Contract 176

Report

Remedial Work to Counter Erosion at Westshore, Napier

November 2003

Prepared for

Napier City Council

Beca

▪ report

Contract 176 Remedial Work To Counter Erosion At Westshore, Napier

Prepared for Napier City Council

By
Beca Carter Hollings & Ferner Ltd

November 2003

2509903/010 Rev B ▪ L2:2217

Executive Summary

Purpose of the Study

The study areas covered by this commission include Variation 2 and Variation 3. Variation 2 is a reach of beach from Scapa Flow to 8 Charles Street, referred to as Whakarire Ave. Variation 3 covers a 5 kilometre reach of beach from 49 Fergusson Street, Bayview to the Esk River. This area is referred to as Bayview.

Based on existing information, the specific objective was to carry out an analysis of both soft and hard engineering options and recommend a preferred option for both Variation 2 and Variation 3 areas. The preferred option would be accompanied by rough order of costs for implementation but would not be advanced beyond a concept plan stage.

Whakarire Ave

Causes of Erosion

Coastal erosion seaward of Whakarire Avenue is primarily due to a lack of sediment supply. Since the construction of the training moles, the reduction in the tidal prism of the Ahuriri Estuary, the loss of direct sediment input from the Tutaekuri River, and the introduction of the Port of Napier breakwater system, natural inputs of sediment into the southern end of the Westshore Beach system have ceased.

The area has also been reclaimed and the orientation of the coastline in this location is not in equilibrium with the incident wave climate. This means that any sand or gravel that is introduced into the local beach system cannot hold its position because the waves drive the sediment northwards.

Existing Seawall

Although the existing seawall structure has remained intact since 1994 and affords some level of protection to the existing coastline, it is not robust enough to be considered as a long term structure for the protection of the coastline.

The existing structure funnels waves into Westshore Beach and causes rapid transport of sediment both offshore and along shore. It does not enable a stable control point or feeder area for the current renourishment scheme.

Options

In addition to the do nothing option, the following coastal protection options were examined:

- Enhanced seawall (See Figure 9: approximate cost of $400,000)

- Enhanced seawall and infilling of backshore (See Figure 10: approximate cost of $525,000)
- Groyne to limit wave focussing (See Figure 11: approximate cost of $750,000)
- Wave spending beach (See Figure 12: approximate cost of $725,000)
- Attached breakwater with beach creation (See Figure 13: approximate cost of $675,000)

Resource consents will be required for all options. In addition to the coastal processes, an application will need to address the effects on existing surfing conditions, reef ecology, heritage values, and landscaping.

Preferred Option

The attached breakwater with beach creation is the preferred option at this stage of the assessment. The new breakwater would act as a headland control and encourage a new beach to be created on the leeward side. This new recreational beach could be created from sand dredged from the Ahuriri entrance channel. It will transition into the gravel and sand beach of southern Westshore. This section of the beach will be more stable and could act as a feeder beach for the nourishment scheme.

Bayview

Causes of Erosion

It would appear that the beneficial effects of the existing nourishment scheme run out at Snapper Park Motor Camp. More monitoring data are required, however, to quantify the extent of its benefits and to confirm if all sections of Bayview are suffering from long-term erosion.

The beach alignment appears to be in dynamic equilibrium as it is perpendicular to the net wave direction. The sediment budget indicates that the main loss of beach material is due to the abrasion of the gravel.

The river channel of the Esk River is mobile. It causes erosion of the nearby beach system when migrating in a southerly direction.

There are no significant coastal protection works in place at Bayview.

Options

In addition to the do nothing option, the following coastal protection options were examined for a 2.5 km beach section:

- Emerging breakwaters (See Figure 16: approximate cost of $24 million)
- Submerged Reefs (See Figure 17: approximate cost of $9 million)

- Revetment Structures (See Figure 18: approximate cost of $5-22 million)
- Renourishment (annual cost of $100,000)

Given the level of existing development and the assessment of erosion potential along this coastline a cautious approach is required. Large scale hard engineering options will be very expensive for this section of the coastline and probably not warranted at this stage.

Preferred Option

Further monitoring of the beach profiles and nearshore area is recommended so as to clearly establish whether long term erosion is persisting.

The coastal hazard zones should be adopted as a planning measure to control future development in the area. A cautious approach is recommended.

Revision History

Revision Nº	Prepared By	Description	Date
A	Stephen Priestley	Draft for review	August 2003
B	Stephen Priestley	Final to Napier City	November 2003

Document Acceptance

Action	Name	Signed	Date
Prepared by	Stephen Priestley		11/03
Reviewed by	Jeremy Gibb/ Richard Frankland	R Frankland	11/03
Approved by	Stephen Priestley		11/03

Table of Contents

Appendices

Tables:

Figures:

1 Introduction

On 15 February 2003, Beca Carter Hollings and Ferner Limited (Beca) was commissioned by Napier City Council to investigate remedial works to counter erosion at Westshore, Napier.

The study areas covered by this commission include Variation 2 and Variation 3. Variation 2 is a reach of beach covering 350 metres from Scapa Flow to 8 Charles Street. Throughout this report this area will be referred to as Whakarire Ave. Variation 3 covers a 5 kilometre reach of beach from 49 Fergusson Street, Bayview to the Esk River. This will be referred to as Bayview in this report.

The specific objectives of this work were to:

- Based on existing information, carry out an analysis of both soft and hard engineering options and recommend a preferred option for both Variation 2 and Variation 3 areas.
- The preferred option would be accompanied by rough order of costs for implementation but would not be advanced beyond a concept plan stage.

The general methodology for this work is generally as covered in the Beca Proposal of 28 January 2003. It has included:

- Briefing by the Coastal Scientist, Dr Jeremy Gibb and Napier City Staff
- Attendance at a public meeting of the Whakarire Street residents.
- Review of reports forwarded by Napier City which include Smith (1993), Koutsos (1993), Gibb (1996), Oldman and Smith (1998), Koutsos (1999), Tonkin and Taylor (2000), ASR (2001), Gibb (2002), Gibb (2003), Koutsos (2003).
- From these reports a general understanding of the history of the area as well as the coastal processes and erosion potential of Westshore Beach was gained.
- From this understanding of the erosion processes a range of soft and hard engineering options were considered for both study areas. These were discussed with council and preferred options were costed in more detail and presented in this final report.

Photographs of the coastline and coastal features are contained in Appendix A.

2 Background

2.1 Location of the Study Areas

Both study areas are contained within a beach system extending from the Ahuriri Harbour, northwards to Tangoro Point covering a coastline length of some 15 kilometres. The location of both study areas is shown in Figure 1 relative to this coastline. Whakarire Ave is immediately north of the Ahuriri Harbour. The Bayview study area is some 5 kilometres north of the harbour.

2.2 History

Much has been written about the natural and anthropogenic changes to the Westshore coastline since Maori occupation of the area. Gibb (1996) provides a succinct history of the area. In relation to the Whakarire Avenue area the following events are highlighted:

- Prior to harbour works and the 1931 earthquake the Ahuriri Harbour was significantly larger and subject to natural fluctuations in width from about 115 to 275 metres and tidal currents of up to 6 to 7 knots, causing changes to the nearshore and foreshore areas adjacent to the entrance.
- Between 1855 and 1875 limestone boulders were removed from the Ahuriri entrance area for ships ballast which destabilised the entrance resulting in an increase in width and a corresponding decrease in depths.
- Between 1876 and 1879 the entrance was confined to a width of 122 metres between 2 training moles resulting in the trapping of the net northerly drift of gravel against the east mole and starving of Westshore Beach of its natural supply of gravel and sand.
- Between 1879 and 1887 gravel and sand commenced bypassing the entrance in the form of an elongated bank. That is, an ebb delta and a scoured basin formed on the nearshore seabed adjacent to the entrance in response to the ebb tide jet with velocities up to 7 knots.
- Between 1882 and 1888 gravels were dredged from the entrance and used to reclaim the western spit by Whakarire Avenue, thereby stabilising the shoreline. A freezing company was located in this area. In 1888 the depositing of dredging behind the western mole was discontinued and the outer beach rapidly began to diminish to the extent that the freezing company needed to protect their works by building groynes.
- Between 1887 and 1890 the weather breakwater for Napier Harbour was constructed, which totally blocked the northerly drift of gravel and deflected the northerly transport of sand. This resulted in the Ahuriri entrance deepening and bypassing of sediments northwards pass the entrance ceased.
- Between 1909 and 1923 protection works were constructed around the Whakarire Avenue reclamation to combat sea erosion.

Beca

2509903/010

Location of Study Areas

Figure 1

- On 3 February 1931 the Hawkes Bay earthquake produced an instantaneous 20 to 65 metre shoreline advance and a 1.8 to 2.1 metre uplift of the gravel barrier and nearshore seabed. Another dramatic effect of the earthquake was the disappearance of most of the shallow Ahuriri Estuary, significantly diminishing the tidal prism, the tidal jet and any nearshore bar system. Some 2,230ha of the 3,845ha estuary was uplifted. As the earthquake reduced river gradients, the Tutaekuri River was redirected from its discharge into the harbour to the south near Awatoto where it remains today.
- From 1931 to the mid 1960's the uplifted beach profile was gradually eroded in the nearshore and formed a well maintained beach system which either advanced or remained in dynamic equilibrium. Frequently during this period Westshore Beach appeared as a sandy beach. By the mid 1960's the benefits of the uplifted foreshore area had diminished and the Westshore Beach system started to noticeably erode.
- Between 1987 and 2002, 155,000 cubic metres of gravel and sand were placed on the nearshore of Westshore Beach and an additional 79,000 cubic metres was placed in the back barrier to renourish the beach system. In general this material has been placed within the beach system north of Whakariere Avenue although in 1995, some 4,500 cubic metres of fine gravel and 10,000 m^3 of fine sand were placed immediately in front of Whakariere Avenue. This material was observed to have moved rapidly to the north.

Within the Bayview study area, the above historical events are also relevant with the following event also highlighted:

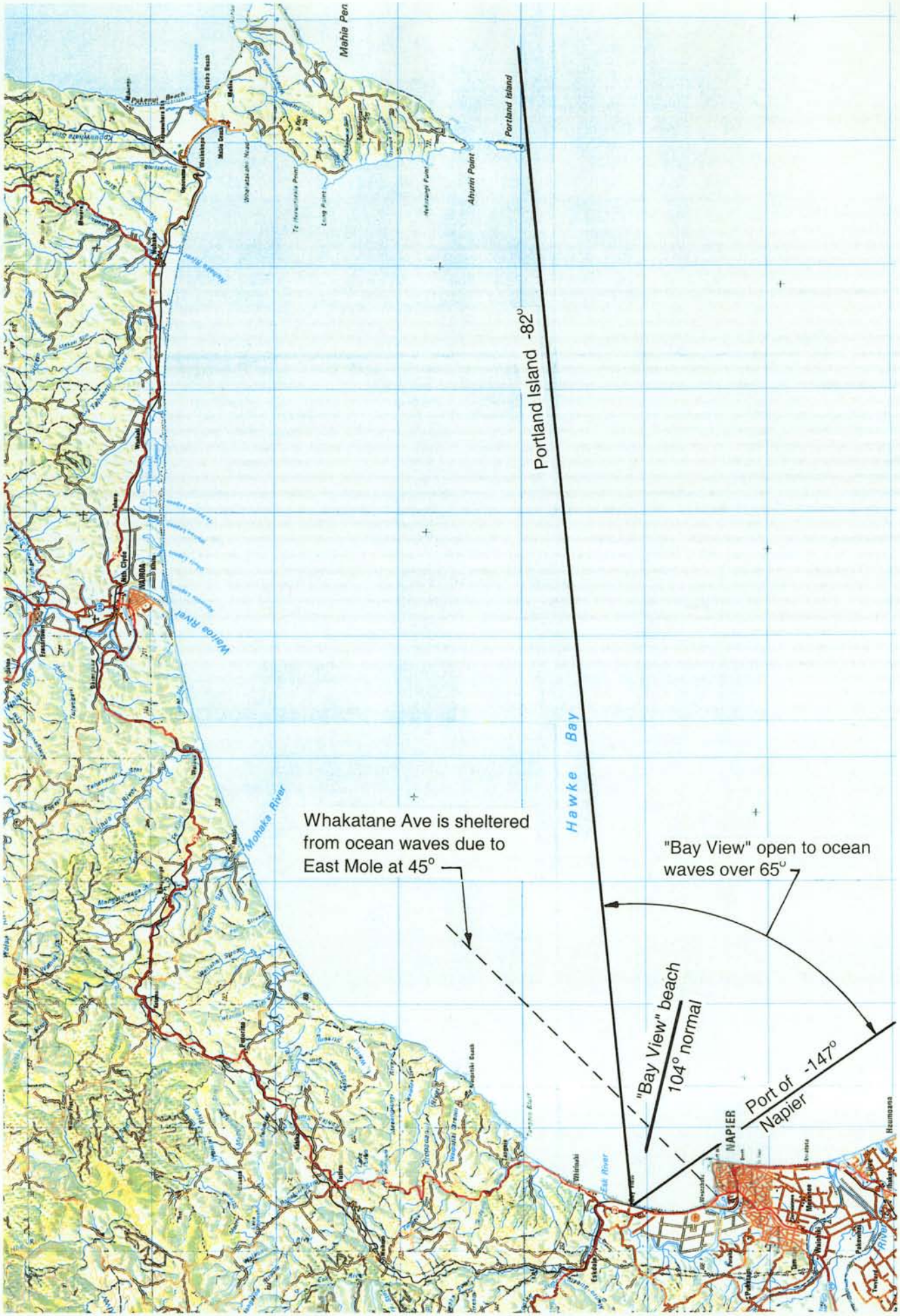
- When Maori settled into the area about 1300 AD they entered the Ahuriri estuary at Bayview, 6.5 kilometres north of the present entrance. That entrance was permanently closed by storm waves in about 1760 and Maori opened the present entrance between 1769 and 1824 where it has remained ever since.

2.3 Coastal Processes

2.3.1 Wave Climate

Waves are the predominant mechanism for gravel and sand transport within the Westshore Beach system. The entire bay is subject to a number of headland constraints. At the southern end of Westshore, Whakariere Avenue is located in lee of the eastern mole, the Port of Napier breakwater and Portland Island, resulting in a sheltered wave environment subject to diffraction from these headlands and local refraction due to the shoaling reef immediately seaward. These features are shown in Figure 2.

There is a natural tendency for a shoreline to be in balance with the incoming wave environment and the natural net supply of sediment to the system. The orientation of the shoreline created on the eastside of the eastern mole is about 30° (true north) normal. It would be expected that a similar orientation (although subject to further refraction) would occur for a beach formed immediately north of Whakariere Avenue. In this area it has



Headland Constraints

2509903/010

Figure 2

been estimated by ASR (2001) that waves are reduced by up to 70% (of wave height) compared to those waves offshore of the Port of Napier.

Further to the north near Bayview the beach is open to the ocean from 82° round to 147°. See Figure 2. The beach is orientated at about 102° to 110° normal, indicating slightly more energetic waves from the east to south east quarter.

From a 20 year numerical wave hindcast by Laing and Gorman (2000) (cited ASR (2000)) and a short term wave rider deployment by ASR (2001) it was found that the most energetic waves are dominated by waves approaching from the nor noreast to east quarter. These data indicate 5% percent exceedance for a significant wave height of 2 m and 50% percent exceedance for a wave height of 0.9 m. The largest open sea, significant wave height in the hindcast record was 6.2 m.

Port of Napier has advised that, based on 2 years of wave data from their wave-buoy, waves larger than 1.0 m are strongly focussed around 75 to 150 °, with waves larger than 2.5 m more focussed around 110 to 115°. This is generally in agreement with the beach orientation at Bayview.

