APPENDIX E: ASSESSMENT OF RECESSION TRENDS, DESIGN WAVE HEIGHTS AND SCOUR LEVELS

Preamble

WRL were retained to provide an assessment of design breaking wave heights and scour levels for the proposed revetment. A copy of this assessment is provided in this appendix.

HKA assisted with the briefing of WRL, as outlined in their assessment. Drawing No. 8A0271-MA-CH700 is provided in this appendix.

An updated assessment of long term recession trends utilising the most recent photogrammetry data was also undertaken by HKA to inform the WRL investigations. Details of this assessment are given below.

Assessment of Long-Term Recession Trends

Longer term sand movement at Old Bar Beach has been examined using photogrammetry[7], by considering the movement of the beach scarp between the first and last dates of photography.

Photogrammetric data provided by OEH

The photogrammetric data collection was undertaken by OEH using their AC3 photogrammetric instrument. The study area for the current investigation is covered by 42 shore-normal beach profiles in four blocks[8], named OB4 to OB7 as per WorleyParsons (2010a), as shown in **Figure E1** and **Figure E2**. Blocks OB4, OB6 and OB7 are characterised by an alongshore profile spacing of 100 m, while a profile spacing of 20 m applies to Block OB5.

Photogrammetry data for Old Bar Beach was collected for the following dates of photography, spanning from 1940 to 2013:

- December 1940 (excluding Block OB7)
- 16 January 1965
- 13 August 1970 (excluding Block OB7)
- 13 June 1981 (excluding Blocks OB6 and OB7)
- 27 November 1981 (Block OB7 only)
- 22 April 1986
- 19 June 1989 (excluding Block OB7)
- 31 August 1991 (excluding Block OB7)
- 20 June 1993
- 31 May 1996 (excluding Block OB7)
- 17 May 2000
- 31 July 2004
- 26 November 2006
- 2009 (excluding Block OB7)
- 18 May 2013

WorleyParsons (2010a) analysed photogrammetry data collected to 2006, i.e. the present assessment considers the most recent photogrammetry data collected in 2009 and 2013.

7 Photogrammetry involves measurement and data acquisition from photographic and other remotely sensed images. It was used in this study to measure historical beach profile changes from vertical aerial photography. This assists in identification of possible recession or accretion trends.
8 A block represents a set of beach profiles that are parallel to each other.

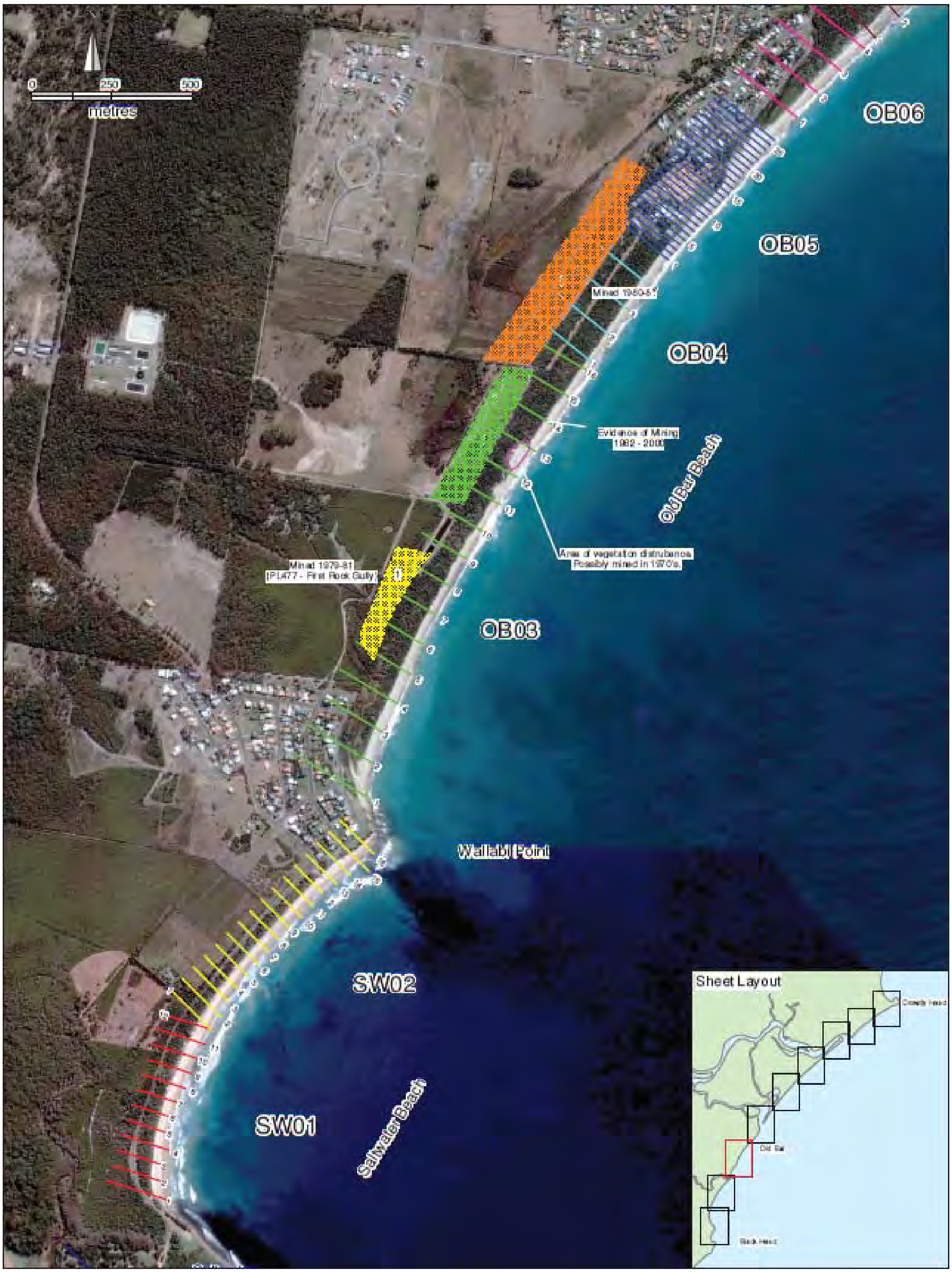

Figure E1 –Location of photogrammetric profiles at Old Bar Beach (southern end)
(Source: WorleyParsons, 2010a)

