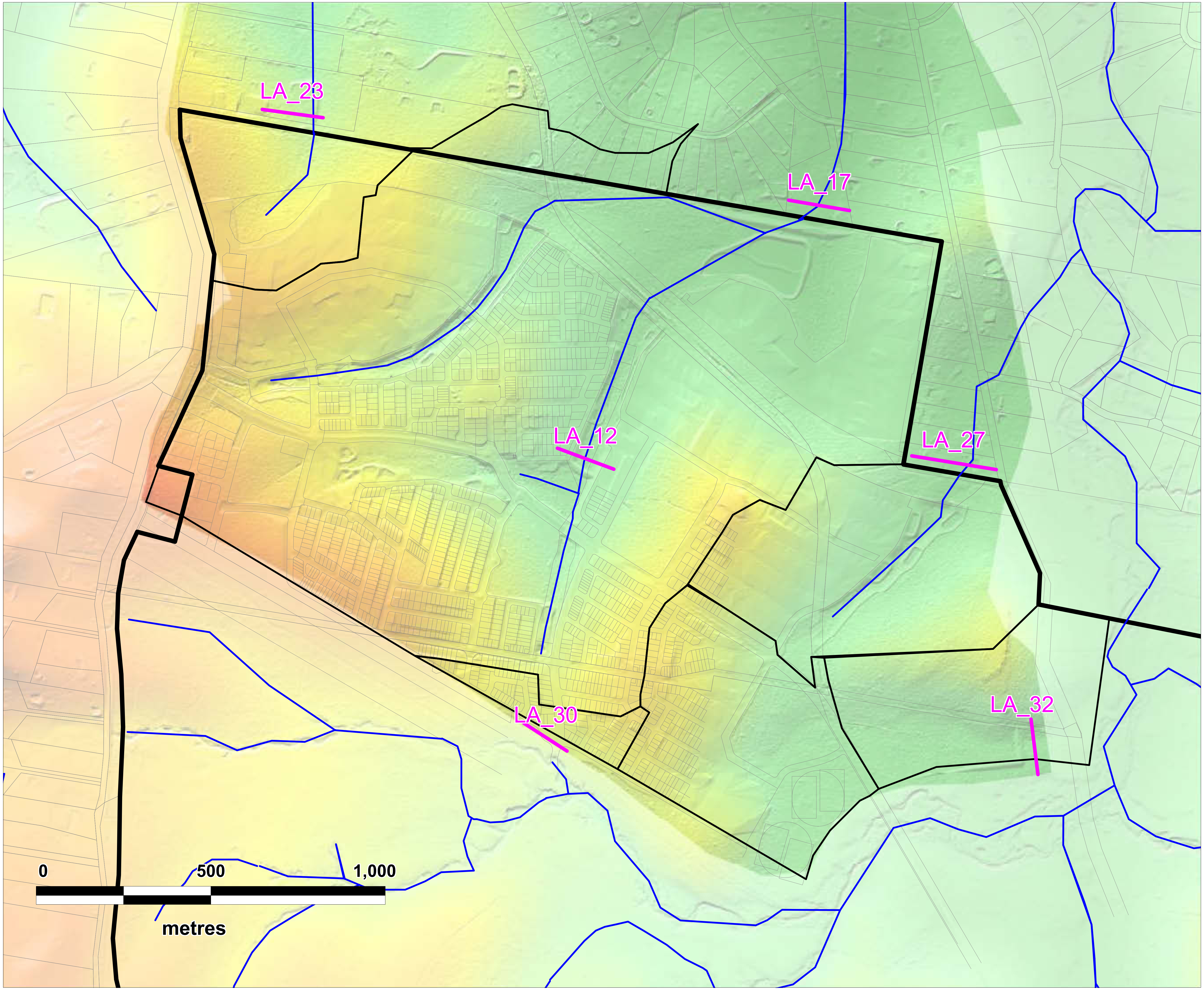


DesignFlow

LEGEND

- Yarrabilba site boundary
- Flow reporting location
- Developed catchment
- Waterway drainage line

FIGURE 19
Flow reporting locations

Yarrabilba Precinct 1 and 2 Stormwater Management Plan

SCALE:
1 : 1000

Date:
1 November 2013

Client:
Lend Lease

Generally, the Rational Method showed over-estimating peak pre-development flows for the smaller 1 and 2 year ARI events with better agreement for the larger ARIs. This is expected as the average velocity assumptions for the natural channel flow time (and therefore time of concentration) have been derived directly for the 100 year ARI event hydraulic model. Under minor events, the flow depth and therefore velocity through the natural channel can be expected to be lower leading to increased time of concentration for minor ARIs (and lower flows). The derivation of differing time of concentration for every ARI was not undertaken as this comparison was intended as a broad evaluation only.

The XPSWMM model can therefore be seen to show satisfactory agreement with the Rational Method for the mid to high ARIs under existing conditions. Under the very minor ARIs, XPSWMM results in more conservative peak flows which was considered appropriate for this assessment.

The developed case (unmitigated) shows satisfactory agreement of peak flows between XPSWMM and Rational Method across all ARIs at all locations.

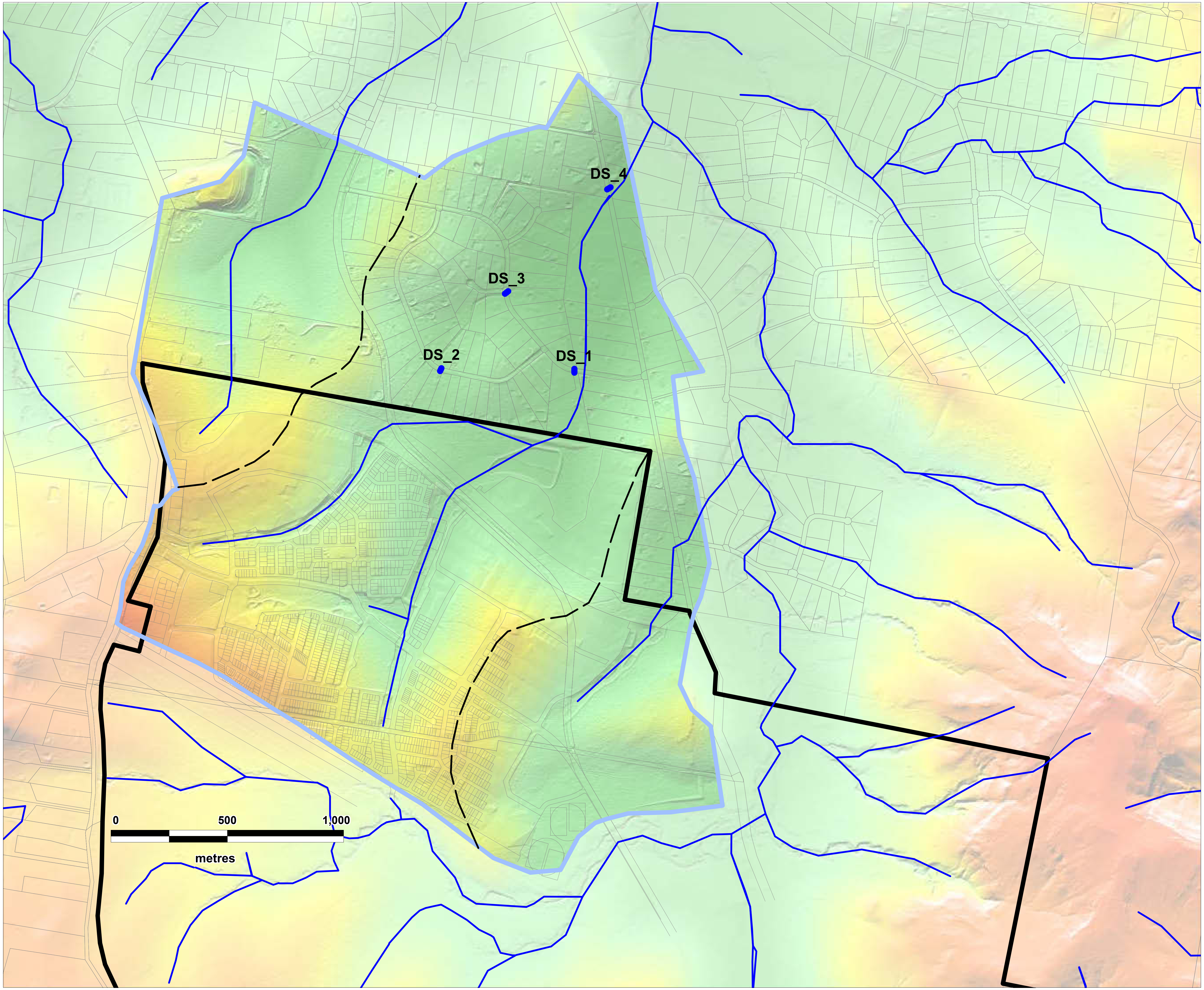
8.6 HYDRAULIC MODELLING

A hydraulic assessment of the site was undertaken using Tuflow (1D/2D linked hydrodynamic model) to determine the flood depths and extent within the existing and developed site (including detention basins). Results from the Tuflow model were compared against the 1D hydraulic module of XPSWMM which was used to verify the pre and post development results at key locations.

8.6.1 Model Terrain and Extent

The terrain model used in the analysis was derived from the Lidar data (supplied by RPS) for the existing site and the combined developed earthworks design. The DEM were derived on a 1.0m grid to allow detailed hydraulic assessment within natural channels and proposed structures. Additional earthworks design DEMs were prepared by DesignFlow for the proposed stormwater treatment measures (predominantly bioretention basins, sediment ponds and wetlands) and overlaid on the developed case DEM to ensure accurate representation of the stage-storage relationship at propose detention basins.

A broader scale 2.0m grid DEM from Logan City Council was used to extend the site DEM further downstream to Quinzeh Creek, in order to test for back water affects from the waterway back upstream to Yarrabilba. Overlapping areas of the two DEMs were compared for consistency, and it was found that the two DEMs aligned very well in the vertical plane, and there was no significant elevation offset between the datasets.



