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WATTLE RANGE COUNCIL

Appendix 2 - Lake George

Groyne Extension Concept Design

301015-03541 – 002

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PROJECT 301015-03541 - APPENDIX 2 - LAKE GEORGE

REV	DESCRIPTION	ORIG	REVIEW	WORLEY-PARSONS APPROVAL	DATE	CLIENT APPROVAL	DATE
A	Issued for internal review	C Adamantidis	A Nielsen	A Nielsen	29-Nov14	N/A	
B	Draft	C Adamantidis	A Nielsen	A Nielsen	29-Nov-14		
C	Final	C Adamantidis	A Nielsen	A Nielsen	28-Oct-15		

CONTENTS

1 METHODOLOGY

This Appendix presents the derivation of the estimated wave conditions in the nearshore area of the Beachport for the conceptual design of the groyne extensions at the outlet to Lake George.

The nearshore design wave estimates were based on the transformation of offshore wave conditions through numerical modelling to the project site. The principal aim of establishing the extreme wave conditions is to provide initial estimates of design wave conditions for the existing coastal structures along the Beachport foreshore. The offshore boundary conditions were based on published extreme wave data collected by offshore wave buoys and metocean hindcasts.

It should be noted that numerical models as applied herein are a tool that can provide estimates of the physical response of the coastal system, based on its calibration and capacity to replicate measured phenomena. The models used herein provide an estimate of the design nearshore water levels and wave heights that would apply at the various coastal structures and are thus applicable for assessment of their structural stability and conceptual design of an upgrade of the structures. It should be noted that there can be departures between a model output and the actual physical response, as not all processes are able to be replicated by numerical approximations. However, the model results provide the best available estimates of the design parameters for assessment of the performance of the coastal structures.

Nearshore wave conditions were derived using a 3rd generation 2D(H) spectral wave model (SWAN). As the spectral wave model is not able to model wave induced setup, a 1D roller model, which is able to model wave induced setup (SBEACH), was used to transform the nearshore wave conditions to the shore. The use of the SWAN and SBEACH models together provides a three dimensional solution algorithm for wave transformation across the surf zone to shore and gives a far better result than that obtained from a 1D solution alone, such as GENESIS or LITPAC.

Armour stability for the existing structures was assessed using the Hudson equations with the derived nearshore design wave height for the 1 year, 10 year and 100 year ARI wave events.

2 WAVE MODELLING

Both SWAN and SBEACH models were used for the wave transformation modelling. The SWAN model (version 40.85) (Delft University of Technology, 2011) was used to derive transformed nearshore wave heights for the range of offshore wave directions and periods comprising the long term wave climate to establish suitable input boundary conditions for the SBEACH surfzone wave transformation model. The SBEACH model was utilised to describe in detail the surfzone wave transformation processes for the determination of nearshore wave setup water levels and wave heights at the structures, necessary for assessment of their structural stability.

The SWAN and SBEACH wave transformation modelling is presented in detail in the Rivoli Bay Study (WorleyParsons 2014).

2.1 SWAN Model Results

The SWAN model was run for an offshore wave height of 1 m (to obtain wave height coefficients), for all wave directions ranging from south to north-east. It was found that the largest wave height coefficients occurred when the offshore wave direction was from the south at Beachport. WRL (2013) found that the highest significant wave heights along the coastline adjacent to western Victoria occur from the west-southwest directions (225 – 270°). WRL (2012) adopted a 10 year ARI significant wave height of 5.9 m for this section of coast for directions between east and south compared with a much higher significant wave height of 9.5 m for waves from the west. Despite waves from the south being lower offshore than waves from the west, due to wave refraction around Penguin Island, southerly waves result in the highest waves at the Beachport shoreline.

Wave transformation coefficients for a peak wave period of 15 s at selected nearshore locations in 3 m water depth at Beachport for the range of offshore wave directions modelled are illustrated in Figure 1. It can be seen from these plots that at Beachport, the peak wave energy arrives at the foreshore when offshore wave direction is from the south.

