



MIRVAC

Stormwater Master Plan

Teviot Road, Greenbank



**PLANS AND DOCUMENTS
referred to in the PDA
DEVELOPMENT APPROVAL**

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www.engeny.com.au

P: 07 3221 7174 | F: 07 3236 2399

Lvl 11, 344 Queen st Brisbane QLD 4000 | PO Box 10183 Brisbane QLD 4000

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M43000_002 GREENBANK STORMWATER MASTER PLAN							
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1. INTRODUCTION

Engeny Water Management (Engeny) has been commissioned by Mirvac to prepare a Stormwater Master Plan (SMP) for the proposed 3,350 lot residential subdivision located at Teviot Rd, Greenbank (the Site). The development sits within the Logan City Council local government area, however the Site falls within a declared Priority Development Area (PDA), with the assessing authority being Economic Development Queensland (EDQ).

The SMP comprises the following:

- *Flood mapping - To inform development boundaries and flood extents.*
- *Hydrologic management – To quantify changes in catchment hydrology as a result of the development and required stormwater infrastructure.*
- *Water quality strategy – To develop a stormwater quality treatment train to demonstrate compliance with relevant guidelines.*

The SMP has been prepared with reference to the following guidelines and planning documents;

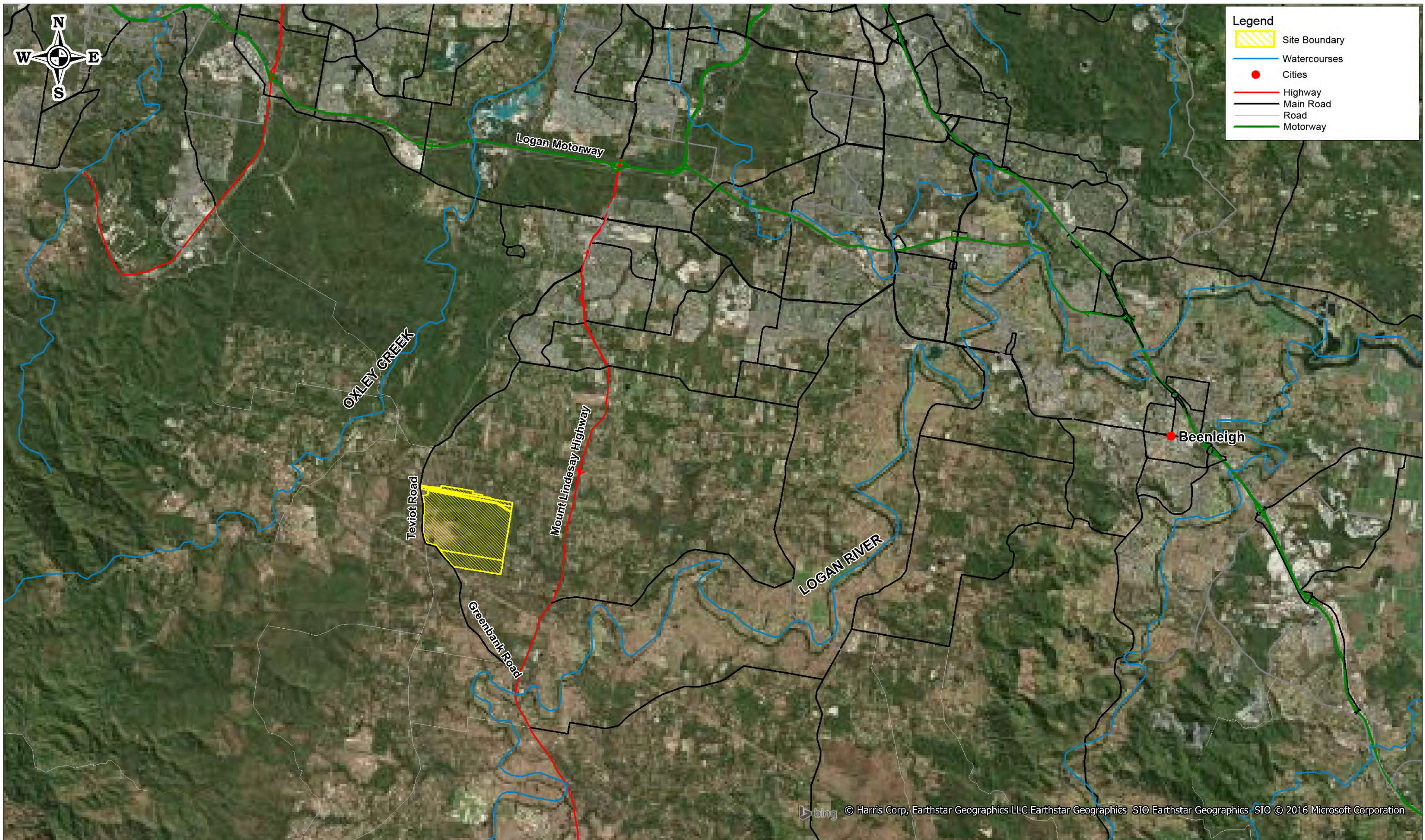
- Logan City Council Planning Scheme (LCC, 2015)
- Queensland Urban Drainage Manual (QUDM) (Department of Energy and Water Supply (DEWS), 2013)
- MUSIC Modelling Guidelines for Southeast Queensland – Version 1, (Water by Design, 2010)
- Framework for the Integration of Flooding and Stormwater Management into Open Space, (Water by Design: 2014)

The SMP has been developed to comply with the State Planning Policy (SPP) (DSDIP, 2014).

2. SITE DETAILS

2.1 Site Location

The Site is located off Greenbank Road and Teviot Road, Greenbank. The Site comprises Lot 434 on RP845844, Lot 9 on S312355 and Lot 205 on RP 854844, is approximately 481 hectares in size and is currently used as farmland. The total yield is anticipated to be in the order of 3,350 lots. The development is proposed to occur over three application areas with all three areas being assessed as part of the Stormwater Master Plan. The Area 1 application has been assessed in isolation as there may be a delay in access to Areas 2 and 3 due to timeframes around EPBC referral and resolution. This means that area 1 can be serviced without accessing the balance of the site. The location of the Site is presented in Figure 2.1.



2.2 Site Encumbrances

The Site is currently encumbered by the following:

- Four primary drainage lines (Tributaries of Norris Creek) that traverse the Site generally in a West- East direction.
- Two existing farm dams that have an impact on attenuation of pre-developed flows (proposed for removal).
- Easements over two services on the northern border of the Site (power and water).
- The requirement for EPBC referral and resolution prior to development of the eastern portion of the site.

2.3 Topography and Drainage

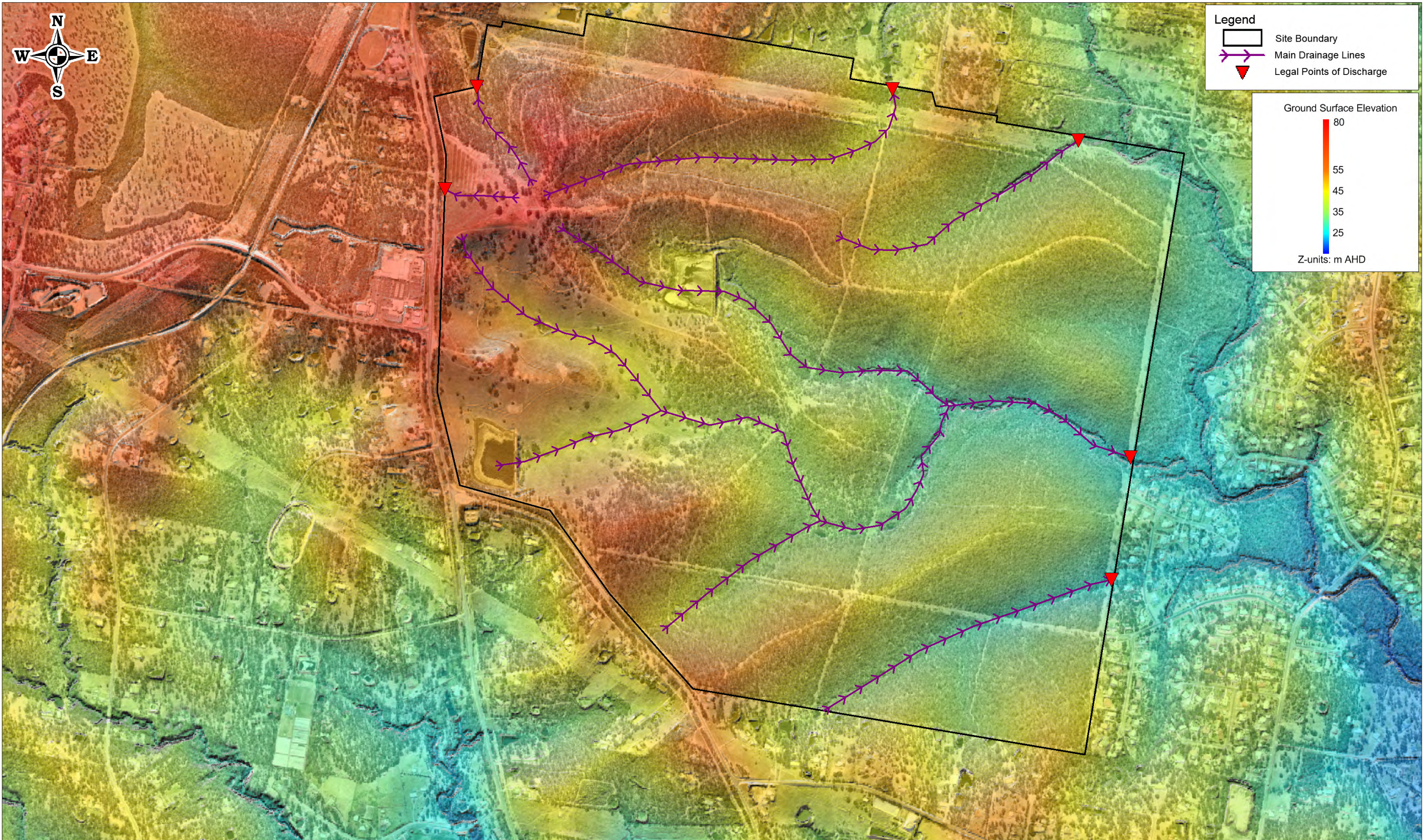
The Site is located in the Logan River catchment and the current site topography and drainage characteristics are described as follows:

- Elevations on the Site range from 25 to 78 m AHD.
- Minimal upstream catchment, the majority of the upstream catchment comprises the Site.
- Drainage lines are well defined and provide the lawful point of discharge for Site runoff at the development boundary. The predominant drainage lines flows from west to east across the Site before discharging into the Logan River through Norris Creek. An additional drainage line is located south of the major drainage line and also flows towards the east. Three minor drainage lines flow across the northern boundary.

Site topography and main drainage lines are shown in Figure 2.2.

2.4 Lawful Points of Discharge

When land is developed, the roof and stormwater runoff from that land and any external catchment (draining through the development site) must be discharged to a defined lawful point of discharge (LPD). Specific locations of LPD will be defined in future application in alignment with the design outcomes and standards presented in Appendix E.



2.5 Proposed Development

The proposed development site is approximately 481 hectares and is shown in Figure 2.3. The predominant post-development land use will be approximately 3,350 residential lots. There will be three main residential land use types:

- Residential – Standard Lots at 15 dwellings per hectare
- Residential – Interface Lots
- Residential – Eco Lots.

In addition, the development will also contain a neighbourhood centre, community facility, school and linear/ neighbourhood and recreational parks. A portion of the site will remain undeveloped as it is identified as forming part of Logan City Council's Conservation Parkland.

Figure 2.3 shows the proposed development layout.



Figure 2.3 Proposed Development Layout

3. FLOOD MANAGEMENT

The Defined Flood Event (DFE) for the Site is the 100 year ARI flood event as stated in the Logan City Council Planning Scheme (LCC, 2015). TUFLOW hydraulic modelling was undertaken to map the existing flood extent and to identify the main drainage lines through the Site.

XPRAFTS hydrologic modelling was undertaken to calculate existing and post-development peak flows for the site and to size appropriate stormwater detention infrastructure along the main drainage lines to ensure that no-worsening of flooding conditions is achieved at the site LPDs.

The flood management assessment was performed with reference to the following standards:

- Logan City Council Planning Scheme (LCC, 2015).
- Queensland Urban Drainage Manual (QUDM) (Department of Energy and Water Supply (DEWS), 2013).
- Engineers Australia (EA), Australian Rainfall and Runoff: A Guide to Flood Estimation.
- Australian Rainfall and Runoff (AR&R) Revision Project 15 – Two Dimensional Modelling in Urban and Rural Floodplains.

3.1 Assessment Approach

XPRAFTS hydrologic modelling and TUFLOW hydraulic modelling was undertaken for the purpose of identifying the constraints of the Site and sizing appropriate stormwater infrastructure to achieve post-development flood attenuation at key reporting locations.

A key consideration for determining existing site flows was the attenuation provided by the farm dams currently located on the Site. An iterative process was adopted to simulate the peak flows from these sub-catchments in the XPRAFTS model to produce accurate existing development peak flow targets for detention sizing. The following methodology was adopted:

- Development of a XPRAFTS existing development scenario hydrology model to produce inflows for the TUFLOW hydraulic model, herein referred to as the initial existing scenario XPRAFTS model.
- Development of a 2D TUFLOW existing development scenario hydraulic model for the purpose of estimating peak flows from catchments containing dam storages and catchment lags.

- Alterations to the XPRAFTS initial existing development scenario model to include TUFLOW simulated flows from the catchments affected by existing dam storages, herein referred to as the final existing scenario model.
- Development of an XPRAFTS post-development scenario hydrology model for the purpose of detention basin sizing, herein referred to as the post-development scenario XPRAFTS model.

The 5, 20 and 100 year ARI flood events have been analysed for the Stormwater Master Plan.

3.2 Flood Modelling

3.2.1 Hydraulic Assessment

TUFLOW Model Development

TUFLOW 2D hydraulic modelling of the existing development scenario has been undertaken for 5, 20 and 100 year ARI flood events to identify main drainage flow paths and to simulate dam storages currently present on the Site.

The model extent covers the streams on Site at their furthest upstream location and extends downstream to Greenhill Road. A 2013 1 m LiDAR data set for the Site was utilised for the topography in the model. A cell size resolution of 2 m was selected to accurately model the dam storage spillways and flow paths existing on the Site and a timestep of 1 second was selected to achieve model stability.

The Manning's roughness applied for varying land use types are summarised in Table 3.1. These values have been adopted in accordance with Table 10.1 of the AR&R Revision Project 15.

Table 3.1 Manning's "n" Roughness Values

Land Use	Manning's "n" Value
Medium Vegetation	0.08
Road	0.02
Open Grass	0.05
Open Water	0.025
Rural Residential	0.12
Medium Residential	0.15

A XP-RAFTS hydrological model was developed to represent the existing scenario and to provide inflows into the hydraulic model. Note that the model utilised to create inflows into the hydraulic model did not contain catchment lagging links or flows from the dams, these were added after the TUFLOW model, and this revised model was utilised for the stormwater mitigation analysis. The development of this model is discussed in Section 3.3.1.

The dam storages on Site have been modelled as full by applying initial water level polygons in the TUFLOW model. These water levels are summarised in Table 3.2.

