WAVERLEY COUNCIL

Waverley

Coastal Risks and Hazards Vulnerability Study

301015-02526-CS-REP-0001

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COASTAL RISKS AND HAZARDS VULNERABILITY STUDY

.

PROJECT 301015-02526 – WAVERLEY COASTAL RISKS AND HAZARD VULNERABILITY STUDY

REV	DESCRIPTION	ORIG	REVIEW	WORLEY-PARSONS APPROVAL	DATE	CLIENT APPROVAL	DATE
A	For Progress Discussion (incomplete - unchecked)	D Gooch/ L Nielsen	-				
B	Draft For Internal Review	D Gooch/ L Nielsen	L Nielsen				
C	Draft For External Review	D Gooch/ L Nielsen	L Nielsen				
0	Final	D Gooch/ L Nielsen	L Nielsen				
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EXECUTIVE SUMMARY

The coastal hazards and climate change vulnerability of the beaches and cliffs of Waverley Council (Council) have been assessed. Generally, the hazard risk to the coastal assets of Waverley is low. The cliff faces and foreshore slopes have a ‘tolerable’ to ‘acceptable’ risk of instability. Under existing conditions and into the near future on-going monitoring and periodic geotechnical assessments are an appropriate method of coastal zone and landslide risk management.

However, projected sea level rises over ensuing decades would be likely to cause reductions in the widths of Waverley’s beaches, resulting in the gradual reduction in the amenity of all the beaches. Further, there would be an increasing risk to the stability of the seawalls during storms as sand is eroded from the beaches. Overtopping of seawalls also would increase with time due to sea level rise. As the relative crest levels and rock levels are lowered larger waves would reach the coastal lots and seawalls, increasing the risk of inundation to foreshore buildings, increasing the risk to pedestrians promenading during storms and increasing the risk to the structure of the seawalls.

More intense storm events and elevated sea levels would result in elevated erosion rates over a greater height of the cliff faces and their bases, which would be likely to lead to localised collapses of undercuts and potential instability of the toe areas of the fill batter slope within Waverley Cemetery.

Lots with the potential to be at risk from coastal or geotechnical hazards have been identified. The following recommendations for management are made:

- Information on coastal/geotechnical hazards be incorporated into Councils planning instruments.
- Notification of lots potentially affected by coastal or geotechnical hazards be undertaken by the inclusion of appropriate annotations on Section 149 Certificates. Wording is suggested.
- Beach nourishment be considered should the projected sea level rise occur and sands are eroded from Waverley’s beaches.
- Signs warning of potential cliff face instability to be provided in all publicly accessible areas along the crest and basal areas and specific sites are nominated.
- Property owners be advised to post similar warning signs where yard areas extend to the cliff edge. Specific sites are nominated.
- Fence lines be provided at several locations as specified.
- Monitoring be undertaken on all the study beaches and identified potential cliff hazard areas and actions are suggested.

- All existing subsurface drains, sewers and any other water carrying pipelines be maintained regularly.

- Council display appropriate signs warning that during extreme storms there is the potential for significant overtopping of seawall promenades. Council take steps to exclude public access to seawall promenades during extreme events.

- Projected inundation levels at Ben Buckler be provided to property owners so that they may take advice in respect of the level of risk that may be presented to development on their Lots and any mitigation measures that may be warranted.

- Owners be advised of the requirement for Council approval for any works that may be undertaken to mitigate any adverse impacts of storm events on their Lots and that prior approval for any contemplated temporary works should be sought.

- Seawalls be monitored and fencing be used to exclude public access to any areas that become unsafe during a severe storm.

- Council continue to regrade sand as it builds up against the seawall to minimise sand drift landward of the seawall.

- Appropriate signage and fencing be erected at Bondi to mitigate the risk of falls where the beach has been lowered during storms.

- Studies be undertaken for Bronte and Tamarama seawalls including:
    - assessment of the fabric and stability (including test pits concrete cores and stability analysis)
    - wave modelling to assess nearshore wave conditions
    - photogrammetric analysis to determine historical beach levels

- The above data be used to refine estimates of storm demand and review longer term trends in beach levels.

- The following further geotechnical work be undertaken:
    - review of monitoring reports
    - re-assess the need for stabilisation measures in light of the above monitoring reports
    - geotechnical re-assessment on a five yearly basis

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1. INTRODUCTION

1.1 Background

Waverley Council (Council) Local Government Area (LGA) is located within the Eastern Suburbs of Sydney, bounded by the Central Business District in the west and the Tasman Sea to the east. It comprises the suburbs of Bondi, Bondi Junction, Bronte, Dover Heights, North Bondi, Rose Bay, Tamarama, Vaucluse and Waverley. The extent of coastline included in this study is shown (orange line) in the location plan (**Figure 1.1**)

Figure 1.1 Location plan, showing study extent (source: Google Earth Pro.)

In the past, development in the Waverley area (including public assets and infrastructure) and neighbouring municipalities (of Woollahra and Randwick) has been threatened or damaged by the action of extreme weather events. With the predicted intensification of these storm events and sea level rise in the coming decades, Council has identified the development of a Coastal Zone Management Plan (CZMP) as an outcome of a coastline hazard definition and climate change vulnerability study as vital to ensure Waverley is resilient to such impacts and challenges.

Currently, there is little history in regards to coastal management studies for the Waverley area. To date there is no CZMP for Waverley's coastal and aquatic environments.

This report provides a Coastal Risks and Hazards Vulnerability Study, as the first stage in preparing a Coastal Zone Management Plan.

1.2 Coastline Management Process and Brief

There is no fixed planning process that councils are required to follow in preparing a CZMP and councils are encouraged to adopt approaches that best suit their circumstances. Council has chosen to break down the preparation of the Plan into smaller stages. Specifically, the brief for this stage of the CZMP has defined the following:

- define the coastline hazards that impact upon each beach embayment and the study areas
- identify historical coastline/property protection works (including emergency works) and assess the likely performance and impact of these works during the design storm event (taken to be the 100 year Average Recurrence Interval (ARI) storm event
- determine the immediate hazard line i.e. the estimated landward extent of beach or cliff erosion from the design storm event plus any zone of reduced foundation capacity
- determine hazard lines for the 2050 and 2100 planning periods assuming both no sea level rise and incorporating sea level rise projections
- determine the oceanic and tidal inundation levels for all affected properties during the range of ARI events for the 2050 and 2100 planning periods
- assess the vulnerability of existing private/public assets and infrastructure within the study area to climate change induced sea level rise
- provide recommendations for the appropriate and achievable management of those risks and for the ongoing retention and/or improvement of public infrastructure, public beach access and beach amenity

At a meeting with Council's steering committee on 15 June and confirmed subsequently in writing by the Office of Environment and Heritage, Council resolved that mapping of properties/lots that have a potential to be affected by coastal hazards would be undertaken instead of mapping hazard lines.

1.3 Scope of this Report

This report summarises the current knowledge and understanding of the coastal processes that operate within the study area. The report examines the coastal hazards that impact Bondi, Bronte and Tamarama beaches and assesses these hazards to determine the immediate, 50 year and 100 year hazard risks. This report also assesses stability hazards along the cliff and bluff areas within Waverley LGA.

The hazards examined are those set out in the New South Wales Government's Guidelines for Preparing Coastal Zone Management Plans (2010), as listed below:

- Beach erosion: storm bite due to a beach erosion event with an average recurrence interval (ARI) of approximately 100 years plus an allowance for reduced building foundation capacity
- Shoreline recession: estimated recession due to sediment budget deficit and projected sea level rise*
- Coastal inundation: estimate of wave run-up level and overtopping of dunes resulting from an extreme ocean storm event*
- Coastal cliff or slope instability: slope stability assessment; see Australian Geomechanics Society (2007)*

* assessed under current conditions and projected 2050 and 2100 conditions.

Additional to these, the report addresses also the following issues:

- sand drift
- stormwater

Information included in each report section is listed below:

Section 2 outlines the site and historical setting

Section 3 examines the coastal processes operating in the study area

Section 4 discusses the coastline hazards affecting the study area

Section 5 discusses the geotechnical hazards affecting the study area

Section 6 provides a summary of the findings of the report and recommendations.

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

2. STUDY AREA

2.1 Site Description

The Waverly LGA is located within the Eastern Suburbs of Sydney and includes the iconic beaches of Bondi, Tamarama and Bronte as well as long stretches of rock cliffs. The LGA is bounded to the east by a cliffed coastline extending from Waverley Cemetery in the south to Clarkes Reserve in the north. A locality plan is shown in **Figure 1.1.**

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.

2.1.1 Cliff Areas

The study area comprises the cliff lines at the following locations:

- Clarke Reserve to North Bondi (including Ben Buckler Headland)
- Southern end of Bondi Beach to the Northern end of Tamarama Beach
- Southern end of Tamarama Beach to the Northern end of Bronte Beach
- The southern end of Bronte Beach to the southern end of Waverley Cemetery (Boundary Street).

A general summary of our observations is presented below and more specific details are provided in **Appendix A**.

The crest areas of the cliff lines within the study area included the rear yards of private properties, grass surfaced reserve areas, cliff top coastal walkways and lookout areas. The crest areas within public areas were often stepped with the steps formed by sandstone bedrock outcrop faces, steep vegetated slopes and the flat areas grass surfaced. The coastal walkways comprised timber framed structures or paved pathways. The crest areas within private properties comprised landscaped rear yards or driveways, with brick, concrete block or rendered retaining walls or fences. In some instances, rear yard areas extended along, or close to, the cliff edges.

The cliff face ranged in height between about 5 m and 35 m to 40 m and, typically, comprised sub-vertical sandstone bedrock faces with occasional ‘step’ features. The cliff faces and their outline (in plan) were controlled by orthogonal sub-vertical joint planes within the sandstone bedrock generally orientated (bearing) approximately north-south (bearings ranging between about 350° and 015°) and east-west (bearings ranging between about 095° and 120°). In some areas the cliff face “zig-zagged” and appeared to be controlled by the orthogonal jointing in the rock mass.

Stepped wave cut rock platforms were evident over the toe areas of the majority of the cliff lines. Often these platforms were covered with numerous sandstone blocks ranging in size from less than 0.5 m × 0.5 m × 0.5 m to in excess of 2 m × 2 m × 4 m. At the interfaces between the cliff lines and the beach areas the bases of the cliff lines were lined by concrete paved walkways (often comprising the roof slabs of stormwater box culverts) and foreshore pools.

2.1.2 Beaches

Each beach within the study area is briefly described below (extracts from Short, 2006). Beaches are listed from south to north (refer to **Figure 2.1**).

***Bronte** – lies 200m south of Tamarama and is 250 m long with large rips. The beach is set in a picturesque valley, occupied by 10 ha Bronte Park…… The beach is fronted by a surf zone that is usually occupied by 2-3 rips, one at either end against the headlands and, at times, a third one in the centre, with the rock pool on the southern head adjacent to the surf club. The southern headland rip is known as the 'Bronte Express' and provides a fast ride out to sea.*

***Tamarama** – is one of several deeply embayed beaches on Sydney's south side. It is only 80 m long, with the narrow sand filled valley behind the beach providing 3 ha of well maintained park and picnic areas. The small beach is wedged between two protruding sandstone headlands and the energetic wave climate ensures that at least one and often two rips are present on the beach with one usually flowing out past the northern rocks.*

***Bondi** – located 7 km south of South Head a with steep rocky coast in between. The northern Ben Buckler headland forms the eastern boundary of 800 m wide south-facing Bondi Bay, with McKenzie Point to the south. The wide 900 m long beach curves between the two headlands and faces southeast. It is backed by a continuous seawall, walkway, beachfront car park and large grassy foreshore reserve including two surf clubs and a bathing pavilion.*

***North Bondi** is protected partially by Ben Buckler headland and curves east of the rocks for 250 m. This headland reduces wave height about 1 m in the corner increasing to the south. The lower waves maintain in a continuous attached bar, cut by a small topographic rip against the northern rocks and rock pool, with rips increasing down the beach.*

The main central-southern section of Bondi Beach continues for another 650 m to the southern rocks and rock pool. It faces southeast into the prevailing waves, which average 1.6 m and maintain a continuous bar usually cut by 2-3 rips and at times separated from the shore by a longshore trough. A persistent large and often strong rip, called the Backpacker Express, runs out against the southern headland.

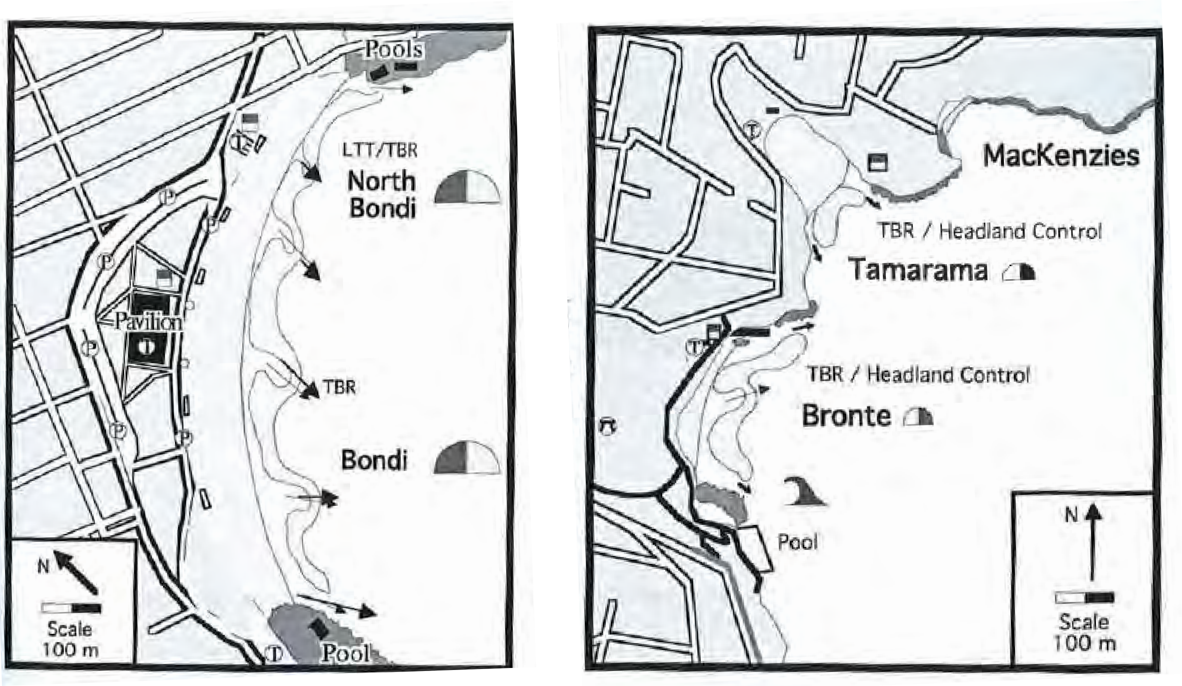

Figure 2.1 Predominant Rips (Source: Short, 2006)

2.1 Historical Setting

2.1.1 Bondi

Before European settlement, there is clear evidence of Indigenous occupation of Bondi. Early British arrivals identified Aboriginal pathways running from Port Jackson to the coast. A midden of shellfish debris and artefacts at the edge of the dunes has now disappeared under modern development, but rock carvings are still present in the area.

The Bondi topography originally consisted of undulating low hills with a large area of high sand dunes stretching from Bondi through to Rose Bay (refer **Figure 2.2**). The ridges were typically bare and comprised exposed sandstone outcrop, while numerous lagoons and streams existed behind the beach and in low lying areas (refer **Figure 2.3**). Some of the streams discharged across the beach to the ocean. The vegetation was predominated by tea tree scrub and native bushes.

Figure 2.2 Bondi 1875, showing vegetated dune field (source: Waverley Council)

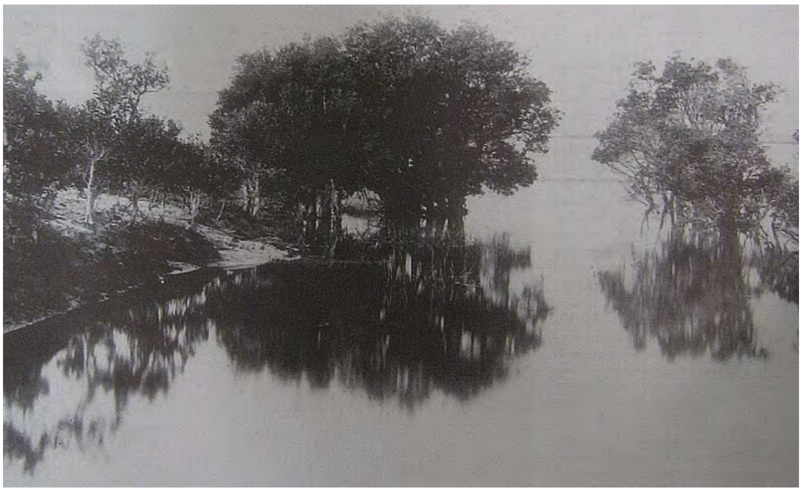

Figure 2.3 Bondi Lagoon 1890's (source: Waverley Council)

The first land grant was taken up by Roberts in 1811 and held by him and his family until 1851. The Hall/O'Brien families took possession of this grant in 1851 until 1882 when part of it was resumed for public use and the Council became the trustee. Other sections were sub-divided for housing. These grants and changes in tenure in the 19^{th} century initiated a new landscape behind the beach, but the beach itself remained in place. In the early 1880's O'Brien was forced to relinquish land under public pressure and the use of this part of the bay for public recreation took off after 1885 (refer **Figure 2.4**). Photos of bathing, picnics and later promenading show an increased use especially after trams came to the area and daylight surfing and swimming in the baths, built on the rock platform, were permitted. In about 1888 Bondi Baths and dressing sheds were constructed.

Figure 2.4 Bondi 1890's (source: Waverley Council)

Grazing of livestock helped to destabilise the vegetation in the 19th century and extensive sand drifts are recorded between the beach and Bellevue Hill some 2-3 km inland by 1920. Shacks, tracks and lagoons are noted as being overwhelmed by moving sands. This became the first area in NSW for experiments in sand stabilization using brush fences and introduced plants (refer **Figure 2.5**).

Figure 2.5 View north showing brush fences and tree planting 1900 (Source: Waverley Council)

At the turn of the century Bondi was a sparsely populated area covered mostly by sand hills and scrub. The beach sloped to large sand hills to the north with only a few buildings on Ben Buckler Headland. Numerous lagoons and streams could still be found in the backbeach area. In the following thirty years much development occurred at Bondi:

1905	First facilities were provided on the beach in the form of surf sheds, and soon after a kiosk was built on the beach
1907	The first pavilion was built on the site of the existing pavilion, and the first pine trees

	were planted in the southern part of Bondi Park
1909	Seawall constructed along the southern section of beach, to act as a barrier to wind driven sand blowing off the beach (refer **Figure 2.6**)
1923	Seawall was extended to the northern end of the beach and the promenade constructed
Mid 1920's	The dune field was mined, levelled and the lagoons drained, allowing land subdivision and residential development resulting in the fully developed urban landscape landward of Bondi Park

Figure 2.6 Bondi 1917 (Source: Waverley Council)

During the Depression, relief work was utilised to construct two piers near the centre of the beach, which extended from the promenade out onto the beach and were connected to the pavilion by tunnels (refer **Figure 2.7**). In 1942, the piers were demolished for coastal defence purposes during war, remnants of which still exist as small viewing platforms adjacent to seawall.

Figure 2.7 Aerial photograph showing piers, 1940 (source: Waverley Council)

One of the most significant storm events that affected the study area occurred in 1974. Further information on storms within the study area is given in **Section 3.1.3. Figure 2.8** shows the 1974 storm damage at the southern end of the beach. **Figure** 2.9 provides a summary of the damage that occurred during the 1974 storm throughout the study area.

Figure 2.8 Southern end of Bondi Beach following 1974 storms

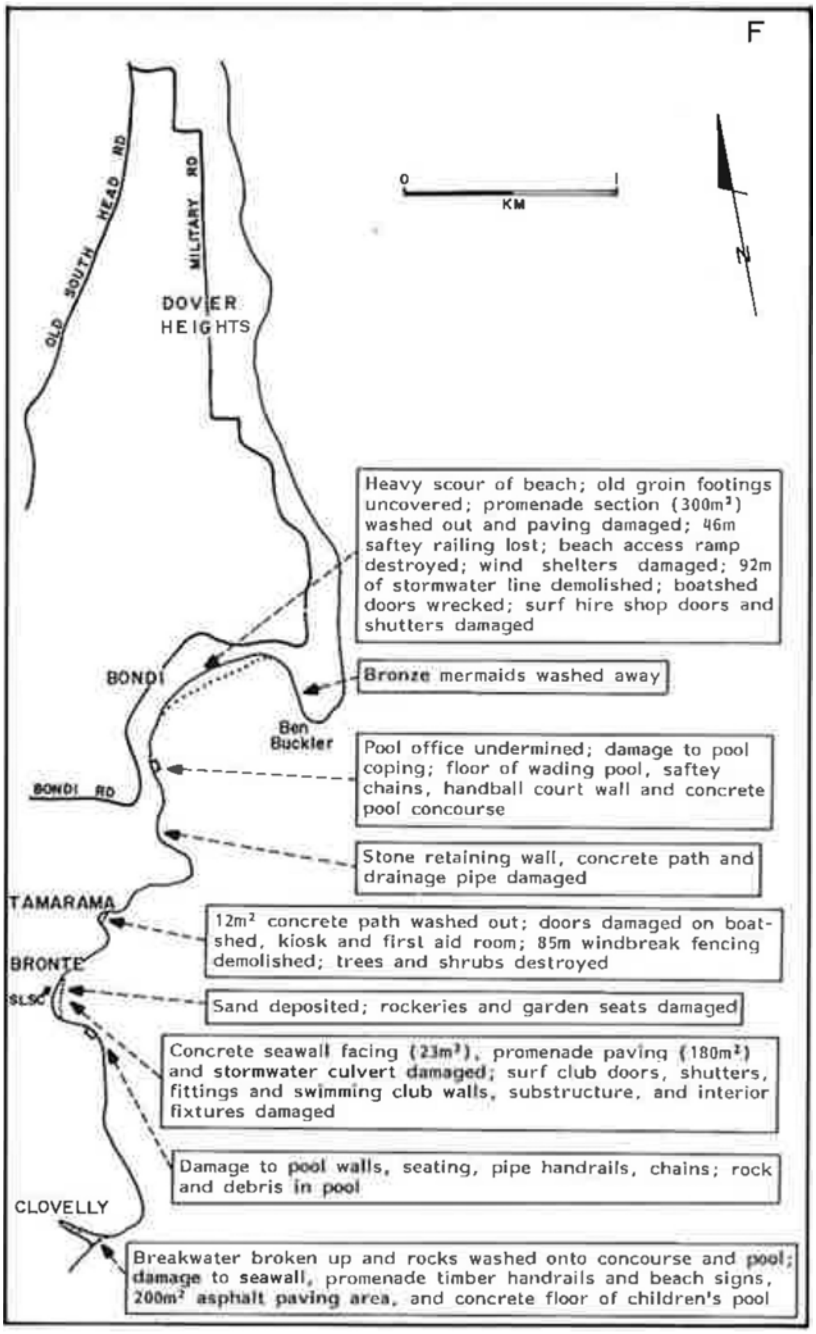

Figure 2.9 1974 Storm damage (Source: Chapman *et al.* 1982)

Following the 1974 storms, improvements were made to the seawall, with the construction of a Reno-mattress toe (refer **Figure 2.11** and **Figure 3.16)**. The work was undertaken in stages from 1987 to 1992, by which time the entire length of the seawall had been reinforced. In the late 1980's the box culvert stormwater outfall was constructed in the south of the beach (refer **Figure 3.6**). Prior to that two large stormwater pipes (1260mm and 1350mm diameter) discharged onto the beach at the southern end, which caused scour during high flows (refer **Figure 2.10**) .

Today Bondi Beach is a highly popular beach for sunbathers, swimmers and surfers. The beach receives many visitors each year and is widely known around the world as an iconic Australian beach.

Figure 2.10 Old stormwater outfall, showing beach scour following storm event August 1986 (source PWD 1988)

Figure 2.11 Construction of Bondi Seawall, Reno-mattress toe protection works 1987 (Source: Lex Nielsen)

Figure 2.12 South Bondi, 2011 (view from the water)

2.1.2 Bronte

The historical data from this section is taken from Council Fact Sheets and www.dictionaryofsydney.org/entry/bronte based on Stan Vesper*, Bronte: The Birthplace of Surf Lifesaving* (Playright Publishing, Sydney, 2006).

No aboriginal middens or carvings have yet been found within the Bronte Park area. However, indigenous people were well established throughout Waverley before European arrival and would have used the area.

In 1836 European arrivals started claiming land in the area. Mortimer Lewis, New South Wales Colonial Architect, became the first to purchase land at Nelson Bay, later known as Bronte. His land included the whole of Bronte Park and the Gully, the shopping strip opposite the park and the area on which Bronte House stands. The property was known as the Bronte Estate.

Council had been petitioning the NSW government since 1863 to resume 14 acres on the beachfront at Bronte for use as a public park. In 1886 the land was purchased and Council was appointed Trustees of Bronte Park. The park was proclaimed in 1887 and two further resumptions of land increased its size. At that time the waterfall in Bronte Gully fed a creek which ran across Bronte Park forming a series of pools. A bridge crossed the creek allowing access to the beach from the park. This creek ran parallel to the beach, then turned and ran across the beach, flowing out to sea at the southern end of the beach near the bogey hole. Early sketches and photos (refer **Figure 2.13** and **Figure 2.14**) show that at this time Bronte Beach was far more extensive that it is today, extending as far back as the mouth of Bronte Gully, with a deeper sand beach covering much of the land occupied today by Bronte Park.

There are two bogey holes (pools made of rings of rocks, with ‘bogey’ believed to be derived from an Indigenous word) at Bronte. The bogey hole adjacent to the Bronte Baths was a popular bathing place before the Bronte Baths were built and, in 1887, it was enlarged and a sea wall built to form the Bronte Baths. Between 1904 and 1908 rocks were cleared to extend the 'Men's Bogey' and linked to the baths by a set of steps. The second bogey hole, on the beach itself, is more well-known and was created originally for women and children. In April 1916 the Bronte Progress Association wrote to Council requesting the removal of rocks from a natural rock shelf at the southern end of the beach to provide a safe bathing place for women and children. The bogey holes remain as some of the beach's attractions.

Figure 2.13 Earliest image of Bronte Park, sketch by Georgiana Lowe, 1845-1849 (Source: Waverley Library Fact Sheets)

Figure 2.14 Bronte Beach 1900-1910 (Source: State Library of NSW)

Between 1914 and 1917 the seawall and promenade were constructed as part of a Bronte Beach improvement scheme. Effectively this cut the beach in half, with the area now behind the promenade being drained, filled in and grassed and becoming part of Bronte Park (refer to **Figure 2.15**).

The creek that runs through Bronte Gully (which used to discharge at the Bogey Hole) is now diverted into a storm water drain coming out at the northern end of the beach.

Today Bronte Beach attracts many sunbathers, swimmers and surfers with the large park, picnic area and good access and parking adding to its appeal. Rips occur along the beach. Similar to Bondi Beach, Bronte Beach also have some issues at times with sand blowing onto the promenade (refer **Figure 2.17**).

Figure 2.15 Bronte Beach June 1935 (Source: State Library of NSW)

Figure 2.16 Bronte Beach 1959 (Source: Waverley Library Fact Sheets)

Figure 2.17 Bronte Beach, June 2011

2.1.3 Tamarama

The historical data from this section is taken from Council Fact Sheets, the Tamarama Plan of Management and www.dictionaryofsydney.org/entry/tamarama.

There is clear evidence of Aboriginal occupation of Tamarama prior to European settlement. There are local midden deposits with an accumulation of shell food refuse and evidence of fireplaces. One particular midden, on a sandstone rock ledge at Tamarama, has been classified by the National Parks and Wildlife Service as a site of archaeological significance because 'beachside shelters with midden deposits are rare on the Tasman Sea coast'. To protect it the exact location of this midden is restricted. There are also rock carvings on the coastal walk at Mackenzies Bay.

Originally, Tamarama Gully was a glen with cascading waterfalls, lush vegetation and a winding creek spilling out through a lagoon at the beach to the sea (refer **Figure 2.18**).

The land on which Tamarama Park is now situated was granted to a J.R. Hatfield in 1839. By the late 1880s, the land surrounding the Glen was subdivided for residential use. In 1888 Council asked the government of the time to dedicate a 100-foot wide reservation along the frontage of Hatfield's original grant as a public recreation reserve. The government refused. Meanwhile, a group of local businessmen had purchased the land and in 1887 opened The Royal Aquarium and Pleasure Grounds. This was an open-air amusement park located behind Tamarama Beach and on the northern headland.

Ownership and management changed several times throughout its existence. In 1906 William Anderson, a theatrical entrepreneur, leased the land, minus a 12-foot strip of coastline, to allow the public access to the beach. He leased further land also in Tamarama Gully and constructed his 20 acre outdoor entertainment area, Wonderland City (Refer **Figure 2.19**). Wonderland City was closed in 1911. Although little visible evidence survives today (with the possible exception of the two paths on the north boundary of the Gully), the NSW Heritage Office still considers the site to be of archaeological significance.

Figure 2.18 Postcard of Tamarama, 1912, showing where the creek lagoon flowed into the ocean (Source: Waverley Library)

Figure 2.19 Wonderland City, Tamarama,1907. The switch back railway can be seen at the back of the bay, and the wire fence at the beach where there is a dense concentration of people (Source: Waverley Library Fact Sheet)

A public park known by the name of 'Tamarama Beach' was proclaimed for Public Recreation on 17 April 1907. Also on that day Council was appointed trustee of Tamarama Beach by notification in the Government Gazette. In 1916, Council tried again to claim an area of Tamarama for public park but was again refused. Finally, on 24 September 1920, Council was able to purchase 7 acres of Tamarama for beach access and parkland. Thus began over 80 years of Council improvements, with the first initiative being a formal landscape layout under a public employment program after World War 1.

A seawall and promenade were constructed in the 1920s and 1930s, and the valley floor was drained and regraded. Tamarama Marine Drive was also built, along with the sandstone wall that supports it (refer **Figure 2.20** and **Figure 2.21**). In 1935 a pedestrian underpass connected the gully and park under Tamarama Marine Drive (refer **Figure 2.21**). Some local residents report that this was removed by public demand.

In the 1950s, the park was separated from the road by timber fencing. Throughout the 1960s and 1970s, more park furniture and facilities were provided. These included the picnic shelters, amenities building, outdoor tables, and revegetation of native species such as Coastal Banksia. The public amenities building was opened in 1984. The lifeguard tower on the beach was completed in November 2000.

Today Tamarama beach attracts sunbathers, swimmers and surfers. The beach and reserve is popular for recreational activities such as picnics and beach volleyball. The beach is known as one of the most dangerous beaches on the east coast due to strong rip currents. A recent photograph of the beach is shown in **Figure 2.22**.

Figure 2.20 Tamarama, 1931, construction of Marine Drive. The seawall and picnic huts can be seen (Source: Waverley library)

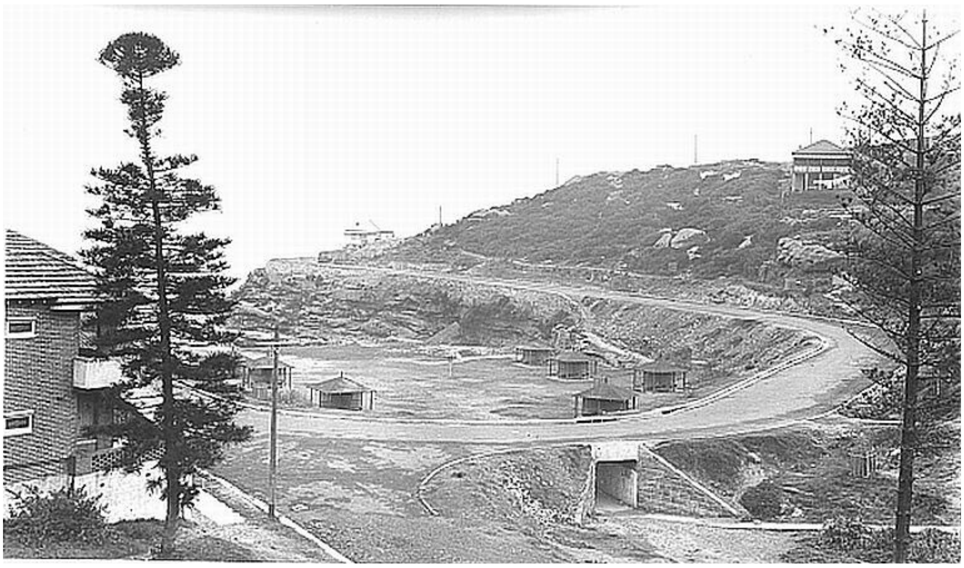

Figure 2.21 ***Tamarama, 1935.*** During the Great Depression road workers were employed under a government publicwork scheme to construct Marine Drive. The tunnel which once linked Tamarama Park with Tamarama Gully can be seen. (Source: Waverley Library Fact Sheet)

Figure 2.22 Tamarama Beach, photo taken on 15 June 2011.

2.2 Cliff/Bluff Areas

The foreshore cliff faces comprise Hawkesbury Sandstone bedrock of Triassic age (around 245 to 210 million years ago). The sandstone represents vast quantities of sediments transported into the Sydney Basin by rivers flowing from the south-west and west. Uplift and deformation of the Sydney basin area probably occurred over several phases and was associated with the opening of the Tasman Sea approximately 60 to 80 million years ago. The present elevation of the Sydney Basin region was achieved by about the mid Tertiary (about 40 to 50 million years ago). This uplift and deformation has led to the observed pattern of jointing and faulting in the rock mass and the intrusion of igneous dykes generally along the dominant joint planes; typically in an approximately east-west direction but with some trending approximately north-south. Weathering and erosion of the sandstone continued with sea level fluctuations from the early Quaternary onwards (commencing around 1.8 million years ago), associated with glacial and inter-glacial periods (sea level low high periods, respectively), having a significant effect on the formation of the present day coastline.

Current sea levels are believed to have been reached around 6,400 years before present (ybp). A glacial period between about 17,000 and 25,000 ybp is believed to have caused a sea level fall of around 130m below present day levels. At the end of this glacial period ice melted and sea levels rose to their current levels in-filling the valleys that now form the Sydney Harbour foreshore we see today. This cycle of varying sea levels is believed to have occurred several times over during the Quaternary (about 1.8 million years ago to present day). The wave cut platforms observed along the bases of many of the cliff faces are likely to have developed during inter-glacial sea level highs. It is believed that the current cliff faces were located some 90kms to the east and the erosion over the last 70 million years has resulted in the recession of the cliff faces to the present coast line.

3. COASTAL PROCESSES

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

- wave climate
- coastal storms
- elevated water levels
- wave induced currents
- sediment transport
- wave runup and overtopping
- climate change

3.1 Wave Climate

3.1.1 Offshore Wave Climate

The study site is located in the south-west Pacific at around 33.5°S and receives waves generated in the Tasman Sea and the Southern Ocean. The annual wave climate is both energetic and highly variable with a distinct seasonality present. Based on a recent analysis of long-term records the months of March and June-July experience the largest average monthly wave heights (Harley et al., 2009). Although moderate waves dominate the climate, large waves (H_s>4 m) and/or low swell may occur in any month (Short and Trenaman, 1992). Extreme events (H_s>6m) occur predominately in autumn and winter. Waves in the region are generated by five typical meteorological systems: east-coast lows, tropical cyclones, mid-latitude cyclones, zonal anticyclonic highs and local summer sea breezes (Short and Trenaman, 1992).

Manly Hydraulics Laboratory (MHL), part of the NSW Department of Service Technology and Administration, operates a network of Waverider buoys in deep water along the NSW coast. Analysis of the collected data allows (among other things) the *significant* wave height (H_s – the average height of the highest 33% of the waves, which often corresponds to the observed wave height) and peak spectral wave period (T_p – the wave period coinciding with the highest energy in a wave record) to be determined.

The nearest Waverider buoy to the study area is located approximately 11 km ESE of Long Reef in Sydney's northern beaches in a water depth of about 85m (MHL, 2005). This buoy is denoted by MHL as the Sydney Waverider Buoy. Wave data collected at this location is considered to be representative of offshore wave conditions that will influence coastal processes in the Waverley LGA.

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

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

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

The occurrences of waves by direction are listed in **Table 3.1**. It is evident that the majority (approximately 65%) of offshore waves propagate from the S-SE sector (i.e. S, SSE and SE cardinal directions). S-SE waves originate from storms and swells originating in the Tasman Sea and Southern Ocean and can occur during any season. Easterly waves (i.e. ESE, E and ENE cardinal directions) make up approximately 30% of the total offshore wave energy. N-NE waves make up approximately 3% of the offshore wave energy and are generated by summer sea breeze systems and tropical cyclones in the Coral Sea. The largest period waves typically occur from the S-SE sector in the winter months.

The directional occurrence of storm waves (H_s exceeding 3m) is also listed in **Table 3-1**. It is evident that the dominant storm wave direction was from the S (about 38% of storm waves), with about 31% from the SSE and 13% from the SE. It can also be noted that waves from E through NE to N only accounted for about 9% of the storm waves.

Table 3-1: Occurrence of waves from each offshore wave direction for the Sydney Waverider Buoy from 1992 to 2010

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

Directional extreme waves for the 1, 50 and 100 year return periods have been estimated for the Sydney region based primarily on the analysis of the directional Sydney Waverider Buoy data. The wave height likely to occur or be exceeded, on average, every 100 years was estimated to be 9.3m. This value compares well to previously reported values for the 100-year return significant wave height for the Sydney region (You, 2007).

Table 3-2 summarises the directional extreme waves as calculated for the region offshore of the study area, based on directional data from the Sydney Waverider Buoy.

Table 3-2: Offshore directional wave extremes for the study region

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

Notes: Location: 33° 46' 54"S 151° 25' 29"E
Water Depth: 85m
The above are the extremes likely to be reached, or exceeded, once on average every 1-year, every 50-years and every 100-years, respectively for the directional sector indicated at the above location.

Beach erosion is strongly linked to the occurrence of high wave conditions with elevated ocean water levels (the latter are discussed in **Section 3.4**). Therefore, inclusion of duration is likely to describe more accurately the severity of a storm in terms of beach erosion, rather than using ARI alone (Lawson and Youll, 1977). Erosion is more likely to be significant when the large waves coincide with a high tide. In general, storms with a duration in excess of 6 hours are likely to coincide with high tide on the NSW coast (Lord and Kulmar, 2001). Therefore, it is 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. Assessing the relationship between H_s, duration and ARI, at the Sydney Waverider buoy it was found that the Sydney offshore 100 year ARI significant wave height exceeded for a duration of 6 hours was about 7.8m. These data have been used with the nearshore wave transformation coefficients to determine nearshore wave runup levels on the foreshores.

The influence of a range of climate oscillations, such as the El Niño/ Southern Oscillation, may help to explain the high variability observed in the offshore wave climate in the Sydney region (Harley et. al., 2009). Climate change may influence future trends in the offshore wave climate (McInnes et. al., 2007). The potential impacts of climate change and the relative time frames are discussed further in **Section 3.6**.

3.1.2 Nearshore Wave Climate

As waves approach the shore, they may be transformed by the processes of refraction, shoaling, diffraction, attenuation, reflection and breaking. Therefore, the nearshore wave climate in the study area has a different wave height and wave direction compared with that offshore.

A previous assessment of nearshore wave conditions in the study region comprised a wave refraction/diffraction analysis of Bondi Beach, presented in PWD (1988). That study showed that nearshore wave coefficients (ratios of nearshore to offshore wave heights), in a nearshore water depth of around 5 m, decreased moving north from around 1.0 at the southern end of the beach to around 0.6 at the northern end (**Figure 3.1**). It can be expected that beach erosion volumes and wave runup levels may vary considerably with such a variation in nearshore wave conditions along the beach. No nearshore wave modelling has been undertaken for Bronte Beach or Tamarama Beach.

The application of nearshore wave heights, for hazard calculations such as overtopping, is discussed further within the relevant sections of **Section 4**.

