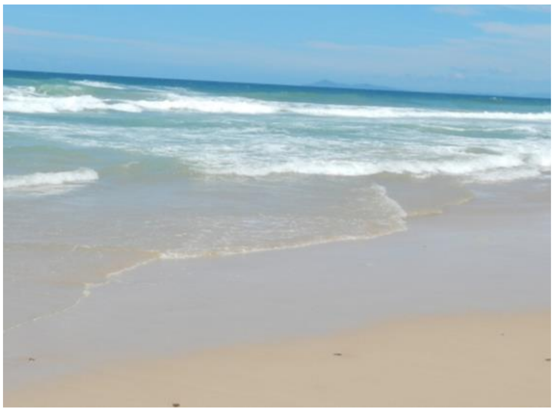
Greater Taree
City Council






Risk Assessment to Define Appropriate Development Setbacks and Controls in Relation to Coastline Hazards at Old Bar

Prepared for Greater Taree Council by Haskoning Australia Pty Ltd

3 April 2014

Issue 2 (Draft)

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1. INTRODUCTION

1.1 Background

Coastal development setbacks in NSW have traditionally been defined through delineation of coastline hazard lines, using a variety of planning periods and hazard zones. However, until recently there has been no known rigorous assessment of the validity of traditional hazard lines in terms of leading to an acceptable risk to property if used as setbacks for new development.

A Collaroy-Narrabeen Beach and Fishermans Beach Coastal Zone Management Plan (CZMP) is being prepared by Haskoning Australia for Warringah Council. As part of that investigation , it was agreed between the study team, Council staff, Councillors, Office of Environment and Heritage (OEH) staff and an external peer reviewer (Mr Bruce Walker of JK Geotechnics) that defining appropriate development setbacks using an acceptable risk approach was valid, reasonable and an improvement on traditional hazard line approaches to defining setbacks. As such, acceptable risk lines were delineated at Collaroy-Narrabeen and Fishermans Beach to define setbacks for future development.

Greater Taree City Council and OEH also supported the use of an acceptable risk approach to assist in defining appropriate development setbacks and controls in relation to coastline hazards at Old Bar, as set out herein. It is emphasised that the setbacks derived herein are applicable to new development. Any setbacks and controls adopted for new development would have no effect on the coastline hazard risk to existing development.

Setbacks were developed for two scenarios, namely for new structures on conventional foundations (such as strip footings or shallow piers) and new structures on piles. It is recognised that although a piled structure may be at an acceptably low risk of damage, other considerations such as loss of surrounding land and the need for access to the structure would make piled development unsuitable as a general control at Old Bar given the extent of at-risk development.

1.2 Study Area

The study area for the assessment herein was intended to be the same as that considered in the Haskoning Australia (2013) investigation *Old Bar Beach Coastal Protection Structure Design Investigation*. That is, it was intended that the study area extended along Old Bar Beach from north of Taree Old Bar Surf Life Saving Club in the north to the MidCoast Water exfiltration ponds in the south.

OEH has derived photogrammetric data (shore-normal beach profiles using aerial photography) at Old Bar Beach for various dates between 1940 and 2013. These are arranged into Blocks (each containing a set of parallel profiles), with Blocks 3, 4, 5, and 7 covering Old Bar Beach (moving south to north respectively). However, available 2013 photogrammetric data at Old Bar Beach did not include Block 3, so to enable consistent analysis the study area only extended to the southern end of Block 4 (thus not including the area near the exfiltration ponds). The alongshore length of the study area is about 1.8km.

The extent of the study area in relation to Old Bar Beach is depicted in Figure 1. Note that Racecourse Creek discharges over the beach immediately seaward of Pacific Parade. Taree Old Bar Surf Life Saving Club (SLSC) is also shown. The location of the photogrammetric Blocks at Old Bar Beach is depicted in Figure 2. Profiles are numbered commencing at 1 at the southern end of each Block, increasing moving north. For example, the northern profile in Block 7 is Profile 8.

Figure 1: Study area at Old Bar Beach

Figure 2: Location of OEH photogrammetric profile Blocks at Old Bar Beach, with first and last profiles in each Block numbered

1.3 Scope

The ‘acceptable risk’ setbacks developed herein are based on coastal erosion caused by meteorological events (“coastal storms”) leading to large waves and elevated water levels, and recession due to net sediment loss and sea level rise . Tsunamis, which have rarer frequencies of occurrence and different driving processes to coastal storms[1], have not been considered.

It was assumed that the entire study area was sandy and erodible. That is, there was no consideration of the effect of any non-sandy subsurfaces nor future protective works.

1.4 Framework

The framework of the adopted approach came from the Australian Geomechanics Society (AGS) procedures for landslide risk management (AGS, 2007a, b), which were developed over a period of more than a decade via a Working Group of experts[2], and have been widely applied in geotechnical engineering practice since 2000[3]. The AGS procedures were also subject to peer review and discussion through the AGS Landslides Taskforce, with 23 members.

That is, the AGS procedures can be considered to be an established, recognised and peer reviewed methodology for defining landslide risk for development assessment. With modification to be appropriate for “sandy beach” coastline hazards, it is considered that the same principles of the AGS procedures can be applied to define acceptable risk for beachfront development, as has been undertaken herein.

In *Guidelines for Preparing Coastal Zone Management Plans* (DECCW, 2010a), one of the Coastal Management Principles is to” adopt a risk management approach to managing risks to public safety and assets”. The approach adopted herein is considered to be consistent with DECCW (2010a), and also consistent with the principles of the joint Australian, New Zealand and International Organisation for Standardization Standard AS/NZS ISO 31000:2009, “Risk management - Principles and guidelines” and Australian Standard AS 5334-2013, “Climate change adaptation for settlements and infrastructure - A risk based approach”.

1.5 Recognition of Uncertainty

It is important to recognise that future climate cannot be predicted precisely, and is subject to not only storm variability, but longer term cycles such as the El Nino / La Nina Southern Oscillation, Pacific Decadal Oscillation, and Interdecadal Pacific Oscillation (IPO).

For example, Helman (2007) has postulated that during negative Interdecadal Pacific Oscillation (IPO) phases, the NSW coast experiences wet periods, major floods, sea level above the long term trend and coastal erosion. Using an 11 year Chebychev filter annual series from 1871 to 2008 (Folland, 2008), a significant past continuous negative IPO period was from 1945 to 1977, and IPO was positive from 1978 to 2000, returning to negative from 2001 to 2008 (although the nature of the filtering was

[1] Tsunamis are typically driven by earthquakes, landslides, large scale collapse of volcanic islands, or asteroid impacts, with earthquakes being the dominant tsunami source in NSW for events more frequent than 500 year average recurrence interval (Somerville et al, 2009).
[2] Mr Bruce Walker, who peer reviewed the Collaroy-Narrabeen Beach and Fishermans Beach risk assessment, was the Working Group Convenor.
[3] Using preceding AGS documents as discussed in AGS (2007a).

such that the 2004 to 2008 period should be regarded with caution). A return to negative IPO combined with additional future projected sea level rise could lead to a future period of enhanced erosion compared to the 1978 to 2000 period.

Future climate can also not be predicted precisely due to ongoing climate change caused by the enhanced greenhouse effect. Climate change effects such as sea level rise are projected by researchers based on various scenarios as to how greenhouse gases and aerosols will be emitted anthropogenically in the future, that is so called “representative concentration pathways” as described by the Intergovernmental Panel on Climate Change (IPCC), for example in IPCC (2013a). These scenarios represent a range of 21st century climate policies and cannot be precisely predicted as they largely depend on political decisions and economic growth.

Furthermore, storm events more severe than adopted design events can occur, or a structure could remain in place for longer than the design life considered herein (thus potentially being exposed to more severe conditions, for example because sea level rise is projected to be ongoing).

Therefore, it must be recognised that any development landward of a particular “acceptable risk” line is not at zero risk (but at acceptably low risk), and damage may be possible both during and particularly beyond the design life. Council should not (and could not) guarantee that development given consent to be sited landward of a particular “acceptable risk” line would never be damaged by coastal processes.

That stated, the approach developed herein is considered to be reasonable and valid for defining acceptable risk to property for new development in the study area, and an improvement on traditional methods of hazard definition. It is recommended that the CZMP covering the study area is updated at least every 5 to 10 years to enable improved understanding to be incorporated as required.

The assessment herein largely relied on WorleyParsons (2010) for defining coastline hazard parameters based on the scope of work agreed with Council. It is considered that additional analysis of these parameters beyond WorleyParsons (2010) would assist in improving the veracity of the assessment outlined herein.

1.6 Risk to Life

Only risk to property is evaluated herein, not risk to life. In the coastal beach context, risk to life related to development in the study area was considered to be acceptably low as:

- coastal storms (large waves and elevated water levels) are generally foreseeable at least 24 hours in advance, with warnings issued by the Bureau of Meteorology;
- a large component of elevated water levels is astronomical tide, which can be accurately predicted decades into the future;
- erosion would generally be expected to be greatest for a few hours near the peak of the tide;
- the progress of erosion on a beach is visible and perceptible, and would not generally be expected to proceed undetected to damage development;
- it is highly unlikely that a landowner would be occupying a dwelling and would be unaware (or would not have been made aware) that this dwelling was at imminent threat of damage;
- the State Emergency Service (SES), if mobilised, has powers to warn and evacuate residents if required (as does NSW Police); and

- Council could request the SES taking on a Combat Agency role if an actual emergency was occurring and it had not already been mobilised.

These factors mean that residents would have a low probability of occupancy and/or loss of life during an actual storm event that could threaten development, and hence have a low risk to life which would satisfy the acceptance criteria given in AGS (2007a).

Also note that Council has a gazetted Emergency Action Subplan under Section 55 of the *Coastal Protection Act 1979* entitled “Greater Taree Coast Emergency Action Plan”[4], including consideration of the study area. This was notified in the *NSW Government Gazette* of 17 August 2012. Some legislative and guideline document changes have occurred since the completion of that study.

1.7 Appendix Structure

The Appendix herein is set out as follows:

- design life is considered in Section 2;
- in Section 3 to 6, risk is considered:
    - risk is defined as the product of likelihood and consequences, with likelihood discussed in Section 3 and consequences (on a structure situated immediately landward of a particular setback position) outlined in Section 4;
    - acceptable risk is defined in Section 5;
    - likelihood lines are delineated for the study area in Section 6, including comparison to traditional hazard lines;
- plots of the determined acceptable risk lines are provided in Section 7;
- the implications of these acceptable risk lines on development controls are outlined in Section 8; and
- discussion on other approaches to risk determination are provided in Section 9.

[4] Available at http://www.gtcc.nsw.gov.au/files/FP_Strategic_Planning/GTCC_Emergency_Action_Plan_V7_06082012_collated.pdf

2. DESIGN LIFE

The risk assessment must be undertaken in the context of a specified design life. This design life governs the planning period over which risks are assessed. That is, risks to structures will be determined as being acceptable or not acceptable on the basis of the risk of damage to the structure at the end of the design life.

Selection of a suitable design life is discussed in Section 9 of AGS (2007a) and Section C9.3 of AGS (2007b), in which it is noted that:

- a design life of at least 50 years would be considered to be reasonable for permanent structures used by people; and
- there is a community expectation that a residential dwelling frequently, with appropriate maintenance, will have a functional life well in excess of 50 to 60 years.

The design life of a structure should be related to the typical design life of its components, such as concrete, steel, masonry and timber. The design life used in various Australian Standards (AS) is as follows:

- in *AS 3600 - Concrete structures*, a 50 years ± 20% design life[5] (that is, 40 year to 60 years) is used in devising durability requirements for concrete structures;
- in *AS 2870 - Residential slabs and footings*, for design purposes the life of a structure is taken to be 50 years for residential slabs and footings construction;
- in *AS 1170.0 - Structural design actions - General principles*, the design life for normal structures is generally taken as 50 years[6];
- in *AS 4997 - Guidelines for the design of maritime structures*, the design life for a normal commercial structure is specified as 50 years[7], and
- in *AS 4678 - Earth-retaining structures*, the design life for earth-retaining structures (structures required to retain soil, rock and other materials) is noted as 60 years for river and marine structures and residential dwellings.