Figure E2 – Location of photogrammetric profiles at Old Bar Beach (northern end)
(Source: WorleyParsons, 2010a)

Methodology

To assess long term recession rates, changes in the position of the RL 4 contour level was determined at each profile over time. The rates were derived by linear regression; that is, by determining the line of best fit (least squares error) in each case[9]. The advantage of using linear regression, rather than simple differences between the first and last dates of photography, is that

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

errors in predicted rates due to variations in beach states are likely to have been minimised. The analysis was performed using scripts developed by HKA in the software package MATLAB[10].

Analysis Periods

Rates of change were determined for both the entire analysis period considered (1940 to 2013) and shorter periods (1965 to 2013, 2000 to 2013 and 2006 to 2013). Consideration of the analysis period commencing in 1965 is considered relevant as a means to help verify the results for the 1940 to 2013 analysis period. It should also be noted that the 1940 data may not be as accurate as the later dates (WorleyParsons, 2010a). The 1965 to 2013 analysis period is therefore considered most appropriate for defining natural beach change over time.

The analysis periods commencing in 2000 and 2006 are useful for assessing more recent beach change, although these results should be treated with caution. In particular, it should be recognised that the analysis of shorter periods can sometimes be confounded due to lack of data, particularly if the data includes significantly eroded or accreted (or erroneous) profiles. It should also be noted that analysis periods commencing in 2000 and 2006 include only five or three dates of photogrammetry data respectively, which is generally considered insufficient to identify any meaningful trends in recent beach change.

Results

Longitudinal plots (that is versus the position along the beach) of the variation in the rate of positional change at RL 4 are given in **Figure E3** for Old Bar Beach.

South and north of the entrance to Racecourse Creek, average contour recession rates of around 0.6 m/year and 0.8 m/year were determined for the analysis periods commencing in 1940 and 1965. Accretion was generally determined within the entrance to Racecourse Creek, however these results should be treated with caution as they are influenced by catchment flows in addition to coastal processes.

Results for the analysis periods commencing in 2000 and 2006 are more variable than the longer periods, which is expected given the reduced datasets used for these analyses. Rates of positional change for the 2000 to 2013 analysis period indicate that recession rates increase (moving north) from around 1.8 m/year to 2.5 m/year just south of the entrance to Racecourse Creek. North of the creek entrance, recession rates generally reduce from around 3 m/year to approximately 0.7 m/year at the northern end of Block OB7, i.e. near the surf club.

[10] MATLAB is a high-level technical computing language and interactive environment for algorithm development, data visualization, data analysis, and numerical computation.

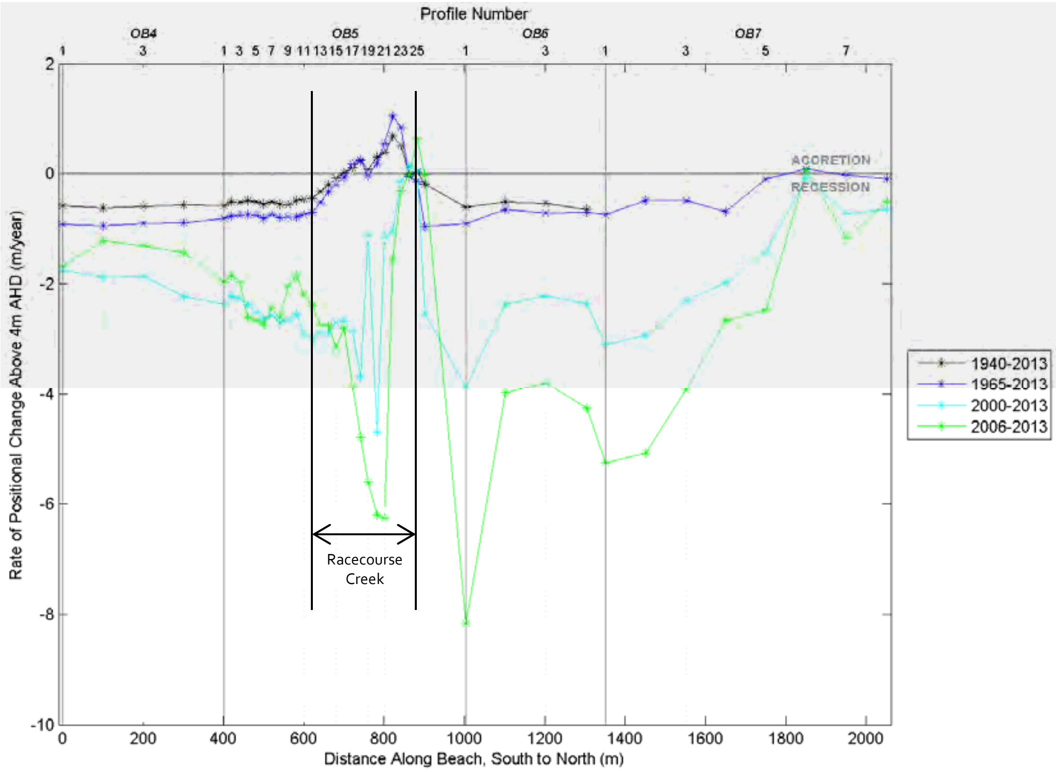

Figure E3 – Rate of positional change at RL 4 at Old Bar Beach

Conclusions

In summary, the long-term recession rate (1965-2013) is likely in the order of 0.8 m/year in the vicinity of Lewis Street (Stage 1 wall zone), possibly reducing towards Racecourse Creek (Stage 2). More recent observations in the Stage 1 zone (2000-2013) suggest that this rate may have increased to around 2.5 m/year.

