WATTLE RANGE COUNCIL

Rivoli Bay Study

301015-03541 – 001

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WATTLE RANGE COUNCIL
RIVOLI BAY STUDY

SYNOPSIS

This study reviews the current management plans, assesses the success and failure of existing stabilisation measures along the beach, compiles and examines all the latest information with respect to technical surveys of the beach compartments at Rivoli Bay and details short, medium and long term strategies to address the erosion threat at Beachport Jetty and the foreshore north of the Lake Frome outlet.

Disclaimer

PROJECT 301015-03541 - RIVOLI BAY STUDY

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EXECUTIVE SUMMARY

Rivoli Bay is located along the south-east coast of South Australia, approximately 400 km south-east of Adelaide. The township of Beachport is located on the northern end of Rivoli Bay, and the township of Southend is located at the southern end. The saline Lake George is located to the north and west of Beachport, and has been connected artificially to the sea since the early 20th century via a constructed channel.

At Beachport, groynes have been constructed to stabilise the shoreline, which had been suffering from erosion in places. Existing groynes on the western side of Rivoli Bay trap sediment on their southern side. However, it is evident from recent aerial photography that the groyne compartments cannot contain all the sand and that the groynes are only partially effective in stabilising the coastline in this area. Notwithstanding these works, some beachfront areas have continued to suffer erosion. The groyne structures have failed to compartmentalise and stabilise the shoreline, being too short and of such a design as to cause sand to be lost into the entrance channel at Lake George. In general, the scale of the sand transport processes is very much larger than the scale of the groyne field designed to stabilise the system.

At the eastern end of the Beachport foreshore, an engineered channel has been constructed, which has created an artificial connection between Lake George and the sea. Through this channel, sand is being drawn by tidal currents into Lake George, creating a sand sink for sand from the beach to the west, and causing siltation and a change in the ecology of Lake George.

At Southend on the southern side of Rivoli Bay, erosion has been experienced also and there has been an attempt at shoreline stabilisation with groynes. An engineered drain has been cut to the sea from Lake Frome, with two training walls at the entrance to the Lake interrupting the south-to-north sediment transport along this foreshore. The most severe erosion at Southend has been experienced downdrift of the Lake Frome entrance training walls, and there is evidence of some sand transport upstream into the channel (although to a lesser extent than what has been seen at Lake George).

Wattle Range Council has engaged WorleyParsons to review the current management plans, assess the success and failure of existing stabilisation measures along the beach, compile and examine all the latest information with respect to technical surveys of the beach compartments at Rivoli Bay and prepare a Management Plan detailing short, medium and long term strategies to address the erosion threat, especially in high risk areas near Beachport Jetty and the foreshore north of the Lake Frome outlet. This study assesses the effectiveness of the existing groynes on Rivoli Bay, the impacts of channel works to Lake George and Lake Frome, the impact of rising sea levels, the need for additional works for the long term protection of the Rivoli Bay Beaches and controls on development on the beachfront.

Community Consultation

Stakeholder and community consultation was carried out in Beachport and Southend in October 2014 and again during March-April 2015, with the draft report presented and placed on public exhibition.

The stakeholder consultation included:

- One-on-one meetings with key stakeholders at Beachport in October 2014;
- Public drop-in information centre attended by Council and WorleyParsons at Beachport and Southend in March 2015
- Stakeholder meetings with Council, Department of Planning, Transportr and Infrastructure (DPTI), Department of Environment, Water and Natural Resources South-East (DEWNR SE) and South Eastern Water Conservation and Drainage Board (SEWCDB) in March 2015;
- Public presentations on the Rivoli Bay and related Lake George studies in Beachport in March 2015;
- Hard copies of the report available at the Beachport Visitors Information Centre and on Council website during duration of the exhibition period which covered March 2015;
- Project information posters on display at Beachport Visitors Information Centre during the exhibition period.

A summary of comments received during the exhibition period included:

- Comments from residents at Southend relating to the direction of net sediment transport at Southend around Cape Buffon, suggesting that large quantities of sand exit the bay around Cape Buffon during the winter storms.
- Comments from the community relating to the management of Southend, in particular opposing the removal of the outlet groynes at Lake Frome due to the risk of erosion on the beach to the west, and comments about the effectiveness of mechanical placement of sand from the western side of the outlet to the eastern side, this having been tried previously and resulting in a loss of this sand through offshore sediment transport.
- Suggestions relating to management of the area immediately surrounding the boat ramp at Beachport to reduce sand ingress into the boat launching area.
- In principle support from key government stakeholders including DPTI, Coastal Protection Board (CPB) and Council for construction of additional Groyne 8A as well as a rock revetment in front of the jetty at Beachport, repair/maintenance of existing groynes.
- Lengthening of groynes at Lake George outlet supported in principle by Lake George Management Committee with the proviso from CPB that sand bypassing be implemented to prevent down-drift beach erosion.

- Retreat of critical infrastructure and application of development restrictions in coastal erosion hazard areas was preferred as a strategy to piling of new development found to be affected by coastal erosion hazard.

Coastal Processes

Rivoli Bay formed as a result of a breach in the Robe Range as sea levels rose by up to 140 m following the most recent glacial maximum, and comprises a beach ridge plain and relatively stable foredune. Following the stabilisation of sea levels to around the present level about 6,000 years before present, sand gradually infilled the Bay and this led to the formation of the beach ridge plain feature seen at the central portion of the Bay today.

Littoral drift (movement of sediment along the shoreline) is active along the Beachport and Southend foreshores due to the predominant wave direction in these zones.

Detailed coastal process models have been developed along the Beachport and Southend foreshores, which describe the dominant wave direction and pathways for sediment movement. These models have been developed on the basis of mathematical wave transformation modelling, data collection, analysis of historical survey and analysis of historical aerial photography. These models provide the baseline understanding of the coastal processes needed to develop a Management Plan for the Bay.

Other coastal processes active within the Bay include:

- short-term coastal erosion including that resulting from severe storms, the behaviour of estuary entrances and slope instability;
- long term coastline recession which can result from more sand leaving the coastal system than arriving, wind-blown sand moving out of the system, climate change and beach rotation; and
- oceanic inundation of low lying areas.

In Rivoli Bay, the shoreline is generally not suffering from long term recession, except in areas which have been modified by human activity. The formation of the beach ridge plain in the centre of the Bay indicates that sand has been building up for thousands of years under natural conditions.

Areas that have suffered long term recession include those areas downdrift of the groynes at the outlet to Lake Frome, the areas in front of the Beachport jetty which have suffered due to the impact of reflections from the vertical seawall, and areas where the bathymetric profile has deepened due to loss of seagrasses. There has also been a loss of sand into the channels connecting Lake George and Lake Frome which continues today.

Climate Change

Climate change has the potential to increase the design wave heights and design water levels at Beachport and Southend, as well as increase wave overtopping levels at Beach Road in the future (due mainly to sea level rise and increased design wave heights).

Wave inundation of the Beachport foreshore at Beach Road is a particular challenge, as the foreshore levels are low compared with the extreme water levels that can occur during storms in Rivoli Bay. Sea level rise due to climate change of up to 1.0 m by 2100 will exacerbate this problem in the future. This wave inundation can lead to damage to infrastructure and impacts on public safety. Engineering intervention will eventually be required to reduce the risk to infrastructure and public safety as the risk will increase in the future due to sea level rise.

The historical human intervention in the coastal processes would be expected to exacerbate the impacts of climate change into the future. Sea level rise would reduce the onshore sediment transport rate compared with that experienced at present, which may reduce the ongoing rate of accretion within the Bay. Without intervention, this sand would continue to be lost from the system by being drawn into Lake George via the constructed channel. The reduced sand supply may lead to exacerbated impacts in the areas currently affected by shoreline recession (i.e. downdrift of the groynes at Lake Frome and at localised areas downdrift of the groynes at Beachport) due to reduced sediment supply to those areas.

Assessment of Coastal Structures

An engineering assessment of each of the groynes and seawalls at Beachport and Southend has been described. The assessment describes:

- the features of the coastal structures as seen during the site visit, including slope, armour materials, size and type, condition including stability and structural integrity of each structure;
- the visible impact of each structure on the surrounding beach and on beach amenity, as gleaned from the site reconnaissance; and
- visual observations relating to the coastal processes within the embayment and their interaction with the erosion protection structures.

The effectiveness of the erosion protection structures against storm events of varying magnitude has been assessed quantitatively with the aid of numerical modelling.

In general, the groynes and revetments at Beachport and Southend are in poor condition and do meet contemporary engineering standards for design and construction. There is considerable damage to the groynes caused by wave action, with dislodged and slumped primary armour layers and erosion of the clay cores of several of these groynes.

The groynes at Beachport, while in poor condition, were generally found to be effective in stabilising the shoreline. The majority of the groynes were bypassing sand continuing to be supplied to the area around Glen Point. Localised erosion impacts were evident downdrift of some of the groynes impacting the beach dunes, timber walkways and, in some areas, threatening to outflank the groynes.

The main timber seawall at Beachport is sometimes subject to severe wave overtopping onto Beach Road, and wave reflections from this seawall have prevented the formation of a usable beach in front of the seawall in the vicinity of the jetty. This wave overtopping caused severe damage to the beachfront promenade during the site inspection on 24 June 2014, with undermining of the concrete pathway.

Sand is bypassing the groynes at the outlet to Lake George, with considerable quantities of sand being carried into the Lake George channel by wave action and tidal currents.

At Southend, the groynes at the outlet to Lake Frome were found to be in particularly poor condition. These groynes have suffered from erosion, loss of armour with miscellaneous rubble used to repair the groynes in places. Severe erosion has occurred in the dunes on the northern side of the outlet, threatening to undermine development. The three groynes north of the Lake Frome outlet have not been effective in stabilising the dune, with little buildup of sand on the south sides of these groynes, indicating that littoral drift may have been rapidly removed because the groynes are too short to trap sand effectively. It is considered that the groynes at the outlet to Lake Frome are considerably reducing the supply of sand to the section of foreshore north of the lake outlet, as sand is not able to bypass the lake outlet.

While the rock revetment adjacent to the Southend jetty was in good condition, severe wave overtopping was observed into the carpark adjacent to the jetty and boat ramp at Southend during the site inspection.

The effectiveness of the layout of the groynes at Beachport and Southend was assessed. The assessment considered the groyne spacing, layout and lengths. It was found that the groynes may not be long enough to compartmentalise the beach effectively. The engineering rule of thumb is that the spacing between groynes should equal two to three times the groyne length. In the vicinity of the Beachport jetty, the ratio between groyne spacing and groyne length is around 7. In this location, the groynes are either too short or their spacing is too large, or both, suggesting that an additional groyne would be of benefit.

Preliminary Coastal Management Scheme

A range of preliminary management options for the foreshores at Beachport and Southend have been canvassed to deal with the key coastal management issues.

The various options for coastal management include short term, medium term and longer term actions. Broadly these options include:

- Extension of the groynes at the channel entrance of Lake George - this would reduce the loss of sand into the Lake outlet and, therefore, reduce the required frequency of dredging of the channel. It would also allow greater ingress of seawater into the Lake thus improving the ability of the Lake to meet its water level target range.
- The layout of the groynes can be improved, particularly in the vicinity of the Beachport jetty, by lengthening the groynes or providing additional groynes in some areas. In particular, provision

of an additional groyne just north of the jetty or extension of the existing groyne north of the jetty would encourage additional sand build-up around the jetty area.

- Provision of a rock revetment in front of the vertical timber seawall near the jetty - this would reduce the wave reflections from the existing seawall, reduce wave runup onto Beach Road and encourage the buildup of sand in this area;
- Review of existing management plan for sand management along Beachport and Southend foreshore to identify triggers for movement of sand between various beach compartments;
- Repair of existing rock groynes where these are inadequate and repair of foreshore areas on the downdrift side of these groynes through ongoing sand management, provision of formalised access, repair of damaged beach accessways and improved dune management techniques;
- Retreat of critical infrastructure landward to prevent damage by coastal processes (e.g. move cabins at Southend landward, mechanically collapse the steep dune escarpment and re-vegetate dune – it is noted that this work was carried out in late 2014);
- Beach nourishment – placement of sand onto the beach in areas experiencing foreshore recession (e.g. the Beachport foreshore near the jetty and the area east of the outlet to Lake Frome) to increase the beach width and reduce wave energy reaching the back of the beach
- Repair of and shortening of the groynes at the outlet to Lake Frome at Southend
- Lengthening of the groynes east of the Lake outlet at Southend
- Replacing damaged or inadequate seawall structures at Beachport and Southend

Each of the above management options will incur a capital and maintenance cost. Some of the identified management options would need to be subject to detailed design and environmental assessment. Development controls on land use and special building requirements (such as coastal setbacks) may be required in areas that are subject to direct coastal hazards of inundation, erosion and recession.

1 INTRODUCTION

1.1 Background

Rivoli Bay is located along the south-east coast of South Australia, approximately 400 km south-east of Adelaide. The township of Beachport is located on the northern end of Rivoli Bay, and the township of Southend is located at the southern end. The saline Lake George is located to the north and west of Beachport, and has been connected artificially to the sea since the early 20^{th} century via a constructed channel.

Rivoli Bay is a geologically young feature, having been formed during the Holocene period when sea levels rose and flooded the Robe range (Short and Hesp, 1980). Parts of the Robe range are close to the sea surface today and are influencing wave patterns in the nearshore, thus influencing sediment transport patterns within the Bay. Over the Holocene epoch since the most recent sea-level still stand (approximately 6,500 years ago), beach ridge plains have formed along the central area of Rivoli Bay and are evidence of shoreline progradation over the last 5000 – 7000 years, with swell waves transporting sand onshore from the bed of the Bay.

At Beachport, groynes have been constructed to stabilise the shoreline, which had been suffering from erosion in places.

As shown in Figure 1, wave patterns at Rivoli Bay are complex and are influenced by both wave refraction and diffraction. The predominant wave climate creates a south-to-north sediment transport along the western foreshore of Rivoli Bay. Existing groynes on the western side of Rivoli Bay trap sediment on their southern side. However, it is evident from recent aerial photography that the groyne compartments are fully bypassing and that the groynes are only partially effective in stabilising the coastline in this area. Notwithstanding these works, some beachfront areas have continued to suffer erosion. The groyne structures have failed to compartmentalise and stabilise the shoreline, being too short and of such a design as to cause sand to be directed offshore and lost in deeper waters or into the entrance channel at Lake George. In general, the scale of the sand transport processes is very much larger than the scale of the groyne field designed to stabilise the system.

At the eastern end of the Beachport foreshore, an engineered channel has been constructed, which has created an artificial connection between Lake George and the sea. Through this channel, sand is being drawn by tidal currents into Lake George, creating a sand sink for sand from the beach to the west, and causing siltation and a change in the ecology of Lake George.

At Southend on the southern side of Rivoli Bay, erosion has been experienced also and there has been an attempt at shoreline stabilisation with groynes. An engineered drain has been cut to the sea from Lake Frome, with two training walls at the entrance to the Lake interrupting the south-to-north sediment transport along this foreshore. The most severe erosion at Southend has been experienced downdrift of the Lake Frome entrance training walls, and there is evidence of some sand transport upstream into the channel (although to a lesser extent than what has been seen at Lake George).

Wattle Range Council has engaged WorleyParsons to review the current management plans, assess the success and failure of existing stabilisation measures along the beach, compile and examine all the latest information with respect to technical surveys of the beach compartments at Rivoli Bay and prepare a Management Plan detailing short, medium and long term strategies to address the erosion threat, especially in high risk areas near Beachport Jetty and the foreshore north of the Lake Frome outlet. This study assesses the effectiveness of the existing groynes on Rivoli Bay, the impacts of channel works to Lake George and Lake Frome, the impact of rising sea levels, the need for additional works for the long term protection of the Rivoli Bay Beaches and controls on development on the beachfront.

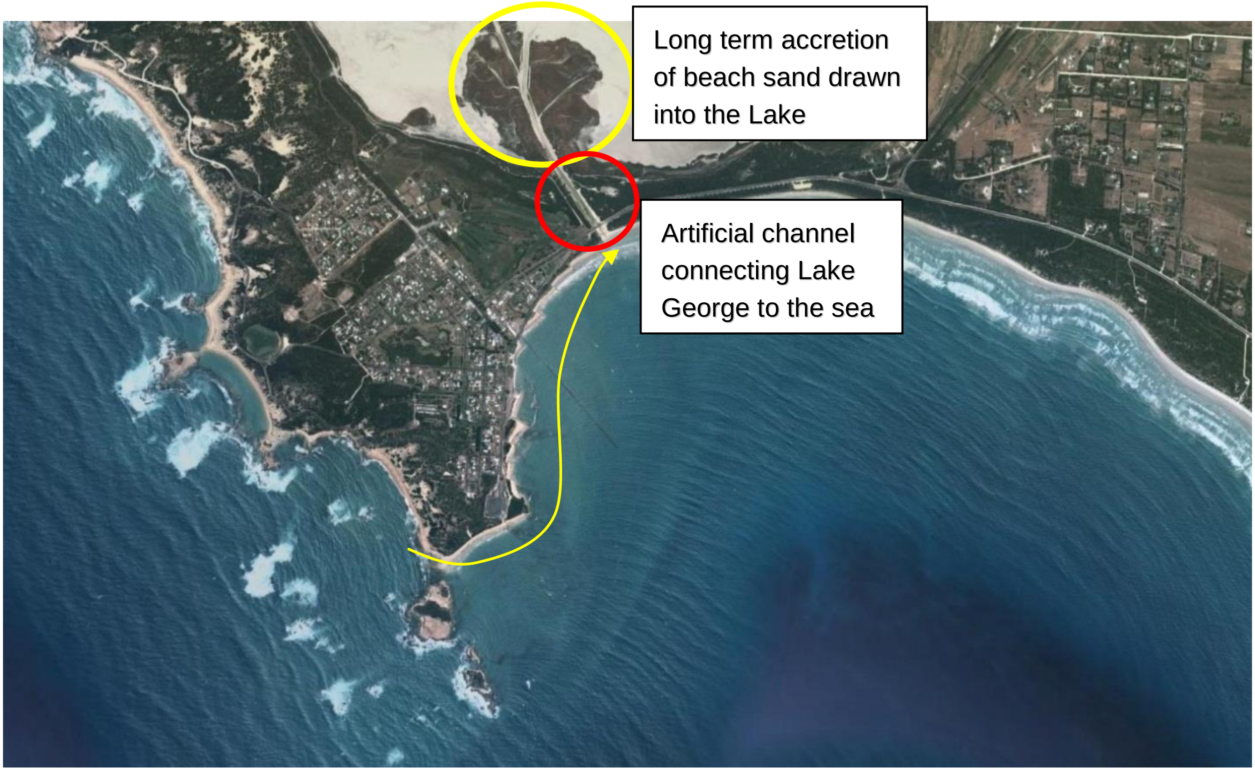

Figure 1 – Aerial photograph of western side of Rivoli Bay indicating sediment transport direction and complex wave patterns (Google Earth).

1.2 Stakeholder and Community Consultation

Stakeholder and community consultation was carried out in Beachport and Southend in October 2014 and again during March-April 2015, with the draft report presented and placed on public exhibition.

The stakeholder consultation included:

- One-on-one meetings with key stakeholders at Beachport in October 2014;

- Public drop-in information centre attended by Council and WorleyParsons at Beachport and Southend in March 2015
- Stakeholder meetings with Council, Department of Planning, Transportr and Infrastructure (DPTI), Department of Environment, Water and Natural Resources South-East (DEWNR SE) and South Eastern Water Conservation and Drainage Board (SEWCDB) in March 2015;
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- Lengthening of groynes at Lake George outlet supported in principle by Lake George Management Committee with the proviso from CPB that sand bypassing be implemented to prevent down-drift beach erosion.
- Retreat of critical infrastructure and application of development restrictions in coastal erosion hazard areas was preferred as a strategy to piling of new development found to be affected by coastal erosion hazard.

2 COASTAL PROCESSES

2.1 Introduction

An understanding of the coastal processes at Rivoli Bay has been developed from the analysis of existing data, data collected specifically for this investigation and site observations complemented with the known understanding of local coastal processes as described in previous studies and as modelled for this investigation.

2.2 Regional coastal processes

Rivoli Bay formed as a result of a breach in the Robe Range as sea levels rose by up to 140 m following the most recent glacial maximum, and comprises a beach ridge plain and relatively stable foredune (Short and Hesp, 1980). Following the stabilisation of sea levels to around the present level about 6,000 years before present, sand gradually infilled the Bay and this led to the formation of the beach ridge plain feature seen today.

A regional coastal process model of Rivoli Bay, illustrated in Figure 3, is based on a review of previous studies, site observations as well as wave modelling undertaken for this project. Note that this model does not include prevailing ocean currents which generally move from north to south along the SE South Australian coastline (and are not shown in Figure 3), but is based on wave-generated sediment transport within the confines of Rivoli Bay only. It should be noted that while wave conditions can transport sand into the Bay, ocean currents may have a role in transporting the finer sediment fractions out of Rivoli Bay, these being deposited in areas further south along the ocean-facing coastline south of Cape Buffon. Figure 3 illustrates the major regional sediment transport pathways, with predominant wave energy vectors as determined from wave transformation modelling superimposed.

Net sediment transport into the Bay is driven by wave action, as the sand is actively mobile due to the shallow depths within Rivoli Bay. The wave climate is directed predominantly from the south-west, driving sediment transport into the Beachport and Southend areas of Rivoli Bay and net onshore movement into the central area of Rivoli Bay. Due to the shallow water depths within the Bay, the sand is able to be mobilised by wave action and has been actively transported onshore since sea levels stabilised following the most recent glacial maximum, around 6,000 years before present. This is evidenced by the presence of an extensive system of dune ridges along the central portion of Rivoli Bay which is more than two kilometres wide at its widest point. Remnant peaks of the Robe Range outcrop as islands offshore from the central portion of Rivoli Bay, influencing local wave patterns and encouraging the formation of a sand lobe or salient along the foreshore at the centre of the Bay (Figure 3). Active sand lobes within the Bay and remnant areas of the Robe Range are represented generally in yellow in Figure 3, which indicates areas shallower than approximately 10 m. These areas may be subjected to active sediment transport as wave generated currents are strong enough in these areas to move the sediment. In general, areas shaded in blue which are deeper than around

12 m, would be subjected to infrequent sediment transport. These areas may act as sand sinks for offshore sediment transport during storms, whereby sand may be lost to the littoral system in these areas due to offshore sediment transport following storm events.

Littoral drift is active along the Beachport and Southend foreshores due to the predominant wave climate in these zones. Along the main central foreshore of Rivoli Bay (i.e. east of Lake George outlet and west of Lake Frome outlet), onshore-offshore sediment transport is the dominant sediment transport mechanism, due to the shoreline alignment in this area being in equilibrium with the dominant nearshore wave direction (as illustrated in Figure 3). Note that Figure 3 is an indicative illustration of nearshore wave-induced net sediment transport, and does not reflect seasonal changes in sediment transport direction or sediment transport due to wider ocean currents in the area. In particular, local anecdotal evidence from discussions with local fishermen has shown that there is a sediment transport pathway out of Rivoli Bay from Southend around Cape Buffon and southward along the shoreline, particularly during the winter months. Close to the shoreline, sediment transport driven by breaking waves and surfzone currents results in a mainly northward sediment transport at Beachport, driven by waves refracted around Penguin Island and Glen Point. Waves from most offshore directions would result in a net northerly flux of wave energy and, hence, sediment transport potential at the shoreline, due to wave refraction. This has been confirmed by the SWAN wave transformation modelling carried out in Appendix 2.

2.2.1 Cross-shore sediment transport

In addition to the longshore sediment transport processes, cross-shore sediment transport occurs as a result of storm events, with sand moving offshore in response to short-term erosion events.

Following storms, ocean swell replaces the sand from the offshore bars onto the beach face where onshore winds move it back onto the frontal dune. This beach building phase, typically, may span many months to several years. Following the build-up of the beach berm and the incipient foredunes, and the re-growth of the sand trapping grasses, it can appear that the beach has fully recovered and beach erosion has been offset by beach building (Figure 2).

However, in some instances, not all of the sand removed from the berm and dunes is replaced during the beach building phase. Sand can be lost to sinks, resulting in longer term ongoing recession of the shoreline. Further, over decadal time scales, changes in wave climate can result in beach rotation. The signature of the medium-term oscillations in sub-aerial beach sand store caused by decadal variations in the Southern Oscillation Index (SOI) and the fluctuations resulting from minor storm events are apparent in the profile data for Beachport.

2.2.2 Sediment Budget

Once the sand has been transported offshore into the surf zone, it may be moved alongshore under the action of the waves and currents and out of the beach compartment. Some of the sand that is transported directly offshore during storms may become trapped in offshore reefs, thereby preventing its return to the beach. Other direct losses of material from the beach may include the inland transport of sand under the action of onshore winds; this mechanism being called aeolian sand transport. Over the longer term, should the amount of sand taken out of the compartment by alongshore processes exceed that moved into the compartment from adjacent beaches or other sources, then there will be a direct and permanent loss of material from the beach and a deficit in the sediment budget for the beach (Figure 4). This will result in an increasing potential for dune erosion during storms and long term beach recession.

Obvious processes that may lead to a deficit in the sediment budget of a beach include the deposition of littoral drift into estuaries (such as what has been occurring at Lake George and Lake Frome), wind-blown sand off the beach (aeolian sand transport causing transgressive dune migration), mining the beach for heavy minerals and beach sand extraction operations. Other processes include the transport of quantities of littoral drift alongshore and out of a beach compartment, which may be larger than any inputs.

The quantification of sediment budgets for coastal compartments is exceedingly difficult. The usual practice is to identify the processes and to quantify the resulting beach recession using photogrammetric and survey techniques. Long term rates of shoreline recession have been quantified by the South Australian Department of Environment, Water and Natural Resources (DEWNR) for the Beachport foreshore using analysis of historical surveyed profile data.

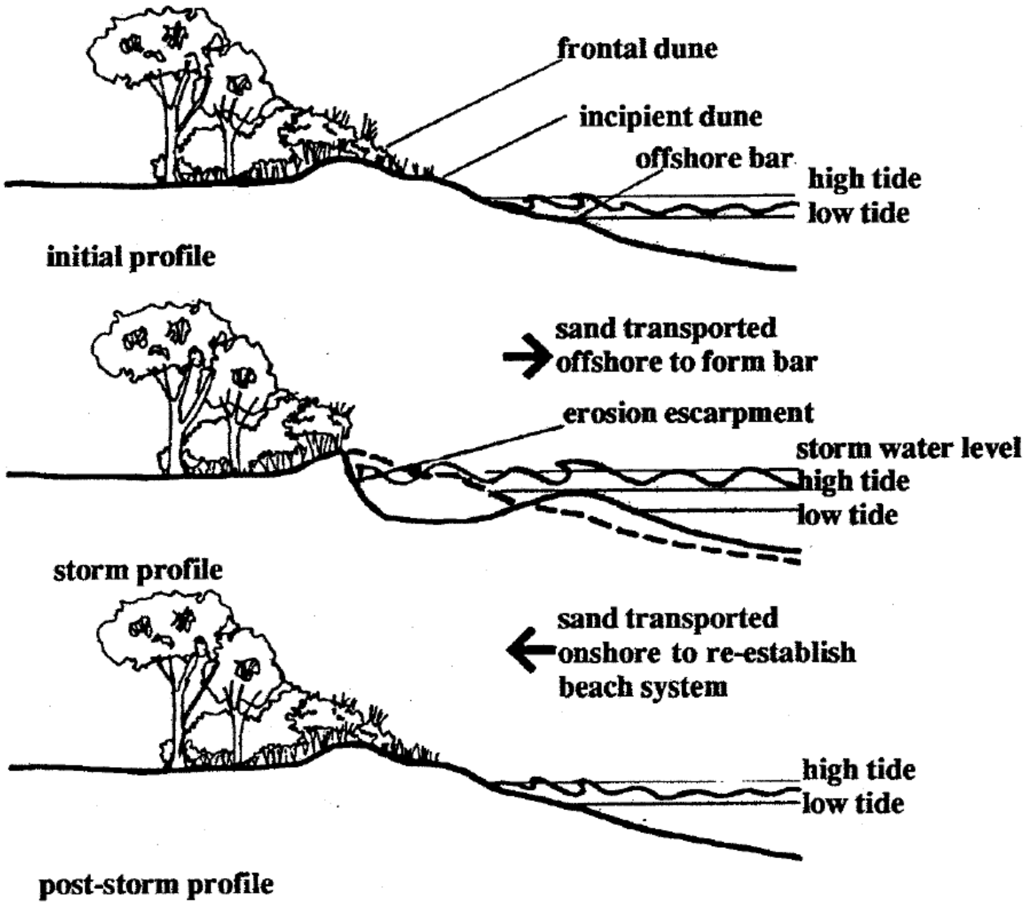


Figure 2 – Beach storm erosion/accretion cycle (after NSW Government, 1990)

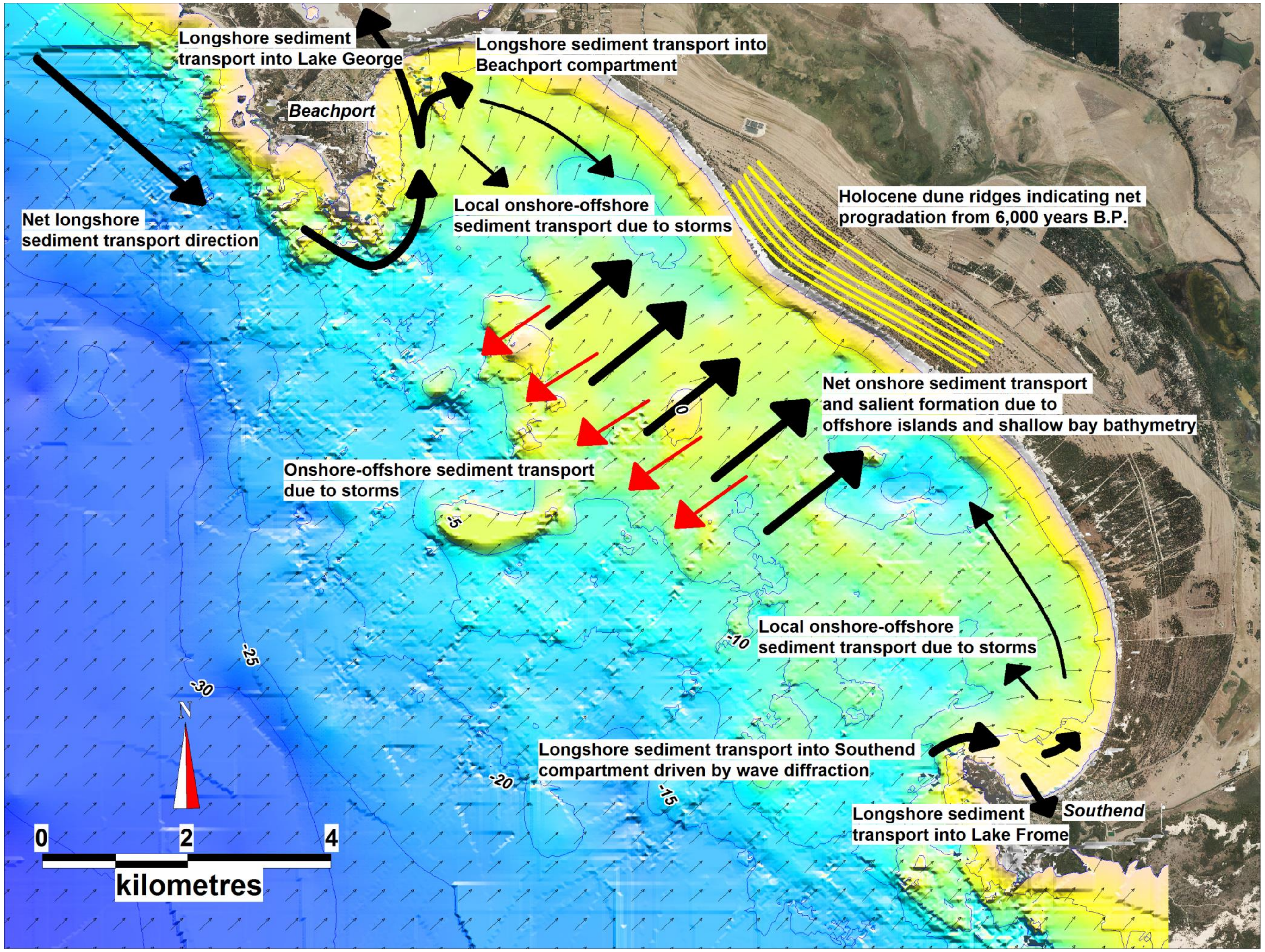


Figure 3 – Regional coastal processes conceptual model

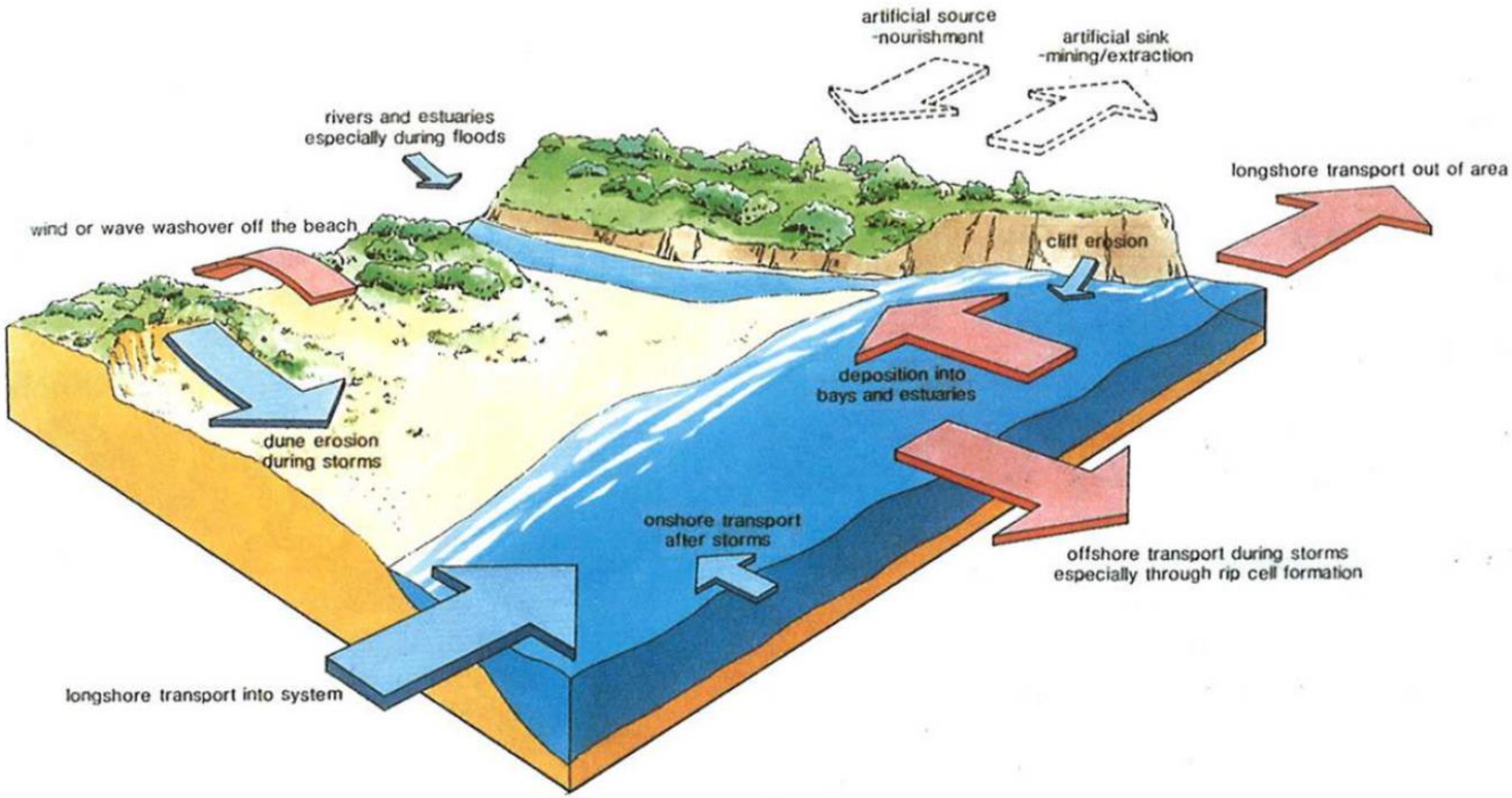


Figure 4 – Sediment Budget schema (after NSW Government, 1990)

2.3 Local coastal processes

2.3.1 Beachport

A local coastal process model of the Beachport area, illustrated in Figure 5, is based on a review of previous studies, site observations as well as wave modelling undertaken for this project. Figure 5 illustrates the sediment transport pathways into the Beachport area, with predominant wave energy vectors as determined from wave transformation modelling superimposed.