The cost of new residential development is amortised for tax purposes over 40 years based on Subdivision 43-25 of the *Income Tax Assessment Act 1997*.

Based on the above, it is considered that a reasonable design life to adopt for devising setbacks and controls for new beachfront development in the study area is between 40 and 60 years. Given the uncertainty in future climate, it is considered to be more appropriate to choose the upper end of this range, and hence a design life of 60 years has been adopted herein[8]. The design life has been applied in 2014, and thus 2074 represents the end of the design life.

[5] Period for which a structure or a structural member is intended to remain fit for use for its intended purpose with appropriate maintenance.

[6] In AS 1170.0, it is noted that for a design life of 50 years and normal structures (Importance Level 2), design event probabilities for structural actions should be 500 year ARI for wind, 150 year ARI for snow and 500 year ARI for earthquake.

[7] For a "special structure/residential" the specified design in AS 4997 is 100 years, but this was in the context of overwater structures (typically multi-unit, such as Walsh Bay 6/7, Woolloomooloo Finger Wharf, and Pyrmont), and the implications of having to carry out repairs over water are different to structures on land such as beachfront development.

[8] Note that for beachfront development in the Pittwater Council Local Government Area, "development must be undertaken in accordance with the acceptable risk management criteria defined in this document [the "Coastline Risk Management Policy for Development In Pittwater", which is Appendix 6 of the Pittwater 21 Development

A landowner may choose to design a structure for a longer design life than 60 years, in which case a site specific risk assessment could be completed by a coastal engineer on behalf of the applicant to define acceptable risks over the selected life.

It should also be recognised that future development applications (after 2014) that reference the acceptable risk lines developed herein would be applying a design life of less than 60 years. On this basis, it is recommended that either:

- Council regularly updates the risk assessment herein; or
- applicants in the study area be required to obtain coastal engineering advice to ensure that acceptable risk has been addressed over a 60 year design life at the time of any development application.

With appropriate legal advice, Council may also want to consider applying trigger conditions to development consents (for example that the consent lapses if the erosion escarpment progresses to within a certain distance of an approved dwelling), to again allow for uncertainty and to take account of the fact that development consents by default do not have a time limit.

It is recommended that the assessment herein is updated at least every 5 to 10 years. This would enable the acceptable risk lines to remain relevant as understanding of coastal processes and climate change (such as sea level rise) develops in the future.

The 60 year design life adopted herein is relevant to new development proposed in existing developed areas. However, consideration of a different planning period may be appropriate for defining setbacks for greenfield development, for example.

Control Plan] for a design project life, taken to be 100 years, unless otherwise justified by the applicant and acceptable to Council". That is, this is an example of a Council that has adopted a more conservative design life than 60 years, namely 100 years.

3. LIKELIHOOD

3.1 AGS Terminology

AGS (2007a, b) used 6 likelihood descriptors, as set out in Column 1 of Table 1[9], along with associated annual exceedance probabilities (AEPs). The AEP is given as both the indicative (single) value reported by AGS (2007a, b) in Column 2, as well as the range (based on notional boundaries between the likelihoods) in Column 3.

For a design life of 60 years, the cumulative probability of an event of that AEP occurring at least once over the design life was determined as per Column 4 of Table 1, using the formula[10]:

$$J = 1 - (1 - P)^L \tag{1}$$

where *P* is the AEP, *L* is the design life (years) and *J* is the probability of the event with an AEP of *P* occurring over the design life. The lower probability limit was associated with each descriptor herein, as per Column 5 of Table 1, which is conservative.

Table 1: Likelihood descriptors and associated probabilities used by AGS (2007a, b)

1 Descriptor	2 Annual Exceedance Probability (indicative value)	3 Annual Exceedance Probability	4 Cumulative probability of event occurring over 60 year design life (range)	5 Designated cumulative probability of event occurring over 60 year design life
Almost Certain	10%	> 5%	> 95.4%	95.4%
Likely	1%	0.5 to 5%	26.0 to 95.4%	26%
Possible	0.1%	0.05 to 0.5%	3.0 to 26.0%	3%
Unlikely	0.01%	0.005 to 0.05%	0.3 to 3.0%	0.3%
Rare	0.001%	0.0005 to 0.005%	0.03 to 0.3%	0.03%
Barely Credible	0.0001%	< 0.0005%	< 0.03%	not used

3.2 Long Term Scenarios Considered

For sea level rise and long term recession , three scenarios have been considered herein, namely:

- a “mild case” estimate, taken to have a 95% probability of exceedance (leading to lower recession);
- a “best” estimate, taken to have a 50% probability of exceedance; and
- a “severe case” estimate, taken to have a 5% probability of exceedance (leading to higher recession).

Calculations to determine the magnitude of the long term recession associated with each of the three scenarios are provided in Sections 3.3.4 and 3.3.5. Rotation was considered but not allowed for as discussed in Section 3.3.6. An uncertainty allowance was also included for each of the three scenarios as described in Section 3.3.7. Storm demand and the spatial extent of erosion, which were

[9] The heading of each column shows the column number.
[10] For example see Laurenson (1987).

not determined in this scenario based manner, are considered in Section 3.3.1/3.3.2 and Section 3.3.3 respectively.

3.3 Coastline Hazard Line Components

3.3.1 Storm Demand

During storms, large waves, elevated water levels and strong winds can cause severe erosion to sandy beaches. Storm demand represents the volume of sand removed from a beach (defined herein as the volume lost above 0m AHD) that could be expected due to a severe storm or from a series of closely spaced storms.

Based on measurements at NSW beaches, Gordon (1987) derived relationships between storm demand and average recurrence interval, at both “high demand” (at rip heads) and “low demand” (away from rip heads) areas respectively. He estimated that the storm demand above 0m AHD was about 220m^3/m for the 100 year average recurrence interval (ARI) event, for exposed NSW beaches at rip heads, and depicted a relationship between storm demand (plotted vertically) and the logarithm of ARI (plotted horizontally) that was linear (Figure 3).

In the *Black Head to Crowdy Head Coastline Hazard Definition Study* (WorleyParsons, 2010), which covered the study area, the 100 year ARI storm demand was adopted as per Gordon (1987) as 220m^3/m (except for a 500m length in the lee of the Urana Bombora where 180m^3/m was adopted, that is from north of the SLSC[11]). The red line in Figure 3 represents the Urana Bombora storm demand for a range of ARI’s[12].

Woodroffe et al (2012) noted that coastal zone managers are increasingly seeking beach erosion hazard (storm demand) predictions within a probabilistic framework to facilitate risk informed decision making. Use of Figure 3 to define storm demand for various ARI’s herein facilitates such an approach.

It is recognised that it has been assumed that the wave climate is stationary in this procedure, and that wave heights and directions may change in the future (compared to the past) under climate change. However, it is considered that insufficient information is presently available to enable any reliable estimation of what these changes may be[13]. Based on our experience investigating open coast NSW beaches, it is considered that the storm demand values adopted herein are likely to be conservative at present for a given ARI, and an uncertainty allowance has been included (Section 3.3.7) to partially account for future potential changes to storm demand. In addition, as noted previously, the CZMP should be regularly reviewed.

[11] That is, only at Block 7 Profiles 6 to 8.
[12] Factoring down Gordon (1987) by 180÷220=0.82.
[13] That stated, it can be noted that Woodroffe et al (2012) considered potential variations to storm wave direction and height in probabilistically assessing future recession at Narrabeen Beach, and did not find significant effects for that location in the scenarios assessed.

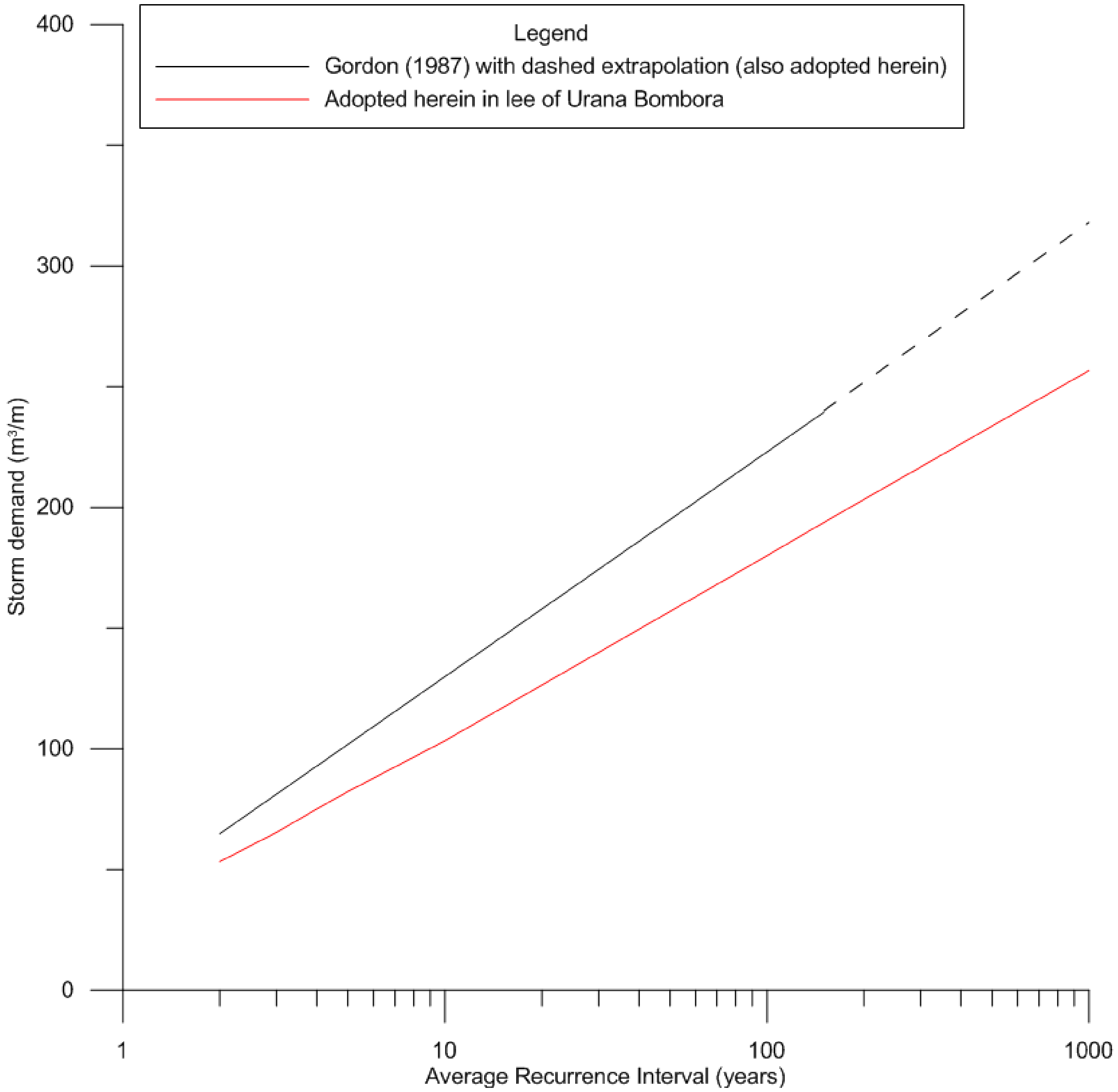

Figure 3: Relationship between storm demand and ARI as developed by Gordon (1987) for "high demand" (rip head) areas, along with adopted values for investigation herein

The question may be asked as to whether Gordon (1987) is sufficiently reliable for use herein. To compare other investigations, Callaghan et al (2008, 2009) developed a method for estimation of storm demand based on joint probability distributions of wave height, storm duration, wave period, tidal anomaly, and wave direction, a so-called Joint Probability Method (JPM) . It can be inferred from these papers that 100 year ARI storm demand values (as applied at Narrabeen Beach in these references) using this JPM were in the order of 220m^3/m to 250m^3/m, consistent with the general value adopted herein. However, there was uncertainty in extrapolating their results to such rare events.

