3.5 LANDUSE CATEGORIES, RAINFALL LOSSES & IMPERVIOUSNESS

Rainfall losses were modelled in xpstorm using the initial loss/continuing loss method, which can be set separately for each defined landuse zone. A USDA Soil Type also needs to be allocated to each landuse type, which the program uses to define attributes for parameters such as soil porosity and hydraulic conductivity.

Soil profile mapping across the site shows there is a fair range of soil types ranging from straight Sand through to Sandy Clay. In recognition that each landuse zone does not necessarily correspond with existing soil types, and the fact that much of the development footprint will need to be filled with unknown material from an external source, an ‘average’ Sandy Loam USDA soil type and 10mm/5mm/hr Initial Loss/Continuing Loss values were applied to pervious areas. 1mm/0mm/hr Initial Loss/Continuing Loss values were applied to impervious areas.

Impervious percentage also varies with landuse, and is summarised in Table 1.

Surface roughness values were applied using the survey data, high quality aerial photography, and the proposed development layout. Applied pre and post development model roughness values and areas are summarised in the following table and figures.

Table 1: Landuse Category Properties

Surface Type	Mannings Roughness	% Impervious
Open Water	0.02	100
Roads	0.025	100
Wetland / Wetland Buffer	0.1	0
Maintained Grass/Pasture	0.05	0
Commercial	0.025	90
Reserves	0.06	10
Rural Residential	0.04	10
Residential	0.03	50

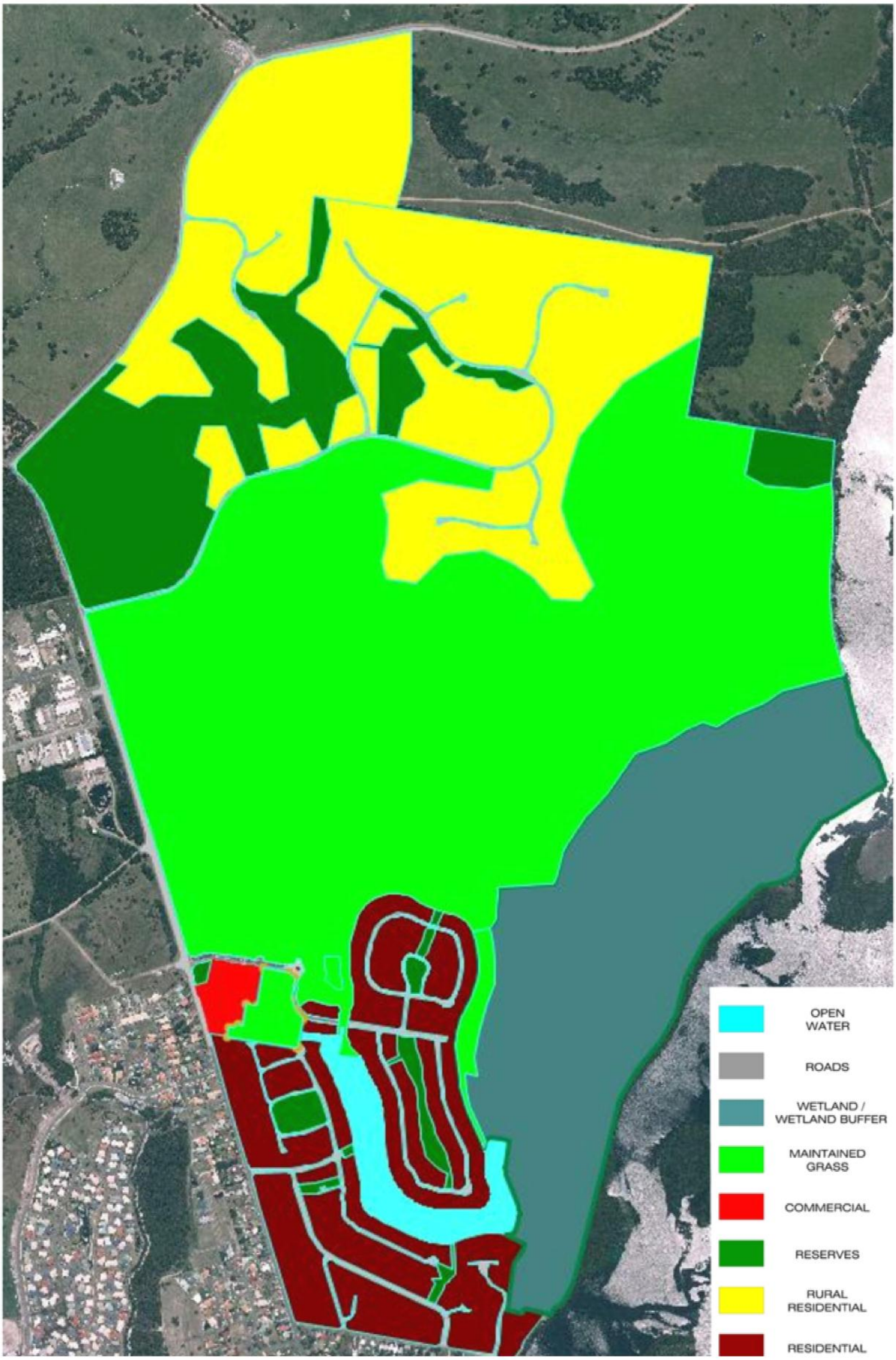

Figure 8 – Existing State Model Roughness Values

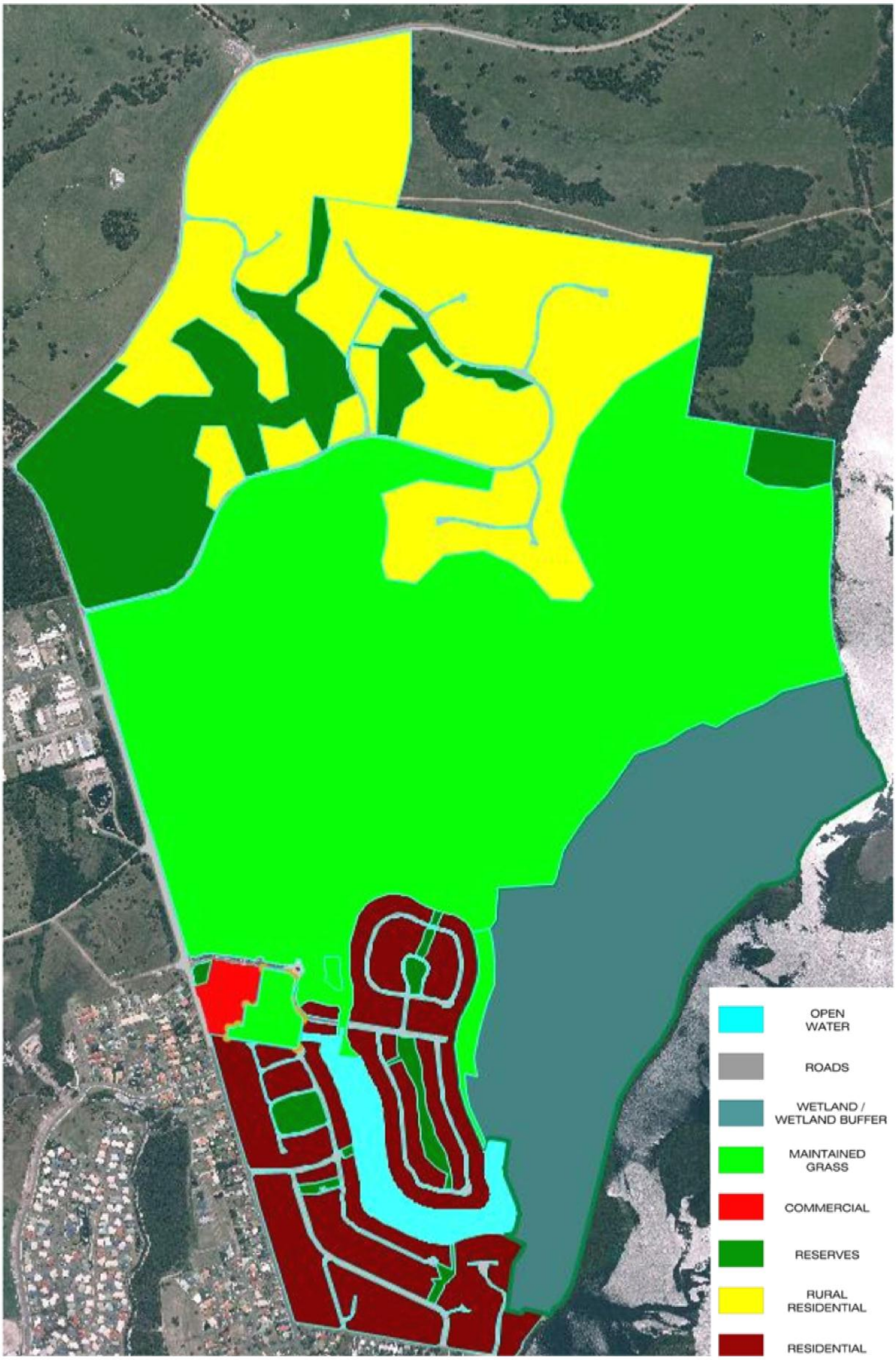

Figure 9 – Design State Model Roughness Values

3.6 TAILWATER CONDITIONS

A constant Head Boundary plane was set up along the full frontage of the Myall River to serve as the downstream boundary control. Various levels were used for different modelling purposes, and following discussions with Great lakes Council's Engineering Department were generally sourced from the recent Port Stephens Design Flood Levels Climate Change Review[1]. This report lists 20yr, 100yr and 'extreme' Design Flood Levels at the Singing Bridge (linking Tea Gardens and Hawks Nest). This level is at the upstream limit of their study.

To account for a hydraulic gradient in the river up to Monkey Jacket from the bridge, Council have added 0.1m to come up with their current 'interim' design flood levels for Riverside. This equates to a level 0.2m above the level stated at Limestone (the location of the river outlet into Port Stephens).

It should be noted that, the current Lower Myall River Flood Analysis[2] shows that the 100yr river flood would add 0.3m at the Riverside site to a 1.0m Limestone tide level, and only 0.1m to a 2.0m Limestone tide. Extrapolation of these values would suggest this difference would be even less when applied to the current 2.6m Design Flood Level from the WMA Water report at Limestone, showing the current assumptions to be conservative by at least 0.15m.

It is understood BMT WBM are currently undertaking a more detailed study of flooding in the Myall River for Great Lakes Council. This study is expected to further refine flood levels in the Myall River, but results from this study are not expected to be finalised until mid-2013. It is expected this could see a reduction in the peak river levels compared to the values used for this report.

Correspondence from Council supporting the adopted tailwater levels and probability combinations can be seen in Appendix C.

A short summary is shown below in Table 2.

Table 2: Tailwater Conditions

Tailwater Condition	Level (m AHD)
Existing Mean High Water	0.5
2100 Mean High Water	1.4
2100 5yr River Level	2.0
2100 100yr River Level	2.8
2100 'Extreme' River Level	3.3

[1] WMA Water (2010), *Port Stephens design Flood Levels Climate Change Review*
[2] Department of Public Works, NSW (1980), *Lower Myall River Flood Analysis*

It should be noted that in all cases the tailwater conditions have been set at a constant steady state peak level. This is a conservative assumption as the primary factor influencing peak river levels is a tidal influence from Port Stephens. In short duration events this would have little impact on the modelled results, whereas in longer duration events the effects would be more pronounced as varying river levels during the tidal cycle may result in higher capacity for discharge and thus lower peak flood levels.

3.7 1D/2D MODELLING AND DTM CREATION

The model was set up as a combined 1D/2D model utilising the xpstorm flood modelling software. xpstorm is an integrated 1D/2D application that utilises a version of the EPA SWMM engine for 1D calculation and the TUFLOW engine for 2D calculations. The application of a 2D analysis should provide more accurate modelling of stormwater behaviour through the proposed Riverside development site.

The 2D domain with 5m grid spacing and accompanying DTM were utilised across the entire 4.4km^2 catchment. Several features were also modelled as 1D structures within the 2D domain with nodes linked directly to the grid, including both existing and proposed drainage culverts.

In order to compare results from different modelling conditions, standard 'head/velocity' points and 'flow line' sections were defined within the model. This included breaking the frontage along the wetland buffer into six 300m sections to ensure that pre and post development flows distributions are maintained.

The locations and a brief description of these points and sections are shown below.

Table 3: Head/Velocity Sample Points

Location	Description
A - E	East-West Branch Floodway
F - J	West Branch Floodway
K	Existing Basin
L - M	Existing Lake
N	Existing Discharge Swales
O - P	'Monkey Jacket' Branch Floodway

Table 4: 'Flow Line' Sections

Location	Description
CREEK	Discharge direct to the Myall River via existing creek inlet
RIVER	Discharge direct to the Myall River
WETLAND 1-6	Segmented discharge into the wetland buffer
LAKE	Discharge into existing basin flowing towards existing lake

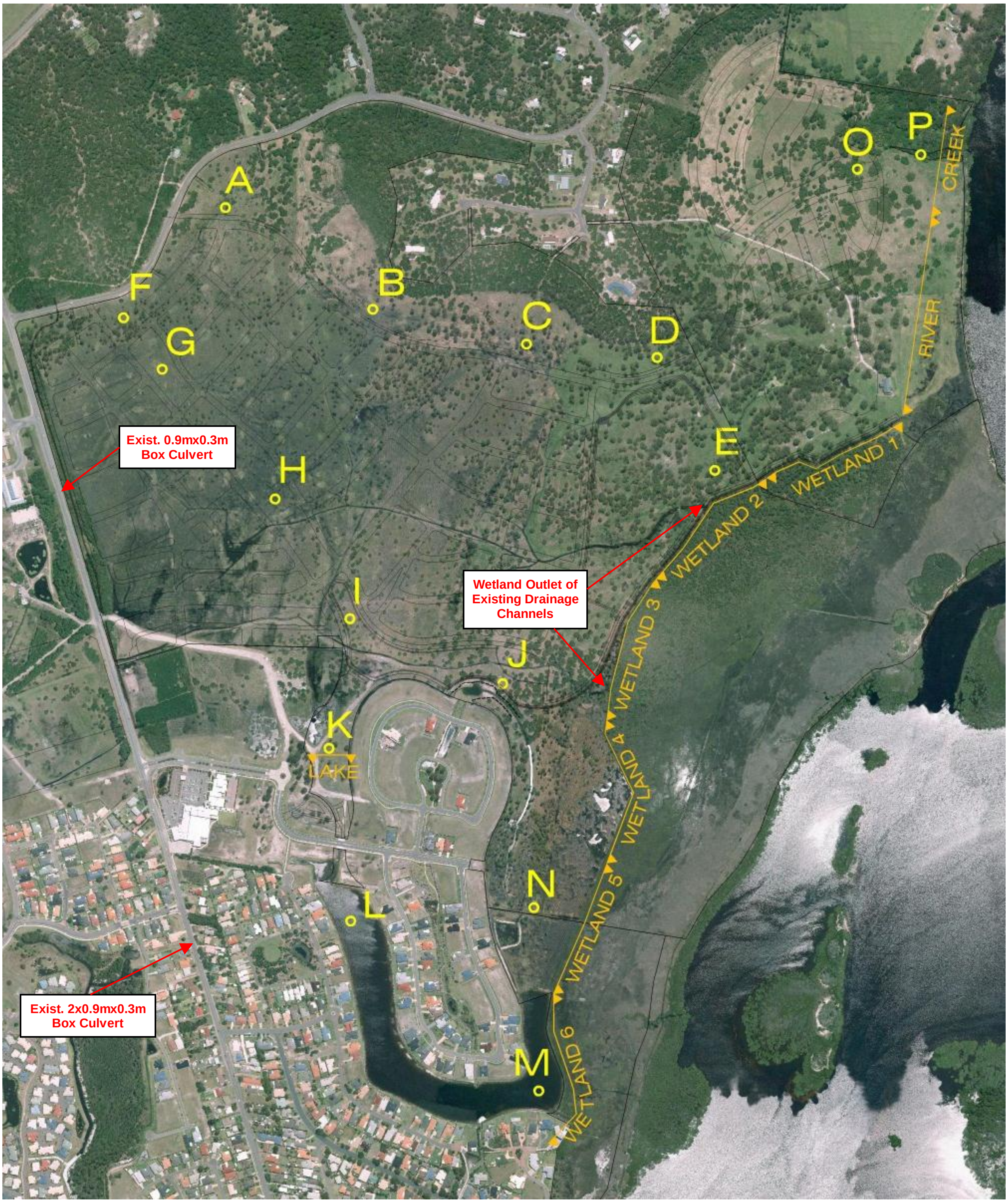

Figure 10 – Head/Velocity Sample Points and Flow Lines

3.7.1 Existing State Model

The majority of the existing state DTM was created from extensive detail survey information. Sections of the upstream hill (Shearwater Estate) were supplemented with contours from topographic maps.

