Assessment and Decision Frameworks for Seawall Structures

2013

Appendix F

Case Study Clontarf

SYDNEY COASTAL COUNCILS GROUP

Coastal Adaptation Decision Pathways Project **(CAP)**

The Sydney Coastal Councils Group (SCCG) is a voluntary Regional Organisation of Councils representing fifteen coastal and estuarine councils in the Sydney region. The Group promotes cooperation and coordination between Members to achieve the sustainable management of the urban coastal environment.

Project Management

Geoff Withycombe
Executive Officer
Sydney Coastal Councils Group

Douglas Lord
Director
Coastal Environment Pty Ltd

Professor Rodger Tomlinson
Director
Griffith Centre for Coastal Management
Griffith University

Project Co-ordination

Dr Ian Armstrong
Project Officer - Climate Change Adaptation
Sydney Coastal Councils Group
Ph: 02 9288.5802
ian@sydneycoastalcouncils.com.au

Australian Government
Department of Climate Change and Energy Efficiency

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Cover image: Coastal seawall. Provided by Douglas Lord

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Appendix F
Case Study – Clontarf

Prepared for

Sydney Coastal Councils Group

Prepared by

Water Research Laboratory
University of New South Wales
110 King Street
Manly Vale NSW 2093

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SYDNEY COASTAL COUNCILS GROUP

Part A	Synthesis Report
Part B	Appendices Appendix A – Literature Review Appendix B – Geotechnical Considerations Appendix C – Economic Considerations Appendix D – Site Field Data Collection Appendix E – Case Study Bilgola Appendix F – Case Study Clontarf Appendix G – Case Study Gold Coast

APPENDIX F PREFACE

This Appendix was prepared by the Water Research Laboratory (WRL) of the University of New South Wales for this Report titled *Assessment and Decision Frameworks for Seawall Structures*. The purpose of the information in this Appendix was to assess the likely design elements applicable to a range of seawalls existing on estuarine beaches at Clontarf in Sydney Harbour, based on existing information and on field inspections. The 'probable' design cross-sections were then subjected to design assessment using appropriate design conditions for current and future sea levels.

The assessment reported in this Appendix should not be construed as a detailed assessment of the adequacy or otherwise of any of the existing seawalls at Clontarf Beach. The study was purely a technical exercise in demonstrating an appropriate methodology for seawall assessment as applied by a leading coastal engineering consultancy. No consideration was given to the economic, wider environmental and community values or planning frameworks associated with managing seawalls. In particular, many of the design assumptions may not be appropriate relying on available information and generic values. They could be refined with more detailed investigation. While it is intended the document will be used widely, with councils in many locations as its intended audience, it cannot be assumed that all seawalls are a council-owned asset and that in instances, there could be multiple ownership/responsibility issues that have not been considered in this assessment as they are outside the scope of this project.

The authors of the WRL report were A. Mariani and I. Coghlan. It has been published by WRL as a single Report WRL2012/13 titled *Seawall Structure Assessment at Bilgola and Clontarf, Sydney, NSW* which includes the information included here as Appendix D, Appendix E and Appendix F. That WRL report was released in September 2012 and can also be viewed in that format.

The information included here has been taken in its entirety from the WRL report and is a true reflection of the original advice provided to the project by the Water Research Laboratory. No additions, edits or changes have been made to their final report, other than minor editorial and layout changes for consistency in appearance. References to sections, figures and tables are to those included within this Appendix or the associated Appendices as quoted.

As appropriate, information from this Appendix has been incorporated or referenced in the main report for this project.

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FIGURES

Glossary

accretion	On a beach, deposition of sediment (typically sand) transported naturally to the location by waves, currents and winds
active beach zone	The section of the beach from the offshore limit of onshore/ offshore sand movement under waves to the landward limit of wave uprush during storms
active slope	The slope of the nearshore area which adjusts to prevailing waves and currents through associated erosion or accretion. Tends to be flatter on an eroding profile and steeper on an accreting profile
aeolian processes	Pertaining generally to sand or sediment moved and deposited by wind above the mean high water mark
altimetry	The measurement of altitude
annual exceedance probability (AEP)	The likelihood that an event of a given size, on average, occurs or is exceeded once each year (e.g. wave height, water level, wind velocity). It may occur several times in one year or may not occur for several years. See also exceedance probability.
barometric setup	The increase in means sea level caused by a change in barometric pressure. If barometric pressure is low (cyclone) then sea level is elevated above normal, if barometric pressure is high (anticyclone) then sea level is depressed below normal average levels
deepwater wave height	Water depth in which the velocity generated by the action of the wave is negligible. Commonly referred to as half the length of the wave in deep water
depth-limited	The maximum height of a wave that can be transmitted and break in a given water depth. Commonly used as a limiting design condition for shoreline structures in exposed coastal locations where the biggest wave reaching the structure is controlled by the water depth at the strucutre. Larger waves will break offshore in greater depths, not reaching the structure as an unbroken wave
diffraction	The bending of a wave front as it reaches shallow water and slows down. Diffraction can result in focussing or spreading of waves which increase in height as they slow down. On a straight, parallel shoreline as the wave length decreases the wave height increases to maintain the wave energy (shoaling)
geotextile	A permeable geosynthetic sheet comprised solely of textiles, used in geotechnical engineering construction. Materials may be either woven or needle punched and are robust Commonly geotextiles provide a filter layer under rock armour or can be fashioned into containers filled with sand used as armour units in a structure
groundwater	Water beneath the surface of the ground, often perched above an impervious layer
incident wave	Wave moving landward at a particular location and time
Mean High Water Springs (MHWS)	The ocean level that is the average of all the twice-daily high tides at spring periods
mean sea level	The average level of the surface of the sea over a long period of time in all stages of oscillation. Also the average level which would exist in the absence of tides. Approximately 0m AHD

nearshore	That section of the shoreline extending from the onshore limit of storm wave action to the landward edge of the offshore region. Commonly defined as the limit of onshore/offshore sediment movement under wave action and typically in a water depth of 10m to 30m
outflank	At the end of a seawall or where gaps exist in a discontinuous seawall, during storm events waves and erosion can penetrate behind the exposed ends of the seawall, causing collapse of the structure from the landward side
overtopping bore	Where a seawall is significantly overtopped, the volume of water travelling inland beyond the crest as a single wave front
photogrammetry	The process of making surveys, maps and measurements using overlapping vertical aerial photography
propagation distance	The distance which a wave has travelled form the original point of origin
recession	The landward movement of a shoreline over time (e.g. receding shoreline). Can be caused by erosion resulting in more sediment leaving a coastal compartment than is entering it, or as a result of sea level rise inundating the shoreline over time
reflected wave	Waves travelling toward a shoreline or structure will be partly dissipated against the structure and partly reflected from the structure. Where reflection is high (such as a vertical seawall or cliff) wave heights immediately seaward may be increased and the depth of wave scour at the sea bed correspondingly increased
refraction	The process by which the direction of a wave train moving in shallow water at an angle to the contours is changed to align itself parallel to the shoreline. That part of the wave in deeper water moves faster than that part in shallower water, causing the wave to bend as it approaches the shore
rubble mound rock armour	The larger size stone intentionally placed on the exposed surface of a seawall or revetment, specifically to resist and dissipate the forces of waves on the structure
scour	Erosion, normally by the action of flowing water or wave action
sea level rise (SLR)	A rise in mean sea level when averaged over an extended time period. In terms of climate change is usually used to describe the predicted or projected increase in the mean sea level that will occur to a future date measured above the 1990 mean sea level
sediment transport	The main agencies by which sediments are moved are gravity (gravity transport); running water (rivers and streams); ice (glaciers); wind; the sea (currents and alongshore drift). Running water and wind are the most widespread transporting agents. In both cases, three mechanisms operate, although the particle size of the transported material involved is very different, owing to the differences in density and viscosity of air and water. The three processes are rolling or traction, in which the particle moves along the bed but is too heavy to be lifted from it; saltation; and suspension, in which particles remain permanently above the bed, sustained there by the turbulent flow of the air or water
significant wave height	The average height of the highest one third of waves recorded in a given monitoring period. Also referred to as H1/3 or Hs. Commonly referenced statistical wave height

stillwater level	The surface of the water if all wave and wind action were to cease. In deep water this level approximates the midpoint of the wave height. In shallow water it is nearer to the trough than the crest. Also called the undisturbed water level
storm demand	That volume of sand located on a beach that can theoretically be eroded and removed offshore by a single storm event or close spaced series of storms. Provides an indication of the susceptibility of a beach to storm erosion
storm surge	The increase in onshore elevation of the mean ocean level associated with a storm. Primarily comprises a tidal component, a barometric component (low pressure) and wind setup caused by strong onshore winds at the shoreline, but does not include wave setup and wave runup
toe level	The level of the seaward base of a seawall
water table	The upper surface of a zone of saturation, where the body of groundwater is not confined by an overlying impermeable formation. Where an overlying confining formation exists, the aquifer in question has no water table
wave period	The time interval occurring between two consecutive wave crests
wave return parapet	A small structure constructed at the crest line of a seawall to limit minor wave overtopping by increasing the crest height. Often retro fitted to existing structures and frequently shaped to maximise the seaward wave reflection of the wave crest
wave runup	The maximum elevation reached by a broken wave against the beach or shoreline structure, measured above the still ocean level. Storm wave runup is a key element in the design of coastal protection works
wave setup	The amount by which the still water level inshore of the breaking wave zone exceeds that outside; in part due to the kinetic energy in the breaking waves being converted into an elevated inshore water level

Acronyms

AEP	Annual Exceedance Probability
AHD	Australian Height Datum -
ARI	average recurrence interval
GPR	Ground Penetrating Radar
H_s	significant wave height
LGA	Local Government Area
MSL	Mean Sea Level
MHWS	Mean High Water Springs
SLR	Sea Level Rise

1. INTRODUCTION

The Water Research Laboratory (WRL) of the University of New South Wales (UNSW) was engaged by Sydney Coastal Councils Group (SCCG) to undertake three case studies assessing existing seawalls in Sydney. At the request of SCCG, each case study was documented as a stand-alone appendix within the main report.

