Horton Coastal Engineering
Coastal & Water Consulting

Coastal Engineering Report for Construction of Upgraded Coastal Protection Works at "Flight Deck", 1114 Pittwater Road Collaroy

Prepared by Horton Coastal Engineering Pty Ltd

for the Owners of Strata Plan No. 1977

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1. INTRODUCTION

The report herein has been prepared as part of a Development Application to Northern Beaches Council for construction of upgraded coastal protection works at 1114 Pittwater Road Collaroy, a 12-level unit block with 36 units, known as "Flight Deck".

In the *Collaroy – Narrabeen Beach Coastal Protection Works Design Specifications* (hereafter denoted as "the *Specifications*") prepared by Northern Beaches Council in 2016, it is stated that:

> "A Basis of Design (BoD) statement shall be prepared as part of the seawall design process and submitted with the Development Application. The BoD shall clearly state all of the design factors, assumptions and qualifications adopted in the design, including specific reference to the above design criteria".

The report herein has been formulated to meet this requirement.

The report author, Peter Horton [BE (Hons 1) MEngSc MIEAust CPEng NER], is a professional Coastal Engineer with 25 years of coastal engineering experience. He has postgraduate qualifications in coastal engineering, and is a Member of Engineers Australia (MIEAust) and Chartered Professional Engineer (CPEng) registered on the National Engineering Register (NER). He is also a member of the National Committee on Coastal and Ocean Engineering (NCCOE) and NSW Coastal, Ocean and Port Engineering Panel (COPEP) of Engineers Australia.

2. GENERAL DESCRIPTION OF PROPOSED DESIGN AND MATERIALS

It is proposed to place an additional layer of basalt armour rock over the existing rock revetment at Flight Deck. As required and as directed by a coastal engineer, any gaps in the existing rock armour would be infilled with armour rock prior to placement of the additional armour. Some existing rock may also need to be removed and repositioned where required, eg where not already well interlocked with other rock. Any loose (non-interlocked) rocks seaward of the existing revetment toe would be repositioned as necessary, moving these rocks landward to be part of the upgraded revetment.

The existing protection works at Flight Deck comprise randomly placed sandstone boulders laid at a relatively flat slope of between about 1:2.4 and 1:3.0 (vertical:horizontal), with an average slope of 1:2.6 and median slope of 1:2.7[1]. For hydraulic stability calculation purposes, the steepest slope of 1:2.4 has been adopted herein and has been assumed to apply over the entire revetment, which is conservative. This is flatter than the steepest allowable hydraulically stable slope of 1:1.5 (vertical:horizontal), which is also the steepest allowable slope in the *Specifications*, and hence exceeds the *Specifications* requirement in this regard.

Observation of the rock boulder sizes exposed after the June 2016 storm indicated an approximate typical dimension of 1m, but significant variability between about 0.3m and 2m in this typical dimension.

The crest level of the existing revetment is at about 4m to 4.5m AHD, with the tiled area seaward of the Flight Deck tower at a level of about 5m AHD.

As depicted on the Drawings, the proposed coastal protection works design comprises placement of a single layer of basalt rock armour with an individual rock median mass of 3.0 tonnes (approximate 1.2m dimension) over the existing revetment. If sandstone was used instead of basalt, the median required mass for a 100-year Average Recurrence Interval (ARI) event would 5.3 tonnes (approximate 1.5m dimension). As the existing revetment is to remain in place, it is not possible to certify that the proposed revetment complies with the *Specifications*, given that:

- the existing sandstone armour rock has a typical mass of about 2.2 tonnes, which is undersized relative to the required 5.3 tonnes sandstone mass; and
- the existing revetment may not have a graded filter design (that is, no secondary armour in 2 layers and then second rock underlayer or geotextile).

However, the overall design is considered to provide an acceptably low risk of damage to Flight Deck from coastal erosion as:

- the existing revetment has a large volume of rock, providing a "thick" and permeable structure;
- the existing revetment almost certainly has no underlying geotextile, which improves revetment permeability performance (CIRIA et al ,2012)

[1] As measured over 13 cross sections using UNSW Water Research Laboratory (WRL) drone survey data captured when the revetment was exposed immediately after the June 2016 storm, on 10 June 2016.

- the placed armour would theoretically be hydraulically stable at the 0-5% damage level for 500-year ARI design conditions[2], and would assist in stabilising the underlying undersized armour (see further discussion on design event below); and
- the Flight Deck tower is on deep piled foundations.

To "compensate" for the retention of the existing revetment with its undersized armour and potential non-graded filter layers, it was decided to "over-design" the additional armour for a 500-year ARI event occurring at the end of the design life (rather than a 100-year ARI event, which is considered to be acceptable for a new revetment). However, compared to the 100-year ARI event, this only increased the median armour size from 2.4 tonnes to 3.0 tonnes, as derived in subsequent sections.

With the area seaward of the Flight Deck tower and extending to the seaward property boundary having been filled after the 1967 storm, there may be in the order of 2,700m^3 of fill (presumably mostly rock) at this location, calculated as a triangular prism with 18m (distance seaward of tower) × 59m (seaward property boundary length) × 5m (approximate height of filling adjacent to tower) volume. This ignores fill that would have been placed under the tower itself.

Assuming all fill is rock, and 37% voids between the rocks, there would be about 1,700m^3 of rock placed at Flight Deck in 1967. This is about 3,700 tonnes of rock assuming a sandstone density of 2,200kg/m^3. For the design herein, it was considered to be unnecessary to remove this large mass of rock in order to achieve a standard filtration design, given that the mass itself will act as some sort of filter, and there is a sufficient volume of rock in the existing revetment for it to remain adequately in place (even with some damage) to protect the piling of Flight Deck (in combination with placement of the additional armour layer).

