ARRAWARRA CARAVAN PARK SUBDIVISION

Flooding and Stormwater Assessment

June 2016

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Flooding and Stormwater Assessment

Prepared by
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on behalf of
Astoria Group

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1.0 Introduction

1.1 Background

Astoria Group Pty Ltd (Astoria) is seeking approval to subdivide Arrawarra Beach Caravan Park (refer to **Figure 1.1**). A Flooding and Stormwater Assessment is required to support the development application. The township of Arrawarra is located on the east coast of New South Wales approximately 30 kilometres north of Coffs Harbour.

The site is situated at the eastern edge of the township of Arrawarra, specifically on Lot 1 DP 789002, Lot 1 DP 26125 and Lot 12 DP 835612. The site is located at the confluence of Arrawarra Creek and Yarrawarra Creek, at the entry of both creeks to the Pacific Ocean. There is a sand bar located at the mouth of both creeks which historically shifts and changes over time with impacts of tides and flooding events. A swampy depression south of the township of Corindi drains part of the Yarrawarra Creek catchment and acts as a sink, only rarely discharging into Yarrawarra Creek.

The proposed development is to be undertaken in a single stage with lots 1 to 13 in the southern portion of the site, and lots 14 to 24 in the northern portion of the site. To protect the site against erosion by wave action, storm surge and currents in Arrawarrra Creek and Yarrawarra Creek a revetment wall consisting of geotextile and rock armour is proposed.

In 2003, Umwelt (Australia) Pty Limited (Umwelt) undertook a Flood Study to determine the peak 1% Annual Exceedance Probability (AEP) flood levels for the site with consideration of extreme oceanic storm surges. Coffs Harbour City Council (Council) now requires that sea level rise associated with climate change be considered in assessing the impacts of development on the local flooding regime in line with NSW OEH *Coastal Risk Management Guide for Incorporating Sea Level Rise benchmarks in Coastal Risk Assessments* (OEH, 2010).

This report has been prepared to update the flooding and stormwater aspects of the proposed subdivision, including consideration of sea level rise. It includes assessment of the 100%, 20%, 10%, 5% and 1% AEP flood levels in conjunction with a range of extreme oceanic storm surge conditions. The combination of conditions modelled has an Annual Exceedance Probability of considerably less than 1%.

