Basis of Design - Beachmere A-Line

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1 Introduction

1.1 Project Definition

The Beachmere A-Line project concerns the development of a 'clear pathway for the approval, construction and maintenance of private coastal protection structures at Beachmere', with a key focus being the practicable protection of natural coastal processes, vegetation and overall foreshore amenity within and adjacent to the project site. The 'A-Line' refers to the continuous alignment of seawall crests extending along the study area shoreline.

This document establishes the rock revetment seawall design basis and develops design criteria for seawall cross-section design, incorporating functional considerations for the area, and is intended to support the A-Line alignment development, and associated policy and implementation.

1.2 Project Background

BMT contributed to a Shoreline Erosion Management Plan (SEMP) for Northern Moreton Bay in 2013-14. Beachmere was identified as a location where the threat of short-term erosion associated with storms was greatest, given the number of developed private and public lots that are situated at the shoreline. Currently the shoreline is semi-protected, with various staggered public and private seawalls constructed along its length, as well as unprotected beach sections which are characterised as both eroding and healthily vegetated. A component of the SEMP recommended the development of a continuous shoreline protective measure.

1.3 Project Objectives

The final deliverable will be to provide the alignment (the so-called A-Line), standard seawall configurations, and approvals pathway for a shoreline protection plan that encompasses the length of Beachmere foreshore.

1.4 Use

The Basis of Design Report (BoD) is a live document for the seawall design. Each design criteria and/or reference in this document will be evaluated and clarified with/by the Client and confirmed in the Final Issue. This document summarises the available information required for project definition to Detailed Design level.

2 General

2.1 Abbreviations

AHD	Australian Height Datum
ARI	Average Recurrence Interval (also often referred to as return period)
AEP	Annual Exceedance Probability
BoD	Basis of Design
CD	Chart Datum
GFRP	Glass Fibre Reinforced Polymer
HAT	Highest Astronomical Tide
Hmax	Maximum wave height
Hsig	Significant wave height
LAT	Lowest Astronomical Tide
MHWN	Mean High Water Neaps
MHWS	Mean High Water Springs
MLWN	Mean Low Water Neaps
MLWS	Mean Low Water Springs
MSL	Mean Sea Level
RL	Reduced Level
SLR	Sea Level Rise
Tm	Mean wave period
Tp	Peak wave period

2.2 Units

Metric system units shall be used for all aspects related to this study and concept design.

3 Relevant Standards and Reference Documents

All aspects of this design consider the requirements of the latest versions of relevant Australian statutory documentation.

The precedence applying for use of the Codes, Standards, Specifications and Regulatory requirements for this project is as follows:

- Regulatory Requirements
- Project Specific Specifications and Standards
- Australian Standards
- International Standards
- Service Authority Standards.

In the event of an inconsistency, conflict, or discrepancy between any of the Standards, Specifications or Regulations, the most stringent and safest requirement applicable to the project will prevail to the extent of the inconsistency, conflict or discrepancy. It is assumed any inconsistencies critical to the design would be brought to the attention of the relevant Project Manager for resolution.

3.1 Statutory guidelines and requirements

The design should conform to the requirements of the following statutory guidelines, including the relevant MBRC Shoreline Erosion Management Plan (SEMP):

- Department of State Development, Infrastructure, Local Government and Planning (2022) State Development Assessment Provisions v3.0 - State code 8: Coastal development and tidal works
- Department of Environment and Science (2022) Guideline: State Development Assessment Provisions - State Code 8: Coastal development and tidal works.
- Department of Environment and Heritage Protection (now Department of Environment and Science). (2013). Operational Policy (*Coastal Protection and Management Act 1995*) - Building and Engineering Standards for Tidal Works.

3.2 Standards and Codes

Unless otherwise specified, the design is to be in accordance with the following relevant standards and guidelines:

- AS4997 Design of Maritime Structures
- AS2758.6 Aggregates and rock for engineering purposes – Guidelines for the specification of armourstone
- AS1170.0 Structural Design Actions – General Principles
- AS1170.1 Structural Design Actions – Permanent, imposed and other actions
- AS1170.2 Structural Design Actions – Wind Loads
- AS1170.4 Structural Design Actions – Earthquake actions in Australia
- AS4678 Earth retaining structures
- AS3600 Concrete Structures
- AS4100 Steel Structures

- AS1428 Design for Access and Mobility
- AS1657 Fixed platforms, walkways, stairways, and ladders
- Queensland Prescribed Tidal Works Code (Coastal Protection and Management Regulation 2017)
- BS6349 Maritime Structures code

3.3 Guidelines and references

The following provides a list of relevant background information, design guidance manuals, and other references relevant to the project:

- Coastal Engineering Manual (CEM). United States Army Corps of Engineers (USACE) (1984-2003)
- CIRIA, CUR, CETMEF - 2007. The Rock Manual - The use of rock in hydraulic engineering. C683, CIRIA London.
- Eurotop (2018). Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application. Van der Meer, J.W., Allsop, N.W.H., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schüttrumpf, H., Troch, P. and Zanuttigh, B., www.overtopping-manual.com
- Goda, Y. (2010). Random Seas and Design of Maritime Structures. World Scientific.
- PIANC MarCom WG 162 - 2016: Recommendations for Increased Durability and Service Life of New Marine Concrete Infrastructure.
- Queensland Urban Drainage Manual - 4th edn., Institute of Public Works Engineering Australia.
- Shore Protection Manual (1984). Waterways Experiment Station, Corps of Engineers, PO Box 631. Department of the Army - US Army Corps of Engineers.
- Van Der Meer. (1998). Application and Stability Criteria for Rock and Artificial Units.

3.4 Project -specific studies and drawings

The following data are available to BMT for this scope of works, with the relevant sources provided in the Appendices.

- Storm Tide Studies:
  - Cardno, 2017. Storm Tide Study Report.
  - Cardno, 2019. Storm Tide Inundation Modelling.
- Geotechnical Data:
  - Douglas Partners, 2013. Report on Geotechnical Investigation – Proposed Drainage Channel – Bay Avenue, Deception Bay
  - Coffey Geotechnics, 2009. Proposed Jetty, Outlet and Boardwalk – Beachmere – Geotechnical Investigations
  - AECOM, 2012. Beachmere Geotechnical Investigation – Site Sketch.
- Survey:
  - 2019 LiDAR survey of Beachmere – Provided by MBRC 10/06/2021
  - 2014 LiDAR survey – Extracted for Beachmere from ELVIS Elevation and Depth Database 03/06/2021

    - GBR30 bathymetry, Geosciences Australia – Extracted for study area from ELVIS Elevation and Depth Database 10/06/2021
    - 2019 Photogrammetry survey ortho-mosaic and DEM, survey dated 23/10/2019 covering Biggs Ave to Bishop Rd. Provided by MBRC 11/06/2021.
    - Survey of Biggs Ave public seawall (files dated 2017). Provided by MBRC 11/06/2021.
    - 2022 drone-based lidar survey – refer Drawing 401847-SK02 (Veris, 2022)
- MBRC Regional Seawall Condition Database
- Historical approvals records – 20 approvals from 1967 – 1999 under Section 86 of the Harbours Act.

3.5 Approvals

The A-line seawall structures are 'prescribed tidal works' and therefore require a Development Permit under the *Planning Act 2016* and *Coastal Protection and Management Act 1995*. To obtain approval, structures need to meet standard criteria which are set out in the following:

- *Coastal Protection and Management Regulation 2017* Schedule 3 Code for assessable development that is prescribed tidal works
- State Development Assessment Provisions State code 8: Coastal development and tidal works
- Moreton Bay Planning Scheme:
    - 8.2.1 Coastal hazard overlay code
    - 9.4.2 Works code.

For specific items within the Schedule 3 code, compliance can be demonstrated only where works are appropriately certified. This requires certification by a Registered Professional Engineer of Queensland (RPEQ) in the area of civil, environmental, geotechnical or structural engineering. This certification can be at the point of design but is also required at the point of construction.

4 Characterisation of existing protection

The shoreline at Beachmere is approximately 7km long on mainland Queensland within Northern Moreton Bay. The beaches are tidally dominated, and characterised by large intertidal banks, narrow beaches and sporadic mangrove colonies. There are various shoreline protection structures in place and contrasting beach segments that are naturally in good condition. During a site visit in June 2021, BMT collected data that is summarised in this section.

199 segments of the shoreline were inspected and recorded, the majority of which aligned with single cadastral land parcels, though some spanned multiple lots (specifically the long public seawall at the southern end of Biggs Avenue). Of these segments, 128 had some sort of shoreline protection works:

- 27 rock wall segments (steep vertical/near-vertical wall with square placed boulders) – see Figure 4.1
- 56 rubble slope segments
- 12 timber wall segments
- 33 segments of other assorted shoreline protection (timber retaining wall, blockwork wall)
- 71 segments identified with no practical protection.


Figure 4.1 Examples of near-vertical gravity type seawalls comprised of placed boulders with mortar (left) and without mortar (right)

Each of the protective structures were given a condition between 1 and 5, as described in Table 4.2. The majority of structures (87%) had a condition rating of 2 or 3, indicating the majority of existing structures have suffered minor or moderate deterioration relative to the as-built condition, and the function is somewhat compromised.

Table 4.2 Distribution of condition ratings for existing structures along the Beachmere shoreline

Condition Rating	Description	Percent of protected shoreline
1	Engineered structure in as-built condition	6%
2	Some deterioration or defects are evident, but function is not significantly affected relative to as-built condition.	46%
3	Serious deterioration in at least some portions of the structure. Function may be inadequate.	39%
4	Extensive deterioration. Barely functional.	2%
5	Dilapidated to the point of offering no protection	7%

Additionally, condition ratings were summarised for each type of coastal protection structure inspected (Table 4.3).

Table 4.3 Distribution of condition ratings by structure type

Condition Rating	Structure type				
	Rock revetment	Concrete seawall	Rubble Slope	Rock Seawall	Other Structure
1	7%	40%	0%	0%	11%
2	85%	40%	41%	100%	20%
3	7%	20%	41%	0%	29%
4	0%	0%	14%	0%	17%
5	0%	0%	4%	0%	23%
Total Count	27	5	56	4	29

Shoreline protection classification has been refined in the structure types in Table 4.3. Rock Revetments denote substantial engineered rock structure, whereas Rubble Slope denotes less formal protection structures constructed of rock. A Rock Seawall is a more vertical structure made from square rocks, while a Concrete Seawall includes sloped and vertical concrete. Other Structures include timber walls/fences, sandbag walls, and other informal, ad-hoc structures including those made from tyres, logs or scattered rubble.

Historical approvals were supplied by MBRC for 21 of the segments, indicating these structures were engineered to standard (reference as-built design drawings for modern approved seawalls are attached in Annex A). Of the remaining seawall segments, there is no confirmation of engineered design.

Sandy buffer protection along the seawall extents was estimated using the 2022 drone survey data. This data was further compared with the 2014 Lidar survey, which captures the intertidal zone to a greater extent than the 2019 Lidar survey. Sub-aerial beach volume offshore from each of the 199 shoreline segments was assessed as the volume greater than 0m AHD, in front of the A-Line position.

The Beachmere A-Line project includes two distinct areas, either side of the Beachmere Conservation Park and adjoining areas:

1. Biggs Avenue to the Sandy Street drain (southern area).
2. Bayside Drive (northern area).

Figure 4.3 and Figure 4.4 illustrate the surveyed variance in sandy buffer for the southern and northern areas respectively, calculated on a series of cross sections extending perpendicularly offshore (Figure 4.2). Volumes of ~30m^2 correspond to a low level single beach slope protection meeting a seawall at ~RL 1.25, which volumes of ~60m^2 are consistent with a more extensive sandy buffer, with dunes rising to ~RL 2.0.

Consistent with visual assessment of beach exposure, it is clear the southern 1km of the Biggs Ave to Bishop Rd reach is relatively exposed with a narrow sandy buffer, and an expectation of limited sheltering from offshore waves impacting the A-Line. North of this extent, there is generally sufficient sandy buffer to limit wave heights under storm surge extremes. The northern area is generally quite well protected, other than the few properties to the south end of Bayside Dr. The Northern Moreton Bay Shoreline Erosion Management Plan – Stage 1 (BMT, 2014) provides further details regarding both the short term storm erosion potential (Section 6.5) and an assessment of shoreline erosion risk (Section 7) for the Beachmere shoreline.

