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The Design Process for a Seawall at Kingscliff Beach: A Life-Cycle Cost Case Study for an Integrated Coastal Protection Asset

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ABSTRACT: *The most common historical approach to mitigate the hazard of erosion in Australia has been to construct a seawall to protect the foreshore. This paper presents two designs for a seawall at Kingscliff Beach, NSW. The designs include consideration of the uncertainty surrounding future beach width and allowance for future adaptation to sea level rise (SLR). During the design process, selection of the expected beach scour level at the toe was one of the most influential decisions affecting overall geometry. This level was heavily dependent on whether or not a commitment can be made to a sustained beach nourishment program to offset the projected effects of climate change. The first design was modest but reliant on ongoing beach nourishment; the second was more robust without the need for nourishment. Selection of the design crest elevation was also important for prevention of failure of the seawall and the safety of pedestrians, vehicles and buildings which may be impacted by wave overtopping. To prevent negative impacts on beach amenity (views and sea breezes) associated with single stage construction with allowance for future SLR, an adaptive response through a two stage construction design was adopted. Stage 1 will consist of the bulk of the proposed works. Stage 2 will be deferred until the extent of measured SLR exceeds a trigger value and will consist of raising the seawall crest at that time. While the two designs were specifically developed for Kingscliff Beach, the approach taken is applicable to many coastal infrastructure developments.*

KEYWORDS: seawall, revetment, nourishment, erosion, sea level rise, adaptation, wave overtopping.

1 Introduction

On an open coastline, many options exist to adapt to the hazards of erosion and recession. The most common historical approach to mitigate the hazard of erosion in Australia has been to construct a seawall to protect the foreshore. During the design process, the most important decision affecting the overall geometry of a seawall may be whether or not a commitment can be made to a sustained beach nourishment program. This paper presents two designs for a seawall at Kingscliff Beach which consider the uncertainty surrounding future beach width and allowance for future adaptation to sea level rise (SLR).

2 Background Information

Kingscliff Beach, located at the southern end of Wommin Bay on the far north coast of NSW (Figure 1), is a section of the Tweed coastline with built assets at immediate risk from coastal hazards. Ongoing erosion in the last few years has resulted in substantial loss of beach amenity and community land. Storm erosion episodes between 2009 and 2012 severely impacted the Kingscliff Beach Holiday Park (KBHP). This section is also affected by moderate ongoing underlying shoreline recession [1].

Figure 1: Location (Aerial Photo 23/06/2008)

To manage the Kingscliff Beach foreshore (Figure 2) in the longer term, Tweed Shire Council (TSC) is considering a combination of several of the following options:

- undertaking various beach works – dune reconstruction and vegetation, fencing, access-ways and stormwater management;
- undertaking beach nourishment between the northern end of Kingscliff Beach Bowls Club (KBBC) and the northern training wall of Cudgen Creek;
- construction of a 420 m long terminal seawall between an existing rock seawall protecting KBBC to the north and an existing secant pile seawall at Cudgen Headland Surf Life Saving Club (CHSLSC) to the south;
- construction of a groyne field between the northern end of KBBC and the northern training wall of Cudgen Creek; and/or
- planned retreat.

WRL was engaged by TSC to prepare two different designs for a long term terminal seawall at Kingscliff Beach. The first terminal seawall design assumed that erosion protection would be provided by ongoing beach nourishment in conjunction with the seawall. The second design assumed that complete erosion protection would be provided by a terminal seawall fronting KBHP without the requirement for beach nourishment.

3 Considerations for an Integrated Coastal Protection Asset

When considering the implementation of integrated foreshore protection works (i.e. beach nourishment in conjunction with a terminal seawall), it is critical to appreciate their interdependence. The location and geometry of a seawall influences the extent (volume and frequency) of beach nourishment required to maintain its structural integrity.

For a given location on a beach system, the expected beach scour level (the seawall design scour level) determines the required penetration of the structure to prevent undermining and a range of potential geotechnical failure modes. The water depth at the seawall (which is a function of the bed elevation and the design water level) determines the maximum depth limited breaking wave height that can reach the structure. The design wave and water level conditions at the structure affect the hydraulic performance (wave runup and overtopping) and stability of the structure (required armour size) which have a direct effect on the capital and maintenance costs.