The present scope of works included the following case studies:

- Remote Sensing Assessment of a Buried Seawall Structure (Bilgola Beach Case Study) , Appendix D to main report)
- Assessment of Open Coast Seawalls (Bilgola Beach Case Study) (Appendix E to main report)
- Assessment of Estuarine Beach Seawalls (Clontarf Case Study) (this report, Appendix F to main report).

Note that the results presented in this report should not be used to assess the suitability or otherwise of any particular structure, nor to determine the suitability of any structure in protecting development at Bilgola Beach. Rather, the case study has been prepared as a practical, useful and usable framework to assist local government in managing and assessing generic seawall structures where no detailed design information is available.

The objective of the investigation included in Appendix D was the trial of a non-intrusive technology (ground penetrating radar (GPR)) to determine several key geometric parameters of an existing buried seawall. This report aimed to verify the reliability and suitability of GPR for this purpose, by comparing the GPR outputs to drilling logs.

The objective of the investigations presented in Appendices E and F was to analyse the suitability of existing seawalls to withstand the occurrence of 1-, 10-, 50- and 100-year-ARI events for present-day conditions and for the 2050 and 2100 planning horizons, including sea level rise projections. The general methodology applied for the assessment of these coastal structures consisted of the following tasks (also presented diagrammatically in Figure 1.1):

- data compilation: an initial data and literature review including review of previous site investigations
- seawall characterisation: establishing relevant engineering design parameters such as crest and toe levels, construction method etc.
- environmental conditions: establishing design parameters in terms of wave and water level conditions and relevant coastal processes such as erosion, recession and inundation
- seawall assessment: a stability assessment with regards specifically to coastal processes
- remedial options: a list of upgrade, replacement and maintenance options, and
- future management: recommendations provided for further monitoring and data collection.

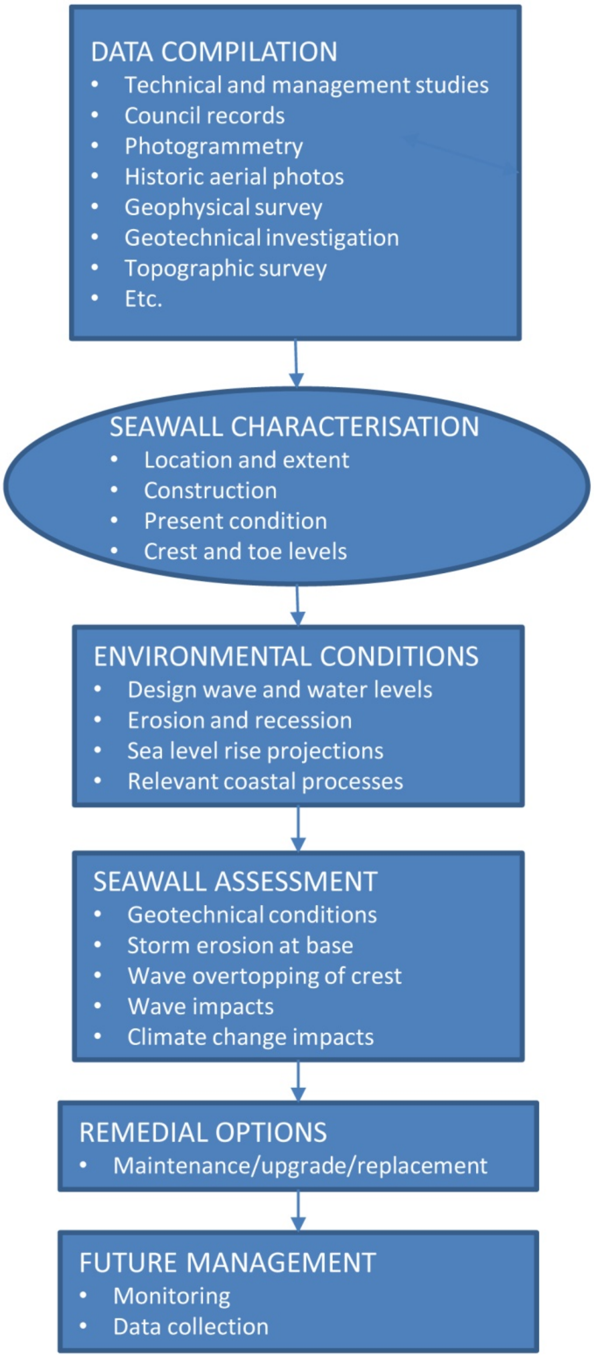

Figure 1.1 Methodology Applied for Seawall Suitability Assessment in Appendices E and F

2. Assessment of Estuarine Beach Seawalls (Clontarf Case Study)

2.1 General

The Water Research Laboratory of the University of New South Wales was engaged by Sydney Coastal Councils Group to undertake an assessment of seawalls on an estuarine or harbour beach. In consultation with SCCG, Clontarf was selected as an appropriate location to undertake the case study on the condition of a variety of existing seawalls. **Note that the results presented in this report should not be used to assess the suitability or otherwise of any particular structure, nor to determine the suitability of any structure in protecting development at Clontarf**. Rather, the case study has been prepared as a practical, useful and usable framework to assist local government in managing and assessing small seawall structures where no detailed design information is available.

Clontarf is part of the Manly Council Local Government Area (LGA) and its coastline includes the sandy foreshore from Sandy Bay at the western end (located 500 metres east of the Spit Bridge, fronting Sandy Bay Road, Holmes Avenue and Monash Crescent) to Clontarf Point at the eastern end (at the end of Monash Crescent). The Clontarf Marina, the netted swimming reserve and Clontarf Reserve are located along the foreshore. Figure 2.1 presents the study area location.

There are several discrete seawall structures along the Clontarf section of coastline. For the purpose of this study the following three sections (from north to south) were assessed:

- Clontarf Seawall 1: the continuous section of seawall running from the western end of Sandy Bay to the Marina. This section of wall is of generally steep sloping construction and constructed of dressed or cut sandstone blocks.
- Clontarf Seawall 2: the vertical concrete section of seawall adjacent to the netted pool enclosure.
- Clontarf Seawall 3: the sloping section of seawall of sandstone construction located at the Monash Crescent public access to the southern segment of Clontarf foreshore.

Figure 2.2 shows the location of the three seawalls within the Clontarf foreshore area. Table 2.1 presents a summary of the seawalls assessed.

Table 2.1 Summary of Assessed Seawalls at Clontarf

Seawall	Location	Construction	Year of Construction	Length (m)
1	Between Sandy Bay and the Marina	Vertical (0-70 m) and steep-sloping at 1H:1V (70-200 m); sandstone blocks set in mortar; concrete capping	early 1950s	200
2	At pool enclosure	Vertical retaining concrete structure; considered as landscaping wall	not reported	60
3	Public access from Monash Crescent	Sloping rock seawall (1H:1V)	not reported	5

