3.9.2 Wetland Peak Flows – Major/Rare Rainfall Events

It is important to show that in rare storm events, the proposed development will not result in damaging flow velocities in the downstream wetland. While other catchment storm/tailwater conditions are modelled further on in this report to determine peak flood levels, these events generally include high tailwater levels. The scenario that will provide the highest resultant velocities through the downstream wetland is a 100yr catchment storm & existing -0.5m AHD Mean Low Water Level tailwater combination.

Note: The peer review of the previous report identified previously a +0.5m MHWM Tailwater was utilised, and that a -0.5m MLWM level would provide a greater head difference. While this change has been made for this report, there is no practical difference as the ground levels along the development/wetland are all above 0.5m AHD.

Furthermore, it was stated that the model extents should be extended across the wetland to the river. This change has been made for this section of this report and discussed in more detail in Section 3.9.5.5.

Note: this was limited to only the 75% of the SEPP14 wetland covered by detailed site survey information – see figures below. While there is not 100% coverage, the coverage includes all the wetland areas closest to the development, and as illustrated in Section 3.9.5.5 the additional model area has no impact on model behaviour, so it is considered sufficient to demonstrate the objectives of this section.

The figures below show peak velocities in the downstream wetland buffer and SEPP14 wetland in the worst case scenario pre and post development, including Velocity Difference Mapping in Figure 35.

The proposed Riverside development will result in no meaningful change to peak velocities in the critical 100yr flood event within the SEPP14 wetland. Figure 35 illustrates that any variation within the wetland extents are less than 0.05m/s. In both pre and post development scenarios, the worst case peak 100yr velocities are in the order of up to 0.2m/s and still at such a low level that they should not lead to any significant increase in levels of vegetation damage or soil erosion within the SEPP14 wetlands.

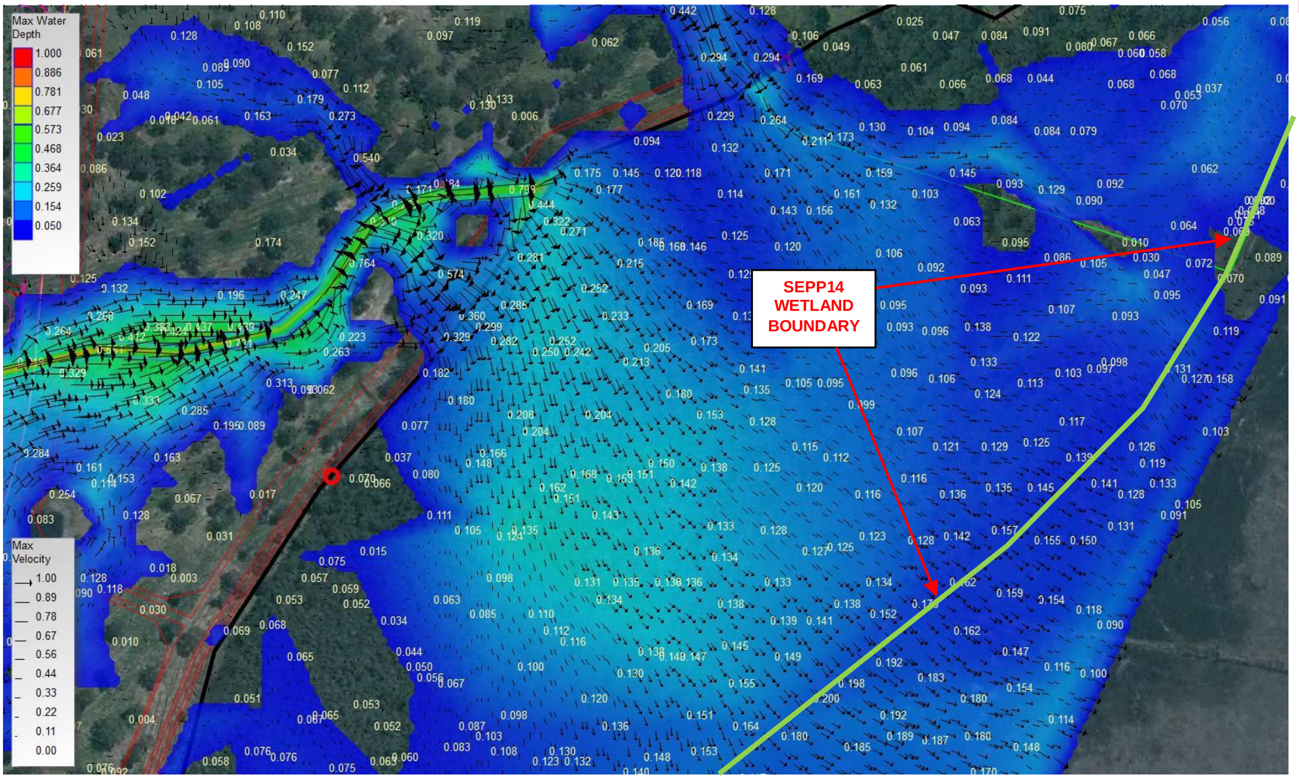

Figure 29 - Existing Critical Flood Depths/Velocities (Velocity values displayed) At Existing Drain Wetland Outlet – 2hr 100yr Storm, Current MLW Tailwater

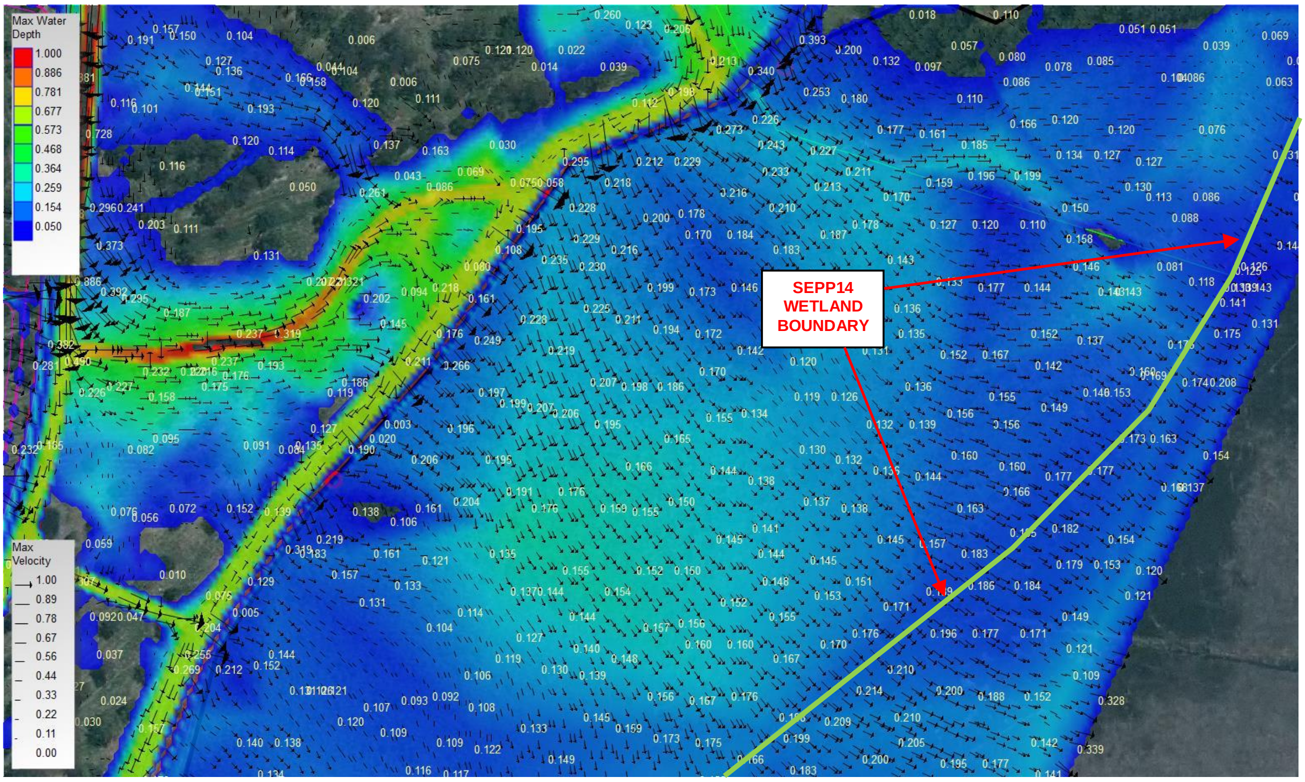

Figure 30 - Design Critical Flood Depths/Velocities (Velocity values displayed) At 'Discharge Culvert A' – 2hr 100yr Storm, Current MLW Tailwater

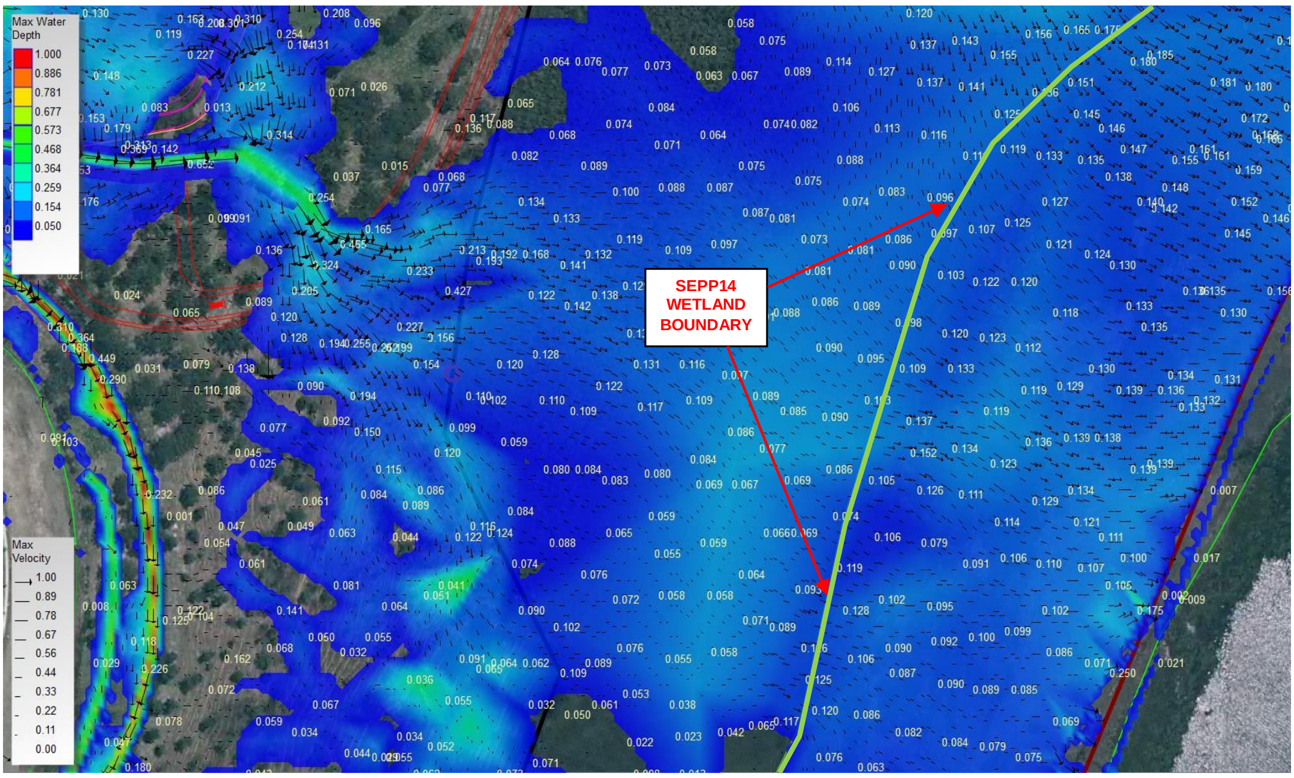

Figure 31 - Existing Critical Flood Depths/Velocities (Velocity values displayed) At Existing Drain Wetland Outlet – 2hr 100yr Storm, Current MLW Tailwater

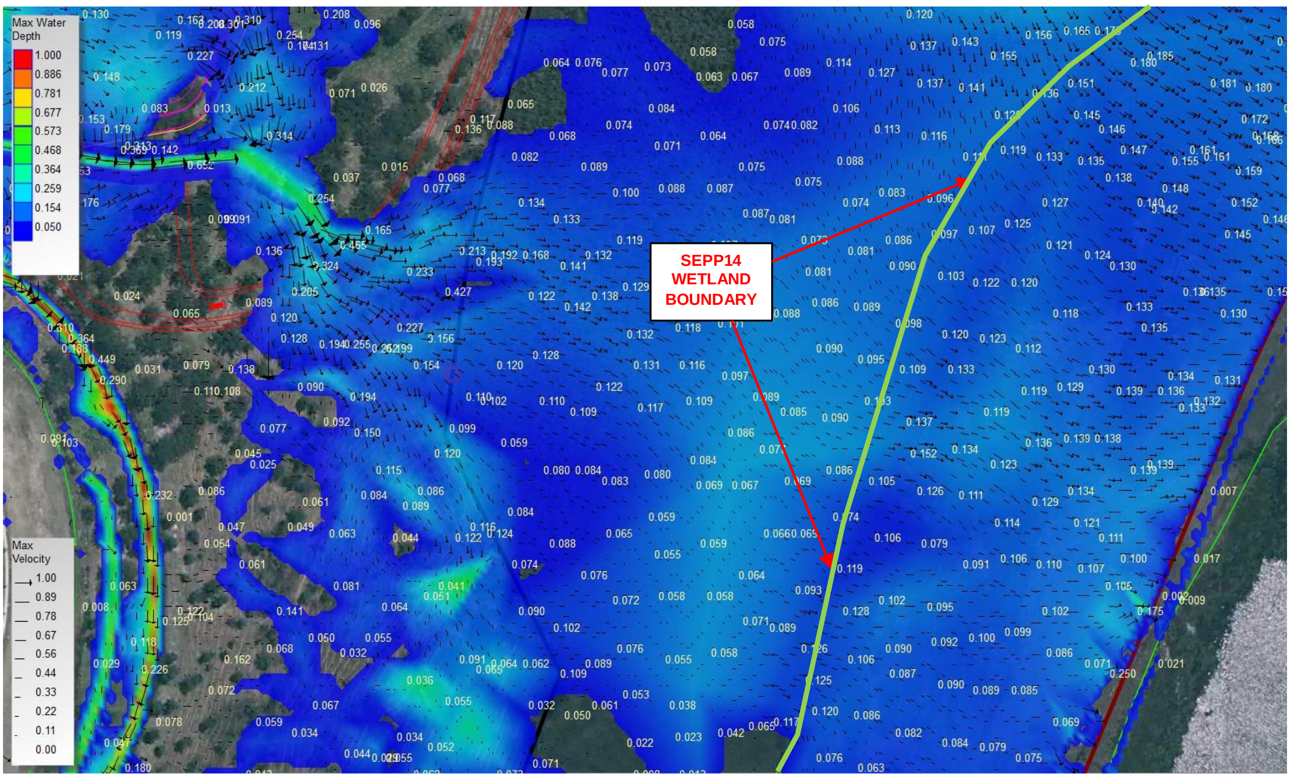

Figure 32 - Design Critical Flood Depths/Velocities (Velocity values displayed) At "Discharge Culvert B' – 2hr 100yr Storm, Current MLW Tailwater

