–	
Saturated Hydraulic Conductivity	mm/hr	100	100-200 mm is preferred	MUSIC v6 Documentation and Help
Filter Depth	m	0.4	–	
TN Content of Filter Media	mg/kg	800	–	
Exfiltration Rate	mm/hr	0	–	
Vegetated Swale Design Inputs				
Length	m	Varied	–	
Bed Slope	%	Varied	–	
Base Width	m	Varied	–	
Top Width	m	Varied	–	

MUSIC INPUT PARAMETER	UNITS	VALUE	NOTES	REFERENCE
Depth	%	Varied		
Vegetation Height	m	0.25		
Exfiltration Rate	mm/hr	0.7		
Wetland Design Inputs				
High Flow Bypass	m^3/s	0		
Surface Area	m^2	Varied	As from Master Plan	
Extended Detention Depth	m	0.35		
Permanent Pool Volume	m^3	Varied	Calculated with average depth of 0.3m	
Exfiltration Rate	mm/hr	0		
Evaporation Loss	%	125		
Outlet Equivalent Pipe Diameter	mm	Varied	Sized to achieve 72-hour notional detention time	
Gross Pollutant Traps				
High Flow By-pass	m^3/s	Varied	Sized for treatment up to 3-month ARI	
Gross Pollutants Inputs & Outputs Concentration	%	90		
Total Suspended Solids Inputs & Outputs Concentration	%	70		
Total Phosphorus Inputs & Outputs Concentration	%	0		
Total Nitrogen Inputs & Outputs Concentration	%	0		

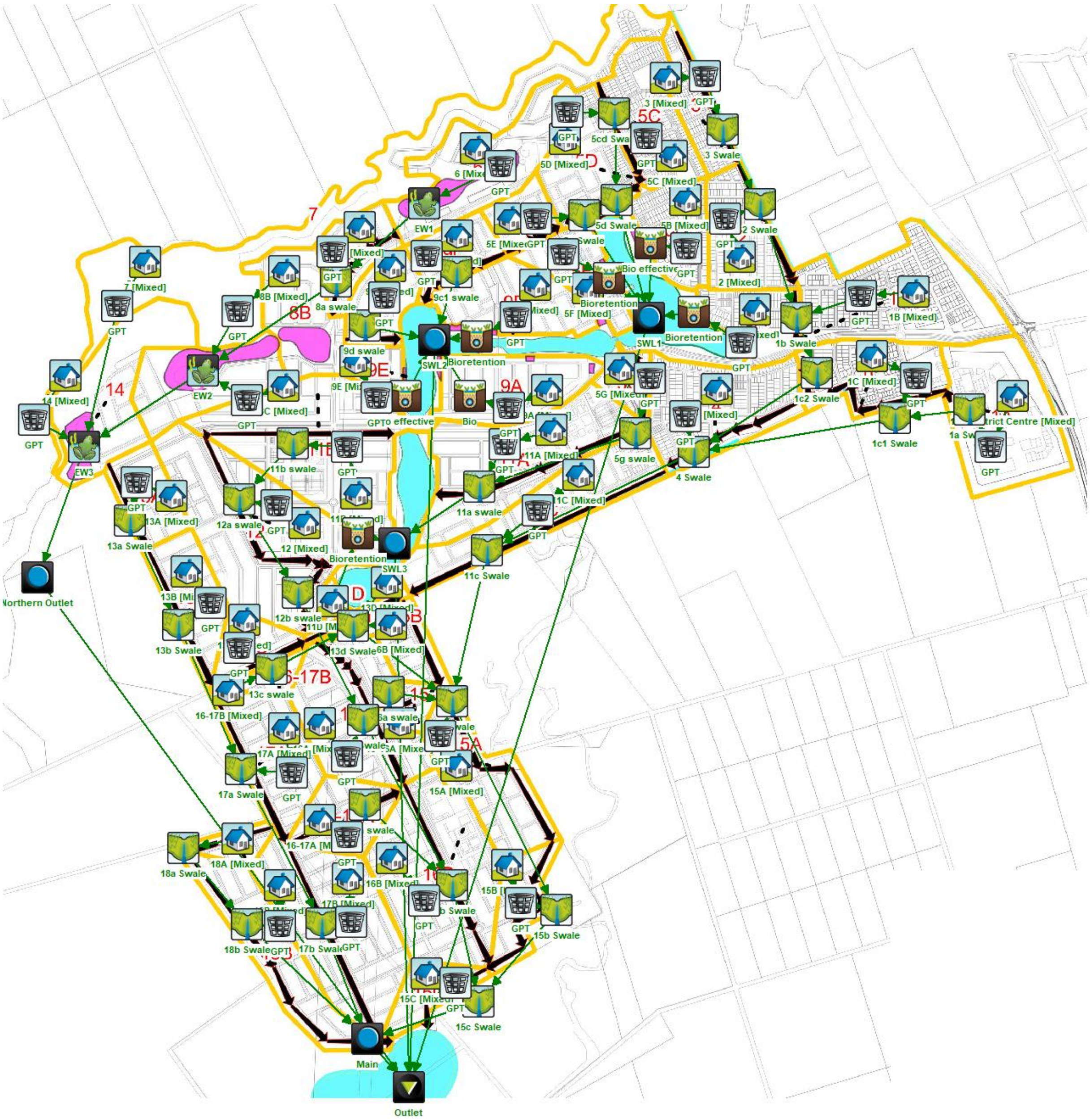

Figure 3 : MUSIC Model Schematic

1.4 Stormwater Treatment Performance Results

The stormwater treatment performance results at the northern outlet from the wetlands, and at the three SWLs and at the main southern outlet are summarised and compared with the required performance criteria in Table 2.

It is also noted that the areas of the wetlands were not assessed to achieve the optimal treatment targets as the wetland areas are also intended as flood zones for the Gawler River flooding.

The results indicate that the overall stormwater treatment systems across the site will comply with the treatment criteria, in addition to meeting all the treatment criteria at each individual outlet.

Table 2: Water Quality Results Compared to Best Practice Standards

POLLUTANT TYPE	TSS	TP	TN	GROSS POLLUTANTS/LITTER
Target percentage reduction (%)	80	60	45	>50 mm and retention in 3-month ARI
Reduction achieved at SWL1 (%)	95.9	73.9	57.4	100% trapped (averaged over the simulated period)
Reduction achieved at SWL2 (%)	96.1	70.5	57.4	100% trapped (averaged over the simulated period)
Reduction achieved at SWL3 (%)	96.8	83.2	66.0	100% trapped (averaged over the simulated period)
Reduction achieved at Northern Outlet (%)	100	100	100	93.4% trapped (averaged over the simulated period)
Reduction achieved at Southern Outlet (%)	97.2	86.8	69.7	100% trapped (averaged over the simulated period)
Reduction achieved at Site Overall (%)	97.3	85.4	70.9	99.2% trapped (averaged over the simulated period)

1.5 Summary

This Master Plan level assessment of the stormwater treatment strategy for Riverlea Estate indicated that stormwater quality discharging from the estate will meet the treatment performance targets as defined in EPA water quality policy and Greater Adelaide Region's WSUD pollutant reduction targets. In addition, it was shown that the proposed treatment strategy also achieves the stormwater treatment targets suitable for discharging into the proposed SWL to not impact the water quality within the lakes.

2 REFERENCES

BMT, 2021. Riverlea Concept Stormwater Quality Management Plan.

WGA, 2022. Riverlea Development – Stages 1 to 12: Stormwater management Plan.

Wallbridge & Gilbert (W&G), March 2009. Riverlea Park Proposal: Stormwater Management: Water, Wastewater and Recycled Water – Technical Paper.

Water Sensitive South Australia, 2021. South Australian MUSIC Guidelines, Adelaide, South Australia.

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Appendix B

Stantec Report – Riverlea – Precinct 2 Land Division Masterplan – Traffic Assessment

Riverlea – Precinct 2

Land Division Masterplan

Traffic Assessment

12/04/2023 - V03 **Ref: 301401258**

PREPARED FOR: Walker Buckland Park

Stantec

Quality Record

Issue	Date	Description	Prepared By	Checked By	Approved By	Signed
1	11/10/22	Final	Paul Morris	Tim Jones	Paul Morris	PSM
2	01/11/22	Minor update to dwelling numbers in Table 4.1. Change to intersection controls in Figure 5.1.	Paul Morris	Tim Jones	Paul Morris	PSM
3	12/4/23	Updated based on Council & DIT comments and revised cross sections	Paul Morris	Tim Jones	Paul Morris	

Contents

1. Introduction

1.1 Background

Riverlea is a major development which will form a new township in the northern area of greater Adelaide. The township will provide approximately 12,000 dwellings, a district centre, neighbourhood centres, educational facilities, mixed use precincts and recreation precincts to cater for 33,000 residents. The development will be undertaken over 20 years.

Key to the development is the street and road network which will provide access for the daily services and needs of the community. A master plan has been prepared for the whole township, however revisions are proposed to Precincts 1 and 2 to commence creation of the township.

Precinct 2 was included in the master plan however it is proposed to revise the layout to integrate better with Precinct 1, which has provided the initial neighbourhood centre, key road network to Port Wakefield Road and associated residential development.

1.2 Purpose of this Report

This report sets out an assessment of the anticipated traffic and transport implications of the proposed development in Precinct 2, including consideration of the:

- existing and estimated traffic conditions surrounding the site;
- traffic generation characteristics of the proposed development;
- proposed access arrangements for the site;
- overview of the layout based on the master plan for Precinct 2;
- transport impact of the development proposal on the surrounding township road network.

1.3 References

In preparing this report, reference has been made to a number of background documents, including:

- Masterplan for the proposed development provided by Walker Corp (dated 4th June 2013)
- Precinct 2 masterplan provided by Walker Corp (August 2022)
- '*Buckland Park Traffic Impact Assessment'* Parsons Brinckerhoff Australia Pty Ltd, 1 April 2009
- Riverlea Precinct 2 Traffic Assessment, GTA Consultants, 2015
- various technical data as referenced in this report
- other documents as nominated.

2. Existing Conditions

The subject site is located within the Riverlea site, which is located adjacent Port Wakefield Road opposite Angle Vale Road. The location of the site can be seen in Figure 2.1.

Figure 2.1: Site and Surrounding Environs

(Photomap courtesy of Walker Corp)

3. Development Proposal

The revised Precinct 2 development is proposed to comprise approximately 3,000 dwellings comprising low and medium density.   A neighbourhood centre, school and sports facility will be included within the site.

Vehicle access to Precinct 2 will be from Riverlea Boulevard which has been constructed thorough Precinct 1 to Port Wakefield Road. The proposed road network will connect to Riverlea Boulevard with various types of intersections to manage the anticipated traffic demands.

The revised precinct road network will comprise distributor, collector and local access roads, and some laneways.

The proposed site layout can be seen in Figure 3.1

Figure 3.1: Revised Precinct 2 Layout (and key intersections)

4. Traffic Assessment

4.1 Previous Assessment

The traffic assessment for the previously approved Riverlea township was undertaken by Parsons Brinkerhoff using a strategic transport model. The assessment was undertaken on the site master plan and did not consider individual precincts. However, the traffic assessment did include traffic generation of the master plan at 5-year intervals based on the anticipated dwelling occupancy.

Precinct 1 has since commenced with traffic management constructed on Riverlea Boulevard including traffic signals for Port Wakefield Road/Angle Vale Road intersection upgrade, and a roundabout at the Guilding Terrace/Riverlea Boulevard intersection.

4.2 Traffic Generation

4.2.1 Design Rates

Based on experience with other land divisions in greater Adelaide, a traffic generation rate of 8 trips per dwelling per day, and 0.85 trips per dwelling per hour (peak hour) as an average across all dwellings provides a robust method of traffic demand estimation. It is noted that in the City of Playford, 76.4% of people travelled to work in a private car, 3.3% took public transport and 1.2% rode a bike or walked. 5.4% worked at home (extract from census 2021 data). Hence car use in the City of Playford is higher than the greater Adelaide average.

As such, this rate has been applied for this assessment which is based on traffic generation of each stage in the precinct ad distribution across the road network in Precinct 2 and connecting to Precinct 1.

It has been assumed the neighbourhood centre will attract traffic from the residents within Riverlea with negligible passing trade from along Port Wakefield Road. Estimates of peak hour and daily traffic volumes are set out in Table 4.1.

Precinct 2 will provide 3132 dwellings (low and medium density) which will result in 25,056 trips per day and 2,664 trips per hour during the peak hours.

It should be noted that some Precinct 1 stages are included in this assessment as they will contribute to the road network at key intersections assessed in this report. This includes 157 dwellings in Stages 4 and 5 which are part of Precinct 1. These are shown in Table 4.2.

The Precinct 1 stages will add 1,256 trips per day and 133 trips per hour to the road network as part of this analysis.

It is noted that whilst the base traffic generation rate has been updated, the traffic generation is consistent with the Traffic Impact Assessment for Buckland Park (2009), and the 2015 Precinct 2 assessment by GTA Consultants, with regards to the anticipated traffic demands of the precinct.

Rates provided within the RTA Guide suggest the neighbourhood centre of 5,550 sq.m total floor area will typically attract 6,750 vehicle trips per day (Thursday). The proposed school is likely to have an attendance of up to 1,000 students. Traffic generation rates for schools as surveyed by GTA indicate a trip generation of 1.34 trips per student per day. Application of this rate suggests the proposed school is likely to attract 1,340 trips per day.

As previously mentioned, the traffic associated with the proposed school and neighbourhood centre are anticipated to be associated with Precinct 2 and not “passing trade” from along Port Wakefield Road. Hence it can be assumed that approximately 30% of all traffic generated by Precinct 2 will be internal to the Precinct 2 site.

Table 4.1: Traffic Generation for Precinct 2

Stage	Detached	Apartments	Total Dwellings	Daily Trips	Peak Hour Trips
8	91		91	728	77
10	90		90	720	77
11	122		122	976	104
12	123		123	984	105
14	143		143	1144	122
15	157		157	1256	133
16	99		99	792	84
17	99	175	274	2192	233
18	92		92	736	78
19	85		85	680	72
20	94		94	752	80
21	121		121	968	103
22A	115		115	920	98
22	110	105	215	1720	183
23	107	35	142	1136	121
24	87		87	696	74
25	111		111	888	94
26	94		94	752	80
27	143		143	1144	122
36	152	35	187	1496	159
37	95		95	760	81
38	101		101	808	86
39	135		135	1080	115
40	105		105	840	89
41	111		111	888	94
		TOTAL	3132	25056	2664

Table 4.2: Precinct 1 Stages adjacent Precinct 2

Stage	Detached	Apartments	Total Dwellings	Daily Trips	Peak Hour Trips
4	86		86	688	73
5	71		71	568	60
		TOTAL	157	1256	133

4.2.2 Distribution and Assignment

The directional distribution and assignment of traffic generated by the proposed development will be influenced by a number of factors, including the:

- configuration of the distributor road network in the immediate vicinity of the site;
- existing operation of intersections providing access between the local, collector and distributor road network;
- surrounding employment centres, retail centres and schools in relation to the site;
- configuration of access points to the site.

Having consideration to the above, it has assumed that 30% of all trips generated will be internal and the remaining 70% will be external to the Riverlea site (that is to and from Port Wakefield Road and Angle Vale Road.

Based on the above, Figure 4.1 and Figure 4.2 indicate the predicted traffic volumes for daily and peak hour periods expected on the road network around Riverlea Boulevard. These volumes have been developed to assist in assessing the proposed intersections for appropriate layouts.

Figure 4.1: Predicted Daily Traffic Volumes

Figure 4.2: Predicted Peak Hour Traffic Volumes

In addition, the directional splits of traffic (i.e. the ratio between the inbound and outbound traffic movements) in the AM and PM peak periods are 90:10 (90% outbound 10% inbound) and 40:60 (40% outbound and 60% inbound) respectively for the external trips.

These AM directional splits have been assumed based on the majority of residential traffic likely to be leaving while the PM directional splits have been assumed based on some residents leaving for dinner or other commitments external to the development while the inbound traffic is residents returning from work.

The internal trip directional splits are assumed to be 50:50 during both peak periods. This internal traffic is likely to be more even with AM directional splits likely to be associated with student drop off and PM directional split likely to be a result of customers at the neighbourhood centre.

The traffic volumes are consistent with the Traffic Impact Assessment (2015) for the traffic demands for Precinct 2 on the distributor road network in Riverlea.

4.2.3 Future Traffic Demands – Ultimate Scenario

As the Riverlea development progresses to the west, there will be additional traffic demands on Riverlea Boulevard. The anticipated traffic volumes will be dependant on the future land uses to the west including additional neighbourhood centres, schools, and employment areas that define an areas level of self-sufficiency (that is ability to remain within that area for daily needs) and reduce external trips. As Riverlea develops further west, the level of self-sufficiency is expected to increase and reduce rate of growth of traffic on Riverlea Boulevard.

For the purposes of this assessment, the same anticipated traffic demands from the west as applied in the 2015 assessment will be used. These were based on the traffic volumes for the ultimate Riverlea site as determined by '*Buckland Park Traffic Impact Assessment*' (Parsons Brinckerhoff Australia Pty Ltd, 1 April 2009). This will provide consistency across assessments.

The additional traffic generation for the analysis from additional development to the west is expressed as additional trips per hour on Riverlea Boulevard for eastbound and westbound flows. These will be added to the Precinct 2 generated Riverlea Boulevard traffic volumes to identify future traffic volumes. These are shown below in Table 4.3**.**

Table 4.3: Ultimate Riverlea Development Additional Traffic

Riverlea Boulevard Direction Flow	Peak - Trips per hour	
	AM	**PM**
Eastbound	+1,248	+534
Westbound	+345	+1,156

*Note: Additional traffic volumes determined by '*Buckland Park Traffic Impact Assessment*' (Parsons Brinckerhoff Australia Pty Ltd, 1 April 2009) as used in the previous Precinct 2 assessment dated 2015

As development occurs to the west, it would be expected that traffic assessments will be revised for intersection on Riverlea Boulevard, as well as monitoring of traffic volumes to ascertain operating conditions actually occurring.

4.3 Traffic Impact

The impact of Precinct 2 traffic on the road network intersections is considered in this section with up to three intersection layout considered as follows:

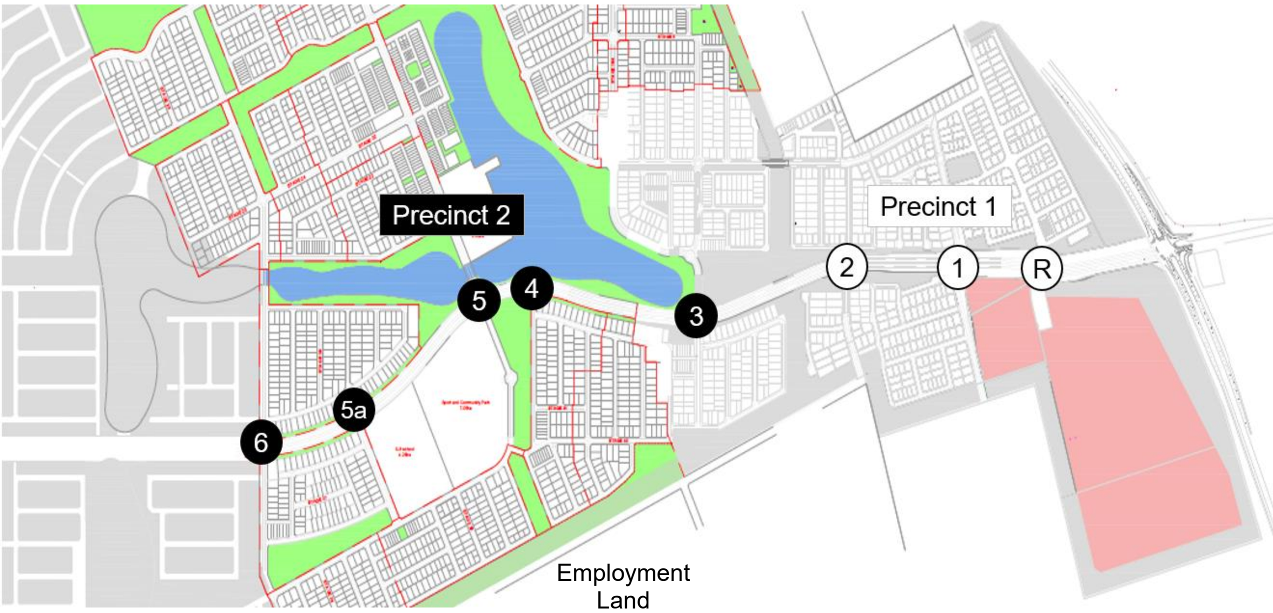
Initial The initial intersection layout proposed for the precinct.

Interim Where applicable, minor upgrades that could be undertaken to maintain the life of the initial intersection.

Ultimate The ultimate layout of the intersection when considering ultimate traffic volumes on Riverlea Boulevard

The impact of the development traffic has been assessed using SIDRA Intersection at key intersections throughout Precinct 2. The key intersection locations are shown in Figure 4.3.

Figure 4.3: Location of Key Intersections



The previous assessment in 2015 included assessment of all intersections from Port Wakefield Road to Precinct 2. Given Precinct 1 has commenced with construction of some intersections, this assessment will only consider the intersections within Precinct 2. A summary of the intersections from previous assessments and new intersections are shown below. The same numbering system has been used to ensure consistency.

Previous key intersections (not part of this assessment):

Port Wakefield Road/Riverlea Boulevard:

Constructed with Precinct 1 as signalised 4-way intersection. Preliminary analysis of this intersection has indicated it is capable of accommodating up to 4,500 allotments which would cater for Precinct 1 and 2 traffic demands. The growth of proposed District Centre may impact this intersection and should be revised as part of planning for District Centre. Future precincts 3 and 4 impacts on this intersection will need to be considered in conjunction with a secondary access to the development area from the south.

The PB Report (2009) indicated that the initial Riverlea Boulevard intersection at Port Wakefield Road will continue to operate satisfactorily for 11 years of development which would equate to approximately 4,740 allotments. This accords with the preliminary analysis of the intersection.

R Precinct 1: Reedy Road intersection. Currently T-junction with Reedy Road to north. Future upgrade as part of recently approved neighbourhood centre with left-turn access to south side of Riverlea Boulevard. Further consideration of the intersection upgrade required for future District Centre proposed to south of Riverlea Boulevard. No further review as part of this report.

