ATTACHMENT A

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BMT WBM

City of Darwin

Coastal Erosion Management Plan

R.B18413.001.04.Final.docx

June 2013

City of Darwin

Coastal Erosion Management Plan

FINAL

Prepared For:	City of Darwin
Prepared By:	BMT WBM Pty Ltd (Member of the BMT group of companies)

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Title :	City of Darwin Coastal Erosion Management Plan
Authors :	Malcolm Andrews, Matthew Eliot
Synopsis :	This report details the investigations, data collation and results of a study into the high shoreline recession rates in some of the coastal areas of Darwin, including cliffs, which is resulting in threats to infrastructure. This report will assist in assessing the causes of erosion and possible mitigation measures.

REVISION/CHECKING HISTORY

REVISION NUMBER	DATE OF ISSUE	CHECKED BY		ISSUED BY	
0	17/4/2012	ME		MJA	
1	10/08/2012	ME		MJA	
2	14/09/2012	ME		MJA	
3	05/02/2013	MB		MJA	
4	11/06/2013	MB		MJA	

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EXECUTIVE SUMMARY

Introduction

The City of Darwin is responsible for the management major sections of the Darwin foreshore including areas with sandy beaches and iconic cliffs. Previous and present-day management issues along this foreshore include access management, ongoing requirements for beach shaping, soft cliff erosion and overwash. The significance and nature of these issues varies along the foreshore according to the local morphology, active foreshore uses and infrastructure. Coastal systems are generally dynamic except in areas where the geology represents significant resistive qualities (rocky shorelines) which characteristically change at a faster rate than many other land systems i.e. beaches will tend to erode and recover whereas rocky shorelines will only erode.

BMT WBM has been engaged by City of Darwin to produce a Coastal Erosion Management Plan for its areas of responsibility. The core objectives of the management plan are to identify sites and infrastructure at risk, assess the processes causing erosion and provide coastal engineering based recommendations for actions to prevent or minimise the erosion. Whilst the purpose of this consultancy is not to investigate the implications of Climate Change, the assessment of sites, risk and recommendations do incorporate the readily foreseeable impacts of rising sea levels. For the purposes of this report, the model of sea level rise used is based on IPCC (2007) which reports 0.3m over 50 years and 0.8m over 100 years.

Background Studies

There have been a number of studies relating to the Darwin shoreline and adjacent coastal zone primarily starting after the dramatic event of Tropical Cyclone Tracy in December 1974. Of particular interest are those which which provide some physical description of the coast and oceanographic processes (Comley 1996-7, Goad 2001, Gray 2002 and SEA 2006-10) as well as geological and geomorphic processes (Young et al 1998, Mulcahy 2001 and Nott et al 2003). Also, a significant amount of recorded physical data (winds, waves, tides and aerial photography) have been captured and this data has been analysed and interpreted with regard to its likely impact on the Darwin shoreline.

The understanding of the interaction of the shoreline (geology/geomorphology) and oceanographic components (waves, tides) at the shoreline is the fundamental basis of understanding drivers and resistance to shoreline recession which ultimately results in a threat to development in the City of Darwin. The Darwin foreshore can be classified, albeit simply, as three different coast types, being sandy beaches, soft rock cliffs and mangrove shore. Each type resists coastal stresses through different mechanisms, which imply their relative susceptibility to future conditions, including severe storm events or projected sea level rise.

The sensitivity of coastal types to water level and wave conditions, including wave direction, requires analysis of a range of environmental conditions. In the absence of a regular wave monitoring program, much of the coastal behaviour is inferred from description of the wind record. In addition to oceanic phenomena, site inspection has indicated that cliff erosion is strongly tied to rainfall runoff. The table below summarises the dynamics associated with each of the coast types and the corresponding measures of sensitivity.

Coast Type	Sandy Beach	Soft Rock Cliffs	Mangrove Shore
Description	Ultra-dissipative macrotidal beach	Variably layered lateritic cliffs with intertidal rock platform	Fringing coastal mangroves, generally fronting low dune
Response to energetic wave conditions	Beach flattening Plan form rotation Dune erosion Dune overtopping / breaching	Dissipation on rock platform (breaking) Talus / block erosion	Dissipation through mangroves (friction) Loss of mangroves
Response to raised sea levels	Landward and upward profile shift	Increased erosion above platform	Mangrove retreat Dune growth
Key sensitivity	High waves coincident with high water levels	Sustained high water levels	High or long period wave conditions
Measure of performance	Dune buffer width and height	Cliff position	Width of mangroves

Risk Assessment

A risk assessment has been undertaken as part of the study to establish a priority ranking for infrastructure protection. The level of risk is taken as the combination of the likelihood of an event occurring and the level of consequence (damage) of the event. This combination was displayed through a risk matrix, from which the level of risk was analysed. Scales for consequence and likelihood and their combination in a risk matrix form the risk criteria.

The likelihood is a combination of the physical processes involved, the regularity of occurrence and the rate of progress to a critical, unacceptable consequence. Estimates of the likelihood risk have been made in this study based on the knowledge of the driving forces (wind, waves, currents, rainfall runoff etc) and the resistive forces (rock characteristics, mitigation measures etc).

Coastal hazards incorporating shoreline erosion and recession and inundation can have varying impacts on coastal lands depending on how that land is valued. For example, the inundation of a private residence may have a short term reversible impact, while cliff failure or recession has a permanent and thus greater consequence.

For management purposes, the consequence of coastal hazards (e.g. from ‘high’ to ‘low’ impacts) will be largely dependent upon the values of private and public assets. The assigning of consequence shall also incorporate the permanency and type of hazards impact, for example short term repairable impacts from inundation, permanent impacts from erosion or undermining of cliffs and seawalls, resulting in risk to life from cliff falls etc.

Concept Actions

Several sites have been identified where high erosion or inundation risks are present. Management options need to be developed to avoid the risk, minimise the risk, or mitigate the risk. Options may reduce the likelihood or the consequence, but more likely, a combination of options will provide the best outcomes. The suite of options will recommend both short-term and long-term issues associated with coastal risk along the City of Darwin coastline. The focus of the management study is to manage risks from coastal hazards to public safety, public and private property, and community and environmental assets.

Cliff Erosion

There are two main locations where the risks associated with ongoing cliff erosion are high. These are East Point and Nightcliff. In both areas the cliff recession appears to exceed the regional average as there is no offshore sloping platform to reduce wave action i.e. in front of the historical military area and either side of the aquatic complex at Nightcliff. However, the cliff recession rate is also locally accelerated by the overflow of stormwater at or near the top of the cliff. The dissolution of the rock by freshwater causes fissures in the cliffs which then concentrate wave energy and exacerbates erosion.

The recommendations for management of cliff erosion have considered the following mitigation strategies:

- Consideration of offshore structures to reduce wave energy approaching the site;
- Use of terminal structures (e.g. seawalls) to reduce erosion at the shoreline;
- Mitigating localised erosion at low water levels by adding rocks to reinforce the platform;
- Reduction of cliff face stormwater discharge to reduce localised accelerated erosion due to formation of fissures which concentrate wave energy; and
- Combinations of the above.

Sandy Beaches

It has been established that the sandy beaches of Mindil Beach and Vesteys Beach are both subject to considerable dynamics in response to variation of environmental conditions but appear to be dynamically stable in the longer term. Dynamic stability means that the beach sand is moved offshore during storm conditions and back onshore during ambient condition with no net change to beach voume or shoreline location. Seasonal high waves and water levels during the wet season commonly produce a beach scarp on the existing dune profile, and under more severe storms, denude the vegetation that has been planted to enhance dune stability. On Mindil Beach, this variation is currently managed through beach scraping and dune reconstruction. Whilst this aids in maintaining the dunes it is sand budget neutral i.e. no extra sand is added to the system nor is it induced to arrive from elsewhere. For Vesteys Beach, the beach and storm scarp are perched above an indurated layer in the inter-tidal zone and sections of the beach scarp has been armoured with riprap adjacent to infrastructure, and left as a near vertical face elsewhere.

Seawalls

A number of seawalls currently exist at several locations along the City of Darwin shoreline. These include Mindil Beach South, Vesteys Beach North and Nightcliff. There is also a low grouted rock revetment at East Point South but this is not of an engineering design standard.

The seawall that exists on Mindil Beach South has been built over several stages primarily as protection to the Sky City Casino. The recently installed rock seawall (approximately 75m in length) appears undamaged with no rock displacement and no signs of overtopping and therefore incorporates reasonable design values. The next rock seawall to the north (approximately 150m in length installed during Casino construction) has suffered rock displacement and possible slumping and has been overtopped indicating that the current seawall is now inadequate and requires maintenance. Adjacent to the north is a geofabric “soft rock” seawall extending a further 70m which is

also inadequate and requires maintenance. An 'end effect' where accelerated erosion occurs adjacent to the wall is present and is expected. This area will need to be maintained after storms.

The management options for slumped and damaged seawalls protecting infrastructure is to maintain or upgrade the seawalls. This design also needs to contain basic elements such as foundation and toe design, geofabric filters and adequate armour rock size. Sea level rise needs to be considered in the design but its implementation (crest elevation) is likely to be delayed until sea level rise is realised.

Foreshore Overwash

Management options for berm overwash lie within the zone between the still water level and the limit of wave runup. These options for consideration include:

- Raising the berm crest by the addition of sand or rock;
- Reducing shore slope to reduce runup (constrained by mangroves);
- Increasing percolation losses to reduce runup e.g. provide coarser material;
- Re-directing the overwash flow to deliver the sand elsewhere e.g. overwash drainage between the berm and the pathway; and
- Raising or relocating the pathway to above the runup level (i.e. rebuild pathway at higher level).

Many of these options are constrained by the width available to carry out works. It is considered that raising the berm crest level is the only viable option with a high likelihood of success. It also has little disruption to existing usage and the natural environment. An increase in the use of rock revetments to reduce overwash will reduce the amenity of these beach areas.

Recommended Actions

Based on the considerations above the following actions have been recommended. The assessments are based on conceptual design arrangements noting the construction practices and materials that are industry standard and have proven successful in adjacent areas and nationally. These options have been costed by a local cost consultant and their full report is given in Appendix A. Generally the costs have included materials, labour, ancillary works and mobilisation/demobilisation costs and exclude GST, Consultants Fees and escalation after 2014.

At East Point North it is recommended that the following actions be undertaken:

- Immediate action to divert as much stormwater discharge as possible from the cliff face. The stormwater may be able to be redirected to adjacent shorelines. Expected cost over 1 year \$311,000.
- Trial loose rock within the main platform fissure only at an estimated cost of \$38,500 over 1 year. If successful continue placement across the site with estimated final cost of \$240,000.
- In the longer term, consider the construction of a seawall at a cost of \$1,450,000 with timing dictated by results of rock trial and re-analysis of the long term cliff recession rate.

At Nightcliff Central it is recommended that the following actions be undertaken:

- Trial loose 3 tonne rock within the platform fissure only at an estimated cost of $25,000 over 1 year.
- Investigate diverting as much stormwater discharge from the cliff face as possible. The stormwater may be able to be redirected to adjacent shorelines. Expected cost over 1 year $286,000.
- Consider the construction of a seawall at a cost of $500,000 with timing dictated by results of drainage works and the loose rock trial.

At Nightcliff North it is recommended that the following actions be undertaken:

- Investigations be initiated to relocate as much stormwater discharge from the cliff face as possible. The stormwater should be redirected to adjacent shorelines with an expected cost over 1 year of $286,000.
- At the same time prepare for the construction of a seawall at a cost of $1,100,000 with timing dictated by results of drainage works and reassessed rates of cliff recession.

<u>High Cost Alternative</u>

In all three locations above the option of an offshore breakwater was considered. Although considered effective these have associated costs in the order of $10,000,000 and will be difficult to construct in a macro-tidal environment. There are also aesthetic and environmental implications.

