Development Plan,
135 Great Alpine Road, Lucknow
Brookfield Lakes Estate
Planning and Environment Act 1987,
East Gippsland Planning Scheme (Approved March 2025).
Development Plan Overlay Schedule 3.
Signed: Chris Stephenson
General Manager Place and Community
Date: 22 April 2025

No.	Drawing/ document	Prepared by	Date	Reference number	Version
1	Development Plan Report	Marshal	Apr-25	35753	5
2	Traffic Engineering	Traffix Group	24/07/2024	G34124L-02B	
3	Servicing Report	Marshal	23/10/2024	12576586	
4	WSUD Function Design Technical Report	Water Tech	28/8/24	19101R01v01	2
5	Revised Development Plan	Marshal	Apr-25		A

marshal

Brookfield Lakes

Great Alpine Road, Lucknow

Development Plan Report - Addendum
Development Plan Overlay – Schedule 3

April 2025

Development Plan,
135 Great Alpine Road, Lucknow
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Planning and Environment Act 1987,
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marshal

TABLE OF CONTENTS

1. INTRODUCTION

An amendment to the development plan has been prepared by Marshal Melbourne Pty Ltd on behalf of the property owner Brookfield Lakes Pty Ltd. This report should be read as an addendum to the initial Development Plan Report prepared by SMEC Urban (July 2011) together with the appended technical which formed part of the initial approved Development Plan.

The purpose of the amendment to Development Plan is to:

- Revise reference to low density residential lots (lots over 1000sqm), to allow for all lots to be delivered in accordance with the General Residential Zone provisions, except those which abut the north western boundary;
- Adjust road layouts to remove cul-de-sacs and replace with an increased permeable layout promoting vehicular and pedestrian connectivity; and
- Provide for a minor increase in the area (108.5sqm) of the northern open space area in which the artefact scatter is located in order to comply with the approved CHMP which applies to the land.

Since the approval of the initial Development Plan the land has been rezoned (re-named) as General Residential and Commercial 1 from the initial zones of Residential 1 and Business 1 Zones (part of the Vic Government zones and provisions reform). More recently (August 2024) the Erosion Management Overlay which applies to the land was decreased from applying to the entirety of the site to only the areas in which the severity of was deemed to require additional management measures.

Importantly to note are that no changes are sought to the General Residential Zone that applies to the majority of the land.

The proposed revisions will result in an approximate increase in the number of possible allotments from 426 to 511 (with an average lot density between 13 to 14 lots per net developable hectare) through efficiencies gained from road alignment and the ability to deliver conventional residential for the majority of the residential area. The approximate lot number increase will result in an approximate average lot size of 550 square metres, with individual lots to vary in size depending on location and site constraints across the Development Plan Area.

Stages 1, 2, 3, 4 and 5 have either been developed or are in the process of being developed at the time of this amendment and this amendment does not seek to change the Development plan as it applies to already titled lots as part of the residential subdivision.

The amendment Development Plan Proposal relies on previously prepared reports and plans other than the appended technical report addendums comprising the following:

- Stormwater Strategy Addendum prepared by WaterTech;
- Traffic Report Addendum prepared by Traffix Group;
- Servicing Plan prepared by Marshal Melbourne;

A full list of the reports and whether they are to be updated through the provision of an addendum is provided in the below table.

Report	Whether amended addendum is relied upon
Flora and Fauna Assessment – Ecology and Heritage Partners (March 2007)	No revisions sought
Traffic Assessment – Traffix Group (March 2011)	Traffic Assessment addendum relied upon prepared by Traffix Group
Cultural Heritage Management Plan – Heritage Insight (February 2010)	No revisions sought
Water Sensitive Urban Design Functional Design Technical Report – Water Technology (May 2011)	An addendum is relied upon prepared by Water Technology
SMEC Servicing Plan	An addendum is relied upon prepared by marshal Melbourne

The Traffic Assessment addendum considers a revised delivery sequence where the delivery of the Eastwood Road Entry with approximately the 401st lot is assessed as an appropriate outcome.

2. PROPOSED REVISIONS

The below illustrates the locations of new road alignments (green arrows), removed road alignments (red arrows) and locations where 1000sqm + lots are no longer proposed to be provided.

3. PLANNING CONTEXT

3.1. Zoning

3.1.1. General Residential Zone

Since the initial Development Plan approval, the Residential 1 Zone components of the land have been re-zoned General Residential Zone as part of the State wide amendments to zoning revisions.

The purposes of the General Residential Zone are:

- To implement the Municipal Planning Strategy and the Planning Policy Framework.
- To encourage development that respects the neighbourhood character of the area.
- To encourage a diversity of housing types and housing growth particularly in locations offering good access to services and transport.
- To allow educational, recreational, religious, community and a limited range of other non-residential uses to serve local community needs in appropriate locations.

3.1.2. Commercial 1 Zone

Since the initial Development Plan approval, the Business 1 Zone components of the land have been re-zoned Commercial 1 Zone as part of the State wide amendments to zoning revisions.

The purposes of the Commercial 1 Zone are:

- To implement the Municipal Planning Strategy and the Planning Policy Framework.
- To create vibrant mixed use commercial centres for retail, office, business, entertainment and community uses.
- To provide for residential uses at densities complementary to the role and scale of the commercial centre.

3.2. Overlays

3.2.3. Erosion Management Overlay

Amendment C167 (approved 1 August 2024) made revisions to the Erosion Management Overlay which applied to the land by reducing the area in which the Erosion Management Overlay applied from applying to the entirety of the site to only the areas in which the severity of was deemed to require additional management measures.

The purpose of the Erosion Management Overlay is:

- To implement the Municipal Planning Strategy and the Planning Policy Framework.
- To protect areas prone to erosion, landslip, other land degradation or coastal processes by minimising land disturbance and inappropriate development.

3.2.4. Vegetation Protection Overly – Scheule 1

The purposes of the Vegetation Protection Overlay is:

- To implement the Municipal Planning Strategy and the Planning Policy Framework.
- To protect areas of significant vegetation.
- To ensure that development minimises loss of vegetation.
- To preserve existing trees and other vegetation.
- To recognise vegetation protection areas as locations of special significance, natural beauty, interest and importance.
- To maintain and enhance habitat and habitat corridors for indigenous fauna.
- To encourage the regeneration of native vegetation.

3.2.5. Development Plan Overlay

The purposes of the Development Plan Overlay are:

- To implement the Municipal Planning Strategy and the Planning Policy Framework.
- To identify areas which require the form and conditions of future use and development to be shown on a development plan before a permit can be granted to use or develop the land.
- To exempt an application from notice and review if a development plan has been prepared to the satisfaction of the responsible authority.

A permit must not be granted to use or subdivide land, construct a building or construct or carry out works until a development plan has been prepared to the satisfaction of the responsible authority.

This does not apply if a schedule to this overlay specifically states that a permit may be granted before a development plan has been prepared to the satisfaction of the responsible authority.

A permit granted must:

- Be generally in accordance with the development plan.
- Include any conditions or requirements specified in a schedule to this overlay.

The development plan may be amended to the satisfaction of the responsible authority.

Schedule 3 to the Development Plan Overlay sets out the requirements for a development plan and the principles to which it must comply. The principles have been included in section 5 of this report with an assessment of each principle against the proposed amendments.

4. MUNICIPAL PLANNING STRATEGY

Clause 02.01 Settlement

In 2017, there were more than 46,000 people living and working in the Shire and this is forecast to grow to about 50,500 by 2027 and 56,000 by 2037. The main towns in East Gippsland are the 'growth area towns' of Bairnsdale, Lakes Entrance and Paynesville. Bairnsdale has the largest population and is the regional retail and service centre for the Shire.

Clause 02.03-1 Growth Area Towns

Bairnsdale is the gateway to East Gippsland and is the largest town in the Shire providing major commercial, retail, educational and public-sector services. Council's strategic directions for the growth area towns are:

- Encouraging population growth and development on fully serviced residential land in the growth area towns.
- Encouraging a range of housing types and styles including higher density dwellings in town centres.
- Encouraging development of East Gippsland as a retirement destination.
- Reinforcing Bairnsdale as the main commercial, retail and service centre for East Gippsland.

5. PLANNING POLICY FRAMEWORK

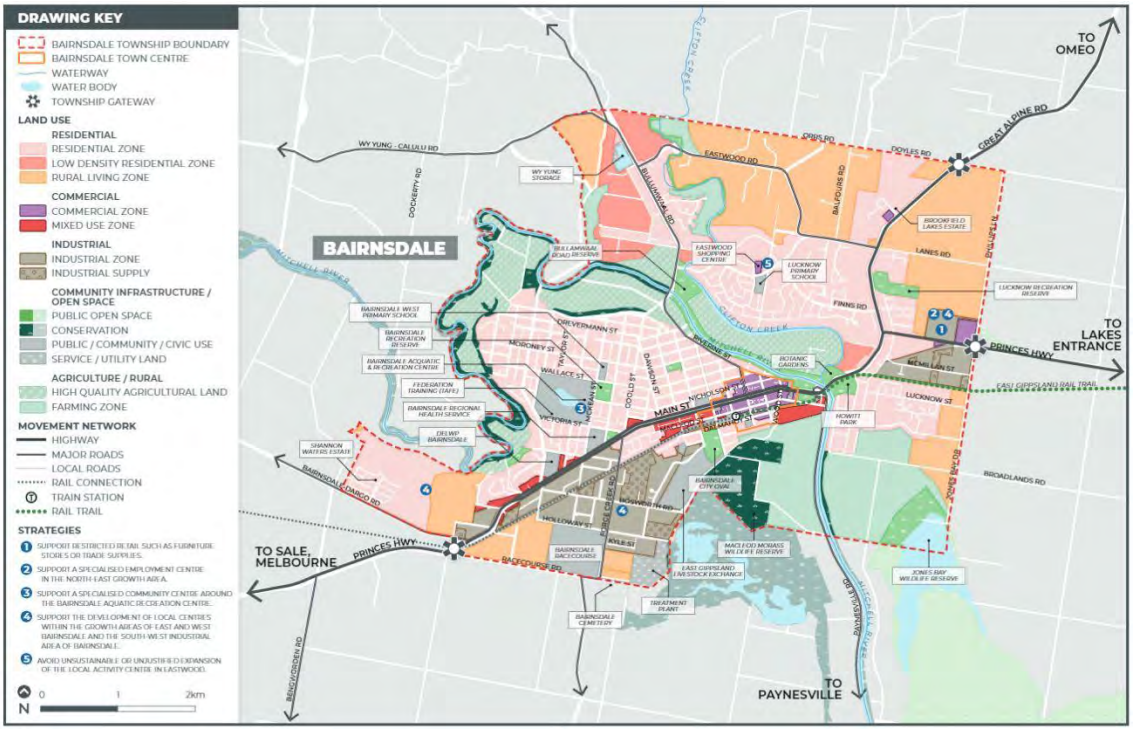
Since the initial Development Plan Approval, the Esat Gippsland Planning Scheme Planning Policy Framework has been standardized and thus the updated Planning Policy Framework is provided below. The revised Planning Policy Framework continues to provide strong support for the development of the land for residential purposes, with the site specifically identified and addressed in the Bairnsdale Framework Plan.

Clause 11.01-1L-02 Growth Area Towns

This policy applies to all land in the Bairnsdale Framework Plan and the East Bairnsdale – Lucknow Precinct Plan to this clause, with the following relevant strategies:

- Facilitate urban development for Bairnsdale within the existing town boundary.
- Encourage residential development in Brookfield Lakes and Shannon Waters Estate.
- Facilitate the East Bairnsdale – Lucknow precinct for urban growth with a mix of activities including residential, local area commercial and industrial.
- Provide roads within the precinct for connectivity, efficient access and dispersal of traffic.
- Preserve remnant vegetation and incorporate it into public open spaces.
- Design drainage in subdivisions to support water management plans.

- Provide urban infrastructure to support new residential and business development.



Source: Bairnsdale Framework Plan – East Gippsland Planning Scheme – Clause 11.01-1L-02

Clause 16.01-1S Housing Supply

The Clause seeks to facilitate well-located, integrated and diverse housing that meets community needs, through the following strategies:

- increase the proportion of housing in designated locations in established urban areas (including under-utilised urban land) and reduce the share of new dwellings in greenfield, fringe and dispersed development areas.
- Encourage higher density housing development on sites that are well located in relation to jobs, services and public transport.
- Identify opportunities for increased residential densities to help consolidate urban areas.
- Facilitate diverse housing that offers choice and meets changing household needs by widening housing diversity through a mix of housing types.

Clause 16.01-1S Housing Supply

The clause seeks to deliver more affordable housing closer to jobs, transport and services, through the following strategies:

- Ensuring land supply continues to be sufficient to meet demand.
- Increasing choice in housing type, tenure and cost to meet the needs of households as they move through life cycle changes and to support diverse communities.

6. CLAUSE 56 PRELIMINARY ANALYSIS

The following is provided as a general high level assessment of the amendment's ability to continue to achieve the requirements of Clause 56 which will be assessed in greater detail at the Planning Permit Application Stage.

Subdivision Site and Context Description and Design Response	The proposed amendments to the road alignments and removal of the majority of the low-density housing lots continues to align with the purpose of the zone which is to provide for the development that respects the character of the area and housing growth in locations offering good access to services. The road alignments in particular will provide for the development of lots which align with the existing topographical features on the land.
Policy Implementation	The continuing development of the land under the proposed amendments will achieve the strategic objectives of the East Gippsland Planning Scheme which make specific reference to encouraging housing at Brookfield Lakes at Clause 11.
Liveable and Sustainable Communities.	The proposed revisions to the development plan which remove multiple cul-de-sacs will contribute to creating a more compact and walkable neighbourhood.
Lot Design	The amendment seeks to provide a more transparent street network through the removal of cul-de-sacs ensuring appropriate street orientations.
Urban Landscape	The proposed amendments do not seek to reduce the amount of open space provided across the site or alter how landscaping of parks or streets is undertaken, which will continue to be delivered on a stage by stage basis.
Access and Mobility Management	The proposed revisions to the development plan which remove multiple cul-de-sacs will contribute walking and cycling objectives through a more transparent street network. Streets will continue to be delivered in accordance with Clause 56 and the Infrastructure Design Manual.
Integrated Water Management	An addendum to the Stormwater Management Plan has been prepared and is enclosed with this amendment to the Development Plan.
Site Management	The Development the site will continue to be undertaken with Council requirements.
Utilities	A servicing plan has been prepared and is enclosed with this amendment to the Development Plan.

7. DEVELOPMENT PLAN ASSESSMENT

This section addresses the amended Development Plan against the provision and requirement of Schedule 3 to the Development Plan Overlay in the East Gippsland Planning Scheme. The table outlines how the amendment seeks to align with each of the Development Plan Principles listed in the Schedule.

Development Principles	Response
A subdivision layout which responds to the features of the land as identified in the Site Analysis Plan.	The overall development area has not been altered significantly by the proposed development plan amendment, with road alignments adjusted to allow improved gradients across allotments as a result of detailed engineering work having now been undertaken.
A subdivision layout which is sympathetic to remnant native vegetation, particularly mature indigenous trees and vegetation, within the abutting road reserves.	No additional native vegetations is proposed to be removed as a result of the revisions proposed by this Development Plan amendment.
A network of open space across the site.	The open space network will remain predominantly the same other than a small increase in the northern artefact scatter site to align with the approved CHMP. The central waterway which remains key to the development plan area will remain as previously approved.
Usable and attractive open space nodes along Goose Gully reserve.	
That new development incorporates water sensitive urban design principles for appropriate drainage solutions.	The revised development plan does not propose an amendment to the previously approved WSUD measures forming part of the Goose Gully reserve.
That indigenous plant species should be used for landscaping of public open space reserves and for revegetation in accordance with the Department of Environment, Land, Water and Planning's *Revegetation Planting Standards.*	The revised development plan does not seek to revise planting schedules or programs and the development of the site will continue to be landscape in accordance with Council approvals and requirements.

That the subdivision layout is designed to facilitate the provision of efficient and effective pedestrian, bicycle and vehicle movements, and to ensure connectivity with the existing public transport network.	The revised development plan will provide for an improved street network, with a reduced number of cul-de-sacs ensuring an improved level of connectivity.
The subdivision provides appropriate linkages and interfaces with adjoining land.	All previously approved connections to abutting properties are retained in the approximate locations of the existing approved development plan.
Development of land in a manner that protects areas prone to erosion, landslip or other land degradation process.	The developable area of the land is not increased by the proposed revisions to the development plan and the site will continue to be developed in accordance with the approved Erosion Management Plan practices.
No direct access for individual lots being provided to the Great Alpine Road.	The revised development plan does not seek to allow for direct access onto the Great Alpine Road.

On other items to be considered under the Development Plan Overlay Schedule 3, the following is noted:

- No revisions are sought to the approved **Cultural Heritage Management Plan** (CHMP), with the proposed revision to the north reserve seeking to align the development plan with the approved CHMP;
- No revisions are sought to the approved **Erosion Management Plan** endorsed under the multi-lot subdivision permit which applies to the land.
- No reduction in the **Public Open Space** is to be provided as part of the development is sought;
- A revised **Stormwater Management Plan** has been prepared and is enclosed to address the revised street network and the removal of refence to the delivery of low density housing;
- A revised **Traffic Management Plan** has been prepared and is enclosed to address the revised street network and the removal of refence to the delivery of low density housing;
- A revised **Servicing Report (infrastructure plan)** has been prepared and is enclosed to address the revised street network and the removal of refence to the delivery of low density housing;

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General Manager Place and Community
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8. CONCLUSION

This Development Plan report has been prepared as an addendum to the initial Development Plan Report prepared by SMEC Urban (July 2011) to support minor amendments to the development plan to provide for an improved road network and removal of the majority of low density (>1000sqm) lots.

The proposed amendments will allow for a more efficient road network and thus residential lot layout, with lots able to be developed in accordance with the current planning zone requirements. The proposed revisions will continue to align with the purposes and expectations of the General Residential Zone and Planning Policy Framework (Clause 11.01-1L-02) to provide for residential development in Brookfield Lakes Estate.

The proposed revisions will still align with the development principles to be implemented as part of the Development Plan approval as required by Schedule 3 to the overlay.

Our Reference: G34124L-02B

24 July 2024

Brookfield Land Co Pty Ltd
c/o Marshal
Suite 201, Level 2 448 St Kilda Road
MELBOURNE VIC 3004

Attention: Tim Watson

Dear Tim,

135 Great Alpine Road, Lucknow – Amended Residential Development Traffic Engineering Assessment

Further to your instructions, please find following our review of the amended residential development at 135 Great Alpine Road, Lucknow.

The following report provides a traffic engineering assessment of the traffic impact associated with the amended application, noting that Traffix Group prepared the traffic report for the currently approved development (Our reference: GRP09519R#2, dated March 2011). More recently, Traffix Group also prepared a traffic addendum that included an updated detailed assessed the site's proposed intersection with Eastwood Drive.

Our assessment is as follows.

Existing Conditions

Subject Site

The subject site is generally bounded by Doyles Road (north), Great Alpine Road (southeast), Eastwood Road (south), and Balfours Road (west) in Lucknow. The subject site is largely vacant, with a number of dwellings having been developed and the site connection (Lomandra Boulevard) with Great Alpine Road having been constructed.

An aerial photograph of the subject site is shown at Figure 1.

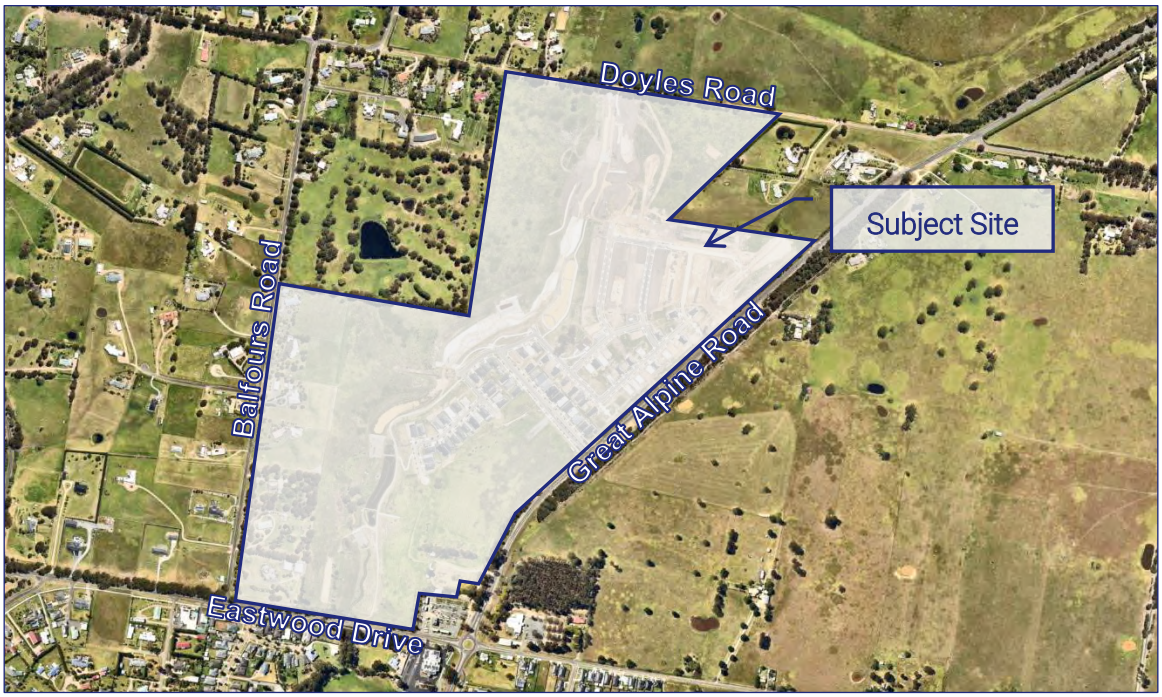

Figure 1: Aerial Photograph *Source: NearMap (Feb 2024)*

Road Network

Great Alpine Road is classified as a 'Transport Zone 2 – Principal Road Network (TRZ2)' road and is an arterial road under the control of the Department of Transport and Planning (DTP). The road is oriented in a general southwest-northeast direction along the eastern boundary of the site.

In the vicinity of the subject site, Great Alpine Road accommodates one through traffic lane in each direction and a 2m wide shoulder on each side of the road. On-street car parking is not permitted along the length of the road past the subject site.

A posted speed limit of 70km/h applies to Great Alpine Road past the subject site.

Existing Traffic Volumes

Traffix Group conducted turning movement counts at the Great Alpine Road/Lomandra Boulevard intersection on Tuesday 23rd April 2024 between the following times:

- 7:30am-9:30am, and
- 4pm-6pm.

The AM and PM peak hours were determined to be between 7:45am-8:45am and 4:30pm-5:30pm, respectively.

The existing traffic volumes at the AM and PM peak hours are shown at Figure 2.

Full output of the turning movement counts are attached at Appendix A.

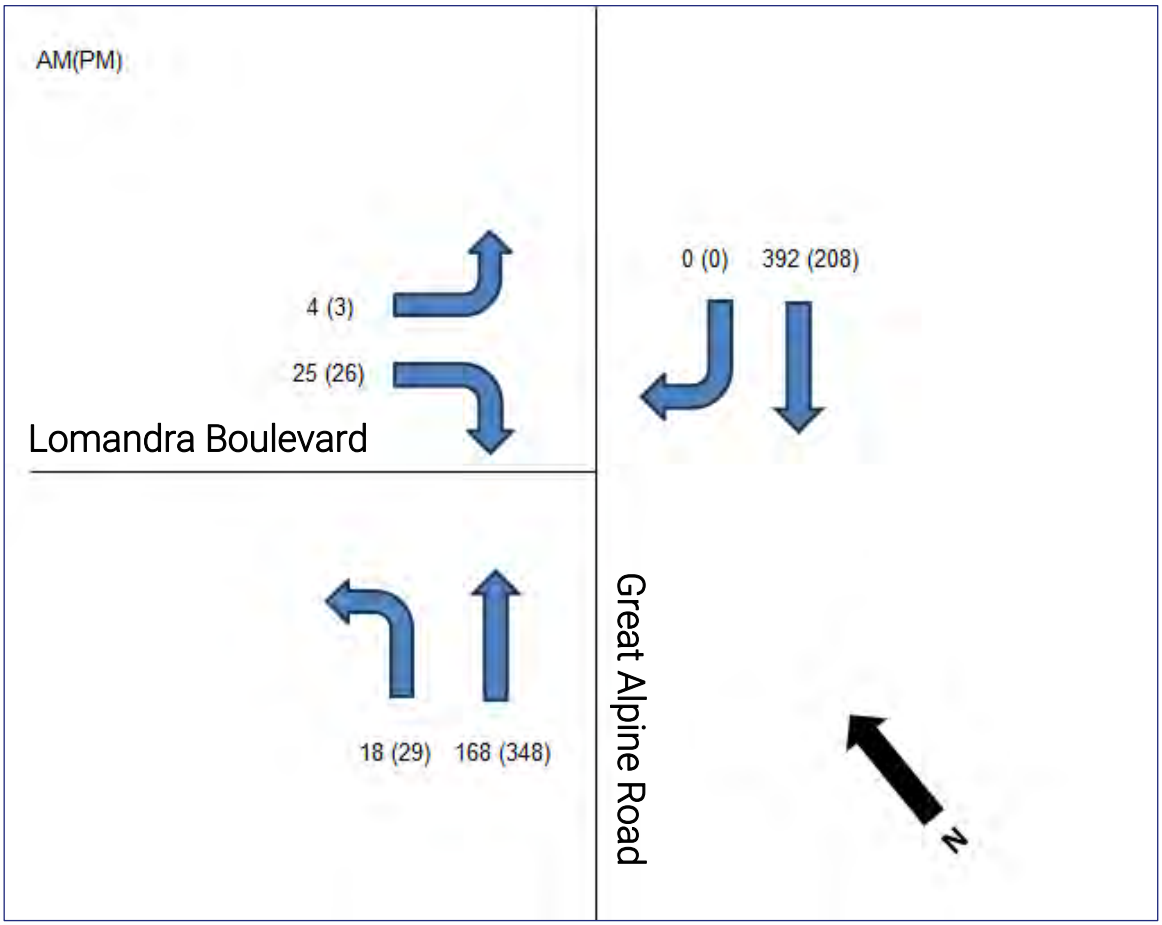

Figure 2: Existing Peak Hour Traffic Volumes - Great Alpine Road/Lomandra Boulevard Intersection

Proposal

The amended proposal consists of the development of 86 additional allotments (from 426 lots to 511 lots). Additionally, the sequence of the development stages has been changed from developing in numerical order to the following:

Stage Number	Sequence
1-5	1-5 (as previously approved)
6	Stage 10
7	Stage 11
8	Stage 8
9	Stage 9
10	Stage 6
11	Stage 7

The amended development plan, prepared by Marshal, are attached at Appendix B.

It should be noted that the other components of the development including the superlot and local activity centre will remain as approved.

Traffic Considerations

Traffic Generation

Residential Subdivision:

A rate of 10 trip ends per household per day is adopted for the residential subdivision and one (1) vehicle trip end per household per day in each of the AM and PM peak hours, consistent with our previous assessment. Based on the amended proposal to develop 511 residential lots, this results in a daily traffic generation of 5,110 vehicle movements or 511 vehicles trips in the AM and PM peak hours.

In the interim stage, prior to the construction of Stage 9 when the site connection with Eastwood Drive is identified, the amended proposal is predicted to generate a total daily traffic generation of 3,980 vehicle movements, including 398 vehicle trips in each of the AM and PM peak hours.

Retirement Village

A total of 810 daily vehicle trip ends, including 81 vehicle trips ends during each of the AM and PM peak hours, will be adopted for the retirement village consistent with the assessment that was undertaken to inform the currently approved scheme. For the purposes of this assessment, we will conservatively assume that the retirement village is fully developed and occupied prior to Stage 9.

Local Activity Centre (LAC)

A total of 906 daily external vehicle trips ends, including 94 external vehicle trip ends during the PM peak hour will be adopted for the LAC consistent with the assessment for the approved scheme. It is noted that the traffic generated by the LAC in the AM peak hour is considered negligible, consistent with what was adopted for the currently endorsed plans. For the purposes of this assessment, we will conservatively assume that the LAC is fully developed and occupied prior to Stage 9.

Traffic Distribution

Due to the increased number of allotments, there will be an increase in the number of vehicle movements turning to and from the site. The additional allotments are generally located in closer proximity of the Great Alpine Road/Lomandra Boulevard intersection which means it will ultimately carry more traffic than predicted as part of the assessment that was undertaken to inform the currently approved scheme. However, the operation of this intersection will be most critical in the interim prior to the Eastwood Road connection being provided[1]. Accordingly, we will analyse the interim scenario, i.e. assuming full

[1] Predictions suggest that in the order of 80-90% of the ultimate traffic generated by the site would use this intersection prior to the Eastwood Road connection being provided compared with in the order of 68% when both connections are provided.

development and occupation of Stages 1-8 prior to the Eastwood Road connection being provided. Furthermore, not only is it highly unlikely that all allotments within Stages 1-8 will be built on before the Eastwood Road connection is operational, but the creation of the Eastwood Drive connection is expected to result in a decrease in motorists turning directly right out of the site on Great Alpine Road, noting that this is the critical movement. Rather, some motorists who will need to undertake that movement in the interim are likely to instead use the future Eastwood Road connection in order to access areas to the south of the site.

Assumptions

- A directional split of 30% in/70% out in the AM peak has been adopted consistent with the assessments for the approved scheme.
- A directional split of 60% in/40% out in the PM peak has been adopted consistent with the assessments for the approved scheme.
- Directional splits have been adopted at the Great Alpine Road/Lomandra Boulevard intersection that are consistent with the turning movement counts results.
- We have conservatively applied a linear growth rate of 2% per annum for a period of 10 years to through traffic on Great Alpine Road, noting that the development of the site has already begun and land to the south is already largely developed.

On this basis of the aforementioned, the predicted in/out traffic distributions at the Great Alpine Road/Lomandra Boulevard intersection are shown at Figure 3 and Figure 4.

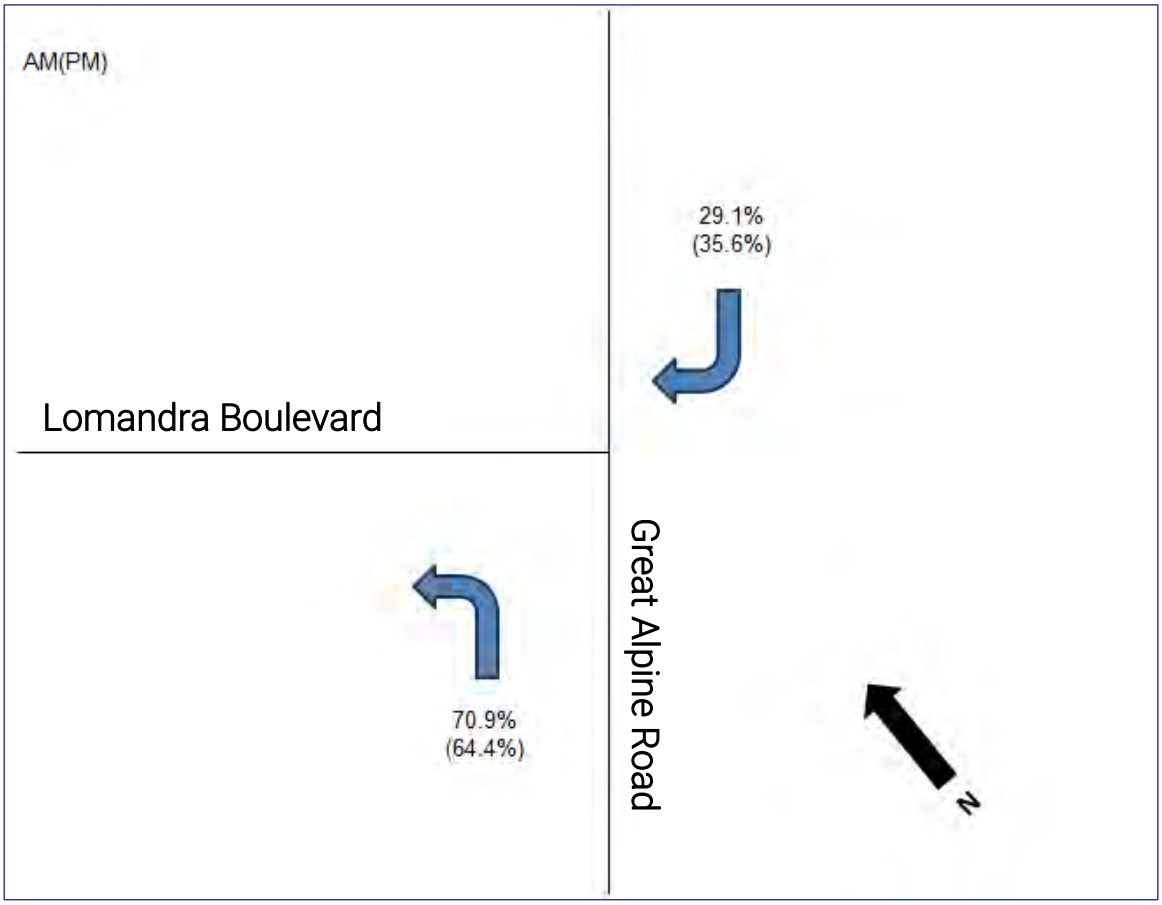

Figure 3: Stage 8 Post Development In Traffic Distribution - Great Alpine Road/Lomandra Boulevard Intersection

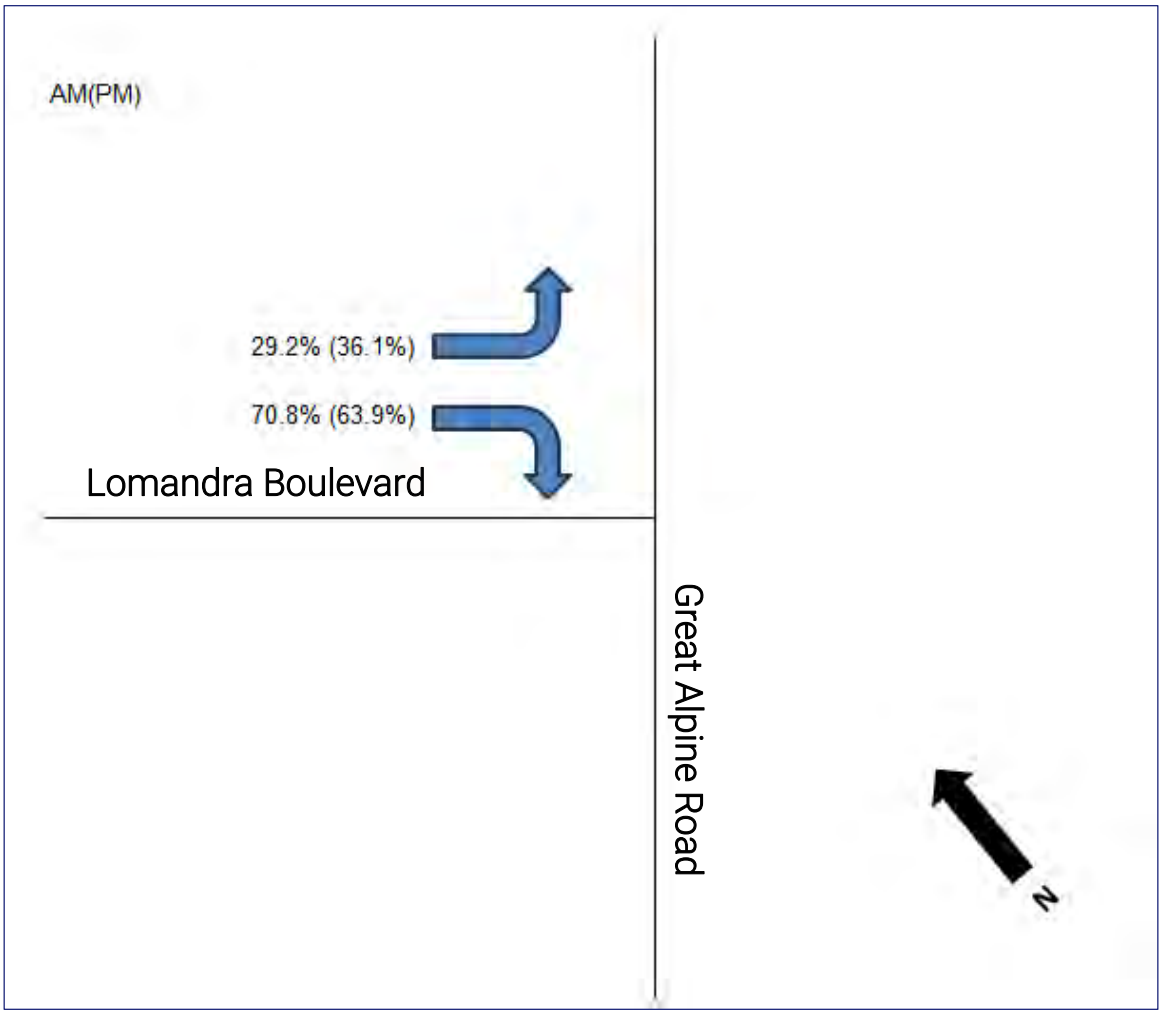

Figure 4: Stage 8 Post Development Out Traffic Distribution - Great Alpine Road/Lomandra Boulevard Intersection

Therefore, the overall post development traffic at the Great Alpine Road/Lomandra Boulevard Intersection is shown at Figure 5.