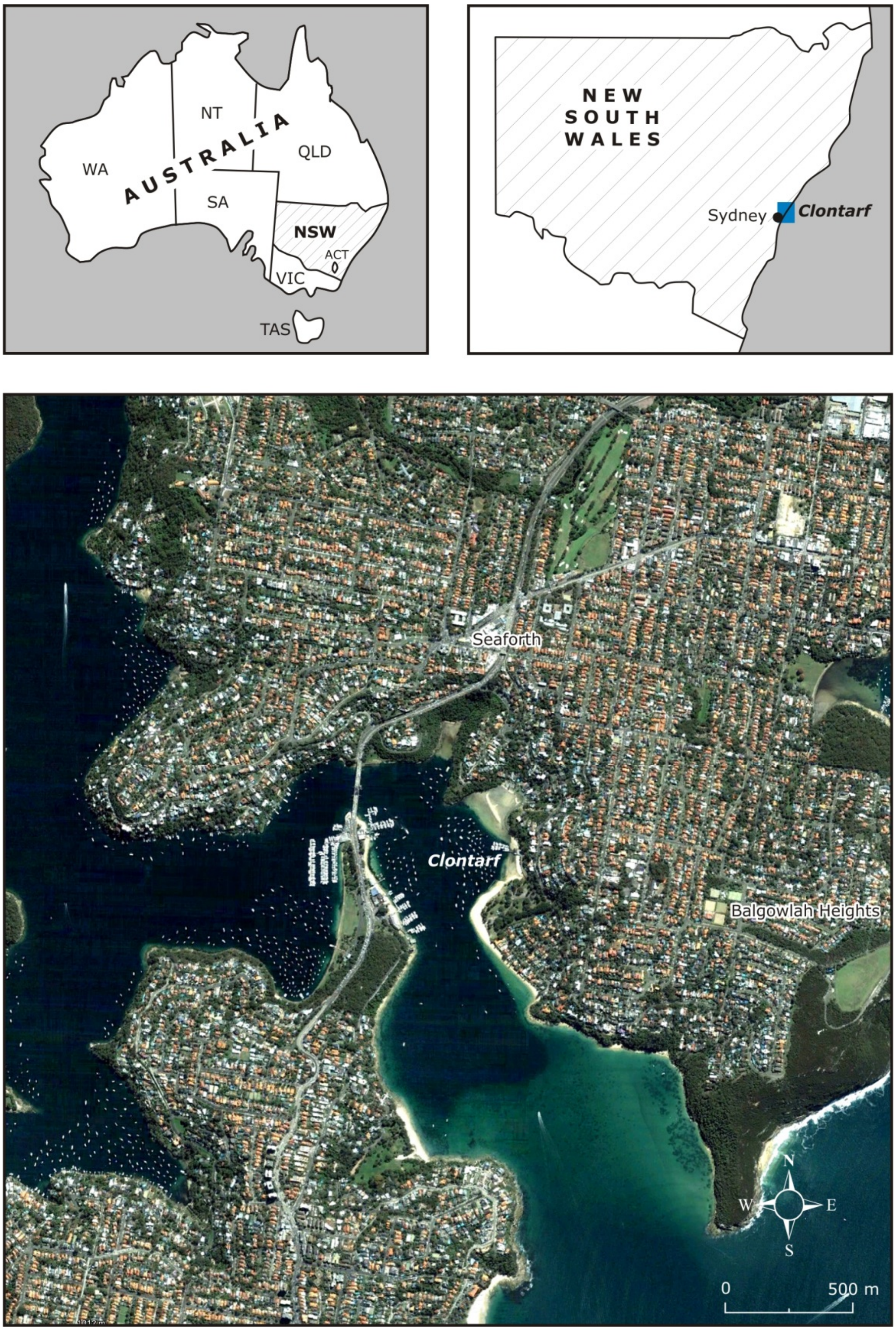

Figure 2.1 Location of Clontarf Sydney NSW

Figure 2.2 Seawall Locations in Clontarf

2.2 Literature Review

A substantial body of literature in the form of consultant and council technical and management reports exists for the Manly Council LGA coastline. All available literature addressing coastal processes, coastal protection works and coastal management within the Clontarf foreshore was consulted, with the most important listed in the following discourse.

2.2.1 Coastal Hazard Definition Studies

A coastal hazard definition report was prepared by the Water Research Laboratory of the University of New South Wales for Manly Council. The report provided information on the coastal hazards relevant to the Ocean Beach and North Harbour sections of the Manly coastline in particular in terms of coastal erosion and coastal recession due to sediment loss and sea level rise. The report consulted for the current study is:

- Mariani, A, Carley J T, Lord, D B and Shand, T D 2012, Identification of Coastal Hazard Risk Areas to Projected Sea Level Rise for the Manly Local Government Area, WRL Technical Report 2011/19 for Manly Council.

2.2.2 Coastal Process Studies

Cardno Lawson Treloar prepared two reports addressing sediment transport processes in the Sydney Middle Harbour area from Clontarf Point to the Spit. A stability assessment of the seawalls along this section of coastline was presented in the reports:

- Cardno Lawson Treloar 2009, Clontarf Sedimentary Processes and Foreshore Stability Study, Sedimentary Processes Report, Manly Council
- Cardno Lawson Treloar 2009, Clontarf Sedimentary Processes and Foreshore Stability Study, Foreshore Stability Report, Manly Council.

2.2.3 Coastline Management Studies

The following coastline management study report was issued by Manly Council:

- Manly Council 2009, North Harbour Coastline Management Study, Final Report February 2009.

2.2.4 Coastal Protection Works Studies

WRL prepared a report for Manly Council providing a risk assessment and a priority list for upgrade or replacement of seawall structures within the Manly LGA. The report produced was:

- Mariani, A and Carley, J T 2012, Manly LGA Seawall Risk Assessment and Plan for Priority Upgrade/Replacement, WRL Technical Report 2012/02 for Manly Council.

2.3 Structure Characterisation

The locations of the three structures investigated, with overview photos of the main features are shown in Figures 2.3, 2.4 and 2.5 and summarised in Figure 2.2. Table 2.1 reports on seawall location, extent and construction.

Representative design cross-sections were prepared for the seawall structures assessed and are presented in Figure 2.6. The sections were based on the review of all relevant documents including council records and technical drawings, consultant reports, geotechnical investigations and topographic surveys. Structure details and information that could not be verified are clearly identified in the cross section figures.

From Sandy Bay to the Clontarf Marina (Seawall 1), the seawall is of vertical to sub-vertical sandstone construction and for much of its length has a poured concrete capping. While of relatively early construction (1950s) it is generally in good condition. The section to the west of the Marina appears to be of earlier construction or has been rebuilt and is ‘bulging’ seaward. No information is available detailing the precise construction of the wall. It appears to be performing adequately at the present time.

Adjacent to the swimming enclosure, the sandstone wall (Seawall 2) is replaced by a vertical concrete wall of unknown cross-section and toe depth. However, based on previous geotechnical investigations, the seawall toe level is located below 0.3 m AHD. In assessing the stability of the seawall (and in the absence of further information), WRL conservatively assumed that the seawall had a toe level of 0.3 m AHD. This section of the wall is generally in poor condition, showing cracking and rotation. Exposed reinforcing steel is rusting. The sections of wall east of the swimming enclosure and within Sandy Bay have a crest level generally at 1.4 m AHD to 1.5 m AHD.

The seawall fronting the Monash Crescent access (Seawall 3) to the beach is of sloping sandstone construction and unknown cross-section and toe depth. As the toe level of Seawall 3 is unknown, no definitive assessment of undermining of this seawall could be undertaken. However, based on the experience with seawalls of similar construction and age located within the Manly LGA coastline, it is unlikely that the seawall toe level would be located below 1.0 m AHD. In assessing the stability of the seawall, WRL assumed that the seawall had a toe level of 1.0 m AHD. The wall is in reasonable condition with a crest level of 1.9 m AHD.

Table 2.2 lists the measured crest elevation, the toe level (when determined by previous geotechnical investigations or assumed based on previous experience), the average sand level against the seawall (inferred from historical photogrammetry analysis) and the present condition of each surveyed wall.

Table 2.2 Clontarf Seawall Characteristics and Present Conditions

Seawall	Location	Crest Level (m AHD)	(1)Toe Level (m AHD)	(2)Ave. Sand Level (m AHD)	(3)Present Condition
1	Between Sandy Bay and the Marina	1.5	-0.4	0.0	Generally reasonable condition, localised erosion of sandstone and mortar, isolated cracks, settling and rotation
2	At pool enclosure	1.4	< 0.3 (0.3 adopted)	1.1	Reasonable condition, isolated forward leaning and corrosion of reinforcing steel
3	Public access from Monash Crescent	1.9	- (1.0 adopted)	1.3	Reasonable condition, weathering of mortar

Notes: (1) as determined by previous geotechnical investigations
(2) as determined by historical photogrammetry analysis
(3) present condition inferred from visual assessment by experienced coastal engineers

Figure 2.3 Clontarf Seawall 1 Between Sandy Bay and Marina 27 July 2001

Figure 2.4 Clontarf Seawall 2 at Pool Enclosure 27 July 2001

Figure 2.5 Clontarf Seawall 3 at Monash Crescent Public Access 27 July 2001

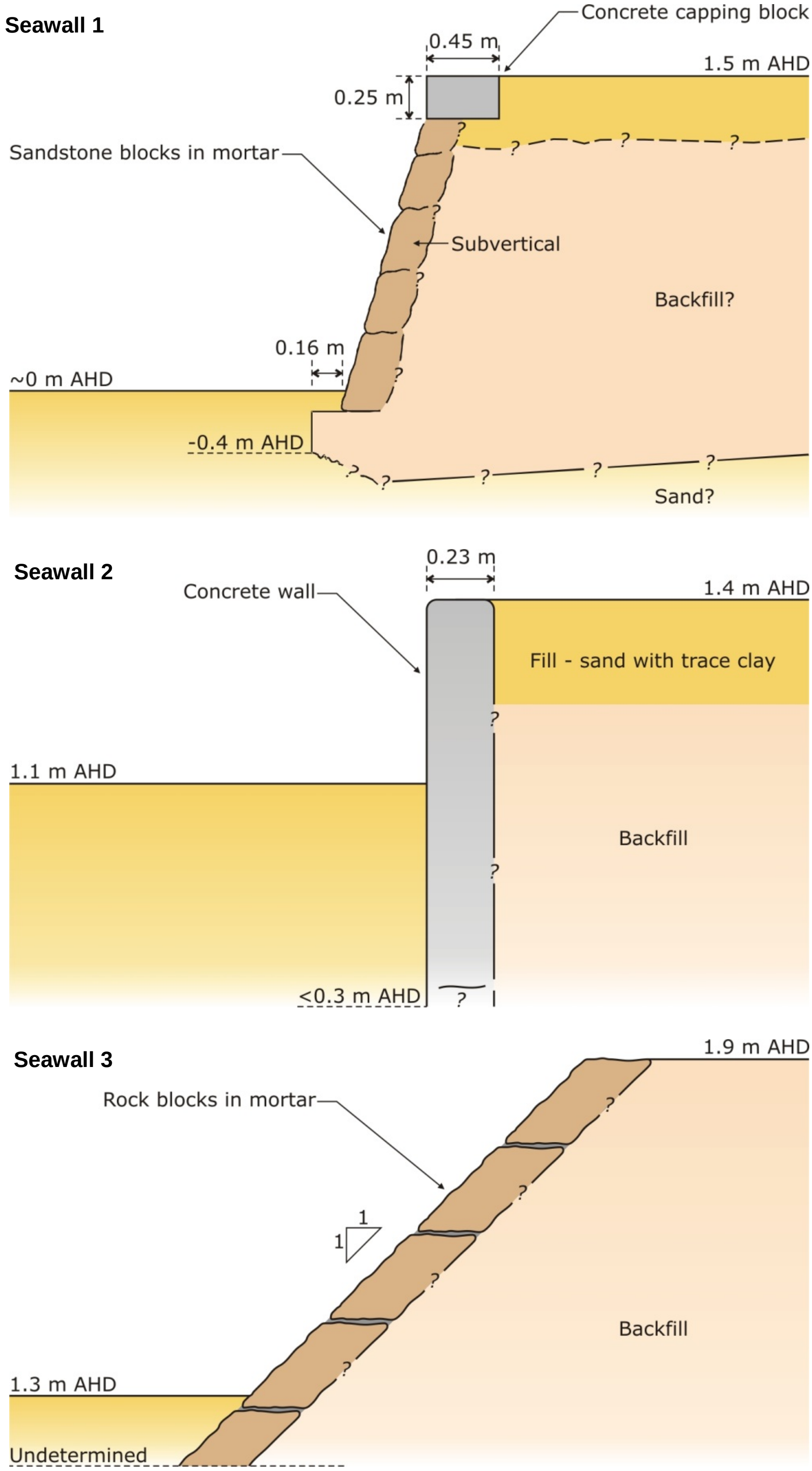

Figure 2.6 Clontarf Seawall Cross-sections

2.4 Environmental Conditions

2.4.1 General

Design parameters for the seawalls assessed include wave and water level conditions and the expected beach scour level at the toe of the structure. The geotechnical conditions at the site which determine the adequacy of existing foundation conditions were outside the scope of this assessment. The toe scour level influences the water depth at the structure which, together with the design water level, determines the maximum depth-limited breaking wave height that can impact 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.