Table 3.2 Dam Storage Initial Water Levels

Dam	Initial Water Level (m AHD)
Northern	46.25
Southern	53.00

Inflows from the initial existing scenario XPRAFTS model were applied as local inflows along the streams in the model extent. The TUFLOW model was simulated for the 5, 20 and 100 year ARI, flood events for the durations ranging between 30 and 120 minutes. Reporting locations were placed directly downstream from the storage spillways to record flow hydrographs from the dam storages.

Flood Constraint Mapping

The 100 year ARI flood extent from the existing scenario hydraulic simulation is provided in Appendix B. This mapping gives an indication of the extent of flooding currently experienced on site and informs required locations of flood mitigation infrastructure.

3.3 Flood Mitigation

The Greenbank development is proposed to occur over three application areas. Flood mitigation requirements for all three areas have been assessed as part of the Stormwater Master Plan. In addition, flood mitigation requirements for Area 1 have been assessed in isolation as there may be a delay in access to Areas 2 and 3 due to timeframes around EPBC referral and resolution.

Detention basins have been proposed at specific locations throughout the Site and sized to achieve non-worsening of peak flood flows at both the Site boundary and further downstream at Greenhill Road. In some instances, portions of catchments upstream of detention basins are proposed to bypass the basins in order to reduce required detention volumes whilst still achieving mitigation objectives. Consequent timing of hydrographs enables bypassed catchment flows to not adversely affect peak discharge downstream of the detention basins.

Details of the existing development and post development hydrology models, locations and sizing of basins, as well as catchments reporting to each basin and the peak discharge assessment locations are discussed in the following sections.

3.3.1 Hydrologic Analysis

Existing Scenario XPRAFTS Model Development

The hydrologic assessment for the existing Greenbank development was undertaken using XPRAFTS to produce runoff hydrographs as an input into the 2D TUFLOW hydraulic model. Catchments were delineated to produce runoff hydrographs for the required inflow boundaries of the 1D/2D hydraulic model.

The XPRAFTS model was developed using the split catchment approach within XPRAFTS. The impervious and pervious portions of the catchment were modelled as separate catchments with 100% and 0% fraction impervious values respectively. Land use areas for the catchment were delineated using aerial imagery and cadastral boundary data. Fraction impervious values for the catchments were adopted from the delineated land use types summarised in Table 3.3. These fractions have been adopted in accordance with Table 4.5.1 of the Queensland Drainage Manual (2013).

Table 3.3 Existing Scenario Fraction Impervious per Land Use

Land Use	Fraction Impervious
Rural	0%
Road	90%
Rural Residential	10%
Open Area	0%
Industrial	90%

The Manning's "n" values were applied globally for pervious and impervious portions of catchment and are summarised in Table 3.4.

Table 3.4 Manning's "n" Value for Existing Development Scenario Hydrological Model

Catchment Type	Manning's "n" Value
Pervious	0.06
Impervious	0.025

Design rainfall depths were generated using the internal IFD calculation tool within XPRAFTS. The parameters used to generate IFD data were sourced from the Bureau of

Meteorology (BoM) website (www.bom.gov.au) and are shown in Table 3.5 using point coordinates at the centroid of the catchment (27.75S, 153.00E).

Table 3.5 Adopted Parameters used to Generate IFD Data

Parameter	Values
$50I_{1h}$	84.87
$50I_{12h}$	15.14
$50I_{72h}$	4.84
$2I_{1h}$	45.61
$2I_{12h}$	7.67
$2I_{72h}$	2.24
Skew	0.14
F2	4.4
F50	17.15

The remaining catchment parameter inputs adopted for the XPRAFTS existing hydrology model are supplied in Appendix A. Note that lags of the catchment links were not included in the initial iteration of the hydrology model; in addition to simulating peak flows from the existing farm dams TUFLOW was utilised to calculate average stream velocities to inform required lags.

Hydrographs were extracted from the TUFLOW hydraulic model for the 5, 20 and 100 year ARI flood events for the durations ranging between 30 and 120 min. These were applied as user input hydrographs on the catchments delineated to the dam storage spillways (Catchments G_32 and G_1) to replace the hydrographs generated by XPRAFTS. The peak discharge from these hydrographs applied in XPRAFTS at the farm dam catchments, as extracted from TUFLOW, for the critical duration envelope are provided in Table 3.6. The farm dam located furthest south is referred to as Farm Dam 1, and the dam located to the north of this one is referred to as Farm Dam 2.

Table 3.6 Peak Discharge Applied to Farm Dam Catchments

Flood Event	Peak Discharge (m ³ /s) - Farm Dam 1	Peak Discharge (m ³ /s) - Farm Dam 2
5 year ARI – 60 min	1.41	0.29

Flood Event	Peak Discharge (m ³ /s) - Farm Dam 1	Peak Discharge (m ³ /s) - Farm Dam 2
5 year ARI – 90min	1.49	0.40
5 year ARI – 120min	1.45	0.50
20 year ARI – 60 min	2.24	0.75
20 year ARI – 90min	2.27	1.10
20 year ARI – 120min	2.22	1.30
100 year ARI – 60 min	3.46	1.98
100 year ARI – 90min	3.40	2.55
100 year ARI – 120min	3.22	2.67

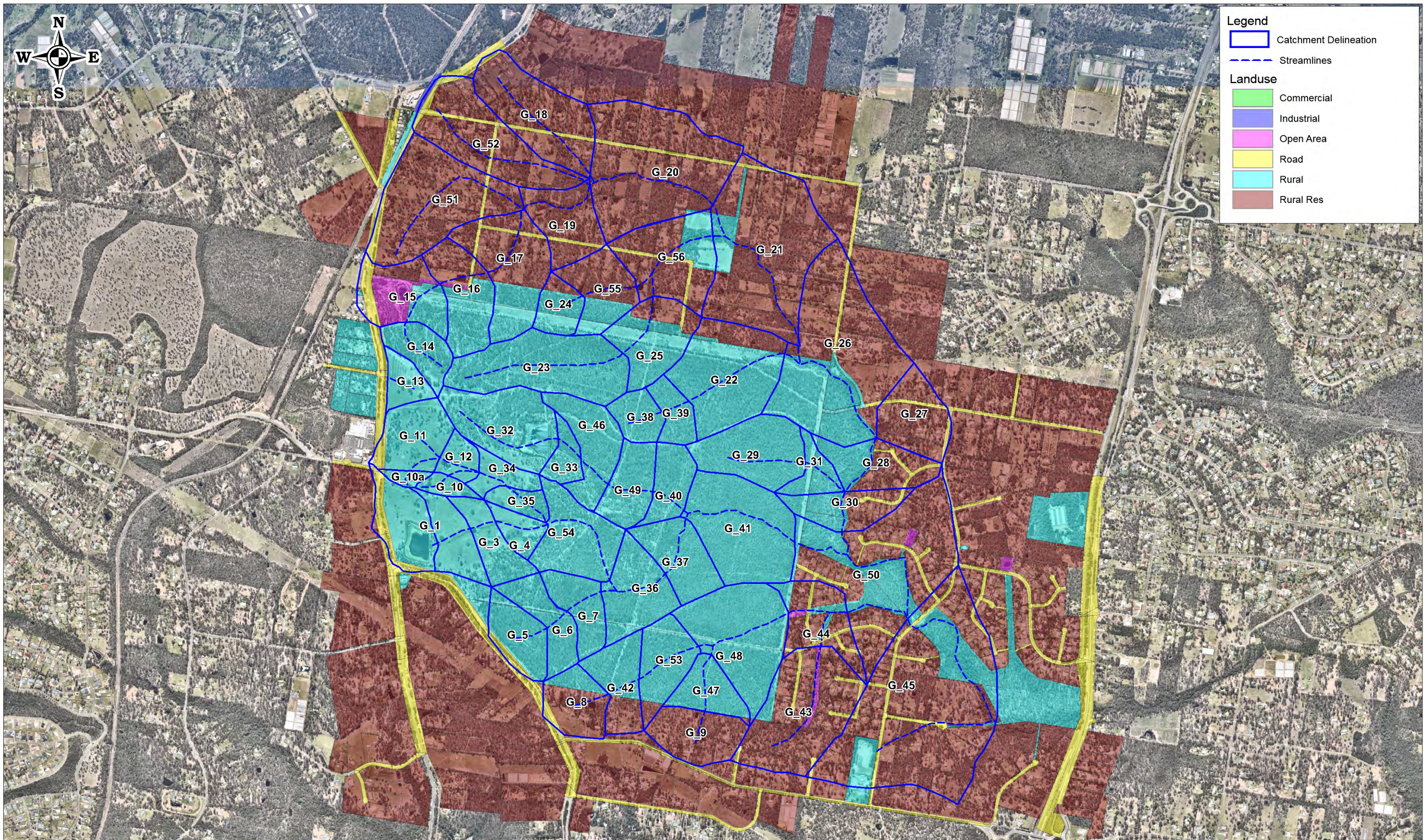
An average stream velocity and length along the node to node flow path was extracted from TUFLOW to calculate the lag times required to be applied between catchments. The final existing scenario XPRAFTS model was simulated for the 5, 20 and 100 year ARI events for durations ranging between 30 and 120 min to produce the peak flow targets for detention basin sizing. All durations ranging from 2 to 100 year ARI will be considered with the application of future Stormwater Management Plans.

The rainfall losses adopted for all simulated design storm events are summarised in Table 3.7. These values align with guidance supplied in AR&R (EA, 1997).

Table 3.7 Rainfall Losses

Catchment Type	Initial Loss (mm)	Continuing Loss (mm/hr)
Pervious catchment	10	2.5
Impervious catchment	1	0

A layout of the existing development catchment delineation and land use is provided in Figure 3.1.



Rational Method Validation

The existing development scenario XPRAFTS hydrological model was validated against the Rational Method at the downstream extent of the XPRAFTS model. A summary of the Rational Method parameters are provided in Table 3.8. Time of concentration was calculated using an average stream velocity of 0.9 m/s; the average stream velocity simulated in the TUFLOW hydraulic model. The C10 value has been selected in accordance with Table 4.5.4 of the Queensland Urban Drainage Manual (2013).

Table 3.8 Rational Method Validation Parameters

Parameter	Value
Catchment Area	1,067 ha
C10 Value	0.70
Time of Concentration	122 mins

A comparison of peak flows at the outlet of the hydrological model to those estimated by the Rational Method are provided in Table 3.9. The difference between the two is less than 5% and therefore validation is achieved.

Table 3.9 Rational Method Validation of XPRAFTS Peak Flows

Flood Event	XPRAFTS Peak Discharge (m ³ /s)	Rational Method Peak Discharge (m ³ /s)	Difference (%)
5 Year ARI	142.1	150.0	-5
20 Year ARI	101.8	99.5	2
100 Year ARI	71.2	69.0	3

Post-Development Scenario XPRAFTS Hydrologic Models

The existing scenario XPRAFTS hydrology model formed the basis for the post-development scenario models. There were two models, one to represent development only occurring in Area 1 for the interim scenario and the second to represent development over the whole of site. The existing scenario sub-catchment delineation was revised to align with the masterplan bulk earthworks – within the boundaries of Area 1 for the Area 1 model, and for the entirety of the development for the ultimate scenario. Additionally, the land use and associated impervious fractions were altered to reflect the two post-development scenarios. These land use and fraction impervious values are summarised in Table 3.10. These fractions have been adopted in accordance with Table 4.5.1 of the Queensland Urban Drainage Manual (2013).

Table 3.10 Post-Development Scenario Fraction Impervious per Land Use

Land Use	Fraction Impervious
Residential – Standard Lots	0.75
Roads	0.9
Rural Residential	0.1
Residential - Interface Lots	0.15
Recreation Park	0.1
Neighbourhood Centre	0.9
Open Area	0
Linear Park	0.02
Rural	0
Neighbourhood Park	0.02
School	0.7
Residential - Eco Lots	0.08
Industrial	0.9

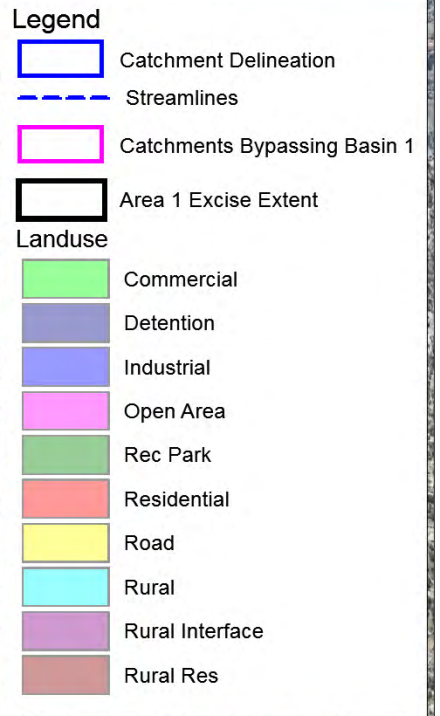
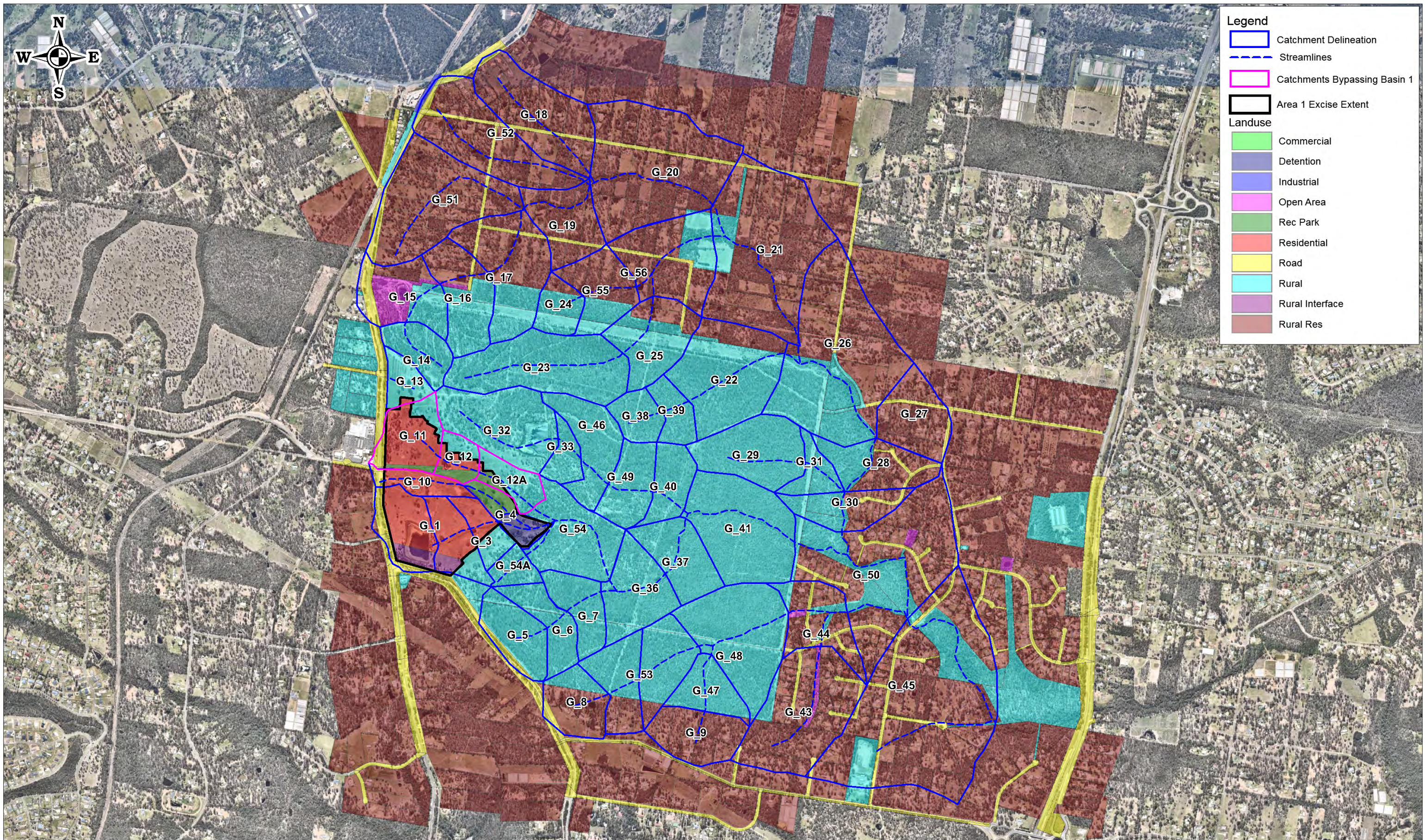
The Manning's "n" values are expected to vary across portions of the Site in the post-development scenarios. The adopted roughness values are summarised in Table 3.11.