<u>Sea Level Rise</u>

The predictions for climate change include a seal level rise of 0.8m in 100 years and a possible increase in storm intensity. For the structures considered above this is likely to have minor impact on rock size but may require the structure crests in be increased in elevation. This would be achieved through topping up of the structures over time.

<u>Sandy Beaches</u>

As the beaches are dynamically stable it is recommended that the current management options be continued. In the longer term, if climate change induced sea level rise is realised then the dunes may need to be raised to suit.

<u>Existing Seawalls</u>

It is recommended that maintenance of the small rock and geofabric seawalls on Mindil Beach South be undertaken to return them to original specification. The design should take into account predicted future sea level rise. The maintenance of the rock wall is expected to cost $120,000 and the geofabric wall $60,000.

The small grouted rock revetment at East Point South also needs to be repaired including improved freshwater drainage at a cost of $542,000.

<u>Overwash</u>

It is considered that raising the berm crest level is the option with a high likelihood of success and with least disruption to existing usage and the natural environment. This option would include the

placement of beach sand along the top of the berm to raise the level by 300mm where overwash is occurring to replicate the natural process at a cost of $16,000.

In the longer term, if predicted sea level rise is realised then the management option will need to be changed as the space available to berm raising by sand is limited. When sea level rise exceeds 0.3m (predicted in 50 years) then the option for path protection will need to change to an option which can be placed at steeper slopes i.e. rock.

At Vesteys Beach South where the path is adjacent to the shoreline it is recommended that the path be relocated or raised. An allowance of $48,000 has been estimated for path relocation or raising.

Monitoring

Although the use of phogrammetry has not be successful in this study it is considered a very useful tool for Council to use to monitor beach and cliff movement in the future and reliably establish the rates of recession for critical areas. The usefulness of the photogrammetry can be greatly increased by defining limited study areas (e.g. Mindil and Vesteys Beaches, East Point military area and Nightcliff) and requesting a high resolution historical analysis by the Department of Lands and Planning. It is expected that this analysis, using quality low level photography (say over the last 40 years) will be able to produce a better indication of recession rates than the existing 0.3m/a average from the existing analyses.

Monitoring will be useful into the future to assess the implications of sea level rise and other climate change processes on the Darwin shoreline.

Summary

The recommended options are summarised in the table below.

Location	Mindil South	Mindil North	Vesteys South	Vesteys North	East Point South	East Point North	Nightcliff South	Nightcliff Central	Nightcliff North	All units	All Units
The Problem	Seawall structures damaged.	Nil	Inundation threat to pathway.	Nil	Damage to low revetment.	Cliff recession.	Inundation threat to pathway.	Cliff recession.	Cliff recession.	Project management to ensure satisfactory completion of management actions.	Maintenance of protection structures and beaches including monitoring by photogrammetry.
Proposed Action	Maintain slumped rock seawalls and rebuild geofabric seawall.	Continue beach scraping after each wet season.	Upgrade pathway to above inundation level.	Maintain rock seawalls.	Repair revetment including freshwater runoff drainage.	Redirect freshwater drainage where possible. Trial loose boulders. If unsuccessful build seawall.	Raise beach berm.	Redirect freshwater drainage where possible. Trial loose boulders. If unsuccessful build seawall.	Redirect freshwater drainage where possible. Build seawall.	Project Management.	Ongoing maintenance and monitoring.
The Outcome	Protection of public and private assets.	Protection of public assets.	Protection of public assets.	Protection of public and private assets.	Protection of public assets.	Protection of heritage assets	Reduction of inundation damage to path and loss of amenity.	Protection of public land, assets and preservation of amenity.	Protection of public land, assets and preservation of amenity.	Scheduled tasks completed on schedule and on budget to the satisfaction of the Council and community.	Protection of public land and, assets, preservation of amenity and understanding of cliff recession rates and beach changes.
Cost Estimates (based on 2012 costing, future years need to allow CPI increases)	Maintenance costs Slumped rock seawall $120k Maintenance costs Geofabric seawall $70k	Beach scraping and dune restoration (current practice)	Capital cost $48K	N/A (routine maintenance budget)	Capital cost $542K	Drainage $311k Loose Rock Trial $38.5k Full loose rock $240k Seawall $1.45M	Sand placed on berm $16k	Drainage $286k Loose Rock Trial $25k Seawall $0.5M	Drainage $300k Seawall $1.1M	5 year Project Management Program $250K/yr	Ongoing program at $250K/yr
Timing	0-1 year	Ongoing	0-1 year	Ongoing	0-1 year	2-5 years	0-1 year	2-5 years	2-5 years	0 – 5 years	Ongoing
Sea Level Rise (0.8m in 100 years)	Raise seawalls progressively 0.8m over 100 years	Raise dunes progressivel y 0.8m over 100 years	Raise berm progressively 0.8m over 100 years	Raise seawalls progressively 0.8m over 100 years	Raise seawalls progressivel y 0.8m over 100 years	Raise seawalls progressivel y 0.8m over 100 years	Raise berm progressivel y 0.8m over 100 years – change to rock	Raise seawalls progressivel y 0.8m over 100 years	Raise seawalls progressivel y 0.8m over 100 years	-	-

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1 INTRODUCTION

The City of Darwin is responsible for the management of sections of Darwin foreshore shown in Figure 1-1. Previous and present-day management issues along this foreshore include access management, beach erosion, ongoing requirements for beach shaping, soft cliff erosion, overwash, inundation and cliff hazard. The significance and nature of these issues varies along the foreshore, according to the local morphology, active foreshore uses and infrastructure.

Coastal systems are generally dynamic except in areas where the geology represents significant resistive qualities (rocky shorelines) which characteristically change at a faster rate than many other land systems i.e. beaches will tend to erode and recover whereas rocky shorelines will only erode.

Management of coastal systems therefore requires adaptive techniques where possible. Councils responsible for managing coastlines face increasing risk when static infrastructure (roads, bridges, buildings and other structures) are located such that they prevent adaptive management i.e., removing the capacity of authorities to ‘plan for change’ when there is not the necessary “room to move’. The Council is forced to consolidate areas (most often with hard engineering solutions) in order to prevent/minimise rates of change and protect public and private infrastructure assets. There are also associated political, planning, construction and cost issues. Several coastal areas under the management of the City of Darwin contain permanent infrastructure where the resistive characteristics of the shoreline are insufficient to prevent damage within the term of the assets life. In these cases, coastal erosion is threatening public and private infrastructure such as paths and roadways at Nightcliff and historical features at East Point.

The City of Darwin foreshore provides a buffer to coastal processes, particularly the extreme inundation and wave action that is possible during tropical cyclones, and to a lesser extent during monsoonal storms. This protective role has been a significant factor in the planning and development of land adjacent to the foreshore. Consequently, relatively intense residential development is focused across the higher topography of Fannie Bay, Nightcliff and isolated higher areas near Mindil Beach and Cullen Bay. Development of the lower lying areas adjacent to Mindil and Vesteys beaches has been limited, with large tracts of (revegetated) bushland, parks and sporting fields. Two sites of more significant infrastructure occur at the Sky City Casino and the Darwin Sailing and Trailer Boat Clubs.

In addition to its primary role as a coastal buffer, the foreshore reserve has been developed to facilitate recreational activities and is highly used in this capacity. Paths located along much of the shore are intensely used as a means of transit, for exercise or as a promenade. Major facilities along the foreshore considered by this Plan are Sky City Casino, Darwin High School, Darwin Museum, Darwin Sailing Club, East Point Historic Precinct, Nightcliff Jetty, Nightcliff Swimming Pool and residential areas. Residential properties are located close to shore at Fannie Bay and Nightcliff. The open air site of Mindil Beach Market also has intense seasonal use.

The City of Darwin has engaged BMT WBM to produce a Coast Erosion Management Plan for its areas of responsibility. The core objectives of the management plan are to identify sites and infrastructure at risk and provide coastal engineering recommendations for structures/actions to prevent or minimise erosion rates. Whilst the purpose of this consultancy is not to investigate the implications of Climate Change, the assessment of sites, risk and recommendations do incorporate

the readily foreseeable impacts of rising sea levels. For the purposes of this report, the model of sea level rise used is based on IPCC (2007) which reports 0.3m over 50 years and 0.8m over 100 years.

In line with the Brief, the study has been divided into three distinct stages.

Part A: Assessment (Chapter 2) – Review and update available information and identify factors contributing to erosion and assess the rate of recession/level of threat based on the work by the Southern Cross University and assessments of the landforms and coastal processes.

Part B: Risk Identification (Chapter 3) – Assess the level of threat/risk at individual locations as well as any major information gaps, identification of values and degraded areas, assessment and recommendation of management options and rank the sites according to vulnerability and infrastructure at risk, map likely erosion and inundation for future timeframes.

Part C: Concept Actions (Chapters 4 and 5) – Recommend actions to address vulnerable sites and indicate budgets and timeframes.

Title:
Locality Plan

Figure:
1-1

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BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

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Approx. Scale

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2 ASSESSMENT

The assessment of the physical processes forcing coastal change has consisted of three main components being:

- Review of historical data and reports to establish the primary physical processes causing change;
- Investigation of photogrammetry to assess critical areas and rates of change; and
- Site visits to assess the effectiveness of existing management options and local issues resulting in the amplification of rates of change.

2.1 Data Collation and Analysis

There have been a number of studies relating to the Darwin shoreline and adjacent coastal zone primarily starting after the dramatic event of Tropical Cyclone Tracy in December 1974. Of particular interest have been a series of studies summarised below which provide some physical description of the coast and oceanographic processes (Comley 1996-7, Goad 2001, Gray 2002 and SEA 2006-10) as well as gelological and geomorphic processes (Young et al 1998, Mulcahy 2001 and Nott et al 2003).

Also, a significant amount of recorded physical data (winds, waves, tides and aerial photography) have been captured and this data is analysed and interpreted with regard to its likely impact on shoreline recession.

The understanding of the interaction of the shoreline (geology/geomorphology) and oceanographic components (waves, tides) at the shoreline is the fundamntal basis of understanding drivers and resistance to shoreline recession which ultimately results in a threat to development of the City of Darwin.

2.1.1 Previous Studies

Coastal Erosion Issues at the East Point and Nightcliff Areas of Darwin

Jones G, Baban S, Pathirana S, Southern Cross University, Lismore NSW. (2008)

This recent study investigated both the geloogical land forms of these critical areas and used remotely sensed data (areial photography and satellite imagery) to assess rates of erosion. The study noted an average recession rate of 0.3m per year but with a high variability and some difficulties with resolution in the data. It also noted that erosion rates were site specific and could not be regionalised with often dramatic collapses resulting in 5-10m recession occurring in high risk areas with little warning.

Two processes were noted being ocean based (waves and tides) and land based (stormwater and sewerage). The report indicated that the interaction of concentrated stormwater flows combined with oceanic wave action caused the more extreme recession rates. The report concludes that a risk based management approach and long term monitoring be established to provide guidance for any engineering based mitigation actions.

Darwin Beach Erosion Surveys

B.W.T Comley, Department of Lands Planning and Environment (1996/7)

Beach profile surveys of Mindil/Vesteys Beach and Casuarina Beach were undertaken by Department of Lands Planning and Environment in 1996 and 1997. Colour photographs of the beaches were captured at the time of survey. Results of the 1997 survey are given in Appendix B.

J.Goad, University of Western Australia (2001) and P. Gray, University of the Northern Territory (2002)

The beach profile surveys undertaken by Comley (1997) were continued by DLPE on a twice-yearly basis until 2001. Goad (2001) analysed the associated beach volume changes, with comparison to behaviour from 1944 to 2001 indicated by historical aerial imagery. Gray (2002) undertook a more detailed analysis of the beach profile data set, considering an array of meteorological and oceanographic processes that might be expected to affect the beaches. Key conclusions of the analysis were to recognize that the beaches experienced both seasonal and inter-annual variability, with apparent rotation and subsequent reversal.