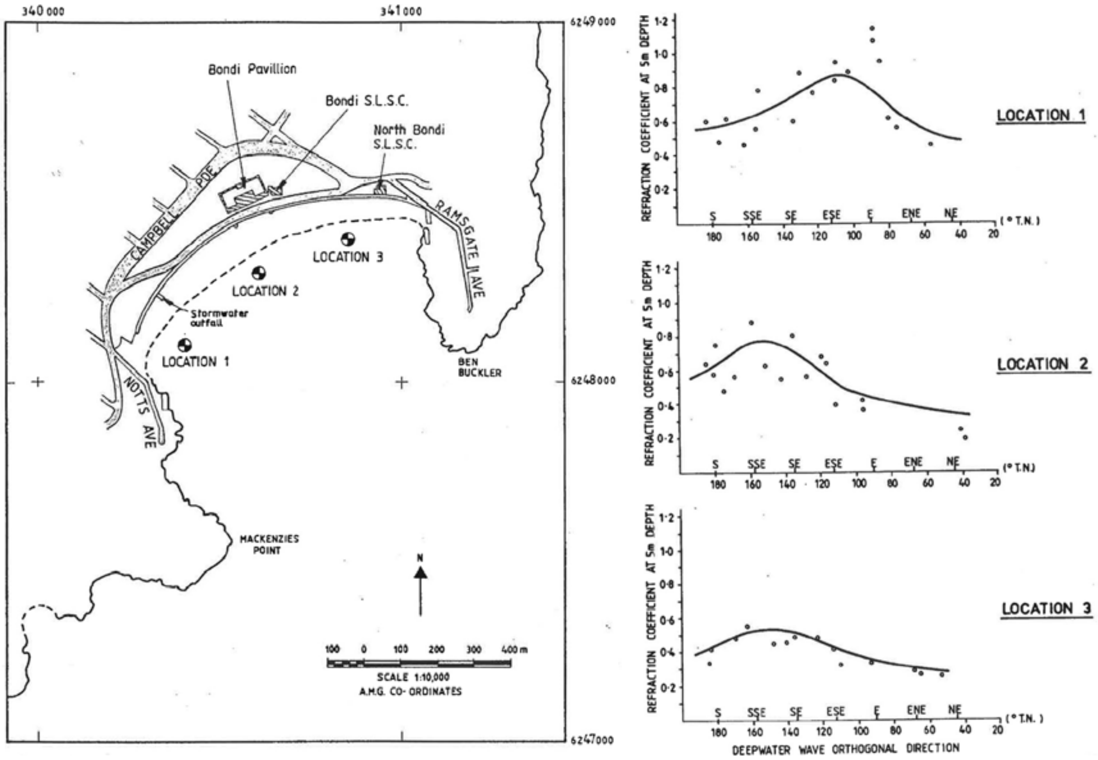

Figure 3.1 Nearshore wave refraction coefficients for 12 s wave period (source PWD 1988)

3.1.3 Coastal Storm Events

The NSW coastline is subject to intense 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 3.1.2**). 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, 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 3-3**.

Table 3-3: 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 assessed 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 Waverley LGA is located in the Central sector.

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

Storm History

PWD (1985b, 1986) listed all Category X, A, B and C storms that were assessed to have occurred between 1880 and 1985. A listing of Category X storms from these references (from the Sydney region) is provided in **Table 3-4**. This includes the estimated significant wave height (H_s) and significant wave period (T_s) calculated by hindcasting for some storms.

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

Table 3-4: Occurrences of Category X storms on the Sydney Coast from January 1880 to December 1985 (based on PWD, 1985b, 1986)

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

Date	Storm Type	H_s (m)	T_s (s)
9-11 March 1958	Easterly trough low		
29 June - 1 July 1958	Continental low		
20-21 July 1959	Continental low		
4-5 October 1959	Continental low		
20-21 May 1966	Southern secondary low		
5-6 September 1967	Continental low	7.7	11.1
13-15 May 1968	Southern secondary low	7.9	11.2
25-26 May 1974	Southern secondary low	8.8	11.8
18-20 March 1978	Easterly trough low	7.7	11.1
31 May – 2 June 1978	Inland trough low	6.9	10.5
7-9 July 1983	Continental low	6.9	14.5
5-8 November 1984	Inland trough low	6.0	12.5

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

Category X storms that have been measured at the Sydney non-directional Waverider buoy (generally from July 1987 to March 1992, but which continued recording until October 2000) and Sydney directional Waverider buoy (from March 1992 to December 2007) are listed in **Table 3-5**.

Table 3-5: Category X storms measured at the Sydney Waverider buoys, 1987-2007

Date	Peak H_s (m)	Mean T_s (s)	Direction
11-13 November 1987	6.8	9.0	SSE
1-3 August 1990	7.2	9.3	SE
24-27 August 1990	6.3	10.3	SSE
25-27 September 1995	6.3	9.9	SE
30 August – 1 September 1996	6.1	10.0	SE
9-12 May 1997	8.4	10.3	SSE
7-10 March 1998	6.0	10.0	SSE
21-25 April 1999	6.2	9.8	E
14-17 July 1999	6.0	10.3	ESE

Date	Peak H_s (m)	Mean T_s (s)	Direction
30 June – 2 July 2000	6.1	9.9	S
27-29 July 2001	7.0	11.0	S
18-22 November 2001	6.2	9.7	SE
29 June – 1 July 2002	6.2	11.1	SSE
18-20 July 2004	6.7	9.8	SSE
22-24 March 2005	6.6	9.2	SE
10-12 July 2005	6.2	10.0	SSE
2-4 June 2006	6.5	10.1	S
11-12 June 2006	6.2	10.6	S
7-10 June 2007	6.9	9.8	SE
16-20 June 2007	6.0	9.2	SE
18-21 July 2007	6.5	10.8	SSE

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

Newspaper Storm Reports

Historical newspaper reports can also provide an indication of storm events and impacts. PWD 1988 included a record of historical newspaper reports of coastal storms and damage in the Sydney area from 1889 to 1988. Below is an extract of this record covering all storms where Bondi, Bronte or Tamarama are mentioned. Key points to note are that overtopping to Bronte Park is mentioned on several occasions, as is damage to coastline structures at all the beaches.

12 to 15/10/1942 *Heavy seas battered Sydney's coast. Waves wash over Bronte Park.*

15 to 20/6/1948 *Huge seas battered coastline. Bondi was swept by mountainous waves over one mile front. Huge seas swept over promenade at Bronte flooding Bronte Park to within a few feet of the roadway.*

23/7/1950 *50 yards of concrete seawall collapsed at Bondi resulting in loss of the promenade due to high tides and intense rainfall. Stormwater runoff scours 20 foot wide gap in main footpath.*

20 to 23/5/1955 *Giant seas in Sydney cause tremendous damage to beaches. Bronte Baths severely damaged by waves.*

14 to 19/3/1959	*Heavy rain drenched Sydney. 30 foot wave overtopped promenade at Bronte and swept 20 yards through the surf club.*
8 to 14/6/1974	*The second storm in four weeks caused enormous damage to Sydney beaches. Severe erosion of Sydney beaches. Storm caused tide to rise 1.5 feet. Seawalls at Bondi and Tamarama suffered $100,000's worth of damage. Ramp at south Bondi destroyed, while in this vicinity a large section of promenade collapsed. Old pier foundations never seen before exposed on Bondi Beach. Bondi baths damaged, seawall of main pool cracked and ladies dressing shed destroyed.* (refer **Appendix E**)
10/3/1975	*Seawall at Bondi undermined in vicinity of stormwater and at North Bondi. The promenade collapsed in the two locations where the wall was undermined.*
15 to 16/6/1978	*100 mph wind gusts caused widespread damage in Sydney. Bondi and Maroubra hardest hit. Seawall at Bondi damaged.*

Analysis of Key Storms Affecting Study Area

The most significant coastal storm that has been recorded to have impacted on the Central Coast of NSW (which includes the Sydney area) is the Category X May 1974 storm (see **Table 3-4**), which was followed by two smaller storms in June 1974 (Category C and Category A). A set of photographs taken during the 1974 storms are included as **Appendix E**. The May 1974 storm was particularly severe as it was accompanied by the highest ever recorded water level along the NSW coast[2]. Coastal erosion depends on far more than just wave height, with factors such as storm duration, water level, wave direction and storm history being important[3].

Most recently the June-July 2007 event comprised 3 Category X storms (see **Table 3-4**), the last of which was preceded by 5 Category B storms. Watson et al (2007) noted that these storms led to the second highest insurance payout and second largest emergency response operation in Australia's history, behind only the 1999 Sydney hailstorm. These storms were described as East Coast Low weather systems.

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

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

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

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

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

3.2 Elevated Water Levels

The factors that 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. Note that sea level is also projected to rise due to climate change.

3.2.1 Tides

The tidal regime of NSW is microtidal semidiurnal with a diurnal inequality. This means that the tidal range is less than 2 m, there are two high tides and two low tides each day and there is a once-daily inequality in the tidal range. The mean tidal range at Sydney is around one meter and the tidal period is around 12.5 hours. The higher spring tides occur near and around the time of new or full moon and rise highest and fall lowest from the mean sea level. The average spring tidal range is 1.3 m and the maximum range reaches 2 m. Neap tides occur near the time of the first and third quarters of the moon and have an average range of around 0.8 m.

3.2.2 Storm Surge and Wave Setup

In NSW, storm surge and wave setup can increase open coast still water levels (within the wave breaking zone) by several metres during storms, with components approximately as large as follows:

- storm surge of 0.6 m (barometric setup of up to 0.3 m to 0.4 m and wind setup of up to 0.2 m to 0.3 m)

- wave setup of up to 1.5 m (typically about 10-15% of the deepwater significant wave height).

This increase in water level is superimposed on the astronomical tide. 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 1.49 m, 1.46 m and 1.41 m AHD respectively. The highest recorded water level at Fort Denison was 1.48 m AHD in May 1974. These levels are representative of astronomical tide and storm surge, but exclude wave setup.

Wave setup, typically, is some15% of the unrefracted deepwater significant wave height. It is manifested as a decrease in water level prior to breaking (with a maximum set down at the break point), while from the break point the mean water surface slopes upward to the point of intersection with the shore where the maximum wave setup occurs.

As extreme water levels at Sydney are representative of conditions in the Waverley LGA, the 100 year ARI water level (including astronomical tide and storm surge) has been adopted as 1.5 m AHD. With a 100 year ARI 6 hour duration offshore significant wave height of 7.8m (refer **Section 3.1**), and assuming wave setup as 15% of this wave height, the 100 year ARI wave setup was determined as 1.2 m. Therefore, a 100 year ARI total design still water level (astronomical tide plus storm surge and wave setup) of 2.7 m AHD has been adopted for this study. This design level does not include climate change considerations which are further discussed in **Section 3.6**.

3.2.3 Sea Level Rise

Sea level rise is a gradual process and will have medium- to long-term impacts. The best national and international projections of sea level rise along the NSW coast are for a rise relative to 1990 mean sea levels of up to 40 cm by 2050 and 90 cm by 2100. There is no scientific evidence to suggest that sea levels will stop rising beyond 2100 or that the current trends will be reversed.

Increasing sea levels have the potential to increase coastal hazards (particularly beach erosion) and flooding risks during major storms. This may affect coastal properties, buildings and infrastructure, recreational facilities, social amenity and coastal access.

The NSW Government has issued NSW sea level rise planning benchmarks, being an increase above 1990 mean sea levels of 40 cm by 2050 and 90 cm by 2100, which has been adopted for this study.

3.3 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 are directed parallel to the shoreline. They are generated by waves breaking at an angle to the shoreline or by wave setup. These currents cause movement of sediment along the shoreline, commonly referred to as littoral drift transport. Due to the variability in wave approach direction at beaches, there may be times when the littoral drift is transported in one direction and at other times when it is transported in the opposite direction.

Rip currents are strong currents that flow seaward from the shore. They comprise the return movement of water which has "piled up" on the shore by incoming waves (wave setup) 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 at the location of rips; that is, opposite rip heads. Accordingly, it is common to distinguish the higher storm erosion demand that can occur at rip heads and the lower storm erosion demand that prevails away from the location of rips.

The existence of longshore currents and rip currents are evident within the study beaches from site observations and from the available historical aerial photography. Rip currents are also well documented and have been given local names such as "back packers express" for the rip in the south of Bondi. Short, 2006 states the following about rips at each of the beaches (see **Figure 2.1**)**:**

Bronte *– usually occupied by 2-3 rips, one at either end against the headlands, and at time a third one in the centre, with the rock pool on the southern head adjacent to the surf club. The southern headland rip is known as the 'Bronte Express' and provides a fast ride out to sea.*

Tamarama *– at least one and often two rips are present on the beach with one usually flowing out past the northern rocks.*

Bondi *– maintains a continuous bar usually cut by 2-3 rips and at times separated from the shore by a longshore trough. A persistent large and often strong rip, called the 'Backpacker Express', runs out against the southern headland.*

While it is apparent that rips form typically at the headlands of the beaches, there is no evidence that the locations of rips 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.

3.4 Sediment Transport

3.4.1 Introduction

Sediments are transported in the littoral zone and Inner Continental Shelf under prevailing ocean currents and waves. In the littoral zone, wave breaking can result in a vigorous stirring and suspension of beach sediments as well as driving strong currents both alongshore and cross shore (rip currents). The combination of these processes can result in significant sediment transport, particularly during storms. Beyond the zone of wave breaking, sediments can be transported under the prevailing ocean currents abetted by wave stirring on the seabed. Wind action is another process that can transport sand away from the beach (aeolian sand transport).

The dynamic nature of beaches is witnessed often during storms when waves remove the sand from the beach face and the beach berm and transport it, by a combination of longshore and rip currents, beyond the breaker zone where it is deposited in the deeper waters as sand bars. During severe storms, comprising long durations of severe wave conditions, the erosion continues into the frontal dune, which is attacked, and a steep erosion escarpment is formed. This erosion process usually takes place over several days to a few weeks.

The amount of sand eroded from the beach during a severe storm will depend on many factors including the state of the beach when the storm begins, the storm intensity (wave height, period and duration), direction of wave approach, the tide levels during the storm and the occurrence of rips. Storm cut is the volume of beach sand that can be eroded from the subaerial (visible) part of the beach and dunes during a *design* storm. Usually, it has been defined as the volume of eroded sand as measured above mean sea level (~ 0 m AHD datum). For a particular beach, the storm cut (or storm erosion demand) may be quantified empirically with data obtained from photogrammetric surveys.

Following storms, ocean swell replaces the sand from the offshore bars onto the beach face where onshore winds move it back onto the frontal dune. This beach building phase, typically, may span many months to several years. Following the build-up of the beach berm and the incipient foredunes, and the re-growth of the sand trapping grasses, it can appear that the beach has fully recovered and beach erosion has been offset by beach building. However, in some instances, not all of the sand removed from the berm and dunes is replaced during the beach building phase. Once the sand has been transported offshore into the surf zone, it may be moved alongshore under the action of the waves and currents and out of the beach compartment. Some of the sand that is transported directly offshore during storms may become trapped in offshore reefs, thereby preventing its return to the beach.

Over the longer term, should the amount of sand taken out of the compartment by alongshore processes exceed that moved into the compartment from adjacent beaches or other sources, then there will be a direct and permanent loss of material from the beach and a deficit in the sediment budget for the beach. This will result in an increasing potential for dune erosion during storms and long term beach recession.

Another process that may lead to a deficit in the sediment budget of a beach includes wind blown sand off the beach (aeolian sand transport causing transgressive dune migration), mining the beach for heavy minerals and beach sand extraction operations. Other processes, which are not so obvious because they occur underwater, include the deposition of littoral drift into estuaries and the transport of quantities of littoral drift alongshore and out of a beach compartment, which may be larger than any inputs.

The quantification of sediment budgets for coastal compartments is exceedingly difficult. The usual practice is to identify the processes and to quantify the resulting beach recession using photogrammetric techniques.

3.4.2 Littoral Drift Transport

Cross-shore Transport

The largest rates of littoral drift transport occur during storms when beach and dune sands are eroded and transported offshore. Storm demand is the volume of sand on the subaerial beach that is eroded during a storm and deposited by cross-shore sand transport processes into deeper water. This volume of sand is measured usually above mean sea level (MSL), which approximates Australian Height Datum (AHD). Knowledge of the storm demand for a beach allows estimation of the amount of material that is required to be held in reserve for a storm 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.

As discussed in Chapman *et al.* (1982) and DECCW (2010), storm demand at any location, at any point in time, is dependent on a number of variables, including the:

- wave height and period as well as the duration of the storm
- state of the beach before the storm
- direction of the storm relative to the orientation of the beach[4]
- magnitude of the storm surge accompanying the event
- amount of wave setup and runup on the beach during and immediately following the storm
- tidal range at the time of the storm
- state of the tide at the peak of the storm

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

- presence of rip cells
- presence and influence of local topography including adjacent headlands or coastal structures, or both, which can modify local wave and current conditions and the supply of sediment
- existence and strength of longshore currents
- sediment grain size of the beach and surf zone
- for embayed beaches, the prevalent stage of the beach rotational cycle due to climatic variability (i.e. Southern Oscillation Index) impacts (Chapman *et al*, 1982 and DECCW, 2010).

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

There are several methods to estimate storm demand in the study area, including:

- analysing measurements of beach erosion that have been collected for Bondi Beach
- comparing measurements of beach erosion that have been collected at other similar beaches
- storm cut numerical modelling
- recently developed statistical joint probability type distribution approaches
- correlating storm demand to relative wave energy along the beaches in the study area.

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

The *Guidelines for preparing Coastal Zone Management Plans* (DECCW, 2010) recognise that the extent of beach erosion depends on a number of factors (refer **Section 5.6.2**). As such, it may be difficult to quantify the potential upper limit of storm demand that may occur in a specific area, and so a more empirical approach is recommended for estimating storm demand. In the absence of site-specific data, for the NSW open coast Nielsen *et al.* (1992) recommended the following maximum design storm erosion demands (above MSL, i.e. above 0 m AHD) for planning purposes:

- 250 m^3/m in fully exposed locations;
- 125 m^3/m for protected embayments.

Photogrammetric data are available for Bondi Beach and the storm demand has been derived empirically from a comprehensive analysis of photogrammetric survey data as documented in PWD

(1988). PWD (1988) defined the storm erosion demand as the difference between the maximum and minimum sand reserves as measured, noting that surveys were available for the immediate post-storm condition following the very severe storms of 1974. The data showed that the storm erosion demand varied along the beach from a maximum value of 194m^3/m at the southern end of the beach, to 108 m^3/m at the central portion and to 41 m^3/m at the northern end. This variation was reflected also in the wave refraction analysis undertaken where maximum wave height coefficients, which define the ratios of the nearshore wave heights to those offshore from which they were generated, varied from around 1.2 at the southern end to 0.55 towards the northern end.

There are no site specific data that allow for an empirical assessment of the seaward extent of cross-shore sediment transport at Bondi Beach, Tamarama Beach or Bronte Beach. However, Nielsen (1992) compiled and documented results from regional field, laboratory and analytical studies in NSW and defined the following limits for cross-shore sediment transport:

- 12 m (±4 m) – the offshore limit of significant beach fluctuations and alongshore transport of littoral drift;
- 22 m (±4 m) – the absolute limit of offshore sand transport under extreme storm events;
- 30 m (±5 m) – the limit of re-working and onshore sediment transport of beach sand under wave action.

ALONGSHORE TRANSPORT

An understanding of the alongshore transport of littoral drift from these beaches can be gleaned from the distribution of sediments as determined from detailed offshore surveys as carried out by the NSW Government Public Works Department (NSW Govt 1989). A detailed map of the regional sediment/rock distribution of the Inner Continental Shelf to a water depth of around 70 m is presented in **Figure 3.2.**

An inspection of **Figure 3.2** shows that the absolute limit of cross-shore sand transport under extreme storms lies within subaequeous rock headlands extending from Shark Point south of Bronte Beach to north of Bondi Beach. Further, however, the limits of alongshore transport of littoral drift indicate that Bondi Beach does not exchange sand with Tamarama Beach and Bronte Beach and, further, these beaches do not lie on a littoral drift coastline.

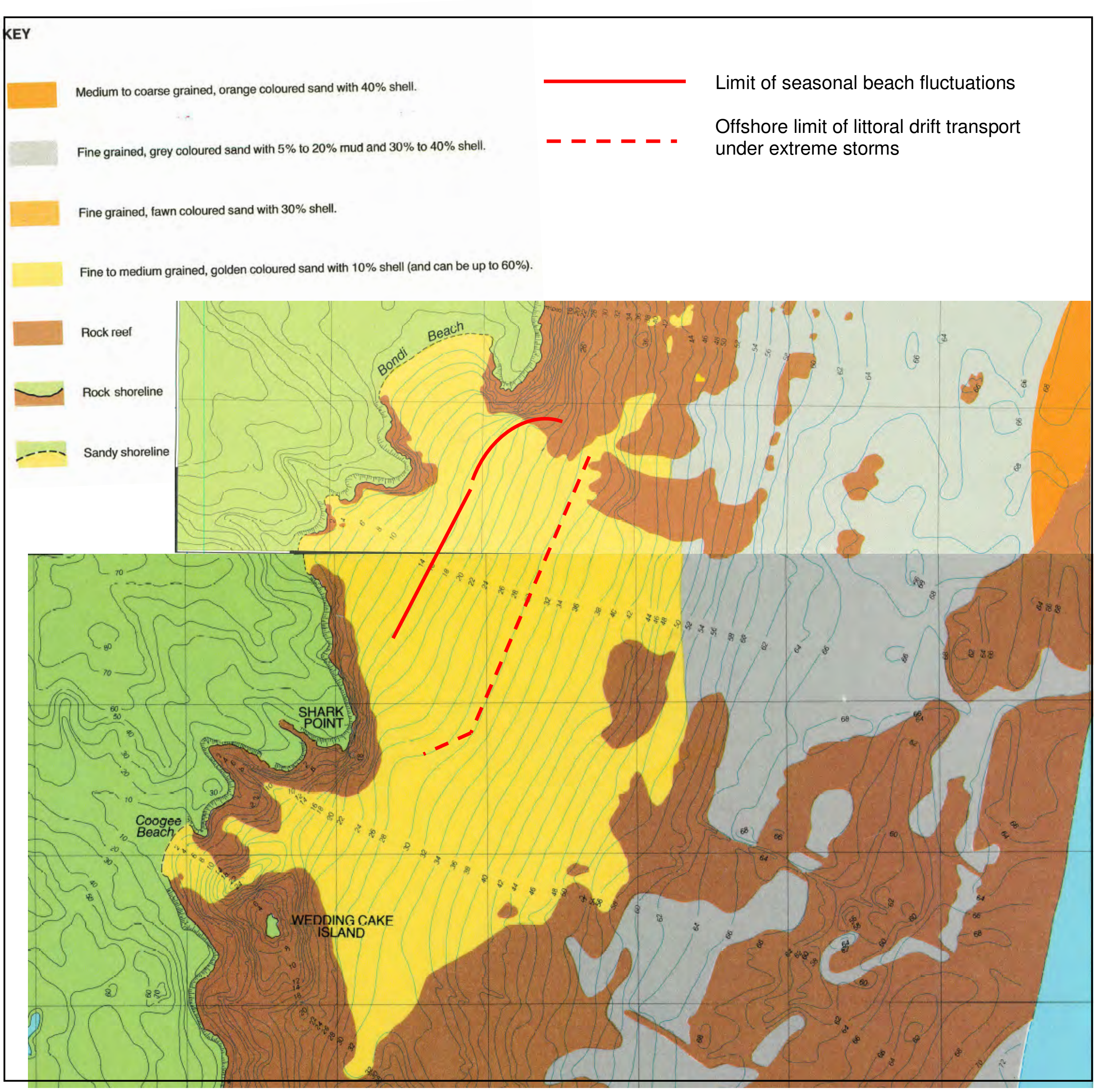

Figure 3.2 Regional seabed map of Inner Shelf sediment/rock distribution between Bondi Beach and Coogee Beach (NSW Govt 1989)

3.4.3 Aeolian Sediment Transport

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

In the past, Bondi Beach in particular was subjected to the removal of beach sand through this process. Old photographs show large bare transgressive sand dunes behind the beach. However, today these have been stabilised by vegetation and development and there are an elevated seawalls landward of the beach along the Waverley beaches. These seawall structures modify the local wind field in such a way so as to cause wind borne sand to drop out of suspension at the sea walls. Therefore, from an overall sediment budget perspective, it is likely that there would be significant sand losses from beaches in the study area due to aeolian processes, notwithstanding that there is some need for sand management practices to deal with sand deposits along and behind the seawalls from time to time. When required, council undertake localised beach scraping, to lower levels of the sand against the seawall, regrading it seaward. In this regard, it is noted that in storms in November 1987 (refer to **Figure 3.3**) and June 2007, windblown sand did move landward into properties and roads, which was probably more a nuisance than a significant sediment loss from the beach. In 2007 the Daily Telegraph reported that "Around 1000 tonnes of sand was scraped up and moved back to Bondi and Tamarama after high winds swept it inland on to parks and roads." Sand build up against the seawall and sand blowing over the seawall was observed by during a site visit on 15/06/11 (refer **Figure 3.4**).

Figure 3.3 Storm at Bondi Beach November 1987

Figure 3.4 Bondi Beach, 15 June 2011

3.4.4 Sediment Transport at Stormwater Systems

There are several stormwater outlets on the Waverley beaches. Historically, the beaches were backed by creeks and lagoons that discharged across the beaches, breaking though the beach berms and causing scour channels during high flows. As the beaches were developed the creeks were piped, the lagoons drained and in-filled, and all stormwater was piped. However, these original stormwater pipes had outfalls at the landward side of the beaches, which meant they still discharged across the beaches, either ponding at the back of the beach during low flows or scouring channels through the beach during high flows. The stormwater ponding and scour channels associated with the old southern stormwater channel at Bondi, can be clearly seen in the earlier historical aerial photographs included in **Appendix C**. At the present day, all stormwater outlets are now constructed immediately adjacent to a headland, with outfall locations predominantly beyond the beach (refer **Figure 3.6** to **Figure 3.9**). **Figure 3.5** presents the PWD 1988 historical photogrammetric profiles at the point of the old stormwater outfall at southern Bondi with the 2005 ALS profile superimposed over the top. This shows clearly how the beach has accreted since the removal of the outfall, with the mitigation of the stormwater scour.

Potential beach management issues associated with stormwater outlets include:

- Localised erosion resulting from stormwater scour
- Localised lowering of beach level as result of stormwater erosion/scour allowing large waves to access back beach area
- Potential reduction in amenity as result of strong flows and reduced water quality

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

Table 3-6 provides a general description of the stormwater outlets on each beach and potential impacts they may have on coastal processes.

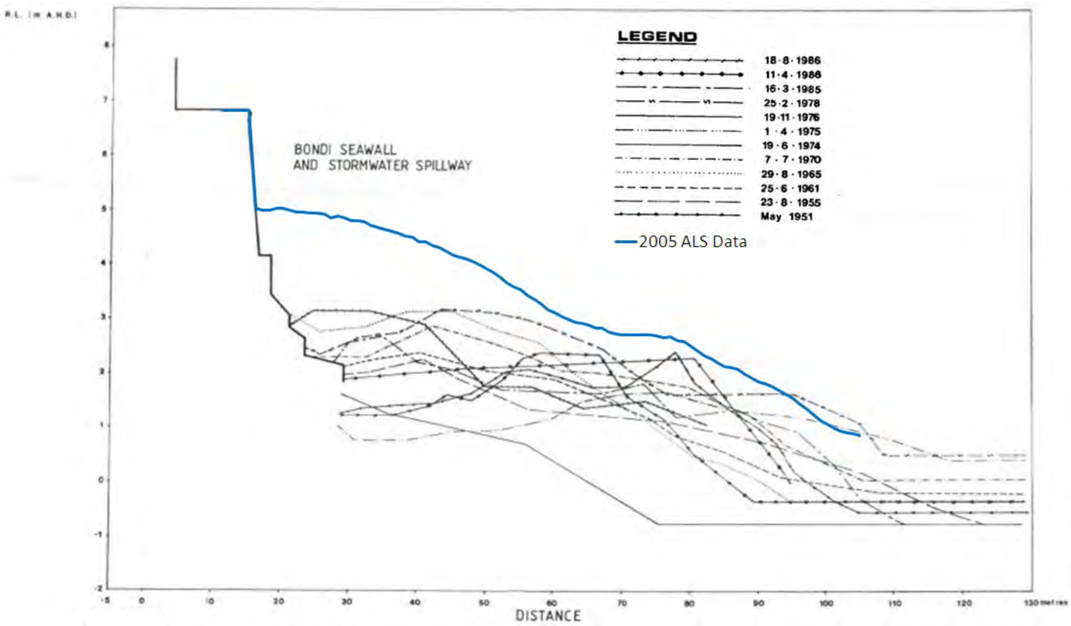

Figure 3.5 2005 ALS data presented over the PWD 1988 Photogrammetric profiles

Table 3-6 Stormwater outlets on open coast beaches

Beach	Location	Description	Impact on Coastal Processes
Bondi	South	2.44m by 1.83m box culvert outfall invert level of 0.31mAHD.	Low potential for localised erosion at outfall location during high flows. Minimal impact on processes
	North	2.44m by 1.675m box culvert outfall invert level at 0.68mAHD	Outfall channel cut into rock, offshore. Minimal impact on processes.
Bronte	North	2.75m by 1.8m box culvert invert level at outfall -0.6mAHD	Potential for localised scour at outfall. Minimal impact on processes
Tamarama	South	2.49m by 1.37m box culvert invert level at outfall 1.31mAHD	Potential for localised scour at outfall. Minimal impact on processes.

Bondi and Bronte stormwater systems discharge onto rock, seaward of the beach sand and, therefore, are unlikely to cause erosion of beach sand and would not be expected to supply any significant quantities of sand to the beach system. Tamarama stormwater system discharges onto sand. However, it extends most of the distance beyond the beach sand and it would be expected that rock would be at a very shallow level at the outfall location. Therefore it is also unlikely to cause significant erosion of beach sand and would not be expected to supply any significant quantities of sand to the beach system.

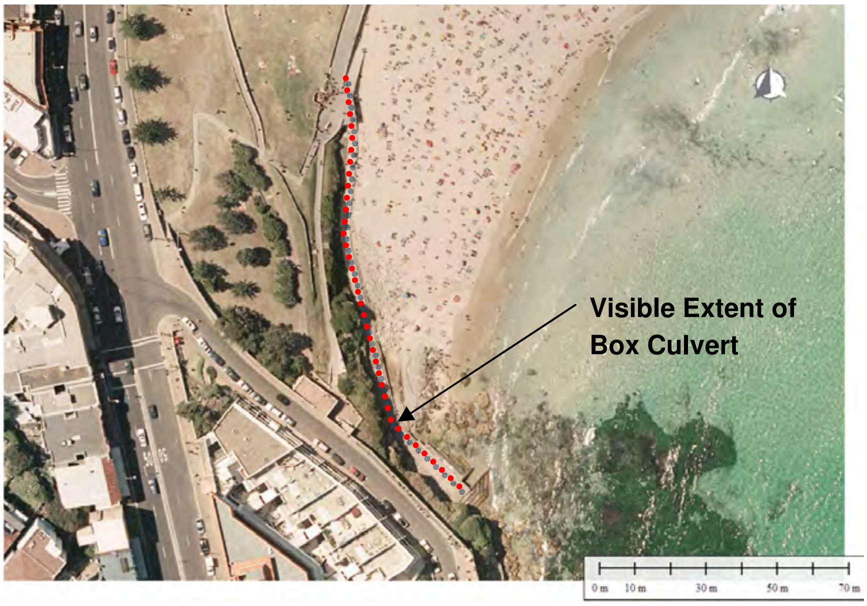

Figure 3.6 Southern end of Bondi Beach stormwater

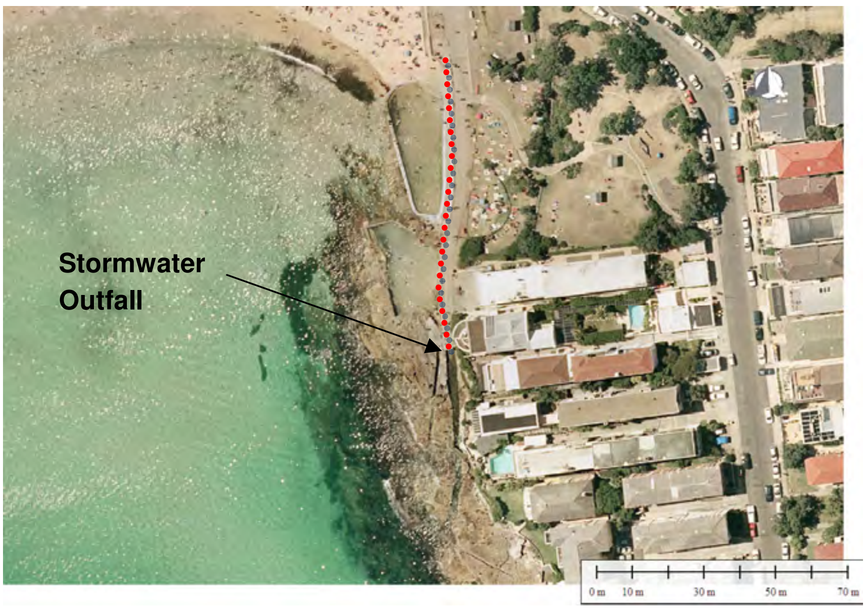

Figure 3.7 Northern end of Bondi Beach stormwater

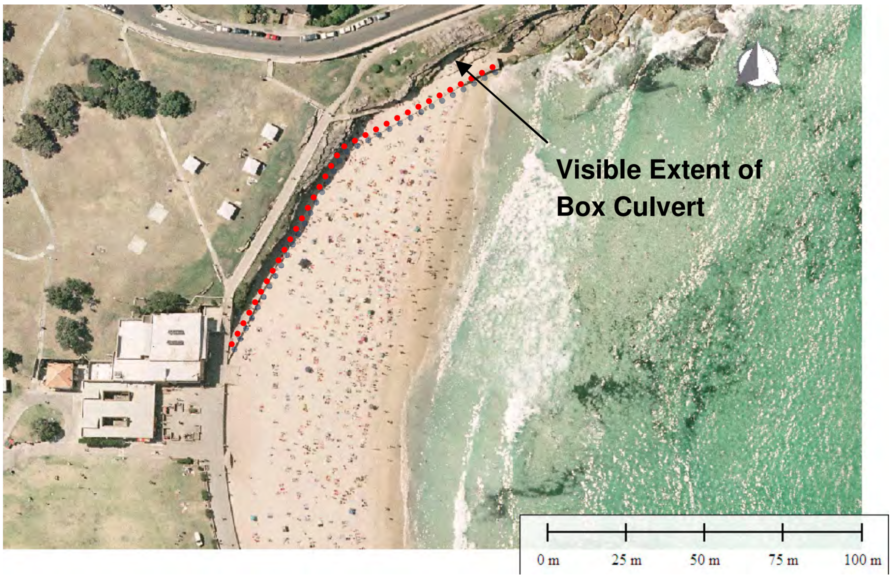

Figure 3.8 Northern end of Bronte Beach stormwater

Figure 3.9 Tamarama Beach stormwater

Surface water runoff from storms of intense rainfall can cause localised flows down access ramps and steps and over the crest of the seawall (**Figure 3.10**). This can cause localised scour along the interface with the beach. During storms in 1986, significant beach scour was caused due to surface water runoff, as shown in (**Figure 3.11**). PWD made recommendations for surface drainage improvements to mitigate this. However, as discussed with Council at the meeting of June 15th, such scour did not pose any problems.

Figure 3.10 Scour in front of Bondi Seawall caused by stormwater runoff

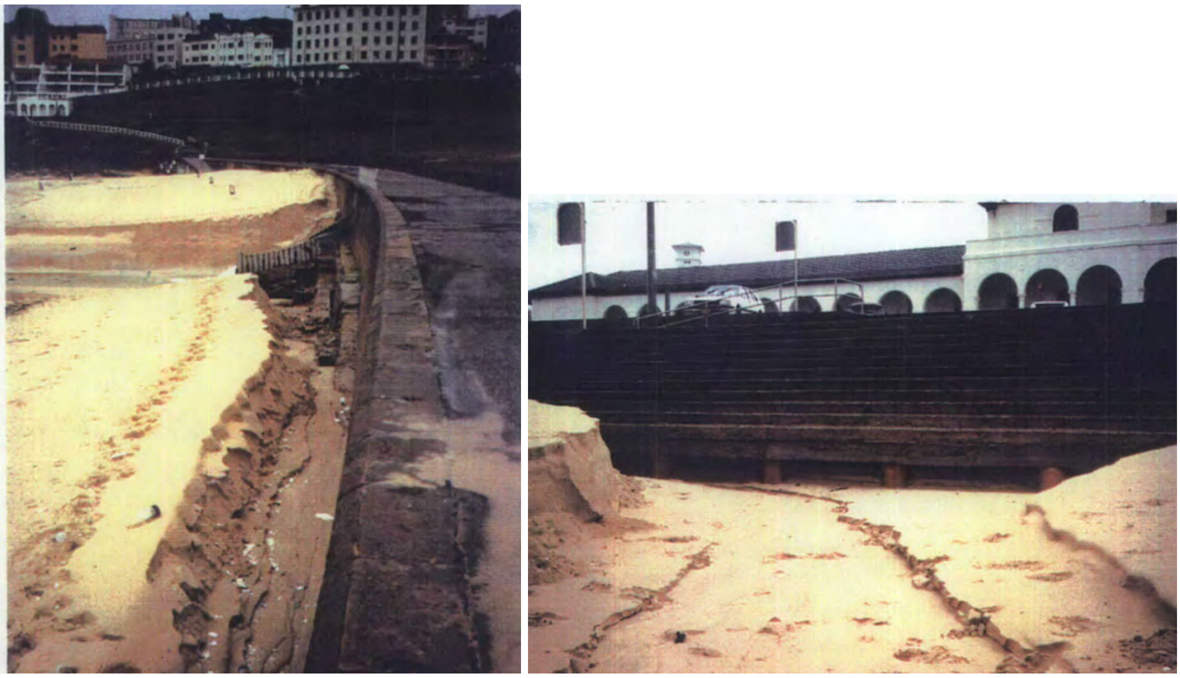

Figure 3.11 Scour along the seawall and steps leading to the pavilion, from surface water runoff during August 1986 storm. Note that the stormwater outlet shown in the left photo no longer exists. (Source: PWD 1988)

3.4.5 Overall Sediment Budget

The overall sediment budget for the study area (Bronte, Tamarama and Bondi) is likely to be essentially closed. That is, there are likely to be no significant net gains or losses of sand into or from this combined beach embayment. This is because there is likely to be no significant transport of sand over the boundaries of the system, namely the:

- northern and southern (headland) boundaries; there are no paths for littoral drift transport over these boundaries due to the presence of rocky headlands and reefs;
- seaward boundary; sand transported offshore in storms would be expected to return in calmer conditions as the nearshore sands extend well past the offshore limits of sand transport; and
- landward boundary; due to the vegetation coverage and seawalls there is little opportunity for sand to be lost or gained from/to the beach system in a landward/seaward direction in this manner.

PWD (1988) also considered that it was unlikely that there was significant long term loss of sediment from Bondi Beach. This was evidenced by a comparison of sand reserves, which showed no long term loss of sand, the compartmentalisation of the beach by rocky headlands and reefs, the stabilisation of the dune field by urban development and the sound beach grooming practices adopted by Council.

3.5 Wave Runup and Overtopping

Runup levels at the study beaches could be in the order of 7 m AHD or above, but this would only be realised if the foreshore was at this runup height or higher. In reality, all the beaches have seawalls and any waves that overtopped the seawalls would fold over the 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.

The seawall at Bondi beach has a crest level which varies from 7 m AHD in the south to 6 m AHD towards the north, dipping to 3.8 m AHD at the very northern extent of the wall. Therefore, it would be expected that during extreme conditions overtopping of the seawall could occur. PWD 1988 made an estimate of runup and overtopping of Bondi seawall. Runup was estimated to be between 7 m AHD (in the north) and 9 m AHD (in the south). Comment was made that photographs taken during the 1974 storm recorded wave run up almost reaching the wall crest at the south end of the beach, where the crest elevation is about 7 m AHD. Beyond the northern extent of the seawall, there are several properties on Ramsgate Avenue that front the ocean and could be subjected to wave overtopping. These properties have levels at their frontage as low as 1 m AHD.

The ALS level data provided by Council suggests that the crest level of Bronte seawall varies from around 3.9 m AHD in the south to 4.8 m AHD in the centre of the beach in front of the amenity block. These levels are well below potential runup levels and therefore overtopping would be expected during extreme events. This is consistent with comments documented in news reports during previous storm events (refer **Section 3.1.3**) including "*Huge seas swept over promenade at Bronte flooding Bronte Park to within a few feet of the roadway (1948)" and "30 foot wave overtopped promenade at Bronte and swept 20 yards through the surf club (1959)"*. Overtopping was also documented in photographs taken during the 1974 storms (**Figure 3.12**).

Figure 3.12 Overtopping at Bronte, 1974 (source: Waverley Council)

The ALS level data provided by Council suggests that the crest level of Tamarama seawall varies from around 5.4mAHD in the south to 5mAHD in the north. These levels are below potential runup levels and therefore overtopping would be expected during extreme events.

