COASTAL ENGINEERING SOLUTIONS

EAST BEACH SEAWALL

WORKS PREPARATION

SEAWALL ASSESSMENT & DESIGN CONCEPTS

April 2013

for

Moyne Shire Council

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EXECUTIVE SUMMARY

Background to this Study

- The physical condition and structural integrity of the seawall along the East Beach frontage of Port Fairy vary significantly along its 2km length - nevertheless the seawall is predominantly vulnerable to damage during storms. Given the importance of this seawall in protecting infrastructure and assets along the East Beach foreshore, Moyne Shire Council has commissioned Coastal Engineering Solutions Pty Ltd to undertake a structural assessment of the seawall.
- In addition to determining the structural integrity of the wall, the assessment has provided a Concept Design for recommended remediation works; along with a prioritisation strategy and indicative costs for implementation of these works.

Existing Condition of the Seawall

- The adequacy of the seawall in providing foreshore protection has been assessed by consideration of its likely performance during a 100 year ARI storm event. Both present-day and future climate change scenarios have been considered. When undertaking the assessment, consideration has been given to the three primary modes of seawall damage/failure, namely:

  erosion of the armour layer - instigated when the rocks on the front face of the wall are not able to withstand the forces applied by waves as they wash against the slope. The rocks are effectively washed off the structure by the waves.

  by undermining - occurs when wave action causes scouring of erodible material at the toe of the armoured slope, causing it to be undermined and to then collapse (even though it may consist of large rocks that would otherwise not have been moved by waves).

  by wave overtopping - caused by waves that wash up over the top of the armoured slope and scour the material immediately behind the wall. The top of the wall is then no longer supported by underlying material and it collapses into the scoured area behind it - lowering the top of the seawall further, allowing greater overtopping, greater scour and rapid progression to structural failure.

- To determine the existing condition of the seawall (and hence its current and future susceptibility to damage/failure) a site investigation was undertaken. This entailed pulling apart sections of the seawall using a large hydraulic excavator. Each section of seawall was then immediately rebuilt once the condition and elements of its construction were inspected and recorded by a coastal engineer specialised in seawall design/construction.

- Aspects examined and recorded included the estimated average size of armour rocks; the range in rock sizes; the number of armour layers; rock placement density and rock interlocking; existence or otherwise of any filter medium (rock or geotextile); toe level; crest level and front face slope.

- The investigations identified ten different seawall categories along the East Beach foreshore - each of varying length and structural condition. these being:

  I. Low seawall backed by dune and park;

  II. Low seawall with dwelling fence line within seawall area;

  III. Low seawall with vegetated dune and dwelling fence line set back from seawall;

  IV. Low seawall with grassed embankment above the wall;

  V. Mid-height seawall with grassed embankment above seawall;

  VI. Mid-height seawall with embankment in front of the SLSC;

  VII. Mid-height seawall with path above the wall, then dune above the path;

  VIII. High steep embankment and seawall with road above;

  IX. Mid-height seawall with vegetated dune above the seawall;

  X. Temporary seawall steps immediately adjacent to recent 2012 seawall upgrade.

- A summary of the primary characteristics of each category is offered in the table on the following page, along with the assessment as to whether or not each category of the existing seawall complies with the 100 year ARI design standard.

A	B	C	C	D	E	F	G	H	I	J
Seawall Category	Excavated Cross Sections	Chainage on wall (metres)	Length of Category (metres)	Typical Armour (tonnes)	Volume of rock (cu.m/m)	Toe level of wall (to AHD)	Crest level of wall (to AHD)	Complies with erosion of armour?	complies with undermining?	complies with overtopping?
I	1 to 3	0-150m	150m	0.25 to 1	5.4	+0.2m	+2.9m	NO	NO	NO
II	4 to 6	150-325m	175m	nil	2.9	+0.4m	+2.7m	NO	NO	NO
III	7 to 14	325-795m	470m	0.25 to 1.5	4.3	+0.4m	+2.3m	NO	NO	NO
IV	15 and 16	795-895m	100m	0.25 to 1.5	4.3	+0.3m	+2.3m	NO	NO	NO
V	17 and 18	895-1015m	120m	0.25 to 1	7.8	+0.1m	+3.6m	NO	NO	YES
VI	19 and 20	1015-1065m	50m	0.25 to 2	12.7	-0.3m	+4.3m	NO	NO	YES
VII	21 and 22	1065-1185m	120m	0.25 to 2	6.6	-0.4m	+3.8m	NO	NO	YES
VIII	23 to 29	1185-1545m	360m	0.25 to 1	9.0	0.0m	+5.1m	NO	NO	YES
IX	30 to 39	1545-2015m	470m	0.25 to 1	5.5	+0.2m	+3.9m	NO	NO	YES
X	steps to new wall	2015-2045m	30m	1	approx 11	approx -1.0m	approx +3	NO	NO	YES

NOTES

Column H : compliance with requirement to mitigate erosion of armour?
Requires the typical armour size in Column D to be at least 2 tonnes, and the volume of rock in column E to be at least 19.7 cu.m. (ie. 35 tonnes) per linear metre

Column I : compliance with requirement to mitigate undermining?
Requires the toe level of the seawall in Column F to be at or below RL-1.25m AHD.

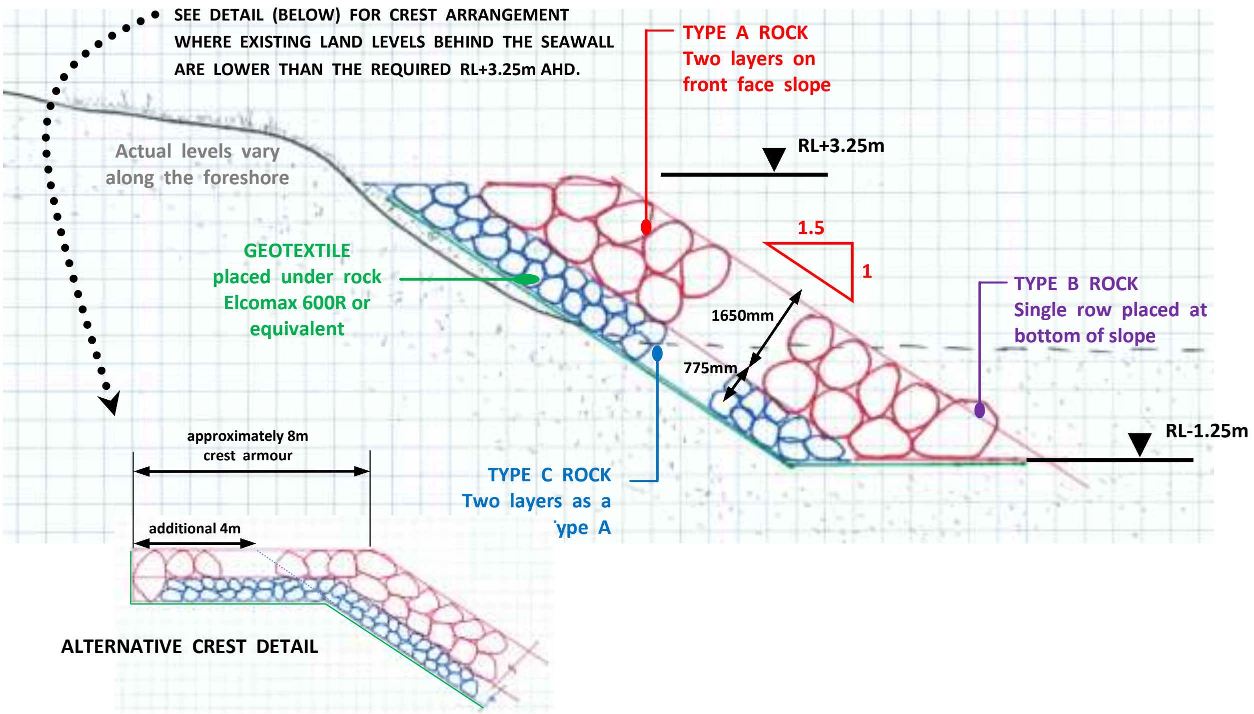
Column J : compliance with requirement to mitigate wave overtopping?
Requires the crest level of the seawall in Column G to be at or above RL+3.25m AHD.

- It is evident that there is a clear need to upgrade the East Beach seawall to an appropriate structural standard of foreshore protection.
- As part of this Study, requirements for remediation have been prioritised based on the combined assessment of the condition of the existing wall and the vulnerability of private and public infrastructure behind the wall. Based on that assessment, it is evident that remediation works could be staged over the coming 20 years.
- Given the existing condition of the seawall, in conjunction with the nature of the foreshore immediately behind the structure itself, a ranking of priority for the necessary remediation works has been compiled and included in the table on the following page.
- It is recommended that the Category VIII length of seawall (between Bourne Avenue and Ritchie Street; ie. between approximate chainages 1185m and 1545m), be upgraded as soon as possible since it has a road and car parking areas very close to the top of the wall. These public assets are currently threatened by seawall failure during severe storm, even under the present-day climate scenario.

Seawall Category	Excavated Cross Sections	Length of Category (metres)	Chainage on wall (metres)	Foreshore type	Remediation priority by 2020	Remediation priority by 2100
I	1 to 3	150m	0-150m	dune & vegetation	LOW	HIGH
II	4 to 6	175m	150-325m	property boundary	HIGH	VERY HIGH
III	7 to 14	470m	325-795m	dune & vegetation	MEDIUM	HIGH
IV	15 and 16	100m	795-895m	embankment / grass	MEDIUM	HIGH
V	17 and 18	120m	895-1015m	embankment / grass	LOW	MEDIUM
VI	19 and 20	50m	1015-1065m	dune & vegetation & SLSC	LOW	MEDIUM
VII	21 and 22	120m	1065-1185m	path (+3.9m AHD) & dune	HIGH	VERY HIGH
VIII	23 to 29	360m	1185-1545m	embankment / road	VERY HIGH	CRITICAL
IX	30 to 39	470m	1545-2015m	dune / path	MEDIUM	HIGH
X	steps to new wall	30m	2015-2045m	dune & vegetation	LOW	MEDIUM

Concept Design

- A Concept Design for upgrading the existing East Beach seawall to an appropriate structural standard has been prepared following consideration of its condition and required structural performance.
- The Concept Design is based on the detailed engineering design of the recently completed upgrade and extension works at the northern-most end of the seawall. The design standard for that completed work accommodates the 100 year ARI event and addresses future climate change scenarios.
- Due to the variable nature of the existing foreshore (both in terms of its landform and use), it is necessary to include a specific detail for those sections where the existing land levels and the top of the seawall are below RL+3.25m AHD. This alternative crest armour arrangement is required to mitigate the effects of green water overtopping during severe storms.



RECOMMENDED CONCEPT DESIGN

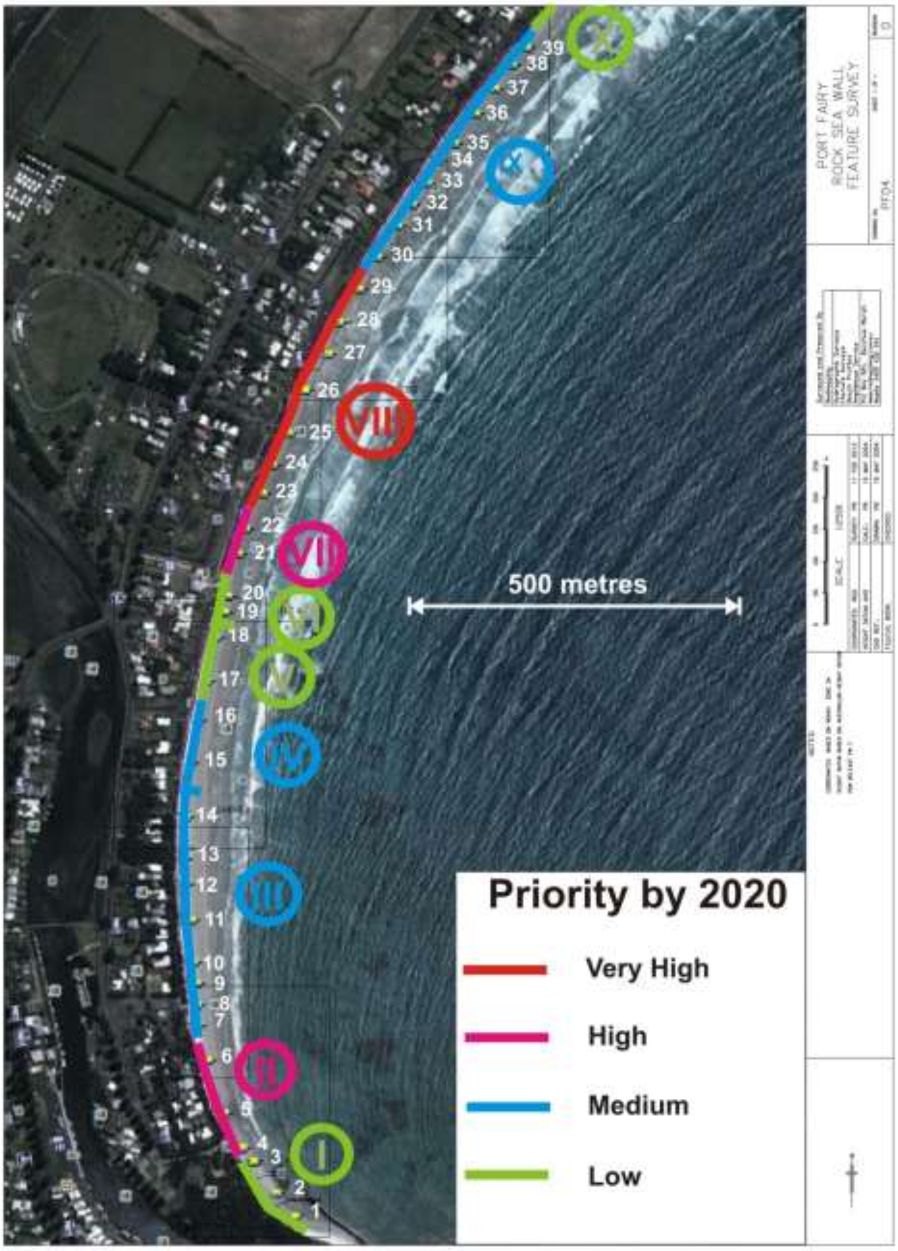
Potential Effects of Seawall Works on Coastal Processes

- The fundamental philosophy applied to the concept design of the East Beach seawall is to have it located behind the active beach system - as close as possible to the alignment of the existing structure.
- The resulting concept is such that the structure does not adversely influence the local beach. Staged works (whereby only discrete sections of the seawall are upgraded) will not adversely affect adjacent sections of the existing seawall.
- During subsequent detailed engineering design, it may become evident that some minor realignment of the seawall may be necessary along short sections of the wall. This aspect will become clearer during the detailed design phase - which will need to consider measures to fully mitigate any potentially adverse implications. Such measures are not expected to be particularly challenging from a technical viewpoint, and are very likely to be achieved through prudent design and careful construction.

Estimated Costs of Seawall Works

- The issue of whether all of the seawall reconstruction work is undertaken at once, or discrete lengths are completed under separate construction contracts spanning several years is controlled by funding issues. Given the scale of the physical works necessary to upgrade the approximately 2kms of existing seawall, it is expected that the works will be undertaken in stages.
- Therefore when preparing cost estimates for the upgrading works, it has been assumed that each of the identified seawall categories will be upgraded as separate construction activities and contracts.
- The table and figure below summarise the overall costs for the upgrading works and shows the localities. The sections of seawall have been ranked in descending order of priority regarding the need to implement the upgrading works. However it is acknowledged that Council should consider the particular order that works within the same priority classification should be undertaken.

Seawall Category	Excavated Cross Sections	Length of Category (metres)	Chainage on wall (metres)	Remediation priority by 2020	Estimated Cost
VIII	23 to 29	360m	1185-1545m	VERY HIGH	$840,000
II	4 to 6	175m	150-325m	HIGH	$395,000
VII	21 and 22	120m	1065-1185m	HIGH	$227,000
III	7 to 14	470m	325-795m	MEDIUM	$848,000
IV	15 and 16	100m	795-895m	MEDIUM	$362,000
IX	30 to 39	470m	1545-2015m	MEDIUM	$864,000
I	1 to 3	150m	0-150m	LOW	$350,000
V	17 and 18	120m	895-1015m	LOW	$260,000
VI	19 and 20	50m	1015-1065m	LOW	$134,000
X	steps to new wall	30m	2015-2045m	LOW	$90,000
ENTIRE LENGTH		2045m	0-2045m		$4,370,000



Seawall Categories and Localities

1 INTRODUCTION

In recent times East Beach at Port Fairy has been experiencing an increasing threat of erosion - a situation which is likely to be further exacerbated by future climate change. In response to the emerging erosion threat, seawall construction works were first implemented in the 1950's - with extension, repair and upgrading of this rock-armoured structure being undertaken intermittently since that time.

The physical condition and structural integrity of the seawall varies significantly along its 2km length, and is vulnerable to damage during storms. Given the importance of this seawall in protecting infrastructure and assets along the East Beach foreshore, Moyne Shire Council has commissioned Coastal Engineering Solutions to undertake a structural assessment of the seawall.

In addition to determining the structural integrity of the wall, the assessment is to provide appropriate concept designs for any necessary remediation works; along with a prioritisation strategy and indicative costs for implementation of works. The extent of the seawall is shown below on Figure 1.

Figure 1 : Extent of the seawall investigations

The natural state of the East Beach foreshore has been considerably modified since settlement of the Port Fairy region. The most significant alterations have been the construction of training works at the Moyne River entrance - so that its confluence with the ocean is now to the north of Griffith Island; and its historical south-west entrance has been closed.

