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WATTLE RANGE COUNCIL

Appendix 1 - Rivoli Bay

Assessment of Existing Coastal Structures

301015-03541 – 001

28 Oct 2015

Infrastructure

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PROJECT 301015-03541 - APPENDIX 1 - RIVOLI BAY

REV	DESCRIPTION	ORIG	REVIEW	WORLEY- PARSONS APPROVAL	DATE	CLIENT APPROVAL	DATE
A	Issued for internal review	<u> </u> C Adamantidis	<u> </u> A Nielsen	<u> </u> A Nielsen	25-Sep-14	<u> N/A </u>	
B	Draft	<u> </u> C Adamantidis	<u> </u> A Nielsen	<u> </u> A Nielsen	26-Sep-14	<u> </u>	
C	Final	<u> </u> C Adamantidis	<u> </u> A Nielsen	<u> </u> A Nielsen	28-Oct-15	<u> </u>	
		<u> </u>	<u> </u>	<u> </u>		<u> </u>	



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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. It should be noted that since that time, some of the structures have been upgraded or lengthened with additional rock armour, particularly the groynes north of the jetty at Beachport. However, this Appendix documents the structures as they existed at the site inspection 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.

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

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



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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.

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 near-vertical 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, sloping 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, 2011).

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.

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 (2011) 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.



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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.

1.3 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 below.



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2 BEACHPORT

The groynes and coastal structures at Beachport were inspected and observations about their performance and characteristics is provided below. The groynes and their associated beaches have been numbered according to the numbering convention used by Wattle Range Council, moving from south to north, as depicted in Figure 1 for Beachport.

2.1 Groyne 2

Groyne 2 is the southernmost groyne along the Beachport foreshore located immediately north of Glen Point. A view along the trunk of Groyne 2 is provided in Figure 2.

The groyne appeared to be armoured with limestone rock, with a median diameter of 500 mm at the tip of the groyne. Littoral drift was bypassing the groyne. At the landward end of the groyne, larger limestone rock armour of median diameter 1000 mm had been placed on the downdrift side. The core of the groyne was exposed in some places with smaller rock fragments visible through the gaps between the larger armour stones. The groyne was observed to be in a poor to fair condition. Sand had built up on the updrift side of the groyne and dune vegetation had become established on top of the groyne as can be seen in Figure 2.



Figure 1 – Groyne and beach identifiers used by Wattle Range Council, Beachport



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Figure 2 – Groyne 2 looking out from downdrift side



Figure 3 – Groyne 2 looking south toward Glen Point



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The dune between Glen Point and Groyne 2 was accreted, despite the presence of an erosion scarp having formed during the storm of 24 June. The dune was relatively well vegetated on both the updrift and downdrift sides of the groyne. Littoral drift was actively transported by wave action from around Glen Point, and around the tip of Groyne 2, as indicated in Figure 4 and Figure 5. There was also some minor erosion observed as a result of an informal access on the downdrift side of the groyne.



Figure 4 – Sand transport from south to north around Glen Point, 25 June



Figure 5 – Sand transport around tip of Groyne 2, 25 June



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2.2 Groyne 3

Groyne 3 was inspected on 25 June 2014 and was considered to be in poor condition. The groyne was covered with sand on the updrift side and was actively bypassing (Figure 6). The groyne comprised limestone armour rock with a median diameter of 1000 mm, but many smaller core/underlayer rocks were observed to have been exposed and the armour has slumped along sections of the groyne trunk (Figure 8). The groyne appeared to have a clay core which has become exposed and devoid of armour (Figure 9). On the downdrift side of the structure, at the landward end of the groyne severe erosion had occurred which is threatening to outflank the groyne on the landward side (Figure 7). Ad hoc rubble and concrete pieces were observed to have been placed at the base of this erosion escarpment, possibly in a previous attempt to arrest the erosion at this location. Given the height of the erosion escarpment on the landward edge of the groyne, it is considered that there is a risk of the erosion scarp collapsing and an associated risk of public injury. The dune within Beach 3 on the downdrift side of the groyne was observed to be suffering from severe erosion, with an erosion scarp around 1.5 m high, formed possibly as a result of the storm event on 24 June (Figure 10).



Figure 6 – Tip of Groyne 3 covered with sand showing sand transport direction



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Figure 7 – Groyne 3 showing eroded area and ad-hoc rubble at downdrift landward side, 25 June



Figure 8 – Groyne 3 looking landward. Note armour rock has slumped along trunk of groyne; clay core and smaller underlayer rock has been exposed and core has been significantly eroded (25 June 2014)



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Figure 9 – Close view of trunk of Groyne 3, showing exposure of small underlayer rock and clay core (25 June)



Figure 10 – View of Beach 3 downdrift of Groyne 3, showing eroded dune (25 June)



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2.3 Groyne 4

Groyne 4 was observed on 25 June 2014 and was considered to be in poor condition. The groyne was covered with sand on the updrift side and was actively bypassing (Figure 11 and Figure 12). Waves were observed to be breaking directly onto the groyne head. The groyne comprised limestone armour rock with a median diameter of 1000 mm, but many smaller core/underlayer rocks were observed to have been exposed, especially on the groyne trunk. There was an eroded dune escarpment on the updrift side approximately 1.5 m high. Rock armour was observed to have been dislodged from the groyne, and the armour appeared to form a very steep side slope on the groyne of approximately 1V:1H (Figure 13). As is the case with Groyne 3, the groyne appeared to have a clay core which has become exposed and devoid of armour (Figure 14). Sand has accumulated within the dune on the updrift side of the groyne, with dune vegetation growing on top of the landward side of the groyne (Figure 14). The dune within Beach 4 on the downdrift side of the groyne was observed to be in good condition, with no erosion scarp visible, despite the occurrence of a severe storm event on the previous day and erosion occurring on the surrounding beaches (Figure 15).



Figure 11 – Tip of Groyne 4 showing sand bypassing over the groyne and dislodged armour stones



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Figure 12 – View of Groyne 4 from updrift side showing sand bypassing over groyne



Figure 13 – View of Groyne 4 trunk from downdrift side, showing steep side slopes, dune vegetation and exposed small armour stones



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Figure 14 – View of Groyne 4 trunk from updrift side showing eroded clay core and dune vegetation



Figure 15 – View of Beach 4 looking updrift toward Groyne 4 showing vegetated dune and beach monitoring pole



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2.4 Groyne 5

Groyne 5 was inspected on 25 June 2014 and was considered to be in good condition. The armour appeared to be well packed with good interlocking between individual stones (Figure 16). The groyne comprised rock armour of median diameter 1200 mm near the tip of the groyne, with a large vegetated dune and build up of sand within Beach 4 on the updrift side. Along the groyne trunk on the downdrift side, smaller rock armour of median diameter 800 mm was visible and erosion was occurring at the landward end of the groyne (Figure 17). There was evidence of breakage of individual armour stones within the groyne, as can be seen in Figure 18. A timber pedestrian walkway extended down onto Beach 5 and this was damaged in the large wave event of 22 June. The crest of the groyne was covered in road base to provide vehicular access to the end of the groyne and beach. Inundation due to wave runup was observed to be impacting on the landward side of this groyne during the large wave event of 24 June (Figure 19).

This groyne was the longest groyne of those on the Beachport foreshore, estimated to be 120 m in length from the aerial photography. At the end of this groyne, a submerged geotextile container breakwater had been constructed in the late 1990's in an attempt to protect the seagrass beds in the lee of the breakwater (i.e., offshore of Beach 5). This breakwater has been monitored by DEWNR between 2004 and 2009 and significant subsidence of the breakwater was noted. However, during the field inspection the breakwater was not visible.

Significant erosion had occurred within the dune at Beach 5, with a two-metre high erosion escarpment visible along the entire length of this beach (Figure 20).



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Figure 16 – Tip of Groyne 5 showing good interlock between armour stones and large sand buildup on updrift side



Figure 17 – Groyne 5 looking along trunk of groyne. Note erosion of road base at crest of groyne and windblown dune sand on groyne trunk



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Figure 18 – Armour stones on Groyne 5 – note breakage of individual armour stones



Figure 19 – Wave runup onto landward end of groyne 24 June 2014



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Figure 20 – Pedestrian accessway, small armour stones and eroded dune escarpment at Beach 5

2.5 Groyne 6 and Groyne 7

Groyne 6 was inspected on 25 June 2014 and was under construction at the time. The groyne extends shore normal for approximately 30 m but then extends shore-parallel for approximately 40 m. The shore-parallel section is currently being extended to provide wave protection for the adjacent boat ramp. The median diameter of armourstones used for the groyne was approximately 1200 mm. The crest of the groyne was covered with crushed rock, to provide access for construction vehicles for placement of rock at the tip of the groyne (Figure 21).

During the inspection on 24 June, waves were observed to be overtopping the shore-parallel section of groyne and swell waves were observed within the boat ramp area (Figure 22).

On the downdrift side of the boat ramp and groyne, a rock revetment extended along the shore for approximately 200 m toward the jetty. This revetment appeared to be in good condition, laid at a slope of 1V:1.5 and with rock armour of median diameter of 1000 mm. Wave overtopping was observed during the storm event of 24 June – with some smaller armour stones having been dislodged and transported into the reserve behind the revetment (Figure 23). The geotextile filter and basalt underlayer rock of median diameter 50 mm was visible in places through the face of the revetment (Figure 24). Several sink holes indicative of wave overtopping damage were observed behind the crest of the revetment (Figure 25). The revetment extended over the location of the former Groyne 7 which is visible in aerial photography prior to 2005. That groyne has since been removed, but a bend in the revetment exists at the former location of the groyne. Erosion was observed in the unprotected fill above the crest of the revetment at the location of the former Groyne 7, caused by



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wave overtopping and rainfall (Figure 26). Beaches 6 and 7 were completely denuded of sand and waves were observed to be breaking directly onto the revetment (Figure 27). At the northern end of this revetment, a timber pedestrian access extended onto the beach and the rock revetment was backed by a vertical timber seawall (Figure 28).



Figure 21 – Looking down trunk of Groyne 6 under construction



Figure 22 – Wave overtopping partially constructed Groyne 6 into boat ramp area, 24 June



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Figure 23 – Evidence of wave overtopping with strewn rock debris thrown up behind revetment



Figure 24 – Geotextile and small underlayer rock visible within armour interstices at rock revetment



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Figure 25 – Cavities behind revetment wall and visible small basalt underlayer rock



Figure 26 – Erosion of unprotected fill at crest at bend in revetment at former Groyne 7 location



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Figure 27 – Direct wave attack onto rock revetment at Beachport, Groyne 6 and boat ramp in background



Figure 28 – Wave attack onto revetment, timber seawall and pedestrian access at northern end of revetment wall, 24 June.



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2.6 Groyne 8

Groyne 8 is a relatively short rock groyne located adjacent to the jetty near the town centre. This groyne comprised large limestone rock armour of median diameter approximately 1500 mm and was observed to be in fair condition. A small fillet of sand was visible on the updrift side of the groyne (Figure 29). The tip of the groyne was submerged and rock armour from the groyne tip appeared to have been dislodged (Figure 30). The groyne was under severe wave attack on the day of the site inspection. The length of this groyne has been estimated from aerial photography at around 30 m.

The timber seawall on the landward side of this groyne had been undermined and was severely damaged in the storm event of 24 June (Figure 31, Figure 32). The clay core of the groyne was exposed on the landward side of the groyne, and was subject to direct wave attack.

On the downdrift side of the groyne, a vertical timber seawall was observed to be under direct wave attack and wave energy was being reflected out to sea. There was no sandy beach in this area, as a result of wave energy being reflected from the vertical seawall. Waves were observed to overtop this seawall and Beach Road in the vicinity of the jetty was subject to severe coastal inundation and wave overtopping on June 24 (Figure 33). Temporary closure of the road was required during this wave inundation event. The timber jetty was damaged by wave attack and required repair during the site inspection.

North of the jetty and at the northern end of the timber seawall, severe erosion had undermined the concrete footpath along the foreshore and damaged stormwater infrastructure on 24 June (Figure 34). Council had installed emergency protection works comprising geotextile sandbags and large limestone rock armour with median diameter 1500 mm in an attempt to protect Beach Road from further erosion damage (Figure 35).



