Shell Cove
Boat Harbour Precinct

Section 75W Application Flood Assessment

June 2017

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Project: SHELL HARBOUR BOAT HARBOUR PRECINCT SECTION 75W APPLICATION FLOOD ASSESSMENT

Rev	Description	Author	Review	Advisian Approval	Date
A	Issued for Internal Review	E Han	W Honour		14/4/2017
B	Issued for Client Review	E Han	W Honour		20/4/2017
C	Final Report	E Han	CRT C Thomas	C Thomas	29/6/2017

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Appendices

1 INTRODUCTION

Frasers Property Australia (FPA) intends to submit a modification to the Shell Cove Boat Harbour Concept Plan Approval (MP 07_0027) under Section 75W of the Environmental Planning and Assessment Act 1979.

The proposed modifications relate primarily to an increase in the density of dwellings across the Boat Harbour Precinct, including changes to the proposed hotel and residential apartment buildings at Precinct D. The modifications also capture various refinements to the road pattern and layout across the Boat Harbour Precinct.

WorleyParsons (now Advisian) completed a Flood Management Assessment in 2009 as part of the Part 3A Concept Plan Application and Environmental Assessment. This incorporated a *'Post Development Flood Analysis Report'*, completed by Cardno Lawson Treloar.

Since then, Advisian has been engaged by FPA to develop a TUFLOW flood model for the purposes of further concept design and development assessment for individual sub-precincts within the greater Boat Harbour Precinct.

The development of this model has involved consultation with Council to confirm its suitability for use in completing a catchment-wide Flood Impact Assessment Report for the Boat Harbour Precinct, which is still in preparation and has been evolving during the DA process for each sub-precinct.

FPA commissioned Advisian to prepare this report which serves to identify and assess the modifications to the Boat Harbour Precinct made since approval of the Part 3A Concept Plan in 2010, with reference to their potential impact on flooding and flood emergency response management.

2 SECTION 75W CHANGES

The Section 75W modifications can generally be described as refinements to the road pattern and layout, and increases in the density of the proposed housing. The following sections detail the changes that are proposed for the Cook Cove development relative to the Part 3A Concept Plan that was approved in 2010.

2.1 Changes to Development Layout

Changes to the development layout since the Part 3A Concept Approval have resulted in modification to the proposed development terrain levels and drainage network (*refer* **Figure 2.1**). These changes include:

- Box culverts beneath Cove Boulevarde between Wetland 3 and Wetland 5 (*constructed*), in lieu of the originally proposed bridge.
- Box culverts beneath Harbour Boulevarde between Wetland 5 and Wetland 6, in lieu of a proposed bridge.
- Changes to the Western and Eastern Overland Flow Channels (OFCs) located at the northern side of the harbour, involving:
  - removal of the road crossing at the upstream end of the Eastern OFC;
  - box culverts at road crossings, in lieu of the previously proposed bridge crossings;
  - incorporation of retaining walls on one side of each of the channels; and,
  - a culverted section that spans the downstream half of the Eastern OFC, in lieu of the originally proposed open-channel.
- Incorporation of a large subsurface culvert along the north side of Precinct E to drain the low point in Harbour Boulevarde adjacent to the Warrigal Estate, in lieu of an overland flow path through the local roads within Precinct E.
- Changes to the layout of Wetland 7 to include a high-flow bypass channel;
- Modification to the Southern Catchment flood conveyance system, to increase the conveyance of flow through sub-surface pipes along Road MC03 and thereby reduce the potential for high hazard flows along the median or roadway.
- Overall minor regrading of various roads and terrain within each sub-precinct.

These modifications to the development have been captured in the latest TUFLOW modelling completed by Advisian, as a part of ongoing work to prepare the development-wide Flood Impact Assessment. In that regard, it should be noted that the masterplan for the overall precinct will continue to evolve as the detailed design for each sub-precinct is progressed. Hence, there will be additions to the TUFLOW model to incorporate these changes as they evolve.

Note that refinements and modifications to water quality treatment measures are covered in a separate report for the Section 75W Application.

2.2 Changes to Development Densities

The following changes to the development density are proposed:

- Increase the maximum number of dwellings from 1,238 to 1,458 and delete the maximum GFA cap of 150,000m^2
- Relocate the proposed Shell Cove Town Centre hotel, increase its maximum height from 9 storeys to 11 storeys, and incorporate flexibility to accommodate serviced and residential apartments in the hotel building
- Remove community and hotel uses from the maximum 22,000m^2 GFA cap for retail/commercial/hotel/community development
- Increase the maximum height of the residential flat buildings from 4 storeys to 6 storeys in parts of the Town Centre and key foreshore locations
- Revise the housing density and typologies across Shell Cove

The above changes are not likely to affect flood behaviour across the Shell Cove Boat Harbour Precinct as the overall footprints of the buildings within each lot have not changed significantly and therefore, do not impact on surface roughness or cause any additional impediment to flow.

As such, changes to lot densities (*specifically the increase in building heights*) are not captured in the flood modelling, other than in the case where they form part of the refined road pattern and layout discussed above.

An increased density of housing may translate to increased resident population. However, this is not expected to impact on flood emergency response, because the development will include flood-free evacuation routes during events up to and including the 100 year ARI event, and a shelter-in-place approach that can be employed during rarer events. Refer to **Section 7** for further information.

3 UPDATED FLOOD MODELLING

Hydrologic and hydrodynamic computer models were developed and used to simulate the behaviour of flooding within the Shell Cove Boat Harbour catchment. The XP-RAFTS hydrologic modelling software package was used to model the hydrologic processes of the catchment draining to the harbour. The hydrologic model was used to provide inputs for a 2-dimensional TUFLOW hydrodynamic model, which was then used to determine key flooding characteristics such as flood levels, flow velocities, floodwater depths and flood hazard throughout the study area.

The TUFLOW model also incorporates a direct rainfall (rainfall-on-the-grid) approach to capture the hydrologic processes within the TUFLOW model domain. The TUFLOW model has been developed in consultation with Shellharbour City Council to ensure the level of modelling rigour meets the expectations of council and is fit for use in the assessment of potential flood impacts.

3.1 Hydrologic Modelling

The XP-RAFTS hydrologic model of the Shell Cove Boat Harbour catchment was developed using information such as a Digital Terrain Model (DTM), land-use zoning maps, and observed housing densities. This information was used to identify the extent of the overall catchment, the delineation of sub-catchments and the connectivity of watercourses within the catchment.

The hydrologic model consists of 26 sub-catchment nodes located upstream of the TUFLOW model extent (*refer* **Figure 3.1**). Each sub-catchment was assigned initial and continuing rainfall losses to simulate rainfall that would be lost from the system; for example, when rainfall is absorbed by pervious surfaces (*refer* **Table 1**).

In accordance with the methodology outlined in the *'Shell Cove Boat Harbour Catchment Flood Study'* (Cardno, 2005), the adopted loss rates for urban areas in **Table 1** (*i.e., most subcatchments*) are the result of applying the following standard losses weighted according to the impervious fraction of the subcatchment:

- Impervious areas: Initial Loss = 1.5 mm, Continuing Loss = 0 mm/hr
- Pervious areas: Initial Loss = 10 mm, Continuing Loss = 2.5 mm/hr

With a typical impervious fraction of urban subcatchments averaging at 60%, the result is an initial loss of 5 mm and continuing loss rate of 1 mm/hr. However, a conservative approach assuming a continuing loss rate of 0.5 mm/hr has been adopted. Guidance from *Australian Rainfall and Runoff (1987 and 1998)* has also been used to estimate loss rates. These loss parameters have been applied across all upstream urban subcatchments.

The major detention basin situated between Hayman Crescent and Norfolk Crescent has been incorporated into the XP-RAFTS model as a detention node. Stage-storage and stage-discharge relationships were developed according to the geometry of the basin and spillway contained in detailed design drawings by BMD Consulting (*Shell Cove Major Detention Basin No.1 – Dam Wall, Spillway and Wetland, dated June 2003*). These relationships are shown graphically in **Appendix A**.

Table 1 XP-RAFTS Hydrologic Model Parameters

Subcatchment Identifier (*refer* Figure 1)	Area (ha)	% Impervious	Catchment Roughness (n)	Initial Loss (mm)	Continuing Loss Rate (mm/hr)
N1	5.21	70	0.025	5	0.5
N2	4.03	70	0.025	5	0.5
N7	2.39	70	0.025	5	0.5
N11	7.88	70	0.025	5	0.5
S1_FIRST	3.15	5	0.025	10	2.5
S1_SECOND	1.22	90	0.025	1.5	0
S1_A	0.73	90	0.025	1.5	0
S2 (Pervious Sub-catchment)	9.33	50	0.025	10	2.5
S2 (Impervious Sub-catchment)	3.54	100	0.025	1.5	0
S2_A	1.26	0	0.025	15	2.5
W0	8.80	50	0.025	5	0.5
W1	9.98	70	0.025	5	0.5
W2	11.66	50	0.025	5	0.5
W3	9.16	70	0.025	5	0.5
W4	7.06	70	0.025	5	0.5
W5	10.40	50	0.025	5	0.5
W6A	8.12	70	0.025	5	0.5
W7A	4.32	70	0.025	5	0.5
W7B	2.66	70	0.025	5	0.5
W8	14.50	70	0.025	5	0.5
W10	15.26	70	0.025	5	0.5
W11	3.57	70	0.025	5	0.5
W11A	3.27	70	0.025	5	0.5
W12	4.73	70	0.025	5	0.5
W13	3.05	50	0.025	5	0.5
W14	8.46	70	0.025	5	0.5
W17	9.27	70	0.025	5	0.5
W19	6.19	70	0.025	5	0.5

For areas downstream of the XP-RAFTS catchments, the TUFLOW hydrodynamic model incorporates hydrologic modelling over a majority of the TUFLOW model domain by way of Direct Rainfall (*or rainfall-on-the-grid*). Further information on the associated hydrologic parameters is contained in the following.

3.2 Hydrodynamic Modelling

A 2-dimensional TUFLOW direct rainfall hydrodynamic model was developed according to a 2-metre grid size to appropriately capture the proposed topography across the Shell Cove Boat Harbour Precinct.

The pre-development DTM used in the hydrodynamic model is shown in **Figure 3.2**, and had been developed according to a survey that was undertaken prior to commencement of the BHP developments.

The post-development DTM is shown in **Figure 3.3**, and has been developed according to a combination of available LiDAR data for previously developed areas and design drawings or Work-As-Executed survey information for more recently completed development precincts immediately west of Harbour Boulevarde.

Changes to the DTM within each sub-precinct and at Wetland 7 have been made since the Part 3A approval. However, the hydrodynamic modelling results show these changes are relatively minor and have not significantly affected the expected flood impacts, as demonstrated in Section 5.1.

Material types, which define the surface roughness and rainfall losses of the land in the hydrodynamic model, were assigned in accordance with roughness values used in previous flood impact assessments for the precinct (*refer* **Figures 3.4** *and* **3.5**). For comparison, the equivalent roughness map for the SOBEK model used in the 2009 Part 3A assessment is included in **Appendix B**. The roughness values have been assigned according to the TUFLOW material types listed in **Table 2**.

It should also be noted that the latest post-development roughness distribution shown in **Figure 3.5** incorporates a generally greater level of detail relative to the 2009 assessment in terms of the following:

- The latest configuration of proposed housing densities;
- Representation of Precinct D town centre buildings as "blocked-out" areas in the TUFLOW model domain, to preclude flow across these areas and thereby simulate the full impediment to flows;
- Simulation of waterbodies within designated wetland areas with higher roughness and zero rainfall losses to account for vegetative cover and day-to-day ponding, respectively.

The initial and continuing rainfall losses applied to the direct rainfall modelling in TUFLOW are outlined in **Table 2**, and have been applied according to the same material type distribution for floodplain roughness (*refer* **Figure 3.4** *and* **3.5**).

A schematic layout of the hydraulic structures in the TUFLOW model is shown in **Figure 3.6** including the bridges, culverts and local drainage lines included in the model. The details for each component have been sourced from relevant design drawings. Since the Part 3A approval, updated concept designs for several road crossings and drainage concept designs have been incorporated into the post-development scenario.

The original Western and Eastern OFC concepts in the Part 3A approval included bridge structures at two road crossings for each channel. The upstream bridge structure for the Eastern OFC has been removed (i.e. the structure is no longer proposed), and the other crossings have since been replaced with triple-cell box culverts. The Eastern OFC is now proposed to incorporate triple-cell box culverts for the lower half of the channel; i.e. extending to the harbour edge (*refer* **Figure 3.6**).

Table 2 TUFLOW Model Material Types and Adopted Modelling Parameters

TUFLOW Material Type Identifier ^	Land Use Description	Roughness (Manning's 'n')	Initial Loss (mm)	Continuing Loss Rate (mm/hr)
1	Roads*	0.015	1.0	0
2	Beach and bay areas / harbour, ocean*	0.02	0	0
3	Well cut grass*	0.035	10.0	2.5
4	Waterway areas, Wetlands	0.06	0	0
5	Vegetated Areas, Dune Areas*	0.06	10.0	2.5
6	Residential Areas (low density)*	0.10	5.0	0.5
7	*Not used*	-	-	-
8	Residential Areas (medium density)	0.15	3.0	0
9	Residential Areas (high density)	0.18	1.0	0
10	Commercial Properties	0.20	1.0	0

** Adopted from Cardno Lawson Treloar (2005), where applicable*
^ Refer Figure 3.5

The originally proposed bridge crossing at Harbour Boulevarde, between Wetland 5 and Wetland 6, has since been replaced with a triple-cell box culvert, as per the design prepared by Cardno (2015).