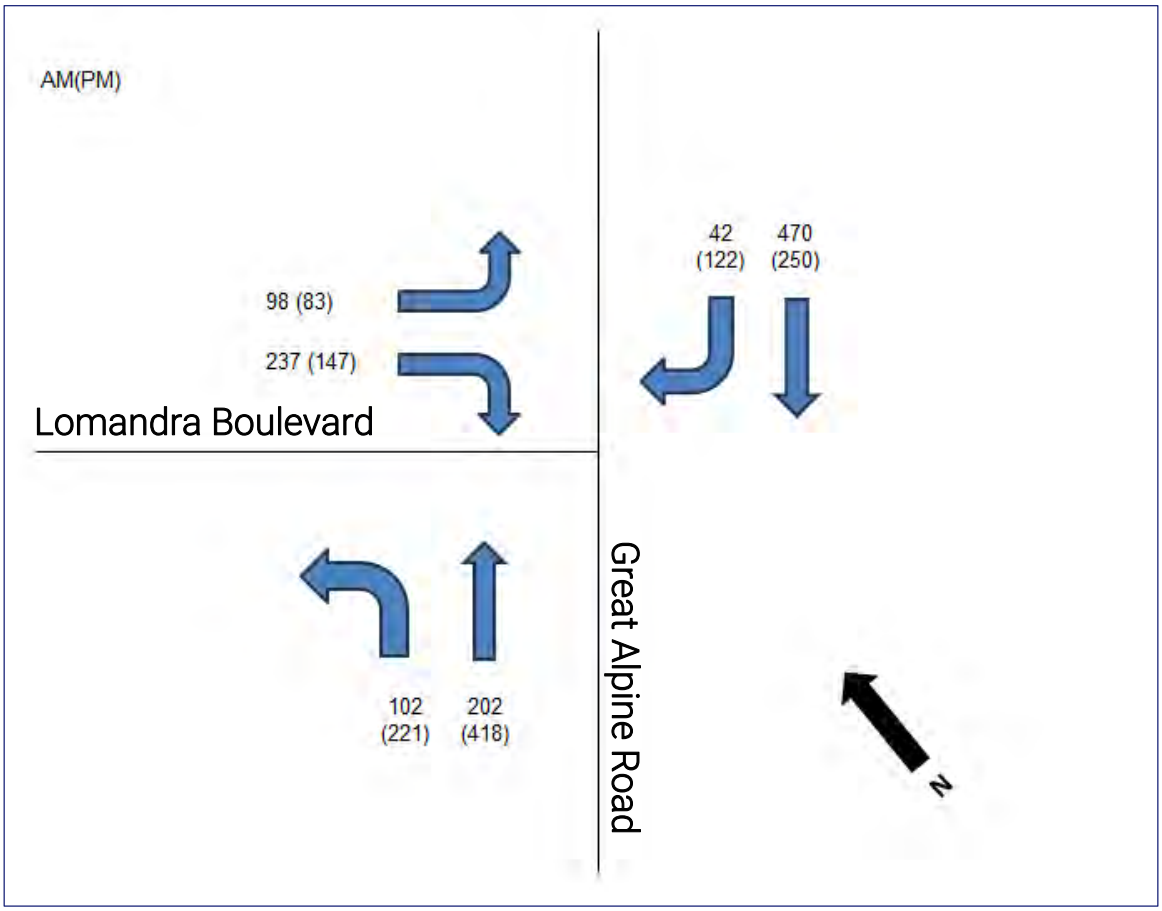

Figure 5: Stage 8 Post Development Traffic Volumes - Great Alpine Road/Lomandra Boulevard Intersection

Intersection Analysis

SIDRA Intersection 9 (SIDRA) is a computer simulation package which assesses the operating performance of intersections and road networks and has been used to undertake an assessment of the conservatively predicated operating conditions of the proposed Great Alpine Road/Lomandra Boulevard intersection.

A summary of the key SIDRA outputs are as follows:

- Degree of Saturation (DoS) – the ratio traffic volume to maximum capacity for a particular turning movement.
- Average Delay (Avg. Delay) – the average delay in seconds for a vehicle making a particular turning movement.
- 95th Percentile Queue (95% Queue) – the 95th percentile queue length is the length in metre which 95 per cent of all observed cycle queues fall below (or 5% exceed) during the peak analysis period.

A DoS of 0.90 for unsignalised intersections is considered the typical 'acceptable limit' for intersection operation, beyond which queues, and delays increase disproportionally with the addition of further traffic.

All SIDRA default values have been adopted except for the critical gap and follow up headway values which have been adopted based on the acceptable range as outlined in the *Guide to Road Design Part 4A: Unsignalised and Signalised Intersections*. The following critical gap and follow up headway values have been adopted:

- 4 seconds and 2 seconds for right turns across a single traffic stream,
- 5 seconds and 3 seconds for left turns into a single traffic stream, and
- 5 seconds and 3 seconds for right turns across two lane/two way traffic streams.

It is important to note that the critical gap and follow up headway values presented above are consistent with what was adopted for the assessment that was undertaken for the currently endorsed.

Full outputs of the SIDRA analysis are attached at Appendix C, with a summary of key results for the AM and PM peak hours presented at Table 1.

Table 1: SIDRA Results Summary – AM & PM Peak Hour

Intersection Leg/Movement		AM Peak Hour			PM Peak Hour		
		DoS	Avg. Delay (s)	95% Queue (veh)	DoS	Avg. Delay (s)	95% Queue (veh)
Great Alpine Road (North)	Through	0.26	0	0	0.14	0	0
	Right	0.03	6	0	0.15	9	1
Lomandra Boulevard (West)	Left	0.70	13	6	0.36	9	2
	Right	0.70	22	6	0.36	12	2
Great Alpine Road (South)	Through	0.11	0	0	0.23	0	0
	Left	0.06	6	0	0.12	6	0

135 Great Alpine Road,
Lucknow

Development Plan,
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General Manager Place and Community
Date: 22 April 2025

As shown in Table 1 and Appendix C, the unsignalised intersection is expected to operate well within the acceptable operating limits under the interim scenario where it is the only access to and from the site based on the amended development sequencing as proposed. In particular, small delays, short queues and low Degree of Saturation is anticipated.

Based on the outcome of our assessments and analysis, we see no traffic engineering reason to suggest that the amended development as proposed should not be approved.

Please contact Nathan Woolcock or Bryan I at Traffix Group if you require any further information.

Yours faithfully,

TRAFFIX GROUP PTY LTD

NATHAN WOOLCOCK (RPE6892)

Director

www.traffixgroup.com.au

Appendix A

Turning Movement Count Results

Development Plan,
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TRANS TRAFFIC SURVEY

trafficsurvey.com.au

TURNING MOVEMENT SURVEY

Intersection of Lomandra Bvd and Great Alpine Rd, Lucknc

GPS -37.805145, 147.651396

Date:	Tue 23/04/24
Weather:	Overcast
Suburban:	Lucknow
Customer:	N/A

North:	Great Alpine Rd
East:	N/A
South:	Great Alpine Rd
West:	Lomandra Bvd

Survey Period	AM:	7:30 AM-9:30 AM
	PM:	4:00 PM-6:00 PM
Traffic Peak	AM:	7:45 AM-8:45 AM
	PM:	4:30 PM-5:30 PM

All Vehicles

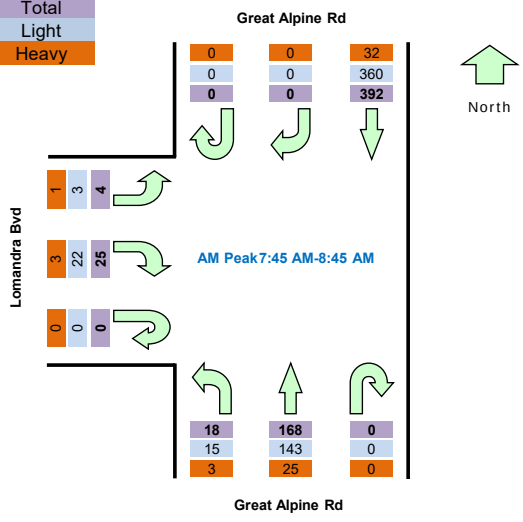
Time		orth Approach Great Alpine			uth Approach Great Alpine			est Approach Lomandra Bv			Hourly Total	
Period Start	**Period End**	**U**	**R**	**SB**	**U**	**NB**	**L**	**U**	**R**	**L**	**Hour**	**Peak**
7:30	7:45	0	1	78	0	42	4	0	4	0	589	
7:45	8:00	0	0	102	0	41	3	0	5	2	607	Peak
8:00	8:15	0	0	94	0	41	6	0	7	1	599	
8:15	8:30	0	0	117	0	28	5	0	7	1	558	
8:30	8:45	0	0	79	0	58	4	0	6	0	505	
8:45	9:00	0	0	84	0	56	2	0	3	0		
9:00	9:15	0	0	55	0	46	3	0	4	0		
9:15	9:30	0	0	58	0	37	7	0	3	0		
16:00	16:15	0	2	55	0	86	5	0	4	0	594	
16:15	16:30	0	0	55	0	79	7	0	5	0	591	
16:30	16:45	0	0	55	0	85	6	0	5	1	614	Peak
16:45	17:00	0	0	48	0	80	6	0	9	1	573	
17:00	17:15	0	0	54	0	83	8	0	4	0	515	
17:15	17:30	0	0	51	0	100	9	0	8	1		
17:30	17:45	0	0	45	0	53	7	1	5	0		
17:45	18:00	0	1	36	0	42	2	0	4	1		

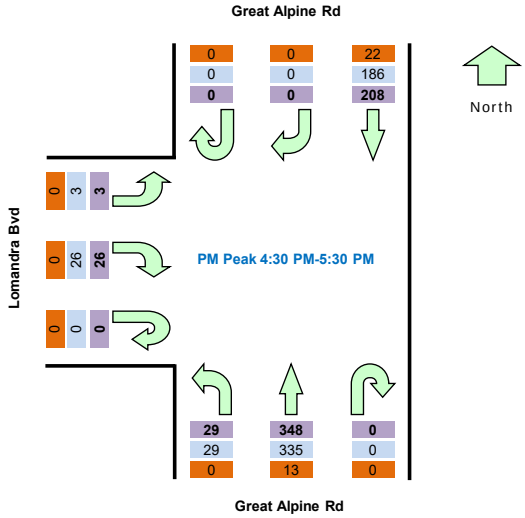
Peak Time		orth Approach Great Alpine			uth Approach Great Alpine			est Approach Lomandra Bv			Peak
Period Start	**Period End**	U	R	SB	U	NB	L	U	R	L	**total**
7:45	8:45	0	0	392	0	168	18	0	25	4	607
16:30	17:30	0	0	208	0	348	29	0	26	3	614

Note: Site sketch is for illustrating traffic flows. Direction is indicative only, drawing is not to scale and not an exact streets configuration.

Graphic

Total
Light
Heavy





Development Plan,
135 Great Alpine Road, Lucknow
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General Manager Place and Community
Date: 22 April 2025

Light Vehicles

Time		orth Approach Great Alpine			uth Approach Great Alpine			est Approach Lomandra Bv		
Period Start	Period End	U	R	SB	U	NB	L	U	R	L
7:30	7:45	0	0	74	0	35	4	0	4	0
7:45	8:00	0	0	94	0	32	3	0	4	1
8:00	8:15	0	0	83	0	35	4	0	6	1
8:15	8:30	0	0	110	0	26	4	0	6	1
8:30	8:45	0	0	73	0	50	4	0	6	0
8:45	9:00	0	0	80	0	46	1	0	3	0
9:00	9:15	0	0	49	0	40	2	0	4	0
9:15	9:30	0	0	51	0	30	5	0	2	0
16:00	16:15	0	2	49	0	82	5	0	4	0
16:15	16:30	0	0	49	0	77	7	0	5	0
16:30	16:45	0	0	45	0	83	6	0	5	1
16:45	17:00	0	0	44	0	77	6	0	9	1
17:00	17:15	0	0	50	0	79	8	0	4	0
17:15	17:30	0	0	47	0	96	9	0	8	1
17:30	17:45	0	0	44	0	53	7	1	5	0
17:45	18:00	0	1	36	0	42	2	0	4	1

Peak Time		orth Approach Great Alpine			uth Approach Great Alpine			est Approach Lomandra Bv			Peak
Period Start	Period End	U	R	SB	U	NB	L	U	R	L	total
7:45	8:45	0	0	360	0	143	15	0	22	3	543
16:30	17:30	0	0	186	0	335	29	0	26	3	579

Heavy Vehicles

Time		orth Approach Great Alpine			uth Approach Great Alpine			est Approach Lomandra Bv		
Period Start	Period End	U	R	SB	U	NB	L	U	R	L
7:30	7:45	0	1	4	0	7	0	0	1	0
7:45	8:00	0	0	8	0	9	0	0	1	1
8:00	8:15	0	0	11	0	6	2	0	1	0
8:15	8:30	0	0	7	0	2	1	0	1	0
8:30	8:45	0	0	6	0	8	0	0	0	0
8:45	9:00	0	0	4	0	10	1	0	0	0
9:00	9:15	0	0	6	0	6	1	0	0	0
9:15	9:30	0	0	7	0	7	2	0	1	0
16:00	16:15	0	0	6	0	4	0	0	0	0
16:15	16:30	0	0	6	0	2	0	0	0	0
16:30	16:45	0	0	10	0	2	0	0	0	0
16:45	17:00	0	0	4	0	3	0	0	0	0
17:00	17:15	0	0	4	0	4	0	0	0	0
17:15	17:30	0	0	4	0	4	0	0	0	0
17:30	17:45	0	0	1	0	0	0	0	0	0
17:45	18:00	0	0	0	0	0	0	0	0	0

Peak Time		orth Approach Great Alpine			uth Approach Great Alpine			est Approach Lomandra Bv			Peak
Period Start	Period End	U	R	SB	U	NB	L	U	R	L	total
7:45	8:45	0	0	32	0	25	3	0	3	1	64
16:30	17:30	0	0	22	0	13	0	0	0	0	35

Development Plan,
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General Manager Place and Community
Date: 22 April 2025

Appendix B

Amended Development Plan

Development Plan,
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Brookfield Lakes Estate
Planning and Environment Act 1987,
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DOYLES ROAD

BALFOUR ROAD

GOOSE GULLY
GOLF COURSE

Shared driveways subject to further design considerations

10m Landscape Buffer

Shared driveways subject to further design considerations

Shared driveways subject to further design considerations

10m Landscape Buffer

GREAT ALPINE ROAD

LOCAL ACTIVITY CENTRE (1.3ha) (Subject to future planning permit application)

Divided Boulevard (Main Entrance) Intersection upgraded as part of Stage 1

SUPER LOT (8.2ha) (Land subject to alternative Development Plan)

Upgrade existing Water Main to accommodate increased demand. Upgrade to East Gippsland Water standards.

10m Landscape Buffer

20m Landscape Buffer

EASTWOOD ROAD

Intersection to be upgraded to Aur & Bac Lane Treatments to East Gippsland Shire as per Traffix Report (Oct 2007)

10m Landscape Buffer

Sewer Connection Point. New Sewer to be constructed by East Gippsland Water from the south as a shared main.

LANES ROAD

TO BAIRNSDALE

Notations

- Refer to Certificate of Title for title particulars and any easements, restrictions, or covenants.
- The PDF copy of this plan has surveyed features on separate layers. These layers may be of assistance with analysis of data on this document.
- Data on this plan may only be manipulated with permission from Marshal Melbourne Pty Ltd.

Vicmap

LEGEND

- Subject Site
- Road Network
- Intersection to be Upgraded
- Water Main
- Sewer Main
- 100 Year Flood Boundary
- Existing Waterway
- Artefact Scatter and 20m Buffer
- Existing Significant Vegetation
- Existing Buildings
- Retarding Basins
- Local Activity Centre
- Superlot (Delivered with Stage 1)
- Residential Lots
- Public Open Space - Gully/Drainage
- Public Open Space - Tree & Artefact Retention
- Public Open Space - Landscape Buffer

marshal
MGA2020 ZONE 55

SCALE: 25 0 50 100m
1 : 2,500 at A1
1 : 5,000 at A3

SHEET 1 OF 1

LEVEL DATUM: N/A
DATE PLAN CREATED: 12/06/2024 REF: 35753

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BROOKFIELD LAKES
135 GREAT ALPINE ROAD
LUCKNOW
EAST GIPPSLAND SHIRE
REF: 35753DP-A

DEVELOPMENT PLAN

Development Plan,
135 Great Alpine Road, Lucknow
Brookfield Lakes Estate
Planning and Environment Act 1987,
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General Manager Place and Community
Date: 22 April 2025

STAGING

Goose Gully Waterway works to be staged as follows

a - Stage 1
b - Stage 2
c - Stage 3
d - Stage 5

Development stage sequencing as follows

1 - Stage 1
2 - Stage 2
3 - Stage 3
4 - Stage 4
5 - Stage 5
6 - Stage 10
7 - Stage 11
8 - Stage 8
9 - Stage 9
10 - Stage 6
11 - Stage 7

Please Note Public open space is designated with the stage in which it will be completed. Earthworks & landscaping may commence within the open space prior to its designated stage. This staging is to ensure that landscaped areas are not disturbed by development occuring adjacent to completed open spaces.

LEGEND

- Stage Boundary
- Major Access Street with Indented Parking (24.0m)
- Major Access Street (18.0m)
- Minor Access Street (16.0m)
- 100 Year Flood Boundary
- Proposed Waterway
- Potential Location of Retarding Basins (subject to further investigation)
- Artefact Scatter and 20m Buffer
- Existing Buildings
- Local Activity Centre
- Superlot (Delivered with Stage 1)
- Residential Lots
- a Public Open Space - Gully/Drainage/Staging No
- Public Open Space - Tree & Artefact Retention
- Public Open Space - Landscape Buffer

MGA2020 ZONE 55

DEVELOPMENT ANALYSIS

Subject Site Area	62.9	ha	%SA
Local Activity Centre	1.3	ha	2%
Superlot	8.2	ha	13%
Open Space	14.7	ha	23%
Goose Gully	11.9	ha	
Tree & Artefact Retention	0.8	ha	
Landscape Buffer	2.0	ha	
Road	10.4	ha	17%
Residential	28.3	ha	45%
Average Lot Size	553	sqm	
Approx Number of Lots	511	lots	

DATE	REVISION	DETAILS	AUTHORISED
31/3/22	Q	Stage 3 Layout updated.	J.T.N
5/8/22	R	Stages 5 & 6 Layout updated, Stage boundaries amended.	K.J.E
13/12/23	S	Stages 6 & 7 layout updated.	K.J.E
7/3/24	T	Stages 10 & 11 layout updated. Stage sequencing amended.	K.J.E
22/4/24	U	Stage 10 & 11 updated. Stage sequencing amended	K.J.E
1/5/24	V	Stage sequencing amended	K.J.E
2/5/24	W	Stage 6 and 7 layout amended	K.J.E
19/6/24	X	Layout and Sequence amended	K.J.E

SCALE: 25 0 50 100m 1:2,500 at A1 1:5,000 at A3

SHEET 1 OF 1

LEVEL DATUM: N/A
DATE PLAN CREATED: 19/06/2024 REF: 35753

marshal

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BROOKFIELD LAKES
135 GREAT ALPINE ROAD
LUCKNOW
EAST GIPPSLAND SHIRE
REF: 35753CP-A, Revision X

OVERALL SUBDIVISION CONCEPT PLAN

Appendix C

SIDRA Output

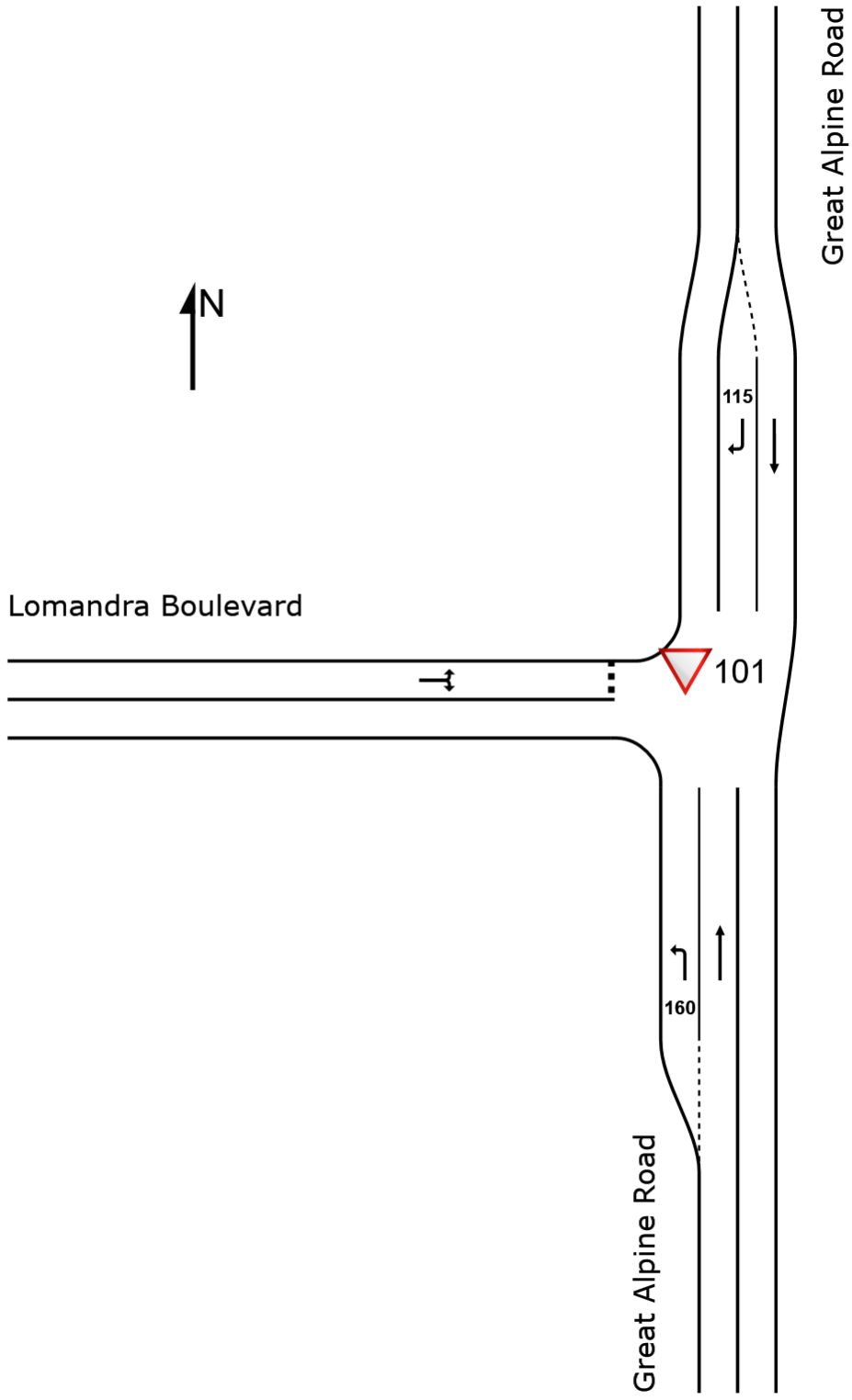
Development Plan,
135 Great Alpine Road, Lucknow
Brookfield Lakes Estate
Planning and Environment Act 1987,
East Gippsland Planning Scheme (Approved March 2025).
Development Plan Overlay Schedule 3.
Signed: Chris Stephenson
General Manager Place and Community
Date: 22 April 2025

SITE LAYOUT

Site: 101 [Great Alpine Road/Main Site Access AM Peak Post Development - 1 Access Point (Site Folder: General)]

New Site
Site Category: (None)
Give-Way (Two-Way)

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



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Date: 22 April 2025

MOVEMENT SUMMARY

Site: 101 [Great Alpine Road/Main Site Access AM Peak Post Development - 1 Access Point (Site Folder: General)]

Output produced by SIDRA INTERSECTION Version: 9.1.2.202

New Site
Site Category: (None)
Give-Way (Two-Way)

Vehicle Movement Performance

Mov ID	Turn	Mov Class	Demand Flows [Total veh/h	HV] %	Arrival 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	Eff. Stop Rate	Aver. No. of Cycles	Aver. Speed km/h
South: Great Alpine Road															
1	L2	All MCs	107	2.0	107	2.0	0.058	5.6	LOS A	0.0	0.0	0.00	0.58	0.00	52.8
2	T1	All MCs	213	5.0	213	5.0	0.111	0.0	LOS A	0.0	0.0	0.00	0.00	0.00	59.9
Approach			320	4.0	320	4.0	0.111	1.9	NA	0.0	0.0	0.00	0.19	0.00	57.3
North: Great Alpine Road															
8	T1	All MCs	495	5.0	495	5.0	0.259	0.1	LOS A	0.0	0.0	0.00	0.00	0.00	59.9
9	R2	All MCs	44	2.0	44	2.0	0.034	6.6	LOS A	0.1	1.0	0.39	0.60	0.39	51.5
Approach			539	4.8	539	4.8	0.259	0.6	NA	0.1	1.0	0.03	0.05	0.03	59.1
West: Lomandra Boulevard															
10	L2	All MCs	103	2.0	103	2.0	0.702	12.7	LOS B	6.2	44.3	0.82	1.15	1.74	44.0
12	R2	All MCs	249	2.0	249	2.0	0.702	22.2	LOS C	6.2	44.3	0.82	1.15	1.74	44.0
Approach			353	2.0	353	2.0	0.702	19.4	LOS C	6.2	44.3	0.82	1.15	1.74	44.0
All Vehicles			1212	3.8	1212	3.8	0.702	6.4	NA	6.2	44.3	0.25	0.41	0.52	53.3

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Options tab).
Vehicle movement LOS values are based on average delay per movement.
Minor Road Approach LOS values are based on average delay for all vehicle movements.
NA (TWSC): Level of Service is not defined for major road approaches or the intersection as a whole for Two-Way Sign Control (HCM LOS rule).
Two-Way Sign Control Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Control Delay: Geometric Delay is included).
Queue Model: SIDRA queue estimation methods are used for Back of Queue and Queue at Start of Gap.
Gap-Acceptance Capacity Formula: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.
Arrival Flows used in performance calculations are adjusted to include any Initial Queued Demand and Upstream Capacity Constraint effects.

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MOVEMENT SUMMARY

Site: 101 [Great Alpine Road/Main Site Access PM Peak Post Development - 1 Access Point (Site Folder: General)]

Output produced by SIDRA INTERSECTION Version: 9.1.2.202

New Site
Site Category: (None)
Give-Way (Two-Way)

Vehicle Movement Performance

Mov ID	Turn	Mov Class	Demand Flows [Total veh/h	HV] %	Arrival 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	Eff. Stop Rate	Aver. No. of Cycles	Aver. Speed km/h
South: Great Alpine Road															
1	L2	All MCs	233	2.0	233	2.0	0.126	5.6	LOS A	0.0	0.0	0.00	0.58	0.00	52.8
2	T1	All MCs	440	5.0	440	5.0	0.231	0.1	LOS A	0.0	0.0	0.00	0.00	0.00	59.9
Approach			673	4.0	673	4.0	0.231	2.0	NA	0.0	0.0	0.00	0.20	0.00	57.2
North: Great Alpine Road															
8	T1	All MCs	263	5.0	263	5.0	0.138	0.0	LOS A	0.0	0.0	0.00	0.00	0.00	59.9
9	R2	All MCs	128	2.0	128	2.0	0.146	8.7	LOS A	0.6	4.4	0.59	0.79	0.59	50.3
Approach			392	4.0	392	4.0	0.146	2.9	NA	0.6	4.4	0.19	0.26	0.19	56.4
West: Lomandra Boulevard															
10	L2	All MCs	87	2.0	87	2.0	0.357	9.0	LOS A	1.6	11.6	0.62	0.89	0.80	49.2
12	R2	All MCs	155	2.0	155	2.0	0.357	11.6	LOS B	1.6	11.6	0.62	0.89	0.80	49.1
Approach			242	2.0	242	2.0	0.357	10.7	LOS B	1.6	11.6	0.62	0.89	0.80	49.1
All Vehicles			1306	3.6	1306	3.6	0.357	3.9	NA	1.6	11.6	0.17	0.34	0.21	55.3

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Options tab).
Vehicle movement LOS values are based on average delay per movement.
Minor Road Approach LOS values are based on average delay for all vehicle movements.
NA (TWSC): Level of Service is not defined for major road approaches or the intersection as a whole for Two-Way Sign Control (HCM LOS rule).
Two-Way Sign Control Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Control Delay: Geometric Delay is included).
Queue Model: SIDRA queue estimation methods are used for Back of Queue and Queue at Start of Gap.
Gap-Acceptance Capacity Formula: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.
Arrival Flows used in performance calculations are adjusted to include any Initial Queued Demand and Upstream Capacity Constraint effects.

Brookfield Lakes Estate Stages 6 to 11

135 Great Alpine Road

Lucknow

Preliminary Servicing Report

23 October 2024

Executive Summary

This servicing report outlines the infrastructure provisions and future requirements for the development of the remaining stages (6 to 11) of the proposed estate located northeast of Bairnsdale, East Gippsland. The subject site is approximately 63 hectares in area and has been partially developed, with stages 1 to 5 already constructed and titled. These earlier stages provide the necessary connection points for the upcoming phases of development.

The key infrastructure for the estate, including drainage, sewerage, potable water, gas, electricity, and telecommunications, has been strategically designed and partially constructed to support the remaining stages. The existing drainage infrastructure, comprising wetlands, sedimentation ponds, and waterways, is in place and will accommodate future stormwater flows with minor upgrades to outlet sizes as recommended by Watertech. Additionally, sewerage and water connections from East Gippsland Water (EGW) will be extended from existing stages, and any additional external capital works will be initiated as the development progresses.

Gas, electricity, and telecommunications infrastructure have similarly been established in earlier stages, with planned extensions to serve future lots. The development will benefit from gas supply via APA's 63mm gas mains, electrical networks supported by AusNet's high-voltage infrastructure, and NBN's fibre optic network for telecommunications.

Environmental impacts have been carefully mitigated through the design and construction of key wetland and waterway infrastructure, with further developments expected to have minimal environmental effects.

In conclusion, the existing infrastructure provides a solid foundation for the remaining stages of the development. This ensures seamless integration of services for future residential lots while maintaining compliance with environmental and regulatory standards.

Contents

1 Existing Condition

The subject site is located 3.5 km to the Northeast of the main Bairnsdale township and is bounded by Eastwood Road to the South, Great Alpine Road to the East, Balfours Road to the West and Doyles Road to the North. The site is approximately 63 hectares in area and is bisected by Goose Gully, that runs south between Eastwood Road and Doyles Road.

Figure 1 - Subject Site Aerial (Source: Nearmap)

Stages 1 to 5 have been constructed and titled, providing connection points for proposed developments. All wetland and waterway works have been constructed to allow for a functional drainage outfall for all future stages.

The remaining stages are largely clear of any existing building or trees except Stage 11 where a patch will need to be retained along with an extended reserve strip along the North boundary to retain the rural landscape.

The subject site is located within East Gippsland Shire Council. Council will be responsible for determining the standards of all road and drainage assets within the site and for the ongoing care and maintenance of these assets. Council adopts the Infrastructure Design Manual (IDM) specifications and standard drawings.

2 Proposed Development

Based on current concept masterplan, CP_A_Z included in Appendix A, the staging order following the construction of the previous five stages is Stage 10, 11, 8, 9, 6 and 7. The remaining stages consists of a total of 295 lots that will need to be serviced from the existing stages.

The land falls steeply centrally into Goose Gully from the East and West title boundary. The proposed stages will need careful consideration towards structures, namely, drainage, sewer and retaining walls to step down the slope to create a functional pad for residential use without compromising the commercial viability.

Figure 2 - Concept Masterplan Ver Z (Source: Marshal)

A local park has been proposed in Northeast corner of future stage 11 to retain and protect an existing river red gum along with the surrounding eucalyptus patch, refer to Figure 3 below.