1 Precinct 1: Guilding Terrace intersection with Riverlea Boulevard has been constructed as a 2-lane roundabout. This intersection will operate satisfactorily with capacity beyond Precinct 2. No further review as part of this report.

2 Precinct 1: Proposed T-junction for residential access. No change to configuration from previous.

Precinct 2 Assessment Intersections (part of this assessment)

3 Proposed 4-way intersection with 2-lane roundabout. Previously provided access to Neighbourhood Centre within this precinct.

4 Proposed T-junction for residential access. No change to configuration from previous.

5 Proposed 4-way intersection in Precinct 2 – Provides access to Neighbourhood Centre and School/Sports Grounds.

5a New intersection - Proposed T-junction adjacent school.

6 Proposed T-junction for residential access. End of Precinct 2.

5. Access

The layout of the street network for the proposed development is based on a modified grid layout, with local streets connecting to a number of key collector streets and then to the distributor road. A modified grid can provide advantages to a residential area in managing traffic to low volumes on each street, limiting the ability for rat-running through the area, managing the speed environment and providing convenient access for walking, cycling and public transport through the area. The proposed road configuration is shown in Figure 5.1 which indicates the road hierarchy and traffic management.

Figure 5.1: Proposed Road Hierarchy

Type Width (m)	Reserve width metres	Carriageway	Travel lanes 3.6	Bicycle lanes 1.5	Median	Indented Parking 2.5	Footpath 2	Bus route
Distributor (Riverlea Blvd.)	36.6	Dual	4	✓ (1.8m)	6m (min)	no	✓ (>2m)	✓
Neighbourhood Centre	28.5	Single (split)	2	✓	3.5m	both sides	✓	✓
Collector A	25	Single	2	✓	×	both sides	✓	✓
Collector B	22	Single	2	×	×	both sides	✓	×
Collector C	20	Single	2	✓	×	reserve side only	✓	✓
Collector D	19	Single	2	×	×	both sides	✓	×
Local Esplanade Roads	16	Single	2	×	×	reserve side only	✓ (1.5m)	×
Local Street	16	Single	2	×	×	on street	✓ (1.5m)	×
Local Esplanade Streets	14	Single	2	×	×	on street	✓ (1.5m)	×
Laneway	9	Single	2 x 3.5m	×	×	×	×	×

5.1 Employment Land

It is understood that future employment lands have been identified to the south of Precinct 2 which will connect to this precinct via the road at Intersection 5. The Employment Land will be approximately 46 hectares in size and provide light industrial and business park uses when developed. It is noted that this area will be developed separately to the residential development once a demand has been developed for it's use. No layout or specification for the land uses has been identified for analysis in this report, however the road to Intersection 5 will be capable of supporting access for the site.

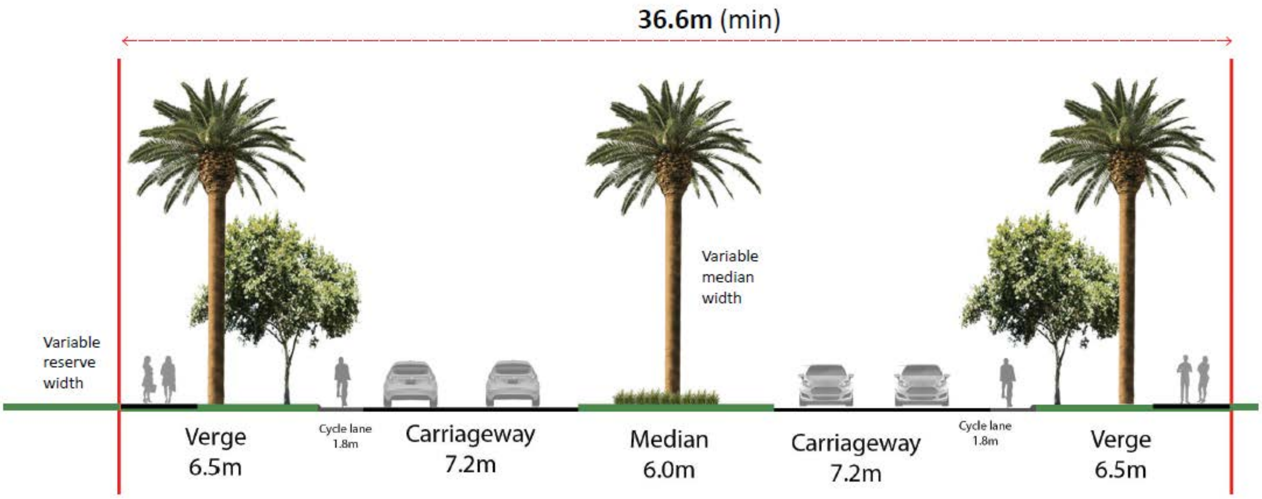
Given the size of the Employment Land site it is appropriate to assume access will be available from Riverlea Boulevard via Intersection 5, and also from Carmelo Road at the southern end of the site. It is assumed that heavy vehicle movements (such as articulated vehicles) would generally access the site from the Carmelo Road access frontage rather than use the Riverlea Boulevard route. Hence the proposed road reserve and cross section for this connection is considered appropriate with Collector A and C cross sections proposed. This would be suitable for access to the employment lands for light vehicles and small heavy vehicles.

The traffic impact of the development of the Employment Lands would be undertaken with any master planning or development applications for the site. This would include analysis of the impact on Riverlea Boulevard and Intersection 5 (and access road) where required.

5.2 Road Cross Sections

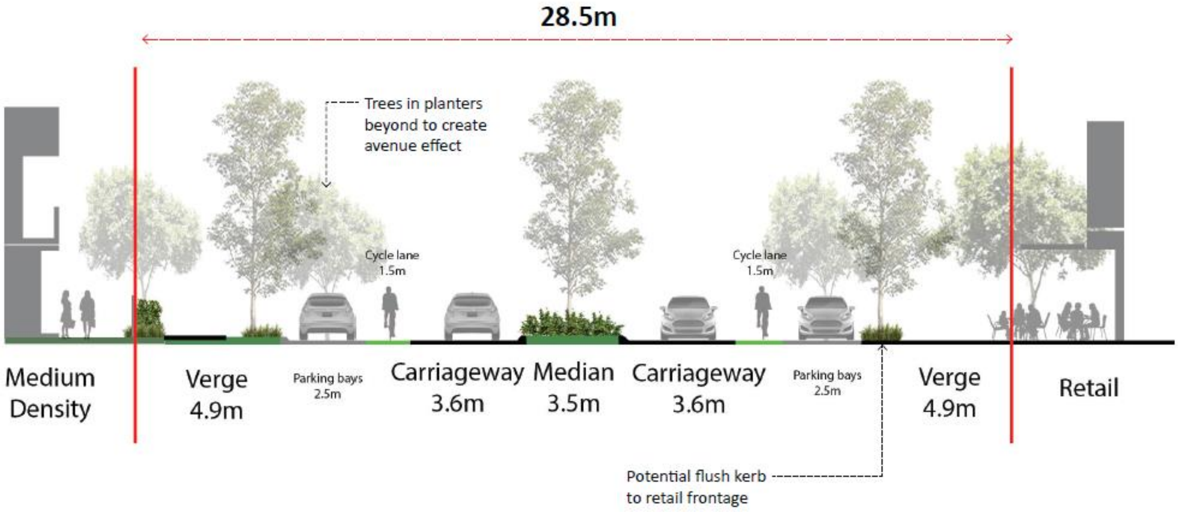
The proposed development will comprise roads of varying widths suited to the function of streets within the network. These align with the proposed street hierarchy as shown in Figure 5.1 previously in this report. Cross sections have been developed in conjunction with the Landscape Plan and are shown in the following figures.

Figure 5.2: Cross Section – Distributor Road



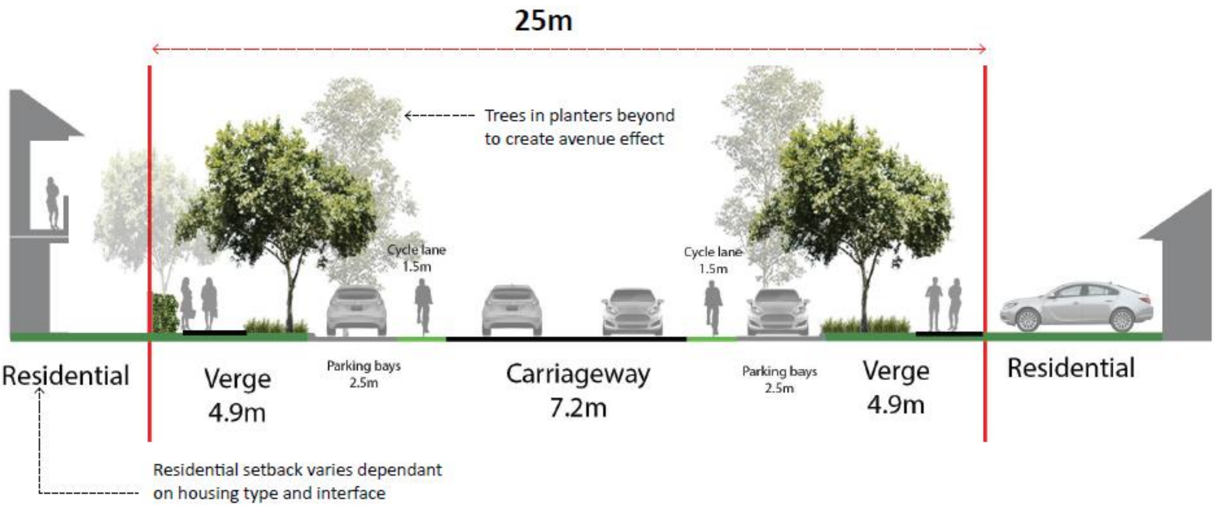
Riverlea Boulevard

Figure 5.3: Cross Section - Neighbourhood Centre Road



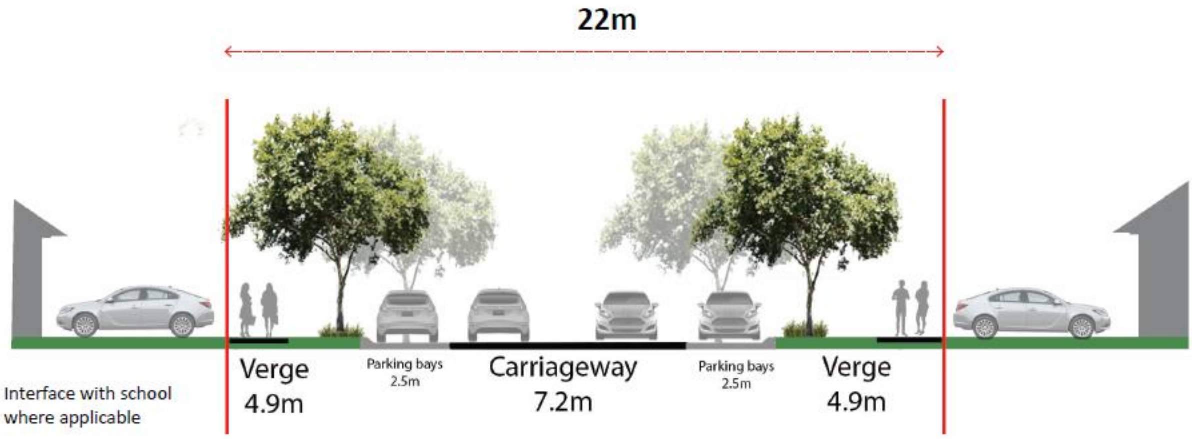
Neighbourhood Centre Retail Avenue

Figure 5.4: Cross Section – Collector Road A



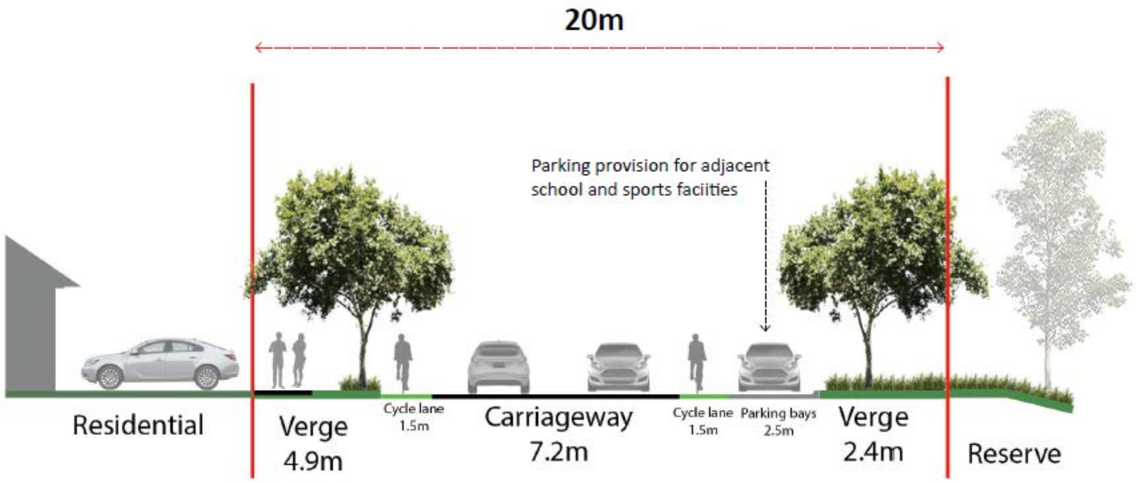
General Residential Collector Road accommodating bus route, cycle lanes and indented car parking

Figure 5.5: Cross Section – Collector Road B



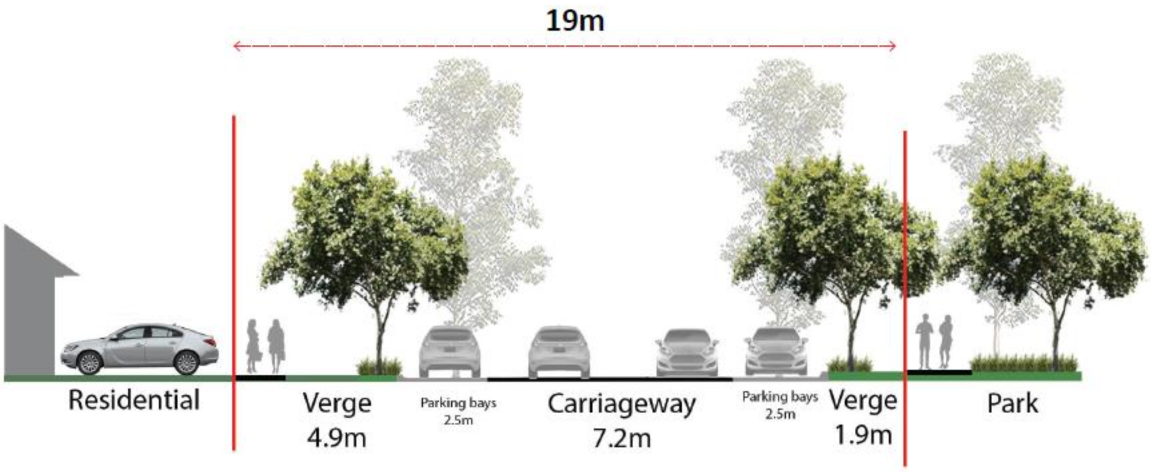
Residential Collector Road accommodating indented car parking and footpaths. Utilised as a 'kiss and ride' school collector road.

Figure 5.6: Cross Section – Collector Road C



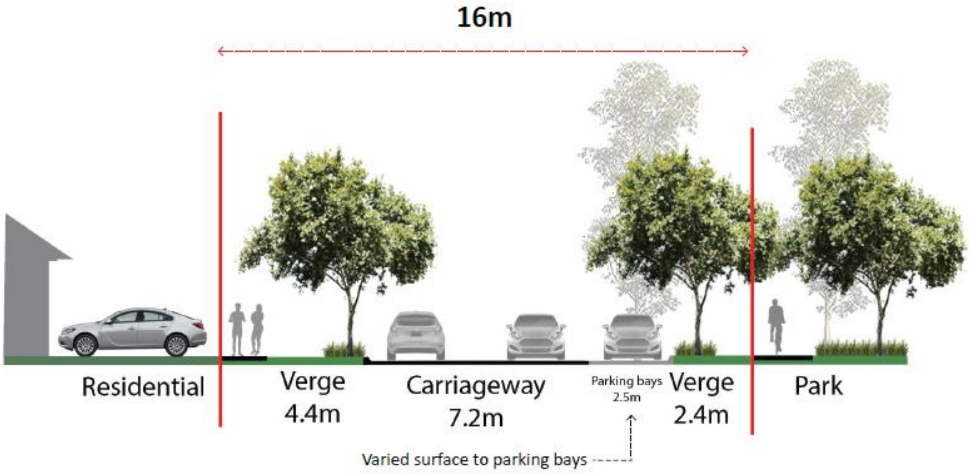
Residential Collector Road alongside drainage reserve where cycle lane connection is required. Includes indented car parking to residential frontages

Figure 5.7: Cross Section – Collector Road D



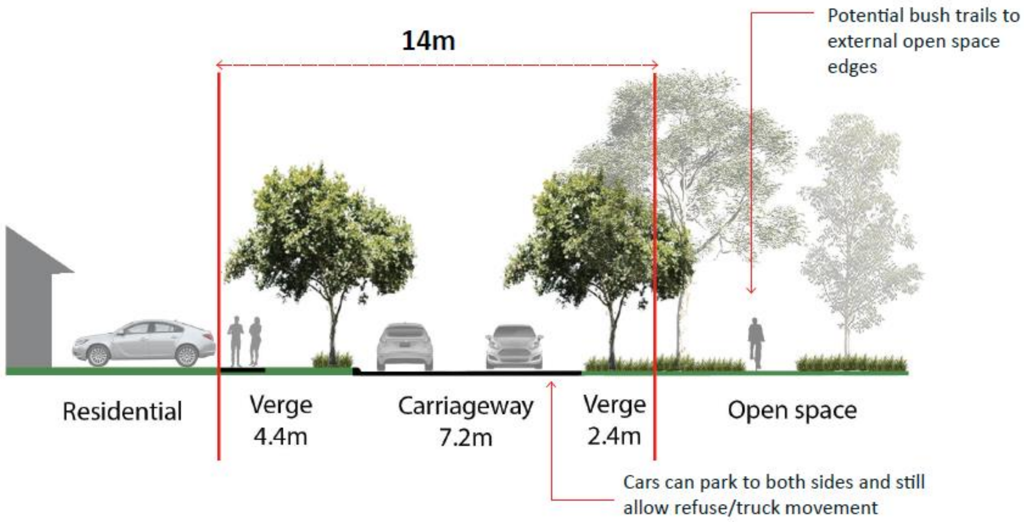
Residential Collector Road alongside major park reserves providing indented car parking bays to both residential and park frontages

Figure 5.8: Local Esplanade Roads (with indented parking)



Local residential roads with optional indented car parking bays to drainage reserve frontage

Figure 5.9: Local Esplanade Roads (on-street parking)



Local residential streets interfacing with open space, external boundaries and reserves, accommodating on-street car parking

Figure 5.10: Local Streets

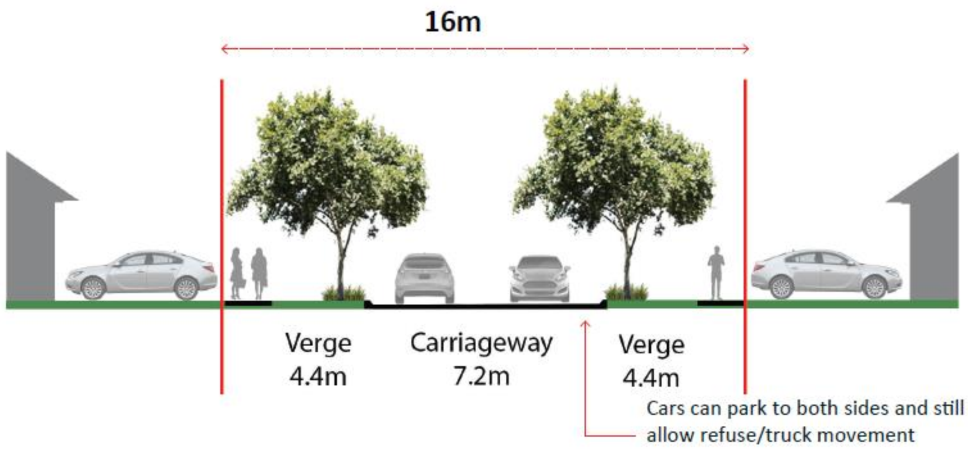


Figure 5.11: Laneways

5.3 T-junctions

The majority of the local street intersections within the proposed development will be controlled by T-Junctions. Realigned T-junctions are proposed at number of locations throughout the development. A realigned T-junction is designed to effect a change in the vehicle travel path thereby slowing traffic via deflection of traffic movements and/or reassignment of priority. These are effective in limiting street lengths and managing speeds on a local road network whilst maintaining a modified grid network. As a result, the safety within the local road network can be improved.

Traffic management measures are required at T-junctions to ensure drivers understand the give-way priority assigned. Generally the right angle bend in conjunction with appropriate kerb alignments will be sufficient however a review in detailed design should consider the following methods to clarify give way priority:

- Give way signs on the minor road approach.
- Pavement marking on the bend for the centreline and parking control.
- Distinctive pavement on the minor road approach.
- Consideration of the radius of bends to ensure suitable turn paths are achieved for the anticipated traffic volumes and vehicle types.

5.4 Roundabouts

A roundabout is an effective form of intersection control and reduces the relative speeds of conflicting vehicles by providing impedance to all vehicles entering the roundabout. A number of roundabout controlled intersections are proposed in Precinct 2, especially where collector roads form four way intersections.

It is recommended that the roundabouts be designed to allow full turning movements for larger vehicles, and in order to cater for semi-trailers a mountable island should be provided. The roundabouts will be required to conform to the relevant standards and guidelines, and the Code, which would be confirmed in detailed design.

5.5 Cul-de-sacs

The development will incorporate circular cul-de-sacs at a number of locations. It is recommended that 18 metre diameter circular cul-de-sacs be provided to enable turning movements by larger vehicles including waste collection vehicles.

5.6 Laneways

Laneways are proposed in a number of locations to provide rear-loaded access to higher density dwellings, for instance row dwellings. The laneways will be wide enough to enable access to garages, provide for rear waste collection.

5.7 Vehicle Speed Management

Austroads Guide to Road Design "*Part 3: Geometric Design*" states a typical acceleration of 1km/h for every 5 metres is possible for private vehicles from a stationary position. Therefore a vehicle can be expected to reach 50km/h (the expected posted speed limit) from a stopped position after 250 metres.

