Our Ref: AWE200056/L001:PDT
Contact: P.D Treloar

16 October 2019

On behalf of D & J Suttie
67 Florence Terrace
Scotland Island NSW 2105

Attention: Steve Crosby

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Dear Steve,

COASTAL ENGINEERING REPORT: 67 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 construction at 67 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 force;

> 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 2069 - DA 01 and 2069 - DA 02, dated February, 2019. No detailed survey of the site has been provided, however, spot elevations and contours, limited to the immediate vicinity of the proposed development, have been provided in drawing DA 01.

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

> The construction of a jetty deck and boat shed. The jetty deck is 6.4 m wide x 7.2 m long and will have a finished floor level (FFL) of +1.8 m AHD. Approximately 1/3 of the deck will sit landward of the existing rock seawall, supported at the rear by a concrete strip footing. Seaward of the seawall, the jetty deck will be supported by concreted piles. A boat shed will sit atop the jetty deck, centred laterally, and setback 1.2 m from the seaward perimeter of the deck. The footprint of the boat shed is 6 m long x 4 m wide and 3 m high, with a FFL of +1.85 m AHD (0.05 m higher than the jetty deck). Approximately 3 to 4 m of the boat shed will be situated seaward of the seawall.

> The construction of an access skid-ramp, seaward of the boatshed and access deck that will allow the launch/retrieval of vessels to and from the water. The ramp will be 3.0 m wide by 6.0 m long. It will slope seawards at a grade of approximately 1:3 (V:H), down to a base level of approximately 0 m AHD (or marginally below). It will be non-floating (fixed in place both horizontally and vertically) and will be supported by concrete piles.

Figure 1-1 Locality Plan

Site Visit

Coastal engineers Dr Doug Treloar and Mr Toby Johnson were escorted to the property by Mr Steve Crosby on Thursday 19th September 2019, arriving at the property around 10.00 am. Weather conditions were sunny, with a clear sky and a moderate westerly breeze. The predicted tidal level at Fort Denison at the time of the visit was approximately +1.5 m LAT (~0.6 m AHD). The property is situated on the central eastern foreshore of Scotland Island (see **Figure 1-1**), about 240 m north of Scotland Island, Eastern Wharf. It has an easterly aspect, and is accessed by boat. **Figures B.1** and **B.2** describe the site – the boat shed would be constructed near the near the pink kayaks on **Figure B.1**.

The position and orientation of the property means that it is in a low energy wave environment, with the property protected from westerly waves by geography. The foreshore faces two significantly sized fetches from the south-east (2.4 km) and north-east (4.6 km) and hence the only significant wave activity at the site will be generated by strong, persistent south-easterly or north-easterly winds blowing over the Pittwater estuary. The foreshore immediately seaward of the property is rocky, with shallow bedrock situated at about +0.5 m AHD (**Figure B.3).** The nearshore then transitions to a sandy seabed, featuring an upper seabed slope of ~1V:10H, before steepening to reach deeper navigable waters about 70 m offshore. An existing timber jetty, extending 70 m seaward, **Figures B.1** and **B.2**, and shared with the adjacent property will remain in place. The proposed boat shed will be situated on the northern side of the jetty with stairs connecting the two structures at the landward extent.

The existing, vertical sandstone rock seawall, which has a crest level of around +1.5 to + 1.7 m AHD, will remain in place – **see Figures B.1 and B.3**. The seawall is founded on bedrock and is comprised of irregularly shaped rocks. The wall is not showing any signs of failure (e.g. slumping, crest erosion, etc) and appears to be in sound condition. However, it should be noted, that the presence of an engineered filter layer, required to prevent loss of sediments through the voids of the wall is unknown. However, the boat shed and skid ramp will not rely on the integrity of the seawall and land behind it for support.

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 analysis 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.

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 deck and boat ramp 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.

