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GREATER TAREE CITY COUNCIL

Black Head to Crowdy Head

Coastline Hazard Definition Study Volume 1: Report

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8-14 Telford Street
Newcastle East NSW 2300 Australia
Tel: +61 2 4907 5300
Fax: +61 2 4907 5333
www.worleyparsons.com
WorleyParsons Services Pty Ltd
ABN 61 001 279 812

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GREATER TAREE CITY COUNCIL
BLACK HEAD TO CROWDY HEAD
COASTLINE HAZARD DEFINITION STUDY VOLUME 1: REPORT

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PROJECT 301017-00051 - BLACK HEAD TO CROWDY HEAD HAZARD DEFINITION STUDY

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1. INTRODUCTION

1.1 Background

The Greater Taree City Council (GTCC) Local Government Area (LGA) is located on the Mid-North Coast of New South Wales, approximately 250km north of Sydney. The LGA is bounded, to the east, by 47km of coastline extending from Black Head in the south to Diamond Head in the north. A locality plan is shown in **Figure 1**. Along this coastline, are the townships of Crowdy Head, Harrington, Manning Point, Old Bar, Wallabi Point, Diamond Beach, Red Head and Black Head.

The brief for the original Old Bar Coastline Hazard Definition Study (completed in draft in late 2008) defined the study area as bounded to the south by Wallabi Point and extending north to the southern arm of the Manning River South Channel Entrance (Farquhar Inlet). This draft report identified significant information/knowledge gaps relating to coastal processes for the entire LGA. The study area was subsequently extended to include some 32 km of coastline between Black Head and Crowdy Head. The extended study area is shown in **Figure 1**. This report covers the extended study area. A future Addendum will cover the southern end of Crowdy Bay.

Coastal processes have threatened sections of the coast within the extended study area. In particular a number of coastal storms have impacted on the coastline adjacent to Lewis Street, Old Bar. Beach erosion has recently and in the past threatened development and beach amenity along Old Bar Beach. Historically, Manning Point Beach has also experienced significant beach erosion.

1.2 Coastline Management Process

The steps involved in formulating a Coastline Management Plan, in accordance with the NSW Government's Coastline Management Manual (1990), in broad terms include the following:

1. Formation of a coastline management committee.

2. Preparation of a Coastline Hazard Definition Study identifying the type, nature and significance of the various coastal processes and hazards affecting the area.

3. Preparation of a Coastline Management Study to identify management options for the area.

4. Preparation of a Coastline Management Plan consisting of the best combination of options for the area.

5. Development of a Program to implement the Plan.

6. Following amendments to the Coastal Protection Act 1979, which came into effect on 7 February 2003, it is now a requirement that Coastline Management Plans are forwarded to the Minister for Environment and Climate Change for approval. If the Plans are approved, they are currently required to be gazetted by the Council.

GTCC formed the Estuary and Coastline Management Committee in 1989 representing the first step in the process outlined above. This report represents the second stage in the process and defines the hazards that affect the coastline and determines the landward limit of beach erosion escarpments due to the cumulative effects of these hazards for various planning periods. It forms an important step in the overall sustainable management of the coastline by providing baseline information to direct future management options. Completion of the Coastline Hazard Definition Study will enable the preparation of the Coastline Management Study and Plan..

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 Black Head and Crowdy Head and assesses these hazards to determine the immediate, 50 year and 100 year hazard lines.

The hazards examined are those set out in the New South Wales Government's Coastline Management Manual (1990), as listed below:

- beach erosion;
- shoreline recession;
- 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;

Section 3 outlines the data used in the preparation of this report;

Section 4 examines the coastal processes operating in the study area;

Section 5 discusses the coastline hazards affecting the study area, quantifying these hazards where possible; and

Section 6 defines the coastline hazards zones.

Section 7 provides a summary of the findings of the report.

Figures for the 32 km coastline have been divided into nine sections at equal scales. A reference in the text, to Figure 3, may actually refer to the nine Figures covering the study area i.e. Figures 3a, 3b, 3c, 3d, 3e, 3f, 3g, 3h and 3i.

It should also be noted that various historical maps, charts and previous reports refer to Wallabi Point and Saltwater Point inconsistently (usually swapping the locations). For the purposes of this report, Wallabi Point is the low headland at the southern extremity of Old Bar Beach and Saltwater Point is the more southern, prominent headland at the southern extremity of Saltwater Beach (see **Figure 1)**.

2. STUDY AREA

2.1 Site Description

The study area for the original Old Bar Coastline Hazard Definition Study, completed in draft in late 2008 was bounded to the south by Wallabi Point and extending north to the southern arm of the Manning River South Channel Entrance (Farquhar Inlet). The study area was subsequently extended to include some 32 km of coastline between Black Head and Crowdy Head. 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 briefly described below (Extracts from Short, 2007). Beaches are listed from south to north.

Black Head Beach *– is 1.6km long, running in a gentle east-facing arc between Red and Black Heads. It is backed by a single foredune with Black Head Lagoon entrance running across the southern end of the beach* (see **Plates 1-4** in **Appendix A**)*. The small Hallidays Point community occupies the northern slopes of Black Head. The beach receives slight protection during southerly waves from Black Head, resulting in an increase in wave height up the beach to a peak at an average of 1.5m at the north end. The beach has persistent rips along the north-central portion and off Red Head, with rips only forming in the south during and following higher waves, which also produce a strong rip against Black Head.*

Diamond Beach *– runs for 5.5km to the south-southwest from Saltwater Point curving around to trend due south at the southern end. Khappinghat Creek drains against the headland in the north. The Khappinghat Nature Reserve occupies the northern half of the beach. The beach is backed by low foredunes, the southern Diamond Head community, with a small creek draining across the southern end* (see **Plates 5-8**)*. It receives waves averaging 1.5m in the north, decreasing at the very southern end owing to slight protection from Red Head and some reefs off the Head. It has a double bar system in the north-centre and a single bar to the south. The inner bar is dominated by rips throughout its length with 20 or more rips common along the beach, as well as a permanent rip against Saltwater Point in the north, which is supplemented by tidal flows from the creek when it is open.*

Between the southern end of Diamond Beach and Red Head are two small moderately protected beaches lying at the base of cliffs and dominated by rocks and reef. The first begins around the rocks that form the southern boundary of Diamond Beach, and trends to the south as a 500m long, double crenulate high tide sand beach fronted by 50m wide intertidal rock flats, while it is backed by vegetated bluffs rising to 20m. A small headland separates it from Shelly Beach, a similar curving 250m long high tide sand beach and vegetated slopes. Red Head forms the southern boundary. It is *composed of red shales interbedded with thin layers of volcanic ash, together with fossil land plant remains.*

***Saltwater Beach** – is a 1.4km long southeast facing beach located between the low northern Wallabi Point and the southern more prominent 18m high head, Saltwater Point* (see **Plates 9** & **10**)*. Saltwater itself refers to the small residential area on the low northern headland, the access road and the national park at the southern end. A double bar system is maintained usually cut by eight rips, together with permanent rips against both heads.*

***Old Bar Beach**– extends southwest for 6km between Farquhar Inlet and Wallabi Point, with a slight foreland at Old Bar in the lee of the rocky Urana Bombora* (see **Plates 11**–**15**)*. A double bar system generally exists with the usually attached inner bar cut by rips every 200-300m. Old Bar is a small, rapidly growing coastal township with an extensive foreshore reserve adjacent to the town centre. Racecourse Creek entrance, reefs directly off the town and the southern headland (Wallabi Point) cause the beach planform to undulate, with two arcs sweeping away from the reef area.*

***Manning Point Beach** – commences on the untrained southern side of the sandy 500m wide Manning River entrance. It extends southwest, initially as a low sandy 2.5km long spit, then as a steep coarse narrow beach for a total of 10km to the southern mouth of the river, called Farquhar Inlet. This is an isolated, relatively high energy beach, with two rip dominated bars, and extensive river channels, bars and currents at each end. The riverfront town of Manning Point is located south of the spit. The remainder of the beach is backed by a narrow eroding dune then the farmland of Mitchells Island.* See **Plates 11** & **17**.

***Harrington Beach** – runs southwest from Crowdy Head for 5.6km to the northern Manning River entrance wall at Harrington. Between 1885 and 1901, sandstone rock was quarried from the headland and transported by railway along the back of the dunes to Harrington, for construction of the northern entrance wall. The dunes were badly destablised by the 1950s. Stabilisation began in the 1980s resulting in the present stable, vegetated system* (see **Plates 18** & **19**)*. The beach generally has a double bar system cut by rips every 200-300m on the inner bar and more widely spaced rips on the outer bar. A strong permanent rip runs out against Crowdy Head.*

2.2 Historical Setting

2.2.1 General

According to Hunziker, 2002, the Old Bar area was first settled in the 1930s. It is reasoned that Old Bar received this name following the construction of spur walls at the Manning River entrance at Harrington. It is believed that these works firmly established the Harrington entrance as the main entrance thereby relegating Farquhar Inlet to the ‘Old Bar’ as in ‘not used anymore’ (though there is no record of any sizable vessel having ever crossed this entrance). The village to the south of the Inlet was gazetted, as recently as 1973, as Old Bar.

The construction of the northern training wall at Harrington entrance commenced in 1895 and was largely completed by 1899. Rock was quarried at Crowdy Headland and brought to Harrington on a specially built 6km railway track. In the 1920s two angled spur walls were added. A southern training wall and breakwater were never commenced due to cost and the coming of the railway (PWD, 1990). **Figure 2** illustrates the various states of the entrance from aerial photography since 1940.

Hunziker (2002) outlines journal evidence of settlement at Scott's Creek, Mitchells Island in 1850 when a timber merchant erected a sawmill on the site of the present approach of the new Scott's Creek Bridge. Another mill, on the northern end of Scott's Creek was known to have preceded this one and was the first on Mitchells Island. Timber was the main industry at the time. Oyster farms began in Lutherie Bay, an area particularly set aside by the Government for soldiers returning after the Great War ended in 1918.

By 1950 the main industries on the Manning were dairying, mixed farming, fishing, oyster growing, butter and cheese manufacture, timber cutting and light secondary industry.

The reef off the swimming beach at Old Bar was gazetted 'Urana Bombora' in the 1980s to commemorate the sinking, on August 31st 1937, of the S.S Urana on the reef about 500m off Old Bar Beach (Hunziker, 2002). Refer **Figure 1**.

A surf life saving club (SLSC) was established at Old Bar in 1928 and a club house was constructed in a similar location to the existing building The Old Bar airstrip was constructed inland of the Old Bar SLSC in the mid 1920s by Mr George Bunyan (refer **Figure 1**). It was officially designated as an aerodrome in 1930 by the Department of Defence, although it had been used as an airstrip for many years prior to that.

One of the largest floods of the Manning River was in July 1866. This flood broke open Farquhar Inlet, which had been closed for some time (PWD, 1990). A large flood was again experienced in 1929, the height at Taree exceeding the 1866 flood height by 0.6m. The flood waters opened the entrance at Old Bar to a width of about 1.2km. The biggest flood since 1929 occurred in 1950. At Taree the flood height was 1.8m lower than in 1929, but further downstream water levels were higher due to heavy seas. The Farquhar Inlet entrance was again opened in 1954, 1977 and 1978 due to flooding of the Manning River.

Most of the historical coastal processes information available on Old Bar is in reference to Farquhar Inlet and its stability. As outlined in SKP (1982) the entrance has been periodically opened and closed depending on estuarine, coastal and man-made forces. **Figure 2** illustrates the various states of the entrance from aerial photography since 1940.

2.2.2 Sand Mining

As noted in Riedel & Byrne (1981), five sand mining leases historically bordered Diamond Beach, namely PML6, ML16, PML5, ML2, and ML1. These included a lease on the beach reserve about 5 km in length (refer **Figure 3a and b).** Sand mining began in 1972 by Rutile Zircon Mines Ltd, at the far south of the lease areas behind the beach reserve. Mining progressed northwards and was completed in 1975. Beach scraping operations were carried out in May and June 1974 along the north end of the beach. Riedel & Byrne note that in 1980 the appearance of the beach was similar to that of 1963 (before mining) and that incipient dunes had reformed naturally or by re-establishment after mining.

Sand mining operations in the Old Bar area commenced in October 1979 at the 5.2 hectare lease (PL477 - First Rock Gully), shown on **Figure 3c** as ***area 1***. This lease was inland of the dune system and was mined during 1979 and 1980. Another 9.3 hectare lease was granted to RZM in September 1980, which like the former was located south of Old Bar and inland of the dune system, shown on **Figure 3c** as ***area 2***. Mining of this lease commenced in 1980 and was completed by April 1981. Evidence of further incidences of mining along Old Bar Beach are suggested by the existence of an old access track which leads to significant depressions on the landward face of the primary dunes (SKP, 1982) and vegetation disturbance shown as ***area 3***, mined between 1982 and 2000. These areas are indicated on **Figure 3c**. Aerial photography indicates vegetative disturbance in this area during the 1970s. Since the late 1970s sand mining operations have been conducted in the backbeach area of Old Bar by Rutile and Zircon Mines (Newcastle) Pty Ltd (RZM). The sand mining operations were undertaken for the extraction of the heavy minerals rutile, zircon, ilmenite and monazite. A series of mining leases were granted to RZM at the southern end of Old Bar Beach, as shown in **Figure 3c**.

Prior to sand mining operations beginning at Old Bar in 1979, RZM had been operating from 1976 on Mitchells Island. A small floating dredge was used to mine a continuous strip behind the primary dune system from Manning Point to Farquhar Inlet, leaving a 40m wide vegetation strip along the dunes as a buffer zone. Rutile and Zircon Mines (Newcastle) also mined the second sand ridge on the middle-southern end of Mitchells Island between May 1976 and October 1979.

Sand mining also occurred on the back beach to the south of Crowdy Head (refer **Figure 3i**).

3. DATA AQUISITION

3.1 Previous Studies/ Literature Review

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.

3.1.1 General

The Old Bar Coastal Erosion Study by Sinclair Knight & Partners (SKP, 1982) was commissioned by the Public Works Department to undertake:

- a historical review of the region;
- a geological/geomorphological assessment of the area;
- photogrammetric analysis of aerial photography; and
- wave and storm surge analysis.

From these assessments a conceptual coastal processes model for the Old Bar coastline was put forward. The main report findings with regard to coastal processes, are summarised below:

- During the past 6000 years there has been a general erosion of sand from the beaches and a net transport of material to the north.
- All present day movement of sediment occurs within a 1.5km strip of the coastline.
- Photogrammetric analysis showed that Old Bar Beach had eroded at an average annual rate of 0.2 to 0.3m/year (between 1940 and 1980), however it was indicated that the rate dropped to zero at the very southern end of the beach and near the Caravan Park;
- Accretion occurred along Mitchells Island (north of Farquhar Inlet) between 1878 (observed from historical maps) and 1940 which was subsequently followed by an average annual erosion rate of up to 2m/year (between 1940 and 1980);
- Geotechnical investigations found that directly in front of the Caravan Park, rock is located only 2m below the beach limiting potential erosion in this area. This hard layer dips rapidly downward to the south where clay was found 4 to 5m below beach level.

- The offshore rock reef and shoals adjacent to the Caravan Park behave like a control structure impacting longshore sediment transport and wave conditions.
- An average beach erosion loss per storm (between 1940 and 1980) for Old Bar Beach was 0.7m, and for Mitchells Island, 2.5m.
- The 1 in 100 year maximum still water level adopted for the study was 2.6m AHD (this includes wave setup).
- The greatest potential for longshore sediment transport occurs along Manning Point Beach (adjacent to Mitchells Island) with a net direction to the north. In front of Old Bar SLSC the energy is considerably less and within the accuracy of the analysis, equal in both directions. Along Old Bar and Saltwater Beaches the dominant direction of transport is in a southerly direction, although the available energy is much lower.
- A conceptual model of sediment transport was developed and indicated that Farquhar Inlet influences the position of the shoreline along Mitchells Island. When the entrance is open at the northern end high erosion rates are predicted along Mitchells Island. As the entrance moves south the erosion rate decreases and accretion is possible, especially when the entrance is closed.
- Identified sediment sources and sinks included:
    - inland losses due to aeolian transport estimated between 12,500 and 19,000m^3 per year from areas where dune vegetation is sparse e.g. public access points at Old Bar, along the southern end of Manning Point Beach and just north of Harrington.
    - gains and losses around Saltwater Point, Wallabi Point and Crowdy Head.
    - short term shortage of sand associated with Farquhar Inlet and Harrington Inlet entrances and episodic supply from these same locations; and
    - short term onshore/offshore movement of sand resulting from storm activity and subsequent movement of sand back to the beach during calmer periods..

The report also looked at the siting of a proposed sewage treatment works and associated trickle discharge effluent disposal areas behind the dunes along Old Bar Beach. The assessment of potential sites identified two areas which were considered unsuitable, namely, near the Old Bar SLSC and midway along Old Bar Beach. This was due to sparse vegetation cover and low level dunes making these locations particularly vulnerable to high erosion rates and/or flood inundation..

3.1.2 Racecourse Creek

Racecourse Creek discharges to Old Bar Beach, and in the past, it has been associated with coastal erosion issues. Historical evidence given in the “Old Bar Coastal Erosion Study” (SKP, 1982) shows that the location of the Racecourse Creek entrance meandered from just north of Rose Street, to as far as 150m south of Rose Street. The investigation found that the southerly movement of the creek entrance is coincident with creek flood flows and was associated with bank and dune erosion. The erosion of the area to the south of Rose Street made this area more susceptible to storm erosion by waves. A Maccaferri Reno Mattress type structure was proposed by SKP to train the creek and provide local protection against erosion.

A “Coastline Management Plan for Racecourse Creek/Lewis Street” was undertaken by GTCC in 1990. The study area extended from Rose Street in the north to the southern boundary of the Lewis Street properties in the south. As part of this plan PWD were commissioned to carry out photogrammetric analysis of the Old Bar area for the period 1940 to 1989. The findings of this analysis are discussed further in **Section 4.7.2**.

PWD also determined 50 and 100 year coastal impact lines for the study area as shown in **Figure 4.** This was based on allowances adopted by the Sub-committee of GTCC’s Coastline Management Committee for short term storm fluctuations, long term recession and Greenhouse Effect induced sea level rise. The details of these allowances were not outlined in the report.

The Coastline Management Plan included assessment of various options for management of the creek and recommended a plan involving the construction of a gabion wall extending to about the frontal dune alignment.

“Racecourse Creek Entrance at Old Bar, A Management Strategy” (AWACS, 1991) was then prepared to provide the necessary engineering information to facilitate the construction of permanent works which both train the creek entrance and reform and stabilise the adjacent dunal area. As part of the design process, various aspects of the coastal processes affecting the area were discussed. The proposed works included the following:

- Stage 1 entrance training works – construction of a gabion mattress and box gabion training wall to route the creek flow across the beach to discharge into the ocean; and

- Stage 2 dune restoration works – the building up of sand reserves in the (then) present entrance channel south of the proposed training wall, stabilised by planting and fencing with beach access provided.

These works have since been undertaken. Based on the proposed design, the location and shape of the gabion wall is indicated on **Figure 5.**

3.1.3 Diamond Beach Coastal Erosion Study

Diamond Beach is approximately 5 km long and is situated some 7 km south of Old Bar and 15 km north of Forster-Tuncurry. Diamond Beach extends from Saltwater Point in the north to Red Head in the south. The "Diamond Beach Coastal Erosion Study" undertaken in 1981 by Riedel & Byrne Consulting Engineers (1981) investigates coastal processes along Diamond Beach. This study was commissioned by the Public Works Department to look at:

- historical shoreline movement including photogrammetric analysis of historical aerial photographs;
- onshore and offshore geology;
- the effects of sand mining and other human-induced changes; and
- coastal processes and a conceptual model of sediment transport at Diamond Beach.

The main findings of the study are summarized below:

- Diamond Beach was eroding at a rate of 0.4 m/year at the north and south and 0.1m/year in the middle (between 1937 and 1975). The rate of recession had not been historically constant but depended on a combination of storm and flood severity.
- Sand mining had only a superficial effect on the Diamond Beach foreshore and a negligible effect on beach stability.
- Diamond Beach was most vulnerable to coastal erosion at the southern end where the foredune is either low or non-existent (see **Plate 6**) and where indurated sands (coffee rock) are at a low level and set back. Approximately 50m north of Diamond Drive the height of the foredune rises (see **Plate 8**) and indurated sands occur under the foredune at a level where they would retard the rate of erosion, particularly localised erosion during severe storms.

3.2 Review of Historical Aerial Photographs

Aerial photography is available for the Black Head to Crowdy Head coastline dating back to 1940. To assist in gaining an understanding of the coastal processes and to assist in photogrammetry analysis (identifying anthropogenic influences such as sand mining activities, beach reshaping and beach accessway construction) a review of the aerial photographs was undertaken. A summary of some of the distinctive features of various dates of photography is provided in **Appendix C**.

3.3 Photogrammetry

A detailed photogrammetric analysis of historical vertical aerial photography (photogrammetry) and interpretation of the results was undertaken by the Department of Environment and Climate Change. This enabled long term recession rates and storm erosion demand to be assessed.

The photogrammetry data consisted of 269 cross-shore profiles in 22 blocks covering a total coastline length of approximately 32 km from Black Head to Crowdy Head. The data covered the period from 1940 to 2006. **Figure 3** shows the location of the photogrammetric profiles.

Appendix B provides further information regarding the photogrammetry including:

- details of the dates of photography;
- a description of the methodology used in the analysis of the photogrammetric data; and
- tables and plots of analysis results.

3.4 Survey Data

An Airborne Laser (ALS) survey of the LGA was conducted on behalf of GTCC between September 30 and October 3, 2005. Council supplied the 0.5m contour data based on this survey information. The accuracy of this data is +/-0.2m.

Based on the supplied survey data and bathymetrical information from the Admiralty Chart AUS810, Port Stephens to Crowdy Head (Department of Defence, 2001), a Digital Terrain Model (DTM) of the study area was created using the 12D software package**.**

This was used to assist in identifying low lying areas subject to coastal inundation and in developing a Conceptual Coastal Processes Model (see **Section 4.12**).

4. 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 4.1**);
- elevated water levels (**Section 4.2**);
- wave runup (**Section 4.3**);
- coastal storms (**Section 4.4**);
- wave induced currents (**Section 4.5**),
- short term onshore/offshore sediment transport (**Section 4.6**);
- longer term sand movement (**Section 4.7**);
- geotechnical conditions (**Section 4.8**); and
- climate change (**Section 4.9**).

4.1 Wave Climate

Manly Hydraulics Laboratory (MHL), part of the NSW Department of Commerce, 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 (amongst other things) the significant wave height (H_s) and peak spectral wave period (T_p) to be determined. For the NSW network, records are collected for 2048s bursts (about 34 minutes) every hour at 0.5s intervals (Lord and Kulmar, 2001). Waverider buoys can be non-directional or directional. Directional buoys allow the predominant wave direction to be determined.

In the vicinity of the study area, a Waverider buoy is located offshore about 20km north east of Crowdy Head. The Crowdy Head Waverider buoy is a non-directional buoy that has been operating since 10 October 1985. Hourly wave data from this wave buoy was sourced from MHL. The data covered the period from 10 October 1985 to 30 April 2008 with an 86% 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. Wave directions have been hindcast by MHL for the period 10 October 1985 to 31 December 1997 based on interpretation of historical synoptic chart information. Limitations in the accuracy of this hindcast method should be considered when using this data.

Based on analysis of the H_s data at Crowdy Head to 30 April 2008, the probability of exceedance of a particular offshore deepwater significant wave height (H_s) is as shown in **Figure 7**. A log normal distribution was used to extrapolate to the 100 year ARI wave height. From the analysis it was calculated that:

- the average wave height is 1.6m, the median or 50th percentile wave height is 1.5m;
- H_s exceeds 3m for about 5% of the time;
- H_s values exceeding 4m occur less than 1% of the time;
- storm conditions with H_s exceeding 5m occur on average once or twice a year;
- the one day per year (i.e. 1/365.25=0.274% Probability of Exceedance) wave height is 4.8m, the 12-hour per year (i.e. 12/(365.25x24)=0.137% Probability of Exceedance) wave height is 5.0m;
- the largest H_s recorded was 7.35m recorded on 04 March 1995 at 13:00hrs with a T_p of 13.5s, the corresponding H_{max} was 11m;
- for storm durations of 1 hour, 6 hours and 12 hours the 100- year Average Recurrence Interval (ARI) wave height is 8.8m, 7.8m and 7.5m respectively (refer **Figure 8**); and
- the average T_p at Crowdy Head is 9.7s, with about 92% of records having a T_p between 6s and 14s

Beach erosion is strongly linked to the occurrence of high wave conditions with elevated ocean water levels (the latter are discussed in **Section 4.2**). Therefore, inclusion of duration is likely to more accurately describe the severity of a storm in terms of beach erosion, rather than using average recurrence interval (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) although the coincidence of high tide with wave height durations of less than 6 hours is still possible. 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 relationship between H_s, duration and ARI, at the Crowdy Head Waverider buoy (for all data collected up to 30 April 2008), is shown in **Figure 8**.

It is evident that the 100 year ARI significant wave height exceeded for a duration of 6 hours at Crowdy Head is about 7.8m. Note that the corresponding wave height in Sydney estimated by Lord and Kulmar (2001) is about 7.5m.

Analysis of the hindcast directional wave data available from 1985 to 1997 is presented in the wave rose plot seen in **Figure 9.** In summary, this analysis indicates that deepwater waves approach the study area proportionally as follows:

- 6% from the NE;
- 11% from ENE;
- 17% from the E;
- 18% for ESE;
- 24% from SE;
- 13% from the SSE; and
- 11% from the S.

Analysis of the hindcast directional wave data for Crowdy Head indicates the weighted (height and period) offshore directional average to be approximately from the SE at 134°N.

As previously discussed, the accuracy of this data is limited by the methodology employed to hindcast the wave direction. Nearshore coastal processes are highly sensitive to wave direction. The lack of high quality measured wave direction data in the local region is considered a significant data gap hindering the understanding of the processes in the study region.

Analysis of the Sydney directional data from 1992 to 1999 indicated that 34% of waves came from the south-southeast, with 17% of waves from the southeast and 14% of waves from the south. Furthermore, the south-southeast direction was dominant for larger waves (Lord and Kulmar, 2001). Installation of directional Waverider buoys (since 1992 in Sydney) has indicated that the predominant wave climate along the NSW coast is from the south-southeast.

Whilst considering the limitations of the Crowdy Head directional data, the comparison with Sydney directional data suggests the possibility of a more easterly average offshore direction for the study region.

4.2 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 4.3**). Note that sea level is also predicted to rise due to climate change (the Greenhouse Effect). This is discussed further in **Section 4.9**.

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 Black Head and Crowdy Head, the 100 year ARI water level (including astronomical tide and storm surge) adopted was 1.5m AHD. With a 100 year ARI offshore significant wave height of 7.8m (**Section 4.1**), and assuming wave setup as 15% of this wave height, the 100 year ARI wave setup was determined as 1.2m. Therefore, a 100 year ARI total design still water level (astronomical tide plus storm surge and wave setup) of 2.7m AHD has been adopted for this study. It is noted that the combination of 100 year ARI wave height and water level is conservative though not unrealistic and as a joint probability analysis is beyond the scope of this study such an assumption is considered appropriate. This design level does not include climate change considerations which are further discussed in **Section 4.9**.

4.3 Wave Runup

Wave runup is site specific, but typically is about 3m to 6m above the elevated still water level (**Section 4.2**) on the NSW open coast. The height of wave runup on beaches depends on many factors including (NSW Government, 1990):

- 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 but are not available for the study area.

Hanslow and Nielsen (1995) provide guidance on calculating wave runup. They found that the runup above the still water level was given by:

$$R = 0.9H_s \left(\frac{L_s}{H_s} \right)^{0.5} \tan \beta \qquad (1)$$

where R is the runup exceeded by 2% of waves, H_s is the significant wave height, L_s is the significant wave length, and $tan\beta$ is the beach slope. The significant wave length is given by:

$$L_s = \frac{gT_s^{\ 2}}{2\pi} \qquad (2)$$

where g is the gravitational acceleration (9.8ms^{-2}) and T_s is the significant wave period. Note: wave setup is implicitly included in this calculation of wave runup.

Along the coastline between Black Head and Crowdy Head the 100 year ARI H_s is 7.8m (**Section 4.1**), and T_s can be assumed to be equal to 12s, as is commonly used in coastal engineering design. Assuming that the beach face slope is equal to 1H:10V, as is common in an eroded profile, the predicted runup above the still water level is 3.8m. With a still water level of 1.5m AHD (**Section 4.2**), the predicted 100 year ARI wave runup level exceeded by 2% of waves is 5.3m AHD. Taking into account predicted sea level rise, the wave runup values are therefore approximately 5.5m, 5.9m and 6.2m AHD (for the low, mid and high range sea level rise scenarios). For planning purposes, it is considered that a runup level of 6.2m AHD should be adopted for the study area, which includes the predicted sea level rise of 0.9m over a planning period of 100 years (high range scenario). Refer to **Section 4.9** for discussion on sea level rise.

Runup levels in the order of 6m AHD 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 the existing dune profile would 'roll back'.

4.4 Coastal Storms

4.4.1 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 4.2**). 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 4-1**.

Table 4-1: Categorisation of coastal storms in NSW by PWD (1985, 1986)

Category	Offshore significant wave height (H_s), 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 (1985), all Category X, A, B and C storms that were predicted to have occurred between 1880 and May 1980 were listed[1], 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 Mid-North sector covered the NSW coastline north from Sugarloaf Point (near Seals Rocks) to Smoky Cape (near South West Rocks), placing the study area within this sector.

[1] However, the only reliable data for statistical analysis was from 1920 to 1944 and 1957 to 1980.

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

4.4.2 Storm Types

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

- 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 10**.

Based on PWD (1985, 1986) it is evident that on average:

- the Mid-North Coast sector receives a relatively high rate of coastal storm incidents with only the Central Coast sector receiving significantly greater coastal storm incidents. This is due to these sectors being influenced by storms originating in both the tropical and southern area, as well as those developing locally;
- easterly trough lows and tropical cyclones are the dominant storm types on the Mid-North Coast[2], however southern secondary lows can also affect the area; and
- most storms on the Mid-North Coast occur in Summer, Autumn and Winter, with June and March being the most prevalent months for storms (tropical cyclones generally only occur between January and April, with easterly trough lows dominating between April and July).

[2] These storm types are predominantly weather systems that come from the north.

4.4.3 Storm History

As noted in **Section 4.4.1**, PWD (1985, 1986) listed all Category X, A, B and C storms that were predicted to have occurred between 1880 and 1985.

Storm history information, derived from the offshore Crowdy Head wave buoy was also obtained from Manly Hydraulics Laboratory, NSW Department of Commerce (MHL, 2008). DECCW is acknowledged as the owner of this data. The information was for events where the significant wave height exceeded 3m since the commissioning of the Waverider buoy in 1985, up until the end of 2007. Further information on this data is given in **Section 4.1**.

A listing of the predicted Category X storms from 1940 to 1985 is given in **Table 4-2**, including the storm type. The Category X storms measured at the Crowdy Head Waverider buoy from October 1985 to January 2008 are listed in **Table 4-3**, with the recorded H_s (at the peak of the storm) and T_s values also shown.

A total of 12 Category X events were recorded at the Crowdy Head Waverider buoy from October 1985 to January 2008. This represents an average of 1 Category X event every 1.9 years, in the 22 years of record. However, the time period between storms was not uniform. For example, there were no Category X storms from 1991 to 1994, and 3 Category X storms in 1990.

Table 4-2: Mid-North Coast Category X storms, 1940 to 1985 (PWD, 1985, 1986)

Date	Storm Type
12-15 October 1942	Easterly Trough Low
10-13 June 1945	Easterly Trough Low
18-19 January 1950	Inland Tough
8 June 1951	Easterly Trough Low
14-15 June 1952	Continental Low
19-22 February 1954	Tropical Cyclone
18-23 February 1957	Tropical Cyclone
20-24 January 1959	Tropical Cyclone

Date	**Storm Type**
29-31 January 1967	Tropical Cyclone
23-24 July 1968	Southern Secondary Low
24-25 August 1969	Easterly Trough Low
22-25 July 1971	Continental Low
18-20 March 1978	Easterly Trough Low

Table 4-3: Category X storms measured at Crowdy Head from 1985 to 2008

Date	**Peak H_s (m)**	**Mean T_s (s)**
8-11 February 1988	6.5	10.6
7-10 March 1990	6.3	10.4
28-30 May 1990	6.7	9.2
12-15 October 1990	6.4	11.1
2-5 March 1995	7.4	9.9
6-8 March 1995	6.3	10.4
9-12 May 1997	6.3	10.1
22-25 April 1999	6.5	11.2
13-17 July 1999	6.8	10.5
28 -29 July 2001	6.3	11.9
29 June - 2 July 2002	6.3	11.7
13 -17 May 2005	6	9.2

Based on the available Crowdy Head Waverider data, as documented in MHL (2008), a total of 24 Category A events were recorded from October 1985 to January 2008. This represents an average of approximately 1.1 Category A events per year, in the 22 years of record. However, the time period between storms was not uniform. For example, there were no Category A storms in 1993, 1997-1998, 2000-2003 and 2006, and three Category A storms in 1989 and 2004.

4.4.4 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[3]. According to Airy theory, the total wave energy in one wavelength per unit crest width is given by (in deep water):

$$E = \frac{\rho g H^2 L_o}{8} \qquad (3)$$

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:

$$L_o = \frac{gT^2}{2\pi} \qquad (4)$$

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 1985 to 2008 period, which had measured wave data, the storminess indicator was calculated using a wave power calculation. The data included durations for each discrete 0.5m increment that H_S exceeded 3m. 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.