Training works for the river entrance commenced in 1870 and were undertaken to improve vessel navigation into Port Fairy Harbour. Various modifications occurred over the years up to the 1950's; but the training walls have had a significant impact on the natural transport of littoral sand which previously moved eastward past Port Fairy.

BMT WBM (2007) estimated that the volume of sand which has been naturally trapped or mechanically placed to the south and west of the training walls is of the order of 500,000 cubic metres. In the absence of any training walls, this sand would have remained within the active littoral regime - and as such would have been available to feed onto East Beach, thereby maintaining a natural flow of sand around the bay. It is probable that East Beach changed from having a stable shoreline to gradually becoming an eroding beach at about the time the training walls were completed.

In the meantime, land fronting East Beach was developed for residential purposes - presumably on the basis that when the early subdivision survey was undertaken, the shoreline was perceived to be stable.

The earliest available aerial photograph was taken in 1947 and shows that an offshore seawall had been built at that time - refer to Figure 2. It appears that there weren't any seawalls along the beach at that point. Anecdotal evidence suggests that seawall construction started in the late 1950's when erosion began to threaten dwellings/allotments along the beachfront.

By the 1960's the seawall had been extended along the entire foreshore length of the East Beach residential subdivision. Since its initial construction, the seawall has experienced significant settlement and has been damaged by storms. Additional armour rock has been placed at various locations on its length, and some rock armour has reputedly been re-distributed from sturdy sections of the wall to weaker areas.

It is evident from a visual inspection of the structure that the seawall is unlikely to withstand significant storm wave action at present-day sea levels; and would likely be significantly damaged or even fail if anticipated climate change scenarios transpire in coming decades.

The purpose of this Study is to determine the existing structural condition of the seawall and to prepare concept designs and cost estimates for appropriate remediation works - to accommodate present-day and future climate scenarios.

Figure 2 : 1947 Aerial photo showing rock walls at that time

This report provides the findings of the Study and has been structured as follows:

- This Section 1, which consists of an introduction and provides some background regarding the commissioned work.
- Section 2 presents a discussion of the mechanisms by which seawalls in general can be damaged and ultimately fail - along with the appropriate measures that are typically incorporated into designs to ensure their structural integrity. The intent of this section is to highlight the important structural elements and performance criteria associated with seawall structures in general, prior to then considering these issues with specific regard to seawall works at East Beach.
- Section 3 provides a description of the design storm event for present day and future sea level rise scenarios to 2100.

- Section 4 then provides an assessment of the current condition of the seawall. The assessment is based primarily on an inspection undertaken on 11th, 12th and 13th February 2013 by Coastal Engineering Solutions senior staff. The inspections were quite detailed and involved excavating through the seawall profile using a hydraulic excavator, and recording the nature and extent of rock in each exposed cross-section using survey techniques.

- Section 5 presents technically viable Concept Designs to rectify structural deficiencies.

- Cost estimates and construction timescales for upgrading of the various lengths of seawall in accordance with the proposed Concept Designs are then presented in Section 6.

- Section 7 lists the various technical references used in discussions throughout this report.

- Appendices which support the technical content of the report are then included.

2 SEAWALL PERFORMANCE

2.1 Background

Prior to presenting discussions regarding the condition of the seawall at East Beach and any upgrading options, it would be informative to consider the way in which such structures can fail. This understanding of the fundamental damage processes on seawalls assists in appreciating the true condition and vulnerability of the walls at East Beach, and guides the subsequent development and selection of appropriate measures to protect the local foreshore.

Consequently this Section 2 discusses general issues regarding seawall performance and structural integrity - whereas how these various issues specifically affect the condition of the seawalls at East Beach are discussed later in Section 4.

Structural failure of rock armoured seawalls can be caused by any of three fundamental mechanisms - or indeed by any combination of these, namely:

- erosion of the armour layer - instigated when the rocks on the front face of the wall are not able to withstand the forces applied by waves as they wash against the slope. The rocks are effectively washed off the structure by the waves.
- by undermining - occurs when wave action causes scouring of erodible material at the toe of the armoured slope, causing it to be undermined and to then collapse (even though it may consist of large rocks that would otherwise not have been moved by waves).
- by wave overtopping - caused by waves that wash up over the top of the armoured slope and scour the material immediately behind the wall. The top of the wall is then no longer supported by underlying material and it collapses into the scoured area behind it - lowering the top of the seawall further, allowing greater overtopping, greater scour and rapid progression to structural failure.

Further discussion of these failure mechanisms is offered below, along with appropriate mitigating measures that can be incorporated into seawall designs or rehabilitation strategies to avoid or reduce their effects.

It is important to appreciate that whilst the following discussions refer to important elements in the design of new seawalls, the same considerations can be applied to evaluating the structural integrity of existing seawalls - such as those at East Beach.

2.2 Erosion of the Armour Layer

This type of damage is illustrated conceptually in Figure 3. It is instigated when the rock armour on the front face of a seawall is not able to accommodate the forces applied by the larger waves in the sea state as they wash against the slope. It can be due to a number of deficiencies:

- the rocks themselves are too small (or they have broken down over time into smaller sizes);
- the placement density is poor (such that rocks are very loosely placed and easily removed);
- there is insufficient rock coverage on the underlying slope (allowing waves to wash out the material in the underlying bank slope);
- the front slope of the wall is too steep (allowing any loose rocks to be easily dislodged by waves and to roll down off the slope).

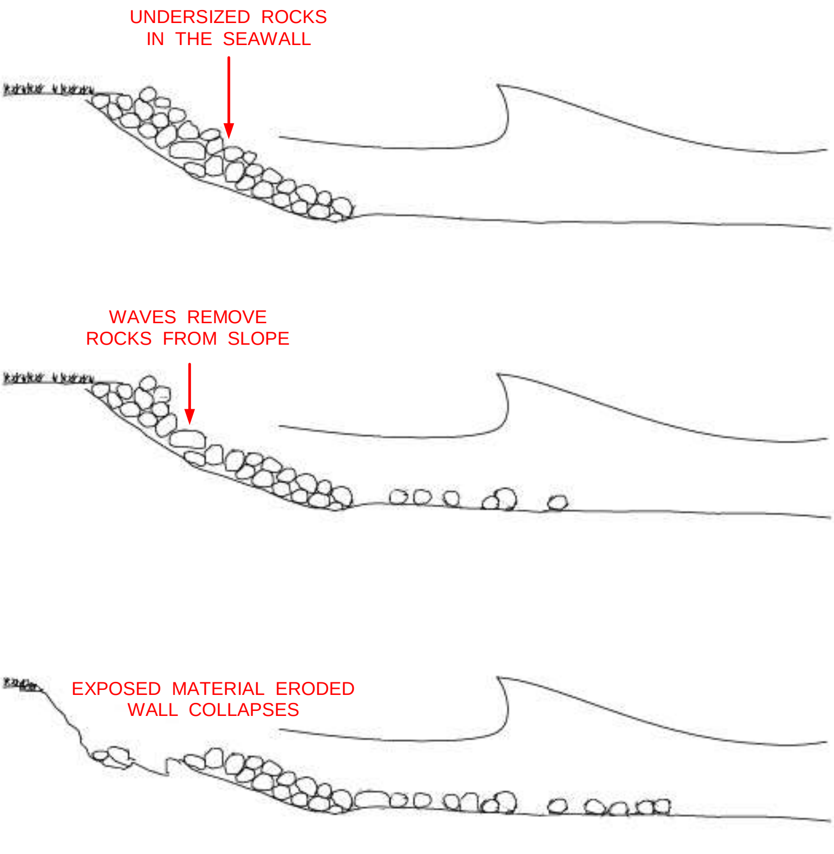


Figure 3 : Damage / Failure Due to Erosion of the Armour Layer

For rock armour structures, a small degree of movement of individual rocks is acceptable. Damage levels of up to 5% may be allowed by designers without the structure being considered as having failed.

Failure of a section of seawall occurs when armour rock is removed from the slope to the extent that the underlying material is exposed. Once waves can wash against the unprotected bank slope and any underlayers of small rock, they can easily scour and remove this material. Collapse of adjoining sections of the armour layer can then be considerable, with failure progressing very rapidly outward from the initial point of failure.

Given the need to adequately protect the underlying bank material, the outer armour on properly constructed seawalls always has at least two layers of armour rock on the slope. In special circumstances single layers of armouring are possible with some precast concrete armour units, such as Accropode or Core-loc systems, which are pattern placed and usually only used when appropriate armour rock is not readily available.

The removal of individual rocks from the face of a seawall under this mode of failure tends not to be caused by those waves in a storm that shoal and break directly onto the rock slope. This is because such waves apply forces that tend to push the rocks into the slope rather than remove them. Whilst this strong impulse from breaking waves can significantly jar individual rocks and potentially loosen their interlocking with surrounding rocks, it may not necessarily remove them from the armouring layer. Instead loose rocks in an armour layer tend to be removed from a seawall by the unbroken waves in the sea state.

The up-rushing and down-rushing of each unbroken wave (as it expends its energy by surging against and through the porous rock slope) is very substantial. It is this "up-slope" and "down-slope" surging of water that removes individual rocks from out of the face of the seawall and rolls them down the slope. Once a rock is removed from the slope, adjoining rocks no longer have the same degree of physical support. The resulting effect is an increased vulnerability of the depleted armour layer to the surging forces running up and down the slope - leading to further removal of rocks and progressive damage leading to structural failure.

Clearly the best way to mitigate this type of action is to ensure that the rocks are large enough and sufficiently interlocked to withstand the uprush and downrush forces applied by waves during storms. The engineering design of seawalls therefore directs considerable focus and effort on ensuring that rocks are correctly sized, and that during construction they are correctly placed as an interlocking matrix on an appropriate slope gradient.

The importance of rock size on seawall stability

The size of rocks required to withstand a particular sea state can be calculated using well established design formulae. Typically the application of these design procedures yields the required weight of individual rocks - since it is the weight of each rock (along with interlocking) that counters the forces trying to remove it from the structure. It is for this reason that designers of marine works will usually specify requirements for armour in terms of rock weight rather than rock dimensions.

The practicalities of rock supply are such that all rocks in a seawall are not the same size - it is inevitable that there will be a range in sizes. So when considering the issue of rock size it is necessary to nominate a representative weight - this is typically the average weight of all of the rocks. Or in other words, it is the weight that 50% of the total number of rocks in the structure exceeds.

The slope of the seawall has a significant bearing on the size of rocks that are necessary to withstand a particular severe wave event. The steeper the slope, the larger is the rock size required. The “angle of repose” is the slope that would form if the rocks were simply dumped into a heap or formed into a seawall by bulk placement techniques. It represents the steepest slope allowable in seawall construction because at this gradient the rocks are at (or near) the point at which they will roll down the slope.

Rather than place armour rocks at this extreme limit of stability, it is usual practice to adopt a maximum allowable slope on the front face of rock armouring of around 1:1½ (ie. 1 vertical unit to 1½ horizontal units, which is approximately 34° from the horizontal).

Flatter slopes enable smaller rocks to be used to counter the design wave forces. However there is a practical limit on just how flat a slope can be built. Gradients flatter than around 1:2½ become quite difficult to construct without the use of specialised equipment. This is because of the requirement for earthmoving equipment placing the rocks to have a long reach out over the constructed slope, particularly when placing rocks on a reasonably high seawall. Despite using smaller rocks, flatter sloped seawalls result in the need for a larger volume of rock armour.

Having determined the size of rocks required to withstand the forces of waves during a storm, it is important that these rocks then don’t breakdown into smaller sizes during the service life of the seawall. It is common for basalt and other volcanic rock types to have inherent joints - most of which were formed in response to shrinkage stresses induced as the lava forming it cooled and solidified. It is similarly common for such joints to contain secondary minerals that would have migrated into the joints during the lava cooling process. Secondary material within the joints can swell or contract in response to moisture and temperature changes.

Consequently when a jointed rock is placed into the marine environment of a seawall (where it is repeatedly subjected to wetting/drying and heating/cooling cycles) the result can be a slow physical degradation of the volcanic rock towards sizes determined by the joint spacing.

The extent of inherent defects is controlled by the geology of the rock source. It is often found that intrusive igneous rock (ie. more slowly cooled and more widely jointed) is a better source of armour than volcanic rock (rapidly cooled and commonly closely jointed).

Petrographic analysis of rock samples and their insitu source can determine whether rocks within existing seawalls (or potential sources of rock for new work) are prone to long-term deterioration in a marine environment.

The importance of rock interlocking on seawall stability

The interlocking of individual rocks within armour layers plays an important role in securing the structural integrity of seawalls.

Simply ensuring that large rocks are used as armour does not ensure that they will not be removed by wave action. Rocks that are not in firm contact with several others in the same layer are therefore loose and are simply sitting on only one or two points of contact - in a potentially unstable position. They can be rocked backwards and forwards by waves surging up and down the face of the seawall. The induced jarring action on these moving rocks can cause them (or others alongside) to fracture under the repeated impacts - with the fragmented rocks then being washed out of the armour and removed from the seawall.

A tightly packed, well interlocked armour layer offers little opportunity for waves to remove individual rocks from the structure. The degree of interlocking within an armour layer is affected by the range of rock sizes that constitute the layer, as well as the shape of the rocks themselves.

As stated previously, it is inevitable that any rock-armoured seawall will consist of a range of rock sizes. If there is a wide range either side of the average, then the interlocking of the preferred size can be compromised. The large number of smaller rocks in a widely graded armour can get in-between and inhibit the firm contact between the larger rock sizes that are required to withstand the wave forces. They may also fill the voids within the armour layer, thereby significantly compromising the seawall's ability to dissipate incoming wave energy.

Similarly, individual very large rocks within a widely graded armour can inhibit effective interlocking by reducing the number of contact points that adjacent rocks might otherwise have with each other - meaning that those rocks aren't as well held within the overall rock matrix because of the presence of the very large rock.

Consequently when specifying the average rock size required for a seawall, designers typically also specify limits on the minimum and maximum rock sizes so as to ensure that interlocking of the completed seawall is not compromised. Internationally accepted design guidelines regarding rock gradings have been developed for this purpose. Construction specifications for rock-armoured marine structures also frequently incorporate strong and clear requirements for individual rocks to be placed so as to be in firm contact with at least three others in the same layer.

Rock shape is also an important consideration in ensuring adequate interlocking within an armour layer. Rocks that are tabular in shape (ie. excessively flat), quite long, and/or cylindrical will not interlock as effectively as cubic or spherical shaped rocks - although very round rocks are not as effective as cubic rocks. Consequently seawall designers will frequently place limits on the shape of rocks (for example, by specifying the maximum allowable ratio of any rock's longest dimension to its shortest dimension).

The importance of rock coverage on seawall stability

Even an armour layer constructed of appropriately sized and interlocked rocks contains significant voids. The void ratio of a properly constructed seawall will typically be around 35% to 40%.

Indeed it is these voids that contribute to the success of a seawall as a coastal defence structure. As waves wash onto and through the armour layer, they lose a significant amount of energy. Consider for example the performance of waves should they encounter a totally impermeable smooth slope on the foreshore. There would be very little loss of wave energy, with such a slope acting as a ramp for the waves to wash over and onto the area behind. A rock armour layer on the other hand absorbs much of the wave energy so that its potential to adversely impinge on the area behind it is significantly reduced.

Nevertheless there is considerable turbulence and movement of water within the voids of a rock armour layer - as a consequence of waves as well as the normal rising and falling of the tides. If the material upon which the rocks are placed is erodible, then it will be washed out between the voids in the outer armour and no longer provide adequate foundation support for the armour layer itself. It is therefore necessary to provide a filter arrangement between the outer armour layer and the material in the underlying bank slope.

Typically this filter is provided by way of an underlayer of smaller rocks - carefully sized to ensure that they themselves aren't washed out between the voids in the outer armour layer, yet still prevent any of the finer bank material from migrating through it. This often requires a geotextile material to be placed directly onto the bank slope beneath the rock underlayer.

Another benefit of this overall filter arrangement is that it improves the overall porosity of the seawall structure - thereby improving its ability to dissipate incoming wave energy.

Summary - mitigating the potential for erosion of the armour layer

To summarise, in order to mitigate the potential for damage or failure of a rock-armoured seawall it is important to ensure that the fundamental aspects discussed above (and listed below) are incorporated into any new structure. Likewise, consideration of these issues with regard to existing seawalls ensures that any assessment of their performance and/or structural integrity addresses the potential for this type of failure mechanism.

- Rocks are to be sized so as to withstand the wave forces associated with storm events.
- As well as the average size of all rocks, the minimum and maximum sizes need to be limited to a reasonably narrow range.
- At least two layers of the specified rock size are to be provided within an outer armour layer.

- All rocks are to be placed so as to be in firm contact with at least three others within the same layer.
- Appropriately designed filter layers are to be included between the primary armour and the bank slope.
- Front slopes should be no steeper than 1 vertical to 1½ horizontal.
- The effect of rock shape on interlocking needs to be considered and limitations imposed.
- Rocks used in seawalls should not contain inherent joints or defects that will cause the rock to breakdown within the local marine environment into smaller sizes.

2.3 Undermining Damage and Failure

The high levels of turbulence generated as incoming waves encounter a seawall can be sufficient to initiate scour at the toe of the seawall. If the seawall is founded at a high level, then this scouring of material in front of it may undermine the foundation of the seawall itself. This failure mechanism is illustrated conceptually in Figure 4.

Undermining causes the rocks in the lower section of the armoured slope to slump or collapse downwards into the scoured foundation, destabilising the upper sections and making the seawall considerably more susceptible to failure by erosion of the armour layer.