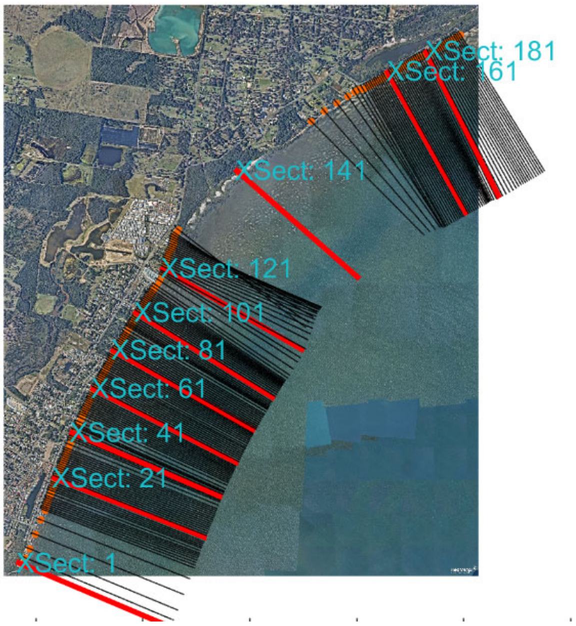


Figure 4.2 Cross sections used for assessment of erosion / accretion

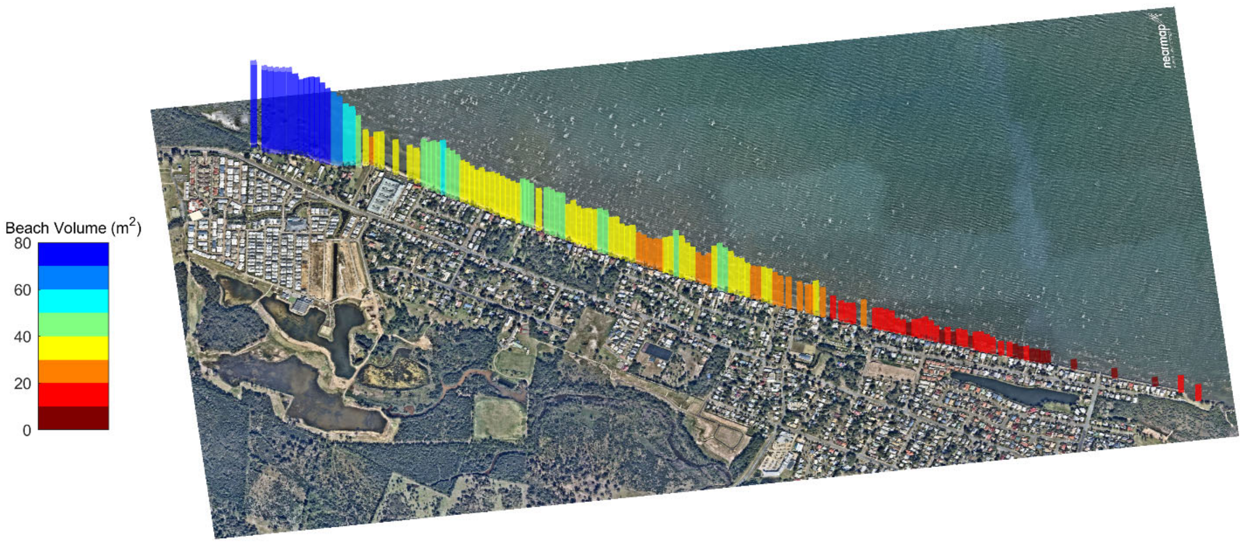


Figure 4.3 Sub-aerial beach volumes (to 2022 survey), Biggs Ave (right) to Bishop Rd (left)

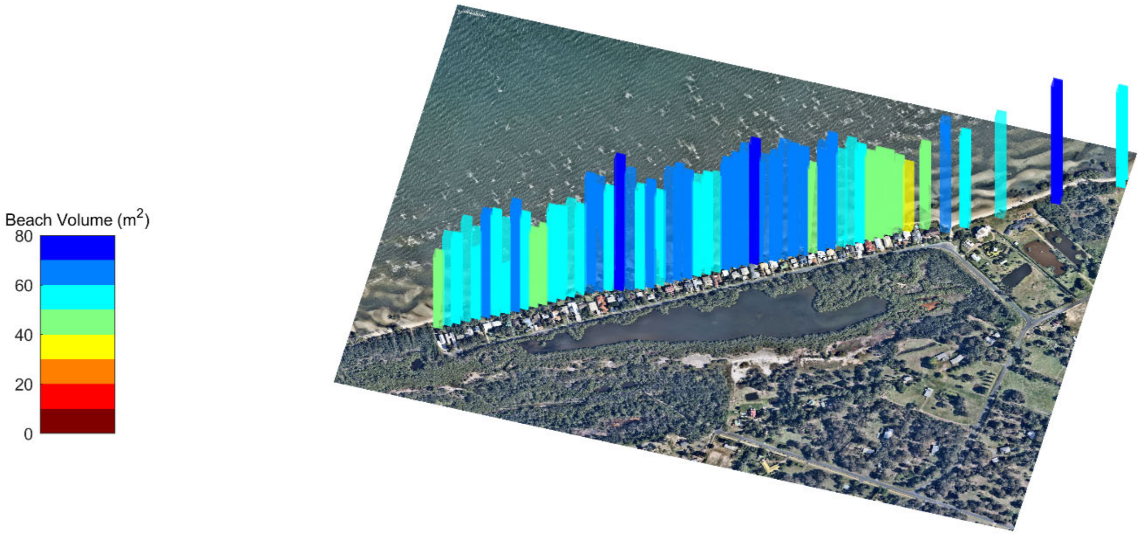


Figure 4.4 Sub-aerial beach volumes (to 2022 survey), Bayside Dr

Between 2014 and 2022, general patterns of erosion and accretion are evident, consistent with the pattern of assessed beach volumes (Figure 4.5).

Towards the southern end of each of the series of private seawalls for Biggs Ave and Bayside Dr, there is notable consistent nearshore erosion, with nearshore accretion to the north of each of these segments. Overall however, the trend is towards sand removal from the beach compartment.

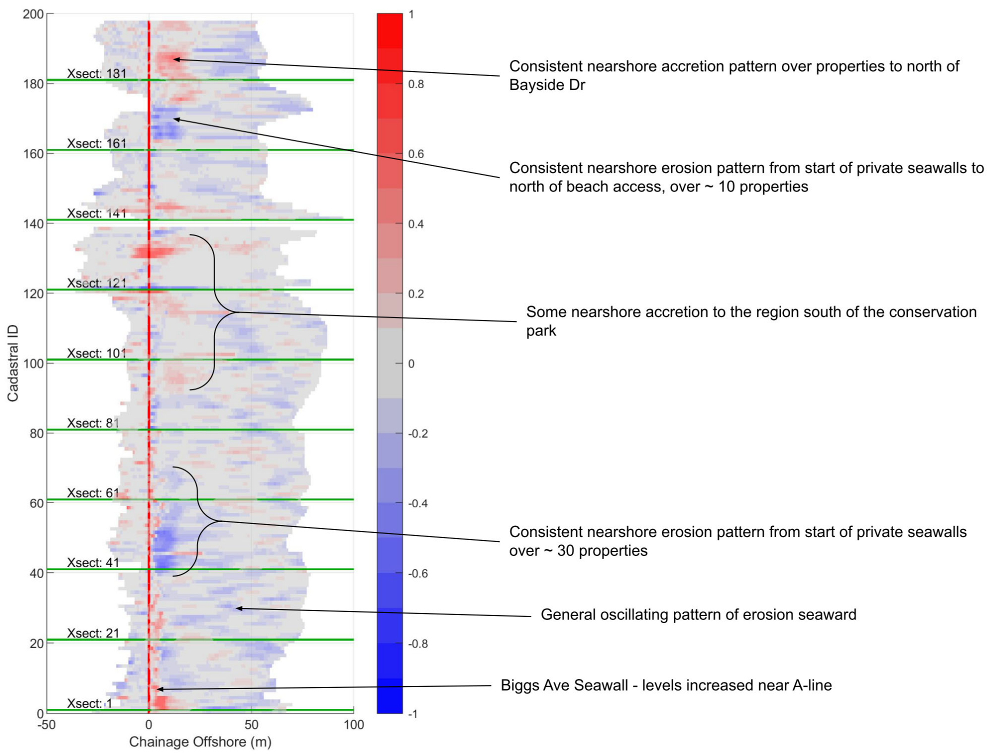


Figure 4.5 Aggregated erosion/accretion pattern on cross sections

5 Functional criteria and constraints

5.1 A-Line alignment and footprint

The southern area adjoins the Moreton Bay Marine Park (MBMP) Deception Bay Habitat Protection Zone (HPZ) while the northern area adjoins the Pumicestone Channel-Godwin Beach Conservation Park Zone (CPZ). Based on the declaration of the MBMP under the *Marine Parks (Declaration) Regulation 2006* Schedule 1, the boundary of the marine park is HAT (s2) but excludes freehold tidal waters and tidal land (s1(3)). Within the exception of road reserves, all properties within both the southern and northern areas are freehold land. Therefore, the MBMP boundary is the seaward boundary of each lot.

The properties in the northern area adjoin a road reserve on their seaward boundary and are understood to have a ‘right-of-line’ boundary. Therefore, the property boundary, and boundary of the marine park, is as mapped on survey plans. By contrast, the properties in the southern area adjoin unallocated state land (USL) and are understood to have an ambulatory boundary which is subject to change with coastal conditions. An indicative ambulatory boundary has been identified by MBRC and the Department of Resource (DoR) based on a desktop assessment. This provides the indicative boundary of the marine park in this area.

The indicative ambulatory boundary along the southern area defined by MBRC and DoR provides the assumed boundary between private and state land. The ambulatory boundary definition generally aligns with existing seawalls (irrespective of approval status) or the vegetation line where a seawall is yet to be constructed. The ambulatory boundary reference was adopted as the A-Line (Figure 5.1).

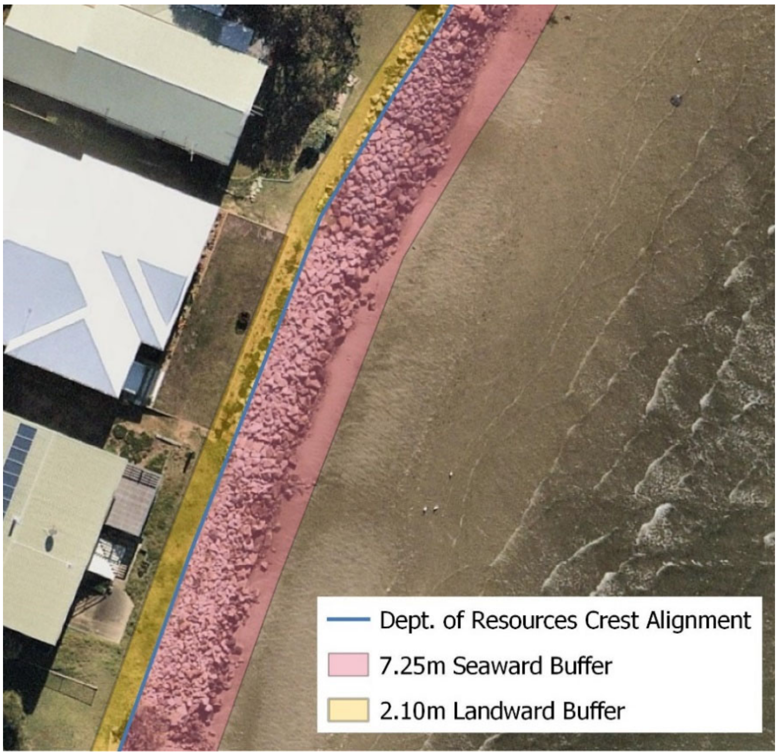


Figure 5.1 Type 2A Seawall Footprint Buffer from Department of Resources Alignment

The A-Line for the properties along Bayside Drive was developed through acknowledgement of existing seawalls, with an aim to situate the seawall footprint entirely on private land where appropriate. This approach was compromised at locations where there were significant existing structures. As seen in Figure 5.2, the southernmost 4 blocks of this segment have existing seawalls. From BMT's site inspection, these were all of good condition (Condition Rating 2), and considering the location of pools on the blocks behind, the A-Line follows the crest of these seawalls. This results in an encroachment of the A-Line total footprint onto the road reserve adjacent these blocks. Over the span of a single cadastral lot, the A-Line then transitions landward to be wholly located on private land.

The A-Line at northern Bayside Drive transitions such that the entire seawall footprint is situated outside the private property boundary (see Figure 5.3). This is due to significant existing lawful structures (e.g. dwellings and pools) located near the seaward lot boundary. A vegetated buffer exists between these properties and shoreline. Seawalls are not expected to be constructed until this vegetated buffer has been eroded.

Figure 5.2 Bayside Drive South A-Line

Figure 5.3 Bayside Drive North A-Line

5.2 Beach Access

Considerations relevant to beach access for both general public and seafaring include:

- Public Amenity – the beach is currently accessed as follows:
  - Biggs Ave South (Public Seawall):
    - 4 x stairways
    - 1 x boat ramp
  - Beachmere Activity Centre:
    - 1 x stairway
  - Beachmere Road Foreshore
    - 1 x stairway
  - McGregor Tce:
    - Beach access path
  - Phillip St:
    - Beach access path
  - Prince St:
    - Beach access path
  - Sandy St:
    - Beach access path
  - Louise Dve:
    - Beach access path
  - Bayside Dve:
    - Beach access path
- Private Amenity:
  - Biggs Rd – many properties have stair access, while in areas where there is a greater volume of sandy buffer, some boat ramps are present for deploying small watercraft.
  - North of Biggs Rd – most properties have either small seawalls with a small number of steps provided; or access via general ambulation across the dunes / scarp.

Figure 5.4 and Figure 5.5 illustrate the beach access locations for Beachmere.

Properties adjacent to the access points at McGregor Tce, Phillip St, Prince St, Sandy St, Louise Dve and Bayside Dve, should further consider works to be conducted jointly with Moreton Bay Regional Council, to establish more formalised beach access and tie-ins between the access points and new seawalls.

In terms of public access, extra boat ramps may not be required, although demand may change as private watercraft access becomes more difficult.

Figure 5.4 Beach access locations – Biggs Ave to Bishop Rd

Figure 5.5 Beach access locations – Bayside Dve

As the A-Line and associated standard design are implemented, it may be sufficient to maintain stair access at each of the current public access points, using much the same approach as currently existing at Biggs Ave south.

In the near term, it is anticipated most residents will desire their current level of access to be maintained. However, it is unlikely private watercraft access can be maintained with the implementation of a full seawall design with crest level >3.5m AHD. Design of seawalls with no sandy buffer and full beach exposure should therefore make allowance for private stairs to be incorporated; and private watercraft access will not be preserved.

5.3 Stormwater Outfalls

Provision should be made in private structures for adequate drainage of stormwater from the finished ground level through the embankment – otherwise, it anticipated collected stormwater will be integrated with public stormwater infrastructure via the property's Legal Point of Discharge.

Several public stormwater outfalls are present:

- 4 x box culvert structures at Beachmere Road Foreshore, mid Coronation Drive, at Phillip St, and at Prince St.
- 1 x duck-billed culvert at Sandy St. As of June 2022, this structure is currently being redesigned as a low-level box culvert structure, similar to the other outfalls.