Table 3.11 Manning's "n" Value for Post-Development Scenario Hydrological Model

Catchment Type	Manning's "n" Value
Pervious – within residential standard lots area	0.04
Pervious – within open space	0.05
Pervious – for all other areas	0.06
Impervious	0.025

The remaining catchment parameter inputs adopted for the XPRAFTS post-development scenario hydrology models are supplied in Appendix A. The post-development catchment lags remain the same as those applied for the existing scenario where catchment delineation remains unchanged. Where the post development catchment delineation

differs from the existing scenario, the lags have been revised for the proposed flow path length. The Interim development scenario catchment delineation and land use can be seen in Figure 3.2. The Ultimate development scenario catchment delineation and land use can be seen in Figure 3.3.



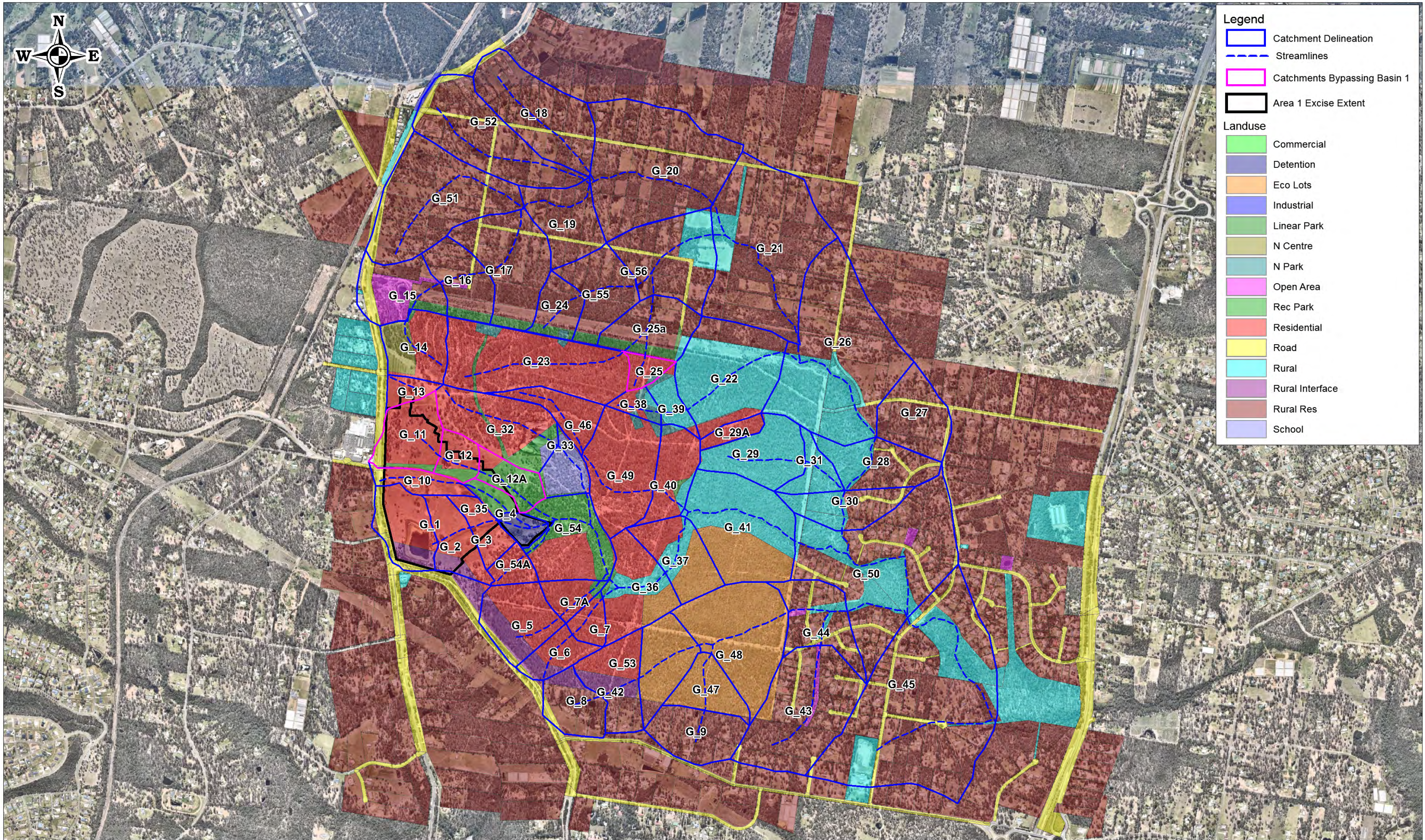


Figure 3.3

3.3.2 Detention Sizing

Interim Development Scenario

Flood mitigation options for Area 1 have been investigated in isolation to those required for the balance of the site as an Interim development scenario solution. This has been done due to potential delays in access to Areas 2 and 3 associated with resolution of the EPBC referral. Flood mitigation will therefore be achieved for Area 1 in isolation.

A detention basin is proposed to be located in the open space at the edge of the Area 1 development. The location of the proposed basin can be seen in Appendix C. The basin was modelled in the post-development scenario XPRAFTS hydrological model immediately downstream of catchment G_4. The XPRAFTS hydrological model used to size the basin was the final existing scenario model with the parameters of catchments within Area 1 altered to the post-development scenario parameters.

A stage-volume curve produced by 12D was utilised to size appropriate outlet configurations for the basin and to calculate maximum stored volumes and water surface levels in the 5, 20 and 100 year ARI flood events. This storage curve is provided in Table 3.12. It is noted that this storage curve does not include the 300mm extended detention depth associated with the bioretention system that is proposed in the base of the detention basin to meet water quality requirements.

Table 3.12 Basin 1 Storage Curve

Elevation (m AHD)	Volume (m ³)
39.9	-
40	372
40.1	755
40.2	1,149
40.3	1,555
40.4	1,972
40.5	2,539
40.6	3,392
40.7	4,532
40.8	5,936
40.9	7,556

Elevation (m AHD)	Volume (m ³)
41	9,362
41.1	11,320
41.2	13,400
41.3	15,566
41.4	17,787
41.5	20,036
41.6	22,310
41.7	24,610
41.8	26,937
41.9	29,290

A summary of the basin outlet configuration for Basin 1 has been provided in Table 3.13.

Table 3.13 Area 1 Detention Basin Configuration

Basin	Outlet Configuration	Spillway Configuration	Crest Level	Indicative Basin Area Required (m ²)	LPD Reporting Location
Basin 1	1 / 1.2x0.6 m RCBC	10m wide at 41.6 m AHD – Depth of 1.2m	42.2 m AHD	30,000	Location 1

To reduce the required detention basin volumes, catchments along the northern edge of the green space (referred to as G11, G12 and G12A on Figure 3.2) have been diverted around the basin by conveying the flow from these catchments to the open space area at the outlet of Basin 1 through their own separate pipe and overland flow arrangement. The basin will achieve non-worsening at the Greenbank development property boundary and further downstream at Greenhill Road. LPD peak discharge reporting locations and the layout of Area 1 and the balance of the site are illustrated on Figure 3.5. This figure also shows the catchments to be diverted around the basin. The basin location is shown in Appendix C.

The basin has been configured to minimise its footprint due to land area constraints associated with the EPBC boundary. This has resulted in a single basin with two distinct basin inverts; a higher grassed area that could be used as an informal kick around area

and a lower section which is set 800mm below the grassed area which serves as both 300mm of bioretention extended detention and 500mm of dedicated detention storage. The water depths within this area will exceed the 1.2m depth in the 20 year ARI flood event however access will be limited through dense planting on both the batters and base of the system. This component of the basin is set at the back of the detention basin which also acts to limit access to the 1 in 2 batters located on the western side of the basin. The configuration and levels of the detention and bioretention systems proposed for Area 1 are shown in Figure 3.4.

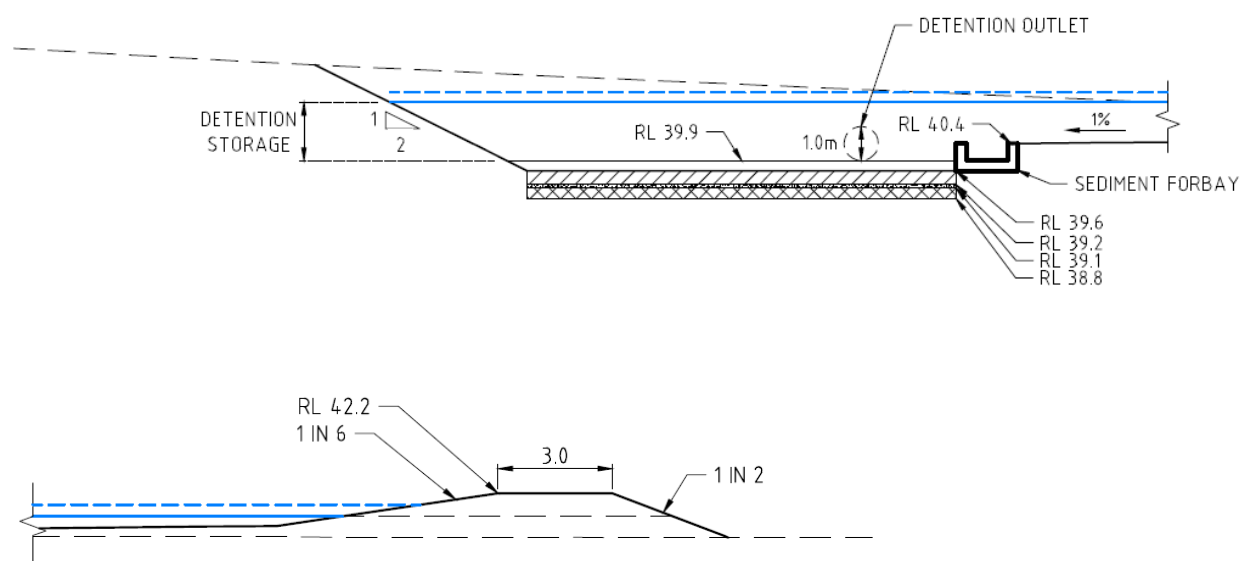


Figure 3.4 Detention and Bioretention System 1 Typical Section



The existing, un-mitigated post-development (no detention basin) and mitigated post-development (with detention basin) peak discharges at the LPD reporting location previously identified are summarised in Table 3.14. The critical duration has been identified by running multiple durations in XPRAFTS for each flood event and the respective peak discharges have been reported for the critical duration only.

Table 3.14 Peak Discharges at LPD Reporting Location for Detention Basin 1

Flood Event	Existing Development Peak Discharge (m ³ /s)	Post-Development Peak Discharge (m ³ /s)	Post-Development Mitigated Peak Discharge (m ³ /s)	Diff. (%)
5 year ARI	23.75	29.3	23.38	-1.6
20 year ARI	34.36	42.29	32.78	-4.6
100 year ARI	47.86	56.95	46.01	-3.9

Figure 3.6 shows a comparison of peak discharge critical duration (90 min) hydrographs for the existing scenario versus the post-development mitigated scenario at the reporting location for Detention Basin 1.

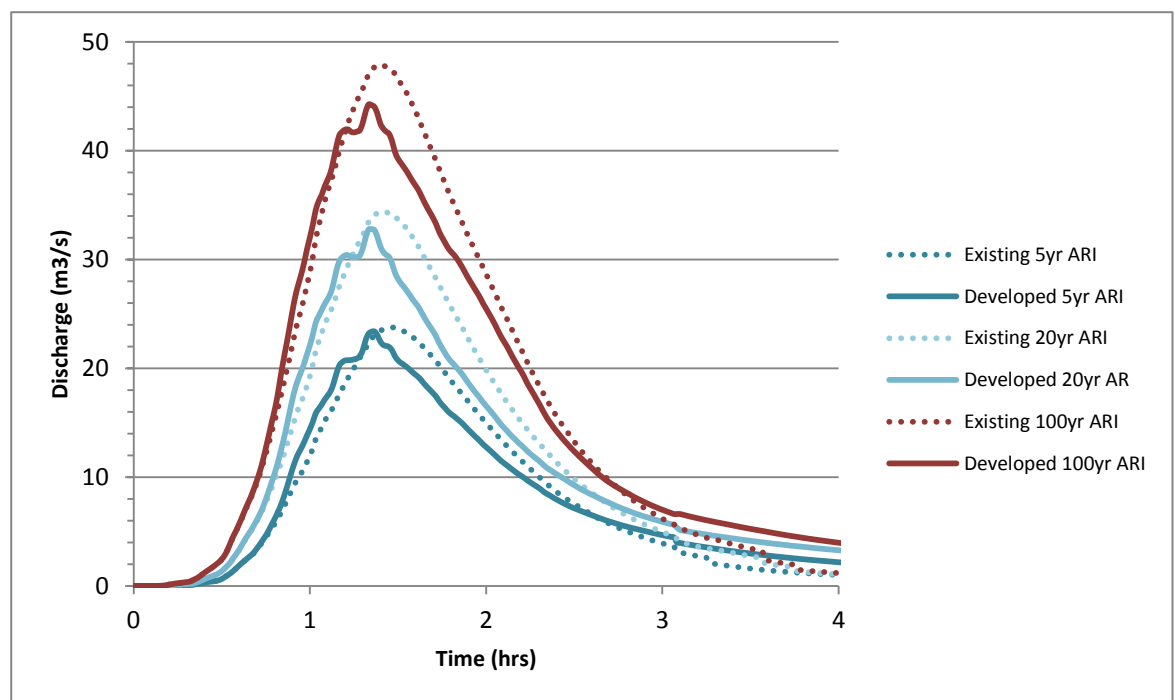


Figure 3.6 Existing vs Post Development Mitigated Discharge at Basin 1 LPD Reporting Location

The maximum water level, depth and stored volumes (excluding water quality component) are also summarised in Table 3.15. As the crest level of the basin is located at 42.2 m AHD, Basin 1 has 350mm of freeboard to the crest in the 100 year ARI event.