Undermining failure can occur during a major storm event that causes significant scouring of the seawall's foundation. However it could well be that the slow gradual removal of material from in front of the wall as a consequence of ambient (ie. day-to-day) conditions could result in the structure being in a vulnerable condition prior to the onset of a storm. Often it is not readily apparent that the level of the beach/seabed in front of a seawall is near to that of its foundation and it is therefore close to being undermined - even by a mild storm event.

Mitigating the potential for undermining of the armour layer

There are basically two ways in which undermining failure can be avoided, namely:

- placing non-erodible material in front of the seawall (ie. toe armour); or
- founding the seawall's armour layer at a depth below the expected level of scour.

The selection of the most appropriate for any particular application is determined primarily by "constructability" issues. That is, which of these two basic options is the easiest and most cost effective to build at particular site.

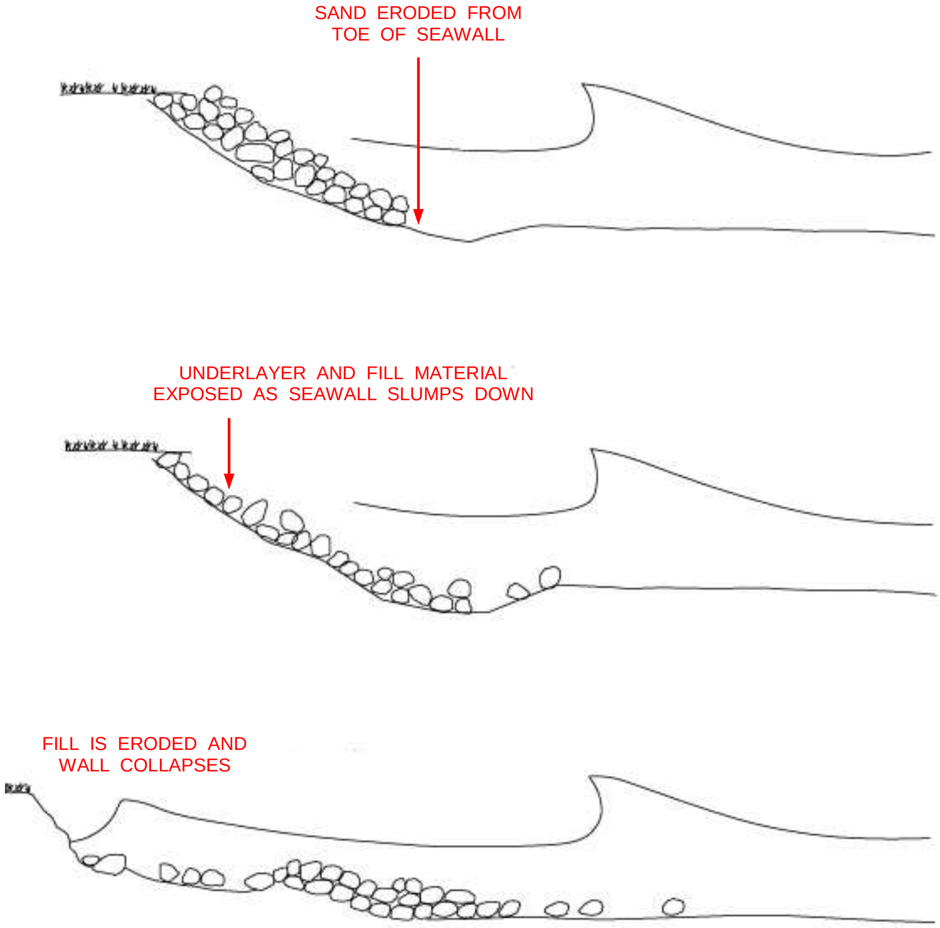


Figure 4 : Damage / Failure Due to Undermining

The most difficult challenge to overcome when providing protection against undermining is the ability to excavate below the surface level (for the subsequent placement of the toe armour, or establishing a deeper foundation level for the seawall). Excavation depths are typically below groundwater and ocean water levels, even during low tides. Where the foreshore / seabed is sandy, such excavations will tend to be unstable due to inflowing water and prone to collapse.

The placement of a horizontal blanket of toe armour is best suited to new seawall construction or seawall repairs that allow placement of the toe armour directly onto the surface immediately in front of the structure. For instance these would be seawalls that have water in front of them at all stages of the tide. This is because it avoids the necessity to excavate.

Adopting this strategy at locations where there is a beach in front of the seawall is not an attractive option, as it adversely affects the foreshore amenity - unless it is buried well below the beach. In which case it becomes necessary to excavate, negating any benefit of simply placing the armour on the surface.

The alternative option of extending the armour layer down to a level below that of the expected scour is best suited to new seawall construction rather than as a simple repair option for existing seawalls, unless the existing seawall is in a very poor condition which would require it to be rebuilt anyway. This is because it would otherwise require the removal of the existing rock slope above the area of the seawall foundation to be deepened, and the reconstruction of the entire slope above the new foundation level.

This option of a deep foundation requires excavation of material that will be below groundwater level and therefore poses the challenges associated with keeping the excavation open for subsequent placement of armour. This was successfully achieved for the recent seawall re-construction at the eastern end of East Beach by only excavating a short length of trench, 5 to 10 metres long and dewatering using the excavator bucket. For sites where the tidal range is high and it is impractical to use such simple excavating and dewatering methods, the construction issue can be overcome by use of light-duty sheet piling or prefabricated shore-trenching frames to temporarily stabilise the excavation.

2.4 Overtopping Damage and Failure

As waves encounter a seawall structure they surge up the slope. If wave run-up levels are high enough during the elevated ocean water levels and strong wave action that occur during severe storms, then the surging water will reach and pass over the crest of the wall. This scenario defines the “green water” overtopping phenomenon where a relatively complete sheet of water surges over the top of the seawall - not just spray. If the material immediately behind the seawall is erodible, then it can be significantly scoured by this green water overtopping.

Scouring of the material supporting the crest of the wall results in collapse of the top section of the armour layer back into the scour hole. This effectively reduces the height of the seawall, thereby increasing the occurrence and severity of subsequent green water overtopping, leading to greater scour, leading to even greater overtopping, etc. This progressive damage can rapidly lead to failure of the seawall. The process is shown conceptually in Figure 5.

The extent of green water overtopping is dependent upon a number of parameters - including the front slope of the seawall, the composition and thickness of the armour layer, and the prevailing wave characteristics (of height, period and incident direction). However the main parameter affecting overtopping rates is the difference in height between the top of the seawall and the ocean water level prevailing at the time of the wave event (ie. the crest freeboard). Clearly a small freeboard results in greater overtopping than a large freeboard.

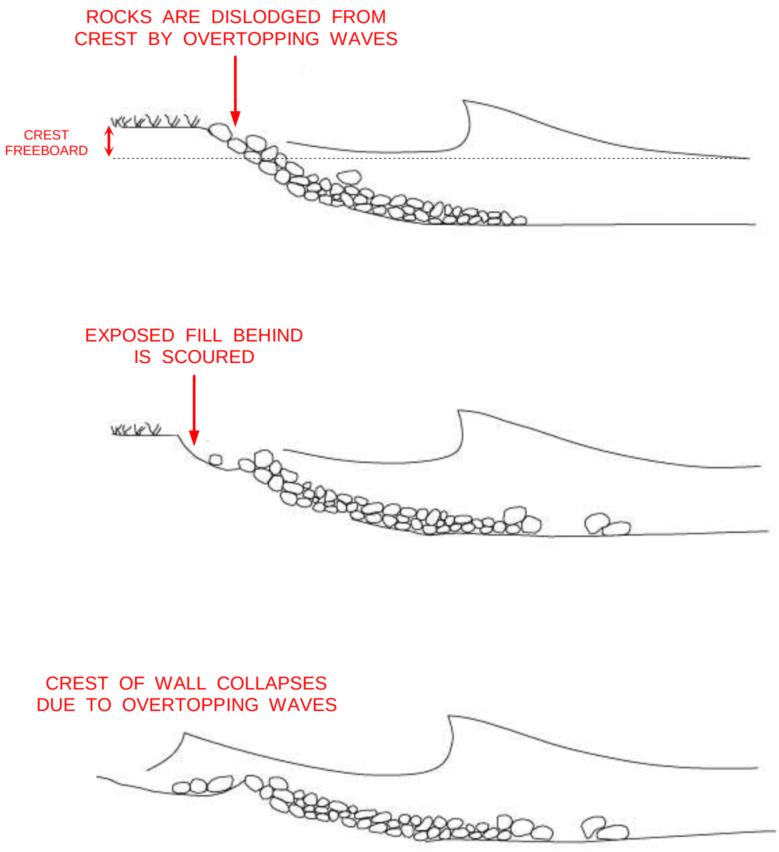


Figure 5 : Damage / Failure Due to Overtopping

There are a number of methods available to calculate the volumes of overtopping (ie. the *overtopping discharges*) for a variety of wave and ocean water conditions, and a variety of seawall types. However when considering the implications to the seawall of the calculated overtopping discharges, the issues are somewhat less exact. Given the current international understanding by engineers and scientists of this complex interaction, it is not possible to give unambiguous or precise limits to tolerable overtopping discharges for all possible conditions.

Nevertheless research of this issue to date enables some guidance to be offered with respect to tolerable overtopping discharges for seawalls such as those at East Beach. The limits listed below for the value Q of overtopping discharge serve as an indication of the need for specific protection to mitigate large overtopping flows (EurOtop, 2007):

$Q < 0.05\ m^3/sec/m$ No damage at rear of seawall crest.

$0.05\ m^3/sec/m < Q < 0.2\ m^3/sec/m$ Damage if the area behind the crest is not paved.

$Q > 0.2\ m^3/sec/m$ Damage even if the area behind the crest is paved.

When designing new seawalls, the extent of overtopping can be estimated and these limits can be considered to ensure that the overtopping water does not have an adverse impact on the land immediately behind the seawall. Likewise they can be used when assessing the performance of existing seawalls under any given storm condition to determine whether green water overtopping will cause damage.

Mitigating the potential for overtopping damage

There are two fundamental ways in which failure by overtopping can be avoided, namely:

- increase the crest freeboard;
- place non-erodible material immediately behind the crest of the seawall (ie. crest armour).

A viable solution for some locations may incorporate features from both of these strategies.

Clearly a means of increasing the freeboard would be for the seawall to have a high crest level. Whilst this may be appropriate at some locations, the height to which a seawall would need to be built so as to contain wave run-up levels to the front slope only (and thereby limit overtopping during severe storm or cyclone events) is typically very high. A high crest may have a significant adverse effect on visual amenity, public access and inhibit the drainage of rainfall runoff from foreshore areas.

The alternative to raising the top of the seawall is to maintain a lower crest level and allow green water overtopping to occur - but place armour rocks in the area immediately behind the seawall. This would prevent this region from scouring and the top of the seawall from failing. The crest armour can be buried below the filled surface level if required and vegetation could be planted over the top. This landscaping would need to be considered as sacrificial - in that it would be washed away during an extreme overtopping event. However the crest armour would remain intact, thereby ensuring the structural integrity of the wall during such an event.

The availability of foreshore land behind a seawall that could be used for crest armour (whether for new seawall construction or as repairs/upgrading of existing walls) may be constrained by existing infrastructure or land tenure. Consequently the optimum solution for mitigating potential damage or failure by overtopping at any particular site is often an appropriate combination of raising the crest as well as armouring the area behind it.

3 THE DESIGN STORM EVENT

3.1 Selection of the Design Event

The preceding section of this report discusses the mechanisms by which rock armoured seawalls can be damaged and ultimately fail. However when considering ways in which these processes can be avoided (either when considering the construction of new seawalls or the rehabilitation / upgrade of existing structures) it is necessary to select a particular storm "event" which the structure must accommodate.

This selection is typically based on an acceptable probability of that event occurring within the length of time that the structure and its components are intended to serve their given purpose (this is termed the *design life* of the structure). The selection of an appropriate *design event* therefore becomes a decision that acknowledges and accepts a particular level of risk that this event (ie. the particular combination of wave conditions and ocean level that the seawall structure is required to accommodate) might be equalled or exceeded within the design life of the structure.

The severity of the design event is quantified by assigning it an *Average Recurrence Interval* (also referred to as a *return period*). This is the average time that elapses between two events that equal or exceed a particular condition. For instance, a 100 year Average Recurrence Interval (ARI) event is one which is expected to be equalled or exceeded on average once every 100 years. However since such events occur randomly in any particular timeframe under consideration (rather than at precise regular or cyclical intervals), they have a probability of occurrence within that time.

Table 3.1 presents a summary of the percentage probability that various ARI events are likely to be equalled or exceeded within particular timeframes. For example a 100 year ARI design event has a 22.1% probability of being equalled or exceeded in any 25 year design life. It also has a 1% chance of occurring or being exceeded in any particular year.

It is standard engineering practice when designing permanent works (as opposed to temporary structures) to adopt design events with ARI much longer than the design life. This philosophy applies to a substantial number of engineered works - whether they are maritime projects, flood mitigation works, buildings, or any other such significant infrastructure or constructed asset.

Given this fundamental engineering design principle, in conjunction with the design life of permanent foreshore protection works typically being no less than 25 years, it is sound practice to adopt at least a 100 year ARI event for the design of such structures.

Number of years within the period	Average Recurrence Interval *(years)*					
	5	10	25	50	100	200
1	*18.1%*	*9.5%*	*3.9%*	*2.0%*	*1.0%*	*0.5%*
2	*33.0%*	*18.1%*	*7.7%*	*3.9%*	*2.0%*	*1.0%*
5	*63.2%*	*39.3%*	*18.1%*	*9.5%*	*4.9%*	*2.5%*
10	*86.5%*	*63.2%*	*33.0%*	*18.1%*	*9.5%*	*4.9%*
25	*99.3%*	*91.8%*	*63.2%*	*39.3%*	*22.1%*	*11.7%*
50	*100.0%*	*99.3%*	*86.5%*	*63.2%*	*39.3%*	*22.1%*
100	*100.0%*	*100.0%*	*98.2%*	*86.5%*	*63.2%*	*39.3%*
200	*100.0%*	*100.0%*	*100.0%*	*98.2%*	*86.5%*	*63.2%*

Table 3.1 : Probability of occurrence of various Average Recurrence Interval Events

In fact adopting a greater ARI than 100 years is often undertaken if the 22.1% chance of it occurring or being exceeded during a 25 year design life is considered to be unacceptable - for instance if it would otherwise result in significant harm to the public, or considerable cost to reinstate or repair damage to property or infrastructure should it occur.

A fundamental consideration in the assessment of the seawall has been the requirements that new foreshore works are to comply with the *Victorian Coastal Strategy, 2008*. That strategy (Victorian Coastal Council, 2008) requires a 100 year ARI event to be accommodated and that the implications of future climate change must be considered.

It is pertinent to note that global sea levels are predicted to rise substantially over the next 50 to 100 years and that sea level rise may be accompanied by an increase in storm intensities. The design ARI event needs to include the implications of both sea level rise and increased "storminess".

The following section of this report offers discussion on the particular wave and ocean water levels that constitute the 100 year ARI design event for the seawall along East Beach.

3.2 Characteristics of the 100 year ARI Design Event

The selection of the 100 year ARI event to be used in the assessment of the East Beach seawall is not a straight forward or simple process, as it consists of a combination of severe waves and extreme ocean water levels. When considering the adequacy of coastal defences it is necessary to consider the likelihood of both conditions occurring simultaneously. The assumption of complete dependence between waves and ocean levels in an analysis of joint occurrence would lead to a conservative assessment - since the 100 year ARI design event would have to comprise a 100 year ARI storm tide level occurring at the same time as the 100 year ARI wave conditions.

Conversely the assumption of complete independence between waves and water levels could lead to under-assessment of structural performance, since any increase in the probability of large waves occurring during storm tide events would have been ignored - clearly an understatement of likely waves. The actual correlation between waves and storm tide levels during a severe storm event will lie between these two extremes of complete dependence and complete independence. Specialist studies are typically required to establish this joint probability.

Wave characteristics and the storm surge can generally be estimated for a storm of any given intensity and size, however the storm tide level depends upon when the peak surge generated by the storm occurs in relation to the astronomical tide. For example, a severe storm which produces high waves and a high surge will not produce a high storm tide if it occurs around the time of low tide. The large surge and severe waves occurring at low tide might result in less wave energy reaching the foreshore (due to the waves breaking in the shallower seabed approaches) than a moderate surge and moderate wave conditions occurring at high tide.

The methodology adopted when assessing the existing East Beach seawall and then developing appropriate options for structural upgrading has been to implement the conservative approach of considering the 100 year ARI design event as consisting of the 100 year ARI wave conditions occurring in conjunction with the 100 year ARI storm tide level.

This approach to the determination of extreme wave and ocean water conditions affecting the East Beach seawall is consistent with the design methodology implemented for the recent extension and structural upgrade of the northern-most end of the seawall that was completed in November 2012.

When selecting appropriate design storm parameters, the findings of the 2009 CSIRO study (McInnes, et al) "*The Effect of Climate Change on Extreme Sea Levels along Victoria’s Coast*" have been used to establish storm tide characteristics and future storm scenarios.

3.2.1 Ocean Water Level

Figure 6 illustrates the primary water level components of a storm tide event. Any increase in ocean water levels as a consequence of future climate change would be in addition to these various natural phenomena. A brief discussion of these components is offered below.

