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GOSFORD CITY COUNCIL

Open Coast and Broken Bay Beaches

Coastal Processes and Hazard Definition Study

24 February 2014

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COASTAL PROCESSES AND HAZARD DEFINITION STUDY

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PROJECT 301020-02641 - OPEN COAST AND BROKEN BAY BEACHES

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EXECUTIVE SUMMARY

The Gosford Council Local Government Area is bounded, to the east, by 14 km of coastal beaches extending from Patonga (within Broken Bay) in the south to Forresters Beach on the open coast in the north. Beach erosion and other hazards associated with coastal storms have caused damage to development and other infrastructure in the study area in the past. At Wamberal Beach, homes have been destroyed during severe storms.

Coastline Hazard Lines (representing the predicted extent of erosion for a severe coastal storm) for the study area were last defined in 1994 for open coast beaches and in 1998 for Broken Bay beaches, and adopted as planning controls for development. However, these lines did not take into account the sea level rise planning benchmarks adopted recently by Gosford City Council at its Ordinary Meeting of 20 August 2013, nor did they make any allowance for reduced foundation capacity as required now by the *Guidelines for Preparing Coastal Zone Management Plans* (OEH 2013).

In recognition of these limitations, Council engaged WorleyParsons to review the adequacy of the hazard lines and to develop hazard lines for 2050 and 2100 future planning periods (as well as the immediate planning period). The revised hazard lines were developed using the parameters summarised in the table below. In the majority of cases, an entirely sandy subsurface was assumed. That is, non-engineered designed existing seawalls designed to protect the entire beach, dumped rock and other works that are presently effective (to varying degrees) in limiting storm demand in the study area were ignored for calculation purposes. Likewise, where the natural bedrock or other inerodible subsurface materials (such as stiff clays) are not defined accurately in the area of active erosional coastal processes were not accounted for in the analysis.

Components of 2008, 2050 and 2100 Hazard Lines

Beach	Location	Design Storm Demand (m^3/m above 0m AHD)	Long Term Recession due to Net Sediment Loss (m)		Long Term Recession due to Sea Level Rise (m)	
			2050	2100	2050	2100
Patonga	Entire Beach	40	0	0	3.4	8.4
Pearl	Block 1	120	0	0	4.8	11.8
	Block 2	120				
	Block 3	120				
	Block 4	150				
Umina	Blocks 1 to 7	100	0	0	0	0
Ocean	Block 1	120				
	Block 2	120				
	Block 3	150				
	Block 4	*200*				

Beach	Location	Design Storm Demand (m^3/m above 0m AHD)	Long Term Recession due to Net Sediment Loss (m)		Long Term Recession due to Sea Level Rise (m)	
			2050	2100	2050	2100
Putty	Block 1	280	0	0	8.5	21.0
	Block 2	200				
MacMasters-Copacabana	Block 1	200	0	0	13.3	32.8
	Block 2	240	4.2	9.2		
	Block 3	240	4.2	9.2		
	Block 4	N/A	N/A	N/A		
	Block 5	280	4.2	9.2		
	Block 6	280→100 moving north	0	0		
Avoca	Blocks 1 to 5	100→250 moving north	0	0	17.0	42.0
	Blocks 6 to 9	250				
Terrigal-Wamberal	Blocks 1 to 2 (Terrigal)	60→140 moving north	N/A	N/A	14.6	36.1
	Blocks 4 to 7 (Wamberal)	250	8.8	18.8		
Forresters	Entire Beach	180	0	0	-	-

The existence of existing protection works (generally buried rock revetments and seawall), and areas with potential inerodible subsurfaces, were ignored. That is, an entirely sandy subsurface was assumed. Coastline hazard lines have not been defined for Terrigal Beach due to the presence of the seawall and rock bluff which have been assumed to represent the landward limit of coastal hazards in this area. Similarly, coastline hazard zones have not been determined seaward of the rock bluff on MacMasters-Copacabana Beach.

To improve coastline hazard estimates, Council may consider requesting additional photogrammetric data for recent dates of photography, particularly for Avoca, Wamberal and Forresters beaches. Geotechnical data needs to be obtained for Forresters Beach to allow for the determination of the Zone of Slope Adjustment and the Zone of Reduced Foundation Capacity.

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1. INTRODUCTION

1.1 Background

The Gosford City Council Local Government Area (LGA) is located on the Central Coast of New South Wales, approximately 50km north of Sydney. The LGA is bounded, to the east, by 14km of coastal beaches extending from Patonga (within Broken Bay) in the south to Forresters Beach on the open coast in the north. A locality plan is shown in **Figure 1**.

Historically, coastal processes have threatened sections of the coast within the study area. In particular, Wamberal Beach on the open coast experienced severe erosion in 1974, 1978, 1986 and 1997. In May-June 1974 many houses were threatened and in June 1978 beach and dune erosion, attributed to an intense rip cell, undermined and destroyed two houses. Damage to public assets and recreational amenity has also been experienced at other beaches in the Gosford area.

In recognition of this threat and the impact on the recreational amenity, in June 1984 Council established a Coastal Committee, comprising Council's technical and professional staff and officers of the then NSW Government Public Works Department (PWD) and the Department of Environment and Planning, to consider the coastal hazards of the City's foreshores and to develop management strategies for its coastal regions. The PWD provided Coastal Engineering Advice in respect of coastal erosion at Wamberal Beach and Avoca Beach (PWD 1985). Later, in 1994, Council commissioned PWD to complete a coastal process investigation for all the open coast beaches (PWD 1994), while Patterson Britton & Partners were commissioned to complete a coastal processes study for Broken Bay beaches in 1998 (Patterson Britton & Partners 1998). Coastline hazard lines (representing the predicted extent of erosion for a severe coastal storm) were defined by these studies and adopted as planning controls for development.

The risk to assets along the Gosford LGA coast is projected to increase due to projected sea level rises. In August 2013, Gosford City Council endorsed a number of climate change scenarios relating to the Central Coast region. The climate change scenarios are intended to present a plausible future state of the climate in the region at different time periods and form the basis for risk assessment in this study. The indicative changes described in the scenarios are relative to the current period defined as the average climate experienced over the 1980 - 2007 period and are based on medium to high end of best available projections. Gosford City Council's adopted sea level rise planning benchmarks are 0.40 m by 2050 and 0.9 m by 2100. Due to the higher sea level rise planning benchmarks than those adopted for previous studies, projected shoreline recession on open coast beaches is expected to increase typically by some 7 m for the 50 years projections and, for the 100 years projections, by some 14 m.

Further, the *Guidelines for Preparing Coastal Zone Management Plans* (OEH 2013), have been adopted by the then Minister for Climate Change and the Environment as Guidelines under Section 55D of the *Coastal Protection Act 1979*, and Councils are required to prepare draft plans in accordance with these Guidelines. These Guidelines require that the beach erosion hazard is defined as a storm bite plus an allowance for reduced foundation capacity. Previously, the erosion hazard incorporated storm bite plus an allowance for slope adjustment only, so the updated Guidelines require a more conservative definition of the erosion hazard.

Council's existing development controls are based on hazard lines that do not take into account the current Gosford Council Sea Level Rise Planning Benchmarks. The existing hazard lines are based on the mid-range projections in IPCC (1990). As such, Gosford City Council engaged WorleyParsons to develop hazard lines for the immediate, 2050 and 2100 future planning periods. This review is documented herein.

Figure 1: View of the Gosford LGA Coastline

1.2 Coastline Management Process

1.2.1 Basic Framework

Since late 2009, there have been a number of legislative changes in NSW and development of guidelines that affect how coastline hazards are managed, as outlined at DECCW (2011).

As noted at DECCW (2011), the basic framework for managing coastline hazards in NSW is through the *NSW Coastal Policy* and *Coastal Protection Act 1979*. This is implemented through local Councils (with financial and technical support from the NSW Government) undertaking coastline hazard studies and developing coastal zone management plans which are used to inform land-use planning, development controls and coastal activities. These plans and related planning schemes should contain a range of suitable management strategies indicating how coastal erosion will be dealt with in the particular LGA (or particular area within the LGA), and how individual landowners of properties at risk can and should respond.

Some of the key documents released and legislative amendments made in NSW in recent years include the following:

- *Coastal Risk Management Guide: Incorporating sea level rise benchmarks in coastal risk assessments* (DECCW 2010a), *Flood Risk Management Guide: Incorporating sea level rise benchmarks in flood risk assessments* (DECCW 2010b) and *NSW Coastal Planning Guideline: Adapting to Sea Level Rise* (Department of Planning 2010) in August 2010, *Guidelines for Preparing Coastal Zone Management Plans* (OEH 2013);
- *Coastal Protection and Other Legislation Amendment Act 2010* being passed by NSW Parliament in October 2010, and largely commencing on 1 January 2011[1];
- *Coastal Protection Regulation 2011* commencing on 3 March 2011, which includes additional requirements that support amendments to the *Coastal Protection Act 1979*, and,
- *Coastal Protection Amendment Act 2012* which commenced on 21 January 2013 and modified the requirements for landowners to place temporary works on their properties.

1.2.2 Coastal Zone Management Plans

Gosford City Council has been directed by the then NSW Minister for Climate Change and the Environment to complete a Coastal Zone Management Plan (CZMP) for the beaches in the Gosford LGA. The document herein will inform the subsequent CZMP.

Based on Section 55C of the *Coastal Protection Act 1979*, a CZMP must make provision for (amongst other matters):

1. protecting and preserving beach environments and beach amenity;
2. emergency actions carried out during periods of beach erosion, including the carrying out of related works, such as works for the protection of property affected or likely to be affected by beach erosion, where beach erosion occurs through storm activity or an extreme or irregular event;

[1] This Act was repealed with effect on 26 February 2011 as the Acts it amended had commenced.

3. ensuring continuing and undiminished public access to beaches, headlands and waterways, particularly where public access is threatened or affected by accretion;
4. where the plan relates to a part of the coastline, the management of risks arising from coastal hazards; and,
5. where the plan proposes the construction of coastal protection works (other than temporary coastal protection works[2]) that are to be funded by the Council or a private landowner or both, the proposed arrangements for the adequate maintenance of the works and for managing associated impacts of such works (such as changed or increased beach erosion elsewhere or a restriction of public access to beaches or headlands).

Item 2 above is of particular relevance to the investigation reported herein. Item 4 (and others) are of equal importance for Coastal Zone management planning and this Coastal Process Hazard Definition Study provides the baseline risk assessment investigation to enable Council to meet the provisions as identified within Section 55C of the *Coastal Protection Act 1979*.

As noted in DECCW (2010c), Coastal Management Principles have been developed to inform strategic considerations in coastal management, including the preparation of CZMPs. Two of the principles most relevant to the investigation reported herein are as follows:

- the priority for public expenditure is public benefit; public expenditure should cost-effectively achieve the best practical long-term outcomes; and,
- adopt a risk management approach to managing risks to public safety and assets; adopt a risk management hierarchy involving avoiding risks where feasible and mitigation where risks cannot be reasonably avoided; adopt interim actions to manage high risks while long-term options are implemented.

1.3 Scope of this Report

This report summarises the current knowledge, to provide an understanding of the coastal processes that operate with the study area. The report examines the coastal hazards that impact the coastline between Patonga and Forresters Beach and assesses these hazards to determine the immediate, 2050 and 2100 hazard lines.

The hazards examined herein include:

- beach erosion;
- shoreline recession;

[2] "Temporary coastal protection works" has a specific meaning in relation to the *Coastal Protection Act 1979*, generally being sand or sandbags temporarily placed on a beach to reduce beach erosion impacts. To distinguish this specific meaning from the general meaning of emergency coastal protection works in coastal engineering practice (being any works implemented to limit coastal erosion in an emergency), the specific meaning is denoted as "Part 4c sand/sandbags ECPW" herein in reference to the Section in the *Coastal Protection Act 1979* in which they are described.

- sand drift;
- coastal inundation;
- stormwater erosion;
- slope instability; and
- climate change.

Information included in each report section is listed below:

Section 2 outlines the geographical and historical setting of the beaches in the study area;

Section 3 contains a review of previous coastal studies related to the study area;

Section 4 outlines the data used in the preparation of this report;

Section 5 examines the coastal processes operating in the study area;

Section 6 discusses the coastline hazards affecting the study area, quantifying these hazards where possible;

Section 7 defines the coastline hazards zones; and

Section 8 provides recommendations for future work; and

Section 9 provides a summary of the findings of the report.

Note that all levels given in this report are in metres to Australian Height Datum (AHD), unless stated otherwise. Zero metres AHD is approximate to mean sea level.

2. STUDY AREA

2.1 Site Description

The study area is bounded by 14km of coastal beaches extending from Patonga (within Broken Bay) in the south to Forresters Beach on the open coast in the north. The study area is shown in **Figure 1**. The study area extends in both the seaward and landward directions from the shoreline to the limit of the active coastal processes operating at present, and in the future over a planning period of up to 100 years. Each beach within the study area is described briefly below (extracts from Short 2007). Beaches are listed from south to north. A selection of general ground level photographs of the study area taken as part of the investigation reported herein is provided in **Appendix A**.

Patonga Beach *– occupies part of the 1km wide mouth of Patonga Creek at Brisk Bay. It forms a curving south to southeast-facing beach backed by a low 200-400m wide sandy barrier then the creek and mangroves of Woody Glen Swamp. The beach is 1.4km long and looks out across Broken Bay, receiving only low swell and local wind waves at the shore. These maintain a relatively steep, narrow high tide beach, fronted by deeper water in the centre. Tidal sand shoals front a smaller creek that crosses the eastern end of the beach; and there are larger shoals at the western entrance to Patonga Creek. Only during very large outside swell do ocean waves reach the beach.*

Pearl Beach *– curves for 1.1km to the south where it faces north in behind Green Point. The beach is backed by a low dune area locked in by high valley sides. The beach faces the east, however all waves must pass through Broken Bay entrance and travel 3km into the bay to reach the beach. This results in waves averaging 0.5m at the northern end, dropping in height to the southern corner. Because of the low average waves and coarse sands the beach is always steep and reflective, with deep water off the shore. High east and southeast swell which periodically reach the beach result in a strong and dangerous shorebreak.*

Umina Beach *– occupies the western 1.2km of Ocean-Umina beach. The waves, clear of the tidal delta initially increase slightly in size to average about 1m, then decrease into the western corner, where the beach turns to face east, and Ettalong creek drains across the beach (see Plates 9 & 10). Along this section the bars remain low and wide, but rips are more common and stronger when the waves are breaking.*

Ocean Beach *– commences at Wagstaff Point, sand entrance to Brisbane Water, and trends west for 1.3km. It is sheltered by Box Head and the tidal delta with usually low waves along the shore, and a reflective beach in lee of the shallow shoals. However waves breaking over the shoals can generate rip currents together with tidal currents flowing out of Brisbane Water.*

Putty Beach *– a slightly curving 1.6km long southeast-facing beach, located between prominent 70m high sandstone headlands, the eastern third of which is the national park, while the densely vegetated slopes behind the western end rise to 130m. The beach is well exposed to southerly waves which* *increase in size towards the western surf club end of the beach. The Putty end usually has an attached bar which continues to Kilcare. Here higher waves and rips are more common, with up to eight rips forming along the beach, including a permanent rip against the western rocks, where there is also a small rock pool.*

***Copacabana-MacMasters Beach** – occupies a 1.4km wide southeast-facing embayment bordered by the prominent sandstone 110m high Tudibaring Head in the north and 90m high Second Point in the south. The beach faces the east-southeast and receives waves that average 1.5m at Copacabana, decreasing to about 1m at MacMasters. These maintain a single bar, which is usually attached along the beach, but cut by 6-8 rips, which decrease in size and intensity to the south, often infilling at MacMasters forming a continuous, attached bar. A strong permanent rip runs out along the northern head, and during high seas a similar rip is formed against the southern head, particularly during summer northeast waves.*

***North Avoca & Avoca Beach** – the 1.7km long beach lies between two prominent 60m high sandstone headlands and faces the east-southeast exposing it to waves averaging 1.5m. Avoca Lake backs the centre of the beach and opens during floods. The beach receives higher waves towards the north and centre where the bar is often detached and usually cut by several rips, including a permanent rip against the northern headland. At Avoca slight protection by the southern headland lowers waves in the southern corner to form a continuous, attached bar. However rips are frequent and a permanent rip runs out against the southern rocks.*

***Terrigal-Wamberal Beach** – is a 2.8km long stretch of sand that trends southwest from the rocks on the north side of Wamberal Lagoon entrance south to Terrigal Lagoon entrance where the beach begins to curve around to the southeast to terminate at the rocks on the southern end of Terrigal Beach, in lee of Broken Head. The beach blocks the entrances to two drowned valleys, now occupied by Wamberal and Terrigal lagoons, which only open during heavy rain.*

The northern 1.5km of Wamberal Beach lies in the Wamberal Lagoon Nature Reserve. The 1km long 20m high foredune, between Wamberal and Terrigal lagoons, has been developed for beachfront housing. South of the lagoon mouth, rocky bluffs then a low dune back the 700m long Terrigal Beach. The northern Wamberal Beach is well exposed with waves averaging 1.5m and up to 15 rips dominating the surf zone. As wave height drops to the south the rips decrease in size, with often a continuous bar forming along Terrigal Beach.

***Forresters Beach** – a 1.5km long, southeast-facing sandy beach located along the base of vegetated bluffs rising to 100m. It is bordered by 130m high Cromarity Hill in the north and Wamberal Point to the south. The entire beach is fronted by extensive rocks and reefs, which abut the shore in the north extending 250m offshore in the south. Wave breaking on the reefs, lower waves at the shore to less than 1m, which maintain a narrow, steep, reflective beach.*

2.2 Historical Setting

2.2.1 General

According to the Central Coast NSW website (2010), the first European settlement of the Gosford district began in the 1820s, with most development occurring in the eastern or coastal sector. Early industry mainly consisted of timber-getting, lime burning, shipbuilding, grazing and citrus orchards. Transport improvements, including construction of the railway network by 1887 and the Pacific Highway in 1930, resulted in a steady increase in urbanisation and transformed the region from a rural community prior to World War II, to that of a city containing some secondary and service industries related to the tourist trade.

Some relevant historical features of each beach in the study area are provided in the following sections.

2.2.2 Patonga Beach

A photograph of Patonga Beach taken in the 1920s is provided in **Figure 2**[3], which shows several properties located along the relatively flat frontal dune. Patterson Britton & Partners (1998) notes that, in the late 1960s the outlet to Patonga Creek meandered to the north eroding into the caravan park. Council constructed a training wall on the northern side of the entrance in 1969/70 to direct flows further to the south, while another wall was constructed in 1971 immediately upstream of the training wall to prevent erosion in this area. Sand accreted against the northern side of the training wall until sand bypassing was re-established in the 1990s.

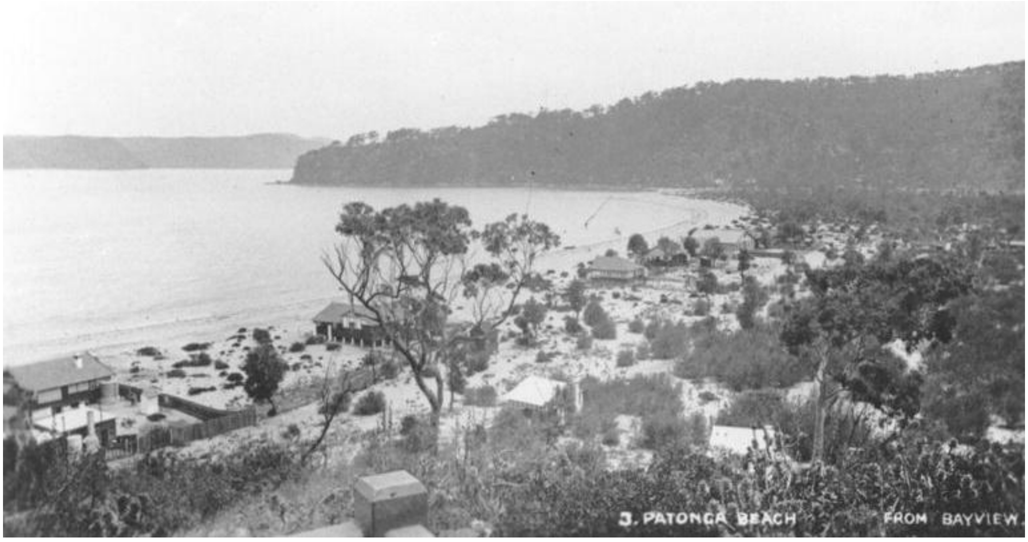

Figure 2: Patonga Beach in the 1920s

[3] Sourced from Gosford City Council Online Photo Archives, http://photosau.com/gosford/scripts/home.asp (accessed 27/1/12) File 002\002540.

2.2.3 Pearl Beach

Despite being first accessed by Europeans in March 1788 when Governor Phillip rowed ashore, Pearl Beach remained unsettled until 1921 when the Rock Davis Estate was bought by real estate developer Charles Staples, who subdivided it into 570 blocks and cut a road below Mt Ettalong (O'Brien 2009). Short (2007) notes that this road was replaced by the existing over the hill route when high seas destroyed part of the lower road. A photograph of Pearl Beach taken in the 1920s is provided in **Figure 3**[4], which shows several properties located along a reasonably well vegetated dune towards the northern end of the beach around Coral Crescent. Development at Pearl Beach has increased rapidly in recent times, with the number of dwellings rising from 90 in 1950 to around 600 today (O'Brien 2009).

Figure 3: Pearl Beach in the 1930s or 1940s

2.2.4 Ocean-Umina Beach

The formation of Ocean-Umina Beach over the previous 9000 years as part of the Woy Woy beach ridge barrier is discussed in **Appendix D**. A photograph of Ocean-Umina Beach taken in the 1920s is provided in **Figure 4**[5], which shows a reasonably wide sandy beach rising to a lightly vegetated foredune. The Surf Lifesaving Clubs at Ocean Beach and Umina Beach were formed in 1921 and 1958 respectively (Short 2007). The construction of a groyne at Ettalong Point following erosion in 1973 has likely enhanced the permanency of accreted sand at the northern end of Ocean Beach

[4] Sourced from Gosford City Council Online Photo Archives, http://photosau.com/gosford/scripts/home.asp (accessed 27/1/12) File 002\002458.
[5] Sourced from Gosford City Council Online Photo Archives, http://photosau.com/gosford/scripts/home.asp (accessed 27/1/12) File 002\002406.

(Patterson Britton & Partners 1998). At the southern end of the beach between Mt Ettalong and the Ettalong Creek entrance, it is understood that rock protection has been placed on the seaward side of the road during severe storm erosion events (Patterson Britton & Partners 1998).

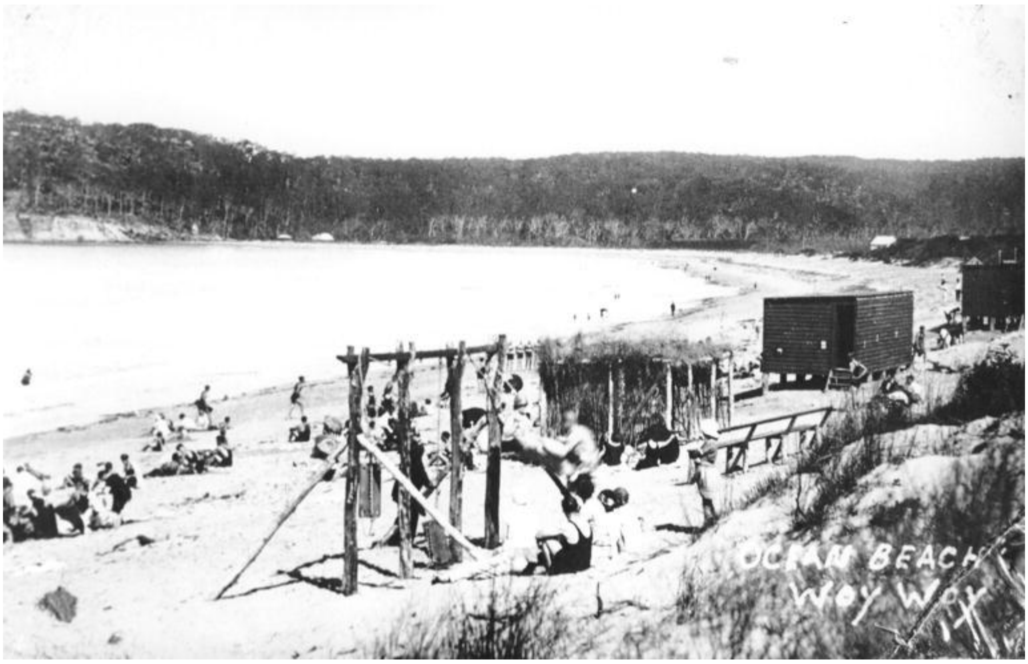

Figure 4: Ocean-Umina Beach in the 1920s

2.2.5 Putty Beach

Killcare Surf Lifesaving Club was formed in 1932 (Short 2007). Sand mining of the frontal dune at Putty Beach was undertaken during the late 1950s and 1960s (Patterson Britton & Partners 1998). The frontal dune was extensively lowered as a result, although subsequent revegetation efforts have succeeded in mitigating previous windblown sand losses (Patterson Britton & Partners 1998). This is discussed further in **Appendix E**.

2.2.6 MacMasters-Copacabana Beach

MacMasters Beach was occupied by the MacMaster family from the 1840s, although development progressed rapidly from 1927 onwards when Banavie Estate was subdivided, including beachfront property on Marine Parade (Gosford City Council Website 2012). Copacabana was previously known as Tudibaring (an Aboriginal word meaning "where the waves pound like a beating heart") until it was subdivided in 1954 and the northern side of Cockrone Lake was subsequently opened up (CentralCoastAustralia.com 2012). Development of MacMasters Beach and Copacabana has occurred rapidly since the 1950s. Short (2007) notes that the Surf Life Saving Clubs at MacMasters Beach and Copacabana Beach were formed in 1946 and 1963 respectively.

2.2.7 Avoca Beach

Avoca Beach became accessible to the public in 1908 when the first bridge was built across Avoca Lake and the Avoca Guest House was constructed at what was then called Moore's Beach (Short 2007). A view of Avoca Beach in around 1926 is provided in **Figure 5**, sourced from the National Library of Australia. Evident in this photograph is a wide sandy beach in front of a densely vegetated foredune which is now occupied predominantly by residential development. Further, the photograph of Avoca Beach taken in 1948 (**Figure 6**) generally indicates that development at North Avoca did not commence until the latter half of the 20th Century, while some development had occurred at Avoca at this time. The Surf Lifesaving Clubs at Avoca and North Avoca were formed in 1929 and 1957 respectively (Short 2007).

Figure 5: Avoca Beach in around 1926

Figure 6: Avoca Beach in 1948
(photo courtesy Gosford City Council)

2.2.8 Terrigal-Wamberal Beach

Photographs of Wamberal and Terrigal beaches taken in the early 1900s are presented in **Figure 7**[6] and **Figure 8**[7] respectively. The photograph of Wamberal Beach (Figure 7) provides evidence of a fairly prominent erosion scarp at the interface of the beach and densely vegetated foredune. Much of this area is currently occupied by residential development along Ocean View Drive and Pacific Street. The photograph of Terrigal Beach (Figure 8) displays a lightly vegetated incipient dune system along much of the beach, which is generally nonexistent today.

Short (2007) notes that the Surf Life Saving Clubs at Terrigal and Wamberal were formed in 1924 and 1950 respectively.

[6] Sourced from State Library of NSW PICMAN database, Digital Order No a106273.
[7] Sourced from State Library of NSW PICMAN database, Digital Order No a106285.

Figure 7: Wamberal Beach in the early 1900s

Figure 8: Terrigal Beach in the early 1900s

The seawall behind Terrigal Beach protects popular recreational amenity and the local business district. Photographs of Terrigal Beach taken in the 1940s, 1988 and 2011 are provided in **Figure 9**, **Figure 10** and **Figure 11**, which show various states of the seawall over time, while it is also noted from observation of a photograph taken in 1972 that a seawall was not present at this time. It can be seen that the seawall was initially positioned further seaward at the southern end, while the photographs taken in 1988 and 1999 show the change from a timber wall to the existing sandstone-coloured concrete block seawall.

Figure 9: Terrigal Seawall in the 1940s
(photo courtesy Gosford City Council)

Figure 10: Terrigal Beach in 1988 showing timber seawall
(Source: MHL 2003)

Figure 11: Construction of the sandstone block seawall at Terrigal Beach, December 1999
(photo courtesy Gosford City Council)

2.2.9 Forresters Beach

Beachfront property development at Forresters Beach did not commence until the 1950s (PWD 1994). Extensive subdivision of the top dune, south headland and immediately behind the dune occurred in the 1960s, which involved levelling the top of the dune and pushing a significant volume of sand seaward, completely altering the dune alignment and shape (PWD 1994). This is discussed further in **Appendix I**.

