<table>
<tr><th></th><th></th><th></th><th>Response</th></tr>
<tr><td>1</td><td>Table 5.3</td><td>The area of C2 results to be 7.29 ha (5.10 + 2.19), which is different from 8.07 ha in Table 5.2.While this might not change the outcome of the study, it is recommended to check the values reported in this table.</td><td>
The areas for Catchment C2 have been incorrectly transposed from excel to word.
We apologise for this error.
The XPstorm modelling has been reviewed and the model has been based on 5.652 ha and 2.422 ha = 8.074 ha
This error can be easily corrected with an amendment to Table 5.3.
Table 5.3 Developed Case – xpstorm Catchment Details
<table>
<tr><th colspan="2" rowspan="2">Catchment ID</th><th colspan="2">Impervious Area</th><th colspan="2">Pervious Area</th></tr>
<tr><th>Area (ha)</th><th>Slope (%)</th><th>Area (ha)</th><th>Slope (%)</th></tr>
<tr><td rowspan="2">External</td><td>Ext A(1)</td><td>1.59</td><td>0.50</td><td>5.85</td><td>0.25</td></tr>
<tr><td>Ext C(1)</td><td>0.63</td><td>0.21</td><td>6.72</td><td>0.21</td></tr>
<tr><td rowspan="5">Internal (Developable Area)</td><td>A</td><td>0.21</td><td>0.50</td><td>0.09</td><td>0.50</td></tr>
<tr><td>B</td><td>12.27</td><td>0.50</td><td>5.26</td><td>0.50</td></tr>
<tr><td>B(Road Access)</td><td>0.09</td><td>1.50</td><td>0.04</td><td>1.50</td></tr>
<tr><td>C1</td><td>0.29</td><td>0.50</td><td>0.12</td><td>0.50</td></tr>
<tr><td>C2</td><td>5.10</td><td>0.50</td><td>2.19</td><td>0.50</td></tr>
<tr><td rowspan="6">Internal (remainder of site area)</td><td>B2</td><td>-</td><td>-</td><td>0.51</td><td>0.23</td></tr>
<tr><td>B3</td><td>-</td><td>-</td><td>0.77</td><td>0.31</td></tr>
<tr><td>C3</td><td>-</td><td>-</td><td>11.04</td><td>0.40</td></tr>
<tr><td>D</td><td>-</td><td>-</td><td>0.58</td><td>0.50</td></tr>
<tr><td>E</td><td>-</td><td>-</td><td>2.61</td><td>0.37</td></tr>
<tr><td>F</td><td>-</td><td>-</td><td>0.62</td><td>0.23</td></tr>
</table>
C2 imp = 5.652, per 2.422
</td></tr>
<tr><td>2</td><td>Table 5.6</td><td>The percentage difference between the two models does not seem correct. The flow value for B3 calculated with the rational method might not be correct.

It is recommended to check all the values reported in this table</td><td>
The peak discharge for Catchment C3 has been incorrectly entered into the table.
We apologise for this error.
The XPstorm modelling has been reviewed and the model produces an expected peak discharge of 0.22 m^3/s for this catchment.
This error can be easily corrected with an amendment to Table 5.6.
Table 5.6 Developed Case – Peak Flow Validation xpstorm vs Rational
<table>
<tr><th colspan="2">Catchment/Point of Discharge</th><th>xpstorm (m^3/s)</th><th>Rational (m^3/s)</th><th>Difference (%)</th></tr>
<tr><td rowspan="2">External</td><td>Ext A(1)</td><td>1.68</td><td>1.45</td><td>14.3</td></tr>
<tr><td>Ext C(1)</td><td>1.00</td><td>1.24</td><td>-19.8</td></tr>
<tr><td rowspan="11">Internal</td><td>A</td><td>0.22</td><td>0.20</td><td>8.9</td></tr>
<tr><td>B</td><td>12.19</td><td>10.42</td><td>14.5</td></tr>
<tr><td>B(Access Road)</td><td>0.10</td><td>0.09</td><td>11.5</td></tr>
<tr><td>B2</td><td>0.12</td><td>0.11</td><td>5.1</td></tr>
<tr><td>B3</td><td>3.91</td><td>0.19</td><td>13.6</td></tr>
<tr><td>C1</td><td>0.30</td><td>0.25</td><td>18.1</td></tr>
<tr><td>C2</td><td>5.64</td><td>5.13</td><td>9.1</td></tr>
<tr><td>C3</td><td>1.66</td><td>1.49</td><td>10.2</td></tr>
<tr><td>D</td><td>0.12</td><td>0.11</td><td>5.1</td></tr>
<tr><td>E</td><td>0.46</td><td>0.41</td><td>11.1</td></tr>
<tr><td>F</td><td>0.12</td><td>0.11</td><td>8.3</td></tr>
</table>
0.22
</td></tr>
<tr><td>3</td><td>Table 6.2</td><td>The last column on the right is titled “High Flow Weir”; however, some of the structures listed in this column are not weirs. Also, some units seem to be missing for C1 – Miles Street Swale.