Net sediment transport into the western section of the Bay is driven by wave action as the sand is actively mobile due to the shallow depths within Rivoli Bay.

The local wave climate drives littoral drift northward along the foreshore at Beachport, as evidenced by build-up of sand on the southern faces of the groynes installed along the Beachport foreshore since the 1960's. It has been estimated that over 500,000 m^3 of sand had been transported into the outlet channel of Lake George until 1980 (Short and Hesp, 1980), forming an extensive flood-tide delta in the southern basin of Lake George. The channel to Lake George was constructed in the early 1920's and, since that time, Lake George has acted as a sink for northward longshore sediment transport along the Beachport foreshore. Localised deepening of the nearshore area around Glen Point has occurred since the 1980's as evidenced in historical profile data collected by DEWNR, possibly as a result of a loss of seagrass beds, mobilising sediment for longshore and onshore-offshore transport.

Onshore-offshore sediment transport occurs locally due to storm events, with sand moving offshore during storm events and gradually being carried onshore again under long low swells following the storm.

A more detailed local coastal processes model of the Beachport foreshore is illustrated in Figure 6. Superimposed on this Figure are the dominant local wave vectors resulting from south-westerly offshore waves as modelled using SWAN in Appendix 2. Local beach alignment angles have been drawn for each beach compartment (i.e. between individual groynes) based on the results of the SWAN modelling, with the beach alignment assumed to be perpendicular to the dominant nearshore wave angle. From this diagram, the following features are apparent:

- The beach compartments between individual groynes are closely aligned to the dominant wave angles indicating that the beaches have reached an equilibrium plan-form alignment with respect to the groynes and local wave climate.
- The beach compartments are all "full" and each groyne is actively bypassing sediment, as seen also in the field inspections.
- The groyne immediately north of the Beachport jetty is too short to allow a stable beach to form in the area around the jetty and adjacent to Railway Terrace.
- The breakwater that was constructed in November 2014 in the vicinity of the boat ramp has allowed littoral drift to bypass this area. However, since construction of this breakwater,

significant volumes of sand have been observed to be accumulating within the boat ramp basin.

- Offshore sediment transport occurs in this area due to wave reflections from the vertical seawall – this leads to a local deepening of the beach profile in this area allowing larger waves to reach this section of foreshore.
- There is an un-even buildup of sand within the various groyne compartments, which is a function of the varying lengths of the groynes.
- The three northernmost groynes appear to be set at a more appropriate spacing and have led to the formation of a beach dune in the area adjacent to the caravan park and Lake George outlet.

2.3.2 Southend

A local coastal process model of the Southend area, illustrated in Figure 7, is based on a review of previous studies, site observations and wave modelling undertaken for this project. Figure 7 illustrates the sediment transport pathways into the Southend area, with predominant wave energy vectors as determined from wave transformation modelling superimposed.

Net sediment transport into the eastern section of the Bay is driven by wave diffraction around Cape Buffon, as the sand here is actively mobile due to the shallow depths within Rivoli Bay.

The local wave climate drives southward sediment transport along the foreshore at Southend, along the rocky foreshore in the vicinity of the boat ramp and jetty and towards the bay foreshores.

Onshore-offshore sediment transport occurs locally due to storm events, with sand moving offshore during storm events and gradually being carried onshore again under long low swells in calmer weather following the storm. Sand can be carried offshore and around Cape Buffon in the winter months, where it leaves Rivoli Bay and moves further south along the coast.

Superimposed on this Figure are the dominant local wave vectors resulting from south-westerly offshore waves as modelled by SWAN in Appendix 2. Local beach alignment angles have been drawn for each beach compartment (i.e. between individual groynes) based on the results of the SWAN modelling, with the beach alignment assumed to be perpendicular to the dominant nearshore wave angle. From this diagram, the following features are apparent:

- The beach compartment west of the outlet to Lake Frome is aligned to the dominant wave direction here, with littoral drift directed toward the outlet to Lake Frome. This beach compartment is “full” with evidence that sand is being directed into the Lake Frome channel.
- The shoreline angles between individual groynes east of the Lake Frome outlet are closely aligned to the dominant wave angles indicating that transport of littoral drift in this section of beach is not as strong as it is at Beachport. The groynes east of the Lake Frome outlet are not as effective as those at the outlet at trapping sediment, due to their short length compared to their spacing. There has been a reduction in sediment supply from the west due to

sediment unable to bypass the outlet to Lake Frome, hence continuing beach recession occurring east of the Lake outlet.

- Longshore sediment transport is the dominant sediment transport mechanism west of Lake Frome and in the area east of the Lake outlet, as evidenced by continuing foreshore recession downdrift of the Lake outlet.

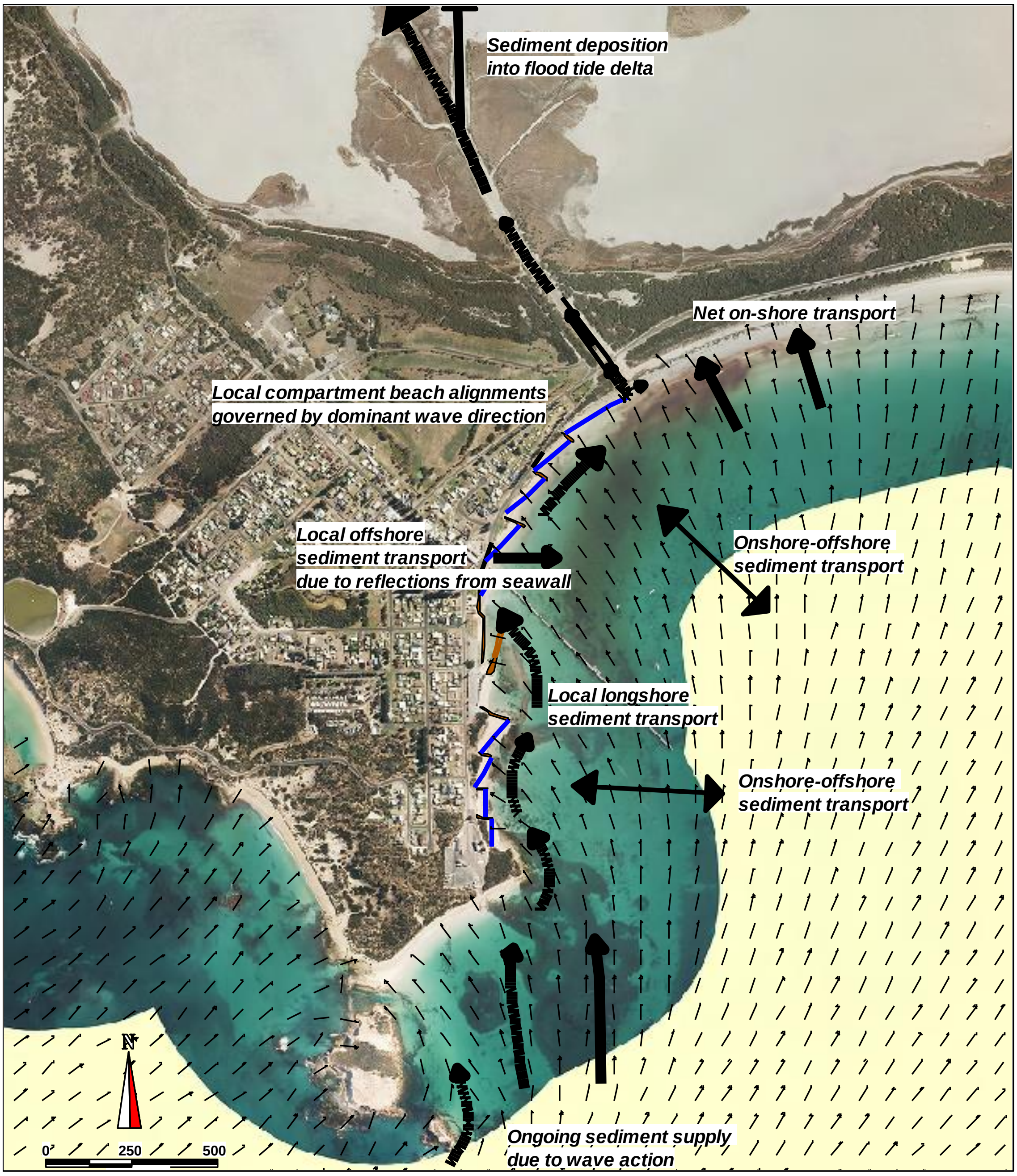


Figure 5 – Coastal process model within Beachport area

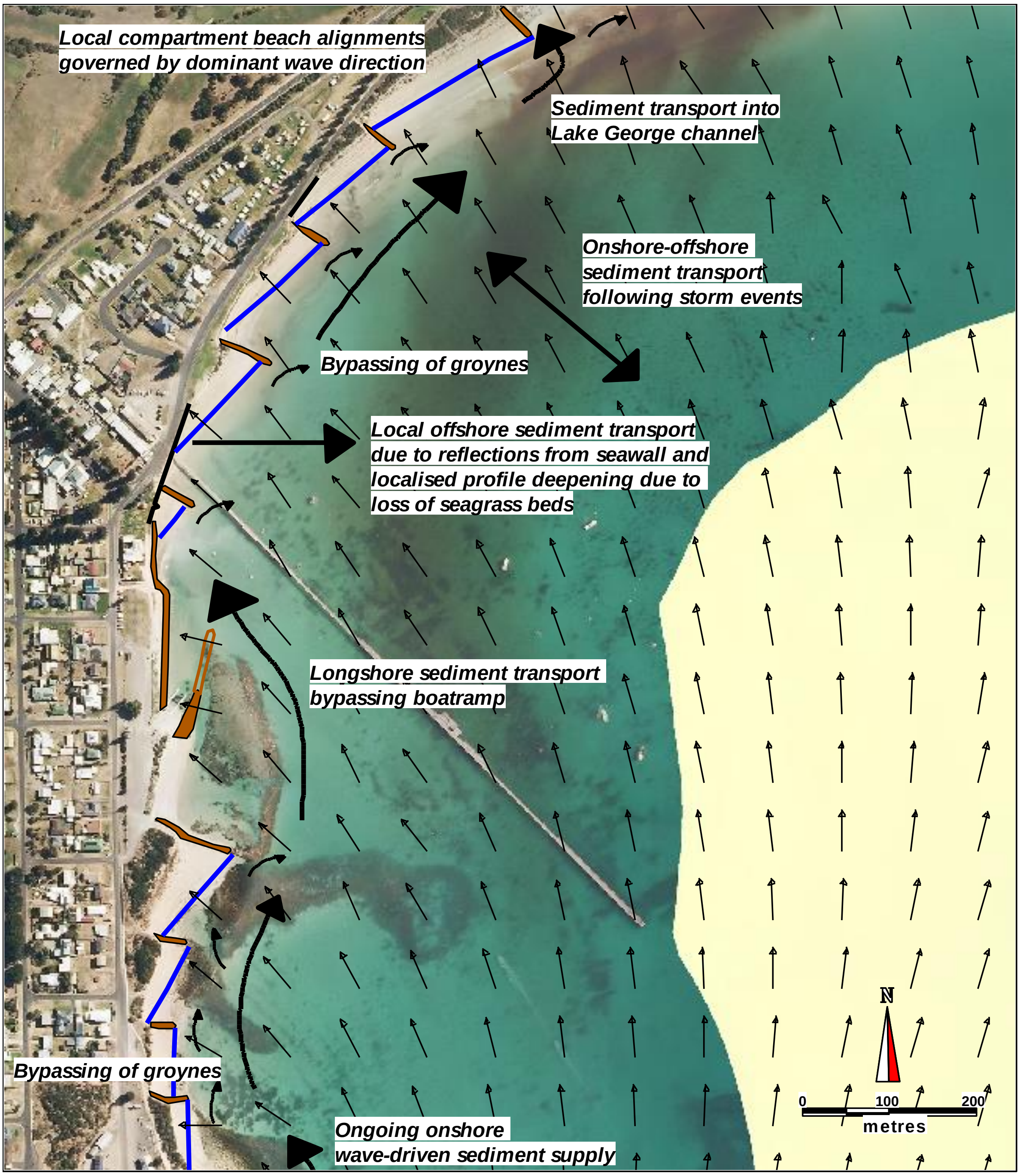


Figure 6 – Detailed coastal process model along Beachport foreshore

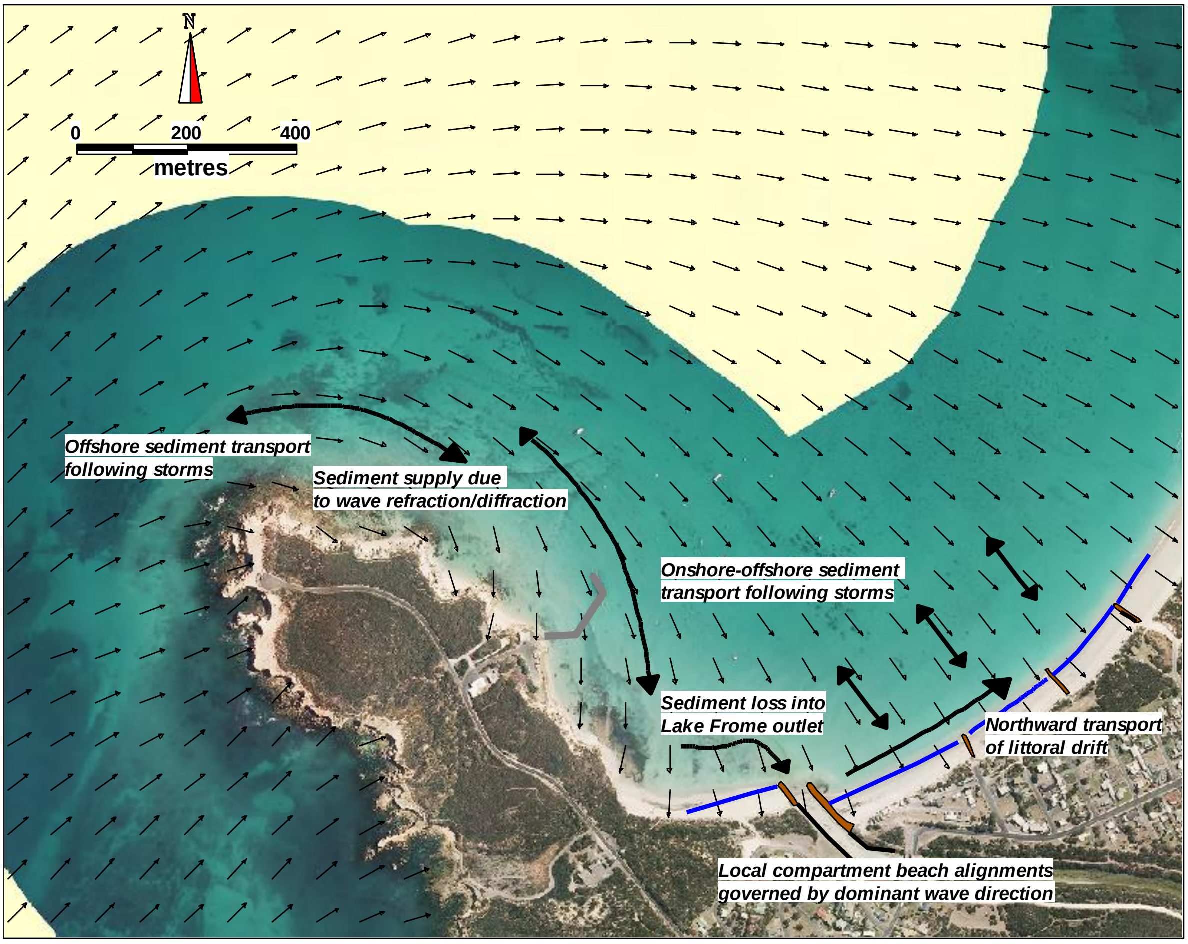


Figure 7 – Southend local coastal processes model

2.3.3 Sediment Sampling

Local sediment samples have been collected at various locations within Beachport and Southend to identify better and understand local sediment transport pathways as well as identify potential sand sources for use in any potential beach nourishment program.

The sand samples were collected from the beaches within the swash zone at low tide, which is where active sand transport takes place. Sand samples were collected also from the outlet channels of Lake George and Lake Frome.

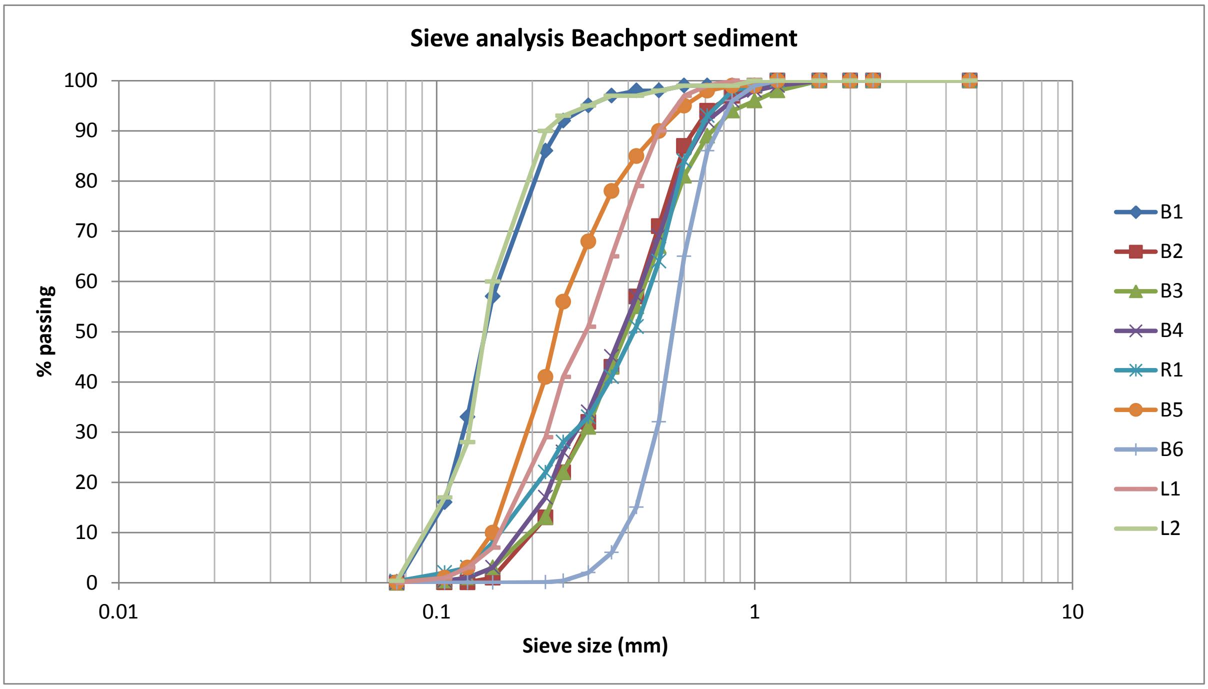
The locations where sand samples were collected are shown in Figure 8 for Beachport and Figure 9 for Southend and the results of the sand sampling are shown in Figure 10.

The following interpretations could be drawn from the sand sampling program:

- At Beachport, the coarsest sediment was seen at Sample B6 (Salmon Hole), due possibly to the higher wave climate at this location;
- Samples B2, B3 and B4 are all very similar and can be classified as medium grained sands, indicating a similar origin for the sand immediately surrounding the boat ramp and immediately north of the jetty area;
- Sample R1 near the centre of Rivoli Bay was also very similar in composition to the sand found along the Beachport foreshore;
- Sample B5 was finer than the sand at the other locations west of Lake George, possibly as a result of this location being more sheltered than other locations along the beach, allowing finer sediment to settle on the beach rather than being carried downdrift by wave-driven currents;
- Sample B1 (immediately east of Lake George outlet) was much finer than the sand west of the outlet, indicating that the coarser fraction of the longshore sediment transport is being carried into the Lake channel and only the finer fraction is able to bypass the channel;
- Sample L1 (in the lower section of the Lake George channel) was slightly finer than but comparable to the sand from the beach west of the channel, indicating that the source of this sand is from the beach to the west;
- Sample L2 (in the upper section of the Lake George channel) was finer than the sand in the lower section of the channel, as would be expected due to “dropping out” of the coarser fractions of the incoming sand transport in the lower section of channel.
- At Southend, Sample S1 and S2 were very similar, indicating that the sand transport pathway here is from the western section of beach into the Lake Frome channel;
- Sample S3 was coarser than Sample S1 and S2 and very similar to the sand in Beachport and at the centre of Rivoli Bay, indicating that the finer fraction was being drawn into the Lake and being more actively transported by the erosion process occurring at this location.

Figure 8 – Beachport sediment sampling locations

Figure 9 – Southend sediment sampling locations



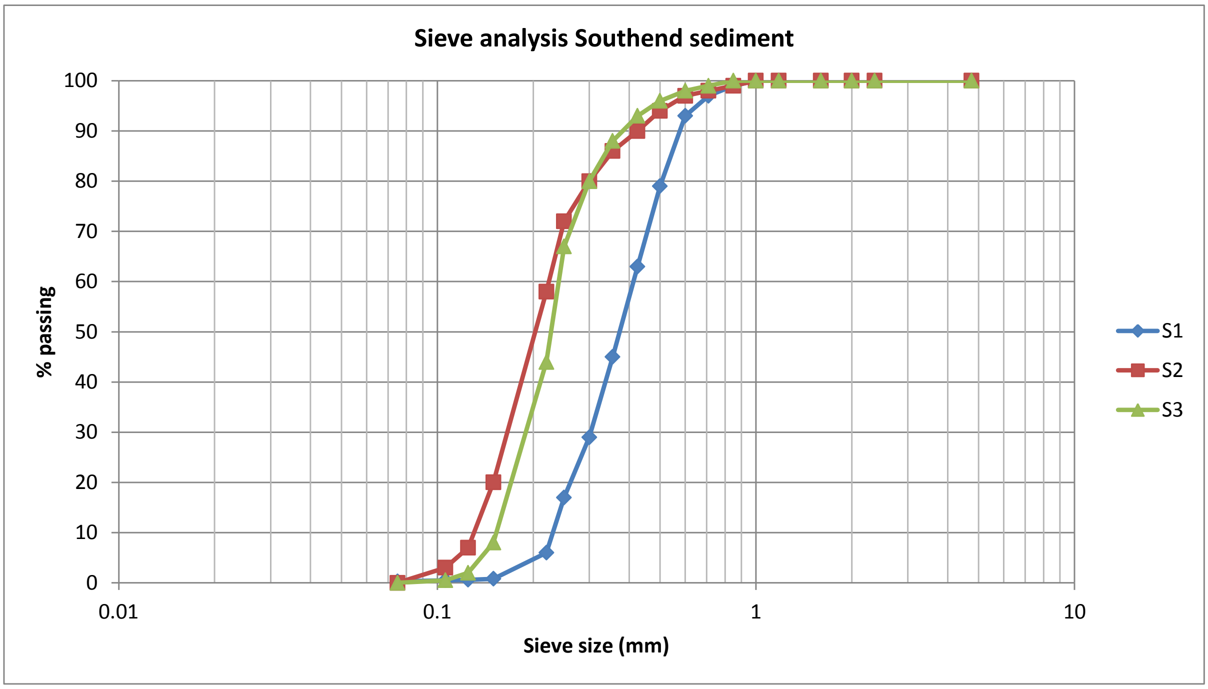


Figure 10 – Sediment sieve analysis for Beachport and Southend

2.3.4 Beach erosion, recession and coastal inundation

The beach is often perceived to be the sandy area between the waterline and the dunes. It includes the beach berm, where sand-binding grasses may exist, and any incipient foredune formations. Typically, however, on an open coast the overall beach system extends from some several kilometres offshore, in water depths of around twenty metres to the back beach dune or barrier region, which may extend up to several hundred metres inland (Figure 11). When examining the coastal processes of a beach system often it is necessary to consider this wider definition.

The principal hazards induced by the coastal processes that are relevant for a coastal hazard risk assessment of the beach along the Rivoli Bay coastline include:

- short-term coastal erosion including that resulting from severe storms, the behaviour of estuary entrances and slope instability;
- long term coastline recession including that resulting from imbalances in the sediment budget, such as aeolian sand transport, climate change and beach rotation; and
- oceanic inundation of low lying areas.

The hydrodynamic forces controlling the rate of these processes and hazards comprise the prevailing wave climate and water levels.

2.3.4.1 Short Term Coastal Erosion

Typically, a beach comprises unconsolidated sands that can be mobilised under certain meteorological conditions. The dynamic nature of beaches is witnessed often during storms when waves remove the sand from the beach face and the beach berm and transport it, by a combination of longshore and rip currents, beyond the breaker zone where it is deposited in the deeper waters as sand bars (Figure 11). During severe storms, comprising long durations of severe wave conditions, the erosion continues into the frontal dune, which is attacked, and a steep erosion escarpment is formed. This erosion process usually takes place over several days to a few weeks. At Southend and Beachport, sections of beach are separated by groynes, forming discrete beach compartments which are partially self-contained.

The amount of sand eroded from the beach during a severe storm will depend on many factors including the state of the beach when the storm begins, the storm intensity (wave height, period and duration), direction of wave approach, the tide levels during the storm and the occurrence of rips. Storm cut is the volume of beach sand that can be eroded from the subaerial (visible) part of the beach and dunes during a design storm. Usually, it has been defined as the volume of eroded sand as measured above mean sea level (~ 0 m Australian Height Datum, AHD datum). For a particular beach, the storm cut (or storm erosion demand) may be quantified empirically with data obtained from photogrammetric surveys, or it may be quantified analytically using a verified numerical model.

The history of severe storm erosion demand for the beaches at Beachport and Southend was unable to be determined due to the lack of suitable pre and post-storm profile surveys and the lack of available data with which to quantify local values of storm erosion demand. However, a study of generic coastal erosion volumes and setbacks covering the entire Australian coastline has been undertaken by the University of NSW Water Research Laboratory (WRL 2012). For the fully-exposed section of coastline covering south-eastern SA, WRL (2012) suggest a value of storm erosion demand of 200 m^3/m be adopted. WorleyParsons (2014) has found that storm erosion demand is related directly to the wave energy reaching the coastline. Therefore, the storm erosion demand will be lower in the more sheltered locations of Rivoli Bay such as at Beachport and Southend. The SWAN model developed for this project would allow for storm erosion demand volumes to be estimated for different sections of the Rivoli Bay foreshore, which can be validated for a future known storm event if pre and post-storm beach profile surveys are available. Measurements carried out by DEWNR found changes in beach profile volumes of up to 100 m^3/m between successive profile surveys at Glen Point. However, most of the documented changes have been over the entire profile and not within the dune. At Railway Terrace in Beachport, beach profile volumes have shown a steady decline within the dune area, although this has documented the long term recession of the dune, the signature of short term erosion due to storm events is not visible in the data.

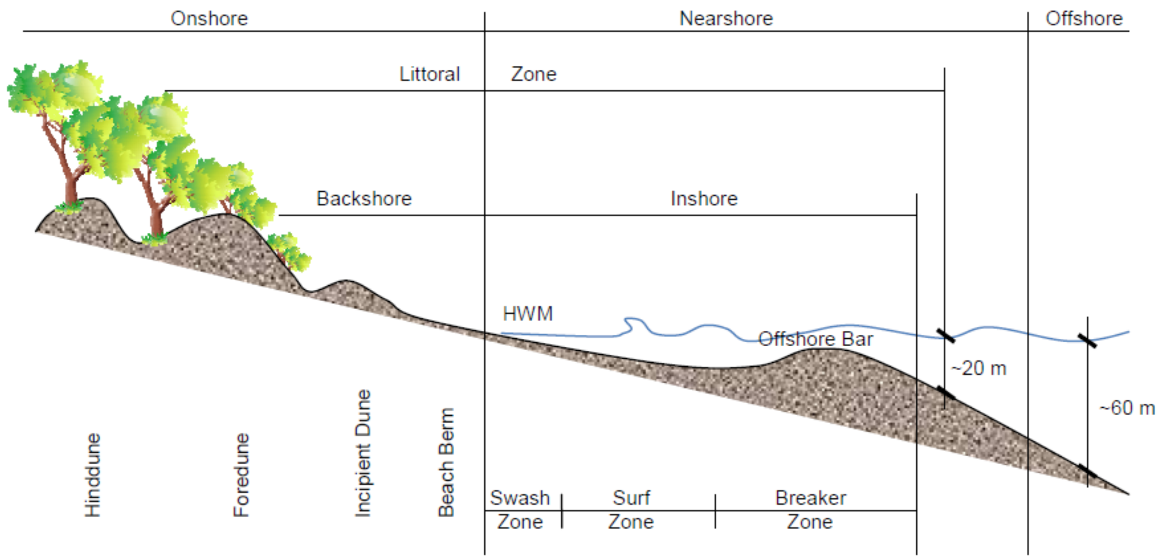


Figure 11 – Beach definition sketch open coast beaches (not to scale)

2.3.4.2 Slope Instability

Slope instability refers to the instability of both sandy dune areas, and rocky cohesive bluffs and headlands.

Following storm cut the dune face dries out and may slump. This results from the dune sediments losing their apparent cohesive properties that come from the negative pore pressures induced by the water in the soil mass. This subsequent slumping of the dune face causes further dune recession.

Dune slumping is treated as a slope instability hazard and can be quantified with stability computations, which can serve as a guide to determining safe setback distances on frontal dunes that are prone to wave attack and slumping during storms.

Typically, the dune erosion hazard is defined as:

- a line delineating the limit of wave impact and dune slumping (*Zone of Wave Impact and Slope Adjustment,* refer Figure 12); and
- a line delineating the limit of the area behind the dune face where the capacity of the sand to support building foundations is reduced because of the sloping dune escarpment (*Zone of Reduced Foundation Capacity*, refer Figure 12).

An illustrative example of how the dune erosion hazard zones would apply is provided in Figure 13, for an area where there was a steep erosion escarpment where infrastructure was considered to be at risk at Southend following the June 2014 storms, immediately east of the outlet to Lake Frome.

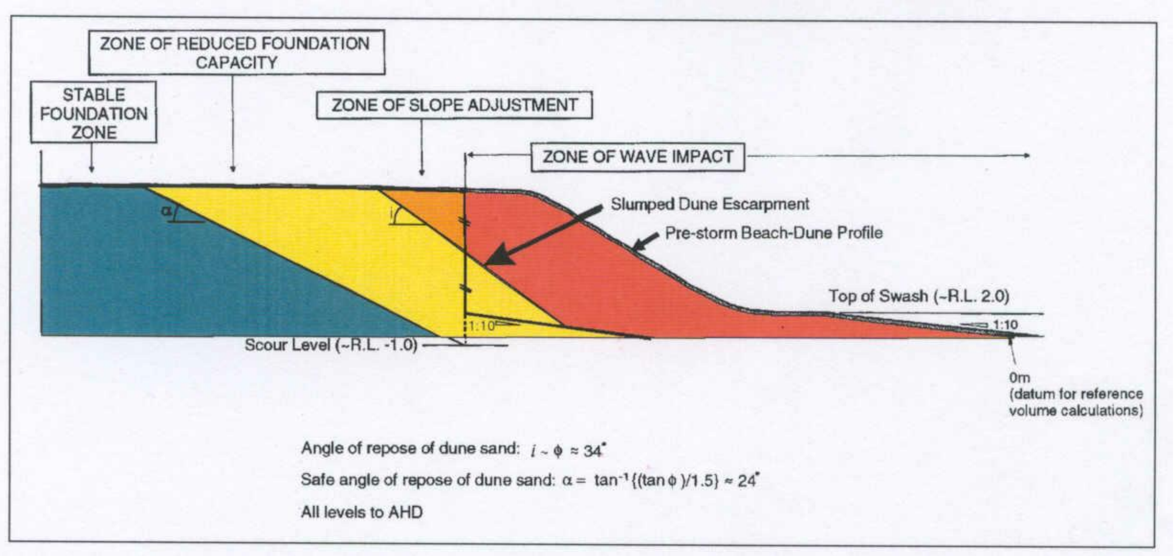


Figure 12 - Schematic representation of dune erosion hazard (after Nielsen et al, 1992)

Bluffs and headlands with varying slope angles and heights are common features along the shore for the areas around Cape Buffon and the coastline west of Glen Point. Potential slope instability in bluffs and headlands constitutes a foreshore hazard, also referred to as a slope instability hazard.

Figure 13 – Example of the application of the dune erosion hazard zones at Southend (June 2014)

Slope instability of bluffs and headlands is a result of the continuing operation of physical processes as well as anthropogenic activities within a particular geological and geomorphological setting in the coastal landscape. The physical processes could include rainfall, climate, rock weathering and disintegration, surface and ground water movement, soil erosion, sea level fluctuation, wave impact and earthquakes. On the other hand, coastal urbanisation and land use causing, for example, destruction of vegetation, either intentionally or otherwise, and the concentration of storm water flows may be regarded as anthropogenic factors. Slope failures in bluffs and headlands (both in rock, cohesive and unconsolidated sediments) are one of several coastal hazards that threaten the coastal community and values. A condition of slope instability may create public safety hazards, threaten existing infrastructure and affect sustainable development and use of coastal areas.

2.3.4.3 Behaviour Of Estuary Entrances

Various coastal hazards can be created by both trained and natural estuary entrances. There are no natural estuary entrances along the Rivoli Bay shoreline. However, two artificial entrances have been constructed, one connecting Lake George to the sea at Beachport and another connecting Lake Frome to the sea at Southend. Both of these entrances are trained along both of their banks by rock training walls and both are controlled by weirs which, typically, are closed off during the summer months and opened in winter, allowing water levels to be controlled within the lakes.

The issues associated with the estuary entrance include the entrainment of sediments into the estuary entrances by tidal currents, changing the hydrological characteristics of the lakes, as well as interruption of longshore sediment transport along the foreshore caused by the entrance training walls.

2.3.4.4 Beach Rotation

Studies of embayed beaches on the Australian coast have identified a sensitivity of shoreline alignment to wave direction (Short et al., 2000). Changes in long term wave climate and multi-decadal fluctuations in offshore wave direction would have an influence on longshore sediment transport and, hence, shoreline alignment within Rivoli Bay, particularly at the extreme ends of the Bay at Beachport and Southend.

The effect of a theoretical change in offshore wave angle of 5° on the shoreline alignment at Beachport and Southend was examined using the shoreline equilibrium model, MEP-BAY and the SWAN modelling in Appendix 2. MEP-BAY is an empirical crenulate bay shoreline model that calculates the idealised shoreline planform of a headland-bay beach in static equilibrium based on the parabolic model (Klein *et al.* 2003).

