Appendix A  Coastal Processes Study
Sandy Point/Conroy Park Coastal
Process Study

Prepared for Port Stephens Council

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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 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)

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<td>Extreme</td>
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<tr>
<td>1% AEP</td>
<td>0.4</td>
</tr>
<tr>
<td>2% AEP</td>
<td>0.4</td>
</tr>
<tr>
<td>5% AEP</td>
<td>0.3</td>
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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)

<table>
<thead>
<tr>
<th>AEP</th>
<th>Ocean Water Level (m AHD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>1.43</td>
</tr>
<tr>
<td>2%</td>
<td>1.47</td>
</tr>
<tr>
<td>1%</td>
<td>1.50</td>
</tr>
</tbody>
</table>

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)

<table>
<thead>
<tr>
<th>AEP</th>
<th>Sandy Point</th>
<th>Corlette Head</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>1.58</td>
<td>1.60</td>
</tr>
<tr>
<td>2%</td>
<td>1.62</td>
<td>1.65</td>
</tr>
<tr>
<td>1%</td>
<td>1.67</td>
<td>1.69</td>
</tr>
<tr>
<td>Extreme</td>
<td>1.70</td>
<td>1.72</td>
</tr>
</tbody>
</table>

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)

<table>
<thead>
<tr>
<th>Site</th>
<th>5% AEP (2050)</th>
<th>1% AEP (2050)</th>
<th>Extreme (2050)</th>
<th>5% AEP (2100)</th>
<th>1% AEP (2100)</th>
<th>Extreme (2100)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy Point</td>
<td>2.0</td>
<td>2.1</td>
<td>2.1</td>
<td>2.5</td>
<td>2.6</td>
<td>2.6</td>
</tr>
<tr>
<td>Corlette Point</td>
<td>2.0</td>
<td>2.1</td>
<td>2.1</td>
<td>2.5</td>
<td>2.6</td>
<td>2.6</td>
</tr>
</tbody>
</table>

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.

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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)

<table>
<thead>
<tr>
<th>Site</th>
<th>5% AEP</th>
<th>1% AEP</th>
<th>Extreme</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy Point</td>
<td>2.4</td>
<td>2.3</td>
<td>2.9²</td>
</tr>
<tr>
<td>Corlette Point</td>
<td>2.2</td>
<td>2.3</td>
<td>2.9²</td>
</tr>
</tbody>
</table>

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 110×10⁶ m³.

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

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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° 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

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
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.
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° 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
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 landmarks 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