It would seem reasonable for the WRL assessment of design breaking wave heights and scour levels to adopt 0.8 m/year for Stage 1 and Stage 2 as representative of a likely estimate of long term sediment budget recession over the next 50 years, increased to say 2.5 m/year to reflect a possible extreme recessional trend based on more recent observations. Recession due to sea level rise would need to be included separately.

5th November 2013

WRL Ref: WRL2013107 LR20131031

COMMERCIAL IN CONFIDENCE

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A/Director, Environmental Programs
Regional Operations Group
Office of Environment and Heritage
NSW Department of Premier and Cabinet
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Dear Ms Gibbs,

Old Bar Revetment: Design Wave Heights and Scour Levels

1. Introduction

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at The University of New South Wales is pleased to provide this Letter Report with the findings of the assessment of design wave heights and scour levels at the proposed Old Bar revetment.

Information on the proposed revetment was provided in the following drawings from Royal HaskoningDHV (RHDHV):

- 8A0271-MA- WORKING PLAN;
- 8A0271-MA-CH700;
- 8A0271-MA-1000; and
- 8A0229-MA-1000.

The methodology to assess scour levels and wave conditions at the proposed structure consisted of the following steps:

- Estimation of extreme offshore waves and water levels;
- Transformation of design waves to structure;
- Inclusion of shoreline recession and beach erosion;
- Estimation of scour levels at toe of structure;
- Estimation of local water level and depth at structure; and
- Estimation of design wave conditions at structure.

2. Coincidence of Extreme Waves and Water Levels

Extreme conditions used for designing coastal structures arise from the combination of large waves, high water levels and eroded sand levels. Detailed studies on the joint coincidence of these factors are not available for the study site.

Shand *et al* (2012) examined the joint probability of waves and tidal anomalies (but not eroded sand levels) for Sydney, with an example shown in Figure 1. It can be seen that for 100 year ARI conditions, the offshore significant wave height (for Sydney) varies by less than 1 m for 100 year ARI conditions, whether the tidal anomaly is 0.0 m or 0.4 m.

For the NSW Mid-North Coast, intense low pressure systems such as east coast lows or tropical cyclones cause the largest waves and most elevated water levels. Sand levels also erode in response to such storms. While the coincidence (phasing) of worst cases of these three variables may not occur simultaneously, there are insufficient studies to fully consider different phasing of each variable.

Therefore, as a conservative estimate, it has been assumed that for the ARI considered, the same ARI be applied to each component. That is, it has been assumed that the 100 year ARI (1 hour duration) wave height and water level coincide, together with the 100 year ARI beach erosion level. This is acknowledged to be conservative, however, well accepted (less conservative) alternative methodologies are not available.

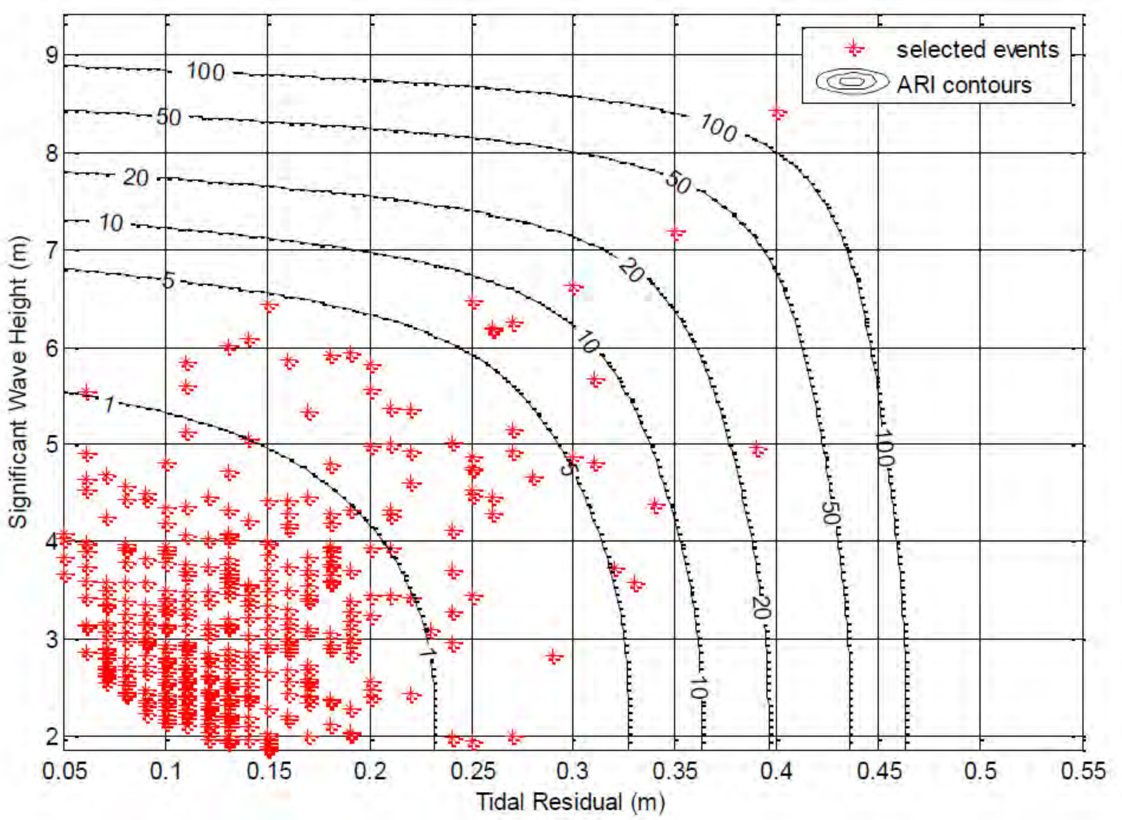

Figure 1: Joint Probability of Waves and Tidal Residuals for Sydney (Shand *et al.*, 2012)

3. Adopted Offshore Design Wave Conditions

3.1 Wave Height

Waves reaching the coast at Old Bar may be modified by the processes of refraction, diffraction, wave-wave interaction and dissipation by bed friction and wave breaking.

