Our Ref: AWE200146/L001:PDT
Contact: P.D Treloar

3 February 2020

On behalf of Scott and Carrie Towers
121 Florence Terrace
Scotland Island NSW 2105

Attention: Steve Crosby

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Dear Steve,

COASTAL ENGINEERING REPORT: 121 FLORENCE TERRACE, SCOTLAND ISLAND

Preamble

This technical letter presents a summary of our analyses undertaken in support of your submission for Design Approval to Northern Beaches (formerly Pittwater), Council in relation to your proposed boatshed and jetty construction at 121 Florence Terrace, Scotland Island (see **Figure 1-1**). This report presents summary findings of the following analyses:-

> Moderate to severe ARI storm tide levels, wave heights and run-up levels;

> Vertical wave impact forces for combined waves and water levels;

> Horizontal wave impact forces;

> Additional safety considerations.

Proposed Development

Details of the proposed development are shown on the drawings prepared by Stephen Crosby & Assoc. Pty. Ltd, presented in **Appendix A**. The drawings are labelled 2128 - DA 01, 2128 - DA 02 and 2128 - DA 03, dated June and September, 2018. A detailed survey of the existing development site, dated May 2018 is also included in **Appendix A**.

The proposed development is multi-faceted, and will consist of:-

> The knock-down and rebuild of the existing boat shed. The new boat shed will feature the same footprint area as the existing structure and will be situated in the same location, approximately 1.8 m landward of the seawall on an existing concrete slab. The proposed boatshed is 6 m (wide) x 4 m (long) and has a proposed finished floor level (FFL) of +1.85 m AHD. The boatshed is situated entirely landward of the seawall.

> The remediation of the collapsed seawall at the northern extent of the boatshed (in the location of a proposed slip-rail and 'putt-putt' storage, including backfilling with suitable material and capping with a concrete slab, interfacing with the adjacent property's seawall.

> The construction of a timber deck to an FFL of +1.80 m AHD (0.05 m below the FFL of the boat shed), atop of the concrete slab landward of the seawall. The proposed deck is approximately 16.6 m (long) x 6.8 m (wide) and extends ~1.3 m south of the lot boundary. It is noted, however, that the existing seawall and concrete slab presently extend slightly south of the property boundary.

> The knock-down and rebuilding of the existing timber jetty. The proposed jetty will feature a similar footprint area as the existing structure, however, the deck level will be raised from ~ +1.5 m AHD to +1.65 m AHD and the alignment of the jetty rotated slightly clockwise to meet perpendicular to the seawall. The jetty will be 1.5 m wide and 20.7 m long, extending over a total of 6 evenly spaced pile bents. At the seawall interface, the landward-most span remains at the deck level of +1.8 m AHD, sloping

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down to the jetty level of +1.65 m AHD, over the second landward-most span. All existing piles will be removed and replaced with new timber piles, concreted into bedrock.

> The installation of a floating pontoon and gangway at the seaward extent of the jetty. The pontoon will be 5 m (long) x 3 m (wide) and will be laterally restrained by 3 piles (top level of +2.67 m AHD) and roller bearings (2 landward, and 1 seaward). Two additional mooring piles will be installed on either side of the pontoon. Based on surveyed elevations, the seabed depth within the mooring area is approximately -2.5 m AHD.

> The construction of a timber skid-ramp in front of the boat shed. The skid ramp is 6 m (long) x 4.3 m (wide) and will extend down to the seabed at about mean sea level (i.e. 0 m AHD). The slope of the ramp is approximately 1V:3.33H and will be supported by the timber deck on the landward end and two timber piles, concreted into bedrock at the seaward end.

> The construction of two sliprails for launching of a timber "putt-putt" boat at the northern most end of the development site. Surveyed drawings indicate the presence of existing slip-rails, however, upon inspection of the site, no such slip rails were observed, suggesting that they may be been removed due to deterioration. Similarly, to the aforementioned marine structures, the slip rails will be also be supported by piles, concreted into bedrock and will extend to an elevation of about -0.93 m AHD (equivalent to LAT). The resulting structure is approximately 20 m long from the edge of the seawall.

Figure 1-1 Locality Plan

Site Visit

Coastal engineer Mr Toby Johnson was escorted to the property by Mr Steve Crosby on Wednesday 8th January 2020, arriving at the property around 10:15 am. Weather conditions were hazy, with a moderate south south-westerly breeze. The predicted tidal level at Fort Denison at the time of the visit was approximately +1.2 m LAT (~ +0.3 m AHD), at the ebb of high tide. The property is situated on the south-eastern foreshore of Scotland Island (see **Figure 1-1**). It has a south-easterly aspect, and is accessed by boat. **Figures B.1** to **B.5** describe the site.

The position and orientation of the property means that it is in a low energy wave environment, with the property protected from westerly and northerly waves by its physiographic features. The foreshore faces two relatively small fetches from the east and south (~1 km), with the largest fetch occurring along the south-eastern sector (~ 2.1 km). Hence, the only significant wave activity at the site will be generated by strong, persistent southerly to easterly winds blowing over the Pittwater estuary. The foreshore immediately seaward of the property is relatively rocky, with surveyed seabed levels at the toe of the existing stone seawall ranging between approximately +0.3 m and +0.6 m AHD (**Figure B.4).** The slope of the seabed between the seawall and the end of the jetty is relatively gradual at about 1V:12H. Beyond the seaward end of the jetty, the seabed rapidly steepens into the navigation channel.

The crest of the existing vertical seawall (**Figure B.4**) has a relatively low elevation, with the level of the capping stones ranging between approximately +0.85 m AHD and +1.17 m AHD. The top level of the wall, however, is marginally higher due to the presence of a concrete slab (see **Figures B.3 to B.5**), which adds about 100 to 300 mm of additional elevation. It should be noted, however, that even with consideration of the concrete slab, the crest level of the seawall remains below the 1 in 100-years ARI design water level of +1.52 m AHD, meaning that some level of inundation beneath the proposed timber decking and green-water overtopping of the deck itself should be expected during such extreme conditions. The toe level of the seawall is unknown, however, observations of shallow bedrock at the front of the wall indicate that it is likely founded on suitable material.