2.4.2 Design Life and Design Event

Establishing the design working life of the assessed seawalls is critical for estimation of subsequent design parameters. The typical design life is 50 years for a normal maritime structure and 100 years for a structure protecting residential developments (AS 4997, 2005). As such, WRL adopted a design life of 100 years for this assessment. The Australian Standard 4997 recommends design significant wave heights for marine structures based on the function and design life of the structure as reproduced in Table 2.3. Note that while this standard covers maritime structures (e.g. wharves and vertical seawalls), it specifically excludes the design of rubble mound rock armoured walls. AS 4997 recommends that the design water levels accompanying these waves should not be below Mean High Water Springs (MHWS).

Table 2.3 Annual Probability of Exceedance of Design Wave Events
(Source: AS 4997)

Structure Description	Design Working Life (Years)			
	5 or less (temporary works)	25 (small craft facilities)	50 (normal maritime structures)	100 or more (special structures/ residential developments)
Structures presenting a low degree of hazard to life or property	1/20	1/50	1/200	1/500
Normal structures	1/50	1/200	1/500	1/1000
High property value of high risk to people	1/100	1/500	1/1000	1/2000

Based on this guideline, selection of the 1-in-200 to 1-in-500-year ARI event would be suitable for seawalls as they may be regarded as 'normal' maritime structures. However, the seawalls assessed are generally smaller structures and are typically only one component of a more comprehensive and holistic foreshore management solution. Additionally, best practice in coastal hazard assessments for local government areas typically considers the 100-year ARI as the design criteria for deriving coastal setbacks and inundation areas. As such, there is a reasonable basis for accepting some reduction in the design conditions. The guideline gives no further direction on the recommended design water level.

A further consideration is that the maximum significant wave height that can reach the structure is a function of design water level. WRL has selected the 1-in-100-year ARI event for both wave conditions (height, period and direction) and water level conditions (tide plus anomaly). Due to depth-limited conditions, the design wave heights at the seawalls could be generated by an event where the recurrence interval of the deep water wave height was lower than 100-year ARI but the water level was higher than 100-year ARI.

2.4.3 Design Wave Conditions

The Clontarf coastline is subject to waves originating from offshore storms (swell) or produced locally (wind waves) within Sydney Harbour and the nearshore coastal zone. Swell waves reaching the seawalls will be modified by the processes of refraction, diffraction, wave-wave interaction, dissipation by bed friction, wave breaking and wind. Locally generated waves undergo generation processes as well as the aforementioned propagation and dissipation processes.

Offshore wave characteristics were derived through statistical analysis of recorded data from the Sydney directional wave buoy and extrapolated to extreme events (Shand et al., 2010). Design offshore wave conditions for the 1-in-1-, 10-, 50- and 100-year ARI events adopted within the present study are shown in Table 2.4. Nearshore wave conditions at the three seawall locations as derived from the application of a wave transformation numerical model (Mariani et al., 2012) are presented in Table 2.5. The nearshore wave conditions were conservatively adopted as the result of a south-south-east swell event with south-westerly winds blowing across Sydney Harbour. The concurrence of these two events was considered plausible and would result in the highest nearshore wave heights at Clontarf.

Table 2.4 Extreme Offshore Wave Conditions (All Directions)
(Source: Shand et al., 2010)

Average Recurrence Interval (years)	One hour exceedance at Sydney offshore buoy	
	Hs (m)	Tp (s)
1	5.9	11.0
10	7.5	12.1
50	8.6	12.7
100	9.0	13.0

Table 2.5 Clontarf Nearshore Extreme Wave Conditions
(Source: Mariani et al., 2012)

Average Recurrence Interval (years)	H_s (m)		
	Between Sandy Bay and Marina (Clontarf north) (m)	At pool enclosure (Clontarf spit) (m)	At private properties along Monash Cres. (Clontarf south) (m)
1	0.2	0.3	0.4
10	0.3	0.5	0.6
50	0.4	0.5	0.6
100	0.4	0.5	0.6

2.4.4 Design Water Levels

The Coastal Risk Management Guide (DECCW, 2010) recommends design elevated water levels for a range of average recurrence intervals, which are presented in Table 2.6. While these design water levels incorporate allowance for tides, barometric setup and (regional) wind setup (i.e. storm surge), wave setup is excluded and was determined through data and modelling (Mariani et al., 2012). Design water levels including wave setup but excluding local wind setup (which is considered to be negligible) for Clontarf foreshores are shown in Table 2.7.

Table 2.6 Design Water Levels, Tide + Storm Surge
(Source: DECCW, 2010)

Average Recurrence Interval (years)	Water Level Excluding Wave Setup and Runup (m AHD)
1	1.2
10	1.3
50	1.4
100	1.4

Table 2.7 Design Water Levels Including Wave Setup Excluding Wave Runup
(Source: Mariani et al., 2012)

Average Recurrence Interval (year)	Design Water Levels Including Wave Setup Excluding Wave Runup (m AHD)
1	1.3
10	1.4
50	1.5
100	1.5

2.4.5 Sea Level Rise Projections

The sea level rise (SLR) projections for the 2050 and 2100 planning periods adopted in this study were derived from the NSW Sea Level Rise Policy Statement (DECCW, 2009a) and are shown in Table 2.8. These benchmarks were established considering the most recent international (Intergovernmental Panel on Climate Change, IPCC, 2007a and 2007b) and national (McInnes, 2007) projections.

Table 2.8 Sea Level Rise Projections
(Source: DECCW, 2010)

Planning Period (year)	(1)Sea Level Rise (m)
2050	0.40
2100	0.90

Notes: (1) increase above 1990 Mean Sea Level

The design still water levels adopted for 2050 and 2100 also require a reduction of 66 mm to accommodate the estimated amount of global average sea level rise that has occurred between 1990 and the present (2012). This is estimated at approximately 3 mm/year from satellite altimetry (DECCW, 2009b).

2.4.6 Erosion and Recession

Beach erosion and recession processes at Clontarf sandy beaches were reported in Mariani et al., (2012). Clontarf beaches are characterised by a low energy wave climate (typically locally generated wind waves) due to their sheltered location within Sydney Harbour. Consequently, estimated storm demand (from photogrammetry and modelling) for the 100-year ARI storm event is consistently lower than for the open ocean coast. At the southern end of the Clontarf foreshore, the sandy stretch from Clontarf Point to Clontarf Reserve was found to be undergoing recession due to sediment loss, while the remaining sandy sections (Clontarf Spit and pool enclosure, and Sandy Bay) of coastline were found to be stable or accreting. Table 2.9 summarises design storm demand and recession rates for Clontarf sandy foreshores as well as estimated active slopes.

Table 2.9 Summary of Design Storm Demand, Recession Rate and Active Slope for Clontarf

Seawall	Volume of Storm Demand (m3/m)	Ongoing Underlying Recession Rate due to Sediment Loss (m/yr)	Active Slope or 'Bruun Factor' (-)
1	10	0.00	11
2	10	0.00	11
3	10	0.10	11

2.5 Seawall Assessment

2.5.1 Overview

The seawalls were assessed with regard to their suitability to withstand the occurrence of the adopted design storm event, i.e. the 100-year ARI event for present-day conditions and for the 2050 and 2100 planning horizons, including SLR projections. The following coastal processes were considered in assessing the likelihood of each seawall to fail:

- erosion of sand in front of the seawall during storm events
- wave impacts due to elevated water levels and large wave conditions, and
- wave overtopping of the seawall due to elevated water levels and storm wave conditions.

The erosion of sand in front of the seawall, in particular if associated with elevated groundwater levels within the seawall backfill (due to overtopping or intense rain events), can lead to geotechnical failure through the following modes:

- undermining, in which the sand or rubble toe level drops below the footing of the wall and the wall then subsides and collapses into the hole
- sliding, in which the entire wall slides seaward

- overturning, in which the wall topples over
- slip circle failure, in which the entire embankment fails
- structure instability due to increased wave impacts, and
- erosion of the backfill, caused by wave overtopping, high watertable levels, or leaching through the seawall.

A detailed geotechnical assessment was beyond the present scope of works, as such the likelihood of failure of the seawalls was assessed only for undermining, structure instability (qualitative only) and erosion of the backfill (wave overtopping). That is, WRL did not examine the likelihood of failure of the seawalls by sliding, overturning or slip circle failure.