As part of a document researched for the 50th anniversary of Flight Deck in 2016[3], evidence (in a letter from John F Webb-Wagg) was provided that the "rock wall contains 25,000 tons of various size rocks and was designed and installed by Mr WG Hunt (a land, mining and engineering surveyor), and at that time a resident of the building, together with Professor Hattersley of the University of NSW". A mass of 25,000 tons (about 25,400 tonnes) would seem excessive for Flight Deck alone, but may be more realistic if this includes the mass of rock placed at Shipmates and Ramsay Street to the north, and 1 Frazer Street, Frazer Street and 1104 and 1106 Pittwater Road Collaroy to the south (all of which had rock placed in 1967).

In a letter believed to have been written by WG Hunt on 26 August 1967 (in "Flight Deck 1966-2016"), it was noted that the Flight Deck tower stands on approximately 180 concrete piles driven to solid rock at depths of about 56 feet (17.1m)[4]. It was also noted that "there is no need whatsoever for any concern as to the possibility of this building ever collapsing…it would be possible, given extraordinary adverse conditions, that the filling under the P1 floor level [basement car park] could possibly be slightly undermined. The wall we have designed is intended purely to prevent the abovementioned undermining and to prevent further erosion of our beach front and at the same time to accelerate the restoration of our beach area. It is our opinion that very rarely will the actual rocks in our proposed wall be visible".

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[2] Although it is recognised that hydraulic stability formulae are generally derived based on there being 2 layers of primary armour. This statement is made on the basis of the new layer only.
[3] "Flight Deck1966-2016", written and researched by Michelle Richmond, Local Studies Historian at the then Warringah Council Library.
[4] Review of structural engineering Drawing No. 63065:1 of Taylor Thompson Whitting, dated 23 November 1963, indicated that two boreholes on the landward side of Flight Deck encountered rock at this depth of 56 feet.

To tie in appropriately with an adjacent (already existing) rock revetment to the south of Flight Deck at 1 Frazer Street and 1112 Pittwater Road Collaroy, that was upgraded after the June 2016 storm, the proposed works will connect to those works. These upgraded works already extend to the southern boundary of Flight Deck.

The executive of the adjacent property to the north of Flight Deck, at Shipmates (1122 Pittwater Road Collaroy), which also has existing protection works, has decided not to proceed with upgraded protection works at this time. The proposed works will thus extend to the northern property boundary of Flight Deck, which projects about 4m north of the northern face of the Flight Deck tower due to the angle of the boundary (see Figure 1). Given this projection, given that the Flight Deck building is on deep piles, and given the conservative new primary armour design as outlined in subsequent sections, it was not considered to be necessary to extend further north (that is, on to the Shipmates land)[5]. That is, extending the proposed works to the northern boundary was considered to provide a sufficient level of protection for the Flight Deck tower, although it is recognised that the land north-east of the north- eastern corner of Flight Deck will not be adequately protected until Shipmates upgrades its works. This is considered to be acceptable, as any erosion at the north-eastern corner of the Flight Deck lot would not be expected to create a significant risk to life or significant risk to non-ancillary built assets (the ancillary assets at the most potential risk at Flight Deck are a fence at the north-eastern corner of the tower, and the surface at the northern end of the tiled area, as well as surrounding lawn and garden areas).

Figure 1: Alignment of northern face of Flight Deck (blue) in relation to property boundaries (yellow) and projection of proposed works north of Flight Deck building (red)

[5] Although it may be required (however slightly) to remove and reposition some of the rock at Shipmates to ensure a reasonable interlock to the new works at Flight Deck.

Immediately seaward of the Flight Deck building, it is not possible to fit the minimum recommended maintenance setback to the landward edge of the revetment of 5m as per the *Specifications*. There is about a 2.3m distance between the landward edge of the revetment and seaward face of the Flight Deck tower. This means that any required maintenance of the revetment would have to be undertaken from the beach, post-storm when sand levels had recovered. This is considered to be acceptable given that the Flight Deck building is on deep piled foundations, and there is unlikely to be any emergency requirement for immediate post-storm repairs to the revetment. That stated, the rock mass has been selected to minimise the potential damage in the design storm, with hydraulic stability calculations undertaken for a 500-year ARI event.

3. PROPOSED DESIGN LIFE OF PROTECTION WORKS

A design life of 60 years has been adopted for the proposed protection works (that is, at the year 2077). As outlined in Horton et al (2014) and Horton and Britton (2015), this design life is considered to be appropriate in relation to beachfront development (that relies on the protection works for protection against erosion/recession over the design life) as:

- it is consistent with Australian Standards applying to the residential development landward of the protection works:
  - in *AS 3600*, a 50 years ± 20% design life[6] (that is, 40 years to 60 years) is used in devising durability requirements for concrete structures;
  - in *AS 2870*, for design purposes the life of a structure is taken to be 50 years for residential slabs and footings construction;
  - in *AS 1170.0 – Structural Design Actions - General Principles*, the design life for normal structures (Importance Level 2, as applies to typical residential development) is generally taken as 50 years; and
  - in *AS 4678,* the design life for earth-retaining structures (structures required to retain soil, rock and other materials) is noted as 60 years for river and marine structures and residential dwellings.
- the cost of new residential development is amortised for tax purposes over 40 years based on Subdivision 43-25 of the *Income Tax Assessment Act 1997*;
- a design life of at least 50 years would be considered to be reasonable for permanent structures used by people (AGS, 2007a, b); and
- this design life of 60 years was adopted in the gazetted *Collaroy-Narrabeen Beach and Fishermans Beach Coastal Zone Management Plan* (CZMP).

A minimum 60-year design life was adopted in the *Specifications*. As noted therein, this design life recognises, among other things, that redevelopment of beachfront properties typically occurs within such a period. In practice, rock incorporated within the proposed works would have a considerably greater life than 60 years based on the durability criteria specified in the design.

