ADAPTIVE DESIGN OF FLYNNS BEACH SEAWALL PORT MACQUARIE

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Abstract

The existing retaining wall along Flynns Beach was constructed in the 1970s and despite serving an effective function for over 30 years, it was not designed as a seawall and is now in a poor condition. A new seawall is required to safeguard proposed foreshore enhancements and protect against coastal erosion and expected sea level rise.

Port Macquarie Hasting Council engaged three professional consultants to develop concept design options and following public exhibition and review by an evaluation panel selected a composite and adaptive design option proposed by NSW Public Works' Manly Hydraulics Laboratory (MHL). The proposed solution comprises a piled concrete stepped seawall integrated with the existing boat ramp and wheelchair access, a partially buried rubble mound rock seawall and a vertical precast concrete panel seawall design.

Key features of the preferred seawall include seating and beach access in front of the surf club, enhanced wave energy dissipation along the car park, maximised beach and parkland amenity in other areas and concrete integrally coloured to match local beach sand. The proposed design is versatile for a variety of community needs and is adaptable to future climate uncertainty, offering a 'No-Regret' strategy to various possibilities of sea level rise.

Community input was sought to refine the initial concept design and to understand key community priorities before undertaking the detailed design as presented in this paper. The composite design is well suited to a staged implementation program that allows for construction over several funding cycles.

Introduction

Flynns Beach is situated at Tuppenny Lane, Port Macquarie, on the mid-north coast of NSW, approximately 380 km north of Sydney (**Figure 1**). The existing retaining wall was constructed from prestressed concrete panels in the 1970s extending approximately 260 m along the foreshore of Flynns Beach. Despite serving an effective function retaining 600 mm to 1800 mm of fill for over 30 years, it was not designed as a seawall and is now in a poor condition. A number of tie back repairs and rock toe stabilisation works (comprising rocks of 2–3 tonnes) have been undertaken to the existing retaining wall by Port Macquarie Hastings Council following instability caused by scour from rainfall runoff and/or beach erosion undermining the retaining wall foundations. Prestressed concrete panels and supporting concrete piles have reached their design life and are suffering from weathering (concrete cancer) with further failures evident (**Figure 1**, insert A). A new seawall is required to safeguard proposed foreshore enhancements and to protect against coastal erosion and expected sea level rise.