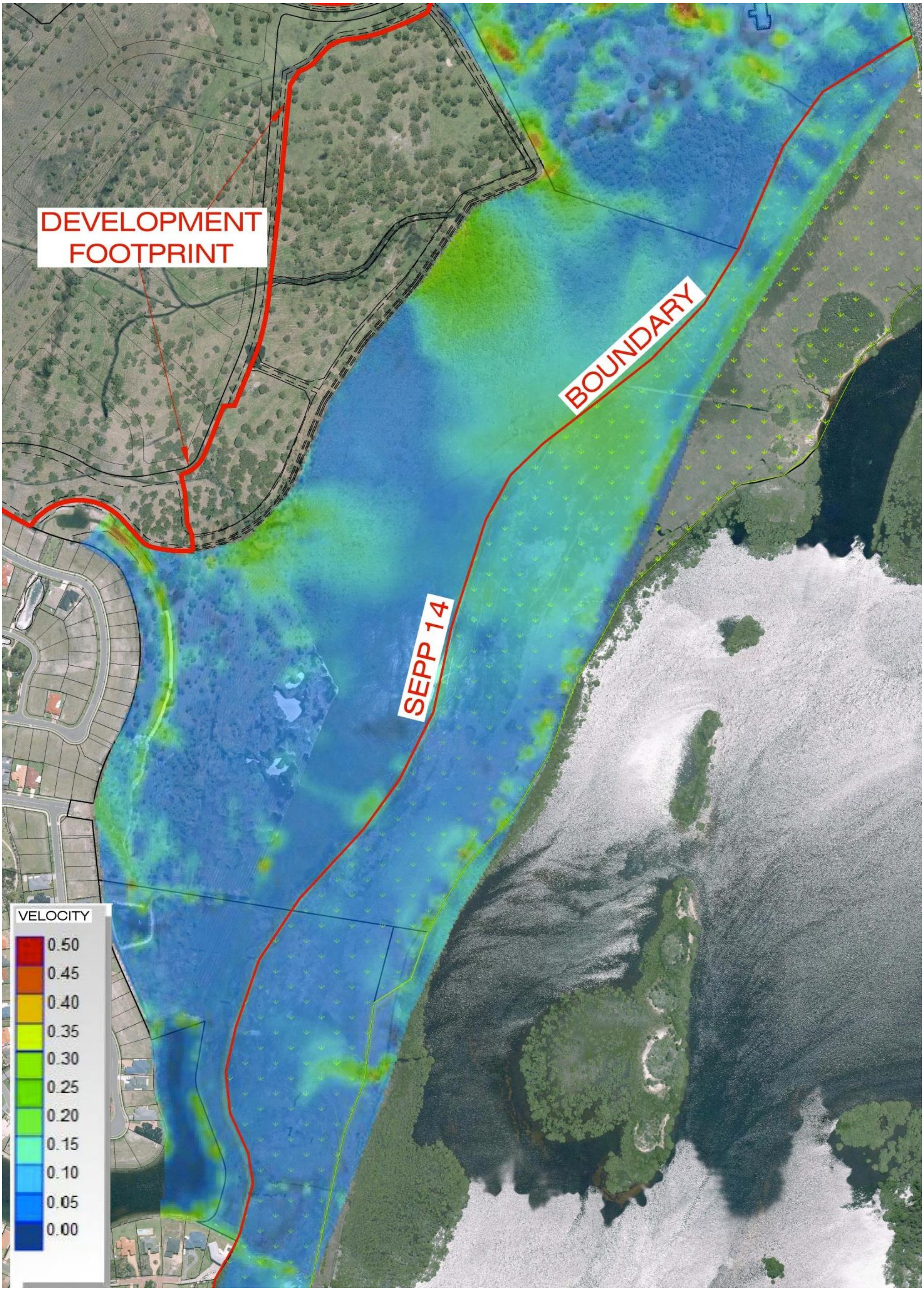

Figure 33 - Peak Flood Velocity Mapping – 2hr 100yr Storm, Current MLW Tailwater

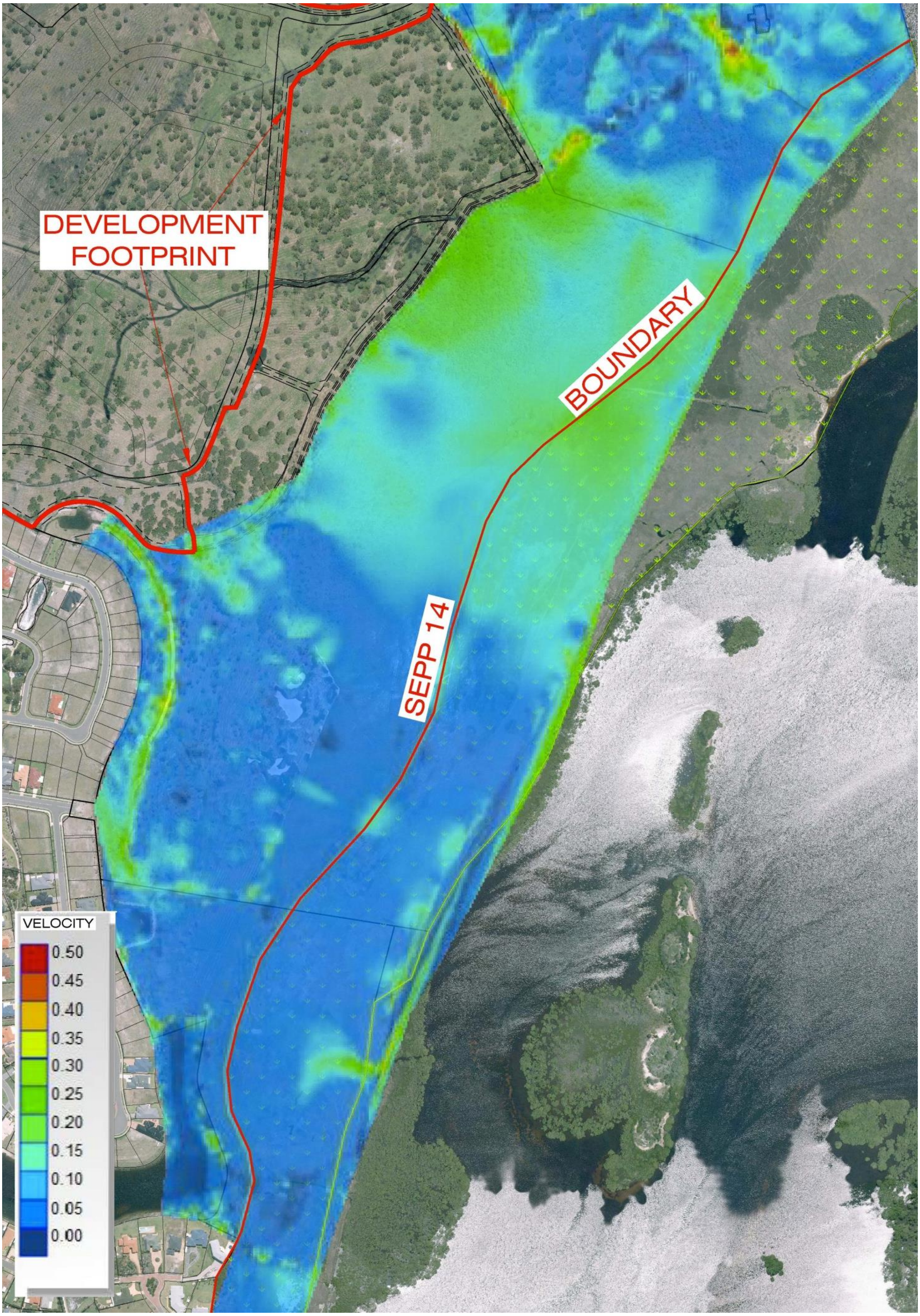

Figure 34 - Peak Flood Velocity Mapping – 2hr 100yr Storm, Current MLW Tailwater

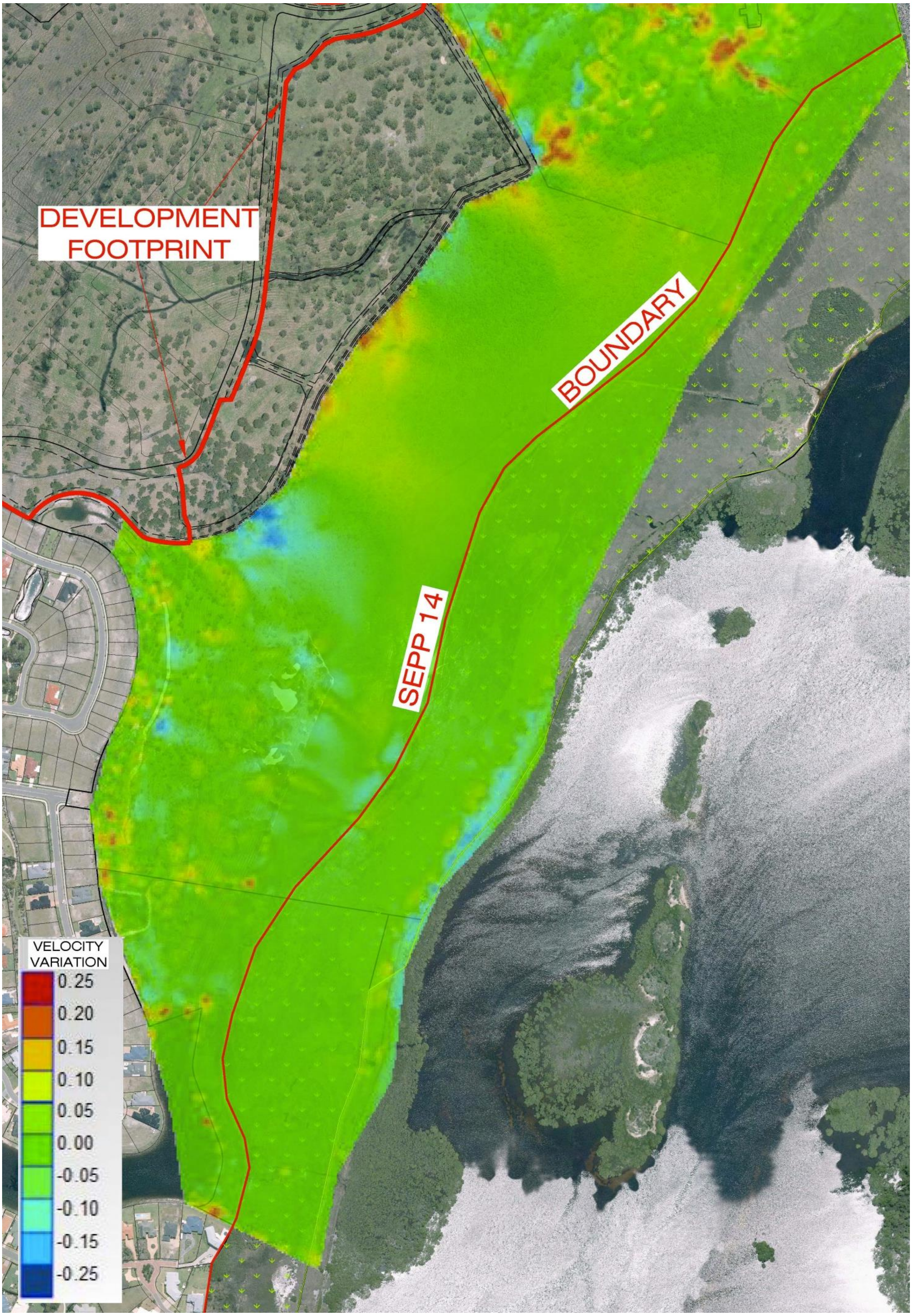

Figure 35 - Peak Flood Velocity Difference Mapping – 2hr 100yr Storm, Current MLW Tailwater

3.9.3 Impact on Flooding of Adjoining Land

It is important to ensure that the proposed development will not adversely affect adjoining properties by increasing the extent or frequency of flooding. A summary of the lands surrounding the development site is shown below;

3.9.3.1 Adjacent Western Lands

The development site is bordered by Myall Street to the West – this road is also the existing catchment watershed, and as such land to the west of the site will not be impacted by the Riverside development.

As discussed in Section 3.3.1, the two existing minor exceptions to this are two small culverts that drain the eastern roadside table drains to the west. Both these culverts will be retained, and will continue operate as they currently do. They cannot be removed as the existing table drains are too low to drain into the Riverside site, but the adjacent Riverside lands will be filled to drain away from the table drains, ensuring no additional flows will be directed west on top of existing flows – ie post development the watershed will be the edge of filling adjacent to the Myall Street table drain, and all Riverside flows will be directed east away from Myall Street.

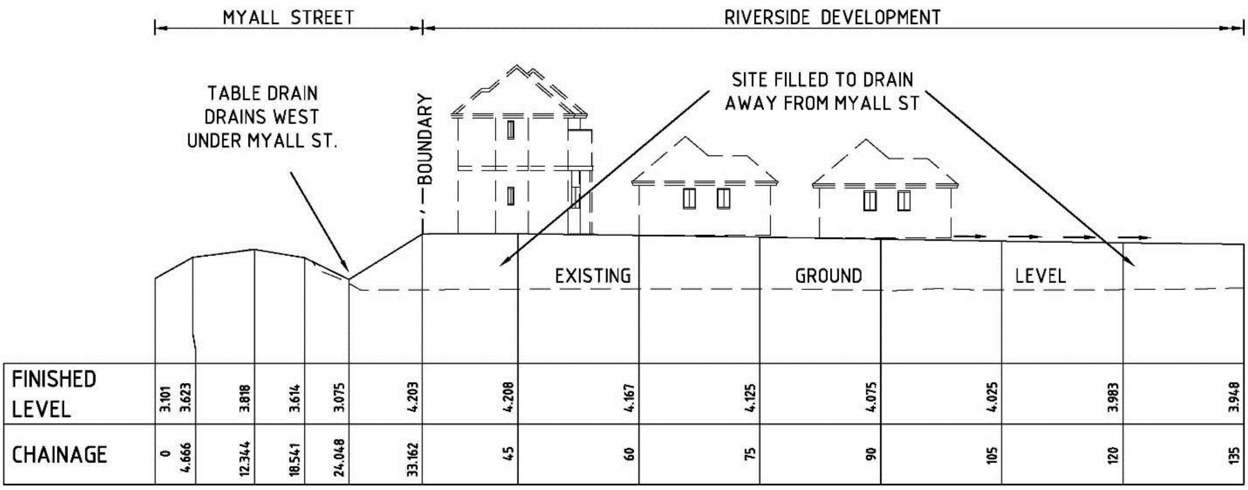

Figure 36 - Cross Section showing fill levels adjacent to Myall Street

3.9.3.2 Upstream Northern Catchment

The lands directly to the north of the Riverside site are significantly elevated and consists of low density development and bushland reserves. It has recently been rezoned by Council, prohibiting any further subdivision.

With the Riverside development footprint pulled further back again from Toonang Drive in this latest modification, there will be zero impact on Toonang Drive or beyond, as shown diagrammatically in the following two figures.

Review of the previous Riverside report incorrectly interpreted the presented results of this section, most likely due to a combination of a lack of understanding of the site extents, and insufficient diagram labelling. The additional annotation on these diagrams hopefully adequately demonstrates that the modelling indicates there is no discernible impacts on Toonang Drive and any lands upstream of the site.