2.5.2 Toe Undermining

The erosion of sand during storm events can cause the reduction of beach levels fronting the seawall and consequently undermine the foundations of the seawall. This can potentially cause failure of the seawall by exposing the toe of the structure to direct wave impact, or by reducing foundation support. For each seawall section, the likelihood of seawall undermining is related to the following factors:

- seawall toe design and toe levels as determined by previous geotechnical investigations or from design drawings (when available)
- seawall slope and porosity
- level of sand against the seawall prior to the commencement of a severe storm
- pre-storm volume of sand above mean sea level seaward of the structure
- storm demand or estimated volume of sand eroded (above mean sea level) during an extreme erosion event, and
- exposure (magnitude, direction and duration) to nearshore wave conditions.

Seawall toe levels, sand levels above the toe of the seawall, average sand volume of the beach fronting the seawall and design storm demand are presented for each assessed structure in Table 2.10. Toe levels for Seawalls 1 and 2 were based on previous geotechnical investigations, while the toe level of Seawall 3 was assumed based on previous experience. Average and minimum beach levels against the base of the seawalls were derived from photogrammetry analysis (Mariani et al., 2012) as were the average sand volumes and storm demand for the beach in Clontarf. It should be noted that the levels and volumes determined with this technique may not represent the full range of conditions which have occurred and depend on the dates of available photos. The storm demand for Clontarf was derived from previous coastal engineering reports as adopted in Mariani et al (2012).

Table 2.10 Summary of Factors Influencing the Likelihood of Toe Undermining at Clontarf

Seawall	Location	Toe level (m AHD)	(2)Beach level against wall (m AHD)		(2)Ave. Sand Volume Available (m3/m)	Storm Demand (m3/m)
			Min.	Ave.		
1	Between Sandy Bay and the Marina	(1)-0.4	0.0	0.0	0.0	10
2	At pool enclosure	(1)0.3	0.5	1.1	10-15	10
3	Public access from Monash Crescent	(3)1.0	0.9	1.3	7.0	10

Notes: (1) as determined by previous geotechnical investigations
(2) from photogrammetry
(3) based on experience with seawalls of similar construction and age located within Manly LGA

A range of methods was considered to determine the design scour level. These are indicated below:

- engineering 'rules of thumb'
- photogrammetry, and
- erosion modelling.

In NSW, a foundation level of approximately -1.0 m AHD is commonly adopted as an engineering rule of thumb for rigid coastal structures located at the back of the active beach area for open coast beaches. This is based on stratigraphic evidence of historical scour levels and observed scour levels during major storms in front of existing permeable and non-permeable seawalls along the NSW coast (Nielsen et al. 1992; Foster et al. 1975). However, the location of the seawall on the beach system influences the extent to which the structure interacts with coastal processes such as waves, and hazards such as erosion. That is, adopting a uniform scour level for all structures and all storm events does not consider their relative risk of toe undermining.

Photogrammetry can be used to investigate historical sand level variations in front of seawalls. However, this method can only be applied if the scour levels are located above 0 m AHD as, depending on the water level at the time of the photo, photogrammetry generally does not extend out to levels below approximately 0 m AHD.

Seawalls are commonly classified using the Weggel (1988) classification system depending on their location within the active beach system (see Figure 2.7 and Table 2.11). By definition, scour due to wave action is not a hazard for Type 1 seawalls. For seawalls which are located high up the beach above the still water level of maximum storm surge (Type 2), empirical scour techniques are not available. CEM (2006) provides design recommendations for scour in front of seawalls due to incident and reflected waves where seawalls are located below the level of storm surge (Types 3-6, as each of the seawalls at Clontarf are at the time of writing). That is, guidance is provided where the sand above the toe of the seawall will be submerged at the commencement of an extreme storm event.

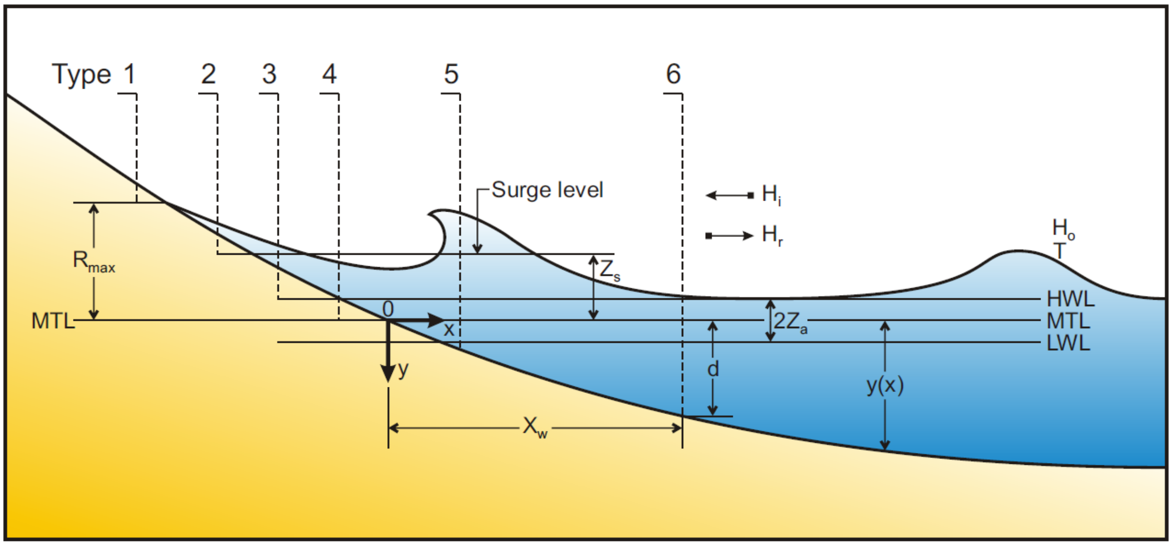

Figure 2.7 Seawall Location According to Weggel Classification
(Source: Weggel 1988)

Table 2.11 Weggel Seawall Classification
(Source: Weggel 1988)

Type	Location of Seawall
1	Landward of maximum level of runup during storms. The wall does not affect either hydraulic or sedimentation processes under any wave or water level conditions, although may affect aeolian processes
2	Above still water level of maximum storm surge and below the level of maximum runup. Exposed only to the runup of waves during storm events
3	Above normal high water and below the still water level of storm surge. Base will be submerged during storms and during exceptionally high astronomical tides but will normally be above water
4	Within the normal tide range; base is submerged at high water
5	Seaward of mean low water; base is always submerged; subjected to breaking and broken waves
6	So far seaward that incident waves do not break on or seaward (of the wall)

Numerical models such as SBEACH (Storm-induced BEach Change) can be used to estimate scour levels in front of seawalls during extreme storm events. It should be noted that while SBEACH does not model wave reflection processes from vertical walls in detail, good agreement with full scale physical model results and other numerical models which include wave reflections has been demonstrated for SBEACH (McDougal et al., 1996).

Since CEM (2006) outlines design recommendations where seawalls are located below the level of storm surge in areas typically dominated by local seas, such as Clontarf, WRL has generally adopted its readily applicable empirical technique for this study. CEM (2006) suggests that the scour depth in front of each of the seawalls is approximately equal to the maximum unbroken incident wave height. When photogrammetry data and/or numerical predictions were also available from the literature, the most conservative scour level prediction has been adopted. Table 2.12 presents estimates of the 100-year ARI scour depth at the toe of the three seawalls in Clontarf. Scour levels were then inferred by applying the scour depth estimate to the long-term weighted average sand level against the seawall.

Table 2.12 100-year ARI Scour Depth Predictions at Clontarf Seawalls

Seawall	Toe Level (m AHD)	(1)Pre-Storm Beach Level at Wall Toe (m AHD)	Predicted Scour for 100-year ARI		Undermining
			Depth(2) (m)	Level(1) (m AHD)	
1	-0.4	0.0	0.4	-0.4	no
2	0.3	1.1	0.3	0.8	no(3)
3	1.0	1.3	0.4	0.9	yes

Notes:
(1) calculated from long-term average sand level against seawall
(2) based on the most conservative predictions using CEM (2006) recommendations and/or photogrammetry analysis and/or numerical modelling predictions (when available from literature)
(3) if pool enclosure is not dredged

Based on the scour predictions, the seawall between Sandy Bay and Clontarf Marina (Seawall 1) is not at risk of failure from undermining during the 100-year ARI storm event (equivalent to 1% annual exceedance probability, AEP). The seawall at the pool enclosure (Seawall 2) will also not be undermined provided that dredging of the pool is not undertaken to critical toe levels. Based on the assumption that the toe level of Seawall 3 is 1.0 m AHD, this structure is at risk of failure from undermining during the 100-year ARI storm event.

While WRL has selected the 100-year ARI event as the 'design' condition for each of the seawalls, scour for more frequent storms (1-, 10- and 50-year ARI events) was also assessed and is presented in Table 2.13. Based on the scour predictions, the seawall between Sandy Bay and Clontarf Marina (Seawall 1) is not at risk of failure from undermining during any present day storms up to 100-year ARI. In the scenario of no dredging being undertaken at the pool enclosure, Seawall 2 would not be at risk of undermining even for the 100-year ARI storm event (1% AEP). The seawall at the Monash Crescent public access (Seawall 3) is not at risk of failure from undermining during the 1- and 10-year ARI storm events, but is for the 50-year ARI event.

Table 2.13 Scour Depth Predictions at Clontarf Seawalls for a Range of ARI

Seawall	Predicted Scour Depth (m)			Predicted Scour Level (m AHD)		
	50-yr ARI	10-yr ARI	1-yr ARI	50-yr ARI	10-yr ARI	1-yr ARI
1	0.4	0.3	0.2	-0.4	-0.3	-0.2
2	0.3	0.2	0.2	0.8	0.9	0.9
3	0.4	0.1	0.0	0.9	1.2	1.3

Figure 2.8 presents for each seawall plots of:

- predicted scour levels for the 1-, 10-, 50- and 100-year ARI storm events
- estimated seawall toe levels, and
- minimum sand levels against the seawall as derived from photogrammetry analysis.