In consideration of the above, roads that provide less than 250 metres of straight sections of road are considered too short for excessive vehicle speeds to occur and act as natural speed control devices. Generally, most streets in the proposed development will be less than 250 metres in length. These streets will generally assist in creating a speed environment of less than 50km/h, and closer to 35km/h where streets are less than 150 metres long.

A number of streets will have a total length greater than 250 metres however, these will be managed with traffic control devices at regular intervals, including intersection treatments such as realigned T-junctions or roundabouts.

Urban design techniques to assist in managing vehicle speeds including tree plantings and house design/driveways, in conjunction with carriageway design techniques will be considered in the context of street design features to manage speeds.

Notwithstanding the above, vehicle speeds within Precinct 2 will be generally managed and can be confirmed in design of the built form for the land division.

5.8 Intersection Sight Distance

In order to provide fundamental safety at intersections, adequate sight distances must be provided at each one. There are three categories of sight distances, these are:

- Approach Sight Distance (ASD)
- Safe Intersection Sight Distance (SISD)
- Minimum Gap Sight Distance (MGSD).

A description and review of each of these sight distances for the proposed development is discussed in the following sections.

Approach Sight Distance (ASD)

ASD is the sight distance required for a driver of a vehicle on a minor road approaching an intersection to observe the holding line for the intersection on the ground. The distance is required such that the driver can observe the holding line, react and stop as required.

Based upon the table provided with the Austroads '*Guide to Road Design Part 4a: Signalised and Signalised Intersections*' (2009, henceforth referred to as Austroads Guide) a design speed of 50km/h has an ASD of 55 metres.

Safe Intersection Sight Distance (SISD)

SISD is the sight distance required for a driver of a vehicle on a major road approaching an intersection to observe a vehicle within the intersection. The SISD is required such that if a vehicle has stopped (i.e. stalled) within an intersection the driver of the approach vehicle on the major road will observe the vehicle and be able to react and stop if required.

Based upon the table provided with the Austroads Guide a design speed of 50km/h has an SISD of 97 metres.

Minimum Gap Sight Distance (MGSD)

MGSD is the sight distance required for a driver of a vehicle on a minor road at the intersection to observe vehicles in the conflicting streams. The distance is required such that the vehicle can view approaching vehicles in order to safely commence the desired manoeuvre.

The MGSD is based upon the number of lanes the vehicle is required to cross, the type of manoeuvre that is required.

Austroads Guide requires a road with a design speed of 50km/h has an MGSD of 69 metres for the critical right turn movement on a two lane/two way road.

Sight Distance Summary

An assessment of the above horizontal sight distances indicates the intersections within the proposed development can provide the minimum requirements. A further sight distance assessment is recommended during detailed design to ensure the horizontal and vertical sight distances are met.

5.9 Street Gradients for Vehicles

It is noted that the current site is very flat and roads will generally be designed with appropriate grades for stormwater management, as opposed to achieving compatibility with existing terrain in undulating environments. Hence, grades of streets are not considered to be an issue within the precinct.

5.10 Parking

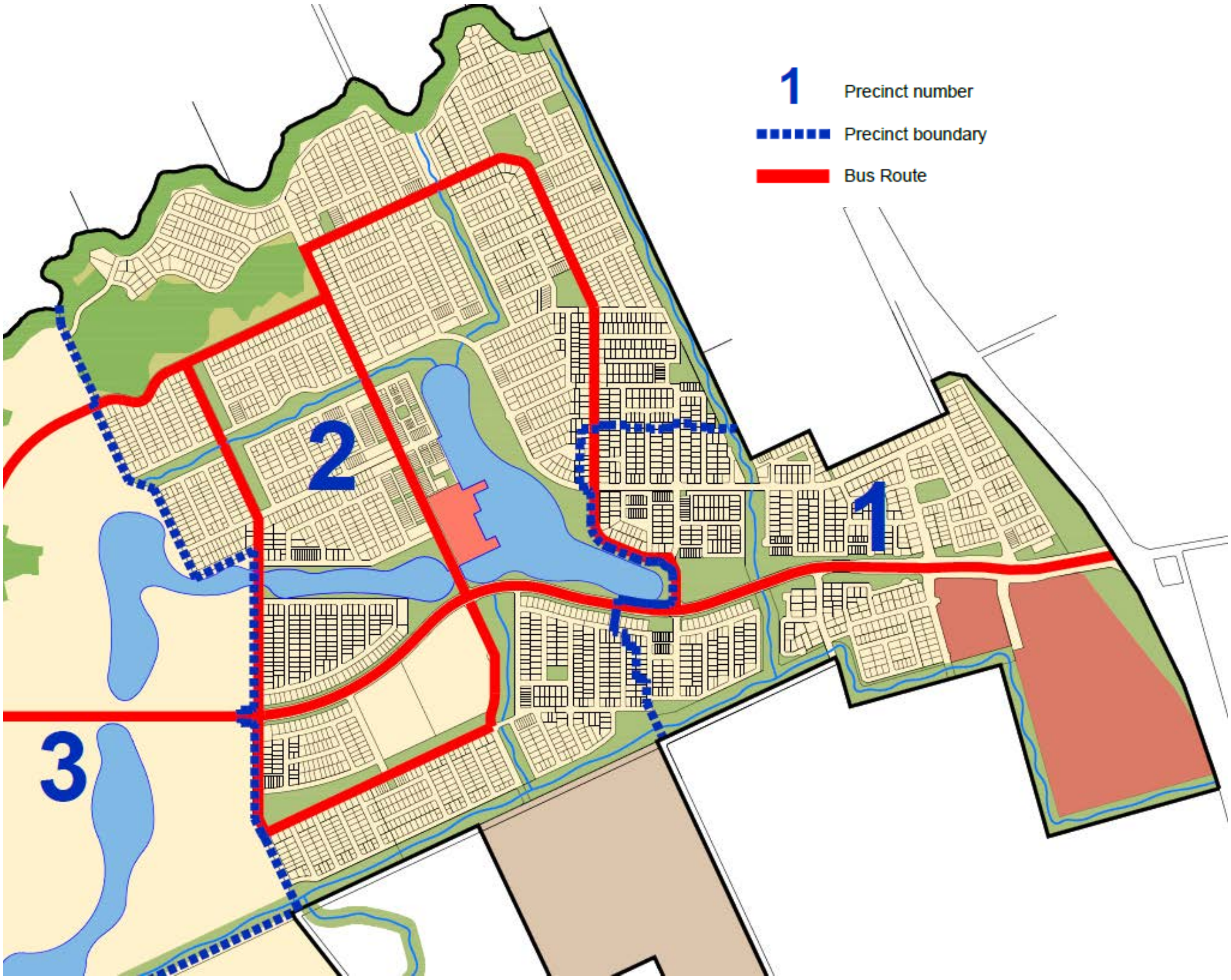
The proposed development will provide a high level of on-street parking which will cater for a minimum of 1 on-street space per 3 dwellings or more based on the proposed road cross sections. These cross sections include a variety on-street parking on the carriageway or indented parking bays.

The frontages of reserves will provide a high level of parking where available. The need for parking at reserves has been considered by an assessment provided in Appendix A.

5.11 Public Transport

Bus routes are proposed to provide public transport access to the Riverlea township. Figure 5.12 indicates the road network to be available for bus services. The actual services will be confirmed on conjunction with agreement from the Department for Infrastructure and Transport. It is envisaged that the proposed bus routes will utilise the distributor and collector roads to provide a bus route that will be within approximately 400 metres of all residential allotments within the Riverlea township.

Figure 5.12: Proposed Bus Routes in Precinct 2



Extract from Walker plan "Overall Bus Routes", 12 April 2023

5.12 Heavy Vehicles

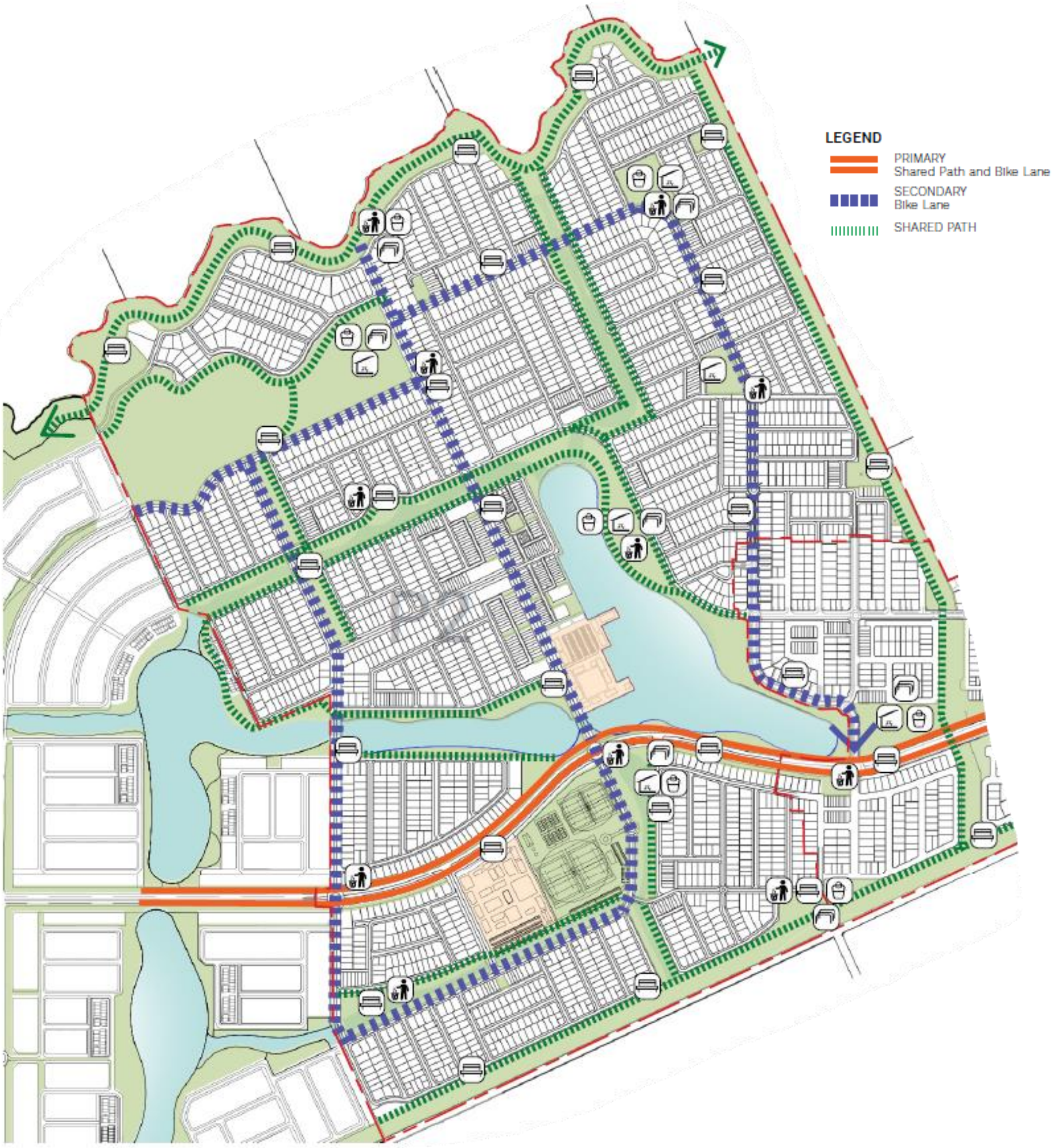
Heavy vehicles will use the proposed road network on an occasional service for waste collection within the proposed residential area. The proposed road network will be capable of providing appropriate access subject to detailed design of intersections and junction to ensure safe and appropriate turning movements are available.

The cul-de-sac streets will enable trucks to turn to enter and exit in a forward direction. The cul-de-sacs should be confirmed in detailed design to ensure adequate space is available.

5.13 Bicycle Access

Bicycle access is proposed with bicycle routes on key collector roads in Precinct 2 as shown in Figure 5.13 where bicycle lanes and/or paths can be considered. These roads will provide key access within and throughout Precinct 2 for bicycles. The low speed design and low volumes on most of the local street network will also facilitate safe bicycle access. The proposed network will provide a high level of accessibility to the neighbourhood centre and school precincts within the site.

Figure 5.13: Proposed Bicycle Routes (extract from Landscape Masterplan)



6. Intersection Assessments

Each intersection has been assessed individually for performance based on anticipated traffic demands. Schematic layouts for each intersection have been prepared to indicate required lane arrangements. Other features such as pedestrian crossings, suitable turn paths for design vehicles and location of traffic signal posts are assumed to be included and to be confirmed in detailed design.

6.1 Intersection 3 Assessment

A roundabout is proposed at this intersection as part of Precinct 1 development (Silverleaf Drive in Stage 4), with 2 lanes for eastbound and westbound traffic on Riverlea Boulevard. A single lane approach for the north and south legs.

The anticipated AM and PM peak hour traffic volumes for Precinct 2 volumes at intersection 3 are shown in Figure 6.1. The Ultimate through volumes on Riverlea Drive are also shown.

Figure 6.1: Intersection 3 – Precinct 2 AM & PM Peak Hour Turning Volumes

Stage 5 Road

		PM	89	5	49			
Riverlea Blvd		AM	89	5	444		Riverlea Blvd	
PM	AM		R	T	L		AM	PM
89	89	L				R	49	444
260	1070	T				T	192	1259
38	38	R				L	3	25
Ultimate			L	T	R			Ultimate
794	2318	AM	38	5	25		537	2415
		PM	38	5	3			

Stage 4 Road

Orange figures indicates the future traffic on Riverlea Boulevard for the Ultimate intersection analysis.

The intersection layouts are shown in Figure 6.2 to Figure 6.4.

Figure 6.2: Intersection 3 – Initial Layout

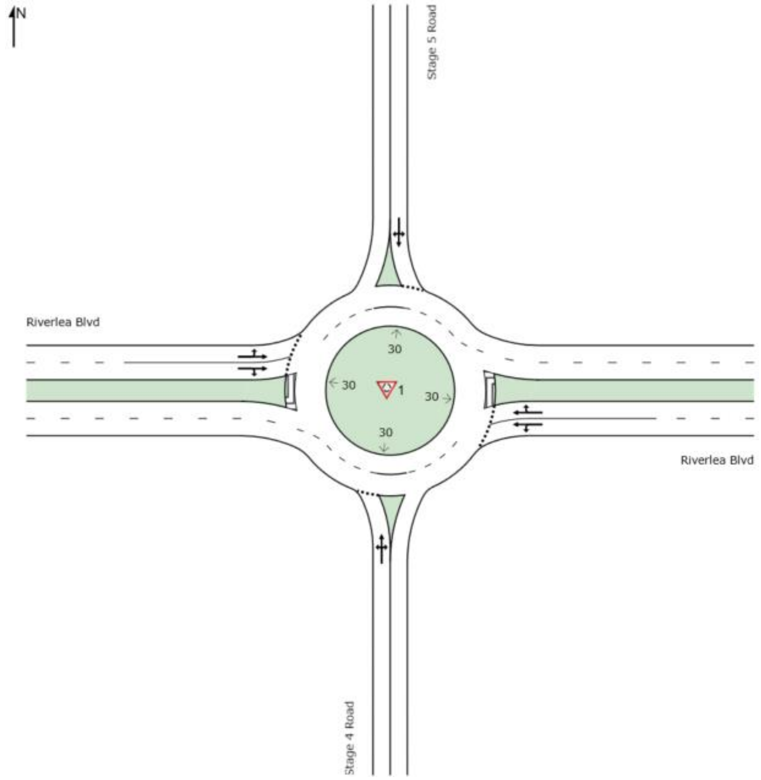


Figure 6.3: Intersection 3 – Interim Upgrade

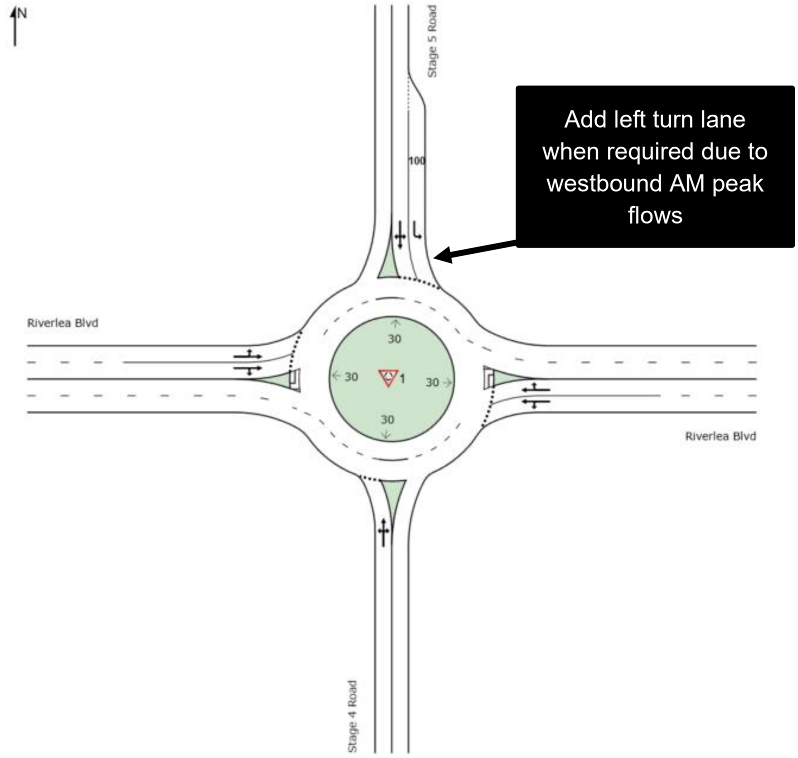
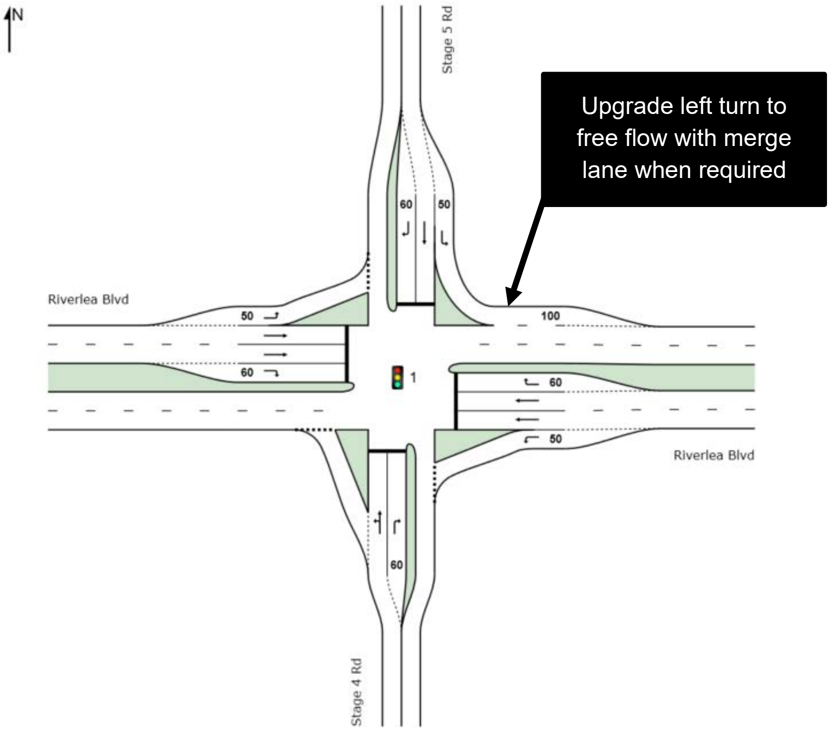


Figure 6.4: Intersection 3 – Ultimate Layout



The movement summary for each of the intersection peak periods are shown in the following tables.