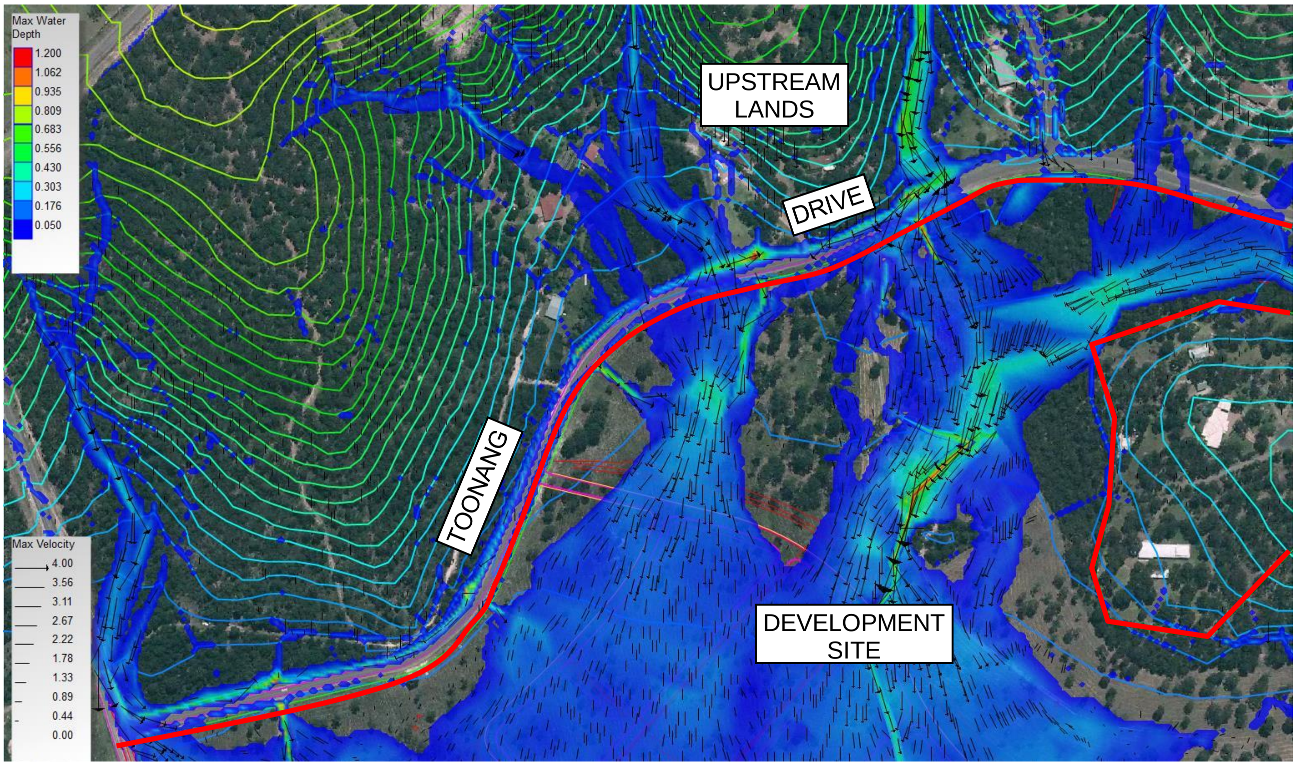

Figure 37 - Pre Development Critical Flood Depths At Existing Toonang Drive Culverts - 2hr 100yr Storm, 2100 5yr Tailwater

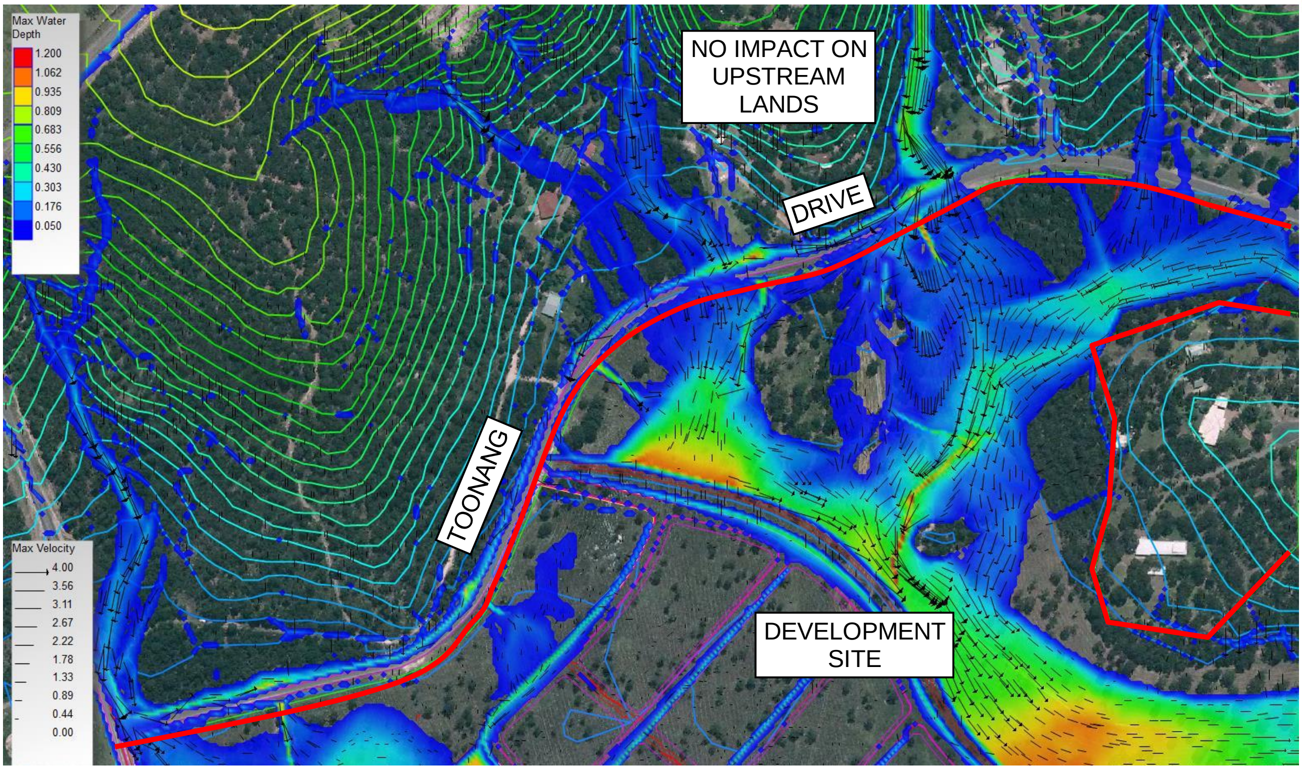

Figure 38 - Post Development Critical Flood Depths At Existing Toonanag Drive Culverts - 2hr 100yr Storm, 2100 5yr Tailwater

3.9.3.3 Adjacent Southern Lands

The existing development to the South drains to either the existing lake, or to existing surface drains running towards the wetland. It is not intended to discharge from the proposed Riverside development directly into the existing drains or lake in minor events. Some additional water will flow into these structures during larger events, and as such it will be important to ensure that the Riverside development does not increase water levels in the lake or wetland.

In order to assess the impact of the proposed development on adjoining development during more major storm events, 5yr and 20yr events were modelled using the current projection of the 2100 MHW tailwater level (1.4m AHD), and the 100yr event was modelled with the 5yr river flood level (2.0).

Table 7: Pre and Post Development 5yr Peak Flood Levels (m AHD)

	3hr		6hr		9hr		12hr	
Location	Pre	Post	Pre	Post	Pre	Post	Pre	Post
K (Bebo)	2.31	1.83	2.32	1.83	2.35	1.87	2.32	1.85
L (Lake)	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41
M (Lake)	1.41	1.40	1.41	1.41	1.41	1.41	1.41	1.40
N (Wetland)	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40

Table 8: Pre and Post Development 20yr Peak Flood Levels (m AHD)

	3hr		6hr		9hr		12hr	
Location	Pre	Post	Pre	Post	Pre	Post	Pre	Post
K (Bebo)	2.36	1.89	2.37	1.90	2.39	1.94	2.37	1.92
L (Lake)	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41
M (Lake)	1.41	1.40	1.41	1.41	1.41	1.41	1.41	1.41
N (Wetland)	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40

Table 9: Pre and Post Development 100yr Peak Flood Levels (m AHD)

	1hr		2hr		3hr		6hr	
Location	Pre	Post	Pre	Post	Pre	Post	Pre	Post
K (Bebo)	2.37	1.19	2.40	2.19	2.40	1.18	2.40	1.18
L (Lake)	1.16	1.16	2.15	2.15	1.16	1.16	1.16	1.16
M (Lake)	1.16	1.16	2.15	2.15	1.16	1.16	1.16	1.16
N (Wetland)	1.15	1.15	2.15	2.15	1.15	1.15	1.15	1.15

It can be seen in these results that in all cases the tailwater is the critical factor influencing flood levels in these scenarios. Ground levels in the wetland between the existing lake and the Myall River range from 0.5m to around 1.2m AHD. Under the adopted tailwater scenarios, the lake and river would become a continuous water body. The model results show that any variation in incoming flows will be absorbed by the shallow but broad connection to the river and substantial detention capacity available in the combined lake-river water body.

Furthermore, construction of the Southern Branch floodway (including the removal of the temporary haul road and diversion berm) will lower the flood level in this area, improving flood conditions for existing lots that currently discharge into this area.

The proposed Riverside development will not increase flood levels in the adjoining lake or wetland buffer area in these rainfall events, and as such will not have a negative impact on flood levels within the adjoining development in these events.

3.9.4 100 yr Maximum Flood Levels

In reference to the recently completed Lower Myall River and Myall Lakes Flood Study by WBM BMT and discussions with Great Lakes Council, it has been determined that it is appropriate to model two design storm / downstream tailwater level combinations in assessing 100yr Flood Planning Levels for Riverside Estate;

- 100yr Event 1; 100yr catchment storm with a 5yr 2100 river level of RL.2.15m AHD
- 100yr Event 2; 5yr catchment storm with a 100yr 2100 river level of RL.2.3m AHD

Both these tailwater levels have been revised from the last Riverside report (previously 2.0m and 2.8m AHD respectively). The combined worst case level will be adopted as the design flood level. Flood Planning Levels can then be determined by Council, usually by adding a further 0.5m freeboard.

As the last Riverside report determined the critical duration for the Flood Planning Level assessment was 2hrs, only this duration has been re-run for this report. For comparison purposes, the previously modelled results are presented next the current results.

General observations;

- Critical peak 100yr flood levels (and related Flood Planning Levels) are either similar, or lower than the previous report. In no case has the critical FPL increased,
- Fill levels are not limited by coverage of minor storm street drainage structures, rather than 100yr Flood Planning Levels,
- In all cases flood flows are restricted to designated floodways and roadways, with no lots being affected by either of the 100yr flood events,
- Complete removal of the ‘eco-tourist’ precinct has significantly reduced 100yr peak flows through the ‘River’ flow region between the Wetland and Monkey Jacket outlet,
- Loss of storage volumes in the wildlife corridor (above Stage 10 & 14) has increased peak flows into the existing Lake (under the ‘Bebo’ bridge) and though ‘Wetland 4’. This does not correlate to any noticeable increased impact on downstream flood levels, as there is significant flow and storage capacity available in both areas.

Table 10: Design State 100yr Peak Flood Levels (critical level highlighted)

	2hr		2hr (Previous Riverside Report)	
Location	Event 1	Event 2	Event 1	Event 2
Flood Level (m AHD)				
A	6.14	5.99	6.33 (+0.19)	6.18 (+0.19)
B	4.12	3.85	4.34 (+0.22)	4.05 (+0.20)
C	3.92	3.62	3.95 (+0.03)	3.68 (+0.06)
D	3.82	3.48	3.78 (+0.04)	3.50 (+0.02)
E	2.18	2.30	2.15 (-0.03)	2.80 (+0.50)
G	2.96	2.61	2.94 (-0.02)	3.00 (+0.39)
H	2.80	2.56	2.80 (-0.00)	2.93 (+0.37)
I	2.22	2.31	2.15 (-0.07)	2.81 (+0.50)
J	2.19	2.31	2.09 (-0.10)	2.81 (+0.50)
K	2.19	2.31	2.07 (-0.12)	2.80 (+0.50)
L	2.15	2.30	2.00 (-0.15)	2.80 (+0.50)
M	2.15	2.30	2.00 (-0.15)	2.80 (+0.50)
N	2.15	2.30	2.00 (-0.15)	2.80 (+0.50)
O	2.92	2.69	2.82 (-0.10)	2.99 (+0.30)
P	2.15	2.30	2.00 (-0.15)	2.80 (+0.50)
Peak Flow (m^3/s)				
Lake	6.6	3.5	4.5	2.3
Creek	4.9	3.0	3.5	2.9
River	1.3	0.8	11	1.3
Wetland 1	2.9	2.0	4.2	1.8
Wetland 2	11.9	5.2	12.0	6.0
Wetland 3	5.4	4.5	5.2	6.5
Wetland 4	6.5	4.0	4.5	5.0
Wetland 5	7.6	4.0	7.5	5.5
Wetland 6	13.6	10.0	14.0	10
Overall Discharge	49	32	38	35

To summarise the peak flood levels, the following plan depicts maximum 100yr flood levels across the Riverside site. Flood Planning Levels should be set 0.5m above these levels.