Prior to the 1931 earthquake a number of sea storm events were reported to overtop the barrier. As noted in Gibb (1996), wave runup events along the coast have not been observed to overtop the gravel barrier following the earthquake.

2.3.2 Currents

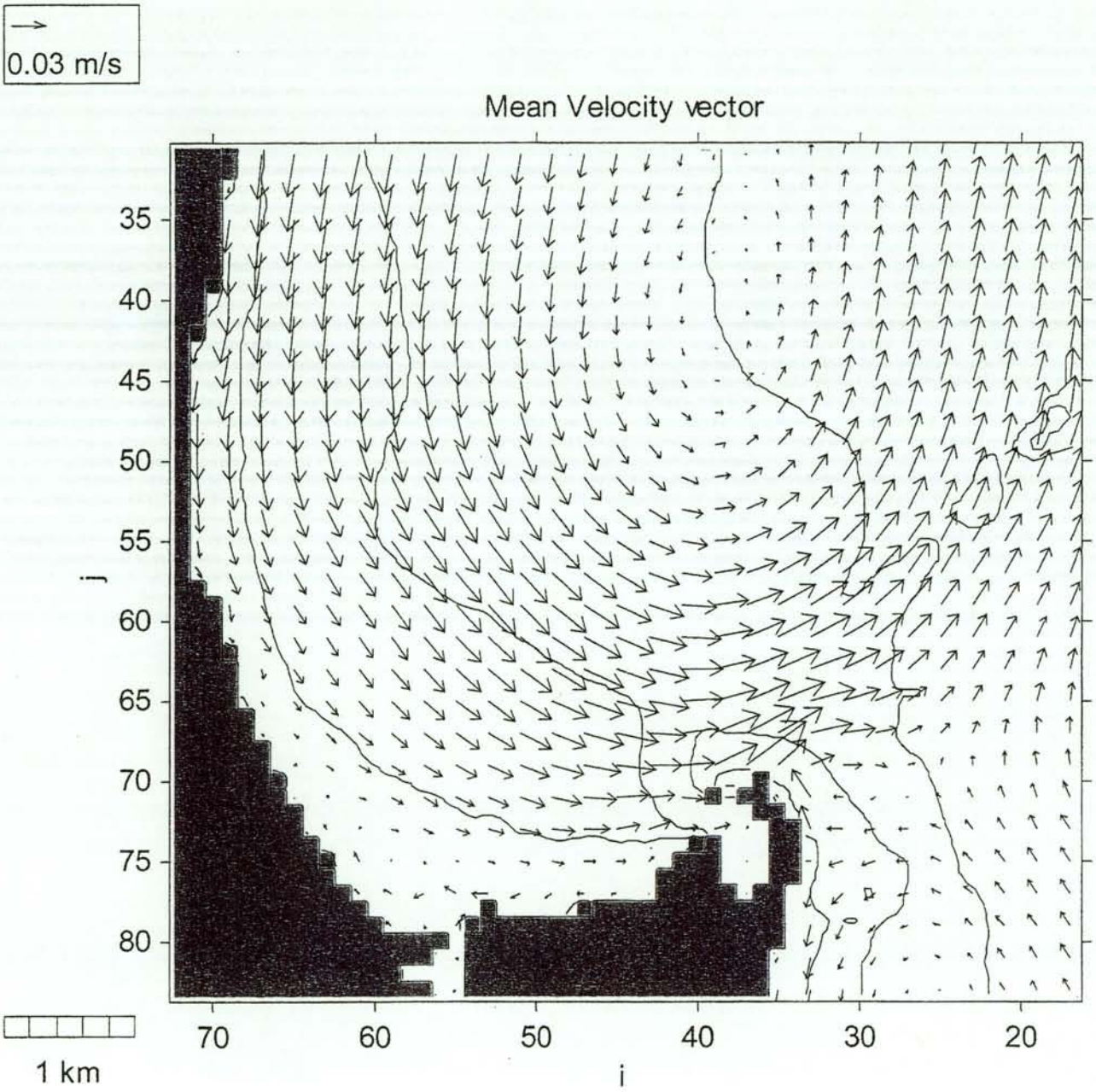
Tidal levels in Hawke Bay are given in Table 1.

Table 1: Tide Levels

Tide State	Chart Datum (m)
Highest on Record[1]	2.3
MHWS	1.6
MHWN	1.5
MSL[2]	0.9
MLWN	0.3
MLWS	0.2
Chart Datum	0

[1] Peter Frizzell (pers.com, 2003) of Port of Napier
[2] Corresponds to Napier City datum of RL 10.0m

Because the tidal currents within Hawke Bay are relatively weak, current circulations tend to be driven by other forcing functions such as wind driven or wave driven events. From a 2 week deployment and calibration of a numerical model, ASR (2001) reported the mean velocity vectors over the Westshore Bay area as shown in Figure 3. This generally shows that the nearshore currents move in a southerly direction from the Bayview area with a strong anticlockwise rotation over the deeper parts of the bay and a weaker clockwise rotation closer to shore nearer the Ahuriri Entrance. In general the currents are relatively weak and less than 0.1m/s.



Source: Figure 6.5 ASR (2000)

Beca

2509903/010

Mean Velocity Vectors

Figure 3

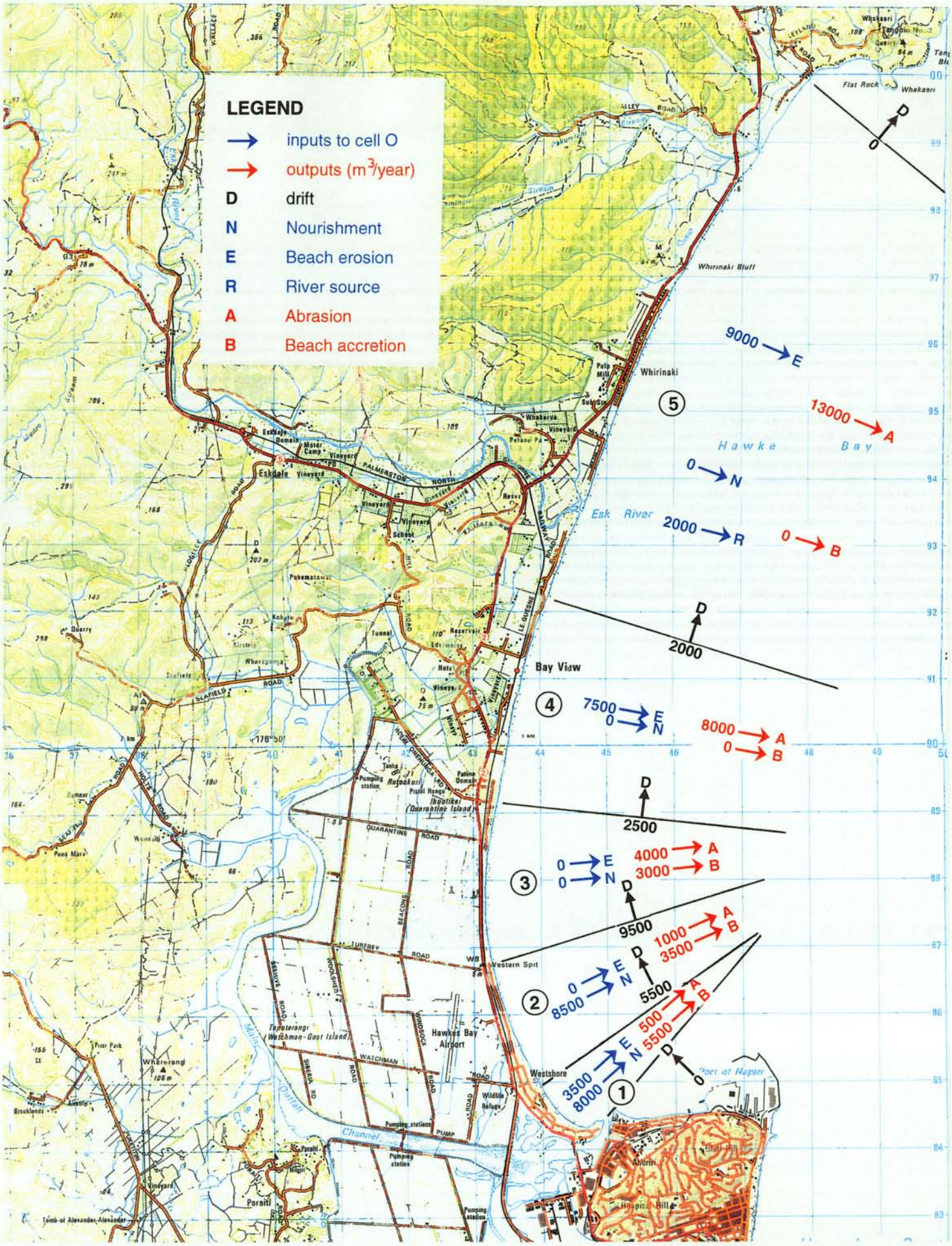
2.3.3 Sediment Transport

The beach system and barrier is generally covered in gravel although during periods of finer weather some sand has been observed to build up within the intertidal area. The nearshore comprises fine to medium sands. The nearshore sands can be thought of as a foundation for the beach, allowing the incident waves to be refracted and shoaled as they move towards the coastline. Loss of nearshore sand material allows the waves to propogate further inshore and be more energetic on the beach system. The gravels tend to have a relatively steep beach profile, a berm and an erosion scarp. They are highly permeable, but generally result in a reflective beach system.

When in dynamic equilibrium the orientation of a beach will be in balance with the predominant wave climate and the sediment being introduced into the beach system. If there were no external sources of sediment in the beach system then it would be orientated perpendicular to the net wave energy direction allowing for the diffraction, refraction and shoaling of waves. If there were sediment being introduced to the system, for example in a northerly direction, then the beach would be slightly orientated from the normal of the net wave energy direction in an anti clockwise direction so as to promote the transport of sediment along the beach system. This rate of sediment transport would be equivalent to the input of sediment to the system.

A number of studies have been undertaken to establish a sediment budget for both the gravel and sand system operating in the area. Figure 4 illustrates the sediment budget for the coarse gravel material above mean sea level as given in Gibb (2003). This also includes the benefits of the beach nourishment system for which an allowance of 16,500 cubic metres a year has been made. (This number varies amongst the various reports ranging from 9,000 cubic metres per year as given by Koutsos (1999) to 16,500 as given by Gibb (2003)). In general terms the budget indicates that the beach nourishment scheme has more or less stabilised the beach south of Bayview. At its present levels of nourishment, the beach has also been assisted by the placement of sand in the nearshore area, by the Port of Napier opposite the Esplanade. At Bayview the benefits of the nourishment are not felt. In this area, it would appear that the general beach alignment is in equilibrium with the incident wave climate. Losses are still experienced, however, in this part of the system primarily due to the lack of sediment inputs to the system and abrasion of the gravels.

ASR (2001) has attempted to provide a sediment budget for the sand system, as illustrated in Figure 5. Approximately 123,000 cubic metres per year bypass the port breakwater some of which completely bypasses the shipping channels and the remainder is trapped within the dredged channels. Some of this material is dredged and placed back into the sediment system. From a comparison of bathymetric records (1954,1981) it would appear that there is a general loss of material in the Westshore nearshore system (ASR, 2001). This process would indicate a net northerly or offshore drift of sand. It is noted that the sediment budget as given in Figure 5 is not necessarily supported by other data. For example, Gibb (2003) has calculated from high resolution bathymetric data in Dump Area "R" (see Figure 6) that material is accumulating and that the material is being transported towards the shore rather than back into the port area which is inferred from the work of



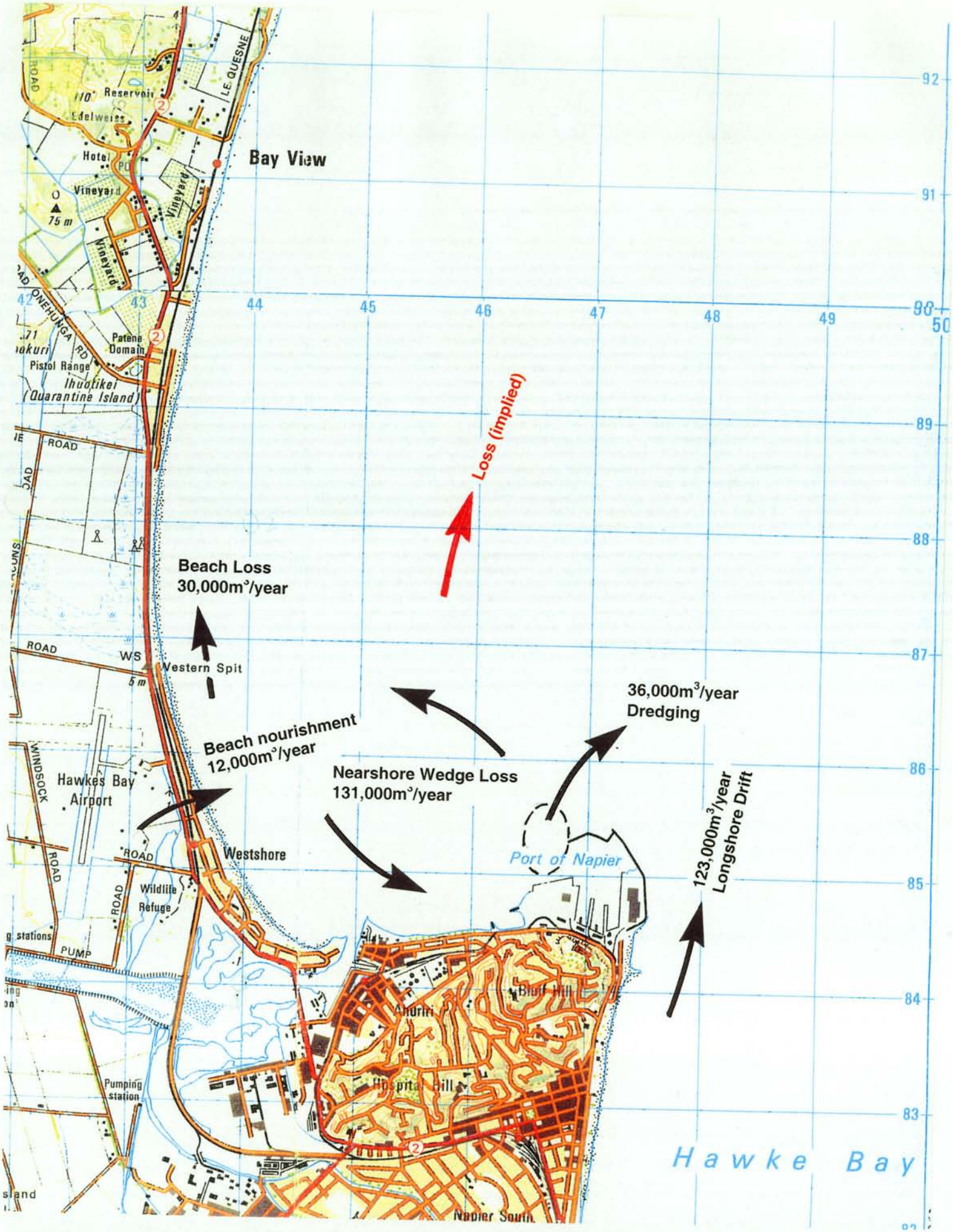
Source: Figure 9; Gibb (2003)

Sediment Budget
(for coarse material above MSL)