Morphology and Process on the Lateritic Coastline near Darwin, Northern Australia

R. W. Young E. A. Bryant University of Wollongong (1998)

This study investigated the coastal morphology near Darwin and found that it is controlled mainly by the gentle warping of a lateritic profile. In synclines the lateritic cuirasse forms extensive shore platforms, but on the anticlines the pallid zone of the weathering profile is eroded by waves, causing the undercut cuirasse to collapse. The dominant modern process on the shore platforms is solutional attack on the laterite, resulting in large depressions. Many of the platforms are covered by relict layers of cemented laterite cobbles transported by waves of high energy. C14 ages on carbonate cement between the cobbles show that one sheet was deposited at about 3700 BP and the other sheet at about 1700 BP. Waves generated during devastating tropical cyclones this century had little effect on the cobble sheets, and they were probably transported onshore by tsunamis originating in the Indonesian archipelago.

The Urban Geology of Darwin, Australia

Jonathan F. Nott Faculty of Science and Engineering, James Cook University, (2003)

This study notes that the latitudinal position of Darwin has strongly influenced the local geology which is dominated by deeply weathered lateritic regolith formed on labile Cretaceous marine sediments. Close to 2 billion years of geological history is lost from this immediate region because the largely horizontally bedded Cretaceous strata unconformably overly folded Proterozoic metasediments. Models of the landscape evolution of this region, and indeed across much of northern Australia, have been based upon the incorrect interpretation of a type section, displaying weathered Cretaceous strata, close to Darwin city. A recent reinterpretation of this section type has shown that the landscape here is a function of deep weathering and structural controls within the Cretaceous strata rather than repeated cycles of uplift and pediplanation during the Cainozoic. The low relief of this landscape, low rates of denudation and preservation of Cainozoic deep weathering profiles and predominantly rocky shore have meant that Quaternary sediments are restricted to shallow alluvial

deposits in creek valleys and marine sediments forming beaches with limited dune and beach ridge development.

Sedimentology and Nearshore Circulation Patterns within Fannie Bay, Darwin Harbour.

C. Mulcahy, Notre Dame University, Australia, Honours Thesis (2001)

Analysis of sediment data and short-term drogue records were used to interpret nearshore processes in Fannie Bay. Description of the sediments, as summarised by Gray (2002) includes:

The principal beach sediments between Emery Point and Lee Point are quartzose and calcic sands on the upper beach, with silts, clays and marine muds in the subtidal zone. The sands vary in size from coarse silts to very fine sand (up to 95%), and are moderately sorted on the beach face. The main sources of sediments include: alluvial sands, silts and clays; skeletal material from the reef flats, subtidal terraces and mangals; and lithoclastic material derived from erosion of the cliffs and rock platforms. The beach sediments are comprised of mainly siliceous (up to 40%) and calcic (30 to 40%) sands with some feldspars (up to 20%) and other material, mainly heavy minerals, mica and lateritic lithoclasts.

Darwin Storm Tide Mapping Study

SEA (2006 and 2010)

These studies describe the assessment of storm tide risks generated by tropical cyclones within the Darwin region, extending from Cape Grose in the west to Gunn Point in the east on the mainland and also the southern facing coast of Bathurst and Melville Island, west of Cape Gambier. The assessed storm tide risk includes the storm surge, the interaction with the normal astronomical tide and the possible effects of breaking wave setup. Localised wave runup effects and cyclic inter-annual variations of tide or mean sea level are not included. In the 2010 study the potential influence of climate change has been tested by considering future sea level rise and changes in tropical cyclone behaviour to the year 2100. Plots showing the significant components that comprise storm induced elevated water levels for Fannie Bay and Nightcliff are shown below in Figure 2-1 and Figure 2-2.

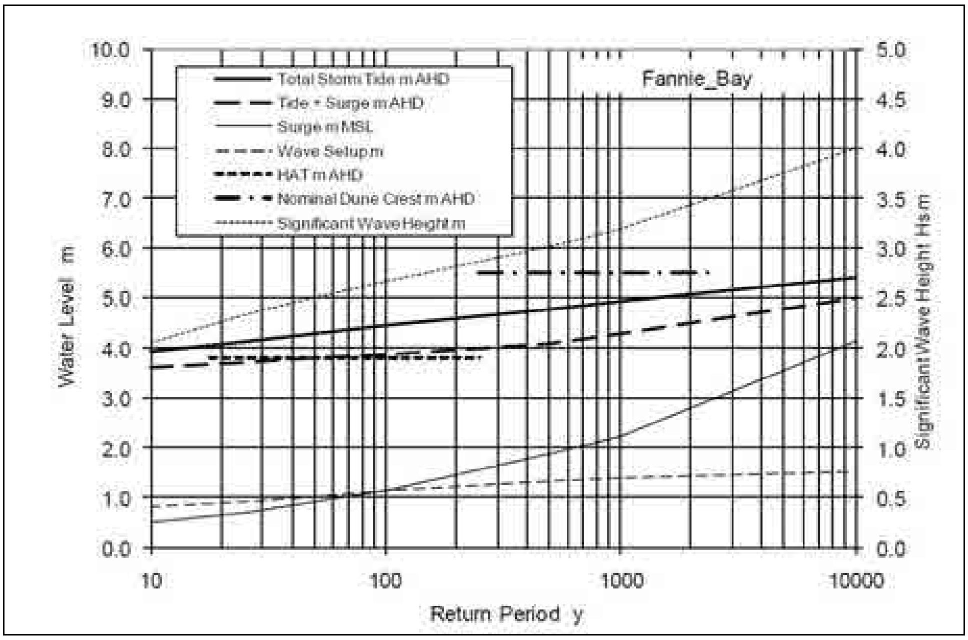


Figure 2-1 Predicted Storm Tide Levels for Fannie Bay

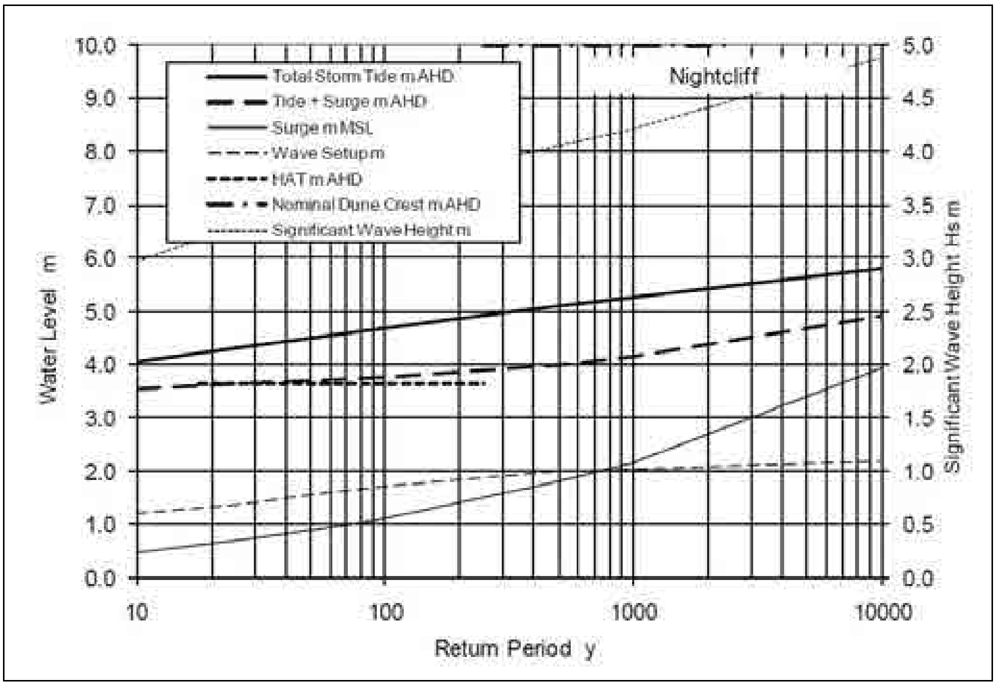


Figure 2-2 Predicted Storm Tide Levels for Nightcliff

2.1.2 Geomorphic Basis for Review of Physical Data

The Darwin foreshore can be classified, albeit simply, as three different coast types, being sandy beaches, soft rock cliffs and mangrove shore (described in Section 2.1.3.6). Each coast type resists coastal stresses through different mechanisms, which imply their relative susceptibility to future conditions, including severe storm events or projected sea level rise. Table 2-1 summarises the dynamics associated with each of the coast types and the corresponding measures of sensitivity.

Table 2-1 Darwin Coast Types and Corresponding Key Sensitivities

Coast Type	Sandy Beach	Soft Rock Cliffs	Mangrove Shore
Description	Ultra-dissipative macrotidal beach	Variably layered lateritic cliffs with intertidal rock platform	Fringing coastal mangroves, generally fronting low dune
Response to energetic wave conditions	Beach flattening Plan form rotation Dune erosion Dune overtopping / breaching	Dissipation on rock platform (breaking) Talus / block erosion	Dissipation through mangroves (friction) Loss of mangroves
Response to raised sea levels	Landward and upward profile shift	Increased erosion above platform	Mangrove retreat Dune growth
Key sensitivity	High waves coincident with high water levels	Sustained high water levels	High or long period wave conditions
Measure of performance	Dune buffer width and height	Cliff position	Width of mangroves

The sensitivity to water level and wave conditions, including wave direction, requires analysis of a range of environmental conditions. In the absence of a regular wave monitoring program, much of the coastal behaviour is inferred from description of the wind record. In addition to oceanic phenomena, site inspection has indicated that cliff erosion is strongly tied to rainfall runoff. Hence a brief summary of historically recorded rainfall is included in 2.1.3.1.

2.1.3 Physical Data

2.1.3.1 Meteorology and Wind Conditions

Darwin is located at approximately 12°25'S and 130°50'E, on the southern side of Beagle Gulf, towards the northwest limit of the Northern Territory mainland. This is central to the Wet-Dry Tropics of Australia, which experience seasonal conditions (Figure 2-3) with a distinct period of the year in which the majority of rainfall occurs (the "Wet" season).

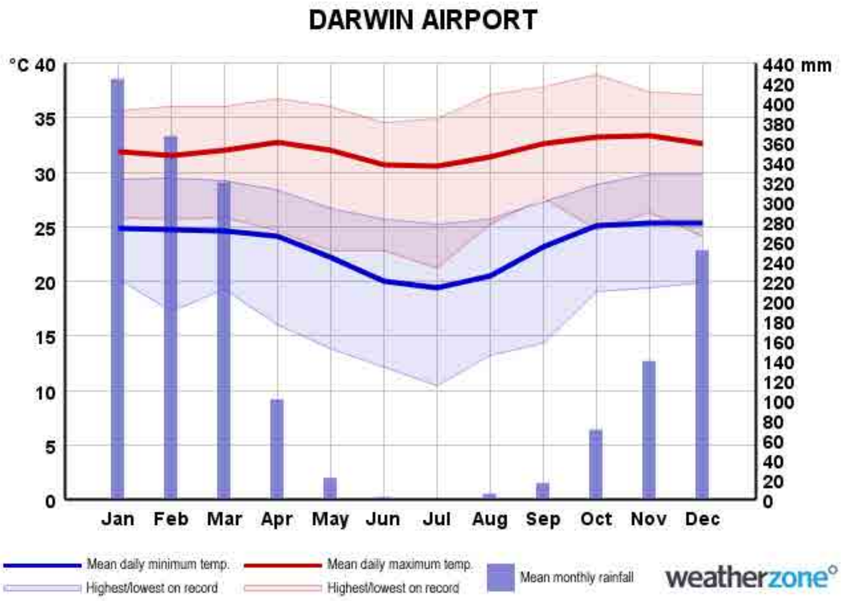


Figure 2-3 Median Monthly Wind and Temperature (Courtesy WeatherZone)

Synoptic conditions are dominated by the tropical low pressure belt, with occasional and diffuse influence from extra-tropical anticyclones during the southern hemisphere winter months, and intense winds during summer months due to the Australian monsoon (Gentilli 1971). Wind conditions are clearly bimodal, with east to southeast and west to northwest bands (Figure 2-4). Comparison of the wind distributions on a monthly basis (Figure 2-5 and Figure 2-6) indicates that the Wet-Dry seasonal pattern of rainfall corresponds to a similarly distinct shift in prevailing winds and active weather systems. An equivalent comparison for winds above 30 km/h (Figure 2-6) shows that the strongest winds mainly occur during January and February, within a narrow directional band.