3.6 Climate Change

3.6.1 Introduction

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

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

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

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

3.6.2 Sea Level Rise

Over the period 1870–2001, global sea levels rose by 20 cm and sea levels are expected to continue rising throughout the twenty-first century. There is no scientific evidence that sea levels will stop rising beyond 2100 or that the current trends will be reversed in the foreseeable future.

Sea level rise is an incremental process and will have medium to long-term impacts. The more extreme national and international projections of sea level rise along the NSW coast are for a rise relative to 1990 mean sea levels of 40 cm by 2050 and 90 cm by 2100 (NSW Govt 2009). However, the Intergovernmental Panel on Climate Change (IPCC) in 2007 also acknowledged that higher rates of sea level rise are possible.

In simple terms, sea level rise will raise the average water level of oceans and estuaries. As the average water level rises, so too will high and low tide levels affecting the natural processes responsible for shaping the NSW coastline. Exactly how the coast and estuaries will respond is complex and often driven by local conditions but, in general, higher sea levels may lead to:

- increased or permanent tidal inundation of land by seawater
- recession of beach and dune systems and, to a lesser extent, cliffs and bluffs
- changes in the way that tides behave within estuaries

- saltwater extending further upstream in estuaries higher saline water tables in coastal areas
- increased coastal flood levels due to a reduced ability to effectively drain low-lying coastal areas.

These physical changes will have an impact on coastal ecosystems, access to and use of public and private lands, historical and cultural heritage values, arable land used for agriculture, freshwater access, public and private infrastructure, and low-lying areas of coastal land that are affected by flooding. Sea level rise will also affect coastal hazards such as beach erosion during storms and coastal flooding.

The NSW Government promotes an adaptive, risk-based approach to managing the impacts of sea level rise. The adaptive risk-based approach recognises that, potentially, there are significant risks from sea level rise and that the accuracy of sea level rise projections will improve over time.

The NSW Government has adopted sea level rise planning benchmarks to support this adaptive risk-based approach. These benchmarks will enable the consistent consideration of sea level rise within this adaptive risk-based management approach. The primary purpose of the benchmarks is to provide guidance supporting consistent considerations of sea level rise impacts, within applicable decision-making frameworks. This includes strategic planning and development assessment under the EP&A Act and infrastructure planning and renewal.

The use of the benchmarks is required when undertaking coastal and flood hazard assessments in accordance with the Coastline Management and Floodplain Development Manuals. It is already a statutory requirement that the preparation of local environmental plans give effect to and be consistent with these manuals. The NSW sea level rise planning benchmarks are an increase above 1990 mean sea levels of 40 cm by 2050 and 90 cm by 2100, with the two benchmarks allowing for consideration of sea level rise over different timeframes. The benchmarks were established by considering the most credible national and international projections of sea level rise and take into consideration the uncertainty associated with sea level rise projections. The Government will continue to monitor sea level rise observations and projections and will review these planning benchmarks periodically, with the next review likely to coincide with the release of the fifth IPCC report, due in 2014.

3.6.3 Other Climatic Change Considerations

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

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

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

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

Given the above uncertainty and difficulty in quantitative prediction, no specific account was taken of any potential changes to storm frequency and intensity, or changes in wave directions[5]. However, this uncertainty should be taken into consideration when assessing the risk and consequences of recession occurring in the future.

3.7 Current shoreline Protection

3.7.1 Bondi

A seawall is continuous along the full length of Bondi Beach, with a crest level that varies from 6.0 to 7.0 m AHD, dipping to 3.8 m AHD at the very northern extent of the wall. A promenade has been constructed against the seawall. The promenade pavement generally consists of concrete slabs cast flush against the top of the seawall. The promenade is typically about 9 m wide but narrows to 5 m at the southern end and widens to over 12 m at the northern end. Steps in the wall allow access to and from the beach in front of the Bondi Pavilion while a number of ramps constructed in front of the wall provide access at other locations.

Along the northern and southern sections of the promenade, dwarf brick retaining walls up to 1 metre high run parallel to the seawall along the landward edge of the promenade. The walls were installed for the purpose of intercepting wind driven sand blown off the beach. A masonry retaining wall up to 2.7 m high supports Queen Elizabeth Drive which runs adjacent to and above the promenade for a length of about 500 m in the centre of the beach.

Investigations on the Bondi seawall structure were undertaken by PWD 1988, this included metal detection of reinforcement, 17 concrete core samples (into the face of the seawall), sediment sampling and site excavations (10 boreholes and 15 test pits). These investigations (refer **Figure 3.14**) suggested that the historical design drawings (**Figure 3.13**) provide a reasonable representation of the seawall. A long section of the seawall , showing the varying toe depth along its length, is included as **Figure 3.17**). The seawall details outlined below are taken from the PWD 1988 report.

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

In 1909 the southern section of the seawall was constructed. This southern section of the seawall (approximately south from the Pavilion) is an unreinforced concrete gravity wall. According to test pits, the toe level of the concrete is around 2 m AHD in some sections and around 4 m AHD in other sections. In some areas bed of sandstone gravel and cobbles up to 300 m thickness was observed beneath the wall. Concrete core tests suggested an average concrete compressive strength of 12 MPa.

The seawall was extended to the northern end of the beach in about 1923. This section of seawall is a reinforced concrete counterfort wall. According to test pits, the toe level of the concrete is around 4 m to 4.7 m AHD for most of its length, with some sections at around 2 m AHD in the north. Investigations suggested that the reinforcement consists of 10 mm diameter bars, with horizontal bar spacing 230 to 300 mm and bar vertical spacing of 400 to 580 mm and cover of 35 to 50 mm. Concrete core tests suggested an average concrete compressive strength of 20 MPa.

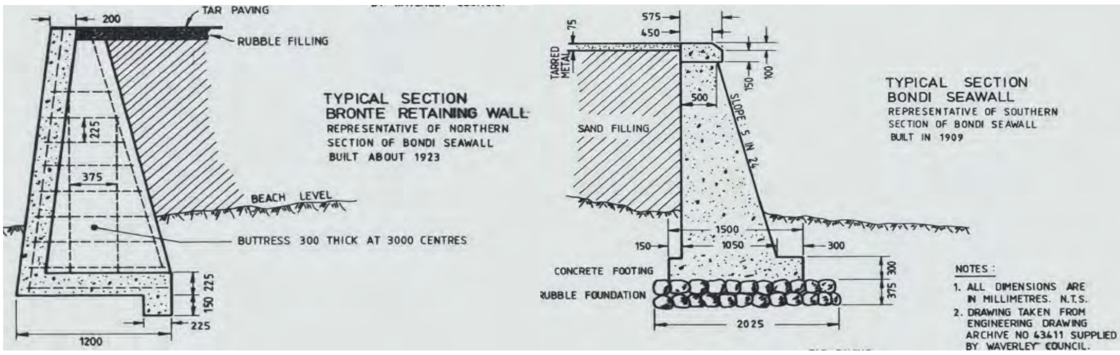

Figure 3.13 Reproduction of Historical Design Drawings of Bondi and Bronte Seawalls (Source: PWD, 1988)

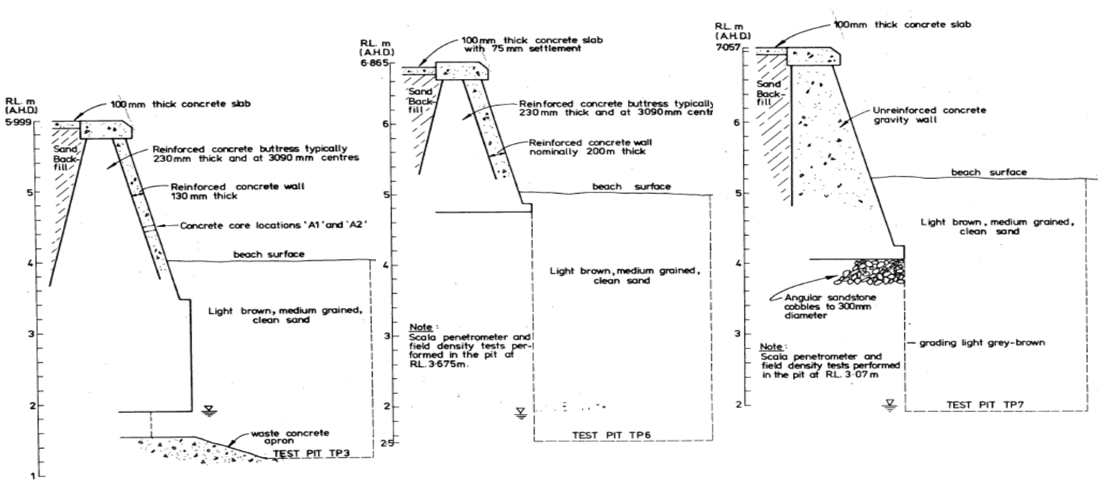

Figure 3.14 Cross Sections from test pits (Source: PWD 1988)

Sandstone bedrock is only a foundation material in the very southern and northern extents of the seawall. Therefore, most of the seawall is founded on sand. The beach and backfill were classified as being poorly graded sand of fine to medium grain size, consistent with beach sand. The backfill material can be clearly seen in **Figure 3.15**, showing failure of the seawall in 1929 recorded as being due to stormwater. PWD 1998 suggested that the variance in the toe level of the seawall could be due to the two different stages in which the seawall was constructed and also due to later repair or reinforcement works (e.g. piling and underpinning), no further details of any repair works were provided.

These original seawall structures still exist today although some repair works have been undertaken. Two key works on the seawall were undertaken in the late 1980's and early 1990's. These included relocating the stormwater outlet in the south of the seawall to the southern headland and construction of a Reno-mattress at the toe of the seawall. The construction drawings for the stormwater improvements, dated 1986, detailed the demolition of the existing concrete apron, infilling and sealing the old outfall pipes and construction of a new concrete box culvert discharging onto the rocks at the southern headland. Details of the Reno-mattress toe protection works were provided by Council (**Figure 3.16**). Construction photos (**Figure 2.11**) and "as constructed" drawings provided by Council indicated that construction along the entire length of the wall was carried out between 1987 and 1992.

Figure 3.15 Bondi Northern Seawall Failure, 1929 (Source: Waverley Council)

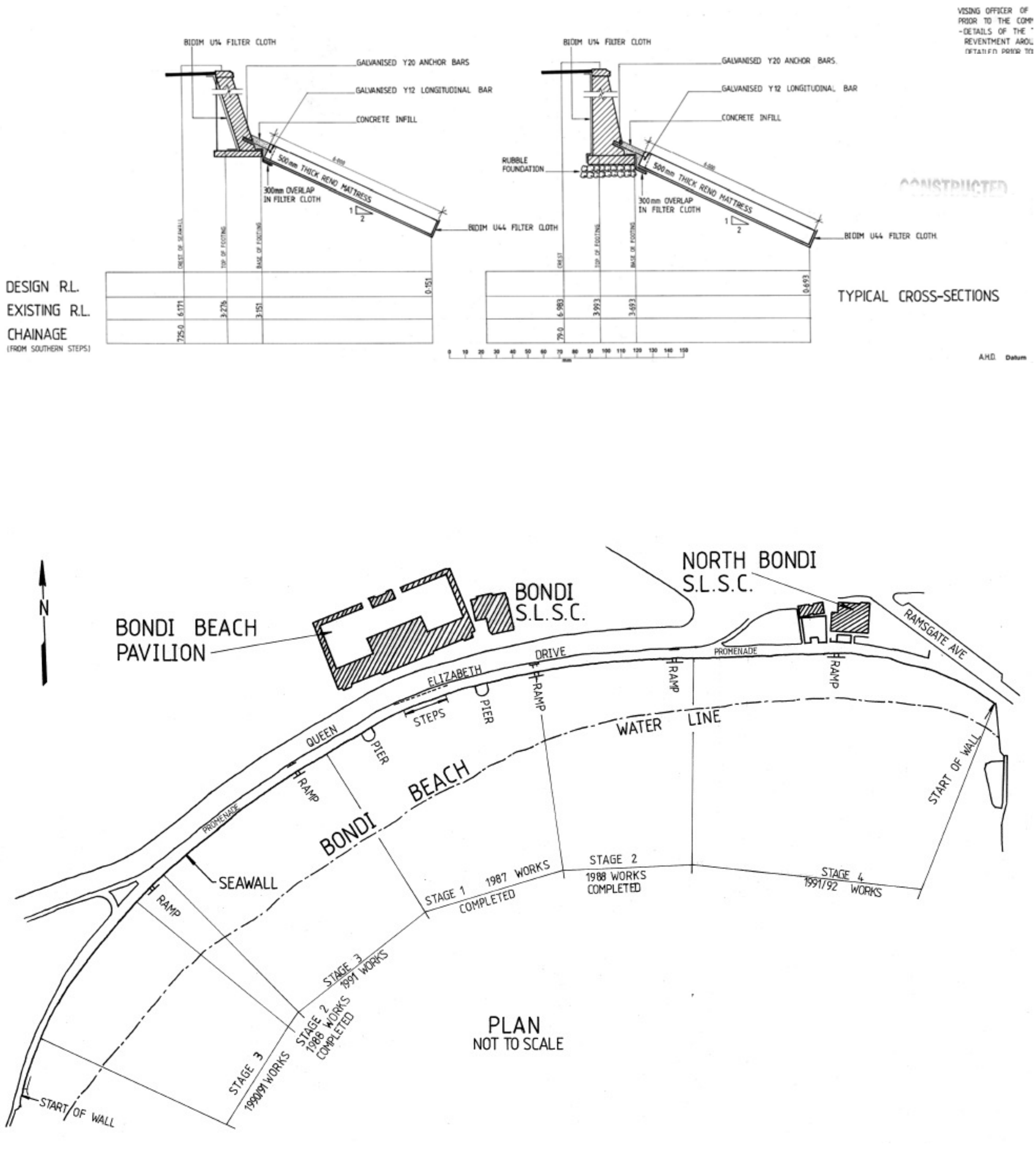

Figure 3.16 Bondi Seawall Toe Improvement works. Typical sections (top); Extent of works (bottom) (Source: Council plan dated 18-1-89)

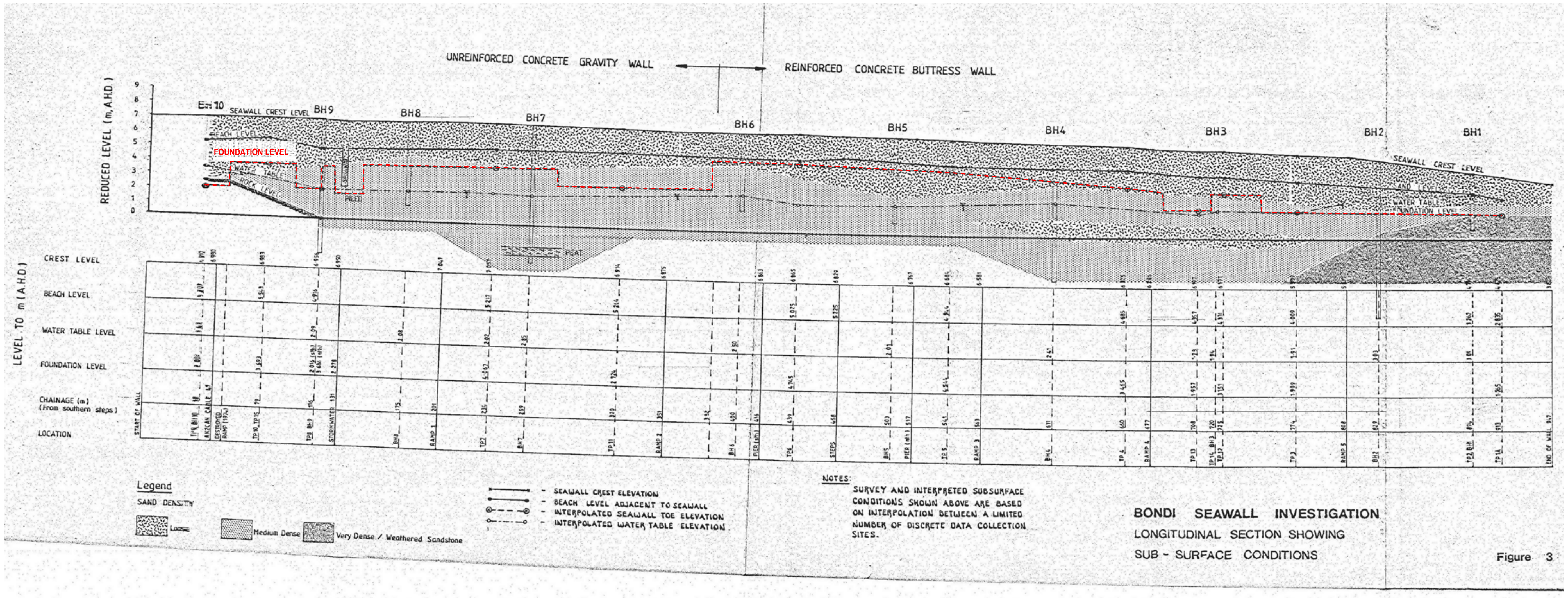

Figure 3.17 Long Section Of Bondi Seawall and Foundations (Source: PWD 1988)

3.7.2 Bronte

The sea wall and promenade were constructed at Bronte Beach between 1914 and 1917. PWD 1988 indicated that the Bronte seawall design was similar to the northern section of Bondi seawall (refer **Figure 3.13**). This suggests that the Bronte seawall is of reinforced concrete counterfort wall construction. The seawall runs the entire length of the beach between the headlands. No data has been able to be sourced on the toe levels of the base of the seawall.

The ALS level data provided by Council suggests that the crest level of Bronte seawall varies from around 3.9 m AHD in the south and 4.8 m AHD in the centre of the beach in front of the amenity block. Just north of the amenity block the seawall returns into the cliff face and terminates. The northern section of the beach is back by the cliff face, without a seawall.

Notes on historical storm events comment on a number of occasions (refer **Section 3.1.3**) that overtopping of Bronte seawall occurred flooding Bronte park, and that the Bronte baths were damaged by waves. However, there is no mention of damage to the seawall and it is not known what (if any) repair works have been undertaken to the seawall since its construction.

3.7.3 Tamarama

The first seawalls were built at Tamarama by Anderson for Wonderland City in 1906/7, but much of these were in ruins by the 1920's. Plans were prepared in 1924, for new works to the Beach (refer **Figure 3.18**) including stormwater and seawall works. The plan shows existing sections of sea wall and a proposed new section of wall linking up with the existing walls. It would seem that the new sea wall was built shortly afterwards, along with several picnic huts (first proposed in 1923), as **Figure 3.19** shows. No information on the cross section design of Tamarama seawall has been able to be sourced.

The ALS level data provided by Council suggests that the crest level of Tamarama seawall varies from around 5.4 m AHD in the south and 5 m AHD in the north.

Notes on historical storm events (refer **Section 3.1.3**) include the comment during the 1974 storms *"seawalls at Bondi and Tamarama suffered $100,000's worth of damage"*. However no information has been able to be sourced on any repair works, which may have been undertaken.

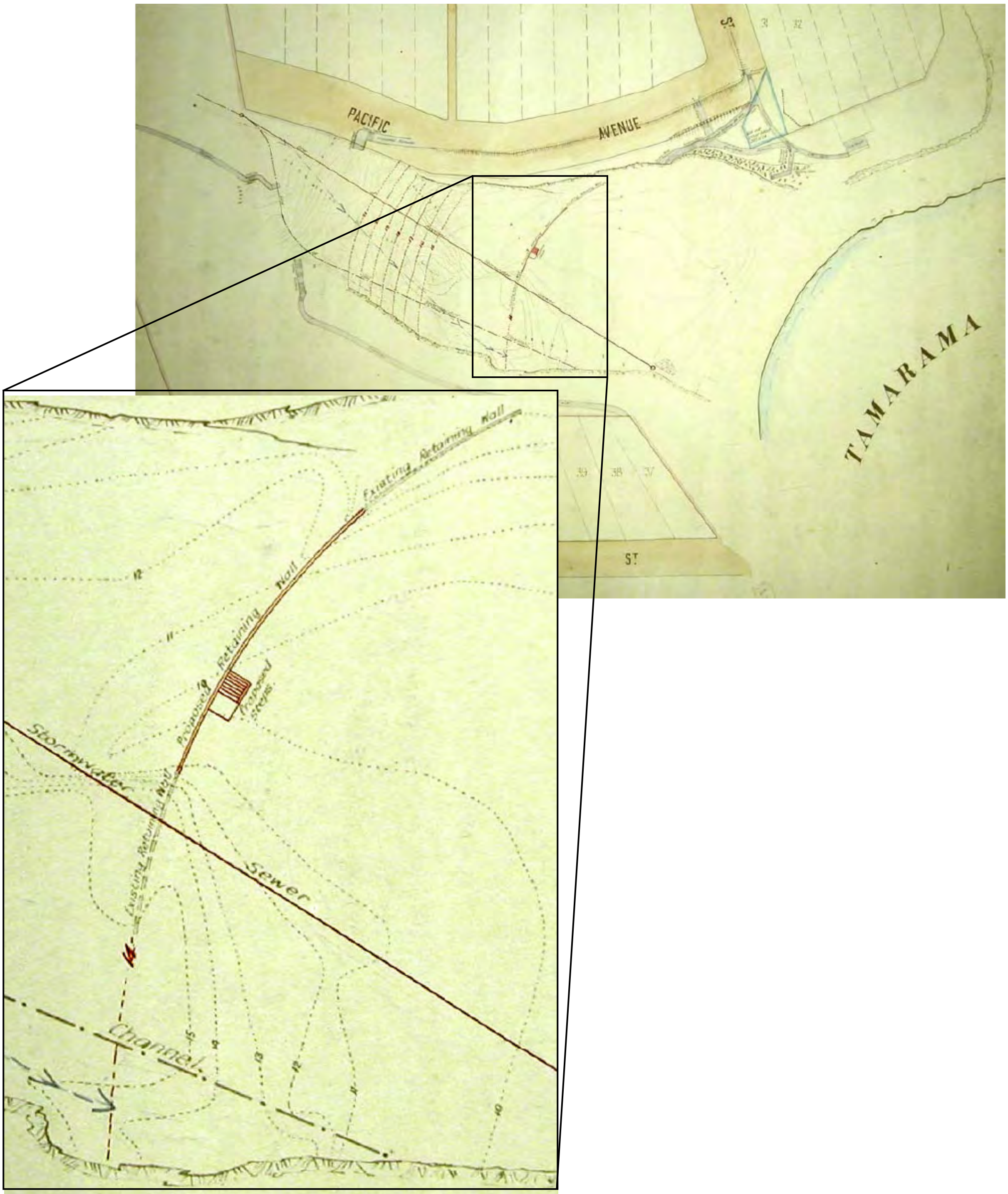

Figure 3.18 Excerpt from a plan showing proposed retaining wall joining two sections of the existing old retaining wall, complete with steps down to the sand, 1924 (Source: Warwick Mayne-Wilson & Ari Anderson Conservation Landscape Architects, Tamarama A Settlement Paradigm, 2010).

Figure 3.19 Tamarama, 1927. A seawall can be seen. (Source: Waverley Library)

4. COASTLINE HAZARD ASSESSMENT

The following hazards are discussed in the following sections:

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

4.1 Beach Rotation

Studies of embayed beaches on the NSW coast have identified a sensitivity of shoreline alignment to mean wave direction that has been linked to the Southern Oscillation Index (SOI). Since 1876, the maximum value of the monthly average of the SOI that has been recorded was +34.8 in August 1917. For much of that year the monthly average of the SOI was above +20 and several very severe storms were experienced along the entire NSW coast from June to November that year. From January to May, 1974, the monthly average of the SOI varied from around +20 to +10, which may have been relevant to the occurrence of the severe storms of May – June 1974.

Goodwin (2005) demonstrated that, since the 1880s, the monthly mid-shelf mean wave direction (MWD) for southeastern Australia has varied from around 125°T to 145°T with a strong annual cycle coupled to mean, spectral-peak wave period. Months and years when a more southerly MWD occurs are accompanied by an increase in the spectral-peak wave period. The most significant multi-decadal fluctuation in the time series was from 1894 to 1914, when Tasman Sea surface temperatures (SST) were 1.0°–1.5°C cooler, monthly and annual wave directions were up to 4°–5° more southerly and, by inference, spectral-peak wave periods were longer when compared with the series since 1915. The sustained shift in wave direction would have had a significant influence on beach and coastal compartment alignment along the NSW coast (Goodwin, 2005).

Studies of beach rotation as a result of variations in the SOI have been undertaken at Narrabeen Beach and Palm Beach (Short *et al.*, 2000; Ranasinghe et al., 2004). Data from Ranasinghe *et al.* (2004) indicated an anti-clockwise rotation of these beaches as a result of a positive value in the SOI and vice versa. A sustained SOI of +10 to +20 (a *La Niña* episode) resulted in an anti-clockwise rotation of Narrabeen Beach by around 0.9° and a sustained SOI of around +15 to +26 resulted in a similar rotation of Palm Beach by around 0.7°. On the other hand, a sustained SOI of −10 to −16 (an

El Niño episode) resulted in a clockwise rotation of Narrabeen Beach by around 1.2° and a sustained SOI of –25 to –38 resulted in a clockwise rotation of Palm Beach by around 0.7°.

These rotations were reflected in the translation of the mean waterline or swash zone of the beach berm and they did not affect the dune alignment. Analysis of 23 years of monthly profiles at Narrabeen Beach showed that rotations accounted for up to 15 m and some 30 m^3/m (above MSL) of the shore-normal beach sand exchange (Short *et al.*, 2000). At Palm Beach, the maximum recession of the swash zone that was recorded over the 2.5 year period was around 10 m (Ranasinghe et al., 2004), which represented the removal of around 20 m^3/m of sub-aerial beach sand store at the extreme ends of the beach. For a given degree of beach rotation, greater recession or progradation of the swash zone and, hence, greater beach sand exchange would be expected on longer beaches.

These beach rotations were considered to be caused by changes to both the mean direction and magnitude of wave energy flux, the signature of which is reflected in the SOI. The larger magnitude of wave energy flux induced greater onshore/offshore sand transport whereas changes in direction affected also alongshore transport rates and directions. During a *La Niña* event (+ve SOI) there were more south-easterly storms whereas during an *El Niño* (–ve SOI) event there was a propensity for more benign east-north-easterly waves.

Both Narrabeen Beach and Palm Beach are exposed open coast beaches and would experience the maximum shift in the mean direction of offshore wave energy flux. On open coast beaches, the *La Niña* events, which are correlated to severe storms, may result in recession of the swash zone at the extreme northern ends of the beaches. This occurs rapidly following the SOI shift (a few months; Ranasinghe *et al.* 2004) and may result in reducing the available sand store on the beach that provides a buffer to the storm erosion demand. However, as the concomitant accretion at the southern end of the beach lags the SOI trend shift considerably (by up to and in excess of 1 year; Ranasinghe *et al.* 2004), this obviates any advantage that the accreted swash zone may accrue to supplying the storm erosion demand.

BONDI BEACH

Bondi Beach is some 850 m long. At Bondi, a variation of ±10º in offshore mean wave direction would result in a variation of around ±2º in the nearshore mean wave direction as a result of wave refraction. If it is assumed that the beach berm would align to the mean nearshore wave direction, a rotation of the beach berm of around ±2º can be expected. This is about double that observed at Narrabeen and Palm Beach (~±1º). Such a variation would result in a plan fluctuation in the waterline along the beach berm of around ±15m at each end of the beach, reducing to 0 m towards the centre. That would represent a variation in beach berm volumes of around ±30 m^3/m at each end of the beach.

The maximum measured fluctuations at the ends of Bondi Beach vary from ±25m^3/m at the northern end to ±92m^3/m at the southern end. Much of this variation can be attributed to storm erosion demand rather than beach rotation.

TAMARAMMA BEACH

Tamaramma Beach is some 100 m long. In the absence of any data for Tamaramma Beach, we have assumed that a variation of ±10º in offshore mean wave direction would result in a variation of around ±2º in the nearshore mean wave direction as a result of wave refraction. Such a variation would result in a fluctuation in the beach berm of around ±2m at each end of the beach, reducing to 0 m towards the centre. That would represent a variation in beach berm volume of around ±4 m^3/m above AHD at each end of the beach. Such changes are insignificant when compared with the cross-shore sand volume changes that occur during storms.

BRONTE BEACH

Bronte Beach is some 230 m long. In the absence of any data for Bronte Beach, we have assumed that a variation of ±10º in offshore mean wave direction would result in a variation of around ±2º in the nearshore mean wave direction as a result of wave refraction. Such a variation would result in a fluctuation in the beach berm of around ±4m at each end of the beach, reducing to 0 m towards the centre. That would represent a variation in beach berm volume of around ±8 m^3/m above AHD at each end of the beach. Such changes are insignificant when compared with the cross-shore sand volume changes that occur during storms.

4.2 Beach Erosion Hazard including Recession

4.2.1 Design Storm Erosion Demand

The design storm erosion demands (above AHD) recommended for each beach for the 100 year ARI event are summarised in **Table 4-1.** These values are based on the maxima recorded at Bondi Beach, including an allowance for beach rotation, and values recommended in Nielsen *at al.* (1992) for open coast beaches where no field data exist.

Table 4-1 Recommended Design Storm Erosion Demand values for study area

Location		Storm Demand (m^3/m above AHD)	Comments
Bondi	South	220	Maximum difference between profile measurements plus 30 m^3/m for beach rotation
	Mid	110	Maximum difference between profile measurements
	North	70	Maximum difference between profile measurements plus 30 m^3/m for beach rotation
Bronte	Mid	250	As no data are available, a maximum value of 250 m^3/m as been adopted from Nielsen *et al.* (1992).
Tamarama	Mid	250	

4.2.2 Impacts of Sea Level Rise

While there has been no recorded long term recession of Waverly Council's beaches, a projected sea level rise has the potential to cause recession of the beach foreshores.

The most widely accepted method of estimating shoreline response to sea level rise is the Bruun Rule (Bruun, 1962; 1983). Bruun (1962, 1983) investigated the long term erosion along Florida's beaches, which was assumed to be caused by a long term sea level rise. Bruun (1962, 1983) hypothesised that the beach assumed an *equilibrium profile* that kept pace with the rise in sea level without changing its shape, by an upward translation of sea level rise (S) and shoreline retreat (R).

The Bruun Rule (**Figure 4.1**) equation is given by:

$$R = \frac{S}{(h_c + B)/L}$$

where: R = shoreline recession due to sea level rise;
S = sea level rise (m)
h_c = closure depth
B = berm height; and
L = length of the active zone.

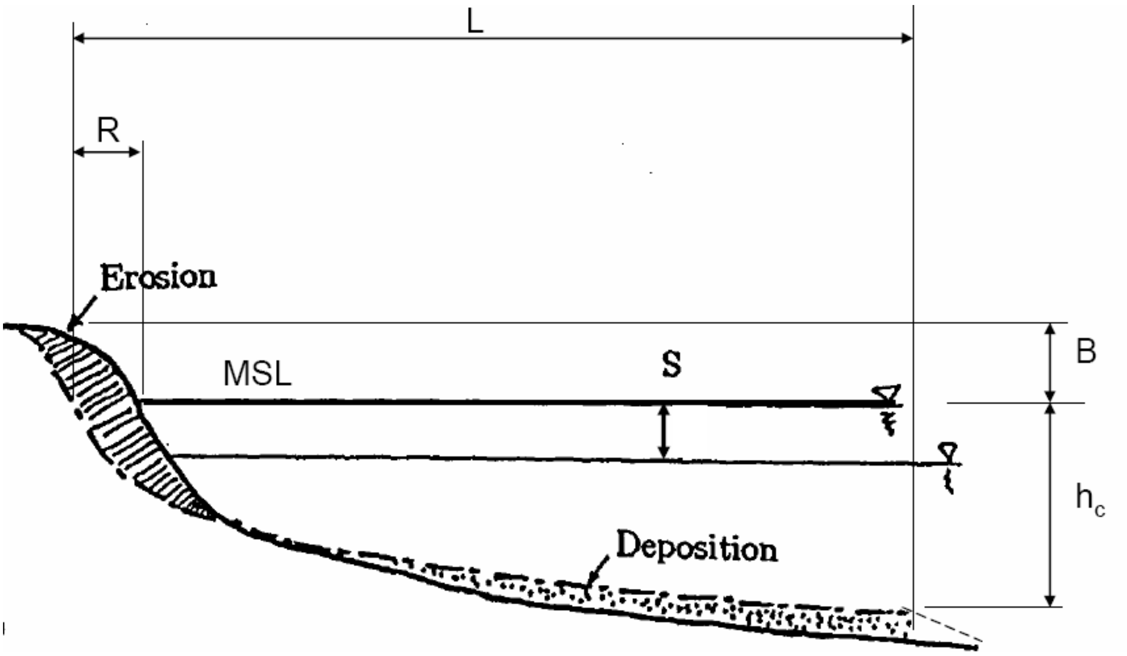

Figure 4.1 Diagram for the Bruun Rule

Berm height is taken to be the average height of the beach berm along the beach (~3 m AHD) and closure depth is the depth at the seaward extent of measurable sand movement. The length of the active zone is the distance offshore along the profile in which sand movement still occurs.

The Bruun rule assumes that the beach profile is in an equilibrium state with the prevailing wave climate. A beach that is in equilibrium with the wave climate will develop a beach profile that has been described variously by Bruun (1954, 1962) and Dean (1991) as:

$$h = Ax^{2/3}$$

where h is the water depth at a distance x offshore.

The parameter, A, is dependent on the sediment fall velocity, w, thus (Dean, 1987):

$$A = 0.067w^{0.44}$$

And the fall velocity, w, can be related to the sediment grain size diameter, d, thus (Hallermeier, 1981):

$$w = 14d^{1.1}$$

Several schema exist, based on analytical and laboratory studies, to determine closure depth and length of the active zone, including Swart (1974) and those of Hallermeier (1981, 1983).

Swart (1974) developed a physically-based schematic model of cross-shore profile development and calibrated this against small scale and full-scale model tests under regular wave conditions. The lower limit of offshore sand transport was defined as:

$$h_m/\lambda_o = 0.0063\exp[4.347H_o^{0.473}/(T^{0.894}D_{50}^{0.093})]$$

Where

h_m	=	water depth at the limit of offshore sand transport (m)
λ_o	=	deepwater wavelength (m)
H_o	=	wave height (m)
T	=	wave period (s)
D_{50}	=	median grain size (m)

In applying this formula to the field Swart (1974) did not provide guidance as to what the appropriate wave parameters should be. Adopting the following typical parameters for extreme storm conditions on the Australian south-eastern seaboard:

H = 10 m
T = 12 s

for the median grain size of 375 µm, as tested for the Bondi Beach sand (PWD 1988), the outer depth for cross-shore transport was calculated to be 26 m.

Hallermeier (1981, 1983) defined a simple zonation of an onshore-offshore beach profile consisting of a *littoral* zone, *shoal* zone or buffer zone, and offshore zone where surface wave effects on the bed are negligible.

Based on an analytical approach, supported by laboratory data and some field data, the two water depths bounding the shoal zone, defined by d_s and d_o are given by:

$$d_s = \frac{2.9H}{(S-1)^{0.5}} - \frac{110H^2}{[(S-1)gT^2]}$$

where
- d_s = water depth bounding the littoral and shoal zones
- H = significant wave height *exceeded 12 hours per year*
- T = associated wave period
- S = specific gravity of the sediment, and
- g = acceleration due to gravity.

and

$$d_o = 0.018 H_{med} T_{med} \left[\frac{g}{(S-1)D_{50}} \right]^{0.5}$$

where d_o is the depth at the boundary of the offshore zone, *H* and *T* are the median *significant* wave height and period parameters and D_{50} is the median grain size.

Statistics from the deepwater Waverider buoy at Botany Bay give the 12 hr/a *significant* wave height H_s = 6.3 m with an associated wave period of T_p = 9.7 s. For these parameters, the water depth bounding the littoral and shoal zones was calculated to be 11.5 m.

To determine the seaward depth limit of the shoal zone, for H_s = 1.5 m, T = 8 s and D_{50} = 0.000375, d_o was calculated to be 27.2 m.

For Bondi Beach, Bronte and Tamarama the nearshore beach slope was found to be about 1V:50H to the limits of littoral drift transport (from around 12 m to 27 m). For the beach sand median grain size of 0.375 mm, the Dean (1987) equilibrium beach slope to the water depth of around 25 m was calculated to be 1V:80H. Therefore, as the actual beach slope to the 27 m water depth, as measured from the charts, was steeper than the calculated equilibrium water depth for the tested beach grain sizes, the measured slope was adopted for the Bruun Rule recession assessment.

Beach recession resulting from sea level rise has been calculated at to be approximately 20m (0.4/(1/50)) for 2050 and 45m (0.9/(1/50)) for 2100 for the Waverley beaches.

4.2.3 Bondi

Three profiles have been assessed (for storm erosion and shoreline recession due to sea level rise) at Bondi, one to the south, one in the middle and one to the north. These sections align with the previous photogrammetry (PWD 1988) as shown in **Figure 4.2**.

The impacts of sand loss in front of the seawall and the impact of the wall stability are discussed further in **Section 4.6**.

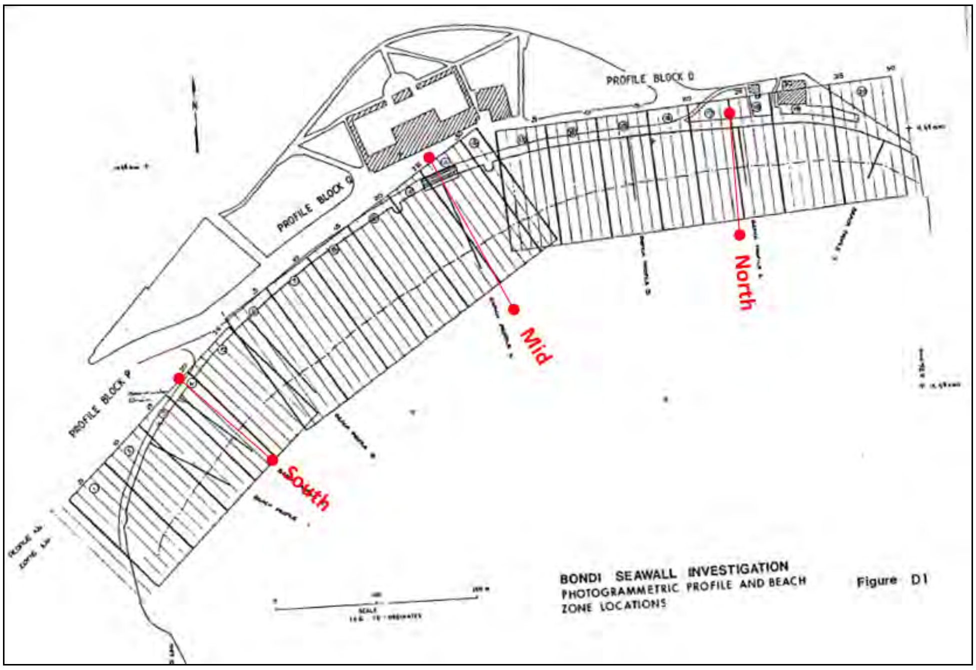

Figure 4.2 Bondi Section Locations, in Relation to PWD 1988 Photogrammetry

South

Figure 4.3 shows the PWD (1988) photogrammetric profiles and also the current profile (from the 2005 ALS data) and an estimate of the 2050 and 2100 profiles (allowing for recession due to sea level rise). The 2005 profile (taken as the immediate profile) shows that the volume of sand in front of the seawall at the south end of the beach is around 300 m^3/m above 0mAHD. This compares with the maximum recorded values presented in PWD (1988) that varied from around 250 m^3/m to 300 m^3/m. For a storm erosion demand of 220 m^3/m, there would still be sand in front of the seawall following severe storm erosion.

In 2050, with a rise in sea level of 0.4 m, the water line would recede some 20 m but the profile would rise by 0.4 m. The volume of sand in front of the wall and above 0.4 m AHD could be around 200 m^3/m. For a storm erosion demand of 220 m^3/m, sand would be eroded back to the seawall causing a risk to the stability of the seawall.

In 2100, with a rise in sea level of 0.9 m, the water line would recede some 45 m from its present day position but the profile would rise by 0.9 m. The volume of sand in front of the wall and above 0.9 m AHD would be 120 m^3/m. For a storm erosion demand of 220 m^3/m, sand would be eroded back to the seawall causing a risk to the stability of the seawall.