There seems to be some differences between the number and size of pits and pipes in the table and the model provided. For example, in the TUFLOW model provided, in Basin C there are 4 of 900x600 pits (RL 1.96 m) for the high flow and the pit 900x900 (RL 1.55 m) is for the low flow. The invert levels at 0.8 m AHD are for the downstream (DS) side of the pipes and not upstream (US). Likewise, differences were found for some of the pits and pipes in Basin B.

It is recommended to check that the sizes and dimensions in Table 6.2 of BIOME (2023) coincide with those used in the model.</td><td>
The last column titled “High Flow Weir” can be amended to “High Flow Structures”.
Units can be added to the dimensions marked in red below.
It is recognised that there is an inconsistency between the outlet configuration included within the TUFLOW model and that relied upon for XPstorm modelling. This inconsistency however has no impact on the TUFLOW results as the structure does not engage during the regional critical 1% AEP. The predicted peak depth in the basin during this event is < 1 m. The crest of the structure is 1.16 m above the basin surface.
XPstorm was relied upon for sizing of the detention basin outlets to mitigate the local storm events (critical duration for the 1% AEP 180 min). The outlet contained within Table 6.2 accurately reflects the structures included within the XPstorm modelling.
The TUFLOW model has been relied upon to assess downstream impacts in a regional context and has a different critical duration for the 1% AEP.
The structures in the TUFLOW can be amended however as discussed above this amendment will not alter the predicted results.
</td></tr>
</table>

<table>
<tr>
<td></td>
<td></td>
<td></td>
<td>
Table 6.2 Outlet Structures
<table>
<tr><th></th><th colspan="2">Low Flow Outlet</th><th>High Flow</th></tr>
<tr><th>Detention Basin ID</th><th>Pit Orifice</th><th>Pit & Pipe Outlet</th><th>High Flow Weir</th></tr>
<tr><td>Basin B</td><td>2x0.2mx0.35m rectangular orifice
@ RL 0.8 m AHD

2x0.2mx0.4m rectangular orifice
@ RL 1.4 m AHD</td><td>2 x 600x900 Pits, Crests @ RL 1.60 m AHD with 1x450mm & 1 x 525mm RCP Pipe outlets (US IL @ RL 0.8 m AHD)

1x450mm x 600mm high RCBC Headwall outlet (US IL @ RL 0.8 m AHD)</td><td>1 x 18 m wide weir, crest @ RL 1.95 m AHD</td></tr>
<tr><td>Basin C</td><td>1x0.2mx0.5m rectangular orifice
@ RL 0.8 m AHD</td><td>1 x 900x900 Pits, Crests @ RL 1.55 m AHD with 1x525 mm RCP outlet (US IL @ RL 0.8 m AHD)</td><td>4 x 900x900 Pits & 1 x 900x600, crest @ RL 1.96 m AHD

3 x 600mm RCP Outlets (USIL @ 0.8 m AHD)</td></tr>
<tr><td>C1 – Miles Street Swale (East)</td><td>-</td><td>-</td><td>1x450 (USIL @ 1.38, 0.6%)</td></tr>
</table>
</td>
</tr>
<tr>
<td>4</td>
<td>Table 6.6</td>
<td>As indicated in Table 6.2, Basin C does not have a weir. It is thus not clear what the last row of this table refers to.</td>
<td>The last column titled “High Flow Weir” can be amended to “High Flow Structures”.</td>
</tr>
<tr>
<td>5</td>
<td>Section 6.2.4 Sensitivity Analysis</td>
<td>It is recommended to specify the rainfall depth of the PMP used and add a figure or a table with the hyetograph of the PMP. The depth of this rainfall event is currently not reported.