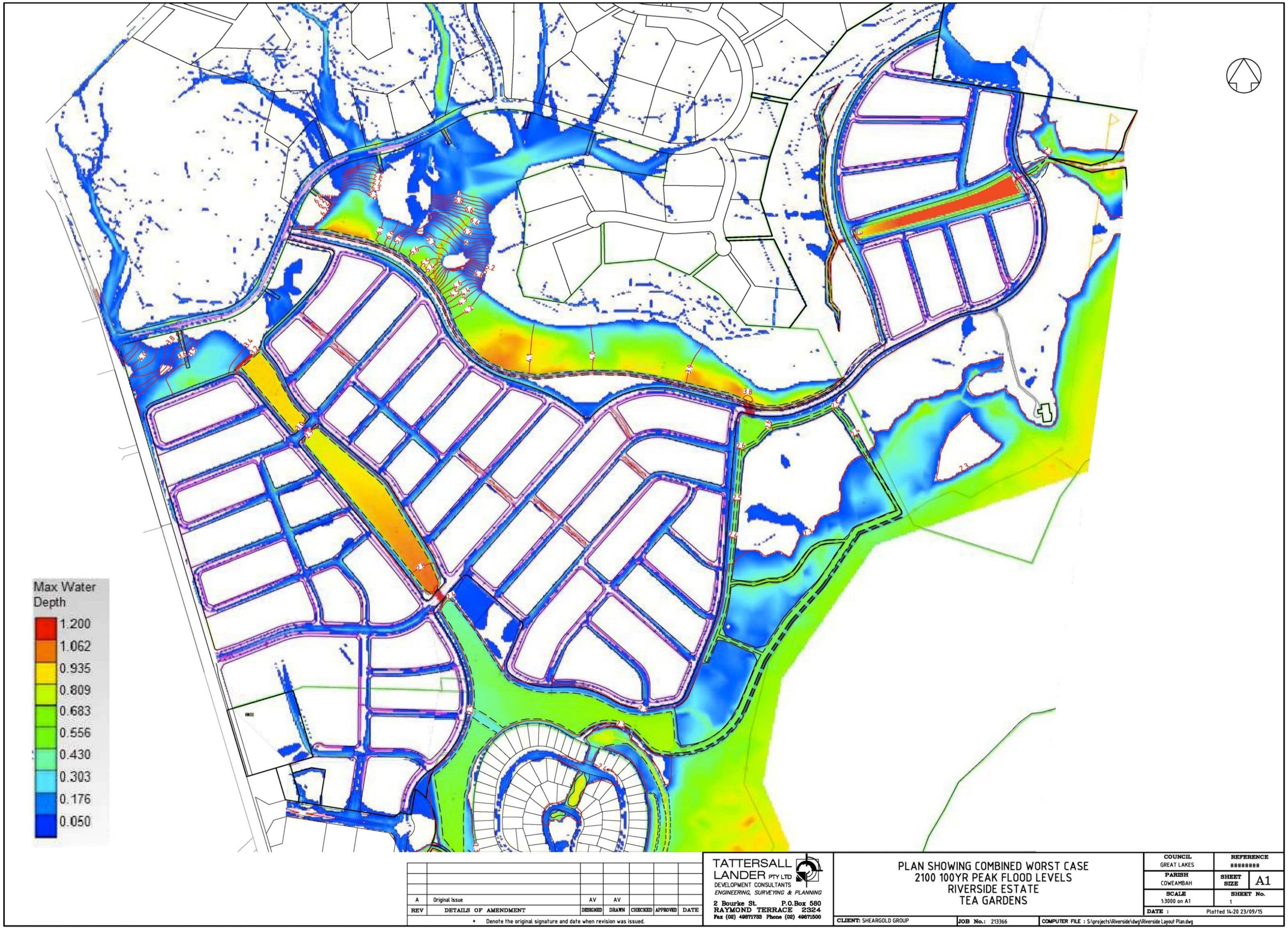

Figure 39 - Combined 2100 100yr Peak Flood Levels

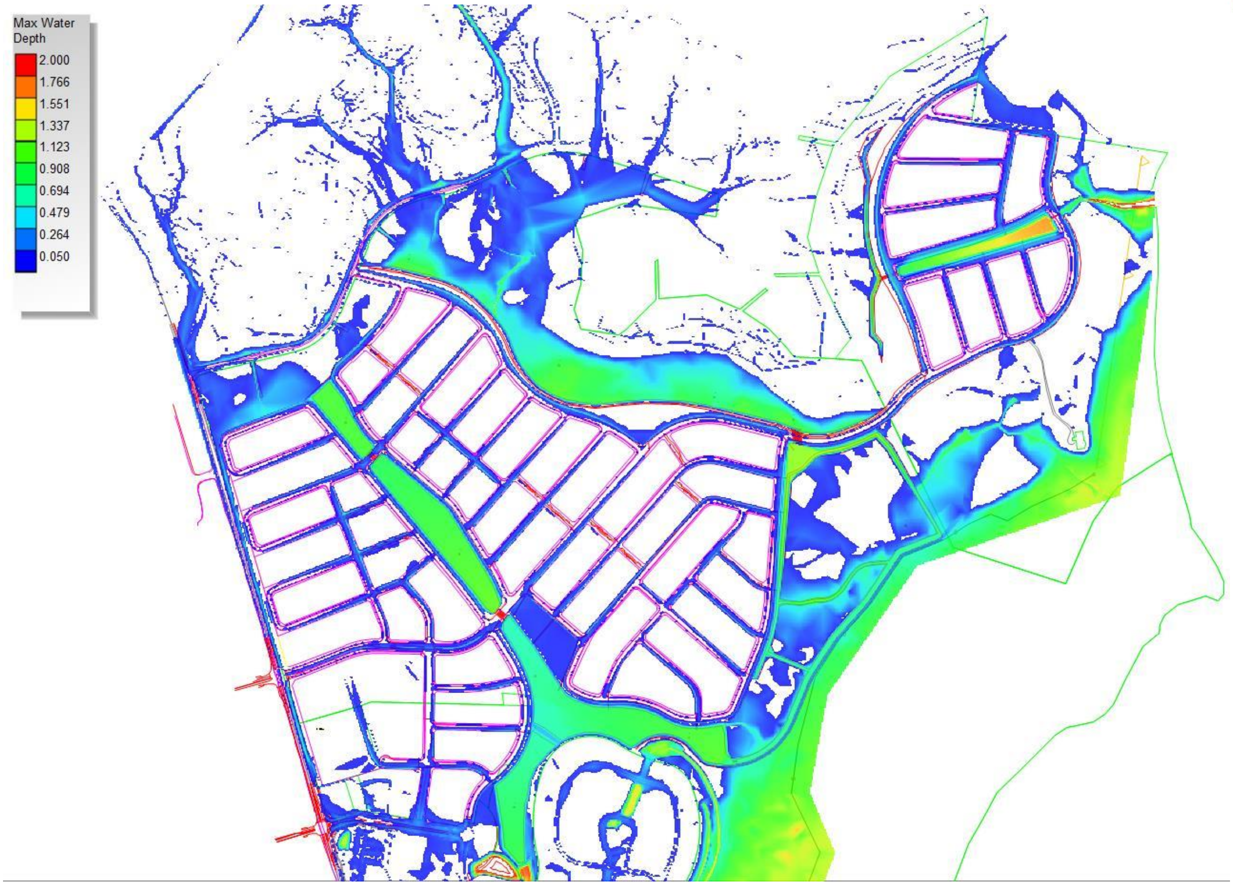

Figure 40 - Flood Depths Across the Developed Riverside Site - Event 1 Critical 2hr Storm – 100yr Storm, 2100 5yr Tailwater

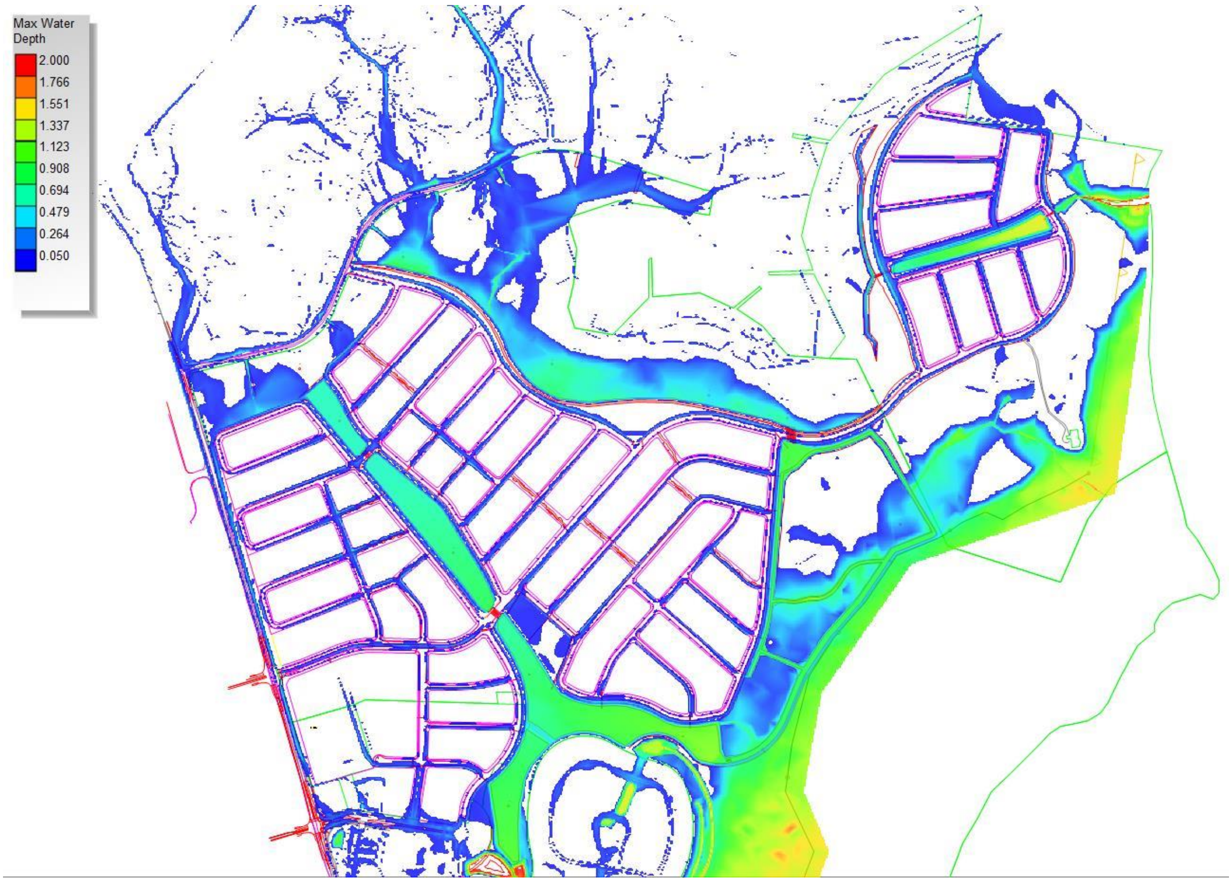

Figure 41 - Flood Depths Across the Developed Riverside Site – Event 2 Critical 2hr Storm – 5yr Storm, 2100 100yr Tailwater

3.9.5 Sensitivity Analysis

As with all modelling techniques, results are reliant on the many variables the modeller enters into the computer model. The adoption of the 2D modelling approach makes results far less reliant on the various generalisations necessary in a 1D model regarding catchment interpretation and representation, or flow conveyance profiles (especially relevant with the variable widths and grades of the floodways in the Riverside development).

There are still a range of other factors that can influence results, including rainfall/infiltration losses, hydraulic roughness, model grid size and potential variation in rainfall intensities (as possibly increased by Climate Change for example).

Justification of various adopted modelling parameters has been explained in earlier sections of this report. While all attempts are made to model with realistic model inputs, or to take conservative assumptions where there is some uncertainty, it is still worth understanding how sensitive model results are to various model inputs.

The following sections present the variation in Developed State 100yr peak flood level results for the critical two hour duration events for a range of different modelling variables.

3.9.5.1 Sensitivity to Initial Rainfall Losses

As described in Section 3.5, the 10mm/hr initial loss and 5mm/hr continuing loss were adopted per the 'Loam Soils' parameters adopted and reviewed as appropriate for the site in previous Cardno reports. It is acknowledged that there are some areas of clay soils within the model area, but equally there are areas of Sand Soils (Cardno applied significantly higher 50mm initial loss and 25mm/hr continuing loss to this soil type). It is considered that the adopted parameters are appropriate and are a conservative approximation of average soil conditions within the model area.

The critical 2hr Developed State 100yr Flood Planning Level models were re-run after varying initial losses to both 0mm and 20mm. While it is considered the adopted Initial Loss parameters are appropriate to best represent site conditions, the 100yr peak flood level models could be described as 'somewhat' sensitive to the Initial Loss parameter, with the maximum variation of +0.07m/-0.10m seen in the East-West diversion channel and +0.07m/-0.12m in the West Branch.

Table 11: Critical 100yr 2hr Peak Flood Level Initial Loss Sensitivity Tests

	Initial Loss = 0mm/hr		Initial Loss = 10mm/hr (Base Scenario)		Initial Loss = 20mm/hr	
Location	Event 1	Event 2	Event 1	Event 2	Event 1	Event 2
Flood Level (m AHD)						
A	6.15 (+0.01)	6.03 (+0.04)	6.14	5.99	6.11 (-0.03)	5.92 (-0.07)
B	4.17 (+0.05)	3.93 (+0.08)	4.12	3.85	4.05 (-0.07)	3.76 (-0.09)
C	3.99 (+0.07)	3.71 (+0.09)	3.92	3.62	3.84 (-0.08)	3.53 (-0.09)
D	3.89 (+0.07)	3.58 (+0.10)	3.82	3.48	3.72 (-0.10)	3.37 (-0.11)
E	2.18 (-0.00)	2.30 (-0.00)	2.18	2.30	2.17 (-0.01)	2.30 (-0.00)
G	3.03 (+0.07)	2.69 (+0.08)	2.96	2.61	2.84 (-0.12)	2.53 (-0.08)
H	2.84 (+0.04)	2.61 (+0.05)	2.80	2.56	2.74 (-0.06)	2.49 (-0.07)
I	2.23 (+0.01)	2.32 (+0.01)	2.22	2.31	2.21 (-0.01)	2.31 (-0.00)
J	2.19 (-0.00)	2.31 (-0.00)	2.19	2.31	2.18 (-0.01)	2.30 (-0.01)
K	2.19 (-0.00)	2.31 (-0.00)	2.19	2.31	2.18 (-0.01)	2.30 (-0.01)
L	2.15 (-0.00)	2.30 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
M	2.15 (-0.00)	2.30 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
N	2.15 (-0.00)	2.30 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
O	2.94 (+0.02)	2.76 (+0.07)	2.92	2.69	2.89 (-0.03)	2.57 (-0.12)
P	2.15 (-0.00)	2.80 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)

It should be noted that as the controlling factor for fill levels is coverage over street drainage and the minimum outlet discharge level of 1.4m AHD, rather than 100yr flood levels. As such, even in the 'worst case' 0mm Initial Loss, the critical 100yr flood level does not encroach onto residential lots, as can be seen in the figure below.

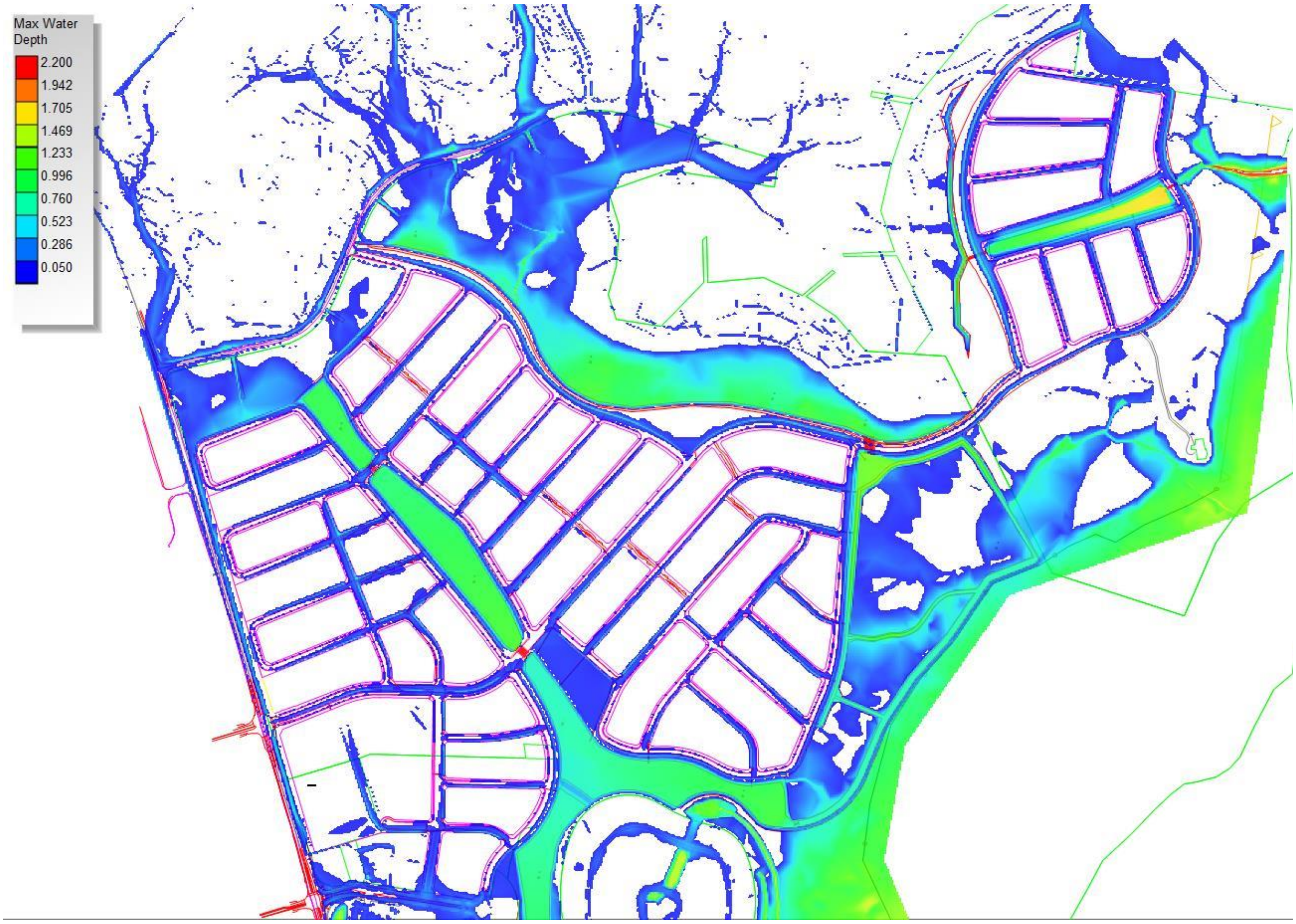

Figure 42 - Flood Depths Across the Developed Riverside Site – 0mm Initial Loss - Event 1 Critical 2hr Storm – 100yr Storm, 2100 5yr Tailwater

3.9.5.2 Sensitivity to Continuing Rainfall Losses

The 2hr Developed State 100yr Flood Planning Level models were re-run with Continuing Losses of 0mm and 10mm. As seen below, the 100yr peak flood level models are relatively insensitive to the Continuing Loss parameter, with the maximum variation +/- 0.03m seen in the East-West diversion channel. While it is considered the adopted Continuing Loss parameters are appropriate to best represent site conditions, the table below demonstrates that this variable has little impact on the critical 100yr flood modelling results.

Table 12: Critical 100yr 2hr Peak Flood Level Continuing Loss Sensitivity Tests

	Continuing Loss = 0mm/hr		Continuing Loss = 5mm/hr (Base Scenario)		Continuing Loss = 10mm/hr	
Location	Event 1	Event 2	Event 1	Event 2	Event 1	Event 2
Flood Level (m AHD)						
A	6.15 (+0.01)	6.00 (+0.01)	6.14	5.99	6.13 (-0.01)	5.98 (-0.01)
B	4.13 (+0.01)	3.88 (+0.03)	4.12	3.85	4.10 (-0.02)	3.83 (-0.02)
C	3.95 (+0.03)	3.65 (+0.03)	3.92	3.62	3.89 (-0.03)	3.59 (-0.03)
D	3.85 (+0.03)	3.51 (+0.03)	3.82	3.48	3.79 (-0.03)	3.44 (-0.04)
E	2.18 (-0.00)	2.30 (-0.00)	2.18	2.30	2.17 (-0.01)	2.30 (-0.00)
G	2.97 (+0.01)	2.62 (+0.01)	2.96	2.61	2.93 (-0.03)	2.59 (-0.02)
H	2.81 (+0.01)	2.57 (+0.01)	2.80	2.56	2.78 (-0.02)	2.54 (-0.02)
I	2.22 (-0.00)	2.32 (+0.01)	2.22	2.31	2.22 (-0.00)	2.31 (-0.00)
J	2.19 (-0.00)	2.31 (-0.00)	2.19	2.31	2.19 (-0.00)	2.31 (-0.00)
K	2.19 (-0.00)	2.31 (-0.00)	2.19	2.31	2.19 (-0.00)	2.31 (-0.00)
L	2.15 (-0.00)	2.30 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
M	2.15 (-0.00)	2.30 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
N	2.15 (-0.00)	2.30 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
O	2.93 (+0.01)	2.71 (+0.02)	2.92	2.69	2.91 (-0.01)	2.67 (-0.02)
P	2.15 (-0.00)	2.30 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)

3.9.5.3 Sensitivity to Hydraulic Roughness

When modelling to match existing conditions, Hydraulic Roughness (Manning ‘’n’) values are often used as a calibration parameter, and adjusted within accepted ranges to calibrate results to measured data. As such model results can be sensitive to chosen parameters. In design flood models such as the Riverside models, calibration is not possible, and Manning ‘n’ values are adopted in line with accepted ranges. Section 3.5 detailed the various Mannings Roughness values adopted for various Landuse Categories within the model. These were assessed and considered appropriate in the previous peer review process.