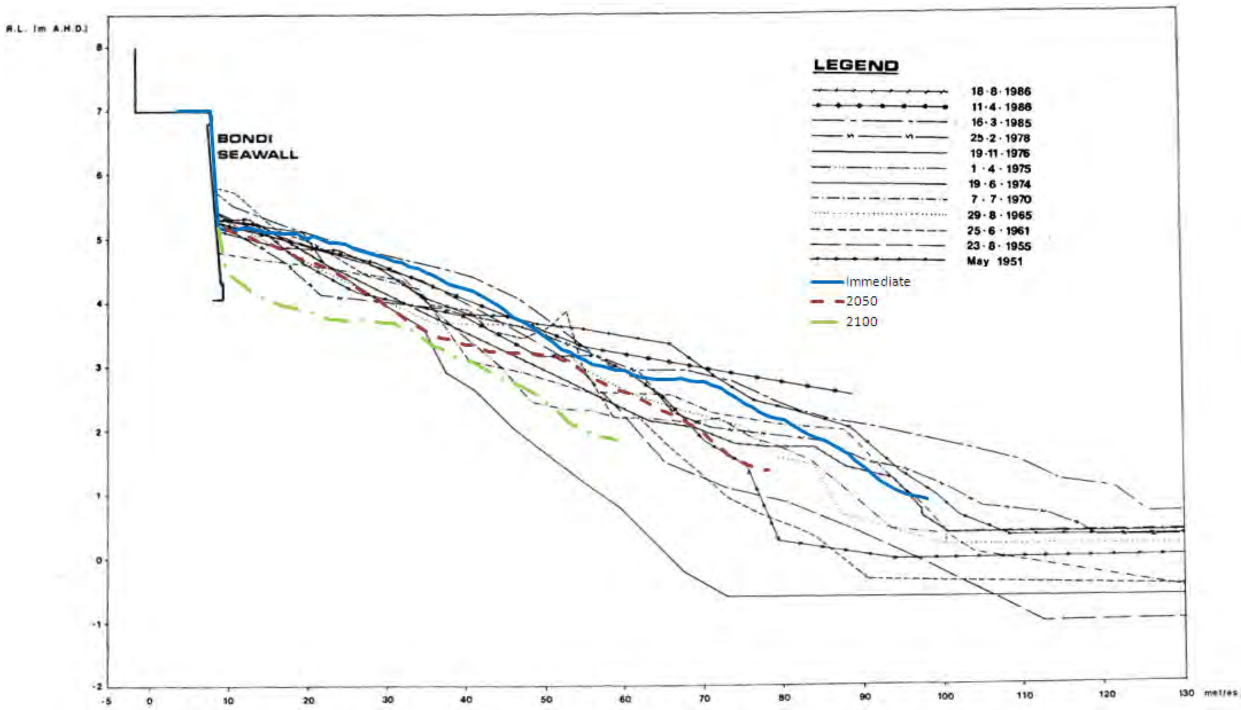

Figure 4.3 Historical and predicted profiles for the south of Bondi

Mid

Figure 4.4 shows the PWD (1988) photogrammetric profiles along with the current profile (from the 2005 ALS data) and an estimate of the 2050 and 2100 profiles (allowing for recession due to sea level rise). The 2005 profile (taken as the immediate profile) shows that the volume of sand in front of the seawall at the centre of the beach is around 300 m^3/m above 0mAHD.This compares with the maximum recorded values presented in PWD (1988) that varied from around 100 m^3/m to 150 m^3/m. For a storm erosion demand of 110 m^3/m there would still be sand in front of the seawall following severe storm erosion.

In 2050, with a rise in sea level of 0.4 m, the water line would recede some 20 m but the profile would rise by 0.4 m. The volume of sand in front of the wall and above 0.4 m AHD could be around 200 m^3/m. For a storm erosion demand of 110m^3/m, there would still be sand in front of the seawall following severe storm erosion.

In 2100, with a rise in sea level of 0.9 m, the water line would recede some 45 m from its present day position but the profile would rise by 0.9 m. The volume of sand in front of the wall and above

0.9 m AHD could be around 120 m^3/m. For a storm erosion demand of 110 m^3/m, sand would be little sand in front of the seawall causing a risk to the stability of the seawall.

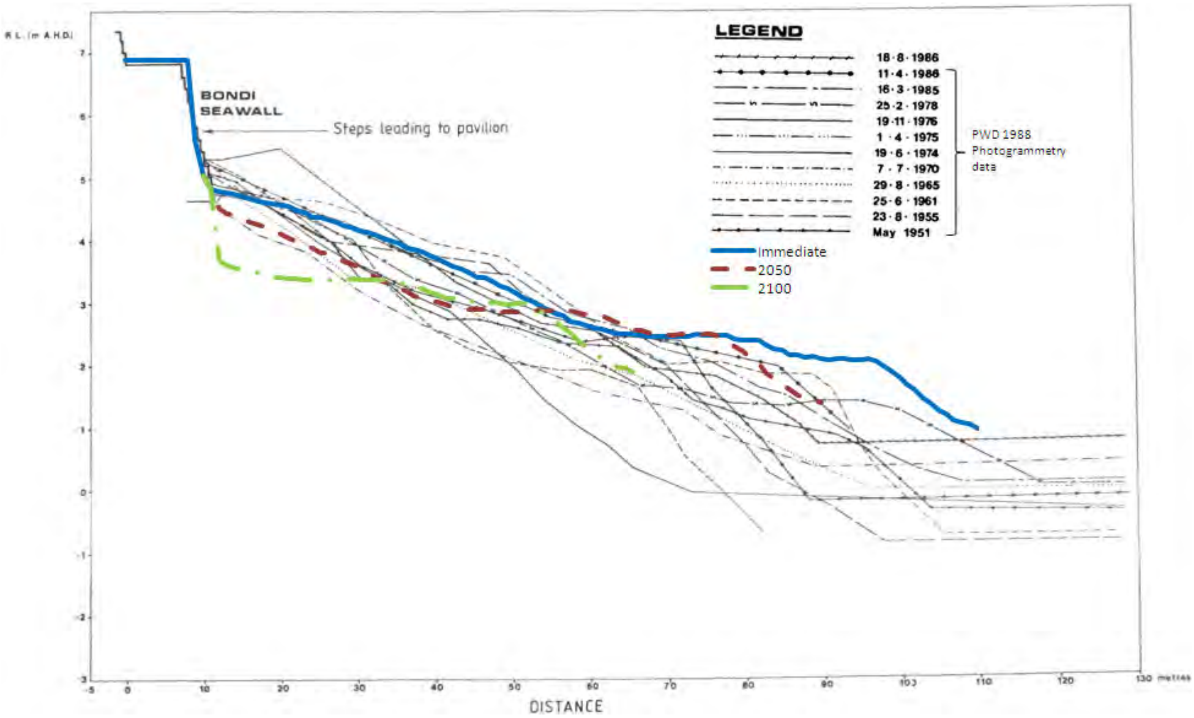

Figure 4.4 Historical and predicted profiles for the centre of Bondi

North

Figure 4.4 shows the PWD (1988) photogrammetric profiles and also the current profile (from the 2005 ALS data) and an estimate of the 2050 and 2100 profiles (allowing for recession due to sea level rise). The 2005 profile (taken as the Immediate profile) shows that the volume of sand in front of the seawall at the north of the beach is around 140 m^3/m above 0 m AHD. This compares with the maximum recorded values presented in PWD (1988) that varied from around 100 m^3/m to 150 m^3/m. For a storm erosion demand of 70 m^3/m there would still be sand in front of the seawall following severe storm erosion.

In 2050, with a rise in sea level of 0.4m, the water line would recede some 20 m but the profile would rise by 0.4 m. The volume of sand in front of the wall and above 0.4 m AHD would be 70 m^3/m. For a storm erosion demand of 70 m^3/m, sand would be eroded back to the seawall causing a risk to the stability of the seawall.

In 2100, with a rise in sea level of 0.9 m, the water line would recede some 45 m from its present day position but the profile would rise by 0.9 m. The volume of sand in front of the wall and above 0.9 m AHD would be 20 m^3/m. For a storm erosion demand of 70 m^3/m, sand would be eroded back to the seawall causing a risk to the stability of the seawall.

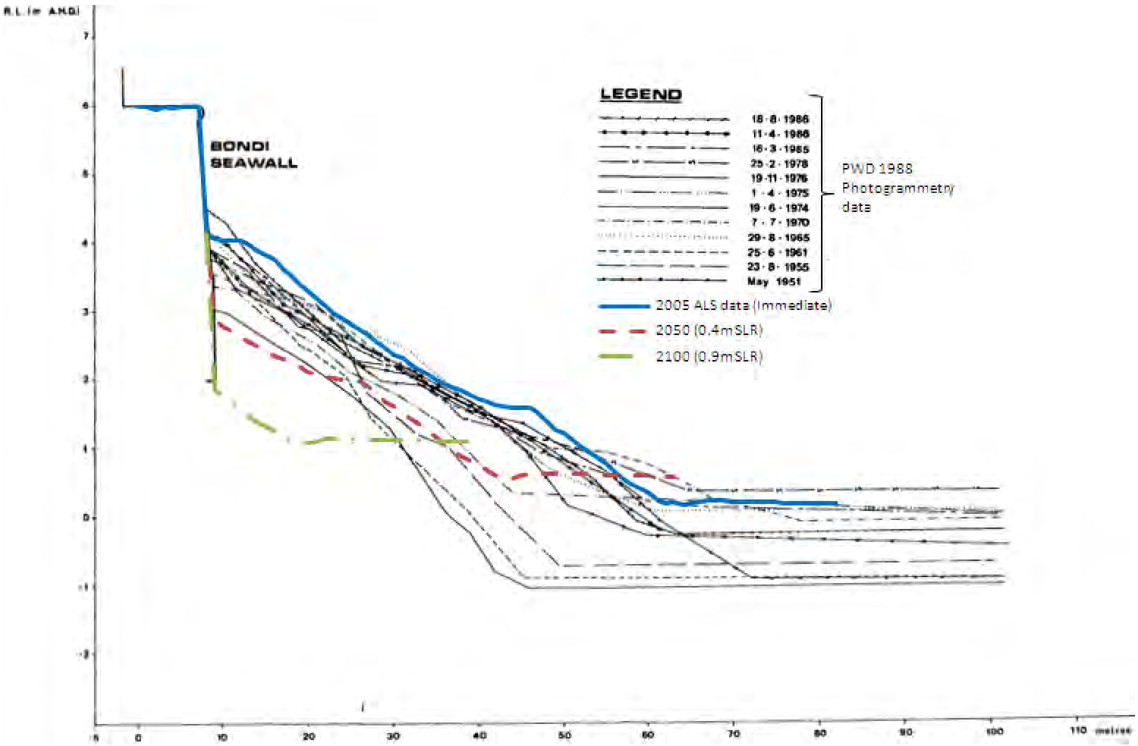

Figure 4.5 Representative profiles for the north of Bondi at present, 2050 and in 2100

4.2.4 Tamarama Beach

Figure 4.6 shows the current profile (from the 2005 ALS data) and an estimate of the 2050 and 2100 profiles (allowing for recession due to sea level rise). The 2005 profile (taken as the immediate profile) shows that the volume of sand in front of the seawall, at the northern end of the beach, is around 220 m^3/m above 0 m AHD. For a storm erosion demand of 250 m^3/m, sand would be eroded back to the seawall causing a risk to the stability of the seawall.

In 2050, with a projected rise in sea level of 0.4 m, the water line would recede some 20 m but the profile would rise by 0.4 m. The volume of sand in front of the wall and above 0.4 m AHD could be around 220 m^3/m. For a storm erosion demand of 250 m^3/m, sand could be eroded back to the seawall causing a risk to its stability.

In 2100, with a rise in sea level of 0.9 m, the water line would recede some 45 m from its present day position but the profile would rise by 0.9 m. The volume of sand in front of the wall and above 0.9 m AHD could be around 170 m^3/m. For a storm erosion demand of 250 m^3/m, sand would be eroded back to the seawall causing a risk to the stability of the seawall.

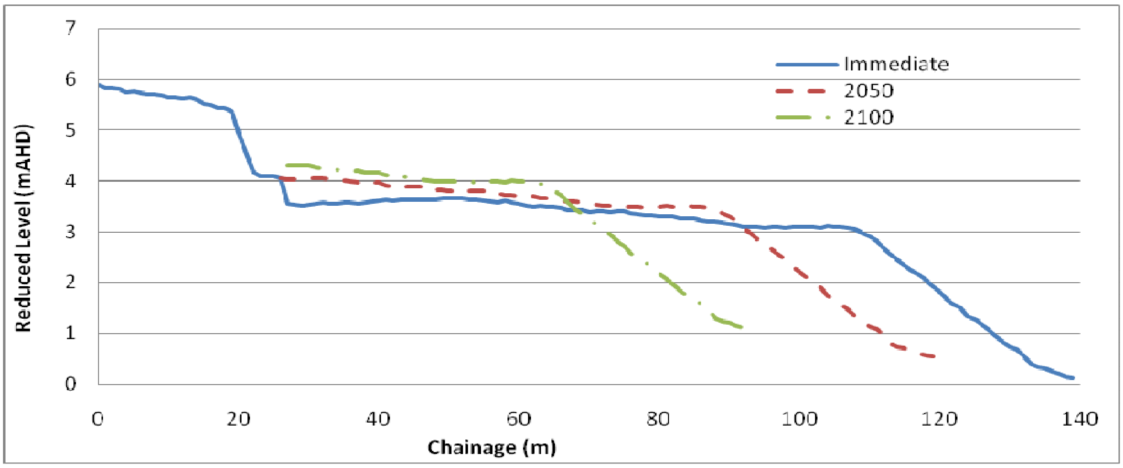

Figure 4.6 Representative profiles for the centre of Tamarama at present, 2050 and in 2100

4.2.5 Bronte Beach

Figure 4.7 shows the current profile (from the 2005 ALS data) and an estimate of the 2050 and 2100 profiles (allowing for recession due to sea level rise). The 2005 profile (taken as the immediate profile) shows that the volume of sand in front of the seawall at the north of the beach is around 210 m^3/m above 0 m AHD. For a storm erosion demand of 250 m^3/m, sand would be eroded back to the seawall causing a risk to the stability of the seawall.

In 2050, with a projected rise in sea level of 0.4 m, the water line would recede some 20 m but the profile would rise by 0.4 m. The volume of sand in front of the wall and above 0m AHD could be around 180 m^3/m. For a storm erosion demand of 250 m^3/m, sand could be eroded back to the seawall causing a risk to the stability of the seawall.

In 2100, with a projected rise in sea level of 0.9 m, the water line would recede some 45 m from its present day position but the profile would rise by 0.9 m. The volume of sand in front of the wall and above 0 m AHD could be around 160 m^3/m. For a storm erosion demand of 250 m^3/m, sand could be eroded back to the seawall causing a risk to the stability of the seawall.

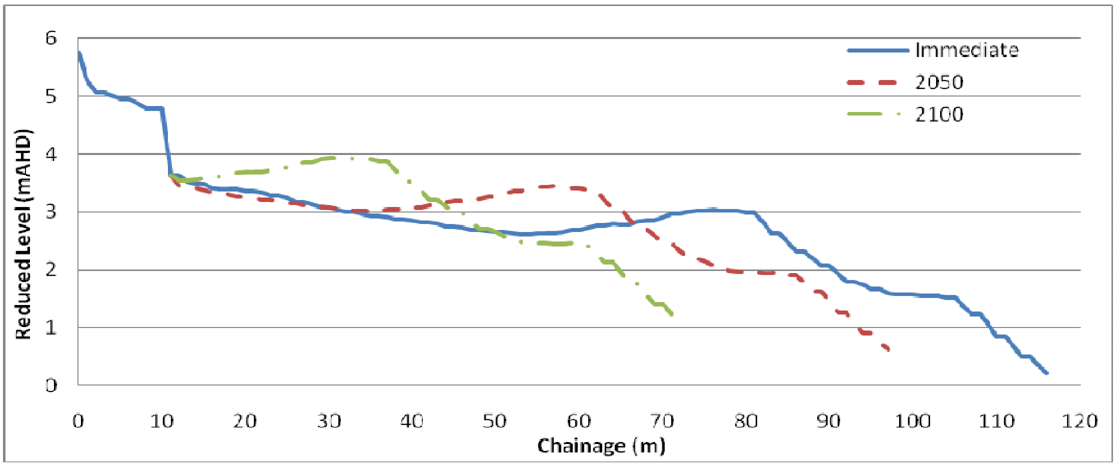

Figure 4.7 Representative profiles for the centre of Bronte at present, 2050 and in 2100

4.3 Sand Drift Hazard

As noted in **Section 3.4.3**, 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 seawalls. 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. Aeolian sand loss is considered to be only of nuisance level and not a significant hazard within the study beaches. It is currently managed by Council through localised beach scraping, when sand levels are high against the seawall, and routine cleaning of GPTs.

4.4 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 3.1.2** namely storm surge (wind setup and the barometric setup) and wave setup. This increased water level may persist for several hours to days. 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 3.6**) would also be included. For the mid and high range sea level rise scenarios, this would bring the total elevated still water level over 100 years to 3.1m and 3.6m respectively. The seawalls at Bondi, Bronte and Tamarama are all above 3.6mAHD, and therefore the areas landward of the seawalls would not be inundated by still water levels alone.

A number of ocean front properties on Ramsgate Avenue could be inundated due to still water levels and wave runup (refer **Figure 4.8).** Levels of inundation have been calculated to be 4.4 m AHD for

2011, 5.4 m AHD for 2050 and 6.6 m AHD for 2100. However, inundation to the buildings is subject to individual floor levels, which could be well above inundation limits.

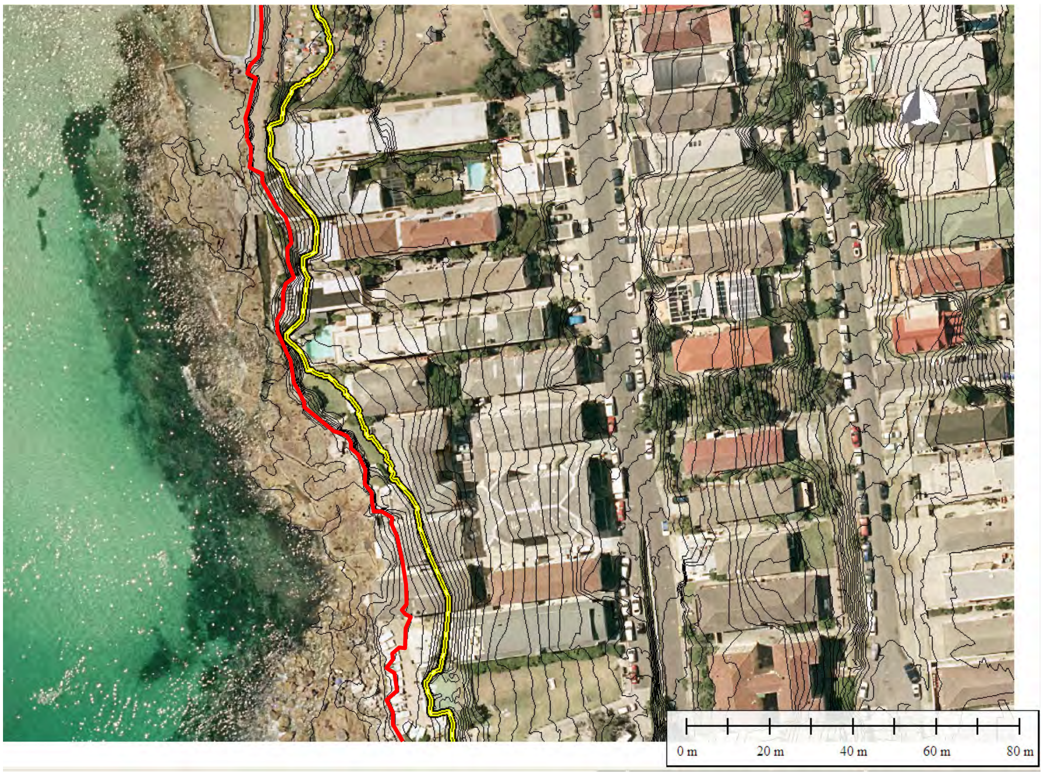

Figure 4.8 Aerial Photograph of Foreshore Properties at Ramsgate Avenue (red line indicates 3mAHD and yellow line indicates 6mAHD, ground levels)

During storm events, individual waves result in further temporary water level increases above the still water level due to the process of wave setup and runup and overtopping (**Section 3.5**). At Bondi Bronte and Tamarama extreme runup heights would not be reached as, instead, waves would overtop the seawall. Therefore, overtopping is discussed for this study in addition to wave runup.

If waves overtopped the seawalls the water would 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 10 m of the foreshore crest. In addition, overtopping is generally episodic, occurring around the peak of the high tide. **Table 4-2** lists the buildings which are along the foreshore of the study beaches and comments on their general potential to be impacted by inundation in an extreme event. The affected areas would become more vulnerable to inundation in the longer term as beach recession occurs and sea level rises. Recommendations for inundation hazard zones and management measures are discussed in **Section 6.2**.

Table 4-2 Ocean Front Properties

Properties	Ground Level[1]	Set Back[2]	Comments[3]
77,79-81, 83, 85, 95, 97, 105, 107 and 111 Ramsgate Avenue	1-3 m AHD	no seawall	The frontages of these properties are not protected by a seawall and are at low levels, therefore are at risk of inundation. Levels of inundation have been calculated to be 4.4 m AHD for 2011, 5.4 m AHD for 2050 and 6.6 m AHD for 2100. However, inundation to the buildings is subject to individual floor levels which could be well above inundation limits.
North Bondi Surf Lifesaving club	6 m AHD	15m	Due to the setback distance, quantities of overtopped water reaching the site would be low.
North Bondi public Amenities	6.5 m AHD	25m	Due to the setback distance, quantities of overtopped water reaching the site would be low if at all.
Bondi Life guard tower	6m AHD	seaward of seawall	There is a risk of inundation; the extent is dependent on the floor levels which are unknown.
Bondi Pavilion and Surf club	8-8.5 m AHD	35m	Due to the setback distance and ground level, overtopped water is unlikely to reach these buildings.
Bondi Icebergs	Café 6 m AHD Ground Floor 8-9 m AHD	no seawall	There is a risk of inundation, however this is reduced as the swimming pool and rock platforms immediately seaward would reduce wave energy before reaching the buildings.
Tamarama Life Guard Tower	-	no seawall	This structure is built on piles with a high floor level. Inundation risk is dependent on these floor levels which are not known, although due to its position it is likely to be subject to ocean spray.
Tamarama Café/Kiosk	5 m AHD	8 m	Due to the low ground level there is a risk of inundation. However due to the set back distance the majority of overtopped water would be shallow sheet flow and the quantity of water reaching the buildings would depend on how the water drained.
Tamarama Public Amenities	6.5 m AHD	18 m	Due to the setback distance, quantities of overtopped water reaching the site would be low.
Bronte Surf Life Saving Club	5.5 m AHD	7 m	Due to the set back distance the majority of overtopped water would be shallow sheet flow. Although the ground level in front of the building is low, the floor levels are raised (stepped access) therefore significantly reducing inundation risk.
Bronte kiosk and public amenities	5 m AHD	20 m	Due to the setback distance the risk of inundation is low, as the overtopped water is likely to have dissipated prior to reaching the building.
Bronte south public amenities	4.5 m AHD	10 m	Due to the low ground level there is a risk of inundation. However the building is set back a significant distance, there is a rocky foreshore in front, and the building floor levels could be above this ground level, therefore inundation risk is reduced.

[1] Ground level is taken immediately seaward of the property based on 2005 ALS data, building floor levels are not known and may be significantly higher

[2] Set back is the approximate distance from the seawall to the seaward edge of the building, estimated from aerial photography

[3] Comments provide general discussion on inundation for an extreme event in 2011

In addition to the inundation of buildings, there is a potential for risk to the safety of pedestrians, vehicles and structures. **Figure 4.10** shows critical overtopping values in relation to damage and risk, suggesting that average values over 0.3 l/s/m are dangerous for pedestrians and values over 50 l/s/m could cause damage to the seawall. To give a general indication of the likely hood of this risk a number of overtopping calculations (applying algorithms from EurOtop, 2007, Wave Overtopping of Sea Defences and Related Structures Assessment Manual) were undertaken. These calculations are intended as an indicative risk assessment and are not suggested as explicit values.

A typical section in the middle of Bondi Beach was assessed for overtopping, with crest level taken as 6.8 m AHD, and the beach level at the toe as the 1974 scour levels. The wave conditions adopted were those for the 6 hour duration wave height that had a 5% risk of being exceeded over the next 50 years. The waves applied were depth limited to the scoured beach level, and a range of wave periods were applied. That is to say that the overtopping estimates relate to extreme wave conditions applied to an eroded beach state. **Figure 4.9** shows a summary of these results for various water levels. It can be seen that for an immediate 100 yr ARI still water level the overtopping value is well over the 0.3 l/s/m suggested as dangerous for pedestrians and for 2050 and 2100 (100 yr still water level) the overtopping increases. As the seawall crests at Bronte and Tamarama are lower than at Bondi, it would be expected that the overtopping would be greater. Recommendations for management measures are discussed in **Section 4.7**.

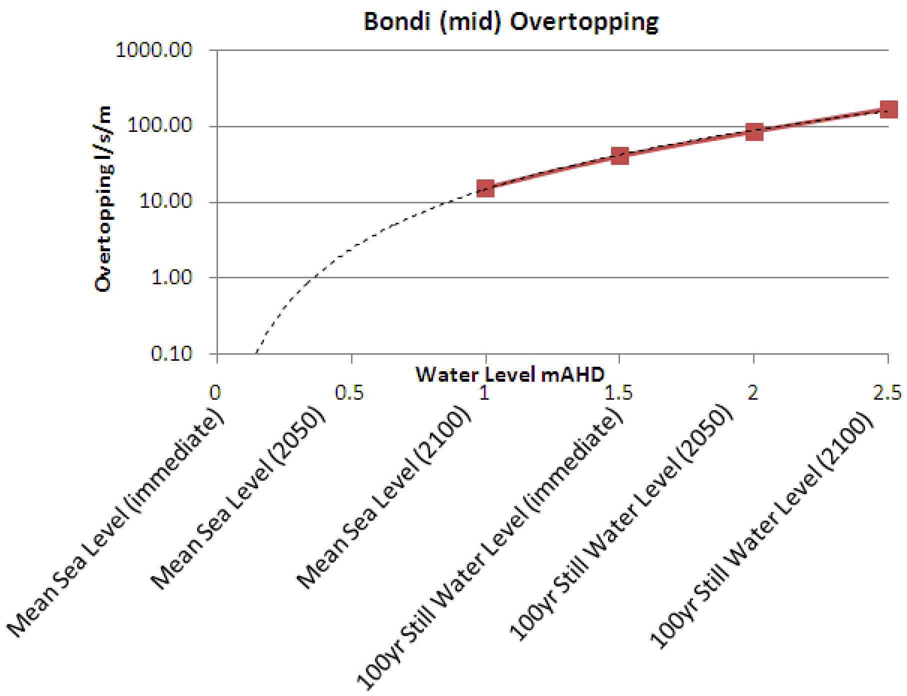

Figure 4.9 Potential Average Overtopping Values for Typical Section in the middle of Bondi, for Extreme Scour and a Range of Water Levels (dashed line in an estimated trend line)

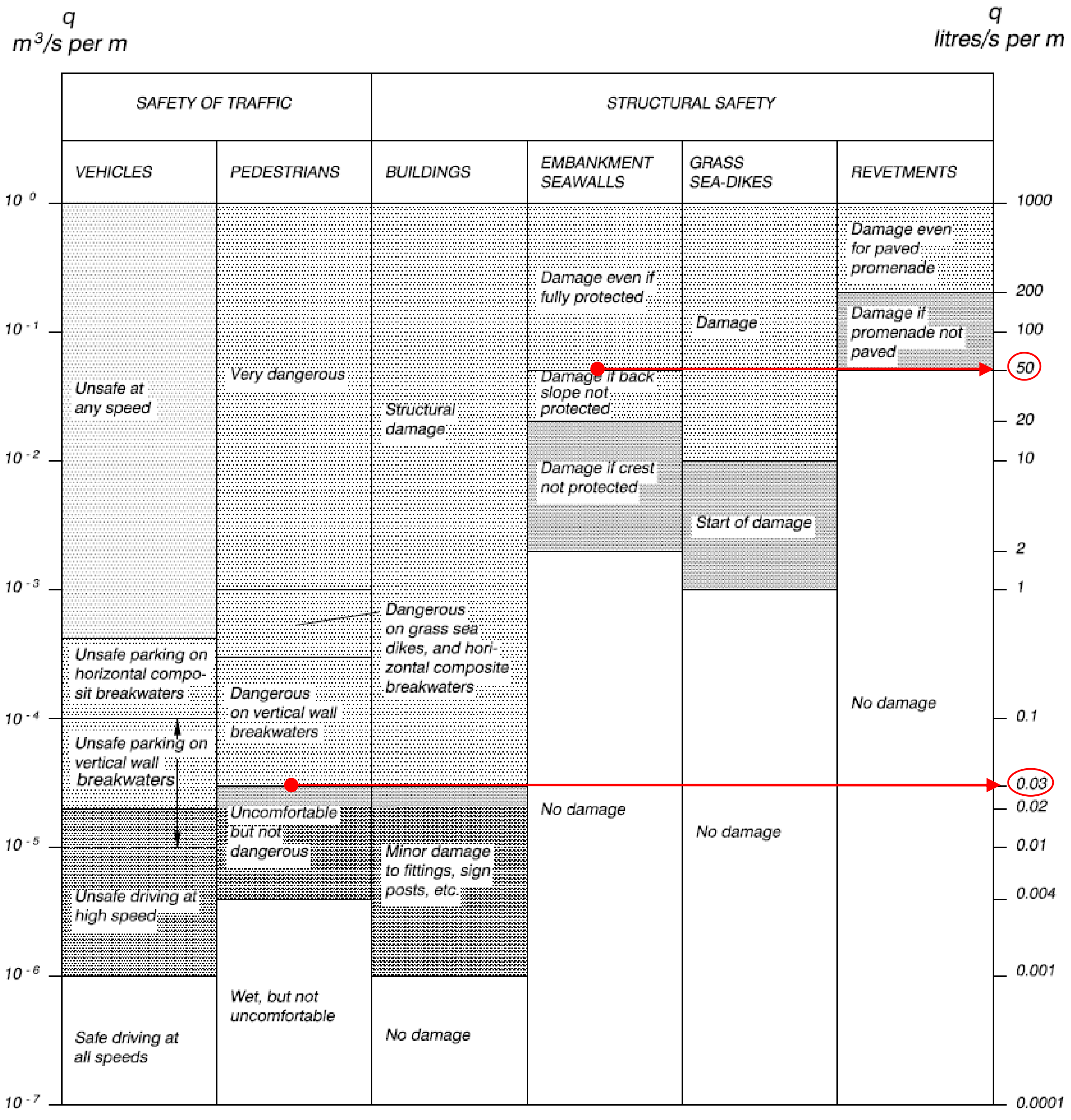

Figure 4.10 Critical Values of Average Overtopping Discharges (Source: CEM)

4.5 Stormwater Hazard

4.5.1 Erosion Hazard

Since the construction of the current stormwater systems, erosion hazard from the outfalls is not considered to be a significant hazard at Bondi, Bronte and Tamarama. Localised surface water runoff from storms of intense rainfall can cause localised flows down access ramps and steps and over the crest of the seawall, which can cause localised scour along the interface with the beach.

4.5.2 Impacts of Sea level Rise

Elevated ocean levels have the potential to increase tail water levels for stormwater drainage, thereby affecting flooding. There are two areas behind Bondi Beach that drain to the ocean there. These are the natural detention basins No. 8 and 10 as shown in **Figure 4.11.** The question has been raised as to whether sea level rise would affect the discharge of stormwater at these two locations and, in particular, the level of flooding experienced at Basin 10.

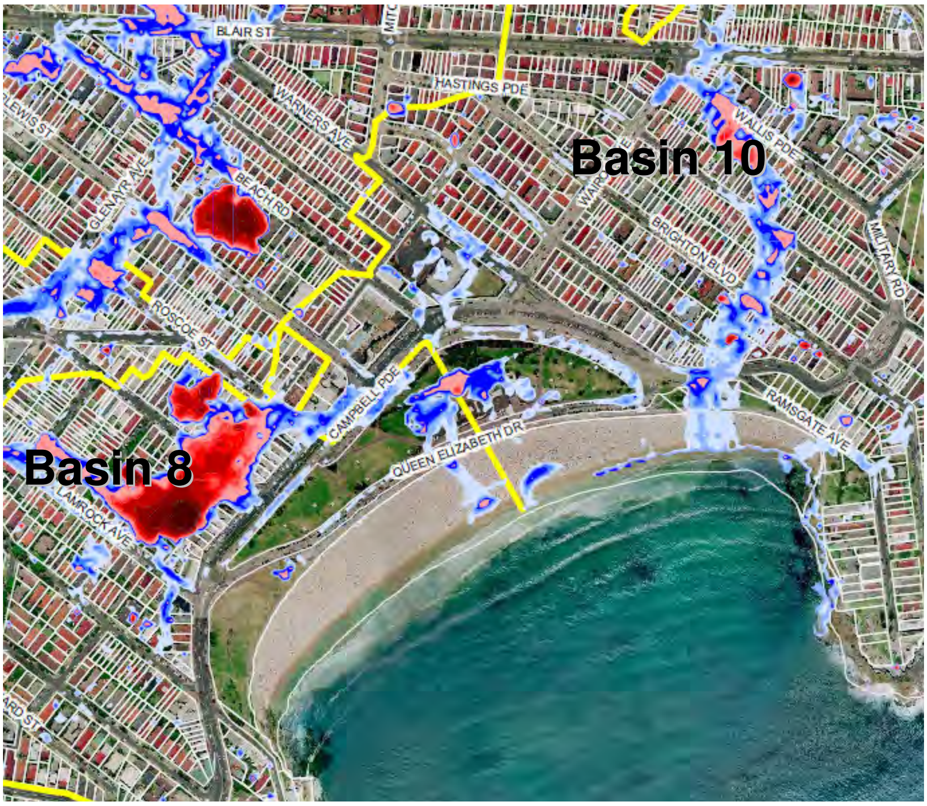

Figure 4.11 Natural detention basins behind Bondi Beach and approximate 100 year ARI flow paths that drain to the coast. (Source: Civic Design, 2007 – see notes on Civic design Map No. B4)

BASIN 10

The piped stormwater drainage system from Basin 10 is some 500 m long. The drainage is constricted to relatively low flows of around 1 m^3/s (**Figure 4.12**), which are insufficient to convey even the 1 year ARI event. Overland flows from Basin 10 range from some 4-5 m^3/s for the 1 year ARI to reach some 10-15 m^3/s for the 100 year ARI events (**Figure 4.13**). The overland flows spill into the ocean over the seawall edge whereas the pipe flow debauches through a 2.44 m × 1.675 m box culvert with an invert at 0.68 m AHD at the northern end of the beach.

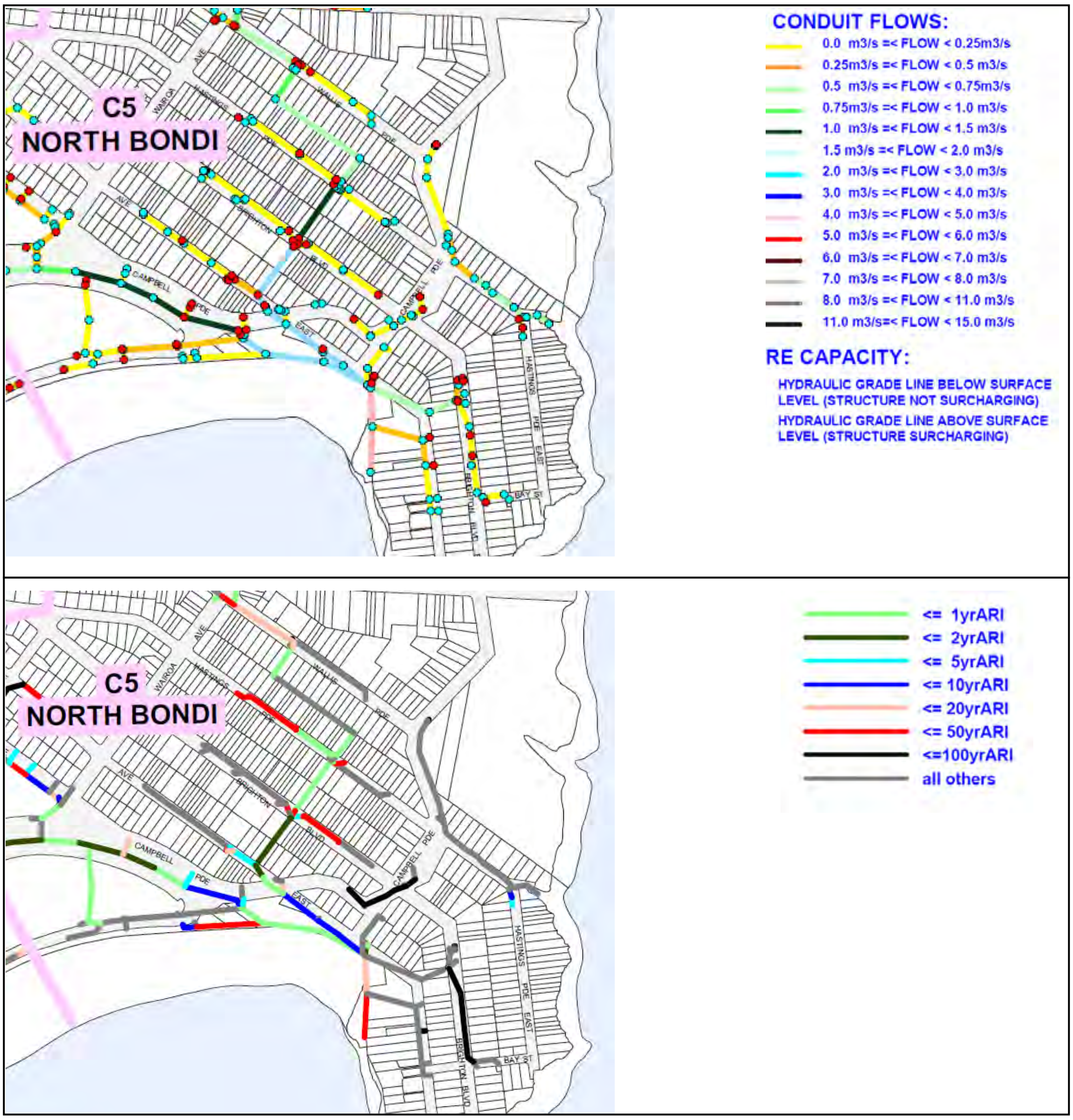

Figure 4.12 Drainage Capacity of conduits from Basin 10 (Source: Civic Design, 2007)

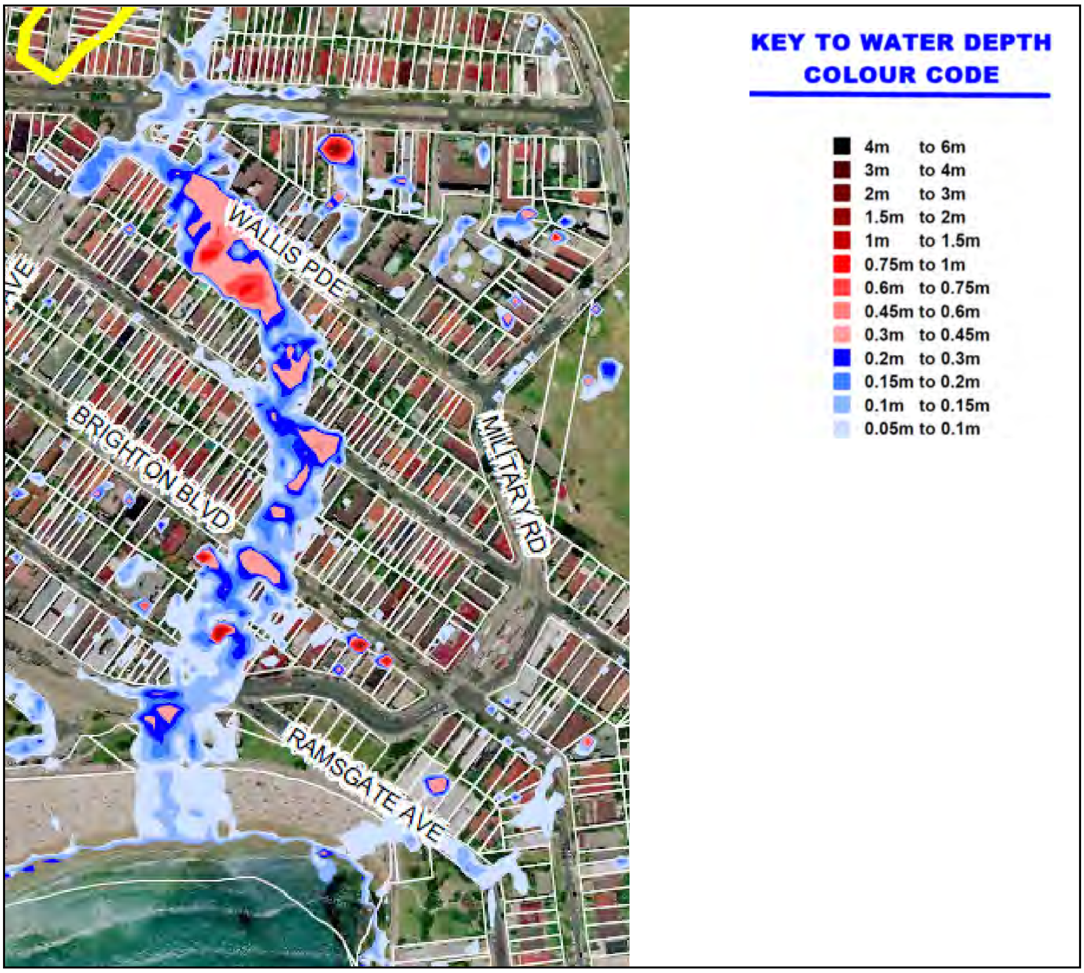

Figure 4.13 Approximate modelling of Overland Flows in 100 year ARI event Source: Civic Design, 2007)

The water level in basin 10 during a 1 year ARI event is around 16.4 m AHD. Under present day conditions, the still water level on the beach during an extreme storm could reach around 2.5 m AHD, which may persist for an hour or two on the top of the tide, flooding the obvert of the box culvert (RL 2.36 m AHD). With climate change sea level rise, this level is projected to increase to 2.9 m AHD for the year 2050 and 3.4 m AHD for the year 2100.