Table 6.1: Intersection 3 – Initial Layout – AM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 4 Road														
1	L2	38	3.0	40	3.0	0.070	3.5	LOS A	0.3	1.9	0.39	0.54	0.39	50.0
2	T1	5	3.0	5	3.0	0.070	3.1	LOS A	0.3	1.9	0.39	0.54	0.39	48.1
3	R2	25	3.0	26	3.0	0.070	8.7	LOS A	0.3	1.9	0.39	0.54	0.39	51.6
Approach		68	3.0	72	3.0	0.070	5.3	LOS A	0.3	1.9	0.39	0.54	0.39	50.4
East: Riverlea Blvd														
4	L2	3	3.0	3	3.0	0.094	4.1	LOS A	0.6	4.0	0.32	0.39	0.32	51.0
5	T1	192	3.0	202	3.0	0.094	3.9	LOS A	0.6	4.0	0.32	0.43	0.32	56.0
6	R2	54	3.0	57	3.0	0.094	9.7	LOS A	0.5	3.8	0.33	0.52	0.33	51.8
Approach		249	3.0	262	3.0	0.094	5.2	LOS A	0.6	4.0	0.32	0.45	0.32	55.0
North: Stage 5 Road														
7	L2	444	3.0	467	3.0	0.802	13.0	LOS A	7.1	50.9	0.87	1.18	1.55	42.5
8	T1	5	3.0	5	3.0	0.802	12.7	LOS A	7.1	50.9	0.87	1.18	1.55	43.5
9	R2	89	3.0	94	3.0	0.802	18.2	LOS B	7.1	50.9	0.87	1.18	1.55	46.4
Approach		538	3.0	566	3.0	0.802	13.9	LOS A	7.1	50.9	0.87	1.18	1.55	43.1
West: Riverlea Blvd														
10	L2	89	3.0	94	3.0	0.416	4.1	LOS A	3.0	21.6	0.30	0.39	0.30	54.8
11	T1	1070	3.0	1126	3.0	0.416	3.9	LOS A	3.0	21.6	0.31	0.40	0.31	56.6
12	R2	38	3.0	40	3.0	0.416	9.7	LOS A	3.0	21.4	0.32	0.41	0.32	53.1
Approach		1197	3.0	1260	3.0	0.416	4.1	LOS A	3.0	21.6	0.31	0.40	0.31	56.3
All Vehicles		2052	3.0	2160	3.0	0.802	6.8	LOS A	7.1	50.9	0.46	0.61	0.64	51.8

Table 6.2: Intersection 3 – Initial Layout – PM Peak Summary

Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
Vehicle Movement Performance														
South: Stage 4 Road														
1	L2	38	3.0	40	3.0	0.103	8.1	LOS A	0.5	3.3	0.77	0.85	0.77	48.4
2	T1	5	3.0	5	3.0	0.103	7.7	LOS A	0.5	3.3	0.77	0.85	0.77	46.5
3	R2	3	3.0	3	3.0	0.103	13.2	LOS A	0.5	3.3	0.77	0.85	0.77	49.9
Approach		46	3.0	48	3.0	0.103	8.4	LOS A	0.5	3.3	0.77	0.85	0.77	48.2
East: Riverlea Blvd														
4	L2	25	3.0	26	3.0	0.629	4.6	LOS A	6.1	44.1	0.50	0.44	0.50	50.1
5	T1	1259	3.0	1325	3.0	0.629	4.5	LOS A	6.1	44.1	0.51	0.48	0.51	55.1
6	R2	444	3.0	467	3.0	0.629	10.4	LOS A	6.1	43.6	0.54	0.58	0.54	50.8
Approach		1728	3.0	1819	3.0	0.629	6.0	LOS A	6.1	44.1	0.52	0.50	0.52	53.8
North: Stage 5 Road														
7	L2	49	3.0	52	3.0	0.149	3.4	LOS A	0.7	4.7	0.43	0.61	0.43	46.0
8	T1	5	3.0	5	3.0	0.149	3.1	LOS A	0.7	4.7	0.43	0.61	0.43	47.2
9	R2	89	3.0	94	3.0	0.149	8.6	LOS A	0.7	4.7	0.43	0.61	0.43	50.6
Approach		143	3.0	151	3.0	0.149	6.7	LOS A	0.7	4.7	0.43	0.61	0.43	48.8
West: Riverlea Blvd														
10	L2	89	3.0	94	3.0	0.193	5.6	LOS A	1.3	9.2	0.62	0.58	0.62	53.5
11	T1	260	3.0	274	3.0	0.193	5.7	LOS A	1.3	9.2	0.62	0.61	0.62	54.7
12	R2	38	3.0	40	3.0	0.193	11.7	LOS A	1.2	8.5	0.63	0.64	0.63	51.2
Approach		387	3.0	407	3.0	0.193	6.3	LOS A	1.3	9.2	0.62	0.60	0.62	54.1
All Vehicles		2304	3.0	2425	3.0	0.629	6.1	LOS A	6.1	44.1	0.54	0.53	0.54	53.4

Table 6.3: Intersection 3 – Interim AM Peak Summary

Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
Vehicle Movement Performance														
South: Stage 4 Road														
1	L2	38	3.0	40	3.0	0.085	4.4	LOS A	0.3	2.3	0.52	0.66	0.52	49.5
2	T1	5	3.0	5	3.0	0.085	4.1	LOS A	0.3	2.3	0.52	0.66	0.52	47.7
3	R2	25	3.0	26	3.0	0.085	9.6	LOS A	0.3	2.3	0.52	0.66	0.52	51.1
Approach		68	3.0	72	3.0	0.085	6.3	LOS A	0.3	2.3	0.52	0.66	0.52	50.0
East: Riverlea Blvd														
4	L2	3	3.0	3	3.0	0.224	4.2	LOS A	1.5	10.8	0.36	0.40	0.36	50.8
5	T1	537	3.0	565	3.0	0.224	4.0	LOS A	1.5	10.8	0.36	0.43	0.36	56.1
6	R2	54	3.0	57	3.0	0.224	9.8	LOS A	1.4	10.4	0.37	0.47	0.37	52.5
Approach		594	3.0	625	3.0	0.224	4.6	LOS A	1.5	10.8	0.36	0.43	0.36	55.7
North: Stage 5 Road														
7	L2	444	3.0	467	3.0	0.897	58.4	LOS E	11.9	85.7	1.00	1.65	2.73	27.9
8	T1	5	3.0	5	3.0	0.897	65.5	LOS E	8.7	62.2	0.99	1.54	2.56	26.7
9	R2	89	3.0	94	3.0	0.897	71.0	LOS F	8.7	62.2	0.99	1.54	2.56	27.8
Approach		538	3.0	566	3.0	0.897	60.6	LOS E	11.9	85.7	0.99	1.63	2.70	27.9
West: Riverlea Blvd														
10	L2	89	3.0	94	3.0	0.840	4.8	LOS A	15.3	110.0	0.62	0.44	0.62	53.2
11	T1	2318	3.0	2440	3.0	0.840	4.8	LOS A	15.3	110.0	0.67	0.46	0.67	54.7
12	R2	38	3.0	40	3.0	0.840	10.7	LOS A	14.4	103.5	0.73	0.48	0.73	51.4
Approach		2445	3.0	2574	3.0	0.840	4.9	LOS A	15.3	110.0	0.67	0.46	0.67	54.6
All Vehicles		3645	3.0	3837	3.0	0.897	13.1	LOS A	15.3	110.0	0.66	0.63	0.91	48.0

Table 6.4: Intersection 3 – Interim PM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 4 Road														
1	L2	38	3.0	40	3.0	0.265	18.8	LOS B	1.5	10.4	0.95	0.97	0.95	42.4
2	T1	5	3.0	5	3.0	0.265	18.5	LOS B	1.5	10.4	0.95	0.97	0.95	41.0
3	R2	3	3.0	3	3.0	0.265	24.0	LOS B	1.5	10.4	0.95	0.97	0.95	43.5
Approach		46	3.0	48	3.0	0.265	19.1	LOS B	1.5	10.4	0.95	0.97	0.95	42.3
East: Riverlea Blvd														
4	L2	25	3.0	26	3.0	0.900	5.8	LOS A	19.1	137.3	0.89	0.56	0.90	48.5
5	T1	2000	3.0	2105	3.0	0.900	6.3	LOS A	20.3	145.4	0.93	0.60	0.96	53.1
6	R2	444	3.0	467	3.0	0.900	13.4	LOS A	20.3	145.4	1.00	0.67	1.07	49.4
Approach		2469	3.0	2599	3.0	0.900	7.6	LOS A	20.3	145.4	0.94	0.61	0.98	52.3
North: Stage 5 Road														
7	L2	49	3.0	52	3.0	0.216	5.0	LOS A	1.0	7.1	0.66	0.81	0.66	45.4
8	T1	5	3.0	5	3.0	0.216	4.7	LOS A	1.0	7.1	0.66	0.81	0.66	46.6
9	R2	89	3.0	94	3.0	0.216	10.2	LOS A	1.0	7.1	0.66	0.81	0.66	49.9
Approach		143	3.0	151	3.0	0.216	8.2	LOS A	1.0	7.1	0.66	0.81	0.66	48.1
West: Riverlea Blvd														
10	L2	89	3.0	94	3.0	0.490	6.2	LOS A	4.3	30.9	0.81	0.62	0.81	52.4
11	T1	794	3.0	836	3.0	0.490	6.4	LOS A	4.3	30.9	0.82	0.67	0.83	53.9
12	R2	38	3.0	40	3.0	0.490	12.6	LOS A	4.1	29.3	0.82	0.72	0.85	50.8
Approach		921	3.0	969	3.0	0.490	6.6	LOS A	4.3	30.9	0.82	0.67	0.83	53.6
All Vehicles		3579	3.0	3767	3.0	0.900	7.5	LOS A	20.3	145.4	0.90	0.64	0.93	52.3

Table 6.5: Intersection 3 – Ultimate AM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 4 Rd														
1	L2	38	3.0	40	3.0	0.072	16.8	LOS B	1.3	9.4	0.45	0.58	0.45	43.3
2	T1	5	3.0	5	3.0	0.072	12.3	LOS A	1.3	9.4	0.45	0.58	0.45	40.9
3	R2	25	3.0	26	3.0	0.189	75.1	LOS F	1.8	13.2	0.96	0.72	0.96	25.6
Approach		68	3.0	72	3.0	0.189	37.9	LOS C	1.8	13.2	0.64	0.63	0.64	34.5
East: Riverlea Blvd														
4	L2	3	3.0	3	3.0	0.002	6.2	LOS A	0.0	0.1	0.12	0.56	0.12	50.1
5	T1	537	3.0	565	3.0	0.196	0.5	LOS A	0.5	3.6	0.03	0.02	0.03	59.5
6	R2	49	3.0	52	3.0	*0.684	88.9	LOS F	4.0	29.1	1.00	0.80	1.15	23.7
Approach		589	3.0	620	3.0	0.684	7.9	LOS A	4.0	29.1	0.11	0.09	0.12	52.8
North: Stage 5 Rd														
7	L2	444	3.0	467	3.0	0.248	25.5	LOS B	0.0	0.0	0.00	0.46	0.00	47.7
8	T1	5	3.0	5	3.0	0.031	67.9	LOS E	0.4	2.5	0.94	0.62	0.94	26.1
9	R2	89	3.0	94	3.0	*0.596	78.2	LOS F	6.9	49.2	1.00	0.79	1.01	25.1
Approach		538	3.0	566	3.0	0.596	34.6	LOS C	6.9	49.2	0.17	0.52	0.18	41.3
West: Riverlea Blvd														
10	L2	89	3.0	94	3.0	0.063	8.6	LOS A	1.4	9.7	0.24	0.60	0.24	51.8
11	T1	2318	3.0	2440	3.0	*0.847	1.1	LOS A	10.2	73.5	0.13	0.12	0.13	58.9
12	R2	38	3.0	40	3.0	0.531	84.8	LOS F	3.0	21.8	1.00	0.73	1.01	24.1
Approach		2445	3.0	2574	3.0	0.847	2.7	LOS A	10.2	73.5	0.15	0.15	0.15	57.3
All Vehicles		3640	3.0	3832	3.0	0.847	8.9	LOS A	10.2	73.5	0.15	0.20	0.16	52.9

Table 6.6: Intersection 3 - Ultimate PM Peak Summary

Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 4 Rd														
1	L2	38	3.0	40	3.0	0.201	6.0	LOS A	0.4	3.0	0.20	0.51	0.20	49.7
2	T1	5	3.0	5	3.0	0.201	1.4	LOS A	0.4	3.0	0.20	0.51	0.20	46.6
3	R2	3	3.0	3	3.0	0.034	79.1	LOS F	0.2	1.6	0.97	0.63	0.97	25.0
Approach		46	3.0	48	3.0	0.201	10.2	LOS A	0.4	3.0	0.25	0.52	0.25	46.4
East: Riverlea Blvd														
4	L2	25	3.0	26	3.0	0.020	6.4	LOS A	0.1	0.5	0.04	0.56	0.04	50.1
5	T1	2415	3.0	2542	3.0	* 0.757	0.5	LOS A	5.3	38.1	0.08	0.07	0.08	59.4
6	R2	444	3.0	467	3.0	0.462	16.0	LOS B	8.1	58.5	0.39	0.74	0.39	44.4
Approach		2884	3.0	3036	3.0	0.757	3.0	LOS A	8.1	58.5	0.13	0.18	0.13	56.4
North: Stage 5 Rd														
7	L2	49	3.0	52	3.0	0.027	5.3	LOS A	0.0	0.0	0.00	0.47	0.00	47.8
8	T1	5	3.0	5	3.0	0.050	74.5	LOS F	0.4	2.7	0.97	0.63	0.97	25.0
9	R2	89	3.0	94	3.0	* 0.882	92.2	LOS F	7.6	54.9	1.00	0.97	1.40	22.9
Approach		143	3.0	151	3.0	0.882	61.8	LOS E	7.6	54.9	0.66	0.79	0.90	28.0
West: Riverlea Blvd														
10	L2	89	3.0	94	3.0	0.093	6.1	LOS A	0.1	1.0	0.04	0.56	0.04	53.8
11	T1	794	3.0	836	3.0	* 0.900	44.2	LOS D	31.2	223.7	0.84	0.82	0.95	34.8
12	R2	38	3.0	40	3.0	0.744	77.6	LOS F	3.2	23.0	0.92	0.91	1.22	25.3
Approach		921	3.0	969	3.0	0.900	41.9	LOS C	31.2	223.7	0.76	0.80	0.88	35.5
All Vehicles		3994	3.0	4204	3.0	0.900	14.1	LOS A	31.2	223.7	0.30	0.35	0.33	48.0

6.1.1 Intersection 3 Analysis Summary

The SIDRA Intersection analysis indicates that the proposed roundabout at Intersection 3 will operate satisfactorily and within capacity for the predicted Precinct 2 traffic volumes.

An analysis of the ultimate traffic volumes has found that the roundabout will continue to operate satisfactorily for the Ultimate AM peak periods based on the addition of a left turn lane on the northern approach.

The roundabout will not, however, be able to accommodate all of the Ultimate PM peak period traffic volumes with significant queueing predicted on the eastern approach of Riverlea Boulevard. Further modelling has found the roundabout will accommodate up to 2000 vehicles per hour westbound on the eastern approach, which equates to about 2/3rds of the Ultimate traffic flow westbound.

Hence, the roundabout should be monitored following further development to the west to determine the timing required for the interim upgrade, and then the Ultimate upgrade to traffic signals.

Traffic signals will be required in the ultimate layout when Riverlea is developed to the west. In particular, a free flowing left turn will be required from Osprey Drive (north leg) to Riverlea Boulevard (east leg) due to the high eastbound flows on Riverlea Boulevard in the AM peak period.

6.2 Intersection 4 Assessment

The anticipated AM and PM peak hour traffic volumes for Precinct 2 volumes at intersection 5 are shown in Figure 6.1.

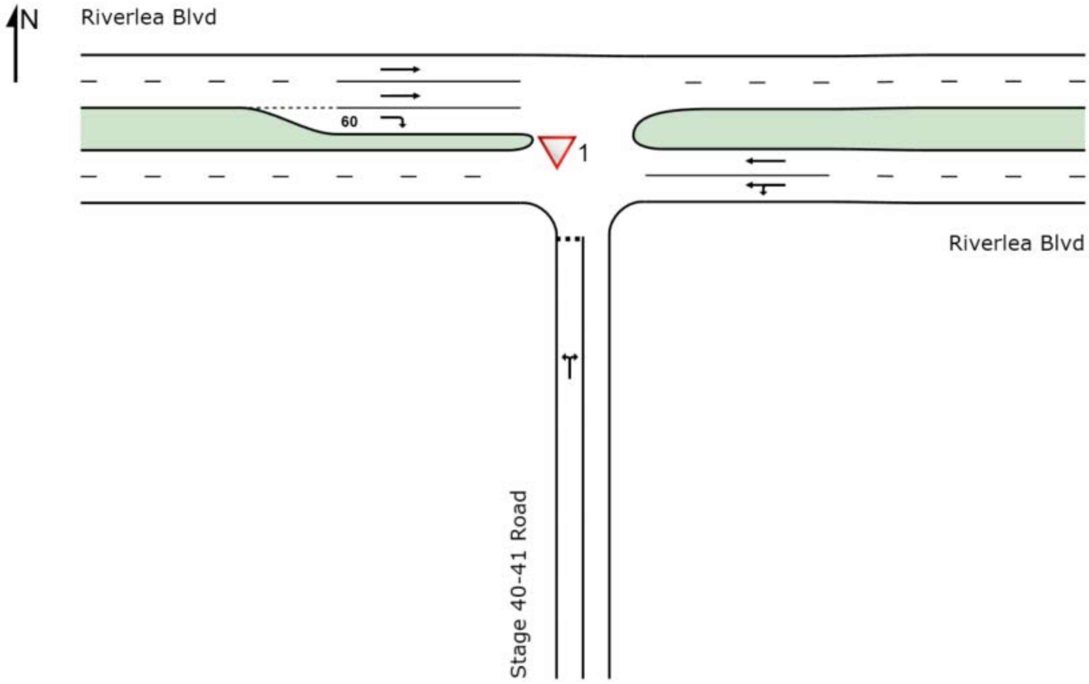
Figure 6.5: Intersection 4 – Precinct 2 AM & PM Peak Hour Turning Volumes

Riverlea Blvd						Riverlea Blvd	
PM	AM					AM	PM
119	1069	T			T	133	1197
14	14	R			L	9	82
Ultimate			L	R			Ultimate
653	2317	AM	14	82		478	2353
		PM	14	9			

Stage 40-41 Road

Orange figures indicates the future traffic on Riverlea Boulevard for the Ultimate intersection analysis.

Figure 6.6: Intersection 4 – T-junction



The movement summary for each intersection and peak period is shown in the following tables.

Table 6.7: Intersection 4 – Initial T-Junction – AM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 40-41 Road														
1	L2	14	3.0	15	3.0	0.223	5.6	LOS A	0.9	6.3	0.64	0.81	0.67	44.6
3	R2	82	3.0	86	3.0	0.223	14.3	LOS A	0.9	6.3	0.64	0.81	0.67	44.4
Approach		96	3.0	101	3.0	0.223	13.0	LOS A	0.9	6.3	0.64	0.81	0.67	44.4
East: Riverlea Blvd														
4	L2	9	3.0	9	3.0	0.039	5.6	LOS A	0.0	0.0	0.00	0.08	0.00	57.5
5	T1	133	3.0	140	3.0	0.039	0.0	LOS A	0.0	0.0	0.00	0.04	0.00	59.7
Approach		142	3.0	149	3.0	0.039	0.4	NA	0.0	0.0	0.00	0.04	0.00	59.5
West: Riverlea Blvd														
11	T1	1069	3.0	1125	3.0	0.294	0.1	LOS A	0.0	0.0	0.00	0.00	0.00	59.8
12	R2	14	3.0	15	3.0	0.009	6.0	LOS A	0.0	0.3	0.25	0.54	0.25	49.0
Approach		1083	3.0	1140	3.0	0.294	0.2	NA	0.0	0.3	0.00	0.01	0.00	59.7
All Vehicles		1321	3.0	1391	3.0	0.294	1.1	NA	0.9	6.3	0.05	0.07	0.05	58.2

Table 6.8: Intersection 4 – Initial T-Junction – PM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 40-41 Road														
1	L2	14	3.0	15	3.0	0.084	23.8	LOS B	0.3	1.8	0.86	0.93	0.86	39.4
3	R2	9	3.0	9	3.0	0.031	16.6	LOS B	0.1	0.8	0.83	0.92	0.83	42.6
Approach		23	3.0	24	3.0	0.084	21.0	LOS B	0.3	1.8	0.85	0.93	0.85	40.6
East: Riverlea Blvd														
4	L2	133	3.0	140	3.0	0.368	5.7	LOS A	0.0	0.0	0.00	0.12	0.00	57.0
5	T1	1197	3.0	1260	3.0	0.368	0.1	LOS A	0.0	0.0	0.00	0.05	0.00	59.3
Approach		1330	3.0	1400	3.0	0.368	0.7	NA	0.0	0.0	0.00	0.06	0.00	59.0
West: Riverlea Blvd														
11	T1	119	3.0	125	3.0	0.033	0.0	LOS A	0.0	0.0	0.00	0.00	0.00	60.0
12	R2	14	3.0	15	3.0	0.047	17.2	LOS B	0.2	1.2	0.83	0.93	0.83	43.1
Approach		133	3.0	140	3.0	0.047	1.8	NA	0.2	1.2	0.09	0.10	0.09	57.6
All Vehicles		1486	3.0	1564	3.0	0.368	1.1	NA	0.3	1.8	0.02	0.08	0.02	58.5

Table 6.9: Intersection 4 – Ultimate T-Junction – AM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 40-41 Road - Stage 1 Turn to Median														
1	L2	14	3.0	15	3.0	0.090	7.2	LOS A	0.4	2.8	0.49	0.64	0.49	48.5
3	R2	82	3.0	86	3.0	0.090	6.2	LOS A	0.4	2.8	0.49	0.64	0.49	48.3
Approach		96	3.0	101	3.0	0.090	6.4	LOS A	0.4	2.8	0.49	0.64	0.49	48.3
East: Riverlea Blvd														
4	L2	9	3.0	9	3.0	0.134	5.6	LOS A	0.0	0.0	0.00	0.02	0.00	58.0
5	T1	478	3.0	503	3.0	0.134	0.0	LOS A	0.0	0.0	0.00	0.01	0.00	59.8
Approach		487	3.0	513	3.0	0.134	0.1	NA	0.0	0.0	0.00	0.01	0.00	59.8
West: Riverlea Blvd														
11	T1	2317	3.0	2439	3.0	0.638	0.4	LOS A	0.0	0.0	0.00	0.00	0.00	59.3
12	R2	14	3.0	15	3.0	0.012	7.0	LOS A	0.0	0.3	0.35	0.59	0.35	48.7
Approach		2331	3.0	2454	3.0	0.638	0.4	NA	0.0	0.3	0.00	0.00	0.00	59.2
SouthWest: Right Turn Staged (from median)														
32a	R1	82	3.0	86	3.0	3.321	2193.5	LOS F	46.4	333.0	1.00	2.15	6.34	1.6
Approach		82	3.0	86	3.0	3.321	2193.5	LOS F	46.4	333.0	1.00	2.15	6.34	1.6
All Vehicles		2996	3.0	3154	3.0	3.321	60.6	NA	46.4	333.0	0.04	0.08	0.19	29.3

Table 6.10:Intersection 4 – Ultimate T-Junction – PM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 40-41 Road														
1	L2	14	3.0	15	3.0	1.000	526.1	LOS F	6.2	44.2	1.00	1.14	1.61	5.7
3	R2	9	3.0	9	3.0	1.000	650.8	LOS F	6.2	44.2	1.00	1.14	1.61	5.7
Approach		23	3.0	24	3.0	1.000	574.9	LOS F	6.2	44.2	1.00	1.14	1.61	5.7
East: Riverlea Blvd														
4	L2	133	3.0	140	3.0	0.686	6.0	LOS A	0.0	0.0	0.00	0.06	0.00	56.9
5	T1	2353	3.0	2477	3.0	0.686	0.5	LOS A	0.0	0.0	0.00	0.03	0.00	58.9
Approach		2486	3.0	2617	3.0	0.686	0.8	NA	0.0	0.0	0.00	0.03	0.00	58.7
West: Riverlea Blvd														
11	T1	653	3.0	687	3.0	0.181	0.0	LOS A	0.0	0.0	0.00	0.00	0.00	59.9
12	R2	14	3.0	15	3.0	1.000	586.4	LOS F	3.3	23.5	1.00	1.12	1.56	5.6
Approach		667	3.0	702	3.0	1.000	12.4	NA	3.3	23.5	0.02	0.02	0.03	49.7
All Vehicles		3176	3.0	3343	3.0	1.000	7.3	NA	6.2	44.2	0.01	0.04	0.02	53.1

6.2.1 Intersection 4 Analysis Summary

Intersection 4 will provide access to the residential area adjacent with a small number of dwellings comparatively. A T-Junction is proposed as the initial intersection which will operate satisfactorily for the development of Precinct 2.