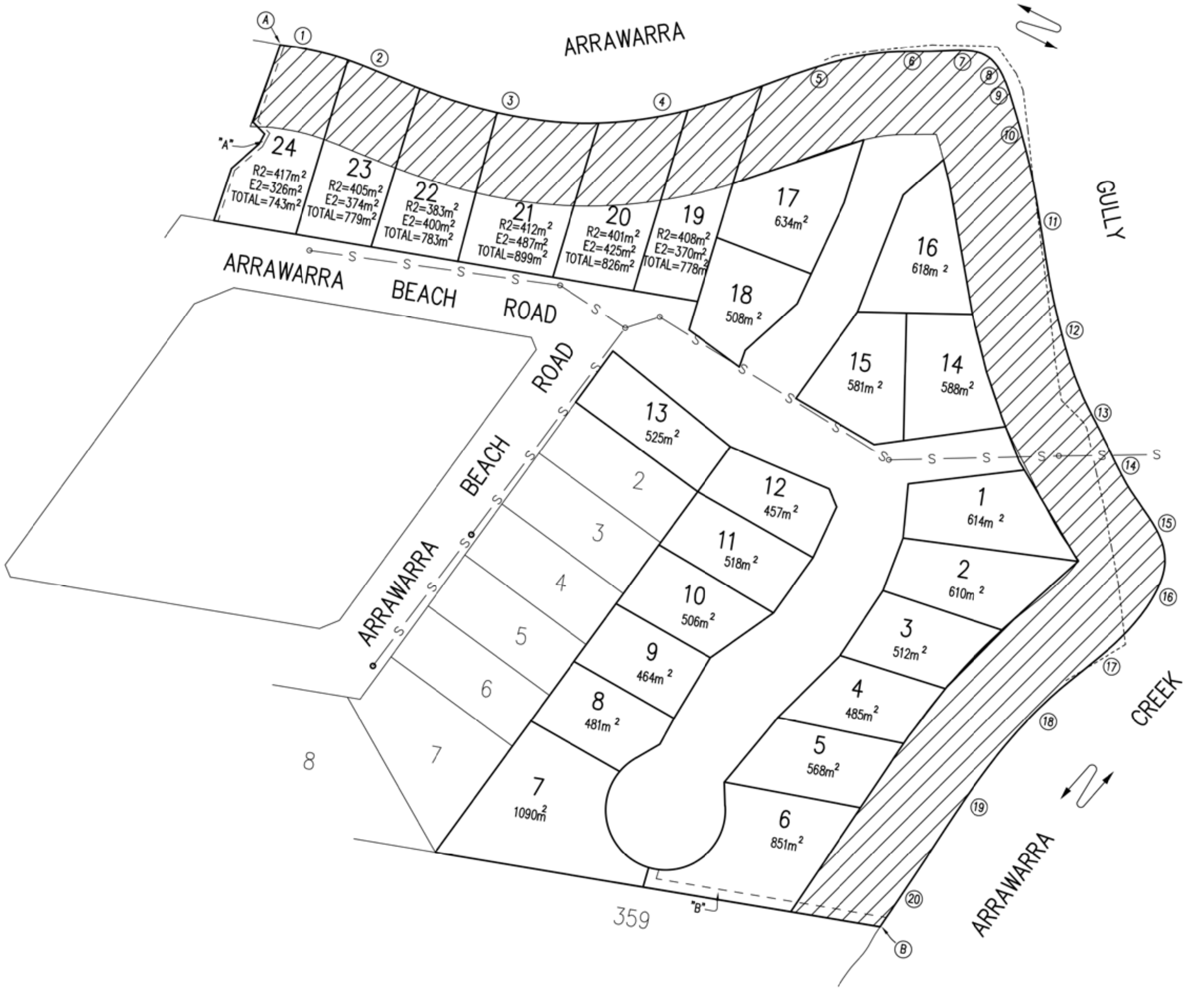


FIGURE 1.1

Proposed Subdivision Layout

Image Source: Astoria Group Pty Ltd (2015)

1.2 Report Structure

This report is structured as follows:

- Review of previous modelling work, including the proposed subdivision layout and a description of modifications and updates.
- Assessment of the design flood elevation and extent with consideration of sea level rise predictions of 0.4 metres and 0.9 metres and extreme oceanic storm surge conditions. The assessment includes identification of flooding issues and potential mitigation measures.
- Stormwater strategy and assessment, including a review of the subdivision catchment's hydrodynamic characteristics and proposed lot scale water sensitive urban design (WSUD) for stormwater detention.

2.0 Review of Previous Flood Study

The previous Flood Study that was prepared by Umwelt in 2003, was undertaken at the time to determine the 1% AEP flood levels for the site and also to consider the sensitivity of flood levels to outlet and tidal conditions.

2.1 Previous Model Inputs

The previous model considered both Arrawarra Creek and Yarrawarra Creek catchments. The model was based on a detailed site survey including cross-sections of both creeks in the vicinity of the site undertaken in 2003. Upstream cross sections were defined based on existing maps and aerial photography. The spatial data used in the model was sourced from:

- Arrawarra Caravan Park Revetment Design, Document No. 31390-001 (SMEC, 2003)
- Arrawarra Caravan Park Revetment Design Report (SMEC, 2003)
- Detailed plan and cross sections based on the ground survey of the project site by Newnham Karl Weir & Partners (April 2003)
- Woolgoolga Y1865-1, Corindi Beach Y1872-7, Y1872-4 1:4,000 Ortho-rectified photographic maps
- Woolgoolga NSW 1:25,000 Topographic Map (map sheet number 9537-4-N).

The sand bar at the mouth of the creeks was included in the model with the dimensions based on survey data and aerial photography. The sand bar historically shifts and changes, however the cross section defined for the purposes of the model was considered a reasonable reflection of average natural conditions for the purposes of the flood study (Umwelt, 2003).

The swampy depression downstream of the Corindi Beach township was modelled as an offline detention basin that only rarely overflows to the creek system.

The previous model is a one dimensional (1D) hydrodynamic model constructed using XP-Storm software. Tailwater effects were included in the model as a downstream boundary condition consistent with studies undertaken for the revetment wall design (SMEC, 2003). Two tailwater scenarios were modelled:

- A low tide of 0.5 metres coinciding with peak flows from the creeks to enable assessment of peak velocities associated with the 1% AEP flood.
- Extreme storm surges of 2.85 mAHD and 3.0 mAHD respectively described in SMEC (2003) coinciding with peak flows from the creeks to enable assessment of peak flood depths associated with the 1% AEP flood.

In addition, a sensitivity analysis was undertaken to assess the impact of changes to the sand bar height and rainfall infiltration rates on flooding at the site.

2.2 Previous Results

For the low tide tailwater condition, modelled flow velocities associated with the 1% AEP event were within the design specification of the sea wall. A maximum velocity of 2.15 m/s was modelled at the outfall of the creeks to the ocean. Peak velocities within the reaches of both creeks were modelled to be less than 1.25 m/s adjacent to and upstream of the site.