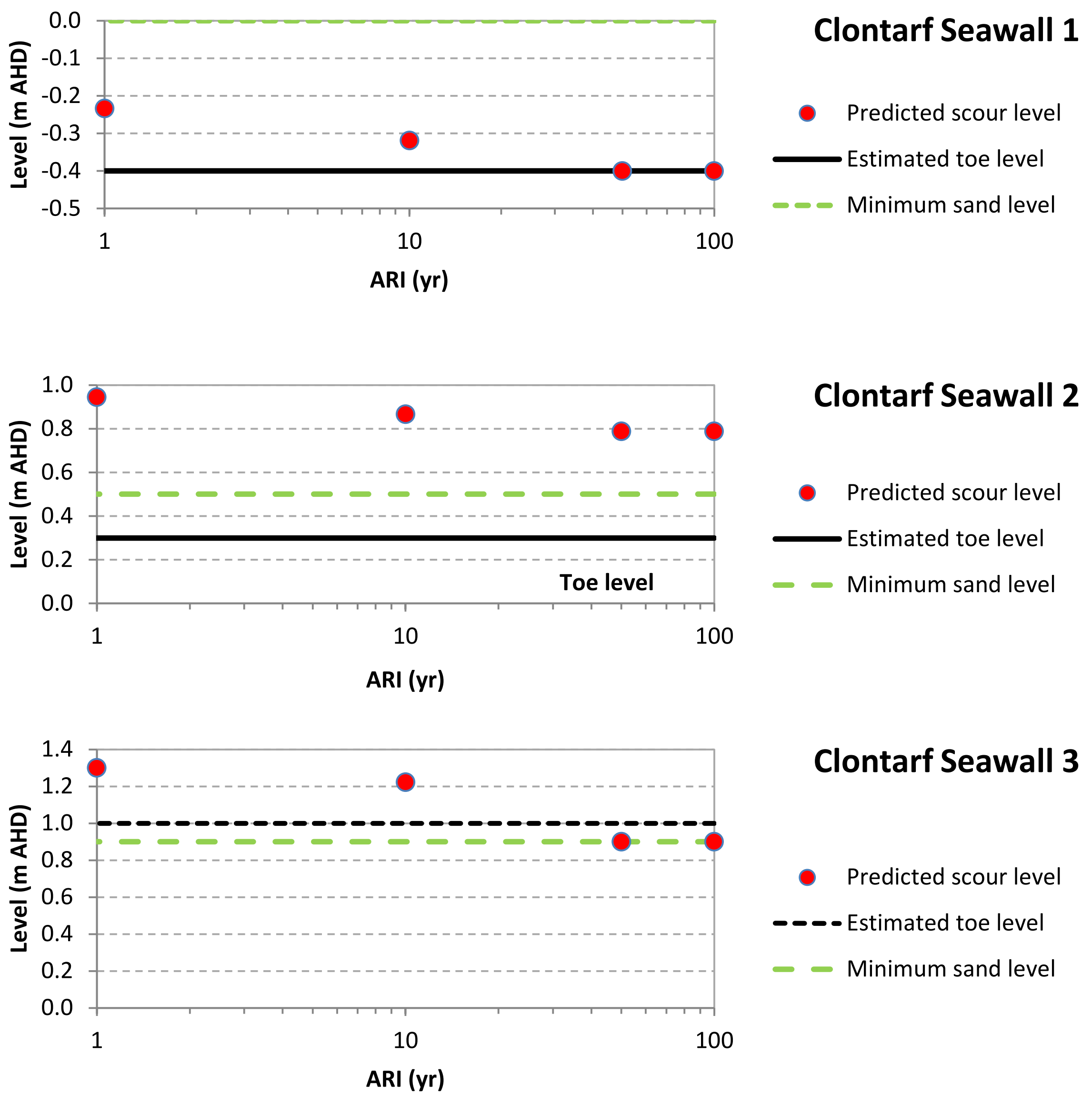

Figure 2.8 Comparison of Predicted Scour Levels with Existing Toe Levels

2.5.3 Wall Stability Under Wave Action

For existing earth-backed, rigid masonry walls such as each of the seawalls at Clontarf, there are no well-known empirical methods for assessing wall stability with respect to direct wave impact. A multi-disciplinary approach would be required involving physical modelling, geotechnical engineering and structural engineering to assess such structures. As such, quantitative assessment of the stability of the Clontarf seawalls was beyond the present scope of works. Observations of historical damage due to wave impact at similar structures within Manly LGA indicate that the most likely damage is dislodgement of the capping stones (Seawalls 1 and 3). Provided that the mortar joints and block integrity are adequately maintained and overtopping is kept within reasonable limits (if unpaved) so that wave impacts do not erode backfill, damage due to wave action is considered to be a lower risk than geotechnical failure modes (such as sliding, overturning or slip circle failure), however, it cannot be entirely excluded. This would require more detailed structural engineering analysis.

2.5.4 Wave Overtopping

Wave overtopping of seawalls is caused by direct (and often violent) impact of waves on the structure. Wave impacts can cause damage to the structure, in particular to freestanding parapets and concrete cappings. More importantly, the water discharged over the seawall crest constitutes a hazard to people and properties located directly behind the seawall.

While empirical estimates of overtopping for coastal structures have improved significantly over the last decade, the available methods are still only useable to provide order of magnitude estimates or for relative comparison purposes. The state-of-the-art empirical technique for estimating overtopping is the EurOtop (2008) 'Overtopping Manual'. WRL has compared predictions of overtopping determined using the methods set out in the manual with several coastal structures physically modelled in wave flumes, and found that in general, the Overtopping Manual provides reasonable predictions (Mariani et al., 2009). However, where precise estimates are required, site specific physical modelling is still recommended.

Overtopping was quantified in terms of volume of water being discharged over the seawall crest and expressed in L/s per metre length of crest. As described in Mariani et al. (2012), wave overtopping was estimated for each structure taking into account the following factors:

- structural characteristics of the seawalls (construction type, crest level, slope etc.)
- design scour levels for the seawalls as derived in Section 2.5.2
- nearshore wave conditions, i.e. wave height and period as derived in Section 2.4.3, and
- elevated water incorporating tides, storm surge and wave setup.

The estimated overtopping rates refer to the zone immediately behind the structure crest and can be related to the published tolerable rates (CEM, 2006, EurOtop, 2008) with regard to structural and people safety. Ranges of mean tolerable overtopping rates for hazards relevant to the study area are presented in Table 2.14 (EurOtop, 2008).

Table 2.14 Limits for Tolerable Mean Wave Overtopping Discharge
(EurOtop, 2008)

Hazard type	Mean Overtopping Discharge (L/s per m)
Aware pedestrian and/or trained staff expecting to get wet	0.1 to 10
Damage to paved promenade behind seawall	200
Damage to grassed promenade behind seawall	50
Structural damage to seawall crest	200
Structural damage to building	(1) 1

Notes: (1) this limit relates to the effective overtopping defined at the building

Table 2.15 presents predicted overtopping rates at each seawall for the design 100-year ARI event. Table 2.16 summarises overtopping rate estimates for 1-, 10- and 50-year ARI events. For the 50- and 100-year ARI events, the seawall between Sandy Bay and the Marina (Seawall 1) will be totally inundated and could suffer structural damage to the crest and paved/grassed areas behind the seawall. Wave overtopping during all modelled storm events would present a hazard for people and vehicles transiting in proximity of the crest of Seawall 1, although it would not be inundated in the 1- and 10-year ARI events. For all modelled storm events except the 1-year ARI event, the structure at the pool enclosure (Seawall 2) would be totally inundated and could suffer structural damage to the crest and paved/grassed areas behind the seawall. Wave overtopping during all modelled storm events would present a hazard for people and vehicles transiting in proximity of the crest of Seawall 2, except for the 1-year ARI event when travel would not be unsafe for vehicles. Seawall 3 (at the Monash Crescent public access would not be inundated for all modelled storm events. However, wave overtopping would again present a hazard for people in proximity of the crest of Seawall 3.

Table 2.15 Predicted Overtopping Rates at Clontarf Seawalls for 100-year ARI Storm Event

Seawall	Crest Level (m AHD)	Hs (m)	Tp (s)	Inundation Level (m AHD)	Q(2) (L/s per m)	(1)Hazard to Pedestrians	(1)Structural Damage
1	1.5	0.4	12.3	1.5	t.i.	very dangerous	yes
2	1.4	0.4	12.3	1.5	t.i.	very dangerous	yes
3	1.9	0.6	13.6	1.5	53	very dangerous	minor

Notes:
Based on EurOtop, 2008
t.i. = totally inundated (i.e. inundation level is greater than or equal to seawall crest level)

Table 2.16 Predicted Overtopping Rates for 1-, 10- and 50-year ARI Storm Events

Seawall	Q (L/s per m)		
	1-yr ARI	10-yr ARI	50-yr ARI
1	13	68	t.i.
2	4	t.i.	t.i.
3	3	14	40

Note: t.i. = totally inundated (i.e. inundation level is greater than or equal to seawall crest level)

2.5.5 Sea Level Rise Impacts

Following assessment of each of the seawalls with regard to their suitability to withstand the occurrence of the present-day storm conditions, WRL considered the stability of the structures for the 2050 and 2100 planning horizons. The same methodology was used to re-assess the likelihood of undermining, structure instability and erosion of the backfill including effects of projected sea level rise. Sea level rise was accounted for in two ways in the revised analysis:

(1) increasing still water levels in accordance with NSW benchmarks, and

(2) adjusting the average beach profiles to account for recession due to sea level rise.

