Appendix A Coastal Processes Study

Whitehead & Associates
Environmental Consultants
197 Main Road Cardiff NSW 2285 Australia
Telephone +61 2 4954 4996 Facsimile +61 2 4954 4996
Email mail@whiteheadenvironmental.com.au

Sandy Point/Conroy Park Coastal Process Study

Prepared for Port Stephens Council

Prepared by Dr David Wainwright, Doug Lord and Benjamin Crawley
Whitehead & Associates Environmental Consultants Pty Ltd
197 Main Road
CARDIFF
NSW 2285

Telephone: 02 4954 4996
Fax: 02 4954 4996
email: davidwainwright@whiteheadenvironmental.com.au

Document Control Sheet

Document and Project Details	
Document Title:	Sandy Point/Conroy Park Coastal Process Study
Author:	David Wainwright (W&A) / Doug Lord (CE)
Project Manager:	David Wainwright
Date of Issue:	28/01/2016
Job Reference:	Report_1415_001_Final.docx
Synopsis:	This report contains a coastal processes study undertaken to inform the development of conceptual management options for the Sandy Point to Conroy Park at Corlette along the southern shoreline of Port Stephens.
Client Details	
Client:	Port Stephens Council
Primary Contact:	Philippa Hill

Document Distribution

Version Number	Date	Status	DISTRIBUTION – NUMBER OF COPIES (p – print copy; e – electronic copy)		
			Client	Other	Other
00	15/07/2015	FIRST DRAFT	1e	-	-
01	02/09/2015	FINAL DRAFT	1e		
02	28/01/2016	FINAL	1e		

Document Verification

Checked by:	***Issued by:***
Doug Lord	David Wainwright

Disclaimer

The information contained in this report is based on independent research undertaken by David Wainwright of Whitehead & Associates Environmental Consultants Pty Ltd (W&A) and Doug Lord of Coastal Environment Pty. Ltd. To our knowledge, it does not contain any false, misleading or incomplete information. Recommendations are based on an appraisal of the site conditions subject to the limited scope and resources available for this project, and follow relevant industry standards. The conclusions made in this report are based on the information gained and the assumptions as outlined. The coastal environment is inherently variable, and the conclusions in this report have a limited lifespan.

Copyright Note

Table of Contents

List of Figures

List of Tables

1 Introduction

This report presents the findings of a coastal processes assessment that was undertaken as part of the Sandy Point / Conroy Park Foreshore Erosion and Drainage Management Plan. It forms Appendix A of the main study report. For a more detailed description of the site locality and purpose of the overall study, readers are referred to the Introduction of that report.

The primary purpose of this assessment is to provide background information on coastal processes, so that informed decisions can be made when designing and evaluating management strategies for the foreshore extending across Sandy Point, westwards to the Anchorage at Corlette, along the southern foreshore of Port Stephens.

The key aims of the Coastal Processes study were to:

1. Identify long term morphology at the site;
2. Calculate longshore transport rates;
3. Determine design water levels and tidal variation at the site;
4. Determine appropriate design current velocities; and
5. Determine nearshore wave conditions for design.

The Coastal Processes Study includes the following:

- Chapter 2: A detailed examination of available background reports;
- Chapter 3: An assessment of existing foreshore structures;
- Chapter 4: Review and analysis of available hydrosurvey and aerial photography;
- Chapter 5: Presentation of a Numerical Model of the Port;
- Chapter 6: Discussion of Design Conditions
- Chapter 7: Summary of Report Findings with reference to the key objectives

2 Background Information

2.1 Introduction

Numerous background studies were sourced and reviewed to determine the baseline understanding of the site. These included:

- *Port Stephens Marina, Corlette. Coastal Processes* (Geomarine Pty. Ltd., 1988)*;*
- *The Anchorage, Corlette, Port Stephens. Environmental Impact Statement* (Gutteridge, Haskins and Davey, 1989);
- A Natural Flushing System for Artificial Harbours; a Case Study of The Anchorage, Port Stephens, Corlette, N.S.W (Nielsen and McCowan, 1994);
- *Port Stephens Flood Study - Stage 2. Design Water Levels and Wave Climate ;*
- *Port Stephens Flood Study - Stage 3. Foreshore Flooding* (Manly Hydraulics Laboratory, 1998)*;*
- *Port Stephens / Myall Lakes Estuary Processes Study* (Manly Hydraulics Laboratory, 1999)*;*
- *Port Stephens and Myall Lakes Estuary Management Plan* (Umwelt (Australia) Pty. Limited, 2000)
- *Port Stephens Foreshore (Floodplain) Management Study* (Webb, McKeown & Associates Pty. Ltd, 2002a);
- *Port Stephens Foreshore (Floodplain) Management Plan* (Webb, McKeown & Associates Pty. Ltd, 2002b);
- *Living on the Edge. A Foreshore Management Plan for Port Stephens* (Umwelt, 2009)*;*
- *Port Stephens Design Flood Levels. Climate Change Review* (WMA Water, 2010);
- *Unique soft coral habitat in a temperate estuary: significance to biodiversity and marine park management* (Poulos, 2011)
- *Review of Studies on Estuarine Morphology and Sediment Movement Conducted in Port Stephens Estuary* (University of Sydney, n.d.)
- *Assessment and Decision Frameworks for Seawall Structures* (Coastal Environment, 2013);

The discussion provided in the remaining sections of this chapter is based upon our review of these documents.

2.2 Geomorphology

2.2.1 Broad Scale Geomorphology

Port Stephens can be broadly separated into two basins, to the east and west of Soldiers Point. The western basin is infilling with fluvial sediments from the Karuah River. In comparison, the eastern basin is affected by marine processes. The ocean entrance, which stretches 1.25 kilometres between Yacaaba Headland on the northern side and Mt Tomaree to the south enables the penetration of swell waves and ocean tides into the eastern basin.

Bathymetry in the eastern basin is dominated by the large flood tide delta, which is slowly moving westwards into the Port. At the present time, the leading edge of the flood tide delta stretches (approximately) along a north-south alignment between Corlette and Pindimar. The wave and climate environment of the study site is governed by the bed elevations, typically less than 10m, across the flood tide delta and the ongoing changes to its channels which convey tides in and out of the Port.

The present form of the Port has evolved during the past 1.8 million years, including the Ice age of the *Pleistocene Epoch*. Ice Ages comprise glacial periods (cooler earth, lower sea levels) and interglacial periods (warmer earth, higher sea levels). The present *Holocene Epoch* stretches from around 10,000 years b.p and contains the tail end of the last period of post glacial sea level rise, which ceased around 6,000 years ago.

The movement of the flood tide delta into Port Stephens is a continuation of ongoing change that was triggered by that rise in sea level, which started some 20,000 years ago, when sea levels were some 120 to 130m below their present level. With this rise, the shoreline gradually moved across the continental shelf, reworking sand westwards, and ultimately (when the ocean reached the present level) forming a series of sandy dune ridges which are evident both to the north and south of Port Stephens. The barriers comprise unconsolidated quartz sands, present as the “Stockton” soil group, comprising beach sands which were deposited to both infill the space between volcanic hills (such as Corlette Head) and fringe the flood tide delta along Corlette and Bagnalls Beaches. These are the same sands which stretch southwards from the study site towards the sand dunes of the Stockton Bight. The sands which fringe Port Stephens are being continually reworked by ongoing change within the flood tide delta, and the resulting modifications to waves and tides within the entrance to Port Stephens.

Within the flood tide delta, the sand is up to 20 to 25m thick, overlying the relict channel of the Karuah River, which used to flow, some 70 to 140m north of the study site, on its way eastwards to the ocean. With the sea level rising up until 6000 years ago, this channel was drowned by the ocean, leading to Port Stephens’ classification as a “drowned river valley” type estuary. Bedrock is shallower at the shorelines of the study area, given the proximity of volcanic hills behind Sandy Point and at Corlette Head. Bedrock dips from south to north.

Simplistically, the flood tide delta can be considered as comprising a relatively flat stoss ‘ramp’ side stretching from the entrance and into the Port, and a much steeper leeward face (or dropover’) where the delta meets the deeper waters of the estuarine basin (i.e. between Corlette and Pindimar). Frolich (2007) argued that, under the action of waves and tides, sediment is presently eroding from the ramp side and being carried over the dropover, lowering the ramp and causing related recession of the beaches which fringe the eastern basin. The estuary processes study for Port Stephens (Manly Hydraulics Laboratory, 1999) also noted that there has been an historical tendency for the recession of sandy shorelines in the Port.

While useful in a very broad sense, there are particular, location specific aspects that need to be considered when looking at implementing foreshore management options with an expected design life span of 25 years.

For example, Geomarine (1988), considered that destruction of “Myall Point” may have been of particular relevance to evolution of the Corlette shoreline. Myall Point was a long sand spit which formed along the eastern edge of the entrance to the Lower Myall River during the 1800's and was subsequently destroy by a severe coastal storm in the late 1920's. Geomarine raised the possibility that the shoals which formed from the redistributed remains of that spit may have altered the patterns of swell wave focussing within the Eastern Basin, with energy particularly focussing on Sandy Point and causing its subsequent erosion. A more detailed examination of the changes in shoals over the last 50 years is presented in Frolich (2007) showing that shoals are continuing to evolve in a complex manner in near vicinity of Myall Point.

2.2.2 Impacts in the Study Area

Corlette Beach, to the west of Sandy Point is around 750m long, stretching between Sandy Point and Corlette Head. Sand movement along Corlette Beach, and within the study area, is overwhelmingly dominated by east to west sand transport. This occurs as a result of tides in the deeper channels, and the impact of refracted oceanic swell waves against the shoreline. Sand which is transported from east to west is ultimately carried over the flood tide delta dropover, settling out in the deep estuarine basin which continues to infill with marine sands. Geomarine (1988) highlighted that there exist no processes to resuspend this sand once it has been carried over the dropover. Temporary reversals of the sand drift direction along Corlette Beach will occur during period of strong westerly winds, however net shoreline transport is dominated by westwards drift (University of Sydney, n.d.). While locally generated wind waves may reverse sand movement along the study foreshores from time to time, these waves do not contain the required energy to reactivate sand lost over the dropover.

Prior to the construction of the Anchorage Marina, Geomarine (1988) estimated that some 28,500m^3 of sand had accumulated on the beach adjacent to the shoreline fronting Corlette Head over 27 years, turning what was once a rocky foreshore into a sandy beach. By examining historical aerial photographs, Geomarine considered that some of this this sand (~6,900) had eroded from the eastern end of Corlette Beach and from a large sand lobe which had previously formed offshore of Sandy Point. This lobe had gradually diminished in size over preceding decades. In other words, the sand which was offshore of Sandy Point in the 1950's had moved westwards, covering a previously rocky shoreline at Corlette Head by the late 1980's. Applying a multiplier of 3 to account for sand below the waterline, Geomarine estimated that an average 3,000m^3/yr of littoral transport, noting that it seemed to have slowed between 1977 and 1986. It was expected that this rate would slow to around 1,000m^3/yr with time.

Construction of the Anchorage Marina in the early 1990's has sheltered the western end of Corlette Beach from waves but, given the overall east to west transport direction, this is unlikely to have had any significant effect on erosion patterns along this Beach. To the east of Sandy Point, Bagnalls Beach is also subject to east to west littoral transport. However, by virtue of its location and alignment, it is less exposed to the penetration of oceanic swell.

Erosion in the study area has been recognised as a problem ever since residential construction began along the foreshore, which was subdivided in 1945. Geomarine (1988) considered it likely that groynes were constructed along the eastern side of Sandy Point following severe storms in July and October, 1959. Furthermore, significant erosion occurred at Sandy Point as a result of the 1974 'Sygna' storm (Webb, McKeown & Associates Pty. Ltd, 2002a).

There is a strong perception within the community that the erosion has accelerated since the 1950's with some considering it to be a result of the construction of marinas along the southern shorelines of the Port, including the Anchorage and d'Albora's Marina at Nelson Bay. However, the lack of reliable reports from before the 1950's makes it very difficult to provide an objective assessment of foreshore variability before this time. To attribute ongoing erosion to foreshore developments, or the destruction of Myall Point, or some other immediately definable and specific cause is likely to only tell a small part of the ongoing story of underlying changes to the flood tide delta. What is necessary for the present project is to recognise that there is a problem with erosion impacting on the foreshores of the study area, and that a resilient, adaptable design is required to provide the flexibility for future uncertainty.

Hydrosurvey and aerial photography, which are reviewed in Section 4 of this report, help to provide a picture of change since the 1950's. It is clear that the foreshore of Sandy Point, which was once "sandy", is now far less sandy and completely armoured by a variety of rock and concrete structures. The pattern of erosion has also progressed from east to west, beginning with structures along the eastern side of Sandy Point, progressing to more recently (last 10 years) additional constructed rock work along the western side of Sandy Point, and stretching to the even more recent (last few years) construction of a "temporary" geotextile sand bag structure fronting the eastern end of Conroy Park. That sand bag structure is now being outflanked by erosion on its western side, continuing the ongoing east to west progression of erosion and recently the addition of further bags to the wall. This pattern is entirely consistent with the well-recognised coastal engineering principle of "downdrift" erosion commensurate with a dominant east to west transport direction.

The east to west transport is also reflected by the behaviour of the present western end of Corlette Beach, adjacent to the Anchorage Marina. Construction of the marina breakwater has interrupted the east to west littoral transport, and sand has accumulated on the updrift (eastern) side of the Marina. This behaviour was predicted as part of the Anchorage Marina EIS (Geomarine Pty. Ltd., 1988), with an estimate of 3,000m^3/yr accumulation provided. Subsequent conditions of consent placed on the development required that a beach nourishment operation would be implemented …

> *"whenever the high water mark against the eastern wall progrades 60m seaward of its present location, or significant subaerial bypassing of the eastern breakwater under waves and current action occurs"*

… and that the sand would be moved to a location along the southern shores of Port Stephens, as directed by Council. The relocation of sand has an important practical purpose, to prevent two stormwater outlets adjacent to, and through the breakwater from being buried by sand. During a site inspection in May, 2015, the study team noted that both of these stormwater outlets were non-functional, due to the build-up of sand. In 2007, Short reported that the beach had accreted some 50m seaward of the original location (Short, 2007).

The subsequent foreshore management plan for Port Stephens also recommended that the accumulated sand could be used to nourish the beach at the eastern end of Corlette Beach to address erosion, highlighting that the earthmoving operation would be relatively simple. Such remedial work would need to be repeated from time to time.

In addition to changes to the immediate foreshore resulting from construction, the leading edge of the prograding flood tide delta was dredged (~170,000m^3) to fill the marina area and to provide a founding bench for the rock breakwaters and thus eliminate the need for excessive and expensive rock work. In addition, some nearshore dredging (~30,000m^3) was required to deepen the Marina area adjacent to the foreshore.

Evidence for the westward movement of sediment offshore of the site was also examined by Geomarine (1988), considering a long record of depths recorded at a nearby sewage outfall which has now been decommissioned. By observing the changes in depth with time, it was calculated that bedforms were moving eastwards at one wavelength (40m) every four years. A sand transport rate of up to 10m^3/m/yr was estimated in the deepest (~ 7-8m) part of the tidal channel offshore of the site. A commensurate growth of the dropover at 0.5 to 1.0m/yr towards the west was also estimated. In the tidal channels, sediments are coarse and very well sorted, reflecting a high energy sediment transporting environment. Closer to shore, sediments are fine to medium grained. Along with the presence of healthy seagrasses, this indicates that current driven transport in the nearshore area is probably limited to the immediate face of the foreshore. At this location, waves impacting the shoreline act to (i) stir up sediments; and (ii) drive a longshore current which transports those suspended sediments. We note that swell waves tend to approach the most severely eroding section of Corlette Beach at an angle of around 45 degrees, which is an optimal condition for beach sediment transport.

2.3 Waves

2.3.1 Swell Waves

Waves are probably the most important physical process affecting the shoreline within the study area. The wave environment includes two key components:

- Ocean swell entering Port Stephens and refracted to impact on the study shoreline;
- Locally generated wind waves, with the largest waves coming from the north-west.

Swell wave heights of over 3.0m can be expected in the immediate entrance of Port Stephens, but are generally less than 0.5m inside the Port (Webb, McKeown & Associates Pty. Ltd, 2002a). That study also indicated that maximum swells and seas of around 1m each could reach Corlette Beach.

Geomarine (1988) noted that swell waves are responsible for most of the movement of sand along the Corlette Beach shoreline, and estimated that swell wave heights along the foreshore could be as much as 10% of those measured offshore, but also indicated that, because of wave direction effects, 10% was at the upper end of the likely range. The direction of swells arriving at the shoreline is considered to be almost constant, as these long period waves adjust towards the alignment of the shoreline as they propagate from the entrance across the flood tide delta. At Bagnalls Beach and along the eastern side of Sandy Point, the swell waves are almost parallel to the present shoreline alignment, which is not conducive to the generation of a longshore current. However, at the eastern end of Conroy Park, swells presently approach from an angle of 45 degrees which is the most efficient direction for generating longshore drift.

Within the Port Stephens Flood Study Stage 2 report (Manly Hydraulics Laboratory, 1997) it was assumed that waves within the study area were approximately 0.04 times the offshore wave heights. This factor was determined from 2 years of wave records at Nelson Bay, with the factor determined for Nelson Bay applied directly to the Bagnalls Beach / Sandy Point / Conroy Park Area. The resulting design waves are presented in Table 1.

Table 1 Swell Wave Climate in the Study Site (from Manly Hydraulics Laboratory, 1997)

Recurrence	Height (m)
Extreme	0.5
1% AEP	0.4
2% AEP	0.4
5% AEP	0.3

However, the patterns of refraction modelled during the present study and other evidence, including data from a storm in April, 2015, indicate that the degree of exposure along the eastern side of Sandy Point may be more pronounced than that at Nelson Bay.

The estuary processes study (Manly Hydraulics Laboratory, 1999) assumed that water depths and shoaling patterns will remain unaffected by a slow rise in mean sea level, arguing that shoal development would match the slow increase in mean sea level. In effect, this would mean that design swell waves won't change significantly as a result of future sea-level rise. This could be seen as non-conservative, particularly given some morphological evidence indicating that the flood tide delta is flattening with time (Frolich, 2007). A brief analysis undertaken by SMEC as background to the foreshore management study (Umwelt, 2009), adopted a design swell wave height of 2.6m for Sandy Point. However, subsequent discussions with the author of that report indicate that this was a simple adoption of the "depth limited" wave that could physically occur at the site. Given the modelling undertaken as part of this study, we consider that this wave height is an overestimate for design purposes.

2.3.2 Wind Waves

As part of the Port Stephens Flood Study (Manly Hydraulics Laboratory, 1997) wind generated wave heights were also estimated utilising a model based on methods outlined in the Shore Protection Manual (CERC, 1984). However, that report indicated that the CERC method requires "10-minute average maximum gust speeds", a term which seems self-contradictory. The CERC method actually specified averaged wind speeds. Furthermore, the extreme wind speeds presented from the Williamtown record seem abnormally high in the Port Stephens Flood Study, and it seems likely that gust wind speeds may have been erroneously applied.

Previous researchers (Geomarine Pty. Ltd., 1988; Manly Hydraulics Laboratory, 1997) have found that the Williamtown wind record is suitable for analysing wind conditions at Port Stephens. For this study the Williamtown record has been considered a reasonable proxy for conditions across Port Stephens.

2.4 Water Surface Elevations

2.4.1 Ocean Water Levels

The tidal elevations within Port Stephens are close to the tidal levels in the ocean. Accordingly ocean water levels tend to control the "still" water level within Port Stephens. Based on an analysis of historical water levels at Sydney, Stage 2 of the Port Stephens Flood Study (Manly Hydraulics Laboratory, 1997) presented the design offshore water levels reproduced in Table 2

Table 2 Design Offshore Water Levels for Sydney (Manly Hydraulics Laboratory, 1997)

AEP	Ocean Water Level (m AHD)
5%	1.43
2%	1.47
1%	1.50

2.4.2 Still Water Levels inside the Port

In addition to the ocean values, wind setup was modelled across the Port for the 100yr ARI for various starting water levels and wind durations. At the study site, wind setup was most pronounced for an easterly wind, with maximum values of 0.12 and 0.13 modelled at Sandy Point and Corlette Head, respectively, for a 2.5 hour duration wind and starting water level of 1.5m AHD. These values were added to derive the combined still water levels (Storm Tide + Flood Runoff + Wind Setup) reproduced in Table 3.

Table 3 Design Still Water Levels for the Study Site (in mAHD Manly Hydraulics Laboratory, 1997)

AEP	Sandy Point	Corlette Head
5%	1.58	1.60
2%	1.62	1.65
1%	1.67	1.69
Extreme	1.70	1.72

In applying these still water levels, Manly Hydraulics Laboratory advised that any subsequent flood planning level (FPL) should include an allowance for freeboard and wave breaking processes against the foreshore. Wave breaking and runup is discussed in Section 2.4.4.

For direct comparison, the corresponding design water level components used in the design of the Anchorage Marina incorporated:

- 1.0m AHD (maximum high tide)

- Storm Surge + 0.5m
- Local Wind Setup + 0.3m;

Resulting in a value of 1.8m AHD, where the difference between this elevation and those presented in Table 3 resulting from the higher estimate of wind set up, derived without the assistance of a numerical model, from the Anchorage EIS.

The estuary processes study argued that wave setup within Port Stephens is not significant. This is consistent with research that has been undertaken since the design of the Anchorage Marina (Dunn et al., 2000; Hanslow and Nielsen, 1992).

2.4.3 Impact of Climate Change

A gradual increase in mean sea level in the ocean will result in a similar increase to mean water level inside Port Stephens. Flood planning levels inside Port Stephens were adjusted by WMA Water to include the present Port Stephens Council allowances for sea-level rise (WMA Water, 2010). This incorporated an allowance of 40cm by 2050 and 90cm by 2100, above 1990 levels, directly added to the design still water levels. They reported the design still water levels for different recurrence interval events as replicated in Table 4.

Table 4 Design Still Water Levels including Sea-level Rise (WMA Water, 2010)[1].

Site	5% AEP (2050)	1% AEP (2050)	Extreme (2050)	5% AEP (2100)	1% AEP (2100)	Extreme (2100)
Sandy Point	2.0	2.1	2.1	2.5	2.6	2.6
Corlette Point	2.0	2.1	2.1	2.5	2.6	2.6

In summary, WMA Water recommended a Flood Planning Level (corresponding to a 1% AEP event) of 2.5m AHD throughout the Estuary, but increasing by 0.4 (to 2.9m AHD) by 2050, and by 0.9 (to 3.4m AHD) by 2100. These FPL's do not include an allowance for wave runup or freeboard.

2.4.4 Wave Runup

Waves impact and run-up the foreshore, and it is the elevation and volume of runup that will affect the design of foreshore structures. In the Anchorage Marina EIS (Geomarine Pty. Ltd., 1988), a design crest height of 2.7m AHD was specified for the breakwaters. The floodplain management study (Webb, McKeown & Associates Pty. Ltd, 2002a) adopted the underlying work of the previous flood studies, and presented design runup levels for sites around Port Stephens. These are replicated in Table 5 and it can be seen that the reported 1% runup level for Sandy Point is lower than the corresponding 5% level, which is counterintuitive.

[1] Rounded to nearest 0.1m in WMA report

Table 5 Design Runup Levels (Manly Hydraulics Laboratory, 1998; in m AHD, without Sea Level Rise)

Site	5% AEP	1% AEP	Extreme
Sandy Point	2.4	2.3	2.9[2]
Corlette Point	2.2	2.3	2.9[2]

Taking a closer look at the foreshore flooding document from which these figures are taken (Manly Hydraulics Laboratory, 1998), we note that the maximum of the following two options was adopted for each site:

- 1% and 5% AEP water level to be combined with the 1% and 5% AEP swell waves (from east to south-west quadrant) plus the 1yr ARI wind waves (from the east counter clockwise to the south west) to estimate 1% and 5% AEP foreshore flood levels; or
- 1yr ARI water level (1.26m) combined with the 1% and 5% AEP wind waves from the worst direction to estimate the 1% AEP and 5% AEP foreshore flood levels.

However, The MHL (1998) study recommends that detailed investigation is probably justified in the eastern basin to address the aspect of wave overtopping. Presently, standard design methods aim to control overtopping volumes, beyond setting crest elevations based on estimated run up levels (Pullen et al., 2007). The impacts are mainly restricted to immediate foreshore areas (~ within 50m of the waterline) however large overtopping volumes can cause a safety issue for the public. This safety issue needs to be appropriately considered in design and in particular where public access or development is close to the crest.

2.5 Currents

Available current and flow data is sparse. A tidal gauging on 29th-30th September 1993, captured a time series of discharge values along a line to the north of Soldiers Point, indicating a total tidal prism of around $110 \times 10^6 m^3$.

Tidal currents were measured by Geomarine (1988) in the vicinity of the (then proposed) Anchorage Marina and estimated that the maximum tidal velocity near the proposed entrance would be around 1.1m/s (depth averaged) or slightly higher after the harbour walls were constructed.

Geomarine also estimated nearshore wind driven currents along Corlette Beach, utilising the results of limited numerical modelling undertaken by PWD in 1987. The assumed relationship for wind driven currents was that the current velocity would be 1/20th of the wind velocity, at a distance 100m from shore. Tidal currents are much stronger than wind driven currents in the vicinity of the study area.

2.6 Ecology

The ecology of the study area was examined as part of the Anchorage Marina EIS (Gutteridge, Haskins and Davey, 1989). The ecological study considered two

[2] The foreshore is overtopped for the extreme events

nearshore areas within the study site, "Area 8" located to the east of the Marina, and "Area 9" to the east of Sandy Point.

At this time, the inshore area had three species of seagrass *Zostera Capricornii, Posidonia Australis* and *Halophila Ovalis.* The presence of well-established seagrasses in the nearshore indicates that sediment transport had ceased at the time of the survey. The seagrasses supported a considerable population of epiphytic algae and many species of invertebrates and fish.

These seagrass beds stretch eastwards along Bagnalls Beach along with patches of the soft coral *Dendronephthya australis* which is restricted to the southern shoreline of Port Stephens (Poulos, 2011). Poulos identified a patch of *D.australis* offshore of Sandy Point, in the vicinity of a steep section of bathymetry immediately to the north of Sandy Point. The soft coral depends on a habitat with strong currents and low wave energy to efficiently feed. However, its presence here also seems to indicate that there is some hard feature such as a rock outcrop or reef which fixes this steep bathymetry, enabling strong currents without carrying the sandy substrate away.

2.7 Practicalities, Planning Constraints, Potential Solutions and Community Aspects

While the immediate foreshore of the study area was subdivided in 1945, the local area surrounding Corlette tripled in population between 1986 and 1996. The Foreshore Management Plan for Port Stephens identified that Conroy Park has potential to be utilised to a much greater extent, particularly for boat based activities, if suitably rehabilitated.

While the study area is within the "General Use Zone" it is still within the bounds of the Port Stephens Marine Park. The area below Mean High Water Mark is owned by the Crown and any works undertaken at the foreshore would likely require land owners consent under the *Crown Lands Act,* 1989.

A number of documents have taken aim at the state of the foreshore surrounding Sandy Point. Problems raised include:

- The seawalls and groynes have not been constructed in accordance with sound coastal engineering principles;
- Armour sizes are inadequate;
- The discontinuous state of the structures leads to concentrations of wave energy and, potentially, unravelling of the structures;
- Vertical sections act to reflect wave energy and induce nearshore scour with the potential for undermining and collapse;
- Access along the surface of the seawall is uneven and dangerous due to the varied types of structures present;
- The height of the seawall around Sandy Point is such that a safety rail would be required;
- Groynes are not large enough to be effective and their impact may actually be detrimental; and
- Placement of rock has been haphazard and is unsightly

The foreshore management plan (Umwelt, 2012) is particularly emphatic on this point stating

> *"Urgent attention is required to rehabilitate the erosion protection works at Sandy Point. This foreshore is used regularly by the public for walking exercise and it would appear that, given the dilapidated nature of the structures and the haphazard construction of the footpath, with uneven surfaces and no guard rails, there is a serious accident waiting to happen there."*

Residents do like rock or concrete protection along their foreshore. While other options such as offshore breakwaters, revegetation, dune reconstruction and beach nourishment could be considered, these are less likely to be acceptable to the community.

Recommendations provided by others in previous documents, and relating to the rehabilitation of the seawalls include:

- Converting vertical seawalls to a sloped revetment of 30 degrees (2H:1V) and don't allow new structures steeper than this to be constructed;
- Remove the Groynes and, potentially recycle this rock for reconstruction of the foreshore revetments;
- Survey the nearshore area to determine levels;
- Rehabilitate the eastern end of Corlette Beach through the construction of a suitable revetment buried in sand sourced from adjacent to the Marina Breakwater; and
- Ensure that crest levels prevent significant overtopping.

There are issues associated with the large scale removal of unauthorised structures including (Umwelt (Australia) Pty. Limited, 2000)

- The costs involved in demolition and reconstruction of a natural foreshore profile;
- Issues associated with identifying the authority responsible for funding and undertaking the work; and
- Objections from individual landowners that see removal as placing their property at risk, particularly when entire foreshore lengths would need to be reconstructed for the works to be effective.

3 Assessment of Foreshore Structures

3.1 Foreshore Structure Inspection and Database

The shoreline of the study area extends approximately 1.1 km from the eastern wall of the Anchorage to the western end of Bagnalls Beach. In the following discussion, chainages are measured in distance east of the Anchorage Breakwater. The western 400m from the centre of Conroy Park to The Anchorage is unprotected and comprises a flat beach and nearshore backed by a low sandy dunes or an erosion escarpment exposing the pre-existing back beach sediments. The western most 250 metres from just east of the stormwater outlet to the Anchorage breakwater has, since the harbour construction, accreted by approximately 60 metres seaward against the wall. A broad, flat dune and beach has built up as the sand moving alongshore from east to west is trapped against the eastern breakwater.

From chainage 250m to chainage 470m (the eastern end of Conroy Park), the beach is realigning and continuing to recede. This has resulted in the loss of mature coral trees and some significant eucalypts along the seaward margin of the park. From about chainage 380m to 470m protection of the eroded bank has been recently undertaken by PSC using geotextile containers, the most recent of these placed in July 2015, in accordance with the NSW Government guidelines for emergency protection works. This work was constructed in an attempt to limit the foreshore recession and overtopping during storms and to protect the remaining significant vegetation in the reserve. Overtopping and erosion at the western end of the first section of this geotextile revetment resulted in scour and undermining and loss of a large eucalypt in the April 2015 storms. The geotextile revetment has been subsequently extended further to the west to try and retain some Coral Trees.

The assessment of the seawalls fronting the properties between Conroy Park and the stormwater drain at the western end of Bagnalls Beach were undertaken over two days in May 2015. The inspections were undertaken by qualified and experienced coastal engineering staff from Whitehead & Associates and Coastal Environment Pty Ltd and utilised the reporting procedures suggested for inspection of seawalls in SCCG 2013 (Appendix B, page 26, “Seawall Preliminary Assessment Form”). The inspections were visual only and included no subsurface investigation or material testing to determine material sizes and composition. In most instances the toe level of existing works was not readily visible or discernible.

For all locations, no design information was available although some residents indicated sections of the walls were constructed based on “engineering advice”. Similarly, for Council constructed sections, no detailed information was available on construction dates, material quantities or concept designs. The details of the wall that could be ascertained were recorded on individual record sheets on a property by property basis and the visible seawall photographed. This more detailed information has been provided separately (digital format) to Council for inclusion within their asset management system.

The following general observations relating to the constructed protection works are relevant:

- The first protection structures around Sandy Point were initially installed in the 1950s and 1960s to protect against perceived recession of the shoreline at that time;
- Construction has continued and been extended along the beach until the present;
- The orientation of the shoreline protected varies through 45^0 with consequent variation in the wave exposure of sections of the foreshore experienced during storm events;
- The walls and groynes constructed are located outside the property boundaries on the crown reserve or the beach and seabed;
- Some walls were constructed by Council, while the majority were initially constructed by individual residents or groups of residents. Some resident constructed walls may have been topped up with rock supplied and placed by Council at a later date;
- Construction materials and techniques are varied and provide differing levels of protection to storm erosion and overtopping from property to property. Materials used include timber sleeper walls (earliest protection), tipped rubble walls (varying sizes and slopes), concrete cubes, mass concrete, brick and geotextile containers;
- Many of the wall sections are showing signs of progressive failure, including: slumping of the rubble walls to a more stable slope (undersize armour stone); loss of armour; and scour holes behind the crest from overtopping;
- Crest levels vary along the wall and at virtually all locations adjacent to Sandy Point, may be overtopped during significant storm events;
- Pedestrian access along the public reserves is varied and depends on the location and width of remaining reserve. The access path may comprise grass, paving, concrete or rubble. The path height and width varies and requires pedestrians to negotiate stairs and boat ramps at different locations; and
- Many properties have individual boat ramps and/or stairs to the beach, constructed on the reserve outside of the property boundaries;

The construction of the existing walls and in particular the lack of design details, the absence of an appropriate toe to limit scour and undermining and the complete absence of any filter layer underlying the armour units, means that it is not possible to certify the adequacy of any section of the existing ad-hoc seawalls as meeting appropriate design standards. While some sections of the revetment are more substantial than others, are exposed to lower wave conditions or are located well seaward of existing residential development, the majority of the protection works are showing signs of failure during storms. Higher crest levels are required and, at present, the protection works are susceptible to scour and piping failures from wave overtopping. The associated dangers of the seawalls, boat ramps, access stairs and pedestrian thoroughfare raises questions of public liability both resulting from storm damage and accident.

There are four key areas of concern in relation to protection of the foreshore at the current location

- Erosion/recession of the shoreline. This has been evident since the mid-20th century when residents along the Sandy Point foreshore commenced construction of protection works to maintain a permanent back beach area, providing a buffer between development and the beach. These works have effectively held the location of this shoreline at the cost of the sandy beach. With the beach experiencing recession and alongshore sand movement through the area, the erosion has translated downdrift and (to the west), over time necessitating extension along the shore of protection works. Without intervention and with a scenario of increasing sea level rise, this trend to foreshore recession will continue for the foreseeable future.
- Loss of the shallow nearshore and sandy beach areas as the nearshore beach profile continues to erode and the protection structures become more prominent. With continuing hardening of the foreshore extending from the western end of the study area, it is likely that the sandy beaches will continue to disappear. Those beaches will re-establish and/or be exposed less frequently over time, unless an integrated management strategy is adopted to address this problem. The existing rock groynes serve this purpose to a limited extent.
- The shoreline recession and loss of the reserve has resulted in a narrow buffer of public land between the private residential boundaries and the seawall crest at Sandy Point. This has compromised the public access along the foreshore and in some locations the access is limited to the crest of the seawall or hard apron, constructed on public land by residents and Council.
- As the beach disappears, the wave impacts are magnified with waves breaking onto what is in some locations a vertical seawall, with resulting, significant wave overtopping. The variable seawall crest levels are in the main too low for the current conditions and this is exacerbated by the construction of boat ramps at many locations along the foreshore on the public reserve. These low points reduce the revetment crest level at these locations and funnel water over the seawall, increasing the inundation of the public reserve and private property. This overtopping has in recent events (April 2015) resulted in scour and piping failures through the walls, loss of armour units from the seawall face and crest, and a risk to the public using the now compromised alongshore access path. In the absence of upgrading of the seawall and with the scenario of increasing ocean levels this situation will continue to deteriorate. More frequent and more severe storm inundation from the Port will occur over time.

3.2 Study Area "Precincts"

For ease of discussion the study area has been divided into six separate precincts on the basis of orientation, exposure to coastal processes and the nature of existing protection works. This facilitates a general discussion of the nature of the hazards, the effectiveness of existing protection works and possible future management strategies. The extent of all six precincts is illustrated in Figure 1.

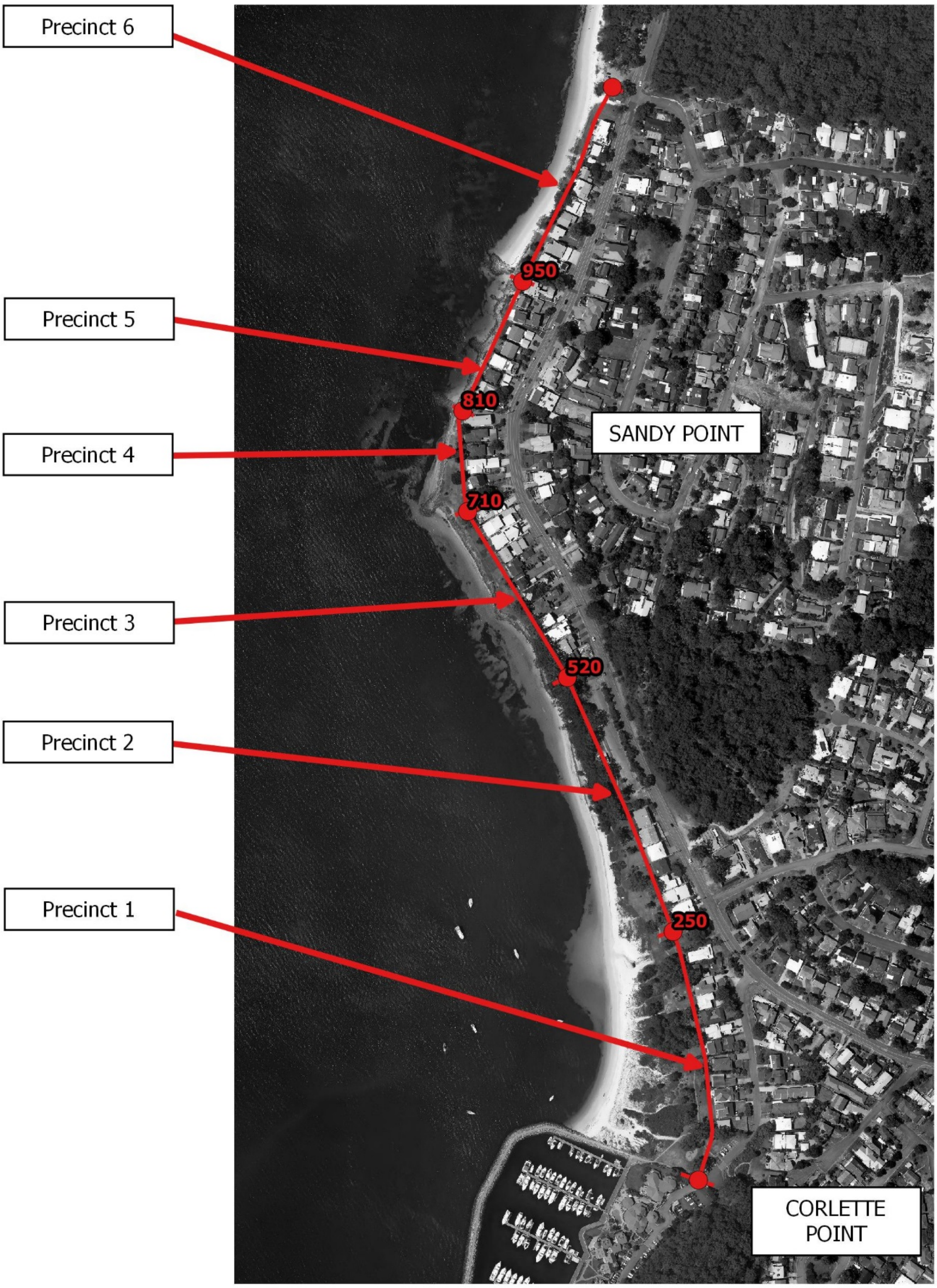


Figure 1: Study Area Precincts

Sandy Point / Conroy Park Coastal Processes Study

Whitehead & Associates
Environmental Consultants

0 100 200 300 m
(Approx Scale)

Revision	A
Drawn	DW
Approved	DW

3.2.1 Precinct One – The Anchorage eastern wall (chainage. 00m) to the east side of the Corlette Point Park stormwater outfall (chainage. 250m).

This section of the beach has accreted following the construction of the eastern harbour wall at the Anchorage in the early 1990s with the trapping of sand which moves naturally along the Sandy Point - Corlette shoreline from east to west under the influence of waves and currents. The accretion forms a triangular fillet with the maximum increase in beach width of approximately 60m against the eastern wall at the present time. The sand accretion is currently covering the stormwater outlets adjacent to the harbour wall which discharge stormwater from the development and the catchment immediately to the south. If allowed to accrete to the extent that it begins to bypasses the harbour wall, sand will then move westward around the harbour and over the face of the flood tide shoal into deeper water off Corlette Head. It is then effectively lost from the beach system.

The amount of accretion over the past two decades decreases with distance east. At the stormwater outlet across Corlette Point Park the accretions is now well seaward of the constructed headwall at the back of the beach. The water discharges across the beach, scouring a narrow channel following rainfall. The headwall is currently around 25m from the high water mark. The sand build-up decreases further to around 0m approximately 75m east of the stormwater outlet (adjacent to #78 Sandy Point Road and the erosion of the foreshore and realignment of the beach dominate from that location to the east.

No foreshore protection works are required to maintain development and crown land within this precinct. The major issues relate to the stormwater drainage outlets (3 of) which are affected by sedimentation. The sand build up also provides an opportunity to source sand on a regular basis which may be transferred to other locations along the southern Port Stephens foreshores east of this location. The original design and approval of for The Anchorage predicted this sand accretion and beach realignment and envisaged the relocation of this sand to address the possible impacts on the stormwater system and to prevent the “loss” of a valuable sand reserve from the active beach system.

Figure 2 Western end of Precinct 1. Sand accretion against the Anchorage marina wall is burying stormwater outlets. *Photo Source:* D Lord, 8th May 2015

Figure 3 Eastern end of Precinct 1. Sand accretion extends to the west of the stormwater drain across Corlette Point Park. *Photo Source:* D Lord, 8th May 2015

3.2.2 Precinct Two - East of the stormwater outfall (chainage. 250) to the western end of rock protection works at the eastern end of Conroy Park (chainage. 520m)

The foreshore between chainage 250 and 470 at the Western end of Conroy Park has remained largely unprotected and fronts the public reserve, providing a sandy beach amenity along the entire length. In recent years the erosion of this foreshore has increased with the high water mark at the base of the escarpment and no usable beach width at high tides. No residential assets are immediately at risk with all development west of Conroy Park set well back from the escarpment. The closest dwelling west of Conroy Park is more than 30m landward of the escarpment crest.

Recently, the major concern has been the erosion of vegetation through Conroy Park with the loss of eucalypts along the shoreline and coral trees which are valued for their summer shade in the reserve. Access to the Beach directly from the park has been comprised although pedestrian access from Conroy Park to Corlette Point Park along the waterfront reserve remains. Council has undertaken emergency foreshore protection through two campaigns, installing geotextile containers as permitted by the current NSW Government guidelines for emergency beach management. These have provided limited protection but do relocate the increased erosion to the western end of the completed wall section (end wall effects). This is evident at Conroy Park where erosion continued to the west of rock protection works prior to the installation of the geotextile bag protection. Again an increase in erosion was evident at the western, unprotected end of the geotextile bags.

Figure 4 Western end of Precinct 2 where accretion of the beach finishes east of the stormwater outlet and erosion of the back beach commences through to Conroy Park. *Photo Source:* D Lord, 8th May 2015

Figure 5 Looking west along precinct 2 from Conroy Park. A sandy beach remains at mid to low tides and is a popular walking area between Conroy Park and the Anchorage.
***Photo Source:* D Lord, 8th May 2015**

In the absence of management works, the erosion to the west of the protection works will continue, requiring extension of the protection to the west through this precinct. Ultimately this will affect access and reduce the sandy beach amenity. There will be a hard line, delineating the back beach (protected) area and the narrowing sandy beach. While residential assets are not at risk in this precinct, a higher priority is the retention of natural vegetation with easy access to a sandy foreshore for recreation. This may require some additional protection possibly coupled with some structural works to retain sand on the beach. Initial and ongoing artificial placement of sand either along the precinct foreshores or further to the east, may help to mimic the situation from previous decades, when there was a sand supply from the east and through the study area.

Figure 6 Precinct 2 eastern end. Geotextile containers have been employed for emergency beach protection along the eastern end of Conroy Park. Increased erosion west of the end of the protection is evident. Additional containers have been placed to extend the wall since this photo. There is little sandy beach width remaining at this location and no beach at mid to high tides. Access to the beach from the park has been affected.
***Photo Source:* D Lord, 8th May 2015**

3.2.3 Precinct Three – From the eastern end of the geo-container protection in Conroy Park (chainage. 520m) to the most western rock groyne (chainage. 710m)

This precinct includes the shoreline from the east end of Conroy Park (adjacent to #70 Sandy Point Road) to the rock groyne adjacent to the reserve immediately west of #46 Sandy point Road at the tip of Sandy Point. The shoreline within this precinct is relatively straight and faces NNE. It is more protected from ocean swells passing through the entrance to Port Stephens than the area further to the east, but has exposure to winter westerly wind waves. The foreshore is mostly protected by tipped rock walls which have not been designed and which vary in their current state of repair and effectiveness. The 12 dwellings behind this foreshore are set back between 13m (western end) and 25m (at #60 Sandy Point Road). The average setback is around 20m. The reserve seaward of the properties is wide and accessible with lawns and some planting maintained by the residents.

Figure 7 Precinct 3 eastern end Conroy Park. Rock protection has been placed along this section of the foreshore to the western most groyne. *Photo Source:* D Lord, 8th May 2015

Figure 8 Precinct 3 looking east to the western groyne. Various access stairs of differing design cross the revetment. The wall is generally steep, with some slumping and loss of armour near the crest, exposing the erodible bank behind. Armour stone size is variable. The western groyne and Port Stephens entrance are at the top of the photo. *Photo Source:* D Lord, 8th May 2015

Figure 9 Precinct 3 looking west from the western groyne to Conroy Park.
***Photo Source:* D Lord, 8th May 2015**

Slumping and some overtopping of these walls are evident. Residential development along this precinct is not immediately at risk. The sandy beach has effectively been replaced by a rock wall extending into the water at most stages of the tide. Patches of sandy beach may appear at lower tides and from time to time depending on weather conditions. Upgrading of the revetment along this section should consider the reinstatement of some sandy beach amenity as appropriate.

3.2.4 Precinct Four - From the western rock groyne (chainage. 710m) to the next rock groyne (chainage. 810m)

The coastal alignment changes through 45^0 at the tip of Sandy Point with the 90m shoreline of Precinct 4 facing ENE towards the entrance of the Port between the rock groyne at #46 Sandy Point Road and the next rock groyne at #38 Sandy Point Road. A small fillet of sand has accreted on the eastern side of this groyne.

The reserve seaward of the properties narrows from west to east with the set back of residences changing from 20m to 10m landward of the seawall crest. This section of the shoreline is vulnerable to wave attack and overtopping with the rock walls generally under designed and the beach all but eroded away. There is a small fillet of sand on the eastern side at the base of the western groyne, indicating the dominant east to west alongshore transport direction and the potential effectiveness of shore normal structures in maintaining a, small sandy beach area. Along the remainder of the precinct the sandy beach is only exposed on the lower tides, if at all.

While this shoreline is less exposed to the ocean swells than precinct five (to the east), there is a need to reconsider the effectiveness of the existing protection which is generally undersized and failing.

Figure 10 Precinct 4 looking east along the revetment face from the western groyne to the next groyne east. Rock armour sizes along this section are variable with significant slumping and loss of armour near the crest, exposing the erodible bank behind. The accreted sand fillet is visible in the lower left hand corner of the photo
***Photo Source:* D Lord, 8th May 2015**

Figure 11 Eastern end of Precinct 4. A boat ramp is located west of the groyne. Rock armour has been grouted to form a smooth wall. The rock groyne has slumped and is too short and low to anchor the beach or provide significant protection. A concrete apron can be seen beyond the groyne in Precinct 5. *Photo Source:* D Lord, 8th May 2015

3.2.5 Precinct Five- From the second most western groyne (chainage. 810m) to the eastern most groyne (chainage. 950m)

Precinct 5 is the most exposed, at risk and vulnerable section of coastline with development close behind a variety of ad-hoc protection works. This precinct includes the foreshore between the second most westerly groyne and the eastern groyne, a length of approximately 150m. There are 9 residences along this precinct from #36 to #20 Sandy Point Road. There is a small but ineffective (height and length) groyne between #28 and #30 Sandy Point Road. The setback from the seaward face of the dwellings varies from about 5m to 12m with little width remaining of the original public reserve.

This limited setback has compromised the alongshore access which is integrated into the seawall crest across some properties and varies in height, width and construction. Following the April 2015 storms the access alongshore through this precinct was unusable in some locations, with damage to the path surface, scour holes and dislodged rocks resulting in sections being taped off to restrict access, pending repair. This section is the most at risk with regular damage to the revetment, and extensive overtopping of the varied protection structures. The effect of wave overtopping is notably exacerbated by boat ramps, with low crest levels and poor drainage from behind the wall also causing issues.

Figure 12 Western end of Precinct 5. A vertical concrete block wall has been constructed in the centre but this precinct is predominantly tipped rock. Little or no sandy beach remains, even at low tide. The public access through Precinct 5 is compromised and wave overtopping during storms is resulting in damage and inundation of property landward of the wall.
Photo Source: **D Lord, 8th May 2015**

Figure 13 Much of the rock wall along Precinct 5 comprises substantial sized stone at a flatter slope. This is the most exposed section of the study area receiving ocean swells entering the Port during storms. At the time of inspection damage to the wall including loss of armour from the face and crest was observed. Scour holes under slabs and through the wall from overtopping was evident at a number of locations. The alongshore access was closed (SES tapes) along several sections. *Photo Source:* D Lord, 8th May 2015

Figure 14 Eastern end of Precinct 5. Sections of the wall have undersized armour, grouted stones and various access stairs and ramps across them. Precinct 5 is the most exposed section of the foreshore and the protection provided is poor for the level of exposure.
***Photo Source:* D Wainwright, 12th May 2015**

3.2.6 Precinct Six – from the eastern most groyne (chainage. 950m) to the western stormwater overflow on Bagnalls Beach (chainage. 1160m)

Precinct six extends from the eastern most groyne adjacent to #20 Sandy Point Road to the western side of the stormwater outlet at the western end of Bagnalls Beach (adjacent to #2 Sandy Point Road). A stormwater line crosses a vacant drainage reserve between #18 and #20 Sandy Point Road. The line runs inside and along the spine of the rock groyne which now serves a double purpose, both providing protection to the Beach to the east and discharging stormwater offshore. The stormwater outlet at the east end of Precinct 6 is also connected to this stormwater line also, with that stormwater outlet conveying excess flows primarily during high rainfall events.

Precinct 6 is more sheltered than areas to the west but is still subject to wave erosion and overtopping as evidenced by the existence of substantial sections of revetment of varying design along the foreshore. The groyne is effectively retaining sand to the east and a sandy beach area fronts the seawall along this precinct under most conditions. Even so, the presence of boat ramps along this precinct allows waves to easily run up and inundate the area behind the foreshore.

Figure 15 Precinct 6. Located to the east of the eastern most groyne, this section just west of Bagnalls Beach is more sheltered than areas westward (Precinct 5) with generally lower protection structures and a build-up of sand on the eastern side of the groyne. The groyne was lengthened and upgraded to carry the stormwater outlet beyond the beach. The type and standard of back beach protection is variable. Overtopping is experienced right along this precinct although much of the development is set further back from the shoreline. *Photo Source:* D Wainwright, 12th May 2015

Figure 16 Precinct 6. Some larger rock has been placed along sections of this precinct and a sandy beach remains seaward of this protection work.
Photo Source: **D Wainwright, 12th May 2015**

3.3 Discussion of Protection Issues

The area west of Conroy Park to Corlette Point Park is fully developed with residential properties (houses and units) along Sandy Point Road. Similarly, the road frontage along Sandy Point Road and landward of the reserve along the foreshore east of Conroy Park to the stormwater drain at the western end of Bagnalls Beach is fully developed. Much of the development immediately adjacent to Sandy Point, and particularly to the east, is dependent on the protection provided by the ad-hoc seawalls constructed and bolstered since homes were constructed here. These structures are mostly under designed. The proximity and value of the residential properties make any option other than the continued protection of the foreshore at or around its present location seem unlikely.

However, the existing works are in need of substantial upgrading to bring the standard of protection to best practice, reduce overtopping and inundation during storm events and to formalise a consistent and accessible public access both along and to the shoreline.

The ownership of much of the shared infrastructure on the public reserve and the consequent responsibility for its maintenance and continued performance are at present unclear. Similarly, potential issues including damage to properties and dwellings, accident and injury to the public, and exacerbation of erosion or inundation on adjacent properties are all areas of continuing uncertainty.

Where properties are close to the seawall crest and the public reserve is narrowest (from the tip of Sandy Point to the east), there is a range of issues that will need to be considered in the design and alignment of any protection works proposed. This includes:

- The current practice of privately constructed boat ramps across the reserve and seawall with lower seawall crests. These compromise the integrity of any protection which can be provided but are highly valued by some property owners.
- The retention of the sandy beach at the base of the seawall. As the walls increase in size, the frequency and extent of the sandy beach area is decreasing. To the east of the eastern groyne, the wave energy is lower and the retained beach and access to that beach is highly valued. At other locations to the west around Sandy Point to Conroy Park, the sandy beach is mainly lost with a steep seawall to the waterline at high tide. At times and in some locations pockets of sandy beach do form, are exposed at low tides, and are valued. Enhancement of the sandy beach amenity could form part of the development of an adequate protection strategy.
- The potential removal and reconstruction of sections of seawall, built by residents at their expense, to bring them up to a current design standard. This may be resisted where residents have undertaken recent works or where they believe the existing protection works are adequate.
- The potential loss of individual access to the shoreline via constructed paths and stairways, often at each property and which do not conform to current design codes for access.

4 Analysis of Historical Aerial Photography and Hydrosurvey

4.1 Aerial Photography

Council and the Office of Environment and Heritage (OEH) provided W&A with multiple aerial photographs of the Corlette/Sandy Point region for our desktop investigation. The aerial photographs, spanning a period from 1953 to 2012, were provided in digital format. The scans were georeferenced and orthorectified for analysis in the geographical information software system “QGIS”.

Orthorectification involves removing the distortion effects of aerial photography from camera tilt and terrain effects. Orthorectification allows for features to be in their true position and allows for more accurate measurement of distances, angles and area. The accuracy of the two processes varied for each photograph due to resolution and a lack of land marks in the older historical photos to georeference to the 2012 satellite image. Despite these issues a reasonable degree of accuracy was achieved. The plan accuracy varied between 0-10 pixels which approximates to ±5.0 metres for the older aerial photographs. Accuracy within more recent aerial photography is within 2.0m,

Following georeferencing (bringing photographs into a common mapping coordinate system), the aerial photographs were used to map and analyse changes to the extent of the following features:

- Seagrass, noting that the limit of seagrasses will not generally grow within the intertidal range (i.e. above ~ -1.0m AHD). When clear and dense seagrass beds are present, the extent of seagrasses is a reasonable proxy for the -1.0m AHD contour;
- A foredune is present along the western end of Corlette Beach, where the dune system has been accreting since construction of the Anchorage Marina. A reasonable proxy for the seaward edge of the foredune is the presence of primary grasses or “light” vegetation. Similarly, the landward edge of the foredune could be interpreted by the presence of denser vegetation or a nominated contour level.
- The presence of “hard” structures has also been mapped from the aerial photographs. This information has helped to ascertain the timing and progression of construction along the shoreline.

The mapped seagrass, vegetation and structural extents along with the corresponding aerial photographs are presented in Appendix A. A description of each photograph is presented in Table 6. Care needs to be taken in interpreting the mapping undertaken here. In particular, the interpreted extents can be affected by the clarity of water, stage of the tide and pixel resolution and reflection off the water surface. In some cases, the resolution is not adequate for clearly locating any structures that may have been present.

Table 6 Aerial Photographs Considered in this Assessment

Year	Original Scale/Resolution	Quality	Useable
1951	1:33,000 Approx.	Beach clearly defined, seagrass not clearly defined	No
1952	1:31,000 Approx.	Beach clearly defined, seagrass not clearly defined	No
1959	1:16,000 Approx.	Beach clearly defined, seagrass not clearly defined	Yes
1963	1:34,000 Approx.	Seagrass, light and dense vegetation reasonably defined, structure extents poorly defined	Yes
1965	1:16,000 Approx.	Seagrass and light vegetation poorly defined, dense vegetation and structure extent reasonably defined.	Yes
1968	1:21,000 Approx.	Seagrass, dense vegetation and structure extent reasonably defined, light vegetation poorly defined.	Yes
1976	1:16,000 Approx.	Seagrass, dense vegetation and structure extent reasonably defined, light vegetation poorly defined.	Yes
1979	1:42,000 Approx.	Seagrass and light and dense vegetation reasonably defined, structure extent poorly defined.	Yes
1986	1:8000 Approx.	High reflection off ocean, seagrass not clearly defined, light and dense vegetation and structure extent well defined.	Yes
1992	1:16000 Approx.	Seagrass, vegetation and structure extents highly defined.	Yes
1996	1:8000 Approx.	Seagrass, vegetation and structure extents highly defined.	Yes
1998	1 pixel: 1m	Seagrass, vegetation and structure extents highly defined.	Yes

Year	Original Scale/Resolution	Quality	Useable
1999	1:8000 Approx.	Seagrass, vegetation and structure extents highly defined.	Yes
2003	1 pixel: 1m	Seagrass, vegetation and structure extents highly defined.	Yes
2005	1:10000 Approx.	Seagrass moderate to highly defined, vegetation and structure extents highly defined.	Yes
2007	1 pixel: 1m	Seagrass, vegetation and structure extents highly defined.	Yes
2012	1 pixel: 1m	Seagrass, vegetation and structure extents highly defined.	Yes

While all of the visible features have been mapped and presented in Appendix A, we have extracted key features from a few years to demonstrate the underlying trends of shoreline evolution since the 1960's.

Figure 17 presents the landward extent of seagrass beds from 1963, 1992 and 2012. The figure indicates that erosion of the eastern end of Corlette Beach, and Accretion of the western end has been ongoing since the 1960's. Of particular interest is the presence of a point about which the beach appears to have "pivoted" around 250m to the east of the present day Marina Breakwater. To the east of that point, Corlette Beach has eroded, and to the west, it has accreted. The key finding from this analysis is that this process has been occurring since *before the Anchorage Marina was constructed*. We do not believe that construction of the Marina has contributed in any significant way to the erosion of eastern end of Corlette Beach. Furthermore, the pattern of less sand being present in the nearshore zone continues around Sandy Point, and indicates that ongoing erosion is largely a function of less sand being available from an *updrift* direction (i.e. Bagnalls Beach or the flood tide shoal).

Figure 18 indicates the approximate dates at which different foreshore protection works have been constructed. Over time, the nature, configuration and alignment of different elements of the constructed works have changed, in response to storms or ongoing erosion. However, it is clear that there were minimal structures present during the 1950's, with the progressive construction of foreshore works occurring from the early 1960's onwards. It appears that settlement of the area during the late 1940's and 1950's, following subdivision in 1945 occurred when the beach was particularly wide as a result of a pulse of sand moving from east to west around Sandy Point. However, in the late 1950's through to the 1970's, coastal storms resulted in a dramatic re-alignment of the shoreline around Sandy Point, as sand which was present in large lobes offshore of the site were progressively moved in a westerly direction, leaving the shoreline relatively denuded of sand and with a comparatively narrower beach.

