Our Ref NA59914038/L001 :sge

Contact P.D. Treloar

Cardno
Shaping the Future

24 September 2013

Mr Colin Scully
14 Macfarlane Street
Davidson NSW 2085

Cardno (NSW/ACT) Pty Ltd
ABN 95 001 145 035

Level 9, The Forum
203 Pacific Highway
St Leonards New South Wales 2065
PO Box 19
St Leonards New South Wales 1590
Australia

Telephone: 02 9496 7700
Facsimile: 02 9439 5170
International: +61 2 9496 7700

Web: www.cardno.com.au

Dear Sir,

COASTAL ENGINEERING REPORT: 55 ROBERTSON ROAD, SCOTLAND ISLAND

Preamble

This technical letter presents a summary of our analyses undertaken in support of your submission for Design Approval to Pittwater Council in relation to your proposed boatshed extension. The report presents summary findings of the following analyses:

- Moderate to severe ARI surge levels, wave heights and run-up levels
- Vertical wave impact forces for combined waves and water levels
- Horizontal wave impact force
- Rock size required for the boulder retaining wall
- Additional safety considerations

Proposed Development

Construction of a boatshed, deck and stairway along the water front is proposed. Details of the proposed development are shown on four drawings prepared by Jack Hodgson Consultants and labelled job number 24417. The drawings are labelled A01-A04 and dated 28th June 2003.

Site Visit

An experienced coastal engineer was escorted to the property by Mr Collin Scully on Tuesday 17th September. Arrival at site was at 9:20am. Weather conditions were fair to sunny, with some light cloud patches and a gentle wind approaching from the South East. The predicted tidal level at Fort Denison at the time of the visit was +0.9m LAT (~0m AHD), or MSL.

The property has a northerly aspect and is accessed by boat. The foreshore at the site is an exposed rock platform at the toe of the property garden, which gently slopes down to about -1m AHD at the end of the boat pier. An embankment (slope approximately 1 in 1) rises steeply from the landward edge of the rock platform, before easing at the northern edge of the house. Slumping has occurred near to the middle of this bank.

The property boat pier and seaward frontages of the adjacent properties were in good visual condition and appeared to be in sound repair. **Figures A1** and **A2** (**Appendix A**) show the northern aspect and foreshore of the property; **Figure A3** shows the rock platform in cross-section relative to the pier deck level.

Figure A4 shows the shore of the property as viewed from the top of the embankment.

Environmental Criteria

Storm Tide and Wave Activity

The level of risk to structures along the foreshore is governed by the severity of impacting waves in combination with the still water level at the time of the peak storm event. Higher water levels provide greater access for waves to impinge upon the foreshore and affect coastal structures. **Figure 1** describes the important coastal processes at the site.

Cardno (2013) [*Pittwater Foreshore Floodplain Mapping of Sea Level Rise Impacts*] provides wave height, period, elevated water levels and wave run-up data throughout the Pittwater estuary for the 100-years Average Recurrence Interval (ARI). Lower return period water levels were taken from other engineering analysis of tide and surge levels at Fort Denison (OEH, 2012). The 100-years ARI significant wave height was calculated to be 1.0m. However, given the relatively sheltered nature of the site, it is possible that a 1.0m storm wave height may occur more frequently than ARI 100-years. Thus for engineering prudence, a 1.0m significant wave height is used as the basis to derive design information in conjunction with a variety of still water levels.