If it is assumed that the water level in the detention basin is controlled by the characteristics of the overland flow and the storm water pipes were flowing full, the pressure in the pipes for the 1 year ARI event could be reduced from around 13.9 m (16.4 m – 2.5 m) to around 13.5 m (16.4 m – 2.9 m) in 2050 and 13 m (16.4 m – 3.4 m) in 2100.

The discharge through the piped stormwater drainage system flowing full will be directly proportional to the square root of the applied head. If the pipelines are all flowing full, the percentage reduction in discharge capacity of the pipes would be 1.5% (√(13.5/13.9)) in the year 2050 to 3.3% (√(13.0/13.9)) in 2100.

The sea level rise would have no impact of the drainage of the overland flow. As the capacity of the storm water system draining Detention Basin 10 is small compared with the rates of overland flows, the impact of the sea level rise on flooding in Detention Basin 10 would be very small if measureable.

BASIN 8

Basin 8 drains through the stormwater system the debauches at the southern end of the beach. The invert level is 0.31 m AHD. The stormwater drainage system has a high capacity and under present day conditions there is little overland flow for events up to the 100 year ARI event (**Figure 4.14** and **Figure Figure** 4.15).

Under present day conditions, the still water level on the beach during an extreme storm could reach around 2.5 m AHD, which may persist for an hour or two on the top of the tide, flooding the obvert of the box culvert (RL 2.14 m AHD). With climate change sea level rise, this level is projected to increase to 2.9 m AHD for the year 2050 and 3.4 m AHD for the year 2100.

The length of the piped storm water drain from Basin 8 is some 500 m and the water surface elevation in the basin varies from some 14.1 m during frequent events to some 15.5 m for extreme events (Civic Design, 2007).

If it is assumed that the storm water pipes were flowing full, the pressure in the pipes for the 1 year ARI event could be reduced from around 11.6 m (14.1 m – 2.5 m) to around 11.2 m (14.1 m – 2.9 m) in 2050 and 10.7 m (14.1 m - 3.4 m) in 2100.

The discharge through the piped stormwater drainage system will be directly proportional to the square root of the applied head. If the pipelines are all flowing full, the % reduction in discharge capacity of the pipes would be 1.8% (√(11.2/11.6)) in the year 2050 to 4.0% (√(10.7/11.2)) in 2100.

These estimates are likely to be conservative as it is unlikely that the culvert would ever be flowing full. The impact of the sea level rise on flooding in Detention Basin 8 would be very small.

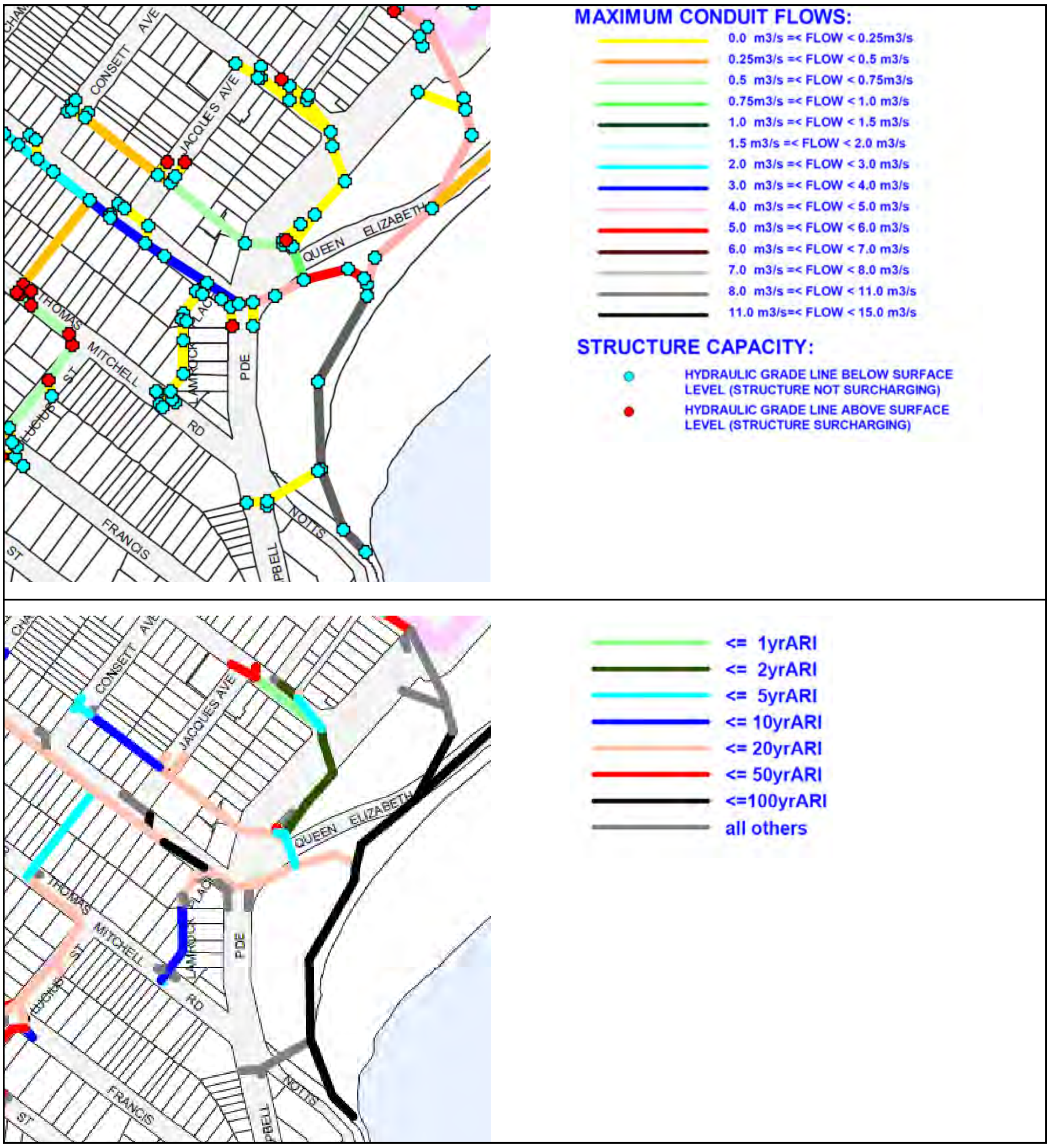

Figure 4.14 Drainage capacity of conduits from Basin 8 (Source: Civic Design, 2007)

Figure 4.15 Approximate Extent of Overland flows in 100 year ARI Event (Source: Civic Design, 2007)

4.6 Beachfront Stability Hazard

For beachfront areas on natural dunes a number of coastline hazard zones can be delineated, based on Nielsen *et al.* (1992), as shown in (**Figure 4.16**).

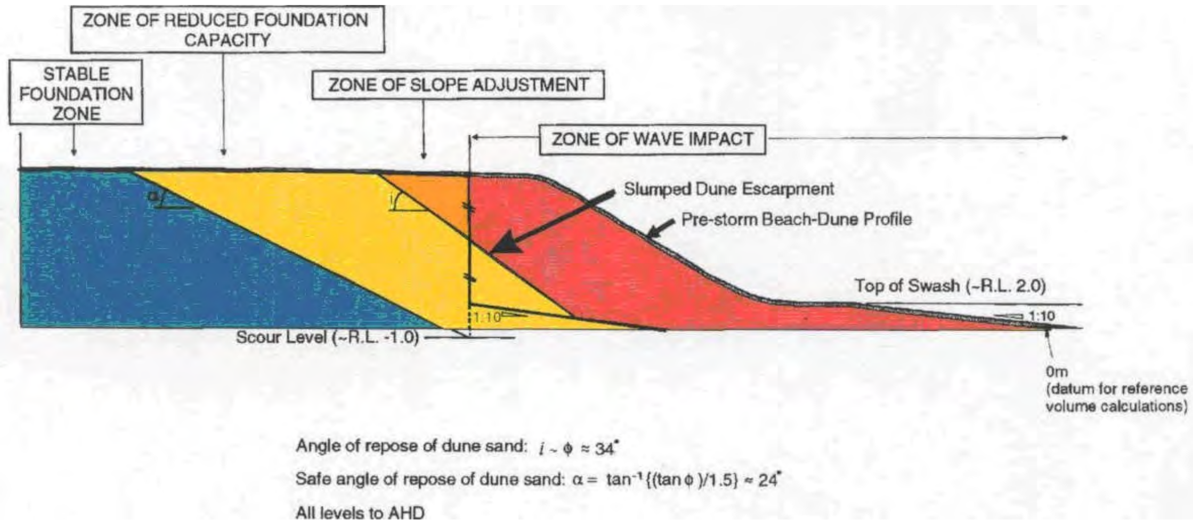

Figure 4.16 Dune Stability Schema (after Nielsen *et al.*, 1992)

The *Zone of Wave Impact* delineates an area where any structure or its foundations would suffer direct wave attack during a severe coastal storm. It is that part of the beach that is seaward of the beach erosion escarpment

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 be transmitted only to 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.

For beaches backed by competent seawalls, the *Zone of Wave Impact* and the *Zone of Slope Adjustment* are defined by the seawall, which doesn't change with time and climate change. However, the global stability of a seawall may be compromised during episodes of extreme scour, which could change with time, resulting in a *Zone of Reduced Foundation Capacity* over an area immediately behind the seawall.

4.6.1 Existing Conditions

PWD (1988) presented a detailed global stability analysis of the Bondi seawall. It was based on a comprehensive site investigation of the foundation and backfill materials and was carried out for each of the different seawall sections.

The stability analysis showed that under fair weather conditions the seawall had high factors of safety (FoS) against collapse (FoS~3 to 4). However, the FoS reduced once the beach level was scoured.

For the scour levels observed following the 1974 storms, the stability analysis undertaken by Public Works (PWD 1988) indicated that the seawall south of Queen Elizabeth Drive had an inadequate FoS and was in danger of collapse. However, all other sections of the wall had an adequate FoS (around 2 to 3), even with elevated water levels behind the seawall. PWD (1988) advised that acceptable factors of safety were determined for the seawall supported by the Reno-mattress revetment if the beach was eroded to the revetment toe.

The Zone of Reduced Foundation Capacity behind the seawall south of Queen Elizabeth Drive was defined by a wedge failure plane in loose sand that had a FoS = 1.5 with scour to RL 0 m AHD, being defined be a slope of 2H:1V. This zone extended some 14 m beyond the crest of the seawall.

No detailed stability analyses have been undertaken for the seawalls at Bronte and Tamarama. While a superficial inspection has not raised any concerns, it is recommended that in due course a stability study be undertaken for these structures.

4.6.2 Year 2050

Taking account of a projected sea level rise, the volume of sand in front of the seawalls would reduce with time. This could lead to increased scour levels in front of the seawalls and, hence, reduced FoS for global stability.

By 2050, the volume of sand in front of the Bondi seawall has been projected to have been reduced by some 50 m^3/m from present day conditions. This is not enough to compromise the stability of the seawall. Nevertheless, the seawall south of Queen Elizabeth Drive would have still an inadequate FoS and be in danger of collapse during a severe storm. Hence, the Zone of Reduced Foundation Capacity behind the seawall south of Queen Elizabeth Drive would extend some 14 m beyond the crest of the seawall.

For the Tamaramma and Bronte seawalls there will be a similar increase in risk with time and we recommended defining a Zone of Reduced Foundation Capacity of 10 m and 8 m (respectively) behind these seawalls.

4.6.3 Year 2100

By 2100, the volume of sand in front of the seawall has been projected to have been reduced by some 100 m^3/m from present day conditions. Under these conditions, scour to the base of the seawall along its entire length can be expected.

In this case, Factors of safety would be reduced to around 1.0, which is inadequate for a public space facility. The Zone of Reduced Foundation Capacity would extend, generally, some 14 m beyond the crest of the seawall.

Again for the Tamaramma and Bronte seawalls there will be a similar increase in risk with time and we recommended defining a Zone of Reduced Foundation Capacity of 10 m and 8 m (respectively) behind these seawalls.

4.6.4 Effect of the Reno-Mattress Toe Protection

The Reno-mattress toe protection along the Bondi seawall was designed to:

- prevent undermining of the seawall
- prevent sand from leaching out from underneath and behind the wall at times of extreme beach scour and through groundwater flow, thereby obviating the undermining of the pavement
- enhance the global stability of the structure during times of extreme beach scour.

The failure analyses presented in PWD (1988) showed clearly the reduction in the global stability of the seawall that can occur by beach lowering during storms, in many cases to unacceptable levels, and that acceptable factors of safety against collapse of the seawall were determined for the seawall supported with the Reno-mattress revetment.

According to PWD (`1988), the Reno-mattress is integral to the stability of the seawall.

4.6.5 Seawall Durability

PWD (1988) presented a thorough investigation of the concrete elements of the Bondi seawall. The seawall comprised two different types of section; an unreinforced concrete gravity wall and a reinforced concrete counterfort wall.

Generally, the site inspection described some cracking and some minor steel corrosion. Reinforcement comprised 10 mm diameter steel bars and cover to reinforcement varied from around 30 mm to 50 mm. The surface of the concrete varied from moderately weathered to significant loss of surficial mortar. Schmidt hammer testing indicated a concrete strength of some 25 MPa to 60 MPa. However, these tests were not considered to be reliable and concrete cores indicated compressive strength of 12 MPa for the gravity wall section, which is considered to be sufficient, and 20 MPa for the reinforced concrete wall sections.

Considering the age of the Bondi seawall structure, the concrete and reinforcement were found to be of surprisingly good quality (PWD, 1988). However, current maritime structures guidelines for reinforced concrete would recommend a minimum bar diameter of 12 mm and minimum cover of 60 mm to 70 mm. Nevertheless, it is considered that the Bondi seawall structure is sound.

While the durability of the Bondi seawall fabric may appear to be in question due to the small reinforcement bar diameter, small cover and low concrete strength, the seawall is not dilapidated and is likely to remain robust for many years hence.

No detailed investigations have been undertaken for the Tamarama and Bronte seawalls. However, they are of a similar vintage and are considered to be in a similar condition to the Bondi seawall. However, it is recommended that the fabric of these seawalls and their structural stability should be investigated.

4.7 Hazard Lines

The immediate hazard line is defined as the “mapped line representing the estimated extent of beach erosion from an extreme oceanic storm event plus any allowance for reduced foundation capacity” (NSW Coastal Planning Guideline: Adapting to Sea Level Rise, NSW Planning 2010). The 2050 and 2100 hazard lines are therefore similar, except that they incorporate any impacts of sea level rise projections.

The hazard lines for Bondi, Bronte and Tamarama have been mapped, figures of which are included in **Appendix G**.

Along the beaches the assumption is that the seawalls will be retained into the future, therefore the erosion hazard limit would be the seawall (i.e. all the hazard lines listed above are in the same location). In line with current policy the reduced foundation capacity landward of the seawall would also be identified. This has been assessed as a distance back from the seawalls, that distance being at a slope of 2H:1V of the retained height of the backfill sand above an extreme scour level of 0 m AHD. That slope provides for a factor of safety for global slope stability of 1.5 for medium sand.

For Bondi the Zone of Reduced Foundation Capacity (and therefore the Coastline Hazard Zone, due to beach erosion and seawall stability) behind the seawall has been estimated to extend some 14 m landward of the crest of the seawall, for the present, 2050 and 2100.

For Bronte, where the seawall is lower, the Zone of Reduced Foundation Capacity (and therefore the Coastline Hazard Zone, due to beach erosion and seawall stability) behind the seawall has been estimated to extend some 8 m landward of the crest of the seawall, for the present, 2050 and 2100.

For Tamarama, the Zone of Reduced Foundation Capacity (and therefore the Coastline Hazard Zone, due to beach erosion and seawall stability) behind the seawall has been estimated to extend some 10 m landward of the crest of the seawall, for the present, 2050 and 2100.

For the cliff areas the hazard line has been identified by Lot instead of a line, cliff hazards are discussed further in **Section 5**. The cliff hazards have been treated differently as the hazard from cliff failure is of a different nature to that of coastal erosion. The exact extent and timing of the cliff failures is difficult to predict and dependent on factors other than coastal processes. An extreme ocean storm my well have no impact on cliff stability. Therefore it is considered more appropriate to identify Lots with the potential for cliff hazards, as a planning control. As the timing of the cliff failures cannot be predicted the Lots identified are for the present, 2050 and 2100 inclusive.

A discussed above a hazard line is defined as the "mapped line representing the estimated extent of beach erosion from an extreme oceanic storm event plus any allowance for reduced foundation capacity". The hazard line therefore does not relate to inundation hazard. Inundation is a temporary process, occurring occasionally and intermittently on a high tide during a storm. It is most often the case that structures can deal with inundation, either being built robustly, t or, simply, by being elevated. For seawalls, a hazard may result from the amount of overtopping discharge, which cannot be defined by a line. For these reasons we have identified lots subject to inundation and inundation levels or discharge values have been given where appropriate. Lots identified are for the present, 2050 and 2100 inclusive.

5. GEOTECHNICAL HAZARD ASSESSMENT

5.1 Geotechnical Hazards

Based on the results of site inspections, detailed in Appendix A, the potential geotechnical hazards for the study area are summarised as:

1. instability of overhang features, wedges or blocks within sandstone bedrock over the following sections of cliff faces:
    a. crest of cliff face
    b. cliff face
    c. base of cliff face
2. instability of natural soil foreshore slopes;
3. instability of fill foreshore slopes;
4. instability of existing stabilisation measures; and
5. instability of retaining structures.

The potential geotechnical hazards 1a to 1c relate to the entire length of cliff faces within the study area and 2 to 5 relate to selected areas of the study area only.

The foreshore cliff faces comprise Hawkesbury Sandstone bedrock of Triassic age (around 245 to 210 million years ago). The sandstone represents vast quantities of sediments transported into the Sydney Basin by rivers flowing from the south-west and west. Uplift and deformation of the Sydney basin area probably occurred over several phases and was associated with the opening of the Tasman Sea approximately 60 to 80 million years ago. The present elevation of the Sydney Basin region was achieved by about the mid Tertiary (about 40 to 50 million years ago). This uplift and deformation has led to the observed pattern of jointing and faulting in the rock mass and the intrusion of igneous dykes generally along the dominant joint planes; typically in an approximately east-west direction but with some trending approximately north-south. Weathering and erosion of the sandstone continued with sea level fluctuations from the early Quaternary onwards (commencing around 1.8 million years ago), associated with glacial and inter-glacial periods (sea level low high periods, respectively), having a significant effect on the formation of the present day coastline.

Current sea levels are believed to have been reached around 6,400 years before present (ybp). A glacial period between about 17,000 and 25,000 ybp is believed to have caused a sea level fall of around 130m below present day levels. At the end of this glacial period ice melted and sea levels rose to their current levels in-filling the valleys that now form the Sydney Harbour foreshore we see today. This cycle of varying sea levels is believed to have occurred several times over during the Quaternary (about 1.6 million years ago to present day). The wave cut platforms observed along the bases of many of the cliff faces are likely to have developed during inter-glacial sea level highs. It is believed that the current cliff faces were located some 90kms to the east and the erosion over the last 70 million years has resulted in the recession of the cliff faces to the present coast line.

The cliff faces have revealed Hawkesbury Sandstone bedrock assessed to be typically distinctly to slightly weathered and generally of medium strength. It is evident that the topography of the majority of the cliff faces has been influenced by the orthogonal joint sets identified during our inspections. At the base of the majority of the cliff faces was a sandstone wave cut platform, generally covered by an abundance of detached blocks from previous rock falls. The blocks were either elongated or "cubic" and suggest that they were derived from collapse of cliff face overhangs and wedges of sandstone bedrock within the cliff face. The detached sandstone blocks ranged in size from less than about $1m^3$ to in excess of about $20m^3$ in size and their shape and size appeared to be controlled by the two principal orthogonal joint sets. The principal orthogonal joint sets were generally orientated approximately north-south (ranging between about 335^o and 035^o) and east-west (ranging between about 95^o and 120^o). We note that some sections of the cliff lines were densely vegetated and at other locations the base of the cliffs sections were covered by detached sandstone blocks, which prevented more detailed observations.

The cliff faces have revealed a number of relatively weak features, including extremely weathered sandstone (XWS) seams, shale bands, fractured zones, weathered igneous dykes, shear zones and clay bands.

The differential weathering and erosion (by wave and wind action) of relatively weak sub-horizontal XWS, shale bands and clay bands over the cliff faces and, in particular the basal portion of the cliff faces is a likely mechanism of cliff line collapses due to undercutting of the more competent sandstone above, followed by toppling and/or basal shear. Basal shear could also occur due to the surcharge load of the less eroded overlying rock mass.

Such a failure mechanism would result in the material collapsing down the cliff face onto the wave cut platform (**Figure 5.1**).

Figure 5.1 Cliff face failure onto wave cut platform

Alternatively, such a failure mechanism would result in the material collapsing and being captured on a flat bench within the cliff face (**Figure 5.2**).

Figure 5.2 Cliff face failure onto cliff bench

In addition, similar differential weathering and erosion (by wave and wind action) of relatively weak sub-horizontal XWS, shale bands and clay bands over the cliff faces has led to the formation of the numerous cliff top overhang features identified along the study area and undercut features at the base and over the cliff face (**Figure 5.3**).

Figure 5.3 Cliff top overhang and undercut features

Collapse of these overhangs has also occurred in a similar manner as described above. The rock fall at Diamond Bay (**Figure 5.4**) is believed to be due to such a mechanism although the form of the rock fall appears to have been controlled at least in part by the relatively weaker mass strength of the rock rather than strictly controlled by the orthogonal joint sets. This may be due to localised alteration of the rock mass due to the elevated heat associated with the intrusion of the igneous dykes to the north and south of the rock fall. The alteration appears to have weakened the overall strength of the rock mass in this area.

Figure 5.4 Collapse of cliff face overhang at Diamond Bay

It also appears that the overhangs migrate up the cliff face with sub-horizontal bedding partings and the orthogonal joint sets controlling each successive block collapse (**Figure 5.5**).

Figure 5.5 Upward migrating overhang

The presence of detached blocks along the wave cut platform is considered to be the product of “recent” (in geological terms) and previous collapses from the cliff faces. The items associated with potential geotechnical hazards 1A, 1B and 1C would be associated with continuation of these natural processes. In addition, we note the presence of sub-vertical features, in particular the igneous dykes and/or shear zones identified over the study area which manifest themselves as sub-vertical sided gullies with similar orientations to the orthogonal joint sets described above.

The igneous dykes and shear zones typically comprise extremely weathered/residual soil materials which are often fractured. These relatively weaker sub-vertical features are also affected by differential weathering and erosion (by wave and wind action) and have resulted in the formation of cliff line gullies (**Figure 5.6**).

Figure 5.6 Cliff line gullies

Crucial to these processes is the rate at which they are occurring. Little evidence is available on the overall rates of occurrence of these forms of instability and the resultant rate of recession of the cliff face. Nonetheless, it is clear that rock falls do occur. There is some evidence on the rates of erosion in the paper by Young & Wray (2000). Rates of erosion, which may be summarised from observations given in this paper, are:

- For recession of the coastal escarpment south of Nowra, the "maximum possible rate of 170m/Ma" has been determined. This corresponds to 0.17mm per year.
- The most rapid rates of recession occur in gorges (usually where undercutting occurs on weaker bands due to waterfall erosion effects) being about 2 to 3km/Ma. This corresponds to a rate of 2mm to 3mm per year.
- There is no data given for the Hawkesbury Sandstone cliff lines in the Sydney area.
- Dragovich (Reference 3) refers to weathering of softer beds causing undercutting of cliff lines. Dragovich quotes Roy as determining an average rate of undercutting of 2mm to 5mm per year, but that the overall rate would be slower due to rock falls protecting the softer bed.

- It is clear from the discussion in Young & Wray, that there will be significant variations in the rates of weathering and that extreme events, such as tsunamis or higher past sea levels during interglacial periods can also be relevant to the rates of recession and cliff line formation.

Based on the above, we could expect the Hawkesbury Sandstone cliff face to erode at less than 1mm per year. As an example, we have taken a 2m x 2m x 2m size block on the side of the cliff face. We have assumed that the block will remain 'stable' provided the horizontal extent of undercutting is less than 1m in from the outer face. Adopting a relatively high (or conservative) rate of erosion of 1mm per year, it would take at least 1000 years before the block would fall from the cliff face.

- In addition, we note that the erosion rate of the relatively weaker shale bands, fractured zones, weathered igneous dykes, shear zones and clay bands would be at accelerated rates as indicated by the formation of gullies and undercut sections of the cliff faces identified over the study area and indicated by the above photographs.

Additional triggers to collapse of blocks and wedges over the cliff faces are:

- Water pressure developed in the sub-vertical joints behind potentially unstable blocks or wedges during and following rainfall events.
- Localised tree root jacking where tree roots penetrate joints at the rear of blocks and wedges over the cliff faces.
- In our opinion, the elements most at risk are:
- Persons (such as residents, recreational users, Council employees etc) at the base of the cliff face, such as users of the Bronte or North Bondi Pools, users of Bondi, Tamarama and Bronte beach areas adjacent to the cliff faces, people on the wave cut platforms (e.g. fishermen, divers, etc).
- Persons (such as residents, recreational users or Council employees) along the crest of the cliff face within private properties, reserve areas, and the coastal paths.
- Sections of residences (including yard areas) located on the tops of the cliffs and situated close to the edge of the cliff faces.
- Existing pathways, stormwater infrastructure, rock pools and services (below and above ground).

The steeply sloping upper vegetated portions of selected cliff faces over the study area are likely to comprise thin natural residual soils with bands of weathered bedrock and/or detached sandstone blocks derived from localised collapse of unstable cliff face features but with little, if any, downslope movement.

Instability of soil slopes over sections of the cliff faces was also identified at selected locations. Instability of such slopes is typically governed by one or more of the following factors; over-steep batter slopes, elevated water pressures within the soils associated with ineffective drainage systems and/or surface water run-off and erosion of the toe of the slopes by wave action.

The most dramatic example of a number of these factors affecting one slope was the fill batter slope infill of the gully feature within Waverley Cemetery (**Figure 5.7**).

Figure 5.7 Fill batter slope at Waverley Cemetery

On-going creep of such soil materials is typical over moderate and steeply sloping sites such as selected areas within the study area. Creep would be indicated by uneven slope surfaces and/or localised sub-vertical back scarp features. In addition, concentrated discharge of surface run-off after heavy or prolonged periods of rainfall can cause localised instability

Existing retaining walls within selected areas close to the cliff edge were in variable condition but generally of relatively low height. Collapse of such walls would be relatively localised and the collapse debris could impact the foreshore area below and/or the retained surface above.

It is important to be mindful that rock falls, soil slumps etc can occur at anytime and it would be difficult to impossible to predict when the identified potential hazards will occur. Also, we cannot predict when an extreme or unusual event may occur (such as an earthquake or 1 in 100 year rainfall event etc) and what impact it would have on the stability of the identified potential hazards.

5.2 Risk Analyses

On the basis of the above and using the information obtained from our site observations we provide below our qualitative assessment of risk to both life and property and additional comments in relation to the potential impact of climate change on risk levels. The assessment has been carried out in accordance with the guidance provided in Reference 1.

5.2.1 Risk To Property

We note that strict application of the assessment of consequences to property, as outlined in Appendix A, requires that “the approximate cost of damage be expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.” We have applied this to our assessment of risk to property but have also included our assessment of the risk of damage to the dwelling and any landscape structures. We also note that we have not made any attempt to quantify any loss of property value due to loss of land as a result of cliff face collapse.

In determining consequences we have used the following information:

- An assumed typical property value (land plus dwelling) based on the median house sale price as outlined in ‘The Sun Herald Property Guide 2010’, dated 21 February 2010. We acknowledge that this is a relatively crude method of value assessment and will vary depending on the size of the lot and whether or not the residence is an individual unit or a detached house and the affect on property values of an ocean view aspect. However, we consider that this is a reasonable estimate.

- Typical costs for building repairs and rock face stabilisation measures.

Table A (Appendix A) summarises a qualitative assessment of each potential landslide hazard and of the consequences to property should the landslide hazard occur, under existing conditions. Based on the above, the qualitative risks to property have been determined. In this regard, we have identified general locations over the study area.

We have assumed "Possible" and "Likely" assessed likelihoods for potential geotechnical hazards 1A, 1B, 1C, 2 and 3 in order to provide a range of risk levels for these potential geotechnical hazards. This is also an attempt to assess the impact of potential geotechnical hazards along the study area cliff faces of variable condition and stability that were, in many instances, unable to be accurately assessed from the base or crest of the cliff faces.

For potential geotechnical hazards 4 and 5 our assessment has been based on their current condition and our knowledge of the installation of a number of the rock face stabilisation measures over the area of the coastal path adjacent to Waverley cemetery. The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A.

The assessed risk to property typically varies between Very Low and Low which would be considered to be 'acceptable', in accordance with the criteria given in *Practice Note Guidelines for Landslide Risk Management*.

However, the assessed risk to property for private residences and stormwater infrastructure at the edge of cliff faces was Moderate which would be considered to be 'tolerable', in accordance with the criteria.

5.2.2 Risk to Life

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal, vulnerability and evacuation factors that have been adopted are given in Table B (Appendix A) together with the resulting risk calculation. We note that we have assumed that the affected person is immediately above or below the specific hazard when it occurs (i.e. spatial probability of 1), which would be regarded as conservative, particularly over longer areas of the study area.

Our assessed risk to life for the person most at risk, under existing conditions ranges between about 10^{-4} and 10^{-10}. These would be considered to be 'tolerable' and 'acceptable', respectively in relation to the criteria given in *Practice Note Guidelines for Landslide Risk Management*. We note that the 'tolerable' risk levels are associated with instability of lookouts/vantage points adjacent to coastal paths and above or below a particular feature when fishing, sun baking or walking on the wave cut platform. The 'tolerable' risk levels were associated with:

- A resident within their dwelling above the cliff top feature (with an assumed "Likely" assessed likelihood),
- A person on a cliff top lookout/vantage point (above a potential cliff face hazard) adjacent to the coastal paths (with an assumed "Likely" assessed likelihood), and
- A person fishing, sun baking or walking on the wave cut platform below a potential cliff face hazard over the general area of the cliff face (with an assumed "Likely" assessed likelihood).

We reiterate that our assessment of risk to life has been based on one person being affected. Where more than one person is affected, the level of risk calculated for an individual increases by a factor equivalent to the number of people present; i.e. for 2 people the level of risk would increase from say 1×10^{-4} to 2×10^{-4} etc. For areas where significant numbers of people may congregate, say over the surrounds of Bronte Pool then the risk levels associated with the increased numbers of people being present (say 10) would, for this study, be over the upper end of the 'tolerable' range.

5.2.3 Impact of Climate Change on Risk Levels

It is difficult to assess the potential impact of predicted more intense storm events as a result of climate change and sea level rise on cliff face stability and no formal studies, to our knowledge, have been completed. However, it is considered reasonable to assume that more intense storm events and elevated sea levels will result in elevated erosion rates over a greater height of cliff face. In addition, salt spray from wave action can be expected to affect a greater height of cliff face. On this basis, in terms of the NSW State Government recommended sea level rise planning benchmarks of an increase above 1990 mean sea levels of 40cm by 2050, and 90cm by 2100 we would expect elevated erosion rates to affect the lower portion of the cliff face.

With regard to our assessed risk levels, increased erosion rates affecting the base of the cliffs would probably lead to localised collapse of undercuts over the basal areas of the cliff lines and potential instability of the toe area of the fill batter slope within Waverley Cemetery. On this basis, we would expect that risk levels associated with potentially unstable features at the bases of the cliffs and the fill batter slope within Waverley Cemetery would, over the next 50 to 100 years, be best represented by the levels of risk to life and property associated with a 'Likely' likelihood. In this instance risk levels would generally be at 'tolerable' levels. However, if the on-going monitoring over the study area reveals evidence of more rapid rates of erosion and deterioration of potentially unstable features, such that a revised risk assessment indicates 'unacceptable' risk levels, then detailed geotechnical advice will be required to determine the scope and extent of stabilisation measures.

5.2.4 Previous Geotechnical Advice

We note that we have completed a number of previous geotechnical assessments over the study area. In general, our advice in reports from 2002 onwards included similar risk assessments as outlined above together with specific detailed design advice. A brief summary is outlined below.

Dover Heights

In 1987 we advised on construction of a pool close to the cliff edge within the rear yard of a property on the eastern side of Lola Street. A cliff top overhang and a shale band within the cliff face about 6m below the crest of the cliff were identified. Our advice recommended drilling of boreholes to investigate the location of sub-vertical defects within the rear yard which would control the lateral (westward) extend of potential instability. Based on the results of the investigation an appropriate location and design of the pool could be determined; we were not involved in any further aspect of the pool development.

In 1999 minor instability of a garden bed fill slope had occurred which affected the rear yard of a property that lined the cliff top adjacent to the small reserve area at the eastern end of Hunter Street. Advice was provided in relation to construction of a new retaining wall to support the fill.

In 2002 we provided geotechnical advice in relation to construction of a portion of the coastal footpath structure within Dover Heights reserve adjacent to the eastern end of George Street. We recommended that the path structure be founded on bedrock and that further assessment of the cliff face be undertaken. We have no information as to whether such further assessment of the cliff face was undertaken.

North Bondi

We have completed geotechnical assessments for proposed developments at 162, 164 to 166, 174, 176, 178 and 180 Hastings Parade. The reports identified various geotechnical features within the upper portion of the cliff faces lining the eastern boundaries of the properties, including cliff top overhangs, sub-vertical defect planes orientated approximately north-south and the above described eroded igneous dyke forming the cliff line gully feature.

Our risk assessments generally indicated a similar range of risk levels to both life and property.

Our advice in relation to reducing risk levels included some or all of the following measures:

- Construct buildings and structures to the west of the zone of influence of any sub-vertical defect orientated approximately north-south (i.e. parallel to the cliff face). Typically the zone of influence related to the lateral spacings of the sub-vertical defects (a maximum of about 4m). Alternatively, structures were recommended to be constructed to cantilever over the defect plane and anchored into sound bedrock to control potential over-turning.

- Provide permanent fences or other barriers to prevent access to the cliff edge.

- Property owners visually monitor the existing cliff line and the cliff top area to check for signs of damage to existing structures and fences, tension cracks developing at the site surface etc. If there are causes for concern then access to the cliff top should be restricted and further geotechnical advice immediately sought. We recommended that any potentially affected neighbours also be informed.

South Bondi

We completed numerous site inspections during the upgrade of the Bondi Icebergs building. The rock cut face lining the south-western side of the site was of poor quality and extensive stabilisation measures were installed included rock anchors, pattern rock bolting and placement of reinforced shotcrete.

Bronte to Tamarama

Between 2003 and 2004 we provided geotechnical advice to Council on construction of the new walkway lining the cliff top. Our assessment also included advice on rock face stabilisation measures including rock bolts, trimming off overhangs and/or underpinning overhangs (including the large overhang below Tamarama SLSC). We note that the walkway has been constructed (with all footings founded on bedrock) but no rock face stabilisation measures were implemented. However, no formal risk assessment was requested or completed.

Extension of Bondi to Bronte Coast Walk; Waverley Cemetery

Between 1999 and 2005 we completed a number of geotechnical investigations and risk assessments and were also involved in witnessing some of the construction of the coastal walk structures.

We identified the most appropriate location of the walkway in terms of risk levels which included bridging over narrow gully features (believed to be eroded igneous dykes), the off-set of the 'on-grade' portion of the walkway around the gully feature should any instability of the fill batter slope occur. We also provided advice on stabilisation measures for rock face undercuts and anchoring of the lookout structure at the crest of the slope within Calga Reserve.

During the construction works there was a report of instability of the fill batter slope within the gully feature although we were not requested to provide any advice. We note that during our recent site inspections there was a temporary 'cyclone' mesh fence lining the crest of the fill batter slope. Furthermore, there appeared to have been a change in the slope profile and a greater amount of concrete debris at the base of the batter slope. It appears that on-going erosion of the 'finer' grained soil materials from the toe of the batter slope is exposing more concrete debris and further instability can be expected over time.

5.2.5 Previous Work by Waverley Council

During the progress meeting held at WC offices on 15 June 2011, WC reported that they had conducted a visual assessment of the cliff faces in 1982. Subsequently, a copy of photographs of the foreshore cliff lines taken in 1982 by WC was provided to the undersigned. The photographs were taken before the works were completed and we understand that the works involved hydraulic rock splitting of overhang features in public reserve areas.

Study of the photographs indicates that the suspected cliff face rock fall at Diamond Bay had occurred before the photographs were taken; the rock debris covering the base of the cliff was evident on the photographs. On this basis, the large cliff face failure had occurred at least 29 years before the current assessment.

5.2.6 Additional Comments

It is recognised that due to the many complex factors that can affect a site, the subjective nature of a risk analysis, and the imprecise nature of the science of geotechnical engineering, the risk of instability for a site cannot be completely removed. It is, however, essential that risk be reduced to at least that which could be reasonably anticipated by the community in everyday life and that landowners be made aware of reasonable and practical measures available to reduce risk as far as possible. Hence, risk cannot be completely removed, only reduced, as removing risk is not currently scientifically achievable.

In preparing our recommendations given below we have assumed that no activities on surrounding land which may affect the risk on the subject site would be carried out. We have further assumed that all buried services are, and will be regularly maintained to remain, in good condition.

We provide below geotechnical advice and recommendations in relation to landslide risk management measures for identified potential geotechnical hazards and the study area as a whole which, if adopted in full, would maintain risk at current 'tolerable' and 'acceptable' levels. These recommendations form an integral part of the Landslide Risk Management Process.

However, it is a matter for Council how they wish to sequence and implement the advice outlined in the following Section 6. In this regard, we note that the advice will be used by WC as a guide to development of emergency responses to safeguard the community, public and private assets and future development from severe coastal storm events.

5.3 Summary

The cliff faces within the study area represent natural features within a foreshore landscape and any associated cliff face instability is also a natural phenomena. Stabilisation of individual potential hazards is likely to be uneconomical, particularly for persons at risk along the base of the cliff faces or crest and upper portions of the cliff face (where access is possible). However, in relation to private property, the potential for cliff face instability may be detrimentally impacted by construction of buildings and other structures due to additional surcharge loadings, unless appropriate additional support to the potentially unstable features has been provided. Furthermore, cliff face instability solely due to natural processes may also detrimentally impact private property and the occupants.

In terms of the hazards associated with cliff lines, properties and parcels of land have been identified as being at various levels of risk (as defined in detail in Appendix A). **Table 5-1** lists private properties which have the potential to be at risk from geotechnical hazard, with the purpose to inform Councils planning advice to property owners, particularly in relation to assessment of new developments. Planning recommendations are discussed further in **Section 6**. Full details of the geotechnical hazard assessment are included in **Appendix A**.