The proposed design life of 60 years is thus appropriate, and meets the minimum requirement in the *Specifications*.

[6] Period for which a structure or a structural member is intended to remain fit for use for its intended purpose with appropriate maintenance.

4. ADOPTED DESIGN PROBABILITY AND RISK USED IN THE DESIGN

A 500-year Average Recurrence Interval (ARI) storm event has been adopted for design. This exceeds the minimum 50-year ARI requirement in the *Specifications*.

A 100-year ARI event would have been adopted if a new revetment was to be constructed, but the rarer 500-year ARI was adopted as the new armour rock is proposed to be placed over an existing revetment with some undersized armour and uncertain filter layers. It was considered that the rock armour designed for a 500-year ARI event, in combination with the existing rock revetment, should acceptably withstand a 100-year ARI storm in combination with the pled building foundations.

It is important to understand that adoption of the 100/500 year ARI storm event for design actually leads to a much rarer storm being able to cause "failure" of the rock revetment. This is because:

- the revetment design is to the 0-5% damage level (generally referred to as the "no damage" condition), whereas failure is generally considered to be the 20% damage level;
- the required rock mass is governed by wave height, which is depth-limited;
- the revetment is only subject to the design wave at the end of the design life, after projected sea level rise (the highest possible water level) has been realised;
- for most of the design life the design wave height cannot occur as it is depth-limited;
- water levels only increase slightly as ARI's become exponentially rarer; and
- rock structures can accommodate some damage without failure.

Considering a single parameter (such as wave height, or water level), a 100-year ARI event has a 45% probability of occurring over a 60-year life. However, this event for a single parameter only has a 1% probability of occurring in year 60 of the design life, which is the only year when the design wave height would be physically able to occur due to being depth-limited in earlier years[7].

In Australian Standard *AS 4997-2005*, a design life of 50 years is recommended for normal maritime structures (specifically excluding rock structures), in conjunction with a 500-year ARI design wave height (this event has a 10% probability of occurring over the design life). However, this does not apply to rock structures (both explicitly within the Standard, and as explained by the logic above). That stated, due to depth-limited conditions it is reiterated that the design wave height cannot occur until the last year of the design life, such that the selection of a 100-year ARI design storm event (based on new armour being hydraulically stable for the 500-year ARI event) is considered to be conservative for the proposed protection works.

Water level, scour and wave calculations for the 100-year and 500-year ARI events are provided in Section 5 and Section 6 respectively. Hydraulic stability calculations for the 100-year ARI and 500-year ARI events are provided in Section 7 and Section 8 respectively.

[7] There is a 10% probability of the design storm occurring in any of the last 10 years of the design life, again only considering a single parameter. However, it should be noted that the sea level rise value adopted (see Section 5.2) is not a certain occurrence, and has an associated probability of exceedance that can be approximately estimated in the order of 50%. Similarly, the scour level adopted (Section 5.4) is not a certain occurrence. That is, use of a 500-year ARI wave height and water level cannot be used to directly imply particular encounter probabilities for structural damage due to the multiple probabilistic factors at play.

5. WATER LEVELS, SCOUR AND WAVES FOR 100 YEAR ARI EVENT

5.1 Present Design Ocean Water Level

Based on Department of Environment, Climate Change and Water [DECCW] (2010), the 100-year ARI ocean water level (in the absence of wave action) as of 2010 is 1.44m AHD. This is also consistent with Manly Hydraulics Laboratory [MHL] (2016a)[8].

5.2 Sea Level Rise

In the *Specifications*, it is noted that sea level rise projections of 0.4m at 2050 and 0.9m at 2100 (both relative to 1990) may be adopted. For the proposed design life of 60 years (at 2077), it would be possible to interpolate between the 2050 and 2100 projections to obtain a sea level rise value of 0.67m AHD relative to 1990[9].

However, based on the *Specifications*, "variations to the above sea level rise projections may be considered. Where a variation is proposed, it shall be supported by a report prepared by a suitably qualified engineer". Given the non-linear rate of sea level rise and conservatism in the above benchmarks, it is considered to be most appropriate to apply a variation to the *Specifications* and to directly derive sea level rise values from Intergovernmental Panel on Climate Change [IPCC] (2013a, b), which is widely accepted by competent scientific opinion. Furthermore, the methodology used to adopt the sea level rise values herein is the same as that used in the CZMP.

With a base year of 2010, as DECCW (2010) water levels were derived at 2010, the sea level rise values presented in Table 1 (at 2077) were determined for various emissions scenarios.

Table 1: Global mean sea level rise (m) from 2010 to 2077 derived from IPCC (2013b)

Emissions Scenario	Exceedance Probability		
	95% exceedance	Median	5% exceedance
SRES A1B	0.27	0.38	0.50
RCP2.6	0.20	0.30	0.41
RCP4.5	0.24	0.35	0.46
RCP6.0	0.24	0.34	0.45
RCP8.5	0.32	0.44	0.58
Average	0.25	0.36	0.48

Taking the median exceedance probability and average of the 5 emissions scenarios, and adding 15% for local sea level rise variation based on IPCC (2013b), a sea level rise value of 0.42m at 2077 (relative to 2010) was derived. Therefore, the 100-year ARI still water level at 2077 based on IPCC (2013b) is 1.86m AHD.

5.3 Design Ocean Water Level at End of Design Life

As noted in Section 5.2, the adopted 100-year ARI still water level at 2077 (at the end of the design life) is 1.86m AHD. Wave setup, caused by breaking waves adjacent to a shoreline, can also increase still water levels, as discussed further in Section 5.6.

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[8] Although MHL (2016a) also found that uncertainties in the extreme value analysis methods adopted to be of the order of +0.1m, which could be applied as an additional increment on the design water level in a more conservative analysis.
[9] This is a sea level rise of 0.61m relative to 2010, discounting historical sea level rise at 3mm/year as recommended in DECCW (2010).