Beach nourishment may be used to raise the expected beach scour level, reduce the overall geometry of a seawall accordingly and offset the projected effects of climate change.

Figure 2: Site Details (Aerial Photo 21/07/2011)

4 Seawall Design Process

4.1 Planning Horizon

Establishing the design working life of the proposed coastal protection works was critical for determination of subsequent design parameters. A nominal design life of 50 years was adopted for the long term terminal seawall and beach nourishment. A further consideration is that the maximum significant wave height that can reach the seawall is a function of design water level due to depth limited wave conditions. The 100 year ARI event was selected for both wave conditions (height, period and direction) and water level conditions (tide plus anomaly).

4.2 Design Scour Level

The design scour level for a seawall is a function of the location of the structure on the beach system, the design storm erosion demand, recession (ongoing and due to sea level rise), sediment strata of the beach system and the slope and porosity of the structure. However, in NSW, a scour level of approximately -1 m AHD is commonly adopted as an engineering rule of thumb for coastal structures located at the back of an active beach area without detailed consideration of site specific factors. Despite considerable research into the processes responsible for wave induced scour, there are no generally accepted empirical techniques for estimating maximum scour depth for structures which are located inside the surf zone and high up the beach. A new methodology for evaluating the beach scour level for seawalls following a storm event(s) equivalent to the design storm erosion demand was developed by WRL and applied at Kingscliff Beach.

To confirm the design scour level for each terminal seawall design, WRL undertook two-dimensional SBEACH modelling of beach erosion with preliminary design scour levels of -1 and -2 m AHD. The SBEACH model is a two-dimensional numerical cross-shore sediment transport and profile change model developed by the United States Army Corps of Engineers [2 and 3]. SBEACH considers sand grain size, the pre-storm beach profile and dune height, plus time series of wave height, wave period and water level in calculating a post-storm beach profile. The process for confirming the design scour level for each structure is shown diagrammatically in Figure 3 and outlined in the following discourse.

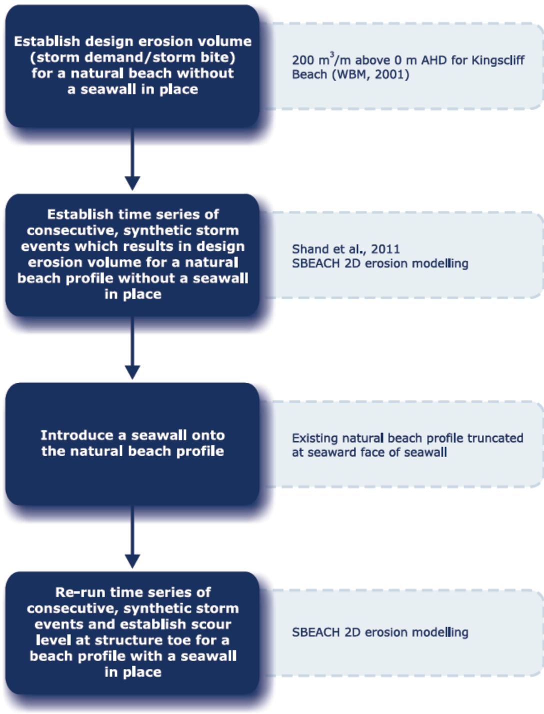

Figure 3: Design Scour Level Process Chart

Firstly, the design erosion volume (storm demand/storm bite) for Kingscliff Beach without a structure in place was established for the 100 year ARI of 200 m^3/m above AHD [1]. Secondly, a time series of consecutive, synthetic storm events [4] was applied in SBEACH without a seawall in place until the change in dune volume matched the adopted storm demand. Thirdly, a seawall was introduced such that erosion of the dune was prevented. Finally, the time series of storm events (which resulted in the adopted storm demand without a seawall) was modelled in SBEACH with a seawall to estimate the scour level at the toe of each seawall design. This process was applied for present day conditions and at the end of the design working life (50 years). Projected SLR was accounted for by increasing still water levels and moving the pre-storm beach profile upward and landward to account for recession due to SLR. The pre-storm profile was also receded further to account for ongoing underlying recession at Kingscliff Beach due to sediment loss.