Callaghan et al (2013) extended the original Callaghan et al (2008, 2009) papers with consideration of two additional storm erosion models, and other developments. They noted an expectation that there was an upper limit to beach erosion on the basis that there was a finite amount of energy available to drive geophysical systems (atmospheric events generating erosion). For the best fitting model, the relationship between storm demand and the logarithm of ARI was found to be linear as per Gordon (1987), up to 1,000 year ARI, although it was considered that a downward concave tail was the most physically realistic. On this basis, adopting a straight line tail as per Figure 3 is likely to be conservative.

There is a "self-limiting" characteristic to beach erosion in that as sand is removed from the upper beach it tends to deposit in offshore bars, which reduces the wave energy reaching the beach. That is, beaches in an eroded state have lower storm demands due to dissipation of wave energy on offshore bars formed during previous erosion events (Harley et al, 2009)[14]. This is evident with the logarithmic horizontal axis in Figure 3.

3.3.2 Application of Storm Demand to Beach Profiles

Nielsen et al (1992) has delineated various coastline hazard zones as discussed below and depicted in Figure 4, assuming an entirely sandy (erodible) subsurface.

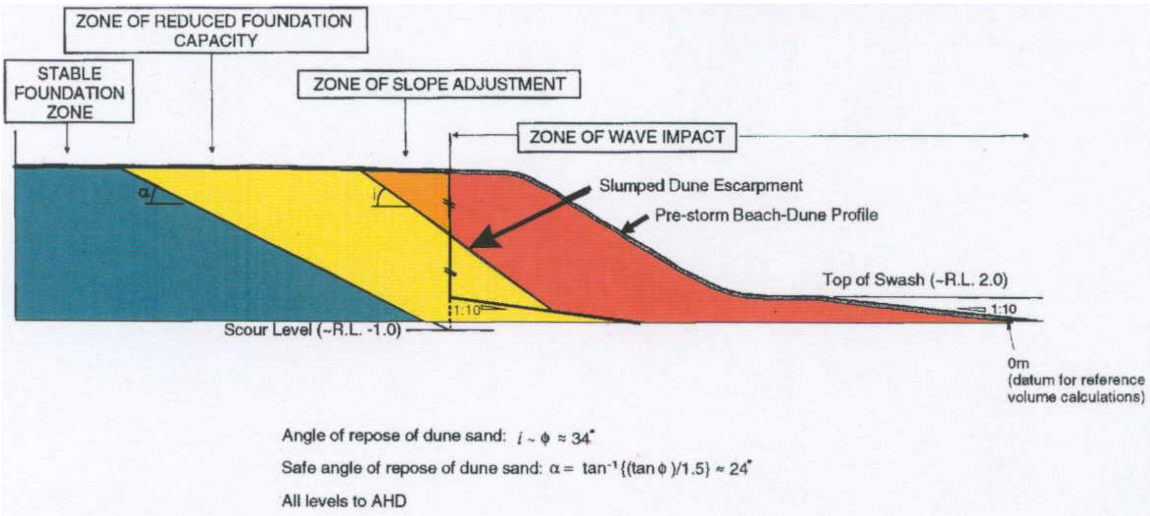

Figure 4: Schematic representation of coastline hazard zones (after Nielsen et al , 1992)

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

A *Zone of Slope Adjustment* (ZSA) is delineated to encompass that portion of the seaward face of the beach that would slump to the natural angle of repose of the beach sand 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* (ZRFC) 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

[14] Or to state it in a different way, relatively more wave energy is required to erode an already eroded beach (Yates et al, 2009).

(1992) recommended that structural loads should only be transmitted to soil foundations outside of this zone (ie 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 ZRFC would be considered to have an inadequate factor of safety.

In WorleyParsons (2010), 2006 profiles were used as the base (pre-storm) profiles, with:

- an additional allowance for recession between 2006 and 2008 to derive 2008 Hazard Lines; and
- the storm demand volume removed from each photogrammetric profile using the method of Nielsen et al (1992) to determine the position (landward edge) of the Zone of Slope Adjustment (ZSA).

The base profiles used for hazard definition herein were 2013 profiles, as these are the most recent available and given such high recession rates along Old Bar Beach it is important to select a recent date. This also removes the need to have an additional allowance for recession as per WorleyParsons (2010). Additional analysis would be required to assess the relative storminess prior to the 2013 photogrammetric data date, and its suitability.

The method of Nielsen et al (1992) was also used herein to define the landward edge of the ZSA. In the method, a ϕ value (natural angle of repose of sand, also known as friction angle) of 30° was adopted herein, with WorleyParsons (2010) not stating the value they used. Kinsela and Hanslow (2013) have suggested that a risk averse approach would be to consider a range of ϕ values between 30° and 35°. However, note that (for example) for a 6m AHD dune elevation, the difference in ZSA position over this ϕ range is only 0.6m, with lower ϕ values giving further landward positions[15].

Effects of the order of 1m in magnitude are not of significance herein. Therefore, no allowance was made for variability in ϕ values herein, and 30 degrees was adopted for all scenarios considered. The adopted 30° value is conservative.

3.3.3 Spatial Extent of Erosion

Although the entire beach is unlikely to be eroded uniformly (erosion tends to be concentrated at rip heads, which are typically a few hundred metres apart), it was conservatively assumed that all locations in the study area would be equally likely to be eroded in any particular storm.

3.3.4 Long Term Recession Due to Net Sediment Loss

Based on analysis of photogrammetric data up to 2006, WorleyParsons (2010) estimated long term recession due to net sediment loss rates in the study area, of:

- zero at Wallabi Point to 1.4m/year at chainage 900m[16] (at exfiltration ponds);
- 1.4m/year from chainage 900m to chainage 1500m;
- transition from 1.4m/year to 0.6m/year from chainage 1500m to southern boundary of Meridian Resort; and
- 0.6m/year from southern boundary of Meridian Resort to the SLSC.

[15] For a 10m AHD dune elevation the difference is 1.2m, and for a 4m AHD dune elevation the difference is 0.3m.
[16] Note that chainages were measured from the southern end of Old Bar Beach near Wallabi Point.

Figure B11 from WorleyParsons (2010) is reproduced in Figure 5, with annotations added in red for the investigation herein reflecting interpretation of the above adopted long term recession due to net sediment loss rates. The notation "OB3", "OB4" etc represents Block numbers of the photogrammetric profiles, that is Old Bar Block 3, etc (Figure 2).

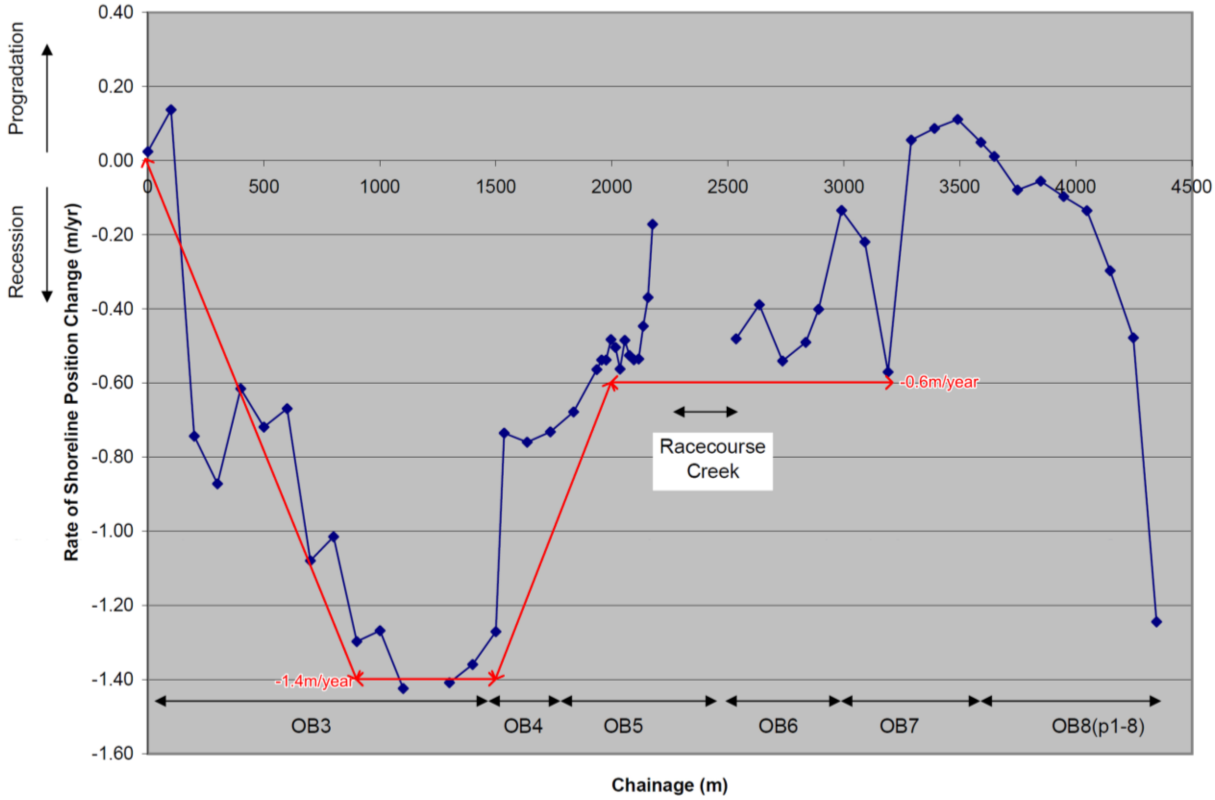

Figure 5: Photogrammetric analysis of movement of 4m contour position modified from WorleyParsons (2010), with interpretation of their adopted rates in red

Haskoning Australia (2013) had the opportunity to analyse photogrammetric data for Blocks 4 to 7 for an additional two dates after 2006, namely in 2009 and 2013. Figure E3 from Haskoning Australia (2013) is reproduced in Figure 6, with annotations added in red for the investigation herein based on analysis of the original data.

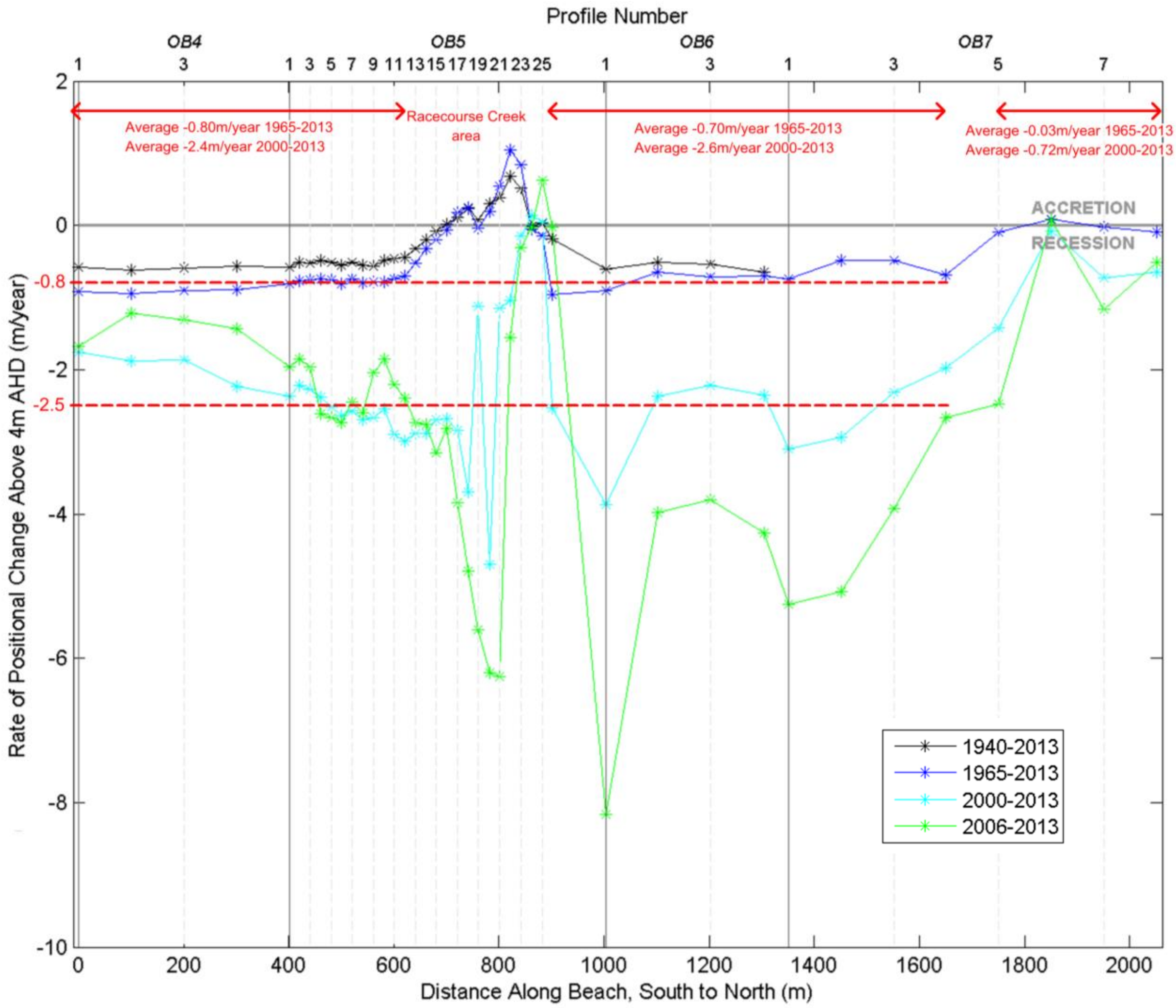

Figure 6: Photogrammetric analysis of movement of 4m contour position modified from Haskoning Australia (2013)