The results shown with the PMP only include a portion of the development, focussing on the Detention Basin B (i.e., Figures 6.4- 6.6). It is recommended to show results for the whole development in addition to those already included in the report.</td>
<td>An additional table can be added within the report to present the PMP rainfall depths used for the PMF assessment of the detention basins.

The PMF assessment was prepared in response to the following information request item from Council:

The Northern Rivers Handbook of Stormwater Drainage Design requires the assessment of the Probable Maximum Flood for detention basins (Section 9, Point 8) – it is unclear if this has been addressed in the latest report.

This information request was specific the detention basins to demonstrate that the proposed high flow weirs are capable of safely conveying the PMF event as a sensitivity test.

In accordance with the Northern Rivers handbook (2019) Section 9-8, the PMF assessment is for flows through the major structures (i.e. the detention basins).

acceptable downstream flowpath i.e. an ‘escape route’ for major system flood flows, must be demonstrated.

8. The Probable Maximum Flood (PMF) is defined as the peak flood derived from routing the Probable Maximum Precipitation (PMP) through the stormwater system. Safe passage of the PMF must be demonstrated on major systems where there is risk to property and / or life (S18.1 ref 4,5,6). Investigation is required for (but not limited to)
• Detention basins / dams and spillways
• Bridges / major culverts
• Public infrastructure such as hospitals

It is not considered necessary to include the PMF results of the entire development as this assessment is specific to detention basin and spillways (Northern Rivers handbook (2019) Section 9-8).</td>
</tr>
<tr>
<td>6</td>
<td>Section 6.3.4 Manning’s coefficient</td>
<td>The TUFLOW model provided uses a value of 0.03 m-1/3 s (grass) across the whole model domain except for areas in the development where the Manning’s coefficient is 0.015 m-1/3 s (roads). The reply to a previous comment on the Manning’s coefficient (item 9 in the previous review) mentions a sensitivity analysis where the coefficient for roads was increased. However, in the current version of the model, the Manning’s coefficient for grass seems to have a value that is different from the value in the reply to item 9 from the previous review and Section 6.3.4.It is recommended to check the TUFLOW model to ensure that the values of the Manning’s coefficient coincide with those in section 6.3.4. Providing a map of the spatial distribution of the materials used in the TUFLOW model would also be helpful.</td>
<td>The Mannings listed in Section 6.3.4 can be altered to 0.03.
This will have not impact on the modelling results as the model relies on a Mannings of 0.03 for grass areas.</td>
</tr>
<tr>
<td>7</td>
<td>Section 6.3.5 Downstream Boundary Conditions</td>
<td>The regional tailwater boundary conditions are not implemented correctly in the TUFLOW model. It is said that “the 39.3% regional tailwater case initial water level has been set equal to the proposed outlet of RL 0.8 m AHD”; it thus seems that the tailwater was only implemented as the initial conditions and not as a tailwater boundary conditions. Tailwater levels should be introduced using a HT (Height versus Time) boundary condition. Given the short duration of local flooding and long duration of regional flooding, a constant level can be assigned. The TUFLOW model uses HQ boundary condition (Flow versus Time) which does not consider water level downstream or any backwater effect. It is therefore recommended to change tailwater boundary condition type to HT type and rerun the TUFLOW model.

Additionally, it is recommended to impose downstream boundary conditions based on the coincident analysis of local and regional flooding, as advised in the previous review by WMAwater (September 2022). For example, The WYURA Flood Impact and Risk Assessment, (BMT, 2023a) provides the 10% AEP post development condition (image below) which can be used to set downstream boundary (as fixed HT level) for the 1% AEP local event. The Lower Clarence Flood Model Update 2022 (BMT, 2023b) modelled 20% AEP event for the existing condition. Both 10% AEP and 20% AEP regional events can be used to provide a sensitivity assessment on the reginal flooding levels.</td>
<td>As the basin outlets level is above the regional tailwater level the application of either a HQ or HT tailwater condition will have negligible effect on results.