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Figure 29 – Groyne 8 showing exposed clay core



Figure 30 – Submerged tip of Groyne 8 showing dislodged rock armour



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Figure 31 – Groyne 8 showing direct wave attack and adjacent undermined timber seawall



Figure 32 – Damaged and undermined timber seawall and erosion



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Figure 33 – Wave overtopping at Beach Road 24 June



Figure 34 – Undermined concrete footpath at northern end of timber seawall



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Figure 35 – Emergency protection works installed 24 June comprising large rock armour and geotextile sandbags

2.7 Groyne 9

Groyne 9 is located south of the jetty. A small beach dune had formed on the updrift side of the groyne between the groyne and the vertical timber seawall adjacent to the jetty, but this dune had been damaged by erosion and a 0.5 m high erosion escarpment had formed here as a result of the June 24 storm event.

The groyne is approximately 60 m long and consists of limestone rock armour with median diameter of 800 mm. Within this groyne, smaller underlayer rock with median diameter of 200 mm was observed (Figure 36). The groyne was observed to be in poor condition, with dislodged rock armour observed on the beach berm on both sides of the groyne. The rock armour along the groyne trunk was observed to be very steep with a slope of 1V:1H.

On the landward end of the northern (downdrift) side of the groyne, a 1.5 m sandy erosion scarp was evident along the landward side of the groyne trunk (Figure 37). Along the Beach 9 foreshore north of the groyne, a dilapidated timber sheet pile structure was observed at the base of the erosion escarpment (Figure 37). The concrete foreshore path in this area had been undermined by erosion with large rock armour (median diameter 1000 mm) placed as emergency coastal protection under the undermined path (Figure 38).



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Figure 36 – Groyne 9 showing mix of small and large armour stones

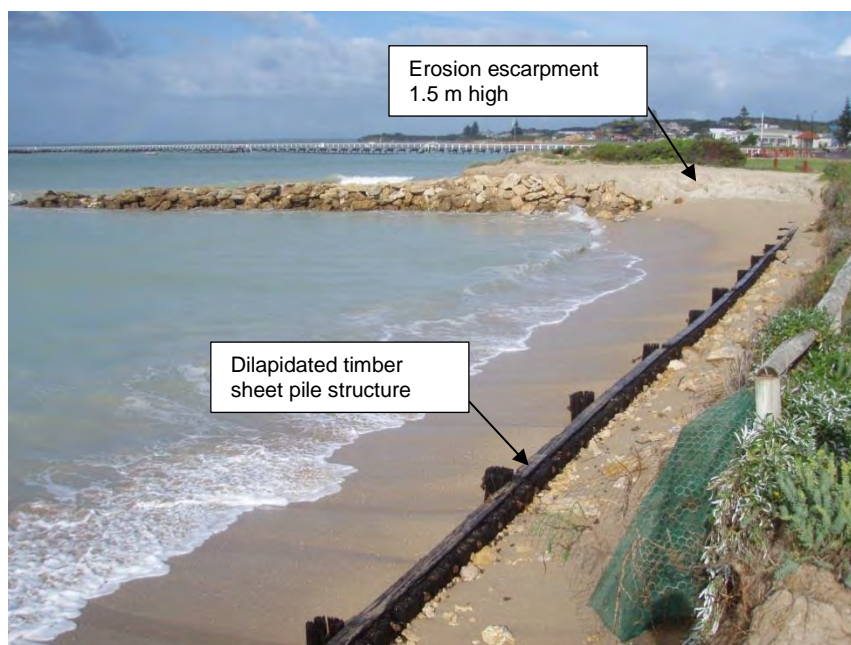


Figure 37 – Looking south toward Groyne 9 showing timber sheet pile wall and erosion escarpment



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Figure 38 – Undermined concrete path along Beach 9 north of Groyne 9 with emergency rock protection

2.8 Groyne 10

Groyne 10 was a relatively short rock groyne (approximately 40 m long) in fair condition (Figure 39). The median diameter of rock armour within this groyne was estimated to be around 1000 mm, with the rock armour comprising limestone. Concrete pieces were observed within the armour of the northern side of the groyne. The tip of the groyne was submerged and not visible. The groyne was partially covered in sand and was actively bypassing (Figure 40). Significant gaps between individual armour stones were visible, these gaps being filled with sand. The beach on the updrift (southern) side was backed by a vegetated dune which had been damaged by erosion, with a dune escarpment 1.5 m high.

A vertical concrete block seawall approximately 50 m long was located immediately north of and adjacent to the groyne. This concrete block seawall was subject to wave overtopping during the storm event of 24 June but otherwise appeared to be in good condition with little movement noticeable in the seawall (Figure 41 and Figure 42). North of the concrete seawall, a vegetated dune extended along the beach to Groyne 11, with this dune in good condition and little erosion evident. The dune was backed by a timber boardwalk which appears to be in good condition.

A stormwater pipe of diameter 200 mm was observed to be discharging at the landward side of the groyne but the exit to the pipe was surrounded by rock armour and no visible erosion damage was observed around the discharge location.



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Figure 39 – Groyne 10 looking south 24 June



Figure 40 – Groyne 10 tip



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Figure 41 – Seawall adjacent to Groyne 10 indicating extent of overtopping onto Beach Road



Figure 42 – Vertical concrete block seawall adjacent to Groyne 10 looking north



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2.9 Groyne 11

Groyne 11 is a relatively short groyne approximately 30 m long (Figure 43). The primary rock armour comprised limestone pieces with a median diameter estimated at 600 mm. Concrete pieces were observed within the armour. Numerous smaller pieces of armour rock were observed on the crest of the groyne, with these armour stones having a median diameter of around 300 mm.

The groyne was in poor condition with many gaps within the armour and exposure of the clay core at the landward side of the groyne. Armour rock had been dislodged from the main trunk of the groyne. Despite the poor condition of the groyne, it was effective in stabilising this section of beach, with a healthy vegetated dune on both sides of the groyne.



Figure 43 – Groyne 11 looking south 25 June



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Figure 44 – Groyne 11 looking inland showing exposed clay core and dislodged armour rock

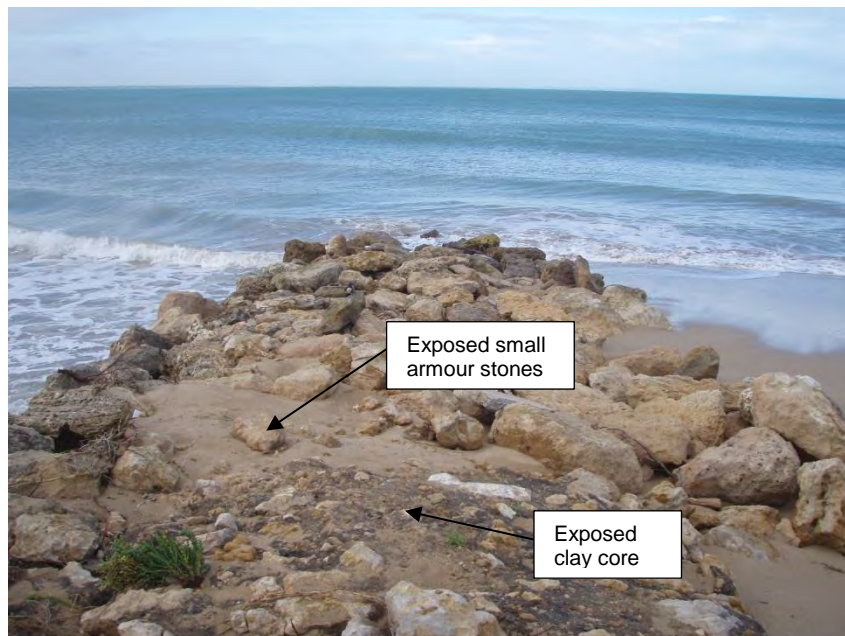


Figure 45 – Groyne 11 tip



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2.10 Groyne 12 and Lake George outlet

Groyne 12 is located on the southern side of the outlet to Lake George and acts as a training wall for the outlet channel (Figure 46). The groyne is estimated to be 50 m long and was in poor condition. Rock armour comprised limestone with a median diameter of 800 mm. There were significant gaps in the rock armour around the tip of the groyne, on both the northern and southern sides of the groyne (Figure 47).

Despite the poor condition of the groyne, the dune on the southern side of the groyne was in good condition, with a wide vegetated foredune and little evidence of erosion. The beach berm had reached the tip of the groyne, indicating that the groyne was bypassing sand into the outlet channel of Lake George. Dune sand had built up on the updrift side of the groyne and carried into the Lake outlet channel by wind action (Figure 48).

On the northern side of the Lake outlet a short rock groyne approximately 10 m long acts as a training wall (Figure 49). This groyne was in fair condition with a median armour diameter of 1000 mm. North of the groyne, the beach dune was in good condition.

At the Lake George outlet, waves were propagating upstream into the channel beyond the road bridge and this was evident particularly during high tide on 24 June (Figure 50). Sand was in suspension being carried upstream into the Lake George channel (Figure 51). Corrosion of the concrete reinforcement and associated spalling of the concrete within the outlet structure piers (Figure 52 and Figure 53) and along the capping beam of the outlet channel (Figure 54) was evident, which could lead to future structural failure of these elements. At the road bridge, corrosion of the steel sheet pile bridge abutments (Figure 55) was evident which should be inspected and tested by a structural engineer.



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Figure 46 – Groyne 12 looking north



Figure 47 – Tip of Groyne 12 – note loss of armour from groyne trunk and exposure of clay core



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Figure 48 – Groyne 12 looking south from Lake George outlet – note build up of dune sand being carried into channel by wind



Figure 49 – Short rock groyne at northern end of Lake George outlet – Lake George outlet structure in background



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Figure 50 – Waves propagating upstream into Lake George channel



Figure 51 – Sand in suspension being carried by waves and tidal currents upstream into Lake George channel



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Figure 52 – Lake George outlet structure with corrosion of concrete reinforcement



Figure 53 – Lake George outlet structure pier with corrosion of concrete reinforcement and concrete spalling



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Figure 54 – Concrete spalling at capping beam along Lake George outlet channel upstream of outlet structure



Figure 55 – Corrosion of sheet pile bridge abutment at road bridge upstream of Lake George outlet structure



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3 SOUTHEND

The groyne and coastal structures at Southend were inspected on 25 June 2014 and observations about their performance and characteristics is provided below. The groyne and their associated beaches have been numbered as depicted in Figure 56 for Southend.

3.1 Revetment walls near Southend boat ramp

There were two separate revetment walls inspected at Southend near the boat ramp. The revetment wall immediately surrounding the jetty was in good condition, was constructed at a slope of 1V:2H and appeared to include a mix of basalt and limestone armour rock (Figure 57). The revetment was curved with one side facing north and the other facing east. The northern face was subject to direct wave attack from waves diffracted around Cape Buffon, whereas the eastern face was subject to oblique waves. While this revetment was in good condition, it was subject to wave overtopping during the storm event of 24 June (Figure 58), with small armour stones and gravel being thrown into the carpark area (Figure 59).

The concrete surface of the boat ramp had suffered some spalling and cracking (Figure 60) and waves were running up the surface of the boat ramp into the carpark on 24 June (Figure 61).

South of the boat ramp, a rock revetment comprising limestone armour of median diameter 1200 mm was subject to oblique wave attack – this revetment was at a slope of 1V:2H and was in poor condition, with the underlying fill material exposed and subject to erosion caused by wave overtopping (Figure 61 and Figure 62). At the southern end of the revetment, rock armour was replaced with ad hoc concrete and building rubble, as shown in Figure 63.



Figure 56 – Groyne structures and identifiers at Southend



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Figure 57 – Sandstone and basalt rock armour revetment near Southend jetty



Figure 58 – Wave overtopping onto revetment at Southend jetty, 24 June



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Figure 59 – Gravel debris indicating extent of wave overtopping in jetty carpark, Southend



Figure 60 – Cracking and spalling of boat ramp surface, Southend



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Figure 61 – Wave runup onto Southend boat ramp, 24 June



Figure 62 – Boat ramp and rock revetment looking south, Southend



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Figure 63 – Ad hoc rubble erosion protection at southern end of revetment near boat ramp

3.2 Groyne 1

Groyne 1 forms a training wall at the western side of the outlet to Lake Frome. The groyne is in poor condition, with damaged rock armour and slumping of the armour (Figure 64). Generally, armour rock was limestone with a median diameter of 800 mm. Smaller armour rock with a median diameter of 500 mm had been exposed, particularly on the eastern side of the groyne within the Lake outlet channel.

The groyne had an exposed clay cap with severely eroded edges, and miscellaneous concrete rubble used as armour on the groyne tip (Figure 65). Slumped armour and exposure of the clay core of the groyne was observed on both sides of the groyne (Figure 66 and Figure 67), although the damage was more severe on the eastern side which is more exposed. An attempt had been made to repair this damage with ad-hoc concrete rubble (Figure 68).