<table>
<thead>
<tr>
<th>Year</th>
<th>Original Scale/Resolution</th>
<th>Quality</th>
<th>Useable</th>
</tr>
</thead>
<tbody>
<tr>
<td>1951</td>
<td>1:33,000 Approx.</td>
<td>Beach clearly defined, seagrass not clearly defined</td>
<td>No</td>
</tr>
<tr>
<td>1952</td>
<td>1:31,000 Approx.</td>
<td>Beach clearly defined, seagrass not clearly defined</td>
<td>No</td>
</tr>
<tr>
<td>1959</td>
<td>1:16,000 Approx.</td>
<td>Beach clearly defined, seagrass not clearly defined</td>
<td>Yes</td>
</tr>
<tr>
<td>1963</td>
<td>1:34,000 Approx.</td>
<td>Seagrass, light and dense vegetation reasonably defined, structure extents poorly defined</td>
<td>Yes</td>
</tr>
<tr>
<td>1965</td>
<td>1:16,000 Approx.</td>
<td>Seagrass and light vegetation poorly defined, dense vegetation and structure extent reasonably defined</td>
<td>Yes</td>
</tr>
<tr>
<td>1968</td>
<td>1:21,000 Approx.</td>
<td>Seagrass, dense vegetation and structure extent reasonably defined, light vegetation poorly defined</td>
<td>Yes</td>
</tr>
<tr>
<td>1976</td>
<td>1:16,000 Approx.</td>
<td>Seagrass, dense vegetation and structure extent reasonably defined, light vegetation poorly defined</td>
<td>Yes</td>
</tr>
<tr>
<td>1979</td>
<td>1:42,000 Approx.</td>
<td>Seagrass and light and dense vegetation reasonably defined, structure extent poorly defined</td>
<td>Yes</td>
</tr>
<tr>
<td>1986</td>
<td>1:8000 Approx.</td>
<td>High reflection off ocean, seagrass not clearly defined, light and dense vegetation and structure extent well defined</td>
<td>Yes</td>
</tr>
<tr>
<td>1992</td>
<td>1:16000 Approx.</td>
<td>Seagrass, vegetation and structure extents highly defined</td>
<td>Yes</td>
</tr>
<tr>
<td>1996</td>
<td>1:8000 Approx.</td>
<td>Seagrass, vegetation and structure extents highly defined</td>
<td>Yes</td>
</tr>
<tr>
<td>1998</td>
<td>1 pixel: 1m</td>
<td>Seagrass, vegetation and structure extents highly defined</td>
<td>Yes</td>
</tr>
<tr>
<td>Year</td>
<td>Original Scale/Resolution</td>
<td>Quality</td>
<td>Useable</td>
</tr>
<tr>
<td>------</td>
<td>--------------------------</td>
<td>---------</td>
<td>---------</td>
</tr>
<tr>
<td>1999</td>
<td>1:8000 Approx.</td>
<td>Seagrass, vegetation and structure extents highly defined.</td>
<td>Yes</td>
</tr>
<tr>
<td>2003</td>
<td>1 pixel: 1m</td>
<td>Seagrass, vegetation and structure extents highly defined.</td>
<td>Yes</td>
</tr>
<tr>
<td>2005</td>
<td>1:10000 Approx.</td>
<td>Seagrass moderate to highly defined, vegetation and structure extents highly defined.</td>
<td>Yes</td>
</tr>
<tr>
<td>2007</td>
<td>1 pixel: 1m</td>
<td>Seagrass, vegetation and structure extents highly defined.</td>
<td>Yes</td>
</tr>
<tr>
<td>2012</td>
<td>1 pixel: 1m</td>
<td>Seagrass, vegetation and structure extents highly defined.</td>
<td>Yes</td>
</tr>
</tbody>
</table>

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 realignment 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

(Approx Scale)
*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.
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 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

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Approved DW

(Approx Scale)
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.

DEM.s 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](image)

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

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Figure 22: Change in Bed Elevations, 2007 to 2015

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Change in Bed Elevation (m)

-2.0 Erosion
-1.0
-0.5
0.0
0.5
1.0
2.0 Accretion
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

<table>
<thead>
<tr>
<th>Distance East from Breakwater (m)</th>
<th>Change in Area (m⁢³/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>282</td>
</tr>
<tr>
<td>48</td>
<td>203</td>
</tr>
<tr>
<td>80</td>
<td>189</td>
</tr>
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<td>112</td>
<td>158</td>
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<td>144</td>
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</tr>
<tr>
<td>176</td>
<td>70</td>
</tr>
<tr>
<td>208</td>
<td>45</td>
</tr>
<tr>
<td>240</td>
<td>13</td>
</tr>
<tr>
<td>Volume of Accumulation</td>
<td>33,800</td>
</tr>
</tbody>
</table>
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

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(Approx Scale)
Figure 25: Model Grid Near Corlette and Wave Analysis Points

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Legend
- Model Cell Boundaries
- Wave Analysis Points
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.
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

Legend

Modelled Significant Wave Height (m)
- 0.4
- 0.5
- 0.6
- 0.7
- 0.8
- 0.9
- 1.0

Vectors indicate direction of wave approach, Colours indicate modelled significant wave height

Location of Video, ~11am, 21st April, 2015

SANDY POINT

CORLETTE POINT
Figure 29: Modelled Peak "Flood" Currents

Sandy Point / Conroy Park Coastal Processes Study

Legend
Current Speed (m/s)