Table 5-1 Properties With Potential Geotechnical Risk

Building Description	Plan	Lot
1 Jensen Avenue	SP 687	-
3 Jensen Avenue	DP 7334	56
5 Jensen Avenue	DP 7334	55
7 Jensen Avenue	DP 7334	54
9 Jensen Avenue	DP 400406	A
11 Jensen Avenue	DP 400406	B
1 Marne Street	SP 30361	-
3 Marne Street	DP 9080	26
5 Marne Street	DP 9080	27
7 Marne Street	DP 19254	24
9 Marne Street	DP 19254	25
11 Marne Street	DP 16375	26
28 Macdonald Street	DP 23177	1
2 Ray Street	DP 976698	37
4 Ray Street	DP 417665	2
51B Lancaster Road	DP 102084	B
20 Hunter Street	DP 7044	15
21 Hunter Street	DP 7044	36
36 Myuna Road	DP 4623	27
31 Myuna Road	DP 4623	26
2 Lola Road	DP 10675	12
4 Lola Road	DP 10675	13
6 Lola Road	DP 10675	14
8 Lola Road	DP 10675	15
10 Lola Road	DP 10675	16
12 Lola Road	DP 10675	17
14 Lola Road	DP 10675	18
12 Douglas Parade	DP 10675	22
21 Douglas Parade	DP 348567	B

Building Description	Plan	Lot
23 Douglas Parade	DP 619746	232
25 Douglas Parade	DP 45691	1
25 Douglas Parade	DP 619746	231
8 Wentworth Street	DP 382476	B
10 Wentworth Street	DP 382476	A
12 Wentworth Street	DP 336579	A
14 Wentworth Street	DP 10090	42
16 Wentworth Street	DP 10090	41
18 Wentworth Street	DP 10090	40
20 Wentworth Street	DP 10090	39
22 Wentworth Street	DP 10090	38
24 Wentworth Street	DP 404933	A
26 Wentworth Street	DP 404933	B
26 Wentworth Street	DP 10090	36
28 Wentworth Street	DP 10090	35
30 Wentworth Street	DP 10090	34
32 Wentworth Street	DP 10090	33
34 Wentworth Street	DP 343564	B
36 Wentworth Street	DP 19465	1
38 Wentworth Street	DP 19465	2
154 Hastings Parade	SP 7883	-
156 Hastings Parade	SP 2178	-
158 Hastings Parade	DP 786	13
160 Hastings Parade	DP 443203	A
162 Hastings Parade	DP 443203	B
164 Hastings Parade	DP 439182	A
166 Hastings Parade	DP 439182	B
168 Hastings Parade	SP 4413	
170 Hastings Parade	DP 786	17
172 Hastings Parade	DP 786	18
174 Hastings Parade	DP 308590	1
176 Hastings Parade	DP 308590	2
178 Hastings Parade	DP 308590	3
180 Hastings Parade	DP 308590	4
182 Hastings Parade	DP 413583	A
184 Hastings Parade	DP 413583	B
186 Hastings Parade	DP 398119	D

Building Description	Plan	Lot
188 Hastings Parade	DP 398119	C
190 Hastings Parade	DP 2905	-
192 Hastings Parade	DP 786	25
194 Hastings Parade	SP 21330	-
196 Hastings Parade	DP 106649	A
198 Hastings Parade	DP 106649	B
200 Hastings Parade	DP 441398	X
202 Hastings Parade	DP 515178	1
204 Hastings Parade	SP 4507	-
206 Hastings Parade	SP 16026	-
208 Hastings Parade	DP 320739	3
1 Bay Street	SP 249	-
3 Bay Street	DP 1123754	1
3A Bay Street	DP 1123754	2
5 Bay Street	DP 331848	C
154 Brighton Boulevard	SP 30225	-
156 Brighton Boulevard	DP 786	6
158 Brighton Boulevard	SP 12058	-
83 Ramsgate Avenue	SP 16621	-
85 Ramsgate Avenue	DP 344571	-
89-91 Ramsgate Avenue	DP 343534	10
95 Ramsgate Avenue	SP 905	-
97 Ramsgate Avenue	SP 1160	-
105 Ramsgate Avenue	SP 1159	-
107 Ramsgate Avenue	SP 5170	-
111 Ramsgate Avenue	SP 22198	-
31 Gaerloch Avenue	DP 9842	36
31 Gaerloch Avenue	DP 9842	35
31 Gaerloch Avenue	DP 9842	34
29 Gaerloch Avenue	DP 415974	B
27 Gaerloch Avenue	DP 415974	A
Tamarama SLSC	DP 1052115	7046

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Climate Change Vulnerability

6.1.1 Beaches

Projected sea level rises of 40 cm by 2050 and 90 cm by 2100 above 1990 mean sea levels (as recommended by the NSW State Government) would be likely to cause reductions in the widths of Waverley's beaches. Calculations indicated that such beach recession would be approximately 20 m by 2050 and 45 m by 2100. A direct impact of this would be the gradual reduction in the amenity of all the beaches. Further, there would be an increasing risk to the stability of the seawalls during storms as sand is eroded from the beaches. Over time, as beaches recede, there would be less sand available during a storm to prevent undermining of the seawalls. As the volume of available sand reduces the stability of the seawalls would become more at risk.

Overtopping of seawalls also would increase with time, due to sea level rise, as the relative crest levels and rock levels are lowered allowing larger waves to reach the Lots and seawalls. Therefore, there would be increased risk of inundation to the foreshore buildings and increased risk to pedestrians and the structure of the seawall crests and walkways.

6.1.2 Cliffs

The cliff faces and foreshore slopes within the study area may be regarded as having a 'tolerable' to 'acceptable' risk of instability. Under existing conditions and into the near future on-going monitoring by Council and periodic geotechnical assessments are an appropriate method of landslide risk management.

It is considered that more intense storm events and elevated sea levels would result in elevated erosion rates over a greater height of cliff face. In addition, salt spray from wave action can be expected to affect a greater height of cliff face. On this basis, in terms of the NSW State Government recommended sea level rise planning benchmarks of an increase in sea level of 40 cm by 2050, and 90 cm by 2100 above 1990 mean sea levels, we would expect elevated erosion rates to affect the lower portions of the cliff faces.

Increased erosion rates affecting the base of the cliffs would be likely to lead to localised collapse of undercuts over the basal areas of the cliff lines and potential instability of the toe areas of the fill batter slope within Waverley Cemetery. We expect that risk levels associated with potentially unstable features at the bases of the cliffs and the fill batter slope within Waverley Cemetery would, over the next 50 to 100 years, be best represented by the levels of risk to life and property associated with a 'Likely' likelihood. In this instance, risk levels generally would be at 'tolerable' levels. However, if the on-going monitoring over the study area reveals evidence of more rapid rates of erosion and deterioration of potentially unstable features, such that a revised risk assessment indicates 'unacceptable' risk levels, then detailed geotechnical advice will be required to determine the scope and extent of stabilisation measures.

6.2 Planning Controls

The Mapping included as figures in **Appendix G** indicates Lots with the potential to be at risk from coastal or geotechnical hazards and lines indicating zones of reduced foundation capacity (Hazard lines). We recommend that this information be incorporated in Councils planning guidelines so that particular sites could be identified and inform Councils planning advice to property owners, particularly in relation to assessment of new developments. On this basis we recommend that should new developments be proposed at these properties, a geotechnical assessment of the cliff face (**Table 5-1** and the reserves as included in the GIS mapping) or coastal assessment (**Table 6-1**) should be required as a mandatory condition of the Development Application process. Land within the zone of reduced foundation capacity should consider the potential for reduced bearing capacity in any structural foundations (i.e. consider piling beyond the zone of reduced foundation capacity), *Nielsen et al (1992)*. That is not to say that development on these lots are affected directly by a geotechnical or coastal hazard but that some portions of the lots may be affected and that further site-specific assessment is recommended.

We recommend that notification of properties known to be potentially affected by coastal or geotechnical hazards is to be undertaken by inclusion on the Section 149 Certificate. This provides advice to current owners as to the potential for coastal or geotechnical risk and the advice transfers to new owners with the sale of the property.

The following wording is recommended for Section 149 notations for planning certificates issued under Section 149 of the Environmental Planning and Assessment Act (1979) with respect to any lands identified in the Coastline Risk Management Policy for Development in Waverley.

Section 149(2) Notation for properties located within Coastline Hazard areas.
"On the information available to Council, the land in question is affected by coastal/geotechnical processes. Restrictions on development in relation to coastline effects apply to this land as set out in Council's Coastline Risk Management Policy for Development in Waverley and Waverley DCP."

Section 149(5) Notation for properties located within Coastline Hazard areas.
"Development along the coast in the Waverley Local Government Area has been threatened, damaged or destroyed by the action of storm waves/cliff falls on a number of occasions in the past. Council may hold records of past storm damage/cliff falls and/or works that occurred at certain locations for particular events."

In line with the above it is recommended that Council develop a Coastline Risk Management Policy for Development in Waverley and annotate the Waverley DCP accordingly.

Seaward of the Ramsgate Avenue properties listed in **Table 6-1**, there are parcels of land which Council indicate are 'residual estate' (DP343534, DP333815, DP1151378, DP1151383, DP338231, DP341546 and DP1151383) i.e. they remain in the private ownership of the people who were owners at the time that the land was subdivided even though the certificates of title show that these parcels were intended to be public land. They are part of a reserve gazetted in Council's name on 6/3/1981. Two lots were transferred to Council's name but these parcels were not. It is recommendation that Council investigate changing the ownership of these residual estates to Crown Land with Council as the trustee or dedication. These areas are rocky reef, at levels of about 1mAHD, which are regularly inundated by waves.

Table 6-1 Properties* Selected for Coastal Inundation Planning Controls

Building Description	Plan	Lot
77 Ramsgate Avenue, Bondi	DP 381954	61
79-81 Ramsgate Avenue, Bondi	DP 344453	B
83 Ramsgate Avenue, Bondi	SP 16621	-
85 Ramsgate Avenue, Bondi	DP 344571	9
89-91 Ramsgate Avenue, Bondi	DP 343534	10
95 Ramsgate Avenue, Bondi	SP 905	-
97 Ramsgate Avenue, Bondi	SP 1160	-
105 Ramsgate Avenue, Bondi	SP1159	-
107 Ramsgate Avenue, Bondi	SP 5170	-
111 Ramsgate Avenue, Bondi	SP 22198	-
North Bondi Surf Lifesaving club	no plan number	
Bondi Life Guard Tower	no plan number	
Bondi Icebergs	DP727777, DP822245	1748, 1556
Tamarama Life Guard Tower	DP1052115	7046
Tamarama café/kiosk	DP1058517	7124
Bronte Surf Life Saving Club	DP1058385	7102
Bronte kiosk and public amenities	DP1058385	7102
Bronte south public amenities	DP1058385 DP93737	7102, 7091

Properties have been selected based on their proximity to the shoreline and ground levels taken from Councils ALS data. To estimate the actual extent of inundation risk to the buildings, floor levels would be required and further assessment undertaken.

6.3 Management Options

6.3.1 Beach Nourishment

Shoreline erosion issues are not unique to the Sydney or the NSW coastline and it has long been held that beach nourishment is, in many cases, the best long-term management strategy to mitigate beach erosion. Hazards associated with storm events and sea-level rise can be alleviated by beach nourishment if sufficient sand deposits are available for nourishment works. Beach nourishment is potential management option for the Waverley Beaches should the projected sea level rise occur and sands are eroded from Waverley's beaches.

The Sydney Coastal Councils Group has undertaken a scoping study to develop the outline of a sand nourishment programme utilising suitable offshore sand deposits for amenity enhancement and to ameliorate increased hazard risk from sea-level rise (Walker et al. 2010). The scoping study found that a nourishment programme accessing offshore sands for Sydney's beaches is viable economically, the main economic benefits being associated with the avoidance of flow-on effects from loss of beach amenity to beach visitors, local residents and businesses and government revenues.

At present the NSW Government has legislated to prohibit the commercial extraction of offshore marine sands. However, the Scoping Study provided a rational basis to inform both the member councils and the NSW Government of the advantages and disadvantages of utilising offshore marine sand sources to facilitate immediate and longer term demands for nourishment purposes in the Greater Metropolitan Region.

As a result of the positive cost-benefit assessment and the favourable environmental and social outcomes, a Strategic Gateway Review was prepared as the first step in the establishment of a business case to NSW Treasury to seek funding to progress the programme.

At present, beach nourishment is not required for Waverley. Should the sea level rise projections come to fruition, then it is recommended that sand nourishment be considered to maintain the beach amenities.

6.3.2 Seawall Works

During times of storm erosion there will be an increasing risk to the stability of the seawalls over time, should the shorelines recede due to sea level rise and as the fabric of the seawalls deteriorates.

A detailed assessment of the fabric and stability of the Bondi seawall was undertaken by PWD in 1988. Following this, improvement works were undertaken to the seawall in the form of a Reno-mattress toe protection structure. Similar improvement works to the seawalls at Bronte and Tamarama could be undertaken also, if deemed necessary. Assessment of the fabric and stability of Bronte and Tamarama seawalls is recommended to ascertain the requirement for any current or future improvement works.

6.3.3 Permanent Warning Signs and Fencing

We recommend that signs warning of potential cliff face instability be provided in all publicly accessible areas along the crest and basal areas of the cliff face within the study area. We recommend that they be placed at least at the following locations:

- Bronte Pool
- The wave cut platforms at the base of the cliff faces where public access is currently feasible. Based on our observations this would include at least the following locations:
  - North Bondi and Ben Buckler Headland
  - Adjacent to the walkway below the north-eastern side of Marks Park
  - Mackenzies Bay to Tamarama
- The cliff edges where public access to the cliff top and the base of the cliff is feasible. Based on our observations this would include at least the following locations:
  - All publicly accessible areas adjacent to Rodney Reserve and Dover Heights Reserve, including any additional localised areas at the ends Hunter Street, Myuna Road, Weonga Road
  - Bondi Golf Course
  - Mackenzies Bay to Tamarama
  - Waverley Cemetery at the gully location

We recommend that property owners be advised to post similar warning signs where yard areas extend to the cliff edge and, in particular, at Marne Street, Jensen Avenue, Ray Street, the eastern ends of Hunter Street, Lola Road, the eastern end of Douglas Parade, the central portion of Wentworth Street (all in Dover Heights) and 154 to 208 Hastings Parade, 1, 3 and 5 Bay Street and 154 to 158 Brighton Boulevard, North Bondi.

We note that there are selected sections of public reserves and other cliff top areas accessible by the public within the study area that are not fenced off. We recommend that fence lines be provided in addition to the above mentioned warning signs at the following locations:

- Reserve area lining the northern side of Diamond Bay
- Waverley Cemetery at the gully location

Consideration should be given to advising Bondi Golf Course to erect fences or, at the very least, warning signs on the basis of the advice in this report.

The damaged fence line at the crest of the cliff over the northern end of Calga Place should also be repaired to prevent access on to the cliff top area above Bronte Pool.

6.3.4 Monitoring

BEACHES

It is recommended that ongoing monitoring be undertaken on all the study beaches. Following extreme events it is recommended that the following be undertaken:

- survey a number of cross sections of the beaches
- photographic record including photographs showing, beach levels, scour against the seawall, exposure of the Reno-mattress (or other structures not normally exposed), extreme water levels and overtopping
- notes on any significant observations, such as location and quantities of overtopping, location of worst scour, etc.

It is understood that the Office of Environment and Heritage (OEH), take aerial photographs of the beaches periodically. These photographs can be accessed and photogrammetry undertaken on request at any time. It is recommended that these data be sourced and analysed periodically to assess ongoing trends at the beaches.

CLIFFS

We recommend that Council monitor the identified potential hazards within the study area on an annual basis and after periods of prolonged or heavy rainfall to assess existing conditions and any indications of deterioration such as debris/boulders on the beach, rock platform, damage to pathways, etc.

Based on previous studies of available rainfall data in relationship to landslide events, in particular the study carried out for the Pittwater area (Walker 2007, Reference 4), we provide the following tentative definition of heavy rainfall and prolonged rainfall:

- Heavy Rainfall: at least 100 mm of rainfall in one day
- Prolonged Rainfall: at least 150mm of rainfall over a 5 day period

These amounts of rainfall represent 1 in 2 year occurrences for the Pittwater area and are considered reasonable for the Waverley area.

It is imperative that such monitoring be documented formally and that the required frequency of reporting (and to whom) is defined clearly. Where incidents of instability have occurred within the monitoring period then, where possible, we recommend that Council provide relevant details within the monitoring reports. These details would include the date of the incident, the weather conditions on the day and leading up to the incident, a location plan, photographs and dimensions of the specific feature (block sizes, width and length of landslip features, crack widths, etc., would also need to be recorded). It is recommended that the monitoring reports be provided to the geotechnical engineers so that further advice can be provided if there were any causes for concern. The need for additional site specific stabilisation measures can then be better assessed.

We recommend that a detailed assessment should be undertaken by an experienced engineering geologist/geotechnical engineer on a 5 yearly basis to assess current conditions with regard to the contents of this report and the on-going inspection monitoring reports. This is of particular importance for the rock below the Bronte to Tamarama coastal path. The following previous J&K reports issued to Council on Bronte Marine Drive to Tamarama Marine Drive are relevant in relation to rock face stabilisation measures:

- Ref. 17666SLrpt dated 20/6/03;
- Ref. 17666SLLet2 dated 17/2/04; and
- Ref. 17666SLfax7, dated 4/6/04.

All existing subsurface drains, sewers and any other water carrying pipelines must be subject to ongoing and regular maintenance by the respective owners. We recommend that this include checking for leaks and damage to the water carrying pipelines by a plumber or similarly qualified professional and appropriate maintenance and repairs completed without delay. We recommend that such maintenance be carried out at no more than five yearly intervals, commencing within 12 months of issue of this report; including provision of a written report confirming scope of work completed and identifying any required remedial measures.

6.4 Emergency Actions

During an extreme coastal storm event there are potential risks to the safety of people and property including wave overtopping (coastal inundation), seawall stability (including scour) and sand drift.

During extreme storms there is the potential for significant overtopping of the seawalls that could impact public safety along the seawall promenades. These walkways run along the top of the seawalls at Bronte, Tamarama and Bronte and are around 7m wide. It is recommended that consideration be given to displaying signage indicating an overtopping hazard. To mitigate this risk, we recommend that Council take steps to exclude public access to seawall promenades during extreme events.

There is a risk of wave runup and inundation to some Lots at Ben Buckler. It is recommended that projected inundation levels be provided to property owners so that they may take advice in respect of the level of risk that may be presented to development on their Lot and any mitigation measures that may be warranted. It is recommended that owners be advised of the requirement for Council approval for any works that may be undertaken to mitigate any adverse impacts of storm events on their Lot, and that prior approval for any contemplated temporary works should be sought.

During extreme storms there are risks to the stability of the seawalls at Tamarama and Bronte with the potential for localised failures. This could be due to a combination of factors including eroded beach profiles, wave forces or localise stormwater runoff. It recommended that the seawalls be monitored and fencing be used to exclude public access to any areas that become unsafe. Once the storm event has passed the seawall should be assessed and appropriate repairs undertaken.

During some storm events, with strong onshore winds, a build up of sand can occur against the seawall and can be blown over the top, impacting the land use behind. Council currently regrade sand as it builds up against the seawall. This should continue to be undertaken as required, to minimise sand drift landward of the seawall.

When the beach at Bondi is in an eroded state the Reno-mattress could become exposed. If exposed there would be a significant fall from the promenade onto the hard Reno-mattress. This could be a public safety hazard as, under normal conditions, there would be a lesser fall onto softer sand. It is recommended that, if this situation occurs, appropriate signage and fencing be erected to mitigate the risk.

6.5 Recommended Further Studies

Beaches

The PWD 1988 report provides significant information of Bondi Beach and Bondi seawall. It is recommended that similar studies be undertaken for Bronte and Tamarama including the following:

- assessment of the fabric and stability of Bronte and Tamarama seawalls (including test pits concrete cores and stability analysis)
- wave modelling to assess nearshore wave coefficients for Bronte and Tamarama
- photogrammetric analysis to determine historical beach levels of Bronte and Tamarama beaches

The above data would be used to refine estimates of storm demand and review longer term trends in beach levels. It would be used also to assess the requirement for improvements to the seawall both now and in the future at Bronte and Tamarama.

Cliffs

With regard to the landslide risk management measures the following further geotechnical work is recommended:

- review of monitoring reports
- re-assess the need for stabilisation measures in light of the above monitoring reports
- geotechnical re-assessment on a five yearly basis

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APPENDIX A – Cliff Assessment

AS/NZS ISO 9001
Certified
Davis Langdon Certification Services

REPORT

TO

WORLEYPARSONS

ON

GEOTECHNICAL ASSESSMENT

OF

EXISTING FORESHORE CLIFFLINES

BETWEEN

CLARKES RESERVE, DOVER HEIGHTS AND BOUNDARY STREET, BRONTE, NSW

5 August 2011

Ref: 24648ZRrpt

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APPENDIX A: LANDSLIDE RISK MANAGEMENT TERMINOLOGY

1 INTRODUCTION

This report presents the results of our geotechnical assessment of the foreshore cliff lines between Clarkes Reserve, Dover Heights and Boundary Street, Bronte, NSW. The assessment was commissioned by Mr Peter Horton of WorleyParsons (WP) in an email dated 22 December. The commission was on the basis of our fee proposal (Ref: P33333ZRprop) dated 9 December 2010.

Whilst the study area covers the entire length of coastline and comprises beach areas and coastal cliff lines, the role of Jeffery & Katauskas (J&K) is to assess only the coastal cliff lines within the study area. WP will address the remainder of the study area.

The geotechnical assessment is to form part of a 'Coastal Risks and Vulnerability Study' required by Waverley Council (WC). Based on the Request For Quotation document dated November 2010 prepared by WC we understand that the purpose of the study is to:

- Understand the potential risks to the community, infrastructure, current systems and natural environments and ecosystems that are associated with the impacts of climate change and sea level rise,
- Provide appropriate information to assist in effective long term sustainable strategic and land use planning within the coastal zone,
- Progressively reduce risk levels to existing public and private assets through the identification and application of appropriate planning mechanisms for both existing and future development,
- Facilitate the dissemination of information and develop measures to assist the Waverley community to manage risks associated with coastal hazards and climate change, and

- Guide the development of emergency responses to safeguard the community and public and private assets and development from severe coastal storm events.
- Assess the potential impact of predicted more intense storm events as a result of climate change and sea level rise.

The above coastline hazard definition and climate change vulnerability study will then assist WC in developing a coastal management plan and meeting their obligations to comply with the coastal zone management and emergency management requirements of the NSW Coastal Erosion Reform Package.

The State Government "Sea Level Rise Policy Statement" finalised in November 2009 identified the following sea level rise planning benchmarks which Council are working towards formally adopting by the end of 2010:

- An increase above 1990 mean sea levels of 40cm by 2050, and
- An increase above 1990 mean sea levels of 90cm by 2100.

We note that in accordance with the NSW Government's Draft Coastal Risk Management Guide for each cliff line our report is required to identify the "immediate hazard line" for cliff erosion from the design storm event plus any zone of reduced foundation capacity together with identification of the "hazard lines" for the 2050 and 2100 planning periods. However, during a progress meeting held at WC offices on 15 June 2011 it was agreed that in terms of the hazards associated with cliff lines, properties and parcels of land would be identified as being at various levels of risk. This information would then be incorporated into WC's planning guidelines so that particular sites could be identified as requiring geotechnical assessment, should new developments be proposed.

We have provided the following information:

- A selection of aerial photographs dated 2011 provided by WP,

- A copy of the draft 'Coastal Risks and hazard Vulnerability Study' (Ref. 301015-02526-CS-REP-0001-DRAFT, dated 14 June 2011) prepared by WP for the progress meeting a WC offices on 15 June 2011, and

- A copy of photographs of the foreshore cliff lines taken in 1982 by WC.

The undersigned initially attended an inception meeting at WC offices with WP on 25 January 2011. The undersigned then inspected the foreshore cliff lines within the study area from a boat on 13 April 2011 then completed land based inspections on 27 and 31 May and 3, 6 and 7 June 2011, in order to assess the existing stability of the foreshore cliff lines within the study area.

Based on the results of our assessment, our review of the above provided information and our previous geotechnical reports completed at a number of the properties lining the cliff lines within the study area, we have prepared this report which includes our site observations, our assessment of current levels of risk to both life and property, landslide risk management measures and additional geotechnical advice in relation to specific areas within the study area.

2 ASSESSMENT PROCEDURE

The cliff lines form an approximately 27km total length of the study area which extends south from the northern boundary of Clarkes Reserve, Dover Heights approximately 31km to Boundary Street, Bronte, which marks the southern boundary of Waverley Cemetery.

The subject site comprised the cliff lines, the foreshore areas lining the toes of the cliff lines and the public walkways and reserve areas lining the crest of the cliff lines. Private properties did not form part of the study area although, where possible, observations were made.

The assessment was completed by a Senior Associate level engineering geologist, from safe vantage points and where access was possible along the crest and toe of the cliff faces and also from a boat. The assessment comprised a detailed walkover inspection of the topographic, surface drainage and geological conditions of the study area and its immediate environs. We note that the dates and times of the land based inspections were selected to optimise low tidal water levels in order to allow access, where possible, along the toes of the cliff lines. Inspections using industrial rope access abseiling techniques were beyond the agreed scope of this assessment.

Any identified potentially unstable features were compared to those of other similar lots in neighbouring locations to provide a comparative basis for assessing the risk of instability affecting the site. The attached Appendix A1 defines the terminology adopted for the risk assessment together with a flow chart illustrating the Risk Management Process based on the guidelines given in AGS 2007(c) (Reference 1).

A summary of our observations is presented in Section 3 below. Our specific recommendations regarding proposed landslide risk management are discussed in Section 6, following our risk assessment.

The attached Figures 1 to 5 present Site Location Plans and are based on aerial photographs sourced from Google Earth and indicate the locations of the photographic plates presented in the attached photographic portfolio.

The features described in Section 3, below have been based on hand held tape measure, inclinometer and compass techniques, where access was possible. Otherwise, the dimensions of features that were inaccessible were estimated using observations made from safe vantage points at the crest or toe of the cliff lines and from the boat. Our observations also compared current conditions to those assessed from observations carried out as part of previous geotechnical investigations and/or assessments of areas of the cliff lines within the study area and other historical photographs. Should any of the features be critical to the proposed landslide risk management measures, we recommend they be located more accurately using instrument survey techniques. Plates 1 to 34 provide a photographic record of the study area and are presented in the attached photographic portfolio.

3 SUMMARY OF OBSERVATIONS

3.1 General

The subject site comprised the cliff lines at the following locations:

- Clarke Reserve to North Bondi (including Ben Buckler Headland),
- Southern end of Bondi Beach to the Northern end of Tamarama Beach,
- Southern end of Tamarama Beach to the Northern end of Bronte Beach, and
- The southern end of Bronte Beach to the southern end of Waverley Cemetery (Boundary Street).

A general summary of our observations is presented below and the following Sections 3.2 to 3.9 provide more specific details for selected areas along the cliff lines. The locations of principal geotechnical features identified in the walkover inspections are presented in the attached photographic portfolio.

The crest areas of the cliff lines within the study area included the rear yards of private properties, grass surfaced reserve areas, cliff top coastal walkways and lookout areas. The crest areas within public areas were often stepped with the steps formed by sandstone bedrock outcrop faces, steep vegetated slopes and the flat areas grass surfaced. The coastal walkways comprised timber framed structures or paved pathways. The crest areas within private properties comprised landscaped rear yards or driveways, with brick, concrete block or rendered retaining walls or fences. In some instances, rear yard areas extended along, or close to, the cliff edges.

The cliff face ranged between about 5m and 35m to 40m vertical height and typically comprised sub-vertical sandstone bedrock faces with occasional 'step' features. The cliff faces and their outline (in plan) were controlled by orthogonal sub-vertical joint planes within the sandstone bedrock generally orientated (bearing) approximately north-south (bearings ranging between about 350° and 015°) and east-west (bearings ranging between about 95° and 120°). In some areas the cliff face "zig-zagged" and appeared to be controlled by the orthogonal jointing in the rock mass.

Over the toe areas of the majority of the cliff lines, stepped wave cut rock platforms were evident which were often covered with numerous sandstone blocks ranging between about less than 0.5m x 0.5m x 0.5m to in excess of 2m x 2m x 4m size.

At the interfaces between the cliff lines and the beach areas the bases of the cliff lines were lined by concrete paved walkways (often comprising the roof slabs of stormwater box culverts) and foreshore pools.

A summary of the more specific principal geotechnical features identified along the subject site is presented below.

3.2 Clarke Reserve to Diamond Bay

The descriptions below should be read in conjunction with Figure 1 and Plate 1.

The sub-vertical cliff face was characterised by an intermittent wave cut platform lining the base of the cliff. Numerous detached blocks covered the wave cut platform to the south-east of the intersection between MacDonald and Marne Streets.

The crest of the cliff within Clarke Reserve and the reserve adjacent to the north-eastern end of Diamond Bay were grass surfaced.

The eastern (rear yard) boundaries of the properties lining the eastern side of Jensen Avenue and Marne Street comprised metal, brick, timber or rendered fences which either lined the edge of the cliff or were set-back up to maximum distance of about 3m from the cliff edge.

Beyond the eastern end of Tower Street a grass surfaced reserve area extended to the cliff edge.

Lining the southern boundary of No. 1 Jensen Avenue a grass surfaced reserve area with a stepped profile extended to a barbeque area at the crest of the cliff. The cliff was lined by a brick fence set-back about 1.5m from the edge of the cliff. A sandstone masonry wall and a bedrock outcrop lined the northern and southern sides of the barbeque area.

Cliff top overhangs were supported by brick columns or walls (about 1m high) below a residence on Marne Street and below the grass surfaced reserve area south-east of MacDonald Street.

Unless otherwise described above, the eastern (cliff edge) side of the public reserve areas were lined by timber fences which occasionally lined the cliff edge but were generally set-back at least 1m from the edge of the cliff.

3.3 Diamond Bay

The descriptions below should be read in conjunction with Figure 1 and Plates 1 and 2.

The sub-vertical cliff face was characterised by a bay feature lined by cliff lines orientated approximately 100°; the northern and southern margins of the bay extended west (inland) to form narrow (at least 4m wide) vegetated gullies. Towards the western end of the northern gully an additional gully feature (about 1.5m wide) extended to the south-west.

The main portion of the cliff face within the bay comprised a cliff line with a curved outline (in plan). The cliff face sloped down to the west at between about 40° and 80°. The central and upper portion of the cliff face had a 'fresh' light grey colour whilst the lower portion was darker grey. The relatively flat wave cut platform at the base of the cliff was covered by a large pile of rock debris including a significant portion of boulder size blocks.

The crest of the cliff comprised a generally grass surfaced reserve area with a concrete paved footpath set-back a considerable distance to the west from the cliff edge. The majority of the northern margin of the reserve area was unfenced and the vegetated cliff top area stepped or sloped down to the south at between about 30°

and 45°. Over the eastern end of the northern margin of the bay, a potentially unstable block of sandstone (defined by an open sub-vertical defect orientated north-east to south-west and a sub-horizontal bedding parting) and an overhang (about 5m long, a maximum of about 1.5m high and which extended back a similar horizontal distance) were present about 1.5m and 5m below the crest of the cliff.

The south-western side of the northern gully was fenced off with signs warning of the cliff edge posted. The fence then extended around the remainder of the reserve area to the western end of the gully lining the southern side of the bay.

The cliff top walkway over the southern side of the bay comprised a timber structure. The upper portion of the cliff face below the timber walkway comprised two overhang features which appeared to extend back a horizontal distance of about 3m.

The timber walkway extended south-west to a stepped concrete paved footpath which extended south-east to the northern end of Ray Street. A sandstone masonry wall (set-back at least 1.5m from the cliff edge) lined the concrete paved stepped walkway and a cliff top overhang feature which extended back a maximum horizontal distance of about 2m was located below the sandstone wall.

3.4 Diamond Bay to Hugh Bamford Reserve

The descriptions below should be read in conjunction with Figures 1, 2 and 3 and Plates 2 to 13.

The sub-vertical cliff face was characterised by an intermittent wave cut platform lining the base of the cliff. Numerous detached blocks only covered selected portions of the wave cut platform.

From the northern end of Ray Street, the crest of the cliff was lined by the rear yards of Numbers 2 to 10 Ray Street. South of No. 10 Ray Street the cliff edge was lined by the grass surfaced Dover Heights Reserve which extended south to Lancaster Road. South of Lancaster Road to Weonga Road the cliff edge was lined by properties at the eastern ends of Lancaster Road, Hunter Street and Myuna Road. From Weonga Road the grass surfaced Rodney Reserve extended south to the northern end of Lola Road. From Lola Road, properties lining the eastern sides of Lola Road, Douglas Parade and Wentworth Street extended to the cliff edge. The grass surfaced Hugh Bamford Reserve then extended south from the south-eastern end of Wentworth Street to the northern margin of the Bondi Sewage Treatment Works.

The observations of the residential properties were limited. However, where observations were possible, we note the following:

- The glass, concrete block, rendered or sandstone masonry walls/fence lines over the north-eastern side of the properties close to the cliff edge on Ray Street appeared to be set-back at least 1m from the cliff edge. An open sub-vertical defect (orientated approximately 010°) extended south below Ray Street.

- South of Lancaster Road to Weonga Road the cliff edge was occasionally lined by rendered walls/fence lines.

- From Lola Road to Wentworth Street the structures within the properties appeared to be set-back from the cliff edge.

The grass surfaced Dover Heights Reserve stepped and sloped down to the south, east and north. A stainless steel fence line was set-back at least 2m from the edge of the cliff. The cliff face was characterised by a near continuous cliff top overhang with at least four ‘cave’ features which extended vertical heights of between about

1.5m and at least 6m. Access restrictions prevented any detailed measurements of these features.

The northern end of grass surfaced Rodney Reserve comprised a children's play area and sports field. The sports field surface was a maximum of about 5m above the edge of the cliff top and the eastern margin sloped down to the east at about 30°. The batter slope showed signs of near surface instability and minor sub-vertical back scarp features revealed sandy soils. A chain link fence was set-back at least 2.5m from the edge of the cliff.

The remainder of the Rodney Reserve formed a relatively flat to gently sloping narrow strip of land with a timber fence set-back a maximum of about 1m from the edge of the cliff.

The grass surfaced Hugh Bamford Reserve comprised a flat grass surfaced sports field with a metal fence set-back at least 5m from the edge of the steep vegetated upper portion of the cliff.

The following additional pertinent features were recorded over the cliff face:

- Between Lancaster Road and the northern end of Rodney Reserve intermittent cliff top overhangs were present. The base of the cliff contained intermittent undercut features which had occasionally been eroded to form 'cave' features. The central section of this portion of the cliff line contained an undercut feature about midway up the cliff face which appeared to extend back a maximum horizontal distance of about 3m.

- Intermittent cliff top overhang features were noted over the southern portion of the sports field forming the northern end of Rodney Reserve.

- To the north of Liverpool Street, a large wedge of sandstone was located over the central portion of the cliff face. The rear of the wedge was defined by a sub-vertical defect orientated approximately 010°. During the boat inspection of 13 April 2011 fishermen were noted on the wave cut platform and ropes were hanging down the cliff face. The actual location of the access point was not readily apparent.

- Over the portion of Rodney Reserve adjacent to the eastern end of Liverpool Street, the short length of south-east facing cliff face contained overhang features located at least about 2m below the crest of the cliff. The overhangs intermittently extended to the south. An undercut feature was also present at the base of the cliff at this location. The flat wave cut platform was covered with rock debris typically of boulder size (maximum size estimated to be about 3m x 2m x 2m). Rock debris was occasionally recorded on the cliff face and there appeared to be preferential erosion of shale bands and/or weaker seams of sandstone within the cliff face.

- An overhang feature towards the crest of the cliff below a property on the eastern side of Wentworth Street was supported by what appeared to be blade wall underpins of brick construction.

- The cliff face below Hugh Bamford Reserve was characterised by an undercut feature over the base of the cliff. Intermittent overhang features were recorded over the remainder of the cliff face and in particular at the base of the vegetated upper portion of the cliff top area. The overhangs appeared to extend back an estimated maximum horizontal distance of about 2m. A flat bench over the lower portion of the cliff face was covered with a number of boulder sized blocks of sandstone.

3.5 Hugh Bamford Reserve and Bondi Golf Course

The descriptions below should be read in conjunction with Figure 3 and Plates 13 and 14.

The southern margin of Hugh Bamford Reserve was characterised by a stepped vegetated slope which was lined by the Bondi Sewage Treatment Works. Access to the Treatment Works was not possible and no further comments have been provided. In addition, we have assumed that comprehensive geotechnical advice was provided to Sydney Water in relation to cliff face stability during construction of the Treatment Works.

South of the Treatment Works the undulating grass surfaced golf course extended south to the properties lining the north-eastern side of Hastings Parade. The pertinent features of the cliff face below the golf course were as follows:

- North of the round tower feature, the central section of the cliff face contained a continuous sub-horizontal shale band (maximum height about 2m) and appeared to have been preferentially eroded to form overhang features within the sandstone at above the shale band. Intermittent overhang features (which extended back a maximum horizontal of about 2m) were recorded towards the top of the cliff. Undercut features were noted immediately above the stepped wave cut platform and appeared to extend back a maximum horizontal distance of about 3m.

- South of the round tower feature the sub-vertical cliff face had a stepped profile with numerous blocks of sandstone (typically boulder size) scattered along the wave cut platform at the base of the cliff and on flat bench features over the lower portion of the cliff face.

- Over the central portion of the cliff face what appeared to be a steeply inclined shear zone sloped down to the south-east at about 45°. The shear zone was between about 1m and 1.5m wide and comprised fractured sandstone and dolerite inclusions within a clayey matrix.

- South from the central portion of the cliff face a sub-vertical gully feature (at least about 1m wide) orientated approximately 350° to 010° was intermittently present within the top surface of the stepped cliff top profile. The gully feature represents an igneous dyke that has been preferentially eroded; the dyke extended south below the rear yards of properties lining the eastern side of Hastings Parade. From the shear zone at the rest of the cliff a steep track (with a poor condition ladder section towards the base) zig-zagged down the cliff face to the wave cut platform; occasional fishermen and Sydney Water personnel were observed along the wave cut platform over the course of our inspections.

- A sub-vertical sided gully feature orientated approximately 120° was recorded over the southern central portion of the cliff face. The crest of the gully within the golf course comprised a curved sub-vertical face (about 2m high) which revealed sandy fill material; a PVC ‘ag’ pipe discharged from the rear (western) end of the sub-vertical sand face.

3.6 North Bondi and Ben Buckler Headland

The descriptions below should be read in conjunction with Figure 3 and Plates 15 to 21.

The sub-vertical cliff face was characterised by an intermittent wave cut platform lining the base of the cliff. Numerous detached blocks (ranging up to 5m x 5m x 3m in size) covered the majority of the wave cut platforms.

From the southern end of Bondi Golf Course the crest of the cliff was lined by the rear yards and buildings within Numbers 154 to 208 Hastings Parade. At the intersection of Hastings Parade and Bay Street the cliff line returned to the south-west and formed the southern end of Ben Buckler Headland. The south-east and south-west facing portion of cliff line was lined by the rear yards and buildings within Numbers 1, 3 and 5 Bay Street, 154 to 158 Brighton Boulevard and the lookout area at the southern end of Ramsgate Avenue (which comprised access roads, walkways, a car park and a grass surfaced area). From the lookout area, the remainder of the cliff face returned to the north and was lined by buildings and structures within Numbers 83 to 111 Ramsgate Avenue then the rock pool and paved surrounds.

The observations of the residential properties were limited. However, where observations were possible and based on our previous inspections for private property owners (under separate commissions), we note the following:

- Currently, the rear yard fence lines and/or timber deck structures, in-ground pool and concrete block or brick landscape structures within properties along Hastings Parade generally either extended to the edge of the cliff or were set-back between about 1m and 4m.

- At No. 208 Hastings Parade, the sandstone masonry and brick fences and building walls were typically set-back less than 0.5m from the edge of the cliff. Overhang features were located over the upper portion of the cliff face, at least about 4m below the crest of the cliff.

- At No. 1 Bay Street the metal fence line was set-back about 0.5m from the edge of the cliff. Approximately 1.5m below the crest of the cliff an overhang feature was present which extended back horizontal distances ranging between about 0.5m and 2m.

- Sandstone masonry walls lined the southern and south-eastern cliff edge margins of Numbers 154 to 158 Brighton Boulevard; a rendered garage within No. 158 also lined the edge of the cliff. Approximately 3m below the crest of the cliff, the cliff face below No. 158 comprised a shotcrete face between about 1.5m and 2m height. To the north-east of the shotcrete face an overhang feature which extended back a maximum horizontal distance of about 3m was recorded and appeared to be supported by a brick underpin and rock bolts.

- Adjacent to No. 107 and 111 Ramsgate Avenue the Ben Buckler Amateur Fish Club concrete paved boat ramp extended north down to the foreshore. The western side of the boat ramp was supported by a sandstone masonry wall (maximum height about 1.7m). A portion of the sandstone wall had been eroded to form an undercut portion about 1.4m long, 0.7m high and which extended back a maximum horizontal distance of about 0.8m.

- Portions of buildings and sandstone masonry walls (maximum height about 3m) over the western end of Numbers 89 to 111 Ramsgate Avenue were founded on sandstone bedrock, either on the top surface of the cliff face or the stepped wave cut platform surface.

- A concrete box culvert (about 1.9m wide and 2.4m high) extended south from the rock pool. The top surface of the box culvert formed a walkway adjacent to the western side of Numbers 83 to 87 Ramsgate Avenue.