Inspection of the seawall also indicated that it is in varying stages of disrepair with observations of core material loss and slumping in some sections, particularly at the northern extent, near to the proposed slipway, where complete failure has occurred (see **Figure B.5**). It is likely that this failure has resulted from the loss of fines from within the core (behind the seawall blocks), due to lack of sufficient filtering, but may also be attributed to wave overtopping or out-flanking which has enabled the removal of material from behind the crest of the wall.

To ensure the longevity of the seawall over the design life of the proposed new boatshed, it is recommended that some remediation work is undertaken to reinstate the loss of material from behind the wall and that an appropriately engineered filter layer (in the form of a geotextile membrane) is installed. Furthermore, to prevent, outflanking of the wall, the seawall should be butted directly to the seawalls of the adjacent properties, providing a seamless coastal defence. It is also recommended that the crest level of the wall (including the width of the concrete slabbing) is raised above the 1 in 100-years ARI design water level of +1.52 m AHD, to reduce wave overtopping, noting that some level of wave overtopping would still occur during the 100-years ARI condition due to the contribution of the wave crest.

It was also noted that at the location of the boat shed, the concrete slab is no longer present (see **Figure B.4**), and appears to by supported by joists spanning between the back of the concrete slab and landward side of the structure. Based on discussions with Steve Crosby, the footprint of the boatshed will be reinstated with suitable backfill material and the concrete slab continued to the location of the proposed retaining wall (landward of the boat shed), providing a suitable foundation for the floor framing of the boat shed. It is, however, recommended that some form of geotechnical analyses is undertaken to ensure that the loads transmitted by the structure to the seawall/retaining wall are sufficiently low to prevent failure of the wall.

Environmental Criteria

Storm Tide and Wave Activity

The level of risk to structures along the foreshore is governed by the severity of waves in combination with the still water level at the time of the peak storm conditions. Higher water levels provide greater ability for waves to impinge upon the foreshore and affect coastal structures. **Figure 1-2** 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 analyses of tide and surge levels at Fort Denison (OEH, 2012). The 100-years ARI significant wave height (H_s) was calculated to be 0.94 m at this site.

It should be noted that all water level scenarios discussed herein are for present day conditions and do not consider projected increases in mean sea level due to climate change. As these structures are uninhabitable and have a relatively low design life (~30 years), the risk to human safety remains low under these projected increases. However, it should be expected that the floor level of the jetty and boatshed may require raising over the design life of the structure to mitigate potential increases in the frequency of inundation and wave overtopping damage that may occur.

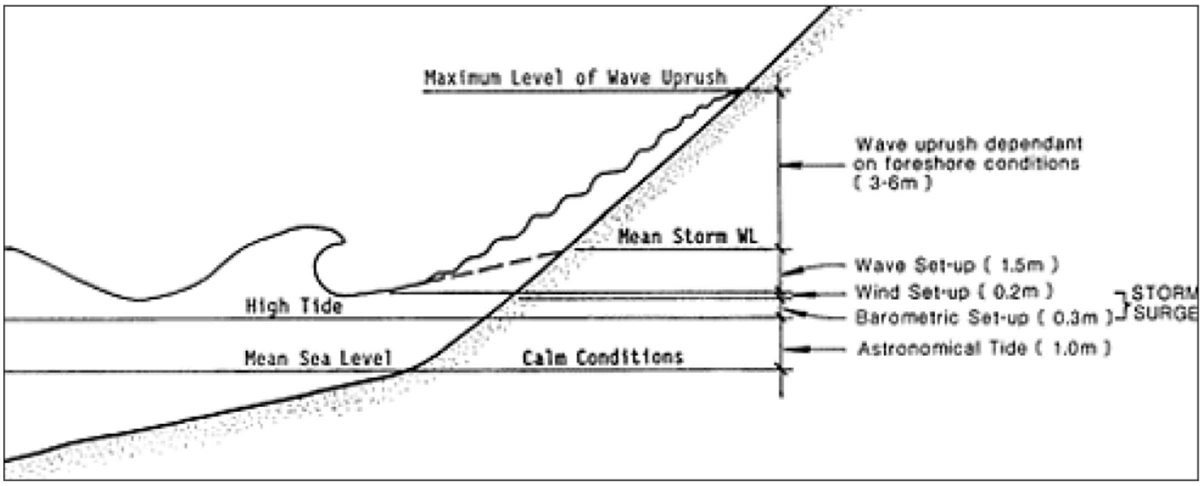

Figure 1-2 Elevated Water Levels during a Storm (NSW Government, 1990)

Design Wave Loading

Introduction

The proposed development is presented in **Appendix A**. The jetty deck and boatshed will be exposed to the following wave forces:-

> Vertical uplift forces on the jetty and skid-ramp from waves propagating underneath the structure;
> Horizontal forces on the boat shed from waves overtopping the seawall;
> Vertical and horizontal wave impact forces on deck units; and
> Horizontal forces on the supporting piers for the jetty, skid-ramp and slip-rails.

The boat shed will be exposed to horizontal forces from waves overtopping the deck. The horizontal forces depend upon deck level, wave parameters and water level. A range of wave uplift force conditions, which depend on the level of the underside of structures, can also occur for structures situated seaward of the seawall and wave uplift forces must be calculated for those cases in order to determine the design loading.

Freeboard

Table 1-1 describes the associated freeboard levels of the jetty and skid-ramp (underside), as shown on architect drawing DA 02 (**Appendix A**), and an assumed timber slat thickness of 75 mm. All potential water levels are considered when calculating the freeboard for these structures, as they are situated entirely over water. Wave loading on the slip-rails is considered to be negligible due to their slender nature (with the exception of the supporting piers). Furthermore, due to the presence of the seawall and concrete slab, uplift loading on the timber deck, landward of the seawall is also considered to be negligible.

The bed level approximately one wavelength (for local sea waves) seaward of the seawall is estimated to be around –0.5 m AHD. At the seaward end of the jetty, the seabed elevation is approximately -2 m AHD. This means that the 100-years ARI significant wave height of 0.94 m, may penetrate the full length of the structure without breaking first.