DesignFlow

LEGEND

- Yarrabilba site boundary
- Tuflow model boundary
- Ridge line
- Existing culvert

FIGURE 20
TUFLOW model terrain and extent

Yarrabilba Precinct 1 and 2
Stormwater Management Plan

SCALE:
1 : 1500

Date:
1 November 2013

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8.6.2 Channel Roughness

An estimate of the roughness through the main open channel flow paths was undertaken using the Modified Cowan method and compared with photographic examples in Appendix C of the *Natural Channel Design Guidelines* (BCC, 2003). Assignment of vegetation groups across the drainage lines was undertaken from review of aerial photography and on-ground visual inspection/verification of the various reaches. For the purpose of this site, the Manning's n values in Table 23 have been derived for the range of open channel conditions observed onsite.

Table 23: Roughness Guidance

Description	Manning's n
Predominantly open grass flow path with minimal trees and brush	0.05
Grass, tree seedlings and light brush, with mature trees (typically regrowth pine) scattered through flow path	0.06
Native undergrowth, low to moderate density trees and some natural debris. Typically broad, shallow flow path profile.	0.07

For the purpose of XP-SWMM hydraulic verification model, the natural channels were represented by 1D links with typical cross sections and slopes derived directly from the DEM. Roughness values were applied to each reach in accordance with the Manning's used on the Tuflow model.

8.6.3 Hydraulic Structures

All hydraulic structures are modelled in Tuflow as 1D structures embedded within the 2D model domain. The existing structures identified in Table 24 were included in the early modelling to ensure the downstream tail water effects could be accurately assessed.

The following culverts downstream of Yarrabilba were inspected and field measurements taken on 16 April 2013. Invert levels were determined by subtracting the culvert depth plus cover from the road crown level (based on DEM value).

Table 24: Existing cross road drainage downstream of site

ID	Location	Type and Size (mm)	US : DS Invert (m AHD)	Status
DS_1	Georgina Dr East	8 x 900 RCP	16.81 : 16.71	Existing
DS_2	Culgoa Crescent	3 x 450 RCP	24.16 : 24.16	Existing
DS_3	Georgina Dr West	4 x 1200 x 900 RCBC	16.71 : 16.61	Existing
DS_4	Steele Rd	3 x 375 RCP	14.93 : 14.83	Existing

The developed case structures were derived from the discharge control requirements for flood detention basins and/or the conveyance of flows under roads. The hydraulic performance of these structures in TUFLOW has been verified against the XP-SWMM 1D hydraulic model to allow rapid testing of configurations ultimately used in the Tuflow model. This is discussed further in Section 8.7.

8.6.4 Inflow hydrographs

Sub-catchment hydrographs were extracted from the XP-SWMM model (runoff mode) as inputs for the TUFLOW flood model. These hydrographs were applied to the lower portion of the sub-catchment channel reach to reflect the discharge hydrograph and location of each sub-catchment once the urban drainage system are constructed. Hydraulic routing and accumulation of sub-catchment inflow hydrographs thus occurred progressively down the catchment/channel.

8.6.5 Boundary Conditions

A series of tests were carried out on the major central catchment (Catchment 2) to determine an appropriate location for the downstream boundary of the flood model, and also to determine any impacts of coincident flooding of Precinct 1 and 2 from Quinzeh Creek. The Precinct 1 and 2 TUFLOW model was extended down to Quinzeh Creek and a number of boundary condition water levels used.

The *Revised Quinzeh Creek Flood Study Letter* (Engeny, 23 August 2013) found flood levels are in the order to 16.5m AHD in the Steele Road area just upstream of the confluence with Quinzeh Creek. This level was initially used as a tailwater level in the TUFLOW model and then adjusted upwards to see if this influenced levels in Precinct 1 and 2. The flood level at the boundary of Precinct 1 and 2 is 21.0 m AHD Chainage 1250 in Figure 21,). The red line represents the DEM surface, and the green line represents the 100 year ARI flood surface. This is well above the flood levels close to Quinzeh Creek of 16.5m AHD (Chainage 0.0).

Therefore, flooding along Quinzeh Creek is not a factor contributing to flood discharge and level estimations within and from the Precinct 1 and 2 catchments.

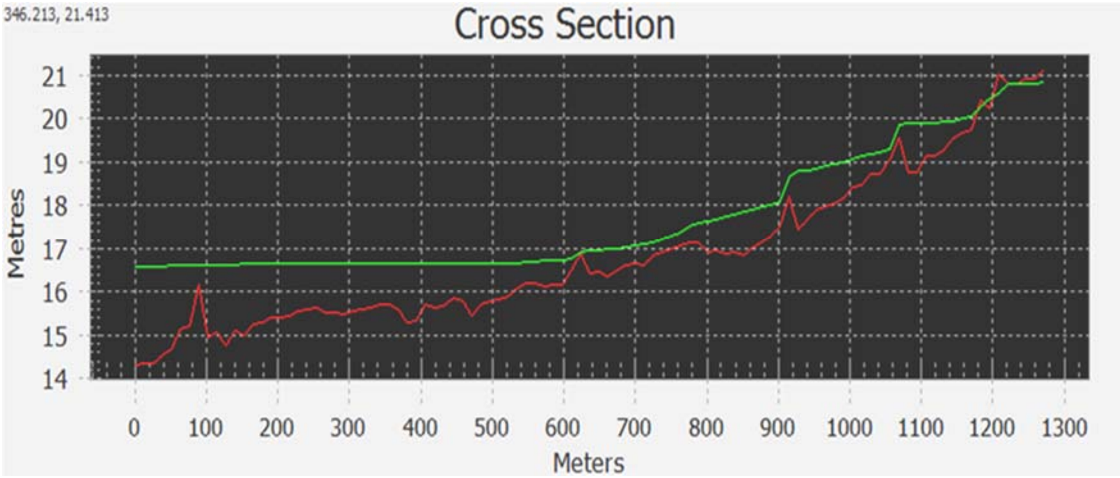
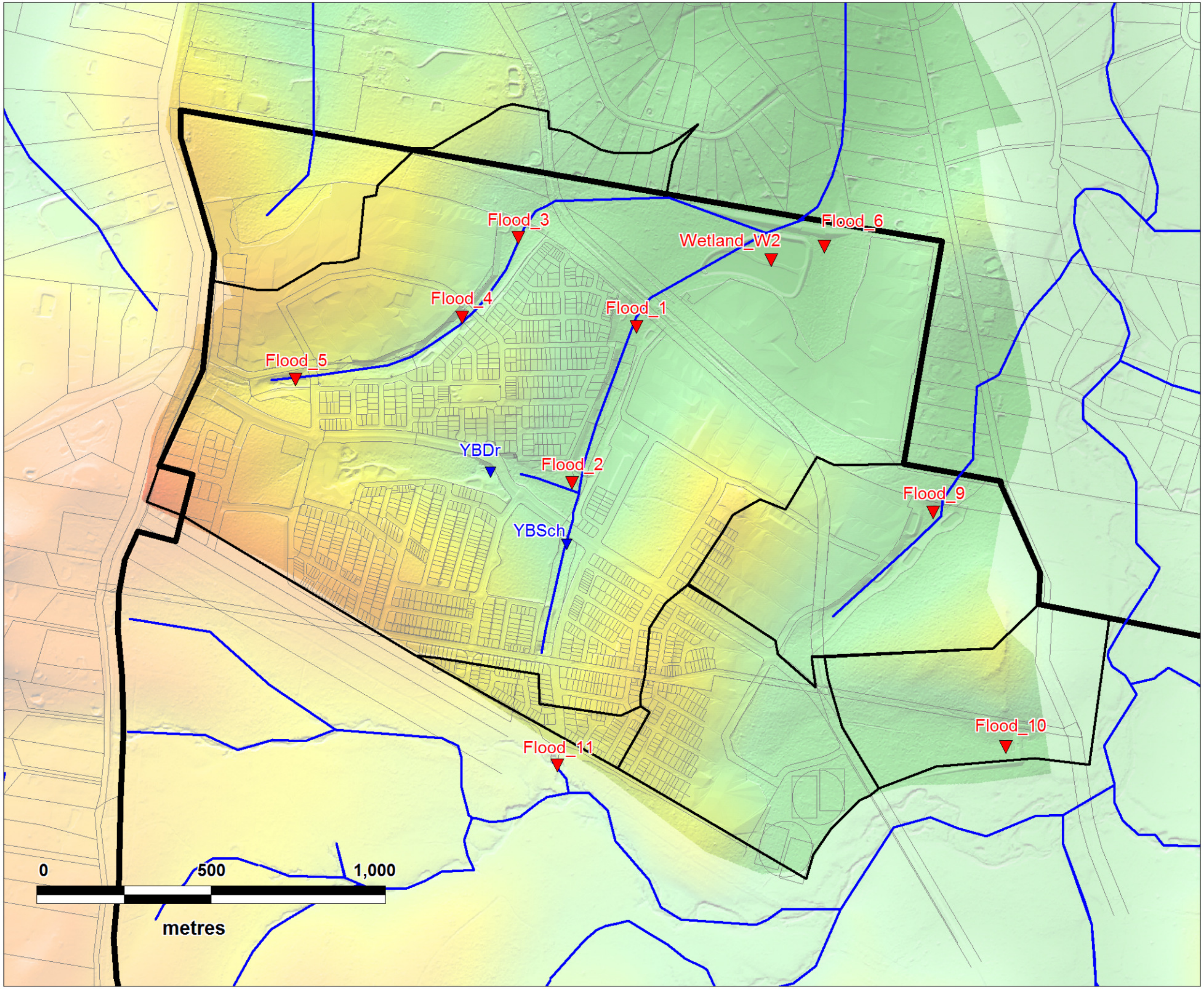


Figure 21: Yarrabilba downstream drainage line long section from Quinzeh Creek

Tailwater levels were established for each catchment from Precinct 1 and 2 from the *Revised Quinzeh Creek Flood Study Letter* (Engeny, 23 August 2013). As illustrated in Figure 20 the TUFLOW model was extended downstream to ensure tailwater conditions did not influence flood levels within Yarrabilba or directly downstream.