[3] 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).

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

$$P = \frac{E}{L_o}\frac{gT}{2\pi} \qquad (5)$$

where all variables have been previously defined (refer to Equation 3). 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)[4].

For the 1985 to 2007 period, E_s was determined for each storm as described above. 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 1986 to 2007 inclusive are shown in **Figure 11**

It is evident that 1988, 1989, 1990 and 1999 were notably stormy years within the 1986 to 2007 period. Conversely, the 1991 to 1998 period was particularly calm with the exception of 1995.

The relationship between the cumulative storm energy per unit wave crest width (E_s) for 1985-2007 (from which the storminess indicator was derived) and a selection of dates of aerial photography listed in **Appendix B** is shown in **Figure 12.**

Note that the storminess indicator is only an approximate measure of beach state, as water level and wave direction are very significant factors in defining the erosive potential of storms. 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.

[4] 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.

4.5 Wave Induced Currents

The most common forms of wave induced currents are longshore currents and rip currents.

Longshore currents occur within the breaker zone and move essentially parallel to the shoreline, they are usually generated by waves breaking at an angle 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, there may be times when the littoral drift is in one direction and at other times when it is in the opposite direction.

There is a net south to north longshore movement of littoral sand within the surf zone of most NSW beaches north of Newcastle. As such, Mid-North Coast beaches are generally supplied by sand from the south and are the source of sand for beaches to the north. This net northward movement of sand is caused by the dominant SSE wave climate (**Section 4.1**) in relation to the general NSW coast orientation of NNE to SSW, and is particularly pronounced in northern NSW as headlands are less prominent. Storm waves can also carry sand around headlands.

Where there is a longshore variation in the rate of longshore sand transport, there will be a net gain or loss of sand from the beach compartment. That is, where more sand is transported out of a beach area than is being brought in over an extended period of time, the beach will erode with the shoreline gradually realigning. The erosion will occur initially in the surf zone where sand transport is greatest, and manifest as beach retreat (recession) following onshore/offshore readjustment of the nearshore profile (WBM, 2003).

On a dominant northwards longshore transport coastline, based on Stephens *et al* (1981), shoreline evolution was predicted to occur as recession commencing at the southern end of the compartment forming an embayment between controlling headlands/features. Within each embayment, recession was also expected to be highest in the southern hook, reducing northwards to negligible rates immediately south of each headland/feature. Long term recession rates were expected to be minimal within small pocket beaches contained between controlling headlands/features. The evolution of zeta form embayments[5] between controlling headlands was considered to be the result of this longshore transport process (see **Figure 13** and **Figure 14**).

[5] Many sections of coastline which are situated in the lee of a headland, feature a curved shoreline geometry. Where sections of coastline are situated between two headlands, and particularly when there is a single, dominant wave direction, the shoreline may likewise assume a curved or "scalloped" shape. In both cases, the curved portion of the shoreline related to the headland(s) is termed a crenulate or "spiral bay". Because of their geometries, these shorelines are also sometimes termed "parabolic," or "zeta-bay" shorelines. The shape results from longshore transport processes which move sediment in the downdrift direction along the down-wave section of the shoreline, and from processes associated with wave diffraction which move sediment in the opposite direction in the immediate lee of the up-wave headland (Rosati *et al*, 2002).

As the shoreline realigned within each compartment, the longshore transport rate was expected by Stephens *et al* (1981) to reduce, with an ultimate reduction in the supply of sand to the next compartment. This would induce a greater transport differential in the next compartment, and cause progressive recession from south to north. The increasing sediment transport rates moving north along the NSW North Coast are consistent with the compartmentisation and zeta form model of Stephens *et al* (1981). It is not possible to determine at what stage the beaches between Black Head and Crowdy Head may be in this process, if at all[6]. The ongoing recession occurring on the beaches between Black Head and Crowdy Head indicates that a stable planform has not developed.

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 a rip typically forms adjacent to the headlands of the various beaches between Black Head and Crowdy Head, 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.

4.5.1 Wave Model

Refer to Addendum (May 2010) for further information, based on new DECCW bathymetric data

A limited investigation into the nearshore wave climate was conducted using numerical wave modelling techniques. The wave modelling was used to transfer typical offshore wave conditions to the nearshore region. This modelling was preliminary only and used for a basic assessment of the nearshore wave patterns and resultant coastal processes under a narrow range of offshore conditions.

Wave modelling was conducted using the STWAVE spectral wave model. STWAVE is a phase-averaged spectral wave model based on the wave action balance equation. STWAVE assumes steady-state conditions over a finite-difference model domain. The model is capable of simulating

[6] It is very difficult to see any evidence of progressive compartmentalisation commencing at the beaches immediately north or south of Old Bar Beach. Farquhar Inlet is an added complexity in this system.

depth-induced wave refraction and shoaling, current-induced refraction and shoaling, depth and steepness-induced wave breaking, diffraction, wave growth because of wind input, and wave-wave interactions.

The STWAVE model domain covered the coastline between Crowdy Head and Black Head, extending offshore to water depths of approximately 70-80m. The model bathymetry, shown in **Figure 6** was based on information contained on navigational charts (AUS 801). A regular rectangular grid with 50m x 50m cells was used for the simulations. The boundary conditions were selected as typical offshore swell wave condition (H_s = 2.0m and T_p=12s) and the model was simulated for two predominant wave directions, east and south-east.

The wave model results, shown in **Figure 15,** highlight the role of the various reefs off the coast in the study area in determining the nearshore wave climate. A dominant feature in the study area is the Urana Bombora. This feature tends to promote wave shoaling and breaking over the reef. The Bombora also acts to focus waves toward the section of the beach in the lee of the Bombora. This location is a popular surfing spot due to this feature. The reefs off Wallabi Point and Saltwater Point also tend to promote wave shoaling and breaking providing some protection from wave action at the southern shoreline. Extensive sand bars have been observed in the lee of these reefs.

Waves simulated from both directions tend to focus at the northern (Urana Bombora) and southern ends of Old Bar Beach, while along the main section of Old Bar Beach wave energy is somewhat reduced. Deeper nearshore bathymetry along this main section of the beach allows swell to penetrate closer to the shore before breaking.

It should be noted that the results of this preliminary wave modelling are limited and can not be used to accurately determine the likely long term wave driven morphological changes at Old Bar. Only a narrow range of offshore wave conditions have been assessed and detailed description of the complex nearshore bathymetry is not currently available. However, the model results do provide information regarding the relative difference in wave heights along the beach and the influence of the bathymetry and control features (such as Urana Bombora and Wallabi Point). This is further discussed in **Section 4.12**.

4.6 Short Term Onshore/Offshore Sediment Transport – Storm Demand

4.6.1 General

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, 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 4.10** for further discussion on aeolian sand movement).

4.6.2 Storm Demand

The amount of sand which can be removed from a beach during a storm event, and transported offshore, is referred to as the "storm demand". This quantity is generally measured above 0m AHD (approximately mean sea level), and is usually expressed as a volume per m length of beach (m^3/m). Knowledge of the storm demand for a beach allows estimation of the amount of material required to be held in reserve for a storm in order to protect a given asset. It also allows 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 is generally measured above 0 m AHD is a reflection of the manner in which the data to describe storm demand has been obtained. Storm demand estimates are typically derived from survey or photogrammetric techniques, where only that portion of the beach above mean sea level is either considered or is visible.

At any location, at any point in time, the storm demand 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;
- 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; and
- state of the tide at the peak of the storm.

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.

4.6.3 Estimate of Storm Demand

The available photography and photogrammetry for the GTCC LGA, does not generally allow an estimate of storm demand to be made, given that no pre-storm to post-storm sequences were captured. Therefore, storm demand must be estimated on the basis of published studies or numerical modelling.

Gordon (1987) estimated that for the exposed NSW beaches the storm demand above 0m AHD for a 100 year ARI event ranged from 140m^3/m to 220m^3/m, for open beaches and rip heads, respectively. In practice, in any one storm, more severe erosion would occur at discrete locations corresponding to the location of major rips.

Numerical modelling techniques are limited in the estimation of storm demand. Typically, two dimensional cross-shore modelling is employed to estimate a storm bite during a synthesised simulation. This can be misrepresentative of actual volumes as complex three dimensional processes (hydrodynamic flow and rip cells) and temporally varying conditions (e.g. a series of closely spaced storms) can not be represented by this simplistic modelling.

In light of these findings and given the uncertainties involved in estimating storm demand, a precautionary approach is deemed appropriate. It is therefore considered that a storm demand of 220m^3/m (consistent with the storm demand for an exposed NSW beach at a rip head (Gordon, 1987)) generally be adopted for all of the beaches, as rip heads could form at any point. Based on engineering judgement a reduced value of 180m^3/m has been conservatively adopted for a 500m stretch of Old Bar Beach, which is afforded some degree of protection, in the lee of the Urana Bombora where a rip head is unlikely to develop.

4.7 Longer Term Sand Movement

4.7.1 General

Longer term sand movement on the beaches between Black Head and Crowdy Head 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. The details of the analysis are included in **Appendix B**.

Caution needs to be exercised in the interpretation of the information due to a number of factors, for example:

- 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.

4.7.2 Previous Assessments of Longer Term Sand Movement

<u>SKP (1982) Assessment</u>

The "Old Bar Coastal Erosion Study" (SKP, 1982) included photogrammetric analysis from 1940 to 1979. The dates of photographs used were November 1940 and January 1941 (considered as one set), January 1965, October 1970 and November 1978. This analysis involved an assessment of the position of the dune escarpment between 1940 and 1979, with the following findings:

- The southern end of Saltwater Beach was stable and the northern end accreted at 0.2m/year (for the period 1965-1979).
- The southern end of Old Bar Beach (at Wallabi Point) was stable and recession rates increased steadily further north to 0.4m/year at approximately the current location of the Meridian Resort. At the Racecourse Creek entrance recession rates increased to a maximum of 1.0m/year. To the north of the creek rates were between 0.2 and 0.5m/year with the exception of the caravan park frontage and the southern bank of the South Arm of the Manning River (some 500m north of the caravan park) where the beach was stable.
- Values were not given for the "Old Bar" (Farquhar Inlet) as they were found to be questionable due to the fluctuating position of the entrance.

- The southern most 1.5km of Mitchells Island eroded at between 1.2 and 2.1m/year. Further north the rate was more consistent, approximately 0.6m/year.

Assessment of the vegetation line, water line and dune blowouts were also undertaken and the following observations made:

- There was a distinct lack of vegetation along the dunes of Old Bar Beach in the early 1940s, with many dune 'blow-outs', particularly between Lewis Street and Farquhar Inlet. This situation progressively improved with vegetative cover being established throughout the frontage by 1980.

- Between 1965 and 1979, the dune escarpment along the entire beach north of Farquhar Inlet to just south of Manning Point receded by approximately 10 to 30m.

PWD (1989) Assessment

PWD were commissioned by GTCC in 1989 to carry out photogrammetric analysis of the Old Bar area as part of the "Coastline Management Plan for Racecourse Creek/Lewis Street". The study area for this plan extended from Rose Street in the north, through to the southern boundary of the Lewis Street properties in the south. This analysis indicated the following:

- A mean coastal recessional trend (1940-1989) of approximately 0.1 m/year. From 1965 to 1989 the mean recession was 0.2 m/year with a maximum of 0.3 m/year at individual locations.

- During the period 1940-1965, Racecourse Creek migrated along a course further south.

- During the period 1940-1965 extensive earthworks were undertaken on land south of, and in the vicinity of, the creek entrance.

- Since 1965 the top of the relic creek embankment had not receded.

- Between 1940-1989 the spit to the north of Racecourse Creek entrance had extended further southwards.

On the open coast south of the creek entrance, the profile information indicated that the frontal dune advanced seaward by up to about 10m during the period 1940-1965. Between 1965 and 1989 the shoreline had progressively receded. Most of the recession had occurred within the 30m road reserve located east of the allotments fronting Lewis Street.

Diamond Beach Coastal Erosion Study

The "Diamond Beach Coastal Erosion Study" (R&B, 1981) included photogrammetric analysis from 1937 to 1980. The dates of photographs used were 1937, 1963, 1970 and 1975. Data for 1980 was also obtained for some sections by ground survey undertaken by the Public Works Department. This analysis involved measurements of beach profiles at 12 sections along Diamond Beach, together with contour plotting of the dune system and the location of erosion scarps along the length of the beach. The findings of the analysis are outlined below:

- The recession rate along the northern and southern parts of the beach was between 0.3 and 0.4m per year.
- In the central part of the beach near an outcrop of 'coffee rock' (refer **Section 4.8.1)** the recession was less than 0.1m per year.
- Within the limits of the study accuracy there was no significant change in the volume of sand on the beach between 1937 and 1975.

4.7.3 Interpretation of most recent DECCW Photogrammetry

Figure 3 illustrates the location of the most recent DECCW photogrammetry profiles for the Study area. Plots of results of the photogrammetric analysis are provided in **Appendix B.**

Trends in shoreline recession can be estimated in two ways:

- by assessment of changes over time in the volume of sand contained within the beach and dune system above 0m AHD *(sediment budget approach)*; and
- by measurements over time of the position of various beach features, such as the position of the back beach erosion escarpment or the position in plan of a certain "cut" level through the foredune.

Both of these approaches have been used for the beaches between Black Head and Crowdy Head as outlined in **Appendix B**. It should be noted that due to the 'poor' quality of the 1940 photography and the sometimes unrealistic profiles derived from this photography, it has not been used in determining long term trends. The longest reliable data set is 1963 or 1965 to 2006.

Plots of block averaged recession rates since 1963/65 have been prepared for all the beaches (refer **Figure B3**, **Figure B6**, **Figure B9** and **Figure B12**). The rates were calculated for the 1963/65 to 2006 period (using all available intermediate dates), and derived by linear regression, that is by determining the line of best fit (least squares error) in each case[7].

[7] This does not imply that there were uniform rates of volume or positional change between the dates of photography.

These plots have all been presented at the same scale so the relative difference between each of the beaches is apparent.

For sand volume calculation purposes, a landward limit near the 2006 sand/vegetation interface was applied to the profiles, in an attempt to discount anthropogenic changes.

The findings of the photogrammetric analysis for each of the beaches are outlined below.

Black Head Beach

An assessment of changes in the volume of sand contained within the beach and dune system at Black Head Beach revealed that there has been a net gain of sand within the beach compartment above AHD (refer **Figure B1**) between 1963 and 2006 of 90,000m^3 or an average of 2100m^3/year, or an average rate of approximately 1.5m^3/m.

An assessment of the movement of a contour level of 3m AHD, indicative of the position of the back beach escarpment at Black Head Beach, (refer **Figure B2**) indicated that the beach has generally been prograding (at rates ranging from 0.1m/year to 0.9m /year), with a couple of isolated areas of minor recession at the southern and northern ends of Block BH3. These exceptions are possible due to anthropogenic changes associated with beach access.

Diamond Beach

An assessment of changes in the volume of sand contained within the beach and dune system at Diamond Beach revealed that there has generally been a minor net gain of sand within the beach compartment above AHD (refer **Figure B4**) between 1963 and 2006 of 21,000m^3 or an average rate of approximately 0.1m^3/m/year. There were some areas where a loss of sand volume had occurred over this period. Most notably this occurred at the northern end of the beach (northwards of the middle of Block D06) and one more isolated example in the middle of the beach (Block D05) possibly due to anthropogenic changes associated with beach access.

An assessment of the movement of various contour levels, indicative of the position of the back beach escarpment, (refer **Figure B5**) indicated minor recession in the south with shoreline position change between -0.1m/year and 0.35m/year (with the exception of the dynamic creek frontage in the south and an isolated beach access location in D03) and minor recession at the northern end (northwards of the middle of Block D06) with shoreline position change ranging from -0.2m/year to 0.1m/year. The central portion was relatively stability with accretion of 0.1m/year, consistent with the presence of 'soft rocks' in this area (refer **Section 4.8.1**). **Figure B6** shows the block averaged change in shoreline position of various contour levels along Diamond Beach from 1963 to 2006. This Figure indicates that the rates of movement (recession) of the beach has been relatively steady over time (with the exception of Blocks D1 and D2, influenced by the creek).

Saltwater Beach

An assessment of changes over time in the volume of sand contained within the beach and dune system at Saltwater Beach revealed that there has been a net loss of sand from the beach compartment above AHD (refer **Figure B7**) between 1963 and 2006 of 41,000m^3 or an average rate of approximately -0.7m^3/m/year. Most of the volume loss occurred within the central portion of the beach (up to -1.9m^3/m/year) with some minor volume gain occurring at the southern and northern extremities.

An assessment of the movement of a contour level of 5m AHD, indicative of the position of the back beach escarpment at Diamond Beach, (refer **Figure B8**) indicated that the beach has generally been receding (at rates up -0.2m/year in the middle).

Figure B9 showing the block averaged change in shoreline position since 1963 indicates that with no recession at the ends of the beach the relative stability (minor recession/erosion) of the beach has been constant over this time period.

Old Bar Beach (to southern edge of Farquhar Inlet)

An assessment of changes in the volume of sand contained within the beach and dune system at Old Bar Beach revealed that there has been a net loss of sand from the beach compartment above AHD (refer **Figure B10**) between 1965 and 2006 of 677,000m^3 or an average rate of approximately -3.3m^3/m/year. Most of the volume loss occurred within the central portion of the beach (from Midcoast Water's treatment plant to Racecourse Creek up to -11m^3/m/year) with some minor volume gain occurring in the lee of Urana Bombora, where the caravan park is situated.

An assessment of the movement of a contour level of 4m AHD, indicative of the position of the back beach escarpment at Old Bar Beach, (refer **Figure B11**) similarly indicated that the beach has generally been receding with rates up to -1.4m/year within a 400m zone directly north of the Midcoast Waters treatment plant with no recession at the southern end of the beach. The rate of recession progressively reduces northwards to around -0.5m/year in front of the Meridian Resort and north of Racecourse Creek to south of the SLSC. In the lee of Urana Bombora minor progradation (0.1m/year) occurred in this period. North of the caravan park recession rates start to increase again up to -1.2m/year at the southern edge of Farquhar Inlet.

Figure B12 showing the block averaged change in shoreline position since 1965 indicates that the rates of shoreline movement have generally been relatively consistent from 1965 to 2000, after which time there is an increase. This is particularly evident for blocks 0B3, 0B4 and 0B5 (i.e. Wallabi Point to Racecourse Creek).

Manning Point Beach

An assessment of changes in the volume of sand contained within the beach and dune system at Manning Point Beach revealed that there has been a net loss of sand from the beach compartment above AHD (refer **Figure B13**) between 1965 and 2006 of 1,385,000m^3 or an average rate of -3.5m^3/m/year. Most of the volume loss occurred within the southern two thirds of the beach (up to -12.2m^3/m/year) with a net volume gain occurring in the northern portion of the beach. Farquhar Inlet and Harrington Inlet have not been considered, as results are not meaningful due to the fluctuating position of the entrances.

An assessment of the movement of a contour level indicative of the position of the back beach escarpment, (refer **Figure B14**) similarly showed recession within the southern two-thirds of the beach (up to -1.9m/year) with net accretion occurring in the northern portion of the beach (up to 2.5m/year).

Figure B12 showing the block averaged change in shoreline position since 1965 indicates that there have been fluctuations in recession in both blocks MP9 and MP10, with greater overall recession occurring in the southern block MP9. The figure also shows that the magnitude of recession occurring within MP9 has generally been greater than that occurring at Old Bar Beach.

Harrington Beach

An assessment of changes in the volume of sand contained within the beach and dune system at Harrington Beach revealed that there has been a net gain of sand within the beach compartment above AHD (refer **Figure B15**) between 1965 and 2006 of 169,000m^3 or an average rate of approximately 0.8m^3/m/year. There was generally a gain in sand volume along this frontage, with the exception of approximately 1500m of shoreline, just north of the middle of the beach where volume losses occurred.

An assessment of the movement of a contour level of 3m AHD, indicative of the position of the back beach escarpment at Harrington Beach, (refer **Figure B16**) similarly indicated that the beach has generally been prograding along the southern half and the northern quarter (at rates ranging from 0.3m/year to 1.0m/year) and receding just north of the middle of the beach (at rates up to -1.2m/year).

Figure B12 showing the block averaged change in shoreline position since 1965 indicates that Harrington Beach in contrast to the Old Bar and Manning Point beaches, receded between 1965 and 1972 then prograded until 1996 and has remained relatively stability to 2006.

Impact of Farquhar Inlet entrance status and Racecourse Creek entrance stabilisation

Initial community and other stakeholder consultation identified two coastal features perceived to have a significant influence on processes in the study area. They are also perceived to possibly hold the answers to the management of the erosion hazards at Old Bar Beach (which has received recent wide spread interest). These two features are

- the status of Farquhar Inlet; and
- works undertaken to stabilise the entrance to Racecourse Creek.

In addition to the long term volumetric and shoreline position analysis of the entire study area, a focused investigation of the possible impact of the status of Farquhar Inlet and the stabilisation works at the entrance to Racecourse Creek was carried out. This involved assessment of three approximately equal time periods, namely:

1986-1993	Farquhar Inlet open. Prior to Racecourse Creek entrance stabilisation
1993-2000	Farquhar Inlet open. Post Racecourse Creek entrance stabilisation
2000-2006	Farquhar Inlet closed. Post Racecourse Creek entrance stabilisation

The sand volume change within these periods is outlined in **Table 4-4**

Table 4-4 Sand volume change for periods 1986-1993, 1993-2000 and 2000-2006.

Section of shoreline	Sand volume change during period (m^3)			Comments
	1986-1993	1993-2000	2000-2006	
Harrington Beach	339,000	246,000	40,000	1986-93 vol estimated from 1986 and 1997 data. 1993-2000 vol estimated from 1986, 1997 and 2006 data. 2000-2006 vol estimated from 1997 and 2006 data.
Harrington spit	-42,000	-5,000	25,000	Estimated through extrapolation of MP10 average. Does not include entrance.
Manning Point Beach Nth (Ch 3000-9400m)	-200,000	-21,000	115,000	MP beach divided in this manner as most plots of recession showed change in behaviour north and south of approx. Ch3000m.
Manning Point Beach Sth (Ch 0-3000m)	46,000	17,000	-305,000	
Farquhar Inlet bar	-150,000	-123,000	156,000	2005 data used rather than 2006. 1986 and 1993 determined from photogrammetric data. 2000 profiles from hydrosurvey and 2005 from lidar survey.
Urana Bombora Nth (0B8 (P1-5))	11,000	-17,000	8,000	
Urana Bombora Sth (0B7)	5,000	-4,100	-10,500	
Old Bar Nth (0B5(P15) to 0B6))	-5,000	3,000	-33,000	Sand redistributed from creek line and lower beach to nourish dunes south of Racecourse Creek. No new sand introduced (AWACS, 1991).
Old Bar Central (0B4 to 0B 5(P1-15))	-20,000	3,000	-70,000	
Old Bar Sth (0B3 (P8-16))	-16,000	-57,000	-84,000	
Old Bar Sth (0B3 (P1-P8))	18,000	-32,000	-19,000	
Total	**-21,000**	**-4,100**	**-238,500**	

Note: shading indicates sand volume gain i.e. accretion. Refer **Figure 3** for location of blocks and profiles, e.g. 0B08 (P15) is Old Bar Beach Block 08, Profile 15

This investigation of the impact of Farquhar Inlet and Racecourse Creek provides some information for assessing the sediment budget between Wallabi Point and Crowdy Head, though it is noted that potentially significant sand loss at Farquhar and Harrington Inlets, aeolian losses, and offshore losses are not accounted for. However, the comparison of the time periods in **Table 4.4** does illustrate some patterns in beach behaviour as follows:

- When Farquhar Inlet was open the southern end of Manning Point Beach accreted whilst the northern end eroded. Harrington Beach also accreted significantly. Conversely, when Farquhar Inlet was closed the reverse occurred i.e. the southern end of Manning Point Beach eroded significantly whilst the northern end accreted. Harrington Beach was still seen to accrete but at an order of magnitude less than when Farquhar Inlet was open.

- When Farquhar Inlet was open the bar or berm separating the Inlet from the ocean was seen to reduce in volume (above 0m AHD). Conversely, when Farquhar Inlet was closed the bar or berm was seen to increase in volume (above 0m AHD).

- When Farquhar Inlet was open (1986-93) the shoreline in the lee of Urana Bombora was generally stable (with minor accretion). When Farquhar entrance was still open (and migrating southwards) this area eroded. Following the closure of the entrance, the area north of Urana Bombora accreted and the area to the south eroded.

- When the entrance was open (1986-1993) Old Bar Beach eroded except the very southern end, which accreted. Since the closure of the entrance, the whole Old Bar frontage has eroded more rapidly except for the very southern end where erosion has slowed but is still significant.

- Following stabilisation of the Racecourse Creek entrance the shoreline directly south of the groyne stabilised. However the southern part of the beach eroded more rapidly.

From the focused volumetric analysis presented above, it is evident that a relationship may exist between the status of Farquhar Inlet and the plan form (related to sediment movements) of the adjacent beach compartments. Further explanation regarding the relationship between Old Bar, Farquhar Inlet, Manning Point Beach, Harrington Inlet and Harrington Beach is outlined in a discussion of the regional coastal processes in **Section 4.12**.

However, the volumetric analysis does not indicate a significant influence resulting from the stabilisation works undertaken at Racecourse Creek entrance. There was short term stabilisation in the immediate vicinity of the works associated with nourishment (beach material redistribution with no external sediment source) and possibly a resulting adjustment of adjacent areas (increased sediment loss). However, following the initial readjustment sediment losses have been significant along the entire length of Old Bar Beach. Stabilisation of the dune area in the immediate vicinity of the creek entrance has been a result of the stabilisation of the creek alignment (relating to catchment flows) and not because of the constructed gabion wall acting as a groyne to stabilise longshore sediment transport. The maintenance of a stable creek entrance location may have a minor influence on coastal processes and beach alignment, acting as a "soft control point" (as occurs with most beach creek outlets). However, this appears to be insignificant relative to regional processes.

4.8 Geotechnical Conditions

4.8.1 Onshore sediments

Geotechnical investigations were undertaken between Saltwater Point and Crowdy Head in 1981 as part of the erosion study of the area (SKP, 1982). Geotechnical investigations were also carried out between Red Head and Farquhar Inlet in 1981 as part of the "Diamond Beach Coastal Erosion Study" (Riedel & Byrne, 1981). The findings of these investigations are summarised below.

The area between Black Head and Crowdy Head consists of Quaternary sediments infilling between a series of bedrock hummocks which are either part of the bedrock hinterland (as in Black Head, Red Head and Wallabi Point) or are isolated inliers of bedrock (as in Saltwater Point, Old Bar and Crowdy Head, refer **Figure 16**). The bedrock is sedimentary including sandstone, mudstones, greywackes, conglomerates and tuffs.

GEOMORPHOLOGICAL CHARACTERISTICS

Five sectors of differing geomorphological character between Saltwater Point and Crowdy Head were identified in (SKP, 1982). These are outlined below.

Sector 1 – Saltwater Point to Farquhar Inlet

The embayments comprise a beach ridge and dune system deposited in front of a bedrock hinterland with an in-filled lagoon behind the barrier system. Offshore rock shoals at Wallabi Point and Old Bar (Urana Bombora) are seaward extensions of the bedrock highs and suggest a degree of compartmentalization which may influence offshore sediment movement.

Sector 2 – Farquhar Inlet

Farquhar Inlet consists of a breached Holocene beach bar with an accumulation of estuarine sediments behind it.

Sector 3 – Farquhar Inlet to Manning Point

Quaternary sediments in this sector are predominantly Pleistocene 'open ocean' type and are bounded to the west, by a bedrock high in the centre of Mitchells Island which is capped by early Pleistocene or late Tertiary gravels.

Sector 4 – Harrington Inlet

Harrington Entrance is characterized by a Holocene breached beach bar behind which estuarine sediments have accumulated. The morphology of Manning Point suggests accumulation of beach sand by northward longshore transport. Gross longshore transport can however occur in either direction and there is a complex interaction of sediment movement around the inlet entrance.

Sector 5 – Harrington to Crowdy Head

Sector 5 is characterized by a wide prograded Pleistocene beach ridge and dune system with an associated lagoon. A broad salient projects out to Crowdy Head. The dune system is mobile in this area with blow outs occurring. The nature of the blow out, in which active dunes are transgressing older dunes rather than older dunes simply degrading, suggests that beach sediment is accumulating and then migrating inland.

GEOLOGICAL CHARACTERISTICS

Borehole investigations (SKP, 1982) revealed the following specific geological characteristics of the Old Bar Beach compartment (all levels given to AHD):

- the 750m stretch of Old Bar Beach from the northern end of Pacific Parade to the caravan park is underlain by shallow bedrock (with gravel layers at approximately RL-0.3m underlain by rock at approximately RL-3m).
- further south along Old Bar Beach there are lenses of heavy gravel patches and soft clays at RL-3 to -4m and firm sandy grey clay of medium plasticity at RL-6 to -8m.
- north of the caravan park a 3-4m layer of clay was encountered from RL-1m. A high plasticity clay was found below this.

During a site inspection (on 13 August 2008) a layer of gravel (from pebbles up to cobble size) was present on most of Old Bar Beach with the exception of the very southern portion (refer **Plate 2** (**Appendix A**)).

The “Diamond Beach Coastal Erosion Study” (Riedel & Byrne, 1981) identified that the Diamond beach area comprises stiff clays (consolidated estuarine mud), overlain by back barrier sands incorporating indurated sand with gravel layers (locally described as ‘soft rock’) which now outcrop along the mid section of the beach under the sand dune escarpment. Dune sands lie on top of the seaward edge of the back barrier sands.

4.8.2 Offshore sediments

From investigations outlined in SKP (1982) offshore sediments from Saltwater Point to Crowdy Head can be categorised into three types as follows:

- Inner nearshore sands – typically clean, light fawn to orange brown, well to very well sorted sand of a fine to medium grain size. The grains are usually sub-angular to sub-rounded, quartzose with a low percentage of rock and shell fragments;

- Outer nearshore sands – characterised as light grey to grey brown, well to very well sorted, very fine to fine, sub-angular quartzose sand with occasional shell fragments; and

- Inner shelf sand – more variable in nature, but generally darker, well to poorly sorted, fine to coarse grained, sometimes muddy with numerous shell fragments and occasional pebbles. Between Saltwater Point and Crowdy Head, the transition between inner and outer nearshore sands generally occurred between the -5 to -10m contours which is roughly 250-300m offshore in the south, and 500m offshore in the north. The transition from outer nearshore to inner shelf sands occurred between -20 to -25m contours, approximately 1 to 1.5 km offshore. Extensive reef areas in the southern area tend to interrupt these generalisations. There were no significant areas of beach sand storage identified other than perhaps Harrington Entrance.

The "Diamond Beach Coastal Erosion Study" (Riedel & Byrne, 1981) identified that at Diamond Beach the nearshore sands occupy a narrow strip parallel to the coastline. There is no evidence of offshore areas of deposition in the form of lobes of nearshore sand extending out from the coast. This indicates that offshore current activity is probably not a dominant feature with respect to nearshore sediment movement in this area. The nearshore sands extend only 250m from the shore and do not appear to be continuous around the reef systems of the headlands.

Figure 17 illustrates the offshore sediment distribution.

4.9 Climate Change

4.9.1 Sea Level Rise

The most recent (Fourth Assessment) estimates for sea level rise due to climate change are provided in Intergovernmental Panel on Climate Change (IPCC) (2007) and Meehl *et al* (2007). Sea level rise estimates were not significantly different to those presented previously in 2001. Based on Meehl *et al* (2007), the latest IPCC sea level rise predictions are shown in **Table 4-5** for the six adopted illustrative emission scenarios (so-called Special Report on Emission Scenarios [SRES] marker scenarios).

Table 4-5: Sea level rise predicted by Meehl et al (2007) from 1980-1999 to 2090-2099 (5% to 95% intervals characterising the spread of model results)

Scenario (IPCC, 2007)	Sea level rise, excluding scaled-up ice sheet discharge	Scaled-up ice sheet discharge	Total sea level rise including scaled-up ice sheet discharge
B1	0.18 to 0.38m	0 00 to 0.09m	0.18 to 0.47m
B2	0.20 to 0.43m	0 00 to 0.11m	0.20 to 0.54m
A1B	0.21 to 0.48m	–0.01 to 0.13m	0.20 to 0.61m
A1T	0.20 to 0.45m	–0.01 to 0.13m	0.19 to 0.58m
A2	0.23 to 0.51m	–0.01 to 0.13m	0.22 to 0.64m
A1FI	0.26 to 0.59m	–0.01 to 0.17m	0.25 to 0.76m

DECC (2007) recommended that low-range, mid-range and high-range sea level rise estimates of 0.18m, 0.55m and 0.91m be adopted for sensitivity analyses in NSW over an approximate 100 year planning period.

The recently released *Draft Sea Level Rise Policy Statement* (DECCW, 2009) adopts a sea level rise planning benchmark. The benchmark's primary purpose is to provide guidance to support consistent consideration of sea level rise impacts, within applicable decision-making frameworks. The NSW sea level rise planning benchmark is an increase above 1990 mean sea levels of 0.40m by 2050 and 0.90m by 2100. The planning benchmark of 0.9m increase by 2100 is similar to the high-range sea level rise estimate of 0.91m previously adopted by DECC (2007).