It was found that the shoreline alignment of Rivoli Bay is determined by the prevailing south-westerly wave angle at the locations where wave diffraction occurs (i.e. at Penguin Island and Cape Buffon, refer Figure 14). Local shoreline alignments at both Beachport and Southend are not very sensitive to changes in offshore wave angle, due to strong wave refraction and diffraction effects. SWAN modelling was used (Appendix 2) to examine the impact of a change in offshore wave climate to the local nearshore wave angle. Should the prevailing south-westerly wave climate become more southerly offshore by 5°, the change in nearshore wave angle near the shoreline at Beachport would be typically around 0.5°, indicating little change in the prevailing shoreline alignment (Figure 15). Similarly at Southend, should the prevailing wave climate become more westerly by 5°, there would be little change in shoreline alignment as the nearshore wave angle would typically change by around 0.5° (Figure 16). The orientation of the beach ridges at the centre of Rivoli Bay are parallel to the present day shoreline, indicating that there has been little change in nearshore wave direction over the Holocene period (approximately 6,000 years before present).

2.3.4.5 Long term recession due to sediment loss

Long term recession due to net sediment loss is a long duration process (period of decades), and can lead to continuing net loss of sand from the beach system. According to the sediment budget concept, this occurs when more sand is leaving than entering the beach compartment. This recession tends to occur when:

- the outgoing longshore transport from a beach compartment is greater than the incoming longshore transport;
- offshore transport processes move sand to offshore “sinks”, from which it does not return to the beach; and/or,
- there is a landward loss of sediment by windborne transport.

Shoreline recession due to net sediment loss should not be confused with beach erosion, which results in a short term exchange of sand between the subaerial and subaqueous portions of the beach, not a net loss from the active beach system. Therefore, shoreline recession is a long term process which is overlain by short term fluctuations due to storm activity.

In Rivoli Bay, the shoreline is generally not suffering from long term recession, except in areas which have been modified by anthropogenic activity. The formation of the beach ridge plain in the centre of the Bay indicates that the embayment has been accreting for thousands of years under natural conditions. Areas that have suffered long term recession include those areas downdrift of the groynes at the outlet to Lake Frome, the areas in front of the Beachport jetty which have suffered due to the impact of reflections from the vertical seawall, and areas where the bathymetric profile has deepened due to loss of seagrasses. There has also been a loss of sediment from the littoral system into the channels connecting Lake George and Lake Frome.

Figure 14 – MEP-BAY calculated shoreline alignment (shown in red) with indicated wave angles, Beachport and Southend

Figure 15 - SWAN calculated change in nearshore wave angle at Beachport should offshore wave angle change by 5°

Figure 16 - SWAN calculated change in nearshore wave angle at Southend should offshore wave angle change by 5°

2.3.5 Climate Change

Climate is the pattern or cycle of weather conditions, such as temperature, wind, rain, snowfall, humidity, clouds, including extreme or occasional ones, over a large area and averaged over many years. Changes to the climate and, specifically, changes in mean sea levels, wind conditions, wave energy and wave direction, can be such as to change the coastal sediment transport processes shaping beach alignments.

Climate change has been defined broadly by the Intergovernmental Panel on Climate Change (IPCC 2007) as any change in climate over time whether due to natural variability or as a result of human activity. Apart from the expected climate variability reflected in seasonal changes, storms, etc., climate changes that are considered herein refer to the variability in average trends in weather that may occur over time periods of decades and centuries. These may be a natural variability of decadal oscillation or permanent trends that may result from such factors as changes in solar activity, long-period changes in the Earth's orbital elements (eccentricity, obliquity of the ecliptic, precession of equinoxes), or man-made factors such as, for example, increasing atmospheric concentrations of carbon dioxide and other greenhouse gases.

The Intergovernmental Panel on Climate Change (IPCC 2007) has indicated that the global average surface temperature has increased over the 20th century by 0.6°C and that this warming will continue at an accelerating rate. This warming of the average surface temperature is postulated to lead to warming of the oceans, which would lead to thermal expansion of the oceans and loss of mass from land-based ice sheets and glaciers. This would lead to a sea level rise which, in turn, may lead to the recession of unconsolidated shorelines. Coastal communities and environments are particularly vulnerable to climate change due to the potential for permanent coastal inundation and increasing coastal hazards associated with changing weather patterns and extreme weather events.

In the longer term, there may be global changes resulting from a postulated warming of the earth due to the accumulation in the atmosphere of certain gases, in particular carbon dioxide, resulting from the burning of fossil fuels (the Greenhouse Effect). The current consensus of scientific opinion is that such changes could result in global warming of 1.5° to 4.5°C over the next 100 years. Such a warming could lead to a number of changes in climate, weather and sea levels. These, in turn, could cause significant changes to coastal alignments and erosion.

2.3.5.1 Sea Level Rise

Global warming may produce also a worldwide sea level rise caused by the thermal expansion of the ocean waters and the melting of some ice caps. According to the Intergovernmental Panel on Climate Change (IPCC, 2013), the upper range estimate for sea level rise for the 21st century is 1.0 m. This is made up of various components, including thermal expansion of the oceans (the largest component), melting of the Greenland and Antarctic ice sheets and melting of land-based glaciers. There is considerable uncertainty in this estimate as it will depend on the future global rates of carbon emmissions. In addition to the effects of climate change, there is also an existing underlying rate of sea level rise that includes the effects of current local rates of isostatic and tectonic land movements.

Mitchell et al. (2001) quantified underlying rates of existing sea level rise at various tide gauge locations around Australia. The sum total of these influences would give an upper bound sea level rise of 0.90 m for a 100 year planning period.

The South Australian Coast Protection Board through the Coastal Erosion, Flooding and Sea Level Rise Standards and Protection Policy (1992) recommends an allowance of 0.3 m for sea level rise to the year 2050, and 1 m by 2100, when considering coastal inundation and long term recession effects and planning for coastal development.

A rising sea level may result in beach recession on a natural beach and an increased potential for dune erosion on a developed beach where the dune line may be being held against erosion by a seawall. The concept of beach recession due to sea level rise is illustrated in Figure 17.

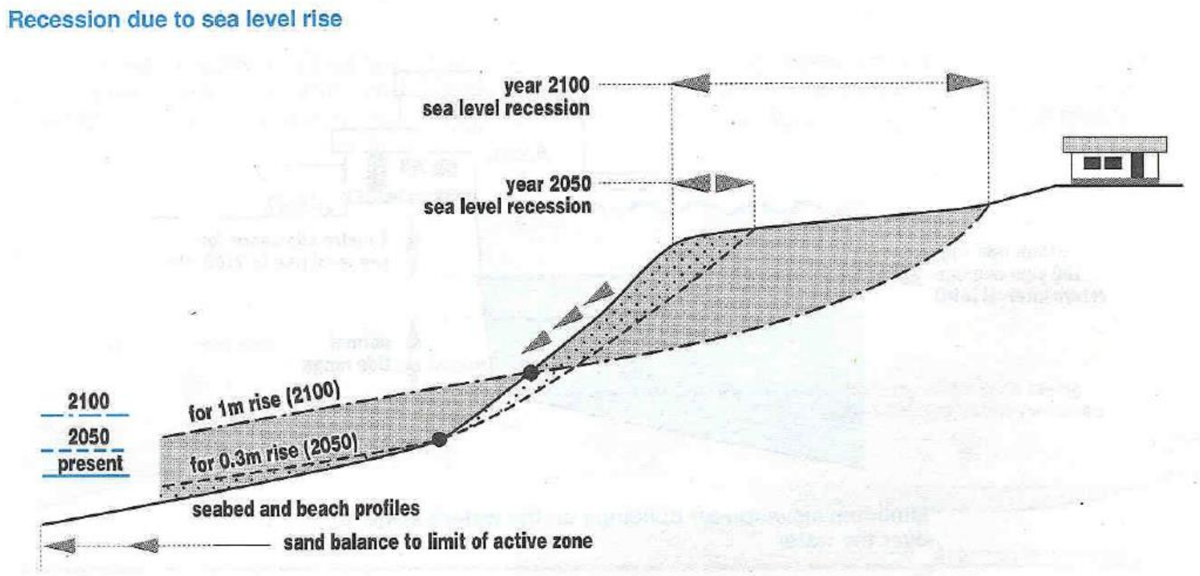


Figure 17 – Concept of beach recession due to sea level rise (SA Government, 1992)

Bruun (1962, 1983) investigated the long term erosion along Florida's beaches, which was assumed to be caused by a long term sea level rise. Bruun (1962, 1983) hypothesised that the beach assumed an *equilibrium profile* with the waves and sediment that kept pace with the rise in sea level, without changing its shape, by an upward translation of sea level rise (*S*) and shoreline retreat (*R*) (Figure 18).

The Bruun Rule equation is given by:

$$R = \frac{S \times L}{(h_c + B)} \tag{1}$$

where: R = shoreline recession due to sea level rise;
S = sea level rise (m)
h_c = closure depth
B = berm height; and
L = length of the active zone.

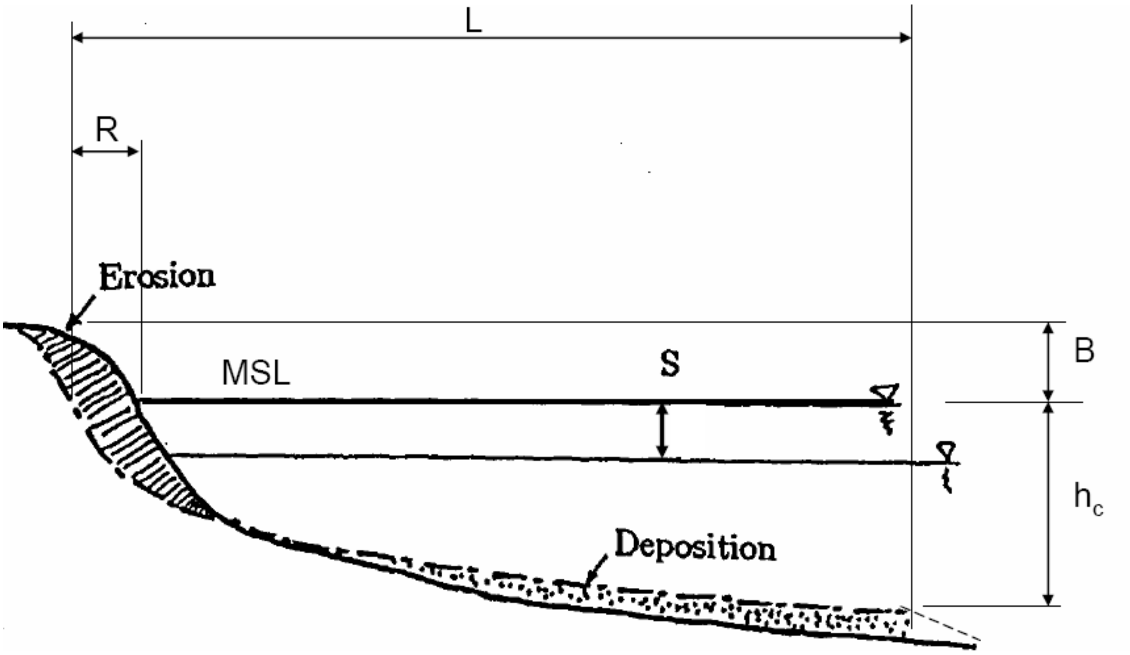


Figure 18 – Illustration of the Bruun Rule

Berm height (B) is taken to be the level of wave runup on the dune and closure depth is the depth at the seaward extent of measurable cross-shore sand transport. The length of the active zone is the distance offshore along the profile in which cross-shore sand transport occurs.

Bruun (1962) states that the depth of closure is "the outer limit for the nearshore littoral drift and exchange zone of littoral material between the shore and the offshore bottom area". According to Bruun, the depth of closure is the water depth beyond which repetitive profile surveys (collected over several years) do not detect vertical sea bed changes, generally considered to be the seaward limit of littoral transport. According to Bruun & Schwartz (1985), the depth can be determined from repeated cross-shore profile surveys, changes in sediment characteristics or estimated using formulas based on wave statistics. It is noted that the depth of closure does not imply the lack of sediment motion beyond this depth. Typical values used by Bruun were 12 m at Florida and 16 m in Denmark.

A synthesis and discussion of the available methods for estimating the depth of closure is provided below, including estimation of the depth of closure for the study area.

Finally, the beach profiles at Rivoli Bay are to be examined against the basic Bruun Rule assumption of the wave-equilibrium profile. Where beach profiles are steeper than the equilibrium profile then the profile slope to the limit of littoral drift transport should be adopted for the application of the Bruun

Rule. Where the beach profile generally is flatter than the equilibrium profile then the profile slope to the point of profile diversion should be adopted for the application of the Bruun Rule.

2.3.5.2 Equilibrium Beach Profile

The fundamental assumption of the Bruun Rule is that the sediments comprising the beach profile are in dynamic equilibrium with the wave climate. The Bruun Rule applies only to such a profile.

Bruun (1954, 1962) proposed a simple power law to describe the relationship between water depth, *h*, and offshore distance, *x*, measured at the mean sea level:

$$h = Ax^m \tag{2}$$

where *m* is an empirical coefficient, commonly adopted as 0.67 (Bruun, 1954, 1962; Dean 1977, Kotvojs and Cowell, 1991), and *A* is a dimensional shape factor, loosely dependent on the grain size but can be derived empirically also. Figure 19 (modified by Dean from Dean, 1987; US Army Corps of Engineers USACE 2002) gives an empirical relationship between *A* and grain size, D.

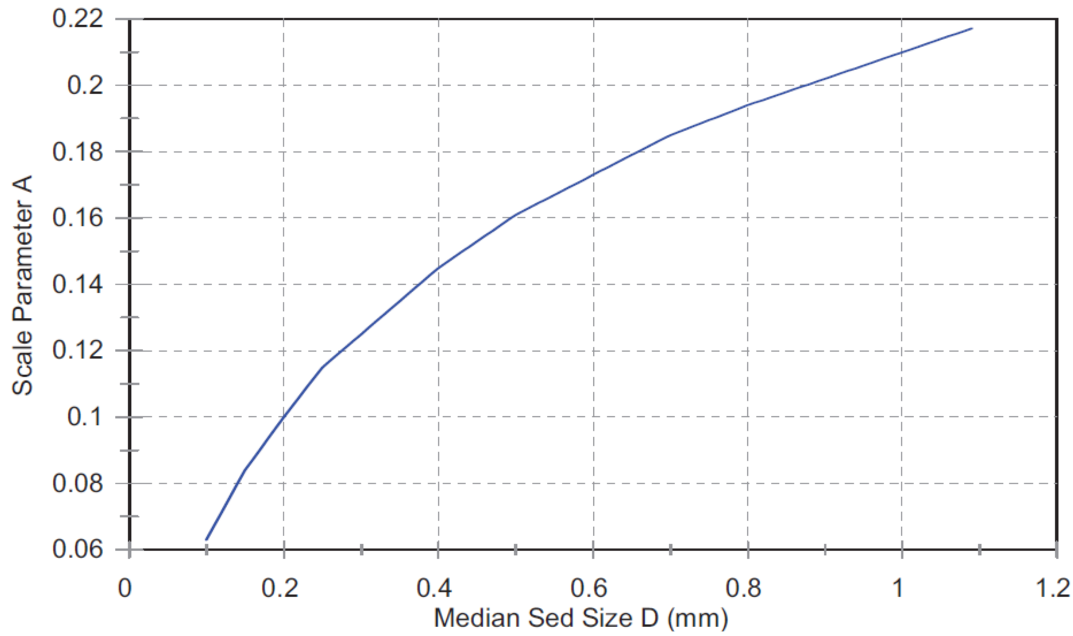


Figure 19 - Relationship between sediment characteristics and the profile scale parameter (US Army Corps of Engineers 2002)

Based on Figure 19, for the median grainsize range found at Rivoli Bay (Figure 10) from 0.15 mm to 0.4 mm, *A* was found to vary from 0.08 to 0.14.

This wave equilibrium profile that forms the basic assumption of the Bruun Rule has a shape that is concave upward. Beach profiles that are concave downward and/or flatter than the equilibrium beach profile are considered to be in an accretionary state whereas steeper profiles are associated with beach erosion.

2.3.5.3 Depth of Closure

Hallermeier (1981, 1983) defined three profile zones, namely the littoral zone, shoal or buffer zone[1], and offshore zone. These zones were defined by two depths, namely:

- an “inner” (closer to shore) depth at the seaward limit of the littoral zone, termed d_l by Hallermeier (1981) and d_s by Hallermeier (1983), and d_{inner} herein; and,
- an “outer” or “lower” (further from shore) depth at the seaward limit of the shoal/buffer zone, termed d_i by Hallermeier (1981) and d_o by Hallermeier (1983), and d_{outer} herein.

From Hallermeier (1983):

$$d_{inner} = \frac{2.9H_e}{\sqrt{S-1}} - 110\left(\frac{H_e^{\,2}}{(S-1)gT_e^{\,2}}\right) \tag{3}$$

where H_e is the effective significant wave height exceeded for 12 hours per year (that is, the significant wave height with a probability of exceedance of 0.137%) and T_e is similarly defined for wave period. Based on measured wave data from Cape du Couedic, H_e is about 6.5 m and T_e is about 14 s. The wave refraction coefficient for the beach at Beachport and Southend as derived from wave transformation modelling in Appendix 2 is around 0.7. From Equation 3 the inner closure depth is thus about 10 m.

From Hallermeier (1983):

$$d_{outer} = 0.018H_mT_m\sqrt{\frac{g}{D(s-1)}} \tag{4}$$

where H_m and T_m are the median wave heights and periods respectively, D is the median sediment diameter and S is the specific gravity of sand (about 2.65). Based on measured offshore wave data, H_m is about 2.7 m, T_m is about 14 s and the wave refraction coefficient for the open coast beaches is around 0.7. For the grain size of around 0.3 mm, from Equation 4 the depth to the outer shoal zone is around 57 m.

According to Hallermeier (1981), “The middle zone is a buffer region where surface wave effects on a sand bed have an intermediate significance. This region is named the shoal zone primarily because the sand transport processes considered here result in deposition of sand from the flanking zones: extreme waves can carry some littoral-zone sand into the landward section of the shoal zone and

[1] Shoal zone in Hallermeier (1981) and buffer zone in Hallermeier (1983).

common waves can carry some offshore-zone sand into the seaward section". That is, the limit of cross-shore transport of littoral sand does not extend far past the inner limit of the shoal zone.

Rijkswaterstaat (1987), approximating the work of Hallermeier (1978, 1981, 1983), found the following simplified estimate for the effective depth of closure, d_c, namely:

$$d_c = 1.75 H_e \tag{5}$$

Therefore, the predicted closure depth from Equation 5 is about 8.6 m.

Analysis of data from the digitised bathymetric profile data provided by DEWNR showed that the nearshore profile was in equilibrium down to a depth of between 2 – 4 m and a profile length varying between 100 and 250 m (Figure 20). The equilibrium profile lengths have been assessed from the beach profile graph. These two characteristics are the coordinates of the last point fitting with the equilibrium profile. Beyond these depths, the profile does not conform to an equilibrium profile. The bathymetric data indicate the presence of a relatively shallow, very wide convex upward sand shoal over which low swell waves propagate shoreward, causing onshore sediment transport. This process has been continuing at Rivoli Bay since the end of the most recent glaciation (around 6,000 years before present). The evidence for extensive progradation of the beach over that time can be seen in the formation of the extensive beach ridge barrier which is the dominant feature over the central portion of the Bay. This would imply that significant long term recession due to sea level rise would likely not be very severe at Rivoli Bay, due to the relatively flat offshore profile compared with the respective equilibrium wave profile. However, sea level rise would reduce the onshore sediment transport rate compared with that experienced at present, which may reduce the ongoing rate of accretion within the Bay. This may lead to exacerbated impacts in the areas currently affected by shoreline recession (i.e. downdrift of the groynes at Lake Frome and at localised areas downdrift of the groynes at Beachport) due to reduced sediment supply to those areas.

As the application of the Bruun Rule is limited to the portion of the profile in equilibrium, the Bruun Rule cannot be applied to the beach at Southend or Beachport as the beach does not conform with the assumptions of the Bruun Rule. It should be noted that the dominant mechanism of sediment transport into the Beachport area is longshore sediment transport which would be expected to continue under sea level rise and that the Bruun Rule schematises the beach response to sea level rise purely as a cross-shore process. In addition, the beaches are stabilised by a series of groynes and seawalls in some locations, so long term recession in Rivoli Bay due to sea level rise is unlikely to occur. It should be noted, however, that recession has been observed along parts of the Beachport and Southend foreshores, with the Coastal Protection Board observing a general erosive trend along the Southend foreshore heading north around the bay for 1 – 2 km. Areas that have suffered long term recession include those areas downdrift of the groynes at the outlet to Lake Frome, the areas in front of the Beachport jetty which have suffered due to the impact of reflections from the vertical seawall, and areas where the bathymetric profile has deepened due to loss of seagrasses.

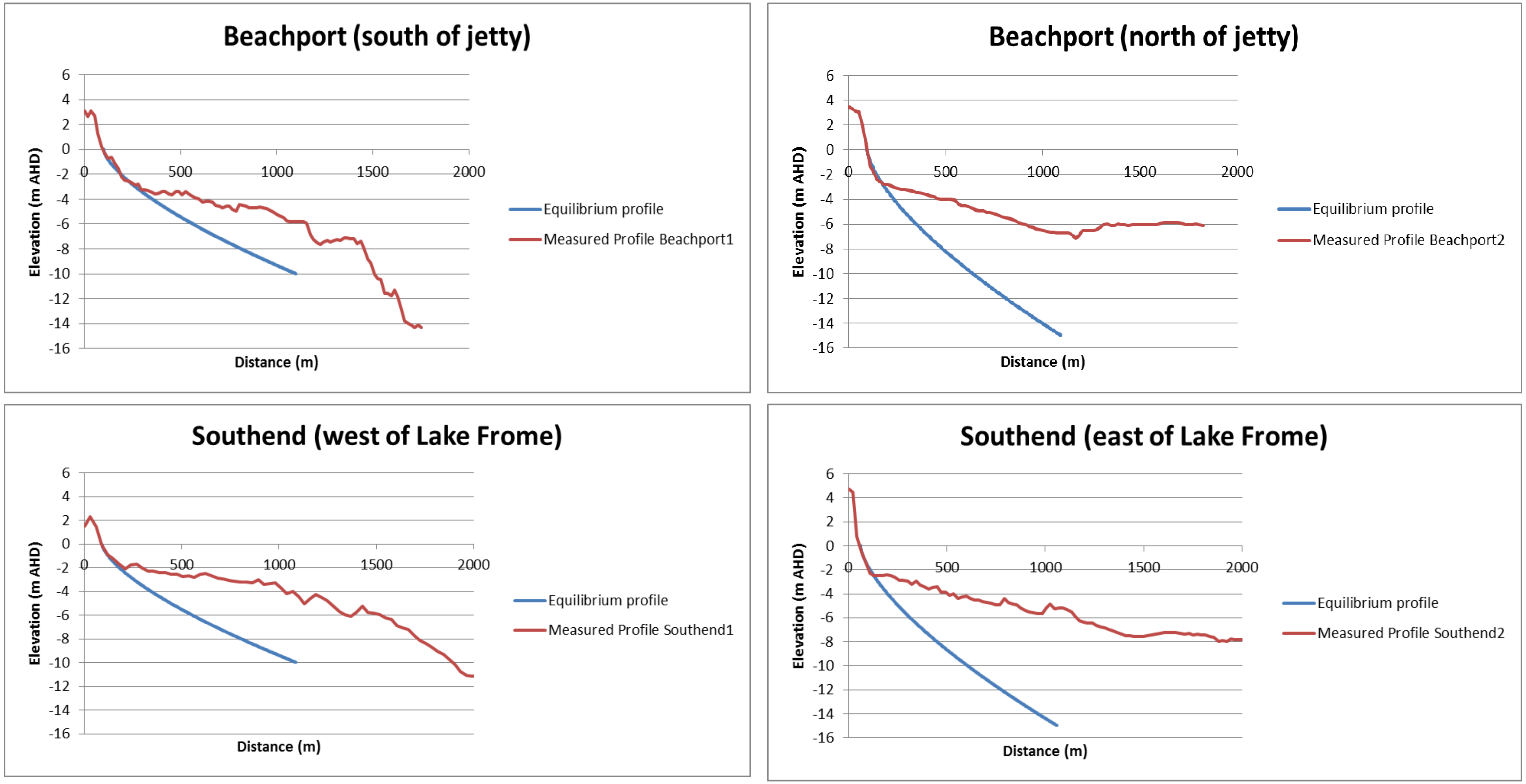


Figure 20 – Measured vs. equilibrium beach profiles at Beachport and Southend. Note the presence of significant quantities of nearshore sand, indicating continuing onshore transport over geological time scales.

2.3.6 Coastal Inundation

An increase in water level at the shoreline results from the breaking action of waves causing what is termed wave setup and wave run-up. Wave setup may be perceived as the conversion of part of the wave's kinetic energy into potential energy. The amount of wave setup will depend on many factors including, among other things, the type, size and periods of the waves, the nearshore bathymetry and the slope of the beach and foreshore. Typically, wave setup on an open-coast beach during severe storms can be around 1 m to 2 m.

The energy of a wave is dissipated finally as the water runs up the beach or shoreline. Wave run-up is the vertical distance the wave will reach above the level of the tide and storm surge and can be several metres. Wave run-up at any particular site is very much a function of the wave height and period, the foreshore profile and slope, surface roughness and other shoreline features on which the breaking waves impinge.

Should dune levels be low or the foreshore not protected by dunes, flooding and damage to structures can result from the coincidence of elevated ocean water levels and wave run-up.

2.3.6.1 Extreme Water Levels

During storms, the ocean water level and that at the shoreline is elevated above the normal tide level. While these higher levels are infrequent and last only for short periods, they may exacerbate any storm damage on the foreshore. Elevated water levels allow larger waves to cross the offshore sand bars and reefs and break at higher levels on the beach. Further, they may cause flooding of low lying areas and increase tail water control levels for river flood discharges.

The components of these elevated water levels comprise the astronomical tide, barometric water level setup, wind setup, wave setup and runup (Figure 21). All of the components do not act or occur necessarily independently of each other but their coincidence and degree of inter-dependence, generally, is not well understood.

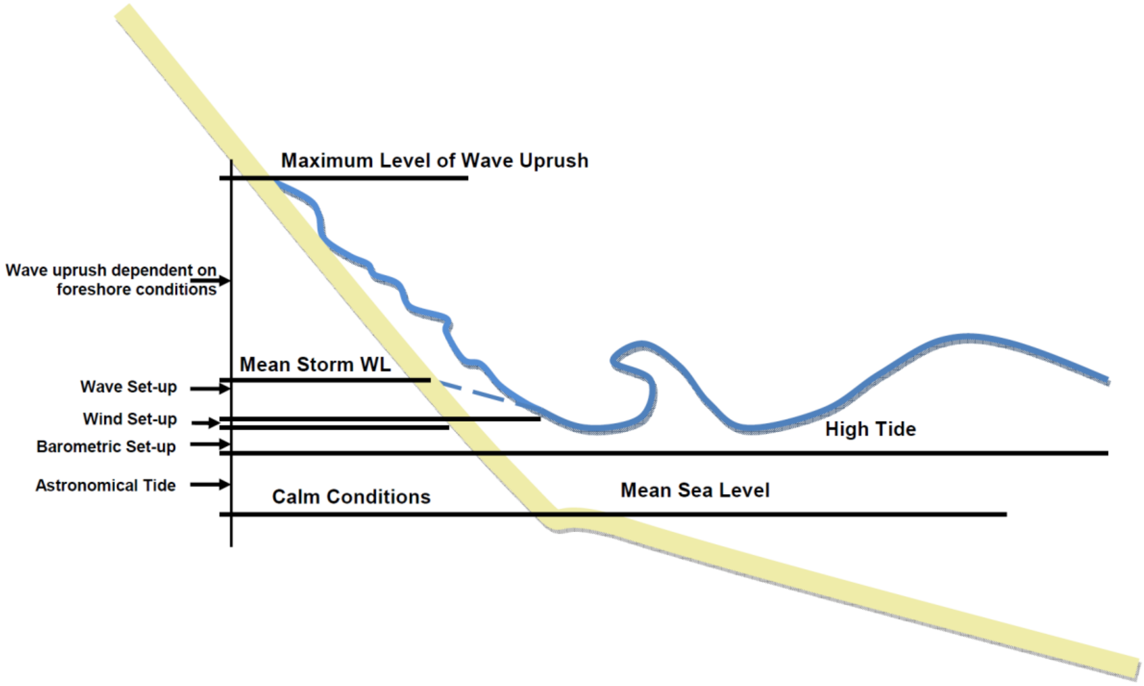


Figure 21 – Illustration of extreme water level components on a coastline

The tides of the south-east SA coast are mixed-semidiurnal type, exhibiting many diurnal characteristics. This means that there are two high tides and two low tides on some days and only a single high and low tide on other days. The mean tidal range is around one metre and the tidal period is around 12.5 hours. Tides vary according to the phases of the moon. The higher spring tides occur near and around the time of new or full moon and rise highest and fall lowest from the mean sea level. The average spring tidal range is 0.8 m and the maximum range reaches 1.6 m. Neap tides occur near the time of the first and third quarters of the moon and have an average range of around 0.4 metres.

Storm surge is the increase in water level above that of the normal tide that results from the low barometric pressures, which are associated with severe storms and cause sea level to rise, and strong onshore winds that pile water up against the coast. McInnes *et al.* (2009) modelled storm surge along the Victorian coast and found that at Portland, near the South Australian border, 100 year Annual Recurrence Interval (ARI) storm surge levels were approximately 0.5 – 0.6 m under present day conditions. This corresponds to a 100 year ARI storm tide height of 1.01 m AHD, with this level increasing under predicted climate change scenarios. Haigh *et al*. (2012) analysed tide gauge data around the entire Australian coastline and estimated a 100 year ARI extreme water level of 1.67 m AHD at Victor Harbour under present day conditions. Return periods for ocean water levels comprising tidal stage and storm surge for Portland as analysed by McInnes *et al.* (2009), which are representative of the study region, are presented in Figure 22 for present day conditions and under various scenarios for climate change.

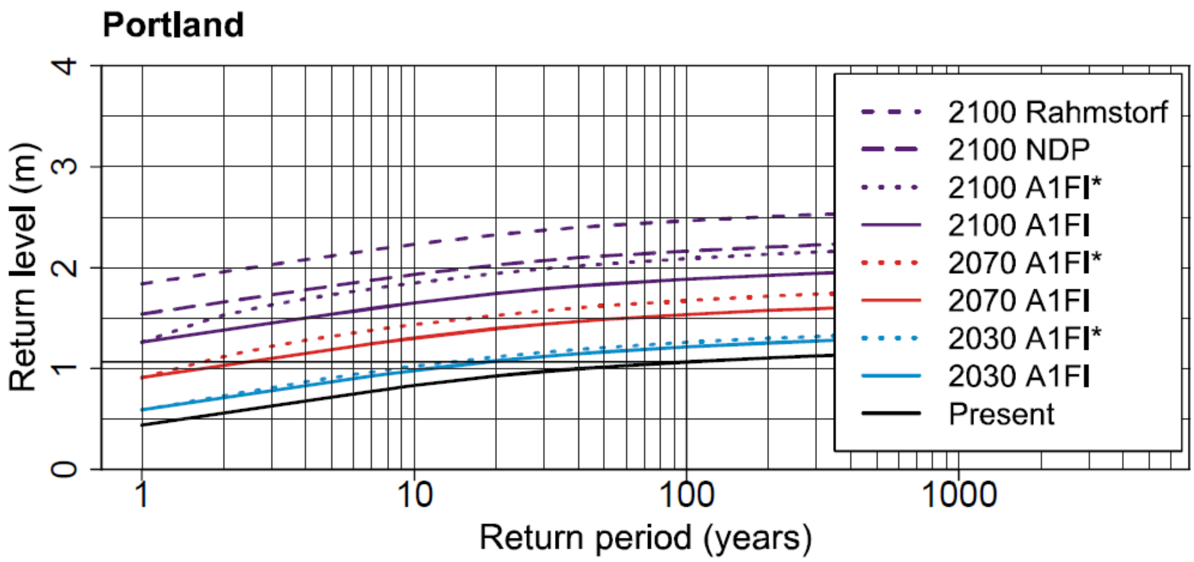


Figure 22 – Modelled storm tide return intervals at Portland, present day and climate change conditions, relative to present day mean sea level (McInnes *et al*. 2009)

An assessment of coastal inundation due to wave run-up for Beachport and Southend has been carried out in Section 4.7.1.

2.3.7 Wave Climate and Storms

The offshore swell wave climate (wave height and period occurrences) has been recorded by the Bureau of Meteorology with a Waverider buoy located at Cape du Couedic, off the south-west coast of Kangaroo Island (approximately 300 km west of Rivoli Bay) for approximately 10 years.

WRL (2013) undertook an analysis of wave buoy data for a study they undertook for Port Fairy, on the western Victorian coast approximately 200 km south-east of Rivoli Bay. For this analysis, WRL found the following extreme significant wave heights and wave periods for the Cape du Couedic wave buoy, which is applicable to the study area:

- 1 year ARI Hs = 7.3 m
- 10 year ARI Hs = 8.4 m
- 100 year ARI Hs = 9.8 m.

WRL (2013) also analysed directional wave data available from directional wave buoys installed at Cape Bridgewater, approximately 150 km south-east of Rivoli Bay, and as derived from global wave models including WAVEWATCH III and ERA-40. They found that the predominant wave direction for extreme waves affecting this portion of the coast is from the south-west sector.

A large storm occurred during the course of the site visit for this study, where offshore swell waves were measured at 8.9 m on Tuesday June 24. That storm resulted in coastal inundation and erosion damage to infrastructure at Beachport, which has been documented in Appendix 1. Because nearshore waves causing dune erosion are depth-limited, wave duration of moderate wave heights becomes a more important factor for dune erosion than peak offshore wave heights of short duration. The storm event was coincident with high spring tides, which maximised the inundation impact and allowed higher nearshore breaking wave heights as observed during the storm.

Such storms, which originate in the Southern Ocean and occur along the SA coastline at irregular intervals, are responsible for episodic events of sand transport and erosion, which are evident when examining historical surveyed profile data

2.3.8 Beach profile changes at Beachport

Analysis of historical beach profiles carried out by the Department of Environment, Water and Natural Resources has found that long term dune recession has not been occurring at Beachport (Figure 23), due to the ongoing supply of sand from the shallower areas of the Bay and from the coastline to the west, with a net gain of around 120 m^3/m over 30 years and a dune-face progradation of around 20 m. The exception to this is at the area adjacent to Railway Terrace at Beachport, with long term recession identified there resulting in a net sand volume loss of 100 m^3/m over 30 years and a dune-face recession of around 5 m (Figure 24). It is considered that wave reflections due to the presence of the vertical seawall may have caused a local deepening of the beach profile here, allowing larger waves to impact the shoreline causing local offshore sediment transport. In addition, the long distance between the groynes, incident wave angle and short length of the groynes in this area does not allow the formation of a beach spanning the entire compartment between these two groynes. It is considered that an additional groyne in this area would allow a beach to be sustained within the jetty area.