For the extreme storm surge condition (3.0 mAHD), the peak 1% AEP flood elevation was estimated at 3.02 mAHD with a peak outflow velocity of 0.31 m/s. Mitigation of the impacts of this flood, in line with Council requirements, dictated that floor levels for the development be at a minimum of 3.52 mAHD, which would likely require filling of the site. Filling of the site to 3.02 mAHD was modelled and results indicated no impact on upstream flood depths or flow velocities for the 1% AEP event.

The sensitivity analysis indicated that the tailwater levels dominate the flood regimes adjacent to the site, essentially drowning the effects of varying rainfall intensity, infiltration rate and sand bar height.

2.3 Modifications and Updates

A review of the previous model (Umwelt, 2003) indicates that the catchment parameters used, including Mannings 'n', initial and continuing losses, and catchment areas are considered appropriate and as such have not been changed for the current study.

The previous results in considering Finished Floor Levels (FFL) for the site are considered to be conservative, since the model was constrained such that the peak runoff occurred concurrently with the peak storm surge conditions resulting in combined flood and storm surge conditions with an AEP of significantly less than 1% (i.e. 0.05%).

In 2015 the NSW Office of Environment and Heritage (NSW OEH) published the *Floodplain Risk Management Guide – Modelling the Interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways.* This guide provides advice on approaches that can be used to derive ocean boundary conditions and design flood levels for flood investigations in coastal waterways considering the interaction of catchment flooding and oceanic inundation for the various classes of estuary waterways found in NSW and likely corresponding ocean boundary conditions. The procedure for modelling ocean level interactions is dependent on the waterway type. For the subject site, the waterway entrance type is defined as "Group 4 Intermittently Closed Estuaries (also known as intermittently closed and open lakes and lagoons (ICOLLs)) i.e. Type C". These are coastal water bodies that become isolated from the sea for extended periods. Based on this waterway entrance type the following modelling scenarios were considered:

- Steady state ocean boundary for Waterway Entrance Type C based on levels obtained from Figure 5.1 of the NSW OEH Guide (for sites north of Crowdy Head)
- Dynamic ocean boundary for Waterway Entrance Type C for each ocean scenario (for sites north of Crowdy Head)
- Dynamic Indicative Spring & Neap Tide Cycles incorporating Indian Springs Low Water (ISLW) and High High Water Springs (Solstice Spring) (HHWS(SS)) for sites north of Crowdy Head

In order to determine flood risk at the site, a range of design events were analysed, as outlined in Table 8.1 of the Floodplain Risk Management Guide (NSW OEH, 2015), reproduced here as **Table 2.1**.

Table 2.1 Combinations of Catchment Flooding and Oceanic Inundation Scenarios

Design AEP for peak levels/ velocities	Catchment Flood Scenario	Ocean Water Level Boundary Scenario	Comment/Reference
50% AEP	50% AEP	HHWS(SS)	Dynamic hydrograph can be taken from Appendix C of the NSW OEH Guide with peak flood to coincide with HHWS (SS) highest peak for highest water levels Peak HHWS (SS) 1.25 mAHD
20% AEP	20% AEP	HHWS(SS)	
10% AEP	10% AEP	HHWS(SS)	
5% AEP	5% AEP	HHWS(SS)	
2% AEP	2% AEP	5% AEP	Dynamic ocean water level boundary hydrograph Appendices A or B of the NSW OEH Guide for relevant waterway type
1% AEP Envelope level	5% AEP	1% AEP	Envelope provides 1% AEP design flood estimate Dynamic ocean water level boundary hydrograph Appendices A or B of the NSW OEH Guide for relevant waterway type
1% AEP Envelope level	1% AEP	5% AEP	
1% Envelope velocity	1% AEP	ISLW	Dynamic hydrograph can be taken from Appendix C of the NSW OEH Guide with peak flood to coincide with ISLW lowest trough for peak velocities in entrance Fixed ISLW approx. -0.95 mAHD
0.5% AEP	0.5% AEP	1% AEP	Dynamic ocean water level boundary hydrograph Appendices A or B of the NSW OEH Guide for relevant waterway type
0.2% AEP	0.2% AEP	1% AEP	
PMF	PMF	1% AEP	
1% AEP Catchment	1% AEP	HHWS(SS)	Suggested envelopes for analysis of catchment flooding only
PMF Catchment	PMF	HHWS(SS)	

Source: Table 8.1 Floodplain Risk Management Guide (NSW OEH, 2015)
HHWS(SS) High High Water Springs (Solstice Spring)
ISLW Indian Springs Low Water

Deriving design or planning flood levels in coastal waterways requires the use of a series of catchment flood and oceanic inundation scenarios to produce an envelope of peak flood levels, as these vary with location. Deriving the peak flood levels for a 1% AEP event, may involve investigating the following scenarios:

- Design 1% AEP oceanic inundation with 5% AEP catchment flooding with coincident peaks, to test peak levels.