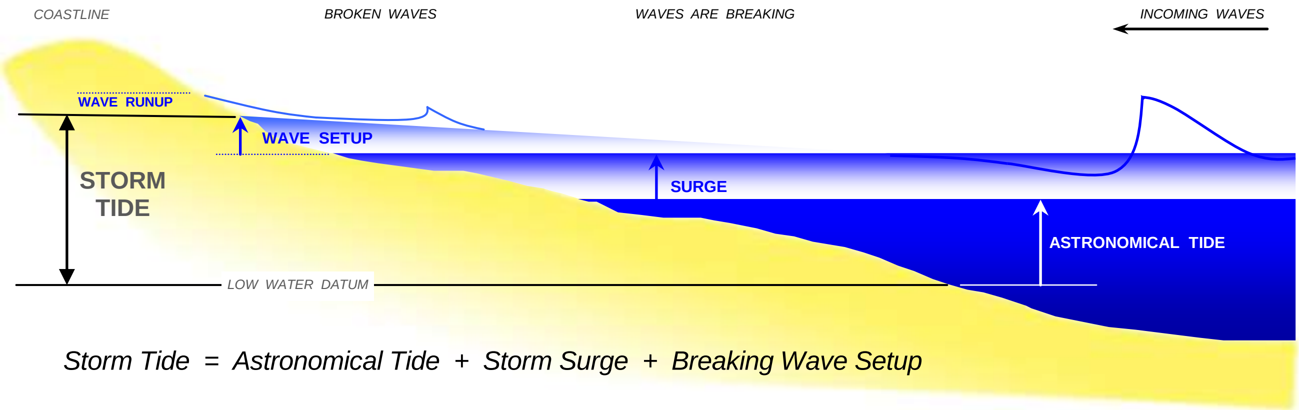


Figure 6 : Components of a Storm Tide Event

Astronomical Tide: The astronomical tide is the normal day-to-day rising and falling of ocean waters in response to the gravitational influences of the sun and the moon. The astronomical tide can be predicted with considerable accuracy. Astronomical tide is an important component of the overall storm tide because if the peak of a severe storm were to coincide with a high spring tide for instance, severe flooding of low lying coastal areas can occur and the upper sections of coastal structures can be subjected to severe wave action.

Storm Surge : This increase in ocean water levels is caused by meteorological effects during severe storms. Strong winds blowing over the surface of the ocean forces water against the coast at a greater rate that it can flow back to sea. Furthermore sea levels can rise locally when a low pressure system occurs over the sea - resulting in what is termed an "*inverted barometer*" effect. A 10mb drop in atmospheric pressure results in an approximate 10 cm rise in sea level. In order to predict the height of storm surges, these various influences and their complex interaction are typically replicated by numerical modelling techniques using computers - such as has been completed for a CSIRO study for Victoria's coast (McInnes et al, 2009).

Breaking Wave Setup: As storm waves propagate into shallower coastal waters, they begin to shoal and will break as they encounter the nearshore region. The dissipation of wave energy during the wave breaking process induces a localised increase in the ocean water level shoreward of the breaking point which is called breaking wave setup. Through the continued action of many breaking waves, the setup experienced on a foreshore during a severe wave event can be sustained for a significant timeframe and needs to be considered as an important component of the overall storm tide on a foreshore.

Wave Runup: Wave runup is the vertical height above the local water level up to which incoming waves will rush when they encounter the land/sea interface. The level to which waves will run up a structure or natural foreshore depends significantly on the nature, slope and extent of the land boundary, as well as the characteristics of the incident waves.

The largely westerly to southerly winds associated with the passage of cold fronts to the south of the Australian mainland have been found to be the primary cause of storm surges along the Victorian coast (McInnes and Hubbert,2003; McInnes et al., 2005). Future climate change scenarios indicate there will be increases in the magnitude of future storm surges due to changes to these meteorological conditions. The CSIRO report (McInnes, et. al. 2009) determines the combined effects of future wind speed changes as well as higher sea levels on 100 year ARI storm tide events at Port Fairy.

Reference to those CSIRO results indicate that the ocean water levels summarised below in Table 3.2 can be used as predictions for 100 year return period storm tide levels at East Beach under future climate change scenarios.

Location	Present-day Climate	Year 2030	Year 2070	Year 2100
Port Fairy	+1.05	+1.25	+1.67	+2.09

(levels are in metres above AHD)

Table 3.2 : Predicted 100 year ARI Tide Levels at East Beach

3.2.2 Wave Conditions

The seabed approach slopes immediately offshore of East Beach are quite flat. As a consequence the nearshore wave climate at the beach is *depth-limited*. This means that it is primarily the depth of water over the nearshore approach slopes of the seabed that determines the characteristics of the waves that reach shore.

As offshore waves propagate shoreward into shallower water, they begin to “feel” the seabed. The decreasing depths in nearshore areas cause the approaching waves to change direction so as to become aligned to the seabed contours and to also shoal up in height until such time as they may break - dissipating their energy as they do so.

Just how much wave energy reaches the shoreline is therefore determined significantly by the depth of water over the seabed approaches. Ocean water levels and the seabed bathymetry are very important aspects in this process of wave energy transmission.

The extent of shoaling/breaking is dependent on a number of characteristics (such as the seabed slope and wave period) but the primary influence is the depth of water through which incoming waves must propagate.

Comprehensive calculations of wave breaking phenomena on the seabed approaches to the East Beach foreshore were undertaken by Coastal Engineering Solutions as part of the detailed engineering design for the recent upgrading and extension of the northern-most end of the East Beach seawall. Those calculations considered the 100 year ARI storm condition and determined the change in wave height distribution within the surf zone due to breaking within a random sea state (using techniques defined in Goda, 2000).

The significant influence that the shallow seabed approaches have on the nearshore wave climate during storms is very apparent from those calculations. The results of this "depth limitation" effect is illustrated in Figure 7, which shows the relationship between *significant wave heights*[1] offshore and those reaching the East Beach shore (when the ocean water level is at the 100 year ARI storm tide level of RL+1.25m AHD predicted for the year 2030).

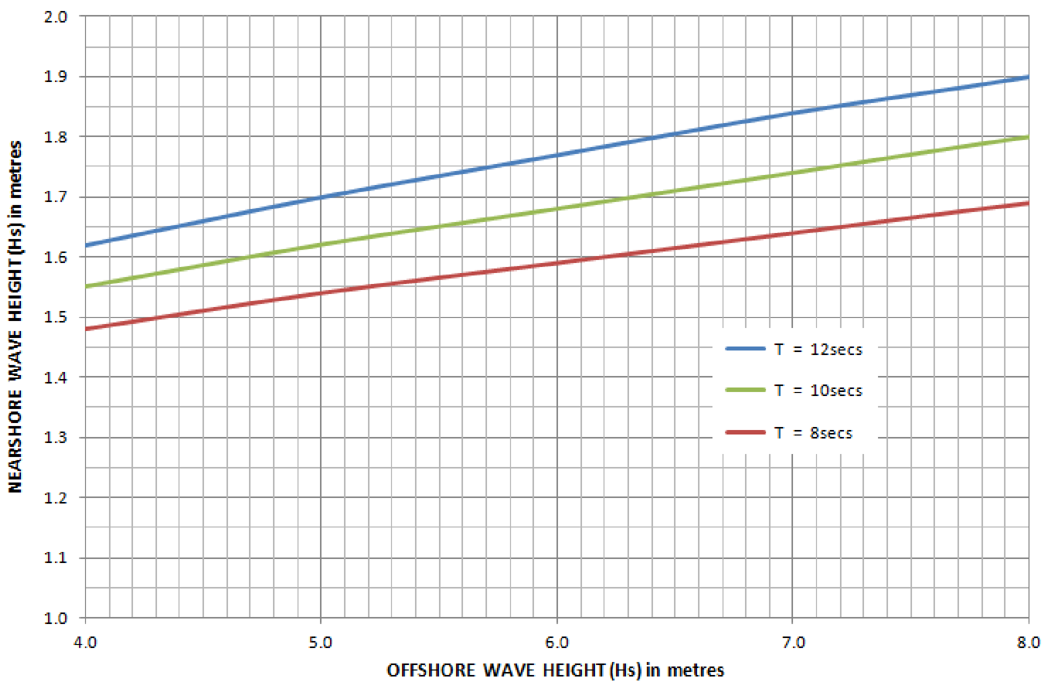


Figure 7 : Relationship between offshore and nearshore waves during the 100 year ARI storm tide (in 2030)

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[1] Due to the random nature and size of waves, the term "*significant wave height*" is used by engineers and scientists to quantify wave heights in a sea state. It represents the average of all of the third highest waves that occur over a particular timeframe. It is typically written as H_s. It is important to appreciate that in deep offshore waters the largest individual wave in the sea state may be up to almost twice the significant wave height.

It is evident that during a storm the depth of water over the nearshore seabed approaches to East Beach causes large waves in the sea state to break - limiting nearshore significant wave heights to less than about 2metres. Even when very large storm waves are generated in deep offshore waters, the waves reaching the East Beach foreshore are limited in height. As can be seen from Figure 7, this relationship between offshore and nearshore waves is also somewhat dependent upon the wave period, but it is primarily the depth of water that determines the wave energy that reaches the foreshore.

It is for this reason that when considering the conditions for the 100 year ARI design event applicable to seawalls on East Beach, the selection of the 100 year ARI storm tide level is critical, whereas the selection of corresponding wave conditions in deep offshore waters is less crucial.

Consideration of the wave breaking phenomenon on the seabed approaches to East Beach result in the wave characteristics presented in Table 3.3 being representative of a 100 year ARI event. The range of possible wave parameters listed in Table 3.3 are the same as those used for the detailed structural design of the recently completed seawall upgrade/extension at the northern end of the East Beach seawall - although for that earlier design, the sensitivity to an increased range of possible wave periods (up to 16 secs) was investigated.

Wave Period	Present-day Climate	Year 2030	Year 2070	Year 2100
8 secs	1.55m	1.65m	1.87m	2.10m
10 secs	1.65m	1.76m	1.97m	2.20m
12 secs	1.75m	1.87m	2.08m	2.35m

Table 3.3 : Predicted 100 year ARI Significant Wave Height at East Beach Seawall

3.3 Accommodation of Future Climate Change

The implications of potential changes to the climatology of the southern regions of Australia as well as possible sea level rise need to be considered and incorporated into structural assessments and designs of foreshore protection works where appropriate.

In reality, any increase in offshore wave characteristics alone are unlikely to result in significant practical changes to the required size of armouring for seawalls. Of greater concern is the potential threat posed by a rise in sea level.

As discussed previously in this report, sea level rise as a consequence of future climate change will result in potentially greater depths of water over the wide flat seabed approach slopes - allowing greater wave energy onto the shore.

The higher ocean level also has the potential for inducing greater wave overtopping on a seawall built to accommodate present-day wave and ocean water conditions. Given that the crest level of any seawall along the East Beach foreshore is likely to be vulnerable to wave overtopping during severe storms, the implications of future sea level rise need to be considered in the assessment and subsequent design of any proposed seawall works.

This is not to suggest that new seawall works need to be constructed to accommodate all aspects of future climate change. It would however be prudent to design and construct the armour layer to accommodate the expected increased wave energy (as a consequence of sea level rise and changed storm climatology). To later increase rock armour characteristics on an existing seawall is extremely difficult and costly - often requiring substantial reconstruction.

On the other hand, the anticipated increase in wave overtopping could be included as subsequent upgrading works if/when it manifests itself. For instance, the design could be undertaken to structurally accommodate present-day overtopping rates by incorporating crest armour and/or raising the crest level, but be cognisant of the possibility that future works (such as a profiled wave wall at the back of the crest armour) may be required in the later years of the seawall's design life. In this way the design and construction would not compromise the options of dealing with increased overtopping due to climate change at a later date.

3.4 Requirements for the Seawall to Accommodate the Design Event

Detailed coastal engineering design techniques were applied to the recently completed seawall upgrading and extension works to the northern-most end of the East Beach seawall. That design work considered the three primary damage/failure mechanisms that could occur within the coastal environment of East Beach (refer to earlier discussions in Section 2) using the same 100 year ARI design parameters as discussed previously in Section 3.2.

The specific structural requirements of that recent seawall upgrade/extension can be used (in conjunction with an assessment of the structural characteristics of the entire length of the seawall) as the basis of determining the ability of the existing structure along East Beach to accommodate the same Design Event.

An earlier study (Coastal Engineering Solutions, 2006) defined the wave climate at a number of nearshore locations along the East Beach foreshore - including that near the recent northern seawall upgrade/extension works. It is evident from that earlier study that the wave conditions (of significant wave height and wave period) during storms did not vary significantly along the East Beach shoreline. The exception to this is near the southern-most end where a semi-submerged offshore rock structure provides some wave protection to the shoreline in its lee.

Given this similarity in the storm wave climate along the 2km long East Beach foreshore, it is appropriate to adopt the structural characteristics of the recent northern seawall upgrade/extension works as being indicative of that required along the East Beach foreshore north of the semi-submerged offshore structure.

The fundamental philosophy adopted during the design of the northern seawall upgrade/extension (and therefore applied to the likely performance of the existing structure) is:

- *to address issues relating to the erosion of the armour layer* – select rock size in accordance with the expected increased wave energy and ocean levels as a consequence of future climate change.
- *to address issues relating to the undermining of the seawall* - determine foundation requirements that accommodate the expected increases in wave energy and sea level as a consequence of future climate change.
- *to address issues relating to the overtopping of the seawall* - determine crest armouring requirements that will accommodate the present-day climate scenario, but be mindful that future climate change and sea level rise may require modifications and/or additional works in future. Consequently new works must not compromise options to accommodate any increased overtopping as actual future climate change influences emerge.

Using the recent upgrading/extension works as the design standard, a summary of the various requirements for the existing seawall to achieve appropriate structural performance are as follows:

- Need approximately 19.7m^3 (ie. approximately 35 tonnes) of rock armouring per linear metre of seawall (including primary armour and underlying filter layer).
- Seawall should be founded at a level no higher than RL-1.25m AHD - so that it is not undermined in future.
- The crest level of the seawall slope should be no lower than RL+3.25m AHD - so that it will not be significantly damaged or fail due to wave overtopping during a severe storm.
- Primary armour must be at least two layers of rock, with 50% of rocks being greater than 2 tonne (with an allowable range in size of 0.5 tonne to 4 tonne).
- Underlayer rock must be in two layers, with an average rock size of 200kg (with an allowable range in size of 50kg to 500kg).
- Minimum thickness of the primary armour layer should be 1650mm; and the minimum underlayer thickness to be 775m.
- Geotextile on underlying bank slope to be Terrafix 600R or equivalent.

4 EXISTING CONDITION OF THE SEAWALL

4.1 Assessment Methodology

The existing seawall at East Beach is approximately 2km long. Figure 8 illustrates the overall extent of the structure. An additional length of about 120 metres was rebuilt and slightly extended at the wall's northern-most end in late-2012 and has therefore been excluded from this assessment of structural condition.

Figure 8 : East Beach seawall and locations selected for structural appraisals

As discussed in Section 2 of this report, damage or failure of a seawall can occur as a consequence of any of three fundamental mechanisms, or indeed by any combination of them. Consequently any assessment of the structural integrity of the East Beach seawall needs to consider the likely performance of the wall against each of these three processes. Any structural shortcomings can then be addressed through appropriately designed remedial works.

The "design standard" adopted for these requirements has been the structural design of the recently completed upgrading/extension works at the northern-most end of the East Beach seawall. That structure was designed to accommodate a 100 year ARI event, and its design made appropriate allowances to accommodate the effects of future climate change.

To determine an appropriate comparison with this design standard (and hence the susceptibility of the existing seawall to damage/failure) a site inspection was undertaken. Aspects examined included the estimated average size of armour rocks; the range in rock sizes; the number of armour layers; rock placement density and interlocking; existence or otherwise of any filter medium (rock or geotextile); toe level; crest level and front face slope.

Given the variable nature of the seawall, this presented somewhat of a challenge since it required categorising different lengths according to their structural form and condition.

The specific methodology used to establish characteristics of the existing seawall has been as follows:

- Thirty-nine (39No.) locations were chosen at approximate 50 metre intervals along the alignment of the East Beach seawall. Their selection was primarily guided by the need to identify sections that were typical or representative of the seawall - bearing in mind the variable nature and condition of the wall along its length. These thirty-nine locations are shown in Figure 8.
- In order to facilitate the assessment, intrusive investigative measures were implemented - namely the pulling apart of sections of the seawall by a hydraulic excavator. Each section of seawall was then immediately rebuilt once the condition and elements of its construction were inspected and recorded by a coastal engineer specialised in seawall design/construction.
- A surveyor was on site throughout the investigation, and for each excavated section the following aspects were identified by survey:
- the existing location of the beach against the sloping front face of the rock seawall;
- the seaward toe of the wall when the sand had been excavated down to the base of the rock armour;

- the landward bottom corner of the seawall where the rock started to slope up the embankment; and
- the top rear side of the rock embankment.
- By considering the survey points at each cross-section it is possible to estimate the volume of rock currently within the seawall.
- A photograph was taken of the seawall at each of these survey steps for each cross section to provide a visual record of the seawall's condition.
- An approximate count was made of individual rocks to then estimate the percentage by volume of the primary armour rock and the underlayer rock.

A summary of the results of the investigations is provided in Table 4.1 on the following page. The complete Site Log of information is included in an Appendix to this report and provides a baseline record of the condition and nature of the seawall along its entire length at the time of the site inspection.