Sensitivity test were undertaken by re-running the critical 2hr Developed State 100yr Flood Planning Level Flood models and applying +25% and -25% to the adopted Manning ‘n’ values. The table below shows the results are fairly insensitive to adopted Manning ‘n’ values;

Table 13: Critical 100yr 2hr Peak Flood Level Hydraulic Roughness Sensitivity Tests

	Manning ‘n’ -25%		Base Scenario		Manning ‘n’ +25%	
Location	Event 1	Event 2	Event 1	Event 2	Event 1	Event 2
Flood Level (m AHD)						
A	6.11 (-0.03)	6.97 (-0.02)	6.14	5.99	6.16 (+0.02)	6.00 (+0.01)
B	4.06 (-0.06)	4.82 (-0.03)	4.12	3.85	4.16 (+0.04)	3.88 (+0.03)
C	3.90 (-0.02)	3.60 (-0.02)	3.92	3.62	3.94 (+0.02)	3.64 (+0.02)
D	3.84 (+0.02)	3.49 (+0.01)	3.82	3.48	3.80 (-0.02)	3.47 (-0.01)
E	2.17 (-0.01)	2.30 (-0.00)	2.18	2.30	2.18 (-0.00)	2.30 (-0.00)
G	2.96 (-0.00)	2.62 (+0.01)	2.96	2.61	2.94 (-0.02)	2.62 (+0.01)
H	2.80 (-0.00)	2.57 (+0.01)	2.80	2.56	2.79 (-0.01)	2.56 (-0.00)
I	2.21 (-0.01)	2.32 (+0.01)	2.22	2.31	2.23 (+0.01)	2.32 (+0.01)
J	2.18 (-0.01)	2.31 (-0.00)	2.19	2.31	2.19 (-0.00)	2.31 (-0.00)
K	2.19 (-0.00)	2.31 (-0.00)	2.19	2.31	2.20 (+0.01)	2.31 (-0.00)
L	2.15 (-0.00)	2.30 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
M	2.15 (-0.00)	2.30 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
N	2.15 (-0.00)	2.30 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
O	2.92 (-0.00)	2.69 (-0.00)	2.92	2.69	2.92 (-0.00)	2.69 (-0.00)
P	2.15 (-0.00)	2.30 (-0.00)	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)

3.9.5.4 Sensitivity to 2D model Grid Spacing

The size of the modelling grid utilised in a 2D flood model is an important factor in the overall behaviour of a 2D model. The chosen grid size needs to be small enough to adequately identify and reflect the important features and of the underlying DTM, while large enough to maintain a reasonable model run time, results file size and model stability (regarding stability - cell depth to width should remain within acceptable ranges as current technology solves in 2D, not 3D). It is an inherent characteristic of a 2D model that to halve the grid size is to multiple run times by a factor of eight.

The previous Riverside model was undertaken with a 5m grid spacing. This was considered appropriate as the model is intended to assess the trunk drainage network, which features channel widths in the order of 30-70m wide. It is true that this may not accurately represent flow behaviour at individual street level, but these areas of the model are really only included as catchment generation area (as demonstrated by the modelling results, 100yr flows are contained within the defined flood channels). To accurately model street level details, minor system street drainage, inter-allotment drains, driveway crossings of swales etc would all need to be included, which is considered beyond the scope of this report.

For this report fewer model runs were done as only previously determined critical durations were re-modelled, allowing a more time-consuming 2.5m grid spacing to be adopted.

The following table shows the difference the adopted grid size makes to the critical 2hr Developed State 100yr Flood Planning Level Flood models. Analysis of the results shows not so much an overall impact on the peak flood levels, but local variances observed at locations in close proximity to the inlets of the highest energy culverts – reflecting the way a higher grid density provides both a better ‘resolution’ in areas of higher gradient, and also more accurately represents the 1D-2D headwall connections.

Table 14: Critical 100yr 2hr Peak Flood Level Grid Size Sensitivity Tests

	2.5m Grid (Base Scenario)		5m Grid	
Location	Event 1	Event 2	Event 1	Event 2
Flood Level (m AHD)				
A	6.14	5.99	6.14 (-0.00)	5.98 (-0.01)
B	4.12	3.85	4.13 (+0.01)	3.86 (+0.01)
C	3.92	3.62	3.90 (-0.02)	3.62 (-0.00)
D	3.82	3.48	3.70 (-0.12)	3.40 (-0.08)
E	2.18	2.30	2.18 (-0.00)	2.30 (-0.00)
G	2.96	2.61	2.89 (-0.07)	2.57 (-0.04)
H	2.80	2.56	2.77 (-0.03)	2.53 (-0.03)
I	2.22	2.31	2.22 (-0.00)	2.31 (-0.00)
J	2.19	2.31	2.19 (-0.00)	2.31 (-0.00)
K	2.19	2.31	2.19 (-0.00)	2.31 (-0.00)
L	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
M	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
N	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)
O	2.92	2.69	2.90 (-0.02)	2.63 (-0.06)
P	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)

3.9.5.5 Sensitivity to the location of the Downstream Boundary

The peer review of the previous modelling suggested that the location of the downstream boundary should be shifted further towards the river, and ideally be placed in the river channel. This would presumably be to assess the impact any flood gradient across the wetland would have on the results.

It was expected that this would have little impact on 100yr flood modelling results, as ground levels across the wetland area between the adopted Head Boundary and the river range from between 0.5m-1.2m AHD, which are significantly flooded by all design tailwater conditions.

Additionally, for the 0.5m tailwater existing case scenarios, the results are relative (comparing pre and post development), and with each model having the same boundary location would not impact on results.

To demonstrate the appropriateness of the selected boundary head condition, the critical 2hr Developed State 100yr Flood Planning Level Flood model was re-run with the boundary Head Boundary shifted to the edge of the DTM adjacent to the river. This included an extra 500,000sq.m of downstream wetland to the model.

No change was seen in the peak flood level results, demonstrating that the originally selected location for the downstream Head Boundary is appropriate.

Table 15: Critical 100yr 2hr Peak Flood Level Downstream Boundary Sensitivity Tests

	Downstream Boundary @ wetland interface (Base Scenario)		Downstream Boundary @ River	
Location	Event 1	Event 2	Event 1	Event 2
Flood Level (m AHD)				
A	6.14	5.99	6.14	5.99
B	4.12	3.85	4.12	3.85
C	3.92	3.62	3.92	3.61
D	3.82	3.48	3.82	3.46
E	2.18	2.30	2.18	2.30
G	2.96	2.61	2.96	2.63
H	2.80	2.56	2.80	2.57
I	2.22	2.31	2.22	2.31
J	2.19	2.31	2.19	2.31
K	2.19	2.31	2.19	2.31
L	2.15	2.30	2.15	2.30
M	2.15	2.30	2.15	2.30
N	2.15	2.30	2.15	2.30
O	2.92	2.69	2.92	2.69
P	2.15	2.30	2.15	2.30

3.9.5.6 Sensitivity to Design Storm Intensity Increases

At present it has become fairly common in the industry to assess the effects of 10%, 20% and 30% rainfall intensity increases on flood modelling results, to cover the possible future impacts of climate change on rainfall intensities. This is in contrast to advice received via Great Lakes Council from the Bureau of Meteorology and Office of Environment and Heritage is that there is currently insufficient evidence to justify including any intensity increases into FPL assessments at present.

The Department of Environment, Climate Change and Water's Flood Risk Management Guide states; *"The (0.5m) freeboard provides a relatively small allowance to accommodate some of the projected increases in rainfall intensity from flood-producing storm events associated with climate change, which have currently not been accurately quantified."*

The critical duration 100yr flood models were re-run to assess the sensitivity of the Flood Planning Level. The results from these models are listed below.

Table 16: Critical 100yr 2hr Peak Flood Level Rainfall Intensity Sensitivity Tests

	Existing Intensities (Base Scenario)		+ 10% Intensities		+ 20% Intensities		+ 30% Intensities	
Location	Event 1	Event 2	Event 1	Event 2	Event 1	Event 2	Event 1	Event 2
Flood Level (m AHD)								
A	6.14	5.99	6.17 (+0.03)	6.02 (+0.03)	6.19 (+0.05)	6.05 (+0.06)	6.22 (+0.08)	6.07 (+0.08)
B	4.12	3.85	4.17 (+0.05)	3.90 (+0.05)	4.23 (+0.11)	3.94 (+0.09)	4.28 (+0.16)	3.99 (+0.14)
C	3.92	3.62	3.99 (+0.07)	3.67 (+0.05)	4.06 (+0.14)	3.72 (+0.10)	4.13 (+0.21)	3.77 (+0.15)
D	3.82	3.48	3.89 (+0.07)	3.54 (+0.06)	3.97 (+0.15)	3.59 (+0.11)	4.04 (+0.22)	3.64 (+0.16)
E	2.18	2.30	2.18 (-0.00)	2.30 (-0.00)	2.19 (+0.01)	2.30 (-0.00)	2.19 (+0.01)	2.30 (-0.00)
G	2.96	2.61	3.05 (+0.09)	2.67 (+0.06)	3.15 (+0.19)	2.74 (+0.13)	3.24 (+0.28)	2.79 (+0.18)
H	2.80	2.56	2.86 (+0.06)	2.60 (+0.04)	2.92 (+0.12)	2.65 (+0.09)	2.98 (+0.18)	2.69 (+0.13)
I	2.22	2.31	2.23 (+0.01)	2.32 (+0.01)	2.25 (+0.03)	2.32 (+0.01)	2.27 (+0.05)	2.33 (+0.02)
J	2.19	2.31	2.20 (+0.01)	2.31 (-0.00)	2.20 (+0.01)	2.31 (-0.00)	2.21 (+0.02)	2.31 (-0.00)
K	2.19	2.31	2.20 (+0.01)	2.31 (-0.00)	2.21 (+0.02)	2.31 (-0.00)	2.22 (+0.03)	2.32 (+0.01)
L	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)	2.15 (-0.00)	2.30 (-0.00)	2.15 (-0.00)	2.30 (-0.00)
M	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)	2.15 (-0.00)	2.30 (-0.00)	2.15 (-0.00)	2.30 (-0.00)
N	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)	2.15 (-0.00)	2.30 (-0.00)	2.15 (-0.00)	2.30 (-0.00)
O	2.92	2.69	2.96 (+0.04)	2.75 (+0.06)	2.99 (+0.07)	2.81 (+0.12)	3.02 (+0.10)	2.86 (+0.17)
P	2.15	2.30	2.15 (-0.00)	2.30 (-0.00)	2.15 (-0.00)	2.30 (-0.00)	2.15 (-0.00)	2.30 (-0.00)

Analysis of these results shows that the highest level increases are up to 0.28m increase in maximum 100yr flood level in the higher sections of the site. In the lower sections of the catchment where tailwater factors dominate the flood planning levels, the impact of the intensity increase is negligible.

It should be noted that as the controlling factor for fill levels on the Riverside site is depth of coverage over street drainage, rather than 100yr flood levels. Even the 'worst case' 30% rainfall intensity increase are well within the Councils 0.5m adopted freeboard applied to Flood Planning Levels, and will not result in the critical 100yr flood level encroaching onto residential lots, as can be seen in the figure below.

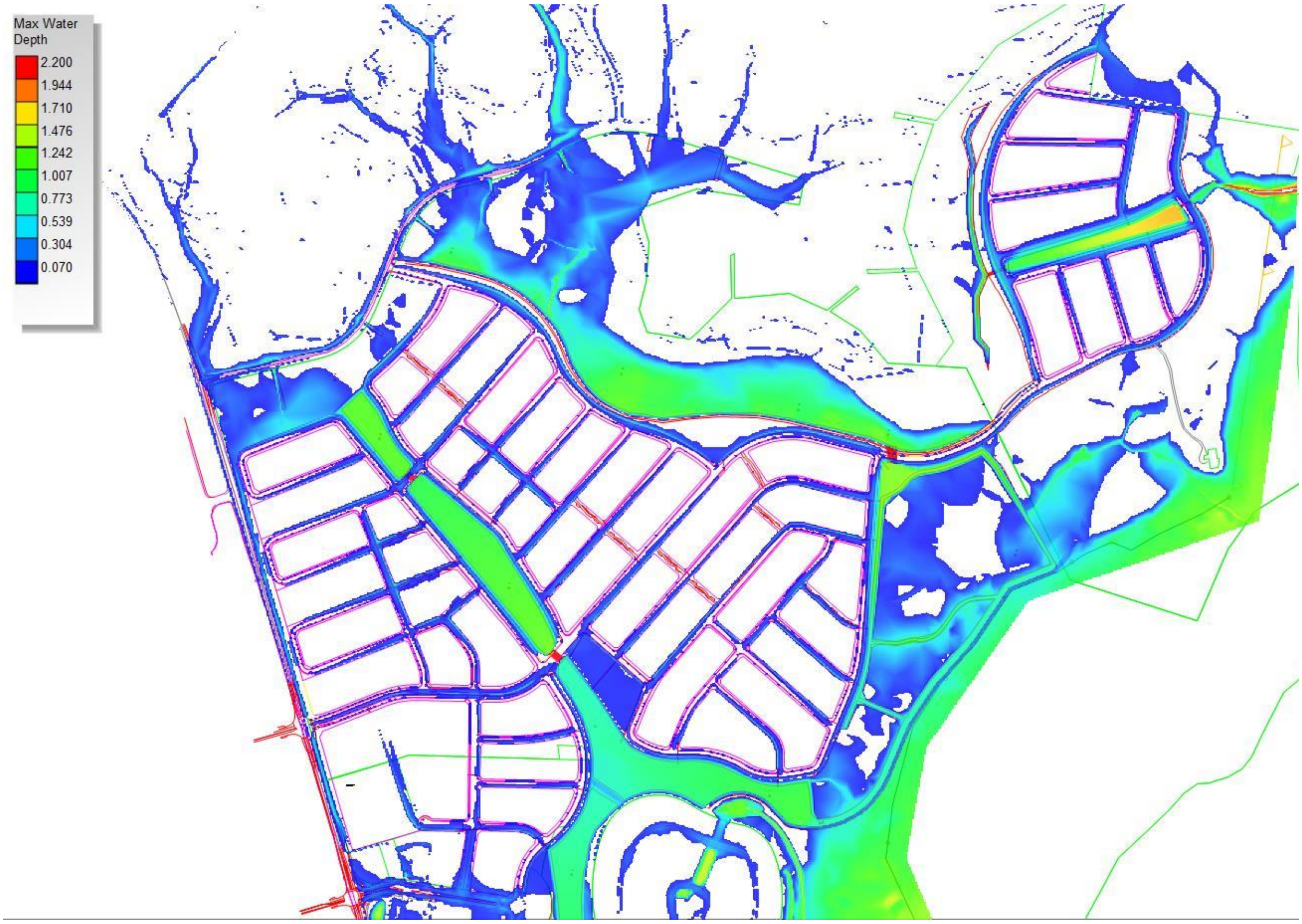

Figure 43 - Flood Depths Across the Developed Riverside Site – Intensities +30% - Event 1 Critical 2hr Storm – 100yr Storm, 2100 5yr Tailwater

The results listed above can be referenced by Great Lakes Council if Flood Planning Levels need to be adjusted in the future due to policy changes in relation to climate change induced rainfall intensity increases.

3.9.6 Public Safety Assessment

Public safety is an important consideration near stormwater management devices. Floodways though urban environments can represent a significant safety risk during storms and times of flood. Their close proximity to residential housing can result in people entering the floodways (either deliberately or by accident). It is important that a proposed floodway is safe by design.