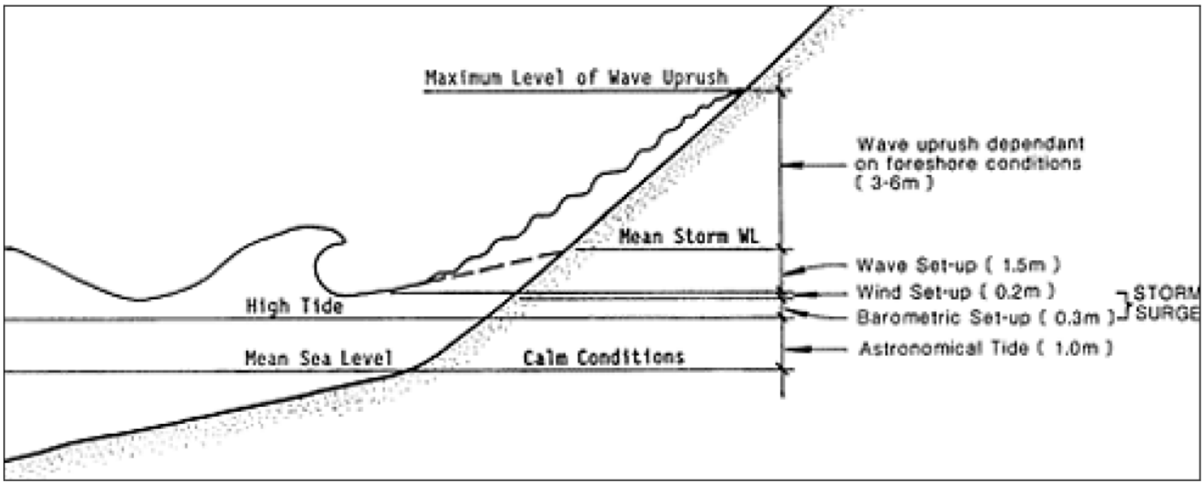

Figure 1-2 Elevated Water Levels during a Storm (NSW Government, 1990)

Design Loading

Introduction

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

- Vertical uplift forces from waves propagating underneath the structures;
- Horizontal forces on the boat shed from waves overtopping the deck;
- Vertical and horizontal wave impact forces on deck units; and
- Horizontal forces on the supporting piers.

The boat shed will be exposed to horizontal forces from waves overtopping the deck. These forces depend upon floor (under-side) level, wave parameters and water level. Hence a range of wave uplift force conditions can occur 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 skid ramp and deck (undersides), 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 of the deck, as this deck is partly over water.

The bed level approximately one wavelength (for local sea waves) seaward of the deck is 0 to -0.5 m AHD. This means that the 100-years ARI significant wave height of 0.94 m, may penetrate to 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 = 1.60 m (Goda, 2010) in non-depth limited conditions. However, given the relatively shallow seabed seaward of the deck and skid ramp, the maximum wave height for all water levels considered herein will be depth limited. Consequently, H_{max} will be more closely constrained to the significant wave height than for non-depth-limited conditions.

Based on a nearshore seabed slope of 1V:10H, the maximum breaking wave height is given as 0.7 x the water depth (Kamphuis, 2010). The resulting maximum breaking wave heights for each scenario are provided in **Table 1-1**.

Maximum water surface elevation at the deck and skid-ramp 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** shows that the deck may be submerged on a wave-by-wave basis at the 1 in 1-years ARI tide level or higher.

Table 1-1 Freeboard of Various Structures

Scenario	Storm Tide (m AHD)	Max Breaking Wave Height, $H_{max,b}$	Storm Tide + 0.5 x H (m AHD)	Freeboard to Underside of Deck (1.72 m AHD)	Freeboard to Underside of Landward End of Skid Ramp (1.72 m AHD)	Freeboard to Underside of Seaward End of Skid Ramp (0.00 m AHD)
MHWS	+0.68	0.83 m	+1.09	+0.63 m	+0.63 m	-1.09 m
HAT	+1.08	1.11 m	+1.63	+0.09 m	+0.09 m	-1.63 m
1-year ARI	+1.24	1.22 m	+1.85	-0.13 m	-0.13 m	-1.85 m
20-years ARI	+1.38	1.32 m	+2.04	-0.32 m	-0.32 m	-2.04 m
100-years ARI	+1.52	1.41 m	+2.23	-0.51 m	-0.51 m	-2.23 m

Note that there will be some reflection of incident waves from the vertical rock wall, and beneath the deck 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. 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 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 estimated breaking wave height rather than the theoretical limit of 1.7 x H_s = 1.56 m, valid only for non-breaking wave conditions.