5.4 Scour Level

A storm scour level of -1m AHD is typically adopted at NSW beaches. This is based on stratigraphic evidence of historical scour levels and observed scour levels occurring during major storms (Carley et al, 2015).

For a rock revetment, the toe level is derived based on the anticipated scour level. In the *Specifications*, a minimum toe level for protection works of -1m AHD was adopted, "although a higher toe level may be considered if there is evidence of an inerodible layer at a level above -1m AHD".

In the year 2000, test pits were excavated at the Flight Deck revetment to determine its characteristics, as reported in Jeffery & Katauskas Pty Ltd (2000). A view of the mostly-buried revetment prior to the excavation, and the test pit locations, is provided in Figure 2. The test pits indicated that a cemented sand layer was present at a level of about 0.5m AHD in the north and -1m AHD in the south, with toe boulders likely to be founded on the cemented sand.

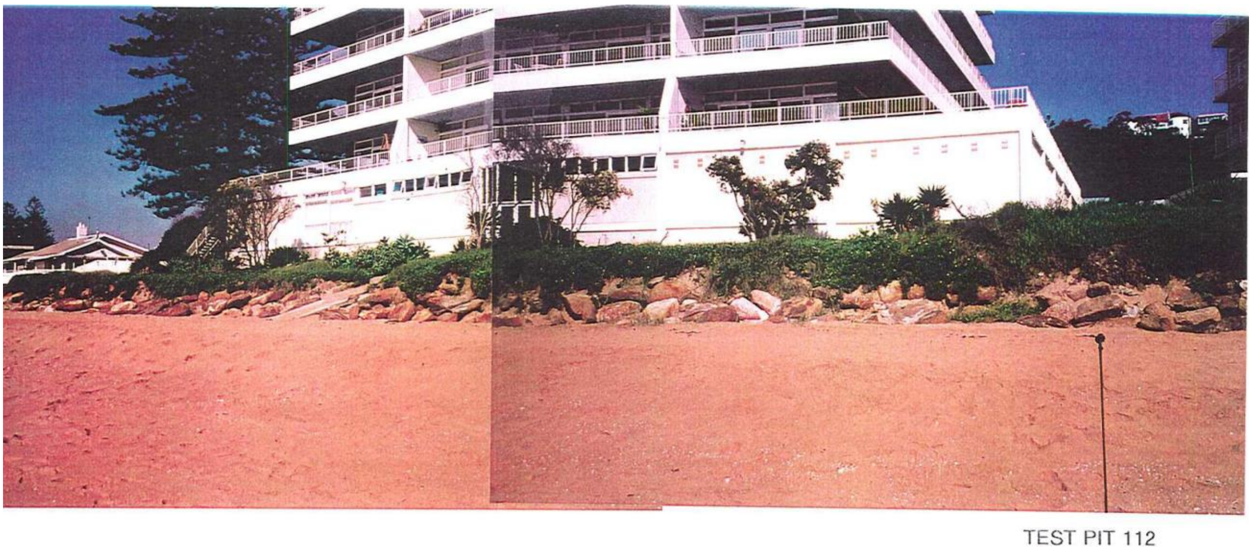

Figure 2: View of Flight Deck in 2000, with Test Pit 112 location of Jeffery & Katauskas (2000) shown (note that Test Pit 113 was excavated near the southern boundary towards the far left)

As evident in Figure 3, cemented sand was visible offshore of the southern end of the subject property after the June 2016 storm. Based on a WRL drone survey, the cemented sand level at this outcrop was about -0.2m to -0.3m AHD.

A Jeffery & Katauskas (2000) borehole at the seaward end of Frazer Street (to the south of Flight Deck), aligned with the seaward face of Flight Deck, found a cemented sand level of 0.2m AHD. A test pit seaward of Frazer Street found a cemented sand level of -0.5m.

Figure 3: Cemented sand visible offshore of southern end of Flight Deck (between red arrows) on 10 June 2016

A Jeffery & Katauskas (2000) borehole at the seaward end of Ramsay Street (to the north of Flight Deck) found a cemented sand level of 0m AHD. Coffey Partners International (1998) completed a test pit seaward of Ramsay Street and found a cemented sand level of around -0.5m AHD.

Douglas Partners (2016) undertook a geotechnical investigation which included drilling of a borehole at Ramsay Street (at an approximate cross-shore position at an alignment consistent with the landward face of the Flight Deck tower). Correcting an approximately estimated surface level of 5m AHD used by Douglas Partners (2016), which based on survey would be closer to 5.6m AHD, this would put dense to very dense sand at -2.1m AHD to 1.7m AHD at this Ramsay Street location. Analysing the cone penetration test results of Douglas Partners (2016), the upper surface of the cemented sand layer can be interpreted to be at 1.1m AHD at that location[10].

[10] As interpreted by Horton Coastal Engineering and reviewed by Paul Roberts of JK Geotechnics.

Based on a WRL drone survey of cemented sand outcrops observed seaward of properties immediately north of Ramsay Street after the June 2016 storm, cemented sand levels about 40m seaward of the Douglas Partners (2016) borehole noted above were about 0.1m AHD. Comparing the Douglas Partners (2016) and WRL cemented sand levels, this would indicate that the upper surface of the cemented sand layer dips at about 1:40 (1 to 2°) moving seaward.

As long-term recession is realised, scour levels may lower at a particular cross-shore position where the beach profile translates landward. In the CZMP, a "best estimate" inverse slope of the active beach profile of 30 was adopted, which would cause 13m of long term recession due to sea level rise over the design life based on the Bruun Rule (using the adopted sea level rise value of 0.42m from Section 5.2), and about 0.4m of lowering assuming no restriction from cemented sand.