Table 3.15 Maximum Water Depth and Volume within Detention Basin 1

Flood Event	Maximum Water Depth (m)	Maximum Water Level (m AHD)	Maximum Stored Volume (m ³)	Approximate Plan Area (m ²)
5 year ARI	1.03	41.33	16,228	22,104
20 year ARI	1.23	41.63	22,997	22,952
100 year ARI	1.55	41.85	28,110	23,527

Cumulative impacts for Area 1 have also been assessed by calculating peak discharges at Greenhill Road (XPRAFTS node G_50). These discharges are summarised in Table 3.16.

Table 3.16 Peak Discharges at Greenhill Road for Area 1

Flood Event	Existing Peak Discharge (m ³ /s)	Post-Development Peak Discharge (m ³ /s)	Post-Development Mitigated Peak Discharge (m ³ /s)	Dif. (%)
5 year ARI	71.15	74.74	70.27	-1.2
20 year ARI	101.77	105.79	99.32	-2.4
100 year ARI	142.07	146.42	137.63	-3.1

Figure 3.7 shows a comparison of peak discharge hydrographs (90 minute critical duration) for the existing scenario versus the post-development mitigated scenario at Greenhill Road for Area 1 development.

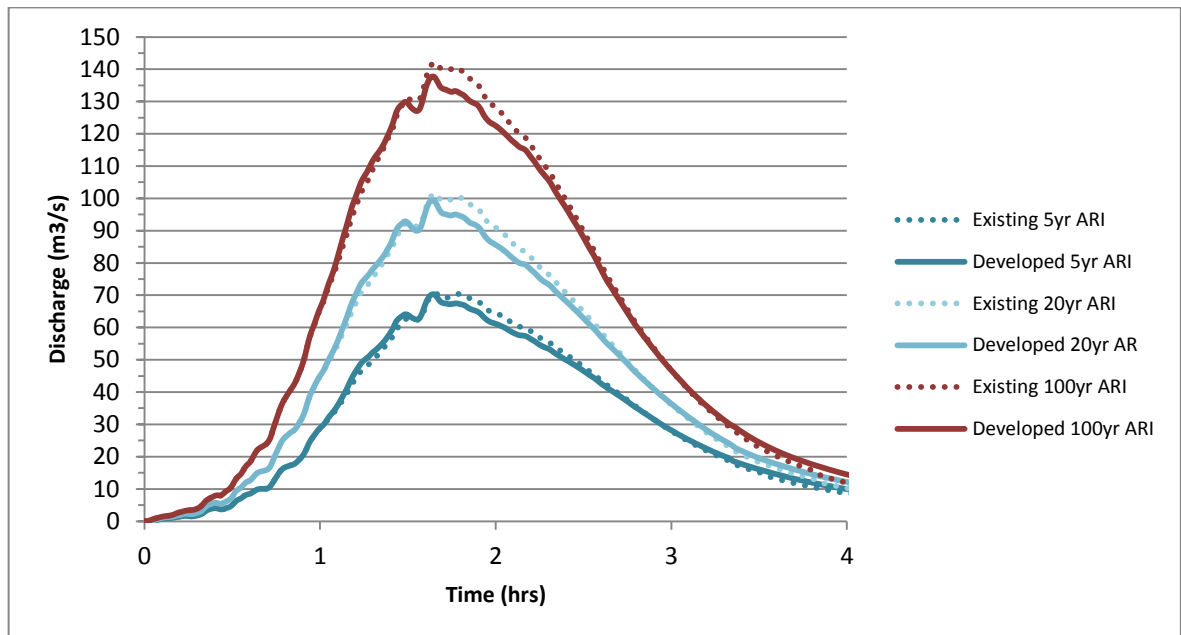


Figure 3.7 Existing vs Post Development Mitigated Discharge at Greenhill Road

Ultimate Development Scenario Detention Basins – Basin 2

For the ultimate development scenario, it is proposed that the interim scenario detention basin (Basin 1) be filled and converted to regional recreation park, with only a portion of the water quality component remaining. To provide mitigation for the catchment located upstream of the ring road in the ultimate scenario, a basin will be constructed utilising the ring road 1 embankment. This basin is referred to as Basin 2. A stage-volume curve produced in 12D was utilised to size appropriate outlet configurations for Basin 2 and to calculate maximum stored volumes and water surface levels in the 5, 20 and 100 year ARI flood events. This storage curve is provided in Table 3.17. It is noted that this storage curve does not include the 300mm extended detention depth associated with the wetland system that is proposed in the base of the detention basin to meet water quality requirements. Catchments G11, G12 and G12A which previously bypassed basin 1 will also discharge into Basin 2.

Table 3.17 Basin 2 Storage Curve

Elevation (m AHD)	Volume (m³)
34.4	-
34.5	2,058
34.6	4,133
34.7	6,225
34.8	8,334

Elevation (m AHD)	Volume (m ³)
34.9	10,461
35	12,606
35.1	14,768
35.2	16,949
35.3	19,147
35.4	21,364
35.5	23,599
35.6	25,852
35.7	28,124
35.8	30,415
35.9	32,725
36	35,054
36.1	37,402
36.2	39,769
36.3	42,156
36.4	44,563
36.5	46,989
36.6	49,435
36.7	51,901
36.8	54,388
36.9	56,910
37	59,558
37.1	62,292
37.2	65,054

Elevation (m AHD)	Volume (m ³)
37.3	67,845
37.4	70,663

A summary of the basin outlet configuration for Basin 2 has been provided in Table 3.18.

Table 3.18 Basin 2 Configuration

Basin	Outlet Configuration	Spillway Configuration	Road Level (m AHD)	Indicative Basin Area Required (m ²)	LPD Reporting Location
Basin 2	3 / 1.8x1.2 RCBC	High Flow Culverts – 9 / 1.2x0.6 RCBC at 36.5 m AHD	37.4 m AHD	33,000	Location 1

The existing, un-mitigated post-development (no detention basin) and mitigated post-development (with detention basin) peak discharge at the LPD reporting location previously identified are summarised in Table 3.19. The critical duration has been identified by running multiple durations in XPRAFTS for each flood event and the respective peak discharge has been reported for the critical duration only.

Table 3.19 Peak Discharges at LPD Reporting Location for Detention Basin 2

Flood Event	Existing Development Peak Discharge (m ³ /s)	Post-Development Peak Discharge (m ³ /s)	Post-Development Mitigated Peak Discharge (m ³ /s)	Diff. (%)
5 year ARI	23.75	40.18	23.64	-0.5
20 year ARI	34.36	54.52	32.24	-6.2
100 year ARI	47.86	72.63	43.65	-8.8

Figure 3.8 shows a comparison of peak discharge hydrographs (90 min critical duration) for the existing scenario versus the post-development mitigated scenario at the reporting location for Detention Basin 2 for the ultimate development scenario.

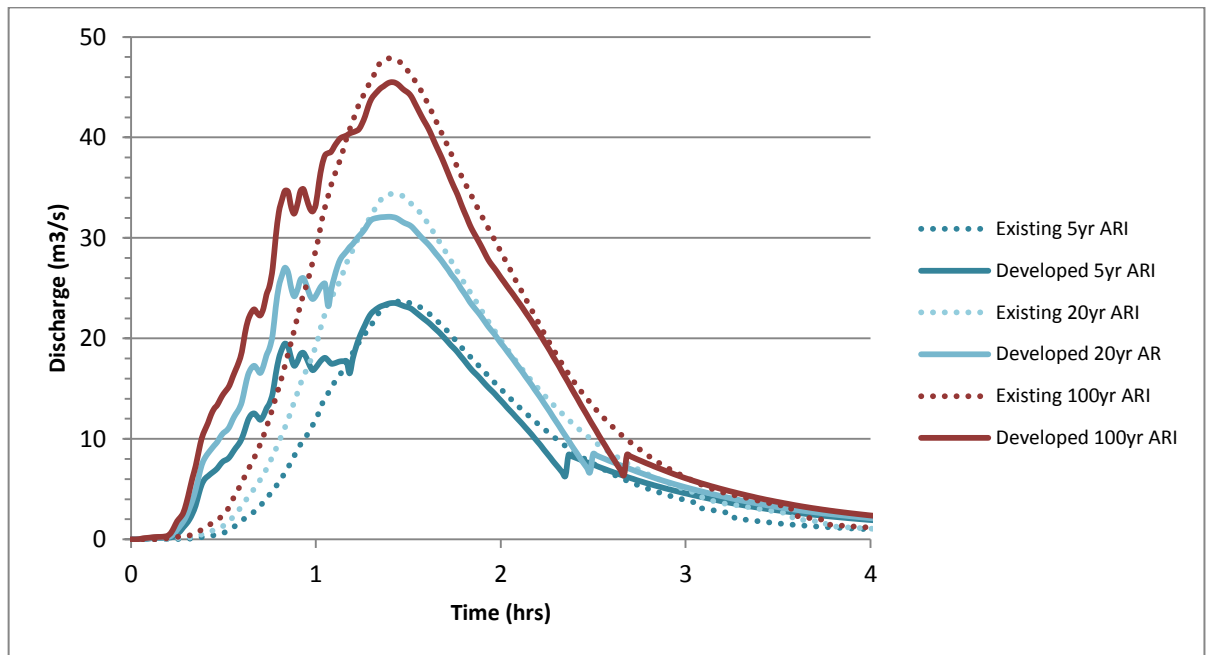


Figure 3.8 Existing vs Post Development Mitigated Discharge at Basin 2 LPD Reporting Location

The maximum water level and stored volumes (excluding water quality component) are also summarised in Table 3.20. As the adjacent road level is located at 37.4 m AHD, Basin 2 has 500 mm of freeboard in the 100 year ARI event. The results show a detention depth of 2.03 m in the 20 year event which exceeds the maximum depth recommended in QUDM. As the entire area of Detention Basin 2 is wetland, access to this area will not be permitted by the public through densely planted batters and safety fencing where required. A detailed safety in design assessment will be undertaken at detailed design to ensure the area is safe during all storm events

Table 3.20 Maximum Water Depth and Volume within Detention Basin 2

Flood Event	Maximum Water Depth (m)	Maximum Water Level (m AHD)	Maximum Stored Volume (m ³)	Approximate Plan Area (m ²)
5 year ARI	1.63	36.02	35,552	23,423
20 year ARI	2.03	36.42	45,046	24,204
100 year ARI	2.50	36.9	56,910	25,772

Ultimate Development Scenario Detention Basins – Basin 3 - 7

For the ultimate development scenario, an additional five basins, in addition to Basin 2, are proposed at varying locations to achieve non-worsening at the Site boundary and at Greenhill Road. These basins were modelled in the post-development scenario XPRAFTS model using simple linear stage-volume curves. More accurate stage-volume curves will be utilised in the future Stormwater Management Plans and stage outlet configurations will be considered to reduce volumes.

These additional basins have been sized appropriately so that a maximum water surface level of 1.5 m above basin invert will not be exceeded in the 20 Year ARI flood event. The area required for the basins has been calculated by conservatively assuming that the 100 year ARI flood volume will be stored in basins only 1.5 m deep whereas in reality this depth will be greater. An additional 25% allowance has been added to the area to allow for embankments and outlet structures etc. for all basins except for basin 2 where this has been reduced to 15% as the basin is to be located against a proposed road embankment.

For Basin 4, catchment G_25 immediately upstream of the basin is proposed to be diverted around the basin to reduce the required volume, this basin is partially located within an electrical and water easement, during detailed design the basin will be designed to ensure compliance with all easement conditions or the basins will be relocated outside the easement if required. A summary of the basin configurations for the total development have been provided in Table 3.21. The basin locations are shown in Appendix D. LPD reporting locations are shown in Figure 3.5 .

The basins proposed will reduce flows at each of the reporting locations to below that of existing catchment conditions. The exception is reporting location 6, which does not require detention due to the limited development reporting to this location.

Table 3.21 Total Development Detention Basin Configurations

Basin	Outlet Configuration	Indicative Basin Area Required (m ²)	LPD Reporting Location
Basin 3	4 / 0.675m RCP	4,900	Location 2
Basin 4	2 / 0.825m RCP	18,100	Location 3
Basin 5	2 / 0.525m RCP	6,200	Location 4
Basin 6	2 / 0.45m RCP	990	Location 5
No detention basin required			Location 6
Basin 7	2 / 0.3m RCP	1,800	Location 7

The existing, post-development and post-development mitigated peak discharges for the 5, 20 and 100 year ARI flood events at the Site boundary locations previously identified are summarised in Table 3.22, Table 3.23 and Table 3.24. The maximum water depths

and stored volumes (excluding any volume requirements for water quality) are also summarised. The critical duration has been identified for each flood event and the respective peak discharges have been reported for the critical duration only.

Table 3.22 5 Year ARI Flood Event Peak Discharges at LPD Reporting Locations for Detention Basins 3-7

Basin	Existing Peak Discharge (m ³ /s)	Post-Development Peak Discharge (m ³ /s)	Post-Development Mitigated Peak Discharge (m ³ /s)	Dif. (%)	Maximum Water Surface Level (m)	Maximum Stored Volume (m ³)
Basin 3	10.37	11.08	9.96	-4.0	1.09	3,630
Basin 4	3.75	13.75	3.33	-11.2	1.07	14,145
Basin 5	1.25	4.96	0.98	-21.6	1.06	4,710
Basin 6	0.90	1.40	0.64	-28.6	0.9	800
Reporting Location 6	3.79	3.15	No detention required			
Basin 7	1.81	2.68	1.78	-1.7	0.86	1,194

Table 3.23 20 Year ARI Flood Event Peak Discharges at LPD Reporting Locations for Detention Basins 3-7

Basin	Existing Peak Discharge (m ³ /s)	Post-Development Peak Discharge (m ³ /s)	Post-Development Mitigated Peak Discharge (m ³ /s)	Dif. (%)	Maximum Water Depth (m)	Maximum Stored Volume (m ³)
Basin 3	15.32	16.27	14.28	-6.8	1.47	4,900
Basin 4	5.49	18.45	4.74	-13.7	1.16	18,041
Basin 5	1.85	6.54	1.23	-33.5	1.4	6,220
Basin 6	1.33	1.84	0.81	-39.1	1.19	1,060
Reporting Location 6	5.56	4.73	No detention required			
Basin 7	2.65	3.76	2.57	-3.0	1.16	1,611

Table 3.24 100 Year ARI Flood Event Peak Discharges at LPD Reporting Locations for Detention Basins 3-7

Basin	Existing Peak Discharge (m³/s)	Post-Development Peak Discharge (m³/s)	Post-Development Mitigated Peak Discharge (m³/s)	Dif. (%)	Maximum Water Surface Level (m)	Maximum Stored Volume (m³)
Basin 3	22.11	22.92	21.52	-2.7	1.75	5,830
Basin 4	8.12	23.59	7.57	-6.8	1.23	21,855
Basin 5	2.65	8.19	2.65	0.0	1.68	7,470
Basin 6	1.91	2.3	1.64	-14.1	1.33	1,180
Reporting Location 6	8	6.77	No detention required			
Basin 7	3.79	5.17	3.57	-5.8	1.54	2,139

Cumulative impacts for the total development have also been assessed by calculating peak discharge at the outlet of G_50, placed at the location of Greenhill Road. These discharges have been summarised in Table 3.25.

An increase of approximately 2.1% is observed at Greenhill Road for the 5 year ARI flood event. This increase will be addressed in future Stormwater Management Plans and is likely to be overcome through detailed design of the detention basins, including staged outlets and by taking waterway storage into consideration.