- Design 5% AEP oceanic inundation with 1% AEP catchment flooding with coincident peaks; to test peak levels.

- Coincidence of ISLW in indicative spring and neap tide cycle with 1% AEP catchment flooding to test peak velocities.

A number of flood events were modelled by simulating certain storm events with tailwater conditions in accordance with the NSW OEH Guide. These flood events and the results of the modelling are detailed in **Section 3.** Ocean water levels were selected in line with locations north of Crowdy Head (the site lies approximately 200 kilometres north of Crowdy Head).

The sea wall design used in the model was updated to reflect the current design using the drawing set ARRAWARRA BEACH REVETMENT_15-849NSW (18 September 2015).

The subdivision layout and sea wall cross sections are shown in **Figures 2.1** to **2.3**.

The tailwater conditions used in the model were updated in line with *Floodplain Risk Management Guide* (NSW OEH, 2015). The model was also run with tailwater conditions reflecting sea level rise conditions of 0.4 metres and 0.9 metres respectively. The topography used in the model was also modified to include the sea wall.

No other changes were made to the existing model.

Legend

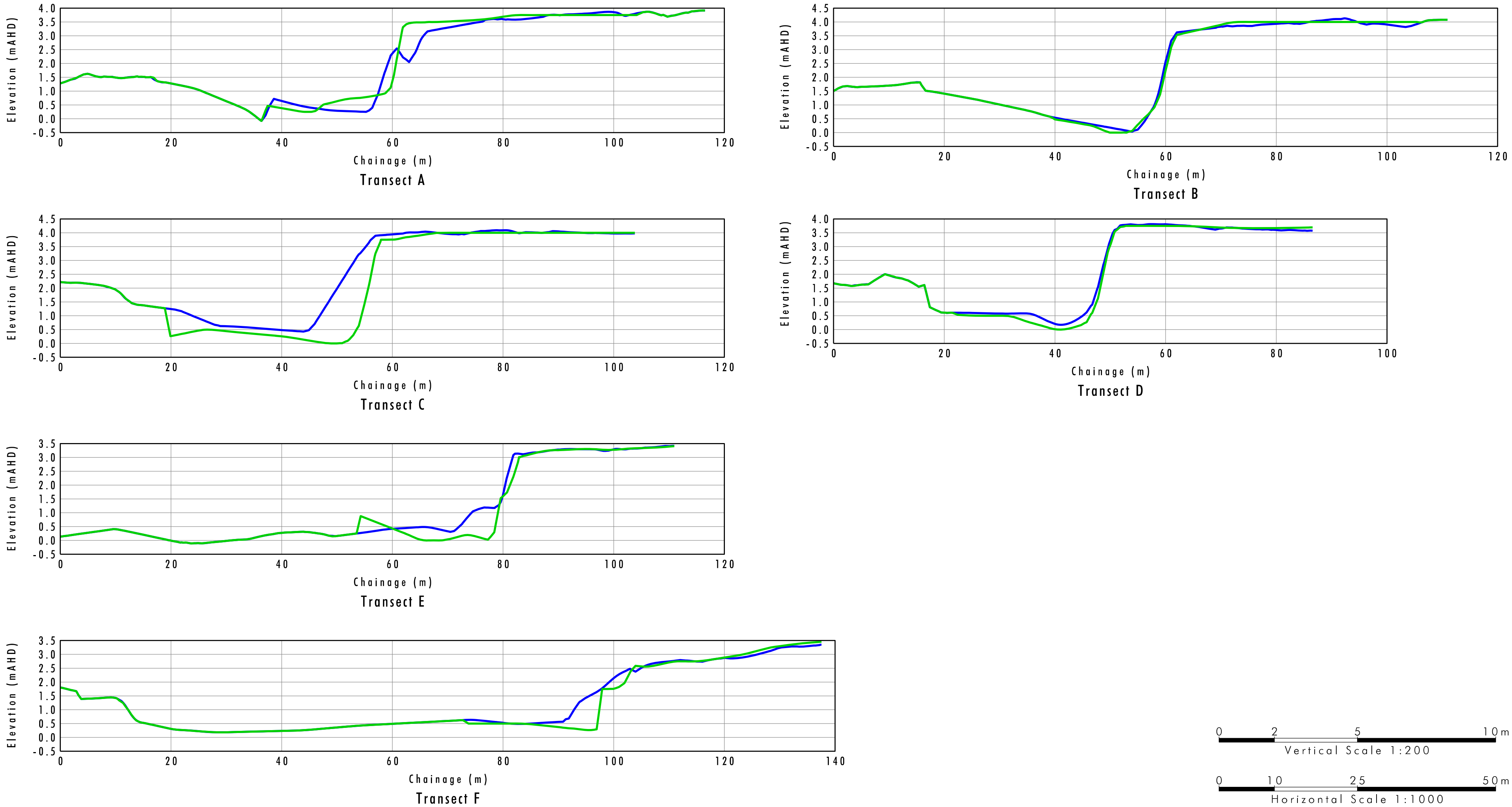
Proposed Subdivision of Arrawarra Caravan Park