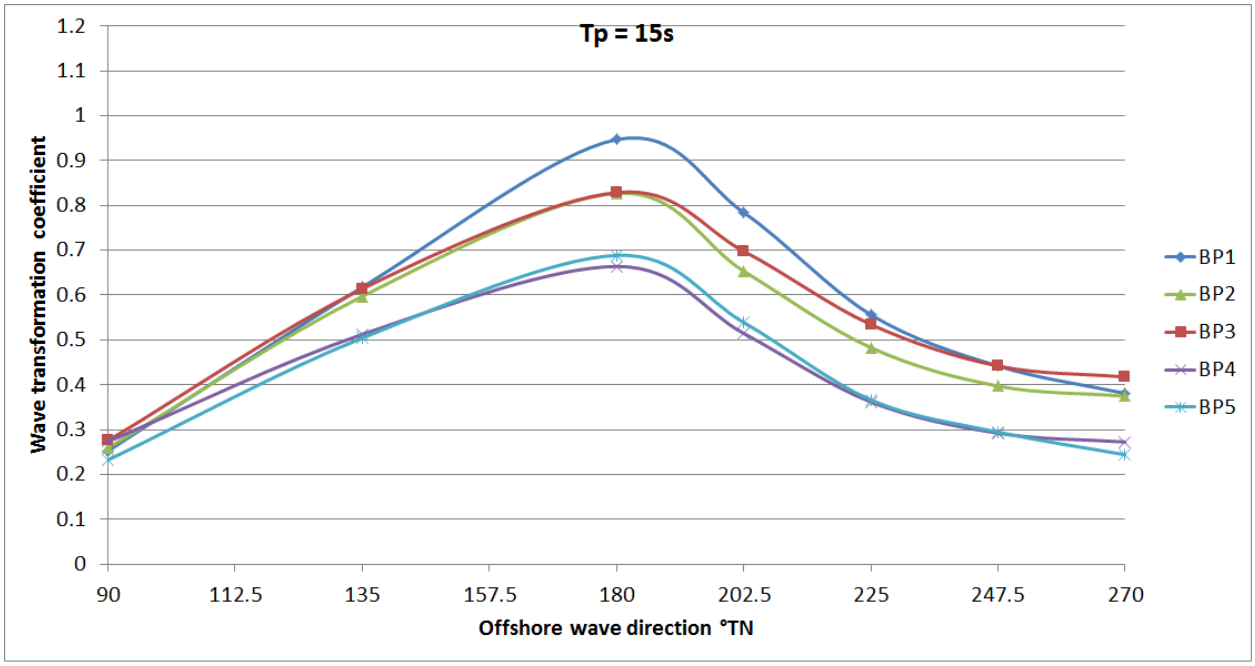


Figure 1 – Nearshore wave transformation coefficients at five locations in Beachport vs. offshore wave direction

2.2 SBEACH Modelling

To establish the wave conditions at the various groynes and revetments, the SBEACH model (Rosati *et al.*, 1993) has been used. SBEACH simulates beach profile change, including the formation and movement of major morphologic features such as longshore bars, troughs, and berms, under varying storm waves and water levels. The model is empirically based and was developed originally from a large data set of net cross-shore sand transport rates and beach profile change observed in large tanks. Along with beach profile changes SBEACH is able to simulate depth induced wave breaking, shoaling, wave generation due to wind and wave induced setup.

There are no site wave data within the study area with which to validate the program. However, the SBEACH algorithms have been validated for the Australian eastern seaboard at numerous sites (Carley, 1992; Carley *et al*., 1998).

The SBEACH modelling is described in detail in the Rivoli Bay Study (WorleyParsons 2014).

2.2.1 Results

As the wave conditions are depth limited, the design wave for the structures would be the largest wave that breaks on the structure. This corresponds to the largest wave that is half a wavelength seaward of the seawall or groyne.