Table 3.25 Peak Discharges at Greenhill Road for Ultimate Development Scenario

Flood Event	Existing Peak Discharge (m³/s)	Post-Development Peak Discharge (m³/s)	Post-Development Mitigated Peak Discharge (m³/s)	Dif. (%)
Q5	71.15	80.46	72.61	2.1
Q20	101.77	112.46	101.24	-0.5
Q100	142.07	153.47	140.72	-1.0

Figure 3.9 shows a comparison of peak discharge hydrographs (90 min critical duration) for the existing scenario versus the post-development mitigated scenario at Greenhill Road for the Ultimate development scenario.

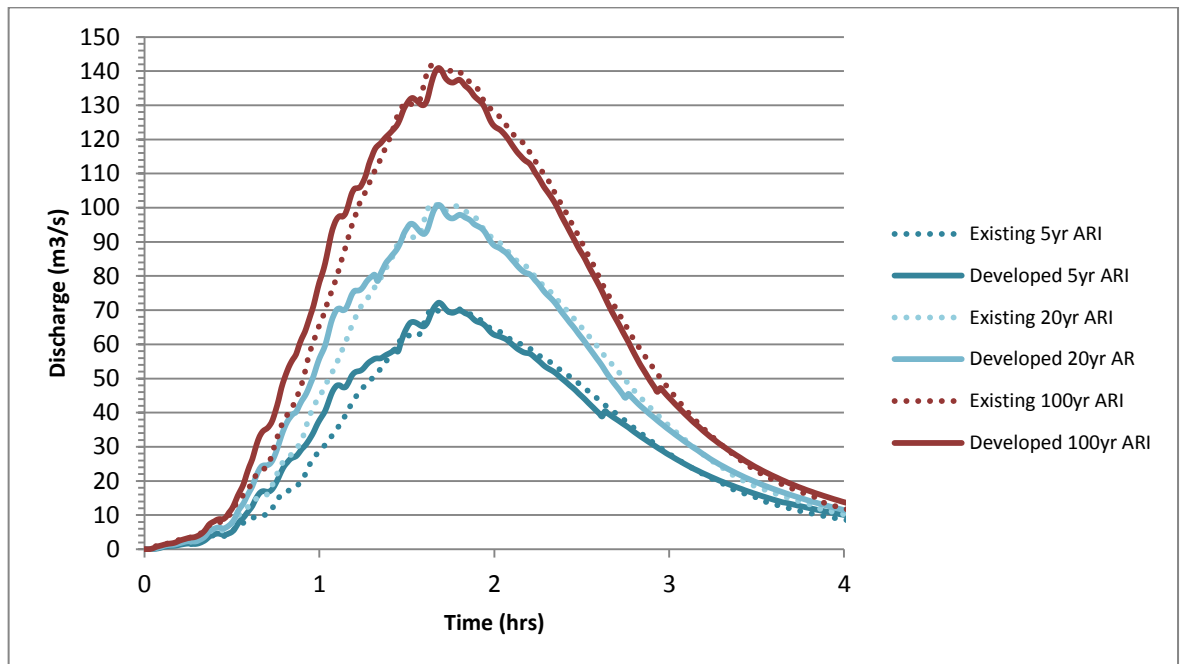


Figure 3.9 Existing vs Post Development Mitigated Discharge at Greenhill Road

A concept layout plan depicting the location of the proposed detention basins discussed above is provided in Appendix D.

3.4 Design Outcomes and Standards

Stormwater quantity design standards and outcomes proposed for adoption across the development are outlined in Appendix E.

4. STORMWATER QUALITY MANAGEMENT

4.1 Water Quality Design Objectives

A conceptual stormwater quality treatment strategy has been developed in accordance with the State Planning Policy (SPP) 2014. Based on the provisions of the SPP, the load based pollutant reduction targets have been used to demonstrate compliance with objectives. Load based targets identified in the SPP were adopted for discharge targets as outlined in Table 4.1.

Table 4.1 Adopted Water Quality Objectives

Pollutant	Minimum Load Reduction Target (%)
Total Suspended Solids (TSS)	80
Total Phosphorus (TP)	60
Total Nitrogen (TN)	45
Gross Pollutants (GP)	90

4.1.1 Climate Data

Climate data for the catchment was sourced from the Bureau of Meteorology (BoM). Rainfall data was obtained from Greenbank Thompson Rd (40659) for the 10 years between 1980 and 1989 at six (6) minute intervals as specified by MUSIC Modelling Guidelines for South East Queensland (Water by Design, 2010).

4.1.2 Catchment Properties

The catchment areas for water quality are based on the proposed development layout. A lumped catchment approach was considered appropriate and adopted for the assessment of the urban residential catchments. Due to the vast areas of open green space within the sports and Recreation Park this area has been excluded from the water quality model.

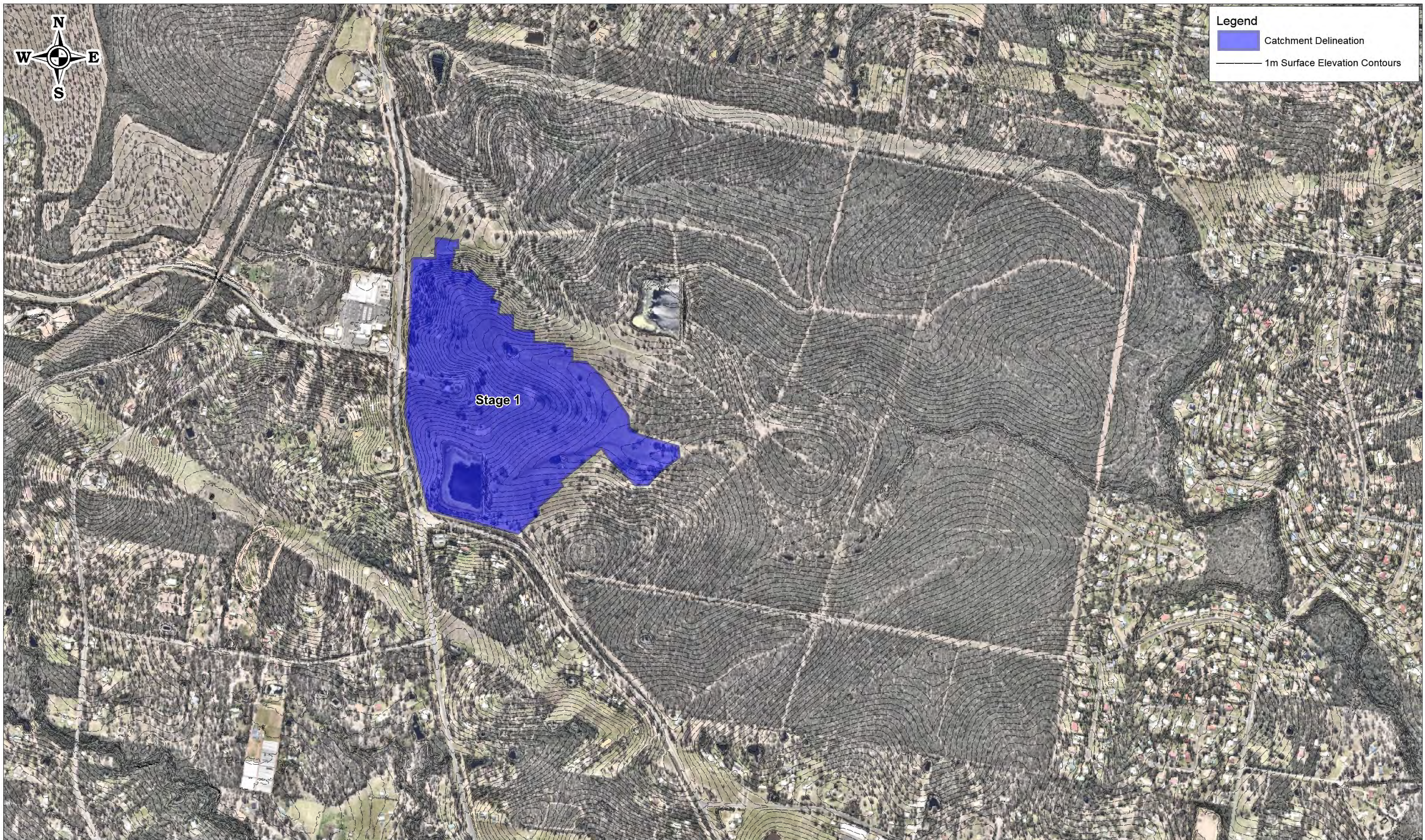
In the case of the two residential interface eco catchments (L & M) a split catchment approach was adopted. Despite the size of the lots in catchment L and M being larger than in the other catchments (1.1 Ha and 5.8 Ha respectively), there are limitations on the developable area of each lot. Due to this restriction the whole area of each lot has not been considered in the water quality analysis; rather a nominal developable Site of 600 m² was assumed for each lot within these catchments. This 600 m² was then split 50/50 between Roof area and Ground area. These assumed areas are included as the catchment area for catchments L and M in Table 4.2.

Rainfall-runoff and pollutant export parameters were adopted from the recommended values identified in Tables 3.7 and 3.8 of MUSIC Modelling Guidelines for South East Queensland (Water by Design, 2010).

Table 4.2 and Figure 4.1 provides a summary of the MUSIC Model parameters adopted with the various land uses throughout the study area. Fraction impervious was adopted based on the recommended values provided by MUSIC Modelling Guidelines for South East Queensland (Water by Design, 2010).

Table 4.2 MUSIC Model Catchment Parameters

Catchment ID	MUSIC Land Use	Area (ha)	Impervious Fraction (%)
Stage 1 Development			
Stage 1 Catchment	Residential	48.8	55 %
Ultimate Development			
A	Residential	54.4	55 % (Based on Table 3.6, Water by Design MUSIC modelling guidelines, 2010)
B		64.9	
C		11.7	
D		39	
E		12.7	
F		3.9	
G		7.05	
H		32.9	
I		5.5	
J		3.3	
K		15.7	
N		3.4	
L	Rural Residential	4.2	Split Catchment Approach
M		0.8	





4.2 Treatment Strategy

The stormwater quality treatment strategy for the Greenbank development takes a three pronged approach to the treatment of runoff from the development. This approach includes the use of a number of water sensitive urban design treatment measures including bioretention systems, a wetland, stormwater harvesting system and waterway enhancement works. Site specific characteristics such as topography, primary pollutant sources, available land area and the existing drainage conditions have all been considered in the development of the master plan strategy.

Further detail into each of the proposed treatment measures for both the interim and ultimate development scenarios are included in the sections below.

4.3 Interim Development Scenario Strategy

To meet water quality requirements for the interim development scenario a bioretention system (B1) is proposed within the base of detention basin 1 (D1). Due to constraints associated with outlet levels this system has been modelled with a 400 mm filter depth as opposed to the 500 mm proposed elsewhere on the site.

Flows entering the detention basin will be directed through an open channel and into a coarse sediment forebay that will act to evenly distributed flows across the basin in a non-erosive manner. During larger flow event the detention volume above the extended detention depth will slowly fill and overflow onto the grassed area, negating any risk of erosion.

4.3.1 Treatment Sizing

Treatment system parameters have been adopted based on the WSUD Technical Design Guidelines for South East Queensland and the Water by Design MUSIC Modelling Guidelines for South East Queensland. The parameters used for the proposed bioretention basin are provided in Table 4.3. The proposed treatment device sizes are included in Table 4.6 and the layout of the MUSIC model is included as Figure 4.3.

A Concept Layout Plan depicting the locations of proposed water quality treatment devices for the interim scenario is included in Appendix C.

Table 4.3 Modelled Ephemeral Wetland Parameters (modelled as Bioretention Basin node)

Parameter	Value
Extended Detention Depth	300 mm
Filter Depth	400 mm
Batter Slope to Natural Surface	1V:4H

Parameter	Value
Orthophosphate Content of Filter Media ¹	3 mg/kg

1 – The orthophosphate content of filter media based on River Sands Pty Ltd product data sheet for Bioretention Filter Media. (Refer Appendix F)

Table 4.4 Proposed Temporary Treatment Measures – Interim Scenario Development

Asset ID	Ephemeral Waterway Filter Area
EW1	3,800 m ²

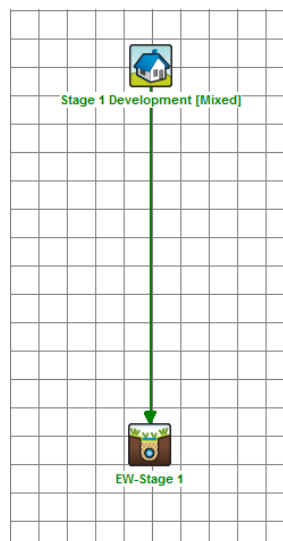


Figure 4.3 MUSIC Model Layout (Interim Scenario Development)

4.3.2 Water Quality Model Results

Results of the MUSIC model are presented in Table 4.5.

Table 4.5 MUSIC Model Results – Stage 1 Development

Pollutant	Load Based Reduction Target (%)	Modelled Reduction (%)
TSS	80	88.6
TP	60	84.6
TN	45	45.3
GP	90	100

Results of the MUSIC modelling indicate that the load based reduction targets have been achieved with the proposed treatment train.

4.4 Ultimate Development

The ultimate development scenario strategy involves the Area 1 detention basin being decommissioned and a portion approximately 1300 m² of bioretention B1 reclaimed as park area. The water quality strategy incorporates a number of WSUD features including a total of 8 bioretention systems, wetland, stormwater harvesting system and waterway enhancement works, as described below.

4.4.1 Bioretention

This treatment strategy proposes the incorporation of eight (8) bioretention systems implemented on a regional scale. Of the eight bioretention systems proposed for this development, three will be located within the base of a detention basin. Where possible the bioretention and detention basins have been combined to reduce the overall footprint of the devices and to minimise the number of assets to be ultimately managed by council.

Bioretention located within detention basins will be incorporated below the invert of the detention storage and therefore the water quality extended detention depth is not included within the detention basin storage curve. Basin inflows will be directed into a coarse sediment forebay that will act to evenly distributed flows across the basin in a non-erosive manner. During larger flow events, the detention volume above the extended detention depth will slowly fill and overflow onto the detention basin surface negating any risk of erosion. Pipe inflows will be managed with adequate scour protection and located far enough away from the bioretention filter media to not pose as an issue. The exact orientation of each system and scour protection requirements will be addressed further at detailed design stage. A concept sketch showing the arrangement of a bioretention basin at the base of the detention basin is included as Figure 4.4.

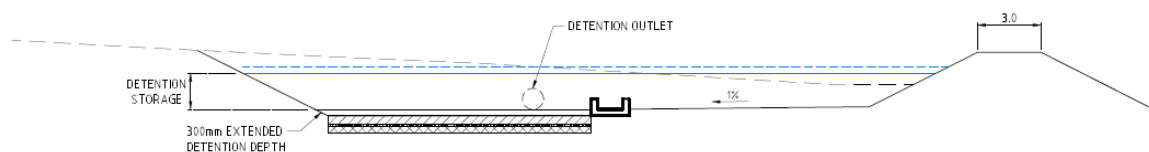


Figure 4.4 Bioretention Basin in Detention Basin – Concept Sketch

4.4.2 Wetland and Water Harvesting

The master plan strategy also proposes the inclusion of a wetland, to be located at the southern end of the new central park within the development. As part of the wetland it is proposed to incorporate a water harvesting system into the treatment train to enable harvesting and reuse of stormwater for irrigation of sports fields within the sports and

recreation park. The wetland and associated infrastructure will consist of an inlet pond (sediment basin), wetland, harvesting pond and a pump/irrigation system.