Cross Section Location

FIGURE 2.1

Sea Wall Design Cross Sections Locations

umwelt



Legend

Original Landform
Sea Wall Design

FIGURE 2.2

Sea Wall Design Cross Sections

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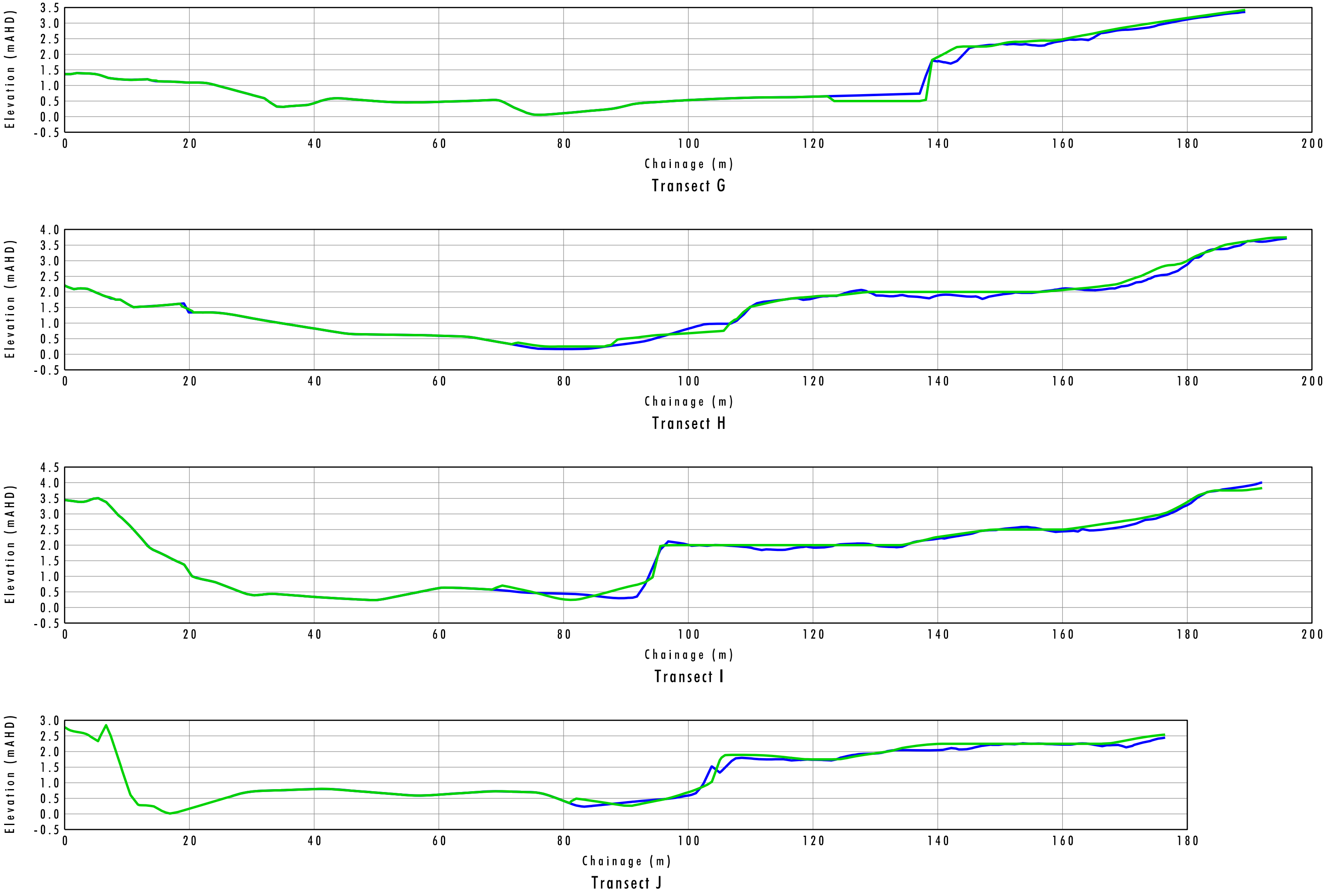

0 2 5 10m
Vertical Scale 1:200

0 10 25 50m
Horizontal Scale 1:1000

Legend
Original Landform
Sea Wall Design

FIGURE 2.3

Sea Wall Design Cross Sections

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3.0 Flooding Assessment

3.1 Baseline conditions

Figures 3.1 to **3.2** show the extents of the modelled flood events for the existing and developed site respectively.