The erosion modelling results indicated that the integrity of a terminal seawall with a design scour level of -1 m AHD would rely on ongoing beach nourishment to prevent undermining. Alternatively, for a 50 year design working life, ongoing beach nourishment is not required for a seawall with -2 m AHD design scour level.

4.3 Design Crest Level

Following establishment of the location of a seawall and its design scour level, the design crest level for a seawall is a function of the degree to which wave overtopping is tolerable. Each project will have specific considerations which may influence the design overtopping rate including:

- pedestrian safety;
- vehicle stability;
- damage to buildings located landward of the crest; and
- ultimate failure of the seawall through damage to paved surfaces behind the seawall.

To confirm the design crest level for each terminal seawall design, WRL estimated the average wave overtopping rates for various crest level options (4, 5, 6 and 7 m AHD) using the empirical techniques set out in the EurOtop "Overtopping Manual" [5] for terminal seawall designs 1 (1V:1.5H slope) and 2 (1V:2.2H slope). This process was undertaken for present day conditions and at the end of the design working life (50 years). The results of the analysis are shown in Table 1 (present day conditions) and Table 2 (at the end of the design working life).

Table 1: Wave Overtopping Rates for Various Crest Levels (100 year ARI at present day)

Crest Level (m AHD)	Mean Overtopping Rate (L/s/m) Design 1 (-1m AHD Toe)	Design 2 (-2m AHD Toe)
4.0	305	239
5.0	90	36
6.0	27	5
7.0	8	1

Table 2: Wave Overtopping Rates for Various Crest Levels (100 year ARI at working life end)

Crest Level (m AHD)	Mean Overtopping Rate (L/s/m) Design 1 (-1m AHD Toe)	Design 2 (-2m AHD Toe)
4.0	665	662
5.0	209	126
6.0	63	22
7.0	20	4

The analysis indicated that wave runup exceeds all crest levels for 100 year ARI conditions. That is, the proposed terminal seawall will be an overtopped structure under design conditions. Accordingly, a tolerable (design) wave overtopping rate must be established. Based on the threshold at which overtopping flows become hazardous to the area of land adjacent to a seawall (as set out in [5 and 6]), an average design overtopping rate of 200 L/s/m (litres per second per metre of seawall crest) was adopted. At this overtopping rate, damage to paved surfaces behind the seawall, which might ultimately lead to failure of the seawall, will be prevented.

The empirical wave overtopping estimates, suggest that a design crest level of 4 m AHD would result in unacceptable overtopping rates (i.e. > 200 L/s/m) under present and projected conditions. While a crest level of 5 m AHD would result in acceptable overtopping rates for the present day for both seawall designs, overtopping would be unacceptable by the end of the working life for design 1. Design crest levels of 6 or 7 m AHD would result in acceptable overtopping rates for both designs throughout their design working life.

Secondary considerations in the selection of the design crest level for a seawall include:

- the total volume of water expected to overtop and pond landward of a seawall during a design event;
- impacts on beach amenity (views to and sea breezes from the ocean);
- the existing crest level of the beach foredune; and
- the crest level(s) of adjacent seawalls interfacing with the proposed seawall.

The foreshore dune crest at the proposed location varies between 3.5 and 5.0 m AHD. The adjacent seawalls fronting the Kingscliff Beach Bowls Club and the Cudgen Headland Surf Life Saving Club have crest elevations of 6.0 and 4.4 m AHD, respectively.

Following consideration of the preceding information, the EurOtop wave overtopping estimates are recommended as the basis of the adopted design crest level. However, site specific physical modelling is recommended to obtain precise wave overtopping estimates. To minimise amenity impacts, it is proposed that the terminal seawall be constructed in two stages; Stage 1 will consist of the bulk of the proposed works (crest elevation +5.0 m AHD), Stage 2 will be deferred until the extent of measured sea level rise necessitates it and will consist of raising the crest with a wave return wall (crest elevation +6.0 m AHD).