The SBEACH model allows the determination of nearshore water level conditions to be estimated, including the effects of wave setup. Based on these water levels, a maximum breaking wave height at each of the groynes was able to be estimated. As the nearshore wave height is controlled by the water depth, there is an increase in wave height in front of the structures with the rarer events, which is due to the influence of wave setup. Given the relatively small water depths involved, it is clear that climate change sea level rise has the potential to increase significantly the size of the incident breaking wave heights.

The most important parameters for assessing the stability of the groynes and revetments are the breaking wave height in front of the structure, the scour level at the structure toe and the water level at the structure. These parameters determine the effectiveness of the existing works, such as the stability of the existing rock armour and the probability that the structures would be overtopped. The results from the SBEACH model provide the variation in these design parameters along the entire foreshore at Beachport. Generally:

- The present day maximum wave height approaching the structures at Beachport ranges from 0.9 – 2.0 m for the 1 year ARI, 1.5 – 2.4 m for the 10 year ARI and 1.9 – 2.7 m for the 100 year ARI storm events. These wave heights do not include shoaling – the breaking wave height (H_b) at the structures would be larger than these due to shoaling, which is derived separately.
- The maximum water levels (including the effects of wave setup at the foreshore) vary from 2.3 – 3.7 m AHD for the 1 year ARI, 2.6 – 4.4 m for the 10 year ARI event and 3.0 – 5.1 m for the 100 year ARI event. Wave setup at the foreshore is significant in the extreme events due to the shallow nature of Rivoli Bay and extensive wave breaking that occurs during these events.

The largest wave heights occur where the coastal structures are located furthest seaward along the beach profile (i.e. at the Beachport jetty), due to the profile being deeper at these locations. The variation in wave height along the foreshore is a function of the nearshore water depth at a point half a wavelength in front of the foreshore structures. Water level variations between profiles are a function of the wave setup calculated by SBEACH at the measurement point half a wavelength in front of the structures. The maximum wave heights obtained from SBEACH (not including shoaling) were found to be around 2.5 m – with shoaling, this would result in a breaking wave height at the structure of around 3.7 m for a 100 year ARI event.

3 HYDRAULIC ARMOUR STABILITY OF ROCK GROYNES

The results of the wave modelling have been used to assess the size of rock required to protect the groyne extensions against wave attack, for the 1 year, 10 year and 100 year ARI storm events.

3.1 Rock Armour Stability

The stability of the primary armour against wave attack has been assessed using the Hudson equation. Another commonly used formulation for rock armour sizing is the Van der Meer equation. This equation, however, is only applicable for deep water conditions (i.e. where the depth in front of the structure is greater than three times the significant wave height in front of the structure, CIRIA, CUR, CETMEF 2007). The conditions at the groynes and revetments are shallow water conditions and the structures will be subject to breaking waves. Hence, the van der Meer formulae are inappropriate for use in assessing the structure stability and the Hudson formula has been used for the calculation of the requisite armour size for the structure.

The Hudson equation is given by:

$$W = \frac{w_r H^3}{K_D (S_r - 1)^3 \cot\theta}$$

where:

W	=	Weight of an individual armour unit in the primary cover layer, kg;
w_r	=	unit saturated surface dry density, kg/m^3
H	=	design wave height at the structure site, m (corresponding to H_{max})
S_r	=	specific gravity of armour unit, relative to the water density at the structure
θ	=	angle of the structure slope, measured in degrees
K_D	=	stability coefficient which depends primarily on the shape of the armour units, roughness of the armour unit surface, sharpness of edges and the degree of interlocking achieved during placement

The above formula is based on comprehensive physical model investigations at the U.S. Army Corps of Engineers.

The variable w_r depends on the properties of the available rock. A flatter slope or higher stability coefficient (K_D) value leads to a decrease in required armour stone weight, W.

Armour units that consist of rough quarried stone will have a higher K_D value than smooth, rounded armour stones. A higher K_D value can be achieved by special placement of the armour stones to achieve a high degree of interlocking. Random placement of the stones leads to a lower value of K_D, which could lead to the required armour stone size W exceeding that available.