Beca

2509903/010

Figure 4

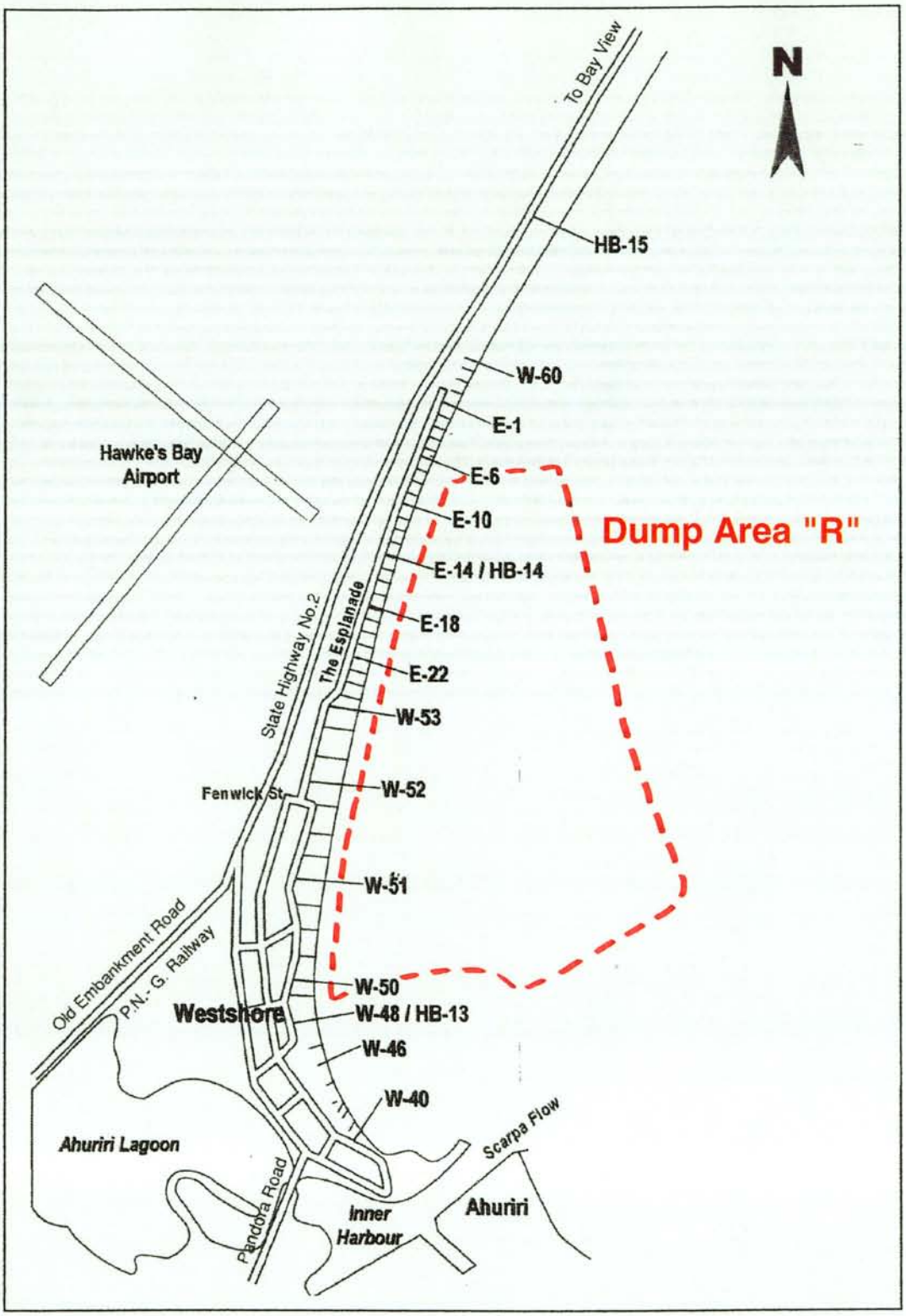


Source: Figure 7.1; ASR (2000)

Total Sediment Budget in Westshore Bay (for all sediments)

2509903/010

Figure 5



Source: Figure 3; Gibb (2003)

Beca

2509903/010

Beach Profile Locations

Figure 6

ASR (2001). Port of Napier also supports the claim that fine sand is being transported towards Westshore Beach. This is a relatively important issue and more field work is required to better understand the sediment budget for the sand material which is important for maintaining the nearshore profile within Hawke Bay.

Discussion with ASR (Shaw Mead, pers.com, 2003) revealed they consider that the coarser fraction of sand is directed from the end of the port breakwater to Bayview. The finer fraction is directed to the port channels and Westshore Bay.

Inspection of Nautical Chart NZ 5712 shows a narrowing of the nearshore area at Bayview, between contours 5 to 10 m CD, compared to the Westshore Bay area. That is, the nearshore profile is steeper in the Bayview area. Coarser material would assist in stabilising a steeper profile.

2.4 Westshore Nourishment Scheme

Gibb (2003) reviewed the performance of the Westshore nourishment scheme and made a number of recommendations for its improvement. If those recommendations were implemented the coastal hazard lines established in 1996 will be adjusted to account for the continued implementation of the nourishment scheme. His recommendations include:

- That nearshore dumping of sand within dump ground "R" should become part of the nourishment scheme and dumping should be as close into the beach system as feasible.
- That the most sustainable source of gravels is from Pacific Beach which can potentially supply 12,000-13,000 cubic metres per year. Nourishment volumes above this level would probably require another source.
- That 12,000 cubic metres be placed on the shore face annually between profiles W40 and W51, as shown in Figure 6, and 2,000 to 3,000 cubic metres be placed between profiles E6 and E18 every two years.
- That the artificial barrier ridge is raised, the beach gradient is reduced, and the level of the swale behind the beach ridge is raised to improve the amenity value and the performance of the nourishment system.
- That the existing monitoring system be enhanced and continued so as to provide quantitative information on the performance and any revisions needed of this scheme.

Since its introduction in 1987, the monitoring data and subsequent analysis has proved that the nourishment scheme has been successful at limiting the historical erosion of the Westshore Beach between Charles Street and the Snapper Park Motor Camp. Variations 2 and 3 which are the study areas for this report are generally both outside the area that benefits from the existing nourishment scheme.

2.5 Previous Options To Reduce Coastal Erosion

Following the 1985 storms, the Hawkes Bay Catchment Board investigated a number of alternatives to protect the coastline from further erosion (Koutsos, 1993). A brief outline of these alternatives is as follows:

Offshore Islands- A series of artificial offshore structures either permeable or solid would reduce wave energy and allow beach material to be deposited in the characteristic curve shape, behind them. These structures would have to be built in water depth of about 5 metres which would make their construction cost ($20 million in 1986) disproportionate to the value of the assets that they would protect.

Groynes- Permeable or impermeable groynes along the beach would tend to reduce the northerly movement of material. However, apart from aesthetic objections to such structures being built on Napier's best swimming beach, it was also considered that their effect could be limited as there is no supply of material from the southern end to maintain a strong littoral movement. The possible shifting of the problem further to the north was also a deterrent.

Shoreline Defences- A solid wall or rock rip-rap along the eroding face was calculated to cost $1,000 per linear metre in 1985. Even putting aesthetics aside, this method was not favoured because ultimately it would result in the complete depletion of the beach due to the reflective action of the breaking waves and the littoral drift to the north. Such a wall would also require periodic extension northwards, because as the beach in front of the well became depleted, the erosion would move in that direction to maintain the longshore drift.

Beach Nourishment- This option, apart from having the lowest initial cost, also satisfied all environmental and technical requirements. It was considered to be the 'natural' approach as it would re-establish the process that man had interrupted with the construction of the Napier Port breakwater. This option would maintain the beach shape and supply the material required for the longshore drift without interruption, and was therefore adopted by the Hawke's Bay Catchment Board and accepted by the Napier City Council which supplied the bulk of the funding for this scheme.

Oldman and Smith (1998) investigated a comprehensive range of potential coastal erosion mitigation measures for the section of beach from the Ahuriri Entrance to Bayview. These measures included groynes, emerging breakwaters, submerged breakwaters, artificial surfing reefs, hardfaced seawalls and nourishment schemes. A summary of their 21 options is contained in Appendix B. Although that study has been criticised by others (Tonkin & Taylor (1999) and the Hawkes Bay Regional Council (1999)) it provides a perspective on the rough order of costs and the impacts of various different options. NIWA concluded that no option gives a complete solution. Moreover any hard engineering option requires some form of beach nourishment in order to maintain the existing coastline. The most expensive options were for emerging breakwaters running parallel to the coast which had a capital cost of over $80 million. The lowest cost options

were maintenance of the renourishment scheme which was estimated to require 10,000 cubic metres per year.

Some coastal works have been built in front of Whakarire Avenue and these are discussed further in Section 3.3.

3 Whakarire Ave: Engineering Options

3.1 Causes of Erosion

The potential for coastal erosion seaward of Whakarire Avenue is primarily due to a lack of sediment supply. Since the construction of the training moles, the reduction in the tidal prism of the Ahuriri Estuary, the loss of direct sediment input from the Tutaekuri River, and the introduction of the Port of Napier breakwater system, natural inputs of sediment into the southern end of the Westshore Beach system have ceased.

The area has also been reclaimed and the orientation of the coastline in this location is not in equilibrium with the incident wave climate. This means that any sand or gravel that is introduced into the local beach system cannot hold its position because the waves drive the sediment northwards.

3.2 Relevant Technical Information

3.2.1 Beach Profiles and Erosion Rates

Beach profiles show the seaward side of the properties on Whakarire Ave being about 3 metres above mean sea level (i.e. 3.9 m CD) and falling down into a lagoon area behind the existing seawall. The seawall is located on the seabed which varies between mean sea level and 0.7 metres below mean sea level (i.e. 0.2 m to 0.9 m CD).

Review of the historical profile data by Gibb (2002) gives an average erosion rate of 0.31m per year, with retreat distances of 8 to 13 m since 1962. According to local residents the erosion of the foreshore area has been arrested since the installation of the seawall in this area.

3.2.2 Design water levels

For the purpose of concept design, the maximum water levels will be based on:

MHWS	1.6 m above CD
Surge setup on open coast	0.7 m
Sea Level Rise	0.4 m
Design Maximum Water Level	2.7 m above CD

Other relevant tide levels are given in Table 1.

3.2.3 Design Wave Events

The immediate nearshore area comprises a rock shelf which promotes strong refraction and shoaling of waves. It is a popular surfing location under certain weather conditions.

Due to the headland constraints at the eastern mole and due to wave focussing on the seaward rock reef, wave attack will be between 10 to 20° (true north). See Figure 7. Wave heights will be restricted to wave breaking conditions on the reef near the existing or any other proposed structures. The wave height is sensitive to the depth of water at the time of adverse sea conditions. Table 2 gives a summary of design wave heights for various sea bed levels for the existing and proposed seawalls.

Table 2: Whakarire Ave: Design Wave Heights

Assumed seabed level (in front of wall)	Still Water Level	Wave Height[1] H_s (m) [2]
0.4 CD[3]	1.6 CD	0.7
	2.0 CD	0.9
	2.3 CD	1.0
	2.7 CD	1.3
-0.6 CD	2.7 CD	1.8

[1] Estimates for the wave heights are based on CIRIA/CUR (1991)
[2] H_s is the significant wave height, which is the average of the highest 1/3 of waves.
[3] CD – Chart Datum (see Table 1)

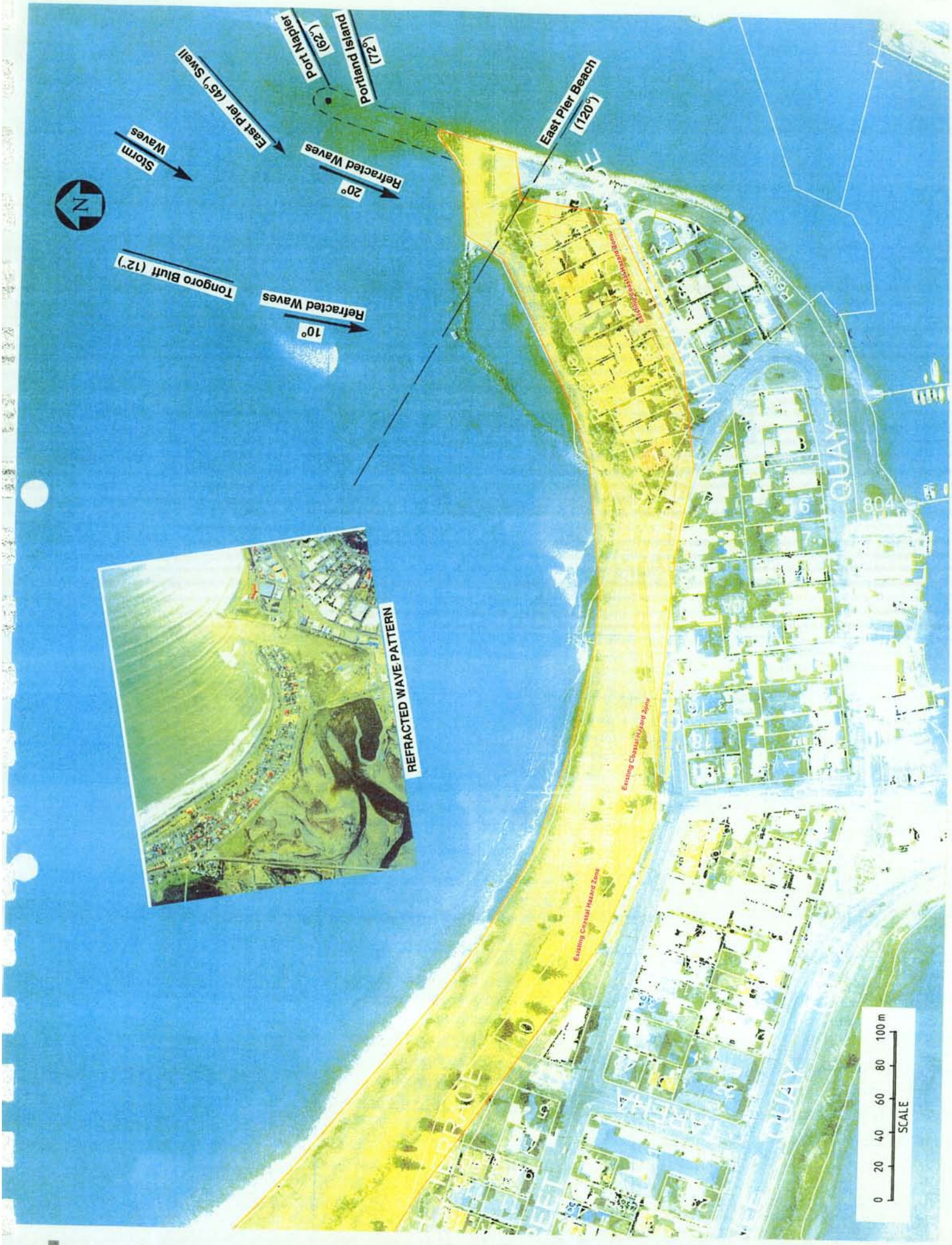
3.3 Existing Seawall

A seawall has been built seawards of Whakarire Avenue as shown in Figures 7. Its alignment was based on protecting a line of old wooden piles . The initial works were constructed about 1994 and comprised of concrete rubble. About 1997, the initial works were dressed with limestone rock armour. Since being in place these works have not suffered any significant damage and have held the shoreline in place (Clive Squire, pers comm, 2003). A lagoon has been created behind the seawall which shows evidence of filling up with fine sand. According to the local residents overwash from the seawall does not impinge on private property.