2.3 Major Storm Events

2.3.1 General

While settlement along the Central Coast commenced in the 1800s, the area has been the focus of continued urban development since the 1960s (Short 2007). The threat of erosion to properties in the Gosford LGA has been demonstrated during storms from this time (MHL 2002). The most significant storm events that affected properties in the study area occurred in 1974 and 1978, while the 1986 storm also resulted in major beach erosion. There have been relatively few major storm events and associated erosion damage over the previous 20 years. An exception to this perhaps is the May 1997 storm, which not only consisted of high seas but had an angle of approach that caused significant damage to the entire NSW coastline (MHL 2002). Coastal storms in 1985 and 1995 also significantly impacted the beaches in the study area.

According to PWD (1985a), the passage of successive storms within a relatively short time period is more likely to result in erosion of the foredune than individual storm events. Importantly, the 1974 storms consisted of two separate but closely spaced events on 25-26 May and 8-14 June respectively, while the 1978 storms consisted of four individual events on 18-20 March, 31 May to 2 June, 15-16 June and 18-21 June respectively. While there are other examples of major storm events impacting the Central Coast region that are of similar magnitude to the 1974 and 1978 storms (e.g. 20-21 May 1966, 5-6 September 1967, 13-15 May 1968, 9 July 1983), no serious damage was reported in these cases.

A quantitative description of storm history on the NSW Central Coast is provided in **Sections 5.4.3** and **5.4.4**. The following sections provide a brief summary of the impacts of some of the most significant storm events on beaches in the study area, including:

- May-June 1974 (see **Section 2.3.2**);
- June 1978 (see **Section 2.3.3**);
- September 1985 (see **Section 2.3.4**);
- August 1986 (see **Section 2.3.5**);
- September 1995 (see **Section 2.3.6**); and
- May 1997 (see **Section 2.3.7**).

The information provided in the following sections was gathered from a number of sources, including past reports, available photographs (including photographs sourced from Gosford City Council archives) and photogrammetry data. It is recognised that the storm events discussed below would have likely impacted each beach in the study area to some degree. However, discussion has only been provided regarding those impacts that can be supported by a suitable reference.

2.3.2 1974 Storms

As described in **Section 5.4.3**, the major storms of May-June 1974 are the most significant coastal storms that have been recorded to have impacted on the Central Coast of NSW. These storms generally produced the most significant erosion along the beaches in the study area, which is evident from the assessment of photogrammetric data as described in **Appendices B** to **I** for each beach.

PWD (1985a) noted that the major storms of May-June 1974 severely damaged one house at the northern end of Wamberal Beach with the collapse of its seaward brick foundation wall which was undermined by dune erosion (**Figure 12**). A photograph of waves impacting the primary dune at Wamberal Beach is provided in **Figure 13**. The severe erosion threatened virtually all beachfront developments at North Avoca (**Figure 14**) and Terrigal-Wamberal, while the State Emergency Services and Australian Army were called in to place rocks, sand bags and other materials in front of the eroding dune face in an effort to halt further shoreline recession. Septic tanks were placed in front of the home units at Pacific Street by private contractors.

Resident surveys undertaken as part of the *Avoca Beach Storm Wave Inundation Study* (Cardno Lawson & Treloar 2007) described oceanic inundation around Ficus Avenue during the 1974 storms. Several properties were inundated in this area, while a fibro cottage was knocked off its piers, and waves were observed travelling upstream through Avoca Lake past the oval.

The severity of the 1974 storm event at MacMasters-Copacabana Beach is evident in **Figure 15**[8], and **Figure 16** which show significant erosion along the entire beach, with all the sand stripped from the southern end of MacMasters Beach, leaving behind the rocky substrate. While the beach had partially recovered by 1975, the erosion scarp in front of Copacabana SLSC is still present at this time (**Figure 17**).

Further, a large amount of the fill on which Killcare Surf Club is located was eroded down to an extensive rock shelf and threatened to undermine the building during the May-June 1974 storms (Patterson Britton & Partners 1998).

It is also understood that beach scraping was undertaken at Pearl Beach to help protect homes in Coral Crescent (**Figure 18**).

Figure 12: Dune Erosion threatening Property at Wamberal Beach, 1974
(Source: PWD 1985b)

[8] Sourced from Gosford City Council Online Photo Archives, http://photosau.com/gosford/scripts/home.asp (accessed 27/1/12) File 000\000611.

Figure 13: Storm waves attacking primary dune at Wamberal Beach, 1974
(photo courtesy Gosford City Council)

Figure 14: Dune Erosion at North Avoca Beach, 1974
(photo courtesy Gosford City Council)

Figure 15: Erosion at MacMasters Beach, 1974

Figure 16: Erosion Scarp at Copacabana Beach, 1974
(photo courtesy Gosford City Council)

Figure 17: Partially Recovered Erosion Scarp at Copacabana Beach, 1975
(photo courtesy Gosford City Council)

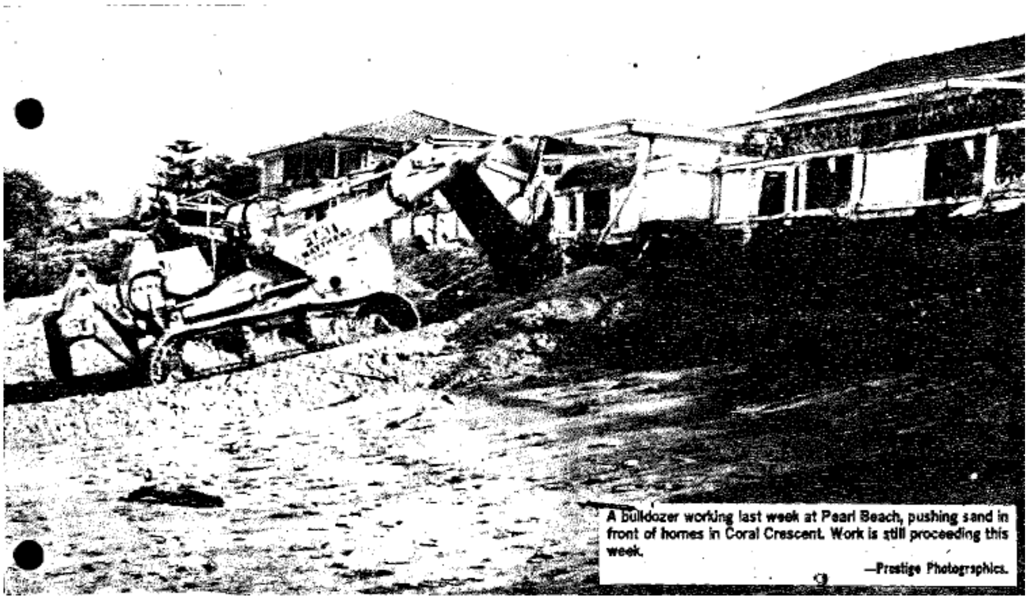

Figure 18: Beach Scraping at Pearl Beach, 1974
(Source: Central Coast Express, 7 June 1974)

2.3.3 1978 Storms

The major storms of June 1978 also produced significant erosion along the beaches in the study area, most notably at Avoca and Terrigal-Wamberal. Following the severe effects of the 1974 storms, many beachfront owners at Avoca and Terrigal-Wamberal constructed a variety of shore protection structures comprising rock rubble, corrugated iron, rubber tyres, besser blocks and concrete walls, while some sprayed the dune face with gunite and others attempted to mitigate erosion by planting vegetation (PWD 1985b). These ad hoc shore protection measures did not appear to be designed or constructed on sound coastal engineering principles, which was demonstrated during the severe storms of June 1978 when two houses collapsed into the ocean at Wamberal Beach (PWD 1985b). **Figure 19** shows the first of these houses after it had collapsed, while the second house on the block to the south was lost later that evening at high tide, and the house on the far left of the photo was saved by relocation landward during the storm (PWD 1985b). Extensive wave inundation also occurred around Ficus Avenue adjacent to Avoca Lake during the severe storms in 1978 (**Figure 20**).

The assessment of photogrammetric data, as described in **Appendices B** to **I** for each beach, also provides evidence of the erosional impacts of the major storms of June 1978, although it is noted that suitable data for the purposes of assessing erosion caused by this event was not available for Patonga, Putty, MacMasters-Copacabana and Forresters beaches. However, as stated previously, it is recognised that the 1978 storms would have likely impacted each of the beaches in the study area to some degree, and indeed it is noted in Geomarine (1989) that severe dune erosion was observed at MacMasters-Copacabana Beach during this event.

Figure 19: Property Collapse at Wamberal Beach, 1978
(Source: News Limited)

Figure 20: Wave overwash at Ficus Avenue in 1978
(photo courtesy Gosford City Council)

2.3.4 1985 Storms

Notable impacts of the storms in September 1985 include significant erosion at:

- Putty Beach (**Figure 21**), which is thought to have been exacerbated by a rip that formed adjacent to the rocks at the southern end; and
- Ocean Beach (**Figure 22**), which resulted in the collapses of a timber beach accessway.

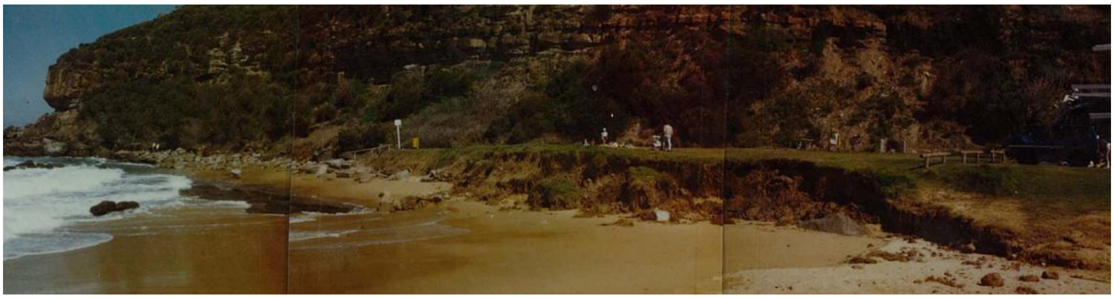

Figure 21: Erosion at Putty Beach, September 1985
(photo courtesy Gosford City Council)

Figure 22: Erosion at Ocean Beach, September 1985
(photo courtesy Gosford City Council)

2.3.5 1986 Storms

The August 1986 storms also resulted in major erosion in the study area. This is borne out in the photogrammetric data assessment, which is described in **Appendices B** to **I** for each beach in the study area. In particular, significant erosion was observed for Pearl, Ocean-Umina, MacMasters, Terrigal-Wamberal and Forresters beaches. Further, this storm event was characterised by offshore waves approaching from the east-south-east, enabling a more direct angle of wave attack to the beaches in Broken Bay and also the southern ends of beaches on the open coast.

Notable features of the August 1986 storm event included:

- Significant erosion exposing rock immediately seaward of properties at Wamberal (**Figure 23**) and North Avoca (**Figure 24**);
- Significant erosion at MacMasters Beach with wave uprush through dune fencing (**Figure 25**);
- Significant erosion at Ocean-Umina Beach with wave uprush through the dune impacting the carpark on The Esplanade (**Figure 26**);
- Strong wave conditions at Pearl Beach (**Figure 27**), with flows between the beach and lagoon crossing the culvert and over Coral Crescent (**Figure 28**);
- Erosion at Pearl Beach leading to subsequent beach scraping undertaken by bulldozers in front of properties along Coral Crescent; and
- Damage to the jetty at Patonga Beach (**Figure 29**), which was subsequently reconstructed to withstand wave uplift forces (Patterson Britton & Partners 1998).

Figure 23: Exposed rock at Wamberal, August 1986
(photo courtesy Gosford City Council)

Figure 24: Erosion near Properties at North Avoca, August 1986
(photo courtesy Gosford City Council)

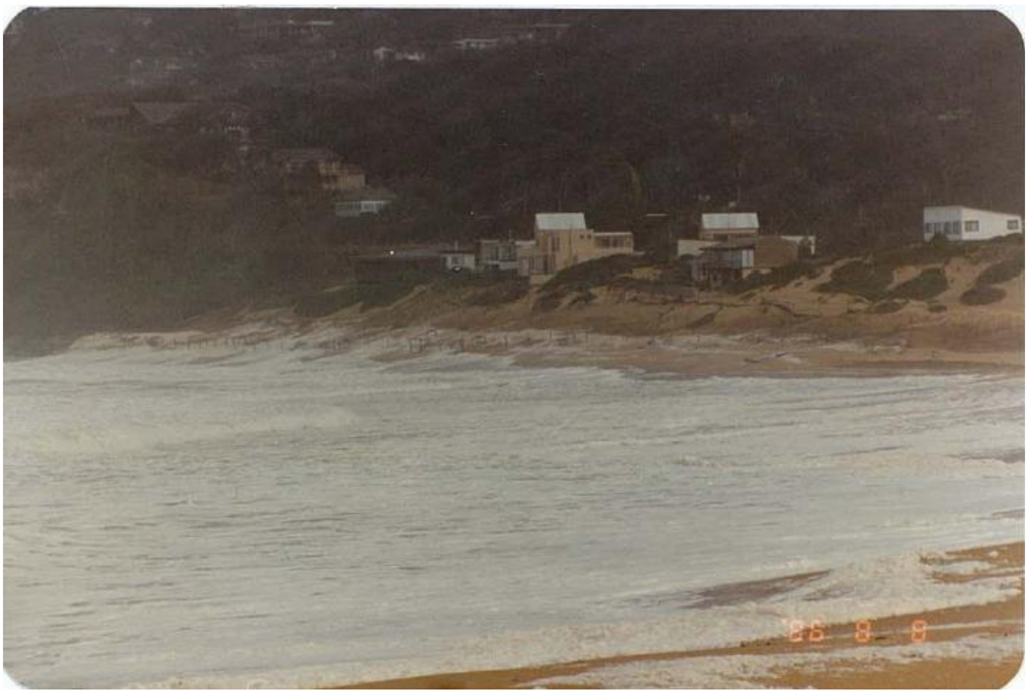

Figure 25: Erosion at MacMasters Beach, August 1986
(photo courtesy Gosford City Council)

Figure 26: Erosion at Ocean-Umina Beach, August 1986
(photo courtesy Gosford City Council)

Figure 27: Storm Conditions at Pearl Beach, August 1986
(photo courtesy Gosford City Council)

Figure 28: Flows across the culvert near the Lagoon Entrance at Pearl Beach, August 1986
(photo courtesy Gosford City Council)

Figure 29: Damage to Jetty at Patonga Beach, August 1986
(photo courtesy Gosford City Council)

2.3.6 1995 Storms

Figure 30 shows storm conditions facing north from Terrigal Beach during September 1995. Evident in this photograph is complete erosion of the beach face up to the old timber seawall, as well as minor overtopping of the wall.

Storm conditions at Wamberal Beach during September 1995 are shown in **Figure 31**. Noteworthy in this photograph is the complete erosion of the foredune area with wave runup occurring all the way to the primary dune area. The opening to Terrigal Lagoon also spans a much greater area compared to typical lagoon opening events.

Figure 30: Storm Conditions facing north from Terrigal Beach, September 1995
(photo courtesy Gosford City Council)

Figure 31: Storm Conditions at Wamberal Beach, September 1995
(photo courtesy Gosford City Council)

2.3.7 1997 Storms

Patterson Britton & Partners (1998) noted the following major impacts on Broken Bay beaches related to the May 1997 storm event:

- Wave runup washed significant quantities of sand into the Killcare Surf Club car park.
- Following this storm event, wave runup debris lines were observed along the seaward boundary fence of the properties at the southern end of Berrima Crescent at Umina Beach, and also at Pearl Beach on the front lawns of northern properties.
- Wave runup was observed to flow through the box culvert under Patonga Drive and onto Eve Williams Memorial Oval. Further, the footpath at the base of the jetty at Patonga Wharf was undermined and significant quantities of sand and debris were washed onto the carpark and road.

3. CHRONOLOGY OF PREVIOUS COASTAL STUDIES RELATING TO STUDY AREA

As part of this study a comprehensive search and review of previous literature was undertaken. The more important reports relevant to the current study are outlined below for Broken Bay and Open Coast beaches respectively.

3.1 Broken Bay Beaches

A Coastal Processes Study for the Broken Bay beaches was completed by Patterson Britton & Partners (1998). The purpose of this study was to quantify the risk to development from coastal hazards and define estuarine processes, ecology and water quality. Community consultation was also undertaken as part of this study. Reference to some of the parameters estimated by Patterson Britton & Partners (1998) has been made elsewhere (as relevant) in the investigation reported herein.

Patterson Britton & Partners (1998) defined the immediate (1997), 50 year (2047) and 100 year (2097) hazard lines for Broken Bay beaches[9]. The 50 year and 100 year hazard lines were based on predicted shoreline recession assuming sea level rise of 0.26m and 0.61m respectively, the "best estimates" (mid-range projections) as advised by NSW Government (PWD 1994).

The *Broken Bay Beaches Coastal Management Plan* was subsequently developed for Broken Bay beaches (including Putty, Ocean-Umina, Pearl and Patonga) and documented in Patterson Britton & Partners (1999). This Plan adopted the hazard lines provided in the Coastal Processes Study and recommended some management actions as being of high priority, which are summarised in Table 1 along with the implementation status of each action.

Table 1: Management actions for Broken Bay beaches

Management Item Source: Patterson Britton & Partners (1999)	Implementation Status Source: Gosford City Council (pers. comm., 6/2/12)
Adopting suitable development controls	Implemented
Consideration of relocating Putty Beach Surf Club landward, or constructing a terminal rock revetment to protect the club	Unknown
Maintenance of dune vegetation and fencing	Implemented and ongoing
Consideration of relocating properties located within the hazard zones landward, including properties along Ocean-Umina Beach	Occurs over time in line with hazard information
Consideration of development of a seawall to protect one at-risk	Not implemented

[9] The study area covered by Patterson Britton & Partners (1998) extends from the southern end of MacMasters Beach incorporating Bouddi National Park, Killcare Beach, Ocean/Umina Beaches (commencing from Ettalong Point), Pearl Beach and Patonga Beach. The local estuaries in the study area included Ettalong Creek, Pearl Beach Lagoon, Green Point Creek, Middle Creek, Patonga Creek and Brisbane Water entrance.

Management Item Source: Patterson Britton & Partners (1999)	Implementation Status Source: Gosford City Council (pers. comm., 6/2/12)
property on the southern side of Green Point Creek	
Purchasing privately owned properties following loss or damage, or on redevelopment any building should be located landward of the 50 year hazard line, piled and indemnify Council against future damage by coastal processes	DCP125 imposes special conditions on development seaward of the 2098 (ie 100 year) hazard lines at all beaches, except at Pearl Beach where a building line has been defined at Coral Crescent.
Formulate guidelines for the opening of all creek entrances on Pearl Beach	Not included in lagoon opening policy. However, Council is about to embark upon a CZMP for Pearl Beach lagoon which will look at entrance management procedures for the northern lagoon entrance.
Formulate guidelines for beach scraping along Pearl Beach for improving access to and along the beach	Beach scraping guidelines developed. Procedure not encouraged.

3.2 Open Coast Beaches

Major coastal storms in 1974 and 1978, which resulted in loss of public and private assets and adversely affected beach amenity (through construction of ad hoc protective works) in NSW, led to attention being focussed on better planning and management of the coastal zone. In 1983, the NSW Public Works Department commenced an overall study to investigate coastal processes operating along the open coastline of the Gosford LGA. The findings of these investigations were reported in PWD (1985b) for Avoca and Terrigal-Wamberal beaches, and PWD (1989) for Forresters Beach. These investigations resulted in Council placing restrictions on further development within the coastal zone deemed to be subject to hazards.

Refraction analysis of Avoca and Terrigal-Wamberal was undertaken by Lawson & Treloar (1984) to determine the nearshore wave height and direction statistics for Avoca and Terrigal-Wamberal.

PWD (1985b) provides a description of findings from seabed mapping, including the locations of bedrock reef, which notes that reef systems extend from around 40m depth adjacent to Avoca Beach, whereas adjacent to Terrigal-Wamberal Beach these reef systems commence at around 25m depth. It is noted also that Terrigal-Wamberal may experience long-term beach volume loss due to sand becoming trapped in offshore reef systems given that a typical beach system extends to 25-30m water depth.

Design storm demands were also calculated in PWD (1985b) for Terrigal-Wamberal and Avoca, which were based on erosion attributable to the 1974 storm events. While coastline hazard mapping was not undertaken (i.e. hazard lines were not defined), design storm demands determined from photogrammetry were applied to a ‘beach full’ profile obtained from 1984 aerial photography to indicate regions over which the dune may be attacked by waves during a severe storm. Wave runup levels along the beaches were also calculated to indicate regions where oceanic inundation may occur.

Nielsen et al (1992) provided a useful summary of studies carried out by PWD in the early 1980s for Avoca and Terrigal-Wamberal beaches.

Coastal Development Manuals were prepared by Council in 1989 for Terrigal-Wamberal Beach (Gosford City Council 1989a), Avoca Beach (Gosford City Council 1989b) and MacMasters Beach (Gosford City Council 1989c). The purpose of these manuals was to define the coastal engineering information that must be considered by those making Development and Building Applications on land fronting the coast. Each manual provided a Hazard Area Map showing blocks that may be affected by coastal processes including erosion and inundation due to severe storm events, and erosion due to long-term sand loss. Specific coastal management issues relevant to these beaches are also provided.

As noted in **Section 1**, the *Gosford Coastal Process Investigation* was documented in PWD (1994). This report updated previous investigations including PWD (1985b) for Avoca and Terrigal-Wamberal beaches, and PWD (1989) for Forresters Beach, while also including data for MacMasters-Copacabana Beach. Coastline hazard mapping of these beaches was presented in PWD (1994), in which hazard lines were estimated for the immediate and 50 year (2045) planning periods. It should be noted that the 50 year hazard line includes shoreline recession due to an assumed sea level increase of 0.26 m. Reference to some of the parameters estimated by PWD (1994) has been made elsewhere (as relevant) in the investigation reported herein.

The *Coastal Management Study and Coastal Management Plan – Gosford City Open Coast Beaches* was completed by WBM Oceanics (1995), which adopted the hazard lines presented in PWD (1994). Coastline management strategies and actions that were adopted included:

- Planned retreat;
- Voluntary purchase under the State Voluntary Purchase scheme for adversely affected coastal properties;
- Beach nourishment;
- Terminal protection; and
- Implementing Council's 'Coastal Frontage – Development and Building Policy' (Development Control Plan No.125) to control, and in some cases prevent, inappropriate coastal development.

Council subsequently commissioned a geotechnical assessment of the open coast ocean beaches (Hudson 1997). This investigation determined the extent of bedrock along the study area and the likely impact (if any) of these non-erodable layers on the future beach recession scenario. Generally, the Hudson (1997) report indicated that nowhere does the basement rock level come higher than 0 m AHD at the toes of the existing dunes. Therefore, there would be very little influence of the rock on the extent of dune erosion, at least for present day conditions. With a rising sea level any influence of basement rock would diminish.

The Unisearch Ltd Water Research Laboratory prepared a detailed Design Report (WRL 1998a) and Technical Specification (WRL 1998b) for construction of a terminal protection structure along Wamberal Beach. The design preferred by Council and the community comprised a 1,350m long

Seabee armoured terminal protection structure with a rock-filled gabion basket and Reno mattress toe. This structure would span the section of beach between the lagoon entrances and extend from a level of -1 m AHD to a crest level of 6 m to 8 m AHD. Physical modelling was undertaken to refine the design (WRL 1997). Relevant design parameters for the structure, including recession rates, storm demand and nearshore wave climate, were based on coastal processes defined in PWD (1994) and WBM (1995).

Manly Hydraulics Laboratory (MHL) simultaneously undertook an assessment of viable sand sources and nourishment requirements for the coastal beaches including Terrigal-Wamberal (MHL 1998). This investigation reviewed all available sand sources, narrowing the options to three alternatives that could provide the sand volumes likely to be required. Further assessment of these potential sources and the technical and economic viability of the nourishment approach was presented in the Stage Two report, which concluded that the most likely source of sand for nourishment of Terrigal-Wamberal Beach would be from large sand reserves located offshore from the embayment in deep water (MHL 2002).

Subsequently, an Environmental Impact Statement (EIS) was prepared by MHL (2003) to assess the likely environmental impacts should Council determine to proceed with construction of a terminal protection structure at Wamberal Beach combined with periodic maintenance sand nourishment. Construction was deemed to be well placed to progress to the point that existing or proposed development would be secured, however this would be subject to appropriate funding and development approval. Further, MHL (2003) noted that while nourishment may prove to be a viable strategy for beach protection, the large volumes and ongoing maintenance requirements are dependent on a suitable, ongoing sand source that needs to be proven and secured before viability of the beach nourishment strategy can be determined. Proposed works assessed by the EIS have not proceeded due to lack of funding and because a sand source has not yet been secured. The economic evaluation that took place in the EIS process found the most viable sand source for nourishment for Wamberal was from local offshore deposits. However, this is not allowable under current state government policy. The overwhelming support of the local community for the construction of the seawall has been gleaned from numerous consultation activities.

4. DATA ACQUISITION AND ANALYSIS

4.1 Photogrammetry

Photogrammetry involves measurement and data acquisition from photographic and other remotely sensed images. It was used in this study to measure historical beach profile changes from vertical aerial photography. This assists in identification of possible recession or accretion trends, selection of appropriate base ("beach full") profiles for coastline hazard definition, and assessment of any beach rotation effects. The methodology used to analyse the photogrammetric data is outlined in **Section 5.6.3**.

The photogrammetric data collection was undertaken by the NSW Office of Environment & Heritage (OEH) using their AC3 photogrammetric instrument.

Details of the photogrammetric analysis undertaken are provided in separate appendices for each beach, as set out below:

- **Appendix B** – Patonga Beach
- **Appendix C** – Pearl Beach
- **Appendix D** – Ocean-Umina Beach
- **Appendix E** – Putty Beach
- **Appendix F** – MacMasters-Copacabana Beach
- **Appendix G** – Avoca Beach
- **Appendix H** – Terrigal-Wamberal Beach
- **Appendix I** – Forresters Beach

4.2 Spatial Data

Council supplied the following spatial datasets to assist in the study reported herein:

- Aerial photography (1997, 2005, 2007 and 2010 aerial photography);
- Cadastre (including Lot and Deposited Plan information);
- Light Detection and Ranging (LiDAR) data (0.5m interval, collected in 2007);
- Stormwater pipes, channels and outlets;
- Waterways (e.g. creek lines);
- Flood extents;

- Existing coastline hazard lines;
- Zoning;
- Vegetation mapping;
- Crown Lands; and
- National Parks.

The 2010 aerial photography has been used in the figures showing the revised coastline hazard lines and inundation extents. The cadastre has also been shown on these figures to indicate individual properties, although it should be noted that Council have advised that there may be some errors with the cadastre information.

The LiDAR data was used to assist in identifying low lying areas subject to coastal inundation, and to define 0 m AHD elevations in the Gosford LGA to assist in the delineation of a landward boundary in the nearshore wave model bathymetry.

5. COASTAL PROCESSES

In this Section, the coastal processes prevalent along the study area coastline are outlined. In particular, details are provided on:

- wave climate (Section 5.1);
- elevated water levels (Section 5.2);
- wave runup (Section 5.3);
- coastal storms (Section 5.4);
- short term onshore/offshore sediment transport (Section 5.5);
- longer term sand movement (Section 5.6);
- climate change (Section 5.7); and
- Lagoon Entrance Processes (Section 5.8).

5.1 Wave Climate

5.1.1 Offshore Wave Climate

The study site is located in the south-west Pacific at around 33.5°S and receives waves generated in the southern Coral and Tasman Seas and the Southern Ocean. The annual wave climate is both energetic and highly variable with a distinct seasonality present with the largest average monthly wave heights experienced in winter. Although moderate waves dominate the climate, large waves (Hs>4 m) and/or low swell may occur in any month (Short and Trenaman 1991). Extreme events (Hs>6m) occur predominately in autumn and winter. Waves in the region are generated by five typical meteorological systems: east-coast lows, tropical cyclones, mid-latitude cyclones, zonal anticyclonic highs and local summer sea breezes (Short and Trenaman 1991).

Manly Hydraulics Laboratory (MHL), part of the NSW Department of Service Technology and Administration, operates a network of Waverider buoys in deep water along the NSW coast. Waverider buoys are spherical floating accelerometers which determine sea level surface displacement based on the double integration of measured vertical accelerations. Analysis of the collected data allows (among other things) the significant wave height (H_s) and peak spectral wave period (T_p) to be determined[10]. For the NSW network, records are collected for 2048s bursts (about 34 minutes) every hour at 0.5s intervals (Lord and Kulmar 2001).

[10] The significant wave height is the average height of the highest one-third of the waves in a particular record. The peak spectral wave period is determined by the inverse of the frequency at which the wave energy spectrum reaches its maximum.

The nearest Waverider buoy to the study area is located approximately 11km east-south-east of Long Reef in Sydney's northern beaches, in a water depth of about 85m (MHL 2011). This buoy is denoted by MHL as the Sydney Waverider Buoy. Wave data collected at this location is considered to be representative of offshore wave conditions that will influence coastal processes in the Gosford LGA.