The floodways in the proposed Riverside development include several features that will ensure public safety;

- Well defined edges – with the inclusion of bio-filters into the streetscape, the standard road profiles adjoining the floodways will include a 3.5m wide vegetated bio-filter between the roadway and the floodway. This vegetation will include tree plantings and dense macrophyte plantings, so even under major flood conditions the biofilters will provide a clear visual and tactile delineation between the roadway and the deeper floodway area,

- Alternate Routes – Generally speaking the grid-like street pattern provides alternative access routes if a particular road crossing becomes flooded by extreme flows or culvert blockages. This should ensure there is always another safe route, and pedestrians and vehicles are not forced to cross flooded roadways,

- Revegetation – it is proposed to utilise the base of the floodways as infiltration/groundwater recharge areas, and the central 20m also being densely reforested, including larger tree species. People entering a flooded floodway will be able to use the vegetation to assist with orientation and stability as they attempt to exit the water.

- Flat grades/wide sections – conforming to the character of the existing site, the floodways will feature very flat grades. This necessitates a wide cross section in order to provide flow capacity. Combined with the high roughness values due to the level of revegetation proposed, the resulting low velocities and depths mean the floodways are inherently safe, in the case that people or vehicles enter them during times of flood.

The following detailed analysis checks safety within the floodways by analysing the velocity-depth product.

3.9.6.1 Flood Hazard Definitions

Flood Hazard categories have been defined in the NSW Government's Floodplain Management Manual (2005);

- High Hazard – possible danger to personal safety, evacuation by truck difficult; able-bodied adults would have difficulty wading to safety; potential for significant structural damage to buildings.
- Low Hazard – should it be necessary, trucks could evacuate people and their possessions; able bodied adults would have little difficulty wading to safety.

The following figures from the Floodplain Management Manual can be used to determine the hazard within floodwaters. The Flood Hazard Mapping presented in this report is prepared specifically in relation to Figure L2 as shown below.

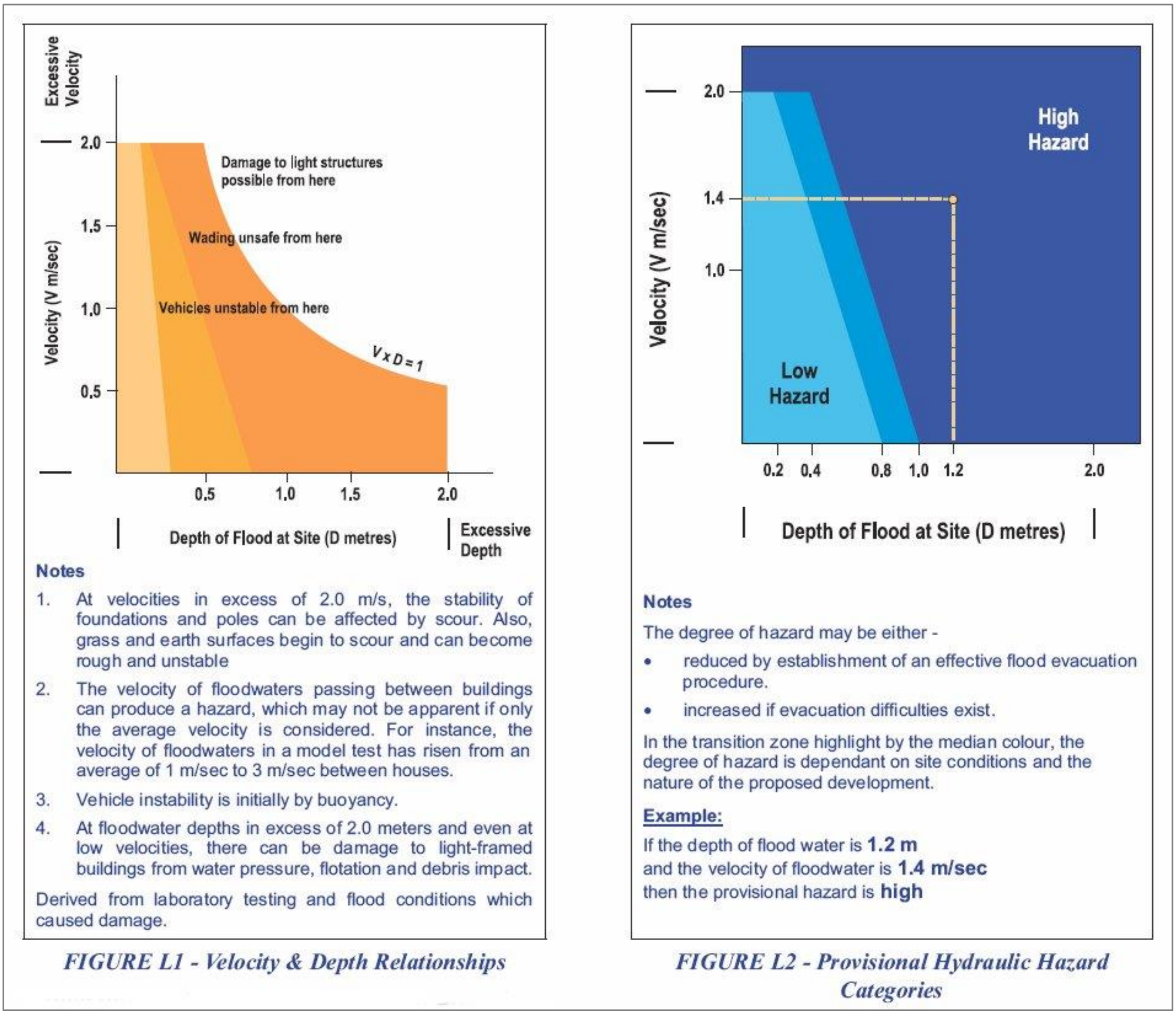

Figure 44 - Hazard Determination (NSW Government, 2005)

3.9.6.2 Minor Storm Assessment

Great Lakes Council's Design Specifications state the minor system in a residential area should be designed for a 5yr ARI event. In storm events up to this limit the design should "provide convenience and safety for pedestrians and traffic….by controlling those flows within prescribed limits".

The critical storm event hazard mapping below shows that High and Intermediate hazard levels are non-existent within the development site, other than within designated drainage channels.

Figure 45 - Critical Flood Hazard Mapping Across the Developed Riverside Site – 2hr 5yr Storm, 2100 MHWM Tailwater

3.9.6.3 Major Storm Assessment

Great Lakes Council's Design Specifications state the major structures should be designed for a 100yr ARI event. Critical 100yr storm event Hazard Mapping is shown below. High and Intermediate hazard levels are only seen in designated drainage channels, where egress is not required during a flood event. No road crossings are affected by dangerous flows.

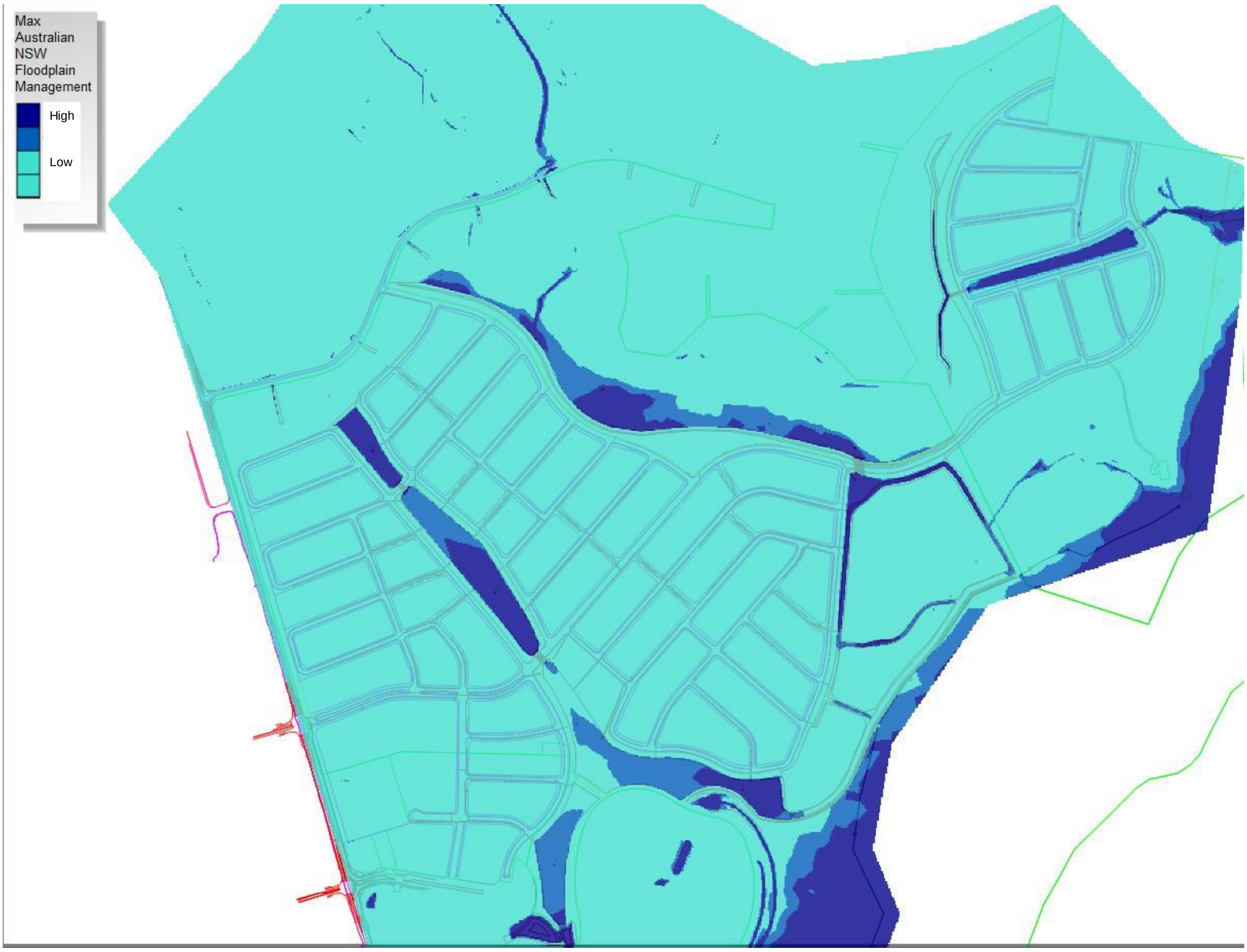

Figure 46 - Critical Flood Hazard Mapping Across the Developed Riverside Site – 2hr 100yr Storm, 2100 5yr Tailwater

3.9.6.4 Impact of Structure Blockage

Modelling of blockages was not considered a requirement of the previous Riverside report as it was not included in any of the previous studies of the site, and was not listed in any of the responses of any of the government agencies in their previous assessments of submissions. It was, however, raised in the peer review process of the last report, so is now analysed below.

Blockage of culverts via vehicle is highly improbable - the hazard assessment above shows that in storms up to and including the 100yr event that there are no flows across any roads strong enough to wash a vehicle into a floodway. Unintentional vehicular entry would also be unlikely due to the design features explained above.

It is acknowledged that the central sections of the floodways are proposed to be thickly vegetated. However, as a general comment, the significantly flat grades and wide cross-sections in the trunk drainage lines (and resultant low flow velocities), would make the transfer of large debris into culvert structures unlikely. The low velocities would, however, also make it unlikely that any debris that did lodge on the culverts would self-clear.

The critical 100yr flood hazard model was re-run with 50% blockage of major culverts on the West Branch, East-West Branch and Monkey Jacket Branch. As a result there are more High Hazard flows within the upstream drainage channels, but these high hazards do not extend onto any roadways or residential lands.

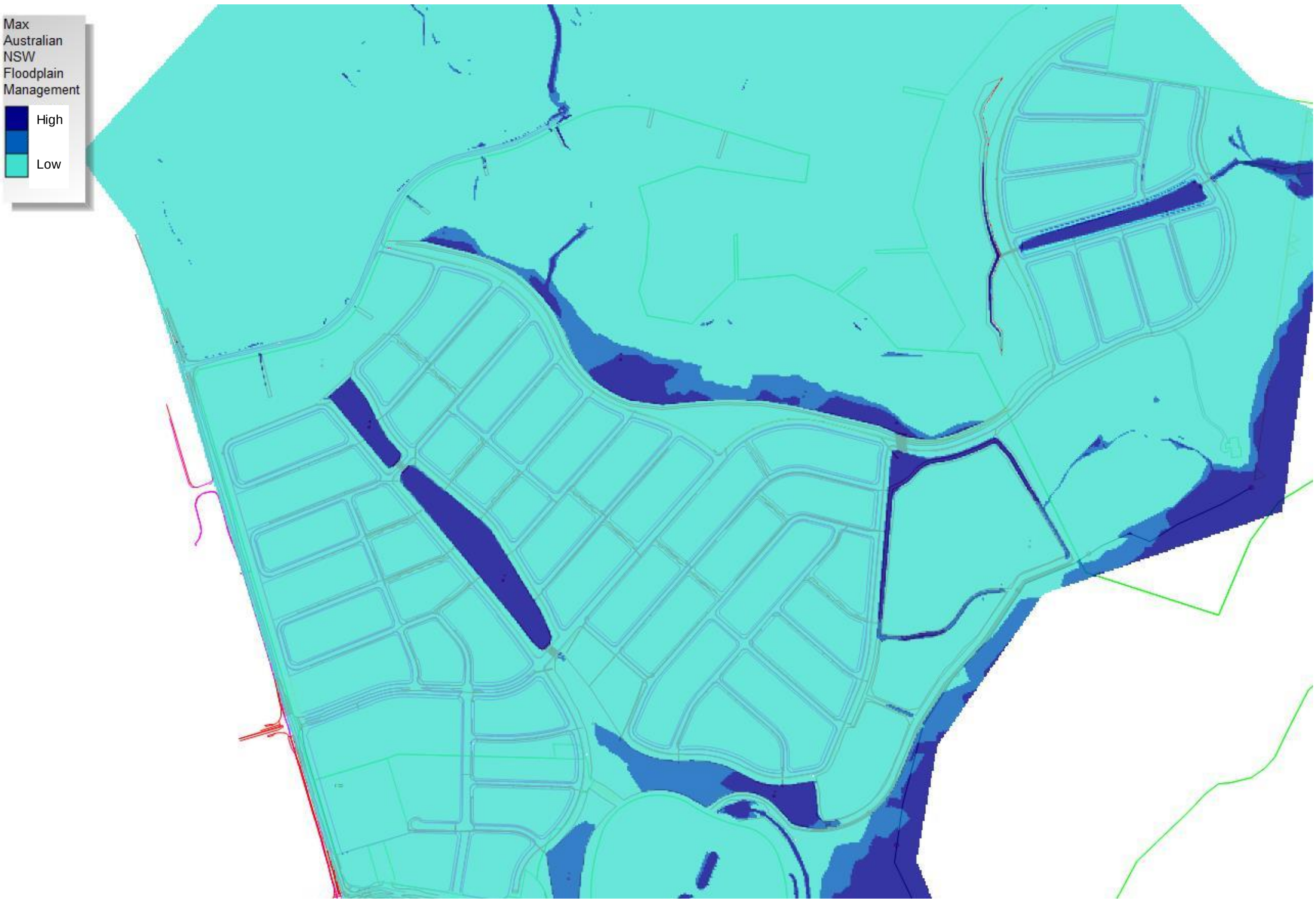

Figure 47 - Critical Flood Hazard Mapping Across the Developed Riverside Site – 2hr 100yr Storm, 2100 5yr Tailwater with 50% culvert blockages

3.9.6.5 PMF Assessment

While once reserved for assessment of major engineering structures (such as major dam projects), or areas designated as high post-disaster importance, assessment of the Probable Maximum Flood is now increasingly requested for lower order engineering designs, such as in this case a residential subdivision.

To put the rarity of such events in some perspective, the most recent significant flooding event in the region, the recent April 2015 ‘Superstorm’, involved approximately and 310mm of rainfall over 48hrs. By comparison the critical PMF storm models 550mm in only 2hrs. This is more than double the highest ever recorded 2hr point rainfall event in NSW, which is 254mm[2].

While theoretical determination and modelling of such a storm event is possible, results should be reviewed with consideration to the probability of the event in mind. The following results will show roads cut within the development and unsafe to cross for periods up to 80 minutes. This should not be seen as a failure as no hazardous flows encroach onto private lands, and while alternate emergency access routes are available, realistically evacuation from the development would be pointless as existing roads surrounding the site (including the Myall Road link to the Pacific Highway) would also likely be impassable for a similar period.