Theoretically, the highest possible wave directly in front of the structure is the maximum wave height (H_{max}), which can be approximated as 1.7 x H_s = 1.7 m (Goda, 2010) in non-depth limited conditions. As previously discussed, seabed elevations along the length of the jetty vary between approximately -2.0 m AHD and +0.6 m AHD. This means that at the seaward end of the structure, the water depth during the 1 in 100-years ARI condition will be 3.52 m, sufficiently deep for the theoretical maximum wave height of 1.6 m to exist. Approximately one wavelength away from the seawall (about half way along the length of the jetty), the seabed elevation is -0.5 m AHD, meaning the water depth during the 1 in 100-years ARI condition will be 2.02 m. The maximum theoretical wave height that can exist in this water depth is 1.52 m, marginally lower than the theoretical maximum of 1.6 m, indicating that the maximum wave height will have been broken and reduced in height.

Because, the propagating wave will typically need about one wavelength of distance to ‘react’ to changing seabed conditions, depth limited wave conditions have been adopted for the landward half of the jetty (designated at the location of the -0.5 m AHD seabed contour) and skid-ramp. Seaward of the -0.5 m AHD contour, non-depth limited wave conditions have been adopted.

Due to uncertainties surrounding nearshore wave heights for ARI scenarios lower than 100-years, maximum depth-induced breaking wave heights are adopted. It should be noted that there is likely some level of conservatism to the height of these estimated depth limited waves and that they may be lower than the theoretical maximum.

Based on a nearshore seabed slope of 1V:12H, the maximum breaking wave height is given as 0.75 x the water depth (Kamphuis, 2010). The resulting adopted maximum wave heights adopted for each scenario are provided in **Table 1-1**.

Maximum water surface elevation at the jetty will be formed from the storm tide plus half of the height of any wave. The freeboard is the vertical height difference between the underside of the deck and the maximum water surface elevation. **Table 1-1** and **Table 1-2** show the storm tide-levels and freeboards for the various structures landward and seaward of the 0.5 m AHD seabed contour, respectively.

Table 1-1 Freeboards for Various Structures Landward of -0.5 m AHD Seabed Contour: Design H_s = 0.94 m, H_{max} = 1.60 m

Scenario	Storm Tide (m AHD)	Adopted Maximum Wave Height, $H_{max,b}$	Storm Tide + 0.5 x H (m AHD)	Freeboard to Underside of Jetty Ramp (Landward End – Top Level +1.65 m AHD)	Freeboard to Underside of Jetty Ramp (Seaward End - Top Level +1.8 m AHD)	Freeboard to Underside of Skid-Ramp (Landward End – Top Level +1.8 m AHD)	Freeboard to Underside of Skid-Ramp (Seaward End - Top Level 0 m AHD)
MHWS	+0.68	0.68 m	+1.02	0.56 m	0.71 m	0.71 m	-1.02 m
HAT	+1.08	1.08 m	+1.62	-0.05 m	0.11 m	0.11 m	-1.62 m
1-year ARI	+1.24	1.24 m	+1.86	-0.29 m	-0.14 m	-0.14 m	-1.86 m
20-years ARI	+1.38	1.38 m	+2.07	-0.50 m	-0.35 m	-0.35 m	-2.07 m
100-years ARI	+1.52	1.52 m	+2.28	-0.71 m	-0.56 m	-0.56 m	-2.28 m

Table 1-2 Freeboards for Various Structures Seaward of -0.5 m AHD Seabed Contour: Design H_s = 0.94 m, H_{max} = 1.52 m

Scenario	Storm Tide (m AHD)	Adopted Maximum Wave Height, $H_{max,b}$	Storm Tide + 0.5 x H (m AHD)	Freeboard to Underside of Jetty Ramp (Landward End – Top Level +1.65 m AHD)	Freeboard to Underside of Jetty Ramp (Seaward End – Top Level +1.8 m AHD)
MHWS	+0.68	1.60 m	+1.48	0.10 m	0.25 m
HAT	+1.08	1.60 m	+1.88	-0.30 m	-0.15 m
1-year ARI	+1.24	1.60 m	+2.04	-0.46 m	-0.31 m
20-years ARI	+1.38	1.60 m	+2.18	-0.60 m	-0.45 m
100-years ARI	+1.52	1.60 m	+2.32	-0.74 m	-0.59 m

Note that there will be some reflection of incident waves from the vertical rock wall, and beneath the jetty deck near the shoreline, this process will increase wave uplift forces.

Vertical Wave Impact Forces on Deck

Vertical forces on the boatshed floor due to *overtopping* waves are negligible due to the presence of the seawall and concrete slab. It is, however, recommended that the crest level of the seawall/slab is increased to a minimum elevation of +1.52 m AHD to help mitigate wave overtopping volumes that may surge underneath the timber decking and flooring of the boatshed.

Vertical forces on the deck in front of the boatshed may be decomposed into *hydrostatic,* due to buoyancy effects, and *kinetic,* due to wave crest impact upon the deck. Total wave impact forces are calculated assuming the maximum wave height that can occur during a storm event of a given significant wave height. The 1 in 100-years ARI significant wave height (calculated at this site as 0.94 m) 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 probable wave height is given by the theoretical limit of 1.7 x H_s = 1.6 m, valid only for non-breaking wave conditions (i.e. seaward of the -0.5 to -0.6 m AHD seabed contour).

Table 1-3 gives the estimated unfactored vertical wave impact forces for the jetty deck and skid-ramp, landward of the -0.5 m AHD seabed contour. 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 0.5 m). **Appendix C** gives the equations and methods used in the calculations. **Table 1-4** presents similar data for jetty deck, seaward of the -0.5 m AHD seabed contour

Table 1-3 Unfactored Vertical Uplift Wave Pressures on Underside of Jetty Deck and Skid-Ramp Landward of -0.5 m AHD Seabed Contour

Scenario	Landward End of Jetty Underside (Top of Deck Level +1.65 m AHD)		Seaward of Jetty Ramp & Skid-Ramp Underside (Top of Deck/Ramp Level +1.8 m AHD)		Seaward End of Skid-Ramp Underside (Top of Ramp Level +0 m AHD)	
	Hydrostatic vertical wave pressure	*Vertical wave Impact pressure*	*Hydrostatic vertical wave pressure*	*Vertical wave impact pressure*	*Hydrostatic vertical wave pressure*	*Vertical wave impact pressure*
MHWS	N/A	**N/A**	N/A	**N/A**	11.3 kN/m^2	**12.5 kN/m^2**
HAT	1.0 kN/m^2	**4.9 kN/m^2**	N/A	**N/A**	16.8 kN/m^2	**14.6 kN/m^2**
1-years ARI	3.2 kN/m^2	**7.6 kN/m^2**	1.7 kN/m^2	**6.0 kN/m^2**	19.1 kN/m^2	**15.3 kN/m^2**
20-years ARI	5.1 kN/m^2	**9.2 kN/m^2**	3.6 kN/m^2	**8.0 kN/m^2**	21.0 kN/m^2	**15.9 kN/m^2**
100-years ARI	7.1 kN/m^2	**10.4 kN/m^2**	5.6 kN/m^2	**9.5 kN/m^2**	22.9 kN/m^2	**16.4 kN/m^2**