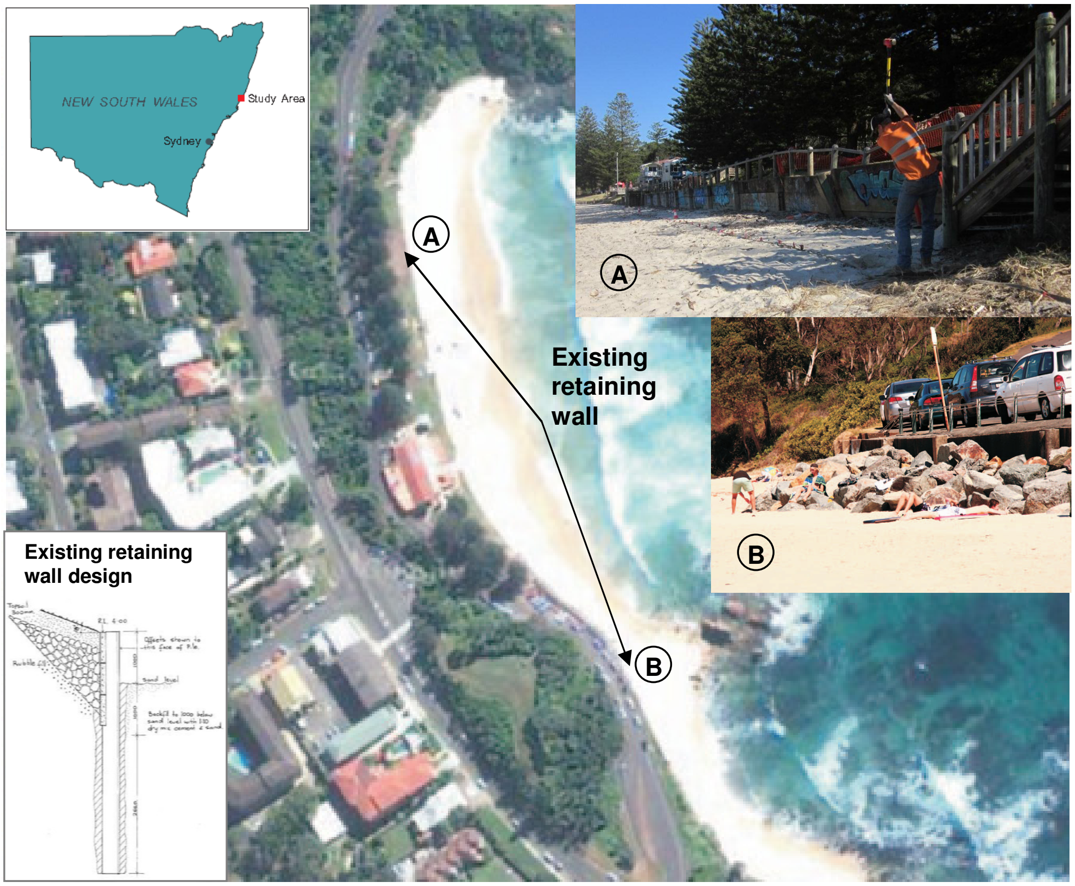

Figure 1: Flynns Beach Existing Retaining Wall, Port Macquarie, NSW

Adopting conventional coastal engineering practice, the new seawall would require a crest level 1.0 to 1.5 metres higher than the existing wall. This would significantly detract from the existing character and serviceability of the beach and surrounding amenities, being twice the height of the existing retaining wall at some locations.

Following development of three different concept design options by three independent professional consultants for community consultation (MHL, 2013b), NSW Public Works' Manly Hydraulics Laboratory (MHL) was engaged by Council to develop the detailed design of the preferred option, providing a versatile solution to meet community needs by being adaptable to future climate uncertainty, offering a 'No Regrets' strategy to various possibilities of sea level rise. The preferred design comprises a concrete stepped seawall integrated with the existing boat ramp and wheelchair access, partially buried rock seawalls at the northern and southern ends and a vertical precast concrete panel seawall. All concrete surfaces are to be integrally coloured to match the local beach sand. Other features of the new seawall include seating and beach access in front of the surf club, enhanced wave energy dissipation along the car park areas and maximised beach and parkland amenity in other areas.

Design Objectives

The following design objectives were developed through community consultation and relevant coastal engineering practice:

- The new seawall should be in keeping with current seawall design practices and offer appropriate scour protection (after CERC 2006).
- The design shall be in accordance with 'Environmentally Friendly Seawalls – A Guide to Improving the Environmental Value of Seawalls and Seawall-Lined Foreshores in Estuaries' (DECC, 1990). The design will also comply with 'Engineering Standards and Guidelines for Maritime Structures' code (NSW Maritime 2005) and AS 4997-2005 Guideline for the Design of Maritime Structures.
- The design should aim to protect the grassed terrace and buildings from potential coastal erosion and projected sea level rises of up to 450 mm over the next 35 years.
- The design should be sensitive to the maintenance of the pristine beach area as well as the protection of the grassed terrace, picnic facilities and buildings including the surf club and cafe which overlook the beach.
- The proposed solution should not unnecessarily reduce the limited beach area, or threaten the Norfolk Island pines or the facilities which have been constructed on the grassed area.
- The design should include both traditional and innovative (composite) approaches.
- The design should allow matching of all existing infrastructure such as footpaths, lighting, kerbs, gutters, car parking, picnic areas and access ways.

The new seawall design should also give consideration to:

- minimising any trip hazards on top of the seawall;
- dissipating wave action to prevent scouring and loss of sand at the toe of the seawall;
- preventing ponding of water on the park in close proximity to the seawall;
- making allowance for stormwater outlets to the seawall; and
- not being visually intrusive or unnecessarily limit public access to the foreshore.

Summary of Relevant Coastal Processes

Flynns Beach is an embayed sandy beach located between two rocky headlands (**Figure 1**) that faces east-north-east and hence it is somewhat protected from the dominant offshore wave direction from the south-south-east (**Figure 2**) in comparison with open ocean beaches on the NSW mid-north coast. Rocky features are present throughout the foreshore area, with a rocky outcrop in the centre of the embayment creating a small salient feature towards the south of the beach. More extensive and shallow rock reefs exist offshore of the beach further south. Flynns Beach is further protected by the offshore reef system that helps to dissipate incident storm waves (through breaking) and acts to perch the beach to create a relatively flat and further dissipative surf zone profile with slopes of 1:100 to 1:150.