The concept for the proposed bridge downstream of Wetland 6 incorporates two clear spans of 9 metres (*with one pier*) totalling a flow width of 18 metres.

The proposed swale within the median of Road MC03 positioned between Precincts A and B shown in the Part 3A Report has been removed. Upon consultation with Council, the agreed concept for the drainage network at Road MC03 includes two stormwater collection basins between Harbour Boulevarde and the existing waste cell immediately to the south. Flows during events up to and including the 100 year ARI event are then conveyed from these basins to the harbour via twin 1800 mm pipe culverts beneath Road MC03 with minimal inundation of the roadway or median.

The hydrodynamic model was run initially with no blockage factor applied to major culverts and bridge structures. Sensitivity testing which involved the application of blockage to these structures is described in Section 8.3.

Hydrographs were extracted from the XP-RAFTS hydrologic model and applied at the inflow polygons at the upstream limits of the TUFLOW model domain (*refer* **Figure 3.1**). A direct-rainfall approach was applied across the majority of the TUFLOW model grid for downstream areas, as shown in **Figure 3.1**.

The hydrodynamic model was used to simulate the following design events:

- 20% Annual Exceedance Probability (AEP) flood (*or 5 year ARI flood*);
- 1% Annual Exceedance Probability (AEP flood (*or 100 year ARI flood*); and,
- Probable Maximum Flood (*PMF*).

The tailwater conditions used in the hydrodynamic model were derived from Cardno's '*Shell Cove Boat Harbour Post Development Flood Analysis*' (*July, 2009*) and *Elliot Lake – Little Lake Flood Study* (*January, 2006*). The method of deriving the tailwater level involved taking into account a 1% exceedance tide level, the effects of sea level rise (*0.55 metres by the year 2050*), wave-setup and also the reduction in wave-setup height caused by the presence of the harbour in the post-development scenario.

The pre-development and post-development tailwater conditions used in the hydrodynamic model are listed in **Table 3**. In order to address the Statement of Commitments associated with the original concept approval, a sea level rise of 0.9 metres for year 2100 was incorporated into the harbour tailwater level for the TUFLOW simulation used to determine Flood Planning Levels. This effectively increases the adopted 100 year ARI harbour level by 0.35 metres.

Table 3 Adopted Downstream Tailwater Conditions in the TUFLOW Models

Scenario	Pre-Development Tailwater Level (mAHD)	Post-Development Tailwater Level (mAHD)
5 Year ARI	1.55	1.55
100 Year ARI	2.35	1.95
100 Year ARI with 0.9m Sea Level Rise (for determining Flood Planning Levels)	N/A	2.30
PMF	2.45	2.05

The hydrodynamic modelling results for the pre-development scenario are discussed in Section 4, and the results for the post-development scenario are discussed in Section 5. Due to the direct-rainfall approach used in the hydrodynamic modelling, it was agreed with Council that a depth filtering criteria be applied to the modelling results, such that depths less than 150 mm are omitted from the flood mapping.

4 EXISTING FLOOD CHARACTERISTICS

4.1 Pre-Development Scenario Modelling Results

Peak flood level and depth mapping was prepared for the pre-development scenario in order to assess existing flooding behaviour and is shown in **Figures 4.1** to **4.6**. Flood hazard mapping, prepared in accordance with the *NSW Floodplain Development Manual (2005)*, is shown in **Figures 4.7** to **4.9**.

In the pre-development scenario, several properties adjacent to the Northern Swale between Sophia Street and Mary Street are expected to be affected during the 5 year ARI and greater events. Peak flood levels expected at these properties are tabulated below in Section 4.2 and shown in **Figures 4.1** to **4.3**. There is low flood hazard associated with the inundation of these properties in events up to and including the 100 year ARI storm (*refer* **Figure 4.8**). However, high flood hazard is expected at some of the properties during the PMF in pre-development conditions.

It should be noted that the ponding that is shown at the properties immediately east of Boollwarroo Parade and the flooding that is shown within Sophia Street is expected to be largely captured by existing stormwater culverts during the 5 and 100 year ARI events. However, such culverts have not been incorporated into the TUFLOW model as their operation is not expected to impact significantly on the downstream flooding. This approach is consistent between pre and post-development scenarios.

4.2 Comparison to Part 3A Concept Flood Modelling Results

A comparison has been made between the pre-development 100 year ARI flood levels from the latest TUFLOW modelling and those from the SOBEK modelling completed as part of the 2009 Flood Assessment for the Part 3A Report (*refer* **Table 4**). The locations of the comparison points listed in the table are shown in **Figure 4.11**.

Table 4 Comparison between 2009 SOBEK and 2017 TUFLOW Modelling Results for the Pre-Development Scenario

Location (*refer* Figure 4.11)	100 Year ARI Flood Level (mAHD)		Difference (m)
	2009 SOBEK	2017 TUFLOW	
SWAMP	3.06	3.04	- 0.02
NS1	3.07	3.05	- 0.02
NS2	3.07	3.05	- 0.02
NS3	3.06	3.04	- 0.02

The comparison shows that the 2017 TUFLOW flood levels are consistent with the 2009 SOBEK modelling results presented in the Part 3A Report, and the minor differences of approximately 20 mm are likely attributed to the different level of topographical detail contained in the models by way of the different model grid resolution (*2 metres in TUFLOW compared to 5 metres in SOBEK*).

Difference mapping has also been prepared to compare the flood levels from the 2017 TUFLOW modelling and the 2009 SOBEK modelling for the pre-development scenario (refer **Figures 4.10** to **4.12**). Areas shown in red indicate an increase in flood levels in the 2017 TUFLOW modelling in comparison to the 2009 SOBEK modelling results, while areas in blue show a decrease in flood levels.

As shown in **Figure 4.10**, in the 5 year ARI event the difference mapping shows that the TUFLOW model produces flood levels that are approximately 170 mm lower across the pre-existing swamp area. The higher flood levels in the SOBEK model are likely attributable to the incorporation of 1-Dimensional elements to represent the existing creek/channel downstream of the swamp (*to the south of the swamp*). In the SOBEK model the top of the channel banks, within the 1D domain and also the adjacent 2D domain, appear to be slightly higher than in the DTM used in the TUFLOW model, which is causing an increased backing-up of flow upstream from the channel in SOBEK.

In the 100 year ARI event, differences in flood levels between the 2016 TUFLOW and 2009 SOBEK modelling results are minimal, as outlined above (*refer* **Figure 4.11** *and* **Table 4**).

For the PMF, the difference mapping shows that the TUFLOW model has generally higher flood levels within the low-lying flat areas of the Shellharbour Swamp (*refer* **Figure 4.12**). This is likely attributed to the smaller 2-metre grid size used in the TUFLOW model, which better enforces the crest of the road embankment of Bass Point Tourist Drive to appropriately restrict the flow across the road. With the larger 5-metre grid size of the SOBEK model, it is possible that some diagonal connections between grids effectively reduce the crest level of flow paths across the road by up to 300 mm, which in turn acts to reduce the level of flooding on the upstream side of the road.

5 IMPACT OF THE PROPOSED DEVELOPMENT

5.1 Post-Development Scenario Modelling Results

Peak flood level and depth mapping was prepared for the post-development scenario and is shown in **Figures 5.1** to **5.6**. Flood hazard mapping prepared in accordance with the *NSW Floodplain Development Manual* (*2005*) is presented in **Figures 5.7** to **5.9**. A comparison between the post-development and the pre-development scenarios is discussed below in **Section 5.3**.

Shellharbour Village

In the post-development scenario, existing properties adjacent to the Northern Swale between Sophia Street and Mary Street are still expected to be affected during the 5 year ARI and greater events, but to a lesser depth when compared to the pre-development scenario. This inundation is associated with low flood hazard in events up to the 100 year ARI (*refer* **Figure 5.8**).

However, these properties are subject to high flood hazard during the PMF, which remains unchanged from pre-development conditions (*compare* **Figures 4.9** *and* **5.9**). Peak flood depths are expected to be reduced due to the construction of the Overland Flow Channels (*OFCs*). The high flood hazard in the PMF and reduction in flood levels are consistent with the mapping provided in the Part 3A Post-Development Flood Analysis. A detailed comparison of pre-development and post-development peak flood levels at the Shellharbour Village is discussed in Section 5.3.

Overland Flow Channels

The Western and Eastern OFCs are expected to convey flows in-bank to the harbour during events up to and including the PMF. The road crossings are not expected to be overtopped during the 100 year ARI event.

During the PMF some overtopping at Brigantine Drive may occur. As high flood hazards are expected at other points along Brigantine Drive during the PMF, the overtopping of the overland flow channels is not expected to cause any further reduction in accessibility to and from these sub-precincts during such an extreme event.

Harbour Boulevarde

During the 100 year ARI flood, localised ponding is expected along Harbour Boulevarde to the east of the Warrigal Site, and to the east of Wetland 3 (*refer* **Figure 5.2**). Such ponding is from local rainfall and is not connected to flows along the designated watercourses. Low flood hazards are associated with these areas of localised ponding during the 100 year ARI event and are not likely to pose a risk to vehicles.

During the PMF, Harbour Boulevarde is expected to be overtopped to the east of Wetland 3 and Wetland 5. Significant flood depths are also expected to the east of the Warrigal Site and at the intersection of Road MC03 and Harbour Boulevarde (*refer* **Figure 5.6**). High flood hazards are associated with these areas during the PMF (*refer* **Figure 5.9**). It is expected that the duration of inundation of these roadways (*to a depth greater than 300 mm*) will be no longer than 1 to 1.5 hours.

A detailed comparison of pre-development and post-development peak flood levels at Harbour Boulevarde and adjacent Wetlands is discussed in Section 5.3.

Cove Boulevarde

During the 100 year ARI event the box culverts connecting Wetland 3 and Wetland 5, beneath Cove Boulevarde, are expected to convey flows without overtopping the roadway.

During the PMF flows are expected to overtop Cove Boulevarde to a depth of approximately 920 mm, and Harbour Boulevarde to a depth of 1.4 metres (*refer* **Figure 5.6**). There is transitional to high hazard associated with the flow over both Cove Boulevarde and Harbour Boulevarde during the PMF, which is similar to the mapping shown in the Part 3A *Post-Development Flood Analysis*. The duration of inundation (*to a depth greater than 300 mm*) at Cove Boulevarde during the PMF will be less than 1.5 hours.

Local Roads within Sub-Precincts

Flooding shown during the 100 year ARI event on local roads and carparks within sub-precincts, such as that within Precinct D and Precinct B, is expected to be largely captured by subsurface drainage networks. Not all pits and pipes have been included in the TUFLOW model. It is understood that the these local pipe drainage networks have sufficient capacity to ensure the depth of flow in the road gutters does not exceed 200 mm during the 100 year ARI storm, as per the requirements of Council's *'Subdivision Drainage Design Code'* (2004).

In the PMF certain roadways are expected to convey a significant volume of flow to the harbour, which acts to minimise flows and hazard across the proposed lots. This includes Road 16, Brigantine Drive, Road 11, Road 10, Road MC01, Road MC03, and a local road within Precinct A, which are all shown to have high flood hazard (*refer* **Figure 5.9**). This is consistent with the mapping shown in the Part 3A Report.

Southern Catchment and Road MC03

Minimal flooding occurs along Road MC03 during the 100 year ARI event, with a maximum depth of 200 mm at localised points (*refer* **Figure 5.5**). The ponding shown is caused only by local inflows from within Precinct A and Precinct B. It should also be noted the TUFLOW modelling incorporates only an indicative representation of the local pipe drainage networks within Precinct A and Precinct B, which likely underestimates the capture of runoff into these pipes.

During the Probable Maximum Flood it is expected that flood depths will only exceed 300 mm at Harbour Boulevarde for a duration of about 1 hour. The flood hazard along Road MC03 is expected to be high during the PMF, which is consistent with the Part 3A flood mapping.

Given the absence of any high hazard flows through the proposed lots across Precincts A and B during the PMF, there will be limited to no structural impact on houses at these lots and the proposed flood emergency response strategy would be to 'shelter-in-place' during the PMF.

5.2 Comparison of Peak Flood Levels to Part 3A Concept Flood Levels

Table 5 provides a comparison between 100 year ARI flood levels along major flow paths from the latest TUFLOW modelling of the Boat Harbour Precinct and those from the SOBEK modelling that was completed as part of the 2009 Post-Development Flood Analysis for the Part 3A approval. The locations of the comparison points listed in the tables are shown in **Figure 5.2**.

Table 5 Comparison between 2009 SOBEK and 2017 TUFLOW Modelling Results for the Post-Development Scenario at Major Flow Paths

Location (*refer* Figure 5.2)	100 Year ARI Flood Level (mAHD) 2009 SOBEK	2016 TUFLOW	Difference (m)
NS1	2.97	2.96	-0.01
NS2	2.99	2.86	-0.13
NS3	2.99	2.80	-0.19
WL3A	8.24	8.14	-0.10
WL3B	9.26	9.14	-0.12
WL3C	8.20	7.94	-0.26
WL5A	7.12	6.75	-0.37
WL5B	6.33	6.06	-0.27
WL5C	6.40	6.82	+0.42
WL5D	6.44	6.03	-0.41
WL6A	5.29	3.57	-1.72
WL6B	3.97	3.40	-0.57
WL6C	3.02	3.34	+0.32

The comparison shows that in all but two locations the 2017 flood modelling generates peak flood levels that are lower than those predicted for the landform that was proposed in 2009. The two locations where peak 100 year ARI flood levels are predicted to increase are highlighted in yellow in **Table 5**. These correspond to the outlet of Wetland 6 to the boat harbour (WL6C) and immediately north of the Cove Boulevarde culverts within the Wetland 3 and 5 high flow channel (WL5C).