8.7 FLOOD DETENTION BASIN DETAILS

Post development flow attenuation is achieved via a number flood detention basins created upstream of road embankments, detention bunds across drainage lines, or interim locations formed by bunding future use areas (Flood Basins 10 and 11) until future precinct flood basins are implemented. Discharge from detention basins is controlled by a range of outlet structures including pits, pipes, culverts and/or weirs. The location of these basins is shown in Figure 22. The two locations indicated in blue (YBDr and YBSch) in Figure 22 while not forming part of the formal detention basin network (as there is minimal detention volume upstream), are included in the hydraulic reporting as they are cross road culverts and influence the flow conveyance through the site. A brief description of each basin is provided in Table 25.



DesignFlow

LEGEND

- Yarrabilba site boundary
- Developed catchment
- Waterway drainage line
- Flood detention
- Culvert

FIGURE 22
Flood detention basins (and culverts)

Yarrabilba Precinct 1 and 2 Stormwater Management Plan

N W E S

SCALE:
1 : 1000

Date:
26 February 2014

Client:
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Table 25: Flood detention basin descriptions

Location	Description
Flood 1	Upstream of future SIC in Catchment 2. Low flow pipes drain to Wetland W2, high flow outlets directly downstream to channel adjacent to Wetland W2 and ultimately to Flood 6 detention basin.
Flood 2	Upstream of a purpose built bund across waterway. (Already constructed)
Flood 3	Upstream of future SIC in the northern waterway in Catchment 2.
Flood 4	Upstream of road embankment across northern waterway in Catchment 2.
Flood 5	Above bioretention basin B4a surface and upstream of road embankment at the upper end of the northern waterway in Catchment 2
Flood 6	Formed by a proposed bund along northern site boundary to manage flows from Catchment 2. Targets management of minor flow discharge downstream (while the balance of the flood basins are targeting the major flows upstream in the catchment) Integrates Wetland W2 internal to the area.
Wetland W2	Receives low flow discharge from Flood 1 and acts as a perched flood basin within Flood 6 zone to provide low flow attenuation for very minor flows. Weir overflow occurs to Flood 6 under high flows.
Flood 9	Upstream of future SIC in Catchment 3. Manages part of future Precinct 3 catchment to the east also.
Flood 10	interim flood detention basin for Catchment X31 and X32 (figure 17) draining to Quinzeh Creek. This basin will only operate until further development occurs to the south and the roads are extended across the waterway (which forms the ultimate broader catchment flood strategy).
Flood 11	Interim flood detention basin serving post developed catchment X30 (Figure 17) until future precinct to the south is developed and regional flood detention strategy is implemented. Created by bund across an existing isolated channel meander
YBDr	Yarrabilba Drive cross road culverts in central waterway in Catchment 2. No significant flood detention occurs upstream of these culverts but included in hydraulic reporting.
YBSch	Yarrabilba Drive cross road culverts adjacent to proposed school in Catchment 2. No significant flood detention occurs upstream of these culverts but included in hydraulic reporting.

Discharge control structures for the detention basins identified in Table 25 are summarised in Table 26.

Table 26: Developed case hydraulic structures

Location	Structure ID	Size (mm)	USIL	DSIL	Length (m)
Flood 1	FL1_Lo	3 x 1050 RCP	24.5	24	100
	FL1_Hi	8 x 1200 RCP	25.9	25.76	70
Flood 2	FL2_Lo	4 x 1200 RCP	28.05	28.01	12.2
	FL2_Weir	20m Weir	29.85	-	
Flood 3	FL3_	3 x 1200 RCP	28.5	28.29	70
Flood 4	FL4_	3 x 1200 RCP	32.8	32.638	34.16
Flood 5	FL5_	900 x 1200 pit* 1500 x 1200 pit* with 1 x 1200 RCP*	43.3 43.8 41.3	- -	85
Wetland W2	W2_pipe	1 x 600 RCP	21	20.5	20
	W2_Weir	20m Weir	22.5	-	-
Flood 6	FL6_Lo	1 x 2400 x 750 RCBC	20.21	20.2	42
	FL6_Weir	70m weir	22.2	-	-
Flood 9	FL9_Lo	1 x 600 RCP	24.5	24.325	70
	FL9_Hi	3 x 900 RCP	26.6	26.39	70
Flood 10	FL10_Lo	1 x 525 RCP	26.5	26.41	30
	FL10_Weir	10m weir	28.4	-	-
Flood 11	FL11_Lo	1 x375 RCP	34.5	34.4	20
	FL11_Hi	2 x 450 RCP	36.1	36.0	20
	FL1_Weir	8m weir	36.75	-	-
YBDr	YBDr_out	3 x 2400 x 750 RCBC	32.01	31.91	31.72
YBSch	YBSch_out	3 x 2700 x 600 RCBC	30.05	29.75	39.04

RCP = Reinforce concrete pipe, RCBC = reinforced concrete box culvert

* Flood 5 outlet structure comprises a two stage pit with a single discharge pipe. Refer below.

The two stage Flood 5 outlet control pit is modelled with a rating curve in Tuflow. This outlet functions as both the bioretention basin treatment control outlet (low level) and the flood discharge control pit(high level). The complexity of modelling this pit in TUFLOW was replaced by a rating curve based on upstream water level (stage-discharge). The rating curve data is provided in Table 27.

Table 27: Basin 5 Outlet Stage-Discharge curve data

Depth above piut crest(m)	Discharge (m^3/s)
0	0
0.1	0.1
0.2	0.2
0.3	0.4
0.4	0.6
0.5	0.9
0.6	1.3
0.7	1.6
0.8	2.0
0.9	2.5
1.0	3.0
1.1	3.4
1.2	3.6
1.3	3.8
1.4	4.0
1.5	4.2
1.6	4.3

8.8 RESULTS

Table 29and Table 30 presents the pre-development, post development unmitigated and post development mitigated peak flood flows at each of the discharge locations from Yarrabilba into downstream property. **The results indicate peak flows are attenuated back to or below existing peak flows for all events (flood objective).**

Peak flows at Local LA_30 increase because no flood detention is proposed at this location in the interim because future flood detention will manage these flows when Precinct 3 is delivered. However, an interim storage can be provided if required by EDQ and it will be in the order to 5000m^3. TUFLOW modelling was not complete for LA_30.

Table 28: Pre-development (Existing) case peak flow at reporting locations

Location	Peak Discharge – TUFLOW (XPSWMM in brackets) m^3/s						
ARI -->	1	2	5	10	20	50	100
LA_17	4.87 (4.93)	9.22 (9.13)	15.56 (14.90)	19.65 (18.90)	25.31 (24.58)	32.96 (31.71)	39.32 (37.97)
LA_23	0.54 (0.52)	1.01 (0.96)	1.66 (1.64)	2.08 (2.05)	2.69 (2.66)	3.49 (3.45)	4.14 (4.10)
LA_27	0.91 (0.88)	1.69 (1.66)	2.82 (2.72)	3.55 (3.42)	4.57 (4.38)	5.94 (5.63)	7.07 (6.70)
LA_30*	(0.24)	(0.4)	(0.64)	(0.81)	(1.07)	(1.38)	(1.65)
LA_32	0.56 (0.63)	1.02 (1.13)	1.96 (1.83)	2.53 (2.27)	3.30 (2.94)	4.34 (3.85)	5.25 (4.59)

* TUFLOW modelling was not completed for this catchment due to its small size and direct inflow characteristics. XP-SWMM results are presented.

Table 29: Unmitigated case peak flow at reporting locations

Location	Peak Discharge (TUFLOW only) m^3/s						
ARI -->	1	2	5	10	20	50	100
LA_17	21.5	29.8	39.9	45.80	53.90	66.90	74.90
LA_23	0.20	0.50	0.90	1.10	1.40	1.80	2.10
LA_27	5.80	7.80	10.10	11.40	13.20	15.60	17.40
LA_30*	(1.18)	(1.54)	(1.97)	(2.22)	(2.56)	(3.00)	(3.33)
LA_32	2.80	4.00	5.40	6.20	7.40	9.10	10.30

* TUFLOW modelling was not completed for this catchment due to its small size and direct inflow characteristics. XP-SWMM results are presented.

Table 30: Developed case (mitigated) peak flow at reporting locations

Location	Peak Discharge – TUFLOW (XPSWMM in brackets) m^3/s						
ARI -->	1	2	5	10	20	50	100
LA_17	4.82 (4.55)	6.05 (10.21)	12.14 (16.04)	15.85 (20.16)	20.63 (25.61)	28.55 (33.31)	33.09 (37.60)
LA_23	0.37 (0.42)	0.66 (0.66)	1.02 (1.01)	1.24 (1.23)	1.53 (1.52)	1.90 (1.90)	2.19 (2.21)
LA_27	0.91 (1.08)	1.42 (1.81)	2.25 (2.87)	2.79 (3.41)	3.44 (4.09)	3.96 (5.13)	4.41 (5.59)
LA_30*	(0.22)	(0.40)	(0.61)	(0.71)	(0.82)	(1.30)	(1.61)
LA_32	0.57 (0.6)	0.62 (0.66)	1.45 (1.51)	2.02 (1.92)	2.57 (2.61)	4.03 (3.76)	4.83 (4.50)

* TUFLOW modelling was not completed for this catchment due to its small size and direct inflow characteristics. XP-SWMM results are presented.

Figure 23 presents pre and post development mitigated 100yr ARI hydrographs for location LA_17 (Catchment 2). The figure presents the 90 minute and 120 minute storms only for clarify as these storms produce the highest flows at this location. The 120 minute storm is also the critical storm for Quinzeh Creek where flows from Catchment 2 flows into the creek. Therefore, as well as managing the peak flows at the boundary of Precinct 1 and 2, the peak flows for the 120minute storm should be attenuated to pre-development rates to minimise the risk of coincident flood further downstream.

The hydrographs show that post development peak flows for both the 90 minute and 120 minute storm are attenuated to below pre development peak flow rates. The attenuation acts to 'flattened out' the hydrograph and delays the peak of the hydrograph by 10-15mins when compared to pre-development, thus actually increasing the warning times for peak flow generation. The same outcomes are found at locations LA_27 (Catchment 3) and LA_32 (Catchment 4) when reviewing the hydrographs.

Based on these findings it is very unlikely there will be an increase in flood flows further downstream on Quinzeh Creek as a result of coincident flooding.