The surf club was located at Flynns Beach in 1929 in preference to adjacent beaches because the beach provides generally safe surf conditions, has less rock and more sand along its length and has good vehicle access, with a beachfront park and car parking along Tuppenny Lane. The existing retaining wall constructed in the 1970s is located above the typical high tide level and hence has no significant adverse impacts on the dominant coastal processes.

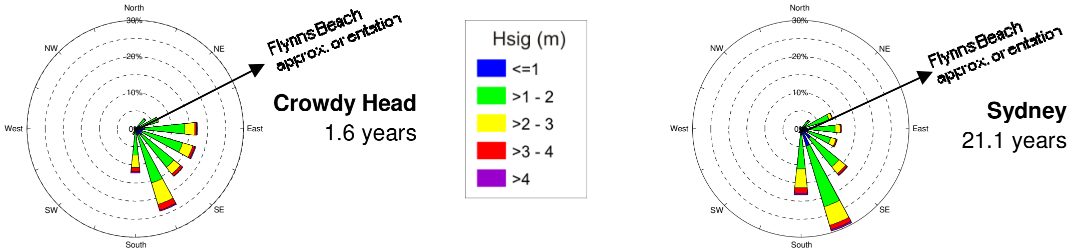

Figure 2: OEH/MHL Offshore Significant Wave Height and Wave Direction (after Kulmar et.al, 2013)

Notwithstanding the protected and dissipative characteristics of Flynns Beach, elevated ocean water levels and storm waves can cause beach erosion, transporting beach sands offshore to further widen the surf zone towards an equilibrium storm profile. Coastal storms can erode the back of the beach (adjacent to the retaining wall) by typically 1 metre and by more than 2 metres during major coastal storms such as in 1974, representing a cross-shore storm demand (or storm bite) of some 20–50 m^3/m. This eroded sand typically returns to the beach during milder wave conditions with no evidence of any significant permanent offshore loss of sand over the reef system occurring in the last 30 to 40 years based on historical aerial photogrammetry analysis (MHL, 2013b) and anecdotal accounts. It is noted, however, that with expected future sea level rise, the incidence of wave energy onto the seawall would become more regular (during typical high tides) and this would increase local beach scour with consequentially reduced beach width if this were not mitigated by a future sand nourishment program to maintain the beach amenity. It is further noted that sand nourishment would be required to maintain the beach amenity in response to sea level rise irrespective of the presence or not of a seawall at the back of the beach due to the limited sandy foreshore area which is naturally constrained by the geology of Flynns Beach.

Sediments on the shoreline are generally mobilised to align the shoreline with the approaching wavefronts. Longshore drift or alongshore transportation of sediments is generated when there is a misalignment between the approaching wave fronts and the shoreline. As a consequence of the dominant incident wave energy direction from the south-south-east (**Figure 2**), the longshore sand transport within the Flynns Beach embayment is estimated to be between 20,000 m^3 per year and 50,000 m^3 per year towards the north (based on measured quantities at similar mid-north coast beaches), with the majority of longshore transport taking place episodically during major coastal storms. Little or no net loss of sand is evident within the embayment during the last 30 to 40 years, however, with incoming and outgoing sand volumes generally matching based on the shoreline position from historical aerial photogrammetry analysis (MHL, 2013b) and anecdotal accounts. Notwithstanding a generally balanced sediment budget indicated for Flynns Beach (with no evidence of long-term beach recession), changes to the subaerial beach width from north to south and vice versa are observed to occur in response to interdecadal variability in the dominant incident wave energy direction (due to ENSO) causing some rotation of the beach alignment. As with cross-shore sediment transport, future beach nourishment may be required to maintain an adequate beach amenity in response to any permanent longer-term change to the dominant incident wave energy direction resulting from Climate Change (CSIRO, 2007).

The relevant coastal processes for design of the new seawall at Flynns Beach are further considered in subsequent Sections.

Design Parameters

Preamble

Design parameters for the new seawall include ocean wave and water level conditions, expected beach scour at the toe of the seawall, and local geotechnical conditions. These parameters inform a range of design decisions that are discussed below. The design of the new seawall is based on a nominal design life of 35 years with design parameters primarily associated with a 35 years Average Recurrence Interval (ARI). Recognising, however, that more extreme conditions can occur within the design life, the serviceability and maintenance requirements have been assessed also for more extreme events including design variables associated with 50-years and 100-years ARI.