A series of five separate culverts under Toonang Drive and the existing lake discharge channel were modelled as 1D structures. Additionally, there are several small existing surface drains on the site, which generally drain west to east towards the wetland. It is expected these drains will have little effect on major storm flows, but will have an impact on smaller rainfall events. The more significant of these drains have also been included as 1D structures within the model.

A Head Boundary line was set up along the complete frontage to the Myall River as the downstream control, and initial water levels were set in the existing basins to each respective permanent water level.

Screenshots from a few critical areas are shown below to illustrate the level of detail and results being achieved.

Note – Direct comparison with results previously achieved by Cardno is not possible due to the differing approaches used to define flow measurement points. Previous studies have lumped the entire wetland buffer area together as a single discharge point. As illustrated in Figure 10, this report has broken the wetland buffer interface into six 300m long sections to more accurately measure flow dispersal along this length. Due to the varying nature of the adjacent catchments, each Wetland flow line have their peaks at different times, and even have different critical storm durations, and as such cannot simply be added together for a direct comparison.

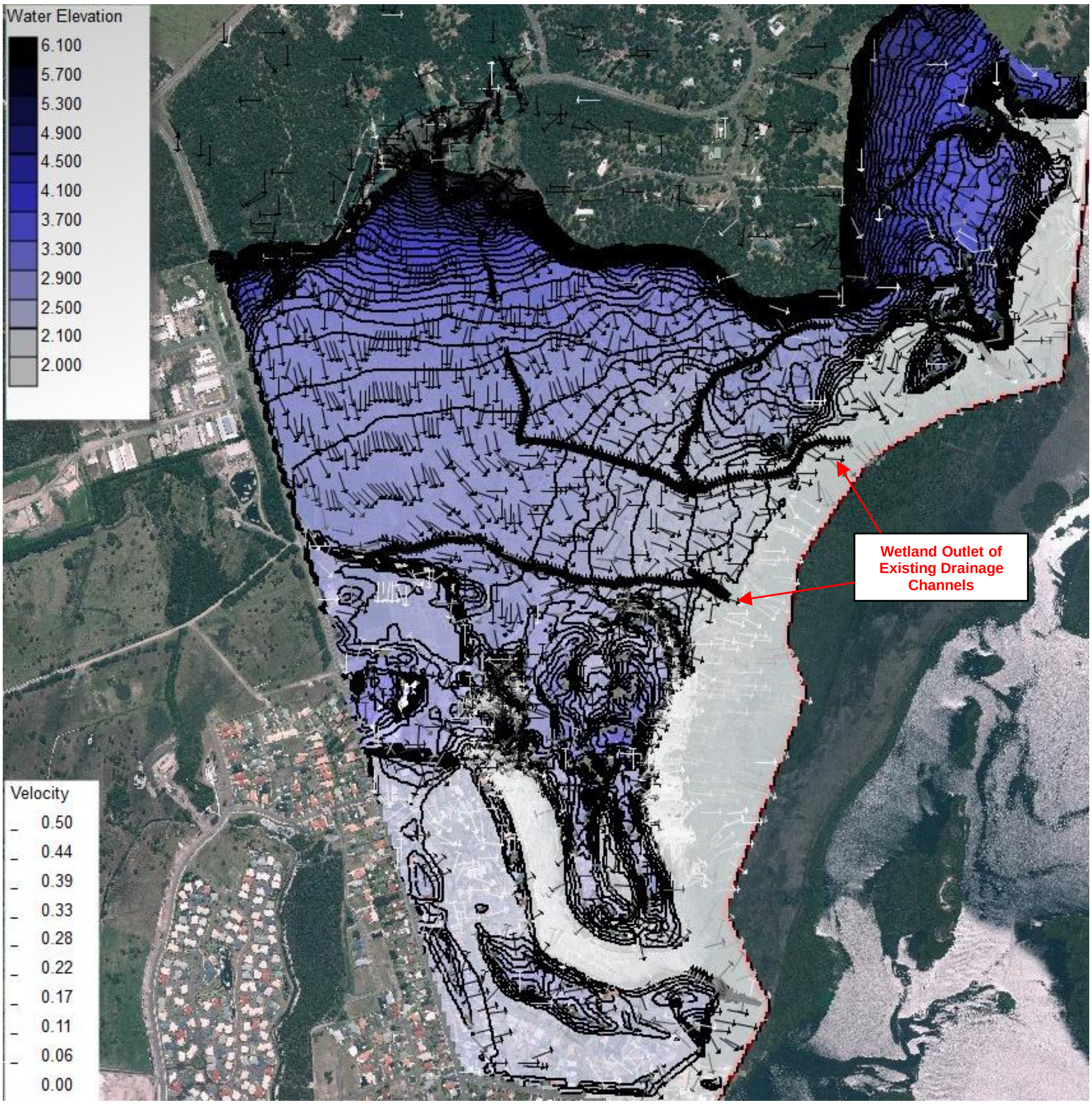

Figure 11 – Flood Contours/Velocity Arrows Across the Existing Riverside Site

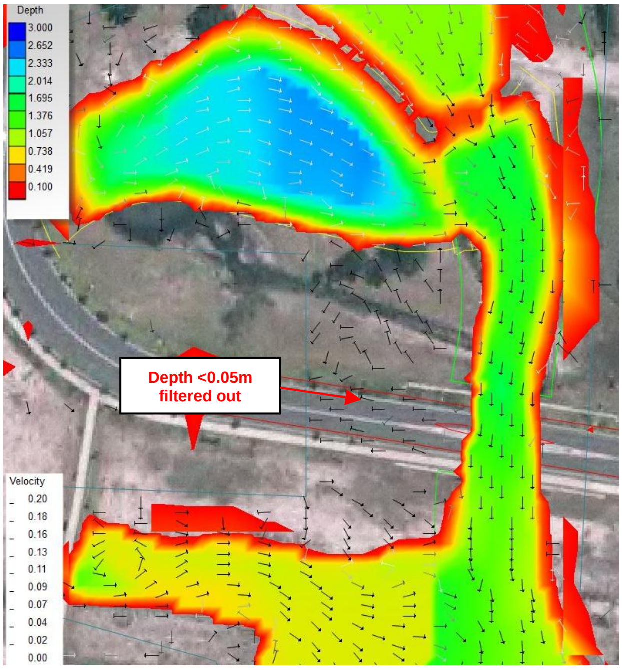

Figure 12 – Flood Depths/Velocity Arrows at the Existing 'Bebo' Bridge and Adjacent Basins/Spillway

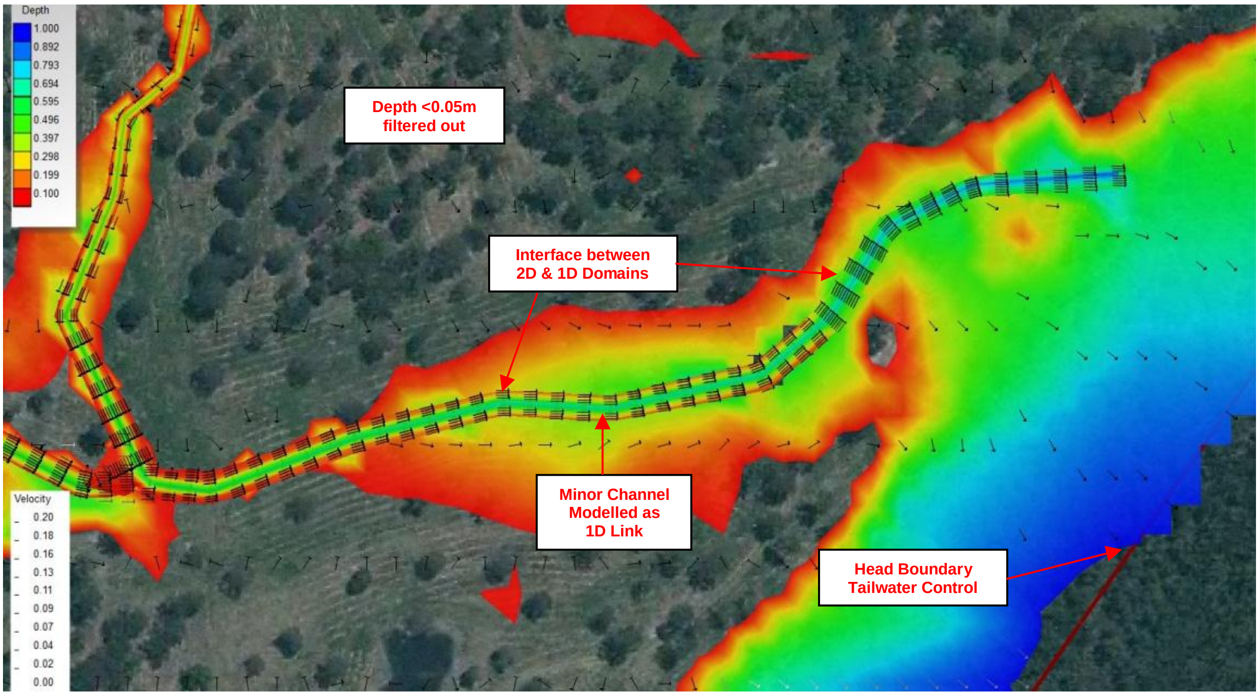

Figure 13 – Flood Depths/Velocity Arrows at an Existing Drainage Channel (modelled in 1D)

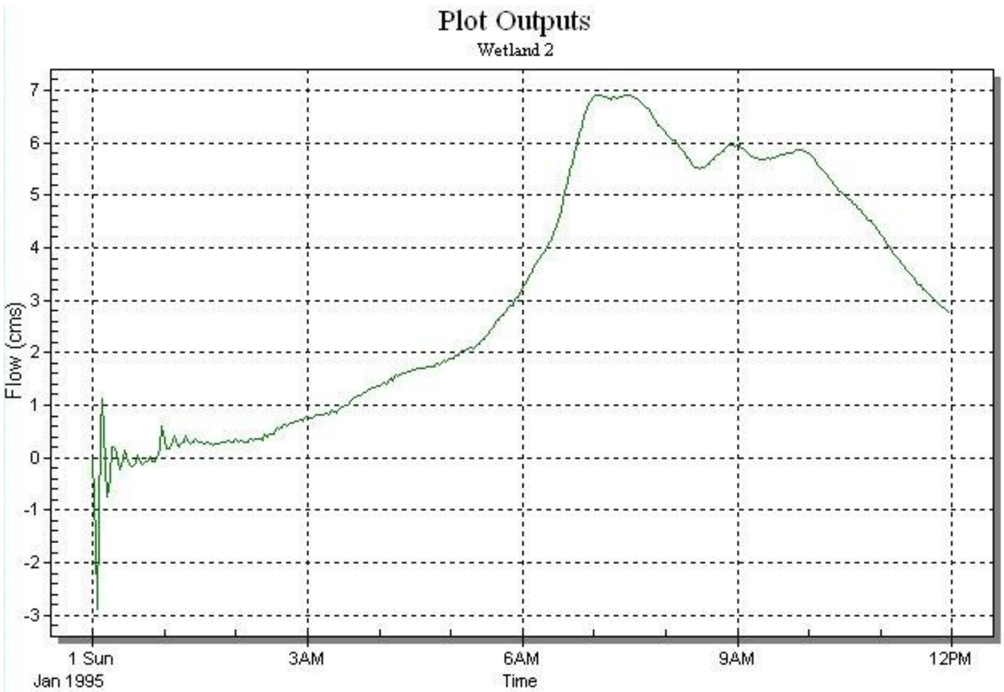

Figure 14 – Typical Flow Line Discharge Hydrograph – 'Wetland 2' 100yr 12hr Storm

3.7.3 Design State Model

The preliminary design DTM surface used in the design state flood models was created using engineering design software to model the proposed ultimate site conditions. In this DTM the drainage features described in Section 2 of this report have all been designed in detail to ensure designs are practical and the flood modelling is as accurate as possible. This included a preliminary design of every street within the development.

As with the existing state model, the design model includes the Toonang Drive culverts and lake outlet as 1D structures. In addition to this, several other proposed culverts were also modelled as 1D structures. The general model area was covered with a 5m grid, with 1D structures or the 'Elevation Shapes' feature used to represent critical structures.

Only major drainage structures were included in both the Existing and Design models. There are extensive street and inter-allotment drainage networks installed in the existing residential and commercial areas adjacent to Riverside. Similar drainage structures will also be features of the final developed site, but their details would not be determined until detail design stage. It was considered beyond the scope of this report to attempt to model the entire (existing and future) drainage network across the entire catchment area.

While significant effort and detail have been applied to the creation of the design state DTM used for this modelling, it is important to realise that this is still a *preliminary* design surface only. There will be extensive refining of fill levels, road grades etc, along with inclusion of the street and interallotment drainage network during the detail design process required at Construction Certificate stage. As such, results in this report are not presented as a 'fait accompli', but rather intended to demonstrate that managing stormwater as proposed in the Riverside development can meet all requirements in regard to maintaining environmental flows to the wetland and safely dealing with minor and major flooding.

Screenshots from a few critical areas are shown below to illustrate the level of detail and results being achieved in the design state flood modelling. Detailed results relating to specific criteria are reported in Section 3.9.