For this investigation, coastline hazards are estimated for the:

- immediate planning period;
- 50 year planning period with a mid-range sea level rise of 0.2m;
- 50 year planning period with a high-range sea level rise of 0.4m;
- 100 year planning period with a mid-range sea level rise of 0.6m; and
- 100 year planning period with a high-range sea level rise of 0.9m[8].

[8] Given the similarity of the 50 year planning period mid range and 100 year planning period (low-range) sea level rise estimates, it was not considered to be necessary to include both scenarios.

The impact of sea level rise on extreme coastal water levels is discussed in **Section 5.5.**

4.9.2 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[9]. However, this uncertainty should be taken into consideration when assessing the risk and consequences of recession occurring in the future.

4.10 Aeolian (Wind) Sand Movement

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

Sand drift is a result of this aeolian 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

[9] A generally conservative approach was used in the estimation of the other coastline hazards.

foredune[10] into the hinddune), and can result in abrasion, burial, blockage and damage to coastal developments (NSW Government, 1990).

It is important to recognise that dune vegetation is necessary to stabilise dune systems and protect them from wind erosion. Should human and vehicular traffic, or fire (for example) impact on the dunes at Old Bar Beach in the future, there is the potential for landward sand drift to occur, with resulting shoreline recession. As noted by the NSW Government (1990), the likely direction of sand drift (where it occurs) on the NSW North Coast is to the north west.

(SKP, 1982) estimated that between Crowdy Head and Harrington breakwater, approximately 11,000m^3 year of material was being transported inland due to the migration of transgressive dune fields. Calculations of the inland transport potential using local wind conditions predicted a sand loss of 0.5m^3/year/m (SKP, 1982) along the frontage between Crowdy Head and Saltwater Point. Isolated cases of inland dune transgression were noted along Old Bar Beach and extensive movement of sand inland along Mitchells Island (up to 6m^3/year/m) between 1940 and 1965 (SKP, 1982).

Dune 'blowouts' have been evident historically at Old Bar and Diamond Beach as discussed in **Section 3.2**. Since the 1940s the vegetative cover on the dunes has improved significantly. There remain some isolated areas where vegetation is sparse, such as the middle of Old Bar Beach (where sand mining historically occurred) and the numerous accessways in front of the SLSC and caravan park. These areas are vulnerable to aeolian processes.

Saltwater and Black Head Beaches show minimal evidence of historical aeolian sediment transport.

Unvegetated areas such as the Farquhar Inlet berm and Manning Point Beach spit are vulnerable to significant aeolian transport losses as sediment is deposited into the estuaries.

4.11 River Entrances

4.11.1 Farquhar Inlet

The Farquhar Inlet Entrance is approximately 2km and is currently not trained. Coffey and Partners (1981) report flood tide delta sediments extending 3km upstream. The inlet is a sediment sink due to the dominance of flood tide flows coupled with wave induced sediment entrainment in the adjacent nearshore and within the inlet channel. Although some ebb tide scour occurs, the lack of an

[10] The foredune is the larger and more mature dune lying between the incipient dune (generally characterised by grasses) and hinddune area (generally characterised by). 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).

entraining agent (such as wave action) in the upstream end of the entrance region results in the ebb tide sediment outflux being less than the flood tide sediment influx.

Inlet behaviour was determined by PWD (1991) to consist of three distinct morphological stages, breakout, constriction and closure. These stages are discussed further below. A model of inlet behaviour mechanisms showing the morphodynamic processes that operate during each stage is presented in **Table 4-6**. **Table 4-7** presents historical conditions of Farquhar Inlet derived from PWD (1987), aerial photographs and anecdotal evidence.

Table 4-6 Morphodynamic Model for the Breakout, Constriction and Closure of Farquhar Inlet (adapted from PWD, 1991)

Stage	Process	Morphology
Breakout	Flooding of the Lower Manning River. Entrance plug overtopped inducing channel scour (natural) or channel excavated through entrance plug (mechanical).	Entrance channel develops. Overall entrance area scoured. Both dependant on magnitude of floodwater discharge.
Constriction	Floodtide and wave induced currents dominate. Rapid ingress of sediment from adjacent nearshore into entrance area. Ebbtide and small floodwater discharges deflected by accreting floodtide shoals. Sediment influx reduces tidal exchange. Low/ moderate wave energy. Southward littoral drift. Aeolian transport.	Accretion within entrance area. Entrance channel shoals and narrows. Ebbtide and small floodwater discharge channel deflected by accreting floodtide shoals. Ebbtide shield develops. Inlet migrates southwards.
Closure	Moderate/high wave energy. Low catchment rainfall/runoff. Neap tides. Abundant sediment supply in adjacent beach. Longshore sediment transport into entrance. Deposition of wave suspended sediment in entrance mouth. Wave overwashing. Swash process. Aeolian transport.	Wave deposited shoal forms in constricted entrance mouth. Development of an overwash fan. Berm formation on seaward face of entrance plug. Aeolian cap develops.

Table 4-7 Farquhar Inlet Entrance Status

Year	Entrance status	Comments
1940	open	very narrow opening at middle
1941	open	narrow opening towards the south
1942	closed	closed despite significant flooding
1945	closed	closed despite significant flooding
1946	closed	closed despite significant flooding
1950	open	Opened mechanically by local residents.
1954	open	significant flooding
1956	open/closed	open for a flood but apparently closed later
1957	open	opened mechanically
1965	open	open at northern end
1969	open	entrance migrating southward
1970	open	open in middle
1972	open	open just south of middle
1973	closed	closed then reopened mechanically
1974	open	two entrances with main entrance at north. Entrance area and main channel scoured.
1976	open	open towards northern end. Southern entrance closed.
1977	open	Open during flood
1978	open	Open during extreme flood
1979-80	open	wide opening extending from middle northwards
1981	open	narrow opening at northern end (infilling from the south)
1985		
1986	open	narrow opening northern end
1989	open	open near middle
1991	open	open near middle
1993	open	open near middle
1996	open	opening hard up against southern end and a man-made opening in progress at northern end
1997	open	wide opening hard up against southern end
2000	open	narrow opening at southern end (sand has been moving southwards along inlet)
2004	closed	
2005	open	opened mechanically – remained open May (photo) to early Oct (lidar survey) -
2006	closed	
2008	open	possibly open in April (pers. Comm. Evan Watterson, Dan Messiter)
2009	open	opened mechanically in Feb.

Breakout of the Manning River at Farquhar Inlet occurs as flood levels in the lower estuary increase and overtop the entrance plug. An incipient scour channel develops as floodwater flows over the entrance plug. High initial breakout velocities in the order of 3m/s scour and widen this channel

(Gordon, 1981). 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 inlet. PWD (1987) reports that the breakout of Farquhar Inlet tends to occur towards the northern end of the Farquhar Park beach spit. Breakout tends to occur in this location as a result of floodwater discharge from South Channel taking a direct route towards the ocean and overtopping the entrance plug region of the beach spit.

Farquhar Inlet has also been mechanically broken out by local authorities and residents, including oyster growers, via the excavation of a starter channel across the entrance plug during a flood event. The success of mechanical breakout operations is dependent on river flood and ocean water levels.

Following a breakout at Farquhar Inlet, flood tide and wave induced currents re-establish dominance within the entrance channel as the breakout discharge diminishes. Net sediment transport into the inlet then results. Ebb tide and small floodwater discharges are deflected around the accreting flood tide delta and the ebb tide shield is formed. The accumulation of sediment within the inlet progressively reduces tidal exchange between the river and the ocean. Subsequently, scouring of the entrance area is progressively reduced, promoting further deposition. A positive feedback system is therefore generated between tidal exchange and the accumulation of sediment promoting inlet constriction.

The Farquhar Inlet entrance has been observed to migrate southwards towards Old Bar during the constriction stage. The rate at which the entrance migrates southward appears to vary significantly as it is dependent on both floodwater discharge and littoral drift. The migration of the entrance to the south is the opposite direction to the net northward littoral drift which dominates the NSW coastline.

A complex relationship appears to exist between the changes to beach erosion patterns of Manning Point Beach, north of Farquhar Inlet, and the southward movement of the closing entrance. Along this stretch of coastline (whilst the entrance is open) a littoral drift direction node or null point (i.e. point where littoral drift changes direction) is induced by the wave refraction around the extensive reef system of Urana Bombora offshore, and the ebb tide delta of Farquhar Inlet. This is similar to what is usually seen downdrift of a headland, or groyne/breakwater. As the entrance moves southward the interaction of Urana Bombora and the ebb tide delta becomes a less significant feature and the nodal point moves closer to the entrance.

The closure of Farquhar Inlet follows the constriction of the entrance area. Closure tends to occur during high wave energy, neap tide and low rainfall episodes, when there is abundant sediment supply in the adjacent beach and nearshore. Following closure, the influence of Urana Bombora and the ebb tide delta becomes an even less significant feature with the nodal point moving closer to the entrance.

This complex coastal process is explained further in a discussion of the regional coastal processes and a conceptual regional processes model in **Section 4.12**.

Sediment budget

It was estimated in the "Manning River Processes Study" (Webb, McKeown & Associates, 1997) that based on the assumption of three 50% AEP events occurring in the river each year, the annual average flood scour at Harrington Inlet would be in the order of 300,000m^3 and for Farquhar Inlet around 200,000m^3. Hydrographic surveys in 1981 and aerial photography before and after the 1978 flood indicate that over 500,000m^3 of sand was scoured from the marine delta at Harrington inlet during that flood, with a smaller but similar amount from Farquhar Inlet (Webb, McKeown & Associates, 1997). The 1978 flood had an AEP of between 1% and 2% (PWD, 1991).

4.11.2 Harrington Entrance

Harrington Entrance has extensive training walls along the northern bank, terminating in a breakwater constructed around 1904 and spur walls constructed around 1927. Harrington Entrance has been dredged since the late 1800s to maintain navigation channels. The majority of these dredging operations were conducted between 1889 and 1950 by the NSW Public Works Department (PWD). These operations ceased as the use of the Manning River by commercial shipping diminished. In excess of 4.3 million tonnes (approximately 1,600,000m^3) of sediment was removed from the Harrington entrance bar and crossing before 1940 (Rolyat Services, 2003). The PWD Annual Reports on dredging operations indicate that in the order of 100,000m^3 of sand was dredged from the Harrington bar each year from 1908 to 1920.

The sediments within the inlet form a flood tide delta depositional feature. The inlet is a sediment sink due to the dominance of flood tide flows coupled with wave induced sediment entrainment in the adjacent nearshore and within the inlet channel. Although some ebb tide scour occurs as for Farquhar Inlet, the lack of an entraining agent (such was wave action) in the upstream end of the entrance region results in the ebb tide sediment outflux being less than the flood tide sediment influx. Net sediment transport into the inlet therefore occurs.

Floodwater discharges from the river temporarily overcome the dominance of flood tide and wave induced currents within the inlet. As such, floodwater discharges induce the removal of sediment from the inlet to the adjacent nearshore. Floodwater discharges scour the inlet and remove a significant quantity of sediment from the entrance spit. The volume of sediment removed from Harrington Inlet during a flood discharge is probably in the order of tens to hundreds of thousands of m^3 (PWD, 1990).

Long-term sediment infilling of Harrington Inlet was assessed in PWD (1990) using historic hydrographic surveys, aerial photographs, and bar and recorded crossing depths The assessment indicated that the degree of shoaling within the inlet had fluctuated considerably over the previous 130 years, associated with:

- Frequency and magnitude of floodwater discharges;
- Movement of the inlet northward and southward alongshore;
- Channel clearance operations;
- Timing of hydrographic surveys in relation to the above.

It was suggested (PWD, 1990) that the Harrington Inlet sediment budget is in a state of dynamic equilibrium over the short term (10-100 years). That is, the degree of inlet shoaling fluctuates around a modal condition to which it returns following major floodwater discharges or influxes of sediment associated with storm wave events. However, on a geological time scale the estuary will continue to infill with catchment, estuarine and marine sediments.

Harrington Inlet has migrated north and south of its present position over the last 180 years. The entrance has been recorded approximately 100m north (prior to the construction of the northern training wall) and 1200m south against the southern training wall. It is probable that inlet migration is driven by the interaction of both littoral drift and floodwater discharges.

Construction of the training walls and channel dredging between Harrington Inlet and Wingham early last century isolated the Harrington Back Channel and Harrington Lagoon from the main river flow, and at least initially provided a deeper more hydraulically efficient entrance for the north channel. Ongoing dredging helped maintain the improved entrance and channel conditions. The entrance works probably increased flows down the north channel. The effect would be to encourage closure of Farquhar Inlet and to restrict tidal ranges and flows along Scotts Creek and South Channel (Webb, McKeown 1997). The effects of dredging are no longer likely to be significant in terms of estuarine processes because of extensive natural sediment movement and shoaling in the Harrington entrance area. However, it is likely that the entrance works still improve Harrington entrance efficiency.

4.12 Regional Processes – Conceptual Model

The study area from Black Head in the south to Crowdy Head in the north (refer **Figure 1**) forms a regional coastal processes compartment. This regional compartment is highly complex due to the presence of two dynamic river entrances, Farquhar Inlet and Harrington Entrance as well as a number of control features in the form of headlands and bomboras such as at Saltwater Point, Wallabi Point and Urana Bombora. As discussed in **Section 4.11.1**, the influence of some of these features can vary in time depending on environmental factors.

A conceptual model of the regional processes occurring between Black Head and Crowdy Head is presented in **Figure 18**. Further explanation of these processes is provided below (from south to north). Sediment transport rates are indicated by estimates of the average yearly volume change of particular areas. The estimates for volume changes for beach areas have been based on photogrammetry analysis of the sub-aerial proportion of the beach profiles (discussed in **Section 4.7.3**) and a 1:3 ratio of sub-aerial to sub-aqueous volumes. That is, the assessed total volume change in **Figure 18** was four times the measured sub-aerial volume change (above AHD) determined from photogrammetry. This ratio is based on the approach of Rijkswaterstaat (1987) for determining sub-aqueous nourishment volumes for seaward advancement of a beach system. Substantial reef systems have been identified in surveys undertaken by others off the study area. These reef systems may act as a restriction to the development of the full sub-aqueous profile reducing the ratio and therefore the total volumes. However, the volumes presented are indicative of the relative sediment movements in the region. Refer to Addendum (May 2010) for further discussion.

Volume changes at significant sediment “sinks” (e.g. Harrington and Farquhar Inlets) have been estimated based on historical evidence (navigational dredging campaigns), information from previous reports and attempts to balance the sediment budget in the conceptual model.

It is important to note that the conceptual model attempts to summarise the long term average condition. Event based conditions may occur which would appear to contradict this model, especially where complex local coastal features exist, e.g. Farquhar Inlet. Conceptual coastline planform change has been indicated in **Figure 18** based on trends derived from photogrametry analysis of the previous 40 years to aid in understanding of the processes. These are not intended to represent the actual predicted future shoreline position. Additionally, climate change impacts have not been considered in the conceptual model.

Black Head to Wallabi Point (**Figure 18a**)

Black Head Beach is a relatively stable closed system with minor long term accretion. Isolated locations of minor historical recession (0.1 to 0.2m/yr) have occurred in the central to northern portion of the beach possibly due to persistent rips in these locations and/or anthropogenic changes associated with pedestrian access. The long term minor accretion is likely to be due to leaky bypassing around Black Head supplying sediment from the south (Nine Mile Beach) consistent with the net northerly littoral transport along the NSW coast. This bypassing is most likely to occur during large southerly storm wave events.

Diamond Beach is generally stable with minor, long term recession occurring in the south (0.1m/year) and north (0.2m/year). The beach has historically been stable in the centre consistent with the presence of exposed indurated sands or ‘soft rocks’ in this area. Diamond Beach may be described as almost being a closed system. The amount of sediment moving into and out of the embayment is small compared to the general longshore drift along the NSW coast. The large reef system off Red Head appears to be acting as a submerged barrier. Subsequently there is likely to be negligible sand supply from the south and refracted wave energy reaching the beach, stabilising the southern end and reducing the net northward movement of sediment. Similarly the reef system at Saltwater Point acts as a submerged barrier at the northern end of the beach minimising the likely bypassing of sediment around this headland. Bypassing may occur under certain conditions such as: a major flood event where Khappinghat Creek breaks through, moving sufficient entrance bar material seaward; or a large south storm wave event, followed by predominantly southerly waves. A negligible amount of Holocene sediment, on or behind, the foredune indicates that aeolian sediment transport does not contribute significantly to the sediment budget. Similarly offshore sediment sampling indicated a negligible amount of sediment is being lost offshore (Reidel & Byrne, 1981).

Saltwater Beach is similarly a relatively closed system. Saltwater Beach has experienced historical recession of 0.2m/year in the central portion of the beach and is generally stable at the ends. A minor long term sediment loss is likely to be due to leaky bypassing of Wallabi Point to the north, or offshore losses during less frequent storm wave events.

<u>Old Bar Beach to Crowdy Head</u> (**Figure 18b**)

Old Bar Beach from Wallabi Point to Farquhar Inlet is a highly complex and unusual system and as such is presenting some interesting behaviour. The Old Bar frontage between Wallabi Point and Farquhar Inlet has been in a continuous state of erosion since the 1940s with very limited periods of recovery of the lower beach and no recovery of the foredunes. Most of the volume loss has occurred within the central portion of the beach (from the Midcoast Water treatment plant to Racecourse Creek) with some minor accretion occurring in the lee of Urana Bombora along the caravan park frontage. Long term recession rates of up to 1.4m/year are evident within the central zone. The rate of recession has increased in recent years, with the most rapid recession occurring between 2000 and 2006.

On the basis of existing data and observations made on site visits, during both ambient and storm conditions, the following attempts to summarise, in general, what is an extremely complex and variable system.

*Ambient Conditions (**Figure 18c(i)**)*

- There is net northward sediment transport on Old Bar Beach during ambient conditions. At the southern extremity of the beach (Wallabi Point) refraction and diffraction effects cause lateral expansion flow to the south, moving sediment locally to cause accretion against Wallabi Point and infilling of First Rock Gully Creek. This is generally consistent with the findings of an investigation into the distribution of the longshore component of wave energy flux in (SKP, 1982).

- Sediment bypassing of Urana Bombora occurs along the inshore zone (between the sub-aerial beach and the rock reef) when wave conditions are such that the influence of the offshore reef systems is not as significant. This can occur in either a northerly or southerly direction on an event by event basis, with a long term net northerly transport. This mechanism represents a net loss of sediment from the beach compartment.

- Site observations and consultation with local boardriders and beach side residents have indicated that, even when wave conditions are relatively mild, nearshore conditions are characterised by significant longshore currents and high suspended sediment load. It is postulated that due to the relatively deep nearshore area (trough) and steep beach face that typifies the beach profile, waves approach the shoreline relatively unrefracted and break in a narrow high energy surf zone (entraining large amounts of sediment). As a result, waves break at a relatively large angle to the beach promoting high longshore currents with a high sediment load. Longshore transport rates are therefore significant compared to what would be expected considering the incident wave conditions.

*Storm Conditions (**Figure 18c(ii)**)*

- Significant sand movement on Old Bar Beach occurs during storm conditions over a relatively short time period.

- With large wave heights and periods (storm conditions) refracted inshore wave directions, may be significantly different to directions offshore due to extensive reef systems in the nearshore zone, (based on limited wave modelling described in **Section 4.5.1)**. Generally, waves approach the inshore zone more from the east (even for SE events). Due to the SW-NE orientation of Old Bar Beach, small changes in wave direction towards the east (and the associated induced currents) may lead to southerly sediment transport along most of the beach.

- Additionally, during large seas a differential in water levels along Old Bar Beach would occur as a result of wave setup on Urana Bombora (higher water levels) due to the focusing of waves breaking on the reef and the width of the surf zone in this area (as seen in the wave model results discussed in **Section 4.5.1**) relative to the rest of the beach. This differential water level is likely to result in a ‘flow’ of water away from the Bombora setting up a southward flow along Old Bar Beach and creating the potential for southward sediment transport along this frontage (augmenting nearshore wave direction effects).

- The beach state at Old Bar during storm conditions is typically a longshore bar and trough in the nearshore, with a relatively steep beach face. For this typical beach state, waves breaking on the outer bar and the steep beach face are generally of the plunging type, either on to the relatively shallow bar, or directly onto the beach face. Both these mechanisms have significant potential for sediment entrainment in the surf zone. Additionally, the steep beach face and relatively low, narrow beach berm (typical of the profile at Old Bar Beach, see

Figure 18c(ii)) combined with elevated water levels (due to storm effects such as wind and wave setup) allow significant wave runup and overwash across the beach berm to the escarpment. This direct attack mobilises significant volumes of sand from the back beach area. Waves breaking at an angle on the outer bar drive water across the bar into the trough feeding the longshore movement of sediment laden water as it attempts to return seaward. Intermediate rip cells form to facilitate this movement seaward.

- Ultimately, at the southern end of the beach a large rip cell has been observed to develop as the rock reef and shoals at Wallabi Point direct the flow seaward into a deeper channel (as seen in the bathymetry presented in **Figure 6**) carrying sediment offshore. This sand may then be effectively lost from the system. Either because it is too deep and too far offshore to be transported landward during lower energy accretion periods, or the time scale for landward movement is orders of magnitude slower than the storm induced removal. The lack of evidence from the photogrammetry analysis of any significant recovery (or accretion) phase occurring on this frontage suggests that this may be the case.

A recent bathymetric survey undertaken by DECCW indicates this to be the case. See Addendum May 2010.

- Urana Bombora causes wave diffraction/refraction to drive sediment transport in the immediate vicinity from the north and south (of the reef) to form a salient behind the reef. This process removes sand from Old Bar Beach to the south at the same time limiting any sediment supply from the north. This volume of sand in the lee of Urana Bombora can be mobilised to the north during ambient conditions (as described in **Figure 18c(i)**) ultimately resulting in a loss of sediment from the system.

- For purely southerly wave events the loss mechanism is similar to ambient conditions. i.e. net transport to the north, lateral expansion flow to the south locally at Wallabi Point. Losses would be expected to be worst in the portion of the beach just north of the effects of lateral expansion flows (in the vicinity of Mid Coast Water treatment plant). The possible loss mechanism by offshore bypassing (rip cells/offshore sediment pathways) of Urana Bombora during high energy southerly events was assessed following hydrosurvey and seabed character observations undertaken by DECCW – see Addendum May 2010.

There a general lack of sand supply to Old Bar Beach. Very limited supply is likely from the south via bypassing of Wallabi Point and offshore reef systems, and a net loss of sand is likely at the northern end as it bypasses Urana Bombora. The bombora is a submerged feature, not an island, and as such is not an absolute control. Its influence varies depending on incident wave heights and wave periods. Sediment built up behind the bombora during large wave heights/periods can subsequently be transported northward and inshore of this feature during smaller, average conditions. There is also evidence to suggest that significant offshore losses (without subsequent recovery) occur during storm sea conditions, as described above. Morphological features, such as the southward deflection of Racecourse Creek along Old Bar Beach, suggest that this may be the dominant transport mechanism (and loss) for sediment in the upper beach area. The overall sediment deficit results in continual erosion of this segment of coastline.

In the absence of detailed regional coastal models, it is only possible to speculate as to what might be causing the increase in the rate of recession in recent years at Old Bar Beach. On the basis of existing data and observations the following is postulated:

- Typically, an increase in the recession or erosion rate would be associated with a period of increased storminess. From the available data, this does not appear to be the case at Old Bar Beach. It is evident that the period from 1986 to 1996 showed minimal acceleration in the recession rate, whereas the period from 1996 to 2006 showed a steep increase in this rate. From **Figure 11** and **Figure 12** it is evident that the later period from 1996 to 2006 was actually slightly less stormy than the previous decade, from 1986 to 1996. It is noted that the energy per unit wave crest width (E_s) that forms the basis of the data presented in **Figure 11**and **Figure 12** does not account for direction or water level. It may therefore be the case that although the wave energy associated with the storms was lower during the more recent period, the storms may have coincided with higher water levels, creating a higher erosion potential. Wave directions during the storms of the more recent period may have also led to a higher erosion potential.

- It is noted that the period of rapid acceleration of the erosion rate, particularly in Blocks OBO4 and OBO5 (2000-2006), has coincided with a period in which the entrance to Farquhar Inlet was closed. Whilst a direct relationship between these two factors would be overly simplistic, there are likely to be some interactions. As outlined in **Section 4.7.3**, an assessment of the available photogrammetric data for the periods 1986-1993, 1993-2000 and 2000-2006 was undertaken to try to identify any relationship between the status of Farquhar Inlet entrance, and the behaviour of Old Bar Beach (and to highlight any effects, of the stabilisation of Racecourse Creek entrance). Farquhar Inlet (described further below) was found to be a significant sediment sink during and following closure. From photogrammetry it is evident that the closure of Farquhar entrance is associated with growth in height and volume of the entrance berm and a reduction in nearshore sand shoals (ebb tide delta). From an inspection of aerial photography significant infilling of the Inlet (through aeolian processes) also occurs during and following closure. This onshore movement of sand reduces sand in the nearshore zone offshore from the entrance, changing the nearshore bathymetry that otherwise combines with Urana Bombora to stabilise (relatively) Old Bar Beach (supplying sand during southerly bypassing, and reducing the rate of northerly bypassing of Urana Bombora) and Manning Point Beach (as a supply for northerly littoral drift). Both Old Bar Beach and Manning Point Beach south were seen to erode significantly during the period of closure of Farquhar Inlet (2000-2006), relative to a similar period when the entrance was open (1986-1993).

Farquhar Inlet influences coastal processes differently depending on whether the entrance is open or closed (and all the varying states in between these two extremes). To try and conceptualised the general impact of this coastal feature on the adjacent coastline, three representative entrance states are been described as follows:

1. open (entrance located in the northern half of the compartment) – **Figure 18d(i)**;
2. closing (entrance generally in the southern half of the compartment) – **Figure 18d(ii)**; and
3. fully closed (i.e. beach state) **- Figure 18d(iii)**.

These states generally correspond to the time periods assessed in **Section 4.7.3.**

Figure 18d(i) is a schematic indication of the "open" state conceptual processes and is characterised by the following:

- A relatively wide entrance opening, extensive ebb tide delta and offshore bars (associated with greater than average catchment flows). Although flows are higher than average, especially immediately following the opening events, the entrance reverts to a wave dominated inlet form.
- Ebb tide delta and offshore bars create "control features" refracting waves and influencing local littoral sediment pathways. Creating a "loose" pocket beach form in between the two adjacent quasi control features (the entrance and salient which forms behind Urana Bombora).
- Net sediment transport is to the north, bypassing the entrance along the ebb delta. However, the two quasi control features stabilise the immediate area due to wave refraction effects. Immediately downdrift of the entrance (Manning Point Beach) sediment is recirculated back towards the inlet as it rejoins the beach from the ebb delta;
- Counter-intuitively, infilling of the entrance occurs from the north due to this recirculation effect and the ebb flow from the estuary starts to concentrate on the southern bank eroding it in the process. The mechanism starts the migration of the entrance to the south.
- As a result, the southern end of Manning Point Beach is stabilised (or progrades). However, this stabilisation also has the effect of starving the northern portion of Manning Point of some sediment supply (compounded by the loss of sediment from the system as infilling of the entrance occurs) leading to recession potential further to the north.
- A sediment transport direction nodal point forms north of the entrance. At this location net sediment transport reverts to northward;

Note: comparison of mechanical openings (May 2008 and May 2009) with that induced by major flooding (mid to late 1970s), see **Figure 2**, suggests that mechanical opening at specific catchment levels do not produce a full "open" state. Breaching the berm before the peak catchment level is reached for flood mitigation, does not maximise the sediment removal and ebb tide delta formation. This reduces the extent and duration of the ebb tide delta and its influence on coastal processes.

Figure 18d(ii) is a schematic indication of the “closing” state conceptual processes and is characterised by the following:

- A narrower entrance opening (though still significant tidal exchange), ebb tide delta and offshore bars.
- Entrance “control feature” having migrated south combines with the influences of Urana Bombora as a more singular feature (i.e. no pocket beach formation in between and a less pronounced salient feature the further the entrance moves south).
- Net sediment transport is to the north, bypassing the entrance along the ebb tide delta. The now singular control feature stabilises the immediate area due to wave refraction effects. Immediately downdrift of the entrance (now the northern entrance compartment) sediment is recirculated back towards the inlet as it rejoins the beach from the ebb tide delta. There is also corresponding southward shift of the sediment transport nodal point and the Manning Point Beach alignment changes.
- Infilling of the entrance continues to occur from the north due to this recirculation effect and the ebb flow from the estuary is concentrated on the southern bank, eroding it in the process. This mechanism continues the migration of the entrance to the south.

Figure 18d(iii) is a schematic indication of the “closed” state conceptual processes and is characterised by the following:

- No entrance opening, beach state forms and ebb tide delta and offshore bars weld to beach, eventually building the beach berm and ceasing to exist.
- Urana Bombora becomes the only (quasi) control feature, salient forms again without the influence of a nearby open entrance. However, this feature is not as efficient at stabilising the littoral drift without the entrance shoals.
- Net sediment transport is to the north. Despite the entrance being closed to flood flow based sediment ingress, the onshore movement of sediment from the ebb tide delta shoals and building of the entrance beach berm appears to create a more efficient continual loss of sand from the coastal system through aeolian losses (evident from historical aerial photographs). This sediment sink acts to starve Manning Point Beach of sediment supply despite apparent increased efficiencies in sediment bypassing inshore of Urana Bombora.

- The area immediately downdrift of the entrance southern portion of (Manning Point) reacts in classic response to a updrift sediment shortage and recedes.

Farquhar Inlet to Crowdy Head

As noted in **Section 4.7.3** the northern third of Manning Point Beach generally appears to prograde while the southern two-thirds recedes. However, as discussed above this trend can be reversed through short term fluctuations as a result of refracted wave patterns influenced by the state of Farquhar Inlet entrance and Urana Bombora. The state of Harrington Inlet entrance and estuary flow is an added complexity influencing the northern portion of Manning Point Beach.

As outlined in **Section 4.7.3**, an assessment of the available photogrammetric data was undertaken to try to identify any relationship between the status of the Farquhar Inlet entrance, and the behaviour of the Old Bar, Manning Point and Harrington Beaches. It was observed that when Farquhar Inlet was open, the southern end of Manning Point Beach accreted whilst the northern end eroded. Harrington Beach also accreted significantly. If Farquhar Inlet entrance is open due to catchment flow, Harrington Inlet would also be in a state of high catchment flow and relatively “open”. This would favour net bypassing of the Harrington Inlet entrance rather than estuary infilling, growth of the ebb tide delta, reduction of Manning Point Beach spit (on the southern side of the entrance as the entrance widens), recession of the northern portion of Manning Point Beach and progradation of Harrington Beach north of the Harrington Inlet entrance. That is, Harrington Inlet entrance and spit areas act less as a sink to beach sediments. This is conceptualised in **Figure 18e(i)**. It is important to note that this can be an extremely complex system which can not be wholly represented by a simple conceptual model. During high flow events, short circuiting can occur resulting in the development of a new entrance location on the spit (to the south). This was the case between 1989 and 1993 (see **Figure 2h**). However more significant bypassing, a larger ebb tide delta and reduction in sand volumes in the Manning Point Beach spit (and related adjacent beach responses) is still evident.

Conversely when Farquhar Inlet was closed the reverse occurred i.e. the southern end of Manning Point Beach eroded significantly whilst the northern end accreted. Harrington Beach was still seen to accrete but this was an order of magnitude less than when Farquhar Inlet was open.

The planform of Manning Point Beach changes in response to the status of the two entrances acting as “soft” control features. When the Farquhar Inlet entrance is open (particularly when it is open at the northern end), its associated delta and wave refraction effects act to stabilise and essentially hold the southern end of Manning Point Beach out (seaward), to the detriment of the beach further north. Additionally, as the Harrington Inlet entrance is permanently open, bypassing is favoured.

When Farquhar Inlet is closed, the ebb tide delta/shoal is reduced and the sediment sink effect of the entrance berm and estuary compartment (under aeolian transport) acts as the southern control point and the southern end of Manning Point Beach recedes. The northern end of the beach progrades as the spit grows and the Harrington Inlet entrance in-fills, acting as the northern control point. Bypassing is reduced significantly. This northern entrance response is conceptualised in **Figure 18e(ii).** The planform of the whole Manning Point Beach (between Farquhar Inlet Entrance compartment and Harrington Inlet) forms a zeta shape (see Figures 13 and 14), typical of a net northerly littoral transport beach, with limited sand supply in the south.

Harrington Beach has historically shown stability with net accretion occurring between 1965 and 2006. Harrington Beach is supplied with sand from the south within the net northerly littoral transport regime of the northern portion of Manning Point Beach/Harrington Beach area. Sediment transported northward from Manning Point Beach in-fills the Harrington Inlet entrance and bypasses to Harrington Beach, particularly during high river flow events. Historically the planform of the beach has responded to the introduction of the Harrington Inlet training wall (around 1904). As is typical of beaches downdrift (north) of trained river entrances on the Mid and North Coasts of NSW, wave refraction/diffraction effects causing lateral expansion currents prograde the southern portion of the beach, with a corresponding recession of the northern portion (rotating the beach). This planform has evolved towards equilibrium with the long term altered wave climate and little change has been evident noting that the photogrammetry extends back to 1940 while the entrance works were completed in approximately 1904. Progradation of the entire beach compartment occurs relative to the net bypassing of the Harrington Inlet entrance.

5. COASTLINE HAZARD ASSESSMENT

5.1 Overview

The New South Wales Government's Coastline Management Manual (NSW Government, 1990) identifies seven hazards, namely:

- beach erosion hazard;
- shoreline recession hazard;
- coastal entrance hazard;
- sand drift hazard;
- coastal inundation hazard;
- stormwater erosion hazard;
- 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.