Waverider buoys can be non-directional or directional. Directional buoys allow the predominant wave direction to be determined. The Sydney Waverider Buoy has been operating since July 1987, originally non-directional, but directional from March 1992 to present[11].

Hourly wave data from the Sydney Waverider Buoy covered the period from 3 March 1992 to 30 April 2010 with an 81% capture rate. The data consisted of H_s, H_{max}, T_z, and T_p for this period where H_{max} is the maximum wave height and T_z is the zero crossing wave period. It was evident that, over the period of record:

- the average H_s was 1.6m, the median or 50th percentile H_s was 1.5m;
- the average T_p was 9.8s;
- H_s exceeded 3m for 5% of the time;
- H_s exceeded 4m, 5m and 6m for 1.3%, 0.3% and 0.1% of the time respectively;
- 59% of T_p values were between 8s and 12s;
- 90% of T_p values were between 6s and 14s; and
- T_p exceeded 14s and 16s for 3.6% and 0.6% of the time respectively.

The occurrence of waves from each direction was as listed in Table 2[12]. It is evident that the majority (approximately 65%) of offshore waves propagate from the S-SE sector (i.e. S, SSE and SE cardinal directions). S-SE waves originate from storms and swells originating in the Tasman Sea and Southern Ocean and can occur during any season. Easterly waves (i.e. ESE, E and ENE cardinal directions) make up approximately 30% of the total offshore wave energy. N-NE waves make up approximately 3% of the offshore wave energy and are generated by summer sea breeze systems and tropical cyclones in the Coral Sea. The largest period waves typically occur from the S-SE sector in the winter months. The median wave direction was 146° (SSE), with the weighted vector average storm wave direction equal to 135° (SE).

The directional occurrence of storm waves (H_s exceeding 3m) is also listed in Table 2. It is evident that the dominant storm wave direction was from the S (about 38% of storm waves), with about 31%

[11] MHL also operates Waverider buoys (with commencement dates in brackets) at Byron Bay (October 1976), Coffs Harbour (May 1976), Crowdy Head (October 1985), Port Kembla (February 1974), Batemans Bay (May 1986) and Eden (February 1978). The Byron Bay and Batemans Bay Waverider buoys are directional, with the other sites having non-directional buoys. Sydney Ports Corporation operates a non-directional Waverider buoy located offshore of Botany Bay, which has been collecting data since April 1971.
[12] The occurrence of waves from SW through W to NNW was only 0.4%, and as these directions do not produce onshore waves, they were excluded from Table 2.

from the SSE and 13% from the SE. The median storm wave direction was 163° (SSE/S), with the weighted vector average storm wave direction equal to 153° (SSE). It can also be noted that waves from E through NE to N only accounted for about 9% of the storm waves.

Table 2: Occurrence of waves from each offshore wave direction for the Sydney Waverider Buoy from 1992 to 2010

Direction	Occurrence (%)	Occurrence for waves with Hs exceeding 3m (%)
N	0.0	0.0
NNE	0.1	0.0
NE	3.1	0.2
ENE	9.1	2.4
E	11.1	6.6
ESE	10.3	7.4
SE	16.4	13.3
SSE	29.9	31.4
S	18.7	37.5
SSW	1.0	1.2

Based on all data collected at the directional Sydney Waverider Buoy to the end of 2004, Kulmar et al (2005) predicted that the 100 year average recurrence interval (ARI) H_s exceeded for a duration of 1 hour and 6 hours offshore of Sydney was 9.5m and 8.5m respectively.

Beach erosion is strongly linked to the occurrence of high wave conditions with elevated ocean water levels (the latter are discussed in **Section 5.2**). Therefore, inclusion of duration is likely to more accurately describe the severity of a storm in terms of beach erosion, rather than using ARI alone (Lawson and Youll 1977). Erosion is more likely to be significant when the large waves coincide with a high tide. In general, storms with a duration in excess of 6 hours are likely to coincide with high tide on the NSW coast (Lord and Kulmar 2001). It is therefore considered that the 6 hour duration is the most appropriate to use for beach erosion and wave runup considerations, and as such has been adopted for use in the investigation reported herein.

The influence of a range of climate oscillations, such as the El Niño Southern Oscillation, may help to explain the high variability observed in the offshore wave climate in the Sydney region (Harley et. al. 2009). Climate change may influence future trends in the offshore wave climate (McInnes et. al. 2007). The potential impacts of climate change and the relative time frames are discussed further in **Section 5.9**.

5.1.2 Extreme Waves

Directional extreme waves for the 1, 50 and 100 year return periods have been estimated for the Sydney region based primarily on the analysis of the directional Sydney Waverider Buoy data collected from 1992 to 2008. Extreme value analysis was conducted, with a set of 340 peak storm wave heights derived from the available data. Care was taken to ensure that each storm event selected was from an independent synoptic event. A number of candidate probability distribution functions were fitted to the peak storm data sample following the method recommended by Goda (2000). A variety of other wave data sources were also used to verify and validate this analysis, including:

- Time series of wave parameters output from the National Oceanic and Atmospheric Administration Wavewatch3 (WW3) global model for a period of 9 years (1997-2005). Output data was taken from the closest WW3 grid point to Sydney, 34.0°S 152.25°E.
- Data from the International Comprehensive Ocean-Atmosphere Data Set, which is a compilation of data from various sources including measurements and visual observations from ships, moored and drifting buoys, coastal stations and other marine platforms. Data for the period from 1956-2007 was extracted over the area from 33°S to 34°S latitude and 152°E to 153°E longitude.

Directional factors were calculated based on a review of the 1 year, 100 year, 0.1% and 0.01% exceedence directional distribution for each dataset. The directional extreme significant wave height values that have been quoted are based on comparison of results from all data sources, taking into account the inherent merit and limitations of each data source, as well as judgement based on knowledge of the general meteorology and oceanography of the area. The peak period (T_p) associated with the extreme wave height is not necessarily the return value of the extreme peak period. The extreme peak periods were estimated using standard empirical relationships based on a typical range of significant wave steepness.

The Weibull 3 parameter distribution (maximum likelihood estimator, peak over threshold of 3m) was found to provide the best fit to the data, with the resulting 1 year, 50 year and 100 year ARI H_s and T_p values from each offshore wave direction listed in Table 3. The wave height likely to occur or be exceeded, on average, every 100 years was estimated to be 9.3m. This value compares well with previously reported values for the 100 year return significant wave height for the Sydney region. Relative wave energy values are also listed in Table 3 for the 100 year return period, with wave energy calculated as $H_s^2T_p^2$ and expressed relative to the maximum wave energy from the SSE direction being equal to unity. It is evident that offshore wave energy is concentrated into the SE, SSE and S directions, with 64% of the total offshore energy coming from these directions.

Table 3: Offshore directional wave extremes for the study region

Return Period	Direction							
	NE	ENE	E	ESE	SE	SSE	S	SSW
1-year								
Significant Wave Height (H_s) (m)	3.0	4.2	4.8	5.0	5.8	6.4	6.1	3.8
Peak energy period (T_p) (s)	7.6	8.9	9.6	9.8	10.5	11.1	10.8	8.5
50-year								
Significant Wave Height (H_s) (m)	4.1	5.7	6.6	6.9	8.0	8.8	8.4	5.2
Peak energy period (T_p) (s)	8.9	10.5	11.2	11.4	12.4	13.0	12.6	10.0
100-year								
Significant Wave Height (H_s) (m)	4.4	6	7	7.3	8.5	9.3	8.8	5.5
Peak energy period (T_p) (s)	9.2	10.7	11.6	11.8	12.7	13.3	13.0	10.2
Relative wave energy	0.11	0.27	0.43	0.48	0.76	1.00	0.86	0.21

5.1.3 Nearshore Wave Climate

As waves approach the shore, they may be transformed by the processes of refraction, shoaling, diffraction, attenuation, reflection and breaking. Therefore, the nearshore wave climate in the study area has a different wave height and particularly wave direction compared to offshore. Typically, waves break in a water depth about equal to the wave height. Estimates by Short (2007) of average wave heights along the study area are given in **Section 2.1**.

Patterson Britton & Partners (1998) estimated the nearshore wave climate for Broken Bay beaches based on wave analyses undertaken in the assessment of the Broken Bay Marine Aggregate Project (PWD 1990). This involved the transfer of limited recorded offshore wave data to an inshore wave recording instrument, although it should be noted that the wave record analysed did not contain any severe storms and therefore may not accurately reflect wave processes during storm conditions[13]. For Pearl and Patonga beaches, wave coefficients for storm conditions were estimated based on wave runup measurements taken during the 1986 storm event.

A refraction/diffraction analysis of MacMasters Beach was undertaken by PWD (1994) using the Refraction-Diffraction (REFDIF) numerical model, while a refraction analysis of Avoca, Terrigal and Wamberal beaches was undertaken by Lawson and Treloar (1984). Refraction/diffraction analysis was not undertaken by PWD (1994) for Forresters Beach due to the complexity of the nearshore bathymetry. However, given the exposure of this beach to all directions of wave attack, they suggested that a relative wave coefficient of around 1.0 should be used to formulate the design wave conditions.

[13] Major storms are generally characterised by elevated water levels and longer wave periods, which influence wave refraction and shoaling processes, which are particularly important for relatively shallow regions such as Broken Bay.

Nearshore wave coefficients previously determined for the beaches in the Gosford LGA, as reported in Patterson Britton & Partners (1998), PWD (1994) and Lawson and Treloar (1984), are summarised in **Table 4**.

Table 4: Previously determined wave coefficients for Gosford LGA beaches

Beach	Location	Wave Coefficient
Patonga	Southern	0.3
	Middle	0.18
	Northern	0.18
Pearl	Southern	0.16
	Middle	0.34
	Northern	0.6
Ocean-Umina	Entire	0.6
Putty	Southern	0.8
	Northern	0.7
MacMasters-Copacabana	Southern	0.81
	Middle	0.96
	Northern	1.02
Avoca	Southern	0.82
	Middle[14]	1.09
	Northern	1.12
Terrigal-Wamberal	Southern	0.35
	Middle[15]	0.76
	Northern[16]	0.94
Forresters	Entire	1.0

To provide a more rigorous analysis of nearshore wave climate in the study area, wave transformation modelling has been undertaken using the SWAN program. SWAN is a spectral, phase-averaging numerical model used for estimating wave parameters in coastal areas for given wind, bottom and current conditions. A detailed description of the model system, the model setup, validation of the model, and model results, is provided in **Appendix J**.

Nearshore wave coefficients determined for the study area from the SWAN model are provided in Table 5, at output locations generally corresponding to those listed in Table 4 (to enable comparison between both sets of results). These results are the maximum wave coefficients calculated for all

[14] Location corresponds to a point near the entrance to Avoca Lagoon.
[15] Location corresponds to a point just north of the entrance to Terrigal Lagoon.
[16] Location corresponds to a point just south of the entrance to Wamberal Lagoon.

directions simulated, and were determined at a water depth of 6.5m AHD. The offshore wave directions corresponding to each coefficient are also provided to give an indication of the wave directions likely to have the most direct impact on each beach. It should also be noted that these results account for shoaling effects to enable comparison with the coefficients provided in Table 4.

Table 5: Peak wave coefficients and corresponding offshore wave directions for Gosford LGA beaches determined from wave transformation modelling

Beach	Location	Peak Wave Coefficient Table 4	Peak Wave Coefficient This Study	Corresponding offshore wave direction
Patonga	Southern	0.30	0.28	S
	Middle	0.18	0.28	S
	Northern	0.18	0.28	S
Pearl	Southern	0.16	0.43	S
	Middle	0.34	0.63	S
	Northern	0.60	0.65	SE
Umina	Southern	0.60	0.43	SE
	Middle	0.60	0.54	SE
	Northern	0.60	0.44	SSE
Ocean		0.60	0.44	SSE
Putty	Southern	0.80	0.88	SE
	Northern	0.70	0.77	S
MacMasters-Copacabana	Southern	0.81	0.85	S
	Middle	0.96	0.90	SE
	Northern	1.02	0.92	E
Avoca	Southern	0.82	0.82	E
	Middle	1.09	0.90	S
	Northern	1.12	0.93	S
Terrigal-Wamberal	Southern	0.35	0.70	E
	Middle	0.76	0.87	S
	Northern	0.94	0.89	S
Forresters	Southern	1.00	1.02	SE
	Northern	1.00	0.96	ESE

Plots of the alongshore variation in 100 year ARI H_s and H^2T (a proxy for wave energy in shallow water) at 6.5m depth for all directions simulated are provided in **Figure 32** and **Figure 33** respectively. The maximum envelope of100 year ARI H_s and H^2T based on all simulated directions is shown in **Figure 34** and **Figure 35** respectively. From these plots, it is evident that:

- offshore waves from the SSE and SE generally produce the largest inshore wave heights in the study area;
- highest wave heights in the study area were determined offshore at Forresters Beach, which is likely related to wave focussing on the rocky reef;
- at Terrigal-Wamberal, Avoca and MacMasters beaches, there is a reduction in wave heights and wave energy moving south, while the potential effect of offshore waves from the ESE and E becomes more pronounced moving south along these beaches;
- the largest inshore wave heights in Broken Bay occur at Pearl Beach, while the lowest inshore wave heights occur at Patonga Beach;
- while it would be expected that Ocean Beach would receive higher wave heights compared to Umina Beach given its increased exposure to wave directions with a southerly component, the lower wave heights and energy determined at Ocean Beach can be attributed to increased wave refraction in shallower waters associated with the entrance shoal; and
- the sudden increase in wave heights at the southern end of Pearl Beach is likely related to wave focussing at Green Point.

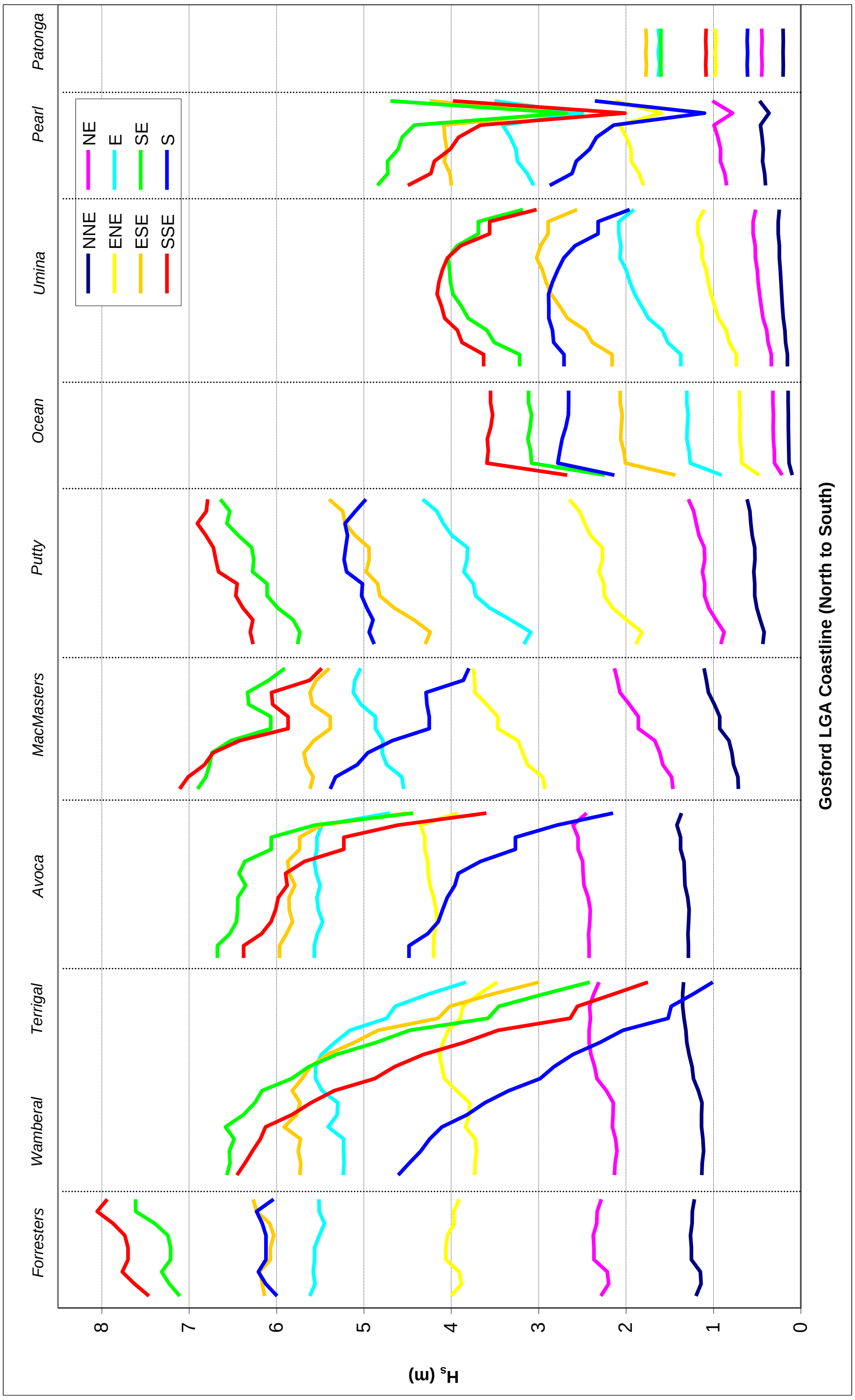

Figure 32: Alongshore variation in 100 year ARI Hs along Gosford LGA coastline

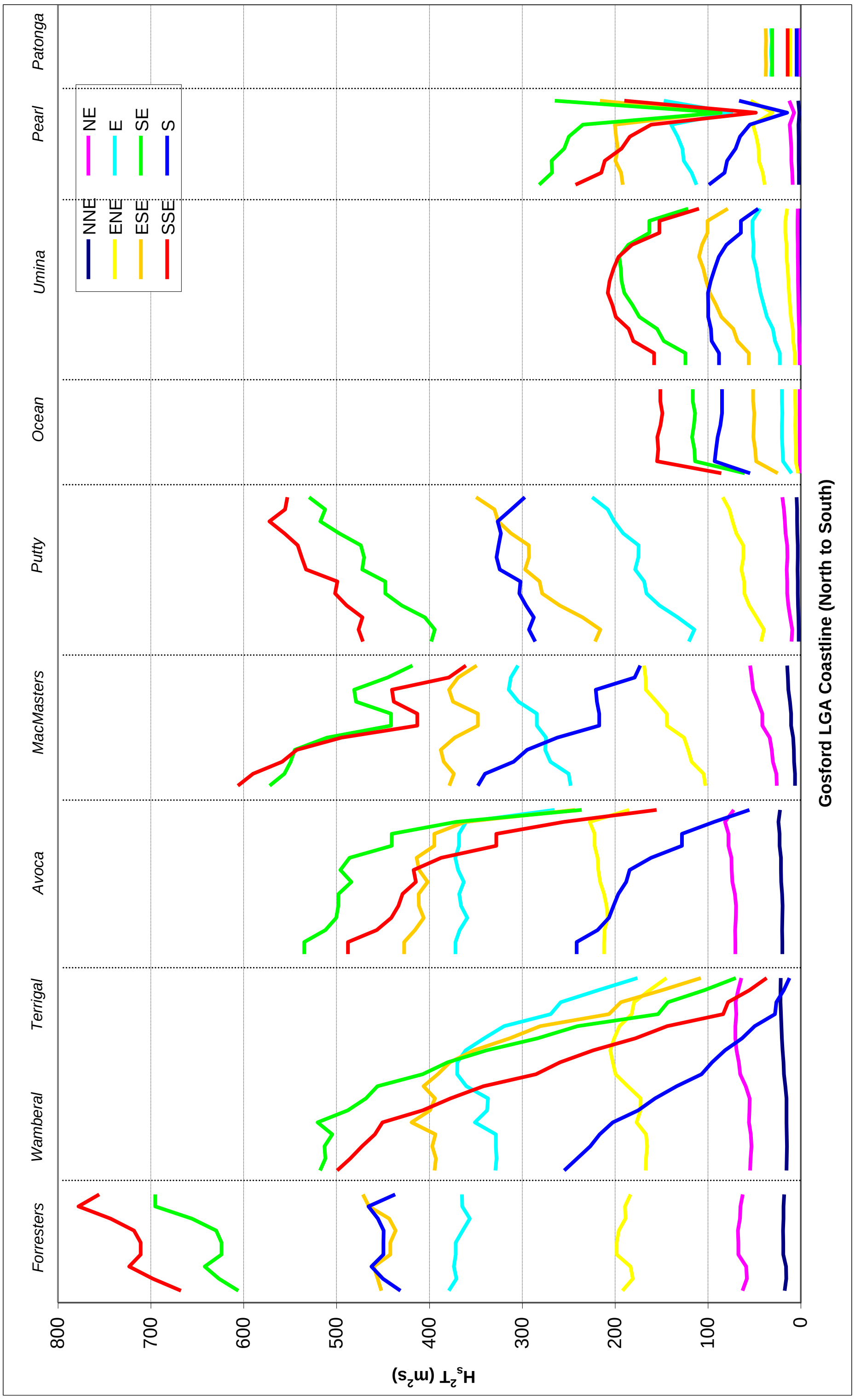

Figure 33: Alongshore variation in 100 year ARI Hs^2T along Gosford LGA coastline

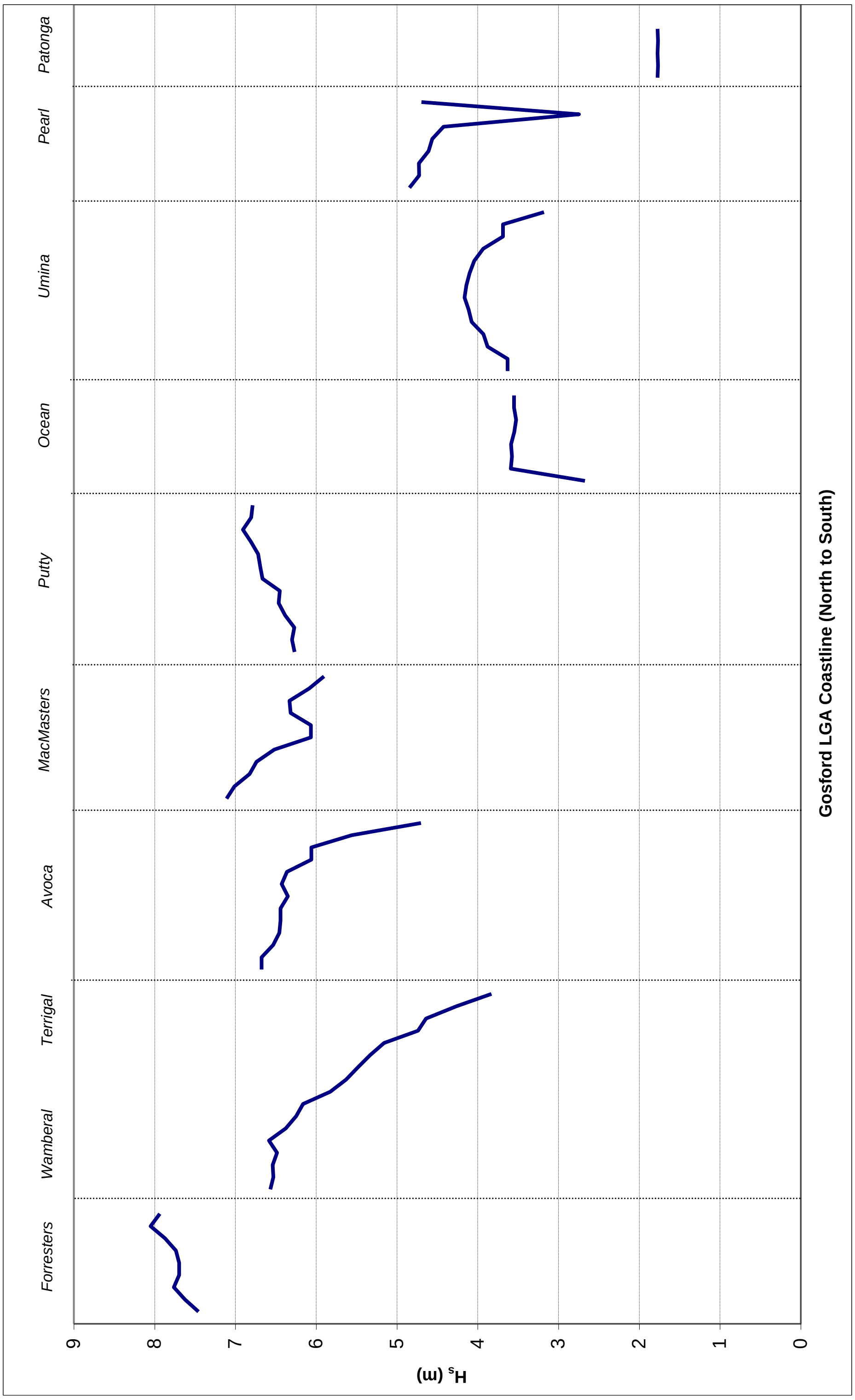

Figure 34: Alongshore variation in maximum 100 year ARI Hs along Gosford LGA coastline

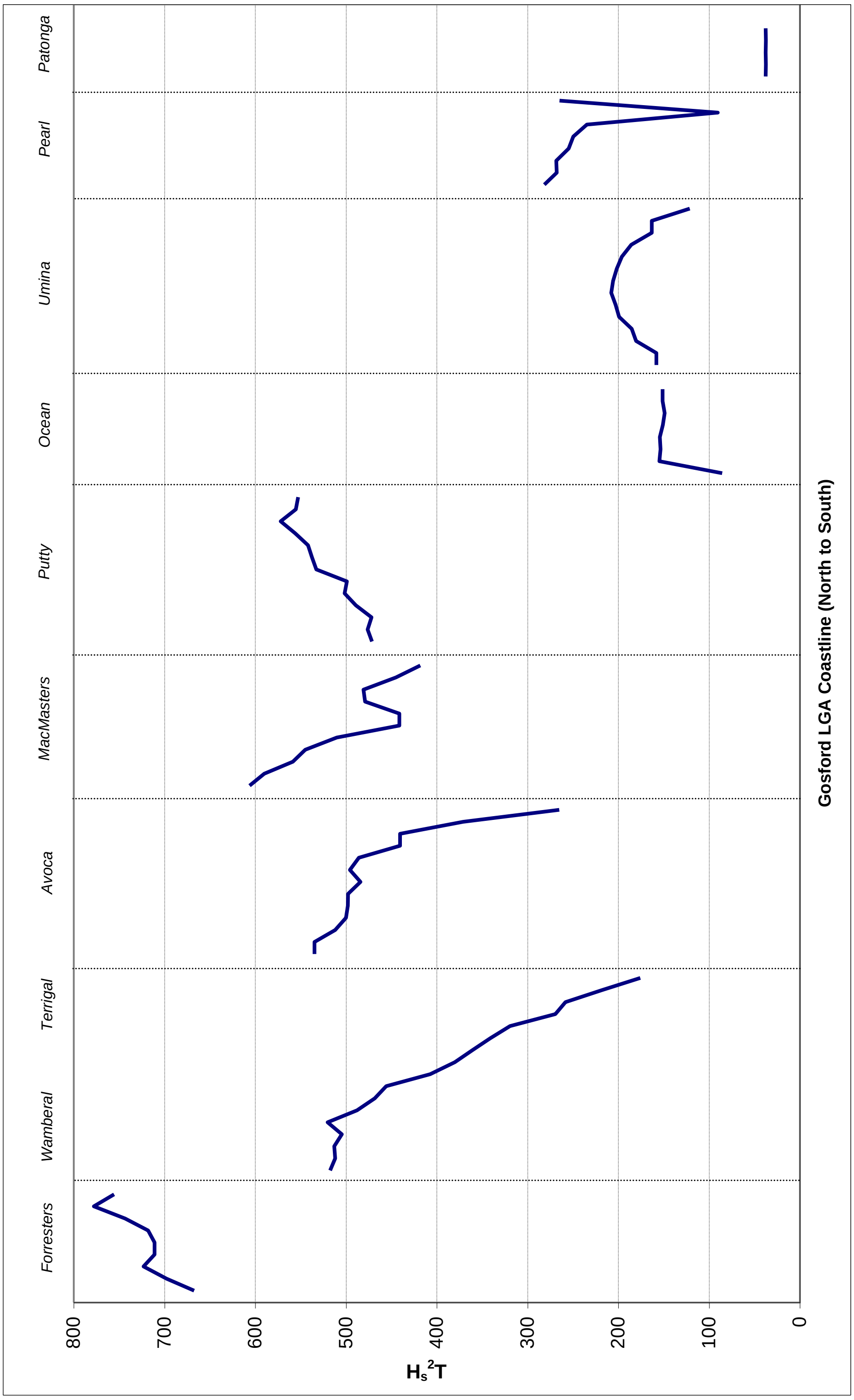

Figure 35: Alongshore variation in maximum in 100 year ARI H_s^2T along Gosford LGA coastline

5.2 Elevated Water Levels

The potential factors which contribute to elevated still water levels on the NSW coast comprise

- astronomical tide;
- storm surge (barometric setup and wind setup); and
- wave setup (caused by breaking waves).