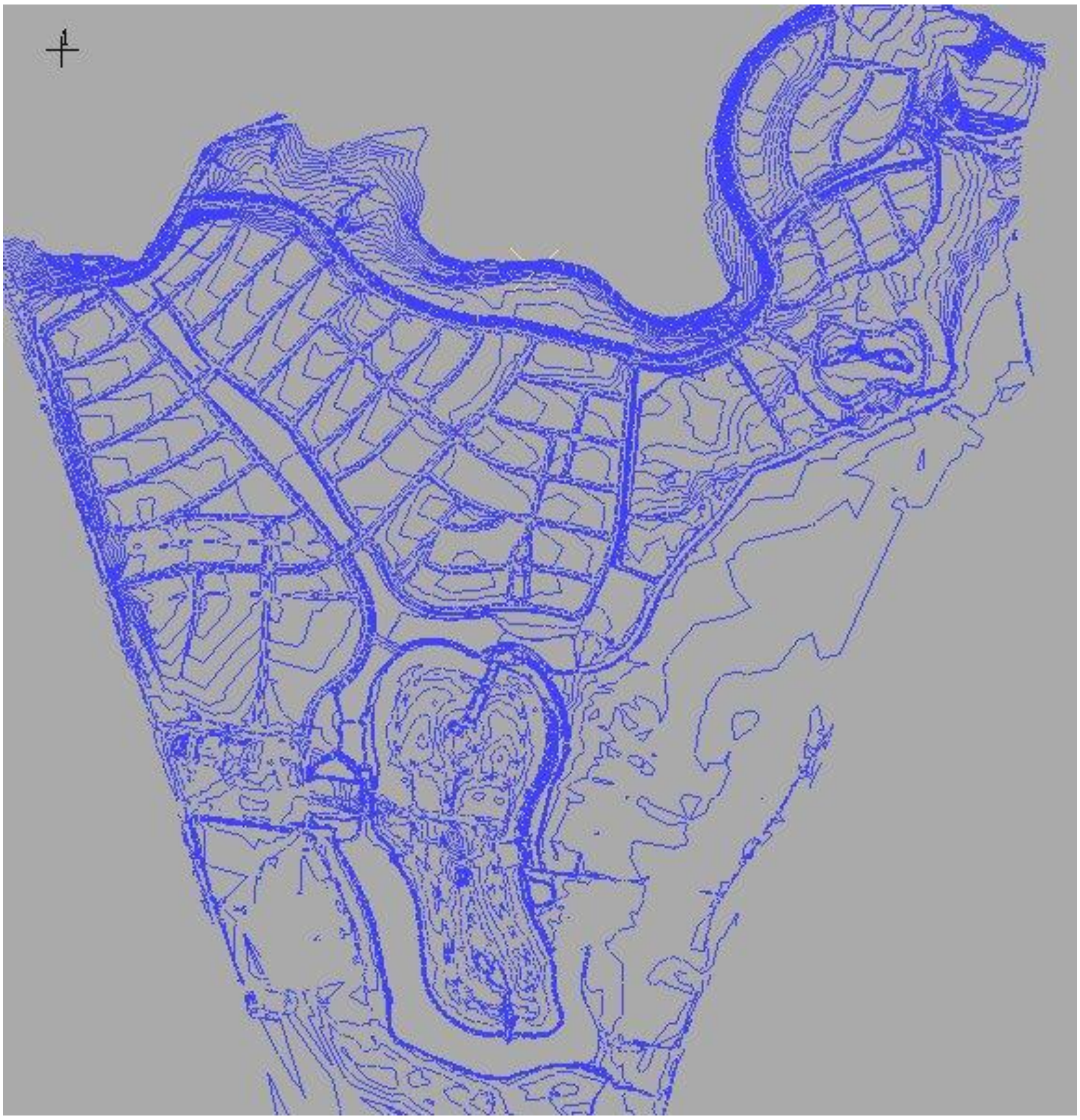

Figure 15 – Preliminary Design DTM Illustration

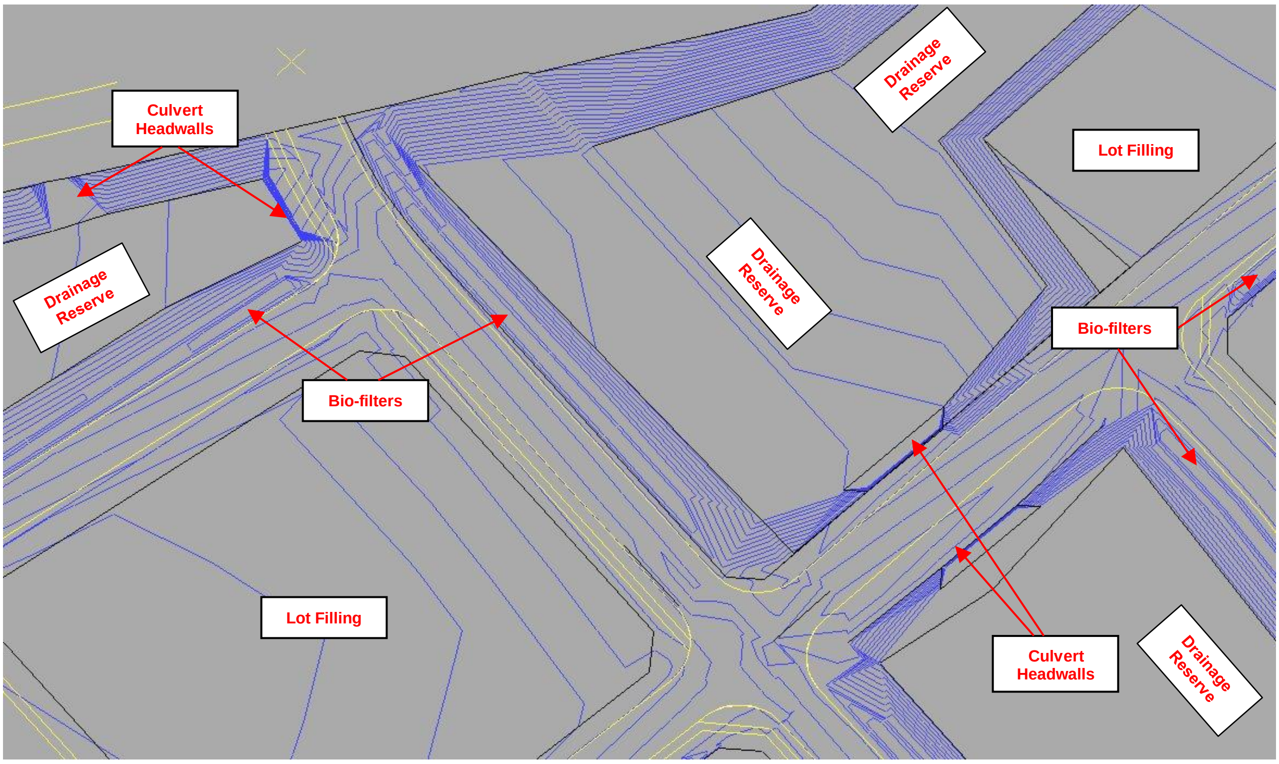

Figure 16 – Sample Detailed Preliminary Design DTM Illustration

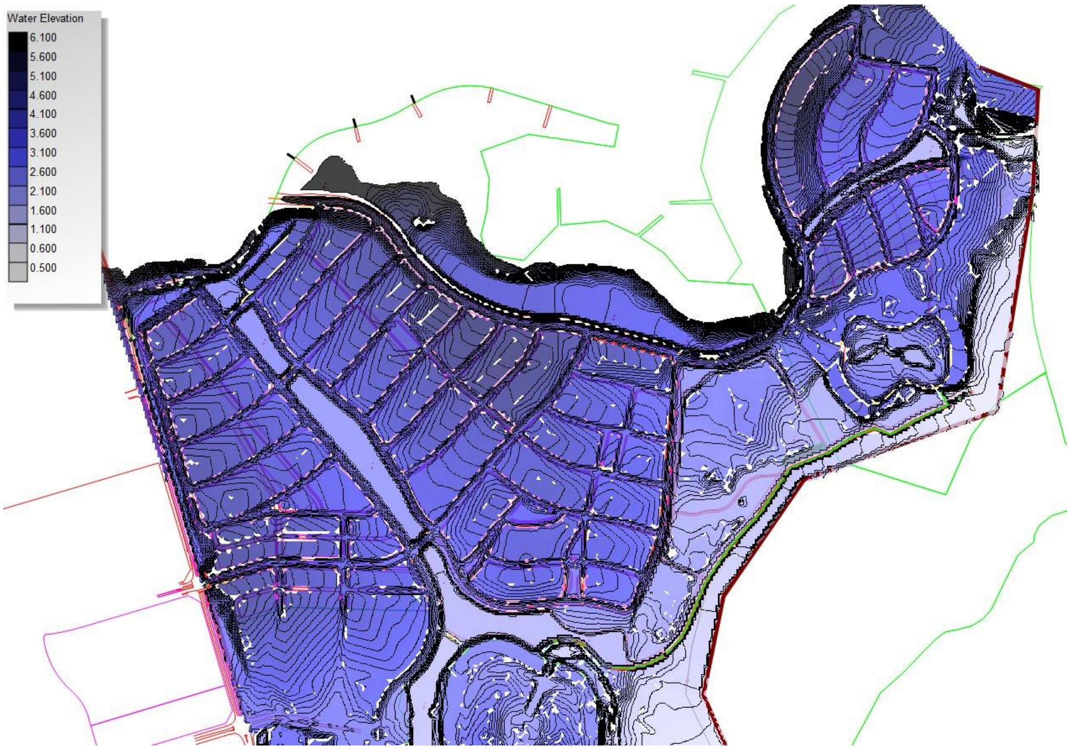

Figure 17 – Flood Levels Across Preliminary Design Riverside Site – 1yr 2hr Storm, 0.5m Tailwater

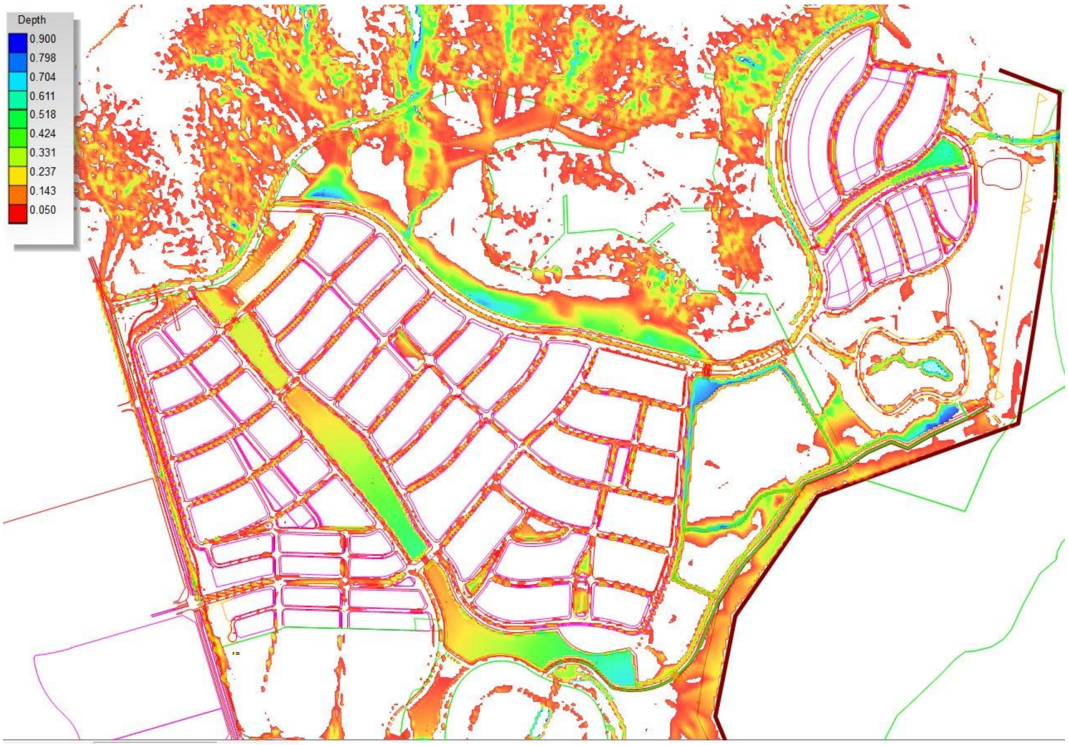

Figure 18 – Flow Depths Across Preliminary Design Riverside Site – 1yr 2hr Storm, 0.5m Tailwater (depth below 0.05m filtered out)

It is important to note that because "rainfall on grid" has been used to generate runoff, flood level plots (such as Figure 12 above) will appear to show flood water covering the entire site. In fact much of this area is covered only by minor surface flows. Unfortunately this is not able to be filtered out in xpstorm. By comparison, Figure 13 shows the same flood event, displaying depths rather than absolute levels.

In this instance minor surface water (<0.05m) can be filtered out, showing flood waters are actually confined to designated flowpaths.

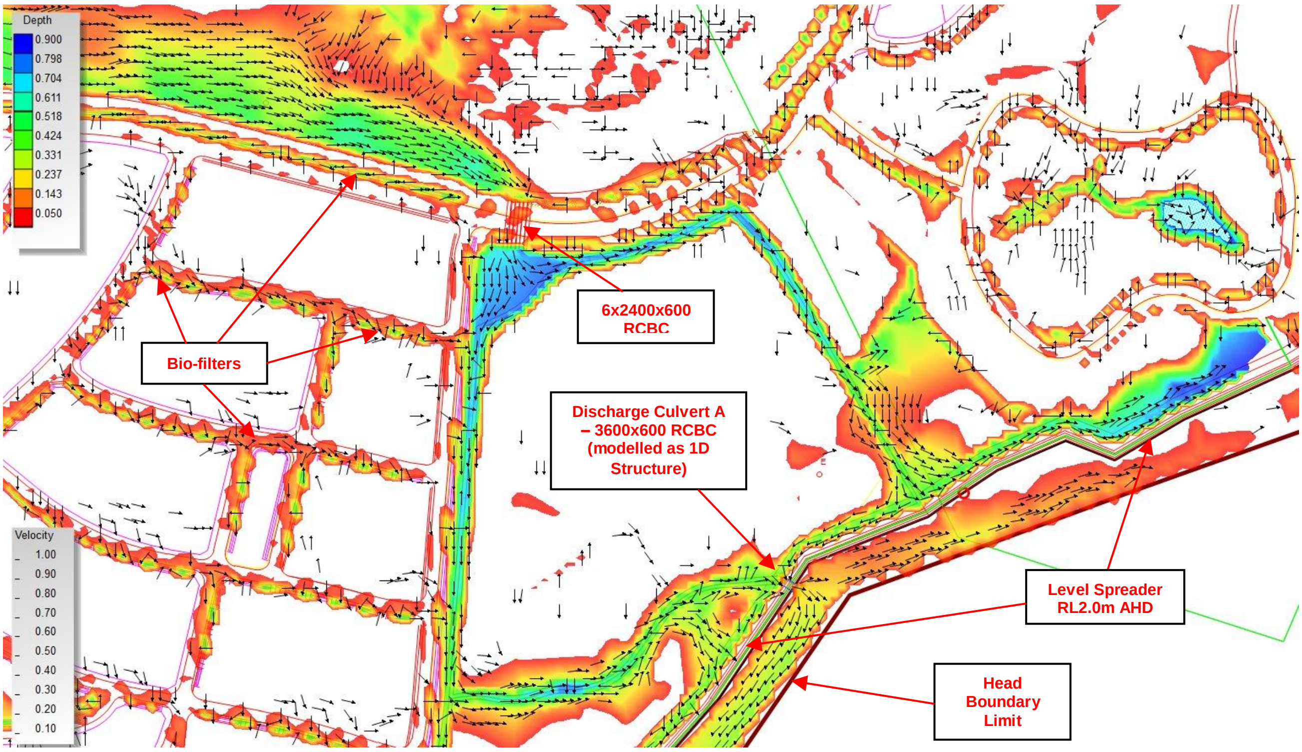

Figure 19 – Flow Depths/Velocity Arrows at Discharge Culvert A – 1yr 2hr Storm, 0.5m Tailwater (Depth<0.05m filtered out) Note – Level spreader is not topped

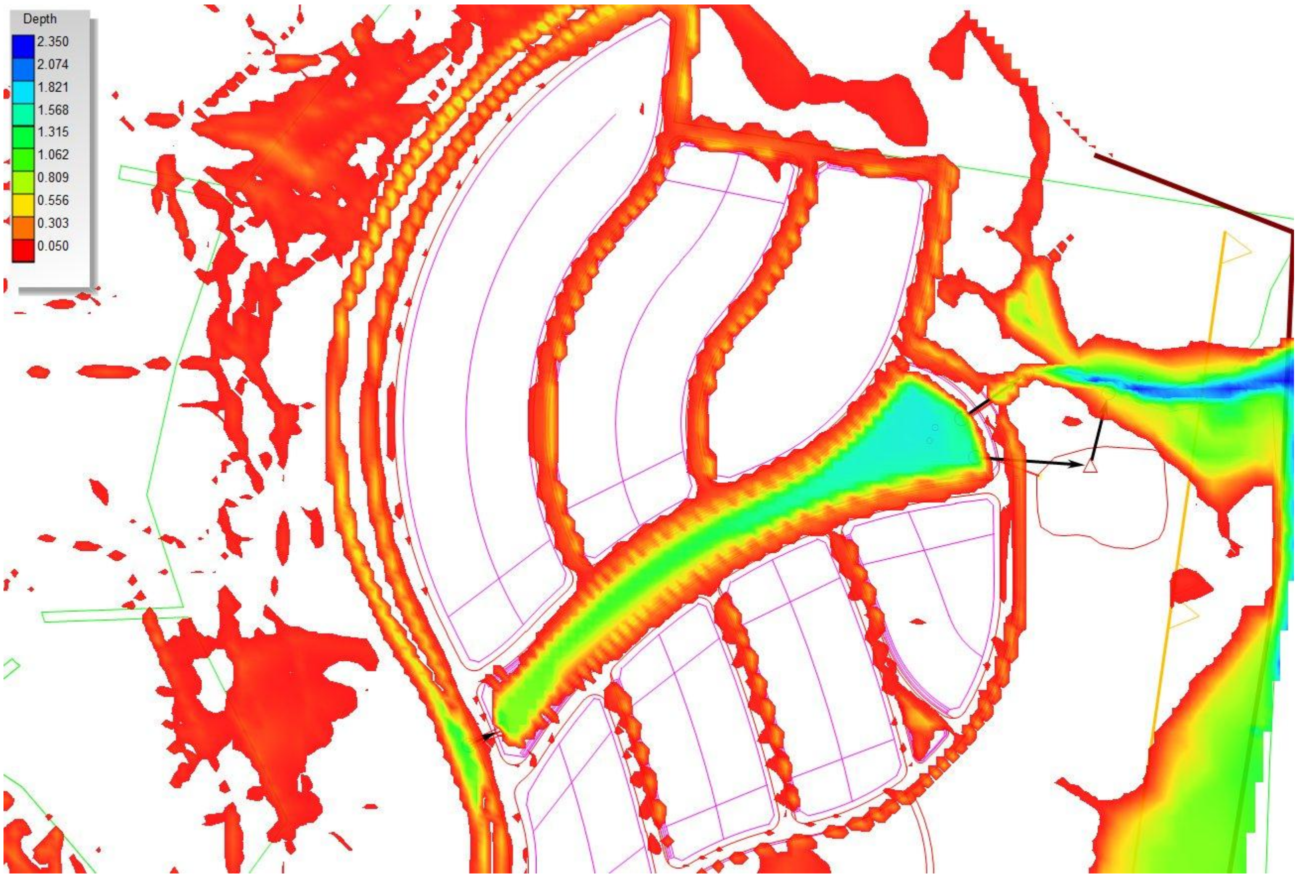

Figure 20 – Flow Depths at the Monkey Jacket Precinct (Depth<0.05m filtered out)

3.8 HYDRAULIC CATEGORY MAPPING

The most current available flood study for the Lower Myall River was produced by the Department of Public Works in 1980[3]. Great Lakes Council have recently commissioned a Lower Myall River and Myall Lakes Flood Study, intended to update the existing report to consider, among other things, climate change, the recently completed Port Stephens Design Flood Level Review, and to utilise more modern analysis techniques. This is a sizable study in its own right, with initial publication of findings not expected until mid-2013.