The increases at these locations occur for the following reasons:

- The 2017 TUFLOW modelling incorporates a bridge structure at the downstream end of Wetland 6 which is approximately 1 metre higher than the terrain that was adopted in this area in the 2009 SOBEK model. The flow constriction caused by the bridge causes the increase in the peak 100 year ARI flood level of 320 mm at location WL6C.
- The terrain adopted in the 2017 TUFLOW model is based on Work as Executed survey of the area that was current at the time of the modelling. The terrain elevation at the Wetland 3 and 5 high flow channel is approximately 400 mm higher in the WAE conditions, when compared to terrain elevations in this area that were adopted in the 2009 SOBEK model. This explains the 420 mm increase in flood levels at location WL5C.

The Harbour Boulevarde design has been adjusted since the Part 3A approval, and therefore a comparison of 100 year ARI ponding depths within Harbour Boulevarde has also been made (*refer* **Table 6**).

The locations of the comparison points listed in the tables are shown in **Figure 5.2**.

Table 6 Comparison between 2009 SOBEK and 2016 TUFLOW Modelling Results for the Post-Development Scenario at Harbour Boulevarde

Location (*refer* Figure 5.2)	100 Year ARI Flood Depth		Difference
	2009 SOBEK	2016 TUFLOW	
HB1	400 mm	400 mm	0 mm
HB2	310 mm	< 150 mm	Up to -310 mm
MC03	1310 mm	270 mm	-1040 mm

The comparison in **Table 5** shows that the SOBEK flood levels are typically similar higher than the TUFLOW results. These differences are primarily attributed to the refinements and modifications to the designs of wetlands, Overland Flow Channels, subsurface drainage networks and grading plans that have occurred since the Part 3A approval.

The reduced 100 year ARI flood levels from the TUFLOW model along the Northern Swale (*i.e., at locations NS1, NS2 and NS3*) are the result of a modified design for the Eastern OFC. For the original Part 3A modelling it was assumed that a bridge would be placed across the channel at the upstream end. Whereas the revised concept for the Eastern Channel does not incorporate a road crossing at this location, resulting in less constriction of flows travelling south to the harbour, which reduces the level of upstream flooding.

The reduction in peak flood levels at the two western pools of Wetland 6 (*i.e., at locations WL6A and WL6B*) are a result of the addition of a weir at the far western side, immediately downstream of the culverts beneath Harbour Boulevarde. This weir was included in the latest design of Wetland 6 in order to provide sufficient hydraulic head for water quality treatment devices. The downstream pools are effectively at a lower elevation thereby resulting in reduced flood levels.

Due to changes in the subsurface drainage design along Harbour Boulevarde and Road MC03, peak flood depths during the 100 year ARI have been reduced at locations *HB2* and *MC03* (*refer* **Table 6**).

5.3 Impact of the Proposed Development

The TUFLOW model results show that the proposed development will generally reduce flood levels at existing properties in Shellharbour Village, at existing properties to the east of Boollwarroo Parade and at Ron Costello Oval when compared to the pre-development scenario. Flows will be conveyed to the boat harbour more efficiently via the network of culverts, swales, overland flow channels, and wetlands. The impacts on peak flood levels for each design event are shown in **Figures 5.10** to **5.12**. Areas shown in yellow shading represent altered flow paths within the development itself due to the proposed development landforms and therefore, have been excluded from the flood level difference mapping.

Existing Properties at Shellharbour Village

A comparison between peak flood levels in the pre-development and post-development scenarios along the southern edge of Shellharbour Village is provided in **Table 7**.

Table 7 Impact on Peak Flood Levels at the Southern Edge of Shellharbour Village

Design Event	Pre-Development Peak Flood Level (mAHD)	Post-Development Peak Flood Level (mAHD)	Flood Level Difference (m)
5 Year ARI	2.87	2.77	- 0.10
100 Year ARI	3.05	2.96	- 0.09
PMF	4.33	4.15	- 0.18

As discussed above, the existing properties in this area are affected by flooding in the 5 year ARI event and larger, in both pre-development and post-development conditions. The proposed Shell Cove development will result in a reduction in peak flood levels at these properties during all events up to and including the PMF. This is attributed to the proposed overland flow channels, which will convey flows to the boat harbour more efficiently than the natural channels and swamp that previously existed.

Ron Costello Oval and Boollwarroo Parade

A reduction in flood levels at Ron Costello, the surrounding parklands and existing properties along Boollwarroo Parade is also expected in the post-development scenario. A reduction in peak flood levels of up to 200 mm is expected during the 100 year ARI event and up to 500 mm in the PMF (*refer* **Figures 5.11** *and* **5.12**).

6 HYDRAULIC CATEGORY MAPPING

Hydraulic category mapping has been prepared for the Shell Cove Boat Harbour Precinct and is provided in **Figures 6.1** to **6.3**. The hazard mapping was produced using the results of the TUFLOW modelling and according to the hydraulic criteria outlined in a report titled, *Shell Cove Boat Harbour Post Development Flood Analysis* (*2009*), which was prepared as part of the Part 3A Concept Approval.

The mapping shows that for the 5 and 100 year ARI events the main flow paths through the Northern Swale, the OFCs, Wetland 3, Wetland 5 and Wetland 6, are categorised as Floodways.

The offline areas in the wetlands are expected will function as Flood Storage during events up to and including the 100 year ARI event. Areas of Flood Fringe are defined as depths less than 200 mm and are therefore, relatively small considering the filtering applied to the mapping which trims depths that are less than 150 mm.

For the PMF there are expected to be additional Floodways along Harbour Boulevarde, the far eastern end of Cove Boulevarde, Road 16, Brigantine Drive, Road 11, Road 10, Road MC01 and Road MC03 and a local road within Precinct A, which is similar to the mapping contained in the Part 3A assessment.

Overall, the hydraulic category mapping is consistent with the mapping presented in the Part 3A assessment, while allowing for minor differences due to changes in design terrain and features.

7 FLOOD EMERGENCY RESPONSE

The flood hazard mapping presented in **Figures 5.7, 5.8 and 5.9** show that high hazard is expected to occur only within the major flow paths at the northern swale, the overland flow channels and wetland areas during the 5 year and 100 year ARI storms. Provided that sufficient pit inlet capacity (*including redundancy for blockage*) is incorporated into the drainage network design for Harbour Boulevarde, Cove Boulevarde and for local roads within the precinct, there will be no high hazard expected along local roads and main access routes leading to and from all sub-precincts within the development.

During the 100 year ARI event there will be clear vehicle access along local roads within all sub-precincts that lead to Harbour Boulevarde, which will allow for safe evacuation in events of up to this magnitude. As discussed above, small areas of ponding occur at Harbour Boulevarde, to the east of the Warrigal site and to the east of Wetland 3. However, these areas of ponding have a low flood hazard and, hence, are unlikely to pose significant risk to vehicles or their occupants. The depth of ponding at the edge of the road carriageway is generally about 300 mm or less to the east of the Warrigal Site, and approximately 500 mm or less to the east of Wetland 3. Such inundation is not linked to major flooding along the designated flow paths.

During the PMF high flood hazards are expected along Road 16, Brigantine Drive, Road 11, Road 10, Road MC01, Road MC03, and a local road within Precinct A (*refer* **Figure 5.9**). The duration of inundation across these roadways (*to a depth greater than 300 mm, where vehicles become unstable*) is expected to be no longer than 1.5 hours at areas of ponding within Harbour Boulevarde and Precinct D, and no longer than 1 hour in all other sub-precincts. Given the relatively short potential time of isolation, a shelter-in-place emergency response will be suitable for the Shell Cove Boat Harbour Precinct during flooding greater than the 100 year ARI storm.

Overall, the increase in development density and changes to layouts within the development will not cause any impediment to the originally proposed evacuation routes during flooding up to the 100 year ARI event, and a shelter-in-place response is still considered appropriate during more extreme events. This approach is consistent with the flood response strategy approved as part of the Part 3A Concept. The associated Statement of Commitments require that *"the Proponent undertakes to implement a Flood Emergency Response which includes <u>remaining on site during PMF events</u> and maintaining safe pedestrian and vehicular access routes out of the Boat Harbour Precinct for events up to the 100yr ARI flood."*

8 SENSITIVITY ANALYSES

8.1 Climate Change

The potential impact of climate change has been considered in the context of the changes in proposed development density and road layouts. Climate change scenarios involving increased rainfall intensities in combination with sea level rise are yet to be modelled using the TUFLOW model. It is intended that such sensitivity testing be completed as part of ongoing modelling to confirm the impacts of each sub-precinct as the individual Development Applications are prepared.

For the Part 3A *Post-Development Flood Analysis* (*Cardno, 2009*) two climate change scenarios were evaluated for the 100 year ARI event. The mid-range climate change scenario incorporated a sea level rise of 0.55 m in the ocean tailwater level with a 20% increase in rainfall intensity, and the high-range climate scenario incorporated a sea level rise of 0.9 m and a 30% increase in rainfall intensity. The modelling results demonstrated that there would be an increase in flood levels of approximately 0.03 to 0.13 m in a mid-range climate change scenario, and 0.04 to 0.36 m in a high-range climate change scenario. It was determined that these impacts would not manifest as significant impacts on the development or existing urban areas. Accordingly, it is unlikely that sea level rise or increases in rainfall intensities associated with climate change would have any material effect on the proposed development.

The changes in the proposed development (*refer* **Figure 2.1**) have been designed such that 500 mm freeboard allowance above the 100 year ARI event is maintained to all development areas. As a result, it is also expected that the mid- and high-range climate change scenarios are also expected to cause increases in 100 year ARI flood levels similar to that reported in the Part 3A approval. Similar increases in flood levels would be expected from TUFLOW modelling of the same climate change scenarios and therefore the findings of the Part 3A reporting would not be altered.

Advisian has previously modelled the 100 year ARI event with a sea level rise of 0.9 m incorporated into the modelled tailwater levels in the boat harbour (*i.e., a tailwater of 2.3 mAHD was adopted*), which was undertaken to determine Flood Planning Levels for Precinct D and Precinct E. The results for these simulations showed that sea level rise would have no significant impact on properties adjacent to the harbour edge or on flood levels across other parts of the development. The effects of sea level rise do not propagate into flow paths such as Wetland 6, as it is raised relative to the harbour. While the downstream portions of the Overland Flow Channels may be affected by sea level rise, these effects do not propagate up to the Northern Swale.

8.2 Hydraulic Roughness of the Floodplain

Sensitivity tests involving changes in hydraulic roughness are yet to be modelled in TUFLOW. The Part 3A Post-development Flood Analysis demonstrated that there would be an increase in flood levels of approximately 0.15 m if the hydraulic roughness was increased by 20% across the development. This is a similar magnitude to that of the mid-range climate change scenario, which was determined to not have significant impact on the development or existing urban areas. Similar increases in flood levels would be expected in the TUFLOW modelling results of a sensitivity test involving increased hydraulic roughness.

8.3 Culvert and Bridge Blockage Sensitivity Testing

A sensitivity test was undertaken to assess the behaviour of flooding in the 100 year ARI event with a blockage factor applied to culvert and bridge structures within the Shell Cove Boat Harbour Precinct.

A blockage factor of 50% was applied to all major culvert crossings of waterways; that is, culverts designed to convey major overland flows under Cove Boulevarde and Harbour Boulevarde and the large box culverts designed to convey flows in the Western and Eastern OFCs.

A blockage factor of 20% was applied to the proposed bridge structure at the downstream side of Wetland 6, as discussed with Council, due to the likelihood that most debris would pass through the relatively large width of each of the two bridge spans (*each 9 m wide*).

It is assumed that the road and urban drainage networks are suitably designed to convey flows (*i.e. no blockage factor was applied to urban drainage pipes*), and that the design of all inlet pits has incorporated suitable allowance for inlet blockage.

Flood level, depth and hazard mapping was prepared for the 100 year ARI blockage scenario, as shown in **Figures 8.1** to **8.3**. Similar to the non-blockage 100 year ARI simulation, almost all flows are contained within the wetlands and OFCs in the blockage scenario. Localised street ponding is expected at the same locations; i.e., to the east of the Warrigal Site and to the east of Wetland 3.

Comparison with Post-Development Flood Levels (Non-Blockage)

The mapping in **Figure 8.4** shows the difference in peak flood levels based on a comparison of the 100 year ARI "blockage scenario" with the 100 year ARI "non-blockage scenario".

As shown, culvert blockage is expected to result in flood level increases of approximately 120 mm immediately upstream of the Brigantine Drive crossing of the Western OFC. However, flood level increases of less than 50 mm are expected upstream of the Western OFC Road 16 crossing. The smaller impact at Road 16 is likely due to the lack of any road crossing at the upstream end of the Eastern OFC, which results in flow being directed to the Eastern OFC. A flood level increase of up to 150 mm is expected along the Northern Swale, and about 230 mm immediately upstream of the Eastern OFC culverts (*at Brigantine Drive*).

The flood level increases at the OFCs due to culvert blockage are generally greater than that presented in the Part 3A Report, and are likely due to the changes from bridge structures to culvert structures. The lesser flood level differences upstream of the Western OFC can be attributed to the removal of the upstream road crossing on the Eastern OFC as part of the latest development layout, which allows an increased volume of water being directed to the Eastern OFC.