Table 1-4 Unfactored Vertical Uplift Wave Pressures on Underside of Jetty Deck Seaward of the -0.5 m AHD Seabed Contour

Scenario	Seaward End of Jetty Ramp Underside (Top of Deck Level +1.8 m AHD)	
	Hydrostatic vertical wave pressure	*Vertical wave impact pressure*
MHWS	N/A	**N/A**
HAT	1.6 kN/m^2	**5.8 kN/m^2**
1-years ARI	3.2 kN/m^2	**7.6 kN/m^2**
20-years ARI	4.6 kN/m^2	**8.8 kN/m^2**
100-years ARI	6.0 kN/m^2	**9.7 kN/m^2**

Wave periods in these conditions are about 3.3 seconds and hence the near-shore wave length is about 15 m. The wave 'crest', or that part of the wave causing these oscillatory uplift loads then might extend over 3 m in the wave propagation direction (along main jetty axis), and across the full width of the jetty. It would be a moving load and is applicable in any area of the jetty ramp. The unfactored uplift wave loads to be applied are those in **bold**.

However, at the shoreline, where the progress of the wave is prevented by the jetty deck and the seawall, these wave loads will be bigger – generally unquantifiable, but a realistic load for that part of the deck is 1.5 x the 'Vertical wave impact pressure' of **Table 1-3** and **Table 1-4**. These loadings may be reduced by using highly permeable decking to dissipate uplift pressures.

Horizontal Wave Impact Forces

Structural elements such as the boat shed walls 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:

$F_{slam} = 0.5.Cs.\rho.A.u^2$ *[kPa]* (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 3.0 m/s using a high-order numerical wave theory solution (Kamphuis, 2010). This corresponds to a slamming force of between 9 and 92 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 9 kN/m^2 should be adopted.

This load should be applied to the supporting piles for the deck, half way up from the seabed to 2.3 m AHD (design water level + wave crest), and to the seaward ends of the jetty deck at their respective seaward cross-member/head stock beam. This includes the piles supporting the jetty, skid-ramp and slip rails.

Horizontal Wave Overtopping Forces on Seaward Boat Shed Walls & Doors

Waves overtopping the deck may cause a horizontal load on the seaward facing boat shed wall. This uniformly distributed load may be approximated by the relationship determined by Camfield (1991), based on the work of Cross (1967), as:

$F_{surge} = 4.5.\rho.g.Hw^2$ [*kN/m*] (2)

where H_w is the overtopping depth at impact.

The unfactored forces for the wave overtopping events are presented in **Table 1-5**, along with the appropriate levels for their application. The FFL of the boat shed is +1.85 m AHD; however, it has been assumed that the exterior area of the building exposed to wave loading, including the flooring joists/door frame etc. may extend down to +1.8 m AHD (i.e. the surface level of the decking).

Still water inundation of the boatshed floor for the 1 in 100-years event is not expected to occur under present-day mean sea level because the FFL is 0.33 m above the design water level. However, on a wave-by-wave basis, overtopping waves may cause temporary flooding, for ARI's greater than the 1 in 1-year ARI case, whereby the design water level is extremely close to that of the seawall crest level. By raising the seawall level to the recommended +1.52 m AHD, still water inundation will be mitigated for ARIs up to the present day 1 in 100-years scenario.

Table 1-5 Unfactored Horizontal Wave Overtopping Force on Boat Shed Walls and Door

Scenario	***Storm tide + 0.5H_{max}***	***Seawall Overtopping Depth***	***Horizontal Surge Force, F_{surge}***	***Force Application Level***
MHWS	1.02 m AHD	-	**N/A**	**N/A**
HAT	1.62 m AHD	-	**N/A**	**N/A**
1-years ARI	1.86 m AHD	0.06 m	**0.2 kN/m**	**1.83 m AHD**
20-years ARI	2.07 m AHD	0.27 m	**3.3 kN/m**	**1.94 m AHD**
100-years ARI	2.28 m AHD	0.48 m	**10.4 kN/m**	**2.04 m AHD**

For design purposes, vertical and horizontal loads should be applied simultaneously.

Design Considerations

An important design consideration is that of the construction of the boat shed doors. Due to the relatively low level of the boat shed floor (+1.85 m AHD), the horizontal wave overtopping forces are relatively large, and it is possible that construction of the doors to resist such forces may not be economical. That is, it may be cheaper to replace the doors after a severe, but rare storm event than to design the doors to withstand it. Building materials should also be suitable to withstand inundation. Drawing DA 03 indicates that the proposed design is that of sliding/stacking doors. Whilst the building material is not specified, it is likely to be either timber, powder coated aluminium, or an alternative corrosion/rust resistant material.

> An additional design consideration is that of drainage of wave overtopping ingress into the boat shed. Generally, the drainage of such ingress can be addressed by one of the following measures:-

> Scuppers at the base of the north, south, and seaward walls of the boat shed;

> Spacing's between the floor slats;

> A slight seaward grade

> No purpose constructed drainage method, but rather overtopping ingress is manually drained from the shed by being swept out after storm events.

Each option has pros and cons, but ultimately the choice is up to the client. Scuppers would allow seawater to drain almost instantly (within seconds to minutes), but would also allow for more overtopping discharge to flow into the boat shed to begin with, particularly for more frequent and less severe events that are unlikely to damage the doors. Spacing between the floor slats also allows for instantaneous drainage; however, it would also allow for spray to intrude into the shed from underneath due to wave action, and is consequently not advised.

Constructing the boat shed floor with a slight seaward grade may be a simple solution, particularly given the expected frequency of access deck overtopping and ingress into the shed. However, this may also impinge upon the intended functionality of the shed, particularly vessel storage, which is likely to be on rails or tracks.