It is likely this study will show the need for significant mitigation works to protect the existing Tea Gardens township from even 100yr post climate change flood impacts, impacts the proposed Riverside development is already designed to cater for.

Any such mitigation works would have a significant effect on the behaviour of the floodway and the resultant flood hazard and hydraulic category mapping, and that the study currently underway would be better placed to accurately define hydraulic categories due to the fact it is a full river catchment study, and can include the impact of any mitigation proposed works.

However, in the interim some preliminary category mapping can still be undertaken for the purposes of this report. The following assessment is based on the original 1980 Department of Public Works Lower Myall River Flood Analysis, and the 2010 WMA Water Port Stephens Design Flood Levels Climate Change Analysis[4].

The Myall River is a unique waterway in that it long (25km), generally narrow (semi) tidal waterway that links two large water bodies – the upstream Myall Lake/Bombah Broadwater and downstream Port Stephens. Having such a large storage area upstream of the Lower Myall (the lakes have a surface area of 113sq.km out of a total 775sq.km catchment) means flood events are significantly delayed and attenuated.

To determine the peak 100yr flood extent, the two relevant flood/tailwater combinations are a;

- 100yr river flood with a 5yr tailwater (Exist = 1.1m AHD, 2100 = 2.0m AHD)
- 5yr river flood with a 100yr tailwater (Exist = 1.9m AHD, 2100 =2 .8m AHD)

The Department of Public Works report shows that in the proximity of the Riverside development, the 100yr river flood adds around 0.3m to a 1.0m tidal level and 0.1m to a 2.0m tidal level. The 20yr flood adds 0.12m to a 1.0m tide and 0.04m to a 2m tide. By extension and extrapolation, this would make the existing flood level for the two combinations above approximately 1.38m AHD and 1.95m AHD. The equivalent 2100 levels would be 2.1m AHD and 2.8m AHD respectively.

---

[3] Department of Public Works, NSW (1980), *Lower Myall River Flood Analysis)*
[4] WMA Water (2010), *Port Stephens Design Flood Levels Climate Change Review*

The three hydraulic categories as defined by the NSW Government Floodplain Development Manual are;

- **Floodways** are areas conveying a significant proportion of the flood flow and where partial blocking will adversely affect flood behaviour to a significant and unacceptable extent.
- **Flood Storage** areas are those areas outside the floodway which, if completely filled with solid material, would cause peak flood levels to increase anywhere by more than 0.1m and/or would cause peak discharge anywhere downstream to increase by more than 10%.
- **Flood Fringe** is the remaining area of land affected by flooding, after Floodway and Flood Storage areas have been defined.

In respect to the above definitions, it is clear the main channel of the Myall River forms a good approximation of the location of the floodway extents.

The existing 1.95m AHD peak 100yr river level does not encroach onto the development footprint proper as illustrated in the figure below.

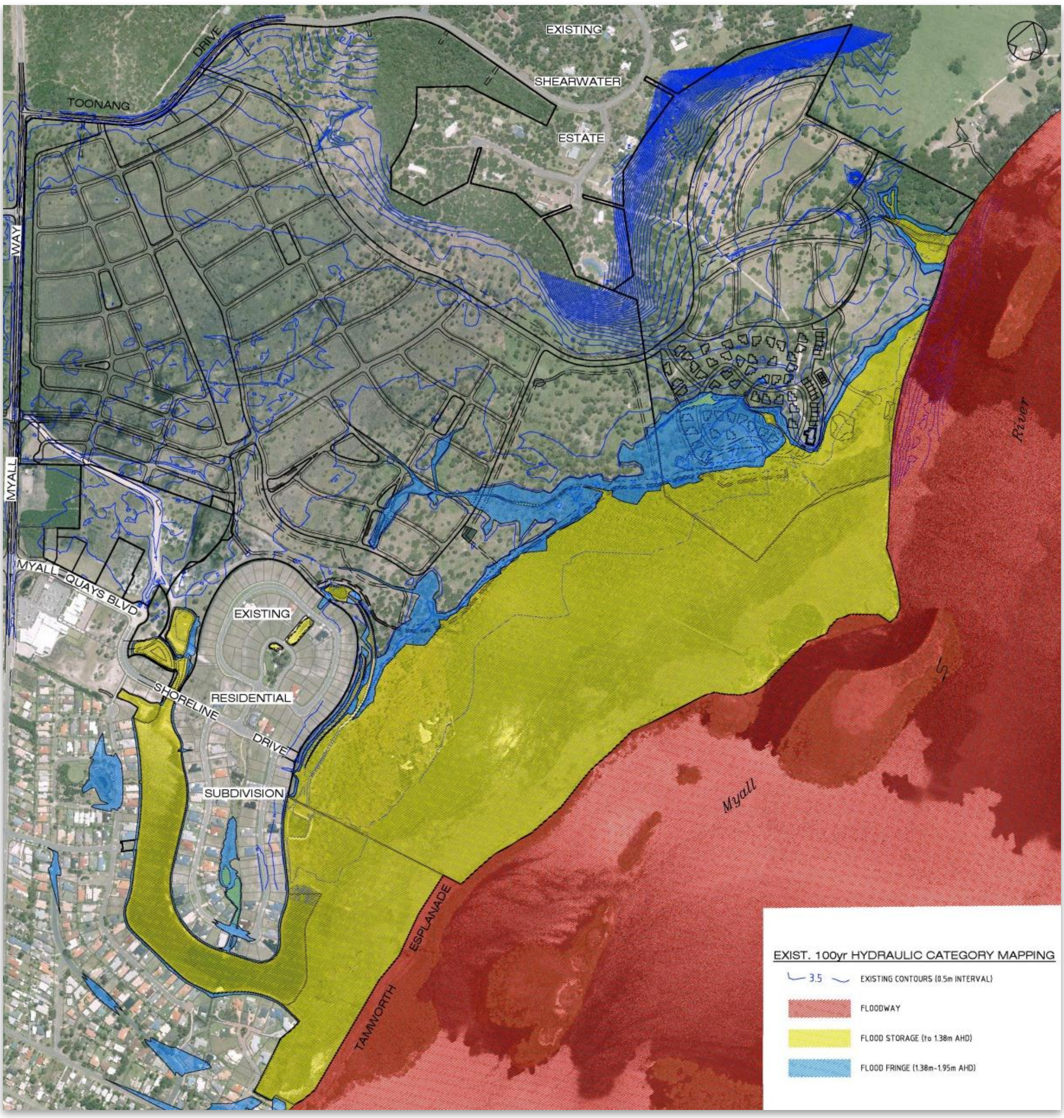

Figure 21 – Interim Pre-Development Existing River Flood Hydraulic Category Mapping

Adopting 2.8m as the critical 2100 river level would see flood waters across a significant proportion of the site if it were to remain undeveloped, up to 0.8m in depth. Within this area the Riverside proposal would see approximately 100,000cu.m net fill within the floodplain.

However, given that this critical level is almost entirely governed by tidal influence rather than river flood conveyance, storage volume lost on the development site is inconsequential compared to the near-infinite volume of Port Stephens and the ocean beyond. As such the entire area can be classified as Flood Fringe, and proposed filling will have negligible impact on flood levels on adjoining land. The results of the design state models in Section 3.9 will further illustrate this point.

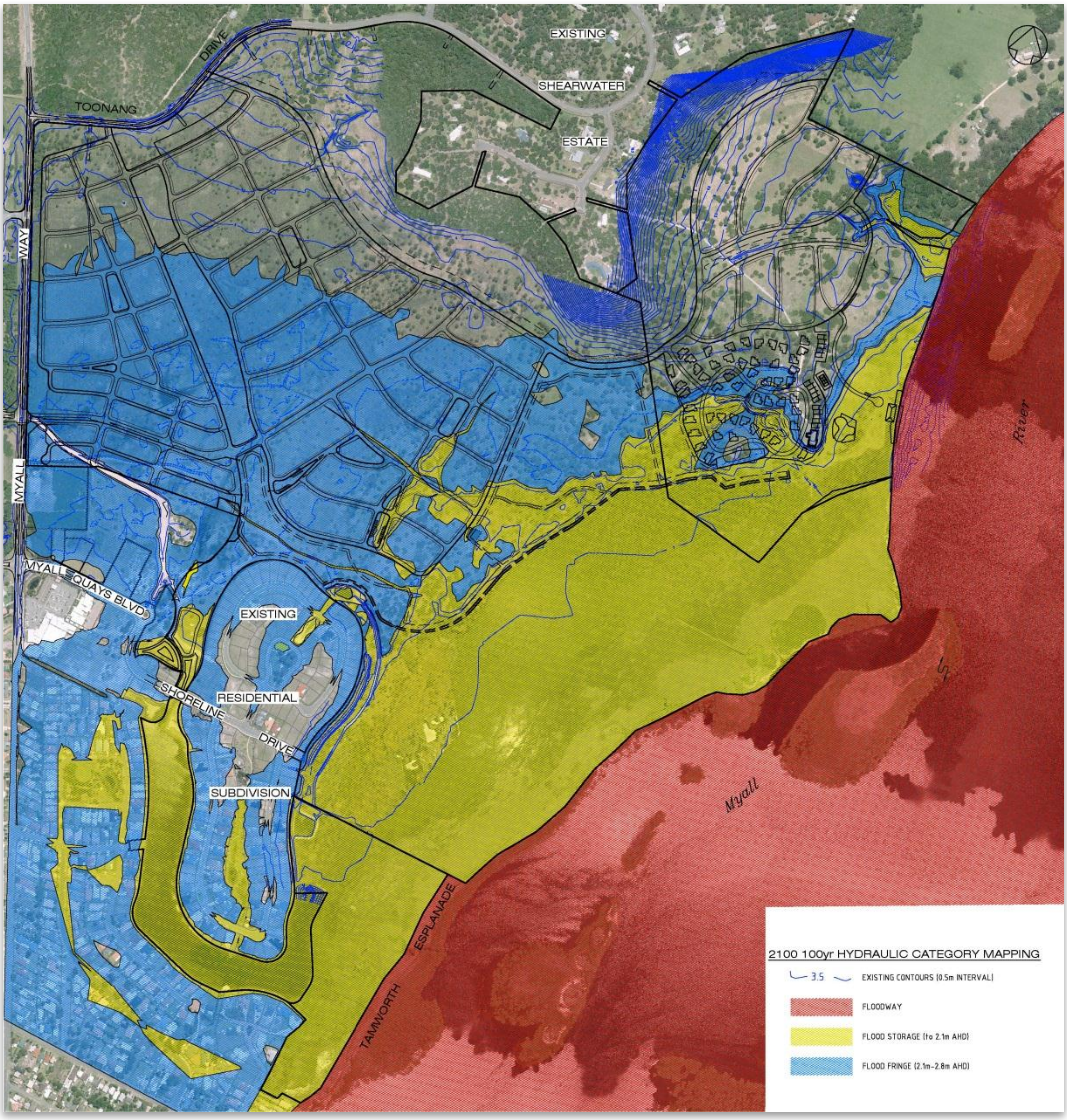

Figure 22 – Interim Pre-Development 2100 River Flood Hydraulic Category Mapping

3.9 DESIGN STORM/TAILWATER CONDITIONS

A full range of storms have been simulated across both the pre and post development models. Both the existing and proposed design catchments are complex, made up of various sub-catchments with varying critical durations. One of the benefits of a 2D analysis is that it allows more accurate modelling of these local features, rather than making broad simplifications required for 1D modelling. To identify the critical events across the site, a full range of durations were modelled for each recurrence interval.

The different events have been modelled for different purposes. A short summary is shown below;

- Maintain existing regular ‘environmental’ flows into the wetland buffer and existing lake – 0.25yr and 1yr rainfall events, existing tailwater conditions,
- Ensure no increase in potentially scouring peak flow velocities within the wetland during major storm events – 100yr storm, existing tailwater conditions
- Maintain existing flood levels in surrounding areas post development – 5yr and 20yr storms with 2100 MHW tailwater levels,
- Determine the relevant “Flood Planning Level” for the proposed development – both 100yr storm/5yr 2100 tailwater and 5yr storm/100yr 2100 tailwater combinations,
- Assess the impact of possible Climate Change induced rainfall intensity increases on the Flood Planning Level assessment
- Emergency response assessment for a ‘worst case’ extreme flood – both PMF storm event/100yr 2100 tailwater and 100yr storm/’extreme’ event tailwater combinations.

3.9.1 Environmental Flows – Replicating Minor/Regular Rainfall Events

Ensuring regular existing discharge rates into the adjoining wetland are replicated post-development will be important in order to maintain wetland health. Both Pre and Post development models were set up to simulate quarterly and annual rainfall events with existing MHW (0.5m AHD) as the downstream tailwater condition.

It is proposed to control the regular release of stormwater into the downstream wetland via a constructed level spreader, which incorporates two concentrated low-flow outlets (box culverts). These culverts are positioned at the same location that existing site drains flow into the wetland (identified on Figures 10 and 11). The dimensions of the discharge culverts and level spreader crest have been provisionally sized with the aid of model results to approximate existing conditions. It is important to note that this is a concept design and further refinement at detail design stage should allow an even closer replication of existing conditions.

The results of both pre and post development quarterly and annual rainfall events are presented below. Critical flow peaks are highlighted in yellow. While pre and post development flows do not match exactly, they are sufficiently close to demonstrate that with further fine-tuning during detail design, the existing flow regime should be able to be replicated to a sufficient degree to match pre development flows.

The only area with significantly varied flows post development is the flows entering the existing lake. This is a reflection of the removal of the temporary diversion drains and haulage road that currently block this flowpath. Review of the 'Wetland 6' results shows that the additional flow into the lake does not directly translate to outlet discharge increases due to the significant detention capacity afforded by the lake. It should also be noted that discharge from the existing lake into the Myall River is via an existing defined drainage channel rather than feeding the fringing wetland.