- 0.10
- 0.20
- 0.30
- 0.40
- 0.50
- 0.60

Vectors indicate direction of tidal current approach, Colours indicate modelled current speed.
Figure 30: Modelled Peak "Ebb" Currents

Sandy Point / Conroy Park Coastal Processes Study

Whitehead & Associates
Environmental Consultants

Legend
Current Speed (m/s)
- 0.10
- 0.20
- 0.30
- 0.40
- 0.50
- 0.60

Vectors indicate direction of tidal current approach, Colours indicate modelled current speed

SANDY POINT
CORLETTE POINT

(Approx Scale)
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)

<table>
<thead>
<tr>
<th>Probability</th>
<th>Sydney ³</th>
<th>Crowdy Head ⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>10% (P=0.1)</td>
<td>2.55</td>
<td>2.48</td>
</tr>
<tr>
<td>1% (P=0.01)</td>
<td>4.19</td>
<td>2.94</td>
</tr>
<tr>
<td>1yr ARI (P≈0.0001)</td>
<td>5.9</td>
<td>5.4</td>
</tr>
<tr>
<td>10yr ARI (P≈0.000001)</td>
<td>7.5</td>
<td>7.0</td>
</tr>
<tr>
<td>100yr ARI (P≈0.0000001)</td>
<td>9.0</td>
<td>8.5</td>
</tr>
</tbody>
</table>

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.

³Directional Record between 1989 and 2009 Analysed
⁴Non Directional Record between 1985 and 2010 Analysed
Table 9  

<table>
<thead>
<tr>
<th>Probability</th>
<th>Hs(m)</th>
<th>Tp(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1yr ARI</td>
<td>5.9</td>
<td>10.7</td>
</tr>
<tr>
<td>10yr ARI</td>
<td>7.5</td>
<td>11.5</td>
</tr>
<tr>
<td>100yr ARI</td>
<td>9.0</td>
<td>12.3</td>
</tr>
</tbody>
</table>

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  

<table>
<thead>
<tr>
<th>Tidal Plane</th>
<th>Level (m AHD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Higher High Water Springs Solstices</td>
<td>0.976</td>
</tr>
<tr>
<td>Mean High Water Springs</td>
<td>0.601</td>
</tr>
<tr>
<td>Mean High Water</td>
<td>0.474</td>
</tr>
<tr>
<td>Mean High Water Neaps</td>
<td>0.348</td>
</tr>
<tr>
<td>Mean Sea Level</td>
<td>-0.038</td>
</tr>
<tr>
<td>Mean Low Water Neaps</td>
<td>-0.423</td>
</tr>
<tr>
<td>Mean Low Water</td>
<td>-0.55</td>
</tr>
<tr>
<td>Mean Low Water Springs</td>
<td>-0.677</td>
</tr>
<tr>
<td>Indian Springs Low Water</td>
<td>-0.945</td>
</tr>
</tbody>
</table>

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 residual⁵ 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)

<table>
<thead>
<tr>
<th>Probability</th>
<th>Fort Denison(^6)</th>
<th>Sydney(^7)</th>
<th>Port Macquarie (Offshore)(^8)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1yr ARI (P~0.0001)</td>
<td>0.36</td>
<td>0.31</td>
<td>0.37</td>
</tr>
<tr>
<td>10yr ARI (P~0.00001)</td>
<td>0.44</td>
<td>0.38</td>
<td>0.48</td>
</tr>
<tr>
<td>100yr ARI (P~0.000001)</td>
<td>0.61</td>
<td>0.47</td>
<td>0.61</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Recurrence Interval (yrs.)</th>
<th>Fort Denison (Sydney Harbour) Water Level (m AHD)</th>
<th>Tomaree (Port Stephens) Water Level (m AHD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1.3</td>
<td>1.28</td>
</tr>
<tr>
<td>20</td>
<td>1.36</td>
<td>1.31</td>
</tr>
<tr>
<td>50</td>
<td>1.40</td>
<td>1.35</td>
</tr>
<tr>
<td>100</td>
<td>1.42</td>
<td>1.37</td>
</tr>
</tbody>
</table>