The influence of Farquhar Inlet and the trained Harrington Inlet on coastal processes is discussed in **Section 4.11.** The influence of Racecourse Creek entrance on local coastal processes was examined in previous reports, see **Section 3.1.2** and **4.7.2**. Also see Section **4.7.3** for a discussion on the impact of Farquhar Inlet entrance on coastal processes and Racecourse Creek entrance stabilisation works. The other minor creeks/ lagoons in the study area currently discharge adjacent to headlands. The impacts of entrance break through have not been examined because potential break through points are located within Crown land/ Council reserves, or in the case of Khappinghat Creek, national park. Accordingly, no development would be under threat due to this hazard. The inundation hazard associated with these creeks/ lagoons has been considered in **Section 5.5**.

5.2 Beach Erosion Hazard

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 4.6.2.** It has been established, based on previous work by other researchers and experience, that the storm demand or beach erosion hazard for the design storm event at Old Bar beach is 220m^3/m, with the exception of a 500m frontage in the lee of Urana Bombora for which a reduced value of 180m^3/m has been adopted on the basis of engineering judgement.

5.3 Shoreline Recession Hazard

The hazard of shoreline recession is the progressive landward shift in the average long term position of the coastline. The two causes of shoreline recession are sediment loss and an increase in sea level.

Sediment Loss - Recession of a sandy beach is the result of a long term and continuing net loss of sand from the beach system. 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
- there is a landward loss of sediment by windborne transport.

Sea Level Rise - A progressive rise in sea level will 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: deeper offshore waters expose the coast to attack by larger waves; the nearshore refraction and diffraction behaviour of waves will change; a significant volume of sediment will move offshore as the beach adjusts to a new equilibrium profile. Sea level rise is discussed in more detail under The Hazards of Climate Change in **Section 5.7**.

Shoreline recession is typically a long term process which in some cases is imperceptible. Its effect on a beach is often masked by the more rapid and dramatic erosion and accretion that accompanies storm events. Consequently, it can be difficult to identify recession from historical data, even if it extends over many years.

The hazard of shoreline recession is the progressive landward shift in the average long term position of the coastline (NSW Government, 1990). The two potential causes of shoreline recession identified above are discussed in **Sections 5.3.1** and **5.3.2** respectively. It is also appropriate to discount the historical recession due to net sediment loss, taking into account the actual sea level rise that occurred during the measurement period from 1963 to 2006, as discussed in **Section 5.3.3**.

5.3.1 Long Term Recession Due to Net Sediment Loss

Trends in shoreline recession can be estimated in two ways:

- by assessment of changes over time in the volume of sand contained within the beach and dune system *(sediment budget approach)*; and
- by measurements over time of the position of various beach features, such as the position of the back beach erosion escarpment or the position in plan of a certain "cut" level through the foredune.

For the beaches between Black Head and Crowdy Head the most appropriate method of calculating long-term shoreline recession is by measurements over time of the position of a certain "cut" level through the foredune.

This assessment was reported in **Section 4.7.3**. With regional processes affecting the area discussed in **Section 4.12**.

Based on this work the following recession rates were adopted as the long term recession due to net sediment loss (refer **Table 5-1**):

Table 5-1 Adopted recession rates for study area

Location	Adopted recession rate (m/year)	Extent	Comment
Black Head Beach	0	Entire beach	Historically stable
Diamond Beach	0.1	Southern end of beach to Ch 2500m	Transition from 0m/yr at southern rocky headland
	0	Ch 2500m to Ch 3000m	Soft rocks (minor control)
	0.2	Ch 3000m to Ch 4800m	Transition to 0m/yr at northern rocky headland

Location	Adopted recession rate (m/year)	Extent	Comment
Saltwater Beach	0.2	Ch 400m to Ch 800m	Transition to 0m/yr at rocky headlands to north and south
Old Bar	1.4	Central portion of the beach from Ch 900m to Ch1500m	Transition to 0m/yr at rocky headland at Wallabi Point
	0.6	From southern boundary of Meridian Resort to SLSC	Transition from 1.4 to 0.6 from Ch1500m to southern boundary of Meridian Resort
	0.3	From SLSC to Farquhar Inlet	
Farquhar Inlet	Recession rate not relevant as sand spit will fluctuate depending on status of opening/closure.		
Manning Point Beach	1.8	From Ch 2500m to Ch 5000m	Transitioning from 0m/yr at Farquhar Inlet
	1.4	From Ch 5000m to Ch 6800m	Transitioning to 0m/yr at Harrington Inlet
Harrington Beach	0	From Harrington Inlet to Ch 3000m	
	0.6	From Ch 3000m to Ch 4000m	Transitioning to 0m/yr at Crowdy Head

Note: Chainages commence at southern end of each beach.

5.3.2 Long Term Recession Due to Sea Level Rise

Bruun (1962) 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 and deposition of this sediment offshore, followed by re-establishment of the original equilibrium profile. This profile is re-established by shifting it landward and upward. The concept is shown graphically in Bruun (1983), and can be described by the equation (Morang and Parson, 2002):

$$R = \frac{S \times B}{h + d_c} \tag{6}$$

where R is the recession (m), S is the long term sea level rise (m), h is the dune height above the initial mean sea level (m), d_c is the depth of closure[11] of the profile relative to the initial mean sea level (m), and B is the cross-shore width of the active beach profile, that is the cross-shore distance from the initial dune height to the depth of closure (m). This means that the recession due to sea level rise is equal to the sea level rise, multiplied by the average inverse slope of the active beach profile.

Nielsen (1994) found that, based on a synthesis of field and laboratory data and analytical studies (particularly offshore of SE Australia), there were consistent limits of sub-aqueous beach fluctuations, namely water depths (relative to AHD) of:

- 12m ± 4m being the limit of significant wave breaking and beach fluctuations;
- 22m ± 4m being the absolute limit of sand transport under cyclonic or extreme storm events; and
- 30m ± 5m being the limit of reworking and onshore transport of beach sized sand under wave action.

The 12m ± 4m depth can be considered to be analogous to the depth of closure for use in the Bruun Rule, given that it is the limit of significant beach fluctuations, and consistent with formulae for its prediction[12].

Rijkswaterstaat (1987), approximating the work of Hallermeier (1978, 1981, 1983), found the following simplified equation for the effective depth of closure, d_c, namely:

$$d_c = 1.75 H_e \tag{7}$$

[11] The depth of closure is the water depth beyond which repetitive profile surveys (collected over several years) do not detect vertical sea bed changes, generally considered to be the seaward limit of littoral transport. The depth can be determined from repeated cross-shore profile surveys or estimated using formulas based on wave statistics. Note that this does not imply the lack of sediment motion beyond this depth (Szuwalski and Morang, 2001).

[12] The 12m ± 4m depth has been used in almost all known investigations that have utilised the Bruun Rule to determine recession due to sea level rise in eastern Australia, where the depth of closure has been related to the limit of significant long term profile changes, and where rock reef has not been present. A rare exception would be Gordon (1991), who estimated a depth of closure of 37m relative to AHD for the coastline in the vicinity of Belmont Ocean Outfall near Newcastle.

where H_e is the effective significant wave height exceeded for 12 hours per year (that is, the significant wave height with a probability of exceedance of 0.137%). For the Crowdy Head Waverider buoy (representative of offshore wave conditions at Old Bar), from **Figure 7**, H_e is about 4.7m. Therefore, the predicted depth of closure at Old Bar from Equation 7, using this value, is 8.2m.

Bruun (1988) suggested a depth of closure of $2H_b$, where H_b is actual breaker height of the highest waves within a certain time period, namely 50 to 100 years according to Dubois (1992). Arbitrarily using H_e (as defined above) for H_b, the predicted depth of closure is 9.4m using this methodology.

Sedimentological data consistently shows distinct changes in the characteristics of sediments with water depth. These changes include variations in grain size, sorting, carbonate content and colour. The boundary between Inner and Outer Nearshore Sand is typically found at about 11-15m depth (relative to AHD), while the boundary to Inner Shelf Sand (also known as Shelf Plain Relict or Palimpest Sand) is usually at 18-26m depth. The boundary between Nearshore (Inner and Outer) Sands and Inner Shelf Sands correspond to those parts of the seabed considered to be active and relict (Nielsen, 1994).

The boundary between Inner and Outer Nearshore Sand in the vicinity of Old Bar, analogous to the depth of closure, was found to be typically at a depth of approximately 5-10m relative to AHD and 250-300m offshore (SKP, 1982). Nielsen (1994) reported that three studies had identified the boundary between Inner and Outer Nearshore Sand at approximately 10m depth (relative to AHD) in the Byron Bay area. Other investigations by Patterson Britton and Partners (2004a) and others (Nielsen, 1994 and Stephens, 2004) along the NSW mid-north and north coast indicated a depth of closure of 10-11m relative to AHD. Given the available evidence from nearby sites, a depth of closure of 10m below AHD was adopted. Available bathymetric information offshore of Old Bar is limited. The hydrographic chart AUS 810[13] covers the region, but is of coarse accuracy (1:150,000 scale) with depth contours in the vicinity of the depth of closure only at 0, 5, 10 and 15m relative to Chart Datum (approx 1m below AHD). From this chart the distance between the 0m and 10m contour is generally between 500 and 750m offshore. Based on a typical width of the upper beach (from 0m AHD to the top of the dune) of 100m, 850m was adopted as the cross-shore width of the active beach profile. From an analysis of the photogrammetry profiles a typical dune height of 8m was adopted. This is equivalent to a value for the active beach slope (X/(h+dc)) of 47. A conservative beach slope of 50 was therefore adopted. The equation for recession due to sea level rise then becomes R=50r.

Inspection of the active beach width (based on a depth of closure of 10m below AHD) throughout the rest of the study area resulted in the following estimates for active beach slopes:

- Black Head Beach – 1 in 47

[13] AUS 810, "Australia – East Coast, New South Wales, Port Stephens to Crowdy Head", published by the Hydrographic Service, Royal Australia Navy, 14 September 2001, New Edition 17 March 2006.

- Diamond Beach - 1 in 30
- Saltwater Beach – 1 in 42
- Manning Point Beach – 1 in 47
- Harrington Beach - 1 in 45

Taking a precautionary approach, an active beach width of 50 was adopted throughout. In **Section 4.9**, it was noted that mid-range and high-range sea level rise estimates of 0.6m and 0.9m would be adopted for sensitivity analyses for the 100 year planning period. Based on the Bruun Rule, this is equivalent to long term recession due to sea level rise of 30m and 45m respectively over 100 years. Over a 50 year planning period, a 0.2m mid-range and 0.4m high-range sea level rise have been adopted, equivalent to 10m and 20m of recession respectively.

It should however be noted that there are a number of limitations to the accuracy of the Bruun Rule, based on the accuracy of the estimate of the depth of closure.

As described above, there are a broad range of techniques available for estimating the closure depth and several (Hallermeier, Birkmeier, Rijkswaterstaat, USACE, Bruun etc) idealised formulae for estimating closure depth, based on offshore wave statistics. All formulae provide differing results.

The various techniques used for estimating the closure depth for application of the Bruun rule are generally dependent on wave data. Limited availability of complete local wave data sets and variations in wave statistics from year to year therefore also limit the accuracy of the Bruun rule results. The use of historical information to assume future wave statistics also limit the application of these techniques. Particularly given postulated changes to wave climate due to climate change.

It is therefore appropriate to consider a sensitivity analysis for this element of the Bruun Rule. It is common for the active beach profile slope to fall in the order of 1:50 to 1:100 for the east coast of NSW. The range of recession due to sea level rise then becomes R=50r to 100r. It is therefore possible for the recession due to sea level rise to be up to double the amount determined using the adopted active beach slope of 1:50.

5.3.3 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, in this case from 1965 to 2006 (the period over which the long term recession rate was determined). If this contribution is significant, the historical recession rates should be adjusted (discounted) to represent recession due to sediment loss only.

This is because, in the prediction of the future position of the coastline, shoreline recession due to net sediment loss and shoreline recession due to sea level rise are calculated separately.

Based on analysis of average annual water levels at Fort Denison in Sydney Harbour from 1887 to 1987, the NSW Government (1990) estimated that the mean sea level rise over the 101 years of record was 0.5mm/year. More recent estimates by Church *et al* (2001) indicated that for the two water level recording stations with the longest records in Australia (both in excess of 80 years), at Sydney and Fremantle, the observed rates of relative sea level rise were 0.86 ± 0.12 mm/year (from 1915 to 1998) and 1.38 ± 0.18 mm/year (from 1897 to 1998) respectively[14]. The Department of Defence (1999), cited in Nielsen *et al* (2001), estimated that the rate of relative sea level rise at Newcastle (on the NSW Central Coast), from 1967 to 1999, was 1.18mm/year. Averaged around Australia, the relative sea level rise from 1920 to 2000 was about 1.2mm/year (CSIRO Marine Research, 2004).

Adopting a rate of relative sea level rise of 0.86mm/year from 1965 to 2006, represented a sea level rise of 35mm over this period. Using the adopted inverse slope of the active beach profile of 50 (**Section 5.3.2**), this was equivalent to a reduction in shoreline recession of about 1.8m, that can be accounted for (that is, subtracted from the calculated total recession, or added to the calculated total accretion). This is equivalent to a recession of 0.04m/year. Given the predicted low rate of recession due to historical sea level rise, together with the uncertainties in the Bruun analysis, no discounting of measured recession, to account for sea level influences, is considered warranted.

5.4 Sand Drift Hazard

As noted in **Section 4.10,** 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[15] into the hinddune), and can result in abrasion, burial, blockage and damage to coastal developments (NSW Government, 1990). The likely direction of sand drift (where it occurs) on the NSW North Coast is to the NW.

As outlined in **Section 4.10** Harrington Beach has historically lost approximately 11,000m^3 year of sand due to the migration of transgressive dune fields. Manning Point Beach has similarly

[14] Corrected for land movement, the absolute rates of sea level rise at Sydney and Fremantle were about 1.2 and 1.6 mm/year respectively.
[15] 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).

experienced extensive movement of sand inland. These areas are vulnerable to aeolian processes. Further south, there are some isolated areas where vegetation is sparse, such as the middle of Old Bar Beach (where sand mining has historically occurred) and the numerous accessways in front of the SLSC and caravan park. These areas are also vulnerable to aeolian processes and future management strategies for these areas will need to take into consideration this potential mechanism of sand loss from the system.

As noted in **Section 4.10**, the unvegetated areas of Farquhar Inlet berm and Manning Point Beach spit are particularly susceptible to aeolian sediment transport losses into the adjacent estuaries, resulting in loss of sediment from the system and subsequent recession of the shoreline.

5.5 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 4.2** namely storm surge (wind setup and the 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. A 100 year ARI total design still water level of 2.7m AHD has been adopted for this study. For long term planning purposes, sea level rise (as outlined in **Section 4.9**) would also be included. For the mid and high range sea level rise scenarios, this would bring the total elevated water level over 100 years to 3.3m and 3.6m respectively.

During storm events individual waves cause further temporary water level increases above the still water level due to the process of wave setup and runup or uprush (**Section 4.3**). The wave runup values adopted were given in **Section 4.3**, namely 5.9 and 6.2m AHD (for the mid and high range 100 year ARI sea level rise scenarios).

The areas potentially affected by coastal inundation have been illustrated in **Figure 19**. From this figure it is evident that the dune system is generally sufficiently high to prevent inundation due to elevated water levels and wave runup. Where the wave runup is likely to overtop the dune a zone of ‘potential overtopping due to wave runup’ is indicated. In the inland creek areas where there is more limited potential for wave activity to penetrate, a zone of ‘potential minor inundation due to wave action’ is depicted. These zones are not intended to accurately describe the inland extent of the water flow as this is a complex process with many variables, but rather indicate areas that are more vulnerable to inundation now, and into the future as the beaches recede and sea level rises. The areas potentially affected by this hazard are outlined below:

1. The entrance to all the creeks in the study area including the Black Head Lagoon, north (Khappinghat Creek) and south ends of Diamond Beach, south end of Old Bar Beach (First Rock Gully Creek) and Racecourse Creek. The shallow creek mouths would be affected by wave runup and the design elevated ocean water levels as outlined above, would penetrate upstream in the creek.

2. The northern half of Black Head Beach (see **Plate 14**) where the dune height is approximately 5m AHD would experience overwash due to wave runup.

3. The southern end of Diamond Beach (see **Plate 6**) including the southern end of Golden Drive (up to Diamond Drive).

4. Near the middle of Old Bar Beach the dune system is low lying as a result of historical sand mining activities (refer **Plate 13**). With a dune height of approximately 5m AHD this area would experience overwash due to wave runup. It is noted also that the area behind the dune system is also low lying at approximately 4.5m AHD.

5. The entrance to Racecourse Creek. The shallow creek mouth would be affected by wave runup and the design elevated ocean water levels as outlined above would penetrate upstream in the creek. The dunes fronting the Lewis Street properties are also low lying and wave runup could potentially reach the current seaward edge of these properties.

6. The various accessways to Old Bar Beach near the SLSC (see **Plate 15**) are low lying and therefore potentially affected by wave runup. Inundation is not currently likely to reach any building assets in this area.

7. The low lying area at the southern end of the entrance to Farquhar Inlet.

8. Some isolated low lying dunal areas along Manning Point Beach. There is also likely to be flooding of the Manning Point Beach coastal areas from elevated water levels within the Manning River. This inundation from both the river and ocean sides may result in a thin strip of dry sand dune along this frontage.

9. Low lying sections of Harrington township would potentially be inundated by elevated ocean water levels penetrating upstream in the Manning River. Inundation around Harrington is affected by the entrance structures and their future maintenance and repair.

10. The southern 1.5km of Harrington Beach with low lying dunes (approximately 5m AHD) would experience overwash due to wave runup.

As discussed in **Section 4.3** runup levels in the order of 6m AHD would only be realised if the foreshore was at this runup height or higher. In reality, any waves that overtopped dunes or creek banks would fold over the foreshore crest and travel as a sheet flow at shallow depth, spreading out and infiltrating over landward areas. A significant reduction in the velocity and depth of runup would be expected within 10m of the foreshore crest. In addition, wave runup and overtopping is generally episodic, occurring around the peak of the high tide. The affected areas would however become more vulnerable to inundation in the longer term as beach recession occurs and sea level rises.

It should be noted that local catchment flooding has not been considered in determining the potential inundation hazard at the creek entrances, as this is outside of the scope of this study.

5.6 Stormwater Erosion Hazard

Most of the stormwater within the study area drains to creeks or rivers with outlets to the ocean as shown in **Figure 19** and summarised below.

- Black Head Lagoon (south end);
- Diamond Beach creeks (south end and north end (Khappinghat Creek));
- Old Bar Beach creeks south end (First Rock Gully Creek) and central (Racecourse Creek); and
- Manning River (Farquhar and Harrington Inlets)

During major stormwater events, discharge from the creeks can cause significant erosion to the beach berm and the nearshore area. This in turn can allow larger waves to attack the beach and can cause migration of the creek entrances (NSW Government, 1990).

In addition, flow from stormwater pipes with outlets on beaches can also potentially scour the surrounding sand, creating erosion zones.

There are constructed outlets at the southern end of Black Head Beach, Diamond Beach and Wallabi Point as shown in **Figure 19** with most of these discharging into creeks. There is also one piped stormwater outlet that discharges directly onto Old Bar Beach. This stormwater pipe and a natural drainage channel are located on the northern side of the viewing platform just north of the SLSC beach access. There is presently significant erosion of the dune associated with this stormwater drain as shown in **Plate 14**. There is also dunal degradation occurring adjacent to this area as a result of the beach access from the viewing platform. The formal access stops on top of the frontal dune and pedestrians have carved a fanned erosion pathway from this point down on to the beach (refer **Plate 15**. In the longer term the combination of stormwater erosion and erosion at the beach access may lead to loss of most of the frontal dune in this location.

As discussed in **Section 3.1.2,** the entrance to Racecourse Creek has historically been associated with significant erosion issues due to the meander of the entrance across the shoreline. The training of the entrance with a gabion wall and associated dune stabilisation works have been largely successful in stabilising the location of the entrance and protecting properties along Lewis Street from the hazard associated with entrance instability. This is evident in photogrammetry profiles in this vicinity, as shown in **Figure 20**.

5.7 Climate Change

A discussion on sea level rise associated with climate change was provided in **Section 4.9**. The possibility of other effects caused by climate change, such as increases in storm intensities, was discussed in **Section 4.9.2**.

Under the projected accelerated sea level rise, it is expected that shoreline recession will occur. This issue was discussed in **Section 5.3.2**, as part of the discussion on shoreline recession hazards.

5.8 Slope and Cliff Instability Hazard

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 21**.

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 which is seaward of the beach erosion escarpment (as defined by the beach erosion hazard, see **Section 5.2**).

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 with the Zone of Reduced Foundation Capacity would be considered to have an inadequate factor of safety.

The coastline hazard zones for the study area are determined in **Section 6**, with the position of the Zone of Slope Adjustment and Zone of Reduced Foundation Capacity defined for the immediate, 50 year and 100 year planning periods.

Within the study there are also a number of rocky headlands which were visually assessed for stability. The rocky cliffs and headland were generally found to be of a stable nature and did not pose a significant hazard in the immediate future. These areas should be monitored for changes to highlight any instability that may evolve as natural weathering and erosion processes continue.

Where the foreshore slope consisted of rock a conservative methodology was adopted for the purposes of defining hazard definition planning lines. This methodology involved assuming a 45^{o} failure slope from the toe of the rock foreshore to define the 100 year hazard. This was based on limited geotechnical data (**Figure 16** and SKP, 1982) and visual inspections. A site specific geotechnical investigation in conjunction with a coastal engineering assessment is recommended during the planning stage of individual greenfield developments, or lot redevelopment. These site specific investigations may reduce the necessity to adopt this conservative approach.

6. DEFINITION OF COASTLINE HAZARD ZONES

In this Section, coastline hazard zones are defined within the study area, based on the cumulative impacts of the coastline hazards outlined in **Section 5**, in relation to erosion and recession.

Two coastline hazard zones are defined, namely the Zone of Slope Adjustment and the Zone of Reduced Foundation Capacity (see **Section 5.8**)[16]. These are defined for the immediate (that is, present post storm), 50 year and 100 year planning timeframes.

For simplicity, the landward limit of the Zone of Slope Adjustment for each of the planning timeframes has been denoted as the "Hazard Line". The position of the 2008 Hazard Line, 2058 Hazard Line and 2108 Hazard Line is thus the predicted position of the back beach erosion escarpment after a 100 year ARI coastal storm in 2008, 2058 and 2108 respectively, including subsequent slumping to a stable angle of repose.[17].

Table 6-1 outlines the components of the final Hazard Lines. The volumes were applied as per Nielsen *et al* (1992); see **Figure 21**, at each of the survey profiles. The methodology of Nielsen et al (2001) recommends that the long term average profile be used, however, given the receding nature of this coastline, the 2008 Hazard Line was determined by removing the storm demand volume from the 2006 photogrammetric profiles. Recent erosion was also accounted for by a landward translation of the immediate hazard line, based on photographic evidence of the recession that has occurred since 2006. Thus, at each profile, a position on the 2008 Hazard Line was determined.

[16] The Zone of Wave Impact was also defined as part of the calculations, but is not depicted in Figure 25.
[17] That is, the Hazard Lines do not represent future predicted shorelines, but future predicted erosion escarpments after a 100 year ARI coastal storm.

Table 6-1: Components of 2008, 2058 and 2108 Hazard Lines

<table>
<tr><th rowspan="2">Location</th><th rowspan="2">Adopted recession rate (m/yr)*</th><th rowspan="2">Storm demand (m^3/m)</th><th colspan="3">Long term recession due to sediment loss (m)</th><th colspan="3">Long term recession due to sea level rise (m)</th></tr>
<tr><th>2008</th><th>2058</th><th>2108</th><th>2008</th><th>2058**</th><th>2108***</th></tr>
<tr><td>Black Head Beach</td><td>0</td><td>220</td><td>0</td><td>0</td><td>0</td><td>0</td><td rowspan="11">10-20</td><td rowspan="11">30-45</td></tr>
<tr><td rowspan="3">Diamond Beach</td><td>0</td><td>220</td><td>0</td><td>0</td><td></td><td>0</td></tr>
<tr><td>0.1</td><td>220</td><td>0</td><td>5</td><td>10</td><td>0</td></tr>
<tr><td>0.2</td><td>220</td><td>0</td><td>10</td><td>20</td><td>0</td></tr>
<tr><td>Saltwater Beach</td><td>0.2</td><td>220</td><td>0</td><td>10</td><td>20</td><td>0</td></tr>
<tr><td rowspan="3">Old Bar</td><td>0.3</td><td>180</td><td>0</td><td>15</td><td>30</td><td>0</td></tr>
<tr><td>0.6</td><td>220</td><td>0</td><td>30</td><td>60</td><td>0</td></tr>
<tr><td>1.4</td><td>220</td><td>0</td><td>70</td><td>140</td><td>0</td></tr>
<tr><td rowspan="2">Manning Point Beach</td><td>1.4</td><td>220</td><td>0</td><td>70</td><td>140</td><td>0</td></tr>
<tr><td>1.8</td><td>220</td><td>0</td><td>90</td><td>180</td><td>0</td></tr>
<tr><td>Harrington Beach</td><td>0.6</td><td>220</td><td>0</td><td>30</td><td>60</td><td>0</td></tr>
</table>

* refer **Table 5-1** for locations of adopted recession rates. There are transition areas at the edges of most zones.
** Based on mid-range and high range sea level rise scenarios of 0.2 and 0.4m.
*** Based on mid-range and high range sea level rise scenarios of 0.6m and 0.9m.

In determining the two types of Hazard Line for 2058 and 2108, the 2008 Zone of Wave Impact positions were translated landward allowing for long term recession due to sea level rise (see Table 6-1), then the positions of the Zone of Slope Adjustment and Zone of Reduced Foundation Capacity were recalculated. 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.

Landward of the 2008, 2058 and 2108 Hazard Lines, there would exist a Zone of Reduced Foundation Capacity (ZRFC), which takes account of the reduced bearing capacity of the sand adjacent to the storm erosion escarpment. In general, structures not piled and located within the ZRFC would be considered to have an inadequate factor of safety. The 2008, 2058 and 2108 ZRFC Lines represent the predicted landward limit of this Zone in 2008, 2058 and 2108 respectively. The position of the ZRFC was determined as outlined in **Section 6** in each case. The ZRFC was on average about 16m landward of the corresponding Hazard Line, with the distance mainly depending on the height of the dune.

Note that the ZRFC was derived assuming a beach profile composed entirely of sand. If there were layers of less erodible or inerodible material, such as stiff clays and/or rock within the ZRFC, then the extent of the Zone could potentially be reduced. Geotechnical engineering advice is recommended if foundations within the ZRFC are proposed.

The immediate, 2058 and 2018 Hazard Lines within the study area are shown in **Figure 22.** It is recommended that if any development is planned seaward of the ZRFC, consideration be given to placement of structures on piers (spread footings or piles) extending into the Stable Foundation Zone as defined by Nielsen et al (1992), unless geotechnical conditions enable reduced foundation depths. A site specific geotechnical investigation in conjunction with a coastal engineering assessment is recommended during the planning stage of any individual lot redevelopment.

7. CONCLUSION

The Study has found that the most significant hazards in the study area are beach erosion and shoreline recession, coastal inundation, stormwater erosion, and climate change. Each of these hazards is outlined below. **Figure 22** illustrates the defined coastline hazard zones within the study area based on the cumulative impacts of the coastline hazards relating to erosion and recession. The determination of these zones is discussed in **Section 6**.

7.1 Beach Erosion and Shoreline Recession

The hazard lines were determined based on the cumulative effects of the 100 year ARI coastal storm erosion, long term recession due to net sediment loss, and long term recession due to sea level rise (refer **Figure 22**) indicate that a number of assets will be impacted by this recession. Assets impacted in the Immediate, 50 year and 100 year planning period are outlined in **Table 7-1**.

Table 7-1: Assets potentially impacted by coastal erosion and shoreline recession

Location	Assets impacted in Planning Period		
	Immediate	**50 year (high range)**	**100 year (high range)**
Black Head Beach	• Black Head SLSC and associated assets (boatramp, rock pool etc); • Stormwater outlets to Black Head Lagoon entrance; • Black Head Lagoon Park facilities and sewerage pumping station; • Pedestrian bridge over Black Head Lagoon to beach; • Main St roadway at Black Head.	• Main St and properties between Albert St and Ocean St (9); • Foreshore row of cabins at Beachfront Holiday Resort (Big 4), Red Head; • Properties at the seaward end of Scenic Avenue (4), Red Head; • Black Head SLSC and associated assets; • Stormwater outlets to Black Head Lagoon entrance; • Black Head Lagoon park facilities and sewerage pumping station; • Pedestrian bridge over Black Head Lagoon to beach.	• Main St properties between Albert St and southern corner(12); • Foreshore row and some second row cabins at Beachfront Holiday Resort (Big 4), Red Head; • Properties at the seaward end of Scenic Avenue (6); • Stormwater drain at Red Head; • Black Head SLSC and associated assets; • Stormwater outlets to Black Head Lagoon entrance; • Black Head Lagoon park facilities and sewerage pumping station; • Pedestrian bridge over Black Head Lagoon to beach; • Watermains along Main St.

Diamond Beach	• Properties on seaward side of southern end of Jubilee Pde (6).	• All properties on seaward side of Jubilee Pde (25); • Seaward row of units/structures within Diamond Beach Holiday Park at northern end of Jubilee Pde; • Stormwater outlet to creek; • Carpark at end of Diamond Dv; • Seaward edge of lots between the holiday park and Australis Resort.	• Jubilee Parade roadway and all properties on seaward side; • Seaward and second row of units/structures within Diamond Beach Holiday Park ; • Stormwater outlet to creek; • Carpark at end of Diamond Dv; • Diamond Beachfront Holiday Units, Diamond Beach Rd, most easterly house/ unit and eastern end of accommodation block; • Diamond Beach Resort, easterly most buildings and eastern end of building parallel to Diamond Beach Rd; • Seaward part of Seashells Beachfront Resort building; • House/buildings on northern side of Seashells Resort main building; • Most seaward buildings in Australis Diamond Beach Resort; • Watermains along Jubliee Pde, to the Diamond Beach Resort and Diamond Beach Holiday Units.

Saltwater Beach	• Stormwater outlet, south side Wallabi Point.	• Properties (4) at Wallabi Point (at seaward end of Marine Drive, Ocean Drive and Saltwater Road); • Saltwater Road; • Water main to rural properties; • Frontage to rural properties; • Stormwater outlet, south side Wallabi Point.	• Properties (9) at Wallabi Point (at seaward end of Marine Drive, Ocean Drive and Saltwater Road); • Stormwater outlet to First Rock Gully Creek entrance; • Saltwater Road; • Water main to rural properties; • Frontage to rural properties; • Stormwater outlet, south side Wallabi Point.
Old Bar Beach	• South end of Pacific Pde roadway; • Seaward yards of properties on seaward side of Lewis St (12); • Buildings on seaward edge of lots (3), southern end of Lewis St; • Stormwater outlet near Taree Old Bar SLSC.	• Midcoast Water exfiltration ponds; • Properties on seaward side of Lewis Street (23 plus Meridian Resort); • Sewer and water mains along Lewis St, in vicinity of Pacific Pde and the eastern end of Rose St; • Properties (12) at the southern end of Pacific Street; • Eastern frontage of Lani's on the Beach caravan park;	• Midcoast Water exfiltration ponds; • All properties on seaward side and landward side of Lewis Street; • Properties (4) on Rose St; • All properties on Pacific Pde (23); • All properties on seaward side of Hall St (14); • Properties fronting Ungala Rd (6); • Sewer and water mains along Lewis St, in vicinity of Pacific Pde and the eastern end of Rose St; • Old Bar Public School;

		• Taree Old Bar SLSC and associated amenities (toilet block/change rooms, part of carpark etc); • Stormwater outlet near Taree Old Bar SLSC.	• Eastern frontage of Lani's on the Beach caravan park; • Taree Old Bar SLSC, associated amenities (including playground) and stormwater outlet.
Manning Point Beach	Nil	• The eastern edge of rural land including several dams; • Midcoast Water Sewage Treatment Plant and associated structures, Manning Point.	• The eastern edge of rural land including several dams; • Properties (2) on Beach Road (houses near middle of beach); • Midcoast Water Sewage Treatment Plant and associated structures, Manning Point; • Properties (16) on seaward side of Manning Street (north of Ocean Parade) including bowling club; • Properties on northern side of Manning St (9) and on Main St at eastern end of block (5); • Eastern half of Easts Ocean Shores Holiday Park; • Manning Point Hideway Holiday Cabins to the south of the Holiday Park.
Harrington Beach	Nil	Nil	Nil

The most vulnerable area currently at risk (i.e. within the “immediate” hazard line) is Old Bar Beach. The majority of nearshore oceanic conditions lead to net loss of sediment from Old Bar Beach. This has been supported through analysis of photogrammetric data showing no significant period of beach recovery (volume or profile change over the period 1963 - 2006). The highest erosion is in the central portion of Old Bar Beach, where a number of complex coastal processes combine to exacerbate the erosion rate. The erosion rate decreases at the southern and northern extremities of the beach, under the stabilising influences of Wallabi Point and Urana Bombora (and the entrance to Farquhar Inlet depending on its open/closed status). Further to this the erosion rate has appeared to increase in recent time. Key findings for the Old Bar Beach (from the interpretation of available data) conceptual model of coastal process include the following.