Table 5: Pre and Post Development Quarterly Peak Flows

	2hr		3hr		6hr		9hr		12hr		18hr	
Location	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post
Peak Flow (m^3/s)												
Lake	0	0.04	0	0.04	0	0.03	0	0.05	0	0.04	0	0.02
Creek	0.1	0.12	0.15	0.12	0.15	0.14	0.22	0.20	0.19	0.21	0.1	0.18
River	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.03	0.03	0.03	0.02	0.01
Wetland 1	0.01	0.03	0.02	0.02	0.03	0.04	0.07	0.05	0.06	0.05	0.05	0.04
Wetland 2	0.3	0.40	0.36	0.41	0.43	0.43	0.57	0.55	0.53	0.50	0.49	0.44
Wetland 3	0.01	0.08	0.01	0.07	0.01	0.04	0.05	0.04	0.10	0.13	0.05	0.12
Wetland 4	0	0	0	0	0	0	0	0	0	0	0	0
Wetland 5	0	0	0	0	0	0	0.01	0.01	0.01	0.01	0.01	0
Wetland 6	0.06	0.05	0.035	0.06	0.17	0.04	0.05	0.01	0.03	0.11	0.02	0.14
Overall Discharge	0.35	0.60	0.5	0.60	0.73	0.65	0.75	0.78	0.82	0.92	0.65	0.87

Table 6: Pre and Post Development Annual Peak Flows

	2hr		3hr		6hr		9hr		12hr		18hr	
Location	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post
Flow (m^3/s)												
Lake	0.03	0.16	0.1	0.13	0.01	0.18	0.01	0.61	0.01	0.67	0	0.56
Creek	0.9	0.57	0.75	0.55	0.76	0.55	0.95	0.70	0.82	0.72	0.52	0.50
River	0.09	0.16	0.09	0.10	0.09	0.10	0.16	0.10	0.12	0.10	0.07	0.06
Wetland 1	0.22	0.16	0.21	0.16	0.23	0.18	0.3	0.26	0.27	0.22	0.2	0.14
Wetland 2	1.12	1.35	1.3	1.39	1.4	1.52	1.75	1.92	1.72	1.80	1.6	1.59
Wetland 3	0.42	0.34	0.47	0.31	0.49	0.35	0.5	0.37	0.54	0.36	0.23	0.35
Wetland 4	0.11	0.05	0.01	0.02	0.06	0.07	0.07	0.10	0.13	0.14	0.11	0.11
Wetland 5	0.03	0.03	0.04	0.04	0.04	0.04	0.07	0.08	0.13	0.07	0.13	0.05
Wetland 6	0.13	0.23	0.18	0.11	0.33	0.22	0.2	0.38	0.51	0.55	0.46	0.48
Overall Discharge	1.8	2.52	2.3	2.19	2.8	2.68	3.25	3.21	3.75	3.34	3.25	2.95

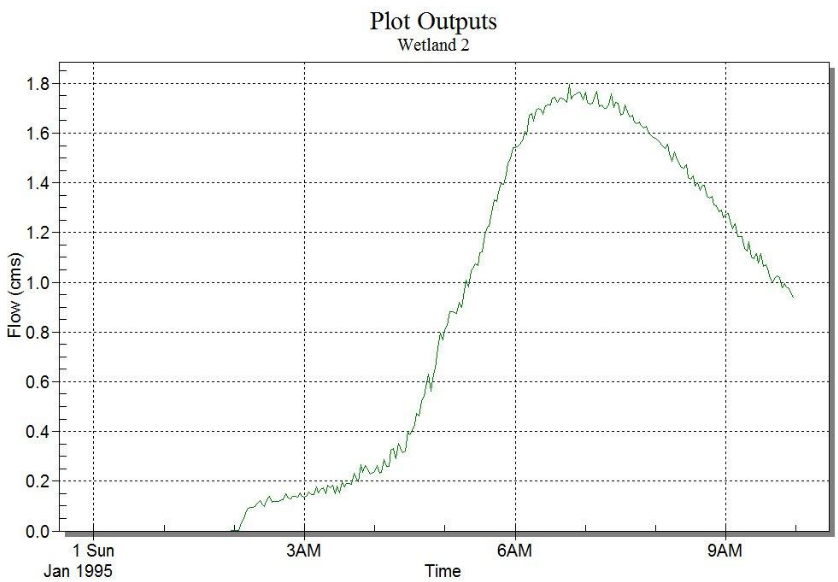

Figure 23 – Example Existing Discharge Hydrograph – 'Wetland 2' Flow Line 1yr 9hr Storm

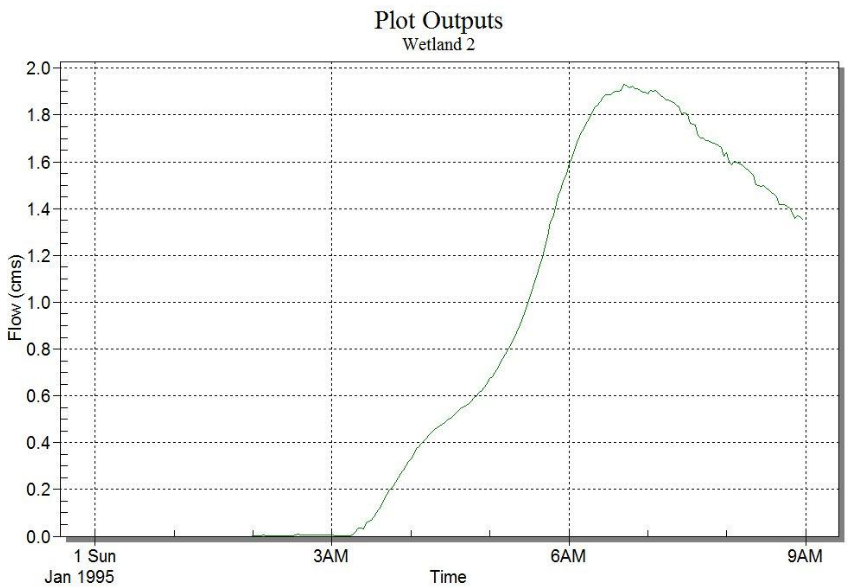

Figure 24 – Example Design Discharge Hydrograph – 'Wetland 2' Flow Line 1yr 9hr Storm

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 tailwater combination.

The figures below show peak velocities in the downstream wetland buffer in the worst case scenario pre and post development. While the proposed Riverside development will increase the total amount of water discharging in a 100yr event, the installation of the level spreader weir along the full frontage of the wetland buffer allows the peak flows to be better distributed along the full downstream interface.

As a result worst case post development peak velocities are equal to or lower than pre-development values. Additionally, it should be remembered that the SEPP14 wetland is still over 250m further downstream from these discharge points.

The proposed Riverside development will not result in an increase in peak flow velocities that could lead to erosion within the SEPP14 wetlands.

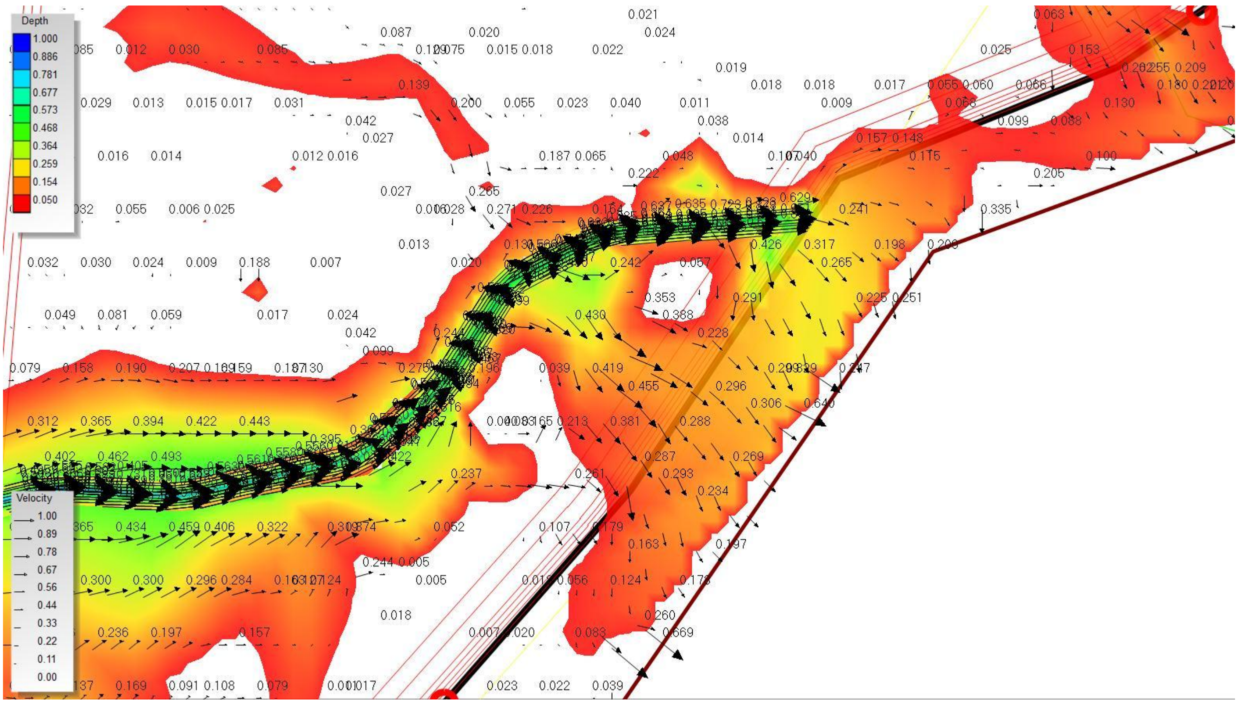

Figure 25 – Critical Flood Depths/Velocities (Velocity values displayed) at Existing Drain Wetland Outlet - 2hr 100yr Storm, Current MHW Tailwater

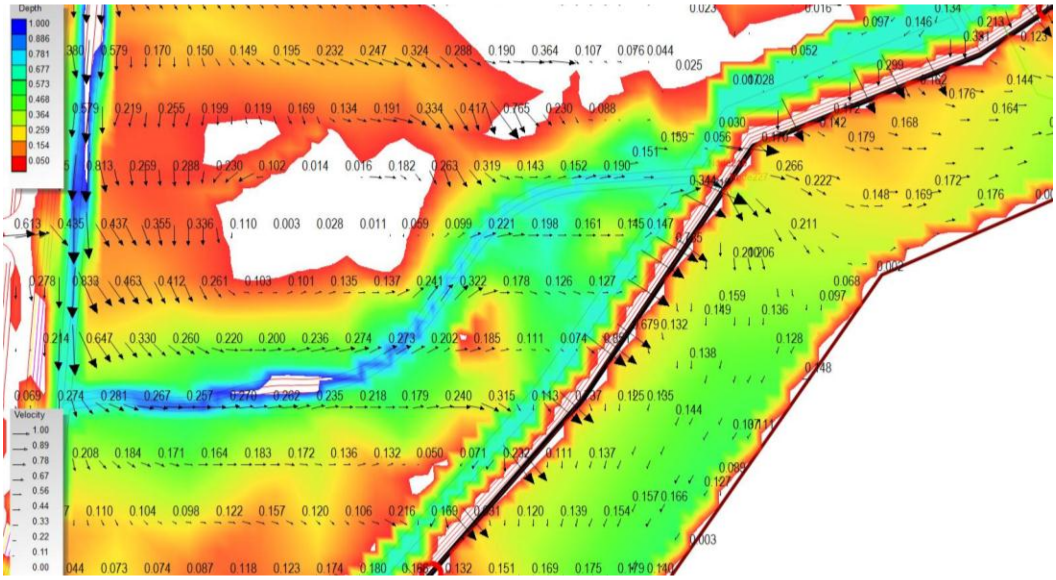

Figure 26 – Critical Flood Depths/Velocities (Velocity values displayed) At 'Discharge Culvert A' - 2hr 100yr Storm, Current MHW Tailwater

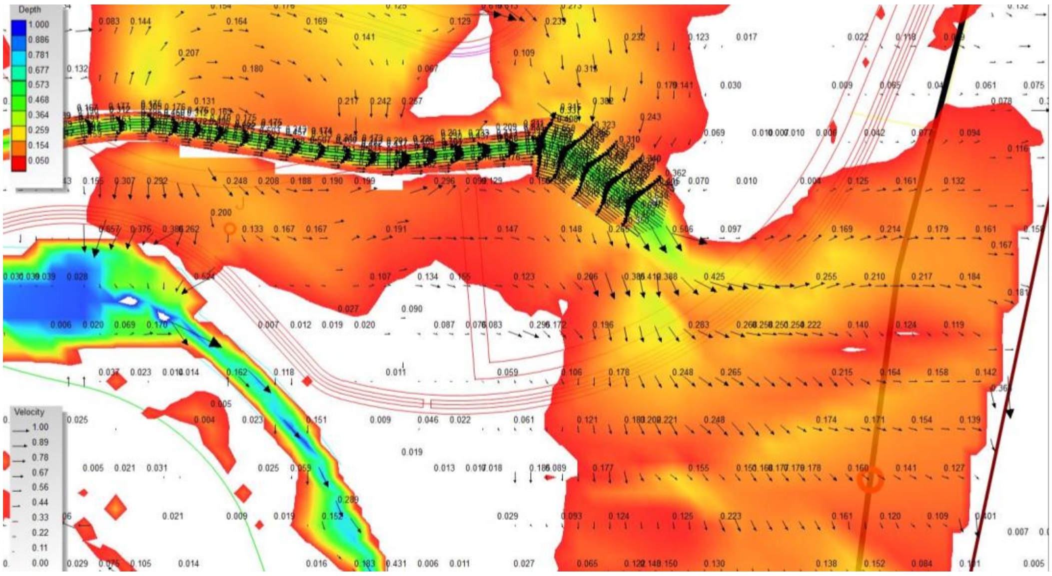

Figure 27 – Critical Flood Depths/Velocities (Velocity values displayed) At 'Discharge Culvert B' - 2hr 100yr Storm, Current MHW Tailwater

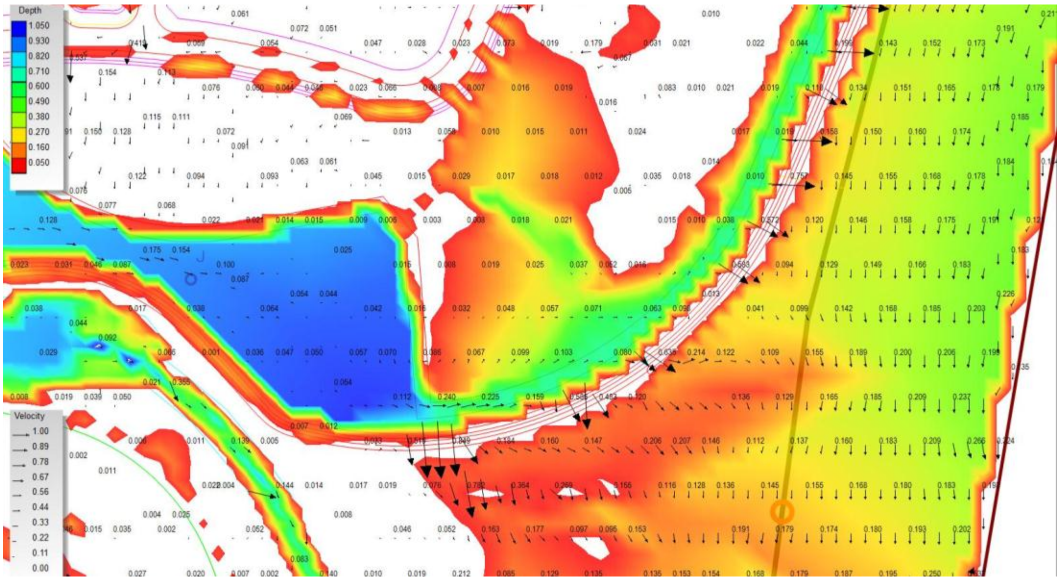

Figure 28 – Critical Flood Depths/Velocities (Velocity values displayed) At 'Discharge Culvert B' - 2hr 100yr Storm, Current MHW Tailwater

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 Upstream Northern Catchment

The upstream catchment to the North is significantly elevated and consists of low density development. The existing culverts under Toonang Drive will remain free draining, and no impact is expected.