As shown in Figure 6, the average historical recession rates were:

- about 0.8m/year from 1965 to 2013, and about 2.4m/year from 2000 to 2013, south of Racecourse Creek;
- about 0.7m/year from 1965 to 2013, and about 2.6m/year from 2000 to 2013, north of Racecourse Creek to Block 7 Profile 4 (just south of the SLSC); and
- about 0.03m/year from 1965 to 2013, and about 0.7m/year from 2000 to 2013, from north of Block 7 Profile 4 (from north of the SLSC).

Three scenarios were considered and applied for long term recession due to net sediment loss herein, namely:

- a "mild case" estimate (95% probability of exceedance) of 0.4m/year in the entire study area except for zero for the most northern 4 profiles (Block 7, Profiles 5-8)[17];

[17] This is equal to the average recession rate over all of Block 4 to Block 7 from 1965 to 2013.

- a "best" estimate (50% probability of exceedance) of 0.8m/year in the entire study area except for 0.1m/year for the most northern 4 profiles[18]; and
- a "severe case" estimate (5% probability of exceedance) of 2.5m/year in the entire study area except for 0.7m/year for the most northern 4 profiles[19].

The "best" and "severe case" estimate rates of 0.8 and 2.5m/year respectively are depicted in Figure 6 as dashed lines. For simplicity, the most northern 4 profiles are denoted as "northern profiles" (covering Block 7, Profiles 5-8), and the remaining profiles are denoted as "southern profiles" (covering Blocks 4, 5 and 6, as well as Block 7 Profiles 1-4).

At Racecourse Creek, which is subject to entrance fluctuations, the same long term recession rate as north and south of the Creek were adopted. Although long term measured recession rates are lower near Racecourse Creek, it is considered that these rates would have been confounded by the entrance state (closed or open) at the time of photogrammetry and therefore cannot be reliably used as an indication of long term recession[20]. Volumetric analysis of photogrammetric data completed by but not reported in Haskoning Australia (2013) indicated that volume losses in the Racecourse Creek area were consistent with those to the north and south for the 2000 to 2013 period.

The adopted rates were assumed to be constant over the design life. In reality, recession would be linked to the occurrence of storms (which can in turn be related to medium term climate variability), but this would be complex to allow for in a statistically meaningful manner, and hence constant rates are considered to be reasonable. This is common practice.

Given that the base beach profiles for hazard definition were dated in 2013, to project long term recession due to net sediment loss to the end of the design life at 2074 this is a period of 61 years. Accordingly, long term recession due to net sediment loss values at 2074 are as listed in Table 2.

Table 2: Adopted long term recession due to net sediment loss values at 2074

Scenario	Long term recession due to net sediment loss at 2074 (m)	
	Southern	Northern
95% exceedance ("mild case")	24	0
50% exceedance ("best" estimate)	49	6
5% exceedance ("severe case")	153	43

3.3.5 Long Term Recession Due to Sea Level Rise

Based on Table AII.7.7 in IPCC (2013b), global mean sea level rises with respect to 1 January 2013 at 1 January 2074 for 4 representative concentration pathways (RCP) scenarios as well as the Special Report on Emissions Scenarios (SRES) A1B scenario used in the previous IPCC assessment (Meehl et al, 2007) are presented in Table 3. It is relevant to use 2013 as the starting year as base profiles for hazard definition were derived in 2013.

The projections were based on results from 21 Atmosphere-Ocean Global Circulation Models for each scenario, with 95% and 5% exceedances also shown (based on the range of model results).

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[18] Both consistent with the average applicable rates from 1965 to 2013 shown in Figure 6.
[19] Both consistent with the average applicable rates from 2000 to 2013 shown in Figure 6.
[20] Mechanical dune field restoration in around 1992 in this area may have also had some effect on long term recession rates.

Assuming each scenario is equally likely, averages over all scenarios are also shown in Table 3. These averages were adopted as the global sea level rise values for use herein.

Table 3: Global mean sea level rise (m) from 2013 to 2074 from IPCC (2013b)

Emissions Scenario	Exceedance Probability		
	95% exceedance	Median	5% exceedance
SRES A1B	0.24	0.35	0.46
RCP2.6	0.18	0.27	0.37
RCP4.5	0.22	0.32	0.42
RCP6.0	0.22	0.31	0.41
RCP8.5	0.30	0.40	0.52
Average	0.23	0.33	0.44

Note that a key assumption in Table 3 is that the 95%, 5% and median exceedances of climate model results represent the corresponding probabilities of future sea level rise. This is considered to be reasonable until any information becomes available from the IPCC to enable an alternative assumption. It is recognised that if future anthropogenic greenhouse gas emissions are closer to any of the particular SRES or RCP scenarios, then averaging all scenarios becomes less relevant. That stated, the variability in model results between the various scenarios is considered to be relatively small.

It is also relevant to consider regional sea level rise variation, that is how the study area sea level rise may vary from the global mean. From Figure 13.21(a) of IPCC (2013b), although the resolution is coarse, it can be estimated that sea level rise in NSW is projected to be 10-20% larger than the global mean at 2081 to 2100. Assuming these increases also apply at 2074 relative to 2013, the following scenarios were adopted from the IPCC (2013b) information, as also summarised in Table 4:

- "mild case" estimate of 10% increase in sea level rise (0.02m) above 95% exceedance global mean in study area (that is, 0.25m sea level rise at 2074);
- "best" estimate of 15% increase in sea level rise (0.05m) above median global mean in study area (that is, 0.38m sea level rise at 2074); and
- "severe case" estimate of 20% increase in sea level rise (0.09m) above 5% exceedance global mean in study area (that is, 0.52m sea level rise at 2074).

Table 4: Adopted sea level rise at 2074 (relative to 2013)

Scenario	Global mean sea level rise from Table 3 (m)	Additional local sea level rise (m)	Adopted total sea level rise at 2074 (m)
95% exceedance ("mild case")	0.23	0.02	0.25
50% exceedance ("best" estimate)	0.33	0.05	0.38
5% exceedance ("severe case")	0.44	0.09	0.52

In Department of Environment, Climate Change and Water [DECCW] (2009a),there was also discussion on regional variation in sea level rise in the context of derivation of NSW sea level rise benchmarks at that time. DECCW (2009a) adopted increases in NSW sea level rise above the global mean of 0.1m at 2050 and 0.14m at 2100 based on upper limit projections.

From examination of the source of this information, namely McInnes et al (2007), it is evident that at 2070 the following projections were made of regional increases in NSW sea level rise above the global mean based on two different climate models (with no information provided as to which model could be considered most likely):

- "Low Mark 2": 0 to 0.04m at both Wooli and Batemans Bay; and
- "High Mark 3" 0.08 to 0.12m at both Wooli and Batemans Bay.

These values are consistent with the IPCC (2013b) values adopted above. Woodroffe et al (2012) used a quadratic polynomial equation to define the variation in local sea level rise in NSW relative to the global mean, and found that at 2074 (relative to 2013) the increase was 0.08m, similar to the 5% exceedance value applied herein.

Linearly interpolating between the 2050 and 2100 sea level rise benchmarks in the former *NSW Sea Level Rise Policy Statement* (DECCW, 2009b)[21], which were relative to 1990, and adjusting to be relative to 2013, the equivalent sea level rise at 2074 from DECCW (2009b) is 0.57m. This is more severe than the 5% exceedance "severe case" value of 0.52m adopted herein. This emphasises that the former *NSW Sea Level Rise Policy Statement* sea level rise benchmarks were closer to current upper limit projections. It is considered that the sea level rise probabilities and risk based framework applied herein is more appropriate than the direct adoption of the former sea level rise benchmarks[22].

Bruun (1962) proposed a methodology to estimate shoreline recession due to sea level rise, the so-called Bruun Rule. It can be described by the equation (Morang and Parson, 2002):

$$R = \frac{S \times B}{h + d_c} \qquad (2)$$

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

In WorleyParsons (2010), the adopted inverse slope of the active beach profile was 50. This value was briefly reassessed herein, and discussion on depth of closure is provided in **Appendix A**. From Appendix A, adopted 95%, 50% and 5% exceedance inverse slopes of the active beach profile are as listed in Table 5. Long term recession due to sea level calculations are also completed in Table 5. The values in Table 5 were adopted as long term recession due to sea level rise estimates for use herein.

[21] Which is no longer NSW Government policy.

[22] Also note that the sea level rise values derived herein were based on the latest 5th IPCC assessment (IPCC, 2013a, b), whereas the DECCW (2009b) benchmarks were derived from the previous 4th IPCC assessment (Meehl et al, 2007).

Table 5: Long term recession due to sea level rise calculations for Old Bar Beach

Scenario	Average inverse slope of active beach profile	Sea level rise at 2074 from Table 4	Long term recession due to sea level rise at 2074 (m) from Equation 2
95% exceedance ("mild case")	15	0.25	4
50% exceedance ("best" estimate)	43	0.38	16
5% exceedance ("severe case")	84	0.52	44

Ranasinghe et al (2012), updating Ranasinghe et al (2009), has developed an alternative method to the Bruun Rule, using a process based model of dune erosion and recovery to derive probabilistic estimates of sea level rise driven coastal recession[23]. Applying the so-called Probabilistic Coastline Recession (PCR) model at Narrabeen Beach, they estimated long term recession due to sea level rise at 2100 for exceedance probabilities varying between 1% and 100%.

Ranasinghe et al (2012) considered that Bruun Rule estimates were far larger than using their PCR model, but work completed as part of the Collaroy-Narrabeen Beach and Fishermans Beach CZMP by Haskoning Australia has indicated that the Bruun Rule and PCR results were similar there. That stated, the approach herein differs to Ranasinghe et al (2012) in that different sea level values were used for different exceedance scenarios, which is considered to be more appropriate given the uncertainty in future sea level rise.

3.3.6 Future Beach Rotation

Based on studies of Palm Beach and Collaroy-Narrabeen Beach in Sydney, there have been attempts (Ranasinghe et al, 2004) to explain beach realignment/rotation in terms of shifts in the Southern Oscillation Index (SOI)[24]. Specifically, Ranasinghe et al (2004) proposed that these beaches rotate clockwise (with the northern end accreting and southern end receding) in El Niño phases (negative SOI). Conversely, it was proposed that these beaches rotate anti-clockwise (with the northern end receding and southern end accreting) in La Niña phases (positive SOI)[25]. In both cases, the beach response at the northern end lagged SOI trend shifts by about 3 months, while the beach response at the southern end lagged SOI trend shifts by about 17 months.

WorleyParsons (2010) did not consider beach rotation in their hazard definition, and additional analysis would be required to determine the significance of the process at Old Bar Beach. In the absence of this analysis, and without evidence that this is a significant proves in the study area, no allowance has been made for beach rotation herein.