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 1-2 gives the estimated unfactored 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 0.5 m). **Appendix C** gives the equations and methods used in the calculations.

Table 1-2 Unfactored Vertical Wave Pressures on Underside of Deck (underside level = 1.52 m AHD)

Scenario	***Jetty and Access Deck & Landward End of Skid Ramp Underside level = 1.72 m AHD***		***Seaward End of Skid Ramp Underside level = 0.0 m AHD***	
	Hydrostatic vertical wave pressure	***Vertical wave Impact pressure***	***Hydrostatic vertical wave pressure***	***Vertical wave impact pressure***
MHWS	N/A	**N/A**	11.0 kN/m^2	**12.3 kN/m^2**
HAT	N/A	**N/A**	16.4 kN/m^2	**14.4 kN/m^2**
1-years ARI	1.3 kN/m^2	**5.4 kN/m^2**	18.6 kN/m^2	**15.1 kN/m^2**
20-years ARI	4.6 kN/m^2	**8.8 kN/m^2**	20.5 kN/m^2	**15.7 kN/m^2**
100-years ARI	5.2 kN/m^2	**9.2 kN/m^2**	22.5 kN/m^2	**16.3 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 boatshed and deck axis), and across the full width of the deck/jetty. It would be a moving load and is applicable in any deck/jetty area. The loads to be applied are those in **bold**.

However, at the shoreline, where the progress of the wave is prevented by the floor and the seawall, 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 pressure' of **Table 1-2**.

<u>Horizontal Wave Impact Forces</u>

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 2.9 m/s using a high-order numerical wave theory solution (Kamphuis, 2010). This corresponds to a slamming force of between 8 and 80 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 8 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 2m AHD (design water level + wave crest), and to the seaward ends of the deck and skid ramp, at their respective seaward cross-member/head stock beam.

<u>Horizontal Wave Overtopping Forces on Seaward Boat Shed Walls & Doors</u>

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:

$Fsurge = 4.5.\rho.g.Hw^2$ [*kN/m*] (2)

where Hw is the overtopping depth at impact.

The unfactored forces for the wave overtopping events are presented in **Table 1-3**, along with the appropriate levels for their application.

Table 1-3 Unfactored Horizontal Wave Overtopping Force on Boat Shed Walls and Door

Scenario	Storm tide + $0.5H_{max}$	Deck Overtopping Depth	Horizontal Surge Force, F_{surge}	Force Application Level
MHWS	1.10 m AHD	-	N/A	-
HAT	1.71 m AHD	-	N/A	-
1-years ARI	1.85 m AHD	-	N/A	-
20-years ARI	2.04 m AHD	0.19 m	1.6 kN/m	1.95 m AHD
100-years ARI	2.24 m AHD	0.39 m	6.7 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 low level of the deck (+1.8 m AHD) and 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. It is not expected that wave overtopping will occur on a frequent (annual) basis, however building materials will need to be suitable to withstand inundation on a less frequent basis . Drawing DA 02 indicates that the proposed design is that of sliding 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 in to the 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 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 after a severe storm event. The wave overtopping inundation depths listed in **Table 1-3** 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 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 1.5 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,