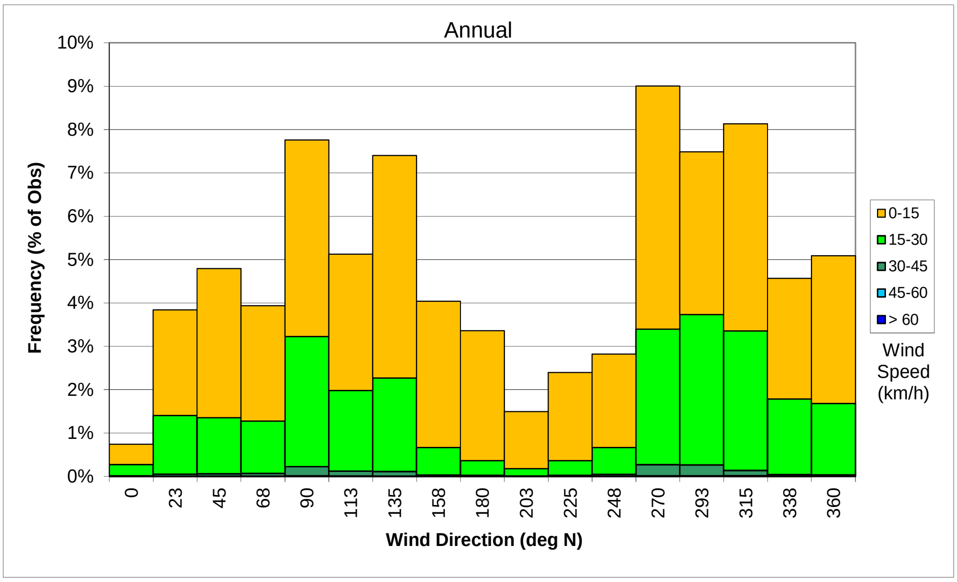


Figure 2-4 Darwin Airport Combined Annual Wind Speed-Direction-Frequency Plot

Table 2-2 Wind Direction Frequency

Dir (deg)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.0	1.0%	1.2%	1.6%	1.3%	0.9%	0.5%	0.8%	0.8%	0.9%	0.9%	0.5%	0.9%
22.5	4.7%	6.6%	7.0%	9.5%	7.1%	5.7%	5.0%	6.4%	4.6%	4.2%	3.3%	4.5%
45.0	4.5%	5.8%	7.1%	8.7%	12.4%	10.9%	11.8%	9.5%	4.4%	4.2%	2.4%	4.5%
67.5	2.5%	3.1%	4.9%	5.1%	9.3%	9.6%	8.6%	5.4%	2.0%	2.0%	1.5%	3.2%
90.0	2.6%	4.0%	6.4%	7.6%	12.1%	17.2%	14.3%	8.0%	3.7%	2.1%	1.3%	1.6%
112.5	1.6%	3.1%	4.0%	4.8%	7.6%	11.4%	7.7%	4.5%	1.5%	1.3%	0.8%	1.2%
135.0	0.8%	2.0%	3.4%	3.8%	5.6%	5.8%	4.9%	3.9%	2.0%	1.4%	0.7%	1.1%
157.5	1.0%	1.8%	3.9%	6.0%	7.4%	8.2%	7.5%	5.6%	2.8%	1.2%	0.8%	0.8%
180.0	1.5%	2.4%	4.7%	7.4%	9.7%	9.4%	10.2%	10.1%	4.2%	2.2%	1.9%	1.5%
202.5	1.6%	2.6%	4.1%	3.9%	4.1%	3.2%	4.4%	5.3%	3.6%	3.1%	1.9%	1.5%
225.0	3.5%	4.8%	6.8%	7.3%	4.4%	3.5%	4.7%	7.5%	9.4%	7.7%	5.1%	3.1%
247.5	12.9%	10.7%	11.0%	8.9%	4.5%	3.3%	5.1%	10.4%	19.7%	21.3%	21.9%	16.6%
270.0	28.9%	20.2%	12.9%	9.0%	4.9%	2.9%	4.5%	9.7%	22.7%	28.2%	34.6%	31.6%
292.5	14.7%	11.8%	7.4%	4.8%	2.3%	1.9%	2.5%	4.6%	8.8%	9.8%	13.3%	14.0%
315.0	8.5%	8.0%	5.2%	3.4%	1.8%	1.3%	1.7%	2.8%	3.5%	4.1%	4.7%	5.7%
337.5	7.1%	7.2%	5.4%	4.2%	1.4%	0.7%	1.5%	2.2%	3.8%	4.2%	3.6%	5.7%
360.0	2.0%	3.4%	2.8%	1.6%	1.0%	0.8%	1.2%	1.0%	0.9%	1.4%	1.2%	1.8%

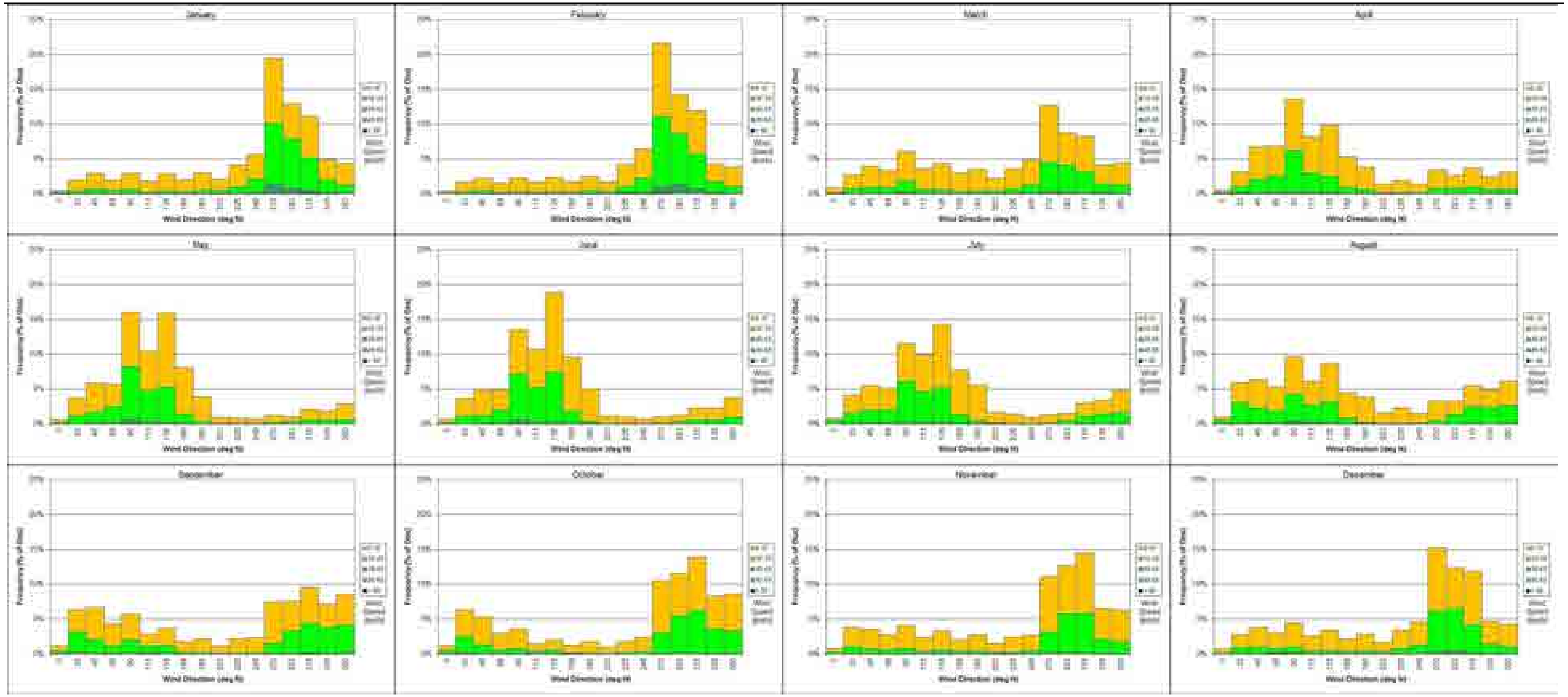


Figure 2-5 Darwin Airport Monthly Wind Speed-Direction-Frequency Plots

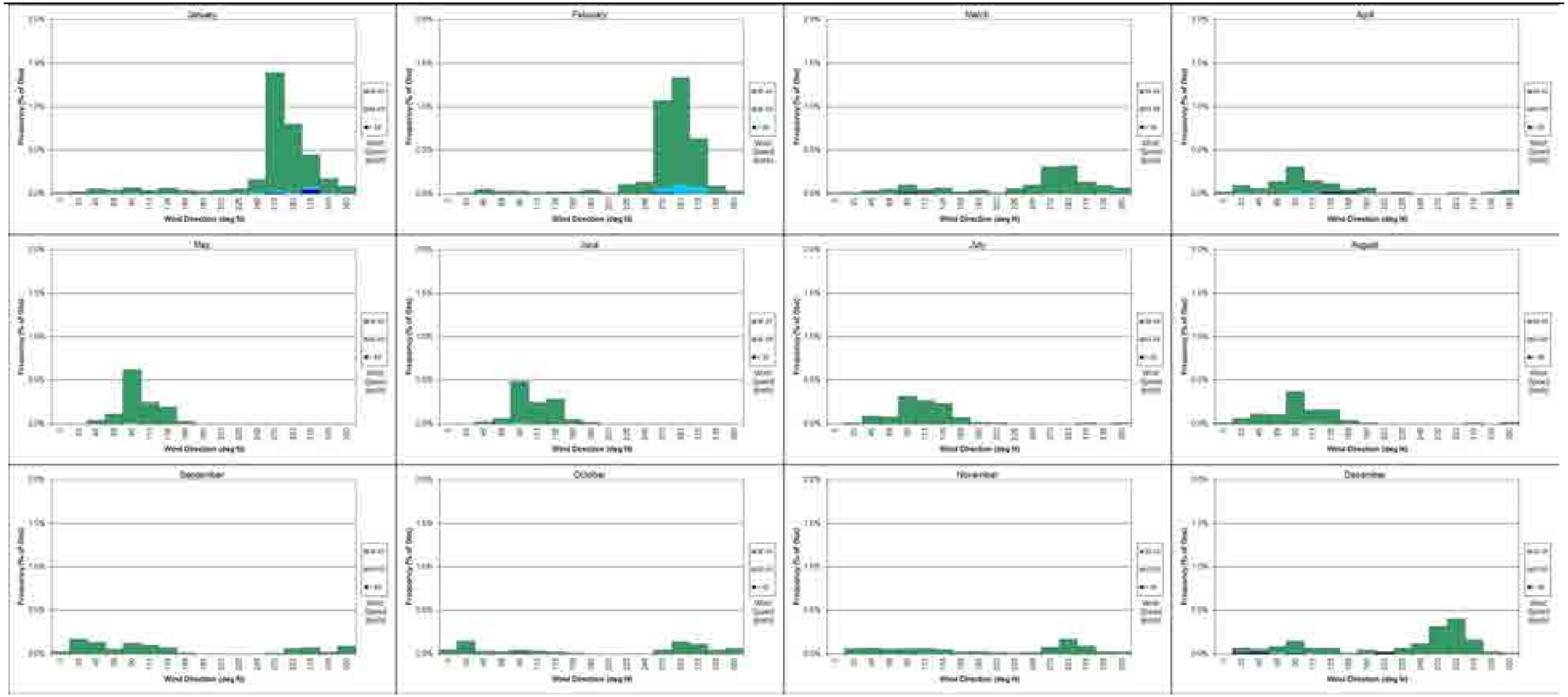


Figure 2-6 Darwin Airport Direction-Speed-Frequency Plots for Winds above 30 km/h

“Dry season” conditions occur from April to August, with prevailing southeasterly trade winds, modulated on a daily basis by the land-sea breeze. “Wet season” conditions occur from October to March, although monsoonal winds and heavy rain usually occur within a shorter period (Kullgren & Kim 2006) as indicated by Figure 2-6. During the Wet, strong northwest monsoon winds are interspersed with weaker break conditions and very occasional tropical cyclones. Inter-annual variability of the monsoon has been previously identified, with changes in onset, strength and persistence (Holland 1986; Evans & Allan 1992; Kim et al. 2006). The relative influence of inter-annual variations in prevailing winds has been evaluated through the use of cumulative wind drift plots (Figure 2-7 and Figure 2-8) These show the consistency of winds being generally from the NE in the morning and SE in the afternoon.