For the scenarios modelled, the maximum flood elevation at the junction of Arrawarra Creek and Yarrawarra Creek occurs for the 1% AEP storm event combined with the 5% AEP dynamic tailwater condition. The elevation of the peak flood depth of this flood event is 2.65 mAHD. It should be noted that the probability of this flood event occurring in any given year (that is, the probability of the 1% AEP storm event coinciding with the 5% AEP ocean tide level) is 0.0005, that is, 1 in 2,000 Average Recurrence Interval. Further, ocean tide levels generally peak sometime after the occurrence of the low pressure system triggering the associated significant storm event. Therefore, in order for the peak ocean level to coincide with the peak runoff event, two or more critical duration storm events would need to occur in quick succession. This is considered to further reduce the likelihood of this occurring.

The results are summarised in **Table 3.1** below. These results incorporate a number of catchment runoff tide scenarios, including longer-duration storm events (72 hours) in order to capture the peak tide level from the time-varying ocean tide data, and shorter duration (2 hours) storm events in order to capture the peak runoff flow rate with a fixed tide level, in line with NSW OEH requirements

The sea wall was also modelled and found to have negligible impact on flood levels for the tailwater conditions modelled. The model results presented in **Table 3.1** include the sea wall.

Table 3.1 Maximum Modelled Flood Elevations at the Junction of Arrawarra Creek and Yarrawarra Creek

Scenario	Storm Event (AEP)	Critical Duration (hrs)	Ocean Tide (AEP)	Tide Peak (mAHD)	Modelled Tidal Cycle[1]	Maximum modelled elevation (mAHD)	Combined Flood Probability
Existing	5%	72	1%	2.65	Dynamic Hydrograph	2.65	0.0005
Existing	1%	72	5%	2.45	Dynamic Hydrograph	2.45	0.0005
Existing	2%	72	5%	2.45	Dynamic Hydrograph	2.45	0.001
Existing	5%	2	HHWS (SS)	1.25	Fixed	2.14	0.05
Existing	10%	2	HHWS (SS)	1.25	Fixed	2.02	0.1
Existing	20%	2	HHWS (SS)	1.25	Fixed	1.92	0.2

Scenario	Storm Event (AEP)	Critical Duration (hrs)	Ocean Tide (AEP)	Tide Peak (mAHD)	Modelled Tidal Cycle[1]	Maximum modelled elevation (mAHD)	Combined Flood Probability
Existing	50%	2	HHWS (SS)	1.25	Fixed	1.74	0.5
Developed	5%	72	1%	2.65	Dynamic Hydrograph	2.65	0.0005
Developed	1%	72	5%	2.45	Dynamic Hydrograph	2.45	0.0005
Developed	2%	72	5%	2.45	Dynamic Hydrograph	2.45	0.001
Developed	5%	2	HHWS (SS)	1.25	Fixed	2.14	0.05
Developed	10%	2	HHWS (SS)	1.25	Fixed	2.02	0.1
Developed	20%	2	HHWS (SS)	1.25	Fixed	1.92	0.2
Developed	50%	2	HHWS (SS)	1.25	Fixed	1.74	0.5

[1]Refer to Comments/References in **Table 2.1**

Table 3.1 shows that the maximum modelled flood depth using the envelope approach (refer to **Section 2.3**) occurs for the 5% AEP storm event combined with the 1% AEP ocean tide condition. The maximum modelled flood depth is 2.65 mAHD. The results show that the ocean tide condition is the dominant factor influencing the maximum modelled flood elevation at the site.

Figure 3.2 below shows the cross sections of the existing site and the developed site in the vicinity of the proposed sea wall.

3.2 Sea Level Rise Scenarios

In October 2009, the NSW Government released a sea level rise policy statement which sets the levels to be used in stormwater and flooding assessment studies. These benchmarks are for a sea level rise of 0.4 metres by the year 2050 and 0.9 metres by the year 2100 above the mean average sea level recorded in 1990. Although no longer explicit policy of the NSW Government, these are the levels used in the Coffs Coast Coastal Processes and Hazard Definition Study (BMT WBM Pty Ltd, 2011), and are adopted here.