4.4 Construction Materials

Terminal seawall designs were prepared from four different construction materials, as follows:

- Rock (greywacke or basalt);
- Sand-filled geotextile containers;
- Concrete (Seabee) armour units; and
- Stepped concrete.

Terminal seawall designs composed of rock are outlined in the following discourse. On the basis of hydraulic stability [7], sand-filled geotextile containers were found to be unsuitable for a terminal seawall if the design working life is 50 years. While the Seabee and stepped concrete designs remain valid options, they have not been included in this paper for brevity. The reader is directed to [8] for discussion of these two material designs.

Conventional rock armoured seawalls commonly referred to as rubble mound structures, comprise two layers of graded primary armour stones overlying another two layers of graded secondary armour stones. Rubble mound slopes of between 1V:1.5H and 1V:3.0H are typically used. Figures 4 and 5 show a typical conventional rock armoured structure acting as a terminal protective structure at Stockton Beach, NSW. Note that this seawall has been designed to allow significant wave overtopping and has a design scour level of -2 m AHD.

Rock for use in seawalls must have high durability and high density. Enquiries with several quarries indicated that there was an abundance of greywacke rock but that the supply of basalt was more limited. Consequently, greywacke was adopted as the primary armour material for design. However, it was not possible to economically acquire greywacke rock in sufficient quantities for construction with a median mass greater than 7.0 t.

Two design cross-sections for greywacke armour were prepared with design scour levels of -1 and -2 m AHD. For the design with a -1 m AHD scour level (Figure 6), a relatively steep slope of 1V:1.5H was adopted to minimise the footprint of the structure and its impact on the existing beach amenity. For the design with a -2 m AHD scour level (Figure 7), a flatter slope of 1V:2.2H was adopted as enquiries indicated that it was not possible to acquire greywacke rock of necessary median mass (and in sufficient quantities) to remain stable on a 1V:1.5H slope.

Figure 4: Rock Seawall Stockton Beach, NSW (Typical Pre-Storm Conditions)

Figure 5: Rock Seawall Stockton Beach, NSW (Typical Post-Storm Conditions)

The rock armour sizing was undertaken using several different empirical methods; Hudson (two formulations) and Van der Meer (deep and shallow water) [6, 9, 10 and 11]. The results of this analysis indicate that the required median armour mass for a design scour level -1 m AHD would be in the range of 1.8 to 4.9 t for initiation of damage to the seawall under design conditions. Similarly, for a scour level -2 m AHD, the required armour mass would be in the range of 2.3 to 8.0 t.

Following consideration of the four empirical techniques, WRL adopted M_{50} = 4.0 t (density ≈ 2650 kg/m^3, D_{n50} = 1.15 m) as the median mass of the primary armour for design 1. For design 2, WRL adopted M_{50} = 7.0 t (density ≈ 2650 kg/m^3, D_{n50} = 1.40 m) as the median mass. Either of these masses should be optimised with physical modelling as the empirical techniques used to derive them are generally very conservative. In WRL's experience, savings in required armour mass are typically of the order of 20 to 30% following a physical modelling program. Comparison of the footprints for designs 1 and 2 emphasises the impact on overall geometry if a beach nourishment program is sustained.