Consideration has given to coincident flooding (local vs regional events), based on a catchment ratio of 10,000 to 1 and with reference to QUDM table BN 8.3.4.1 – Suggested ARIs for coincidental occurrence (Figure below). The following event combinations have been considered:
• local 1% AEP on a regional 39.3% AEP; and
• regional 1% on a 39.3% local.

Tailwaters of 10%AEP and 20%AEP would be representative of a catchment ratio of around 1:100 and 1:1000 respectively which is not representative of the local catchment to the regional catchment in this case.</td>
</tr>
</table>

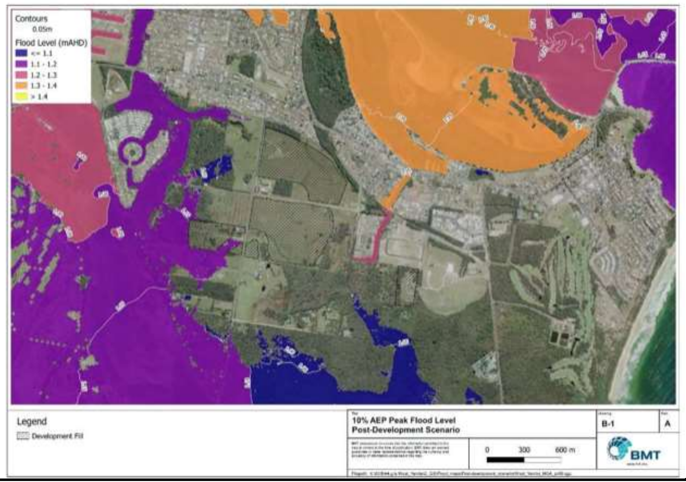

Table BN 8.3.4.1 – Suggested ARIs for coincidental occurrence

Area ratio	10% AEP (10 yr ARI) design		1% AEP (100 yr ARI) design	
	Main stream	Tributary	Main stream	Tributary
10,000 to 1	1	10	2	100
	10	1	100	2
1,000 to 1	2	10	10	100
	10	2	100	10
100 to 1	5	10	25	100
	10	5	100	25
10 to 1	10	10	50	100
	10	10	100	50
1 to 1	10	10	100	100
	10	10	100	100

Sourced from: U.S. Department of Transport, Urban Drainage Design Manual (2009)

8

Section 6.3.6 Hydrology

This section states: "The TUFLOW model incorporates a rain-on-grid methodology for the catchment with rainfall and infiltration parameters as described in the above sections".

However, it is not clear where the infiltration parameters used in the TUFLOW model are described in previous sections. In the model provided, IL and CL values are called in the material.csv file from the file rainfall_losses.trd, and values of IL and CL change with AEP. It appears that the storm loss values were not applied properly in the model. This could result in a more conservative estimate as more rainfall is applied. In the case WMAwater run (1% AEP, 45-minutes, and TP10), it is expected the total rainfall of the design event to be 78.75 mm (i.e., 105 mm/h from the design rainfall multiplied to 0.75 hours = 45 minutes). With IL = 3.4 mm and CL = 1.5 mm/h, the total rainfall input in the model should be lower than 78 mm; however, the total rainfall input implemented in the TUFLOW model results to be 78.72 mm. This might be due to the way the rainfall input is implemented, as the TUFLOW manual says that "rainfall losses in the materials files are not applied to global rainfall boundaries" (i.e., Global Rainfall BC).It is recommended to check that the rainfall inputs in the TUFLOW model are consistent with the reported IFD curves and the intended rainfall losses.

The reference to the TUFLOW manual is noted, however subsequent TUFLOW release notes have clarified the implementation of losses for global rainfall, see:

6.4 Changes for Global Rainfall Losses and Material Rainfall Losses

Build 2020-10-AA fixes a discrepancy between how rainfall losses are applied to global rainfall boundaries between Classic and HPC.

As per the TUFLOW manual Global Rainfall losses (.tbc commands Global Rainfall Initial Loss == and Global Rainfall Continuing Loss ==) are only applied to Global Rainfall Boundaries for both Classic and HPC. These commands must occur before a Global Rainfall BC is applied.