For design purposes, depth limited wave conditions must be determined at a plunging distance (or plunge length) seaward of the toe of the proposed works. Based on Coastal Engineering Research Center (1984) and Smith and Kraus (1991), the plunging distance is approximately equal to 10m. For a 1:40 cemented sand slope, and assuming a cemented sand level of -0.2m AHD at the toe of the proposed works, the cemented sand level 10m seaward of the works is about -0.5m AHD. Adding the 0.4m of lowering from long term recession due to sea level rise, the bed level at a plunging distance from the works is -0.9m AHD.

Although it is expected that cemented sand (where present) would restrict this lowering, the level and extent of cemented sand at a location 10m seaward of the works is unknown. On this basis, it has been assumed herein that the scour level at the plunging distance is -1m AHD, consistent with general NSW beach scour levels and similar to the value determined above. The design ocean depth (excluding wave setup) is thus 2.86m at the plunging distance.

At the revetment, placing the toe on the cemented sand layer (expected to be at around -0.2m AHD), along with the placement of larger toe rocks as per the Drawings to continue to provide support to the revetment even if some toe settlement occurs, is considered to provide a suitable toe level and to meet the intent of the *Specifications* (given the evidence of an inerodible layer at a level above -1m AHD).

5.5 Ocean Waves

Extreme value offshore wave conditions have recently (since the June 2016 storm) been re-evaluated for Sydney by Louis et al (2016), based on offshore Waverider buoy records. They determined 100-year ARI offshore significant wave heights (H_s) of 9.5m and 8.7m for 1-hour and 6-hour durations respectively.

Beach erosion and relatively large wave run-up is strongly linked to the occurrence of high wave conditions with elevated ocean water levels, so erosion and run-up are more likely to be significant when large waves coincide with a high tide. Consistent with MHL (2016b), a 6-hour duration is considered to be appropriate for design, as storms with a duration of 6 hours are likely (50% probability) to coincide with high tide on the NSW coast (which is a prerequisite for elevated water levels to occur). A 1-hour duration only has an 8% probability of coinciding with high tide. Therefore, an offshore H_s (or H_o) of 8.7m was adopted herein.

In adopting 100-year ARI wave conditions herein, it was assumed that the design water level and wave can occur at the same time, which is conservative. Shand et al (2012) found that considering the joint probability of waves and tidal residuals for Sydney, the wave height for the joint 100-year ARI event reduced by about 10% as the tidal residual increased from 0.05m to 0.4m (with the latter necessary to achieve the design water level). That stated, adopting joint 100-year ARI water level and wave conditions is not entirely unreasonable, as elevated waves and water levels can be generated by the same weather systems.

A design peak spectral wave period (T_p) of 13s was adopted, based on Shand et al (2011), who determined the associated wave period for the 100-year ARI H_s event at Sydney as 13.0s (± 0.7s considering 90% confidence intervals).

5.6 Wave Setup and Design Depth

Goda (2010a) has presented a relationship between wave setup at the shoreline, wave steepness, and beach slope. For a T_p of 13s, the deepwater wavelength (L_o) is 264m, and hence for an H_o of 8.7m the wave steepness is 0.03m. The beach slope offshore of the subject properties, between -40m and -10m AHD, is approximately 1:100 (MHL, 2016b). For this slope and wave steepness, Goda (2010a) estimated that wave setup at the shoreline was 11% of H_o.

However, it is conservative to apply the full quantum of shoreline wave setup to define the depth limited breaking wave height on the proposed revetment, as the design wave is applied at a plunging distance offshore of the revetment[11]. With a depth (h) of 2.86m at the plunging distance, based on Goda (2010a) then h/H_o is 0.3, and wave setup at the plunging location is 9% of H_o. Therefore, setup at the plunging location is 0.8m, and the design depth at the plunging location is 3.7m.

As it used in wave overtopping calculations (see Section 10), the present day depth at the plunging location was determined as the present design ocean water level (1.44m AHD, see Section 5.1), plus an allowance for scour down to -1m AHD as per Section 5.4, plus approximate wave setup of 0.8m as above. This gave the present-day 100-year ARI depth at the plunging location as 3.2m.

5.7 Design Wave Height at Structure

The method of Goda (2010a, b) for incipient breaking of significant waves was employed with the following parameters:

- water depth of 3.7m as defined in Section 5.6;
- L_o of 264m based on a wave period of 13s; and
- beach slope of 1:30, which is the bed slope down to the “inner Hallermeier” depth that was adopted in the CZMP.

This gave an H_s for incipient breaking of 2.3m (with a breaker index of 0.6), which was adopted as the design wave height at the structure.

Using the methodology in Battjes and Groenendijk (2000) for wave height distributions in the shoaling and breaking zone, $H_{10\%}$[12] and $H_{2\%}$ values of 2.8m and 3.0m were derived as these respective design wave heights at the structure. As it used in wave overtopping calculations (see Section 10), a present-day 100-year ARI H_s for incipient breaking of 2.0m was calculated using the Goda (2010a, b) methodology.

[11] Furthermore, the subject properties may not be fully exposed to the design offshore wave height, due to sheltering provided by Long Reef headland from wave directions from the S to SE.
[12] Denoted as $H_{1/10}$ by Battjes and Groenendijk (2000).

6. WATER LEVELS, SCOUR AND WAVES FOR 500 YEAR ARI EVENT

6.1 Present Design Ocean Water Level

Log-extrapolating beyond the 100-year ARI event in DECCW (2010), the 500-year ARI ocean water level (in the absence of wave action) as of 2010 is about 1.51m AHD. Extrapolating beyond the 100-year ARI event in [MHL] (2016a), the corresponding level is about 1.52m, which was adopted for design.

6.2 Sea Level Rise

As per Section 5.2, a sea level rise value of 0.42m at 2077 (relative to 2010) was used. Therefore, the 500-year ARI still water level at 2077 based on IPCC (2013b) is 1.94m AHD.