WRL, in conjunction with New South Wales Office of Environment and Heritage (OEH, formerly DECCW) have completed an assessment of coastal storms and extreme waves for NSW which involves the identification of all measured coastal storms during the period 1971 – 2009 and derivation of directional design storm events for annual recurrence intervals of 1 to 100 years (Shand *et al.* 2010a). The results from the study for the wave buoy at Crowdy Head and two adjacent wave buoys at Sydney and Coffs Harbour are shown in Table 1.

Table 1: Extreme Offshore Wave Conditions (All Directions) (source Shand *et al.,* 2010a)

Average Recurrence Interval ARI (year)	**One Hour Exceedance H_S** (m)		
	Sydney	**Crowdy Head**	**Coffs Harbour**
1	5.9	5.4	5.2
10	7.5	7.0	6.7
50	8.6	8.0	7.7
100	9.0	8.5	8.1

The capture rates for the three wave buoys are 84.5% (Sydney), 85.6% (Crowdy Head) and 84.7% (Coffs Harbour). WRL has adopted the offshore significant wave heights from the Crowdy Head wave buoy. Note that this assumption does not have a substantial outcome on the design wave conditions at the structure, due to the depth limited nature of waves at the structure toe.

3.2 Wave Period

WRL, in conjunction with the Australian Climate Change Adaptation Research Network for Settlements and Infrastructure (ACCARNSI), reviewed Australian storm climatology and previous extreme wave analyses undertaken using instrument and numerical model data (Shand *et al.,* 2011). Importantly, the study defined the peak spectral wave period during storm events around the Australian coast. The nearest location to the subject site where this analysis was undertaken was Sydney, with results presented in Table 2. The peak spectral wave period associated with 1 in 100 year ARI offshore significant wave heights was adopted as 13 s for this study.

Table 2: Associated Wave Period for Extreme Wave Events (source Shand *et al.,* 2011)

Average Recurrence Interval ARI (year)	**Peak T_P** (s)
	Sydney
1	11.0
10	12.1
50	12.7
100	13.0

3.3 Wave Direction

The closest directional wave buoys to the study site (with long records) are Sydney and Byron Bay. It is noted that all NSW wave buoys are now directional, but do not yet have sufficient data for detailed analysis. In the aforementioned study by WRL (Shand *et al.,* 2010a), WRL also examined the influence of wave direction on extreme storm wave height along the NSW coast. Results showed that for wave events arriving from north of 90°, the extreme values were approximately 75% of the *'all direction'* values, wave events from the east to south-east were approximately 5% lower than the 'all direction' values and waves arriving from south to south-east were typically 100% of the 'all direction' values. WRL adopted the south to south-east direction as the design direction.

4. Wave Transformation

Waves travelling from offshore to the subject site are influenced by the processes of refraction, shoaling, diffraction, friction and breaking. Prior to the breakpoint, wave transformation can be represented by the equation:

$$H_{s\ nearshore} = K\ H_{os} \qquad (1)$$

where $H_{s\ nearshore}$ is the nearshore significant wave height (prior to breaking)
K is a combined coefficient of refraction, diffraction friction and shoaling
H_{os} is deepwater significant wave height

While some wave transformation modelling was undertaken by WorleyParsons (2010), the output was not readily available in a format for application to 100 year ARI conditions. In the absence of a comprehensive numerical wave modelling study for the area, WRL adopted a K value of 1. This is considered realistic for large waves from the south-east. Offshore wave heights are used as deep water input into the Dally *et al.* (1984) surf zone model to estimate wave setup. The nearshore wave height method of Goda (2007) has been used to derive breaker indices and wave heights at the structures.

5. Design Water Levels

5.1 Storm Tide (Astronomical Tide + Anomaly)

Elevated water levels consist of (predictable) tides, which are forced by the sun, moon and planets (astronomical tides), and a tidal anomaly. Tidal anomalies primarily result from factors such as wind setup (or setdown) and barometric effects, which are often combined as "storm surge". Additional anomalies occur due to "trapped" long waves propagating along the coast. Water levels within the surf zone are also subject to wave setup and wave runup.

Design storm surge levels (astronomical tide + anomaly) are recommended in the Coastal Risk Management Guide (DECCW, 2010) based on data from the Fort Denison tide gauge in Sydney and reproduced in Table 3. This is based on approximately 100 years of data at the Fort Denison tide gauge which is not subject to wave setup or river flow effects. However, these levels are primarily applicable in the Newcastle - Sydney – Wollongong area and analysis of local tidal records on the NSW Mid-North Coast is recommended.

Table 3: Design Water Levels Tide + Storm Surge Newcastle – Sydney – Wollongong

Average Recurrence Interval ARI (year)	**Water Level Excl. Wave Setup and Runup** (m AHD) (source DECCW, 2010)
1	1.24
10	1.35
50	1.41
100	1.44

The elevated water levels in Table 3 can be supplemented with additional analyses for other tide gauges in NSW Mid-North Coast undertaken by MHL (2010). However, it should be noted that these are generally based only on approximately 20 years of data and many of the northern NSW tide gauges are subject to river flow effects. The elevated water levels for mid and northern NSW locations (from central estimates in Appendix B of MHL, 2010) are reproduced in Table 4.