The wetland is proposed to be combined with Detention Basin D2 which is to be located in the same area. Once again the 300mm extended detention depth required for water quality requirements within the wetland has not been included in the detention basin storage curve.

The harvesting pond will be located at the far end of the wetland to ensure stored stormwater has undergone the maximum amount of treatment through the wetland. The harvesting pond will be separated from the wetland to ensure that water used for harvesting purposes does not impact on the drought resilience of the wetland by drawing down wetland levels. This will be achieved through an outlet structure that will only allow flows from the wetland into the harvesting pond above the wetlands natural water surface level. The harvesting pond batters will be densely planted to minimise access and any visual amenity issues caused by the frequent fluctuation in water level.

4.4.3 Waterway Enhancement

The final water quality treatment device proposed as part of the master plan strategy is Riparian Revegetation. Four sub-catchments have been identified as suitable for the substitution of conventional water quality treatment devices for the revegetation of existing riparian habitats. The revegetation of downstream waterways will be used to offset the SPP water quality treatment objectives for these catchments. The sub-catchments identified as suitable for this approach are C, G, L and M.



Figure 4.5 Existing Creek Line Downstream of Catchments 7 & 13

As part of this strategy it is proposed to revegetate 4 m² of riparian habitat for every 1 m² of bioretention filter media required to meet the SPP treatment objectives for the catchment according to the MUSIC model for the development. As part of the revegetation it is proposed to undertake all the earthworks required for successful vegetation establishment along the relevant sections of waterway. Waterway stabilisation works may be undertaken in lieu of revegetation. Detailed plans for the riparian revegetation are to be included as part of the stormwater management plans for the relevant development stages.

The approach to waterway revegetation, including the identification of the waterway extents to be addressed as part of this work, shall be determined in stage specific Stormwater Master Plans. The success of the riparian revegetation will be assessed based on plant performance against nominated attrition rates and the stability of stabilised areas. A monitoring plan to assess these conditions is to be developed, as part of the future Stormwater Management Plans.

We believe this innovative approach to water quality treatment will offer the following additional benefits to council:

- Increase in waterway stability
- Reduction in maintenance activities due to sedimentation in downstream drains and channels
- Reduced regular maintenance as this work will replace four WSUD assets all of which would require ongoing maintenance activities
- Improved natural habitat
- Improvements to water quality associated with filtration of water through vegetation, shading of the waterway and reduced sediment loads associated with instream erosion.

4.5 Treatment Sizing

Treatment system parameters have been adopted based on the WSUD Technical Design Guidelines for South East Queensland and the Water by Design MUSIC Modelling Guidelines for South East Queensland. The parameters used for the proposed bioretention basins; ephemeral wetland; wetland and harvesting pond are provided in Table 4.6, Table 4.7 and Table 4.8 respectively. The proposed treatment device sizes are included in Table 4.9 and the layouts of the MUSIC models are included as Figure 4.6.

Table 4.6 Modelled Bioretention Basin Parameters

Parameter	Value
Extended Detention Depth	300 mm
Filter Depth	500 mm
Batter Slope to Natural Surface	1 in 4 (V:H)
Orthophosphate Content of Filter Media ¹	3 mg/kg

1 – The orthophosphate content of filter media based on River Sands Pty Ltd product data sheet for Bioretention Filter Media. (Refer Appendix F)

Table 4.7 Modelled Wetland Parameters

Parameter	Value
Extended Detention Depth	400 mm
Inlet Pond Volume	750 m ³
Wetland Surface Area	16,500 m ²
Notional Detention Time	15 Hours

Table 4.8 Modelled Harvesting Pond parameters

Parameter	Value
Extended Detention Depth	200 mm
Permanent Pool Volume	3000 m ³
Harvesting Pond Area	1,500 m ²
Water Harvesting Rate	58,660 kL/Year
Water Harvesting Distribution	PET
% of Demand met from Harvesting	70%

1 – Harvesting rate based on 7000 kL per year per hectare of activity area (8.34 Ha.), which has been derived from a similar project with Ipswich City Council .

Incorporation of a stormwater harvesting system into the sports park will have many benefits including;

- Management of any perceived or real increase in base flows
- A reduction on potable demand (70% of water demand met through harvesting)
- Reduction in the requirement for fertilisers as stormwater naturally contains nutrients
- Reduction in the size of measures required to meet water quality objectives
- Improvement in the overall quality of sports fields.

Table 4.9 Proposed Treatment Measures – Ultimate Development

Asset ID	Bioretention Filter Area	Ephemeral Wetland Area	Wetland Area (Incl. Inlet Pond, Wetland and Harvesting Pond)	Equivalent Riparian Revegetation Area
EW1	-	2,500 m ²	-	-
W2	-	-	20,000 m ²	-
RP3*	850 m ²	-	-	3,400 m ²
B4	2,900 m ²	-	-	-
B5	1,000 m ²	-	-	-
B6	290 m ²	-	-	-
RP7*	490 m ²	-	-	1,960 m ²
B8	2,370 m ²	-	-	-
B9	420 m ²	-	-	-
B10	260 m ²	-	-	-
B11	1,150 m ²	-	-	-
RP12*	300 m ²	-	-	1,200 m ²
RP13*	70 m ²	-	-	280 m ²
B14	250 m ²	-	-	-
Total	8,640 m² – 1,710 m² = 6,930 m²	2,500 m²	20,000 m²	6,840 m²

* Note Bioretention Area is intended to be substituted for Riparian Revegetation Area, refer to Section 4.4.3 for details.

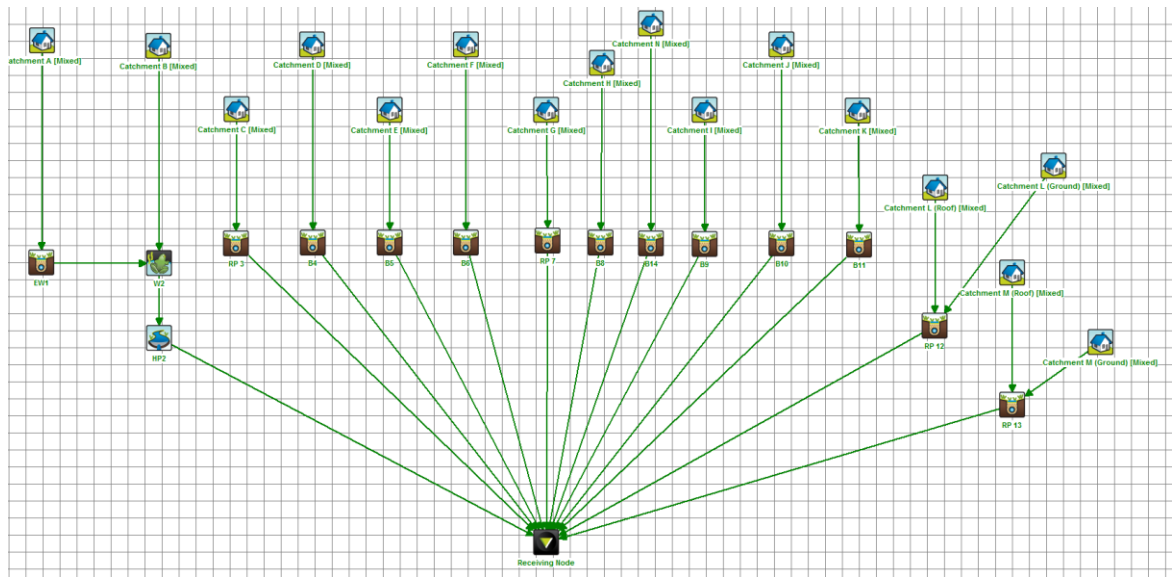


Figure 4.6 MUSIC Model Layout (Ultimate Development)

A Concept Layout Plan depicting the locations of proposed water quality treatment devices for the ultimate scenario is included in Appendix D.

4.6 Water Quality Model Results

Results of the MUSIC model are presented in Table 4.10.

Table 4.10 MUSIC Model Results – Ultimate Development

Pollutant	Load Based Reduction Target (%)	Modelled Reduction (%)
TSS	80	81.5
TP	60	73.5
TN	45	45.0
GP	90	100

Results of the MUSIC modelling indicate that the load based reduction targets have been achieved with the proposed treatment train.

4.7 Design Standards

Stormwater quality design standards and outcomes proposed for adoption across the development are outlined in Appendix E.

5. CONCLUSION

The master plan for the proposed Greenbank development has been developed to achieve the required stormwater management objectives as outlined in SPP 2014. The key objectives of this master plan included:

- Stormwater quality modelling to develop a stormwater quality treatment train to demonstrate compliance with SPP water quality objectives.
- Hydraulic assessment to quantify the Site's flooding constraints due to overland flow, provide mitigation measures to achieve "no worsening" at suitable reporting locations, and provide a connection to existing drainage channels.

Key stormwater quality management design outcomes for the Site are as follows:

- Seven Bioretention Basins are proposed to be incorporated within the development; these basins have a total treatment area of 6,690 m². Four of these bioretention basins will be incorporated into the base of a detention basin sited at the same location.
- One Ephemeral Wetland is proposed to be included at the eastern end of the new Central Park. This ephemeral wetland will be constructed to 3,800 m² as a temporary treatment measure during Stage 1 construction. Once the ultimate development has been approved the temporary structure can be reduced in size to the permanent treatment area of 2,500 m².
- One Wetland and an associated harvesting pond are proposed to be included at the southern end of the new Central Park. It is proposed to channel outflows from the wetland into the harvesting pond, where they will be captured for use as irrigation water for the neighbouring sports park.
- Four catchments have been identified as suitable for the application of riparian revegetation rather than conventional water quality treatment through bioretention. In these areas it is proposed to revegetate 4 m² of the existing riparian waterway for every 1 m² of bioretention filter media required to meet the SPP water quality objectives in the music model. The results of this modelling have determined a total of 7,320 m² of existing riparian water way shall be revegetated in order to achieve the SPP water quality treatment objectives for these catchments.
- The details of the riparian revegetation are to be developed, as part of the stormwater management plans for the relevant development stages.

Key stormwater quantity management design outcomes for the Site as follows:

- One detention basin is proposed for the Interim development scenario, to be located within the open space area within the Area 1 excise extent. The stored volume in the 100 year ARI event for this basin is 28,110 m³. This detention basin achieves mitigation of post development flows from development within Area 1.

- For the ultimate development scenario, a total of 6 detention basins are required to mitigate increased runoff from the remainder of the development. Basin 1 proposed as part of the interim development scenario will be filled, with this portion of the site now serviced by Basin 2.

The Master Plan has achieved the stormwater management objectives through the use of detention basins, bioretention, ephemeral wetland, regular wetland with stormwater harvesting, and riparian revegetation. The proposed treatment train will not only mitigate additional pollutant loads as results of the development, but will also significantly improve the existing stormwater water quality. This Masterplan is intended to set a stormwater management benchmark for future developments of this type.

6. REFERENCES

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2. Department of Energy and Water Supply, "Queensland Urban Drainage Design Manual", 2013 (Provisional);
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8. Logan City Council 2015, Logan City Council Planning Scheme
9. Healthy Waterways 2006, WSUD Technical Design Guidelines for South East Queensland.
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11. Water by Design 2012, Bioretention Technical Design Guidelines, Version 1.

APPENDIX A

Catchment Parameters

Table A1 Existing Scenario Subcatchment Parameters

Sub-catchment ID	Total Area [ha]	Catchment Slope [%]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]
G_1	11.58	3.90	0.06	0
	3.15	3.90	0.025	100
G_2	10.59	5.48	0.06	0
	0.00	5.48	0.025	100
G_3	9.98	6.00	0.06	0
	0.00	6.00	0.025	100
G_4	5.58	1.60	0.06	0
	0.00	1.60	0.025	100
G_5	10.41	4.71	0.06	0
	1.13	4.71	0.025	100
G_6	11.07	5.48	0.06	0
	0.00	5.48	0.025	100
G_7	10.53	3.26	0.06	0
	0.00	3.26	0.025	100
G_8	10.54	3.44	0.06	0
	1.52	3.44	0.025	100
G_9	12.61	3.88	0.06	0
	1.43	3.88	0.025	100
G_10	3.08	6.05	0.06	0
	0.00	6.05	0.025	100
G_11	13.40	4.41	0.06	0
	1.09	4.41	0.025	100

Sub-catchment ID	Total Area [ha]	Catchment Slope [%]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]
G_12	4.97	5.29	0.06	0
	0.00	5.29	0.025	100
G_13	5.97	3.91	0.06	0
	0.07	3.91	0.025	100
G_14	9.01	3.95	0.06	0
	0.08	3.95	0.025	100
G_15	13.94	3.14	0.06	0
	3.11	3.14	0.025	100
G_16	15.96	5.02	0.06	0
	0.95	5.02	0.025	100
G_17	22.06	2.34	0.06	0
	2.69	2.34	0.025	100
G_18	22.35	2.08	0.06	0
	3.73	2.08	0.025	100
G_19	13.00	3.02	0.06	0
	2.03	3.02	0.025	100
G_20	42.17	1.92	0.06	0
	6.14	1.92	0.025	100
G_21	46.61	1.08	0.06	0
	5.13	1.08	0.025	100
G_22	30.00	1.37	0.06	0
	0.57	1.37	0.025	100
G_23	30.91	1.98	0.06	0

Sub-catchment ID	Total Area [ha]	Catchment Slope [%]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]
	0.00	1.98	0.025	100
G_24	7.16	5.08	0.06	0
	0.13	5.08	0.025	100
G_25	16.88	1.80	0.06	0
	0.70	1.80	0.025	100
G_26	50.11	1.33	0.06	0
	4.38	1.33	0.025	100
G_27	12.87	4.60	0.06	0
	2.57	4.60	0.025	100
G_28	13.04	2.99	0.06	0
	1.75	2.99	0.025	100
G_29	19.80	2.29	0.06	0
	0.00	2.29	0.025	100
G_30	10.74	2.47	0.06	0
	1.24	2.47	0.025	100
G_31	5.99	2.73	0.06	0
	0.00	2.73	0.025	100
G_32	22.67	4.06	0.06	0
	0.00	4.06	0.025	100
G_33	5.35	2.80	0.06	0
	0.00	2.80	0.025	100
G_34	6.03	4.41	0.06	0
	0.00	4.41	0.025	100

Sub-catchment ID	Total Area [ha]	Catchment Slope [%]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]
G_35	4.63	1.60	0.06	0
	0.00	1.60	0.025	100
G_36	16.76	2.69	0.06	0
	0.00	2.69	0.025	100
G_37	14.18	1.72	0.06	0
	0.00	1.72	0.025	100
G_38	7.11	4.45	0.06	0
	0.00	4.45	0.025	100
G_39	6.74	2.86	0.06	0
	0.00	2.86	0.025	100
G_40	9.36	3.53	0.06	0
	0.00	3.53	0.025	100
G_41	31.04	1.40	0.06	0
	0.00	1.40	0.025	100
G_42	12.50	5.32	0.06	0
	0.42	5.32	0.025	100
G_43	29.18	2.18	0.06	0
	4.80	2.18	0.025	100
G_44	12.66	1.90	0.06	0
	2.68	1.90	0.025	100
G_45	69.13	2.34	0.06	0
	10.77	2.34	0.025	100
G_46	15.13	2.92	0.06	0