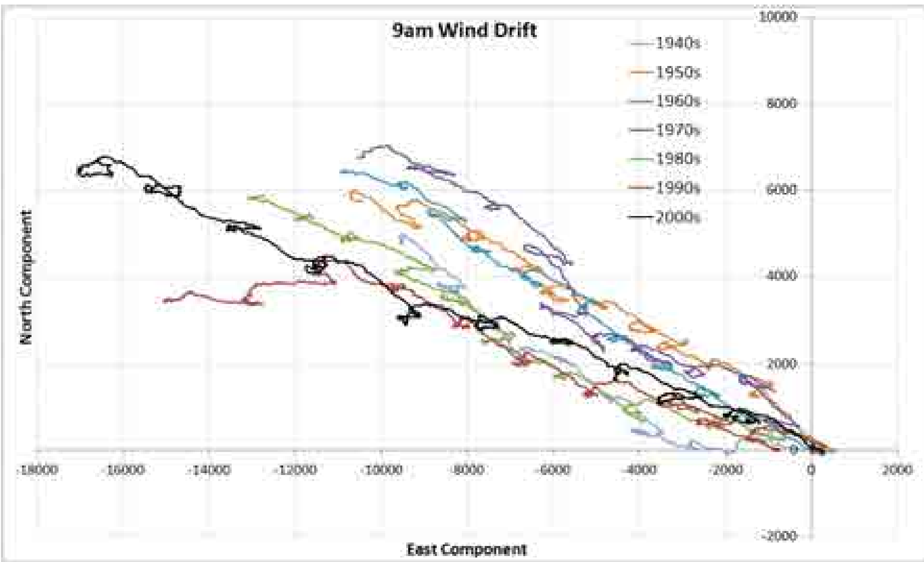


Figure 2-7 Cumulative Wind Drift for 9am Winds

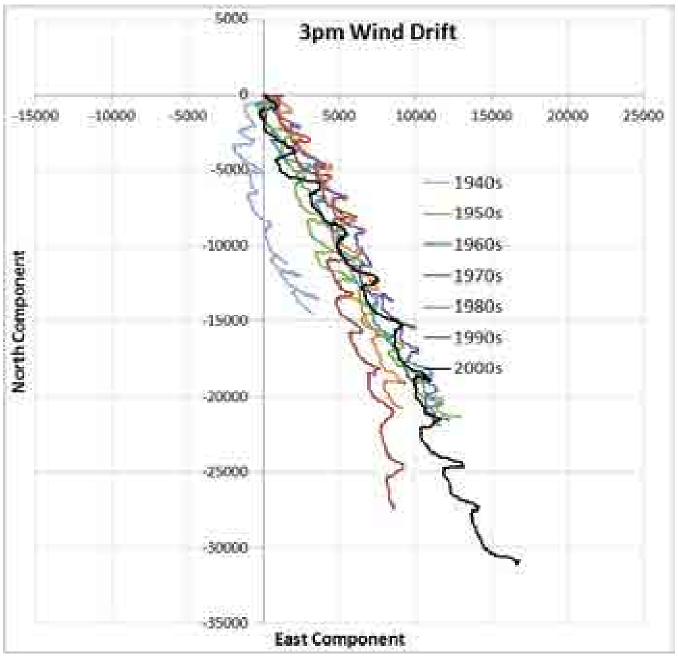


Figure 2-8 Cumulative Wind Drift for 3pm Winds

Monsoons and monsoonal storms are the most frequent cause of strong winds, often exceeding 30 km/h, and under extreme conditions, reaching up to 70 km/h. The frequency of these events, their comparative persistence and onshore (W-NW) direction determines that they play a significant role in coastal dynamics. In contrast, tropical cyclones are capable of more intense conditions but are also less frequent, transient and variable in direction (Figure 2-9).

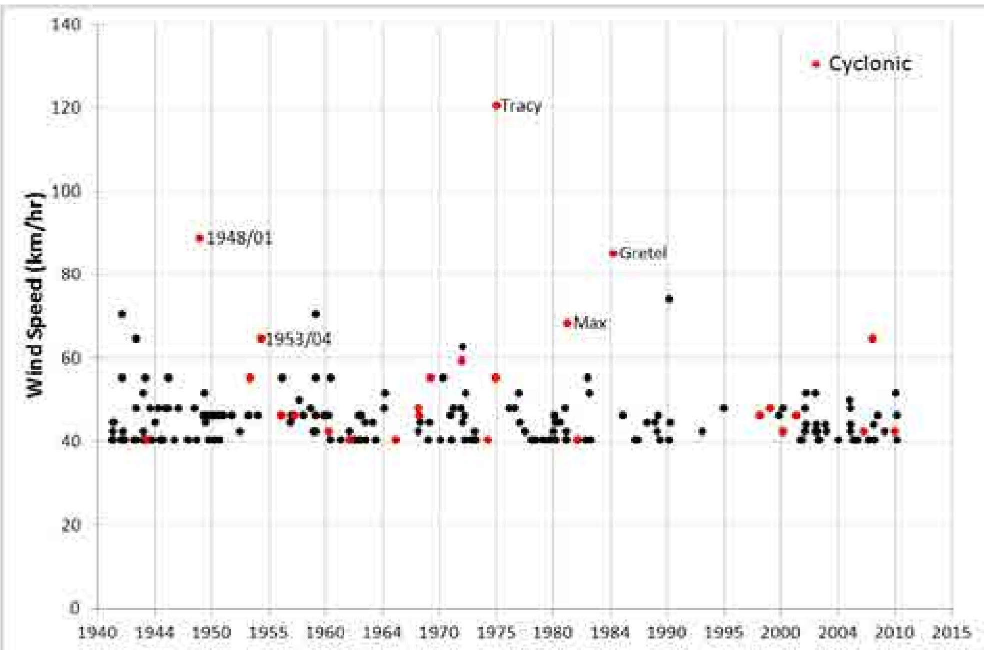


Figure 2-9 Record of Strong Wind Events

Tropical cyclones in the Darwin region are moderately frequent (with respect to global tropical storm frequency), with an average of three tropical cyclones crossing the Northern Territory per year. Approximately four cyclones per decade pass within 100km of Darwin, and would therefore be capable of producing extreme conditions. Wind speed above 120 km/h was observed during TC Tracy, which caused extensive damage to Darwin, despite a relatively small synoptic scale (Bureau of Meteorology 1977; Callaghan & Smith 1998; Guillaume et al. 2010).

2.1.3.2 Tropical Cyclones

Historic impacts of tropical cyclones within the Northern Territory have previously been summarised by Murphy (1984), with brief meteorological summaries for more recent events available from the Bureau of Meteorology[1]. Following Coleman (1972) and Lourensz (1981), the Bureau of Meteorology maintains a tropical cyclone database which may be interrogated using an online tool[2] to assist with statistical analysis and storm surge modelling.

Key characteristics of tropical cyclones located within the Darwin region include:

- Cyclones are mainly generated in the nearby ocean basins, including the southern part of the Arafura Sea, the eastern part of the Timor Sea and the northern part of the Gulf of Carpentaria.

[1] http://www.bom.gov.au/cyclone/about/northern.shtml#history
[2] http://www.bom.gov.au/cyclone/history/index.shtml

Occasionally cyclones have been generated in the Coral Sea and retained cyclonic structure across the Gulf;

- A number of cyclones have been first identified as adopting a tropical storm structure when over land (landphoons);
- The region's tropical cyclones are comparatively small in scale, which corresponding to the short travel times and the corresponding limit for storm system growth;
- As a result, the majority of systems in the region have low intensity, with 84% having central pressure above 990hPa when within 500km of Darwin; and
- More intense tropical cyclones (<990hPa) generally had an extended east to west path. TC Tracy is the only exception, developing in the Arafura Sea and moving southward.

Tropical cyclones that passed within 100km of Darwin have been identified through interrogation of the Bureau of Meteorology cyclone database as summarised below (Table 2-3). Murphy (1984) further identifies historic records of tropical cyclone impacts in 1882, 1897, 1915, 1917, 1919 and 1937.

Table 2-3 Tropical Cyclones passing within 100km of Darwin

Decade	Date	Name	Cyc ID	Min CP (hPa)	Near Point (km)	Near Bearing (°N)	Near CP (hPa)	Near Speed (km/hr)	Direction (°N)	Avg Speed (km/hr)
	3/03/44	Unnamed	1943/03	970	81	143	998	n/a	n/a	n/a
1940s	01/12/48	Unnamed	1948/01	995	51	359	1000	n/a	n/a	n/a
1950s	10/04/54	Unnamed	1953/04	976	77	80	n/a	13	173	12
	22/03/60	Unnamed	1959/06	965	63	187	1003	26	278	25
	8/03/64	Carmen	1963/07	996	63	327	996	14	234	17
	2/12/64	Flora	1964/02	992	86	97	997	8	246	13
	28/02/65	Marie	1964/09	994	80	72	998	10	232	13
	29/12/65	Gisele-Amanda	1965/03	995	57	137	997	15	233	12
	19/01/68	Bertha	1967/09	995	47	236	997	16	237	15
1960s	4/03/69	Audrey- Bonnie	1968/13	991	85	154	994	18	249	19
	5/12/71	Kitty	1971/02	1000	70	319	1000	18	201	21
	3/12/74	Selma	1974/04	980	81	302	980	15	79	15
1970s	24/12/74	Tracy	1974/05	950	7	330	950	10	120	8
	11/03/81	Max	1980/12	960	17	351	993	8	246	13
	3/12/81	Amelia	1981/03	996	54	23	1000	38	262	21
	21/12/81	Unnamed	1981/04	990	50	58	1005	30	273	33
	2/03/84	Ferdinand	1983/17	980	69	29	1004	n/a	n/a	n/a
1980s	13/04/85	Gretel	1984/17	984	38	264	984	19	236	13
	11/01/90	Sam	1989/04	966	63	327	1003	n/a	n/a	n/a
	19/02/95	Bobby	1994/02	925	73	49	1005	28	253	27
1990s	16/03/99	Vance	1998/10	910	23	322	1002	8	259	10
	7/03/03	Craig	2002/06	976	86	227	1004	n/a	n/a	n/a
	15/03/04	Fay	2003/08	910	40	358	997	19	273	32
	13/03/05	Ingrid	2004/07	924	29	356	964	11	273	13
	25/04/06	Monica	2005/12	916	53	199	998	17	314	22
2000s	5/01/08	Helen	2007/6	975	n/a	n/a	975	n/a	n/a	n/a

2.1.3.3 Water Levels

Darwin experiences a large tidal range, generally with two tidal cycles per day (semi-diurnal, macrotidal conditions). These tides are strongly influenced by astronomic forcing, producing several regular cycles, the best-recognised being the fortnightly spring-neap cycle (Figure 2-10). Other major tidal cycles at Darwin include the bi-annual cycle, which peaks in March and September, and the 4.4-year cycle of lunar perigee (sub-harmonic). Non-tidal processes including surges and mean sea level variation are generally much smaller than the tides, but they may have significant effect upon the frequency with which inundation thresholds are reached. Key non-tidal processes include seasonal mean sea level variation, inter-annual variation correlated to the El Nino-Southern Oscillation phenomenon, monsoonal surges and tropical cyclone storm surge.