Seawall design should consider tie-in to these structures to avoid damage or negative impacts. Adaptation works to existing stormwater outfall structures are considered a separate scope of works. Refer to Section 9 for further details of tie-in works.

5.4 Adjacent Structures

New seawalls will interface with several existing public seawalls including the southern end of Biggs Avenue, the Beachmere Activity Centre and at the eastern end of Main Street. Additionally, many approved and unapproved structures have been constructed on private properties along the proposed A-Line position. Appropriate treatments will be required at the interfaces between existing and new structures to ensure that the new seawall can be constructed without damaging the existing structures, and so that the new seawall does not negatively impact the existing structures.

Refer to Section 9 for further details of tie-in works.

5.5 Construction Methodology

The construction methodology will likely consist of standard land-based works during suitable tide windows. Staged, localised bunding and de-watering to facilitate construction works at the seawall toe may also be required.

Overall, the site and site access options are quite constrained. Public beach access locations at the Biggs Ave (south) public seawall boat ramp, and at McGregor Tce, Prince St, Sandy St, Louise Dve, and Bayside Dve, may be the only locations at which plant is able to move to the beach-side for construction operations (see Figure 5.4 and Figure 5.5).

Individual properties may be able to offer land-side access to heavy plant; but it is likely this will not be typical, limiting land-side plant to smaller vehicles.

Accordingly, seawall designs should consider the feasibility of construction predominantly from beach-side with tidal-controlled access. High-level construction sequence should consider:

- Services location and isolation
- Implementation of pedestrian and traffic management plan
- Site establishment
- Core construction works phase
- Works certification
- Demobilisation

5.6 Maintenance Provisions

Public beach access ramps should be designed to handle the occasional truck and excavator loading which may occur for maintenance activities on the seawall.

For Biggs Ave to Bishop Rd (refer Figure 5.4), the current boat ramp integrated with Biggs Ave Public Seawall may be suitable for access by maintenance vehicles. Locations at McGregor Tce, Phillip St, Prince St, and Sandy St may also be amenable for access by maintenance vehicles subject to minor engineering works. The Biggs Ave boat ramp could then be expected to service the Biggs Ave seawall in addition to the southernmost private residences, while the four noted beach access points to the north are likely adequate to service private residences from Coronation Dve north.

The distance between Biggs Ave boat ramp and McGregor Tce access path is ~1700m. Properties within this segment should consider whether land-side access might be provided as part of the design if this should prove more cost-effective and of lower impact than requiring maintenance vehicles to traverse a significant distance alongshore.

The northern segment adjacent to Bayside Dve should be adequately serviced by access points at Louise Dve and north Bayside Dve, again noting that minor engineering works may be required at these access point to permit passage of maintenance vehicles.

5.7 Safety Performance Criteria

Coastal Hazards

The design should seek to achieve an appropriate level of protection from wave overtopping commensurate with the planned usage characteristics of the wall. For private residences it may not be necessary to design to pedestrian safety limits. However, if the crest is designed to be frequently trafficked, then pedestrian safety should be considered.

The reference seawall has been designed to achieve pedestrian safety during a 1yr ARI event, including allowance for future sea level rise.

5.8 Amenity

The seawall design should seek to maintain, as far as practicable, unimpeded views seaward. However, this may be in direct conflict with other project requirements, including protection from wave overtopping. If a crest wall is incorporated into the structure, the height should be select to minimize the impact on seaward views whilst achieving the required overtopping protection. Future adaptation of the crest wall (increasing the height of the wall in stages to match future SLR) may also be a means of achieving the project objectives.

5.9 Safety in Design

A Safety in Design (SiD) report and accompanying risk register were prepared for the design, construction, and maintenance/demolition phase of construction of the reference design. A copy of the register is provided within this document (see Appendix B). The key extreme risks that were identified during the construction and commissioning phase of the project are summarised below:

- Instability of the slope and structure and adjacent structures during deconstruction of the existing seawalls to be replaced.
- Unsuitable tides, waves, weather, or lighting conditions making it hazardous to continue construction and damaging partially completed works.
- Pedestrians accessing worksite and are injured by unstable slopes, rocks, or excavations.

6 Geotechnical data

Geotechnical studies provided by MBRC are summarised in Table 6.1 below, and their spatial extent illustrated in Figure 6.1.

It is generally expected this will characterise the geotechnical conditions of the Biggs Ave – Bishop Rd extent, and most likely Bayside Dr to the north – however, further DCP testing would be required for larger seawall approvals remote to Biggs Ave. The studies were conducted to support a range of surficial structures including the Biggs Ave Public Seawall, and as a result the characterisation does not extend to large depths. The Sandy Street investigation by Douglas Partners extends inland from the coastline, while the AECOM investigation extends along the seawall frontage (on the beach), and the Coffey investigation focusses nearshore with a few boreholes located inland.

Table 6.1 Historical geotechnical investigations at Beachmere

Study	Site	Tests	Results
Coffey Geotechnics (2009)	Proposed Jetty and Boardwalk – assumed location at northern end of Biggs Ave	6 x DCP Boreholes	Transition from loose to very dense sands over upper 6m of alluvium
AECOM (2011)	Lehman Park and Beachmere Activity Centre	14 x DCP Boreholes	Transition from loose to very dense sands. Upper 1.6m are loose; medium depth to ~3.5m; dense > 3.5m
Douglas Partners (2013)	Sandy Street easement from Bishop Rd towards Deception Bay	5 x DCP Boreholes	Loose to medium dense sands to 3m depth

In the region adjacent to the toe of the Biggs Ave seawall, the studies are broadly consistent, in that alluvial sand layers varying from very loose near the surface (RL2.0 - RL1.0) to medium dense (RL1.0 – RL-1.0) to dense/very dense (below RL -1.0) were reported (see Figure 6.2).

At the Lehman Park location, the Coffey investigation extends to greater depths (generally to 6m). The material was characterised by Coffey as follows:

'At the jetty site, the profile consists predominantly of sands to the full depth of the investigation. A thin band of firm silty clay was observed in BH1 depth of about 3.4 metres. The sands were very loose to loose in consistency in the upper profile becoming medium dense to dense (and very dense at some locations) with depth'.

This aligns generally with the Biggs Ave results, though information on the relative levels is lacking. It is further noted by Coffey that '*an embedment depth of 0.9 metres is recommended for the revetment wall*' (boulder).

Landside of the seawall, the Sandy St tests indicate the presence of natural medium dense sand and silty sand over the surficial 3m (BH1). The Coffey boreholes also indicate the presence of medium dense alluvial materials, with imported fill towards the surface.

These results are useful for understanding the likely composition of the seawall sites. However, site-specific geotechnical testing will be required for individual private seawalls, to characterise material at the seawall toe and landside.

It is expected that below the surficial alluvial layers described by this historical testing, some form of cemented (indurated) sands will be encountered, possibly overlying several metres of very dense sand and muddy sand, before reaching Landsborough sandstone at about -8m MSL (Brooke et al, 2008).

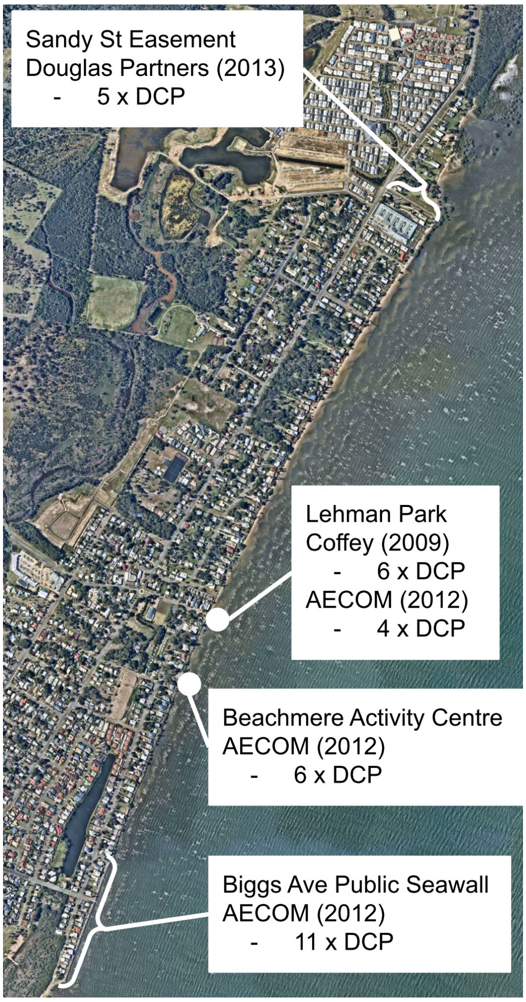


Figure 6.1 Historical geotechnical investigation sites – Biggs Ave to Bishop Rd

For the purposes of the reference design, the following assumptions have been made to support global stability assessments:

- Landside:
  - Depths of 0 – 1.0m – loose sand (Φe = 29 deg, Ce = 0 kPa)
  - Depth of 1.0m to RL -1.0m – medium dense silty sand (Φe = 35 deg, Ce = 20 kPa)
  - RL -1.0m to RL -3.0m – very dense sand (Φe = 40 deg, Ce = 0 kPa)
  - Below RL -3.0m – very dense and coherent material (Φe = 40 deg, Ce = 20 kPa)
- Seaside:
  - Depths of 0 – 1.0m – loose sand (Φe = 29 deg, Ce = 0 kPa)
  - Depth of 1.0m to RL -1.0m – medium dense sand (Φe = 35 deg, Ce = 0 kPa)

- RL -1.0m to RL -3.0m – very dense sand (Φe = 40 deg, Ce = 0 kPa)
- Below RL -3.0m – very dense and coherent material (Φe = 40 deg, Ce = 20 kPa)

Top RL	2.00 m	*Drop height*	510mm	*Tested by*	POS/CC	*Checked by*	
Easting	504919.6	*Hammer mass*	9 kg	*Tested on*	26/08/2011	*Checked on*	
Northing	6997898.48	*Tip type*	Conical				

RL [m]	DEPTH [m]	Number of Blows	COMMENTS
1.9	0.1	0	
1.8	0.2	0	
1.7	0.3	5	
1.6	0.4	0	
1.5	0.5	0	
1.4	0.6	0	
1.3	0.7	1	
1.2	0.8	2	
1.1	0.9	6	
1.0	1.0	7	
0.9	1.1	7	
0.8	1.2	9	
0.7	1.3	6	
0.6	1.4	4	
0.5	1.5	3	
0.4	1.6	3	
0.3	1.7	3	
0.2	1.8	6	
0.1	1.9	5	
0.0	2.0	4	
-0.1	2.1	3	
-0.2	2.2	2	
-0.3	2.3	3	
-0.4	2.4	4	
-0.5	2.5	4	
-0.6	2.6	3	
-0.7	2.7	7	
-0.8	2.8	12	
-0.9	2.9	11	
-1.0	3.0	18	
-1.1	3.1	21	
-1.2	3.2	18	
-1.3	3.3	20	Test terminated
-1.4	3.4	0	
-1.5	3.5	0	
-1.6	3.6	0	
-1.7	3.7	0	

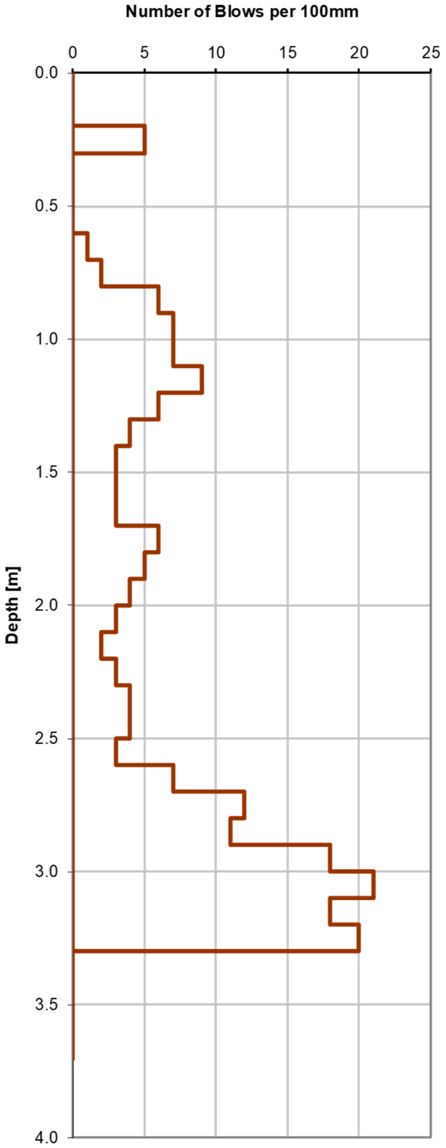


Figure 6.2 Typical DCP test results from the Biggs Ave segment (AECOM, 2011) at the toe

7 Design criteria

7.1 Design life

The design life is defined as the period for which the structure will remain fit for use for its intended purpose with appropriate maintenance. Due to the changing climate and sea level rise, this period might also include planned adaptations to ensure the design function of the assets match the design demand.

One of the main objectives of the seawall is to provide protection of private residential properties which may be in close proximity to the walls. Residential, timber houses typically have a design life of 50-years with other ancillary structures (sheds, swimming pools, etc.) being generally being designed for 20 years or less. In addition, these structures are typically built on shallow foundations and require the presence and stability of the underlying foundation material. To retain the material, and therefore ensure the stability and structural integrity of the residential structures, the new seawall should adopt a design life that matches or exceeds the design life of the structures it protects.

Minimisation of the Equivalent Annual Cost (annualised whole-of-life costs) for a new seawall asset should be considered, and it will likely indicate the adoption of longer design lives is a cost-efficient practice.

Table 7.1 indicates typical adopted design lives in structural and coastal design. The reference seawall design has assumed a 100yr design life.