Section	Typical Armour Size (Tonne)	Armour %	Underlayer % (typical)	Toe level of rock m(AHD)	Rear base level of rock m(AHD)	rock meets beach m(AHD)	Top of rock level m(AHD)	Total volume of rock m³/m length of wall	Comment on land behind
1	1/4 to 1	60	40	0.1	0.2	1.8	1.9	4.8	dune/veg
2	1/4 to 1	50	50	0.3	0.8	1.3	4.7	6.6	dune/veg
3	1/4 to 1	40	60	0.3	0.7	1.3	2.1	4.7	dune/veg
4	nil	0	100	0.6	0.6	1.3	3	2.7*	} fence re
5	nil	0	100	0.3	0.5	1.2	2.3	3.4*	} private
6	1/4 to 1	20	80	0.2	0.5	1.3	1.9	2.5*	} land
7	1/4 to 2	80	20	0.7	1.1	1.8	2.9	5.0	dune/veg
8	1/4 to 2	80	20	0.4	0.8	1.4	2.5	4.2	dune/path
9	1/4 to 2	70	30	0.3	0.8	1.3	2.2	4.6	dune/veg
10	1/4 to 1	70	30	0.2	0.5	1.3	1.7	3.8	dune/veg
11	1/4 to 1	60	40	0.2	0.5	1.4	2.3	3.8	dune/veg
12	1/4 to 1	10	90	0.4	0.8	1.5	2.4	4.9	dune/veg
13	1/4 to 1	60	40	0.4	0.5	1.7	2.2	3.7	dune/veg
14	1/4 to 1	40	60	0.3	0.6	1.5	2.3	4.1	dune/veg
15	1/4 to 1	40	60	0.3	0.6	1.3	2.1	4.0	dune/grass
16	1/4 to 2	50	50	0.2	0.6	1.5	2.6	4.6	dune/grass/road
17	1/4 to 2	30	70	0.2	0.3	1.4	3.2	8.0	dune/grass
18	1/4 to1/2	70	30	0	0.3	1.4	4	7.6	dune/grass
19	1/4 to 2	50	50	-0.3	0.3	1.3	4.2	12.1	dune/veg
20	1/4 to 2	50	50	-0.4	0	1.4	4.4	13.3**	dune/veg/SLSC
21	1/4 to 2	50	50	-0.4	-0.1	1.3	3.8	6.6	path (RL3.9)/dune
22	1/4 to 2	50	50	0.1	0.1	1.3	3.5	7.7	path (RL3.9)/dune
23	nil	0	100	0.1	0.5	1.3	5.8	8.1	embankment/road
24	1/4 to 1	60	40	-0.1	-0.1	1.4	2.5	5.3	embankment/road
25	1/4 to 2	50	50	-0.1	0	1.3	6	8.7	embankment/road
26	1/4 to 1	10	90	-0.1	0.5	1.3	6.6	13.8	embankment/road
27	1/4 to 1	10	90	0	0.6	1.4	6.7	7.3	embankment/road
28	1/4 to 1	10	90	0	0.5	1.4	5.4	8.4	embankment/road
29	1/4 to 2	60	40	0.1	0.2	1.3	4.5	11.3	dune/path
30	1/4 to 1	50	50	0.2	0.8	1.5	3.4	4.3	dune/path
31	1/4 to 1	50	50	0.4	0.7	1.3	3.6	5.1	dune/path
32	1/4 to 1	50	50	0.3	0.8	1.4	3.3	4.4	dune/path
33	1/4 to 1	50	50	0.4	0.6	1.4	3.1	4.6	dune/veg
34	1/4 to 1 1/2	50	50	0.3	0.9	1.4	3.8	5.6	dune/veg
35	1/4 to 1	50	50	0.3	1	1.4	4	5.5	dune/veg
36	1/4 to 1	50	50	0.2	0.9	1.4	3.5	6.1	dune/veg
37	1/4 to 1	30	70	0.1	1.1	1.4	3.9	6.7	dune/veg
38	1/4 to 1 1/2	30	70	0.1	1	1.4	3.9	6.7	dune/veg
39	1/4 to 1 1/2	30	70	0	1	1.6	3.8	6.4	dune/veg

Table 4.1 : Summary of characteristics of the existing seawall

Notes:
** property fence line straddles the seawall – investigation limited to seaward side of the fence.*
*** there appears to be a considerable amount of buried rock behind the seawall – not disturbed.*

4.2 Seawall Categorisation

The standard of construction of the East Beach seawall is extremely variable. Seawall types have been identified by reference to the investigated cross-sections shown in Figure 8 and their characteristics as listed in Table 4.1.

The site investigations were undertaken when beach levels along the entire length of the foreshore were seasonally high. Typically beach sand had to be excavated to a depth of around 1 metre to 1.5 metres to reach the foundation of the seawall.

The investigations identified ten different seawall categories along the East Beach foreshore, namely:

I. Low seawall backed by dune and park;

II. Low seawall with dwelling fence line within seawall area;

III. Low seawall with vegetated dune and dwelling fence line set back from seawall;

IV. Low seawall with grassed embankment above the wall;

V. Mid-height seawall with grassed embankment above seawall;

VI. Mid-height seawall with embankment in front of the SLSC;

VII. Mid-height seawall with path above the seawall, then dune above the path;

VIII. High steep embankment and seawall with road above;

IX. Mid-height seawall with vegetated dune above the seawall;

X. Temporary seawall steps immediately adjacent to recent 2012 seawall upgrade.

A discussion of each of these categories is offered below.

4.2.1 Low seawall backed by dune and park reserve

Excavated cross sections 1, 2 and 3 (having an approximate foreshore length of 150 metres) fall into this category. The seawall is backed by Battery Point Reserve and generally has a low-level vegetated dune located behind it. Figure 9 illustrates the typical nature of this section of the seawall.

This section of foreshore experiences some protection afforded by a low-level offshore rock structure which was apparently constructed prior to 1947 - since this is the date of the earliest aerial photograph in which the structure can be seen.

The seawall consists of loosely placed rocks with no real structural form. It is estimated that the placement density is not greater than $1.5 t/m^3$. The total weight of rock in this seawall is therefore about 1,200 tonnes - with approximately 50% being small armour rock (of 0.25 tonne to 1 tonne) and the remaining 50% being underlayer rock.

The need for remedial work is not urgent since there is no essential infrastructure behind the wall which would be at risk should this section fail. Furthermore the presence of the offshore rock structure dissipates some wave energy before it reaches this section of the seawall.

Nevertheless should the predicted future sea level rise of 0.8 metres occur by the year 2100, it is expected that the offshore structure will provide a much reduced benefit; and that the seawall would be breeched with a possible breakthrough to the Moyne River if the wall was left in its present condition.

Figure 9 : Typical seawall for Section 1 to 3

4.2.2 Low seawall, fence line within seawall area

Excavated cross sections 4, 5 and 6 (with an approximate foreshore length of 175 metres) fall into this category. It appears that fence lines on property boundaries are located along the top of the seawall. Excavations by the hydraulic excavator avoided moving material at (or landward) of any fences so as to not damage them.

Therefore it Is not known whether there is any suitable rock armour buried landward of the fence - serving as either supplementary foreshore defence or as a potential source for re-constructing or repairing the seawall.

This section of foreshore is not protected by the offshore rock structure - which terminates approximately opposite cross section 4. Figure 10 shows the seawall and the proximity of the property fence line.

It is pertinent to note from a Google Earth image of the area (refer to Figure 11) that there was originally a road along the seaward side of existing foreshore buildings. The implication is that the local foreshore eroded some 50 metres following the construction of the Moyne River training walls. It also suggests that this section of the foreshore is vulnerable during large storms, even at present-day sea levels.

Figure 10 : Seawall at Section 5

All of the rocks are loosely packed along this section of the seawall. The total mass of rock located seaward of the fence line is estimated to be about 800 tonnes along the 175 metre length of foreshore represented by Sections 4 to 6.

There is effectively no primary armour rock present; and most of the existing rock could only be reused as underlayer material. It is likely that any seawall re-construction on this section of foreshore would need to be on public land and therefore located seaward of the fence line. This is an area of high priority if the Council wishes to prevent the collapse of the seawall and the subsequent erosion of the land behind the fence.

4.2.3 Low seawall, vegetated dune with fence line set back from seawall

This particular seawall type occurs between Sections 7 and 14 - having an overall length of approximately 470 metres. Here the fence along the property boundary is set back from the seawall.

Except for a location near Section 12, there is a considerable quantity of primary armour rock in the seawall – estimated as 60% to 70% of the overall amount. The seawall in the vicinity Section 12 has only about 10% as primary armour rock.

Figure 11 : Aerial photograph showing road seaward of Sections 4 to 6

As can be seen in Figure 12, the seawall is presently almost completely buried by a low vegetated foredune. The quantity of rock in the seawall is typically about 4.3m^3 per linear metre of foreshore. Again the rock armour appears to be loosely placed, and the total quantity of rock in this 470 metre length of seawall is estimated at 3,000 tonne.

The seawall is very low and has a typical crest elevation of +2.2m AHD. During severe winter storms, much of the sand on the beach can be removed from in front of the seawall. However, the seawall dissipates some incoming wave energy under the present-day climate scenario. Due to green water overtopping there may nevertheless be some erosion of the foreshore behind the seawall, but the dwellings which are well set back from the wall are not threatened.

Given a predicted sea level rise of 0.8 metres by the year 2100, it is likely that there will not be sufficient sand to allow the dune to naturally reform. Therefore under such a future climate change scenario, waves overtopping the seawall could cause significant erosion of the foreshore and adversely impact on dwellings located behind the seawall.

Figure 12 : Seawall at Section 10

4.2.4 Low seawall with grassed embankment above the wall

Sections 15 and 16, being typical of an approximate foreshore length of 100 metres, fall into this classification. The embankment behind this particular length of seawall does not contain any large shrubs or trees and is primarily covered by grass. The dwellings located behind the seawall have a clear view of the sea. Figure 13 illustrates the typical foreshore characteristics.

The top of the seawall is at about +2.5 metres AHD, but the volume of rock in the wall is estimated at only about 4.5m^3 per linear metre of wall. The toe of the seawall is located about 1.2 metres below the beach level (as illustrated in Figure 14). The beach level is lower than for the foreshore to the south; and the rock appears to be slightly more densely placed. A placing density of 1.6t/m^3 is estimated, which results in about 700 tonnes of rock within this 100 metre length of foreshore. The type of rock is equally divided between primary armour (of 0.25 tonnes to 2 tonne size) and underlayer rock.

Figure 13 : Seawall between Sections 15 and 16

Figure 14 : Toe of the seawall at Section 16

Whilst this section of foreshore/seawall appears visually to be quite different to that of Sections 7 to 14 immediately to the south, the protection provided by the structure and the threat posed to dwellings behind the seawall are similar.

That is, for the present-day climate scenario there will be periods of time following storms where the sand level on the beach will be lowered by over a metre. This will allow larger waves to reach (and potentially overtop) the seawall - resulting in local scour of the embankment. However, such erosion is not likely to impact directly on adjacent dwellings. For the year 2100 climate scenario, erosion may be quite severe and begin to adversely affect private properties.

4.2.5 Mid-height seawall with grassed embankment above

Excavated cross sections 17 and 18 (having an approximate foreshore length of 120 metres) fall into this category. The crest of the seawall is higher here than it is to the south - with the average crest level being about +3.5 metres AHD. Visually the seawall and embankment are similar to the adjacent Sections 15 and 16. Figure 15 shows the seawall in this area (with the rock excavated from the seawall at Section 17 in the foreground).

Figure 15 : Typical seawall at Sections 17 to 18 (with excavated rock in foreground)

Overall the rock consists of 50% primary armour (of 0.25 tonnes to 1 tonne) and 50% underlayer rock. With most of the rock located within the originally constructed seawall, it is estimated that the density of placement is 1.6t/m^3, resulting in a rock quantity of some 1,500 tonnes over the 120 length of this particular foreshore.

Whilst the crest level of the seawall is sufficiently high to limit wave overtopping under present-day conditions, the thickness of the rock layers and the size of the armour rock are not sufficient to prevent damage during severe storm events. The extent of damage during a severe storm is likely to be modest - with erosion of the embankment limited to only a few metres.

As for all of the seawall to the south of this location, the existing seawall is likely to fail under storm conditions associated with future climate change influences of the year 2100. The dwellings are well set back from the seawall and whilst there will be an increasing threat emerging in future years, the dwellings are not expected to be undermined. Seawall upgrade works for Section 17 to 18 should therefore have lower priority than those for Sections 1 to 16 to the south.

4.2.6 Mid-height seawall with embankment in front of the SLSC

This is the most robust length of seawall within the investigation area. Excavated cross sections 19 and 20 (with an approximate length of 50 metres) are representative of this seawall type, which extends to the southern side of the boat ramp. The main features of this seawall type are:

- crest level of the wall being approximately +4.3 metres AHD;
- armour rock having a range in size of 0.25 tonne to 2 tonne;
- 50% of the rock is primary armour and the balance is underlayer rock.

The volume of rock in the seawall is estimated to be in excess of 12 m^3 per linear metre, which equates to about 19t/m length. However this quantity of rock is less than that recently installed in the seawall works at the northern-most end of the beach, where approximately 35t/m of rock was used. The total quantity of rock along this reach is about 1,000 tonnes.

Figure 16 shows that the depth to the seawall toe is almost 2 metres below the beach level. The seawall is adequate for the present-day climate scenario since damage is estimated to be limited to around 10% - 15% during a design storm. This damage is likely to occur due to the limited interlocking of the existing armour; and the armour rock not being consistently two layers thick. Nevertheless this section of seawall could be readily repaired; and any erosion behind the wall initiated by overtopping is expected to be minimal.

Under the future climate scenario of the year 2100, the wall would sustain substantial structural damage but erosion is not expected to adversely affect the SLSC.

Figure 16 : Seawall at Section 20 (with surveyor standing at the excavated toe of the seawall)

4.2.7 Mid-height seawall, path above the wall, dune above the path

This seawall type extends from the eastern end of the foreshore fronting the SLSC up to a ramp to the west of public toilet and shower facilities. Sections 21 and 22 (with an approximate length of 120 metres) are typical of this seawall classification.

Its main characteristic is a pedestrian pathway located within about 1 metre of the top of the wall. This is illustrated in Figure 17. It is likely that the original dune was grassed as part of landscaping works; and the parking area and roadway constructed along the crest of this dune. The rock in this length of seawall is again about 50% primary armour (of 0.25 tonne to 2 tonne) and 50% underlayer rock.

However the volume of rock in the seawall (per linear metre of wall) is about 11 tonnes of rock per metre length of wall – estimated at 1,300 tonne in total for this wall type.

The structural performance of this seawall will be similar to that for Sections 17 and 18 - which implies that the pedestrian pathway is at risk during a severe storm under present-day climate conditions.

Given the higher sea level and increased wave energy associated with the year 2100 climate scenario, the pathway would be lost and erosion of the dune could be sufficiently severe as to threaten the parking area behind it.

Figure 17 : Seawall from above Section 21 showing proximity of pathway

4.2.8 High steep embankment and seawall with road above

Excavated cross sections 23 to 29 (having an approximate foreshore length of 360 metres) fall into this category. This particular seawall type is considered to be the most vulnerable along the whole study length. This is because the embankment slope above and behind the seawall is very steep; and there is infrastructure located near the top of the embankment.

The embankment slope is typically around the natural angle of repose for granular material, which is too steep for a seawall to be stable. Furthermore (except for near cross sections 24 and 25) the rock is small; and there is virtually no suitable primary armour rock within the structure. Figure 18 and Figure 19 illustrate the steepness of the wall and the location of infrastructure in close proximity to the crest of the embankment.

Figure 18 : Steep embankment above the seawall at the amenities building

Figure 19 : Steep embankment and seawall (with the road at the crest)

It is estimated that the quantity of rock in this section of the wall is around 14 tonnes per metre of its length, yielding a total of approximately 5,000 tonnes. However, because of the steep slope of the wall and the proximity of the infrastructure at the top of the embankment, it is unlikely to be practical to retrieve the existing rock for seawall reconstruction. Removal of the rock could result in the embankment slipping, thereby damaging or removing the pavement area above it.

Considering the small armour rock along most of this seawall section, it is a little surprising that the extent of damage during past storms has not been greater.

A reason for this is likely to be due to the armour around mean sea level being at least 2 metres thick. Consequently this length of seawall has performed as a *dynamically stable* structure, with individual rocks being moved by storm waves off the upper slope down to the toe area. However, this natural re-profiling means that the seawall is now structurally diminished above a level of about +2metres AHD. The implication is that if a storm with an ARI of around 100 years occurred during winter (when the normal beach level is often a metre lower than the level during summer) then the upper part of the seawall would fail. The embankment above the seawall would then no longer be supported and could also fail. The road and paved areas would then be at risk of damage or loss.

For the elevated sea levels and wave energy associated with the year 2100 climate scenario, it is very likely that this section of seawall would fail; the pavement would collapse adjacent to the top of the embankment; and erosion would adversely affect the amenities building.

4.2.9 Mid-height seawall with vegetated dune above

Excavated cross sections 30 to 39 (with an approximate length of 470 metres) are representative of the remaining length of the East Beach seawall. It has fairly uniform characteristics in terms of structural form, embankment height and the vegetation cover on the upper sections of embankment.

Most of this seawall has experienced structural settlement. Additional armour rock has previously been placed on the crest of the seawall - with the apparent intent of preventing further erosion of the dune/embankment face behind and above the seawall. Figure 20 illustrates the typical characteristics of this seawall type where the rock at the top of the seawall is smaller and of a different (less weathered) appearance than the rock on the lower part of the seawall.

About 70% of the rock in this seawall type has an equal mix of primary armour (of 0.25 tonne to 1 tonne) and underlayer rock. The other 30% of the seawall contains smaller rock, with the percentage of primary armour rock being only around 30%.

The quantity of rock per metre length of seawall is approximately 9 tonnes, resulting in a total yield of about 4,000 tonnes of rock over this 450 metre segment of seawall.