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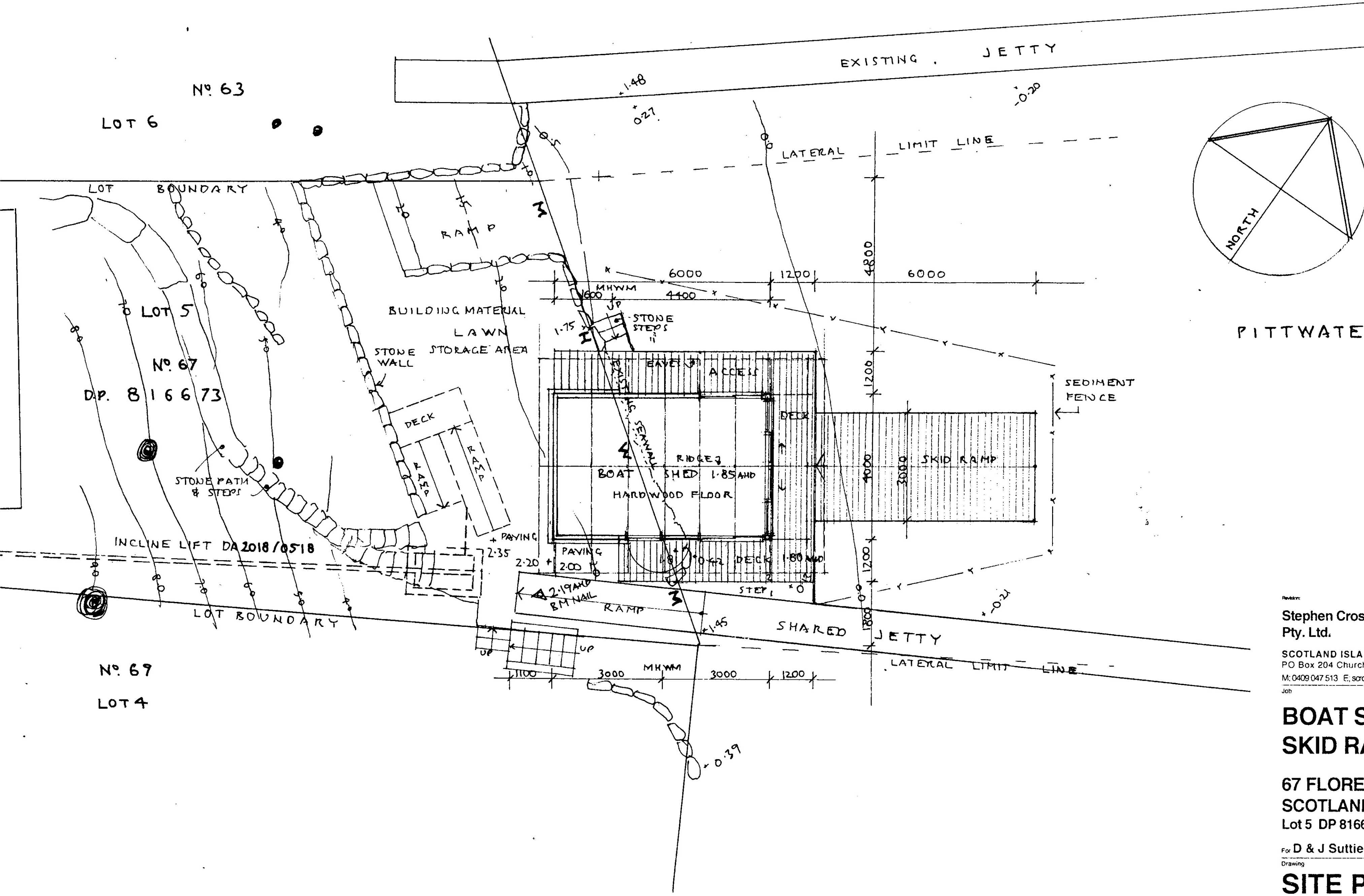
Appendix A
Development Plans and Site Survey