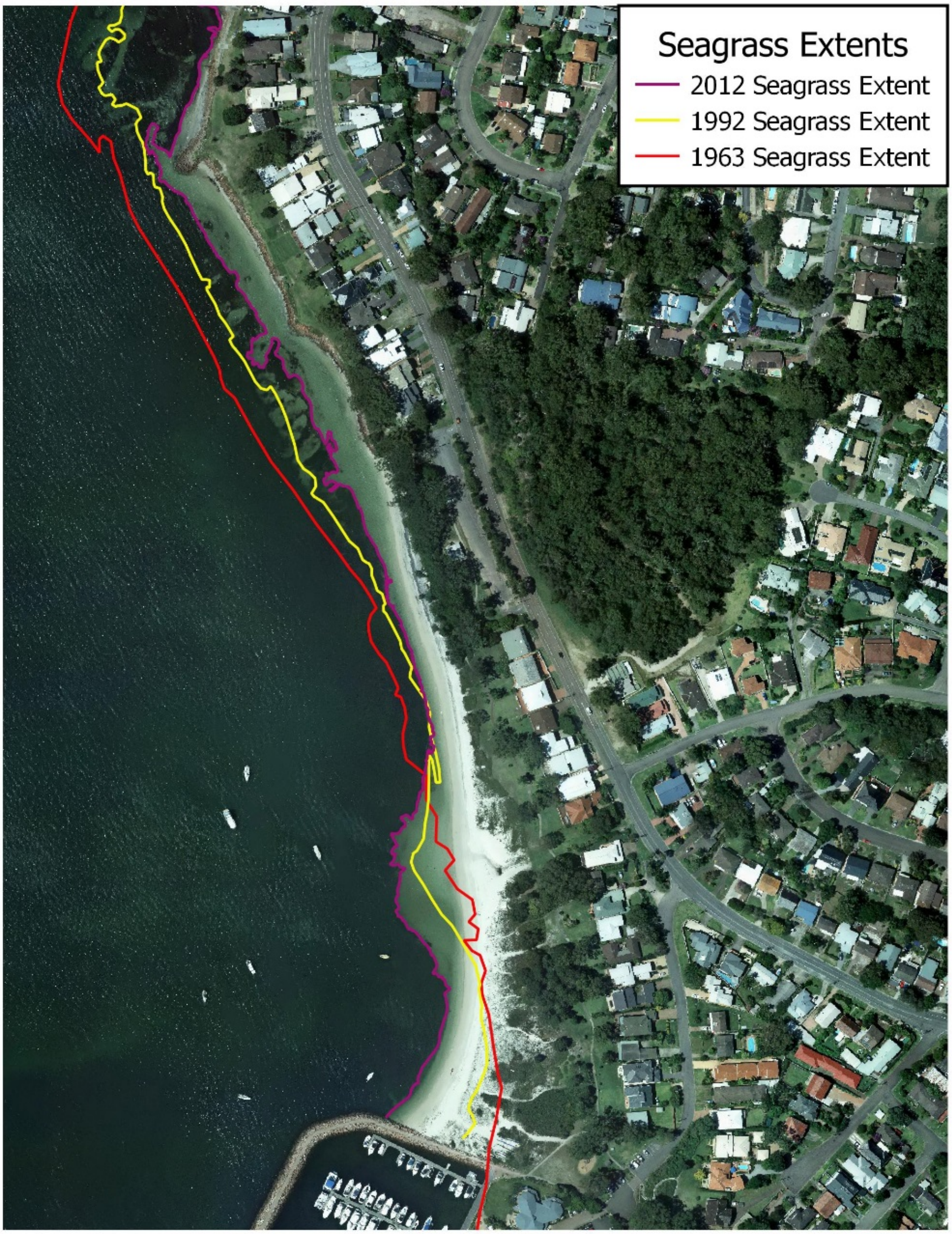


Figure 17: Movement of Landward Edge of Seagrass Beds Over Time

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

0 50 100 150 200 m

(Approx Scale)

N

Revision	A
Drawn	BC
Approved	DW

*From shoreline alignment appears structures were here from 1963, however they become more prominent with time.
**Progressive construction tended to be from east to west.

Figure 18: Approximate Construction Dates for Foreshore Structures

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

N

Whitehead & Associates
Environmental Consultants

0 50 100 150 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

As this shoreline adjustment took place, a variety of structures were built to address ongoing erosion. This began on the eastern side of Sandy Point, and progressed westwards. This pattern of foreshore protection is entirely consistent with an east to west longshore drift, noting that "edge effects" of coastal protection structures tend to result in increased erosion of a beach on the downdrift side of the structure. This east to west progression of the erosion continues today, with the most recent construction of sand filled geotextile revetments fronting Conroy Park (June 2015).

The evolution of an alternative proxy measurement for foreshore alignment is presented in Figure 19. In that figure, the "light" vegetation, which approximates the seaward location of the foredune along the western end of Corlette Beach, has been mapped for 1979, 1992 and 2012. Similarly to Figure 17, Figure 19 indicates that progradation of the Beach had begun in this location prior to the construction of the Anchorage. Similarly, there is an apparent point about which the beach has pivoted or "rotated" around 250m to the east of the Anchorage breakwater.

4.2 Hydrosurvey

4.2.1 Data Sources

Four hydrosurvey data sets were obtained and examined for this study, and these are described below.

1969 Hydrosurvey

The 1969 hydrosurvey was prepared by the NSW Department of Public Works and covers the area to the east of Soldiers Point. The final product comprises a set of 65 detailed sheets (44 outside the Port and 21 inside the Port). These are accompanied by two 1:12000 scale compilation sheets. The less detailed compilation sheets were the only ones available for the present study. The 1969 survey formed the basis for much of chart *AUS209*, published by the Australian Hydrographic Service. The contours on AUS209 were digitised and used as a basis for the 1969 Digital Elevation Model (DEM) developed during this study. To the west of Soldiers Point, contours on this chart date from 1920 (Admiralty Chart 1070).

2007 Hydrosurvey

The 2007 hydrographic survey dates from October and November of that year with the outputs comprising an index sheet and 12 detailed sheets covering the area from just outside the entrance to Port Stephens, westwards to include Salamander Bay. Survey lines were spaced at 50m typically, but relaxed to 100m to the west of Corlette Point. This survey is far more detailed than the 1969 survey. In generating the 2007 DEM, the point data file (x, y, z) coordinates were imported to GIS and these were used in place of the 1969 information where the more up to date information was available.

2011 Multi-beam Echo sounding

OEH undertook limited multi-beam echo sounding along the southern side of Port Stephens during 2011. This survey did not provide broad coverage of the Port, but is useful in that it provided a high resolution of soundings through the deepest parts of the southern tidal channel of the Port. From this survey, bedform patterns are discernible, and it is clear that sediment transport is to the west within this channel, consistent with the findings of previous researchers.

Figure 19: Change in Dune Vegetation Extent over Time

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

0 50 100 150 200 m
(Approx Scale)

N

Revision	A
Drawn	BC
Approved	DW

2015 Hydrosurvey (This Study)

As part of the present study, hydrosurvey was undertaken by McGlashan and Crisp Pty. Ltd., offshore of the study area. Shore normal transects, spaced at 20m were surveyed to distances of between 600 and 700 metres offshore. This survey was combined with a high accuracy UAV (drone) survey of ground elevations onshore at the study site to derive a continuous DEM of the transition between onshore and underwater elevations at the site. The resulting survey plans and CAD files were provided to Council as a deliverable for the overall project and will form a suitable basis for detailed design.

DEMs derived from all four data sources were generated using a multi-level B-spline interpolation algorithm from within the geographical information systems software environment QGIS. These models are presented in Appendix B.

4.2.2 Comparisons and Interpretation

Based on the 2015 hydrosurvey (Figure 20), bed elevations offshore of the site vary from 0 at the shoreline, down to elevations of around -10.0m AHD some 400 – 500 metres offshore of the site. The nearshore variation of elevation changes has a different character on the east and western sides of Sandy Point.

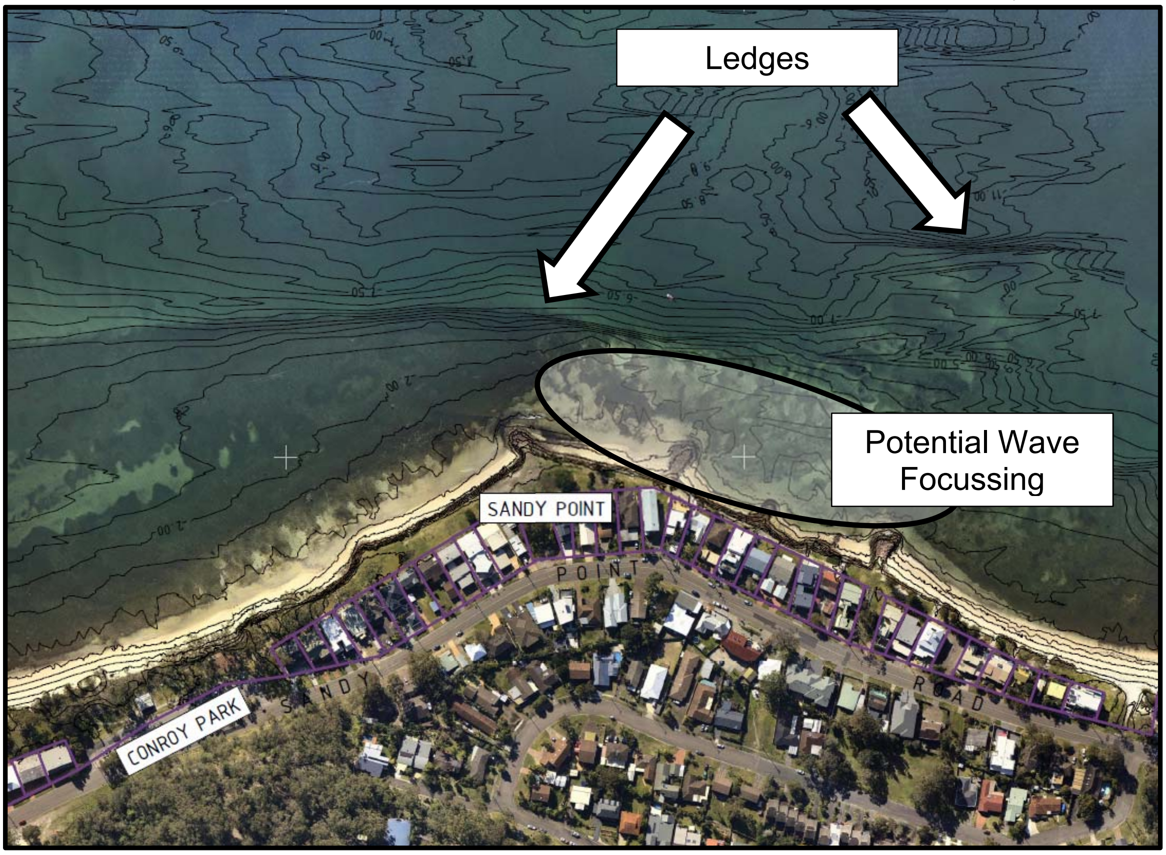


Figure 20 Extract from 2015 Survey Offshore of Sandy Point

Offshore from the eastern side of Sandy Point the bathymetry contains a number of ledges and drop overs which effect steep localised falls of 3-5 metres in the bathymetry. Based on the patterns of seagrass present in these areas, it appears likely that these features cause localised wave focussing and scouring of seagrasses to the east of Sandy Point. Conversely, to the west of Sandy Point, seagrass patches are denser and broader. This appears to result from Sandy Point providing a sheltering effect on the nearshore zone offshore from Conroy Park, by sheltering from wave energy, and by deflecting currents to ensure that the main channel flows, typically, more than 100m to the north of Corlette Beach.

Two comparisons between different dates of data were made. Firstly, a long term, broad scale comparison was made between 1969 and 2007. Noting that detailed sounding information was not available, and to avoid a second stage of interpolation, measured differences were calculated along the contour lines of the 1969 survey. A surface was then interpolated between those differences. The resulting difference map is provided as Figure 21, noting that areas of accretion are presented as positive values, and areas of erosion as negative values.

Figure 21 shows a few readily explainable features, such as dredging offshore of the Anchorage Marina where the spoil was used to fill the platform on which the Marina was constructed. Furthermore there are a number of areas where tidal channels have been actively depositing sand over the leading edge of the flood tide delta. Immediately offshore from Sandy Point, the analysis indicates an accumulation of sand, particularly around the point and western shorelines. This seems counter intuitive, given that most evidence points towards overall erosion of the beach in this area. However, beaches with higher energy tend to flatten out and it is possible that more sand could be present in the nearshore area, while the immediate foreshore is receding. Furthermore, the differences in detail between the 1969 and 2007 hydrosurveys could be causing some bias in interpolation throughout this area. Another confounding factor is the ongoing patterns of lobes of sand moving around Sandy Point during the second half of the 20th century. The exact configuration of the immediate foreshore shoals in 1969 is not clear from the survey plans. This apparent accretion is less than 1.0m and may just reflect the accuracy of the comparison available given the paucity of data in the earlier survey.

Nevertheless, the offshore pattern is governed by lowering of the tidal channels, and apparent accretion upon some shoaled areas. Offshore of Conroy Beach, the bed elevations have remained relatively stable. This is reflected by the healthy seagrasses that have flourished in this area. Slow, ongoing changes in the bed elevations offshore of the study site will have a governing impact on the foreshores over time. Any continued deepening or southward movement of the channel will increase the likelihood of erosion of the Sandy Point foreshores.

Secondly a more recent and localised analysis of elevation changes was made by subtracting the 2007 elevations model from the 2015 elevations model. The calculation was limited by the extent of the 2015 hydrosurvey. The resulting difference map is provided as Figure 22. Figure 22 indicates that any significant recent changes offshore of the study site show, almost uniformly, erosion. These include erosion adjacent to the foreshores of Sandy Point and Conroy Park, and further offshore, in the tidal channel. The pattern is consistent with the “lowering” of the ramp side of the flood tide delta, and transport of those sand westwards, likely depositing on the prograding face of the flood tide delta immediately to the north of Corlette Headland. We expect that the bed may continue to lower offshore of the study site in coming decades.

However, the present rate of lowering is unlikely to significantly affect the way in which currents and waves will impact the study shoreline over the 25 year design life for which foreshore protection measures are to be considered.

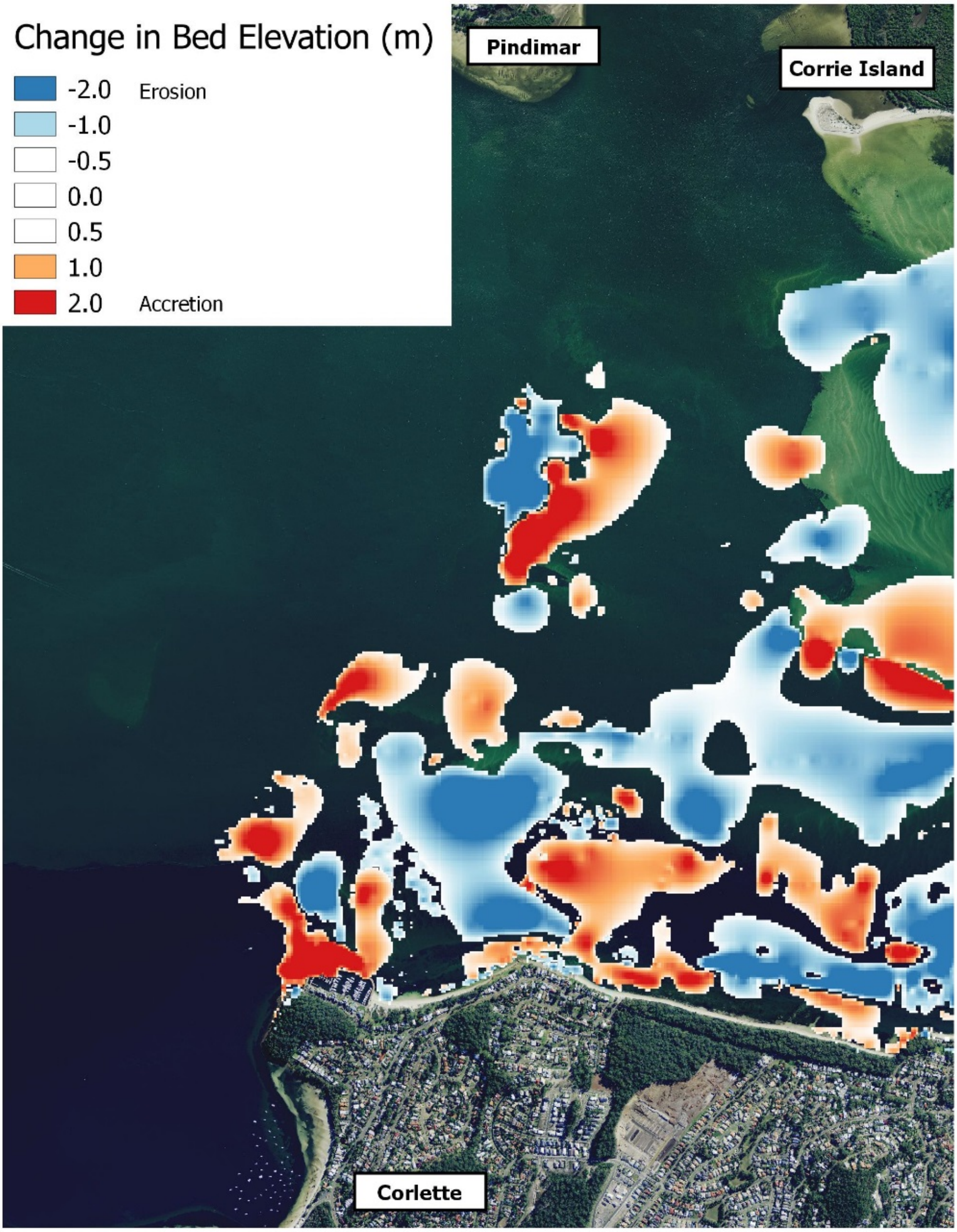


Figure 21: Change in Bed Elevations, 1969 to 2007

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

Revision	A
Drawn	BC
Approved	DW

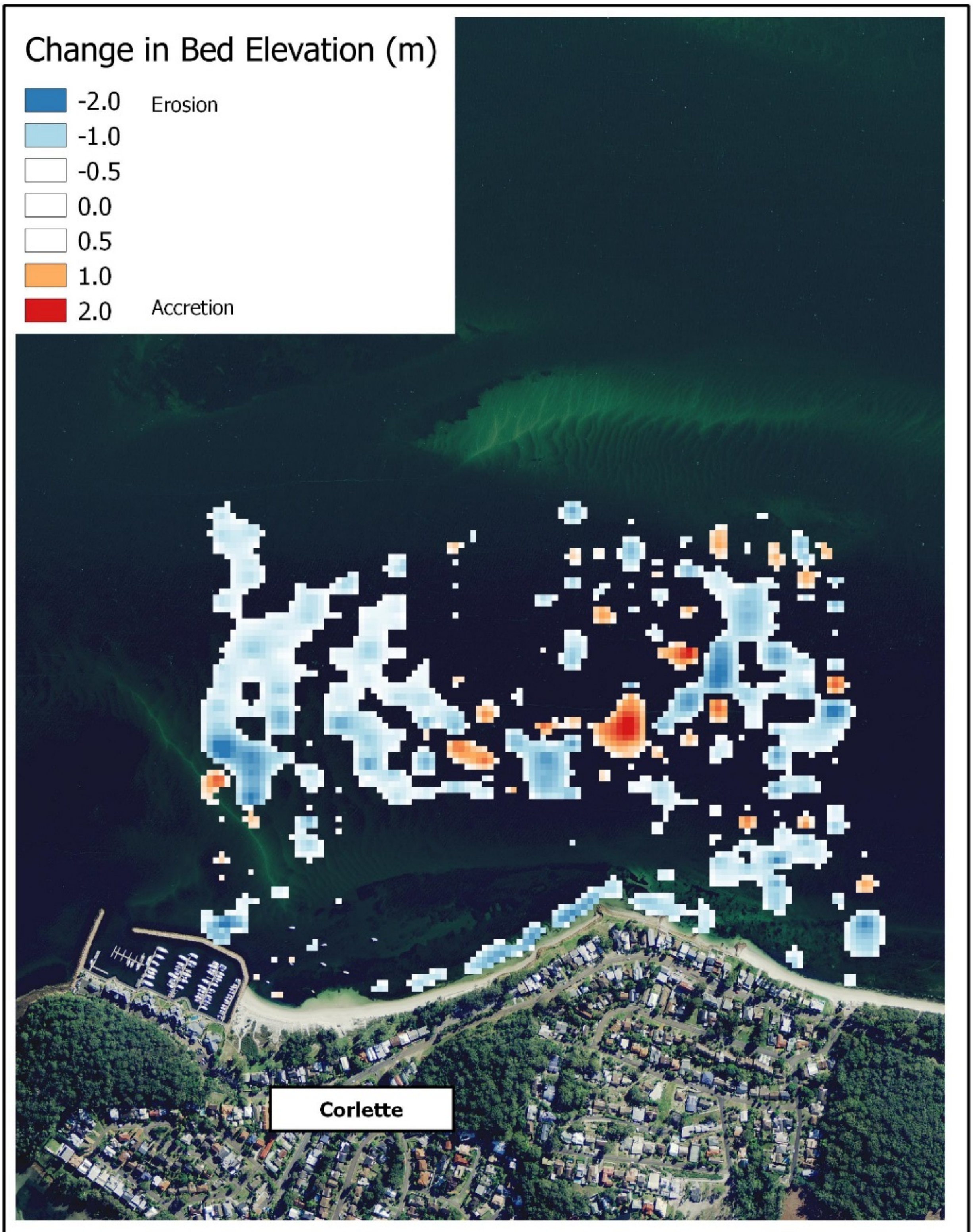


Figure 22: Change in Bed Elevations, 2007 to 2015

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

0 100 200 300 400 500 m
(Approx Scale)

N

Revision	A
Drawn	BC
Approved	DW

4.3 Estimation of Sand Transport Rates

It is clear from the aerial photographs that sand has accumulated against the eastern breakwater of The Anchorage since its construction in the early 1990's. We have estimated the amount of sand that has accumulated since construction and, by extension, have estimated an average annualised sediment transport rate along the beach over the past two decades.

From Figure 17 a pivot point about which the beach has apparently "rotated" can be identified around 250m east of the breakwater. To the west of this point, the beach has accreted, and to the east of this point, the beach has eroded. The rate of sand accumulation to the west of the pivot point has been estimated, and this used to estimate the average annual sand transport rate between 1992 and 2012.

Eight shore normal cross sections (~30m spacing) were established along the length of beach over which accretion has occurred during the past 2 decades. The shape of the present beach profile was estimated from a combination of the 2007 hydrosurvey and onshore lidar data provided by Port Stephens Council for the purpose of this project. The distance which the profile has moved seaward over this time was estimated based on the extent of the seagrass. Furthermore, based on the 2007 hydrosurvey, the profile was noted to flatten out at around -4.0m AHD. Accordingly, only volumes of accretion above -4.0m AHD were considered. Assuming that the overall profile has maintained a similar shape, which is reasonable given that the overall swell wave climate has not changed, profiles along each of the cross-sections for both 1992 and 2012 were determined. An example of the result for Cross-Section 1 (closest to the Anchorage Marina) is presented in Figure 23. By 2012, the profile adjacent to the Marina Breakwater had moved an estimated 50m northwards from its location in 1992.

The difference in area between the two profiles in Figure 23 illustrates the cross-sectional of accretion that has occurred adjacent to the breakwater. This area was calculated for each of the eight cross-sections, and then multiplied by the distance between the cross-sections to estimate the value of accretion to the west of the pivot point in the beach. The change in area at each cross-section, and total volume of accretion are presented in Table 7. Using these figures, around 33,800m^3 of sand is estimated to have accumulated along the western section of Corlette Beach between 1992 and 2012.

This equates to around 1700m^3 per year of sand passing the pivot point of Corlette Beach and agrees broadly with the estimates of Geomarine (1988) who indicated an average 3,000m^3/yr of littoral transport historically, but with an expectation that the rate would reduce to around 1,000m^3/yr with time. This sand comes from either sand transport around Sandy Point and/or erosion of the shoreline and near shore profile from Sandy point to the area of accretion at The Anchorage. The volume estimate provides a sound basis for considering sand placement and beach nourishment options.

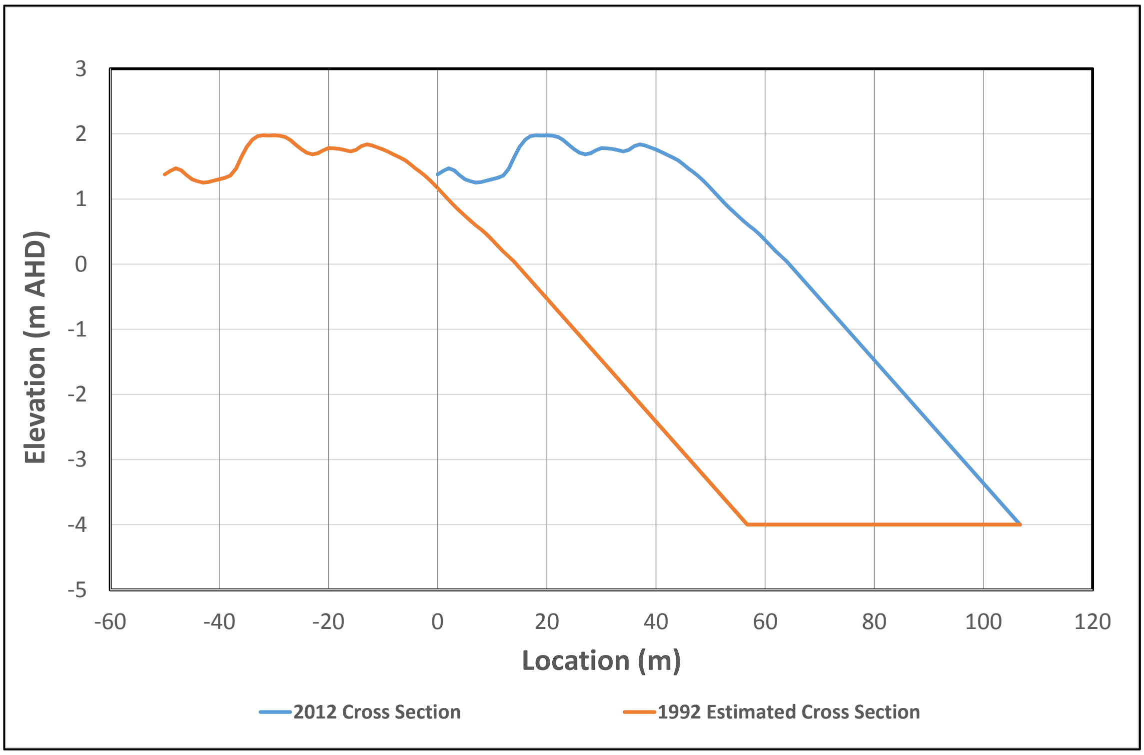


Figure 23 Example of Derived Cross Section Profiles for Transport Analysis

Table 7 Cross Section Areas and Total Volume

Distance East from Breakwater (m)	Change in Area (m^3/m)
16	282
48	203
80	189
112	158
144	244
176	70
208	45
240	13
Volume of Accumulation	33,800

5 Numerical Modelling

5.1 Introduction

A numerical model was built based on the Delft3d open source modelling software. That software is capable of simulating two dimensional (depth averaged) and three dimensional flow, sediment transport and morphology, waves, water quality and ecology. The wave component within the Delft3d modelling suite is provided via a link to the widely applied SWAN spectral wave model.

For this particular application, the model has been used in 2-dimensional (depth averaged) mode with the interactions between waves and wind as follows:

- Wind has been applied across the water surface of the hydrodynamic model to generate wave driven currents;
- Wind is applied to the water surface of the wave model to simulate the generation and growth of waves by wind across Port Stephens and in the open ocean;
- An ocean boundary is adopted for the application of ocean tides, which raise and lower the ocean water level and drive tides in and out of Port Stephens, generating currents and altering water levels inside the Port in accordance with the bathymetry used as input to the model;
- The same ocean boundary is adopted for the input of oceanic swell wave conditions, representative of depth conditions where waves are recorded offshore of NSW. The SWAN model propagates these offshore waves in through the mouth of Port Stephens, were they are altered primarily by the effects of shoaling, friction, refraction and diffraction, before they reach the study shoreline at a greatly reduced height and changed direction, but with relatively minor changes to the wave period;
- The wave model is used to calculate forces that act to both set-up water levels at the shoreline, and drive longshore currents.

The model bathymetry utilised for this study comprises the completed “2015” bathymetry as discussed in Section 4.2.1.

5.2 Configuration

The present open source version of Delft 3d undertakes its calculations on a curvilinear grid, comprising two sets of orthogonally intersecting lines (m & n lines) that bend slowly in space. These lines form approximately rectangular cells that vary slowly in size in the m and n directions. Orthogonality (lines intersecting at right angles) and smoothness (rate of change of size between adjacent cells) need to be kept within reasonable bounds to build a successful model.

The model built for this project extends over the entire surface of Port Stephens, and extends into the ocean to the 100m contour Figure 24. At this depth, it is reasonable to input waves as measured by the wave recorder network maintained offshore of NSW. The model covers large portions of the tributaries of Port Stephens, including Tilligerry Creek and the lower reaches of the Lower Myall and Karuah Rivers, although those areas are not targeted specifically for analysis at this time and the resolution there is less than around the study area. The model could be readily adapted for other locations in the future.

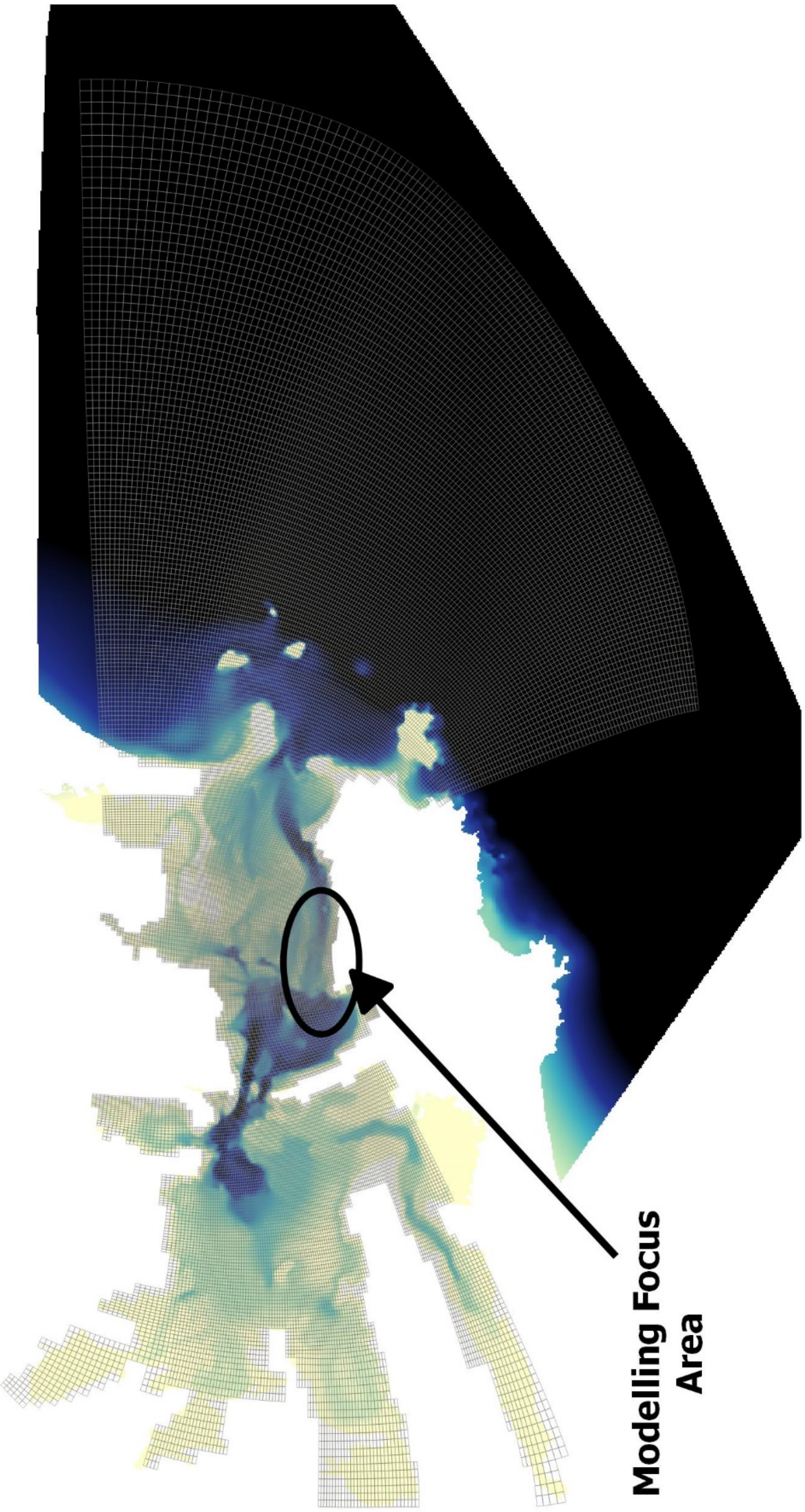


Figure 24: Model Extent and Configuration

N

Sandy Point / Conroy Park Coastal Processes Study

Whitehead & Associates
Environmental Consultants

0 2.5 5 7.5 10 12.5 km

(Approx Scale)

Revision	A
Drawn	DW
Approved	DW

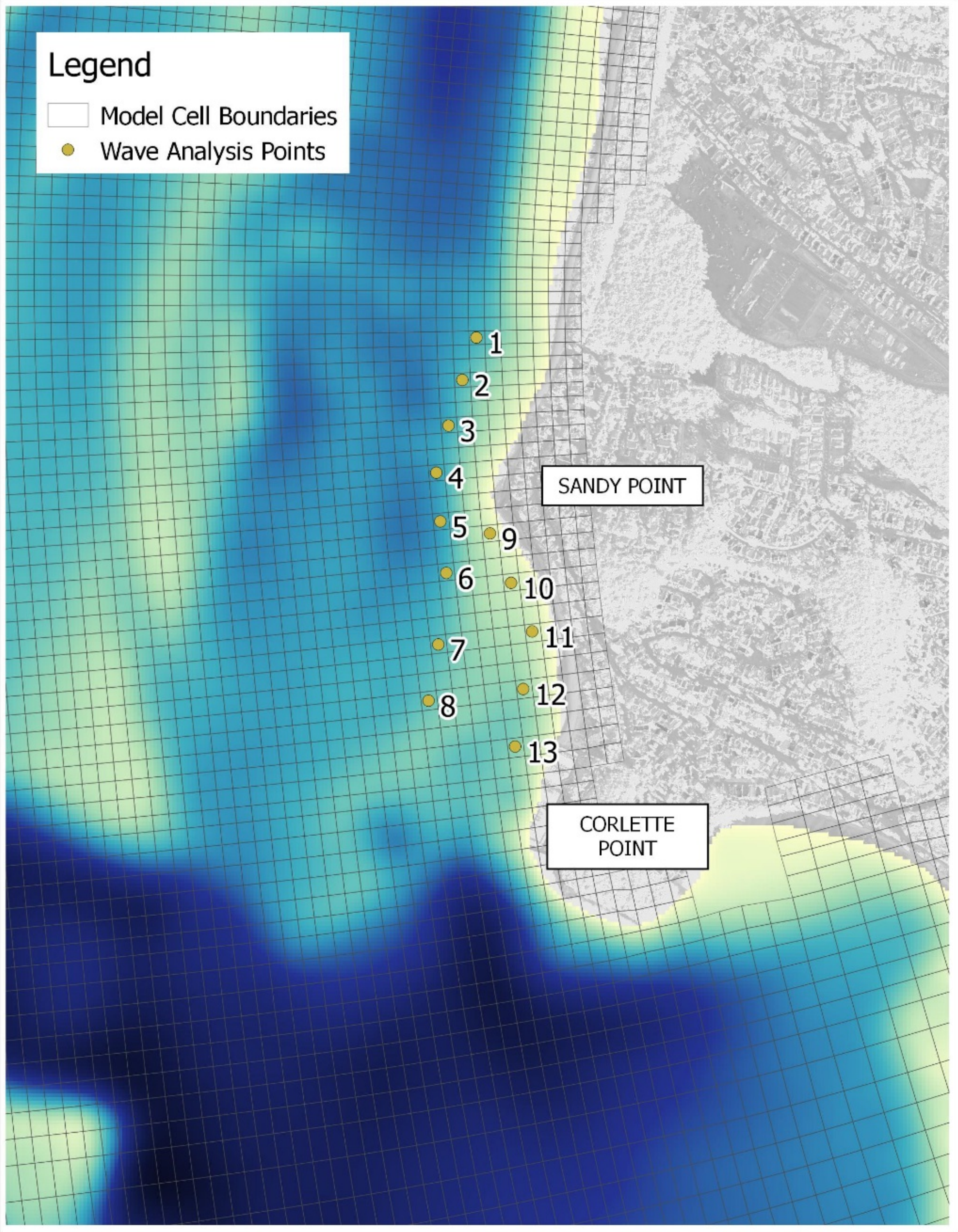


Figure 25: Model Grid Near Corlette and Wave Analysis Points

Sandy Point / Conroy Park Coastal Processes Study

Whitehead & Associates
Environmental Consultants

Revision	A
Drawn	DW
Approved	DW

The fringes of the model domain, where cells are coloured white in Figure 24 are included to ensure a reasonable representation of water storage at elevated water levels, particularly in conjunction with projected sea level rise. Lidar data, provided by Port Stephens Council were utilised to determine low lying ground elevations in those areas. The largest grid cells, along the ocean boundary of the model, have sides of up to 400m long. Around the study area the grid cells are finer, with side lengths of around 40 to 50m (Figure 25). Figure 25 also shows locations where results have been extracted from the wave model to help assess appropriate design conditions for any foreshore management options, as detailed in Section 6

5.3 Calibration

Before using the model, it is necessary to be comfortable with its ability to replicate real world conditions. Classically, any model should go through a two-step process involving:

- Calibration, where the model parameters are adjusted within reasonable bounds such that the model predicts field measurements over a given period of time; and
- Validation, where a second set of independent measurements is used to test the parameters arising from the model calibration.

In practice, there is some iteration between these processes resulting from the paucity of reliable data, particularly during significant storm events. We are unaware of any detailed and recent measurements in the near vicinity of the study site. Therefore, limited information was available to complete the two step process.

Initially, we have tested model performance against the recent, April, 2015 storm, being representative of the present configuration of the flood tide delta, and conditions that are experienced during an extreme storm, which is of most interest to the present study. Wave and tide data were obtained from Manly Hydraulics Laboratory, which manages tide and wave collection instruments in NSW on behalf of the Office of Environment and Heritage. Wind data were obtained from the Bureau of Meteorology for the Williamtown recorder. The Tomaree “ocean” tide gauge record was applied at the ocean boundary, as were the wave conditions from the Sydney recorder. Wind was applied uniformly across the surface of the model.

There was only one other tide gauge data set available inside the Port Stephens entrance, at Mallabula to the west of Soldiers Point. The following parameter values were adopted to achieve the “best fit” to the modelled water levels as follows:

- Manning Roughness of 0.017 (uniform across model domain), which is at the low end but within an acceptable range;
- Horizontal eddy viscosity of 100m^2/s, which is at the higher end of the recommended range;
- Subgrid scale turbulence was modelled using the standard large eddy simulation formulation and parameters as recommended in the modelling documentation; and
- Standard wind drag coefficients were adopted.

A time step of 0.05 minutes was adopted to achieve stability. A comparison of the resulting modelled and measured tide levels at the Mallabula gauge site is provided in Figure 26 covering a range of tidal cycles over 9 days.

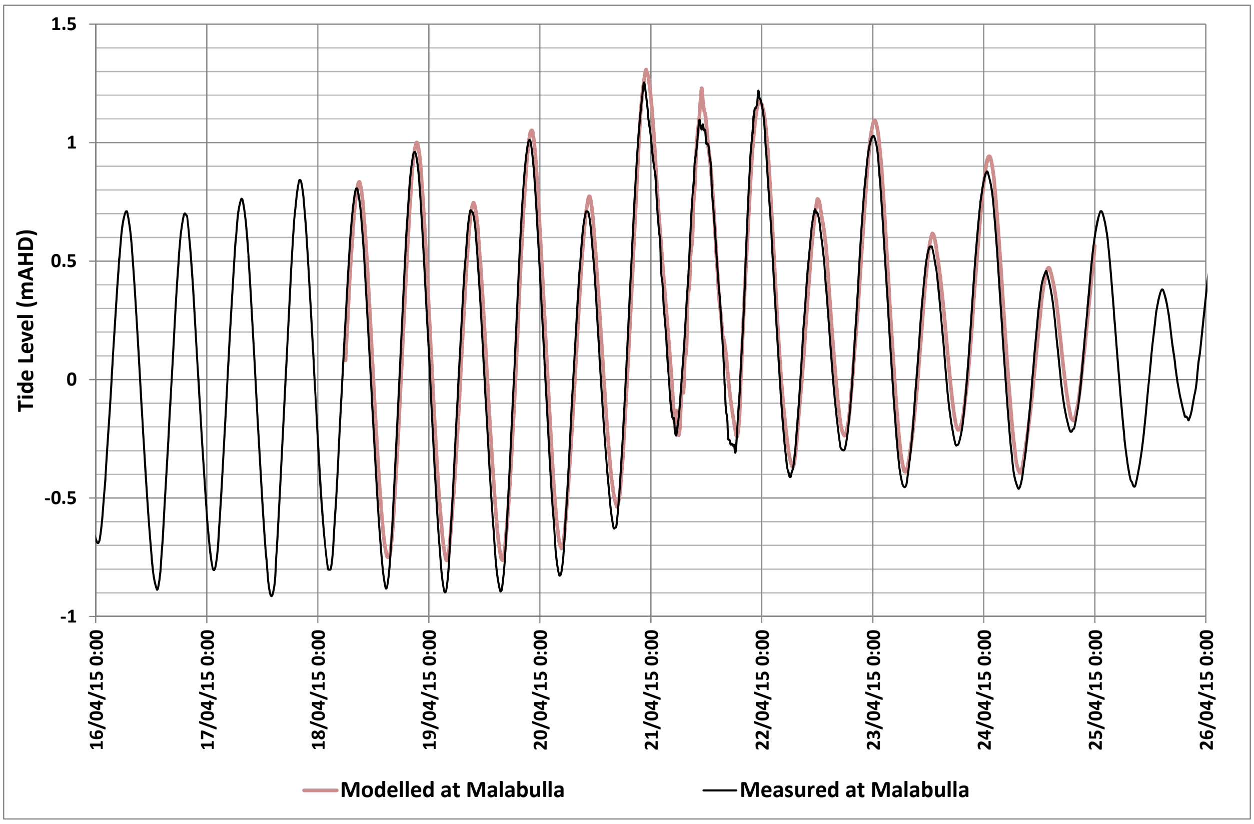


Figure 26 Modelled and Measured Tides at Mallabula Gauge, April 2015 Storm

A close look at the results indicates the following:

- The modelled values tend to be around 5cm higher than the measured values at both the tidal peaks and troughs; and
- The modelled high and low tides tend to occur slightly later than the corresponding measured high and low tides.

This type of mismatch, although reasonably minor, could normally be adjusted by lowering the friction coefficients within the model. However, as discussed above, the adopted coefficient is reasonably low already, and further downward adjustment is not prudent. It appears most likely that the application of the Tomaree Tide gauge, which actually sits inside the entrance to Port Stephens, as the ocean boundary, would account for much of this mismatch. It is likely that the ocean tide has been transformed by the time it propagates to the Tomaree gauge location.

Even so, the performance of the model is reasonable for storm conditions, considering the purpose to which it is to be applied in this study (deriving design conditions for conceptual design). Future users of the model would need to judge the sufficiency of the model for their particular purpose before application.

We are unaware of any robust wave measurements undertaken in the vicinity of the study area. Following the April 2015 storm and the community consultation undertaken as part of the overall project, we were provided with videos captured by the community during various storm events. In particular, one video captured around 27s of footage at around 11am on 21st April, showing waves breaking across the foreshore of No. 36 Sandy Point Road. Stills from that video are reproduced here as Figure 27. Importantly this video was captured very close to the time at which a record wind speed of over 25m/s was measured at Williamtown, with the wind approaching from a bearing of 150 degrees.

It is difficult to estimate a representative wave height from the video, although it appears that the actual wave height offshore at the time would be of the order of 1m, but probably less. As shown by the bottom left frame in Figure 27, the wave approaches from slightly east of shore normal, breaking across the eastern end of the foreshore fronting No. 36 first before peeling westwards along the structure. This corresponds to a wave approaching from almost north east.

The model prediction at 11am on 21st April is shown in Figure 28. That figure demonstrates wave heights around 0.8m offshore of the site, and waves approaching from east which are shore normal, as witnessed on site. This provides confidence that the modelled wave heights are reasonable, and that the model is capable of predicting extreme design wave conditions, including the influence of very strong winds.

As waves approach the foreshore they increase rapidly in height. This process is not accurately replicated by the SWAN wave model and neither is wave breaking. For this reason, the apparent decrease in wave height close to shore in Figure 28 is not matched by the actual waves which shoal and break. In considering waves at the foreshore for design, separate methods are needed to take this into account during the design process.

Current speed data are sparse. Nielsen & McGowan (1994), undertook modelling of the flushing pipes at the Anchorage Marina, and collected some current velocity data near the entrance to the Marina, indicating that peak ebb and flood current speeds were both around 0.4m/s. However, limited information is provided within that paper to discern the type of tide being considered, although there is some evidence that a tide range of around 1.6-1.7m was applied, some 10% larger than the mean spring tide at this site.

To test the model, a tide ranging from -0.75 to 0.9m was used in the model, and the current patterns and speeds that developed with such a tide were examined. For that simulation, winds and waves were not included. Figure 29 shows the predicted flood tide currents offshore of the study site. Notably, the current “streamlines” contract around Sandy Point and Corlette Head, causing faster flows in these areas, whereas tidal currents inshore adjacent to Corlette Beach are relatively static. While the 50m grid size of the model is insufficient to replicate exact current patterns around the Anchorage Marina, it can be seen that the model predicts currents in the vicinity of 0.3 to 0.4m/s near the entrance to the Marina, with currents up to 0.6m/s or more with distance offshore. A very approximate rule of thumb would suggest that currents of 0.25m/s are capable of entraining and transporting sand size coastal sediments in NSW. The corresponding figure for ebb tide currents (Figure 30) indicates a very similar pattern to the flood tide currents, albeit in the reversed direction. If anything, the current speeds on ebb tides are marginally slower than those on the Flood Tide.

The model does a reasonable job of predicting currents in this area, possibly tending towards a slight under prediction of velocities.

Figure 27 Stills from Video of Foreshore Overtopping, 21st April, 2015.
(Top Left: Approaching, Top Right: Breaking, Bottom Left: Impact, Bottom Right: Backwash)

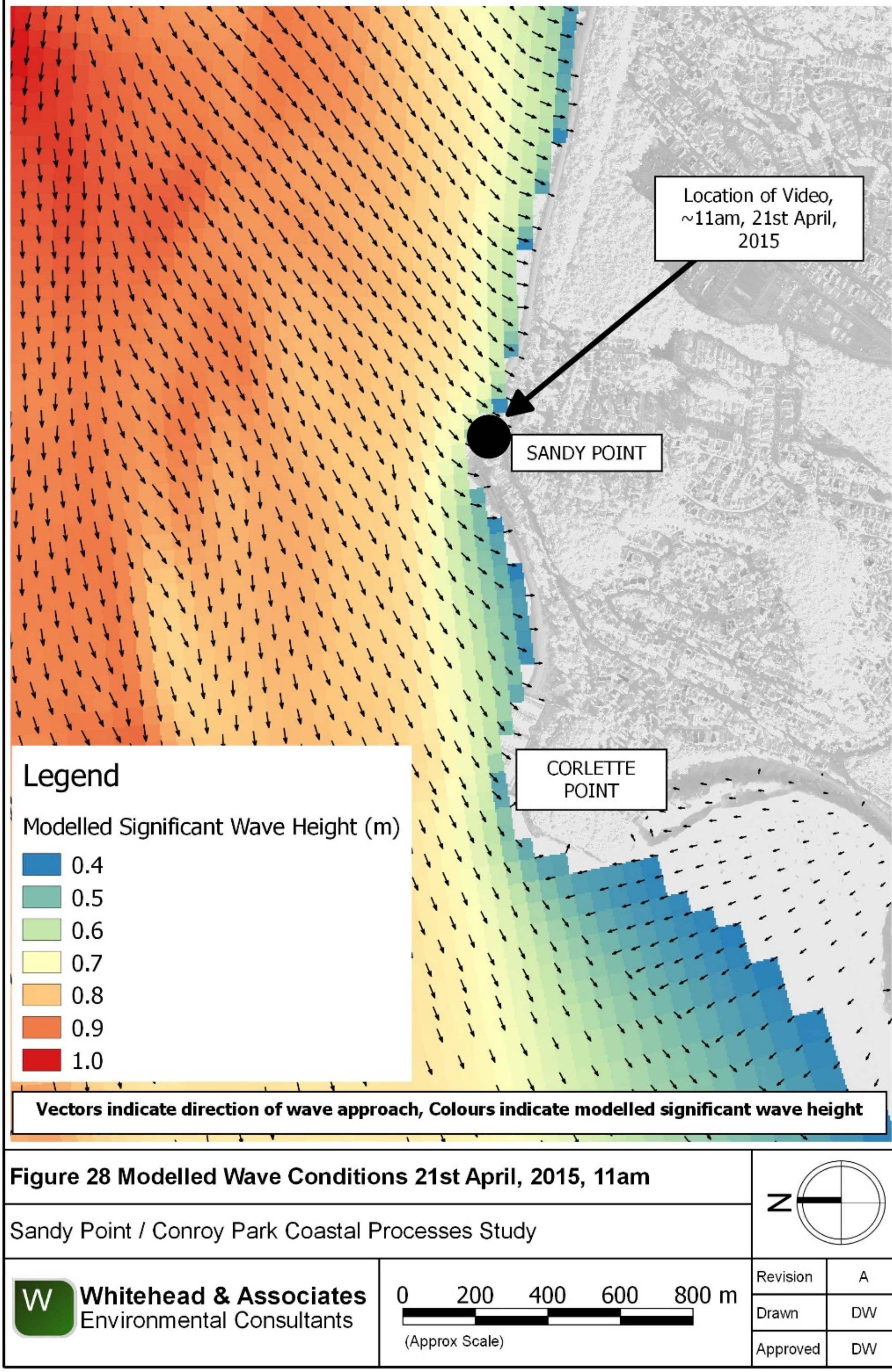


Figure 28 Modelled Wave Conditions 21st April, 2015, 11am

Sandy Point / Conroy Park Coastal Processes Study

Whitehead & Associates
Environmental Consultants

Revision	A
Drawn	DW
Approved	DW

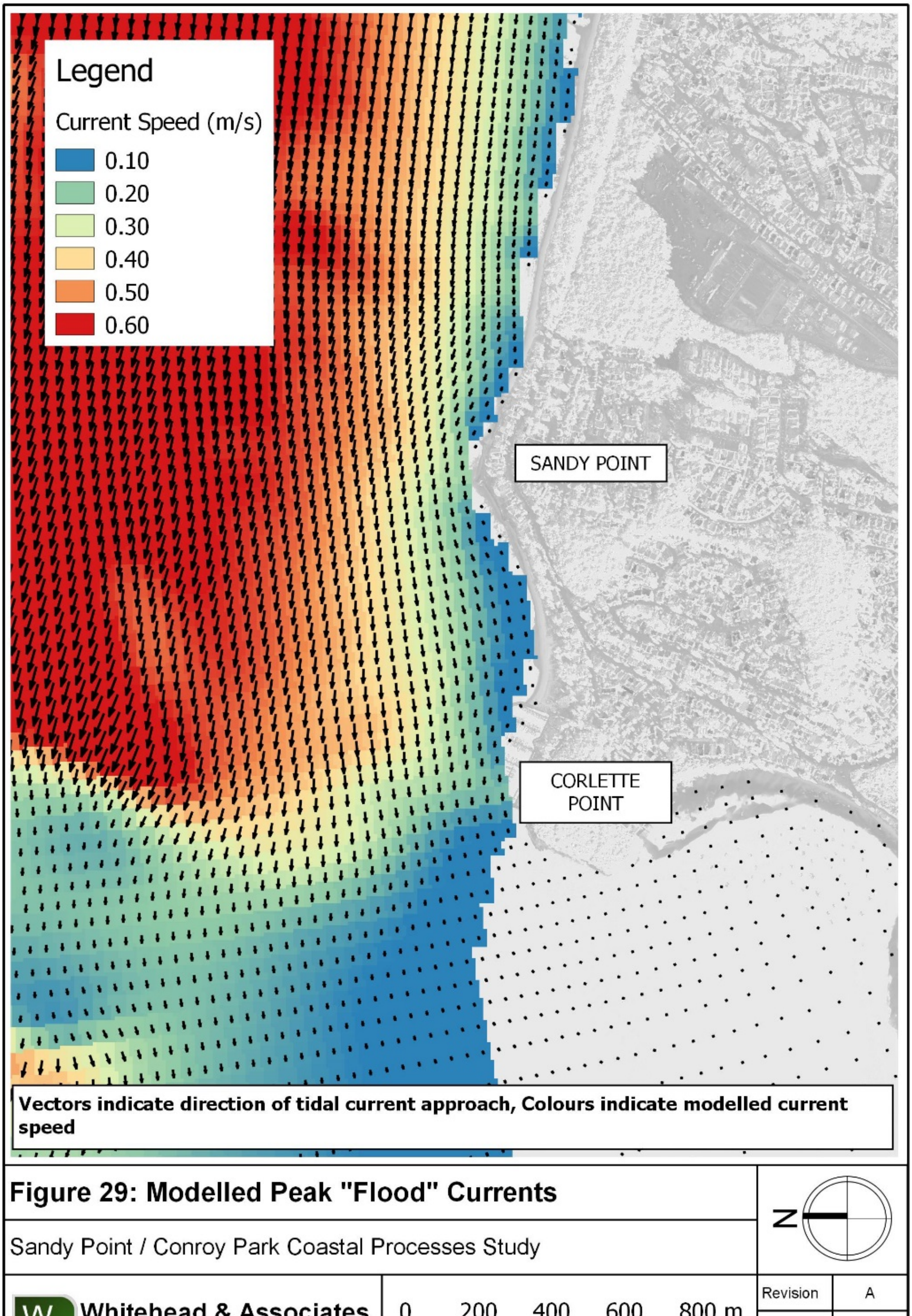


Figure 29: Modelled Peak "Flood" Currents

Sandy Point / Conroy Park Coastal Processes Study

Whitehead & Associates
Environmental Consultants

Revision	A
Drawn	DW
Approved	DW

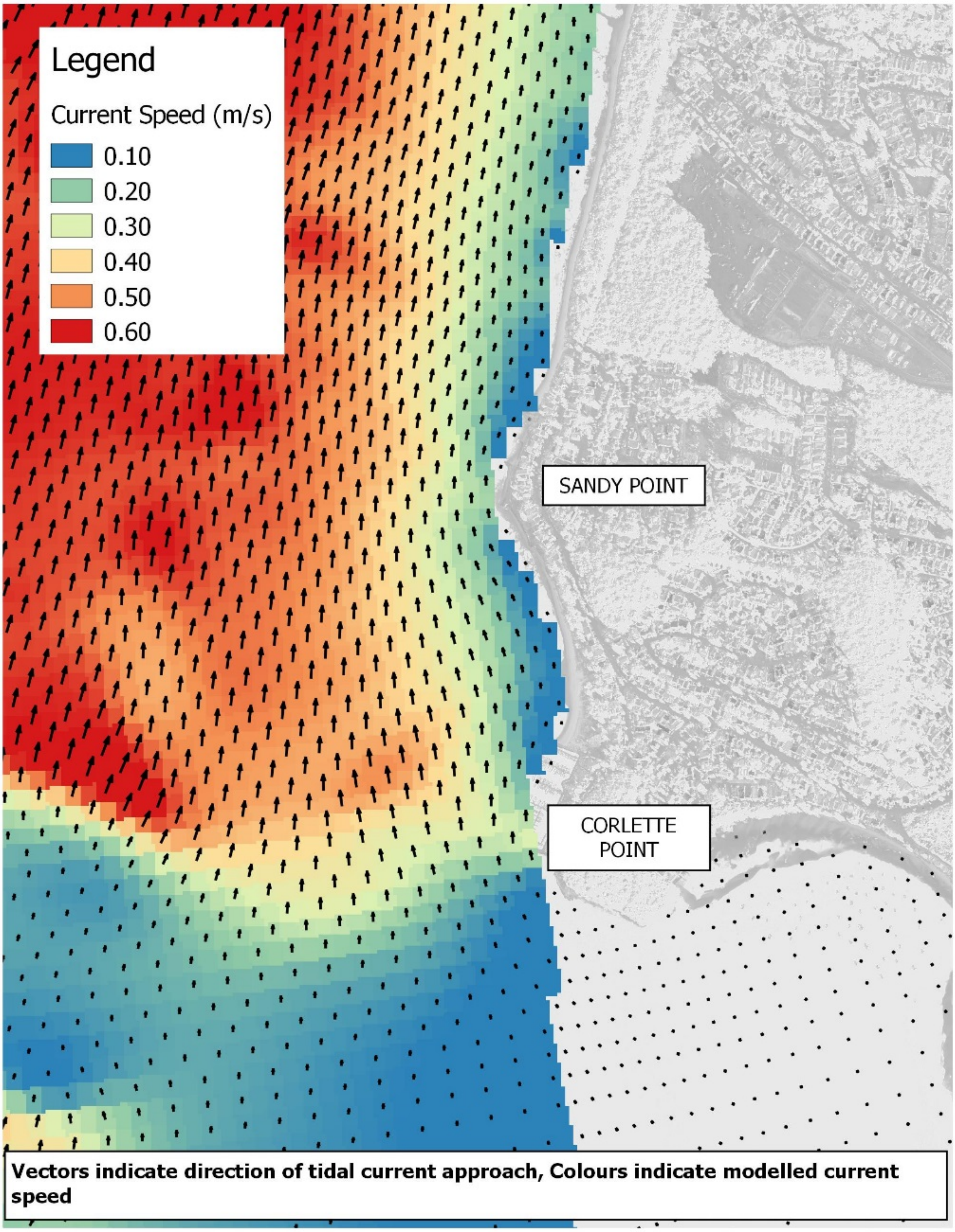


Figure 30: Modelled Peak "Ebb" Currents

Sandy Point / Conroy Park Coastal Processes Study

N

Whitehead & Associates
Environmental Consultants

0 200 400 600 800 m

(Approx Scale)

Revision	A
Drawn	DW
Approved	DW

6 Design Conditions

6.1 Offshore Waves and Water Levels

6.1.1 Offshore Waves

Design “storm” conditions typically comprise both an elevated water level and the action of high waves. Shand et al. (2012) undertook a Joint Probability Assessment of these parameters for available records along the NSW coast. From the Crowdy Head and Sydney records, they present Wave Buoy Extreme values as shown in Table 8.

Table 8 Estimated Extreme Offshore Significant Wave Heights (m)

Probability	Sydney[3]	Crowdy Head[4]
10% (P=0.1)	2.55	2.48
1% (P=0.01)	4.19	2.94
1yr ARI (P~0.0001)	5.9	5.4
10yr ARI (P~0.00001)	7.5	7.0
100yr ARI (P~0.000001)	9.0	8.5

Table 8 illustrates a long recognised feature of the NSW wave climate: that the central NSW coast, around Sydney tends to have a more severe, stormier wave climate than the coast to the north (e.g. Crowdy Head) or south. While the average of both sites could be taken to approximate conditions offshore of Port Stephens, a slightly conservative approach is to adopt the more severe, Sydney conditions and this is the approach taken herein. In terms of the actual waves experienced at the Sandy Point / Conroy Park site, the wave heights are greatly reduced, as shoaling, refraction and friction losses affect waves as they propagate into Port Stephens towards Corlette.

With respect to wave direction Callaghan et al. (2008) studied 30 years of coastal storms at Sydney. They found that there was a tendency for extreme storms to have waves approaching from between 150 and 170˚ (i.e. from south of south east), with less extreme storms clustering around 175˚. They were, however, unable to determine a robust relationship between storm wave height and direction. For the present study, waves between 130 degrees and 180 degrees were investigated in setting the design storm wave heights inside the Port, noting that the offshore direction and period of the wave are very important determinants of the degree of focussing and refraction once waves propagate inside the entrance to Port Stephens.

Callaghan et al. (2008) also presented a statistically derived expression for the “expected” or most likely value for the period associated with a particular storm wave height. Adopting that expression, and the extreme Sydney Wave Heights from Table 8, appropriate periods for the design waves were derived.

[3] Directional Record between 1989 and 2009 Analysed
[4] Non Directional Record between 1985 and 2010 Analysed

Table 9 Expected Wave Period for Adopted Design Offshore Wave Heights

Probability	H_s(m)	T_p(s)
1yr ARI	5.9	10.7
10yr ARI	7.5	11.5
100yr ARI	9.0	12.3

6.1.2 Offshore Water Levels

Astronomical tide levels have been reported by Manly Hydraulics Laboratory (2012). The tidal planes for the closest ocean tide gauge at Tomaree are presented in Table 10. These values are based on an annual average covering 20 years of data between 1990 and 2010.