Table 4: Extreme Water Levels for Northern NSW Tide Gauges (based on MHL, 2010)

Location	**100 year ARI** (m AHD)
Tweed Heads	1.56
Brunswick Heads	1.48
Ballina	1.58
Yamba	1.43
Coffs Harbour	1.47
Port Macquarie	1.55
Crowdy Head	1.45
Sydney (Fort Denison)	1.44
Adopted for this study	1.44

The adopted 100 year ARI extreme water level conditions (excluding wave setup) for the design is 1.44 m AHD.

5.2 Wave Setup

Wave setup and runup are intrinsically dependent on the determination of the nearshore wave conditions.

To determine the wave setup at the proposed structure, the effective offshore significant wave height H_s was adjusted to the root mean square wave height H_{RMS} according to CIRIA (2007) in Equation (1).

$$H_{RMS} = 0.706 \times H_S \qquad (2)$$

This wave height was applied as a boundary condition to the Dally *et al.* (1984) two-dimensional surf zone model. The bathymetric profile (Ch700) for the model was provided by the RHDHV. The corresponding 100 year ARI peak spectral wave period and storm tide water level were also applied.

The corresponding wave setup and setup water surface level at the different contours along Ch700 profile were determined as shown in Table 5.

Table 5: Wave Setup Estimates Using Dally et al. (1984)

Contour (m AHD)	**Wave Setup** (m)	**Setup Water Surface Level** (m AHD)
2	1.23	2.67
1	1.02	2.46
0	0.91	2.35
-1	0.82	2.26
-2	0.72	2.16
-3	0.47	1.91
-4	0.26	1.70

5.3 Sea Level Rise

Mean sea level on the NSW coast is presently rising at between 1 and 3 mm/year (Watson, 2011; DECCW, 2010). Depending upon the scenario adopted, mean sea level is projected to increase by up to 0.9 m by 2100 by which time it would be rising at 13 mm/year (NCCOE, 2012).

The 2030 and 2050 planning scenarios were considered over the 50 year project life. WorleyParsons (2010) adopted a 0.4 m high-range sea level rise (SLR) for the 2050 planning scenario. Accordingly, WRL adopted 0.4 m of SLR for 2050 (approximately at the end of planning period) and 0.2 m SLR for 2030 (approximately mid-way through the planning period). Corresponding extreme water level conditions (excluding wave setup) were calculated based on the recommendations in DECCW (2010), i.e. by discounting the estimated amount of global average sea level rise that has occurred between 1990 and present:

- 2030, 100 year ARI: 1.57 m AHD;
- 2050, 100 year ARI: 1.77 m AHD.

The (now withdrawn) NSW Government sea level rise policy and risk management guide (DECCW 2009; 2010) reported that global sea level is currently rising at approximately 3 mm/year.

Beach recession due to sea level rise has been considered for the 50 year project life. WorleyParsons (2010) adopted a Bruun Factor of 50 for Old Bar. That is, the beach is estimated to recede by a factor of 50 times the sea level rise. For the subject structure, WRL has assumed that the beach recedes 10 m and 20 m respectively for the 2030 and 2050 planning scenarios.

6. Underlying Recession and Sea Level Rise Recession

The most recently completed analysis of historic photogrammetric data by RHDHV informed the following allowances for recession in the study area:

- Ongoing underlying recession of 0.8 m/year (best/central estimate); and
- Ongoing underlying recession of 2.5 m/year (extreme estimate).

The adopted design profiles for estimating depth limited waves at the structure utilise the 2013 initial profile (at Ch700) and include the horizontal recessions shown in Table 6.

Table 6: Underlying and SLR Recession

Planning Scenario	Underlying Recession Rate (m/yr)	Underlying Recession (m)	SLR (m)	SLR Recession (m)	Total Recession (m)
2030	0.8	13.6	0.2	10	23.6
2050	0.8	29.6	0.4	20	49.6
2030	2.5	42.5	0.2	10	52.5
2050	2.5	92.5	0.4	20	112.5

7. Storm Erosion

WorleyParsons (2010) estimated design (nominally 100 year ARI) storm demand of 220 m^3/m above AHD consistent with the storm demand for an exposed NSW beach at a rip head (Gordon, 1987). While this volume would normally be applied to a long term average or accreted initial profile, due to the high observed rate of recession at Old Bar, the storm erosion was applied to the most recent (2013) profile.

8. Reference Profile

Profile data was provided by the RHDHV. The profile used was at Chainage 700 m (RHDHV Drawing No 8A0271-MA-1000) corresponding to Profile 16, Block Y in the OEH photogrammetry dataset. The sub-aerial profile (landward of approximately 0 m AHD) was from a laser scan survey conducted in September 2013 while the offshore profile (seaward of approximately -3 m AHD) was from a hydrosurvey undertaken in September-December 2009 (Figure 2).

A Dean (1977) equilibrium profile for a median sand size of 0.30 mm (Surf Life Saving Australia database) was used to join the two profiles (approximately 70 m missing).

For the 2030 and 2050 planning scenarios the structure, the reference profile has been modified as shown in Table 6.

When considering scour levels, the 100 yr ARI storm demand of 220 m^3/m above AHD was considered.

9. Design Scour Level

A range of options were canvassed regarding determination of the design scour level. These are indicated below:

- Engineering "rules of thumb";
- Photogrammetry;
- Erosion modelling;
- Published data on profile change such as Gordon (1978) and Chapman and Smith (1983).
- Other allowances using a Dean (1977) equilibrium profile.