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General Manager Place and Community
Date: 22 April 2025

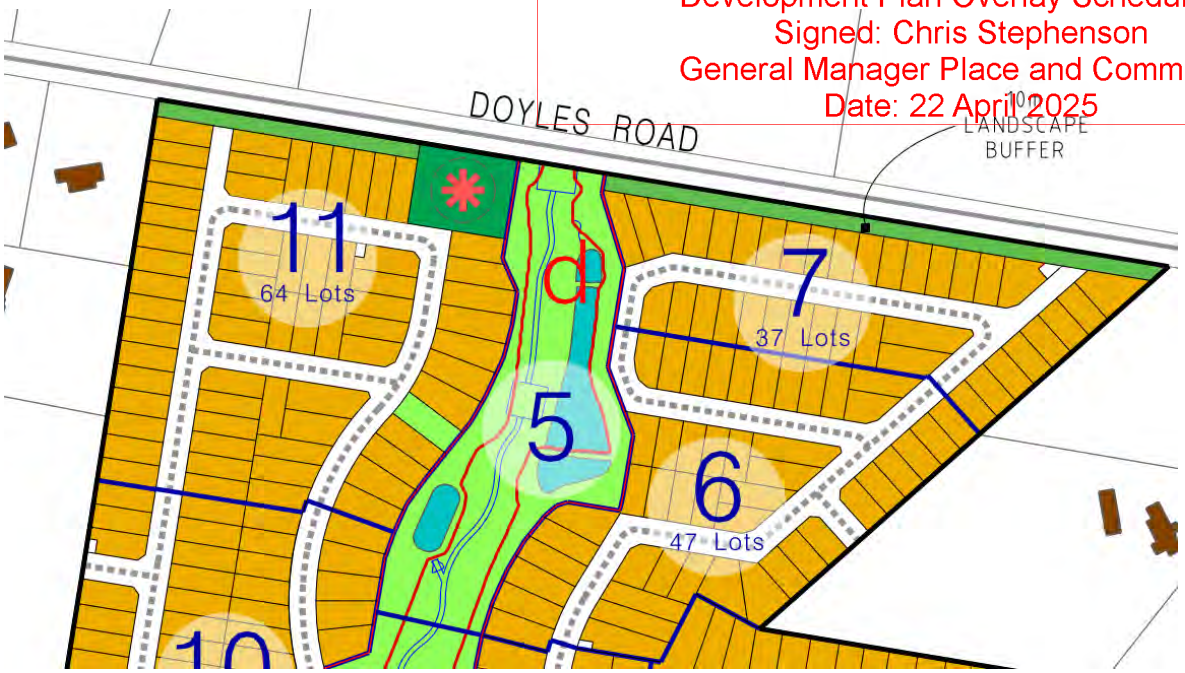

Figure 3 - Concept Masterplan Ver Z, zoom (Source: Marshal)

Construction of stage 10 being the next stage after stage 5 will require a set of culverts to be constructed to allow road connections to be made including utilities for stage 10 and 11. A similar size culvert will need to be constructed as part of stage 8 to allow road connections for stages 8 and 9. The Superlot and Local Activity Centre will both be serviced by the wider development.

3 Drainage

3.1 Existing Infrastructure

All major drainage infrastructure for the estate have already been completed. As advised by our drainage consultant Watertech in Appendix C – SWMS, overland flows are contained within the estate with the primary external catchment concentrating into Goose Gully Creek.

The existing infrastructure includes major waterway improvements for Goose Gully Creek, three offline wetlands, three sedimentation ponds, and an online lake. These features are designed to meet the requirements of both the Catchment Management Authority (CMA) and the local council, following Melbourne Water's design guidelines.

3.2 Proposed Infrastructure

The proposed stages drain into the existing waterway which includes a series of sedimentation pond and wetlands.

Due to the increased number of lots from 241 to 295, the increased density of development has altered the performance of the various basins within the development. Specifically, the increased density has led to changes in peak discharge and top water levels experienced within the proposed storages. Watertech have advised an increase in outlet size into the existing wetland and eventually the waterway will be required, please see this upsizing requirement in Table 1 below. The timing of these upsizing works can be completed in each relevant stage contributing to the respective wetlands.

Table 1 - Outlet Sizing into Wetland - Upsizing (Source: Watertech)

Name	Previous pipe size (mm)	Updated pipe size (mm)
SP W1A	0.6	0.75
SP W2A	0.45	0.525
SP W2B	0.45	0.525
SP W3A	0.525	0.6
SP W3B	0.525	0.6

The proposed drainage infrastructure is further detailed in Appendix C - Stormwater Management Strategy prepared by Watertech (August 2024) which will be included for endorsement as part of the development plan submission.

The drainage infrastructure will provide water quality treatment for the catchment in accordance with Urban Stormwater Best Practice Environmental Management Guidelines. These guidelines require all stormwater runoff from new residential developments be treated to ensure compliance with the following criteria:-

- 80% removal of suspended solids
- 45% removal of total phosphorus, and
- 45% removal of total nitrogen annual loads
- 70% gross pollutants

Overland Flow Paths

Proposed roads within the development will need to cater for continuous and uninterrupted overland flows and be designed to ensure compliance with floodway safety criteria providing freeboard to the proposed residential lots.

Council - Drainage

The Stormwater Management Strategy that addresses the subject land's drainage outfall, attenuation of stormwater to pre-developed flows (if required) and water quality treatment. The strategy must also meet the requirements of CSIRO Best Practice Environmental Management Guidelines for Urban Stormwater prior to discharge to receiving waterways.

Filling

Minor isolated local filling of the subject land will be required to ensure 600mm freeboard is maintained to the 1 in 100 year flood level of the proposed waterway. See Figure 4 below showing the indicative Q100 levels in the central waterway..

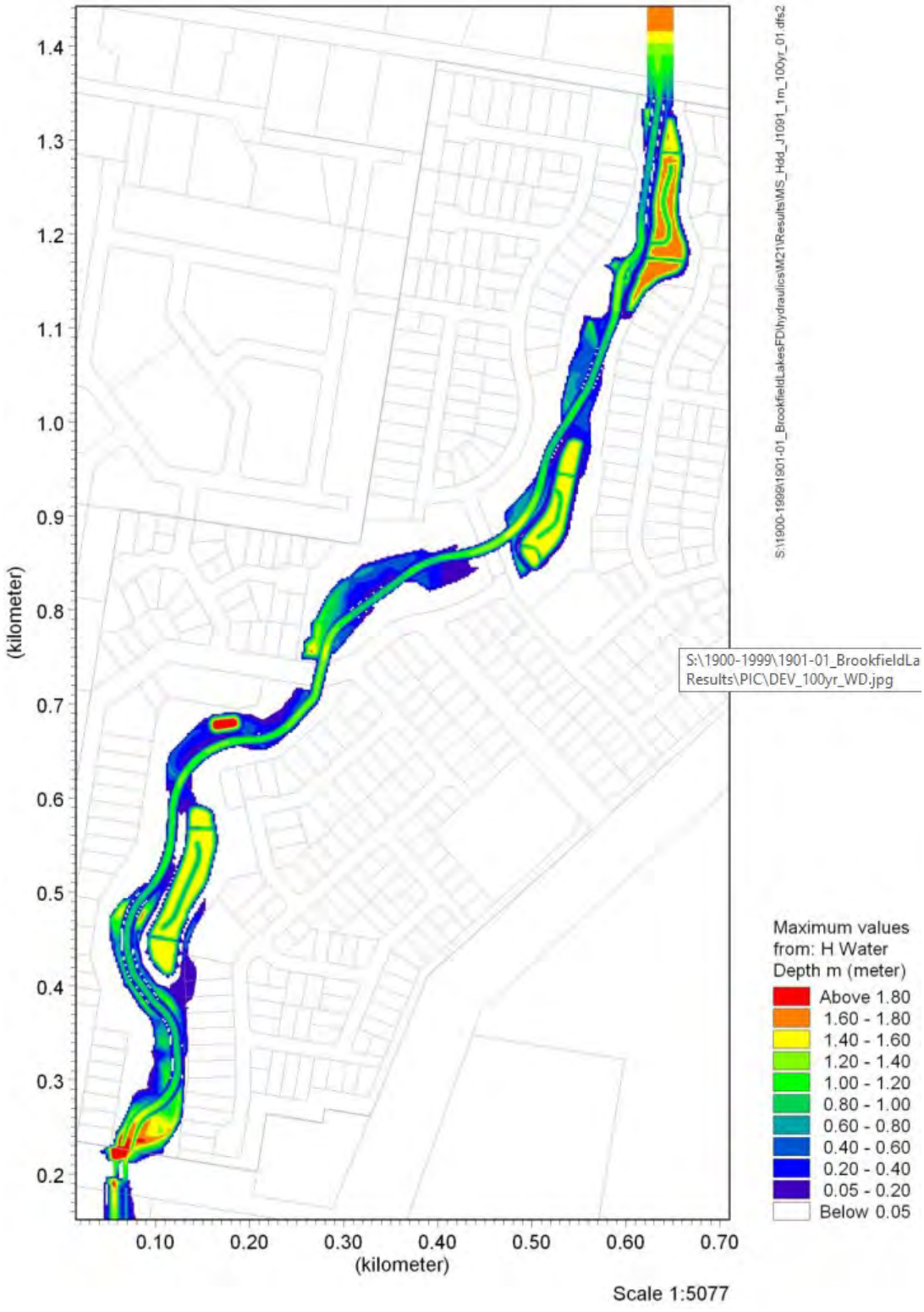

Figure 4 - Waterway Q100 depth (Source: Watertech)

4 Sewerage

4.1 Existing Infrastructure

East Gippsland Water (EGW) is the authority responsible for providing sewerage facilities for the proposed development. EGW has previously issued a technical memorandum by GHD, Appendix D, detailing the works and cost-sharing arrangements between the developer and EGW. The memorandum specifies that a series of external capital works will be triggered once the development, including any internal superlot, reaches certain stages of development. Upon completion of Stage 5, the development will have a total of 216 lots.

Stage 10 will contribute an additional 54 lots based on the proposed CMP A_Z. The remaining stages (11, 8, 9, 6, and 7) will be subject to the completion of the aforementioned external capital works before further statements of compliance for both sewer and water can be issued.

At the time of writing this report, no other developments within the estate, aside from the internal Brookfield Lakes stages, have the necessary permits and approvals in place to connect to the existing EGW infrastructure.

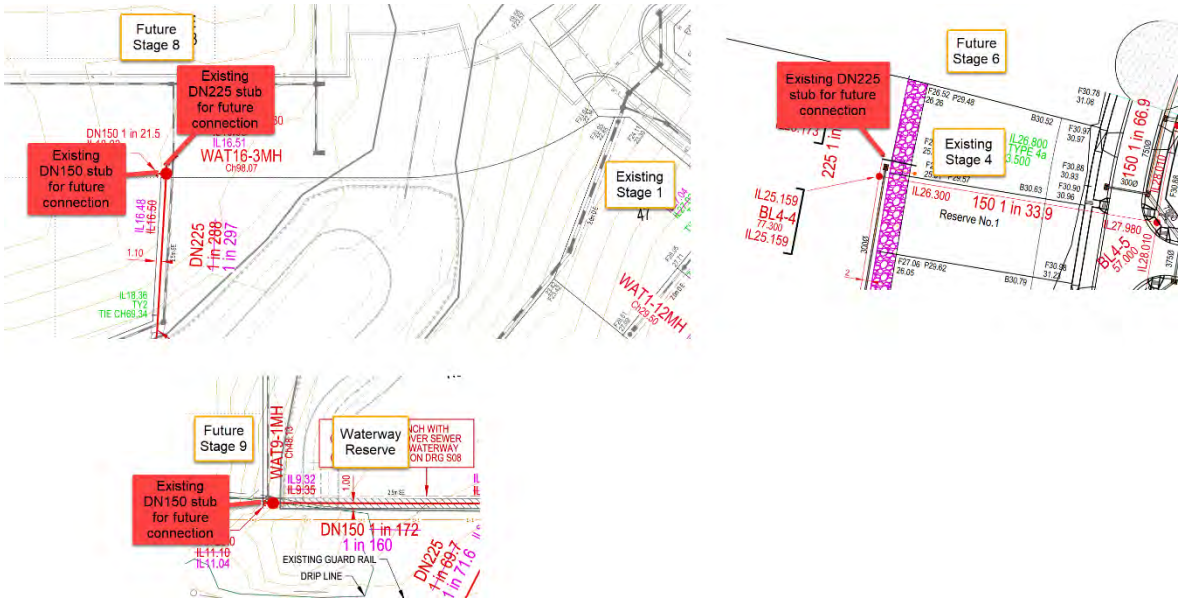

Figure 5 shows connection location from existing stages

Referring to Figure 5, the remaining stages 6 to 11 can be serviced from existing sewer constructed as part of stage 1 and stage 4. A DN225 and DN150 stub can be connected into structure WAT16-3MH to service future stage 10, 11 and 8. Existing DN150 from structure WAT9-1MH can be connected to service Stage 9, while a DN225 from structure BL4-4 can be connected to service future stages 6 and 7.

4.2 Proposed Infrastructure

All connection points can be found in existing stubs constructed as part of stage 1.

Figure 6 - Proposed Sewer Network (Source: Marshal)

Further extensions of local sewerage assets are required within the proposed development to service the remaining internal stages within the sewerage catchment. These assets will be provided progressively as development proceeds to service residential lots by extending appropriately sized gravity reticulation sewers.

EGW do not have a set of authority standards for design and construction reference Brookfield Lakes will be designed and constructed in accordance with the Water Industry standards and notably:-

- Sewers shall be located in accordance with WSA 02-2014-3.1 Clause 5.2.4
- Sewers shall only be located along the rear boundary of private lots where reasonable access to the sewer is maintained, as defined in WSA 02-2014-3.1 Clause 5.2.4.3.
- For sewers proposed to be located along the rear boundary of private lots, a Plan of Subdivision showing building envelopes must be submitted with the sewer design drawings to verify that access requirements can be met.
- Where compliance with access requirements cannot be verified at the design stage, the sewer must be located in public or road reserve. Western Water may offer dispensation to this requirement where only a small portion of allotments do not meet access requirements.
- Maintenance holes shall not be located in the rear of private property without reasonable access, unless they are required to accommodate lateral reticulation sewers at the end of a pod of lots in accordance with MRWA Standard MRWA-S-108.

5 Water Supply

5.1 Existing Infrastructure

East Gippsland Water (EGW) is the authority responsible for providing potable water facilities for the proposed development. EGW has previously issued a technical memorandum by GHD, Appendix D, detailing the works and cost-sharing arrangements between the developer and EGW. The memorandum specifies that a series of external capital works will be triggered once the development, including any internal superlot, reaches certain stage of development. Upon completion of Stage 5, the development will have a total of 216 lots.

Stage 10 will contribute an additional 54 lots based on the proposed CMP A_Z. The remaining stages (11, 8, 9, 6, and 7) will be subject to the completion of the aforementioned external capital works before further statements of compliance for both sewer and water can be issued.

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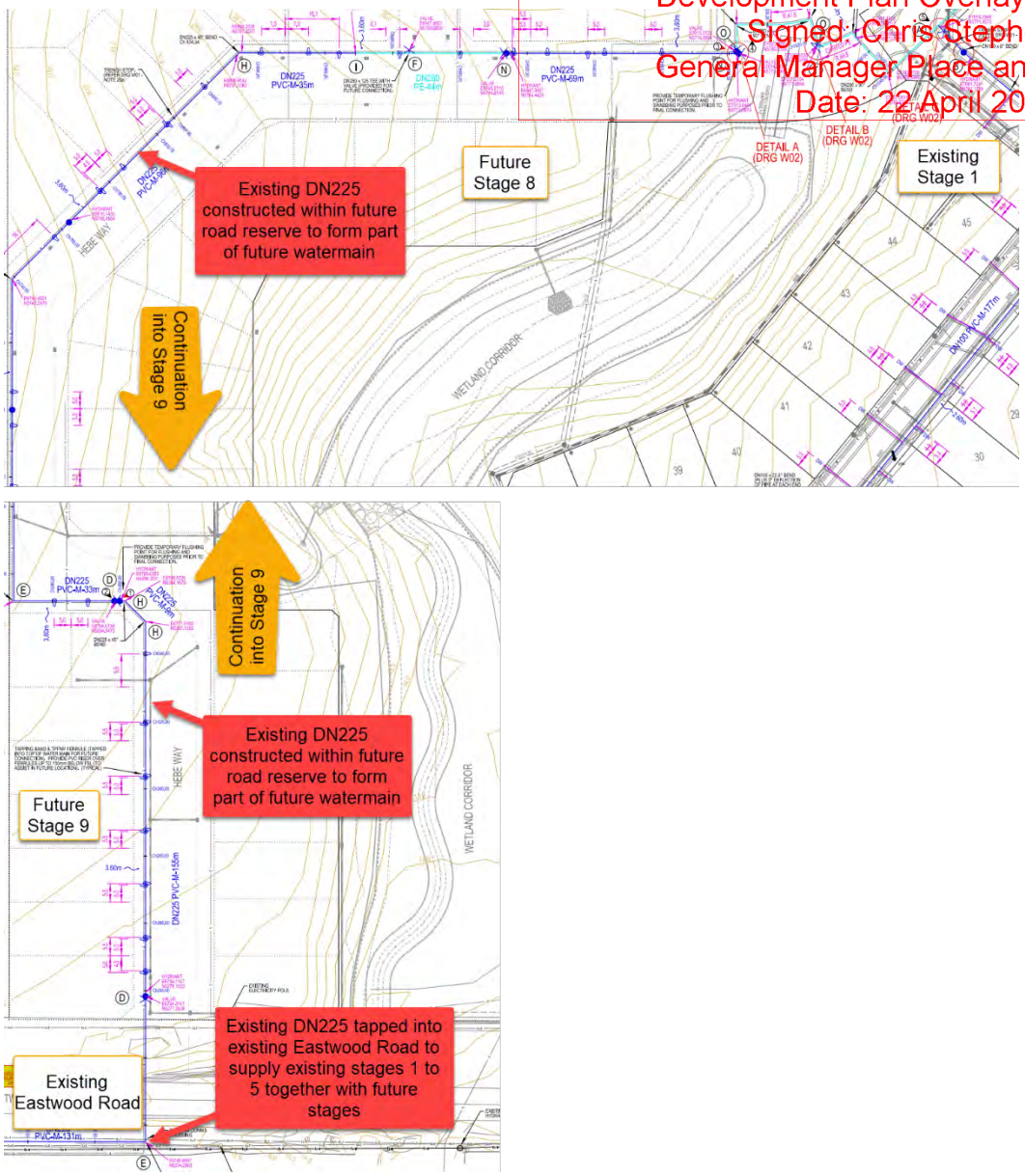

Figure 7 Existing stage 1 watermain supply to Brookfield Estate

The above Figure 7 shows the already constructed DN225 main as part of stage 1 that taps into the existing watermain located within Eastwood Road that services existing stage 1 to 5. The main runs along the proposed stage 8 and 9 along the road reserve that can be eventually tapped into during the construction of the relevant stages.

Figure 8 Stage 10, 11, 6 and 7 connection points
An existing DN150 washout can be continued to service future stages 10 and 11. A polyethylene DN180 can be continued from stage 4 to service future stages 6 and 7.

5.2 Proposed Infrastructure

Drinking water supply for the remaining stages will be provided progressively by extensions of the existing mains from the adjoining existing stages, notably from stage 2 across the waterway to stage 10 through to stage 11, and from stage 4 into stage 6 through to stage 7. Both stage 8 and 9 already have an existing main constructed as part of stage 1, most of this existing water main runs through the main road within the verge within the road reserve of the two stages.

150mm and 100mm dia. reticulated potable water mains will be extended from the key East Gippsland Water assets through the proposed development to service the individual lots as development proceeds.

A PRV has been confirmed to be required and delivered as part of the stage 10 works. EGW has agreed that cost of construction will be shared with the developer.

6 Recycled Water Supply

6.1 Existing Infrastructure

There is no recycled water supply network in the surrounding area to provide connectivity to the estate and is not required as an essential infrastructure.

6.2 Existing Infrastructure

APA is the authority responsible for the provision of gas supply to the Bairnsdale area including the subject land. All road connection points will have 63mm gas mains available for connection from the existing stages 1 to 5.

6.3 Proposed Infrastructure

Connection to the existing assets from existing stages similar to watermain connection for stages 10, 11, 6 and 7.

Stage 8 will extend from existing stage 2 and further extend into future stage 9.

7 Electricity

7.1 Existing Infrastructure

Existing stages 1 to 5 has been constructed and will provide necessary connection points to future stages 6 to 11.

7.2 Proposed Infrastructure

Connection is anticipated to be made to AusNet's existing/proposed high voltage networks in the adjoining development to the north and west.

Further extensions of underground high voltage and low voltage cables and provision for substation sites located within all future stages will be required. These electrical assets will be provided progressively as development proceeds from the north and west to service residential lots.

AusNet will contribute to the cost of reticulating high voltage infrastructure under current policy. According to the endorsed electrical masterplan, all future remaining will require and electrical substation, refer to excerpt below that marks their location, extracted from Appendix E.

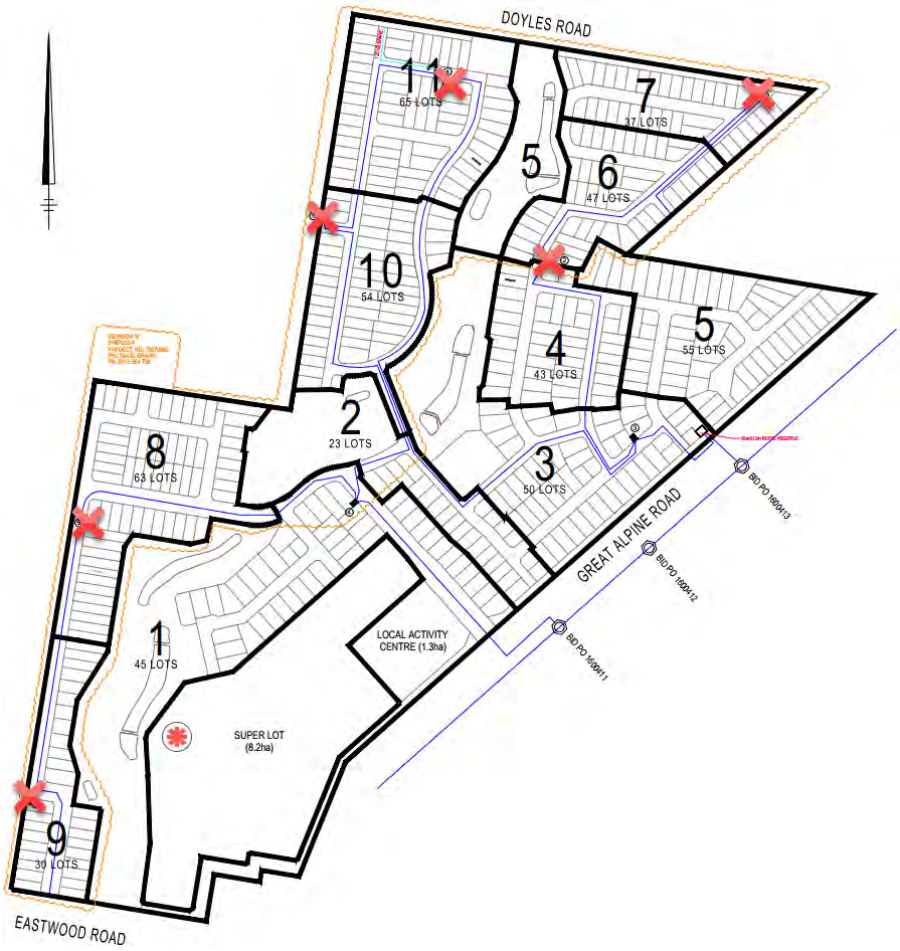

Figure 9 Future substation locations marked on every stage

8 Communications

8.1 Existing Infrastructure

Existing stages 1 to 5 has been constructed and will provide necessary connection points to future stages 6 to 11.

.

8.2 Proposed Infrastructure

An agreement will be entered into with NBN to supply the proposed development with fibre optic services to the home.

The provider will extend the fibre optic network to the future stages from existing stages 1 to 5. Pipe & pit infrastructure will progressively be constructed as development proceeds from the west to service residential lots. The fibre network infrastructure will be designed and constructed with sufficient capacity to service the full demands of the future development.

9 Environmental Impact

The majority of the key infrastructure works required to service the proposed development including sewerage, potable and recycled water main extensions, gas supply, underground electrical and telecommunications extensions from adjoining developments are all expected to have very minimal environmental impact.

All key wetland & waterway infrastructure have been constructed that were careful design to minimise the impact of the construction of these assets upon the sensitive conservation area.

10 Conclusion

The proposed development for the remaining stages 6 through 11 is well-supported by existing infrastructure, ensuring a smooth transition for future growth. With essential services such as drainage, sewerage, water supply, gas, electricity, and telecommunications already in place from stages 1 to 5, the foundation for the next phases of construction has been firmly established. The drainage system, designed in compliance with environmental guidelines, has undergone the necessary enhancements to support the increased density and will continue to provide effective water management for future stages. The waterway infrastructure, including wetlands and sedimentation ponds, ensures sustainable stormwater management, with minor modifications to outlet sizing as per recommendations from Watertech.

Sewerage and potable water networks, managed by East Gippsland Water (EGW), have been extended progressively from earlier stages, offering connection points for the new development. Future stages will benefit from these established networks, with any additional works triggered once certain development milestones are reached. Gas and electrical infrastructure, both essential services, will similarly be extended from existing stages, with AusNet contributing to the high-voltage infrastructure requirements and substations planned to accommodate the future growth.

Telecommunication services will be provided by NBN, extending from the current network, ensuring that future residential lots are equipped with high-speed fibre optic services. Overall, the environmental impact of these infrastructure expansions is expected to be minimal, particularly with key drainage, wetland, and waterway works already completed. This has been done with careful consideration of the local ecosystem, particularly Goose Gully, ensuring that the development progresses in an environmentally responsible manner.

In summary, the established infrastructure within the development site forms a solid foundation for the remaining stages, allowing the continuation of high-quality residential development while adhering to environmental and regulatory standards. The careful planning and phased approach to extending services will ensure that future residents have access to reliable utilities and amenities, all within a framework of sustainable growth.

Appendices

A – A_Z Concept Master Plan

B – Road Network Plan

C – SWMS - WaterTech

D – GHD Technical Memorandum

E – Endorsed Electrical Masterplan

Development Plan,
135 Great Alpine Road, Lucknow
Brookfield Lakes Estate
Planning and Environment Act 1987,
East Gippsland Planning Scheme (Approved March 2025).
Development Plan Overlay Schedule 3.
Signed: Chris Stephenson
General Manager Place and Community
Date: 22 April 2025

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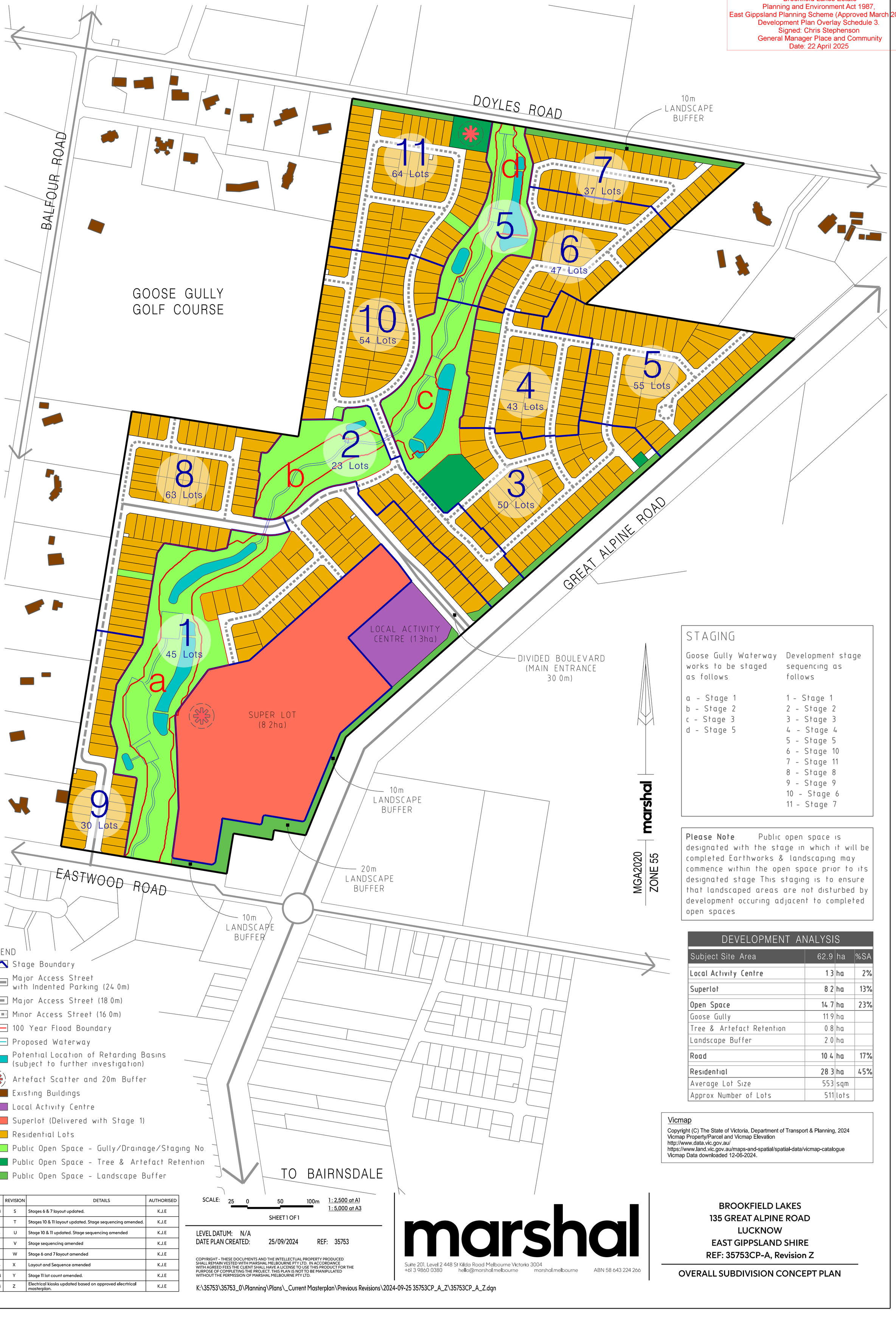

STAGING

Goose Gully Waterway works to be staged as follows	Development stage sequencing as follows
a - Stage 1	1 - Stage 1
b - Stage 2	2 - Stage 2
c - Stage 3	3 - Stage 3
d - Stage 5	4 - Stage 4
	5 - Stage 5
	6 - Stage 10
	7 - Stage 11
	8 - Stage 8
	9 - Stage 9
	10 - Stage 6
	11 - Stage 7

Please Note: Public open space is designated with the stage in which it will be completed. Earthworks & landscaping may commence within the open space prior to its designated stage. This staging is to ensure that landscaped areas are not disturbed by development occuring adjacent to completed open spaces.

DEVELOPMENT ANALYSIS

Subject Site Area	62.9	ha	%SA
Local Activity Centre	1.3	ha	2%
Superlot	8.2	ha	13%
Open Space	14.7	ha	23%
Goose Gully	11.9	ha	
Tree & Artefact Retention	0.8	ha	
Landscape Buffer	2.0	ha	
Road	10.4	ha	17%
Residential	28.3	ha	45%
Average Lot Size	553	sqm	
Approx Number of Lots	511	lots	

LEGEND
- Stage Boundary
- Major Access Street with Indented Parking (24.0m)
- Major Access Street (18.0m)
- Minor Access Street (16.0m)
- 100 Year Flood Boundary
- Proposed Waterway
- Potential Location of Retarding Basins (subject to further investigation)
- Artefact Scatter and 20m Buffer
- Existing Buildings
- Local Activity Centre
- Superlot (Delivered with Stage 1)
- Residential Lots
- a Public Open Space - Gully/Drainage/Staging No
- Public Open Space - Tree & Artefact Retention
- Public Open Space - Landscape Buffer

MGA2020 ZONE 55

SCALE: 25 0 50 100m — 1:2,500 at A1, 1:5,000 at A3

SHEET 1 OF 1

LEVEL DATUM: N/A
DATE PLAN CREATED: 25/09/2024 REF: 35753

DATE	REVISION	DETAILS	AUTHORISED
13/12/23	S	Stages 6 & 7 layout updated.	K.J.E
7/3/24	T	Stages 10 & 11 layout updated. Stage sequencing amended.	K.J.E
22/4/24	U	Stage 10 & 11 updated. Stage sequencing amended	K.J.E
1/5/24	V	Stage sequencing amended	K.J.E
2/5/24	W	Stage 6 and 7 layout amended	K.J.E
19/6/24	X	Layout and Sequence amended	K.J.E
19/8/24	Y	Stage 11 lot count amended.	K.J.E
25/9/24	Z	Electrical kiosks updated based on approved elecrtrical masterplan.	K.J.E

K:\35753\35753_0\Planning\Plans_Current Masterplan\Previous Revisions\2024-09-25 35753CP_A_Z\35753CP_A_Z.dgn

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BROOKFIELD LAKES
135 GREAT ALPINE ROAD
LUCKNOW
EAST GIPPSLAND SHIRE
REF: 35753CP-A, Revision Z

OVERALL SUBDIVISION CONCEPT PLAN

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Brookfield Lakes WSUD Functional Design Technical Report

August 2024

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

DOCUMENT STATUS

Version	Doc type	Reviewed by	Approved by	Date issued
v01	Report	SEL	CMB	16/09/11
v02	Report	AM	AM	28/08/24

PROJECT DETAILS

Project Name	190101R01v01
Client	Watsons
Client Project Manager	Charles Ho
Water Technology Project Manager	Yafei Zhu
Report Authors	Yafei Zhu, Sabrina Guazzo
Job Number	1901-01
Report Number	R02
Document Name	1901-01R01v02.docx

Cover Photo: Existing farm dam on the proposed development site

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

Water Technology was engaged by Watsons Pty Ltd to develop a functional design of stormwater treatment system for the proposed development of the Brookfield Lakes, Bairnsdale (Figure 1). This study follows from the Stormwater Management Plan prepared by Water Technology in 2008. The functional design will consist of major waterway improvement for the Goose Gully Creek within the study site, 3 offline wetlands, 3 sedimentation ponds and 1 online lake, as shown in Figure 2. This design meets both the CMA's and Council's requirements and follows Melbourne Waters design guidelines. This report will outline the key design considerations, and it is recommended to be viewed along with the Brookfield Lakes WSUD Functional Design Drawings.