Sand had built up on the western side of the groyne to the groyne tip, and a wide vegetated dune has formed in this area. The built-up sand was considered to be mobile and sand may be being blown into the channel of Lake Frome over the top of the groyne (Figure 68).



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Figure 64 – Groyne at southern end of Lake Frome outlet. Note buildup of sand, slumped armour and use of concrete pieces as armour.



Figure 65 – Tip of groyne at southern end of Lake Frome outlet. Note use of concrete rubble as armour.



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Figure 66 – Tip of groyne at southern side of Lake Frome outlet. Note eroded core and groyne cap as well as exposure of small armour stones on southern side of groyne



Figure 67 – Groyne at southern side of entrance to Lake Frome. Note eroded clay cap and exposure of smaller armour stone on northern side of groyne.



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Figure 68 – Groyne at southern side of entrance to Lake Frome, looking inland. Note poor condition of clay capping and core on northern side, use of concrete rubble to repair erosion damage and groyne being covered by mobile dune sand

3.3 Groyne 2

Groyne 2 forms a training wall at the eastern side of the outlet to Lake Frome. The groyne is in very poor condition, with damaged rock armour and slumping of the armour observed along the entire groyne length (Figure 69). The primary rock armour was generally limestone which had a median diameter of around 1200 mm but large areas of exposed core rock were visible with median diameter 200 mm. The clay core and capping of the groyne was severely eroded especially on the more exposed northern side of the groyne (Figure 70). Pieces of building rubble were within the groyne armour at several places along the groyne.

Severe dune erosion was observed on the northern side of the groyne – this erosion was threatening to undermine several cabins within the caravan park on the northern side of the lake outlet (Figure 71). Debris was observed on the beach downdrift of the groynes which could pose a danger to the safety of beach users (Figure 72).

Waves were observed to be breaking directly onto the groyne during the site inspection (Figure 73).



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Figure 69 – Severe erosion on northern side of groyne at northern side of Lake Frome outlet. Note exposure of small core rock and erosion of the clay core.



Figure 70 – Groyne at northern side of Lake Frome outlet. Note severe erosion of groyne trunk on northern side and exposure of small armourstones.



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Figure 71 – Erosion at dune downdrift (north) of Lake Frome outlet threatening cabins



Figure 72 – Eroded foreshore and exposed debris



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Figure 73 – Waves breaking at groyne on northern side of Lake Frome outlet

3.4 Groyne 3

Groyne 3 comprised a mix of limestone and basalt armour rock with a median diameter of 1200 mm. The groyne was in relatively good condition, approximately five metres wide, with good interlocking between individual armour stones (Figure 74). This groyne has a lookout facility and two sets of timber stairways leading down onto the beach on either side of the groyne. The foreshore escarpment on the landward side of this groyne is relatively high, affording views of the beach, jetty and the surrounding area (Figure 75). The foreshore on both sides of this groyne has been impacted severely by erosion and is characterized by a steep dune escarpment.

A timber stairway beach access located along the beach between Groynes 2 and 3 had been damaged severely by waves (Figure 76).



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Figure 74 – Groyne 3 – note armour in good condition and interlocking between individual stones



Figure 75 – View of Groyne 3 looking south. Note timber stairway and lookout at crest of groyne.



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Figure 76 – Damaged pedestrian access onto beach south of Groyne 3

3.5 Groyne 4

Groyne 4 comprises a groyne with limestone primary armour with a median diameter of 1200 mm. The groyne is approximately 5 m wide, and is bounded on the southern side by a vertical timber face. While the groyne appeared to be in good condition, numerous smaller armourstones with a median diameter of 200 mm were exposed and some armour stones appear to have been dislodged and deposited on the southern side of the groyne (Figure 77). There was no formal beach access available at this location.

The groyne was subject to severe wave attack and the tip of the groyne was being severely overtopped on the day of the site inspection. Although the groyne appeared to be in good condition, the groyne did not appear to be very effective as there was no significant buildup of sand on either side of the groyne. A steep vegetated dune escarpment was evident on both sides of the groyne. Rock armour had been placed at the toe of the dune escarpment on the northern side of the groyne which appears to have been effective in reducing erosion of the dune (Figure 78).



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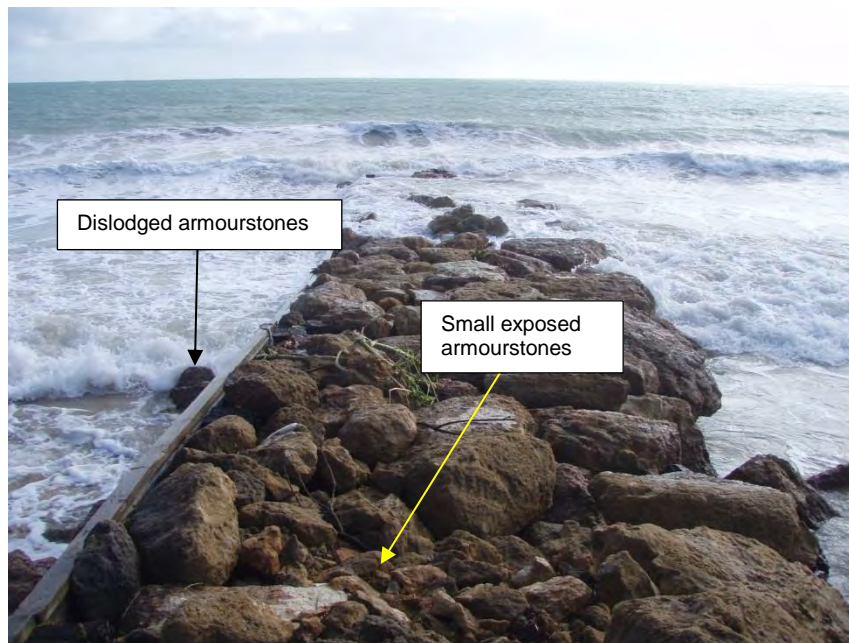


Figure 77 – Groyne 4 – note exposure of smaller armourstones, overtopping of the tip of the groyne and dislodgement of some smaller armourstones on the southern side of the groyne.



Figure 78 – Looking landward toward beach face. Note rock protection installed at toe of dune and lack of formal access onto the beach at this location.



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3.6 Groyne 5

Groyne 5 is the northernmost groyne along the Southend foreshore. This groyne consisted of basalt rock armour with a median diameter of 1200 mm. The groyne was bounded on its southern side by a timber wall (Figure 79). The tip of the groyne was in good condition, with good interlocking between individual armourstones and few visible signs of movement (Figure 80).

Vehicle access onto the beach in this location has been undermined and was closed off at the time of the site inspection (Figure 81). Despite the groyne being undermined on its landward side, the groyne was in fair condition. Some gaps in the armour were observed within the trunk of the groyne (Figure 82). Erosion at the landward end of the northern side of the groyne was severe at the time of the site inspection, threatening to outflank the groyne.

Immediately south of this groyne, a building within the Rivoli Bay Sailing Club area was close to the edge of the erosion escarpment (Figure 83).



Figure 79 – Groyne 5



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Figure 80 – Tip of groyne 5



Figure 81 – Undermining of Groyne 5 at landward side



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Figure 82 – Looking landward along Groyne 5



Figure 83 – Building at Rivoli Bay Sailing Club close to dune escarpment



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4 SUMMARY

In general, the groynes and revetments at Beachport and Southend were in poor condition and do not appear to 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.

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.



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5 REFERENCES

USACE (2011), "Coastal Engineering Manual – Part VI, EM 1110-2-1100, U.S. Army Corps of Engineers, September.



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Appendix 2 - Rivoli Bay Wave Transformation Modelling

301015-03541 – 001

28 Oct 2015

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PROJECT 301015-03541 - APPENDIX 2 - RIVOLI BAY

REV	DESCRIPTION	ORIG	REVIEW	WORLEY- PARSONS APPROVAL	DATE	CLIENT APPROVAL	DATE
A	Issued for internal review	C Adamantidis	A Nielsen	A Nielsen	25-Sep-14	N/A	
B	Draft	C Adamantidis	A Nielsen	A Nielsen	26-Sep-14		
C	Final	C Adamantidis	A Nielsenq	A Nielsen	28-Oct-15		



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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.

The model results were used to assess the armour stability for the existing structures – this has been done in the main report.



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2 WAVE MODELLING

2.1 Modelling Approach

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.

SWAN is a third generation phase averaged two-dimensional (horizontal – 2D(H)) wind wave model and is capable of simulating a range of physical processes such as:

- Wave generation by wind
- Shoaling
- Refraction due to current and depth
- Diffraction
- Three and four wave interactions
- Wave dissipation due to white capping, bottom friction, and depth induced breaking.

Wave diffraction around Cape Buffon and Penguin Island is expected to be an important process in the transformation of offshore waves to the site. While diffraction is not able to be modelled explicitly in phase averaged wave models such as SWAN, which has been applied to this study, recent advances include diffraction by utilising a phase-decoupled refraction-diffraction approximation. This approach has been shown to give realistic approximations to wave diffraction around large obstacles such as headlands as is considered here. The diffraction approximation has been applied to the SWAN modelling described in this section.

To transform the offshore waves to the site, three separate computational grids were used, these being a fairly coarse grid covering the entire Rivoli Bay area with a grid spacing of 200 metres and two nested grids with resolutions of 40 m covering the areas around Beachport and Southend.

The extent of these grids overlaid on the model bathymetry is depicted in Figure 1 and a numerical description of the three grids is given in Table 1. The finest resolution grid of 40 m is sufficiently fine to represent the scale of the wave transformation processes, given that the wavelength of the waves being simulated is greater than 100 m.



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Bathymetric data used in the model was sourced from navigational charts of the area and bathymetric surveys carried out by the Department of Environment, Water and Natural Resources (DEWNR).

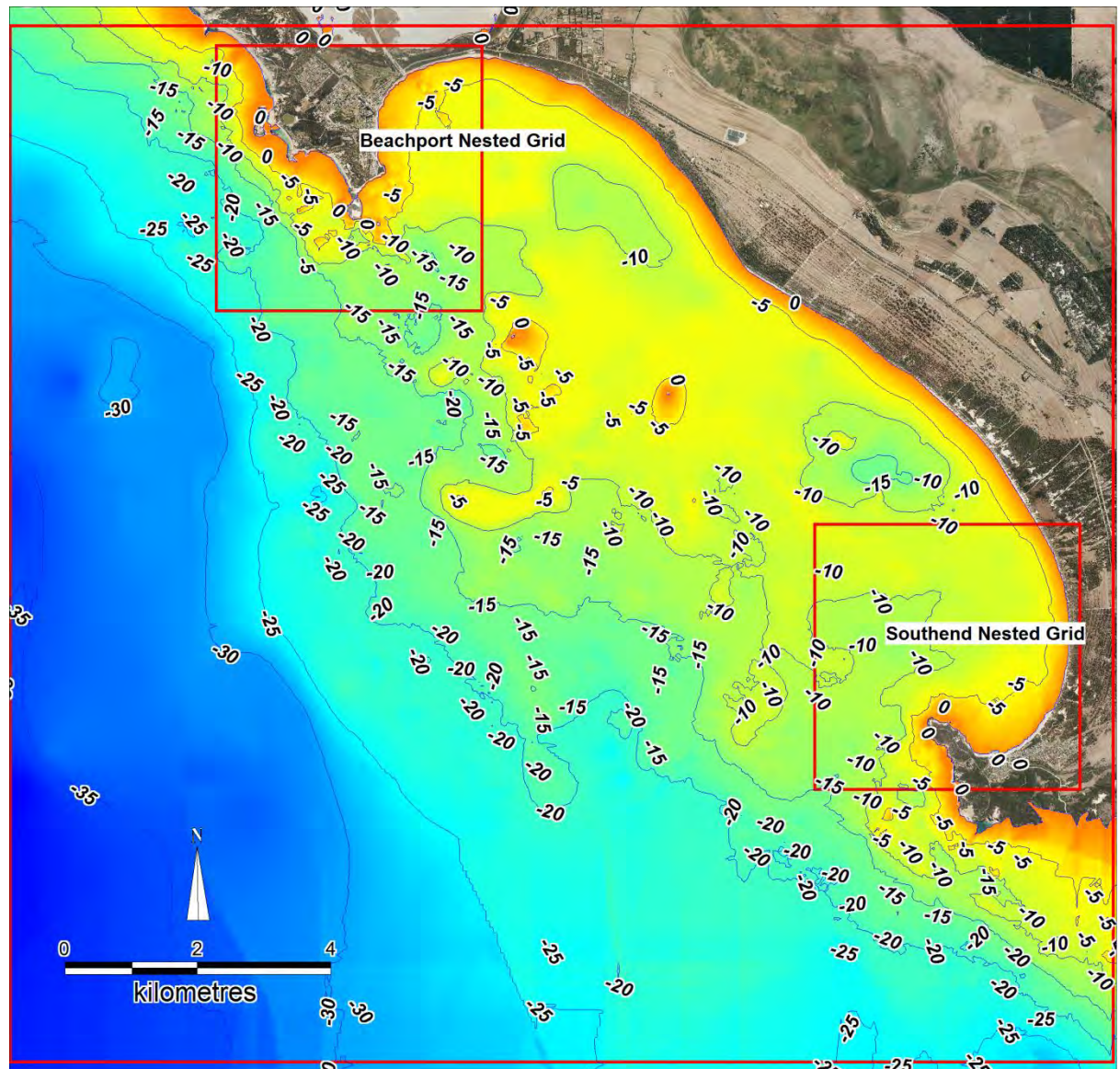


Figure 1 - Model bathymetry and extent of the computational grids



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Table 1: Numerical SWAN grids