Table 10 Annual Averaged Tidal Planes (Tomaree)

Tidal Plane	Level (m AHD)
Higher High Water Springs Solstices	0.976
Mean High Water Springs	0.601
Mean High Water	0.474
Mean High Water Neaps	0.348
Mean Sea Level	-0.038
Mean Low Water Neaps	-0.423
Mean Low Water	-0.55
Mean Low Water Springs	-0.677
Indian Springs Low Water	-0.945

The joint probability analysis of Shand et al. (2012) concluded that, in the absence of sufficient data to enable a more comprehensive analysis at a particular site, complete dependence of offshore significant wave heights and tidal residual[5] should be assumed. While many smaller storms do not show any real dependence, Shand et al. noted that there does appear to be a correlation near the extremes, with the largest measured tidal residuals corresponding to the largest measured wave heights.

Bearing this in mind, the extreme values of tidal residuals described in Shand et al. (2012) are presented in Table 11

[5]The tidal residual is the amount by which the offshore water level exceeds the predicted astronomical tide.

Table 11 Estimated Extreme Tidal Residuals (m)

Probability	Fort Denison[6]	Sydney[7]	Port Macquarie (Offshore)[8]
1yr ARI (P~0.0001)	0.36	0.31	0.37
10yr ARI (P~0.00001)	0.44	0.38	0.48
100yr ARI (P~0.000001)	0.61	0.47	0.61

Due to the apparent anomalies between Fort Denison and Sydney, which should be statistically similar, Manly Hydraulics Laboratory (MHL) was contacted to discuss this issue. MHL was in the process of updating water level analyses, including water levels during storms, along the NSW coast. Draft outputs from that report (MHL 2236) were provided to us by Ben Modra from MHL.

Charts showing total water levels for different recurrence intervals were provided for both the Tomaree Gauge and Fort Denison. Values were extracted from these charts and are presented in Table 12. It was noted that present analysis shows more consistency between the two Gauges in Sydney Harbour

Table 12 Ocean Water Levels for Various Recurrence Intervals

Recurrence Interval (yrs.)	Fort Denison (Sydney Harbour) Water Level (m AHD)	Tomaree (Port Stephens) Water Level (m AHD)
5	1.3	1.28
20	1.36	1.31
50	1.40	1.35
100	1.42	1.37

Herein, considering the slight tidal amplification present in Port Stephens, and the much longer record available for Fort Denison, the analysis for the Fort Denison Gauge data is considered an appropriate basis for foreshore protection design.

At the present time, PSC has adopted an allowance for sea level rise in line with the benchmark values adopted by the NSW government in 2009, and subsequently withdrawn in 2011. Local Councils have been advised to investigate and make their own decision regarding the projections that they should adopt, and many local councils in NSW have retained the original benchmark values. The values comprise a 0.4m rise in mean sea level between 1990 and 2050 with a further 0.5m by 2100. Of importance to the present study is that a 25 year planning time frame was set by Council (i.e. designed to be serviceable until 2040). Interpolating the 40/90 values via a second order polynomial, results in applying a rise of close to 32cm between 1990 and 2040. However, to make this meaningful for design, it needs to be adjusted to be relative to AHD. Wainwright et al. (2014) presented analysis of the Fort Denison tidal record and estimated that mean sea level was around 3cm above AHD in 1990. Accordingly, we estimate that an appropriate allowance for mean sea level in 2040 will be 35cm AHD and advise that this should be used for design.

[6]Record between 1914 and 2011
[7]Record between 1987 and 2011
[8]Record between 1984 and 2011

6.2 Nearshore Water Levels

Astronomical tide levels have been reported by Manly Hydraulics Laboratory (2012). The tidal gauge at Mallabula, to the west of Soldiers Point and some 9km west of Sandy Point was commissioned in 1992. Manly Hydraulics Laboratory reported annually averaged tidal planes as presented in Table 13.

Table 13 Annual Averaged Tidal Planes (Mallabula)

Tidal Plane	Level (m AHD)
Higher High Water Springs Solstices	1.08
Mean High Water Springs	0.69
Mean High Water	0.588
Mean High Water Neaps	0.443
Mean Sea Level	0.009
Mean Low Water Neaps	-0.431
Mean Low Water	-0.57
Mean Low Water Springs	-0.709
Indian Springs Low Water	-0.995

A comparison of these tidal planes indicates that there is a small amplification of the tides, with the tidal ranges at Mallabula higher than those at Tomaree. Furthermore, the mean sea level at Mallabula is higher than at Tomaree, indicating a slight "pumping" up of the tide. Overall, at the high tide levels of importance to designing foreshore management strategies, there is a difference of around 10cm. Most of the bathymetric features which would contribute to these modifications exist downstream (i.e. east) of the site. Therefore, it is reasonable to adopt the tidal planes at Mallabula as an underlying basis for design water levels. Similarly, adding 0.1m to the still water levels presented in Table 12 will include an allowance for the tidal amplification effects that are felt at the site.

It can be reasonably assumed that mean water level, and all tidal planes within the Port will rise by a similar amount to those in the open ocean, as a result of climate change driven sea level rise. Utilising Council's adopted values, Mean Sea Levels are projected to be around 0.35m AHD in 2040, which is the timeframe for planning required by Port Stephens Council for this project.

6.3 Nearshore Waves

6.3.1 Introduction

The design wave climate near the shoreline of Sandy Point and Conroy Park comprises two notably different types of waves:

- Modified ocean swell which propagates through the entrance to Port Stephens and is significantly refracted and affected by frictional losses before approaching the study shoreline. These waves tend to have a low height in the vicinity of Corlette, even during extreme events, but shoal (rear up) significantly as they approach the shoreline. The degree to which they shoal is affected by the wave period, with longer period waves shoaling to a greater extent; and

- Waves generated by winds acting locally over the surface of Port Stephens. The shoreline is exposed to wind waves approaching from the west, north and east. Southerly winds will generate offshore waves at the site. While these locally generated waves can be quite high, they have short periods and are not subject to shoaling to the same extent as refracted oceanic swell.

In order to obtain design wave conditions close to the shoreline, both types of waves have been simulated using the numerical model described in Section 5, with details of those simulations provided in the following two sections. Model results were extracted for the 13 locations shown on Figure 25 to ascertain whether there was a difference in nearshore design wave heights at different locations.

6.3.2 Swell Waves

Swell wave simulations were executed for the conditions presented in the 1, 10 and 100yr recurrence interval conditions presented in Table 8. In addition to these three recurrence intervals, intermediate wave heights and periods were also simulated. In all cases, a following wind was also applied to the model. By applying this wind to the surface of the model, additional wave height growth between the offshore location (where waves are recorded, and the statistics are based) and the inside of Port Stephens is represented. Testing showed that this following wind could contribute significantly to the refracted swell wave heights simulated inside the Port and, based on our model testing of the April 2015 storm, must be included to get reasonable results. The wind speed was selected to match the direction and recurrence interval of the swell wave simulated at the model boundary. This is appropriate, as a commensurately strong wind is required to generate a wave of a given recurrence interval. In summary, there were 5 base wave conditions considered as presented in Table 14. Note that 3 different directions were considered, between 130 and 180 degrees based on the findings of Callaghan et al. (2008). While the wave height used was the same, the wind speed was varied based on the extreme analysis undertaken of the Williamtown wind record.

Table 14 Swell Wave Conditions Modelled

ARI	Hs	Tp	Wind Speed (From 130 degrees)	Wind Speed (From 155 degrees)	Wind Speed (From 180 degrees)
1	5.9	10.7	13 0	13.4	15.1
Intermediate	6.7	11.1	14.5	14.9	15.7
10	7.5	11.5	16.0	16.3	16.3
Intermediate	8.25	11.9	18.0	18.6	16.5
100	9.0	12.3	20.1	20.8	16.7

In total, there were 15 different conditions simulated, and these were executed for an extreme (and unrealistic) 12.5 hour tidal cycle varying between -0.95 and 1.60m AHD. The tide signal was constructed by adding a 0.6m surge on top of a tide varying between ISLW and HHWSS in the ocean. The surge rose and fell completely in sync with the tide. The purpose of this synthesised water level was twofold:

- To investigate swell wave propagation over a wide range of water levels; and

- To investigate an extreme current condition and how that might affect focussing of wave energy due to wave-current interaction, which is built into the model.

Following completion of the simulation, the maximum wave height resulting from the 12.5hr of each tidal cycle was analysed to find the maximum wave height at each of the 13 locations considered and this was the nearshore design wave condition adopted at that location for each of the modelled conditions from Table 14.

A full summary of the outputs for all 13 sites is tabulated in Appendix C. The results indicate that swell waves at the site are highest when waves approach from a more easterly direction (i.e. 130 degrees). Along the eastern side of Sandy Point, the 100 year swell wave condition is around 0.8m, whereas for a 1yr ARI the wave heights are around 0.6m. Waves approach from between 75 and 80 degrees at this location. At point 8, the wave results are not fully representative of the swell condition, and there are anomalous results present due to the generation of wind waves from inside Salamander Bay. Otherwise, along the western side of Sandy Point, the swell wave heights are smaller ranging from around 0.4m for a 1yr ARI up to 0.7m for a 100yr ARI Event. Waves to the west of Sandy Point approach from between 45 and 55 degrees.

In comparison, MHL (1997) predicted design swell waves of around 0.4m for a 100yr ARI event and 0.5m for an extreme event. One possible reason for this was the reliance on pure swell conditions input to the boundary of the model. We have found that wind on coastal waters can significantly increase wave heights from between the depths where ‘offshore’ waves are typically measured in New South Wales and the coastline. Testing of our model without the following wind indicated that similar design values to those presented by MHL were obtained.

6.3.3 Wind Waves

Local wind waves have been tested in the model, with extreme wind conditions determined from the Williamtown wind record. Previous studies have highlighted an excellent correlation between the record at Williamtown, and temporary wind records collected during studies at Jimmy’s Beach (Geomarine, 1988; MHL, 1997). Accordingly it is reasonable to apply the record at Williamtown for this purpose. The Williamtown Recorder has an elevation of 9m. It is standard to adjust the wind to match an equivalent wind at 10m height. The adjustment from 9m to 10m elevation results in an increase in wind speed of 1.5%.

Using the data record provided by BoM for Williamtown, all records were grouped into 16 bins equally spaced around the compass and then a statistical fit to the generalised extreme value distribution undertaken using the maximum likelihood method.

Overall, the fit was good for most directions of interest, although there were anomalies for winds from North to East. Technically, the derived shape parameter for the distribution in these directions (ξ) returned positive values in a number of instances. This appears physically implausible for winds being generated from one ‘type’ of meteorological system. The presence of significant large outliers in a number of these N⇨E directional bins suggests that there may be more than two types of meteorological systems generating winds from this quadrant, one of which results in the most extreme of events.

Regardless, the positive value of ξ results in higher wind speeds at the higher recurrence intervals, and the values from the analysis have been adopted as moderately conservative.

Table 15 Extreme Wind Analysis Results

Direction	ARI (years)					
	10	20	25	30	50	100
W	18.6	19.3	19.6	19.7	20.2	20.7
WNW	21.6	22.5	22.7	22.9	23.4	24.0
NW	18.7	19.6	19.9	20.1	20.6	21.3
NNW	15.2	16.4	16.7	17.0	17.7	18.4
N	12.9	15.2	16.0	16.7	18.7	21.8
NNE	11.1	12.3	12.7	13.0	13.9	15.2
NE	11.9	12.8	13.1	13.4	14.1	15.2
ENE	12.9	13.6	13.8	13.9	14.4	15.0
E	14.1	15.3	15.7	16.0	17.0	18.4

A simulation was executed with these wind speeds from each of the directions being considered. The simulation was continued over a full range of tidal elevations, utilising the tidal boundary described in Section 6.4 to examine the impact of water levels on the generation of these local waves. Outputs were derived at the same locations as described in Section 6.3.2. The maximum wave simulated at each location over the entire tidal cycle was selected for design, and the results are tabulated in Appendix C.

The results show that wind waves from West North West are the largest for almost every location and recurrence interval considered, except for the rarest events at Point 2, where a North Westerly wave is the largest. Design waves vary between 0.6 and 0.8m for all locations and recurrence intervals.

The maximum height waves approach from a direction that will impact almost normal to the western side of Sandy Point. However, along the eastern side (Output Points 1 through 4) a northerly wave, which approaches in a more shore normal direction, may be more appropriate for designing against wind waves. Wind waves are unlikely to refract significantly to impact on the eastern side of Sandy Point.

The wind waves modelled here are significantly smaller than those presented in the part 2 of the Port Stephens Flood Study (MHL, 1997), which estimated wind waves of over 2.0m for the 100yr ARI event. As noted in Section 2.3.2, we consider this to have been most likely caused by an overestimate of the design wind speeds particularly from the North West and west, which the present study has shown to be the critical direction for wind waves.

6.3.4 Use of Modelled Waves in Design

The waves presented in the preceding two sections have been extracted at locations at least 100m and in depths of at least 7m, offshore of the immediate shoreline around the study site. In order to develop conditions at the immediate shoreline, it is necessary to consider the wave transformation processes that will alter the waves as they traverse the surf zone and impact upon the foreshore. The complex wave breaking and surf zone dynamics are not well replicated by the numerical model. The design of foreshore treatments, such as revetments and sea walls, needs to consider the way in which these waves interact with the structures. The more detailed propagation of these waves and their interaction with the shoreline is to be considered during the design of foreshore treatments as part of the design of management options during latter stages of this study.

6.4 Current Velocities

Current circulation patterns and peak velocities have been investigated using the numerical model. A repeating, theoretical tide, representing the largest oceanic astronomical tidal range was applied as the ocean boundary condition, with no wind or waves applied and the maximum depth averaged ebb and flood current speeds calculated in the vicinity of the study site. These are presented in Figure 31 and Figure 32 for flood and ebb tide currents respectively.

Figure 19 demonstrates that the current speeds are increase with distance offshore, reaching 0.6m/s approximately 300m north of Corlette in the centre of the east-west aligned tidal channel. Currents are comparatively slower around Corlette Point. This is related to the offshore bathymetry at Corlette. As the flood tide flows around Sandy Point, it diverges off Sandy Point and follows the line of a steep shelf which carries it away from Corlette Beach, as indicated by Figure 19. As the tidal flows continue west past Corlette Head, the depth averaged speed decreases due to the sudden increase in depth at the tidal dropover.

Similarly, the Ebb tide in Figure 20 moves fastest approximately 300m offshore from Corlette and is slowest around Corlette Point. The Ebb tide current drains Salamander Bay resulting in a convergence of flow and acceleration around Corlette Point, which again acts to divert tidal currents from Corlette Beach. Accordingly, tidal currents tend to be faster around the northern tip of Sandy Point, but slower along Corlette Beach.

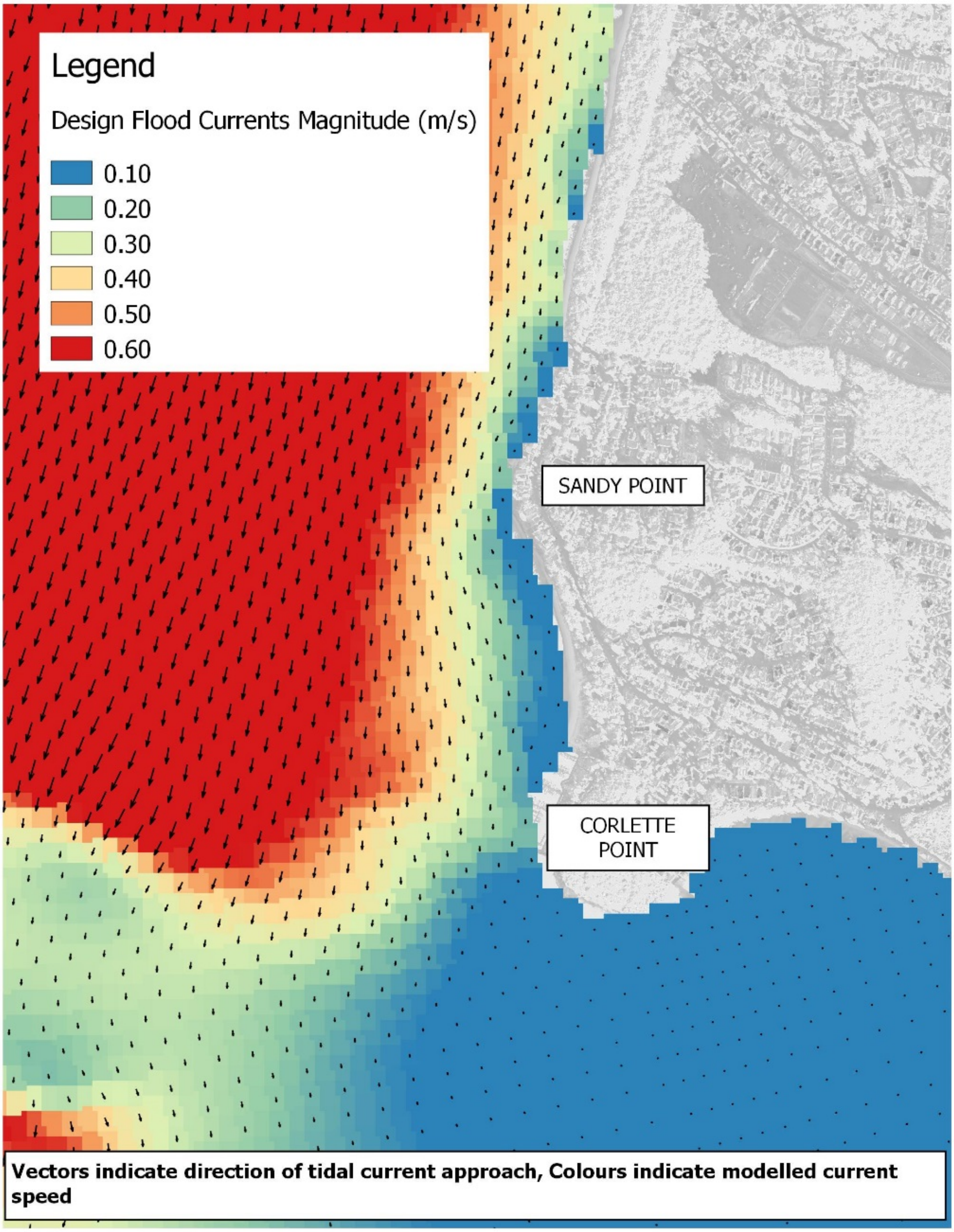


Figure 31: Design Flood Tide Currents

Sandy Point / Conroy Park Coastal Processes Study

Whitehead & Associates
Environmental Consultants

Revision	A
Drawn	DW
Approved	DW

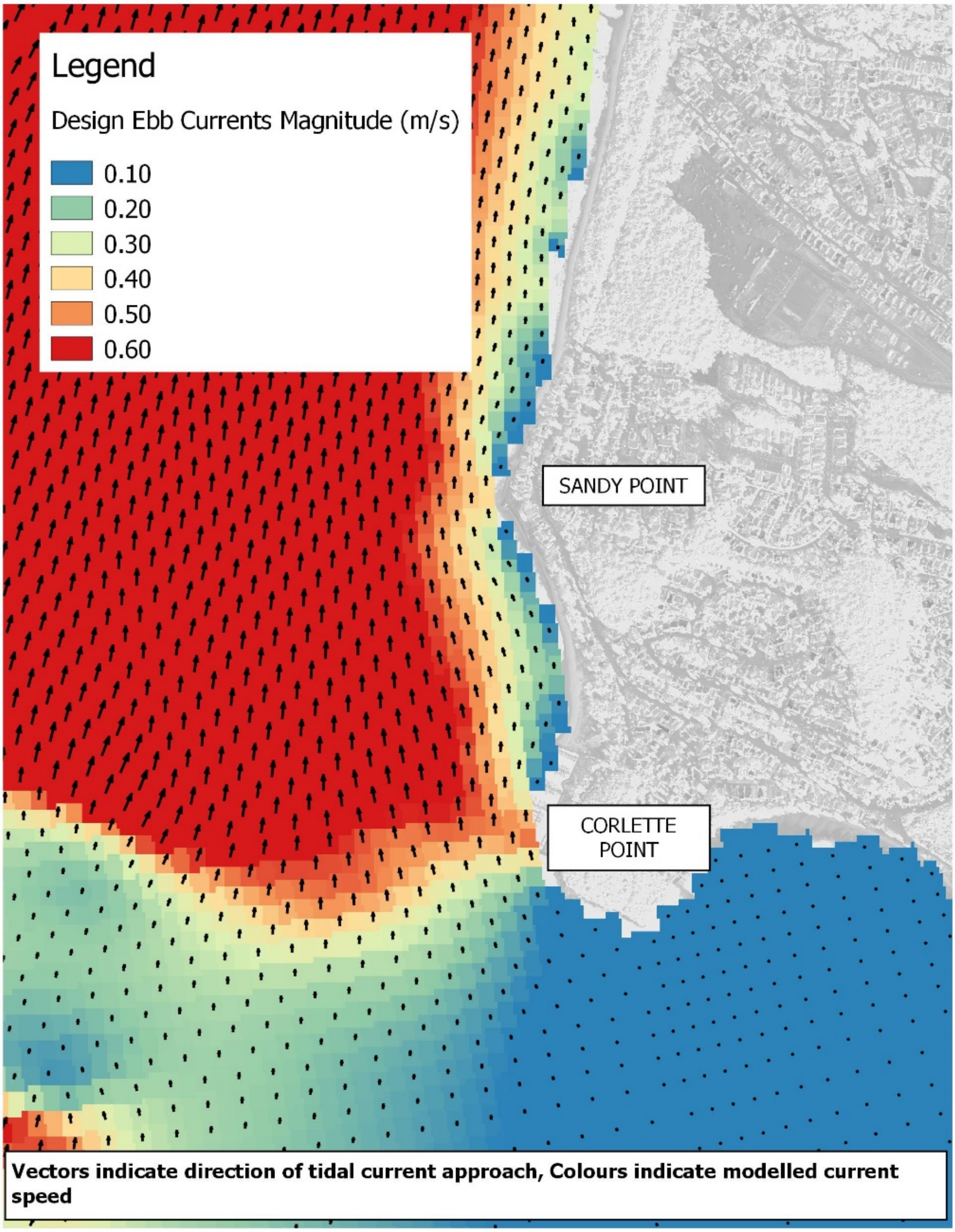


Figure 32: Design Ebb Tide Currents

Sandy Point / Conroy Park Coastal Processes Study

N

Whitehead & Associates
Environmental Consultants

0 200 400 600 800 m
(Approx Scale)

Revision	A
Drawn	DW
Approved	DW

7 Summary

This Coastal Processes report was prepared to inform the development of appropriate management strategies for the Sandy Point / Conroy Park Foreshores within Corlette, to the south of Port Stephens.

Firstly, the report examined the geomorphological context of the study area and found that the foreshore, along with most foreshores in the lower (eastern) basin of Port Stephens, is strongly influenced by behaviour across the estuarine flood tide delta. The flood tide delta is a large sand body which defines bathymetry in the lower basin and is slowly moving into the Estuary, at estimated rates of between 0.5 and 1.0m per year. The present, prograding (leading) face of the delta exists approximately between Corlette Head and Pindimar.

The study foreshore, extending from the western end of Bagnalls Beach, has suffered from intermittent and presently ongoing erosion since the area was first settled in the late 1940's and 1950's. At the time that subdivision occurred, a large sandy lobe existed offshore of Sandy Point, providing a wide sandy beach and obviously affecting the naming of the point. Within a decade of initial settlement, however, this lobe had eroded to such an extent that foreshore erosion was becoming a problem to residents. Initial settlement of the point was undertaken without a clear understanding of the variability or processes acting along this length of foreshore. The original sand lobe was clearly not a permanent feature, and Sandy Point is located on a receding shoreline. The historical siting of development within coastal areas that have subsequently proven to be “at-risk” is not uncommon along the NSW coast, but it means that there are now a number of complex management issues to be addressed.

Historically, the foreshore has been subject to intermittent periods of erosion, when no sandy beach was present, and periods when plenty of sand was present. This has been caused by the intermittent transport of pulses of sand from east to west along the foreshore. In the past two decades, the situation has tended more towards a lack of sandy foreshore, with the exception of the western end of Corlette Beach, adjacent to “The Anchorage” marina, where sand has been accumulating.

Since the late 1950’s, the need to protect the foreshore has been clear, with protective structures appearing along the eastern side of Sandy Point from the early 1960's. Over time, the extent and magnitude of protective works has increased, with the protected length of foreshore extending westwards as time passed. This is consistent with an ongoing east to west sediment transport along the shoreline, forced by ocean swell which enters the Port and is refracted across the flood tide delta to impact on the study shorelines.

At the present time, the most westward of this sequence of structures is a sand filled geotextile sand bag wall fronting the eastern end of Conroy Park. Erosion will continue to the west of this structure with time unless appropriate management actions are taken to arrest it. To the west of geotextile sand bag wall, Corlette Beach has shown a pattern of erosion over the past 20 years, although that eroded sand is accreting on the beach adjacent to “The Anchorage” marina. This results in an apparent “pivot” point about which the beach has rotated, with that point located around 250m to the east of “The Anchorage”. The beach is presently adjusting to be “in equilibrium” with the incoming swell wave direction and, if allowed to continue, will likely erode the majority of Conroy Park, the adjacent car park and road. The rate at which this is occurring would slow with time and, if allowed to continue, such extensive erosion would likely take many decades. Even so, erosion of Conroy Park is a contemporary problem, with the previous sandy beach having been lost and foreshore vegetation being progressively undermined and lost to wave action. The area fronting Conroy Park is referred to as “Precinct 2” in our study, and is one of the key areas of concern for ongoing management.

A group of around 9 residences on the eastern side of Sandy Point are highly exposed to overtopping of the foreshore by refracted ocean swell waves. This appears to have been the case for a number of decades, and may be influenced by the focussing of wave energy by ledges in the nearshore bathymetry, where those ledges may result from the underwater outcropping of underlying geology. Foreshore structures in this area, while substantial, do not meet the standard of engineering that is normally applied in professional coastal engineering practice in NSW at the present time.

Anecdotally, we understand that some of these structures are presently overtopped several times per year, although this would vary from property to property, as the nature and effectiveness of the foreshore protection varies substantially. The nature of some of these structures presents an impediment to foreshore access by the public, given that a public reserve exists between the shoreline and the properties which the structures aim to protect. The area fronting this group of residences is referred to as “Precinct 5” in our study, and is also key area of concern for ongoing management.

Elsewhere, the structures are less substantial, but also have significant problems with design, the most notable being over-steepness, lack of filter, insufficient crest elevations and lack of a structural toe. Perhaps of more concern is the presence of numerous boat ramps along the foreshore which present a significant weakness for foreshore protection and allow the runup of waves and inundation of the foreshore reserve and residential yards during moderate wave conditions. During numerous site inspections undertaken during this study, we have noted the deposition of sand on the landward side of removable barriers installed in an attempt to prevent boat ramp runup reaching residential buildings. These measures are apparently only effective to a small degree and it is highly doubtful that they would prove effective during relatively frequent storms.

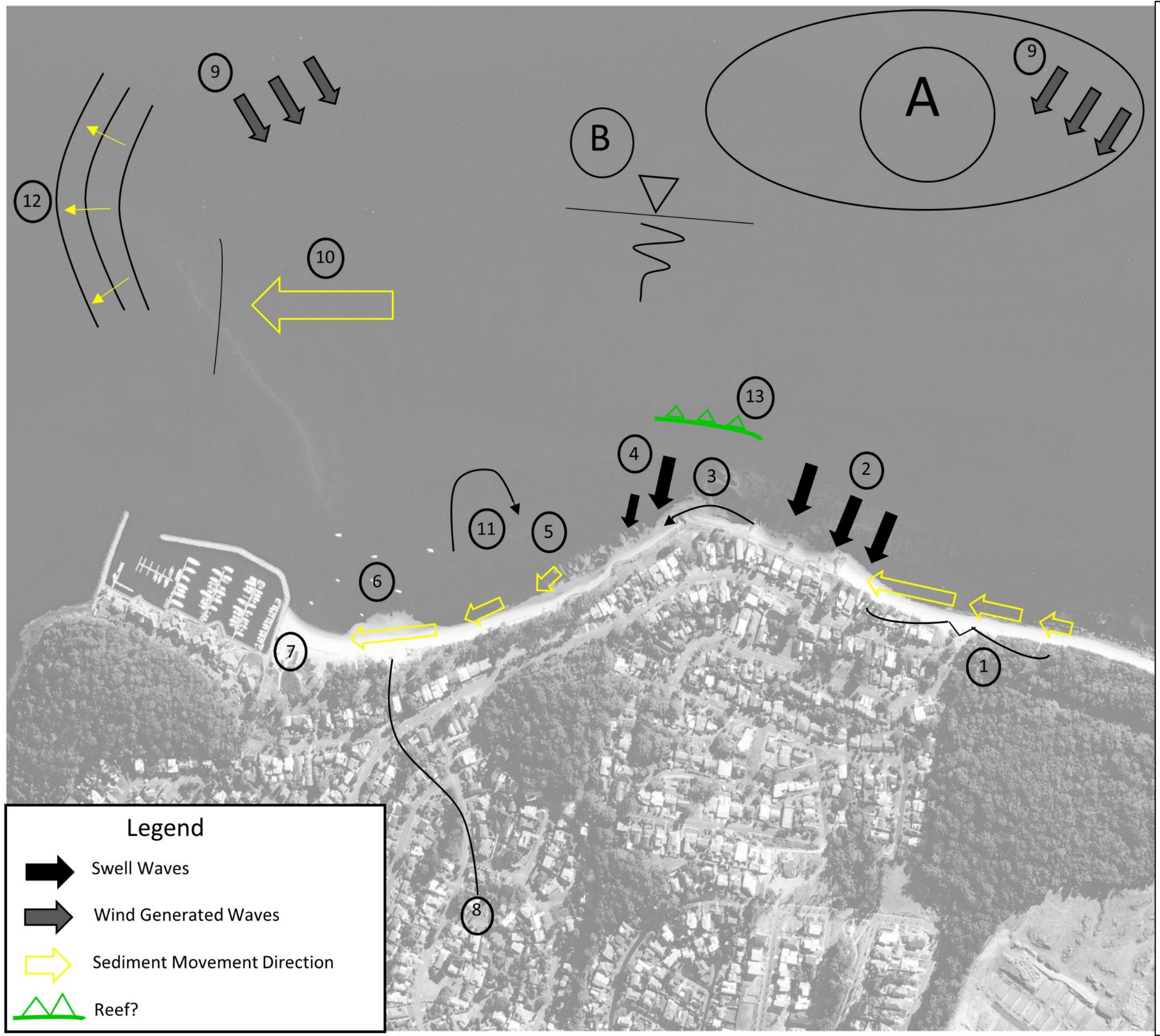
The analysis of aerial photography and hydrosurvey data as part of this study has validated the findings of previous investigations, namely that:

- Sand movement is from east to west;
- There is less sand offshore of Sandy Point than there used to be in the past, with no imminent respite expected from this as a result of natural processes;
- Erosion will continue at the western end of structures lining this length of foreshore without some intervention; and
- Sand will continue to move and accumulate adjacent to the breakwater of the Anchorage Marina at rates of between 1000 and 2000 m^3/yr on average.

Also notable, is the slow lowering of the bed offshore of the study site. This is consistent with expected ongoing processes associated with the flood tidal delta in Port Stephens. While we expect this to continue, the incremental impact on waves and currents and the immediate nearshore bathymetry of the study site is expected to be minimal over the design time frame established for this project (25 years).

A numerical model was developed as part of the study, and used to estimate design conditions. While the model appears to reasonably replicate real world conditions, and provides results that are broadly consistent with previous studies in the area, we note that insufficient data exist to validate the model. We consider that deployment of wave/current meters in the vicinity of the site would be a useful exercise prior to detailed design, to enable proper validation of the model and to give more confidence in the design values simulated by the model.

A conceptual model summarising the coastal processes surrounding the site has been prepared. This model is presented as Figure 33



A. Long Term changes to the Flood Tide Delta of Port Stephens cause ongoing change to wave refraction and current patterns. These affect patterns of erosion and deposition within the study area.
B. The water levels in the Port here are largely governed by ocean water levels (and tides). A rise in mean sea level will result in an equivalent rise in ocean water levels during storms.

1. Eastern Bagnalls Beach is relatively sheltered from ocean swell waves, however, exposure increases with distance westwards.
2. At western Bagnalls Beach refracted swell waves approach perpendicular to the beach resulting in a small amount of east to west longshore drift. Waves are particularly focussed on the eastern end of Sandy Point. Groynes do act to trap some sand, but they are sub-optimal.
3. A lobe of sand which existed at Sandy Point has diminished over the past 60 years, driven by swell wave induced east to west sediment transport, this has resulted in the exposure of residential properties to quite severe wave conditions. We cannot reliably say that this sand will be replenished from the East in future. Properties here are now subject to significant threats. Structures in this area are variable and not to an acceptable coastal engineering standard. Overtopping is severe for relatively frequent storms
4. Around the western side of Sandy Point, waves are more oblique, becoming increasingly optimal at the eastern end of Conroy Park driving erosion at that location.
5. The eastern 1/3rd of Corlette Beach, fronting Conroy Park is subject to an increasing sediment transport rate with distance westward. Therefore, erosion is present in front of Conroy Park at the present time, and expected to become more severe over the coming decades.
6. The western 2/3rds of Corlette Beach are now accreting, due to the interruption of sand by the Anchorage Marina breakwater. Sand has accumulated at an average rate of around ~1,000 m^3/year since construction. This is as predicted. Due to this point and point (5), Corlette Beach is seen to be rotating to be more aligned with incoming swell waves (and minimise transport). It is not in equilibrium with the present swell environment.
7. Sand has accreted to the point where stormwater outlets are now buried adjacent to the Anchorage. This will exacerbate stormwater flooding upstream.
8. This major stormwater outlet, erodes sand from the beach face, depositing in the nearshore. The process is not of significant concern, and any pollutants carried here do not seem to be affecting seagrasses (with the exception of smothering). The sand deposited in the nearshore is still part of the littoral transport, as the long period swell waves are strong enough to move the sand.
9. Wind generated waves from the NW and NE are of minor concern. They don't contribute as much to sand movement as swell waves, but can cause some nuisance overtopping of foreshores, and fretting of already destabilised eroding shorelines, such as at Conroy Park.
10. Offshore, the tidal channel is slowly deepening. It transports sand at 10m^3/m/year.
11. Wind driven circulations offshore of Corlette Beach have only minor impacts on sand transport .
12. Tidal Delta "Drop Over" is accreting in this location at a rate of 0.5-1.0m/year.
13. Apparent reef (persistent steep bathymetry) in this area concentrates currents around Sandy Point, also contributes to east to west movement of sand.

Figure 33: Conceptual Coastal Processes Model

Sandy Point / Conroy Park Coastal Processes Study

N

Whitehead & Associates
Environmental Consultants

0 500 1000 1500 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

8 References

Callaghan, D.P., Nielsen, P., Short, A.D., Ranasinghe, R., 2008. Statistical simulation of wave climate and extreme beach erosion. Coast. Eng. 55, 375–390. doi:10.1016/j.coastaleng.2007.12.003

CERC, 1984. Shore protection manual, 4th ed. U.S. Army Coastal Engineering Research Center, Corps of Engineers, Washington, D.C.

Coastal Environment, 2013. *“Assessment and Decision Frameworks For Seawall Structures”.* Part A Synthesis Report, Part B Appendices. April 2013 Final report on a Coastal Adaptation Decision Pathways Project funded by the Australian Government, prepared for the Sydney Coastal Councils Group Inc. ISBN 978-0-980208-4-5.

Dunn, S.L., Nielsen, P., Madsen, P.A., Evans, P., 2000. Wave setup in river entrances, in: Edge, B.L. (Ed.), . Presented at the Proceedings of the 27th International Conference on Coastal Engineering, ASCE, New York, pp. 3432–3445.

Frolich, M., 2007. Recent Morphological Evolution of the Port Stephens Flood Tide Delta. University of Sydney, Sydney.

Geomarine Pty. Ltd., 1988. Port Stephens Marina, Corlette. Coastal Processes (Preliminary, Final is attached to Anchorage EIS No. 88001.01.003).

Gutteridge, Haskins and Davey, 1989. The Anchorage. Corlette, Port Stephens. Environmental Impact Statement (No. 211/022602/00).

Hanslow, D., Nielsen, P., 1992. Wave setup on beaches and in river entrances. Presented at the Proceedings of the 23th International Conference on Coastal Engineering, ASCE, pp. 240–252.

Manly Hydraulics Laboratory, 2012. OEH NSW Tidal Planes Analysis: 1990-2010 Harmonic Analysis (No. MHL2053).

Manly Hydraulics Laboratory, 1999. Port Stephens / Myall Lakes Estuary Processes Study (No. MHL913).

Manly Hydraulics Laboratory, 1998. Port Stephens Flood Study - Stage 3 Foreshore Flooding (Final Draft No. MHL880).

Manly Hydraulics Laboratory, 1997. Port Stephens Flood Study - Stage 2. Design Water Levels and Wave Climate (Final No. MHL759).

Nielsen, A., McCowan, A.D., 1994. A Natural Flushing System for Artificial Harbours; a Case Study of The Anchorage Port Stephens, Corlette, N.S.W. Trans. Inst. Eng. Aust. Multidiscip. Eng. GE18, 41–48.

Poulos, D., 2011. Unique soft coral habitat in a temperate estuary: significance to biodiversity and marine park management (Honours). University of Technology, Sydney.

Pullen, T., Allsop, N., Bruce, T., Kortenhaus, A., Schüttrumpf, H., Van der Meer, J., 2007. EurOtop wave overtopping of sea defences and related structures: assessment manual.

Shand, T.D., Wasko, C.D., Westra, S., Smith, G.P., Carley, J.T., Peirson, W.L., 2012. Joint Probability Assessment of NSW Extreme Waves and Water Levels. University of New South Wales, School of Civil and Environmental Engineering.

Short, A.D., 2007. Beaches of the New South Wales coast: A guide to their nature, characteristics, surf and safety. Australian Beach Safety and Management Program, Sydney.

Umwelt, 2012. Coastal Zone Management Plan for the Shoalhaven Coastline - Appendices (No. 2239/R04_V3).

Umwelt, 2009. Living on the Edge. A Foreshore Management Plan for Port Stephens (Draft).

Umwelt (Australia) Pty. Limited, 2000. Port Stephens and Myall Lakes Estuary Management Plan (No. 1287/R04/V2).

University of Sydney, n.d. Review of Studies on Estuarine Morphology and Sediment Movement Conducted in Port Stephens Estuary.

Wainwright, D.J., Lord, D., Watson, P., Lenehan, N., Ghetti, I., 2014. “Widely Accepted by Competent Scientific Opinion” Sea Level Projections for the Shoalhaven and Eurobodalla Coast. Presented at the 13rd NSW Coastal Conference, Ulladulla.

Webb, McKeown & Associates Pty. Ltd, 2002a. Port Stephens Foreshore (Floodplain) Management Study. Prepared for Port Stephens and Great Lakes Councils.

Webb, McKeown & Associates Pty. Ltd, 2002b. Port Stephens Foreshore (Floodplain) Management Plan. Prepared for Port Stephens and Great Lakes Councils.

WMA Water, 2010. Port Stephens Design Flood Levels Climate Change Review (Final). Prepared for Port Stephens and Great Lakes Councils.

Appendix A Aerial Photographs

Figure A1: 2012 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

N

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

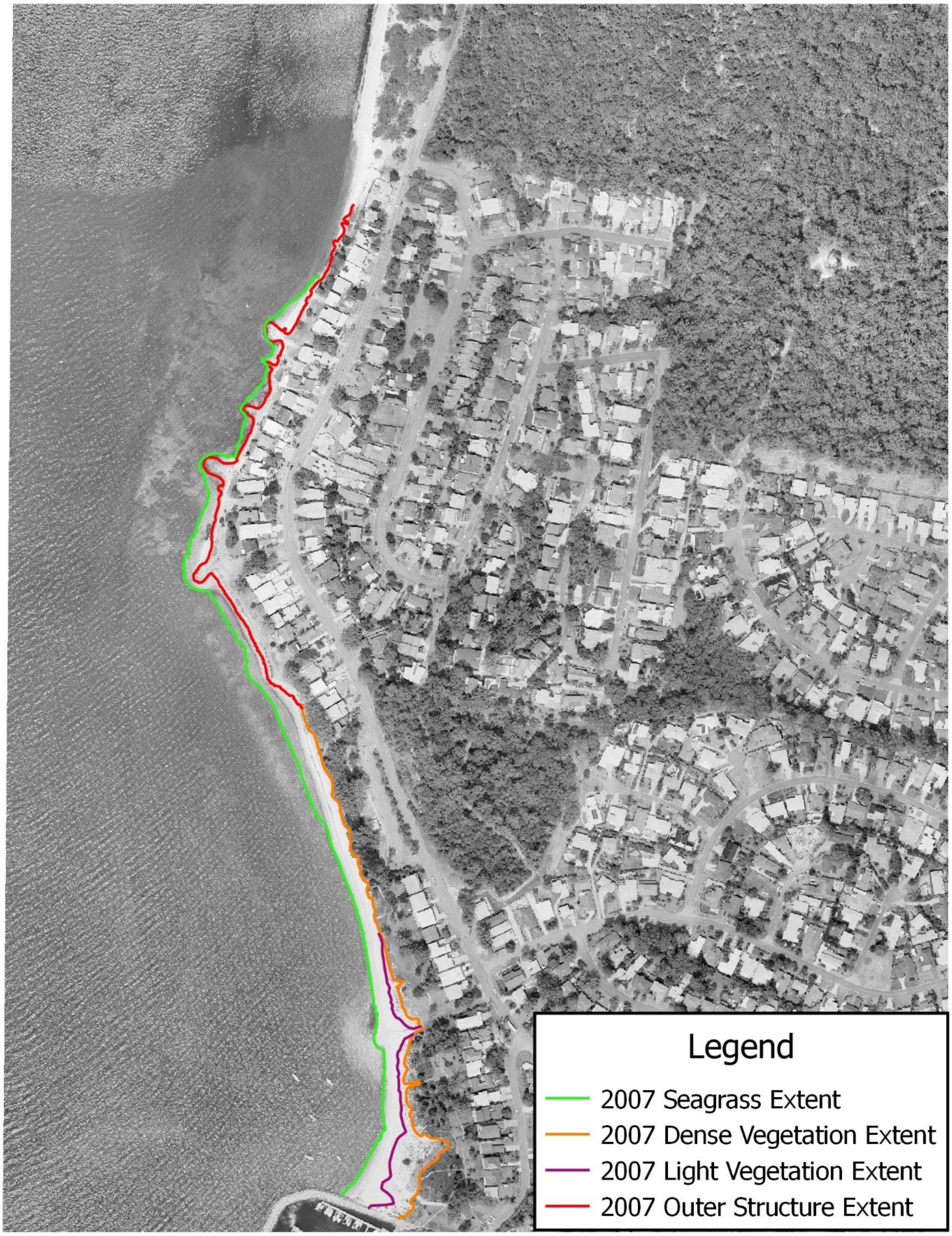
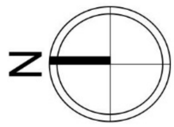