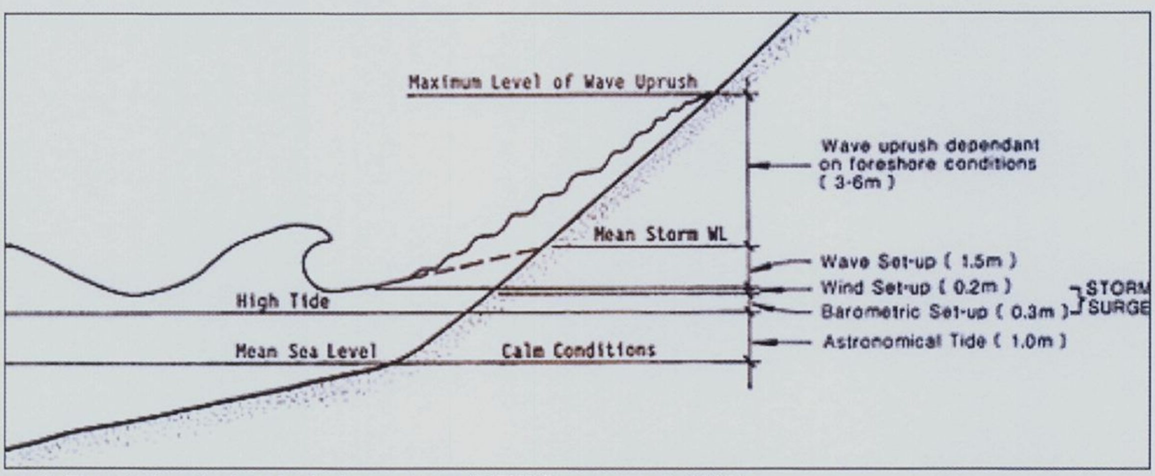

Figure 1: Elevated water levels during a storm (NSW Government, 1990)

Wave Run-Up

The level of wave run-up achieved relative to the still water level is dependent on the foreshore type and the height of the foreshore edge (crest level of natural shoreline or a structure). All the storm tide levels considered in **Table 1** exceed the crest level of the rock platform comprising the foreshore fronting the property. Thus, during storm tides, waves will be able to directly surge over the foreshore crest and onto the steeply sloping bank fronting the property, albeit with some attenuation due to the reduction in depth below the wave as it crosses the rock platform. **Table 1** shows storm tide levels and estimated run-up levels assuming waves breaking on a rocky foreshore with a 1 in 5 slope.

Table 1: Storm tide and estimated wave run-up levels (present – day sea level)

Scenario	'Frequent' (HAT)	'Moderate' (1-years ARI)	'Severe' (20-years ARI)	'Extreme' (100-years ARI)
Storm Tide Level	+1.08m AHD†	+1.24m AHD*	+1.38m AHD *	+1.53m AHD††
Wave height (H_{m0})	1.0m	1.0m	1.0m	1.0m
Wave run-up	+1.88m AHD	+2.04m AHD	+2.18m AHD	+2.33m AHD

† Sand Point Tide Gauge; *Fort Dennison (OEH, 2012); †† Cardno 2013

Water level and total wave run-up levels given in **Table 1** ignore projected Sea Level Rise estimates. These are +0.4m to the year 2050 and 0.9m to the year 2100. These levels may be added to the storm tide and wave run-up levels given in **Table 2.**

Design Risk

Freeboard

It is likely that the floor of the boat-shed will be affected by wave uplift forces. These forces depend upon floor level, wave parameters and water level. Hence a range of conditions can occur and wave uplift forces must be calculated for those cases in order to determine the design loading.

Table 2 describes the associated freeboard levels of the boat-house floor assuming a floor level of +1.7m AHD, as given in Architect drawing 24417-A03. It should be noted that the floor level remains above the present-day 100-years ARI still water level. However, the floor level submerges at the 1-year ARI level (based on analysis at Fort Denison water level data) with the inclusion of a 1.0m incident significant wave. Assuming a less severe incident wave height of 0.5m, the boat-shed floor remains dry at the 20-years ARI level (neglecting contributions from wave reflection).