Details on the local geotechnical conditions at the site are important in determining an adequate foundation design for the seawall structure. The expected level of beach scour determines the depth below the existing surface level to which the seawall must penetrate in order to prevent undermining of the structure. The design toe scour level will also determine the maximum depth of water possible at the toe of the seawall. The level of toe scour expected, together with the design water level, determines the maximum depth-limited breaking wave that can impact on the seawall. Design wave and water levels at the structure affect the design of the rock armour and stability of the structure, determine wave forces on the vertical wall sections, and affect the hydraulic performance of the seawall with regard to wave runup and wave overtopping (defining the necessary seawall crest levels).

Given the importance of the depth-limited design wave conditions for the seawall design, the adopted design water level and in particular the design allowance for projected sea level rise will have a significant effect on the seawall dimensions and resulting cost. Adopting conventional coastal engineering practice with an upper end projection of sea level rise, the new seawall would require a crest level 1.0 m to 1.5 m higher than the existing wall. This would significantly detract from the existing character and serviceability of the beach and surrounding amenities and incur potentially unnecessary cost should these upper end projections not materialise. The proposed design philosophy takes into consideration the adaptable nature of the proposed design to offer a 'No Regrets' strategy that caters for climate uncertainty and considers adaptation strategies that cater for various sea level rise possibilities.

Allowance for Local Sea Level Rise

Most people associate climate change with a man-made (anthropogenic) cause, but there is compelling evidence that the earth's climate is in a constant state of change. The relatively slow rate of change and strong yearly as well as inter-decadal variations in climate means that climate change can go unnoticed in a lifetime.

All data and observations indicate sea levels are rising, whether man-induced, natural or a combination of both. It is not clear whether sea level rise is accelerating as projected by models and implied by recent satellite observations as reliable sea level records are too short to validate models, and inter-annual/inter-decadal variability can easily mask or bias implied long-term trends. Despite great uncertainty with respect to future predictions, Councils and other agencies cannot ignore sea level rise to demonstrate 'good faith' in managing their liability on the coast – the challenge is to what extent and how to do this effectively without creating false liabilities should projections not materialise within the allotted planning horizons.

The rates of sea level variability measured along the NSW coast over the past 20 years are consistent with expected IPO cycle influences and are consistent with longer-term records from Fort Denison and satellite observations indicating sea level rates of rise of between 1 mm/year and 3 mm/year (MHL, 2013a). The previous planning benchmarks from NSW Government (2009) have been set aside (despite the NSW Chief Scientist and Engineer stating the science was reasonable) to allow Councils to adopt the best available knowledge for their local conditions. The previous benchmarks comprising a sea level rise of 40 cm by 2050 and 90 cm by 2100 were selected to represent upper bound projections because trends since the year 2000 indicated global emissions will exceed the highest IPCC (2007) scenarios.

New coastal structures need to be designed either to cope with the potential wave conditions expected in their economic lifetime or, more reasonably given that the future is uncertain, designed so they can be readily adapted. A risk-based approach is warranted to deal with the inherent uncertainty of climate change and sea level rise projections. This further underlines the importance of ongoing monitoring to improve projections and measure the effectiveness of planning decisions and signal triggers to adapt. Depending on the level of risk, upper bound projections of sea level rise should be considered in designing effective adaptation measures, and where adaptation is impracticable, upper bound projections should be adopted.

Moving beyond simply a risk-based approach as suggested in AS5334-2013, assessment of adaptation options should further consider the principles described by Hallegatte (2009) comprising in priority order:

(i) selecting 'No-Regret' strategies that yield benefits whether or not Climate Change or sea level rise takes place;
(ii) favouring reversible and flexible options;
(iii) buying 'Safety Margins' in new investments;
(iv) promoting soft adaptation strategies, including long-term potential; and
(v) reducing decision time horizons (or effective asset life).

In the case of Flynns Beach, the new seawall will be constructed for present conditions, with adaptive measures built into the design so they are of negligible cost and easy to implement without need for redesign and have well defined trigger conditions. Coupled with potential future sand nourishment to preserve beach amenity in response to sea level rise, the adaptive seawall design satisfies all of the above criteria.