Whilst much of this seawall length has been rehabilitated previously by the placement of small rock at the top of the wall, it is still of an insufficient standard to withstand a 100 year ARI storm under present-day conditions.

This section of wall would be damaged and erosion of the embankment behind it is likely to occur - but the extent of erosion is unlikely to impact on the dwellings behind the seawall. For the year 2100 climate scenario, there will be significant structural damage, and erosion may directly threaten adjacent properties.

The structural standard of this seawall type lies between that of the wall at Sections 16 and 17 and that of Sections 18 and 19. Consequently its priority for remedial works also falls between the priorities for those other seawall types.

Figure 20 : Typical seawall – Sections 30 to 39

4.2.10 Temporary seawall steps adjacent to 2012 seawall upgrade

This 30 metre length of seawall is located between the northern-most beach access steps and the seawall works that were completed in November 2012. Given its short length, some rehabilitation works were undertaken as part of the excavation and reinstatement of the seawall conducted for this structural assessment.

This short length of the East Beach seawall had previously been dismantled in anticipation of the earlier seawall works (undertaken in November 2012) being extended southward to the steps. This did not transpire at that time due to funding constraints.

The opportunity was therefore taken to reinstate this length of seawall to at least the same (or slightly better) condition than the original wall. The rehabilitation was implemented by:

- excavating the beach down to a similar toe level as had existed prior to the earlier removal of rock;
- placing available geotextile material longitudinally on this length of seawall. This resulted in a 4 metre width of geotextile on the embankment slope;
- placing underlayer rock on the geotextile; and then
- placing the available primary armour rock.

Unlike the upgrading works completed in November 2012 on the seawall to the immediate north, no buttress rocks were installed at the toe of the seawall. Likewise the armour rock size was closer to 1 than to the 2 tonne specified for the upgrading.

Figure 21 shows the resulting rehabilitated seawall. It has a similar crest elevation as the works completed in November 2012, but the thickness of the armour is less. This particular section of the seawall will need to be dismantled and rebuilt to the structural standard of the 2012 seawall, but that work has a low priority.

Figure 21 : Restored seawall between beach access steps and western end of 2012 seawall

4.3 Summary of Seawall Condition

The overall seawall length of about 2,045 metres has been categorised into ten types (classified as Category I to Category X) - each of varying length and structural condition. These are summarised overleaf in Table 4.2, along with the assessment as to whether or not each section of the existing seawall complies with the 100 year ARI design standard. The notes in Table 4.2 provide guidance as to the compliance criteria.

It is pertinent to note that there is no section of the existing seawall which has been constructed with a geotextile beneath armour rock. Such an arrangement inhibits the natural leaching of sand from the embankment behind the seawall through the rock armoured slope. The seawall was primarily constructed along the face of an existing sand dune; and therefore the placement of an adequate filtering arrangement is a high priority.

A	B	C	C	D	E	F	G	H	I	J
Seawall Category	Excavated Cross Sections	Chainage on wall (metres)	Length of Category (metres)	Typical Armour (tonnes)	Volume of rock (cu.m/m)	Toe level of wall (to AHD)	Crest level of wall (to AHD)	Complies with erosion of armour?	complies with undermining?	complies with overtopping?
I	1 to 3	0-150m	150m	0.25 to 1	5.4	+0.2m	+2.9m	NO	NO	NO
II	4 to 6	150-325m	175m	nil	2.9	+0.4m	+2.7m	NO	NO	NO
III	7 to 14	325-795m	470m	0.25 to 1.5	4.3	+0.4m	+2.3m	NO	NO	NO
IV	15 and 16	795-895m	100m	0.25 to 1.5	4.3	+0.3m	+2.3m	NO	NO	NO
V	17 and 18	895-1015m	120m	0.25 to 1	7.8	+0.1m	+3.6m	NO	NO	YES
VI	19 and 20	1015-1065m	50m	0.25 to 2	12.7	-0.3m	+4.3m	NO	NO	YES
VII	21 and 22	1065-1185m	120m	0.25 to 2	6.6	-0.4m	+3.8m	NO	NO	YES
VIII	23 to 29	1185-1545m	360m	0.25 to 1	9.0	0.0m	+5.1m	NO	NO	YES
IX	30 to 39	1545-2015m	470m	0.25 to 1	5.5	+0.2m	+3.9m	NO	NO	YES
X	steps to new wall	2015-2045m	30m	1	approx 11	approx -1.0m	approx +3	NO	NO	YES

NOTES

Column H : compliance with requirement to mitigate erosion of armour?
Requires the typical armour size in Column D to be at least 2 tonnes, and the volume of rock in column E to be at least 19.7 cu.m. (ie. 35 tonnes) per linear metre

Column I : compliance with requirement to mitigate undermining?
Requires the toe level of the seawall in Column F to be at or below RL-1.25m AHD.

Column J : compliance with requirement to mitigate wave overtopping?
Requires the crest level of the seawall in Column G to be at or above RL+3.25m AHD.

Table 4.2 : Summary of existing seawall characteristics and structural performance

5 OPTIONS FOR UPGRADING THE SEAWALL

5.1 Prioritisation of Remedial Works

The preceding Section 4 presented an assessment of the condition of the existing East Beach seawall and its' likely ability to accommodate a 100 year ARI storm event. That assessment was undertaken through consideration of present-day climate conditions and the predicted climate scenario in the year 2100. It is evident that the existing seawall is of a variable standard along its length, but is generally of inadequate structural condition - with many inherent defects.

It is apparent from the detailed examination of the seawall, that the deficiencies in the wall have been progressing throughout the structure despite intermittent remedial works. Such works appear to have been undertaken on a reactive basis - to repair local failures as they emerge. The requirement for this type of reactive remedial works will increase in frequency and extent as the structural condition of the wall deteriorates.

Such “patch up” works do not address the extensive structural shortcomings that are inherent in the seawall. Nor do they improve the standard of the wall to the extent that it can withstand a severe storm event.

There is a clear need to upgrade the seawall at East Beach to an appropriate structural standard of foreshore protection. As part of this Study, requirements for remediation have been prioritised based on the combined assessment of the condition of the existing wall and the vulnerability of private and public infrastructure behind the seawall. Based on this assessment, it is evident that remediation works could be staged over the coming 20 years.

Given the existing condition of the seawall (summarised in Table 4.2) in conjunction with the nature of the foreshore immediately behind the structure itself, a ranking of priority for the necessary remediation works has been compiled. This is presented overleaf in Table 5.1 and in Figure 22 for remediation priorities up to 2020.

It is strongly recommended that the Category VIII length of seawall (from Cross Section 23 to 29), be upgraded as soon as possible since it has a road and car parking areas very close to the top of the wall. These public assets are currently threatened by failure during severe storm, even under the present-day climate scenario.

Other seawall remedial works are less urgent. The sections of seawall in front of private dwellings (Category II, between Cross Sections 4 and 6); and that protecting the pedestrian pathway (Category VII, between Cross Sections 21 and 22) have a high risk of failure but the consequences for present-day climate conditions are not as severe.

Seawall Category	Excavated Cross Sections	Length of Category (metres)	Chainage on wall (metres)	Foreshore type	Remediation priority by 2020	Remediation priority by 2100
I	1 to 3	150m	0-150m	dune & vegetation	LOW	HIGH
II	4 to 6	175m	150-325m	property boundary	HIGH	VERY HIGH
III	7 to 14	470m	325-795m	dune & vegetation	MEDIUM	HIGH
IV	15 and 16	100m	795-895m	embankment / grass	MEDIUM	HIGH
V	17 and 18	120m	895-1015m	embankment / grass	LOW	MEDIUM
VI	19 and 20	50m	1015-1065m	dune & vegetation & SLSC	LOW	MEDIUM
VII	21 and 22	120m	1065-1185m	path (+3.9m AHD) & dune	HIGH	VERY HIGH
VIII	23 to 29	360m	1185-1545m	embankment / road	VERY HIGH	CRITICAL
IX	30 to 39	470m	1545-2015m	dune / path	MEDIUM	HIGH
X	steps to new wall	30m	2015-2045m	dune & vegetation	LOW	MEDIUM

Table 5.1 : Summary of seawall remediation priorities

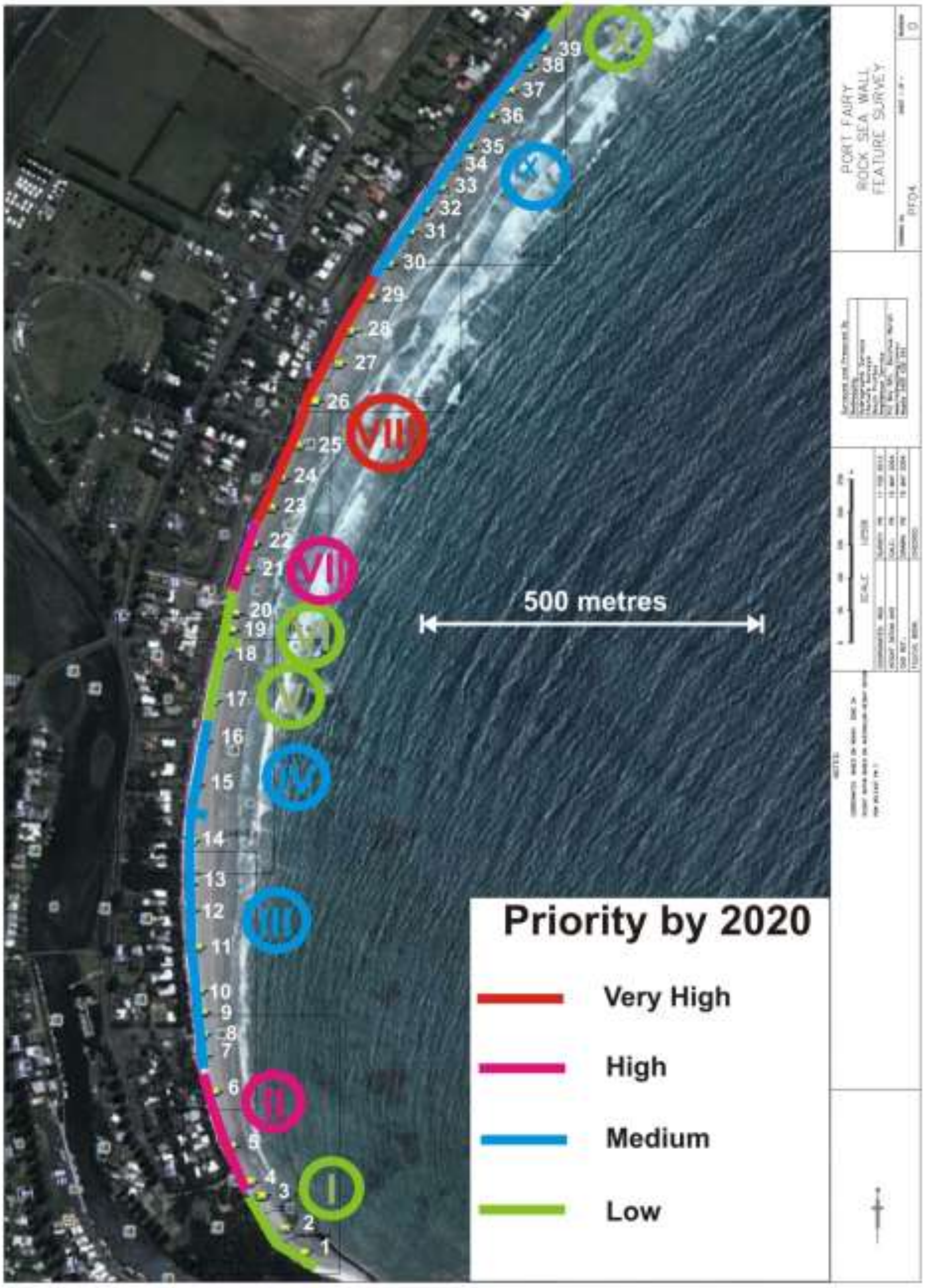

Figure 22 : Seawall Categories and Priorities for Remediation to 2020

5.2 Conceptual Design for Upgrading the Existing Seawall

A Concept Design for upgrading the existing East Beach seawall to an appropriate structural standard has been prepared following consideration of its condition and required structural performance. The Concept Design is based on the detailed engineering design of the recently completed upgrade and extension works at the northern-most end of the seawall. The design standard for that completed work accommodates the 100 year ARI event and addresses future climate change scenarios.

Nevertheless, the structural details offered herein for the remaining 2km length of the East Beach seawall can only be considered conceptual at this stage. The design wave conditions adopted have been interpreted from a previous coastal study in the area (Coastal Engineering Solutions, 2007). Additional investigations for specific wave conditions along with beach response modelling would be required to prepare final engineering design and construction documentation for seawall works.

The fieldwork undertaken for this study, in conjunction with previous investigations (WRL, 2010) regarding the extent of underlying rock, offers sufficient details regarding the characteristics of existing rock seawall. However there needs to be a detailed survey undertaken prior to preparing any detail design of remediation works for each section of the seawall. This would best be undertaken once a commitment/funding has been formally made for remediation of specific sections of seawall.

The Concept Design offers the opportunity to prepare indicative construction cost estimates to guide decision-making when considering appropriate improvements to foreshore protection along East Beach. The estimated costs are discussed in the subsequent Section 6 of this report.

5.2.1 Mitigating Damage by Erosion of the Armour Layer

Various methods for calculating the size of rock armour under wave attack have been proposed by coastal engineers in the past few decades. The decision as to which mathematical technique is the most appropriate has been the subject of much deliberation, however most practitioners are now generally agreed that the formulae originally developed by van der Meer (1988) are the most appropriate. They are based upon an extensive series of physical model tests, which included a wide range of incident wave conditions, nearshore bathymetry, core / underlayer permeabilities, and rock characteristics. This was the design technique applied to the northern upgrade/extension works completed on the East Beach seawall in late-2012.

Consequently the design techniques attributed to van der Meer have been applied in development of the Concept Design for the remainder of the East Beach seawall.

In doing so, the extent of damage that is deemed to be acceptable under the 100 year ARI design criteria has been selected as 5%. This is in keeping with widely accepted practice when designing rock armoured works.

The slope of the armour layer has been selected as 1 vertical to 1.5 horizontal. Previous experience shows that a flatter gradient is more difficult to build without specialised long-reach excavators. Flatter seawall slopes require a greater volume of rock product, and also have significantly greater impact on the visual amenity of the foreshore.

The following average rock sizes are required as the primary outer armouring to accommodate the 100 year ARI design criteria:

- present-day scenario : two layers of 1.5 tonne rocks
- future climate scenario : two layers of 2 tonne rocks

Rather than place armour with an average rock size of 1.5 tonnes and then experience damage if/when future climate change increases the wave loadings on the seawall (which would then require expensive upgrading of the armour), it is prudent to initially place the slightly heavier 2 tonne armour. This is further justified given that the effects of future climate change will be gradual. Hence the future inadequacy of 1.5 tonne armour will not necessarily manifest itself immediately following a particular storm event, but rather as an on-going and increasing need to undertake repairs as incident wave energy gradually increases.

Based on this philosophy of accommodating future storm wave energy, the required armouring of the seawall for conceptual design purposes consists of the following:

- Seawall slope: 1 vertical to 1.5 horizontal
- Primary armour: Two layers of rock, 50% by number greater than 2 tonne (allowable range in size 0.5 tonne to 5 tonne).
- Minimum layer thickness of primary armour: 1650mm
- Underlayer rock: Two layers, average size 150kg (allowable range in size 50kg to 500kg)
- Minimum underlayer thickness: 775mm
- Minimum rock density: 2,650 kg/m3
- Suitability of all rock for application in marine works confirmed by petrographic analyses
- Geotextile on underlying bank slope: Elcomax 600R or an approved equivalent

These required characteristics to mitigate damage and possible structural failure due to erosion of the armour layer are shown in Figure 23.

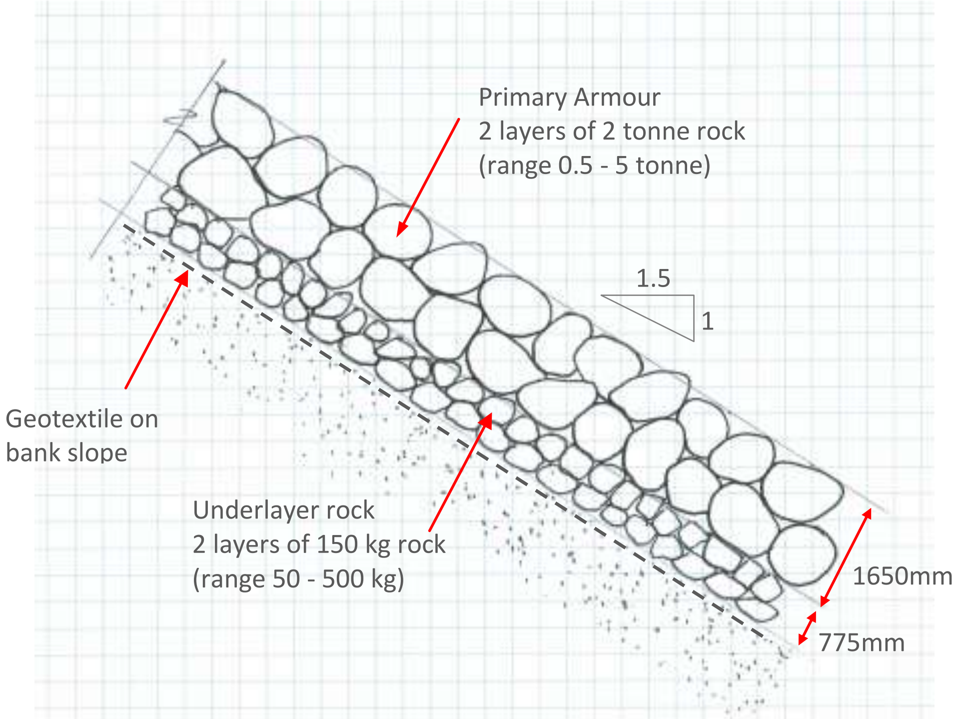

Figure 23 : Required Design for Mitigating Failure by Erosion of the Armour Layer

5.2.2 Mitigating Damage by Undermining

Numerical modelling of beach response was undertaken as part of the detailed coastal engineering design of the recent northern seawall upgrade/extension.