Table 7.1 Design Life

Structure Type	Design Life	Reference
Ancillary Structures	25 yr	
Private Residences	50 yr	AS 4997-2005 (normal maritime structures)
Reference Seawall Design	100 yr	Adopted for MBRC Public Seawalls

7.2 Design Reliability

Design performance requirements are typically specified through a targeted design life and associated exceedance probability over the design life. Under stationary conditions, this maybe specified as an Annual Exceedance Probability (AEP) or Average Recurrence Interval (ARI = 1/AEP).

The probability of exceedance of the design event (EP) over the design life under non-stationary conditions (e.g., changing climatic intensity and sea level rise) is calculated as follows:

$$Pr(X > x) = 1 - \prod_{i=1}^{N} \left(1 - AEP(t)\right)$$

$$AEP(t) = 1 - \exp\left(\frac{-1}{ARI(t)}\right)$$

Where N is the design life in years, and ARI(t) is the time varying exceedance level (including design adaptations) at the relevant design limit.

Adopted rates of SLR over the design life are discussed in Section 8.

7.3 Design actions

7.3.1 General

As outlined in the guidance to Performance Outcome 10(3) of the State code 8: Coastal development and tidal works, the seawall design should comply with the relevant Australian Standards, which may include: AS 1170.0, AS 1170.1, AS 1170.2, AS 1170.4, AS 4997 and AS 4678 (DES 2019). As such, the structure classification and general design events to be considered in the design shall be determined in accordance with these standards, plus relevant design guidelines including QUDM.

In addition to the requirements of the referenced standards, the design shall also meet the minimum requirements outlined in Part G. Seawalls of the Operational Policy (Coastal Protection and Management Act 1995) - Building and Engineering Standards for Tidal Works (DEHP 2013), which are summarised in Table 7.2.

Table 7.2 Minimum design requirements – DEHP (2013) – Part G. Seawalls

Parameter	Value
Design storm event	2% AEP (or better) – Waves and Water levels
Overtopping	Overtopping permitted provided structural stability is unaffected
Scour	Toe of wall shall accommodate 50 years of long-term erosion and toe level must be less than or equal to LAT
Sea Level Rise	+0.3 meters
Seawall slope	Designed to minimise wave reflection
Terminal ends	Designed to minimise 'end effects'

7.3.2 Environmental loads

Environmental loading conditions shall be determined in accordance with the recommendation provided in AS 1170.0 and AS 4997. The proposed requirements are provided in Table 7.3.

Table 7.3 General design actions - seawall & crest wall

Parameter	Value	Reference
AS 1170.0-2002		
Structure classification	Importance Level 1 (walls)	Table 3.2
Wind	1/250 AEP*	Table 3.3
Earthquake	1/250 AEP*	Table 3.3
AS 4997-2005		
Structure classification	Function category 1	Table 5.4
Waves & Water Levels	1/200 AEP+	Table 5.4
Currents	Not defined	Sect 5.5

***100-year design life**

+50-year design life

It is noted that Section 5 of AS 4997-2005 provides guidance on appropriate return periods for design wave events and that these should be combined with water levels equal to or higher than MWHS. At this project site, waves are depth limited, so the appropriate design wave event is effectively governed by the water levels. As such, the same event recurrence interval applies, as the two variables are considered dependant.

Wave Overtopping Design Actions

The design criteria and associated design events for wave overtopping are provided in Table 7.4.

Pedestrian safety (SLS) criteria may not be applicable to private seawalls depending on the crest treatment; this criteria has however been adopted for the reference seawall design.

If ancillary structures or private residences are located within 10m of the seawall crest, Structural Stability – Adjacent Structures (ULS) criteria will be applicable to the seawall (note however that the relevant design life is that applicable to the adjacent structure). No allowance has been made for adjacent structures in the reference seawall design.

Table 7.4 Wave overtopping design performance criteria

Performance criteria	Event description	Limiting overtopping rate	Damage level	Reference
Pedestrian safety (SLS)	EP < 63% in any year of design life* (1 yr ARI)	< 10 -20 l/s/m (Hm0 ≈ 1m) < 1 l/s/m (Hm0 ≈ 2m)	-	Table 3.3 EurOtop II
Structural Stability – Adjacent Structures (ULS)	EP < 39% over design life* EP < 2% in any year of design life*	< 1 l/s/m	No damage	Table 3.2 EurOtop II
Structural stability - Seawall (ULS)	EP < 39% over design life* EP < 2% in any year of design life*	< 100-200 l/s/m	Minor damage acceptable	Table VI-5-6 CEM

*Events include the effects non-stationary SLR over the design life (Section 8)

As noted in Table 7.4, no damage to the structure should occur as a result of the Structural Stability – Adjacent Structure ULS event. However, some minor damage would be acceptable under the Structural Stability – Seawall ULS event, provided that overall structural stability and integrity is maintained.

Rock Armour Design Actions

The design criteria and associated design events are provided in Table 7.5, with reference to Van Der Meer (1998) Application and Stability Criteria for Rock and Artificial Units.

Table 7.5 Rock armour performance criteria

Performance criteria	Event description	Limiting Damage Rate (Sd)	Damage level	Reference
Structural stability (SLS)	EP < 87% over design life* EP < 5% in any year of design life*	2	No damage	Table 1 – Van Der Meer (1998)

Performance criteria	Event description	Limiting Damage Rate (Sd)	Damage level	Reference
Structural stability (ULS)	EP < 39% over design life* EP < 2% in any year of design life*	3	Minor damage acceptable	Table 1 – Van Der Meer (1998)

*Events include the effects of non-stationary SLR over the design life (Section 8)

As noted in Table 4, no damage to the structure should occur as a result of the 'Serviceability Limit State' (SLS) event. However, some minor damage would be acceptable under the noted 'Ultimate Limit State' (ULS) event, provided that overall structural stability and integrity is maintained.

Scour Protection Design

The structure's toe protection, plus the overall structure, shall be designed to resist a 1 in 200-year design scour event. This event will notionally be taken as the maximum scour resulting from the nominated 200-year ARI wave and water level event (per the ULS event applicable to wave overtopping).

Crest Wall Design (Stability)

The design criteria and associated design events for assessing crest wall stability against wave and water level actions are provided in Table 7.6.

Table 7.6 Crest wall performance criteria associated with wave and water level actions

Performance criteria	Event description	Limiting Factor of Safety (FoS)	Damage level	Reference
Structural stability - Sliding (ULS)	EP < 39% over design life* EP < 2% in any year of design life*	1.5	No damage	USACE (2003)
Structural stability - Overturning (ULS)	EP < 39% over design life* EP < 2% in any year of design life*	1.5	No damage	USACE (2003)

*Events include the effects non-stationary SLR over the design life (Section 8)

Other design assessments to be considered are within the typical purview of footing/soil retaining structural assessments, and include:

- Bearing Pressure
- Stability against surcharge (refer Section 7.3.3)

Over the design life of the structure, crest adaptation measures may be required to meet the increasing stability demands due to sea level rise.

7.3.3 Geotechnical

Appropriate geotechnical design actions shall be determined in accordance with the recommendations provided in AS 1170.0, AS 4997 (Sections 5.8 and 5.11) and AS 4678 (noting that this standard is not explicitly applicable to this seawall, which is categorised as a 'revetment' under the standard).

Further guidance is provided by Mollaert et al (2020) which synthesises design approaches from Eurocode 7, ROM, USACE, and the CIRIA Rock Manual. This approach applies Limit Equilibrium methods based on traditional Factors of Safety (FoS) (Table 7.6). These recommendations for limiting

FoS generally in alignment with the Queensland Government Department of Main Roads – Geotechnical Design Standard.

Table 7.7 Required factor of safety for slope stability analysis (wave and water level scenarios)

Soil Condition	Water Level + Wave Design Case	Stability Factor of Safety (Circular Failure)
Short Term (Undrained)	LAT	1.3
	LAT, MSL, HAT + W_1	1.3
	WL_{200} + W_{200}	1.05
	MSL + S	1.05
Long Term (Drained)	Low WL_{200}	1.5
	LAT, MSL, HAT + W_1	1.5
	WL_{200} + W_{200}	1.1
	MSL + S	1.1

Where the numerical subscript refers to the selected extreme event level; S = seismic; W = wave; WL = water level

Surcharge scenarios should also be considered as follows for ambient water level and wave conditions:

Surcharge loading:

- Construction phase - Short term - 'rapid draw-down' (Top of wall -> MLWS) + construction surcharge.
- Operational phase - Short term - 'rapid draw-down' (Top of wall -> MLWS) + maintenance surcharge.
- Operational phase - Long term - 'rapid draw-down' (Top of wall -> MLWS) (no surcharge).

In accordance with Section 5.11 of AS 4997, the applied surcharge shall not be less than 5.0 kPa for the above scenarios.

The reference seawall design has assumed the following vehicles apply:

- Construction surcharge corresponds to a 20t excavator with a 5m exclusion buffer from the design wall crest.
- Maintenance surcharge corresponds to a light motor vehicle of up to 3t tare, with axles spaced at > 1800mm.

Seismic:

Seismic load cases should consider a minimum pseudo-static horizontal acceleration of 0.10g consistent with AS 1170.4, given the low magnitude seismic events in the area. Vertical and horizontal pseudo-static coefficients are adopted as follows:

- a_{eq} = 0.65 * a_h = 0.65 * 0.10 = 0.065g
- a_v = +/- 0.5 * a_h

No surcharge is applied above the seawall during seismic conditions.

Hydrostatic water pressure management

The design is to be a rock armour structure - the porosity of this type of structure allows for free-drainage between the ocean and land side of the structure, and no need for hydrostatic water pressure management. Nevertheless, should any impermeable barriers be incorporated in the seawall design appropriate weepholes or similar should be incorporated to manage the build-up of hydrostatic water pressures.

7.3.4 Structural

Structural loads shall be determined based on the resultant loads from geotechnical, environmental, and other load sources, with load combinations determined in accordance with AS 1170.1, AS 4997 and AS 4678 (as appropriate).

Live loads for structural design:

- Pedestrian Live Load – 5 kPa area load applied to landward side of seawall
- Light Motor Vehicles up to 3t tare – allowing for 20 kN point load over an area 150x150mm, at a spacing > 1800mm.

7.3.5 Local drainage

Local surface water drainage design shall generally follow the principles and guidance provided in QUDM. Stormwater runoff from within the embankment area should be drained toward the sea as diffuse surface flows. Suitable crossfall shall be incorporated into the crest detail to achieve this, and concentration of flows should be avoided where possible.

If a crest wall is utilised, a suitable narrow gap should be provided at appropriate intervals (in the range 3-5m intervals) in the crest wall to 'outlet' local surface water drainage.

7.3.6 Durability requirements

The structure should be designed in accordance with the guidance provided in the following standards:

- AS2758.6-2019 - rock armour.
- Section 4 AS3600-2018 - concrete (i.e., crest wall, and any additional footpaths or access stairs).
- ACI 440.1R-15 - GFRP reinforcement.

For rock armour at elevations up to HAT + 1m (2.36m AHD), the exposure classification is "High Risk - salt water intertidal and splash zone - High energy" in accordance with AS2758.6 Table 1. Rock armour material requirements, including grading, rock shape, abrasion resistance, particle density are specified in AS2758.6 Section 9.

For reinforced concrete up to 2.36m AHD, the exposure classification is "C2 - tidal/splash zone" in accordance with AS3600-2018. Above 2.36m AHD, the exposure category is C1 for concrete elements. Section 6.3.1 of AS 4997-2005, suggests consideration should be given to alternative strategies available to reduce the opportunity for chlorides to cause reinforcement to corrode in any concrete elements incorporated into the structure. This includes the use of plain concrete members (un-reinforced members), fibre-reinforced concrete, and alternative reinforcement materials including glass fibre reinforced polymer (GFRP) or stainless steel. Where plain steel reinforcement cannot be avoided, the affected members shall be designed based on the requirements for this classification and the earlier noted design life of 100 years.

8 Design Parameters

8.1 Water Levels

8.1.1 Tidal Water Level

Tidal plane levels at Beachmere (Caboolture River) are documented by Maritime Safety Queensland, and reproduced below.

Table 8.1 Tidal Planes at Beachmere (MSQ, 2021)

Tidal Plane	m (LAT)	m (AHD)
HAT	2.62	1.36
MHWS	2.08	0.82
MSL	1.21	-0.05
MLWS	0.36	-0.90
LAT	0	-1.26

HAT modelling was conducted by Cardno (2017), which indicated expected HAT levels corresponding to 2050 and 2100 SLR values of 0.30m and 0.80m respectively as shown in Table 8.2:

Table 8.2 Highest Astronomical Tide (HAT) future levels

	Present Day	2050	2100
MSL increase from present day	-	+0.30 m	+0.80 m
HAT increase from present day	-	+0.38 m	+0.89 m
HAT level	1.40 m AHD	1.74 m AHD	+2.25 m AHD

8.1.2 Storm Tide Levels

The most recent storm tide studies covering the Beachmere region have been conducted by Cardno in 2017 and 2019. The latter study was directed toward storm tide inundation modelling, while the former study provides a more comprehensive set of extreme cyclonic and non-cyclonic extremes, including associated wave heights. The 2017 report has been referenced herein, noting the differences at the study site in the 2019 report are minimal.

Storm tide levels are presented in terms of Average Recurrence Intervals (ARI) and are reproduced for Beachmere in Table 8.3.

Table 8.3 Storm Tide Levels at Beachmere MBC-022 (Cardno, 2017), present day and 2100

ARI (years)	Present Day (m AHD)	2100 (m AHD)
20	1.87	2.67
50	1.93	2.75
100	2.01	2.83
200	2.44	3.39
500	2.83	3.82
1000	3.05	4.05

For ARI > 100 years (rarer events), the cyclonic extremes dominate, as evidenced by the sharp increase in extreme water levels at ARI = 200 years.