Table 2: Freeboard heights for storm tide levels plus 0.5m and 1.0m significant wave height. Additional surface elevation due to wave height is 0.5 x significant wave height. Contribution due to SLR projections may be added as required

Scenario	Storm Tide (m AHD)	Storm tide + 0.5 x Hs (Hs = 0.5m)	Freeboard	Storm Tide + 0.5 x Hs (Hs = 1.0m)	Freeboard
MHWS	+0.68	+0.93m AHD	+0.77m	1.18m AHD	+0.52m
HAT	+1.08	+1.33 AHD	+0.37m	1.58m AHD	+0.12m
1-year ARI	+1.24	+1.49m AHD	+0.21m	1.74m AHD	-0.04m
20-years ARI	+1.38	+1.63m AHD	+0.07m	1.88m AHD	-0.18m
100-years ARI	+1.53	+1.78m AHD	-0.08m	2.03m AHD	-0.33m

Vertical Wave Impact Forces

Vertical forces on the boat-shed floor may be decomposed to *hydrostatic* due to buoyancy effects and *kinetic* due to the wave crest impacting upon the boat shed floor. Wave impact forces are calculated assuming the maximum wave height that can occur during a storm event of a given significant wave height. The significant wave height is a statistical parameter that can be considered to be the mean value of the highest third of waves observed in a time–series of waves. In this instance, the maximum allowed wave height is 1.4m due to wave breaking in the near-shore area, as well as limiting wave steepness.

Wave impact forces are typically calculated on a statistical basis assuming the four highest wave events that can occur during a time-series of 1000 waves. Assuming a peak storm-duration of say two hours and a wave period of 3.6 seconds leads to 8 wave events per storm that will equal or exceed the forces calculated below. In this instance, however, the maximum allowable wave height is limited by depth-induced wave breaking and other considerations. Therefore the 'maximum' wave height would be expected to occur much more frequently, approximately once every 50 waves. This is because the maximum wave height is more closely constrained to the 'significant' wave height value than expected from a purely Gaussian distribution.

Table 3 gives the estimated vertical wave impact forces. It should be noted that these estimates represent an upper-range, conservative estimate, and should be reduced pro-rata if considering less significant wave activity (say 0.5m). **Appendix B** gives the equations and method used in the calculations.

Table 3: Vertical wave forces

Scenario	Storm tide + $0.5H_{max}$	Freeboard	Hydrostatic vertical wave force	Dynamic factor (upper bound)	Vertical wave impact force
MHWS	1.37m AHD	+0.33m	N/A	N/A	N/A
HAT	1.77m AHD	-0.07m	0.7 kN/m^2	6.25	4.4 kN/m^2
1-years ARI	1.93m AHD	-0.23m	2.1 kN/m^2	3.02	7.0 kN/m^2
20-years ARI	2.07m AHD	-0.37m	3.7 kN/m^2	2.26	8.4 kN/m^2
100-years ARI	2.22m AHD	-0.52m	5.2 kN/m^2	1.83	9.6 kN/m^2

Wave periods in these conditions are about 2.5 seconds and hence the near-shore wave length is about 10m. The wave 'crest', or that part of the wave causing these oscillatory uplift loads then might extend over 2.5m in the wave propagation direction (along the jetty and main boat-shed axis), and across the full width (about 4m) of the boat-shed. It would be a moving load and is applicable in any boat-shed floor area. However, at the shoreline, where the progress of the wave is prevented by the floor and shoreline, these wave loads will be bigger – generally unquantifiable, but a realistic load for that part of the floor is 1.5 x the 'Vertical wave impact force' of Table 3. These design waves can generally only propagate from the north-east to north-west sector.

Horizontal Wave Impact Forces

Structural elements such as the boat shed walls, stairs and piles can be subject to high loads due to waves breaking or slamming upon the structure as they become submerged. The force is defined as:

$$Fs = 0.5\ Cs.\rho.A.u^2 \qquad (1)$$

where A is the area of the vertical surface subject to the wave crest, u is the peak horizontal fluid velocity in the wave crest, and Cs is a 'slamming coefficient' in the range 2 – 20 (Tickell, 1994).