In order to assess the impact of sea level rise, the sea level rise scenarios described above have been added to the peak ocean level of the maximum modelled flood event determined using the envelope approach described above (2.65 mAHD, refer to **Sections 2.3** and **3.1**). This approach is considered appropriate given that the results show that the ocean tide condition dominates the maximum modelled flood elevation at the site. An additional analysis of the various storm/tide combinations used in the envelope approach is therefore considered to be unwarranted.

Figure 3.2 shows the extents of the maximum modelled flood (that is, 5% AEP storm event combined with 1% AEP ocean tide condition) with static tailwater conditions of 3.05 mAHD and 3.55 mAHD in line with sea level rise scenarios of 0.4 metres and 0.9 metres respectively, relative to 1990 mean sea levels for the existing topography (i.e. pre-development). For the 0.4 metre sea level rise scenario, the maximum modelled flood elevation at the creek junction is 3.08 mAHD, while for the 0.9 metre sea level rise scenario, the maximum modelled flood elevation is 3.58 mAHD. These results are summarised in **Table 3.2** below.

Table 3.2 Maximum Modelled Flood Elevations at the junction of Arrawarra Creek and Yarrawarra Creek – Sea Level Rise Scenarios

Scenario	Storm Event (AEP)	Modelled tailwater level (mAHD)	Maximum modelled elevation, developed site (mAHD)
Sea level rise 0.4 m over baseline static tailwater level	1%	3.05	3.08
Sea level rise 0.9 m over baseline static tailwater level	1%	3.55	3.58

Image Source: Google Earth, CNES/Astrium (2015)

0 100 200 400m
1:7500

Legend

- Proposed Subdivision of Arrawarra Caravan Park
- 1% AEP Flood Extent (1% AEP Catchment Flood and 5% AEP Ocean Water Level)
- 1% AEP Flood Extent (5% AEP Catchment Flood and 1% AEP Ocean Water Level)
- 2% AEP Flood Extent
- 5% AEP Flood Extent
- 10% AEP Flood Extent
- 20% AEP Flood Extent
- 50% AEP Flood Extent

FIGURE 3.1

1, 5, 10, 20 and 100% AEP Flood Extents, Existing Landform

Image Source: Google Earth, CNES/Astrium (2015)

0 100 200 400m
1:7500

Legend

- Proposed Subdivision of Arrawarra Caravan Park
- 1% AEP Flood Extent (1% AEP Catchment Flood and 5% AEP Ocean Water Level)
- 1% AEP Flood Extent (5% AEP Catchment Flood and 1% AEP Ocean Water Level)
- 2% AEP Flood Extent
- 5% AEP Flood Extent
- 10% AEP Flood Extent
- 20% AEP Flood Extent
- 50% AEP Flood Extent

FIGURE 3.2

1, 5, 10, 20 and 100% AEP Flood Extents, Existing Landform with Sea Wall Designed

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4.0 Stormwater Strategy and Assessments

4.1 Introduction and objectives

The proposed Stormwater Strategy aims to meet Council's Water Sensitive Urban Design (WSUD) targets with regard to volumes and flows:

- Target 1 – Frequent flow management: capture and manage the first 10 millimetres of runoff from all impervious surfaces of the proposed development.
- Target 2 – Waterway stability management: limit the post-development peak 1 hour, 1 year ARI event discharge to the predevelopment peak 1 hour, 1 year ARI event discharge.
- Target 3 – Stormwater quality management: to ensure stormwater quality is considered in the planning of new developments.

The proposed Stormwater Strategy consists of lot-scale stormwater management (including pro-rata detention for public hardstand areas within the proposed development) in the form of rainwater detention tanks. It is proposed that rainwater tanks be installed on each property to capture at least 75% of runoff from the roof area. The potential benefits of the inclusion of rainwater tanks that are emptied by a 20 millimetre diameter outlet on the runoff response of the development area was investigated.

4.2 Rainwater tank sizing

Lot scale stormwater controls (rainwater tanks) would provide detention and storage for stormwater generated by the roof areas within in each lot and would provide pro-rata detention for other public hardstand areas (that is, roads and pavements). Based on the proposed site layout, pre- and post-development runoff scenarios based on average equivalent lot size were modelled using the rational method. Average equivalent lot size was calculated based on the average lot size and the total road reserve area. In assessing post-development flows, it was assumed that 75% of the roof area would drain to the rainwater tank.

The post-development scenario modelled was the one hour duration, one year ARI storm event in line with Council's Water Sensitive Urban Design Target 2. It was found that a standard size 3.3 kL tank would reduce post development flows to the pre-development scenario. The next greater standard size tank (5 kL) would reduce post-development flows to approximately 2% less than the pre-development scenario. Larger rainwater tanks would allow for both storage and detention. For example, a 10 kL rainwater tank could provide 5 kL stormwater detention and 5 kL for storage and re-use.