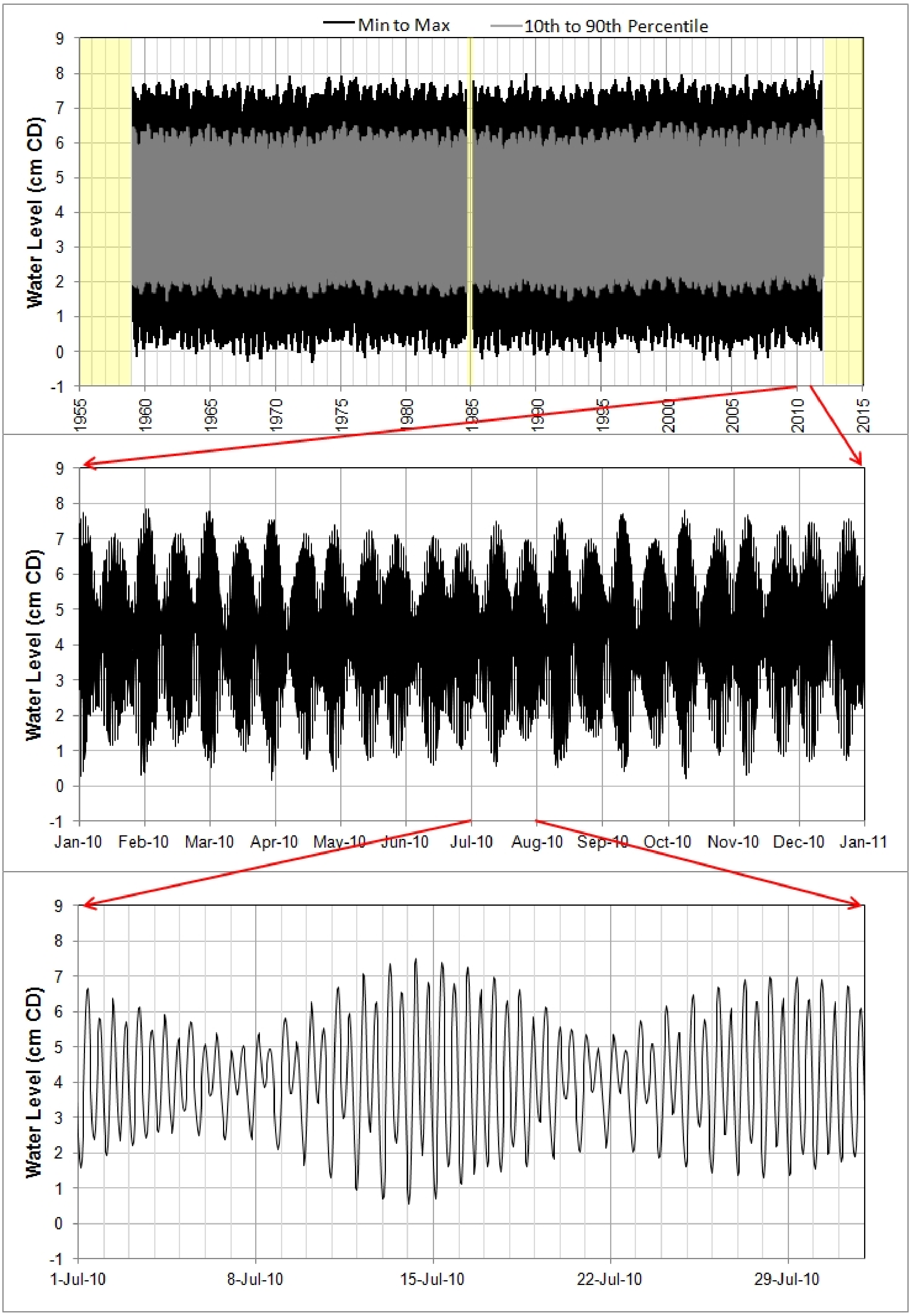


Figure 2-10 Illustration of Observed Water Levels at Three Scales

Darwin has a long history of water level measurement, with installation of a fixed automatically logging tide gauge in 1955 (Hamon 1963)[3]. From 1990, the gauge has been supplemented by a high quality "seaframe" gauge, which simultaneously measures weather, water level and water temperature. This gauge forms part of the array of instruments deployed for the Australian Baseline Sea Level Monitoring Program[4]. Harmonic analysis of the tide gauge record has been undertaken to provide a set of tidal constituents, which are in turn used to describe a set of tidal planes (Table 2-4). Care must be taken when interpreting these planes, as they do not take into account non-tidal phenomena such as long duration ocean surging and may produce a false indication of likelihood when considered over time scales of less than 19 years due to inter-annual and seasonal tidal cycles.

Table 2-4 Darwin Tidal Planes

Extracted from the Australian National Tide Tables (Department of Defence 2010)

Tidal Plane	Observational Definition	Harmonic Definition	Level
Highest Astronomic Tide (HAT)	The highest level of water which can be predicted to occur under any combination of astronomical conditions (over the 19-year tidal epoch)	$Zo + \Sigma j \|Aj\|$	+8.2m CD +4.0m AHD
Mean High Water Spring Tide (MHWS)	The average of all high water observations at the time of spring tide over a period of time (preferably 19 years)	Zo + M2 + S2	+7.1m CD +2.9m AHD
Mean High Water Neap Tide (MHWN)	The average of all high water observations at the time of neap tide over a period of time (preferably 19 years)	Zo + |M2-S2|	+5.1m CD +0.9m AHD
Mean Sea Level (MSL)	The average of all daily high and low water observations over a period of time (preferably 19 years)	Zo	+4.2m CD +0.0m AHD
Mean Low Water Neap Tide (MHWN)	The average of all low water observations at the time of neap tide over a period of time (preferably 19 years)	Zo - |M2-S2|	+3.3m CD -0.9m AHD
Mean Low Water Spring Tide (MHWS)	The average of all low water observations at the time of spring tide over a period of time (preferably 19 years)	Zo - M2 + S2	+1.4m CD -2.8m AHD
Lowest Astronomic Tide (LAT)	The lowest level of water which can be predicted to occur under any combination of astronomical conditions (over the 19-year tidal epoch)	$Zo - \Sigma j \|Aj\|$	+0.1m CD -4.1m AHD

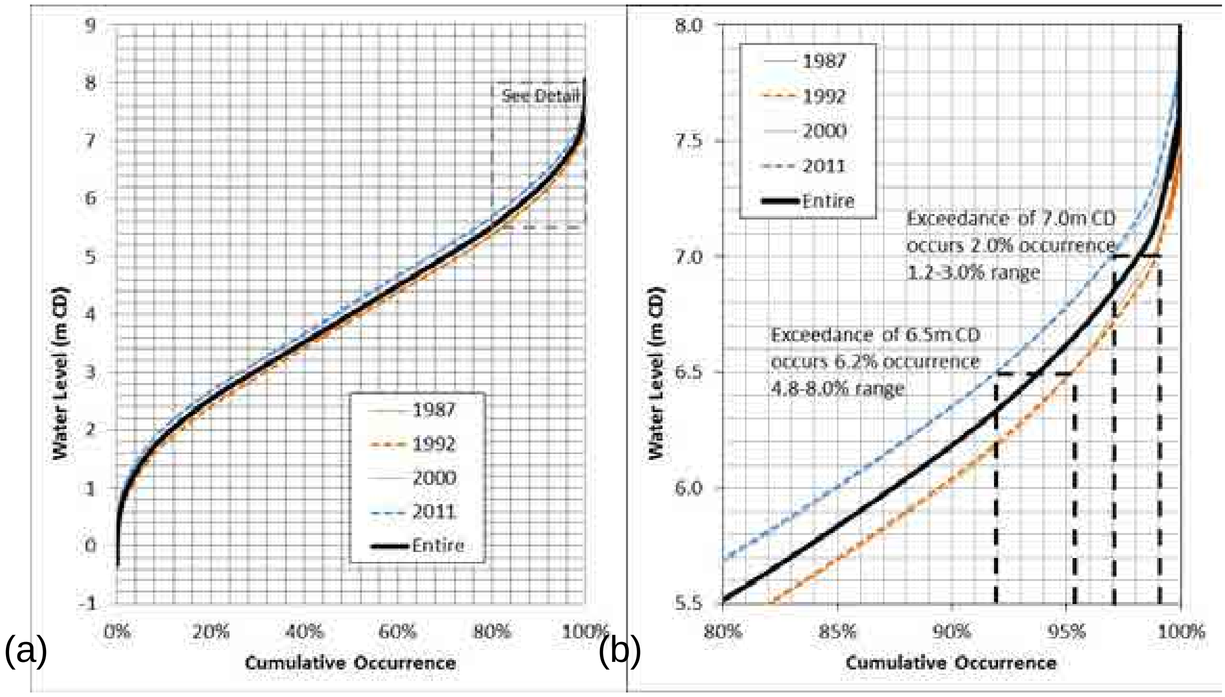
Notes: 1. CD refers to chart datum which is the low water datum used on historical navigational charts – usually close to Lowest Astronomical Tide
2. AHD refers to Australian Height Datum which is a National Datum close to Mean Sea Level.

Note that the biannual tidal cycle was enhanced during 2010, as this corresponds to a peak in the perigean cycle. The difference between equinoctial (March/September) and solsticial (June/December) spring tides varies from 0.3 to 1.0m with a 4.4 year cycle, which is the first sub-harmonic of the cycle of lunar perigee.

The frequency of water level exceeding natural thresholds is significant, particularly for the soft rock cliffs. The tide gauge data set (1959-2011) has been used to generate a water level occurrence plot, illustrating the macrotidal character (Figure 2-11a). Year-by-year plots vary marginally from this distribution, in response to both inter-annual tidal cycles and mean sea level variation. The largest

[3] http://www.nt.gov.au/nreta/naturalresources/water/surfacewater/telemeteredsites/description.jsp?Site_Id=g8150029
[4] http://www.bom.gov.au/oceanography/projects/abslmp/abslmp.shtml

perturbations from the long-term occurrence plot were associated with strong El Nino or La Nina conditions, modulated by the tidal cycle phase. This produced enhanced inundation during 2000 and 2011, and reduced inundation during 1987 and 1992. Whilst this variation is small compared with the tide range, it has a more significant effect upon the occurrence of high water levels, representing a ±25% and ±50% change in occurrence for 6.5m CD and 7.0m CD thresholds (Figure 2-11b).



Note: CD refers to chart datum which is the datum used on historical navigational charts – usually close to Lowest Astronomical Tide

Figure 2-11 Water Level Occurrence from Darwin Tide Gauge

There is also considerable seasonal variation of inundation occurrence for high water levels (Figure 2-12), developed through a combination of tide, surge and mean sea level variation). The seasonal variation is relatively mild near spring tide high water levels (6.5m CD) but becomes increasingly significant with higher tide levels.

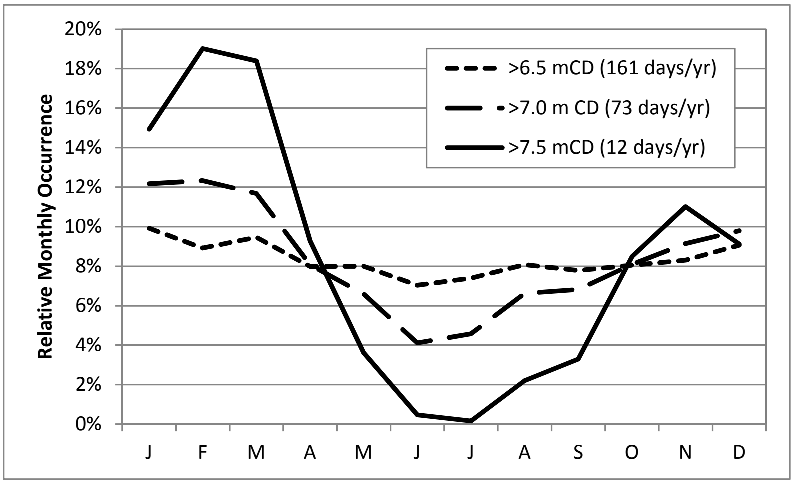


Figure 2-12 Relative Monthly Occurrence of High Water Levels

Table 2-5 provides a summary of the phasing of inundation components. This suggests that high water levels require coincident phasing of multiple components, during the overlapping period between high storminess (January to March) and high tidal conditions (February to April).