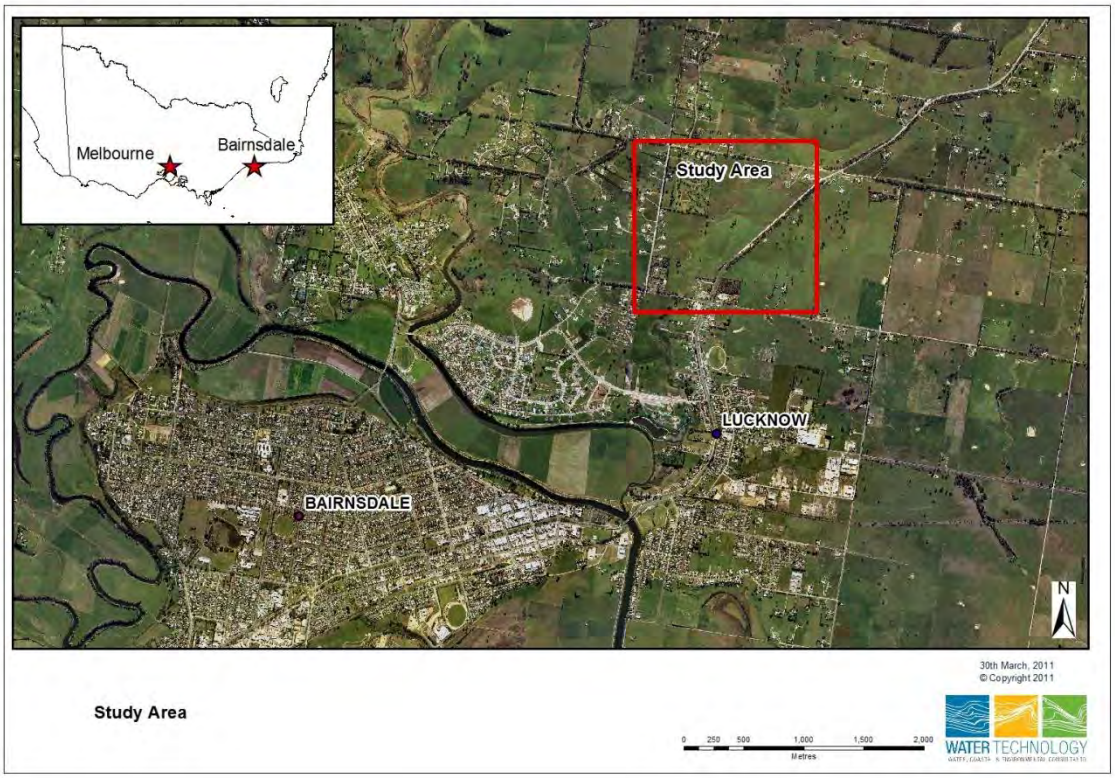


Figure 1 Location Map

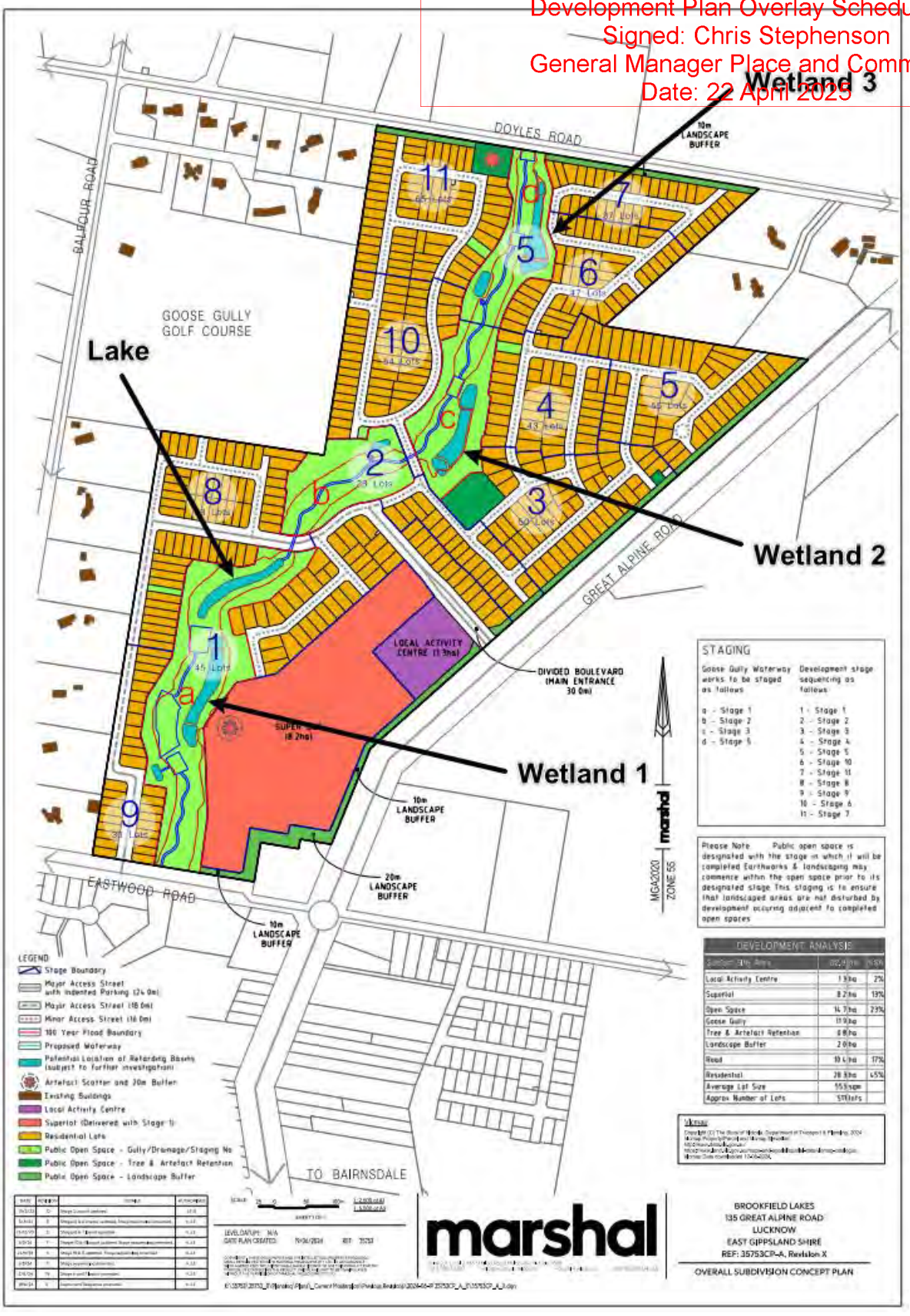


Figure 2 **Proposed development**

2. WATERWAY IMPROVEMENT

The major waterway for the study area is Goose Gully Creek, intersecting the site in a north-south direction with very steep side slopes. Goose Gully Creek is an ephemeral stream and at the time of site inspections (April 2008 and April 2011) it contained no permanent water other than small farm dams (Figure 3**Error! Reference source not found.**). A number of minor gully lines traverse the site, the most major of which provides a flow path from Goose Gully Golf Course into Goose Gully Creek. Goose Gully Creek receives flow from a predominantly agricultural upstream catchment, with an approximate catchment area of 700 hectares at the end of the proposed site. An accompanying Waterway Management Plan accompanies this stormwater management technical report, detailing the short, medium and long term management goals and improvement of the waterway.

Figure 3 Goose Gully Creek

The waterway improvement works include:

- Reduce the waterway's vulnerability to erosion by easing the channel slope and increasing flow area; five rock chutes are proposed;
- A low flow channel with three month flow capacity following the existing channel alignment is proposed;
- One online lake is to be constructed upstream of the second rock chute; it will not only provide the benefits of flood storage, flow velocity reduction and stormwater quality improvement but also act as mosquito control;
- A exclusion zone next to the first wetland will be planned to prevent any work within the area so that a number of trees with high ecological values will be protected;

- Dense vegetation will be planted within the floodplain.

Refer the Waterway Management Plan for more details.

2.1 Rock chutes

Rock chutes were employed to prevent erosion from in the design. The rock chutes were sized using the industry standard modelling program, CHUTE, with design flow 15m^3/s (5yr ARI). The specifications of the rock chutes can be seen in DWG NO. 20.

2.2 Lake

An online lake is designed to provide flood storage, reduce erosion potential, mosquito control and water quality benefit. The total storage volume is approximately 4500 m^3. A cumulative frequency analysis for the lake storage has been undertaken with 50-years of rainfall and evapo-transpiration data. There are a number of farm that dams have been built upstream of the site. Therefore the analysis looked at the scenario if no upstream runoff enters in to lake. Figure 4**Error! Reference source not found.** shows that the size of the lake is considered to be sustainable as the worst case scenario shows no dry outs. The basin is at least half full 30% of the time with no upstream flow inputs, and 80% of the time with the upstream inputs.

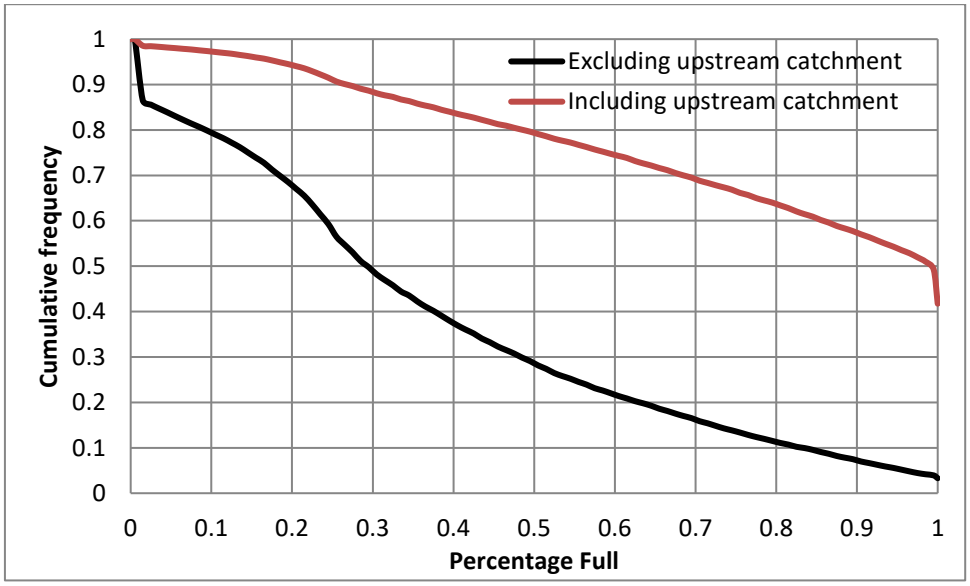


Figure 4 Cumulative frequency analysis for lake storage

3. SEDIMENTATION POND

3.1 Inlet

The sedimentation pond is used as a pre-treatment to settle out coarse sediments and dissipate energy. Appropriate rock protection and energy dissipation structures will need to be applied at the entry to the sediment pond. The structures will need to be designed to dissipate the velocity below 1m/s (0.5m/s preferred) within the sediment pond.

3.2 Outlet

Based on the topographic constraints of the site, only the 5 year flow through drainage pipe network and a very small portion of the overland flow will be captured by the sedimentation ponds. Therefore, no spillway was included in the design. The outlet structure has been design to cater for the 10 year

ARI flow. The outlet structure will use standard precast square grated drainage pit which have been checked against both weir flow and orifice flow conditions. A flat outlet pipe is connected to the outlet pit discharging in to a rock chute. The rock chute was sized using CHUTE. All the calculations have been attached in Appendix A.

3.3 Maintenance

The maintenance of the sediment pond is achieved by allowing the pond to free drain down to 0.5m below the NWL which is the expected sediment level for clean out. A maintenance pipe is connected to the outlet pit which is sized to drain water to the required level within 12 hours. The clean out frequency for the sediment pond is based on the area of the pond and the loads generated off the catchment. Using a sediment load of 1.6m^3/ha/yr (Willing and Partners, 1992) and a Gross Pollutant Load of 0.4m^3/ha/yr (Alison et al, 1998) the clean out frequency is shown in **Table 3-1**. An area will be constructed for dewatering of removed sediments from the sedimentation ponds.

Table 3-1 Clean out frequency for the sedimentation ponds

	Sedimentation Pond		
Clean out frequency (year)	SP1	SP2	SP3
	16	11	7

As can be seen the average clean out frequency of the sedimentation ponds is approximately 10 years, minimising maintenance costs to council.

4. WETLAND

Each of the three wetlands consists of two inlet zones and one macrophyte zone. The inlet zones act as basically sedimentation ponds which remove the coarse sediments. The macrophyte zone is a heavily vegetated water body which can effectively remove pollutants including dissolved contaminants from stormwater.

4.1 Inlet zone

The inlet zone will act as a sedimentation pond. The design procedure is same as outlined in section 3. However, the entry pit and pipe was designed with the 1 year ARI flow. The normal water level (NWL) of the inlet zone is 200mm above that of the macrophyte zone that will ensure a sufficient head difference. An overflow weir of 5 year ARI capacity is to be placed between the inlet and macrophyte zones. The scheduled clean out frequency for the inlet zones is shown in **Table 4-1**.

Table 4-1 Clean out frequency for the sedimentation ponds at wetland inlet zone

	Sedimentation Pond		
Clean out frequency (year)	W1A/ W1B	W2A/ W2B	W3A/ W3B
	10	16	10

Again, the sedimentation ponds on the wetland have been designed to minimise council maintenance requirements.

4.2 Macrophyte zone

The macrophyte zone consists of a series of shallow and deep ponds which promote vertical and horizontal diversities for various vegetation species. A bund with top at the total extended detention (TED) level is to be constructed in the centre of the macrophyte zone so that the hydraulic efficiency could be increased, similar to a baffle structure.

4.3 Outlet structure

Outlet pipes for the wetlands are connected to an outlet pit containing a PVC pipe with small orifice holes which determine the detention time within the wetland that is critical to strip nutrients from the inflow. The Riser outlet structure is designed for each wetland to provide for a 72 hour notional detention time for effective treatment. The calculations are attached in Appendix B.

It is noted that some outlet pipes from some sedimentation basins prior to wetlands have changed with the recent layout change as per table below. The same results are reported in Appendix B with more details.

Name	Previous pipe size (mm)	Updated pipe size (mm)
SP W1A	0.6	0.75
SP W2A	0.45	0.525
SP W2B	0.45	0.525
SP W3A	0.525	0.6
SP W3B	0.525	0.6

4.4 Maintenance

Maintenance drain with a manual valve is included in the design. It will allow free draining of the inlet zone and macrophyte zone down to 500mm below NWL within 12 hours.

4.5 Additional storage

In addition to a 300mm freeboard, another 300mm above the TED level for each wetland has been included in the design. The storage is considered to provide flood attenuation storage and reduce the runoff frequency (**Table 4-2**). The spillways for the wetlands were designed in CHUTE and specifications can be viewed in the design drawings.

Table 4-2 Additional storage in the wetland systems

	Wetland 1	Wetland 2	Wetland 3
Storage above NWL	2474	2120	2124
Storage above TED	1040	851	891

5. UPDATED HYDROLOGY

A RORB model has been implemented to study the hydrology of the site and external catchments. The model schematic that reflects the RORB model is shown in Figure 5 that was updated to reflect the increased density of the proposed development.

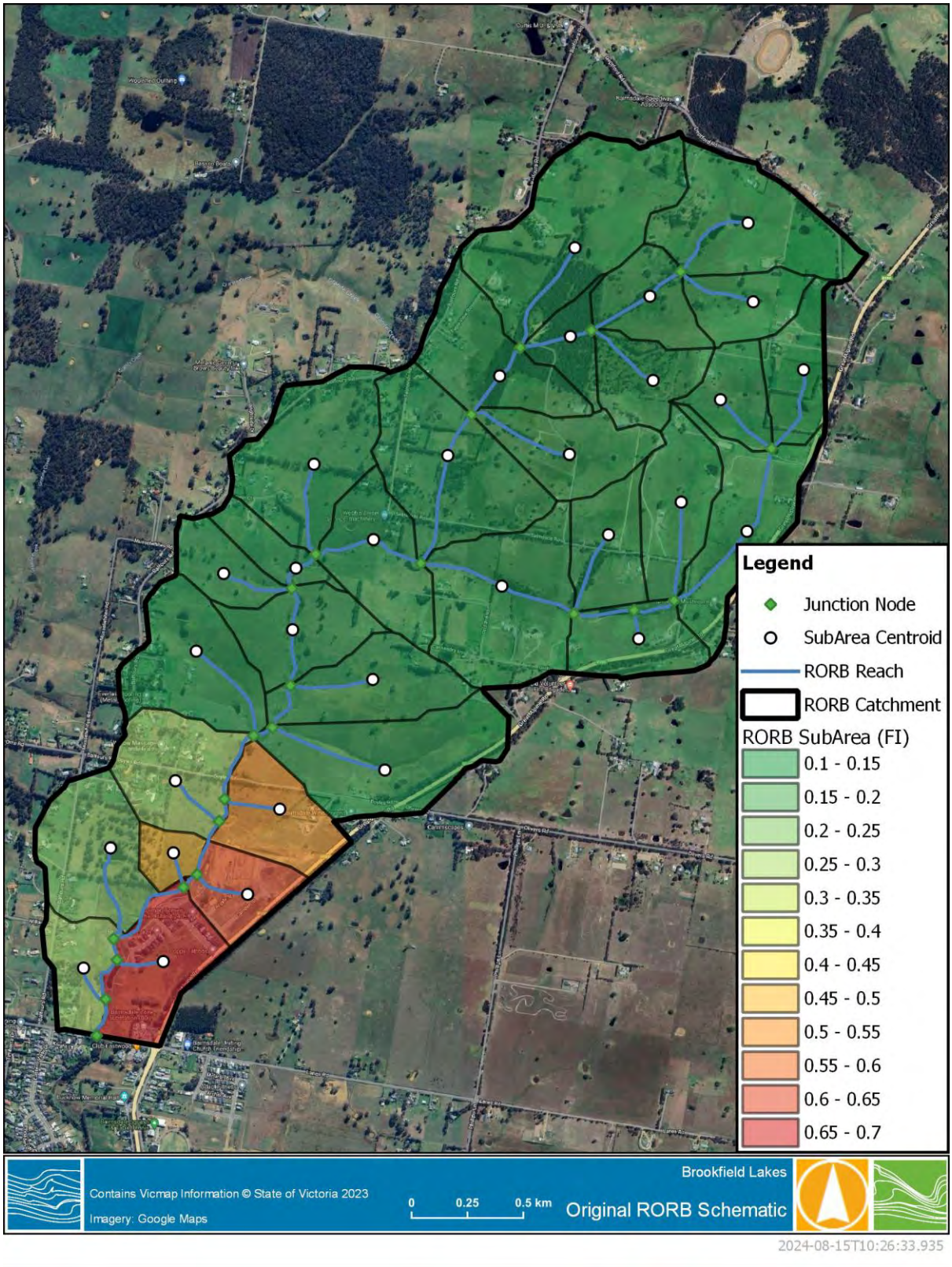


Figure 5 Model schematic

5.1 ARR19 – Sensitivity

Previous modelling of the hydrology for the site was completed with ARR87 methodologies, however the new ARR19 methodologies and data was adopted to ensure that the latest guidelines have not significantly changed the expected outcomes. This analysis included the following:

- Extraction of latest IFD rainfall data.
- Adoption of multiple temporal patterns.
- Replacement of run-off coefficient with continuing loss. A continuing loss of 3 mm/hr was adopted, slightly reduced from the value provided by ARR datahub (3.4 mm/hr).

The updated model was run using a Monte Carlo analysis and flows rates shown in Table 5-1. The results demonstrate the difference in the methodologies applied, with the ARR87 results validated to the rational method. This was common practice and involved adjusting the kc parameter so results aligned with the rational method. This method is no longer best practice and if further analysis was to be undertaken the kc parameter would be re-calculated.

Importantly the ARR19 results shown adopting the previous kc value are an overestimate, and directly the result of the kc value applied. For example, adjusting the kc value to a more commonly used value Pearse et al, resulted in the design flows aligning. Design flows also aligned well with flow estimates from RFFE 2021. Therefore, it is considered that it is appropriate to adopt the original model parameters and methodology with the ARR87 data.

The updated model was simulated adopting rainfall data from ARR87 and the model parameters as provided in Table 5-2.

Table 5-1 Existing Conditions – Peak Flow Comparison for Differing Methodologies

AEP / ARI	ARR87 (m3/s)	ARR19 (m3/s)	Adjusted Kc (m3/s)	RFFE 2021 (m3/s)
Kc Value	2.86	2.86	4.29	N/A
1 in 5	15.2	13.1	10.4	6.2
1 in 10	17.8	18.1	14.7	10.3
1 in 20	21.1	23.4	18.3	15.2
1 in 50	24.8	30.8	24.7	22.5
1 in 100	28.3	37.3	29.1	28.6

Table 5-2 Model Parameters

ARR87 Parameters	Value	Model Parameters	Value
2y 1hr	19.78	Kc	2.86
2y 12hr	4.79	m	0.8
2y 72hr	1.47	Initial Loss	15.0 mm
50y 1hr	38.39	RoC	0.6
50y 12hr	8.83		
50y 72hr	3.06		
Skew	0.3		
F2 Value	4.22		

ARR87 Parameters	Value	Model Parameters	Value
F50 Value	15.35		

5.2 Results

The model results demonstrated the following:

- The increased density of development has little influence on the peak flow along Goose Gully for all design events and irrespective of on-site mitigation. This is because the development is located at the downstream end of the catchment and the increased efficiency of runoff reduces the overlapping of flows, reducing the peak flow experienced when unmitigated. However, flow mitigation is still required at the lot scale as the development will have increased the run-off entering Goose Gully. The results are provided in **Table 5-3**.

The increased density of development has increased the peak discharge and top water level experienced within the proposed storages. The changes to discharge and peak storage are provided in

- **Table 5-4**, note peak storage was used as the levels in RORB do not relate to on-ground levels. The location of the storages is provided in Figure 1.

Importantly the wetland outlet arrangements were designed to different catchment areas and sized using the rational method. The purpose of

- **Table 5-4** is to provide a comparison of the RORB modelling, further discussion around the outlet arrangement is provided in Section 4.3.

Table 5-3 Peak Flow Comparison – Goose Gully

AEP / ARI	Existing Conditions (m3/s)	Developed (Unmitigated) (m3/s)	Developed (Mitigated) (m3/s)
1 in 5	15.2	14.8	15.1
1 in 10	17.8	17.2	17.6
1 in 20	21.1	20.4	20.9
1 in 50	24.8	24.1	24.6
1 in 100	28.3	27.4	28.0

Table 5-4 Storage Performance – 1 in 100 YR Results

Basin	Design Event	Design Flow Rate (m3/s)	Previous Performance			Updated Performance		
			Peak Discharge (m3/s)	Peak Storage Volume (m3)	Spillway Engaged	Peak Discharge (m3/s)	Peak Storage Volume (m3)	Spillway Engaged
Wetland 3	1 in 100	2.28	2.24	2,970	Y	2.42	3,040	Y
Wetland 2	1 in 100	1.85	2.06	1,940	Y	2.37	2,030	Y
Wetland 1	1 in 100	2.63	3.53	3,140	Y	4.29	3,370	Y
Lake 1	1 in 5	15.0	24.43	3,760	Y	24.53	3,760	Y

5.3 Outcomes

Based upon the hydrology review the following outcomes have been identified:

- The increased density of development has not influenced the peak flow along Goose Gully, due to the upstream catchment been significantly larger.
- The increased density of development has altered the performance of the various basins within the development. These changes in relation to the previous methodology is provided in Section 5. The changed performance in the RORB modelling has not altered peak flows along Goose Gully.
- As peak flows have not been affected by the changes, there is no change to the previous assumptions and methodology applied to the Rock Chute designs within Goose Gully. Assuming the design criteria and methodology was previously accepted there is not requirement to undertake further assessment.
- Peak flows and discharge from the development through proposed storages has changed as a result of the development, the implications of this is discussed in Section 5.2.

6. FLOOD MODELLING

The previous hydraulic modelling undertaken adopted a single inflow at the upstream end of the development along Goose Gully. As the hydrology has not changed there was no need to re-run the hydraulic modelling. The previous result of the hydraulic modelling is provided in Figure 6 to Figure 7.

The waterway was modelled in Mike21 for both existing and developed conditions. The modelling was completed to ensure that velocities within the channel and through the wetland did not exceed scour values. This preliminary modelling confirmed that the proposed design will work efficiently without threatening the both the remaining habitat, and the proposed design features. The modelling also confirmed the size of all proposed culverts within the development.

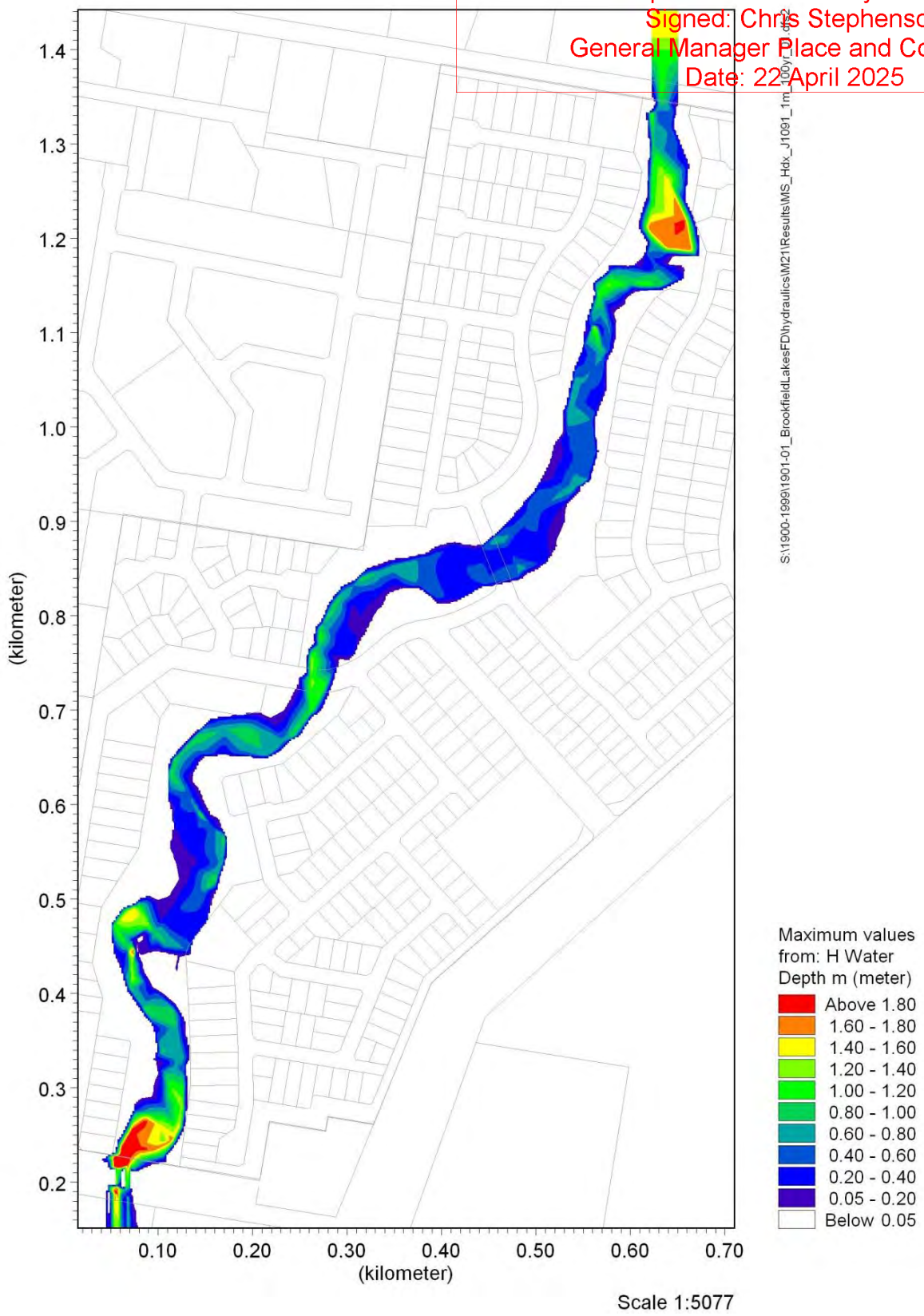


Figure 6 Brookfield Lakes 100y ARI existing conditions

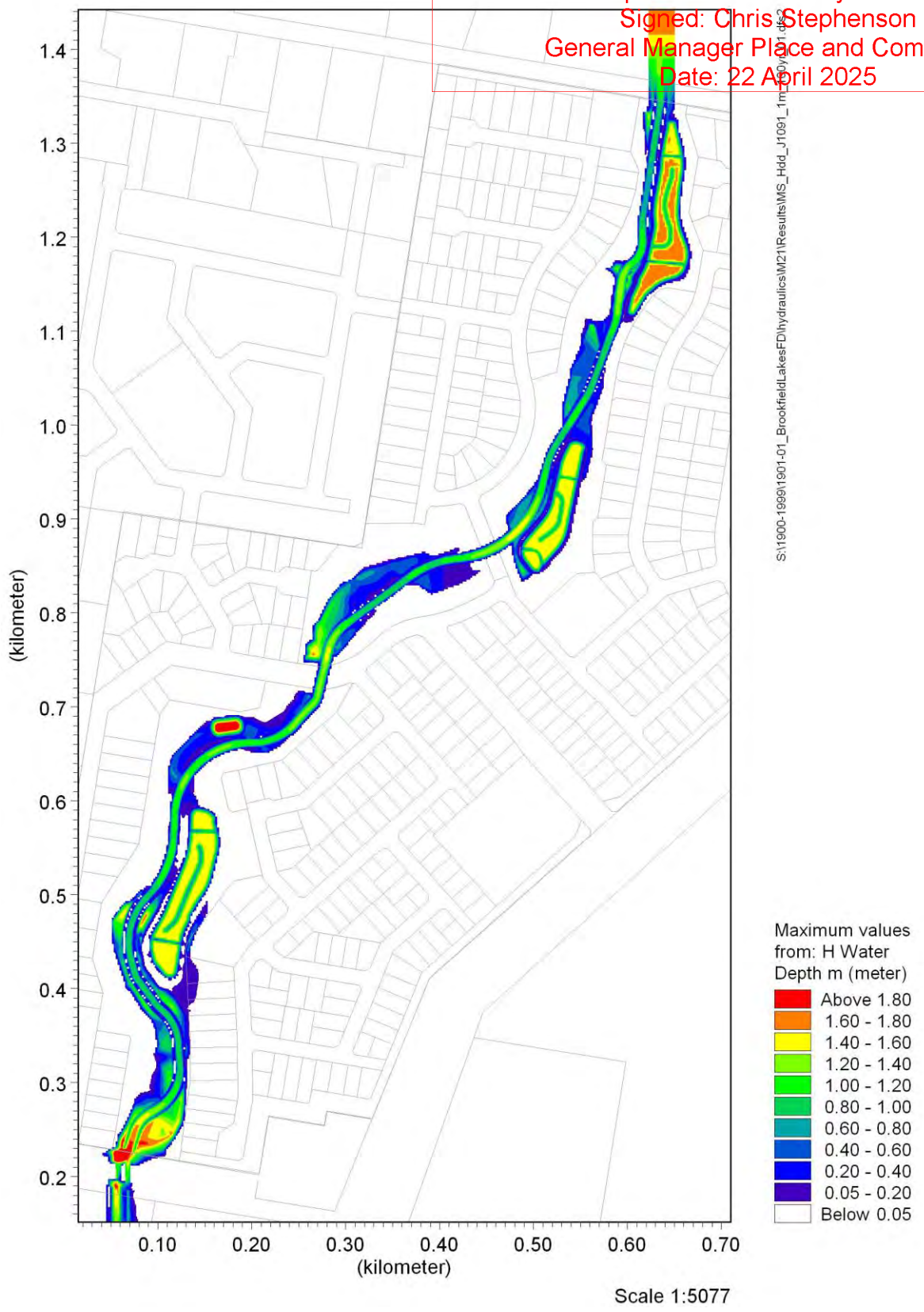


Figure 7 Brookfield Lakes 100y ARI preliminary developed conditions

7. RUNOFF FREQUENCY

As a result of waterway improvement, wetland and sedimentation ponds construction, it is expected that more than 18,000m^3 effective flood plain storage will be provided. In the Water Technology (2008) report, it states 7,300m^3 storage will be required to retard the developed flow back to existing level. Runoff frequency analysis (**Figure 8**) conducted in MUSIC shows that only negligible increase of runoff frequency is likely occur due to the development, and mostly the increases are for the flows less than 0.2 m^3/s.

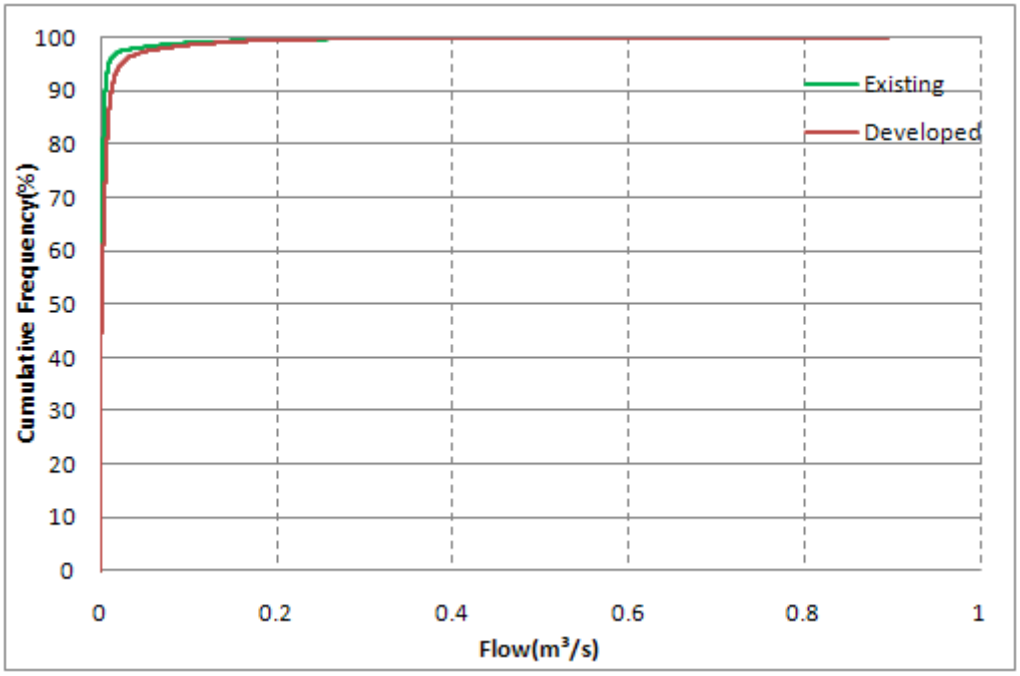


Figure 8 Cumulative runoff frequency

8. WATER QUALITY

The treatment performance of the WSUD features for the proposed residential development has been evaluated using CRC for Catchment Hydrology MUSIC model (**Figure 9** & **Table 8-1**). Specific catchments draining to each of the WSUD features has been schematised. It is apparent from **Table 8-2** that the water quality objectives are exceeded by the proposed stormwater treatment train and hence the design meets Best Practice.