6.3 Design Ocean Water Level at End of Design Life

As noted in Section 6.2, the adopted 100-year ARI still water level at 2077 (at the end of the design life) is 1.94m AHD. Wave setup, caused by breaking waves adjacent to a shoreline, can also increase still water levels, as discussed further in Section 6.6.

6.4 Scour Level

As discussed in Section 5.4, a bed level of -1m AHD was adopted at a location 10m seaward of the proposed works for the 100-year ARI event. For the 500-year ARI event, a bed level of -1.2m AHD was adopted, that is allowing for 0.2m of additional scour given the larger waves for this event. The 500-year ARI design ocean depth (excluding wave setup) is thus 3.14m at the plunging distance.

6.5 Ocean Waves

Log-extrapolating beyond the 100-year ARI event in Louis et al (2016), the 500-year ARI H_s for a 6-hour duration is 9.9m. A T_p of 13s was adopted as per Section 5.5.

6.6 Wave Setup and Design Depth

Applying wave setup as 9% of H_o as per Section 5.5, setup at the plunging location is 0.9m, and the design depth at the plunging location is 4.03m.

6.7 Design Wave Height at Structure

Using the method of Goda (2010) as per Section 5.7, the 500-year ARI H_s for incipient breaking was determined to be 2.5m. Using the methodology in Battjes and Groenendijk (2000), $H_{10\%}$ and $H_{2\%}$ values of 3.0m and 3.2m were derived as these respective design wave heights at the structure.

7. ROCK REVETMENT HYDRAULIC STABILITY DESIGN (100 YEAR ARI EVENT)

7.1 Preamble

The most comprehensive and up to date guidance on rock revetment design is provided in the *Rock Manual* (CIRIA et al, 2012). For design of revetments in shallow water conditions (as applies for the proposed revetment), four different methodologies for rock armour sizing are outlined therein, namely:

1. the Hudson formula;
2. a modified Hudson formula proposed by van der Meer (1988);
3. van der Meer formulae for shallow water conditions; and
4. van Gent et al (2004) stability formula.

Method 3 was preferred herein (see Section 7.2), with checks of the other methodologies as discussed in Section 7.3.

7.2 Method 3 (van der Meer Formulae for Shallow Water)

Method 3 was adopted herein with parameters as follows:

- H_s of 2.3m and $H_{2\%}$ of 3.0m (from Section 5.7);
- T_p of 13s (from Section 5.5), with $T_{m-1,0}$ assumed to be T_p ÷1.1;
- number of waves (N) in storm calculated assuming a 2.5 hours storm duration, given that waves are depth-limited and elevated water levels can only occur at high tide (for a T_p of 13s, which is a $T_{m-1,0}$ of 11.8s, N is about 580 using a correction for less than 1,000 waves[13]);
- (basalt) rock density of 2,650kg/m^3;
- (sea) water density of 1,025kg/m^3;
- structure slope of 1:2.4 (vertical:horizontal);
- notional permeability of structure of 0.4;
- coefficient c_{pl} of 7.25, and c_s of 1.05, as applies for a 5% damage limit; and
- S_d of 2 as applies for 0-5% damage (start of damage) for armourstone in a double layer.

Method 3 is the only one of the formulae considered herein to include wave period as an input parameter. Under sensitivity testing of a range of wave periods from 6s to 14s, it was found that the design T_p of 13s produced the largest required armour size, and so was adopted as the critical case.

For the critical case, the median primary armour mass was determined to be 2.4 tonnes.

7.3 Other Methodologies

7.3.1 Method 1 - Hudson

For the Hudson methodology, parameters were adopted as follows:

- $H_{10\%}$ of 2.8m (from Section 5.7);
- stability coefficient (K_D) of 2.0 (for breaking waves); and

[13] The actual number of waves in 2.5 hours for this $T_{m-1,0}$ of 11.8s is 760.

- rock and sea water densities, and structure slope, as per Section 7.2.

The median primary armour mass was determined to be 3.1 tonnes.

7.3.2 Method 2 - Modified Hudson (van der Meer, 1988)

For the modified Hudson methodology, parameters were adopted as follows:

- H_s of 2.3m (from Section 5.7);
- stability coefficient (K_D) of 4.0 (for permeable core); and
- rock and sea water densities, and structure slope, as per Section 7.2.

The median primary armour mass was determined to be 0.7 tonnes. Realisation of a permeable core in practice is likely given the extent of rock fill in the revetment and lack of geotextile. For an impermeable core assumption with K_D equal to 1.0, the median primary armour mass was determined to be 4.7 tonnes. Assuming an intermediate K_D of 2.0 as per Section 7.3.1, the median primary armour mass was determined to be 2.4 tonnes.

7.3.3 Method 4 – van Gent et al (2004)

For the van Gent et al (2004) methodology, parameters were adopted as follows:

- H_s of 2.3m (from Section 5.7);
- N determined as per Section 7.2;
- S_d of 2 as applies for 0-5% damage (start of damage) for armourstone in a double layer;
- core particle diameter of 60mm (a coarse gravel); and
- rock and sea water densities, and structure slope, as per Section 7.2.

The median primary armour mass was determined to be 1.7 tonnes. Note that if the core particle diameter is increased to 220mm, the median primary armour mass reduces to 1.2 tonnes. If sand is used as the core, the median primary armour mass increases to 2.0 tonnes.

8. ROCK REVETMENT HYDRAULIC STABILITY DESIGN (500 YEAR ARI EVENT)

8.1 Preamble

As per Section 7.1, four different methodologies for rock armour sizing were considered, namely:

1. the Hudson formula;
2. a modified Hudson formula proposed by van der Meer (1988);
3. van der Meer formulae for shallow water conditions; and
4. van Gent et al (2004) stability formula.