Geotechnical Conditions

The geotechnical conditions at the seawall are an important factor in the structure's foundation design and may affect other design parameters such as the depth of scour possible along the length of the seawall. The presence and/or depth of bedrock below the surface as illustrated in **Figure 3** from investigations undertaken by Coffey (2013) may present challenges in construction of piled structures, while the bearing capacity of the subsurface material may also affect post-construction settlement of heavier structures. Coffey (2013) describes Flynns Beach as comprising marine and aeolian sand, with boreholes indicating that clay derived from serpentinite is present near the surf club at around 1.1 m below the surface. Regarding piling conditions, the interpretations indicate that piles of around 6 m to 8 m could be achieved near the central section of the beach; however piles outside these areas are unlikely to provide a viable means of cantilever support for the seawall. Rock comprising highly weathered serpentinite can be found at relatively shallow depth at both the northern and southern extents of the beach, limiting the ability to use deep piles as foundations, however, these conditions favourably limit the extent of toe scour and provide a solid foundation for the rubble mound structures to be used in these areas. Combined with the existing foreshore layout and community amenity values, the geotechnical conditions drive the practicability of the seawall design.

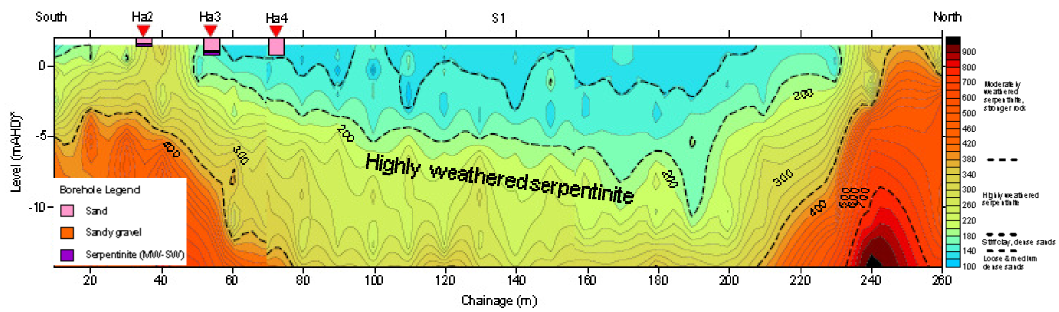

Figure 3: Flynns Beach S-wave seismic velocity results

Design Scour Levels

During storm conditions beaches scour to levels below the normal beach level. The depth of scour is an important parameter as this depth governs the height of breaking waves possible at the back of the beach, sand escarpment or against a nearshore structure. The height of the maximum waves that can reach the shore affects the stability of coastal structures and resulting wave runup levels and wave overtopping rates. In NSW, a design scour level of 0.0 m to -1.0 m relative to Australian Height Datum (AHD) is commonly adopted for rigid coastal structures located at the back of an open ocean beach area, dependent on the degree of storm wave exposure of the beach. These scour levels are based on stratigraphic evidence of historical scour levels and observed scour levels occurring during major storms in front of seawalls along the NSW coast (Nielsen et.al., 1992; Foster et.al., 1975).

Except where bedrock further limits beach scour, a design scour level of -0.5 m AHD has been adopted for Flynns Beach on the basis that the beach is somewhat protected by its orientation and offshore reef systems creating a relatively flat, perched beach profile. Anecdotal observations at Flynns Beach over the past 30 to 40 years have not witnessed toe scour levels below approximately +0.5 m AHD along the existing retaining wall, indicating the adopted design scour level to be conservative. Nevertheless, all structural elements have been designed with toe scour protection down to at least -1.0 m AHD as a safety measure.

Design Water Levels

The water depth at the seawall affects both the wave energy that reaches the structure and the rate of wave overtopping, so water levels and the design scour level are key design parameters. Water levels during a storm are the result of the regular tidal regime, storm-induced wind and wave setup and other tidal anomalies that may raise or lower the water level. Further, long period changes to the water level, such as sea level rise, will have an impact on the seawall design performance that should be appropriately considered in the design.

A detailed review of water levels across NSW coastal tide gauges was undertaken by MHL (2013c) which includes extreme value analysis of the gauge data to produce design water levels of difference exceedance probabilities. The Port Macquarie tide gauge, located within the river entrance, is considered the most representative tide gauge for Flynns Beach. Extreme value analysis results as reported by MHL (2013c) have been adopted for design (see **Table 1**). The Port Macquarie gauge is not materially affected by wave setup and this must be separately added to the stillwater levels to derive the design water levels for the seawall design. Wave setup is dependent on a number of hydraulic and geometric factors, the most important of these being the incident wave height. Based on field measurements during major storms, it is generally accepted that wave setup on the NSW open coast is 10% to 15% of the offshore significant wave height (H_s). A conservative value of 15% of H_s was adopted to determine wave setup (see **Table 1**) based on the offshore wave climate as described below.