The most widely known model for beach response to future sea level rise is that of Bruun (1962, 1983 and 1988). The Bruun model (as separately defined from the *Bruun Rule*) assumes that as sea level is raised, the equilibrium profile is moved upward and landward conserving mass and original shape. The limitations of this methodology are well recognised (Ranasinghe et al., 2007), particularly on an estuarine or harbour beach, and were taken into consideration. However, no robust and scientifically recognised alternative currently exists, and the application of the Bruun rule is currently supported by State Government Policy (DECCW, 2010). Accordingly, to re-assess scour levels in 2050 and 2100, WRL raised and receded each of the long-term weighted average beach profiles by the values shown in Table 2.17. The average beach profile seaward of Seawall 3 was also receded further to the values shown in Table 2.17 to account for ongoing underlying recession at this location due to sediment loss (Tables 2.18 and 2.19). Note that the 'Bruun Factor' is essentially a cross-shore slope. Response to sea level rise is then simply a translation of the existing nearshore profile up a regional slope. Where the adjusted profile intersected with the face of each seawall (for non-buried structures), the profile landward of the seawall remain unchanged for the analysis.

Table 2.17 Beach Profile Response to Sea Level Rise Projections

Planning Period (year)	(1)Sea Level Rise above 1990 Level (m)	(2)Sea Level Rise above Present Level (m)	Vertical Profile Rise (m)	'Bruun Factor' (-)	Horizontal Profile Recession (m)
2050	0.40	0.33	0.33	11.0	3.7
2100	0.90	0.83	0.83	11.0	9.2

Table 2.18 Beach Profile Response to Underlying Sediment Loss

Seawall	Ongoing Underlying Recession due to Sediment Loss		
	Horizontal Recession Rate (m/yr)	2050 (m)	2100 (m)
1	0.0	0.0	0.0
2	0.0	0.0	0.0
3	0.1	3.8	8.8

Table 2.19 Total Allowance for Underlying and Sea Level Rise Recession (Horizontal)

Seawall	2050			2100		
	Underlying (m)	SLR (m)	Total (m)	Underlying (m)	SLR (m)	Total (m)
1	0.0	3.7	3.7	0.0	9.2	9.2
2	0.0	3.7	3.7	0.0	9.2	9.2
3	3.8	3.7	7.5	8.8	9.2	18.0

While obviously influencing beach recession, sea level rise will also exacerbate the process of erosion on sandy beaches. Further to any reduction in pre-storm sand levels at the toe of the seawalls from recession to due sea level rise and underlying recession, increased still water levels will also allow higher waves to impact the structures and generate deeper scour compared to present-day conditions. Accordingly, the risk of seawall instability due to undermining will increase over time. Table 2.20 summarises estimates of scour levels for 2050 and 2100.

Table 2.20 Scour Level Predictions for 2050 and 2100 Planning Horizons

Seawall	Toe level (m AHD)	2050 Predicted Scour Levels (m AHD)				2100 Predicted Scour Levels (m AHD)			
		1-yr ARI	10-yr ARI	50-yr ARI	100-yr ARI	1-yr ARI	10-yr ARI	50-yr ARI	100-yr ARI
1	-0.4	-0.6	-0.7	-0.8	-0.8	-1.1	-1.2	-1.3	-1.3
2	0.3	0.4	0.2	0.1	0.1	-0.1	-0.3	-0.4	-0.4
3	1.0	-0.1	-0.3	-0.4	-0.4	-1.7	-1.9	-2.0	-2.0

Based on the scour predictions, Seawalls 1, 2 and 3 are at risk of failure from undermining during all modelled 2050 and 2100 storms, except for Seawall 2 in the 1-year ARI event in 2050. It should be noted that by 2050, Seawall 3 is at risk of undermining from recession (underlying and due to SLR) alone; additional scour from storms will not be required to cause failure. By 2100, all of the seawalls at Clontarf are at risk from undermining from recession alone. Note that it has been assumed that the sandy foreshores fronting Seawalls 1 and 2 are stable or accreting under present-day conditions (underlying recession is zero). As such, the estimated time to failure for these walls is considered conservative as any long-term underlying accretion is likely to counteract recession due to SLR.

No assessment of wall stability has been undertaken for the seawalls at Clontarf due to the nature of the walls. However, note that Seawall 1 was built during the 1950s and would have a service life of approximately 100 years, which would be reached by 2050 and exceeded by 2100. The year of construction for Seawalls 2 and 3 remains unknown.

Due to their low crest levels, with the projected sea level rise for the 2050 and 2100 planning horizons, the three assessed seawalls will be subject to inundation more frequently. Table 2.21 summarises estimates of mean overtopping rates for 2050 and 2100. Each of the seawalls would be totally inundated in all modelled events in 2050 and 2100 (except for Seawall 3 in the 1- and 10-year ARI events in 2050) and could suffer structural damage to the crest and paved/grassed areas behind the seawall due to overtopping bores. While Seawall 3 is not likely to be inundated by the 1- and 10-year ARI events in 2050, it is predicted that the wall will suffer structural damage to the crest and grassed area in its lee. Wave overtopping during all modelled events in 2050 and 2100 would present a hazard for people and vehicles (if access permits) transiting in proximity of the seawall crests.

Table 2.21 Predicted Overtopping Rates for 2050 and 2100 Planning Horizons

Seawall	2050 Predicted Overtopping Rates L/s per m)				2100 Predicted Overtopping Rates (L/s per m)			
	1-yr ARI	10-yr ARI	50-yr ARI	100-yr ARI	1-yr ARI	10-yr ARI	50-yr ARI	100-yr ARI
1	t.i.	t.i.	t.i.	t.i.	t.i.	t.i.	t.i.	t.i.
2	t.i.	t.i.	t.i.	t.i.	t.i.	t.i.	t.i.	t.i.
3	90	196	t.i.	t.i.	t.i.	t.i.	t.i.	t.i.

Note: t.i. = totally inundated (i.e. inundation level is greater than or equal to seawall crest level)

2.5.6 Summary

WRL has undertaken a detailed assessment for each of the three seawalls at Clontarf with regard to their suitability to withstand the occurrence of 1-, 10-, 50- and 100-year ARI events for present-day conditions and for the 2050 and 2100 planning horizons, including sea level rise projections. The likelihood of failure of the seawalls was assessed for undermining, structure instability (qualitative only) and erosion of the backfill (wave overtopping) though excluded geotechnical considerations. Each of the three structures was found to be at risk of failure by one of the assessed modes under present-day design conditions. A summary detailing if structural failure is likely to occur is presented in Table 2.22 for each seawall within each planning period and for each storm event considered.

Table 2.22 Summary of Seawall Assessment for Present Day, 2050 and 2100 Planning Horizons

Planning Horizon	ARI	Structural Failure					
		Seawall 1		Seawall 2		Seawall 3	
		Scour	OT	Scour	OT	Scour	OT
Present Day	1	no	no	no	no	no	no
	10	no	no	no	yes	no	no
	50	no	yes	no	yes	yes	no
	100	no	yes	no	yes	yes	no
2050	1	yes	yes	no	yes	yes	no
	10	yes	yes	yes	yes	yes	yes
	50	yes	yes	yes	yes	yes	yes
	100	yes	yes	yes	yes	yes	yes
2100	1	yes	yes	yes	yes	yes	yes
	10	yes	yes	yes	yes	yes	yes
	50	yes	yes	yes	yes	yes	yes
	100	yes	yes	yes	yes	yes	yes

To consider the likely time to failure for each of the seawalls at Clontarf it is helpful to consider the probability of encountering one of the modelled storm events (1-, 10-, 50- and 100-year ARI). The 'encounter probability' is the chance of a given ARI event occurring during the service life a seawall. Tabulated values for this probability are presented in Table 2.23. Ignoring the variable length of service already provided by each of the seawalls (i.e. commencing calculations from 2012), it can be seen that the probability of encountering 1-and 10-year ARI events by 2050 and 2100 is near certain. However, the probability of encountering a 100-year ARI storm event by 2050 is 32% and by 2100, 59%. While this analysis has its limitations, it is useful for considering the planning horizon over which failure may be likely. Note that this technique assumes that the consequences of each respective ARI storm are stationary. As noted earlier, this is not the case under sea level rise projections and the consequences (and hence risk) for a given ARI storm will increase with time.

Table 2.23 Encounter Probability for ARI Event and Service Life

ARI		Encounter Probability (%) for ARI Event (years)			
		1	10	50	100
AEP		63.21	9.52	1.98	1.00
Service Life (Years)	Calendar Year				
1	2013	63.21	9.52	1.98	1.00
2	2014	86.47	18.13	3.92	1.98
5	2017	99.33%	39.35	9.52	4.88
10	2022	100.00	63.21	18.13	9.52
20	2032	100.00	86.47	32.97	18.13
38	2050	100.00	97.76	53.23	31.61
50	2062	100.00	99.33	63.21	39.35
88	2100	100.00	99.98	82.80	58.52
100	2112	100.00	100.00	86.47	63.21

Considering the mechanism by which structural failure is first likely to occur and the likely time to failure (to within a planning horizon) for each of the seawalls at Clontarf, a summary presenting the most likely outcomes is presented in Table 2.24. This indicates that wave overtopping is the most likely failure mechanism for Seawalls 1 and 2. Structural instability due to toe undermining is the predicted failure mechanism for Seawall 3. The time to failure for each of the seawalls is expected to be within the Present Day to 2050 planning horizons.