Method 3 was preferred herein (see Section 8.2), with checks of the other methodologies as discussed in Section 8.3.

8.2 Method 3 (van der Meer Formulae for Shallow Water)

Method 3 was adopted herein with parameters as per Section 7.2, except with H_s of 2.5m and $H_{2\%}$ of 3.2m. For the critical case, the median primary armour mass was determined to be 3.0 tonnes.

8.3 Other Methodologies

8.3.1 Method 1 - Hudson

For the Hudson methodology, parameters were adopted as per Section 7.3.1, except with $H_{10\%}$ of 3.0m (from Section 6.7). The median primary armour mass was determined to be 3.8 tonnes.

8.3.2 Method 2 - Modified Hudson (van der Meer, 1988)

For the modified Hudson methodology, parameters were adopted as per Section 7.3.2, except with H_s of 2.5m (from Section 6.7). The median primary armour mass was determined to be 0.9 tonnes. For an impermeable core assumption with K_D equal to 1.0, the median primary armour mass was determined to be 5.7 tonnes. Assuming an intermediate K_D of 2.0 as per Section 7.3.1, the median primary armour mass was determined to be 2.9 tonnes.

8.3.3 Method 4 – van Gent et al (2004)

For the van Gent et al (2004) methodology, parameters were adopted as per Section 7.3.3, except with H_s of 2.5m (from Section 6.7). The median primary armour mass was determined to be 2.1 tonnes. If the core particle diameter is increased to 220mm, the median primary armour mass reduces to 1.5 tonnes. If sand is used as the core, the median primary armour mass increases to 2.4 tonnes.

9. ADOPTED DESIGN

9.1 Masses and Gradings

A median primary armour mass of 3.0 tonnes was adopted for design, as reflected on the Drawings, consistent with the calculated Method 3 mass for the 500-year ARI event (Section 8.2). This is less than the Method 1 mass of 3.8 tonnes (Section 8.3.1, as expected given Hudson is known to be conservative), similar to the Method 2 mass of 2.9 tonnes assuming an intermediate K_D of 2.0 (Section 8.3.2), and larger than the Method 4 mass of 2.1 tonnes assuming a core particle diameter of 60mm (Section 8.3.3).

Although 3.0 tonnes is less than the minimum mass for igneous rock of 3.8 tonnes in the *Specifications*, as noted therein "variations to [this]...rock size...may be considered. Where a variation is proposed it shall be supported by a report proposed by a suitably qualified engineer". The variation can be supported due to the relatively flat (1:2.4) slope of the revetment, with this slope accounted for in the hydraulic stability calculations. The 3.8 tonnes mass in the *Specifications* was based on a slope of 1:1.5, which requires a larger mass than the flatter 1:2.4 slope.

A grading of 0.75 to 1.25 times the median mass was adopted for the primary armour, consistent with the *Specifications*.

9.2 Thickness of Layers and Meaning of Dimensions on Drawings

The minimum requirement in the *Specifications* is for 2 rocks of thickness in primary and secondary layers. As it is proposed to add a single layer of primary armour to the existing revetment, it is not strictly possible to verify that this requirement has been met. However, given the existing thickness of rock layers and use of oversized additional armour, the proposed works are considered to provide an acceptably low risk of damage to the Flight Deck building (which has deep piled foundations).

The thickness of a single layer of rock is determined as $k_t \times D_{n50}$, where k_t is the layer thickness coefficient (and depends on rock shape and the type of packing), and D_{n50} is the median dimension of an equivalent cube giving the median rock mass (CIRIA et al, 2012). For the drawings, the added armour thickness was determined using a k_t value of 1.0, as recommended in Burcharth and Hughes (2011) for rough quarrystone with random placement in 2 layers.

In construction, different thicknesses may be obtained, and the thickness measuring technique (method of survey) will need to be agreed with the contractor. CIRIA et al (2012) noted that based on recent research, the k_t value of 1.0 is likely to be an overestimate. For standard or dense placement, and blocky rock, a k_t value of 0.91 is noted for a spherical foot staff survey method, and 0.96 for a highest point survey method. Corresponding values for irregular (not blocky) rock are 0.87 and 0.92 respectively.

Masses stated on the drawings are based on hydraulic stability formulae, with dimensions only approximate, and expressed on the drawings as square opening sieve size dimensions (calculated as $1.15 \times D_{n50}$). Note that in CIRIA et al (2012), square opening sieve size dimensions are calculated as $1.19 \times D_{n50}$.

9.3 Rock Durability

Notes are included on Drawing S.01 to specify required rock durability criteria, consistent with the *Specifications* for igneous rock.

10. WAVE OVERTOPPING AND REVETMENT CREST LEVEL

As per the Drawings, a minimum revetment elevation of 5.7m AHD has been adopted (with a maximum crest elevation of 6.0m AHD to allow a construction tolerance).

The Neural Network for Wave Overtopping Predictions (van Gent et al, 2007) associated with EurOtop (van der Meer et al, 2016), Version 2.04 (March 2016), was utilised to calculate average wave overtopping rates in a 100-year ARI storm at the proposed works for both present conditions and in 2077 at the end of the design life. Input parameters are summarised in Table 2. Note that the mean wave period was derived as the peak spectral wave period of 13s divided by 1.1. The roughness coefficient was derived from Table 6.2 of van der Meer et al (2016) for rocks in 2 layers with a permeable core. The crest and armour freeboard in Table 2 was devised based on a 5.7m AHD crest elevation.