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)

<table>
<thead>
<tr>
<th>Tidal Plane</th>
<th>Level (m AHD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Higher High Water Springs Solstices</td>
<td>1.08</td>
</tr>
<tr>
<td>Mean High Water Springs</td>
<td>0.69</td>
</tr>
<tr>
<td>Mean High Water</td>
<td>0.588</td>
</tr>
<tr>
<td>Mean High Water Neaps</td>
<td>0.443</td>
</tr>
<tr>
<td>Mean Sea Level</td>
<td>0.009</td>
</tr>
<tr>
<td>Mean Low Water Neaps</td>
<td>-0.431</td>
</tr>
<tr>
<td>Mean Low Water</td>
<td>-0.57</td>
</tr>
<tr>
<td>Mean Low Water Springs</td>
<td>-0.709</td>
</tr>
<tr>
<td>Indian Springs Low Water</td>
<td>-0.995</td>
</tr>
</tbody>
</table>

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>
<thead>
<tr>
<th>ARI</th>
<th>Hs</th>
<th>Tp</th>
<th>Wind Speed (From 130 degrees)</th>
<th>Wind Speed (From 155 degrees)</th>
<th>Wind Speed (From 180 degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.9</td>
<td>10.7</td>
<td>13.0</td>
<td>13.4</td>
<td>15.1</td>
</tr>
<tr>
<td>Intermediate</td>
<td>6.7</td>
<td>11.1</td>
<td>14.5</td>
<td>14.9</td>
<td>15.7</td>
</tr>
<tr>
<td>10</td>
<td>7.5</td>
<td>11.5</td>
<td>16.0</td>
<td>16.3</td>
<td>16.3</td>
</tr>
<tr>
<td>Intermediate</td>
<td>8.25</td>
<td>11.9</td>
<td>18.0</td>
<td>18.6</td>
<td>16.5</td>
</tr>
<tr>
<td>100</td>
<td>9.0</td>
<td>12.3</td>
<td>20.1</td>
<td>20.8</td>
<td>16.7</td>
</tr>
</tbody>
</table>

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 $\xi$ 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**

<table>
<thead>
<tr>
<th>Direction</th>
<th>ARI (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10</td>
</tr>
<tr>
<td>W</td>
<td>18.6</td>
</tr>
<tr>
<td>WNW</td>
<td>21.6</td>
</tr>
<tr>
<td>NW</td>
<td>18.7</td>
</tr>
<tr>
<td>NNW</td>
<td>15.2</td>
</tr>
<tr>
<td>N</td>
<td>12.9</td>
</tr>
<tr>
<td>NNE</td>
<td>11.1</td>
</tr>
<tr>
<td>NE</td>
<td>11.9</td>
</tr>
<tr>
<td>ENE</td>
<td>12.9</td>
</tr>
<tr>
<td>E</td>
<td>14.1</td>
</tr>
</tbody>
</table>

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

Legend
Design Flood Currents Magnitude (m/s)

- 0.10
- 0.20
- 0.30
- 0.40
- 0.50
- 0.60

Vectors indicate direction of tidal current approach, Colours indicate modelled current speed.
Figure 32: Design Ebb Tide Currents

Sandy Point / Conroy Park Coastal Processes Study

Vectors indicate direction of tidal current approach, Colours indicate modelled current speed
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
Figure 33: Conceptual Coastal Processes Model

Sandy Point / Conroy Park Coastal Processes Study

Legend
- Swell Waves
- Wind Generated Waves
- Sediment Movement Direction
- Reef?

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 focused 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 m³/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 10^7 m³/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 0.5-1.0 m/year.
13. Apparent reef (persistent steep bathymetry) in this area concentrates currents around Sandy Point, also contributes to east to west movement of sand.
References


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