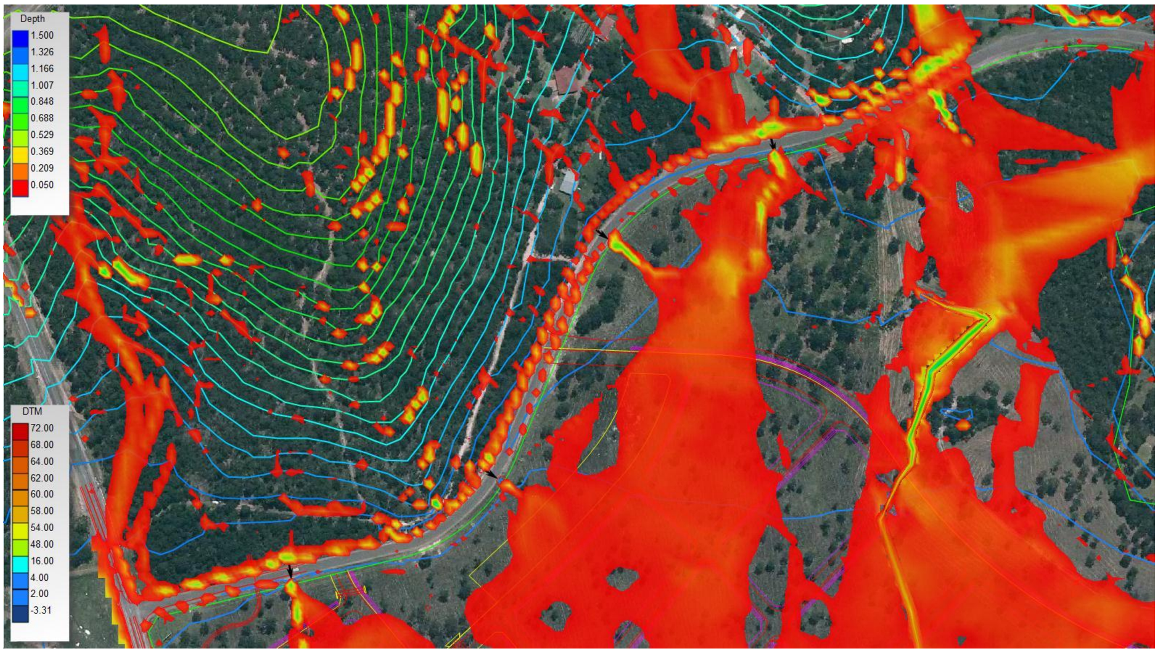

Figure 29 – Pre Development Critical Flood Depths At Existing Toonang Drive Culverts - 2hr 100yr Storm, 2100 5yr Tailwater (Depth<0.05m filtered out)

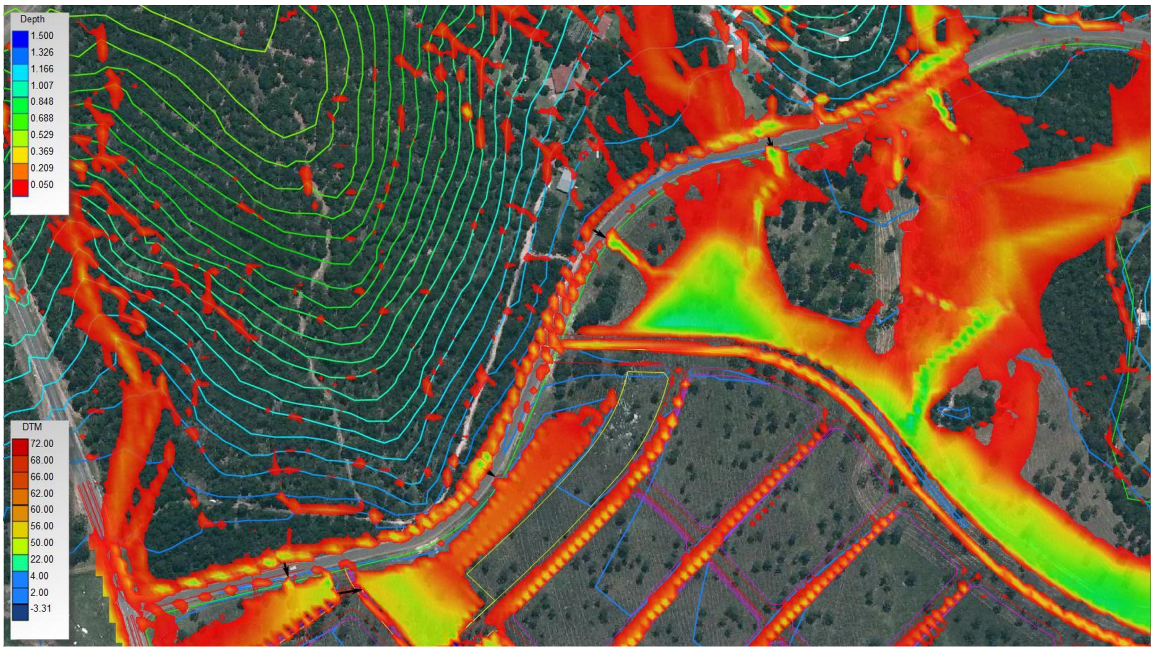

Figure 30 – Post Development Critical Flood Depths At Existing Toonanag Drive Culverts - 2hr 100yr Storm, 2100 5yr Tailwater (Depth<0.05m filtered out)

3.9.3.2 Adjacent Western Lands

The development site is bordered by Myall Street to the West – this road is also the catchment watershed, and as such land to the west of the site will not be impacted by the Riverside development. Proposed construction works as part of the Riverside development will partially raise Myall Street and further define this as the catchment boundary.

As discussed in Section 3.3.1, the two existing minor exceptions to this are two small culverts that drain the eastern roadside table drain to the west. The more northern of these culverts is adjacent to the Riverside development, and will be removed as part of the development construction works. The southern culvert under Myall Street will remain drains a small discrete catchment to the west. This catchment has no connection to the east, and as such the proposed Riverside Estate will have no impact on it.

Photograph 2 – Small existing culvert will be removed adjacent to Riverside development

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).

This 1.4m AHD 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 this tailwater scenario, the lake and river would become a continuous water body at mean high water if no other mitigating devices were introduced to protect the existing adjacent township from flooding. It is expected that any variation in incoming flows will be negated by the shallow but broad connection to the river and substantial detention capacity available in the 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.

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

	2hr		3hr		6hr		9hr		12hr		18hr	
Location	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post
K (Bebo)	2.28	1.89	2.30	1.91	2.31	1.91	2.31	1.95	2.31	1.92	2.30	1.90
L (Lake)	1.40	1.40	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41
M (Lake)	1.40	1.40	1.41	1.41	1.41	1.41	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	1.40	1.40	1.40	1.40

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

	2hr		3hr		6hr		9hr		12hr		18hr	
Location	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post
K (Bebo)	2.34	1.95	2.35	1.96	2.35	2.00	2.37	2.02	2.35	1.98	2.33	1.93
L (Lake)	1.42	1.42	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.41	1.40	1.40
M (Lake)	1.41	1.41	1.42	1.41	1.42	1.41	1.41	1.41	1.41	1.41	1.40	1.40
N (Wetland)	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40

It can be seen that the proposed Riverside development will not have a notable impact on flood levels in the adjoining lake or wetland buffer area in these rainfall events. The Riverside development will not have a negative impact on flood levels within the adjoining development.

3.9.4 100 yr Flood Planning Level

In reference to the recently completed Port Stephens Design Flood Levels Climate Change Review by WMA Water 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.2.0m AHD
- 100yr Event 2; 5yr catchment storm with a 100yr 2100 river level of RL.2.8m AHD

Correspondence from Council supporting the adopted tailwater levels and probability combinations can be seen in Appendix C.

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.

A general observation of modelling results shows that Event 1 was the critical scenario only in the highest parts of the Riverside site, near Toonang Drive. Event 2 was critical across the majority of the development site (ie modelled flood levels were higher than Event 1). Flood flows are restricted to designated floodways and roadways, with no lots being affected by either of the 100yr flood events.

Table 9: Developed State 100yr Peak Flood Levels

Location	1hr		2hr		3hr		6hr		9hr		12hr	
	Event 1	Event 2	Event 1	Event 2	Event 1	Event 2	Event 1	Event 2	Event 1	Event 2	Event 1	Event 2
Flood Level (m AHD)												
A	6.31	6.16	6.33	6.18	6.26	6.12	6.22	6.12	6.20	6.11	6.20	6.11
B	4.30	4.04	4.34	4.05	4.18	3.96	4.21	3.93	4.09	3.92	4.09	3.92
C	3.94	3.67	3.95	3.68	3.84	3.62	3.82	3.10	3.78	3.60	3.77	3.60
D	3.76	3.49	3.78	3.50	3.66	3.45	3.63	3.44	3.60	3.44	3.59	3.44
E	2.15	2.80	2.15	2.80	2.14	2.80	2.14	3.80	2.13	2.80	2.13	2.80
F	3.03	3.02	3.04	3.02	2.92	2.96	2.91	2.94	2.92	2.92	2.91	2.92
G	2.78	2.99	2.94	3.00	2.87	2.94	2.88	2.92	2.89	2.91	2.88	2.91
H	2.14	2.93	2.80	2.93	2.77	2.90	2.78	2.89	2.80	2.88	2.77	2.88
I	2.08	2.81	2.15	2.81	2.14	2.80	2.15	2.80	2.16	2.81	2.15	2.81
J	2.06	2.81	2.09	2.81	2.08	2.80	2.09	2.80	2.09	2.81	2.09	2.81
K	2.00	2.81	2.07	2.80	2.06	2.80	2.07	2.80	2.07	2.81	2.07	2.81
L	2.00	2.80	2.00	2.80	2.00	2.80	2.00	2.80	2.01	2.81	2.01	2.81
M	2.00	2.80	2.00	2.80	2.00	2.80	2.00	2.80	2.01	2.80	2.01	2.80
N	2.00	2.80	2.00	2.80	2.00	2.80	2.00	2.80	2.00	2.80	2.00	2.80
O	2.82	2.98	2.82	2.99	2.75	2.97	2.73	2.97	2.73	2.97	2.68	2.96
P	2.00	2.80	2.00	2.80	2.00	2.80	2.00	2.80	2.00	2.80	2.00	2.80
Peak Flow (m^3/s)												
Lake	4.2	2.3	4.5	2.3	4.3	1.8	4.4	1.6	4.6	1.5	4.4	1.6
Creek	3.7	2.8	3.5	2.9	3.2	1.1	3.1	1.0	2.9	0.9	3.0	1.0
River	1.7	1.1	11	1.3	0.8	1.0	0.7	0.9	0.6	0.8	0.6	0.9
Wetland 1	4.3	2.0	4.2	1.8	4.2	1.8	4.2	1.8	3.8	1.7	3.6	1.5
Wetland 2	14	5.5	12	6.0	14	5.2	12	5.0	10	4.9	12	4.0
Wetland 3	5.2	6.0	5.2	6.5	5.2	5.0	5.6	4.9	5.0	4.5	4.8	4.8
Wetland 4	4.0	4.5	4.5	5.0	3.0	3.2	3.2	2.7	3.0	2.4	3.4	2.5
Wetland 5	6.5	5.0	7.5	5.5	4.5	4.0	4.6	3.0	4.5	2.5	4.5	3.2
Wetland 6	9.0	10	13	10	7.0	8.0	7.5	8.0	7.0	5.0	8.0	5.0
Overall Discharge	33	30	38	35	36	20	35	20	34	18	33	19

To summarise the peak flows above, the following plan depicts recommended Flood Planning Levels across the Riverside site.