Table 2.24 Predicted Failure Mechanism and Timing for Clontarf Seawalls

Seawall	Location	Predicted Failure	
		Mechanism	Timing
1	Between Sandy Bay and the Marina	Overtopping	Present Day-2050
2	At pool enclosure	Overtopping	Present Day-2050
3	Public access from Monash Crescent	Toe Undermining	Present Day-2050

It is also helpful to qualitatively consider the likely outcomes from a major storm occurring at present and within the 2050 and 2100 planning horizons. Since the 100-year ARI storm is the adopted design event, the likely impacts of such a storm are considered in the following discourse based on the assumptions set out in the preceding sections.

At the present time, a scenario involving a 100-year ARI event would likely involve a single storm with a duration of almost six days. It is assumed that the Clontarf foreshore would be in a relatively accreted state prior to the commencement of the storm. Large, long period waves generated offshore from a low pressure system combined with a high storm surge on a spring tidal cycle would cause extensive erosion of the beach. Gale force onshore winds and intense rainfall would also be expected. The rate of erosion and extent of inundation would be greatest for approximately 2 hours at each high tide (occurring approximately every 12.5 hours).

Undermining of Seawall 1 is not expected, although the predicted scour level will be approximately equivalent to the toe level. However, failure of this structure is expected to be induced by wave overtopping. Seawall 1 is expected to suffer structural damage to the crest and the pavement and bitumen in its lee due to extensive overtopping bores. Ongoing erosion following the failure of Seawall 1 may threaten Sandy Bay Road. Up until the point of failure of Seawall 1, wave overtopping is likely to be unsafe for pedestrians and vehicles near the structure crest. Failure of Seawall 1 may also lead to outflanking of coastal protections works leeward of Clontarf Marina.

The performance of Seawall 2 during the 100-year ARI event is predicted to be generally similar to Seawall 1 except that the scour level is predicted to remain above the toe. Failure of the structure is again expected to be induced by wave overtopping. Ongoing erosion following the failure of Seawall 2 is likely to damage the foreshore at Clontarf Reserve. Up until the point of failure of this structure, wave overtopping is likely to be unsafe for pedestrians near the structure crest.

At the public access from Monash Crescent, Seawall 3 is expected to fail due to toe undermining. Once the sand level in front of the wall is reduced below the toe level, the structure is likely to subside and collapse seaward. Prior to the failure of Seawall 3, wave overtopping is likely to be unsafe for pedestrians near the structure crest. Most importantly, the failure of Seawall 3 may also lead to outflanking of the adjacent private seawalls. This could potentially lead to the progressive unravelling of all coastal protection works along this stretch of foreshore, but it is likely that this would occur over several months following the initial failure of Seawall 3. If this were to eventuate, ongoing erosion following the failure of these works may threaten the leeward buildings on private property (if they are not founded on rock or piled foundations, and if emergency protection were not provided). It is also likely that stormwater drainage systems would be affected by 'quasi-static' inundation (excluding wave runup but including storm surge and wave setup) during the event.

The same scenario involving a 100-year ARI event occurring in 2050 is also considered. The details of the storm event are assumed to be the same except that the mean sea level is higher. It is assumed that the Clontarf shoreline would be approximately 5 m narrower (where underlying recession is zero) than the present day accreted state prior to the commencement of the storm, and that Seawall 1 has not yet failed between 2012 and 2050. Since Seawall 2 is likely to fail due to wave overtopping in events as frequent as the 10-year ARI and Seawall 3 is likely to be undermined from recession alone, it is assumed that by 2050 these structures have failed and/or been replaced by more robust structures designed and constructed to conventional coastal engineering standards. As such, Seawalls 2 and 3 are not considered in this qualitative discussion. For Seawall 1, failure may be induced by toe undermining and/or wave overtopping. Once the sand level in front of the wall is reduced below the toe level, the structure is likely to subside and collapse seaward. The structure is also expected to suffer structural damage to the crest and the pavement and bitumen in its lee due to extensive overtopping bores. Ongoing erosion following the failure of Seawall 1 may threaten Sandy Bay Road.

Finally, the occurrence of a 100-year ARI event in 2100 is discussed. The details of the storm event are again unchanged except that the mean sea level is higher still. It is assumed that the Clontarf shoreline would be approximately 10 m narrower than the present-day accreted state and that all existing seawalls at Clontarf have failed and/or been replaced between 2012 and 2100.

2.6 Remedial Options

2.6.1 Emergency and Short Term

In the event of major erosion of a wall toe or collapse of a seawall, sand-filled geotextile containers (geocontainers) are recommended as an emergency measure to prevent wall undermining or to prevent further loss of the infrastructures defended. The implementation of geocontainers in an emergency situation would require a degree of advance planning.

2.6.2 Medium and Long-term Structural Options

In the longer term, it is recommended that modifications be made to each of the seawalls prior to their exposure to a storm event which is likely to cause structural failure. This should consider the likely time to failure presented in Table 2.24. The following structural options implying upgrade or replacement of the seawall are considered viable:

- rock or piled toe protection would reduce the risk of undermining failure, but (subject to detailed geotechnical investigation) may not prevent other types of geotechnical failure
- a wave return parapet would reduce the risk associated with wave overtopping during storm events
- ensuring a strong pavement and drainage leeward of the crest of the seawalls will assist in reducing damage from overtopping in more frequent (10–50-year ARI) events
- a new seawall, comprising deeper toe levels and/or toe protection and higher crest level, subject to detailed design and stakeholder acceptance
- reinforcing the walls with a new face and deeper toe
- changing the conditions which cause erosion (installation of offshore breakwater and/or beach nourishment).

Under present-day conditions, it is recommended that wave return parapets could be added leeward of the crests of Seawalls 1 and 2 to reduce the risk associated with wave overtopping during a design event. These additional protective works could also have a secondary function as a landscape feature. For Seawall 1, this feature could be incorporated with the pavement design landward of the crest. Seawall 3 has a high toe level and is at risk of toe undermining, its service life could be extended with the addition of rock or piled toe protection.

Additional works could be planned for the 2050 planning horizon. It is recommended that a new seawall be constructed to replace Seawall 1 as it has reached the end of its design life. For Seawall 2, the addition of rock or piled toe protection is recommended and a wave return parapet is also suggested to be added to Seawall 3.

Other than the installation of an offshore breakwater and/or beach nourishment, no remedial upgrade options are likely to be practical to mitigate the risk of structural failure for Seawalls 2 and 3 for the 2100 planning horizon. It is recommended that new seawalls be constructed to replace these structures in time to come, preferably before the structures become damaged and impromptu emergency works are required.

2.7 Future Management

The coastal protection structures at Clontarf have been assessed as a case study of existing seawalls on an estuarine or harbour beach. From this review, generic information to inform local government in managing and assessing seawall structures may be derived. It is recommended that the following aspects of existing structures be measured, monitored and recorded to assist in future management of harbour beach seawalls and adjacent beach areas:

- seawall crest level
- seawall toe level
- sand level at the seawall
- slope of seawall (if non-vertical)
- estimated rock dimensions and type (if flexible rubble mound seawall)
- condition of blockwork and mortar (if masonry seawall)
- estimated depth and propagation distance for overtopping bores during a storm
- presence of rock scour protection at the toe, and
- presence of bedrock below seawall toe.

Seawall managers are encouraged to maintain photographic and written records of significant storm events (including an inventory of any damage). Further to this information, if an estuarine beach seawall is to be assessed to the same level of detail as that for the case study at Clontarf, the following information should be collected:

- any available drawings (conceptual, detailed design, 'as-built')
- extent of structure in plan (including outflanking protection)
- extent of structure below the sand level (i.e. test pit excavation, borehole drilling, GPR)
- characteristic sand sample from the beach (mean grain size diameter)
- cross-shore photogrammetry profiles and/or recent beach survey by traditional means
- information regarding human intervention in beach processes (dredging/nourishment)
- nearshore bathymetric data, and
- confirmation of sub-surface structure (multiple rock layers, geotextile underlayer).

However, in practical terms it should be noted that there are no well-known empirical methods for assessing wall stability for existing earth-backed, rigid masonry walls with respect to direct wave impact. A multi-disciplinary approach would be required involving physical modelling, geotechnical engineering and structural engineering to assess such structures.

For an existing flexible rubble mound seawall, it is recommended that a physical modelling program be undertaken to provide a more robust assessment of its stability rather than the use of empirical techniques alone (though these empirical techniques are not unreasonable).

When existing seawalls (such as earth-backed, rigid masonry structures and flexible rubble mound structures) are examined rigorously, significant unknowns are likely to remain, requiring various assumptions to be made. Accordingly, it is unlikely that a professional coastal engineer would issue a certificate declaring the suitability of such a structure (including the three seawalls at Clontarf) to provide adequate protection during a design event, without clarification of these unknowns.

2.8 Conclusions

The assessment of three existing seawalls located along the Clontarf foreshore was undertaken with regard to storm erosion and inundation hazards. The structures were characterised in terms of the most relevant engineering features based on the consultation of available literature and field investigation reports. Environmental conditions were defined for the Clontarf coastline, in particular design wave and water level conditions were established at each structure. The absence of toe protection and the high toe levels generally made the structures vulnerable to undermining due to storm erosion and ongoing recession. The risk associated with the reduction of sand levels at the toe of the structures is likely to be exacerbated by sea level rise. Wave overtopping considering the safety of people and seawall structural integrity was also assessed. The inundation hazard was also estimated to be very relevant for these structures due to the low crest levels. Recommendations were provided for remedial options for each seawall as well as generic advice for the management of estuarine beach seawalls.

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Sydney Coastal Councils Group Inc.

councils caring for the coastal environment

Level 14, Town Hall House

456 Kent Street

GPO Box 1591

SYDNEY NSW 2001

t: +61 2 9246 7326

f: +61 2 9265 9660

e: info@sydneycoastalcouncils.com.au

w: www.sydneycoastalcouncils.com.au.com.au