Prior to the 2020-10-AA Build, TUFLOW Classic did not apply rainfall losses in the materials file to any global rainfall boundaries, this is the behaviour documented in the TUFLOW manual. However, TUFLOW HPC did apply these losses. A summary of the loss application for both Classic and HPC is outlined in the table below for Builds prior to 2020-10-AA:

	Solution Scheme	
	Classic	HPC
Global Rainfall Losses	Applied	Applied
Material Rainfall Losses	Not Applied	Applied
Both Global and Materials Losses	Only Global Applied	Both losses applied

For the 2020-10-AA Build, if rainfall losses have been specified in the materials file these are applied to global rainfall and any Global Rainfall losses are ignored. A WARNING 2244 is output if Global losses are also specified. This is summarised in the table below for builds 2020-10-AA onwards

	Solution Scheme	
	Classic	HPC
Global Rainfall Losses	Applied	Applied
Material Rainfall Losses	Applied	Applied
Both Global and Materials Losses	WARNING 2244, Material Losses Applied	WARNING 2244, Material Losses Applied

Therefore losses are taken off the 78.72 mm internally by TUFLOW prior to application of excess to the grid, which appropriately accounts for the losses.

No change to the modelling inputs are required.

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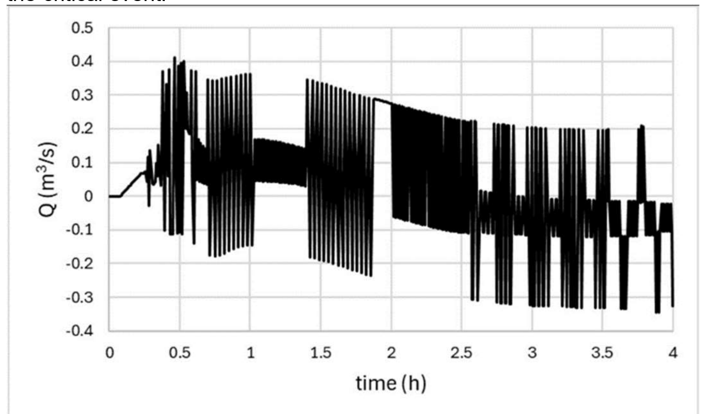
Section 6.3.7 Critical Storm Duration

It is recommended to specify the selected temporal pattern in addition to the critical duration.

It is also stated that the culverts along Carrs Drive were used to determine the critical design storm. The scenario with 1% AEP, 45-minute duration, and TP10 in pre-developed conditions led to a very noisy flow in one of the culverts along Carrs Drive (culv3; see figure below).It is recommended to check the results of the model with the parameters used in the report to ensure a sound selection of the critical event.



The interrogation line used for selection of critical storms is located upstream of Carrs Road culverts, and not Culv3 as suggested. Culv3 is not the discharge point for the site and is located to the north-west of the site. This is a minor culvert with little effect on flooding behaviour at the site.

No change to the modelling approach is considered necessary.

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6.3.9 Post-Development TUFLOW Results

Some of the maps shown in this section (Figures 6.14-6.16) are slightly different from the ones that were obtained running the provided TUFLOW model. This might be due to the different values of the Manning's coefficient and the temporal pattern used in the simulations and the model provided.

It is recommended to clarify the differences between these models to facilitate our understanding of these differences.

It is hard to comment on non-specific "differences" without any examples of what the difference are. As suggested it is likely that these differences are due to the use of a different temporal pattern. The TUFLOW model relies upon the following temporary patterns:

<table>
<tr><td></td><td></td><td></td><td>
<table>
<tr><th></th><th>dur</th><th>tp</th></tr>
<tr><td>0.5EY</td><td>15</td><td>9</td></tr>
<tr><td>0.2EY</td><td>120</td><td>6</td></tr>
<tr><td>10% AEP</td><td>120</td><td>5</td></tr>
<tr><td>5% AEP</td><td>120</td><td>5</td></tr>
<tr><td>2% AEP</td><td>120</td><td>5</td></tr>
<tr><td>1% AEP</td><td>45</td><td>10</td></tr>
</table>
</td></tr>
<tr><td>11</td><td>Table 7.5</td><td>It is noted that the sum of the land use area for C1 is 0.51 ha, which is different from Table 5.2, where the area of C1 is 0.41 ha, consistent with drawing DWG-201 in Appendix A.