At Glen Point, there has been a net loss of sediment in the nearshore zone over the 30 years between 1977 and 2008, although the beach dune has continued to build up over that time (Figure 25). It is considered that loss of seagrass beds in the nearshore zone since the mid 1980's has allowed this sand to become more mobile in response to nearshore currents driven by waves, with some of the mobile sediment being driven on-shore onto the dune and some being transported northward toward the caravan park area and outlet to Lake George.

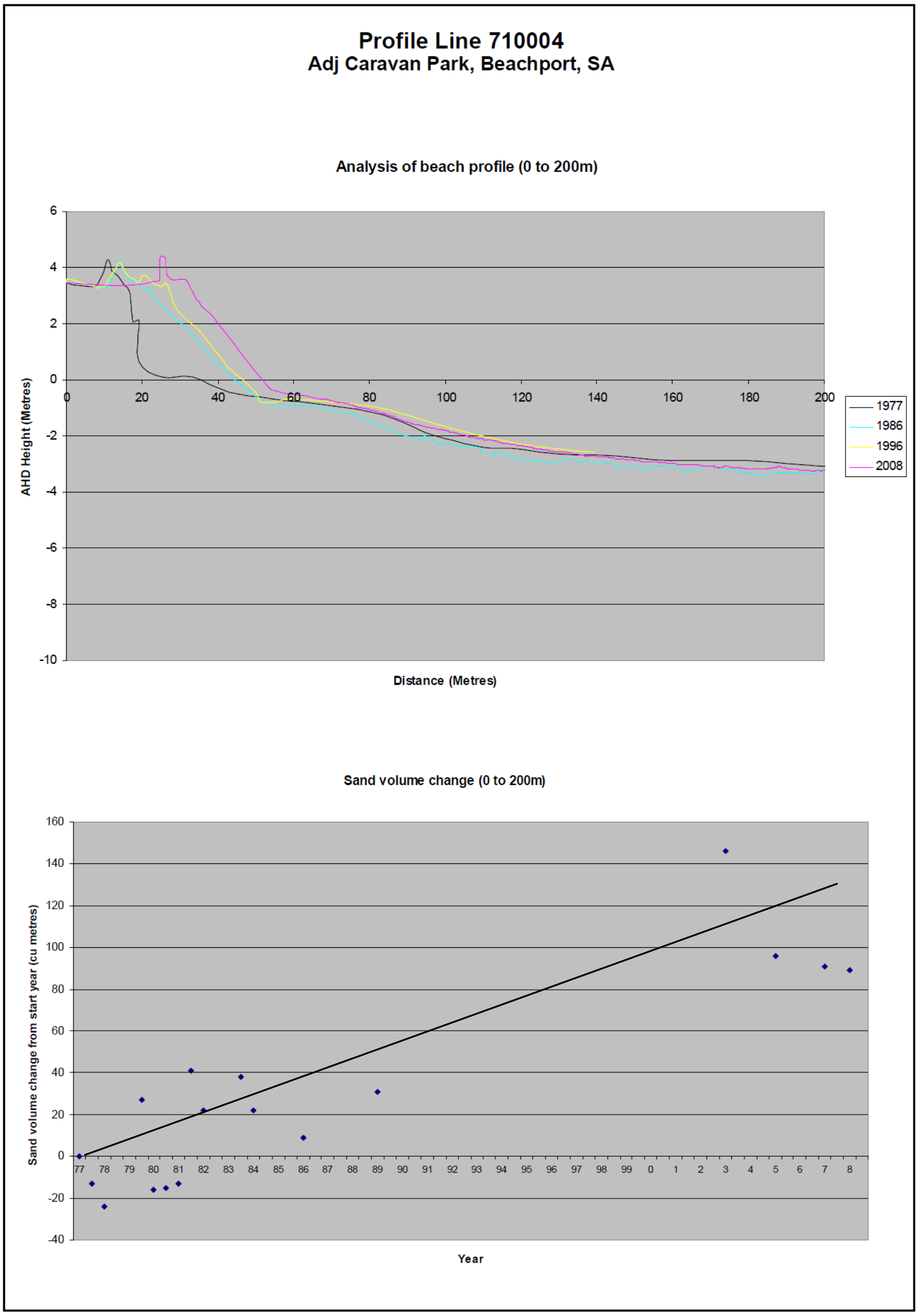


Figure 23 - DEWNR analysis of historical beach profiles adjacent to Caravan Park (eastern end Beachport), indicating long term accretion (trend line added)

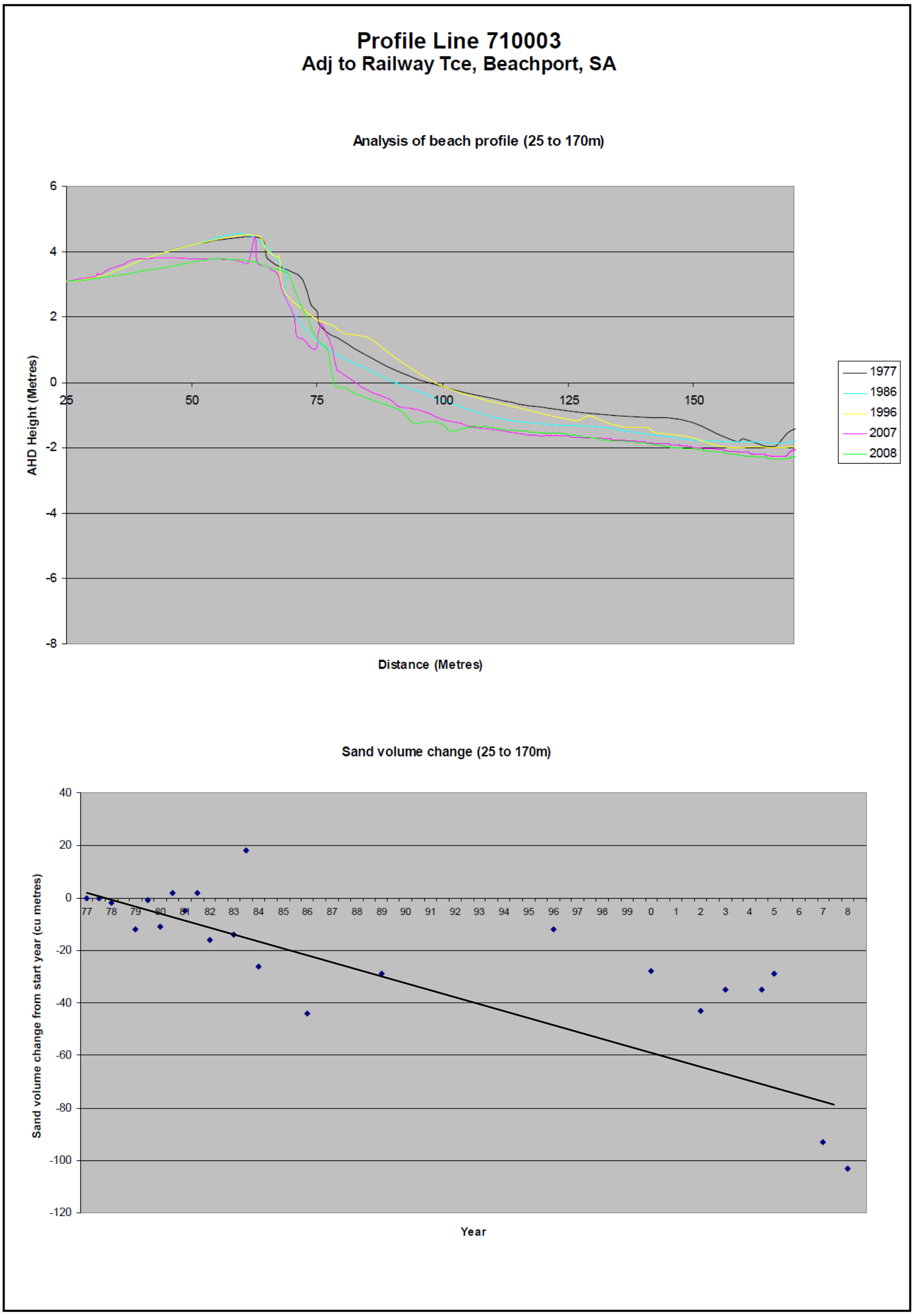


Figure 24 – DEWNR analysis of historical beach profiles adjacent to Railway Terrace (Beachport), indicating long term recession (trend line added)

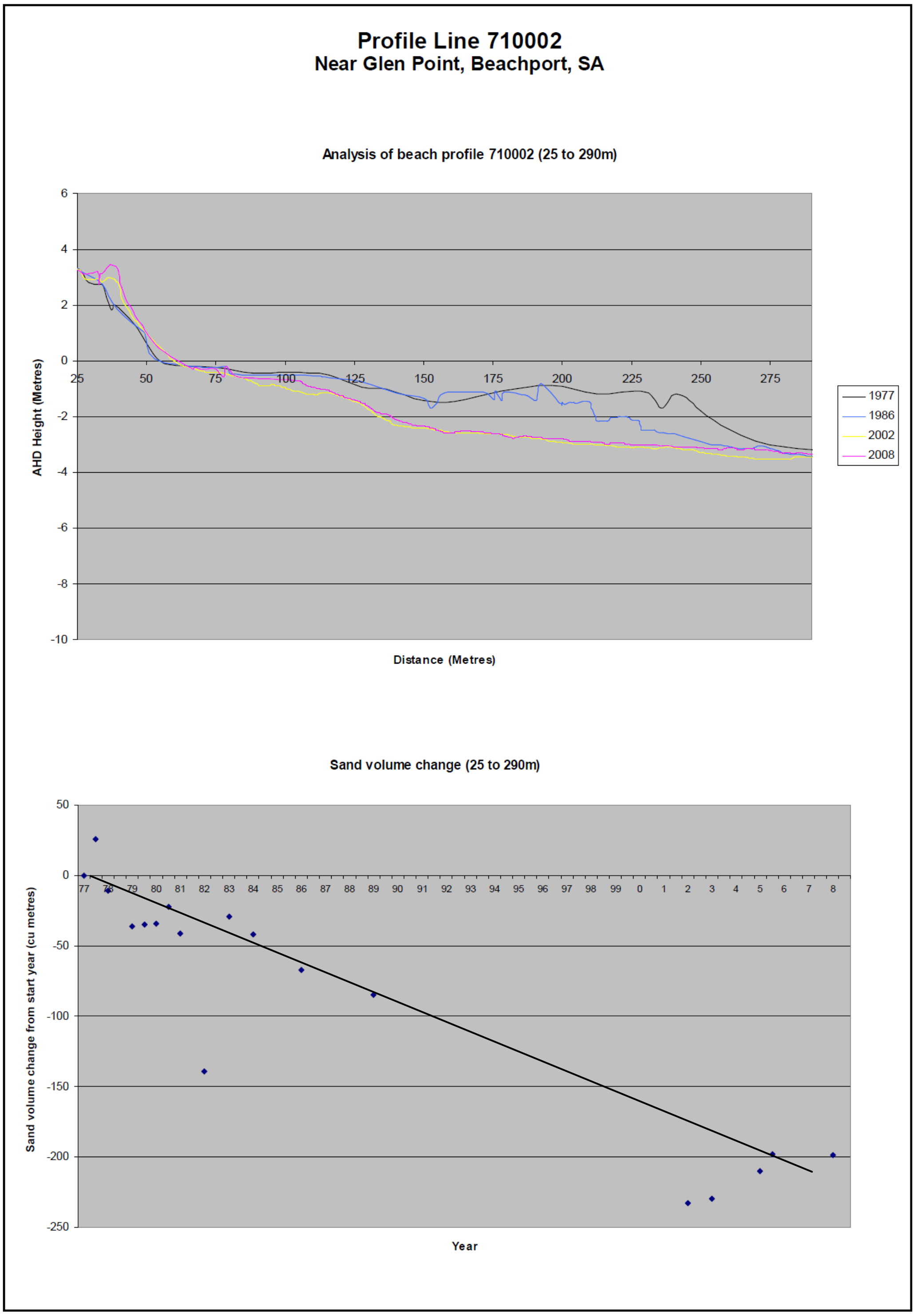


Figure 25 – DEWNR analysis of historical beach profiles adjacent to Glen Point (Beachport), indicating long term loss of profile volume from the nearshore area (trend line added)

3 ASSESSMENT OF EXISTING COASTAL STRUCTURES

3.1 Introduction

This section of the report presents an overview of the existing coastal structures at Beachport and Southend, based on a site inspection carried out in June 2014. Each coastal structure is described in terms of its stability, structural integrity and condition.

In particular, this section:

1. Describes the features of the coastal structures as seen during the site visit, including slope, armour materials, size and type, condition including stability and structural integrity of each structure.
2. Describes the visible impact of each structure on the surrounding beach and on beach amenity, as gleaned from the site reconnaissance.
3. Describes visual observations relating to the coastal processes within the embayment and their interaction with the erosion protection structures.

The site inspection was undertaken from the public area of the beach and a detailed photographic record was captured.

It should be noted that a number of renewal or upgrade works were carried out between June 2014 and March 2015. While these works were inspected in March 2015, they were not described in detail. These works include:

- extension of Groyne 5A (boat ramp),
- works on the seawall in front of the jetty,
- works in placing rock protection on beaches 8 and 9 and
- works to repair Groynes 8, 9 and 10.

The effectiveness of the erosion protection structures against storm events of varying magnitude has been assessed quantitatively in Section 4 with the aid of numerical modelling documented in Appendix 2.

For the evaluation of the coastal zone management works, the works at Beachport and Southend have been considered separately.

The condition of each structure as gleaned from the site inspection was defined as follows:

- **Good condition** – structure armour intact, with little or no displacement of armour units. Little or no visible slumping of the structure crest. No visible deformation of structure profile. No gaps observed between structure and retained material. No settlement or cracking of the area immediately behind the structure and no visible loss of retained material through the structure's armour.

- **Fair condition** – Structure has suffered some minor damage but is still providing some degree of erosion protection. Some deformation of the structure's profile or minor weathering of individual armour units but no displacement of individual units from the structure. No loss of retained material through the structure and no large gaps in the structure's armour. No excessive slumping of the structure's crest or toe.
- **Poor condition** – Structure has suffered extensive damage or is not providing erosion protection effectively. Structure may have suffered slumping, displacement of some armour units from the structure's face, erosion behind the structure or some loss of retained material through the structure. Structural properties are not appropriate for the coastal engineering conditions experienced at the structure based on visual assessment.
- **Failed condition** – Structure is not providing any erosion protection. Structure has largely collapsed with armour units displaced and retained material having washed through the structure. Erosion of the coastline behind the structure is continuing or has resumed.

3.1.1 Documented Structure Features

Each of the identified coastal structures was inspected in detail during the site visit. Generally, the coastal structures documented were rubble mound groynes or flexible sloping revetments or seawalls, comprising rock armour. Seawalls are structures designed to prevent or alleviate overtopping or flooding of the land and the structures behind, due to storm surges and waves. They also work to reduce coastal erosion and hold the coastline in place. Similar to seawalls, revetments are a more specific structural type with a similar purpose of protecting the shoreline from wave-induced erosion by placing an erosion resistant cover directly on an existing slope or embankment (USACE, 2002).

The main features of each erosion protection structure documented during the site visit included:

- Type of structure – flexible revetment, or rubble-mound groyne
- Crest level or height of crest above beach berm
- Slope of structure face (measured on-site and referenced back to the available survey information)
- Armour size, condition, grading
- Toe condition (where toe was visible)
- Apparent interaction of structure with adjacent shoreline and coastal processes (i.e. the apparent impact of the structure on the adjacent shoreline) as gleaned from visual observations
- Apparent risk to public as a result of observed instability of rock armour and other materials comprising the erosion protection structures
- Impact of the structure on the local beach amenity.

3.1.2 Failure Mechanisms

The term “failure” may imply a total or partial collapse of a structure. However, the term “failure” in the context of coastal engineering structures and their design performance, is defined by USACE (2002) as “*Damage that results in structure performance and functionality below the minimum anticipated by design*”. Design failure occurs when either the structure as a whole, including its foundation, or individual structure components cannot withstand load conditions within the design criteria. Design failure occurs also when the structure does not perform as anticipated.

Several modes of “failure” have been documented for coastal structures in general, with some of these mechanisms observed in the Rivoli Bay embayment during the site visit.

Each of the groyne and revetment structures at each precinct was inspected on 24-25 June 2014 and the following observations were made.

3.2 Conditions during the field inspection

During the course of the field inspection, a deep low pressure system and associated cold front were affecting the study area, with an offshore *significant* wave height (average of the highest one-third of waves in the record) measured at 8.9 m and maximum wave height exceeding 14 m at the Cape du Couedic Waverider buoy operated by the Bureau of Meteorology. This event resulted in very large waves breaking directly onto the coastal structures at Beachport, as well as damage to the vertical seawall near the jetty, undermining of the concrete walkway along the dune north of the jetty, severe beach erosion along the foreshore, wave overtopping onto Beach Road and damage to the jetty itself. Severe coastal erosion had also occurred at Southend, north of the outlet to Lake Frome, as a result of this storm event. This event reached its peak intensity at approximately 10 am on 24 June 2014. Conditions had improved by 25 June, providing an opportunity to assess the coastal structures in more detail.

The detailed inspection of the coastal structures at Beachport and Southend is documented in Appendix 1. The findings from this inspection are summarised below.

3.3 Summary

In general, the groynes and revetments at Beachport and Southend were in poor condition and do not meet contemporary engineering standards for design and construction. There was considerable damage to the groynes caused by wave action, with dislodged and slumped primary armour layers and erosion of the clay cores of several of these groynes.

The groynes at Beachport, while in poor condition, were generally found to be effective in stabilising the shoreline. The majority of the groynes were bypassing sand continuing to be supplied to the area around Glen Point. Localised erosion impacts were evident downdrift of some of the groynes impacting the beach dunes, timber walkways and, in some areas, threatening to outflank the groynes.

The main timber seawall at Beachport was subject to severe wave overtopping onto Beach Road, and wave reflections from this seawall had prevented the formation of a usable beach in front of the seawall in the vicinity of the jetty. This wave overtopping caused severe damage to the beachfront promenade during the site inspection on 24 June, with undermining of the concrete pathway.

Sand bypassed the groynes at the outlet to Lake George, with considerable quantities of sand being carried into the Lake George channel by wave action and tidal currents.

At Southend, the groynes at the outlet to Lake Frome were in particularly poor condition. These groynes have suffered from erosion, loss of armour with miscellaneous rubble used to repair the groynes in places. Severe erosion has occurred in the dunes on the northern side of the outlet, threatening to undermine development. The three groynes north of the Lake Frome outlet have not been effective in stabilising the dune, with little buildup of sand on the south sides of these groynes, indicating that littoral drift may have been rapidly removed because the groynes are too short to trap sand effectively. It is considered that the groynes at the outlet to Lake Frome are considerably reducing the supply of sand to the section of foreshore north of the lake outlet, as sand is not able to bypass the lake outlet. During the community consultation for the project, comments were received from the community relating to the management of Southend, in particular opposing the removal of the outlet groynes at Lake Frome due to the risk of erosion on the beach to the west. In addition, comments were received about the effectiveness of mechanical placement of sand from the western side of the outlet to the eastern side, this having been tried previously and resulting in a loss of this sand through offshore sediment transport.

While the rock revetment adjacent to the Southend jetty was in good condition, severe wave overtopping was observed into the carpark adjacent to the jetty and boat ramp at Southend during the site inspection. Ad hoc rubble was observed to have been placed at the southern end of the revetment, which would not be effective erosion protection.

4 QUANTITATIVE ASSESSMENT

4.1 Methodology

This section presents the derivation of the estimated wave conditions in the nearshore area of the Beachport and Southend and the assessment of the structural stability of the existing works.

The nearshore design wave estimates were based on the transformation of offshore wave conditions through numerical modelling to the project site. The principal aim of establishing the extreme wave conditions is to provide initial estimates of design wave conditions for the existing coastal structures along the Beachport and Southend foreshore, to enable the structural stability of the various structures to be assessed. The offshore boundary conditions were based on published extreme wave data collected by offshore wave buoys and metocean hindcasts.

It should be noted that numerical models as applied herein are a tool that can provide estimates of the physical response of the coastal system, based on its calibration and capacity to replicate measured phenomena. The models used herein provide an estimate of the design nearshore water levels and wave heights that would apply at the various coastal structures and are thus applicable for assessment of their structural stability and conceptual design of an upgrade of the structures. It should be noted that there can be departures between a model output and the actual physical response, as not all processes are able to be replicated by numerical approximations. However, the model results provide the best available estimates of the design parameters for assessment of the performance of the coastal structures.

Nearshore wave conditions were derived using a 3rd generation 2D(H) spectral wave model (SWAN). As the spectral wave model is not able to model wave induced setup, a 1D roller model, which is able to model wave induced setup (SBEACH), was used to transform the nearshore wave conditions to the shore. The use of the SWAN and SBEACH models together provides a three dimensional solution algorithm for wave transformation across the surf zone to shore and gives a far better result than that obtained from a 1D solution alone, such as GENESIS or LITPAC.

Armour stability for the existing structures was assessed using the Hudson equations with the derived nearshore design wave height for the 1 year, 10 year and 100 year ARI wave events.

Wave runup onto the structures was based on algorithms provided in the Shore Protection Manual (CERC 1984), and wave overtopping was assessed using the algorithms of Owen (1980).

4.2 Wave Modelling

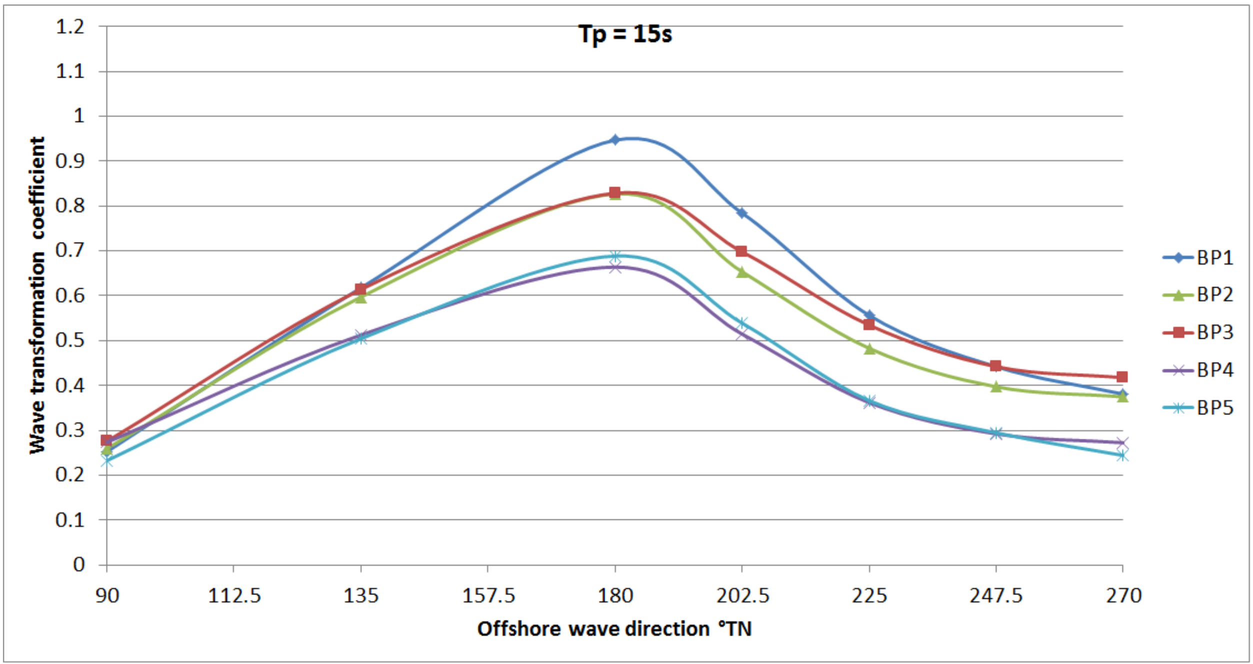
Both SWAN and SBEACH models were used for the wave transformation modelling. The SWAN model (version 40.85) (Delft University of Technology, 2011) was used to derive transformed nearshore wave heights for the range of offshore wave directions and periods comprising the long term wave climate to establish suitable input boundary conditions for the SBEACH surfzone wave transformation model. The SBEACH model was utilised to describe in detail the surfzone wave transformation processes for the determination of nearshore wave setup water levels and wave heights at the structures, necessary for assessment of their structural stability.

The SWAN and SBEACH wave transformation modelling is presented in detail in Appendix 2.

4.2.1 Model Results

The SWAN model was run for an offshore wave height of 1 m (to obtain wave height coefficients), for all wave directions ranging from south to north-east. It was found that the largest wave height coefficients occurred when the offshore wave direction was from the west at Southend and from the south at Beachport. WRL (2013) found that the highest significant wave heights along the coastline adjacent to western Victoria occur from the west-southwest directions (225 – 270°). WRL (2012) adopted a 10 year ARI significant wave height of 5.9 m for this section of coast for directions between east and south compared with a much higher significant wave height of 9.5 m for waves from the west. Despite waves from the south being lower offshore than waves from the west, due to wave refraction around Penguin Island, southerly waves result in the highest waves at the Beachport shoreline. At Southend, waves from the west were found to result in the highest waves at the shoreline, with southerly waves being subject to strong wave refraction around Cape Buffon.

Wave transformation coefficients for a peak wave period of 15 s at selected nearshore locations in 3 m water depth at Beachport and Southend for the range of offshore wave directions modelled are illustrated in Figure 26. It can be seen from these plots that at Beachport, the peak wave energy arrives at the foreshore when offshore wave direction is from the south, and at Southend, peak wave energy arrives at the foreshore when offshore wave direction is from the west.



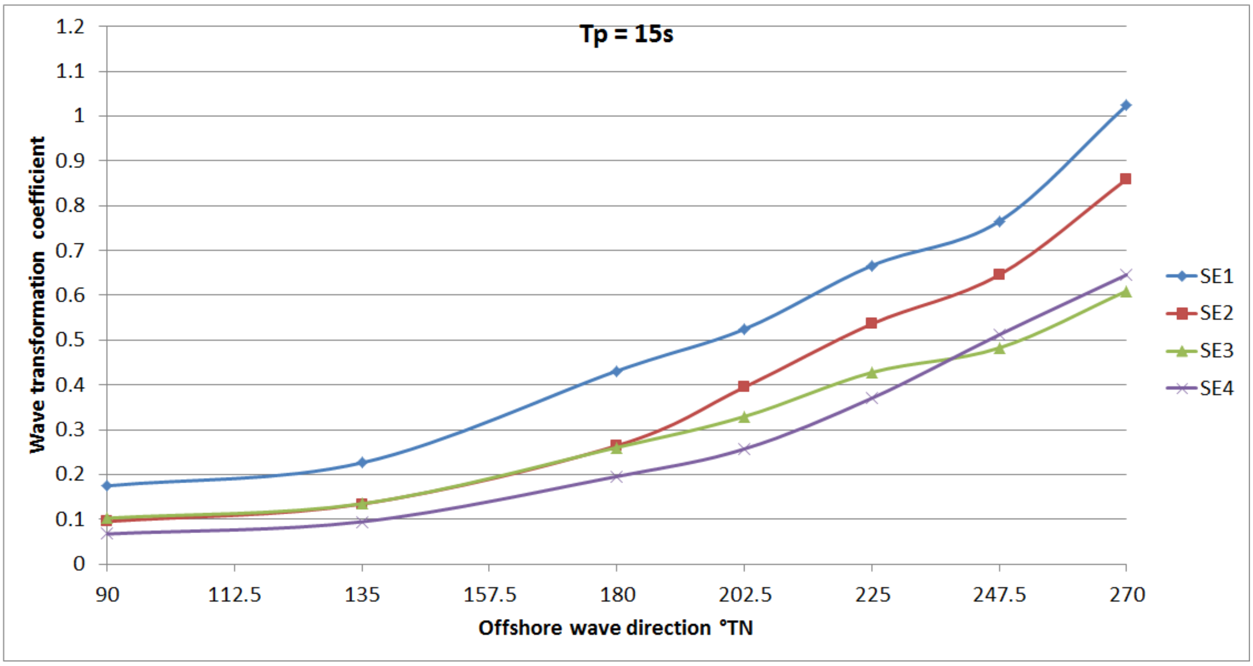


Figure 26 – Nearshore wave transformation coefficients at five locations in Beachport (top) and four locations in Southend (bottom) vs. offshore wave direction

4.3 SBEACH Modelling

To establish the wave conditions at the various groynes and revetments, the SBEACH model (Rosati *et al.*, 1993) has been used. SBEACH simulates beach profile change, including the formation and movement of major morphologic features such as longshore bars, troughs, and berms, under varying storm waves and water levels. The model is empirically based and was developed originally from a large data set of net cross-shore sand transport rates and beach profile change observed in large tanks. Along with beach profile changes SBEACH is able to simulate depth induced wave breaking, shoaling, wave generation due to wind and wave induced setup.

There are no site wave data within the study area with which to validate the program. However, the SBEACH algorithms have been validated for the Australian eastern seaboard at numerous sites (Carley, 1992; Carley *et al*., 1998).

The SBEACH modelling is described in detail in Appendix 2.

4.3.1 Results

As the wave conditions are depth limited, the design wave for the structures would be the largest wave that breaks on the structure. This corresponds to the largest wave that is half a wavelength seaward of the seawall or groyne.

The SBEACH model allows the determination of nearshore water level conditions to be estimated, including the effects of wave setup. Based on these water levels, a maximum breaking wave height at each of the groynes was able to be estimated. As the nearshore wave height is controlled by the water depth, there is an increase in wave height in front of the structures with the rarer events, which is due to the influence of wave setup. Given the relatively small water depths involved, it is clear that climate change sea level rise has the potential to increase significantly the size of the incident breaking wave heights. This is examined later in the report.

The most important parameters for assessing the stability of the groynes and revetments are the breaking wave height in front of the structure, the scour level at the structure toe and the water level at the structure. These parameters determine the effectiveness of the existing works, such as the stability of the existing rock armour and the probability that the structures would be overtopped. The results from the SBEACH model provide the variation in these design parameters along the entire foreshore at Beachport and Southend. Generally:

- The present day maximum wave height approaching the structures at Beachport ranges from 0.9 – 2.0 m for the 1 year ARI, 1.5 – 2.4 m for the 10 year ARI and 1.9 – 2.7 m for the 100 year ARI storm events. These wave heights do not include shoaling – the breaking wave height (H_b) at the structures would be larger than these due to shoaling, which is derived separately.

- The largest wave heights along the Beachport foreshore occur immediately adjacent to the Beachport jetty. This is due to the deeper water available in this location immediately seaward of the vertical seawall at Beachport.

- The largest wave heights along the Southend foreshore occur adjacent to the Southend jetty. This is due to the more exposed nature of this location where less wave refraction occurs than at the Southend beach foreshore.

- The maximum water levels (including the effects of wave setup at the foreshore) vary from 2.3 – 3.7 m AHD for the 1 year ARI, 2.6 – 4.4 m for the 10 year ARI event and 3.0 – 5.1 m for the 100 year ARI event. Wave setup at the foreshore is significant in the extreme events due to the shallow nature of Rivoli Bay and extensive wave breaking that occurs during these events.

The largest wave heights occur where the coastal structures are located furthest seaward along the beach profile (i.e. at the Beachport jetty), due to the profile being deeper at these locations. The variation in wave height along the foreshore is a function of the nearshore water depth at a point half a wavelength in front of the foreshore structures. Water level variations between profiles are a function of the wave setup calculated by SBEACH at the measurement point half a wavelength in front of the structures.

The maximum wave heights obtained from SBEACH (not including shoaling) were found to be around 2.5 m – with shoaling, this would result in a breaking wave height at the structure of around 3.7 m for a 100 year ARI event.

4.4 Hydraulic Armour Stability of Rock groynes and revetments

Primarily there are two types of coastal protection structures in Beachport and Southend:

- Rock revetments (immediately adjacent to the boatramps at Beachport and Southend), and
- Rock groynes.

The results of the wave modelling have been used to check the hydraulic stability of the existing coastal structures in the Rivoli Bay embayment against wave attack, for the 1 year, 10 year and 100 year ARI storm events.

4.4.1 Rock Armour Stability

The stability of the primary armour against wave attack has been assessed using the Hudson equation. Another commonly used formulation for rock armour sizing is the Van der Meer equation. This equation, however, is only applicable for deep water conditions (i.e. where the depth in front of the structure is greater than three times the significant wave height in front of the structure, CIRIA, CUR, CETMEF 2007). The conditions at the groynes and revetments are shallow water conditions and the structures will be subject to breaking waves. Hence, the van der Meer formulae are inappropriate for use in assessing the structure stability and the Hudson formula has been used for the calculation of the requisite armour size for the structure.

The Hudson equation is given by:

$$W = \frac{w_r H^3}{K_D (S_r - 1)^3 \cot\theta}$$

where:

W	=	Weight of an individual armour unit in the primary cover layer, kg;
w_r	=	unit saturated surface dry density, kg/m^3
H	=	design wave height at the structure site, m (corresponding to H_{max})
S_r	=	specific gravity of armour unit, relative to the water density at the structure
θ	=	angle of the structure slope, measured in degrees
K_D	=	stability coefficient which depends primarily on the shape of the armour units, roughness of the armour unit surface, sharpness of edges and the degree of interlocking achieved during placement

The above formula is based on comprehensive physical model investigations at the U.S. Army Corps of Engineers.

The variable w_r depends on the properties of the available rock. A flatter slope or higher stability coefficient (K_D) value leads to a decrease in required armour stone weight, W.

Armour units that consist of rough quarried stone will have a higher K_D value than smooth, rounded armour stones. A higher K_D value can be achieved by special placement of the armour stones to achieve a high degree of interlocking. Random placement of the stones leads to a lower value of K_D, which could lead to the required armour stone size W exceeding that available.

Incorporated within the K_D value are variables such as the angle of incidence of wave attack, size and porosity of the underlayer material, revetment crest width and the extent of the revetment slope below the still water level. Table 1 gives recommended values of K_D to use for different situations (after Coastal Engineering Research Center CERC, 1984).

Table 1 – K_D values for Determining Quarrystone Weight*

Armour Units (Quarrystone)	Number of layers 'n'	Placement	Slope Cotangent	Structure Trunk		Structure Head	
				Breaking Wave	Non-breaking Wave	Breaking Wave	Non-breaking Wave
Smooth rounded	2	Random	1.5 – 3.0	1.2	2.4	1.1	1.9
Smooth rounded	>3	Random		1.6	3.2	1.4	2.3
Rough Angular	1	Random			2.9		2.3
Rough Angular	2	Random	1.5	2.0	4.0	1.9	3.2
			2.0			1.6	2.8
			3.0			1.3	2.3
Rough Angular	>3	Random		2.2	4.5	2.1	4.2
Rough Angular	2	Special		5.8	7.0	5.3	6.4
Parallelpiped	2	Special		7.0 – 20.0	8.5 – 24.0		
Graded Angular		Random		2.2	2.5		

*After CERC, 1984

The results from the Hudson analysis assume that no damage to the profile is allowed (static design). This means that there is no difference in the structure cross-section before and after a storm. If 0 – 5% of the armour stones are displaced between the crest and a level of one wave height below still water, this corresponds to "no damage" according to the Hudson formulation and would be acceptable for design (CIRIA, CUR, CETMEF 2007).

From Table 1, a revetment consisting of two layers of rough angular armour stones randomly placed and subject to breaking waves corresponds to a K_D value of **2.0**. This value has been adopted for the analysis of the rock revetment and groyne structures within Rivoli Bay.