Inspection of the seawall indicates that the limestone dressing is mainly confined to the leeward side. It is a variable material, ranging from 0.6m to 1.2m in size, and would not normally be classified as rock armour because it is relatively soft. Some of the limestone dressing has moved and small fragments of limestone were found on the landward side of the lagoon. The rear of the properties contains friable fill and rubble material, which could have been part of the original reclamation.

A detrimental aspect of this seawall is that it funnels waves into Westshore Beach. This results in a retreat of the shoreline and the rapid transport of gravel (including nourishment material) in an offshore and northerly direction. That is, the effect of the seawall is to not allow Westshore Beach to create a permanent control north of Charles Street, or a feeder area for the nourishment scheme.

During a significant storm event with elevated sea levels, it is likely that the existing seawall would fail, particularly the limestone dressing material. The structure has a



Whakarire Ave Wave Directions & Constraints

2509903/010

Figure 7

relatively low elevation (up to 2.8 m above CD) and would be easily over topped. Based on the procedures given in CIRIA/CUR (1991) it is estimated that the existing seawall would fail if a significant storm event coincided with a sea level of 400 mm above mean high water springs (ie. 2 m above CD). The existing large concrete rubble is likely to stay in place but breaching and movement of this material could be expected in a large storm.

The revetment section adjacent to the western mole, which is currently used as a carpark/observation area, is significantly higher than the seawall (4.5 m above CD). The exact composition of the revetment is not known although concrete rubble can be seen in the outer face. This revetment section is not likely to receive the same level of wave attack as the seawall because the seabed is higher (at just below mean sea level) and is slightly in the lee of the wave attack because of the western mole. The revetment structure should be able to resist wave attack and wave over topping in moderate/extreme events. This section of revetment should be regularly inspected and repaired as required but no major works are proposed.

3.4 Do nothing option

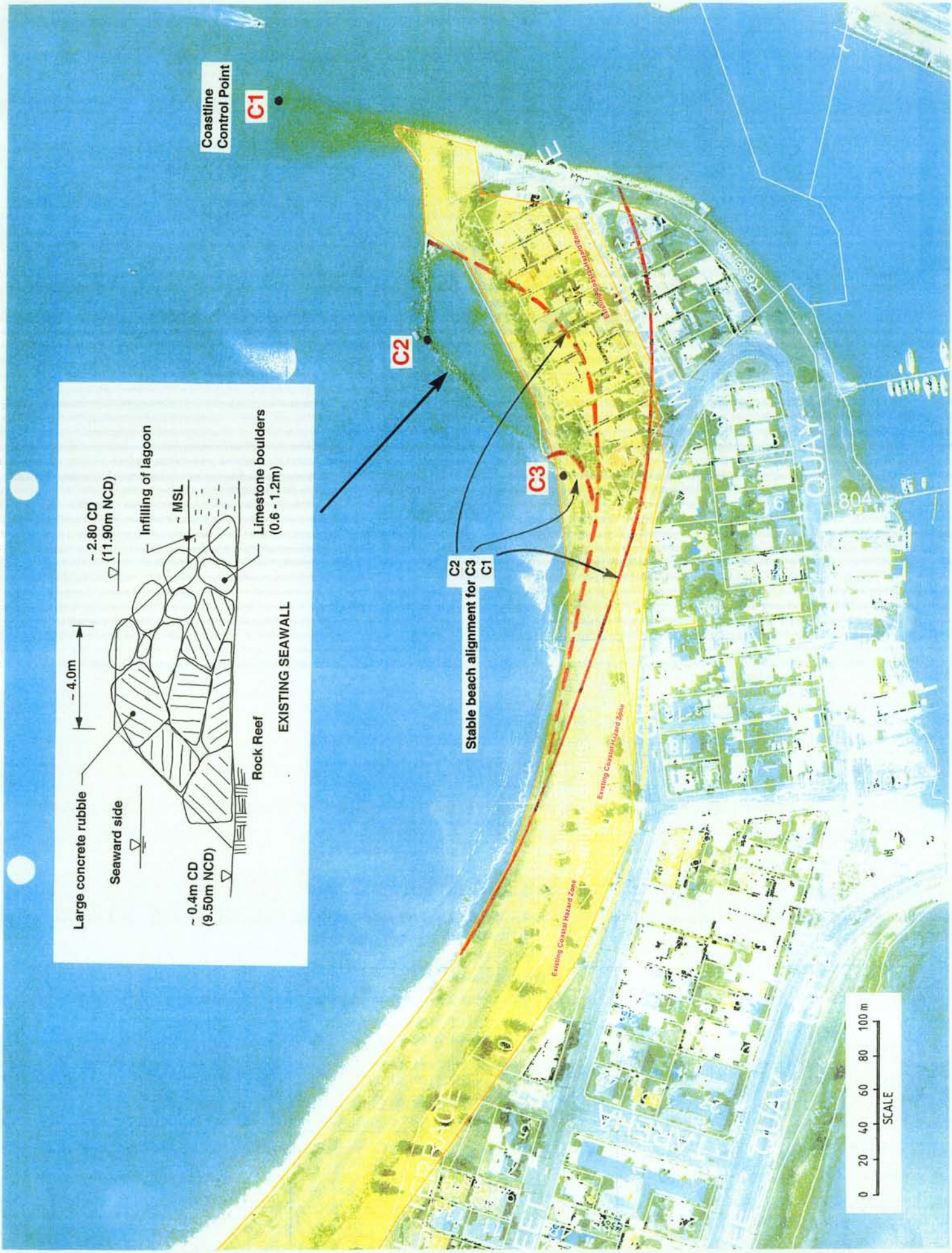
Given that there is no supply of sediment from the south and that the net wave direction is approximately 10 to 20°, Whakarire Avenue would be eroded without protection works. The degree of coastal erosion would depend on the stable control point for the coastline. A number of potential control points are illustrated in Figure 8. Control point C1 is the outer end of the western mole, control point C2 is the knee point in the existing seawall alignment, and control point C3 is at the south western end of the existing seawall. Potential erosion planform profiles are illustrated in Figure 8, based on a crenulate bay being created behind each of these control points with the wave approach direction from 10 to 20° (Silvester & Hsu, 1993). For this assessment , it is assumed that the beach will be renourished north of Charles Street and that shoreline will remain generally in its present location.

If the existing seawall were ineffective in the long term there is the potential for the coastline to retreat such that the houses located on the northern side of Whakarire Avenue would be lost. If the control point moved to the knee in the existing seawall (i.e. point C2) then the coastline would retreat such that half the houses on the northern side would be lost. If the seawall remained intact the coastline would not retreat into the houses on Whakarire Avenue in the long term, provided the existing nourishment scheme continued.

The cost associated with this option would be the clean up of the existing seawall if it failed and the loss of private and public property. Given the historical rates of erosion, it would take many decades for a shoreline to become stable.

3.5 Coastal Protection Options

A number of coastal protection options have been considered in order to protect the land at Whakarire Avenue, as well as potentially enhance the coastal processes in the area. The options considered in detail are described below.



Whakarire Ave: Do Nothing Option

Beca

2509903/010

Figure 8

Other options, such as narrowing the entrance of the Ahuriri Estuary have also been considered. It is expected, however, that such works would be very expensive and compromise existing navigations through the entrance, if the entrance were narrowed to the extent where an ebb delta formed outside the entrance. Furthermore substantial work would still be required in the nearshore area of Whakarire Avenue because its existing alignment would not be able to hold any sediment that bypassed any shoal formation.

In assessing the performance of each option, the focus is on land protection and enhancement of coastal processes. Effects on reef ecology, surfing conditions, landscape and heritage values have not been considered. Also it is assumed that the existing nourishment scheme will continue and that Westshore Beach will remain in its existing location.

For each of the options described below, a rough order of cost is provided for the construction works. More details on the availability of suitable construction materials, their unit costs, and total construction costs are given in Appendix C.

3.5.1 Enhancing existing seawalls (Option W1)

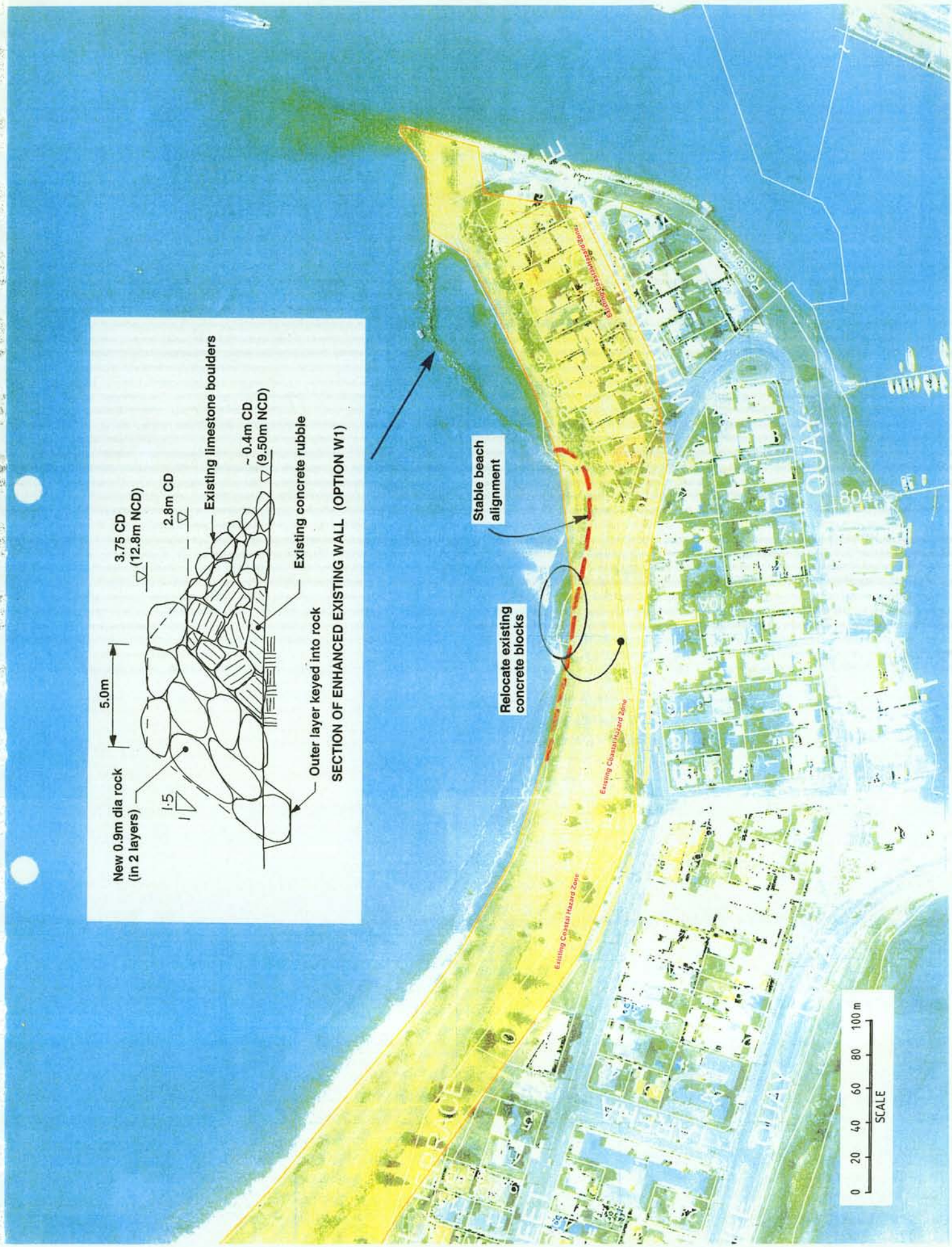
As discussed in Section 3.3, the existing seawall is not considered robust enough to perform satisfactorily in the long term, particularly when allowances for sea level rise are taken into account. Enhancement of the structure is required in order for it to perform adequately for a still water level at 2.7 metres above CD. This would require the existing structure to be raised by 1.0 metres and for the top width to be 5 metres. New rock armour would be imported to form the outer layer which would also be keyed into the rock reef. The final level of the seawall will be generally be lower than the rear of the existing properties.

A layout of this option and enhanced seawall section is shown in Figure 9. The large concrete blocks opposite Charles Street would also be removed to encourage the coastline to form a more continuous plan profile. However with this option there is still the risk that waves will funnel into the Westshore Beach and cause a discontinuity where the wall runs out. The estimated cost of this option is $400,000.

3.5.2 Enhanced Wall and infilling of backshore area (Option W2)

This option would be similar to Option W 1 except that the existing lagoon would be infilled so as to extend the existing reserve area and create a more formal public walkway. Creating a swale through the reserve area will be important to take overspill during extreme storm events. A swale would be aligned at the edge of the existing properties and be drained down to Westshore Beach.

This option is illustrated in Figure 10. Its benefits and disadvantages will be similar for option W1 except that in gaining any resource consents an application may be required for a reclamation. Construction costs are estimated at $525,000.



Whakarire Ave: Enhanced Seawall

Beca

2509903/010

Figure 9

Whakarire Ave: Enhanced Seawall & Infilling of Backshore

Beca

2509903/010

Figure 10

3.5.3 Groyne to limit wave focusing (Option W3)

Because the existing alignment of the seawall causes funnelling of waves onto Westshore Beach it would be better to align the seawall either parallel or perpendicular to the predominant direction of the wave attack. Such an option is illustrated in Figure 11. The existing seawall parallel to the beach would be extended and a new groyne created from Westshore Beach to meet this extended seawall. The waves approaching Whakarire Avenue will generally be dissipated and reflected off the new structure rather than being redirected towards Westshore Beach. In order for the beach profile on Westshore Beach to be continuous it is recommended that the existing concrete blocks be relocated to the backshore area. The cost estimate for this option is $750,000.