As requested by the Department of Planning and Infrastructure, PMF events were modelled across the Riverside catchment under both of the following combined probability scenarios;

- PMF Event 1; PMF catchment storm with a 100yr 2100 river level of RL.2.3m AHD
- PMF Event 2; 100yr catchment storm with an ‘extreme’ 2100 river level of RL.2.5m AHD

Both these tailwater levels have been revised down (previously 2.8m and 3.3m AHD) per the recently completed BMT WBM Lower Myall Flood Study. While the adopted ‘extreme’ event is technically not a PMF event, this value represents the best data available and is considered sufficient for this purpose.

The following figures illustrate critical duration PMF hazard mapping across the entire site, plus detailed mapping at critical locations areas of the development. Complete Flood Depth and Flood Hazard mapping plans for all PMF flood event are shown in Appendix D.

Minor inundation (<0.3m) can be seen on some residential land. The worst case PMF storm does top the East-West Branch Flood Levee and flows down residential streets. In some cases High Hazard flows are seen within the streets, but in no instances does High Hazard flow extend into the lots.

[2] Bureau of Meteorology - Wantabadgery Station, 29/8/1902

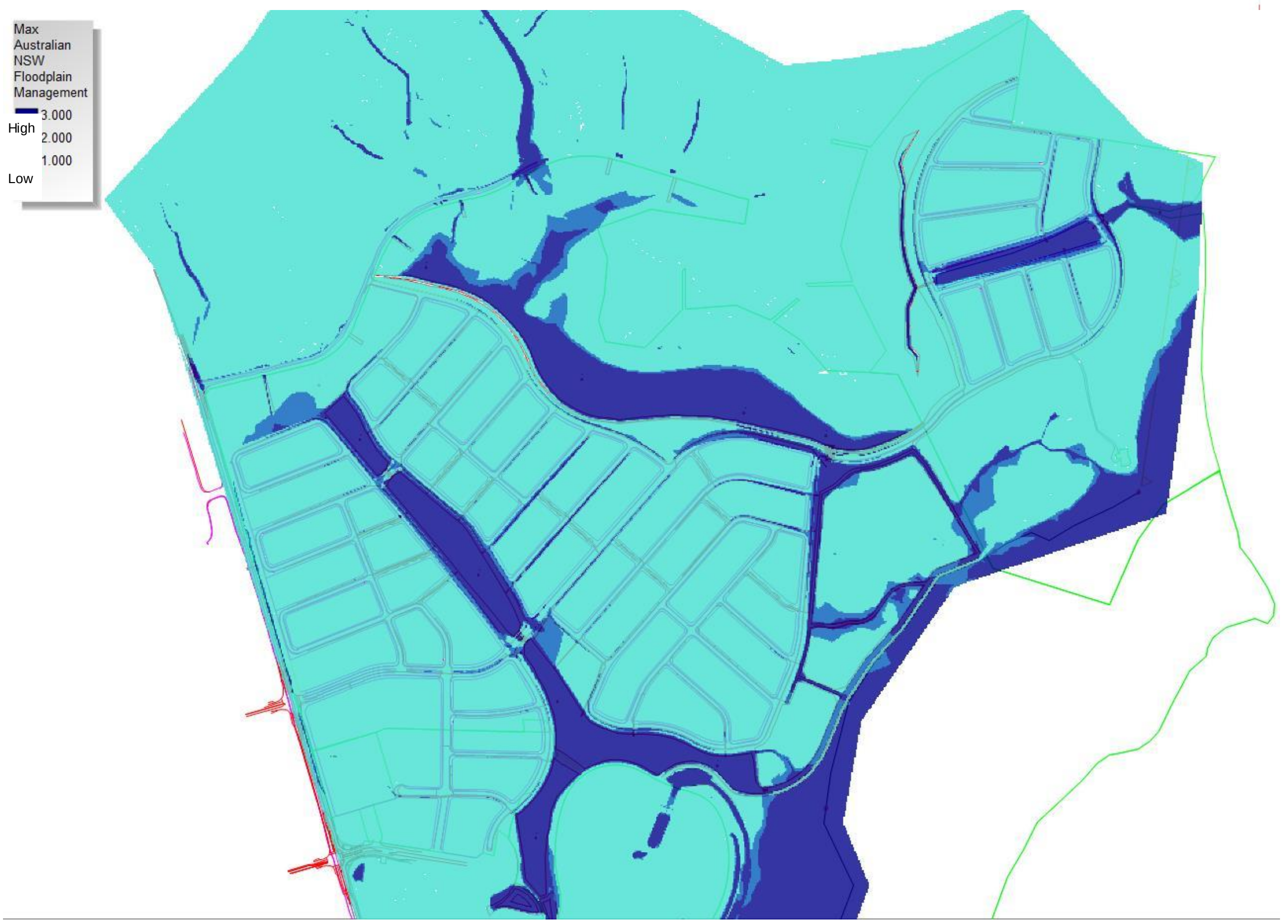

Figure 48 - Critical Flood Hazard Mapping Across the Developed Riverside Site – 1hr PMF Storm, 2100 100yr Tailwater

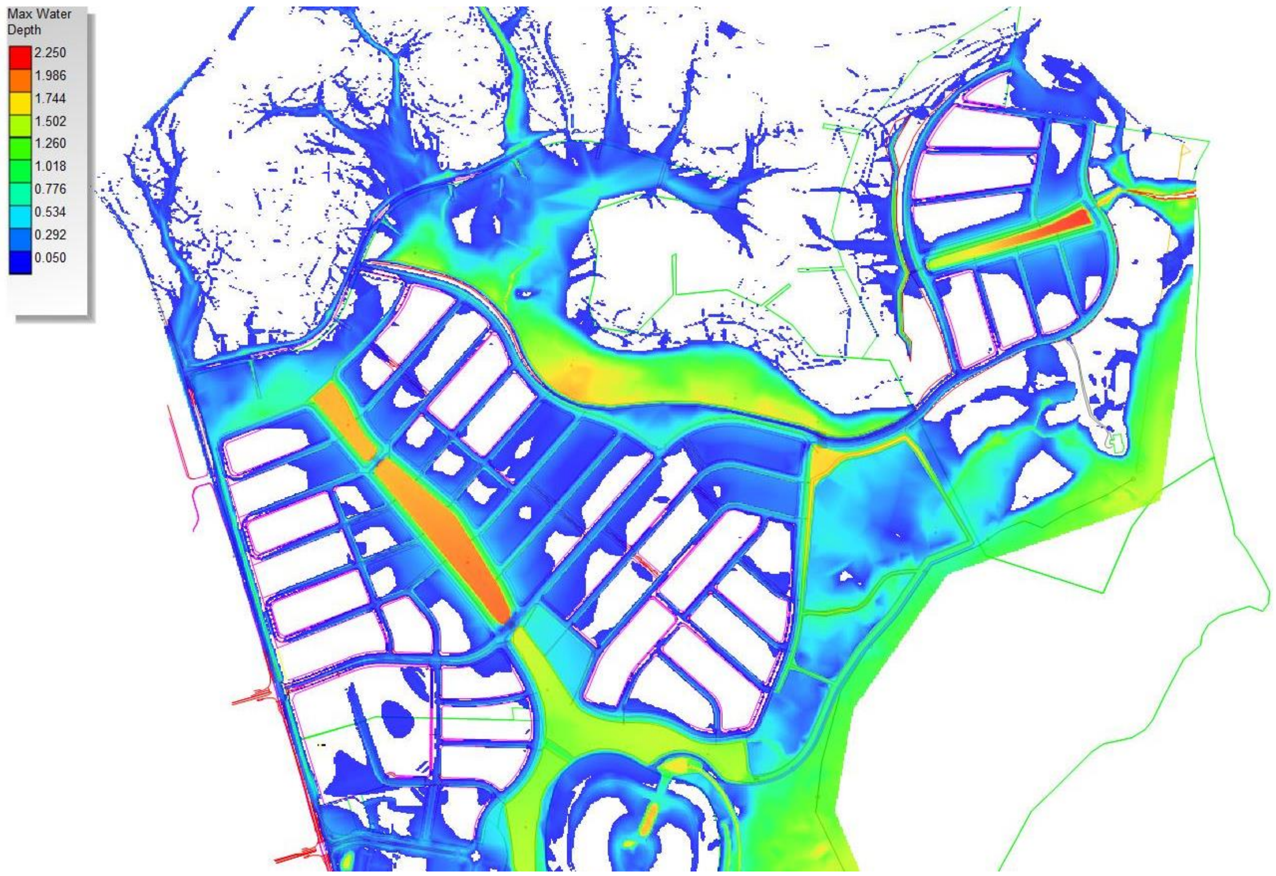

Figure 49 – Critical Flood Depth Mapping Across the Developed Riverside Site – 1hr PMF Storm, 100yr Tailwater

The following critical areas are worst affected by the PMF event, and require further discussion;

- **West Branch Floodway**

Some road crossings of the West Branch will become submerged and dangerous for pedestrians to cross in the 1hr, 2hr and 3hr PMF events, but as the proposed landform rises away from the floodway, there are safe evacuation routes available via other streets that will not require crossing of dangerous floodwaters. In the worst case (2hr PMF), the road crossing will be cut by High Hazard flows for up to 80 minutes (t=34-112min).

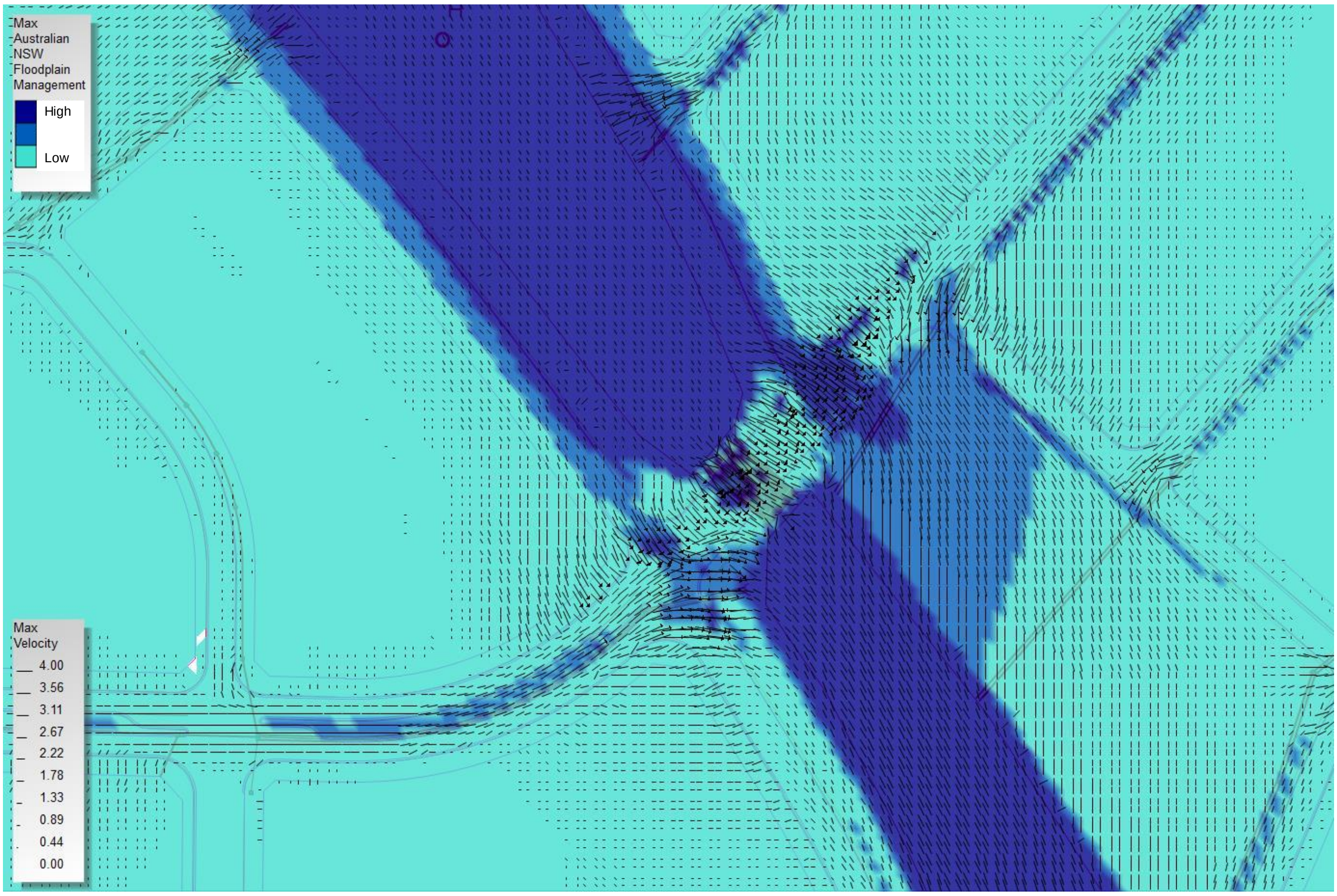

Figure 50 - Critical Flood Hazard Mapping Across the West Branch Crossing - 1hr PMF Storm, 2100 100yr Tailwater

- **East-West Branch Floodway**

The East-West Branch Floodway has been designed to contain the worst case 100yr storm flows (including freeboard). The PMF, being a significantly larger storm, exceeds this capacity and the flood levee becomes a weir over a 700m length, across the northern edges of Stages 4, 5 and 7 and road access to 15. Damage to the levee is unlikely as the worst case 'weir' flow depths are less than 0.3m and the concrete cycleway along the top of the flood levee will help armour it against flow induced scouring. Additionally, the east-west branch holds no permanent water, and only contains flows over the duration of a storm event, making any piping failure unlikely.

The excess water that tops the weir is controlled in the adjacent road profile and directed away to the downstream drainage regime without significantly impacting on residential lands. High Hazard flows are mapped within the East-West Branch and adjacent roadways, but in no case do they extend onto private lands.

These High Hazard flows will also cut access to the Monkey Jacket precinct for a period in the 1hr, 2hr and 3hr PMF. In the worst case (2hr PMF storm) the Flood Hazard will be High in the road reserve for up to 50min (t=34-84min). At this level the area would be impassable by trained rescue workers and vehicles alike.

Ultimately access to the Monkey Jacket precinct will also be available via the collector road connection to the proposed North Shearwater residential subdivision to the north, providing another possible evacuation option. Since the previous Riverside report, further progression of the North Shearwater development provides even more certainty that the 'Monkey Jacket' precinct will have this additional evacuation route available. In addition to this, existing public access corridors between existing lots linking to Toonang Drive and Petrel Place will also remain post-development and will provide additional safe emergency evacuation options.

It should also be noted that the results show almost the entire site consists of higher ground, which will remain Low Hazard level in even the most extreme possible flood scenario. This land will are available for refuge during the PMF event until flood water recedes to again provide safe access to the flooded sections of the street network.