Grid	Rivoli Bay	Beachport	Southend
Origin (MGA 54)	E407400, N5836900	E410500, N5848200	E419500, N5841000
Length in X direction (m)	16,600	4,000	4,000
Length in Y direction (m)	15,600	4,000	4,000
$\Delta x, \Delta y$ (m)	200,200	40,40	100,100
No. of grid points	83x78	100x100	100x100

2.1.1 Boundary Conditions

To transform the offshore wave climate to the site, 24 unique combinations of wave height, direction and period were defined based upon the offshore design storm wave conditions and applied as boundary conditions to the model. The model was run with an offshore wave height of 1 m to simulate the effects of refraction and diffraction and obtain wave transformation coefficients at five locations along the Beachport foreshore and four locations along the Southend foreshore. The offshore wave data were derived from the Cape du Couedic Waverider Buoy, off the south-western tip of Kangaroo Island, approximately 300 km west of Rivoli Bay.

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 H_s = 7.3 m
- 10 year ARI H_s = 8.4 m
- 100 year ARI H_s = 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.



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2.1.2 Model Validation

No site-specific validation of the wave transformation model was possible due to a lack of suitable nearshore and offshore wave measurements. However, Nielsen and Adamantidis (2003) have validated successfully the SWAN numerical algorithms for the NSW coast *via* a comparison of numerical results with a comprehensive field data set obtained at Broken Bay, north of Sydney. As the only opportunity to calibrate the model lies in varying the bathymetric boundary conditions, if the bathymetry is known and schematised at an appropriate resolution then the model can be expected to give realistic results.

Table 2: Applied offshore wave conditions.

Sector	ARI (yr)	H_{m0}^2 (m)	T_P^3 (s)	Wave and wind direction (°)
E	n/a	1.0	10	90
	n/a	1.0	15	90
	n/a	1.0	20	90
SE	n/a	1.0	10	135
	n/a	1.0	15	135
	n/a	1.0	20	135
S	n/a	1.0	10	180
	n/a	1.0	15	180
	n/a	1.0	20	180
SSW	n/a	1.0	10	202.5
	n/a	1.0	15	202.5
	n/a	1.0	20	202.5
SW	n/a	1.0	10	225
	n/a	1.0	15	225
	n/a	1.0	20	225
	1	7.3	14	225
	10	8.5	14	225
	100	9.8	14	225
WSW	n/a	1.0	10	247.5
	n/a	1.0	15	247.5
	n/a	1.0	20	247.5
W	n/a	1.0	10	270
	n/a	1.0	15	270

² H_{m0} refers to the significant wave height, or the mean of the highest third of the waves in a wave group, computed on the basis of a spectrum.

³ T_P is the peak wave period, or wave period with the highest energy.



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2.1.3 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 and vectors for southerly offshore waves at Beachport and westerly offshore waves at Southend are shown in Figure 2 and Figure 3.

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 4. The locations at which the nearshore wave transformation coefficients are derived are shown in Figure 2 and Figure 3. 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.



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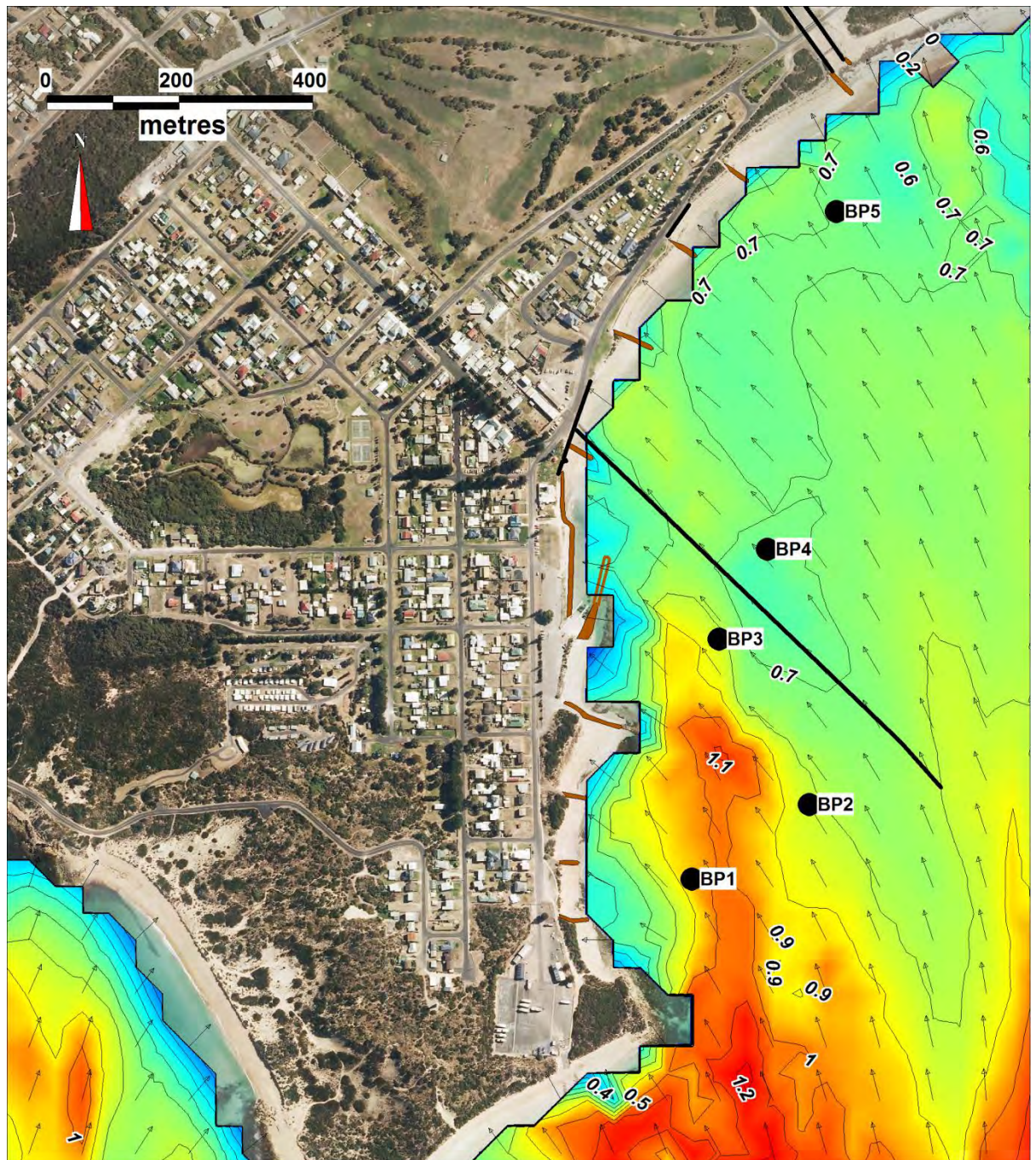


Figure 2 - Wave transformation coefficients, $H_{mo} = 1\text{m}$, $T_p = 15\text{s}$ from the south, Beachport



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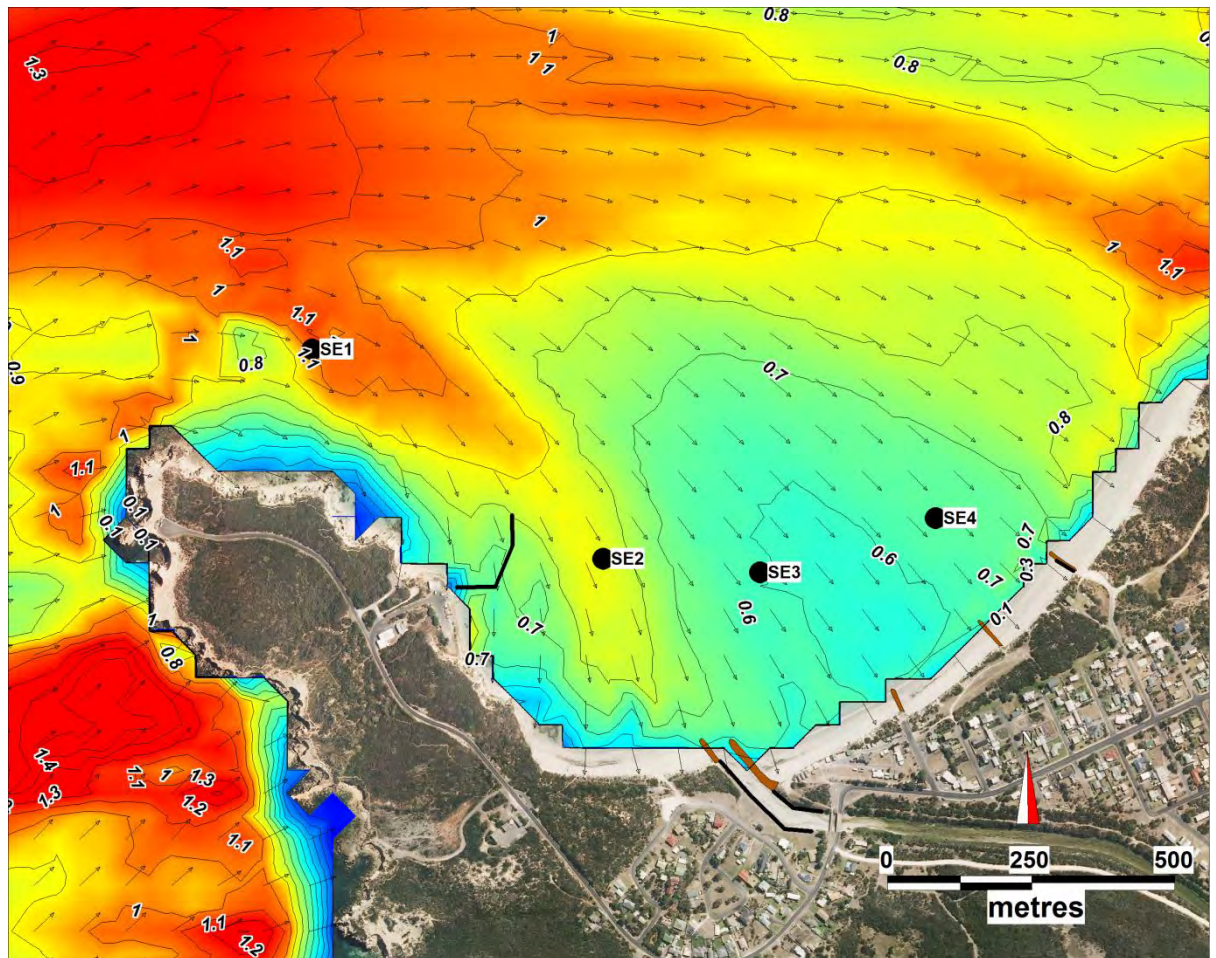


Figure 3 - Wave transformation coefficients, $H_{mo} = 1\text{m}$, $T_p = 15\text{s}$ from the west, Southend