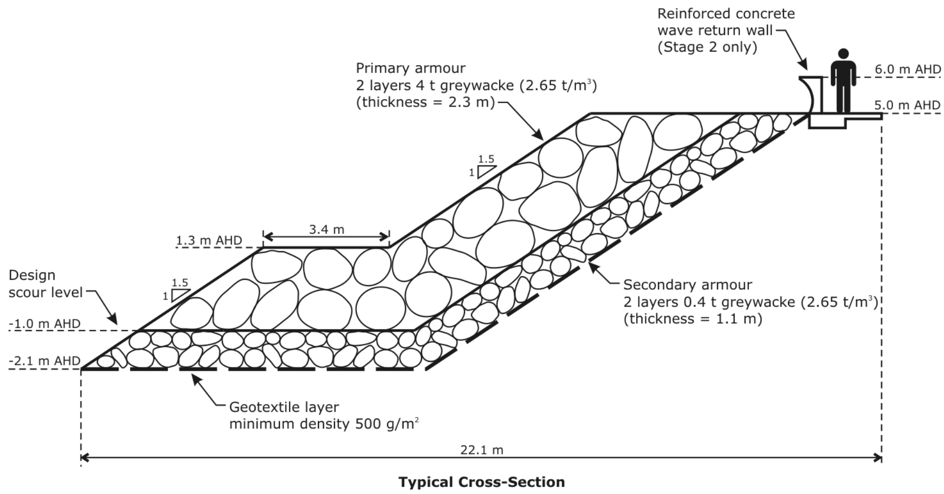

Figure 6: Rock Seawall Design 1 (Design Scour Level -1 m AHD, Reliant on Beach Nourishment)

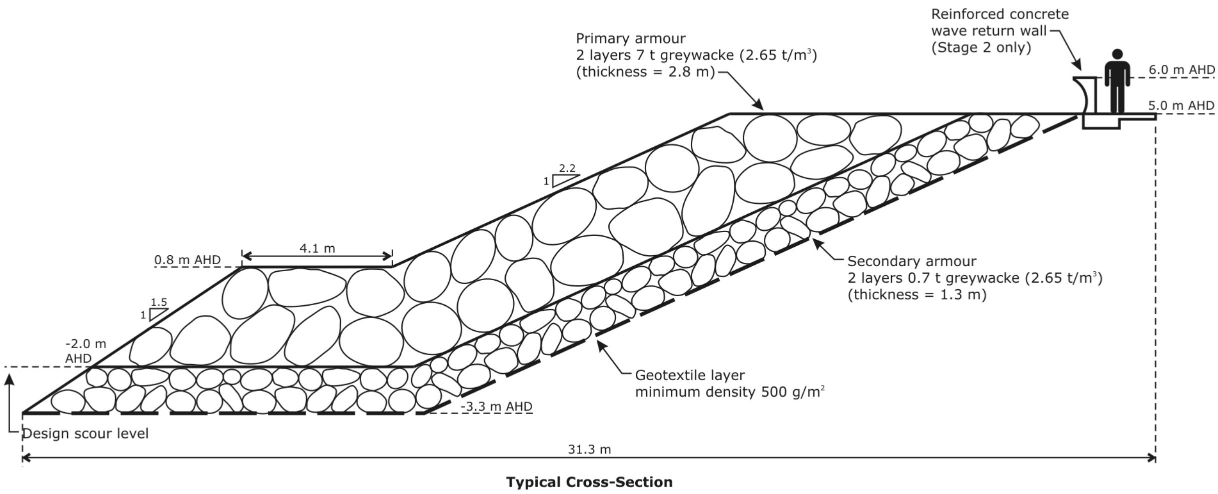

Figure 7: Rock Seawall Design 2 (Design Scour Level -2 m AHD, No Beach Nourishment)

5 Beach Nourishment

5.1 General

Beach nourishment involves the placement of large quantities of good quality sand on a beach to advance it seaward. WRL assessed the required extent of beach nourishment to maintain the beach above the design scour level for the first seawall design. For comparitive purposes, the extent of beach nourishment required to provide erosion protection without a terminal seawall was also assessed. It was assumed that Kingscliff Beach would be nourished between the northern end of Kingscliff Beach Bowls Club and the northern training wall of Cudgen Creek (total length 1,110 m). That is, nourishment would be undertaken directly offshore of the proposed seawall and in regions to the north and south of the seawall.