Sub-catchment ID	Total Area [ha]	Catchment Slope [%]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]
	0.00	2.92	0.025	100
G_47	9.75	3.86	0.06	0
	0.03	3.86	0.025	100
G_48	29.08	2.42	0.06	0
	0.04	2.42	0.025	100
G_49	11.69	4.80	0.06	0
	0.00	4.80	0.025	100
G_50	46.20	2.69	0.06	0
	6.54	2.69	0.025	100
G_51	29.97	2.04	0.06	0
	5.40	2.04	0.025	100
G_52	22.82	1.61	0.06	0
	5.46	1.61	0.025	100
G_53	13.26	5.32	0.06	0
	0.12	5.32	0.025	100
G_54	18.79	1.35	0.06	0
	0.00	1.35	0.025	100
G_55	13.01	2.87	0.06	0
	1.02	2.87	0.025	100
G_56	20.59	2.16	0.06	0
	2.97	2.16	0.025	100
G10_a	1.87	5.33	0.06	0
	0.06	5.33	0.025	100

Table A2 Post-Development Scenario (Area 1 Development) Subcatchment Parameters

Sub-catchment ID	Total Area [ha]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]	Catchment Slope [%]
G_1	4.5	2.66	0.046	0
	9.07	2.66	0.025	100
G_2	5.22	4.31	0.044	0
	5.18	4.31	0.025	100
G_3	3.9	6	0.048	0
	2.81	6	0.025	100
G_4	0.36	1.6	0.05	0
	3.31	1.6	0.025	100
G_5	11.26	4.71	0.06	0
	1.36	4.71	0.025	100
G_6	10.15	5.48	0.06	0
G_7	10.53	3.26	0.06	0
G_8	10.54	3.44	0.06	0
	1.52	3.44	0.025	100
G_9	12.61	3.88	0.06	0
	1.43	3.88	0.025	100
G_10	1.48	1.83	0.043	0
	4.7	1.83	0.025	100
G_11	5.37	3.99	0.046	0
	7.86	3.99	0.025	100
G_12	3.94	3.66	0.05	0
	1.76	3.66	0.025	100

Sub-catchment ID	Total Area [ha]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]	Catchment Slope [%]
G_13	5.59	3.91	0.059	0
	0.29	3.91	0.025	100
G_14	8.9	3.95	0.06	0
	0.08	3.95	0.025	100
G_15	13.94	3.14	0.06	0
	3.11	3.14	0.025	100
G_16	15.96	5.02	0.06	0
	0.95	5.02	0.025	100
G_17	22.06	2.34	0.06	0
	2.69	2.34	0.025	100
G_18	22.35	2.08	0.06	0
	3.73	2.08	0.025	100
G_19	13	3.02	0.06	0
	2.03	3.02	0.025	100
G_20	42.17	1.92	0.06	0
	6.14	1.92	0.025	100
G_21	46.61	1.08	0.06	0
	5.13	1.08	0.025	100
G_22	30	1.37	0.06	0
	0.57	1.37	0.025	100
G_23	30.91	1.98	0.06	0
G_24	7.16	5.08	0.06	0
	0.13	5.08	0.025	100

Sub-catchment ID	Total Area [ha]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]	Catchment Slope [%]
G_25	16.88	1.8	0.06	0
	0.7	1.8	0.025	100
G_26	50.11	1.33	0.06	0
	4.38	1.33	0.025	100
G_27	12.87	4.6	0.06	0
	2.57	4.6	0.025	100
G_28	13.04	2.99	0.06	0
	1.75	2.99	0.025	100
G_29	19.8	2.29	0.06	0
G_30	10.74	2.47	0.06	0
	1.24	2.47	0.025	100
G_31	5.99	2.73	0.06	0
G_32	23.79	4.06	0.06	0
G_33	5.35	2.8	0.06	0
G_12A	7.02	2.85	0.058	0
	0.24	2.85	0.025	100
G_35	2.89	2.85	0.05	0
	0.49	2.85	0.025	100
G_36	16.75	2.69	0.06	0
G_37	14.18	1.72	0.06	0
G_38	7.11	4.45	0.06	0
G_39	6.74	2.86	0.06	0
G_40	9.36	3.53	0.06	0

Sub-catchment ID	Total Area [ha]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]	Catchment Slope [%]
G_41	31.04	1.4	0.06	0
G_42	12.5	5.32	0.06	0
	0.42	5.32	0.025	100
G_43	29.18	2.18	0.06	0
	4.8	2.18	0.025	100
G_44	12.66	1.9	0.06	0
	2.68	1.9	0.025	100
G_45	69.13	2.34	0.06	0
	10.77	2.34	0.025	100
G_46	15.13	2.92	0.06	0
G_47	9.75	3.86	0.06	0
	0.03	3.86	0.025	100
G_48	29.08	2.42	0.06	0
	0.04	2.42	0.025	100
G_49	11.69	4.8	0.06	0
G_50	46.2	2.69	0.06	0
	6.54	2.69	0.025	100
G_51	29.97	2.04	0.06	0
	5.4	2.04	0.025	100
G_52	22.82	1.61	0.06	0
	5.46	1.61	0.025	100
G_53	13.26	5.32	0.06	0
	0.12	5.32	0.025	100

Sub-catchment ID	Total Area [ha]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]	Catchment Slope [%]
G_54	20.2	1.35	0.06	0
G_55	13.01	2.87	0.06	0
	1.02	2.87	0.025	100
G_56	20.59	2.16	0.06	0
	2.97	2.16	0.025	100
G_54A	4.62	4.07	0.06	0
	0.001	4.07	0.025	0

Table A3 Post-Development Scenario (Ultimate Development) Subcatchment Parameters

Sub-catchment ID	Total Area [ha]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]	Catchment Slope [%]
G_1	4.5	0.045	0	2.66
	9.07	0.025	100	2.66
G_2	3.78	0.04	0	4.31
	6.61	0.025	100	4.31
G_3	1.7	0.04	0	6
	5	0.025	100	6
G_4	0.36	0.05	0	1.6
	3.31	0.025	100	1.6
G_5	6.19	0.043	0	3.78
	8.91	0.025	100	3.78
G_6	2.93	0.041	0	3.91
	4.17	0.025	100	3.91

Sub-catchment ID	Total Area [ha]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]	Catchment Slope [%]
G_7	1.2	0.041	0	4.04
	3.27	0.025	100	4.04
G_8	8.85	0.057	0	3.44
	1.78	0.025	100	3.44
G_9	12.61	0.06	0	3.88
	1.43	0.025	100	3.88
G_10	1.47	0.043	0	1.83
	4.71	0.025	100	1.83
G_11	3.56	0.043	0	3.99
	9.68	0.025	100	3.99
G_12	2.26	0.043	0	3.66
	3.43	0.025	100	3.66
G_14	2.6	0.04	0	4.01
	10.18	0.025	100	4.01
G_15	11.67	0.056	0	3.14
	3.33	0.025	100	3.14
G_16	11.14	0.051	0	5.02
	1.47	0.025	100	5.02
G_17	20.25	0.055	0	2.61
	3.29	0.025	100	2.61
G_18	22.35	0.06	0	2.08
	3.73	0.025	100	2.08
G_19	13	0.06	0	3.02

Sub-catchment ID	Total Area [ha]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]	Catchment Slope [%]
	2.03	0.025	100	3.02
G_20	42.17	0.06	0	1.92
	6.14	0.025	100	1.92
G_21	46.61	0.06	0	1.08
	5.13	0.025	100	1.08
G_22	28.03	0.058	0	1.37
	2.29	0.025	100	1.37
G_23	9.84	0.04	0	1.97
	24.97	0.025	100	1.97
G_24	7.28	0.045	0	5.08
	0.78	0.025	100	5.08
G_25	1.15	0.041	0	2.76
	2.46	0.025	100	2.76
G_26	50.1	0.06	0	1.33
	4.39	0.025	100	1.33
G_27	12.87	0.06	0	4.6
	2.57	0.025	100	4.6
G_28	13.04	0.06	0	2.99
	1.75	0.025	100	2.99
G_29	13.61	0.06	0	2.29
	2.32	0.025	100	2.29
G_30	10.72	0.06	0	2.47
	1.26	0.025	100	2.47

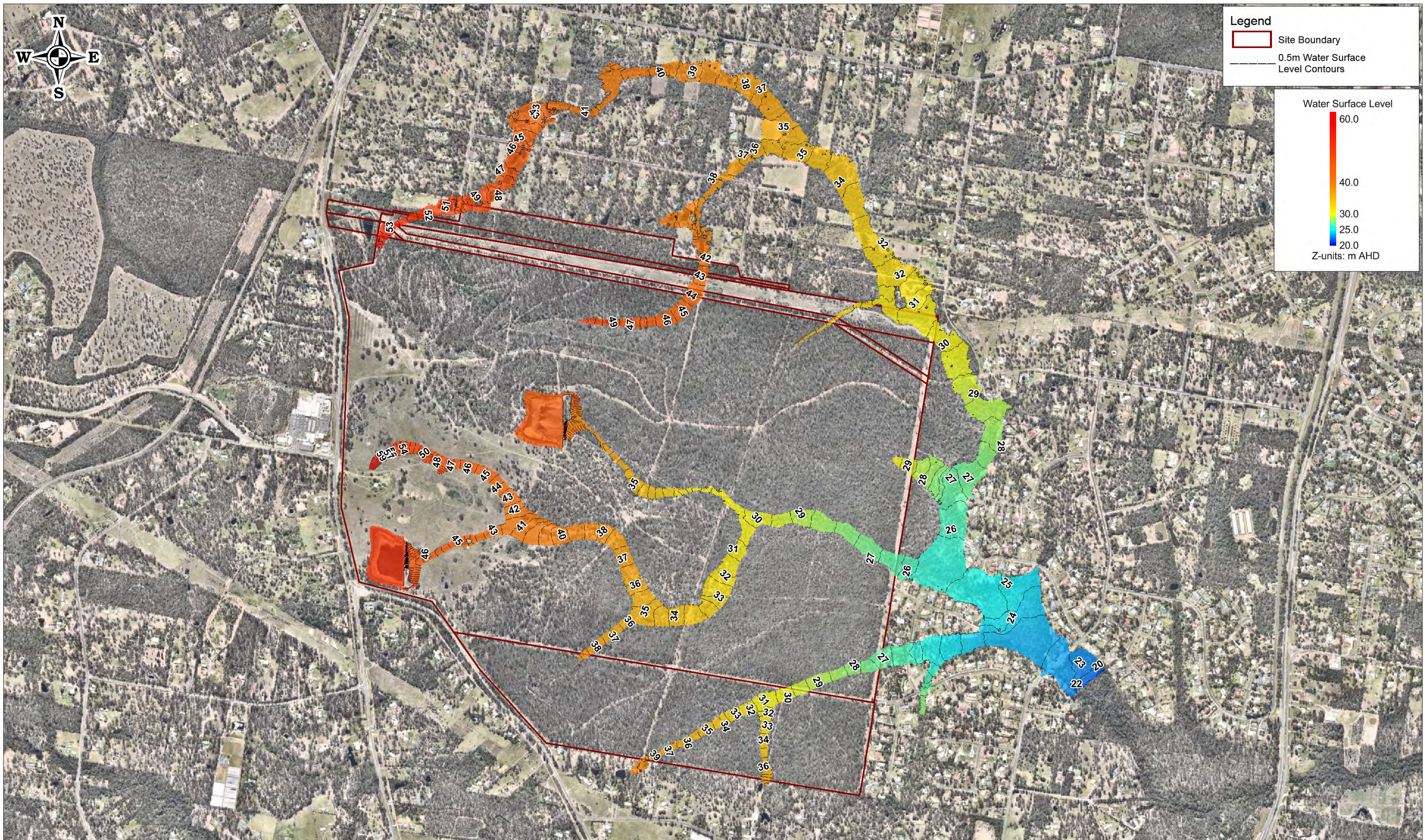
Sub-catchment ID	Total Area [ha]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]	Catchment Slope [%]
G_31	5.96	0.06	0	2.73
	0.02	0.025	100	2.73
G_32	8.08	0.042	0	1.89
	13.41	0.025	100	1.89
G_33	2.49	0.05	0	1.07
	4.66	0.025	100	1.07
G_35	2.9	0.05	0	2.85
	0.49	0.025	100	2.85
G_36	9.58	0.048	0	2.69
	7.65	0.025	100	2.69
G_37	10.38	0.054	0	1.72
	3.8	0.025	100	1.72
G_38	2.24	0.053	0	4.31
	1.44	0.025	100	4.31
G_39	4.69	0.056	0	2.86
	0.39	0.025	100	2.86
G_40	4.38	0.043	0	3.53
	7.22	0.025	100	3.53
G_41	30.07	0.06	0	1.4
	0.97	0.025	100	1.4
G_42	8.59	0.047	0	5.32
	5.77	0.025	100	5.32
G_43	29.06	0.06	0	2.18

Sub-catchment ID	Total Area [ha]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]	Catchment Slope [%]
	4.92	0.025	100	2.18
G_44	12.58	0.06	0	1.9
	2.77	0.025	100	1.9
G_45	69.13	0.06	0	2.34
	10.77	0.025	100	2.34
G_46	2.2	0.04	0	2.83
	6.61	0.025	100	2.83
G_47	8.98	0.06	0	3.86
	0.8	0.025	100	3.86
G_48	26.77	0.06	0	2.42
	2.36	0.025	100	2.42
G_49	6.41	0.04	0	2.96
	17.09	0.025	100	2.96
G_50	46.18	0.06	0	2.69
	6.56	0.025	100	2.69
G_51	29.97	0.06	0	2.04
	5.4	0.025	100	2.04
G_52	22.82	0.06	0	1.61
	5.46	0.025	100	1.61
G_53	12.28	0.06	0	5.32
	1.1	0.025	100	5.32
G_54	11.91	0.046	0	0.76
	7.52	0.025	100	0.76

Sub-catchment ID	Total Area [ha]	Catchment Manning's 'n' [n value]	Percentage Impervious [%]	Catchment Slope [%]
G_55	13.27	0.054	0	2.87
	1.56	0.025	100	2.87
G_56	20.59	0.06	0	2.16
	2.97	0.025	100	2.16
G_13	0.93	0.041	0	2.25
	2.82	0.025	100	2.25
G_25a	12.74	0.048	0	1.99
	1.65	0.025	100	1.99
G_12A	5.73	0.048	0	2.85
	1.54	0.025	100	2.85
G_54A	1.11	0.04	0	5.96
	3.32	0.025	100	5.96
G_7A	1.72	0.041	0	2.28
	3.74	0.025	100	2.28
G_29A	0.87	0.04	0	2.5
	2.59	0.025	100	2.5