Calculations of profile response under 100 year ARI future storm conditions suggest that the beach in front of the existing seawall is likely to erode down to approximately RL-0.35m AHD.

The toe of the seawall will need to extend below this predicted scour level. Ideally the toe should be located at a depth of at least one rock diameter below this level to ensure adequate foundation of the armour layer during extreme scour conditions. Given the requirement for 2 tonne rocks in the armour layer, this equates to a toe depth of RL-1.25m AHD - which is more than 0.5 metres below the level of the Lowest Astronomical Tide at East Beach.

The philosophy adopted to ensure that future undermining of the toe does not compromise the structural integrity of the seawall is therefore to found the seawall at RL-1.25m AHD.

There are a number of different ways in which to provide scour protection at the toe of a seawall. One method which seems to have a preferred application in European structures is to provide a relatively wide scour blanket that extends out in front of the toe of the seawall.

A disadvantage of this type of protection is that it can be very difficult (and therefore very expensive) to construct. It requires the excavation of a deep wide trench in front of the seawall below groundwater levels - and will therefore be inherently unstable. Experience shows that such excavations collapse and only remain open for very short times. Light-weight sheet piling or trench-shoring techniques can be applied to keep short lengths of toe excavation open for subsequent backfilling with toe armour, however this can be extremely expensive. In reality, the difficulty in constructing a toe scour blanket often results in having to accept a constructed outcome that is actually less than that expected, thereby potentially compromising the integrity of the protection.

Another method of providing protection against undermining of the toe is to stabilise the base of the armour layer without necessarily providing a large degree of protection against scour. This entails placing large rocks at the base of the armour layer to form a buttress foundation for the slope above it.

The challenge of excavating the beach/seabed to accommodate the placement of the large individual rocks remains, however the extent of the excavation is considerably less than that required for a scour blanket. Therefore it can also be open for shorter times to facilitate placement of individual rocks.

Consequently the toe armour arrangement adopted for this Concept Design is to place a row of large rocks at the toe of the armour layer to form a foundation no higher than RL-1.25m AHD upon which the armour layer is then constructed. Each rock in buttress row should be no less than 4 tonnes in weight. This arrangement to mitigate damage and possible structural failure due to undermining of the armour layer is shown conceptually in Figure 24.

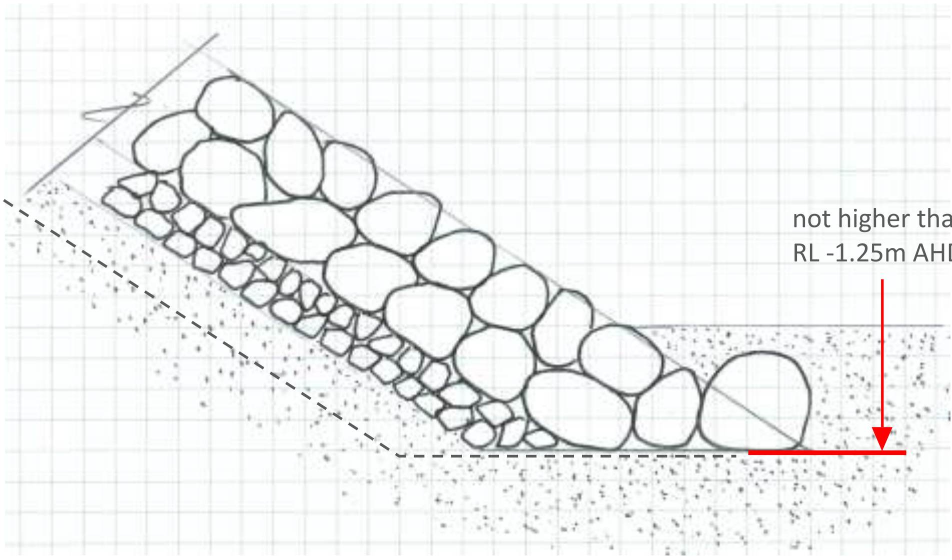

Figure 24 : Required Design for Mitigating Failure by Undermining

5.2.3 Mitigating Damage by Overtopping

Given that the intent of the East Beach seawall is to prevent overtopping waves from eroding the unprotected soil slope behind the seawall, it will be necessary to limit overtopping discharges to a maximum of 0.05 m^3/sec/m. Ideally this is achieved by ensuring the crest of the armoured seawall slope is sufficiently high so as to ensure wave overtopping is less than this upper limit.

When assessing wave overtopping performance of the proposed seawall, the comprehensive calculation techniques outlined in the "*Wave Overtopping of Sea Defences and Related Structures : Assessment Manual*" (EurOtop, 2007) have been applied for conditions associated with the 100 year ARI Design Event at East Beach.

This results in the requirement to have the crest of the seawall no lower than RL+3.25m AHD. Surveys undertaken at the cross sections where the existing seawall was pulled apart and reinstated by a hydraulic excavator during fieldwork for this study identified the varying crest level of the structure. The results of these surveys, along with visual observation of the foreshore, suggest that this high crest level can be achieved when upgrading those sections of the existing seawall north of approximately Cross Section 17.

However the approximately 895m long length of seawall to the south of this location has a foreshore level lower than RL+3.25m AHD. The provision of adequate overtopping protection represents a significant challenge in the design of upgrading works for this southern length of the East Beach seawall. This is because the existing landform; close proximity of private property; established foreshore vegetation; and public infrastructure located immediately behind the existing seawall alignment; all place a considerable constraint on what can be constructed in this area.

Detailed surveys of existing levels, property boundaries, significant vegetation, road alignments, drainage paths and other such infrastructure need to be obtained for the detailed engineering design phase to guide the careful design of crest details to mitigate damage by overtopping in these areas.

Nevertheless the most viable means of providing the necessary overtopping protection along the southern portion of the seawall is likely to maintain the existing underlying landform and either widen the seawall crest and/or slightly realigning the wall so that it is further seaward.

The extent of crest armour placement depends upon the level of the foreshore it is to protect. Where the foreshore is low, the crest armour needs to extend further back from the top of the seawall. Calculations of overtopping and crest armour requirements indicate that at the locations where the wall height is below RL+3.25m AHD, rocks should be placed for an additional distance of approximately 4 metres behind the seawall. This results in an overall crest armour width of approximately 8 metres measured from the top of the seawall front slope.

Crest armour can be buried behind the seawall to improve the visual and recreational amenity of the foreshore. It is a structural element of the seawall which is only utilised during severe overtopping events. Any fill placed to cover the crest armour would need to be a granular or sandy material that could be readily washed out of the voids by overtopping waves - so that the performance of the crest armour in dissipating overtopping flows is not compromised.

The best arrangement of filling over the crest armour would be to have a landscaped environment (similar to that of sand foredunes) appropriately vegetated with dune species. Alternatively the nature of the existing foreshore could be reinstated over the top of the crest armour.

However it is important to appreciate that such “landscaping” over the crest armour would be sacrificial, in that it would be removed by overtopping waves during severe events - nevertheless the crest armour (and hence the seawall itself) would remain structurally sound.

The arrangement to mitigate damage and possible structural failure due to overtopping of the seawall is shown conceptually in Figure 25. It includes the placement of a row of larger rocks at the landward edge of any required crest armour to act as a buttress to maintain its integrity.

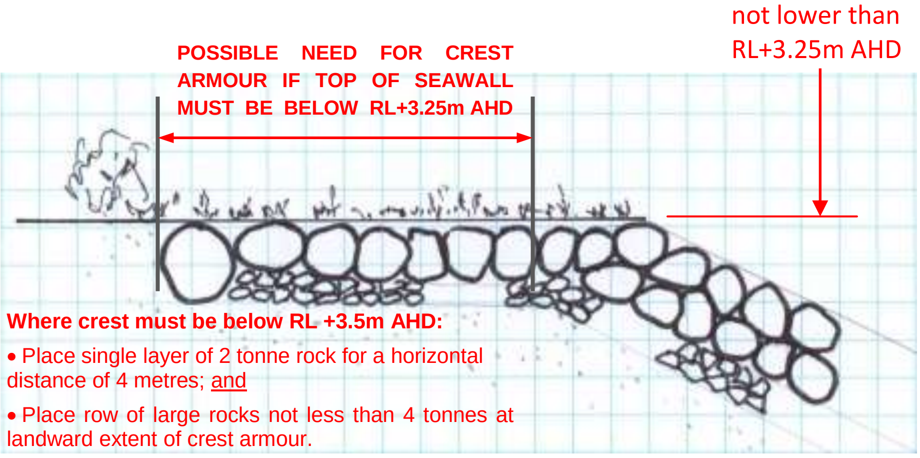

Figure 25 : Required Design for Mitigating Failure by Overtopping

5.3 Recommended Seawall Cross Section

The various measures discussed in the preceding sections for mitigating the damage/failure that a seawall on the East Beach foreshore can experience as a consequence of the 100 year ARI storm event have been incorporated into a recommended cross section for the structure. The resulting concept design is illustrated in Figure 26.

Due to the variable nature of the existing foreshore (both in terms of its landform and use), it is necessary to include a specific detail for those sections where the existing land levels and the top of the seawall are below RL+3.25m AHD. This alternative crest armour arrangement is required to mitigate the effects of green water overtopping during severe storms.

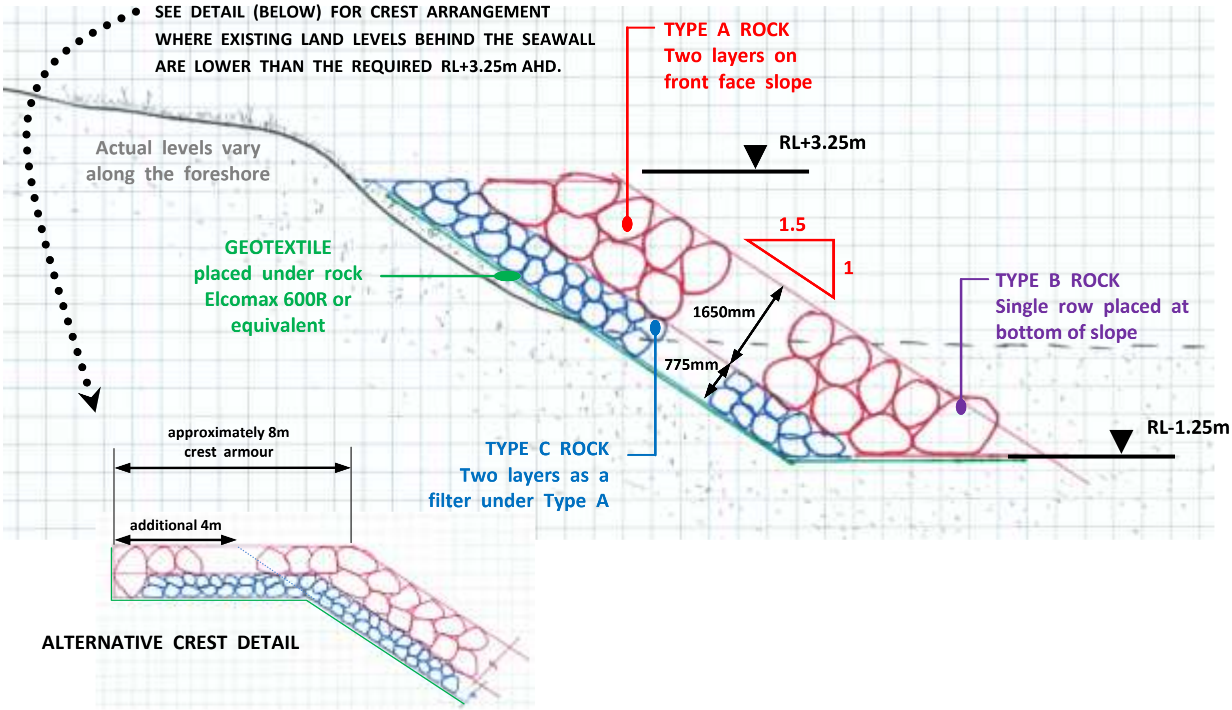

Figure 26 : Conceptual Seawall Design (for 100 year ARI Design Event)

5.4 Construction Methodology

Seawall construction is a specialised construction activity. It should not be viewed as a simple or straight-forward earthworks activity simply requiring the bulk placement of different rock sizes on a slope. On the contrary, it requires large rocks to be individually placed onto the slope to achieve appropriate interlocking; works would be frequently constrained by tides and sea conditions; construction access and work areas on foreshores will be significantly confined; there are quite tight constraints on the standard of rocks that are to be supplied; and the construction activity can be quite disruptive to foreshore amenity and the community's use.

It is expected that the reconstruction works at East Beach would be undertaken by a suitably experienced contractor, and that the evaluation of tenders for the work would acknowledge such experience as a distinct advantage.

Repairs and reconstruction of the seawall will entail removing all of the rock from the existing structure; preparing the underlying bank slope; placing geotextile; placing the rock underlayers and then placing the primary outer armour layer (including toe protection and crest armour).

The stripping of rock from the existing seawall and construction of the design profile would occur on relatively short sections - with construction activity progressing along the length of the seawall. This would reduce the vulnerability of sections of the foreshore that were exposed by the removal of existing armour.

Rocks removed from the existing seawall that comply with the requirements for armour to be placed in the reconstructed seawall could be reused in the new works. Rocks that did not meet the specification would need to be stockpiled for later removal, or disposed of offsite.

The difficulty for both the designer and the constructor is knowing beforehand the quantity of rock in the existing seawall which can be reused. This will have a significant bearing on the cost and (to a lesser extent) the construction timescale.

Whilst estimates of existing rock size and quantities have been undertaken during site investigations for this Study, contracts for the upgrading of the seawall should be prepared such that any benefits obtained by reusing actual rock quantities from the existing seawalls can be accurately measured and incorporated into the final contract fee.

5.5 Potential Effects of Seawall Works on Coastal Processes

The fundamental philosophy for the concept design of the East Beach seawall is to have it located behind the active beach system as close as possible to the alignment of the existing structure.

During ambient sea and weather conditions, waves will dissipate their energy on the sandy beach in front of the structure. However during particularly severe storm conditions, the beach can erode to the extent that waves will wash directly against the rock-armoured seawall. The Concept Design has been prepared to accommodate the wave loadings associated with a 100 year ARI storm.

During severe storms when the seawall is directly exposed to wave attack, the reflected wave energy contributes to the lowering of the beach in front of the structure. Beach sand is moved offshore during the storm, and deposited near the break-point of the incoming waves - thereby forming an nearshore sandbank within the surf zone. Once the storm has passed, subsequent milder wave conditions re-works the sand on the offshore sandbank back onto the beach. The timescale of this natural restoration of the eroded beach is typically much greater than that of the erosion event itself.

Historically East Beach has recovered naturally from significant erosion events, and the proposed upgrading works on the existing seawall alignment will not adversely affect the coastal processes that facilitate this natural recovery.

Staged works (whereby only discrete sections of the seawall are upgraded) will not adversely affect adjacent sections of the existing seawall.

A potentially adverse effect could occur if the new seawall works resulted in a plan alignment that was significantly seaward of the existing seawall. This could result in partial impedance of the longshore transport of sand, thereby depleting the supply of sand to the immediately downdrift section of seawall. This could result in undermining failure of the downdrift section during a severe storm.

It is apparent from the recent site inspections that it may be inevitable there will have to be a new alignment on two sections of the seawall, namely:

- Chainage 150m - 325m. Excavated cross sections 4, 5 and 6 (with an approximate foreshore length of 175 metres). It appears that fence lines on property boundaries are located along the top of the seawall, consequently any new upgrading works may need to be slightly seaward of the existing alignment to avoid being constructed in private property.

- Chainage 1185m - 1545m. Excavated cross sections 23 to 29 (having an approximate foreshore length of 360 metres). The embankment slope above and behind the seawall is very steep; and there is infrastructure located near the top of the embankment. Therefore it is unlikely that any rock in the existing structure could be removed without the risk of the embankment above it collapsing. It may be necessary to undertake upgrading works directly against the existing front slope of the seawall - resulting in a slightly seaward alignment.

The expected realignment in both cases will be minor. Nevertheless the detailed engineering design of upgrading works in these two areas will need to consider measures to fully mitigate any potentially adverse implications to the seawall immediately downdrift. Such measures are not expected to be particularly challenging from a technical viewpoint, and are very likely to be achieved through prudent design and careful construction.

6 ESTIMATED COSTS OF SEAWALL WORKS

The issue of whether all of the seawall reconstruction work is undertaken at once, or discrete lengths are completed under separate construction contracts spanning several years is controlled by funding issues. Given the scale of the physical works necessary to upgrade the approximately 2kms of existing seawall, it is expected that the works will be undertaken in stages.

Therefore when preparing cost estimates for the upgrading works, it has been assumed that each of the identified seawall categories will be upgraded as separate construction activities and contracts.

6.1 Unit Rates

The unit rates presented in Table 6.1 for various work elements have been compiled from recent seawall projects of comparable size to that proposed at East Beach. They are considered appropriate rates for contracted works as of early-2013.