3.3.7 Uncertainty Allowance

Three scenarios were considered and applied at Old Bar Beach to account for future rotation and uncertainty (for example, in future changes to storminess and wave directions) over the design life, namely:

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[23] Note that OEH does not support the use of this methodology.
[24] The SOI is calculated from the monthly or seasonal fluctuations in the air pressure difference between Tahiti and Darwin. The method used by the Australian Bureau of Meteorology is the Troup SOI which is the standardised anomaly of the Mean Sea Level Pressure difference between Tahiti and Darwin (Bureau of Meteorology, 2005).
[25] It was also found that La Niña phases were associated with more energetic (erosive) conditions.

- a “mild case” estimate (95% probability of exceedance) of zero additional translation;
- a “best” estimate (50% probability of exceedance) of 5m additional landward translation; and
- a “severe case” estimate (5% probability of exceedance) of 20m additional landward translation.

Engineering judgement was used in adopting these values, based on the typical magnitude of existing variability of beach width on NSW beaches.

3.3.8 Combined Effects

The combination of long term recession due to net sediment loss (Section 3.3.4), long term recession due to sea level rise (Section 3.3.5) and beach rotation and uncertainty (Section 3.3.6) gives the total landward translations listed in Table 6[26].

Table 6: Adopted landward translations to define recession and rotation effects at 2074

Scenario	Recession and Rotation Allowance at 2074 (m)	
	Southern	Northern
95% exceedance (“mild case”)	28	4
50% exceedance (“best” estimate)	70	27
5% exceedance (“severe case”)	216	106

The translations were included after the storm demand was applied as discussed in Section 3.3.1 and 3.3.2. This translation was achieved by shifting the Zone of Wave Impact position and recalculating the position of the Zone of Slope Adjustment at the elevation relative to the shifted position. It is recognised that this approach is simplistic as it assumes that the storm erosion and recession occur instantaneously, whereas in reality recession would occur first (with some uncertainty as the how the dune morphology may change over time, for example whether it would ‘roll back’ the dune or cut into it[27]) and then the storm demand volume would be removed from profiles different to those in 2013.

Kinsela and Hanslow (2013) have discussed this issue, noting that “it may not be conservative to expect that the development of coastal morphology will maintain pace with projected rapidly accelerating sea level rise”. This issue could be considered in future revisions of the CZMP if further information becomes available on potential dune responses to sea level rise, but it can be noted that the adopted methodology ignores any ‘roll back’ and is thus generally conservative.

[26] It is recognised that several events of the same probability (eg 5% exceedance) were combined to define an overall scenario with the same probability (eg 5% exceedance long term recession due to net sediment loss, combined with 5% exceedance long term recession due to sea level rise and 5% exceedance uncertainty allowance, to define the overall 5% exceedance scenario. This is not statistically valid (the combination of events has a lower probability that the individual events themselves), but more rigorous statistical analysis would need to be undertaken (such as bivariate analysis, see Footnote 32 on page 26) for this to be addressed. It should be recognised that the scenario probabilities adopted herein are only approximate.

[27] In addition, sea level rise would be expected to cause the dune crest to rise in elevation in response as it translates landwards.

4. CONSEQUENCES

AGS (2007a, b) used 5 consequence descriptors. These descriptors were related to the percentage of damage caused to a property due to a landslide event, relative to the market value of the property (land plus structures), as listed in Table 7.

Table 7: Consequence descriptors from AGS (2007a, b)

Descriptor	Approximate cost of damage	Description
Catastrophic	> 100%	Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.
Major	40% to 100%	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.
Medium	10% to 40%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage
Minor	1% to 10%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works
Insignificant	< 1%	Little damage

For the investigation reported herein, it was considered that the appropriate consequence descriptor for storm erosion leading to a slumped erosion escarpment immediately seaward of a structure on conventional foundations[28] was "minor". Although a structure immediately landward of a slumped escarpment may not be damaged at all, in recognition of the structure being in a Zone of Reduced Foundation Capacity (Nielsen et al, 1992) and hence having a lower factor of safety, it was considered that there was the potential for some damage.

For development on appropriately engineered piled foundations, it was considered that the appropriate consequence descriptor for structures immediately landward of the slumped erosion escarpment was "insignificant". Indeed, a structure could be well seaward of the slumped erosion escarpment and be designed with piled foundations to not be damaged for a suitably low probability event (structures can be designed to be at acceptable risk in the ocean itself)[29].

Given that hazard lines are defined herein at the landward edge of the Zone of Slope Adjustment, if used as setback lines for development this is thus equivalent to setting the consequences at that line as "minor" for development on conventional foundations and "insignificant" for development on piled foundations.

AGS (2007a, b) defines the approximate cost of damage (as per Table 7) to include:

- the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the erosion which has occurred and professional design fees; and
- consequential costs such as legal fees and temporary accommodation.

[28] Note that some practitioners distinguish "foundations" from "footings", with the latter being the structural element (such as a pier) and the former being the ground material that this structural element bears upon. However, to be consistent with Nielsen et al (1992), the term "foundations" is used herein to refer to the structural element.

[29] However, as discussed in Section 1.1, piled development would be unsuitable as a general control at Old Bar given the extent of at-risk development and difficulty in providing access to that development in the long term.

It is recognised that the land seaward of a structure sited landward of a particular setback line (for example, backyards of beachfront development) may be eroded in coastal storms, and that this does have consequences on the use of that land and landowner beach access, and may damage minor structures such as fences, decks, clothes lines and the like. This loss of land may also affect land values (a consequential loss) and have some reinstatement costs[30].

However, given that the focus of the investigation reported herein was on defining acceptable risk for new structures approved as part of the development assessment process, it was considered most appropriate to only consider risk to those structures that would be considered as part of a development application to Council, for which consequential losses are likely to be minimal given limited damage to the approved structures.

For consequences to be "insignificant" for piled development, any loss of land amenity would need to be mitigated by adoption of additional measures as discussed in Section 7.2. Furthermore, in adopting future coastline management options for Old Bar, it would be necessary to consider broader issues than just acceptable risk to new development.

In adopting the consequences descriptors of "minor" for development on conventional foundations and "insignificant" for development on piled foundations, it is assumed that there are no additional coastline hazards landward of the slumped erosion escarpment. Such hazards could include wave runup and overtopping forces on structures, or inundation of floor areas, that lead to damage. It is recognised that these hazards would need to be managed as part of defining acceptable risk to development, for example through ensuring ground floor levels are at least 0.5m above adjacent ground levels and appropriate regard has been made for these effects in the design. It is recommended that applicants in the study area be required to obtain coastal engineering advice to address issues of acceptable risk to new development from inundation in relation to design and construction, where required.

[30] However, it should be recognised that coastal land can "naturally" recover after storm events, with sand that had moved offshore in the storm returning to build the beach back up under calmer conditions after the storm. That is, any loss of land values may be temporary, and reinstatement costs may not be significant if the landowner can wait for natural recovery. That stated, with such high recession rates at Old Bar Beach, recovery may be limited.

5. ACCEPTABLE RISK

A risk matrix is presented in AGS (2007a, b), as shown in Figure 7. For example, if the consequences of a particular “unlikely” event were “minor”, then the risk would be considered “low”.

Figure 7: AGS (2007a, b) risk matrix

Likelihood	Consequence				
	Catastrophic	**Major**	**Medium**	**Minor**	**Insignificant**
Almost Certain	Very High	Very High	Very High	High	Medium
Likely	Very High	Very High	High	Medium	Low
Possible	Very High	High	Medium	Medium	Very Low
Unlikely	High	Medium	Low	Low	Very Low
Rare	Medium	Low	Low	Very Low	Very Low
Barely Credible	Low	Very Low	Very Low	Very Low	Very Low

AGS (2007a, b) defined “acceptable risk” as follows:

> “A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable”.

A key aspect of the AGS (2007a, b) approach is that they defined the acceptable level of risk for new development as being “low” risk (or lesser, that is “very low”) as per the matrix in Figure 7. This was based on review of the limited literature available, extensive discussion amongst the AGS Working Group, and consideration of the annualised cost of damage to property. AGS (2007a, b) concluded that:

> “most informed home owners are likely to be risk averse as a result of appreciation of the consequences at a family or personal level, almost regardless of the likelihood of the event. This risk aversion suggests that Low Risk to Property is an appropriate recommendation for acceptable risk to the regulator for domestic dwellings which are of Importance Level 2 (as defined in the BCA [Building Code of Australia])”.

Note that AGS (2007a, b) considered that the acceptable risk level was “low” for structures of both:

- Importance Level 2 (such as low-rise residential construction)[31]; and
- Importance Level 3 (such as buildings and facilities where more than 300 people can congregate in one area, schools of greater than 250 people, health care facilities with a capacity of 50 or more residents, power generating facilities, water treatment and waste water treatment facilities).

For structures of Importance Level 4 (such as buildings and facilities designated as essential facilities or with special post-disaster functions, medical emergency or surgery facilities, emergency service facilities (fire, rescue, police etc.), the designated acceptable risk level was “very low”. Additional assessment would be require to define acceptable risk lines for these areas.

[31] For structures of Importance Level 1 (such as minor temporary facilities), the designated acceptable risk level was “medium”.

Given that “low” risk can be considered acceptable for typical structures in the study area, it follows from Figure 7 that:

- the “unlikely” likelihood line can define the acceptable risk setback for new development that is constructed on conventional foundations (since, as noted in Section 4, this has “minor” consequences); and
- the “likely” likelihood line can define the acceptable risk setback for new development that is constructed on piled foundations (since, as noted in Section 4, this has “insignificant” consequences).

6. DELINEATION OF LIKELIHOOD LINES IN STUDY AREA

6.1 Procedures Considered

Two procedures were applied to define likelihood lines ("almost certain", "likely", "possible", "unlikely" and "rare" as per Table 1) in the study area, namely:

- Type 1: a storm event occurring at any time over the design life, ignoring recession[32]; and
- Type 2: a storm event occurring in the last year of the design life, after the full magnitude of recession as per Table 6 had been realised.

The storm event probabilities are different in these procedures. For Type 1, the event can occur at any time over the design life, so for example a 0.5% AEP (200 year ARI) event has a 26% probability over the design life in Type 1 for a 60 year life. However, a 0.5% AEP event is treated as 0.5% probability for Type 2, which when multiplied by the recession scenario probability (for example 50% for the "best" estimate) gives the probability over the design life (0.25% in this example).

That is, once recession is included, the probability of the event occurring in the last year (only) of the design life is considered (as per Type 2), and the event probability is much lower than the probability of occurring at any time during the design life (as per Type 1).

As noted in Section 3.3.2, likelihood lines were defined at the landward edge of the Zone of Slope Adjustment, with the storm demand volume (Section 3.3.1) applied to 2013 profiles.

The calculation methodologies for the Type 1 and Type 2 procedures are described in Section 6.2 and Section 6.3 respectively.

It is recognised that more advanced statistical approaches and Monte Carlo modelling could be undertaken to refine the estimates provided herein. It is recommended that these approaches are considered in the future as understanding develops of the appropriate probability distributions to adopt in these analyses.

6.2 Storm Event Occurring any Time Over Design Life, Ignoring Recession (Type 1)

Based on the relationships between likelihood and AEP from Table 1, the conversion from AEP to ARI as follows[33]:

$$ARI = \frac{-1}{ln(1-AEP)} \qquad (3)$$

[32] Recession was not included in the Type 1 procedure adopted herein. It was assumed that the design event occurred at any time over the design life, but the recession component was not included. In reality, the design storm can occur at any time over the design life, and the recession depends on the year of the event. For example, a 0.5% AEP event could occur in say Year 1, or Year 20, or Year 60, and the probability of that event occurring is 0.5% in each case. However, the recession component would vary in each case. As a future refinement to this investigation, it may be possible to model the bivariate distribution of the joint probability of the storm erosion and recession to consider both processes in a Monte-Carlo modelling exercise.
[33] Where ARI is in years, and AEP is expressed as a decimal (for example, 6.6% becomes 0.066).

and the relationships between ARI and storm demand from Figure 3, storm erosion volumes for the “almost certain”, “likely”, “possible”, “unlikely” and “rare” likelihoods were determined as shown in Table 8.