It is recommended to check the values used in different models to ensure that they are consistent with each other and the plans.</td><td>The areas within Table 5.2 are incorrect and were not updated from previous versions.
We apologise for this error.

The catchment areas relied upon in the modelling are correct.</td></tr>
<tr><td>12</td><td>Section 7.5.1 Treatment Measures</td><td>In relation to the site constraints for biofilters, it is said that the HAT level at the Yamba Gauge No 1a is 1 mAHD. Please, confirm that this is correct. From Figure 6.8 of the report, taken from the Northern Rivers Local Gov Handbook, it looks like the HAT might be higher than 1m. MHL tidal planes for Station 204454 also recommends HAT= 1.11 mAHD.</td><td>HAT within Section 7.5.1 has been calculated as Chart Datum of 1.91 – 0.91 = RL 1.0 m AHD.

0.91 is a conversion factor applied to Chart Datum to produce AHD levels. See note below Figure 1A - Appendix D of Northern Rivers Handbook.

FIGURE 1A – CLARENCE RIVER TIDAL GRADIENT

NOTES
1 Level Datum – Iluka Port Datum (Zero Iluka Gauge = 12.60 BM N°.2).
Subtract 0.91m for AHD values
2 Source – NSW PWD Clarence River Waterway Planning Study 1976
(Soros Longworth & McKenzie) Figure 11a
ISBN 7240 2675 4
PWD 78006</td></tr>
<tr><td>13</td><td>Table 7.11</td><td>The reference to Table 7.7 within this table should refer to Table 7.12.</td><td>The reference in the table is incorrect and can be updated.

Table 7.11 Site Constraint Bioretention Design Recommendations
<table>
<tr><th>Constraint</th><th>Recommendations</th></tr>
<tr><td>Presence of PASS</td><td>Bioretention to be lined (liner specifications to be provided by Geotechnical Engineer) to prevent interaction with PASS.</td></tr>
<tr><td>Shallow Groundwater</td><td>7.12
Minimum design levels and freeboard contained within Table 7.7 (and Table 7 of Water By Design 'Bioretention Technical Guidelines – Version 1.1) are to be maintained.

If required Bioretention to be lined to prevent interaction with Groundwater (liner specifications to be provided by Geotechnical Engineer)</td></tr>
<tr><td>HAT</td><td>Minimum design levels and freeboard contained within Table 7.7(and Table 7 of Water By Design 'Bioretention Technical Guidelines – Version 1.1) are to be maintained.</td></tr>
<tr><td>Invert Constrained</td><td>Consider Type 1 drainage profile configuration for Bioretention measures incorporating a saturated zone. (Refer to Section 3.5.1.3 of Water By Design 'Bioretention Technical Guidelines – Version 1.1)</td></tr>
</table>
</td></tr>
<tr><td>14</td><td>Figure 7.3</td><td>It is not clear what weir flow refers to, as the Detention Basin C does not have a weir.</td><td>Weir flow refers to flow over the crest of the outlet pit.</td></tr>
<tr><td>15</td><td>Appendix H DCP</td><td>For objective 01 (Ensure stormwater management associated with the WYURA…), check that the TUFLOW model correctly accounts for the regional FIRA (BMT, 2023a).- Table H1: the requirement of item “Water efficient landscaping to be implemented” seems different from the DCP.</td><td>It is unclear what issue is being raised within this comment.</td></tr>
<tr><td>16</td><td>General</td><td>One of the events listed in the .tef file of TUFLOW model was 01.0pCC. Climate change is not mentioned in the report, and it is recommended to be included in the sensitivity analysis.</td><td>Additional modelling scenarios can be added undertaken if considered necessary.</td></tr>
<tr><td>17</td><td>General</td><td>It is recommended to provide flood maps with the TUFLOW model results in an Appendix to show the whole model domain. Results in pre- and post-development for at least the 39.3% and 1% AEPs and PMF should be provided for water depths, velocities, and hazard.</td><td>These results can be added as suggested if considered necessary.</td></tr>
</table>