- The rock pool was lined by concrete walls ranging between about 0.5m and 2.2m high and accessed from the walkway by concrete steps. Sections of the concrete were spalling and hairline to 3mm wide cracks were recorded over the pool walls and steps.

The pertinent features of the cliff face over this portion of the study area were as follows:

- The southern, western and northern site boundaries of Numbers 154, 156 and 158 Hastings Parade, respectively were lined by stepped sub-vertical cliff faces lining the sides and rear of a gully feature orientated approximately 115°. The upper portion of the rear (western) gully face comprised a sandstone masonry wall with a concrete stormwater pipe (about 0.6m diameter) pipe which discharged down the gully. The gully was infilled with numerous large detached sandstone blocks (maximum size about 7m x 4m x 3m) and localised areas of sandy fill materials with building rubble and various other forms of litter and man-made debris.

- The cliff face adjacent to Numbers 164 and 166 Hastings Parade contained a gully feature similar to the one described along the golf course to the north and represents a portion of the southern extension of the igneous dyke.

- The intermittent undercut feature present over the eastern and south-eastern facing portions of the North Bondi/Ben Buckler Headland ranged between about

1m and 3m height and extended back horizontal distances ranging between about 0.5m and 4m.

- The cliff top overhangs present over the eastern, south-western and south-eastern facing portions of the North Bondi/Ben Buckler Headland extended back a maximum horizontal distance of about 3m.

- The cliff faces (maximum height about 4m) below the western end of Numbers 89 to 105 Ramsgate Avenue contained undercut features which ranged between about 0.5m and 3m high and extended back a maximum horizontal distance of about 3m. Selected lengths of the undercuts were supported by parallel sandstone masonry walls founded on the sandstone bedrock wave cut platform surface.

3.7 South Bondi to Tamarama

The descriptions below should be read in conjunction with Figure 4 and Plates 22 to 26.

The sub-vertical cliff face was characterised by an intermittent wave cut platform lining the base of the cliff. A number of detached blocks (ranging up to about 2m x 2m x 1m in size) covered selected portions of the wave cut platforms.

The rendered Bondi Baths and Bondi Icebergs Club (Icebergs) complex lined the cliff face over the western end of the southern side of Bondi Beach. The coastal path extended south-east from the Icebergs building and initially comprised a stepped timber walkway supported on concrete columns then a concrete paved undulating pathway which curved around minor headlands and bay features to Mackenzies Bay just north of Tamarama Beach. From the western end of Mackenzies Bay the pathway was set-back between about 2m and 5m from the cliff edge and extended south around the headland to Tamarama Surf Life Saving Club (SLSC). The SLSC was set-back at least 5m from the edge of the cliff.

The seaward side of the portion of the coastal path west from the northern end of Hunter Park to below the north-western portion of Marks Park was supported by a sandstone masonry wall (maximum height about 1.5m). Occasional sections of the wall (about 0.6m high and 1m long) were missing and the wall backfill had been eroded out to form an undercut section which extended back a maximum horizontal distance of about 1.1m.

The seaward side of the walkway lining the northern side of Mackenzies Bay was supported by a sandstone masonry wall (about 2m maximum height) which was founded on a sandstone bedrock face. The base of the wall was about 4m above the wave cut platform. The rock face contained a number of undercuts which ranged between about 0.6m and 1.2m height and which extended back horizontal distances ranging between about 1m and 4m.

The southern side of the path adjacent to the cliff face below the north-eastern portion of Marks Park was lined by stacked sandstone and sandstone masonry walls (maximum height about 1.3m).

Upslope and to the south of the Icebergs building the north-eastern side of Hunter Park comprised a vegetated area (about 6m high) that sloped down to the north-east at about 40°; occasional sandstone bedrock outcrops and detached sandstone blocks were evident over the vegetated area.

The north-western and south-eastern portions of Marks Park sloped down to the north-east and south-east at about 30°, respectively and comprised uneven grass surfaced slopes. The north-eastern portion of Marks Park was supported by a concrete wall (maximum height about 3m).

Stepped and vegetated slopes also lined the northern and southern portions of the rear (western) section of Mackenzies Bay.

The remainder of this portion of the study area comprised stepped and sub-vertical cliff faces end The Mackenzies Point headland comprised a grass surfaced cliff top reserve. The pertinent features of the cliff faces were as follows:

- Below the eastern end of Hunter Park an overhang extended over the path which was about 5m high and extended back a maximum horizontal distance of about 3m. An overhang adjacent to the seaward side of the path at this location was about 4m high and extended back a maximum horizontal distance of about 3m.

- Undercut sections were present beneath the wall supporting the pathway and were a maximum of about 1.5m high and extended back a maximum horizontal distance of about 2.5m.

- Over the remainder of the length of pathway the cliff face beneath the pathway contained undercut features which ranged between about 1m and 3m height and extended back a maximum horizontal distance of about 5m. Occasionally the undercuts were eroded to form a 'cave' feature approximately 5m x 5m and which appeared to extend back a horizontal distance of about 10m.

- Adjacent to the southern side of the pathway below the north-eastern portion of Marks Park an overhang feature (about 1.4m high and which extended back a maximum horizontal distance of about 3m) was supported by rendered masonry underpins about 0.35m x 0.35m.

- Two overhanging sections of the cliff face below the walkway beyond the southern side of Marks Park were supported by concrete columns (0.3m diameter) about 2m maximum height.

- The base of the cliff face forming the headland between Mackenzies Bay and Tamarama Beach contained a near continuous undercut feature which was typically about 1m high and extended back horizontal distances of between about 0.5m and 4m. The crest of this cliff face contained a number of cliff top overhangs.

- The cliff face below the Tamarama SLSC contained an overhang feature that was about 15m long, a maximum height of about 5m and which extended back a maximum horizontal distance of about 5m. To the west, the cliff face lined the northern side of Tamarama Beach and the base of the cliff contained undercut features with some extremely weathered sandstone revealed and which was partially covered by a poor condition concrete facing.

- Behind the timber life guard station the cliff face contained an undercut feature about 2.5m high and which extended back a horizontal distance of about 1m. The undercut was supported by a sandstone masonry wall which was showing signs of erosion.

3.8 Tamarama to Bronte

The descriptions below should be read in conjunction with Figure 5 and Plates 26 to 29.

The sub-vertical cliff face that extended south around the headland from the southern side of Tamarama Beach to the northern side of Bronte Beach had a stepped profile. The base of the cliff only had short lengths of wave cut platform adjacent to the interfaces with the beach areas.

The upper vegetated portion of the cliff face was lined by a footpath located on the seaward side of Bronte Marine Drive and Tamarama Marine Drive. A timber fence lined the cliff edge side of the footpath and traces of stacked sandstone walls (maximum height about 3m) below the fence appeared to support portions of the footpath.

The vegetated upper portion of the cliff face generally sloped down to the foreshore at about 30°.

Intermittent undercut features were recorded over the central and lower portions of the cliff face and were typically of the order of 1m to 4m high and extended back horizontal distances of between about 0.5m and 2.5m.

The cliff face lining the northern side of Bronte Beach contained a continuous undercut ranging between about 1.5m and 4.5m height and which extended back a maximum horizontal distance of about 6m. Concrete steps extended up to Bronte Marine Drive from the eastern end of this cliff face. The base of the cliff face lining the northern side of Bronte Beach comprised the roof of a concrete stormwater box culvert (about 3.2m wide); the box culvert discharged onto the beach.

3.9 Bronte to Boundary Street

The descriptions below should be read in conjunction with Figure 5 and Plates 30 to 34.

The sub-vertical cliff face extended south around the headland from the rock pool located over the southern side of Bronte Beach to the southern side of Waverley Cemetery (Boundary Street).

The rock pool comprised sandstone masonry and concrete walls (maximum height about 2.5m) founded on sandstone bedrock. Vertical hairline to 6mm wide cracking was evident and some erosion along the cracks was also recorded.

A concrete walkway extended up from the south-eastern corner of the rock pool and the seaward side was lined by a timber fence. Above the walkway, the crest of the cliff was lined by Calga Place and a timber and chain link fence was set-back at least 1m from the edge of the cliff. In this area there were damaged sections of the fence line.

The base of the sub-vertical cliff face between the Bronte Beach rock pool and the northern end of the timber and stainless steel coastal walkway structure within Calga Reserve was lined by a wave cut platform. Occasional detached blocks of sandstone were evident over the wave cut platform surface.

Calga Reserve comprised an undulating grass surfaced area which sloped down to the east to the cliff edge at between about 2° and 20°. Localised steeper slopes extended down to the south from the south-eastern corner of Calga Reserve, and the north-eastern portion of Calga Reserve was essentially flat with sandstone bedrock outcropping at the surface. A timber lookout structure was located at the commencement of the coastal footpath steps at the edge of the cliff.

The thickly vegetated cliff top area lining the eastern side of Calga Reserve comprised an uneven thickly vegetated area that generally sloped down to the east at between about 30° and 45°. Stepped areas were also evident and appeared to comprise short portions of dilapidated log retaining walls, concrete walls or stacked sandstone walls, detached blocks of sandstone and outcrop faces. The sub-vertical outcrop faces below the upper sloping vegetated portion contained undercut features that had been provided with rendered concrete block blade wall underpins.

The sub-vertical cliff face over the length of the coastal walkway was dissected by a gully that has been infilled by repeated end tipping of sandy materials excavated from burial sites, plus building rubble comprising large fragments of concrete, sandstone and portions of masonry walls. Numerous large fragments of concrete covered the toe area of the fill slope. The concave fill slope was about 14m high and sloped down to the west at a maximum of about 65°. The fill slope surface contained sub-vertical portions of between about 1m to 2m vertical height formed by back scarp features of localised near surface instability affecting the fill materials. Sub-vertical cliff faces (about 12m to 13m high) formed the northern and southern margins of the gully.

The coastal walkway structure was set-back at least 1m from the edge of the cliff and an 'on-grade' paved portion lined the western side of the aforementioned gully feature. The landward side of the coastal path was lined by the sandstone masonry Waverley Cemetery boundary walls.

At the base of the stepped northern portion of the coastal path, the top of the cliff was characterised by a relatively flat area which comprised a 'hanging swamp' protected nature reserve area. South of the 'hanging swamp' to the gully feature the sandstone cliff top was generally flat. To the south of the gully feature, the cliff top generally comprised a stepped sandstone surface with sloping surfaces controlled by cross bedding planes which sloped down to the north at between about 10° and 20°.

Cliff top overhang features extended back horizontal distances of between about 3m and 5m.

The base of the cliffline contained undercut features of between about 1.5m and 8m in height and which extended back horizontal distances of between about 4m and 10m.

Close to the southern end of the study area, a sub-vertically sided gully feature about 1m to 5m wide extended the full height of the cliff and appeared to be controlled by parallel sub-vertical joint planes orientated about 120°. The gully extended back into the cliff and based on published geological mapping information this gully is believed to mark the line of an igneous dyke

4 POTENTIAL GEOTECHNICAL HAZARDS

Based on the results of our inspections, the potential geotechnical hazards for the study area are summarised and outlined below.

1. Instability of overhang features, wedges or blocks within sandstone bedrock over the following sections of cliff faces:
   A. Crest of Cliff Face
   B. Cliff Face
   C. Base of Cliff Face
2. Instability of natural soil foreshore slopes.
3. Instability of fill foreshore slopes.
4. Instability of Existing Stabilisation Measures.
5. Instability of Retaining Structures.

The potential geotechnical hazards 2 to 5 relate to selected areas of the study area and potential geotechnical hazard 1A to 1C relate to the entire length of cliff faces within the study area.

5 GEOTECHNICAL ASSESSMENT

5.1 General Overview

The foreshore cliff faces comprise Hawkesbury Sandstone bedrock of Triassic age (around 245 to 210 million years ago). The sandstone represents vast quantities of sediments transported into the Sydney Basin by rivers flowing from the south-west and west. Uplift and deformation of the Sydney basin area probably occurred over several phases and was associated with the opening of the Tasman Sea approximately 60 to 80 million years ago. The present elevation of the Sydney Basin region was achieved by about the mid Tertiary (about 40 to 50 million years ago). This uplift and deformation has led to the observed pattern of jointing and faulting in the rock mass and the intrusion of igneous dykes generally along the dominant joint planes; typically in an approximately east-west direction but with some trending approximately north-south. Weathering and erosion of the sandstone continued with sea level fluctuations from the early Quaternary onwards (commencing around 1.8 million years ago), associated with glacial and inter-glacial periods (sea level low high periods, respectively), having a significant effect on the formation of the present day coastline.

Current sea levels are believed to have been reached around 6,400 years before present (ybp). A glacial period between about 17,000 and 25,000 ybp is believed to have caused a sea level fall of around 130m below present day levels. At the end of this glacial period ice melted and sea levels rose to their current levels in-filling the valleys that now form the Sydney Harbour foreshore we see today. This cycle of varying sea levels is believed to have occurred several times over during the Quaternary (about 1.6 million years ago to present day). The wave cut platforms observed along the bases of many of the cliff faces are likely to have developed during inter-glacial sea level highs. It is believed that the current cliff faces were located some 90kms to the east and the erosion over the last 70 million years has resulted in the recession of the cliff faces to the present coast line.

The cliff faces have revealed Hawkesbury Sandstone bedrock assessed to be typically distinctly to slightly weathered and generally of medium strength. It is evident that the topography of the majority of the cliff faces has been influenced by the orthogonal joint sets identified during our inspections. At the base of the majority of the cliff faces was a sandstone wave cut platform, generally covered by an abundance of detached blocks from previous rock falls. The blocks were either elongated or "cubic" and suggest that they were derived from collapse of cliff face overhangs and wedges of sandstone bedrock within the cliff face. The detached sandstone blocks ranged in size from less than about $1m^3$ to in excess of about $20m^3$ in size and their shape and size appeared to be controlled by the two principal orthogonal joint sets. The principal orthogonal joint sets were generally orientated approximately north-south (ranging between about 335° and 035°) and east-west (ranging between about 95° and 120°). We note that some sections of the cliff lines were densely vegetated and at other locations the base of the cliffs sections were covered by detached sandstone blocks, which prevented more detailed observations.

The cliff faces have revealed a number of relatively weak features, including extremely weathered sandstone (XWS) seams, shale bands, fractured zones, weathered igneous dykes, shear zones and clay bands.

The differential weathering and erosion (by wave and wind action) of relatively weak sub-horizontal XWS, shale bands and clay bands over the cliff faces and, in particular the basal portion of the cliff faces is a likely mechanism of cliff line collapses due to undercutting of the more competent sandstone above, followed by toppling and/or basal shear. Basal shear could also occur due to the surcharge load of the less eroded overlying rock mass.

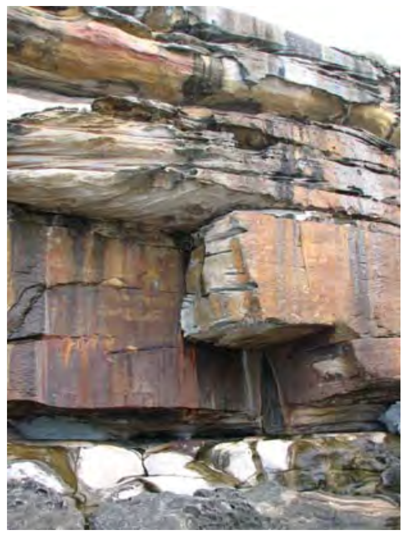
Such a failure mechanism would result in the material collapsing down the cliff face onto the wave cut platform:

Alternatively, such a failure mechanism would result in the material collapsing and being captured on a flat bench within the cliff face

In addition, similar differential weathering and erosion (by wave and wind action) of relatively weak sub-horizontal XWS, shale bands and clay bands over the cliff faces has led to the formation of the numerous cliff top overhang features identified along the study area and undercut features at the base and over the cliff face.

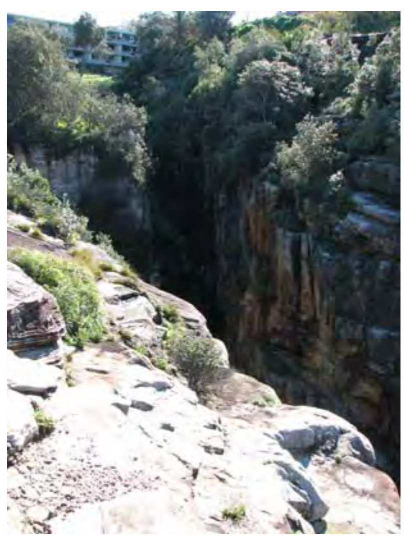
Collapse of these overhangs has also occurred in a similar manner as described above. The rock fall at Diamond Bay is believed to be due to such a mechanism although the form of the rock fall appears to have been controlled at least in part by the relatively weaker mass strength of the rock rather than strictly controlled by the orthogonal joint sets. This may be due to localised alteration of the rock mass due to the elevated heat associated with the intrusion of the igneous dykes to the north and south of the rock fall. The alteration appears to have weakened the overall strength of the rock mass in this area.

It also appears that the overhangs migrate up the cliff face with sub-horizontal bedding partings and the orthogonal joint sets controlling each successive block collapse.



The presence of detached blocks along the wave cut platform is considered to be the product of “recent” (in geological terms) and previous collapses from the cliff faces. The items associated with potential geotechnical hazards 1A, 1B and 1C (as described in Section 4, above) would be associated with continuation of these natural processes. In addition, we note the presence of sub-vertical features, in particular the igneous dykes and/or shear zones identified over the study area which manifest themselves as sub-vertical sided gullies with similar orientations to the orthogonal joint sets described above.

The igneous dykes and shear zones typically comprise extremely weathered/residual soil materials which are often fractured. These relatively weaker sub-vertical features are also affected by differential weathering and erosion (by wave and wind action) and have resulted in the formation of the cliff line gullies described in Section 3, above.



Crucial to these processes is the rate at which they are occurring. Little evidence is available on the overall rates of occurrence of these forms of instability and the resultant rate of recession of the cliff face. Nonetheless, it is clear that rock falls do occur. There is some evidence on the rates of erosion in the paper by Young & Wray (2000), Reference 2. Rates of erosion, which may be summarised from observations given in this paper, are:

- For recession of the coastal escarpment south of Nowra, the "maximum possible rate of 170m/Ma" has been determined. This corresponds to 0.17mm per year.
- The most rapid rates of recession occur in gorges (usually where undercutting occurs on weaker bands due to waterfall erosion effects) being about 2 to 3km/Ma. This corresponds to a rate of 2mm to 3mm per year.
- There is no data given for the Hawkesbury Sandstone clifflines in the Sydney area.
- Dragovich (Reference 3) refers to weathering of softer beds causing undercutting of cliff lines. Dragovich quotes Roy as determining an average rate of undercutting of 2mm to 5mm per year, but that the overall rate would be slower due to rock falls protecting the softer bed.
- It is clear from the discussion in Young & Wray, that there will be significant variations in the rates of weathering and that extreme events, such as tsunamis or higher past sea levels during interglacial periods can also be relevant to the rates of recession and cliff line formation.

Based on the above, we could expect the Hawkesbury Sandstone cliff face to erode at less than 1mm per year. As an example, we have taken a 2m x 2m x 2m size block on the side of the cliff face. We have assumed that the block will remain 'stable' provided the horizontal extent of undercutting is less than 1m in from the outer face. Adopting a relatively high (or conservative) rate of erosion of 1mm per year, it would take at least 1000 years before the block would fall from the cliff face.

In addition, we note that the erosion rate of the relatively weaker shale bands, fractured zones, weathered igneous dykes, shear zones and clay bands would be at accelerated rates as indicated by the formation of gullies and undercut sections of the cliff faces identified over the study area and indicated by the above photographs.

Additional triggers to collapse of blocks and wedges over the cliff faces are:

- Water pressure developed in the sub-vertical joints behind potentially unstable blocks or wedges during and following rainfall events.

- Localised tree root jacking where tree roots penetrate joints at the rear of blocks and wedges over the cliff faces.

In our opinion, the elements most at risk are:

- Persons (such as residents, recreational users, Council employees etc) at the base of the cliff face, such as users of the Bronte or North Bondi Pools, users of Bondi, Tamarama and Bronte beach areas adjacent to the cliff faces, people on the wave cut platforms (e.g. fishermen, divers, etc).

- Persons (such as residents, recreational users or Council employees) along the crest of the cliff face within private properties, reserve areas, and the coastal paths.

- Sections of residences (including yard areas) located on the tops of the cliffs and situated close to the edge of the cliff faces.

- Existing pathways, stormwater infrastructure, rock pools and services (below and above ground).

The steeply sloping upper vegetated portions of selected cliff faces over the study area are likely to comprise thin natural residual soils with bands of weathered bedrock and/or detached sandstone blocks derived from localised collapse of unstable cliff face features but with little, if any, downslope movement.

Instability of soil slopes over sections of the cliff faces was also identified at selected locations. Instability of such slopes is typically governed by one or more of the following factors; over-steep batter slopes, elevated water pressures within the soils associated with ineffective drainage systems and/or surface water run-off and erosion of the toe of the slopes by wave action.

The most dramatic example of a number of these factors affecting one slope was the fill batter slope infill of the gully feature within Waverley Cemetery.

On-going creep of such soil materials is typical over moderate and steeply sloping sites such as selected areas within the study area. Creep would be indicated by uneven slope surfaces and/or localised sub-vertical back scarp features. In addition, concentrated discharge of surface run-off after heavy or prolonged periods of rainfall can cause localised instability

Existing retaining walls within selected areas close to the cliff edge were in variable condition but generally of relatively low height. Collapse of such walls would be relatively localised and the collapse debris could impact the foreshore area below and/or the retained surface above.

It is important to be mindful that rock falls, soil slumps etc can occur at anytime and it would be difficult to impossible to predict when the identified potential hazards will occur. Also, we cannot predict when an extreme or unusual event may occur (such as an earthquake or 1 in 100 year rainfall event etc) and what impact it would have on the stability of the identified potential hazards.

5.2 Risk Analyses

On the basis of the above and using the information obtained from our site observations we provide below our qualitative assessment of risk to both life and property and additional comments in relation to the potential impact of climate change on risk levels. The assessment has been carried out in accordance with the guidance provided in Reference 1.

5.2.1 Risk To Property

We note that strict application of the assessment of consequences to property as outlined in Reference 1 and Appendix A requires that "the approximate cost of damage be expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures." We have applied this to our assessment of risk to property but have also included our assessment of the risk of damage to the dwelling and any landscape structures. We also note that we have not made any attempt to quantify any loss of property value due to loss of land as a result of cliff face collapse.

In determining consequences we have used the following information:

- An assumed typical property value (land plus dwelling) based on the median house sale price as outlined in 'The Sun Herald Property Guide 2010', dated 21 February 2010. We acknowledge that this is a relatively crude method of value assessment and will vary depending on the size of the lot and whether or not the residence is an individual unit or a detached house and the affect on property values of an ocean view aspect. However, we consider that this is a reasonable estimate.

- Typical costs for building repairs and rock face stabilisation measures.

The attached Table A summarises our qualitative assessment of each potential landslide hazard and of the consequences to property should the landslide hazard occur, under existing conditions. Based on the above, the qualitative risks to property have been determined. In this regard, we have identified general locations over the study area.

We have assumed "Possible" and "Likely" assessed likelihoods for potential geotechnical hazards 1A, 1B, 1C, 2 and 3 in order to provide a range of risk levels for these potential geotechnical hazards. This is also an attempt to assess the impact of potential geotechnical hazards along the study area cliff faces of variable condition and stability that were, in many instances, unable to be accurately assessed from the base or crest of the cliff faces.

For potential geotechnical hazards 4 and 5 our assessment has been based on their current condition and our knowledge of the installation of a number of the rock face stabilisation measures over the area of the coastal path adjacent to Waverley cemetery. The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A.

Table A indicates that the assessed risk to property typically varies between Very Low and Low which would be considered to be ‘acceptable’, in accordance with the criteria given in Reference 1.

However, Table A indicates that the assessed risk to property for No. 2 Queenscliff Road and No. 7 Pavilion Street was Moderate which would be considered to be ‘tolerable’, in accordance with the criteria given in Reference 1.

5.2.2 Risk To Life

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal, vulnerability and evacuation factors that have been adopted are given in the attached Table B together with the resulting risk calculation. We note that we have assumed that the affected person is immediately above or below the specific hazard when it occurs (i.e. spatial probability of 1), which would be regarded as conservative, particularly over longer areas of the study area.

Our assessed risk to life for the person most at risk, under existing conditions ranges between about 10^{-4} and 10^{-10}. These would be considered to be ‘tolerable’ and ‘acceptable’, respectively in relation to the criteria given in Reference 1. We note that the ‘tolerable’ risk levels are associated with instability of lookouts/vantage points adjacent to coastal paths and above or below a particular feature when fishing, sun baking or walking on the wave cut platform. The 'tolerable' risk levels were associated with:

- A resident within their dwelling above the cliff top feature (with an assumed "Likely" assessed likelihood),

- A person on a cliff top lookout/vantage point (above a potential cliff face hazard) adjacent to the coastal paths (with an assumed "Likely" assessed likelihood), and

- A person fishing, sun baking or walking on the wave cut platform below a potential cliff face hazard over the general area of the cliff face (with an assumed "Likely" assessed likelihood).

We reiterate that our assessment of risk to life has been based on one person being affected. Where more than one person is affected, the level of risk calculated for an individual increases by a factor equivalent to the number of people present; i.e. for 2 people the level of risk would increase from say 1×10^{-4} to 2×10^{-4} etc. For areas where significant numbers of people may congregate, say over the surrounds of Bronte Pool then the risk levels associated with the increased numbers of people being present (say 10) would, for this study, be over the upper end of the 'tolerable' range.

5.2.3 Impact of Climate Change on Risk Levels

It is difficult to assess the potential impact of predicted more intense storm events as a result of climate change and sea level rise on cliff face stability and no formal studies, to our knowledge, have been completed. However, it is considered reasonable to assume that more intense storm events and elevated sea levels will result in elevated erosion rates over a greater height of cliff face. In addition, salt spray from wave action can be expected to affect a greater height of cliff face. On this basis, in terms of the NSW State Government recommended sea level rise planning benchmarks of an increase above 1990 mean sea levels of 40cm by 2050, and 90cm by 2100 we would expect elevated erosion rates to affect the lower portion of the cliff face.

With regard to our assessed risk levels, increased erosion rates affecting the base of the cliffs would probably lead to localised collapse of undercuts over the basal areas of the cliff lines and potential instability of the toe area of the fill batter slope within Waverley Cemetery. On this basis, we would expect that risk levels associated with potentially unstable features at the bases of the cliffs and the fill batter slope within Waverley Cemetery would, over the next 50 to 100 years, be best represented by the levels of risk to life and property associated with a 'Likely' likelihood. In this instance risk levels would generally be at 'tolerable' levels. However, if the on-going monitoring over the study area reveals evidence of more rapid rates of erosion and deterioration of potentially unstable features, such that a revised risk assessment indicates 'unacceptable' risk levels, then detailed geotechnical advice will be required to determine the scope and extent of stabilisation measures.

5.2.4 Previous Geotechnical Advice

We note that we have completed a number of previous geotechnical assessments over the study area. In general, our advice in reports from 2002 onwards included similar risk assessments as outlined above together with specific detailed design advice. A brief summary is outlined below.

Dover Heights

In 1987 we advised on construction of a pool close to the cliff edge within the rear yard of a property on the eastern side of Lola Street. A cliff top overhang and a shale band within the cliff face about 6m below the crest of the cliff were identified. Our advice recommended drilling of boreholes to investigate the location of sub-vertical defects within the rear yard which would control the lateral (westward) extend of potential instability. Based on the results of the investigation an appropriate location and design of the pool could be determined; we were not involved in any further aspect of the pool development.

In 1999 minor instability of a garden bed fill slope had occurred which affected the rear yard of a property that lined the cliff top adjacent to the small reserve area at the eastern end of Hunter Street. Advice was provided in relation to construction of a new retaining wall to support the fill.

In 2002 we provided geotechnical advice in relation to construction of a portion of the coastal footpath structure within Dover Heights reserve adjacent to the eastern end of George Street. We recommended that the path structure be founded on bedrock and that further assessment of the cliff face be undertaken. We have no information as to whether such further assessment of the cliff face was undertaken.

North Bondi

We have completed geotechnical assessments for proposed developments at 162, 164 to 166, 174, 176, 178 and 180 Hastings Parade. The reports identified various geotechnical features within the upper portion of the cliff faces lining the eastern boundaries of the properties, including cliff top overhangs, sub-vertical defect planes orientated approximately north-south and the above described eroded igneous dyke forming the cliff line gully feature.

Our risk assessments generally indicated a similar range of risk levels to both life and property as outlined in Sections 5.2.2 and 5.2.2.

Our advice in relation to reducing risk levels included some or all of the following measures:

Construct buildings and structures to the west of the zone of influence of any sub-vertical defect orientated approximately north-south (i.e. parallel to the cliff face). Typically the zone of influence related to the lateral spacings of the sub-vertical defects (a maximum of about 4m). Alternatively, structures were recommended to be constructed to cantilever over the defect plane and anchored into sound bedrock to control potential over-turning.

Provide permanent fences or other barriers to prevent access to the cliff edge.

Property owners visually monitor the existing cliff line and the cliff top area to check for signs of damage to existing structures and fences, tension cracks developing at the site surface etc. If there are causes for concern then access to the cliff top should be restricted and further geotechnical advice immediately sought. We recommended that any potentially affected neighbours also be informed.

South Bondi

We completed numerous site inspections during the upgrade of the Bondi Icebergs building. The rock cut face lining the south-western side of the site was of poor quality and extensive stabilisation measures were installed included rock anchors, pattern rock bolting and placement of reinforced shotcrete.

Bronte to Tamarama

Between 2003 and 2004 we provided geotechnical advice to Waverley Council on construction of the new walkway lining the cliff top. Our assessment also included advice on rock face stabilisation measures including rock bolts, trimming off overhangs and/or underpinning overhangs (including the large overhang below Tamarama SLSC). We note that the walkway has been constructed (with all footings founded on bedrock) but no rock face stabilisation measures were implemented. However, no formal risk assessment was requested or completed.

Extension of Bondi to Bronte Coast Walk; Waverley Cemetery

Between 1999 and 2005 we completed a number of geotechnical investigations and risk assessments and were also involved in witnessing some of the construction of the coastal walk structures.

We identified the most appropriate location of the walkway in terms of risk levels which included bridging over narrow gully features (believed to be eroded igneous dykes), the off-set of the 'on-grade' portion of the walkway around the gully feature should any instability of the fill batter slope occur. We also provided advice on stabilisation measures for rock face undercuts and anchoring of the lookout structure at the crest of the slope within Calga Reserve.

During the construction works there was a report of instability of the fill batter slope within the gully feature although we were not requested to provide any advice. We note that during our recent site inspections there was a temporary 'cyclone' mesh fence lining the crest of the fill batter slope. Furthermore, there appeared to have been a change in the slope profile and a greater amount of concrete debris at the base of the batter slope. It appears that on-going erosion of the 'finer' grained soil materials from the toe of the batter slope is exposing more concrete debris and further instability can be expected over time.

5.2.5 Previous Work By Waverley Council

During the progress meeting held at WC offices on 15 June 2011, WC reported that they had conducted a visual assessment of the cliff faces in 1982. Subsequently, a copy of photographs of the foreshore cliff lines taken in 1982 by WC was provided to the undersigned. The photographs were taken before the works were completed and we understand that the works involved hydraulic rock splitting of overhang features in public reserve areas.

Study of the photographs indicates that the suspected cliff face rock fall at Diamond Bay had occurred before the photographs were taken; the rock debris covering the base of the cliff was evident on the photographs. On this basis, the large cliff face failure had occurred at least 29 years before the current assessment.

5.2.6 Additional Comments

It is recognised that due to the many complex factors that can affect a site, the subjective nature of a risk analysis, and the imprecise nature of the science of geotechnical engineering, the risk of instability for a site cannot be completely removed. It is, however, essential that risk be reduced to at least that which could be reasonably anticipated by the community in everyday life and that landowners be made aware of reasonable and practical measures available to reduce risk as far as possible. Hence, risk cannot be completely removed, only reduced, as removing risk is not currently scientifically achievable.

In preparing our recommendations given below we have assumed that no activities on surrounding land which may affect the risk on the subject site would be carried out. We have further assumed that all buried services are, and will be regularly maintained to remain, in good condition.

We provide below geotechnical advice and recommendations in relation to landslide risk management measures for identified potential geotechnical hazards and the study area as a whole which, if adopted in full, would maintain risk at current 'tolerable' and 'acceptable' levels. These recommendations form an integral part of the Landslide Risk Management Process.

However, it is a matter for Council how they wish to sequence and implement the advice outlined in the following Section 6. In this regard, we note that the advice will be used by WC as a guide to development of emergency responses to safeguard the community, public and private assets and future development from severe coastal storm events.

6 GEOTECHNICAL ADVICE

6.1 General

The cliff faces within the study area represent natural features within a foreshore landscape and any associated cliff face instability is also a natural phenomena. Stabilisation of individual potential hazards is likely to be uneconomical, particularly for persons at risk along the base of the cliff faces or crest and upper portions of the cliff face (where access is possible). However, in relation to private property, the potential for cliff face instability may be detrimentally impacted by construction of buildings and other structures due to additional surcharge loadings, unless appropriate additional support to the potentially unstable features has been provided. Furthermore, cliff face instability solely due to natural processes may also detrimentally impact private property and the occupants.

6.2 Landslide Risk Management

We provide below various measures which, if adopted in full, seek to manage and, where appropriate, maintain risk to at least generally considered ‘tolerable’ levels. These recommendations form an integral part of the Landslide Risk Management Process and will also assist in guiding the development of emergency responses to safeguard the community, public and private assets and future developments from severe coastal storm events.

6.2.1 Warning Signs and Fence Lines

We consider that signs warning of potential cliff face instability be provided in all publicly accessible areas along the crest and basal areas of the cliff face within the study area. We recommend that they be placed at least at the following locations:

- Bronte Pool,
- The wave cut platforms at the base of the cliff faces where public access is currently feasible. Based on our observations this would include at least the following locations:
    - North Bondi and Ben Buckler Headland,
    - Adjacent to the walkway below the north-eastern side of Marks Park, and
    - Mackenzies Bay to Tamarama.

- The cliff edges where public access to the cliff top and the base of the cliff is feasible. Based on our observations this would include at least the following locations:
    - All publicly accessible areas adjacent to Rodney Reserve and Dover Heights Reserve, including any additional localised areas at the ends Hunter Street, Myuna Road, Weonga Road.
    - Bondi Golf Course,
    - Mackenzies Bay to Tamarama, and
    - Waverley Cemetery at the gully location.

We also recommend where yard areas extend to the cliff edge and in particular Marne Street, Jensen Avenue, Ray Street, the eastern ends of Hunter Street, Lola Road, the eastern end of Douglas Parade, the central portion of Wentworth Street (all in Dover Heights) and 154 to 208 Hastings Parade, 1, 3 and 5 Bay Street and 154 to 158 Brighton Boulevard, North Bondi property owners be advised to post similar warning signs.

We note that there are selected sections of public reserves and other cliff top areas accessible by the public within the study area that are not fenced off. We recommend that fence lines be provided in addition to the above mentioned warning signs at the following locations:

- Reserve area lining the northern side of Diamond Bay, and
- Waverley Cemetery at the gully location.

Consideration should be given to advising Bondi Golf Course to erect fences or, at the very least warning signs on the basis of the advice in this report.

In addition, the damaged fence line at the crest of the cliff over the northern end of Calga Place should also be repaired to prevent access on to the cliff top area above Bronte Pool.

6.2.2 Monitoring

Council should monitor the identified potential hazards within the study area on an annual basis and after periods of prolonged or heavy rainfall in order to assess existing conditions and any indications of deterioration such as debris/boulders on the beach, rock platform, damage to pathways etc.

Based on previous studies of available rainfall data in relationship to landslide events, in particular the a study carried out for the Pittwater area (Walker 2007, Reference 4), we provide the following tentative definition of heavy rainfall and prolonged rainfall:

- Heavy Rainfall: at least 100mm of rainfall in one day, and
- Prolonged Rainfall: at least 150mm of rainfall over a 5 day period.

These amounts of rainfall represent 1 in 2 year occurrences for the Pittwater area and are considered reasonable for the Waverley area.

It is imperative that such monitoring be formally documented and that the required frequency of reporting (and to whom) is clearly defined. Where incidents of instability have occurred within the monitoring period then where possible we suggest that Council provide relevant details within the monitoring reports. These details would include the date of the incident, the weather conditions on the day and leading up to the incident, a location plan, photographs and dimensions of the specific feature (block sizes, width and length of landslip features, crack widths etc would also need to be recorded). The monitoring reports should be provided to the geotechnical engineers so that if there are any causes for concern, further advice can be provided. The need for additional site specific stabilisation measures can then be better assessed.

In addition, on a 5 yearly basis, a detailed assessment should be undertaken by an experienced engineering geologist/geotechnical engineer to assess current conditions with regard to the contents of this report and the on-going inspection monitoring reports. This is of particular importance for the rock below the Bronte to Tamarama coast path where our recommendations in relation to rock face stabilisation measures were not adopted.

6.2.3 Sewer and Stormwater Drainage

All existing subsurface drains, sewers and any other water carrying pipelines must be subject to ongoing and regular maintenance by the respective owners. We recommend that this include checking for leaks and damage to the water carrying pipelines by a plumber or similarly qualified professional and appropriate maintenance and repairs completed without delay. Such maintenance should be carried out at no more than five yearly intervals, commencing within 12 months of issue of this report; including provision of a written report confirming scope of work completed and identifying any required remedial measures.

6.3 Proposed Developments Within Private Properties

We note that in accordance with the NSW Government's Draft Coastal Risk Management Guide for each cliff line our report is required to identify the "immediate hazard line" for cliff erosion from the design storm event plus any zone of reduced foundation capacity together with identification of the "hazard lines" for the 2050 and 2100 planning periods. However, during a progress meeting held at WC offices on 15 June 2011 it was agreed that in terms of the hazards associated with cliff lines, properties and parcels of land would be identified as being at various levels of risk. This information would then be incorporated in WC's planning guidelines so that particular sites could be identified and inform WC's planning advice to property owners, particularly in relation to assessment of new developments. On this basis we recommend that should new developments be proposed at the following properties tabulated below, a geotechnical assessment of the cliff face should be required as a mandatory condition of the Development Application process:

Address	Plan	Lot
1 Jensen Avenue	SP 687	-
3 Jensen Avenue	DP 7334	56
5 Jensen Avenue	DP 7334	55
7 Jensen Avenue	DP 7334	54
9 Jensen Avenue	DP 400406	A
11 Jensen Avenue	DP 400406	B
1 Marne Street	SP 30361	-
3 Marne Street	DP 9080	26
5 Marne Street	DP 9080	27
7 Marne Street	DP 19254	24
9 Marne Street	DP 19254	25
11 Marne Street	DP 16375	26
28 Macdonald Street	DP 23177	1
2 Ray Street	DP 976698	37
4 Ray Street	DP 417665	2
51B Lancaster Road	DP 102084	B
20 Hunter Street	DP 7044	15
21 Hunter Street	DP 7044	36
36 Myuna Road	DP 4623	27

Address	Plan	Lot
31 Myuna Road	DP 4623	26
2 Lola Road	DP 10675	12
4 Lola Road	DP 10675	13
6 Lola Road	DP 10675	14
8 Lola Road	DP 10675	15
10 Lola Road	DP 10675	16
12 Lola Road	DP 10675	17
14 Lola Road	DP 10675	18
12 Douglas Parade	DP 10675	22
21 Douglas Parade	DP 348567	B
23 Douglas Parade	DP 619746	232
25 Douglas Parade	DP 45691	1
25 Douglas Parade	DP 619746	231
8 Wentworth Street	DP 382476	B
10 Wentworth Street	DP 382476	A
12 Wentworth Street	DP 336579	A
14 Wentworth Street	DP 10090	42
16 Wentworth Street	DP 10090	41
18 Wentworth Street	DP 10090	40
20 Wentworth Street	DP 10090	39
22 Wentworth Street	DP 10090	38
24 Wentworth Street	DP 404933	A
26 Wentworth Street	DP 404933	B
26 Wentworth Street	DP 10090	36
28 Wentworth Street	DP 10090	35
30 Wentworth Street	DP 10090	34
32 Wentworth Street	DP 10090	33
34 Wentworth Street	DP 343564	B
36 Wentworth Street	DP 19465	1
38 Wentworth Street	DP 19465	2
154 Hastings Parade	SP 7883	-
156 Hastings Parade	SP 2178	-
158 Hastings Parade	DP 786	13
160 Hastings Parade	DP 443203	A
162 Hastings Parade	DP 443203	B
164 Hastings Parade	DP 439182	A
166 Hastings Parade	DP 439182	B
168 Hastings Parade	SP 4413	
170 Hastings Parade	DP 786	17
172 Hastings Parade	DP 786	18
174 Hastings Parade	DP 308590	1
176 Hastings Parade	DP 308590	2
178 Hastings Parade	DP 308590	3
180 Hastings Parade	DP 308590	4
182 Hastings Parade	DP 413583	A
184 Hastings Parade	DP 413583	B
186 Hastings Parade	DP 398119	D
188 Hastings Parade	DP 398119	C
190 Hastings Parade	DP 2905	-
192 Hastings Parade	DP 786	25
194 Hastings Parade	SP 21330	-
196 Hastings Parade	DP 106649	A
198 Hastings Parade	DP 106649	B
200 Hastings Parade	DP 441398	X
202 Hastings Parade	DP 515178	1
204 Hastings Parade	SP 4507	-
206 Hastings Parade	SP 16026	-
208 Hastings Parade	DP 320739	3
1 Bay Street	SP 249	-

Address	Plan	Lot
3 Bay Street	DP 1123754	1
3A Bay Street	DP 1123754	2
5 Bay Street	DP 331848	C
154 Brighton Boulevard	SP 30225	-
156 Brighton Boulevard	DP 786	6
158 Brighton Boulevard	SP 12058	-
83 Ramsgate Avenue	SP 16621	-
85 Ramsgate Avenue	DP 344571	-
89-91 Ramsgate Avenue	DP 343534	10
95 Ramsgate Avenue	SP 905	-
97 Ramsgate Avenue	SP 1160	-
105 Ramsgate Avenue	SP 1159	-
107 Ramsgate Avenue	SP 5170	-
111 Ramsgate Avenue	SP 22198	-
31 Gaerloch Avenue	DP 9842	36
31 Gaerloch Avenue	DP 9842	35
31 Gaerloch Avenue	DP 9842	34
29 Gaerloch Avenue	DP 415974	B
27 Gaerloch Avenue	DP 415974	A
Tamarama SLSC	DP 1052115	7046

6.4 Conclusions

On the whole, the cliff faces and foreshore slopes within the study area may be regarded as having a 'tolerable' to 'acceptable' risk of instability. In our opinion, we consider that under existing conditions and into the near future on-going monitoring by Council and periodic geotechnical assessments are an appropriate method of landslide risk management.