Design Wave Conditions

Although the design wave height for the Flynns Beach seawall is depth-limited, the offshore wave climate is important to determine wave setup, which affects the design water depth. Further, the design breaking wave height at the structure is affected also by the wave period that is characterised also by the offshore wave conditions.

Deepwater offshore wave data is collected by MHL for the NSW Office of Environment and Heritage (OEH) to provide essential input to design, construction and performance monitoring of coastal zone projects in NSW. Offshore wave data statistics from the Crowdy Head Waverider buoy (**Figure 4**) located some 45 km south of Flynns Beach has been used to determine design conditions and wave setup. The buoy was first deployed in 1985 providing wave statistics from a record of 28 years, with wave directional data since 2011 (**Figure 2**).

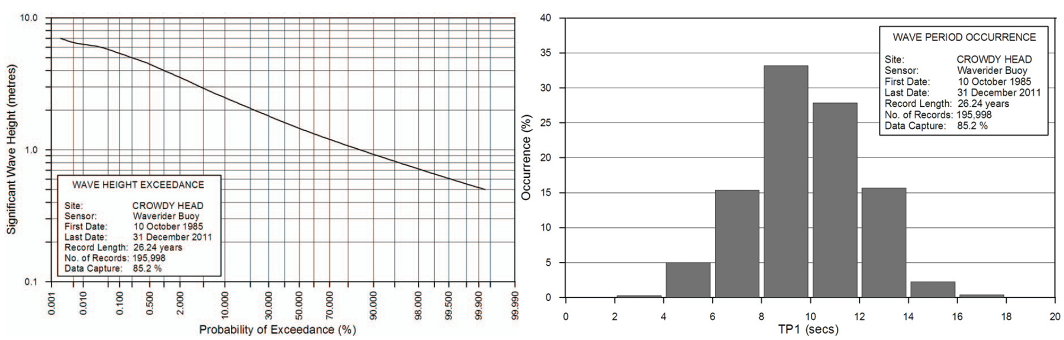

Figure 4: Offshore Significant Wave Height (H_s) Exceedance and Distribution of Peak Spectral Wave Period, OEH/MHL Crowdy Head Waverider Buoy

The location where waves break in the surf zone is dependent on wave heights, wave period and water depth. Elevated water levels during coastal storms allow larger waves to reach further inshore before breaking, with the largest wave that may affect a seawall being determined by the nearshore bed slope, water depth in front of the seawall and the offshore wave characteristics.

The relatively flat nearshore slope of Flynns Beach results in the largest storm waves breaking offshore of the proposed seawall, creating depth-limited design wave conditions at the structure. Design breaking wave heights at the seawall for each nominal return period as shown in **Table 1** were calculated from the offshore wave data, design water levels and a scour depth to -0.5 m AHD using the empirical formulations given by LeMehaute and Koh (1967) and Singamsetti and Wind (1980).

Summary of Design Parameters

A summary of the key design parameters adopted for Flynns Beach to determine the seawall crest height, level to which toe protection is required and rock armour size are shown in **Table 1**. The design of the seawall elements is primarily based on the parameters associated with a 35 years ARI, with serviceability, maintenance and adaptation considerations given to the impacts of more extreme events and the impacts of sea level rise (SLR).

Table 1: Adopted Seawall Design Parameters

Design Parameter	35-year ARI	50-year ARI	100-year ARI
No SLR allowance			
Still Water Level (m AHD)	1.39	1.41	1.45
Wave Setup (m)	1.10	1.13	1.20
Design Water Level (m AHD)	2.49	2.54	2.65
Nearshore Design Water Depth* (m)	2.99	3.04	3.15
→Design Breaking Wave Height (m)	2.01	2.04	2.18
450 mm SLR allowance			
Design Water Level + 450 mm SLR (m AHD)	2.94	2.99	3.10
→Design Breaking Wave Height (m)	2.31	2.34	2.49
900 mm SLR allowance	3.39	3.44	3.55
Design Water Level + 900 mm SLR (m AHD)			
→Design Breaking Wave Height (m)	2.62	2.64	2.81

* with -0.5 m AHD design toe scour level.