Figure A2: 2007 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan



Whitehead & Associates
Environmental Consultants

Revision	A
Drawn	BC
Approved	DW

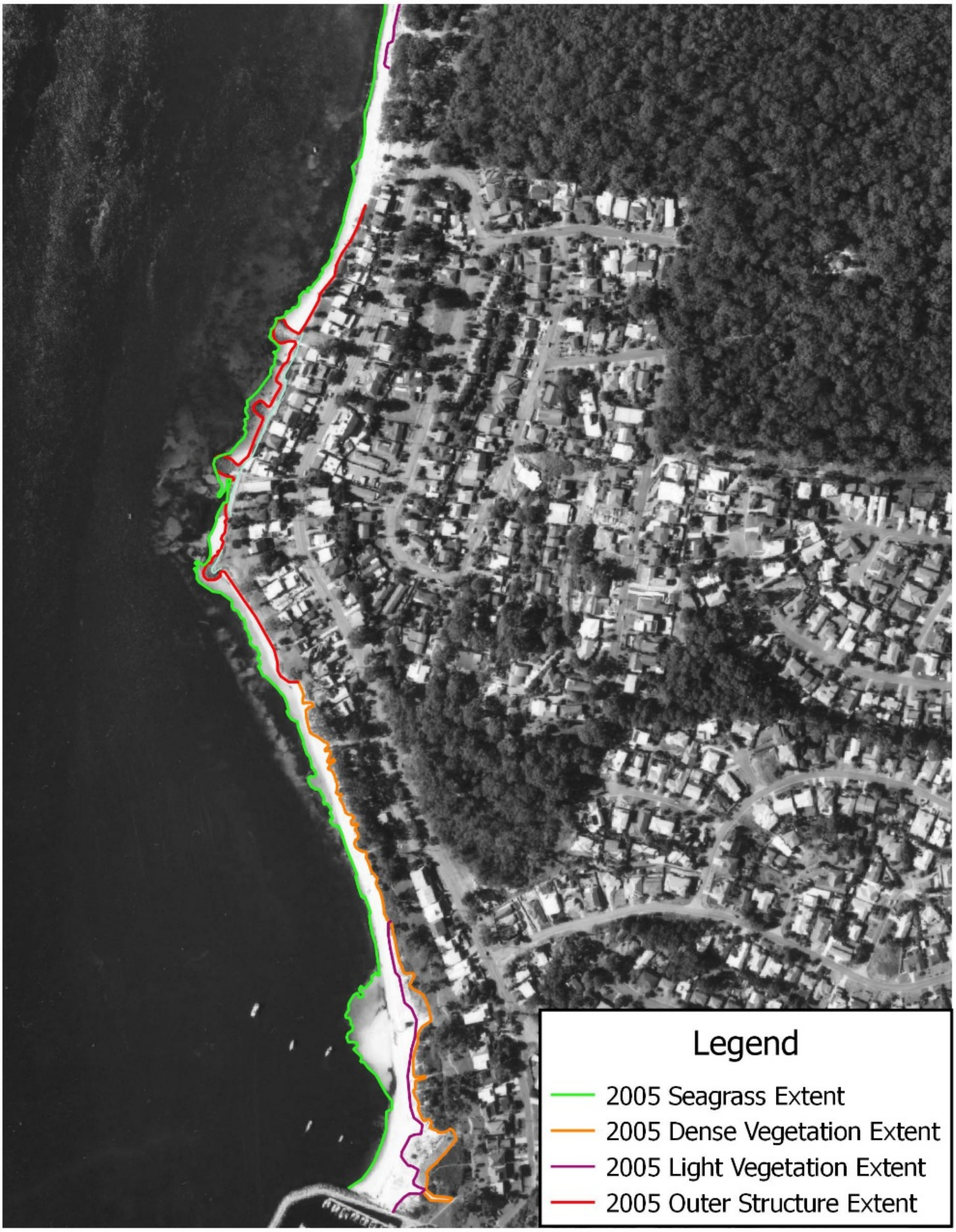


Figure A3: 2005 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

Revision	A
Drawn	BC
Approved	DW

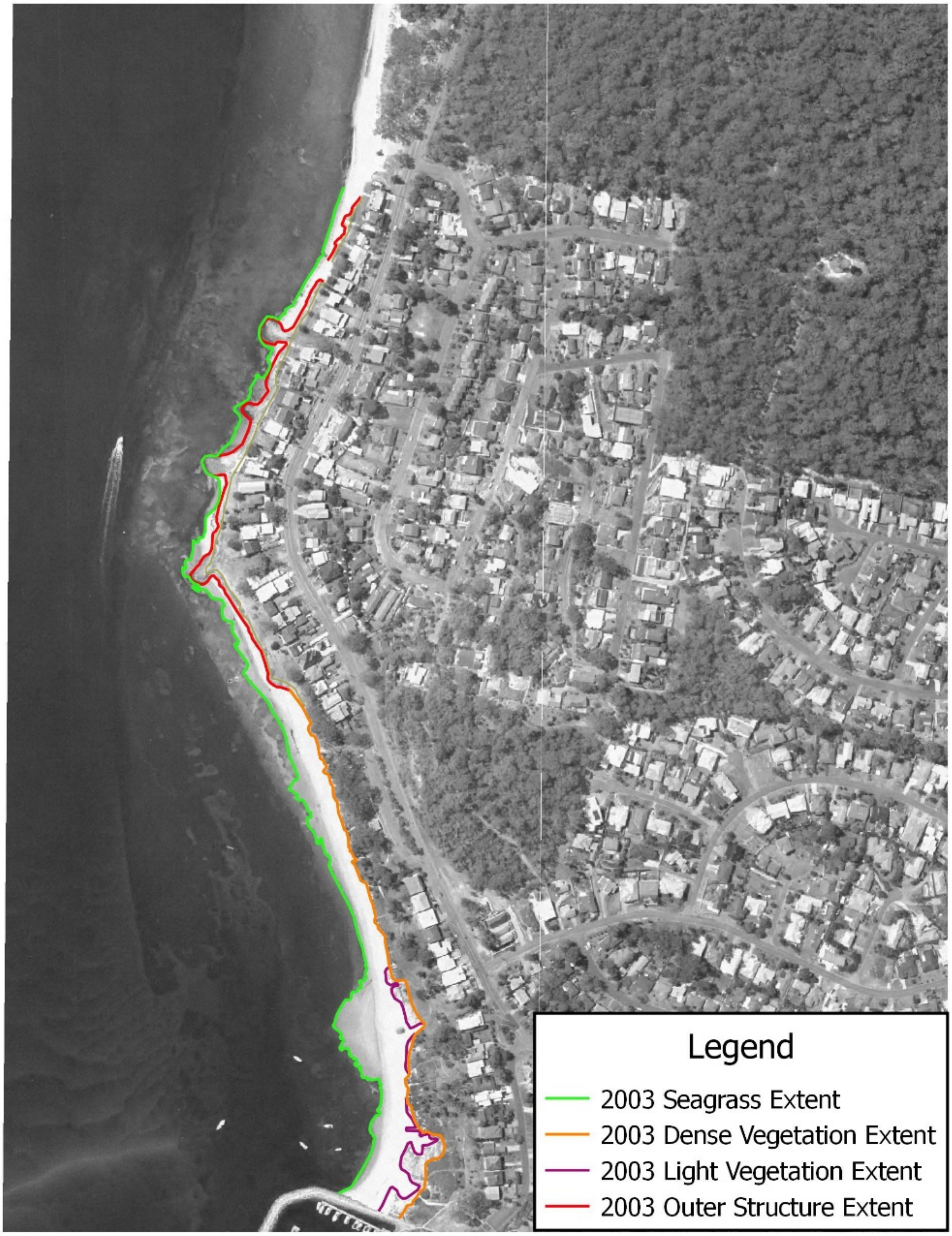


Figure A4: 2003 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

N

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

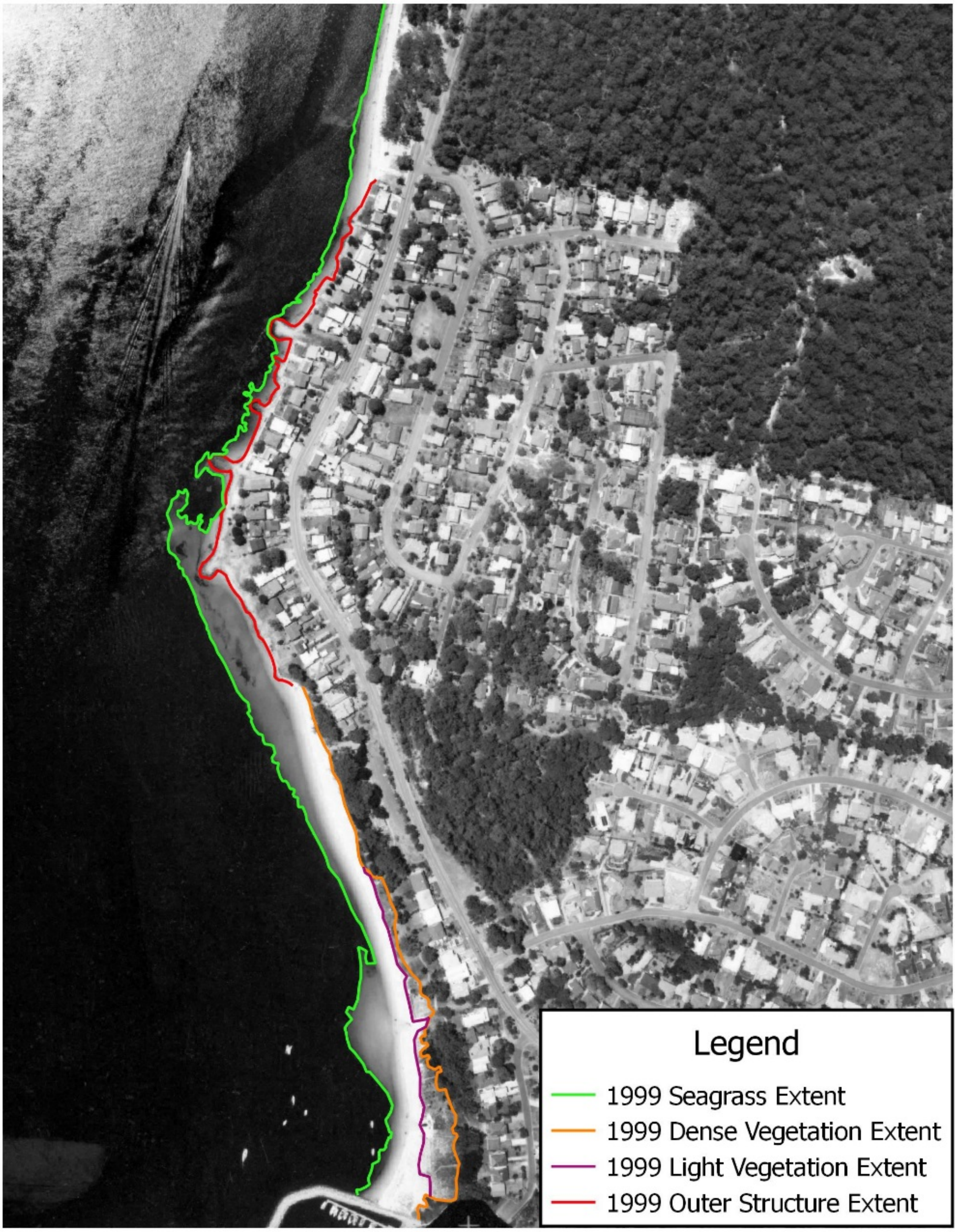


Figure A5: 1999 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

N

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

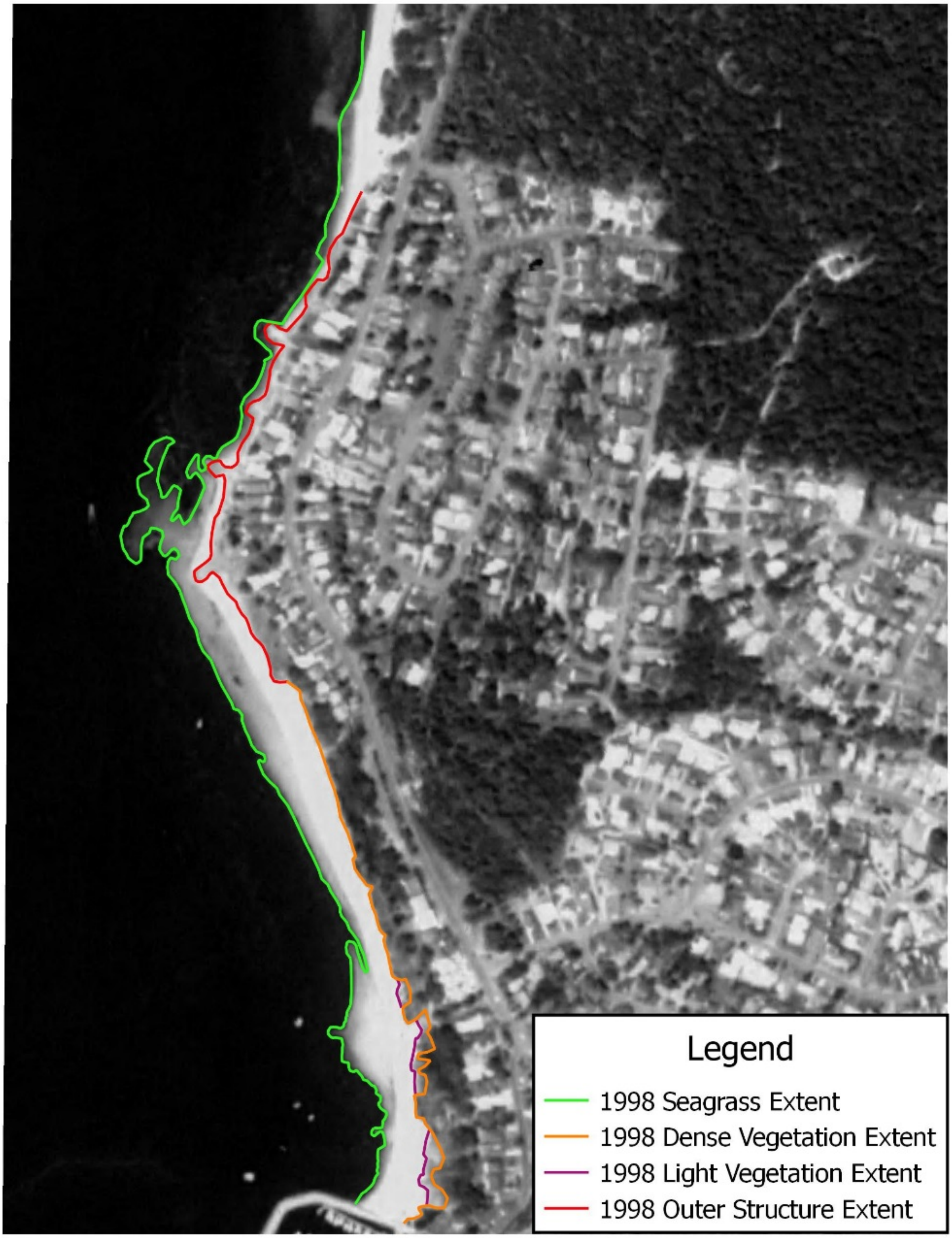


Figure A6: 1998 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

N

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

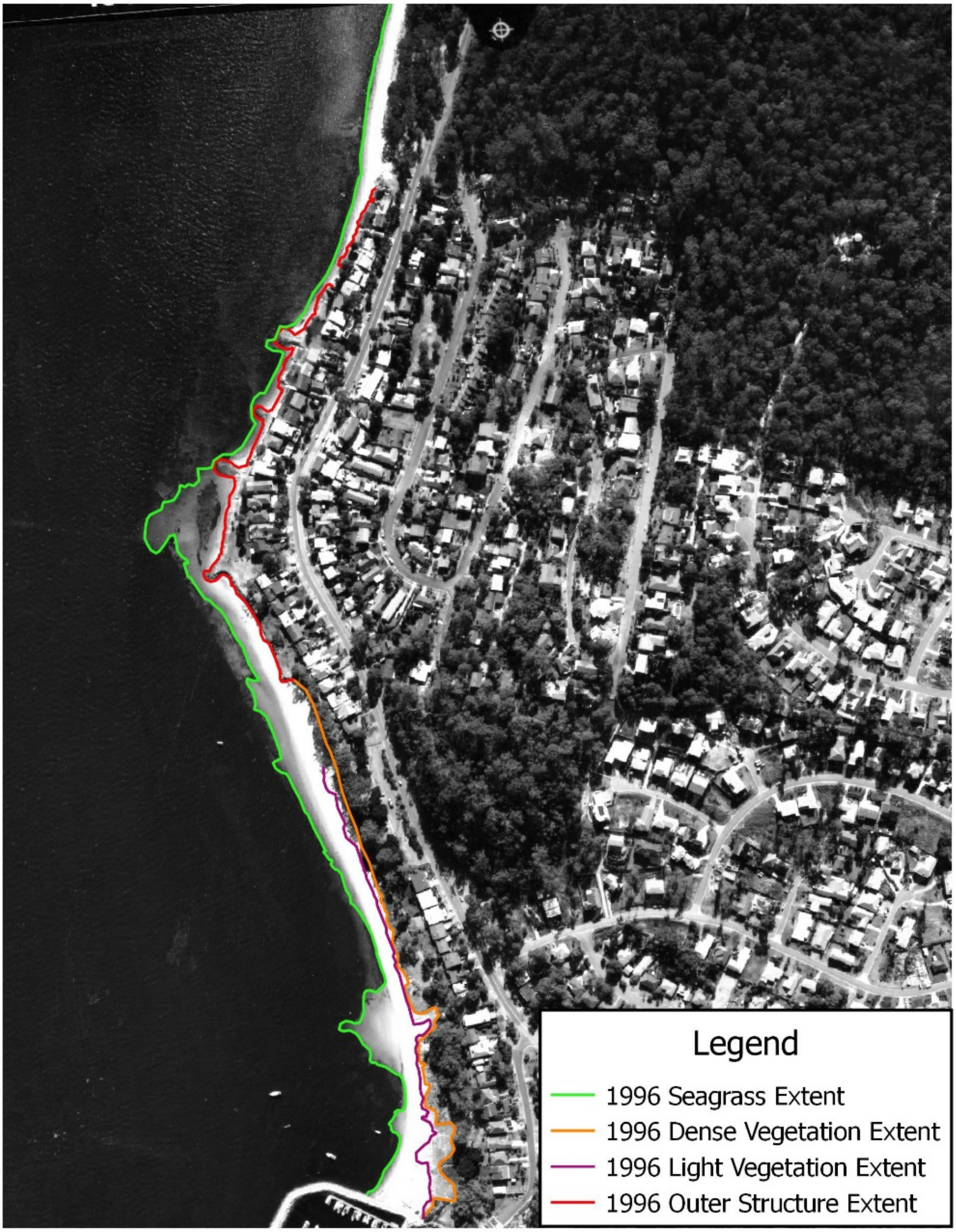


Figure A7: 1996 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

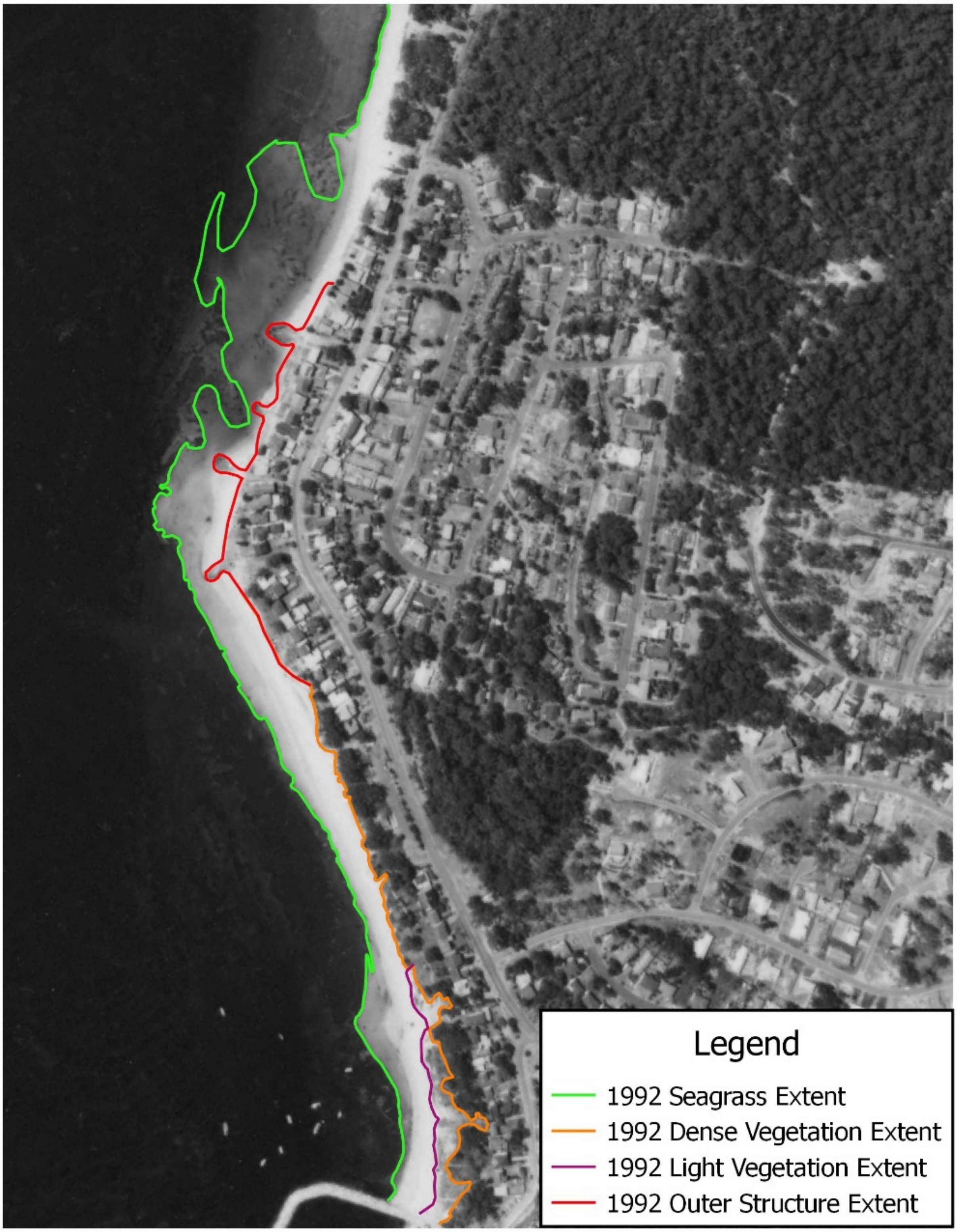


Figure A8: 1992 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

N

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

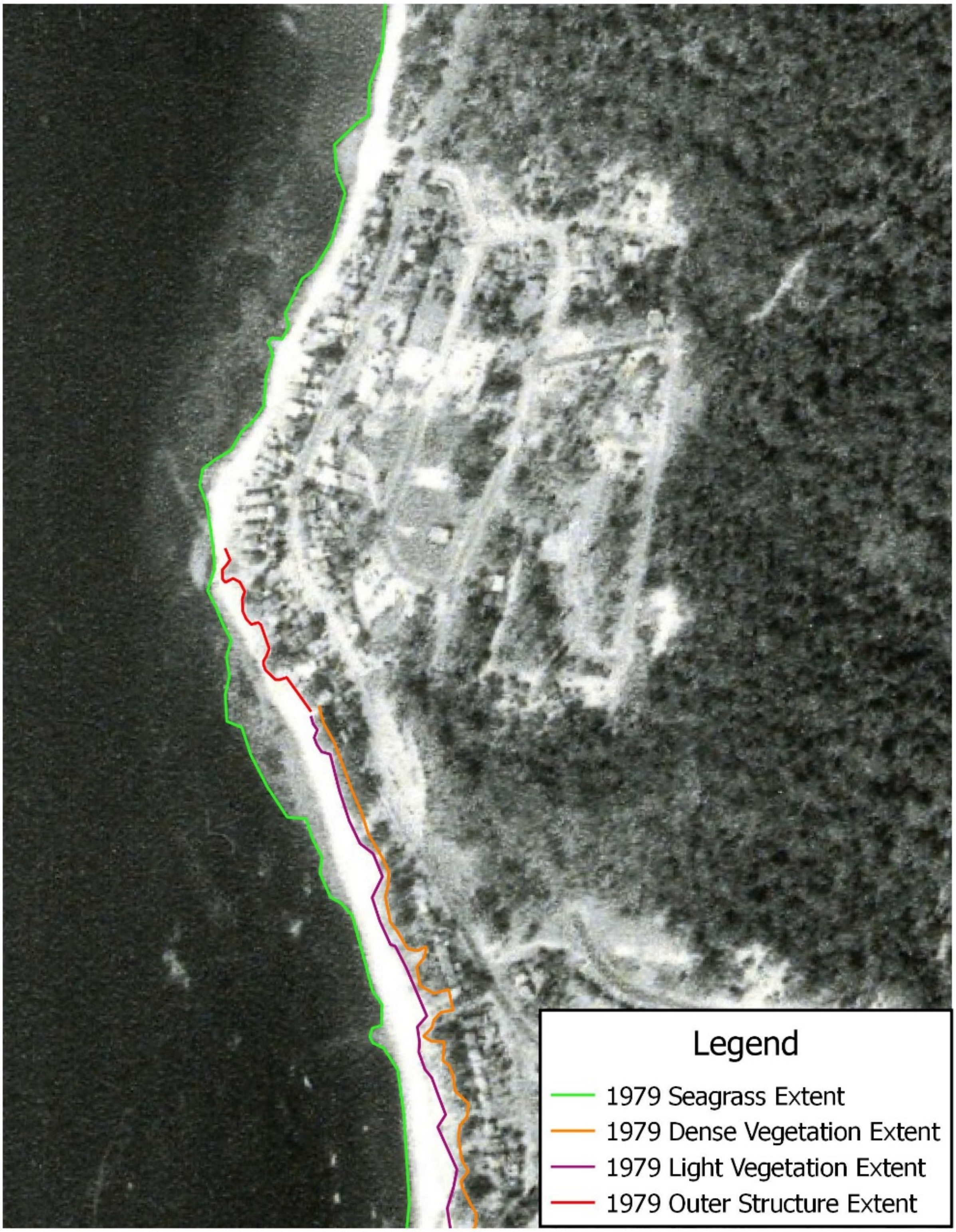


Figure A9: 1979 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

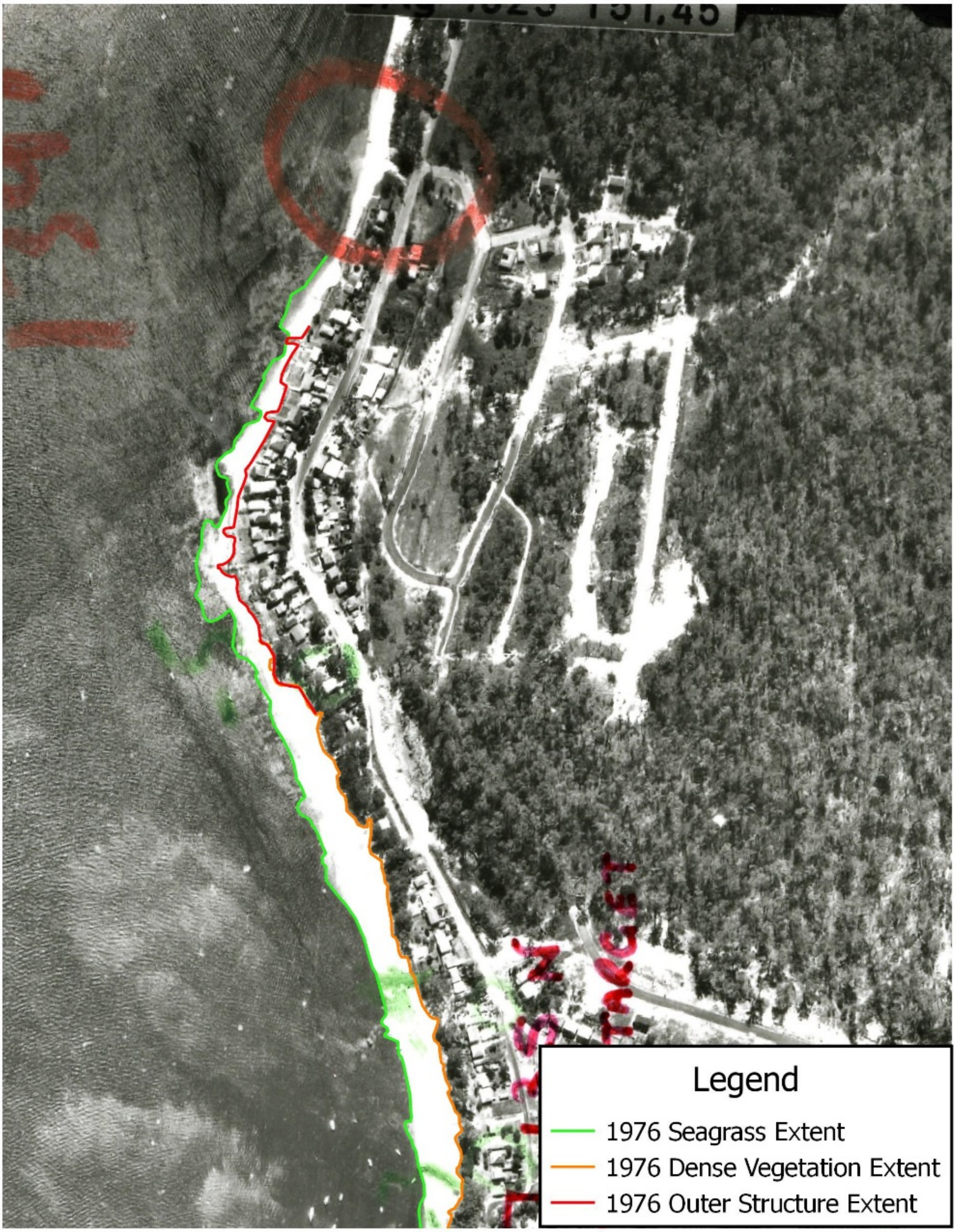


Figure A10: 1976 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

N

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

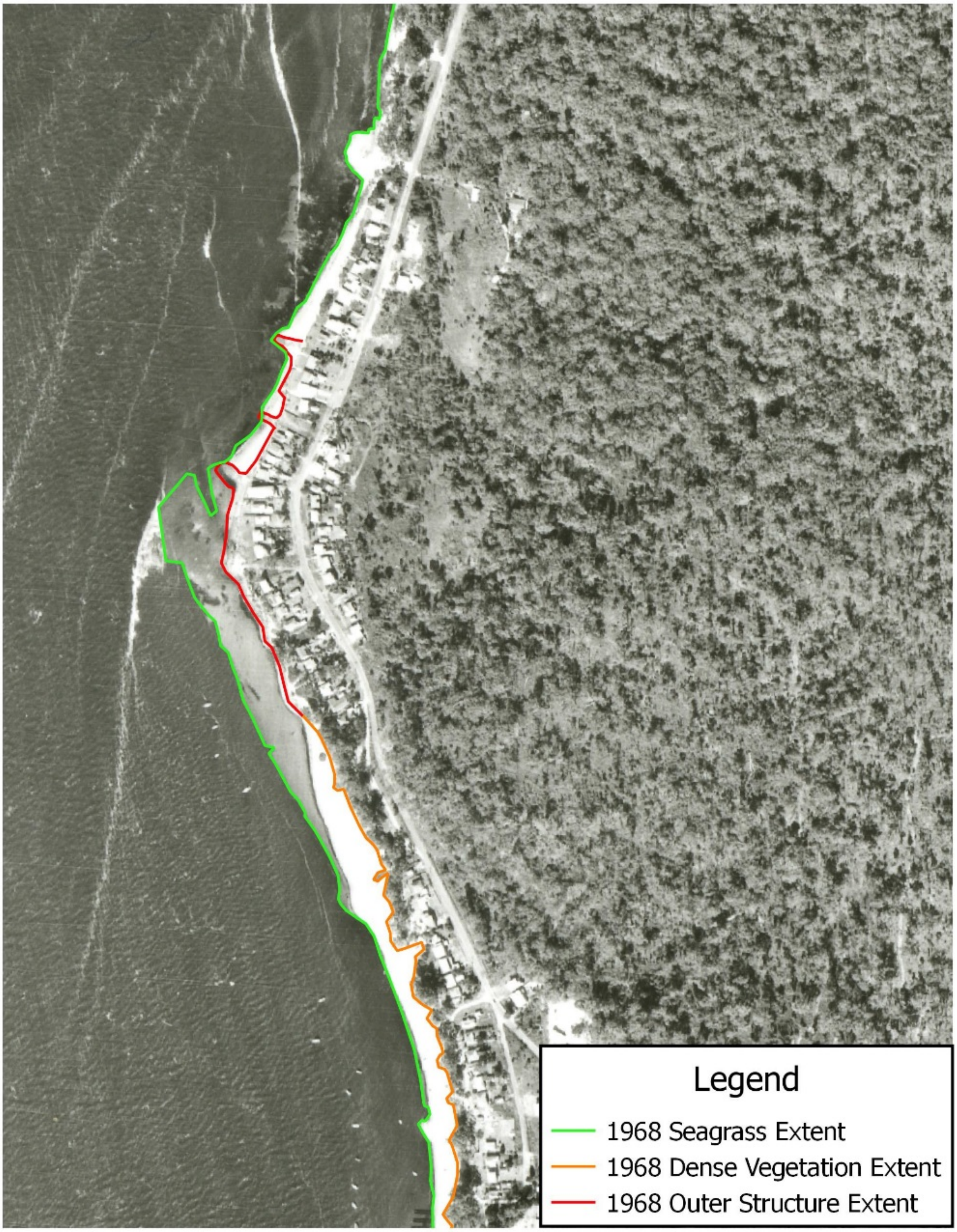


Figure A11: 1968 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Revision	A
Drawn	BC
Approved	DW

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

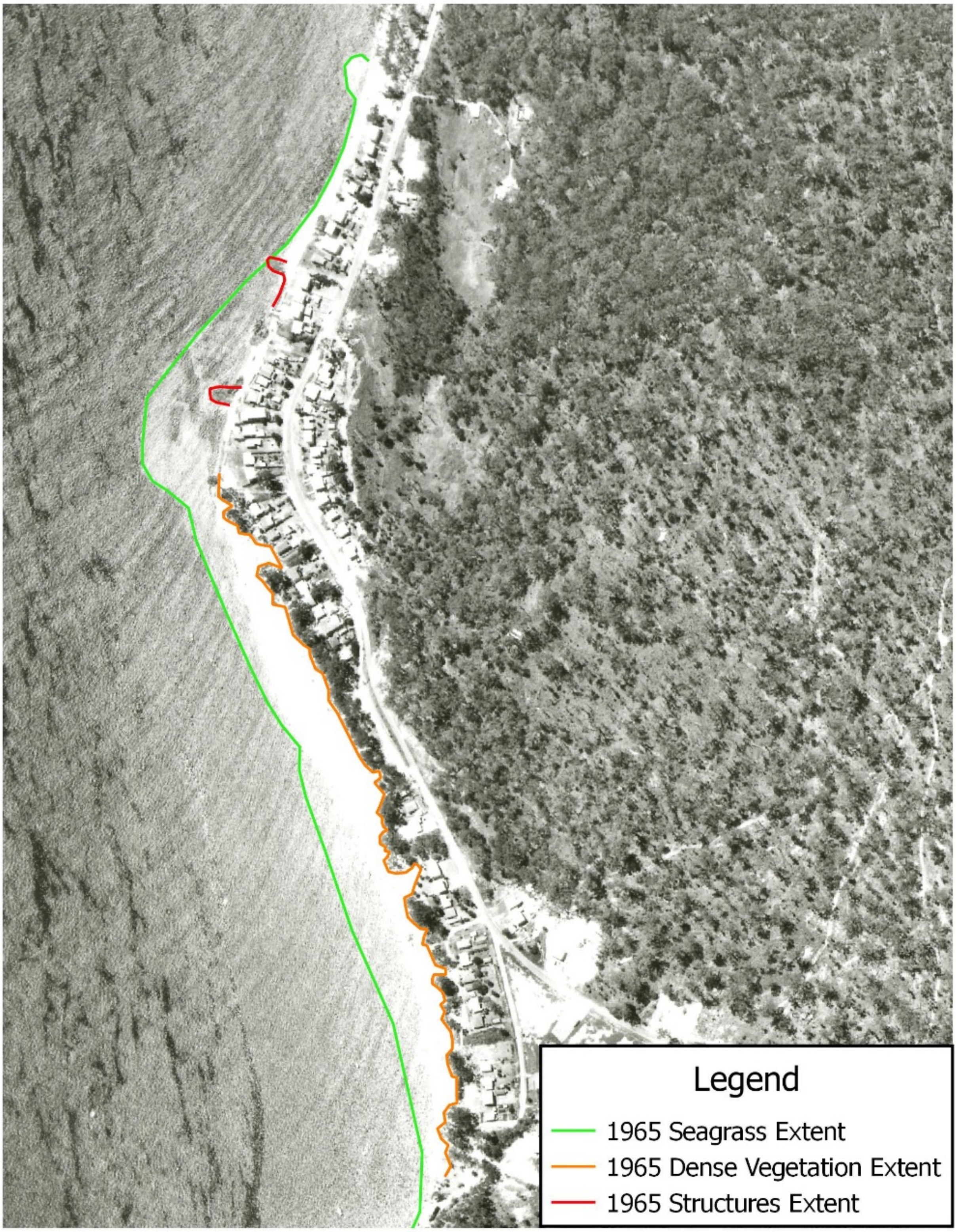


Figure A12: 1965 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

N

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

Figure A13: 1963 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

N

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

Figure A14: 1959 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

N

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

Figure A15: 1952 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

Revision	A
Drawn	BC
Approved	DW

Figure A16: 1951 Shoreline Features

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

0 75 150 225 300 m

(Approx Scale)

N

Revision	A
Drawn	BC
Approved	DW

Appendix B Digital Elevation Models

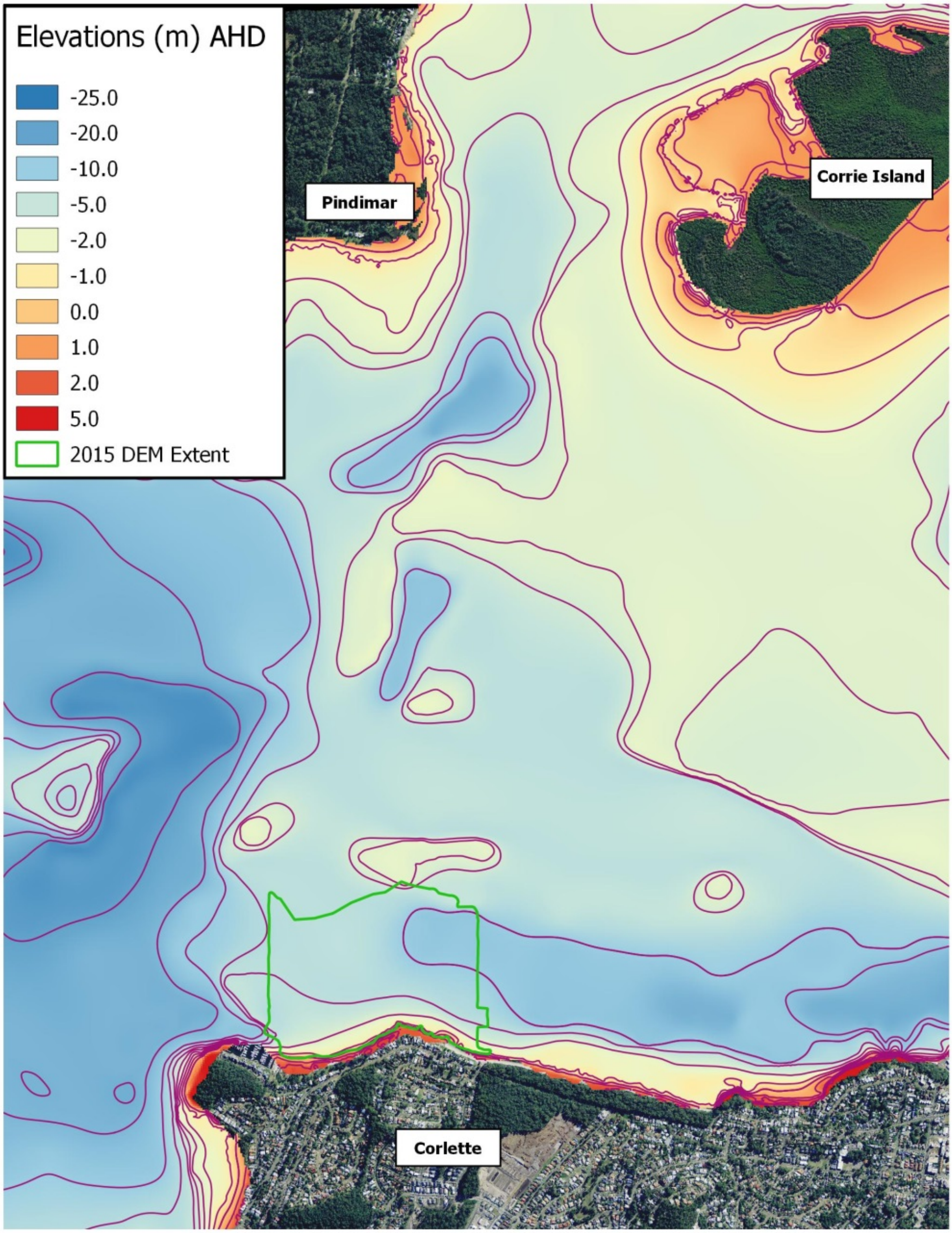


Figure B1: 1969 Corlette Digital Elevation Model

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

0 500 1000 1500 m
(Approx Scale)

N

Revision	A
Drawn	BC
Approved	DW

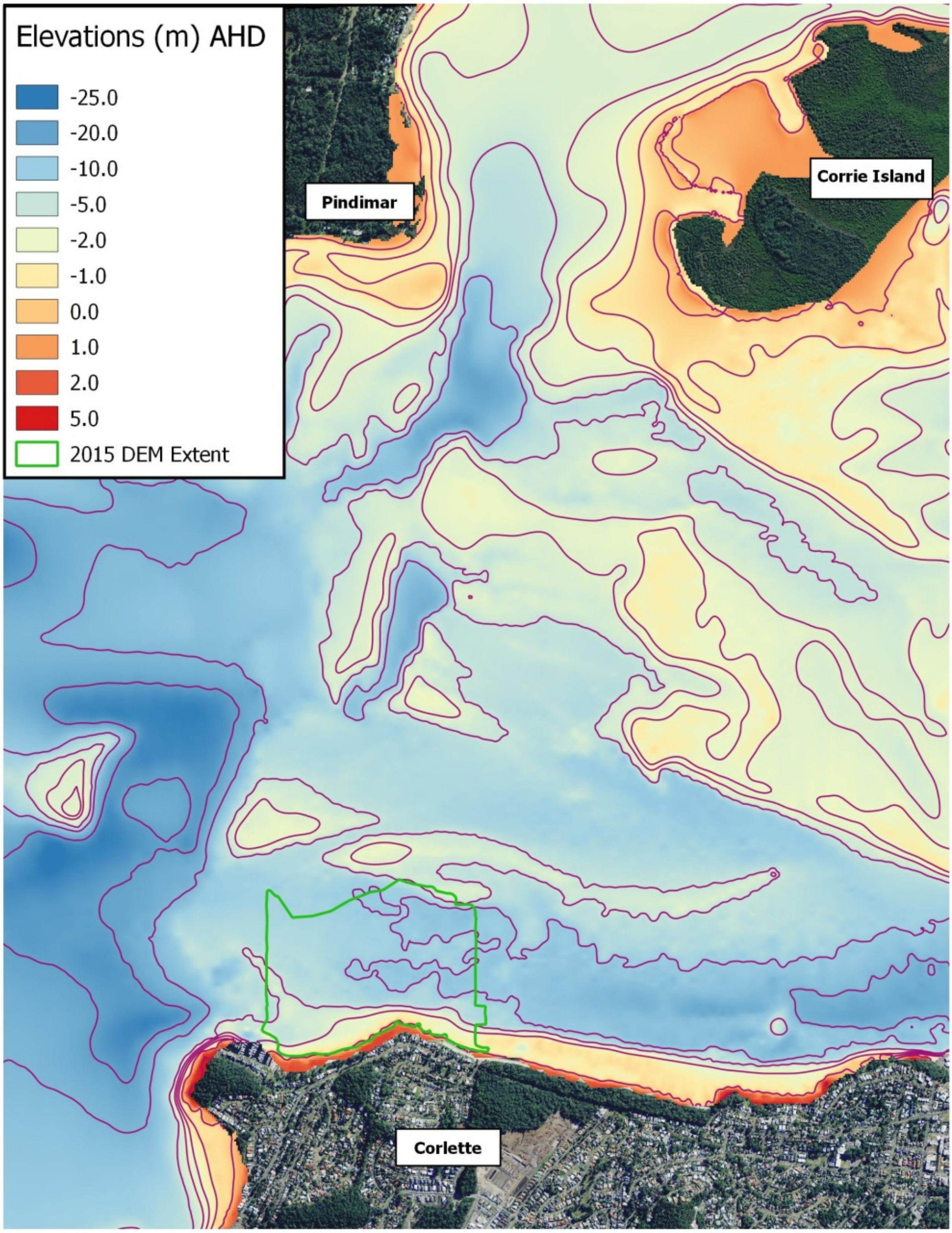


Figure B2: 2007 Corlette Digital Elevation Model

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

0 500 1000 1500 m

(Approx Scale)

Revision	A
Drawn	BC
Approved	DW

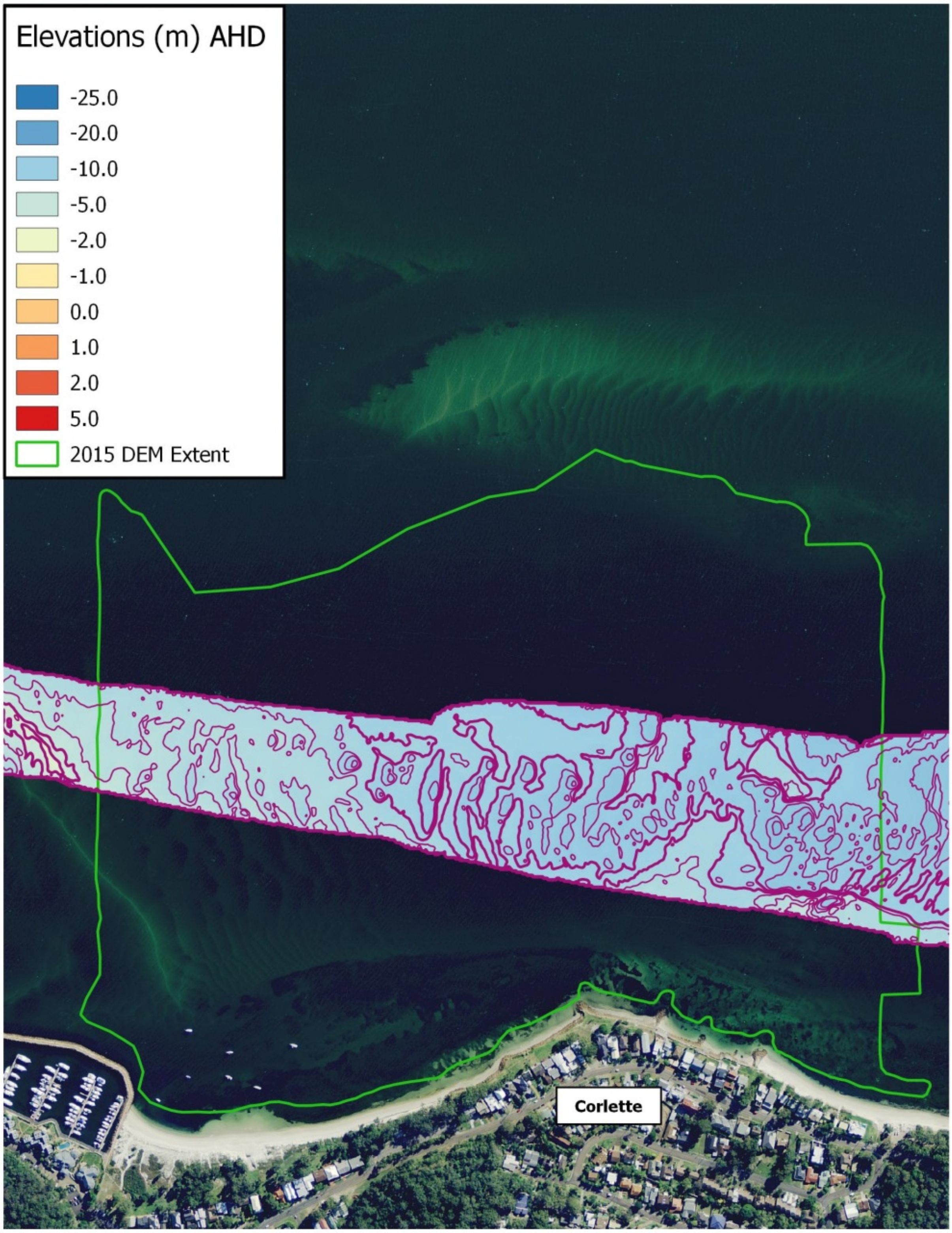


Figure B3: 2011 Corlette Digital Elevation Model

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates
Environmental Consultants

Revision	A
Drawn	BC
Approved	DW

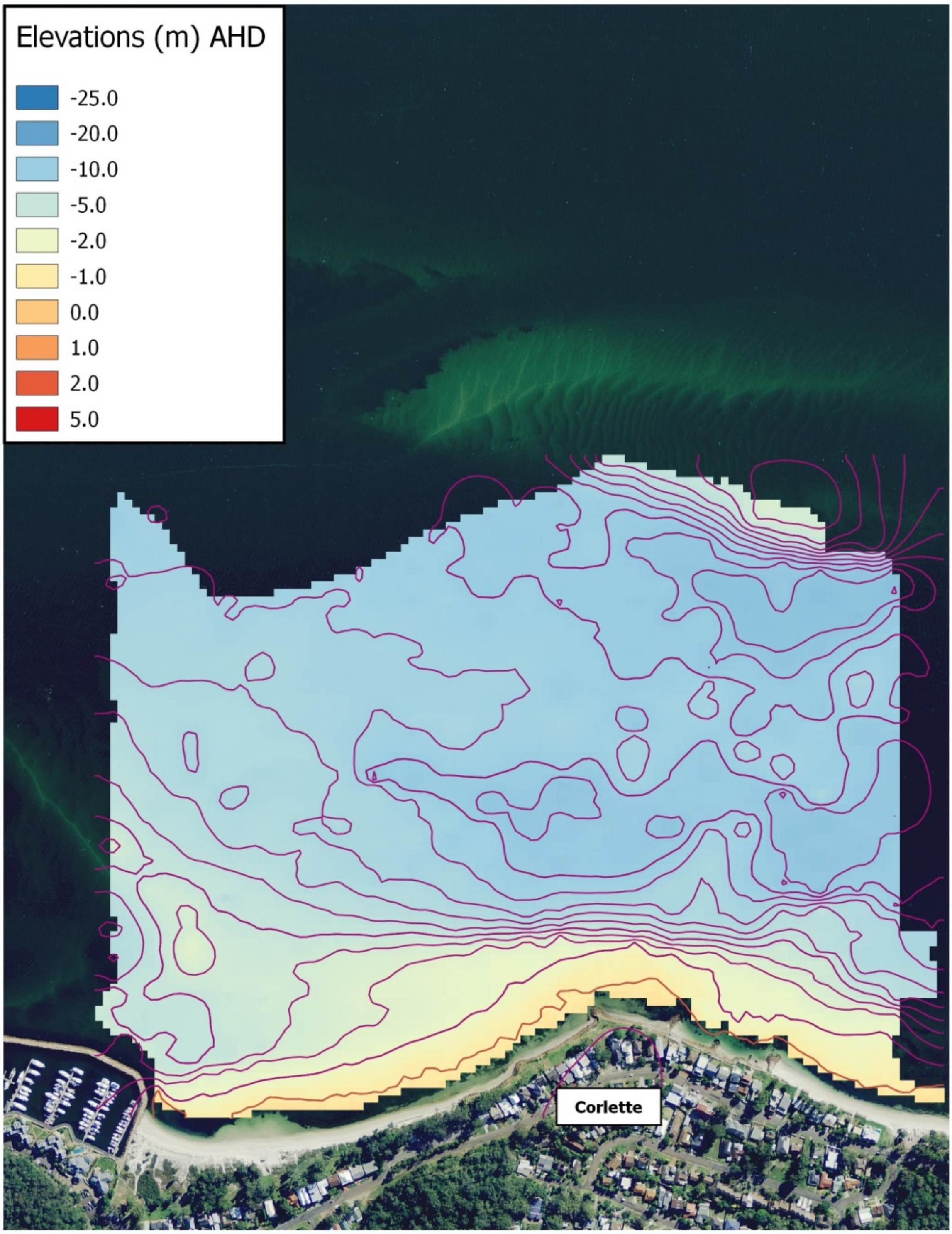


Figure B4: 2015 Corlette Digital Elevation Model

Sandy Point/Conroy Park Foreshore Erosion and Drainage Management Plan

Whitehead & Associates Environmental Consultants	0 100 200 300 400 m (Approx Scale)	Revision	A
		Drawn	BC
		Approved	DW

Appendix C Modelled Design Wave Conditions

Swell Modelling Results

	Swell			Wind			Point 1		Point 2		Point 3		Point 4		Point 5		Point 6		Point 7	
ARI	Hs	Tp	Dir	Spreading	Speed	Dir	Hs	Dir	Hs	Dir	Hs	Dir	Hs	Dir	Hs	Dir	Hs	Dir	Hs	Dir
1	5.9	10.7	130	5	13	130	0.6	77.4	0.6	77.6	0.6	80.0	0.6	80.9	0.6	80.4	0.5	80.1	0.6	81.0
1	5.9	10.7	155	5	13.4	155	0.5	77.9	0.5	78.0	0.5	80.7	0.5	81.7	0.5	81.0	0.5	80.7	0.5	81.8
1	5.9	10.7	180	5	15.1	180	0.4	95.6	0.4	97.0	0.4	98.7	0.4	100.2	0.4	100.1	0.4	102.1	0.4	105.4
Intermediate (≈ 5)	6.7	11.1	130	5	18	130	0.6	75.9	0.6	76.3	0.6	78.5	0.6	79.2	0.6	78.4	0.6	78.2	0.6	79.2
Intermediate (≈ 5)	6.7	11.1	155	5	18.6	155	0.5	77.2	0.5	77.5	0.5	80.1	0.5	81.0	0.5	80.0	0.5	79.9	0.5	81.3
Intermediate (≈ 5)	6.7	11.1	180	5	16.5	180	0.4	72.9	0.4	73.6	0.4	82.3	0.4	83.2	0.4	81.5	0.4	74.7	0.4	82.8
10	7.5	11.5	130	5	14.5	130	0.7	75.3	0.7	75.8	0.7	77.8	0.7	78.6	0.7	77.6	0.7	77.3	0.7	78.7
10	7.5	11.5	155	5	14.9	155	0.6	71.9	0.6	72.2	0.6	79.1	0.6	80.0	0.6	74.2	0.6	73.9	0.6	75.8
10	7.5	11.5	180	5	15.7	180	0.5	70.9	0.5	71.6	0.5	80.4	0.5	81.4	0.5	73.2	0.4	72.2	0.5	80.4
Intermediate (≈50)	8.25	11.9	130	5	20.1	130	0.7	76.3	0.8	76.9	0.8	79.0	0.8	80.0	0.8	79.2	0.7	78.6	0.7	79.5
Intermediate (≈50)	8.25	11.9	155	5	20.8	155	0.7	72.5	0.7	72.8	0.7	74.8	0.7	75.9	0.7	74.9	0.7	74.6	0.7	76.1
Intermediate (≈50)	8.25	11.9	180	5	16.7	180	0.5	69.0	0.5	69.7	0.5	72.2	0.5	79.3	0.5	71.2	0.5	70.2	0.5	72.8
100	9	12.3	130	5	16	130	0.8	77.4	0.8	78.1	0.8	80.1	0.8	80.9	0.8	79.9	0.8	79.3	0.8	80.2
100	9	12.3	155	5	16.3	155	0.7	73.3	0.7	73.7	0.7	73.4	0.7	74.4	0.7	73.3	0.7	73.3	0.7	75.0
100	9	12.3	180	5	16.3	180	0.5	67.5	0.5	68.1	0.5	70.5	0.5	71.9	0.5	69.6	0.5	68.5	0.5	71.2

	Swell			Wind			Point 8		Point 9		Point 10		Point 11		Point 12		Point 13	
ARI	Hs	Tp	Dir	Spreading	Speed	Dir	Hs	Dir	Hs	Dir	Hs	Dir	Hs	Dir	Hs	Dir	Hs	Dir
1	5.9	10.7	130	5	13	130	0.3	110.9	0.5	55.2	0.4	55.2	0.4	56.0	0.4	55.2	0.5	55.2
1	5.9	10.7	155	5	13.4	155	0.3	116.2	0.5	52.6	0.4	55.8	0.4	55.2	0.4	55.8	0.4	66.7
1	5.9	10.7	180	5	15.1	180	0.3	98.6	0.4	52.7	0.3	51.3	0.3	50.1	0.3	51.3	0.4	98.2
Intermediate (≈ 5)	6.7	11.1	130	5	18	130	0.4	108.4	0.5	47.3	0.5	48.1	0.5	48.1	0.5	48.1	0.5	66.0
Intermediate (≈ 5)	6.7	11.1	155	5	18.6	155	0.3	113.5	0.5	46.2	0.4	47.0	0.4	46.6	0.4	47.0	0.5	64.0
Intermediate (≈ 5)	6.7	11.1	180	5	16.5	180	0.4	84.9	0.4	45.7	0.3	45.3	0.3	44.7	0.3	45.3	0.4	62.7
10	7.5	11.5	130	5	14.5	130	0.4	107.0	0.6	44.1	0.5	43.2	0.5	42.8	0.5	43.2	0.6	60.5
10	7.5	11.5	155	5	14.9	155	0.3	120.6	0.5	41.9	0.5	43.3	0.4	40.5	0.5	43.3	0.5	61.2
10	7.5	11.5	180	5	15.7	180	0.4	89.8	0.4	42.0	0.4	41.3	0.4	39.4	0.4	41.3	0.4	59.9
Intermediate (≈50)	8.25	11.9	130	5	20.1	130	0.4	109.0	0.7	44.8	0.6	46.7	0.6	46.4	0.6	46.7	0.6	62.1
Intermediate (≈50)	8.25	11.9	155	5	20.8	155	0.4	107.3	0.6	47.0	0.6	48.7	0.5	48.2	0.6	48.7	0.6	63.4
Intermediate (≈50)	8.25	11.9	180	5	16.7	180	0.4	83.2	0.4	38.4	0.4	38.6	0.4	36.4	0.4	38.6	0.4	57.0
100	9	12.3	130	5	16	130	0.5	107.8	0.7	46.8	0.6	49.0	0.6	47.9	0.6	49.0	0.7	63.0
100	9	12.3	155	5	16.3	155	0.4	117.3	0.7	46.5	0.6	48.0	0.6	47.6	0.6	48.0	0.6	62.0
100	9	12.3	180	5	16.3	180	0.4	88.7	0.5	36.0	0.4	34.1	0.4	34.0	0.4	34.1	0.4	52.5

Wind Wave Modelling Results Points 1 through 7

Degrees	Wind Direction	Wind Speed	ARI	Point 1			Point 2			Point 3			Point 4			Point 5			Point 6			Point 7		
		(m/s)		Hs	Tp	Dir	Hs	Tp	Dir	Hs	Tp	Dir	Hs	Tp	Dir	Hs	Tp	Dir	Hs	Tp	Dir	Hs	Tp	Dir
450	E	14.1	10	0.4	2.3	79.5	0.4	2.5	90.7	0.4	2.5	91.5	0.4	2.5	90.3	0.4	2.4	88.2	0.4	2.4	85.1	0.4	2.5	83.0
427.5	ENE	12.9	10	0.3	2.1	60.9	0.2	1.2	198.7	0.4	2.1	66.0	0.4	2.1	66.2	0.4	2.1	61.6	0.4	2.1	58.2	0.4	2.1	59.4
405	NE	11.9	10	0.3	1.9	340.0	0.2	1.3	229.8	0.3	1.9	337.8	0.3	1.9	336.6	0.3	1.9	337.3	0.3	1.9	338.6	0.3	1.9	339.0
382.5	NNE	11.1	10	0.6	2.4	333.0	0.2	1.4	285.5	0.6	2.4	330.7	0.6	2.4	329.9	0.6	2.4	330.1	0.6	2.4	331.2	0.6	2.4	331.8
360	N	12.9	10	0.6	2.4	326.8	0.3	1.9	319.8	0.6	2.4	321.9	0.6	2.4	319.5	0.6	2.3	320.5	0.5	2.3	320.9	0.5	2.3	320.9
337.5	NNW	15.2	10	0.6	2.5	317.5	0.5	2.3	326.5	0.6	2.6	311.1	0.7	2.6	307.4	0.7	2.5	307.5	0.6	2.5	307.1	0.6	2.5	306.4
315	NW	18.7	10	0.6	2.1	327.8	0.5	1.9	327.8	0.6	2.2	322.0	0.6	2.2	318.8	0.6	2.2	317.7	0.6	2.2	317.8	0.6	2.2	315.2
292.5	WNW	21.6	10	0.6	2.5	308.4	0.6	2.1	321.8	0.7	2.6	301.2	0.7	2.6	298.3	0.7	2.5	298.4	0.7	2.5	297.5	0.7	2.5	297.1
270	W	18.6	10	0.5	2.1	306.8	0.4	1.8	328.6	0.5	2.1	298.1	0.5	2.1	295.4	0.5	2.1	294.6	0.5	2.1	294.5	0.5	2.1	294.6
450	E	15.3	20	0.5	2.7	84.6	0.5	2.7	86.4	0.5	2.7	87.2	0.5	2.8	86.2	0.5	2.7	83.2	0.5	2.7	79.8	0.5	2.7	78.5
427.5	ENE	13.6	20	0.2	1.6	131.1	0.3	1.6	128.2	0.3	1.6	121.5	0.3	1.7	105.0	0.3	1.6	97.3	0.3	1.7	91.6	0.3	1.7	95.8
405	NE	12.8	20	0.2	1.3	229.3	0.2	1.3	231.1	0.2	1.3	230.2	0.2	1.3	230.4	0.2	1.4	232.3	0.2	1.1	247.2	0.2	1.4	231.8
382.5	NNE	12.3	20	0.2	1.6	278.0	0.2	1.4	275.0	0.2	1.6	274.1	0.2	1.6	275.7	0.2	1.6	273.2	0.2	1.6	269.7	0.2	1.6	268.6
360	N	15.2	20	0.4	2.2	338.3	0.4	2.2	336.9	0.4	2.2	335.5	0.4	2.2	334.0	0.4	2.1	335.7	0.4	2.1	337.3	0.4	2.1	338.1
337.5	NNW	16.4	20	0.5	2.4	328.7	0.5	2.4	327.1	0.5	2.4	324.2	0.5	2.4	321.5	0.5	2.3	322.9	0.5	2.2	321.8	0.5	2.2	322.7
315	NW	19.6	20	0.6	2.3	315.0	0.5	1.9	327.9	0.6	2.4	309.5	0.6	2.4	307.5	0.6	2.3	307.2	0.6	2.3	307.5	0.6	2.3	307.9
292.5	WNW	22.5	20	0.7	2.6	309.0	0.6	2.1	323.2	0.7	2.6	301.9	0.7	2.7	298.9	0.7	2.6	299.0	0.7	2.6	298.3	0.7	2.6	297.9
270	W	19.3	20	0.5	2.1	308.1	0.4	1.8	329.0	0.5	2.2	299.3	0.5	2.2	296.3	0.5	2.1	295.6	0.5	2.1	295.4	0.5	2.1	295.4
450	E	15.7	25	0.5	2.7	84.0	0.5	2.7	85.8	0.6	2.8	86.7	0.6	2.8	85.6	0.5	2.7	82.5	0.5	2.7	79.0	0.6	2.8	77.8
427.5	ENE	13.8	25	0.3	1.7	108.0	0.3	1.7	109.1	0.3	1.8	105.1	0.3	1.8	92.3	0.3	1.8	84.6	0.3	1.8	80.1	0.3	1.8	84.0
405	NE	13.1	25	0.2	1.3	227.6	0.2	1.3	231.9	0.2	1.3	231.5	0.2	1.4	232.2	0.2	1.4	232.3	0.2	1.4	232.3	0.2	1.4	231.8
382.5	NNE	12.7	25	0.2	1.6	286.9	0.2	1.5	282.8	0.2	1.6	283.1	0.2	1.7	284.1	0.2	1.6	281.8	0.2	1.6	279.1	0.2	1.6	277.8
360	N	16	25	0.5	2.3	339.7	0.5	2.3	338.4	0.5	2.3	336.9	0.5	2.3	335.7	0.5	2.2	337.7	0.5	2.2	339.3	0.4	2.2	340.4
337.5	NNW	16.7	25	0.5	2.4	328.8	0.5	2.4	327.1	0.5	2.4	324.2	0.5	2.4	321.6	0.5	2.3	323.1	0.5	2.2	321.8	0.5	2.2	322.6
315	NW	19.9	25	0.6	2.3	315.2	0.6	2.5	315.5	0.6	2.4	309.7	0.6	2.4	307.6	0.6	2.4	307.4	0.6	2.3	307.7	0.6	2.3	308.0
292.5	WNW	22.7	25	0.7	2.6	309.2	0.6	2.1	323.4	0.7	2.7	302.2	0.7	2.7	299.1	0.7	2.6	299.2	0.7	2.6	298.5	0.7	2.6	298.1
270	W	19.6	25	0.5	2.1	307.9	0.5	1.8	327.1	0.5	2.2	299.1	0.5	2.2	296.1	0.5	2.2	295.3	0.5	2.1	295.1	0.5	2.1	295.2
450	E	16	30	0.5	2.7	83.7	0.6	2.8	85.5	0.6	2.8	86.4	0.6	2.8	85.3	0.6	2.8	82.0	0.6	2.7	78.5	0.6	2.8	77.4
427.5	ENE	13.9	30	0.3	1.8	101.2	0.3	1.8	103.0	0.3	1.8	99.9	0.3	1.9	88.8	0.3	1.9	81.5	0.3	1.9	77.2	0.3	1.9	80.8
405	NE	13.4	30	0.2	1.2	249.2	0.2	1.2	257.7	0.2	1.2	244.0	0.2	1.2	256.8	0.2	1.2	255.9	0.2	1.2	254.8	0.2	1.2	246.7
382.5	NNE	13	30	0.2	1.6	297.4	0.2	1.5	291.7	0.2	1.6	293.5	0.2	1.7	293.8	0.2	1.7	292.8	0.2	1.6	291.4	0.2	1.7	290.2
360	N	16.7	30	0.5	2.3	340.2	0.5	2.2	334.9	0.5	2.3	337.6	0.5	2.3	336.5	0.5	2.2	338.4	0.5	2.2	339.8	0.5	2.2	340.9
337.5	NNW	17	30	0.5	2.4	328.8	0.5	2.4	327.1	0.5	2.4	324.2	0.5	2.4	321.6	0.5	2.3	323.0	0.5	2.2	321.6	0.5	2.2	322.2
315	NW	20.1	30	0.6	2.4	315.2	0.6	2.5	315.6	0.6	2.4	309.7	0.6	2.4	307.6	0.6	2.4	307.4	0.6	2.4	307.7	0.6	2.3	308.0
292.5	WNW	22.9	30	0.7	2.6	309.3	0.6	2.2	323.4	0.7	2.7	302.4	0.7	2.7	299.3	0.7	2.6	299.3	0.7	2.6	298.6	0.7	2.6	298.2
270	W	19.7	30	0.5	2.1	307.8	0.5	1.8	327.0	0.5	2.2	299.0	0.5	2.2	296.1	0.5	2.2	295.3	0.5	2.1	295.1	0.5	2.1	295.2
450	E	17	50	0.6	2.8	83.3	0.5	2.3	91.1	0.6	2.8	85.9	0.6	2.9	84.7	0.6	2.8	81.4	0.6	2.8	77.8	0.6	2.9	76.7
427.5	ENE	14.4	50	0.4	2.0	85.9	0.4	2.1	87.8	0.4	2.1	86.6	0.4	2.2	80.5	0.4	2.2	75.4	0.4	2.2	71.8	0.4	2.2	73.3
405	NE	14.1	50	0.2	1.5	55.2	0.2	1.6	48.0	0.2	1.6	52.9	0.2	1.7	57.1	0.2	1.6	51.3	0.2	1.6	48.2	0.3	1.7	56.1
382.5	NNE	13.9	50	0.3	1.8	332.3	0.3	1.7	324.7	0.3	1.8	328.4	0.3	1.8	328.5	0.3	1.8	330.7	0.3	1.8	332.5	0.3	1.8	332.8
360	N	18.7	50	0.5	2.3	335.0	0.5	2.2	328.6	0.5	2.3	331.9	0.5	2.3	330.4	0.5	2.2	331.7	0.5	2.2	332.6	0.5	2.2	333.1
337.5	NNW	17.7	50	0.5	2.4	327.8	0.5	2.4	326.2	0.5	2.4	323.3	0.5	2.4	320.7	0.5	2.3	321.8	0.5	2.2	320.3	0.5	2.2	320.9
315	NW	20.6	50	0.6	2.4	315.3	0.6	2.5	315.7	0.6	2.5	309.6	0.6	2.5	307.4	0.6	2.4	307.3	0.6	2.4	307.6	0.6	2.4	307.9
292.5	WNW	23.4	50	0.7	2.6	309.7	0.6	2.2	323.7	0.7	2.7	302.8	0.7	2.7	299.6	0.7	2.6	299.6	0.7	2.6	298.9	0.7	2.6	298.4
270	W	20.2	50	0.5	2.2	306.9	0.5	1.9	325.7	0.5	2.2	298.1	0.5	2.3	295.0	0.5	2.2	294.2	0.5	2.2	294.0	0.5	2.2	293.9
450	E	18.4	100	0.6	2.7	85.0	0.5	2.3	94.6	0.6	2.8	87.7	0.6	2.8	86.6	0.6	2.8	83.1	0.6	2.7	79.5	0.6	2.8	78.9
427.5	ENE	15	100	0.4	2.3	79.5	0.4	2.3	81.5	0.4	2.4	80.9	0.4	2.5	77.6	0.4	2.5	73.3	0.4	2.5	69.7	0.4	2.5	70.0
405	NE	15.2	100	0.3	2.1	60.9	0.3	2.1	63.7	0.4	2.1	66.0	0.4	2.1	66.2	0.4	2.1	61.6	0.4	2.1	58.2	0.4	2.1	59.4
382.5	NNE	15.2	100	0.3	2.0	344.5	0.3	1.9	338.9	0.3	2.0	342.3	0.3	2.0	341.2	0.3	2.0	343.1	0.3	1.9	344.8	0.3	1.9	345.7
360	N	21.8	100	0.6	2.5	338.5	0.6	2.4	331.8	0.6	2.5	335.1	0.6	2.5	333.8	0.6	2.4	335.1	0.6	2.4	336.2	0.6	2.4	336.9
337.5	NNW	18.4	100	0.6	2.4	326.8	0.6	2.4	325.2	0.6	2.4	321.9	0.6	2.4	319.5	0.6	2.3	320.5	0.6	2.2	319.1	0.5	2.2	319.7
315	NW	21.3	100	0.7	2.5	315.4	0.6	2.6	315.7	0.7	2.5	309.7	0.7	2.5	307.4	0.7	2.5	307.4	0.7	2.5	307.8	0.7	2.5	308.0
292.5	WNW	24	100	0.7	2.6	310.0	0.6	2.2	324.1	0.7	2.7	303.1	0.7	2.7	299.9	0.8	2.7	299.7	0.7	2.7	299.1	0.7	2.7	298.6
270	W	20.7	100	0.5	2.2	305.4	0.5	1.9	324.1	0.6	2.3	296.8	0.6	2.3	293.8	0.6	2.3	292.8	0.6	2.2	292.5	0.5	2.2	292.4

Wind Wave Modelling Results Points 8 through 13

Degrees	Wind Direction	Wind Speed	ARI	Point 8			Point 9			Point 10			Point 11			Point 12			Point 13		
		(m/s)		Hs	Tp	Dir	Hs	Tp	Dir	Hs	Tp	Dir	Hs	Tp	Dir	Hs	Tp	Dir	Hs	Tp	Dir
450.0	**E**	**14.1**	**10**	0.5	2.6	82.2	0.4	1.8	69.3	0.3	1.7	77.5	0.3	1.6	80.7	0.4	2.0	55.6	0.4	2.2	65.5
427.5	**ENE**	**12.9**	**10**	0.4	2.2	61.4	0.3	1.9	29.2	0.3	1.8	21.5	0.3	1.8	25.6	0.3	2.0	41.7	0.3	2.0	50.3
405.0	**NE**	**11.9**	**10**	0.3	1.9	338.0	0.3	1.9	337.5	0.3	1.9	338.8	0.3	1.9	341.2	0.3	1.9	341.9	0.3	1.9	341.4
382.5	**NNE**	**11.1**	**10**	0.6	2.4	331.8	0.5	2.3	329.8	0.5	2.3	330.2	0.5	2.3	333.2	0.6	2.3	333.8	0.6	2.4	333.7
360.0	**N**	**12.9**	**10**	0.5	2.3	321.3	0.4	2.0	324.1	0.5	2.0	322.5	0.4	2.0	327.8	0.5	2.1	327.4	0.5	2.2	327.9
337.5	**NNW**	**15.2**	**10**	0.6	2.5	306.5	0.5	2.1	319.3	0.5	2.2	315.6	0.5	2.1	322.0	0.6	2.2	319.0	0.6	2.3	318.9
315.0	**NW**	**18.7**	**10**	0.6	2.2	313.2	0.5	2.1	312.5	0.5	2.1	313.1	0.5	2.1	318.2	0.5	2.1	316.8	0.6	2.1	322.7
292.5	**WNW**	**21.6**	**10**	0.6	2.5	297.3	0.6	2.3	307.1	0.6	2.3	307.1	0.6	2.2	312.9	0.6	2.3	310.3	0.6	2.3	309.2
270.0	**W**	**18.6**	**10**	0.5	2.1	294.5	0.4	2.0	303.1	0.4	1.9	304.5	0.4	1.9	311.6	0.4	1.9	309.6	0.4	1.9	308.2
450.0	**E**	**15.3**	**20**	0.6	2.8	78.1	0.4	1.9	61.8	0.4	1.8	65.1	0.4	1.8	65.3	0.4	1.9	67.9	0.4	2.4	72.0
427.5	**ENE**	**13.6**	**20**	0.3	1.7	97.9	0.2	1.5	27.9	0.2	1.3	173.9	0.2	1.2	203.1	0.2	1.5	118.6	0.3	1.5	126.0
405.0	**NE**	**12.8**	**20**	0.2	1.4	230.9	0.2	1.2	235.7	0.2	1.2	235.2	0.2	1.2	234.1	0.2	1.2	226.7	0.2	1.2	226.8
382.5	**NNE**	**12.3**	**20**	0.2	1.6	267.9	0.2	1.4	273.3	0.2	1.3	273.9	0.2	1.3	278.3	0.2	1.3	273.8	0.2	1.3	278.3
360.0	**N**	**15.2**	**20**	0.4	2.1	338.0	0.4	2.0	332.5	0.4	2.0	333.6	0.4	2.0	336.2	0.4	2.0	336.4	0.4	2.1	340.2
337.5	**NNW**	**16.4**	**20**	0.5	2.2	323.1	0.5	2.1	323.5	0.5	2.1	324.2	0.5	2.1	328.3	0.5	2.1	327.7	0.5	2.1	327.2
315.0	**NW**	**19.6**	**20**	0.6	2.3	308.1	0.6	2.2	313.0	0.6	2.2	313.4	0.5	2.1	318.5	0.6	2.2	317.1	0.6	2.2	316.4
292.5	**WNW**	**22.5**	**20**	0.7	2.6	297.9	0.6	2.4	308.0	0.6	2.4	307.8	0.6	2.3	313.6	0.6	2.3	311.0	0.6	2.4	310.1
270.0	**W**	**19.3**	**20**	0.5	2.1	295.4	0.5	2.0	304.4	0.5	1.9	305.6	0.4	1.9	312.7	0.4	1.9	310.7	0.5	2.0	309.2
450.0	**E**	**15.7**	**25**	0.6	2.8	77.5	0.4	2.0	59.0	0.4	1.8	62.5	0.4	1.8	62.0	0.4	2.0	66.0	0.4	2.4	70.8
427.5	**ENE**	**13.8**	**25**	0.3	1.9	87.0	0.2	1.6	43.2	0.2	1.5	25.7	0.2	1.5	34.0	0.3	1.6	91.8	0.3	1.6	105.0
405.0	**NE**	**13.1**	**25**	0.2	1.4	230.9	0.2	1.3	236.1	0.2	1.2	235.9	0.2	1.2	235.6	0.2	1.2	227.5	0.2	1.2	227.0
382.5	**NNE**	**12.7**	**25**	0.2	1.7	277.1	0.2	1.4	280.8	0.2	1.4	281.7	0.2	1.3	286.6	0.2	1.4	283.2	0.2	1.4	288.4
360.0	**N**	**16.0**	**25**	0.4	2.2	340.3	0.4	2.1	333.8	0.4	2.1	334.7	0.4	2.1	337.2	0.4	2.1	337.5	0.4	2.1	341.5
337.5	**NNW**	**16.7**	**25**	0.5	2.2	323.0	0.5	2.1	323.5	0.5	2.1	324.2	0.5	2.1	328.2	0.5	2.1	327.7	0.5	2.2	327.2
315.0	**NW**	**19.9**	**25**	0.6	2.3	308.2	0.6	2.2	313.3	0.6	2.2	313.6	0.6	2.1	318.7	0.6	2.2	317.1	0.6	2.2	316.4
292.5	**WNW**	**22.7**	**25**	0.7	2.6	298.1	0.6	2.4	308.1	0.6	2.4	308.0	0.6	2.3	313.7	0.6	2.4	311.2	0.6	2.4	310.4
270.0	**W**	**19.6**	**25**	0.5	2.1	295.3	0.5	2.0	304.3	0.5	2.0	305.4	0.4	1.9	312.6	0.5	1.9	310.6	0.5	2.0	309.2
450.0	**E**	**16.0**	**30**	0.6	2.9	77.1	0.4	2.0	57.8	0.4	1.9	60.4	0.4	1.9	59.6	0.4	2.3	65.1	0.5	2.5	70.1
427.5	**ENE**	**13.9**	**30**	0.3	2.0	83.7	0.2	1.6	44.3	0.2	1.6	30.0	0.2	1.6	35.5	0.3	1.6	83.4	0.3	1.6	98.2
405.0	**NE**	**13.4**	**30**	0.2	1.2	238.8	0.2	1.2	271.5	0.2	1.2	236.2	0.2	1.2	239.8	0.2	1.2	229.4	0.2	1.2	228.4
382.5	**NNE**	**13.0**	**30**	0.2	1.7	289.1	0.2	1.5	289.7	0.2	1.4	291.2	0.2	1.4	295.9	0.2	1.4	294.9	0.2	1.4	300.5
360.0	**N**	**16.7**	**30**	0.5	2.2	340.9	0.4	2.1	333.8	0.4	2.1	334.5	0.4	2.1	337.0	0.4	2.1	337.5	0.5	2.2	337.3
337.5	**NNW**	**17.0**	**30**	0.5	2.2	322.7	0.5	2.1	323.3	0.5	2.1	323.8	0.5	2.1	328.1	0.5	2.2	327.4	0.5	2.2	327.0
315.0	**NW**	**20.1**	**30**	0.6	2.3	308.3	0.6	2.2	313.4	0.6	2.2	313.7	0.6	2.1	318.8	0.6	2.2	317.3	0.6	2.2	316.6
292.5	**WNW**	**22.9**	**30**	0.7	2.6	298.2	0.6	2.4	308.4	0.6	2.4	308.2	0.6	2.3	313.9	0.6	2.4	311.4	0.6	2.4	310.6
270.0	**W**	**19.7**	**30**	0.5	2.1	295.2	0.5	2.0	304.3	0.5	2.0	305.4	0.4	1.9	312.4	0.5	2.0	310.5	0.5	2.0	309.1
450.0	**E**	**17.0**	**50**	0.6	2.9	76.4	0.4	2.1	56.3	0.4	2.0	55.2	0.4	2.0	55.2	0.4	2.3	63.8	0.5	2.5	69.4
427.5	**ENE**	**14.4**	**50**	0.4	2.3	75.1	0.3	1.9	46.2	0.3	1.8	37.5	0.3	1.8	38.8	0.3	1.8	64.1	0.3	1.8	76.2
405.0	**NE**	**14.1**	**50**	0.3	1.7	63.0	0.2	1.5	5.5	0.2	1.5	352.8	0.2	1.5	1.5	0.2	1.5	30.1	0.2	1.6	43.1
382.5	**NNE**	**13.9**	**50**	0.3	1.8	331.8	0.2	1.6	323.6	0.2	1.6	325.5	0.2	1.6	329.0	0.3	1.6	328.9	0.3	1.7	336.0
360.0	**N**	**18.7**	**50**	0.5	2.2	332.8	0.5	2.0	327.0	0.5	2.0	327.9	0.5	2.0	330.8	0.5	2.1	331.1	0.5	2.1	330.9
337.5	**NNW**	**17.7**	**50**	0.5	2.2	321.4	0.5	2.1	322.2	0.5	2.1	322.7	0.5	2.1	327.0	0.5	2.2	326.5	0.5	2.2	326.1
315.0	**NW**	**20.6**	**50**	0.6	2.4	308.2	0.6	2.2	313.7	0.6	2.2	313.9	0.6	2.2	319.1	0.6	2.2	317.5	0.6	2.3	316.8
292.5	**WNW**	**23.4**	**50**	0.7	2.6	298.5	0.6	2.4	308.8	0.6	2.4	308.6	0.6	2.3	314.3	0.6	2.4	311.6	0.6	2.4	310.8
270.0	**W**	**20.2**	**50**	0.5	2.2	294.1	0.5	2.0	303.7	0.5	2.0	304.5	0.5	1.9	311.6	0.5	2.0	309.4	0.5	2.0	308.0
450.0	**E**	**18.4**	**100**	0.6	2.8	79.3	0.4	2.1	59.2	0.4	2.0	57.5	0.4	2.0	56.8	0.4	2.3	65.8	0.5	2.5	71.4
427.5	**ENE**	**15.0**	**100**	0.5	2.5	71.5	0.3	2.1	45.2	0.3	2.0	38.8	0.3	2.0	39.7	0.4	2.0	55.6	0.4	2.2	65.5
405.0	**NE**	**15.2**	**100**	0.4	2.2	61.4	0.3	1.9	29.2	0.3	1.8	21.5	0.3	1.8	25.6	0.3	2.0	41.7	0.3	2.0	50.3
382.5	**NNE**	**15.2**	**100**	0.3	1.9	345.4	0.3	1.9	337.5	0.3	1.9	338.8	0.3	1.9	341.2	0.3	1.8	344.5	0.3	1.9	346.7
360.0	**N**	**21.8**	**100**	0.6	2.4	336.8	0.5	2.3	329.8	0.5	2.3	330.2	0.5	2.3	333.2	0.6	2.3	333.8	0.6	2.4	333.7
337.5	**NNW**	**18.4**	**100**	0.5	2.2	320.2	0.5	2.1	320.9	0.5	2.1	321.4	0.5	2.1	325.8	0.5	2.1	325.3	0.5	2.2	325.0
315.0	**NW**	**21.3**	**100**	0.7	2.5	308.2	0.6	2.3	314.1	0.6	2.3	314.2	0.6	2.2	319.4	0.6	2.3	317.8	0.6	2.3	317.1
292.5	**WNW**	**24.0**	**100**	0.7	2.7	298.6	0.6	2.4	309.2	0.7	2.4	308.9	0.6	2.3	314.6	0.7	2.4	311.9	0.7	2.4	311.2
270.0	**W**	**20.7**	**100**	0.5	2.2	292.6	0.5	2.1	302.5	0.5	2.1	303.2	0.5	2.0	310.0	0.5	2.0	307.7	0.5	2.1	306.3

Appendix B Drainage Processes Study

Stormwater Drainage and Water Quality Assessment

for Sandy Point/Conroy Park Foreshore

Prepared by: Jason Armstrong & Mark Passfield

SEEC Reference: 15000047-SWMP-04

1st April 2016

Strategic Environmental & Engineering Consulting

Strategic Environmental and Engineering Consulting

PO Box 1098, Bowral NSW 2576
telephone: (02) 4862 1633 • fax: (02) 4862 3088 • email: reception@seec.com.au

Document Certification

This report has been developed based on agreed requirements as understood by SEEC at the time of investigation. It applies only to a specific task on the nominated lands. Other interpretations should not be made, including changes in scale or application to other projects.
Any recommendations contained in this report are based on an honest appraisal of the opportunities and constraints that existed at the site at the time of investigation, subject to the limited scope and resources available. Within the confines of the above statements and to the best of my knowledge, this report does not contain any incomplete or misleading information.