Table 2-5 Seasonal Phasing of Inundation Components

	J	F	M	A	M	J	J	A	S	O	N	D
Tide												
MSL												
Cyclones												
Storms												
Combined												

Shading indicates periods when conditions are prone to cause elevated water levels (lighter shading indicates lower influence).

A portion of the seasonal pattern of inundation occurrence is explained by mean sea level variation, which generally peaks around February-March and is lowest around July-August. However, this variation is small (0.2-0.4m) and on its own is insufficient explanation. Mean sea level variation is strongly affected by the El Nino-La Nina phenomena, which causes a net vertical movement of 0.2-0.3m, as illustrated by the difference between El Nino conditions in 1987 and La Nina conditions in 2011 (Figure 2-13). There is also modulation of the signal according to monsoon intensity: although 2005 had conditions conducive to high mean levels, the weak monsoon produced only a damp peak during the Wet season.

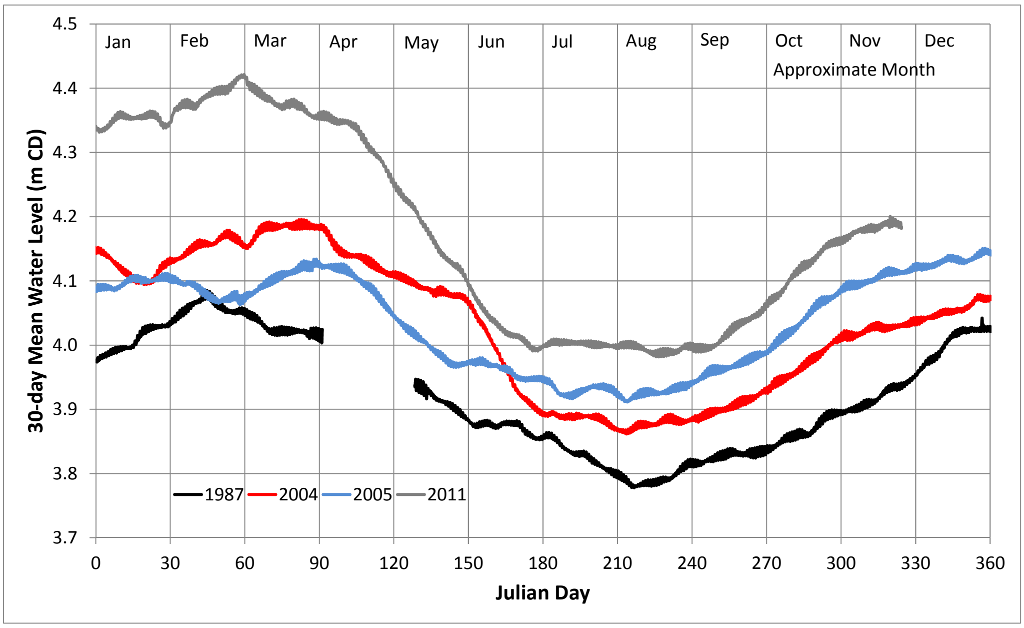


Figure 2-13 Seasonal Sea Level Cycle Variations

2.1.3.4 Storm Surge Modelling

Initial settlement in Darwin occupied higher terrain, however subsequent expansion of the city put pressure on the available development space. Following the impact of TC Tracy the growing capacity of storm surge modelling was recognised as an integral tool for coastal planning in Darwin.

A sequence of storm surge models have been applied to Darwin over that last 3 decades with progressively improved technology and information. The significant studies include:

- Hopley & Harvey (1976);
- Maritime Works Branch & Department of Housing & Construction (1981);
- Lawson & Treloar & Blain-Bremner Williams (1984);
- VIPAC, Bureau of Meteorology Special Services Unit, GEMS & Acer-Vaughan (1994);
- Systems Engineering Australia (SEA) Pty Ltd (2006); and
- Systems Engineering Australia (SEA) Pty Ltd (2010).

Excluding the first preliminary estimates (Hopley & Harvey 1976), the modelling results have produced a general downward trend in the estimates of extreme water levels (Table 2-6). This is understood to be a result of improved modelling techniques and greater information about tropical cyclones in the region. The general decline is offset in the most recent study (SEA 2010) through inclusion of a 0.8m allowance for sea level rise by 2100. Approximate levels for each of the studies are summarised in Table 2-6, recognising that there is a degree of spatial variability along the Darwin coast.

Table 2-6 Estimates of Extreme Storm Water Levels

Where spatial variability is indicated, Fannie Bay has been used as a reference site.

Study	10 yr ARI	100 yr ARI	1000 yr ARI	Note
Hopley & Harvey (1976)		8.5m CD	9.5m CD	Preliminary estimate
Lawson,Treloar & BBW (1984)		9.6m CD	10.8m CD	
VIPAC et al (1994)	8.2m CD	9.4m CD	10.6m CD	
SEA (2006)		8.6m CD	9.8m CD	
SEA (2010) “2010”		8.8m CD	9.3m CD	
SEA (2010) “2100”		9.5m CD	9.8m CD	Incl. SLR allowance

2.1.3.5 Wave Climate

Darwin is well sheltered from the generation of waves by winds from the north or east by its position in Beagle Gulf. Consequently there is a strong seasonal bias in the Darwin wave climate, with low wave energy during the Dry season, which has prevailing southeast winds; and moderate to high wave energy during the Wet season, when strong west to northwest winds may occur associated with monsoonal troughs, storms and occasional tropical cyclones. The potential for tropical cyclones to generate the most significant conditions has prompted a hindcast modelling approach towards the definition of extreme wave conditions, consequently with limited wave measurement.

Historical wave data for Darwin and the wider Northern Territory is limited, with Hamilton (1997) identifying a single Waverider buoy deployment near Larrakeyah from 1979 to 1982, otherwise data is limited to lighthouse wave estimates. The Larrakeyah dataset was interpreted by Byrne (1988) in his description of Darwin coastal processes, and formed a basis for the validation of hindcast

modelling by GHD-Macknight (1997). The latter study was used to define design wave criteria for Cullen Bay Marina, with a 100-year ARI cyclonic significant wave height estimated to be up to 4.5m, larger than the 3-3.5m height interpolated by Byrne (1988) from the wave measurements.

There have been a number of wave modelling exercises completed as part of Environmental Impact Assessments and environmental auditing within the Darwin region. These include the Ichthys Project (URS 2002; HR Wallingford 2010; APASA 2010), East Arm Wharf Expansion (Acer-Vaughan & Consulting Environmental Engineers 1993; URS 2011) and NT Power-Water (Valentine et al. 2011). In the majority of cases, the wave climate has been hindcast based upon validation from a short period of wave measurement of 2-4 weeks during the calm Dry season.

HR Wallingford (2010) use the results of a global wave hindcast database to provide an estimate of offshore wave conditions, identifying the significant difference in wave energy and direction between the Wet and Dry seasons (Table 2-7).

Table 2-7 Offshore Hindcast Wave Conditions

Season	Median Conditions	<10% Occurrence	Maximum Hindcast
Dry Season	<0.5m Hs	1.2m Hs	3.8m Hs
Wet Season	<0.5m Hs	1.8m Hs	5.2m Hs

These statistics from HR Wallingford (2010) are reported for an offshore point located towards the western end of Beagle Gulf, which is the viable limit of a global hindcast model. They are not considered representative of conditions at Darwin as significant energy loss and transformation will occur in the shallow offshore conditions. The annual offshore wind rose for this offshore location shows a dominant westerly direction to the waves (Figure 2-14). This direction also has the longest fetch, between Cox Penninsula and Bathurst Island and therefore generates the largest waves at the Darwin coastline. The propagation of a 5m westerly wave inshore is given in Figure 2-15 and shows on a regional scale that an equally distributed wave height is predicted for the exposed coast. Detailed bathymetry is required to better define wave heights and directions close to the shore where structures may be planned e.g. seawalls and offshore breakwaters. Sufficienly detailed bathymetry was not available for this study and hence design wave conditions for proposed structures could not be modelled.

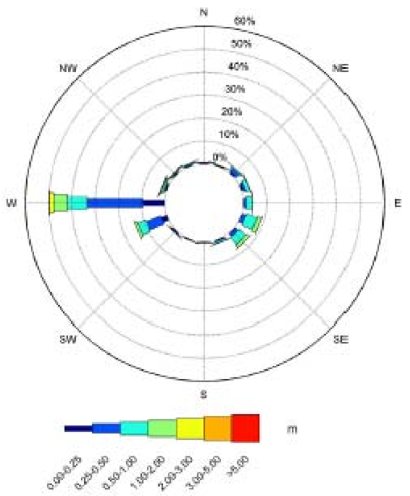


Figure 2-14 Wave Rose Offshore from Darwin (HRW 2010)

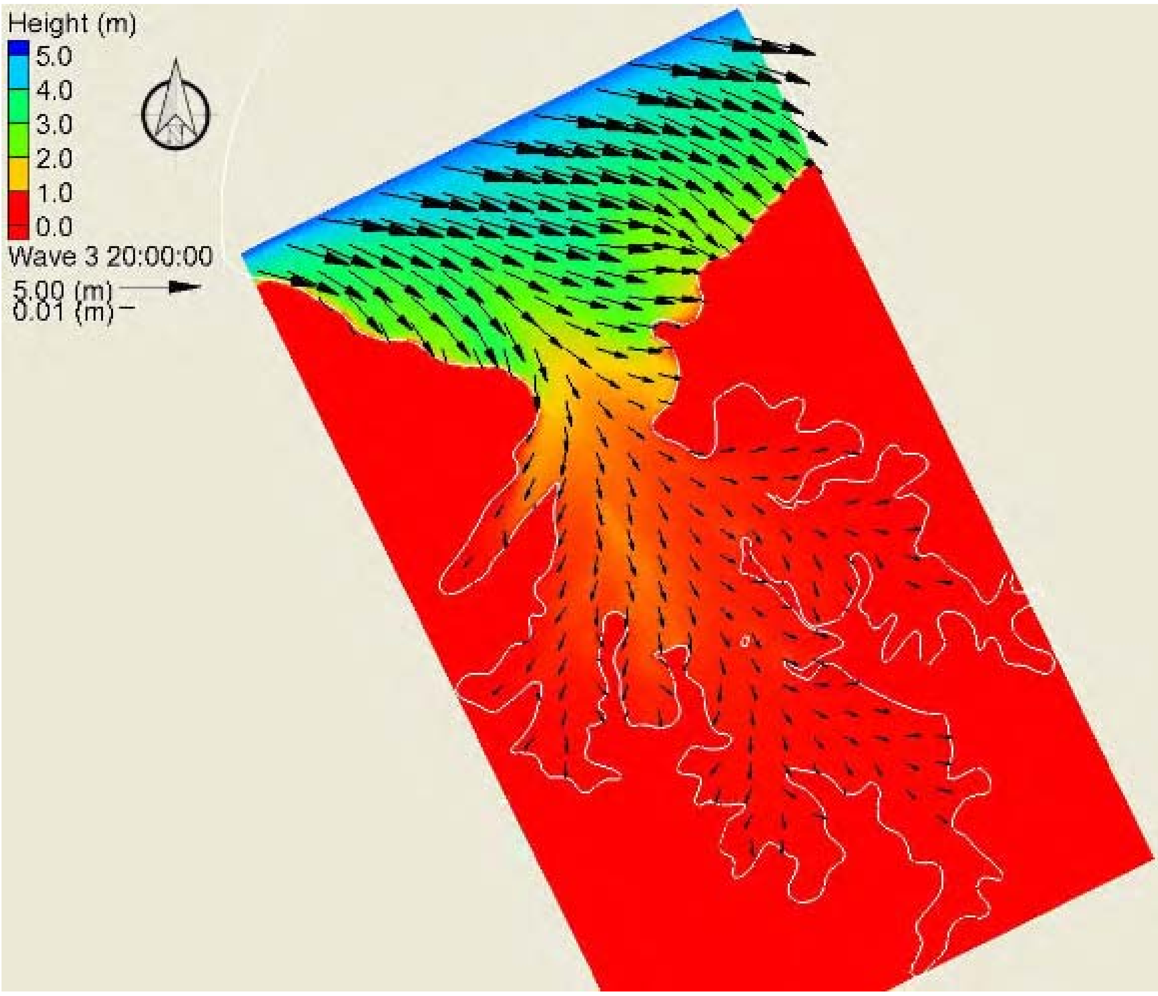


Figure 2-15 Propagation of 5m Westerly Wave into Darwin (URS 2011)