It should be noted that the adopted design parameters assume coincidence of design water levels and offshore wave conditions. This is a conservative assumption as the joint probability of extreme water level and wave height occurrence is lower than the probability of occurrence for either of the individual conditions occurring independently. A degree of correlation does exist between extreme water levels (with the exception of tidal components) and wave heights as both can result from a single storm system and so it is appropriate to consider these two conditions in combination. Furthermore, the coincident design assumption only has a secondary effect on the design waves due to the depth-limited condition at the structure.

Seawall Makeup, Crest Levels and Alignment

Figure 5 shows the proposed new seawall crest levels, alignment and footprint of the main seawall elements from south to north comprising:

Type B1 – rock rubble mound armour with crest level at 4.2 m AHD;

Type C2 – vertical concrete panel and tied steel piles with crest level at 4.0 m AHD;

Type A1 – concrete steps with buried sheet pile toe and crest level at 6.3 m AHD;

Type A2 – concrete steps with buried sheet pile toe and crest level at 4.5 m AHD;

Type C1 – vertical concrete panel and steel piles with crest level at 4.5 m AHD; and

Type B2 – rock rubble mound armour with crest level at 4.5 m to 3.5 m AHD.

The new seawall will be located within 0.5 m (seaward) of the existing retaining wall to maximise the beach amenity while maintaining highly valued parkland areas. The proposed new seawall crest levels have been determined to meet serviceability requirements (wave overtopping) and to fit in with existing ground and landscape levels. The new seawall also includes features that further improve the foreshore amenity which is highly valued by the community. The overall design philosophy adopted caters for 35 years ARI design conditions excluding allowance for sea level rise with an assessment of seawall performance for 50 years and 100 years ARI conditions and adaptation strategies developed where necessary to cater for upper bound sea level rise projections.

Figure 5: Proposed New Flynns Beach Seawall Alignment and Makeup

The performance of the proposed new seawall design under different design conditions has been assessed in terms of tolerable wave overtopping limits as described in the EuroTop Overtopping Manual which is summarised in **Table 2**.

Table 2: Wave Overtopping Limits

Hazard Type and Reason	Mean Discharge (L/s/m)
Damage to paved or armoured promenade behind seawall	200
Damage to grassed or lightly protected promenade or reclamation cover	50
Vehicles – driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicles not immersed	10-50
Pedestrians – trained staff, well shod and protected, expecting to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway	1-10
Pedestrians – aware pedestrian, clear view of the sea, not easily upset or frightened, able to tolerate getting wet, wider walkway	0.1

Estimated wave overtopping rates for the different sections of the proposed new seawall are summarised in **Table 3** to **Table 7** inclusively. Wave overtopping rate estimates for the 4.5 m crest stepped seawall (Type A2; **Table 4**) are compared with a 5.3 m crest level stepped seawall that incorporates an adaptation strategy comprising a 0.8 m vertical back wall to cater for possible sea level rise projections as shown in

Table 8. **Table 5** is indicative of expected reductions in wave overtopping for the vertical concrete panel and anchored steel pile seawall with a crest level of 4.0 m AHD (Type C2; **Table 6**) when adapted by adding an additional 0.5 m high concrete panel to cater for upper bound sea level rise projections. Estimated wave overtopping rates should be tested using physical modelling as part of the proposed adaptation strategy design validation, particularly for the stepped seawall where no directly similar measurements are available.

Table 3: Wave Overtopping Rates Stepped Seawall (Type A1; 6.3 m AHD Crest)

6.3 m AHD Crest Height	35-year ARI	50-year ARI	100-year ARI
Overtopping rate (L/m/s)	4	4	16
Overtopping rate including 450 mm SLR (L/m/s)	16	17	50
Overtopping rate including 900 mm SLR (L/m/s)	48	50	126

Table 4: Wave Overtopping Rates Stepped Seawall (Type A2; 4.5 m AHD Crest)

4.5 m AHD Crest Height	35-year ARI	50-year ARI	100-year ARI
Overtopping rate (L/m/s)	42	44	109
Overtopping rate including 450 mm SLR (L/m/s)	133	137	290
Overtopping rate including 900 mm SLR (L/m/s)	333	343	635

Table 5: Wave Overtopping Rates Vertical Seawall (Type C1; 4.5 m AHD Crest)

4.5m AHD Crest Height	35-year ARI	50-year ARI	100-year ARI
Overtopping rate (L/m/s)	10	11	19
Overtopping rate including 450 mm SLR (L/m/s)	35	37	66
Overtopping rate including 900 mm SLR (L/m/s)	117	121	216