Table 8: Storm demands at Old Bar Beach corresponding to various likelihoods for Type 1 procedure

Likelihood	Cumulative probability over design life (%)	AEP (%)	ARI (years)	Storm demand (m^3/m) Block 7 Profiles 6-8	Storm demand (m^3/m) everywhere except Block 7 Profiles 6-8
Almost Certain	95.4%	5	20	130	160
Likely	26%	0.5	200	200	250
Possible	3%	0.05	2,000	280	350
Unlikely	0.3%	0.005	20,000	360	440
Rare	0.03%	0.0005	200,000	430	540

These respective storm demand volumes were applied at Old Bar Beach as per Section 3.3.1 and 3.3.2. This defined the landward edge of the Zone of Slope Adjustment, which in turn defined the likelihood line for the five likelihoods considered as shown in Figure 8. For each likelihood line, the description applies at the line and seaward to the next seaward line. For example, the “possible” line has a “possible” likelihood, as does the area seaward of that line to immediately landward of the “likely” line[34].

[34] Note that the “barely credible” likelihood is represented by the area landward of the “rare” likelihood line.

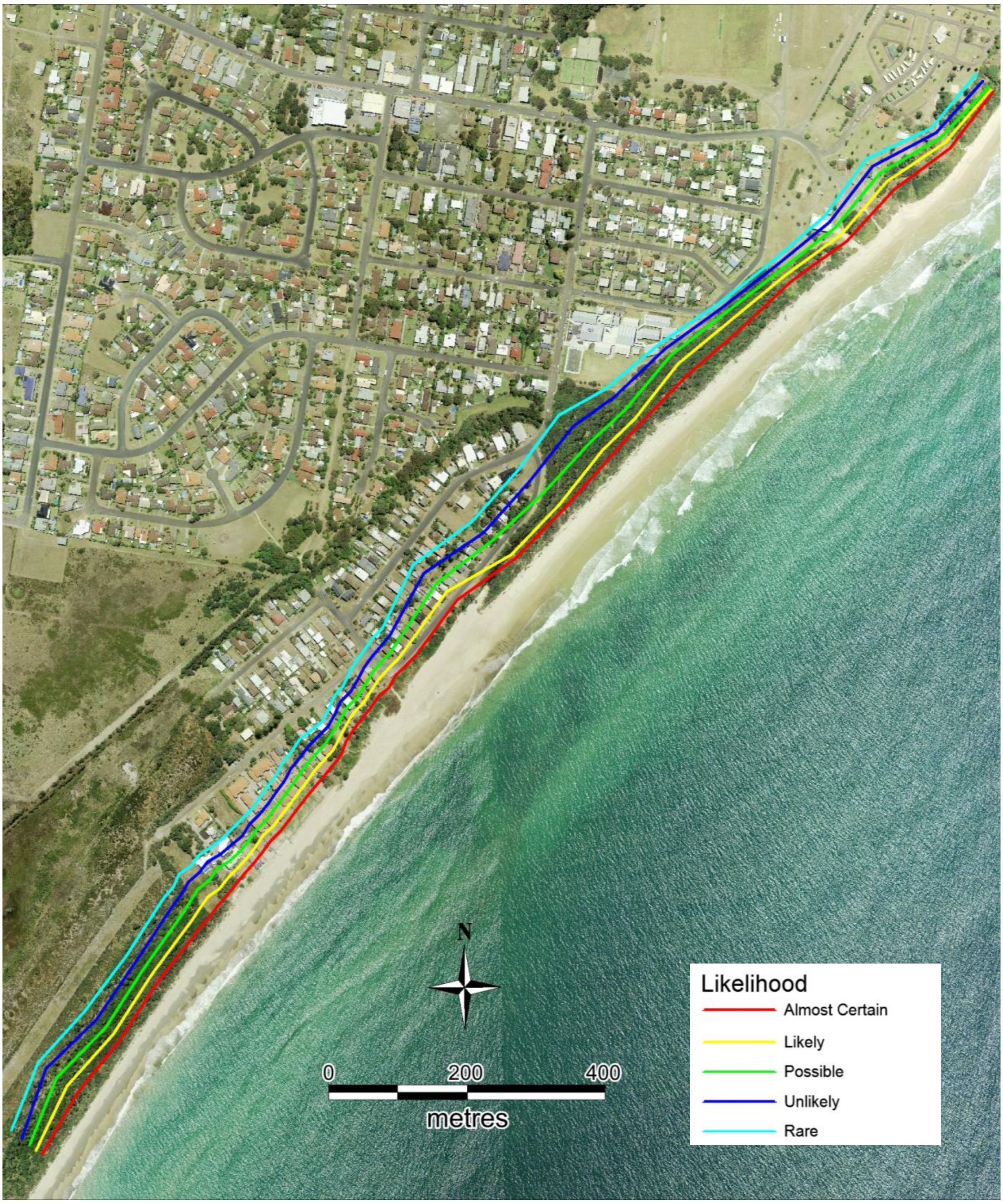

Figure 8: Likelihood lines at Old Bar Beach for Type 1 procedure (no recession included)

6.3 Storm Event Occurring in Last Year of Design life, With Recession (Type 2)

For Type 2, the procedure adopted herein has been to consider the probability of a particular storm erosion volume occurring in the last year of the design life (after long term recession has been realised). This is appropriate as it is equally likely that a particular storm of probability *P* occurs in

2014 or 2074 (ignoring any potential increases in the severity or frequency of storms under climate change), and the later a storm of probability *P* occurs in the design life the further landward it would extend due to greater prior recession.

The first step in this procedure was to define the storm event probability (AEP) for a storm occurring in the last year of the design life after recession had occurred. This required the storm event AEP (probability), when multiplied by the relevant probability for the scenario (for example, 50% for the 50% exceedance “best” estimate scenario), being equal to the cumulative probability over the design life associated with the particular likelihood (see Table 9).

For example, for the “unlikely” likelihood, the required cumulative probability over the design life is 0.3%. For the 50% exceedance (“best” estimate) scenario, the storm demand event AEP is 0.6% (0.006 × 0.5 ×100 = 0.3%). “N/A” entries in Table 9, denoting “not applicable”, mean that the AEP was greater than 100% and hence undefined.

In multiplying the probabilities together it was assumed that the storm event and recession scenarios are independent. These processes are not completely independent, as coastal storms are mostly driven by weather patterns leading to large waves and elevated water levels, while recession would be driven by net sediment loss (in turn dependent on waves and water levels, particularly during storms) and sea level rise (where water level is a factor). However, assuming independence is considered to be a conservative approach.

Table 9: Storm event probabilities that would achieve particular likelihood probabilities for the three exceedance scenarios considered

Likelihood	Cumulative probability of event occurring over design life	Storm demand event AEP (%)		
		95% exceedance	50% exceedance	5% exceedance
Almost Certain	95.4%	N/A	N/A	N/A
Likely	26%	27	52	N/A
Possible	3%	3	6	59
Unlikely	0.3%	0.3	0.6	6
Rare	0.03%	0.03	0.06	0.6

In Table 10, the storm event probabilities in Table 9 were converted to ARI’s using Equation 3. The “Almost Certain” likelihood was not included as it cannot be defined using the Type 2 procedure give that AEP’s exceed 100%.

Table 10: Storm event ARIs corresponding to events in Table 9

Likelihood	Cumulative probability of event occurring over design life	Storm demand event ARI (years)		
		95% exceedance	50% exceedance	5% exceedance
Likely	26%	3	1	N/A
Possible	3%	32	16	1
Unlikely	0.3%	320	170	16
Rare	0.03%	3200	1670	170

Based on Figure 3, the storm demand volumes corresponding to these events were determined as shown in Table 11. For Block 7 Profiles 6-8, these volumes would be multiplied by 0.82.

Table 11: Storm demand volumes for Old Bar Beach (except Block 7 Profiles 6 to 8) corresponding to events in Table 10

Likelihood	Cumulative probability of event occurring over design life	Storm demand volume (m^3/m)		
		95% exceedance	50% exceedance	5% exceedance
Likely	26%	80	50	N/A
Possible	3%	180	150	40
Unlikely	0.3%	270	240	150
Rare	0.03%	370	340	240

These respective storm demand volumes were applied at Old Bar Beach as per Section 3.3.1 and 3.3.2. This defined the landward edge of the Zone of Slope Adjustment. The setback for the particular scenario (95%, 50% or 5% exceedance) was then applied as per Table 6, to define the likelihood lines for the four likelihoods considered. The critical events (that produced the most landward setback) for each likelihood are highlighted in red in Table 11, namely the 5% exceedance ("severe case") scenarios.

The likelihood lines for the 95%, 50% and 5% exceedance recession scenarios are shown in Figure 9, Figure 10 and Figure 11 respectively. It is evident that although a particular likelihood line (eg "possible") theoretically has the same probability of occurrence for each of the three (95%, 50% and 5%) exceedance recession scenarios, the likelihood lines move progressively landward over these scenarios (with 5% producing the most landward lines). This was due to the relatively large recession rates at Old Bar, such that rarer recession scenarios produce relatively more landward lines than rarer storm demands.

Figure 9: 95% exceedance recession scenario likelihood lines at Old Bar Beach for Type 2 procedure

Figure 10: 50% exceedance recession scenario likelihood lines at Old Bar Beach for Type 2 procedure

Figure 11: 5% exceedance recession scenario likelihood lines at Old Bar Beach for Type 2 procedure

6.4 Comparison to Traditional Hazard Lines

WorleyParsons (2010) delineated Immediate (at 2008), 2058 (Mid-Range and High-Range) and 2108 (Mid-Range and High-Range) Coastline Hazard Lines in the study area. These were defined at the landward edge of the Zone of Slope Adjustment. The Immediate, 2058 Mid-Range and 2108 Mid-Range Hazard Lines are shown in Figure 12[35]. It should be recognised that the WorleyParsons hazard lines would generally be further landward if based on the same profiles as used for hazard definition herein, namely the 2013 profiles.

A plot of the 50% exceedance scenario Type 2 likelihood lines (for “likely”, “possible”, “unlikely” and “rare”) and Type 1 “almost certain” likelihood line is also provided in Figure 12 for comparison.

It is evident that the traditional Immediate ZSA is similar to (or seaward of) the “almost certain” line. The traditional 2058 ZSA is generally similar to the “likely” likelihood line. The traditional 2108 ZSA is landward of the “rare” likelihood in the southern portion of the study area, but is similar to the “possible” and likely” likelihoods at some locations moving further north.

It is considered that the likelihood lines derived herein are more reliable and relevant for planning purposes than the traditional hazard lines.

[35] Note that the lines were derived based on GIS information supplied by Council. There appears to be a discrepancy between the Immediate Lines plotted in WorleyParsons (2010) and herein, with the latter further seaward at some locations.