- Significant sediment losses may occur in both directions (to the north, or to the south and offshore)

- The beach is not in equilibrium with the incident wave climate because of limited sediment supply both offshore and downdrift (reef systems and headlands) and significant sediment sinks removing sand from the system. Accordingly, the beach profile is susceptible to losses during major events. The beach alignment/state will continue to adjust until an equilibrium is approached through further embayment (i.e. nearshore conditions are such that sand relocated to offshore bars in storm events returns to the beach foreshore during quieter periods and there is no net loss of sand from the system. In a sediment supply limited system this amounts to a significant reduction in long shore transport potential (wave induced longshore currents)

- Entrance stabilisation works at Racecourse Creek do not act as a significant control on the loss of sediment from Old Bar Beach (it is noted that this was not the intention of the works).

- The nearshore delta, when Farquhar Inlet is open in combination with complex wave processes in the vicinity of Urana Bombora, appear to moderate the loss of sediment from Old Bar Beach. When the inlet is closed, Urana Bombora (in isolation) does not appear to be as effective in this regard. However, continual mechanical opening of the entrance does not produce the same delta as would form from a major flood event, limiting the effect and duration of benefits to the coast. Manning Point Beach is also influenced significantly by processes at both Manning River entrances (Farquhar Inlet and Harrington Inlet).

- Significant data gaps inhibit the understanding of this complex system, particularly detailed nearshore bathymetric survey data and offshore directional wave data. DECCW have commissioned a nearshore survey to assist in addressing some of the data gaps. This data is expected to be available for the management study and, as a consequence, coastal processes will need to be revisited in assessing the viability of management options.

The most significant implication of the sediment limited character of Old Bar Beach is that any engineering works implemented to manage erosion/recession would require a massive initial beach nourishment campaign and a commitment to ongoing, periodic maintenance to be effective and to maintain beach amenity. Furthermore, engineered stabilisation works and beach nourishment at Old Bar Beach (with the most likely source being Farquhar Inlet) could potentially cause increased erosion/recession issues along Manning Point Beach unless this section of beach was also nourished (or nourishment sand was sourced externally). The cost effectiveness of these and other management measures will be examined in the management study.

Without intensive management of sediment processes, stabilisation of one particular area of coastline will lead to instability elsewhere. This is evident from the natural fluctuations seen at Manning Point Beach and Old Bar Beach in response to the varying degree of control offered to the system by the Farquhar Inlet entrance ebb tide delta (or lack of) and Urana Bombora.

7.2 Coastal Inundation

A number of areas and assets have been identified as being particularly vulnerable to erosion and inundation due to low lying topography (as shown in **Figure 19**). These areas include:

- The entrance to all the creeks in the study area including Black Head Lagoon, Khappinghat Creek south end of Diamond Beach, First Rock Gully Creek and Racecourse Creek. The shallow creek mouths would be affected by wave runup and at the design elevated ocean water level ocean water would penetrate upstream in the creek.

- The northern half of Black Head Beach where the dune height is approximately 5m AHD (this area would experience overwash due to wave runup).

- The southern end of Diamond Beach including the southern end of Golden Drive (up to Diamond Drive).

- Near the middle of Old Bar Beach where the dune system is relatively low lying as a result of historical sand mining activities. With a dune height of approximately 5m AHD this area would experience overwash due to wave runup. It is noted that the area behind the dune system is also low lying at approximately 4.5m AHD.

- The dunes fronting the Lewis Street properties which are relatively low lying (wave runup could potentially reach the current seaward edge of these properties).

- The various accessways to the beach near the Taree Old Bar SLSC are low lying and therefore potentially affected by wave runup. Inundation is not currently likely to reach any building assets in this area.

- The low lying area at the southern end of the entrance to Farquhar Inlet (at or near 4WD access points to the beach).

- Some isolated low lying dunal areas along Manning Point Beach. There is also likely to be flooding of the Manning Point Spit from elevated water levels within the Manning River. This inundation from both the river and ocean sides may result in a thin strip of dry sand dune in this area.

- Low lying sections of the Harrington township would potentially be inundated by elevated ocean water levels penetrating upstream in the Manning River. Impacts at Harrington are dependent on future maintenance and repairs to the entrance works.

- The southern 1.5km of Harrington Beach which has relatively low lying dunes (approximately 5m AHD) would experience overwash due to wave runup.

7.3 Stormwater Erosion

A stormwater pipe and a natural drainage channel are located on the northern side of the viewing platform just north of the Taree Old Bar SLSC beach access. There is presently significant erosion of the dune associated with this stormwater drain as shown in **Plate 14**. Dunal degradation has also occurred adjacent to this area, as a result of informal beach access from the viewing platform. The formal access stops on top of the frontal dune and pedestrians have carved a fanned erosion pathway from this point down on to the beach. In the longer term the combination of the stormwater erosion and from pedestrian access erosion may lead to loss of most of the frontal dune in this location.

7.4 Climate Change

Recession of all beaches is predicted in the future due to sea level rise associated with climate change. The estimated predicted shoreline recession due to sea level rise for the study area is in the order of 10 to 20m over the next 50 years, and 30 to 45m (mid-range to high-range scenarios) over the next 100 years.

Another potential outcome of the climate change is an increase in the frequency and intensity of storm events. 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 at different locations in the compartment.

Given the above uncertainty and difficulty in quantifying any change, potential changes to storm frequency and intensity, or changes in wave directions were not specifically addressed.[18] However, this uncertainty should be taken into consideration when assessing the risk and consequences of recession occurring in the future. The potential for climate change related recession needs to be continually reviewed as more information becomes available.

7.5 Forthcoming Data and Recommended Further Studies

During this investigation a number of data gaps were identified.

DECCW are currently planning to undertake a bathymetric survey of the coastline between Wallabi Point and Farquhar Inlet to obtain further detail of the nearshore zone, the rock reef features and any unusual offshore sand features. This data may be utilised in a wave model to further investigate the refractive/diffractive influence of Urana Bombora, other reef features and the Farquhar Inlet entrance shoals. This would then enable the sediment transport processes to be better defined on each of the beaches, shedding further light on the relationship between Farquhar Inlet, and the adjacent beaches of Old Bar and Manning Point. The bathymetric data will also reveal any unusual offshore features that may indicate the destination of permanent offshore sand losses from the system. This information is necessary for assessing the viability of management options put forward in the management study.

Other recommended data collation includes the following:

- Directional wave data at Crowdy Head to better appreciate the influence of wave energy direction on the erosion/recession of the beaches in the study area. This will become increasingly important in adaptive management of the beaches if the angle of approach of the predominant wave climate changes due to the climate change.

- Pre and post storm beach profiling to enable the storm demand volume to be better estimated.

- Repeat bathymetric surveys between Wallabi Point and Farquhar Inlet for comparative analysis to identify changes.

- Hydrographic survey of Farquhar Inlet and nearshore shoals to further define their contribution to the sediment budget through various phases (open constricted and closed).

[18] A generally conservative approach was used in the estimation of other coastline hazards.

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Addendum
May 2010

Introduction

This Addendum was prepared following the completion of a recent detailed bathymetric survey undertaken by Department of Environment Climate Change and Water (DECCW) and subsequent additional coastal processes investigations. It provides additional information to further develop the understanding of coastal processes through a conceptual model, (summarised in **Section 4.12** of the Black Head to Crowdy Head - Coastline Hazard Definition Study), particularly with regard to the Old Bar Beach compartment. The addendum should be read in conjunction with **Section 4.12** of the Black Head to Crowdy Head - Coastline Hazard Definition Study. Figures which accompany this Addendum are appended following the text.

Detailed Bathymetric Survey

Hydrographic surveyors from the Water and Coastal Science Section of DECCW undertook a detailed hydrosurvey of the offshore area between Saltwater Point and 3.5km north of the entrance to Farquhar Inlet. At the time of the survey work Farquhar Inlet was open. The objective of the detailed survey was to resolve the complex bathymetry and rock reef features at the site to: provide input to additional coastal process modelling; identify possible sediment pathways; and guide any further investigations necessary. **Figure 1A** (attached) is a rendered isometric image of the hydrosurvey data indicating the bathymetric character of the surveyed area.

From the detailed survey data presented in **Figure 1A** the following key features are identified:

- Despite the visible (above sea level) beach form indicating otherwise, Old Bar Beach has the submarine form of an embayed beach (**Figures 1A and 2A** indicate this form).
- The significant continuous submarine rock reef features at Wallabi Point and Urana Bombora form the embayment (see **Figure 2A**).
- The geological feature that forms Urana bombora (including the reef form that is commonly known as “second corner reef” on the northern side of the feature) is the most significant bedform of the area, rising significantly above the general bed level for a distance of 2.5km offshore and then re-emerging above the general bed level to be influential for a further 2km.
- Based on geology maps, anecdotal evidence and the detailed bathymetry collected; it is likely that the bedrock feature of Urana Bombora continues inland and is located at shallow depths below the beach sand, but the extent is unknown.
- The general form of the bathymetry on either side of Urana Bombora is similar to that of a headland. On the southern side of Urana Bombora the bathymetry has the form of a sediment

fillet, indicating that this feature traps sediment moving south to north in the compartment, and the beginnings of an indented "zeta" curve form is evident to the north;

- The zeta form is modified on the northern side by the presence of the estuarine delta of the entrance to Farquhar Inlet;
- Further north of the influence of the entrance to Farquhar Inlet (along Manning Point Beach), rock bedforms are less frequent and less significant. The bedform is regular and a classic long zeta shaped beach and nearshore form occurs uninterrupted, typical of longshore drift dominated beaches. A definite longshore trough and bar system can be indentified;
- As Urana Bombora is not an emergent feature above approximately -2.5m AHD, a beach connects Old Bar Beach to the entrance berm of Farquhar Inlet. Analysis of the detailed bathymetry indicates that this beach region is the most significant sediment pathway for littoral drift bypassing of Urana Bombora, as the submarine rock feature forms a barrier to movement out to depths in excess of 15m (greater than depths at which significant longshore sediment transport ceases to occur). Also, during storm events overtopping of the Bombora by suspended sediment (due to wave action) in flow provides a bypassing pathway;
- At the southern end of the Old Bar Beach embayment it appears that a small compartment is formed between the main Wallabi Point reef and a second submarine reef outcrop approximately 500m to the north.
- Running along the northern side of the second Wallabi Point reef outcrop a trench feature exists;
- There are no immediately obvious sand lobes indicating an offshore deposition of nearshore sand. However, further analysis (see **Figure 3A**) does suggest some evidence of flattening of the bathymetry (indicating deposition) at the offshore extremity of the trench feature where infilling between rock outcrops may be occurring.*
- The presence of a sediment fillet form at the northern end of the embayment against Urana Bombora at relatively significant depths indicates this may also be an area of sediment deposition due to offshore transport. Infilling between the irregular rock bedform of Urana Bombora may also be occurring.*
- Comparison of the cross-shore profiles (see **Figure 4A**) located in the centre of Old Bar Beach and along Manning Point Beach (north of Farquhar Inlet Entrance) reveals that below -8m AHD (approximately the seaward limit of the littoral zone) the offshore slope at Old Bar is significantly shallower. This may be the result of different dominant sediment transport mechanisms for the adjacent beach compartments. Offshore transport dominated at Old Bar Beach (with sediment being transported and lost beyond the littoral zone during storm events) and longshore transport dominated at Manning Point Beach.*

*Note - All of the evidence relating to offshore deposition locations is not absolutely conclusive because of the obvious presence of bedrock outcrops which may also be present subsurface.

A comparison of the most recent hydrosurvey with hydrosurvey data collected in the past was attempted to assess changes over long periods of time. However, the relatively coarse data resolution of historical surveys and the complexity of the bedform in the area made this task impractical because of the obvious inaccuracy of the previous collected data in providing a realistic representation of the bedform. For future understanding and management of this complex beach compartment it would be advantageous to complete repeat detailed surveys of the area.

Additional Coastal Processes Modelling

The detailed bathymetry collected was used to further develop and refine a MIKE21 coupled hydrodynamic wave and flow model of the region. Originally the model was developed to investigate wave refraction processes at the site and was based on coarse bathymetry digitised from the Hydrographic chart AUS 801 (see **Section 4.5.1** of the Black Head to Crowdy Head - Coastline Hazard Definition Study). The detailed coupled wave/flow model was used to investigate a number of event based conditions to gain a greater understanding and test hypothesis based on community consultation and site observations of the complex hydraulic nature of the region. In particular to investigate the likelihood of significant erosion (and ongoing recession) of Old Bar Beach occurring due to the development of major rip cells transporting sediment (suspended due to high energy wave action) offshore during major storm events was investigated. A spectral wave model was used to transfer offshore wave conditions into the nearshore zone. Radiation stresses calculated from the wave model were coupled to the flow model to drive flow.

It is important to note that the modelling undertaken was of a steady state nature and can not represent significant nearshore temporal processes, such as surf beat and the growth and decay of minor rip cells.

Selected offshore wave cases from a comprehensive array of test cases are presented in **Figure 5A** (“average” conditions) and **Figure 6A** (“extreme storm” conditions) to illustrate the findings of this investigation.

Figure 5A Series - “Average” Conditions

Farquhar Inlet was modelled as open with an ebb tide shoal present. Based on long term wave statistics, an average ocean condition was parameterised for application to the model as follows:

Significant Wave Height (Hs) = 1.5m,

Peak Wave period (Tp) = 10 secs,

Sensitivity to wave direction was undertaken by applying this wave condition at wave directions from South through to North East in 22.5 degree increments. Wave Direction is indicated on each Figure.

From the model results presented in the **Figure 5A** series the following key features are indicated:

- During average conditions it is only the most southerly of wave events that promote northward flowing longshore currents inshore of Urana Bombora. Weak flow reversal due to wave refraction and breaking processes on the Bombora is evident for the SSE direction and is more significant for the SE and ESE directions.
- The formation of a large scale rip cell in which there is a significant component of the flow directed offshore due to competing longshore flow directions emanating from Wallabi Point and Urana Bombora, is evident for the SE and ESE directions. This flow feature forms immediately to the south of Racecourse Creek entrance for SE wave directions, and further south against the second reef outcrop at Wallabi Point for ESE wave directions. Note: depth averaged current velocities in the rip cell are of the order of 0.1m/s or less and are not likely to carry significant volumes of sediment.
- for offshore wave directions North of East, the flow patterns are similar to that indicated for an Easterly direction (predominantly north to south flow).

Figure 6A Series - “Extreme Storm” Conditions

An extreme storm condition was synthesised for application to the model as follows:

Hs = 5m,

Tp = 12 secs,

Wind Speed = 10m/s from the same direction as waves,

Sensitivity to wave direction was undertaken by applying this wave condition for wave directions from South through to North East in 22.5 degree increments. Wind and wave direction is indicated on each Figure.

From the model results presented in the **Figure 6A** series the following key features are indicated:

- Flow features develop in a similar fashion to that described above for average conditions, relative to offshore wave direction. However, during more extreme events the modelling indicates that both South and SSE events promote northward flowing longshore currents inshore of Urana Bombora.
- For events with a SE wave direction a large scale rip cell (in which there is a component of the flow directed offshore) forms immediately to the south of Racecourse Creek entrance. However, due to the large SE waves interacting with Urana Bombora at significant distances from the beach it appears there is a partial recirculation of this flow within the beach “compartment” as flow is directed back to shore at Urana Bombora. Note: 2D depth averaged

current velocities in the rip cell are of the order of 0.5m/s and are likely to carry significant volumes of sediment.

- For events with an ESE wave direction the formation of a large scale rip cell (in which the flow is directed offshore due to competing longshore flow directions emanating from Wallabi Point and Urana Bombora) is significant. The offshore flow appears to coincide with the trench feature immediately north of the second Wallabi Point reef outcrop, described in the Section above. No recirculation within the beach compartment occurs. Note: 2D depth averaged current velocities in the rip cell are of the order of 0.5m/s and are likely to carry significant volumes of sediment.
- For offshore wave directions North of East, the flow patterns are similar to that indicated for an Easterly direction (predominantly north to south flow).

Sensitivity to Wave Period - a range of wave periods was investigated for both the average and extreme storm conditions and typically the same flow patterns were observed regardless of wave period. However with increasing wave period, flow velocities increased and any large scale flow feature (such as rip cells) increased in size and extent. This influenced the location along the beach at which they formed and the offshore extent.

Discussion

Analysis of bathymetric features and numerical modelling of specific wave events has indicated the following:

- The hypothesis that significant amounts of sediment could be lost from the Old Bar Beach system due to the formation of a large rip cells carrying sediment offshore during large storms is supported.
- The location of the rip cell head is generally in the central to southern portion of the beach adjacent to where the most significant recession rates have been identified for Old Bar Beach by the Black Head to Crowdy Head - Coastline Hazard Definition Study.
- Wave direction of the storm event may be a significant factor in whether sediment carried by the rip cell is predominantly lost to the offshore zone or partially recirculated within the nearshore beach compartment.
- Bedload sediment bypassing of Urana Bombora is likely to occur inshore of the rock feature. Elevated water levels during storm events would facilitate easier inshore bypassing (for the most southern and E - NE events). This could occur in both directions. The amount of bypassing is likely to be influenced by the beach state on either side of the Bombora (including the open/closed status of the entrance to Farquhar Inlet) at the time of the particular event.

- The subsurface rock feature landward of Urana Bombora is a significant factor in the relative stability of the beach in this location, rather than purely wave diffraction/refraction effects (i.e. it acts as a subterranean/subaqueous headland rather than an submerged offshore reef);
- Wave direction is a significant factor in the mechanism and amount of sediment transport that occurs at Old Bar Beach. As a result, seasonal and decadal fluctuations in loss rates are likely.
- When the storm wave direction is from the SE sector due to the complexities of the wave driven flow structure which occurs as result of Urana Bombora, significant volumes of sediment in suspension may be transported offshore to the north due to the formation of a major rip cell (despite nearshore currents flowing north to south). This volume of sediment appears to be deposited against the southern side of Urana Bombora (evidenced by the fillet form described above) or is recirculated to the nearshore zone by flow across Urana Bombora. This recirculated flow is likely to result in sediment being transport both south and north of the Bombora at the shoreline. As a result, there is a net loss of sediment from Old Bar Beach (both offshore and through transport to the north of Urana Bombora).
- For the case when the storm wave direction is from ESE sector (wave height = 5m, and wave period = 12s), the large rip cell which forms has a tendency to transport sediment offshore at the southern end of the beach with no partial recirculation to the nearshore zone, resulting in a larger net loss from Old Bar Beach than for SE storm events. This feature was observed during a site visit on 21 April 2009 for which significant erosion of Old Bar Beach occurred. Recorded wave direction during this event at Sydney was ESE (Note: wave recording at Crowdy Head does not include wave direction. Given the significant sensitivity of coastal processes at the site to wave direction it would be advantageous if the data collection station was upgraded to include wave direction for future investigations).

A number of sediment loss mechanisms from the Old Bar Beach compartment have been identified. Storm related events are recognised as the dominant driver for significant losses. Due to the submarine embayed form of Old Bar Beach and resultant large rip cell formation during storm events from the SE and ESE direction, offshore transport is the dominant loss mechanism. This observation is consistent with Wright and Short (1983) who reported that for "most of the New South Wales beaches which occupy embayed compartments bounded by protruding headlands…..the most significant removal of sand, when it occurs, is associated with transport processes acting normal rather than parallel to the shore. During high energy events, these processes may move sand offshore to depths of 10 to 20m from which it requires 2 years or more to return."

However, as opposed to a completely embayed beach where longshore sediment transport processes are not significant in terms of losses, for storm events with wave directions other than the SE and ESE sectors, the submarine form of Urana Bombora is less effective in influencing the refraction of wave energy, and longshore transport is the dominant mechanism. Inshore bypassing of Urana Bombora occurs under these storm wave conditions. Overtopping due to suspended sediment load in flows is another bypassing mechanism.

Also, during periods of low energy wave action when sediment transport occurs primarily at the beach face and the immediate nearshore area, the influence of the northern “headland” of the embayment, Urana Bombora, is significantly reduced in terms of a barrier to sediment transport due this feature not being emergent above -2.5m AHD. As a result, net losses of sediment due to littoral drift south to north also occur at Old Bar Beach when the dominant process is oblique wave direction at the shoreline promoting sediment transport by beach drifting*, not wave generated currents.

* Beach drifting occurs as a result of the zig zag motion of sediment along the beach face caused by the uprush from breaking waves running obliquely up the beach face and the backwash returning under gravitational action straight down the beach face.

Implications for Management Options

The mechanisms for loss need to be considered when assessing the effectiveness of structural management options. Bathymetric features which are the reason for the hydraulic character of the compartment are of a regional scale. Accordingly, engineered structures would need to be of a similar scale to influence the driving sediment loss mechanisms or mitigate sediment transport. The scale and cost of engineering required needs to be considered relative to the value of property and assets at risk from long term recession.

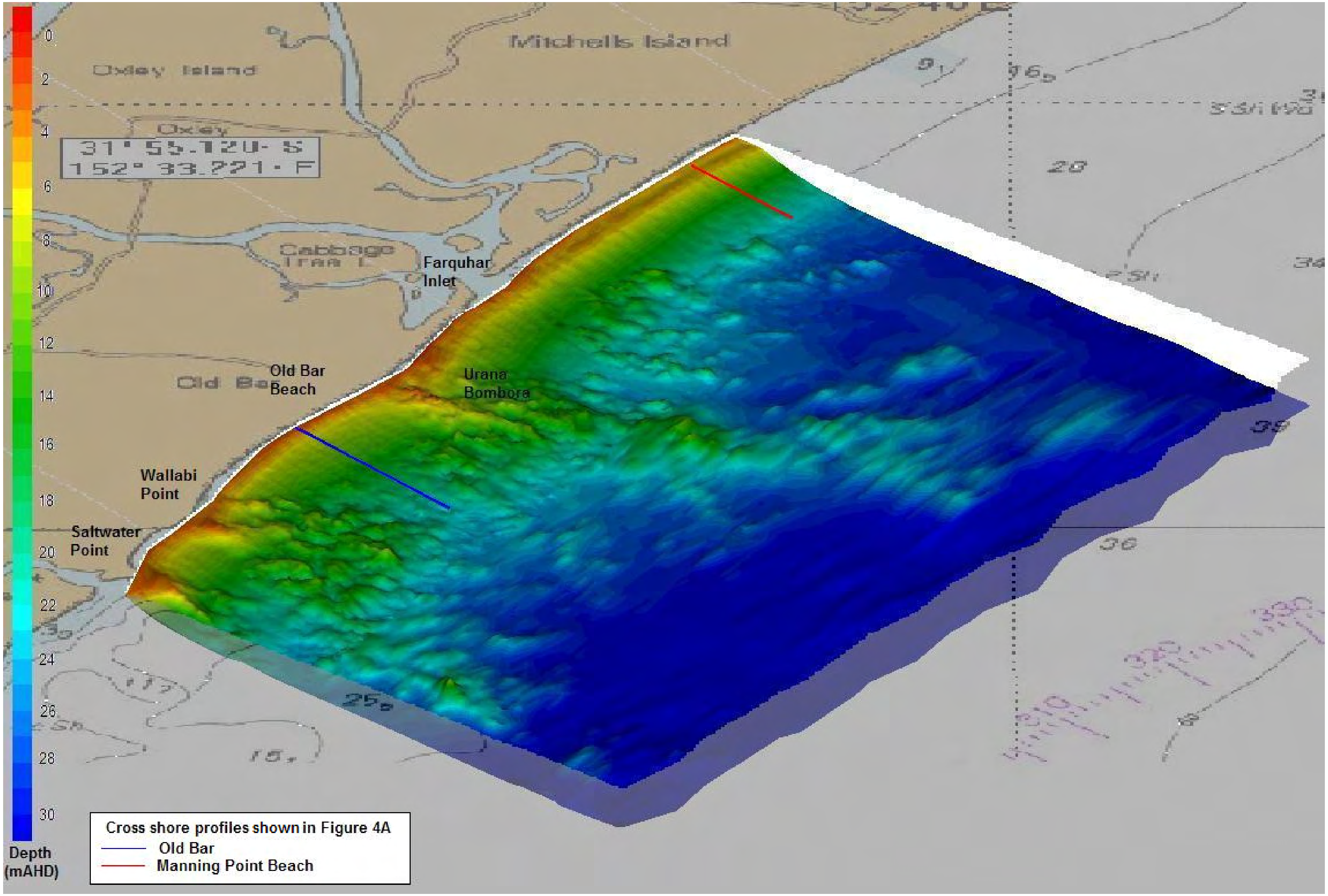
Depending on the wave direction, dominant wave period, temporal and local scale fluctuations during particular storm events and ambient conditions, structures may be relatively effective for one particular situation, neutral for another and counter effective for another. Similarly, this may be the case for general seasonal and decadal fluctuations.

Engineered mitigation by the replacement of lost sediment through beach nourishment may be effective. However, this would require massive initial beach nourishment and a commitment to regular maintenance nourishment to keep pace with significant recession rates (likely to increase due to rising sea levels). The likely source of nourishment sand would be from Farquhar Inlet. As indicated by investigations into sediment loss mechanisms from Old Bar Beach, this may lead to a net loss of sediment from Manning Point Beach north of Urana Bombora as a significant proportion of sediment placed regularly on Old Bar Beach is likely to be transported and lost offshore, rather than being transported back into Farquhar Inlet or bypassing the entrance to the north.

It may be feasible from a technical and coastal processes point of view to dredge sediment lost offshore in the Old Bar embayment (i.e. beyond approximately 8m AHD depth) to recirculate onto the beach. This would require further investigation. However, at this point in time, Government policy on offshore dredging rules out this option.

References

L. D. Wright and A. D. Short, “Morphodynamics of beaches and surf zones in Australia,” in Handbook of Coastal Processes and Erosion. Boca Raton, FL: CRC, 1983, pp. 35–64.



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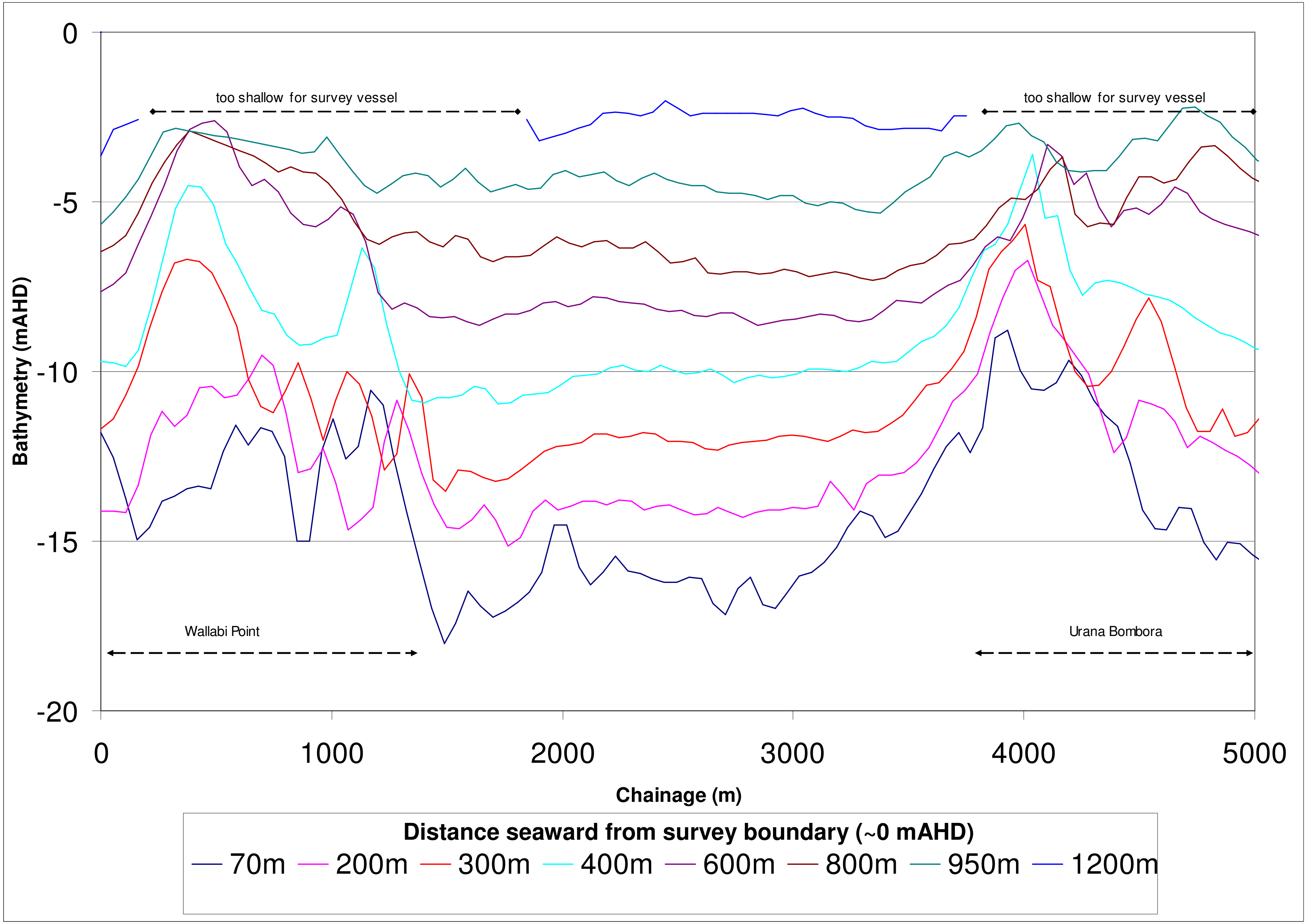
Nearshore Bathymetry

Figure 1A



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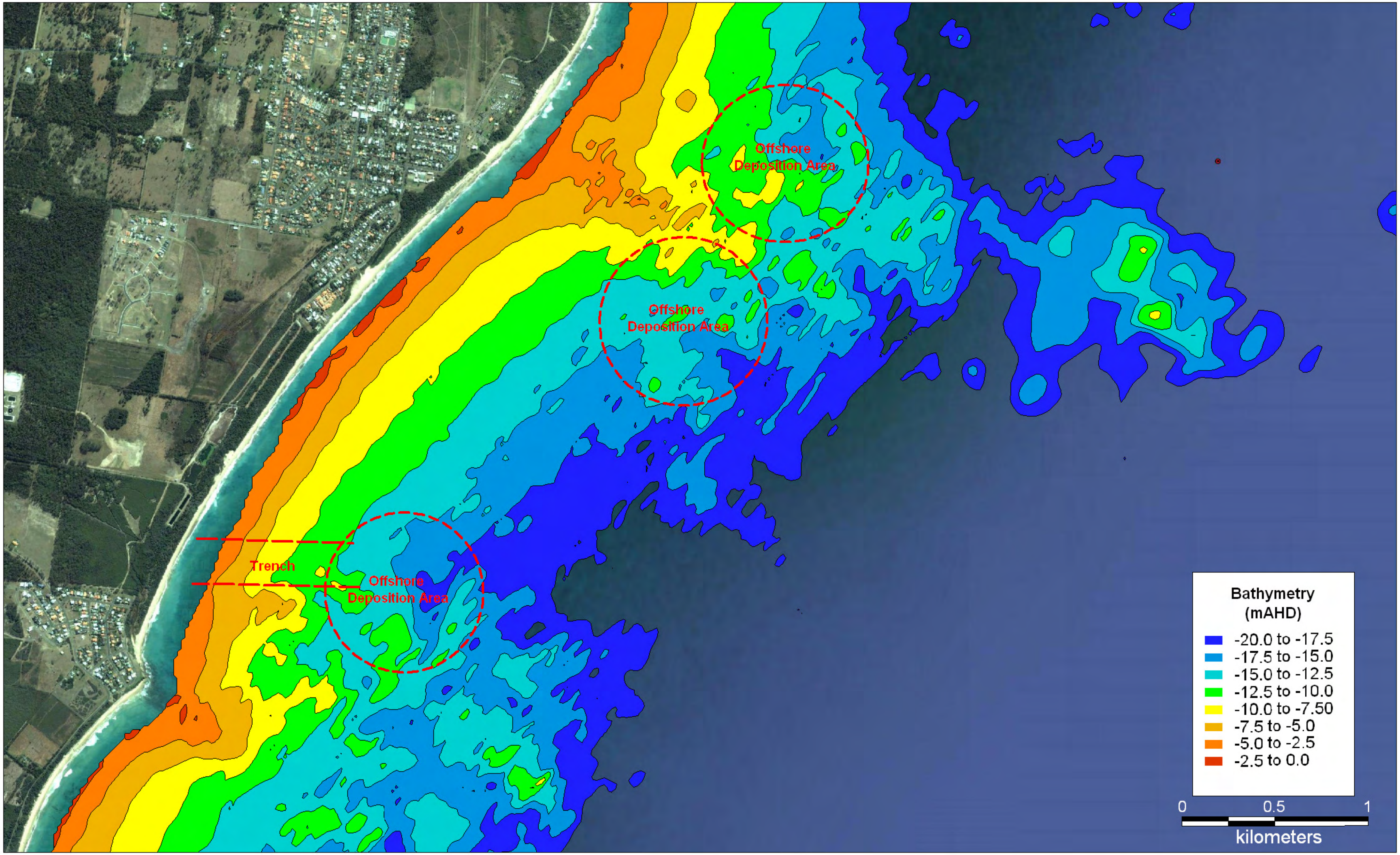
Alongshore Hydrographic Survey Transects