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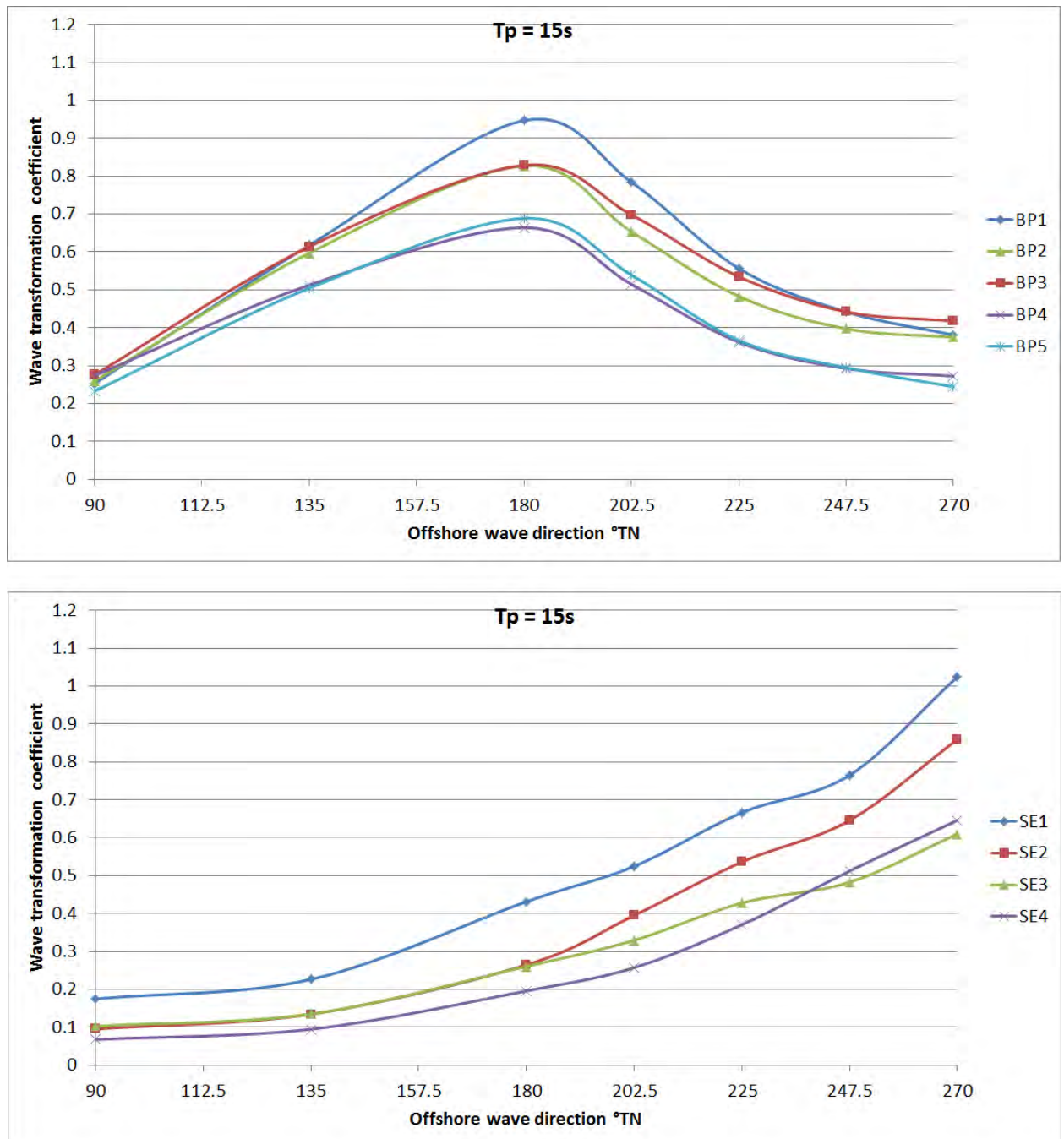


Figure 4 – Nearshore wave transformation coefficients at five locations in Beachport (top) and four locations in Southend (bottom) vs. offshore wave direction



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3 SBEACH MODELLING

Offshore extreme wave conditions were transformed to the Beachport and Southend nearshore using the SWAN wave model. The SWAN modelling has indicated that the extreme waves originating from the south results in the largest waves closer to shore at Beachport, and from the west at Southend. As the SWAN model does not account for wave setup and the wave conditions used to assess the stability of the coastal structures are expected to be depth limited, directly extracting the results from the SWAN model would be likely to result in an underestimation of the design conditions. Nevertheless, it was required to run the SWAN model to determine the *unrefracted deepwater wave height*, which is required for input to the SBEACH model, as described in the following.

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.

The model is founded on extensive large wave tank and field data measurements and analysis (Rosati *et al.*, 1993; CERC, 1984). The model accepts as data:

- surveyed beach profiles;
- time-varying water levels;
- regular or irregular wave heights and periods;
- wave angles;
- wind speeds and wind directions; and
- an arbitrary grain size in the fine-to-medium sand range.

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).

3.1 Model Setup

3.1.1 Bathymetry

The model bathymetry was based on beach profile data provided by the Department of Environment, Water and Natural Resources (DEWNR). The SBEACH profiles modelled are indicated in Figure 5 and Figure 6. Where a seawall or revetment is located within the beach profile, this was simulated in



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APPENDIX 2 - RIVOLI BAY

WAVE TRANSFORMATION MODELLING

the model. The SBEACH model allows the calculation of storm scour at each profile, thus the wave conditions and wave setup derived at each beach profile take into account scoured conditions as determined by the SBEACH model.

As the wave conditions are depth limited, the design wave for the various coastal structures would be the largest wave that breaks on the structure.

An example profile is shown below in Figure 7 showing the pre-storm beach profile, post-storm profile, local water level including wave setup and local maximum wave height.

A total of 9 SBEACH profiles were simulated, with five along the Beachport foreshore as shown in Figure 5, and four along the Southend foreshore as shown in Figure 6.



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Figure 5: SBEACH profiles Beachport



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Figure 6 – SBEACH profiles Southend



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WAVE TRANSFORMATION MODELLING

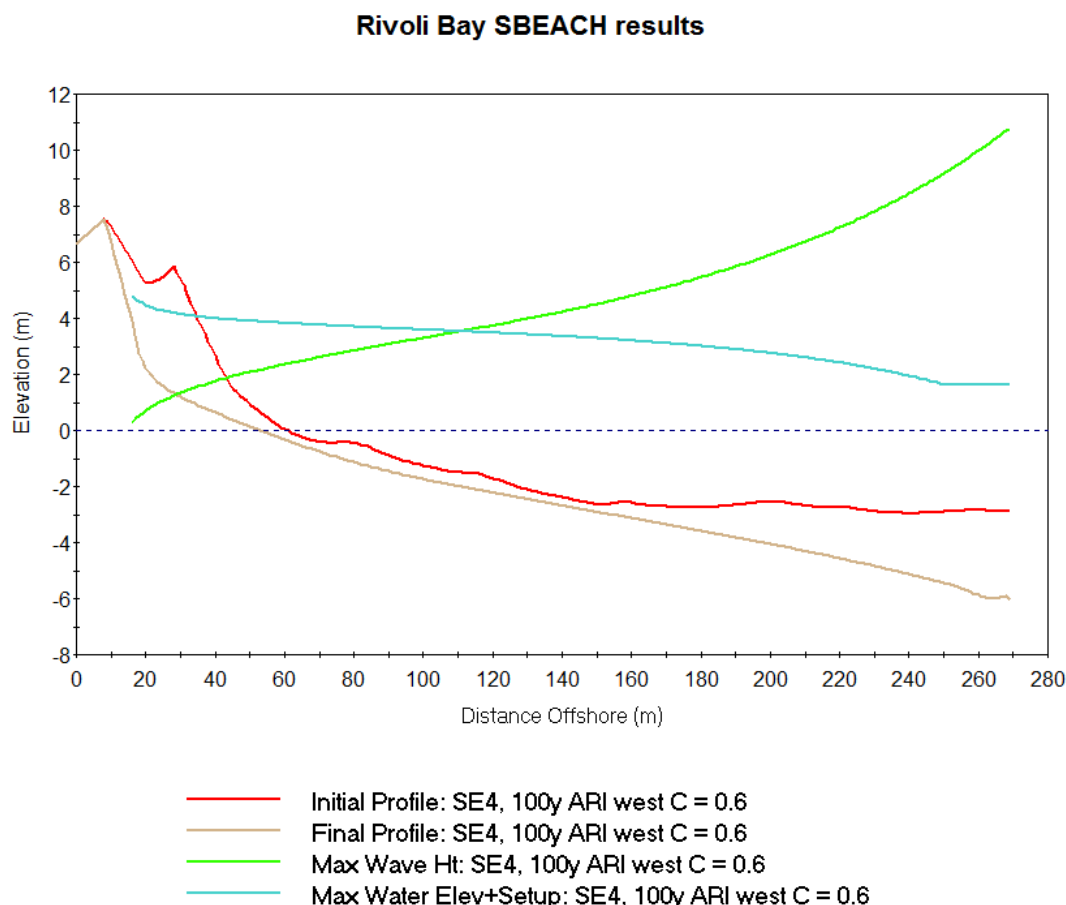


Figure 7: Example SBEACH profiles and results

3.2 Boundary Conditions

Boundary conditions were applied to the SBEACH model in the form of wave and water level time series at the seaward boundary of each profile.

Wave and water level boundary conditions were derived following the procedure outlined in Nielsen and Adamantidis (2007). This approach combines various duration extreme wave heights of the same return period (i.e. the 7 day, 6 day, 5 day, 4 day, 3 day, 2 day, 1 day, 12 hour, 6 hour, 3 hour and 1 hour exceedance wave heights for a return period of 100 years were combined to generate a time series of wave heights at the boundary for the 100 year ARI storm). The wave transformation coefficients derived at the seaward end of each profile from the SWAN model were applied to the 100 year, 10 year and 1 year ARI offshore wave heights at the seaward end of each profile, to account for the processes of wave refraction, diffraction and wave setup.



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Water levels consist of a tidal variation and a storm surge (or anomaly). The input time series for the tidal anomaly has been developed in a similar manner to the waves (i.e. by combining various duration extreme anomalies of the same return period). The anomaly time series was then added to predicted tidal variations during a spring tide (a high tide level of 0.8 m AHD) to derive the input water level time series. As spring tides occur twice a month, the use of a spring tide represents a conservative but credible approach for design which represents the worst condition that could occur in practice.

3.3 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.

Table 3 presents the wave and water level conditions extracted from the SBEACH model for the 1, 10 and 100 year average return intervals (ARI) at each profile. The location of the profiles is shown in Figure 5 and Figure 6. It should be noted that the variation in water levels between profiles shown in Table 3 are derived from the model's calculation of wave setup for each profile. These values provide an indicative range of water levels that may be expected across the study area and should be considered accordingly.

Table 3 - SBEACH results. Wave conditions are taken half a wavelength in front of the structure. Water levels are provided in metres above AHD and Hs (max) is the maximum local significant wave height in metres.

Cross section	Distance to structure	1 Year ARI		10 Year ARI		100 Year ARI	
		Max WL	Hs(max)	Max WL	Hs(max)	Max WL	Hs(max)
BP1	20	2.84	1.35	3.38	1.76	3.89	2.20
BP2	20	2.53	1.40	2.96	1.66	3.44	1.93
BP3	20	2.48	1.97	2.98	2.28	3.44	2.49
BP4	20	2.33	1.87	2.63	2.06	2.97	2.27
BP5	30	2.28	1.72	2.70	2.16	3.10	2.65
SE1	0	2.44	1.83	2.93	2.22	3.40	2.57
SE2	0	3.74	0.90	4.43	1.54	5.11	2.06
SE3	30	3.11	1.54	3.53	1.93	4.03	2.23
SE4	40	2.91	1.90	3.46	2.39	4.00	2.92

As shown in Table 3, the nearshore wave height is controlled by the water depth, there is an increase in wave height in front of the structure 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



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WAVE TRANSFORMATION MODELLING

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 Table 3 provide the variation in these design parameters along the entire foreshore at Beachport and Southend. It can be seen that:

- The present day maximum wave height approaching the structures 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 at profile BP3 (i.e. immediately adjacent to the Beachport jetty - refer Figure 5 for the location of this profile). 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 at profile SE1 (i.e. adjacent to the Southend jetty - refer Figure 6 for the location of this profile). 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) varies 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.