In addition to these construction activities there will be other costs associated with works undertaken on each section of seawall. These would include the detailed engineering design and documentation for the final works; tendering; site supervision; contract administration; and mobilisation / demobilisation by the contractor. These various activities have been estimated for each specific section of seawall upgrade.

Activity	Unit	Rate
Recover & stockpile existing armour	tonne	$3.25
trim slope	sq.m.	$2.20
excavate beach sand for toe	cu.m.	$5.75
supply & place geotextile	sq.m.	$10.50
supply additional Type C armour	tonne	$24.25
supply additional Type B armour	tonne	$20.00
supply additional Type A armour	tonne	$17.50
placeType C armour	tonne	$15.75
place Type B armour	tonne	$26.25
place Type A armour	tonne	$21.50
backfill/ reinstate beach	cu.m.	$1.25

Table 6.1 : Summary of Unit Rates for Construction Activities

6.2 Cost Estimates

6.2.1 Category I - chainage 0m to 150m

Activity	Quantity	Unit	Rate	Amount
GENERAL				
Mobilisation		lump sum	$5,000	$5,000
Prepare site access etc.		lump sum	$5,000	$5,000
Remove access & reinstate foreshore		lump sum	$7,500	$7,500
De-mobilisation		lump sum	$5,000	$5,000
As Constructed Drawings		lump sum	$1,000	$1,000
			subtotal	**$23,500**
PHYSICAL WORKS				
Recover & stockpile existing armour	1,200	tonne	$3.25	$3,900
trim slope	575	sq.m.	$2.20	$1,265
excavate beach sand for toe	2,290	cu.m.	$5.75	$13,168
supply & place geotextile	1,775	sq.m.	$10.50	$18,638
supply additional Type C armour	1,395	tonne	$24.25	$33,829
supply additional Type B armour	685	tonne	$20.00	$13,700
supply additional Type A armour	3,415	tonne	$17.50	$59,763
placeType C armour	1,995	tonne	$15.75	$31,421
place Type B armour	685	tonne	$26.25	$17,981
place Type A armour	4,015	tonne	$21.50	$86,323
backfill/ reinstate beach	2,290	cu.m.	$1.25	$2,863
			subtotal	**$282,849**
MANAGEMENT				
Detailed survey		lump sum		$3,500
Detailed design & documentation		lump sum		$25,000
Project Management		lump sum		$15,000
			subtotal	**$43,500**
			Overall Estimated Cost	$349,849
			ESTIMATED COST *(including GST)*	**$350,000**

6.2.2 Category II - chainage 150m to 325m

Activity	Quantity	Unit	Rate	Amount
GENERAL				
Mobilisation		lump sum	$5,000	$5,000
Prepare site access etc.		lump sum	$5,000	$5,000
Remove access & reinstate foreshore		lump sum	$7,500	$7,500
De-mobilisation		lump sum	$5,000	$5,000
As Constructed Drawings		lump sum	$1,000	$1,250
			subtotal	**$23,750**
PHYSICAL WORKS				
Recover & stockpile existing armour	800	tonne	$3.25	$2,600
trim slope	605	sq.m.	$2.20	$1,331
excavate beach sand for toe	2,715	cu.m.	$5.75	$15,611
supply & place geotextile	2,025	sq.m.	$10.50	$21,263
supply additional Type C armour	1,460	tonne	$24.25	$35,405
supply additional Type B armour	810	tonne	$20.00	$16,200
supply additional Type A armour	4,535	tonne	$17.50	$79,363
placeType C armour	2,025	tonne	$15.75	$31,894
place Type B armour	810	tonne	$26.25	$21,263
place Type A armour	4,535	tonne	$21.50	$97,503
backfill/ reinstate beach	2,715	cu.m.	$1.25	$3,394
			subtotal	**$325,825**
MANAGEMENT				
Detailed survey		lump sum		$3,500
Detailed design & documentation		lump sum		$25,000
Project Management		lump sum		$16,500
			subtotal	**$45,000**
			Overall Estimated Cost	$394,575
			ESTIMATED COST *(including GST)*	**$395,000**

6.2.3 Category III - chainage 325m to 795m

Activity	Quantity	Unit	Rate	Amount
GENERAL				
Mobilisation		lump sum	$10,000	$10,000
Prepare site access etc.		lump sum	$7,500	$7,500
Remove access & reinstate foreshore		lump sum	$7,500	$7,500
De-mobilisation		lump sum	$7,500	$7,500
As Constructed Drawings		lump sum	$1,500	$1,500
			subtotal	**$34,000**
PHYSICAL WORKS				
Recover & stockpile existing armour	3,000	tonne	$3.25	$9,750
trim slope	1,450	sq.m.	$2.20	$3,190
excavate beach sand for toe	6,015	cu.m.	$5.75	$34,586
supply & place geotextile	4,590	sq.m.	$10.50	$48,195
supply additional Type C armour	4,105	tonne	$24.25	$99,546
supply additional Type B armour	1,795	tonne	$20.00	$35,900
supply additional Type A armour	8,410	tonne	$17.50	$147,175
placeType C armour	5,155	tonne	$15.75	$81,191
place Type B armour	1,795	tonne	$26.25	$47,119
place Type A armour	10,360	tonne	$21.50	$222,740
backfill/ reinstate beach	6,015	cu.m.	$1.25	$7,519
			subtotal	**$736,911**
MANAGEMENT				
Detailed survey		lump sum		$7,500
Detailed design & documentation		lump sum		$32,500
Project Management		lump sum		$37,500
			subtotal	**$77,500**
			Overall Estimated Cost	$848,411
			ESTIMATED COST *(including GST)*	**$848,000**

6.2.4 Category IV - chainage 795m to 895m

Activity	Quantity	Unit	Rate	Amount
GENERAL				
Mobilisation		lump sum	$5,000	$5,000
Prepare site access etc.		lump sum	$5,000	$5,000
Remove access & reinstate foreshore		lump sum	$5,000	$5,000
De-mobilisation		lump sum	$5,000	$5,000
As Constructed Drawings		lump sum	$1,250	$1,250
			subtotal	**$21,250**
PHYSICAL WORKS				
Recover & stockpile existing armour	700	tonne	$3.25	$2,275
trim slope	580	sq.m.	$2.20	$1,276
excavate beach sand for toe	2,435	cu.m.	$5.75	$14,001
supply & place geotextile	1,850	sq.m.	$10.50	$19,425
supply additional Type C armour	1,725	tonne	$24.25	$41,831
supply additional Type B armour	725	tonne	$20.00	$14,500
supply additional Type A armour	3,820	tonne	$17.50	$66,850
placeType C armour	2,075	tonne	$15.75	$32,681
place Type B armour	725	tonne	$26.25	$19,031
place Type A armour	4,170	tonne	$21.50	$89,655
backfill/ reinstate beach	2,435	cu.m.	$1.25	$3,044
			subtotal	**$304,570**
MANAGEMENT				
Detailed survey		lump sum		$3,500
Detailed design & documentation		lump sum		$17,500
Project Management		lump sum		$15,500
			subtotal	**$36,500**
			Overall Estimated Cost	$362,320
			ESTIMATED COST *(including GST)*	**$362,000**

6.2.5 Category V - chainage 895m to 1015m

Activity	Quantity	Unit	Rate	Amount
GENERAL				
Mobilisation		lump sum	$5,000	$5,000
Prepare site access etc.		lump sum	$5,000	$5,000
Remove access & reinstate foreshore		lump sum	$5,000	$5,000
De-mobilisation		lump sum	$5,000	$5,000
As Constructed Drawings		lump sum	$1,250	$1,250
			subtotal	**$21,250**
PHYSICAL WORKS				
Recover & stockpile existing armour	1,500	tonne	$3.25	$4,875
trim slope	605	sq.m.	$2.20	$1,331
excavate beach sand for toe	2,085	cu.m.	$5.75	$11,989
supply & place geotextile	1,975	sq.m.	$10.50	$20,738
supply additional Type C armour	730	tonne	$24.25	$17,703
supply additional Type B armour	310	tonne	$20.00	$6,200
supply additional Type A armour	2,405	tonne	$17.50	$42,088
placeType C armour	1,480	tonne	$15.75	$23,310
place Type B armour	310	tonne	$26.25	$8,138
place Type A armour	3,153	tonne	$21.50	$67,790
backfill/ reinstate beach	2,085	cu.m.	$1.25	$2,606
			subtotal	**$206,766**
MANAGEMENT				
Detailed survey		lump sum		$3,500
Detailed design & documentation		lump sum		$17,500
Project Management		lump sum		$10,500
			subtotal	**$31,500**
			Overall Estimated Cost	$259,516
			ESTIMATED COST *(including GST)*	**$260,000**

6.2.6 Category VI - chainage 1015m to 1065m

Activity	Quantity	Unit	Rate	Amount
GENERAL				
Mobilisation		lump sum	$5,000	$5,000
Prepare site access etc.		lump sum	$5,000	$5,000
Remove access & reinstate foreshore		lump sum	$5,000	$5,000
De-mobilisation		lump sum	$5,000	$5,000
As Constructed Drawings		lump sum	$1,250	$1,250
			subtotal	**$21,250**
PHYSICAL WORKS				
Recover & stockpile existing armour	1,000	tonne	$3.25	$3,250
trim slope	270	sq.m.	$2.20	$594
excavate beach sand for toe	935	cu.m.	$5.75	$5,376
supply & place geotextile	885	sq.m.	$10.50	$9,293
supply additional Type C armour	160	tonne	$24.25	$3,880
supply additional Type B armour	140	tonne	$20.00	$2,800
supply additional Type A armour	910	tonne	$17.50	$15,925
placeType C armour	660	tonne	$15.75	$10,395
place Type B armour	140	tonne	$26.25	$3,675
place Type A armour	1,410	tonne	$21.50	$30,315
backfill/ reinstate beach	935	cu.m.	$1.25	$1,169
			subtotal	**$86,672**
MANAGEMENT				
Detailed survey		lump sum		$3,500
Detailed design & documentation		lump sum		$17,500
Project Management		lump sum		$5,000
			subtotal	**$26,000**
			Overall Estimated Cost	$133,922
			ESTIMATED COST *(including GST)*	**$134,000**

6.2.7 Category VII - chainage 1065m to 1185m

Activity	Quantity	Unit	Rate	Amount
GENERAL				
Mobilisation		lump sum	$5,000	$5,000
Prepare site access etc.		lump sum	$5,000	$5,000
Remove access & reinstate foreshore		lump sum	$5,000	$5,000
De-mobilisation		lump sum	$5,000	$5,000
As Constructed Drawings		lump sum	$1,250	$1,250
			subtotal	**$21,250**
PHYSICAL WORKS				
Recover & stockpile existing armour	1,300	tonne	$3.25	$4,225
trim slope	515	sq.m.	$2.20	$1,133
excavate beach sand for toe	1,775	cu.m.	$5.75	$10,206
supply & place geotextile	1,680	sq.m.	$10.50	$17,640
supply additional Type C armour	610	tonne	$24.25	$14,793
supply additional Type B armour	265	tonne	$20.00	$5,300
supply additional Type A armour	2,032	tonne	$17.50	$35,560
placeType C armour	1,260	tonne	$15.75	$19,845
place Type B armour	265	tonne	$26.25	$6,956
place Type A armour	2,680	tonne	$21.50	$57,620
backfill/ reinstate beach	1,775	cu.m.	$1.25	$2,219
			subtotal	**$175,497**
MANAGEMENT				
Detailed survey		lump sum		$3,500
Detailed design & documentation		lump sum		$17,500
Project Management		lump sum		$9,000
			subtotal	**$30,000**
			Overall Estimated Cost	$226,747
			ESTIMATED COST *(including GST)*	**$227,000**

6.2.8 Category VIII - chainage 1185m to 1545m

Activity	Quantity	Unit	Rate	Amount
GENERAL				
Mobilisation		lump sum	$10,000	$10,000
Prepare site access etc.		lump sum	$7,500	$7,500
Remove access & reinstate foreshore		lump sum	$8,500	$8,500
De-mobilisation		lump sum	$7,500	$7,500
As Constructed Drawings		lump sum	$1,500	$1,500
			subtotal	**$35,000**
PHYSICAL WORKS				
Recover & stockpile existing armour	*none*	tonne	$3.25	
trim slope	1,890	sq.m.	$2.20	$4,158
excavate beach sand for toe	6,510	cu.m.	$5.75	$37,433
supply & place geotextile	6,165	sq.m.	$10.50	$64,733
supply additional Type C armour	4,620	tonne	$24.25	$112,035
supply additional Type B armour	970	tonne	$20.00	$19,400
supply additional Type A armour	9,840	tonne	$17.50	$172,200
placeType C armour	4,620	tonne	$15.75	$72,765
place Type B armour	970	tonne	$26.25	$25,463
place Type A armour	9,840	tonne	$21.50	$211,560
backfill/ reinstate beach	6,510	cu.m.	$1.25	$8,138
			subtotal	**$727,883**
MANAGEMENT				
Detailed survey		lump sum		$8,000
Detailed design & documentation		lump sum		$32,500
Project Management		lump sum		$36,500
			subtotal	**$77,000**
			Overall Estimated Cost	$839,883
			ESTIMATED COST *(including GST)*	**$840,000**

6.2.9 Category IX - chainage 1545m to 2015m

Activity	Quantity	Unit	Rate	Amount
GENERAL				
Mobilisation		lump sum	$10,000	$10,000
Prepare site access etc.		lump sum	$7,500	$7,500
Remove access & reinstate foreshore		lump sum	$7,500	$7,500
De-mobilisation		lump sum	$7,500	$7,500
As Constructed Drawings		lump sum	$1,500	$1,500
			subtotal	**$34,000**
PHYSICAL WORKS				
Recover & stockpile existing armour	10,840	tonne	$3.25	$35,230
trim slope	2,080	sq.m.	$2.20	$4,576
excavate beach sand for toe	7,170	cu.m.	$5.75	$41,228
supply & place geotextile	6,795	sq.m.	$10.50	$71,348
supply additional Type C armour	2,850	tonne	$24.25	$69,113
supply additional Type B armour	1,070	tonne	$20.00	$21,400
supply additional Type A armour	9,080	tonne	$17.50	$158,900
placeType C armour	5,090	tonne	$15.75	$80,168
place Type B armour	1,070	tonne	$26.25	$28,088
place Type A armour	10,840	tonne	$21.50	$233,060
backfill/ reinstate beach	7,170	cu.m.	$1.25	$8,963
			subtotal	**$752,071**
MANAGEMENT				
Detailed survey		lump sum		$7,500
Detailed design & documentation		lump sum		$32,500
Project Management		lump sum		$38,000
			subtotal	**$78,000**
			Overall Estimated Cost	$864,071
			ESTIMATED COST *(including GST)*	**$864,000**

6.2.10 Category X - chainage 2015m to 2045m

Activity	Quantity	Unit	Rate	Amount
GENERAL				
Mobilisation		lump sum	$5,000	$5,000
Prepare site access etc.		lump sum	$7,500	$7,500
Remove access & reinstate foreshore		lump sum	$5,000	$5,000
De-mobilisation		lump sum	$5,000	$5,000
As Constructed Drawings		lump sum	$1,250	$1,250
			subtotal	**$23,750**
PHYSICAL WORKS				
Recover & stockpile existing armour	770	tonne	$3.25	$2,503
trim slope	175	sq.m.	$2.20	$385
excavate beach sand for toe	575	cu.m.	$5.75	$3,306
supply & place geotextile	445	sq.m.	$10.50	$4,673
supply additional Type C armour	90	tonne	$24.25	$2,183
supply additional Type B armour	70	tonne	$20.00	$1,400
supply additional Type A armour	150	tonne	$17.50	$2,625
placeType C armour	330	tonne	$15.75	$5,198
place Type B armour	70	tonne	$26.25	$1,838
place Type A armour	700	tonne	$21.50	$15,050
backfill/ reinstate beach	575	cu.m.	$1.25	$719
			subtotal	**$39,878**
MANAGEMENT				
Detailed survey		lump sum		$3,500
Detailed design & documentation		lump sum		$17,500
Project Management		lump sum		$5,000
			subtotal	**$26,000**
			Overall Estimated Cost	$89,628
			ESTIMATED COST *(including GST)*	**$90,000**

6.3 Summary of Cost Estimates

Table 6.2 provides a summary of the overall costs for the upgrading works. Refer to Figure 22 for the locality of the seawall categories. The sections of seawall have been ranked in descending order of priority (as discussed in Section 5.1). However it is acknowledged that Council will consider the particular order that the works within the same priority classification be undertaken.

Seawall Category	Excavated Cross Sections	Length of Category (metres)	Chainage on wall (metres)	Remediation priority by 2020	Estimated Cost
VIII	23 to 29	360m	1185-1545m	VERY HIGH	$840,000
II	4 to 6	175m	150-325m	HIGH	$395,000
VII	21 and 22	120m	1065-1185m	HIGH	$227,000
III	7 to 14	470m	325-795m	MEDIUM	$848,000
IV	15 and 16	100m	795-895m	MEDIUM	$362,000
IX	30 to 39	470m	1545-2015m	MEDIUM	$864,000
I	1 to 3	150m	0-150m	LOW	$350,000
V	17 and 18	120m	895-1015m	LOW	$260,000
VI	19 and 20	50m	1015-1065m	LOW	$134,000
X	steps to new wall	30m	2015-2045m	LOW	$90,000
ENTIRE LENGTH		2045m	0-2045m		$4,370,000

Table 6.2 : Summary of Cost Estimates and Recommended Implementation Priorities

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APPENDIX A

SITE LOG OF SEAWALL INVESTIGATIONS