The orbital velocity at the wave crest was calculated to be 2.6m/s using a high-order numerical wave theory solution (Fenton, 1999). This corresponds to a slamming force of between 7 and 70 kN/m^2 of structure surface area exposed to oncoming waves. Considering the relatively sheltered aspect of the site, and the overall shape of the sections presented to the on-coming waves, it is more likely that the impact forces will be at the low end of the range considered. Hence a uniform load of 7kN/m^2 applied up to 2.2m AHD along the northern boat-shed wall is to be included.

For design purposes, vertical and horizontal loads should be applied simultaneously. Because these design waves can propagate from north-east to north-west, the estimated horizontal loads must be applied to the eastern and western walls in addition to the northern wall.

Rock Size for Rubble Retaining Wall

Rock size calculations have been undertaken using Van der Meer (1998), assuming a rock slope of 1V in 1.5H, an incoming significant wave height of 1.0m (mean period of 2.6s) and a design water depth at the toe of 1.6m during the 100-years ARI storm surge. **Table 4** gives the estimated range of rock sizes required for the boulder retaining wall to withstand the 100-years ARI wave and storm water level. A suitable geofabric filter will be required to prevent the soil layer from being eroded from beneath the boulders. If the soil at the revetment is not sand, then soil should be excavated and be replaced by a 1m thick/wide layer of coarse sand. Rocks should have an SG of 2.6 or more and have no cracks

Table 4: Rock sizing required to withstand projected 100-years ARI combined water level and significant wave 5eight

Description	Grade	Diameter (m)	Mass (kg)
Primary Armour	50% Passing	0.66	750
	15% Passing	0.54	410
	85% Passing	0.81	1380

Additional safety considerations

The conditions presented above would be expected to last for less than 6 hours per return period considered because water level is dominated by the astronomical tide and wind direction changes over such a duration. Although this represents a relatively small amount of time in comparison to the overall length of the year, we note the following safety considerations should be taken in to account:

- The location, aspect and exposure of the boat shed to oncoming storm waves makes it unsuitable for habitation purposes
- Power supplies (interior) should be located *at least* 1 metre above the floor level of the boat shed. Exterior fittings should be at 1.5m above the floor level to avoid contact with splashing waves.
- The potential for component fatigue (wear and tear) should be recognised for the less severe, but more frequent, wave impact loadings.

Yours faithfully

P. D. Treloar

P.D. Treloar – Senior Principal
for **Cardno (NSW/ACT) Pty Ltd**

References

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Tickell, R.G. (1994): Wave forces on structures. *In*: Abbott & Price (Ed). *Coastal, Estuarial and Harbour Engineers reference book*. Chapter 28, pp 369 – 380.

Appendix A: Site Photographs

Figure A1:Looking south towards the property from the end of the boat pier (east aspect)

Figure A2: Looking south towards the property from the end of the boat pier (west aspect)