Individual waves also cause temporary water level increases above the still water level due to the process of wave runup or uprush (see Section 5.3). Note that sea level is also predicted to rise due to climate change (the Greenhouse Effect). This is discussed further in **Section 5.7.2**.

In NSW, open coast still water levels (within the wave breaking zone) can increase by up to about 2.1m above normal levels during storms due to storm surge and wave setup, with components approximately as large as follows:

- storm surge of 0.6m (barometric setup of up to 0.3m to 0.4m and wind setup of up to 0.2m to 0.3m); and
- wave setup of up to 1.5m (typically about 10-15% of the deepwater significant wave height).

This increase in water level is superimposed on the astronomical tide, which typically varies between about –1m AHD (approximately equivalent to Indian Springs Low Water or Lowest Astronomical Tide, LAT) and 1m AHD (approximately equivalent to Highest Astronomical Tide, HAT) along the NSW coast, with 0m AHD close to mean sea level. On the NSW coast, Mean High Water Springs is about 0.6m AHD, Mean High Water is about 0.5m AHD, and Mean High Water Neaps is about 0.4m AHD. If a severe storm continued for a day, it would be expected that two high tides would occur during this time. Ignoring wave effects, the highest absolute water level that might be experienced in a storm would be when the maximum storm surge occurred at the same time as the HAT.

Water levels have been recorded at Fort Denison in Sydney Harbour for over 100 years, and are representative of NSW open coast water levels near Sydney (in the absence of waves). The data from 1914 onwards is considered to be reliable. Based on a joint probability analysis of tide and storm surge (assumed as independently occurring events), for the May 1914 to December 1991 data set, MHL (1992) predicted that the 100 year, 50 year and 20 year ARI water levels at Fort Denison were 1.49m, 1.46m and 1.41m AHD respectively. The highest recorded water level at Fort Denison was 1.48m AHD in May 1974. These levels are representative of astronomical tide and storm surge, but exclude wave setup.

Assuming extreme water levels in Sydney were representative of conditions between Patonga (within Broken Bay) in the south to Forresters Beach in the north, a 100 year ARI water level (including astronomical tide and storm surge) of 1.5m AHD can be adopted.

Wave setup can be expected to vary along the study area depending on wave exposure. Assuming wave setup as being equal to 15% of the 100 year ARI significant wave height (see Figure 34), wave setup values and total design still water levels can be estimated as shown in **Table 6**. Note that these design levels do not include climate change considerations which are discussed in **Section 5.7.2**.

Table 6: Approximate present day 100 year ARI wave setup and total design still water levels for beaches in the Gosford LGA

Beach	100 year ARI H_s (m)	100 year ARI wave setup (m)	100 year ARI total design still water level (m)
Patonga	1.8	0.3	1.8
Pearl	4.8	0.7	2.2
Umina	4.2	0.6	2.1
Ocean	3.6	0.5	2.0
Putty (south)	6.8	1.0	2.5
Putty (north)	6.3	1.0	2.5
MacMasters	6.0	0.9	2.4
Copacabana	7.1	1.1	2.6
Avoca	5.0	0.8	2.3
North Avoca	6.7	1.0	2.5
Terrigal	4.0	0.6	2.1
Wamberal	6.6	1.0	2.5
Forresters	8.0	1.2	2.7

5.3 Wave Runup

5.3.1 Measured Wave Runup

Wave runup is site specific, but typically reaches a maximum level of about 7m AHD on the open NSW coast at present. The height of wave runup on beaches depends on many factors, including:

- wave height and period;
- the slope, shape and permeability of the beach;
- the roughness of the foreshore area; and
- wave regularity.

Wave runup can be difficult to predict accurately due to the many factors involved. Anecdotal evidence and the surveying of debris lines following a storm event usually provide the best information on wave runup levels.

Overtopping of the dune crest at the southern end of MacMasters Beach occurred during 1974 when the lower part of MacMasters Surf Club was inundated. Parts of the carpark and recreational park further up the beach were also inundated during the 1974 storm (PWD 1994). Further, a wave runup level of around 5m AHD was measured at the southern end of Avoca Beach during the 1974 storm (CLT 2007).

Extensive wave runup was also observed to occur during the August 1986 storm event, as shown in **Figure 36** for Avoca Beach, which shows inundation to be most pronounced at the location of a stormwater outlet.

Figure 36: Wave Runup at Avoca Beach during August 1986 Storm
(Photo Courtesy Gosford City Council)

Following the August 1986 storm, Council surveyed the extent of runup on a number of beaches in the Broken Bay area (Patterson Britton & Partners 1998). A photograph showing a survey at Killcare SLSC in September 1985 is provided in **Figure 37**.

Runup levels determined following the 1986 storm event are presented in **Table 7** and were generated by the following storm characteristics:

- deepwater offshore significant wave height 7.5m
- peak ocean still water level (0.22m above predicted levels) 1.06m AHD
- wave period 9 to 13 s

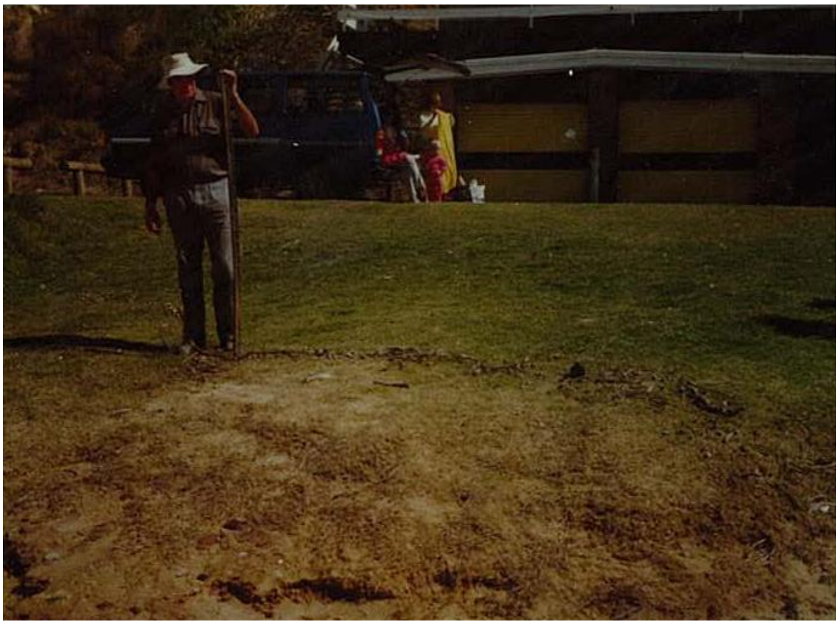

Figure 37: Measurement of debris line 1m below floor level at Killcare SLSC, September 1985
(photo courtesy Gosford City Council)

Table 7: Measured wave runup levels at Broken Bay beaches during August 1986 storm (after Patterson Britton & Partners 1998)

Beach	Location	Wave Runup Level (m AHD)
Patonga	Southern end	3.8
	Middle	3.1 to 3.2
Pearl	Southern end	3.8
	Middle	5.1
	Northern	6.4
Ocean-Umina	No data available	-
Putty	Southern end – Surf Club	5

The May 1997 storm was characterised by higher offshore wave heights (H_s = 8.5m) but occurred over a shorter duration than the August 1986 storm. Overtopping of the dunes occurred at Patonga, the northern end of Ocean Beach, the southern end of Umina Beach at Berrima Crescent and at the Killcare Surf Club. No measurements were taken of debris line levels. At Berrima Crescent, the level of the road as determined from the photogrammetric beach profiles varies from about 3.5m to 4.5m AHD.

It was noted in PWD (1994) that inundation of Avoca Lagoon has occurred several times over the previous 20 years, indicating dune overtopping for events significantly below the design storm. CLT

(2007) also notes that general observations have been made of infrequent flooding at the car park of Avoca Beach SLSC since 1993.

A Storm Wave Inundation Study was undertaken by Cardno Lawson Treloar (CLT 2007) to examine the inundation hazard for Avoca Beach. This study utilised modelling techniques calibrated by anecdotal evidence to define high and low level oceanic inundation hazard zones for Avoca Beach and the lagoon entrance area. This study identified the following:

- the risk of oceanic inundation was greatest near the carpark west of the Avoca Beach SLSC and the car park near Ficus Avenue. Wave overtopping at those sites may typically occur about once every 10 years.
- properties along the southern and northern entrance channel are only affected by oceanic inundation when the entrance is open;

It should be noted that this study adopted a mean sea level rise (SLR) of 0.3m by 2105 whereas the current SLR prediction for 2100 is 0.9m (as discussed further in **Section 5.7.2**).

5.3.2 Calculated Wave Runup

A comprehensive assessment of wave runup was undertaken for each beach in the Gosford LGA, while an assessment of potential inundation levels near the entrances to Cockrone Lagoon, Avoca Lake, Terrigal Lagoon and Wamberal Lagoon was also completed. The results of these assessments are provided in **Appendix K**, which also describes the methodology employed. The design wave runup levels (exceeded by 2% of waves) are summarised in **Table 8**.

Table 8: Design Wave Runup Levels (m AHD)

Beach	Location	Adopted 2011 wave runup level (m AHD)
Patonga	Southwest	2.5
	Northeast	2.0
Pearl	South	3.0
	Central	6.0
	North	7.0
Ocean-Umina	South	6.0
	Central	5.0
	North	4.0
Putty	Southwest	7.0
	Northeast	3.0
MacMasters-Copacabana	South	6.0
	Central	7.0

Beach	Location	Adopted 2011 wave runup level (m AHD)
	North	7.0
Avoca	South	4.0
	Central	5.0
	Lagoon entrance	2.5
North Avoca	Central	5.0
	North	7.5
Terrigal-Wamberal	South	4.0
	Central	6.0
	North	7.0
Forresters		8.0

Runup levels outlined above would only be realised if the foreshore was at this runup height or higher. In reality, any waves that overtopped dunes or creek banks in the study area would fold over the foreshore crest and travel as a sheet flow at shallow depth, spreading out and infiltrating over landward areas. Accordingly a significant reduction in the velocity and depth of runup would be expected within about 10m from the foreshore crest.

In the long term, as a beach receded, it could be postulated that the present dunal barrier would disappear, with the new shoreline taking on the existing topography landward of the present dune. This is considered to be unlikely from an understanding of the morphological response of beaches. The existing dune crest levels are a complex response to a variety of factors including beach sand characteristics, exposure to wind and wave action, and local topographic controls, all of which are likely to be relatively constant irrespective of the shoreline position in the long term; i.e., it is considered more likely that the existing dune profile would 'roll back'. However, as most of the coastal strip is fully developed and many private dwellings are within the active zone, it is unlikely that the dune crest will be allowed to roll back naturally in response to coastal processes.

Further discussion on the implications of wave runup on coastal inundation is provided in **Section 6.5**.

5.4 Coastal Storms

5.4.1 General

The NSW coastline is subject to intense tropical and non-tropical storms at irregular intervals. The drop in atmospheric pressure and the winds and waves that accompany these storms can cause the ocean to rise above its normal level (see **Section 5.2**). If this occurs concurrently with high astronomical tides, there is the potential for:

- coastal erosion (in particular as the storm waves dissipate energy closer to the shoreline with the increased water levels); and/or
- overwash into low-lying coastal areas (PWD 1985).

PWD (1985, 1986) categorised coastal storms to indicate the potential of a storm to generate abnormal water levels along the NSW coastline. The categories were discretised on the basis of offshore significant wave heights, as shown in **Table 9**.

Table 9: Categorisation of coastal storms in NSW by PWD (1985, 1986)

Category	Offshore significant wave height (Hs), m
X	$H_s \geq 6$
A	$5 \leq H_s < 6$
B	$3.5 \leq H_s < 5$
C	$2.5 \leq H_s < 3.5$

Category X and A storms were those expected to lead to coastal erosion and damage to coastal facilities. According to PWD (1985, 1986), Category X storms were characterised by damage to coastal installations, severe erosion, and serious disruption to shipping. Category A storms were characterised by erosion or other damage to coastal installations and disruption to shipping.

In PWD (1985a), all Category X, A, B and C storms that were predicted to have occurred between 1880 and May 1980 were listed[17], along with a description of the storm generating mechanism and characteristics, and wave heights and periods (for selected storms). Estimates were given for each of four coastal sectors in NSW, namely North, Mid-North, Central and South. The Central sector covered the study area, with the sector extending from Sugarloaf Point (near Seal Rocks), to just south of Jervis Bay.

Similarly, in PWD (1986b), all Category X, A, B and C storms that were predicted to have occurred between May 1980 and December 1985 were listed.

Callaghan and Helman (2008) have also prepared a description of storms that have affected the east coast of Australia from 1770 to 2008. They noted that over the last 30 years there had been a relatively low number of storms. They also found that severe storms over this 239 year period tended to occur when the ENSO (El Niño Southern Oscillation Index) was positive (that is, in La Niña phases) and the IPO (Interdecadel Pacific Oscillation) was negative.

5.4.2 Storm Types

PWD (1985a) recognised six different major storm types which impacted on the NSW coast, namely:

[17] However, the only reliable data for statistical analysis was from 1920 to 1944 and 1957 to 1980 due to a bias in reporting between 1944 and 1957.

- tropical cyclones;
- easterly trough lows;
- inland trough lows;
- continental lows;
- southern secondary lows; and
- anti-cyclonic intensification.

Typical synoptic patterns for tropical cyclones, easterly trough lows, inland trough/continental lows, and southern secondary lows are shown in **Figure 38**.

Based on PWD (1985, 1986), the spatial variation in occurrence of these six storm types along the NSW coast is shown in **Figure 39**, with monthly variations in storm occurrence on the Central Coast shown in **Figure 40**.

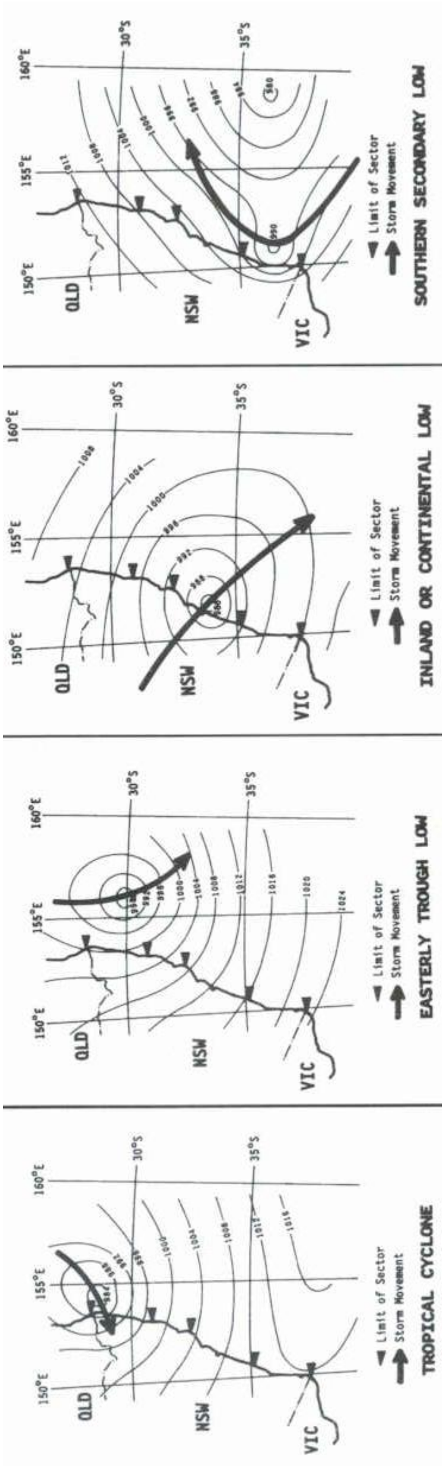

Figure 38: Typical synoptic patterns for tropical cyclones, easterly trough lows, inland trough / continental lows, and southern secondary lows (after NSW Government 1990)

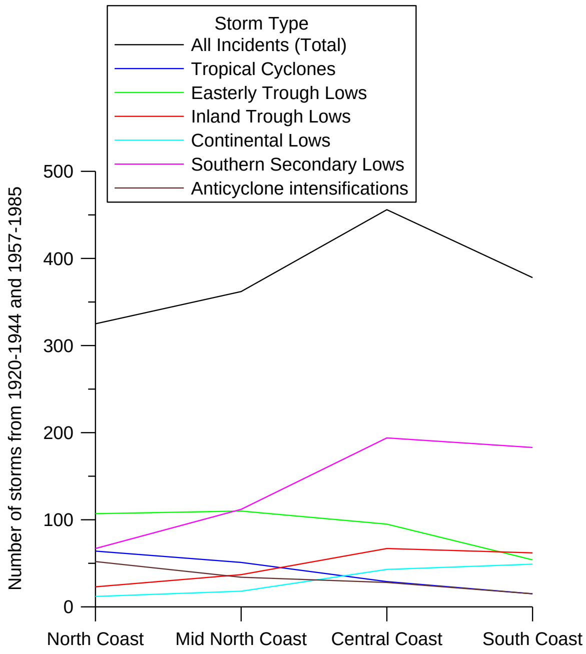

Figure 39: Variation in occurrence of different major storm types (significant wave height exceeding 2.5m) along the NSW coast (based on PWD 1985, 1986)

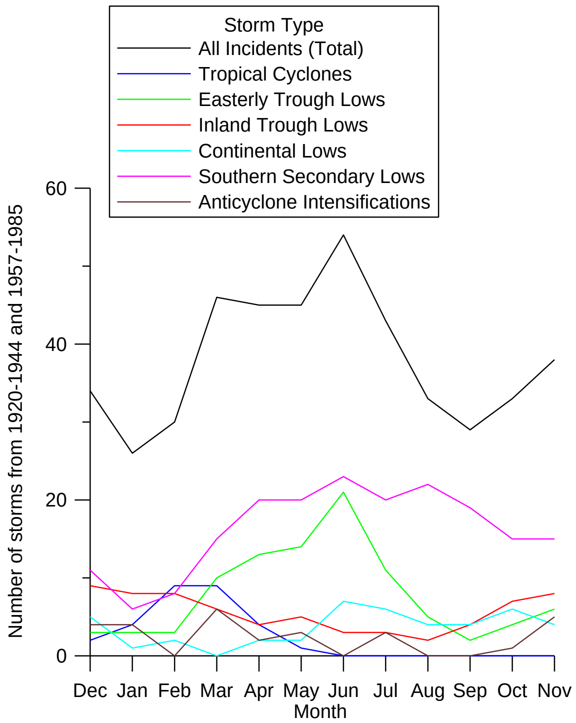

Figure 40: Monthly variation in storm occurrence (significant wave height exceeding 2.5m), Central Coast of NSW (based on PWD 1985, 1986)

It is evident that, on average:

- the Central Coast (incorporating the study area) and South Coast have more storms than areas further north in NSW;
- southern secondary lows and easterly trough lows are the dominant storm types on the Central Coast[18]; and,
- most storms on the Central Coast occur in Autumn and Winter, in particular due to the prevalence of southern secondary lows and easterly trough lows during these seasons.

This seasonal variation can be confirmed by analysis of the Sydney directional Waverider buoy data collected from 1992 to 2007. The relative wave energy[19], which has a good correlation with beach erosion, for storms exceeding 3m for each month is shown in **Figure 41**.

[18] Note that the NSW Government (1990) presented the information in Figure 40 based only on data to 1980. By using the more accurate data set from 1980-1985, a large number of southern secondary lows and other Category B and C events were picked up for the investigation reported herein, that were not included in the PWD (1985b) study.

[19] A relative wave energy was defined relative to a value of 1.0 for an average month. Therefore, values exceeding 1.0 indicate months with greater wave energy than the monthly average. Conversely, values less than 1.0 indicate months with less wave energy than the monthly average.

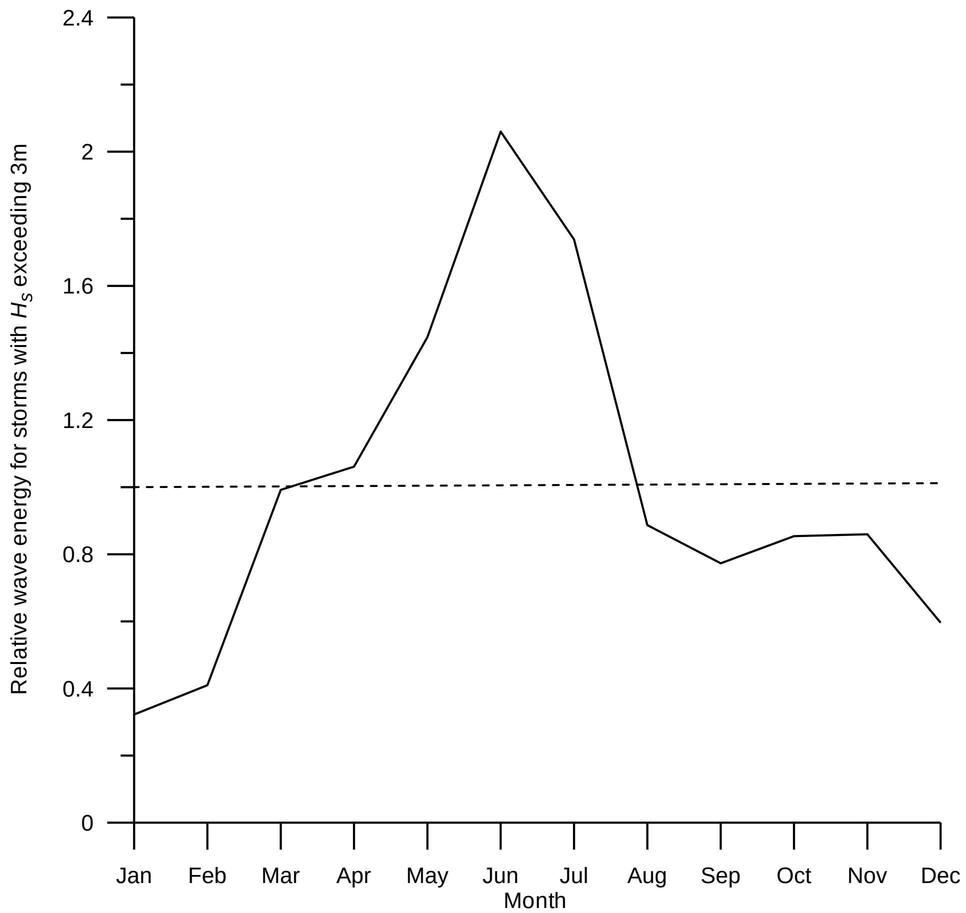

Figure 41: Relative monthly wave energy for storms offshore of Sydney, based on data collected from 1992 to 2007 (Average = 1.0 – dashed abscissa).

It is evident that the Autumn and Winter seasons have been the most stormy, with June, July and May having been the most stormy months. The Winter period has been more than three times more stormy than Summer, with January and February being the least stormy months. Autumn and Winter combined has been more than twice as stormy as Spring and Summer combined.

5.4.3 Storm History

As noted in Section 5.4.1, PWD (1985a, 1986a) listed all Category X, A, B and C storms that were predicted to have occurred between 1880 and 1985. A listing of Category X storms from these references (from the Central Coast region) is provided in **Table 10**. This includes the estimated significant wave height (H_s) and significant wave period (T_s) calculated by hindcasting for some storms.

Table 10: Occurrences of Category X storms on the Central Coast from January 1880 to December 1985 (based on PWD 1985, 1986)

Date	Storm Type	H_s (m)	T_s (s)
23-24 September 1892	Easterly trough low		
12-16 June 1896	Easterly trough low		
5-19 August 1899	Southern secondary low		
1-5 August 1908	Inland trough low		
12-14 January 1911	Tropical cyclone		
14-17 July 1912	Inland trough low		
13-15 May 1913	Inland trough low		
18-20 September 1917	Easterly trough low		
15-21 May 1919	Easterly trough low / anticyclone intensification		
8-13 December 1920	Easterly trough low		
22-24 July 1921	Easterly trough low	7.2	10.8
25-28 June 1923	Continental low	7.2	10.8
25-26 March 1926	Inland trough low	7.2	10.7
16-20 May 1926	Easterly trough low / anticyclone intensification	6.6	10.3
15-19 April 1927	Easterly trough low	8.4	11.6
13-14 June 1928	Continental low	8.4	11.6
6-8 July 1931	Southern secondary low	6.9	10.5
7-8 July 1932	Southern secondary low	6.4	10.1
2-3 February 1934	Tropical cyclone	7.1	10.6
18-20 June 1935	Southern secondary low	7.4	11.0
19-23 June 1937	Easterly trough low	8.0	11.3
19-23 June 1937	Easterly trough low	8.0	11.3
28-30 September 1940	Southern secondary low	6.4	10.1
12-15 October 1942	Easterly trough low	6.4	10.1
10-13 June 1945	Easterly trough low		
14-15 June 1952	Continental low	7.2	10.8
2-5 January 1954	Southern secondary low		
19-22 February 1954	Tropical cyclone	7.4	10.9
9-10 June 1956	Continental low		
18-23 February 1957	Tropical cyclone		
22-24 August 1957	Southern secondary low		
9-11 March 1958	Easterly trough low		

Date	Storm Type	H_s (m)	T_s (s)
29 June - 1 July 1958	Continental low		
20-21 July 1959	Continental low		
4-5 October 1959	Continental low		
20-21 May 1966	Southern secondary low		
5-6 September 1967	Continental low	7.7	11.1
13-15 May 1968	Southern secondary low	7.9	11.2
25-26 May 1974	Southern secondary low	8.8	11.8
18-20 March 1978	Easterly trough low	7.7	11.1
31 May – 2 June 1978	Inland trough low	6.9	10.5
7-9 July 1983	Continental low	6.9	14.5
5-8 November 1984	Inland trough low	6.0	12.5

Prior to the installation of the Sydney Waverider buoy, the closest MHL Waverider buoy to Sydney was at Port Kembla. A single Category X storm was measured at Port Kembla from January 1986 to June 1987, the period after completion of the PWD (1986a) analysis and prior to the Sydney Waverider buoy installation. This occurred on 5-11 August 1986, with a peak H_s of 6.8m and mean T_s of 10.4s.

Category X storms that have been measured at the Sydney non-directional Waverider buoy (generally from July 1987 to March 1992, but which continued recording until October 2000) and Sydney directional Waverider buoy (from March 1992 to December 2007) are listed in Table 11. The "storm rank in terms of energy" is ranked from highest (most energy) to lowest based on the storm energy (which is a proportional to wave height squared, wave period squared, and storm duration).

Table 11: Category X storms measured at the Sydney Waverider buoys, 1987-2007

Date	Peak H_s (m)	Mean T_s (s)	Direction	Storm energy (MJ/m)	Storm rank in terms of energy
11-13 November 1987	6.8	9.0	SSE	36,697	25
1-3 August 1990	7.2	9.3	SE	27,038	48
24-27 August 1990	6.3	10.3	SSE	42,769	13
25-27 September 1995	6.3	9.9	SE	40,616	17[20]
30 August – 1 September 1996	6.1	10.0	SE	39,803	18[21]
9-12 May 1997	8.4	10.3	SSE	74,063	1

[20] Full storm not recorded at directional Waverider (Category A), so non-directional values used, although note that full storm was not recorded at the latter either.
[21] Full storm not recorded at directional Waverider, so non-directional values used.

Date	Peak H_s (m)	Mean T_s (s)	Direction	Storm energy (MJ/m)	Storm rank in terms of energy
7-10 March 1998	6.0	10.0	SSE	29,319	45[20]
21-25 April 1999	6.2	9.8	E	57,473	3
14-17 July 1999	6.0	10.3	ESE	49,654	8
30 June – 2 July 2000	6.1	9.9	S	44,978	11
27-29 July 2001	7.0	11.0	S	32,624	32[22]
18-22 November 2001	6.2	9.7	SE	64,787	2[22]
29 June – 1 July 2002	6.2	11.1	SSE	48,470	10
18-20 July 2004	6.7	9.8	SSE	34,965	29[22]
22-24 March 2005	6.6	9.2	SE	30,450	39
10-12 July 2005	6.2	10.0	SSE	13,776	113[22]
2-4 June 2006	6.5	10.1	S	41,387	15
11-12 June 2006	6.2	10.6	S	25,466	53
7-10 June 2007	6.9	9.8	SE	57,841	4[22]
16-20 June 2007	6.0	9.2	SE	40,626	16[22]
18-21 July 2007	6.5	10.8	SSE	38,664	22

It can be noted that of the largest 10 storms (in terms of storm energy) from 1987 to 2007, only 6 were Category X, while 3 were Category A (6-11 August 1988, 14-17 August 2002, 1-5 March 1995), and 1 was Category B (4-10 March 2001).

It is evident that over the 1880 to 2007 period there were 65 Category X storms, that is 1 Category X event every 2 years (on average). However, the time period between storms has not been uniform. For example, there were no Category X storms from 1880-1891, 1900-1907, 1946-1951, 1960-1965, 1969-1973 and 1979-1982. Also, there were 3 Category X storms in 2007, and two in 1926, 1937, 1954, 1957, 1958, 1959, 1978, 1990, 1999, 2001, 2005 and 2006.