To calculate the required stone diameter from the weight, it has been assumed that the bulk density of the rock boulders in the revetments within the Rivoli Bay embayment is 2300 kg/m^3 (pers. comm. Wattle Range Council). The assumed density is based on the specific gravity of the locally-sourced limestone/sandstone rock typically used for construction of the coastal structures within Rivoli Bay as measured by staff from Wattle Range Council.

The results of the Hudson analysis are provided in Table 2 for Beachport and Table 3 for Southend, which shows the calculated median primary armour diameter needed for hydraulic stability against wave attack for the 1 year ARI storm event. These diameters are compared with the actual median armourstone diameters determined from the results of the site inspection.

It should be noted that this analysis has not taken into account other factors of importance in design of the rock boulder structures such as crest level, toe level, armour grading, presence of a filter layer or porosity into account. It can be seen that, for all the rock revetment and groyne structures, that the rock armour would be hydraulically unstable for wave heights at the structure resulting from an eroded beach profile, for storm events greater than or equal to a 1 year ARI.

Table 2 – Hudson Analysis for coastal structures at Beachport

Structure No.	SBEACH Profile	Average Slope 1V:XH	Median Boulder diameter (mm)	H_b[1] 1yr	Hudson W_{50} 1yr (kg)	Hudson D_{50}[2] 1yr (mm)	Estimated Design standard
G2 - G5	BP1	2	1000	1.4	900	800	~1yr ARI
Revetment near boat ramp	BP2	2	1000	1.7	1500	1000	~1yr ARI
G8 and jetty area	BP3	2	1000	2.2	3100	1200	< 1yr ARI
G9	BP4	2	1000	2.4	4000	1300	< 1yr ARI
G10 - G12	BP5	2	1000	2.2	3200	1200	< 1yr ARI

Table 3 – Hudson Analysis for coastal structures at Southend

Structure No.	SBEACH Profile	Average Slope 1V:XH	Median Boulder diameter (mm)	H_b[1] 1yr	Hudson W_{50} 1yr (kg)	Hudson D_{50}[2] 1yr (mm)	Estimated Design standard
Revetments near boat ramp	SE1	2	1000	2.3	3800	1300	<1yr ARI
Groynes at Lake Frome outlet	SE2	2	1000	1.8	1800	1000	~1yr ARI
Groyne immediately north-east of Lake Frome outlet	SE3	2	1000	1.6	1200	900	~ 1yr ARI
Two northernmost groynes	SE4	2	1000	2.3	3700	1300	< 1yr ARI

1. Breaking wave height is calculated using linear wave theory at a point approximately 10 m in front of the structure (i.e. equal to the plunge width of the wave), for the water depth resulting from scour and wave setup determined for the 1 year ARI. The breaking wave height includes a calculated factor for wave shoaling. The breaking wave height at the toe of the structure will be reached for offshore deepwater wave heights much lower than the 1 year ARI.

2. Required median diameters for rock armour for the structures have been derived assuming a bulk density of 2300 kg/m^3 for the locally available rock.

The maximum breaking wave height at the structure toe is dependent on the scour level and the local water level which is influenced by wave, wind and barometric setup. The SBEACH model has predicted a scour level at the structure toe for the 1 year ARI event which would allow the maximum breaking wave heights shown in Table 2 and Table 3 to reach the structures. Even under median offshore wave heights (of around 1.5 m) in scoured conditions, maximum breaking wave heights at the structure can be sufficient to cause some damage to the existing armour layers according to the Hudson analysis. The interpretation of these results is that the existing structures currently meet approximately a 1 year ARI design standard, if the beach is in an eroded or scoured state and if no damage is permitted to occur to the structures.

If greater levels of damage were considered, CERC (1984) outlines the equivalent wave height at the structure for use in the Hudson analysis that would result in a particular level of damage to the cover layer. The results of the damage analysis indicate that in a 10 year ARI event with an eroded beach profile, the groynes would suffer around 30% - 40% damage to the cover layer. The groynes along Southend east of Lake Frome outlet are most exposed in a 10 year ARI storm event but were also observed to be the most robustly built of the groynes and were observed to be in good condition in the field.

The level of observed damage to most of the groyne structures is in accord with this assessment, with approximately 30 – 40% of the primary armour of the structure having been dislodged from the groynes along the Beachport foreshore. As the groynes have been in place for approximately 40 – 50 years, they will have been exposed to several, if not many, storm events over this time.

4.4.2 Temporary Geotextile Container Revetment – Beachport

The hydraulic stability of geotextile container revetments has been examined by Coghlan *et al.* (2009) through a series of physical model tests. The Beachport geotextile container revetment was installed as a temporary emergency protection measure and comprises 0.75 m^3 geotextile units. Coghlan *et al.* (2009) found the significant wave height at the structure which would cause *initial damage* (defined as 0 – 2% damage or displacement of the individual geotextile units from the face of the structure). For a geotextile revetment, as opposed to a rock revetment, displacement of individual units from the structure would lead to a more rapid collapse of the structure, as the geotextile units are completely removed from the structure face, leading to exposure of the underlying layers. In contrast, larger levels of damage to a rock revetment may be acceptable, because a rock revetment can generally accommodate re-shaping of the structure face, while still providing effective erosion protection. Damage in the context of a rock revetment structure means that individual rock armour units can move around on the structure face (but are not necessarily removed from the structure).

For a spectral peak wave period of 12 seconds, the significant wave height at the structure that would cause initial damage is around 1.3 m (for a structure slope of 1V:1.5H, Figure 27).

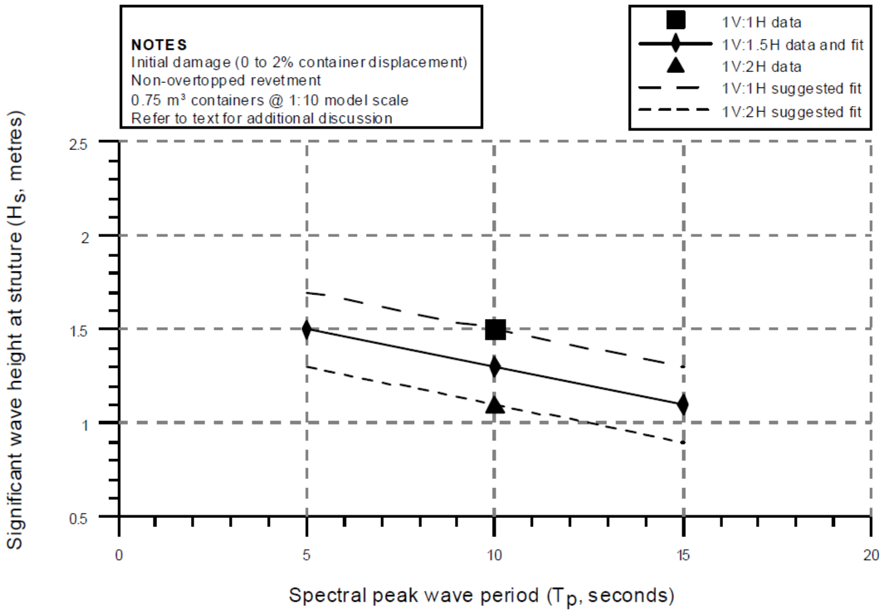


Figure 27 – Hydraulic stability of geotextile container revetments (Coghlan *et al.*, 2009)

From Figure 27, the wave height threshold for initial damage would be exceeded for all storm events greater than or equal to the 1 year ARI event, for conditions where the beach is eroded and the revetment is exposed to direct wave attack.

Similarly, should the beach berm in front of the structure erode away in a storm, the structure would have a low Factor of Safety against geotechnical slip failure, due to the steepness of the structure slope and low angle of friction between the geotextile bags, the underlying geotextile underlay and the sand slope. The observed structure slope of 1V:2H is steeper than the internal friction angle between two geotextile surfaces of 20° (Nielsen & Mostyn, 2011). This means that, once the sand in front of the structure is scoured away in a large storm, the structure would be at risk of suffering from slip failure.

The geotextile sand bag structure has been found to be incompetent for a storm event greater than 1 year ARI from both hydraulic and geotechnical stability considerations, should the frontal dune erode away (i.e. based on an eroded profile). For the conditions experienced at Beachport, therefore, this type of structure is suitable only as a temporary protection measure.

4.4.3 Summary

For the structures at Beachport and Southend, generally, when the beach is in an eroded state, the primary armour layers are unstable hydraulically against direct wave attack for all storm events greater than a 1 year ARI event. This means that some damage to the structure armour would be expected. In a 10 year ARI event, some parts of the structures could expect to incur 30 – 40% damage to the primary armour layer. This represents failure of the structure.

It should be noted, however, that other factors also are important in determining the robustness of a particular structure, including crest height, toe level, armour grading, presence of a filter layer, armour and structure porosity.

4.5 Layout of Groyne Schema

The spacing between groynes should equal two to three times the groyne length from the berm crest to the seaward end (CERC, 1984). For Beachport south of the boat ramp, the groynes generally conform to this rule. However, in the vicinity of the jetty, the ratio between groyne spacing and groyne length is around 7. In this location, the groynes are either too short or their spacing is too large, or both, suggesting that an additional groyne would be of benefit. North of the jetty, the ratio between groyne spacing and groyne length is around four to five, which suggests here also that the groynes are too short and/or there should be more of them.

Given the very shallow bathymetry of the Bay, sand can be transported by wave-driven longshore currents to levels of at least -3 m AHD. At these depths, the active littoral zone extends up to 200 m offshore of the beach. As the groynes extend to levels less than 1 m below AHD, they are subject to bypassing, indicating that they may not be long enough to compartmentalise the beach effectively. The groyne lengths and spacings relative to the bathymetry at Beachport are illustrated in Figure 28.

At Southend, the ratio between groyne spacing and groyne length is around three, with the groynes extending to around -1.5 m AHD. Bypassing is most pronounced around the westernmost groyne with sand being drawn into the Lake Frome channel, and a reduced sand supply reaches the beach to the east of the Lake Frome outlet.

It is considered that the appropriate seaward limit for the groynes would depend on a balance between capital cost and ongoing sand nourishment maintenance requirements. Despite the groynes having been designed and constructed in an ad-hoc manner over the past few decades, there has been an ongoing alongshore supply of sand from the west around Penguin Island and the groynes at Beachport have been effective in creating a stable plan-form profile along the foreshore.

DEWNR (2012) analysed beach bathymetry data at Beachport between 2002 and 2011 to determine changes in beach volumes over this time. These plots have shown ongoing accretion at the northernmost groyne compartments near the outlet to Lake George (indicating movement of sand northward over time from the groyne compartments to the south), with ongoing net losses for the groyne compartments fronting the jetty area, losses from up-drift of the boat ramp area and deposition down-drift of the boat ramp. These losses may be due to an increase in the mobilisation of sand as a result of loss of seagrass beds in the nearshore region over recent years. The DEWNR analysis is shown in Figure 29.

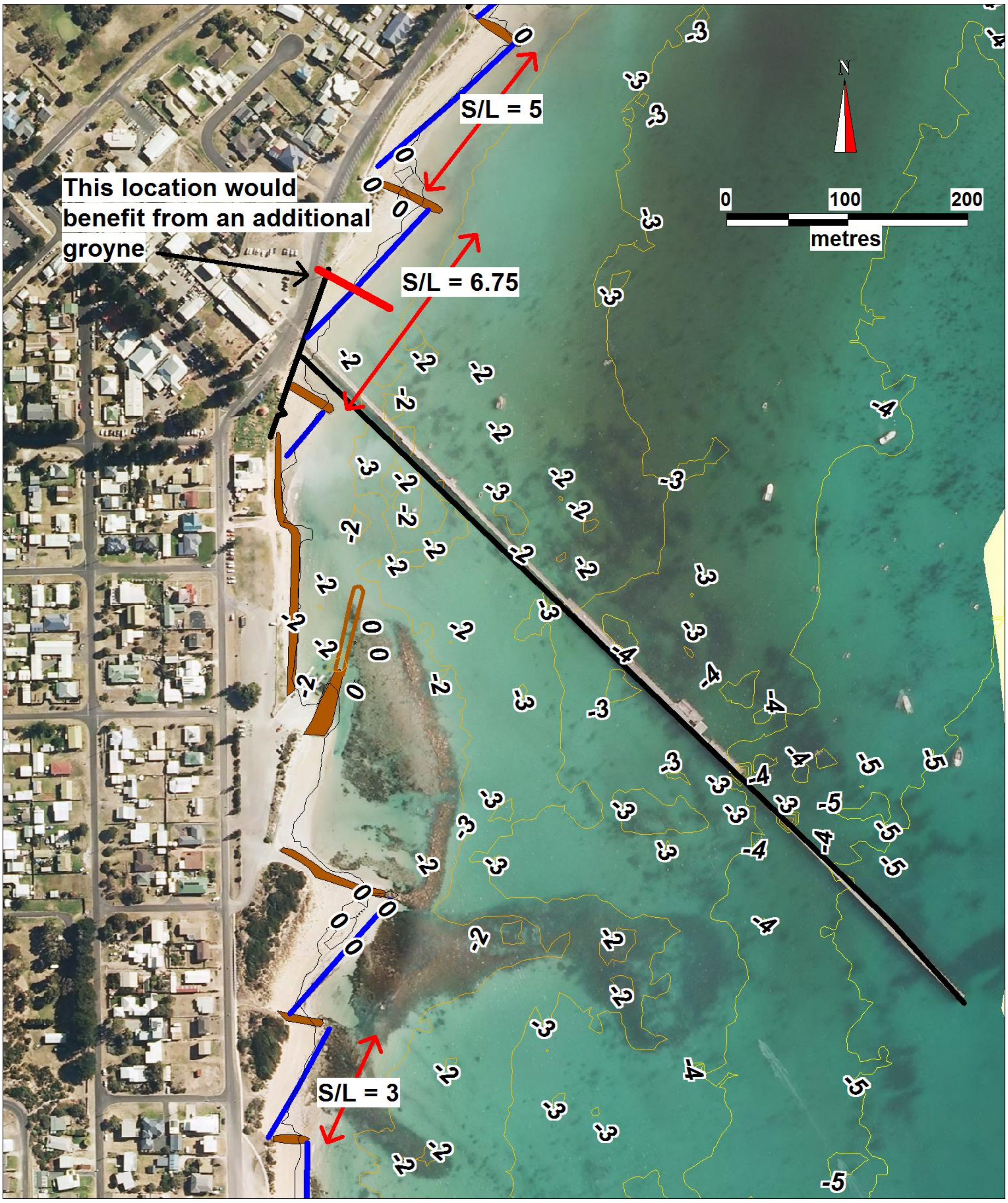


Figure 28 – Groyne spacing vs. length and bathymetry, Beachport

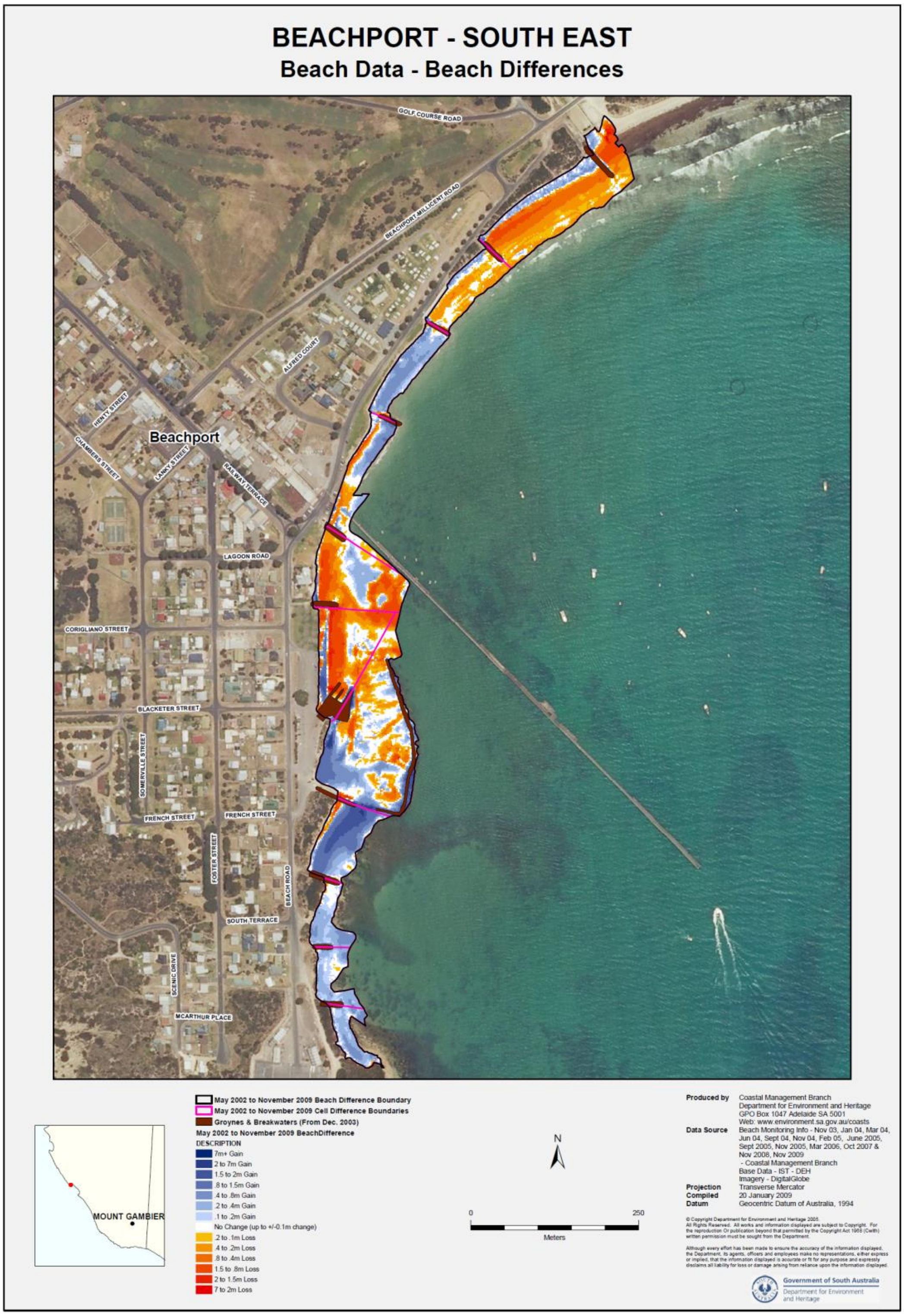


Figure 29 – Beach difference analysis, Beachport 2002 – 2011 (DEWNR, 2012)

4.6 Analysis of Historical Aerial Photography

Historical aerial photography of Rivoli Bay has been observed to ascertain the changes that have occurred along the Beachport and Southend foreshores over time. Aerial photographs dating between 1946 and 2013 were examined.

4.6.1 Beachport

4.6.1.1 1940s and 1950s

Aerial photography shows that the initial groynes along the Beachport foreshore were constructed sometime between 1951 and 1962. It was clear that, prior to construction of the groynes, beach erosion was already occurring along the Beachport foreshore in the vicinity of the jetty, especially in front of the vertical seawall where, at present, there is no beach. The beach width was very narrow at this location. Extensive seagrass beds were visible in these early photographs. South of the jetty, the beach was relatively uniform in planform shape and a vegetated dune was evident in this area on the seaward side of Beach Road in 1946 and 1951.

In the lower area of Lake George there was a flood tide delta observed in the early stages of formation, which was much smaller than the present day delta. North of the Beachport township large mobile dunes were encroaching onto the western foreshore of Lake George and threatening the Pool of Siloam. At the centre of Rivoli Bay mobile sand dunes were transgressing over the beach ridge plain area.

4.6.1.2 1960s

Three groynes south of the jetty and one north of the jetty were apparent in the 1962 aerial photography. However, the beach width had reduced considerably along the southern portion of the beach compared with the 1951 photography.

4.6.1.3 1970s

By 1975, a further five groynes were apparent along the beachfront south of Blacketer Street, constructed perhaps to stabilise the southern portion of the beach, which appeared to be eroding. Despite the presence of the groynes near to and north of the jetty, the beach from the jetty area northwards had failed to accrete and an erosion escarpment was visible in the 1975 photograph. By 1978, the groyne immediately north of the jetty had been removed and some sand appeared to have built up at the area around the jetty. The southern end of the beach had been stabilised but the beaches north of the jetty were devoid of sand.

4.6.1.4 1980s

By the mid 1980s, seagrass cover throughout the Beachport area had begun to decline and sand had built up to the south of the jetty area. However, the area around the jetty and further to the north still appeared to be depleted.

The large transgressive sand dunes visible in the earlier photography had been stabilised by the late 1980s.

4.6.1.5 1990s

By 1997, sand had begun to build up around the jetty area and in the two beach compartments immediately to the north.

4.6.1.6 2000s

In the 2004 aerial photograph, the new geotextile breakwater (designed to protect the remaining seagrass in the lee of the breakwater) and new boat ramp south of the jetty were visible and sand was visible in all the beach compartments (although the beach was narrow in the vicinity of the jetty). By 2013, the outline of the geotextile breakwater was still visible but it was evident that the breakwater had begun to subside and was no longer emergent. Sand had built up along the northern end of the beach, near the outlet to Lake George.

Shoreline changes along the beachfront foreshore are indicated in Figure 31, Figure 32 and Figure 34. Historical aerial photographs at Beachport are provided in Appendix 4.

4.6.1.7 Summary

The groynes have been largely successful in stabilising the southern end of the beach which had suffered from erosion in the 1960s. Since the 1980s, the northern end of the beach has undergone sand build up, indicating that a slug of sand was made available for sediment transport since that time and has moved north along the foreshore. This slug of sand may have been mobilised from the nearshore areas south of the boat ramp as a result of the ongoing decline in seagrass cover. That the beaches south of the boat ramp have remained stable since construction of the groynes may be a result of continuing sand supply from around Glen Point, or onshore movement of nearshore sand that had been recently mobilised since the 1980s due to the decline in seagrass cover.

Groynes have been constructed and removed over time – notably, a groyne immediately north of the boat ramp existed in the 1960’s and early 1970’s but was removed in the late 1970s. The area in front of the vertical seawall at Beachport has been subject to frequent loss of sand over the historical record.

Large scale works associated with the construction of the boat ramp and geotextile breakwater were undertaken in the early 2000s, with the shoreline response varying considerably as a result of these works. In particular, a large buildup of sand has occurred immediately south of the boat ramp, and the area in the vicinity of the jetty appears to have become depleted. Many of these changes were observed by stakeholders and communicated to the project team during the community consultation, together with suggestions as to the sediment transport pathways around the boat ramp and how this can best be managed. Large scale changes were observed within the lower basin of Lake George also, notably the build-up of the extensive flood tide delta between the 1940s and 1980s.

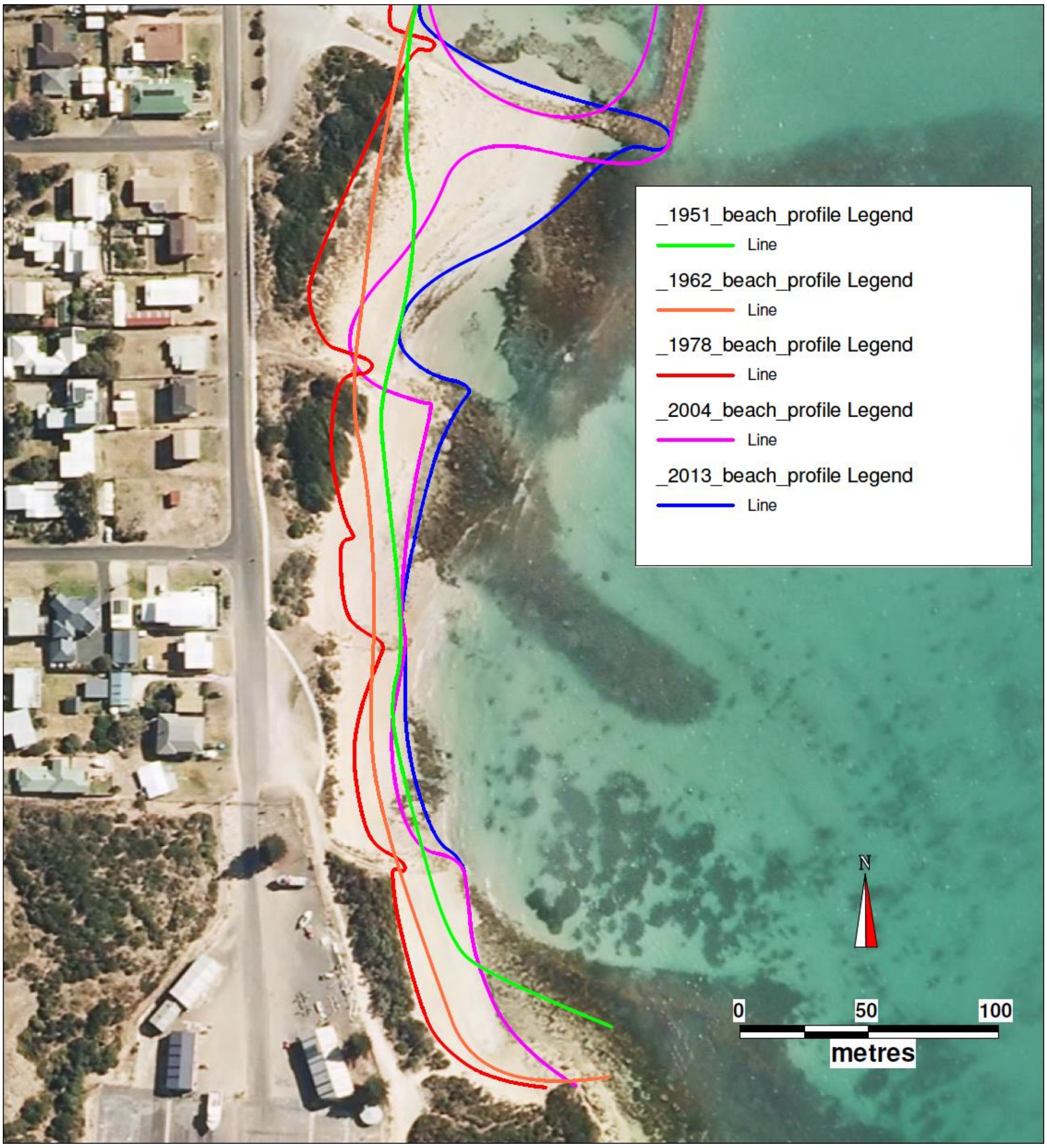


Figure 30 – Beachport analysis of aerial photography, Beachport (south)

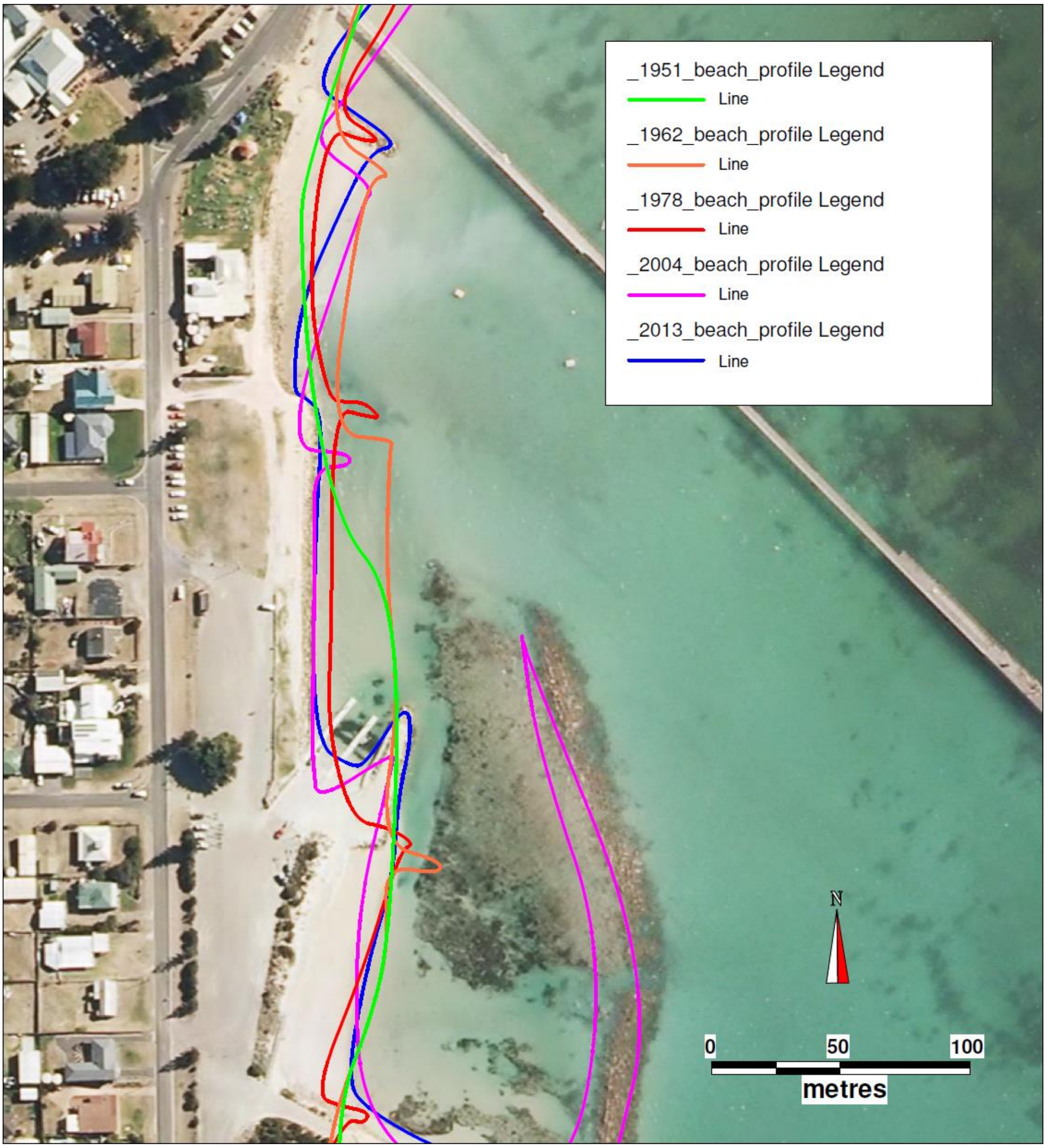


Figure 31 – Beachport aerial photography analysis – Beachport central

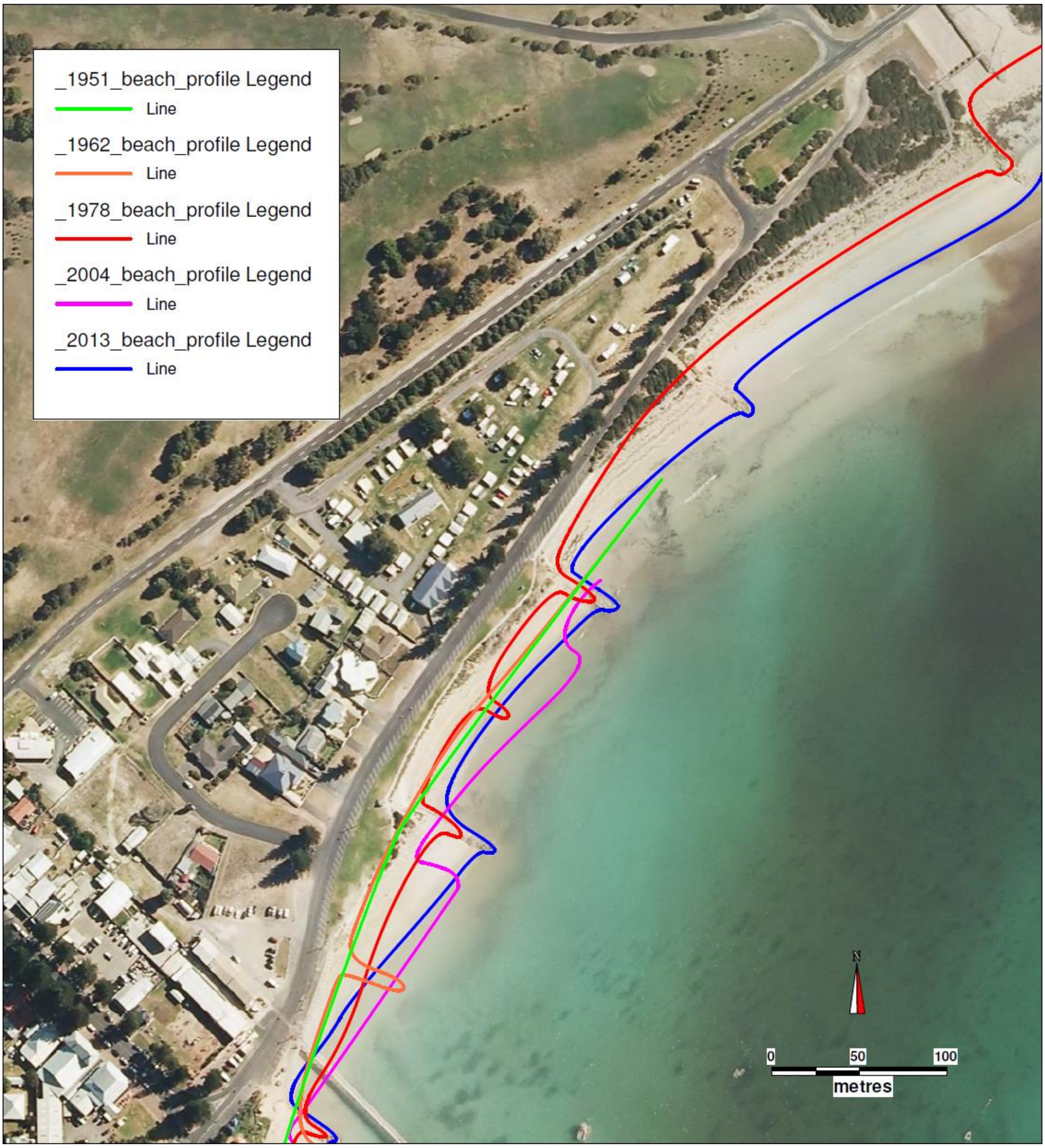


Figure 32 – Beachport aerial photography analysis – Beachport (north)

4.6.2 Southend

At Southend, in the 1946 aerial photography, the outlet to Lake Frome was untrained, and was observed to open out onto the beach. The present-day flow regulator and bridge were not visible in the 1946 photograph. There was no development at Southend at that time, and a large mobile wind-blown dune extended from Cape Buffon to the beach at Rivoli Bay. Seagrass beds were observed within the Bay in this area.

By 1978, development had commenced in the Southend area but there were no groynes along the beach. The beach was uniform in shape and the outlet to Lake Frome was closed over with sand.

Between 1982 and 1987, a single rock groyne had been constructed on the eastern side of the outlet to Lake Frome, but the beach plan form had not begun to change as a result of the construction of the groyne.

By 1992, a second groyne had been constructed on the western side of the outlet to Lake Frome and the beach to the east of the Lake outlet had begun to recede.

By 2000, three additional groynes had been constructed along the foreshore to the east of the Lake outlet, perhaps to stabilise the foreshore there, and the planform of the beach had begun to respond to the presence of the groynes.

During the community consultation comments from residents at Southend were received relating to the direction of net sediment transport at Southend around Cape Buffon, suggesting that large quantities of sand exit the bay around Cape Buffon during the winter storms. This view is supported by the 2000 aerial photography, which appears to show a plume of sediment laden water stretching from the outlet at Lake Frome toward Cape Buffon.

Comments were also received from the community opposing the removal of the outlet groynes at Lake Frome due to the risk of erosion on the beach to the west. Evidence from the aerial photography shows that the beach may well recede following removal of the Lake Frome groynes, as the shoreline was considerably closer to the roadway at Bridges Drive.

Historical aerial photography of Southend is shown in Figure 33.