Jason Armstrong & Mark Passfield
SEEC
1st April 2016

Issue	Author	Change Notes	Authorised	Date
01	JA, MP	Draft Issue	M.P	29th June 2015
02	JA, MP	Final Draft	M.P	25th August 2015
03	JA, MP	Revised Draft	M.P	28th January 2016
04	JA, MP	Final Issue	M.P	1st April 2016

TABLE OF CONTENTS

1 Introduction

SEEC has been commissioned by Whitehead and Associates on behalf of Port Stephens Council to provide this Stormwater Drainage and Water Quality Assessment.

This study has been prepared in accordance with the guidelines and recommendations set out in ***Australian Rainfall & Runoff (1998***) and ***Port Stephens Council Infrastructure Specification (2006).***

This assessment outlines the procedures used to determine storm flows for the 5 year *'minor'* and 100 year *'major'* ARI storm events throughout each catchment contributing flows to the five existing drainage outfalls located along the Sandy Point to Conroy Park foreshore. It also estimates mean annual flow volumes and sediment and pollutant loads from the total catchment at each of the five outfall locations.

This assessment is based on a desktop study of upstream catchment areas. The accuracy of this assessment is controlled by the level of detail obtainable from the desktop study and a visual site inspection. Any results, assumptions or conclusions provided within this report are suitable only for the purpose of assessing the five outfalls and should not be used for any other reason.

2 Project Description

This assessment is part of a project to undertake an Erosion and Drainage Management Plan of the Sandy Point to Conroy Park Foreshore Area. It will be used as input into a management plan for the holistic sustainable management and protection of the foreshore, its homes, Conroy Park Beach and the immediate aquatic environment. Council's objectives for the study area are to balance the public access and recreational amenity needs of the community with the environmental values of the area and the protection of private assets.

3 Site Description

3.1 Location and General Topographic Situation

The study area is located within the township of Corlette (Figure 1). Corlette is located along the southern foreshore of Port Stephens. The terrain within the catchments of the study area ranges from moderately to very steep, but the foreshore area itself is almost flat.

Figure 1 – Site Location & Catchment Plan

3.2 Catchment Area Description

The study area contains three main catchments areas (Figure 1) that are described as follows:

3.2.1 *Catchment CA1*

This catchment has an area of 29 Ha with the majority of this catchment consisting of approximately 85% urban land with some small areas of approximately 4 Ha made up of steep terrain and parkland that are undeveloped. Grades range from 2 – 5% slope within the foreshore area and rise to between 15 – 25% heading back towards the escarpment.

3.2.2 *Catchment CA2*

This catchment is similar to catchment CA1 and has an area of 29.4 Ha. The majority of this catchment consists of approximately 87% urban land with some small areas of approximately 3.8 Ha made up of steep terrain and parkland that are undeveloped. Grades are also similar to catchment CA1 and range from 2 – 5% slope within the foreshore area and rise to between 15 – 25% heading back towards the escarpment.

3.2.3 *Catchment CA3*

This is the smallest catchment and has an area of 4.3 Ha located mainly around the foreshore area. Approximately 77% of the catchment is urban with the remaining areas consisting of foreshore reserve area and a steeper undeveloped reserve located to the west. This has similar grades to catchment CA1 & CA2, relatively flat with grades ranging from 2 – 8% along the foreshore and with grades of 15 – 30% grade at the steeper sections.

3.3 Existing Stormwater Discharge Locations

There are five main stormwater discharge locations located within the study area. These are shown on Figure 1 as Outfalls 1 to 5. A description of each is given below.

3.3.1 *Outfall 1*

Outfall 1 is located within catchment CA1. It consists of a surcharge pit and overland flow path across the beach reserve (Photo 1). It is located at a lowpoint within Sandy Point Road and serves as a relief point during localised flooding of Sandy Point Road during large storm events. The surcharge path requires regular maintenance for it to work effectively and it had recently been cleaned out prior to our inspection.

Photo 1 – Looking south towards surcharge pit at Outfall 1 location.

3.3.2 *Outfall* 2

This is a major discharge point located within catchment CA1. It consists of two, 1200mm diameter reinforced concrete pipes under a rock groyne wall (Photo 2).

Photo 2 - Outfall 2 Looking north

3.3.3 *Outfall 3*

Outfall 3 is located within catchment CA2. It is a major discharge point consisting of three 1200mm diameter concrete pipes discharging directly onto Corlette Beach (Photo 3). The pit immediately upstream of the outlet contains a surcharge outlet, surcharging larger flows during major storms into an existing concrete apron.

Photo 3 – Outfall 3 Looking South East

3.3.4 *Outfalls 4 & 5*

Both of these outfalls are located next 'The Anchorage' Marina and within catchment CA3. Outlet 4 is a 600mm diameter reinforced concrete pipe and was completely blocked during the site inspection (Photo 4).

Outfall 5 is a 375mm reinforced concrete pipe located under the eastern breakwater of the marina. This was partial blocked during our site inspection (Photo 5) and was missing a flap valve.

Photo 4 – Outfall 4, completely blocked.

Photo 5 – Outfall 5, partially blocked with flap valve removed.

4 Hydrological Modelling

4.1 Design Parameters

A hydrological model of the study area was developed using *DRAINS* urban stormwater drainage modelling software. *DRAINS* uses the *ILSAX* hydrological model. The design data used to develop the model was taken from the following information.

4.1.1 *ILSAX Model Data*

The following parameters were used within the ILSAX model.

Parameter	Value
Impervious Area Depression Storage (mm)	1
Supplementary Area Depression Storage (mm)	1
Pervious Area Depression Storage (mm)	5
Soil Type	2 (Moderate Infiltration Rate)
AMC (Antecedent Moisture Condition)	3

4.1.2 *Rainfall Data*

The Intensity Frequency Duration (IFD) rainfall data for the site was produced from the 'Bureau of Meteorology's Rainfall IFD Data System' which is based on data presented in Australian Rainfall and Runoff (1987) Book 2 for Corlette (Appendix 1). This information was input into the *DRAINS* model.

4.1.3 *GIS Data*

GIS information supplied by Port Stephens Council was imported into *Autocad Civil 3D.* This data included existing contour levels (0.5m intervals), lot boundaries and existing stormwater drainage pits and pipe locations and sizes. Pit depths at the start, major intersection points and at the ends were checked during a site inspection of the study area.

A three-dimensional model of each of the pipe networks was developed to represent the existing drainage infrastructure using *Advanced Road Design,* which is an add-on application for *Civil 3D*. There are four stormwater drainage pipe networks:

(i) **Network 1** in Catchment CA1 contains drainage Outfalls 1 and 2;
(ii) **Network 2** in Catchment CA2 contains Outfall 3;
(iii) **Network 3** in Catchment CA3 contains Outfall 4; and
(iv) **Network 4** in Catchment CA3 contains Outfall 5.

4.1.4 *Sub-Catchment Areas*

Each of the catchments described in Section 3.2 were broken down into sub-catchment areas to each of the existing stormwater pits using Autocad. The areas, slope lengths and grades were exported into the *DRAINS* model using the *Advanced Road Design* software. Impervious area within the urbans areas of the catchment was set at 60% in accordance with Port Stephens Council's *Handbook for Drainage Criteria Section D5.06* for Zone 2a – normal residential zoned land.

4.1.5 *Pit Blockage Factors*

Pit blockage factors of 50% for sag pits and 20% for pits on-grade were specified in the DRAINS model in accordance with Council's *Infrastructure Design Specification – D5 Stormwater Drainage Table D5.2.* Note that outlets 4 and 5 where completed blocked with sand at the time of inspection.

4.1.6 *Overland Flow Paths*

Overland flow paths from surcharging pits were modelled using a typical roadway cross-section for surcharge paths along roads and with a generic cross-section for surcharge paths along property easements. Slope lengths were taken directly from the DTM during the software transfer process from *Civil 3D* into *DRAINS.*

4.2 Stormwater Modelling Results

4.2.1 *Resultant Flows at Outfalls*

The resultant flows from the *DRAINS* modeling for the peak (worst case) 5-year and 100-year ARI storm events at each of the stormwater outfalls previously described in Section 3.3 are shown below in Tables 1 and 2. They show the total flows for each catchment broken into pipes flow and overland flow along the surcharge path, including the storm duration.

Table 1 - 5 Year ARI (Minor System) Results

Outfall No.	5 Year ARI –Pipe (m^3/s)	Pipe Flow Peak Storm Duration (mins)	5 Year ARI Overland Flow (m^3/s)	Overland Flow Peak Storm Duration (mins)	Total Flowrate (m^3/s)	Max. Velocity (m/s)
1	0.328	25	0	25	0.328	0.328
2	3.28	25	0.405	60	3.685	3.26
3	4.2	60	0.007	60	4.207	3.27
4	0.096	60	0	60	0.096	1.32
5	0.114	25	0	25	0.114	2.17

Table 2 - 100 Year ARI (Major System) Results

Outfall No.	100 Year ARI –Pipe (m^3/s)	Pipe Flow Peak Storm Duration (mins)	100 Year ARI Overland Flow (m^3/s)	Overland Flow Peak Storm Duration (mins)	Total Flowrate (m^3/s)	Max. Velocity (m/s)
1	0.641	60	0.679	120	1.32	0.59
2	4.19	60	1.12	60	5.31	3.48
3	5.52	45	0.193	90	5.713	3.44
4	0.321	60	0.02	60	0.341	1.81
5	0.231	20	0.134	25	0.365	2.50

4.2.2 *Stormwater Pipe Capacity*

Overland (surcharge) flow quantities and their locations are shown in Appendices 2 to 5 and referenced on Drawing 15000047_P01_SWMP01 (Appendix 6). These are the numbers shown in red and are shown for both the 5 year and 100 year ARI storm events in m^3/s. There are numerous pits that surcharge during a 5-year storm event which shows the existing piped stormwater system is significantly under-sized when compared to current Council and Australian Standards. These locations have also been identified on drawing 15000047_P01_SWMP01 (Appendix 6).

5 Stormwater Quality Modelling

5.1 Introduction

The estimated sediment and pollutant loads are modelled using MUSIC (Model for Urban Stormwater Improvement Conceptualisation), developed by eWater. The model is appropriately set up using inputs as in Tables 3, 4 and 5. Statistics are produced in MUSIC for the following parameters:

- Flow (ML/yr)
- TSS - Total Suspended Solids (kg/yr)
- TP - Total Phosphorus (kg/yr)
- TN - Total Nitrogen (kg/yr)
- Gross Pollutants (kg/yr)

5.2 Climate Data

Creation of a MUSIC catchment file requires an associated meteorological data file. In this case data provided by Port Stephens Council via *MUSIC LINK* has been used. The data file used was the *"default catchment, sandy soils, Williamtown RAAF"*. Rainfall and evapotranspiration statistics are in Table 3 and a time-series graph is in Figure 2.

Table 3 - Rainfall and PET statistics

Meteorological Data Statistics

	Rainfall/6 Minutes	Evapo-Transpiration
mean	0.012	3.817
median	0.000	3.270
maximum	10.570	5.870
minimum	0.000	1.770
10 percentile	0.000	1.770
90 percentile	0.000	5.840

	Rainfall	Evapo-Transpiration
mean annual	1013	1394

Close Print...

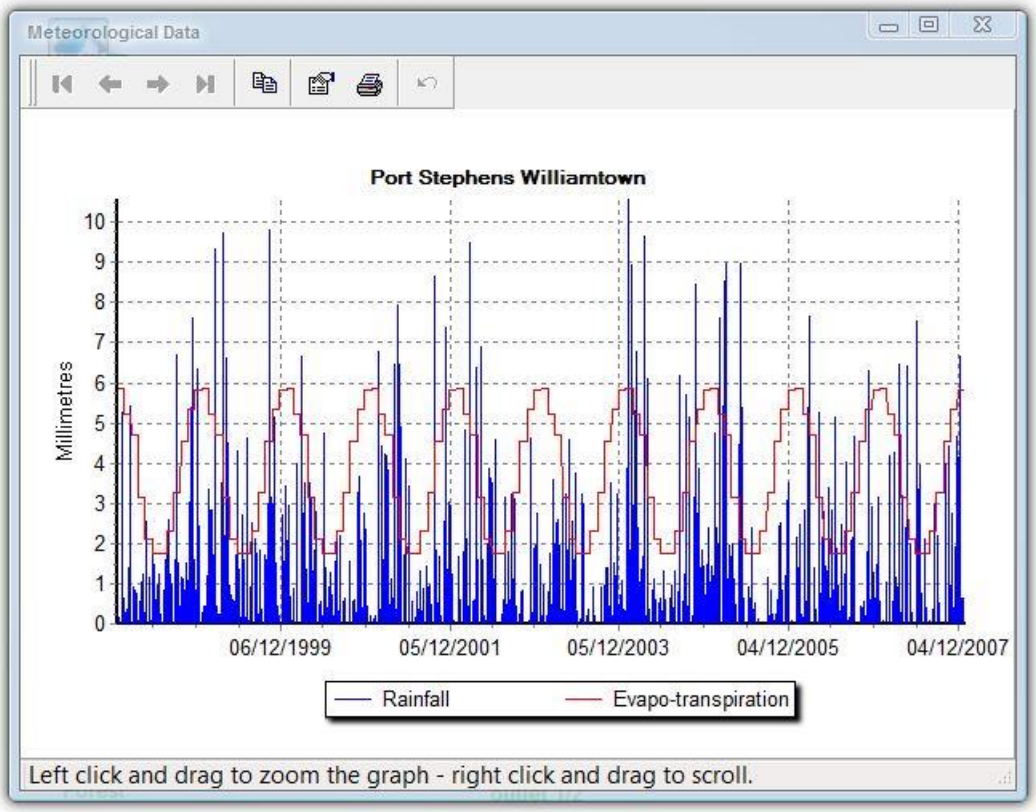


Figure 2 - Rainfall and PET Time Series Graph

5.2.1 Node Parameters

Table 4 presents the storm flow concentration parameters for the MUSIC model. They are derived from SMCMA (2010).

Table 4 - Storm flow concentration parameters used in MUSIC

	TSS mean (log mean)	TSS std dev (log std dev)	TP mean (log mean)	TP std dev (log std dev)	TN mean (log mean)	TN std dev (log std dev)
Urban Land	141 (2.15)	2.09 (0.32)	0.251 (-0.6)	1.78 (0.25)	2 (0.3)	1.55 (0.19)
Forest	39.8 (1.6)	1.58 (0.2)	0.08 (-1.1)	1.66 (0.22)	0.89 (-0.05)	1.74 (0.24)

The pervious area parameters for both pre and post modelling are given in Table 5. They are based on the method described in Section 3.6.3 of SMCMA (2010), see also Section 5.2.2.

Table 5 - Pervious area parameters used in MUSIC

Parameter	Value
Soil storage capacity	170
Initial storage	30
Field capacity	70
Infiltration capacity coefficient	210
Infiltration capacity exponent	4.7
Groundwater initial depth	10
Daily recharge rate	50
Daily base flow rate	5
Daily deep seepage rate	0

5.2.2 Catchment Hydrology Check

To check the model's hydrological calibration the outflow from a calibration node with 55% *effective* imperviousness[1] was checked against the Annual Runoff Fraction (Figure 3). The model's annual rainfall is 1,013 mm so the annual runoff fraction should be about 0.6 which equals 6.08 ML/ha/yr. The calibration node's actual runoff is 6.02 ML/y which is within 1%.

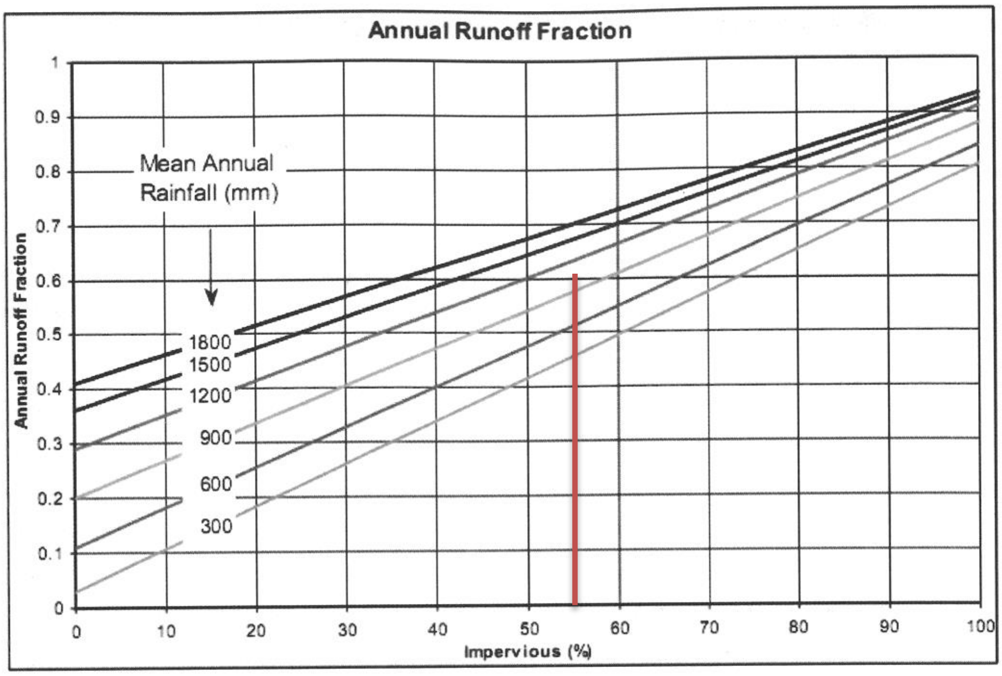


Figure 3 - Annual Runoff Fraction

[1] Reference: Table 3-3 in SMCMA, 2010. Effective imperviousness is different to actual imperviousness as it only accounts for impervious surfaces that are directly connected to the stormwater system

5.3 Model Input

The MUSIC model is divided into the five piped-networks as detailed in Section 3.3. Table 6 gives the breakup of each catchment which are divided into urbanised land and reserves (modelled with a forest source node).

Table 6 – Catchment Areas

	Catchment Area	Forest (ha) (100% pervious)	Urban (ha) (55% impervious)
Outlet 1	3.24	3	0.24
Outlet 2	25.29	-	25.29
Outlet 3	27.5	3	24.5
Outlet 4	1.66	-	1.66
Outlet 5	1.29	0.79	0.5
Totals	59.11	8.11	51

The MUSIC model schematic is shown in Figure 4.

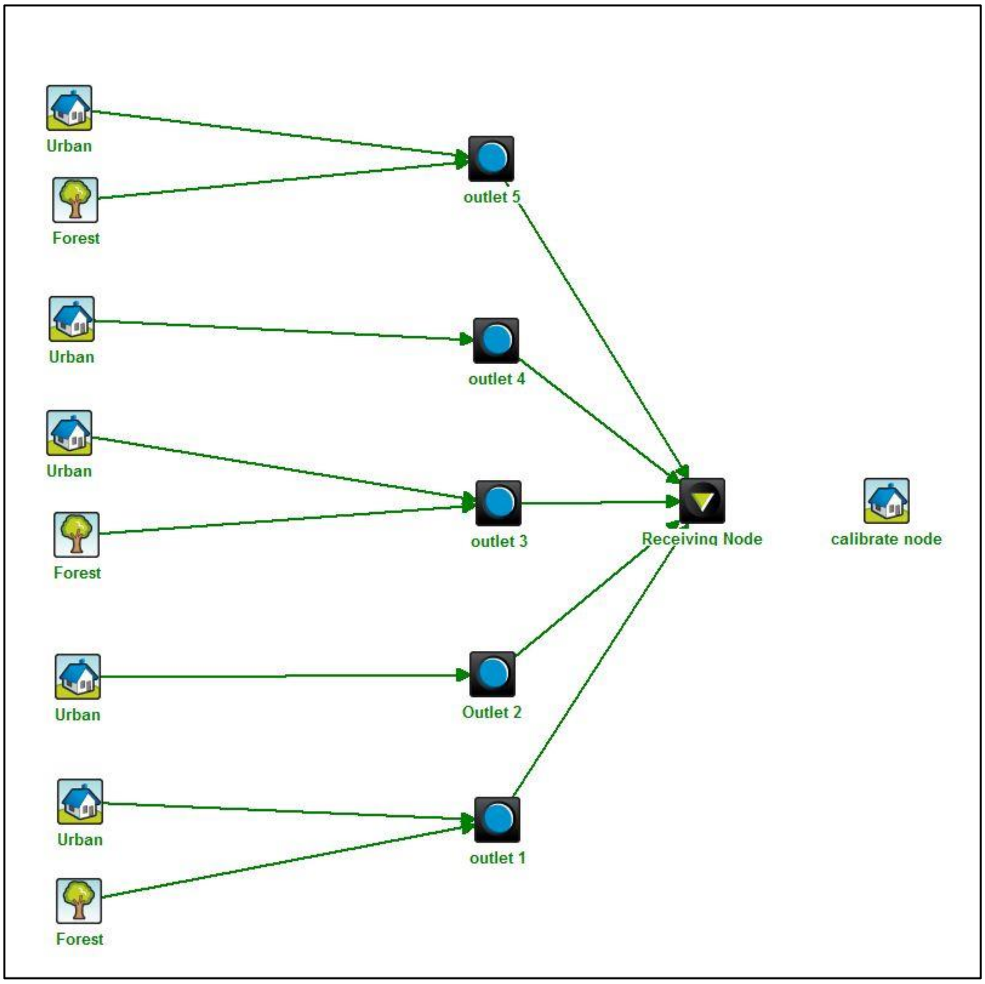


Figure 4 – MUSIC Model Schematic

5.4 MUSIC Results

The results of the modelling are given on Table 7.

Table 7 – Estimated Mean Annual Flow and Pollutant Loads

	Outlet 1	Outlet 2	Outlet 3	Outlet 4	Outlet 5	Total
Mean Annual Flow (ML/y)	11.6	152	158	10	5.7	337.3 ML/y
Total Suspended Solids (kg)	332	23800	24300	1630	511	50573 kg/y
Total Phosphorous (kg)	2.97	271	270	18.4	5.6	567.97 kg/y
Total Nitrogen (kg)	10.7	314	313	20.8	8.2	666.7 kg/y
Gross Pollutants (kg)	41.4	4360	4230	286	86.3	9003.7 kg/y

6 Stormwater Outfall Management

Part of this assessment is to determine which stormwater outfalls would benefit from upgrading works to help reduce the amount of erosion that is occurring along the foreshore and, in turn, help reduce maintenance costs in the long term. The outfalls that would benefit from some engineering works are discussed below.

6.1 Outfall 1 – Catchment CA1

6.1.1 *Design Considerations*

Outfall 1 as described in Section 3.3.1 is part of Network 1 as identified within the *DRAINS* model. As previously discussed it serves as a relief/surcharge point within catchment CA1 to relieve some of the localised flooding along Sandy Point Road during large storm events; therefore its removal would not be an option.

The main issue with Outfall 1 is backing up of sand into the overflow channel from wave action and tidal surges during large storms events, which blocks off the overland flow path. It is therefore critical that Council regularly clean-out and maintain this overflow channel. Another issue is that, being a surcharge pit, it and the pipe system it serves are always charged (full of stormwater). Therefore, the pit regularly surcharges stormwater into the beach reserve adding to erosion problems.

Based on the resultant flows from the *DRAINS* analysis, the combined 5 Year and 100 Year flows from this outfall only represent 20 percent of the total flow from catchment CA1 (Tables 1 & 2). Considering this, and also due to existing pipe invert levels, it would be impractical to retro-fit a gross pollution trap (GPT) to reduce gross pollutants. However, two practical options for upgrading this outfall are discussed in the following.

6.1.2 *Design Options*

(i) <u>Option 1</u> – Fill in and regrade the existing overflow channel. Construct a rock lined swale drain from the surcharge outlet down to the beach front. Topsoil and revegetate either side of newly lined swale drain. Refer to drawing 15000047_P01_SK01 in Appendix 7 for details.

(ii) <u>Option 2</u> – As for option 1 above with the addition of a rock-filled filtration trench located under the swale drain. Weep holes would then be installed through the outlet wall of the surcharge pit into the filtration trench to help reduce the water level in the upstream piped drainage system. A sump and trash screen would also have to be installed around the weep holes to help reduce the chance of blockage. Refer to drawing 15000047_P01_SK02 (Appendix 7) for details.

6.2 Outfall 2 – Catchment CA1

6.2.1 *Design Considerations*

This is one of two major outfalls within the foreshore. As previous described in section 3.3.2 it is located under an existing groyne and discharges directly into Port Stephens. It is currently working effectively and was unblocked during the time of inspection and did not seem to be contributing to any local erosion. The *DRAINS* analysis shows this outlet to be under sized for the 5 Year storm event (Tables 1 & 2) with surcharging of the system evident upstream in Sandy Point Road. Although being undersized, it would be impractical and costly at this stage to try and augment the existing piped drainage system.

Being the major discharge point from catchment CA1, Outfall 2 carries a considerable amount of suspended sediment and pollutants from the upstream urban areas. This is summarised in Table 7. An option to resolve this is discussed below.

6.2.2 *Gross Pollution Trap*

This outlet would benefit from retro-fitting two Gross Pollutant Traps (GPTs) upstream of the outlet within the existing Council reserve. Twin Humeguard HG40B GPTs (one for each 1200mm diameter outlet) were modelled in MUSIC with the predicted pollutant reductions shown in Table 8. The Humeguard was chosen due to its efficiency working in high tailwater conditions. Refer to drawing 1500047_P01_SK03 (Appendix 7) for details.

Table 8 – Outfall 2 - Mean Annual Pollutant Load Reductions

Mean Annual Loads - Generic GPT

	Inflow	Outflow	% Reduction
Flow (ML/yr)	152	152	0.0
Total Suspended Solids (kg/yr)	24.2E3	14.4E3	40.6
Total Phosphorus (kg/yr)	278	218	21.5
Total Nitrogen (kg/yr)	316	281	11.3
Gross Pollutants (kg/yr)	4.36E3	476	89.1

6.3 Outfall 3 – Catchment CA2

6.3.1 *Design Considerations*

This is the second major outfall and is located in the centre of Corlette Beach. The outfall discharges across a substantial width of beach causing significant erosion and loss of sand from the beach front. The *DRAINS* analysis shows that outlet is under-sized for the 5 Year storm event (Tables 1 & 2) with surcharging of the system and localised flooding evident upstream in Sandy Point Road.

Like Outfall 2, Outfall 3 carries a considerable amount of suspended sediment and pollutants from the upstream urban areas within catchment CA3. This is summarised in Table 7. Two practical options for upgrading this outfall are discussed in the following.

6.3.2 *Design Options*

(i) Option 1 – Install two GPTs upstream of the outlet within the existing Council reserve. These would also need to be twin Humeguard HG40B GPTs as discussed for Outfall 2. The GPTs would need to be arranged differently to those at Outfall 2. This is due to there being three 1200mm diameter pipes at Outfall 3. Therefore, each outside pipe would be connected to a Humeguard via a new junction pit and the central outlet would need to be raised to act as an overflow weir during large storm events. The two GPTs were modelled in MUSIC with the predicted pollutant reductions shown in Table 9 below. Refer to drawing 1500047_P01_SK04 (Appendix 7) for details.

Table 9 - Outfall 3 - Mean Annual Pollutant Load Reductions

Mean Annual Loads - Generic GPT

	Inflow	Outflow	% Reduction
Flow (ML/yr)	158	158	0.0
Total Suspended Solids (kg/yr)	24.3E3	14.4E3	40.9
Total Phosphorus (kg/yr)	270	212	21.6
Total Nitrogen (kg/yr)	312	277	11.2
Gross Pollutants (kg/yr)	4.23E3	458	89.2

(ii) Option 2 – Install Humeguard GPTs as for Option 1 but also extend the three 1200mm diameter concretes pipes a minimum of 80m into the bay of Port Stephens. A 3 metre wide rock groyne would then be constructed over the newly extended pipe line. Refer to drawing 1500047_P01_SK05 (Appendix 7) for details.

6.4 Outfalls 4 & 5 – Catchment CA3

Outfalls 4 and 5 as described in Section 3.3.4 have the smallest catchments and contribute the least amount of suspended sediment and gross pollutants compared with the other catchments.

Outfall 4 was completely buried and Outfall 5 was partially blocked at the time of inspection. In the absence of any works to extend these outlets, a regular maintenance effort will be required to prevent burial by beach sand as it accretes against the breakwater.

7 Conclusion

The existing stormwater pipe network is significantly undersized throughout the catchments. There is little that can be done to relieve this without the costly exercise of augmenting the entire piped drainage system downstream of the problem areas. However, Council should ensure overland flow paths through properties are clearly defined and clear of obstructions such as vegetation or illegal structures.

The estimated total mass of sediment exported from the five outlets is 50.73 tonnes per annum, with Outfalls 2 and 3 accounting for approximately 47% each. The remaining three outlets have minor sediment loads. Retro-fitting of GPTs at Outfalls 2 and 3 would achieve significant reductions in suspended sediments and gross pollutants into the bay as discussed in Sections 6.2 and 6.3.

8 References

Engineers Australia (1998). *Australian Rainfall and Runoff, A Guide to Flood Estimation, Volume 1 and Volume 2.*

Port Stephens Council (2006). *Infrastructure Specification – Design Specification Series Part 1.*

Port Stephens Council. Topographic details (0.5 meter contours).

SMCMA (2010). *Draft NSW MUSIC Modelling Guidelines. Sydney Metropolitan Catchment Management Authority.* (MUSIC Modeling Reference).

Watercom Pty Ltd *DRAINS Version 2015.07.*

9 Appendices

9.1 Appendix 1 – Rainfall Intensity Frequency Duration Information

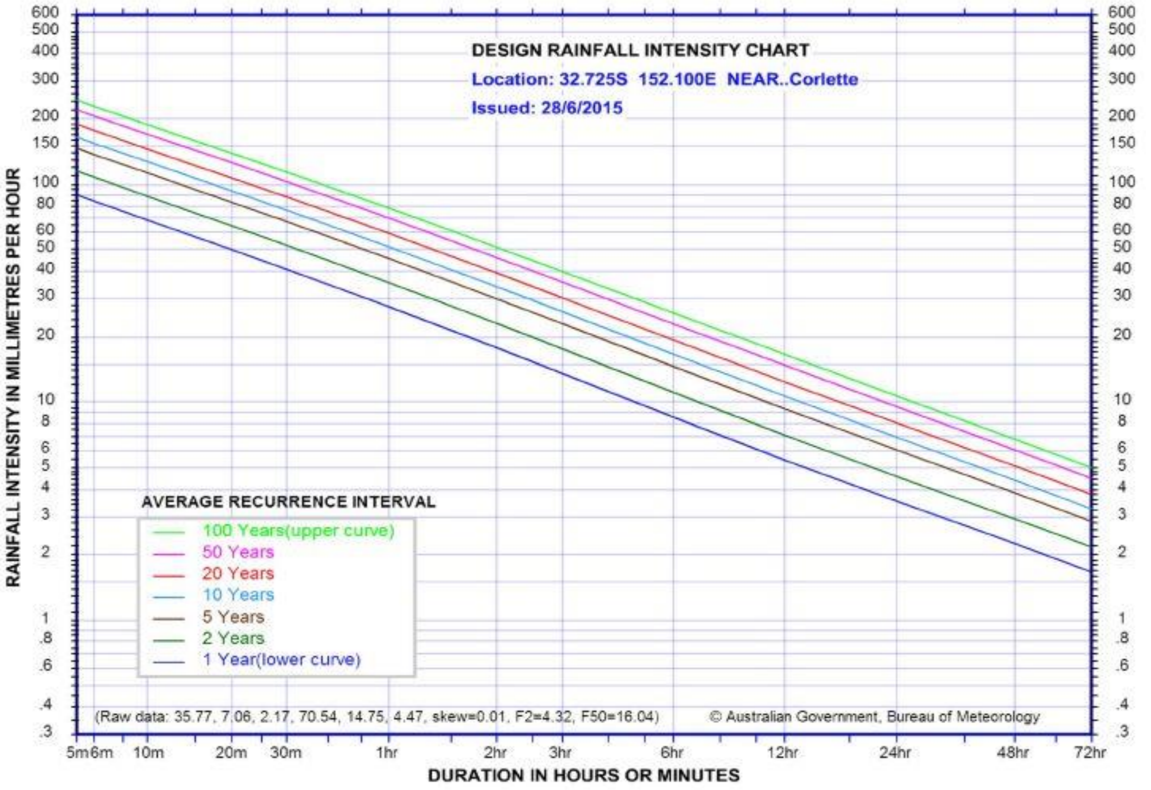
Intensity-Frequency-Duration Table

Location: 32.725S 152.100E NEAR.. Corlette Issued: 28/6/2015

Rainfall intensity in mm/h for various durations and Average Recurrence Interval

	Average Recurrence Interval						
Duration	**1 YEAR**	**2 YEARS**	**5 YEARS**	**10 YEARS**	**20 YEARS**	**50 YEARS**	**100 YEARS**
5Mins	89.7	115	147	165	189	220	244
6Mins	83.9	108	137	154	177	206	228
10Mins	68.6	88.2	113	127	145	170	188
20Mins	50.2	64.6	82.8	93.3	107	126	139
30Mins	40.8	52.6	67.5	76.2	87.7	103	114
1Hr	27.5	35.5	45.8	51.8	59.8	70.2	78.1
2Hrs	17.8	23.0	29.8	33.9	39.2	46.1	51.4
3Hrs	13.6	17.6	23.0	26.1	30.2	35.7	39.9
6Hrs	8.55	11.1	14.6	16.6	19.3	22.9	25.7
12Hrs	5.43	7.07	9.33	10.7	12.4	14.8	16.6
24Hrs	3.52	4.57	6.04	6.91	8.05	9.55	10.7
48Hrs	2.25	2.92	3.84	4.39	5.10	6.04	6.77
72Hrs	1.67	2.17	2.85	3.25	3.78	4.48	5.02

(Raw data: 35.77, 7.06, 2.17, 70.54, 14.75, 4.47, skew=0.01, F2=4.32, F50=16.04)