Incorporated within the K_D value are variables such as the angle of incidence of wave attack, size and porosity of the underlayer material, revetment crest width and the extent of the revetment slope below the still water level. Table 1 gives recommended values of K_D to use for different situations (after CERC, 1984).

Table 1 – K_D values for Determining Quarrystone Weight*

Armour Units (Quarrystone)	Number of layers 'n'	Placement	Slope Cotangent	Structure Trunk		Structure Head	
				Breaking Wave	Non-breaking Wave	Breaking Wave	Non-breaking Wave
Smooth rounded	2	Random	1.5 – 3.0	1.2	2.4	1.1	1.9
Smooth rounded	>3	Random		1.6	3.2	1.4	2.3
Rough Angular	1	Random			2.9		2.3
Rough Angular	2	Random	1.5	2.0	4.0	1.9	3.2
			2.0			1.6	2.8
			3.0			1.3	2.3
Rough Angular	>3	Random		2.2	4.5	2.1	4.2
Rough Angular	2	Special		5.8	7.0	5.3	6.4
Parallelpiped	2	Special		7.0 – 20.0	8.5 – 24.0		
Graded Angular		Random		2.2	2.5		

*After CERC, 1984

The results from the Hudson analysis assume that no damage to the profile is allowed (static design). This means that there is no difference in the structure cross-section before and after a storm. If 0 – 5% of the armour stones are displaced between the crest and a level of one wave height below still water, this corresponds to "no damage" according to the Hudson formulation and would be acceptable for design (CIRIA, CUR, CETMEF 2007).

From Table 1, a revetment consisting of two layers of rough angular armour stones randomly placed and subject to breaking waves corresponds to a K_D value of **2.0**. This value has been adopted for the analysis of the rock revetment and groyne structures within Rivoli Bay.

To calculate the required stone diameter from the weight, it has been assumed that the bulk density of the rock boulders in the revetments within the Rivoli Bay embayment is 2300 kg/m^3 (pers. comm. Wattle Range Council). The assumed density is based on the specific gravity of the locally-sourced limestone/sandstone rock typically used for construction of the coastal structures within Rivoli Bay as measured by staff from Wattle Range Council.

Based on a 10 year ARI design standard, the required parameters for the groyne rock size are provided in Table 2. The conceptual design of the groyne is provided in Figure 2.

Table 2 – Hudson Analysis for 10 year ARI

Structure No.	SBEACH Profile	Average Slope 1V:XH	H_b[1] 10yr	Hudson W_{50} 10yr (kg)	Hudson D_{50}[2] 1yr (mm)	Estimated Design standard
G10 - G12	BP5	2	2.7	5700	1500	10yr ARI

1. Breaking wave height is calculated using linear wave theory at a point approximately 10 m in front of the structure (i.e. equal to the plunge width of the wave), for the water depth resulting from scour and wave setup determined for the 1 year ARI. The breaking wave height includes a calculated factor for wave shoaling. The breaking wave height at the toe of the structure will be reached for offshore deepwater wave heights much lower than the 1 year ARI.
2. Required median diameters for rock armour for the structures have been derived assuming a bulk density of 2300 kg/m^3 for the locally available rock.

3.2 Underlayers

Secondary armour layers are typically designed to have a median mass about one-tenth of the primary armour rock mass. Core layers are typically designed using filter rules to control their sizing to prevent wash through of fine materials through the overlying armour layers. These aspects would be considered further in the detailed design phase.

Geotextile is recommended at the base of the groyne to prevent sand being washed through the groyne, which would result in settling of the groyne over time.