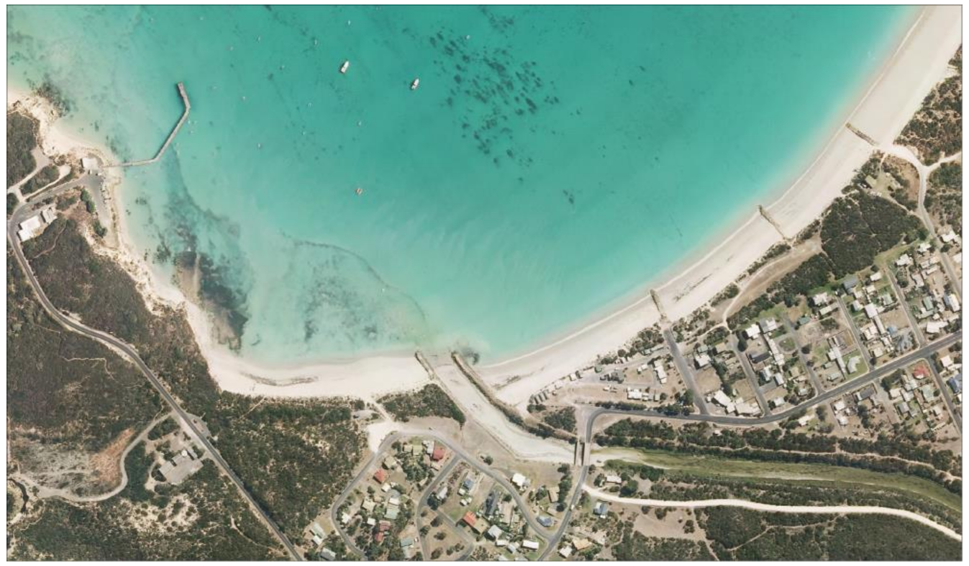
1946

1978

1987

1992

2000



2013

Figure 33 – Historical aerial photography at Southend (supplied by DEWNR)

4.7 Crest Levels of seawalls and frontal dunes

Wave inundation due to wave overtopping of the existing revetments, seawalls and frontal dunes at Beachport and Southend has been assessed using the available data on ground levels and results from the nearshore wave transformation modelling.

At Beachport, there is the potential for wave overtopping to occur over the vertical seawall and onto Beach Road near the jetty during storms, such as that which occurred on 24 June 2014. Overtopping of the dunes along the beachfront also is likely during storms, causing inundation of the land behind the beach.

At Southend, inundation of the boat ramp and jetty carpark occurs during storms and parts of the dune system experience coastal inundation due to wave runup.

4.7.1 Wave Runup and overtopping

Wave runup refers to the maximum vertical height above the still water level to which a wave would rise on a structure or beach dune. Wave overtopping refers to the flow of water caused by wave runup reaching the crest of the slope or seawall, measured in cubic metres per unit length of foreshore. Wave overtopping can be dangerous for pedestrian or vehicular traffic, as illustrated in Figure 34.

Estimates of the maximum wave runup onto the foreshore that could be expected from 1 year, 10 year and 100 year ARI waves, calculated using the SBEACH model results, were made using the software program Automated Coastal Engineering Software (ACES). The runup calculation was based on algorithms provided in the irregular wave runup on beaches formulation in ACES, which is based on Mase (1989). The wave runup is based on the local beach slope in the nearshore region, wave setup as calculated using SBEACH and significant wave height and period.

The calculated 2%, 10% wave runup levels (i.e., runup levels exceeded by 2% and 10% of waves in a particular storm) are shown Table 4. It can be seen that, at Beachport, the predicted runup levels exceed the local crest levels of the dunes. Therefore, for storm events greater than a 1 year ARI storm event, wave overtopping of the local foreshore could be expected to occur along most of the length of the Beachport foreshore. In some areas, this wave overtopping may extend onto the roadway behind the foreshore as witnessed during the storm event of 24 June 2014.

At Southend, wave overtopping of the local revetments into the boat ramp carpark may be expected in a design storm event, as well as overtopping of the dune area west of the entrance to Lake Frome. East of the Lake Frome entrance, waves would not be expected to overtop the dune crest in storm events up to the 100 year ARI.

Wave overtopping is one of the most common causes of revetment failure which is relevant for the revetments at the Southend carpark and Beachport boat ramp. If the crest is not sufficiently high or wide, waves overtopping the revetment can lead to erosion of the retained material on the landward side. As well as posing a hazard to pedestrians and vehicles, this can cause subsequent collapse of the top of the structure. Depending on the erodibility of the material immediately behind the structure and the landuse of the area protected, a small amount of overtopping volume may be acceptable in some circumstances. Figure 34 provides recommended allowable overtopping volumes under various situations.

Table 4 – Wave runup results from ACES for each section of beach, 1 yr, 10 yr, 100 yr ARI, m AHD

		1 Year ARI			10 Year ARI			100 Year ARI		
Cross section*	Crest level	2% Runup	10% Runup	Sig. Runup	2% Runup	10% Runup	Sig. Runup	2% Runup	10% Runup	Sig. Runup
BP1	3.5	4.77	4.57	4.18	5.36	5.15	4.75	5.82	5.61	5.21
BP2	4.4	7.24	6.83	5.99	7.73	7.32	6.48	8.35	7.92	7.05
BP3	4.0	5.40	5.13	4.60	5.91	5.64	5.11	6.70	6.41	5.83
BP4	3.0	5.05	4.78	4.24	5.50	5.23	4.69	5.89	5.62	5.08
BP5	4.1	4.67	4.41	3.91	5.05	4.79	4.29	5.63	5.36	4.81
SE1	2.0	5.74	5.43	4.83	6.27	5.96	5.36	7.07	6.74	6.09
SE2	3.0	5.03	4.87	4.57	5.69	5.53	5.23	6.32	6.16	5.86
SE3	5.2	4.21	4.04	3.72	4.68	4.51	4.19	5.17	5.00	4.66
SE4	7.4	4.98	4.65	3.99	5.73	5.36	4.62	6.27	5.87	5.08

*Cross section locations are mapped in Appendix 2. BP1 – BP5 are locations moving north along the beach at Beachport from Glen Point to the Lake George outlet. SE1 to SE4 are locations moving east from the Southend jetty to the easternmost groyne.

The duration of the wave overtopping depends upon how quickly the water can drain away from the crest area. As a rough indication, if the crest of the embankment is free draining, the overtopping would drain away relatively quickly (within seconds). Not every wave would result in overtopping. However, it may be possible to have two or more consecutive waves overtop the foreshore crest area – in this case, if the water has not drained away prior to arrival of the next wave, the overtopping depth and duration in the area landward of the crest of the foreshore would increase.

Several methods of estimating wave overtopping are available, including a neural network web-based model as described in the EurOtop manual (HR Wallingford 2007). However, all these methods typically provide, at best, an order of magnitude estimate of the overtopping volumes. Given that local foreshore crest levels are low in relation to the maximum wave runup levels (Table 4), wave overtopping volumes during storm events are likely to be significant and are likely to exceed levels which are dangerous for pedestrians and vehicular traffic and levels which would cause damage to paved promenades, particularly at Beach Road near the jetty area and at the boatramp carpark at Southend.

In the absence of physical model testing, overtopping for the various sections of foreshore at Beachport and Southend was estimated using the algorithms of Owen (1980), as described in the Hong Kong Port Works Manual (Government of Hong Kong, 2003).

Owen (1980) derived the following formula to describe overtopping (from Government of Hong Kong, 2003):

$$R_* = R_c / \left(T_m \left(g H_{1/3} \right)^{0.5} \right) \qquad (0.05 < R_* < 0.30)$$
$$Q_* = A \exp(-B R_* / r)$$
$$Q = Q_* T_m g H_{1/3}$$

where	R_c	=	Freeboard between still water level and crest of structure (m)
	$H_{1/3}$	=	Significant wave height at the toe of the structure (m)
	T_m	=	Mean wave period at the toe of the structure (s)
	r	=	Roughness coefficient
	g	=	Acceleration due to gravity (m/s^2)
	A,B	=	Empirical coefficients dependent on cross section
	Q	=	Mean overtopping discharge rate per metre run of seawall (m^3/s/m)
	Q_*	=	Dimensionless mean overtopping discharge
	R_*	=	Dimensionless freeboard

For a permeable crest, the overtopping can be reduced by a reduction factor which is a factor of the significant wave height and crest width, given below:

$$C_r = 3.06 \exp\left(-1.5 C_w / H_{1/3}\right)$$

where C_w = Crest width of the structure (m)

If $C_w / H_{1/3}$ is less than 0.75, C_r may be assumed as 1.

Using the above algorithms, overtopping discharge was estimated for the foreshore slopes at the various locations along the Beachport and Southend foreshore. The results of the overtopping estimation were that for the Beachport foreshore at Beach Road and the Southend jetty carpark, overtopping volumes would reach between 0.5 m^3/s/m and 1 m^3/s/m in a 100 year ARI storm event, which exceeds levels which are dangerous for pedestrians and vehicular traffic and levels which would cause damage to paved promenades according to Figure 34. Even in more frequent storm events such as the 1 year ARI storm event, overtopping volumes may be as high as 0.4 m^3/s/m at the Beachport foreshore at Beach Road and the Southend jetty carpark.

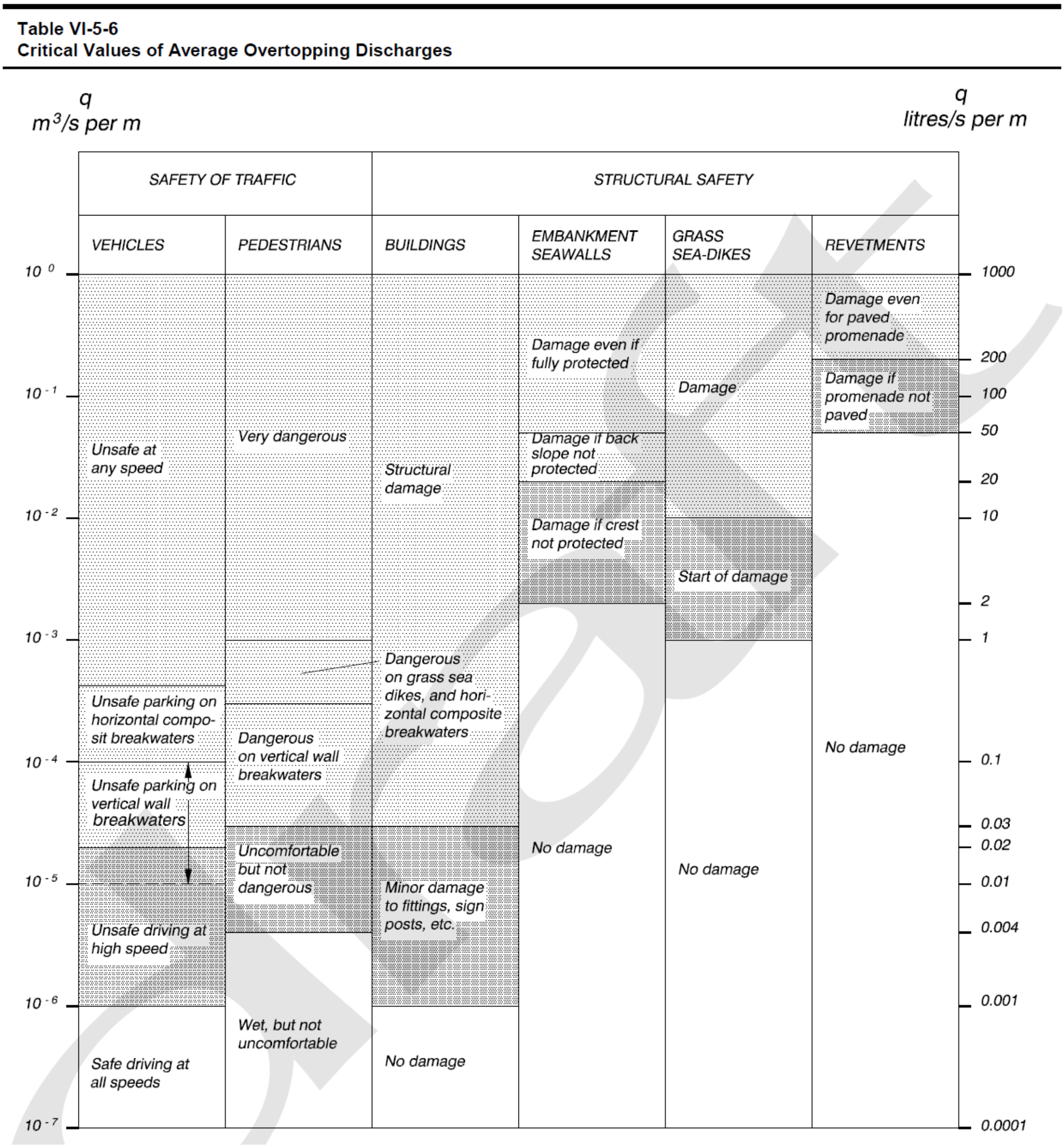


Figure 34 – Critical values of wave overtopping discharge (after Coastal Engineering Manual CERC 2003)

A higher sea level in the future would result in higher levels of wave runup onto the foreshore structures, and higher rates of wave overtopping in the future, if the crest levels are not designed to take account of the potential for sea level rise over the structure design life.

To reduce wave overtopping to a level that would improve pedestrian and vehicle safety, one or more of the following measures could be put into place:

- Increase foreshore dune or seawall crest levels – as an approximation, crest level would need to increase to above the significant runup level in Table 4 plus an appropriate allowance for freeboard and future sea level rise due to climate change
- Increase crest width for rock revetments – a wider crest is able to absorb wave overtopping back into the revetment armour
- Provide a wave return structure at the crest of the seawall in the vicinity of the jetty to reduce the volume of overtopping. An example of such a structure is provided in Figure 35, below.

Practical non-engineering measures could also be introduced to improve pedestrian and vehicle safety in the event of wave overtopping, including:

- Restricting public access to parts of Beach Road and the Southend boat ramp carpark during a large storm event (this was implemented during the storm event of 24 June 2014)
- Use of warning signs to advise the public of the danger to public safety due to wave overtopping.

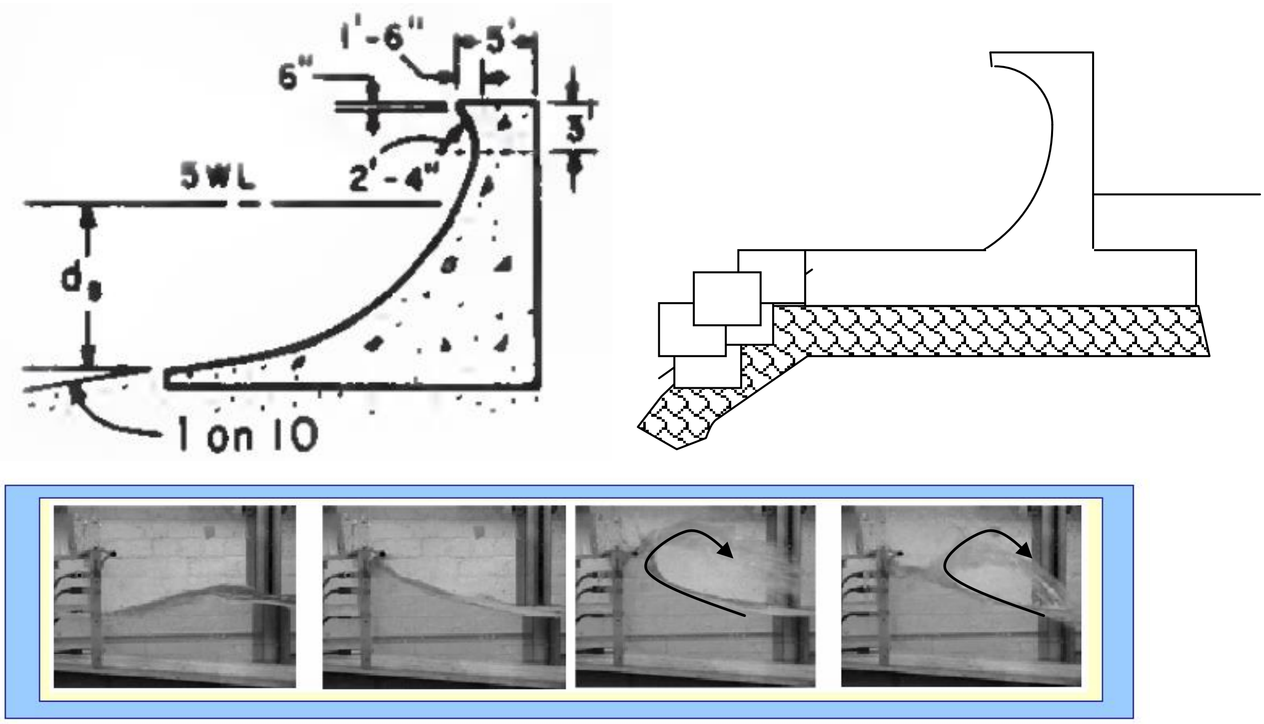


Figure 35 – Example of effect of recurved wave return wall on reducing overtopping of a seawall structure, and typical design detail (after EurOtop Overtopping Manual 2007 and CERC, 1984)

4.7.2 Scour Potential

During severe storm events, the beach berm will scour to low levels, with natural beach scour levels having been found to reach approximately -1 m AHD on open coast beaches (Nielsen *et al.*, 1992). Greater scour can occur in front of reflective seawalls, with Nielsen *et al.* (1992) quoting a level of -2 m AHD often used for design purposes. According to beach profile data provided by DEWNR, scour levels are lowest in the vicinity of the vertical seawall in the vicinity of the jetty at Beachport. Anecdotally from site observations, the area in front of the vertical seawall was observed to be submerged at low tide, indicating a scour level for this area below the low tide level or around -1 m AHD.

4.8 Climate Change

In the longer term, there may be global changes resulting from a postulated warming of the earth due to the accumulation in the atmosphere of certain gases, in particular carbon dioxide, resulting from the burning of fossil fuels (the Greenhouse Effect). According to the Intergovernmental Panel on Climate Change (IPCC, 2013), the upper range estimate for sea level rise for the 21st century is 1.0 m (Figure 36). This is made up of various components, including thermal expansion of the oceans (the largest component), melting of the Greenland and Antarctic ice sheets and melting of land-based glaciers. The South Australian Coast Protection Board through the Coastal Erosion, Flooding and Sea Level Rise Standards and Protection Policy (1992) recommends an allowance of 0.3 m for sea level rise to the year 2050, and 1 m by 2100, when considering coastal inundation and long term recession effects and planning for coastal development.

The impact of climate change on the coastal processes at Rivoli Bay are discussed in Section 2.3.5.

Where the shoreline is held in place by a seawall, there is a reduced capacity for the nearshore profile to be supplied with sand to maintain an equilibrium beach profile, as there is no sand store available in the dunes behind the beach. A future deepening of the beach profile as a result of beach erosion caused by sea level rise may therefore lead to higher waves being able to reach the foreshore structures, especially in the area in front of the Beachport seawall.

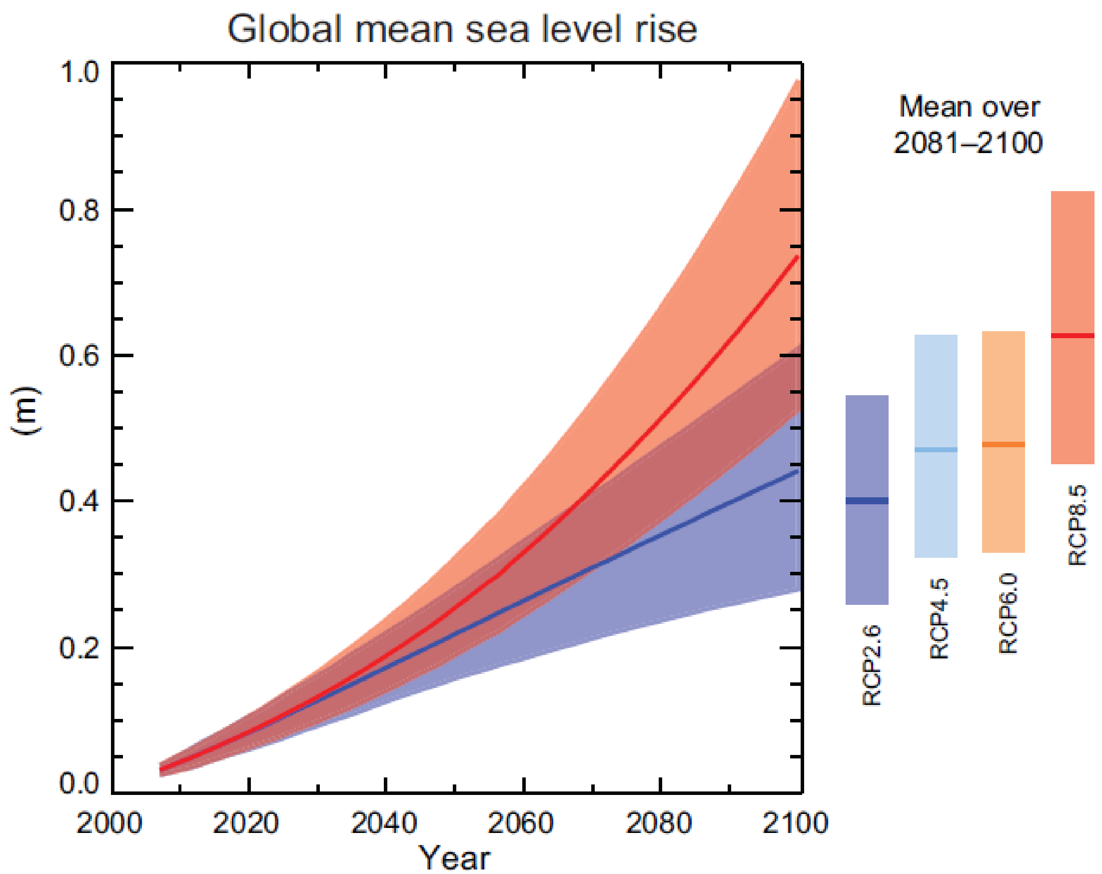


Figure 36 - Projected sea level rise between 2000 and 2100 (after IPCC, 2013)

The main aspects of Climate Change affecting the Beachport and Southend foreshores are as follows:

- Sea level rise of 0.3 m by 2050 and 1.0 m by 2100 (South Australian Coast Protection Board) resulting in future higher wave heights and runup levels onto the foreshore structures and dunes and greater volumes of wave overtopping
- Increased risk of erosion in front of the seawalls due to reduced onshore sand supply caused by sea level rise
- Higher waves able to reach the more reflective foreshore structures, particularly the vertical seawall at Beachport and revetments within the Southend jetty carpark caused by deepening of the nearshore profile.

4.8.1 Effect of Climate Change on Beachport and Southend seawalls

The main design parameters for the seawalls that would be affected by climate change are as follows:

- **Incident wave height** – this would be expected to increase at the structure face, due to a deepening of the nearshore profile if the rise in the nearshore seabed does not keep pace with the rise in sea level. The incident wave height at the structure could also be influenced by future long-term changes in offshore wave height and direction. The updated incident wave height at the structure can be estimated using the SBEACH model.
- **Scour Level –** Increased scour may be expected immediately in front of the vertical seawall at Beachport, due to higher wave energy reaching the structure as a result of sea level rise if the rise in nearshore bed level does not keep pace with the rate of rise in sea level. Updated scour levels can be estimated using the SBEACH model.
- **Crest Level –** As the sea level rises, the crest level with respect to mean sea level and, hence, the freeboard of the crest of the structure over the wave runup level will decrease over time. This would result in increasing frequency, depth and discharge of wave overtopping.

The effect of an increase in sea level on the incident wave height at the Beachport vertical seawall was modelled in SBEACH, for the 2050 planning horizon with 0.3 m sea level rise assuming no change in the bottom profile in front of the seawall and assuming that there would be no future change in offshore wave height and direction as a result of climate change. However, there is considerable uncertainty with regard to future changes in wave climate.

The results of this model run are provided in Figure 37. It can be seen that the water level as well as incident wave height at the structure would increase, leading to an increase in the level of wave runup at the structure by a higher value than the quantum of the sea level rise alone. Thus, increasing the crest of the structure by the quantum of expected future sea level rise would not be sufficient to prevent an increase in wave overtopping onto the road in the future. If the 0.3 m sea level rise by 2050 combined with additional scour is realised, the impact of a 1 year ARI event in the future on wave overtopping and local scour would be similar to the impact of a 10 year ARI in the present day.

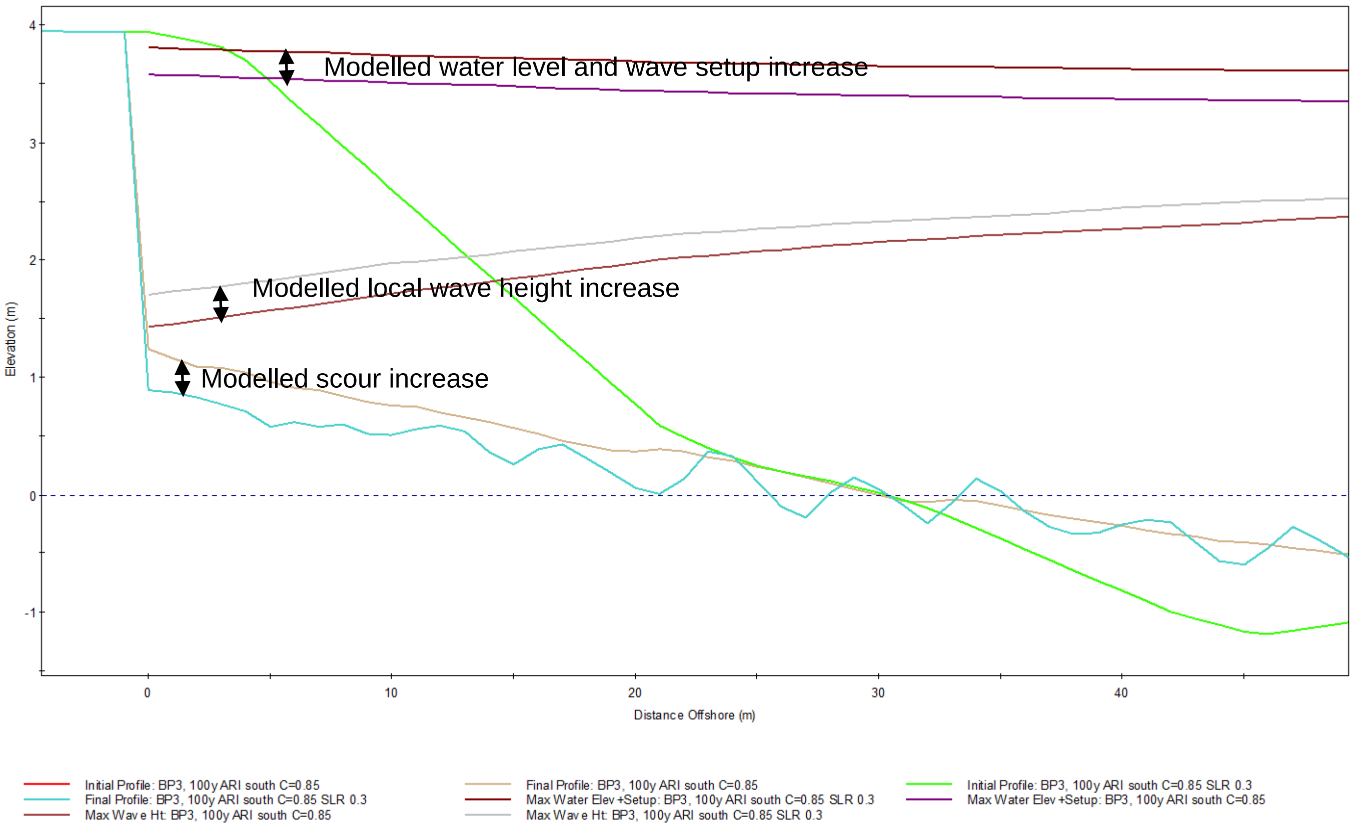


Figure 37 – SBEACH model run for Beachport vertical seawall near jetty, illustrating impact of sea level rise on local wave height, water level and scour

4.9 Summary

A quantitative assessment of the structural stability of the coastal works at Beachport and Southend and the coastal processes within Rivoli Bay has been carried out using wave modelling and analysis of existing data. It was found that:

- The rock armour at many of the groynes at Beachport and Southend is hydraulically unstable, with parts of the structure armour assessed to be subject to 30 – 40% damage under wave action for storm events greater than the 1 year ARI event on a severely depleted profile.
- The lack of sand in the jetty area at Beachport, coupled with low crest levels and reflective seawall result in frequent wave overtopping onto Beach Road in storm events greater than or equal to the 1 year ARI storm event as defined in this report. Overtopping volumes that would occur in such events would be at a level which would be dangerous for pedestrians and vehicles, and cause structural damage to the revetment in places. Structural damage would occur in a 100 year ARI event even in the areas where the promenade is paved. This situation is exacerbated by the layout and spacing of the existing groyne field and would be improved by provision of an additional groyne north of the jetty.
- The groyne field at Southend has been largely ineffective with erosion being exacerbated east of the Lake Frome outlet as a result of construction of the groynes there in the 1980's.
- Climate change has the potential to increase the design wave heights and design water levels at Beachport and Southend (due mainly to increased depths at the structure toe), as well as increase wave overtopping levels at Beach Road in the future (due mainly to sea level rise and increased design wave heights).

Design of a coastal management scheme for Beachport and Southend would need to take the above factors into account.

5 PRELIMINARY MANAGEMENT SCHEME

A range of preliminary management options for the foreshores at Beachport and Southend are presented in this section.

The options presented below have been formulated based on an assessment of the coastal processes within Rivoli Bay and the perceived coastal management issues at Beachport and Southend. Given that the coastal processes and issues within the Bay are well understood, discrete practical management actions developed specifically for Beachport and Southend have been discussed herein, rather than presenting discussion on the full range of generic coastal management options.

5.1 Main coastal management issues

The main issues identified through this investigation at Beachport and Southend that need to be addressed include the following:

- **Poor stability of existing groyne structures and temporary protection works at Beachport against wave action.** For the structures at Beachport and Southend, the primary armour layers generally are hydraulically unstable against direct wave attack for all storm events greater than a 1 year ARI event, when the beach is in an eroded state. This means that some damage to the structure armour would be expected. In a 10 year ARI event, some parts of the structures could expect to incur 30 – 40% damage to the primary armour layer. All the groynes at Beachport, and the two groynes at the outlet to Lake Frome at Southend, were assessed to be in poor condition.
- The area in front of the vertical seawall at Beachport is subject to **severe wave overtopping** onto Beach Road under adverse conditions and wave reflections, which does not allow buildup of a beach at this location;
- There is **ongoing entrainment of dune sand into Lake George** which has led to loss of sand from the littoral system, loss of conveyance within the outlet channel and ongoing siltation of the lower basin of Lake George;
- At Southend there is **severe dune erosion** in the area immediately east of the outlet to Lake Frome, where buildings within the caravan park are currently at the edge of the erosion escarpment and are under threat;
- The carpark at the boat ramp at Southend is subject to **wave overtopping** in large storms which can be dangerous for pedestrians and vehicles;
- Entrainment of sand has occurred into the outlet to Lake Frome with a reduced sediment supply to the beach east of the Lake Frome outlet.

Past coastal management issues that have recently or are in the process of being addressed include:

- Excessive **buildup of sand at the boat ramp** at Beachport – a breakwater has been constructed around the boat ramp which was designed to serve the dual purpose of protecting the ramp against wave action while directing littoral drift transport further north to bypass the boat ramp. However, since construction of the new breakwater, sand has begun to settle in the boat ramp basin, this sand being carried in by wave generated currents and sourced from Beach 4.
- **Loss of seagrass cover** within the Beachport area – this has occurred in recent years with an attempt to arrest the decline of the seagrasses through construction of a geotextile breakwater offshore from the boat ramp. This attempt was partially successful in protecting the seagrass beds, but the geotextile breakwater structure has now reached the end of its design life and is no longer providing protection to the seagrasses.

5.2 Preliminary Coastal Management Options

The various options for coastal management to address the identified issues that have been canvassed in this section of the report include:

- **Option 1 - Extension of the groynes at the channel entrance of Lake George** beyond the surf zone - this would reduce the littoral drift transport into the Lake outlet and, therefore, reduce the required frequency of dredging of the channel and slow down the growth of the flood tide delta. It would also allow greater ingress of seawater into the Lake thus improving the ability of the Lake to meet its water level target range.
- **Option 2 - The layout of the groynes** can be improved, particularly in the vicinity of the jetty, by lengthening the groynes or providing additional groynes in some areas. In particular, provision of an **additional groyne** just north of the jetty or extension of the existing groyne north of the jetty - this would encourage additional sand build-up around the jetty area;
- **Option 3 -** Provision of a **rock revetment** in front of the vertical timber seawall near the jetty - this would reduce the wave reflections from the existing seawall, reduce wave runup onto the road and encourage the buildup of sand in this area;
- **Option 4 and 5** - Review of existing management plan for **sand management** along Beachport and Southend foreshore to identify triggers for movement of sand between various beach compartments;
- **Option 6 - Repair of existing rock groynes** where these are inadequate (i.e. Southend Lake Frome outlet groynes) and repair of foreshore areas on the downdrift side of these groynes through ongoing sand management, provision of formalised access, repair of damaged beach accessways and improved dune management techniques;
- **Option 7 - Shortening of the groynes at the outlet to Lake Frome at Southend**
- **Option 8 - Lengthening of the groynes east of the Lake outlet at Southend**
- **Option 9 - Retreat** of critical infrastructure landward to prevent damage by coastal processes (e.g. critical infrastructure at Beachport and cabins at Southend) and **Review of planning**

controls for development that is subject to coastal hazards of beach erosion, recession, inundation and slope instability, for example, identifying setbacks for cabin development at the Southend Caravan Park and identifying building infrastructure criteria for new development in areas subject to coastal hazards;

- **Option 10 - Beach nourishment** – placement of sand onto the beach in areas experiencing foreshore recession (e.g. the Beachport foreshore near the jetty and the area east of the outlet to Lake Frome) to increase the beach width and reduce wave energy reaching the back of the beach
- **Option 11** - Management of inundation risk at Beachport and Southend by building concrete wave return wall at Beachport and raising revetment at Southend boat ramp by 1 m
- **Seawalls** - replacing damaged or inadequate seawall structures at Beachport and Southend with one of various seawall types, including:
  - Bulkhead walls
  - Rigid Gravity Structures
  - Rigid Sloping Revetments
  - Semi-rigid sloping pattern-placed unit revetments
  - Flexible near-vertical mass gravity structures
  - Flexible sloping rock rubble revetments
  - Flexible sloping sandbag revetments
  - Flexible sloping rock mattress revetments
- **Do nothing** – take no action

The various options are canvassed below and discussed in terms of their effectiveness and ease of implementation.

5.3 Option 1 - Extension of groynes at channel outlet to Lake George

Continuing sand entrainment into the Lake George outlet channel over time has led to the siltation of the lower basin area and outlet channel of Lake George and the growth of a flood tide delta over time, with up to 500,000 m^3 of sand having built up between the 1950's and the early 1980's, as evidenced by examination of historical aerial photography. Through sand sampling conducted for this study it is evident that sand has been continuing to be deposited into the outlet channel at Lake George and that the source of this sand is from the beach to the south of the outlet.

The Lake water levels are managed by means of a weir at the outlet, with logs at the weir typically removed in the early winter to allow ocean flows into the lake, and replaced again in spring to retain the water within the Lake during the summer months when freshwater inflows into the lake are low.