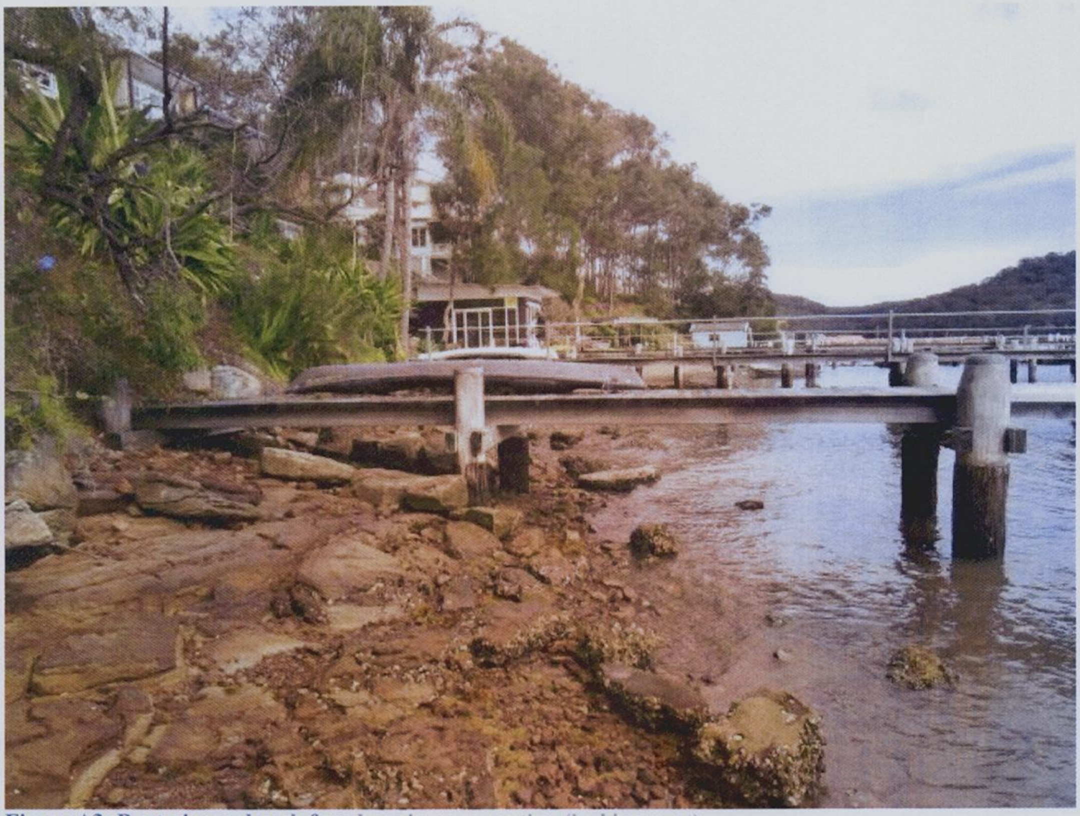

Figure A3: Boat pier and rock foreshore in cross-section (looking west).

Figure A4: Location of planned boat shed (to right of centre pier).

Appendix B: Calculation of vertical wave forces

Following the method of McConell et al (2004):

The 'basic wave force' , F_v^*, is calculated for a wave reaching the predicted maximum crest elevation, η_{max} whilst assuming no (water) pressure on the reverse side of the structural element. F_v^* is defined by a simple pressure distribution using hydrostatic pressures p_1 and p_2 at the top and bottom (respectively) of the particular element being considered:

$$p_1 = [\eta_{max} - (b_s + c_1)].\rho g \tag{B1}$$

$$p_2 = [\eta_{max} - c_1].\rho g \tag{B2}$$

Where $[\eta_{max} - c_1]$ represents the clearance height of the maximum crest elevation above the lower surface of the structural element; b_s is the thickness of the deck flooring ρ is water density (assumed 1025kg/m^3) and g is the acceleration due to gravity (9.81m/s).

Integrating over the underside area of the deck allows approximation of the basic vertical wave force as

$$F_v^* = (length) \times (width) \times p_2 \tag{B3}$$

The dynamic component of the vertical force from the wave impacting on the structure is given by:

$$\frac{F_{vd(+or-)}}{F_v^*} = \frac{a}{\left[\frac{(\eta_{max} - c_1)}{H_s}\right]} \times C \tag{B4}$$

Where H_s is the significant wave height (defined in this study as 1.0m); a and b are empirical coefficients defined in this instance as 0.82 and 0.61, respectively; and C is a factor to calculate the lower (C=0.5) and upper (C=1.5) limits of the data. In this instance, only the upper limit of the data is applied.

The dynamic component represents the kinetic impact force of the wave hitting the structure. Typically very high forces are generated that last only for a short period of time (order of 0.1s). Therefore repetitive striking can act as a 'hammer', over time loosening joists, bolts, nails and other joined structure elements.