Figure 2A



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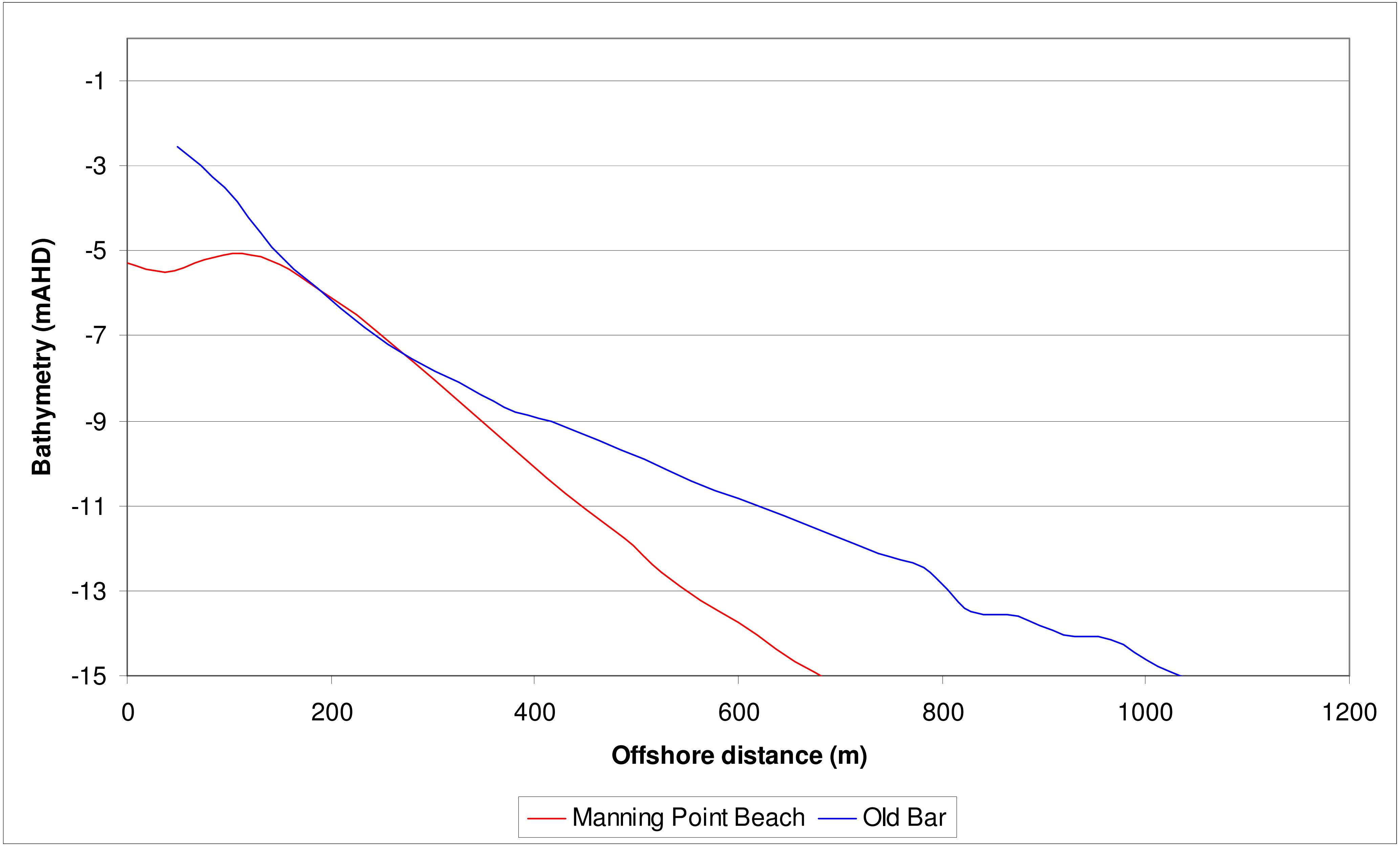
Possible Offshore Sediment Transport Features

Figure 3A



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Comparison of Cross Shore Profiles

Figure 4A

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Wave Direction - S

Figure 5A (i)

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Wave Direction – SSE

Figure 5A (ii)

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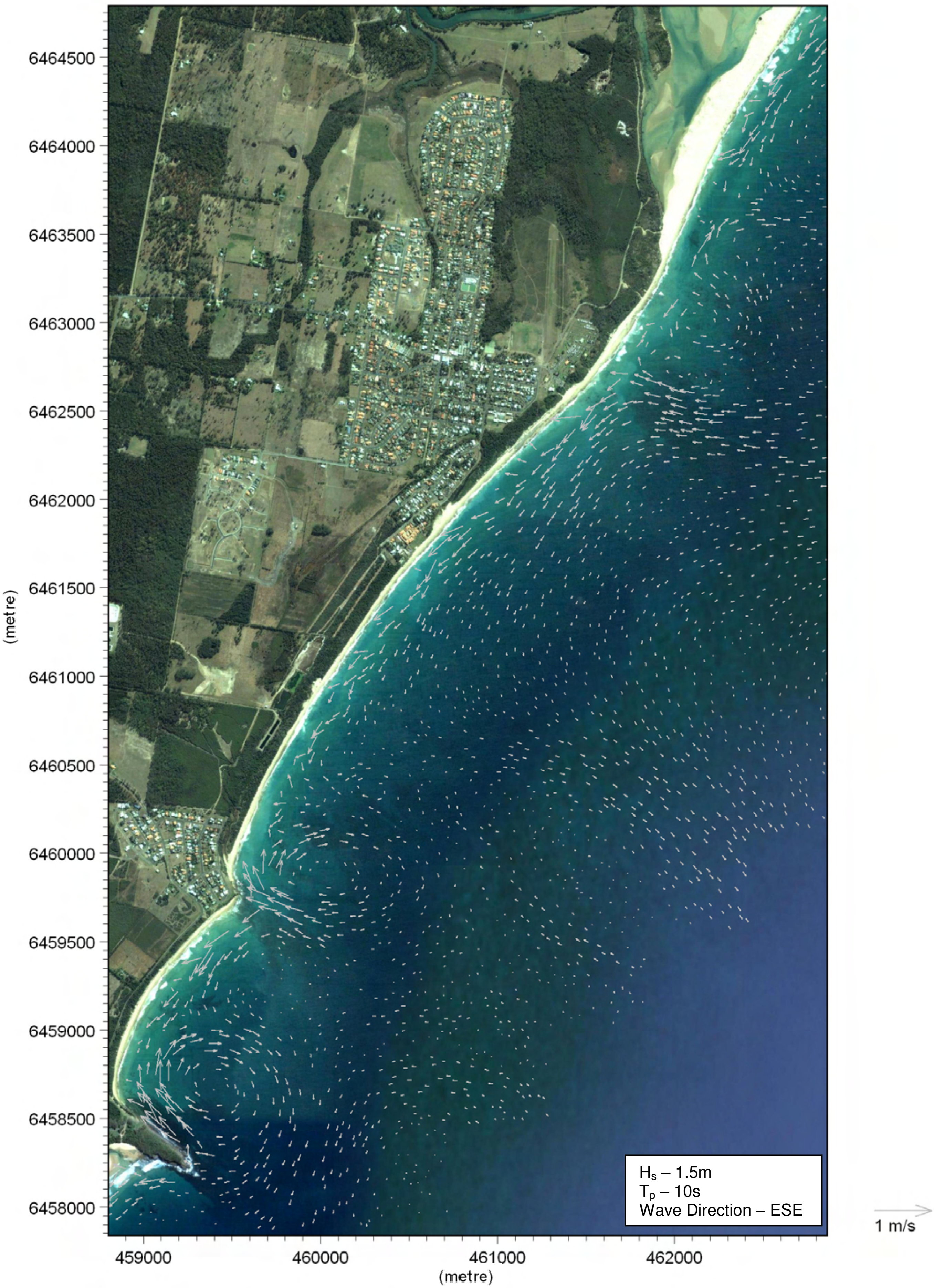
Wave Direction - SE

Figure 5A (iii)



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Wave Direction - ESE

Figure 5A (iv)

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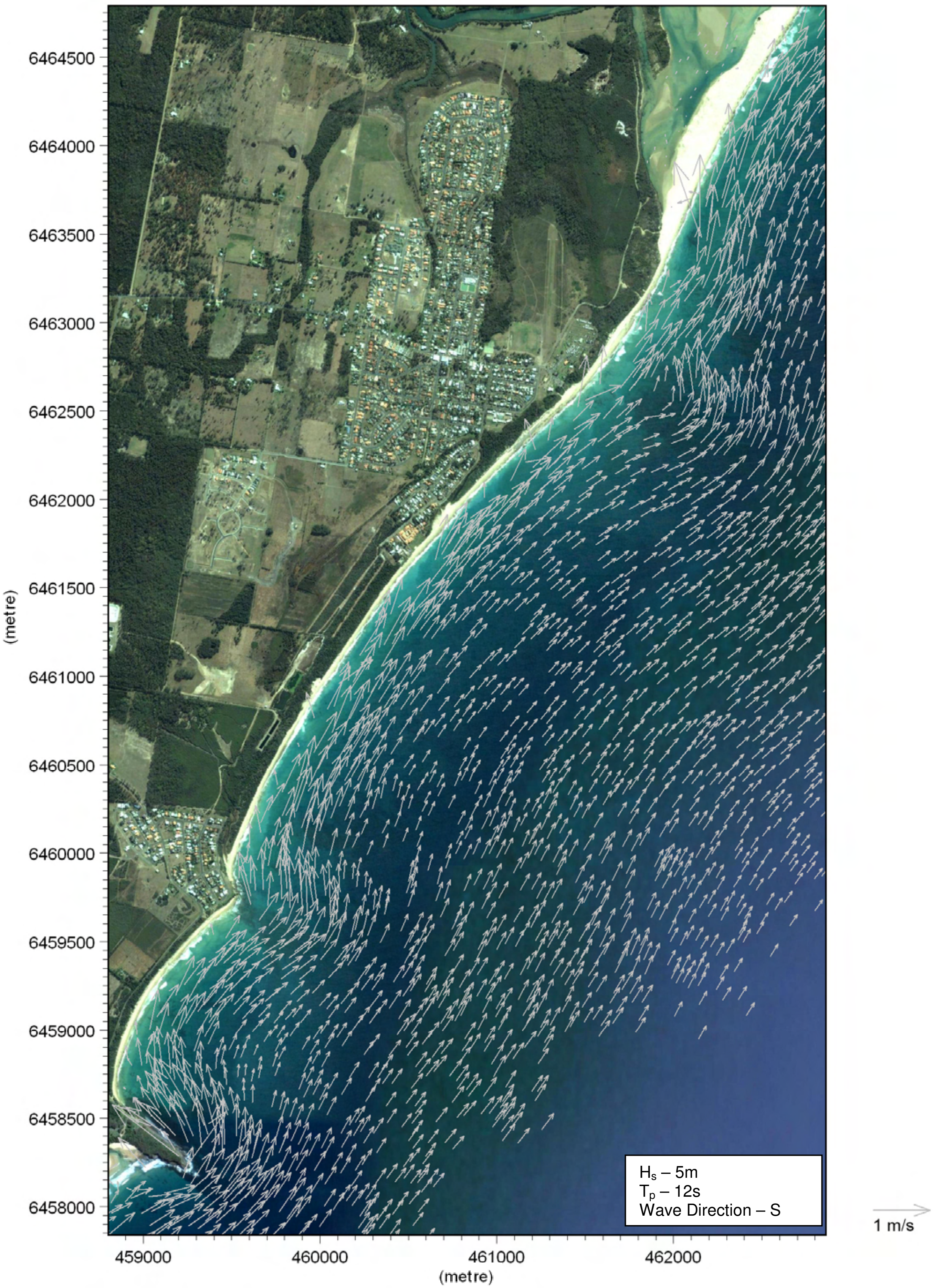
Wave Direction - E

Figure 5A (v)



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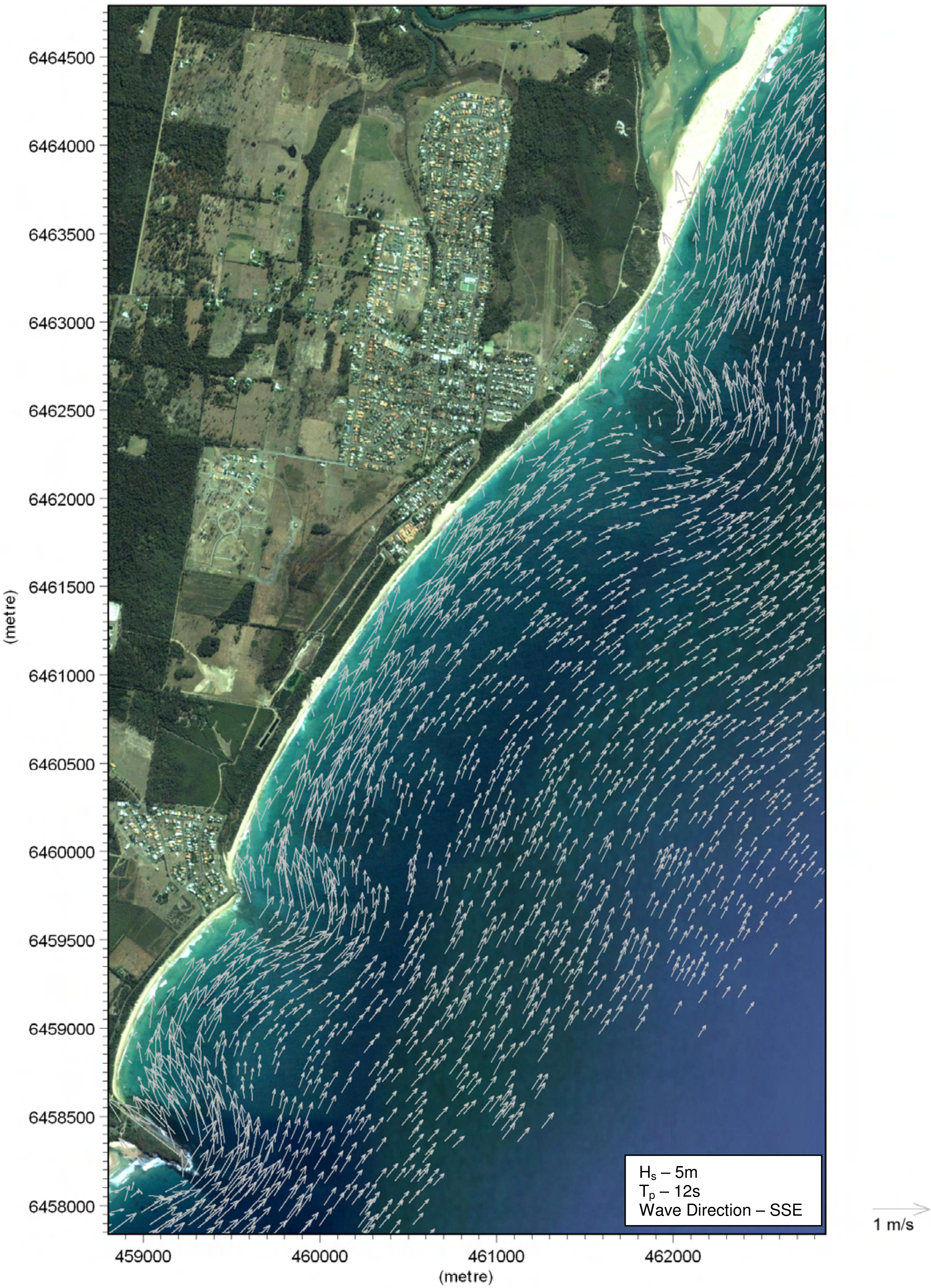
Wave Direction - S

Figure 6A (i)



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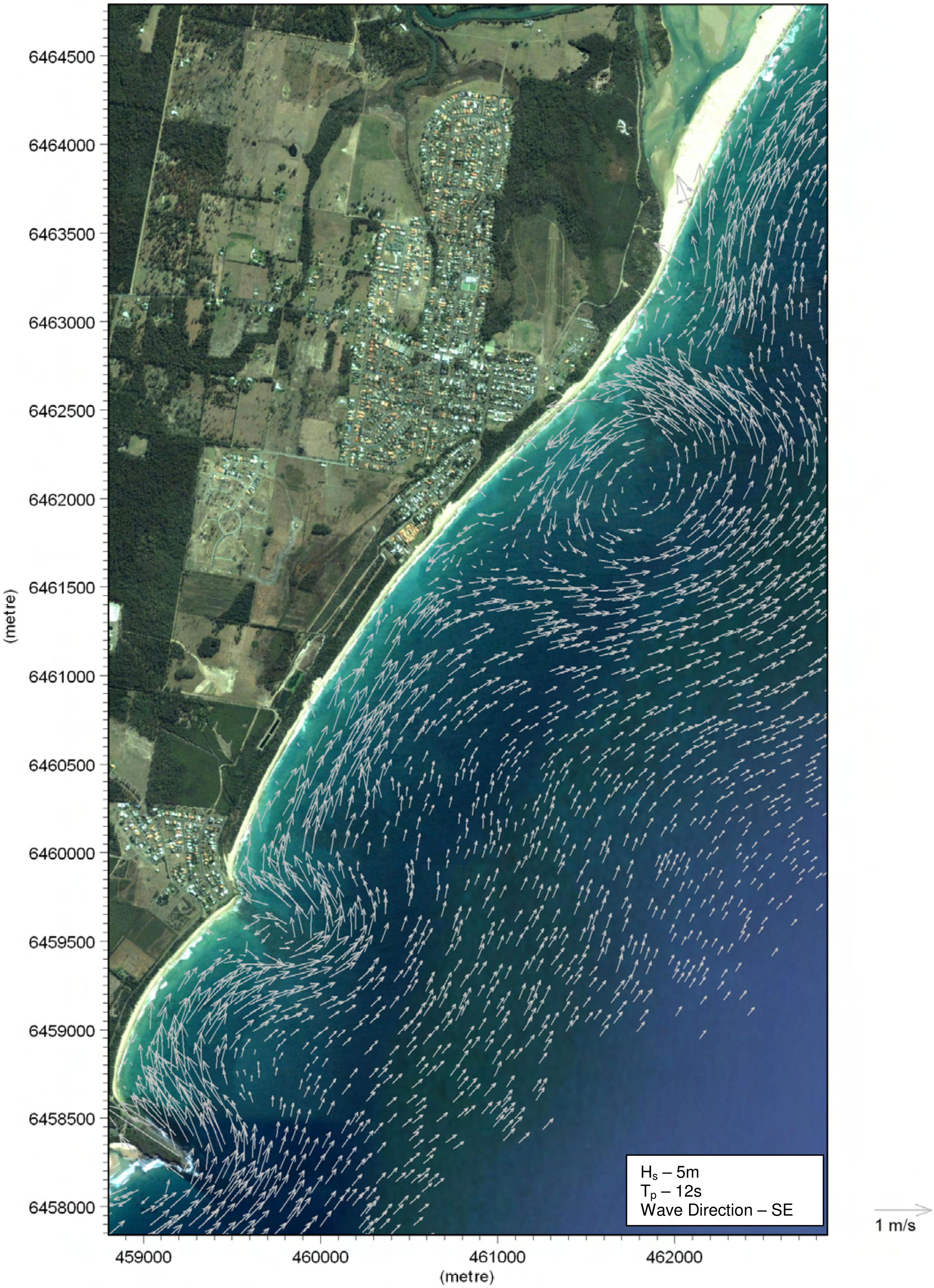
Wave Direction - SSE

Figure 6A (ii)



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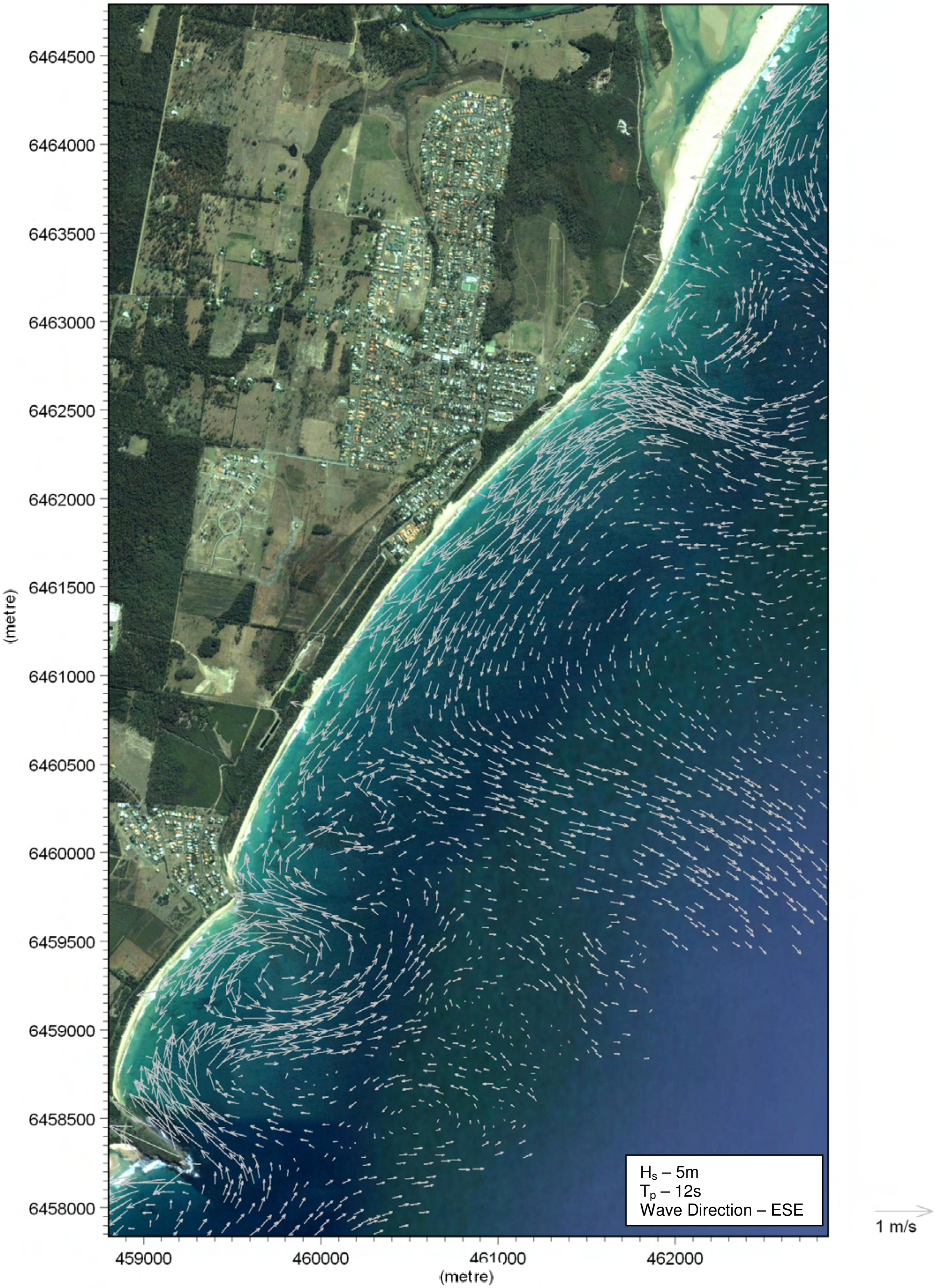
Wave Direction - SE

Figure 6A (iii)



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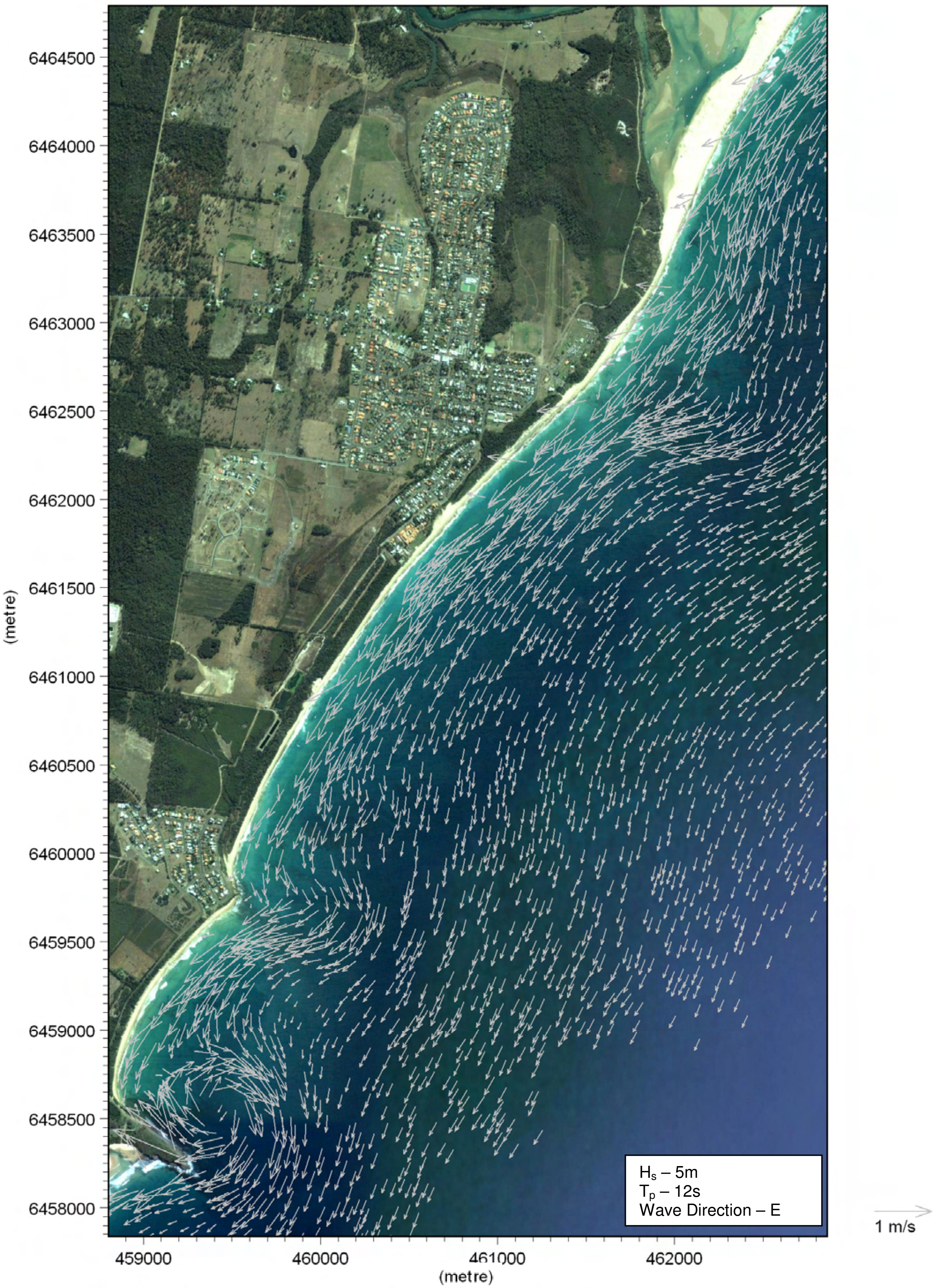
Wave Direction - ESE

Figure 6A (iv)



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Wave Direction - E

Figure 6A (v)

Appendices

GREATER TAREE CITY COUNCIL
BLACK HEAD TO CROWDY HEAD
COASTLINE HAZARD DEFINITION STUDY VOLUME 1: REPORT

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APPENDIX B – PHOTOGRAMMETRIC DATA ASSESSMENT

B1 Introduction

Photogrammetry is the science of measurement and data acquisition from photographic and other remotely sensed images. It was employed in this study to detect and measure historical shoreline changes from suitable vertical aerial photography.

The photogrammetric analysis was undertaken by the Department of Environment and Climate Change (DECCW) using their AC3 photogrammetric instrument.

This appendix describes the methodology used in the photogrammetric analysis as well as providing the "raw" results, but does not contain any interpretation of these results.

B2 Photogrammetry Provided by DECCW

The photogrammetry for Black Head to Crowdy Head was supplied to WorleyParsons by DECCW. The photogrammetry data consisted of 269 cross-shore profiles in 22 blocks covering a total coastline length of approximately 32 km. The data covered the period from 1940 to 2006, details of the dates of photography are outlined below in **Section B.3.**

Each block of cross-shore profiles cover a section of the overall coverage. **Figure 3** shows the locations of each cross-shore profile within each of the blocks. A summary of the each block is provided in **Table B.1.**

Table B.1. Summary of Photogrammetry Cross-shore Profile Blocks

Block Number	Length of Coastline	Number of Profiles	Profile Spacing	Geographical Coverage
BH1	300	6	50	Black Head beach south
BH2	500	10	50	Black Head Beach central
BH3	500	10	50	Black Head Beach north
D1	200	4	50	Diamond Beach south
D2	300	6	50	Diamond beach south

D3	500	10	50	Diamond beach south
D4	500	10	50	Diamond beach central
D5	800	16	50	Diamond beach central
D6	1200	12	100	Diamond beach north
D7	600	6	100	Diamond beach north
D8	1000	10	100	Diamond beach north
SW1	600	12	50	Saltwater Beach south
SW2	800	16	50	Saltwater Beach north
OB3	1530m	16	100m	Wallabi Point to Old Bar Beach
OB4	400m	4	100m	Middle section of Old Bar Beach
OB5	500m	26	20m	Lewis Street and Racecourse Creek
OB6	400m	4	100m	Northern Old Bar Beach
OB7	817m	8	100m	SLSC and caravan park
OB8	1500m	16	100m	Northern end of beach, and Farquhar Inlet.
MI9	4000	20	200	Manning Point Beach south
MI10	6000	30	200	Manning Point Beach north
CH1	3400	17	200	Harrington to Crowdy Head

B3 Aerial Photography

- The vertical aerial photography used in the photogrammetric analysis was selected by DECCW. The first date of photography used was 1940 with the most recent photography being 2006. The photography was selected on the following basis:

- coverage of area;
- time between photography;
- photo quality; and
- scale.

A summary of the photography used in the analysis is provided below in **Table B.2.** Details of the quality of the photography used are also outlined. In this table ‘Photo quality’ refers to the definition of detail under magnification and ‘Model control’ refers to how well the photography can be set up in order to obtain quantitative data.