The siltation of the channel has led to the requirement for the SEWCDB to periodically dredge the outlet channel to allow the exchange of ocean water with the Lake, as the siltation of the channel does not allow efficient exchange of the water between the Lake and the sea, making it difficult to maintain the lake levels within the required target ranges specified in the Lake George Plan of Management. The SEWCDB has limited resources for dredging, and dredging of the Lake George outlet channel is not considered to be a high priority for them.

The configuration of the outlet channel was therefore examined to determine whether there is any scope to modify the channel dimensions to improve the tidal flushing of the lake, improve the hydraulic efficiency of the channel and therefore reduce the requirement for maintenance dredging of the outlet channel and ongoing siltation of the Lake. This was done by means of an Escoffier analysis of the outlet channel, which is detailed in Appendix 3. It was found that it would not be feasible to modify the outlet channel such that the cross-sectional area is in a stable equilibrium, as the channel would need to be excavated to a much greater width and depth than exists at present and severe bank erosion could result. In addition, increasing the channel dimensions to a level required to achieve a stable equilibrium would lead to increasing tidal ranges within the lake and lead to a change in the ecology of the lake due to the increased tidal range and modified salinities.

In conjunction with the modification of the Lake outlet channel, the option of extending the outlet groynes beyond the surf zone to reduce the supply of sediment to the Lake was examined. This would provide the dual purpose of creating a build-up of sand immediately up-drift of the Lake outlet, which can be used periodically as a source of sand to nourish the other beach compartments within the Beachport area as required.

This option has been discussed an examined in detail in the Lake George Study (WorleyParsons 2015).

A similar option has been put forward in past studies, such as suggested by Tonkin & Associates (1997), associated with the potential construction of a boat ramp at this location.

5.3.1 Longshore Sediment Transport and Outlet Stability

Further impacting the stability of the Lake outlet is the supply of littoral drift to the outlet from the south. Bruun (1978) presents a method for evaluating the stability of a tidal inlet where the inlet stability is graded in terms of the ratio between the tidal prism (Ω) and the total longshore sediment transport flux (M_{tot}). If the ratio between Ω and M_{tot} is greater than around 150, conditions at the lake ocean inlet are relatively good with a small offshore bar and good tidal flushing. For conditions where this ratio is less than around 50, the ocean inlet conditions are considered to be “poor” – i.e. the outlet is considered to be unstable and tends to close over due to input of littoral drift.

For the Lake George outlet, the ratio of tidal prism to sediment transport flux is estimated to be around 20. This ratio may be increased by reducing the supply of littoral drift transport (M) to the lake outlet, thus improving the stability of the outlet. To do this, an effective method would be to increase the length of the groynes at the lake outlet such that the sediment transport bypassing the groynes is lower than is currently the case. This would provide the benefit of reducing the sand supply to the lake. An additional benefit is that a source of sand becomes available that can be used from time to time to nourish the other beach compartments along the Beachport foreshore where erosion may be experienced as a result of storms. Given the low freshwater inputs into the lake, the lake entrance could be dredged as required, with the weir kept open year-round to allow tidal flows in and out of the lake, with the required frequency of dredging reduced due to the reduced sediment supply as a result of lengthening the groynes at the lake outlet.

A potential management scheme for the outlet to Lake George is presented in Figure 38.

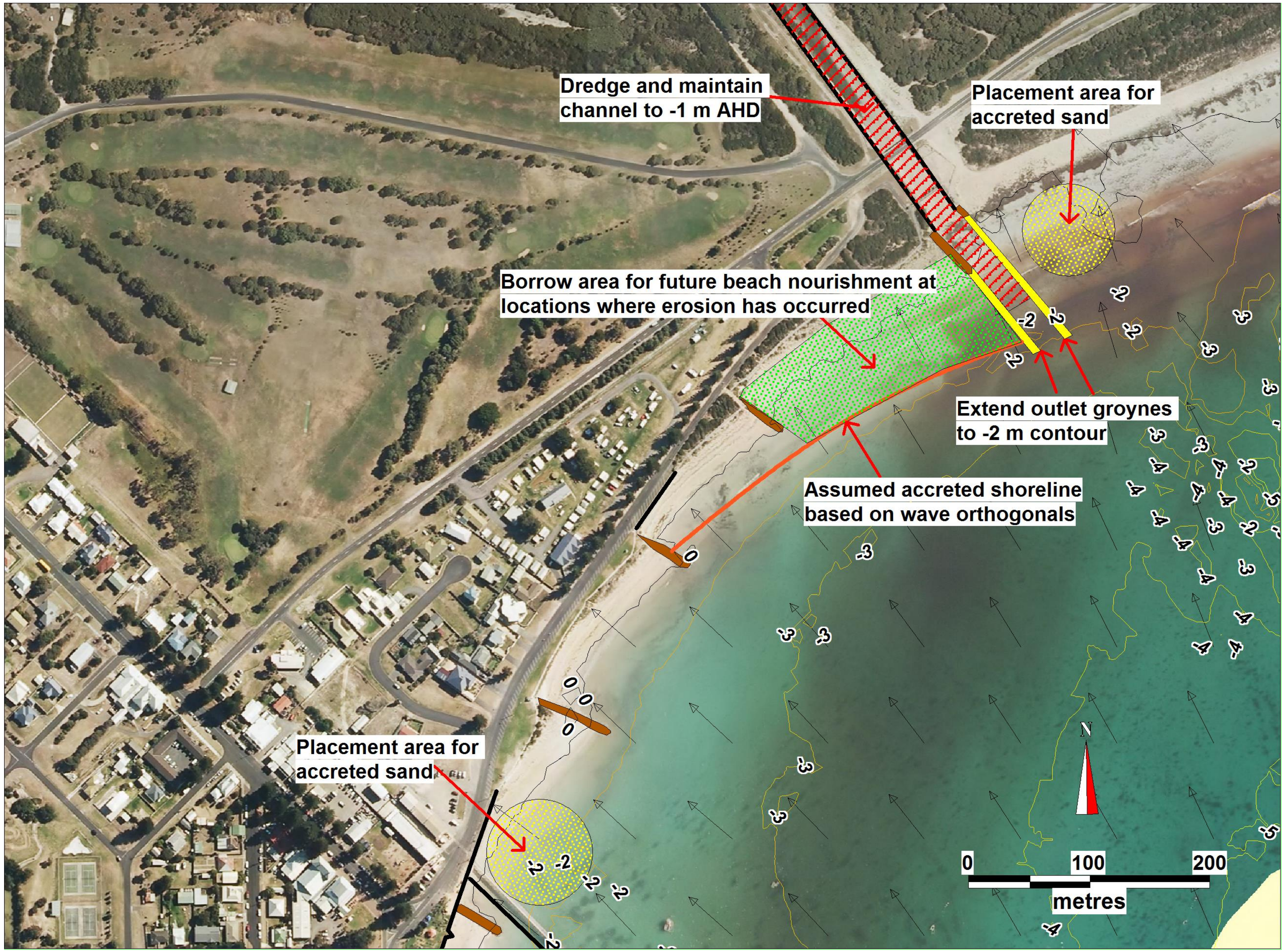


Figure 38 – Potential management scheme for Lake George outlet area

5.4 Option 2 – Improve layout of groynes and provision of additional groyne at Beachport

It is considered that there would be benefit from providing an additional groyne in the area immediately north of the jetty as shown in Figure 39. It is suggested that the additional groyne extend to approximately the -2 m AHD bathymetric contour and be designed such that it would withstand the wave climate experienced at that location. An indicative conceptual design for the groyne is provided in Figure 40. The indicative mass of the rock has been calculated using the Hudson formulation for the locally available rock as discussed in Section 4.4.

It is understood that the SA Department of Transport is considering the construction of an additional groyne in this vicinity which would be consistent with this option.

A new breakwater has been constructed around the boat ramp, downdrift of Beach 4, with the construction completed in November 2014. Since the breakwater was completed, sand has accumulated in the boat ramp basin. This sand is being transported into the boat ramp area by wave generated currents and wave diffraction around the breakwater tip.

During the community consultation for this project, suggestions to reduce the accumulation of sand into the boat ramp area were received, including extension of Groyne 5 to reduce sand supply from Beach 4 to the tip of the new breakwater.

It is considered that a short extension of Groyne 5 to “close” the sand transport pathway at the tip of the groyne and over the remains of the geotextile breakwater (as shown in Figure 39), together with sand management of Beach 4, would reduce (but not completely eliminate) the sand supply to the tip of the breakwater and reduce the accumulation of sand in the boat ramp area.

5.5 Option 3 - Provision of upgraded seawall at jetty

At the vertical seawall in front of the jetty at Beachport, reflections from the seawall have led to a loss of the beach from in front of the wall. The local deepening of the profile has led to increased wave energy reaching this area and has led to overtopping onto Beach Road on a regular basis at this location. Provision of the additional groyne as discussed above in conjunction with placement of sand at this location would reduce the wave energy in this area thereby reducing the frequency of wave overtopping onto Beach Road. Provision of sand would also improve the stability of the jetty itself as well as improve the local amenity of this area by providing a beach which can be used by the local community.

In a large storm, however, this sand can be eroded away, thus exposing the vertical seawall resulting in loss of the beach from this area. To improve the beach amenity and improve the stability of the vertical wall (by reducing scour and wave reflections), a rock revetment can be provided in front of the wall, which would absorb wave energy, reduce wave overtopping onto the road and allow a beach to be maintained in this area for longer. This could be implemented by extending the existing rock revetment northward to encompass the area currently fronted by vertical seawall.

In lieu of a rock revetment, various options for reconstruction of the existing vertical timber seawall are available and are discussed below. These options consider the types of structures and construction methodologies.

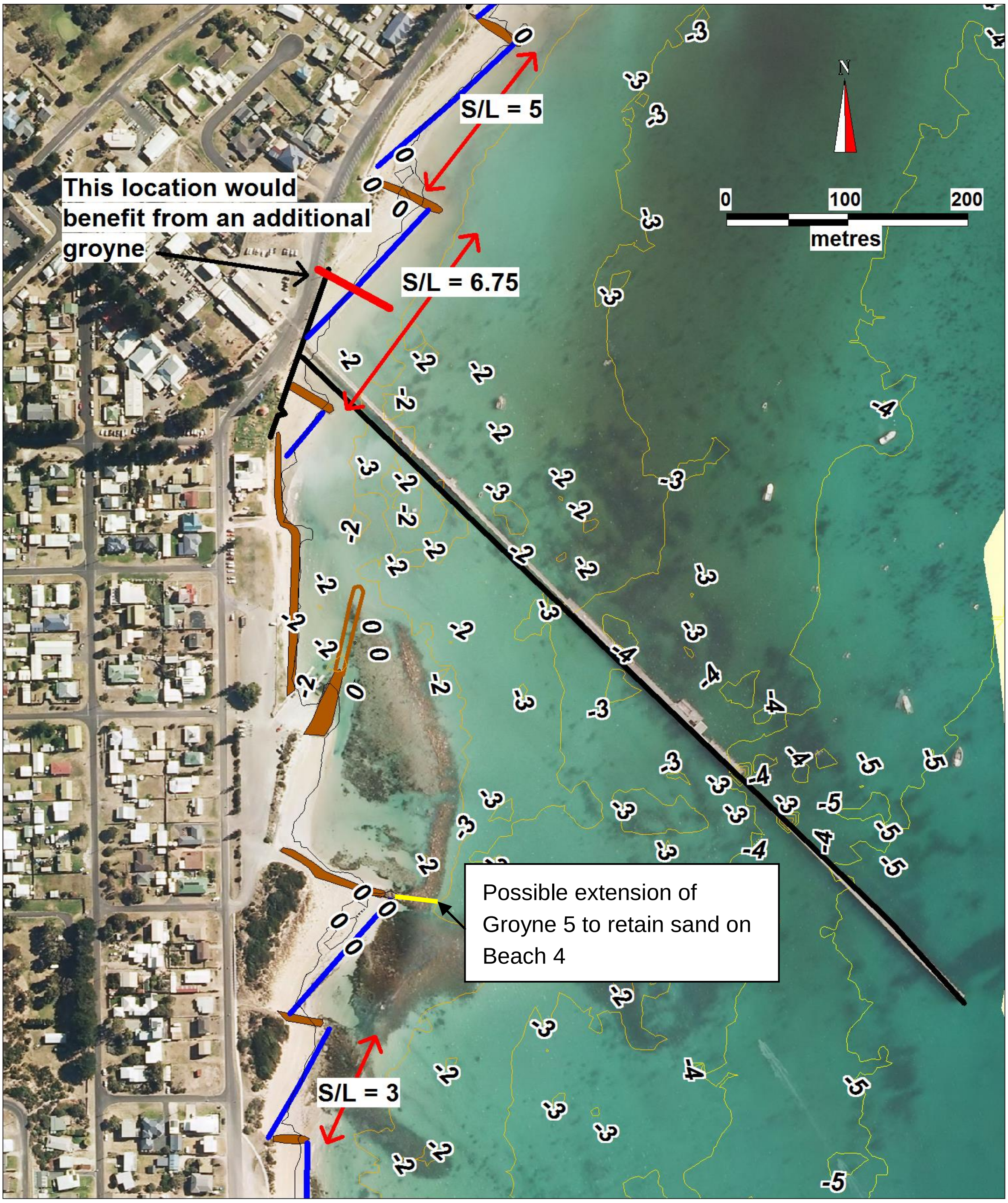
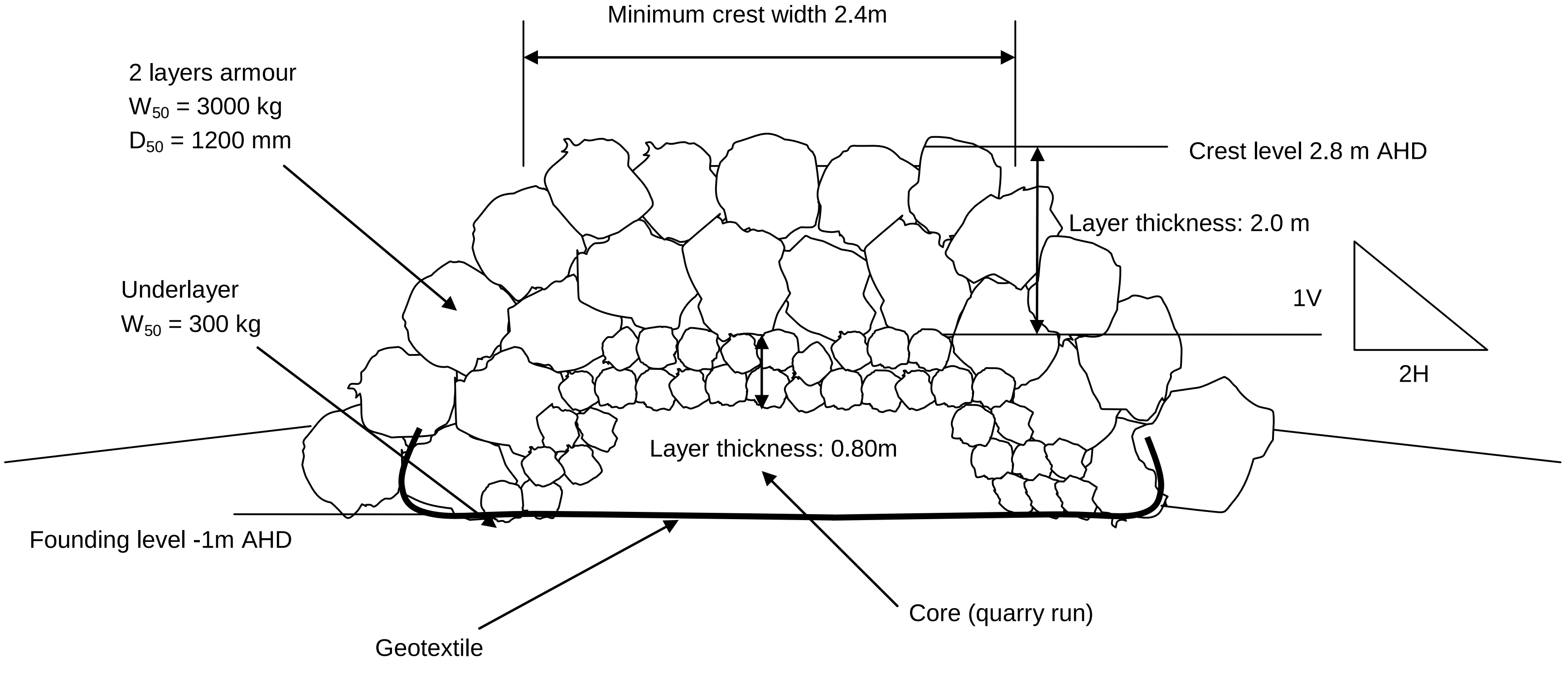


Figure 39 – Suggested location of additional groyne, Beachport



SECTION VIEW

Figure 40 – Conceptual design of groyne cross section for additional groyne north of jetty area

5.5.1 Rigid Gravity Structures

Rigid gravity structures can be near-vertical or stepped structures. They are typically concrete or blockwork walls and have been used as promenades on beaches (Figure 41 and Figure 42).

The advantages of rigid gravity structures for this site are:

- Near-vertical structures can have a smaller seawall footprint;
- Stepped structures can provide seating and can easily incorporate access; and
- Limited maintenance required.

The disadvantages of rigid gravity structures for this site are:

- Rigid near-vertical gravity structures tend to reflect rather than dissipate wave energy and could lead to erosion of the beach as is experienced currentlty. A rigid stepped gravity structure would be better able to dissipate wave energy;
- Rigid structures are more prone to catastrophic failure. If the structures are not piled, they would be unable to accommodate settlement or adjustment of the underlying material;
- Can be expensive; and
- Rigid stepped structures can require maintenance in the tidal zone for marine growth.

Rigid near-vertical and stepped gravity structures can be considered further for this site. Consideration would need to be given to whether piles are required.

The crest level for this site is up to approximately 4.0 m AHD. Therefore the structure for this site could be up to 4 to 6 m high with consideration to the design scour level of -2 m.

Figure 41 – Rigid near-vertical sandstone gravity wall.

Figure 42 –Rigid stepped concrete gravity wall.

5.5.2 Rigid Sloping Revetments

Rigid sloping revetments are popular on promenades, especially where there is very heavy pedestrian traffic, such as on main tourist beaches. The facing can be a concrete slab or interlocked bricks, concrete or rock blocks (Figure 43).

However, generally they are unable to accommodate settlement or adjustment of the underlying materials.

The advantages of rigid sloping revetments for this site are:

- Relatively thin and comprise components that can be transported readily to site;
- Stairs can be incorporated into sloping revetments with minimal protrusion seaward and landward of the revetment, allowing unobstructed access along pathways and foreshore; and
- Limited maintenance required.

The disadvantages of rigid sloping revetments for this site are:

- Rigid structures are more prone to catastrophic failure as they are unable to accommodate settlement or adjustment of the underlying material;
- Large wave run-up onto the promenade and Beach Road in a large storm. A wave return wall could be at the crest used to limit wave run-up; and
- Little dissipation of wave energy.

Rigid sloping revetments with a wave return wall can be considered for the site. However, a semi-rigid sloping pattern-placed unit revetment is likely to be more suitable because it can dissipate wave energy and can tolerate some movement or displacement without total collapse.

Figure 43 –Promenade and rigid sloping revetment with access stairs.

5.5.3 Semi-Rigid Sloping Pattern-Placed Unit revetments

Sloping pattern-placed unit revetments comprise units that can tolerate some movement or displacement without total collapse (Figure 44). These revetments can dissipate wave energy at the back of beaches and along the foreshore.

Pattern-placed units can be more stable than randomly placed units, and can result in the use of lighter individual units. These revetments can be useful where site constraints limit the use of randomly placed units.

The advantages of sloping pattern-placed unit revetments for this site are:

- Relatively thin and comprises components that can be transported readily to site;
- Tolerance for some movement or displacement without total collapse;
- Provide dissipation of wave energy;
- Stairs can be incorporated into sloping revetments with minimal protrusion seaward and landward of the revetment, allowing unobstructed access along pathways and foreshore; and
- Limited maintenance required.

The disadvantages of sloping pattern-placed unit revetments for this site are:

- Some wave run-up onto the promenade and Beach Road in a large storm. A wave return wall could be at the crest used to limit wave run-up;
- Less tolerance for movement and displacement of units than flexible structures; and
- Pattern placement can be time consuming during construction.

Sloping pattern-placed unit revetments with a wave return wall can be considered for the site.

Figure 44 –Example of Seabee seawall.

5.5.4 Flexible Near-Vertical Mass Gravity Structures

Flexible near-vertical mass gravity structures can comprise various materials including sandbags, rock boulders and gabion units (Figure 45 and Figure 46). These near-vertical mass gravity structures have a smaller footprint than sloping structures and can be effective in reducing the encroachment on beaches.

Sandbags

Sandbags comprise geotextile containers filled with sand. There is a range of container sizes. The larger containers are typically 2.5 m^3 with dimensions of 600 x 1800 x 2400 mm.

Preliminary stability analysis of a sandbag gravity wall by Nielsen and Mostyn (2011) shows the width of the wall would be approximately the same as the height of the wall.

Sandbags are mainly for wave climates with significant wave height of less than 2.5 m and therefore would not be suitable for this site, except for use as emergency protection as has already been implemented.

Rock Boulders

Large rock boulders could be used to construct a mass gravity structure at a steeper slope than a flexible sloping rock rubble revetment. The rock boulder gravity wall would result in a smaller over footprint than a flexible sloping rock rubble revetment, but would likely require a thicker rock layer. The thickness and slope would be subject to detailed design and stability analysis.

Gabion Units

Woven mesh Gabions and Reno Mattresses consist of woven mesh units that are laced together and filled with stones to form monolithic structures. Unlike other conventional river bank protection structures, their structure and design allows for a certain degree of flexibility, allowing the units to deform to differing settlements whilst still maintaining their functionality.

A weakness of the gabion and reno mattress systems can be their susceptibility to mechanical abrasion. The mesh baskets that house the stones are coated with protective substances for protection and durability, such as alloy and PVC coatings. Gabion and reno mattress units are not recommended for the open coast as these coatings can be easily scratched or damaged and corrosion of the exposed wire case will occur rapidly. Weaknesses in the casings can lead to a loss of structural integrity which may result in a catastrophic failure of the structure.

There are also safety issues with sharp wire exposed on the beach.

Flexible near-vertical rock gravity structures can be considered for this site.

Figure 45 – Sandbag gravity seawall (source: Geofabrics Australasia)

Figure 46 – Left: Rock boulder gravity seawall; Right: Gabion gravity seawall

5.5.5 Flexible Sloping Rock Rubble Revetments

Flexible sloping rock rubble revetments can comprise armour layer, underlayer, core, filter layers or geotextiles. The design of rock rubble revetments can be limited by the size, shape and quality of rock available from nearby quarries.

The advantages of flexible sloping rock rubble revetments for this site are:

- Tolerance for a significant degree of displacement and shifting;
- Provide dissipation of wave energy;
- Potential reuse of existing rock on site;
- Easier to repair; and
- Easier to adapt to sea level rise.

The disadvantages of sloping pattern-placed unit revetments for this site are:

- Could create habitats for vermin and trap rubbish;
- Hazardous pedestrian access over revetment; and
- A larger footprint compared to other options.

Flexible sloping rock rubble revetments can be considered for the site.

Figure 47 – Existing rock revetment south of Beachport jetty

5.5.6 Flexible Sloping Sandbag Revetments

Sloping sandbag revetments are being used increasingly on beachfronts. Sandbags are mainly for wave climates with significant wave height of less than 2.5 m and therefore would not be suitable for this site except if used as emergency protection works.

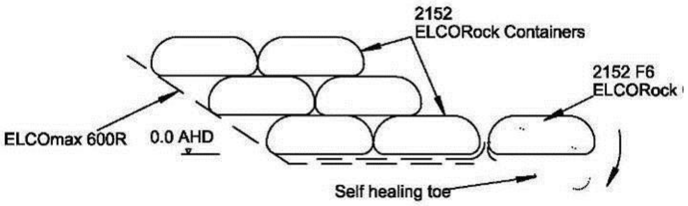


Figure 48 – Sandbag revetment (source: Geofabrics Australasia)

5.5.7 Flexible Sloping Rock Mattress Revetments

Flexible sloping rock mattress revetments are commonly used in the rehabilitation and protection of riverine environments. They consist of woven mesh units that are connected together and filled with rock.

Reno mattress systems units are not recommended for the ocean coast as the coatings and casings for the units could easily be damaged and lead to catastrophic failure.

Figure 49 – Left: Sloping Reno-mattress revetment under construction (Source: NSW Gov., 1990).

5.5.8 Preferred Options for Construction

Based on the above, the following preferred options would be considered further to reduce the reflection of wave energy in the jetty area:

- sloping pattern-placed unit revetments with a wave return wall; and
- flexible sloping rock rubble revetments.

5.6 Option 4 and 5 - Sand Management

Currently, Beachport has a Sand Management Plan for the town beaches at Beachport (Wattle Range Council, 2013). This Management Plan establishes a pro-active sand management program for redistributing sand between the various groyne compartments which is to be established in conjunction with extension of the breakwater associated with the boat ramp. The Plan stipulates operational requirements for movement of sand and regular 12-monthly survey requirements for beaches 4, 5, 7, 8 and 9.

Generally, the Plan recommends sand be collected from Beach 4 (south of the boat ramp) and from the boat ramp basin (to keep the boat ramp area navigable). Trigger points for removal of sand from Beach 4 are related to sand levels as measured on Beach 4 by a monitoring pole. When sand levels at the pole reach 0.5 m AHD, the Plan recommends that the beach be excavated by 0.5 m to the bottom of the marker, or 0 m AHD. The Plan also recommends inspecting the pole monthly. The Plan recommends placement of the material on beaches 7 and 8 and recommends removal of the sand by excavator and carting the sand by truck.

The existing Plan of Management is considered to be an effective way to manage sand build up and depletion within the town beaches at Beachport provided it is implemented effectively and reviewed as required.

Should the measures of an additional groyne north of the jetty and lengthening the groynes at the outlet to Lake George be implemented, the existing Sand Management Plan can be updated to reflect the availability of a new source of sand (i.e., on the up-drift side of the Lake outlet) and mechanical bypassing at the lake outlet (to prevent excessive beach erosion north of the lake outlet). Placement at beaches 7 and 8 may not be required to be done as frequently if a new groyne is constructed immediately north of the jetty, as this would be expected to improve the stability of the beach in this area.

Grain size analysis of the sand in the various beach compartments indicates that the sand would be suitable for nourishment of the depleted areas.

It is considered that a similar Plan of Management can be implemented for Southend as well, with the operational requirements based on those already in place for Beachport. The Plan of Management for Southend would involve monitoring the beach levels for the beach immediately west of the outlet to Lake Frome with a monitoring pole similar to that used for Beach 4 at Beachport, then placement of this sand using mechanical equipment onto the beach berm in the area immediately downdrift of the Lake outlet (where severe dune erosion is currently taking place). Dredging of the existing buildup of sand within the Lake Frome outlet could be carried out also using land-based excavator equipment with placement of this sand onto the beach immediately east of the lake outlet. A proposed sand management scheme for the Southend area is provided in Figure 50.

5.6.1 Sand Placement and Dune Management

Sand can be pushed or placed by dozer or excavator against the toe of the dune escarpment (Beach scraping), to enhance the natural beach building process of long, low ambient swell wave conditions bringing sand back onto the beach. This would result in a build-up of material at the toe of the eroding embankment, thereby slowing down the rate of erosion. The placement of sand should not result in a steepening of the local beach profile, as the sand would redistribute itself to a flatter nearshore profile due to the fine grained nature of the native sand. Rather, the beach scraping should be carried out in such a way so that the beach profile slope is maintained and to provide toe protection to the existing eroded beach embankment. An example of this process is illustrated in Figure 51.

Placed sand can be subject to an ongoing dune management program, such as has already been successfully implemented at the northernmost beaches within the Beachport area. Dune management is the combination of activities to maintain vegetative cover on the foredune to prevent sand blowing inland where it is lost from the coastal system. Key elements of successful dune management include dune management planning, reconstruction, re-vegetation, protection and maintenance. To be most successful, dune management programs require the community to be aware of, and actively or passively support, dune management works. The engraving of community member's names on the boardwalk posts as observed throughout Beachport is considered to be an effective way of raising community awareness and involving the local community in dune management activities.

5.7 Option 6 - Repair and maintenance of existing groynes

It is considered that the existing groynes at Beachport, while mostly in poor condition, have mainly been effective in stabilising the foreshore and upgrade of the groynes with larger rock to better withstand the local wave climate may not be an efficient use of funds. The exception to this is the groyne immediately updrift of the boat ramp, although this groyne was found to be in good condition when compared to the remaining groynes along the beachfront.

Damage to many of the groynes and associated beach erosion on the immediate downdrift side of the groynes was observed during the site inspection – it is considered that this damage can be repaired on a case-by case basis with localised minor works to improve public safety. For example, works can be implemented on the downdrift side of Groyne 2 at Beachport to improve public safety and beach access at the vertical escarpment, and prevent outflanking of the groyne in this area (Figure 52).

Repair of the damaged groynes at the outlet to Lake Frome could be undertaken to prevent further ingress of sand into the Lake Frome outlet channel. Any new groynes (such as those proposed near the jetty and near the outlet to Lake George) should be constructed according to an engineered design which considers the wave climate and coastal processes as described in this report.

While major capital investment to upgrade the existing groynes may not be warranted due to the high initial capital outlay, ongoing minor maintenance of the groynes as is currently undertaken by Council and the local community to improve public safety has been relatively effective and is considered to be a more affordable strategy for managing the foreshore in the short – medium term.

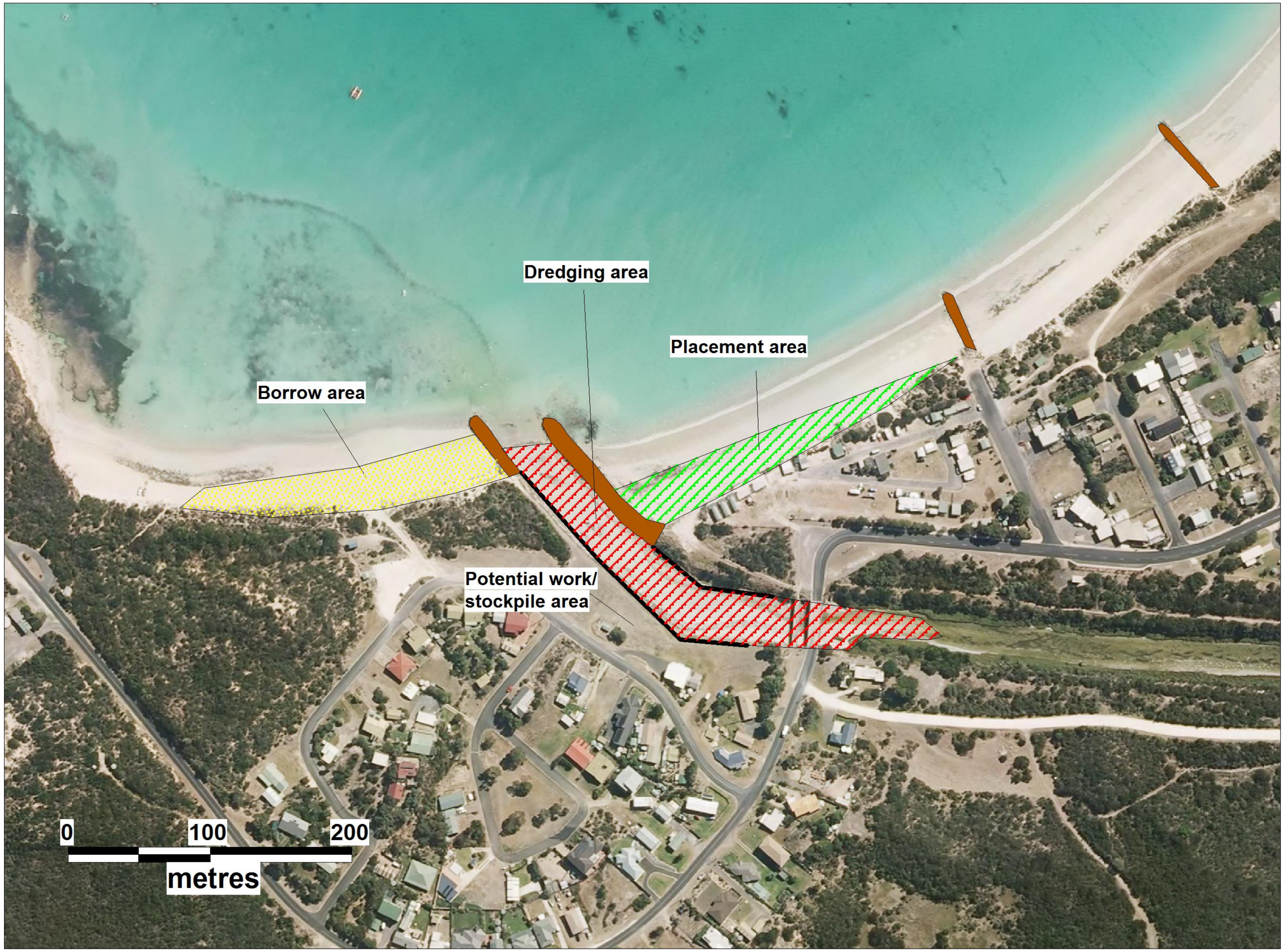


Figure 50 – Potential Southend sand management scheme

Figure 51 – Example of beach scraping of sand against eroded dune escarpment

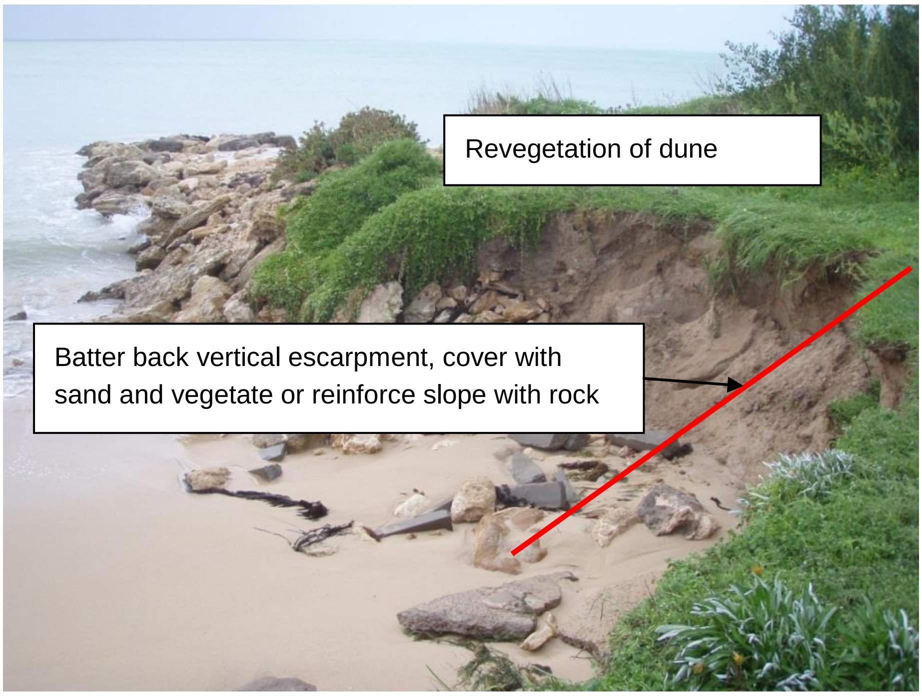


Figure 52 – Potential minor works to improve public safety at Groyne 2

5.8 Option 7 and 8 - Removal or shortening of the Lake Frome outlet groynes at Southend

Evidence from historical aerial photography and wave modelling has shown that the groynes at Southend, particularly east of the Lake Frome outlet, have not had a beneficial impact on this section of beach and that the main sediment transport pathway in this area is onshore-offshore transport. Further, the groynes at the outlet to Lake Frome have trapped a large quantity of sand to their west, with severe beach recession having occurred on the eastern side of the lake outlet in this area, threatening caravan park infrastructure.