5.4.4 Analysis of Key Storms Affecting Study Area

As noted in Section 2.3, the key storms to affect the study area occurred in:

- May-June 1974;
- May-June 1978;
- September 1985;
- August 1986;

[22] Full storm was not recorded.

- September 1995; and
- May 1997.

The storms in June-July 2007 also affected the study area.

The most significant coastal storm that has been recorded to have impacted on the Central Coast of NSW is the Category X May 1974 storm (see Table 10), which was followed by two smaller storms in June 1974 (Category C and Category A). The May 1974 storm was particularly severe as it was accompanied by the highest ever recorded water level along the NSW coast[23]. Coastal erosion depends on far more than just wave height, with factors such as storm duration, water level, wave direction and storm history being important[24].

The 1978 storms were also particularly severe, resulting in the collapse of two homes at Wamberal Beach (see Section 2.3.3). As noted in Section 2.3.1, the passage of successive storms within a relatively short time period is more likely to result in erosion of the foredune than individual storm events (PWD 1985). It is therefore important to note that, during 1978, Category X storms occurred on 18-20 March, 30 May to 2 June, and 15-16 June, while Category A storms occurred on 28-30 January, 20-21 May, and 23 August.

The September 1985 event was a Category A storm, while the August 1986 event was a Category X storm (measured at Port Kembla) with a peak H_s of 6.8m and mean T_s of 10.4s.

The September 1995 event was a Category A storm, while the Category X storm in May 1997 was associated with the highest wave energy for all storms recorded at the Sydney Waverider Buoy between 1987 and 2007 (see Table 11).

The June-July 2007 event comprised three Category X storms (see Table 10), the last of which was preceded by 5 Category B storms. Watson et al (2007) noted that these storms led to the second highest insurance payout and second largest emergency response operation in Australia's history (at that time), behind only the 1999 Sydney hailstorm. These storms were described as East Coast Low weather systems, and were particularly severe in the Newcastle area.

It is evident that damaging storms in the study area have generally been or been preceded by sequences of storms, often not particularly severe storms in isolation. A key factor in the erosiveness of a storm, besides the storm energy, is also the water level occurring during the storm.

Watson et al (2007) considered that the 7-10 June 2007 event had an ARI of about 4 to 10 years, as compared to the May 1974 event with a ARI of 20 to 70 years (based on the magnitude and duration of H_s). They also noted that peak water levels were about 0.5m higher in the 1974 storm. However, they did not attempt rigorous analysis of the ARI of the sequence of the June-July 2007 storms.

[23] Chapman et al (1982) also noted that the February to April 1974 period was erosive (there were 5 storms with H_s exceeding 2.5m during this period), causing a general lowering of beach profiles prior to the May to June storms, thus contributing to the severity of the latter events.

[24] In terms of wave height and duration at Sydney, the May 1974 storm was approximately a 20 year to 70 year ARI event, for storm durations between 1 and 24 hours (Lord and Kulmar, 2001). However, when the storm history and elevated water level is considered, the event can be considered to be of lower probability (greater severity).

It can be concluded that the study area has been subject to damaging coastal storms in the past, and can thus be expected to again be exposed to such storms at irregular intervals in the future. These storms are most likely to occur in Autumn and Winter, and are least likely to occur in Summer, but can generally occur at any time.

5.4.5 Storminess Indicator

Particular dates of aerial photography and photogrammetric data can be referenced against dates of coastal storms. This provides an approximate measure of the likely beach state (accreted, average or eroded) at the time of photography, which can assist in the interpretation of photogrammetry and other observations.

Shoreline erosion can be expected to correlate with wave energy, more so than wave height or wave period alone[25]. According to Airy theory, the total wave energy in one wavelength per unit crest width is given by (in deep water):

$$E = \frac{\rho g H^2 L_o}{8} \qquad (1)$$

where (in SI units) ρ is the water density (kg/m^3), g is the gravitational acceleration (m/s^2), H is the wave height (m) and L_o is the deepwater wavelength (m), given by:

$$L_o = \frac{gT^2}{2\pi} \qquad (2)$$

where T is the wave period. Therefore, E has units of kgms^{-2}, equivalent to Newtons (N) or Joules/metre (J/m). It can be seen that E is proportional to H^2T^2.

For the 1880 to 1985 period, H^2T^2 was determined based on PWD (1985b, 1986). A storminess indicator was determined for each year of record as the sum of H^2T^2 for the year divided by the average yearly H^2T^2 for the data set. Therefore, a storminess indicator value less than 1 is indicative of accreted beach states, a value of 1 is indicative of average beach states, and a value greater than 1 is indicative of eroded beach states[26].

The years of 1959, 1974, 1966 and 1967 were particularly stormy. Conversely, the 1940–1944, 1947–1950, 1961–1965 and 1969–1973 periods were particularly calm.

[25] This has been shown to be particularly true in the assessment of erosion caused by boat wakes, especially if significant rather than peak parameters are used (Patterson Britton & Partners, 1995a). It has also been applied in open coast studies such as at Bate Bay (Patterson Britton & Partners, 2001), and at sheltered beaches such as Fishermans Beach at Collaroy (Geomarine, 1991).

[26] Note that a storminess indicator value of 1 should not be considered as a threshold for erosion/accretion. The storminess indicator is intended to be a relative measure of likely beach states.

For the 1988 to 2007 period, the storminess indicator was calculated from available measured wave data from Sydney. The data included the duration that H_s exceeded 3m to 8m, in 0.5m increments. Therefore, duration could be considered as well as wave height and period. Inclusion of duration in the analysis would be expected to provide a better measure of the wave energy or power associated with a storm than using H^2T^2 alone.

In deep water, wave power (P) for an individual wave is given by:

$$P = \frac{E}{L_o} \frac{gT}{2\pi} \tag{3}$$

where all variables have been previously defined (refer to Equation 1). In the SI system, the units of P are kgms^{-2}/s, or N/s, or W/m (Watts per m wave crest width). Including duration as a multiplier, the units become Ws/m, or J/m, thus representing total storm energy per unit wave crest width (denoted as E_s)[27].

For the 1988 to 2007 period, E_s was determined for each storm. A storminess indicator was determined for each year of record as the sum of E_s for the year divided by the average yearly E_s for the data set. The storminess indicators determined for each of the calendar years from 1988 to 2007 inclusive are shown in **Figure 42**.

[27] The various durations and wave heights for each 0.5m increment range were summed as the median H_s of each range, multiplied by the duration that the wave height was in that range.

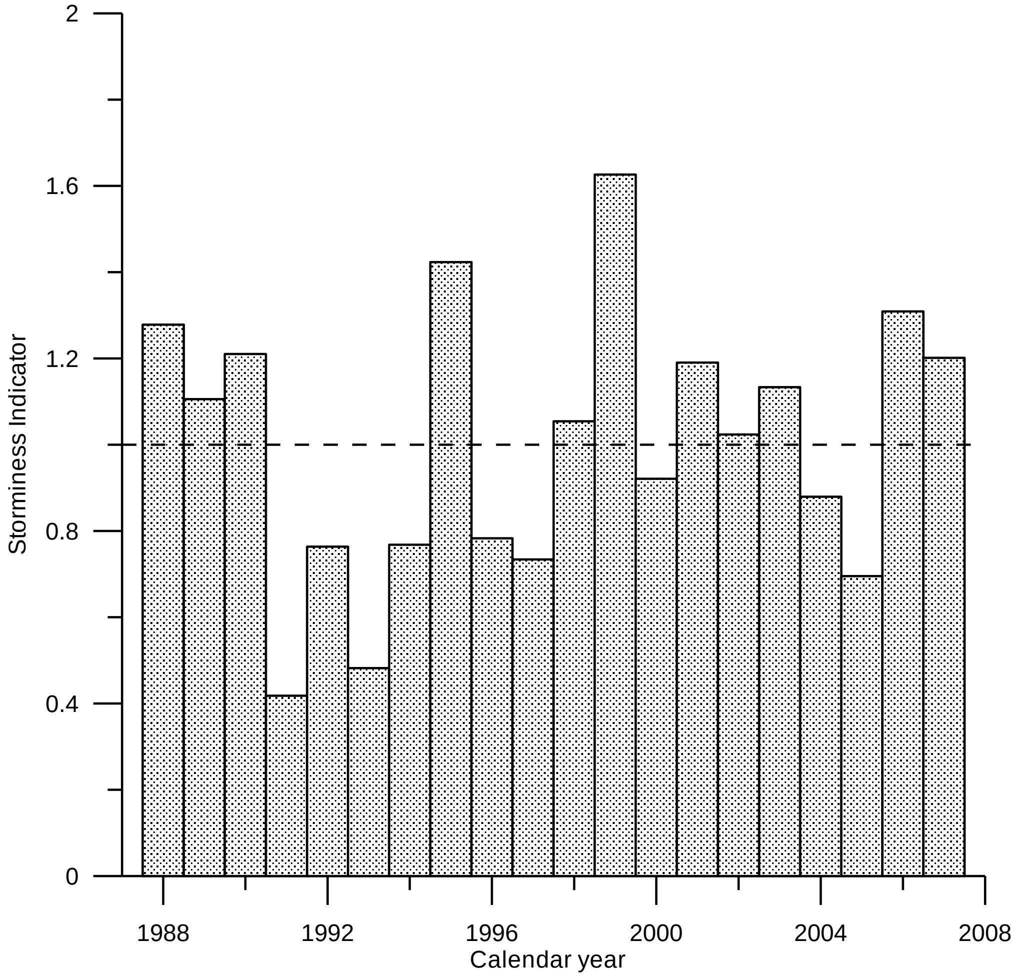

Figure 42: Yearly storminess indicators for 1988 to 2007, based on Sydney Waverider buoy data

It is evident that 1988-1990, 1995, 1999 and 2006 were notably stormy years within the 1988 to 2007 period. Conversely, the 1991-1994 and 1996-1997 periods were particularly calm.

Note that the storminess indicator is only an approximate measure of beach state, as water level is a very significant factor in defining the erosiveness of storms. Furthermore, for the 1940-1985 period, storm duration was not included, and the storms were only predicted and not measured. With an understanding of these limitations, the "storminess indicator" is still considered to be a reasonable measure of the likely beach state for each date of photography.

5.5 Sediment Transport

5.5.1 Preamble

In the region between where waves break and the shoreline, two processes can result in net sediment transport, namely longshore sediment movement and onshore / offshore (termed cross-shore) sediment movement. These transport processes are discussed in Section 5.5.2 and Section 5.5.3 respectively.

Net sediment transport can also occur due to movement of windblown sand, as discussed in Section 5.5.4. The various stormwater systems and lagoon entrances may also contribute sediment to or capture sediment from the beach system, as discussed in Section 5.5.5.

To assist in determining sediment movements, analysis of beach profile (photogrammetric) data has been undertaken, as described in Section 4.1.

5.5.2 Longshore Sediment Transport

Longshore sediment transport is associated with longshore currents. Longshore currents occur between where waves break and the shoreline, and are generated by (NSW Government 1990):

- waves breaking at an angle to the shoreline;
- feeder currents to rip cells; and,
- longshore variations in water level resulting from nearshore wave conditions and wind stress.

Longshore currents essentially move parallel to the shoreline. These currents cause movement of sediment along the shoreline, commonly referred to as littoral drift. Due to the variability in wave approach direction at beaches (and other wind and wave conditions), there may be times when the littoral drift is in one direction and at other times when it is in the opposite direction.

Based on analysis of historical beach profile data (see Section 5.6), it is unlikely that there has been extensive net longshore sediment transport at the open coast beaches along the Gosford LGA coastline in the last 60 or so years. PWD (1985b) could not identify any sources of sediment to the Avoca and Terrigal-Wamberal beach compartments, noting that the extensive offshore reef systems would likely divert any southerly or northerly alongshore drift supply further offshore. These reef systems would also prevent any onshore movement of sand from the inner Continental Shelf under low swell conditions. Similarly, the extensive rocky reef systems offshore of Forresters Beach and MacMasters Beach would limit longshore transport of sand from the south or north, while it should also be noted that MacMasters Beach is heavily embayed by prominent headlands to the north and south, which further reduces the likelihood of longshore sediment transport.

Sediment transport processes within Broken Bay are dominated by estuarine circulations, particularly in the vicinity of major creek and lagoon entrances. In particular, the wide ranging sand shoals fringing the entrance to Brisbane Water has a large influence on sand volume fluctuations at Ocean Beach. Further, however, as the wave energy entering Broken Bay is refracted and diffracted widely, the predominant swell characteristics comprise long low waves. This results in a net shoreward wave-induced bottom current that pushes sand ashore, which explains partly why the Woy Woy beach ridge barrier is the widest Holocene depositional feature in NSW. This process has been operating for some 10,000 years and geological dating and the recent photogrammetric data indicate that sediment accretion of this feature continues to the present day (see Appendix D).

Rip currents are strong currents which flow seaward from the shore. They comprise the return movement of water which is "piled up" on the shore by incoming waves and wind. The rip consists of three parts: the feeder currents flowing parallel to shore inside the breakers; the neck, where the feeder currents converge and flow through the breakers in a narrow band or "rip"; and the head where the current widens and slackens outside the breaker line.

As the "rip" is a locally deeper channel through the sand bars, larger waves can reach the shoreline opposite rip heads. Accordingly, it is common to distinguish the higher storm erosion demand which can occur at rip heads and the lower storm erosion demand which prevails away from rip locations.

While it is apparent from aerial photography that, typically, a rip forms adjacent to the headlands of the various beaches in the study area, there is no evidence that the rip locations are "fixed" elsewhere along the beaches. Consequently, for purposes of assessing the possibility of increased storm erosion demand at rip heads, it is necessary to assume that a rip could form at any location along the beach.

5.5.3 Onshore/Offshore Sediment Transport

Onshore/offshore (also known as cross-shore) sand movement is caused by natural variations in wave climate and water level. The offshore movement of sand is usually referred to as storm erosion. This onshore/offshore movement of sand results in short term fluctuations in the width of the beach profile.

During storms with relatively large waves, the beach is cut by storm waves with beach sand moving offshore to form bars in the surf zone. This process typically occurs over a period of hours to days. When extended periods of calmer waves occur, the material held in these bars migrates onshore to re-build the beach berm. Depending on the magnitude of the preceding storm, this beach building process can occur over a time scale of days to years.

Onshore/offshore sand movement can also be caused by wind, particularly manifested as landward sand drift into dune areas (see **Section 5.5.4** for further discussion on aeolian sand movement).

The amount of sand that can be removed from a beach during a storm event (or series of closely spaced storms), and transported offshore, is referred to as the "storm demand" (Chapman at al. 1982). Generally, this quantity is measured above 0m AHD (approximately mean sea level), and is expressed usually as a volume per metre length of beach (m^3/m). Knowledge of the storm demand for a beach allows the estimation of the amount of material required to be held in reserve for a storm to protect a given asset. It allows also the estimation of the degree to which a beach would be eroded, or cut back, in a storm for a given pre-storm beach profile.

The reason that the storm demand generally is measured above 0m AHD is a reflection of the manner in which the data to describe storm demand has been obtained. Storm demand estimates are derived typically from survey or photogrammetric techniques, where only that portion of the beach above mean sea level is either considered or is visible.

As discussed in Chapman et al. (1982), storm demand at any location, at any point in time, is dependent on a number of variables, including the:

- wave height and period as well as the duration of the storm;
- state of the beach before the storm;
- direction of the storm relative to the orientation of the beach[28];
- magnitude of the storm surge accompanying the event;
- amount of wave setup and runup on the beach during and immediately following the storm;
- tidal range at the time of the storm;
- state of the tide at the peak of the storm;
- presence of rip cells;
- presence and influence of local topography including adjacent headlands or coastal structures, or both, which can modify local wave and current conditions and the supply of sediment;
- existence and strength of longshore currents;
- sediment grain size of the beach and surf zone; and
- for embayed beaches, the prevalent stage of the beach rotational cycle due to climatic variability (i.e. Southern Oscillation Index) impacts (Chapman et al 1982 and DECCW 2010).

Chapman et al. (1982) considered that major erosion generally occurred during a phase of erosive conditions, with a final culminating storm.

Because the actual storm demand is a complex function of these variables, it is usual to express the storm demand in terms of an average recurrence interval (ARI), that is the storm demand for a 50 year ARI event, or 100 year ARI event, for example. In this report, the storm demand is estimated for a storm having an ARI of 100 years.

5.5.4 Aeolian Sediment Transport

Aeolian sand transport can occur at beaches when (usually) dry sand is entrained by aeolian (wind) processes, particularly if the dunes are not densely covered by vegetation or protected by a seawall.

[28] Chapman et al (1982) noted that the occurrence of unusual conditions, out of phase with the normal, can cause damaging erosion along the coastline, as well as extreme erosive conditions.

Along the Gosford open coast beaches there is generally coverage of some dune vegetation or an elevated seawall landward of the beach, although vegetation coverage is limited in some areas due to the proximity of development to the beach. Therefore, from an overall sediment budget perspective, there is likely to be minimal sand loss from beaches in the study area due to aeolian sand movement.

Given that most of the study area is developed landward of dunal areas, there would not be expected to be any significant aeolian sediment transport supplying the study area with sand.

The importance of the stabilisation provided by dune vegetation cannot be understated. It is important to recognise that dune vegetation is necessary to stabilise dune systems and protect them from wind erosion into the future. Should human and vehicular traffic, or fire (for example) impact on the dunes in the study area in the future, there is the potential for landward sand drift to occur, with resulting shoreline recession.

Dune stabilisation works (including log and wire fencing and access control) are present along the majority of the open coast. Notable exceptions that would benefit from installation of fencing and access control for dune stabilisation include the following locations:

- Northern portion of Avoca Beach;
- Copacabana Beach (north of Cockrone lagoon) where the existing fence is almost buried allowing access onto the dunes. Additional formalised access in this location would also be beneficial.
- Northern portion of Copacabana Beach (north of Copacabana Beach SLSC)
- Northern end of Wamberal Beach (north of the entrance to Wamberal Lagoon);
- Isolated properties north of the entrance to Terrigal Lagoon;
- Properties north of the entrance to Pearl Beach Lagoon and south of Green Point Creek;
- Ocean Beach and some sections of Umina Beach.

5.5.5 Sediment Transport at Stormwater Systems

There are several stormwater outlets of varying scale and condition on the beaches of the Gosford LGA. In general, these drains will flow only during or immediately following periods of heavy rainfall. Potential beach management issues associated with the stormwater outlets include:

- Localised erosion resulting from stormwater scour;
- Loss of vegetation associated with stormwater flows;
- Localised lowering of beach level as a result of stormwater erosion/scour allowing larger waves to access the back beach area;
- Potential reduction in amenity as result of strong flows and reduced water quality;
- Aesthetic impact of structures on the beach;

- Impact on longshore sediment transport of structures extending across the beach;
- Accumulation of fines and organic matter around outlet.

Table 12 provides a general description of the stormwater outlets on the beaches in the study area and potential impacts they may have on coastal processes. Photographs of these outlets have been included in Appendix A and are referenced in Table 12. Note that stormwater from surrounding catchments is also discharged via lagoons and creeks at most beaches in the Gosford LGA, which can also impact coastal processes. This is described further in Section 5.8.

Table 12: Stormwater outlets on beaches in the Gosford LGA

Beach	Location	Description	Impact on coastal processes
Patonga Beach	Bay Street (**Figure A4**)	Box culvert under road with bank stabilising rock work	Minimal
	Between jetty and boat ramp (**Figure A8** and **Figure A9**)	Pipe discharging onto back of beach parallel to shoreline	Local scour and lowering of beach levels from depression caused by stormwater channel. Limited impact on processes.
Pearl Beach	Entrance to Pearl Beach Lagoon (**Figure A20**)	Culvert under road with three openings, and rock protection on bank adjacent to property	Minimal
Ocean-Umina	Southern corner of Umina Beach (**Figure A27**)	Pipe discharging onto vegetation at back of beach near toilet block	Minimal
Putty Beach	South end (south of SLSC) (**Figure A45**)	Two pipes discharging onto sand with surrounding rock/concrete/gabion to stabilise bank	Potentially large scour across beach creating local depression in profile. Limited impact on processes. Potential future erosion leading to loss of fill material.
	South end (350m north-east of SLSC) (**Figure A50**)	Unknown outlet configuration (inaccessible), discharging through dune system at back of beach.	Potentially large scour across beach creating local depression in profile. Limited impact on processes (**Figure A51**).
	Middle of beach (550m north-east of SLSC) (**Figure A52**)	Assumed naturally discharging flows through dune system at back of beach.	Potentially large scour across beach creating local depression in profile. Limited impact on processes.
	Middle of beach (1km north-east of SLSC) (**Figure A53**)	Assumed naturally discharging flows through dune system at back of beach.	Potentially large scour across beach creating local depression in profile. Limited impact on processes.
	North end (**Figure A54**)	Unknown outlet configuration (inaccessible), discharging through dune system at back of beach.	Potentially large scour across beach creating local depression in profile. Limited impact on

Beach	Location	Description	Impact on coastal processes
			processes.
MacMasters Beach	South end (north of SLSC) (**Figure A61**)	Pipe discharging onto vegetated sand	Local scour. Minimal impact on processes. Continued scour may give rise to future threat of outflanking of large pine tree and undermining of carpark.
	100m north of southern outlet (**Figure A62**)	Pipe discharging onto concrete with flow decelerating features and rock headwall.	Potentially large scour across beach creating local depression in profile (refer **Figure A63**). May exacerbate erosion and runup access to back beach region.
	North end (north of Copacabana SLSC) (**Figure A70**)	Large box culvert under road and additional double pipe outlet with concrete headwall further seaward. Rock wall protection on nth bank.	Potentially large scour across beach creating local depression in profile. May exacerbate erosion and runup access to back beach region.
Avoca Beach	South end (approximately 250m north of SLSC) (**Figure A76**)	Pipe discharging approximately 1m above beach level onto sand (and miscellaneously dumped rock/concrete)	Local scour. Limited impact on processes. May exacerbate erosion and runup access to back beach region (**Figure A77**). Potential future threat to pine tree.
	South end (approximately 430m north of SLSC) (**Figure A78**)	Broken pipe discharging directly onto sand	Local scour. Minimal impact on processes. May exacerbate erosion and runup access to back beach region (**Figure A79**). Potential future threat to pine tree.
	Northern corner (**Figure A87**)	Channel with concrete wall on north side into a pipe	Local scour. Minimal impact on processes
Terrigal-Wamberal Beach	South end of Terrigal Beach (**Figure A90**)	Box culvert with seven openings and surrounding rock protection	Local scour. Minimal impact on processes
	Rock bluff on southern bank of lagoon	Pit discharging down rock (indurated sand) face	Local erosion of rock bluff. Minimal impact on processes
Forresters Beach	Middle of beach, opposite Forresters Reef (**Figure A111**)	Damaged pipe elevated on piles 4m above beach level	Drop causes significant local scour. Minimal impact on processes, may exacerbate erosion and runup access to back beach region (**Figure A112**).

Given the relatively small size of the stormwater systems discharging onto the beaches in the Gosford LGA, movements of sand at these locations are generally manifested by localised depressions in the beach surface (that is, a temporary loss of sand from the subaerial portion of these beaches during rainfall-runoff events) and are unlikely to be significant in terms of the overall sediment budget. The stormwater systems would not be expected to supply any significant quantities of sand to the beach system.

5.5.6 Sediment Transport at Lagoon and Creek Entrances

Stormwater is also discharged onto the beaches in the study area through the various lagoon and creek entrances, including:

- Patonga Creek (Patonga Beach);
- Green Point Creek, Middle Creek and Pearl Beach Lagoon (Pearl Beach);
- Ettalong Creek (Umina Beach);
- Cockrone Lagoon (MacMasters-Copacabana Beach);
- Avoca Lake (Avoca Beach); and
- Terrigal and Wamberal lagoons (Terrigal-Wamberal Beach).

In general, these creeks and lagoons discharge across the beaches, breaking though the beach berms and causing scour channels during high flows. Similar to stormwater outlets, movements of sand at lagoon and creek entrances are generally manifested by localised depressions in the beach surface and are unlikely to be significant in terms of the overall sediment budget. Further, these systems would not be expected to supply any significant quantities of sand to the beach system.

Lagoon entrance processes are discussed further in Section 5.8.

5.6 Longer Term Sand Movement

5.6.1 General

Longer term sand movement on the Gosford LGA beaches has been examined using photogrammetry, by considering the movement of certain features such as the beach scarp between the first and last dates of photography. Details of the photogrammetric analysis undertaken are provided in **Appendices B** to **I** for each beach in the study area.

Caution needs to be exercised in the interpretation of the photogrammetric data due to a number of factors, such as:

- the relatively short period of historical data (it is implicitly assumed that coastal processes over this period are representative of the longer term situation);
- the frequency and severity of storms over the time span for which the volume changes were measured;
- the typically large fluctuations in sand volumes due to storms which can often mask longer term trends; and

- the influence of sea level rise which causes a reduction in the volume of sand above AHD and which has been operative over the period of the photographic record.

The longer term trends in sand movement are discussed below. The findings of the three previous assessments are also outlined.

5.6.2 Previous Assessments of Longer Term Sand Movement

Patterson Britton & Partners (1998) measured variations in sand volumes for each beach in the Broken Bay area (including Patonga, Pearl, Ocean-Umina and Putty) utilising photogrammetric techniques, based on data collected between 1941 and 1996. Based on these investigations, it was recommended that the long term trend be adopted as stable for each of these beaches. The following was also noted:

- sediment transport at Patonga Beach has been significantly influenced by construction of the training wall in 1969/70, while slugs of sand have been observed depositing material in the lee of Dark Corner;
- sand volumes at Pearl Beach recovered relatively quickly (i.e. within 3 years) following the August 1986 storm event, which was probably aided by beach scraping;
- sediment transport at Ocean Beach is quite complicated, and is significantly influenced by the phase of the estuarine cycle, with sand from the entrance shoal being transported into the nearshore area and then onto the beach during storms following a building phase on the shoal;
- further detailed investigations of the marine and estuarine cycles and their relative contributions to beach sand volumes at Ocean-Umina Beach were recommended; and
- sand volumes along Putty Beach were observed to have remained relatively stable since sand mining activities occurred in the 1950s and 1960s, although a review of this finding was recommended as more data post sand mining becomes available.

PWD (1985b) outlined mechanisms that can result in long-term sand losses from Terrigal-Wamberal and Avoca beaches, including:

- Net northerly alongshore sediment transport offshore over the rugged reef systems;
- Aeolian sand transport under the action of onshore winds;
- Infilling of lagoon systems; and
- (*Terrigal-Wamberal only*) Offshore sand transport due to rip currents during severe storms out onto the offshore reef system at around 25m water depth from where it cannot return to the active beach system (refer **Section 3.2**).

Further, as discussed in Section 5.5.2, PWD (1985b) could not identify any sources of sediment to the Avoca and Terrigal-Wamberal beach compartments.

PWD (1985b) determined long term average sand losses from Terrigal-Wamberal and Avoca beaches by comparing sand volumes on the beach at 1965 and 1984 (rather than linear regression techniques using several dates of data). It was considered that the beaches were in accreted states at both these times so the differences between sand volumes would reflect the rate of long-term sand loss. Recession rates of 0.44 and 0.64m/year were determined for North Avoca and Avoca respectively, while recession rates of 0.06 and 0.4-0.5m/year were determined for Terrigal and Wamberal respectively using this approach. However, subsequent analyses of additional photogrammetry data, as reported in PWD (1994) and the study herein, indicate that the 1984 profile in fact represents a relatively eroded beach state. As such, long term sand losses reported in PWD (1985b) have not been considered further.

PWD (1994) undertook an assessment of long term sand movement utilising photogrammetric techniques. The survey covered the period from 1941 to 1994 including up to 13 dates of aerial photography. Long term trends were calculated by examining average scarp movement as well as average volume change above the beach berm for the period of record. The following results were determined:

- MacMasters Beach was found to have receded over the period of measurement, with a maximum movement of 28m north of the lagoon entrance (or 0.55m/year or 2.5m^3/m/year above 4m AHD). Rates in the central portion of the beach (south of the lagoon entrance) were lower (in the order of 0.25m/year or 1.4m^3/m/year above 4m AHD).
- A design recession rate of 0.2m/year was adopted for Avoca Beach. Higher rates (around 0.47m/year) were determined in Block 7 (some 300m north of lagoon). Peaks also occurred in Blocks 2 and 3 associated with stormwater outlets. Accretion was found to be occurring at the northern end of the beach (Block 8) (possibly associated with a severe storm cut to this area prior to 1941).
- Terrigal-Wamberal Beach was found to have receded over the period of measurement, with considerable variability along the beach. Maximum recession rates of 0.3m/year were measured at several points along the beach, although generally the recessional trend was around 0.1m/year. Accretion was noted in Block 4 (between Terrigal and Wamberal lagoon entrances) and Block 7 (just south of Wamberal lagoon entrance). A design recession rate of 0.1m/year was adopted for the entire beach.
- Forresters Beach was found to have receded over the period of measurement, with peak recession rates measured either side of Forresters Reef. The maximum rate of scarp movement was measured in Block 2 north of the reef where movement of almost 0.5 m/year occurred (or approximately 3 m^3/m/year above 3.6 m AHD). A slightly slower rate of 0.4 m/year occurred south of the reef (Block 1) (or 2.5 m^3/m/year above 3.6 m AHD). Between these two peaks rates were as low as 0.1m/year.