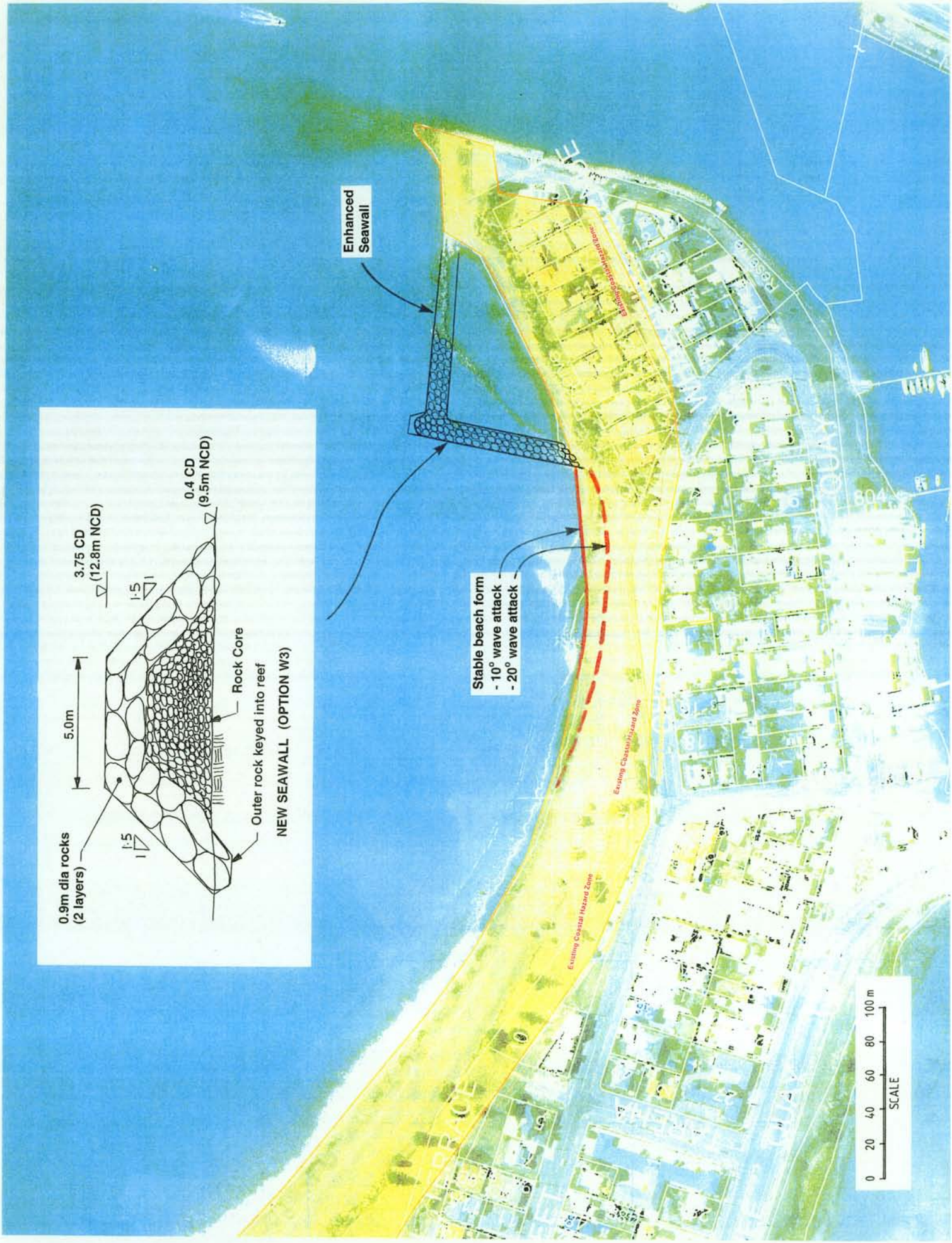
3.5.4 Wave spending beach (Option W4)

This option would entail installing a groyne type structure over the existing concrete blocks and creating a new beach between that location and the existing seawall. This option is illustrated in Figure 12. The performance of the new beach is difficult to assess because of the wave funnelling effects from the existing seawall will still be evident. It may require that the new beach be created of large size gravel in order to be able to dissipate redirected waves. Rip currents could also be generated by the new groyne structure during significant wave events. The cost estimate for this option is $725,000.

3.5.5 Attached breakwater with beach creation (Option W5)

This option would entail enhancing the existing seawall parallel to the coastline and extending it out 100 metres as shown in Figure 13. The benefits of this option is that it will allow incident wave energy to be dissipated and reflected off the seawall which would encourage a sheltered area to be formed in the lee of the seawall. It should form a crenulate shaped beach which would marry in with the existing beach, provided the large concrete blocks were removed from the shoreline. The new beach could be created from sand material that is frequently dredged from the Ahuriri entrance channel. It is likely that any sand beach created immediately in the lee of the seawall would transition into the gravel beach. An estimated cost of this option is $675,000.

The extended breakwater section is relatively expensive because it is founded in deeper water, about 1 metre deeper than the existing seawall. This requires a seawall section significantly larger in size, as illustrated in Figure 13.

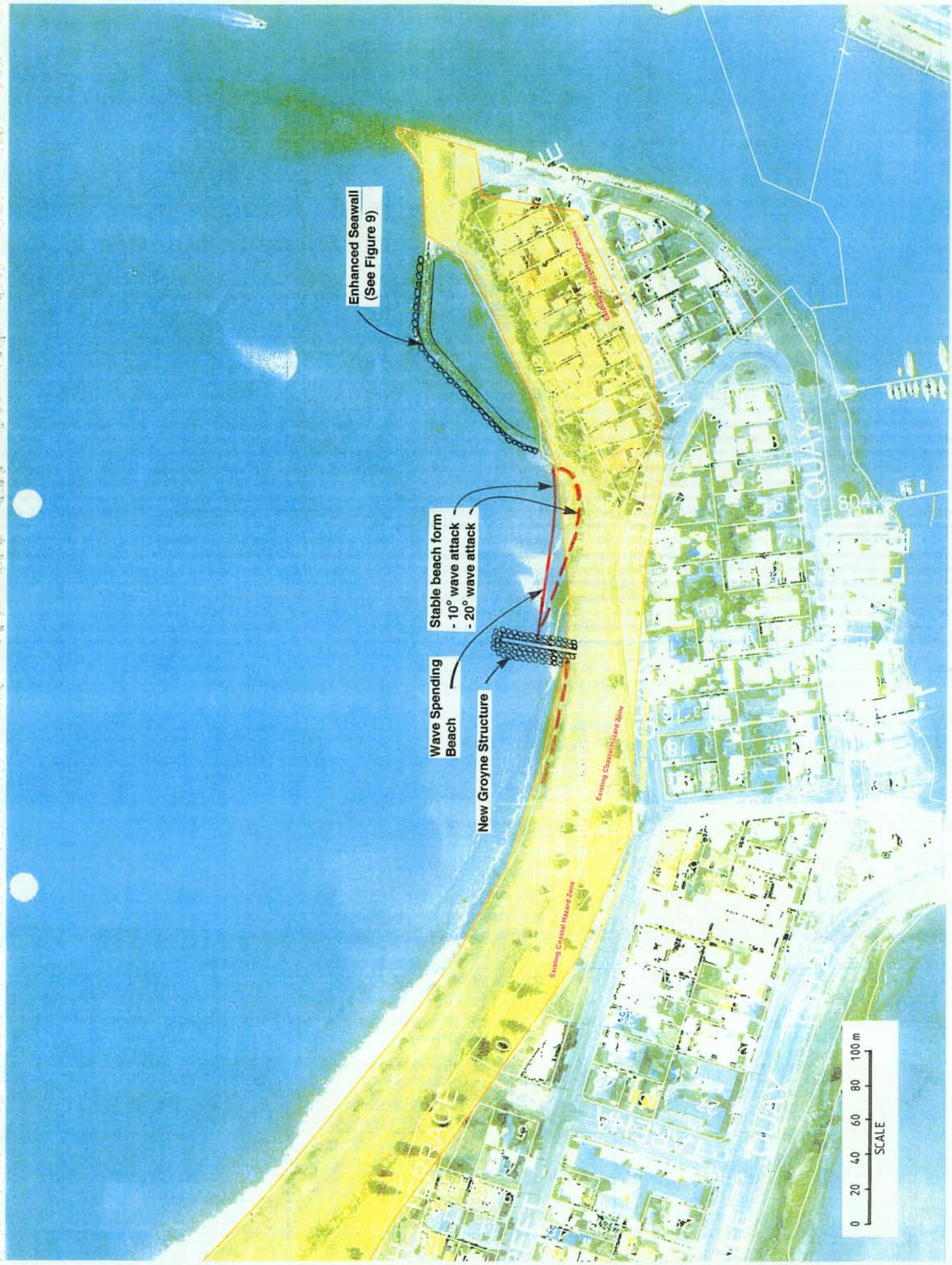


Whakarire Ave: Groyne to Limit Wave Focusing

Beca

2509903/010

Figure 11

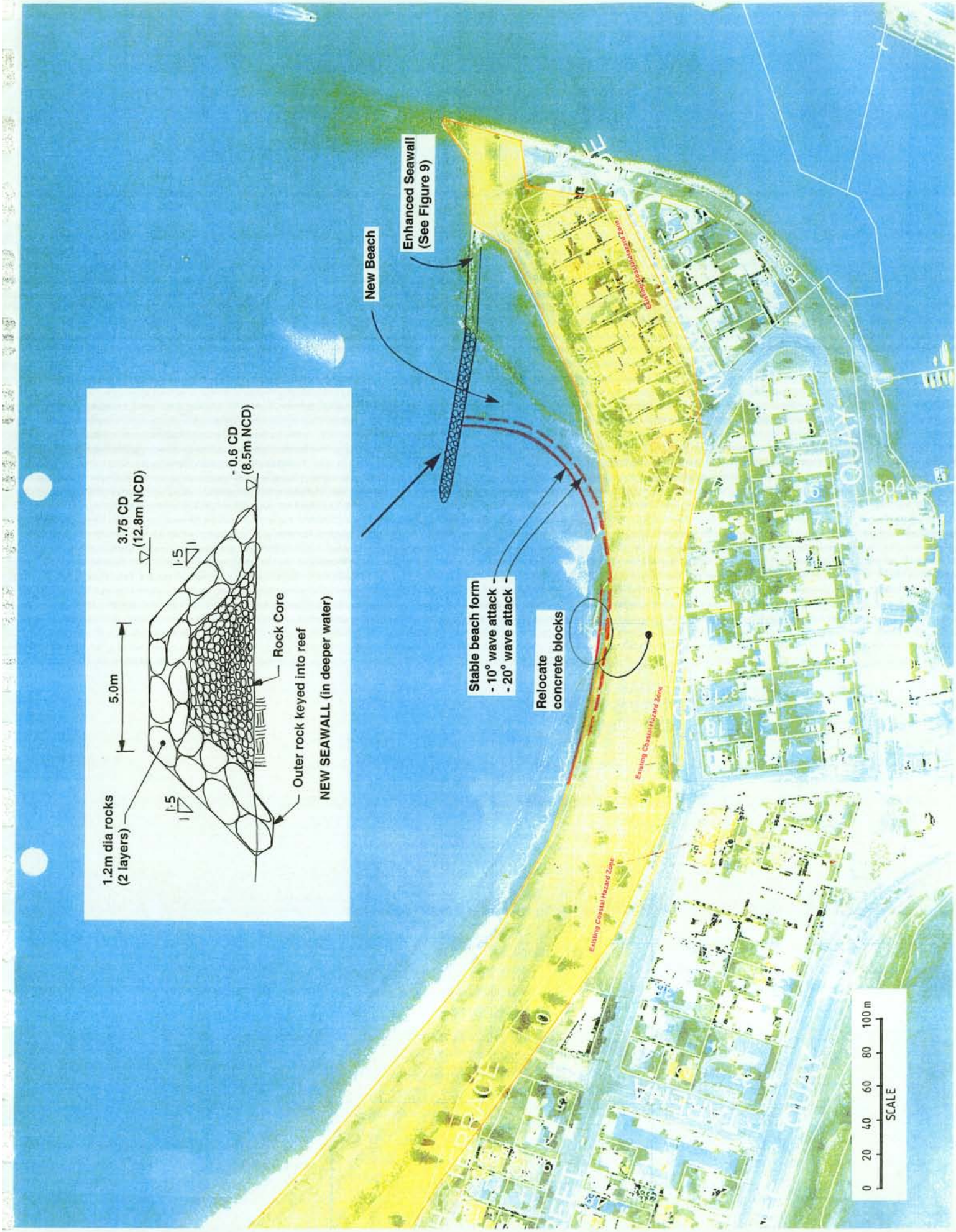


Whakarire Ave: Wave Spending Beach

beca

2509903/010

Figure 12



Whakarire Ave: Attached Breakwater with Beach Creation

2509903/010

Figure 13

4 Bayview: Engineering Options

4.1 Causes of Erosion

According to Gibb (2003) the beneficial effects of the existing nourishment scheme run out at Snapper Park Motor Camp. Therefore most of the Bayview study area is outside the influence of the current nourishment scheme.

Moreover the beach alignment in the study area is in dynamic equilibrium as it is perpendicular to the net wave direction. Open sea waves to this location have a window of 82 to 147°, controlled by the headlands at Portland Island and the Port of Napier breakwater. Local refraction will further narrow this window. The alignment of the beach over the 5 km reach varies between 102 to 110° degrees normal. According to Gibb (2003) there is little sediment input into this system, as shown in Figure 4, with about 2,000 cubic metres per year entering and leaving it. The sediment budget indicates that the main loss is due to the abrasion of the gravel. The net effect is that sediment is being lost from this sub-system within Hawke Bay.

The river channel of the Esk River is mobile. It causes erosion of the nearby beach system when migrating in a southerly direction (Gibb, 1996).

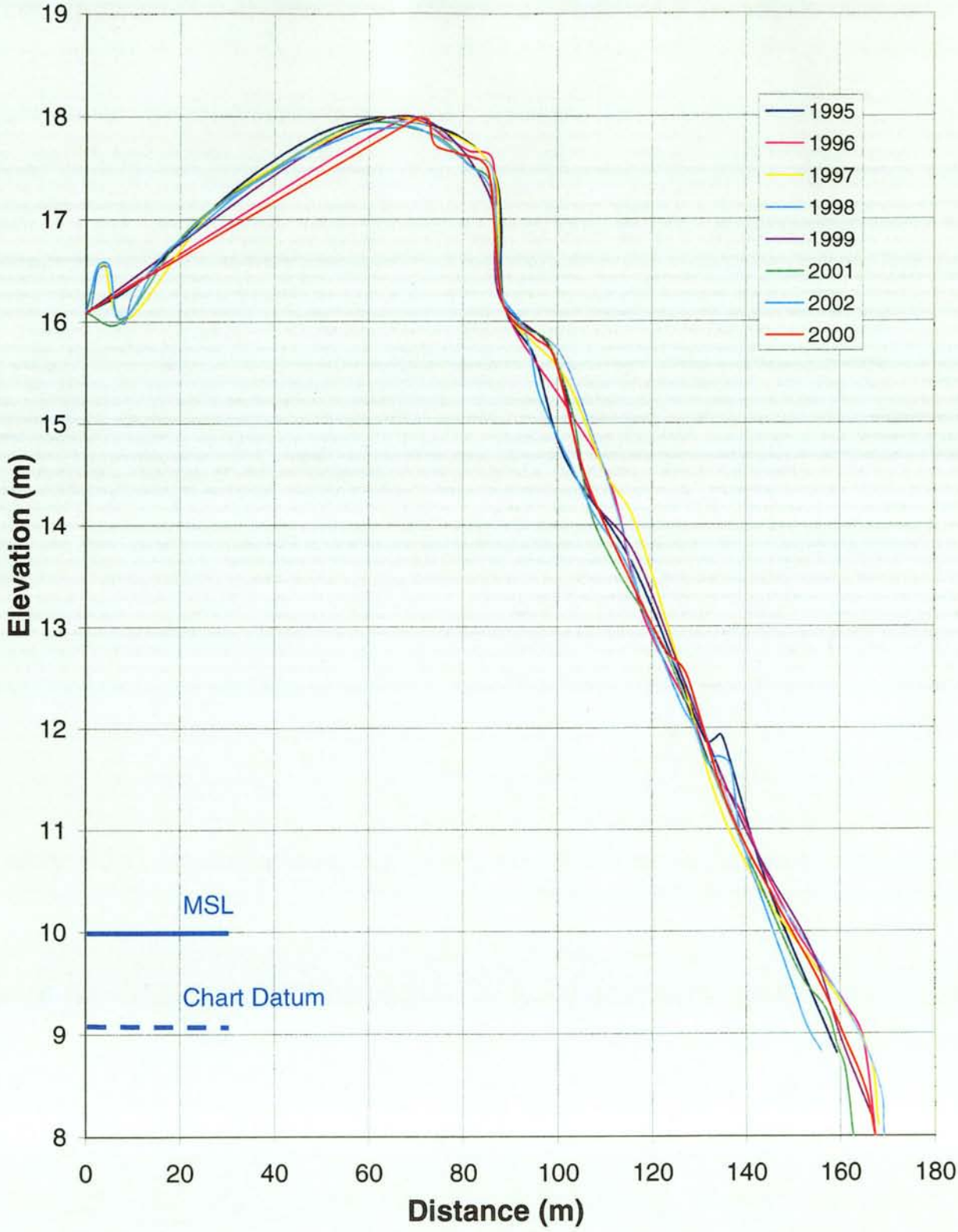
Less appears to be known about the sediment budget within the nearshore which provides the foundation for the beach system. Further work would be required to know whether the nearshore is accreting or eroding within the study area. It is understood that a detailed bathymetric survey of Hawke Bay is to be undertaken in the near future. This survey, when compared to other historical surveys, will give an indication of how the nearshore is behaving. Loss of sediment from the near shore will allow higher energy waves to propagate into the beach system which would further exacerbate abrasion and also lower the beach face due to the general lowering of the nearshore area. Accretion in the nearshore could potentially have the opposite impact.