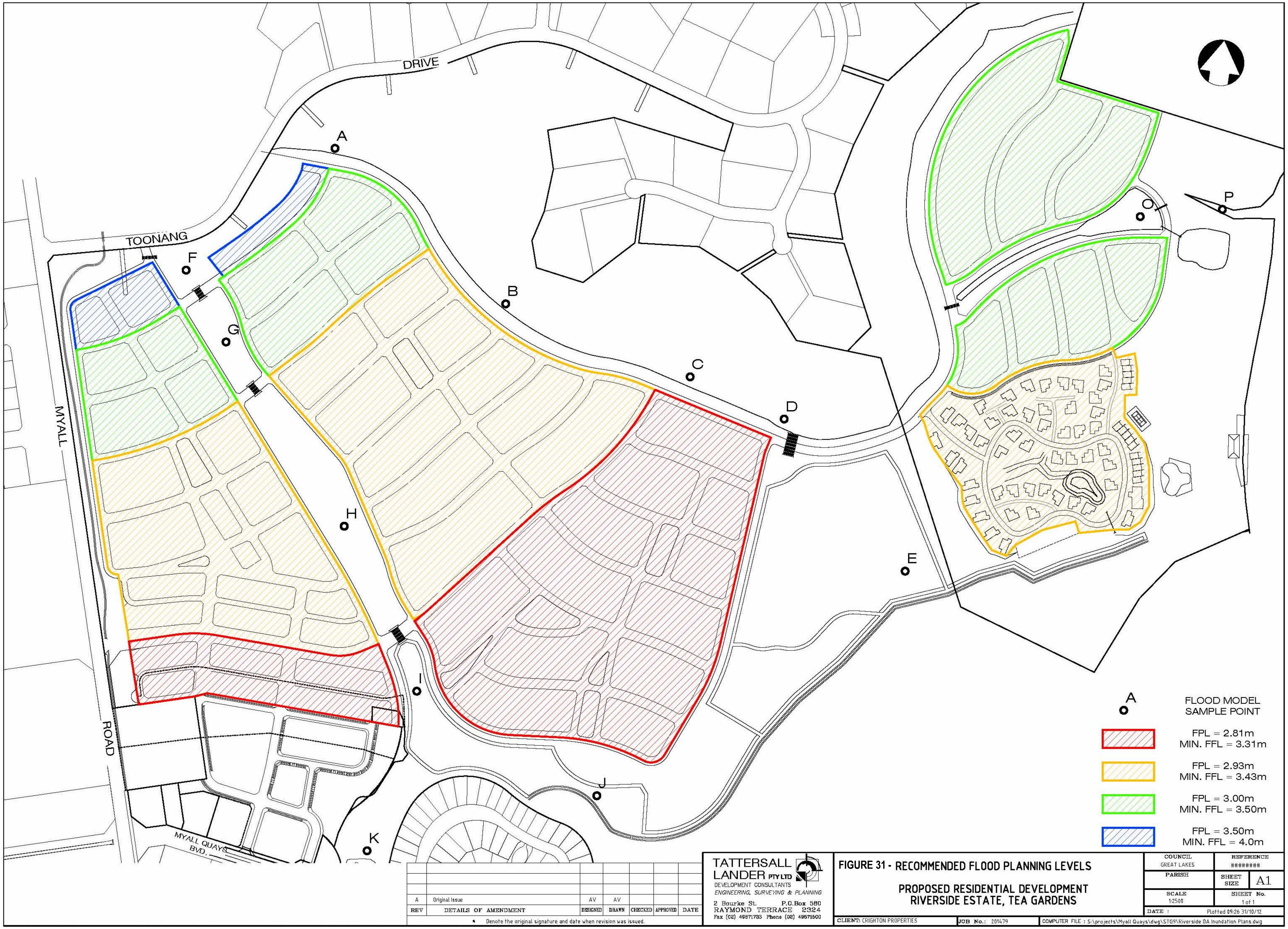

FIGURE 31 - RECOMMENDED FLOOD PLANNING LEVELS

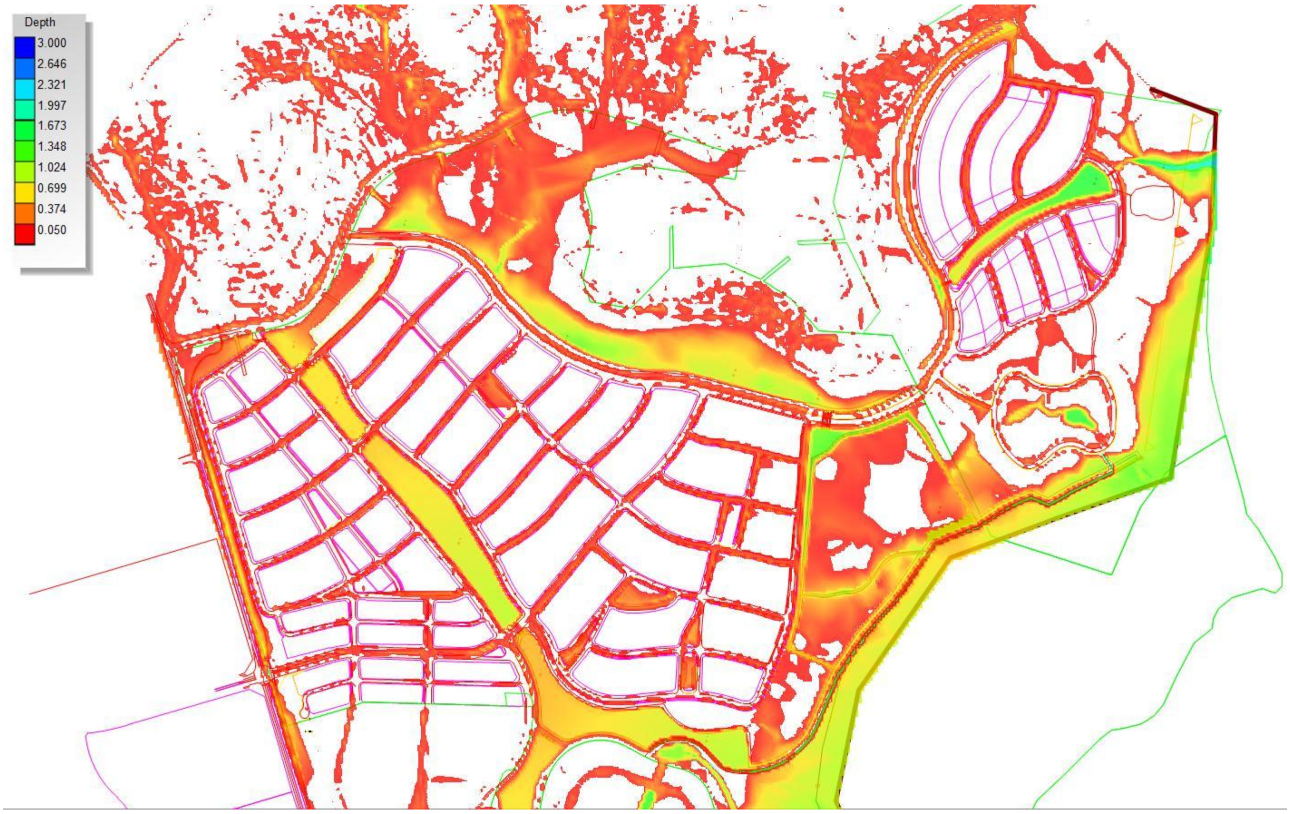

Figure 32 – Flood Depths Across the Developed Riverside Site - Event 1 Critical 2hr Storm – 100yr Storm, 2100 5yr Tailwater (Depth<0.05m filtered out)

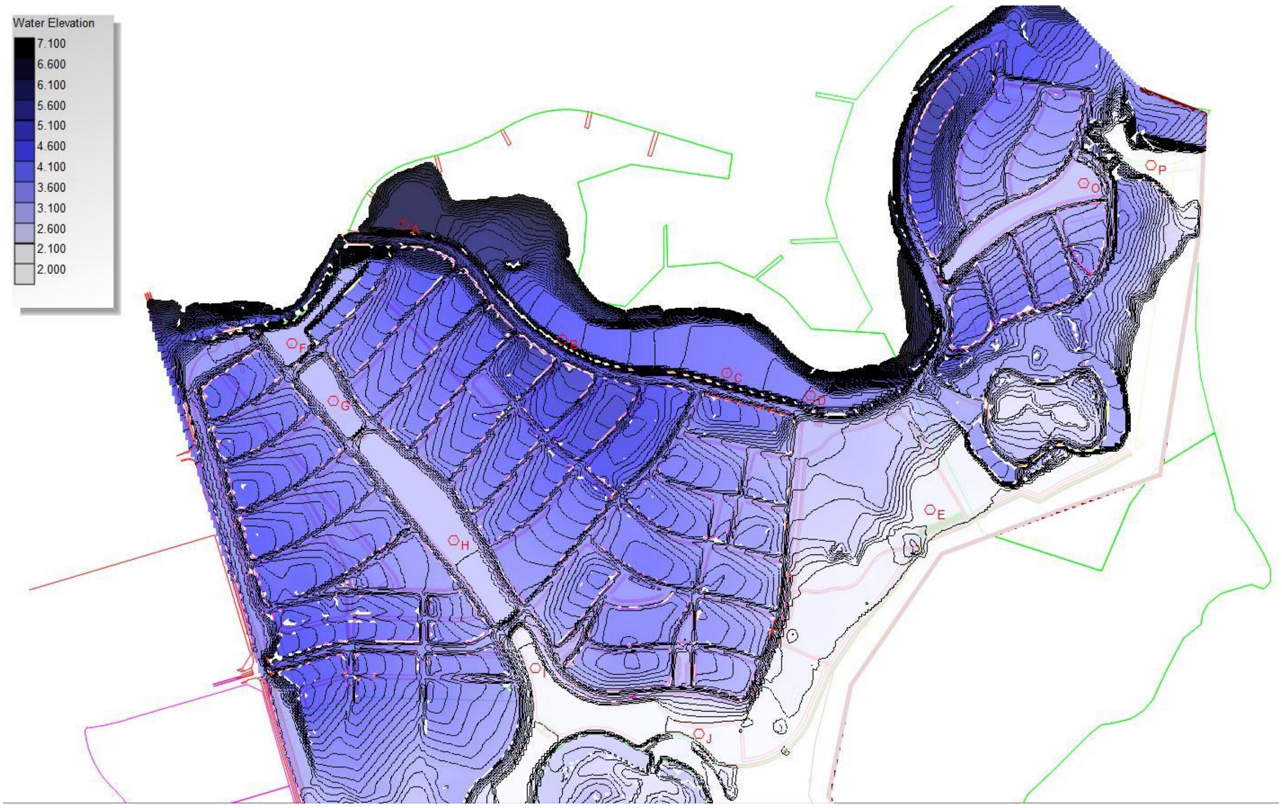

Figure 33 – Flood Level Contours Across the Developed Riverside Site – Event 1 Critical 2hr Storm – 100yr Storm, 2100 5yr Tailwater

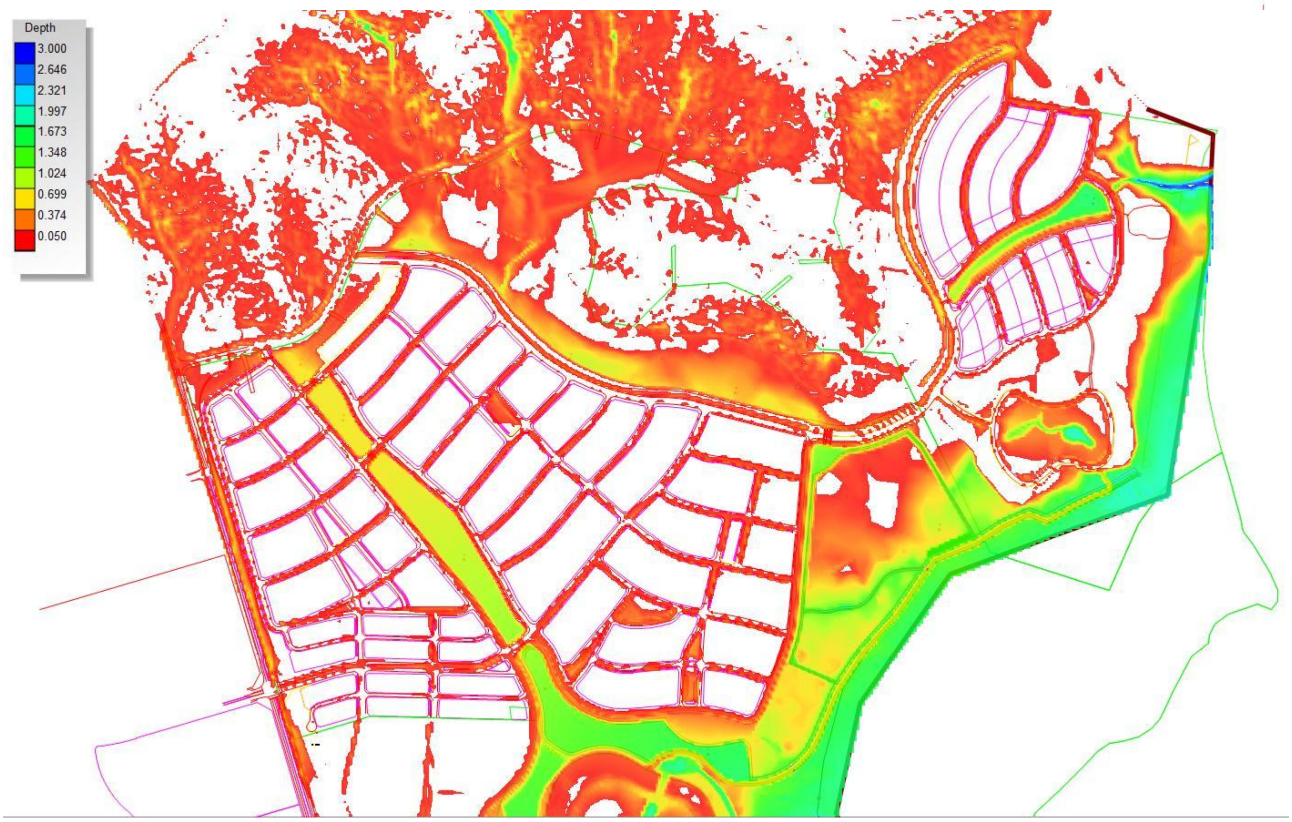

Figure 34 – Flood Depths Across the Developed Riverside Site – Event 2 Critical 2hr Storm – 5yr Storm, 2100 100yr Tailwater (Depth<0.05m filtered out)

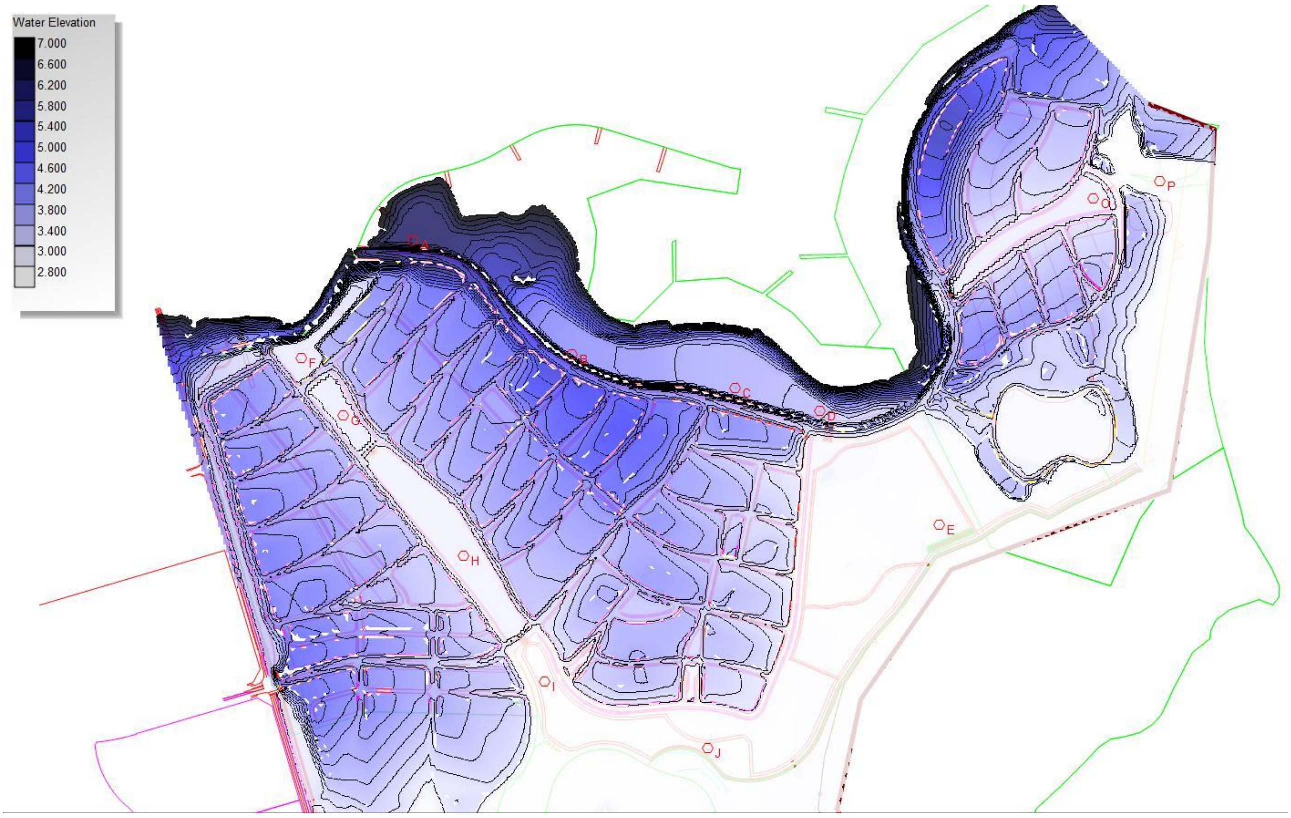

Figure 35 – Flood Level Contours Across the Developed Riverside Site – Event 2 Critical 2hr Storm – 5yr Storm, 2100 100yr Tailwater

It should be noted 100yr models are utilising 2100 downstream tailwater conditions. This results in discharges significantly higher than those shown in previous modelling work conducted by Cardno. In this work, Cardno's 100yr modelling used considerably lower receiving water levels from their own river level modelling, whereas this current modelling has used 2100 river levels as adopted by Council from the WMA Water Port Stephens Study[5].

The effect of this is that the significantly higher tailwater has already inundated the majority of the drainage network, negating any of the existing and proposed storage capacity. Incoming flows are quickly translated downstream through what is essentially a large, deep water body covering the existing lake, wetland, wetland buffer and any constructed floodway areas. As a result the 2hr event becomes critical in all areas of the catchment, rather than the 9hr storm that may be critical in other conditions where detention structures need to be filled before peak flows are achieved.

This is not, however, of great significance, as this increased flow rate does not translate into increased flood levels on adjoining properties. The 2100 100yr river level of 2.8m AHD has the capacity, if left unmitigated, to inundate the majority of the floodplain (including much of the developed township of Tea Gardens), so the increased flow rates are easily 'lost' in the vast storage of the floodplain – the extent of this modelled work only includes downstream lands as far as Coupland St, and does not include the vast floodplain to the East across to Mungo Brush Road on the other side of the river, and still the maximum increase in flood levels post development is recorded as 5mm or less (which is realistically considered beyond the accuracy of the modelling) - see reference points E, J, K, L, M and N.

3.9.4.1 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. However, advice received via Great Lakes Council from the Bureau of Meteorology and Office of Environment and Heritage is that there is insufficient evidence to justify including any intensity increases 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."

Irrespective of this advice, a series of flood models were run to illustrate the sensitivity of the Flood Planning Level to a 30% increase in rainfall intensities - the current 'worst case' plausible increase. The results from these models are listed below.

[5] WMA Water (2010) *Port Stephens Design Flood Levels – Climate Change Review*

Analysis of these results show that the highest level increases occur in the main East-West floodway (greatest at Point D – 0.28m increase), but that these increased flood levels are still contained by the proposed flood embankment. The largest differences within the development site are 0.22m (West Branch, Point F) and 0.23m (Monkey Jacket, Point O). In the lower sections of the catchment where tailwater factors dominate the flood planning levels, the impact of the intensity increase is negligible.