Culvert blockage will lead to flood level increases of up to 670 mm immediately upstream of the Harbour Boulevarde culverts at Wetland 5, and up to 620 mm immediately upstream of the Cove Boulevarde culverts to the north of Wetland 3. Flood level increases in the offline areas within Wetland 3 and 5 are also expected, but are not overtopped into adjacent development areas.

A minor flood level decrease of approximately 130 mm is expected in Wetland 6, downstream of the blocked culverts beneath Harbour Boulevarde. No significant flood level differences are expected in other areas within the development.

These increases in peak flood levels do not cause a significant impact on properties proposed within the Shell Cove development as there is at least 1.5 m of freeboard provided to the channel banks of the Eastern and Western OFCs and 800 mm to the southern bank of the Northern Swale.

Comparison with Pre-Development Flood Levels

The mapping presented in **Figure 8.5** shows the differences in flood levels between the 100 year ARI blockage scenario (post-development) and the pre-development scenario. A reduction in flood levels of at least 90 mm along the Northern Swale and at existing properties at Shellharbour Village is still expected in the post-development scenario (*even with culvert blockage*) when compared to the pre-development scenario. Larger flood level decreases are expected at the eastern end of the Northern Swale.

Accordingly, the existing Shellharbour Village properties will not be affected due to potential blockage of proposed culverts and bridges.

9 CONCLUSIONS

The assessments undertaken as a part of this report have shown that the Section 75W changes to the terrain, layout and increased development density within the Shell Cove Boat Harbour Precinct will not result in any adverse flood impacts beyond that documented in the Post-Development Flood Analysis (*Cardno, 2009*) prepared as a part of the Part 3A Concept Approval.

A comparison of pre-development and post-development flood modelling results of the Shell Cove Boat Harbour Precinct has demonstrated that the development is not expected to adversely affect flooding at any proposed or existing properties during the 5 year and 100 year ARI events and the Probable Maximum Flood (*in respect of Shellharbour Village*).

Additional sensitivity analyses completed as part of the original Part 3A assessment for climate change and floodplain roughness indicated that the effects would not manifest as a significant impact on the development or existing urban areas. The proposed Section 75W modifications to the development are not expected to alter these outcomes.

A sensitivity test which assessed 100 year ARI flood behaviour in the event of culvert and bridge blockage showed that floodwaters would be predominantly contained within the existing/proposed wetlands and watercourses. An exception applies to the Northern Swale, where existing Shellharbour Village properties are affected by flooding in both the pre-development and post-development scenarios. However, the modelling results have determined that the post-development peak flood levels are reduced when compared to pre-development modelling results, even in the culvert blockage scenario.

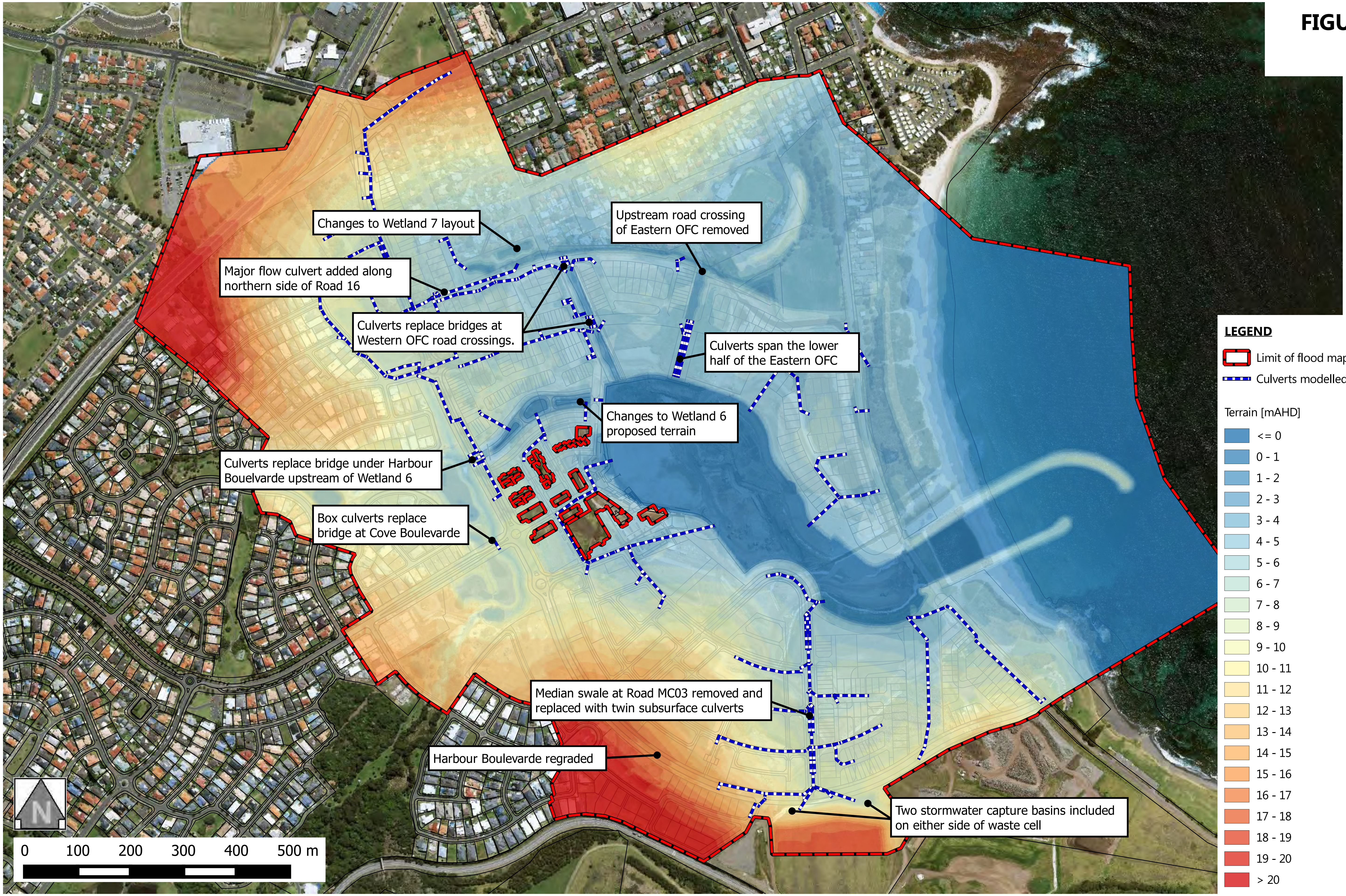
The above assessment has demonstrated that the post-development flood mapping remains consistent with a flood emergency response strategy comprising the possibility of evacuation during events up to and including the 100 year ARI event and shelter-in-place during larger events.

10 REFERENCES

- Cardno (*July 2009*), '*Shell Cove Boat Harbour Post Development Flood Analysis*'
- Cardno (*January 2006*), '*Elliot Lake – Little Lake Flood Study*'
- Cardno Lawson Treloar (*November 2005*), '*Shell Cove Boat Harbour Catchment Flood Study*'
- Cardno (*September 2015*), '*Culvert at Harbour Boulevarde*'
- NSW Government (*August 2010*), '*NSW Coastal Planning Guideline: Adapting to Sea Level Rise*'
- NSW Government (*April 2005*), '*Floodplain Development Manual: the management of flood liable land*'; ISBN 0 7347 5476 0
- Shellharbour City Council (*November 2004*), '*Development Design Specification D5 – Subdivision Drainage Design*'
- WorleyParsons (*July 2009*), '*Shell Cove Boat Harbour Precinct Flood Management Assessment*'

Report Figures

FIGURE 2.1



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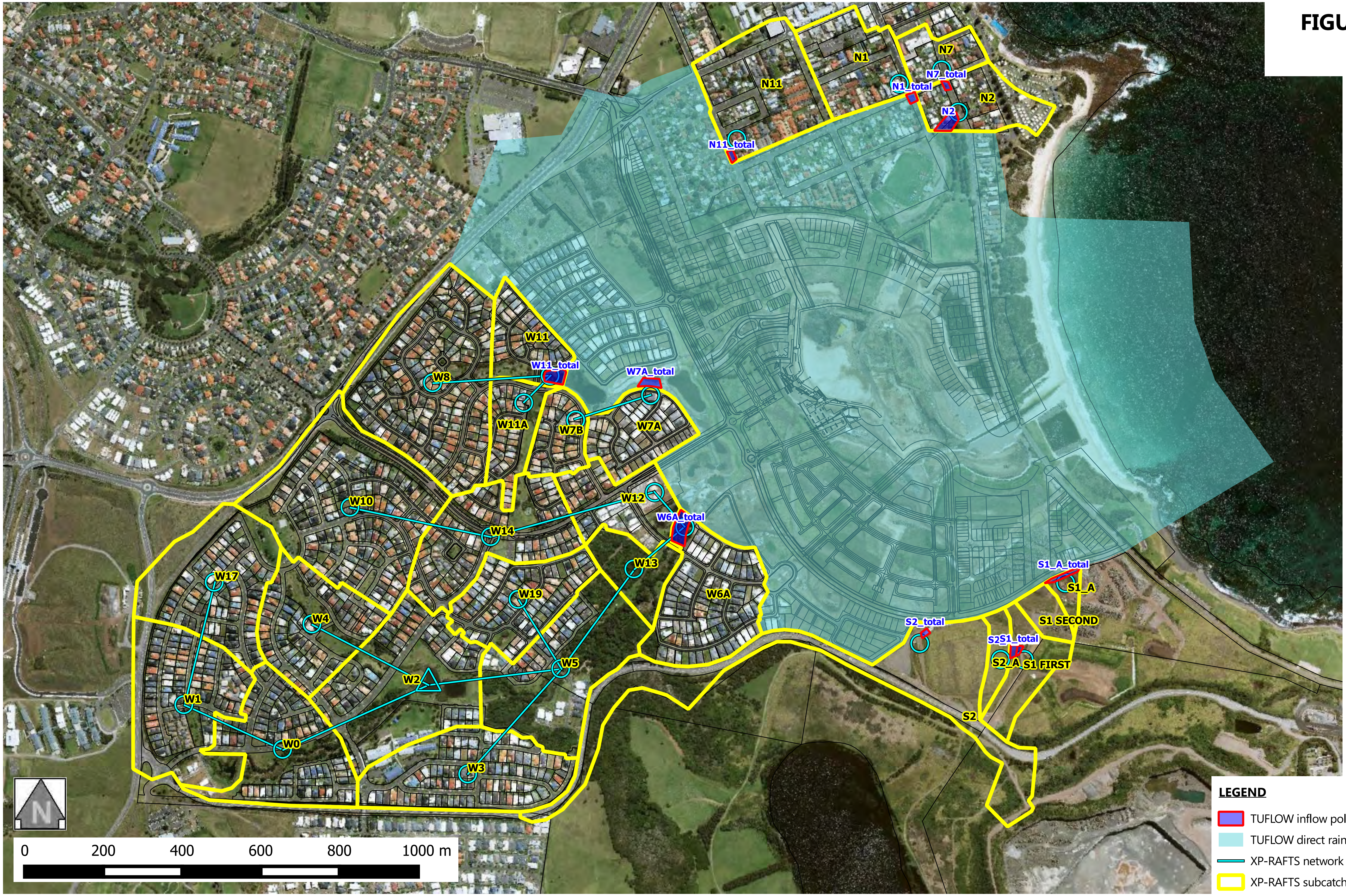
SHELL COVE BOAT HARBOUR PRECINCT
CHANGES TO DRAINAGE SYSTEM CONFIGURATION SINCE THE PART 3A APPROVAL

S75W Draft 2017-03-08 Figure 2.1 - Changes to Development
2017-03-03_ShellCove_Section75W.qgs