5.6.3 Interpretation of OEH Photogrammetry

The photogrammetric data provided by OEH was analysed, in particular to determine long term recession rates due to net sediment loss[29]. To assess long term recession rates, changes in volume and in the position of particular contour levels were determined at each profile over time. This procedure is often denoted as Profile Area Volume (PAV) analysis. Volumes were determined using scripts developed by WorleyParsons in the software package MATLAB[30].

For the study reported herein, volumes above 0m AHD are given. The volume above 0m AHD was used as it was defined in many profiles without necessity for extrapolation, or only required extrapolation over a relatively short distance. Furthermore, volumes above 0m AHD were used because it is the typical datum level used in the method of Nielsen et al (1992). Selecting a higher level would have discarded relevant data. Profiles were generally extrapolated (if required) by continuing the profiles at the same average slope for the block and year as measured between the last two most seaward points in the profiles (generally near 0m AHD).

In the analysis, both the complete profiles (extending to the landward limit) were considered, as well as a landward truncation to a position in the vicinity of the sand/vegetation interface (or seawall location) as visible in 2010 aerial photography. Applying a landward truncation is relevant, as changes to profiles landward of this location can largely be considered to be related to anthropogenic processes (rather than natural coastal processes) [31].

In addition to volumes, the position of particular elevations over time was determined. These elevations must be low enough to be defined in most profiles, high enough to minimise unwanted noise, and reasonably representative of long term coastal processes seaward of the influence of most anthropogenic effects. These elevations were generally determined on a block by block basis for each beach in the study area.

For each of the profiles, the rate of change of the volume above 0m AHD, and the rate of change of the position of the relevant elevation, was determined. The rates were derived by linear regression; that is, by determining the line of best fit (least squares error) in each case[32]. The advantage of using linear regression, rather than simple differences between the first and last dates of photography, is that errors in predicted rates due to variations in beach states are likely to have been minimised.

Details of the photogrammetric analysis undertaken are provided in **Appendices B** to **I** for each beach in the study area.

[29] Given the period between each photography date, analysis to determine storm demand was not considered to be warranted. Pre-storm and post-storm sequences must be captured for such analysis to be reliable.
[30] MATLAB is a high-level technical computing language and interactive environment for algorithm development, data visualization, data analysis, and numerical computation.
[31] It is acknowledged that storm cuts may have extended landward of the landward truncation in severe storms. Aeolian processes may have also extended landward of the landward truncation. That stated, applying a landward truncation is still a relevant tool in assessing the changes in the main beach areas over time.
[32] This does not imply that there were uniform rates of volume or positional change between dates of photography.

5.6.4 Adopted Long Term Recession Rate Due to Net Sediment Loss

Measurement of historical long term variation in beach volume and beach position at each beach in the study area is useful in the consideration of the likelihood of any future long term recession at these beaches. Based on the analysis of beach profile data, as detailed in **Appendices B** to **I**, the design recession rates outlined in **Table 13** have been adopted

Table 13: Adopted Long Term Recession Rate due to Net Sediment Loss

Beach	Design Recession Rate (m/year)
Patonga	0 (stable)
Pearl	0.05
Ocean-Umina	0 (stable)
Putty	0 (stable)
MacMasters-Copacabana	0 (Blocks 1 and 6) 0.1 (Blocks 2, 3 and 5)
Avoca	0 (stable)
Terrigal Wamberal	N/A[33] 0.2
Forresters	0

In comparison with the design recession rates adopted in previous studies (see Section 5.6.2), the following can be noted regarding the design recession rates adopted herein:

- higher design recession rates have been recommended by this study for Pearl and Wamberal beaches;
- lower design recession rates have been recommended by this study for Avoca, MacMasters-Copacabana and Forresters beaches; and
- the recommended design recession rates have not been changed for Patonga, Ocean-Umina and Putty beaches.

[33] Design recession rates were not determined for Terrigal Beach because coastline hazard lines are not required in this area due to the presence of a seawall and ongoing commitment to protection of this important business district (refer Section H4.3 in Appendix H for further details).

5.7 Climate Change

5.7.1 Introduction

Climate is the pattern or cycle of weather conditions, such as temperature, wind, rain, snowfall, humidity, clouds, including extreme or occasional ones, over a large area and averaged over many years. Changes to the climate and, specifically, changes in mean sea levels, wind conditions, wave energy and wave direction, can be such as to change the coastal sediment transport processes shaping beach alignments.

Climate change has been defined broadly by the Intergovernmental Panel on Climate Change (IPCC 2001) as any change in climate over time whether due to natural variability or as a result of human activity. Apart from the expected climate variability reflected in seasonal changes, storms, etc., climate changes that are considered herein refer to the variability in average trends in weather that may occur over time periods of decades and centuries. These may be a natural variability of decadal oscillation or permanent trends that may result from such factors as changes in solar activity, long-period changes in the Earth's orbital elements (eccentricity, obliquity of the ecliptic, precession of equinoxes), or man-made factors such as, for example, increasing atmospheric concentrations of carbon dioxide and other greenhouse gases.

The signature of climate variability over periods of decades is seen in the Southern Oscillation Index (SOI), a number calculated from the monthly or seasonal fluctuations in the air pressure difference between Tahiti and Darwin. Sustained negative values of the SOI usually are accompanied by sustained warming of the central and eastern tropical Pacific Ocean, a decrease in the strength of the Pacific Trade Winds and a reduction in rainfall over eastern and northern Australia. This is called an El Niño episode. During these episodes, a more benign easterly wave condition is expected on the NSW coast. Positive values of the SOI are associated with stronger Pacific trade winds and warmer sea temperatures to the north of Australia, popularly known as a La Niña episode. Waters in the central and eastern tropical Pacific Ocean become cooler during this time. Together, these give an increased probability that eastern and northern Australia will be wetter than normal and, during these episodes, severe storms may be expected on the Australian Eastern seaboard.

Over much longer time frames, the Intergovernmental Panel on Climate Change (IPCC 2001) has indicated that the global average surface temperature has increased over the 20th century by 0.6°C and that this warming will continue at an accelerating rate. This warming of the average surface temperature is postulated to lead to warming of the oceans, which would lead to thermal expansion of the oceans and loss of mass from land-based ice sheets and glaciers. This would lead to a sea level rise which, in turn, would lead to recession of unconsolidated shorelines. Coastal communities and environments are particularly vulnerable to climate change due to the potential for permanent coastal inundation and increasing coastal hazards associated with changing weather patterns and extreme weather events.

5.7.2 Sea Level Rise

The possibility of global climate change accelerated by increasing concentrations of greenhouse gases, the so-called Greenhouse Effect, is now widely accepted by the scientific and engineering communities. This is predicted to cause globally averaged surface air temperatures and sea levels to rise.

The latest (Fourth Assessment) Intergovernmental Panel on Climate Change (IPCC) estimates of future sea level rise were released in 2007 (IPCC 2007). However, these estimates were not presented in as convenient a form as in the previous (Third) IPCC assessment (IPCC 2001a, b).

Given the similarity of the sea level rise estimates presented in the Third and Fourth IPCC Assessments, the Third Assessment results are generally presented below for clarity.

The global average sea level rises predicted by IPCC (2001a, b) between 1990 and 2100 are shown in **Figure 43**. The different curves displayed represent six illustrative emission scenarios (covering a wide range of the main demographic, economic and technological driving forces of future greenhouse gas and sulphur emissions), assumed in the Atmosphere-Ocean General Circulation Models (AOGCMs) used to develop the sea level estimates. It should be emphasised that the actual sea level rise that would occur is uncertain due to approximations in the modelling used to develop the estimates, plus the fact that the results are dependent on the emissions scenario adopted (which would vary depending on a variety of economic and political influences, with no specific probability assigned to any particular scenario). However, note that global average temperature and sea level are projected to rise under all emission scenarios.

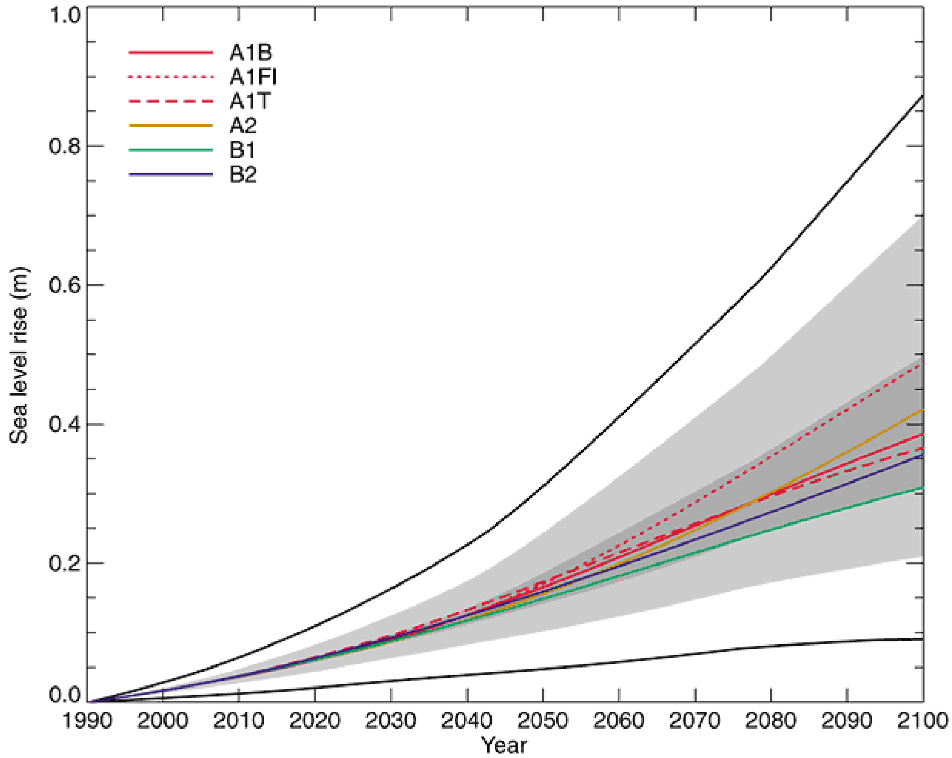

Figure 43: Global average sea level rise predicted due to climate change from 1990 to 2100, based on various emission scenarios (IPCC 2001b)

It should be noted that approximately 60mm of sea level rise occurred between 1990 and 2010 (DECCW 2010a), which is in line with the upper limit of sea level rise projections shown in Figure 43.

The State Government, through its Stage 1 Coastal Reforms which came into effect in January 2013, stipulated that "Councils should consider information on historical and projected future sea level rise which is widely accepted by competent scientific opinion." To this end, Gosford Council, at its Ordinary Meeting of 20 August 2013, endorsed a number of climate change scenarios relating to the Central Coast region. The climate change scenarios are intended to present a plausible future state of the climate in the region at different time periods and form the basis for risk assessment in this study. The indicative changes described in the scenarios are relative to the current period defined as the average climate experienced over the 1980 - 2007 period and are based on medium to high end of best available projections.

The scenarios were first published in 2010, in a report commissioned by the Hunter and Central Coast Regional Environmental Strategy (HCCREMS) called, Potential Impacts of Climate Change on the Hunter, Central and Lower North Coast of NSW (HCCREMS, 2010). That report was informed by a range of different sources, the most significant of which was a detailed analysis of historical climate variability in the Hunter, Central and Lower North Coast region of NSW (Blackmore & Goodwin, 2010). The methodology adopted by Blackmore and Goodwin in their analysis determined projected changes in key climate parameters using a weather typing approach to statistical downscaling from the CSIRO Mk3.5 Global Climate Model.

The scenarios (see Table 14) include consideration of future climate as it relates to:

- Sea Level rise and storm surge
- Extreme rainfall, flooding and storms
- Fire weather
- Average and extreme temperatures
- Average rainfall and water availability.

Table 14 illustrates Gosford Council's climate change scenarios for sea level rise and storm surge as adopted by Council.

Table 14: Climate Change Scenarios for Gosford (Sources: Blackmore & Goodwin, 2009, 2010; CSIRO, 2007; Macadam, McInnes and O'Grady, 2007; CSIRO, 2007b)

Climate Variable	Current[1] (indicative)	Indicative change[2] (relative to current)		Comments
		2050	2100	
1. Sea level rise and storm surge				
Sea level		↑ 0.4m	↑ 0.9m	Latest projections indicate SLR of up to 1.4m by 2100
Storm tide – max height, 1:100 ARI (average recurrence interval)	1.4m	1.8m	2.3m	Based on NSW design still water levels - excludes wave setup
Storm tide – ARI (1.4 m)	1:100	1:1	na	Limited regional modelling of recurrence intervals has been undertaken to date

Key

↑ increase; ↑↑ greater increase

↓ decrease, ↓↓ greater decrease

1. Current - average 1977-2007

2. Indicative change - based on 'most likely' projections

The relative contribution of sea level rise to elevated water levels on open coast beaches is indicated on the schematic provided in **Figure 44**. However, it should be noted that the wave setup and wave runup components of water level vary between each beach in the study area, as described in Section 5.2 and Section 5.3.2 respectively. For example, wave setup estimates vary between 0.3m (Patonga Beach) and 1.2m (Forresters Beach), while wave runup estimates vary between 2m AHD (Patonga Beach) and 8m AHD (Forresters Beach).

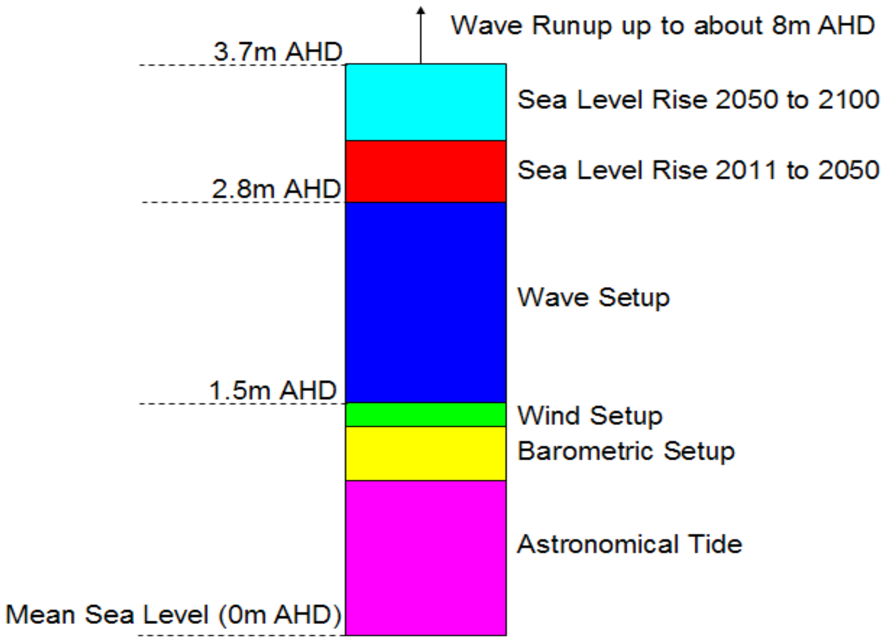

Figure 44: Components of elevated water levels typical for open coast beaches in NSW

Gosford Council's sea level rise planning benchmarks can be used for purposes such as incorporating the projected impacts of sea level rise on predicted flood risks and coastline hazards. The sea level rise scenario is to be used in all relevant strategic processes whereby all relevant strategic documents are to incorporate the adopted sea level rise planning level.

For the investigation reported herein, coastline hazards are estimated for the:

- immediate planning period;
- 2050 planning period with sea level rise of 0.4m (as per Table 14);
- 2100 planning period with sea level rise of 0.9m (as per Table 14).

However, it should be noted that there is considerable uncertainty regarding these values, and future sea level rise could be smaller or larger than predicted.

It is generally expected that recession of the open coast will occur under conditions of accelerated sea level rise, which is discussed further in **Section 6.3.3**.

5.7.3 Other Climatic Change Considerations

Another potential outcome of the Greenhouse Effect is an increase in the frequency and intensity of storm events.

Modest to moderate increases in average and maximum cyclone intensities are expected in the Australian region in a warmer world. However, cyclone frequency and intensity are strongly associated with the El Niño/Southern Oscillation (ENSO) phenomenon. How this phenomenon will vary in a warmer world is currently unknown (CSIRO 2001; CSIRO Marine Research 2001).

Mid latitude storms have been predicted to increase in intensity but decrease in frequency with global warming (CSIRO 2002), due to a reduction in equator to pole temperature gradients. However, as with tropical cyclones, climate modelling at present lacks the resolution to accurately predict changes associated with global warming.

If overall weather patterns change as a result of global warming, there is potential for changes in the angle of approach of the predominant wave climate (CSIRO 2007). For some beaches this may cause realignment of the shoreline, with resulting recession and accretion.

Given the above uncertainty and difficulty in quantitative prediction, no specific account was taken of any potential changes to storm frequency and intensity, or changes in wave directions[34]. However, this uncertainty should be taken into consideration when assessing the risk and consequences of recession occurring in the future.

There have been attempts (Ranasinghe et al 2004) to explain beach rotation in terms of shifts in the Southern Oscillation Index (SOI)[35]. Specifically, Ranasinghe et al (2004) proposed that beaches rotate clockwise (with the northern end accreting and southern end receding) in El Niño phases (negative SOI). Conversely, it was proposed that beaches rotate anti-clockwise (with the northern end receding and southern end accreting) in La Niña phases (positive SOI)[36].

It has been postulated that, as a result of the greenhouse effect, El Niño conditions will be favoured in the future (Cai and Whetton 2000; Boer et al 2004), thus favouring clockwise beach rotation. In the study area, this would most likely have a negative effect on the southern ends of the open coast beaches[37].

The impact of sea level rise on extreme coastal water levels is discussed in **Section 6.5.**

[34] A generally conservative approach was used in the estimation of the other coastline hazards.
[35] The SOI is calculated from the monthly or seasonal fluctuations in the air pressure difference between Tahiti and Darwin. The method used by the Australian Bureau of Meteorology is the Troup SOI which is the standardised anomaly of the Mean Sea Level Pressure difference between Tahiti and Darwin (Bureau of Meteorology, 2005).
[36] It was also found that La Niña phases were associated with more energetic (erosive) conditions.
[37] Beaches located within Broken Bay are not subject to variations in approach directions of storm waves, as experienced at ocean beaches where rotation has been observed.

5.8 Lagoon Entrance Processes

Four of the beaches within the study area are backed by lagoons, namely Cockrone (behind MacMasters Beach), Avoca, Wamberal and Terrigal (refer **Figure 1**). All four of the lagoons are intermittently open to the oceans and are classed as Intermittently Closed and Open Lakes or Lagoons (ICOLLs).

When closed, the lagoons are separated from the ocean by the beach berm. Breakout of the lagoon entrances occurs as flood levels in the lower estuary increase and overtop the berm, or Council mechanically open the lagoon to alleviate flooding or to allow flushing of the lagoon for water quality purposes. Full breakout channel development typically takes 6 hours, although this is dependent on the magnitude and duration of the flood and prevailing ocean water level. A breakout may be of short duration if the floodwater discharge is not sufficient to significantly scour the entrance channel and/or coincides with high wave conditions, which can rapidly transport sediment back into the lagoon.

Table 15 provides a summary of the frequency of mechanical lagoon breakouts, the average duration that the entrance remains open, the managed berm height (m AHD) and the trigger levels for the entrance opening (Cardno Lawson Treloar 2010).

Table 15 Mechanical Entrance Openings between 1976 & 2007

	Wamberal	Terrigal	Avoca	Cockrone
Average number per year	2.7	12.6	3.2	2.5
Average Duration Entrance Open (days)	10	8	21	9
Managed berm height (m AHD)	2.6-2.7	1.7	2.7-2.8	3.3-3.5
Trigger level for entrance opening (m AHD)	2.4	1.23	2.09	2.53

Cardno Lawson Treloar (2010) examined lagoon processes including entrance dynamics and shoreline recession for Cockrone, Avoca, Terrigal and Wamberal lagoons. These issues have therefore not been assessed further in the present report and the main findings of Cardno Lawson Treloar (2010) are summarised below:

- Management of the entrance for flood mitigation purposes has had a significant impact on estuarine hydraulics, with flow on effects for water quality, sediment transport and ecological processes. The breakout levels in the entrance management policy are determined by the levels at which property inundation starts to occur. In the case of Terrigal Lagoon, it is thought that the rate of breakouts under natural conditions would likely be higher due to slower rates of berm re-building. However, development of low lying lands around the foreshores means that the let out level adopted is quite low. Therefore, Council is required to let out the entrance on a regular (monthly) basis. This has likely resulted in very significant modification of natural water level processes, such that the variation in water levels is much more truncated than those observed for the other three lagoons. While entrance management practices would also be having a similar effect on hydraulics in the other lagoons, the magnitude of the impact is much greater for Terrigal.

- An assessment of the existing bank condition identified that the lagoon foreshores are relatively stable, with only isolated areas of erosion. The assessments of future erosion risk and shoreline recession indicate a potential for bank erosion to occur due to storm waves during very rare storm events. However, it is important to note that the level of risk will be highly dependent upon the bank condition in a specific location, with foreshore vegetation assisting in stabilising the shoreline and reducing the risk of erosion. This highlights the need for ongoing protection of foreshore vegetation and maintenance of any protection works. In the short term, shoreline erosion is more likely to occur in relation to human activities where, for example, people access the banks and/or waterways. Therefore, management of recreational usage of the lagoons is key ensuring the ongoing stability of the lagoon banks.

- A total of around 1 million kg of sediments are being delivered to four lagoons from the surrounding catchments every year. Whilst this gradual infilling of the lagoons over geological time is a natural process, the rate is likely to be higher than would occur for an undeveloped catchment.

- Breakout modelling for each lagoon system was undertaken to define the scour characteristics of the breakouts for planning purposes. The resulting entrance conditions following the 100-year ARI storm (in 2010 and 2050) are presented in Figure K1 to Figure K4 in Appendix K.

Entrance processes for these lagoons are described further in **Appendix K**, including:

- entrance migration;
- the effects of lagoon opening on surf zone processes;
- entrance scour; and
- an assessment of breakout modelling undertaken by Cardno Lawson Treloar (2010);

6. COASTLINE HAZARD ASSESSMENT

6.1 Overview

The potential coastline hazards that could impact on the beaches in the Gosford LGA are defined in subsequent sections, namely:

- beach erosion hazard;
- shoreline recession hazard;
- sand drift hazard;
- coastal inundation hazard;
- stormwater erosion hazard; and
- climate change; and
- slope and cliff instability hazard.

Each of the above hazards is discussed in turn in the following sections. The assessment of the hazards often draws upon the information set out in the preceding sections.

6.2 Beach Erosion Hazard

6.2.1 Preamble

During storms, large waves, elevated water levels and strong winds can cause severe erosion to sandy beaches (NSW Government 1990). The hazard of beach erosion relates to the limit of erosion that could be expected due to a severe storm, or from the effects of a series of closely spaced storms.

The erosion can be measured in terms of the volume of sand transported offshore or in terms of the landward movement of a significant beach feature. The volume is usually expressed in terms of cubic metres per metre run of beach (m^3/m), as measured above Mean Sea Level (MSL) or Australian Height Datum (AHD). The significant beach feature is usually taken to be the back beach erosion escarpment.

The beach erosion hazard is analogous to the “storm demand” discussed in **Section 5.5.3**. There are several methods to estimate storm erosion demand in the study area, including:

- analysing measurements of beach erosion that have been collected for the Broken Bay and open coast beaches;
- comparing measurements of beach erosion that have been collected at other similar beaches;

- storm cut numerical modelling;
- recently developed statistical joint probability type distribution approaches; and
- correlating storm demand to relative wave energy along the beaches in the study area.

6.2.2 Measurements for Broken Bay and Open Coast Beaches

Previous estimates of storm erosion demand, based on analysis of photogrammetric data, are provided in several studies for beaches in the study area, most notably Patterson Britton & Partners (1998) for Broken Bay beaches, and PWD (1985; 1994) for open coast beaches. Storm demand was also assessed in the study herein utilising similar photogrammetric techniques, with results detailed for each beach in Appendices B to I. A summary of the main findings of these investigations are presented below. It should be noted that the maximum storm demands outlined in Patterson Britton & Partners (1998) formed the basis for design storm demand values that were adopted in the existing Coastal Management Plan for Broken Bay beaches (Patterson Britton & Partners 1999), while the maximum storm demands outlined in PWD (1994) formed the basis for design storm demand values that were adopted in the existing Coastal Management Plan for open coast beaches (WBM 1995).

PATONGA BEACH

Patterson Britton & Partners (1998) compared July 1972 and July 1975 beach profiles at Patonga Beach to estimate the storm demand attributable to the major storm event in June 1974. Maximum storm erosion demands (calculated above 0m AHD and landward of the 2m AHD contour) measured along Patonga Beach included:

- Block A – 140m^3/m (in front of the tennis courts on Bay Street);
- Block B - 80m^3/m (near the intersection of Bay Street and Brisk Street);
- Block C - 50m^3/m; and
- Block D - 85m^3/m (near the boat ramp).

In comparison, the storm demand analysis undertaken herein (see Appendix B), which compared July 1972 and July 1975 beach profiles also, determined a maximum storm erosion demand of around 35m^3/m at the southern end. However, an apparent accretion of around 45m^3/m was measured just to the south of this location, which suggests that the measured volume changes at the southern end of Patonga Beach are influenced by infilling behind the training wall constructed in 1969-70. Moving south to north from this location, erosion values generally increase from around zero to 30m^3/m for the 1972 to 1975 analysis period.

The reasons for the significant differences between the results presented herein and those reported previously in Patterson Britton & Partners (1998) are not known. Given that the same photography was analysed for both studies, it appears that there must have been differences in the photogrammetry data supplied for each study. However, given that the original data has been lost, it has not been possible to check that data. The data analysed herein was checked by OEH and was certified by OEH to be within acceptable limits (Source: R. Clout, pers. comm. 20/9/11). As such, it has been assumed that inaccurate data was provided for the 1998 study, which seems likely because it is unlikely that storm erosion values up to 140m^3/m would be realised at a beach as well protected as Patonga.

PEARL BEACH

A storm erosion study of Pearl Beach was undertaken by Geomarine, as reported in Gosford City Council (1990a). This investigation measured a maximum storm erosion demand of 125m^3/m at the northern end of the beach. Further, the results of this study provided evidence that, for protected beaches, there is a direct relationship between storm erosion and wave height squared.

Patterson Britton & Partners (1998) determined storm erosion demands for Pearl Beach attributable to the major storm events in June 1974[38] and August 1986[39]. Maximum storm demands (calculated above 0m AHD and landward of the 2m AHD contour) measured along Pearl Beach included:

- Block 1 – 110m^3/m (near the outlet of Green Point Creek);
- Block 2 - 65m^3/m (near the outlet of Middle Creek);
- Block 3 - 80m^3/m; and
- Block 4 - 135m^3/m (near the entrance to Pearl Beach Lagoon).

In comparison, the storm erosion demand analysis undertaken herein (see Appendix C), which estimated storm demand attributable to the 1974, 1978 and 1986 storm events, determined similar maximum storm demands at Pearl Beach, including:

- Block 1 – 105m^3/m;
- Block 2 - 60m^3/m;
- Block 3 - 80m^3/m; and
- Block 4 - 120m^3/m.

It is evident that the various creek and lagoon outlets influence storm erosion volumes at Pearl Beach. For example, even though the wave heights are likely to be higher in Block 2 than Block 1, higher storm demands were measured in Block 1 near the entrance to Green Point Creek. This may be because the Green Point Creek channel tends to meander prior to a storm, which has a substantial effect on beach erosion volumes at the southern end of the beach (Patterson Britton & Partners 1998).

[38] Determined from analysis of aerial photographs taken in April 1971 and June 1974
[39] Determined from analysis of aerial photographs taken in September 1985 and August 1986

OCEAN-UMINA BEACH

Patterson Britton & Partners (1998) determined storm erosion demands for Umina and Ocean Beach attributable to the major storm events in June 1974[40], June 1978[41], August 1986[42], and May 1996[43]. In general, the storms in 1974 and 1978 produced the highest erosion along Umina Beach while the storms in 1996 produced the highest erosion along most of Ocean Beach.