It is suggested the rock groynes at the Lake Frome outlet could be removed or shortened, and the rock re-used to lengthen the remaining groynes east of the Lake Frome outlet, repair some of the groynes at Beachport, construct the proposed additional groyne on the northern side of the Beachport jetty, or raise the existing revetment near the Southend boat ramp. This would allow sand currently trapped on the western side of the lake outlet to by-pass the outlet, thereby restoring the beach in the area in front of the caravan park and reducing the erosion risk. The Lake outlet would then be allowed to open and close naturally in response to flow conditions in Lake Frome and beach conditions. The three eastern-most groynes along the foreshore at Southend could be retained to help prevent loss of sand to the east, helping to stabilise the sand compartments east of the Lake outlet and allowing sand to be scraped along the beach to replace any sand lost when short term erosion due to storms takes place. Lengthening these groynes may improve their ability to trap sand on the beach. However, as the sand bypassing is likely occurring some distance offshore of the groynes, substantial lengthening would be required to reduce the volume of sand being bypassed, which would require substantial investment.

There is a risk with this option that erosion of the beach on the western side of Lake Frome outlet could occur, as the outlet groynes have currently led to a stable beach forming at this location. Furthermore, previous experience with manual bypassing of sand around the Lake Frome outlet has resulted in the deposited sand being removed very quickly, possibly through offshore then along-shore transport.

Due to the risk of increasing erosion on the beach to the west of the lake outlet, feedback during the community consultation indicated that the local community is generally opposed to the removal of these groynes. However, it is noted that these groynes are currently in very poor condition and can be shortened, with the armour at their ends upgraded in order for them to withstand the local incident wave climate. Should these groynes be allowed to deteriorate, there is a risk that erosion of the beach to the west would result and that rock armour from the existing groynes would be dislodged onto the adjacent beach. These groynes are currently under the jurisdiction of the SEWCDB.

There is a risk also that lengthening the groynes east of the outlet would not be very effective and that these groynes would continue to be bypassed, with offshore sand transport continuing to occur (and perhaps increased) by the lengthening of the groynes.

Should the groynes at the lake outlet be removed, management of the Lake outlet would need to be carried out, including monitoring of the beach berm level at the outlet and mechanically opening the outlet when the beach berm reaches a certain trigger value, to prevent flooding of low-lying land in the Lake Frome catchment upstream. The above concept is illustrated in Figure 55.

While the issues at Lake George and Lake Frome appear similar, the recommended management options at each of these areas is markedly different. Lengthening the Lake George groyne is recommended to reduce additional sand ingress into Lake George and stabilise the updrift beach compartments where urban infrastructure exists. At Lake Frome there is infrastructure on the beach compartments on the downdrift side of the outlet groynes which have been subject to erosion as a result of the groynes. Also, the dynamics of the channel at Lake Frome and Lake George are different and shortening the groynes at Lake Frome would not result in ingress of large volumes of sand into the channel as has happened at Lake George.

It is noted that an Adaptation Study for Southend (funded by Council and the CPB) will look at the long term future of Southend and whether significant investment in coastal protection can be warranted for that precinct.

5.9 Option 9 - Retreat of Critical Infrastructure

Beach erosion can lead to direct damage to built assets such as dwellings, water and sewer infrastructure, roads, fencing and public amenities. Direct damage could be catastrophic, such as the destruction of dwellings or loss of life; or could be less serious, such as the temporary loss of services or damage to dune fencing, which can be restored for a known monetary cost. A loss of beach amenity can occur on a temporary basis due to beach erosion, which can have a direct impact on the economy or perceived values at the locality. Direct damage to natural assets such as beach dune ecology can also occur as a result of beach erosion – these systems are often resilient and may recover fully over time.

In areas where infrastructure is considered to be at risk from erosion or coastal inundation, retreat of that infrastructure inland from the coast may be considered as an option. A “retreat” approach recognises that coastal processes and coastline hazards are impacting on the coastline, and that the nature of this impact is likely to worsen in the future. For example, the cabins within the caravan park east of the Lake Frome outlet were close to the edge of the dune escarpment following the storms of June 2014 and were considered to be at risk due to slope adjustment and short-term storm erosion within the dune, as discussed in Section 2.3.4. While the conditions in this area could be improved by placement of additional sand from west of Lake Frome, it is considered that as the cabins were at risk from coastal processes, the risk could be mitigated by moving them landward away from the erosion escarpment and collapsing the steep dune escarpment. This work was carried out in late 2014 and was able to be achieved without purchase of additional land and as the cabins are of lightweight construction, the existing use of the caravan park area is considered to be compatible with the coastal hazard in this area.

For other areas, the coastal risk may presently be low but may increase in the future, for example areas expected to suffer from long term beach recession due to future sea level rise. In these areas, the ability of the community to maintain infrastructure and keep existing properties in their current locations may begin to decline in the future. Infrastructure such as water supply, electricity and sewer may become increasingly exposed to coastal erosion, and eventually it will be more difficult to maintain services for some of the more exposed seaside locations. With future coastal erosion and beach recession due to sea level rise, it may be more difficult to maintain, for example, Beach Road in the future as portions of the roadway may be lost due to future coastal erosion if the roadway is not protected. Eventually, if no action is taken, loss of structural integrity of seaside buildings may result.

It is noted that the notion of "retreat" depends upon the availability of an alternative location to retreat to – in some areas, infrastructure can retreat landward within the same beachfront lot but in others this may not be possible and the infrastructure may need to be abandoned –through voluntary or compulsory purchase of the infrastructure at threat.

5.9.1 Review of Planning Controls - Coastal Setbacks

Required setbacks for properties within the coastal hazard areas can be defined in the local planning instruments. Wattle Range Council has a Development Plan which outlines development controls for areas within the coastal zone, including the coastal frontage along Rivoli Bay. Hazard risk minimisation measures are included in the Development Plan, with the Plan stipulating that development is to be protected against the "1 in 100 year average return interval flood extreme sea level (tide, stormwater and associated wave effects combined), plus an allowance for land subsidence for 50 years at that site". The development also stipulates minimum levels for new commercial, residential, tourism or industrial development and associated infrastructure needs to be protected against sea level rise.

The Wattle Range Development Plan also stipulates that:

"Development should be set back a sufficient distance from the coast to provide an erosion buffer which will allow for at least 100 years of coastal retreat for single buildings or small scale developments, or 200 years of coastal retreat for large scale developments (i.e. new townships)", unless coastal protection measures are in place.

For Rivoli Bay, although these measures present a sound basis for controlling development in vulnerable coastal areas, the magnitude of coastal retreat in the various localities has not been accurately defined to enable the development control provisions to be put into practice. The Beachport town centre is considered to be protected by coastal protection works, although it has been shown to be vulnerable to coastal inundation and shoreline recession in areas where the groynes are too short to adequately to stabilise the shoreline. At Southend, coastline recession is particularly evident in the area east of the entrance to Lake Frome, so development controls may need to be defined for that precinct.

If urban development is shown to be in an area vulnerable to future coastal recession and slope instability, erosion of the dune in front of the existing house could occur leading to the house being affected by reduced foundation capacity in the future. This would require knowledge of the long term recession rate at the beach (including through measured beach profile trends and expected recession due to sea level rise), geotechnical properties of the dune sand, and volume of sand that would be eroded from the beach escarpment in a design storm event, such as a 1 in 100 year ARI storm erosion event. It is usually not appropriate to locate development where the erosion and recession risks are shown to be unacceptably high.

However, should existing development be located in a zone vulnerable to future beach recession and erosion, the structural integrity of any new dwelling would be assured if the building is supported on deep piling foundations designed in accordance with Nielsen, AF; Lord, DB and HG Poulos (1992), “Dune Stability Considerations for Building Foundations”, Australian Civil Engineering Transactions, Institution of Engineers Australia, Volume CE34, No. 2, June, pp. 167 173. Such piles would need to be designed to account for forces induced by the collapsing soil mass as well as wave impact. It is recommended that the piles are founded sufficiently deep to ensure that due consideration to storm events larger than the designated hazard has been allowed for. As a guide, piles would be required to approximately 5 m below AHD.

This concept is illustrated in Figure 54, below.

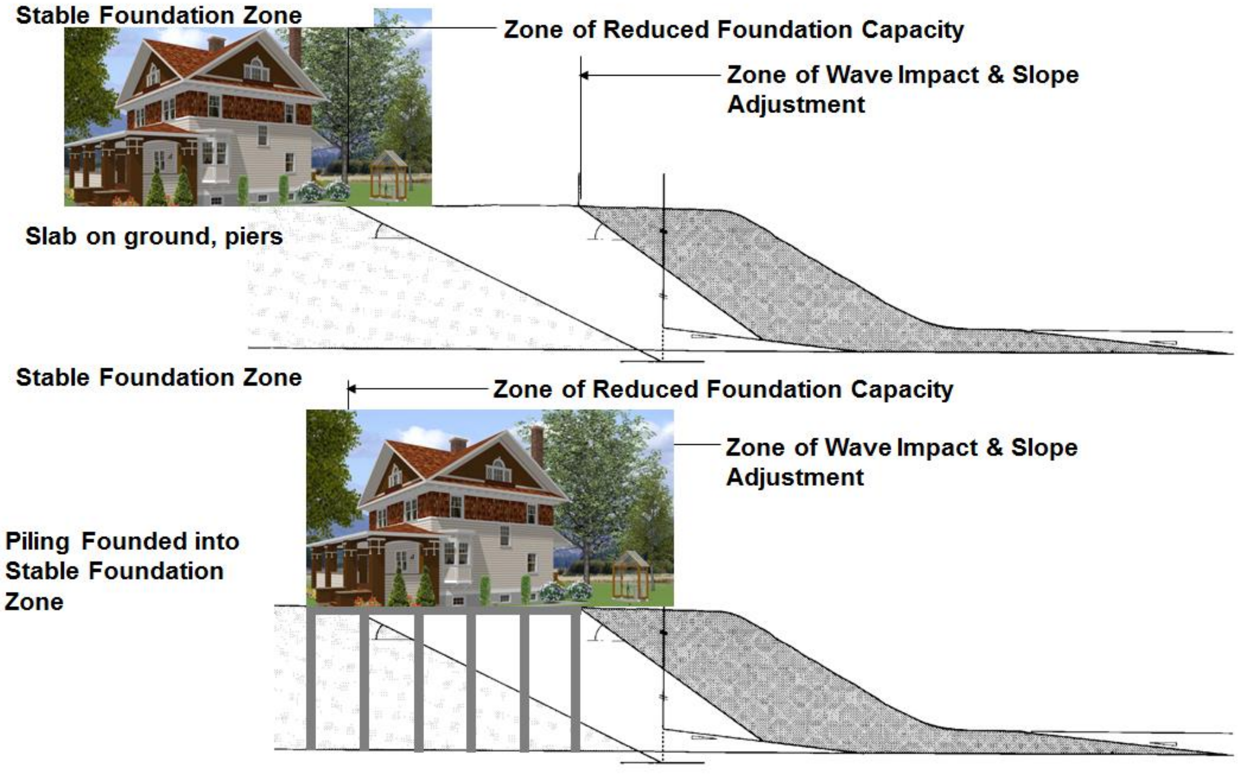


Figure 53 – Use of deep piled foundations to guarantee future structural integrity of new development in areas subject to future shoreline recession.

It is considered that setbacks are a viable management option to avoid the risk, particularly for erosion and long term recession, where they allow the development of a property to still occur (i.e. the setback is not so great as to render a property unable to be developed). Where the required setback under the local planning instrument would be so great as to render properties in a particular precinct undevelopable, alternative management strategies need to be considered for that precinct.

This class of management options can change the consequence of the hazard in a particular area. Examples of this type of approach are to relocate critical infrastructure landward where possible, or making modifications to existing infrastructure to reduce the quantum of damage that could occur following the design storm event.

5.10 Option 10 - Beach Nourishment

Beach nourishment could be considered for areas where beach volumes have been depleted due to storm erosion, or for areas suffering from long term recession. Beach nourishment would increase the existing beach width and nourish the nearshore seabed. The beach nourishment would create a greater buffer of sand to storm events. The existing sand management scheme for Beachport is a form of beach nourishment, with sand being moved from one beach compartment to another within the same littoral system. Bathymetric data show that there is a large volume of sand in the nearshore within Rivoli Bay beyond approximately 3 m depth. This source of sand could be accessed by dredging equipment to undertake beach nourishment should coastal erosion occur in the future through a reduction in onshore sediment supply to the Bay. Consideration would need to be given to the operation of a dredge in the high wave energy environment of Rivoli Bay. Testing of the sand would be required for compatibility with the native beach sand, as well as a rigorous environmental assessment.

5.11 Option 11 - Management of Inundation Risk

Inundation risk (such as for the low-lying areas fronting Beach Road) can be managed in the following ways:

- using construction materials that would not be adversely damaged by inundation, such as concrete floors;
- placing electrical equipment, wiring, or any other service pipes and connections that could be damaged by water at a suitably high level;
- storing goods or materials that could potentially be water damaged or water polluting at a suitably high level;
- using impact resistant construction materials in areas that may be subject to direct wave action; and,
- maintaining seawalls seaward of development at a suitably high crest level.

Such measures can be included as additional clauses in Council's Development Plan for new development within the coastal zone where appropriate.

At Beach Road in Beachport, the provision of a rock revetment in front of the existing seawall and/or the provision of a wave return wall along the promenade (Figure 54) may reduce the risk from overtopping but would not be expected to eliminate it completely. When storms occur, an appropriate emergency response plan should be developed for this area (such as roping off areas which present a danger to the public, temporary closure of the road to vehicular traffic and implementation of emergency protection works), which outlines clear responsibilities and triggers for action. Emergency response actions were put into place by Council officers at Beachport in response to the storm event of 24 June 2014 and it is considered that this response mitigated much of the potential damage that could have occurred as a result of that storm event.

In areas such as the Southend boat ramp carpark, the risk due to wave overtopping of the existing seawall may be mitigated by raising the crest level of the existing revetment, which would reduce the frequency and volume of wave overtopping into the carpark. Given the high storm water levels within Rivoli Bay as a result of wave setup it is not considered practical to eliminate overtopping completely, but it can be reduced significantly by raising the crest level of the existing revetment by at least 1 m. Warning signs can also be installed to inform people of the risk from wave overtopping in this area.

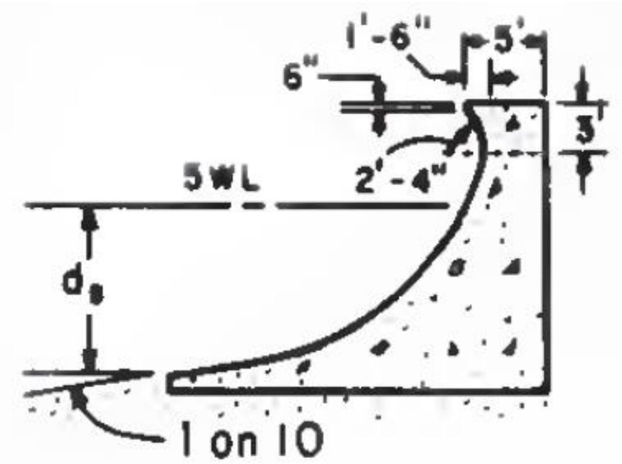


Figure 54 – Cross section of a typical wave return wall (Shore Protection Manual 1984).

5.12 Costing

Indicative cost estimates have been prepared for each of the aspects of the foreshore management options described above.

In the preparation of these cost estimates, consideration has been given to our experience gained from the completion of a number of similar coastal/maritime projects. In particular, we have drawn upon in-house costing information from relevant projects and supplemented this as-required through enquiries with representatives from local quarries and contractors that specialise in this type of work.

The indicative cost estimates for the proposed foreshore works are summarised in Table 5. The cost estimates incorporate 30% contingency and an allowance for detailed design where warranted. The cost estimate excludes GST, project management fees, authority approval fees and allowances for Contractor's risk. The cost estimate does not include allowance for design growth, escalation, procurement and construction management. These estimates are contained in Appendix 5 of this document. It should be noted that these cost estimates are indicative only and are likely to change based on the detailed design and variations in market forces.

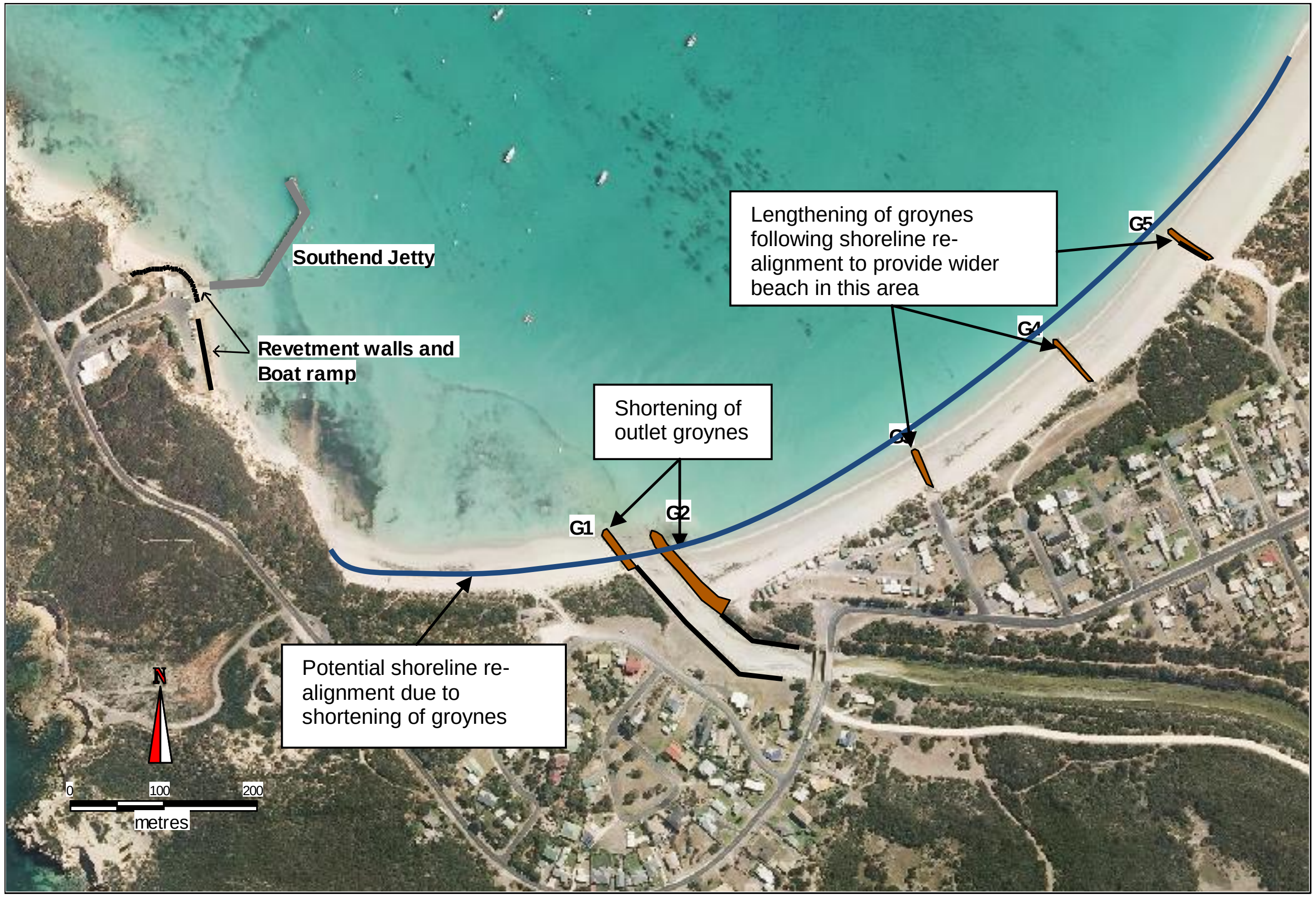


Figure 55 – Potential option to shorten Lake Frome outlet groynes at Southend

Table 5 – Cost estimates for potential coastal management options at Beachport and Southend

No.	Option	Timeframe (short, medium, long term)	Budget Estimate (AUD)
1	Extension of the groynes at the channel entrance of Lake George by 100 m	Medium term	$927,295
2	Provision of additional groyne at Beachport 80 m long	Medium	$455,418
3	Provision of rock revetment in front of the vertical timber seawall near the jetty	Medium	$992,836
4	Sand management at Southend (i.e. excavate from beach west of groynes and within channel and deposit east of lake outlet). Cost does not include existing sand management at Beachport	Short	$390,520
5	Additional sand management at Beachport if groynes at Lake George are extended	Medium	$390,520
6	Repair existing groynes and minor works to improve safety at downdrift side of groynes at Beachport	Short	$217,315
7	Shorten two of the five existing groynes at Southend, redistribute sand along beach, excavate sand from lake channel and vegetate dune. Rock can be reused to repair existing groynes at Beachport or top up revetment at Southend boat ramp (potentially saving up to $200,000 in material costs)	Short	$576,160
8	Extension of the groynes east of the channel entrance of Lake Frome by around 30 - 35 m each (total of 100 m)	Long	$674,898
9	Retreat of critical infrastructure /implementation of planning controls	As needed	N/A
10	Beach nourishment at Beachport (assuming 500 m length of foreshore to be nourished using dredged nearshore sand with 200m^3/m)	Long	$1,143,350
11	Management of inundation risk at Beachport and Southend by building concrete wave return wall at Beachport and raising revetment at Southend boat ramp by 1 m	Medium	$509,340

5.13 Management Options Appraisal

Each of the management measures described above have been ranked based on cost, with higher cost measures receiving a lower cost ranking.

Each of the management measures was then ranked based on the perceived environmental, social and public safety benefits as well as the perceived support for the option from key stakeholders based on the results of the community and stakeholder consultation. This score is subjective. For example, Option 10 “Management of inundation risk at Beachport and Southend” received a relatively high score of 7/10 for benefit, as the public safety aspect of this was considered to be significant given the inundation event that occurred in June 2014. Conversely, a lower score of 5/10 was assigned for Option 8 “Extension of the groynes east of the channel outlet” as the benefit of undertaking these works is not very clear.

Each of the management measures was then ranked based on the benefit scores from highest to lowest.

The ranking of each management measure for cost and benefit was then averaged, to obtain an overall rank for each option.

A recommended timeframe and priority for each measure was provided based on how urgent each of the measures was considered to be, and whether any particular measures need to be preceded by others.

The recommended management measures, budget estimate, timeframe, priority, cost ranking, benefit ranking and overall ranking are presented in Table 6 in order of overall rank, together with some justification for the scores assigned to each measure. The top five ranking options are highlighted in green in Table 6.

Table 6 – Options appraisal, timeframe, priority and budget estimates

No.	Option	Timeframe (short, medium, long term)	Budget Estimate (AUD)	Priority	Ranking (cost)	Score/10 (benefit – environmental, social, public safety)	Rank (benefit)	Overall rank	Comment
6	Repair existing groynes and minor works to improve safety at downdrift side of groynes at Beachport	Short	$217,315	High	1	7	3	1	These works to repair the damaged foreshores in the lee of the groynes would improve public safety at the existing groynes. Ongoing maintenance rather than upgrade of the groynes would be needed.
2	Provision of additional groyne at Beachport 80 m long	Short	$455,418	High	4	8	1	2	Would allow sand to build up in front of jetty area reducing erosion, improving beach amenity and reducing wave overtopping onto Beach Road
9	Retreat of critical infrastructure/implementation of planning controls	As required	N/A	High	N/A	7	3	3	Would only be expected to be required if an existing asset is at risk. Setbacks can be imposed on individual beachfront property owners if the use of their land is compatible with the coastal hazard e.g. Southend caravan park. Coastal hazard needs to be accurately quantified prior to implementing planning controls.
11	Management of inundation risk at Beachport and Southend by building concrete wave return wall at Beachport and raising revetment at Southend boat ramp by 1 m	Medium	$509,340	Medium-high	5	7	3	4	This would improve public safety but not eliminate overtopping completely. Works can be staged to save on cost (e.g. Beachport first, then Southend or vice-versa)
5	Additional sand management at Beachport if groynes at Lake George are extended	Long	$390,520	Low	2	6	7	5	This activity would be dependent on the extension of the groynes at the outlet to Lake George and is to be coordinated with the existing sand management scheme at Beachport
3	Provision of rock revetment in front of the vertical timber seawall near the jetty	Short	$992,836	High	9	8	1	6	Would reduce the wave erosion in front of the jetty area by allowing absorption of wave energy. Best undertaken in conjunction with additional groyne at northern side of outlet and sand placement at this compartment
1	Extension of the groynes at the channel entrance of Lake George by 100 m	Long	$927,295	Low	8	7	3	7	These works would improve the stability of the Lake outlet and reduce the frequency of dredging of lake channel. Would also allow sand to accumulate providing source of sand for placement at other eroded sections of the beach. Can be done at a later date
4	Sand management at Southend (i.e. excavate from beach west of groynes and within channel and deposit east of lake outlet).	Medium	$390,520	Medium	2	5	9	7	Would improve the erosion risk for the foreshore east of the Lake outlet. Would need to be an on-going activity and can be based on the existing management plan at Beachport. May not be very effective
7	Shorten two of the five existing groynes at Southend, redistribute sand along beach, excavate sand from lake channel and vegetate dune. Rock can be reused to repair existing groynes at Beachport or top up revetment at Southend boat ramp (potentially saving up to $200,000 in material costs)	Short	$576,160	High	6	6	7	9	Rock recovered from the existing groynes can be used to repair groynes at Beachport, construct new groynes and/or provide rock for extension of the entrance groynes at Lake George potentially saving up to $200,000 in material costs. Removal of the groynes at Lake Frome could impact land users upstream and would require ongoing sand management at the Lake outlet
8	Extension of the groynes east of the channel entrance of Lake Frome by around 30 - 35 m each (total of 100 m)	Long	$674,898	Low	7	5	9	10	Extend the groynes to the -2m AHD contour to improve stabilisation of the beach, but may not be very effective.
10	Beach nourishment at Beachport (assuming 500 m length of foreshore to be nourished using dredged nearshore sand with 200m³/m)	Long	$1,143,350	Low	10	5	9	11	Access nearshore sand store if long term beach recession occurs in the future and sand no longer moves onshore naturally. Would require dredging equipment to be mobilised - undertake mass sand nourishment in one large operation

6 CONCLUSIONS AND RECOMMENDATIONS

This report has documented a review of the effectiveness of beach management works on the foreshore of Rivoli Bay at Beachport and Southend and has investigated actions to improve the coastal management at the foreshores.

Erosion had been experienced in some areas with excessive accumulation of sand occurring in other areas. There has been significant wave damage occasioned to some of the rock groynes at both Beachport and Southend. Construction of rock groynes at Southend to help stabilise the beach and Lake outlet may have exacerbated erosion in some areas.

A detailed assessment of the coastal processes along various sections of the foreshore was carried out to develop a better understanding of the sediment transport pathways and to determine where sand can be sourced for remedial works without adversely impacting other areas along the foreshore.

An assessment of the coastal structures and rocky foreshores was undertaken through visual inspection and a review of the design parameters. It was found that the groynes and revetments were inadequate to withstand the design conditions as well as ineffective in stabilising some parts of the beach. Generally, the rock armour is too small and the groynes are too short.

Investigated actions to manage the foreshore mainly comprise a redistribution of sand from areas where sand has accumulated excessively to those areas where erosion has occurred. This could be undertaken exclusively using land-based equipment. Other actions canvassed involve construction of longer groynes at the outlet to Lake George, removal or shortening of the groynes at Southend and construction of an additional groyne at Beachport. Ongoing monitoring of the performance of the scheme would need to be undertaken, as some of the recommended actions (particularly future re-distribution of sand) may need to be repeated in the future. A framework for implementing planning controls for the coastal areas of Beachport and Southend is already in place. However, the degree of coastal hazard risk needs to be quantified prior to these controls being able to be implemented.

Budget estimates, timeframes, priority, assessment of relative benefits and overall ranking of each of the proposed measures have been carried out and is presented in Table 6.

7 REFERENCES

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GLOSSARY

Accretion	The accumulation of (beach) sediment, deposited by natural fluid flow processes.
ACES	A computer program, developed by the US Army Corps of Engineers, that is used to determine, among other things, levels of wave runup on natural beaches.
Aeolian	Adjective referring to wind-borne processes.
AHD	Australian Height Datum, approximately equal to mean sea level
ARI	Annual Recurrence Interval
Astronomical tide	The tidal levels and character which would result from gravitational effects, e.g. of the Earth, Sun and Moon, without any atmospheric influences.
Backshore	(1) The upper part of the active beach above the normal reach of the tides (high water), but affected by large waves occurring during a high. (2) The accretion or erosion zone, located landward of ordinary high tide, which is normally wetted only by storm tides.
Bar	An offshore ridge or mound of sand, gravel, or other unconsolidated material which is submerged (at least at high tide), especially at the mouth of a river or estuary, or lying parallel to, and a short distance from, the beach.
Bathymetry	The measurement of depths of water in oceans, seas and lakes; also the information derived from such measurements.
Beach profile	A cross-section taken perpendicular to a given beach contour; the profile may include the face of a dune or sea wall, extend over the backshore, across the foreshore, and seaward underwater into the nearshore zone.
Berm	A nearly horizontal plateau on the beach face or backshore.
Breaker zone	The zone within which waves approaching the coastline commence breaking, typically in water depths of around 2 m to 3 m in fair weather and around 5 m to 10 m during storms
Breaking depth	The still-water depth at the point where the wave breaks.
Chart datum	The plane or level to which soundings, tidal levels or water depths are referenced, usually low water datum.
Coastal processes	Collective term covering the action of natural forces on the shoreline, and the nearshore seabed.
CPB	Coastal Protection Board
Datum	Any position or element in relation to which others are determined, as datum point, datum line, datum plane.
Deep water	In regard to waves, where depth is greater than one-half the wave length. Deep-water conditions are said to exist when the surf waves are not affected by conditions on the bottom, typically in water depths of around 60 m to 100 m.
DEWNR	SA Department of Environment, Water and Natural Resources
DPTI	SA Department of Planning, Transport and Infrastructure
Dunes	Accumulations of wind-blown sand on the backshore, usually in the form of small hills or ridges, stabilised by vegetation or control structures.
Dynamic equilibrium	Short term morphological changes that do not affect the morphology over a long period.

Ebb tide	A non-technical term used for falling tide or ebb current. The portion of the tidal cycle between high water and the following low water.
Elevation	The distance of a point above a specified surface of constant potential; the distance is measured along the direction of gravity between the point and the surface.
Erosion	On a beach, the carrying away of beach material by wave action, tidal currents or by deflation.
Flood tide	A non-technical term used for rising tide or flood current. In technical language, flood refers to current. The portion of the tidal cycle between low water and the following high water.
Geomorphology	That branch of physical geography that deals with the form of the Earth, the general configuration of its surface, the distribution of the land, water, etc.
High water (HW)	Maximum height reached by a rising tide. The height may be solely due to the periodic tidal forces or it may have superimposed upon it the effects of prevailing meteorological conditions. Nontechnically, also called the high tide.
ICOLL	An acronym for Intermittently Closed or Open Lake or Lagoon
Inshore	(1) The region where waves are transformed by interaction with the sea bed. (2) In beach terminology, the zone of variable width extending from the low water line through the breaker zone.
Inshore current	Any current inside the surf zone.
Inter-tidal	The zone between the high and low water marks.
Littoral	(1) Of, or pertaining to, a shore, especially a seashore. (2) Living on, or occurring on, the shore.
Littoral currents	A current running parallel to the beach, generally caused by waves striking the shore at an angle.
Littoral drift	The material moved parallel to the shoreline in the nearshore zone by waves and currents.
Littoral transport	The movement of littoral drift in the littoral zone by waves and currents. Includes movement both parallel (long shore drift) and perpendicular (cross-shore transport) to the shore.
Longshore	Parallel and close to the coastline.
Longshore drift	Movement of sediments approximately parallel to the coastline.
Low water (LW)	The minimum height reached by each falling tide. Non-technically, also called low tide.
Mean high water (MHW)	The average elevation of all high waters recorded at a particular point or station over a considerable period of time, usually 19 years. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value. All high water heights are included in the average where the type of tide is either semidiurnal or mixed. Only the higher high water heights are included in the average where the type of tide is diurnal. So determined, mean high water in the latter case is the same as mean higher high water.
Mean high water springs (MHWS)	The average height of the high water occurring at the time of spring tides.
Mean low water (MLW)	The average height of the low waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

Mean low water springs (MLWS)	The average height of the low waters occurring at the time of the spring tides.
Mean sea level	The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings.
Morphology	The form of a river/estuary/lake/seabed and its change with time.
Nearshore	In beach terminology, an indefinite zone extending seaward from the shoreline well beyond the breaker zone.
Rip current	A strong current flowing seaward from the shore. It is the return of water piled up against the shore as a result of incoming waves. A rip current consists of three parts: the feeder current flowing parallel to the shore inside the breakers; the neck, where the feeder currents converge and flow through the breakers in a narrow band or "rip"; and the head, where the current widens and slackens outside the breaker line.
Runup	The rush of water up a structure or beach on the breaking of a wave. The amount of run-up is the vertical height above still water level that the rush of water reaches. It includes wave setup.
SBEACH	A computer program, developed by the US Army Corps of Engineers, that is used to determine, among other things, wave transformation across the surf zone, beach and dune erosion and levels of wave runup on natural beaches.
Setup	Wave setup is the elevation of the nearshore still water level resulting from breaking waves and may be perceived as the conversion of the wave's kinetic energy to potential energy.
SEWCDB	South Eastern Water Conservation and Drainage Board
Shoal	(1) (noun) A detached area of any material except rock or coral. The depths over it are a danger to surface navigation. (2) (verb) To become shallow gradually.
Shore	That strip of ground bordering any body of water which is alternately exposed, or covered by tides and/or waves. A shore of unconsolidated material is usually called a beach.
Shoreface	The narrow zone seaward from the low tide shoreline permanently covered by water, over which the beach sands and GRAVELS actively oscillate with changing wave conditions.
Shoreline	The intersection of a specified plane of water with the shore.
***Significant* wave**	A statistical term relating to the one-third highest waves of a given wave group and defined by the average of their heights and periods.
***Significant* wave height**	Average height of the highest one-third of the waves for a stated interval of time.
Spring tide	A tide that occurs at or near the time of new or full moon, and which rises highest and falls lowest from the mean sea level (MSL).
Storm surge	A rise or piling-up of water against shore, produced by strong winds blowing onshore. A storm surge is most severe when it occurs in conjunction with a high tide.
Sub-aerial beach	That part of the beach which is uncovered by water (e.g. at low tide sometimes referred to as drying beach).
Surf zone	The nearshore zone along which the waves become breakers as they approach the shore.
Swell	Waves that have traveled a long distance from their generating area and have been sorted out by travel into long waves of the same approximate period.

Tide	The periodic rising and falling of the water that results from gravitational attraction of the moon and sun acting upon the rotating earth. Although the accompanying horizontal movement of the water resulting from the same cause is also sometimes called the tide, it is preferable to designate the latter as tidal current, reserving the name tide for the vertical movement.
USACE	US Army Corps of Engineers