Table 10: Comparison of Developed State 100yr Peak Flood Levels with 5yr Tailwater Level (Flood Planning Level Scenario 1) resulting from Existing and Possible +30% Intensities

	1hr		2hr		3hr		6hr		9hr		12hr	
Location	Exist	+30%	Exist	+30%	Exist	+30%	Exist	+30%	Exist	+30%	Exist	+30%
	Flood Level (m AHD)											
A	6.31	6.4	6.33	6.41	6.26	6.34	6.22	6.29	6.20	6.26	6.20	6.27
B	4.30	4.49	4.34	4.51	4.18	4.35	4.21	4.24	4.09	4.20	4.09	4.20
C	3.94	4.15	3.95	4.17	3.84	4.03	3.82	3.96	3.78	3.91	3.77	3.90
D	3.76	4.04	3.78	4.06	3.66	3.88	3.63	3.79	3.60	3.73	3.59	3.72
E	2.15	2.18	2.15	2.19	2.14	2.18	2.14	2.17	2.13	2.16	2.13	2.14
F	3.03	3.25	3.04	3.23	2.92	3.11	2.91	3.10	2.92	3.08	2.91	3.08
G	2.94	3.14	2.94	3.13	2.87	3.05	2.88	3.05	2.89	3.04	2.88	3.03
H	2.78	2.94	2.80	2.96	2.77	2.93	2.78	2.94	2.80	2.92	2.77	2.90
I	2.14	2.22	2.15	2.23	2.14	2.22	2.15	2.22	2.16	2.22	2.15	2.21
J	2.08	2.12	2.09	2.13	2.08	2.12	2.09	2.12	2.09	2.12	2.09	2.12
K	2.06	2.10	2.07	2.11	2.06	2.10	2.07	2.10	2.07	2.10	2.07	2.10
L	2.01	2.01	2.01	2.01	2.01	2.01	2.01	2.01	2.01	2.01	2.01	2.01
M	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.01	2.01	2.01	2.01	2.01
N	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
O	2.82	3.02	2.82	3.03	2.75	2.97	2.73	2.96	2.73	2.94	2.68	2.89
P	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00

Given the current uncertainty surrounding the possibility of rainfall intensity increases, and the fact these values represent the upper limit of the estimated intensity increases, these possible flood level variations are still well within the Council's adopted 0.5m freeboard applied to the adopted Flood Planning Levels.

The results listed above should 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.

3.9.5 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.

3.9.5.1 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.

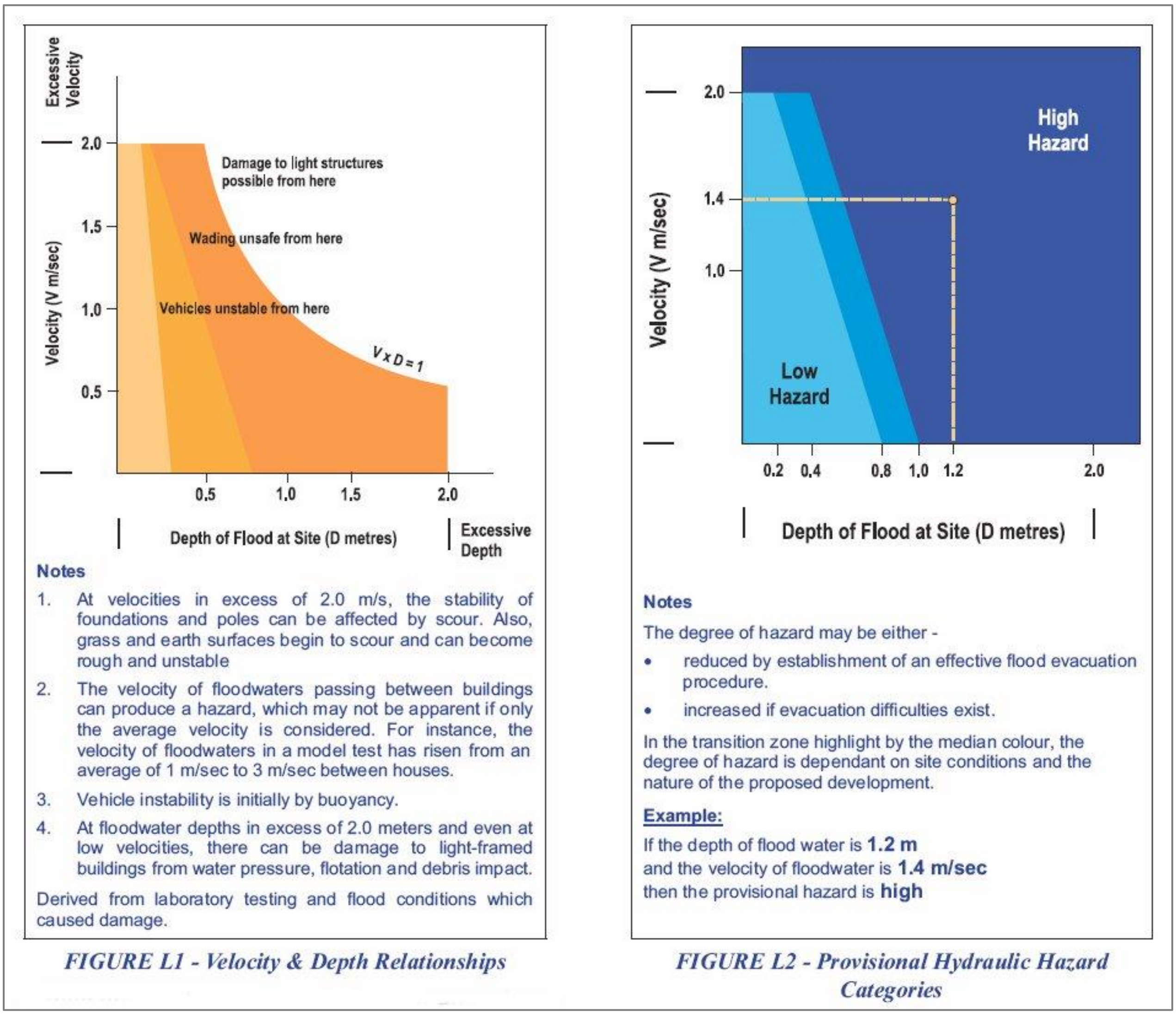

Figure 36 – Hazard Determination (NSW Government, 2005)

Additionally, the recently prepared Australian Rainfall and Runoff – Project 10: Appropriate Safety Criteria for People (2010) offers the following summary floodway safety

<table>
<tr><th>DV (m^2s^{-1})</th><th>Infants, small children (H.M ≤ 25) and frail/older persons</th><th>Children (H.M = 25 to 50)</th><th>Adults (H.M > 50)</th></tr>
<tr><td>0</td><td>Safe</td><td>Safe</td><td>Safe</td></tr>
<tr><td>0 – 0.4</td><td rowspan="5">Extreme Hazard; Dangerous to all</td><td>Low Hazard[1]</td><td rowspan="2">Low Hazard[1]</td></tr>
<tr><td>0.4 – 0.6</td><td>Significant Hazard; Dangerous to most</td></tr>
<tr><td>0.6 – 0.8</td><td rowspan="3">Extreme Hazard; Dangerous to all</td><td>Moderate Hazard; Dangerous to some[2]</td></tr>
<tr><td>0.8 – 1.2</td><td>Significant Hazard; Dangerous to most[3]</td></tr>
<tr><td>> 1.2</td><td>Extreme Hazard; Dangerous to all</td></tr>
</table>

[1] Stability uncompromised for persons within laboratory testing program at these flows (to maximum flow depth of 0.5 m for children and 1.2 m for adults and a maximum velocity of 3.0 ms^{-1} at shallow depths).
[2] Working limit for trained safety workers or experienced and well equipped persons (D.V < 0.8 m^2s^{-1})
[3] Upper limit of stability observed during most investigations (D.V > 1.2 m^2s^{-1})

Figure 37 – Flow Hazard Regime for Infants, Children and Adults (Engineers Australia, 2010)

3.9.5.2 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 dangerous flows (VxD>0.4) are almost non-existent and only seen in the East-West Drainage Branch and other drains external to the main residential areas.

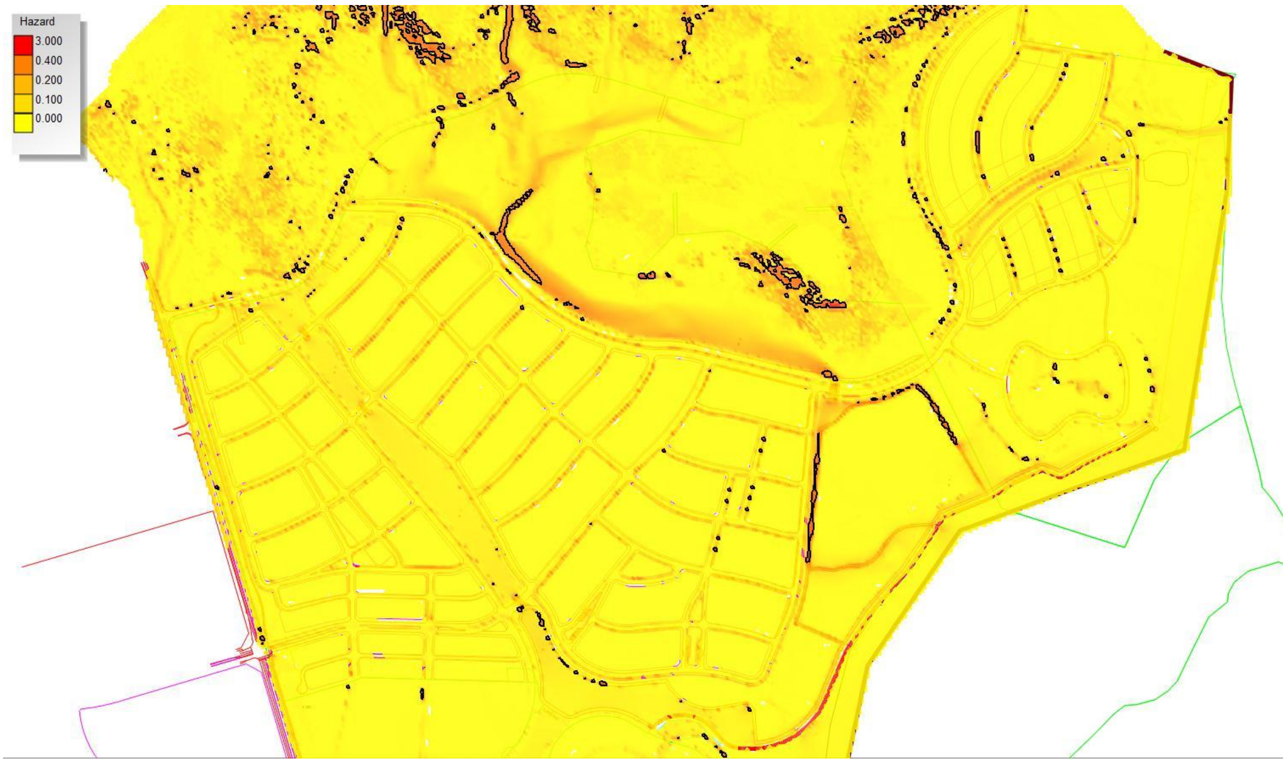

Figure 38 – Critical Flood Hazard Mapping Across the Developed Riverside Site – 2hr 5yr Storm, 2100 MHW Tailwater (VxD=0.4 contour highlighted)

3.9.5.3 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. Potentially dangerous flows (VxD>0.4) are only seen in the East-West Drainage Branch and other drains which are both external to the development, where egress is not required during a flood event. No road crossings are affected by dangerous flows.

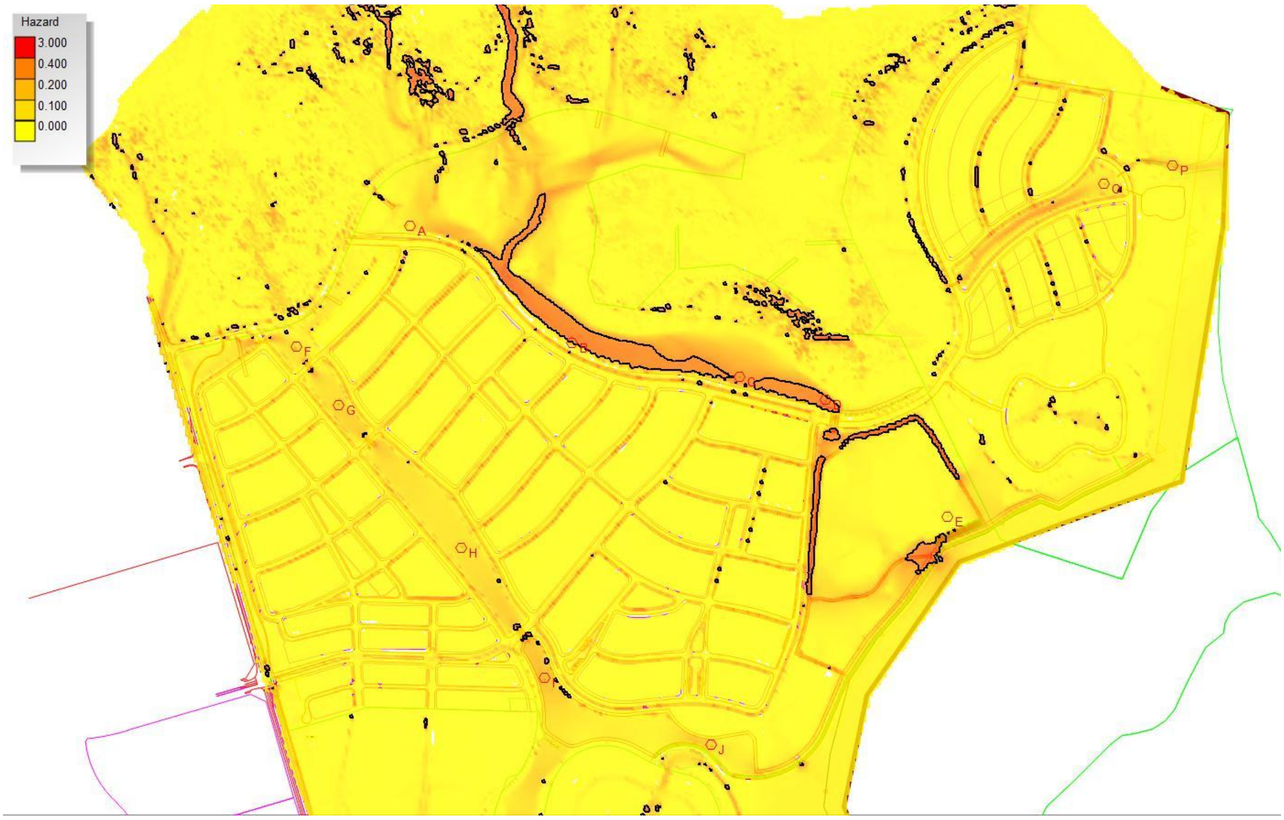

Figure 39 – Critical Flood Hazard Mapping Across the Developed Riverside Site – 2hr 100yr Storm, 2100 5yr Tailwater (VxD=0.4 contour highlighted)

3.9.5.4 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.

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.2.8m AHD
- PMF Event 2; 100yr catchment storm with an ‘extreme’ 2100 river level of RL.3.3m AHD

Tailwater heights were adopted from the Singing Bridge levels in the WMA Water report[6], plus 0.1m to account for river gradient. While the adopted ‘extreme’ event is technically not a PMF event, this value represents the most current data available at present. Both the adopted tailwater levels and the combined probability scenarios were discussed with and agreed to by Great Lakes Council.

The following figures illustrate flood depths and hazard details in the critical areas of the development for their respective critical duration PMF flood event. Complete Flood Depth and Flood Hazard mapping plans for all PMF flood event are shown in Appendix E.

Minor inundation (>0.2m) 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 into the West Branch. Flow depths along all local streets are limited to a maximum of 0.3m, and dangerous flow hazard (VxD>0.4) are generally only seen in the designated floodway channels.

[6] WMA Water (2010) *Port Stephens Design Flood Levels – Climate Change Review*