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Job

BOAT SHED & SKID RAMP

67 FLORENCE TCE. SCOTLAND ISLAND
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For D & J Suttie

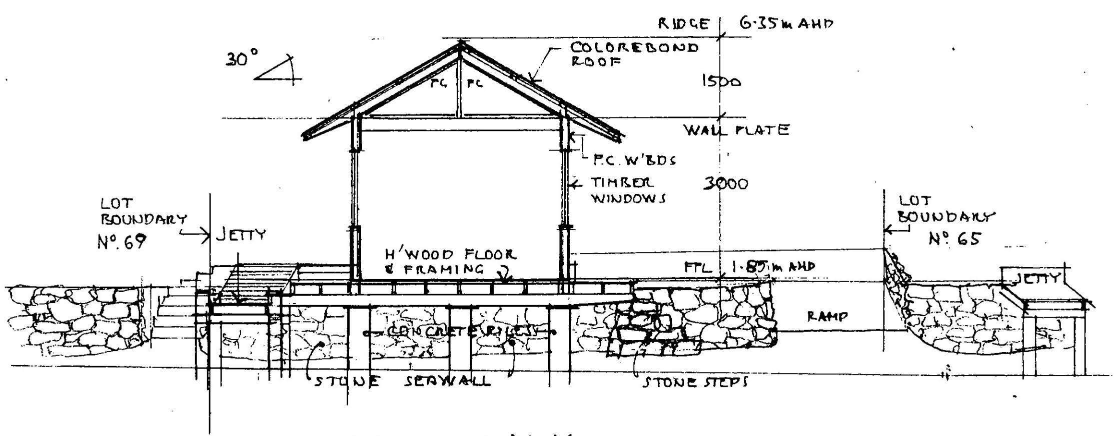
Drawing

SITE PLAN FLOOR PLAN

Scale	1:100 AT A3
Date	FEBRUARY 2019
Drawn	S.C.

Drawing Number

2069 - DA 01



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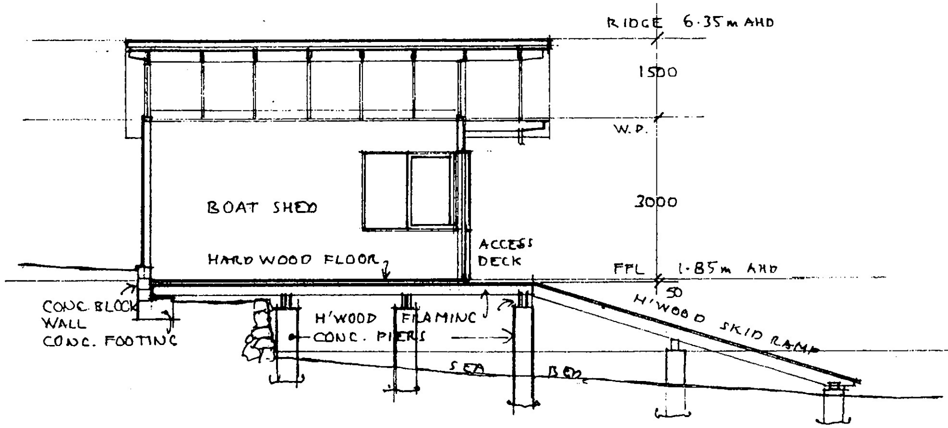

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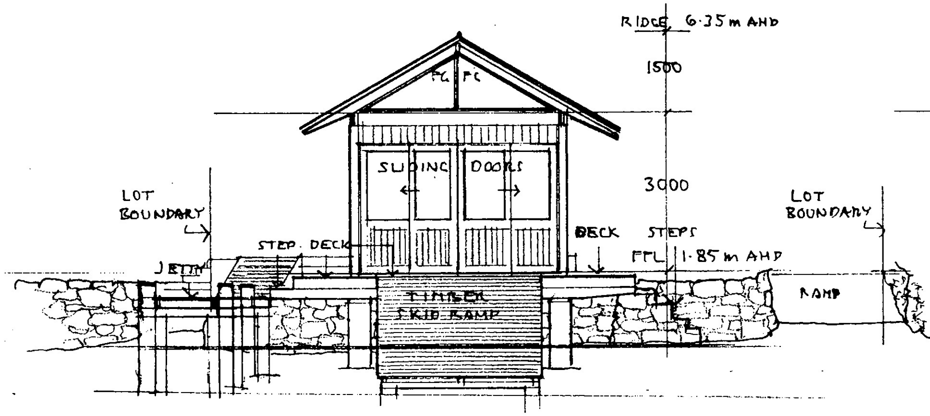