Table 8-1 MUSIC modelling inputs

Catchment	Area (ha)	Ex FI (%)	Dev FI (%)
CAT1	2.297	10	75
CAT2	2.571	10	10
CAT3	5.6	10	75
CAT4	4.817	10	75
CAT5	14.147	10	40
CAT6	4.44	10	75
CAT7	3.075	10	75
CAT8	6.51	10	75
CAT9	4.215	10	80
CAT10	2.352	10	10
CAT11	7.785	10	75
CAT12	1.965	10	80
CAT13	1.34	10	75
CAT14	2.089	10	10
CAT15	5.146	10	10
CAT16	0.291	10	75

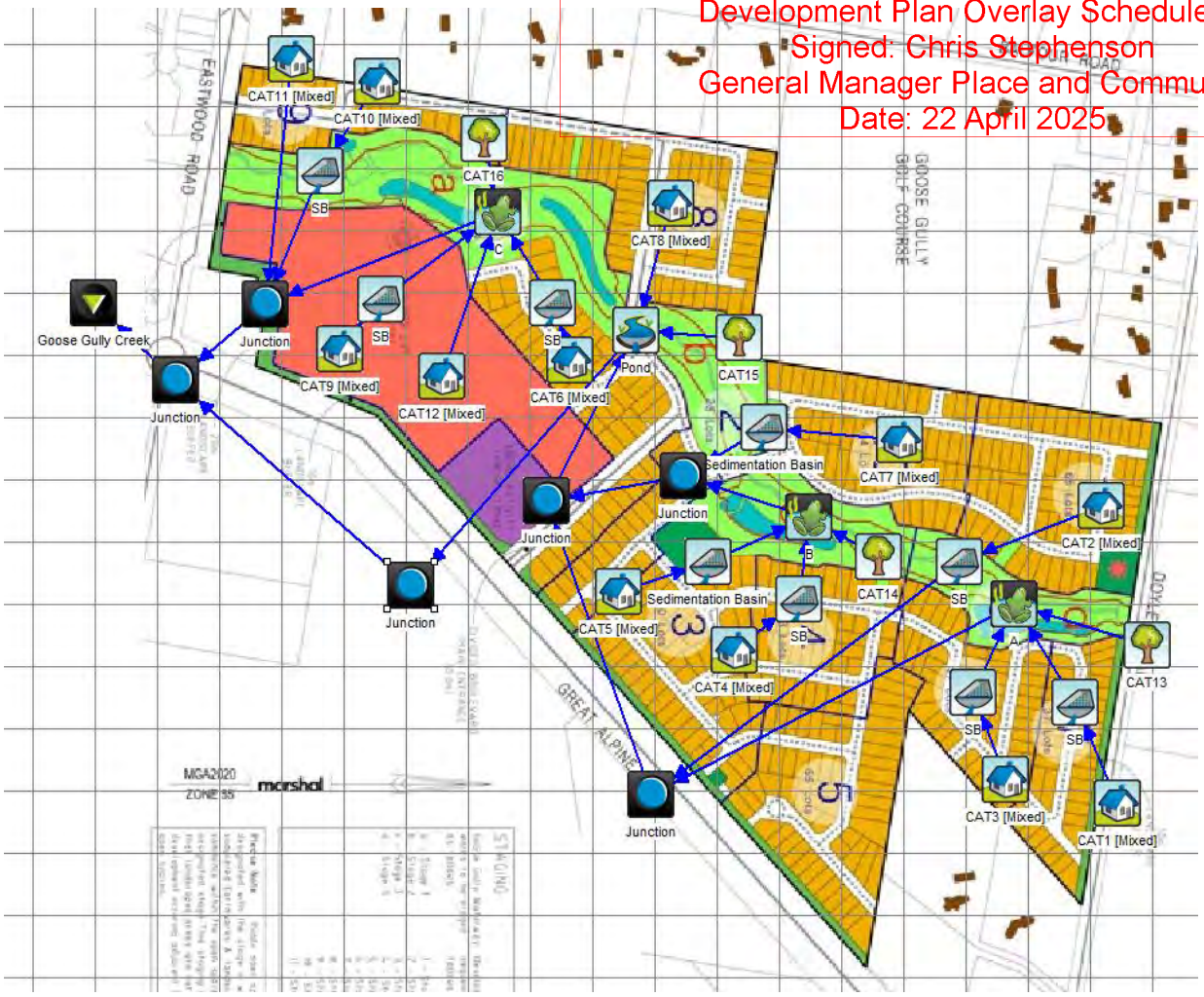


Figure 9 MUSIC model layout

Table 8-2 MUSIC modelling results

		No WSUD treatment features	With WSUD treatment features	
Parameter	**Target Reduction**	**Annual pollutant load**	**Annual pollutant load**	**Reduction**
Suspended Solids (TSS)	80%	61100 kg/yr	8040 kg/yr	86.8 %
Total Phosphorus (TP)	45%	127 kg/yr	33.7 kg/yr	73.4%
Total Nitrogen (TN)	45%	881 kg/yr	431 kg/yr	51.1%
Gross Pollutants	70%	10100 kg/yr	230 kg/yr	97.7%

APPENDIX A SEDIMENTATION POND CALCULATIONS

Note: As the design has been raised vertically post original calculation, differences may be found in the elevations between design drawings and the calculations. However, the relative levels in elevations have not changed and thus the sizing does not change.

SP1

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	220	m^3
Permanent pool Depth	1	m

Hydrology

A(ha)	5.6
$t_c = 7.6A^{0.38}$	14.63
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	22

Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.14**
1	1	0.8	31.474	**0.28**
2	2	0.85	41.524	**0.39**
3	5	0.95	55.748	**0.59**
4	10	1	65.748	**0.73**
5	20	1.05	78.122	**0.91**
6	50	1.15	96.122	**1.23**
7	100	1.2	110.496	**1.47**

	outlet pit	*P35*	
Weir Flow condition	**$p=Q_{des}/(B\times C_w\times H^{1.5})$**		
P(perimeter of the outlet pit,m)	3.39	?	equivalent size
B(Blockage factor)	0.5		**900x900**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.73**	
Orifice Flow condition	**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**		
A_o(Orifice Area(m2)	0.87	?	equivalent size
C_d(Orifice Discharge Coeffieicnet)	0.6		**1300x900**
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.73**	
OverFlow Weir			
Weir Flow condition	**$p=Q_{des}/(C_w\times H^{1.5})$**		
P(perimeter of the outlet pit,m)	5.26	?	

C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	1.41	
Maintenance drain			***p163***
$A_o=Q/(C_d x(2gh)^{0.5})$			
Ao(Orifice Area m2)	0.003		
Diameter(mm)	33	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.33		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.005		
Outlet Pipe			***p164***
Outlet Control	head loss = $(Ke+Kex)\times V^2/2g+ S_f\times L$		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.6	m
RCP pipe radius =		0.3	m
Design flow (ARI) =	10	0.73	m^3/s
Wetted perimeter =		1.89	m
Area =		0.28	m^2
Hyd radius =		0.15	m
V =		2.58	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		8.3	
$S_{f(\text{ Friction slope})}$ =		0.0141	
Head loss =		0.625	m
Total Head at TED	OK	1.400	m
NWL		25.500	m
Outlet Pipe upstream Invert		24.5	m
Outlet Pipe Downstream Invert/WL		24.2	m
Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$		OK
pipe dia =	0.6	m	
Number of pipes	1		
Area =	0.28278	m^2	

	Elevation	H (m)	Q(m^3/s)
Invert	24.500	0.000	0.000
NWL	25.500	0.700	0.629
TED	25.900	1.100	0.788
Spillway Invert	26	1.200	0.823

Sp2

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	186	m^3
Permanent pool Depth	1	m

Hydrology

A(ha)	3.075
$t_c = 7.6A^{0.38}$	11.65
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	19

Index	ARI	Fy	I(mm/hr)	Q(m3/s)
0	0.5			**0.08**
1	1	0.8	34.860	**0.17**
2	2	0.85	46.136	**0.24**
3	5	0.95	62.707	**0.36**
4	10	1	73.061	**0.44**
5	20	1.05	87.414	**0.56**
6	50	1.15	107.768	**0.75**
7	100	1.2	124.768	**0.91**

	outlet pit	*P35*	
Weir Flow condition	**p=Q_{des}/(B×C_w×H^1.5)**		
P(perimeter of the outlet pit,m)	2.07	?	equivalent size
B(Blockage factor)	0.5		**600x600**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.44**	
Orifice Flow condition	**A_o=Q_{des}/(B×C_d×(2gH)^0.5)**		
A_o(Orifice Area(m2)	0.53	?	equivalent size
C_d(Orifice Discharge Coeffieicnet)	0.6		**900x600**
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.44**	
	OverFlow Weir		
Weir Flow condition	**p=Q_{des}/(C_w×H^1.5)**		
P(perimeter of the outlet pit,m)	3.26	?	
C_w(weir coefficient)	1.7		

H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	0.91	
Maintenance drain			***p163***
$A_o=Q/(C_d x(2gh)^{0.5})$			
Ao(Orifice Area m2)	0.003		
Diameter(mm)	30	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.33		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.004		
Outlet Pipe			***p164***
Outlet Control	head loss = $(Ke+Kex)\times V^2/2g+ S_f\times L$		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.45	m
RCP pipe radius =		0.225	m
Design flow (ARI) =	10	0.44	m^3/s
Wetted perimeter =		1.41	m
Area =		0.16	m^2
Hyd radius =		0.1125	m
V =		2.80	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		20	
$S_{f(\text{ Friction slope})}$ =		0.0244	
Head loss =		1.085	m
Total Head at TED	OK	1.275	m
NWL		23.300	m
Outlet Pipe upstream Invert		22.3	m
Outlet Pipe Downstream Invert/WL		22.2	m
Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$		OK
pipe dia =	0.45	m	
Number of pipes	1		
Area =	0.159064	m^2	

	Elevation	H (m)	Q(m^3/s)
Invert	22.300	0.000	0.000
NWL	23.300	0.775	0.372
TED	23.700	1.175	0.458
Spillway Invert	24	1.475	0.513

SP3

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	166	m^3
Permanent pool Depth	1	m

Hydrology

A(ha)	1.965
$t_c = 7.6A^{0.38}$	9.82
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	17

Index	ARI	Fy	I(mm/hr)	Q(m3/s)
0	0.5			**0.06**
1	1	0.8	37.481	**0.12**
2	2	0.85	49.546	**0.16**
3	5	0.95	67.704	**0.25**
4	10	1	79.704	**0.31**
5	20	1.05	95.056	**0.39**
6	50	1.15	117.704	**0.53**
7	100	1.2	136.056	**0.64**

	outlet pit	*P35*	
Weir Flow condition	**p=Q_{des}/(B×C_w×H^1.5)**		
P(perimeter of the outlet pit,m)	1.44	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.31**	
Orifice Flow condition	**A_o=Q_{des}/(B×C_d×(2gH)^0.5)**		
A_o(Orifice Area(m2)	0.37	?	equivalent size
C_d(Orifice Discharge Coeffieicnet)	0.6		**900x600**
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.31**	

OverFlow Weir		
Weir Flow condition	**p=Q_{des}/(C_w×H^1.5)**	
P(perimeter of the outlet pit,m)	2.27	?
C_w(weir coefficient)	1.7	

H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	0.64	
Maintenance drain			***p163***
$Ao=Q/(C_d x(2gh)^{0.5})$			
Ao(Orifice Area m2)	0.003		
Diameter(mm)	28	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.33		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.004		
Outlet Pipe			***p164***
Outlet Control	head loss = $(Ke+Kex)\times V^2/2g+ S_f\times L$		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.375	m
RCP pipe radius =		0.1875	m
Design flow (ARI) =	10	0.31	m^3/s
Wetted perimeter =		1.18	m
Area =		0.11	m^2
Hyd radius =		0.09375	m
V =		2.81	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		20	
$S_{f(\text{ Friction slope})}$ =		0.0313	
Head loss =		1.229	m
Total Head at TED	OK	1.513	m
NWL		17.300	m
Outlet Pipe upstream Invert		16.3	m
Outlet Pipe Downstream Invert/WL		16	m
Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$		OK
pipe dia =	0.375	m	
Number of pipes	1		
Area =	0.110461	m^2	

	Elevation	H (m)	Q(m^3/s)
Invert	16.300	0.000	0.000
NWL	17.300	0.813	0.265
TED	17.700	1.213	0.323
Spillway Invert	17.7	1.213	0.323

APPENDIX B WETLAND CALCULATIONS

Wetland 1

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	809.482	m^3
Permanent pool Depth	0.5	m

Wetland Outlet structure

A(ha)	16.444
$t_c = 7.6A^{0.38}$	22.02
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	30

Index	ARI	Fy	I(mm/hr)	Q(m3/s)
0	0.5			**0.34**
1	1	0.8	25.786	**0.67**
2	2	0.85	33.881	**0.94**
3	5	0.95	45.174	**1.40**
4	10	1	52.952	**1.72**
5	20	1.05	62.952	**2.15**
6	50	1.15	75.976	**2.85**
7	100	1.2	87.952	**3.44**

	OverFlow Weir		
Weir Flow condition	**$p=Q_{des}/(C_w \times H^{1.5})$**		
P(perimeter of the outlet pit,m)	12.31	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	3.44	
	Maintenance drain		***p163***
	$Ao=Q/(C_d x(2gh)^{0.5})$		
Ao(Orifice Area m2)	0.030		
Diameter(mm)	99	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.17		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	33.090	L/s	
Total Permanent Volume of connected SediPond	620		
Wetland	**Outlet**	***Pipe***	***p164***
Outlet Control	head loss = $(Ke+Kex) \times V^2/2g + S_f \times L$		

$S_f = Q^2n^2/A^2R^{4/3}$

Number of pipes		1	
pipe dia =		0.225	m
RCP pipe radius =		0.1125	m
Design flow (max from the riser or maintenance discharge) =		0.033	m3/s
Wetted perimeter =		0.71	m
Area =		0.04	m^2
Hyd radius =		0.05625	m
V =		0.83	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		20	
$S_{f(\text{ Friction slope})}$ =		0.0054	
Head loss =		0.162	m
Total Head at TED	OK	1.288	m
NWL		25.000	m
Outlet Pipe upstream Invert		24.5	m
Outlet Pipe Downstream Invert/WL		24	m

Check for Inlet Control

$Q = 0.6 \times A \times (2gH)^{0.5}$

pipe dia =	0.225	m
Number of pipes	1	
Area =	0.039766	m^2

	Elevation	H (m)	$Q(m^3/s)$
Invert	24.500	0.000	0.000
NWL	25.000	0.388	0.066
TED	25.400	0.787	0.094
Spillway Invert	25.7	1.088	0.110

Riser sizing

				Q1	Q2	Q3	Q4	total	Notional
Orifice Positions (invert level)				0	0.1	0.2	0.3	Flow	Detention Time
Orifice Diameter (mm)				20	20	20	20	(L/s)	(hrs)
Number				2	3	1	1		
Elevation	Volume	Relative Depth	Volume						
25	809.482	0	0	0				0.00	
25.1	985.638	0.1	176	0.501				0.50	97.64
25.2	1182.853	0.2	373	0.728	0.752			1.48	70.08
25.3	1399.684	0.3	590	0.900	1.092	0.251		2.24	73.11
25.4	1633.274	0.4	824	1.043	1.349	0.364	0.251	3.01	76.09

Sedimentation pond W1A

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	456	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 1

A(ha)	14.147			
$t_c = 7.6A^{0.38}$	20.80			
$^{10}I_{1(mm/hr)}$	28.90			
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152			
FI	0.75			
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713			
ColNum	28			
Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.30**
1	1	0.8	26.520	**0.59**
2	2	0.85	34.880	**0.83**
3	5	0.95	46.640	**1.24**
4	10	1	54.200	**1.52**
5	20	1.05	64.400	**1.89**
6	50	1.15	79.400	**2.56**
7	100	1.2	90.600	**3.05**

	Entry pit *P35*		
Weir Flow condition	**$p=Q_{des}/(B\times C_w\times H^{1.5})$**		
P(perimeter of the outlet pit,m)	2.76	?	equivalent size
B(Blockage factor)	0.5		**900x600**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.59**	
Orifice Flow condition	**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**		
A_o(Orifice Area(m2)	0.71	?	equivalent size
C_d(Orifice Discharge Coeffieicnet)	0.6		**900x900**
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.59**	

	OverFlow Weir	
Weir Flow condition	**$p=Q_{des}/(C_w\times H^{1.5})$**	
P(perimeter of the outlet pit,m)	8.16	?
C_w(weir coefficient)	1.7	

H(depth of water above the Crest of outlet pit, m)	0.2		
Q_{des}(Design flow m3/s)----ARI	5	1.24	
	Maintenance drain		***p163***
	$A_o=Q/(C_d x(2gh)^{0.5})$		
Ao(Orifice Area m2)	0.006		
Diameter(mm)	45	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.40		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.011		
	Inlet pipe		***p164***
Outlet Control	head loss = $(Ke+Kex)\times V^2/2g+ S_f\times L$		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.75	m
RCP pipe radius =		0.375	m
Design flow (usually 1 yr ARI) =	1	0.59	m^3/s
Wetted perimeter =		2.36	m
Area =		0.44	m^2
Hyd radius =		0.1875	m
V (less than 0.05 ideally)=		1.35	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		8	
$S_{f(\text{ Friction slope})}$ =		0.0029	
Head loss =		0.161	m
NWL	OK	25.2	m
Outlet Pipe Invert		24.289	m
Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$		OK
pipe dia =	0.75	m	
Number of pipes	1		
Area =	0.44184375	m^2	

	Elevation	H (m)	Q(m^3/s)
Pipe Invert	24.289	0.000	0.000
NWL	25.2	0.536	0.860
FULL	25.6	0.936	1.136
Other Levels	25.8	1.136	1.252
Other Levels	26	1.336	1.357

Sedimentation pond W1B

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	164	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 2

A(ha)	2.297
$t_c = 7.6A^{0.38}$	10.42
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	18

Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.07**
1	1	0.8	36.563	**0.13**
2	2	0.85	48.451	**0.19**
3	5	0.95	65.727	**0.28**
4	10	1	77.302	**0.35**
5	20	1.05	92.302	**0.44**
6	50	1.15	114.453	**0.60**
7	100	1.2	132.029	**0.72**

	Entry pit	*P35*	
Weir Flow condition		**$p=Q_{des}/(B\times C_w\times H^{1.5})$**	
P(perimeter of the outlet pit,m)	0.62	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.13**	
Orifice Flow condition		**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**	
A_o(Orifice Area(m2)	0.16	?	equivalent size
C_d(Orifice Discharge Coeffieicnet)	0.6		**450x450**
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.13**	

	OverFlow Weir	$p=Q_{des}/(C_w \times H^{1.5})$	
Weir Flow condition			
P(perimeter of the outlet pit,m)	1.87	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.2		
Q_{des}(Design flow m3/s)----ARI	5	0.28	
	Maintenance drain		***p163***
		$Ao=Q/(C_d x(2gh)^{0.5})$	
Ao(Orifice Area m2)	0.002		
Diameter(mm)	27	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.40		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.004		
	Inlet pipe		***p164***
Outlet Control	head loss = $(Ke+Kex)\times V^2/2g+ S_f \times L$		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.375	m
RCP pipe radius =		0.1875	m
Design flow (usually 1 yr ARI) =	1	0.13	m^3/s
Wetted perimeter =		1.18	m
Area =		0.11	m^2
Hyd radius =		0.09375	m
V (less than 0.05 ideally)=		1.20	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		8	
$S_{f(\text{ Friction slope})}$ =		0.0058	
Head loss =		0.157	m
NWL	OK	25.2	m
Outlet Pipe Invert		24.668	m
Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$		OK
pipe dia =	0.375	m	
Number of pipes	1		
Area =	0.110461	m^2	

	Elevation	H (m)	Q(m^3/s)
Pipe Invert	24.668	0.000	0.000
NWL	25.2	0.345	0.172
FULL	25.6	0.745	0.253
Other Levels	25.8	0.945	0.285
Other Levels	26	1.145	0.314

Wetland 2

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	835.921	m^3
Permanent pool Depth	0.5	m

Wetland Outlet structure

A(ha)	9.257			
$t_c = 7.6A^{0.38}$	17.70			
$^{10}I_{1(mm/hr)}$	28.90			
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152			
FI	0.75			
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713			
ColNum	25			
Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.21**
1	1	0.8	28.737	**0.42**
2	2	0.85	37.826	**0.59**
3	5	0.95	50.592	**0.88**
4	10	1	59.592	**1.09**
5	20	1.05	70.592	**1.36**
6	50	1.15	86.592	**1.83**
7	100	1.2	99.888	**2.20**

OverFlow Weir			
Weir Flow condition	**p=Q_{des}/(C_w×H^1.5)**		
P(perimeter of the outlet pit,m)	7.87	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	2.20	
Maintenance drain			***p163***
	Ao=Q/(C_dx(2gh)^0.5)		
Ao(Orifice Area m2)	0.035		
Diameter(mm)	106	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.17		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	37.961	L/s	
Total Permanent Volume of connected SediPond	804		

Wetland	Outlet	Pipe	p164
Outlet Control	head loss = (Ke+Kex)xV²/2g+ SfxL		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.225	m
RCP pipe radius =		0.1125	m
Design flow (max from the riser or maintenance discharge) =		0.038	m3/s
Wetted perimeter =		0.71	m
Area =		0.04	m^2
Hyd radius =		0.05625	m
V =		0.95	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		20	
$S_{f(\text{ Friction slope})}$ =		0.0071	
Head loss =		0.213	m
Total Head at TED	OK	1.288	m
NWL		22.000	m
Outlet Pipe upstream Invert		21.5	m
Outlet Pipe Downstream Invert/WL		21	m

Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$			
pipe dia =		0.225	m	
Number of pipes		1		
Area =		0.039766	m^2	
	Elevation		H (m)	Q(m^3/s)
Invert		21.500	0.000	0.000
NWL		22.000	0.388	0.066
TED		22.400	0.787	0.094
Spillway Invert		22.7	1.088	0.110

				Q1	Q2	Q3	Q4	total	Notional
Orifice Positions (invert level)				0	0.1	0.2	0.3	Flow	Detention Time
Orifice Diameter (mm)				20	20	20	20	(L/s)	(hrs)
Number				2	2	2	1		
Elevation	Volume	Relative Depth	Volume						
22	835.921	0	0	0				0.00	
22.1	1004.928	0.1	169	0.501				0.50	93.68
22.2	1191.487	0.2	356	0.728	0.501			1.23	80.34
22.3	1394.698	0.3	559	0.900	0.728	0.501		2.13	72.91
22.4	1611.999	0.4	776	1.043	0.900	0.728	0.251	2.92	73.79

Sedimentation pond W2A

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	338	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 1

A(ha)	4.44
$t_c = 7.6A^{0.38}$	13.39
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	21

Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.12**
1	1	0.8	32.770	**0.23**
2	2	0.85	43.313	**0.32**
3	5	0.95	58.218	**0.49**
4	10	1	68.218	**0.60**
5	20	1.05	81.826	**0.76**
6	50	1.15	100.435	**1.02**
7	100	1.2	116.044	**1.22**

	Entry pit *P35*		
Weir Flow condition		**$p=Q_{des}/(B\times C_w\times H^{1.5})$**	
P(perimeter of the outlet pit,m)	1.07	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.23**	
Orifice Flow condition		**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**	

A_o(Orifice Area(m2)	0.27	?	equivalent size
C_d(Orifice Discharge Coeffieicnet)	0.6		600x600
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.23**	

OverFlow Weir

Weir Flow condition		$p=Q_{des}/(C_w \times H^{1.5})$
P(perimeter of the outlet pit,m)	3.20	?
C_w(weir coefficient)	1.7	
H(depth of water above the Crest of outlet pit, m)	0.2	
Q_{des}(Design flow m3/s)----ARI	5	0.49

Maintenance drain *p163*

$Ao=Q/(C_d x(2gh)^{0.5})$

Ao(Orifice Area m2)	0.005	
Diameter(mm)	38	?
C_d(Orifice Discharge Coeffieicnet)	0.6	
h(1 thrid of the permanent pool depth m)	0.40	
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.008	

Inlet pipe *p164*

Outlet Control head loss = $(Ke+Kex)\times V^2/2g+ S_f \times L$

$S_f = Q^2n^2/A^2R^{4/3}$

Number of pipes		1	
pipe dia =		0.525	m
RCP pipe radius =		0.2625	m
Design flow (usually 1 yr ARI) =	1	0.23	m^3/s
Wetted perimeter =		1.65	m
Area =		0.22	m^2
Hyd radius =		0.13125	m
V (less than 0.05 ideally)=		1.06	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		10	
$S_{f(\text{ Friction slope})}$ =		0.0029	
Head loss =		0.115	m
NWL	OK	22.2	m
Outlet Pipe Invert		21.560	m

Check for Inlet Control $Q = 0.6\times A\times(2gH)^{0.5}$ OK

pipe dia = 0.525 m

Number of pipes			
Area =	0.216503438 m^2		
	Elevation	H (m)	Q(m^3/s)
Pipe Invert	21.560	0.000	0.000
NWL	22.2	0.378	0.354
FULL	22.6	0.778	0.507
Other Levels	22.7	0.878	0.539
Other Levels	22.8	0.978	0.569

Sedimentation pond W2B

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	466	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 2

A(ha)	4.817			
$t_c = 7.6A^{0.38}$	13.81			
$^{10}I_{1(mm/hr)}$	28.90			
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152			
FI	0.75			
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713			
ColNum	21			
Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.12**
1	1	0.8	32.306	**0.25**
2	2	0.85	42.681	**0.35**
3	5	0.95	57.375	**0.52**
4	10	1	67.375	**0.64**
5	20	1.05	80.563	**0.81**
6	50	1.15	98.750	**1.08**
7	100	1.2	113.938	**1.30**

	Entry pit	*P35*	
Weir Flow condition		**$p=Q_{des}/(B\times C_w\times H^{1.5})$**	
P(perimeter of the outlet pit,m)	1.15	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.25**	
Orifice Flow condition		**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**	
A_o(Orifice Area(m2)	0.29	?	equivalent size

C_d(Orifice Discharge Coeffieicnet)	0.6	600x600	
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.25**	

OverFlow Weir			
Weir Flow condition		**p=Q_{des}/(C_w×H^1.5)**	
P(perimeter of the outlet pit,m)	3.42	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.2		
Q_{des}(Design flow m3/s)----ARI	5	0.52	

Maintenance drain			***p163***
		Ao=Q/(C_dx(2gh)^0.5)	
Ao(Orifice Area m2)	0.006		
Diameter(mm)	45	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.40		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.011		

Inlet pipe			***p164***
Outlet Control	head loss = (Ke+Kex)×V^2/2g+ S_f×L		
	S_f = $Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.525	m
RCP pipe radius =		0.2625	m
Design flow (usually 1 yr ARI) =	1	0.25	m^3/s
Wetted perimeter =		1.65	m
Area =		0.22	m^2
Hyd radius =		0.13125	m
V (less than 0.05 ideally)=		1.14	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		10	
$S_{f(\text{ Friction slope})}$ =		0.0033	
Head loss =		0.132	m
NWL	OK	22.2	m
Outlet Pipe Invert		21.543	m

Check for Inlet Control	$Q = 0.6 \times A \times (2gH)^{0.5}$		OK
pipe dia =	0.525	m	
Number of pipes	1		
Area =	0.216503	m^2	

	Elevation	H (m)	Q(m³/s)
Pipe Invert	21.543	0.000	0.000
NWL	22.2	0.395	0.361
FULL	22.6	0.795	0.513
Other Levels	22.7	0.895	0.544
Other Levels	22.8	0.995	0.574

Wetland 3

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	922.799	m^3
Permanent pool Depth	0.5	m

Wetland Outlet structure

A(ha)	14.586			
$t_c = 7.6A^{0.38}$	21.04			
$^{10}I_{1(mm/hr)}$	28.90			
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152			
FI	0.8			
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.7504			
ColNum	29			
Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.32**
1	1	0.8	26.374	**0.64**
2	2	0.85	34.666	**0.90**
3	5	0.95	46.348	**1.34**
4	10	1	53.957	**1.64**
5	20	1.05	63.957	**2.04**
6	50	1.15	78.871	**2.76**
7	100	1.2	89.914	**3.28**

	OverFlow Weir		
Weir Flow condition	**$p=Q_{des}/(C_w \times H^{1.5})$**		
P(perimeter of the outlet pit,m)	11.74	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	3.28	
	Maintenance drain		***p163***
	$Ao=Q/(C_d x(2gh)^{0.5})$		
Ao(Orifice Area m2)	0.049		
Diameter(mm)	125	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.17		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	53.329	L/s	
Total Permanent Volume of connected SediPond	1381		
Wetland	**Outlet**	***Pipe***	***p164***
Outlet Control	head loss = $(Ke+Kex) \times V^2/2g + S_f \times L$		

$S_f = Q^2n^2/A^2R^{4/3}$

Number of pipes		1	
pipe dia =		0.225	m
RCP pipe radius =		0.1125	m
Design flow (max from the riser or maintenance discharge) =		0.053	m3/s
Wetted perimeter =		0.71	m
Area =		0.04	m^2
Hyd radius =		0.05625	m
V =		1.34	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		20	
$S_{f(\text{ Friction slope})}$ =		0.0141	
Head loss =		0.420	m
Total Head at TED	OK	0.788	m
NWL		15.000	m
Outlet Pipe upstream Invert		14	m
Outlet Pipe Downstream Invert/WL		14.5	m

Check for Inlet Control

$Q = 0.6 \times A \times (2gH)^{0.5}$

pipe dia =	0.225	m
Number of pipes	1	
Area =	0.039766	m^2

	Elevation	H (m)	$Q(m^3/s)$
Invert	14.000	0.000	0.000
NWL	15.000	0.888	0.100
TED	15.400	1.288	0.120
Spillway Invert	15.7	1.588	0.133

				Q1	Q2	Q3	Q4	total	Notional
Orifice Positions (invert level)				0	0.1	0.2	0.3	Flow	Detention Time
Orifice Diameter (mm)				20	20	20	20	(L/s)	(hrs)
Number				3	2	2	2		
Elevation	Volume	Relative Depth	Volume						
15	922.799	0	0	0				0.00	
15.1	1130.671	0.1	208	0.752				0.75	76.81
15.2	1363.203	0.2	440	1.092	0.501			1.59	76.78
15.3	1618.85	0.3	696	1.349	0.728	0.501		2.58	74.98
15.4	1893.972	0.4	971	1.565	0.900	0.728	0.501	3.69	73.03

Sedimentation pond W3A

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	458	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 1

A(ha)	7.785			
$t_c = 7.6A^{0.38}$	16.58			
$^{10}I_{1(mm/hr)}$	28.90			
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152			
FI	0.75			
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713			
ColNum	24			
Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.18**
1	1	0.8	29.639	**0.37**
2	2	0.85	39.108	**0.51**
3	5	0.95	52.424	**0.77**
4	10	1	61.424	**0.95**
5	20	1.05	72.847	**1.18**
6	50	1.15	89.271	**1.58**
7	100	1.2	103.271	**1.91**

	Entry pit *P35*		
Weir Flow condition	**$p=Q_{des}/(B\times C_w\times H^{1.5})$**		
P(perimeter of the outlet pit,m)	1.70	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.37**	
Orifice Flow condition	**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**		
A_o(Orifice Area(m2)	0.44	?	equivalent size

C_d(Orifice Discharge Coeffieicnet)	0.6	900×600	
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.37**	

OverFlow Weir

Weir Flow condition		**$p=Q_{des}/(C_w \times H^{1.5})$**	
P(perimeter of the outlet pit,m)	5.05	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.2		
Q_{des}(Design flow m3/s)----ARI	5	0.77	

Maintenance drain *p163*

$Ao=Q/(C_d x(2gh)^{0.5})$

Ao(Orifice Area m2)	0.006		
Diameter(mm)	45	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.40		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.011		

Inlet pipe *p164*

Outlet Control

head loss = $(Ke+Kex)\times V^2/2g + S_f \times L$

$S_f = Q^2 n^2 / A^2 R^{4/3}$

Number of pipes		1	
pipe dia =		0.6	m
RCP pipe radius =		0.3	m
Design flow (usually 1 yr ARI) =	1	0.37	m^3/s
Wetted perimeter =		1.89	m
Area =		0.28	m^2
Hyd radius =		0.15	m
V (less than 0.05 ideally)=		1.29	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		8	
$S_{f(\text{ Friction slope})}$ =		0.0035	
Head loss =		0.156	m
NWL	OK	15.2	m
Outlet Pipe Invert		14.444	m

Check for Inlet Control — $Q = 0.6 \times A \times (2gH)^{0.5}$ — OK

pipe dia =	0.6	m
Number of pipes	1	
Area =	0.28278	m^2

	Elevation	H (m)	Q(m³/s)
Pipe Invert	14.444	0.000	0.000
NWL	15.2	0.456	0.506
FULL	15.6	0.856	0.695
Other Levels	16	1.256	0.842
Other Levels	17	2.256	1.129

Sedimentation pond W3B

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	278	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 2

A(ha)	6.51			
$t_c = 7.6A^{0.38}$	15.49			
$^{10}I_{1(mm/hr)}$	28.90			
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152			
FI	0.75			
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713			
ColNum	23			
Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.16**
1	1	0.8	30.613	**0.32**
2	2	0.85	40.415	**0.44**
3	5	0.95	54.025	**0.66**
4	10	1	63.538	**0.82**
5	20	1.05	75.538	**1.02**
6	50	1.15	93.051	**1.38**
7	100	1.2	107.051	**1.66**

	Entry pit *P35*		
Weir Flow condition		**$p=Q_{des}/(B\times C_w\times H^{1.5})$**	
P(perimeter of the outlet pit,m)	1.47	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.32**	
Orifice Flow condition		**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**	
A_o(Orifice Area(m2)	0.38	?	equivalent size

C_d(Orifice Discharge Coeffieicnet)	0.6	900x600	
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.32**	

OverFlow Weir			
Weir Flow condition		**p=Q_{des}/(C_w×H^1.5)**	
P(perimeter of the outlet pit,m)	4.35	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.2		
Q_{des}(Design flow m3/s)----ARI	5	0.66	

Maintenance drain			***p163***
		Ao=Q/(C_dx(2gh)^0.5)	
Ao(Orifice Area m2)	0.004		
Diameter(mm)	35	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.40		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.006		

Inlet pipe				***p164***
Outlet Control		head loss = (Ke+Kex)×V^2/2g+ S_f×L		
		S_f = $Q^2n^2/A^2R^{4/3}$		
Number of pipes			1	
pipe dia =			0.6	m
RCP pipe radius =			0.3	m
Design flow (usually 1 yr ARI) =	1		0.32	m^3/s
Wetted perimeter =			1.89	m
Area =			0.28	m^2
Hyd radius =			0.15	m
V (less than 0.05 ideally)=			1.12	m/s
Ke(inlet loss coe) =			0.5	
Kex(outlet loss coe) =			1	
n =			0.013	
L(length of culvert)=			8	
$S_{f(\text{ Friction slope})}$ =			0.0026	
Head loss =			0.116	m
NWL	OK		15.2	m
Outlet Pipe Invert			14.484	m

Check for Inlet Control	$Q = 0.6 \times A \times (2gH)^{0.5}$		OK
pipe dia =	0.6	m	
Number of pipes	1		
Area =	0.28278	m^2	

	Elevation	H (m)	Q(m³/s)
Pipe Invert	14.484	0.000	0.000
NWL	15.2	0.416	0.485
FULL	15.6	0.816	0.679
Other Levels	16	1.216	0.829
Other Levels	17	2.216	1.119

D – East Gippsland Water – Technical Memorandum

Technical Memorandum

August 28, 2023

To	East Gippsland Water	**Contact No.**	03 5150 4439
Copy to	Georgina Kingsbury	**Email**	gkingsbury@egwater.vic.gov.au
From	Julienne De Los Reyes	**Project No.**	12576586
Revision	0 – S4 – Final For Client Use	**Reviewed**	M Evans
Project Name	Land Development Water Servicing Studies		
Subject	Bairnsdale and Paynesville – Brookfield Lakes		

1. Introduction & purpose

GHD has been engaged by EGW to analyse the works required to service future growth, including the developments proposed for Brookfield Lakes, to assist in determining cost-sharing arrangements between existing and future customers and developments.