Table 4.1 shows the pre and post development peak discharges for the one hour, one year ARI event for the average equivalent lot.

Table 4.1 Pre- and Post-Development Peak Discharge Rates

Tank size (kL)	Pre-development peak discharge (m^3/s)	Post-development peak discharge (no tank) (m^3/s)	Post-development peak discharge (with tank) (m^3/s)
3.3	0.50	0.70	0.50
5.0	0.50	0.70	0.49

Other WSUD elements may be considered during future development applications that could further improve the runoff response from each lot. Such measures may include infiltration trenches, landscaped ponding areas and grass swales to receive runoff from the impervious areas such as the surplus roof area and rainwater tank overflow, driveway and paved areas. Lot-scale infiltration trenches could also increase the soil moisture storage across the lot area, which would in turn reduce the need for supplementary watering of plants and grasses and improve soil health. The consequent increases in infiltration and evapotranspiration losses from infiltration trenches would further reduce the runoff expected from each lot area and address water quality aspects of WSUD.

5.0 Conclusion

5.1 Previous Work

The Arrawarra Creek and Yarrawarra Creek catchments were initially modelled by Umwelt in 2003 in order to determine peak 1% AEP flood levels and flow velocities at the Arrawarra Beach Caravan Park. The modelling indicated that the site would be impacted by this flood event. The model was modified to accommodate filling of the site to 3.02 mAHD. Results of this modelling indicated that with fill in place, flooding would be restricted to the creek system, with no impacts predicted to occur to the upstream flood regime.

The initial model was inherently conservative in setting a finished floor level (FFL) determined by the 1% AEP flood level as it used a tailwater condition that exceeds the maximum observed water level at the closest tidal monitoring by station by over 50%. The modelled levels reflected potential inundation levels during extreme ocean surge events in combination with the 1% AEP flood event. In addition, the previous work did not include assessment of the impacts of climate change on the local flooding regime.

5.2 Updated Modelling and Assessment

The existing model was updated to incorporate tailwater effects based on the guideline *Floodplain Risk Management Guide – Modelling the Interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways* (NSW Office of Environment and Heritage, 2015). The model was also updated to include the sea wall designed to mitigate the impacts of erosion at the site. In addition to a baseline scenario, sea level rises of 0.4 metres and 0.9 metres were modelled.

The envelope approach described in NSW Office of Environment and Heritage (2015) was used to determine the design flood elevation at the site by modelling a variety of storm/ocean level conditions (refer to **Sections 2.3** and **3.1**). It was found that the combination of the 5% AEP storm event and the 1% AEP ocean tide condition resulted in the maximum modelled flood elevation of 2.65 mAHD at the site. This reflects the dominance of the ocean tide condition in dictating flood levels at the site.

Council's flood planning criteria stipulate that FFLs are to be at least 500 millimetres above the peak 1% AEP flood level. With baseline conditions (that is, no sea level rise), a FFL of 3.15 mAHD would meet Council's flood planning criteria. Part of the site lies below this level, particularly in the eastern area. This level is based on the 5% AEP storm event coinciding with the 1% AEP ocean tide level and as such has a probability of only 0.0005 of occurring in any one year.

The previous study (Umwelt, 2003), based on extreme ocean surge tailwater conditions, recommended filling the site to 3.02 mAHD and elevating floor levels to 3.52 mAHD. This study found that filling the site to 3.02 mAHD would have negligible impact on the upstream flooding regime (refer to **Section 5.1**).

Based on the updated modelling it is suggested that the FFL for the site should be no less than 3.15 mAHD. The updated modelling indicates that this level will provide 500 millimetres freeboard to floor levels with the design flood event. FFL's 500 millimetres above the 50 year and 100 year sea level rise scenarios would need to be no less than 3.58 mAHD and 4.08 mAHD respectively.

5.3 Stormwater Strategy

This report also proposes a Stormwater Strategy with the objective of meeting Council's Water Sensitive Urban Design (WSUD) targets for volumes and flows. The proposed Strategy consists of lot-scale detention via rainwater tanks of minimum 3.3 kL capacity and capturing runoff from 75% of total roof area. Lot scale modelling indicated that this configuration would reduce post-development flows to the pre-development scenario.

6.0 References

BMT WBM Pty Ltd, 2011. *Coffs Coast Coastal Processes and Hazards Definition Study Volume 1.*

NSW Office of Environment and Heritage (OEH), 2010. *Coastal Risk Management Guide for Incorporating Sea Level Rise benchmarks in Coastal Risk Assessments*

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Umwelt (Australia) Pty Limited, 2003. *1 in 100 Year Flood Study - Arrawarra Beach Caravan Park*.

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