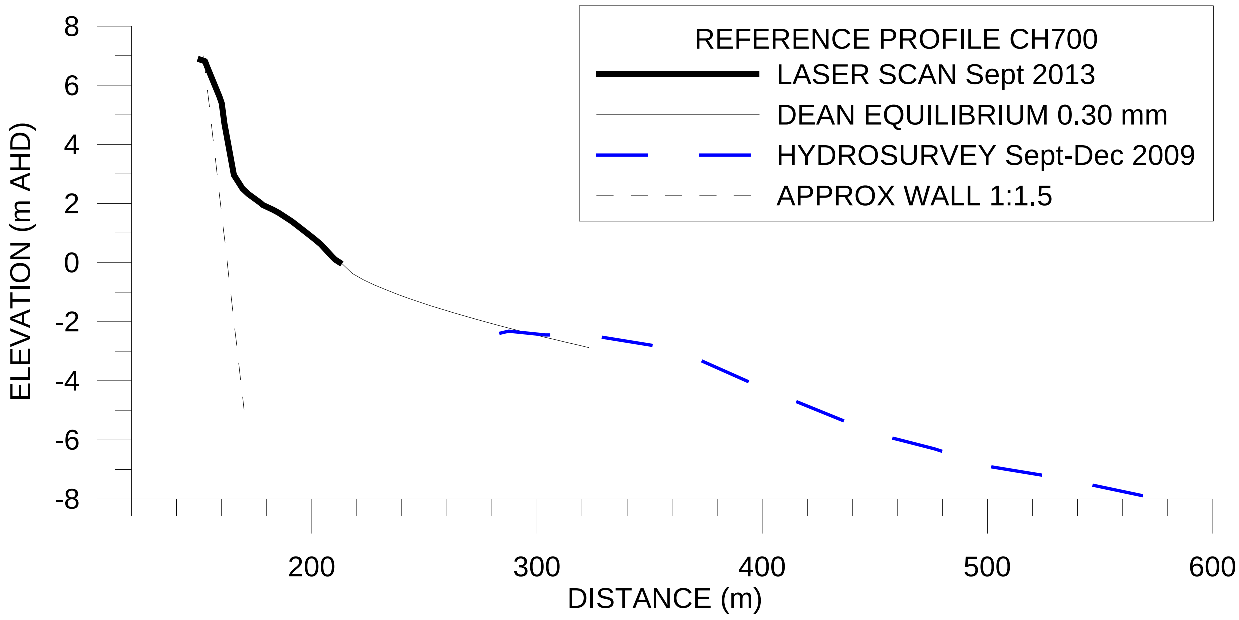

Figure 2: Chainage 700 m Reference Profile based on 2013 and 2009 Data (source RHDHV)

9.1 Rules of Thumb

In NSW, the scour level of approximately -1.0 m AHD is commonly adopted as an engineering rule of thumb for rigid coastal structures located at the back of the active beach area, with -2 m AHD frequently adopted for vertical coastal structures due to increased wave reflections. This is based on stratigraphic evidence of historical scour levels and observed scour levels occurring during major storms in front of existing permeable and non-permeable seawalls along the NSW coast (Nielsen *et al*. 1992; Foster *et al*. 1975). While not directly applicable to Old Bar, for seawalls constructed on the NSW Maritime Authority's land a minimum allowance of 0.6 m for scour from the seaward face of the seawall is required unless the seawall is founded on rock (NSW Maritime, 2005).

9.2 Photogrammetry

Photogrammetry, depending on the water level at the time of the aerial photograph, generally does not extend out to levels below approximately 0 m AHD, so cannot be used to determine extreme historical scour levels.

9.3 SBEACH Modelling

In additional to wave setup modelling, WRL undertook two-dimensional modelling of beach erosion using SBEACH (version 4.03). The SBEACH model is a two-dimensional numerical cross-shore sediment transport and profile change model developed by the United States Army Corps of Engineers, Coastal Engineering Research Center. Details of the model are given in Larson and Kraus (1989) and Larson, Kraus and Byrnes (1990). SBEACH considers sand grain size, the pre-storm beach profile and dune height, plus time series of wave height, wave period and water level in calculating a post-storm beach profile.

The process for confirming the design scour level for each structure using SBEACH is outlined in the following discourse.

Firstly, the design erosion volume (storm demand/storm bite) for Old Bar without a structure in place was established for the 100 year ARI of 220 m^3/m above AHD (as recommended in WorleyParsons, 2010). Secondly, a time series of consecutive, synthetic storm events (Shand *et al*., 2011) was applied in SBEACH without a structure in place until the change in dune volume matched the adopted storm demand. Thirdly, a structure was introduced such that erosion of the dune is prevented. Finally, the time series of storm events (which resulted in the adopted storm demand without a structure in place) was modelled in SBEACH with a structure in place to estimate the scour level at the toe of each structure design.