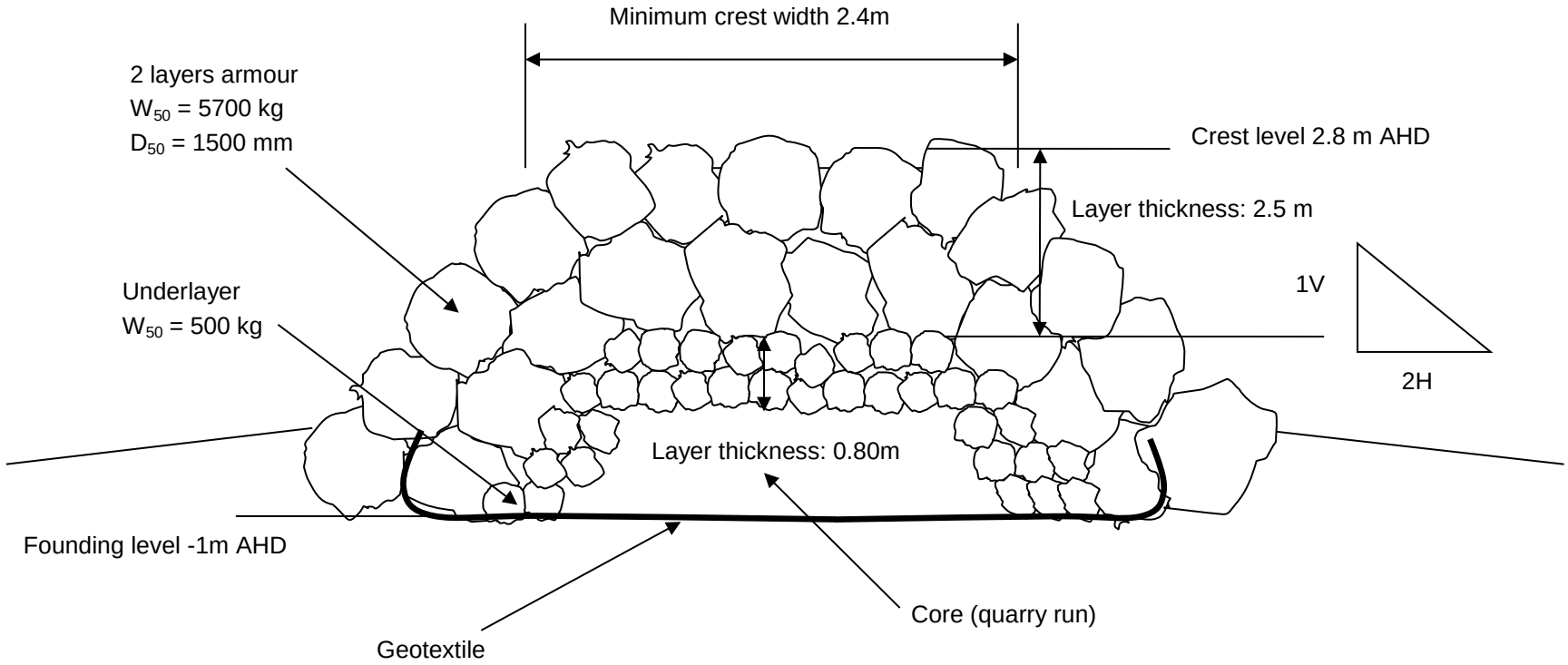
3.3 Toe level

Toe levels have been set based on expected beach scour levels, to prevent the groynes being undermined by wave action. The allowance for scour provides for a toe level of -1 m AHD.

3.4 Length of groynes

It is considered that the appropriate seaward limit for the groynes would depend on a balance between capital cost and ongoing sand nourishment maintenance requirements. Given the very shallow bathymetry of Rivoli Bay, sand can be transported by wave-driven longshore currents to levels of at least -3 m AHD. At these depths, the active littoral zone extends up to 200 m offshore of the beach. As the groynes extend to levels less than 1 m below AHD, they are subject to bypassing, indicating that they may not be long enough to compartmentalise the beach effectively.

It is considered for the Lake George outlet that an appropriate compromise between cost and ongoing maintenance of sand in the outlet channel downstream of the regulator structure would be to extend the groynes to the -2 m bathymetric contour, or by around 100 m.



NOT TO SCALE

SECTION VIEW

Figure 2 - Conceptual design for groyne extension at Lake George outlet

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