9.2 Appendix 2 – *DRAINS* Results for Network 1

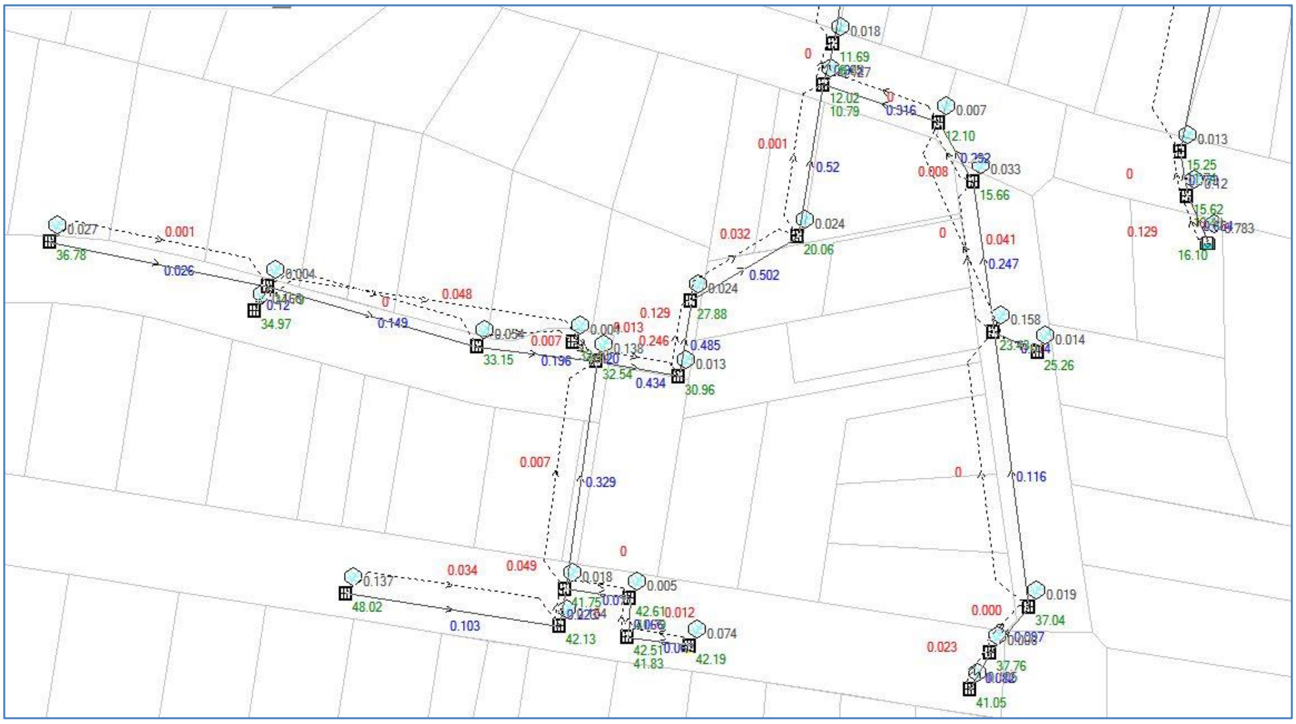


Figure 5 – Network 1 – 5 year Overland Flow Results – Detail 1

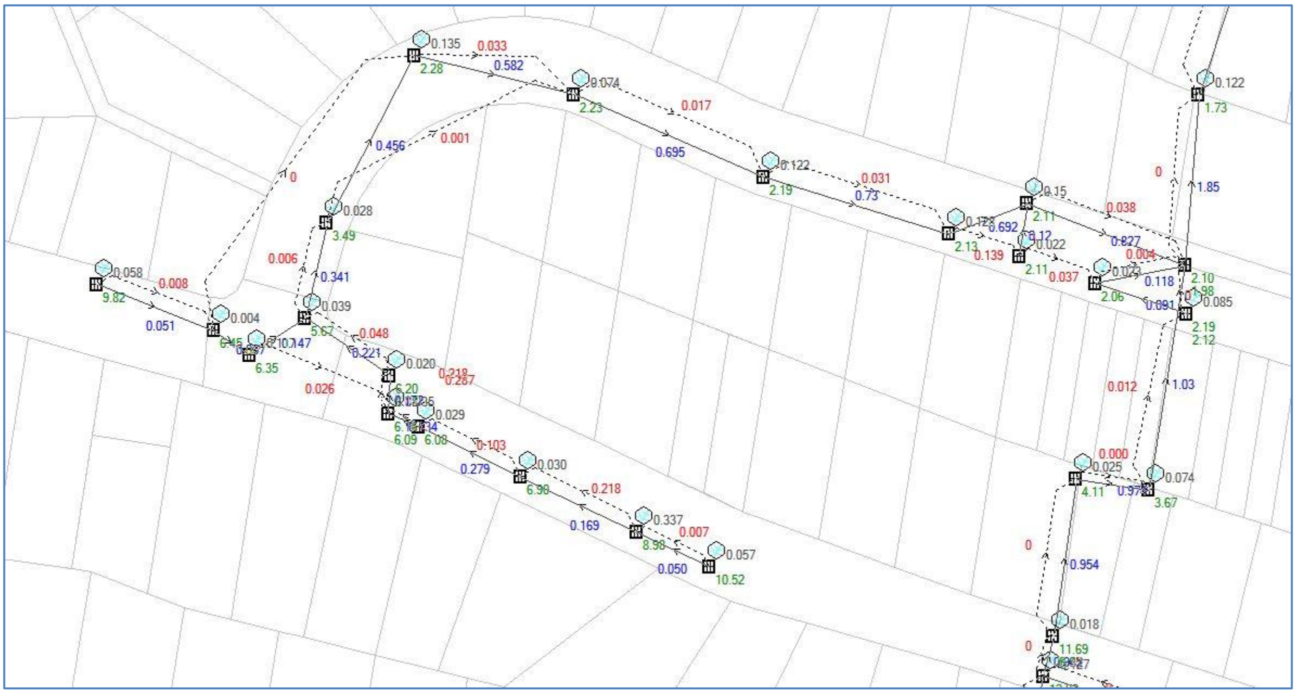


Figure 6 - Network 1 – 5 year Overland Flow Results – Detail 2

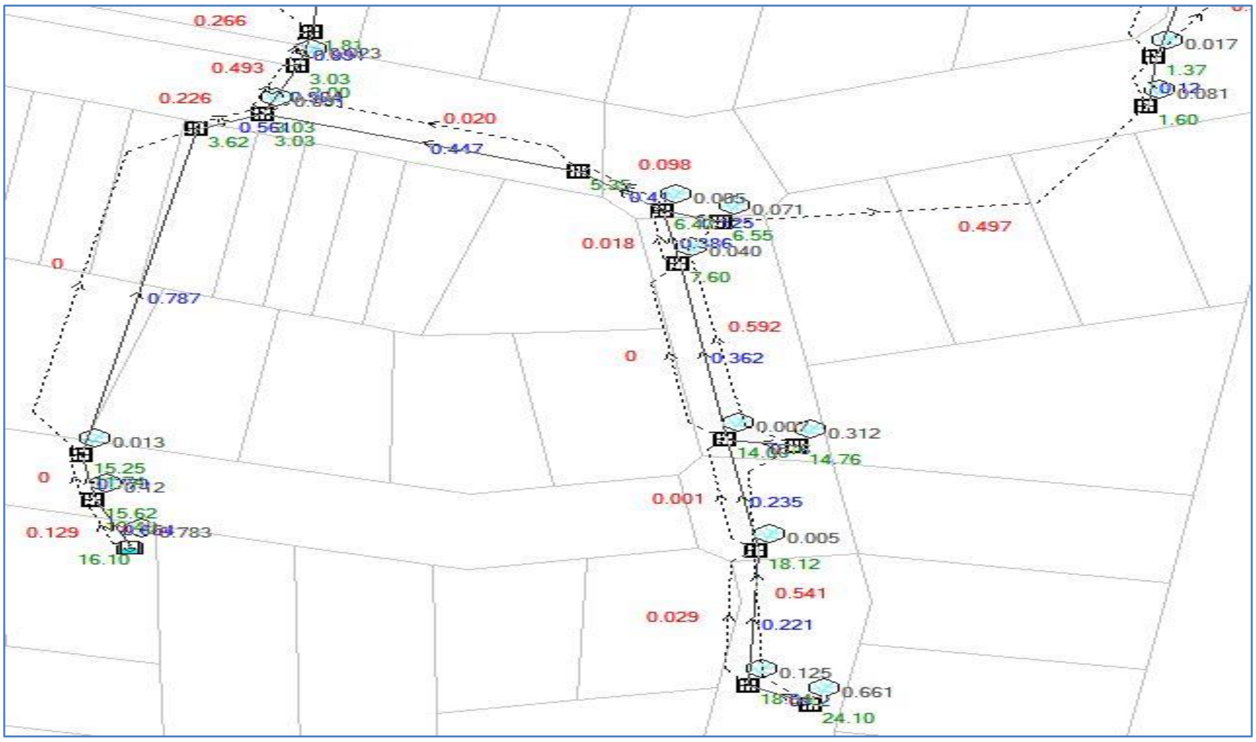


Figure 7 - Network 1 – 5 year Overland Flow Results – Detail 3

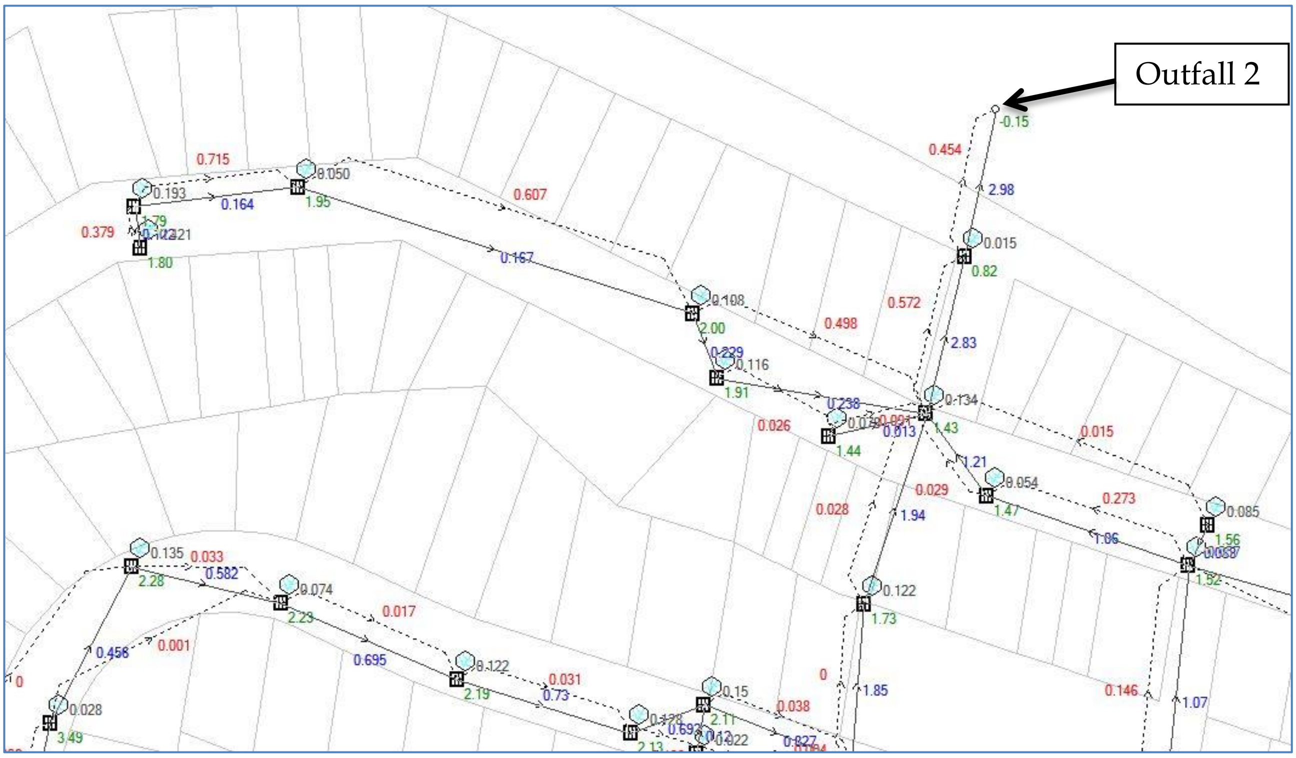


Figure 8 - Network 1 – 5 year Overland Flow Results – Detail 4

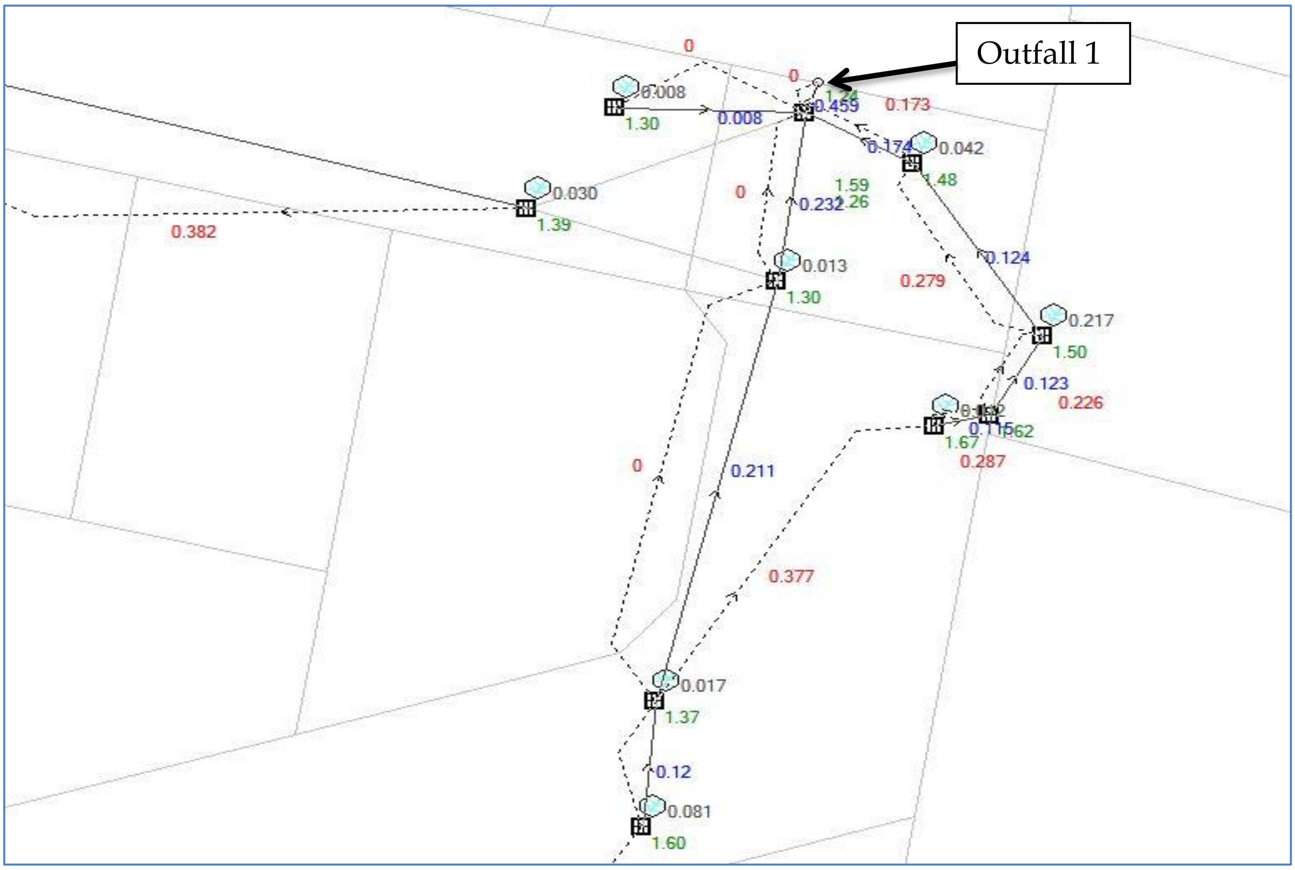


Figure 9 - Network 1 – 5 year Overland Flow Results – Detail 5

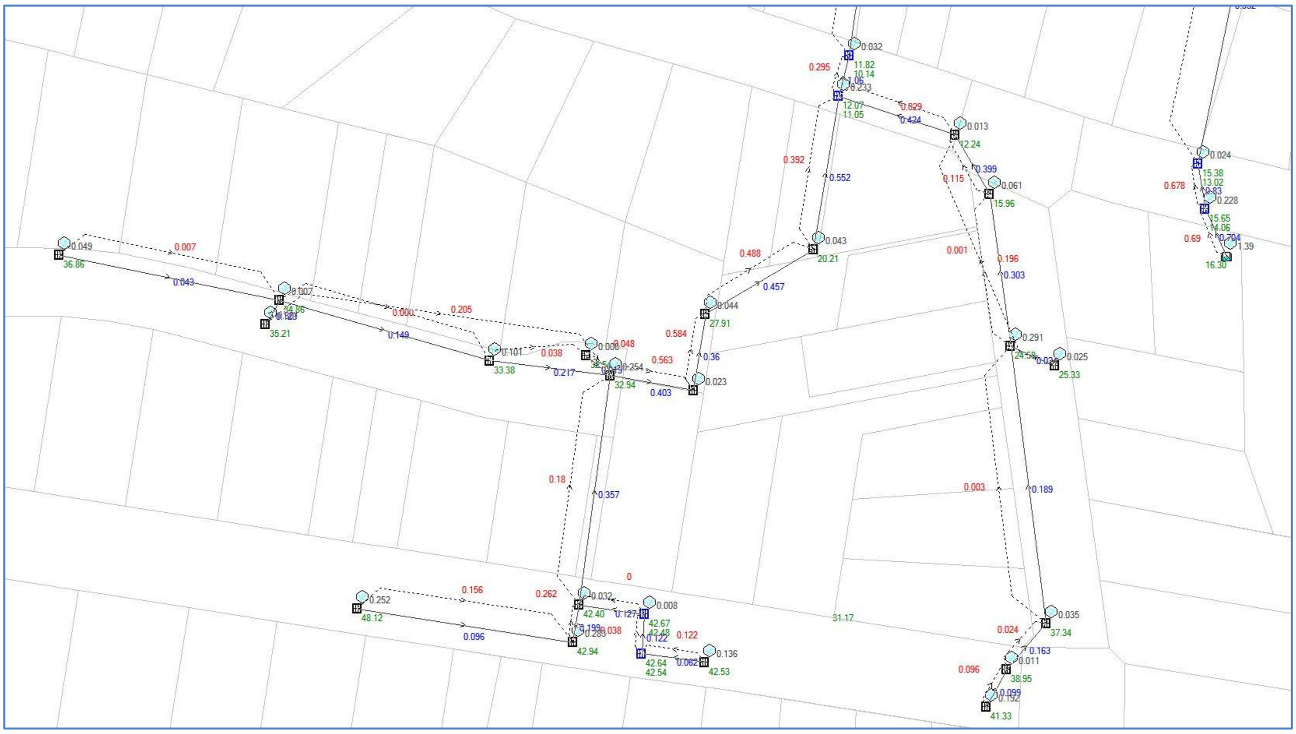


Figure 10 – Network 1 – 100 year Overland Flow Results – Detail 1

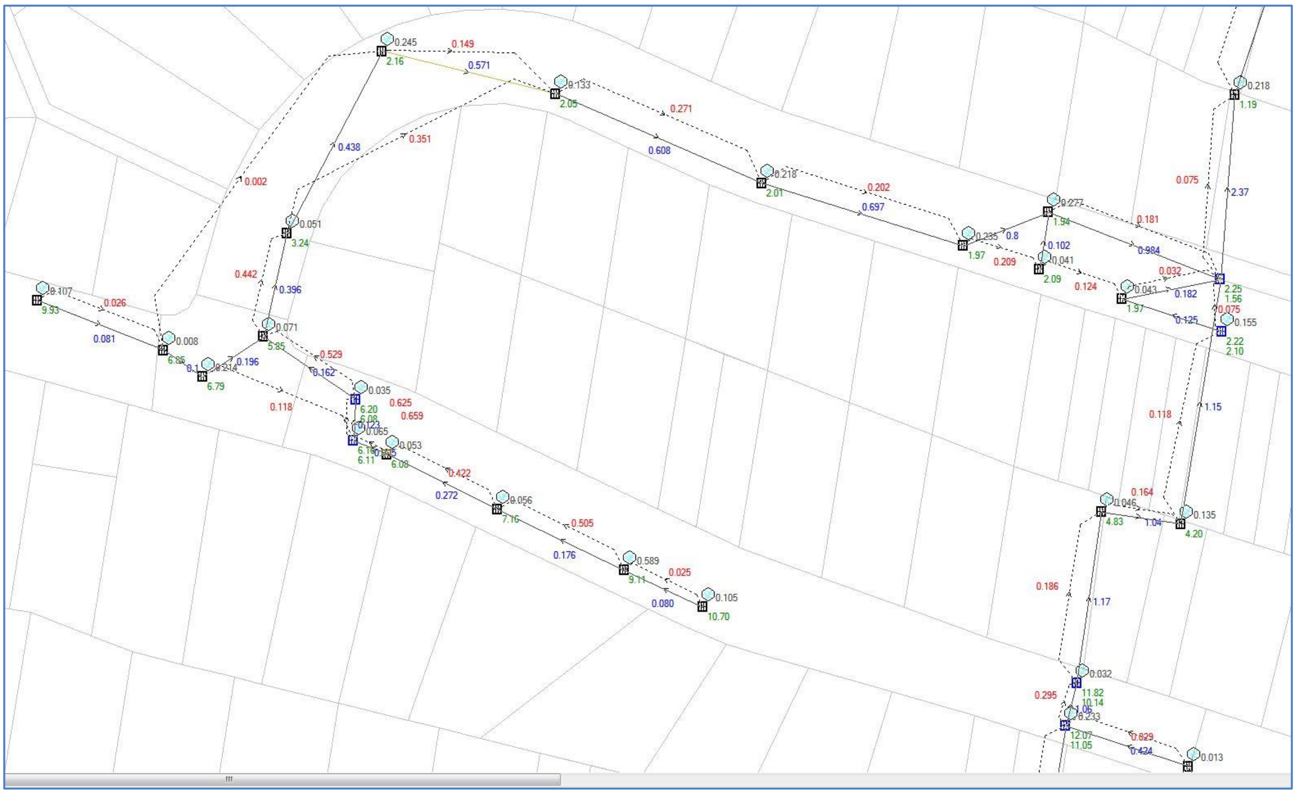


Figure 11 – Network 1 – 100 year Overland Flow Results – Detail 2

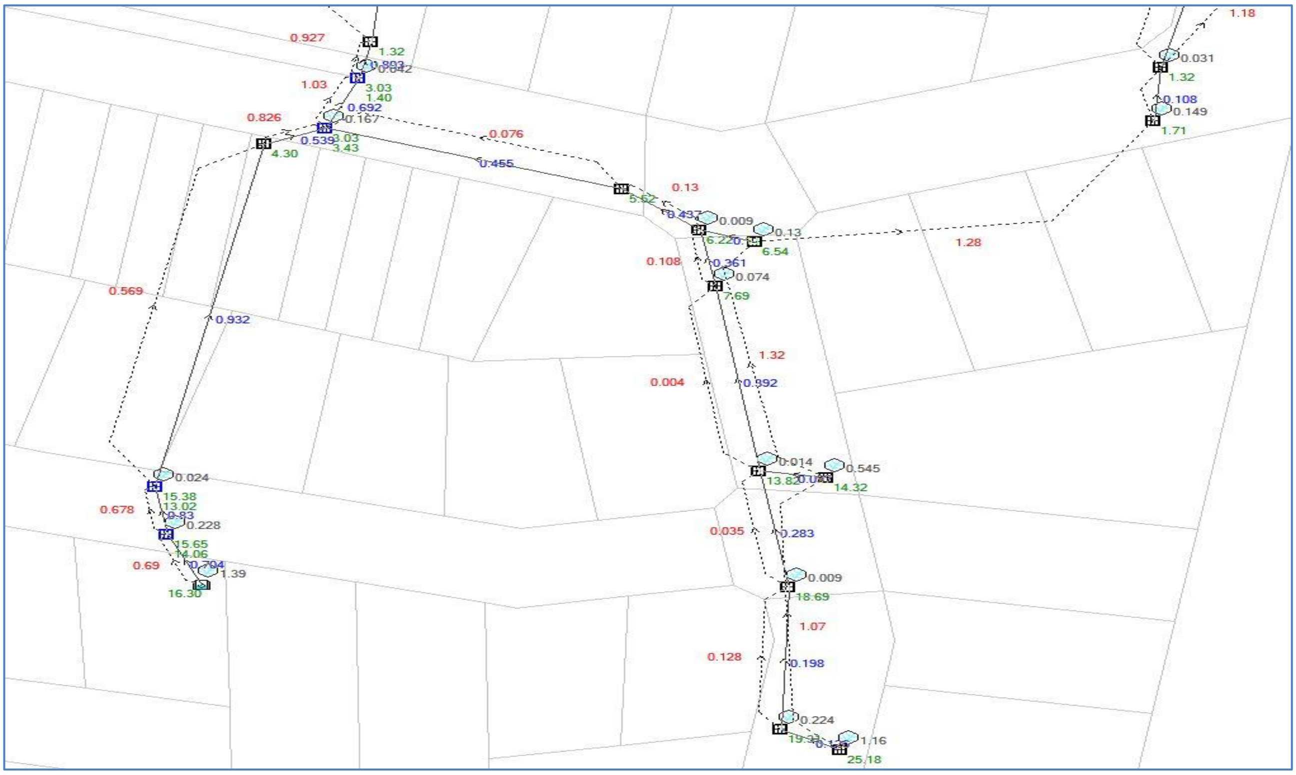


Figure 12 – Network 1 – 100 year Overland Flow Results – Detail 3

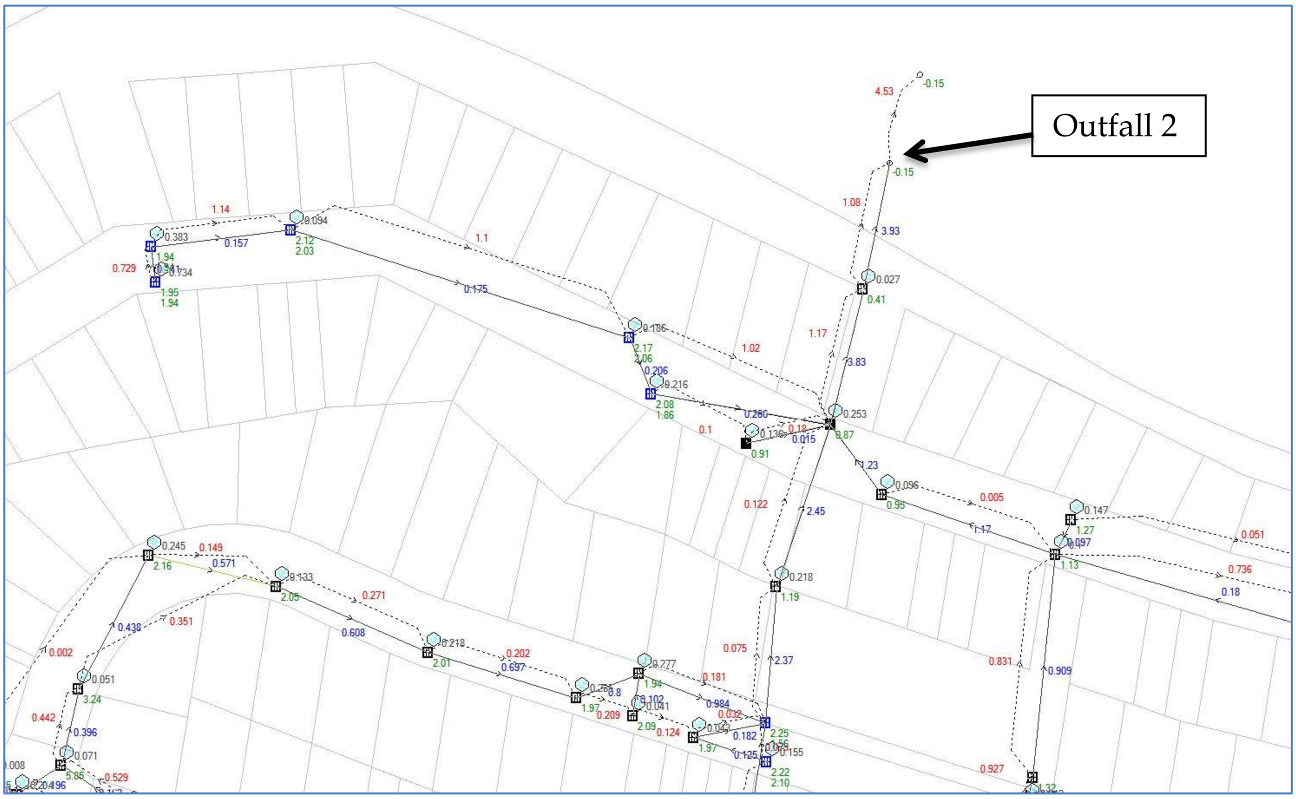


Figure 13 – Network 1 – 100 year Overland Flow Results – Detail 4

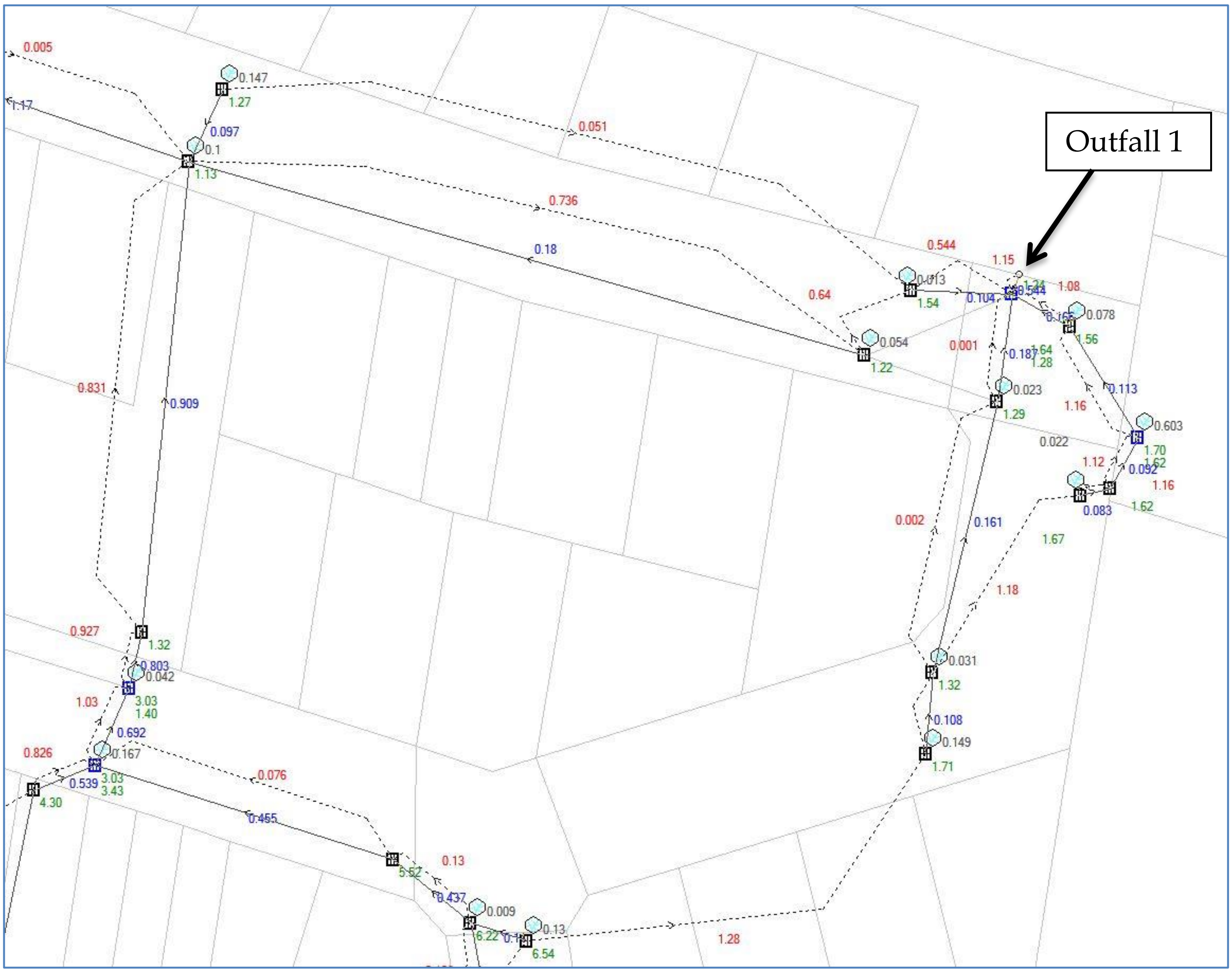


Figure 14 – Network 1 – 100 year – Overland Flow Results – Detail 5

9.3 Appendix 3 - *DRAINS* Results for Network 2

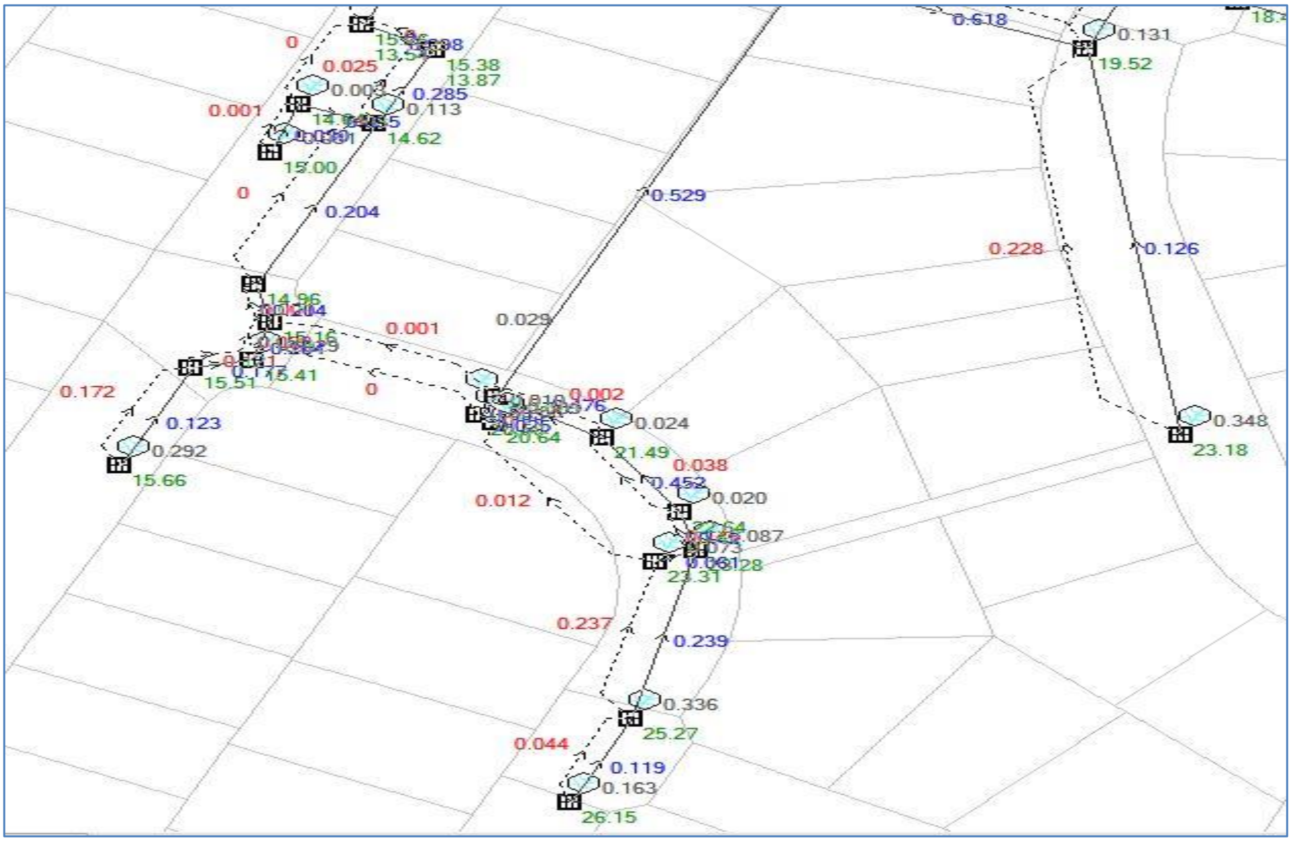


Figure 15 - Network 2 – 5 year Overland Flow Results – Detail 1

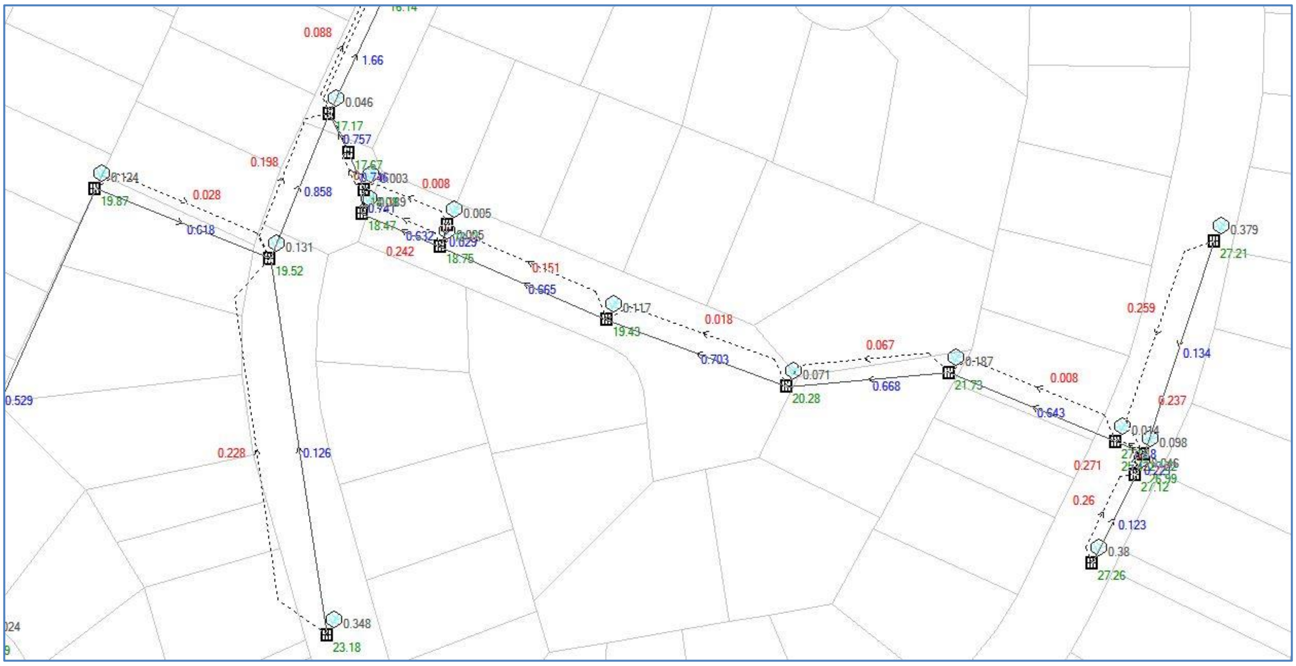


Figure 16 - Network 2 – 5 year Overland Flow Results – Detail 2

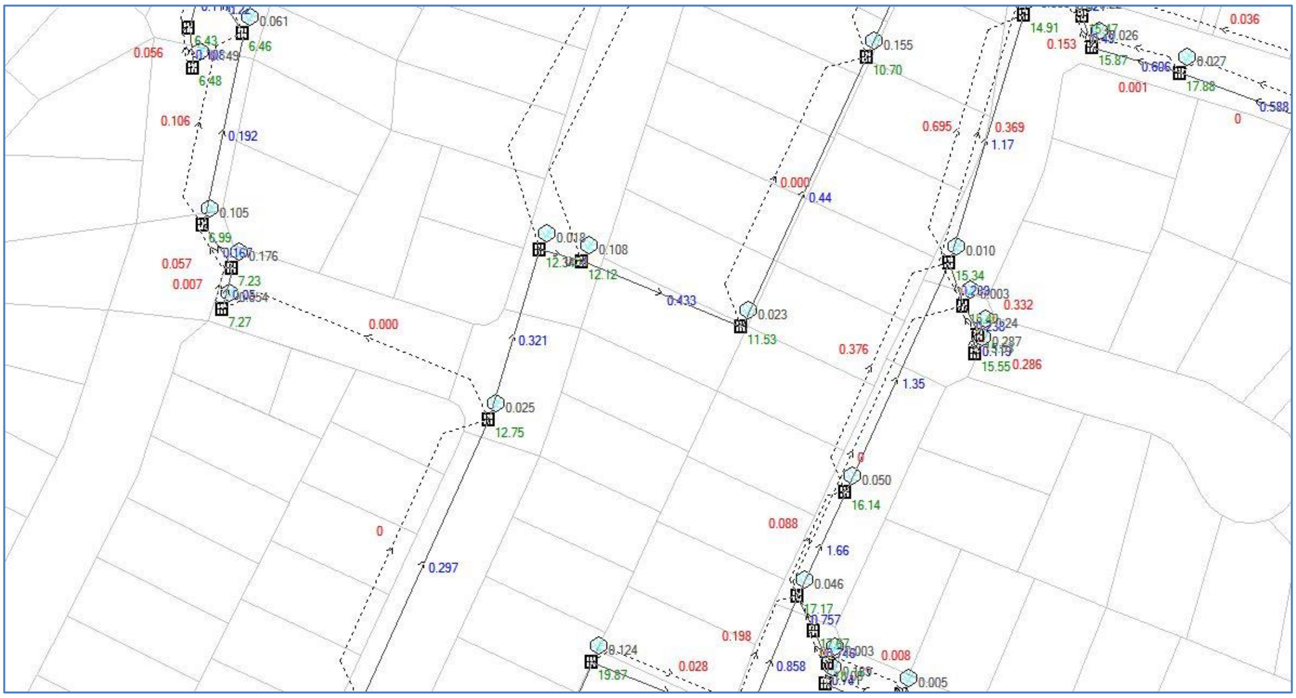


Figure 17 - Network 2 – 5 year Overland Flow Results – Detail 3

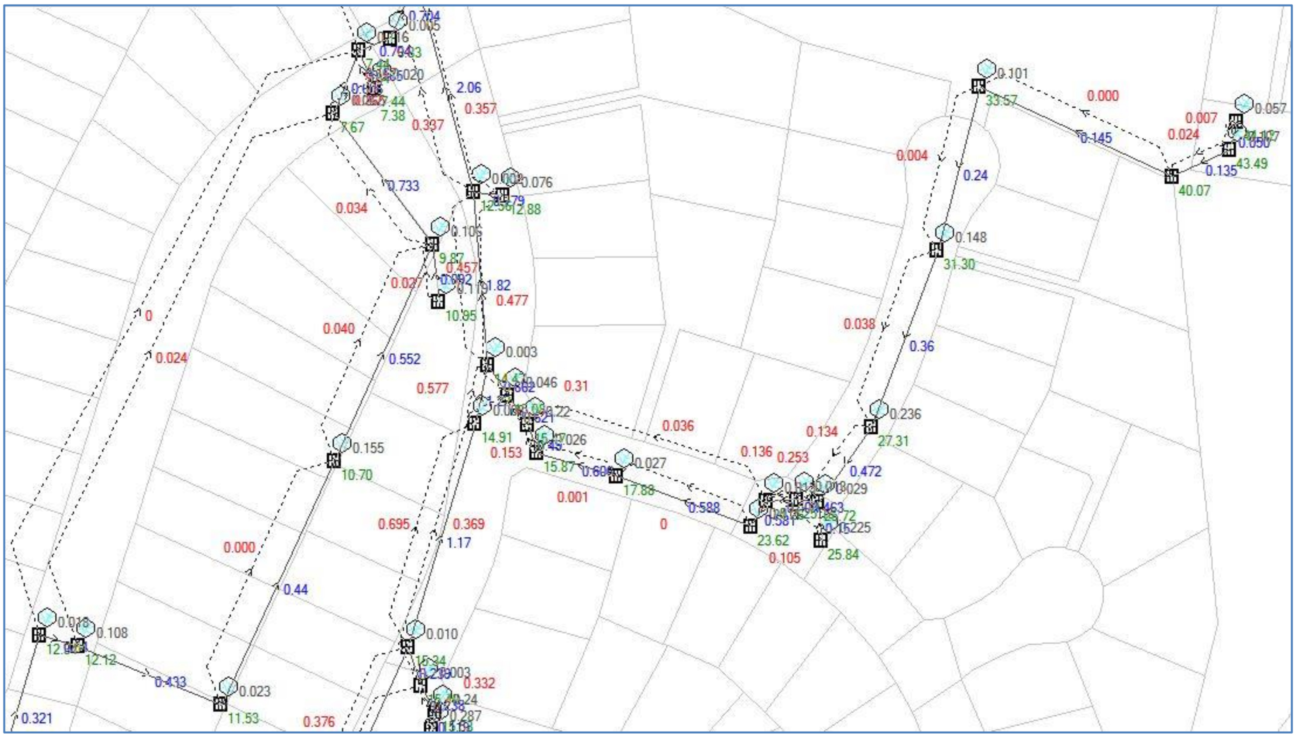


Figure 18 - Network 2 – 5 year Overland Flow Results – Detail 4

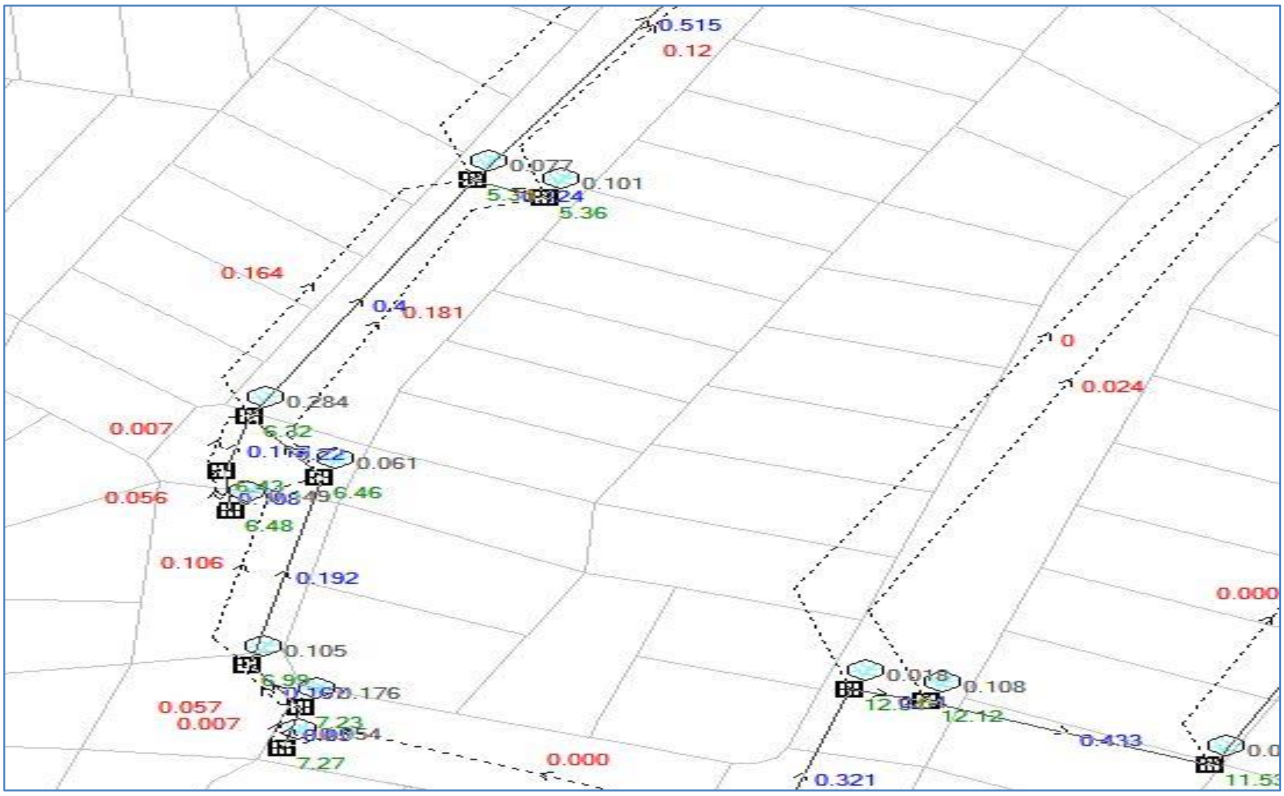


Figure 19 - Network 2 – 5 year Overland Flow Results – Detail 5

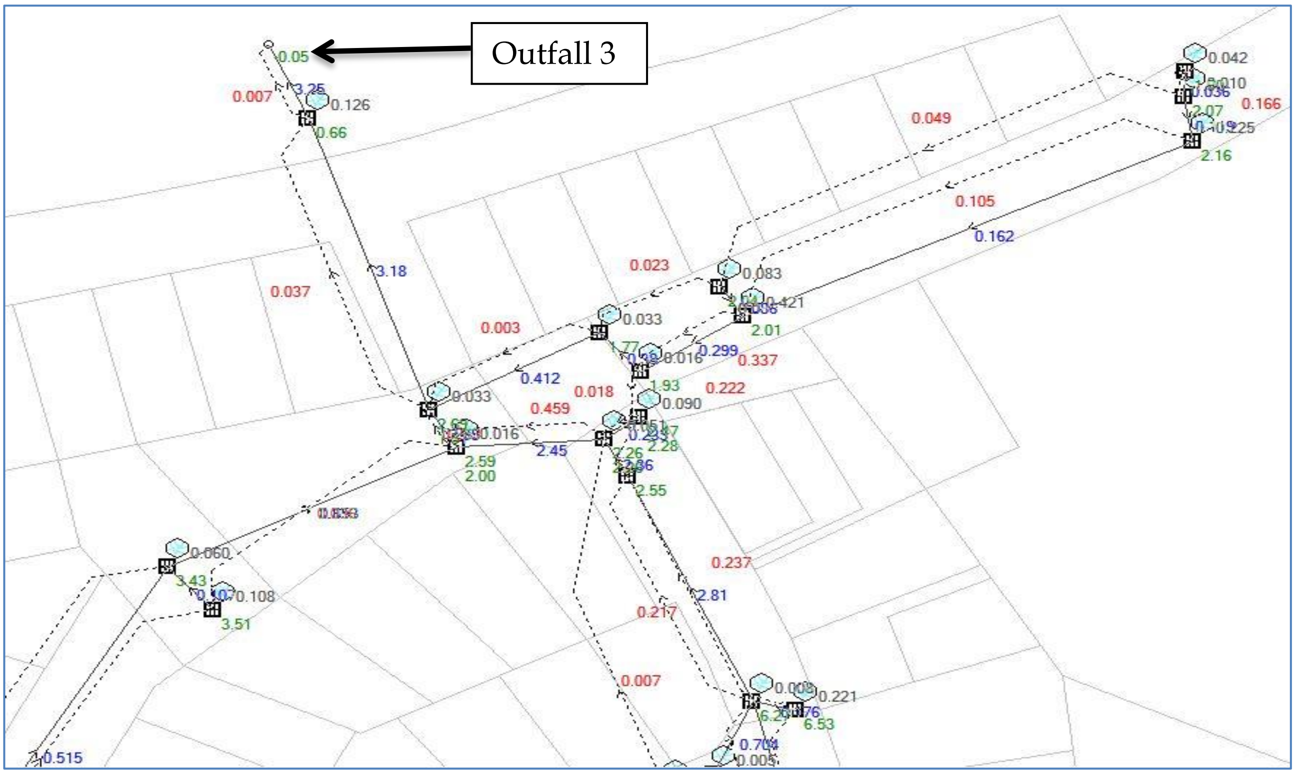


Figure 20 - Network 2 – 5 year Overland Flow Results – Detail 6

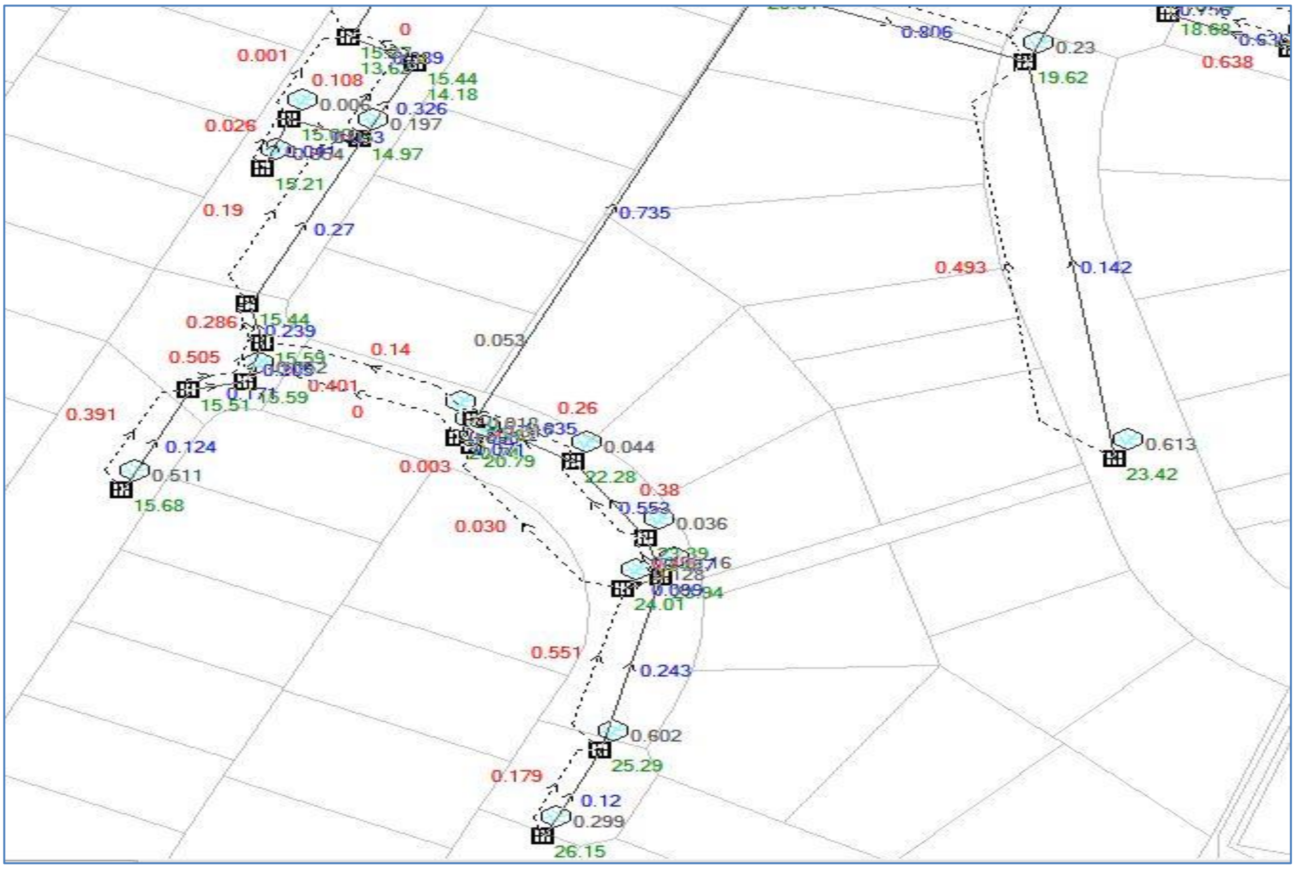


Figure 21 - Network 2 – 100 year Overland Flow Results – Detail 1

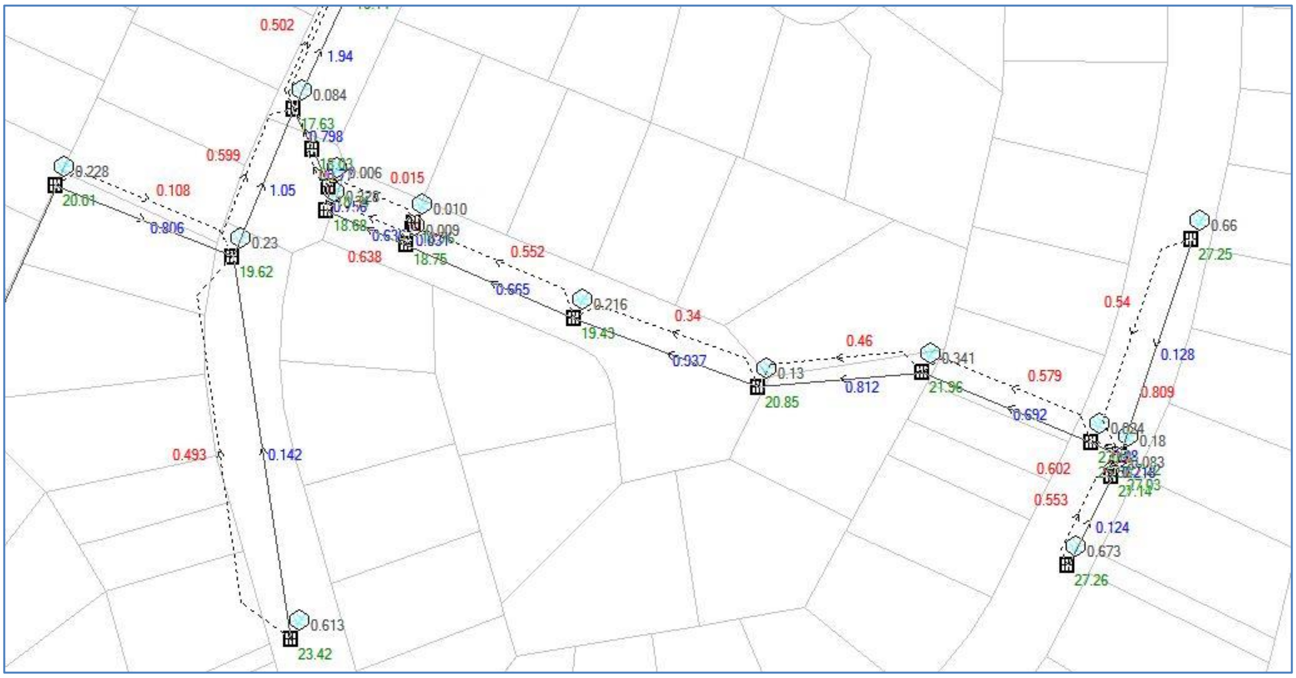


Figure 22 - Network 2 – 100 year Overland Flow Results – Detail 2

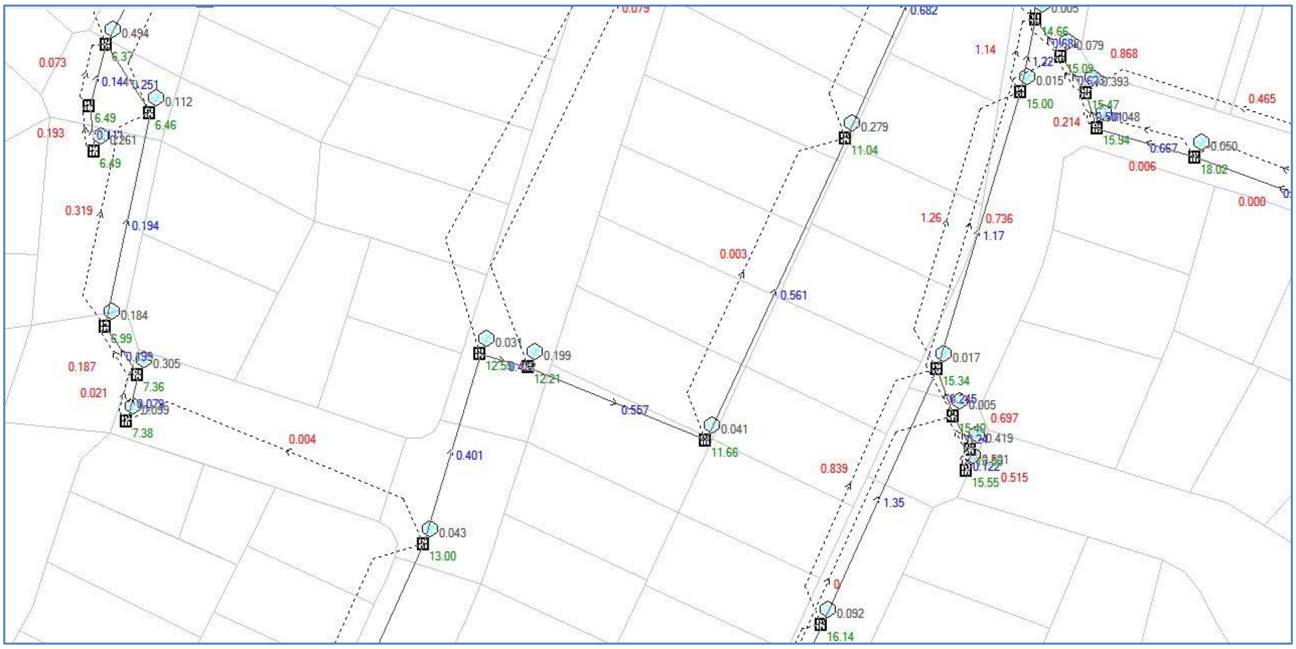


Figure 23 – Network 2 – 100 year Overland Flow Results – Detail 3

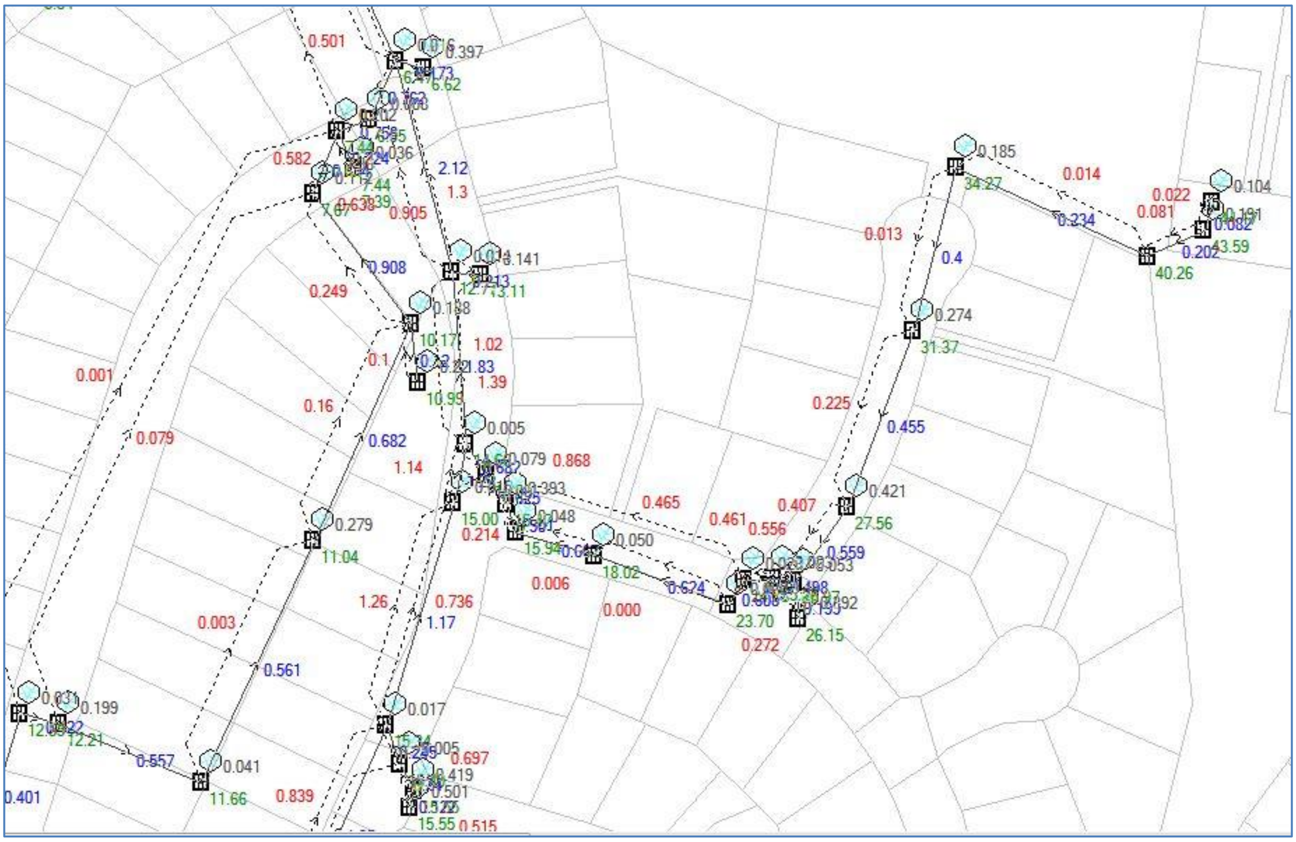


Figure 24 - Network 2 – 100 year Overland Flow Results – Detail 4

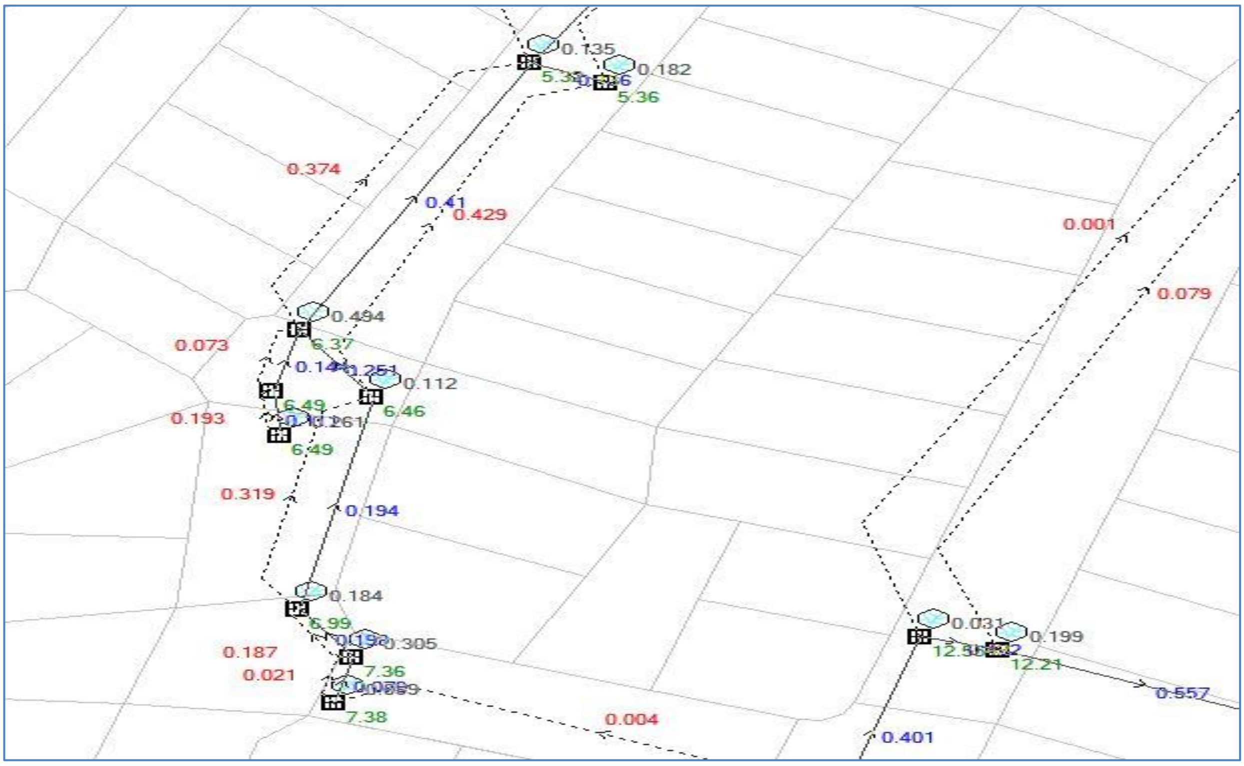


Figure 25 - Network 2 – 100 year Overland Flow Results – Detail 5

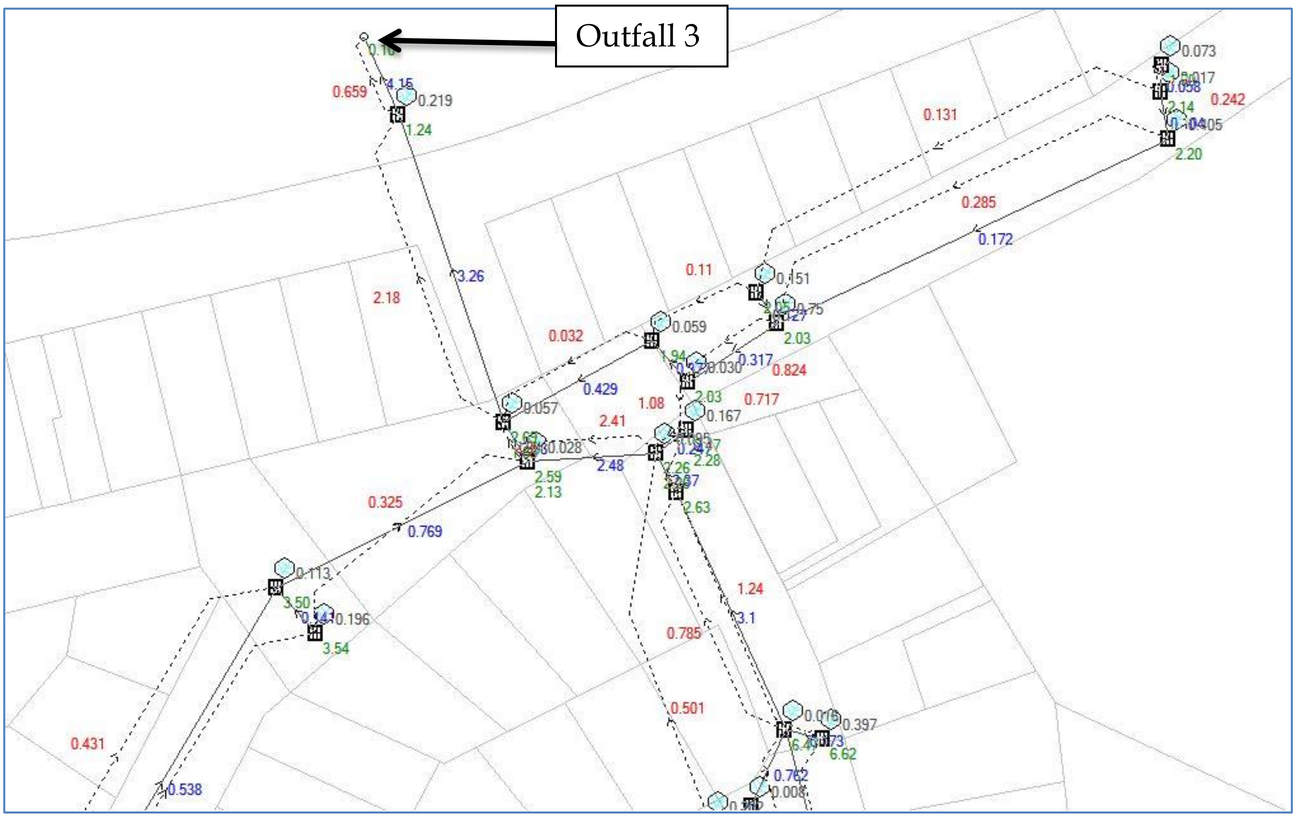


Figure 26 - Network 2 – 100 year Overland Flow Results – Detail 6

9.4 Appendix 4 - *DRAINS* Results for Network 3

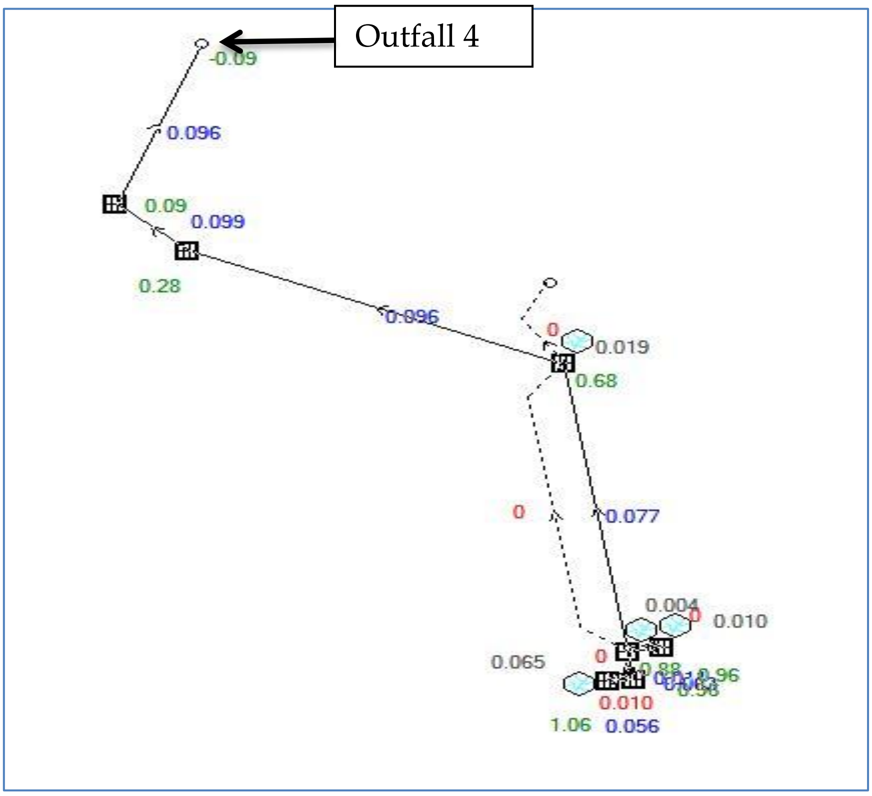


Figure 27 - Network 3 – 5 year Overland Flow Results

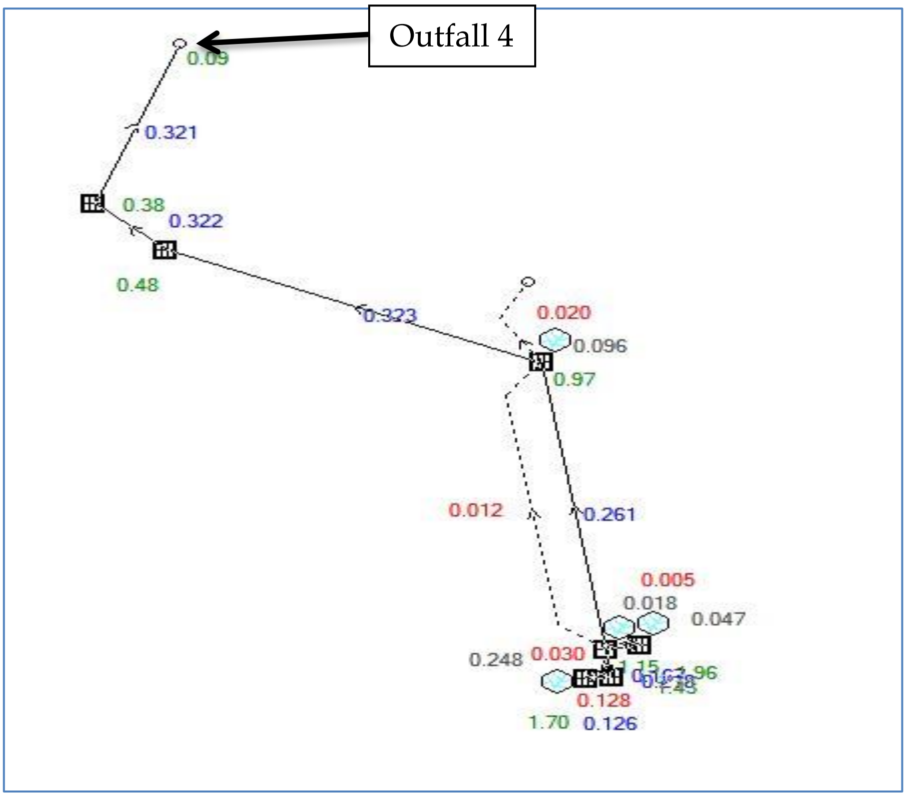


Figure 28 - Network 3 – 100 year Overland Flow Results

9.5 Appendix 5 - *DRAINS* Results for Network 4

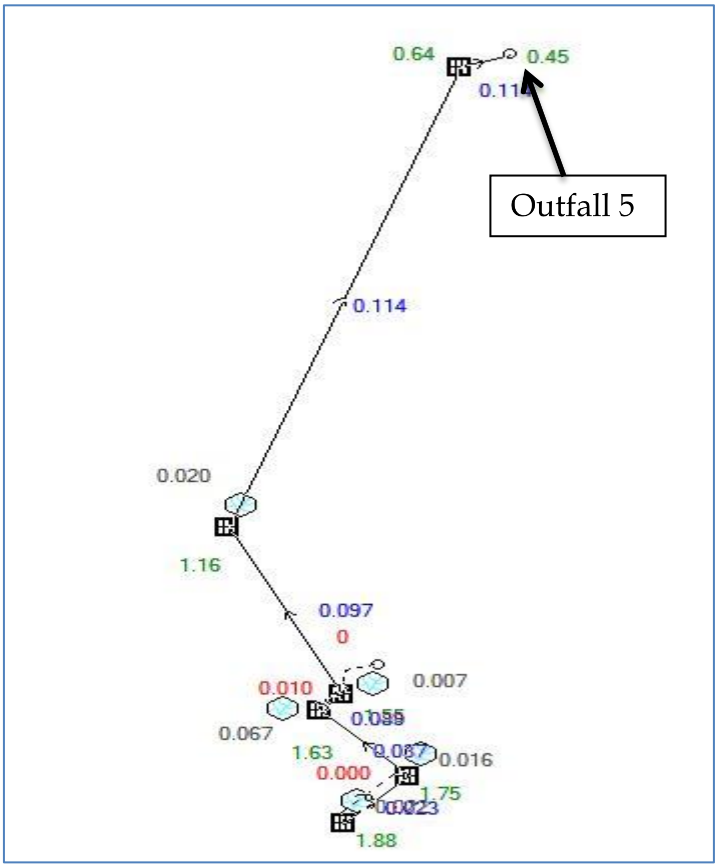


Figure 29 - Network 4 – 5 year Overland Flow Results

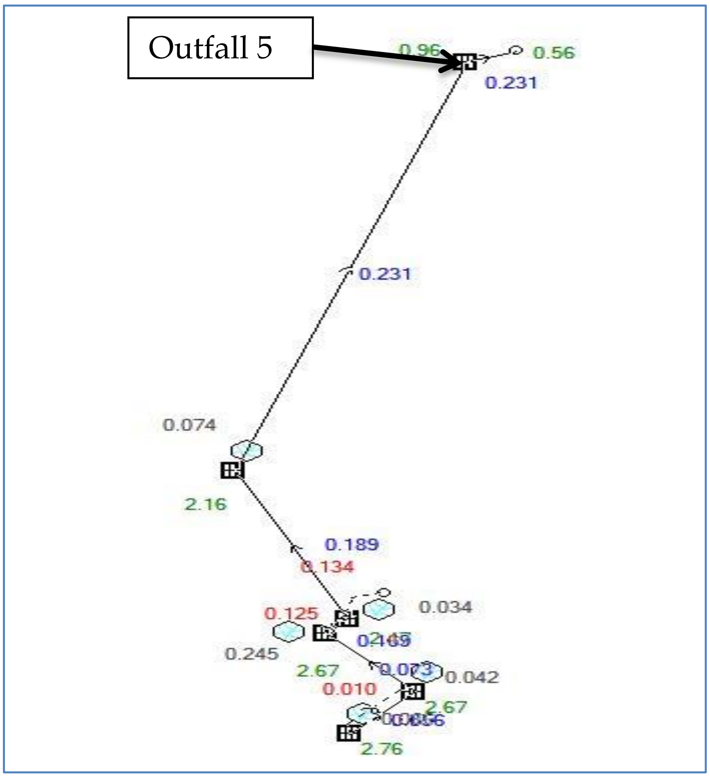
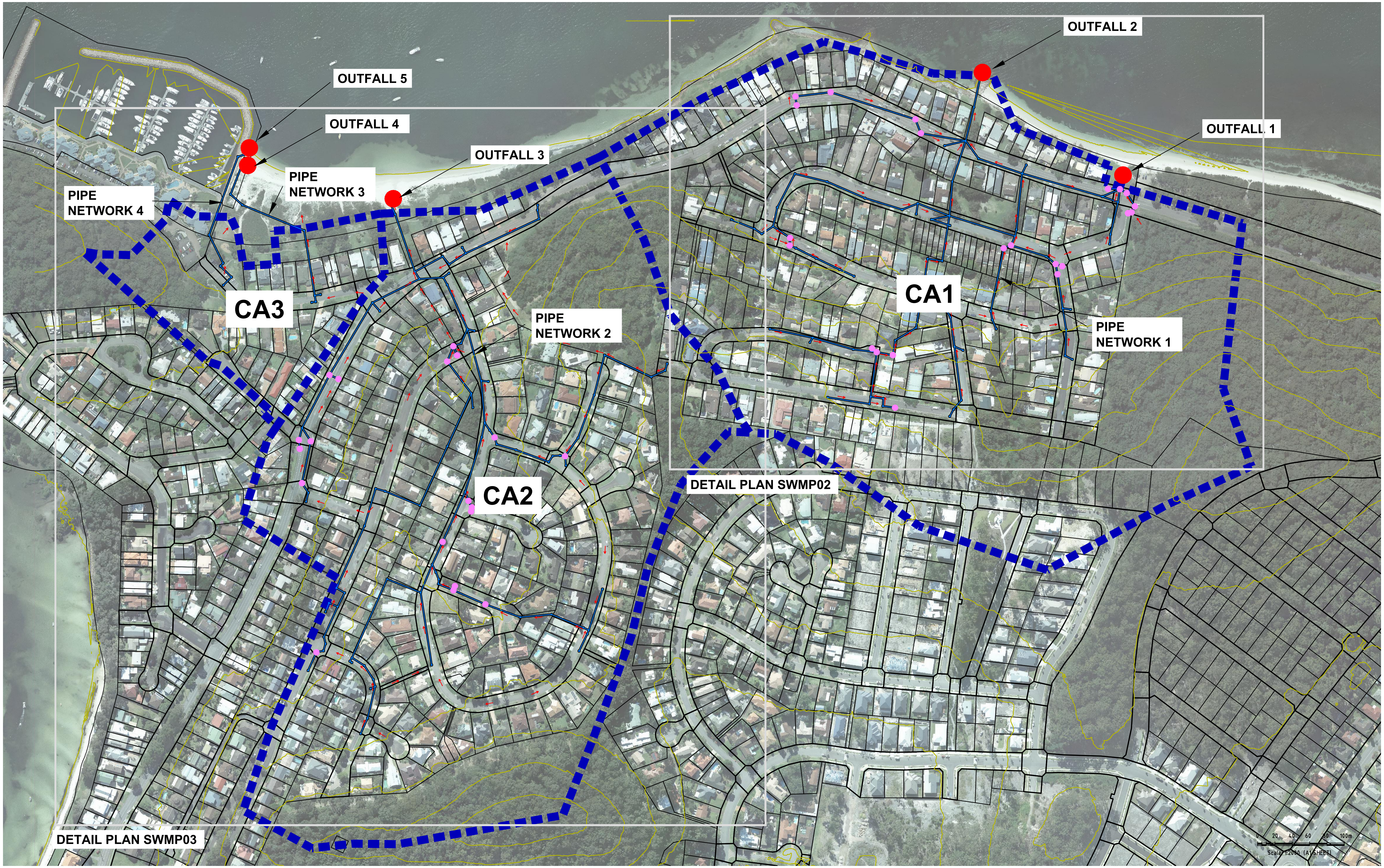