Table 2: Input parameters for Neural Network for Wave Overtopping Predictions

Parameter	Value	
	Present-day	2077
Angle of wave attack (°)	0.0	0.0
Water depth in front of structure (m), see Section 5.6	3.2	3.7
Significant wave height at the toe of structure (m), see Section 5.7	2.0	2.3
Mean wave period (s)	11.8	11.8
Water depth at the toe of structure (m)	3.2	3.7
Width of toe (m)	0.0	0.0
Roughness coefficient	0.4	0.4
Angle of down slope (cotangent)	2.7	2.7
Angle of upper slope (cotangent)	2.7	2.7
Crest freeboard in relation to SWL (m)	3.5	3.0
Berm width (m)	0.0	0.0
Water depth at the berm of the structure (m)	0.0	0.0
Berm slope (tangent)	0.0	0.0
Armour freeboard in relation to SWL (m)	3.5	3.0
Armour width (m)	0.0	0.0

The base case resulting mean overtopping discharges were 2.2L/s/m for the present day, and 13L/s/m at 2077. Increasing the crest elevation of the revetment to 6.0m AHD reduced mean overtopping discharges to 1.4 and 7.6L/s/m respectively. Sensitivity testing was undertaken with alteration to roughness and slope values, without significant changes in overtopping discharges.

Historically, based on the previous (2007) version of EurOtop, a 50L/s/m overtopping discharge would have been considered a threshold for damage to a grassed or lightly protected promenade or reclamation cover. That is, based on the 2007 version of EurOtop, the estimated mean overtopping discharges over the design life would not have been considered to be damaging even to grass cover landward of the revetment. The limit for damage of a paved promenade in the 2007 EurOtop version was 200L/s/m.

In the latest version of EurOtop (van der Meer et al, 2016), there is more of a focus on linking tolerable overtopping with the peak volume, and hence on the wave height that causes the overtopping, thus changing the limits for tolerable overtopping. For a grass covered crest and landward slope, maintained and closed grass cover and with H_{m0} (spectral significant wave height) of between 1m and 3 m (as applies here), a limit of 5L/s/m was adopted. On this basis, the tiled area landward of the proposed revetment may incur some overtopping damage in the 100-year ARI storm towards the end of the design life.

In van der Meer et al (2016), the overtopping limit for damage to weaker building elements such as windows and doors is 1L/s/m. Therefore, it would be necessary to barricade the glass entry door on the seaward side of the Flight Deck tower to prevent its damage from wave overtopping in severe storms. It is recommended that the Flight Deck Executive has readily deployable materials and equipment available to barricade the entry if severe storms are forecast. There should be sufficient warning time for such storms, with the time of the most severe overtopping predictable as it would coincide within around two hours of high astronomical tide. Otherwise, the predicted wave overtopping should not cause any structural damage to the Flight Deck tower.

Any landscaping damage that occurs as a result of erosion and/or wave overtopping is not considered to be a significant issue, with the owners able to restore damaged landscaped areas after any significant storms. The works themselves would not expected to be damaged by overtopping for the design storm over the design life, nor the Flight Deck tower, except at the glass doors (unless they were barricaded).

To adapt to projected sea level rise by increasing the height and width of the armour crest, it would be possible to remove the most seaward portion of the tiled area and place additional armour rock, and/or to place additional armour on the revetment slope. It would also be possible to construct a vertical concrete wall on the landward side of the revetment as an additional barrier to wave overtopping. At 2077 and based on the design projections, raising the crest to 6.5m AHD reduces the mean overtopping discharge to 0.7L/s/m.

Overtopping is not a significant issue in terms of inundation of the internal areas of Flight Deck, if the entry door is barricaded in severe storms. Even if the barricade failed, the entry area has a tiled floor and concrete walls, and significant damage from inundation of this area would not be expected. The entry steps rise to 6.5m AHD, reducing the likelihood of inundation propagating beyond the tiled entry.

With regard to safety of humans, a tolerable limit 1L/s/m (for H_{m0} of 2m) is noted in van der Meer et al (2016) for people at a revetment crest with a clear view of the sea. A range of 1 to 10L/s/m was adopted in the 2007 version of EurOtop for pedestrians (trained staff, well shod and protected, expecting to get wet). For their safety, it would therefore be necessary for people to avoid the tiled area landward of the revetment crest in severe storms. Barricading of the entry door would reduce the likelihood of people accessing this area.

In the *Specifications*, a minimum crest level of 6.5m AHD is specified, "although a lower crest level may be considered. Where a lower crest level is proposed it shall be supported by a report prepared by a suitably qualified engineer". As reported above, a minimum crest level of 5.7m AHD, as proposed, would give satisfactory overtopping discharges over the design life based on van Gent et al (2007). On this basis, as any overtopping of the revetment at 5.7 to 6m AHD is unlikely to cause significant damage, a variation to the minimum crest level is considered to be justified. Furthermore, a higher crest level than 6.0m AHD would be visually intrusive (being 1m above the tiled area and 1.5m above the current rock crest). It is considered that the 6m AHD maximum crest level is the right balance between protection and amenity. It also ties in more appropriately with the existing revetment at 1112 Pittwater Road and 1 Frazer Street which has a crest level of 5.4m AHD.

11. GLOBAL STABILITY

In the *Specifications*, it is stated that "the seawall shall have a minimum factor of safety of 1.5 against global slope stability failure". This has been demonstrated in a separate report prepared by JK Geotechnics.

In the *Specifications*, it is stated that:

> "A geotechnical investigation shall be conducted at the property as part of the seawall design process to confirm, among other things, the extent of existing rock protection. The investigation shall be carried out by a suitably qualified engineer. The investigation shall include, as a minimum, excavation of three test pits along the seaward property boundary with the pits generally aligned perpendicular to the seaward property boundary".

This is not required at the subject property, as the June 2016 storm did a "natural" excavation to reveal the nature of the existing rock protection. The June 2016 erosion revealed the seaward extent of the rock revetment, and enabled measurement of the existing revetment slope, which are two key design parameters. Historical information would indicate that the tiled area seaward of the Flight Deck tower is underlain by rock, but lack of space for access of a drilling rig did not allow further investigation of this.

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