Patterson Britton & Partners (1998) notes that Umina Beach recovered dramatically after the 1974 storm, regaining more sand in 3 years than existed in 1971. A maximum storm erosion demand of 210m^3/m was measured at the entrance to Ettalong Creek. Storm erosion demands (calculated above 0m AHD and landward of the 2m AHD contour) along all other sections of Umina beach was generally less than 100m^3/m, and included:

- Block 1 – 65 m^3/m (south of the entrance to Ettalong Creek);
- Block 2 – 100 m^3/m (the influence of Ettalong Creek extends about 140m to the north resulting in this relatively high storm demand);
- Block 3 – 45 m^3/m;
- Block 4 – 45 m^3/m;
- Block 5 – 70 m^3/m;
- Block 6 – 90 m^3/m (near the Berith Street drainage outlet); and
- Block 7 – 60 m^3/m (near the Trafalgar Avenue drainage outlet).

Maximum storm erosion demands measured along Ocean Beach included:

- Block 1 – 60 m^3/m;
- Block 2 – 100 m^3/m;
- Block 3 – 130 m^3/m (near the intersection of The Esplanade and Barrenjoey Road); and
- Block 4 – 190 m^3/m (near the entrance shoal).

In comparison, the storm erosion demand analysis undertaken herein (see Appendix D), which estimated storm demand attributable to the 1974, 1978 and 1986 storm events, determined similar maximum storm demands at Ocean-Umina Beach, including:

- Entrance to Ettalong Creek – 100m^3/m;
- Northern end of Umina Beach (Block 6) - 90m^3/m;

[40] Determined from analysis of aerial photographs taken in April 1971 and June 1974
[41] Determined from analysis of aerial photographs taken in December 1977 and August 1978
[42] Determined from analysis of aerial photographs taken in August 1978 and August 1986
[43] Determined from analysis of aerial photographs taken in April 1993 and June 1996

- Southern end of Ocean Beach (Block 1) - 60m^3/m; and
- Northern end of Ocean Beach (Block 6) - 225m^3/m

Patterson Britton & Partners (1998) notes that the changes in sand volumes along Umina Beach are complicated by the potential supply of sand to the northern half of the beach from the ebb tide delta shoal at the entrance to Brisbane Water. Indeed, the following observations should be noted regarding erosion at the northern end of Ocean Beach:

- While the 1996 storm event produced relatively significant erosion along most of Ocean Beach, a large accretion of sand was measured at the northern end, possibly due to deposition of sand from the entrance shoal or from southern sections of the beach.
- Conversely, the 1974 storm event was associated with accretion along all of Ocean Beach except the northern end where significant erosion was measured, gradually increasing from zero to 190m^3/m moving north. This would suggest that both tidal flows and waves combine to exacerbate erosion in this area.

Putty Beach

Patterson Britton & Partners (1998) compared April 1972 and July 1975 profiles of Putty Beach to estimate the storm erosion demand attributable to the major storm event in June 1974. Maximum storm erosion demands (calculated above 0m AHD and landward of the 2m AHD contour) measured along Putty Beach invariably occurred at stormwater or creek outlets where runoff contributed to erosion of the beach berm, and included:

- Block E – 110 m^3/m;
- Block F – 140 m^3/m;
- Block G – 160 m^3/m; and
- Block H – 60 m^3/m.

In comparison, the storm erosion demand analysis undertaken herein (see Appendix E), which also compared July 1972 and July 1975 beach profiles, determined a maximum storm erosion demand of around 140m^3/m at the southern end (near the northern end of the carpark). Storm erosion demand around 80m^3/m were determined in the middle section of the beach, increasing to around 130m^3/m moving north, while a maximum storm erosion demand of around 85m^3/m was measured in the protected northern corner.

These results are somewhat different from those reported in Patterson Britton & Partners (1998), which may be related to differences in the photogrammetry data supplied for each study.

In general, it can be seen that storm erosion generally increases to the north as exposure to the south and south easterly swell directions increases. Reduced erosion at the northern end of Putty Beach is most likely a result of some wave protection offered by West Reef.

Patterson Britton & Partners (1998) notes that measured storm demands are likely underestimated because the beach would have recovered to some degree up to the date of the 1975 photography. Further, the placement of fill seaward of the Surf Club following the storm would also lead to an underestimation of storm demand at the southern end. It is noted also that the rock shelf itself would have offered some protection against storm erosion, and that large rocks placed at the top of the shelf would have provided further protection against erosive waves during the 1974 storm event. Given that erosion was observed to occur back to the Surf Club above the rock shelf, storm demand at this location could have been up to 160m^3/m.

MacMasters-Copacabana Beach

PWD (1994) determined storm erosion demands for MacMasters-Copacabana Beach attributable to the major storm events in June 1974[44] and August 1986[45]. Maximum storm erosion demands (calculated above 0m AHD and landward of the 2m AHD contour) along all sections of the beach were found to occur due to the 1974 storm event, and included:

- South of the rock bluff – 120m^3/m;
- Between the rock bluff and lagoon entrance - 220m^3/m; and
- North of the lagoon entrance - 280m^3/m.

Similar values were also reported in Gosford City Council (1989b), which determined a maximum storm erosion demand value of 275m^3/m for MacMasters Beach due to the 1974 storm event.

In comparison, the storm erosion demand analysis undertaken herein (see Appendix F), which estimated storm demand attributable to the 1974 and 1986 storm events, determined reasonably similar maximum storm erosion demands at MacMasters-Copacabana Beach, including:

- South of the rock bluff – 105m^3/m;
- Between the rock bluff and lagoon entrance - 140m^3/m; and
- North of the lagoon entrance - 290m^3/m.

It is noted that the southern end of MacMasters-Copacabana Beach is underlain by cobbles, as evidenced in **Figure 15** which shows the underlying substratum at the southern end of the beach following the May-June 1974 storm event. The underlying substratum may limit the degree of erosion that can occur in this area, partly accounting for the lower storm erosion demand observed in this location when compared with the remainder of the beach north of the rock bluff. It is considered that a subsurface investigation in this area could allow a refined estimate of the storm erosion demand and extent of Zone of Reduced Foundation capacity to be developed for this section of beach.

[44] Determined from analysis of aerial photographs taken in April 1972 and June 1974
[45] Determined from analysis of aerial photographs taken in August 1984 and August 1986

Avoca Beach

PWD (1985b) determined storm erosion demands for Avoca Beach attributable to the major storm events in the late 1960s[46], June 1974 and June 1978, while PWD (1994) determined storm erosion demands for the June 1974[47], June 1978[48] and August 1986[49] storm events. In general, PWD (1985b) and PWD (1994) produced similar results, with maximum storm erosion demands found to occur due to the 1974 storm event. Maximum storm erosion demand values for various sections of Avoca Beach include:

- Southern end of Avoca Beach – 60m^3/m;
- Just south of the lagoon entrance - 160m^3/m (NB: photogrammetric data in this location was only available for the 1986 storm in PWD (1994);
- North of the lagoon entrance - 205m^3/m.

Similar values were also reported in Gosford City Council (1989a), which determined a maximum storm demand value of 200m^3/m for Avoca Beach due to the 1974 storm event.

In comparison, the storm erosion demand analysis undertaken herein (see Appendix G), which estimated storm demand attributable to the 1974, 1978 and 1986 storm events, determined reasonably similar maximum storm erosion demands at Avoca Beach, including:

- South of the lagoon entrance - 120m^3/m;
- North of the lagoon entrance - 225m^3/m.

A storm erosion demand of 225m^3/m at North Avoca Beach is considered to provide a better estimate of potential storm erosion in this area (compared with previous measurements which are slightly lower), given its exposure to swell waves and measurements at similarly exposed nearby beaches. This is discussed further in Section 6.2.6.

Terrigal-Wamberal Beach

PWD (1985b) determined storm erosion demands for Terrigal-Wamberal Beach attributable to the major storm events in the late 1960s[18], June 1974 and June 1978, while PWD (1994) determined storm erosion demands for the June 1974, June 1978 and August 1986 storm events. In general, PWD (1985b) and PWD (1994) produced similar results, with maximum storm erosion demands found to occur due to the 1974 storm event, except in isolated areas as described further below. Maximum storm erosion demand values for various sections of Terrigal-Wamberal Beach include:

[46] Aerial photographs taken in June 1965 and July 1967 separate the storm events on 20-21 May 1966, 5-6 September 1967, and 13-15 May 1968 (PWD, 1985a).
[47] Determined from analysis of aerial photographs taken in April 1972 and June 1974
[48] Determined from analysis of aerial photographs taken in April 1972 and June 1974
[49] Determined from analysis of aerial photographs taken in April 1972 and June 1974

- Terrigal Beach (i.e. south of the entrance to Terrigal Lagoon) - varied from around 60m^3/m at the southern end to a maximum of around 110m^3/m immediately south of the lagoon entrance;
- Wamberal Beach (i.e. north of the entrance to Terrigal Lagoon) - The maximum beach volume loss for the 1978 storm was around 220m^3/m, which occurred as a result of a very severe rip located north of the entrance to Terrigal Lagoon where two houses were lost and the escarpment retreated by up to 20m (near the intersection of Ocean View Drive and Pacific Street). The maximum beach volume loss for the 1974 storm was around 235m^3/m, which was measured just south of the entrance to Wamberal Lagoon.

In comparison, the storm erosion demand analysis undertaken herein (see Appendix H), which estimated storm demand attributable to the 1974, 1978 and 1986 storm events, determined similar maximum storm erosion demands at Terrigal-Wamberal Beach, including:

- Terrigal Beach - 60m^3/m;
- Wamberal Beach - 250m^3/m.

FORRESTERS BEACH

PWD (1994) determined storm erosion demands for Forresters Beach attributable to the major storm events in June 1974 and August 1986. Maximum storm demands along the southern and northern sections of the beach during these respective storm events included:

- Southern end – 175m^3/m (due to the 1974 storm event); and
- Northern end – 165m^3/m (due to the 1986 storm event).

Gosford City Council (1990a) reported higher storm demands for Forresters Beach than those outlined above, with maximum values of 200m^3/m and 250m^3/m attributed to the 1974 and 1986 storm events respectively.

In comparison, the storm erosion demand analysis undertaken herein (see Appendix I), which estimated storm demand attributable to the 1974 and 1986 storm events, determined similar maximum storm erosion demands at Forresters Beach, including:

- Southern end – 145m^3/m (due to the 1974 storm event); and
- Northern end – 155m^3/m (due to the 1986 storm event).

These analyses indicated that the effects of each of these storms varied significantly from one end of the beach to the other. For example, the changes in beach profile following the 1974 storm event were consistent with erosion at the southern end while the northern end accreted slightly. Conversely, in response to the 1986 storm event it appears that Forresters Beach experienced erosion at the northern end while the southern end accreted slightly.

6.2.3 Measurements at Other Beaches

Based on field measurements, Gordon (1987) estimated that the storm erosion demand above 0m AHD was about 220m^3/m for the 100 year ARI event, for exposed NSW beaches at rip heads. This reduces to 140m^3/m for "low demand" areas away from rip heads. In practice, in any one storm, more severe erosion would occur at discrete locations corresponding to the location of major rips. However, rips would be likely to form anywhere on each of the open coast beaches, meaning that it would be reasonable to assume a rip-related storm demand at any location along these beaches.

6.2.4 Storm Cut Numerical Modelling

Time-dependant, process-based numerical beach erosion models such as SBEACH (US Army Corps of Engineers), UNIBEST-TC (Delft Hydraulics) and LIT-CROSS (Danish Hydraulics Institute) can be used to estimate storm demand on a given cross-shore beach profile. For a particular offshore wave and water level time series, and provided with sediment data such as mean grain size, these models can be used to estimate a post-storm profile.

However, it should be noted that numerical modelling techniques are limited in the estimation of storm demand. Previous experience has shown this to be misrepresentative of actual volumes. Complex three dimensional processes (hydrodynamic flow and rip cells) and temporally varying conditions (e.g. a series of closely spaced storms) cannot be represented by simplistic modelling.

Carley and Cox (2003) used SBEACH to estimate storm erosion demand for Narrabeen Beach, which has similar exposure to offshore wave conditions as the open coast beaches in the Gosford LGA. They found that simulating a 100 year ARI storm produced a storm erosion demand of only 110m^3/m, while the simulation of three consecutive storms produced between 180m^3/m and 240m^3/m of erosion (above 0m AHD). However, they noted that rigorous treatment of the probabilities of such sequential storms was the major hurdle for the practical application of time dependant beach erosion models in engineering practice.

6.2.5 Statistical Methods

Callaghan et al (2008, 2009) have developed a method for estimation of storm demand based on joint probability distributions of wave height, storm duration, wave period, tidal anomaly, and wave direction. It can be inferred from these papers that 100 year ARI storm demand values at open coast beaches using this joint probability method are in the order of 220 to 250m^3/m. However, there is considerable uncertainty in extrapolating their results to such rare events.

6.2.6 Correlating Storm Demand to Wave Energy

As discussed in Section 5.6.2, shoreline erosion can be expected to correlate with inshore wave energy. As noted in Section 5.1.3, inshore wave energy varies along each of the beaches in the study area, particularly at protected sections of the beaches (see **Figure 33** and **Figure 35**).

Robust techniques are available to estimate the variation in predicted storm demand based on wave energy considerations. An example of this is presented in WorleyParsons (2009), which predicts the variability in storm demand and maximum fluctuation in beach width for Collaroy-Narrabeen Beach. A similar approach has been adopted herein, using the validated results from the nearshore wave transformation modelling undertaken (see Section 5.1.3 and Appendix J).

Setting the average wave energy at 6.5m depth at the northern end of Wamberal Beach to be equivalent to a storm demand of 250m^3/m, the variation in predicted storm demand along each beach in the study area (excluding Forresters, as discussed below) is depicted in **Figure 45**. Currently adopted design storm erosion demands are also plotted for comparison with these results.

GOSFORD CITY COUNCIL
OPEN COAST AND BROKEN BAY BEACHES
COASTAL PROCESSES AND HAZARD DEFINITION STUDY

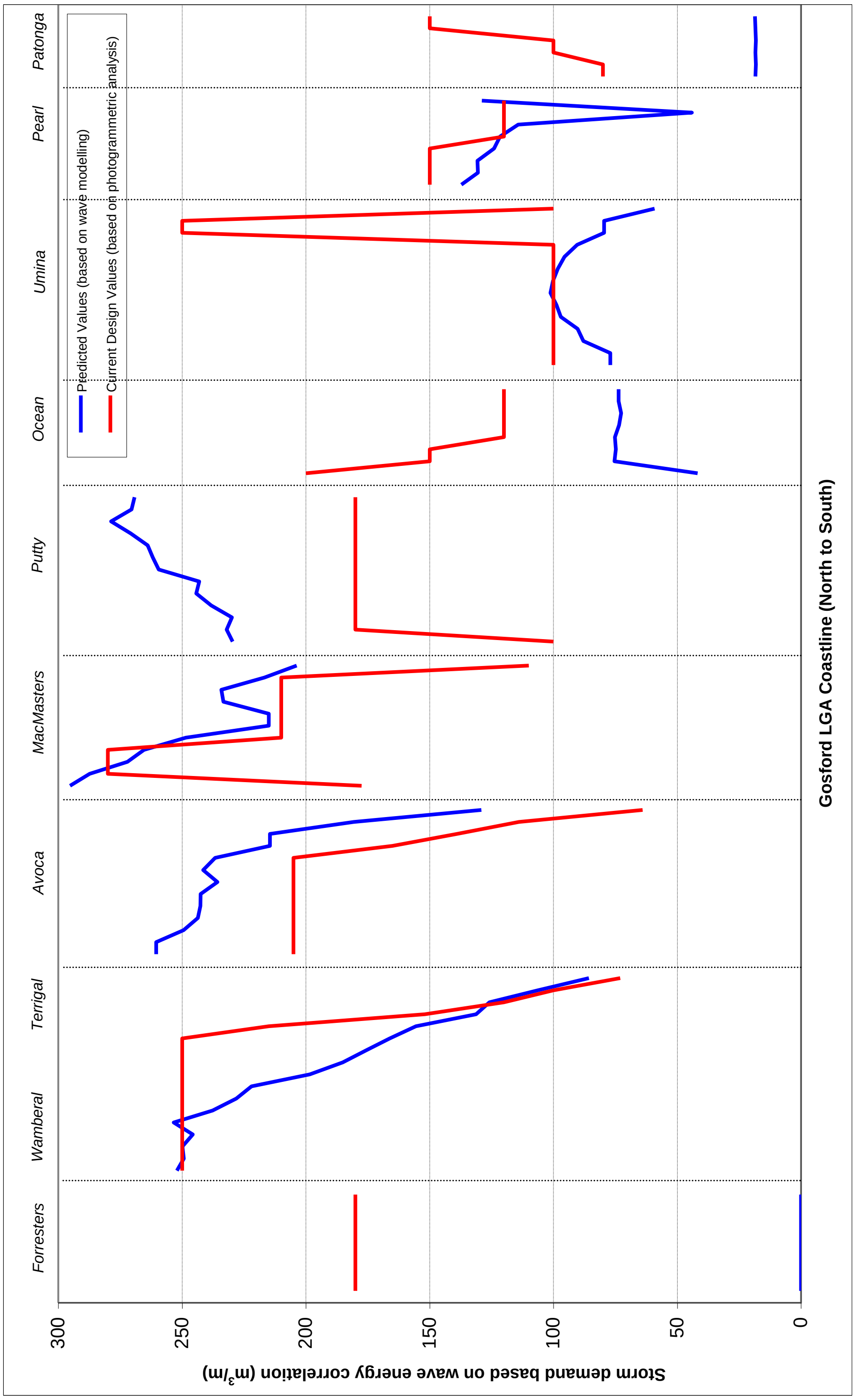

Figure 45: Variation in predicted storm demand (based on wave energy considerations) along Gosford LGA coastline, compared with current design values

It is evident that the predicted storm erosion demand values provide general agreement with currently adopted design values for most beaches. The following can be noted for each beach:

- Forresters Beach was not assessed using this approach because the wave modelling results at this location appear to be influenced by wave focussing at the rock reef where wave breaking (and energy dissipation) is likely to be greatest. Due to the complex bathymetry at Forresters Beach, wave propagation to the shoreline (inshore of the rocky reef) could not be modelled adequately.
- The reduction in the predicted storm erosion demand moving south along Terrigal-Wamberal Beach is generally consistent with the current design values.
- The predicted storm erosion demand at Avoca Beach is quite similar to that for Terrigal-Wamberal, with storm erosion demands predicted to decrease from around 250 to 125m^3/m moving south. This result is expected given the similar exposure to swell waves at both beaches, although it is noted that the southern end of Avoca Beach is slightly more exposed to critical wave directions compared with Terrigal Beach. However, the currently adopted design values are somewhat lower than the predicted values, providing scope for revising the design values (see Section 6.2.7).
- The predicted storm erosion demand at MacMasters-Copacabana Beach shows good agreement with the current design values, including a reduction in the likelihood of storm erosion moving south.
- The relative exposure of Putty Beach to southerly waves has resulted in predicted storm erosion demand values increasing from around 230 to 280m^3/m moving south. These values are much higher than the currently adopted design values (100m^3/m at the northern corner and 180m^3/m along the remainder of the beach). As noted in Patterson Britton & Partners (1998), the measured storm erosion demands are likely underestimated because the beach would have recovered to some degree up to the date of the 1975 photography. As such, it would be reasonable to consider revising the design storm erosion demand values for Putty Beach (see Section 6.2.7).
- Predicted storm erosion at Ocean Beach is somewhat lower than the currently adopted design values. This is likely related to the effect of the entrance shoal in the model, which caused increased wave refraction near Ocean Beach and a subsequent reduction in wave heights (compared with Umina Beach, which is less exposed to wave attack). However, in reality, a major storm event could lead to significant erosion of the entrance shoal, which would be allow for more direct propagation of wave energy to Ocean Beach, as observed in 1974. As such, the current design storm erosion demand values for Ocean Beach are likely appropriate.
- The predicted storm erosion demand at Umina Beach shows good agreement with the current design value of 100m^3/m. The design storm erosion demand of 250m^3/m at the entrance to Ettalong Creek is associated with the potential for increased scour in this area

during significant rainfall-runoff and storm events, which was not accounted for in the wave transformation modelling.

- The predicted storm erosion demand at Pearl Beach shows good agreement with the current design values, including a reduction in the likelihood of storm erosion moving south.
- A predicted storm erosion demand of around 20m^3/m was determined for Patonga Beach, which is significantly lower than the currently adopted design values which increase from 80 to 150m^3/m moving north. Based on these results, it would be reasonable to consider revising the design storm erosion demand values for Patonga Beach (see Section 6.2.7).

As noted in **Section 6.2.4**, storm demands are also strongly influenced by the location of major rips. While correlating storm demand to wave energy can provide a reasonably accurate estimation of the expected variability in storm demand along a beach, a more conservative approach is generally adopted which recognises, among several other factors, that rips can usually form anywhere on a beach. Nevertheless, consideration should still be given to the degree of exposure to wave energy, particularly in well protected corners.

6.2.7 Recommended Design Storm Demand

The *Guidelines for preparing Coastal Zone Management Plans* (OEH 2013) recognise that the extent of beach erosion depends on a number of factors (refer **Section 5.6.2**). As such, it may be difficult to quantify the potential upper limit of storm demand that may occur in a specific area, and so a more empirical approach is recommended for estimating storm demand. For the NSW open coast, Watson (2005) recommends the following maximum storm demands (above MSL, i.e. above 0m AHD) for planning purposes in the order of:

- 200-250 m^3/m in fully exposed locations; and
- 130m^3/m for protected embayments.

In areas where insufficient information is available to establish an empirical storm demand based on site-specific information, a default value of 250m^3/m has been adopted in previous investigations along the NSW coast as a conservative estimate for exposed open coast locations. However, as demonstrated in Section 6.2.2 and Section 6.2.6, a reasonably comprehensive analysis has been undertaken for each beach in the study area.

In light of the findings outlined above, the recommended storm erosion demands (above 0m AHD) for each beach in the study area are outlined below for the 100 year ARI event. These values are summarised in **Table 16**.

Table 16: Recommended design storm erosion demand values for the study area

Beach	Location	Design Storm Demand (m^3/m above 0m AHD)	Comments
Patonga	Entire Beach	40	Considered to be consistent with maximum measured values (see Section 6.2.2), and reasonably conservative compared with predicted values (see Section 6.2.6).
Pearl	Block 1	120	Consistent with currently adopted design storm demand values. Based on good agreement between values measured in this study with previously measured values (see Section 6.2.2), and also good agreement between measured values and predicted values (see Section 6.2.6).
	Block 2	120	
	Block 3	120	
	Block 4	150	
Umina	Blocks 1 to 7	100	Consistent with currently adopted design storm demand values (except at entrance to Ettalong Creek – a storm erosion demand of 250m³/m was previously adopted but this is not considered to be necessary given that the highest measured value at this location is 100m³/m, see Section 6.2.2).
Ocean	Block 1	120	
	Block 2	120	
	Block 3	150	
	Block 4	200	
Putty	Block 1	280	Higher than previous design values, based on predictions using wave modelling results (see Section 6.2.6). Likely conservative.
	Block 2	200	
MacMasters-Copacabana	Block 1	200	Slightly higher than currently adopted design storm demand values, based on predictions using wave modelling results (see Section 6.2.6). Likely conservative.
	Block 2	240	
	Block 3	240	
	Block 4	N/A	Lagoon entrance area.
	Block 5	280	Consistent with currently adopted design storm demand values.
	Block 6	280→100 moving north	It is considered that the potential for storm erosion in the northern corner decreases moving north due to wave protection offered by the rocky reef.
Avoca[50]	Blocks 1 to 5	100→250 moving north	Slightly higher than currently adopted design storm demand values, based on predictions using wave modelling results (see Section 6.2.6), and also slightly higher measured values at North Avoca (see Section 6.2.2) in this study, i.e. 225m³/m compared with 205m³/m measured previously . Also ensures consistency with design values adopted for similarly exposed nearby beaches, e.g. Terrigal-Wamberal. Likely conservative.
	Blocks 6 to 9	250	
Terrigal-Wamberal[51]	Blocks 1 to 2 (Terrigal)	60→140 moving north	Consistent with currently adopted design storm demand values. Based on good agreement

[50] Note that storm demand calculations were not undertaken for Block 5 at Avoca Beach (lagoon entrance area)

Beach	Location	Design Storm Demand (m^3/m above 0m AHD)	Comments
	Blocks 4 to 7 (Wamberal)	250	between values measured in this study with previously measured values (see Section 6.2.2), and also good agreement between measured values and predicted values (see Section 6.2.6).
Forresters	Entire Beach	180	Consistent with currently adopted design storm demand values. Based on good agreement between values measured in this study with previously measured values (see Section 6.2.2).

6.2.8 Erosion at Lagoon Entrances

Breakout modelling studies of Wamberal Lagoon, Terrigal Lagoon, Avoca Lake and Cockrone Lake have been undertaken by Cardno Lawson Treloar (2010). The 100 year ARI breakout event was modelled to provide information for the coastal hazard study. A detailed description of this study and its findings is included in Appendix K.

It is noted that the modelling results for each lagoon do not show the full extent of entrance scour that can be expected at the lagoon entrances or, indeed, the extent of scour that has occurred there in recent times. As such, the lines of fringing vegetation at each of the entrances, being well beyond the 100 year ARI erosion hazard as defined by the modelling, have been adopted for lagoon entrance scour hazard mapping. Scour beyond those lines for the 100 year ARI condition is not indicated by the modelling.

A more detailed assessment of entrance behaviour over time may be useful for better defining the erosion hazard in these areas. This could involve the collection and analysis of photogrammetric data along bank-normal profile lines in each of the lagoon entrances, similar to that undertaken for the beaches. Further, an assessment of future shoreline migration at each entrance area due to projected sea level rise could also be undertaken to define areas likely to be impacted. These recommendations are discussed further in Section 8.

It is considered that bank protection works evident at the entrance to Patonga Creek (see Figure A1 in Appendix A) and Ettalong Creek (see Figures A30 and A31 in Appendix A) would provide adequate protection to surrounding development (i.e. the holiday parks at both locations, and development on the southern banks of both creek entrances) from erosion related to scour events. Further, future shoreline migration of these entrance areas is not expected to occur due to projected sea level rise, based on findings discussed in Section 6.3.3.

[51] Note that storm demand calculations were not undertaken for Block 3 at Terrigal-Wamberal Beach (lagoon entrance area)

6.3 Shoreline Recession Hazard

6.3.1 Preamble

The hazard of shoreline recession is the progressive landward shift in the average long term position of the coastline (NSW Government 1990). Two potential causes of shoreline recession are net sediment loss, and an increase in sea level, as outlined in Sections 6.3.2 and 6.3.3 respectively. It is also appropriate to discount the historical recession due to net sediment loss, due to actual sea level rise that occurred during the measurement period from 1941 to 2006, as discussed in Section 6.3.4.

6.3.2 Long Term Recession Due to Net Sediment Loss

Long term recession due to net sediment loss is a long duration (period of decades), and continuing net loss of sand from the beach system. According to the sediment budget concept, this occurs when more sand is leaving than entering the beach compartment. This recession tends to occur when:

- the outgoing longshore transport from a beach compartment is greater than the incoming longshore transport;
- offshore transport processes move sand to offshore “sinks”, from which it does not return to the beach; and/or,
- there is a landward loss of sediment by windborne transport (NSW Government 1990).

Shoreline recession due to net sediment loss should not be confused with beach erosion, which results in a short term exchange of sand between the subaerial and subaqueous portions of the beach, not a net loss from the active beach system. Shoreline recession is therefore a long term process which is overlain by short term fluctuations due to storm activity.

The long term recession rates adopted for each beach in the study area were provided in Table 13 in Section 5.6.4.

6.3.3 Long Term Recession Due to Sea Level Rise

A progressive rise in sea level may result in shoreline recession through two mechanisms: first, by drowning low lying coastal land, and second, by shoreline readjustment to the new coastal water levels. The second mechanism is probably the more important since deeper offshore waters expose the coast to attack by larger waves, the nearshore refraction and diffraction behaviour of waves may change, and a significant volume of sediment may move offshore as the beach seeks a new equilibrium profile (NSW Government 1990).

(Bruun 1962; 1983) proposed a methodology to estimate shoreline recession due to sea level rise, the so-called Bruun Rule. The Bruun Rule is based on the concept that sea level rise will lead to erosion of the upper shoreface, followed by re-establishment of the original equilibrium profile. This profile is re-established by shifting it landward.

A detailed description of the Bruun Rule and its application to each beach in the study area is provided in **Appendix L**, including calculations of shoreline recession due to projected sea level rise for the 2050 and 2100 planning periods, which are summarised in **Table 17**.