Figure 30 - Network 4 – 100 year Overland Flow Results

9.6 Appendix 6 – Plan of Study Area Showing Pipe Networks – 15000047_P01_SWMP01

9.7 Appendix 7 – Concept Design Plans



REV	DATE	DES.	DRN.	APP.	REVISION DETAILS
00	01/04/16	J.A	A.J.B	J.M.A	FINAL ISSUE
B	28/01/16	J.A.	A.J.B.	J.M.A.	DRAFT ISSUE FOR CONSULTATION
A	28/08/15	J.A.	N.L.	J.M.A.	DRAFT ISSUE

DRAWING STATUS

DESIGN BY	J.A.
DRAWN BY	N.L.
FINAL APPROVAL	J.M.A.
SCALE: (on A1 Original)	1:2000

DRAFT

North

CLIENT

PORT STEPHENS COUNCIL

SEEC
Suites 7 & 8, 68-70 Station Street
PO Box 1098, Bowral NSW 2576.
(t) 02 4862 1633
(f) 02 4862 3088
email: reception@seec.com.au
WWW.SEEC.COM.AU

PROJECT TITLE

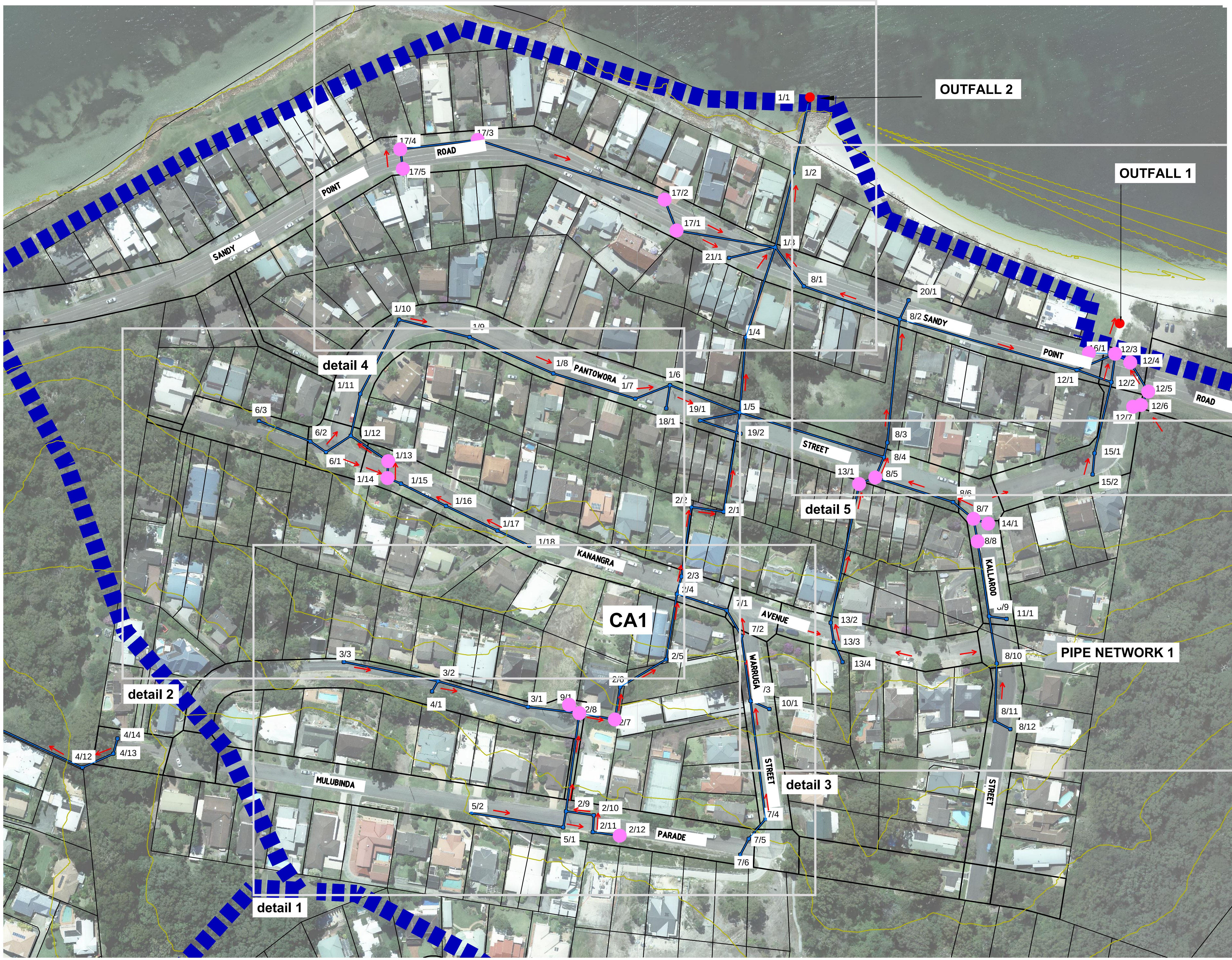
EROSION AND DRAINAGE MANAGEMENT PLAN FOR SANDY POINT/CONROY PARK CORLETTE

DRAWING TITLE

OVERALL STUDY AREA PLAN SHOWING CATCHMENT AREAS

PROJECT NO.	SUB-PR NO.	DRAWING NO.	REV
15000047	P01	SWMP01	00

Plot Date: Friday, 1 April 2016 11:37:03 AM CAD File Name: I:\15000047 Sandy Point_Conroy Park Foreshore Erosion and Drainage Management Plan\Drawings\15000047_P01_C_REV 00.dwg



LEGEND

	CATCHMENT BOUNDARY
2/11	PIT NUMBER
	SURCHARGING PIT IN A 5YR STORM
	APPROXIMATE OVERLAND FLOW PATH

DISCLAIMER
OVERLAND FLOW PATHS SHOWN ARE INDICATIVE ONLY AND ARE NOT BASED ON DETAILED FLOOD MODELING. THEY ARE BASED ON VISUAL SITE INSPECTION AND INTERPRETATION DEPENDING ON AVAILABLE SITE ACCESS AT THE TIME OF INSPECTION.

REV	DATE	DES.	DRN.	APP.	REVISION DETAILS
00	01/04/16	J.A	A.J.B	J.M.A	FINAL ISSUE
B	28/01/16	J.A.	A.J.B.	J.M.A.	DRAFT ISSUE FOR CONSULTATION
A	28/08/15	J.A.	N.L.	J.M.A.	DRAFT ISSUE

DRAWING STATUS	
DESIGN BY	J.A.
DRAWN BY	N.L.
FINAL APPROVAL	J.M.A.
SCALE: (on A1 Original)	AS SHOWN

DRAFT

North

CLIENT
PORT STEPHENS COUNCIL

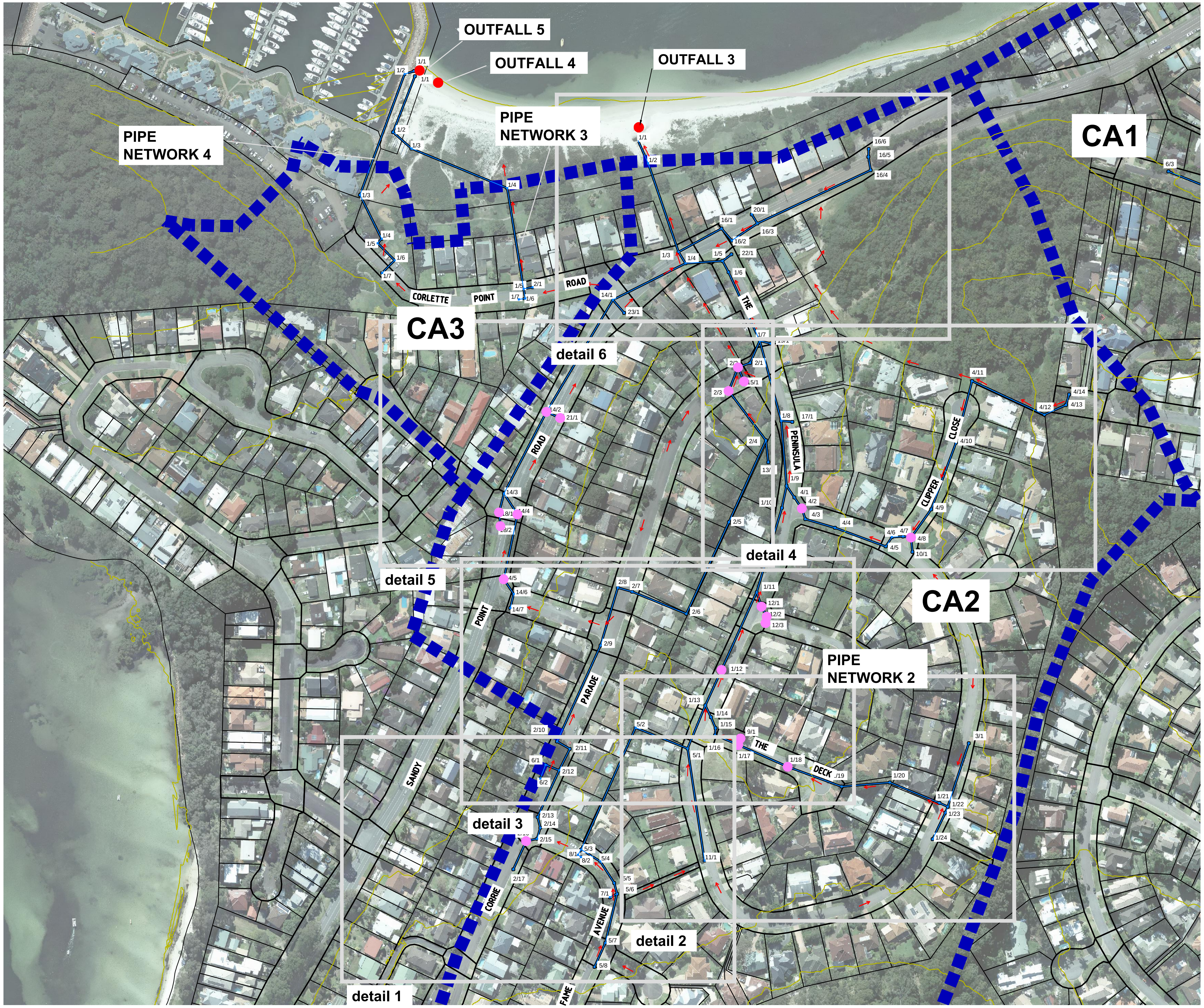
SEEC
Suites 7 & 8, 68-70 Station Street
PO Box 1098, Bowral NSW 2576.
(t) 02 4862 1633
(f) 02 4862 3088
email: reception@seec.com.au
WWW.SEEC.COM.AU

PROJECT TITLE
EROSION AND DRAINAGE MANAGEMENT PLAN FOR SANDY POINT/CONROY PARK CORLETTE

DRAWING TITLE
DETAIL PLAN
CATCHMENT CA1
PIPE NETWORK 1

PROJECT NO.	SUB-PR NO.	DRAWING NO.	REV
15000047	P01	SWMP02	00

Plot Date: Friday, 1 April 2016 11:37:12 AM CAD File Name: I:\15000047 Sandy Point_Conroy Park Foreshore Erosion and Drainage Management Plan\Drawings\15000047_P01_C_REV 00.dwg



LEGEND

- CATCHMENT BOUNDARY
- 2/11 PIT NUMBER
- SURCHARGING PIT IN A 5YR STORM
- APPROXIMATE OVERLAND FLOW PATH

DISCLAIMER
OVERLAND FLOW PATHS SHOWN ARE INDICATIVE ONLY AND ARE NOT BASED ON DETAILED FLOOD MODELING. THEY ARE BASED ON VISUAL SITE INSPECTION AND INTERPRETATION DEPENDING ON AVAILABLE SITE ACCESS AT THE TIME OF INSPECTION.

0 15 30 45 60 75m
Scale: 1:1500 (A1 SHEET)

REV	DATE	DES.	DRN.	APP.	REVISION DETAILS
00	01/04/16	J.A.	A.J.B	J.M.A	FINAL ISSUE
B	28/01/16	J.A.	A.J.B.	J.M.A.	DRAFT ISSUE FOR CONSULTATION
A	28/08/15	J.A.	N.L.	J.M.A.	DRAFT ISSUE

DRAWING STATUS
DESIGN BY J.A.
DRAWN BY N.L.
FINAL APPROVAL J.M.A.
SCALE: (on A1 Original) 1:1500

DRAFT

North

CLIENT
PORT STEPHENS COUNCIL

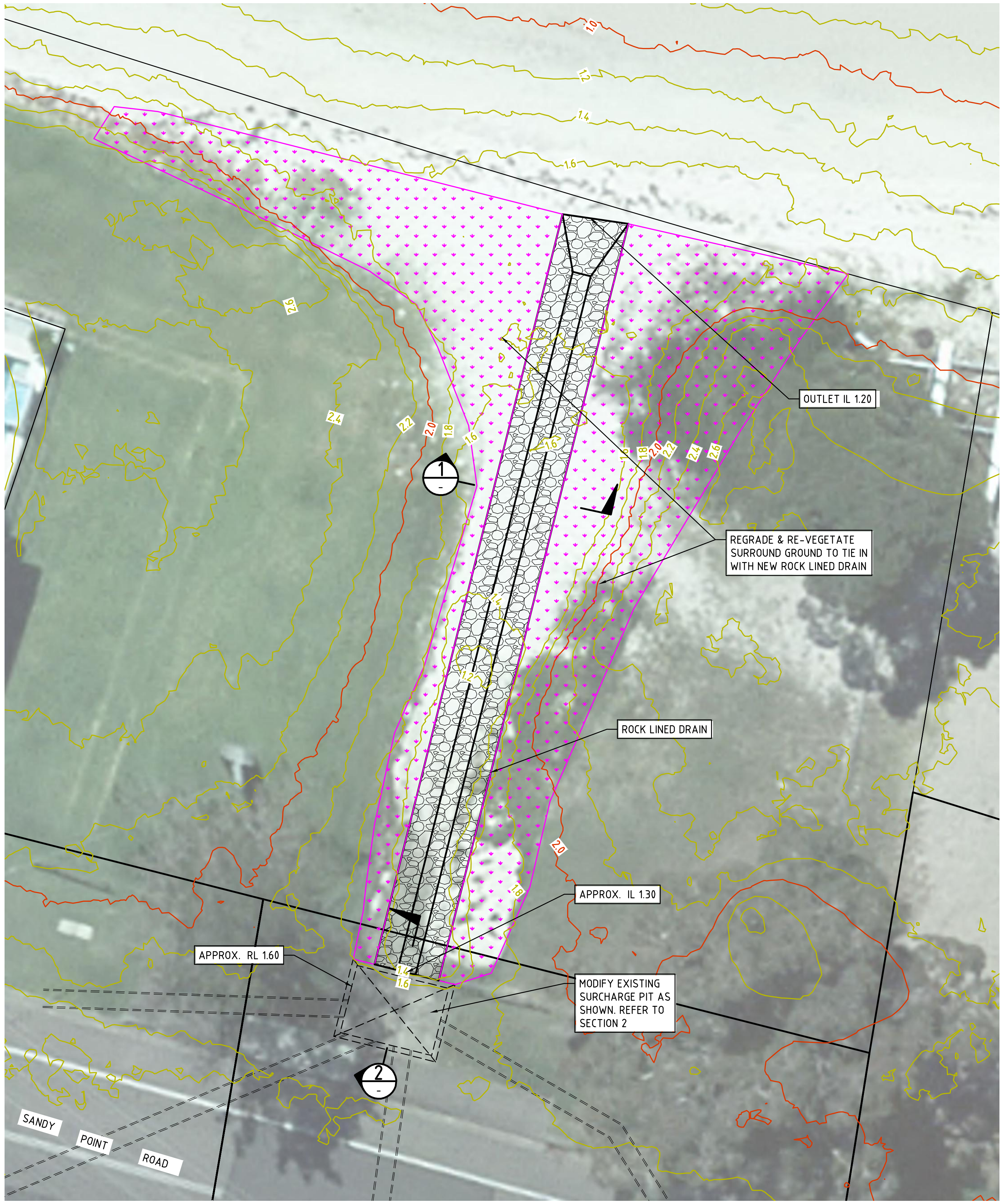
SEEC
Suites 7 & 8, 68-70 Station Street
PO Box 1098, Bowral NSW 2576.
(t) 02 4862 1633
(f) 02 4862 3088
email: reception@seec.com.au
WWW.SEEC.COM.AU

PROJECT TITLE
EROSION AND DRAINAGE MANAGEMENT PLAN FOR SANDY POINT/CONROY PARK CORLETTE

DRAWING TITLE
DETAIL PLAN
CATCHMENTS CA2 & CA3
PIPES NETWORKS 2, 3 & 4

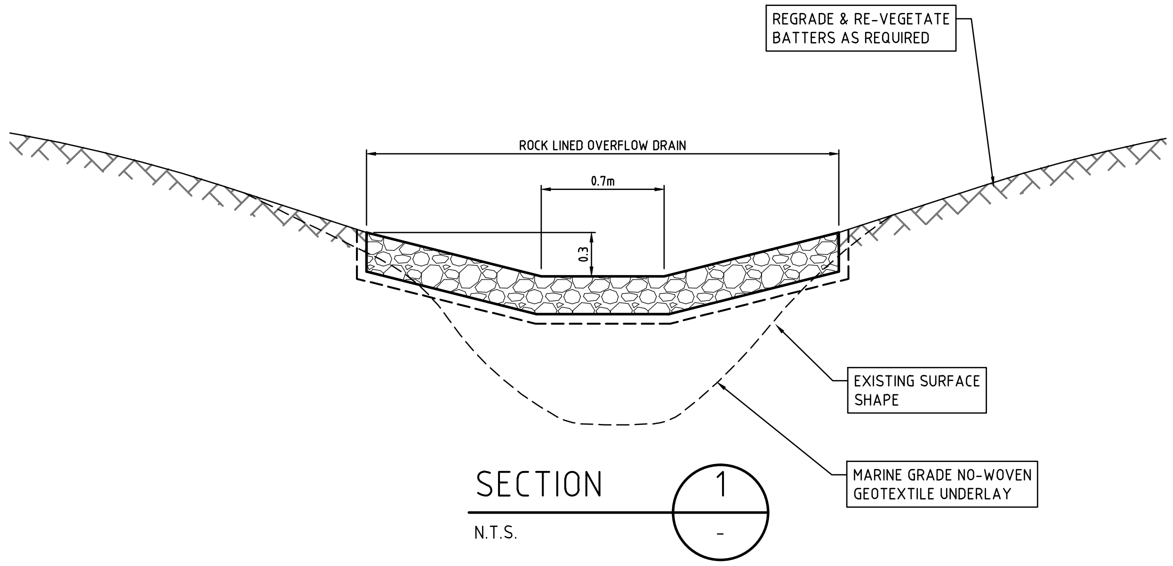
PROJECT NO.	SUB-PR NO.	DRAWING NO.	REV
15000047	P01	SWMP03	00

Plot Date: Friday, 1 April 2016 11:37:20 AM CAD File Name: I:\15000047 Sandy Point_Conroy Park Foreshore Erosion and Drainage Management Plan\Drawings\15000047_P01_C_REV 00.dwg

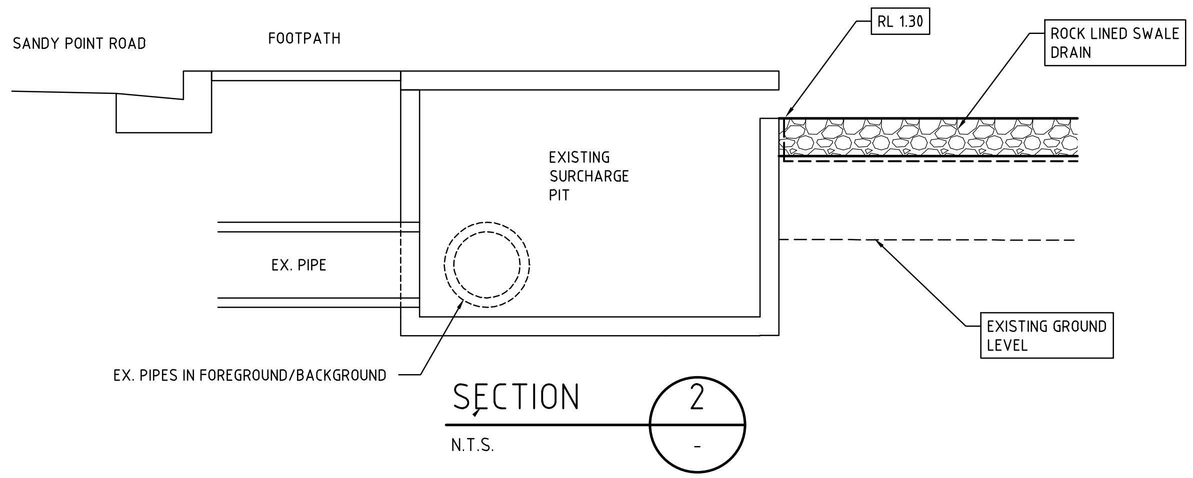


OUTFALL 1 DETAIL PLAN

Scale: 1:100 (A1 SHEET)



TYPICAL SECTION THROUGH ROCK LINED DRAIN



TYPICAL SECTION THROUGH EXISTING SURCHARGE PIT

Scale: 1:20 (A1 SHEET)

REV	DATE	DES.	DRN.	APP.	REVISION DETAILS
00	01/04/16	J.M.A	M.R	J.A	FINAL ISSUE
A	25/08/15	J.M.A.	N.L.	J.A.	DRAFT

DRAWING STATUS

DESIGN BY	J.A.
DRAWN BY	N.L.
FINAL APPROVAL	J.M.A.
SCALE: (on A1 Original)	AS SHOWN

FINAL

North

CLIENT: PORT STEPHENS COUNCIL

SEEC

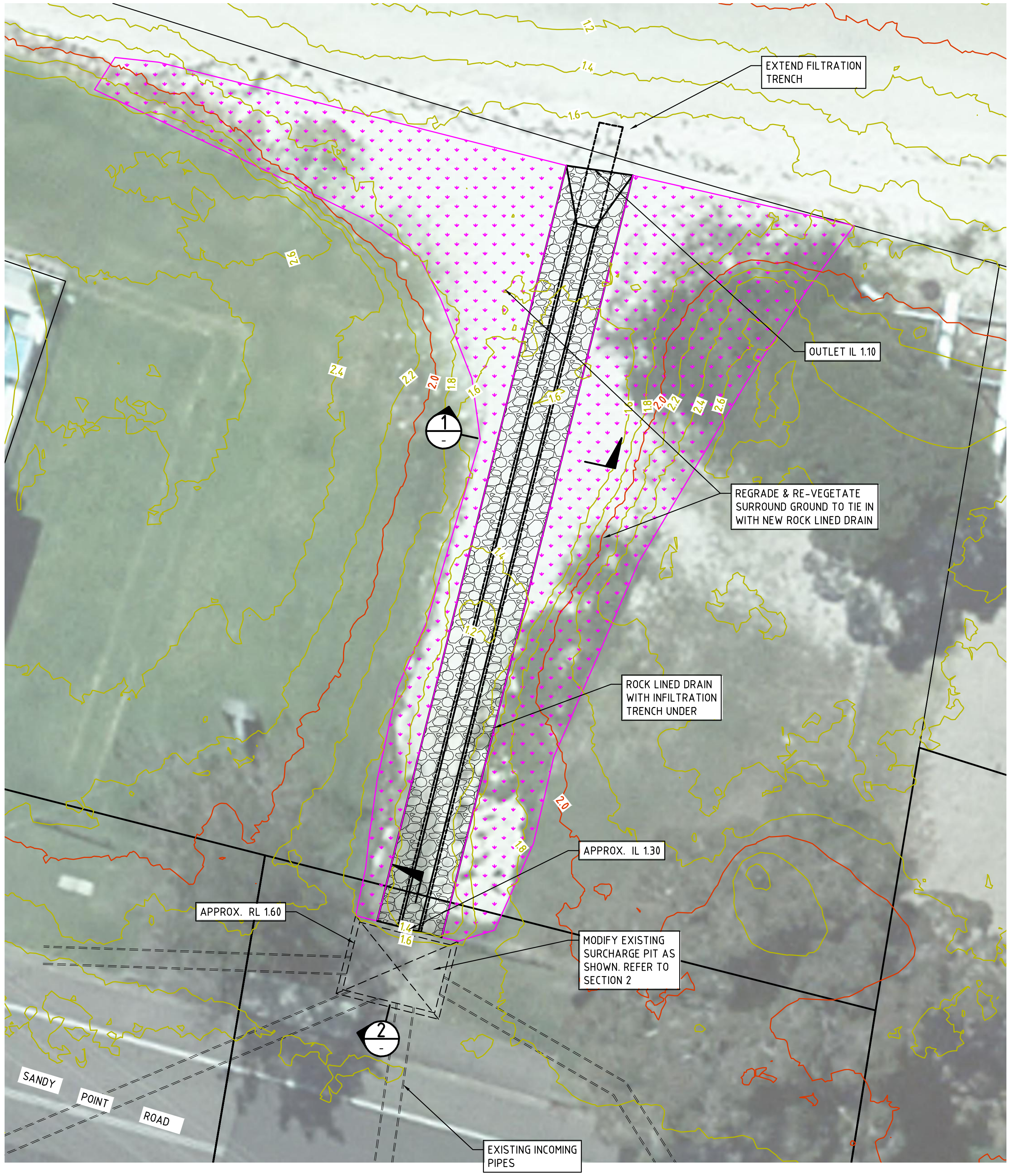
Suites 7 & 8, 68-70 Station Street
PO Box 1098, Bowral NSW 2576.
(t) 02 4862 1633
(f) 02 4862 3088
email: reception@seec.com.au
WWW.SEEC.COM.AU

PROJECT TITLE: EROSION AND DRAINAGE MANAGEMENT PLAN FOR SANDY POINT/CONROY PARK CORLETTE

DRAWING TITLE: OUTFALL 1 CONCEPT DESIGN OPTION 1

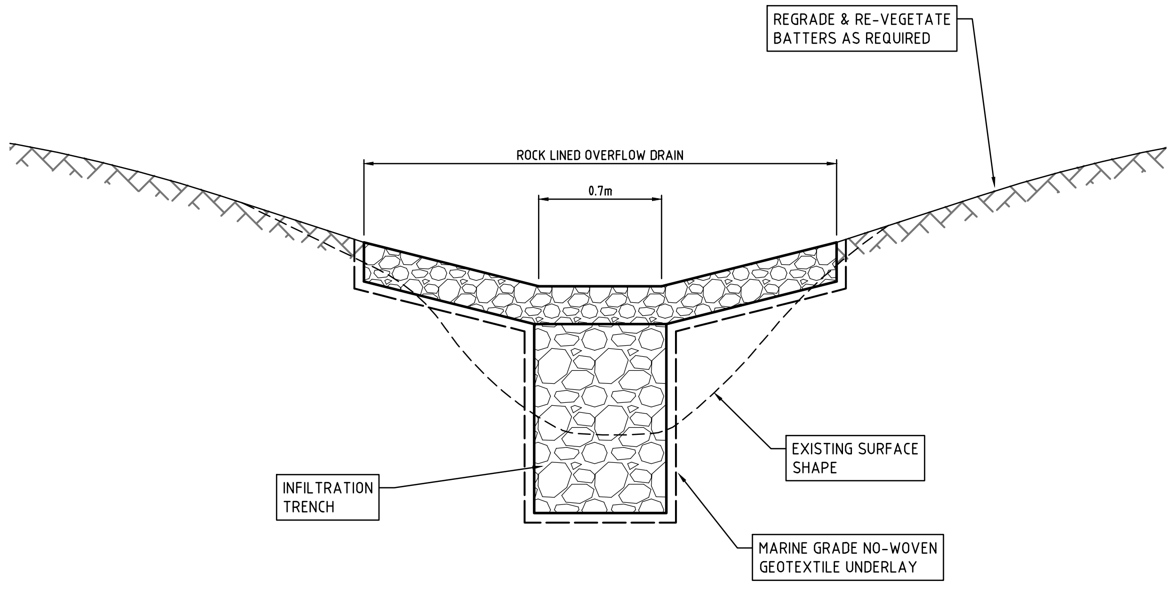
PROJECT NO.	SUB-PR NO.	DRAWING NO.	REV
15000047	P01	SK01	00

Plot Date: Friday, 1 April 2016 11:13:58 AM CAD File Name: I:\15000047 Sandy Point_Conroy Park Foreshore Erosion and Drainage Management Plan\Drawings\15000047_P01_SK_REV 00.dwg



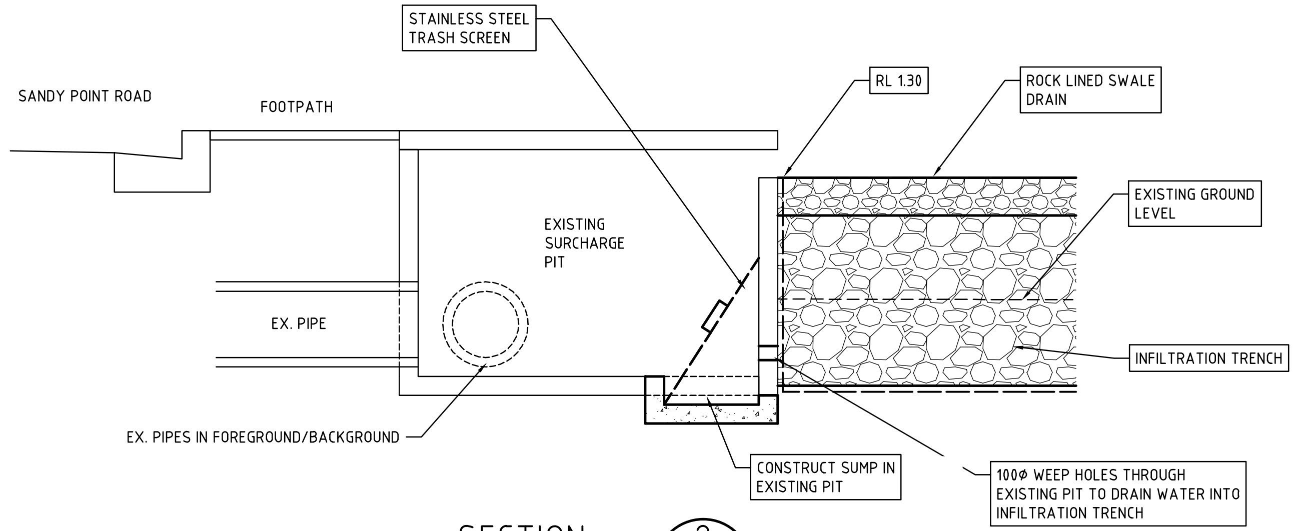
OUTFALL 1 DETAIL PLAN

0 1 2 3 4 5m
Scale: 1:100 (A1 SHEET)



SECTION 1
1:20

TYPICAL SECTION THROUGH ROCK LINED DRAIN



SECTION 2
1:20

TYPICAL SECTION THROUGH EXISTING SURCHARGE PIT

0 0.2 0.4 0.6 0.8 1.0m
Scale: 1:20 (A1 SHEET)

REV	DATE	DES.	DRN.	APP.	REVISION DETAILS
00	01/04/16	J.M.A	M.R	J.A	FINAL ISSUE
A	25/08/15	J.M.A.	N.L.	J.A.	DRAFT

DRAWING STATUS
DESIGN BY J.A.
DRAWN BY N.L.
FINAL APPROVAL J.M.A.
SCALE: (on A1 Original) AS SHOWN
FINAL

North

CLIENT
PORT STEPHENS COUNCIL

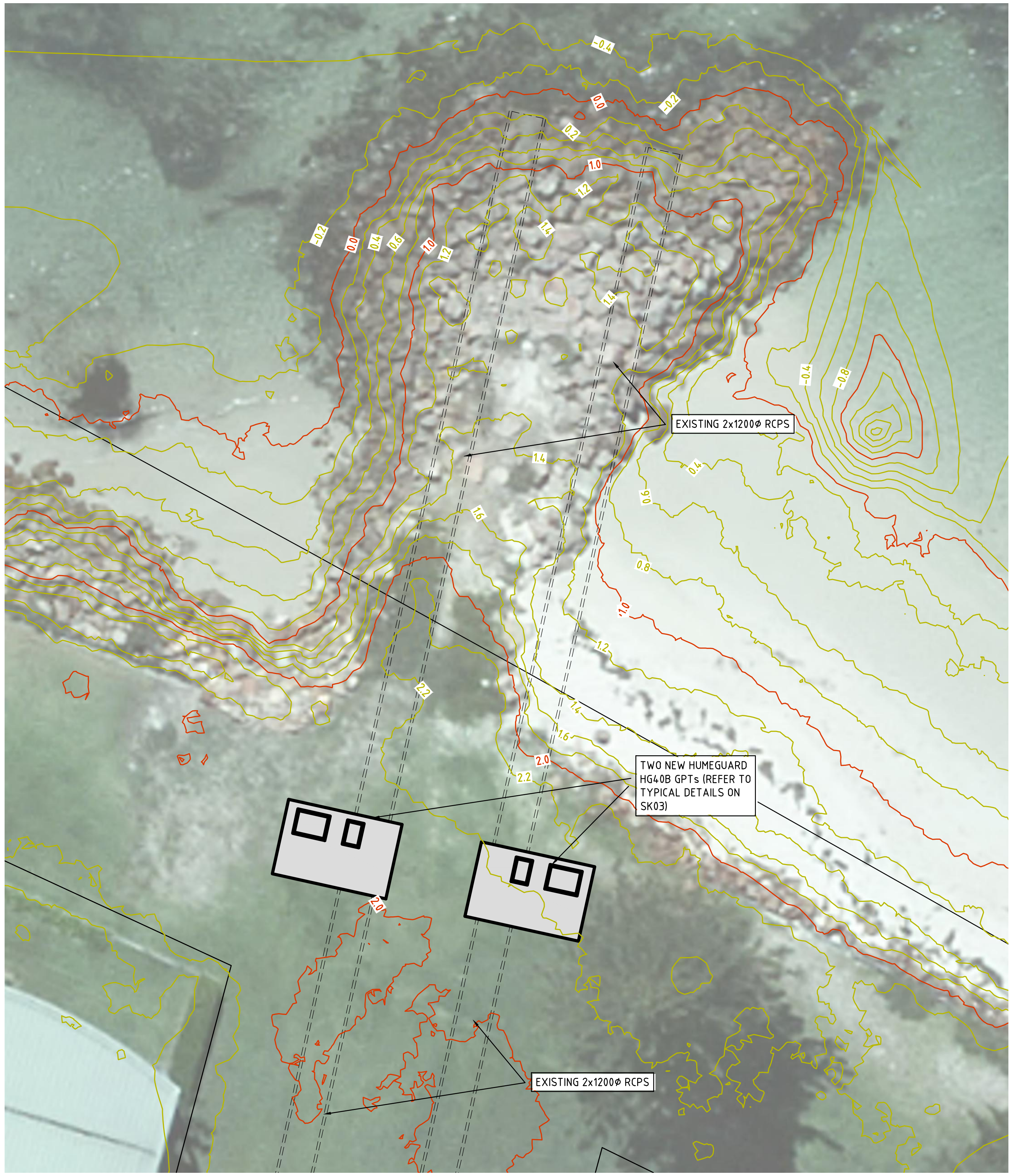
SEEC
Suites 7 & 8, 68-70 Station Street
PO Box 1098, Bowral NSW 2576.
(t) 02 4862 1633
(f) 02 4862 3088
email: reception@seec.com.au
WWW.SEEC.COM.AU

PROJECT TITLE
EROSION AND DRAINAGE MANAGEMENT PLAN FOR SANDY POINT/CONROY PARK CORLETTE

DRAWING TITLE
OUTFALL 1 CONCEPT DESIGN OPTION 2

PROJECT NO.	SUB-PR NO.	DRAWING NO.	REV
15000047	P01	SK02	00

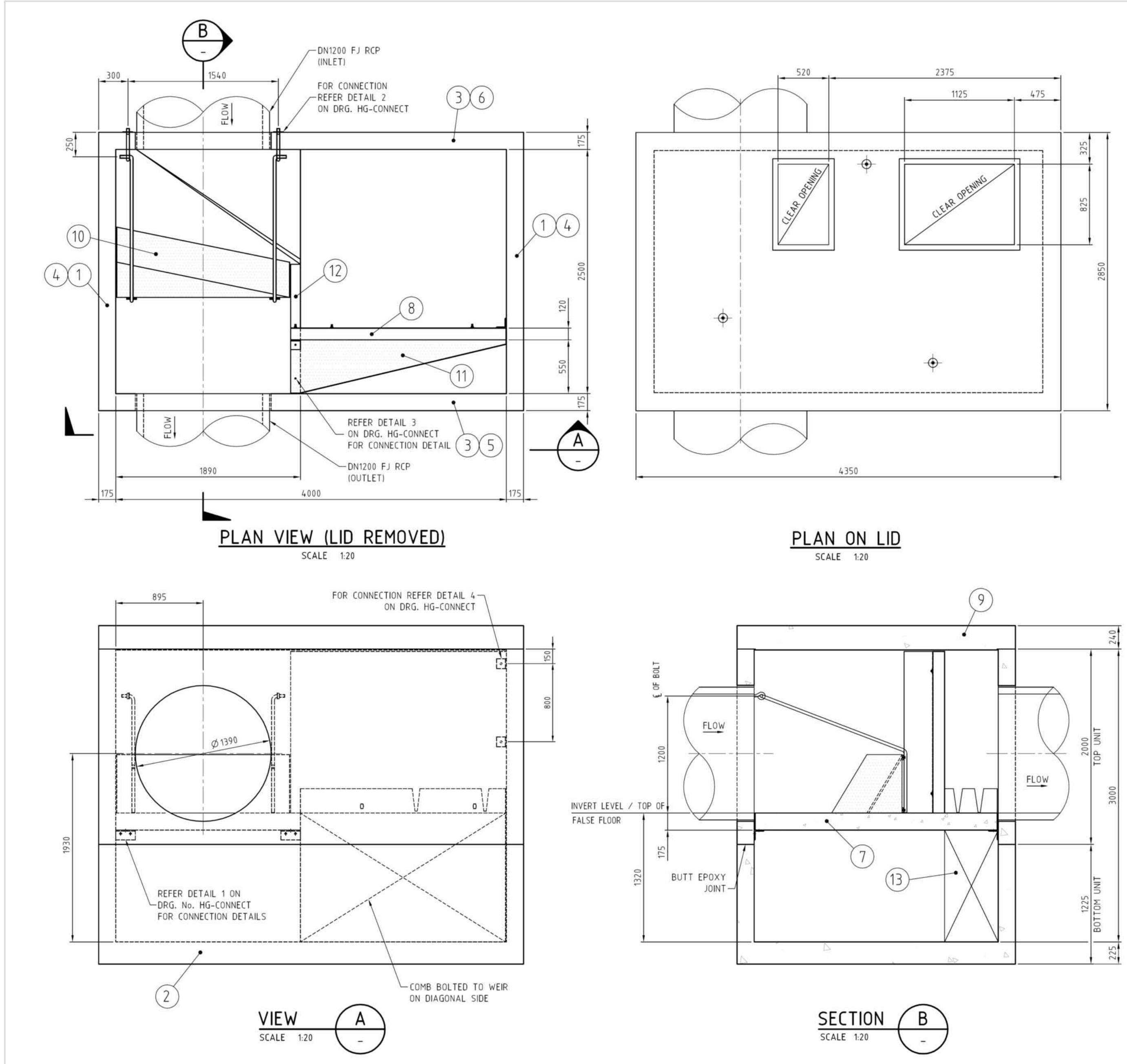
Plot Date: Friday, 1 April 2016 11:14:16 AM CAD File Name: I:\15000047 Sandy Point_Conroy Park Foreshore Erosion and Drainage Management Plan\Drawings\15000047_P01_SK_REV 00.dwg



OUTFALL 2 DETAIL PLAN

0 1 2 3 4 5m

Scale: 1:100 (A1 SHEET)



TYPICAL DETAIL OF HUMEGUARD HG40B/L

NOT TO SCALE

REV	DATE	DES.	DRN.	APP.	REVISION DETAILS
00	01/04/16	J.M.A	M.R	J.A	FINAL ISSUE
A	25/08/15	J.M.A.	N.L.	J.A.	DRAFT

DRAWING STATUS	
DESIGN BY	J.A.
DRAWN BY	N.L.
FINAL APPROVAL	J.M.A.
SCALE: (on A1 Original)	AS SHOWN
FINAL	

North

CLIENT
PORT STEPHENS COUNCIL

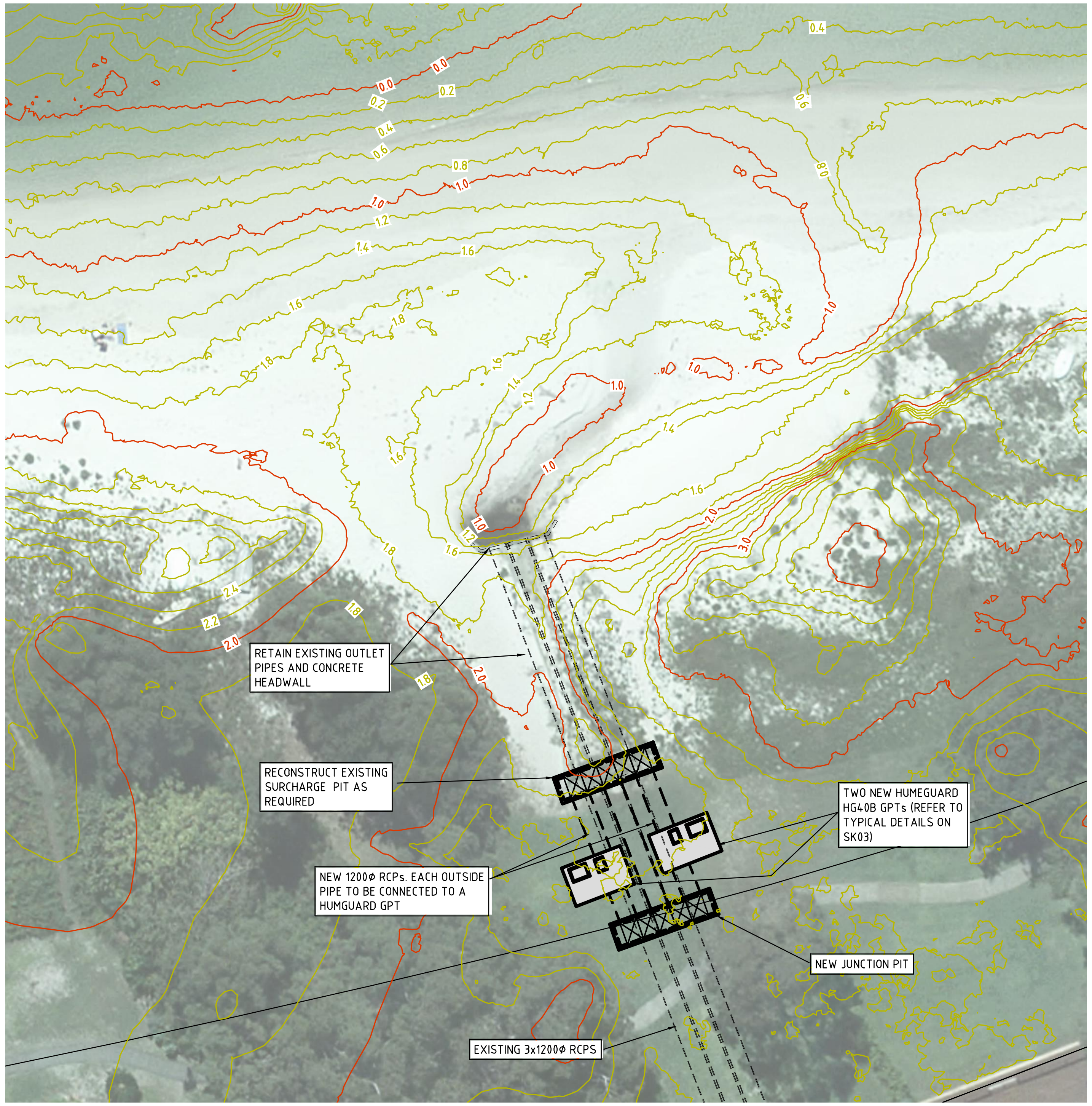
SEEC
Suites 7 & 8, 68-70 Station Street
PO Box 1098, Bowral NSW 2576.
(t) 02 4862 1633
(f) 02 4862 3088
email: reception@seec.com.au
WWW.SEEC.COM.AU

PROJECT TITLE
EROSION AND DRAINAGE MANAGEMENT PLAN FOR SANDY POINT/CONROY PARK CORLETTE

DRAWING TITLE
OUTFALL 2 CONCEPT DESIGN

PROJECT NO.	SUB-PR NO.	DRAWING NO.	REV
15000047	P01	SK03	00

Plot Date: Friday, 1 April 2016 11:14:31 AM   CAD File Name: I:\15000047 Sandy Point_Conroy Park Foreshore Erosion and Drainage Management Plan\Drawings\15000047_P01_SK_REV 00.dwg



OUTFALL 3 DETAIL PLAN

0 2 4 6 8 10m
Scale: 1:200 (A1 SHEET)

REV	DATE	DES.	DRN.	APP.	REVISION DETAILS
00	01/04/16	J.M.A	M.R	J.A	FINAL ISSUE
A	25/08/15	J.M.A.	N.L.	J.A.	DRAFT

DRAWING STATUS	
DESIGN BY	J.A.
DRAWN BY	N.L.
FINAL APPROVAL	J.M.A.
SCALE: (on A1 Original)	AS SHOWN

FINAL

North

CLIENT
PORT STEPHENS COUNCIL

SEEC

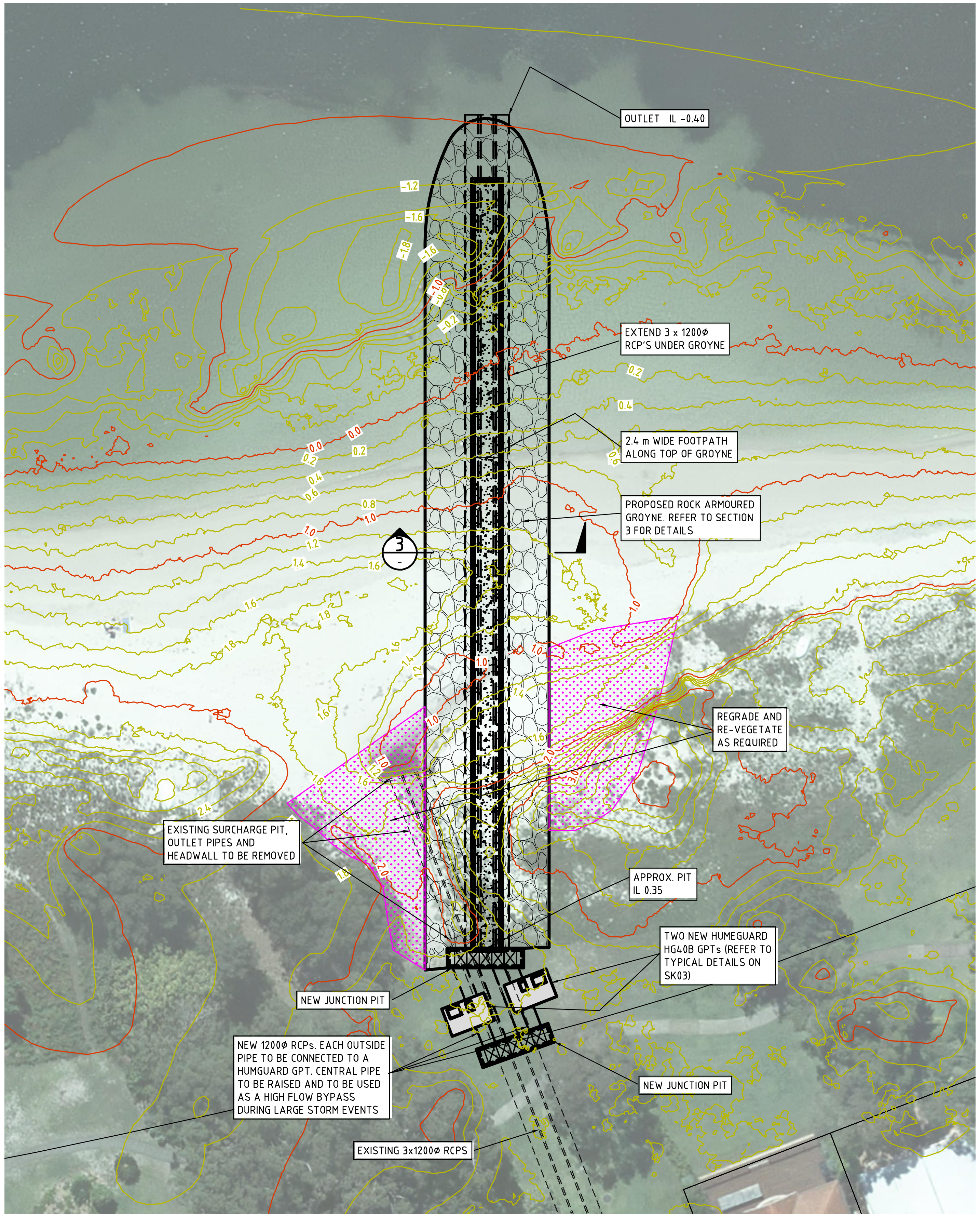
Suites 7 & 8, 68-70 Station Street
PO Box 1098, Bowral NSW 2576.
(t) 02 4862 1633
(f) 02 4862 3088
email: reception@seec.com.au
WWW.SEEC.COM.AU

PROJECT TITLE
EROSION AND DRAINAGE MANAGEMENT PLAN FOR SANDY POINT/CONROY PARK CORLETTE

DRAWING TITLE
OUTFALL 3 CONCEPT DESIGN OPTION 1

PROJECT NO.	SUB-PR NO.	DRAWING NO.	REV
15000047	P01	SK04	00

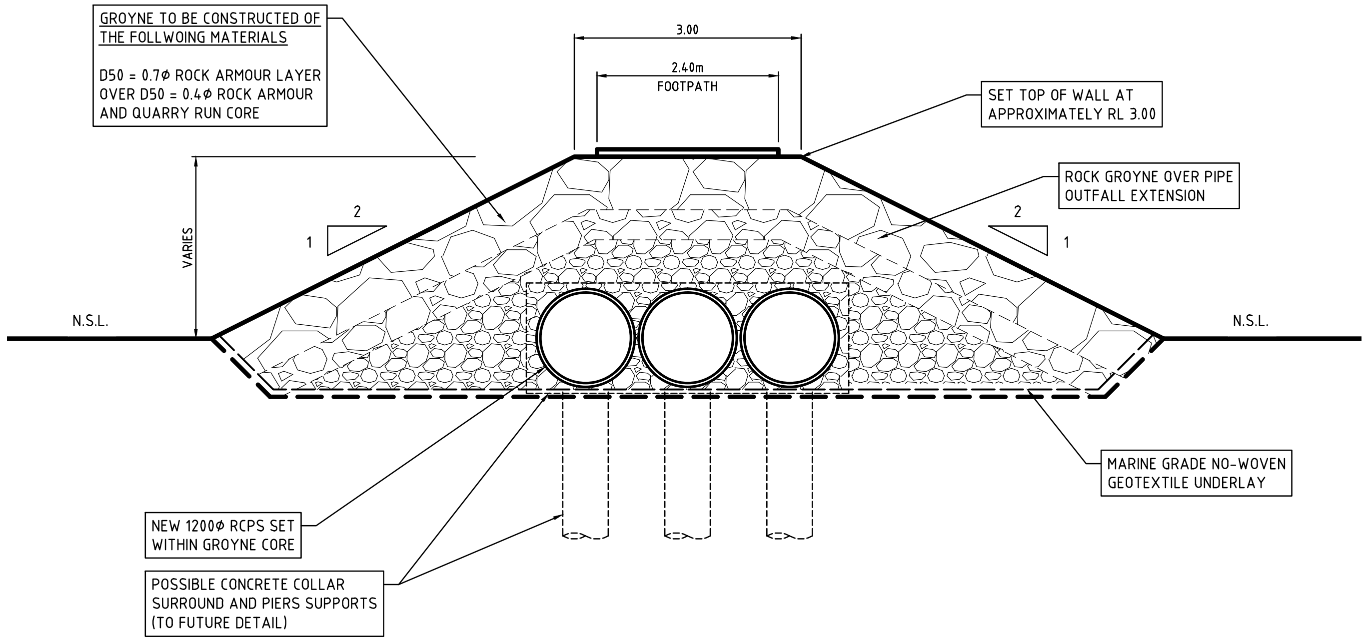
Plot Date: Friday, 1 April 2016 11:14:40 AM CAD File Name: I:\15000047 Sandy Point_Conroy Park Foreshore Erosion and Drainage Management Plan\Drawings\15000047_P01_SK_REV 00.dwg



OUTFALL 3 DETAIL PLAN

0 2.5 5 7.5 10 12.5m

Scale: 1:250 (A1 SHEET)



SECTION 3
1:50

TYPICAL SECTION THROUGH GROYNE

0 0.5 1 1.5 2 2.5m

Scale: 1:50 (A1 SHEET)

REV	DATE	DES.	DRN.	APP.	REVISION DETAILS
00	01/04/16	J.M.A	M.R	J.A	FINAL ISSUE
A	25/08/15	J.M.A.	N.L.	J.A.	DRAFT ISSUE

DRAWING STATUS
DESIGN BY J.A.
DRAWN BY N.L.
FINAL APPROVAL J.M.A.
SCALE: (on A1 Original) AS SHOWN

FINAL

North

CLIENT
PORT STEPHENS COUNCIL

SEEC
Suites 7 & 8, 68-70 Station Street
PO Box 1098, Bowral NSW 2576.
(t) 02 4862 1633
(f) 02 4862 3088
email: reception@seec.com.au
WWW.SEEC.COM.AU

PROJECT TITLE
EROSION AND DRAINAGE MANAGEMENT PLAN FOR SANDY POINT/CONROY PARK CORLETTE

DRAWING TITLE
OUTFALL 3 CONCEPT DESIGN OPTION 2

PROJECT NO.	SUB-PR NO.	DRAWING NO.	REV
15000047	P01	SK05	00

Plot Date: Friday, 1 April 2016 11:14:53 AM CAD File Name: I:\15000047 Sandy Point_Conroy Park Foreshore Erosion and Drainage Management Plan\Drawings\15000047_P01_SK_REV 00.dwg

Appendix C Summary of Collected Ground and Hydro Survey Data

Background

As part of the "Sandy Point / Conroy Park Foreshore Erosion and Drainage Management Plan, a full survey of the foreshore and nearshore area was undertaken. The final survey plans are attached here, although a number of digital products were also provided to Port Stephens Council for future use as required. These digital products are detailed in Table C1.

Table C 1 Description of Digital Survey Data Provided to Council

File Name	Description
conpk_150524_points.laz	Compressed point cloud (laser scan) file format of photogrammetrically derived UAV survey data. The point cloud includes all 3d points for around 1 block back from the foreshore. Prepared by Propeller Aerobotics.
con_model.dxf conpk_half_model.dxf conpk_quarter_model.dxf	AutoCAD text file formats of triangulated ground surface covering the same area as the point cloud. Objects such as houses and trees have been removed to provide a digital elevation model of the study area. The files are large and different resolutions are provided for convenience. Prepared by Propeller Aerobotics.
150707_PRT_STEPHENS.dwg	AutoCAD version of final survey plan derived from the ground models prepared by Propeller Aerobotics and hydrographic survey undertaken by McGlashan and Crisp.
150707_PRT_STEPHENS_EAST.pdf 150707_PRT_STEPHENS_WEST.pdf 150707_PRT_STEPHENS_COMPLETE.pdf	.pdf plots of three views from the corresponding .dwg file covering the eastern end, western end and complete study area respectively. These figures are presented in the following pages
conpk_150524_ortho_Georefenced.tif	Very high resolution stitched vertical image of the study area derived from the drone survey. Geotiff format referenced to MGA 56.

LGA PORT STEPHENS		Drawn F.H.	LEVEL BOOK
Locality		Checked J.C.	
DATUM MGA/AHD	AZIMUTH MGA	Date 03/07/2015	COMPUTER FILE
		Scale 1/3000 @A2	

McGLASHAN & CRISP Pty Ltd
CONSULTING SURVEYORS
117 VICTORIA STREET, TAREE 2430. Ph:02 65521566. DX 7009
EMAIL: admin@mcglashancrisp.com.au

FILE No. F993	JOB No.
No. IN SET	SHEET No.