5.2 Environmental Conditions and Coastal Processes

As with the terminal seawall, a design working life of 50 years was adopted for the beach nourishment program. The two beach nourishment options were designed to provide protection against the 100 year ARI erosion event. Littoral drift transport at Kingscliff Beach is generally northward but occasionally southward [12]. Patterson [13] suggests that the net annual longshore sand transport at the southern end of Kingscliff Beach (Sutherland Point) is 518,000 m^3/year northward. The “best estimate” of the rate of ongoing underlying shoreline recession at Kingscliff Beach is 0.20 m/year [1]. A ”Bruun Factor” of 50 [1] and depth of closure elevation of -15 m AHD [14] were adopted by WRL. A median sediment particle size (d_{50}) of 0.30 mm was adopted as the median sand grain size based on previously collected sediment samples [14].

5.3 Design of Nourished Beach Profiles

A 660,000 m^3 reserve of sand in the Tweed River (Area 5) has been identified as being potentially suitable for nourishment of Kingscliff Beach [15]. Sediment sampling suggests that the sand from this location would be a suitable match for the existing sand at Kingscliff Beach in terms of colour and grading (0.30 mm d_{50}). The preliminary design of sand extraction operations from the Tweed River (Area 5) and delivery via pipeline to Kingscliff Beach was previously investigated by KBR [15]. The study by KBR recommended that a temporary-type pipeline be deployed and an optimal pipeline route was identified (Figure 8).

Figure 8: Preferred Pipeline Route for Sand Nourishment Delivery (Source: KBR [15])

In the design of the nourished profiles for Kingscliff Beach, no volume adjustments due to grain size differences were necessary since the proposed nourishment sand (borrow sand) is an exact match for the native sand. The Equilibrium Beach Profile Method was used for preliminary estimates of required fill volume. This method assumes that the beach profile is in equilibirum with the wave climate, that is, there is no net cross-shore sediment transport and is recommended when mutliple historical bathymetric surveys are unavailable [11].

For beach nourishment in conjunction with terminal seawall design 1, a minimum level of sand against the seawall toe of -1 m AHD is required to prevent undermining. WRL undertook iterative SBEACH modelling for the 100 year ARI event to prepare several representative nourished profiles. Borrow sand was added to the modelled beach profiles until the scour level did not reach -1 m AHD during the event. The results indicated that nourishment is only required below 0 m AHD to maintain the structural integrity of the seawall. That is, no beach nourishment is required on the sub-aerial beach.

For the comparitive case, where protection for the 1 in 100 year ARI erosion event is only provided by beach nourishment and a terminal seawall is not constructed, the protection offered by any existing structures is also ignored. That is, during the 100 year ARI event, erosion would not extend landward of the existing foreshore alignment. The majority of the sand required for this case is located below mean sea level.

A third case, involving beach nourishment for the purpose of maintaining an acceptable beach width for amenity, was assessed but has not been included in this paper for brevity.

5.4 Required Nourishment Volumes

For the two beach nourishment cases considered, Table 3 outlines the initial and ongoing volumes of sand required to provide 100 year ARI erosion event protection for Kingscliff Beach. An ongoing nourishment campaign interval of 10 years was adopted to estimate ongoing nourishment requirements over 50 years. Following initial beach nourishment, ongoing nourishment is requirement to offset the projected effects of recession due to climate change and ongoing underlying recession. It can be seen that over the design life, the sand required to provide erosion protection without a terminal seawall is approximately five times that with a seawall and exceeds the reserve of sand available in the Tweed River (660,000 m^3).

Table 3: Nourishment Volumes over 50 Years

	Beach Nourishment Volume (m^3)	
Project Stage	Seawall #1 + Nourishment	Nourishment Only
Initial	20,200	809,800
Ongoing	195,000	195,000
Total	215,200	1,004,800

6 Life-Cycle Costs

This section outlines the relative capital and maintenance costs for the terminal seawall designs developed and the two beach nourishment cases assessed. Other than consideration of construction and maintenance costs, the assessment was technical in nature and did not examine economic and social benefits (i.e. the value of the protected assets and beach amenity). Detailed cost estimate breakdowns for these coastal protection assets have not been included in this paper for brevity. The reader is directed to [8] for itemised discussion of these assumptions.