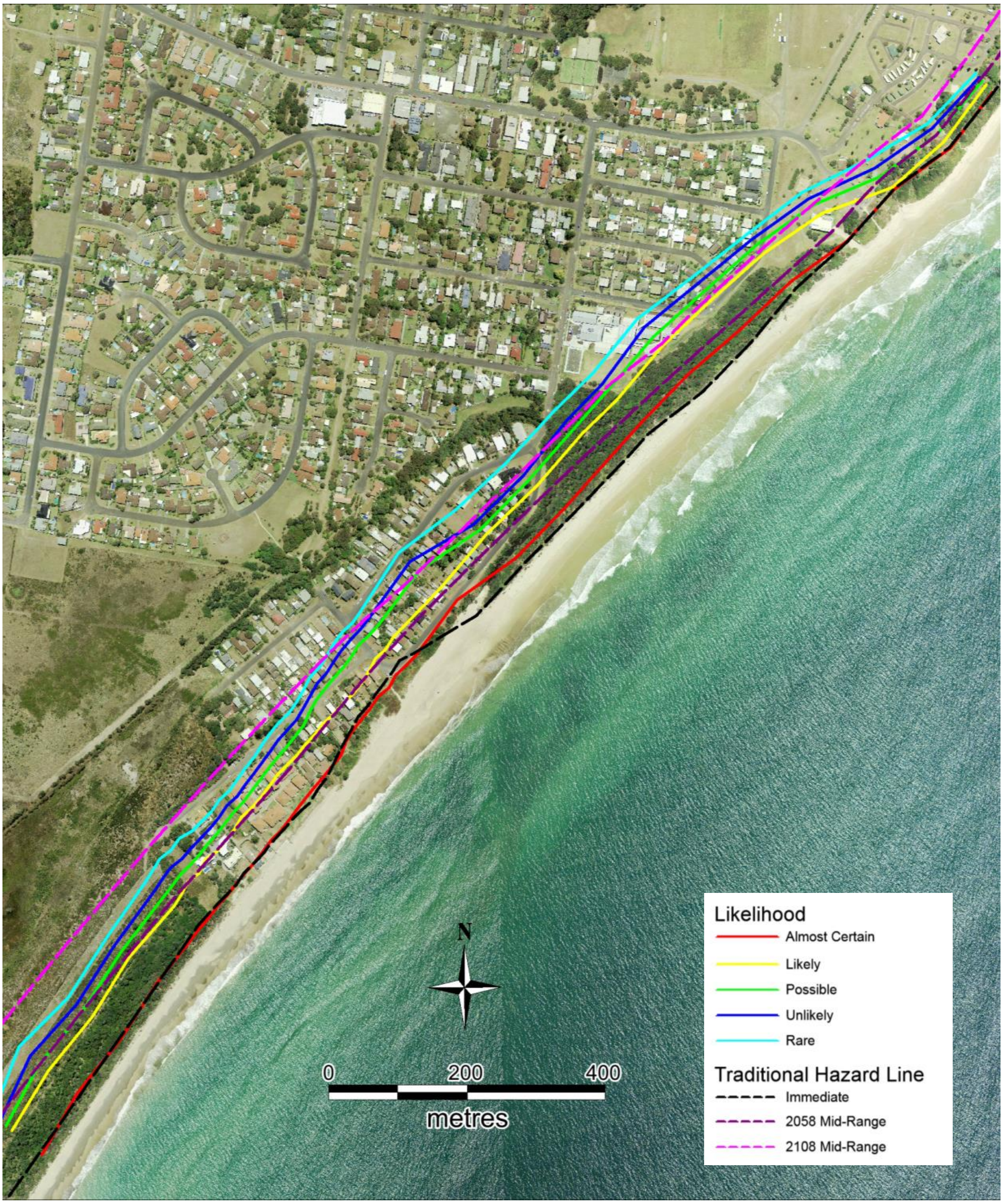

Figure 12: 50% exceedance scenario Type 2 likelihood lines, and Type 1 "almost certain" likelihood line, compared to traditional hazard lines from WorleyParsons (2010)

7. PLOTS OF ACCEPTABLE RISK LINES DETERMINED IN STUDY AREA

7.1 Acceptable Risk Lines

As described in Section 5:

- the "unlikely" likelihood line is the acceptable risk setback for new development on conventional foundations; and
- the "likely" likelihood line is the acceptable risk setback for new development constructed on piled foundations.

The (Type 2) 5% exceedance recession scenario "unlikely" likelihood line has been adopted to define acceptable risk for new development on conventional foundations herein. It was considered that for structures without piling where coastal erosion/recession undermining the structure could lead to considerable damage, a more risk averse outlook was appropriate (particularly given the uncertainty in future recession rates).

The (Type) 2 50% exceedance recession scenario "likely" likelihood line has been adopted to define acceptable risk for new development on piled foundations herein. A less risk averse outlook was considered appropriate in this case given the significant reduction in likely damage for an undermined piled structure.

Plots of the acceptable risk lines to define the setback for new development on conventional foundations and on piled foundations in the study area are provided in Figure 13 To be at acceptable risk, new development would need to be landward of these lines as relevant to the foundation type, unless protective works were constructed. This would mean that new development would not be recommended seaward of the acceptable risk line for piled development unless protective works were constructed[36].

[36] Given the implications of these controls (significant number of lots at unacceptable risk unless piled, and significant number of lots where new development would not be permitted), additional coastline hazards analysis may be warranted as discussed in Section 1.5.

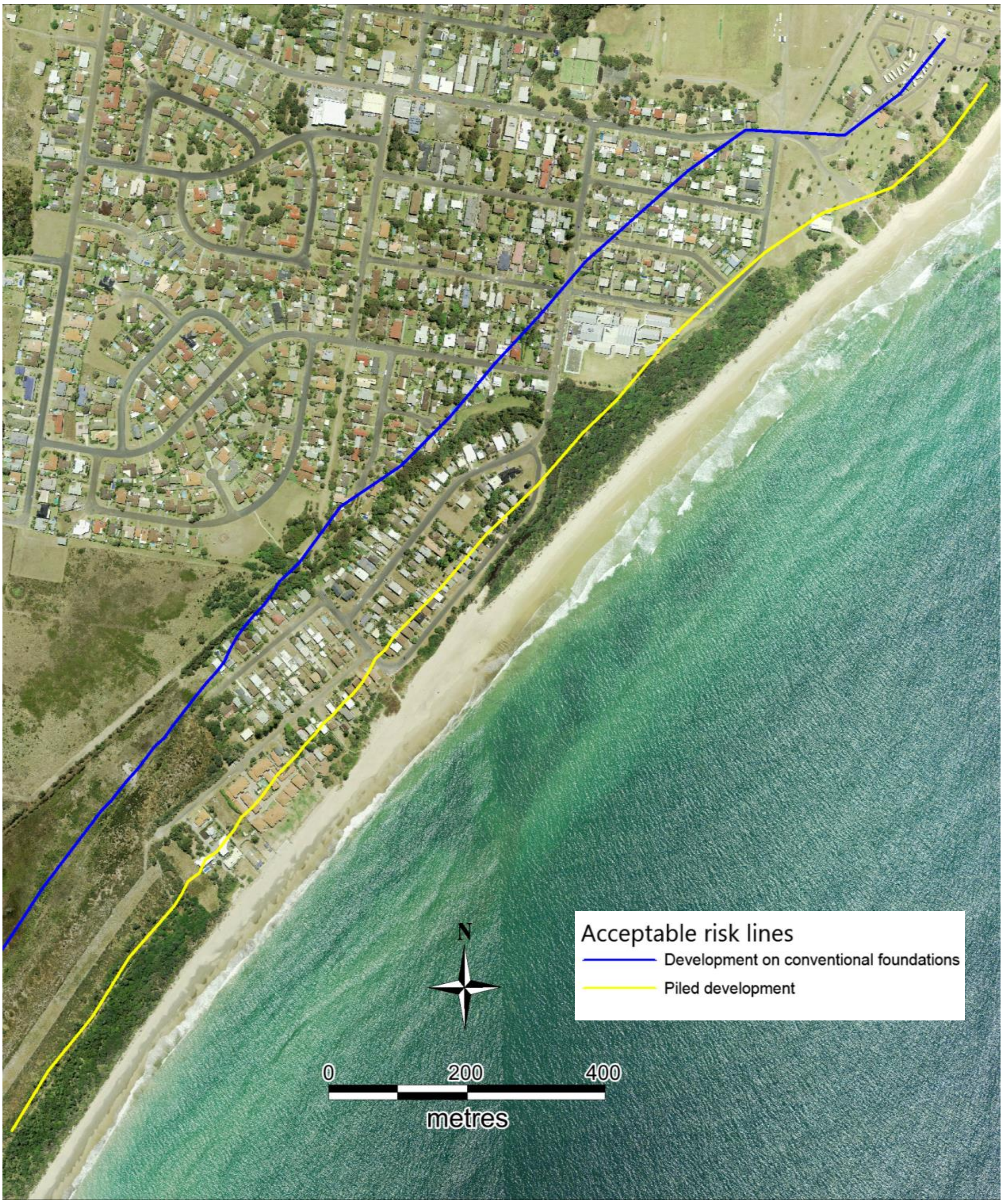

Figure 13: Acceptable risk setback lines determined at Old Bar Beach

7.2 Implications for Piled Development

It is reiterated that piled development is not considered to be an appropriate general control for new development at Old Bar, as discussed in Section 1.1. To emphasise why piled development is inappropriate, note the following:

- erosion of the land surrounding piled structures would be expected to impact on the amenity of the lot;
- to be at acceptable risk, piled access to a future redeveloped dwelling would be required if that access point was seaward of the acceptable risk line for development on conventional foundations; and
- at most lots this would mean that road access to the property would also need to be piled.

Given the area of development seaward of the acceptable risk line for development on conventional foundations at Old Bar, it is evident that such an approach (of piling access etc) would be cost prohibitive and impractical.

8. IMPLICATIONS FOR DEVELOPMENT CONTROLS

As noted in Section 7, if the acceptable risk line for piled development was adopted as a minimum setback, new development would not be permissible at a significant number of lots. If the acceptable risk line for development on conventional foundations was adopted, a large number of lots would require new piled structures if redeveloped.

However, given the considerable number of lots seaward of the acceptable risk line for development on conventional foundations, it would be impractical to adopt a piled development control. This is because at these lots, although the main dwelling structure on piled foundations would be at acceptable risk, vehicular and/or pedestrian access to this structure from the road would be at unacceptable risk unless it was also piled. It would not be appropriate (given the cost to Council of piling road access, and loss of land surrounding dwellings) to essentially create a township that would eventually be suspended over the ocean.

Council is considering alternative options for dealing with the risk to existing and future development, such as:

- construction of protective works, or
- (dealing with the risk to future development only) placing controls on new development such that this must be relocated or removed when an erosion escarpment gets within a certain trigger distance of the structure.

It is beyond the scope of the investigation herein to consider the merits of these or other coastline management options. This is most appropriately undertaken by considering engineering, legal, social, environmental, beach amenity, beach access and economic factors in a broader context than just controlling new development. That stated, it is evident from the study herein that a significant number of lots are not at acceptable risk at Old Bar, and therefore that action needs to be undertaken to reduce risks.

9. OTHER APPROACHES TO RISK DETERMINATION

The approach to defining acceptable risk herein was developed by the authors as an extension to WorleyParsons (2012a, b), in which they (in previous employment) completed a relative risk assessment to Warringah's coastal structures. This risk assessment work has also been described in Horton et al (2011) and Roberts and Horton (2011).

Familiarity with and further review of the AGS (2007a, b) procedures, recognition of the limitations of the traditional hazard lines approach, review of Australian Standards on risk[37], and support in *Guidelines for Preparing Coastal Zone Management Plans* (DECCW, 2010a) for a risk management approach led to development of the approach used herein for the Collaroy-Narrabeen Beach and Fishermans Beach CZMP and adaption to Old Bar . This approach was seen as rational and robust.

Although others have defined likelihood hazard lines (for example, in the *Coffs Harbour Coastal Processes and Hazards Definition Study*), these have been defined qualitatively without reference to defined probabilities, and are not considered to be consistent with AGS (2007a, b) probabilities.

Jongejan et al (2011) considered the use of setback lines as a form of risk mitigation at Collaroy-Narrabeen Beach. They noted that defining appropriate setback lines for land-use planning purposes was a balancing act, but found that it was unclear what level of protection was facilitated by current setback lines, and whether this was sufficient from an economic perspective.

Jongejan et al (2011) presented an economic model to determine what setback lines would be optimal from an economic perspective. They concluded that:

- it is useful to define setback lines on the basis of their exceedance probabilities (as has been undertaken herein)[38];
- the approach required probabilistic estimates of coastal erosion volumes (as has been undertaken herein);
- an order of magnitude 1% AEP event produced an "economically efficient" setback line without sea level rise; and
- long term uncertainties (for example due to climate change) influenced the exceedance probability of "economically efficient" setback lines but only to a limited extent.

Jongejan et al (2011) used the Callaghan et al (2008) and Ranasinghe et al (2009) procedures in their analysis, to obtain probabilistic hazard lines.

Woodroffe et al (2012) further applied the Jongejan et al (2011) procedure to develop "economically efficient" setback lines for Collaroy-Narrabeen Beach. They found that these setbacks lines were located near to Ocean Street and Pittwater Road in the study area. It is considered that the approach adopted herein is more appropriate for defining acceptable risk to development from a Council perspective at this point in time.