Table B.2.Summary of the Aerial Photography used in Photogrammetry

Year	Date	Photo scale (1:X)	Photo quality	Model control	Blocks Covered																					
					BH 1	BH2	BH3	D1	D2	D3	D4	D5	D6	D7	D8	SW1	SW2	OB3	OB4	OB5	OB6	OB7	OB8	MI9	MI10	CH1
1940	Dec 1940	17,000	Poor	Reasonable												✓	✓	✓	✓	✓	✓			✓	✓	
1963	23/11/63	40000	Reasonable	Reasonable	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓									
1965	16/01/65	40,000	Reasonable	Reasonable												✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
1970	13/08/70	16,000	Good	Good				✓	✓	✓	✓	✓	✓	✓	✓			✓	✓	✓						
1972	24/2/72	40000	Reasonable	Reasonable	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓									✓	✓	✓
1976	02/12/76	40000	Reasonable	Poor	✓	✓	✓	✓	✓	✓	✓	✓												✓	✓	
1981	13/06/81	25000	Reasonable	Reasonable	✓	✓	✓																			
1981	27/11/81	10,000	Good	Good	✓	✓	✓												✓	✓		✓	✓	✓	✓	
1986	22/04/86	10000	Good	Good	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓

Year	**Date**	**Photo scale (1:X)**	**Photo quality**	**Model control**	**Blocks Covered**																					
					BH1	BH2	BH3	D1	D2	D3	D4	D5	D6	D7	D9	SW1	SW2	OB3	OB4	OB5	OB6	OB7	OB8	MI9	MI10	CH1
1989	19/06/89	10,000	Good	Good															✓	✓	✓					
1991	31/08/91	25,000	Good	Reasonable															✓	✓	✓					
1993	20/06/93	10,000	Good	Good												✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	
1996	31/05/96	10000	Good	Good	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓				✓	✓	✓					
1997																										✓
2000	17/05/00	10,000	Good	Good															✓	✓	✓			✓	✓	
2004	31/07/04	10,000	Good	Good												✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	
2006	11/11/06	10000	Good	Good	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓											
2006	26/11/06	10,000	Good	Good												✓	✓		✓	✓	✓	✓	✓	✓	✓	✓

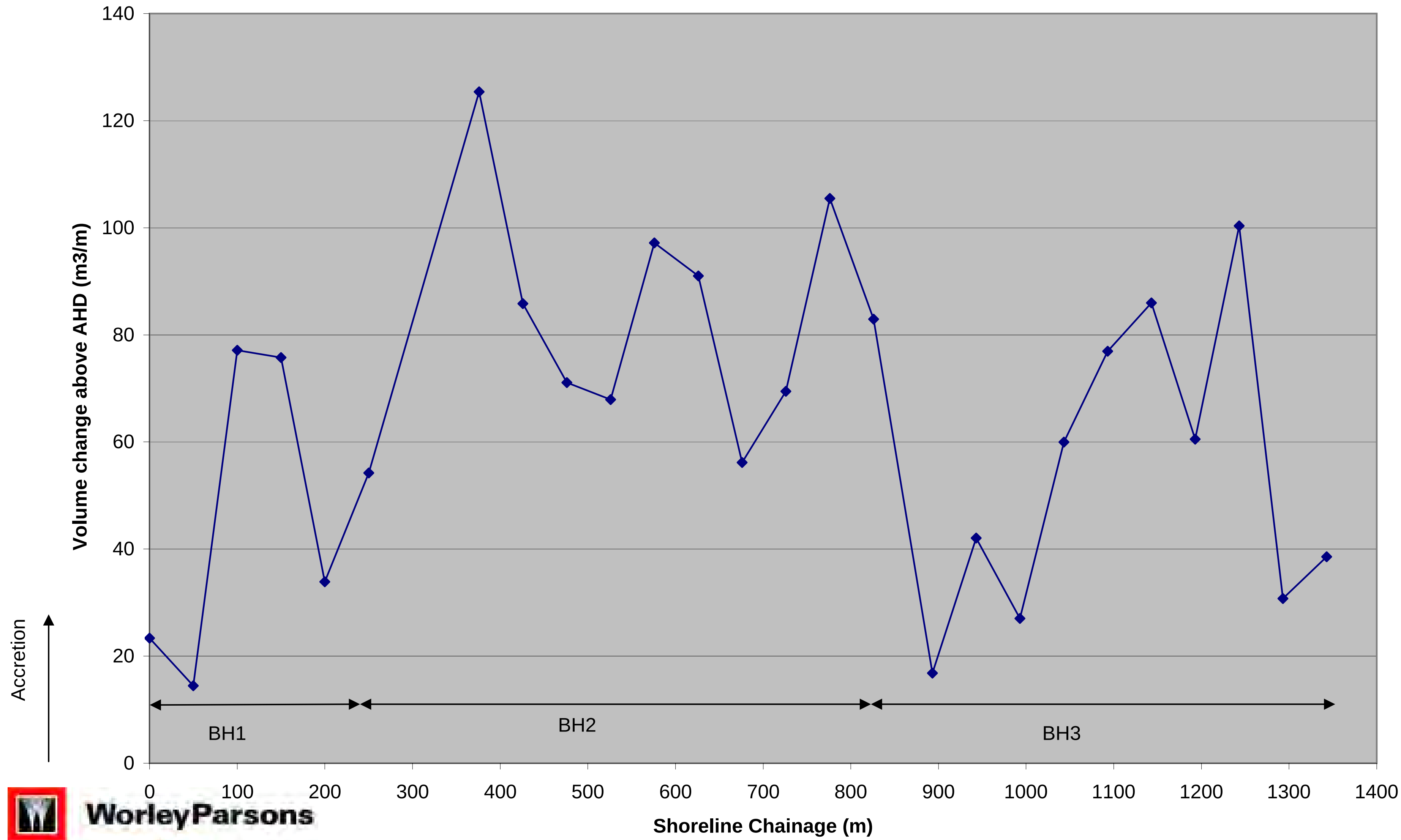
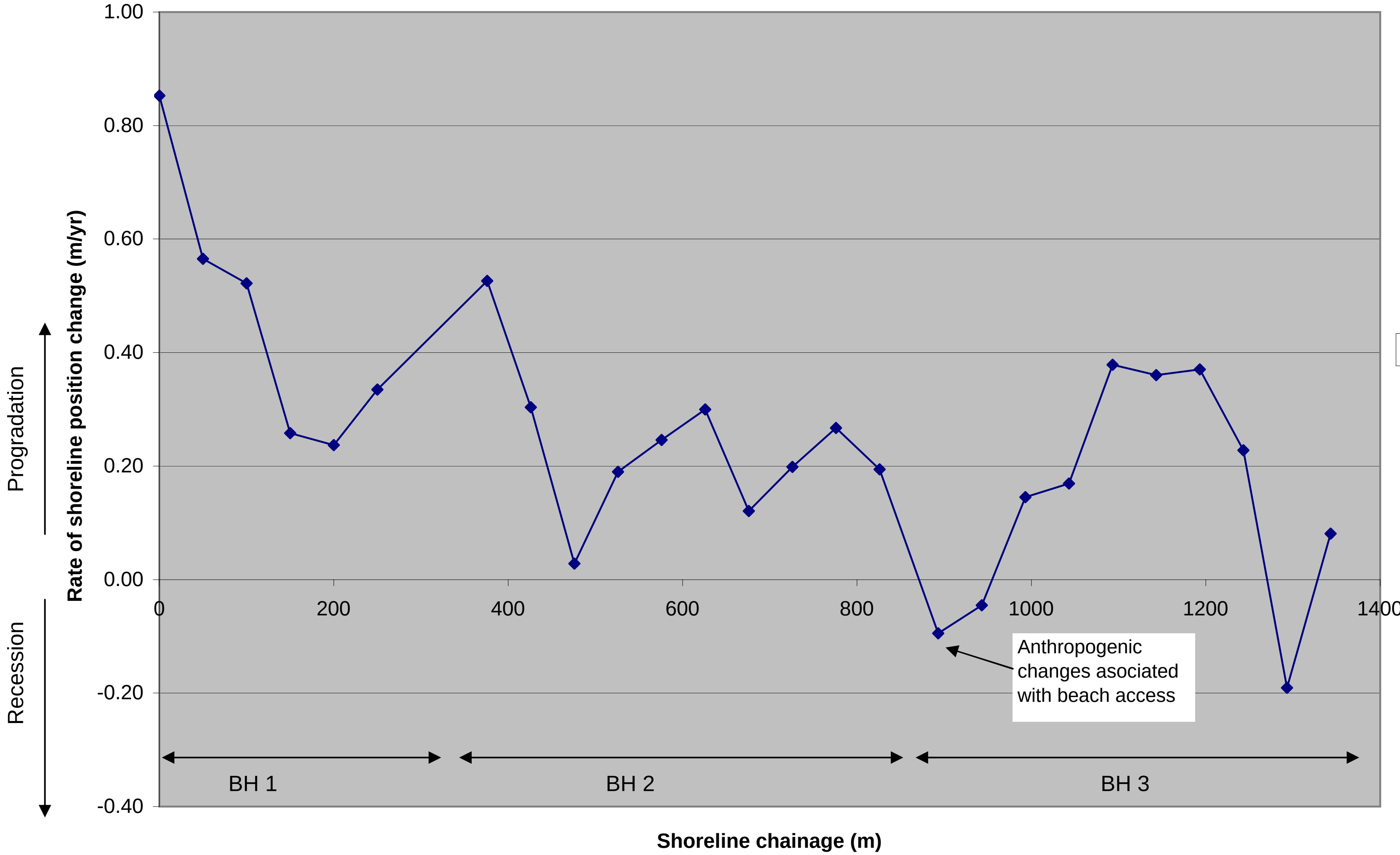


Figure B1
Black Head Beach
Net Volume Change 1963-2006



Note: * 3m AHD contour generally used. 1m AHD contour used for profiles 1 to 3 of B1.

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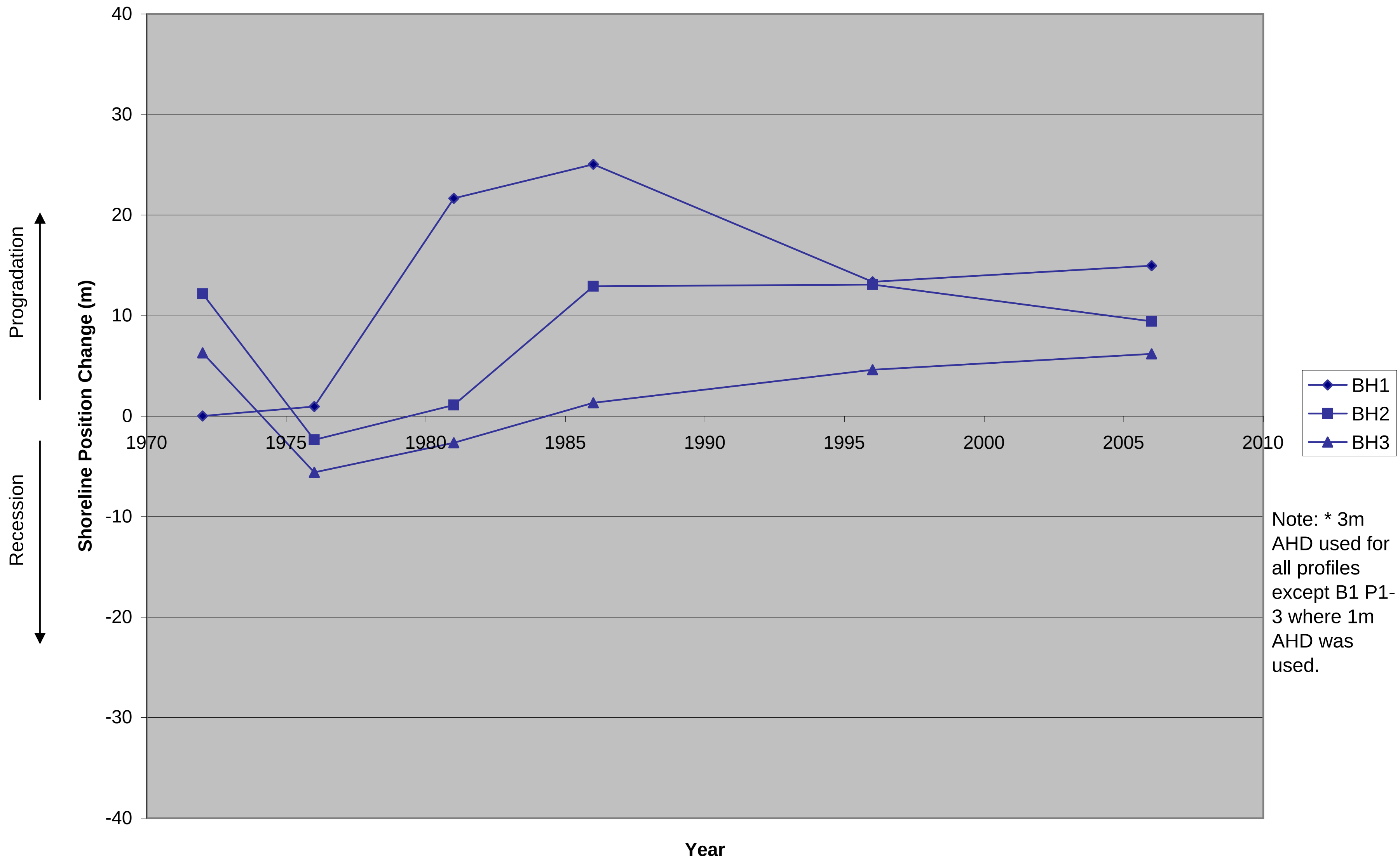
Figure B2
Black Head Beach
Annual Rate of Shoreline Position* Change



Note: * 3m AHD used for all profiles except B1 P1-3 where 1m AHD was used.

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Figure B3
Black Head Beach
Block Averaged Shoreline Position* Change relative to 1963

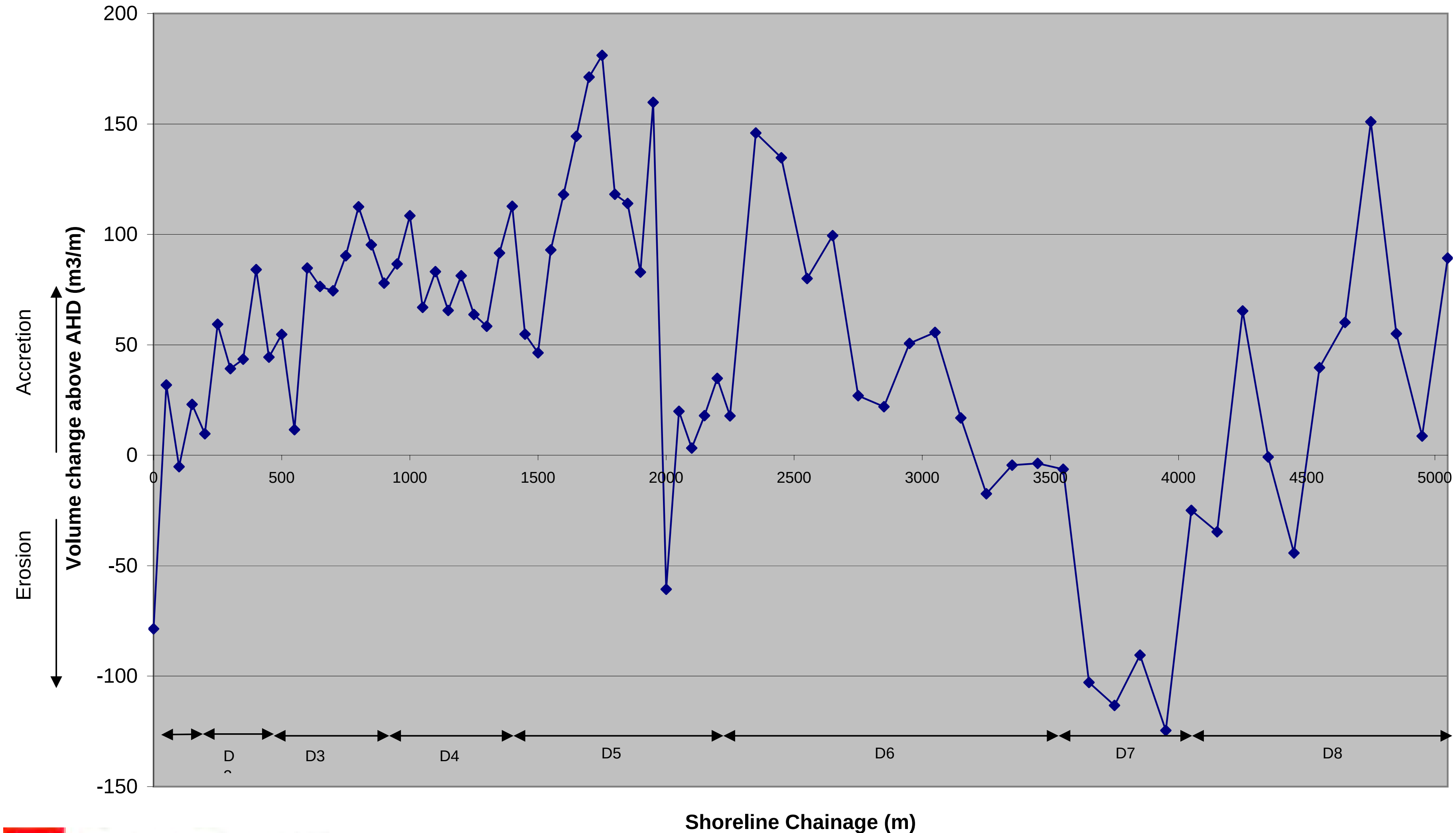

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Figure B4
Diamond Beach
Net Volume Change 1963-2006

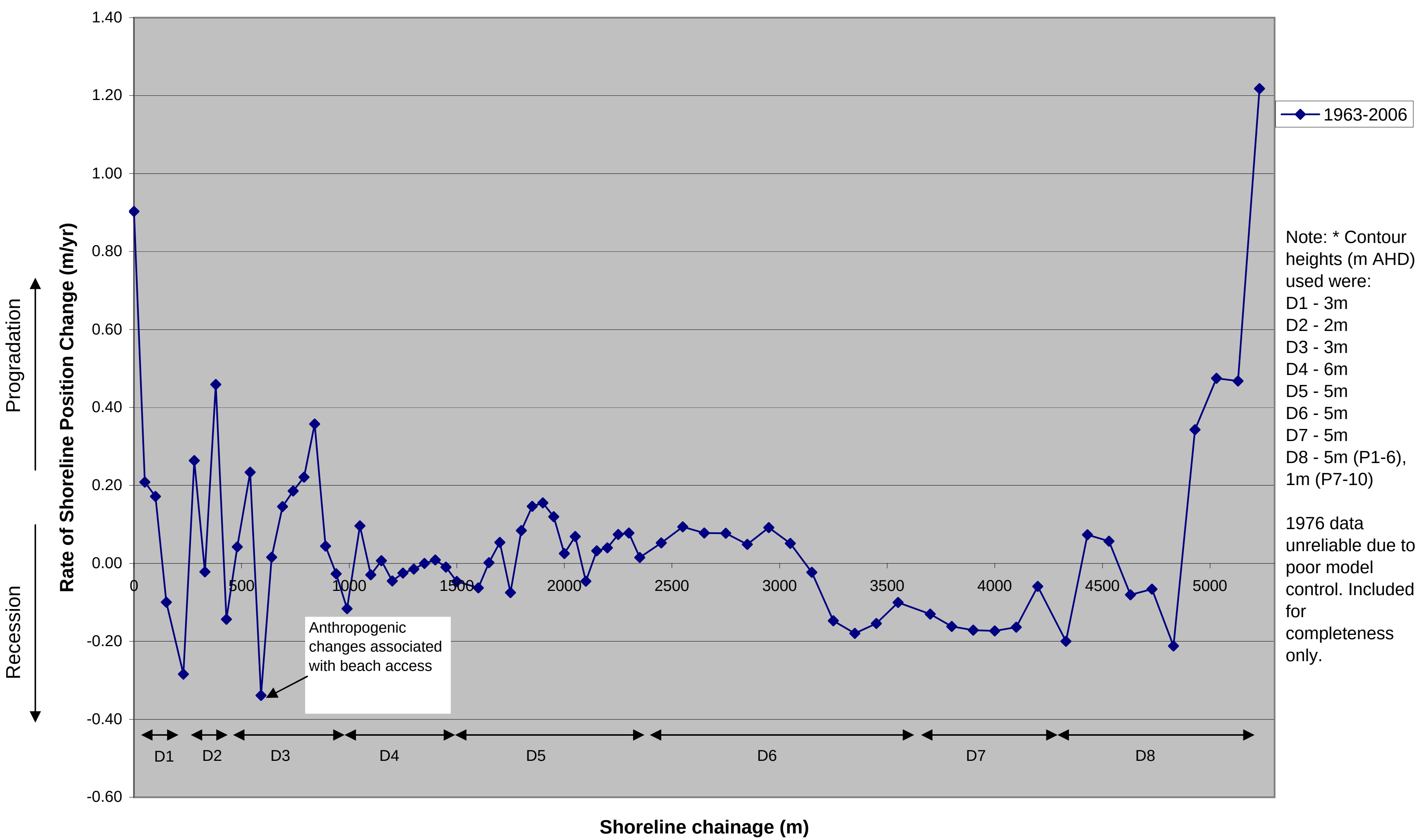

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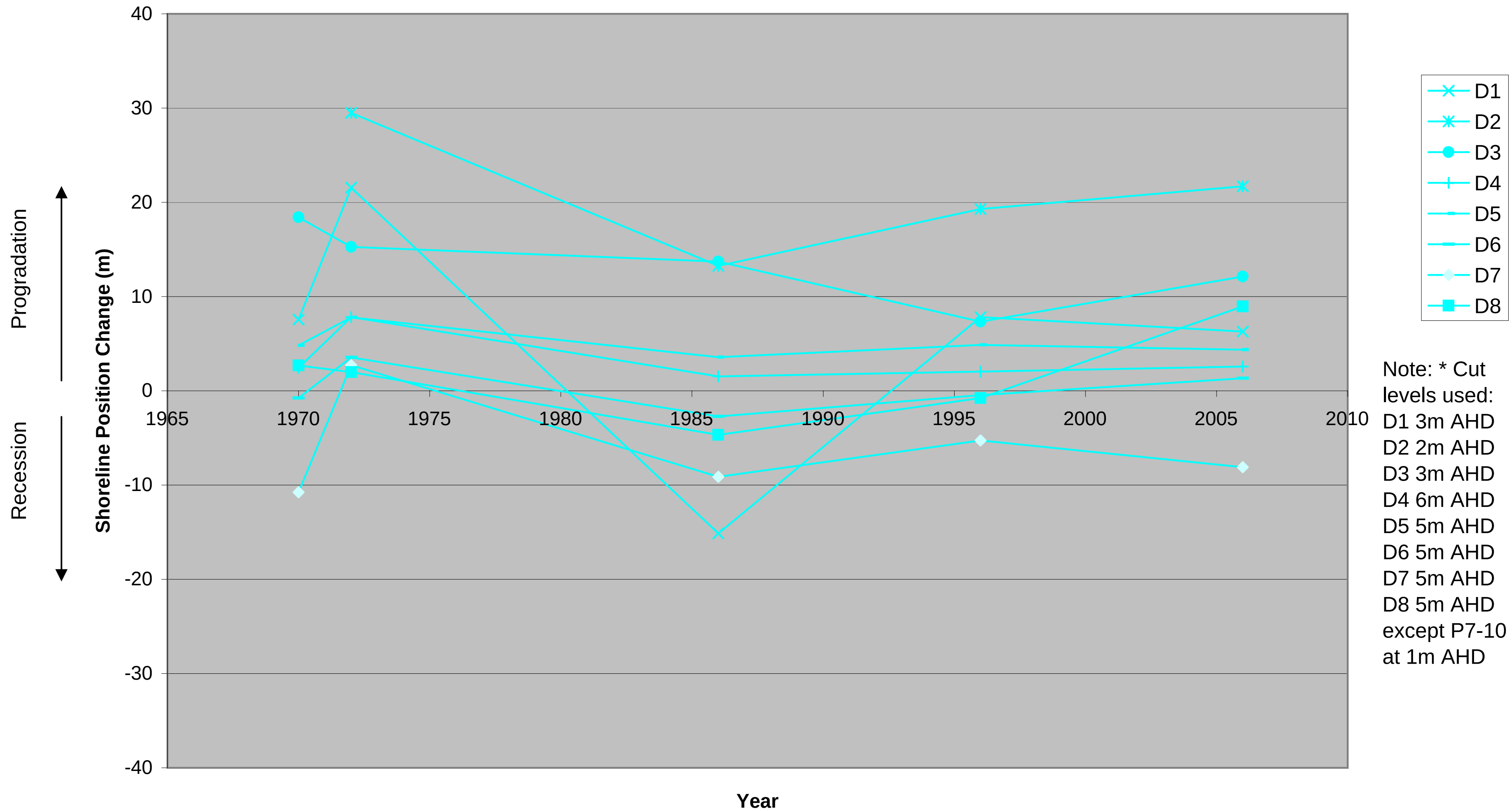
Figure B5
Diamond Beach
Annual Rate of Shoreline Position* Change



Note: * Cut levels used:
D1 3m AHD
D2 2m AHD
D3 3m AHD
D4 6m AHD
D5 5m AHD
D6 5m AHD
D7 5m AHD
D8 5m AHD except P7-10 at 1m AHD

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Figure B6
Diamond Beach
Block Averaged Shoreline Position* Change relative to 1963

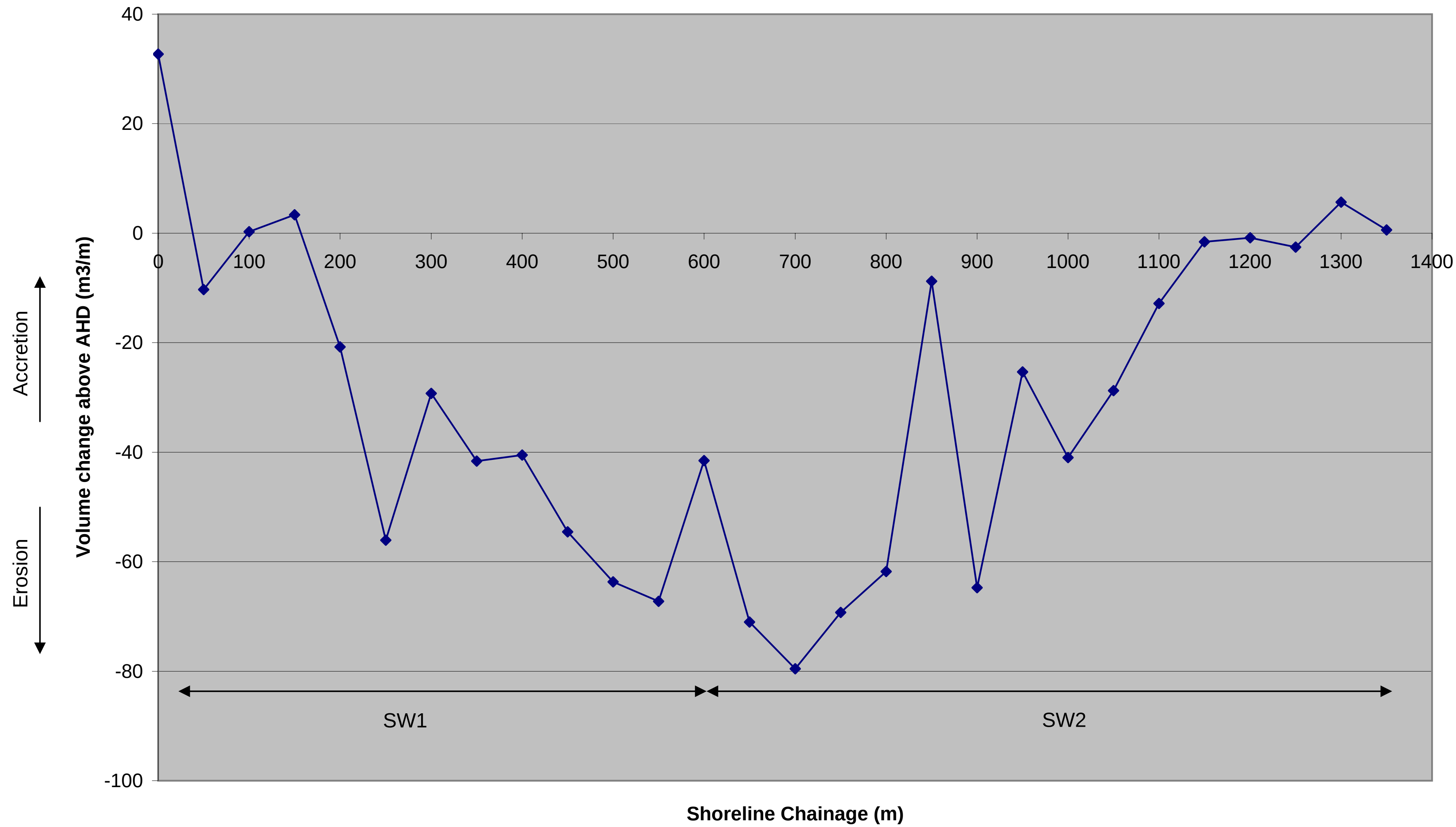

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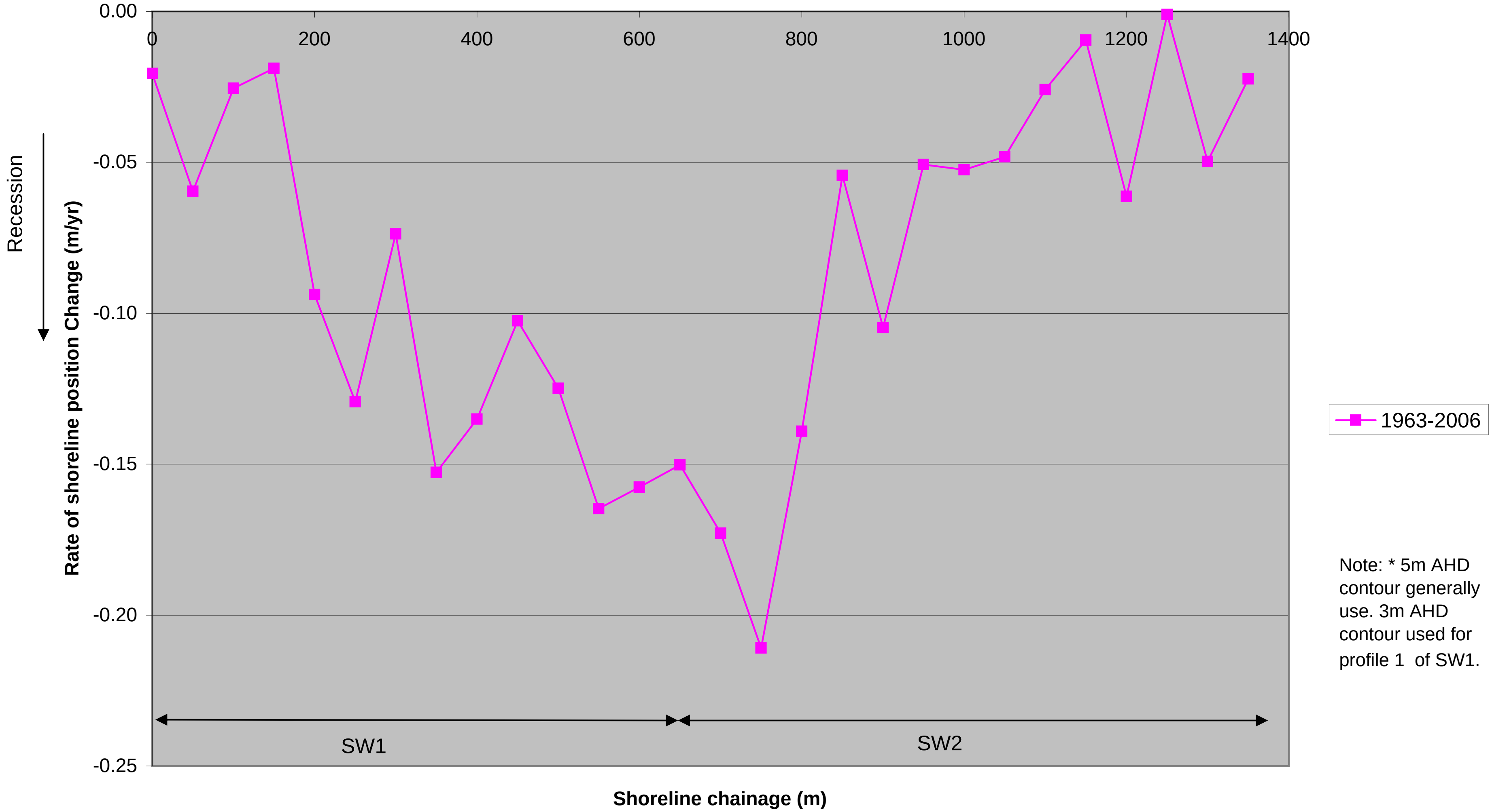
Figure B7
Saltwater Beach
Net Volume Change 1963-2006

O:\301017-00051 - 7257 Old Bar Coastal Study and Managemet Plan (DM)\03_Data\Photogrammetry\Anaylsis\Excel Files\04 Saltwater Bch\Analysis\Contour RecessionSW.xls
6/08/2009



Note: * 5m AHD contour generally use. 3m AHD contour used for profile 1 of SW1.