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APPENDIX 2 - RIVOLI BAY
WAVE TRANSFORMATION MODELLING**

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Appendix 3 - Rivoli Bay

Lake George Tidal Inlet analysis

301015-03541 – 001

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**WATTLE RANGE COUNCIL
APPENDIX 3 - RIVOLI BAY
LAKE GEORGE TIDAL INLET ANALYSIS**

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A	Issued for internal review	C Adamantidis	A Nielsen	A Nielsen	25-Sep-14	N/A	
B	Draft	C Adamantidis	A Nielsen	A Nielsen	26-Sep-14		
C	Final	C Adamantidis	A Nielsen	A Nielsen	28-Oct-15		



**WATTLE RANGE COUNCIL
APPENDIX 3 - RIVOLI BAY
LAKE GEORGE TIDAL INLET ANALYSIS**

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APPENDIX 3 - RIVOLI BAY

LAKE GEORGE TIDAL INLET ANALYSIS

1 ESCOFFIER ANALYSIS OF LAKE GEORGE OUTLET

The cross-sectional stability of a tidal inlet has been first analysed by Escoffier in 1940. Escoffier's theory analysed the permanency of the stability of the entrance depending on its size and the maximum current speed at the entrance. The relationship between cross-sectional area and maximum velocity is illustrated in Figure 1. When the maximum velocity is equal to the equilibrium velocity, the cross-sectional flow area is in equilibrium and the entrance is stable. When the maximum velocity is lower than the equilibrium velocity, the current is not strong enough to move the sediments carried into the inlet by littoral drift and the sediments will be deposited into the entrance reducing the cross-sectional area. When the maximum velocity is higher than the equilibrium velocity, the sediment transport capacity of the inlet currents will be larger than the volume of sediment carried into the inlet entrance by littoral drift and the entrance will therefore erode and the cross-sectional area will increase. From this and Figure 1, it can be observed that:

- if the cross-sectional area A is lower than A_1 ($A < A_1$), the sediments will deposit into the entrance and the entrance will tend to close over time;
- if $A > A_e$, the sediments will be deposited into the inlet entrance and the entrance cross-sectional area will reduce until $A = A_e$;
- if $A = A_1$, the equilibrium is unstable. If there is any storm depositing or removing some sediment from the entrance, the entrance would either close or widen to reach the equilibrium area A_e ; and
- if $A = A_e$, the equilibrium is stable. If a storm deposits or removes some sediments from the entrance it will recover to reach $A = A_e$.

If the cross sectional area is higher than the value which yields the maximum velocity in Figure 1, the entrance cross sectional area will increase and the entrance will erode until $A = A_e$.



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LAKE GEORGE TIDAL INLET ANALYSIS**

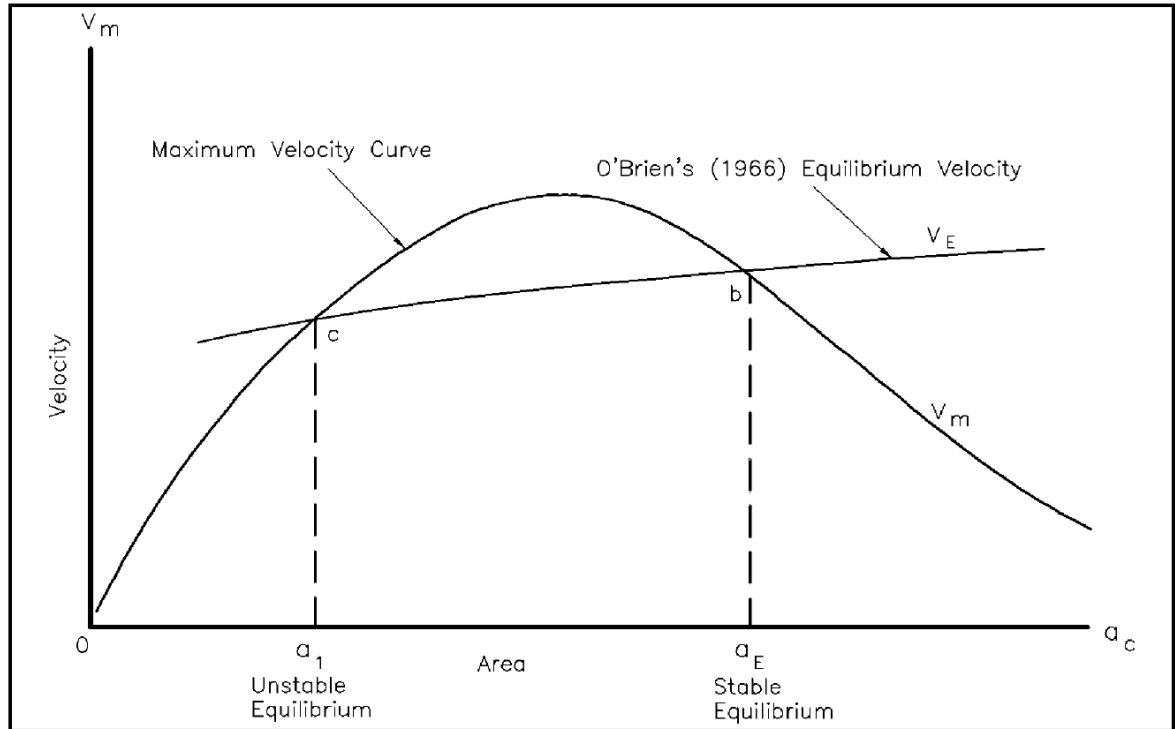


Figure 1 - Escoffier (1940) curve, maximum and equilibrium velocities versus inlet cross-sectional area (C.E.M., Chap. II-6)

However, the equilibrium cross-sectional area may be subject to long term changes. O'Brien (1969) determined a relationship between cross-sectional area and tidal prism. Assuming a maximum velocity $u_i = \hat{u}_i \sin(\omega t)$ with ω the angular frequency of the tide and t the time, it follows:

$$\Omega_i = \frac{A_i \hat{u}_i T}{\pi} \quad (1)$$

with Ω_i = Tidal Prism of the estuary

A_i = Cross-sectional area of the inlet entrance

T = Tidal period

This formula is usually presented as:

$$A = C\Omega^n \quad (2)$$



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LAKE GEORGE TIDAL INLET ANALYSIS

with C and n free parameters. These parameters can be obtained by analysing the correlation between the tidal prism and the cross-sectional area of several estuaries located within the same region.

O'Brien (1969)'s empirical relationship used $C = 4.69 \times 10^{-4}$ and $n = 0.85$, in imperial units, for the values of the exponents C and n in Equation 2 above. In the absence of site-specific data, this cross-sectional area-tidal prism relationship has been used for the Lake George inlet.

The stability of the Lake George outlet was analysed using the Channel Equilibrium Area model created by the Coastal Inlets Research Program (CIRP) from the US Army Corps of Engineers. This model allows the determination of the Escoffier curve and therefore the estimation of the channel equilibrium area based on the channel and outlet characteristics, the tidal prism and the tidal parameters. The parameters used in the calculation are as follows:

- Fundamental Ocean Tide Amplitude – this value is one half of the tidal range and the spring tide is typically used (around 0.45 m for Lake George);
- Ocean Overtide Amplitude – M4 tidal constituent amplitude (0.001 for Beachport);
- Tidal Period (24.84 hrs for diurnal tides);
- Tidal Mean Basin Surface Area – Surface area of the lake and estuary (estimated to be around 60,000,000 m²);
- Hydraulic Radius – average depth of the channel (estimated to be around 2 m after the channel has been dredged);
- Channel width – width of the minimum cross sectional area (estimated from aerial photography to be around 25 m)
- Channel length – estimated to be around 1250 m from aerial photography;
- Channel area – estimated to be around 50 m²
- Entrance loss coefficient K_{en} – value from 0.05 for a relatively streamlined inlet, to 0.25 for an inlet with dual jetties (i.e. 0.25 used here);
- Exit loss coefficient K_{ex} – a value of 1.0 for the exit loss coefficient describes a relatively deep bay and complete loss of kinetic head – smaller values can be tried during calibration); and
- Manning's Coefficient (n value) – this bed resistance parameter may have typical values between 0.025 and 0.05 for inlets (a value of 0.025 was used in the calculation).

The results of the analysis are provided in Figure 2. These results were compared with five-minute water level data within the middle basin of Lake George provided by the South Eastern Water Conservation and Drainage Board for September 2007, when it was known from weir management history provided by the South Eastern Water Conservation and Drainage Board that the weir was open, and a tidal signal was clearly discernible in the water level data. It was found that the CEA model correctly predicted the amplitude of the tidal signal within the Lake based on the channel



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APPENDIX 3 - RIVOLI BAY

LAKE GEORGE TIDAL INLET ANALYSIS

properties provided. It should be noted that the model is only able to predict the tidal amplitude within the Lake, but does not take into account factors such as freshwater inputs to the lake, barometric setup and tidal setup due to friction through the entrance channel which would influence the mean level within the lake.

The Escoffier curve resulting from the model is illustrated in Figure 3. From this figure, it is observed that the existing channel is unstable, with a cross-sectional area too low to maintain an open channel with the channel tending to close over time. Given the very low tidal range within the lake and therefore the low tidal prism, as well as the various dimensions of the channel entrance, the current velocities will be low and this would result in sand deposition within the entrance over time. Therefore the lake entrance tends to reduce and would eventually close if dredging is stopped. Flood events may increase the flow at the entrance and generate erosion that would deepen the entrance temporarily.

Should the channel geometry be changed such that the cross-sectional area be increased to a value greater than around 600 m² over its entire length (and the channel be allowed to modify its dimensions naturally – i.e. is not constrained by the training walls), the channel would be in an unstable equilibrium and would tend to increase its cross-sectional area until a stable equilibrium is reached (i.e. around 3000 m²). Under such conditions, the channel would be in an unstable scouring mode (i.e. it would continue to scour until the stable equilibrium is reached). It is not considered feasible to modify the 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 a change in the ecology of the lake due to the increased tidal range and modified salinities.



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LAKE GEORGE TIDAL INLET ANALYSIS

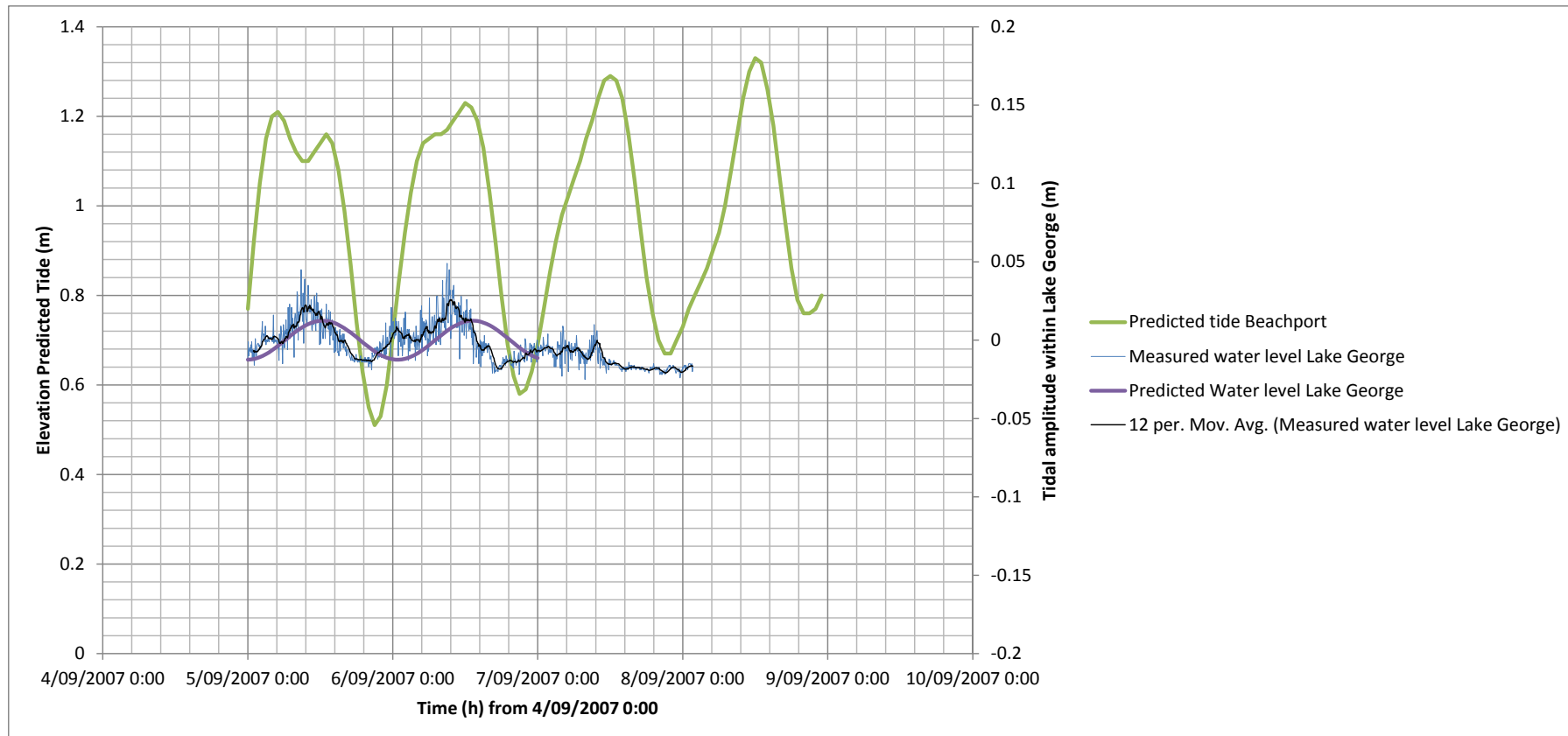


Figure 2 – Comparison of measured lake levels within the middle basin of Lake George with predicted ocean tide and predicted lake tidal signal from CEA model