The most practical solution may be to not include any purpose constructed drainage method, and to manually sweep out overtopping discharge water after a severe storm event. The wave overtopping inundation depths listed in **Table 1-5** are temporary (on a wave by wave basis), and after storm events the inundation depths can be expected to drain down to around a few centimetres, even without purpose-constructed drainage. Manual removal would therefore not be expected to be too onerous.

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 a year, we note the following safety considerations should be taken into 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 1.5 m 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 loading cases on structural elements and fixings.

> Structural design needs to consider likely corrosion processes.

Yours sincerely,

D. W. Treloar

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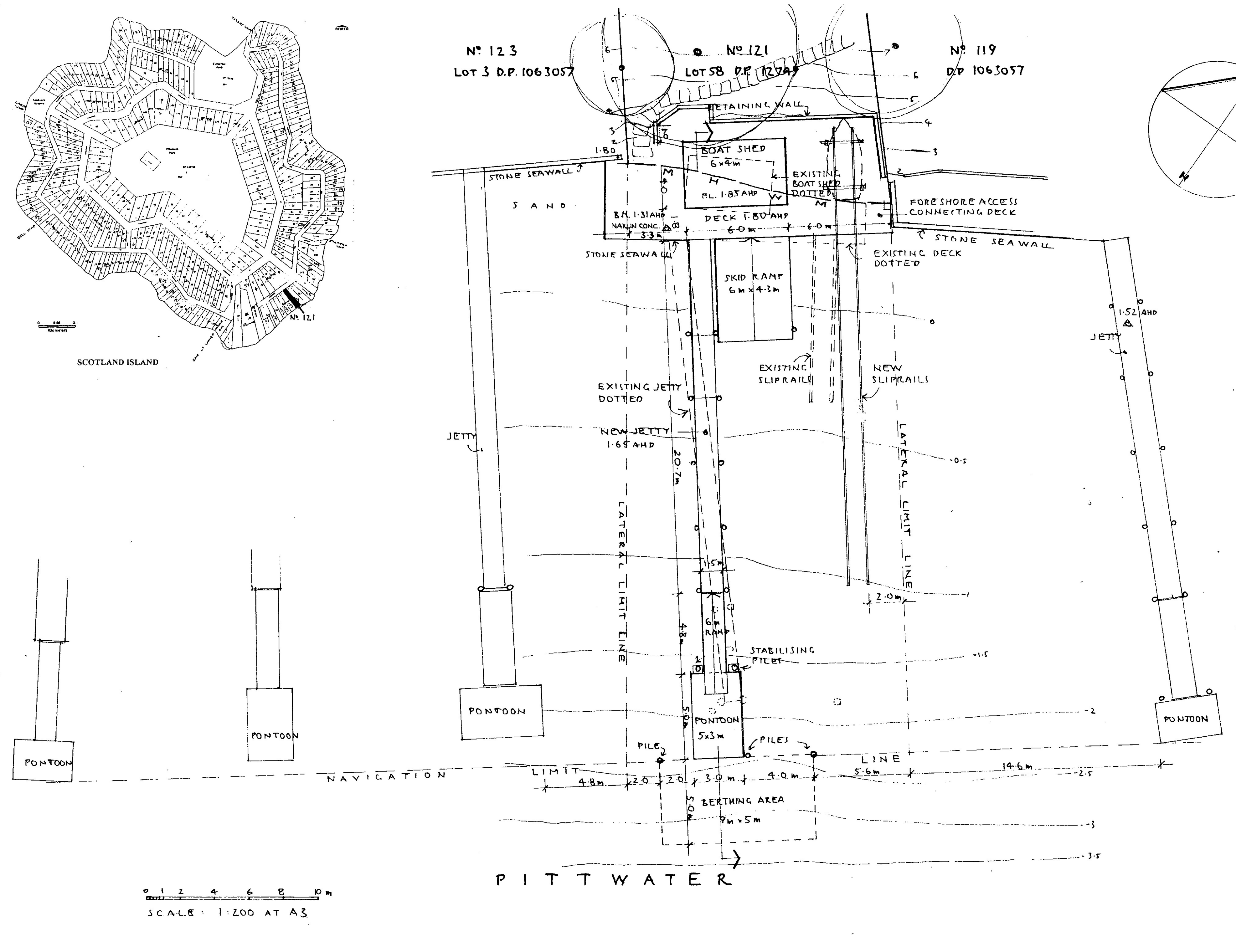
Appendix A
Development Plans and Site Survey



Revision: Date:

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Job

BOAT SHED & JETTY

121 FLORENCE TCE SCOTLAND ISLAND

Lot 58 DP 12749

For S & C TOWERS

Drawing

SITE PLAN

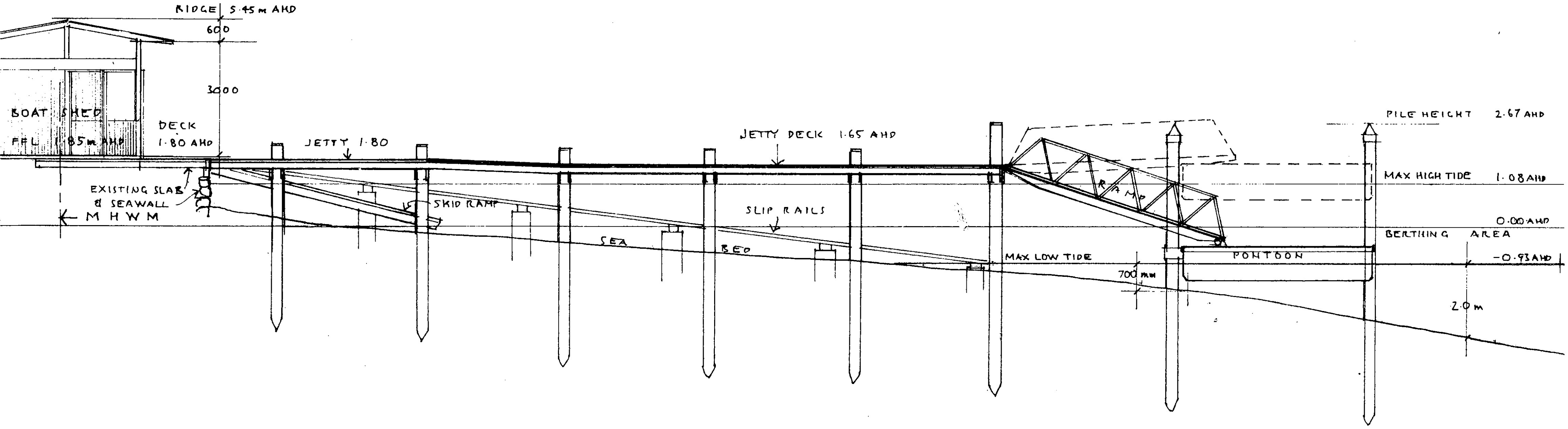
Scale 1 : 200 AT A3

Date JUNE 2018

Drawn S.C.