8.1.3 Sea Level Rise

Sea level rise magnitudes to be applied in the design of this project are presented in Table 8.4.

Table 8.4 Sea level rise magnitudes – MBRC CHAS Phase 2 (Water Technology, 2017)

Year	MSL Increase
2030	+0.15m
2050	+0.325m
2070	+0.50m
2100	+0.80m
2120	+1.20m

Note that this varies from values adopted for the Cardno (2017) study. The values in Table 8.4 are considered more conservative and are adopted herein.

8.2 Design Conditions

8.2.1 Joint Wave and Water Level

Key design conditions for the Beachmere seawalls will be driven by peak water levels, producing the most severe overtopping and erosive conditions.

Table 8.4 reproduces the design wave conditions nearshore for the southern segment of Beachmere detailed in the BMT numerical modelling report (R.A10995.003.00). It is suggested these conditions are also applied to the northern beach segment.

Assumptions underpinning these design conditions are as follows:

1. Beach profile during storm conditions is assumed to be of maximum depth controlling wave breaking to 0.0m AHD (refer BMT design report for Beachmere Seawalls, R.A10995.005.00).

2. Water levels at the structure toe were assessed by adding wave setup (developed from Stockdon et al, 2006) to produce an enhanced water level for propagating wave conditions.

3. Depth-limited wave cutoffs for several foreshore conditions were intersected with the joint probability contours for non-cyclonic conditions, and compared with the cyclonic extreme ordinates:

    a. For non-cyclonic conditions, the maximum intersection between the contour and the depth-limited wave line constitutes the peak wave capable of arriving at the seawall toe. This value has been tabulated along with the joint water level and the median associated peak wave period. Note that for some contours, there may be ordinates of higher water level and lower wave height which are of greater concern for overtopping or crest stability. Where all joint probability conditions are non-breaking, the conditions corresponding to the largest wave height on the contour have been tabulated.

    b. For cyclonic conditions, the depth-limited wave ordinates have been tabulated. The associated peak wave period assumes the same wave steepness as the incident waveform.

Table 8.5 Extreme nearshore non-cyclonic water levels & waves, z_b = 0.0m AHD

Storm tide event	TWL (mAHD)	Significant Wave Height, Hm0 (m)	Peak Wave Period, Tp (s)
Present Day Conditions (circa 2000)			
ARI 1	1.34	0.79	4.3
ARI 10	1.51	0.89	4.6
ARI 20	1.55	0.92	4.6
ARI 50	1.59	0.94	4.6
ARI 100	1.62	0.96	4.7
ARI 100 (Cyclonic)	1.72	1.01	4.1
ARI 200 (Cyclonic)	2.44	1.44	4.8
ARI 500 (Cyclonic)	2.84	1.67	5.1
ARI 1000 (Cyclonic)	3.05	1.80	5.2
2100 Conditions (+0.86m from year 2000)			
ARI 1	1.82	1.07	4.9
ARI 10	2.06	1.22	5.2
ARI 20	2.12	1.25	5.4
ARI 50	2.19	1.29	5.6
ARI 100	2.24	1.32	5.6
ARI 100 (Cyclonic)	2.60	1.54	4.8
ARI 200 (Cyclonic)	3.36	1.98	5.2
ARI 500 (Cyclonic)	3.78	2.23	5.4
ARI 1000 (Cyclonic)	4.00	2.36	5.5

8.2.2 Local Scour Hole

A localised scour hole of depth 1.35m has been applied in the design of the reference seawall. Refer to BMT design report for Beachmere Seawalls for further discussion (R.A10995.005.00).

9 General Design Details

9.1 Seawall Detailing

Secondary Armour

The secondary armour functions to transition from primary armour dimension down to the natural fill grading curve, whilst contributing to establishing target permeability levels.

Geometric relationships for this transition are discussed in the Rock Manual (2007):

- $D_{15,armour}$ / $D_{85,filter}$ < 4
- Indicative mass – $M_{50,filter}$ is between 1/15 to 1/10 of $M_{50,armour}$

Tertiary Filter

The tertiary filter layer serves to retain in-situ material whilst negotiating transition from secondary armour dimension. The typical approach currently is to use a geotextile filter as a membrane between the native fill or sand and the secondary armour, to prevent internal material erosion but allow permeation.

Potential weaknesses of this approach include:

- Friction between the geotextile and the retained sand can be lower and needs to be considered for global stability.
- Care during construction is essential to ensure that the filter layer is effectively continuous and that the geotextile isn't damaged during installation.
- Significant settlement can result in tearing of the geotextile, compromising the filter layer during service conditions.

The following requirements are suggested for the geotextile:

- Unit Weight 1000g/m^2
- Grab tensile strength 1000N in any plane
- Trapezoidal tear resistance 600N
- Water permeability 30l/m^2

The Rock Manual recommends the geotextile should be an approved proprietary geotextile with a locally available product. Non-woven needle-punched staple-fibre geotextile Texcel 1200R manufactured by Geofabrics is considered suitable for marine application and meets the required specifications.

9.2 Toe Level and Detail

Modern seawall design practice typically recommends additional detailing to the toe region of rubble mound seawalls (refer CIRIA, 2007 for recommended toe designs).

It is recommended that the toe of the seawall be established at the anticipated scour depth (ys) level of the existing firm foundational material, with a toe extension treatment consistent with the 'scour potential' for the site (Ref. Section 6.3.4.1).

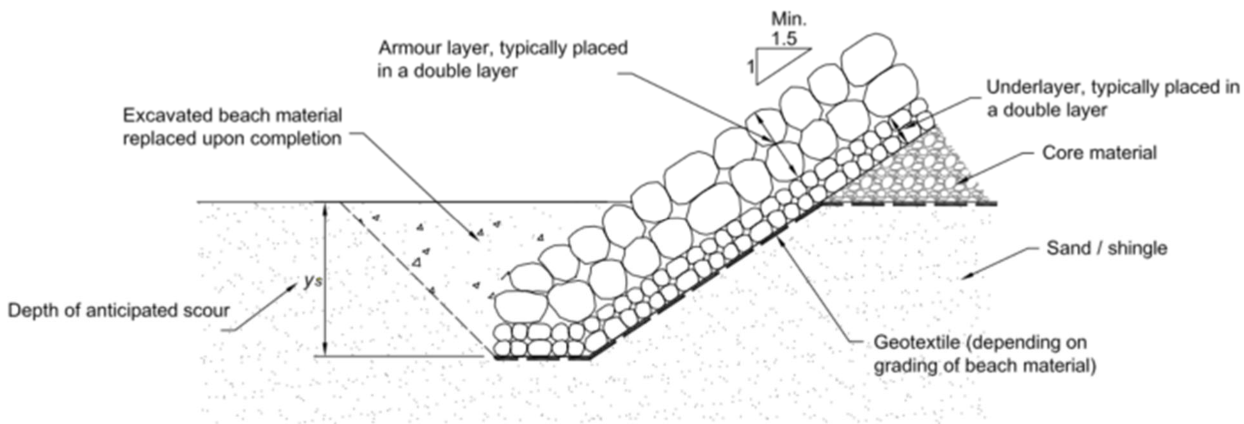


Figure 6.61 *Toe detail 3a: sand or gravel foreshore with low scour potential*

Figure 9.1 CIRIA Rock Manual toe detail on sand/gravel foreshores, low scour potential

9.3 Crest Detail

Seawall with Rear Side Grass Cover

Generally, it is anticipated that private seawalls at Beachmere will be seeking to transition the seawall crest to rear side grass cover, consistent with the current state of most properties. Key features of this type of crest detail should include:

- Crest (primary armour) width > $3k_tDn50$ (refer CIRIA Section 6.3.4.2)
- Material extents and transitions which minimise:
  c. Potential for stripping of surface materials due to overtopping flows:
    - Surface stability can be managed through extension of underlay around primary armour to finished surface level, to assist venting wave generated pressures and achieve much reduced scour potential of surface flows.
    - The geotextile should conform to the rear side of the underlay extension and cut back along the existing surface to provide a further limiter on erosion depth of the rear side grass surface.

  d. Potential for erosion and washout of fines and native material through the coarse primary armour:
    - Providing a similar sequence of materials (primary > secondary > core + geotextile) to the body of the seawall is an appropriate approach to minimise internal material erosion.

Seawall with Crest Wall

The other key crest detail likely to be applied at Beachmere is a crest wall. The reference design has taken the crest wall design from Biggs Ave Public Seawall as the basis geometry/arrangement. Detailing issues are similar to those detailed above for seawalls with rear side grass cover; however, the physical barrier of the crest wall offers a convenient approach for terminating the seawall layers:

- Crest (primary armour) width > $3k_t$Dn50 (refer CIRIA Section 6.3.4.2)
- Crest wall founding elevation can be made coincident with the base of the secondary armour layer, and the geotextile terminated at the rear face of the crest wall base (under a blinding concrete layer). This arrangement ensures resistance to internal erosion.
- Finished surface treatment at the rear of the crest wall should be detailed to resist erosion due to overtopping flows, and filtered drains provided through the crest wall to dissipate overtopped flow volumes.
- If a path is provided, additional drainage measures through the wall may be required.

Crest Reprofiling

A future reprofiled seawall should still meet the recommended crest width of $3k_t$Dn50. With a crest only overlay, this suggests the present day crest width should be greater than $3k_t$Dn50 in accordance with the basic geometry of the wall.

9.4 Seawall Interfaces

Termination in Native Dune Material

Seawall terminations to native fill should be designed to present flanking due to seawall end effects. General guidance on end effects is not extensive – WRL reviewed the literature on this topic for the NSW Department of Environmental and Heritage in 2012 and advised that the work of Komar and McDougal (1988) remains the best guidance on the topic. The relevant geometries are provided in Figure 9.2.

The alongshore extent of the end erosion is approximately 70% of the seawall length (up to a maximum of 500m), while the additional cross shore erosion is approximately 10% of the seawall length.

For a typical isolated private seawall having a frontage of 20m, a fronting seawall would have an adjacent alongshore effect of ~14m and an additional cross shore effect of ~2m.

For a continuous sequence of seawalls of length > 500m, the cross shore effect would therefore be ~50m.

In practice, this is likely to necessitate the following arrangements:

- Isolated Private Seawall on a single allotment – substantial seawall returns will be necessary to negotiate the cross shore effect and also allow for transition of the seawall profile from the toe level back to native bed levels. The result is a similar geometry to that indicated in Figure 9.2, resembling a breakwater head.
- Private Seawall at the end of a sequence of continuous seawalls – the required seawall return may need to extend >50m back from the A-line along the edge of the property to provide adequate protecting against flanking erosion. This is obviously a substantial imposition for a property of this nature.

It is likely to be advantageous for multiple properties to build their seawalls at the same time and to subsidise the additional cost of the seawall terminations whilst avoiding the cost of individual frontage terminations within the property confines.

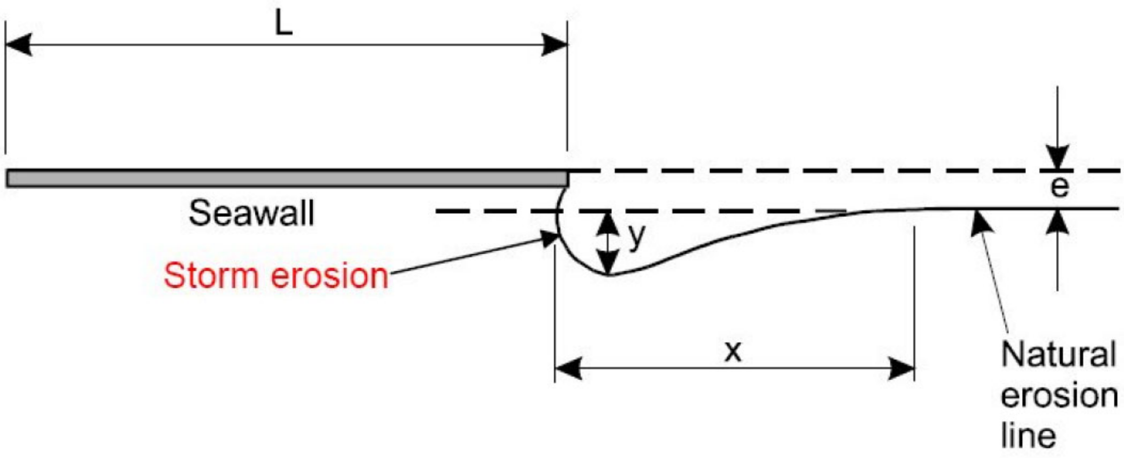


Figure 9.2 End Effects of Seawall (top) and example of successful and unsuccessful seawall terminations for private residences on the Gold Coast (bottom) (reproduced from WRL, 2012)

Interface with Existing Seawalls

There is a wide range of possible interfaces with existing seawalls, consistent with the number of inspected types (see Section 4), and it would be impractical to address each of these specifically within this document.

For reference, as-constructed drawings for the following modern approved seawalls are included in Appendix A:

- Biggs Ave Public Seawall

- Beachmere Activity Centre
- Lehman Park

General principles for seawall tie-in are spoken to in the notes section of the reference seawall drawings, and are listed here with further explanatory text.

The seawall tie-in to existing adjacent approved seawalls shall:

- Conform with appropriate standards and guidelines for structural and geotechnical engineering and armour stability:
  - Changes in wall slope may produce localised global stability issues.
  - Strength and stability of existing walls must be maintained during excavation and construction.
  - Consideration should be given to assumed design surcharge loads from adjacent seawalls in establishing the project basis of design.
  - Vertical wall elements (eg. Sheet piles) may be present and consideration should be given to deconstruction/removal vs inclusion in this instance.
  - Where crest walls are present, tie-ins should not compromise the existing structural capacity.
  - Differing armour size or type may control the transition strategy as the existing design intent should be retained.
- Account for durability – demolition or excavation should not interfere with the durability approach for the existing seawalls – for example by compromising reinforcement cover.
- Account for site drainage – different wall types may rely on varied subsurface drainage strategies, and there is the potential for disjoint at the new seawall wall interface to produce flow concentrations and associated performance issues.