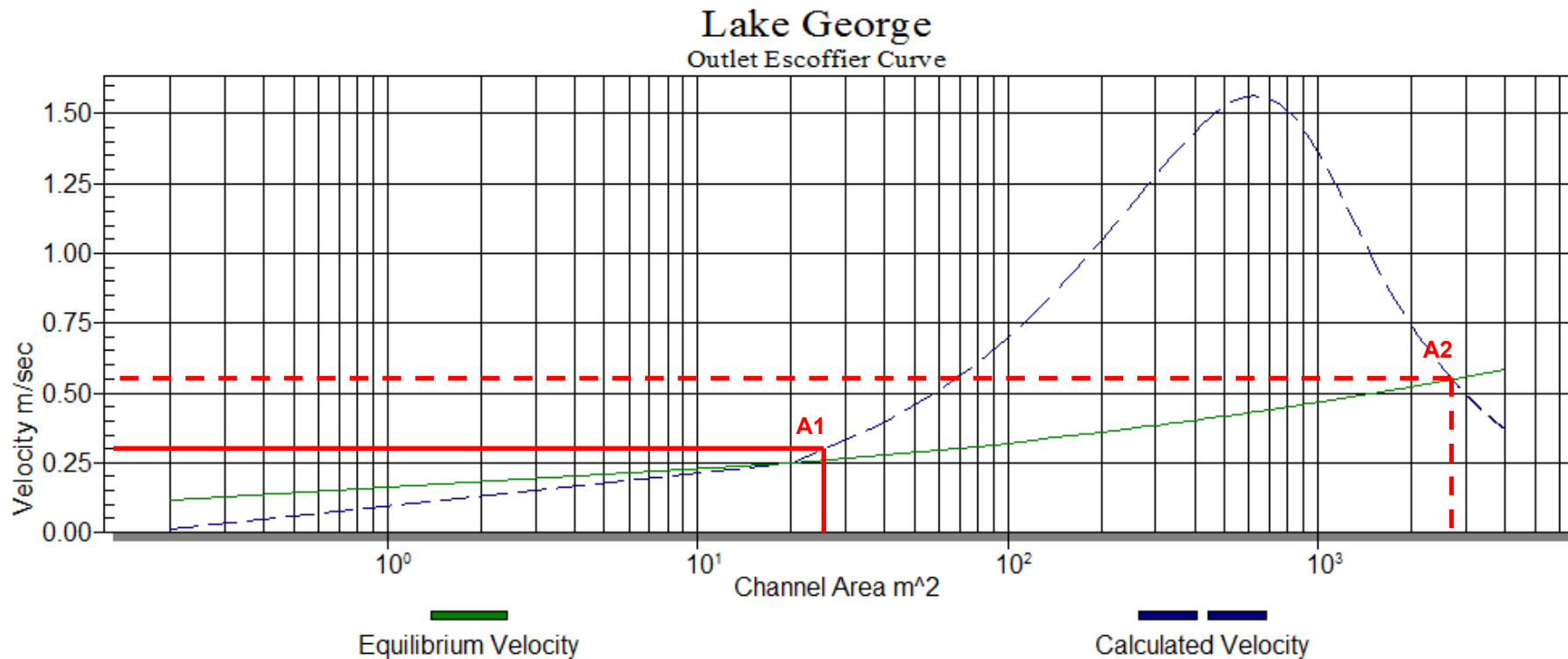


Figure 3 – Escoffier curve for Lake George outlet



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APPENDIX 3 - RIVOLI BAY
LAKE GEORGE TIDAL INLET ANALYSIS**

2 REFERENCES

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Appendix 4 - Rivoli Bay

Beachport Historical Aerial Photography

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BEACHPORT HISTORICAL AERIAL PHOTOGRAPHY**

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A	Issued for internal review	C Adamantidis	A Nielsen	A Nielsen	25-Sep-14	N/A	
B	Draft	C Adamantidis	A Nielsen	A Nielsen	26-Sep-14		
C	Final	C Adamantidis	A Nielsen	A Nielsen	28-Oct-15		



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BEACHPORT HISTORICAL AERIAL PHOTOGRAPHY



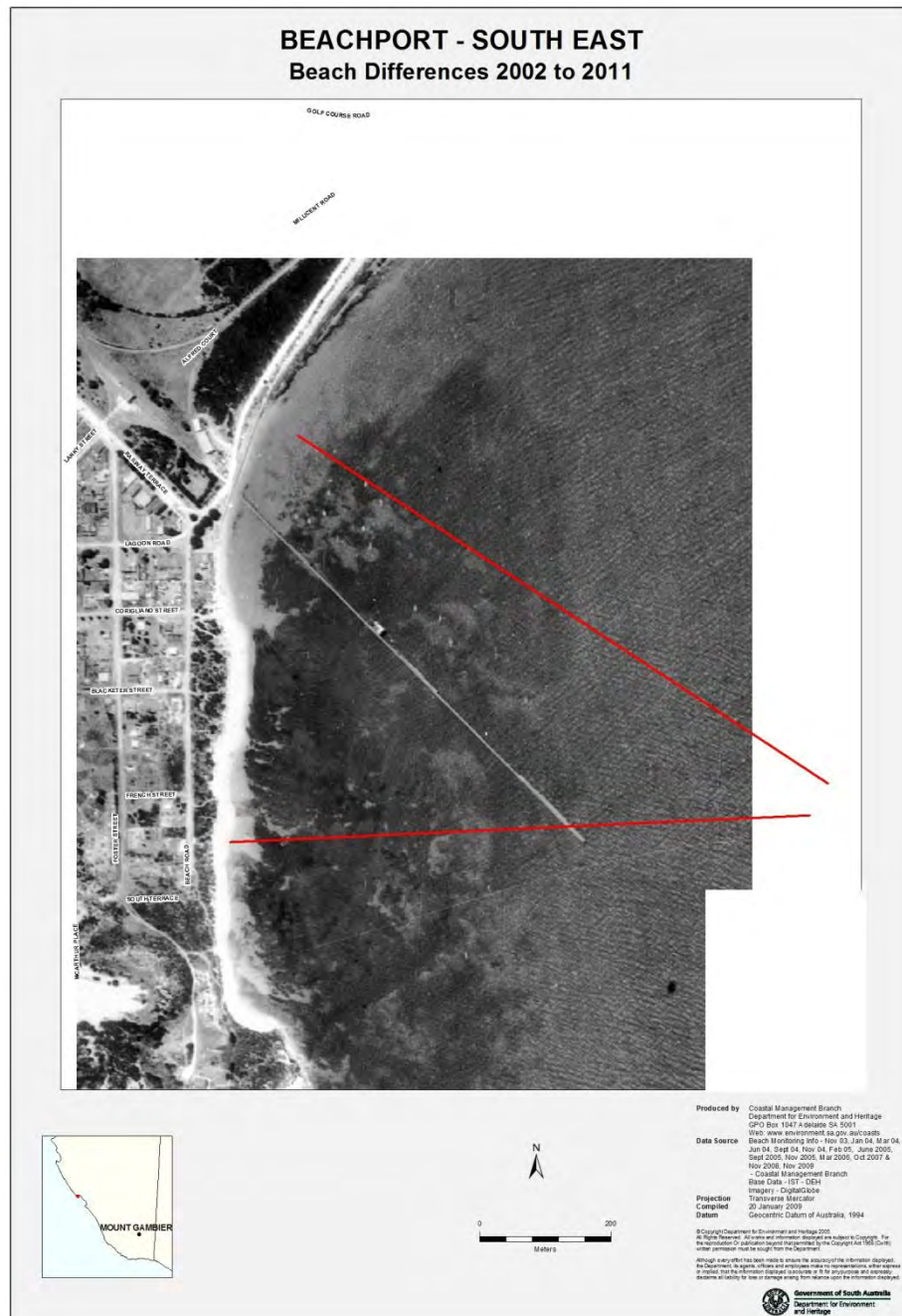
Beachport Feb 1946



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BEACHPORT HISTORICAL AERIAL PHOTOGRAPHY



Beachport 1951



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BEACHPORT HISTORICAL AERIAL PHOTOGRAPHY



Beachport 1962



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BEACHPORT HISTORICAL AERIAL PHOTOGRAPHY



Beachport 1975



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Beachport 1984



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Beachport 2004



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BEACHPORT HISTORICAL AERIAL PHOTOGRAPHY



Beachport 2008



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BEACHPORT HISTORICAL AERIAL PHOTOGRAPHY



Beachport 2013



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Appendix 5 - Rivoli Bay

Indicative Cost Estimates

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APPENDIX 5 - RIVOLI BAY
INDICATIVE COST ESTIMATES**

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Concept Design - Cost Estimate (Option 1)

Project No.: 301015-03541
Project Name: Rivoli Bay Coastal Management Options

Item	Description	Unit	Quantity	Rate	Cost	Source of Rate	Comments
1	Preliminaries						
1.1	Site establishment	Item		Lump Sum	\$ 15,000.00	WP internal	
1.2	Deployment and maintenance of environmental control provisions	Item		Lump Sum	\$ 5,000.00	WP internal	
1.3	Construction survey	Item		Lump Sum	\$ 5,000.00	WP internal	
2	Excavation of existing embankment						
2.1	Excavation+Backfill (embankment)	m ³	0	\$ 69.30	\$ -	Rawlinsons 2012 p 672	Excavate in sand for foundations
3	Extend two rock groynes by 100 m						
3.1	Rock (large, supply & place)	m ³	3,014	\$ 120.00	\$ 361,728.00	Based on quoted rate of \$70/tonne (\$120/m3)	
3.2	Crushed filter rock	m ³	880	\$ 30.20	\$ 26,576.00	Rawlinsons 2012 p677	
3.3	Labour (5 workers for 16 weeks)	hour	3,200	\$ 75.00	\$ 240,000.00		
3.4	Approvals, Detailed Design				\$ 50,000.00		
4	Site Disestablishment						
4.1	Site Disestablishment	Item		Lump Sum	\$ 10,000.00	WP internal	
			Sub-total		\$ 713,304		
			Contingency	30%	\$ 213,991		
			Total		\$ 927,295		

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This cost estimate is based on WorleyParsons' experience and judgement as a firm of practising professional engineers familiar with the construction industry.
This cost estimate can NOT be guaranteed as we have no control over Contractor's prices, market forces and competitive bids from tenderers.
This cost estimate excludes GST, design fees, project management fees, and authority approval fees.

Concept Design - Cost Estimate (Option 2)

Project No.: 301015-03541
Project Name: Rivoli Bay Coastal Management Options

Item	Description	Unit	Quantity	Rate	Cost	Source of Rate	Comments
1	Preliminaries						
1.1	Site establishment	Item		Lump Sum	\$ 15,000.00	WP internal	
1.2	Deployment and maintenance of environmental control provisions	Item		Lump Sum	\$ 5,000.00	WP internal	
1.3	Construction survey	Item		Lump Sum	\$ 5,000.00	WP internal	
2	Excavation of existing embankment						
2.1	Excavation+Backfill (embankment)	m ³	0	\$ 69.30	\$ -	Rawlinsons 2012 p 672	Excavate in sand for foundations
3	Additional groyne at Beachport 80 m long						
3.1	Rock (large, supply & place)	m ³	1,206	\$ 120.00	\$ 144,691.20	Based on quoted rate of \$70/tonne (\$120/m3)	
3.2	Crushed filter rock	m ³	352	\$ 30.20	\$ 10,630.40	Rawlinsons 2012 p677	
3.3	Labour (5 workers for 8 weeks)	hour	1,600	\$ 75.00	\$ 120,000.00		
3.4	Approvals, Detailed Design				\$ 40,000.00		
4	Site Disestablishment						
4.1	Site Disestablishment	Item		Lump Sum	\$ 10,000.00	WP internal	
			Sub-total		\$ 350,322		
			Contingency	30%	\$ 105,096		
			Total		\$ 455,418		

Disclaimer

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This cost estimate excludes GST, design fees, project management fees, and authority approval fees.