The operation of the intersection, in particular right turns from South to the East (during AM Peak Periods) will deteriorate as traffic volumes increase in Riverlea Boulevard. This is demonstrated in the SIDRA summaries where the Degree of Saturation for the right turn movement from Stage 40-41 Road has been calculated at over 1 indicating loss of capacity and poor ability to turn right when considering Ultimate traffic flows.

Given the proximity of the intersection to Intersection 5, it is likely that there will be more gaps than able to be considered by SIDRA. However, there is opportunity for vehicles from this stage to use Intersection 5 or Intersection 3 to turn right onto Riverlea Boulevard. The alternative access and relatively low volumes at this street does not warrant a significant upgrade of the intersection into the future.

6.3 Intersection 5 Assessment

Intersection 5 is proposed to be a four-way intersection linking between the Neighbourhood Centre to the north and school/sports precinct to the south of Riverlea Boulevard. This intersection is a key location for access in this precinct, in particular for pedestrian and cyclist movements to and from retail/commercial, school and sporting uses. The anticipated AM and PM peak hour traffic volumes for Precinct 2 volumes at intersection 5 are shown in Figure 6.1. There will be high traffic volume of vehicle turning left from NCe Road to travel east on Riverlea Boulevard in the AM Peak, and return to turn right into NCe Road in the PM peak.

Figure 6.7: PM Peak Hour Turning Movement Volumes – Intersection 5

Nce Road

		PM	91	55	127	274		
Riverlea Blvd		AM	91	55	685	832	Riverlea Blvd	
PM	AM		R	T	L		AM	PM
117	117	L				R	91	649
99	482	T				T	70	462
100	100	R				L	47	85
Ultimate			L	T	R		Ultimate	
633	1730	AM	41	51	86		415	1618
		PM	41	51	10			

Sport Road

Orange figures indicates the future traffic on Riverlea Boulevard for the Ultimate intersection analysis.

The Initial and Ultimate intersection layouts are shown in Figure 6.2 and Figure 6.4 respectively.

Figure 6.8: Intersection 5 – Initial Roundabout Option

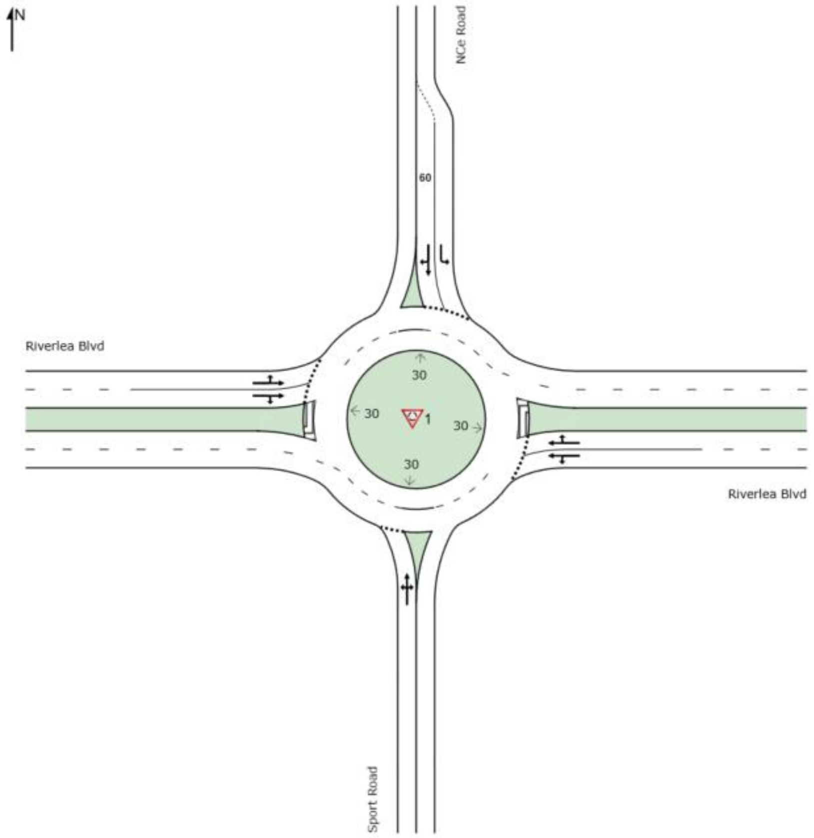


Figure 6.9: Intersection 5 – Initial Traffic Signals Option

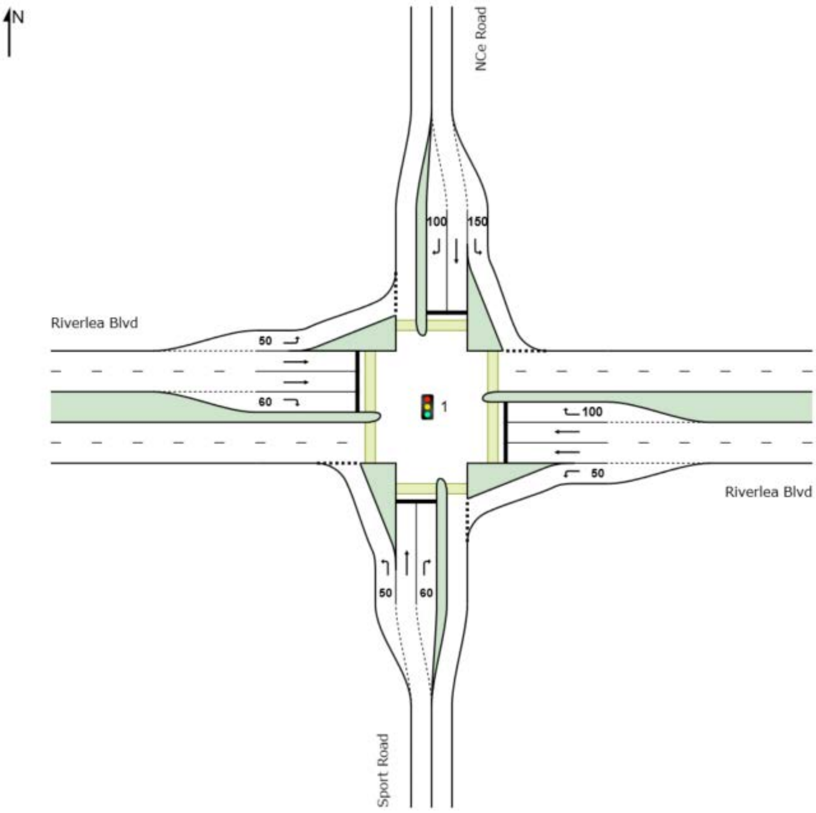
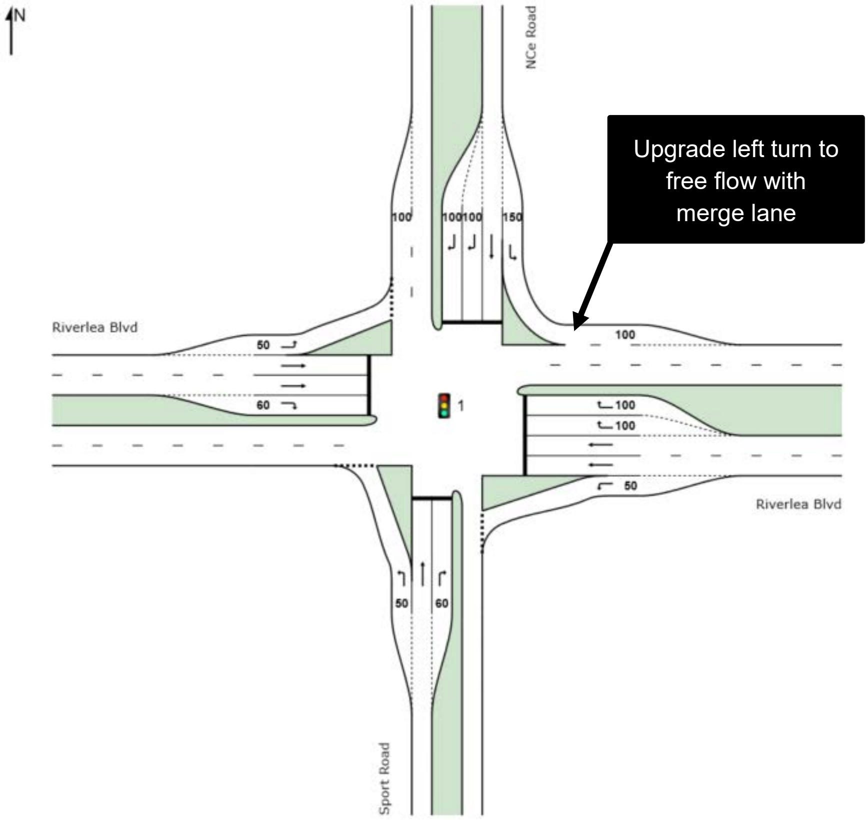


Figure 6.10: Intersection 5 – Ultimate Traffic Signals



The movement summary for each intersection and peak period is shown in the following tables.

Table 6.11:Intersection 5 – Initial Roundabout – AM Peak Summary

Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
Vehicle Movement Performance														
South: Sport Road														
1	L2	41	3.0	43	3.0	0.176	3.3	LOS A	0.8	5.4	0.38	0.54	0.38	49.6
2	T1	51	3.0	54	3.0	0.176	3.0	LOS A	0.8	5.4	0.38	0.54	0.38	47.7
3	R2	86	3.0	91	3.0	0.176	8.5	LOS A	0.8	5.4	0.38	0.54	0.38	51.2
Approach		178	3.0	187	3.0	0.176	5.8	LOS A	0.8	5.4	0.38	0.54	0.38	49.8
East: Riverlea Blvd														
4	L2	47	3.0	49	3.0	0.086	4.5	LOS A	0.5	3.4	0.40	0.46	0.40	50.7
5	T1	70	3.0	74	3.0	0.086	4.3	LOS A	0.5	3.4	0.40	0.47	0.40	56.1
6	R2	91	3.0	96	3.0	0.086	10.3	LOS A	0.5	3.3	0.42	0.63	0.42	49.8
Approach		208	3.0	219	3.0	0.086	7.0	LOS A	0.5	3.4	0.41	0.54	0.41	51.9
North: NCe Road														
7	L2	685	3.0	721	3.0	0.691	7.1	LOS A	5.9	42.1	0.77	0.96	1.01	46.0
8	T1	55	3.0	58	3.0	0.241	5.5	LOS A	1.0	7.2	0.60	0.78	0.60	46.3
9	R2	91	3.0	96	3.0	0.241	11.0	LOS A	1.0	7.2	0.60	0.78	0.60	49.4
Approach		831	3.0	875	3.0	0.691	7.4	LOS A	5.9	42.1	0.74	0.93	0.94	46.4
West: Riverlea Blvd														
10	L2	117	3.0	123	3.0	0.280	4.6	LOS A	1.7	12.5	0.43	0.47	0.43	54.2
11	T1	482	3.0	507	3.0	0.280	4.6	LOS A	1.7	12.5	0.44	0.51	0.44	55.5
12	R2	100	3.0	105	3.0	0.280	10.5	LOS A	1.7	12.1	0.46	0.55	0.46	51.8
Approach		699	3.0	736	3.0	0.280	5.4	LOS A	1.7	12.5	0.44	0.51	0.44	54.7
All Vehicles		1916	3.0	2017	3.0	0.691	6.5	LOS A	5.9	42.1	0.56	0.70	0.65	50.1

Table 6.12:Intersection 5 – Initial Roundabout – PM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Sport Road														
1	L2	41	3.0	43	3.0	0.313	6.9	LOS A	1.5	11.1	0.76	0.87	0.77	47.8
2	T1	51	3.0	54	3.0	0.313	6.6	LOS A	1.5	11.1	0.76	0.87	0.77	46.0
3	R2	86	3.0	91	3.0	0.313	12.1	LOS A	1.5	11.1	0.76	0.87	0.77	49.2
Approach		178	3.0	187	3.0	0.313	9.3	LOS A	1.5	11.1	0.76	0.87	0.77	47.9
East: Riverlea Blvd														
4	L2	85	3.0	89	3.0	0.487	5.4	LOS A	3.7	26.2	0.59	0.54	0.59	49.8
5	T1	462	3.0	486	3.0	0.487	5.2	LOS A	3.8	27.3	0.59	0.54	0.59	55.2
6	R2	649	3.0	683	3.0	0.487	10.5	LOS A	3.8	27.3	0.56	0.67	0.56	49.2
Approach		1196	3.0	1259	3.0	0.487	8.1	LOS A	3.8	27.3	0.57	0.61	0.57	51.4
North: NCe Road														
7	L2	127	3.0	134	3.0	0.122	3.5	LOS A	0.5	3.9	0.41	0.48	0.41	47.6
8	T1	55	3.0	58	3.0	0.125	2.8	LOS A	0.6	4.2	0.40	0.54	0.40	47.3
9	R2	91	3.0	96	3.0	0.125	8.3	LOS A	0.6	4.2	0.40	0.54	0.40	50.6
Approach		273	3.0	287	3.0	0.125	5.0	LOS A	0.6	4.2	0.40	0.51	0.40	48.5
West: Riverlea Blvd														
10	L2	117	3.0	123	3.0	0.217	8.2	LOS A	1.6	11.8	0.83	0.77	0.83	52.4
11	T1	99	3.0	104	3.0	0.217	8.4	LOS A	1.6	11.8	0.83	0.79	0.83	52.8
12	R2	100	3.0	105	3.0	0.217	14.9	LOS B	1.5	10.4	0.82	0.85	0.82	48.2
Approach		316	3.0	333	3.0	0.217	10.4	LOS A	1.6	11.8	0.83	0.80	0.83	51.1
All Vehicles		1963	3.0	2066	3.0	0.487	8.2	LOS A	3.8	27.3	0.60	0.65	0.61	50.6

Table 6.13:Intersection 5 – Initial Traffic Signals – AM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Sport Road														
1	L2	41	3.0	43	3.0	0.033	6.2	LOS A	0.3	1.9	0.31	0.57	0.31	49.2
2	T1	51	3.0	54	3.0	0.172	28.5	LOS B	1.6	11.8	0.90	0.68	0.90	36.1
3	R2	86	3.0	91	3.0	0.560	40.6	LOS C	3.2	23.2	1.00	0.79	1.05	33.8
Approach		178	3.0	187	3.0	0.560	29.2	LOS C	3.2	23.2	0.81	0.71	0.84	37.1
East: Riverlea Blvd														
4	L2	47	3.0	49	3.0	0.038	6.1	LOS A	0.1	0.5	0.07	0.56	0.07	50.2
5	T1	70	3.0	74	3.0	0.093	25.0	LOS B	1.0	6.9	0.78	0.58	0.78	42.6
6	R2	91	3.0	96	3.0	0.508	40.1	LOS C	3.3	23.9	0.99	0.77	0.99	34.3
Approach		208	3.0	219	3.0	0.508	27.4	LOS B	3.3	23.9	0.71	0.66	0.71	39.7
North: NCe Road														
7	L2	685	3.0	721	3.0	0.650	10.4	LOS A	12.3	88.0	0.71	0.78	0.71	43.8
8	T1	55	3.0	58	3.0	*0.186	28.5	LOS C	1.8	12.8	0.90	0.68	0.90	36.1
9	R2	91	3.0	96	3.0	*0.593	40.8	LOS C	3.4	24.7	1.00	0.80	1.08	33.7
Approach		831	3.0	875	3.0	0.650	15.0	LOS B	12.3	88.0	0.75	0.77	0.76	41.9
West: Riverlea Blvd														
10	L2	117	3.0	123	3.0	0.095	6.1	LOS A	0.2	1.4	0.08	0.57	0.08	53.8
11	T1	482	3.0	507	3.0	*0.640	28.9	LOS C	8.0	57.5	0.93	0.79	0.96	40.7
12	R2	100	3.0	105	3.0	*0.559	40.5	LOS C	3.7	26.5	1.00	0.79	1.03	34.1
Approach		699	3.0	736	3.0	0.640	26.8	LOS B	8.0	57.5	0.80	0.75	0.82	41.3
All Vehicles		1916	3.0	2017	3.0	0.650	21.9	LOS B	12.3	88.0	0.77	0.75	0.78	40.9

Table 6.14:Intersection 5 – Initial Traffic Signals – PM Peak Summary

Vehicle Movement Performance

Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Sport Road														
1	L2	41	3.0	43	3.0	0.041	6.2	LOS A	0.3	2.4	0.25	0.56	0.25	49.2
2	T1	51	3.0	54	3.0	*0.474	55.3	LOS D	2.9	20.5	1.00	0.74	1.00	28.6
3	R2	10	3.0	11	3.0	0.098	57.8	LOS E	0.5	3.9	0.97	0.67	0.97	29.2
Approach		102	3.0	107	3.0	0.474	35.8	LOS C	2.9	20.5	0.69	0.66	0.69	34.5
East: Riverlea Blvd														
4	L2	85	3.0	89	3.0	0.062	7.4	LOS A	0.8	5.4	0.25	0.61	0.25	49.3
5	T1	462	3.0	486	3.0	0.248	5.6	LOS A	2.7	19.3	0.23	0.20	0.23	55.0
6	R2	649	3.0	683	3.0	*0.732	23.0	LOS B	22.9	164.2	0.71	0.80	0.71	40.7
Approach		1196	3.0	1259	3.0	0.732	15.2	LOS B	22.9	164.2	0.49	0.56	0.49	45.9
North: NCe Road														
7	L2	127	3.0	134	3.0	0.088	5.6	LOS A	0.8	5.4	0.21	0.56	0.21	46.6
8	T1	55	3.0	58	3.0	0.438	53.9	LOS D	3.0	21.7	1.00	0.75	1.00	28.9
9	R2	91	3.0	96	3.0	*0.762	62.3	LOS E	5.3	38.3	1.00	0.89	1.24	28.2
Approach		273	3.0	287	3.0	0.762	34.2	LOS C	5.3	38.3	0.63	0.71	0.71	34.8
West: Riverlea Blvd														
10	L2	117	3.0	123	3.0	0.159	13.0	LOS A	2.3	16.7	0.48	0.68	0.48	48.9
11	T1	99	3.0	104	3.0	0.345	52.1	LOS D	2.7	19.1	0.99	0.73	0.99	32.5
12	R2	100	3.0	105	3.0	*0.733	61.7	LOS E	5.8	41.3	1.00	0.86	1.18	28.5
Approach		316	3.0	333	3.0	0.733	40.7	LOS C	5.8	41.3	0.80	0.75	0.86	35.3
All Vehicles		1887	3.0	1986	3.0	0.762	23.3	LOS B	22.9	164.2	0.58	0.62	0.60	41.2

Table 6.15:Intersection 5 – Ultimate Traffic Signals AM Peak Summary

Vehicle Movement Performance

Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Sport Road														
1	L2	41	3.0	43	3.0	0.031	8.4	LOS A	0.7	4.9	0.28	0.57	0.28	47.8
2	T1	51	3.0	54	3.0	0.677	82.9	LOS F	4.2	30.1	1.00	0.80	1.13	23.7
3	R2	86	3.0	91	3.0	0.801	86.9	LOS F	7.1	51.1	1.00	0.90	1.24	23.8
Approach		178	3.0	187	3.0	0.801	67.7	LOS E	7.1	51.1	0.83	0.79	0.99	26.9
East: Riverlea Blvd														
4	L2	47	3.0	49	3.0	0.037	5.8	LOS A	0.0	0.4	0.02	0.55	0.02	50.5
5	T1	415	3.0	437	3.0	*0.847	74.4	LOS F	17.0	121.9	1.00	0.92	1.15	27.1
6	R2	91	3.0	96	3.0	0.238	68.6	LOS E	4.0	28.5	0.93	0.74	0.93	27.2
Approach		553	3.0	582	3.0	0.847	67.6	LOS E	17.0	121.9	0.91	0.86	1.02	28.3
North: NCe Road														
7	L2	685	3.0	721	3.0	0.383	13.7	LOS A	0.0	0.0	0.00	0.46	0.00	47.7
8	T1	55	3.0	58	3.0	*0.730	83.7	LOS F	4.6	32.7	1.00	0.83	1.19	23.6
9	R2	91	3.0	96	3.0	*0.847	89.3	LOS F	7.7	55.1	1.00	0.94	1.32	23.4
Approach		831	3.0	875	3.0	0.847	26.6	LOS B	7.7	55.1	0.18	0.54	0.22	40.4
West: Riverlea Blvd														
10	L2	117	3.0	123	3.0	0.080	7.0	LOS A	1.0	7.4	0.18	0.60	0.18	53.2
11	T1	1730	3.0	1821	3.0	*0.880	7.1	LOS A	24.1	172.9	0.36	0.35	0.38	53.8
12	R2	100	3.0	105	3.0	0.101	22.2	LOS B	3.5	25.4	0.50	0.68	0.50	41.1
Approach		1947	3.0	2049	3.0	0.880	7.9	LOS A	24.1	172.9	0.36	0.38	0.37	52.9
All Vehicles		3509	3.0	3694	3.0	0.880	24.7	LOS B	24.1	172.9	0.42	0.52	0.47	42.0

Table 6.16:Intersection 5 – Ultimate Traffic Signals PM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Sport Road														
1	L2	41	3.0	43	3.0	0.062	6.9	LOS A	0.4	2.7	0.32	0.58	0.32	48.8
2	T1	51	3.0	54	3.0	0.384	43.5	LOS D	2.3	16.4	0.99	0.74	0.99	31.6
3	R2	10	3.0	11	3.0	0.079	46.3	LOS D	0.4	3.1	0.96	0.67	0.96	32.2
Approach		102	3.0	107	3.0	0.384	29.1	LOS C	2.3	16.4	0.72	0.67	0.72	36.8
East: Riverlea Blvd														
4	L2	85	3.0	89	3.0	0.091	8.6	LOS A	0.6	4.4	0.24	0.61	0.24	48.6
5	T1	1618	3.0	1703	3.0	0.745	1.3	LOS A	4.8	34.7	0.15	0.14	0.15	58.7
6	R2	649	3.0	683	3.0	*0.906	43.4	LOS D	15.6	112.0	1.00	0.97	1.25	33.4
Approach		2352	3.0	2476	3.0	0.906	13.2	LOS A	15.6	112.0	0.39	0.38	0.45	48.2
North: NCe Road														
7	L2	127	3.0	134	3.0	0.071	5.1	LOS A	0.0	0.0	0.00	0.47	0.00	47.8
8	T1	55	3.0	58	3.0	*0.414	43.7	LOS D	2.5	17.7	0.99	0.74	0.99	31.5
9	R2	91	3.0	96	3.0	*0.720	51.0	LOS D	4.3	31.0	1.00	0.86	1.21	30.9
Approach		273	3.0	287	3.0	0.720	28.2	LOS B	4.3	31.0	0.53	0.65	0.61	37.2
West: Riverlea Blvd														
10	L2	117	3.0	123	3.0	0.107	7.9	LOS A	0.7	5.1	0.20	0.60	0.20	52.4
11	T1	633	3.0	666	3.0	0.595	28.7	LOS C	11.3	80.9	0.85	0.73	0.85	40.9
12	R2	100	3.0	105	3.0	*0.943	74.5	LOS F	6.5	46.4	1.00	1.13	1.83	26.0
Approach		850	3.0	895	3.0	0.943	31.2	LOS C	11.3	80.9	0.78	0.76	0.88	39.4
All Vehicles		3577	3.0	3765	3.0	0.943	19.1	LOS B	15.6	112.0	0.50	0.50	0.57	44.5

6.3.1 Intersection 5 Analysis Summary

Intersection 5 will provide access to the proposed Neighbourhood Centre (to the north) and Sports Fields/School to the south. It will have a mix of traffic movements in conjunction with high flows on Riverlea Boulevard. Pedestrian access should be considered at this intersection with crossings on each side of the intersection.