Some of the potential limitations of Woodroffe et al (2012) included that:

[37] Namely AS/NZS ISO 31000:2009, "Risk management - Principles and guidelines", AS 5334-2013, "Climate change adaptation for settlements and infrastructure - A risk based approach", the draft "Risk management guidelines, Companion to AS/NZS ISO 31000:2009 (Revision of HB 436:2004)" (DR HB 436) and the document HB 327:2010, "Communicating and consulting about risk".
[38] Also supported by Kinsela and Hanslow (2013).

- setbacks were defined based on economic criteria only, as opposed to the approach herein of defining acceptable risk on the basis of probabilities and consequences (which embody an economic consideration) over an appropriate design life compared to a standard developed rigorously by AGS (2007a, b);
- they assumed that those that suffer damage from storm erosion would be compensated by a third party (government, charity or other) that is unable to collect a premium for its explicit or implicit guarantee, whereas it is expected that in practice landowners would bear entirely the financial consequences of any damage to their properties;
- the economic model utilised a number of “doubtful constants” which were noted as imprecise and subject to debate, such as the discount rate and rate of return, and it was assumed that there were no market imperfections;
- there was no consideration of an appropriate design life; and
- there was no consideration of the effect of measures to reduce risk (such as piling and protective works) in the economic model.

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Appendix A:
Estimation of Depth of Closure for Use in the Bruun Rule

CONTENTS

A1. INTRODUCTION

There are a number of methods to estimate the depth of closure for use in the Bruun Rule, as described in this Appendix. These include methods based on:

- wave characteristics (Section A2); and
- sedimentological data (Section A3).

A synthesis and discussion of the available methods is provided in Section A4.

References are included in the main report.

A2. METHODS BASED ON WAVE CHARACTERISTICS

Bruun (1988) suggested a depth of closure of $3.5H_b$, where H_b is the actual breaker height of the highest waves within a certain time period, namely 50 to 100 years according to Dubois (1992). However, Bruun (1988) also noted that a closure depth of $2H_b$ was appropriate for a Danish case study.

Kulmar et al (2005) predicted that the 100 year average recurrence interval (ARI) significant wave height (H_s)[1] exceeded for a duration of 1 hour and 6 hours offshore of Sydney (likely to be similar to Old Bar) was 9.5m and 8.5m respectively. Shand et al (2011) estimated a 100 year ARI 1 hour duration H_s of 9.0m offshore of Sydney, and 8.5m offshore of Crowdy Head (the latter being closer to Old Bar).

Adopting a 100 year ARI wave height of 8.5m, the Bruun (1988) suggested closure depths are between 17m and 30m.

Hallermeier (1981, 1983) defined three profile zones, namely the littoral zone, shoal or buffer zone[2], and offshore zone. This thus defined two closure depths, namely:

- an "inner" (closer to shore) closure depth at the seaward limit of the littoral zone, termed d_l by Hallermeier (1981) and d_s by Hallermeier (1983), and d_{inner} herein; and,
- an "outer" or "lower" (further from shore) closure depth at the seaward limit of the shoal/buffer zone, termed d_i by Hallermeier (1981) and d_o by Hallermeier (1983), and d_{outer} herein.

From Hallermeier (1981):

$$d_{inner} = 2.28H_e - 68.5\left(\frac{H_e^{\,2}}{gT_e^{\,2}}\right) \qquad (1)$$

where H_e is the effective significant wave height exceeded for 12 hours per year (that is, the significant wave height with a probability of exceedance of 0.137%), and T_e is similarly defined for wave period. Based on measured Crowdy Head offshore wave data (Shand et al, 2011), H_e is about 5.3m and T_e is about 18s, and from Equation 1 the inner closure depth is thus about 12m.

From Hallermeier (1983):

$$d_{outer} = 0.018H_m T_m \sqrt{\frac{g}{D(s-1)}} \qquad (2)$$

where H_m and T_m are the median wave heights and periods respectively, D is the median sediment diameter (about 0.3mm in the study area[3]), and S is the specific gravity of sand (about 2.6). Based on measured Crowdy Head offshore wave data (Shand et al, 2011), H_m is about 1.5m and T_m is about 9.5s, and from Equation 2 the outer closure depth is thus about 37m.

[1] The significant wave height is the average height of the highest one-third of the waves in a particular record.
[2] Shoal zone in Hallermeier (1981) and buffer zone in Hallermeier (1983).
[3] As derived from database associated with Short (2007).

In the *Coastal Risk Management Guide*, DECCW (2010b) recommended the use of the outer closure depth when using the Bruun Rule in the absence of readily available information on active profile slopes at a location under consideration.

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

$$d_c = 1.75 H_e \qquad (3)$$

Therefore, the predicted (inner) closure depth from Equation 3 is about 9.3m.

A3. METHODS BASED ON SEDIMENTOLOGICAL DATA

Sedimentological data consistently shows distinct changes in the characteristics of sediments with water depth offshore of NSW (Nielsen, 1994). These changes include variations in grain size, sorting, carbonate content and colour.

There are two distinctive sediment units immediately offshore of the NSW shoreline, namely Nearshore Sand, and (further offshore and coarser) Inner Shelf Sand (also known as Shelf Plain Relict or Palimpsest Sand). Nearshore Sand is further subdivided into Inner and Outer Nearshore Sand units.

For beaches fully exposed to the offshore wave climate, the boundary between Inner and Outer Nearshore Sands is typically found at about 11m to 15m depth (relative to AHD), while the boundary to the nearshore edge of Inner Shelf Sand is usually at 18m to 26m depth. The boundary between Nearshore Sands and Inner Shelf Sands corresponds to those parts of the seabed considered to be active and relict respectively. That is, there is no exchange of Nearshore Sands with those of the Inner Shelf.

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

- 12m $\pm$ 4m being the limit of significant wave breaking and beach fluctuations (consistent with the Inner/Outer Nearshore Sand Boundary and inner Hallermeier depth);
- 22m $\pm$ 4m being the absolute limit of sand transport under cyclonic or extreme storm events (consistent with the inshore Inner Shelf Sand boundary); and,
- 30m $\pm$ 5m being the limit of reworking and onshore transport of beach sized sand under wave action (consistent with the outer Hallermeier depth).

WorleyParsons (2010) presented evidence that in the vicinity of Old Bar, the Inner/Outer Nearshore Sand boundary (inner Hallermeier depth) was at about -10m AHD.

A4. SYNTHESIS AND DISCUSSION

The methods based on wave characteristics (Section A2) and sedimentological data (Section A3) indicated “inner” closure depth values are in the order of 12m ± 4m in the study area, with “outer” closure depth values in the order of 35m.

Closure depths can also be determined from examination of bathymetry, generally coinciding with changes in slope of the offshore seabed profile. Based on a 2009 hydrosurvey supplied by OEH, the seabed profile (shore-normal) offshore of the Meridian Resort (assumed to be representative of the study area) was as shown in Figure A1.

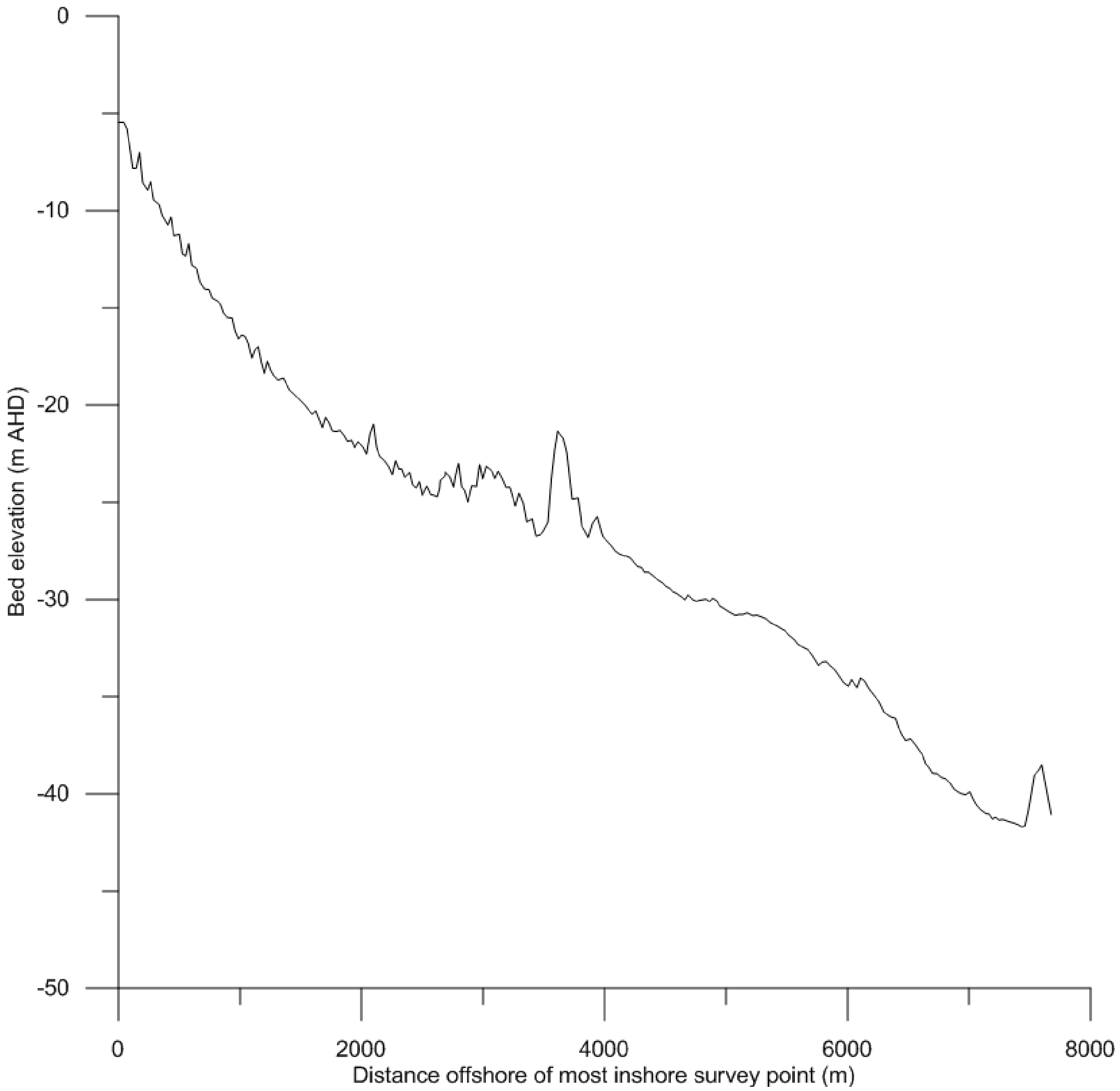

Figure A1: Seabed profile offshore of Meridian Resort based on 2009 hydrosurvey

It is evident that there was a distinct change in slope at about -24m AHD, with a flattening offshore (presumably associated with a rock reef). This 24m depth was therefore used as the outer Hallermeier depth. An inner Hallermeier depth of 12m was adopted.

Based on the 2009 hydrosurvey, the distance from the shoreline (0m AHD) to:

- a bed level of -12m AHD was about 750m; and
- a bed level of -24m AHD was about 2,600m.

Assuming a dune crest height of 8m AHD over a subaerial beach width of 100m as per WorleyParsons (2010), the inverse slopes of the active beach profile are 43 and 84 to the inner and outer Hallermeier depths respectively.

Depth of closure is time-scale and space-scale dependent, and generally increases with time-scale (Capobianco et al, 1997). It is considered that although closure depths in the order of 24m may eventually be realised, this is more likely to be over time scales of centuries (ie beyond 2100). The inverse slope of the active beach profile of 84 (outer Hallermeier depth) was therefore associated with a 5% exceedance scenario. The inverse slope of the active beach profile of 43 (inner Hallermeier depth) was associated with a 50% exceedance scenario.

The inverse slope of the active beach profile for the 95% exceedance scenario was assumed to be equal to the approximate average inverse slope of the subaerial beach face (swash area) based on analysis of photogrammetric data, namely 15.