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Figure B8
Saltwater Beach
Annual Rate of Shoreline Position* Change

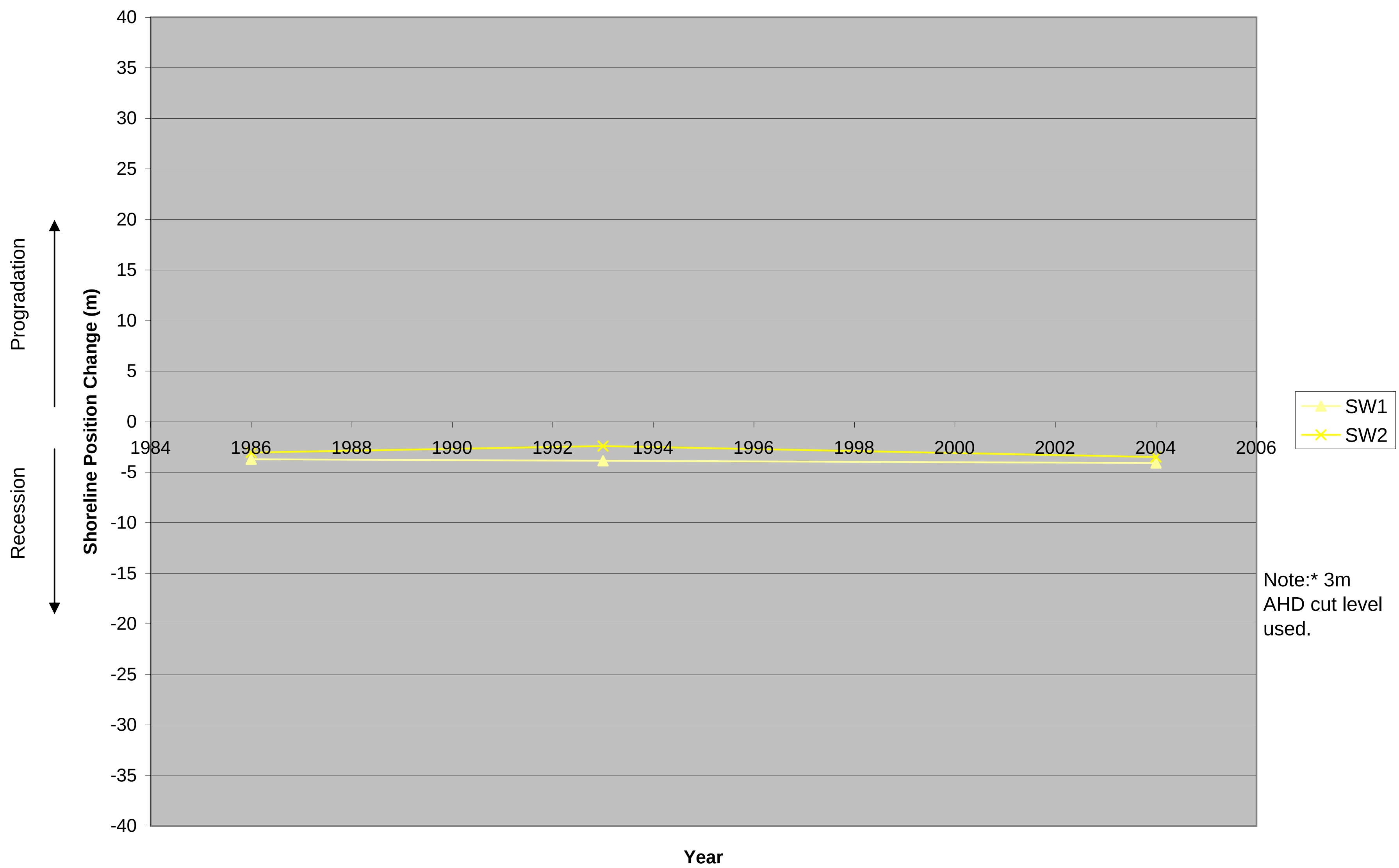

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Figure B9
Saltwater Beach
Block Averaged Shoreline Position* Change relative to 1963

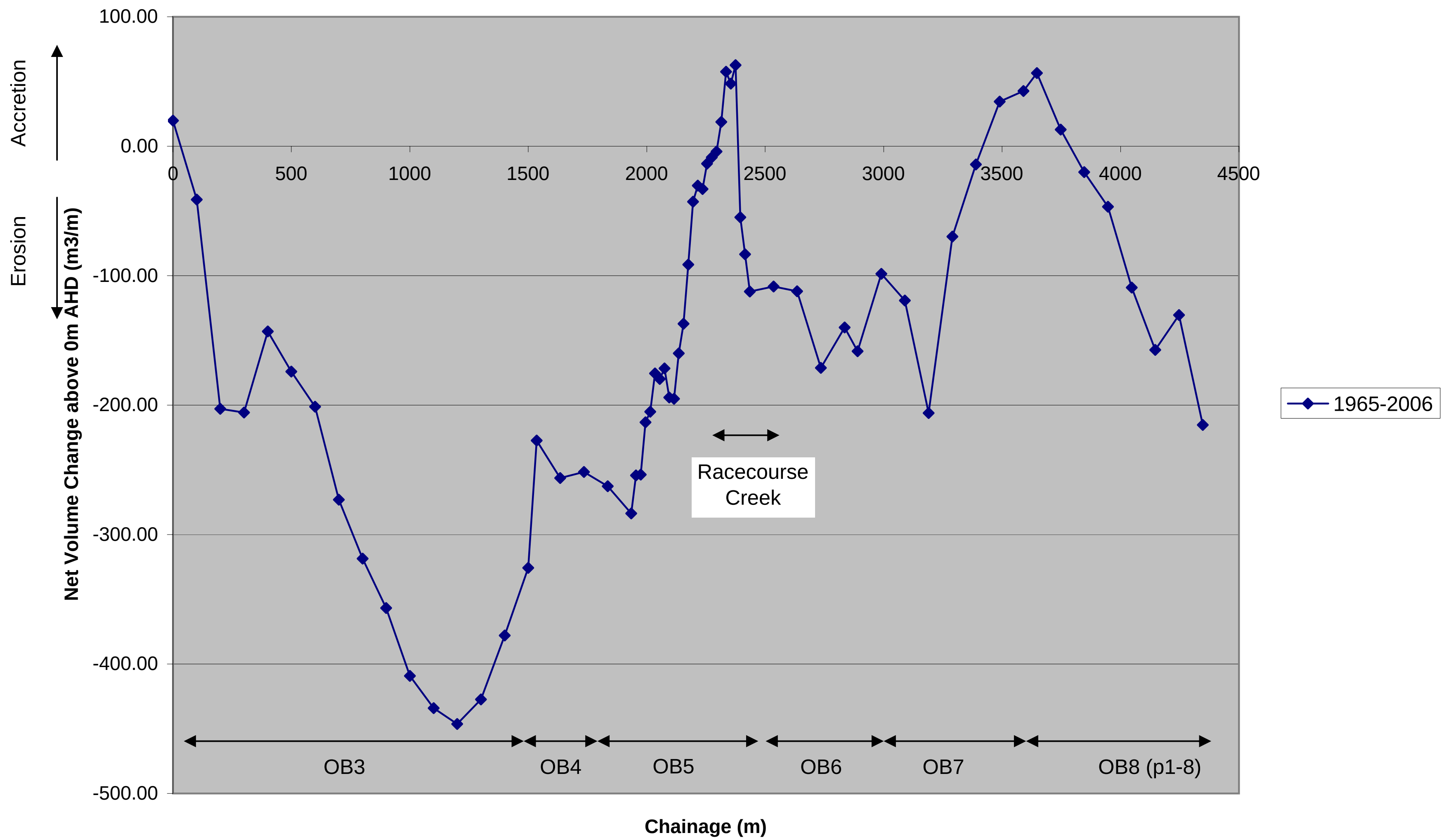

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Figure B10
Old Bar Beach
Net Volume Change 1965-2006

O:\301017-00051 - 7257 Old Bar Coastal Study and Managemet Plan (DM)\03_Data\Photogrammetry\Anaylsis\Excel Files\03 Old Bar\Extended scope\Recession_Anaylsis_ekw_v7.xls
6/08/2009

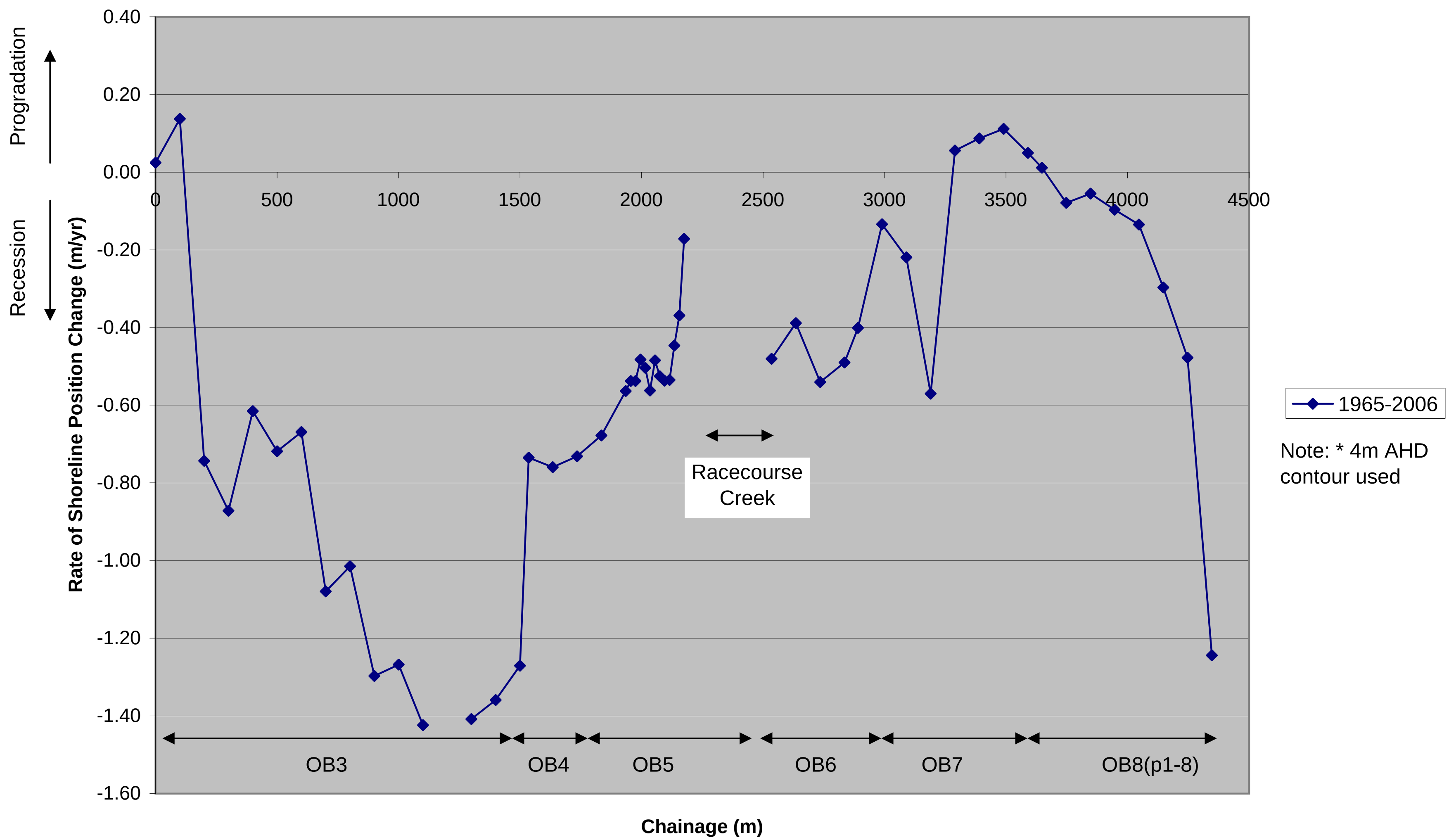

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Figure B11
Old Bar Beach
Annual Rate of Shoreline Position* Change

O:\301017-00051 - 7257 Old Bar Coastal Study and Managemet Plan (DM)\03_Data\Photogrammetry\Anaylsis\Excel Files\Blck Average recession plot.xls
6/08/2009

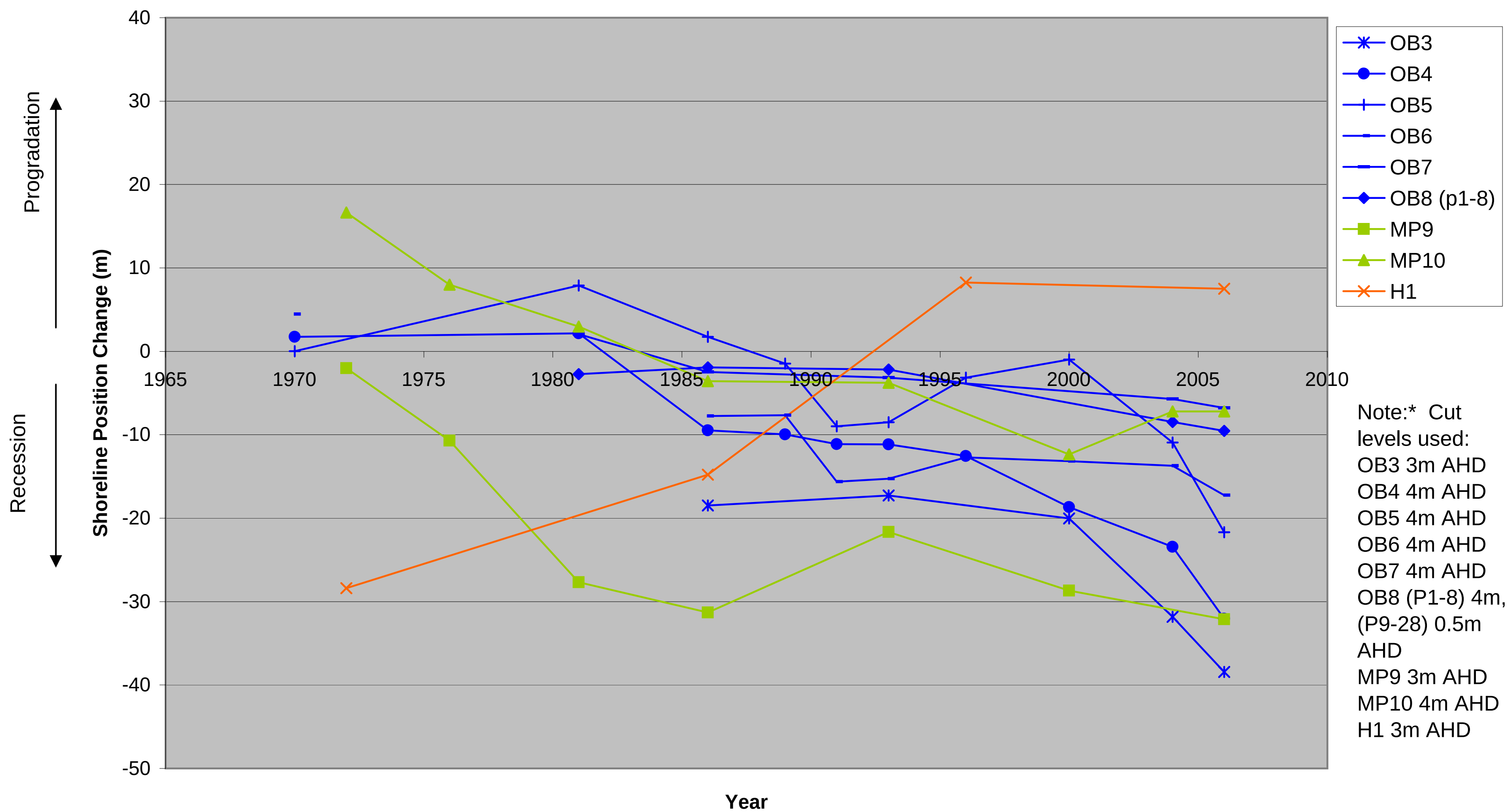

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Figure B12
Old Bar, Manning Point and Harrington Beaches
Block averaged Shoreline Position* Change relative to 1965

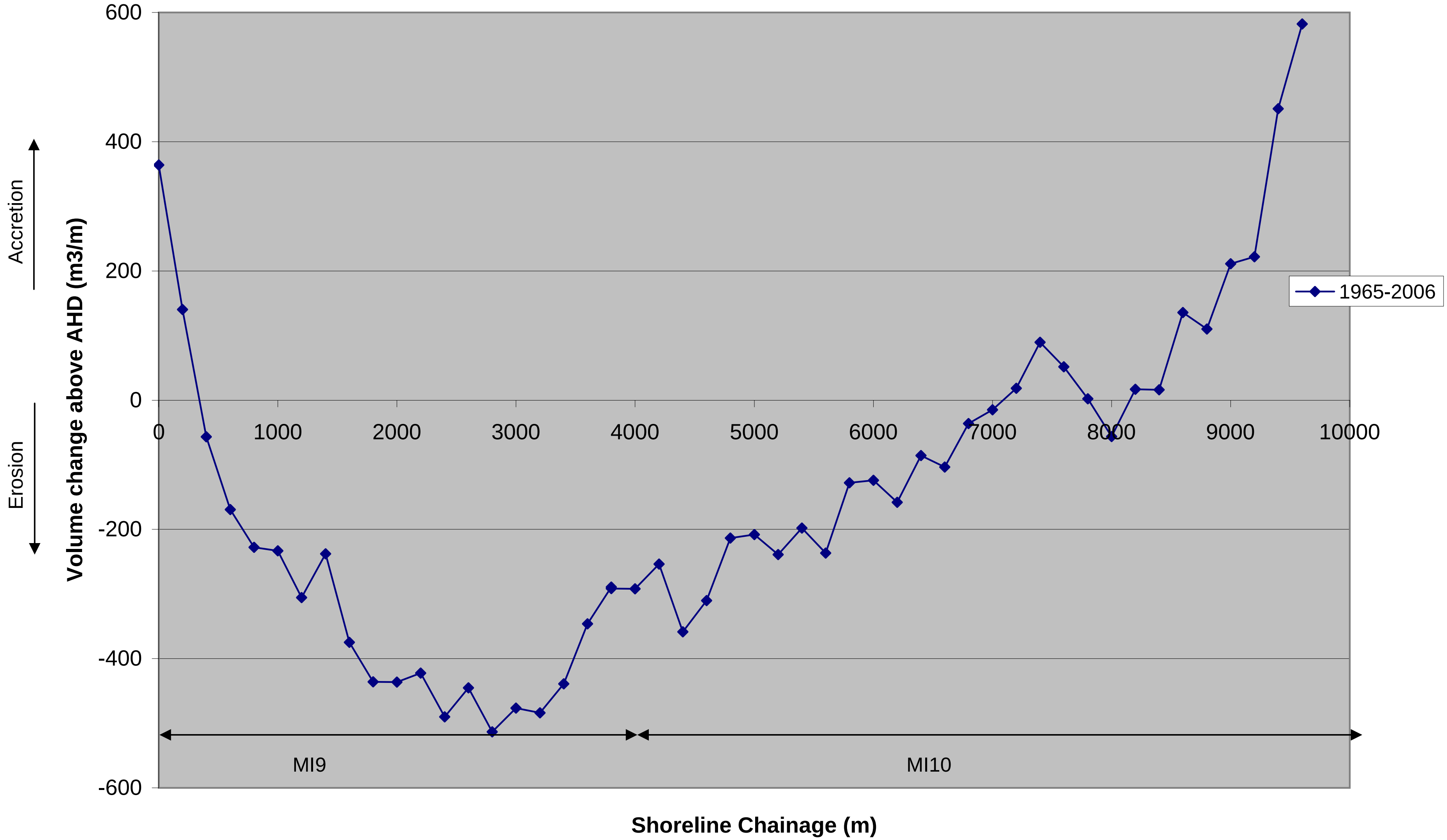

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Figure B13
Manning Point Beach
Net Volume Change 1965-2006

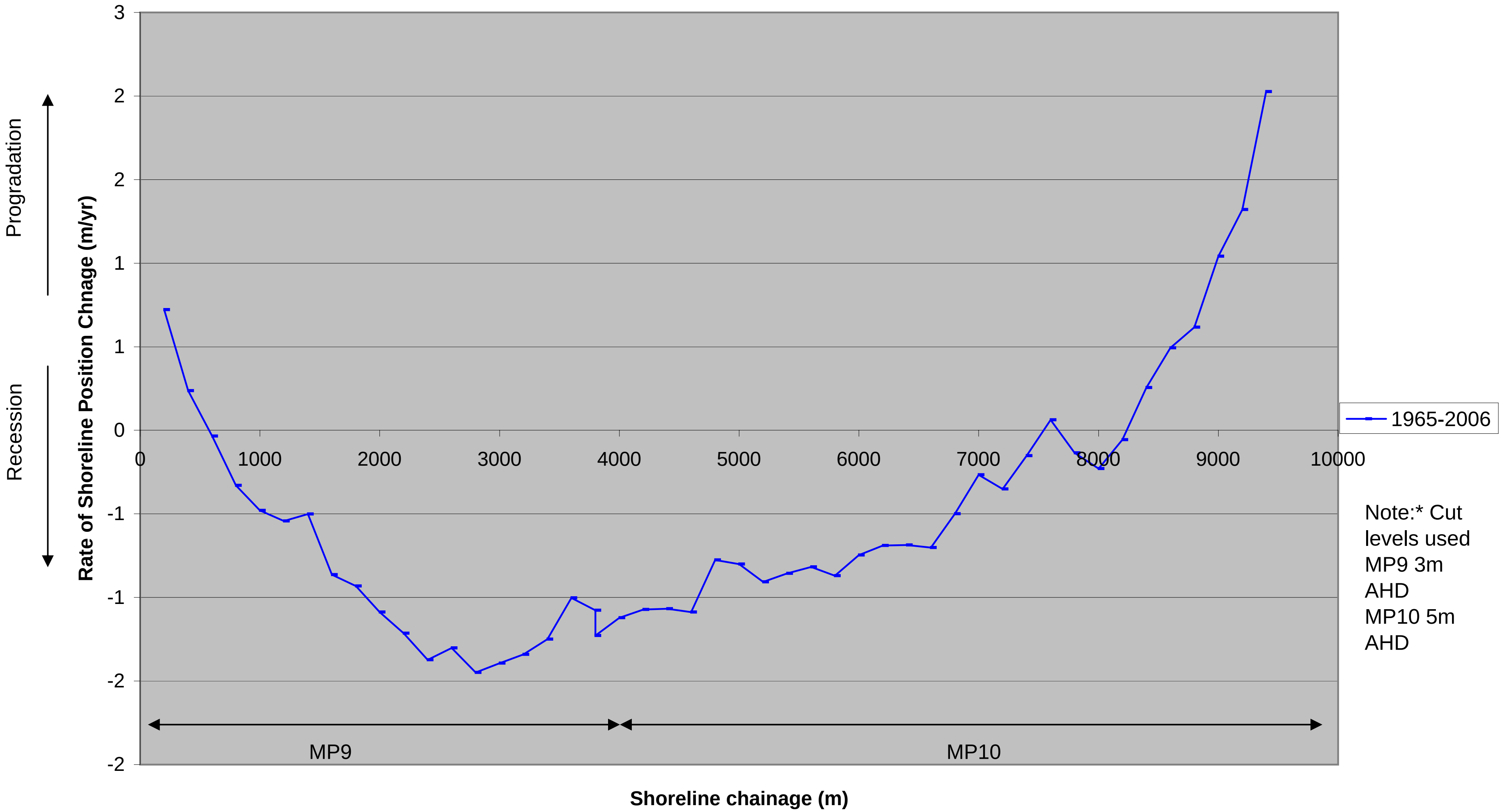

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Figure B14
Manning Point Beach
Annual Rate of Shoreline Position* Change

O:\301017-00051 - 7257 Old Bar Coastal Study and Managemet Plan (DM)\03_Data\Photogrammetry\Anaylsis\Excel Files\01 Harrington Bch\Analysis\VolumeCH.xls
6/08/2009

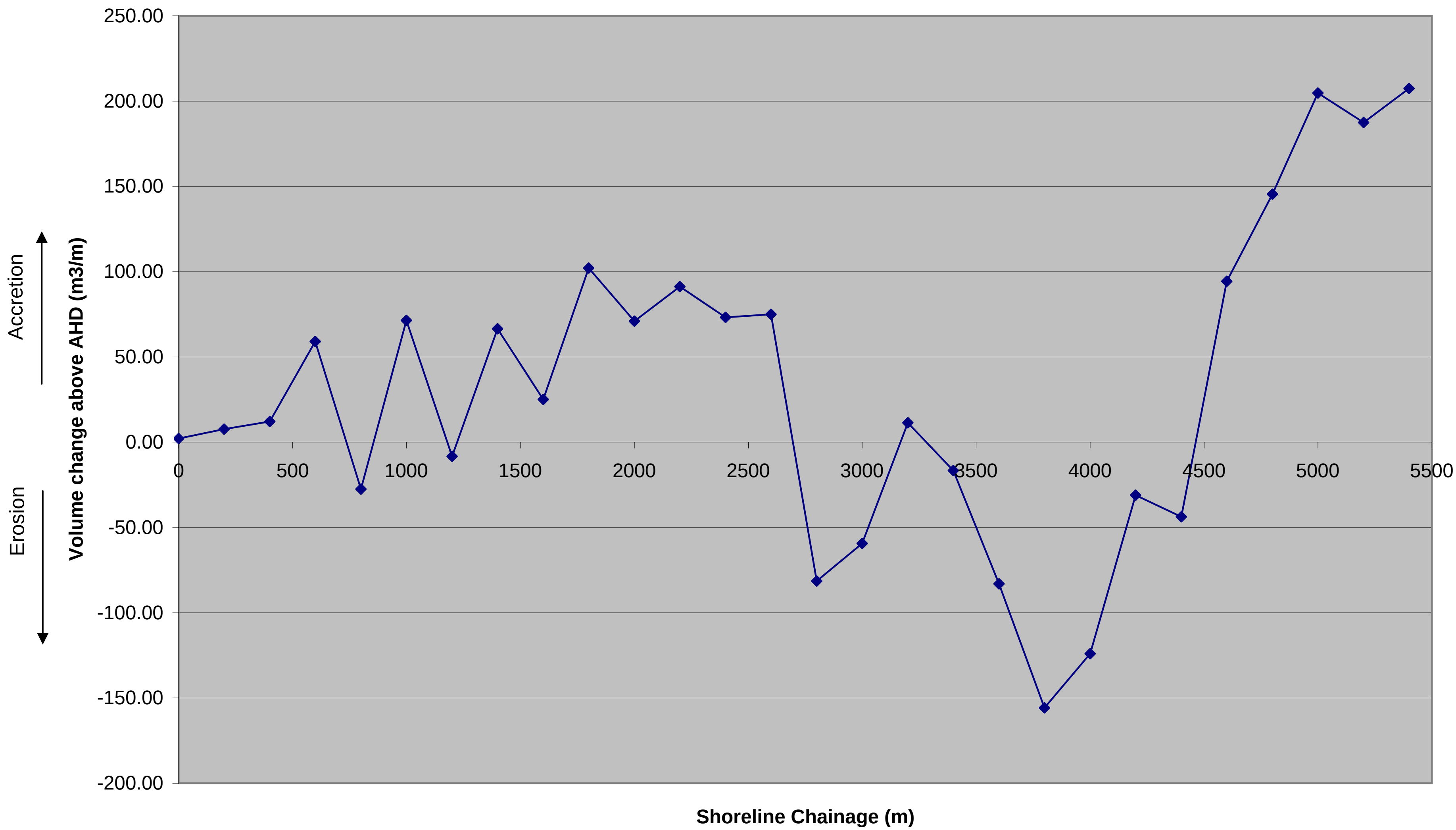

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Figure B15
Harrington Beach
Net Volume Change 1965-2006

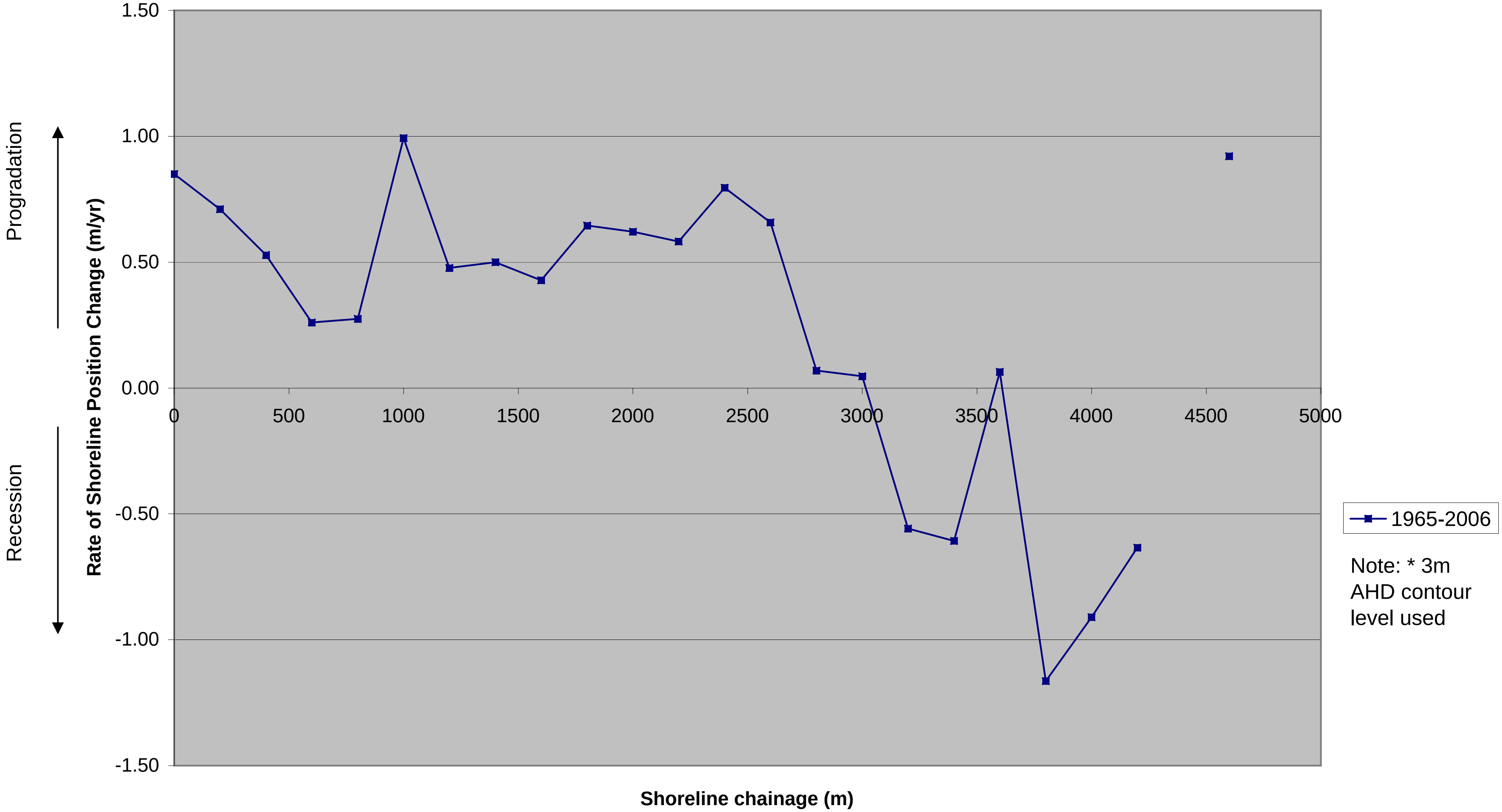

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Figure B16
Harrington Beach
Annual Rate of Shoreline Position* Change

APPENDIX C – Review of Historical Aerial Photographs

Aerial photography is available for the Black Head to Crowdy Head coastline dating back to 1940. A summary of some of the distinctive features of various dates of photography is set out below on a beach by beach basis (north to south).

C.1 Harrington Beach

1940 - Well vegetated healthy dunes in north – some aeolian transport evident. Many progressive rows of dunes behind evident

1965 - Sand mining evident (exposed sand on dune behind road) in northern part of beach. Blow outs evident particularly in south (back to road in one location)

1969 - Large blowouts evident throughout – particularly north and south ends

1976 - Blowout covering southern portion of beach. Revegetation of previously exposed area behind road. Vegetation generally patchy throughout with scares evident due to localised blowouts and sand mining. Beach locally held out at end of road at north end.

1979 - Scarp evident along back beach. Huge blowout (aeolian) sand still covering southern area back to road

1980 - Huge blow out (aeolian) sand still covering southern area back to road. Otherwise, vegetation improving on dune. Beach almost full to end of breakwater in south.

1986 - Beach almost full to end of breakwater in south. Light vegetation coverage on large blowout in south. Feature still evident at end of road in north holding beach out locally.

1989 - Vegetation thickening. Vegetation at south more dense. Numerous rips. Bar offshore of feature just south of road (in north)

1993 - Rock evident offshore of road in north. More vegetation over blowout in south

1996 - South end of beach has receded back along training wall. Vegetation good. Lagoon filling in and sand overtopping training wall – sediment pathway against training wall for aeolian transport.

2000 - South end of beach has receded back along training wall. Vegetation good. Lagoon filling in and sand overtopping training wall – sediment pathway against training wall for aeolian transport.

2004 - Rock/reef feature offshore of road (north) evident and gravel in nearshore zone and on the beach.

C.2 Harrington Inlet

Refer also **Figure 2** and **Section 4.11.2**

1965 - Channel in middle (not up against training wall). Sand stopping training wall back at lagoon. Plumes of sand offshore of spit down to Manning Point. Some vegetation on back edge of spit

1969 - Channel in middle, a lot of sand to north of channel

1976 - More sand overtopping training wall, channel still in middle. More extensive blow out covering southern quarter of beach.

1983 - Channel against training wall.

1986 - Channel against training wall

1989 - Channel against training wall

1993 - Narrow channel against training wall and larger channel in middle. Large build up of sand (dry) in between. Vegetation has extended further north along spit and widened/thickened

1997 - Wide channel in middle – extending from vegetation; area of spit large deposit of sand against training wall, setback from Harrington coastline.

C.3 Manning Point Beach

1996 - Large sediment plumes offshore of Manning Point evident

C.4 Farquhar Inlet

Refer also **Figure 2** and **Section 4.11.1**

1997 - Wide opening south end

2004 – Inlet closed

C.5 Old Bar Beach

1940 - distinct lack of vegetation along the dune system The vegetation is patchy, isolated and limited to the dune crests. Numerous dune blow-outs resulting from aeolian transport are evident, particularly between Racecourse Creek and Farquhar Inlet.

1970 -vegetation of the Old Bar dunes from Wallabi Point to the central portion of the beach was similarly very patchy at this time with a number of dune blow-outs. This is likely to be associated with historical sand mining activities in the area as discussed in **Section** 2.2.2.

1980 - vegetation was then firmly established along the dunes and further northwards to Farquhar Inlet. The only exceptions were the sand mining area behind the dune midway along Old Bar beach and popular access points at the SLSC and caravan park.

1983 - South creek closed

1993 – Racecourse Creek groyne constructed (20-6-93) and dune nourished. South creek closed.

1996 - Some light vegetation on Racecourse Creek dune. Large rip head just north of Racecourse Creek. Sand lobe evident off SLSC shoreline. South creek closed

1997 - Suspended sediment evident near shore at Urana Bombora (SLSC to caravan park) appears to be moving in southerly direction.

2000 - More vegetation on Racecourse Creek dune. Large rip and sediment plume off Meridian Resort. Suspended sand evident at blocks 06, 07 and 08 up to Farquhar Inlet channel at south end of Farquhar Inlet. South creek closed

2004 - Gravel on beach. A lot of suspended sediment between Racecourse Creek and caravan park. South creek closed. Gravel on beach. Large rip off low lying area near Midcoast Water treatment plant basins.

C.6 Saltwater Beach

1970 - ENE swell, large rip at south end – plume and bypassing to south

1972 - NNE swell, large rip at south end – plume and bypassing to south

1981 - NNE swell, large rip at south end – plume evident

1996 - Large rip at north end and in middle. ESE swell

2000 - A lot of gravel on beach (except southern ¼) Large rips at north and south ends with sediment plumes

2006 - Rips and plumes at north and south ends. NE swell

C.7 Diamond Beach

The main features identified in the review of historical photographs of Diamond Beach were:

- The small creek at the south end of Diamond Beach has at various times flowed either northwards (up to 130m) or southwards (up to 150m) after it crossed the foredune.
- Khappinghat Creek mouth was open only in the 1974 aerial photographs.
- Two extensive blowouts of the frontal dune are evident in the 1937 photography. These became progressively revegetated with time.
- Following the establishment of Diamond Beach Estate (1963) there was some loss of vegetation on the frontal dune along the estate.

1996 - Extensive sand extraction evident in middle of beach behind dune

2004 - Recovering from sand extraction. Mostly vegetated. Creeks closed

2006 - More vegetation on sand excavation area. North creek closed. South creeks together but closed

C.8 Black Head Beach

1980 - Black Head Lagoon closed at back of beach

1981 - Black Head Lagoon almost extending to water, hugging back beach

1983 - Black Head Lagoon closed

1986 - Black Head Lagoon closed

1989 - Black Head Lagoon closed

2004 - Black Head Lagoon just dribbling through to beach. Rock offshore of whole beach evident

2008 - Black Head Lagoon closed