The total capital cost estimate for initial construction of a terminal seawall with a design scour level of -1 m AHD combined with beach nourishment is approximately $8.1M (Table 4). Similarly, the estimated initial costs for a seawall with a -2 m AHD scour level without beach nourishment is $7.8M. In comparison, for equivalent erosion protection against the 100 year ARI event without a terminal seawall, the initial costs for beach nourishment are approximately $21.9M. Note that the initial lineal rate cost estimates for construction of terminal seawall designs 1 and 2 only are $12,000 and $17,500 per metre of structure, respectively.

Table 4: Initial Capital Cost Estimates

Project Asset	Cost Estimate ($M ex GST) Nourish Only	Seawall #1 + Nourish	Seawall #2 Only
Seawall	0.0	5.4	7.8
Nourish	21.9	2.8	0.0
Total	21.9	8.1	7.8

Under design conditions, a small amount of damage to the greywacke rock armour in the terminal seawall is acceptable. An allowance for structural maintenance of the rock rubble and cleaning to remove rubbish and rodents to maintain pedestrian safety is required. The average annual cost for maintenance of the rock rubble options was estimated to be approximately 1.0% of the initial capital cost for the terminal seawall only (i.e. excluding beach nourishment costs). As shown in Table 5, this equates to maintenance costs for the terminal seawall designs of approximately $54,000 per year (design scour level -1 m AHD) and $78,000 per year (scour level -2 m AHD). The maintenance cost estimate derived for ongoing beach nourishment is approximately $185,000 per year (commencing 10 years after initial nourishment).

Table 5: Ongoing Maintenance Cost Estimates

Project Asset	Cost Estimate ($K/year ex GST) Nourish Only	Seawall #1 + Nourish	Seawall #2 Only
Seawall	0	54	78
Nourish	185	185	0
Total	185	239	78

To estimate the net present cost for maintenance of the terminal seawall and ongoing beach nourishment, a period of 30 years with a discount rate of 7% was used. Even though the working life of the seawall is 50 years, the maximum project period of 30 years was used for appraisal of long term maintenance costs in line with the upper limit recommended in NSW treasury guidelines [16]. The estimated net present costs for seawall maintenance and nourishment to offset ongoing recession are set out in Table 6. The total 30 year net present cost for a terminal seawall (design 1) combined with beach nourishment is approximately $10.1M. The net present cost for a terminal seawall (design 2) without beach nourishment is $8.7M. For erosion protection with beach nourishment only, the net present cost over 30 years is $23.2M.

Table 6: Net Present Costs over 30 Years

Project Stage	Cost Estimate ($M ex GST) Nourish Only	Seawall #1 + Nourish	Seawall #2 Only
Initial	21.9	8.1	7.8
Ongoing	1.3	2.0	1.0
Total	23.2	10.1	8.7

The initial capital costs for construction of a terminal seawall only, are significantly lower for a -1 m AHD design scour level ($5.4M) relative to a -2 m AHD design ($7.8M). However, the key difference is that initial and ongoing beach nourishment is necessary in conjunction with terminal seawall design 1. The initial capital costs for construction including beach nourishment are similar for designs 1 and 2 ($8.1M and $7.8M), but the life-cycle cost ($8.7M) for a terminal seawall without beach nourishment is lower than with nourishment ($10.1M). Note that nourishment may bring benefits not considered in engineering costs.

The net present cost for providing erosion protection with beach nourishment only is up to 2.7 times the cost to provide equivalent protection with a terminal seawall. However, it is acknowledged that this protection strategy would provide an acceptable beach width more regularly than would otherwise occur.

7 Summary

Two different designs (with and without beach nourishment) were prepared for a long term terminal seawall to provide erosion protection for the Kingscliff Beach Holiday Park. Following establishment of the proposed seawall location, the most important decision affecting its overall geometry is whether or not a commitment can be made to a sustained beach nourishment program. The adopted beach scour level (the seawall design scour level) is reliant upon this decision, determines the toe level of the structure, influences the design crest level and establishes the required primary armour size.