The purpose of this memorandum is to document the upgrades required to service the new development, and the corresponding allocation of the infrastructure costs to the developer and EGW.

This memorandum focuses on the contribution required for the Brookfield Lakes development.

1.1 Scope and limitations

The costs associated with the upgrades required to service the development are calculated based on EGW's rate database and cost sharing calculation method, which seeks to fairly distribute the costs of infrastructure between current and future customers, depending on whether a project is required to maintain existing customer service levels, provide adequate service to new and underway developments, or create provision for future developments that are anticipated based on land-use planning data available to EGW.

2. Growth

2.1 Estimated ultimate growth for Brookfield Lakes

The Brookfield Lakes Development is shown in Figure 1. A total of 426 lots have been added in the model as part of the analysis of future system demands as a result of Brookfield Lakes, and an estimation of the potential number of customers that may be connected within the 'superlot' noted on the development plan. Additionally, the future system was modelled with a number of other future known and anticipated developments near the Brookfield Lakes development, based on land use planning available to EGW. The model also contains current customer demands. These are relevant to the cost-sharing assessments, given some assets may serve both Brookfield Lakes customers and other future and existing customers.

With these additional demands, the model was used to identify where upgrades are required in the network and to what degree to which upgrades are triggered by the planned development.

Development Plan,
135 Great Alpine Road, Lucknow
Brookfield Lakes Estate
Planning and Environment Act 1987,
East Gippsland Planning Scheme (Approved March 2025).
Development Plan Overlay Schedule 3.
Signed: Chris Stephenson
General Manager Place and Community
Date: 22 April 2025

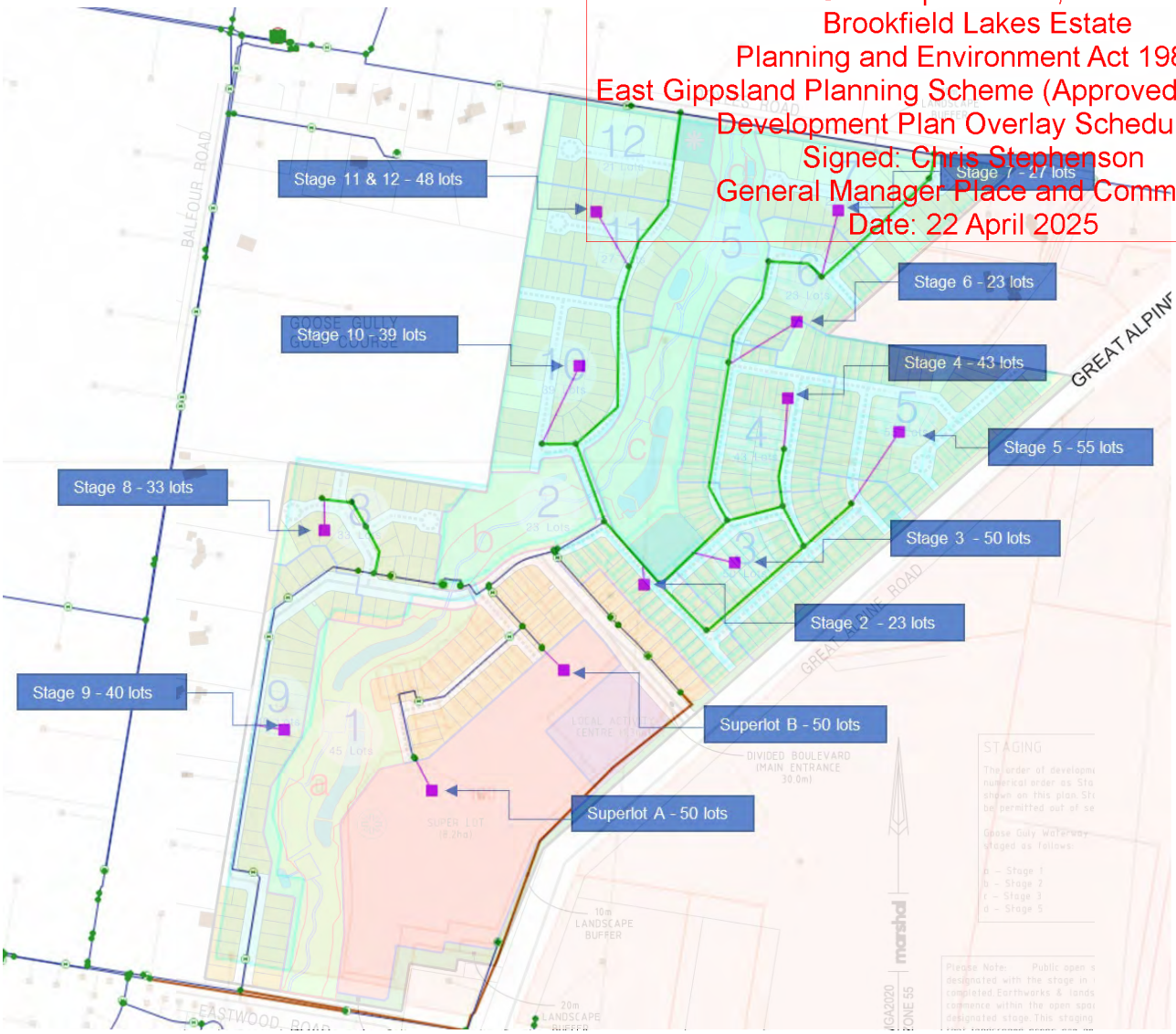


Figure 1 *Brookfield Lakes Development Staging based on REF:3573CP-A, Revision R*

3. Modelling Basis

Hydraulic modelling was utilised to assess impacts of the future demands in the current network. As a basis of planning, technical guides such as Water Services Association of Australia (WSAA) 2011 were used as a guide as to what level of service should be maintained in the system. The planning criteria incudes:

- A minimum pressure of 20 metres at the customer meter
- A maximum pressure of 80 metres at the customer meter
- A maximum headloss gradient of **3 m/km** for transfer mains (DN225 or larger)
- A maximum headloss gradient of **5 m/km** for reticulation mains (DN150 or smaller)
- A maximum velocity of **2 m/s**

Note that high headlosses in the mains does not automatically trigger a pipe upgrade. The minimum pressure in the area remains the primary basis for system upgrades.

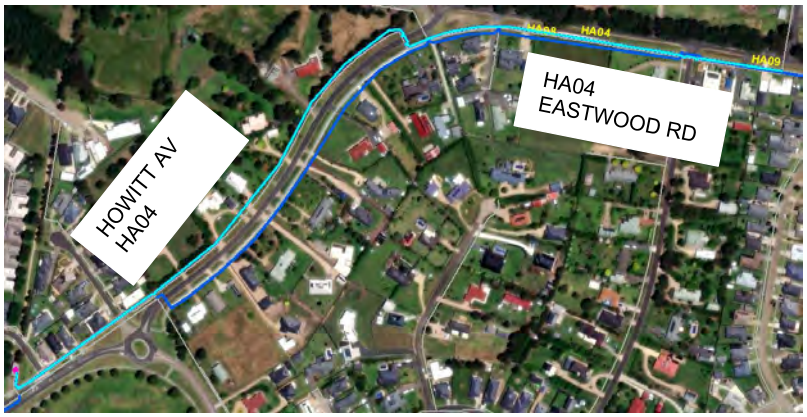
4. Proposed Upgrades and Cost Sharing Allocation

4.1 Proposed Infrastructure Upgrades

Infrastructure upgrades are typically triggered where low pressures in the system are expected to occur due to the future increase in demands. EGW aims to meet WSAA-recommended service pressures, including maintaining a minimum of 20 metres pressure and a maximum 80 metres to all customers in the system, and further seeks to avoid significant reduction in the level of service experienced at existing customer locations. In order to provide or maintain levels of service, the following infrastructure upgrades are necessary.

Table 1 Required system upgrades as a result of Brookfield Lakes Development

Description	Image	Timing
Add PRV at the DN225 entrance to Brookfield Lakes Development (HA00) Required to ensure that the maximum pressures in the current system up to Brookfield Stages 1 to 5 do not exceed 80 m.	 ^PRV (pink) at southern entrance to Brookfield Lakes	ASAP to supply Stages 1 to 5
Upgrade Howitt Av WPS to 83 L/s at 38 m (HA01) Required to supply Brookfield Lakes development		Required when the number of customers in Brookfield Lakes development exceeds 300 (including connections within superlot) or by Stage 9 (if no superlot connections).

Description	Image	Timing
New DN200 pipeline along Eastwood Rd (HA07), up Great Alpine Rd (HA10) and then duplicating the reticulation into east entrance of Brookfield Lakes (HA06) Required to supply Brookfield Lakes development.		Required when the number of customers in Brookfield Lakes development exceeds 300 (including connections within superlot) or by Stage 9 (if no superlot connections).
Duplicate DN300 pipeline along Howitt Ave/ Eastwood Rd (HA04)		Required when the number of customers in Brookfield Lakes development exceeds 400 (including connections within superlot) or by Stage 12 (if no superlot lots).

Description	Image	Timing
Duplicate DN300 pipeline along Howitt Ave except bridge crossing (HA12-1, HA12-3)	 	Required when the number of customers in Brookfield Lakes development exceeds 400 or by Stage 12.

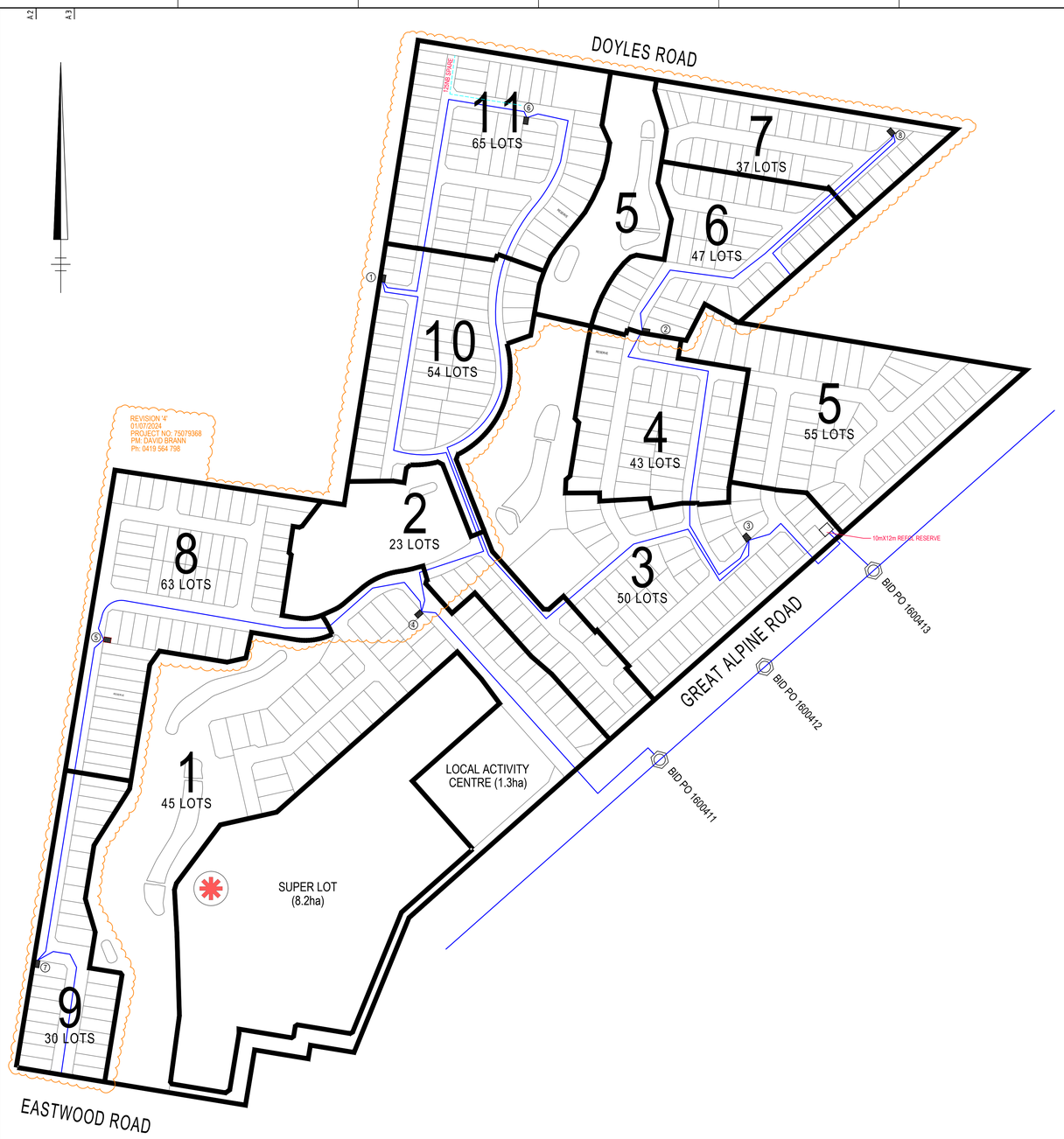
4.2 Cost Sharing Allocation

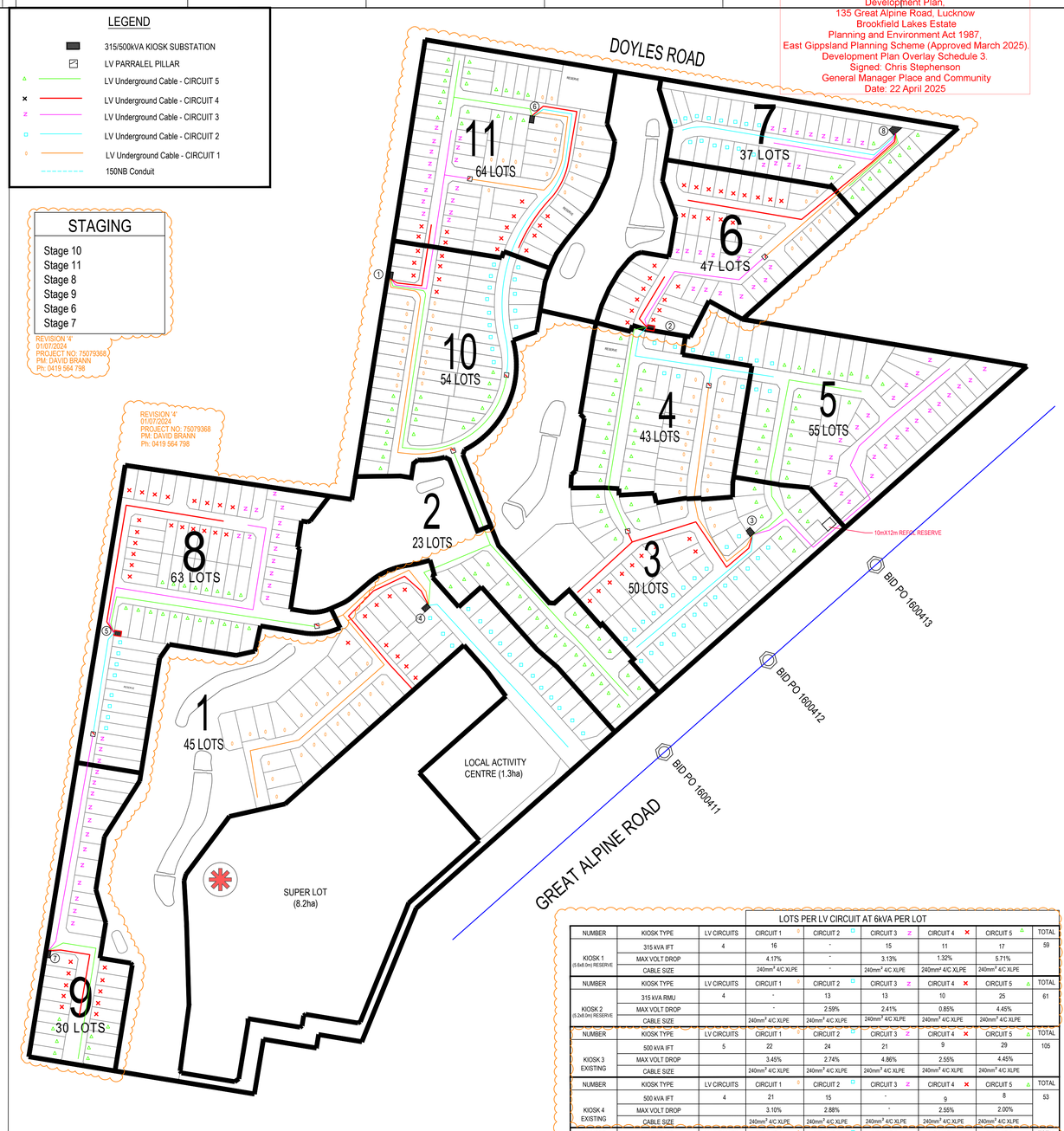
The costs that will be incurred by the need to maintain the desired level of service to existing customers as well as future customers were estimated. A portion of the estimated cost was allocated to the developer based on the number of customers that will benefit from the infrastructure. The table below summarizes the costs and how it was allocated among the developer, EGW, and possible future developers. The total cost of the projects are high level estimates only and is subject to project design. The percentage allocations will be used to determine cost sharing in cases where the total cost does change.

Table 2 Cost Sharing Allocation

Item	Sub-project name	Benefitted Subsystem/ Area	Brookfield Lakes Customers served by asset	Existing + Other Future Growth Customers served	Total Cost	Brookfield Lakes Developer Cost Allocation	EGW Cost Allocation	Cost at each stage for developer
HA00	Add PRV at the DN225 entrance to Brookfield Lakes Development	Brookfield Lakes	426	861	$ 790,000	33%	67%	ASAP to supply Stages 1 to 5: **$ 261,500**
HA01	Upgrade Howitt Avenue WPS to 83 L/s at 38 m	Brookfield Lakes	426	1020	$ 496,120	29%	71%	When lots exceeds 300 (incl superlot connections) or by Stage 9: **$ 341,300**
HA06	Duplicate pipelines DN200 (65m) in Brookfield Lakes	Brookfield Lakes	426	861	$ 41,080	33%	67%	
HA07	New pipeline in Howitt Ave zone DN200 (303m) along Eastwood Rd	Brookfield Lakes	426	861	$ 191,496	33%	67%	
HA10	New pipeline DN200 (565m) extended Howitt Ave zone, along Great Alpine Rd	Brookfield Lakes	426	861	$ 357,080	33%	67%	
HA04	Duplicate pipelines DN300 (1078m) in Howitt Ave/Eastwood Rd	Brookfield Lakes	426	1020	$ 1,021,944	29%	71%	When lots exceeds 400 (incl superlot connections) or by Stage 12: **$ 408,600**
HA 12-1 & HA 12-3	Duplicate pipelines DN300 (1195m) along Howitt Ave except for bridge crossing	Brookfield Lakes	426	4063	$ 1,132,860	9%	91%	

E – Endorsed Electrical Masterplan





LEGEND

- 315/500kVA KIOSK SUBSTATION
- LV PARRALEL PILLAR
- LV Underground Cable - CIRCUIT 5
- LV Underground Cable - CIRCUIT 4
- LV Underground Cable - CIRCUIT 3
- LV Underground Cable - CIRCUIT 2
- LV Underground Cable - CIRCUIT 1
- 150NB Conduit

STAGING

Stage 10
Stage 11
Stage 8
Stage 9
Stage 6
Stage 7

REVISION '4'
01/07/2024
PROJECT NO: 75079368
PM: DAVID BRANN
Ph: 0419 564 798

Development Plan
135 Great Alpine Road, Lucknow
Brookfield Lakes Estate
Planning and Environment Act 1987,
East Gippsland Planning Scheme (Approved March 2025).
Development Plan Overlay Schedule 3.
Signed: Chris Stephenson
General Manager Place and Community
Date: 22 April 2025

50 25 0 50 100 150 200 250

SCALE OF METRES 1 : 2500

LEGEND

- 185mm² 3/c A.X.HC 'H' 22kV UNDERGROUND CABLE (UNLESS OTHERWISE SHOWN) (H/V)
- 185mm² 4/c SA.X.WC LV UNDERGROUND CABLE (UNLESS OTHERWISE SHOWN) (LV)
- 16mm² 4/c C.X.V. LV UNDERGROUND SERVICE CABLE (UNLESS OTHERWISE SHOWN) (LVS)
- 2c-6mm² PUBLIC LIGHTING CABLE
- SUPERVISORY CABLE
- STRAIGHT JOINT (SJ)
- LV MAINS TEE JOINT (MTJ)
- LV SERVICE TEE JOINT (STJ)
- INSULATED END (IE)
- SEALED END (SE)
- U/G CABLE CUT END
- CABLE CONDUIT (125NB = 125mm INSIDE Dia.)
- CABLE CONDUIT (63mm OUTSIDE Dia. UNLESS OTHERWISE SHOWN)
- CABLE HEAD POLE- (CHP)
- STEEL COLUMN & PUBLIC LIGHT
- SERVICE PIT
- PROTECTIVE POST (BOLLARD) PARALLELING PILLAR (P/P)
- PILLAR (P)
- 44/145 HOUSE No / LOT No
- SUBTRANSMISSION POLE
- HV & LV POLE
- HV POLE
- LV POLE
- SINGLE POLE SUBSTATION
- 0.75 DEPTH TO TOP OF CABLE
- 0.2 DEPTH TO TOP OF OBSTRUCTION
- DL DATUM LINE
- PROJ PROJECTION
- (N),(R),(F) UNCONFIRMED WORKS FROM PREVIOUS REVISIONS FOR DIFFERENT PROJECTS
- HV CABLE TERMINATION
- LV CABLE TERMINATION

(N) NEW
(E) EXISTING
(F) FUTURE
(R) REMOVE
BL BUILDING LINE
BOK BACK OF KERB
CENTRE LINE
TP TURNING POINT
SP/D SPARE DUCT
CSW CENTRE STORM WATER
N.O. NORMALLY OPEN
N.C. NORMALLY CLOSED
DE DUCT END

ATTENTION

CONSTRUCTION PLAN ONLY
DO NOT USE FOR CABLE LOCATION

FOR UNDERGROUND CABLE LOCATIONS CONTACT:
METROPOLITAN AREA: MELBOURNE ONE CALL SERVICE - PHONE 1100
OUTSIDE METROPOLITAN AREA: YOUR NEAREST AusNet OFFICE

NOTE

FOR GENERAL INSTALLATION NOTES AND CONDITIONS
REFER TO "NOTES AND CONDITIONS FOR UNDERGROUND RESIDENTIAL DISTRIBUTION CONSTRUCTION PLANS"

NOTES :

1. DESIGN BY DEVELOPER
2. CONSTRUCTION BY DEVELOPER
3. MAXIMUM DEMAND (MD) FOR EACH LOT IS 6kVA / LOT

LOTS PER LV CIRCUIT AT 6kVA PER LOT

NUMBER	KIOSK TYPE	LV CIRCUITS	CIRCUIT 1 △	CIRCUIT 2 □	CIRCUIT 3 Z	CIRCUIT 4 ×	CIRCUIT 5 △	TOTAL
KIOSK 1 (5.6x8.0m) RESERVE	315 kVA IFT	4	16	-	15	11	17	59
	MAX VOLT DROP		4.17%	-	3.13%	1.32%	5.71%	
	CABLE SIZE		240mm² 4/C XLPE	-	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	
NUMBER	KIOSK TYPE	LV CIRCUITS	CIRCUIT 1 △	CIRCUIT 2 □	CIRCUIT 3 Z	CIRCUIT 4 ×	CIRCUIT 5 △	TOTAL
KIOSK 2 (5.2x8.0m) RESERVE	315 kVA RMU	4	-	13	13	10	25	61
	MAX VOLT DROP		-	2.59%	2.41%	0.85%	4.45%	
	CABLE SIZE		240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	
NUMBER	KIOSK TYPE	LV CIRCUITS	CIRCUIT 1 △	CIRCUIT 2 □	CIRCUIT 3 Z	CIRCUIT 4 ×	CIRCUIT 5 △	TOTAL
KIOSK 3 EXISTING	500 kVA IFT	5	22	24	21	9	29	105
	MAX VOLT DROP		3.45%	2.74%	4.86%	2.55%	4.45%	
	CABLE SIZE		240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	
NUMBER	KIOSK TYPE	LV CIRCUITS	CIRCUIT 1 △	CIRCUIT 2 □	CIRCUIT 3 Z	CIRCUIT 4 ×	CIRCUIT 5 △	TOTAL
KIOSK 4 EXISTING	500 kVA IFT	4	21	15	-	9	8	53
	MAX VOLT DROP		3.10%	2.88%	-	2.55%	2.00%	
	CABLE SIZE		240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	
NUMBER	KIOSK TYPE	LV CIRCUITS	CIRCUIT 1 △	CIRCUIT 2 □	CIRCUIT 3 Z	CIRCUIT 4 ×	CIRCUIT 5 △	TOTAL
KIOSK 5 (5.2x8.0m) RESERVE	315 kVA RMU	4	-	8	13	20	19	60
	MAX VOLT DROP		-	1.31%	4.29%	4.22%	4.22%	
	CABLE SIZE		240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	
NUMBER	KIOSK TYPE	LV CIRCUITS	CIRCUIT 1 △	CIRCUIT 2 □	CIRCUIT 3 Z	CIRCUIT 4 ×	CIRCUIT 5 △	TOTAL
KIOSK 6 (5.6x8.0m) RESERVE	315 kVA IFT	4	18	14	-	9	18	59
	MAX VOLT DROP		3.24%	4.90%	-	2.62%	2.16%	
	CABLE SIZE		240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	
	KIOSK TYPE	LV CIRCUITS	CIRCUIT 1 △	CIRCUIT 2 □	CIRCUIT 3 Z	CIRCUIT 4 ×	CIRCUIT 5 △	TOTAL
KIOSK 7 (5.2x8.0m) RESERVE	315 kVA RMU	3	-	-	15	10	8	33
	MAX VOLT DROP		-	-	3.50%	1.40	2.06%	
	CABLE SIZE		240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	
	KIOSK TYPE	LV CIRCUITS	CIRCUIT 1 △	CIRCUIT 2 □	CIRCUIT 3 Z	CIRCUIT 4 ×	CIRCUIT 5 △	TOTAL
KIOSK 8 (5.2x8.0m) RESERVE	315 kVA RMU	5	10	14	12	14	11	61
	MAX VOLT DROP		2.50%	3.48%	1.98%	3.77%	1.07%	
	CABLE SIZE		240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	240mm² 4/C XLPE	

REVISION '4'
01/07/2024
PROJECT NO: 75079368
PM: DAVID BRANN
Ph: 0419 564 798

BEFORE YOU DIG www.byda.com.au
Zero Damage - Zero Harm

ORIGINAL DRAWING IN COLOUR

REFERENCE DRAWINGS

REV	DESCRIPTION	(COMPANY) Project No.	DRAWN	DATE
4	UPDATED STAGES 6-11 LOT BOUNDARIES, REVISED LV & HV CIRCUITS UPDATED KIOSK 1 AND 5 LOCATIONS, ADDED KIOSKS 6-8	(P/PLANT) 75079368	N.N.	2/7/24
3	UPDATED AS PER STAGE 3 & 4 WORKS	(P/PLANT) 75081833	Y.S.	23/4/24
2	KIOSK 1 LOCATION UPDATED ON HV PLAN TO MATCH LV	(P/PLANT) 75079368	G.C.	2/2/22
1	FIRST OBJECTIVE ISSUE.	(P/PLANT) 75079368	G.C.	4/2/22

Surveyor: N/A
Drawn: N.NICOLAOU 02/07/2024
Survey/Drafting Checked: J.TEMPLETON 24/07/2024
Designed: D.MARCHESANI 24/07/2024
Endorsed: Andrew Webber 02-08-2024

OVERALL MASTERPLAN
BROOKFIELD LAKES - OVERALL
GREAT ALPINE ROAD
LUCKNOW

Project Manager: DAVID BRANN
Project Manager PH: 0419 564 798
ORIGINAL Scale: 1:2500
Vic/Roads/Melway: MELWAY
Legacy No
Drawing No: VK5-0340547-001
Rev: 4.0

AusNet

CADD File Name: C:\PPBox\Box\Projects\06. URD\17696 Brookfield Estate - Masterplan\Design\Approved\VK5-0340547-001.dwg
Date Plotted: 5/08/2024

B1 - 679 x 960

[UNCONTROLLED WHEN PRINTED]

B1
AutoCAD

Development Plan,
135 Great Alpine Road, Lucknow
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Signed: Chris Stephenson
General Manager Place and Community
Date: 22 April 2025

Brookfield Lakes WSUD Functional Design Technical Report

August 2024

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

DOCUMENT STATUS

Version	Doc type	Reviewed by	Approved by	Date issued
v01	Report	SEL	CMB	16/09/11
v02	Report	AM	AM	28/08/24

PROJECT DETAILS

Project Name	190101R01v01
Client	Marshal
Client Project Manager	Charles Ho
Water Technology Project Manager	Yafei Zhu
Report Authors	Yafei Zhu, Sabrina Guazzo
Job Number	1901-01
Report Number	R02
Document Name	1901-01R01v02.docx

Cover Photo: Existing farm dam on the proposed development site

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Notting Hill VIC 3168

Telephone (03) 9558 9366
Fax (03) 9558 9365

ACN No. 093 377 283
ABN No. 60 093 377 283

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

Water Technology was engaged by Marshal Pty Ltd to develop a functional design of stormwater treatment system for the proposed development of the Brookfield Lakes, Bairnsdale (Figure 1). This study follows from the Stormwater Management Plan prepared by Water Technology in 2008. The functional design will consist of major waterway improvement for the Goose Gully Creek within the study site, 3 offline wetlands, 3 sedimentation ponds and 1 online lake, as shown in Figure 2. This design meets both the CMA's and Council's requirements and follows Melbourne Waters design guidelines. This report will outline the key design considerations, and it is recommended to be viewed along with the Brookfield Lakes WSUD Functional Design Drawings.

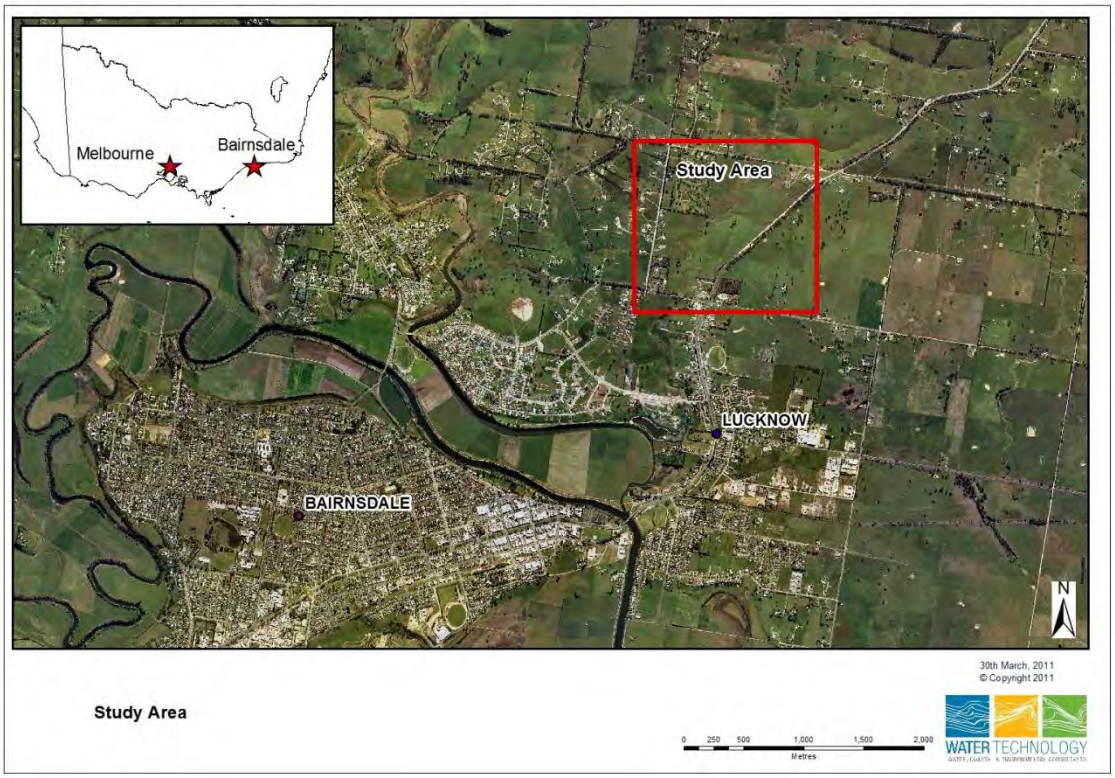

Figure 1 Location Map

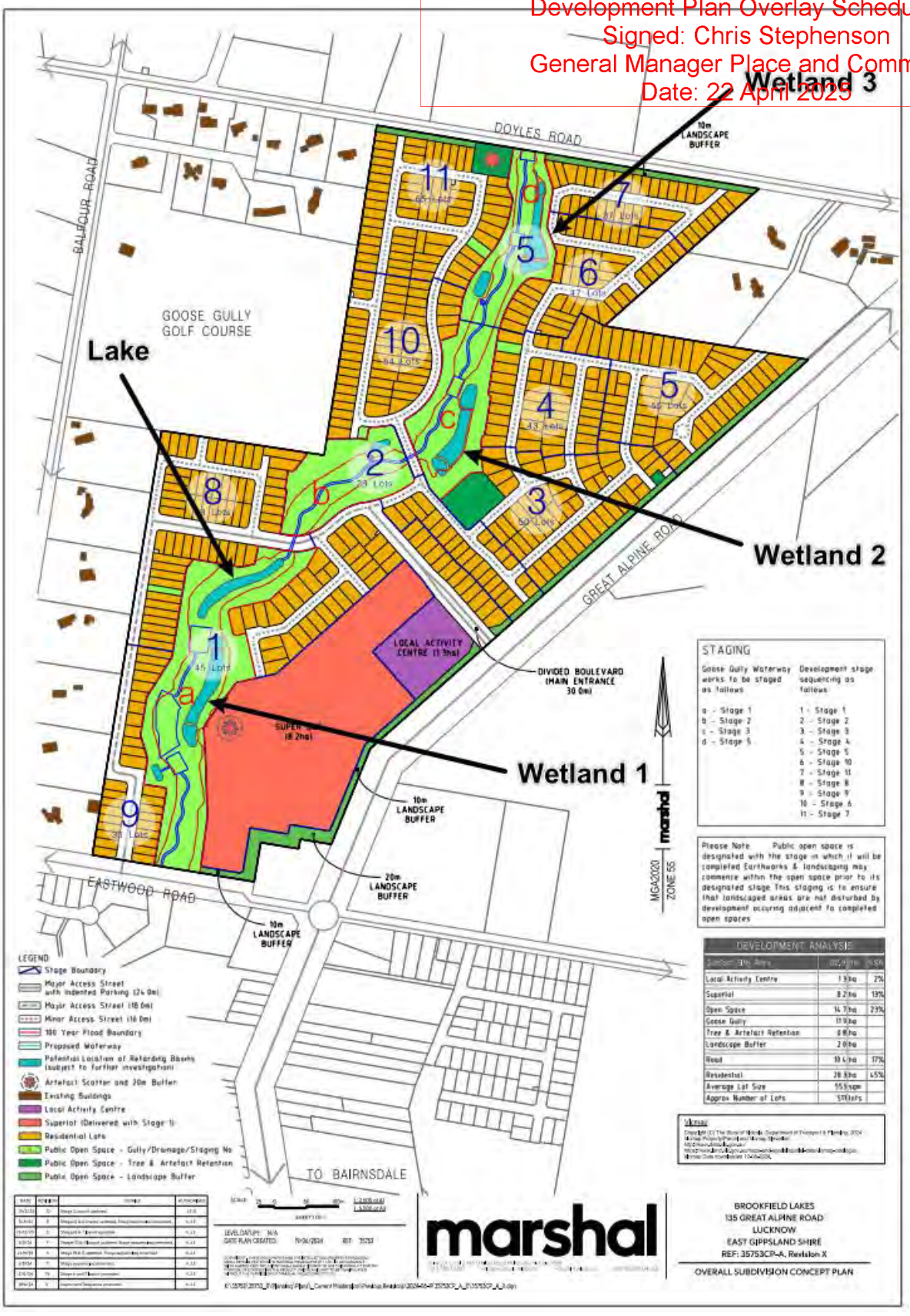

Figure 2 **Proposed development**

2. WATERWAY IMPROVEMENT

The major waterway for the study area is Goose Gully Creek, intersecting the site in a north-south direction with very steep side slopes. Goose Gully Creek is an ephemeral stream and at the time of site inspections (April 2008 and April 2011) it contained no permanent water other than small farm dams (Figure 3**Error! Reference source not found.**). A number of minor gully lines traverse the site, the most major of which provides a flow path from Goose Gully Golf Course into Goose Gully Creek. Goose Gully Creek receives flow from a predominantly agricultural upstream catchment, with an approximate catchment area of 700 hectares at the end of the proposed site. An accompanying Waterway Management Plan accompanies this stormwater management technical report, detailing the short, medium and long term management goals and improvement of the waterway.