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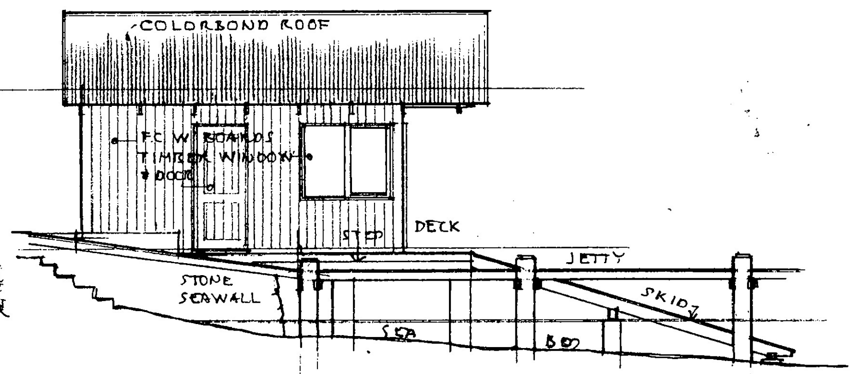

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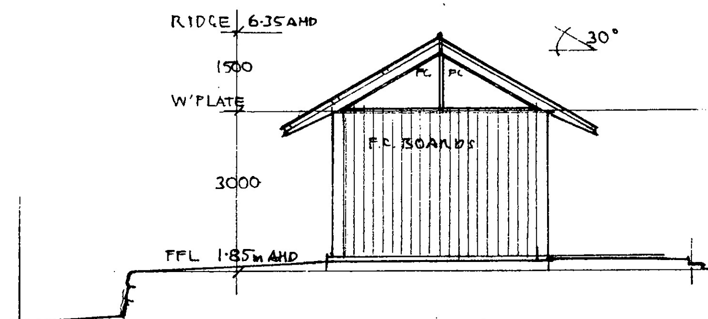

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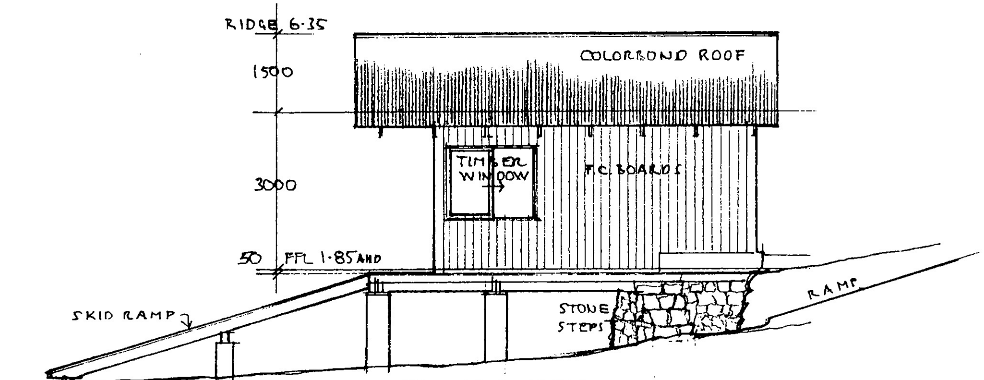

NORTH-WEST

Revision: Date:

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BOAT SHED & SKID RAMP

67 FLORENCE TCE.
SCOTLAND ISLAND
Lot 5 DP 816673

For D & J Suttie

Drawing

SECTIONS & ELEVATIONS

Scale	1:100 AT A3
Date	FEBRUARY 2019
Drawn	S.C.
Drawing Number	

2069 - DA 02

Appendix B
Site Photographs

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67 Florence Terrace − Coastal Engineering Report

Existing Jetty and Seawall
Looking Landward

Appendix B.1

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67 Florence Terrace − Coastal Engineering Report

Existing Jetty
Looking Seaward

Appendix B.2

67 Florence Terrace − Coastal Engineering Report

Existing Seawall
Evidence of Shallow Bedrock

Appendix B.3

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:

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

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

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

$$F_v^* = (length) \times (width) \times p_2 \qquad \text{(C3)}$$

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

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

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.