Table 17: Values of Long Term Recession due to Sea Level Rise

Beach	Inverse Slope	Long Term Recession due to Sea Level Rise (m)	
		2050	**2100**
Patonga	10	3.4	8.4
Pearl	14	4.8	11.8
Ocean-Umina	N/A	0	0
Putty	25	8.5	21.0
MacMasters-Copacabana	39	13.3	32.8
Avoca	50	17.0	42.0
Terrigal-Wamberal	43	14.6	36.1
Forresters	-	-	-

It can be seen that significant long term recession due to sea level rise is not projected to occur at Ocean-Umina Beach, which is due to the relatively flat offshore profile compared with the respective equilibrium wave profile, as discussed in Appendix L. The beach fronts the Woy Woy beach ridge barrier, which is the largest Holocene sand barrier in New South Wales, which has been building for in excess of 8,000 years (Nielsen & Roy, 1981; Chapman et al., 1982; Roy & Boyd, 1996) and which photogrammetric data in this study has indicated is continuing to grow to the present. The reason that it is growing is because it is fronted by a relatively shallow, very wide convex upward sand shoal over which low swell waves propagate shoreward, causing onshore sediment transport. In line with a risk adverse planning approach, The NSW Office of Environment and Heritage has advised of the potential for a low risk of foreshore recession at Ocean-Umina Beach under the sea level rise projections considered in this study. However, WorleyParsons' opinion is that the fundamental geological coastal processes prevailing within Broken Bay and at Ocean-Umina Beach would not be changed under the sea level rises projected in this study and that onshore sand transport to Ocean-Umina Beach would continue, albeit at a lower rate than that occuring at present.

Long term recession has not been projected for Forresters Beach. At Forresters Beach, rock is ubiquitous from -2 m AHD in the nearshore zone and the dune is underlain in the main by consolidated bluff material and rock. Drilling at Forresters Beach (Hudson 1997) indicated rock levels above 5 m AHD and higher for the development along Kalakau Avenue north of the intersection of Boos Road. South of Bluewave Crescent the rock levels appear at around 0 m AHD with clay layers at higher elevations and generally up to around RL 9.0 m AHD. As Forresters Beach has geomorphological features that are very different from the duned beaches of Wamberal, Avoca, etc., it is unreasonable to expect that it will respond similarly to these adjacent beaches under a rising sea level. The Bruun Rule cannot be applied to Forresters Beach due to the shallow rock in the nearshore zone and clay material to RL 9 m AHD within the dune. It is likely that the level of rock and clay would preclude any long term recession under sea level rise at Forresters Beach.

It should also be noted that, for the assessment of long term recession due to sea level rise, it was necessary to discount sea level rise that has already occurred relative to the average values between 1980 and 2007. This is because the adopted 0.4m sea level rise at 2050 is defined to be relative to the average values between 1980 and 2007 (see Section 5.7.2). As described by OEH (2013a), between 1993 and 2009, the estimated rate of rise was 3.2 ± 0.4 mm per year from satellite altimetry data and 2.8 ± 0.8 mm per year from tidal records. To account for this, the actual sea level rise applied in using the Bruun Rule is 0.4 minus 0.06, that is 0.34m (2050), and 0.9 minus 0.06, that is 0.84m (2100).

It is noted that the Bruun Rule has been questioned in the scientific literature, for example by Cooper & Pilkey (2004) and Ranasinghe et al. (2007) to name two. However, no alternative tools for practical application in the engineering community have been presented. The Bruun Rule is based on rational coastal engineering principles and has been applied in this hazard assessment in cognizance of the fundamental assumptions upon which it was based to estimate projected long term recession due to sea level rise.

6.3.4 Discounting of Historical Recession Rates

Shoreline recession rates determined from historical data may be influenced by any sea level rise which occurred in the period of the historical record, i.e. the period over which the long term recession rate was determined. That is, although any long term recession that has occurred over the historical record would mainly be expected to be related to net sediment loss, given that there has also been some sea level rise over the historical record it can be argued that any historical long term recession has been partially caused by long term recession due to sea level rise.

Averaged around Australia, the relative sea level rise from 1920 to 2000 was about 1.2 mm/year (CSIRO Marine Research 2004). Adopting this rate for the period over which the long term recession rate was determined, and using the Bruun Rule, an equivalent shoreline recession can be accounted for. This approach has been adopted, where appropriate, in the assessment of long term recession trends for each beach in the study area, as described in Appendices B to I.

6.4 Sand Drift Hazard

As noted in Section 5.5.4**,** sand drift is a result of this aeolian wind movement of beach sediment, and as such can be controlled to a large extent by the presence of a well vegetated foredune. Sand drift leads to a number of hazards depending on the volume of sand involved. For low sand volumes, sand drift is only of nuisance value. However, for high sand volumes it can represent a permanent loss of sand from the active beach system, thereby causing shoreline recession (if the sand moves landward beyond the foredune[52] into the hinddune), and can result in abrasion, burial, blockage and damage to coastal developments (NSW Government 1990).

As outlined in Section 5.5.4, beaches in the study area are characterised by coverage of dune vegetation or an elevated seawall landward of the beach, although vegetation coverage is limited in some areas due to the proximity of development. Further, dune stabilisation works (including log and wire fencing and access control) are present along most of the open coast. In particular, dune revegetation undertaken at Putty Beach following the completion of sand mining activities in the 1950s and 1960s were noted to be successful in mitigating previous windblown sand losses (Patterson Britton & Partners 1998). The sand drift hazard in the study area is likely to be minimal

6.5 Coastal Inundation Hazard

Coastal inundation is the flooding of coastal lands by ocean waters, which is generally caused by large waves and elevated water levels associated with severe storms. Severe inundation is an infrequent event and is normally of short duration, but it can result in significant damage to both public and private property (NSW Government 1990).

The components which give rise to elevated still water levels at times of storms have been referred to in Section 5.2, namely storm surge (including wind setup and barometric setup) and wave setup. This increased water level may persist for several hours to days and can inundate low lying beach areas and coastal creeks. The 100 year ARI total design still water level adopted for each beach in the study area is summarised in Table 6 (see Section 5.2), which varies between 1.8m AHD (Patonga Beach) and 2.7m AHD (Forresters Beach) throughout the study area. For long term planning purposes, sea level rise (as outlined in Section 5.7.2) would also be included.

During storm events, individual waves result in further temporary water level increases above the still water level due to the process of wave setup and runup or uprush (Section 5.3). A comprehensive assessment of wave runup was undertaken for each beach in the Gosford LGA is described in Appendix K. The wave runup values adopted for each beach in the study area are summarised in Table 8 (see Section 5.3.2).

Appendix K also contains an assessment of potential inundation levels near the entrances to Cockrone Lagoon, Avoca Lake, Terrigal Lagoon and Wamberal Lagoon. The inundation hazard at these areas can occur as a result of ocean stormwave (coastal) inundation and/or catchment derived flooding. As such, the assessment of potential inundation levels at the lagoon entrances involved comparison of inundation levels caused by wave runup with the 1% flood level determined near the entrance at each lagoon, and adopting the higher value as the potential inundation level. Adopted inundation levels for each lagoon entrance are summarised in **Table 18**, which also shows whether these levels are related to wave runup (coastal) or catchment derived flooding.

[52] The foredune is the larger and more mature dune lying between the incipient dune (generally characterised by grass vegetation coverage) and hinddune area (generally). Foredune vegetation is characterised by grasses and shrubs. Foredunes provide an essential reserve of sand to meet erosion demand during storm conditions. During storm events, the foredune can be eroded back to produce a pronounced dune scarp (NSW Government, 1990).

Table 18: Adopted Inundation Levels at Cockrone Lagoon, Avoca Lake, Terrigal Lagoon and Wamberal Lagoon entrances

Lagoon/Lake	Location	Adopted Inundation Level (m AHD)	Coastal or Flooding
Cockrone	South Bank	3.8	Flooding
	North Bank	3.8	Flooding
Avoca	South Bank	3.2	Flooding
	North Bank	4.5 – 5.0	Coastal
Terrigal	North Bank	3.3	Coastal & Flooding
Wamberal	South Bank	4	Coastal
	North Bank	3.5 – 4.0	Coastal

It is noted that potential inundation levels near the entrances to Patonga Creek and Ettalong Creek are most likely to be dominated by elevated ocean storm surge levels rather than catchment derived flooding, given the small catchments of these creeks. As flood studies have not been completed for these creek systems, the 1% flood levels near the entrances cannot be stated. However, it can be expected that flood levels at these entrance areas would be controlled by elevated ocean still water levels, i.e. 1.5m AHD for the 100 year ARI design still water level (see Section 5.2). This level does not include wave setup, which is likely to be minimal in the entrance areas. While some backwater effects may further elevate water levels in the entrance areas, this is not likely to be significant given the relatively small catchment areas drained by Patonga Creek and Ettalong Creek respectively.

Given that ground levels near development at the entrances to Patonga Creek and Ettalong Creek are at a minimum of around 4.0 m AHD, it is likely that the potential for inundation in these areas is dominated by wave runup (coastal) processes, rather than catchment derived flooding. As such, inundation levels determined in the wave runup assessments undertaken for each beach (see Appendix K and Table 8 in Section 5.3.2) can be used to define the inundation hazard in these areas.

Lots potentially affected by inundation are indicated on the hazard maps provided in Section 7 for each beach in the study area.

6.6 Stormwater Erosion Hazard

During major stormwater runoff events, stormwater collected from back beach areas and discharging into coastal waters can cause significant erosion to the beach berm. This in turn can allow larger waves to attack the beach and can cause migration of the stormwater discharge entrance if not structurally contained (NSW Government 1990). Flow from stormwater pipes and outlets on beaches have the potential to scour the surrounding sand, creating erosion zones.

In the study area, most of the stormwater drains to creeks or lagoons, with outlets to the ocean as summarised in Table 12 (see Section 5.5.5). While scour can occur around stormwater outlets, due account of this hazard has been made in the selection of the storm demand value, where appropriate (e.g. Umina Beach and Pearl Beach, see Section 6.2). Within the limitation of the spacing of photogrammetric profiles for hazard definition, natural long-term lowering of beach berms surrounding stormwater outlets is explicitly accounted for in the volumetric analysis defining hazard line positions.

6.7 Climate Change

A discussion on sea level rise associated with climate change was provided in **Section 5.7.2**. The possibility of other effects caused by climate change, such as increases in storm intensities, was discussed in **Section 5.7.3**.

Under the projected accelerated sea level rise, it is expected that shoreline recession will occur at most beaches in the study area, excluding Ocean-Umina beach. This issue was discussed in **Section 6.3**, as part of the discussion on shoreline recession hazards.

6.8 Slope Instability

Beach slope and cliff instability hazards relate to the possible structural incompetence of these features, and associated potential problems with the foundations of buildings, seawalls and other coastal works (NSW Government 1990).

The study area is composed largely of sandy beach and dune areas within the active coastal zone. For such areas, based on Nielsen et al (1992), a number of coastline hazard zones can be delineated as shown in **Figure 46**.

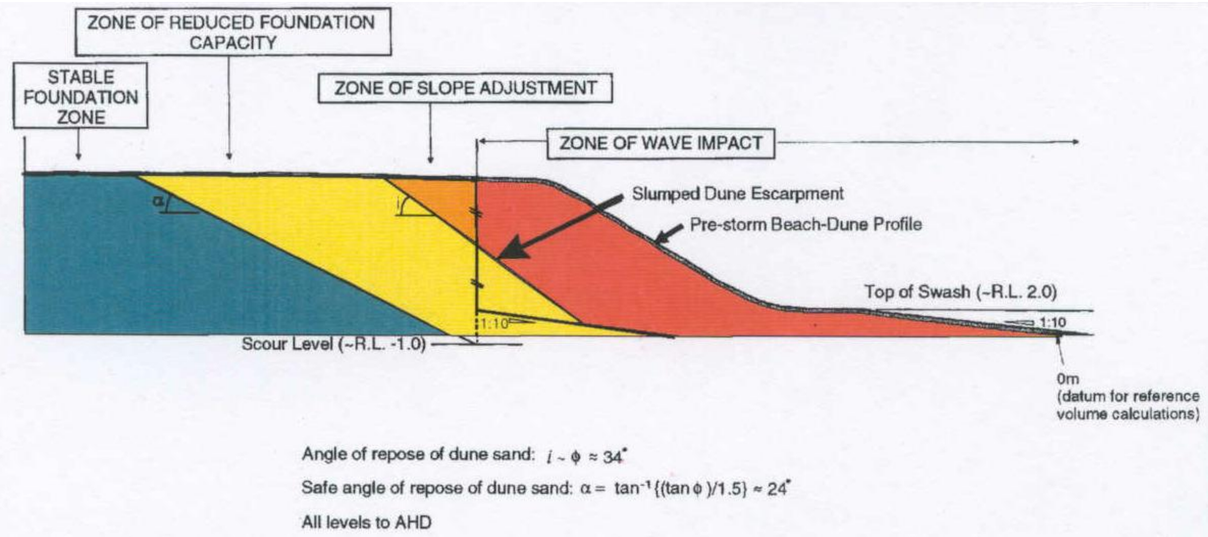

Figure 46: Schematic representation of coastline hazard zones (after Nielsen et al 1992)

The *Zone of Wave Impact* delineates an area where any structure or its foundations would suffer direct wave attack during a severe coastal storm. It is that part of the beach that is seaward of the beach erosion escarpment (as defined by the beach erosion hazard, see **Section 6.2**).

A *Zone of Slope Adjustment* is delineated to encompass that portion of the seaward face of the beach that would slump to a natural angle of repose following removal by wave erosion of the design storm demand. It represents the steepest stable beach profile under the conditions specified.

A *Zone of Reduced Foundation Capacity* for building foundations is delineated to take account of the reduced bearing capacity of the sand adjacent to the storm erosion escarpment. Nielsen et al (1992) recommended that structural loads should only be transmitted to soil foundations outside of this zone (i.e. landward or below), as the factor of safety within the zone is less than 1.5 during extreme scour conditions at the face of the escarpment. In general (without the protection of a terminal structure such as a seawall), dwellings/structures not piled and located within the Zone of Reduced Foundation Capacity would be considered to have an inadequate factor of safety.

Recently, the NSW Government Office of Environment and Heritage re-defined the coastal erosion hazard line that Councils must use for hazard definition studies (OEH 2013). Previously, the landward boundary of the Zone of Slope Adjustment was used to define the hazard line position, while OEH (2013) requires that hazard lines be defined taking into account an allowance for reduced building foundation capacity (i.e. at the landward boundary of the Zone of Reduced Foundation Capacity). This moves the line landward from the previously defined location, which was the landward boundary of the Zone of Slope Adjustment (Figure 47). The difference depends of the height of the dune and is some 10 m for a 6 m high dune and some 20 m for a 12 m high dune.

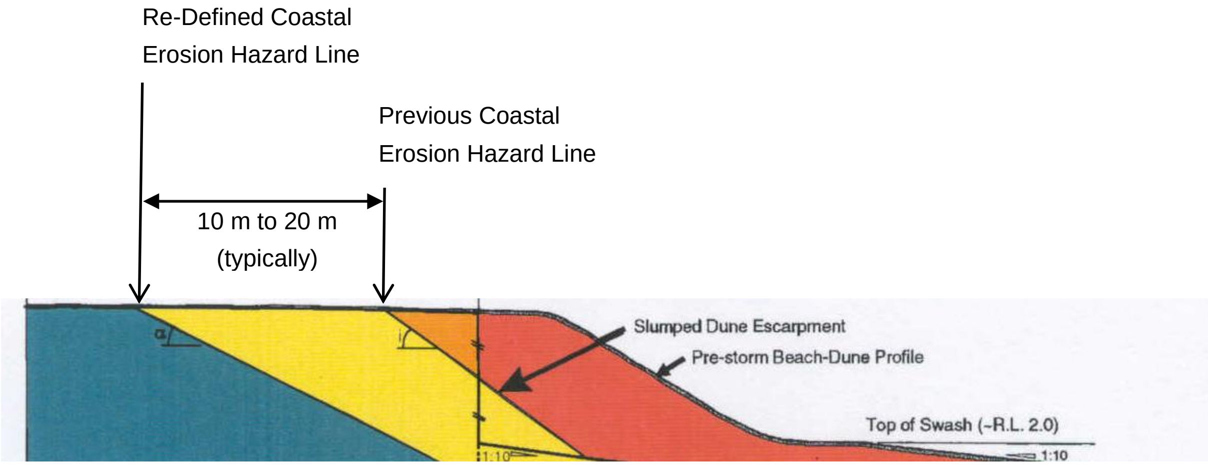

Figure 47: Re-definition of the erosion hazard line

The coastline hazard zones for the study area are determined in **Section 7**, with the position of the Zone of Slope Adjustment and Zone of Reduced Foundation Capacity defined for the immediate, planning period, and the Zone of Reduced Foundation Capacity defined for the 2050 and 2100 planning periods.

Geomarine (1991) found that typical ϕ values were 30° for loose sand, 35° for medium dense sand, and 38° for dense sand. A ϕ value of 33° was adopted for hazard line definition for defining the current hazard lines, which is consistent with the value previously adopted for the study area by PWD (1994) and Patterson Britton & Partners (1998).

A scour level of -1m AHD was adopted for all beaches in the study area, except Patonga, Pearl and Ocean-Umina beaches where a scour level of -0.5m AHD was adopted due to the reduced likelihood for scour at these beaches (based on lower incident wave heights). Similarly, a swash level of 2m AHD was adopted for all beaches in the study area, except Patonga, Pearl and Ocean-Umina beaches where a swash level of 1.5m AHD was adopted.

It should be noted that the presence of clay or other inerodible material in the dune profile is not accounted for in the method described above, which assumes the entire dune profile to consist of sand. Based on geotechnical information collected throughout the study area[53], it is known that clay lenses and rock (located above the scour level of -1m AHD) exists at some locations, which could reduce the extent of the erosion hazard determined herein. In particular, these features are prevalent in the dunes at Forresters Beach. Here the dune is underlain in the main by consolidated bluff material and rock. Drilling at Forresters Beach (Hudson 1997) indicated rock levels above 5 m AHD and higher for the development along Kalakau Avenue north of the intersection of Boos Road. South of Bluewave Crescent the rock levels appear at around 0 m AHD with clay layers at higher elevations

[53] Geotechnical investigations conducted at Avoca, Terrigal-Wamberal and Forresters beaches are outlined in Hudson (1997). Borehole data has also been collected for several coastal engineering assessments at individual properties in the study area.

and generally reaching elevations around RL 9.0 m AHD. Due to the presence of clay and rock at elevated levels within the dune, the coastal hazard zone schema of Nielsen et al. (1992) (Figure 46) cannot be applied to Forresters Beach. The *Zone of Slope Adjustment* and the *Zone of Reduced Foundation Capacity* cannot be determined for Forresters Beach without knowing the geotechnical soul properties of the dune matrix, which can be determined only from subsurface drilling, undisturbed sampling and laboratory testing. These factors preclude the determination of hazard lines for Forresters Beach. A site specific geotechnical investigation in conjunction with a coastal engineering assessment is recommended during the planning stage of any individual lot redevelopment.

7. DEFINITION OF COASTLINE HAZARD ZONES

Coastline hazard lines within the study area have been defined for immediate, 2050 and 2100 planning periods. In all cases, an entirely sandy subsurface was assumed. That is, existing seawalls, dumped rock and other works that are effective (to varying degrees) in limiting storm demand in the study area were ignored for calculation purposes. Similarly, natural bedrock or other inerodible subsurface materials (such as stiff clays) were not accounted for in the analysis in projected zones of coastal erosion. Therefore, coastline hazard lines have not been defined for Terrigal Beach due to the presence of the seawall and rock bluff, which have been assumed to represent the landward limit of coastal hazards in this area. Similarly, coastline hazard zones have not been determined seaward of the rock bluff on MacMasters-Copacabana Beach or at Forresters Beach where bedrock and clay is ubiquitous in the dune, making it impossible to determine the hazard lines.

Table 19 outlines the adopted hazard parameters for each beach in the study area. The position of the 2011 Coastline Hazard Line, 2050 Coastline Hazard Line and 2100 Coastline Hazard Line is thus the predicted position of the back beach erosion escarpment after a 100 year ARI coastal storm in 2011, 2050 and 2100 respectively, including subsequent slumping to a stable angle of repose and an allowance for reduced foundation capacity[54].

It can be seen that total shoreline recession (due to both net sediment loss and sea level rise) is not predicted to occur at Ocean-Umina Beach. As such, the 2050 and 2100 Coastline Hazard Lines at this beach are equivalent to the 2011 Coastline Hazard Lines.

Plans showing the predicted positions of the Coastline Hazard Lines for each beach (except Forresters Beach) and lots that are affected potentially by inundation along the beach front and at lagoon entrance areas are provided in **Appendix M**, as set out below:

- **Figures M1** and **M2** (Patonga Beach);
- **Figures M3** and **M4** (Pearl Beach);
- **Figures M5** and **M6** (Ocean-Umina Beach);
- **Figures M7** and **M8** (Putty Beach);
- **Figures M9** and **M10** (MacMasters-Copacabana Beach);
- **Figures M11** and **M12** (Avoca Beach);
- **Figures M13** and **M14** (Terrigal-Wamberal Beach); and
- **Figures M15** and **M16** (Forresters Beach).

[54] That is, the Hazard Lines do not represent future predicted shorelines, but future predicted erosion escarpments after a 100 year ARI coastal storm.

Two coastline hazard zones are defined, namely the Zone of Slope Adjustment and the Zone of Reduced Foundation Capacity (see **Section 6.8**)[55] for the immediate (that is, present post storm). The Zone of Reduced Foundation Capacity only is defined for the 2050 and 2100 planning timeframes.

[55] The Zone of Wave Impact was also defined as part of the calculations, but is not depicted in the hazard maps.

Table 19: Components of 2011, 2050 and 2100 Hazard Lines

Beach	Location	Design Storm Demand (m^3/m above 0m AHD)	Long Term Recession due to Net Sediment Loss (m)		Long Term Recession due to Sea Level Rise (m)	
			2050	2100	2050	2100
Patonga	Entire Beach	40	0	0	3.4	8.4
Pearl	Block 1	120	0	0	4.8	11.8
	Block 2	120				
	Block 3	120				
	Block 4	150				
Umina	Blocks 1 to 7	100	0	0	0	0
Ocean	Block 1	120				
	Block 2	120				
	Block 3	150				
	Block 4	200				
Putty	Block 1	280	0	0	8.5	21.0
	Block 2	200				
MacMasters-Copacabana	Block 1	200	0	0	13.3	32.8
	Block 2	240	4.2	9.2		
	Block 3	240	4.2	9.2		
	Block 4	N/A	N/A	N/A		
	Block 5	280	4.2	9.2		
	Block 6	280→100 moving north	0	0		
Avoca	Blocks 1 to 5	100→250 moving north	0	0	17.0	42.0
	Blocks 6 to 9	250				
Terrigal-Wamberal	Blocks 1 to 2 (Terrigal)	60→140 moving north	N/A	N/A	14.6	36.1
	Blocks 4 to 7 (Wamberal)	250	8.8	18.8		
Forresters	Entire Beach	180	0	0	-[56]	-

[56] Rock is ubiquitous in the nearshore zone and rock and clay have been found at elevated levels within the foredune, making it impossible to determine long term recession from projected sea level rise.

The landward limit of the Zone of Reduced Foundation Capacity for each of the planning timeframes has been denoted as the "Hazard Line". This is consistent with the *Guidelines for Preparing Coastal Zone Management Plans* (OEH 2013), which require that the beach erosion hazard is defined as a storm bite plus an allowance for reduced foundation capacity.

The previously adopted hazard line for the study area was defined at the Zone of Wave Impact. This previously adopted hazard line (denoted as the "Existing Planning Line") is also shown in Figure M1 to Figure M16 for reference. The previously adopted hazard lines for beaches in the study area correspond with the following:

- Wamberal – Immediate Hazard Line (but landward of proposed revetment footprint)
- Terrigal, Avoca and MacMasters-Copacabana – 2045 Hazard Line
- Pearl – 2098 except along a building line at Coral Crescent
- Putty, Ocean-Umina and Patonga – 2098 Hazard Line

In determining the two types of Hazard Line for 2050 and 2100, the 2011 Zone of Wave Impact positions were translated landward allowing for long term recession due to sea level rise (see Table 19, then the positions of the Zone of Slope Adjustment and Zone of Reduced Foundation Capacity were recalculated.

The hazard lines were determined over the full extent of photogrammetric data, with positions calculated at each photogrammetric profile location. Note that some smoothing of the Hazard Lines was undertaken to avoid significant localised fluctuations in the erosion escarpment position that would be unlikely to be sustained in practice.

8. RECOMMENDATIONS FOR FUTURE WORK

A summary of the recommendations for future work that may be considered for implementation by Council (with support from other agencies as appropriate), is provided as follows:

- Photogrammetric data assessment and acquisition at suitable resolution utilising more recent aerial photography, particularly at Avoca Beach, Terrigal-Wamberal Beach and Forresters Beach, ensuring that all areas of interest, such as the lagoon entrance areas and Killcare SLSC, are covered comprehensively.
- A more detailed assessment of entrance behaviour over time to better define the erosion hazard in these areas by the collection and analysis of photogrammetric data along bank-normal profile lines in each of the lagoon entrances, similar to that undertaken for the beaches.
- Geotechnical data be obtained for Forresters Beach to allow for the determination of the Zone of Slope Adjustment and the Zone of Reduced Foundation Capacity.

9. CONCLUSIONS

This report has presented a detailed technical study and risk assessment using an updated photogrammetric database to quantify empirically the coastal hazards of the Gosford open coast and Broken Bay beaches. Numerical wave transformation modelling has been used to estimate the relative wave energy at each of the beaches and help verify the empirically derived values of storm cut adopted for each beach.

The photogrammetric data has indicated that the beaches are generally not undergoing long term sediment loss, with low rates of long term recession recorded at Pearl, Wamberal and MacMasters/Copacabana beaches. However, the prognosis for a future sea level rise, as a result of global warming, could increase the rate of long term recession. An analysis of long term beach recession due to sea level rise has been undertaken for each beach using the Bruun Rule, with reference to site-specific sedimentological and nearshore bathymetric data.

Coastline hazard lines (representing the predicted extent of erosion for a severe coastal storm) for the study area were last defined in 1994 for open coast beaches and in 1998 for Broken Bay beaches, and adopted as planning controls for development. However, these lines did not take into account the sea level rise planning benchmarks adopted recently by Gosford Council, nor did they make any allowance for reduced foundation capacity as required now by the *Guidelines for Preparing Coastal Zone Management Plans* (OEH 2013).

This report has developed revised coastal hazard lines for 2050 and 2100 future planning periods (as well as the immediate planning period). The revised coastal hazard assessment has found:

- 48 lots in Patonga may be subject to coastal inundation due to wave runup, and the main jetty and boatramp carparks may be at immediate coastal hazard risk. Parts of the coastal access road could be at longer term coastal erosion risk
- 38 beachfront lots at Pearl Beach may be to coastal inundation due to wave runup. Two dwellings and parts of 32 beachfront lots lie within the Immediate Zone of Slope Adjustment, with an additional 21 lots affected by reduced foundation capacity by 2100. Parts of the beachfront access road may be at risk from coastal erosion by 2050
- Two lots at Ocean-Umina Beach may be subject to coastal inundation due to wave runup, with no lots within the Zone of Slope Adjustment or Zone of Reduced Foundation Capacity
- At Putty Beach, the surf club building may be at immediate risk from coastal erosion, with the carparks at the western and eastern ends becoming at risk from coastal erosion by 2100. One lot is partially within the 2100 Zone of Reduced Foundation Capacity
- At MacMasters/Copacabana, one lot was found to be subject to coastal inundation from wave runup. The access road and carpark at the southern end of the beach may be at immediate risk from coastal erosion. Thirteen lots have a portion partially within the Immediate Zone of

Slope Adjustment. The coastal road at the northern end of the beach may be at risk from coastal erosion by 2050. A further 42 lots have a portion partially seaward of the 2100 Zone of Reduced Foundation Capacity limit.

- At Avoca Beach, all the oceanfront lots and lagoon-front lots on the southern side of the lagoon are subject to coastal inundation due to wave runup. A further 15 lots on the northern side of the lagoon entrance and 11 lots at the northern end of the beach are subject to coastal inundation hazard. Approximately 10 lots have a portion within the Immediate Zone of Slope Adjustment, with all oceanfront lots at Avoca being subject to coastal erosion by 2100
- At Terrigal Beach, coastline hazard lines have not been defined, as it has been assumed that the seawall and adjacent rock bluff represent the landward limit of coastal hazards. At Wamberal, over 75 oceanfront lots are affected by the Immediate Zone of Slope Adjustment. There is the potential for Ocean View Drive to be impacted by coastal erosion by 2050, with subsequent breakthrough into Terrigal Lagoon possible. All oceanfront lots at Wamberal have been assessed as subject to coastal inundation hazard.
- At Forresters Beach, no lots are subject to coastal inundation. The Zone of Wave Impact and Slope Adjustment and the Zone of Reduced Foundation Capacity could not be determined for Forresters Beach due to a lack of geotechnical data.

To improve coastline hazard estimates, Council may consider requesting additional photogrammetric data for recent dates of photography, particularly for Avoca, Wamberal and Forresters beaches. Geotechnical data needs to be obtained for Forresters Beach to allow for the determination of the Zone of Slope Adjustment and the Zone of Reduced Foundation Capacity.

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