Concept Design - Cost Estimate (Option 3)

Project No.: 301015-03541
Project Name: Rivoli Bay Coastal Management Options

Item	Description	Unit	Quantity	Rate	Cost	Source of Rate	Comments
1	Preliminaries						
1.1	Site establishment	Item		Lump Sum	\$ 15,000.00	WP internal	
1.2	Deployment and maintenance of environmental control provisions	Item		Lump Sum	\$ 5,000.00	WP internal	
1.3	Construction survey	Item		Lump Sum	\$ 5,000.00	WP internal	
2	Excavation of existing embankment						
2.1	Excavation+Backfill (embankment)	m ³	960	\$ 69.30	\$ 66,528.00	Rawlinsons 2012 p 672	Excavate in sand for foundations
3	Rock revetment at Beachport 120 m long (crest level 4 m AHD)						
3.1	Rock (large, supply & place)	m ³	3,360	\$ 120.00	\$ 403,200.00	Based on quoted rate of \$70/tonne (\$120/m3)	
3.2	Crushed filter rock	m ³	960	\$ 30.20	\$ 28,992.00	Rawlinsons 2012 p677	
3.3	Labour (5 workers for 12 weeks)	hour	2,400	\$ 75.00	\$ 180,000.00		
3.4	Approvals, Detailed Design				\$ 50,000.00		
4	Site Disestablishment						
4.1	Site Disestablishment	Item		Lump Sum	\$ 10,000.00	WP internal	
			Sub-total		\$ 763,720		
			Contingency	30%	\$ 229,116		
			Total		\$ 992,836		

Disclaimer

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Concept Design - Cost Estimate (Option 4)

Project No.: 301015-03541
Project Name: Rivoli Bay Coastal Management Options

Item	Description	Unit	Quantity	Rate	Cost	Source of Rate	Comments
1	Preliminaries						
1.1	Site establishment	Item		Lump Sum	\$ 15,000.00	WP internal	
1.2	Deployment and maintenance of environmental control provisions	Item		Lump Sum	\$ 5,000.00	WP internal	
1.3	Construction survey	Item		Lump Sum	\$ 5,000.00	WP internal	
2	Excavation of existing sand source						
2.1	Excavate from beach berm at Southend and deposit east of Lake P	m ³	30,000	\$ 7.15	\$ 214,500.00	Rawlinsons 2012 p 672	Excavate in sand to reduce levels and deposit, spread and level
2.2	Cartage, 1 km	m ³	30,000	\$ 0.53	\$ 15,900.00	Rawlinsons 2012 p 672	
3	Dunal Revegetation						
3.1	Supply and planting of ground cover vegetation and trees	m ²	5,000	\$ 7.00	\$ 35,000.00	Rawlinsons 2012 p229	Ground cover including planting (avg cost)
4	Site Disestablishment						
4.1	Site Disestablishment	Item		Lump Sum	\$ 10,000.00	WP internal	
			Sub-total		\$ 300,400		
			Contingency	30%	\$ 90,120		
			Total		\$ 390,520		

Disclaimer

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Concept Design - Cost Estimate (Option 5)

Project No.: 301015-03541
Project Name: Rivoli Bay Coastal Management Options

Item	Description	Unit	Quantity	Rate	Cost	Source of Rate	Comments
1	Preliminaries						
1.1	Site establishment	Item		Lump Sum	\$ 15,000.00	WP internal	
1.2	Deployment and maintenance of environmental control provisions	Item		Lump Sum	\$ 5,000.00	WP internal	
1.3	Construction survey	Item		Lump Sum	\$ 5,000.00	WP internal	
2	Excavation of existing sand source						
2.1	Excavate from beach berm at Lake George outlet and from within	m ³	30,000	\$ 7.15	\$ 214,500.00	Rawlinsons 2012 p 672	Excavate in sand to reduce levels and deposit, spread and level
2.2	Cartage, 1 km	m ³	30,000	\$ 0.53	\$ 15,900.00	Rawlinsons 2012 p 672	
3	Dunal Revegetation						
3.1	Supply and planting of ground cover vegetation and trees	m ²	5,000	\$ 7.00	\$ 35,000.00	Rawlinsons 2012 p229	Ground cover including planting (avg cost)
4	Site Disestablishment						
4.1	Site Disestablishment	Item		Lump Sum	\$ 10,000.00	WP internal	
			Sub-total		\$ 300,400		
			Contingency	30%	\$ 90,120		
			Total		\$ 390,520		

Disclaimer

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Concept Design - Cost Estimate (Option 6)

Project No.: 301015-03541
Project Name: Rivoli Bay Coastal Management Options

Item	Description	Unit	Quantity	Rate	Cost	Source of Rate	Comments
1	Preliminaries						
1.1	Site establishment	Item		Lump Sum	\$ 15,000.00	WP internal	
1.2	Deployment and maintenance of environmental control provisions	Item		Lump Sum	\$ 5,000.00	WP internal	
1.3	Construction survey	Item		Lump Sum	\$ 5,000.00	WP internal	
2	Excavation of existing embankment and landscaping						
2.1	Excavation+Backfill (embankment)	m ³	50	\$ 69.30	\$ 3,465.00	Rawlinsons 2012 p 672	Excavate in sand for foundations
3.1	Supply and planting of ground cover vegetation and trees	m ²	100	\$ 7.00	\$ 700.00		
3	Repair existing groynes (i.e. top up) and reinforce downdrift sides of groynes						
3.1	Rock (large, supply & place)	m ³	400	\$ 120.00	\$ 48,000.00	Based on quoted rate of \$70/tonne (\$120/m3)	
3.2	Crushed filter rock	m ³	-	\$ 30.20	\$ -	Rawlinsons 2012 p677	
3.3	Labour (5 workers for 4 weeks)	hour	800	\$ 75.00	\$ 60,000.00		
3.4	Approvals, Detailed Design				\$ 20,000.00		
4	Site Disestablishment						
4.1	Site Disestablishment	Item		Lump Sum	\$ 10,000.00	WP internal	
			Sub-total		\$ 167,165		
			Contingency	30%	\$ 50,150		
			Total		\$ 217,315		

Disclaimer

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Concept Design - Cost Estimate (Option 7)

Project No.: 301015-03541
Project Name: Rivoli Bay Coastal Management Options

Item	Description	Unit	Quantity	Rate	Cost	Source of Rate	Comments
1	Preliminaries						
1.1	Site establishment	Item		Lump Sum	\$ 2,000.00	WP internal	Small scale operation
1.2	Deployment and maintenance of environmental control provisions	Item		Lump Sum	\$ 5,000.00	WP internal	
1.3	Construction survey	Item		Lump Sum	\$ 5,000.00	WP internal	Not needed
2	Removal of four existing Southend groynes						
2.1	Excavate sand from around groynes	m ³	4,000	\$ 30.70	\$ 122,800.00	Rawlinsons 2012 p 673	
2.2	Rip out rock	m ³	2,000	\$ 19.00	\$ 38,000.00	Rawlinsons 2012 p 672	
3	Redistribute sand along beach						
3.1	Excavate from beach berm at Southend and deposit east of Lake Frome	m ³	30,000	\$ 7.15	\$ 214,500.00	Rawlinsons 2012 p 672	Excavate in sand to reduce levels and deposit, spread and level
3.2	Cartage, 1 km	m ³	30,000	\$ 0.53	\$ 15,900.00	Rawlinsons 2012 p 672	
4	Dunal Revegetation						
4.1	Supply and planting of ground cover vegetation and trees	m ²	5,000	\$ 7.00	\$ 35,000.00	Rawlinsons 2012 p229	Ground cover including planting (avg cost)
5	Site Disestablishment						
4.1	Site Disestablishment	Item		Lump Sum	\$ 5,000.00	WP internal	
			Sub-total		\$ 443,200		
			Contingency	30%	\$ 132,960		
			Total		\$ 576,160		

Disclaimer

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Concept Design - Cost Estimate (Option 8)

Project No.: 301015-03541
Project Name: Rivoli Bay Coastal Management Options

Item	Description	Unit	Quantity	Rate	Cost	Source of Rate	Comments
1	Preliminaries						
1.1	Site establishment	Item		Lump Sum	\$ 15,000.00	WP internal	
1.2	Deployment and maintenance of environmental control provisions	Item		Lump Sum	\$ 5,000.00	WP internal	
1.3	Construction survey	Item		Lump Sum	\$ 5,000.00	WP internal	
2	Excavation of existing embankment						
2.1	Excavation+Backfill (embankment)	m ³	0	\$ 69.30	\$ -	Rawlinsons 2012 p 672	Excavate in sand for foundations
3	Extend three rock groynes by total of 100 m						
3.1	Rock (large, supply & place)	m ³	1,507	\$ 120.00	\$ 180,864.00	Based on quoted rate of \$70/tonne (\$120/m3)	
3.2	Crushed filter rock	m ³	440	\$ 30.20	\$ 13,288.00	Rawlinsons 2012 p677	
3.3	Labour (5 workers for 16 weeks)	hour	3,200	\$ 75.00	\$ 240,000.00		
3.4	Approvals, Detailed Design				\$ 50,000.00		
4	Site Disestablishment						
4.1	Site Disestablishment	Item		Lump Sum	\$ 10,000.00	WP internal	
			Sub-total		\$ 519,152		
			Contingency	30%	\$ 155,746		
			Total		\$ 674,898		

Disclaimer

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Concept Design - Cost Estimate (Option 10)

Project No.: 301015-03541
Project Name: Rivoli Bay Coastal Management Options

Item	Description	Unit	Quantity	Rate	Cost	Source of Rate	Comments
1	Preliminaries						
1.1	Site establishment	Item		Lump Sum	\$ 2,000.00	WP internal	
1.2	Deployment and maintenance of environmental control provisions	Item		Lump Sum	\$ 5,000.00	WP internal	
1.3	Deploy dredger base cost	Item		Lump Sum	\$ 57,500.00	Rawlinsons 2012 p 673	
2	Excavation of existing sand source						
2.1	Excavate from below low tide level	m ³	100,000	\$ 4.95	\$ 495,000.00	Rawlinsons 2012 p 673	Excavate in sand to reduce levels and deposit, spread and level
2.2	Level dredged sand on site	m ³	100,000	\$ 2.80	\$ 280,000.00	Rawlinsons 2012 p 673	200 m3/m for 500 m length of beach
3	Dunal Revegetation						
3.1	Supply and planting of ground cover vegetation	m ²	5,000	\$ 7.00	\$ 35,000.00	Rawlinsons 2012 p229	10 m width 500 m length of beach
4	Site Disestablishment						
4.1	Site Disestablishment	Item		Lump Sum	\$ 5,000.00	WP internal	
			Sub-total		\$ 879,500		
			Contingency	30%	\$ 263,850		
			Total		\$ 1,143,350		

Disclaimer

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Concept Design - Cost Estimate (Option 11)

Project No.: 301015-03541
Project Name: Rivoli Bay Coastal Management Options

Item	Description	Unit	Quantity	Rate	Cost	Source of Rate	Comments
1	Preliminaries						
1.1	Site establishment	Item		Lump Sum	\$ 15,000.00	WP internal	
1.2	Deployment and maintenance of environmental control provisions	Item		Lump Sum	\$ 5,000.00	WP internal	
1.3	Construction survey	Item		Lump Sum	\$ 5,000.00	WP internal	
2	Concrete wave return wall at Beachport 120 m long						
2.1	Reinforced Concrete 1.4 m ³ /m	m ³	168	\$ 1,000.00	\$ 168,000.00	Rawlinsons 2012 p 672	Excavate in sand for foundations
2.2	Labour (5 workers for 4 weeks)	hour	800	\$ 75.00	\$ 60,000.00		
2.3	Approvals, Detailed Design				\$ 20,000.00		
3	Raise rock revetment at Southend by 1 m, 120 m long						
3.1	Rock (large, supply & place)	m ³	240	\$ 120.00	\$ 28,800.00	Based on quoted rate of \$70/tonne (\$120/m3)	
3.2	Crushed filter rock	m ³		\$ 30.20	\$ -	Rawlinsons 2012 p677	
3.3	Labour (5 workers for 4 weeks)	hour	800	\$ 75.00	\$ 60,000.00		
3.4	Approvals, Detailed Design				\$ 20,000.00		
4	Site Disestablishment						
4.1	Site Disestablishment	Item		Lump Sum	\$ 10,000.00	WP internal	
			Sub-total		\$ 391,800		
			Contingency	30%	\$ 117,540		
			Total		\$ 509,340		

Disclaimer

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