Drawing Number

2128 - DA 01



Revision: Date:

Stephen Crosby & Assoc.
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SCOTLAND ISLAND
PO Box 204 Church Pt. NSW 2105
M 0409 047 513 E: scrosby@internode.on.net

Job

BOAT SHED
& JETTY

121 FLORENCE TCE
SCOTLAND ISLAND

Lot 58 DP 12749

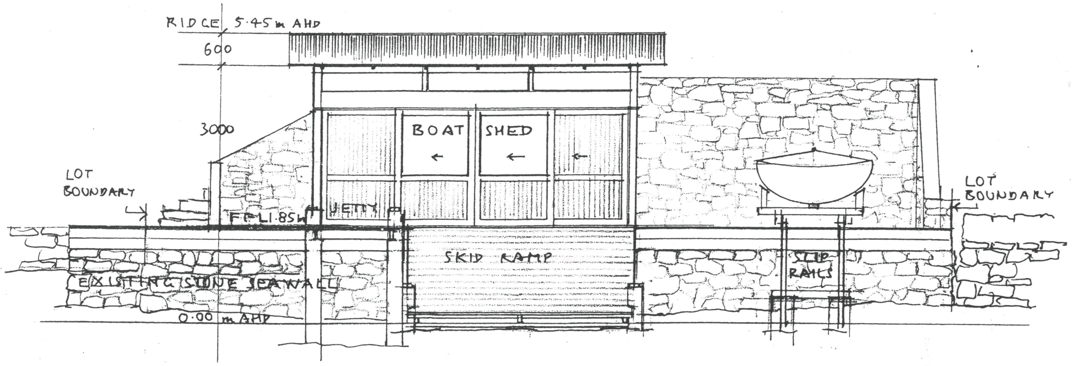
For S & C TOWERS

Drawing

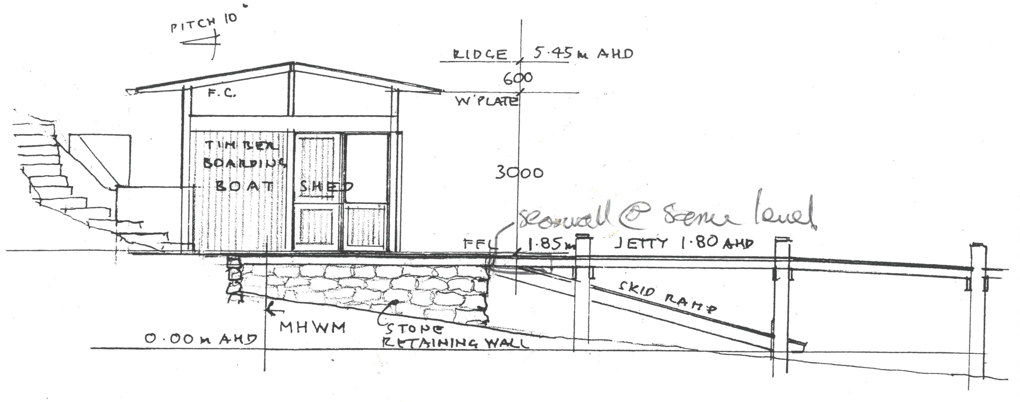
SECTION

Scale 1 : 100 AT A3
Date JUNE 2018
Drawn S.C.
Drawing Number

2128 - DA 02



SOUTH-EAST ELEVATION 1:100



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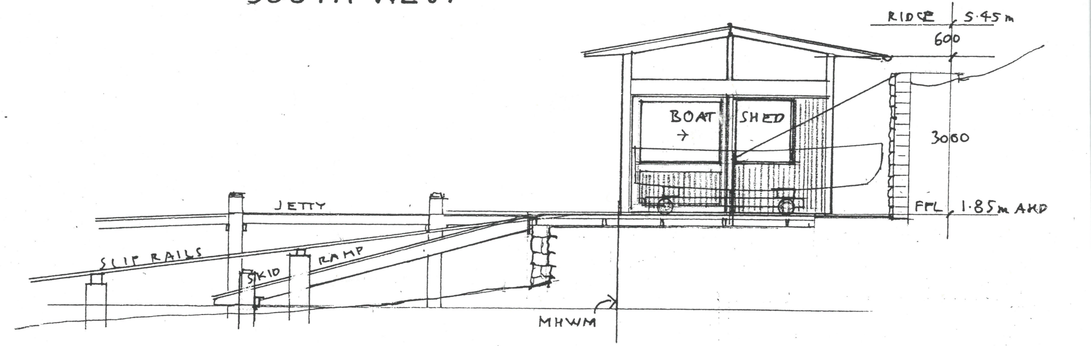