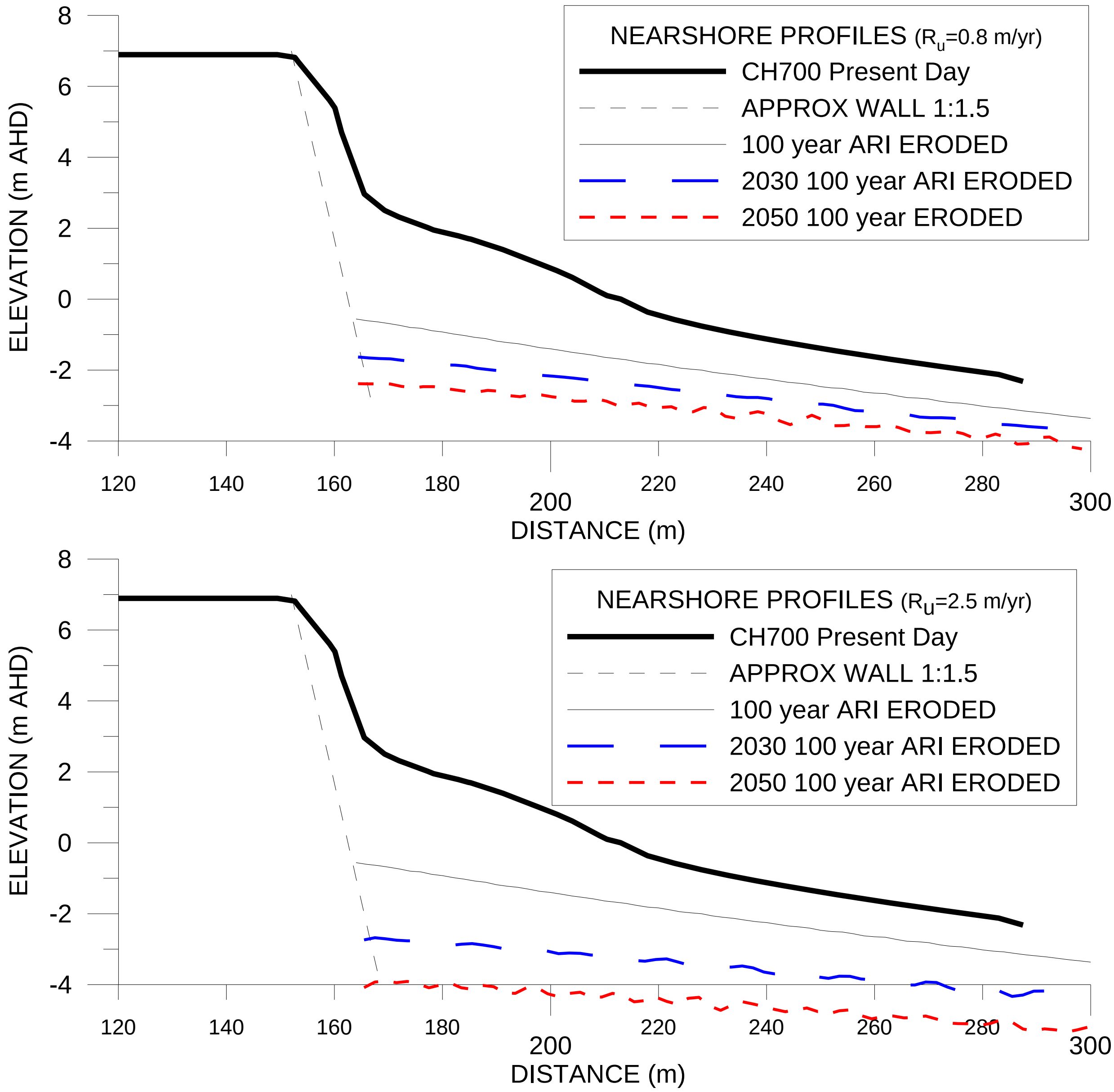

Figure 3: Initial and Eroded Profiles at Seawall using SBEACH for Present Day and Future Planning Scenarios (using 0.8 and 2.5 m/yr Underlying Recession)

The SBEACH modelling found scour levels fronting the structures as shown in Table 7 and Figure 3. For future planning scenarios (2030 and 2050) two rates of ongoing underlying recession were adopted: 0.8 m/year and 2.5 m/year (Section 6).

Table 7: Scour Levels as Estimated using SBEACH

Planning Scenario	**Scour Levels** (m AHD)
Present Day	-0.6
2030 (0.8 m/yr)	-1.6
2050 (0.8 m/yr)	-2.4
2030 (2.5 m/yr)	-2.7
2050 (2.5 m/yr)	-4.1

9.4 Published Profile Change

Gordon (1987) published the expected range of vertical change on the NSW coast as a function of average sand levels. Chapman and Smith (1983) introduced the concept of "swept prism" based on approximately 9 years of ongoing measurements on the Gold Coast. Results from these methods are shown in Table 8. For Old Bar, assuming an average sand level against the structure of +4 m AHD, the minimum expected sand level at the structure from these methods is 1.25 m AHD. This estimate is considered un-conservative and improbable due to the high recession rates characterising the study site.

Table 8: Vertical Change of Reference Elevations from Field Measurements

	Vertical Change from Reference			
Average sand level (m AHD)	**Gordon (1987) High Demand** (m)	**Gordon (1987) Low Demand** (m)	**Chapman and Smith (1983)** (m)	**Minimum estimated sand level from these references** (m AHD)
+4	± 2.75	± 2.0	± 2.25	1.25
+2	± 2.5	± 1.9	± 2.75	-0.75
0	± 2.25	± 1.8	± 2.75	-2.75

9.5 Adopted Scour Depth Fronting Structure

The estimated scour level from a range of techniques is shown in Table 9. The SBEACH modelling (and the rule of thumb) specifically consider enhanced scour due to the structures and is recommended as the basis of the adopted scour level. The SBEACH values were modified by using a Dean (1977) equilibrium profile and receding it (Section 8) to account for ongoing recession and sea level rise. A plunge length (SPM, 1984 and Section 3) of 10 m has also been allowed on the Dean (1977) equilibrium profile.

Table 9: Scour Levels at Toe of Structure (Underlying Recession of 0.8 m/yr)

Method	Scour Level (m AHD)		
	Present Day	2030	2050
SBEACH modelling (assuming 0.8 m/yr underlying recession)	-0.6	-1.6	-2.4
*Chapman and Smith (1983)	1.75	-0.75	-2.75
*Gordon (1987)	1.25	-0.5	-2.25
Rule of thumb	-1.0	-	-

Notes: *values are presented with minor rounding and assuming +4, +2 and 0 m AHD average sand level against structure respectively for present day, 2030 and 2050

10. Nearshore Wave Heights

For the 100 year ARI wave, water level and eroded profile condition, depth limited nearshore wave heights were estimated using the method of Goda (2007) for significant wave heights and Battjes and Groenendijk (2000) for $H_{10\%}$ and $H_{2\%}$. Results are summarised in Table 10 and Table 11.