CLIENT: PORT STEPHENS COUNCIL

PROJECT: SURVEY OF CORLETTE

TITLE: PLAN SHOWING HYDROGRAPHIC & TOPOGRAPHIC CONTOURS BETWEEN SANDY POINT AND ANCHORAGE RESORT

NOTES:
CONTOUR INTERVAL 0.5 M
PHOTO UNDERLAY DATING TO AUGUST 2014 AND FIXED TO BEST FIT
DATUM TO AHD ORIGIN SSM8265 RL 2.331
TOPOGRAPHIC CONTOURS DERIVED BY PHOTOGRAMMETRY
HYDROGRAPHIC CONTOURS DERIVED FROM SOUNDINGS

LGA PORT STEPHENS Locality DATUM MGA/AHD AZIMUTH MGA	Drawn F.H. Checked J.C. Date 07/07/2015 Scale 1/1000 @A2	LEVEL BOOK COMPUTER FILE	McGLASHAN & CRISP Pty Ltd CONSULTING SURVEYORS 117 VICTORIA STREET, TAREE 2430. Ph:02 65521566. DX 7009 EMAIL: admin@mcglashancrisp.com.au	FILE No. F993 No. IN SET	JOB No. SHEET No.	CLIENT PORT STEPHENS COUNCIL PROJECT SURVEY OF CORLETTE	TITLE PLAN SHOWING HYDROGRAPHIC & TOPOGRAPHIC CONTOURS EAST OF SANDY POINT

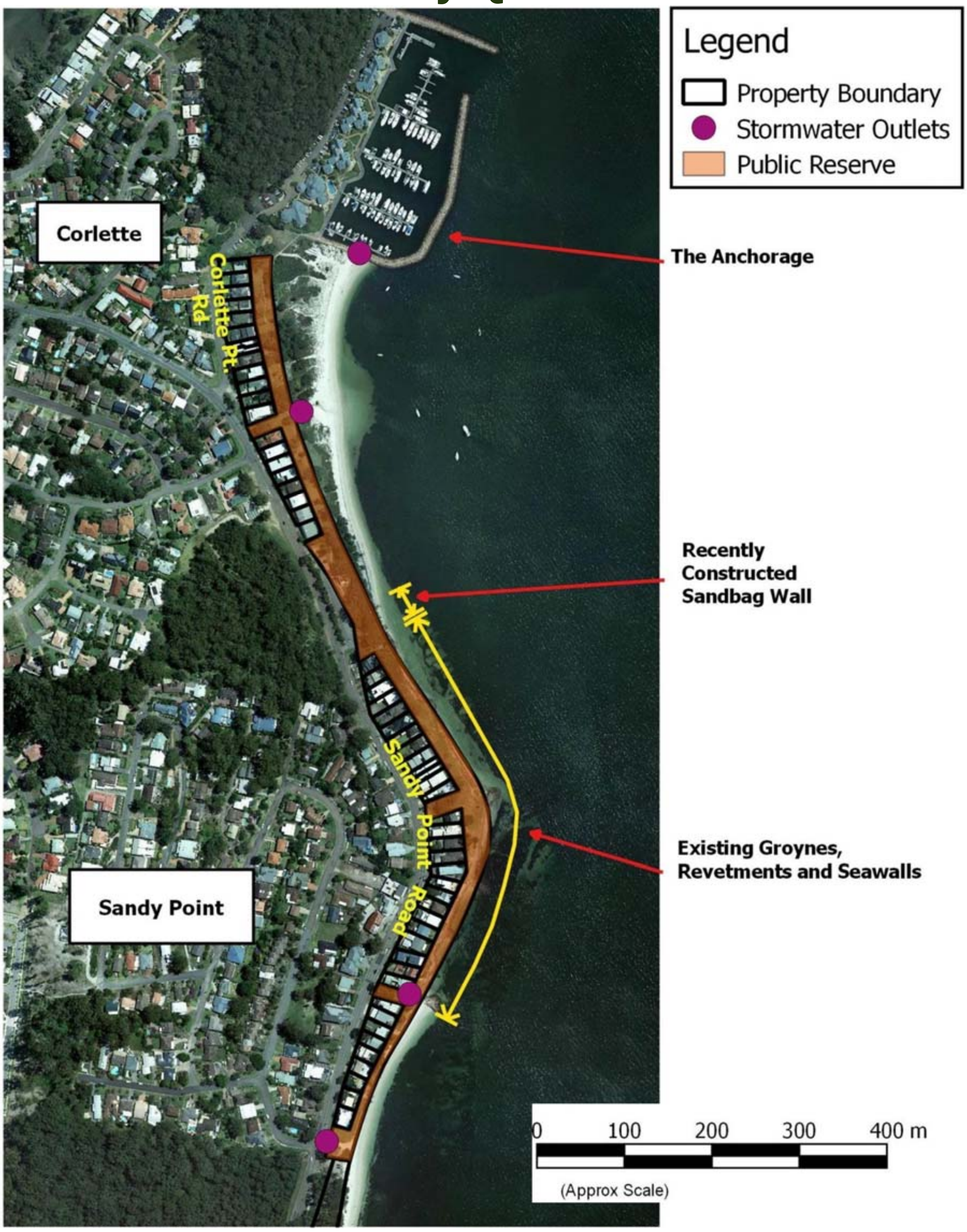
NOTES:
CONTOUR INTERVAL 0.5 M
PHOTO UNDERLAY DATING TO AUGUST 2014 AND FIXED TO BEST FIT
DATUM TO AHD ORIGIN SSM8265 RL 2.331
TOPOGRAPHIC CONTOURS DERIVED BY PHOTOGRAMETRY
HYDROGRAPHIC CONTOURS DERIVED FROM SOUNDINGS

LGA	PORT STEPHENS
Locality	
DATUM	MGA/AHD
AZIMUTH	MGA
Drawn	F.H.
Checked	J.C.
Date	07/07/2015
Scale	1/1000 @A2
LEVEL BOOK	
COMPUTER FILE	
FILE No.	F993
No. IN SET	
JOB No.	
SHEET No.	
CLIENT	PORT STEPHENS COUNCIL
PROJECT	SURVEY OF CORLETTE
TITLE	PLAN SHOWING HYDROGRAPHIC & TOPOGRAPHIC CONTOURS EAST OF THE ANCHORAGE RESORT

McGLASHAN & CRISP Pty Ltd
CONSULTING SURVEYORS
117 VICTORIA STREET, TAREE 2430. Ph:02 65521566. DX 7009
EMAIL: admin@mcglashancrisp.com.au

Appendix D Community Questionnaire

Sandy Point - Conroy Park
Foreshore Erosion & Drainage Management Plan
Community Questionnaire



Address for returning questionnaires:
Philippa Hill
Coast and Estuaries Officer
Port Stephens Council
PO Box 42, Raymond Terrace, NSW 2324

Alternatively, responses can be emailed to philippa.hill@portstephens.nsw.gov.au

Background

Foreshore erosion at Sandy Point and along the Conroy Park shoreline (east of the Anchorage) has been evident for a number of decades. Various protection measures have been constructed over time and recently, Council has further addressed the problem by extending the protection westwards in front of Conroy Park (using geotextile 'sand bags') in accordance with NSW Government policy. Further interim works are currently being investigated to protect trees at the western end of the sand bags. Private residential property along this foreshore is separated from the waterway by a public reserve, to enable continuous public access along the foreshore. Continuing foreshore erosion, wave overtopping during storms, reduction in public access along the foreshore and flooding and scour at stormwater outlets all remain as issues.

Port Stephens Council has commissioned a study to examine the causes for the erosion, evaluate the effectiveness of existing structures and to evaluate possible options to address these ongoing issues.

As part of the study, we are seeking input from the local community to better understand these issues and to develop and assess options that would match broad community expectations. The information collected from the community will be combined with information contained in previous studies and further investigations to be undertaken as part of this study. This questionnaire has been prepared for residents in and around the Sandy Point and Conroy Park foreshores to get feedback from the local community. However, it can be completed by anyone with an interest in this stretch of foreshore.

In addition to the questionnaire, a limited number of follow up interviews will be conducted and there is an option at the end of this questionnaire for owner/residents with beach front properties, to indicate whether you would like to be interviewed.

A reply paid envelope has been provided for your convenience. Please return completed questionnaires to the address on the first page by 22 of May so that your answers can be considered during the study. It is not necessary to answer all questions, but any information will be helpful. Names and addresses will not be made public but responses may be published in full.

Who Are You?

1. What is your current residential address?

____

2. Are you a permanent resident in the foreshore study area?

- ☐ Owner and Occupier
- ☐ Absentee Owner
- ☐ Tenant
- ☐ No, but I'm an interested community member

3. How long have you lived near Sandy Point?

- ☐ < 2 years
- ☐ 2 – 5 years
- ☐ 5 - 10 years
- ☐ 10 – 20 years
- ☐ >20 years (please tell us how long): ____________________

4. How do you use the beaches and waterway between Sandy Point and the Anchorage? (Select more than one if appropriate)

- ☐ Passive Recreation (e.g. walking, shore line fishing, photography)
- ☐ Active recreation (e.g. boating/sailing, swimming, canoe/paddle board/kite, boat fishing)
- ☐ Other (Please describe) ______________________________

____

____

What else do you value about the foreshore?

____

____

____

____

____

Changes to the Foreshores

5. In the time that you have lived here, what changes have you seen along the Sandy Point/Conroy Park shoreline and beaches? i.e. erosion, recession, changes in water depth, vegetation, seagrass, drainage issues (e.g. beach erosion, flooding) etc.

____

____

____

____

____

____

____

6. Do you have any further information (such as historical photographs, reports or documents) which you can provide to the study team?

- [ ] Yes
- [ ] No

If yes, please return copies of this information with your response. Alternatively a study team member can contact you to discuss further - Please provide your telephone number). We are particularly interested in historical photographs you may have of the foreshore and its use/change. Photos of interest should be of known locations and with the approximate year known. **Do not enclose original photos** with your response (copies, not to be returned are OK) Alternatively, you may indicate whether you have photos which you think may be relevant and which you would like to show us.

7. Do you think the changes have become more pronounced in recent years, or are they slowing down?

8. What do you think has caused any identified changes to the foreshores?

9. What would you identify as the main issues which need to be addressed through a management plan for the beaches and shoreline in the area? (ranking)

- [ ] Foreshore erosion and recession
- [ ] Ocean inundation
- [ ] Loss of public access
- [ ] Stormwater drainage and flooding
- [ ] Other:

Please expand and note specific locations if applicable (next page):

Management Options

10. What would you like to see in the future management of the foreshore? (please rank)

- ☐ Rock revetment (sloping rock wall)
- ☐ Sand nourishment
- ☐ More native low vegetation
- ☐ More shade
- ☐ More public access through the reserve
- ☐ Better public access to the water
- ☐ Improved public safety
- ☐ Other, please expand below….

What do you <u>NOT</u> want to see on the foreshore? (please rank)

- ☐ Rock revetment (sloping rock wall)
- ☐ Sand nourishment
- ☐ More native low vegetation
- ☐ More shade
- ☐ More public access through the reserve
- ☐ Better public access to the water
- ☐ Improved public safety
- ☐ Other, please expand below….

11. What do you think would be the benefits of your preferred options from Question 10?

- ______
- ______
- ______
- ______
- ______

Further Contact

12. If you are a waterfront owner/ resident in the area, would you like to take part in a follow up interview?

☐ Yes ☐ No

If yes, can you please provide your name, telephone number so that a member of our study team can contact you?

Name: ______ **Phone Number:** ______

Please use the following lines if you wish to expand on any answers provided above.

______

Appendix E Multi Criteria Analysis Results

Methodology

A long list of feasible options was determined for the six precincts and these were assessed using a multi criteria assessment method. The criteria against which the options were assessed for each precinct were:

- **Public Access**: Referring to either an existing level of use by the public for recreation, and whether this is presently difficult, threatened or could be improved or impeded;
- **Public Safety**: Referring to whether a particular option could either improve or negatively affect safety of the public when using the foreshore;
- **Recreation / Boating**: Referring to whether options are likely to improve or detract from recreational amenity of the foreshore;
- **Foreshore Protection From Erosion**: Referring to whether the particular option would significantly improve protection of the foreshore from erosion;
- **Foreshore Protection From Overtopping**: Referring to whether the particular option would significantly improve protection of the foreshore from overtopping;
- **Impact on Coastal Processes**: Referring to whether the option would have a positive or negative impact on broader coastal processes in adjacent precincts;
- **Seagrasses / Ecology**: Referring to whether the option would tend to enhance or detract from nearshore seagrass habitat;
- **Provision of a Sandy Beach**: Referring to whether the option tends to enhance the provision of a sandy beach, which is seen by many in the community as desirable;
- **Enhancement of Dune / Native Vegetation**: Referring to whether the option would tend to create opportunities to create or enhance coastal dunes & vegetation;
- **Management of Stormwater**: Referring to whether the option would tend to improve the handling of stormwater issues, including water quality, the amount of sand scoured from the beach and ease of maintenance;
- **Aesthetics**: Referring to whether the option would tend to improve or detract from the general appearance of the foreshore and associated beaches;
- **Residential Security**: Referring to whether the option would tend to adversely impact the privacy of residents and/or affect the potential for burglary / theft;
- **Adaptability**: Referring to whether the option incorporates the ability to adapt to changing conditions, such as the movement of the flood tide delta affecting wave focussing along the foreshore, or a rise in mean sea level; and
- **Ease of Construction**: Referring to whether the option involves difficult, in-water construction or whether there is limited foreshore access, which would increase the risk of unforeseen costs during construction.

A total of six individuals, including three members of the study team, and three Council staff members were provided with lists of these 14 criteria and asked to grade the importance of those issues for each of the six precincts using the following scale:

- A – Critically Important;

- B - Very Important;
- C – Important;
- D - A Bit Important; and
- E - Not Important / Irrelevant.

Values of A through E were converted to values of 4 through 0 respectively for subsequent calculation. All individuals that took part had been either involved in consultation activities as part of the project, or had experience in management of foreshores and drainage within the study area.

The long list of feasible options are summarised in the following sections. Again, three engineers from W&A and CE were asked to score how well the options performed against each of the 14 criteria. In this instance the following scale was adopted:

- -2 – Addresses issue well;
- -1 – Somewhat addresses issue;
- 0 – Irrelevant / has neutral impact;
- +1 – Has somewhat negative impact; and
- +2 – Makes the situation significantly worse

For each issue/option combination, the average issue importance and option performance scores were multiplied together, considering the responses of all participants. These were then totalled to give an overall score for each of the options. The overall score is representative of the level of benefit that would result from that option. For each precinct the options were subsequently ranked.

The outcomes of the multi-criteria analysis are presented below. However, this analysis has some weaknesses, for example:

Different individuals will interpret the scoring/ranking criteria differently; and

The analysis does not incorporate the compatibility of options between precincts.

The ranking of each option in the multi criteria analysis, and further consideration of limitations are discussed below, along with a "ball park" estimate of costs. Considering all aspects, three final short-listed "schemes", comprising compatible treatments in adjacent precincts is then provided.

Precinct 1 Options

Option 1: Do nothing. This was the highest scoring option for Precinct 1, with the majority of benefits seen to accrue from the continued provision of a sandy beach and the aesthetics at the western end of Corlette Beach. Particular problems will arise from continued accumulation of sand at the stormwater outlets adjacent to the breakwater.

<u>Benefit Score (Rank)</u>: 12.1 (1/5)

Option 2: Removal of sand. Sand has accumulated in this area since breakwater construction and the sand here could be used for beneficial purposes elsewhere. However, removal of the sand will impact on the aesthetics, public access and would potentially cause loss of dune vegetation.

Benefit Score (Rank): -1.33 (3/5)

Option 3:Construct groyne to convey stormwater. This would prevent sand from the beach face being jetted into the nearshore zone and smothering seagrasses during stormwater runoff events. While it would likely be excellent at addressing stormwater issues, it would have negative impacts on aesthetics, public access and safety.

Benefit Score (Rank):-2.56 (4/5)

Option 4: Option 3 + Option 2. This option has the combined impacts of options 2 and 3, and given that both of those options have an assessed negative benefit score, this option is the most poorly scoring of all options considered for Precinct 1.

Benefit Score (Rank):-4.67 (5/5)

Option 5: No sand removal; extend stormwater lines adjacent to The Anchorage. This is similar to the Option 1, although the interaction of the stormwater extension with beach access, safety and continued widening of the beach has resulted in it having a comparatively lower benefit score. Even so, this option should be considered.

Benefit Score (Rank):7.39 (2/5)

Precinct 2 Options

Option 1: Do nothing. Doing nothing will cause the foreshore to have continued problems with overtopping and erosion, retaining a situation that has issues with public safety, lacks a sandy beach and is suboptimal for recreation and aesthetics.

Benefit Score (Rank): -24.10 (10/10)

Option 2: Nourish with sand from next to Anchorage. While this option has a negative impact on Precinct 1, it is beneficial for Precinct 2 and is ranked second for this Precinct. This option vastly improves on the do nothing option in relation to public access, public safety, foreshore protection, overtopping and aesthetics.

Benefit Score (Rank): 29.24 (1/10)

Option 3: Nourish with sand from edge of delta dropover. This option scores similarly to Option 2, with the exception that it is better for coastal processes along the entire length of Corlette Beach (i.e. not taking from one end to provide sand to the other).

Benefit Score (Rank): 25.99(4/10)

Option 4: Nourishment plus construction of groyne at S/W outlet in Precinct 1. Again, scores similarly to Options 2 and 3, with the exception that the groyne improves the handling of stormwater and the stability of the Beach in Precinct 2. There is, however, a negative aesthetic impact. While this option scores well for this precinct, construction of a groyne is less favourable for Precinct 1.

<u>Benefit Score (Rank)</u>: 29.23 (2/10)

Option 5: Progressive construction of a rock revetment, as required. This option will protect the foreshore from erosion and overtopping, but will not achieve the benefits obtained for recreation and dune/native vegetation that would arise from having a sandy beach.

<u>Benefit Score (Rank)</u>: 5.38 (7/10)

Option 6: Progressive construction of a geotextile sandbag wall. This option was considered aesthetically poor and does not provide significantly better long term protection from erosion and overtopping over time. Again, it will not achieve the benefits of having a sandy beach.

<u>Benefit Score (Rank)</u>: -9.88 (9/10)

Option 7: Progressive rock revetment and construction of groyne at S/W outlet in Precinct 1. Similar benefits to option 5, with the exception that stormwater is managed better and the groyne acts to stabilise the beach to a larger extent within Precinct 2.

<u>Benefit Score (Rank)</u>: 12.47 (5/10)

Option 8: Offshore breakwater: There have been difficulties in achieving effective outcomes using offshore breakwaters, multipurpose or surfing reefs in Australia and around the world. Particularly in a situation such as Corlette Beach, with oblique waves and a strongly bimodal wave climate, we doubt that this would be an effective solution.

<u>Benefit Score (Rank)</u>: - 7.33 (8/10)

Option 9: Groyne at western end of Conroy Park, with more groynes constructed as required: This would retain a beach of some width, would presumable eventually improve stormwater handling in Precinct 1, although progressive downdrift erosion would continue. While a properly designed groyne field would provide effective protection from erosion and overtopping, a broad sandy beach such as that which was present in the past is unlikely to be achieved and the aesthetics of the area would continue to be compromised.

<u>Benefit Score (Rank)</u>: 5.90 (6/10)

Option 10: Option 9 + Nourishment: This would drastically improve Option 9, by providing a sandy beach, with its attendant benefits along this foreshore.

<u>Benefit Score (Rank)</u>: 27.41 (3/10)

Precinct 3 Options

Option 1: Do nothing. The do nothing option is seen as problematic for public safety and overtopping. Furthermore, the existing structure is failing to adequately protect the foreshore from erosion and the overall foreshore interface is unsightly. This option ranks most poorly of the 9 considered for Precinct 3.

<u>Benefit Score (Rank)</u>: -4.64 (9/9)

Option 2: Relocate fence away from crest, fill gaps in revetment and repair obvious failures. This option comprises an aesthetic treatment of the revetment structure and measures to improve public safety which is an issue due to the high, steep nature of the revetment. It does not address the issues associated with erosion of this foreshore.

<u>Benefit Score (Rank)</u>: -3.17 (8/9)

Option 3: Remove stairs, ramps and revetment crossings and rationalise public access. This option includes additional steps to remove unsafe access points across the revetment and is good from the point of view of public access and safety. Even so, it does not address foreshore erosion adequately.

<u>Benefit Score (Rank)</u>: 11.80 (6/9)

Option 4: Options 2 & 3 plus batter back foreshore and reconstruct revetment. This option achieves the positive outcomes of the previous two options, while making a considered effort towards eliminating problems with foreshore erosion. It does not, however provide a sandy beach at the foreshore.

<u>Benefit Score (Rank)</u>: 21.73 (3/9)

Option 5: Options 2 & 3 plus construct revetment seaward to avoid loss of public reserve: Within Precinct 3, the foreshore reserve is wide, meaning that the additional costs associated with reclaiming part of the foreshore to accommodate a reconstructed revetment is not warranted. It provides similar protection to Option 4, but would be more difficult to construct.

<u>Benefit Score (Rank)</u>: 17.03 (4/9)

Option 6: Options 2 & 3 plus bolster and extend Groyne A. This provides some additional protection from overtopping, and is beneficial to the retention of sand in Precinct 4. It does not provide the level of protection from overtopping and erosion provided by options 4 and 5.

<u>Benefit Score (Rank):</u> 12.87 (5/9)

Option 7: Option 6 plus batter back and reconstruct revetment. This combines the benefits of Options 4 and 6, resulting in an outcome which is ranked second in terms of benefits for Precinct 3.

Benefit Score (Rank): 22.94 (2/9)

Option 8: Repair, bolster and extend Groyne A plus nourishment. This option is less favourable than, say, options 4,5 or 7, as it does not provide the level of protection from erosion as those other options, and does not robustly address safety issues.

Benefit Score (Rank): 9.08 (7/9)

Option 9: Options 4 & 8, plus construction of two artificial headlands. The two 'artificial headlands' considered here are rhythmic protrusions that have formed in this length of foreshore as the shoreline has historically adjusted to the prevailing wave climate. One is located on the eastern end of Conroy Park and the other midway between Groyne A and Conroy Park. Bolstering these, by building them out slightly further will result in more definite pocket beaches that could be nourished. However, these would require nourishment on a fairly regular basis. In terms of benefits, this is the most favourable option, but also one of the most expensive ones.

Benefit Score (Rank): 34.76 (1/9)

Precinct 4 Options

Option 1: Do Nothing. This option is problematic from foreshore erosion, overtopping and coastal processes points of view. The foreshore is also unsightly and there are issues with public safety, residential security and public access.

Benefit Score (Rank): -15.80 (6/6)

Option 2: Rebuild and bolster foreshore revetment (limited scope for battering back, some reclamation will be required). Bolstering the foreshore revetment will provide better protection from erosion and overtopping whilst also improving public access and safety in the area. Construction would be somewhat difficult making this option less favourable than the first ranked Option 5.

Benefit Score (Rank): 11.16 (3/6)

Option 3: Groyne A, extend and reconstruct. This option is not favourable due to the difficulty in construction and relatively low level of protection it provides from overtopping.

Benefit Score (Rank): -4.73 (5/6)

Option 4: Groyne B, extend and reconstruct. This option is also not favourable for similar reasons to Option 3.

Benefit Score (Rank): -4.21 (4/6)

Option 5: Options [2] + [3] + [4]. Option 5 scored the highest due to its positive impact on public access, public safety, the protection it provides from erosion and overtopping and its provision of a sandy beach. Even though this option scored the highest overall, it would require substantial construction effort.

Benefit Score (Rank): 22.39 (1/6)

Option 6: [5] + Nourish Beach. Option 6, while scoring reasonably was seen to result in the potential smothering of seagrass beds and having a possible negative effect on residential security by encouraging higher usage of the foreshore in a residential area.

Benefit Score (Rank):13.60 (2/6)

Precinct 5 Options

Option 1: Do Nothing. This option is problematic for public access and safety and scores poorly against foreshore protection from erosion and overtopping. This option is also poor for recreation purposes, lack of a sandy beach and residential security.

Benefit Score (Rank): -24.22 (8/8)

Option 2: Remove boat ramps, reconstruct and raise walls in present location, replacing with uniform rock revetment. This option achieves good protection of the foreshore from erosion and overtopping and is positive for public safety. The difficulty of construction and lack of adaptability cause this option to rank poorly.

Benefit Score (Rank): 3.96 (6/8)

Option 3: [2] + Reclamation to provide for 2.4m path landward of crest + allowance to adapt (raise) crest by 0.35m. This option would have a positive impact on public access and safety while also providing protection to the foreshore from erosion and overtopping. The option is seen to improve the visual appearance of the precinct. The adaptability of the works were also seen as a positive. However, this option would be difficult to build.

Benefit Score (Rank): 23.93 (2/8)

Option 4: [2] + Provision for robust pathway around front of revetment. Option 4 scored similarly to option 3, however it was seen to have a greater positive impact on residential security and aesthetics. This option would pose significant construction challenges.

Benefit Score (Rank): 24.27 (1/8)

Option 5: Extend groynes “D” and (”B”) and nourish beach between these two groynes. Creating a sandy beach approximately 120m long in front of the revetment increases protection from erosion and overtopping, improves the aesthetics of the area and would positively impact overall coastal processes. This option does not improve public access and safety and will also be difficult to construct.

Benefit Score (Rank): 0.51 (7/8)

Option 6: Extend groynes “D” and (”B”), construct enclosing revetment with reclamation of enclosed area. This option achieves positive outcomes for public access and safety and foreshore protection from erosion and overtopping. The difficulty of construction is problematic for this option.

Benefit Score (Rank): 17.97 (3/8)

Option 7: “Mega” nourishment, in vicinity of groynes “C”, “D” and offshore. With monitoring and possible adoption of a structural solution in future. Providing a sandy beach in front of the precinct provides protection from erosion and overtopping and also has a positive impact on coastal processes. The sandy beach is also aesthetically pleasing and positive for recreation. Construction would be difficult. Encouraging public use of the foreshore may cause issues for residential security.

Benefit Score (Rank): 15.19 (4/8)

Option 8: [2] + Extend groyne D. Option 8 had similar positives to Option 7, however the additional works on the groyne increases public safety (protection from overtopping) and accessibility (widening of beach). Due to construction difficulty, this option scored lower than Option 7.

Benefit Score (Rank): 7.53 (5/8)

Precinct 6 Options

Option 1: Do Nothing. Option 1 scored the lowest for Precinct 6. The only positive for this option was that the existing sandy beach is seen as positive. Taking no action in this precinct will not improve foreshore protection nor address the issues associated with public safety.

Benefit Score (Rank): -4.61 (8/8)

Option 2: Remove boat ramps and replace with low wall, adaptable if required in future. Replacing boat ramps with a low wall will protect the backshore from inundation therefore improving public safety. A uniform, properly engineered structure would improve protection from erosion and overtopping. This option is also seen to improve the aesthetics of Precinct 6.

Benefit Score (Rank): 16.70 (2/8)

Option 3: [2] except build wave deflector wall along edge of pathway. Option 3 was deemed to have the same positive impacts as Option 2 although it was not as visually appealing as 2.

Benefit Score (Rank): 16.02 (3/8)

Option 4: Extend groyne "D" and provide ongoing nourishment. By extending the groyne and providing ongoing nourishment a wider, more consistent sandy beach would be provided

Benefit Score (Rank): 23.92 (1/8)

Option 5: Formalise stormwater by raising bed to elevation of outlet and providing an infiltration trench. This is the second best ranked of the stormwater outlet options. It will reduce the amount of sand scoured from the beach during runoff events and improve aesthetics of the area. It is also relatively easy to construct.

Benefit Score (Rank): 11.69 (5/8)

Option 6: Replace stormwater channel with GPT and infiltration trench on present alignment. This is similar to Option 5, with added benefits relating to the prevention of litter entering the waterway and expected slightly less scouring of sand from the beach. It will pose some difficulties for construction.

Benefit Score (Rank):6.31 (6/8)

Option 7: Carry stormwater pipe across beach on low groyne with crossing for pedestrians. This is the least favoured of the stormwater management options, primarily because of its impact on aesthetics and the visual and pedestrian barrier it would create across the western end of Bagnalls Beach.

Benefit Score (Rank):-0.98 (7/8)

Option 8: Disconnect the eastern stormwater line. Considering that the stormwater line acts primarily as a relief outlet during storms, this line could potentially be disconnected. This would somewhat remove the discontinuity in the beach, and improve aesthetics, concentrating flow through the main stormwater line which exist through Groyne D. This would require additional modelling and consideration of the capacity of the adjacent stormwater system, and some provision for overland flow would need to be made here to account for this sitting in a topographic sag point.

Benefit Score (Rank):13.48 (4/8)

Appendix F Tabulated Design Parameters

Summary of Structural Design Conditiosn

	Design Wave Conditions Per Precinct (1)								
	50yr			100yr			200yr		
Precinct	Hs	Hmax	Tp	Hs	Hmax	Tp	Hs	Hmax	Tp
1&2	1.18	1.57	12.1	1.23	1.63	12.3	1.29	1.70	12.5
3	1.18	1.66	12.1	1.29	1.72	12.3	1.34	1.82	12.5
4,5 & 6	1.22	1.73	12.1	1.32	1.78	12.3	1.38	1.86	12.5

(1) Waves offshore of site estimated using Delft 3d, Brought to Foreshore using Goda (2000) relationships.

Armour Sizing - Hudsons Equation (as presented in CIRIA, 2007)															
						Zero Damage Condition – 50 yr Event					20% Damage Condition 200 yr Event				
Precinct	Rock Density (2)	Water Density	Kd	Slope (1 in X)	Relative Buoyancy	Sd	Hs (m)	Dn50 (m)	M50 (kg)	Ds50 (m)	Sd	Hs	Dn50	M50	Ds50
1&2	2560	1025	4	1.5	1.497561	2	1.18	0.56	447	0.69	14	1.29	0.45	240	0.56
3	2560	1025	4	1.5	1.497561	2	1.18	0.56	441	0.69	14	1.34	0.47	271	0.59
4,5 & 6	2560	1025	4	2	1.497561	2	1.22	0.52	366	0.65	14	1.38	0.44	221	0.55

(2) Based on Rock Density provided by Boral Quarries at Seaham

Overtopping – Average Discharge (Eurotop Manual, 2007)										
Precinct	Offshore Wave, 50 yrs (m)	Degrees from Normal Approach	Still Water Level (50 yrs + SLR)	Runup Crest Level (m AHD)	Rc (m)	gammaf	gammab	RHS	LHS Denom	q (l/s/m)
1&2	1.18	0	1.88	2.3	0.42	0.4	1	0.03	4.02	0.10
3	1.18	0	1.88	2.3	0.42	0.4	1	0.03	3.99	0.10
4,5 & 6	1.22	0	1.88	2.35	0.47	0.4	1	0.02	4.20	0.09

Overtopping Maximum Volume for Single Wave (Eurotop Manual, 2007)													
Precinct	Crest Level	Ac	Dn	Pov	Storm Duration Peak (s)	Tz	Nw	Now	a	Allowable Volume (l/m)	P (allowable)	No. Waves Exceeding	Vmax
1&2	2.65	0.77	0.56	0.14	14400	6.9	2088	291	4.33	50	0.9981	0.6	43.8
3	2.65	0.77	0.56	0.14	14400	6.9	2088	288	4.30	50	0.9981	0.5	43.4
4,5 & 6	2.65	0.77	0.56	0.16	14400	6.9	2088	402	2.74	50	0.9999	0.1	29.9

Scour Depth (CIRIA, 2007)									
Precinct	Structure Slope	Design Wave Height (200yr Hs)	Design Wave Height (200yr Hmax)	Tm	Fictitious Offshore Steepness	Surf Similarity (based on mean wave)	Breaker Type	Reflection Coefficient	Scour Depth
Precincts 1&2	0.67	1.29	1.70	7.13	0.02	5.24	Surging	0.48	0.82
Precinct 3	0.67	1.34	1.82	7.13	0.02	5.13	Surging	0.48	0.87
Precincts 4 to 6	0.50	1.38	1.86	7.13	0.02	3.79	Surging	0.40	0.74

Appendix G 3d Visualisation of Management Options

PORT STEPHENS COUNCIL
Whitehead & Associates Environmental Consultants
NSW GOVERNMENT Office of Environment & Heritage
LIGHTBOX

Sandy Point / Conroy Park Foreshore Erosion and Drainage Management Plan
Precinct 2: Existing condition

Port Stephens Council
Whitehead & Associates Environmental Consultants
NSW Government Office of Environment & Heritage
Lightbox Landscape Architecture & 3D Visualisation

Sandy Point / Conroy Park Foreshore Erosion and Drainage Management Plan

Precinct 2: Scheme 1

PORT STEPHENS COUNCIL

Whitehead & Associates Environmental Consultants

NSW GOVERNMENT Office of Environment & Heritage

LIGHTBOX LANDSCAPE ARCHITECTURE & 3D VISUALISATION

Sandy Point / Conroy Park Foreshore Erosion and Drainage Management Plan

Precinct 2: Scheme 2

Whitehead & Associates
Environmental Consultants

Sandy Point / Conroy Park Foreshore Erosion and Drainage Management Plan
Precinct 2: Scheme 3

Whitehead & Associates
Environmental Consultants

Sandy Point / Conroy Park Foreshore Erosion and Drainage Management Plan
Precinct 5:Existing Condition

Whitehead & Associates Environmental Consultants

Office of Environment & Heritage

Sandy Point / Conroy Park Foreshore Erosion and Drainage Management Plan

Precinct 5: Scheme 1

Port Stephens Council
Whitehead & Associates Environmental Consultants
NSW Government Office of Environment & Heritage
Lightbox Landscape Architecture & 3D Visualisation

Sandy Point / Conroy Park Foreshore Erosion and Drainage Management Plan
Precinct 5: Scheme 2

Whitehead & Associates Environmental Consultants

Office of Environment & Heritage

Sandy Point / Conroy Park Foreshore Erosion and Drainage Management Plan

Precinct 5: Scheme 3

Appendix H Cost Estimates

Summary of Cost Estimates and Application of Contingencies and Inflation: Scheme 1 (Exclusive of GST)

Description	Base Cost	Contingency	Inflation[2]	Total	Adopt	Annual Maintenance (Structural)	Annual Maintenance (Nourishment)
Precinct 1: Relocate Sand[1]	$ 67,840.00	$ 13,568.00	$ 3,392.00	$ 84,800.00	85K		$ 8,480.00
Precinct 2: Construct Groyne (Western end, Conroy Park)	$ 407,658.29	$ 81,531.66	$ 20,382.91	$ 509,572.86	0.51M	$ 509.57	
Precinct 3: Demolish and Reconstruct Revetment, Make Safe	$ 849,873.87	$ 169,974.77	$ 42,493.69	$ 1,062,342.34	1.1M	$ 1,062.34	
Precinct 4: Rebuild Foreshore Revetment (Some Reclamation)	$ 341,156.76	$ 68,231.35	$ 17,057.84	$ 426,445.95	0.43M	$ 426.45	
Precinct 5: Demolish and Rebuild, including Reclamation	$ 1,026,481.17	$ 205,296.23	$ 51,324.06	$ 1,283,101.47	1.3M	$ 1,283.10	
Precinct 6: Demolish and Rebuild, (Minor Reclamation)	$ 645,480.97	$ 129,096.19	$ 32,274.05	$ 806,851.21	0.81M	$ 806.85	

[1]The relocated sand is used to nourish Precincts 2 and 3. The total cost for this operation is included under costs for Precinct 1

[2]Base Cost relates to estimates relevatn to the end of 2014. A 5% inflation rate has been applied in accordance with Rawlinson's Quarterly update to their *Australian Construction Handbook* from July, 2015. That places the resulting

Cost Estimate: Scheme 1, Precinct 1

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, insall sediment curtain/environmental controls.	1	Item	2000	2000	
1.2	Remove and clean up at end of work	1	Item	2000	2000	
Subtotal						$ 4,000.00
2	**Sand Excavation and Placement (7 day Operation)**					
2.1	Scraper Hire (2 of)	112	hours	300	33600	
2.2	Dozer Hire (2 of)	112	hours	270	30240	
Subtotal						$ 63,840.00
Total						**$ 67,840.00**

Cost Estimate: Scheme 1, Precinct 2

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$ 10,000.00
2	**Groyne Construction**					
2.1	Acquire and Place Core (Above MSL)	445	T	50	22250	
2.2	Acquire and Place Secondary Armour (Above MSL)	180	T	75	13500	
2.3	Acquire and Place Primary Armour (Above MSL)	1123	T	140	157191	
2.4	Acquire and Place Core (Below MSL)	970	T	70	67900	
2.5	Acquire and Place Secondary Armour (Below MSL)	500	T	85	42500	
2.6	Acquire and Place Primary Armour (Below MSL)	502	T	180	90317	
2.7	Construct Path (40m Long)	40	m	100	4000	
Subtotal						$ 397,658.29
Total						**$ 407,658.29**

Cost Estimate: Scheme 1, Precinct 3

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Demolition**					
	(Costs of Demolition Assumed Offset by gains from not having to acquire Secondary Armour)					
Subtotal						$0.00
3	**Excavation**					
3.1	Batter Back and Prepare Slope.	5200	cu.m	5	26000	
Subtotal						$26,000.00
4	**Revetment Construction**					
4.1	Geotextile	2600	sq.m	10	26000	
4.2	Place Secondary Armour (Above MSL)	1360	T	65	88407	
4.3	Acquire and Place Primary Armour (Above MSL)	3406	T	100	340594	
4.4	Place Secondary Armour (Below MSL)	1251	T	70	87543	
4.5	Acquire and Place Primary Armour (Below MSL)	1903	T	120	228331	
4.6	Construct Path	200	m	100	20000	
4.7	Construct Fence	200	m	115	23000	
Subtotal						$ 813,873.87
Total						**$ 849,873.87**

Cost Estimate: Scheme 1, Precinct 4

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Demolition**					
	(Costs of Demolition Assumed Offset by gains from not having to acquire Secondary Armour)					
Subtotal						$0.00
3	**Excavation**					
	Batter Back and Prepare Slope.	1800	cu.m	5	9000	
Subtotal						$9,000.00
4	**Revetment Construction**					
4.1	Geotextile	900	sq.m	10	9000	
4.2	Place Secondary Armour (Above MSL)	413	T	65	26866	
4.3	Acquire and Place Primary Armour (Above MSL)	1105	T	100	110452	
4.4	Place Secondary Armour (Below MSL)	563	T	70	39394	
4.5	Acquire and Place Primary Armour (Below MSL)	1062	T	120	127444	
4.6	Construct Path	90	m	100	9000	
Subtotal						$ 322,156.76
Total						**$ 341,156.76**

Cost Estimate: Scheme 1, Precinct 5

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Demolition, Stockpiling and Disposal**					
2.1	Demolish Existing Structures and Stockpile Reuseables	450	cu.m	20	9000	
2.2	Dispose of Non-reuseable materials to Landfill	596	T	285	169931	
Subtotal						$178,931.25
3	**Revetment Construction**					
3.1	Core (Above MSL)	2494	T	60	149625	
3.2	Place Secondary Armour (Above MSL)	689	T	47.5	32722	
3.3	Acquire and Place Primary Armour (Above MSL)	1310	T	140	183378	
3.4	Place Core (Below MSL)	1247	T	70	87281	
3.5	Place Secondary Armour (Below MSL)	938	T	57.5	53932	
3.6	Acquire and Place Primary Armour (Below MSL)	1770	T	180	318611	
3.7	Construct Path	120	m	100	12000	
Subtotal						$ 837,549.92
Total						**$ 1,026,481.17**

Cost Estimate: Scheme 1, Precinct 6

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Demolition, Stockpiling and Disposal**					
2.1	Demolish Existing Structures and Stockpile Reuseables	405	cu.m	10	4050	
2.2	Dispose of Non-reuseable materials to Landfill	537	T	285	152938	
Subtotal						$156,988.13
3	**Excavation**					
	Batter Back and Prepare Slope.	2160	cu.m	5	10800	
Subtotal						$10,800.00
4	**Revetment Construction**					
4.1	Geotextile	1080	sq.m	10	10800	
4.2	Place Secondary Armour (Above MSL)	532	T	37.5	19965	
4.3	Acquire and Place Primary Armour (Above MSL)	1182	T	100	118239	
4.4	Place Secondary Armour (Below MSL)	824	T	42.5	35005	
4.5	Acquire and Place Primary Armour (Below MSL)	2214	T	120	265684	
4.6	Construct Path	180	m	100	18000	
Subtotal						$467,692.85
Total						**$ 645,480.97**

Summary of Cost Estimates and Application of Contingencies and Inflation: Scheme 2 (Exclusive of GST)

Description	Base Cost	Contingency	Inflation[1]	Total	Adopt	Annual Maintenance (Structural)	Annual Maintenance (Nourishment/Sand Clearing)
Precinct 1: Install Twin Gross Pollutant Traps. Maintenance clearing of sand from outlets 4 and 5 required	$ 304,000.00	$ 60,800.00	$ 15,200.00	$ 380,000.00	0.38M	$ 6,000.00	$ 5,000.00
Precinct 2: Nourish with imported sand	$ 210,000.00	$ 42,000.00	$ 10,500.00	$ 262,500.00	0.26M		$ 21,000.00
Precinct 3: Demolish and reconstruct revetment, Make safe. Reconstruct and extend groyne A. Nourish with imported Sand	$ 1,296,561.15	$ 259,312.23	$ 64,828.06	$ 1,620,701.43	1.65M	$ 1,545.70	$ 7,500.00
Precinct 4: Rebuild foreshore revetment (Some reclamation). Reconstruct and extend groyne B	$ 727,844.04	$ 145,568.81	$ 36,392.20	$ 909,805.04	0.91M	$ 909.81	
Precinct 5: Demolish upper part of Revetment (down to 0.5m AHD). Reconstruct upper part of revetment to engineering standard.	$ 1,782,450.27	$ 356,490.05	$ 89,122.51	$ 2,228,062.83	2.23M	$ 2,153.06	$ 7,500.00
Precinct 6: Demolish and remove ramps. Stormwater outlet channel filled and shallow dish drain with infiltration trench provided. Reconstruct line of stone between back of beach and foreshore reserve. 'Mega' nourishment of beach.	$ 676,384.13	$ 135,276.83	$ 33,819.21	$ 845,480.16	0.85M	$ 6,595.48	$ 25,000.00

[1]Base Cost relates to estimates relevant to the end of 2014. A 5% inflation rate has been applied in accordance with Rawlinson's Quarterly update to their *Australian Construction Handbook* from July, 2015. That places the resulting estimates as current at the end of 2015.

Cost Estimate: Scheme 2, Precinct 1

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, insall sediment curtain/environmental controls.	1	Item	2000	2000	
1.2	Remove and clean up at end of work	1	Item	2000	2000	
Subtotal						$ 4,000.00
2	**Stormwater**					
2.1	Gross Pollutant Traps	2	Item	150000	300000	
Subtotal						$ 300,000.00
Total						**$ 304,000.00**

Cost Estimate: Scheme 2, Precinct 2

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
1.3	Mobilise / Demobilise Dredging Plant	1	Item	60000	60000	
Subtotal						$ 70,000.00
2	**Nourishment**					
2.1	Dredge and Pump Ashore (Cutter Suction)	14000	cu.m	6.5	91000	
2.2	Spread to Design Profile	14000	cu.m	3.5	49000	
Subtotal						$ 140,000.00
Total						**$ 210,000.00**

Alternative 2: Import from Local Quarry

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$ 10,000.00
2	**Nourishment**					
2.1	Sand Delivery to Site	14000	cu.m	40.25	563500	
2.2	Spread to Design Profile	14000	cu.m	3.5	49000	
Subtotal						$ 612,500.00
Total						**$ 622,500.00**

Based on these figures, Trucking Sand from a local quarry is much more expensive than Dredge Nourishment, which is substantially more expensive than moving sand from next to The Anchorage

Cost Estimate: Scheme 2, Precinct 3

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Demolition**					
	(Costs of Demolition Assumed Offset by gains from not having to acquire Secondary Armour for revetment)					
Subtotal						$0.00
3	**Excavation**					
	Batter Back and Prepare Slope.	5200	cu.m	5	26000	
Subtotal						$26,000.00
4	**Revetment Construction**					
4.1	Geotextile	2600	sq.m	10	26000	
4.2	Place Secondary Armour (Above MSL)	1360	T	65	88407	
4.3	Acquire and Place Primary Armour (Above MSL)	3406	T	100	340594	
4.4	Place Secondary Armour (Below MSL)	1251	T	70	87543	
4.5	Acquire and Place Primary Armour (Below MSL)	1903	T	120	228331	
4.6	Construct Path	200	m	100	20000	
4.7	Construct Fence	200	m	115	23000	
Subtotal						$813,873.87
5	**Bolster Groyne 'A'**					
5.1	Acquire and Place Core (Above MSL)	222	T	60	13338	
5.2	Acquire and Place Secondary Armour (Above MSL)	222	T	75	16678	
5.3	Acquire and Place Primary Armour (Above MSL)	1336	T	140	186995	
5.4	Acquire and Place Core (Below MSL)	485	T	70	33915	
5.5	Acquire and Place Secondary Armour (Below MSL)	535	T	85	45444	
5.6	Acquire and Place Primary Armour (Below MSL)	502	T	180	90317	
Subtotal						$386,687.28
6	**Nourishment**					
6.1	Dredge and Pump Ashore	6000	cu.m	6.5	39000	
6.2	Spread to Design Profile	6000	cu.m	3.5	21000	
	(Note: Establishment Disestablishment Costs are included in Precinct 2)					
Subtotal						$60,000.00
Total						**$ 1,296,561.15**

Cost Estimate: Scheme 2, Precinct 4

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Demolition**					
	(Costs of Demolition Assumed Offset by gains from not having to acquire Secondary Armour)					
Subtotal						$0.00
3	**Excavation**					
	Batter Back and Prepare Slope.	1800	cu.m	5	9000	
Subtotal						$9,000.00
4	**Revetment Construction**					
4.1	Geotextile	900	sq.m	10	9000	
4.2	Place Secondary Armour (Above MSL)	413	T	65	26866	
4.3	Acquire and Place Primary Armour (Above MSL)	1105	T	100	110452	
4.4	Place Secondary Armour (Below MSL)	563	T	70	39394	
4.5	Acquire and Place Primary Armour (Below MSL)	1062	T	120	127444	
4.6	Construct Path	90	m	100	9000	
Subtotal						$322,157
5	**Bolster Groyne 'B'**					
5.1	Acquire and Place Core (Above MSL)	222	T	60	13338	
5.2	Acquire and Place Secondary Armour (Above MSL)	222	T	75	16678	
5.3	Acquire and Place Primary Armour (Above MSL)	1336	T	140	186995	
5.4	Acquire and Place Core (Below MSL)	485	T	70	33915	
5.5	Acquire and Place Secondary Armour (Below MSL)	535	T	85	45444	
5.6	Acquire and Place Primary Armour (Below MSL)	502	T	180	90317	
Subtotal						$386,687.28
Total						**$ 727,844.04**

Cost Estimate: Scheme 2, Precinct 5

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	10000	10000	
1.2	Remove and clean up at end of work	1	Item	10000	10000	
Subtotal						$20,000.00
2	**Demolition, Stockpiling and Disposal**					
2.1	Demolish Existing Structures and Stockpile Reuseables	450	cu.m	20	9000	
2.2	Dispose of Non-reuseable materials to Landfill	447	T	285	127448	
Subtotal						$136,448.44
3	**Revetment Construction**					
3.1	Geotextile	1500	sq.m	10	15000	
3.2	Place Secondary Armour (Above MSL)	411	T	47.5	19513	
3.3	Acquire and Place Primary Armour (Above MSL)	1328	T	140	185856	
3.4	Construct Path	120	m	100	12000	
Subtotal						$232,369.15
4	**Reconstruct and Extend Groyne D**					
4.1	Acquire and Place Core (Above MSL)	595	T	60	35716	
4.2	Acquire and Place Secondary Armour (Above MSL)	254	T	75	19061	
4.3	Acquire and Place Primary Armour (Above MSL)	1526	T	140	213709	
4.4	Acquire and Place Core (Below MSL)	315	T	70	22045	
4.5	Acquire and Place Secondary Armour (Below MSL)	611	T	85	51936	
4.6	Acquire and Place Primary Armour (Below MSL)	573	T	180	103219	
Subtotal						$445,685.84
5	**Complete Rebuild of Groyne C**					
5.1	Acquire and Place Core (Above MSL)	902	T	80	72124	
5.2	Acquire and Place Secondary Armour (Above MSL)	318	T	95	30180	
5.3	Acquire and Place Primary Armour (Above MSL)	1908	T	220	419785	
5.4	Acquire and Place Core (Below MSL)	630	T	100	62985	
5.5	Acquire and Place Secondary Armour (Below MSL)	764	T	115	87833	
5.6	Acquire and Place Primary Armour (Below MSL)	717	T	300	215040	
Subtotal						$887,946.83
6	**Nourish**					
6.1	Dredge and Pump Ashore	12500	cu.m	6.5	39000	
6.2	Spread to Design Profile	12500	cu.m	3.5	21000	
	(Note: Establishment Disestablishment Costs are included in Precinct 2)					
Subtotal						$60,000.00
Total						**$1,782,450.27**

Cost Estimate: Scheme 2, Precinct 6

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Demolition, Stockpiling and Disposal**					
2.1	Demolish Existing Structures and Replace Reuseable material	405	cu.m	10	4050	
2.2	Dispose of Non-reuseable materials to Landfill	537	T	285	152938	
Subtotal						$156,988.13
3	**Nourishment**					
3.1	Dredge and Pump Ashore	20000	cu.m	6.5	130000	
3.2	Reshape	20000	cu.m	3.5	70000	
Subtotal						200000
4	**Construction of Drain with Infiltration Trench**					
4.1	Grade and place Geotextile	171	sq.m	10	1710	
4.2	Place Rock Fill	106.02	T	50	5301	
4.3	Lay Turf and maintain for nominal period	159	sq.m	15	2385	
Subtotal						$ 9,396.00
5	**Gross Pollutant Traps**					
5.1	Install two Gross Pollutant Traps upstream of Groyne D	2	Items	150000	300000	
Subtotal						$ 300,000.00
Total						**$ 676,384.13**

Summary of Cost Estimates and Application of Contingencies and Inflation: Scheme 3 (Exclusive of GST)

Description	Base Cost	Contingency	Inflation[1]	Total	Adopt	Annual Maintenance (Structural)	Annual Maintenance (Nourishment/Sand Clearing)
Precinct 1: Retain Sand. Construct groyne across beach to convey stormwater line. Install Twin Gross Pollutant Traps in foreshore reserve. Maintenance clearing of sand from outlets 4 and 5 required.	$ 1,044,772.32	$ 208,954.46	$ 52,238.62	$ 1,305,965.40	1.3M	$ 1,305.97	$ 5,000.00
Precinct 2: Nourish with imported sand	$ 210,000.00	$ 42,000.00	$ 10,500.00	$ 262,500.00	0.26M		$ 21,000.00
Precinct 3: Demolish and reconstruct revetment, Make safe. Reconstruct and extend Groyne A. Enhance existing "Headlands"	$ 2,164,441.70	$ 432,888.34	$ 108,222.09	$ 2,705,552.13	2.7M	$ 2,630.55	$ 7,500.00
Precinct 4: Rebuild foreshore revetment (Some reclamation). Extend groyne B and nourish between groynes A and B	$ 754,844.04	$ 150,968.81	$ 37,742.20	$ 943,555.04	0.94M	$ 909.81	$ 3,375.00
Precinct 5: Demolish and rebuild, including reclamation. All boat ramps demolished. Pedestrian access around front of new revetment.	$ 1,221,154.92	$ 244,230.98	$ 61,057.75	$ 1,526,443.65	1.55M	$ 1,526.44	
Precinct 6: Demolish and rebuild, (minor reclamation). All boat ramps removed. Allow for construction of wave deflector wall in future. Stormwater outlet channel filled and shallow dish drain with infiltration trench provided	$ 654,876.97	$ 130,975.39	$ 32,743.85	$ 818,596.21	0.82M	$ 818.60	

[1]Base Cost relates to estimates relevent to the end of 2014. A 5% inflation rate has been applied in accordance with Rawlinson's Quarterly update to their *Australian Construction Handbook* from July, 2015. That places the resulting estimates as current at the end of 2015.

Cost Estimate: Scheme 3, Precinct 1

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, insall sediment curtain/environmental controls.	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$ 10,000.00
2	**Groyne Construction**					
2.1	Acquire and Place Core (Above MSL)	445	T	60	26676	
2.2	Acquire and Place Secondary Armour (Above MSL)	179	T	75	13446	
2.3	Acquire and Place Primary Armour (Above MSL)	1123	T	140	157191	
2.4	Acquire and Place Core (Below MSL)	969	T	70	67830	
2.5	Acquire and Place Secondary Armour (Below MSL)	497	T	75	37312	
2.6	Acquire and Place Primary Armour (Below MSL)	502	T	180	90317	
Subtotal						$ 392,772.32
3	**Stormwater Extension**					
3.1	Screw Piers	60	Item	900	54000	
3.2	Stormwater Lines	180	m	1600	288000	
3.3	Gross Pollutant Traps	2	Item	150000	300000	
Subtotal						$ 642,000.00
Total						**$ 1,044,772.32**

Cost Estimate: Scheme 3, Precinct 2

Alternative 1: Dredging

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
1.3	Mobilise / Demobilise Dredging Plant	1	Item	60000	60000	
Subtotal						$ 70,000.00
2	**Nourishment**					
2.1	Dredge and Pump Ashore (Cutter Suction)	14000	cu.m	6.5	91000	
2.2	Spread to Design Profile	14000	cu.m	3.5	49000	
Subtotal						$ 140,000.00
Total						**$ 210,000.00**

Alternative 2: Import from Local Quarry

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$ 10,000.00
2	**Nourishment**					
2.1	Sand Delivery to Site	14000	cu.m	40.25	563500	
2.2	Spread to Design Profile	14000	cu.m	3.5	49000	
Subtotal						$ 612,500.00
Total						**$ 622,500.00**

Based on these figures, Trucking Sand from a local quarry is much more expensive than Dredging, which is substantially more expensive than acquiring sand from adjacent to The Anchorage

Cost Estimate: Scheme 3, Precinct 3

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Demolition**					
	(Costs of Demolition Assumed Offset by gains from not having to acquire Secondary Armour for revetment)					
Subtotal						$0.00
3	**Excavation**					
	Batter Back and Prepare Slope.	5200	cu.m	5	26000	
Subtotal						$26,000.00
4	**Revetment Construction**					
4.1	Geotextile	2600	sq.m	10	26000	
4.2	Place Secondary Armour (Above MSL)	1360	T	65	88407	
4.3	Acquire and Place Primary Armour (Above MSL)	3406	T	100	340594	
4.4	Place Secondary Armour (Below MSL)	1251	T	70	87543	
4.5	Acquire and Place Primary Armour (Below MSL)	1903	T	120	228331	
4.6	Construct Path	200	m	100	20000	
4.7	Construct Fence	200	m	115	23000	
Subtotal						$813,873.87
5	**Bolster Groyne 'A'**					
5.1	Acquire and Place Core (Above MSL)	222	T	60	13338	
5.2	Acquire and Place Secondary Armour (Above MSL)	222	T	75	16678	
5.3	Acquire and Place Primary Armour (Above MSL)	1336	T	140	186995	
5.4	Acquire and Place Core (Below MSL)	485	T	70	33915	
5.5	Acquire and Place Secondary Armour (Below MSL)	535	T	85	45444	
5.6	Acquire and Place Primary Armour (Below MSL)	502	T	180	90317	
Subtotal						$386,687.28
5	**Two New Fishtail Headlands**					
5.1	Acquire and Place Core (Above MSL)	889	T	60	53352	
5.2	Acquire and Place Secondary Armour (Above MSL)	445	T	75	33357	
5.3	Acquire and Place Primary Armour (Above MSL)	2671	T	140	373990	
5.4	Acquire and Place Core (Below MSL)	1938	T	70	135660	
5.5	Acquire and Place Secondary Armour (Below MSL)	1069	T	85	90888	
5.6	Acquire and Place Primary Armour (Below MSL)	1004	T	180	180634	
Subtotal						$867,880.56
6	**Nourishment**					
6.1	Dredge and Pump Ashore	6000	cu.m	6.5	39000	
6.2	Spread to Design Profile	6000	cu.m	3.5	21000	
	(Note: Establishment Disestablishment Costs are included in Precinct 2)					
Subtotal						$60,000.00
Total						**$ 2,164,441.70**

Cost Estimate: Scheme 3, Precinct 4

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Demolition**					
	(Costs of Demolition Assumed Offset by gains from not having to acquire Secondary Armour)					
Subtotal						$0.00
3	**Excavation**					
	Batter Back and Prepare Slope.	1800	cu.m	5	9000	
Subtotal						$9,000.00
4	**Revetment Construction**					
4.1	Geotextile	900	sq.m	10	9000	
4.2	Place Secondary Armour (Above MSL)	413	T	65	26866	
4.3	Acquire and Place Primary Armour (Above MSL)	1105	T	100	110452	
4.4	Place Secondary Armour (Below MSL)	563	T	70	39394	
4.5	Acquire and Place Primary Armour (Below MSL)	1062	T	120	127444	
4.6	Construct Path	90	m	100	9000	
Subtotal						$322,157
5	**Bolster Groyne 'B'**					
5.1	Acquire and Place Core (Above MSL)	222	T	60	13338	
5.2	Acquire and Place Secondary Armour (Above MSL)	222	T	75	16678	
5.3	Acquire and Place Primary Armour (Above MSL)	1336	T	140	186995	
5.4	Acquire and Place Core (Below MSL)	485	T	70	33915	
5.5	Acquire and Place Secondary Armour (Below MSL)	535	T	85	45444	
5.6	Acquire and Place Primary Armour (Below MSL)	502	T	180	90317	
Subtotal						$386,687.28
6	**Nourishment**					
	Dredge and Pump Ashore	2700	cu.m	6.5	17550	
	Spread to Design Profile	2700	cu.m	3.5	9450	
	(Note: Establishment Disestablishment Costs are included in Precinct 2)					
Subtotal						$ 27,000.00
Total						**$ 754,844.04**

Cost Estimate: Scheme 3, Precinct 5

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Demolition, Stockpiling and Disposal**					
2.1	Demolish Existing Structures and Stockpile Reuseables	450	cu.m	20	9000	
2.2	Dispose of Non-reuseable materials to Landfill	596	T	285	169931	
Subtotal						$178,931.25
3	**Revetment Construction**					
3.1	Geotextile	1500	sq.m	10	15000	
3.2	Place Secondary Armour (Above MSL)	689	T	47.5	32722	
3.3	Acquire and Place Primary Armour (Above MSL)	1310	T	140	183378	
3.4	Place Core (Below MSL)	0	T	70	0	
3.5	Place Secondary Armour (Below MSL)	938	T	57.5	53932	
3.6	Acquire and Place Primary Armour (Below MSL)	1770	T	180	318611	
Subtotal						$603,643.67
4	**Footbridge**					
4.1	Piers	45	Item	1500	67500	
4.2	Deck, Ballustrade, Rails etc.	408	sq.m	885	361080	
Subtotal						$428,580.00
Total						**$1,221,154.92**

Cost Estimate: Scheme 3, Precinct 6

Item No.	Item Description	Qty	Unit	Rate ($)	Capital Cost	Sub-Total
1	**Site Establishment/Disestablishment**					
1.1	Erect regulatory signs, setup, plant hire, install sediment curtain/environmental controls. Establish Stockpile	1	Item	5000	5000	
1.2	Remove and clean up at end of work	1	Item	5000	5000	
Subtotal						$10,000.00
2	**Demolition, Stockpiling and Disposal**					
2.1	Demolish Existing Structures and Stockpile Reuseables	405	cu.m	10	4050	
2.2	Dispose of Non-reuseable materials to Landfill	537	T	285	152938	
Subtotal						$156,988.13
3	**Excavation**					
	Batter Back and Prepare Slope.	2160	cu.m	5	10800	
Subtotal						$10,800.00
4	**Revetment Construction**					
4.1	Geotextile	1080	sq.m	10	10800	
4.2	Place Secondary Armour (Above MSL)	532	T	37.5	19965	
4.3	Acquire and Place Primary Armour (Above MSL)	1182	T	100	118239	
4.4	Place Secondary Armour (Below MSL)	824	T	42.5	35005	
4.5	Acquire and Place Primary Armour (Below MSL)	2214	T	120	265684	
4.6	Construct Path	180	m	100	18000	
Subtotal						$467,692.85
5	**Construction of Drain with Infiltration Trench**					
5.1	Grade and place Geotextile	171	sq.m	10	1710	
5.2	Place Rock Fill	106.02	T	50	5301	
5.3	Lay Turf and maintain for nominal period	159	sq.m	15	2385	
Subtotal						$9,396.00
Total						**$ 654,876.97**