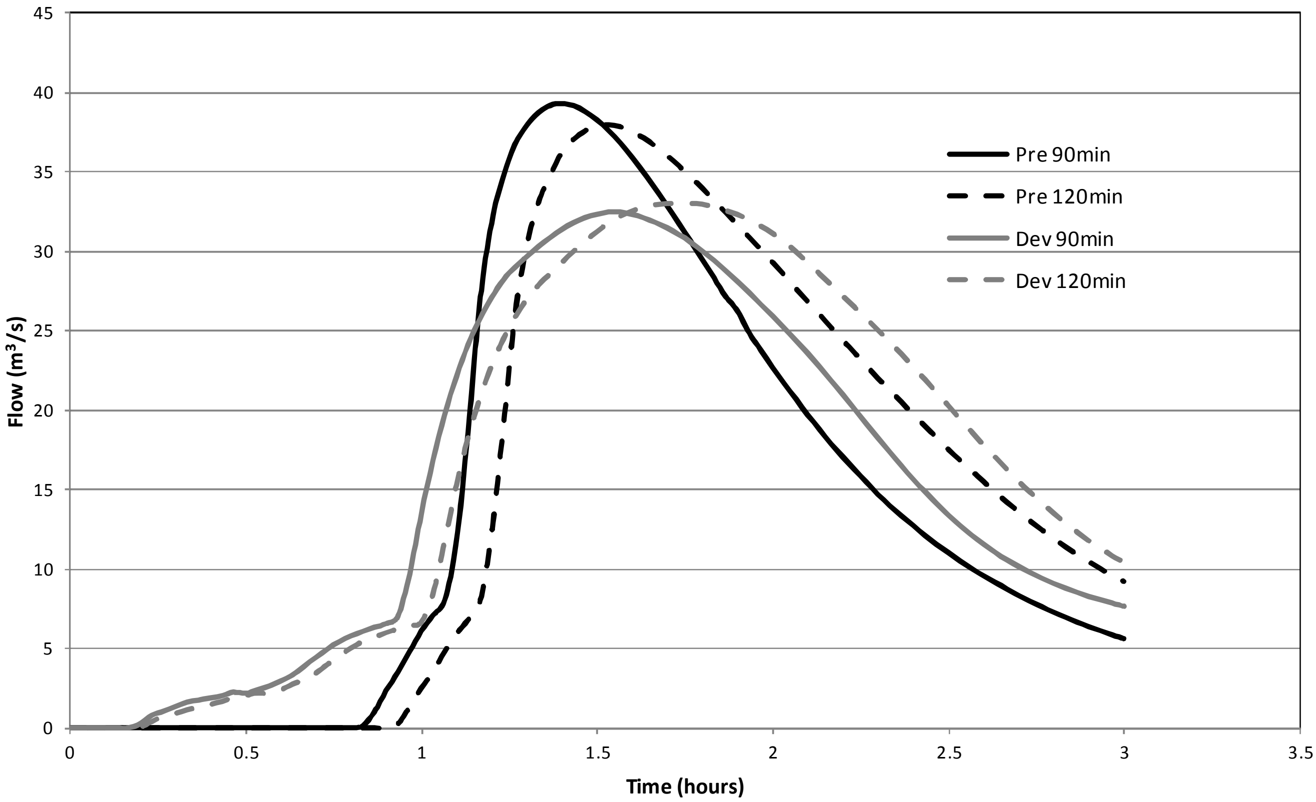
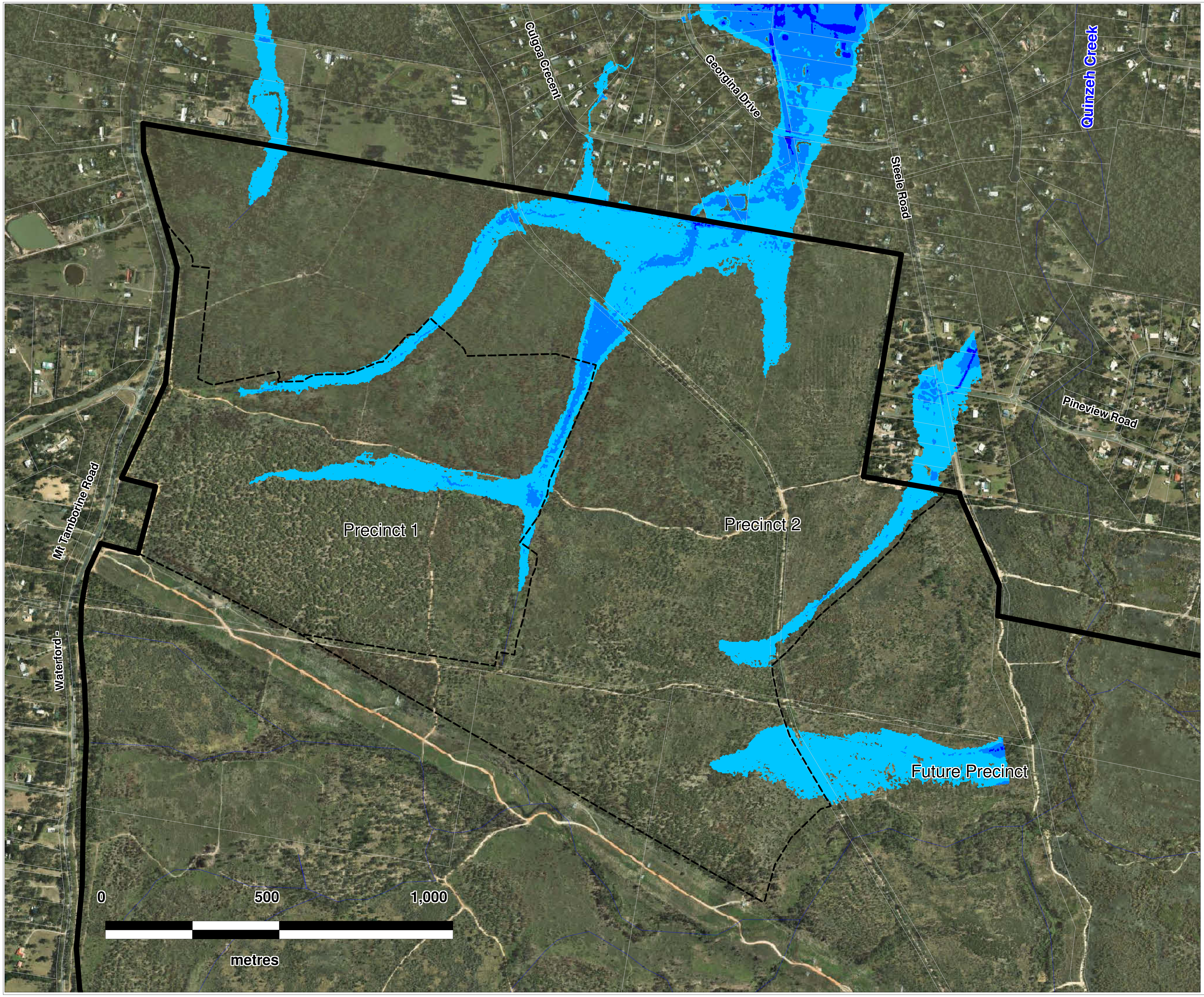


Figure 23: Pre and post development 100yr ARI hydrographs LA_17 (Catchment 2)

Figure 24 presents the pre-development 100yr ARI flood levels and extent across Precinct 1. Figure 25 presents the post development 100yr ARI flood levels and extent. Appendix D presents the flood maps for all the flood events. There are increases in flood levels within Precinct 1 and 2, particularly in the flood detention basins and behind roads. **The development earthworks levels have to ensure roads and allotment levels are above the 100 year ARI (flood objective).**

Flood levels at key locations are summarised in Table 31 and detention storage details are provided in Table 32.



DesignFlow

LEGEND

Boundaries

Yarrabilba Site

Precinct

Depth (m)

< 0.5

0.5 - 1.0

1.0 - 1.5

> 1.5

FIGURE 24
Pre-development 100yr ARI Flood Level and Extent

Yarrabilba Precinct 1 and 2 Stormwater Management Plan

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1 : 1000

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1 November 2013

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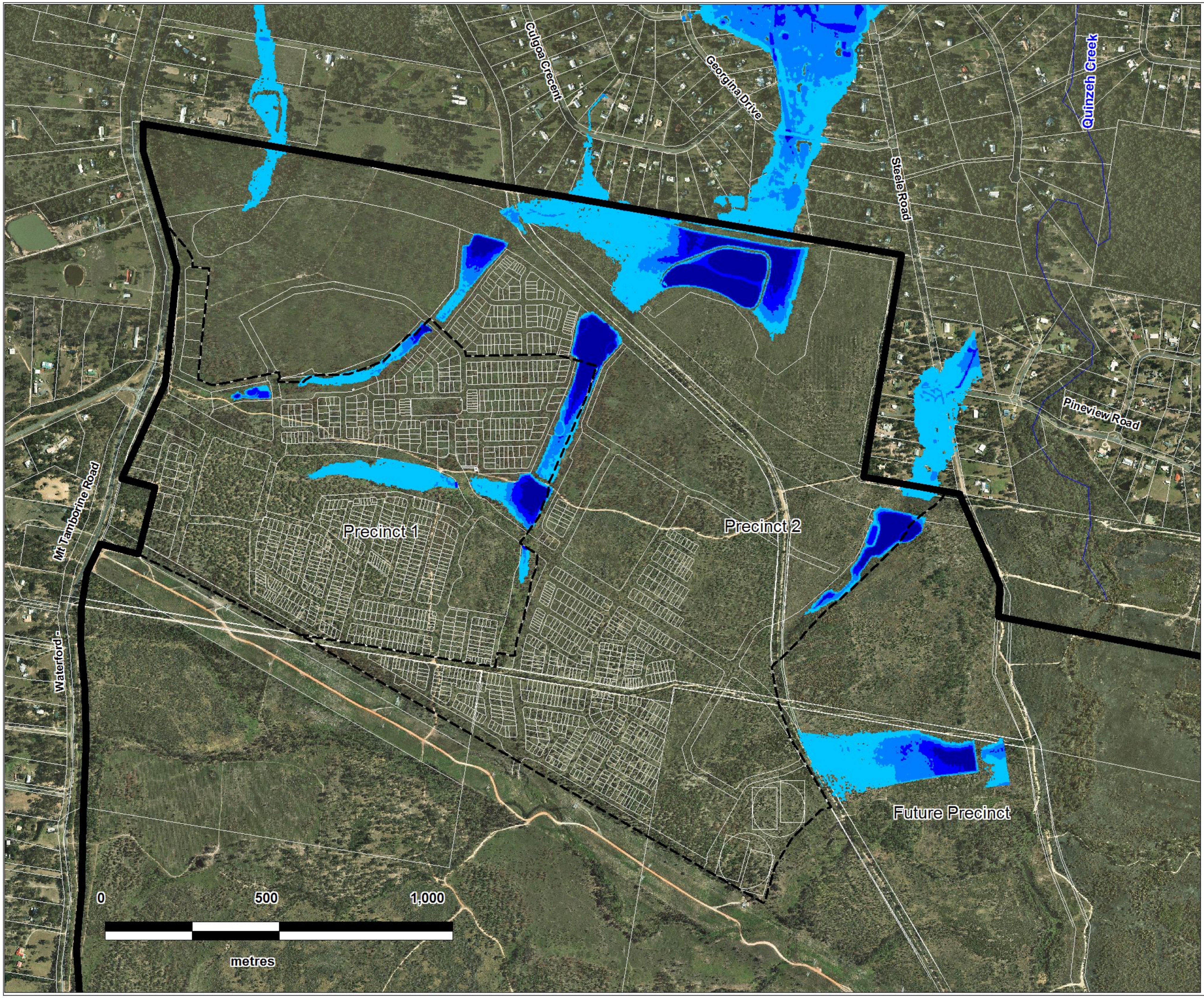