FIGURE 3.1



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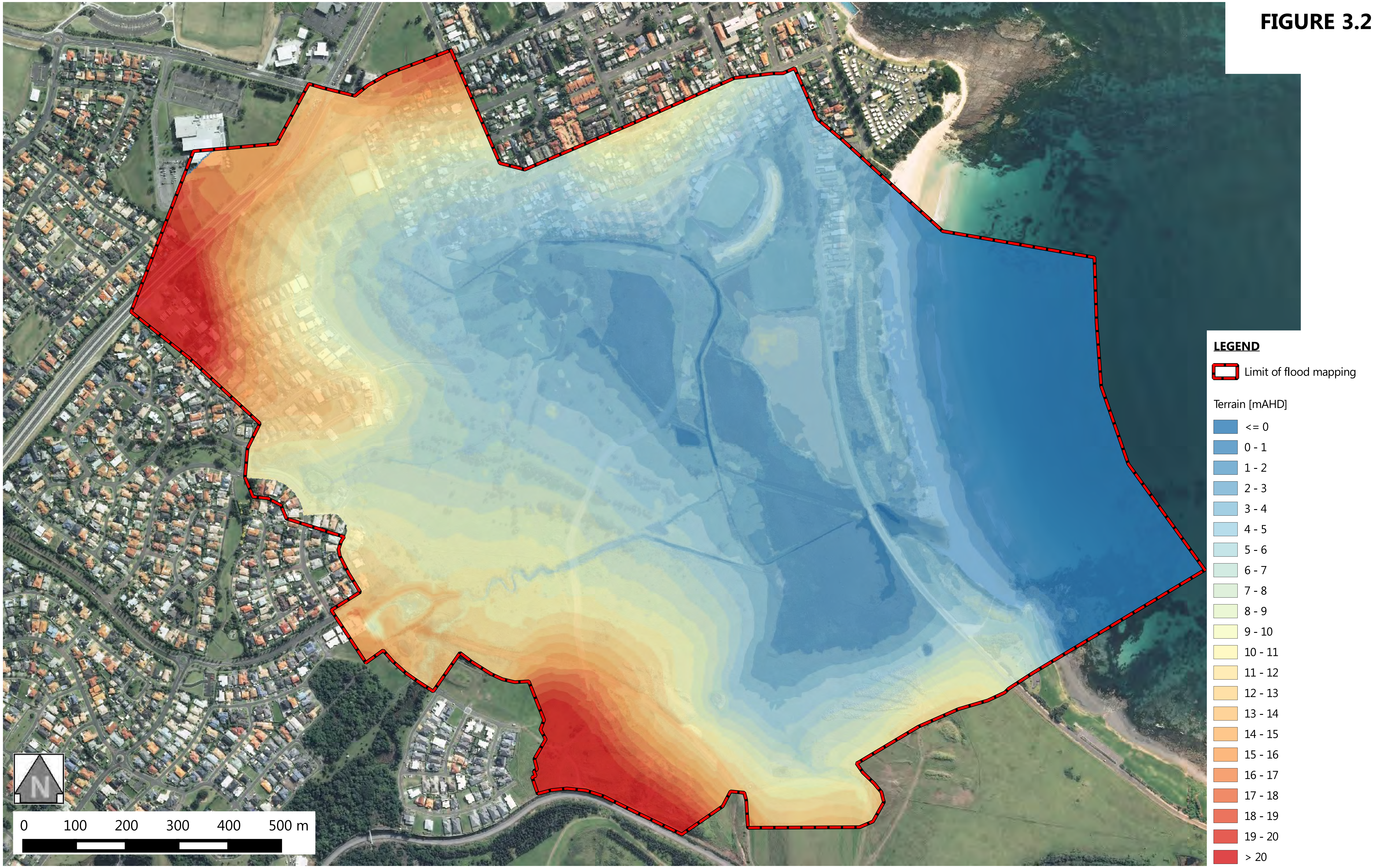
SHELL COVE BOAT HARBOUR XP-RAFTS CATCHMENT DELINEATION AND TUFLOW INFLOW POLYGONS

S75W Draft 2017-03-08 Figure 3.1 - RAFTS and Inflows
2017-03-03_ShellCove_Section75W.qgs

FIGURE 3.2



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S75W Draft 2017-03-08 Figure 3.2 - Pre-Dev Terrain
2017-03-03_ShellCove_Section75W.qgs

SHELL COVE BOAT HARBOUR TUFLOW MODEL TERRAIN
[PRE-DEVELOPMENT SCENARIO]

FIGURE 3.3

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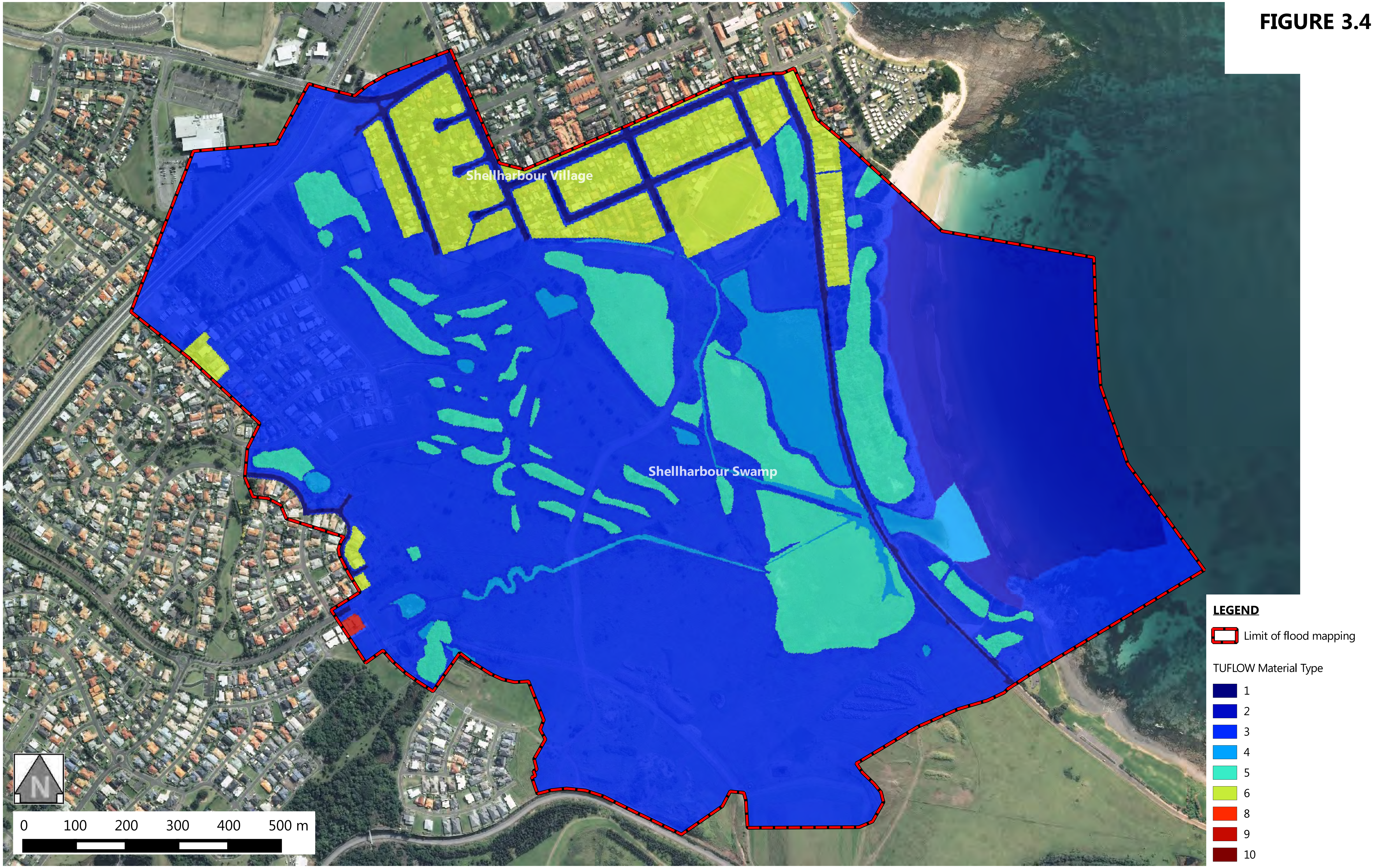
S75W Draft 2017-03-08 Figure 3.3 - Post-Dev Terrain
2017-03-03_ShellCove_Section75W.qgs

SHELL COVE BOAT HARBOUR TUFLOW MODEL TERRAIN [POST-DEVELOPMENT SCENARIO]

FIGURE 3.4



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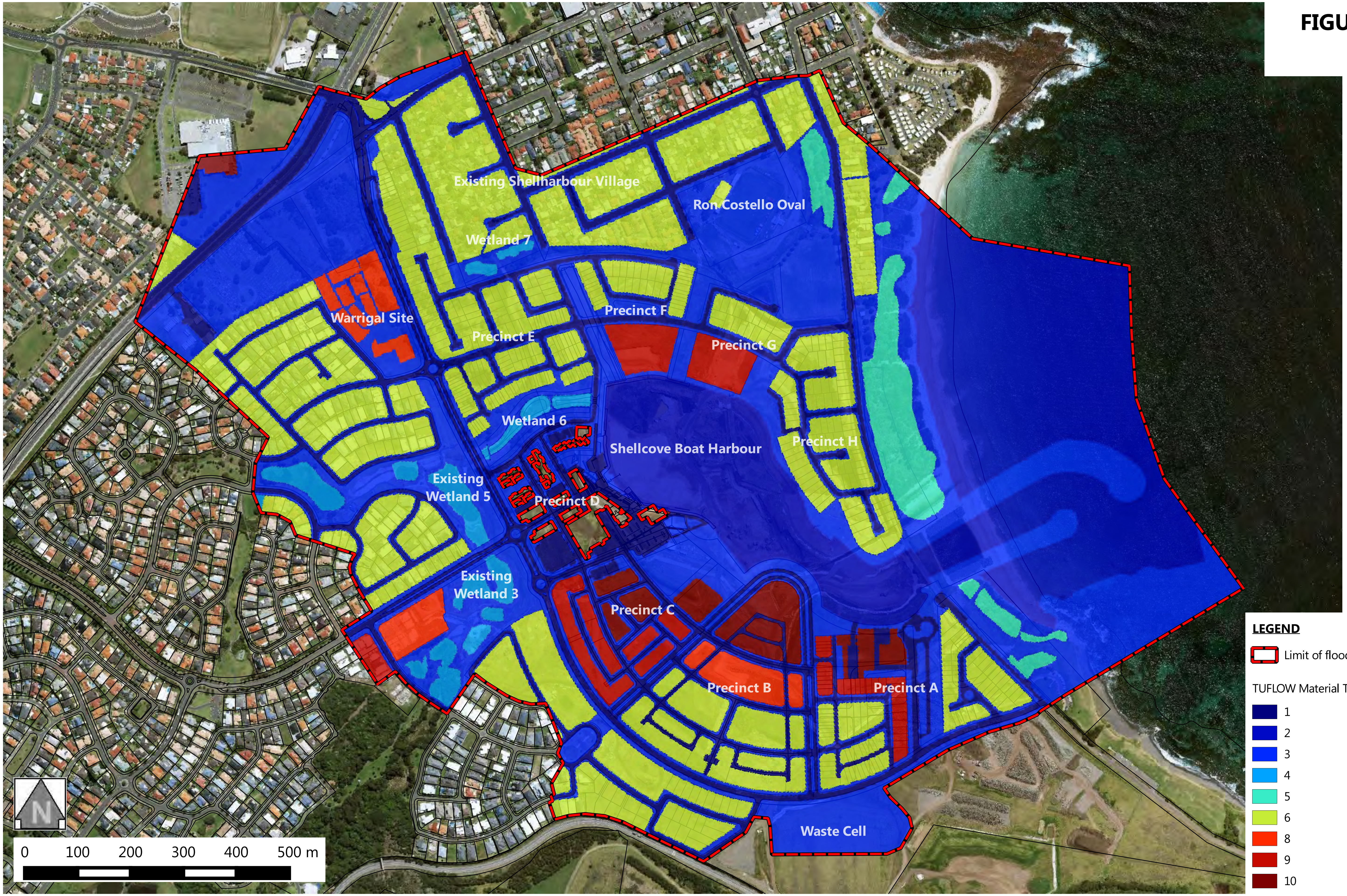
SHELL COVE BOAT HARBOUR TUFLOW MATERIAL TYPES [PRE-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 3.4 - Pre-Dev Material Types
2017-03-03_ShellCove_Section75W.qgs

FIGURE 3.5



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S75W Draft 2017-03-08 Figure 3.5 - Post-Dev Material Types
2017-03-03_ShellCove_Section75W.qgs

SHELL COVE BOAT HARBOUR TUFLOW MATERIAL TYPES
[POST-DEVELOPMENT SCENARIO]

FIGURE 3.6

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SHELL COVE BOAT HARBOUR TUFLOW MODEL CULVERT NETWORK [POST-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 3.6 - Post-Dev Culverts
2017-03-03_ShellCove_Section75W.qgs

FIGURE 4.1

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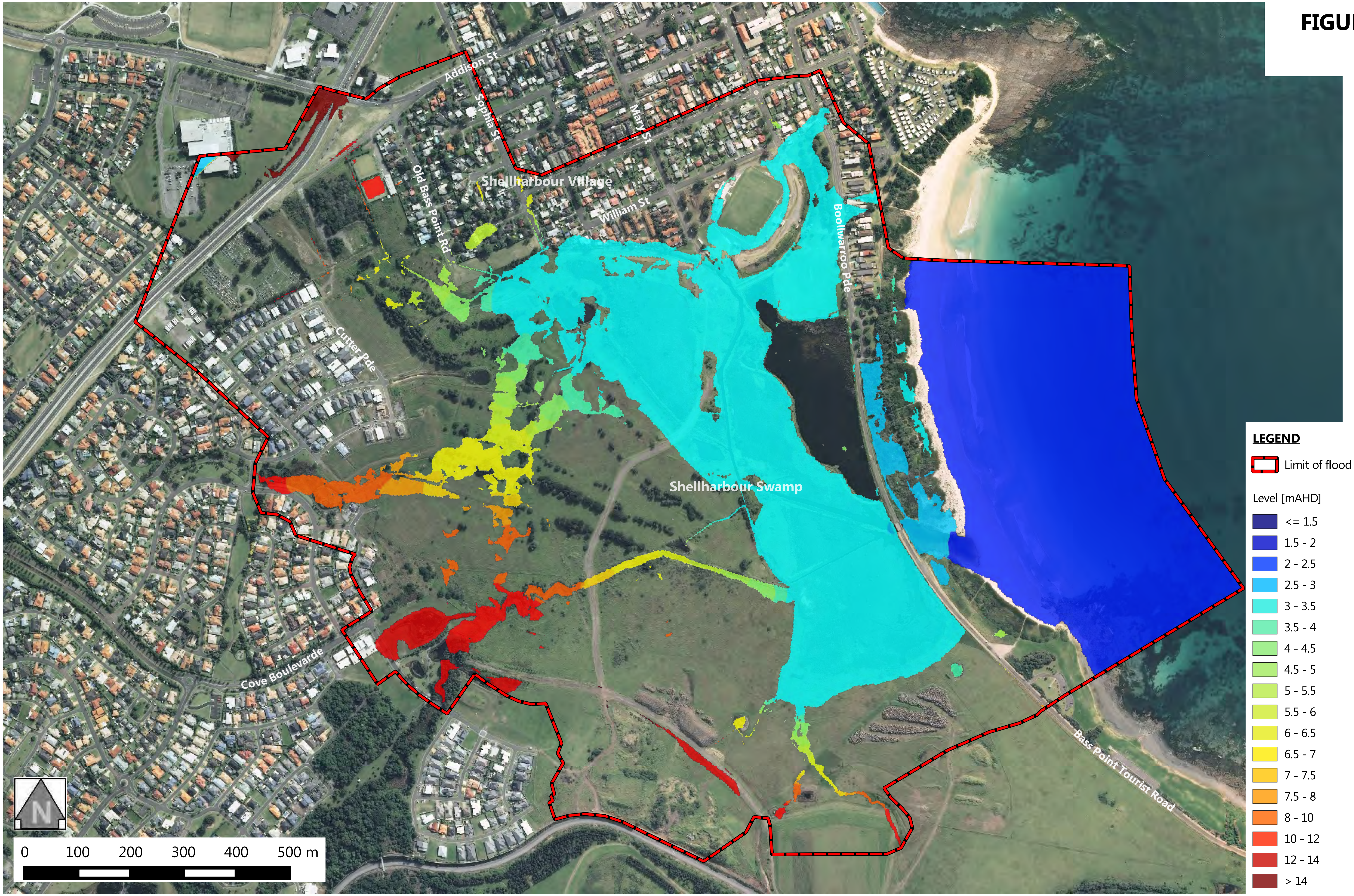
PEAK FLOOD LEVELS FOR THE 5 YEAR ARI EVENT [PRE-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 4.1 - 5yr Levels [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 4.2