A roundabout could be provided similar to Intersection 3. There will be a high volume of left turns from NCe Road (north) to Riverlea Boulevard (east) which will require a left turn lane to provide appropriate level of service. Given the nearby school and sports fields, a roundabout would not provide the best pedestrian access as traffic volumes grow on Riverlea Boulevard. Similar to Intersection 3, a roundabout will struggle to cope with future westbound PM peak period flows, with long queues predicted in modelling the longer term roundabout. A roundabout at this location would not operate beyond part development of Precinct 3 to the west without significant modifications, including a bypass lane from NCe Road to Riverlea Boulevard (east) for eastbound traffic for the AM Peak period. The PM Peak period could operate longer possibly up to development of Precinct 3 only.

An alternative to improve pedestrian access would be to provide traffic signals as the Initial Intersection. This would provide appropriate traffic capacity whilst providing a high level of pedestrian access across Riverlea Boulevard. A slightly smaller signalised intersection (compared to the ultimate layout) could be provided initially with single right turn lane on Riverlea Boulevard.

Traffic signals will be required in the ultimate layout. In particular, a free flowing left turn will be required from NCe Road (north) to Riverlea Boulevard (east) due to the high eastbound flows on Riverlea Boulevard in the AM peak period. Traffic signals utilising a high frequency cycle (that is shorter cycle time) will maintain traffic capacity more effectively and will assist with pedestrian access with more frequent phases occurring.

6.4 Intersection 5A Assessment

Intersection 5A is located adjacent the proposed school and provides access for residential stages to the south of Riverlea Boulevard. The intersection will initially be an unsignalised T-junction. The anticipated AM and PM peak hour traffic volumes for the intersection are shown in Figure 6.11.

Figure 6.11: Intersection 5A - PM Peak Hour Turning Movement Volumes

Riverlea Blvd							Riverlea Blvd	
PM	AM						AM	PM
86	365	T				T	158	599
49	49	R				L	104	161
Ultimate			L	T	R		Ultimate	
620	1613	AM	49		112		503	1755
		PM	49		56			

School Road

Orange figures indicates the future through traffic on Riverlea Boulevard for the Ultimate intersection analysis.

The Initial and Ultimate intersection layouts are shown in Figure 6.12 and Figure 6.13 respectively.

Figure 6.12: Intersection 5A – Initial T-Junction

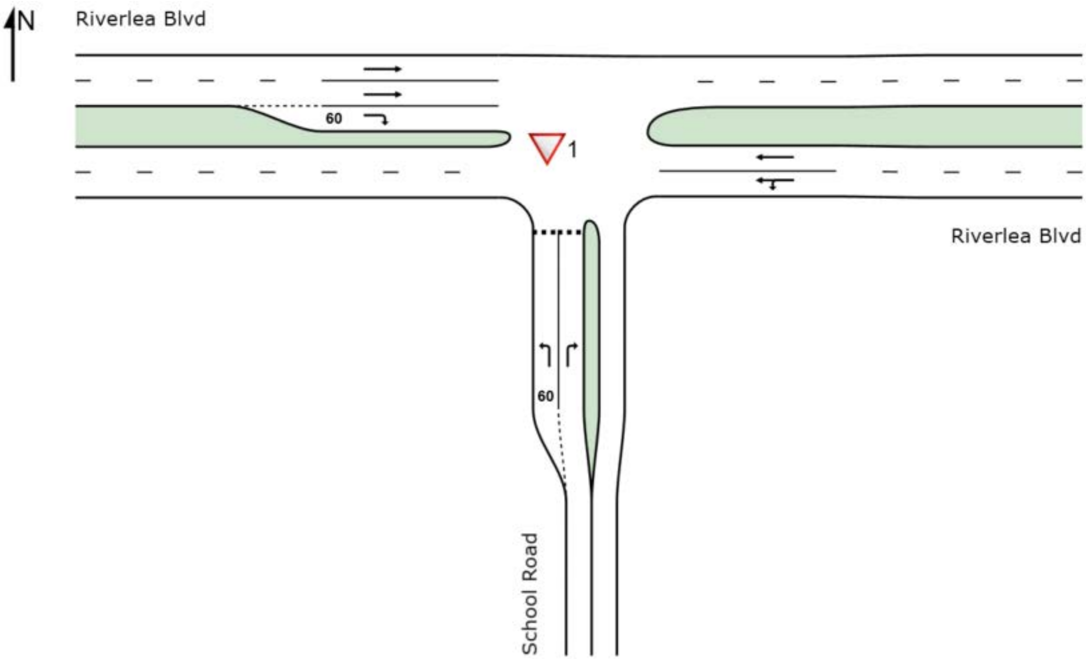
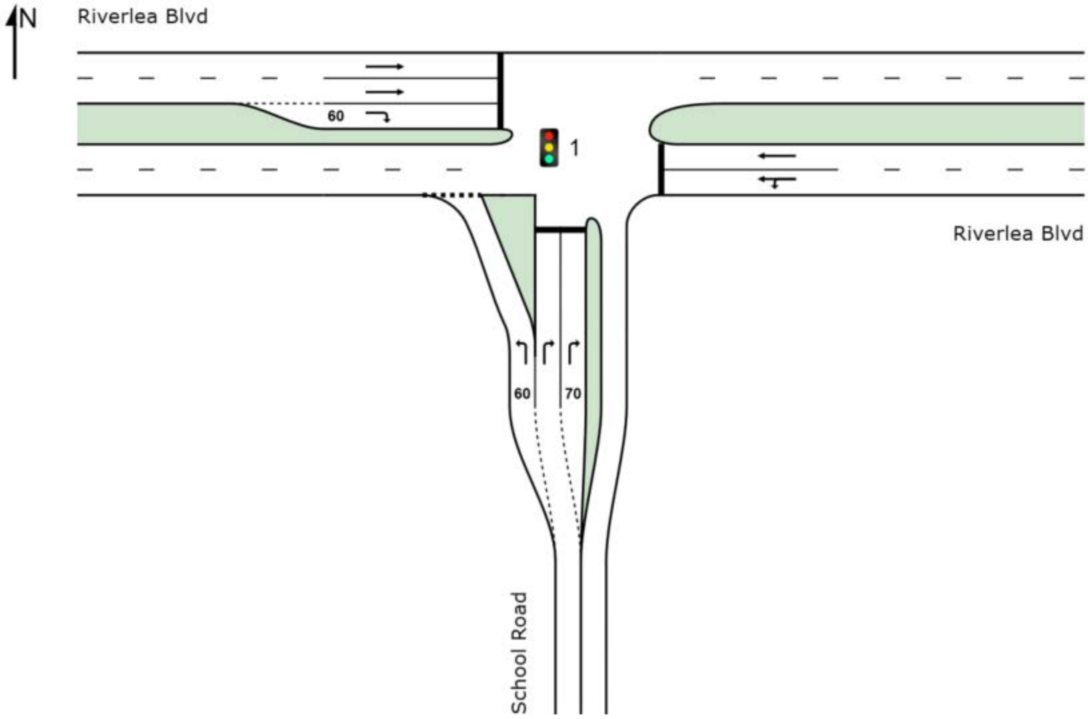


Figure 6.13: Intersection 5A – Ultimate Traffic Signals



The movement summary for each intersection peak period is shown in the following tables.

Table 6.17: Intersection 5A – Initial T-Junction – AM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: School Road														
1	L2	49	3.0	52	3.0	0.055	5.7	LOS A	0.2	1.4	0.32	0.56	0.32	48.8
3	R2	112	3.0	118	3.0	0.125	7.4	LOS A	0.5	3.9	0.57	0.73	0.57	47.7
Approach		161	3.0	169	3.0	0.125	6.9	LOS A	0.5	3.9	0.50	0.68	0.50	48.0
East: Riverlea Blvd														
4	L2	104	3.0	109	3.0	0.074	5.6	LOS A	0.0	0.0	0.00	0.47	0.00	54.3
5	T1	158	3.0	166	3.0	0.074	0.0	LOS A	0.0	0.0	0.00	0.07	0.00	59.3
Approach		262	3.0	276	3.0	0.074	2.2	NA	0.0	0.0	0.00	0.23	0.00	57.2
West: Riverlea Blvd														
11	T1	365	3.0	384	3.0	0.101	0.0	LOS A	0.0	0.0	0.00	0.00	0.00	60.0
12	R2	49	3.0	52	3.0	0.037	6.5	LOS A	0.2	1.2	0.36	0.57	0.36	48.9
Approach		414	3.0	436	3.0	0.101	0.8	NA	0.2	1.2	0.04	0.07	0.04	58.4
All Vehicles		837	3.0	881	3.0	0.125	2.4	NA	0.5	3.9	0.12	0.24	0.12	55.7

Table 6.18:Intersection 5A – Initial T-Junction – PM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: School Road														
1	L2	49	3.0	52	3.0	0.104	10.1	LOS A	0.4	2.6	0.61	0.81	0.61	46.3
3	R2	56	3.0	59	3.0	0.081	8.8	LOS A	0.3	2.4	0.63	0.79	0.63	46.8
Approach		105	3.0	111	3.0	0.104	9.4	LOS A	0.4	2.6	0.62	0.80	0.62	46.6
East: Riverlea Blvd														
4	L2	161	3.0	169	3.0	0.211	5.6	LOS A	0.0	0.0	0.00	0.25	0.00	56.0
5	T1	599	3.0	631	3.0	0.211	0.1	LOS A	0.0	0.0	0.00	0.09	0.00	59.1
Approach		760	3.0	800	3.0	0.211	1.2	NA	0.0	0.0	0.00	0.13	0.00	58.4
West: Riverlea Blvd														
11	T1	86	3.0	91	3.0	0.024	0.0	LOS A	0.0	0.0	0.00	0.00	0.00	60.0
12	R2	49	3.0	52	3.0	0.067	9.4	LOS A	0.3	1.9	0.61	0.77	0.61	47.4
Approach		135	3.0	142	3.0	0.067	3.4	NA	0.3	1.9	0.22	0.28	0.22	54.7
All Vehicles		1000	3.0	1053	3.0	0.211	2.4	NA	0.4	2.6	0.09	0.22	0.09	56.4

Table 6.19:Intersection 5A – Ultimate Traffic Signals – AM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: School Road														
1	L2	49	3.0	52	3.0	0.051	4.9	LOS A	0.2	1.2	0.11	0.52	0.11	50.0
3	R2	112	3.0	118	3.0	* 0.486	66.2	LOS E	3.5	25.3	1.00	0.75	1.00	27.0
Approach		161	3.0	169	3.0	0.486	47.6	LOS D	3.5	25.3	0.73	0.68	0.73	31.4
East: Riverlea Blvd														
4	L2	104	3.0	109	3.0	0.250	6.1	LOS A	0.5	3.4	0.03	0.22	0.03	52.1
5	T1	503	3.0	529	3.0	0.250	0.5	LOS A	0.5	3.5	0.03	0.10	0.03	58.8
Approach		607	3.0	639	3.0	0.250	1.4	LOS A	0.5	3.5	0.03	0.12	0.03	57.5
West: Riverlea Blvd														
11	T1	1613	3.0	1698	3.0	* 0.533	0.4	LOS A	2.0	14.6	0.05	0.04	0.05	59.6
12	R2	49	3.0	52	3.0	0.262	53.5	LOS D	2.6	18.6	0.86	0.73	0.86	30.3
Approach		1662	3.0	1749	3.0	0.533	1.9	LOS A	2.6	18.6	0.07	0.06	0.07	58.0
All Vehicles		2430	3.0	2558	3.0	0.533	4.8	LOS A	3.5	25.3	0.10	0.12	0.10	54.8

Table 6.20:Intersection 5A – Ultimate Traffic Signals – PM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: School Road														
1	L2	49	3.0	52	3.0	0.349	26.7	LOS B	2.8	20.3	0.71	0.70	0.71	38.6
3	R2	56	3.0	59	3.0	* 0.349	49.1	LOS D	2.8	20.3	0.87	0.71	0.87	30.9
Approach		105	3.0	111	3.0	0.349	38.7	LOS C	2.8	20.3	0.79	0.70	0.79	34.1
East: Riverlea Blvd														
4	L2	161	3.0	169	3.0	0.766	6.4	LOS A	4.6	33.2	0.09	0.17	0.09	52.6
5	T1	1755	3.0	1847	3.0	* 0.766	0.8	LOS A	4.7	33.5	0.09	0.12	0.09	58.8
Approach		1916	3.0	2017	3.0	0.766	1.3	LOS A	4.7	33.5	0.09	0.13	0.09	58.2
West: Riverlea Blvd														
11	T1	620	3.0	653	3.0	0.201	0.2	LOS A	0.5	3.3	0.03	0.02	0.03	59.8
12	R2	49	3.0	52	3.0	* 0.807	55.5	LOS D	2.7	19.0	0.86	0.76	1.00	29.7
Approach		669	3.0	704	3.0	0.807	4.3	LOS A	2.7	19.0	0.09	0.08	0.10	55.6
All Vehicles		2690	3.0	2832	3.0	0.807	3.5	LOS A	4.7	33.5	0.12	0.14	0.12	56.0

6.4.1 Intersection 5A Summary

The SIDRA intersection analysis indicates that the proposed unsignalised T-junction at Intersection 5A would operate satisfactorily and within capacity for the predicted Precinct 2 traffic volumes.

Given the location adjacent a school, there may be a need for traffic signals to facilitate pedestrian crossing and safety at the intersection, rather than installing a mid-block crossing to the east for instance. Being a T-junction, the efficiency can deteriorate if higher flows occur on Riverlea Boulevard. The intersection should be monitored to determine when the upgrade should occur based on additional development to the west.

The School Road does link back to intersection 5 which has a higher capacity and would provide for connectivity back to the neighbourhood centre to the north. This may become a loop circuit for people delivering children to school.

The ultimate intersection for traffic signals will have a higher capacity to cater for higher traffic flows on Riverlea Boulevard. Monitoring of the intersection and consideration with development to the west will determine when the traffic signals will be required.

6.5 Intersection 6 Assessment

Intersection 6 will initially be at the end of the Riverlea development, with a T-junction proposed to connect to residential stages to the north and south. Longer-term Riverlea Boulevard will continue west which will require a 4-way intersection to be appropriately managed.

The anticipated AM and PM peak hour traffic volumes for Precinct 2 volumes at intersection 5 are shown in Figure 6.14.

Figure 6.14: Intersection 6 –AM & PM Peak Hour Turning Volumes

Stage 36 Road

		PM	0	5	102			
No Connection		AM	0	5	348		Riverlea Blvd	
PM	AM		R	T	L		AM	PM
0	0	L				R	102	348
0	0	T				T	0	0
0	0	R				L	13	47
			L	T	R			
534	1248	AM	0	5	47		345	1156
		PM	0	5	13			

Stage 37 Road

Orange figures indicates the future traffic on Riverlea Boulevard for the Ultimate intersection analysis.

The Initial and Ultimate intersection layouts are shown in Figure 6.15 and Figure 6.16 respectively.

Figure 6.15: Intersection 6 – Initial – T-Junction

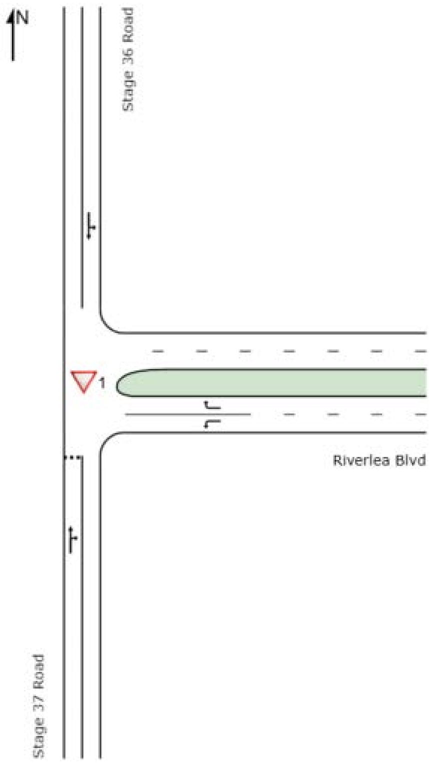
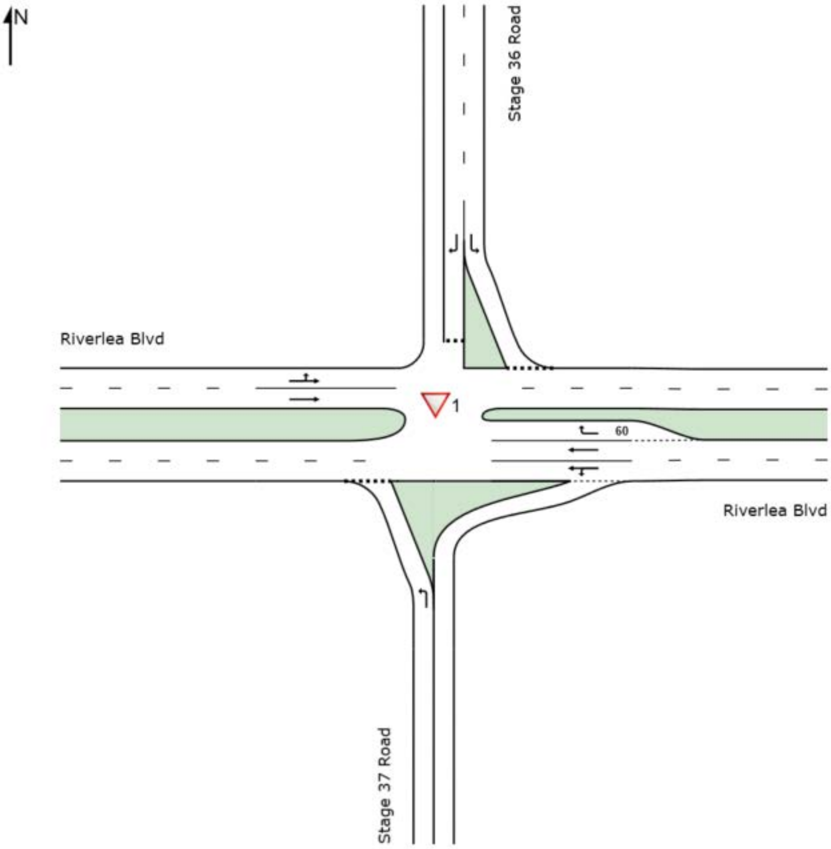


Figure 6.16: Intersection 6 – Ultimate Intersection



The SIDRA movement summary for each intersection and peak period is shown in the following tables.