Transition shall be considered at the seawall:

- General wall and toe – depth, seaward face and landward face of the primary and secondary armour should be transitioned into the new seawall in such a way as to avoid reductions in design capacities of the adjacent seawalls. For example, if the existing seawall were of a steeper slope than the new seawall, then the transition should take place from the property boundary and extend towards the existing seawall.
- Crest – crest walls should be contained wholly within the property boundaries of the developing property. Tie-ins should consider continuity of structural system and surface drainage.
- Geotextile / Native earth interface – geotextile protection to the native earth interface should overlap with geotextile from adjacent seawalls.

Interface with Boat Ramps

The Biggs Ave Public Seawall provides details for transition between a rock armour seawall and a typical boat ramp for the area. It is recommended this interface detail is used as a design starting point (Figure 9.3). Key features include:

- Termination points of crest walls and seawall control lines – offset from the external margins of the boat ramp to allow suitable transition slope between crest levels and boat ramp levels.
- Slope of boat ramp and required cross shore cutback in adjacent seawalls.
- Material extents.

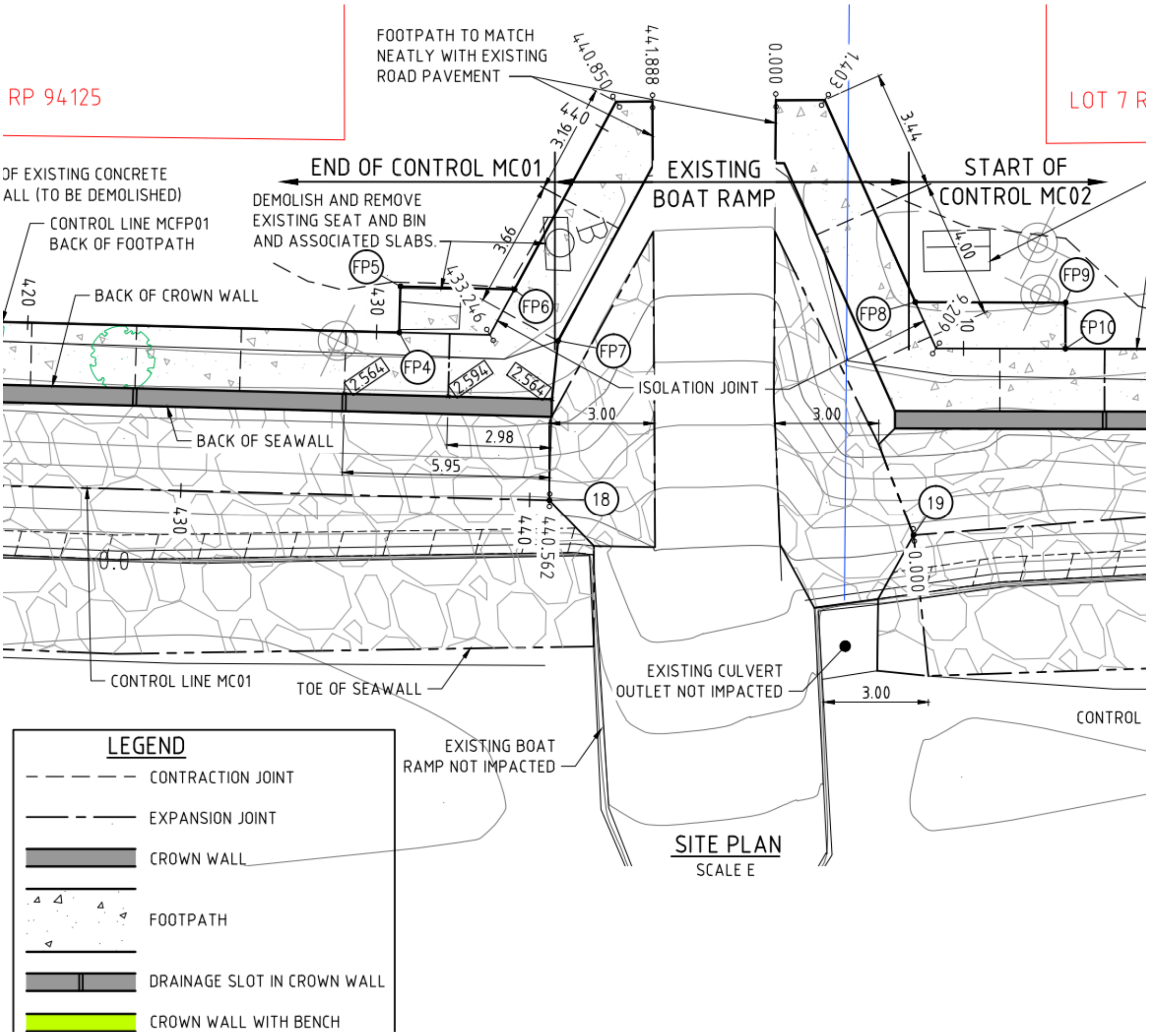
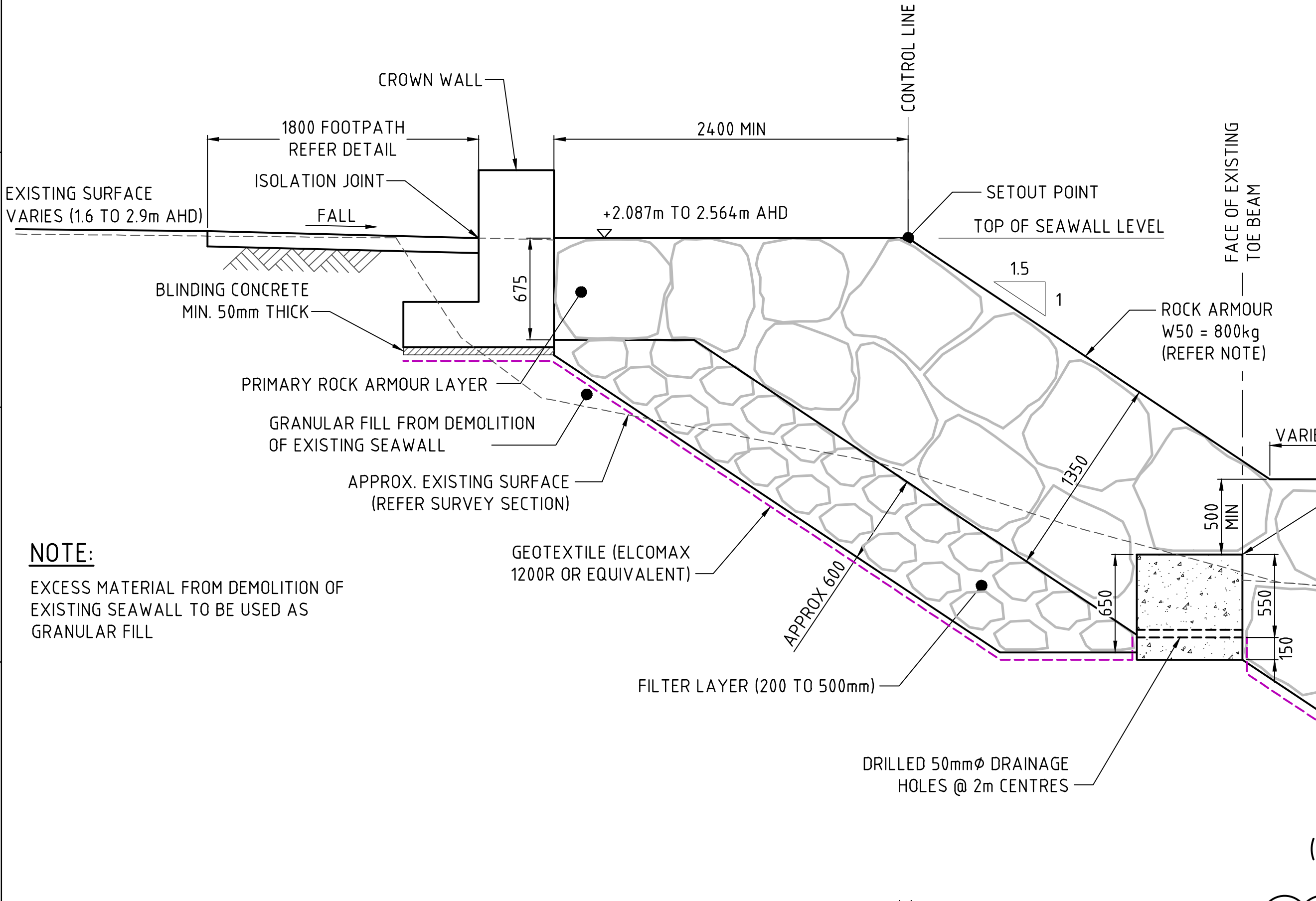
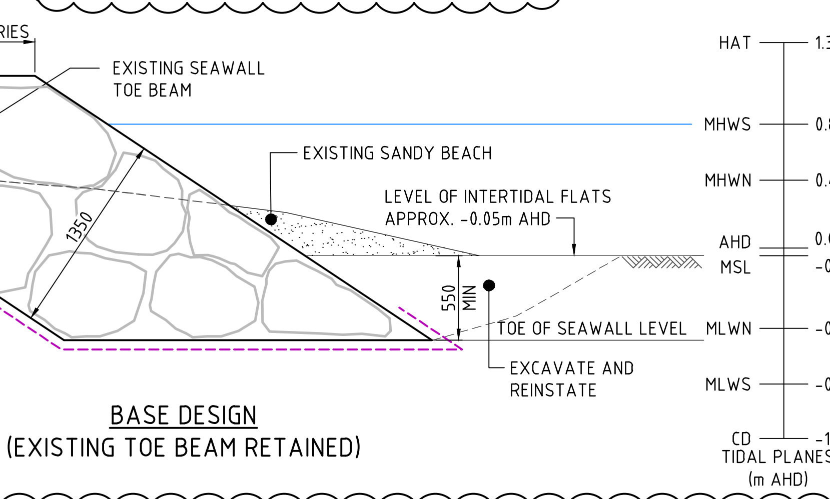


Figure 9.3 Biggs Ave Public Seawall – Interface with existing boatramp (AECOM, 2014)

Annex A Biggs Ave Public Seawall - Drawings

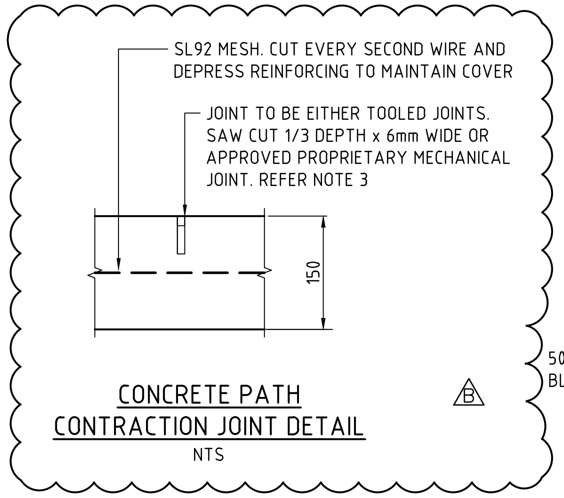




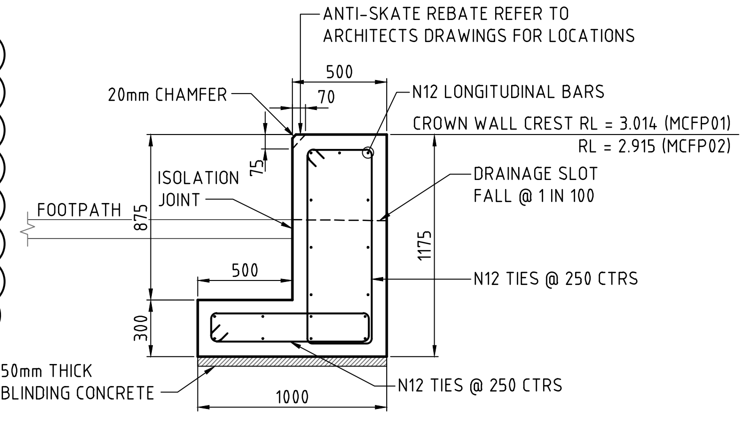
NOTE:

EXCESS MATERIAL FROM DEMOLITION OF EXISTING SEAWALL TO BE USED AS GRANULAR FILL

BASE DESIGN
(EXISTING TOE BEAM RETAINED)