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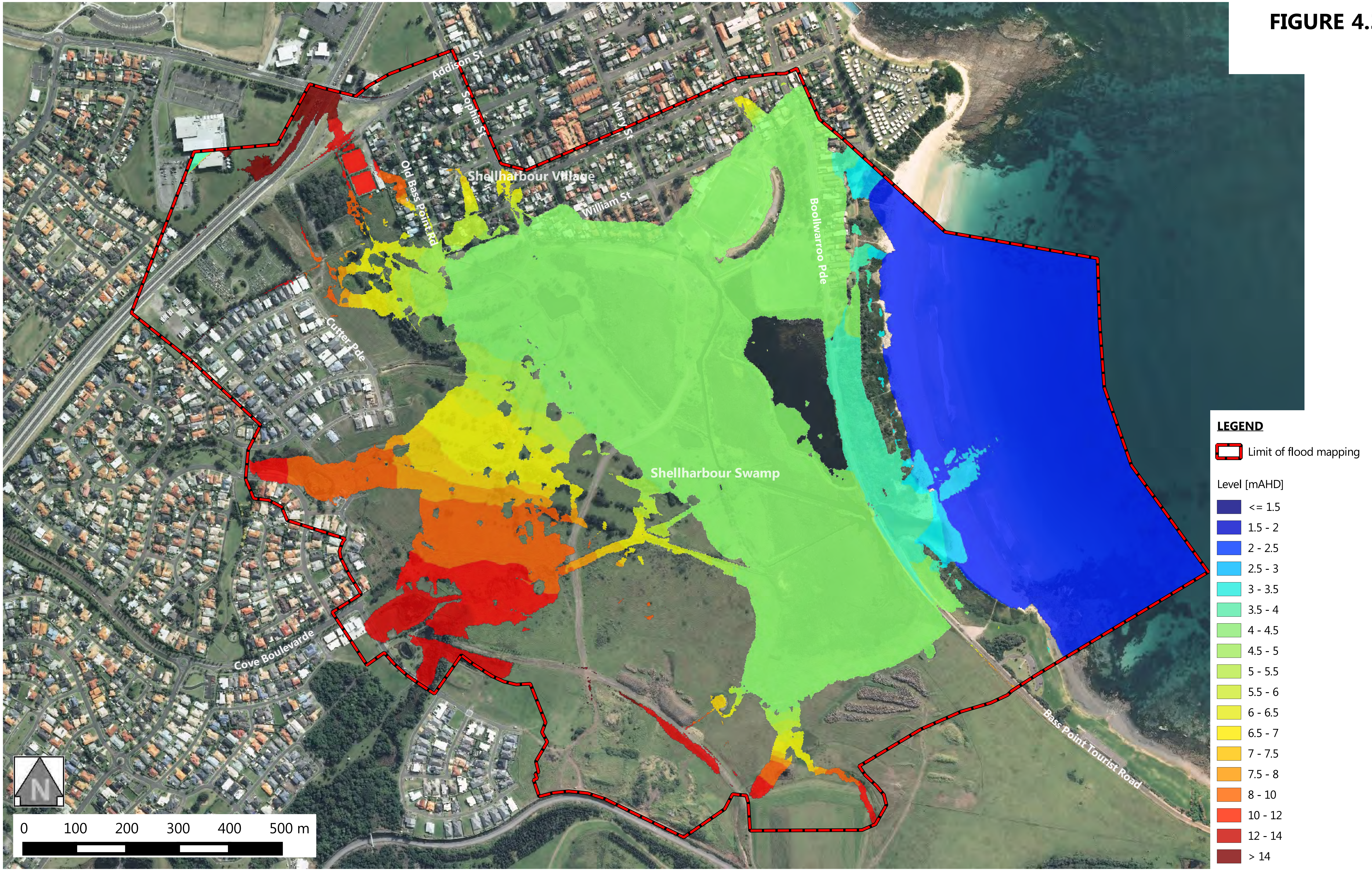
PEAK FLOOD LEVELS FOR THE 100 YEAR ARI EVENT [PRE-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 4.2 - 100yr Levels [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 4.3



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PEAK FLOOD LEVELS FOR THE PROBABLE MAXIMUM FLOOD [PRE-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 4.3 - PMF Levels [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 4.4

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PEAK FLOOD DEPTHS FOR THE 5 YEAR ARI EVENT [PRE-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 4.4 - 5yr Depths [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 4.5

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PEAK FLOOD DEPTHS FOR THE 100 YEAR ARI EVENT [PRE-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 4.05 - 100yr Depths [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 4.6

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PEAK FLOOD DEPTHS FOR THE PROBABLE MAXIMUM FLOOD [PRE-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 4.6 - PMF Depths [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 4.7

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5 YEAR ARI PROVISIONAL FLOOD HAZARD [PRE-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 4.7 - 5yr Hazards [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 4.8

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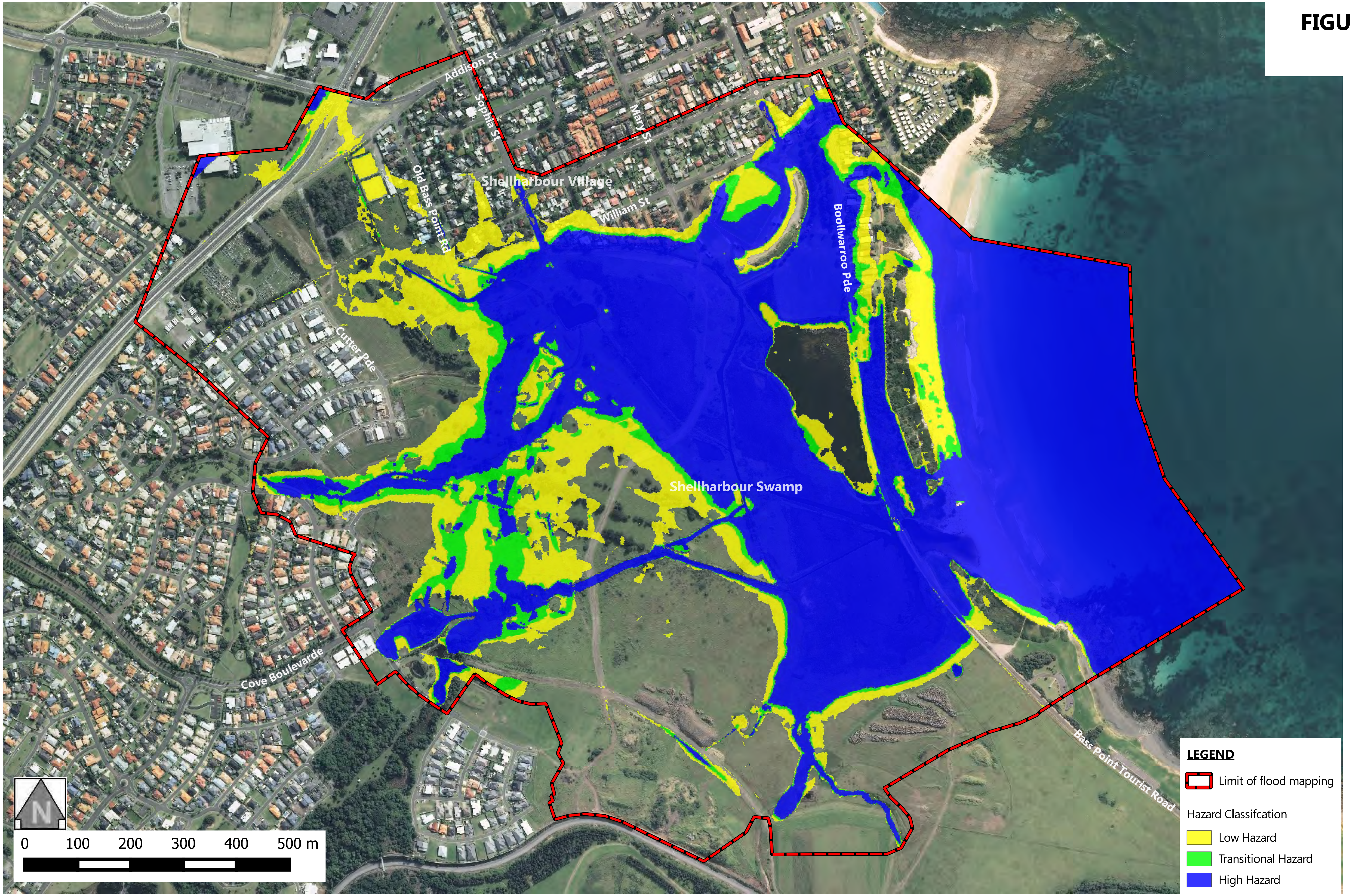
S75W Draft 2017-03-08 Figure 4.8 - 100yr Hazards [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

100 YEAR ARI PROVISIONAL FLOOD HAZARD
[PRE-DEVELOPMENT SCENARIO]

FIGURE 4.9



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PROVISIONAL PROBABLE MAXIMUM FLOOD HAZARD [PRE-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 4.9 - PMF Hazards [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 4.10

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S75W Draft 2017-03-08 Figure 4.10 - 5yr Level SOBEK Diff [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

COMPARISON OF 2017 TUFLOW AND 2009 SOBEK MODELLING RESULTS FOR THE 5 YEAR ARI EVENT [PRE-DEVELOPMENT SCENARIO]

FIGURE 4.11

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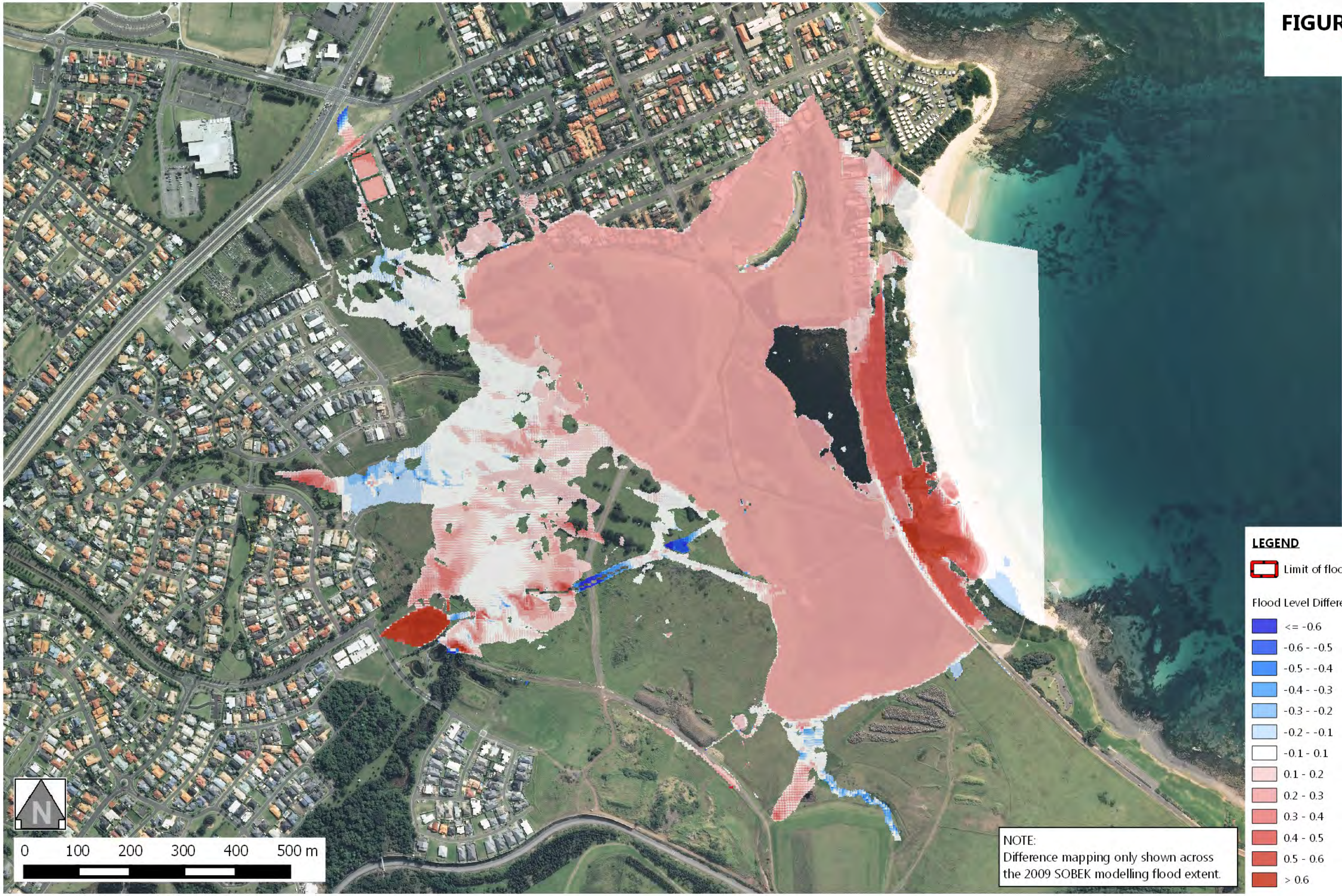
S75W Draft 2017-03-08 Figure 4.11 - 100yr Level SOBEK Diff [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