Table 6.21:Intersection 6 – T-junction – AM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 37 Road														
2	T1	5	3.0	5	3.0	0.048	4.0	LOS A	0.2	1.5	0.42	0.60	0.42	46.0
3	R2	47	3.0	49	3.0	0.048	6.4	LOS A	0.2	1.5	0.42	0.60	0.42	48.4
Approach		52	3.0	55	3.0	0.048	6.2	LOS A	0.2	1.5	0.42	0.60	0.42	48.2
East: Riverlea Blvd														
4	L2	13	3.0	14	3.0	0.008	5.6	LOS A	0.0	0.0	0.00	0.58	0.00	53.5
6	R2	102	3.0	107	3.0	0.059	5.6	LOS A	0.0	0.0	0.00	0.59	0.00	53.3
Approach		115	3.0	121	3.0	0.059	5.6	NA	0.0	0.0	0.00	0.59	0.00	53.3
North: Stage 36 Road														
7	L2	348	3.0	366	3.0	0.205	4.6	LOS A	0.0	0.3	0.01	0.52	0.01	49.7
8	T1	5	3.0	5	3.0	0.205	0.5	LOS A	0.0	0.3	0.01	0.52	0.01	47.1
Approach		353	3.0	372	3.0	0.205	4.5	NA	0.0	0.3	0.01	0.52	0.01	49.7
All Vehicles		520	3.0	547	3.0	0.205	4.9	NA	0.2	1.5	0.05	0.54	0.05	50.3

Table 6.22:Intersection 6 – T-junction – PM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 37 Road														
2	T1	5	3.0	5	3.0	0.017	5.0	LOS A	0.1	0.5	0.47	0.58	0.47	46.0
3	R2	13	3.0	14	3.0	0.017	6.4	LOS A	0.1	0.5	0.47	0.58	0.47	48.4
Approach		18	3.0	19	3.0	0.017	6.0	LOS A	0.1	0.5	0.47	0.58	0.47	47.7
East: Riverlea Blvd														
4	L2	47	3.0	49	3.0	0.027	5.6	LOS A	0.0	0.0	0.00	0.58	0.00	53.5
6	R2	348	3.0	366	3.0	0.201	5.6	LOS A	0.0	0.0	0.00	0.59	0.00	53.2
Approach		395	3.0	416	3.0	0.201	5.6	NA	0.0	0.0	0.00	0.58	0.00	53.2
North: Stage 36 Road														
7	L2	102	3.0	107	3.0	0.063	4.7	LOS A	0.0	0.4	0.06	0.48	0.06	49.7
8	T1	5	3.0	5	3.0	0.063	1.6	LOS A	0.0	0.4	0.06	0.48	0.06	47.1
Approach		107	3.0	113	3.0	0.063	4.6	NA	0.0	0.4	0.06	0.48	0.06	49.6
All Vehicles		520	3.0	547	3.0	0.201	5.4	NA	0.1	0.5	0.03	0.56	0.03	52.2

Table 6.23:Intersection 6 – Ultimate – AM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 37 Road														
1	L2	5	3.0	5	3.0	0.005	6.3	LOS A	0.0	0.1	0.26	0.51	0.26	53.2
Approach		5	3.0	5	3.0	0.005	6.3	LOS A	0.0	0.1	0.26	0.51	0.26	53.2
East: Riverlea Blvd														
4	L2	13	3.0	14	3.0	0.100	5.7	LOS A	0.0	0.0	0.00	0.04	0.00	59.4
5	T1	345	3.0	363	3.0	0.100	0.0	LOS A	0.0	0.0	0.00	0.02	0.00	59.8
6	R2	102	3.0	107	3.0	0.324	18.4	LOS B	1.1	8.0	0.83	0.97	0.97	42.5
Approach		460	3.0	484	3.0	0.324	4.3	NA	1.1	8.0	0.18	0.23	0.21	54.8
North: Stage 36 Road														
7	L2	348	3.0	366	3.0	0.556	12.3	LOS A	4.0	28.9	0.71	1.04	1.18	45.5
9	R2	5	3.0	5	3.0	0.136	93.8	LOS F	0.4	2.6	0.96	0.98	0.96	23.3
Approach		353	3.0	372	3.0	0.556	13.5	LOS A	4.0	28.9	0.72	1.04	1.18	44.9
West: Riverlea Blvd														
10	L2	5	3.0	5	3.0	0.345	5.7	LOS A	0.0	0.0	0.00	0.00	0.00	58.0
11	T1	1248	3.0	1314	3.0	0.345	0.1	LOS A	0.0	0.0	0.00	0.00	0.00	59.8
Approach		1253	3.0	1319	3.0	0.345	0.1	NA	0.0	0.0	0.00	0.00	0.00	59.8
All Vehicles		2071	3.0	2180	3.0	0.556	3.3	NA	4.0	28.9	0.16	0.23	0.25	55.5

Table 6.24:Intersection 6 – Ultimate – PM Peak Summary

Vehicle Movement Performance														
Mov ID	Turn	INPUT VOLUMES [Total veh/h	HV] %	DEMAND FLOWS [Total veh/h	HV] %	Deg. Satn v/c	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh. veh	Dist] m	Prop. Que	Effective Stop Rate	Aver. No. Cycles	Aver. Speed km/h
South: Stage 37 Road														
1	L2	5	3.0	5	3.0	0.007	8.6	LOS A	0.0	0.2	0.51	0.61	0.51	51.9
Approach		5	3.0	5	3.0	0.007	8.6	LOS A	0.0	0.2	0.51	0.61	0.51	51.9
East: Riverlea Blvd														
4	L2	47	3.0	49	3.0	0.333	5.7	LOS A	0.0	0.0	0.00	0.04	0.00	59.2
5	T1	1156	3.0	1217	3.0	0.333	0.1	LOS A	0.0	0.0	0.00	0.02	0.00	59.6
6	R2	348	3.0	366	3.0	0.432	9.7	LOS A	2.4	17.3	0.54	0.84	0.70	47.2
Approach		1551	3.0	1633	3.0	0.432	2.4	NA	2.4	17.3	0.12	0.20	0.16	56.3
NorthEast: Right Turn Thru Median(from N to W) - Stage 2 turn														
26a	R1	5	3.0	5	3.0	0.416	356.1	LOS F	1.1	7.7	0.99	1.01	1.07	8.7
Approach		5	3.0	5	3.0	0.416	356.1	LOS F	1.1	7.7	0.99	1.01	1.07	8.7
North: Stage 36 Road (inc Right Turn Stage 1 to Median)														
7	L2	102	3.0	107	3.0	0.158	7.9	LOS A	0.6	4.4	0.52	0.71	0.52	48.1
9	R2	5	3.0	5	3.0	0.294	227.0	LOS F	0.8	5.5	0.98	1.01	1.04	12.6
Approach		107	3.0	113	3.0	0.294	18.1	LOS B	0.8	5.5	0.55	0.72	0.55	42.6
West: Riverlea Blvd														
10	L2	5	3.0	5	3.0	0.148	5.6	LOS A	0.0	0.0	0.00	0.01	0.00	58.1
11	T1	534	3.0	562	3.0	0.148	0.0	LOS A	0.0	0.0	0.00	0.01	0.00	59.9
Approach		539	3.0	567	3.0	0.148	0.1	NA	0.0	0.0	0.00	0.01	0.00	59.9
All Vehicles		2207	3.0	2323	3.0	0.432	3.4	NA	2.4	17.3	0.12	0.18	0.14	55.5

The above analysis includes staged right turn from Stage 36 Road (north) where vehicles will use the median to store and wait between eastbound and westbound traffic streams on Riverlea Boulevard. This provides a more realistic assessment of the intersection operation.

6.5.1 Intersection 6 Summary

The ultimate layout for the intersection will be a 4-way intersection with the extension of Riverlea Boulevard to the west. Given there will be very few traffic movements north-south across Riverlea Boulevard, and also there would also be very little traffic to and from the west, it is recommended that the intersection remain unsignalised with only certain turning movements provided.

Stage 37 has alternative access for right turns in (from the west) and right turns out (to the east) via Intersection 5A to the east. Based on the above, this road can be limited to left turn in and out only.

Stage 36 on the northern side can retain right turn movements as it will have heavier turning movements to and from the east (right turn in and left turn out). Right turns to the west on Riverlea Boulevard can be maintained with staged turns anticipated through the median.

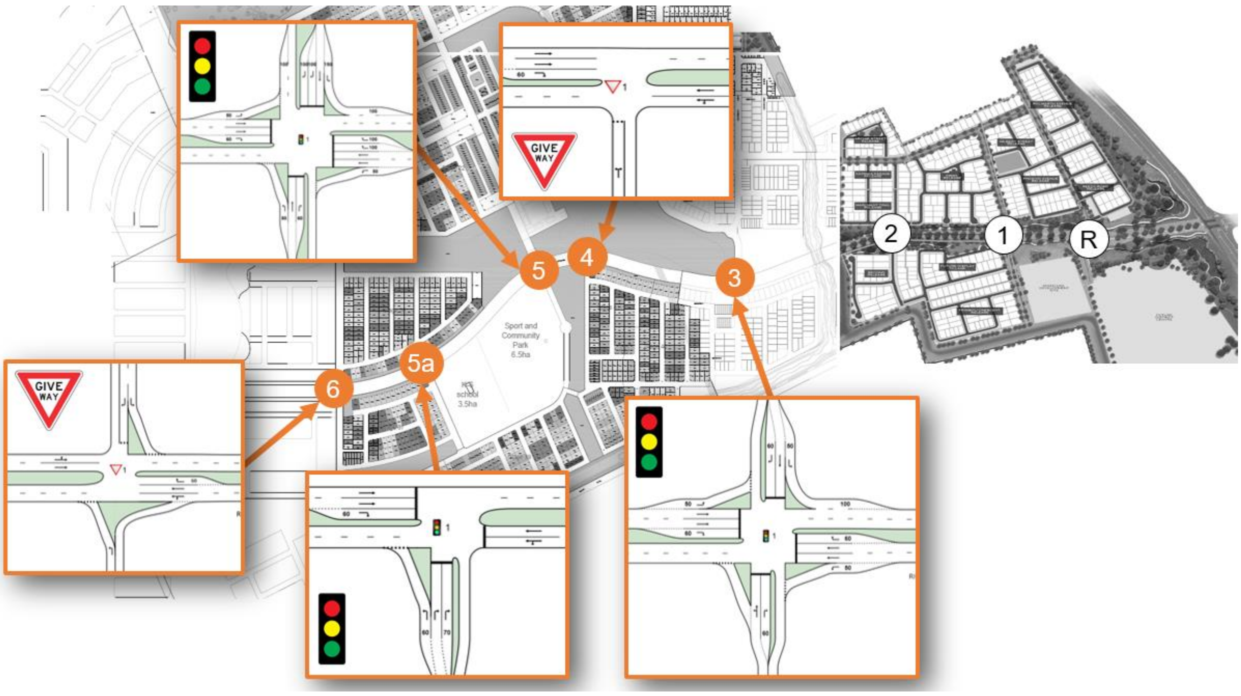
This intersection arrangement will maintain Riverlea Boulevard at 2 lanes in each direction based on higher flows from the west.

6.6 Intersection Summary

The analysis of the intersections in Precinct 2 for the Initial and Ultimate layouts is summarised in the figures below with the recommended intersection layouts.

Initial Intersections

Ultimate Intersections



6.7 Intersection Upgrade Timing

The likely need to upgrade the intersections from interim to ultimate based on future development to the west for Precinct 3 and 4 has been reviewed as part of the intersection analysis. For this assessment, it should be noted that the additional traffic volumes assumed to be from the west (from the whole development) has been developed from the original PB modelling which considered a secondary access and high level of self-sufficiency in each precinct with schools, employment and activity centres. In simple terms this equates to about 3,000 dwellings if no secondary connection is provided.

Hence it is likely overall that the intersections would need to be upgraded prior to full occupation of Precinct 3 assuming it will be similar size to Precinct 2. This assumption is made on the basis that a secondary access would not be available until Precinct 4 for which planning would occur during the development of Precinct 3. It would be assumed that a secondary connection would be provided prior to full occupation of Precinct 3. The analysis generally indicates intersections will need upgrading by 50% of the occupation of Precinct 3 (or about 1500 dwellings in addition to Precinct 2 dwellings). The above assumes Precinct 2 is complete and occupied.

Given the above, the assessment of intersections has found the following:

Intersection 3

Initial –up to 50% of Precinct 3 complete and occupied but interim upgrade likely required as per below

Interim - (additional left turn lane north leg) from 25% of Precinct 3 (due to increased AM Peak flows)

Ultimate – from 50% of Precinct 3 complete and occupied (due to PM Peak period queue lengths)

Intersection 4

Initial and Ultimate will be the same intersection.

Intersection 5

Initial roundabout or signals – can remain until about 50% of Precinct 3 occupied

Ultimate – from 50% of Precinct 3 complete and occupied

Intersection 5a

Initial – up to 50% of Precinct 3 complete and occupied

Ultimate – from 50% of Precinct 3 complete and occupied

However school traffic will likely seek traffic signals for safe crossings by children and right turns by parents. It may be recommended with the timing of the school development.

Intersection 6

Initial – Precinct 2 only

Ultimate – required when Precinct 3 connected to west

Please note this is a guide only and will be dependant on actual traffic volume growth following further development to the west.

7. Conclusions

Based on the analysis and discussions presented within this report, the following conclusions are made:

1. The proposed Precinct 2 development will include approximately 3,100 residential dwellings with associated neighbourhood centre, educational and recreational facilities within a modified grid network and key access routes to Riverlea Boulevard.

2. Precinct 2 will generate some 25,000 vehicle trips per day which is consistent with the Traffic Impact Assessment prepared for the master plan in 2009, and for Precinct 2 in 2015.

3. A review of the proposed intersections on Riverlea Boulevard has identified the initial intersection layouts which will cater for Precinct 2 traffic demands, and ultimate intersection layouts which will cater for future traffic demands of Riverlea as it is developed to the west.

4. Previous analysis has found that the Precinct 1 intersections will be able to cater for the traffic demands of Precinct 2, and similarly preliminary analysis of the Port Wakefield Road / Riverlea Boulevard intersection will be capable of handling the increase demand of Precinct 2 within existing capacity of the intersection. These intersections should be reviewed as part of planning of Precinct 3 to confirm continued suitable operation.

5. The central intersection (5) will provide access to both the neighbourhood centre precinct (to the north) and school precinct (to the south) and is recommended to have traffic signals as an initial option to better accommodate the anticipated traffic movements, but also safer pedestrian and cyclist movements compared to a roundabout.

6. Intersection 6 (at the western end of the precinct) would become T-junctions with no traffic control of Riverlea Boulevard traffic required (which will assist in limiting the impact on through traffic), with limited movements to the south due to the smaller precinct proposed. The development of connections further to the west as part of future precincts may require further consideration of the traffic control of this intersection in the future.

7. The upgrade of the intersection to the ultimate configurations shown will be dependant on timing of future stages to the west, and should be reviewed as part of the planning and design of these stages to assist in identifying upgrade requirements. Generally the initial intersections will be capable of accommodating approximately 50% of Precinct 3 traffic demands.

8. The configurations of the street network will be conducive to a low speed environment of less than 40km/h on the minor streets, and 50km/h on collector streets which will link to Riverlea Boulevard.

9. The street network will be planned to accommodate bus services when required, with road carriageways suitable for bus travel through the precinct. The actual routes are yet to be confirmed.

Appendix A

Riverlea – Precinct 2

Parks & Reserves

Parking Review

28/11/2022 **Ref: 301401258**

PREPARED FOR:

Walker Buckland Park

PREPARED BY:

Stantec Australia

Stantec

Quality Record

Issue	Date	Description	Prepared By	Checked By	Approved By	Signed
1	28/11/22	Final - updated	Paul Morris	Richard Frimpong	Paul Morris	

Contents

1. Introduction

1.1 Background

Parks and reserves in Council areas have in recent times been the focus of upgrades with active and passive uses including playgrounds, fitness equipment, sports facilities, BBQ, space for gatherings, and nature enjoyment. Car parking demands at parks and reserves has not been well defined with parking often requiring some form of management whether it be on-street or off-street parking areas. Many parks and reserves are located in residential areas with parking impacting the available parking for residents.

Car parking at parks and reserves can become an issue for the management authority with various uses that will generate traffic and parking demands. Both passive and active activities will generate traffic and parking, with users often beyond the walking distance of surrounding residents.

Activities can include:

- Open grass area for recreation;
- Picnic and barbeque;
- Natural playspace;
- Multipurpose activity space;
- Fitness stations;
- Wetland interaction and viewing decks;
- Multi-purpose court activities (e.g. tennis, basketball, etc);
- Multiple pathway loops suitable for a range or recreation activities (e.g. walking, jogging, children learning to ride a bike, etc).

It is understood that there are over 15 separate parks and reserves proposed for Precinct 2.

A review of the parks and reserves for Precinct 2 is proposed to identify anticipated activities and consider the parking demands associated with these activities will enable consideration of on-street and off-street parking. The available on-street parking demands for adjacent residential areas will be assessed and compared to potential parking demand and availability for the parks and reserves. Recommendations for parking supply at each reserve can then be considered. Additional matters to be considered are disability permit parking, and future proofing for the provision of Electric Vehicle charging.

Stantec has been commissioned to conduct a car parking study for public open spaces associated with the Precinct 2 Development located in Riverlea Park.

1.2 Purpose of this Report

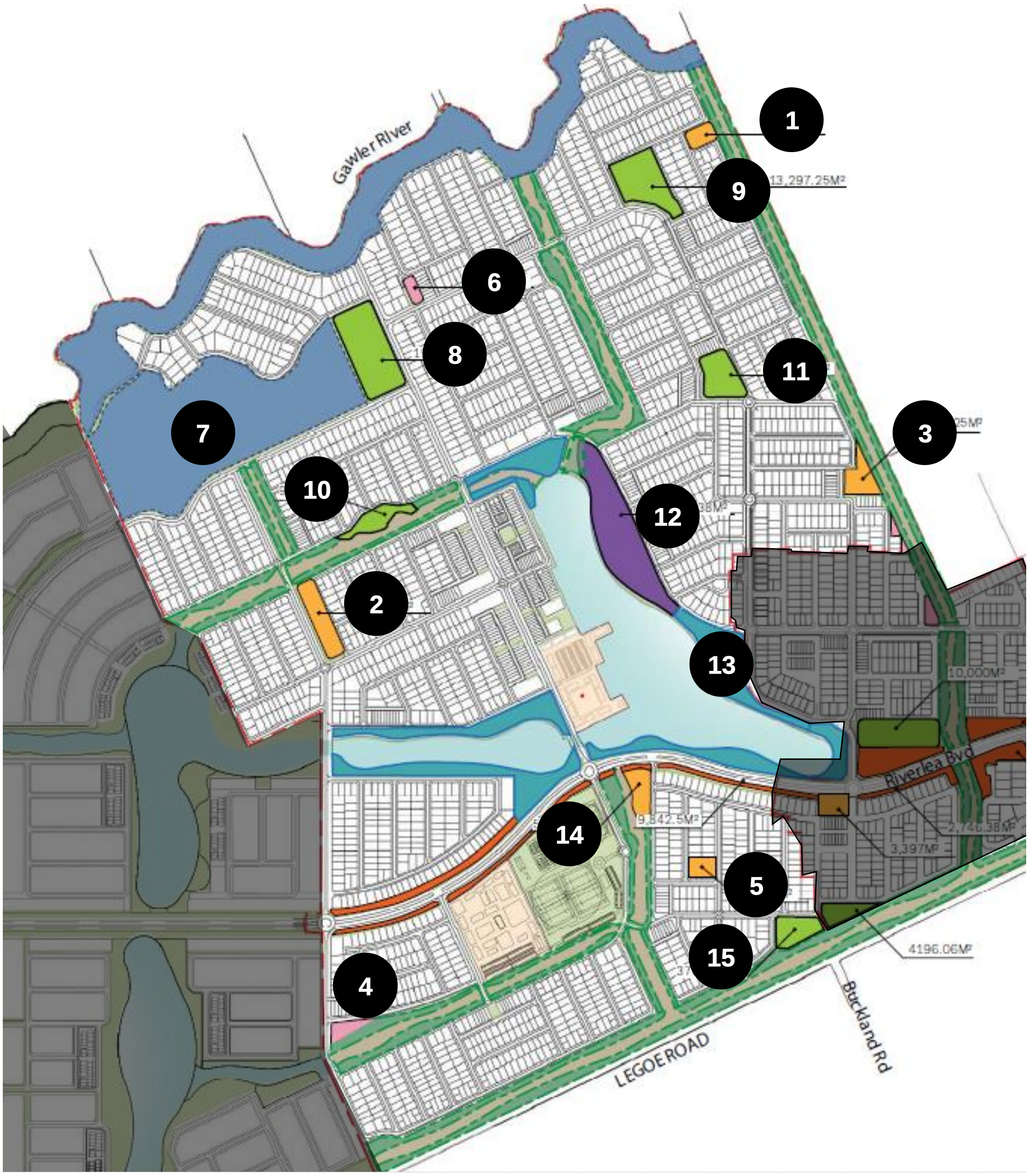
This report sets out an assessment of the anticipated parking implications of the parks and reserves proposed in Precinct 2, including consideration of the following:

- Identify all proposed parks and reserves for location/use/activities.
- Identify available parking supply on road network for each location
- Assess parking demands from adjacent uses (residential, commercial, etc).
- Assess parking demand and availability at each reserve location and identify any potential shortfalls/conflicts.
- Provide recommendations for parking supply at each site with consideration of on-street and off-street supply.

2. Precinct 2 Reserves

The location of the 15 reserves in Precinct 2 is shown in Figure 1.

Figure 1: Reserve Locations



No.	Park Name	No.	Park Name
1	Local Park 1	8	Karra Park
2	Local Park 2	9	Canoe Park
3	Local Park 3	10	Pebbles Park
4	Local Park 4	11	Island Park
5	Local Park 5	12	Lakes Park
6	Minor Park	13	Dragonfly Park
7	River And Conservation Areas	14	Local Park 6
		15	Honeyeater Park

3. On-Street Car Parking Review

The indicative capacity of on-street car parking has been identified for each reserve along the abutting/adjacent road frontages. The on-street car parking requirement was then calculated based on the requirement set out within the SA Planning and Design Code for residential allotments. The remaining balance of on-street car parking that remained and accordingly the amount of car parking spaces that remained available for the open spaces. This is shown in Table 1.

It is noted that Karra Park and the River & Conservation Area open spaces were combined as they directly abutted one another. This was also the case for Lakes Park and Dragonfly Park.