11. Summary of Adopted Design Conditions

The adopted design conditions presented above are summarised Table 10 and Table 11.

Table 10: Summary of Design Conditions Estimated for 100 yr ARI Assuming 0.8 m/yr of Underlying Recession

Variable	Present Day	2030	2050
Underlying beach recession (0.8 m/yr)	n/a	13.6 m	29.6 m
Sea level rise	n/a	0.2 m	0.4 m
Additional beach recession due to SLR	n/a	10 m	20 m
Design storm demand (above AHD)	220 m^3/m	220 m^3/m	220 m^3/m
Design offshore significant wave height (Hso)	8.5 m	8.5 m	8.5 m
Design offshore significant wave direction	South-east	South-east	South-east
Wave transformation coefficient (K)	1.0	1.0	1.0
Design still water level (excluding wave setup)	1.44 m AHD	1.57 m AHD	1.77 m AHD
Design spectral peak wave period Tp	13 s	13 s	13 s
Inshore wave setup at 0 m AHD contour	0.91 m	0.91 m	0.91 m
Design nearshore water level at 0 m AHD contour	2.35 m AHD	2.48 m AHD	2.68 m AHD
Inshore wave setup at −1 m AHD contour	0.82 m	0.82 m	0.82 m
Design nearshore water level at −1 m AHD contour	2.26 m AHD	2.39 m AHD	2.59 m AHD
Inshore wave setup at −2 m AHD contour	0.72 m	0.72 m	0.72 m
Design nearshore water level at −2 m AHD contour	2.16 m AHD	2.29 m AHD	2.49 m AHD
Eroded toe elevation of structure (Ch700,X=164m, SBEACH)	-0.6 m AHD	-1.6 m AHD	-2.4 m AHD
Plunge length of breaking wave	~10 m	~10 m	~10 m
Design nearshore water depth for structure (ds)	3.1 m (-0.8 m AHD contour)	4.0 m (-1.7 m AHD contour)	4.9 m (-2.5 m AHD contour)
Breaker index for Hs (Goda, 2007)	0.58	0.57	0.56
Design Hs at structure (Goda, 2007)	1.79 m (-0.8 m AHD contour)	2.30 m (-1.7 m AHD contour)	2.71 m (-2.5 m AHD contour)
Breaker index for $H_{10\%}$ (Battjes and Groenendijk, 2000)	0.71	0.73	0.74
$H_{10\%}$ at structure (Battjes and Groenendijk, 2000)	2.20 m	2.94 m	3.60 m
Breaker index for $H_{2\%}$ (Battjes and Groenendijk, 2000)	0.75	0.77	0.78
$H_{2\%}$ at structure (Battjes and Groenendijk, 2000)	2.30 m	3.10 m	3.80 m

Table 11: Summary of Design Conditions Estimated for 100 yr ARI Assuming 2.5 m/yr of Underlying Recession

Variable	Present Day	2030	2050
Underlying beach recession (2.5 m/yr)	n/a	42.5 m	92.5 m
Sea level rise	n/a	0.2 m	0.4 m
Additional beach recession due to SLR	n/a	10 m	20 m
Design storm demand (above AHD)	220 m^3/m	220 m^3/m	220 m^3/m
Design offshore significant wave height (Hso)	8.5 m	8.5 m	8.5 m
Design offshore significant wave direction	South-east	South-east	South-east
Wave transformation coefficient (K)	1.0	1.0	1.0
Design still water level (excluding wave setup)	1.44 m AHD	1.57 m AHD	1.77 m AHD
Design spectral peak wave period Tp	13 s	13 s	13 s
Inshore wave setup at 0 m AHD contour	0.91 m	0.91 m	0.91 m
Design nearshore water level at 0 m AHD contour	2.35 m AHD	2.48 m AHD	2.68 m AHD
Inshore wave setup at −1 m AHD contour	0.82 m	0.82 m	0.82 m
Design nearshore water level at −1 m AHD contour	2.26 m AHD	2.39 m AHD	2.59 m AHD
Inshore wave setup at −2 m AHD contour	0.72 m	0.72 m	0.72 m
Design nearshore water level at −2 m AHD contour	2.16 m AHD	2.29 m AHD	2.49 m AHD
Eroded toe elevation of structure (Ch700, X=164m, SBEACH)	-0.6 m AHD	-2.7 m AHD	-4.1 m AHD
Plunge length of breaking wave	~10 m	~10 m	~10 m
Design nearshore water depth for structure (ds)	3.1 m (-0.8 m AHD contour)	4.9 m (-2.8 m AHD contour)	6.1 m (-4.1 m AHD contour)
Breaker index for Hs (Goda, 2007)	0.58	0.56	0.55
Design Hs at structure (Goda, 2007)	1.79 m (-0.8 m AHD contour)	2.72 m (-2.8 m AHD contour)	3.38 m (-4.1 m AHD contour)
Breaker index for $H_{10\%}$ (Battjes and Groenendijk, 2000)	0.71	0.77	0.82
$H_{10\%}$ at structure (Battjes and Groenendijk, 2000)	2.20 m	3.74 m	5.00 m
Breaker index for $H_{2\%}$ (Battjes and Groenendijk, 2000)	0.75	0.81	0.86
$H_{2\%}$ at structure (Battjes and Groenendijk, 2000)	2.30 m	3.95 m	5.28 m

12. Summary

Thank you for the opportunity to provide this report. Please contact James Carley in the first instance should you require further information.

Yours faithfully,

G P Smith
Manager

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