DesignFlow

LEGEND

Boundaries

Yarrabilba Site

Precinct

Depth (m)

< 0.5

0.5 - 1.0

1.0 - 1.5

> 1.5

Figure 25
Post-development Mitigated 100yr ARI Flood Inundation

Yarrabilba Precinct 1 and 2 Stormwater Management Plan

SCALE: 1 : 1000

Date: 2 December 2013

Client: Lend Lease

Table 31: Developed case peak water levels at key locations

Location	Peak Water Level mAHD						
ARI -->	1	2	5	10	20	50	100
Flood 1	25.95	26.27	26.51	26.61	26.75	26.98	27.11
Flood 2	29.20	29.43	29.72	29.85	29.99	30.16	30.24
Flood 3	29.92	30.13	30.41	30.55	30.73	31.03	31.19
Flood 4	33.77	33.95	34.14	34.25	34.43	34.65	34.80
Flood 5	43.58	43.61	43.66	44.14	44.14	43.78	44.28
Flood 6	21.65	22.19	22.33	22.38	22.44	22.52	22.56
Wetland W2	22.67	22.77	22.81	22.82	22.82	22.86	22.87
Flood 9	26.79	27.05	27.30	27.42	27.57	27.79	27.94
Flood 10	28.12	28.36	28.53	28.59	28.63	28.74	28.79
Flood 11	36.13	36.35	36.52	36.61	36.74	36.84	36.88
YBDr	32.48	32.59	32.69	32.75	32.83	32.98	33.05
YBSch	30.66	30.79	30.90	31.08	31.19	31.35	31.44

* Flood 10 and 11 are interim flood detention basins only. Refer to Table 25

Table 32: Peak 100 year ARI volume in detention basins

Basin	Water Level (mAHD)	Depth (m)	Storage Volume (m3)
Flood 1	27.11	2.78	37,023
Flood 2	30.24	2.23	17,142
Flood 3	31.19	2.45	14453
Flood 4	34.80	1.99	4721
Flood 5	44.28	2.12	4385
Flood 6	22.56	2.38	43264
Wetland W2	22.87	2.31	36842
Flood 9	27.94	3.54	26552
Flood 10	28.79	2.30	33138
Flood 11	36.88	2.39	3,576

8.8.1 Safety Considerations

Some of the flood detention storage depths are up to 3.5m. Logan City Council requires detention basins to be designed in accordance with QUDM (2013) with regard to public safety. Specifically, it is desirable (where suitable land is available) to maintain flooded depths to 1.2m for the 1 in 20 AEP and greater if possible. However, (as QUDM states), *in cases where this is neither practical nor economical, and the provision of a detention basin is considered to be better on safety grounds than other alternatives, greater depths may be acceptable. Notwithstanding this, designers are responsible for investigating the overall safety risks associated with the basin, design of the basin and the surrounding landscape in a manner that minimises safety risks and satisfying any safety requirements specified by the local government.*

Given the topography of the Yarrabilba site and the conservative approach to hydrologic modelling (i.e. high initial loss for pre-development scenario vs zero initial loss for post developed), it is not practical nor economical to achieve the 1.2m depth.

In order to quantify the areas onsite that exceed the preferred depth of 1.2m, Appendix E provides mapping of these areas for both the 20 year and 100 year ARI events. To manage safety risk in these areas the following has been adopted for design:

- 1 in 6 batters used where flood depths in are greater than 1.2m
- system will be planted out with vegetation and trees to discourage access
- warning signage provided
- depth indicators to be installed.
- Areas with slopes steeper than 1 on 4 will require steps and a handrail to assist egress (however this is an undesirable outcome and will be designed to meet dot point 1 as far as practical)

(note: Fencing is not recommended by QUDM unless the flood risk is extreme)

There will be no passive or active recreation proposed 'within' in the low areas of flood detention basins that will be subject to flood depths greater than 1.2m or will have a depth x velocity product (d.V) greater than $0.6m^2/s$.

Detailed design of all flood detention zones will also be undertaken to ensure that safety is maintained under non-design operating conditions. Specifically, high level outlets, discharge weirs or road crossings (where applicable) will be designed to ensure that under 100% blockages of low level outlets and/or 50% blockage of screened road culverts (in accordance with QUDM) under the major design storm, the depth x velocity product of overflows at these locations does not exceed $0.6\ m^2/s$. Where this can't be achieved, public access must be restricted in accordance with QUDM.

8.9 SENSITIVITY ANALYSIS

The flood modelling has, so far as practical, attempted to be consistent with the parameters adopted by *Quinzeh Creek Design Discharge Estimate* (WRM, 2013) and *Revised Quinzeh Creek Flood Study Letter* (Engeny, 23 August 2013) for the Council modelling. Some testing of modelling parameters (Bx, Mannings etc.) was undertaken as part of the model establishment. However, in the end all the parameters adopted for Precinct 1 and 2 were consistent with WRM (2013) and Engeny (23 August 2013)

The parameter which was altered was the initial loss for pre-development condition for the reasons described earlier in this report. A range of initial loss parameters we tested in XP-SWMM for all the catchment. The peak flows for each of the losses are presented in Table 33. As expected increasing the initial loss parameter reduces peak flows with the largest reduction occurring for the 1 year ARI event when going from 20mm to 30mm. The drop from 30mm to 40mm is less significant. This is because at about the 30mm initial loss the peak flow is generated by the 6 hour event in the 1 yr flood. This long duration is less influenced by the initial loss. Importantly the proposed flood management strategy results in flows which are generally less or very close to all the peak flows presented below including the initial loss of 40mm.

Table 33: Peak flows for different initial losses

Location	Peak Discharge – XPSWMM m^3/s					
ARI -->	1			100		
IL -->	20mm	30mm	40mm	20mm	30mm	40mm
LA_17	7.56	4.93	4.57	43.31	37.97	32.85
LA_23	0.82	0.52	0.50	4.69	4.10	3.56
LA_27	1.35	0.88	0.82	7.59	6.70	5.80
LA_30*	0.32	0.24	0.20	1.87	1.65	1.39
LA_32	0.91	0.63	0.53	5.11	4.59	3.95

Further sensitivity assessment for initial losses was undertaken by Engeny in February 2014 based on the broader Quinzeh Creek flood model. This was undertaken on the January 2013 flood event which as used to verify the Quinzeh Creek Model initial loss parameters previously.

The assessment tested the impact of changes to initial loss (IL) on the Yarrabilba site (within the broader model) from the existing 38mm (area weighted loss) to 5mm, 30mm and 49mm. The results from this found that, as expected, the reduced IL at Yarrabilba had little effect on the 2103 calibration event peak flows or timing of hydrographs at all reporting locations immediately downstream of Yarrabilba as well as at the downstream confluence, and in fact, the high IL of 49 also had negligible impact, further reinforcing the discussion above that the peak flows and timing are dominated by longer duration events where initial losses are not significant to the peak flow generation. Correspondence from Engeny summarising the assessment is provided in Appendix F. Further regional flood modelling will be ongoing as the detailed design of flood detention basins occurs to ensure that all elements of the Quinzeh Creek hydrology are not worsened as a result of development. Council is to provide the regional flood model and data to allow this occur.

9 WATERWAY STABILITY

9.1 DESIGN OBJECTIVE

The waterway stability objectives adopted for the site are based on protecting waterways and drainage lines from erosion as a result of increased flow from urban development. The waterway stability objective has been set as part of the *Yarrabilba - Stormwater Infrastructure Master Plan* (DesignFlow, 2012) for a range of situations as shown in Table 34.

Given the state of the waterways in Precinct 1 and 2 (mostly Average-Poor condition), the second and third objectives highlighted in blue have been adopted for Precinct 1 and 2. The first objective has been used as a stretch target for the internal waterways where possible.

Table 34: Derived waterway stability objectives

Waterway classification	Waterway Stability Objectives
Site waterway reach in good condition* (i.e. well vegetated and stable waterway)	Limit post development 1 year ARI flows in waterway to pre-development conditions** OR Ensure the stream erosion index remains below 1.5 when comparing pre and post development flows.
Site waterway reach in poor condition* (i.e. poorly vegetated or active erosion)	As per above OR Rehabilitate waterway to convey the <u>post</u> development 1 year ARI flows without the risk of erosion**
External waterways	Limit 1 year ARI flows at site boundary for critical duration event (60 minutes or longer) to pre-development conditions

* Refer Figure 3 for preliminary waterway condition

** Localised increases in 1 year ARI flows can be accepted provided the erosion criteria for sandy vegetated soils (50% cover for native grasses) is achieved in accordance with QUDM

*** The objective supports the rehabilitation of degraded waterways and allows local increases in 1 year ARI provided the rehabilitation design provides an appropriately stable waterway for the increased 1 year ARI flows.