There are no significant coastal protection works in place.

4.2 Relevant Technical Information

4.2.1 Existing beach profiles and erosion rates

Existing beach profiles are illustrated in Figure 14 for profile HB 17. Gibb (2002) has reviewed the historical profile data as well as aerial photographs to give the erosion rates listed in Table 3. It would appear that from the Fergusson South to Le Quesne Road that erosion has been identified over the period 1962-2001. Whereas from Le Quesne Road to 1 km from the Esk River mouth the beach is in dynamic equilibrium and has not recorded erosion over the period 1936 to 2001. Near the river mouth, the beach has suffered erosion due to river migration.



Bayview: Beach Profile at Station 45A
(HB-17 between Rogers Rd and Le Quesne Rd)

Beca

2509903/010

Figure 14

For any coastal erosion assessment it is important to establish whether a coastline is in an erosional state because it is only then that engineering works can be justified. It is stressed that the coastal reach in dynamic equilibrium along Le Quesne Road is based on an analysis period from 1936 to 2001 which is the period just after the 1931 earthquake up until now. It is known that Westshore Beach was in the state of accretion from the period of the earthquake up until 1960's. So it may well be that Le Quesne Road is now in a state of erosion, if the coastal reach were analysed from the early 60's through to 2001. It is not known why the 1936-2001 period was selected but if there were aerial photographs taken in the mid 60's along Le Quesne Road then these should be analysed to assess whether long term erosion is occurring. It is important to recognise that short term fluctuations, both erosional and accretion, will occur within any beach system but that any short term fluctuations would not justify engineering works.

Based on the information in Table 3, this assessment will only consider engineering options from Fergusson South through to Le Quesne Road, approximately covering a reach of some 2.5 kilometres. Any engineering works to combat erosion near the Esk River mouth would need to be based on understanding of the historical river migration. River training schemes could be considered but have not been included as part of this assessment.

Table 3: Erosion Rates at Bayview (Gibb, 2002)

	Location of Station	Distance North of Harbour (km)	Period of Analysis	Beach barrier retreat (-) or advance (+)	Rate (m/yr)
33	Ferguson St South	4.9	1962-2001	-6.5	-0.17
34	Ferguson St South	5.1	1962-2001	-8.0	-0.21
35	Ferguson St South	5.3	1962-2001	-9.0	-0.23
36	Profile HB-16, Fannin St	5.535	1962-2001	-5.0	-0.13
37	Ferguson St North	5.7	1962-2001	-10.0	-0.26
38	Ferguson St North	5.9	1962-2001	-9.0	-0.23
38a	Snapper Park Motor Camp	6.0	1962-2001	-8.0	-0.21
GILL RD/ROGERS RD AREA					
39	Snapper Park Motor Camp	6.1	1962-2001	-8.0	-0.21
40	Gill Rd South	6.20	1962-2001	-11.0	-0.28
41	Gill Rd North	6.30	1962-2001	-12.0	-0.30
42	Mer Place	6.44	1962-2001	-12.0	-0.30
43	Rogers Rd	6.55	1962-2001	-12.0	-0.30
44	Reserve	6.80	1962-2001	-12.5	-0.32
45	Reserve	7.05	1962-2001	-14.0	-0.36
45a	Profile HB-17	7.11	1962-2001	-14.0	-0.36
46	Reserve	7.30	1962-2001	-10.0	-0.15
47	Reserve	7.43	1936-2001	-7.0	-0.11
LE QUESNE RD AREA					
48	Pig Styes 1936 Aerial	7.55	1936-2001	-3.0	-0.05
48a	Le Quesne Rd 1936 Aerial	7.68	1936-2001	0.0	0.00
49	Le Quesne Rd 1936 Aerial	7.80	1936-2001	4.0	0.06

	Location of Station		Distance North of Harbour (km)	Period of Analysis	Beach barrier retreat (-) or advance (+)	Rate (m/yr)
50	Le Quesne Rd	1936 Aerial	8.05	1936-2001	1.5	0.02
51	Le Quesne Rd	1936 Aerial	8.30	1936-2001	0.0	0.00
52	Le Quesne Rd	1936 Aerial	8.55	1936-2001	11.0	0.17
52a	Profile HB-18	1936 Aerial	8.68	1936-2001	13.0	0.20
53	Anthony Place	1936 Aerial	8.80	1936-2001	24.0	0.37
53a	Le Quesne Rd	1936 Aerial	8.93	1936-2001	5.0	0.08
54	Thurley Place		9.05	1936-2001	-9.0	-0.14
55	Le Quesne Road		9.18	1936-2001	-7.0	-0.11
56	Le Quesne Road		9.30	1936-2001	-12.0	-0.19
57	Le Quesne Road		9.43	1936-2001	-29.0	-0.45
58	Barrier North		9.55	1936-2001	-21.0	-0.32
58a	Barrier North		9.65	1936-2001	-47.0	-0.72
59	Barrier North		9.75	1936-2001	-51.0	-0.79
60	Barrier North		9.85	1936-2001	-29.0	-0.45
61	Esk River Mouth		10.00	1936-2001	-13.5	-0.21

4.2.2 Design Water Levels

For the purpose of concept design, the maximum water levels will be based on:

MHWS	1.6 m above CD
Surge set-up on open coast	0.7 m
Sea Level Rise	0.4 m
Design Maximum Water Level	2.7 m above CD

Other relevant tide levels are given in Table 1.

4.2.3 Design Wave Events

For the structures being considered in the engineering options wave heights will be restricted to the wave breaking conditions at the site of the structure. Offshore, the design significant wave height will be in the order of 7 metres but such a wave would break considerably offshore from Bayview. Wave height estimates for the various locations of the structures, as discussed in Section 4.3, are given in Table 4 based on CIRIA/CUR (1991).

Table 4: Bayview: Design Wave Heights

Assumed seabed level (in front of wall)	Seabed slope	Still Water Level	Wave Height[1] H_s (m) [2]
-5m CD[3]	1:100	2.7 CD	4.2
0.7 CD	1:10	2.7 CD	2.4

[1] Estimates for the wave heights are based on CIRIA/CUR (1991)
[2] H_s is the significant wave height.
[3] CD – Chart Datum (see Table 1)

4.3 Do Nothing Option

With no mitigation, the coastline within Bayview will erode with the erosion scarp retreating an average of 30 metres over the next 100 years from Ferguson South to Le Quesne Rd. Hazard maps reflecting this retreat (but also allowing for sea level rise, a factor of safety, and short term fluctuations) are contained in Napier City maps, based on Gibb (2002). The cost associated with this scenario is the loss of property. It is also noted that if the beach does retreat then this will have some updrift and downdrift effects as those reaches of the beach reorientate to accommodate any eroded shoreline.

In 1986, just south of Snapper Rock camp site at the intersection of Fannin Road, a house had installed its own protection works as it was suffering erosion. Since that time, the beach has accreted substantially to cover those protection works.

It may be that the benefits of the Westshore nourishment scheme have yet to be appreciated over the whole of the Bayview area. Only further beach profile monitoring of this area will indicate whether the benefits will flow any further northward than Snapper Rock camp site.

The information available to date indicates that Napier City need to be cautious with any future development in the area if no action is taken.

4.4 Coastal Protection Options

Coastal protection options are discussed below as a means of combating erosion if long term coastal erosion were proven.

For the purpose of this study only options over the southern 2.5 kilometres of the study area (Variation 3) have been considered. Similar options could be extrapolated over the northern 2.5 kilometres to gain an appreciation of the total overall cost for the study area. Near the Esk River some careful training works could assist in minimising channel migration, thereby reducing potential erosion of the local barrier and beach system.

More details on the cost of each option are contained in Appendix C.

4.4.1 Emerging Breakwaters (Option B1)

Emerging breakwaters are structures located in the nearshore that are visible above the water line. Wave energy can directly dissipate on the breakwater structure. Also the change in wave pattern through defraction and refraction encourages the build up of sediment immediately in the lee of the structure. This can be a salient or a tombolo feature depending on the distance of the structure from the coast. This salient relationship is given in Figure 15, based on the work of Silvester and Hsu (1991).

Constructing breakwaters without reshaping of the coast to allow for the salient formation will result in accretion of material in some areas and the erosion of material in others. For the layout illustrated in Figure 16, in which the breakwaters are 250 m long, 250 m offshore and spaced at 1250 metres, the salient are likely to form offshore by about 100 m. This would result, however, in the retreat of the coastline by some 30 metres in between the emerging breakwaters. In order to overcome this adverse effect, sediment would need to be imported to give the formation shown in Figure 16. It would require the importation of some 200,000 cubic metres of material.

Wave energy will still be dissipated on the beach. If it assumed that abrasion loss is proportional to wave energy then the layout shown in Figure 16 would reduce the abrasion loss by 22%. Therefore ongoing nourishment of the beach would be required in order to hold the new coastline.

It is estimated that the cost of the emerging breakwater system would be about $24 million and would have annual operating cost of $75,000 for sediment renourishment. Ongoing costs for maintaining the breakwater structures would probably be small in the initial years but more significant 20 years out with storm damage and also abrasion of the armour units.

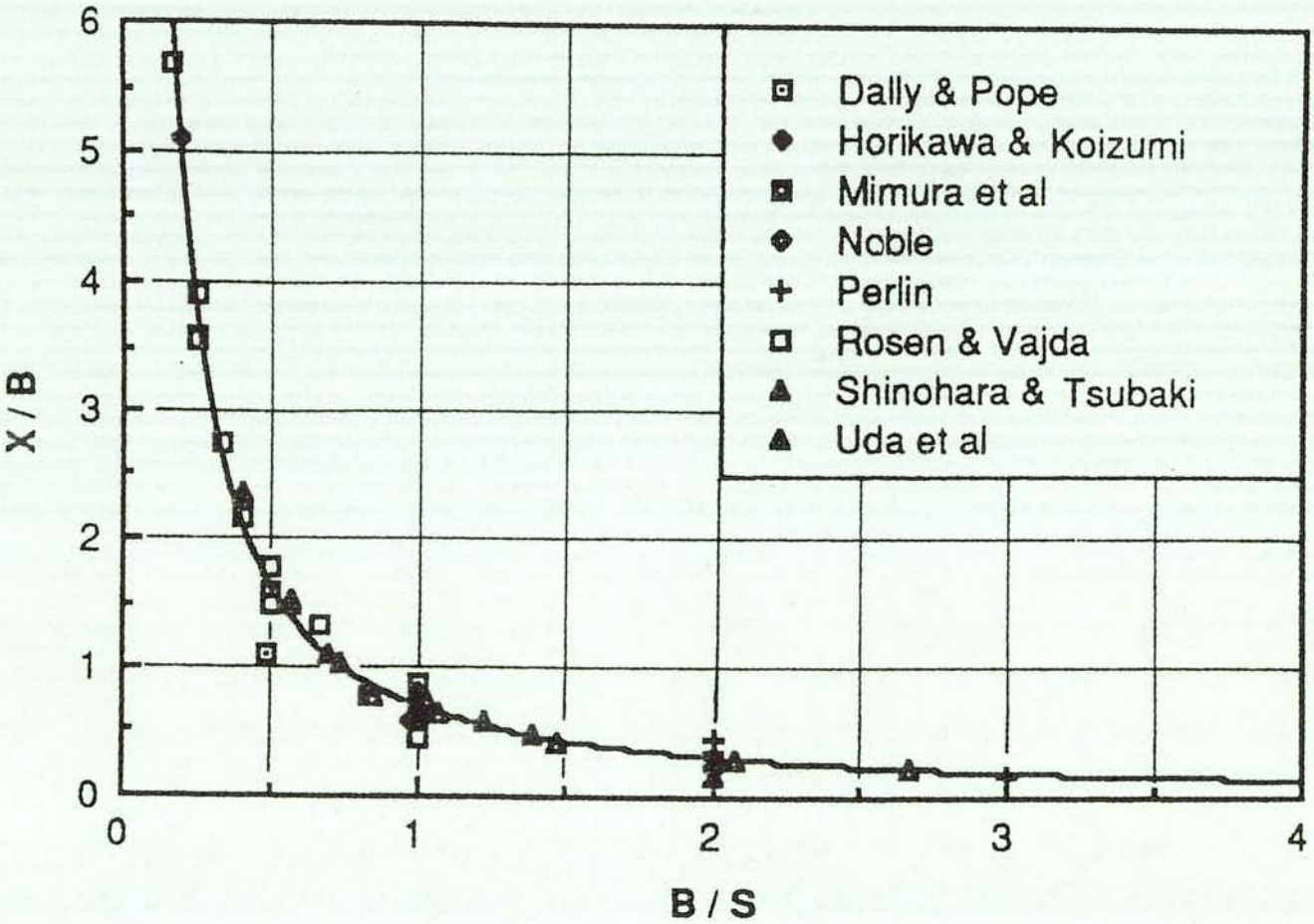
4.4.2 Submerged Reefs (Option B2)

Submerged reefs are similar to emerging breakwaters except that they encourage rotation of the waves so as to create a more stable coastline. Salients will form behind the submerged reefs and follow approximately the same relationship as given in Figure 15.

Advantages of submerged reef systems over emerging breakwaters are that they can enhance surfing conditions, promote marine life and the structures are significantly less expensive (commensurate with an increase in vulnerability). A potential submerged reef system and layout is given in Figure 17. Some 200,000 cubic metres of material would need to be imported to form the new coastline, prior to implementation otherwise some retreat in between the reefs could be expected. Estimated construction cost for the 3 submerged reefs and the initial nourishment along the 2.5 kilometres of coastline is $9 million with an annual beach renourishment cost of $75,000.