Table 6: Wave Overtopping Rates Vertical Seawall (Type C2; 4.0 m AHD Crest)

4.0 m AHD Crest Height	35-year ARI	50-year ARI	100-year ARI
Overtopping rate (L/m/s)	18	19	34
Overtopping rate including 450 mm SLR (L/m/s)	68	71	127
Overtopping rate including 900 mm SLR (L/m/s)	258	270	491

Table 7: Wave Overtopping Rates Rouble Mound Seawall (Type B1; 4.2 m AHD Crest)

4.2 m AHD Crest Height	35-year ARI	50-year ARI	100-year ARI
Overtopping rate (L/m/s)	0.1	0.1	0.2
Overtopping rate including 450 mm SLR (L/m/s)	2.4	2.6	4.1
Overtopping rate including 900 mm SLR (L/m/s)	23	25	35

Table 8: Wave Overtopping Rates Stepped Seawall with 0.8 m Wave Return Wall for Sea Level Rise Adaptation (Type A2; 5.3 m AHD Crest)

5.3 m AHD Crest Height	35-year ARI	50-year ARI	100-year ARI
Overtopping rate (L/m/s)	15	16	46
Overtopping rate including 450 mm SLR (L/m/s)	52	54	133
Overtopping rate including 900 mm SLR (L/m/s)	141	145	310

Tuppenny Lane at the southern end of the beach is to be protected by a 3.0 tonne rubble mound rock armoured seawall with a crest level of 4.2 m AHD (Type B1; **Figure 5**). This will comprise rock armour of a similar size to the existing ad hoc rubble mound rock wall that is currently mitigating undermining of the existing concrete retaining wall, but with a raised crest level to reduce wave overtopping to acceptable levels. The road-seawall interface incorporates a dish drain with appropriate capability for expected wave overtopping rates.

The existing retaining wall crest level south of the clubhouse is too low for coastal protection purposes. A slightly raised vertical concrete panel and steel pile seawall is proposed with a minimum crest level of 4.0 m AHD (Type C2). The 4.0 m crest is assessed to be adequate to tolerate minor wave overtopping as expected under present sea level conditions. This section of seawall can be raised to 4.5 m AHD (or higher) by dropping in additional panels between the steel piles should sea level rise projections eventuate.

An emergency vehicle access ramp is proposed between the rock rubble mound section (Type B1) and the vertical panel seawall (Type C2) to allow access to the beach from Tuppenny Lane. The ramp curves around the southernmost Norfolk Island pine and provides two disabled parking spaces. The new ramp meets the guideline slopes for accessibility (AS1428.1-2009) providing direct access from the disabled parking bays to both the beach and clubhouse area.

A signature feature of the proposed design is the stepped seawall in front of the clubhouse (Type A1), providing a broad seating area, access to the beach, and an attractive frontage with a wider step promenade that continues north past the lifeguard tower and becomes the seawall crest (Type A2).

A new vertical concrete panel and steel pile seawall with a crest level 4.5 m AHD (Type C1) commences after the promenade at a similar level to the existing retaining wall. Tieback anchors are utilised for some piles where bedrock is too shallow to achieve the necessary cantilevered embedment depth towards the northern end.

At the very northern end of the seawall, bedrock is too shallow for a piled seawall (**Figure 3**) and a buried rubble mound rock structure (Type B3) is proposed based on practicable and economic grounds. This structure comprises 3.0 tonne rock armour diminishing through a three-stage transition to 0.5 tonnes at the northernmost end where bedrock and sheltering reduce the design breaking wave height. This area is currently covered with light vegetation that would be restored by covering the seawall with sand after construction to act as a terminal protective structure.

Key Outcomes

The adaptive design saves on initial capital cost while allowing future augmentation to cater for different levels of possible sea level rise without spending unnecessary funds should sea level rise not progress as far as currently projected. The composite design maximises the utility and amenity of this highly valued area, being well suited also to a staged construction program that matches Council's budget over two financial years. Where adaptive design solutions cannot be provided, conventional coastal design warrants upper bound sea level rise projections to be adopted in design (a precautionary principle). A planned adaptive approach as proposed for the new Flynns Beach seawall offers vastly improved serviceability with significant cost savings.

It is noted that additional landscaping will be required as part of the adaptation strategies that Council is developing. It is further noted that sand nourishment comprising importation of suitable beach sand may be required to maintain the subaerial beach amenity if sea level rise projections eventuate.

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