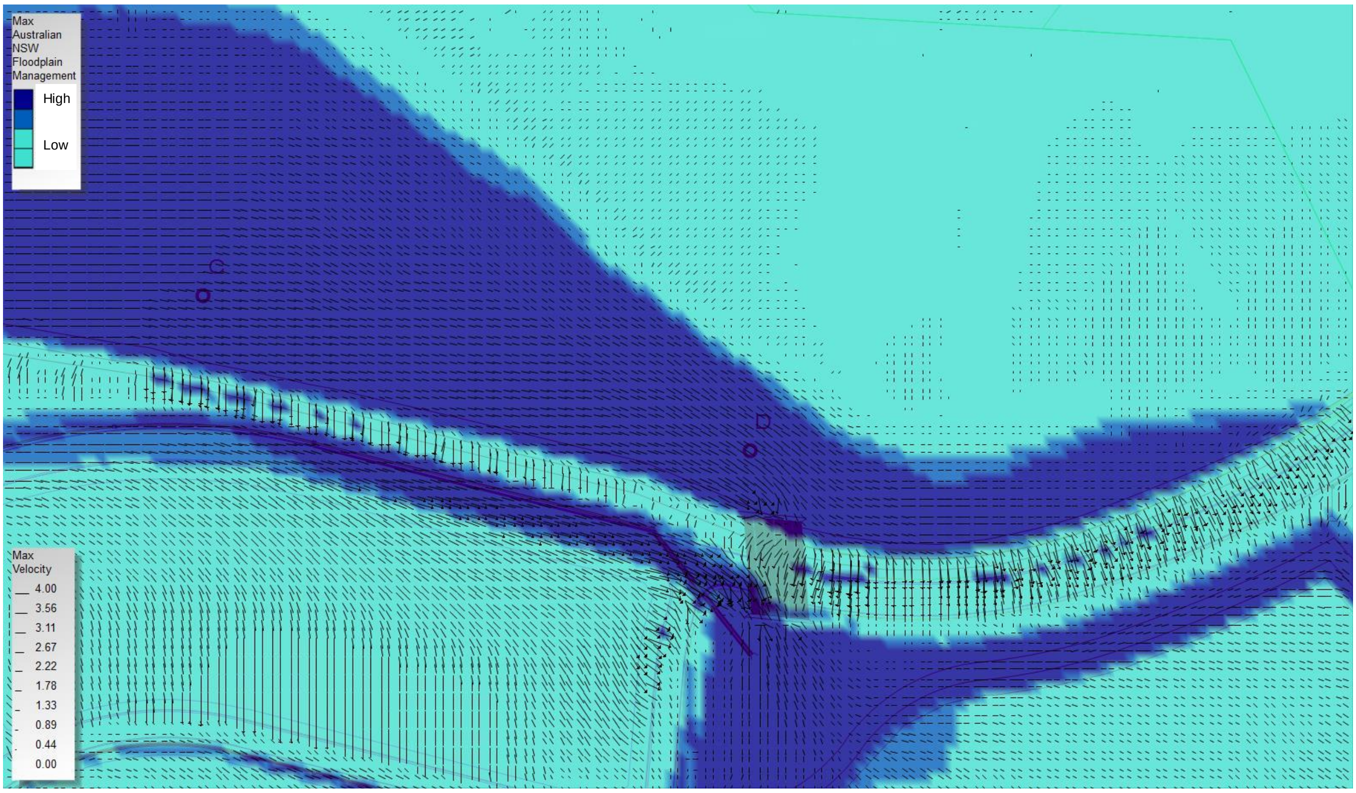

Figure 51 - Flood Hazard Mapping Across the East-West floodway linking to the Monkey Jacket Precinct - 2hr PMF Storm, 2100 100yr Tailwater

- **Monkey Jacket Floodway**

High Hazard flows in the Monkey Jacket Precinct are contained within the designated floodways. Some Intermediate Hazard flows can be seen on adjacent roadways, but do not encroach onto residential lands.

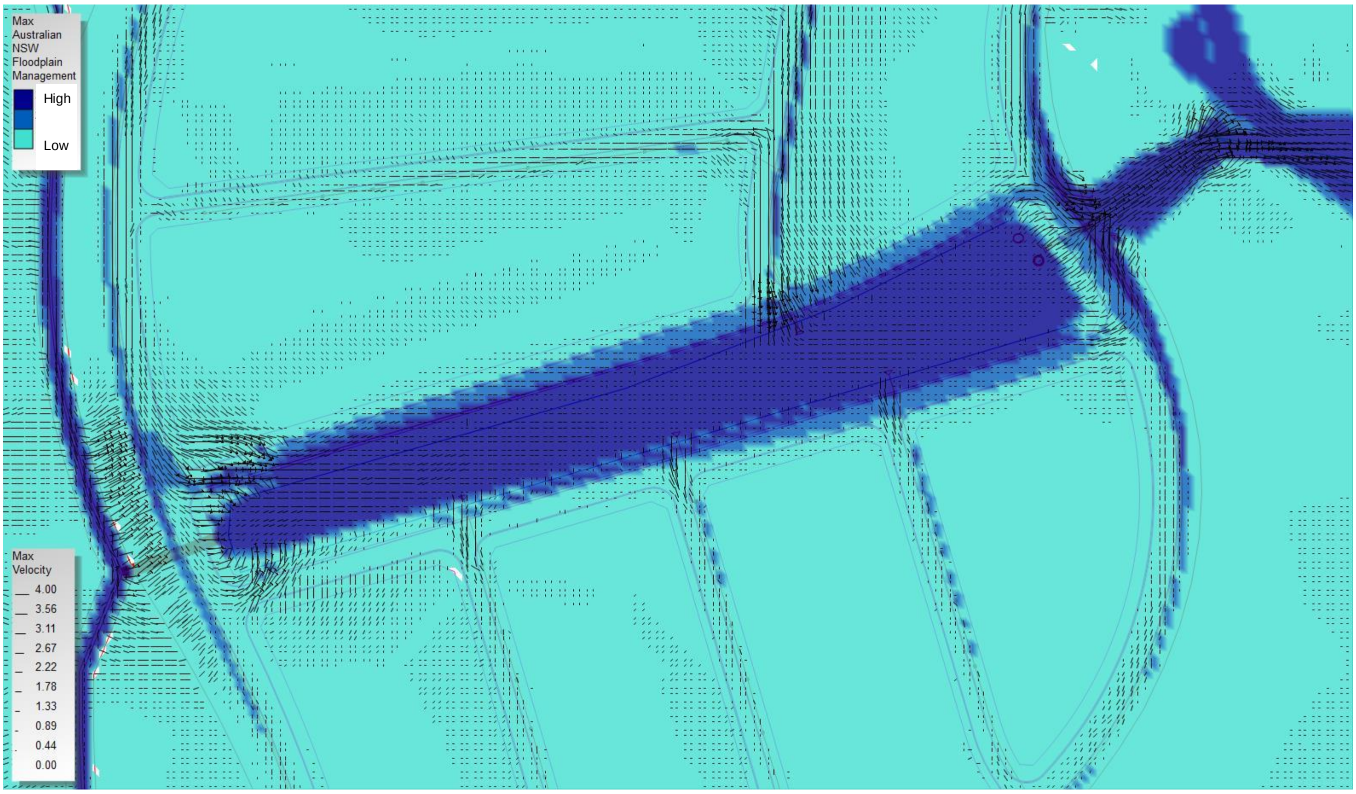

Figure 52 - Flood Hazard Mapping across the Monkey Jacket Floodway – 1hr PMF Storm, 2100 100yr Tailwater

3.9.6.6 'Dam Break' Assessment

As discussed in Section 2.2, the East-West Branch will temporarily hold stormwater flows while diverting them around the development site. The top of the berm has been designed to accommodate at least 0.4m freeboard above the peak 100yr flood levels. As illustrated in Figure 39, the diversion berm that creates the channel holds up to 1m of water behind it during the critical 100yr storm event.

In a topping failure scenario (see PMF Section 3.9.6.5), the berm fails as a weir over the length of the lower, flatter section (up to 700m long in the critical PMF storm). It is considered that neither piping or scour failures are likely, but it is noted that intercepted flows do approach the berm at right angles near Point A (Figures 38 and 39).

The critical 100yr and PMF rainfall events were re-run to include a 'Dam Break' just as the flow depth peaks in the East-West Branch adjacent to Point A. The following figures illustrate the impact of these peak flows suddenly diverting through the development site.

It can be seen that High Hazard flows are contained within the roadside biofilters in the 100yr event 'Dam Break' scenario, and does not extend out onto the roadway nor onto private lands. This impact would be lessened further by the effects of the street pipe drainage network, not included in this modelling.

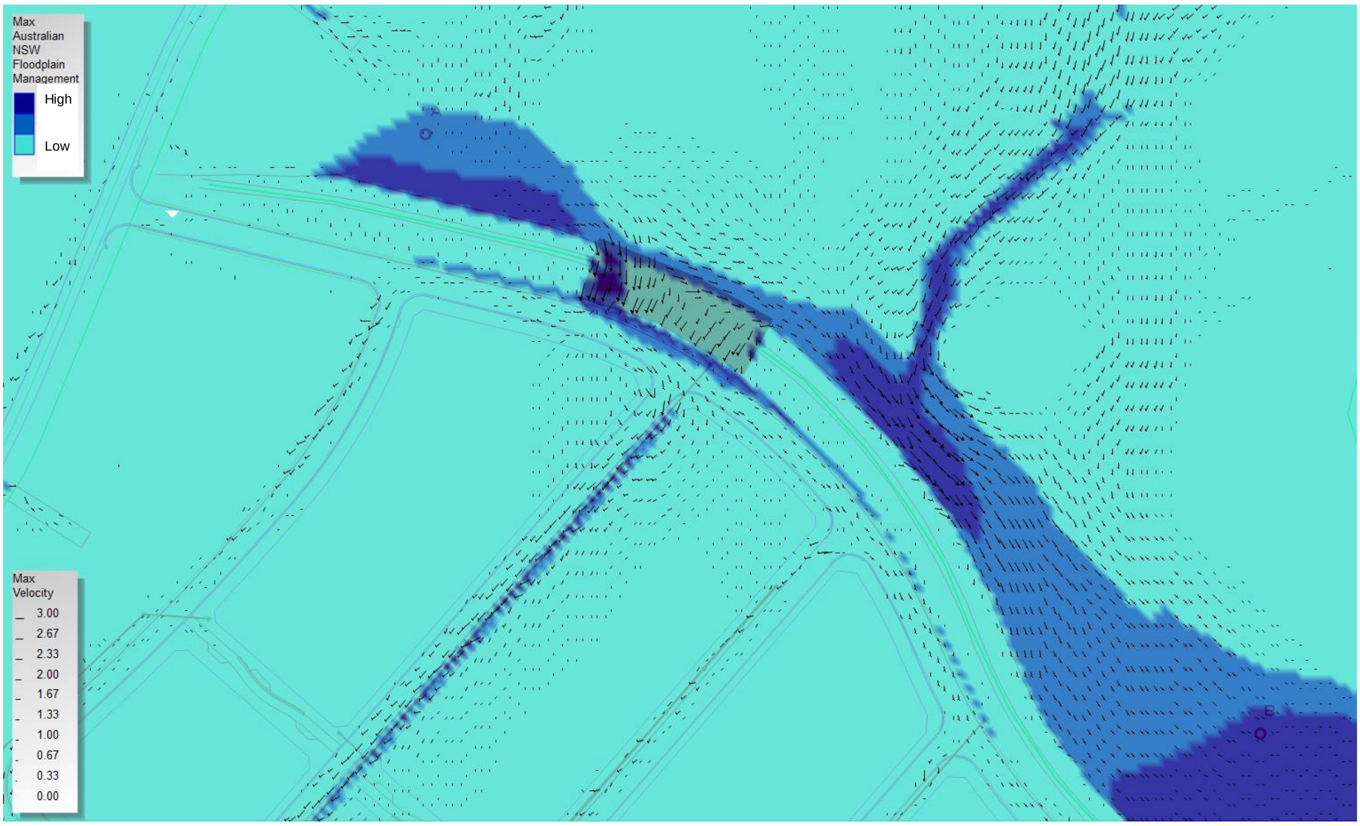

Figure 53 - Flood Hazard Mapping for Point A Dam Break Scenario – 2hr 100yr Storm, 2100 5yr Tailwater

In the PMF "Dam Break' scenario high hazard flows are seen within the roadway and biofilters, but do not extend onto private lands. Additionally, the High Hazard is restricted locally to the area around the break, meaning anyone swept off their feet as the break occurs will be swept a maximum of 40m before Low Hazard conditions are encountered and they will be able to regain their feet. It is considered this is an appropriate level of safety considering the extremely unlikely occurrence of both a PMF event and embankment failure occurring.

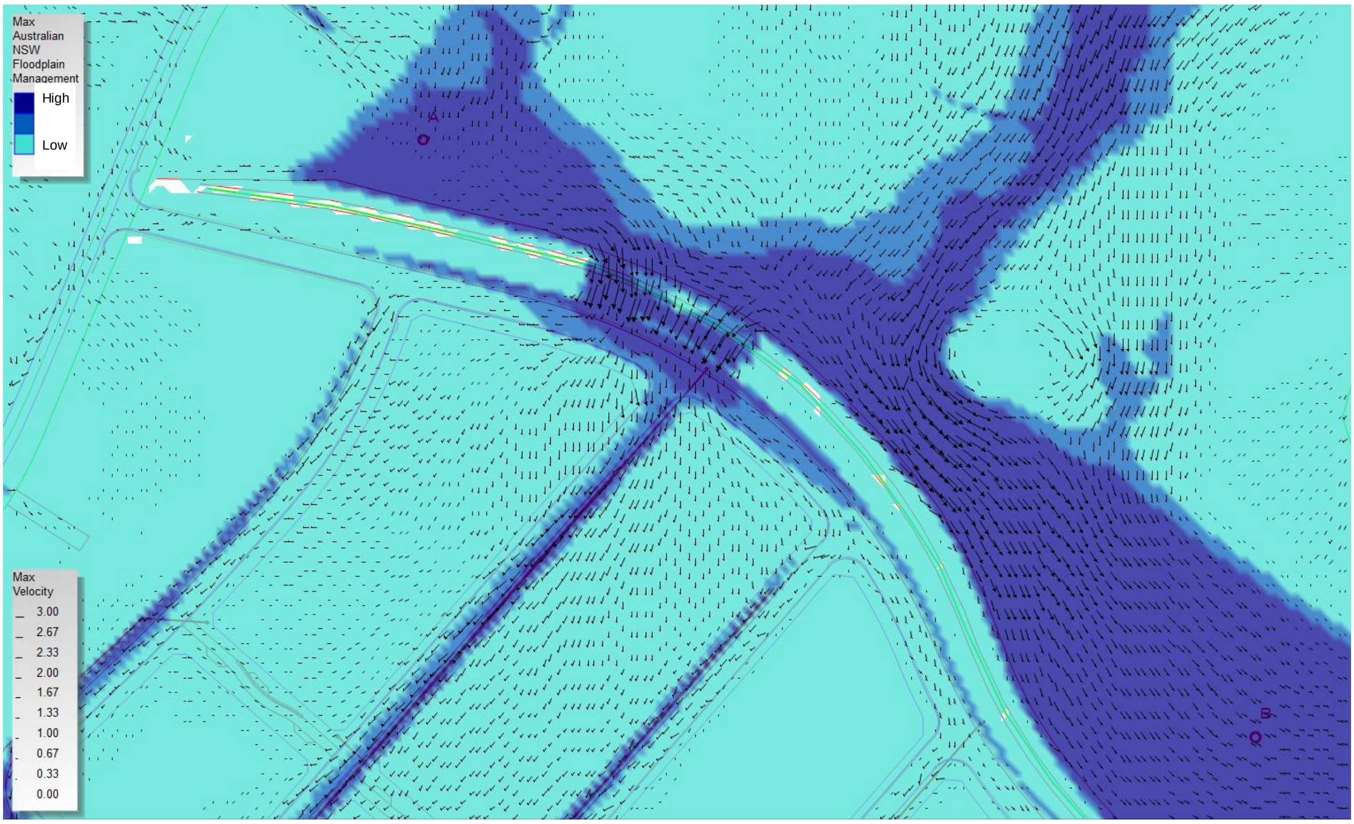

Figure 54 - Flood Hazard Mapping for Point A Dam Break Scenario – 2hr PMF Storm, 2100 100yr Tailwater

4.0 SUMMARY AND CONCLUSIONS

This report updates and supplements the previous Stormwater Management Report completed for the Riverside Estate Concept Plan Approval. It demonstrates that the current (modified) Riverside development proposal, including the proposed drainage regime summarised in Appendix A, will not have an adverse impact on the flood behaviour on or around the site, and proposed developed areas will remain essentially flood free and safe for all future residents.

Additional checks on the hydrologic model, and adopted model parameters - including initial losses, continuing losses, roughness values, model grid size, downstream boundary location and potential climate change induced rainfall intensity increases – have been conducted and illustrate that the overall model is of a suitable robustness and accuracy for the determination of existing and proposed flood behaviour and setting of Flood Planning Levels.

Specifically;

- The combination of the proposed storage and low flow discharge structures ensure regular ‘environmental’ flows into the wetland buffer are maintained post-development,

- High flow discharge via the level spreader over the full downstream frontage of the site ensures the development will not result in any increase of potentially damaging 100yr peak flow velocities in the downstream wetland,

- Existing flood levels in surrounding areas are not adversely impacted post development,

- The proposed development includes sufficient lot filling/floodway capacities to allow all lots to remain flood free in the design 100yr event,

- The ‘worst case’ Probable Maximum Flood assessment demonstrates the proposal sufficiently caters for the safety of all future residents.

5.0 REFERENCES

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