9.2 STRATEGY

The waterway stability objectives for Precinct 1 and 2 will be achieved through the following initiatives as presented in Figure 26:

- **Flood detention** (flows exiting Yarrabilba) – The 1 year ARI flows at the boundary of the site will be attenuated within the flood detention storages to ensure the pre-development flows are maintained. The location and size of the storage is provided in Section 8.7. The outlet structures from all the detention systems are staged to ensure the 1 year ARI flow is attenuated as well as larger flows up to 100 year ARI.

- **Waterway remediation** (within Yarrabilba) – Although flow attenuation is provided along the length of the waterways, there are waterway reaches within Yarrabilba where flows will increase as a result of development. This is unavoidable and creates a risk of erosion. Furthermore, there are signs of existing erosion in some of the waterways as a result of previous landuse practices.

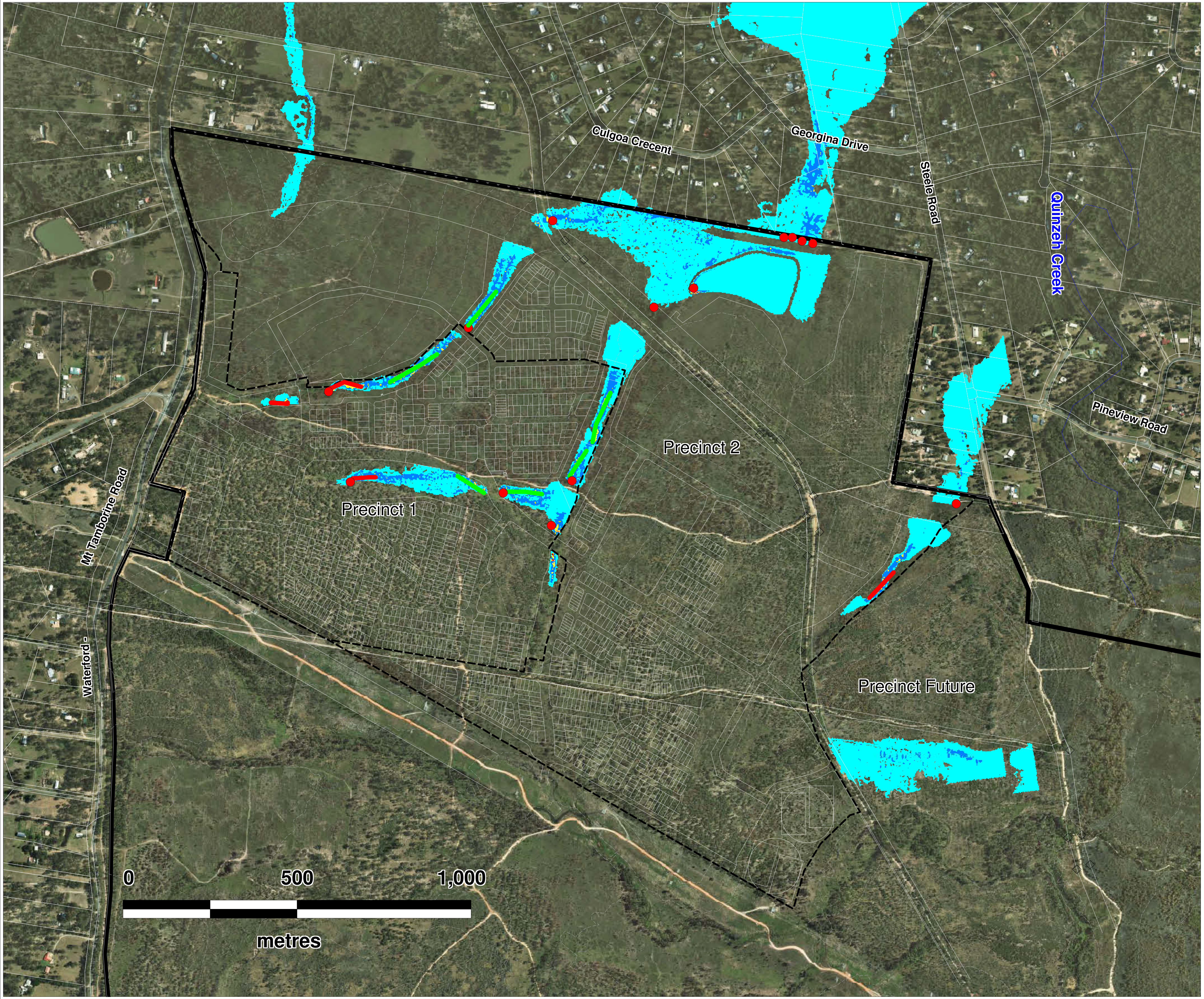
  Waterway remediation is proposed for essentially all the waterways within Precinct 1 and 2. The waterway remediation concept is described in Section 5.3 and involves mostly revegetation with additional stabilisation provided in the zones where criteria listed in Table 35 is not achieved. The areas of additional stabilisation have been identified through interpretation of the 2-dimensional flood modelling presented in Section 8.6.

DesignFlow are developing the concept presented in Figure 26 into a waterway remediation strategy with Lend Lease at the time of writing this document. This involves further interpretation of the waterway stabilisation criteria and flood modelling along with assessment of bed shear stress (over both design storms and continuous simulation) to inform planting and stabilisation. The scope of this work is too detailed to be presented in this report.

Table 35: Internal waterway stabilisation criteria

Parameter	Criteria* (above which additional stabilisation is required beyond revegetation, based on 50% vegetation coverage on erodible soils)
1 and 2 year ARI velocity (QUDM, 2013) (ACARP, 2002)	1.0m/s
50 year ARI (QUDM, 2013)	1.5m/s

* Criteria based on 50% vegetation coverage on erodible (i.e. sandy) soils



DesignFlow

LEGEND

- Yarrabilba Site
- Precinct 1 and 2
- Outlet rock scour protection
- Waterway rock scour protection
- Revegetate and observe (for local scour)

FIGURE 27
Waterway Stabilisation Concept

Yarrabilba Precinct 1 and 2
Stormwater Management Plan

N S E W

SCALE:
1 : 1000

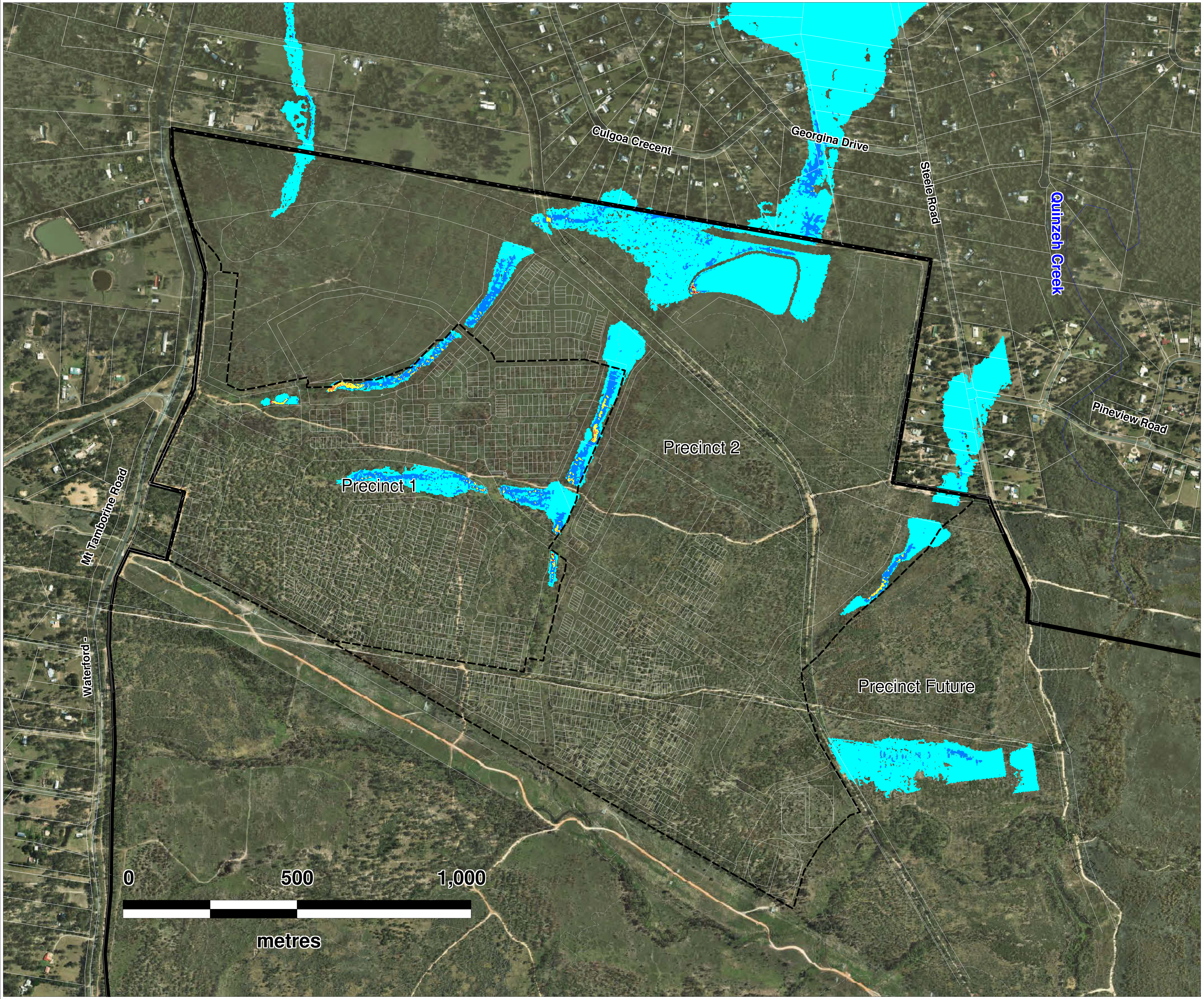
Date:
1 November 2013

Client:
Lend Lease

9.3 MODELLING

All the modelling required for the waterway stability objectives was completed using the flood modelling presented in Section 8.6. As presented in Table 28 and Table 30 the 1 year ARI flow at the boundary of Precinct 1 and 2 have been preserved at pre-development rates. This ensures the 'external waterways' component of the waterway stability objective is achieved.