APPENDIX B

Existing Flood Extent Mapping



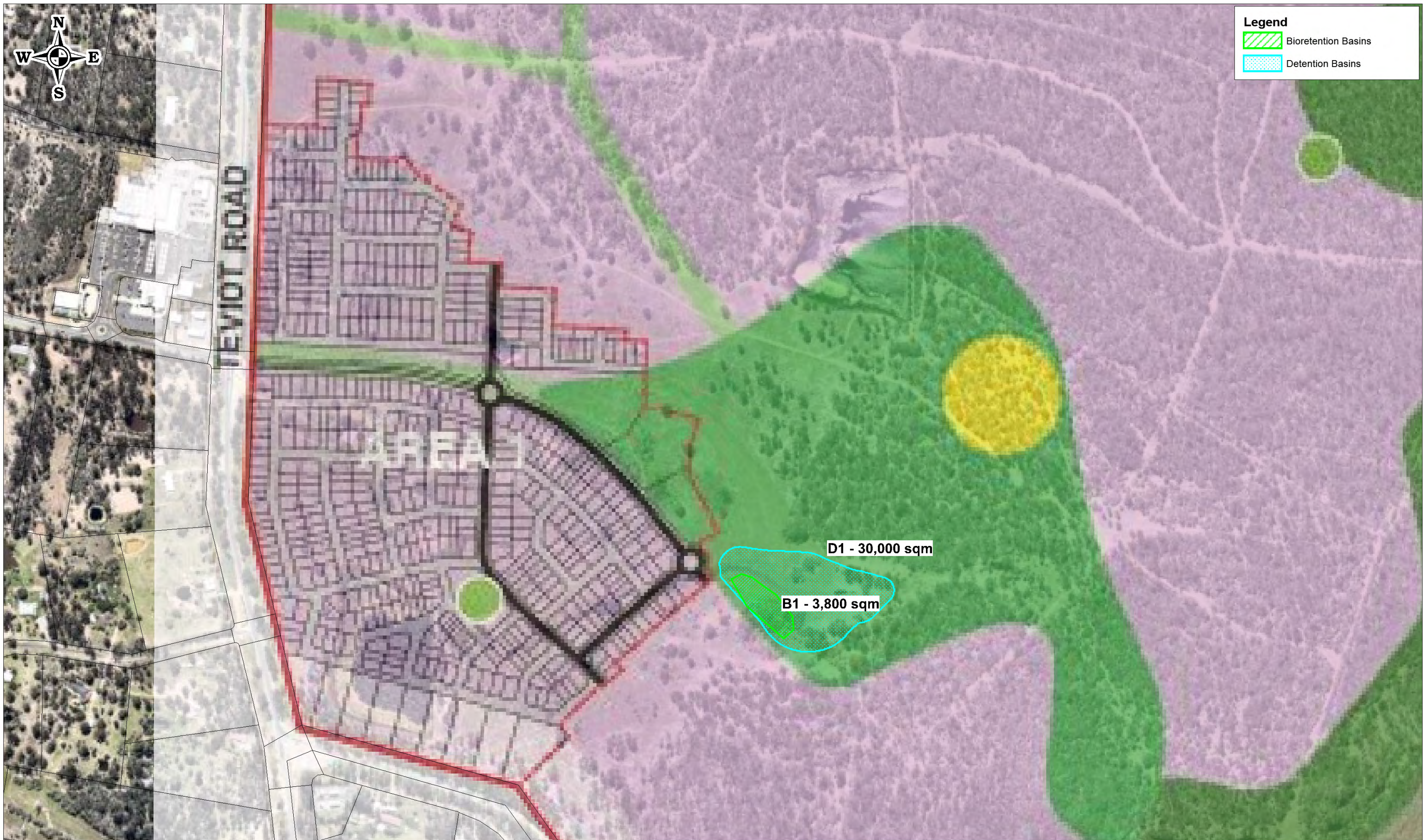
Greenbank Stormwater Management

Q100 Existing Flood Extent

Figure B1

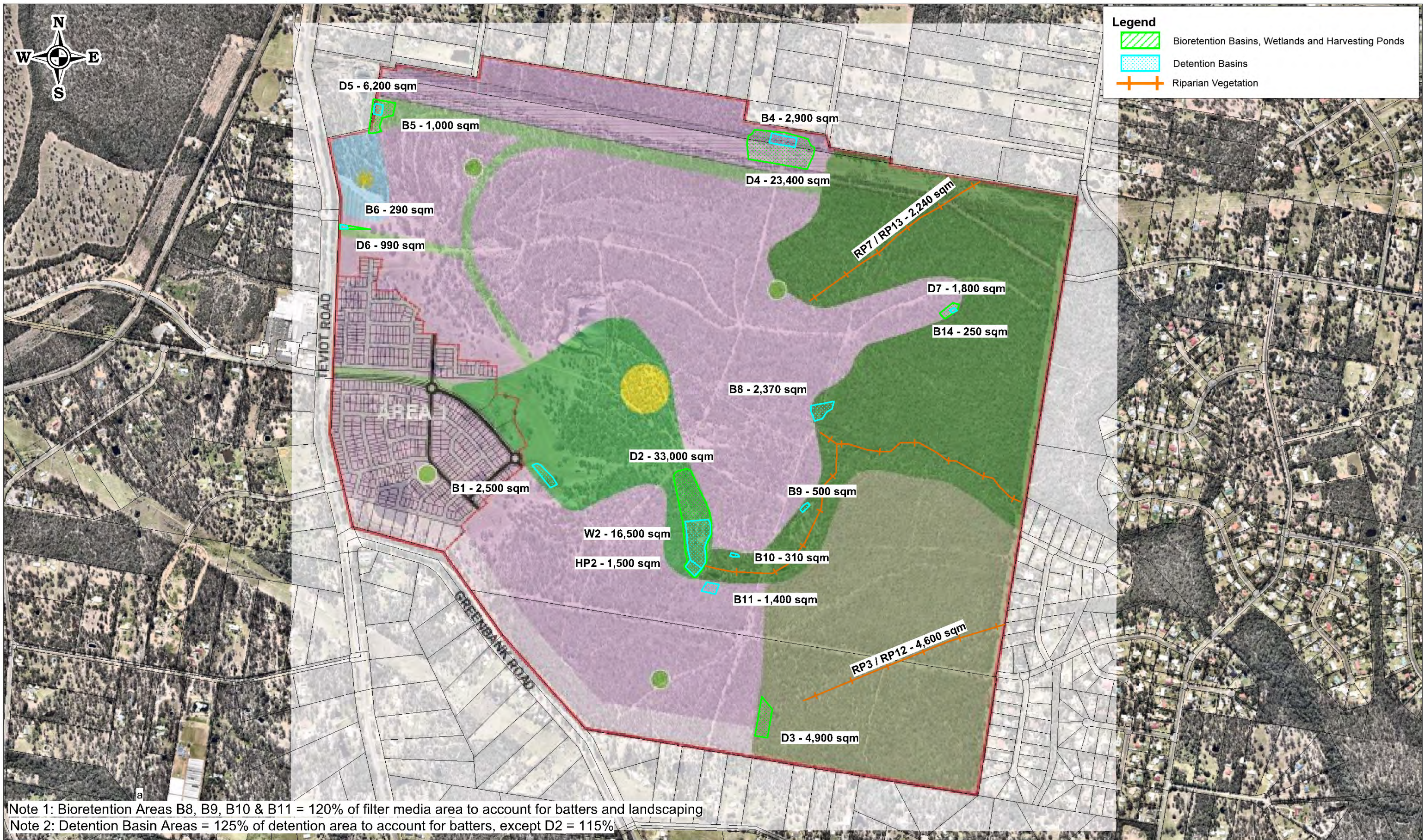
APPENDIX C

Concept Layout Plan Interim Development Scenario



APPENDIX D

Concept Layout Plan Ultimate Development Scenario



APPENDIX E

Design Standards and Outcomes

APPENDIX D – Design Outcomes & Design Standards / Resources

Element	Design Outcome
Lawful Point of Discharge	The point of discharge for outlet works is considered lawful where discharge is to waterways, drainage channels, or drainage features within; ▪ The balance land of the subject development site; ▪ Land to be dedicated / transferred to Council, or otherwise to be under the control of Council; ▪ Reconfigured allotments within the subject development site where access to the discharge infrastructure is secured by an easement (if relevant); or ▪ Private allotments outside the subject development site where access to the discharge infrastructure is secured by an easement (if relevant). The point of discharge from the development site is considered lawful where discharge is to existing waterways, drainage channels, or drainage features at the site boundary
No Worsening	No worsening is defined as; ▪ No increase in peak discharge from the development boundary for the 1:2, 1:5, 1:10, 1:20, 1:50 & 1:100 AEP flood events. ▪ No increase in flood levels external to the development boundary for the 1:2, 1:5, 1:10, 1:20, 1:50 & 1:100 AEP flood events. ▪ No change to timing of peak flow without appropriate runoff routing modelling for the 1:2, 1:5, 1:10, 1:20, 1:50 & 1:100 AEP flood events. ▪ No increase in frequency and duration of discharge for the minor storm (i.e. 1:1 year AEP flood event) ▪ No reduction to capacity of external drainage system When assessing stormwater quantity, the point of measurement is defined as the point of discharge from the development site rather than individual points of discharge from balance parcels.
Owners Consent	▪ Downstream property owners consent is not required if the requirements for Lawful Point of Discharge and No Worsening are met (where No Worsening is measured at the boundary of the development site and at Greenhill Rd).
Minimum Habitable Floor Level	▪ The minimum habitable floor level for buildings (not gazebos or structures open on 3 or more sides) is to be the 1:100 AEP + 500mm
Flood Immunity - Essential Service Infrastructure	▪ Essential service infrastructure likely to fail when inundated will be set above the 1:100 AEP flood level.
Flood Immunity - Roads	▪ Depth Velocity products in accordance with QUDM. ▪ Maximum depth over roads of 300mm in 1:100 AEP event.
Quantity Modelling Sensitivity Assumptions	▪ Variations of 10% for rainfall intensity, Manning's roughness and tailwater. ▪ QUDM blockages adopted for hydraulic structures

Quantity Modelling Event Assumptions	- Masterplan applications are to be assessed against a 1:5, 1:20 & 1:100 AEP flood event - Individual stormwater management plans are to be assessed against a 1:2, 1:5, 1:10, 1:20, 1:50 & 1:100 AEP flood events for the critical storm duration.
Infrastructure design	- Stormwater assets in parkland or other public land must be designed to not look like a hydraulic structure but rather it must have special character. This will involve using variable slopes, retaining upstream gullies, camouflaging inlets and outlet structures and similar (a rectangular or geometrically shaped basin is generally undesirable). - Safe egress and adequate signage is to be considered in accordance with a risk based approach. - Overland flow paths are to be designed to convey the 1:100 AEP flood event. - Stormwater detention basins / bioretention basins and the like are to be sited on land which is to be dedicated / transferred to Council, or otherwise under the control of Council.
Stormwater detention	- Stormwater detention may be required to manage stormwater discharge by limiting post development peak flows to the pre development flow rates for all design storm events. - Stormwater detention may be achieved using road embankments or online storage in areas outside of the conservation area. - Stormwater detention system are to be free draining.
Water Quality Modelling	- Water quality modelling to be undertaken in accordance with the Water by Design MUSIC modelling guidelines. - Waterway rehabilitation and stabilisation is acceptable as an offset to traditional WSUD measures. - SPP water quality objectives adopted where traditional WSUD measures are proposed.
Design	- Stormwater management systems used within waterway buffers must be vegetated stormwater management systems (earth and vegetation) which provide stormwater management and ecological function. - Design of channel works as part of development maximises the use of natural channel design principles where possible.
Notes: - Development outcomes are acceptable where in accordance with the design outcomes above, or part thereof. - Alternate design outcomes may be proposed and deemed acceptable.	

Design Standards / Resources

Queensland Urban Drainage Manual 2013 (provisional)

Logan City Council Planning Scheme Policy 5 – Infrastructure 2015.

Queensland Environmental Protection (Water) Policy 2009

Queensland State Planning Policy July 2014

Water by Design: Concept Guidelines for WSUD 2009

Water by Design: WSUD Technical Design Guidelines in SEQ 2006
Water by Design: Bioretention Technical Design Guidelines 2012
Water by Design: Music Modelling Guidelines 2011
Water by Design: Framework for the Integration of Flooding and Stormwater Management into Open Space 2016
Queensland Urban Drainage Manual 2013 (provisional)
Australian Rainfall and Runoff (ARR) 1987/1999
Brisbane City Council Natural Channel Design Guideline 2003
<i>Notes:</i> - <i>The Design Standards / Resources above are considered acceptable Standards / Resources.</i> - <i>Development outcomes in accordance with the Design Standards / Resources, or part thereof, are considered acceptable outcomes.</i> - <i>Alternate Standards / Resources may be proposed and deemed acceptable.</i>

APPENDIX F

Bioretention Filter Media – Data Sheet

PRODUCT DATA SHEET

This is a typical analysis of material produced and does not constitute as a Product Certification for material supply

Product: River Sands Cert BioMedia (V1)

BioRetention Filter Media

Manufactured to Meet:

- The Facility for Advance Water Biofiltration Guidelines (V 3.01 June 2009)
- The Healthy Waterways / Water By Design Specifications (V1, October 2012)

Main Applications:

BioRetention Filter Media for BioRetention Systems to be used with BioRetention Transition Layer and Drainage Layer to assist in removing pollutants and supporting vegetation.

Indicative Grade Analysis:

Sieve Standards: ASTM C136, AS1141.11.1

Sampling Methods: ASTM D75, AS1141.3.1

Note:

Indicative grading only. Grading analysis will vary over time due to changes in raw feed.

Technical Properties:

Analysis	FAWB Specification	Result
Organic Matter (%)	>3-10 (Ideally 5-10)	6.8 %
Saturated Hydraulic Conductivity (Ksat) (mm/hr)	100-600mm	150 – 300 mm/hr
Electrical Conductivity (1:5) (dS/m)	<1.2	0.30 dS/m
pH (1:5) in H2O (pH units)	5.5-7.5	6.2
Total Nitrogen (mg/kg)	< 1000 mg/kg	600.0 mg / kg
Phosphate as PO4 (mg/kg)	<80 mg / kg	3.0 mg / kg

Grading Properties:

The below attached Particle Size Distribution (PSD) is a recommended range and is of secondary importance compared with saturated hydraulic conductivity.

Size (mm)	FAWB Indicative PSD	Result
<0.05	2-6%	3.3%
0.05-0.15	5-30%	8.7%
0.15-0.25	10-30%	32.1%
0.25-1.0	40-60%	55.5%
1.0-2.0	7-10%	0.3%
2.0-3.35	<3.0%	0.1%
>3.35		0.1%
D85 (Filter Media Layer)	AS 1141.11.1	0.8mm
5 x D85 (Filter Media Layer)	AS 1141.11.1	4.0mm
Fines <0.002mm	< 2 %	✓

To meet 'bridging criteria' the smallest 15% of the sand particles in the transition layer should bridge with the largest 15% of the filter media layer. As per section 4.3.2. Transition Layer (Where using alternate materials please check bridging criteria before use.)

D15 (Transition Layer) <= 5 x D85 (Filter Media Layer)	0.26-0.41mm (D15 River Sands BioTrans 1mm & BioTrans) <= 4.0mm (5 x D85 River Sands BioMedia)	✓
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River Sands operates and maintains a Quality Certified system in accordance with AS/NZS ISO 9001:2008.

The Quality system is third party certified by BSI/NCSI.

River Sands also maintains a registered NATA laboratory.

Registration # 6898

Registration # 10470

Testing results achieved at time of manufacture 28.06.13:3AHS,13/26751

Better Products, Outstanding Results



SERVICE CENTRES

Geebung	Phone 07 3865 6888	Fax 07 3865 8777
Gold Coast	Phone 07 5597 0423	Fax 07 5597 0424
Sunshine Coast	Phone 07 5438 8900	Fax 07 5438 8899
Sydney	Phone 02 9677 1056	Fax 02 9677 1352
Melbourne	Phone 03 9311 9225	Fax 03 9311 9445