Figure 3 Goose Gully Creek

The waterway improvement works include:

- Reduce the waterway's vulnerability to erosion by easing the channel slope and increasing flow area; five rock chutes are proposed;
- A low flow channel with three month flow capacity following the existing channel alignment is proposed;
- One online lake is to be constructed upstream of the second rock chute; it will not only provide the benefits of flood storage, flow velocity reduction and stormwater quality improvement but also act as mosquito control;
- A exclusion zone next to the first wetland will be planned to prevent any work within the area so that a number of trees with high ecological values will be protected;

- Dense vegetation will be planted within the floodplain.

Refer the Waterway Management Plan for more details.

2.1 Rock chutes

Rock chutes were employed to prevent erosion from in the design. The rock chutes were sized using the industry standard modelling program, CHUTE, with design flow 15m^3/s (5yr ARI). The specifications of the rock chutes can be seen in DWG NO. 20.

2.2 Lake

An online lake is designed to provide flood storage, reduce erosion potential, mosquito control and water quality benefit. The total storage volume is approximately 4500 m^3. A cumulative frequency analysis for the lake storage has been undertaken with 50-years of rainfall and evapo-transpiration data. There are a number of farm that dams have been built upstream of the site. Therefore the analysis looked at the scenario if no upstream runoff enters in to lake. **Figure 4Error! Reference source not found.** shows that the size of the lake is considered to be sustainable as the worst case scenario shows no dry outs. The basin is at least half full 30% of the time with no upstream flow inputs, and 80% of the time with the upstream inputs.

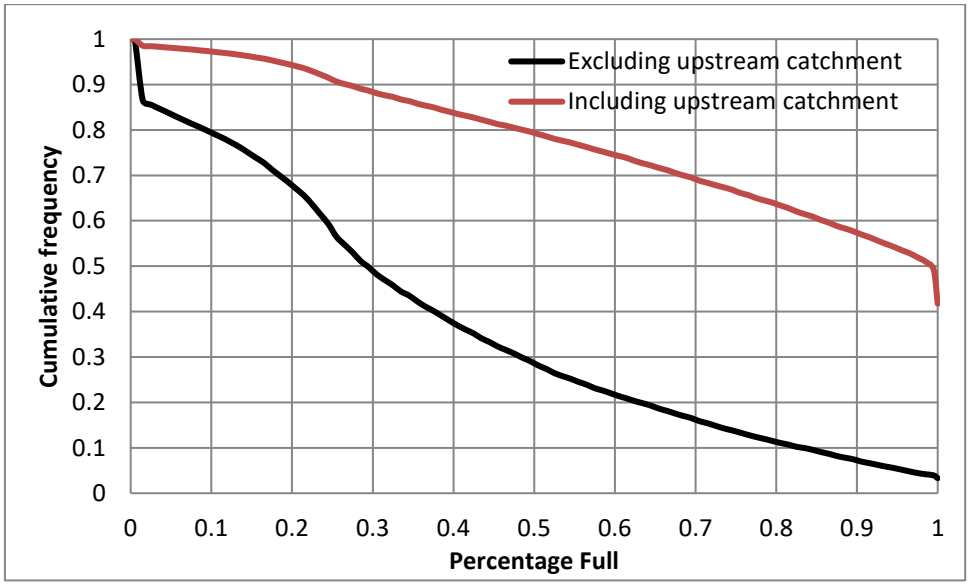

Figure 4 Cumulative frequency analysis for lake storage

3. SEDIMENTATION POND

3.1 Inlet

The sedimentation pond is used as a pre-treatment to settle out coarse sediments and dissipate energy. Appropriate rock protection and energy dissipation structures will need to be applied at the entry to the sediment pond. The structures will need to be designed to dissipate the velocity below 1m/s (0.5m/s preferred) within the sediment pond.

3.2 Outlet

Based on the topographic constraints of the site, only the 5 year flow through drainage pipe network and a very small portion of the overland flow will be captured by the sedimentation ponds. Therefore, no spillway was included in the design. The outlet structure has been design to cater for the 10 year

ARI flow. The outlet structure will use standard precast square grated drainage pit which have been checked against both weir flow and orifice flow conditions. A flat outlet pipe is connected to the outlet pit discharging in to a rock chute. The rock chute was sized using CHUTE. All the calculations have been attached in Appendix A.

3.3 Maintenance

The maintenance of the sediment pond is achieved by allowing the pond to free drain down to 0.5m below the NWL which is the expected sediment level for clean out. A maintenance pipe is connected to the outlet pit which is sized to drain water to the required level within 12 hours. The clean out frequency for the sediment pond is based on the area of the pond and the loads generated off the catchment. Using a sediment load of 1.6m^3/ha/yr (Willing and Partners, 1992) and a Gross Pollutant Load of 0.4m^3/ha/yr (Alison et al, 1998) the clean out frequency is shown in **Table 3-1**. An area will be constructed for dewatering of removed sediments from the sedimentation ponds.

Table 3-1 Clean out frequency for the sedimentation ponds

	Sedimentation Pond		
Clean out frequency (year)	SP1	SP2	SP3
	16	11	7

As can be seen the average clean out frequency of the sedimentation ponds is approximately 10 years, minimising maintenance costs to council.

4. WETLAND

Each of the three wetlands consists of two inlet zones and one macrophyte zone. The inlet zones act as basically sedimentation ponds which remove the coarse sediments. The macrophyte zone is a heavily vegetated water body which can effectively remove pollutants including dissolved contaminants from stormwater.

4.1 Inlet zone

The inlet zone will act as a sedimentation pond. The design procedure is same as outlined in section 3. However, the entry pit and pipe was designed with the 1 year ARI flow. The normal water level (NWL) of the inlet zone is 200mm above that of the macrophyte zone that will ensure a sufficient head difference. An overflow weir of 5 year ARI capacity is to be placed between the inlet and macrophyte zones. The scheduled clean out frequency for the inlet zones is shown in **Table 4-1**.

Table 4-1 Clean out frequency for the sedimentation ponds at wetland inlet zone

	Sedimentation Pond		
Clean out frequency (year)	W1A/ W1B	W2A/ W2B	W3A/ W3B
	10	16	10

Again, the sedimentation ponds on the wetland have been designed to minimise council maintenance requirements.

4.2 Macrophyte zone

The macrophyte zone consists of a series of shallow and deep ponds which promote vertical and horizontal diversities for various vegetation species. A bund with top at the total extended detention (TED) level is to be constructed in the centre of the macrophyte zone so that the hydraulic efficiency could be increased, similar to a baffle structure.

4.3 Outlet structure

Outlet pipes for the wetlands are connected to an outlet pit containing a PVC pipe with small orifice holes which determine the detention time within the wetland that is critical to strip nutrients from the inflow. The Riser outlet structure is designed for each wetland to provide for a 72 hour notional detention time for effective treatment. The calculations are attached in Appendix B.

It is noted that some outlet pipes from some sedimentation basins prior to wetlands have changed with the recent layout change as per table below. The same results are reported in Appendix B with more details.

Name	Previous pipe size (mm)	Updated pipe size (mm)
SP W1A	0.6	0.75
SP W2A	0.45	0.525
SP W2B	0.45	0.525
SP W3A	0.525	0.6
SP W3B	0.525	0.6

4.4 Maintenance

Maintenance drain with a manual valve is included in the design. It will allow free draining of the inlet zone and macrophyte zone down to 500mm below NWL within 12 hours.

4.5 Additional storage

In addition to a 300mm freeboard, another 300mm above the TED level for each wetland has been included in the design. The storage is considered to provide flood attenuation storage and reduce the runoff frequency (**Table 4-2**). The spillways for the wetlands were designed in CHUTE and specifications can be viewed in the design drawings.

Table 4-2 Additional storage in the wetland systems

	Wetland 1	Wetland 2	Wetland 3
Storage above NWL	2474	2120	2124
Storage above TED	1040	851	891

5. UPDATED HYDROLOGY

A RORB model has been implemented to study the hydrology of the site and external catchments. The model schematic that reflects the RORB model is shown in Figure 5 that was updated to reflect the increased density of the proposed development.

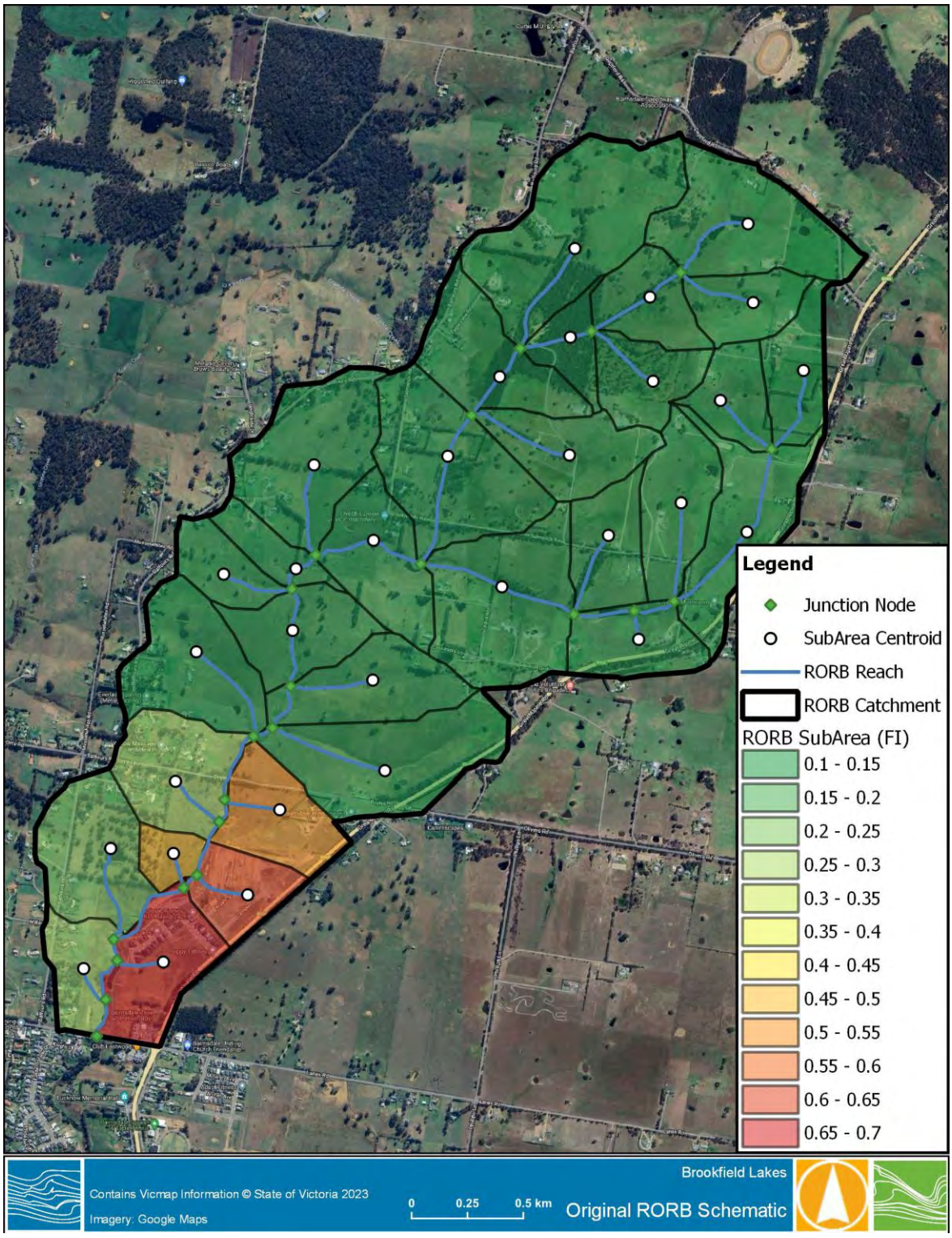

Figure 5 Model schematic

5.1 ARR19 – Sensitivity

Previous modelling of the hydrology for the site was completed with ARR87 methodologies, however the new ARR19 methodologies and data was adopted to ensure that the latest guidelines have not significantly changed the expected outcomes. This analysis included the following:

- Extraction of latest IFD rainfall data.
- Adoption of multiple temporal patterns.
- Replacement of run-off coefficient with continuing loss. A continuing loss of 3 mm/hr was adopted, slightly reduced from the value provided by ARR datahub (3.4 mm/hr).

The updated model was run using a Monte Carlo analysis and flows rates shown in Table 5-1. The results demonstrate the difference in the methodologies applied, with the ARR87 results validated to the rational method. This was common practice and involved adjusting the kc parameter so results aligned with the rational method. This method is no longer best practice and if further analysis was to be undertaken the kc parameter would be re-calculated.

Importantly the ARR19 results shown adopting the previous kc value are an overestimate, and directly the result of the kc value applied. For example, adjusting the kc value to a more commonly used value Pearse et al, resulted in the design flows aligning. Design flows also aligned well with flow estimates from RFFE 2021. Therefore, it is considered that it is appropriate to adopt the original model parameters and methodology with the ARR87 data.

The updated model was simulated adopting rainfall data from ARR87 and the model parameters as provided in Table 5-2.

Table 5-1 Existing Conditions – Peak Flow Comparison for Differing Methodologies

AEP / ARI	ARR87 (m3/s)	ARR19 (m3/s)	Adjusted Kc (m3/s)	RFFE 2021 (m3/s)
Kc Value	2.86	2.86	4.29	N/A
1 in 5	15.2	13.1	10.4	6.2
1 in 10	17.8	18.1	14.7	10.3
1 in 20	21.1	23.4	18.3	15.2
1 in 50	24.8	30.8	24.7	22.5
1 in 100	28.3	37.3	29.1	28.6

Table 5-2 Model Parameters

ARR87 Parameters	Value	Model Parameters	Value
2y 1hr	19.78	Kc	2.86
2y 12hr	4.79	m	0.8
2y 72hr	1.47	Initial Loss	15.0 mm
50y 1hr	38.39	RoC	0.6
50y 12hr	8.83		
50y 72hr	3.06		
Skew	0.3		
F2 Value	4.22		

ARR87 Parameters	Value	Model Parameters	Value
F50 Value	15.35		

5.2 Results

The model results demonstrated the following:

- The increased density of development has little influence on the peak flow along Goose Gully for all design events and irrespective of on-site mitigation. This is because the development is located at the downstream end of the catchment and the increased efficiency of runoff reduces the overlapping of flows, reducing the peak flow experienced when unmitigated. However, flow mitigation is still required at the lot scale as the development will have increased the run-off entering Goose Gully. The results are provided in **Table 5-3**.

The increased density of development has increased the peak discharge and top water level experienced within the proposed storages. The changes to discharge and peak storage are provided in

- **Table 5-4**, note peak storage was used as the levels in RORB do not relate to on-ground levels. The location of the storages is provided in Figure 1.

Importantly the wetland outlet arrangements were designed to different catchment areas and sized using the rational method. The purpose of

  - **Table 5-4** is to provide a comparison of the RORB modelling, further discussion around the outlet arrangement is provided in Section 4.3.

Table 5-3 Peak Flow Comparison – Goose Gully

AEP / ARI	Existing Conditions (m3/s)	Developed (Unmitigated) (m3/s)	Developed (Mitigated) (m3/s)
1 in 5	15.2	14.8	15.1
1 in 10	17.8	17.2	17.6
1 in 20	21.1	20.4	20.9
1 in 50	24.8	24.1	24.6
1 in 100	28.3	27.4	28.0

Table 5-4 Storage Performance – 1 in 100 YR Results

Basin	Design Event	Design Flow Rate (m3/s)	Previous Performance			Updated Performance		
			Peak Discharge (m3/s)	Peak Storage Volume (m3)	Spillway Engaged	Peak Discharge (m3/s)	Peak Storage Volume (m3)	Spillway Engaged
Wetland 3	1 in 100	2.28	2.24	2,970	Y	2.42	3,040	Y
Wetland 2	1 in 100	1.85	2.06	1,940	Y	2.37	2,030	Y
Wetland 1	1 in 100	2.63	3.53	3,140	Y	4.29	3,370	Y
Lake 1	1 in 5	15.0	24.43	3,760	Y	24.53	3,760	Y

5.3 Outcomes

Based upon the hydrology review the following outcomes have been identified:

- The increased density of development has not influenced the peak flow along Goose Gully, due to the upstream catchment been significantly larger.
- The increased density of development has altered the performance of the various basins within the development. These changes in relation to the previous methodology is provided in Section 0. The changed performance in the RORB modelling has not altered peak flows along Goose Gully.
- As peak flows have not been affected by the changes, there is no change to the previous assumptions and methodology applied to the Rock Chute designs within Goose Gully. Assuming the design criteria and methodology was previously accepted there is not requirement to undertake further assessment.
- Peak flows and discharge from the development through proposed storages has changed as a result of the development, the implications of this is discussed in Section 5.2.

6. FLOOD MODELLING

The previous hydraulic modelling undertaken adopted a single inflow at the upstream end of the development along Goose Gully. As the hydrology has not changed there was no need to re-run the hydraulic modelling. The previous result of the hydraulic modelling is provided in Figure 6 to Figure 7.

The waterway was modelled in Mike21 for both existing and developed conditions. The modelling was completed to ensure that velocities within the channel and through the wetland did not exceed scour values. This preliminary modelling confirmed that the proposed design will work efficiently without threatening the both the remaining habitat, and the proposed design features. The modelling also confirmed the size of all proposed culverts within the development.

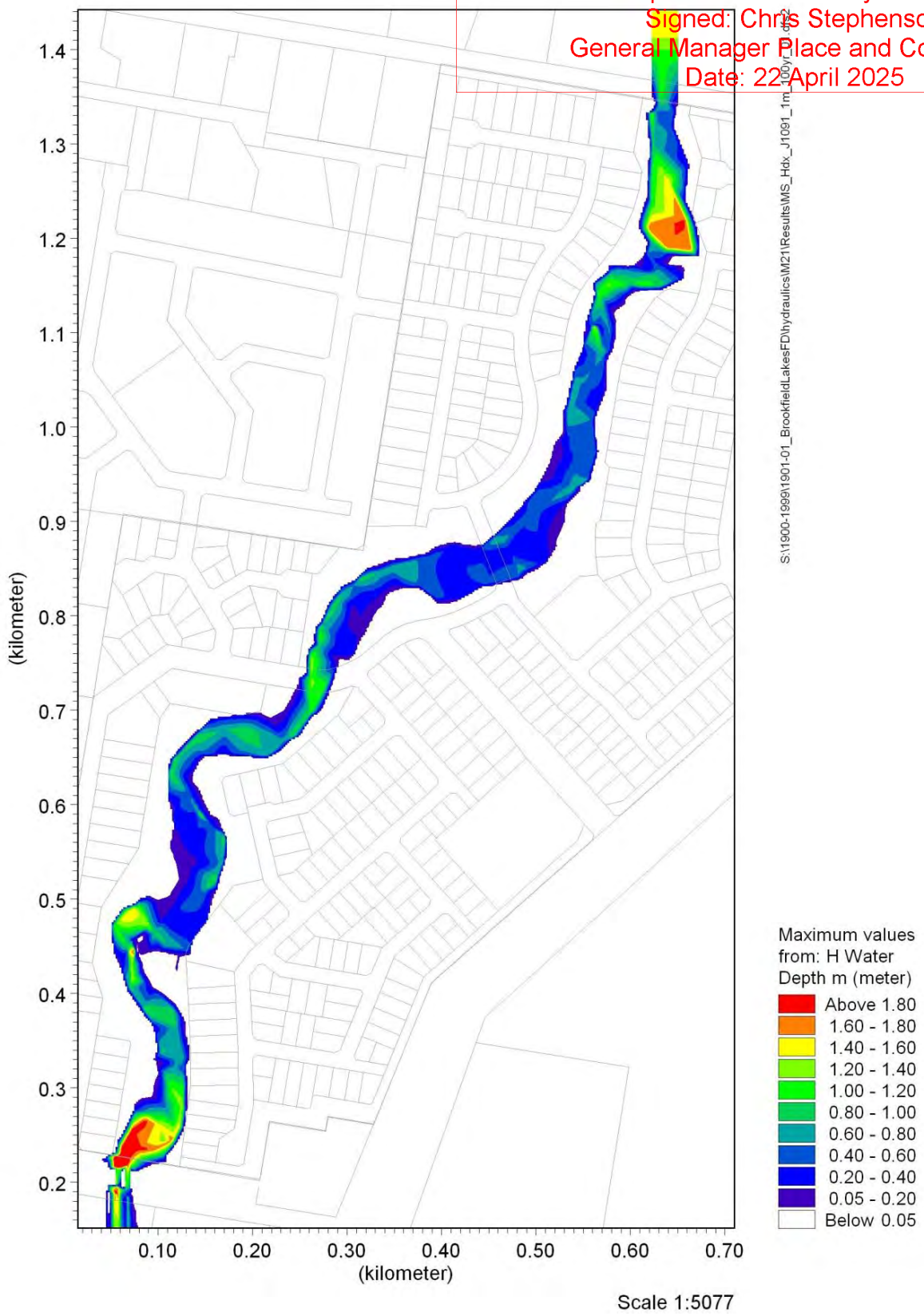

Figure 6 **Brookfield Lakes 100y ARI existing conditions**

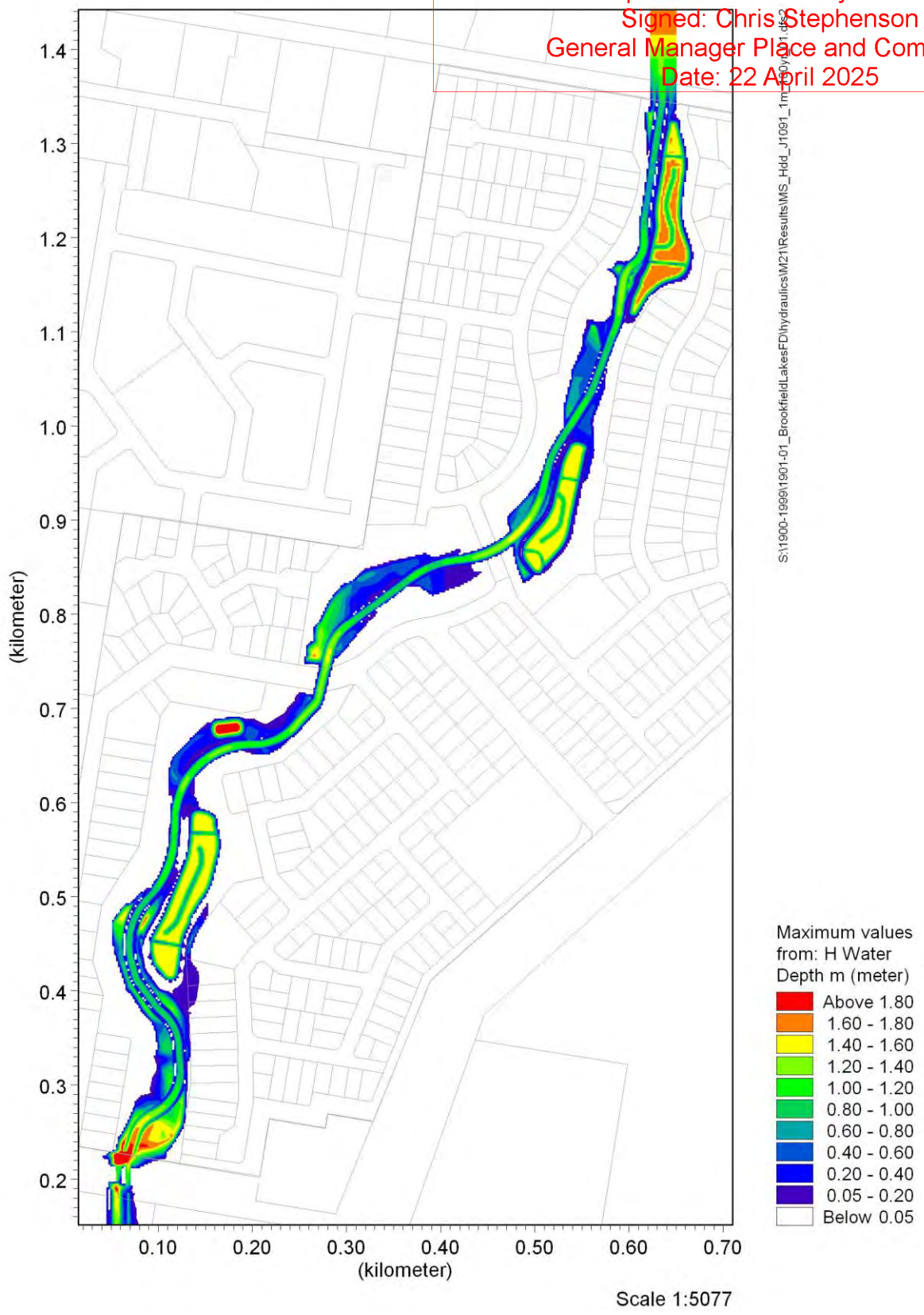

Figure 7 **Brookfield Lakes 100y ARI preliminary developed conditions**

7. RUNOFF FREQUENCY

As a result of waterway improvement, wetland and sedimentation ponds construction, it is expected that more than 18,000m^3 effective flood plain storage will be provided. In the Water Technology (2008) report, it states 7,300m^3 storage will be required to retard the developed flow back to existing level. Runoff frequency analysis (**Figure 8**) conducted in MUSIC shows that only negligible increase of runoff frequency is likely occur due to the development, and mostly the increases are for the flows less than 0.2 m^3/s.

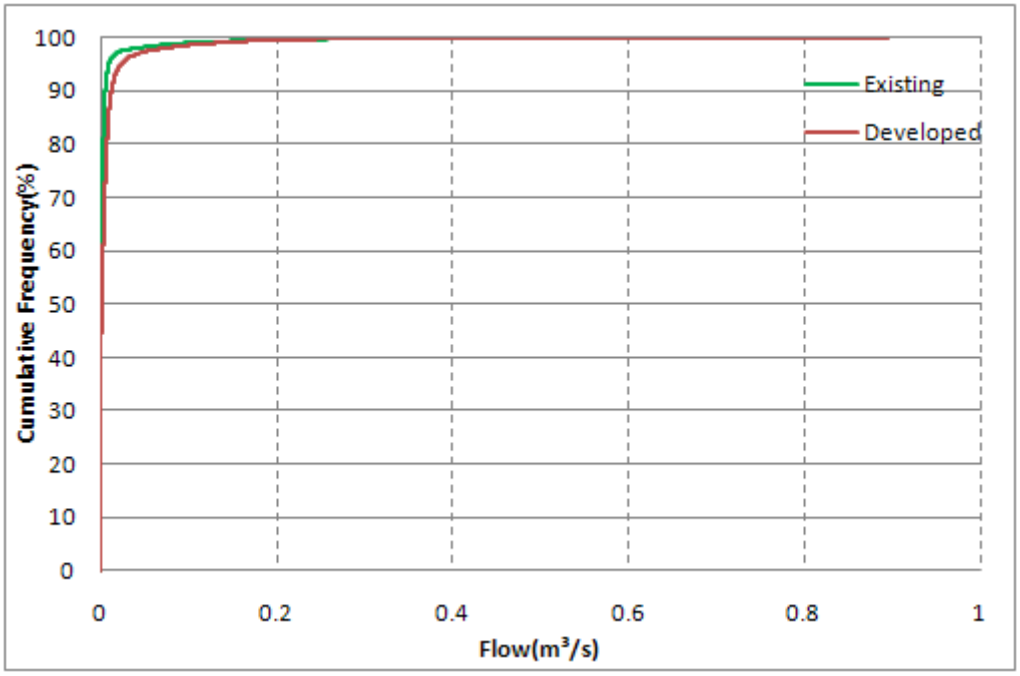

Figure 8 **Cumulative runoff frequency**

8. WATER QUALITY

The treatment performance of the WSUD features for the proposed residential development has been evaluated using CRC for Catchment Hydrology MUSIC model (**Figure 9** & **Table 8-1**). Specific catchments draining to each of the WSUD features has been schematised. It is apparent from **Table 8-2** that the water quality objectives are exceeded by the proposed stormwater treatment train and hence the design meets Best Practice.

Table 8-1 MUSIC modelling inputs

Catchment	Area (ha)	Ex FI (%)	Dev FI (%)
CAT1	2.297	10	75
CAT2	2.571	10	10
CAT3	5.6	10	75
CAT4	4.817	10	75
CAT5	14.147	10	40
CAT6	4.44	10	75
CAT7	3.075	10	75
CAT8	6.51	10	75
CAT9	4.215	10	80
CAT10	2.352	10	10
CAT11	7.785	10	75
CAT12	1.965	10	80
CAT13	1.34	10	75
CAT14	2.089	10	10
CAT15	5.146	10	10
CAT16	0.291	10	75

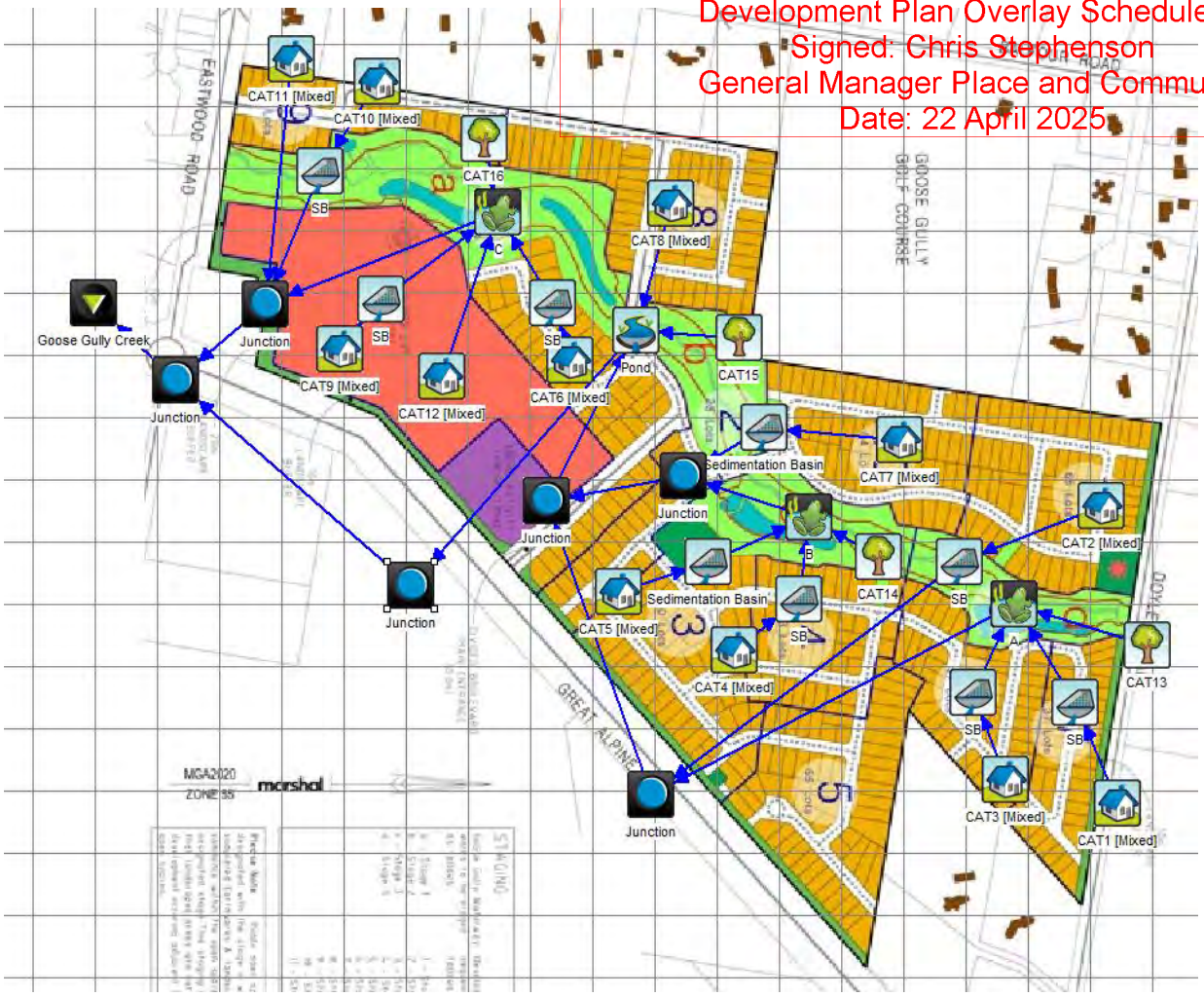

Figure 9 MUSIC model layout

Table 8-2 MUSIC modelling results

		No WSUD treatment features	With WSUD treatment features	
Parameter	**Target Reduction**	**Annual pollutant load**	**Annual pollutant load**	**Reduction**
Suspended Solids (TSS)	80%	61100 kg/yr	8040 kg/yr	86.8 %
Total Phosphorus (TP)	45%	127 kg/yr	33.7 kg/yr	73.4%
Total Nitrogen (TN)	45%	881 kg/yr	431 kg/yr	51.1%
Gross Pollutants	70%	10100 kg/yr	230 kg/yr	97.7%

APPENDIX A SEDIMENTATION POND CALCULATIONS

Note: As the design has been raised vertically post original calculation, differences may be found in the elevations between design drawings and the calculations. However, the relative levels in elevations have not changed and thus the sizing does not change.