COMPARISON OF 2017 TUFLOW AND 2009 SOBEK MODELLING RESULTS FOR THE 100 YEAR ARI EVENT [PRE-DEVELOPMENT SCENARIO]

FIGURE 4.12



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S75W Draft 2017-03-08 Figure 4.12 - PMF Level SOBEK Diff [Pre-Dev]
2017-03-03_ShellCove_Section75W.qgs

COMPARISON OF 2017 TUFLOW AND 2009 SOBEK MODELLING RESULTS FOR THE PROBABLE MAXIMUM FLOOD [PRE-DEVELOPMENT SCENARIO]

FIGURE 5.1

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PEAK FLOOD LEVELS FOR THE 5 YEAR ARI EVENT
[POST-DEVELOPMENT SCENARIO]

FIGURE 5.2

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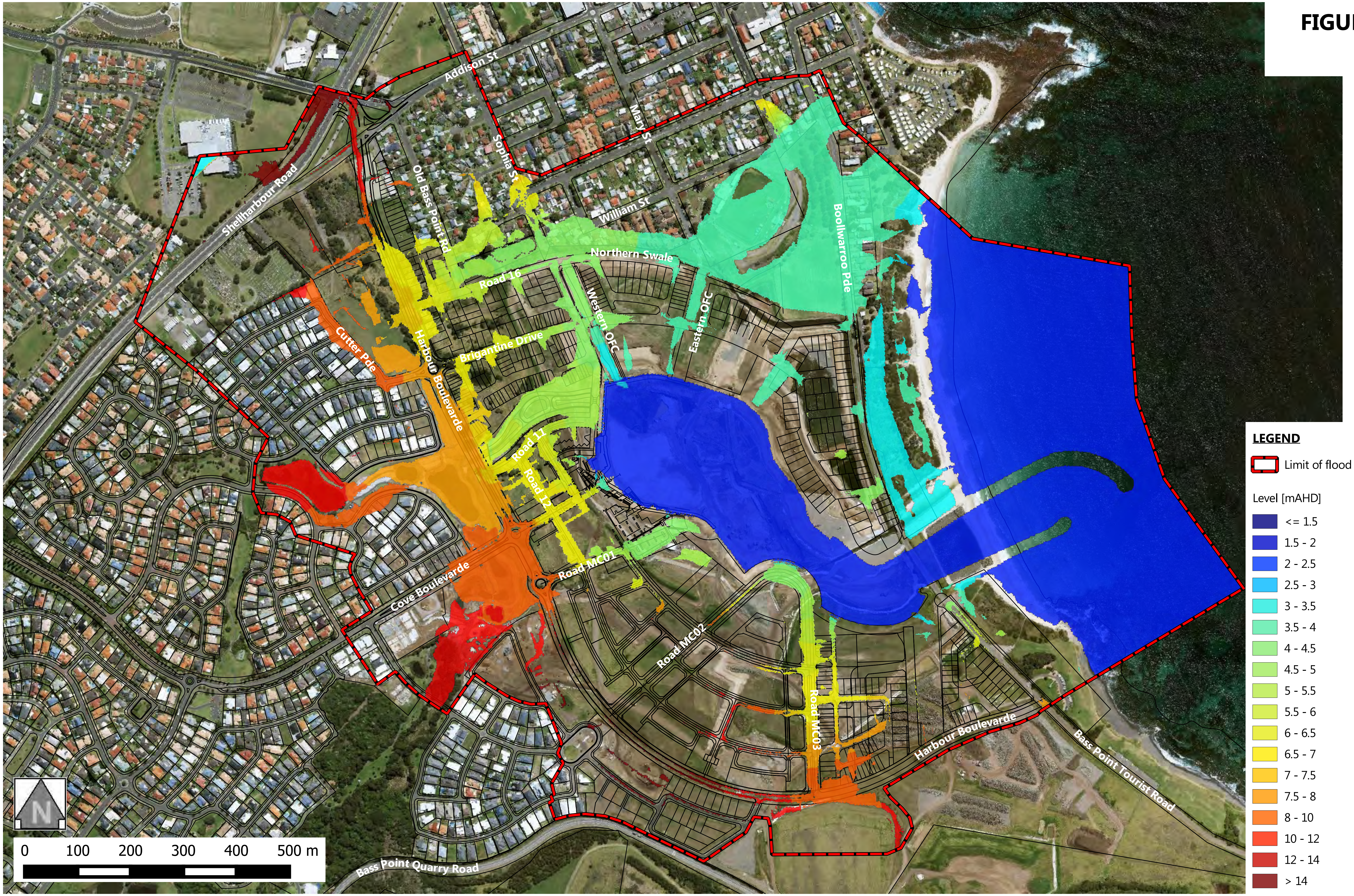
PEAK FLOOD LEVELS FOR THE 100 YEAR ARI EVENT [POST-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 5.02 - 100yr Levels [Post-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 5.3



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PEAK FLOOD LEVELS FOR THE PROBABLE MAXIMUM FLOOD
[POST-DEVELOPMENT SCENARIO]

FIGURE 5.4

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S75W Draft 2017-03-08 Figure 5.04 - 5yr Depths [Post-Dev]
2017-03-03_ShellCove_Section75W.qgs

PEAK FLOOD DEPTHS FOR THE 5 YEAR ARI EVENT
[POST-DEVELOPMENT SCENARIO]

FIGURE 5.5

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S75W Draft 2017-03-08 Figure 5.05 - 100yr Depths [Post-Dev]
2017-03-03_ShellCove_Section75W.qgs

PEAK FLOOD DEPTHS FOR THE 100 YEAR ARI EVENT [POST-DEVELOPMENT SCENARIO]

FIGURE 5.6

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PEAK FLOOD DEPTHS FOR THE PROBABLE MAXIMUM FLOOD [POST-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 5.06 - PMF Depths [Post-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 5.7

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5 YEAR ARI PROVISIONAL FLOOD HAZARD [POST-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 5.07 - 5yr Hazards [Post-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 5.8

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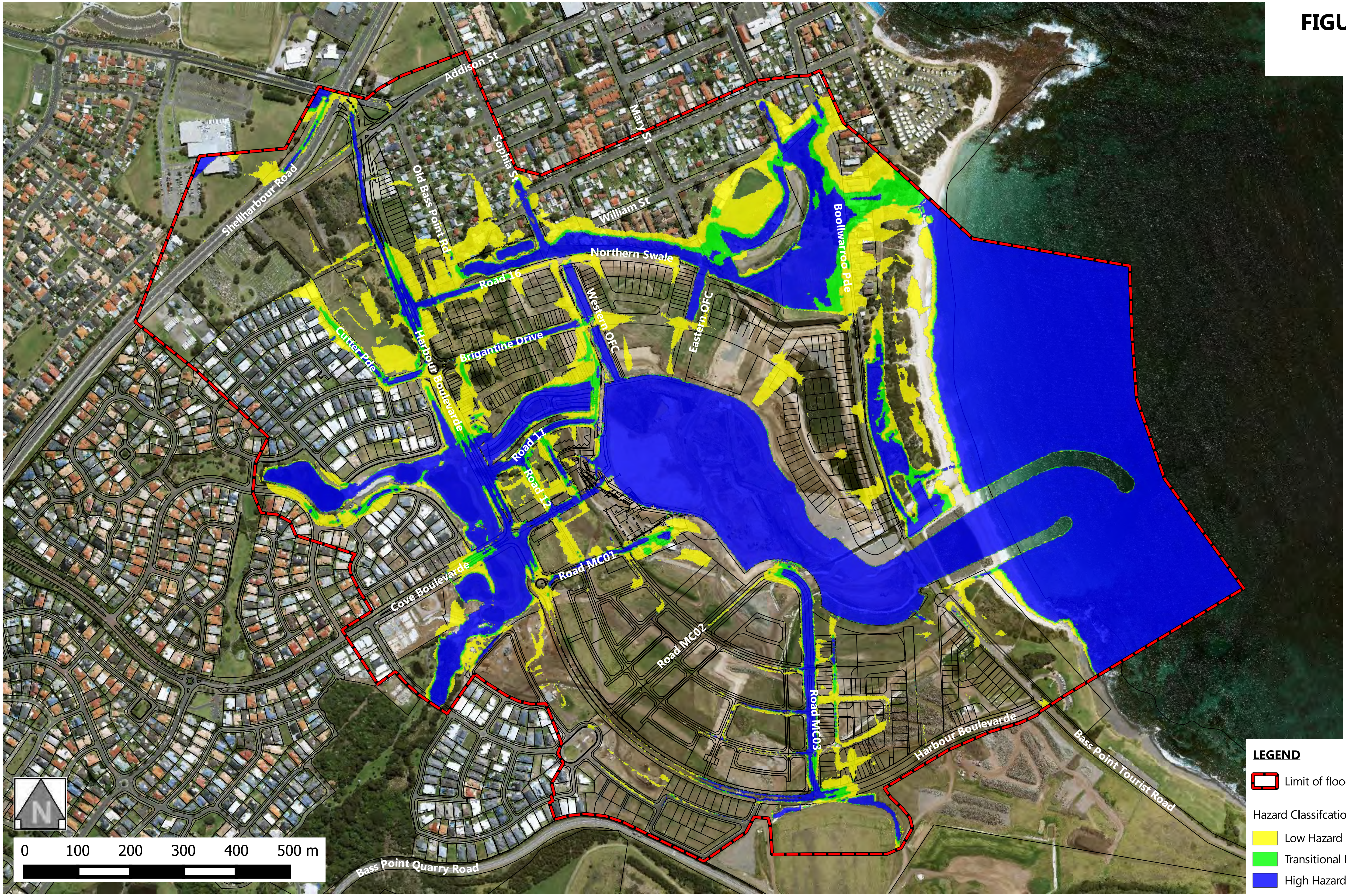
S75W Draft 2017-03-08 Figure 5.08 - 100yr Hazards [Post-Dev]
2017-03-03_ShellCove_Section75W.qgs

100 YEAR ARI PROVISIONAL FLOOD HAZARD [POST-DEVELOPMENT SCENARIO]

FIGURE 5.9



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S75W Draft 2017-03-08 Figure 5.09 - PMF Hazards [Post-Dev]
2017-03-03_ShellCove_Section75W.qgs

PROVISIONAL PROBABLE MAXIMUM FLOOD HAZARD [POST-DEVELOPMENT SCENARIO]

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IMPACT OF DEVELOPMENT ON PEAK FLOOD LEVELS DURING THE 5 YEAR ARI EVENT

S75W Draft 2017-03-08 Figure 5.10 - 5yr Level Imapacts
2017-03-03_ShellCove_Section75W.qgs

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S75W Draft 2017-03-08 Figure 5.11 - 100yr Level Imapacts
2017-03-03_ShellCove_Section75W.qgs

IMPACT OF DEVELOPMENT ON PEAK FLOOD LEVELS DURING THE 100 YEAR ARI EVENT

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S75W Draft 2017-03-08 Figure 5.12 - PMF Level Impacts
2017-03-03_ShellCove_Section75W.qgs

IMPACT OF DEVELOPMENT ON PEAK FLOOD LEVELS DURING THE PROBABLE MAXIMUM FLOOD

FIGURE 6.1

Aerial Imagery © Land and Property Information 2017

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HYDRAULIC CATEGORY MAPPING FOR THE 5 YEAR ARI EVENT [POST-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 6.1 - 5yr Hyd Cats [Post-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 6.2

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HYDRAULIC CATEGORY MAPPING FOR THE 100 YEAR ARI EVENT [POST-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 6.2 - 100yr Hyd Cats [Post-Dev]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 6.3

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HYDRAULIC CATEGORY MAPPING FOR THE PROBABLE MAXMIMUM FLOOD [POST-DEVELOPMENT SCENARIO]

S75W Draft 2017-03-08 Figure 6.3 - PMF Hyd Cats [Post-Dev]]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 8.1