In relation to proposed private developments within properties identified as having a cliff edge frontage, inclusion of a mandatory requirement for geotechnical assessment of the cliff face within the property will assist in managing risk levels.

As outlined above, it is difficult to predict when an instability event (rock fall etc) will occur and in such a foreshore setting some instability is inevitable as, in the main, such instability events are a result of natural processes. However, the assessed risk levels under existing conditions do not warrant construction of any significant stabilisation measures at this stage although as conditions change over time, and possibly exacerbated by the impact of climate change on storm events and sea levels, such measures may become necessary.

Construction of stabilisation measures along the cliff faces could result in a poor aesthetic solution if not completed carefully. Negative community reaction may also occur. For private land owners affected by potentially unstable sections of cliff faces such considerations may be of lesser importance compared to protection of themselves and their property. In terms of regulatory authorities we are unclear as to who would have overall responsibility for approval of cliff face stabilisation measures. We recommend that Council seek appropriate advice in this regard.

6.5 Further Geotechnical Work

With regard to the above range of landslide risk management measures, the following further geotechnical work may be required:

- Review of monitoring reports.
- Re-assess the need for stabilisation measures in light of the above monitoring reports.
- Geotechnical re-assessment on a five yearly basis.

7 GENERAL COMMENTS

It is possible that the subsurface soil, rock or groundwater conditions encountered during implementation of the landslide risk management measures may be found to be different (or may be interpreted to be different) from those inferred from our surface observations in preparing this report. Also, we have not had the opportunity to observe surface run-off patterns during heavy rainfall and cannot comment directly on this aspect. If conditions appear to be at variance or cause concern for any reason, then we recommend that you immediately contact this office.

Should you have any queries regarding this report, please do not hesitate to contact the undersigned.

Reviewed By:

Paul Roberts
Senior Associate

Agi Zenon
Senior Associate

For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

REFERENCES

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2. Young, R W & Wray, R A L (2000). *"The Geomorphology of Sandstone in the Sydney Region"*, published in "Sandstone City", ed. McNally, G H & Franklin, B J, Monograph No 5, Geological Society of Australia, pp.55-73.

3. Dragovich, D (2000). *"Weathering Mechanisms and Rates of Decay of Sydney Dimension Sandstone"*, published in "Sandstone City", ed. McNally, G H & Franklin, B J, Monograph No 5, Geological Society of Australia, pp.74-82.

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TABLE A
SUMMARY OF RISK ASSESSMENT TO PROPERTY UNDER EXISTING CONDITIONS

Potential Landslide Hazard	**1: Instability of overhang or undercut features, wedges or blocks within sandstone bedrock over the cliff faces**			**2: Instability of natural soil foreshore slopes**	**3: Instability of fill foreshore slopes**	**4: Instability of Existing Stabilisation Measures**	**5: Instability of Existing Retaining Structures**
	A: Crest of Cliff Face	**B: Cliff Face**	**C: Base of Cliff Face**				
Locations Affected	**A. Private Properties at the edge of cliff faces. B. Public Reserve Areas. C. Coastal paths/lookouts. D. Rock Pools. E. Stormwater Infrastructure.**	**C. Coastal paths. D. Rock Pools. E. Stormwater Infrastructure.**	**C. Coastal paths. D. Rock Pools. E. Stormwater Infrastructure.**	**A. Private Properties at the edge of cliff faces. B. Public Reserve Areas. C. Coastal paths/lookouts.**	**B. Public Reserve Areas. C. Coastal paths/lookouts.**	**A. Private Properties at the edge of cliff faces. C. Coastal paths/lookouts.**	**A. Private Properties at the edge of cliff faces. B. Public Reserve Areas. C. Coastal paths/lookouts.**
Assessed Likelihood	Possible Likely	Possible Likely	Possible Likely	Possible Likely	Possible Likely	Unlikely	Possible
Assessed Consequences	Minor (A & E) Insignificant (B, C & D)	Minor (E) Insignificant (C & D)	Insignificant	Insignificant	Insignificant	Minor	Minor (A, B & C)
Risk	(Possible Likelihood) Moderate (A & E) Very Low (B, C & D)	(Possible Likelihood) Moderate (E) Very Low (C & D)	(Possible Likelihood) Very Low	(Possible Likelihood) Very Low	(Possible Likelihood) Very Low	Very Low	Moderate (A, B & C)
	(Likely Likelihood) Moderate (A & E) Low (B, C & D)	(Likely Likelihood) Moderate (E) Low (C & D)	(Likely Likelihood) Moderate	(Likely Likelihood) Low	(Likely Likelihood) Low		
Comments	For Potential landslide hazards 1A and 1B, instability assumed not to affect crest area of cliff face. For Potential landslide hazards 4 and 5, retaining walls and stabilisation measures assumed to be engineer designed. Assessed consequences based on approximate cost of damage as a percentage of the approximate value of the land affected, where applicable, the approximate value of the property (land and the structure affected) and, in all cases, the cost of remediation.						

TABLE B
SUMMARY OF RISK ASSESSMENT TO LIFE UNDER EXISTING CONDITIONS

Potential Landslide Hazard	1: Instability of overhang features, wedges or blocks within sandstone bedrock over the cliff faces	2: Instability of natural soil foreshore slopes	3: Instability of fill foreshore slopes	4: Instability of Existing Stabilisation Measures	5: Instability of Existing Retaining Structures
Locations Affected	A. Private Properties at the edge of cliff faces. B. Public Reserve Areas. C. Coastal paths/lookouts. D. Rock Pools. F. Beach areas.	A. Private Properties at the edge of cliff faces. B. Public Reserve Areas. C. Coastal paths/lookouts. F. Beach areas.	B. Public Reserve Areas. C. Coastal paths/lookouts. F. Beach areas.	A. Private Properties at the edge of cliff faces. C. Coastal paths/lookouts.	A. Private Properties at the edge of cliff faces. B. Public Reserve Areas. C. Coastal paths/lookouts. F. Beach areas.
Assessed Likelihood	Possible Likely	Possible Likely	Possible Likely	Unlikely	Possible
Indicative Annual Probability	10^{-3} 10^{-2}	10^{-3} 10^{-2}	10^{-3} 10^{-2}	10^{-4}	10^{-3}
Persons at Risk	1. Person in yard area above feature. 2. Person in residence above feature. 3. Person on edge of cliff in public area. 4. Person on coastal path. 5. Person below feature in pool area, on rock platform and beach at base of cliff.	1. Person in yard area above feature. 2. Person in residence above feature. 3. Person on edge of slope in public area. 4. Person on coastal path. 5. Person below feature on rock platform and beach at base of cliff.	1. Person on edge of slope in public area. 2. Person on coastal path. 3. Person below feature on rock platform and beach at base of cliff.	1. Person in yard area above feature. 2. Person on edge of slope in public area. 3. Person on coastal path. 4. Person below feature on rock platform at base of cliff.	1. Person in yard area above feature. 2. Person in residence above feature. 3. Person on coastal path. 4. Person below feature on rock platform or public reserve area below feature.
Duration of Use of Area Affected (Temporal Probability)	1. 0.02 2. 0.3 3. 0.03 (lookout) or 0.06 (fishing) 4. 4×10^{-5} 5. 0.06 (sun baking, fishing) 4×10^{-5} (walking adjacent to pool, on rock platform and beach at base of cliff)	1. 0.02 2. 0.3 3. 0.03 (lookout) or 0.06 (fishing) 4. 4×10^{-5} 5. 0.06 (sun baking, fishing) 4×10^{-5} (walking on rock platform and beach at base of cliff)	1. 0.03 (lookout) or 0.06 (fishing) 2. 4×10^{-5} 3. 0.06 (sun baking, fishing) 4×10^{-5} (walking on rock platform and beach at base of cliff)	1. 0.02 2. 0.03 (lookout) or 0.06 (fishing) 3. 4×10^{-5} 4. 0.06 (fishing) 4×10^{-5} (walking on rock platform at base of cliff)	1. 0.02 2. 0.3 3. 4×10^{-5} 4. 0.06 (sun baking, fishing) 4×10^{-5} (walking on rock platform or public reserve area below feature)
Probability of Not Evacuating Area Affected	1. 0.4 2. 0.4 3. 0.4 4. 0.1 5. 0.5 (sun baking, fishing) 0.1 (walking adjacent to pool & on beach) 0.4 (walking on rock platform)	1. 0.1 2. 0.1 3. 0.4 4. 0.1 5. 0.5 (sun baking, fishing) 0.1 (walking on beach) 0.4 (walking on rock platform)	1. 0.4 2. 0.1 3. 0.5 (sun baking, fishing) 0.1 (walking on beach) 0.4 (walking on rock platform)	1. 0.4 2. 0.4 3. 0.1 4. 0.5 (fishing) 0.4 (walking on rock platform)	1. 0.1 2. 0.1 3. 0.1 4. 0.5 (sun baking, fishing) 0.1 (walking on public reserve area) 0.4 (walking on rock platform)
Vulnerability to Life if Failure Occurs Whilst Person Present	1. 1 2. 0.4 3. 1 4. 0.1 5. 1	0.1 (low volume of landslip debris)	0.1 (low volume of landslip debris)	1. 1 2. 1 3. 0.1 4. 1	0.1 (low volume of landslip debris)
Risk for Person Most at Risk	(Possible Likelihood) 1. 8×10^{-6} 2. 4.8×10^{-5} 3. 1.2×10^{-5} (lookout) 2.4×10^{-5} (fishing). 4. 4×10^{-10} 5. 3×10^{-5} (sun baking, fishing), 6×10^{-6} (walking adjacent to pool & on beach) 2.4×10^{-5} (walking on rock platform) (Likely Likelihood) 1. 8×10^{-5} 2. 4.8×10^{-4} 3. 1.2×10^{-4} (lookout) 2.4×10^{-4} (fishing). 4. 4×10^{-9} 5. 3×10^{-4} (sun baking, fishing), 6×10^{-5} (walking adjacent to pool & on beach) 2.4×10^{-4} (walking on rock platform)	(Possible Likelihood) 1. 2×10^{-7} 2. 3×10^{-6} 3. 1.2×10^{-6} (lookout) 2.4×10^{-6} (fishing). 4. 4×10^{-10} 5. 3×10^{-6} (sun baking, fishing), 6×10^{-7} (walking adjacent to pool & on beach) 2.4×10^{-6} (walking on rock platform) (Likely Likelihood) 1. 2×10^{-6} 2. 3×10^{-5} 3. 1.2×10^{-5} (lookout) 2.4×10^{-5} (fishing). 4. 4×10^{-9} 5. 3×10^{-5} (sun baking, fishing), 6×10^{-6} (walking adjacent to pool & on beach) 2.4×10^{-5} (walking on rock platform)	(Possible Likelihood) 1. 1.2×10^{-6} (lookout) 2.4×10^{-6} (fishing). 2. 4×10^{-10} 3. 3×10^{-6} (sun baking, fishing), 6×10^{-7} (walking adjacent to pool & on beach) 2.4×10^{-6} (walking on rock platform) (Likely Likelihood) 1. 1.2×10^{-5} (lookout) 2.4×10^{-5} (fishing). 2. 4×10^{-9} 3. 3×10^{-5} (sun baking, fishing), 6×10^{-6} (walking adjacent to pool & on beach) 2.4×10^{-5} (walking on rock platform)	1. 8×10^{-7} 2. 1.2×10^{-6} (lookout) 2.4×10^{-6} (fishing). 3. 4×10^{-11} 4. 3×10^{-6} (sun baking, fishing), 1.6×10^{-9} (walking on rock platform)	1. 2×10^{-7} 2. 3×10^{-6} 3. 1.2×10^{-6} (lookout) 2.4×10^{-6} (fishing). 4. 4×10^{-10} 5. 3×10^{-6} (sun baking, fishing), 4×10^{-10} (walking on public reserve area) 1.6×10^{-9} (walking on rock platform)

Notes

Sun baking, fishing occupancy based on 3hrs per day for 6 months of year: about 0.06,
Standing above or below cliff top overhang, occupancy based on 30 minutes per day for 9 months of year: about 0.03,
Person walking on access path, beach or rock platform at base of cliff, occupancy based on average walking rate of 4 seconds per 5m length per day for 9 months of year: about 4×10^{-5},
Person in rear yard, occupancy based on 0.5hrs per day: about 0.02, and
Person in residence, occupancy based on 8hrs per day: about 0.3.

Ref: 24648ZR Figure 1

Not to scale

Image sourced from *Google Earth*

SITE LOCATION PLAN

To be read in conjunction with text of report.

24648ZR • FIGURE 1

Not to scale

Image sourced from *Google Earth*

SITE LOCATION PLAN

To be read in conjunction with text of report.

24648ZR • FIGURE 2

Not to scale

Image sourced from *Google Earth*

SITE LOCATION PLAN

To be read in conjunction with text of report.

Not to scale

Image sourced from *Google Earth*

SITE LOCATION PLAN

To be read in conjunction with text of report.

Ref: 24648ZR Figure 5

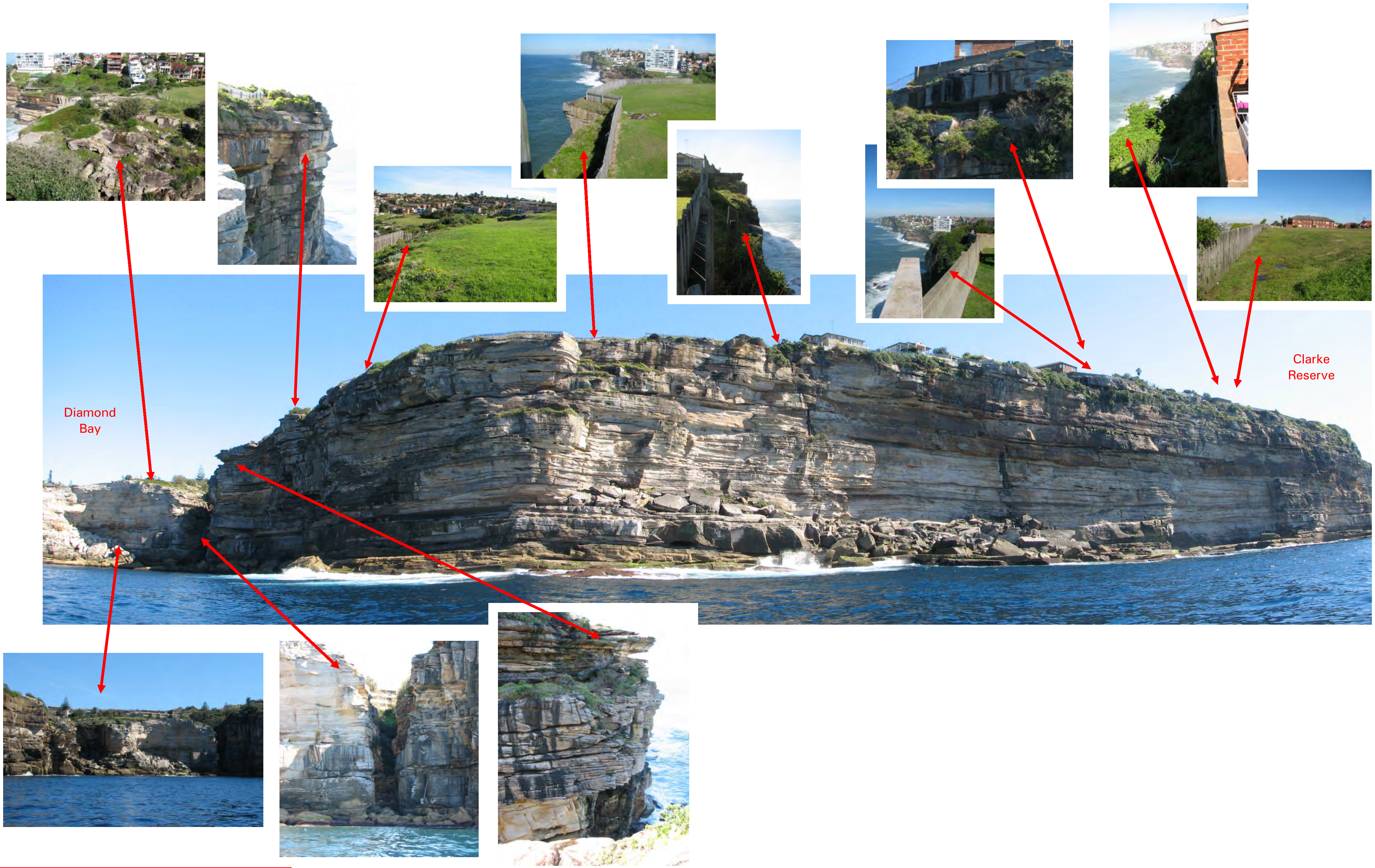
Not to scale

Image sourced from *Google Earth*

SITE LOCATION PLAN

To be read in conjunction with text of report.

24648ZR • FIGURE 5



To be read in conjunction with text of report.

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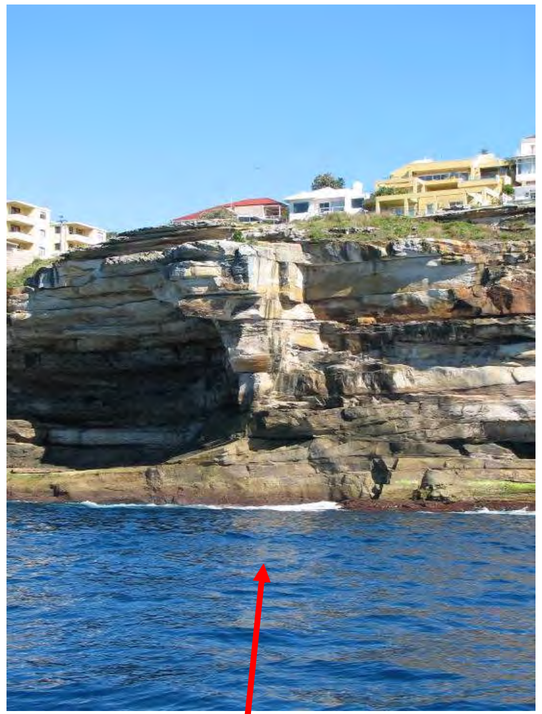
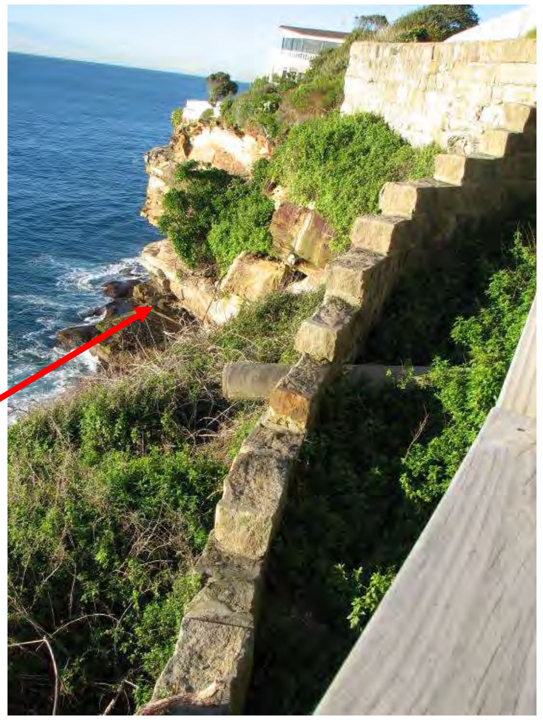
Dover Heights Reserve

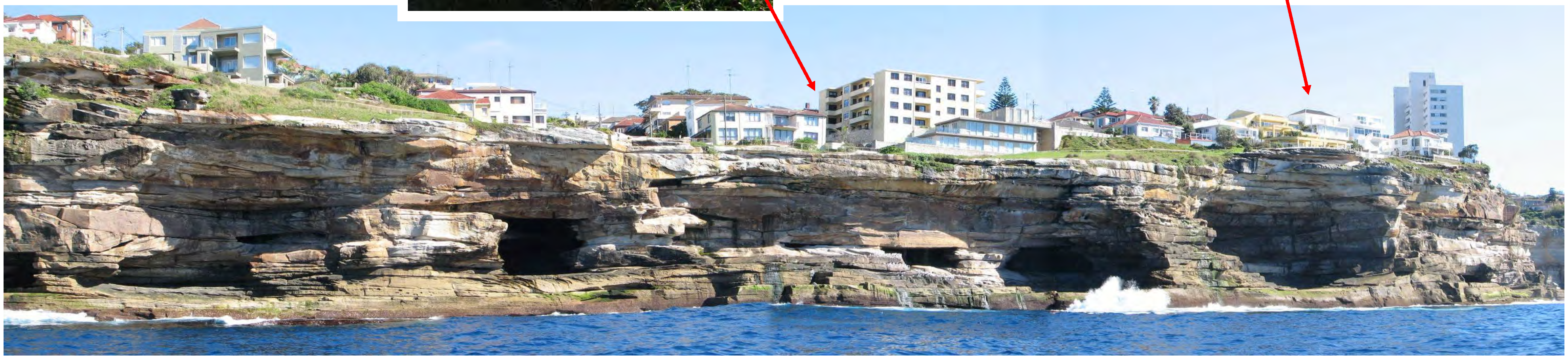




Diamond Bay

To be read in conjunction with text of report.

Dover Heights Reserve

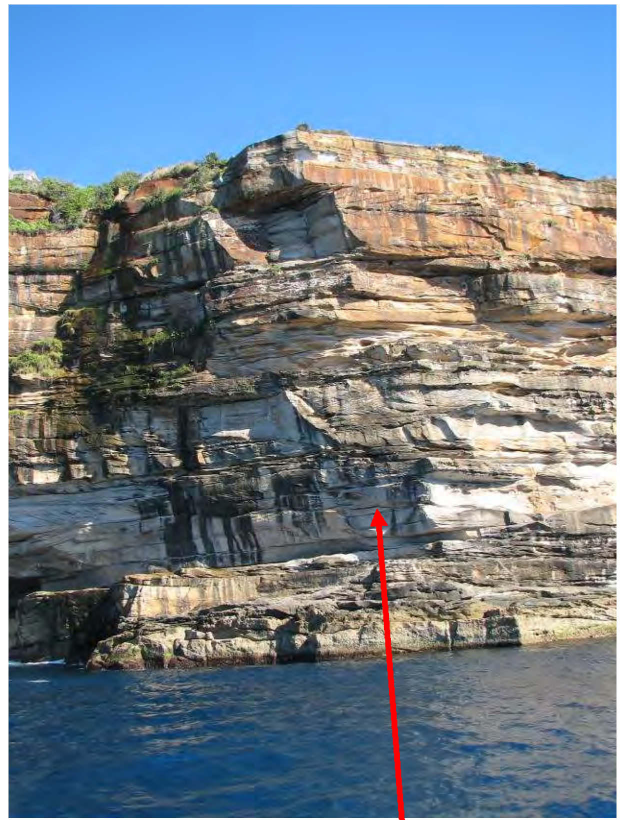


To be read in conjunction with text of report.

Dover Heights
Reserve

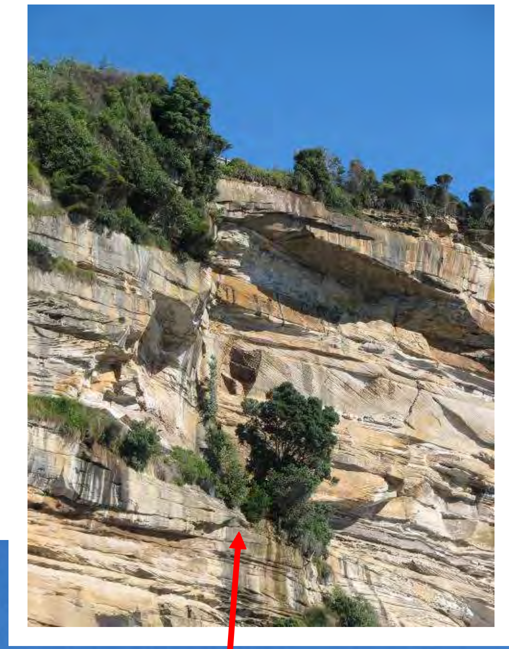
Eastern end of Lancaster Road

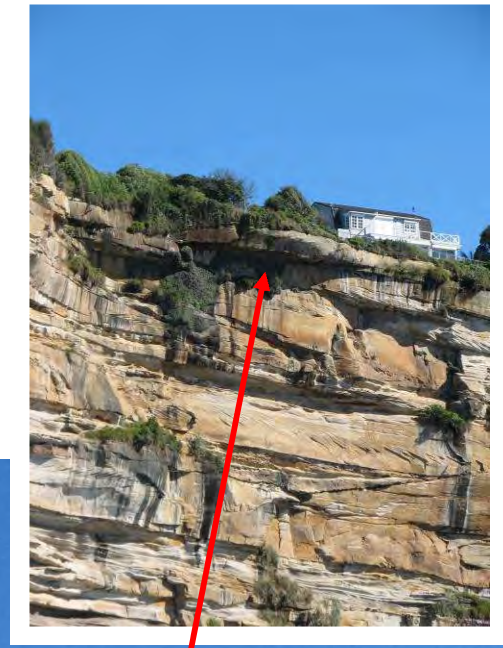
Coastal path over southern end of Dover Heights Reserve



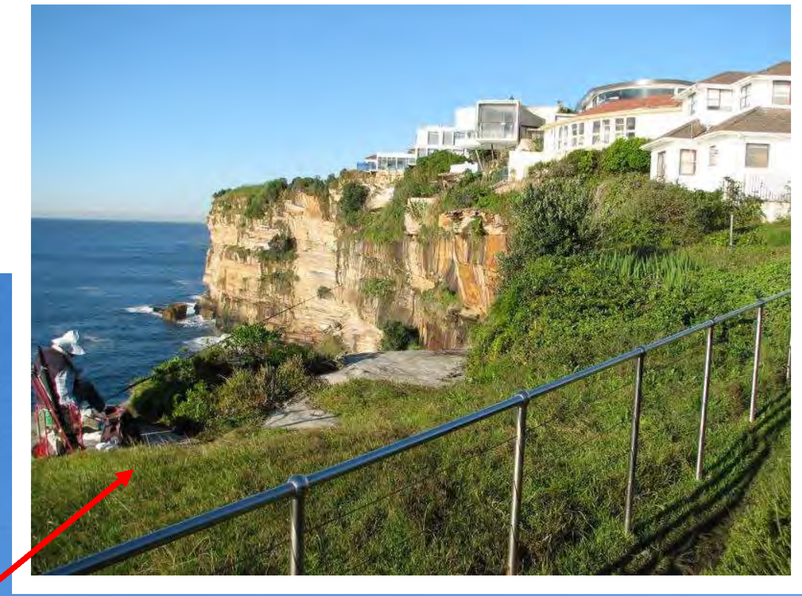
To be read in conjunction with text of report.

Northern end of Rodney Reserve





Eastern end of Lancaster Road

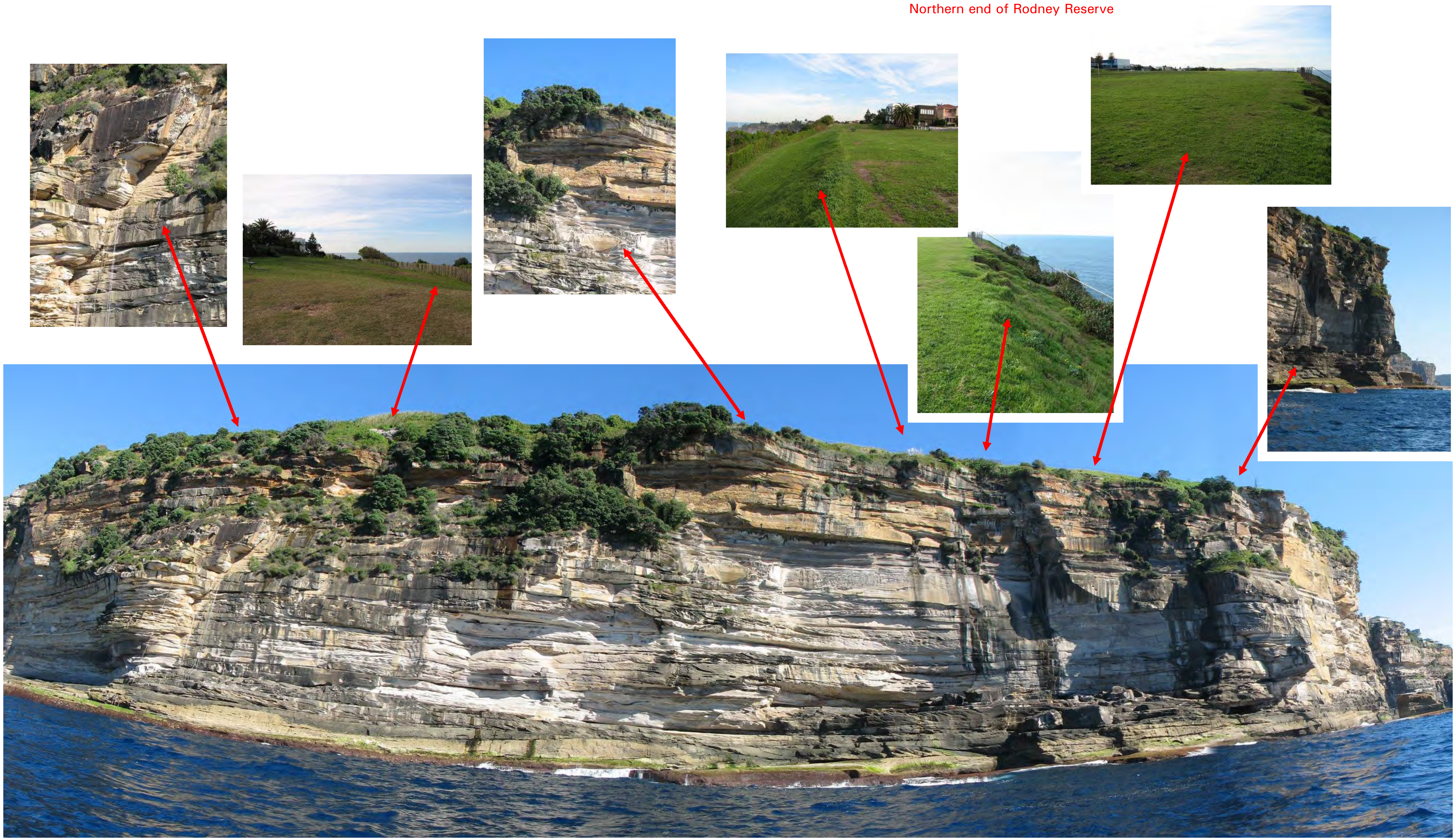


Cliff face below Hunter Street, Lancaster & Myuna Road

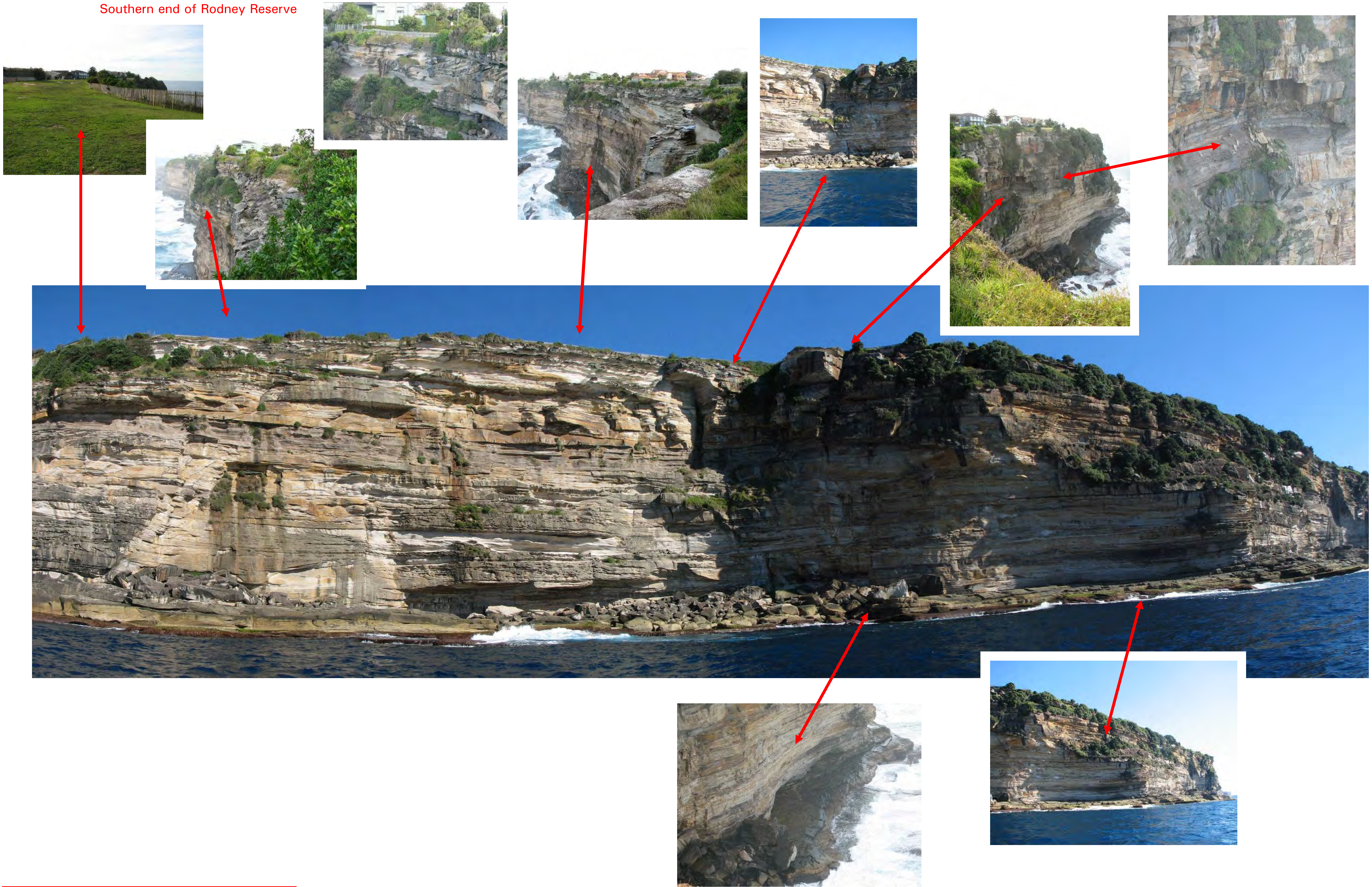
To be read in conjunction with text of report.

Sports field over northern end of Rodney Reserve

To be read in conjunction with text of report.



To be read in conjunction with text of report.



To be read in conjunction with text of report.

Cliff face below Wentworth Street & Lola Road

To be read in conjunction with text of report.

Hugh Bamford
Reserve

Bondi Sewage Treatment Works

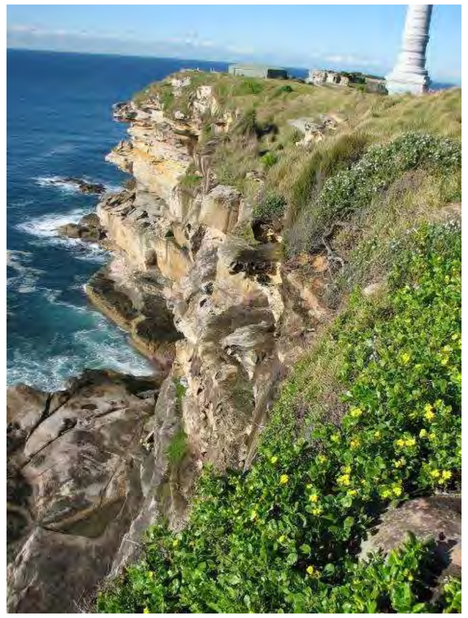
Trace of igneous dyke

Cliff face below Bondi Golf Club

To be read in conjunction with text of report.

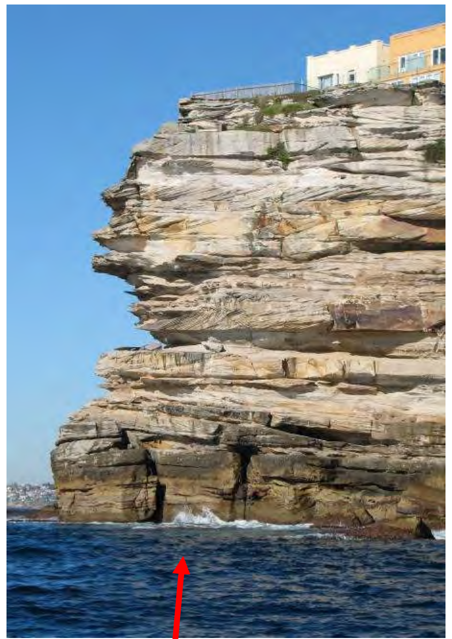
Cliff face below Bondi Golf Club

Instability of soil slope

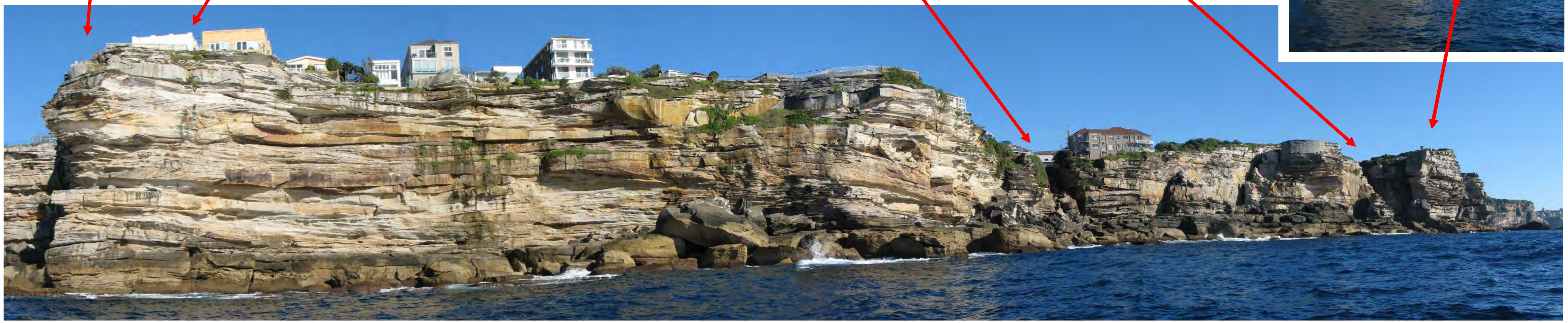


To be read in conjunction with text of report.

To be read in conjunction with text of report.



Gully adjacent to 154, 156 & 158 Hastings Parade



Cliff face below Hastings Parade

To be read in conjunction with text of report.

To be read in conjunction with text of report.