Table 1: On-Street Car Parking Availability

No.	Park Name	No. Dwellings Fronting	On-Street Car Parking Provision	On-Street Car Parking Rate	SA P & D Code On-Street Car Parking Requirement	Available Parking for Reserve
1	Local Park 1	8	27	SA Planning and Design Code Minimum 0.33 on-street spaces per dwelling on the site	3	24
2	Local Park 2	24	74		8	66
3	Local Park 3	17	21		6	15
4	Local Park 4	7	28		3	21
5	Local Park 5	9	22		3	13
6	Minor Park	11	4		4	-
7 & 8	River And Conservation Areas & Karra Park	41+	164+		14	150+
9	Canoe Park	22	74		8	66
10	Pebbles Park	8	17		3	14
11	Island Park	26	47		9	38
12 & 13	Lakes Park & Dragonfly Park	51	156		17	139
14	Local Park 6	5	47		2	45
15	Honeyeater Park	9	22		3	19

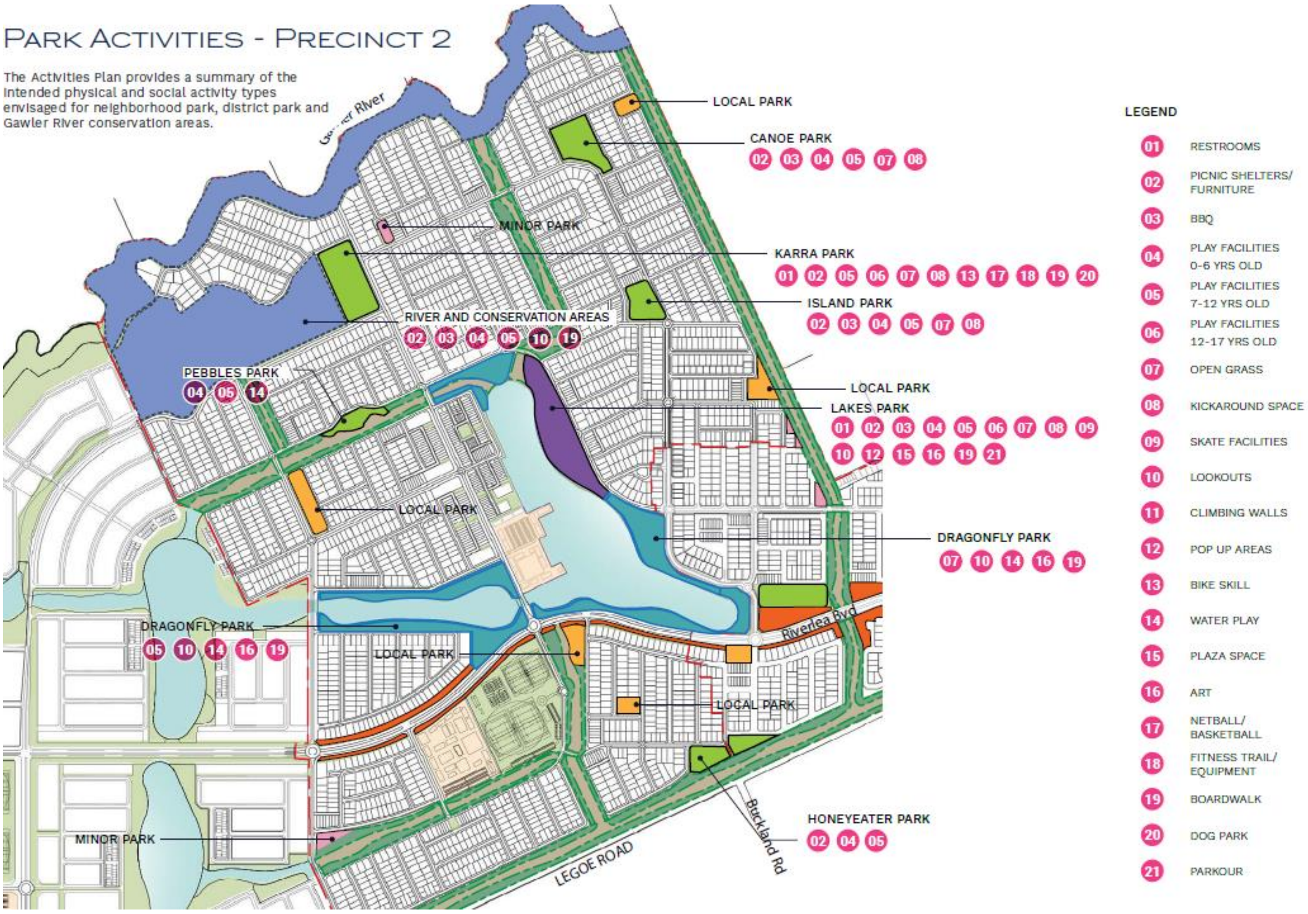
Based on the above, each of the public reserves were considered to have a reasonable balance of on-street car parking available for public use. However, it should be noted:

1. Some public reserves have high parking and traffic demand facilities which may result in significant on-street car parking at peak times.
2. The high demand for on-street car parking at public reserves is not ideal from a residential amenity perspective and does create a traffic impact for local streets.

4. Assessment Methodology

An assessment has been conducted for each of the public reserves to confirm the anticipated car parking demand. Each of the public reserves comprised one or more of the facilities that are listed in Table 2.

Table 2: Proposed Public Reserve Facilities



Each of the facilities were then assessed on the catchment size that the facility is expected to attract (Table 3) and the expected attendance to the facility (Table 4).

Table 3: Catchment – Parking Assessment Criteria

CATCHMENT	DESCRIPTION
LOCAL	Attract generally local residents within a short walk from the facility (within 300 metres)
NEIGHBOURHOOD	Generally attracts local residents within a moderate walk to short drive's distance (within 500 metres)
DISTRICT	Attracts local residents, but also people from outside the catchment area, which are required to drive (within 2 kilometres)

Table 4: Attendance – Parking Assessment Criteria

Attendance	Description
Low	Facility doesn't draw people, but rather is ancillary to other facilities
Moderate	Attracts a reasonable amount of people to the facility, but generally only within the immediate local catchment
High	Attracts a larger number of people both the local, district and regional catchments.

Both these criterion were then used to assess the anticipated parking demand for each facility, ranging from negligible, low, moderate and high, with an associated score ranking. This is shown in Table 5.

Table 5: Parking Demand – Parking Assessment Criteria

Parking Demand	Rating	Description
None	0	Facility unlikely to generate car parking demand
Low	1	Facility generates low car parking demand
Moderate	2	Facility generates moderate car parking demand
High	3	Facility generates high car parking demand

Based on the above, Table 6 considers the anticipated parking demand for each facility type, based on the catchment size, attendance and associated car parking demand.

Table 6: Facility Rating Matrix

No	FACILITY	CATCHMENT	ATTENDANCE	RATING
1	Restroom	Neighbourhood	Moderate	2
2	Picnic Shelters / Furniture	Local	Low	1
3	Bbq	Neighbourhood	Moderate	2
4	Play Facilities (0 – 6 Years)	Local	Moderate	2
5	Play Facilities (7 – 12 Years)	Local	Moderate	2
6	Play Facilities (13 – 17 Years)	Neighbourhood	Moderate	2
7	Open Grass	Local	Negligible	0
8	Kickaround Space	Local	Negligible	0
9	Skate Facilities	Neighbourhood	Moderate	2
10	Look Outs	District	Low	1
11	Climbing Walls	Neighbourhood	Moderate	2
12	Pop Up Areas	District	High	3
13	Bike Skill	Neighbourhood	Moderate	2
14	Water Play	Neighbourhood	Moderate	2
15	Plaza Space	Local	Negligible	0
16	Art	Local	Negligible	0
17	Netball / Basketball	Neighbourhood	Moderate	2
18	Fitness Trail / Equipment	District	High	3
19	Boardwalk	District	High	3
20	Dog Park	District	High	3
21	Parkour	Neighbourhood	Moderate	2

5. Reserve Assessment

Each reserve was assigned as either a minor, small, medium or a large park, which was determined based on the number of facilities each public reserve provided in addition to the points scored based on the facility parking demand (as assigned in Table 6). The methodology for assigning the classification of public reserves is set out in Table 7, while Table 8 considers the classification for each of the reserves.

Table 7: Public Reserve Classification

Park Classification	Rating	Description
MINOR	0	Park unlikely to generate car parking demand as it will only be used by adjacent residents. There are no car parking or traffic generators Car Parking can be accommodated on-street
SMALL	1 – 6	Park may generate a low car parking demand Park contains a small number of uses, which are low traffic and car parking generators Car Parking can be accommodated on-street
MEDIUM	7 – 12	Park may generate a medium car parking demand. Parking contains a moderate number of uses, some of which are moderate to high car parking and traffic generators Indented Car Parking should be considered On-Street DDA Car Parking should be considered
LARGE	13 +	Facility generates high car parking demand. Park comprises a number of uses, plenty of which are moderate to high traffic and car parking generators. Off-Street Car Parking should be considered On-Street DDA Car Parking should be considered Electric Vehicle parking / charging should be considered

Table 8: Public Reserve Assignment

No.	PARK NAME	FACILITIES	RATING	CLASSIFICATION
1	Local Park 1	Minimal	0	Minor
2	Local Park 2	Minimal	0	Minor
3	Local Park 3	Minimal	0	Minor
4	Local Park 4	Minimal	0	Minor
5	Local Park 5	Minimal	0	Minor
6	Minor Park	Minimal	0	Minor
7	River And Conservation Areas	Picnic Shelters / Furniture BBQ Play Facilities (0-6 Years) Play Facilities (7-12 Years) Lookout Boardwalk	11	Medium
8	Karra Park	Restrooms Picnic Shelters / Furniture Play Facilities (7-12 Years) Play Facilities (13-17 Years) Open Grass Kickaround Space Bike Skill Netball / Basketball Fitness Trail / Equipment Boardwalk Dog Park	20	Large
9	Canoe Park	Picnic Shelters / Furniture BBQ Play Facilities (0-6 Years) Play Facilities (7-12 Years) Open Grass Kickaround Space	7	Medium
10	Pebbles Park	Play Facilities (0-6 Years) Play Facilities (7-12 Years) Water Play	6	Small
11	Island Park	Picnic Shelters / Furniture BBQ Play Facilities (0-6 Years) Play Facilities (0-6 Years) Open Grass Kickaround Space	7	Medium
12	Lakes Park	Restrooms Picnic Shelters / Furniture BBQ Play Facilities (0-6 Years) Play Facilities (7-12 Years) Play Facilities (13-17 Years) Open Grass	19	Large

No.	PARK NAME	FACILITIES	RATING	CLASSIFICATION
		Kickaround Space Skate Facilities Lookouts Pop Up Areas Plaza Space Art Boardwalk Parkour		
13	Dragonfly Park	Open Grass Lookout Water Play Art Boardwalk	6	Small
14	Local Park 6	Minimal	0	Minor
15	Honeyeater Park	Picnic Shelters / Furniture Play Facilities (0-6 Years) Play Facilities (7-12 Years)	5	Small

With respect to above:

- Karra Park and Lakes Park were classified as large public reserves. As River & Conservation Areas and Dragonfly Park directly abutted these large public reserves respectively, these were amalgamated into the parking assessment.
- Canoe Park and Island Park were classified as medium public reserves.
- The remaining parks were deemed either small or minor public reserves.

6. Conclusions

Based on the above, the following conclusions have been made:

1. Karra Park / River and Conservations Areas was assessed together as a large public reserve recommended with:
    a. On-site car parking provision.
    b. Indented on-street car parking bays should also be considered to complement the on-site car parking.
    c. On-site parking for people with disabilities.
    d. Electric vehicle charging facilities should be considered.
2. Lakes Park / Dragonfly Park was assessed together as a large public reserve recommended with:
    a. On-site car parking provision.
    b. Indented on-street car parking bays should also be considered to complement the on-site car parking.
    c. On-site parking for people with disabilities.
    d. Electric vehicle charging facilities should be considered.
3. Canoe Park and Island Park are considered medium public reserves and therefore recommended with:
    a. On-street indented car parking.
    b. On-street parking for people with disabilities.
4. The remaining parks were classified as minor or small parks. On-street car parking will generally be sufficient however indented parking is recommended in some instances due to neighbouring uses. Parking for people with disabilities and electric vehicle charging facilities are not considered necessary at these locations.

A summary of the park analysis is shown in Table 9.

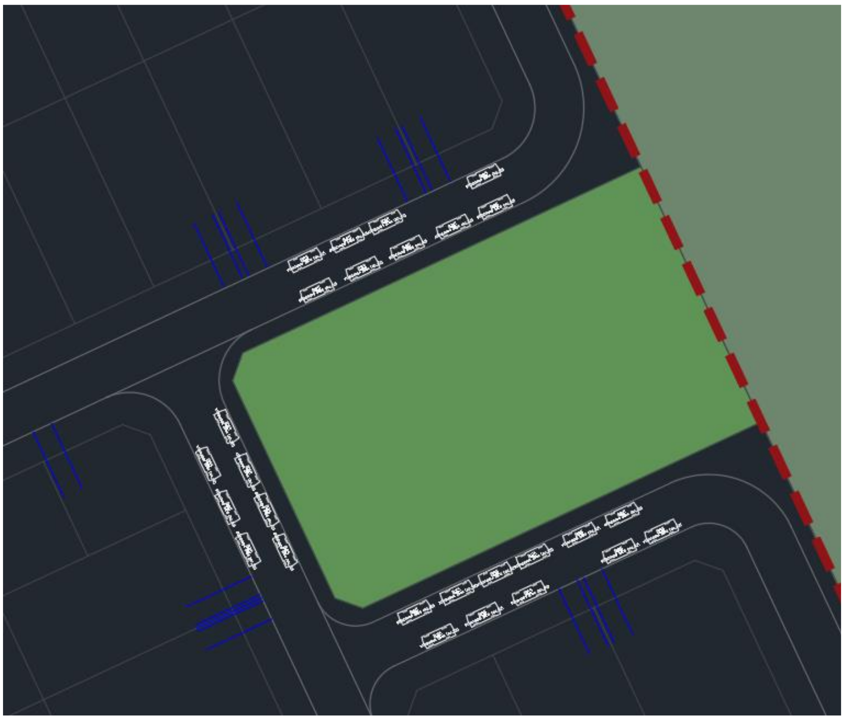
Table 9: Public Reserve Parking Recommendations Summary

Park	Rating	Classification	On-Street Car Parking	Indented On-Street Car Parking	Indented On-Street DDA Car Parking	On-Site Car Parking	On-Site DDA Car Parking	Electric Vehicle Parking / Charging
Local Park 1	0	Minor	✓					
Local Park 2	0	Minor	✓					
Local Park 3	0	Minor	✓					
Local Park 4	0	Minor	✓					
Local Park 5	0	Minor	✓					
Minor Park	0	Minor	✓					
River And Conservation Areas / Karra Park	31	Large		✓		✓	✓	✓
Canoe Park	7	Medium		✓	✓			
Pebbles Park	6	Small	✓					
Island Park	7	Medium		✓	✓			
Lakes Park / Dragonfly Park	25	Large		✓		✓	✓	✓
Local Park 6	0	Small		✓				
Honeyeater Park	5	Small	✓					

Appendix A Park Analysis

1. LOCAL PARK

- Classified as MINOR park
- Parking Demand not anticipated
- On-street car parking adequate

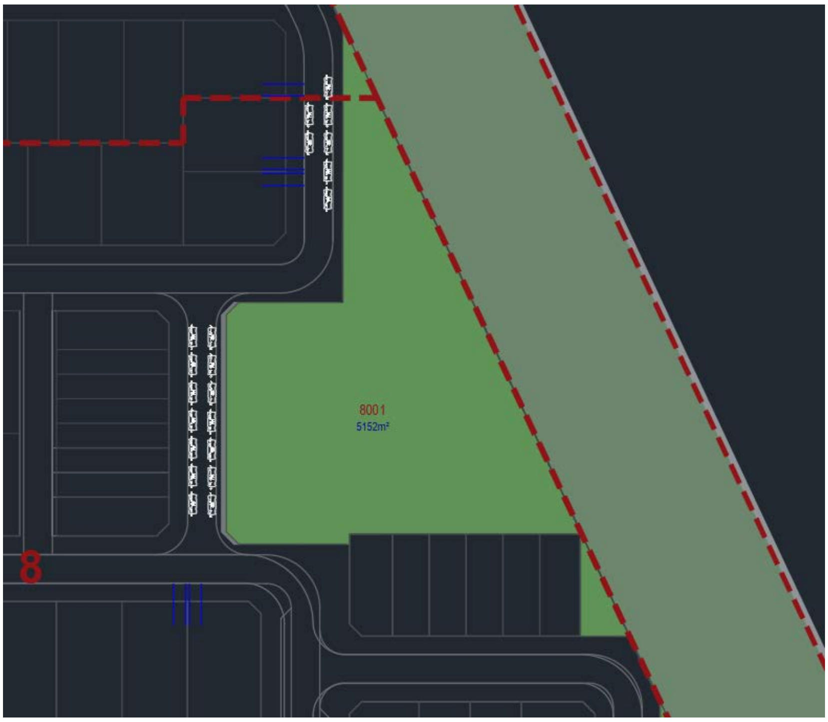


2. LOCAL PARK

- Classified as MINOR park
- Parking Demand not anticipated
- On-street car parking adequate

3. LOCAL PARK

- Classified as MINOR park
- Parking Demand not anticipated
- On-street car parking adequate

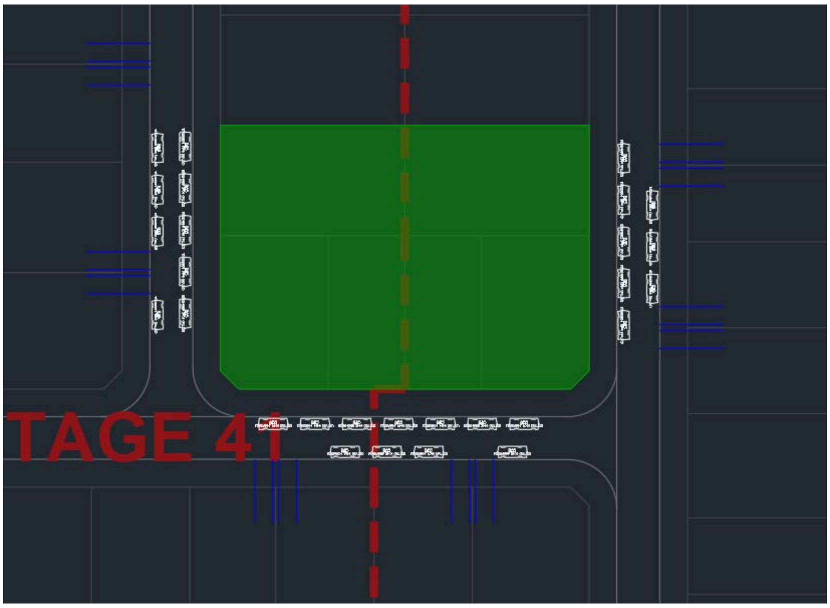


4. LOCAL PARK

- Classified as MINOR park
- Parking Demand not anticipated
- On-street car parking adequate

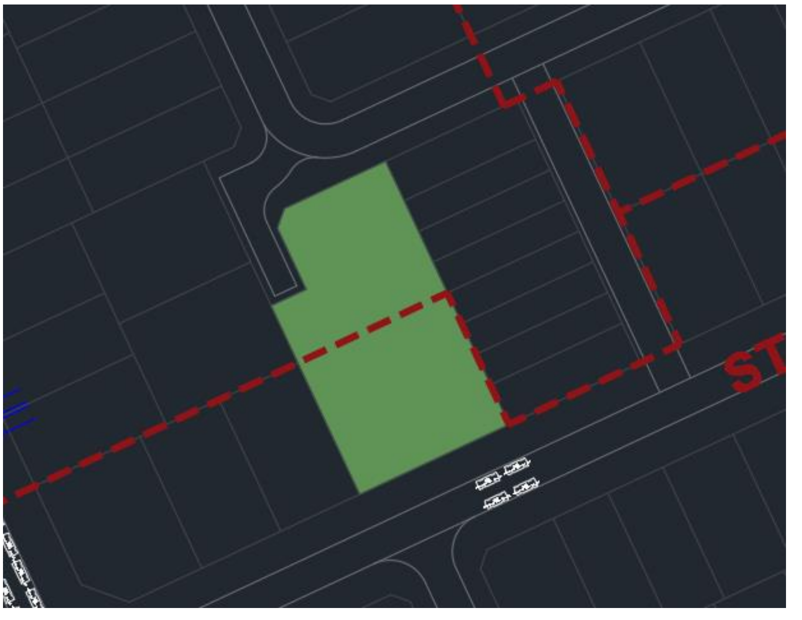
5. LOCAL PARK

- Classified as MINOR park
- Parking Demand not anticipated
- On-street car parking adequate



6. LOCAL PARK

- Classified as MINOR park
- Parking Demand not anticipated
- On-street car parking adequate



7 & 8. RIVER & CONSERVATION AREA AND KARRA PARK

- Combined for the purpose of assessment, as both parks abut each other
- Classified as LARGE park, with a combined rating of 31
- On-Street Car Parking may cause congestion on surrounding streets, and complaints from residents
- On-Site Parking recommended, which could facilitate both parks
- Off-Street DDA car parking recommended
- Provision for Electric Vehicle parking / charging to be considered
- Some indented on-street car parking could complement the parks well, particularly if there are larger events

9. CANOE PARK

- Classified as MEDIUM park, with a rating of 7
- Moderate parking demand anticipated
- On-Street DDA car parking to be considered
- Parking demand likely to be accommodated on-street, but indented is recommended:
  - Ensure easier traffic flow, and lesser impact on local residents
  - Safer park access arrangements

10. PEBBLES PARK

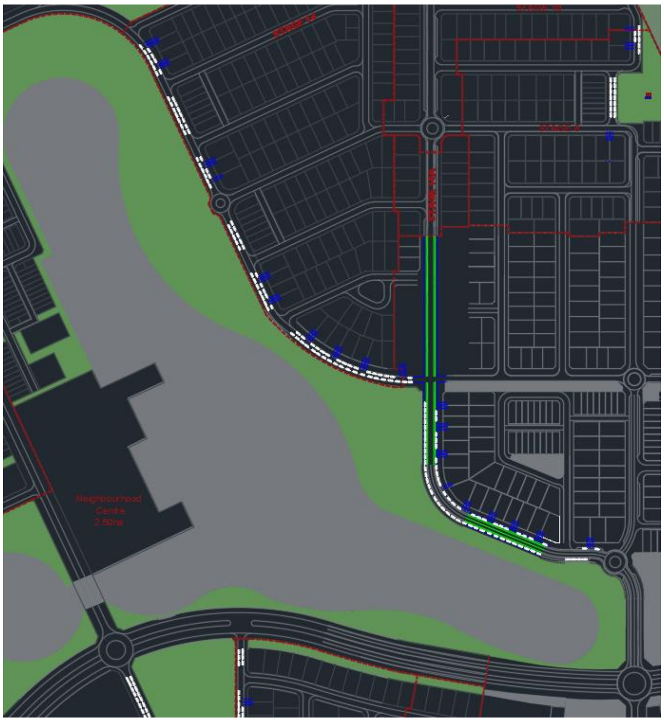
- Classified as SMALL park with a rating of 6
- Parking demand anticipated is minor
- On-street car parking adequate

11. ISLAND PARK

- Classified as MEDIUM park with a rating of 7
- Moderate parking demand anticipated
- On-Street DDA car parking to be considered
- Parking demand likely to be accommodated on-street, but indented is recommended:
  - Ensure easier traffic flow, and lesser impact on local residents
  - Safer park access arrangements

12 & 13. LAKES PARK AND DRAGONFLY PARK

- Combined for the purpose of assessment as parks directly abut each other
- Classified as LARGE park with a combined rating of 25
- Some indented on-street car parking already provided
- On-Street Car Parking only may cause congestion on surrounding streets, and complaints from residents
- On-Site Parking recommended, which could facilitate both parks
- On-site DDA car parking required
- Provision for Electric Vehicle parking / charging to be considered
- Some indented on-street car parking could complement the parks well, particularly if there are larger events



14. LOCAL PARK 6

- Classified as SMALL park with a rating of 6
- While a low parking demand is anticipated, indented is recommended:
  - Site is adjacent a proposed Sports and Community Park, which may generate considerable car parking in itself
  - Ensure easier traffic flow, and lesser impact on local residents
  - Safer park access arrangements

15. HONEYEATER PARK

- Classified as SMALL park with a rating of 6
- Parking demand anticipated is minor
- On-street car parking adequate

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