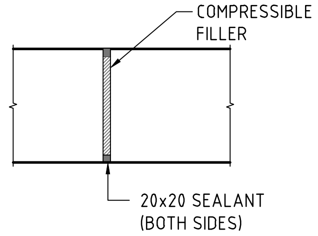


CONCRETE PATH
CONTRACTION JOINT DETAIL
NTS

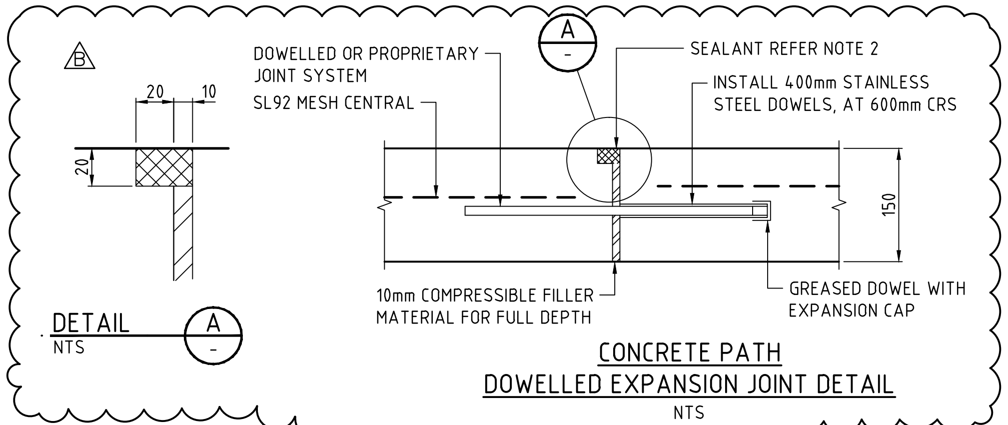


TYPICAL CROWN WALL SECTION
SCALE H

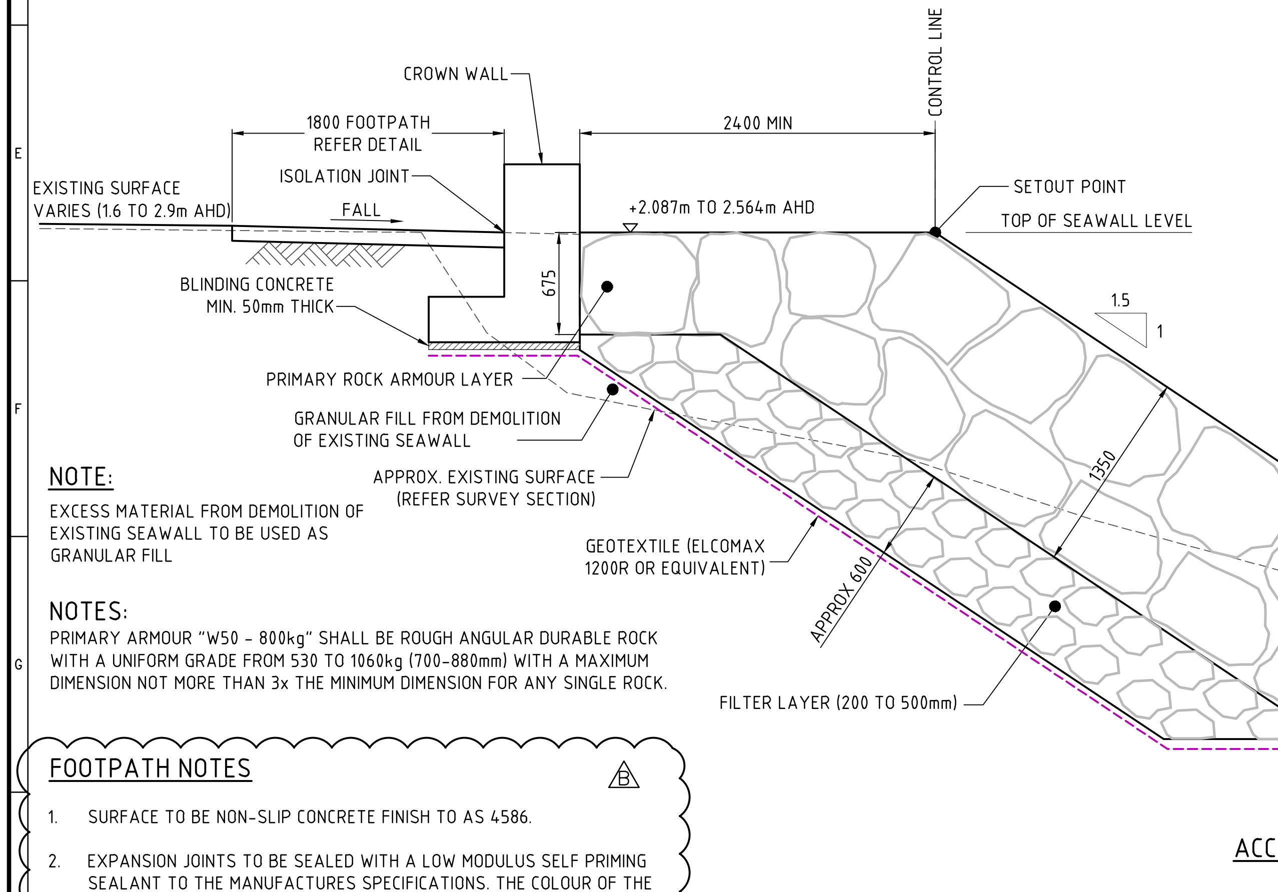
NOTE: CROWN WALL TO HAVE FORMED KEY JOINTS PROVIDED AT 6m CTRS. U.N.O. WALL UPSTAND IS TO HAVE 100mm WIDE DRAINAGE SLOTS AT JOINT LOCATIONS. CONCRETE STRENGTH TO BE 40mPa, COVER TO REINFORCEMENT SHALL BE 70mm.

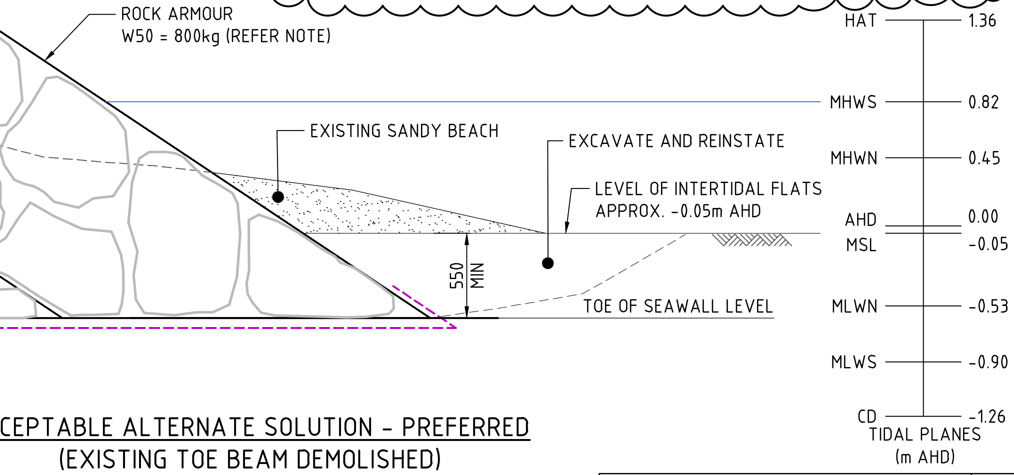


ISOLATION JOINT DETAIL
SCALE J



CONCRETE PATH
DOWELLED EXPANSION JOINT DETAIL
NTS





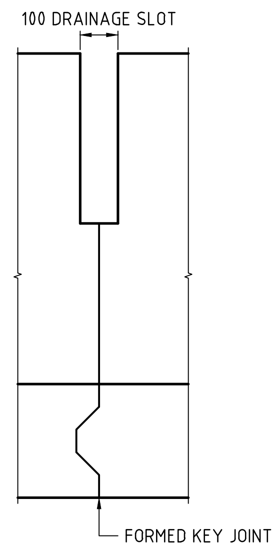
NOTE:

EXCESS MATERIAL FROM DEMOLITION OF EXISTING SEAWALL TO BE USED AS GRANULAR FILL

NOTES:

PRIMARY ARMOUR "W50 - 800kg" SHALL BE ROUGH ANGULAR DURABLE ROCK WITH A UNIFORM GRADE FROM 530 TO 1060kg (700-880mm) WITH A MAXIMUM DIMENSION NOT MORE THAN 3x THE MINIMUM DIMENSION FOR ANY SINGLE ROCK.

ACCEPTABLE ALTERNATE SOLUTION - PREFERRED
(EXISTING TOE BEAM DEMOLISHED)



CROWN WALL
KEYED JOINT DETAIL
SCALE J

FOOTPATH NOTES

1. SURFACE TO BE NON-SLIP CONCRETE FINISH TO AS 4586.
2. EXPANSION JOINTS TO BE SEALED WITH A LOW MODULUS SELF PRIMING SEALANT TO THE MANUFACTURES SPECIFICATIONS. THE COLOUR OF THE SEALANT IS TO MATCH THE ADJOINING SURFACE FINISH.
3. SAW CUT JOINTS ARE TO BE UNDERTAKEN BETWEEN 4 TO 12 HOURS AFTER LAYING DEPENDING ON THE CONDITIONS.
4. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS SHOWN OTHERWISE.

AECOM
AECOM Australia Pty Ltd A.B.N. 20 093 846 925

REVISION:	FOR CONSTRUCTION
DATE:	15.05.2014
APPROVED:	S.BETTINGTON RPEQ: 5251
INTERNAL DRG No:	60266237-215

Moreton Bay Regional Council

ENGINEERING, CONSTRUCTION & MAINTENANCE

AUTHORISATION

TONY MARTINI, Director	12/03/13
DAVID MEYER, Manager	12/03/13

PROJECT INFORMATION

Project Number: 42041
Coordinator / Team Leader
Design: MBRC / Checked: 04/2012
Surveyor / Survey Date
Survey Datum: MGA (GDA94) Zone 56 / AHD
Horizontal / Vertical

Scales
SCALE G 0 500 1000 1250mm; 1:50 (A3) 1:25 (A1)
SCALE H 0 200 400 600 800 1000mm; 1:40 (A3) 1:20 (A1)
SCALE J 0 100 200 300 400 500 mm; 1:20 (A3) 1:10 (A1)

ISSUE

Revisions

Rev	Name	Date
D	Appd:	Date
C	Appd:	Date
B	K. BOURKE, Appd:	01.10.15
A	S.BETTINGTON, Appd:	03.07.15
Original Issue		15.05.14

PROJECT DESCRIPTION

BEACHMERE SEAWALLS

BIGGS AVENUE SEAWALL REHABILITATION

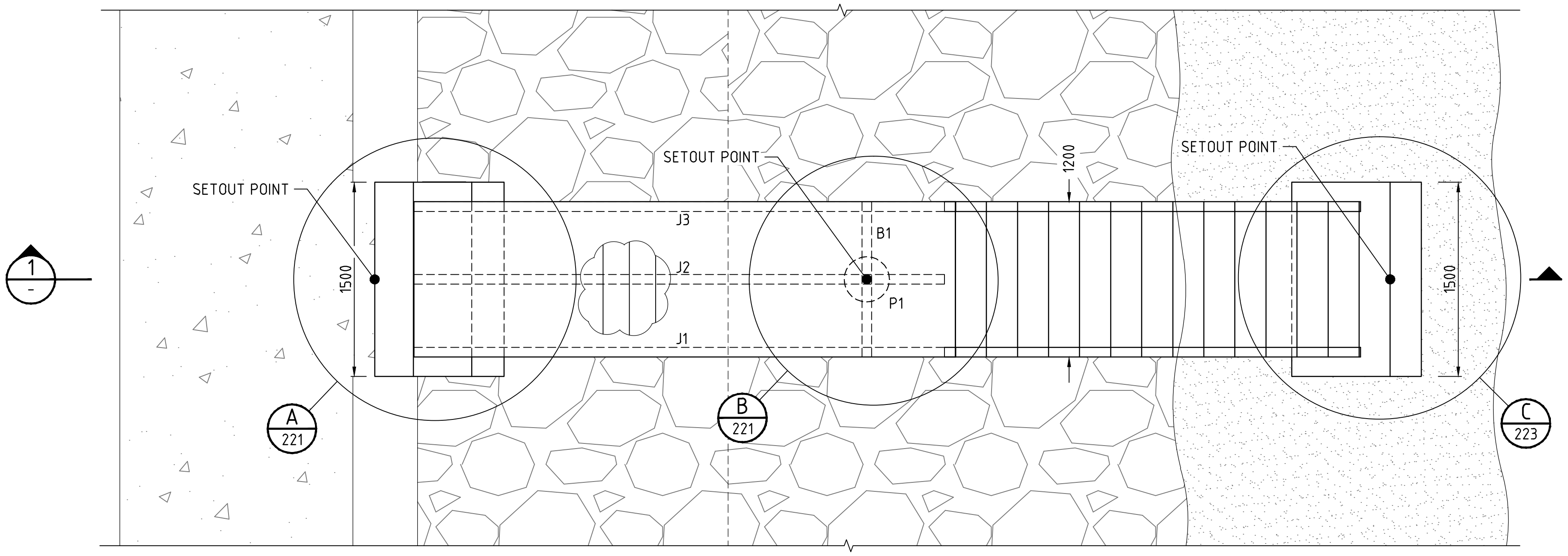
TYPICAL DETAILED SECTION

DRAWING NUMBER

42041-215 B

Drawing Number / Rev.
Drawing of / Original Plan Size A1

XREFS: A1 Drawing Frame.dwg
File name: P:\604X\60432432\5. CAD\20-Sheets\60266237-215.dwg Date: Oct 01, 2015 - 11:53am



TYPICAL STAIR ARRANGMENT PLAN
SCALE G

LEGEND:

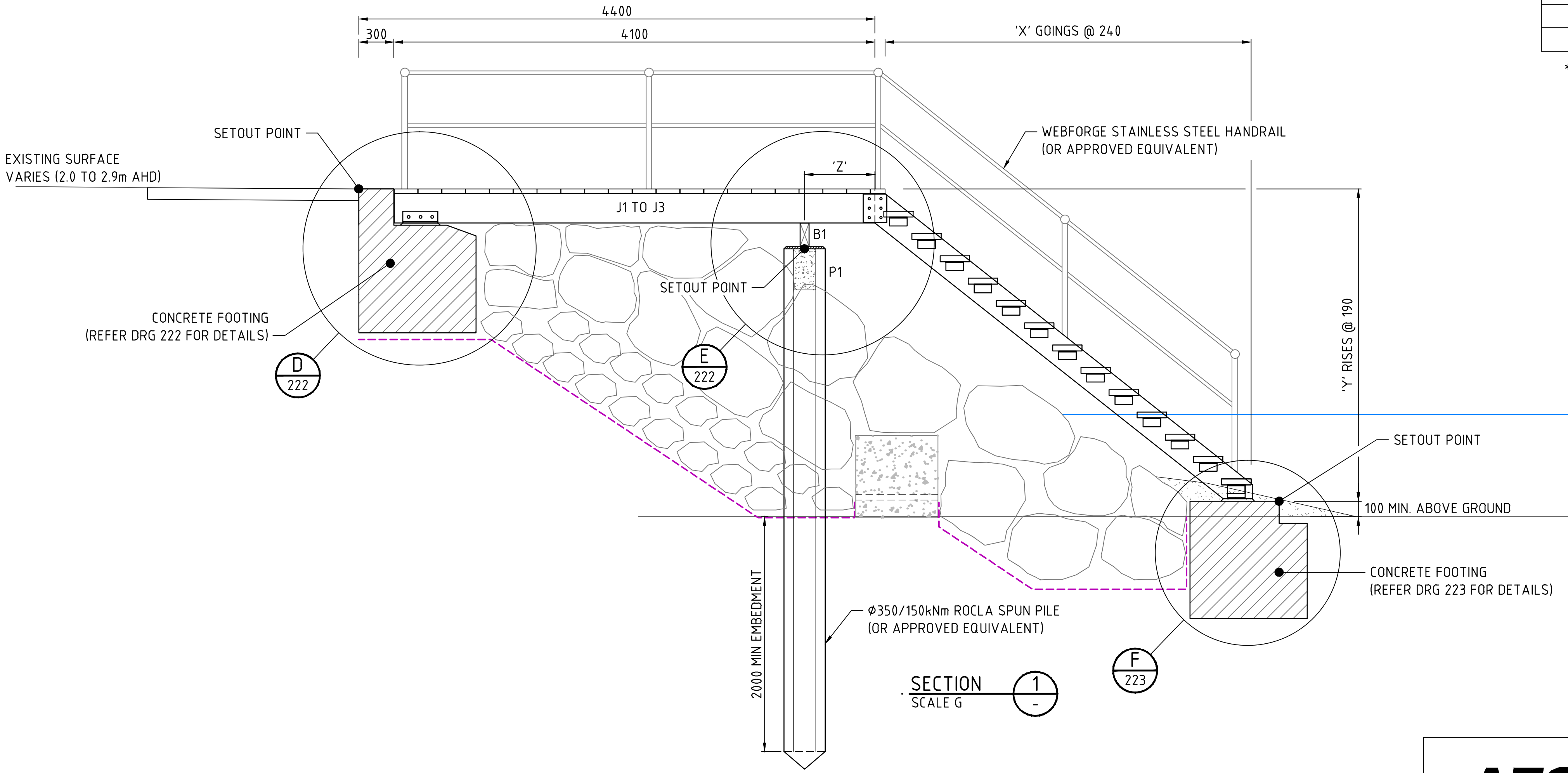
J1 TO J3	DENOTES 250x75 TIMBER JOIST
P1	DENOTES Φ350/150kNm ROCLA SPUN PILE
B1	DENOTES 200x75 TIMBER BEAM

NOTES:

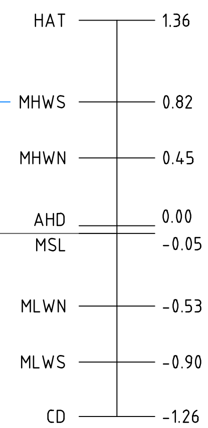
1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH GENERAL NOTES DRAWING 42041-225.
2. EQUAL RISE = 190mm
   EQUAL GOING = 240mm
   No. OF GOING = No. OF RISE - 1
3. ALL TIMBER RECESS FOR CUP HEAD BOLTS SHALL BE FILLED WITH BITUMEN.
4. ALL HORIZONTAL CONTACT SURFACE BETWEEN TIMBERS TO TIMBERS & TIMBER TO CONCRETE SHALL BE PROTECTED WITH A LAYER OF WATER PROOF MEMBRANE.
5. ALL BRACKETS ARE STAINLESS STEEL GRADE 316.
6. ALL BOLTS AND SCREWS ARE STAINLESS STEEL GRADE 316.
7. REFER TO SITE PLANS FOR ACCESS STAIRS SETOUT COORDINATES & LEVELS.
8. TGSI 'S INSTALLED TO TOP OF STAIRS TO COMPLY WITH AS 1428.2

ACCESS STAIRS	X	Y	Z
S1	14	15	700*
S2	15	16	900*
S3	15	16	1000*
S4	15	16	600*

* LOCATION OF PILE TO BE CONFIRMED ON SITE



SECTION 1 -
SCALE G



TIDAL PLANES
(m AHD)

Moreton Bay Regional Council

ENGINEERING, CONSTRUCTION & MAINTENANCE

AUTHORISATION

TONY MARTINI	12/03/13
Director	Date
DAVID MEYER	12/03/13
Manager	Date

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Surveyor / Survey Date
Survey Datum
MGA (GDA94) Zone 56 — Horizontal; AHD — Vertical

Scales: SCALE G 0 500 1000 1250mm; 1 : 50 (A3) 1 : 25 (A1)

ISSUE

Revisions

Rev	Appd:	Date
D		
C		
B		
A	S.BETTINGTON	03.07.15

Original Issue — Date: 15.05.14

PROJECT DESCRIPTION

BEACHMERE SEAWALLS

BIGGS AVENUE ACCESS STAIRS

GENERAL ARRANGEMENT

AECOM
AECOM Australia Pty Ltd A.B.N. 20 093 846 925

REVISION:	FOR CONSTRUCTION
DATE:	15.05.2014
APPROVED:	S.BETTINGTON RPEQ: 5251
INTERNAL DRG No:	60266237-220

DRAWING NUMBER

42041-220 A
Drawing Number — Rev.
Drawing of — Original Plan Size A1

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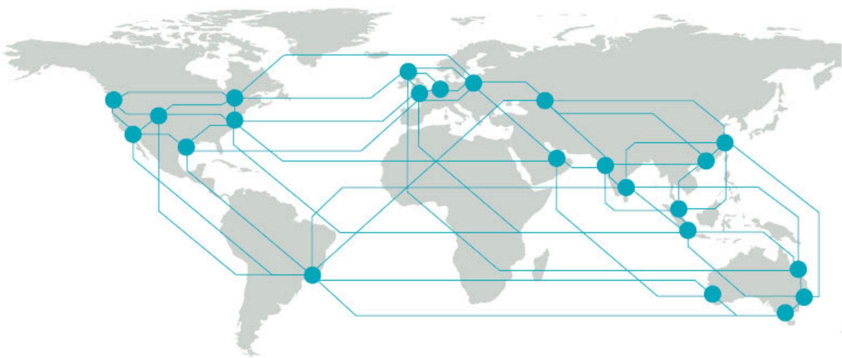
Annex B Safety in Design Register

Beachmere Reference Design Seawall - Safety in Design Risk Register

Date of Issue:

Identify Hazard

No	Risk	Categories	Consequence- Description	C- Rating	Likelihood- Description	L- Rating	Nature and effect of existing controls	Risk Rating	Risk Allocation	Risk Cost	Possible Treatment Options	Preferred Option	Likelihood	Risk Rating after Treatment (Residual)	Person Responsible	Timetable to Implement/Status
Project Delivery Risks - Design Phase																
PD2	Rock armour is undersized and is not hydraulically stable under design wave and water level conditions.	Project Delivery - Design Phase	Early damage to revetment, leading to loss of footpath and damage to embankment	Minor	Unlikely due to relatively standard design procedure for szing rock armour	Unlikey		Low			Design team apply standard design procedure and make conservative consumptions		Rare	Low		
PD3	Material around toe of revetment is scoured during storm leading to rock instability.	Project Delivery - Design Phase	Early damage to revetment, leading to loss of footpath and damage to embankment	Minor	Unlikely due to conservative assumptions of scour potential during design process.	Unlikey		Low			Design team apply standard design procedure and make conservative consumptions		Rare	Low		
PD4	Geotechnical failure of revetment through either bearing capacity, global stability, settlement or earthquake.	Project Delivery - Design Phase	Early damage to revetment, leading to loss of footpath and damage to embankment	Minor	Unlikely due to relatively standard design procedure for assessing geotechnical stability.	Unlikely		Low			Design team apply standard design procedure and make conservative consumptions		Rare	Low		
Project Delivery Risks - Construction and Commissioning Phase																
PD5	Instability of existing slope and embankment during construction activities	Project Delivery - Construction Phase	Localised or global slope slip causing injury or death to workers. Public and infrastructure may also be affected on top of embankment.	Substantial	Possible depending on the methodology and equipment used by the contractor	Possible		Very High			Contractor to develop safe work methods to ensure geotechnical stability of site. Also consider closing top of embankment during works.		Rare	High		
PD6	Rocks not being placed in a stable configuration during revetment construction.	Project Delivery - Construction Phase	Loss of materials and/or time.	Minor	Possible depending on the methodology of the rock placement.	Possible		Moderate			Contractor to ensure placed armour rock is stable with three points of contact to other rocks.		Rare	Low		
PD7	Rock stockpile becoming unstable or unexpected movement when adding, removing or moving rocks	Project Delivery - Construction Phase	Injury or death to workers.	Substantial	Possible but unlikely depending on the site arrangement adopted by the contractor	Unlikely		Very High			Contractor to have appropriate procedure to handle large rocks		Rare	High		
PD8	Pedestrians accessed worksite and are injured by unstable slopes, rocks, excavations.	Project Delivery - Construction Phase	Injury or death to public.	Substantial	Possible given the close proximity to commerical buildings, public spaces and the general public	Possible		Very High			Contractor to ensure the site is properly secured		Rare	High		
PD9	Unsuitable tides, waves, weather or lighting conditions making it hazardous to continue construction.	Project Delivery - Construction Phase	Increased risk of a safety incident occuring.	Substantial	Possible to encounter hazardous conditions at some period during construction.	Possible		Very High			Contractor to stop construction during hazardous weather conditions and include allowance in project schedule for potential weather delays		Rare	High		
PD10	Movement of trucks and heavy machinery causing traffic accidents	Project Delivery - Construction Phase	Injury to pedestrians and/or workers; multi vehicle accidents.	Substantial	Possible given the relatively limited access around the site.	Possible		Very High			Contractor to ensure provide protected pedestrian access around the site.		Rare	High		
PD11	Soil/rock located under heavy construction machinery is unstable, causing machine to fall/slide/tip.	Project Delivery - Construction Phase	Injury or death to workers; damage to revetment; loss of equipment.	Substantial	Relatively unlikely due to known soil conditions around the site	Unlikely		Very High			Design team to verify geotechnical capacity under construction loading. Contractor to confirm geotechnical conditions on site and verify proposed construction methodology. Contractor to report any unexpected ground movement during constructior		Rare	High		
PD12	Storm damage during incomplete works (rock armour revetment damaged, casting of concrete elements damaged).	Project Delivery - Construction Phase	Loss of time and materials; potential for materials to be spread with environmental impacts.	Minor	Possible depending on local weather conditions.	Possible		Moderate			Contractor to plan most vulnerable stages of construction outside cyclone season. Contractor to monitor forecasts and enact pre-storm preparations if storm is approaching.		Unlikely	Low		
PD13	Environmental incidents (e.g. fuel and oil spills) contaminating the site.	Project Delivery - Construction Phase	Local soil and water contamination.	Major	Unlikey depending on contractor's work methodology.	Unlikely		High			Contractor to ensure safe working methodology and have spill response equipment and plans in place.		Rare	High		
Project Delivery Risks - Operations Phase																
PD15	Damage to rock revetment (i.e. movement of rocks) if storms exceed design event.	Project Delivery - Operations	Displaced armour units	Moderate	By definition, rare that design event is exceeded	Rare		Moderate			N/A		Moderate	Moderate		
PD16	Armour rock does not perform as expected due to breakages, weathering, etc.	Project Delivery - Operations	Structural integrity of revetment diminishes; more likely to be damaged during storms	Moderate	Possible if incorrect rock is used.	Unlikely		High			Design team to specify technical requirement of rock armour and ensure testing is performed to confirm suitability		Rare	Moderate		
PD17	Use of heavy machinery to replace/reprofile rocks on unstable ground after storm event	Project Delivery - Operations	Injury or death to workers; damage torevetment; loss of equipment.	Substantial	Unlikely but possible if assessment of ground conditions is not performed prior to works.	Unlikely		Very high			Assessment of ground conditions to be performed if there have been large changes at the site due to storms.		Rare	High		
PD18	Pedestrians walking over revetment structure; slips, trips and falls, or being knocked off by wave overtopping	Project Delivery - Operations	Injury or death to public.	Substantial	Rare, given balustrade will block access to rocks and water in this area	Rare		High			Signage to be installed to specify no pedestrian access permitted onto the revetment		Rare	High		
PD19	Swimmers and/or vessels coming into contact with rocks, since revetment will be partially submerged during high tides	Project Delivery - Operations	Injury or death to public.	Substantial	Rare, given balustrade will block access to path from water	Rare		High			N/A		Rare	High		
Approval																
AP1	Delay associated with Approval Process	Approval	Delay in start of construction of project leading to area being unprotected for a longer period of time	Minor	Possible, depending on the proposed design	Unlikely	Planning and engagement with regulators	Low								
Environmental Risks																
E7	Contractor does not follow communication protocols, condition requirements, notification requirements leading to a breach of conditions	Environmental	Non compliance with approval conditions; potential project delays	Moderate	Possible depending on how serious the contractor takes conditions and responsibilities	Possible	Contract to stipulate communication, reporting and protocols; effective project management of the contractor by MBRC	Moderate			Contractor to ensure communication protocols, condition requirements, notification requirements are adhered to.		Rare	Moderate		
E8	Air, noise, waste or other nuisance complaints from the general public or waterway users	Environmental	Non compliance with approval conditions; potential project delays	Minor	Possible but unlikely given the short duration of projects in close proximity to residences	Unlikely	Contractor to ensure all plant and equipment maintained in good working order; waste management in the ODMP	Low			N/A		Unlikely	Low		



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