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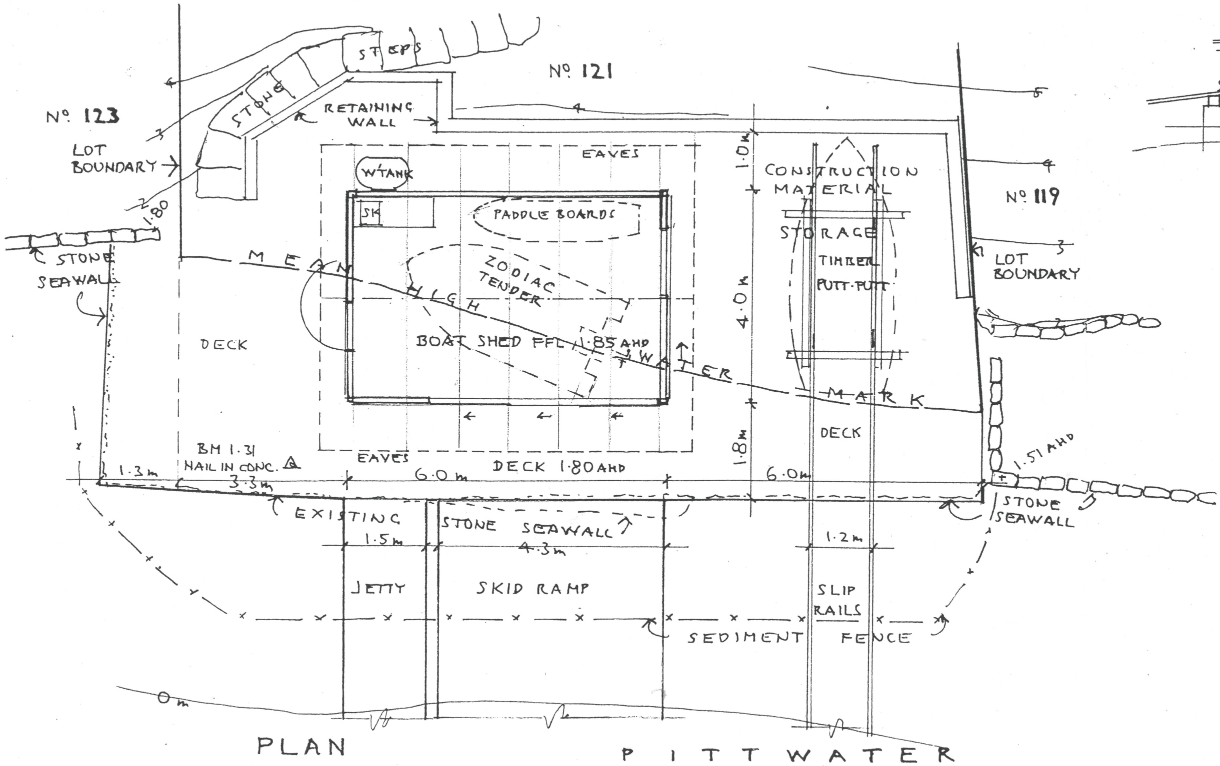
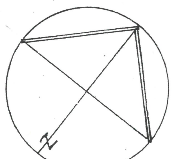


PLAN



Revision: Date:

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Job

BOAT SHED & JETTY

121 FLORENCE TCE
SCOTLAND ISLAND
Lot 158 DP 12749

For S & C TOWERS

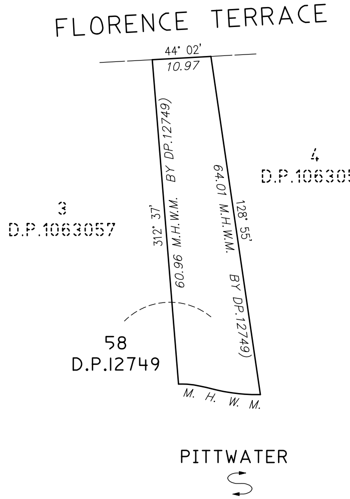
Drawing

BOAT SHED PLAN & ELEVATIONS

Scale	1 : 100 AT A3
Date	SEPTEMBER 2018
Drawn	S.C.
Drawing Number	2128 - DA 03

LEGEND

BB	BOTTOM OF BANK
BW	BOTTOM OF WALL
CON	CONCRETE
DK	DECK
JET	JETTY
NS	NATURAL SURFACE
PIER	PIER
PIL	PILE
POST	POST
RMP	RAMP
RR	ROOF RIDGE
STR	STAIRS
TB	TOP OF BANK
TG	TOP OF GUTTER
TR	TREE-DIA,SPREAD,HEIGHT
TRK	TOP OF ROCK
TRW	TOP OF RETAINING WALL
TW	TOP OF WALL



LOCALITY SKETCH
NOT TO SCALE

· BOUNDARIES HAVE NOT BEEN DEFINED (TITLE DIMENSIONS ONLY) - BOUNDARY DEFINITION IS SUBJECT TO FURTHER SURVEY.

· IF CONSTRUCTION ON OR NEAR BOUNDARIES IS REQUIRED IT IS RECOMMENDED THAT THE BOUNDARIES OF THE LAND BE MARKED.

· TREE SIZES ARE ESTIMATES ONLY.

· THIS PLAN HAS BEEN PREPARED FOR THE EXCLUSIVE USE OF SCOTT & CARRIE TOWERS

· RELATIONSHIP OF IMPROVEMENTS TO BOUNDARIES IS DIAGRAMMATIC ONLY. WHERE OFFSETS ARE CRITICAL THEY SHOULD BE CONFIRMED BY FURTHER SURVEY.

· EXCEPT WHERE SHOWN BY DIMENSION LOCATION OF DETAIL WITH RESPECT TO BOUNDARIES IS INDICATIVE ONLY.

· CRITICAL SPOT LEVELS SHOULD BE CONFIRMED WITH SURVEYOR.

· THIS PLAN IS ONLY TO BE USED FOR THE PURPOSE OF DESIGNING NEW CONSTRUCTIONS.

· CONTOURS SHOWN DEPICT THE TOPOGRAPHY. EXCEPT AT SPOT LEVELS SHOWN, THEY DO NOT REPRESENT THE EXACT LEVEL AT ANY PARTICULAR POINT. ONLY SPOT LEVELS SHOULD BE USED FOR CALCULATIONS OF QUANTITIES WITH CAUTION.

· CONTOUR INTERVAL - 0.5 metre. - SPOT LEVELS SHOULD BE ADOPTED.

· POSITION OF RIDGE LINES ARE DIAGRAMMATIC ONLY (NOT TO SCALE).

· THE INFORMATION IS ONLY TO BE USED AT A SCALE ACCURACY OF 1:100.

· DO NOT SCALE OFF THIS PLAN / FIGURED DIMENSIONS TO BE TAKEN IN PREFERENCE TO SCALED READINGS.

· IF ACCURATE TRUE NORTH IS REQUIRED A FURTHER SURVEY WOULD BE NECESSARY.