In terms of managing waterway stability within internal waterways, the peak velocities for the 1 year ARI and 50 year ARI have been extracted from the TUFLOW modelling and are presented in Figure 27 and Figure 28 respectively. The figures indicate that velocities are below the criteria listed in Table 35 for the majority of the internal waterways indicating that the waterways should remain stable provided they are revegetated. The locations where velocities are elevated above the criteria in Table 35 are highlighted in the figures and will require additional stabilisation. The details of the stabilisation (i.e. rock protection, bed level control etc.) is being resolved as part of the waterway remediation strategy for Precinct 1 and 2.



DesignFlow

LEGEND

Boundaries

Yarrabilba Site

Precinct

Velocity (m/s)

< 0.5

0.5 - 1.0

1.0 - 1.5

1.5 - 2.0

> 2.0

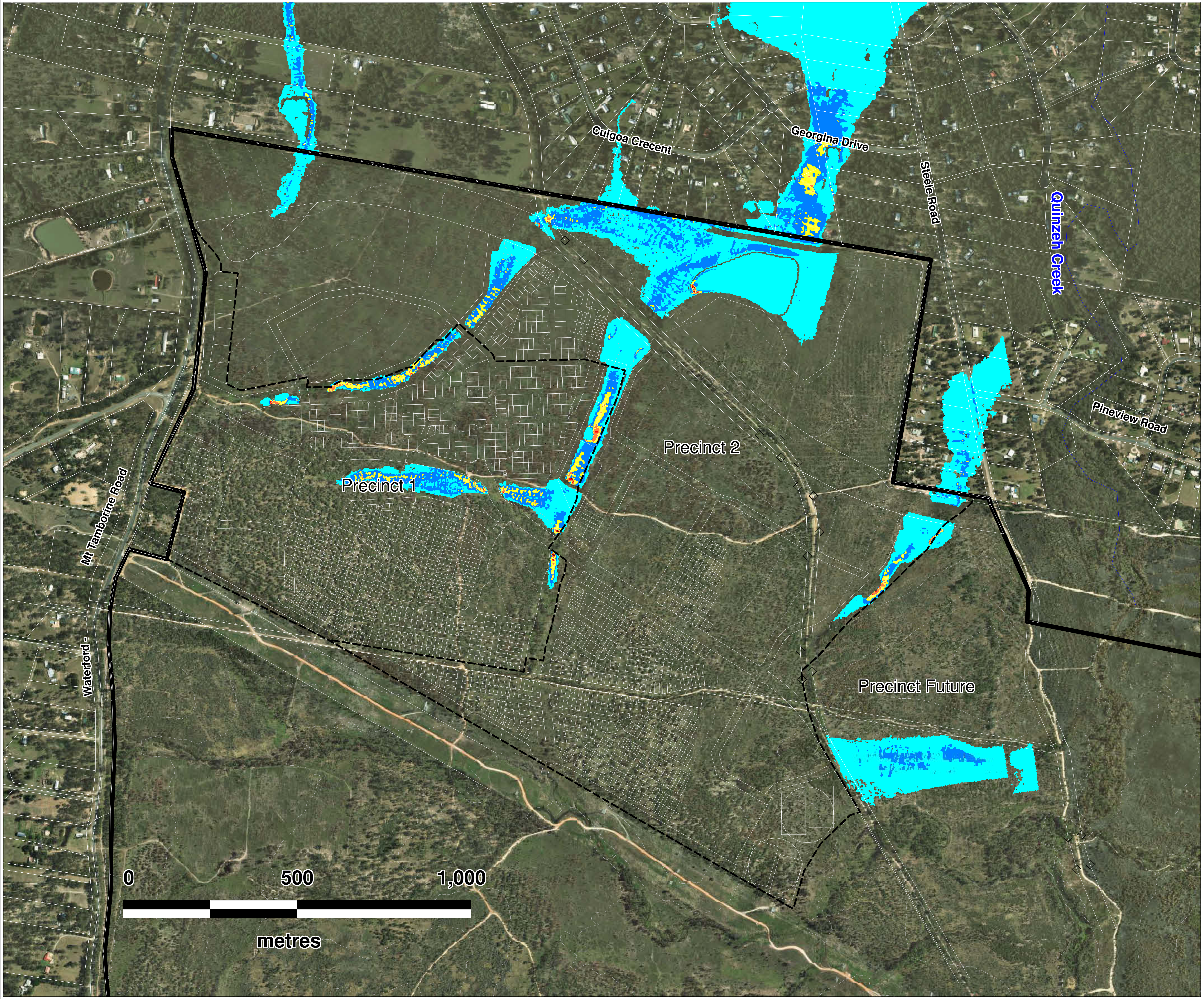
FIGURE 27
1 year ARI Flow Velocities

Yarrabilba Precinct 1 and 2
Stormwater Management Plan

SCALE:
1 : 1000

Date:
1 November 2013

Client:
Lend Lease



DesignFlow

LEGEND

Boundaries
- Yarrabilba Site
- Precinct

Velocity (m/s)
- < 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- > 2.0

FIGURE 28
50 year ARI Flow Velocities

Yarrabilba Precinct 1 and 2
Stormwater Management Plan

SCALE:
1 : 1000

Date:
1 November 2013

Client:
Lend Lease

10 CONCLUSIONS AND RECOMMENDATIONS

A stormwater management strategy has been developed for Yarrabilba Precincts 1 and 2 that delivers best practice objectives for urban stormwater quality improvement, waterway stability and remediation, frequent flow management and flood management. The strategy complies with the *Yarrabilba Stormwater Infrastructure Master Plan* (DesignFlow, 2012) and the *Yarrabilba Total Water Cycle Management Plan* (DesignFlow, 2012).

Key features of the strategy include:

- Local and regional erosion and sediment control during construction including regular auditing.
- Adoption of suitable waterway buffers to preserve key waterways.
- Remediation of all waterways through minor stabilisation and significant revegetation to create waterway systems endemic to the region.
- Creation of stormwater management systems which integrate with the existing waterways and form part of the remediation of the waterways. The treatment systems include sediment basins, remediated waterways, bioretention systems, constructed wetlands and ephemeral wetlands.
- A series of flood detention basins throughout the development to manage flood flow exiting the site to downstream private property.
- Stormwater harvesting and other drainage works to manage nuisance flows in downstream property.

Preliminary earthworks plan has been developed for stormwater treatment system to illustrate that there is sufficient space for the systems within the development layout.

Lend Lease do recognise that further work is required to refine the strategy. This will involve monitoring or boreholes recently installed in Catchment 2 to refine the frequent flow management strategy. Furthermore, a waterway remediation strategy is to be resolved for all the waterway in Precinct 1 and 2 which accounts for stabilisation requirements and revegetation of the poor and average reaches.

It is expected that minor adjustments to the stormwater management designs may occur as part of detailed design for the Operational Works certification process. Any changes will be clearly identified and explained in the form of "WSUD Functional Design Reports" for each of the development stages. The reports will provide detailed description of the design and associated calculations along with detailed MUSIC modelling. Design development and detailed design of the WSUD Strategy will be undertaken in accordance with WSUD Technical Design Guidelines for SEQ.

11 REFERENCES

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APPENDIX A: CONCEPTUAL EARTHWORKS PLANS

LEGEND

LAYOUTS

KEYMAP

SCALE 1:10,000

1:10,000

0 200 400 600m

REVISION	DESCRIPTION	DRAFTED INITS	DRAFTED DATE	APPROVED INITS	APPROVED DATE
A	PRELIMINARY ISSUE	SL	26.02.14	SL	26.02.14

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DESIGNED	SL
DATE	26.02.14
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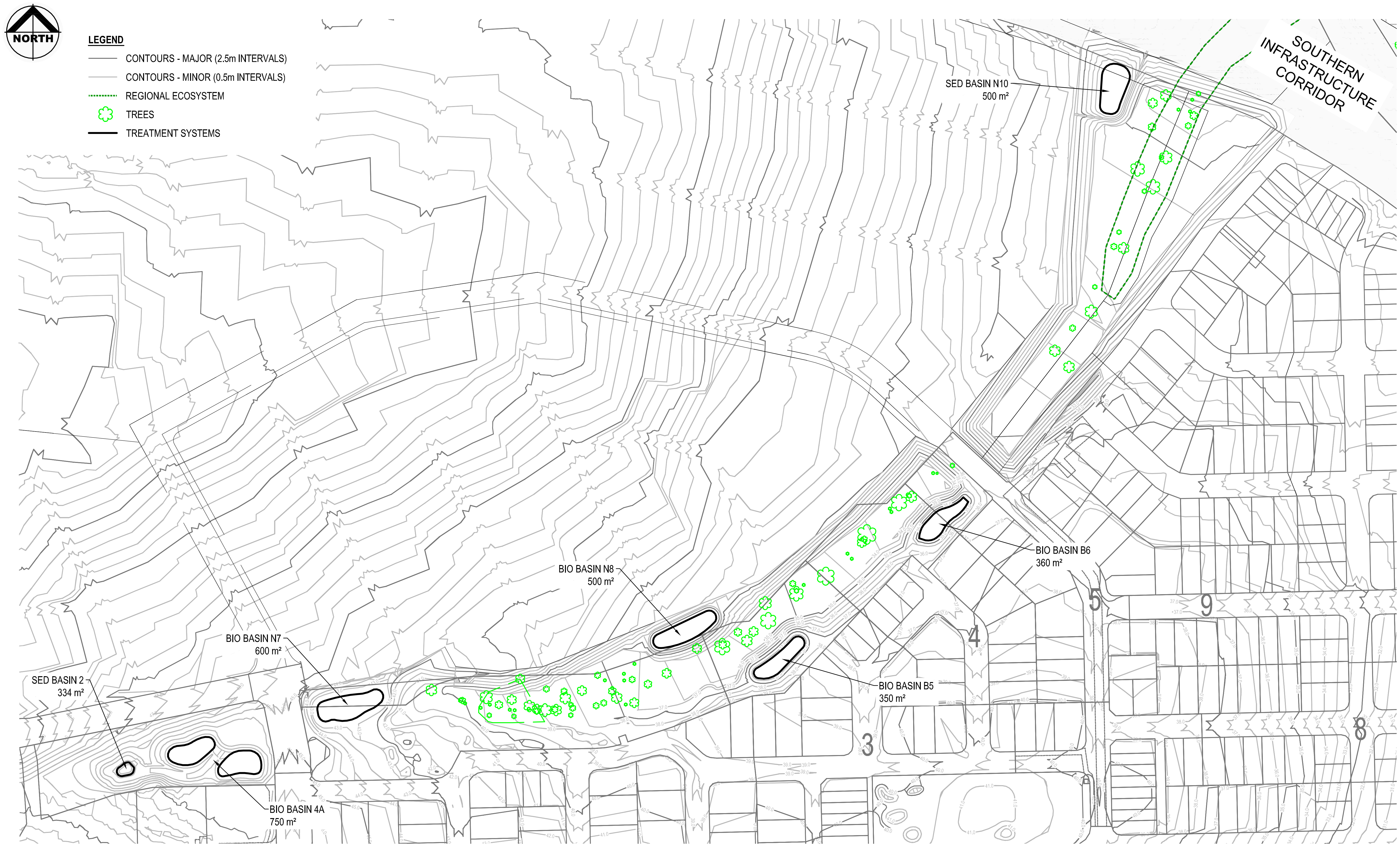
SIZE	A3
PROJECT	YARRABILBA
DRAWING TITLE	CONCEPTUAL EARTHWORKS
PROJECT No	4205
DRAWING No	D0001
REVISION	A