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S75W Draft 2017-03-08 Figure 8.1 - 100yr Levels [Post-Dev Blockage]
2017-03-03_ShellCove_Section75W.qgs

PEAK FLOOD LEVELS FOR THE 100 YEAR ARI EVENT
[BRIDGE & CULVERT BLOCKAGE SCENARIO]

FIGURE 8.2

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S75W Draft 2017-03-08 Figure 8.2 - 100yr Depths [Post-Dev Blockage]
2017-03-03_ShellCove_Section75W.qgs

PEAK FLOOD DEPTHS FOR THE 100 YEAR ARI EVENT [BRIDGE & CUVLERT BLOCKAGE SCENARIO]

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S75W Draft 2017-03-08 Figure 8.3 - 100yr Hazards [Post-Dev Blockage]
2017-03-03_ShellCove_Section75W.qgs

100 YEAR ARI PROVISIONAL FLOOD HAZARD
[BRIDGE & CUVLERT BLOCKAGE SCENARIO]

FIGURE 8.4

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IMPACT OF CULVERT BLOCKAGE ON PEAK FLOOD LEVELS DURING THE 100 YEAR ARI EVENT

S75W Draft 2017-03-08 Figure 8.4 - 100yr Blockage WL Diff [Base Case]
2017-03-03_ShellCove_Section75W.qgs

FIGURE 8.5

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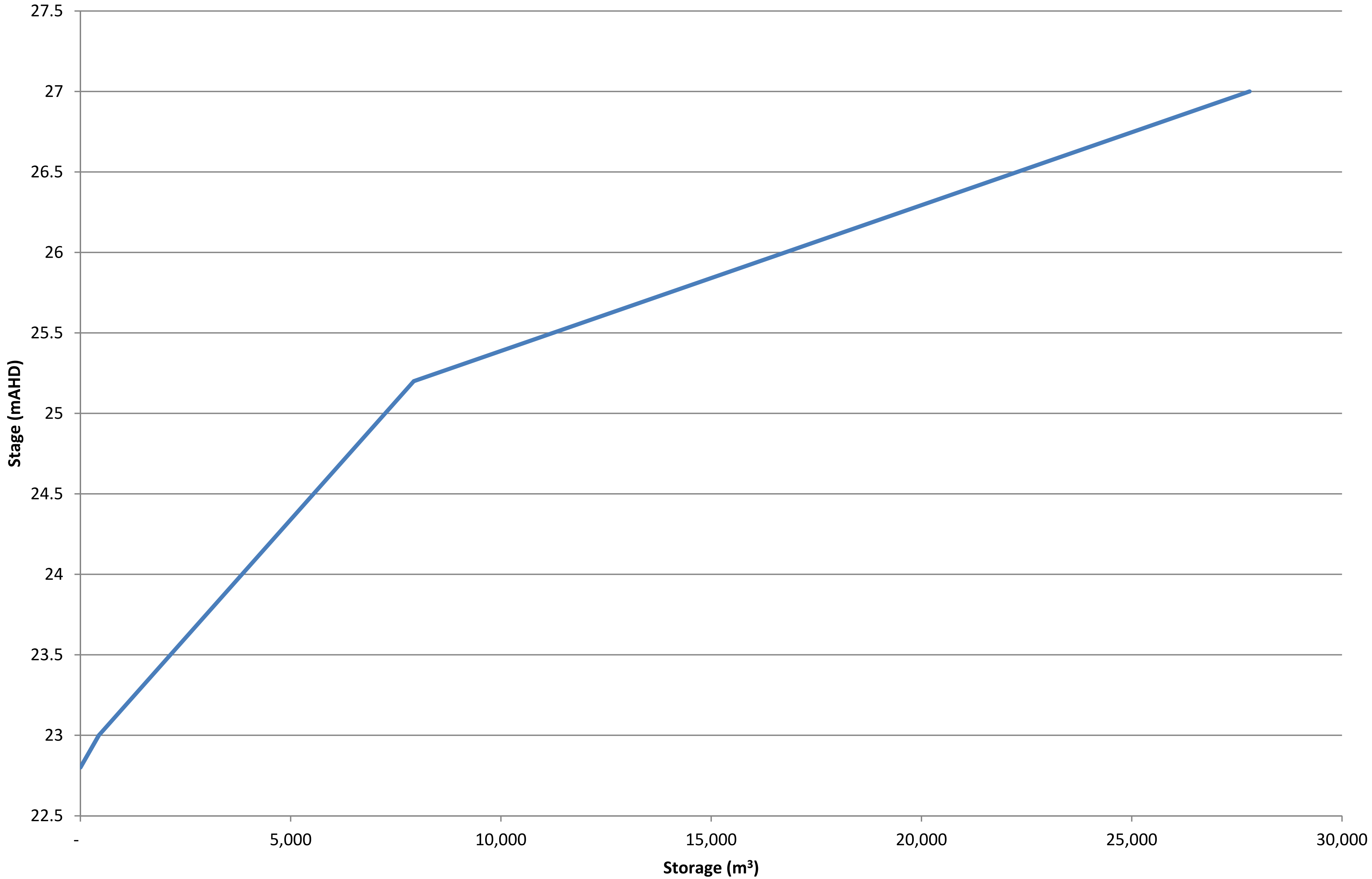
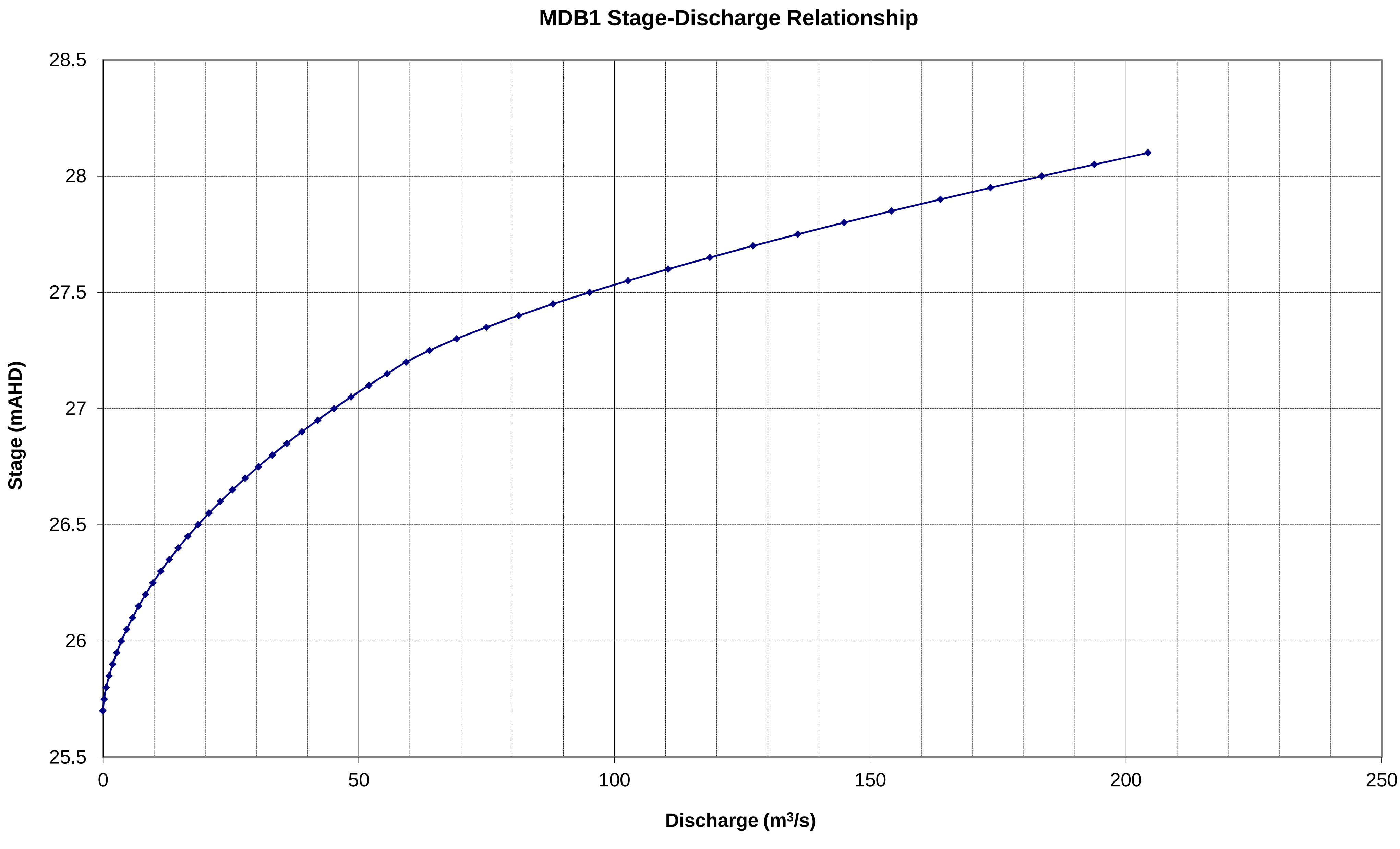
IMPACT OF DEVELOPMENT ON PRE-DEVELOPMENT FLOOD LEVELS DURING THE 100 YEAR ARI EVENT [WITH CULVERT BLOCKAGE]

S75W Draft 2017-03-08 Figure 8.5 - 100yr Blockage WL Diff [Pre-Ex]
2017-03-03_ShellCove_Section75W.qgs

Appendix A: Major Detention Basin 1 Stage-Storage & Storage-Discharge Relationships

MDB1 Stage-Storage Relationship




MDB1 Stage-Discharge Relationship
Stage (mAHD)
28.5
28
27.5
27
26.5
26
25.5
0
50
100
150
200
250
Discharge (m³/s)

Appendix B: 2009 SOBEK Model Roughness

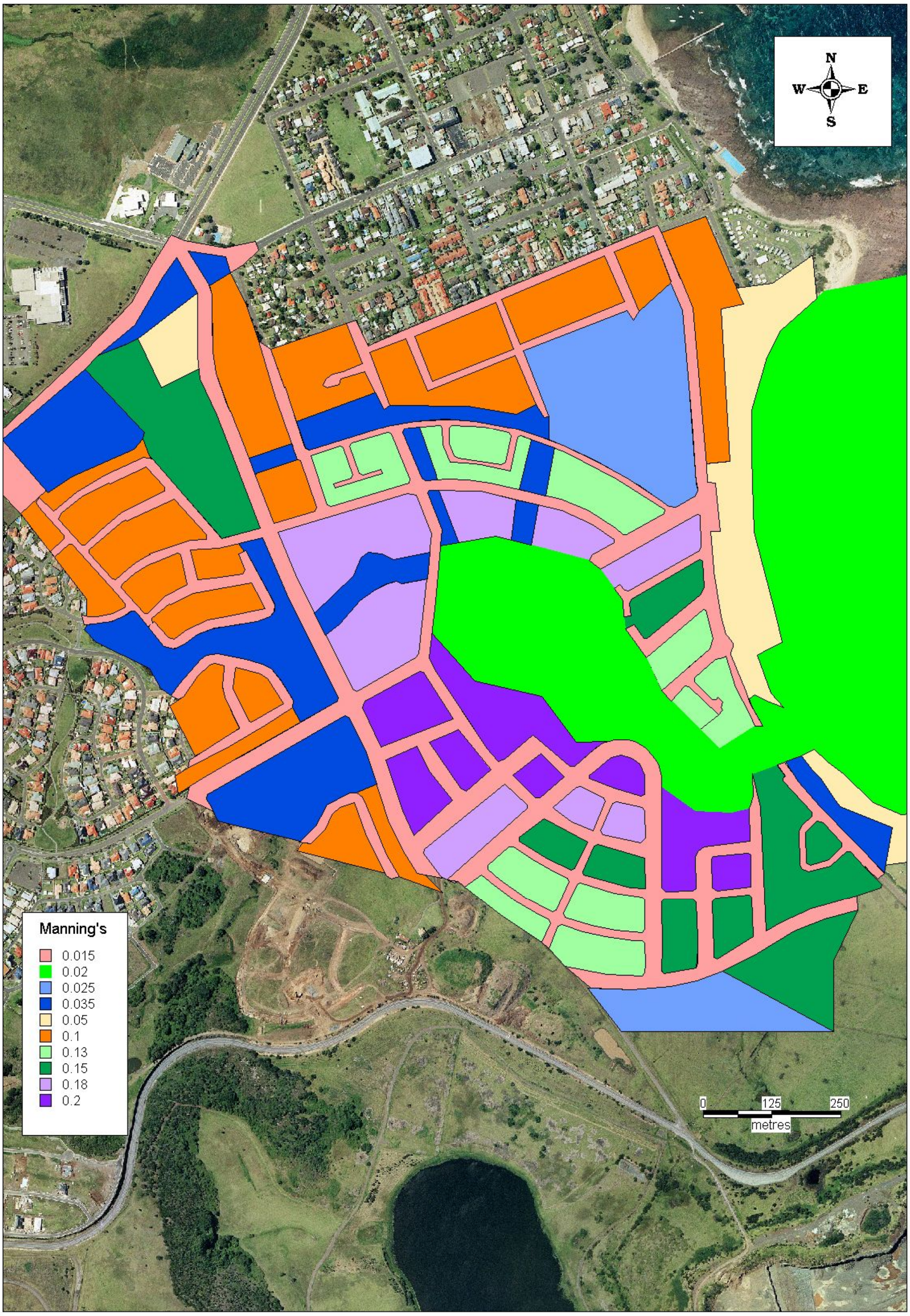


Figure 5.2 2D Roughness Values Adopted