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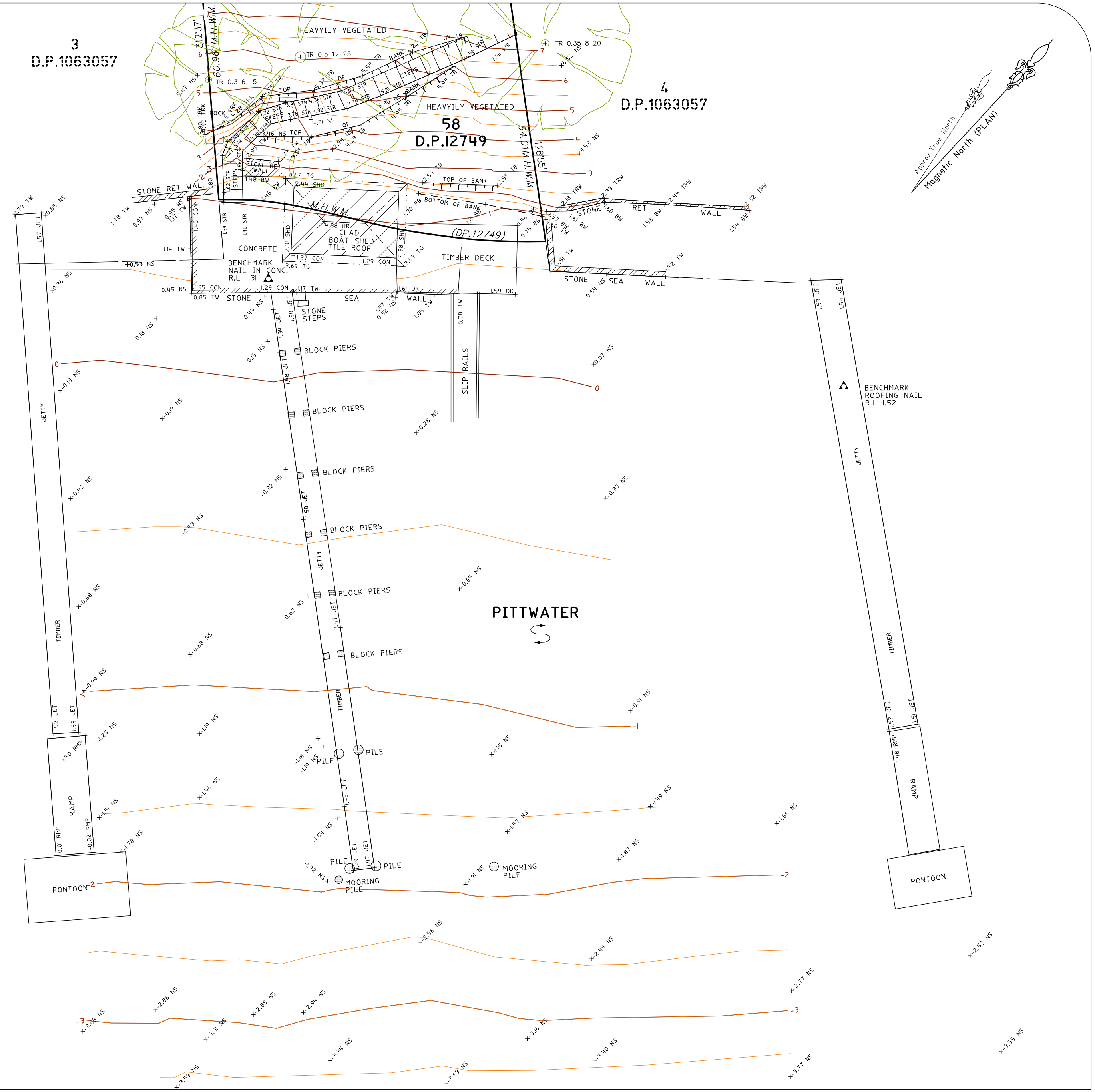

TITLE INDICATES THAT LOT 58 IN D.P.12749 IS SUBJECT TO:

- RESERVATIONS AND CONDITIONS IN THE CROWN GRANT(S).
- D508770 COVENANT (NOT INVESTIGATED)
- 8709148 RIGHT OF FOOTWAY VARIABLE WIDTH APPURTENANT TO THE LAND ABOVE DESCRIBED AFFECTING THE PART SHOWN VARIABLE WIDTH IN PLAN WITH 8709148

Waterview
SURVEYING SERVICES

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0474 843 180

Drawing Title
DETAIL AND LEVELS OVER
PART OF 121 FLORENCE STREET
SCOTLAND ISLAND NSW 2105
BEING LOT 58 IN DP.12749

Client Details
SCOTT & CARRIE TOWERS

Vertical Datum
DATUM: AUSTRALIAN HEIGHT DATUM (AHD)

PROJECT: 814	PAGE 1 OF 1
Date of survey 09/05/2018	Drawing No. 814detail 1
Scale 1:100 @ A1	Rev. A

A	FIRST ISSUE	17/05/18

Appendix B
Site Photographs

Cardno

AWE200146 Jan 2020

Y:\2304\Projects_AWE\FY20\0146_121_Florence_Terrace

121 Florence Terrace, Scotland Island

Existing Jetty and Boatshed
Looking Landward

Appendix B.1

121 Florence Terrace, Scotland Island

Existing Jetty
Looking Seaward

Appendix B.2

AWE200146 Jan 2020

Y:\2304\Projects_AWE\FY20\0146_121_Florence_Terrace

Cardno

AWE200146 Jan 2020

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121 Florence Terrace, Scotland Island

Existing Boatshed
From Southern Side

Appendix B.3

121 Florence Terrace, Scotland Island

Existing Seawall and Slab

Appendix B.4

AWE200146 Jan 2020

Y:\2304\Projects_AWE\FY20\0146_121_Florence_Terrace

AWE200146 Jan 2020

Y:\2304\Projects_AWE\FY200146_121_Florence_Terrace

122 Florence Terrace, Scotland Island

Existing Seawall Failure

Appendix B.5

Appendix C
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 p1 and p2 at the top and bottom (respectively) of the particular element being considered:

$p_1 = [\eta_{max} - (b_s + c_1)].\rho g$ (C1)

$p_2 = [\eta_{max} - c_1].\rho g$ (C2)

Where $[\eta_{max} - c_1]$ represents the clearance height of the maximum crest elevation above the lower surface of the structural element; bs is the thickness of the deck flooring ρ is water density (assumed 1025kg/m3) 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

$F_v^* = (length) \times (width) \times p_2$ (C3)

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

$\frac{F_{vd(+or-)}}{F_v^*} = \frac{a}{\left[\frac{(\eta_{max}-c_1)}{H_s}\right]} \times C$ (C4)

Where Hs 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.