4.4.3 Coastal Revetments (Options B3 & B4)

A revetment structure could be included within the back berm area so as to minimise the loss of land in behind the revetment. Over time the beach will still continue to erode



B = length of reef
S = distance from reef to shoreline
X = distance from reef to salient

Source: Silvester and Hsu (1991)

Beca

Salient Relationship for Offshore Reefs

2509903/010

Figure 15

Bayview: Emerging Breakwaters

2509903/010

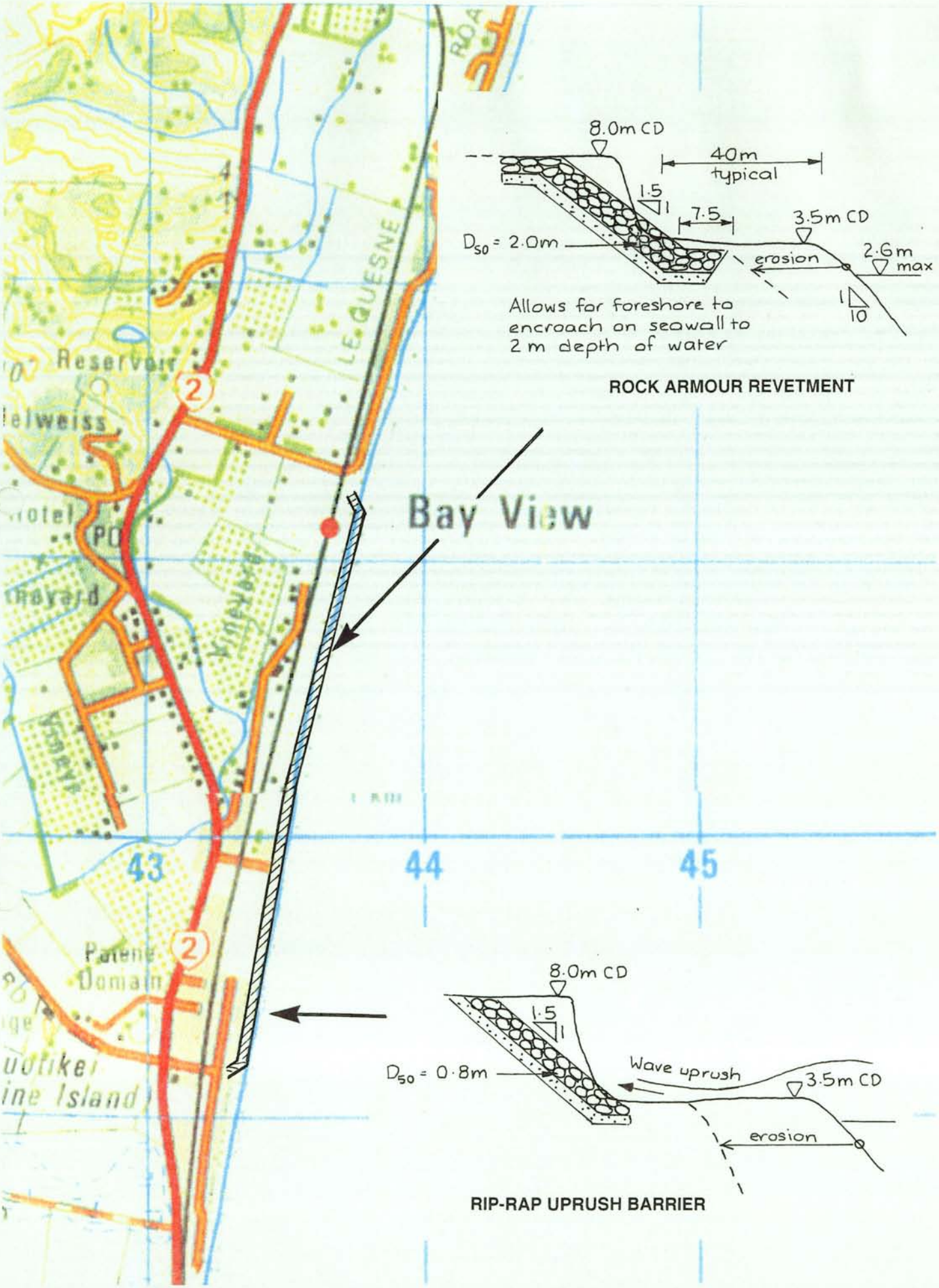
Figure 16

Bayview: Submerged Reef

beca

2509903/010

Figure 17



Bayview: Revetment Structures

beca

2509903/010

Figure 18

because of the abrasion and will, at some time in the future, make contact with the revetment structure. At this point the toe of the structure will lower providing further protection but ongoing erosion will inevitably create a hardening of the coastline and a complete loss of the beach. In effect, it will become a headland. This has consequences for the areas either side of the revetment as they too will retreat in response to erosion which may be exacerbated at the transition between the hardened structure and the softer coastline. It is not the revetment that causes the erosion. The erosion process was already happening before the structure was there. It is just that eventually the coastline will become completely hardened with little or no beach.

Two types of structures can be used for revetments and these are illustrated on Figure 18. The rock armour revetment would resist a certain amount of direct wave attack and would survive effectively for water depths up to 2 metres. If the coastline were eroded beyond that point it would begin to undermine the structure and dislodge the armour. The cost to implement this type of structure along 2.5 kilometres of coastline is estimated at $22 million.

A simpler structure could be built as an uprush barrier that would provide hardening of the scarp area and protect the land but would only be able to manage wave uprush with velocities in the order of 5 m/s. This structure would not be as robust as the rock armour revetment but it would take many years before the eroding coastline would encroach on it. Such a structure is estimated to cost $5 million.

4.4.4 Beach Renourishment (Option B5)

As shown in Figure 4 the loss of course material from the beach system is estimated at 8,000 cubic metres per year, mainly from abrasion. If a similar amount of material were introduced into the system on a regular basis this would offset the abrasion loss.

Importing an annual quantity of 8,000 cubic metres would cost in the order of $96,000 per year which, when capitalised at an 8% discount rate, would have an equivalent life time cost of $1.5 million.

Because there appears to be little littoral drift along this section of the coastline placement of the material will need to be more specific than the Westshore nourishment scheme. The material will need to be graded out to a predetermined profile roughly conforming to the existing beach shape.

4.4.5 Other Options

Groynes along this section of the coastline will not be effective as there is minimal littoral drift in the system. Moreover, they would not reduce the amount of abrasion because the increase in coastline length would be negligible compared to the existing situation. Groynes could encourage the loss of material offshore through the rip currents that develop at the ends of the structures.

Dumping of sediments in the nearshore would be beneficial for this section of the coastline as it would encourage the breaking of waves further offshore and reduce the wave energy on the coastline. Further information on the type of nearshore material and whether it is in an accretion or erosion phase would need to be established before this option could be seriously considered. Surplus sediment would need to be identified for this purpose and may not readily be available.

5 Conclusions

5.1 Whakarire Avenue

Although the existing seawall structure has remained intact since 1994 and affords some level of protection to the existing coastline, it is not robust enough to be considered as a long term structure for the protection of the coastline.

The existing structure funnels waves into Westshore Beach and causes rapid transport of sediment both offshore and along shore. It does not enable a stable control point or feeder area for the current renourishment scheme.

Modest solutions are available which would provide both protection of the properties along Whakarire Avenue and enhance the coastal processes. For example, Option W5 would encourage the creation of a bathing beach on the lee side of the attached breakwater which would then marry into the existing gravel beach of Westshore.

Little more investigation work is required for the design of structures because the design waves will be depth limited. A survey of the seabed area along the proposed alignment is required, along with some geotechnical investigations of the rock reef to establish the best method for keying the structure into the rock.

Resource consents will be required for any new structure. In addition to the coastal processes, it would particularly need to address the effects on:

- existing surfing conditions
- reef ecosystem
- heritage values
- landscaping

5.2 Bayview

Historical analyses of beach profiles and aerial photographs indicate that there is some long term erosion at the southern end of Bayview and that the coastline is in dynamic equilibrium over the northern section except near the Esk River (Gibb, 2002).

Further monitoring is necessary to confirm the status of the coastline. An immediate action should be to obtain aerial photographs from the mid 1960s and compare it with the latest aerial photographs to confirm or otherwise the extent of stability over the northern section of the Bayview area. Ongoing monitoring of the coastline would also indicate if there is any lag in the benefits that may be afforded to this section of the coastline from the Westshore nourishment scheme.

It is understood that detailed bathymetry work may soon be undertaken in the nearshore area. Comparison of that data with historical hydrographic records would indicate whether the nearshore area is accreting or eroding. If it were eroding or stable, offshore disposal of sediments would assist in reducing abrasion losses.

Given the level of existing development and the assessment of erosion potential along this coastline a cautious approach is required. Large scale hard engineering options will be very expensive for this section of the coastline and probably not warranted at this stage.

If on-going monitoring indicates that there is long term persistent erosion along the Bayview coastline then it would appear that renourishment of the shoreline would be the most cost effective and environmentally acceptable solution. This will require regular inputs of sediment into the system and shaping it within the existing beach profile.

5.3 Recommended Preferred Options

Based on existing information and the assessment of the coastal processes carried out in this report, the following options are recommended:

5.3.1 Whakarire Avenue

Option W5 is an attached breakwater as illustrated in Figure 13. The new breakwater would act as a headland control and encourage a new beach to be created on the leeward side. This new beach could be created from sand dredged from the Ahuriri entrance channel. It will transition into the gravel and sand beach of southern Westshore. This section of the beach will be more stable and could act as a feeder beach for the nourishment scheme.

There is the opportunity to create a higher backshore area by filling in the existing lagoon which may have wider benefits to the community. This could only be confirmed after consultation.

5.3.2 Bayview

From data available to date, it would appear that some sections of this coastline are stable and others are experiencing erosion.

Further monitoring of the beach profiles and nearshore area is recommended so as to clearly establish whether long term erosion is persisting.

The coastal hazard zones should be adopted as a planning measure to control future development in the area. A cautious approach is recommended.

6 References

- ASR, 2001, *Westshore Coastal Process Investigation*, for Napier City Council
- CIRIA/CUR, 1991, *Manual on the use of rock in coastal and shoreline engineering*, CIRIA Special Publication 83, CUR Report 154.
- Gibb J, 2003, *Review of the Westshore Nourishment Scheme – Napier City*, for Napier City Council by Coastal Management Consultancy, , Ref: CR 2002/6
- Gibb J, 1996, *Entrance and Esk River Mouth* for Napier City Council by Coastal Management Consultancy, Ref: C.R 96/2.
- Gibb J, 2002, *Review of the 1996 Coastal Hazard Zone between Ahuriri Entrance and Esk River Mouth* for Napier City Council by Coastal Management Consultancy Ref: CR 2002/1.
- Hawke's Bay Regional Council, 1994, *Coastal Processes, Nearshore Suspended Sediment in Hawke Bay*, Ref T.S 94/3.
- Koutsos P T, 1993 *The Westshore Beach Nourishment Scheme* for IPENZ Conference
- Koutsos P T, 1999, *Westshore Coastal Protection, Evaluation of Public Submissions* for Napier City Council.
- Koutsos P T, 2003. *Evaluation of the Westshore Beach Renourishment Scheme*, for Foreworld Developments Ltd.
- Laing A K, Gorman R M (2000), *The Ocean Wave Climate Around New Zealand From Satellites and Modelling*, in Water and Atmosphere
- Oldman JW, Smith R K, 1998,*Westshore Coastal Protection Options Study* for Napier City Council by NIWA, Ref: NCC80201/1
- Silvester R, Hsu , 1991, *Coastal Stabilisation*
- Smith R K, 1993, *Westshore Beach Erosion Review* for Napier City Council by NIWA Ecosystems, Ref: Consultancy Report No. NAP004.
- Tonkin & Taylor Ltd, 2000, *Review of the options and methods of analysis for Coastal Protection at Westshore*, for Napier City Council Ref: 17980.

▪ Appendix A

Photographs

Photo 1 – Whakarire Ave : Existing Seawall

Photo 2 – Whakarire Ave : Lagoon and shoreline

Photo 3 – Whakarire Ave : Westshore Beach

Photo 4 – Whakarire Ave : Existing concrete blocks

Photo 5 – Bayview : Beach at 29 Fergusson St

Photo 6 – Bayview : Beach at 48 Le Quesne Rd

Photo 7 – Bayview : Backshore at 83 Le Quesne Rd

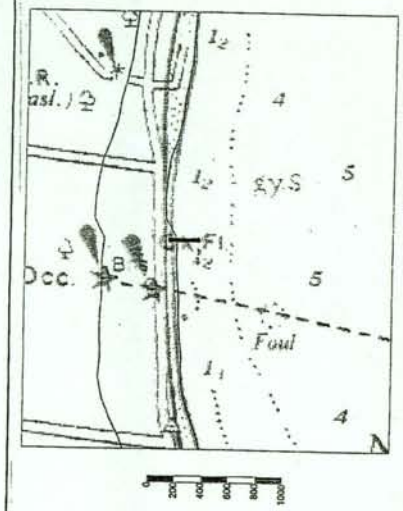
Photo 8 – Bayview : Backshore and berm at 87 Le Quesne Rd

▪ Appendix B

Summary of NIWA (1998)

Single 200m groin north of the Beacons

Option 1



Max seaward migration **49m**

Max landward migration **38m**

Length of coast which migrates landward **1680m**

Length of coast which migrates seaward **1700m**

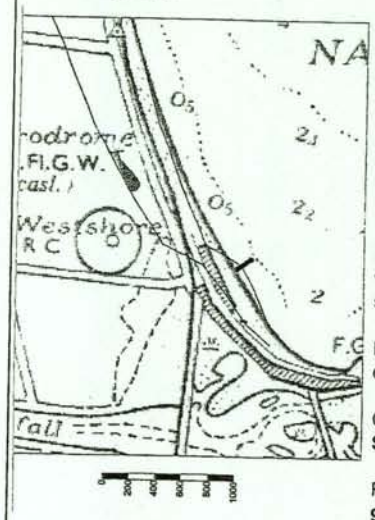
Maintenance cost **$2,800pa**

Capital cost **$700,000**

Renourishment **9600m³/annum**

Single 200m groin near the Surf Life Saving Club

Option 2



Max seaward migration **96m**

Max landward migration **85m**

Length of coast which migrates landward **1570m**

Length of coast which migrates seaward **830m**

Maintenance cost **$2,800pa**

Capital cost **$700,000**

Renourishment **9600m³/annum**

Three 200m groins centred around the Beacons

Option 3

Max seaward migration **67m**

Max landward migration **55m**

Length of coast which migrates landward **3120m**

Length of coast which migrates seaward **3420m**

Maintenance cost **$8,400pa**

Capital cost **$2,100,000**

Renourishment **9000m³/annum**

Five 200m groins

Option 4

Max seaward migration **94m**

Max landward migration **81m**

Length of coast which migrates landward **4650m**

Length of coast which migrates seaward **4080m**

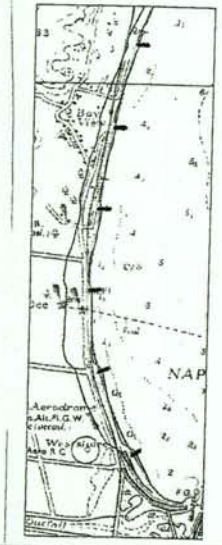
Maintenance cost **$14,000pa**

Capital cost **$3,500,000**

Renourishment **8400m³/annum**

Six 200m groins

Option 5



Max seaward migration **94m**

Max landward migration **81m**

Length of coast which migrates landward **5030m**

Length of coast which migrates seaward **4730m**

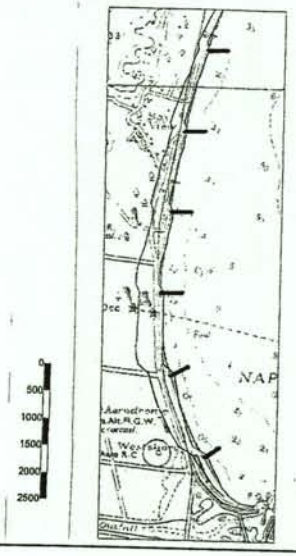
Maintenance cost **$16,800pa**

Capital cost **$4,200,000**

Renourishment **8100m³/annum**

Six 400m groins

Option 6



Max seaward migration **103m**

Max landward migration **89m**

Length of coast which migrates landward **5020m**

Length of coast which migrates seaward **4800m**

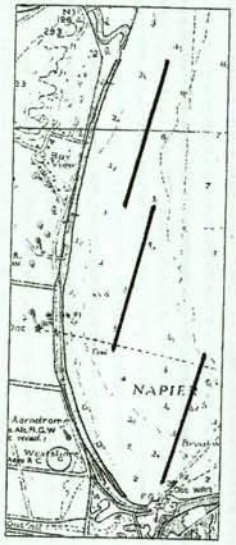
Maintenance cost **$26,400pa**

Capital cost **$6,600,000**

Renourishment **7800m³/annum**

Three Emerged Breakwaters running NNE

Option 7



Max seaward migration **78m**

Max landward migration **51m**

Length of coast which migrates landward **5300m**

Length of coast which migrates seaward **7900m**

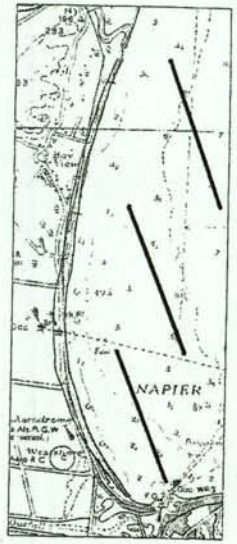
Maintenance cost **$332,000pa**

Capital cost **$83,000,000**

Renourishment **3500m³/annum**

Three Emerged Breakwaters running NSE

Option 8



Max seaward migration **78m**

Max landward migration **50m**

Length of coast which migrates landward **5950m**

Length of coast which migrates seaward **7000m**

Maintenance cost **$332,000pa**

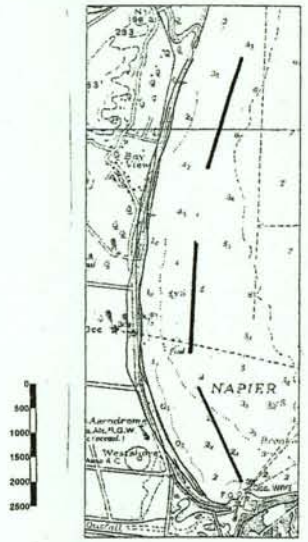
Capital cost **$83,000,000**

Renourishment **4300m³/annum**

2509903\CPOptionsNIWA1998.xls
30/05/2003

Figure B1

Three emerged breakwaters running shore parallel



Option 9

Max seaward migration **87m**

Max landward migration **67m**

Length of coast which migrates landward **5800m**

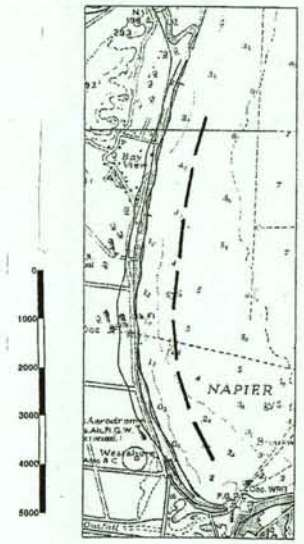
Length of coast which migrates seaward **7100m**