KEYMAP

SCALE 1:2,500

1:2,500

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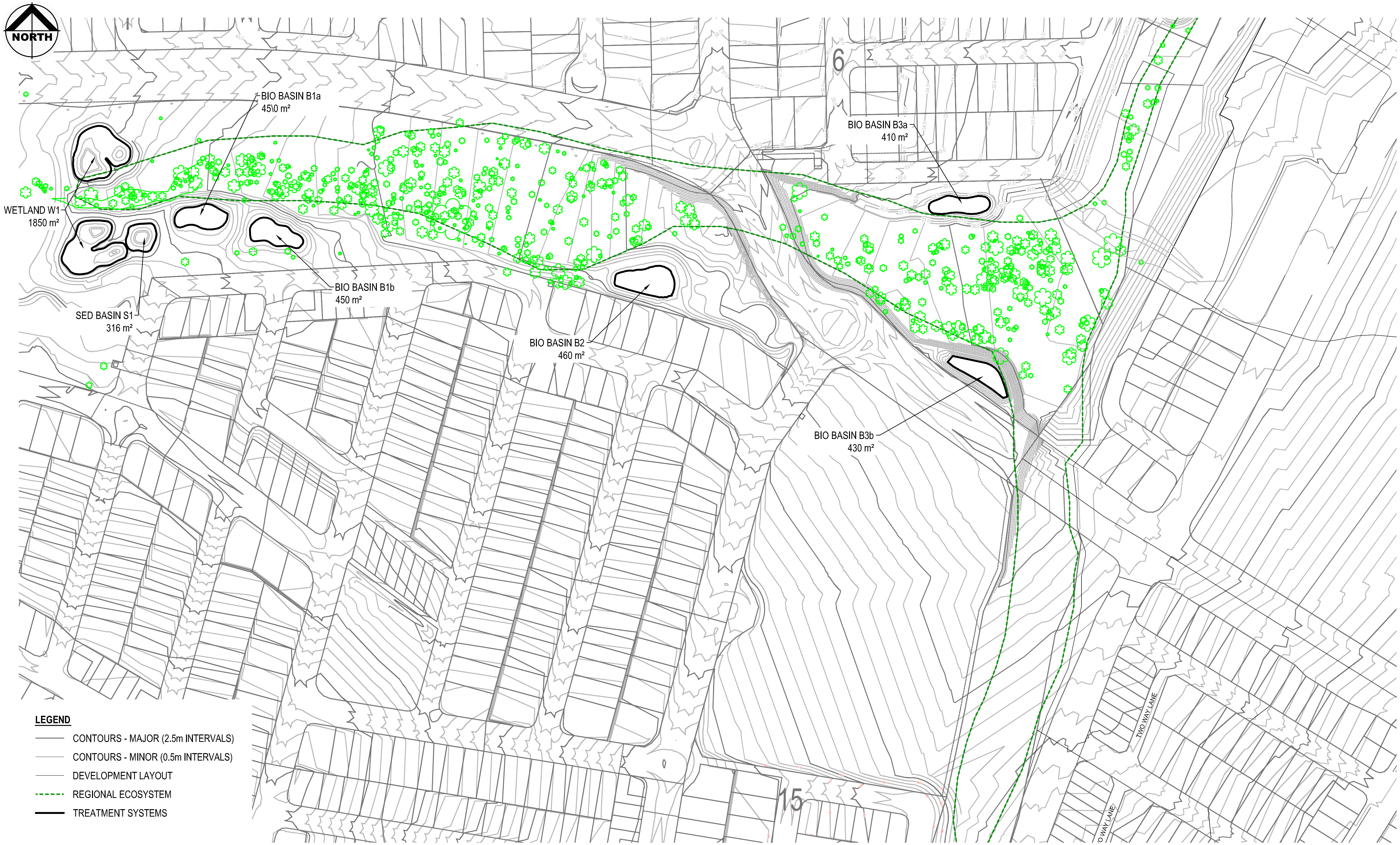
PROJECT No: 4205
DRAWING No: D0002
REVISION: A



KEYMAP

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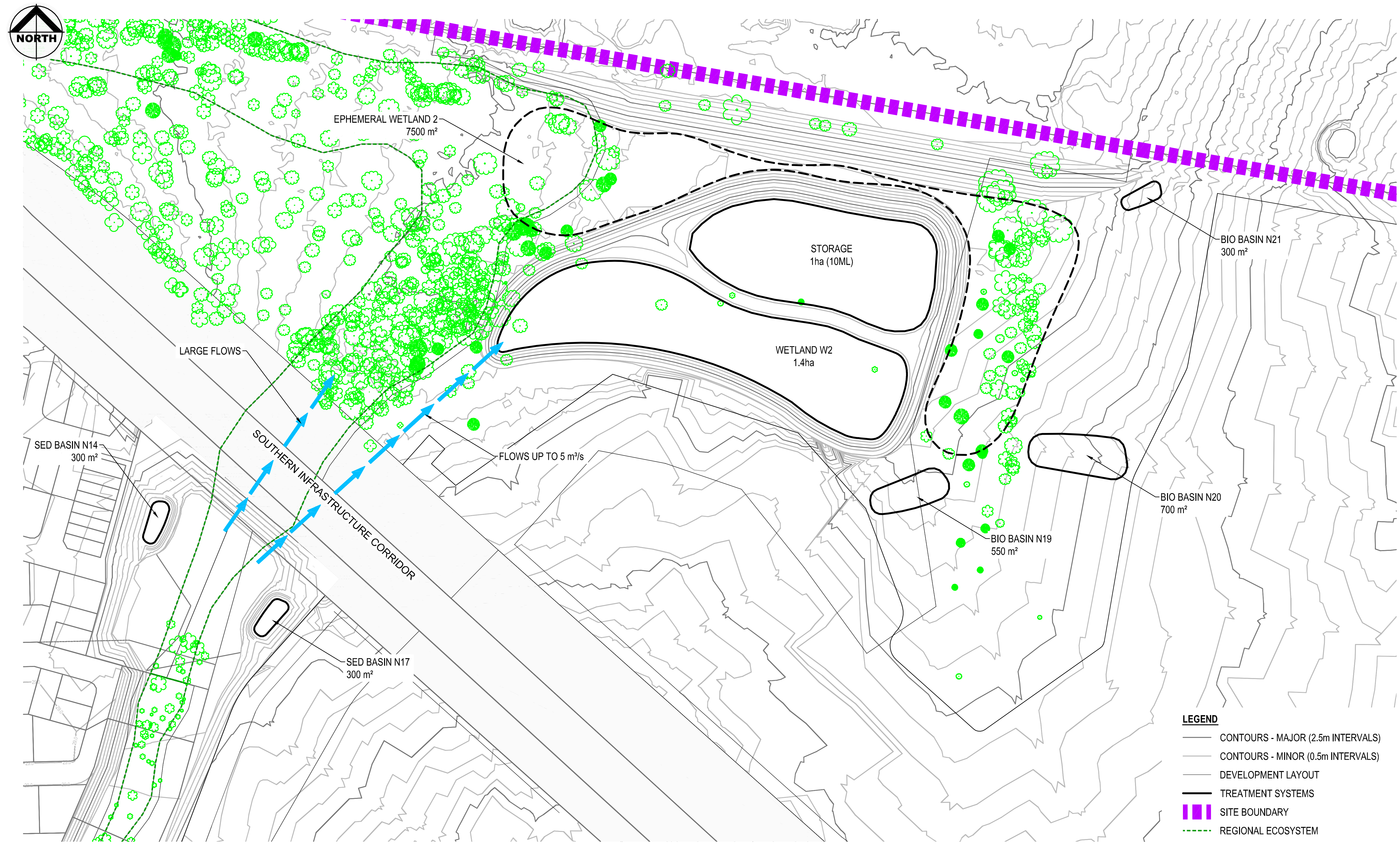
PROJECT	YARRABILBA
DRAWING TITLE	CONCEPTUAL EARTHWORKS LAYOUT 2
PROJECT No	4205
DRAWING No	D0003
REVISION	A



LAYOUT 3

SCALE 1:2,500

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DRAWING TITLE	CONCEPTUAL EARTHWORKS LAYOUT 3
PROJECT No	4205
DRAWING No	D0004
REVISION	A