When considering the implementation of integrated foreshore protection works (i.e. beach nourishment in conjunction with a terminal seawall), it is critical to appreciate their interdependence. Beach nourishment may be used to raise the expected beach scour level at a seawall, reduce its overall geometry accordingly and offset the projected effects of recession due to climate change and ongoing underlying recession.

Two-dimensional SBEACH erosion modelling was undertaken to confirm the design scour level for terminal seawall designs with (-1 m AHD) and without (-2 m AHD) ongoing beach nourishment for a 50 year working life and the 100 year ARI event.

To confirm the design crest level for each terminal seawall design, the average wave overtopping rates were estimated for various crest level options using empirical EurOtop techniques. This process was undertaken for present day conditions and at the end of the design working life (50 years). To prevent negative impacts on beach amenity (views and sea breezes) associated with single stage construction with allowance for future SLR, an adaptive response through a two stage construction design was adopted. For seawall designs with and without nourishment, Stage 1 will consist of the bulk of the proposed works (crest elevation +5.0 m AHD), Stage 2 will be deferred until the extent of measured sea level rise exceeds a trigger value and will consist of raising the crest with a wave return wall (crest elevation +6.0 m AHD) at that time.

The terminal seawall design reliant on ongoing beach nourishment (design scour level -1 m AHD) has a relatively small footprint and a median primary armour rock mass of 4 t. Alternatively, the terminal seawall design without beach nourishment (design scour level -2 m AHD) has a relatively large footprint and a primary armour mass of 7 t.

Since the crest level and median primary armour rock masses for both terminal seawall designs have been established through empirical techniques, site specific physical modelling is recommended to obtain precise wave overtopping estimates and optimise the required armour mass.

Iterative SBEACH modelling for the 100 year ARI event was undertaken to prepare several representative nourished profiles. For beach nourishment in conjunction with terminal seawall design 1, borrow sand was added to the modelled beach profiles until the scour level did not reach -1 m AHD during the event. For comparitive purposes, the extent of beach nourishment required to provide erosion protection without a terminal seawall was also assessed. The required intial nourishment volumes to prevent undermining of terminal seawall design 1 are modest compared to equivalent erosion protection event without a terminal seawall (i.e. large scale beach nourishment). However, construction of a terminal seawall without beach nourishment will generally narrow the existing dry beach area compared to that currently seaward of the Kingscliff Beach Holiday Park resulting in less beach amenity than present.

Capital and maintenance costs were prepared for both seawall designs and the two beach nourishment cases assessed. The initial capital costs for construction of a terminal seawall only, are significantly lower for a -1 m AHD scour level ($12,000/metre) relative to a -2 m AHD design ($17,500/metre). However, the life-cycle cost for terminal seawall design 1 ($10.1M) is higher than for design 2 ($8.7M), since initial and ongoing beach nourishment is required to maintain the structural integrity of design 1. Further to this, a commitment to ongoing beach nourishment is required for 50 years following the initial beach nourishment to maintain long term erosion defence reliability. While implementing beach nourishment in conjunction with a terminal seawall will provide effective coastal protection, if future funding or sand sources are uncertain, there is a higher risk associated with selection of a seawall design reliant on ongoing beach nourishment.

If site specific erosion modelling had not been undertaken and the engineering rule of thumb (-1 m AHD scour level) adopted instead, the resultant terminal seawall would have a higher life-cycle cost and be reliant on initial and ongoing beach nourishment throughout its design working life.

The net present cost for providing erosion protection without a terminal seawall is up to 2.7 times the cost to provide equivalent protection with a seawall. However, this would provide an acceptable beach width more regularly than would otherwise occur.

While the two terminal seawall designs with and without nourishment were specifically developed for Kingscliff Beach, the approach taken to investigate capital and ongoing costs related to climate change is applicable to many coastal infrastructure developments.

If beach nourishment is undertaken at Kingscliff Beach, the construction of a groyne field would minimise the loss of placed sand towards the northern end of Wommin Bay from alongshore spreading. Inclusion of a groyne field with beach nourishment would reduce the cost, volume and interval of ongoing nourishment campaigns.

Acknowledgements

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