Maintenance cost **$316,000pa**

Capital cost **$79,000,000**

Renourishment **4600m³/annum**

Seven submerged breakwaters running shore parallel



Option 10

Max seaward migration **78m**

Max landward migration **47m**

Length of coast which migrates landward **6360m**

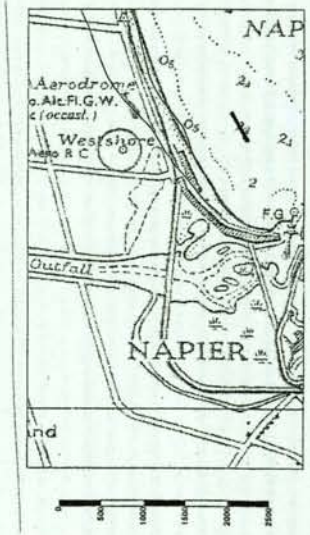
Length of coast which migrates seaward **4780m**

Maintenance cost **$44,000pa**

Capital cost **$11,000,000**

Renourishment **6500m³/annum**

Single submerged breakwater at the Surf Life Saving Club



Option 11

Max seaward migration **83m**

Max landward migration **46m**

Length of coast which migrates landward **2650m**

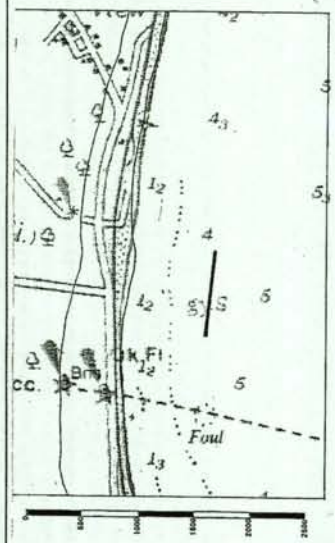
Length of coast which migrates seaward **2100m**

Maintenance cost **$2,100pa**

Capital cost **$540,000**

Renourishment **9100m³/annum**

Single submerged breakwater to the north of the Beacons



Option 12

Max seaward migration **81m**

Max landward migration **45m**

Length of coast which migrates landward **2000m**

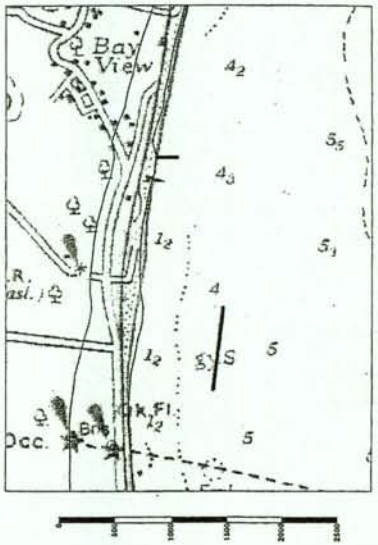
Length of coast which migrates seaward **1800m**

Maintenance cost **$6,000pa**

Capital cost **$1,500,000**

Renourishment **9400m³/annum**

Option 12 plus associated groin



Option13

Max seaward migration **82m**

Max landward migration **42m**

Length of coast which migrates landward **2600m**

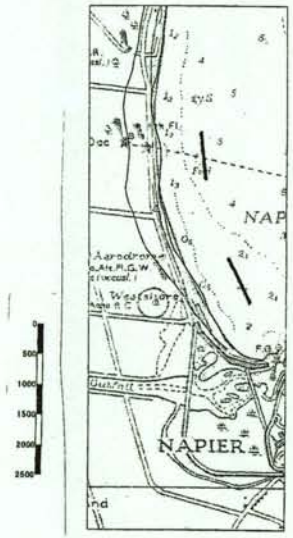
Length of coast which migrates seaward **2100m**

Maintenance cost **$8,800pa**

Capital cost **$2,200,000**

Renourishment **9100m³/annum**

Two submerged breakwaters at Westshore



Option 14

Max seaward migration **80m**

Max landward migration **49m**

Length of coast which migrates landward **3050m**

Length of coast which migrates seaward **2500m**

Maintenance cost **$12,000pa**

Capital cost **$3,000,000**

Renourishment **8800m³/annum**

Option 14 plus associated groins

Option 15

Max seaward migration **78m**

Max landward migration **49m**

Length of coast which migrates landward **2700m**

Length of coast which migrates seaward **3500m**

Maintenance cost **$17,600pa**

Capital cost **$4,400,000**

Renourishment **8300m³/annum**

Figure B2

Coastal Protection Options (NIWA, 1998)

2800m seawall south of Bay View Option 16

Max seaward migration **60m**

Length of coast which migrates landward **6550m**

Length of coast which migrates seaward **2850m**

Maintenance cost
$64,800pa rubble
$162,000pa seawall
$14,800pa geotextile

Capital cost
$16,200,000 rubble
$40,500,000 seawall
$3,700,000 geotextile

Renourishment
9200m^3/annum

1500m seawall south of Bay View Option 17

Max seaward migration **34m**

Length of coast which migrates landward **8300m**

Length of coast which migrates seaward **1500m**

Maintenance cost
$34,000pa rubble
$85,000pa seawall
$8,200pa geotextile

Capital cost
$8,500,000 rubble
$21,250,000 seawall
$2,050,000 geotextile

Renourishment
9700m^3/annum

450m seawall south of Bay View Option 18

Max seaward migration **21m**

Length of coast which migrates landward **8900m**

Length of coast which migrates seaward **450m**

Maintenance cost
$11,600pa rubble
$29,000pa seawall
$1,600pa geotextile

Capital cost
$2,900,000 rubble
$7,250,000 seawall
$395,000 geotextile

Renourishment
9700m^3/annum

2850m seawall centred around the Beacons Option 19

Max seaward migration **93m**

Length of coast which migrates landward **9020m**

Length of coast which migrates seaward **2850m**

Maintenance cost
$64,800pa rubble
$162,000pa seawall
$14,800pa geotextile

Capital cost
$16,200,000 rubble
$40,500,000 seawall
$3,700,000 geotextile

Renourishment
8900m^3/annum

2850m seawall centred around the Gap Option 20

Max seaward migration **72m**

Length of coast which migrates landward **9210m**

Length of coast which migrates seaward **2850m**

Maintenance cost
$64,800pa rubble
$162,000pa seawall
$14,800pa geotextile

Capital cost
$16,200,000 rubble
$40,500,000 seawall
$3,700,000 geotextile

Renourishment
9600m^3/annum

2850m seawall centred at Westshore Option 21

Max seaward migration **17m**

Length of coast which migrates landward **9730m**

Length of coast which migrates seaward **2850m**

Maintenance cost
$64,800pa rubble
$162,000pa seawall
$14,800pa geotextile

Capital cost
$16,200,000 rubble
$40,500,000 seawall
$3,700,000 geotextile

Renourishment
9900m^3/annum

▪ Appendix C

Rough Order of Costs

Appendix C – Rough Order of Costs

C1 Introduction

Obtaining large rock in the Napier area is difficult, particularly if hard dense rock up to 2 metres diameter is required. Beca has been in contact with Winstone Aggregates which have advised that they could supply significant volumes of rock armour from the Tauhara Quarry in Taupo. This rock has a density between 2.4 to 2.6 tonnes per cubic metre and could be supplied at a cost of $90 per cubic metre.

Filter rock that requires grading between 100-250mm may also need to be sourced from outside the Napier region. Beca has been advised that, for a reasonably quantity of this material, it could be delivered to site for $80 per cubic metre. For estimate purposes a blend of local and out of region rock will be assumed at a supply rate of $40 per cubic metre.

To take delivery and place the rock on land it would be an additional $10 per cubic metre, and to place underwater would be $15 per cubic metre.

If concrete armour were used, then the rule of thumb is that it would cost in place 50 to 100 percent above the supply rate for concrete. For 30 MPa concrete, Certified Concrete in Napier has advised that the supply rate is $155 per cubic metre.

Pea gravel is delivered to the Westshore site currently from Pacific Beach for a rate of about $5 per cubic metre. In the discussions with Winstone Aggregate they could supply pea gravel from a river quarry at about $20 per cubic metre and as beach run material at $10 per cubic metre. Costs could be reduced if royalties were not payable as the works would be for the benefit of the Napier community. For estimate purposes of it will be assumed that gravel for beach nourishment, other than from Pacific Beach, can be supplied to site at $10 per cubic metre.

There are a number of 40 tonne concrete blocks located adjacent to Whakarire Avenue that should be removed into the backshore area as they are interrupting the beach profile. It is understood that a 100 tonne digger located in the Region would be able to shift these concrete blocks. Establishment costs would be $5000 and its hourly rate is $500. We have allowed a nominal amount of the $25,000 to move the concrete blocks.

All the above costs relate to the supply and placement of materials and equipment. In addition a preliminary and general (P&G) item of about 10% should be added to allow for this item. Because the project is at a conceptual stage, other items will need to be included that have not been allowed for to date and for the risk elements within the project, a contingency allowance of 15% has been included in the cost estimates for land based works and 25% for marine based work (eg working off a barge).

All costs exclude GST, resource consents and engineering.

C2 Options for Whakarire

Option W1: Enhanced Existing Wall

Unit Cost of Wall (per m length) of rock armour

■ 16 m^3 @ \$100/ m^3	1600
■ 1 m^3 of key @ \$50/ m^3	50
	\$1650/m

Total Construction Costs

■ 175 m of wall @ \$1650/m	289,000
■ Relocation of concrete blocks	25,000
■ Contingency & PG (25%)	78,000
	\$392,000

Option W2: Enhanced Seawall & Infill Backshore Area

Total Construction Costs

■ 175 m of wall @ \$1650/m	289,000
■ Relocation of concrete blocks	25,000
■ Infill backshore 8,500 m^3 @ \$12/ m^3	102,000
■ Contingency & PG (25%)	104,000
	\$520,000

Option W3: Groyne to limit wave focussing

Unit Cost of new wall (to 0.4m CD)

■ 26 m^3 of rock armour @ \$100/ m^3	2600
■ 13 m^3 of rock core @ \$50/ m^3	650
■ 1 m^3 of key @ \$50/ m^3	50
	\$3300

Total Construction Costs

■ 150 m of new wall @ \$3300/m	495,000
■ 50 m of amended wall @ \$1650/m	83,000
■ Relocation of concrete blocks	25,000
■ Contingency \$ PG (25%)	150,000
	\$753,000

Option W4: Wave Spending Beach

Total Construction Costs

■ 175 m of amended wall @ $1650/m	289,000
■ 50 m of wall as per W3 @ $3300/m	165,000
■ 6000 m^3 of gravel @ $22/ m^3	132,000
■ Contingency & PG (25%)	146,000
	$732,000

Option W5: Attached Breakwater For Beach Creation

Unit Cost of new deeper wall (to –0.6 m CD)

■ 40 m^3 of rock armour @ $100/ m^3	4000
■ 12 m^3 of rock core @ $50/ m^3	600
■ 1 m^3 of key $50/ m^3	50
	$4650/m

(Note: if concrete armour were used in place of the rock armour the unit cost would be similar or slightly less than the above costs).

Total Construction Costs

■ 50m of amended wall @ $1650/m	82,000
■ 100m of new wall @ $4650/m	465,000
■ Contingency & PG (25%)	137,000
	$684,000

C3 Options For Bayview

Option B1: Emerging Breakwaters

Unit Cost (per m length)

■ 53 m^3 of concrete units @ \$275/ m^3	14,600/m
■ 50 m^3 of rock filter @ \$55/ m^3	2,800/m
■ 100 m^3 of rock core @ \$35/ m^3	3,500/m
	\$20,900/m

Total Construction Costs

Length of breakwater over 2.5 km (3 No @ 250m)

■ 750 m @ \$20,900/m	15,700,000
■ 200,000 m^3 of beach material @ \$12/ m^3	2,400,000
■ Contingency & PG (35%)	6,300,000
	\$24,400,000

Annual Cost = 6,200 m^3 @ \$12/ m^3 \$74,000

Option: B2 Submerged Reef

Unit Cost (per m length)

■ 80 m^3/m @ \$70/ m^3	\$5600/m

Total Construction Costs

■ 750m at \$5600/m	4,200,000
■ 200,000 m^3 of beach material @ \$12/ m^3	2,400,000
■ Contingency & PG (35%)	2,300,000
	\$8,900,000

Annual Cost = 6,500 m3 or renourishment material @ \$12/m^3 \$78,000

Option B3: Rock Armour Revetment

Unit Cost (per m length)

■ 60 m^3 of rock armour @ \$100/ m^3	6000
■ 20 m^3 of rock filter @ \$50/ m^3	1000
	\$7,000/m

Total Construction Costs

▪ 2500m @ $7,000/m	17,500,000
▪ Contingency & PG (25%)	4,400,000
	$21,900,000

Option B4: Rock Uprush Barrier

Unit Cost (per m length)

▪ 14 m^3 of rock armour @ $100/ m^3	1400
▪ 3.5 m^3 of rock filter @ $50/ m^3	180
	$1580/m

Total Construction Cost

▪ 2500m of wall @ $1580/m	3,950,000
▪ Contingency & PG (25%)	990,000
	$4,940,000

Option B5: Annual Nourishment

▪ 8,000 m^3 of renourishment material @ $12/ m^3	$96,000/ m^3/year
▪ with contingency	$120,000/ m^3/year
Capitalised at 8% discount rate	$1,500,000