SP1

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	220	m^3
Permanent pool Depth	1	m

Hydrology

A(ha)	5.6
$t_c = 7.6A^{0.38}$	14.63
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	22

Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.14**
1	1	0.8	31.474	**0.28**
2	2	0.85	41.524	**0.39**
3	5	0.95	55.748	**0.59**
4	10	1	65.748	**0.73**
5	20	1.05	78.122	**0.91**
6	50	1.15	96.122	**1.23**
7	100	1.2	110.496	**1.47**

	outlet pit	*P35*	
Weir Flow condition	**$p=Q_{des}/(B\times C_w\times H^{1.5})$**		
P(perimeter of the outlet pit,m)	3.39	?	equivalent size
B(Blockage factor)	0.5		**900x900**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.73**	
Orifice Flow condition	**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**		
A_o(Orifice Area(m2)	0.87	?	equivalent size
C_d(Orifice Discharge Coeffieicnet)	0.6		**1300x900**
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.73**	
OverFlow Weir			
Weir Flow condition	**$p=Q_{des}/(C_w\times H^{1.5})$**		
P(perimeter of the outlet pit,m)	5.26	?	

C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	1.41	
Maintenance drain			***p163***
$A_o=Q/(C_d x(2gh)^{0.5})$			
Ao(Orifice Area m2)	0.003		
Diameter(mm)	33	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.33		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.005		
Outlet Pipe			***p164***
Outlet Control	head loss = $(Ke+Kex)\times V^2/2g+ S_f\times L$		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.6	m
RCP pipe radius =		0.3	m
Design flow (ARI) =	10	0.73	m^3/s
Wetted perimeter =		1.89	m
Area =		0.28	m^2
Hyd radius =		0.15	m
V =		2.58	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		8.3	
$S_{f(\text{ Friction slope})}$ =		0.0141	
Head loss =		0.625	m
Total Head at TED	OK	1.400	m
NWL		25.500	m
Outlet Pipe upstream Invert		24.5	m
Outlet Pipe Downstream Invert/WL		24.2	m
Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$		OK
pipe dia =	0.6	m	
Number of pipes	1		
Area =	0.28278	m^2	

	Elevation	H (m)	Q(m^3/s)
Invert	24.500	0.000	0.000
NWL	25.500	0.700	0.629
TED	25.900	1.100	0.788
Spillway Invert	26	1.200	0.823

Sp2

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	186	m^3
Permanent pool Depth	1	m

Hydrology

A(ha)	3.075
$t_c = 7.6A^{0.38}$	11.65
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	19

Index	ARI	Fy	I(mm/hr)	Q(m3/s)
0	0.5			**0.08**
1	1	0.8	34.860	**0.17**
2	2	0.85	46.136	**0.24**
3	5	0.95	62.707	**0.36**
4	10	1	73.061	**0.44**
5	20	1.05	87.414	**0.56**
6	50	1.15	107.768	**0.75**
7	100	1.2	124.768	**0.91**

	outlet pit	*P35*	
Weir Flow condition	**p=Q_{des}/(B×C_w×H^1.5)**		
P(perimeter of the outlet pit,m)	2.07	?	equivalent size
B(Blockage factor)	0.5		**600x600**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.44**	
Orifice Flow condition	**A_o=Q_{des}/(B×C_d×(2gH)^0.5)**		
A_o(Orifice Area(m2)	0.53	?	equivalent size
C_d(Orifice Discharge Coeffieicnet)	0.6		**900x600**
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.44**	
	OverFlow Weir		
Weir Flow condition	**p=Q_{des}/(C_w×H^1.5)**		
P(perimeter of the outlet pit,m)	3.26	?	
C_w(weir coefficient)	1.7		

H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	0.91	
Maintenance drain			***p163***
$A_o=Q/(C_d x(2gh)^{0.5})$			
Ao(Orifice Area m2)	0.003		
Diameter(mm)	30	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.33		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.004		
Outlet Pipe			***p164***
Outlet Control	head loss = $(Ke+Kex)\times V^2/2g+ S_f\times L$		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.45	m
RCP pipe radius =		0.225	m
Design flow (ARI) =	10	0.44	m^3/s
Wetted perimeter =		1.41	m
Area =		0.16	m^2
Hyd radius =		0.1125	m
V =		2.80	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		20	
$S_{f(\text{ Friction slope})}$ =		0.0244	
Head loss =		1.085	m
Total Head at TED	OK	1.275	m
NWL		23.300	m
Outlet Pipe upstream Invert		22.3	m
Outlet Pipe Downstream Invert/WL		22.2	m
Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$		OK
pipe dia =	0.45	m	
Number of pipes	1		
Area =	0.159064	m^2	

	Elevation	H (m)	Q(m^3/s)
Invert	22.300	0.000	0.000
NWL	23.300	0.775	0.372
TED	23.700	1.175	0.458
Spillway Invert	24	1.475	0.513

SP3

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	166	m^3
Permanent pool Depth	1	m

Hydrology

A(ha)	1.965
$t_c = 7.6A^{0.38}$	9.82
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	17

Index	ARI	Fy	I(mm/hr)	Q(m3/s)
0	0.5			**0.06**
1	1	0.8	37.481	**0.12**
2	2	0.85	49.546	**0.16**
3	5	0.95	67.704	**0.25**
4	10	1	79.704	**0.31**
5	20	1.05	95.056	**0.39**
6	50	1.15	117.704	**0.53**
7	100	1.2	136.056	**0.64**

	outlet pit	*P35*	
Weir Flow condition	**p=Q_{des}/(B×C_w×H^1.5)**		
P(perimeter of the outlet pit,m)	1.44	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.31**	
Orifice Flow condition	**A_o=Q_{des}/(B×C_d×(2gH)^0.5)**		
A_o(Orifice Area(m2)	0.37	?	equivalent size
C_d(Orifice Discharge Coeffieicnet)	0.6		**900x600**
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	10	**0.31**	
OverFlow Weir			
Weir Flow condition	**p=Q_{des}/(C_w×H^1.5)**		
P(perimeter of the outlet pit,m)	2.27	?	
C_w(weir coefficient)	1.7		

H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	0.64	
Maintenance drain			***p163***
$Ao=Q/(C_d x(2gh)^{0.5})$			
Ao(Orifice Area m2)	0.003		
Diameter(mm)	28	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.33		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.004		
Outlet Pipe			***p164***
Outlet Control	head loss = $(Ke+Kex)\times V^2/2g+ S_f\times L$		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.375	m
RCP pipe radius =		0.1875	m
Design flow (ARI) =	10	0.31	m^3/s
Wetted perimeter =		1.18	m
Area =		0.11	m^2
Hyd radius =		0.09375	m
V =		2.81	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		20	
$S_{f(\text{ Friction slope})}$ =		0.0313	
Head loss =		1.229	m
Total Head at TED	OK	1.513	m
NWL		17.300	m
Outlet Pipe upstream Invert		16.3	m
Outlet Pipe Downstream Invert/WL		16	m
Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$		OK
pipe dia =	0.375	m	
Number of pipes	1		
Area =	0.110461	m^2	

	Elevation	H (m)	Q(m^3/s)
Invert	16.300	0.000	0.000
NWL	17.300	0.813	0.265
TED	17.700	1.213	0.323
Spillway Invert	17.7	1.213	0.323

APPENDIX B WETLAND CALCULATIONS

Wetland 1

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	809.482	m^3
Permanent pool Depth	0.5	m

Wetland Outlet structure

A(ha)	16.444
$t_c = 7.6A^{0.38}$	22.02
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	30

Index	ARI	Fy	I(mm/hr)	Q(m3/s)
0	0.5			**0.34**
1	1	0.8	25.786	**0.67**
2	2	0.85	33.881	**0.94**
3	5	0.95	45.174	**1.40**
4	10	1	52.952	**1.72**
5	20	1.05	62.952	**2.15**
6	50	1.15	75.976	**2.85**
7	100	1.2	87.952	**3.44**

	OverFlow Weir		
Weir Flow condition	**$p=Q_{des}/(C_w \times H^{1.5})$**		
P(perimeter of the outlet pit,m)	12.31	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	3.44	
	Maintenance drain		***p163***
	$Ao=Q/(C_d x(2gh)^{0.5})$		
Ao(Orifice Area m2)	0.030		
Diameter(mm)	99	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.17		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	33.090	L/s	
Total Permanent Volume of connected SediPond	620		
Wetland	**Outlet**	***Pipe***	***p164***
Outlet Control	head loss = $(Ke+Kex) \times V^2/2g + S_f \times L$		

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$S_f = Q^2n^2/A^2R^{4/3}$

Number of pipes		1	
pipe dia =		0.225	m
RCP pipe radius =		0.1125	m
Design flow (max from the riser or maintenance discharge) =		0.033	m3/s
Wetted perimeter =		0.71	m
Area =		0.04	m^2
Hyd radius =		0.05625	m
V =		0.83	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		20	
$S_{f(\text{ Friction slope})}$ =		0.0054	
Head loss =		0.162	m
Total Head at TED	OK	1.288	m
NWL		25.000	m
Outlet Pipe upstream Invert		24.5	m
Outlet Pipe Downstream Invert/WL		24	m

Check for Inlet Control

$Q = 0.6 \times A \times (2gH)^{0.5}$

pipe dia =	0.225	m
Number of pipes	1	
Area =	0.039766	m^2

	Elevation	H (m)	Q(m^3/s)
Invert	24.500	0.000	0.000
NWL	25.000	0.388	0.066
TED	25.400	0.787	0.094
Spillway Invert	25.7	1.088	0.110

Riser sizing

				Q1	Q2	Q3	Q4	total	Notional
Orifice Positions (invert level)				0	0.1	0.2	0.3	Flow	Detention Time
Orifice Diameter (mm)				20	20	20	20	(L/s)	(hrs)
Number				2	3	1	1		
Elevation	Volume	Relative Depth	Volume						
25	809.482	0	0	0				0.00	
25.1	985.638	0.1	176	0.501				0.50	97.64
25.2	1182.853	0.2	373	0.728	0.752			1.48	70.08
25.3	1399.684	0.3	590	0.900	1.092	0.251		2.24	73.11
25.4	1633.274	0.4	824	1.043	1.349	0.364	0.251	3.01	76.09

Sedimentation pond W1A

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	456	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 1

A(ha)	14.147			
$t_c = 7.6A^{0.38}$	20.80			
$^{10}I_{1(mm/hr)}$	28.90			
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152			
FI	0.75			
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713			
ColNum	28			
Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.30**
1	1	0.8	26.520	**0.59**
2	2	0.85	34.880	**0.83**
3	5	0.95	46.640	**1.24**
4	10	1	54.200	**1.52**
5	20	1.05	64.400	**1.89**
6	50	1.15	79.400	**2.56**
7	100	1.2	90.600	**3.05**

	Entry pit *P35*		
Weir Flow condition	**$p=Q_{des}/(B\times C_w\times H^{1.5})$**		
P(perimeter of the outlet pit,m)	2.76	?	equivalent size
B(Blockage factor)	0.5		**900x600**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.59**	
Orifice Flow condition	**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**		
A_o(Orifice Area(m2)	0.71	?	equivalent size
C_d(Orifice Discharge Coeffieicnet)	0.6		**900x900**
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.59**	

	OverFlow Weir	
Weir Flow condition	**$p=Q_{des}/(C_w\times H^{1.5})$**	
P(perimeter of the outlet pit,m)	8.16	?
C_w(weir coefficient)	1.7	

H(depth of water above the Crest of outlet pit, m)	0.2		
Q_{des}(Design flow m3/s)----ARI	5	1.24	
	Maintenance drain		***p163***
	$A_o=Q/(C_d x(2gh)^{0.5})$		
Ao(Orifice Area m2)	0.006		
Diameter(mm)	45	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.40		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.011		
	Inlet pipe		***p164***
Outlet Control	head loss = $(Ke+Kex)\times V^2/2g+ S_f\times L$		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.75	m
RCP pipe radius =		0.375	m
Design flow (usually 1 yr ARI) =	1	0.59	m^3/s
Wetted perimeter =		2.36	m
Area =		0.44	m^2
Hyd radius =		0.1875	m
V (less than 0.05 ideally)=		1.35	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		8	
$S_{f(\text{ Friction slope})}$ =		0.0029	
Head loss =		0.161	m
NWL	OK	25.2	m
Outlet Pipe Invert		24.289	m
Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$		OK
pipe dia =	0.75	m	
Number of pipes	1		
Area =	0.44184375	m^2	

	Elevation	H (m)	Q(m^3/s)
Pipe Invert	24.289	0.000	0.000
NWL	25.2	0.536	0.860
FULL	25.6	0.936	1.136
Other Levels	25.8	1.136	1.252
Other Levels	26	1.336	1.357

Sedimentation pond W1B

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	164	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 2

A(ha)	2.297
$t_c = 7.6A^{0.38}$	10.42
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	18

Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.07**
1	1	0.8	36.563	**0.13**
2	2	0.85	48.451	**0.19**
3	5	0.95	65.727	**0.28**
4	10	1	77.302	**0.35**
5	20	1.05	92.302	**0.44**
6	50	1.15	114.453	**0.60**
7	100	1.2	132.029	**0.72**

	Entry pit	*P35*	
Weir Flow condition		**$p=Q_{des}/(B\times C_w\times H^{1.5})$**	
P(perimeter of the outlet pit,m)	0.62	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.13**	
Orifice Flow condition		**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**	
A_o(Orifice Area(m2)	0.16	?	equivalent size
C_d(Orifice Discharge Coeffieicnet)	0.6		**450x450**
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.13**	

	OverFlow Weir	$p=Q_{des}/(C_w \times H^{1.5})$	
Weir Flow condition			
P(perimeter of the outlet pit,m)	1.87	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.2		
Q_{des}(Design flow m3/s)----ARI	5	0.28	
	Maintenance drain		***p163***
		Ao=Q/(C_dx(2gh)^0.5)	
Ao(Orifice Area m2)	0.002		
Diameter(mm)	27	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.40		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.004		
	Inlet pipe		***p164***
Outlet Control	head loss = $(Ke+Kex)\times V^2/2g+ S_f \times L$		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.375	m
RCP pipe radius =		0.1875	m
Design flow (usually 1 yr ARI) =	1	0.13	m^3/s
Wetted perimeter =		1.18	m
Area =		0.11	m^2
Hyd radius =		0.09375	m
V (less than 0.05 ideally)=		1.20	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		8	
$S_{f(\text{ Friction slope})}$ =		0.0058	
Head loss =		0.157	m
NWL	OK	25.2	m
Outlet Pipe Invert		24.668	m
Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$		OK
pipe dia =	0.375	m	
Number of pipes	1		
Area =	0.110461	m^2	

	Elevation	H (m)	Q(m^3/s)
Pipe Invert	24.668	0.000	0.000
NWL	25.2	0.345	0.172
FULL	25.6	0.745	0.253
Other Levels	25.8	0.945	0.285
Other Levels	26	1.145	0.314

Wetland 2

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	835.921	m^3
Permanent pool Depth	0.5	m

Wetland Outlet structure

A(ha)	9.257			
$t_c = 7.6A^{0.38}$	17.70			
$^{10}I_{1(mm/hr)}$	28.90			
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152			
FI	0.75			
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713			
ColNum	25			
Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.21**
1	1	0.8	28.737	**0.42**
2	2	0.85	37.826	**0.59**
3	5	0.95	50.592	**0.88**
4	10	1	59.592	**1.09**
5	20	1.05	70.592	**1.36**
6	50	1.15	86.592	**1.83**
7	100	1.2	99.888	**2.20**

OverFlow Weir			
Weir Flow condition	**p=Q_{des}/(C_w×H^1.5)**		
P(perimeter of the outlet pit,m)	7.87	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	2.20	
Maintenance drain			***p163***
	Ao=Q/(C_dx(2gh)^0.5)		
Ao(Orifice Area m2)	0.035		
Diameter(mm)	106	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.17		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	37.961	L/s	
Total Permanent Volume of connected SediPond	804		

Wetland	Outlet	Pipe p164	
Outlet Control	head loss = $(Ke+Kex)xV^2/2g + S_f xL$		
	$S_f = Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.225	m
RCP pipe radius =		0.1125	m
Design flow (max from the riser or maintenance discharge) =		0.038	m3/s
Wetted perimeter =		0.71	m
Area =		0.04	m^2
Hyd radius =		0.05625	m
V =		0.95	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		20	
$S_{f(\text{ Friction slope})}$ =		0.0071	
Head loss =		0.213	m
Total Head at TED	OK	1.288	m
NWL		22.000	m
Outlet Pipe upstream Invert		21.5	m
Outlet Pipe Downstream Invert/WL		21	m

Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$			
pipe dia =		0.225	m	
Number of pipes		1		
Area =		0.039766	m^2	
	Elevation		H (m)	Q(m^3/s)
Invert		21.500	0.000	0.000
NWL		22.000	0.388	0.066
TED		22.400	0.787	0.094
Spillway Invert		22.7	1.088	0.110

				Q1	Q2	Q3	Q4	total	Notional
Orifice Positions (invert level)				0	0.1	0.2	0.3	Flow	Detention Time
Orifice Diameter (mm)				20	20	20	20	(L/s)	(hrs)
Number				2	2	2	1		
Elevation	Volume	Relative Depth	Volume						
22	835.921	0	0	0				0.00	
22.1	1004.928	0.1	169	0.501				0.50	93.68
22.2	1191.487	0.2	356	0.728	0.501			1.23	80.34
22.3	1394.698	0.3	559	0.900	0.728	0.501		2.13	72.91
22.4	1611.999	0.4	776	1.043	0.900	0.728	0.251	2.92	73.79

Sedimentation pond W2A

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	338	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 1

A(ha)	4.44
$t_c = 7.6A^{0.38}$	13.39
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	21

Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.12**
1	1	0.8	32.770	**0.23**
2	2	0.85	43.313	**0.32**
3	5	0.95	58.218	**0.49**
4	10	1	68.218	**0.60**
5	20	1.05	81.826	**0.76**
6	50	1.15	100.435	**1.02**
7	100	1.2	116.044	**1.22**

	Entry pit *P35*		
Weir Flow condition	**$p=Q_{des}/(B\times C_w\times H^{1.5})$**		
P(perimeter of the outlet pit,m)	1.07	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.23**	
Orifice Flow condition	**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**		

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			equivalent size
A_o(Orifice Area(m2)	0.27	?	600x600
C_d(Orifice Discharge Coeffieicnet)	0.6		
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.23**	

OverFlow Weir		
Weir Flow condition	**$p=Q_{des}/(C_w \times H^{1.5})$**	
P(perimeter of the outlet pit,m)	3.20	?
C_w(weir coefficient)	1.7	
H(depth of water above the Crest of outlet pit, m)	0.2	
Q_{des}(Design flow m3/s)----ARI	5	0.49

Maintenance drain		***p163***
	$Ao=Q/(C_d x(2gh)^{0.5})$	
Ao(Orifice Area m2)	0.005	
Diameter(mm)	38	?
C_d(Orifice Discharge Coeffieicnet)	0.6	
h(1 thrid of the permanent pool depth m)	0.40	
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.008	

Inlet pipe				***p164***
Outlet Control	head loss = $(Ke+Kex)\times V^2/2g+ S_f \times L$			
	$S_f = Q^2n^2/A^2R^{4/3}$			
Number of pipes		1		
pipe dia =		0.525	m	
RCP pipe radius =		0.2625	m	
Design flow (usually 1 yr ARI) =	1	0.23	m^3/s	
Wetted perimeter =		1.65	m	
Area =		0.22	m^2	
Hyd radius =		0.13125	m	
V (less than 0.05 ideally)=		1.06	m/s	
Ke(inlet loss coe) =		0.5		
Kex(outlet loss coe) =		1		
n =		0.013		
L(length of culvert)=		10		
$S_{f(\text{ Friction slope})}$ =		0.0029		
Head loss =		0.115	m	
NWL	OK	22.2	m	
Outlet Pipe Invert		21.560	m	
Check for Inlet Control	$Q = 0.6\times A\times(2gH)^{0.5}$			OK
pipe dia =	0.525	m		

Number of pipes			
Area =	0.216503438 m^2		
	Elevation	H (m)	Q(m^3/s)
Pipe Invert	21.560	0.000	0.000
NWL	22.2	0.378	0.354
FULL	22.6	0.778	0.507
Other Levels	22.7	0.878	0.539
Other Levels	22.8	0.978	0.569

Sedimentation pond W2B

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	466	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 2

A(ha)	4.817
$t_c = 7.6A^{0.38}$	13.81
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	21

Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.12**
1	1	0.8	32.306	**0.25**
2	2	0.85	42.681	**0.35**
3	5	0.95	57.375	**0.52**
4	10	1	67.375	**0.64**
5	20	1.05	80.563	**0.81**
6	50	1.15	98.750	**1.08**
7	100	1.2	113.938	**1.30**

	Entry pit	*P35*	
Weir Flow condition		**$p=Q_{des}/(B\times C_w\times H^{1.5})$**	
P(perimeter of the outlet pit,m)	1.15	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.25**	
Orifice Flow condition		**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**	
A_o(Orifice Area(m2)	0.29	?	equivalent size

C_d(Orifice Discharge Coeffieicnet)	0.6	600x600	
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.25**	

OverFlow Weir			
Weir Flow condition		**p=Q_{des}/(C_w×H^1.5)**	
P(perimeter of the outlet pit,m)	3.42	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.2		
Q_{des}(Design flow m3/s)----ARI	5	0.52	

Maintenance drain			***p163***
		Ao=Q/(C_dx(2gh)^0.5)	
Ao(Orifice Area m2)	0.006		
Diameter(mm)	45	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.40		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.011		

Inlet pipe				***p164***
Outlet Control		head loss = (Ke+Kex)×V^2/2g+ S_f×L		
		S_f = $Q^2n^2/A^2R^{4/3}$		
Number of pipes		1		
pipe dia =		0.525	m	
RCP pipe radius =		0.2625	m	
Design flow (usually 1 yr ARI) =	1	0.25	m^3/s	
Wetted perimeter =		1.65	m	
Area =		0.22	m^2	
Hyd radius =		0.13125	m	
V (less than 0.05 ideally)=		1.14	m/s	
Ke(inlet loss coe) =		0.5		
Kex(outlet loss coe) =		1		
n =		0.013		
L(length of culvert)=		10		
$S_{f(\text{ Friction slope})}$ =		0.0033		
Head loss =		0.132	m	
NWL	OK	22.2	m	
Outlet Pipe Invert		21.543	m	

Check for Inlet Control	$Q = 0.6 \times A \times (2gH)^{0.5}$		OK
pipe dia =	0.525	m	
Number of pipes	1		
Area =	0.216503	m^2	

	Elevation	H (m)	Q(m³/s)
Pipe Invert	21.543	0.000	0.000
NWL	22.2	0.395	0.361
FULL	22.6	0.795	0.513
Other Levels	22.7	0.895	0.544
Other Levels	22.8	0.995	0.574

Wetland 3

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	922.799	m^3
Permanent pool Depth	0.5	m

Wetland Outlet structure

A(ha)	14.586			
$t_c = 7.6A^{0.38}$	21.04			
$^{10}I_{1(mm/hr)}$	28.90			
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152			
FI	0.8			
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.7504			
ColNum	29			
Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.32**
1	1	0.8	26.374	**0.64**
2	2	0.85	34.666	**0.90**
3	5	0.95	46.348	**1.34**
4	10	1	53.957	**1.64**
5	20	1.05	63.957	**2.04**
6	50	1.15	78.871	**2.76**
7	100	1.2	89.914	**3.28**

	OverFlow Weir		
Weir Flow condition	**$p=Q_{des}/(C_w \times H^{1.5})$**		
P(perimeter of the outlet pit,m)	11.74	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.3		
Q_{des}(Design flow m3/s)----ARI	100	3.28	
	Maintenance drain		***p163***
	$Ao=Q/(C_d x(2gh)^{0.5})$		
Ao(Orifice Area m2)	0.049		
Diameter(mm)	125	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.17		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	53.329	L/s	
Total Permanent Volume of connected SediPond	1381		
Wetland	**Outlet**	***Pipe***	***p164***
Outlet Control	head loss = $(Ke+Kex) \times V^2/2g + S_f \times L$		

$S_f = Q^2n^2/A^2R^{4/3}$

Number of pipes		1	
pipe dia =		0.225	m
RCP pipe radius =		0.1125	m
Design flow (max from the riser or maintenance discharge) =		0.053	m3/s
Wetted perimeter =		0.71	m
Area =		0.04	m^2
Hyd radius =		0.05625	m
V =		1.34	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		20	
$S_{f(\text{ Friction slope})}$ =		0.0141	
Head loss =		0.420	m
Total Head at TED	OK	0.788	m
NWL		15.000	m
Outlet Pipe upstream Invert		14	m
Outlet Pipe Downstream Invert/WL		14.5	m

Check for Inlet Control

$Q = 0.6 \times A \times (2gH)^{0.5}$

pipe dia =	0.225	m
Number of pipes	1	
Area =	0.039766	m^2

	Elevation	H (m)	$Q(m^3/s)$
Invert	14.000	0.000	0.000
NWL	15.000	0.888	0.100
TED	15.400	1.288	0.120
Spillway Invert	15.7	1.588	0.133

				Q1	Q2	Q3	Q4	total	Notional
Orifice Positions (invert level)				0	0.1	0.2	0.3	Flow	Detention Time
Orifice Diameter (mm)				20	20	20	20	(L/s)	(hrs)
Number				3	2	2	2		
Elevation	Volume	Relative Depth	Volume						
15	922.799	0	0	0				0.00	
15.1	1130.671	0.1	208	0.752				0.75	76.81
15.2	1363.203	0.2	440	1.092	0.501			1.59	76.78
15.3	1618.85	0.3	696	1.349	0.728	0.501		2.58	74.98
15.4	1893.972	0.4	971	1.565	0.900	0.728	0.501	3.69	73.03

Sedimentation pond W3A

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	458	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 1

A(ha)	7.785
$t_c = 7.6A^{0.38}$	16.58
$^{10}I_{1(mm/hr)}$	28.90
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152
FI	0.75
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713
ColNum	24

Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.18**
1	1	0.8	29.639	**0.37**
2	2	0.85	39.108	**0.51**
3	5	0.95	52.424	**0.77**
4	10	1	61.424	**0.95**
5	20	1.05	72.847	**1.18**
6	50	1.15	89.271	**1.58**
7	100	1.2	103.271	**1.91**

	Entry pit *P35*		
Weir Flow condition	**$p=Q_{des}/(B\times C_w\times H^{1.5})$**		
P(perimeter of the outlet pit,m)	1.70	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.37**	
Orifice Flow condition	**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**		
A_o(Orifice Area(m2)	0.44	?	equivalent size

C_d(Orifice Discharge Coeffieicnet)	0.6	900×600	
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.37**	

	OverFlow Weir		
Weir Flow condition		**p=Q_{des}/(C_w×H^1.5)**	
P(perimeter of the outlet pit,m)	5.05	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.2		
Q_{des}(Design flow m3/s)----ARI	5	0.77	

	Maintenance drain		***p163***
		Ao=Q/(C_dx(2gh)^0.5)	
Ao(Orifice Area m2)	0.006		
Diameter(mm)	45	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.40		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.011		

	Inlet pipe			***p164***
Outlet Control	head loss = (Ke+Kex)×V^2/2g+ S_f×L			
	S_f = $Q^2n^2/A^2R^{4/3}$			
Number of pipes		1		
pipe dia =		0.6	m	
RCP pipe radius =		0.3	m	
Design flow (usually 1 yr ARI) =	1	0.37	m^3/s	
Wetted perimeter =		1.89	m	
Area =		0.28	m^2	
Hyd radius =		0.15	m	
V (less than 0.05 ideally)=		1.29	m/s	
Ke(inlet loss coe) =		0.5		
Kex(outlet loss coe) =		1		
n =		0.013		
L(length of culvert)=		8		
$S_{f(\text{ Friction slope})}$ =		0.0035		
Head loss =		0.156	m	
NWL	OK	15.2	m	
Outlet Pipe Invert		14.444	m	

Check for Inlet Control	$Q = 0.6 \times A \times (2gH)^{0.5}$		OK
pipe dia =	0.6	m	
Number of pipes	1		
Area =	0.28278	m^2	

	Elevation	H (m)	Q(m³/s)
Pipe Invert	14.444	0.000	0.000
NWL	15.2	0.456	0.506
FULL	15.6	0.856	0.695
Other Levels	16	1.256	0.842
Other Levels	17	2.256	1.129

Sedimentation pond W3B

Dimensions		
Ext. Det. Depth	0.4	m
Permanent pool Volume	278	m^3
Permanent pool Depth	1.2	m

SediPond outlet Structure 2

A(ha)	6.51			
$t_c = 7.6A^{0.38}$	15.49			
$^{10}I_{1(mm/hr)}$	28.90			
C^1_{10} = 0.1 + (0.7 - 0.1) * ($^{10}I_1$ - 25) / (70 - 25)	0.152			
FI	0.75			
C_{10} = 0.9 * f +C^1_{10} * (1 - f)	0.713			
ColNum	23			
Index	**ARI**	**Fy**	**I(mm/hr)**	**Q(m3/s)**
0	0.5			**0.16**
1	1	0.8	30.613	**0.32**
2	2	0.85	40.415	**0.44**
3	5	0.95	54.025	**0.66**
4	10	1	63.538	**0.82**
5	20	1.05	75.538	**1.02**
6	50	1.15	93.051	**1.38**
7	100	1.2	107.051	**1.66**

	Entry pit *P35*		
Weir Flow condition		**$p=Q_{des}/(B\times C_w\times H^{1.5})$**	
P(perimeter of the outlet pit,m)	1.47	?	equivalent size
B(Blockage factor)	0.5		**450x450**
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.32**	
Orifice Flow condition		**$A_o=Q_{des}/(B\times C_d\times(2gH)^{0.5})$**	
A_o(Orifice Area(m2)	0.38	?	equivalent size

C_d(Orifice Discharge Coeffieicnet)	0.6	900x600	
H(depth of water above Centroid of the orifice, m)	0.4		
Q_{des}(Design flow m3/s)----ARI	1	**0.32**	

OverFlow Weir			
Weir Flow condition		**p=Q_{des}/(C_w×H^1.5)**	
P(perimeter of the outlet pit,m)	4.35	?	
C_w(weir coefficient)	1.7		
H(depth of water above the Crest of outlet pit, m)	0.2		
Q_{des}(Design flow m3/s)----ARI	5	0.66	

Maintenance drain		***p163***	
		Ao=Q/(C_dx(2gh)^0.5)	
Ao(Orifice Area m2)	0.004		
Diameter(mm)	35	?	
C_d(Orifice Discharge Coeffieicnet)	0.6		
h(1 thrid of the permanent pool depth m)	0.40		
Q(required flow rage to drain the volume of the permanet poll in 12 hours m3)	0.006		

Inlet pipe		***p164***	
Outlet Control	head loss = (Ke+Kex)×V^2/2g+ S_f×L		
	S_f = $Q^2n^2/A^2R^{4/3}$		
Number of pipes		1	
pipe dia =		0.6	m
RCP pipe radius =		0.3	m
Design flow (usually 1 yr ARI) =	1	0.32	m^3/s
Wetted perimeter =		1.89	m
Area =		0.28	m^2
Hyd radius =		0.15	m
V (less than 0.05 ideally)=		1.12	m/s
Ke(inlet loss coe) =		0.5	
Kex(outlet loss coe) =		1	
n =		0.013	
L(length of culvert)=		8	
$S_{f(\text{ Friction slope})}$ =		0.0026	
Head loss =		0.116	m
NWL	OK	15.2	m
Outlet Pipe Invert		14.484	m

Check for Inlet Control	$Q = 0.6 \times A \times (2gH)^{0.5}$		OK
pipe dia =	0.6	m	
Number of pipes	1		
Area =	0.28278	m^2	

	Elevation	H (m)	Q(m³/s)
Pipe Invert	14.484	0.000	0.000
NWL	15.2	0.416	0.485
FULL	15.6	0.816	0.679
Other Levels	16	1.216	0.829
Other Levels	17	2.216	1.119

Development Plan,
135 Great Alpine Road, Lucknow
Brookfield Lakes Estate
Planning and Environment Act 1987,
East Gippsland Planning Scheme (Approved March 2025).
Development Plan Overlay Schedule 3.
Signed: Chris Stephenson
General Manager Place and Community
Date: 22 April 2025

DOYLES ROAD

BALFOUR ROAD

GOOSE GULLY
GOLF COURSE

Shared driveways subject to further design considerations

10m Landscape Buffer

Shared driveways subject to further design considerations

Shared driveways subject to further design considerations

10m Landscape Buffer

GREAT ALPINE ROAD

LOCAL ACTIVITY CENTRE (1.3ha) (Subject to future planning permit application)

Divided Boulevard (Main Entrance) Intersection upgraded as part of Stage 1

SUPER LOT (8.2ha) (Land subject to alternative Development Plan)

Upgrade existing Water Main to accommodate increased demand. Upgrade to East Gippsland Water standards.

10m Landscape Buffer

20m Landscape Buffer

EASTWOOD ROAD

Intersection to be upgraded to Aur & Bac Lane Treatments to East Gippsland Shire as per Traffix Report (Oct 2007)

10m Landscape Buffer

Sewer Connection Point. New Sewer to be constructed by East Gippsland Water from the south as a shared main.

LANES ROAD

TO BAIRNSDALE

MGA2020 ZONE 55

marshal

SCALE: 25 0 50 100m
1 : 2,500 at A1
1 : 5,000 at A3

SHEET 1 OF 1

LEVEL DATUM: N/A
DATE PLAN CREATED: 8/04/2025 REF: 35753

Notations

- Refer to Certificate of Title for title particulars and any easements, restrictions, or covenants.
- The PDF copy of this plan has surveyed features on separate layers. These layers may be of assistance with analysis of data on this document.
- Data on this plan may only be manipulated with permission from Marshal Melbourne Pty Ltd.

Vicmap

Vicmap Property/Parcel and Vicmap Elevation
http://www.data.vic.gov.au/
https://www.land.vic.gov.au/maps-and-spatial/spatial-data/vicmap-catalogue
Vicmap Data downloaded 12-06-2024.

LEGEND

- Subject Site
- Road Network
- Intersection to be Upgraded
- Water Main
- Sewer Main
- 100 Year Flood Boundary
- Existing Waterway
- Artefact Scatter and 20m Buffer
- Existing Significant Vegetation
- Existing Buildings
- Retarding Basins
- Local Activity Centre
- Superlot (Delivered with Stage 1)
- Standard Residential Lots (<1000sqm)
- Low Density Residential Lots (1000-2000sqm)
- Public Open Space - Gully/Drainage
- Public Open Space - Tree & Artefact Retention
- Public Open Space - Landscape Buffer

DATE	REVISION	DETAILS	AUTHORISED
7/4/25	A	Stage 11 layout amended.	K.J.E.

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BROOKFIELD LAKES
135 GREAT ALPINE ROAD
LUCKNOW
EAST GIPPSLAND SHIRE
REF: 35753DP-A, Revision A

DEVELOPMENT PLAN