2.1.3.6 Geology and Morphology

Darwin is located on Shoal Bay Peninsula, which is one of five rocky peninsulas inter-leaved with large tidal estuaries located on the southern side of Beagle Gulf. Present-day surface morphology is dominated by Cretaceous siliceous siltstone features, with comparatively sparse Quaternary sediments. These overlie unconformable and weathered Proterozoic deposits, in turn above a Precambrian basement (Nott 2003). The siltstone contains several different layers of material, which are generally consistent in layering sequence (Figure 2-16), but variable in thickness and present day levels, which has previously been interpreted as a result of warping and subsequent deposition (Hays 1967). A more recent interpretation is that the strata variability results from a combination of weathering, including topographic influence and the unconformable structure of the Proterozoic deposits (Nott 2003).

Figure 2-16 Typical Strata showing Saprolite above Cretaceous Shore Platform (from Nott 2003)

The geology provides a framework upon which more mobile sedimentary structures are located. At all scales, these structures are strongly influenced by the macrotidal environment experienced at Darwin. The influence of high rainfall and runoff is less apparent, with most drainage systems being highly modified, through channel realignment, interception or stormwater drainage structures.

The relative tidal influence upon morphology is important, as it indicates areas that are capable of responding to surges, seasonal and inter-annual tidal modulations and sea level variability. The following tidal features are apparent:

- At the largest relevant scale, Darwin Harbour is a large tidal estuary to the west of Darwin. Tidal exchange causes significant flows in and out of the harbour which are topographically controlled, but also these flows are affected by density structure (Williams et al. 2006);
- Tidal flows from Darwin Harbour contribute to the formation and maintenance of Fannie Bay sand bar, which extends approximately 4km north of Cullen Bay Marina (Byrne 1988);
- Beaches considered within this Plan are all macrotidal in structure, with gently sloping planar surfaces, cut by tidal channels (Short 2006);
- Ludmilla and Rapid creeks both have a characteristic 'tidal creek' structure (Perillo 2009); and
- The presence and distribution of mangroves is strongly dependent upon tidal flows, with some species also reliant upon seasonal freshwater influx.

There is less evidence of tidal influence upon terrestrial landforms, which is largely due to the extensive modification that has been undertaken for land-use purposes. This includes several environmental rehabilitation works, including construction of Lake Alexander and Vesteys Lake. Consequently these onshore lowlands do not exhibit the 'natural' structure typical of macrotidal coastal wetlands.

The coastal morphology between Cullen Bay and Rapid Creek may be distinguished, albeit simply, into three general types:

- Soft rock cliffs comprised of porcelanite (siliceous siltstone) with underlying sandstone, variably overlain with 'wormeaten' soft rock (saprolite) or detrital sediments. This coast type is present along Nightcliff, East Point, Fannie Bay and the Gardens, near Darwin High School and the Museum;
- Sandy beaches, with a gently sloping planar structure characteristic of the macrotidal conditions. Tidal drainage channels are typically present, often linked to surface runoff pathways. Sub-tidal and intertidal rock platforms and gravel deposits occur occasionally; and
- Mangrove coast, which has very flat topography of sands and detrital muds. This coast type occurs perched on rock platforms at East Point and Nightcliff, and is contiguous with the mangrove complexes across the muddy coastal and creek foreshores at Ludmilla.

For each of the three coast types, the presence of a lithified coastal platform, generally sandstone, affects the local morphology. Although not always present platforms are commonly present as a sub-tidal or inter-tidal extensions of rock cliffs, , and are also present on sandy beaches (Vesteys Beach and Fannie Bay) as well as mangrove coast (East Point and Nightcliff).

The role of rock to determine the local morphology has previously been highlighted (Young & Bryant 1998), particularly expressed as a result of dips (synclines) and rises (anticlines) of the most erosion resistant upper layer of the siltstone deposit (the cuirasse). Where the cuirasse is low, wave and tide action form intertidal shore platforms; and where it is high, the softer material beneath the cuirasse is gradually undercut, forming cliffs. The relative highpoints at Emery Point, East Point and Nightcliff provide coastal promontories, which provide structural control for the softer shores along Mindil Beach, Fannie Bay and Ludmilla. These shores are approximately aligned with the dominant wave direction.

2.1.3.7 Bioturbation

There has been anecdotal evidence (Peter Whelan, Entomologist, *pers comm*) that rock boring molluscs are present at East Point. They were first noticed these at East Point about 50 years ago in the same position as today and occur at the bottom of soft porcelanite cliffs. The boring action transforms the rock to a honey comb structure that may be more easily abraded by sand and other agents. Borings of rock are sometimes visible on the sand below fresh holes, indicating a relatively rapid boring action.

No literature has been found suggesting a rate of erosion of cliffs due to this action. In this study it has been assumed that the action of waves and tides is the more dominant process.

2.1.3.8 Photogrammetric Analysis

The Northern Territory Department of Land and Planning holds a library of vertical aerial photography of the Darwin region. Samples of this photography for East Point are shown in Figure 2-17 and Figure 2-18. As a routine service the Department uses automated photogrammetry to obtain elevation information on recent photography and the service can be requested for historical photography. For this study the 2011, 1999/2000, 1974 and 1954 photography was investigated. The rational for choosing these dates is based on energetic meteorological periods as shown in Appendix D.

Figure 2-17 1945 Photography of East Point Military Area

Figure 2-18 2011 Photography of East Point Military Area

It is understood that the photogrammetry is carried out automatically on a regular grid which is geo-similar for each date i.e. the point inspected elevations will be at the same geographical point for each photo. However, the photogrammetry we received for all dates had the points for about 15m each side of the cliff line deleted and replaced by break lines as shown in Figure 2-19. This may have been reasonable except that the break lines were not carefully located to pick up the cliff line. Also upon close inspection of a critical area at East Point adjacent to the historical military site, where cliff erosion is threatening these artefacts, the geo-referencing of the historical date was inaccurate as shown in Figure 2-20.

Figure 2-19 2011 Photography of East Point Military Area with Grid and Breaklines

Figure 2-20 Figure Showing Geo-referencing Difference between 1974 and 2011

It is understood that if a detailed study using photogrammetry was required then more accurate analyses can be applied to achieve reliable results. If this were the case then cross sections, as shown in Figure 2-21, at the cliff line could be used to asses the magnitude of recession between historical dates and then by interpretation the long term rate of recession.

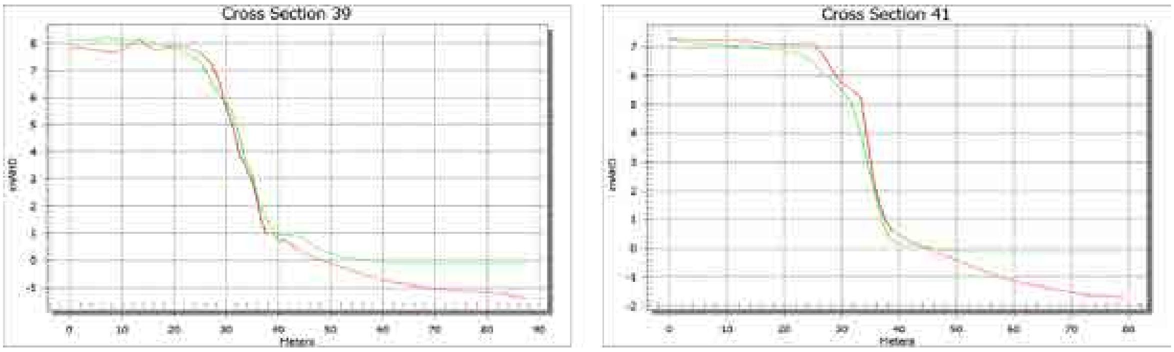


Figure 2-21 Typical Cross Section at East Point Showing 2000 and 2011 Surfaces

As a further example of the problems with geo-referencing, the cliff lines for 1954 and 2001 were digitised then overlaid on other photographic dates as shown in Figure 2-22, Figure 2-23, Figure 2-24 and Figure 2-25.

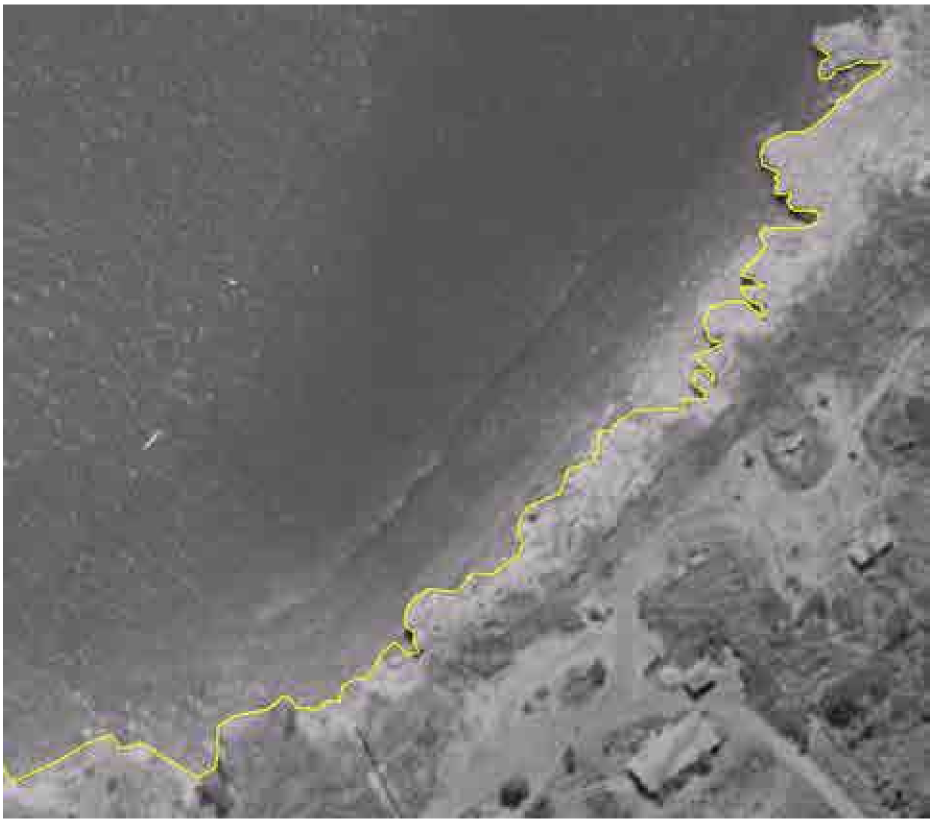


Figure 2-22 1945 Photography with 1945 Cliff Lane

Figure 2-23 2011 Photography with 2011 Cliff Lane

Figure 2-24 2011 Photography with 1945 Cliff Line

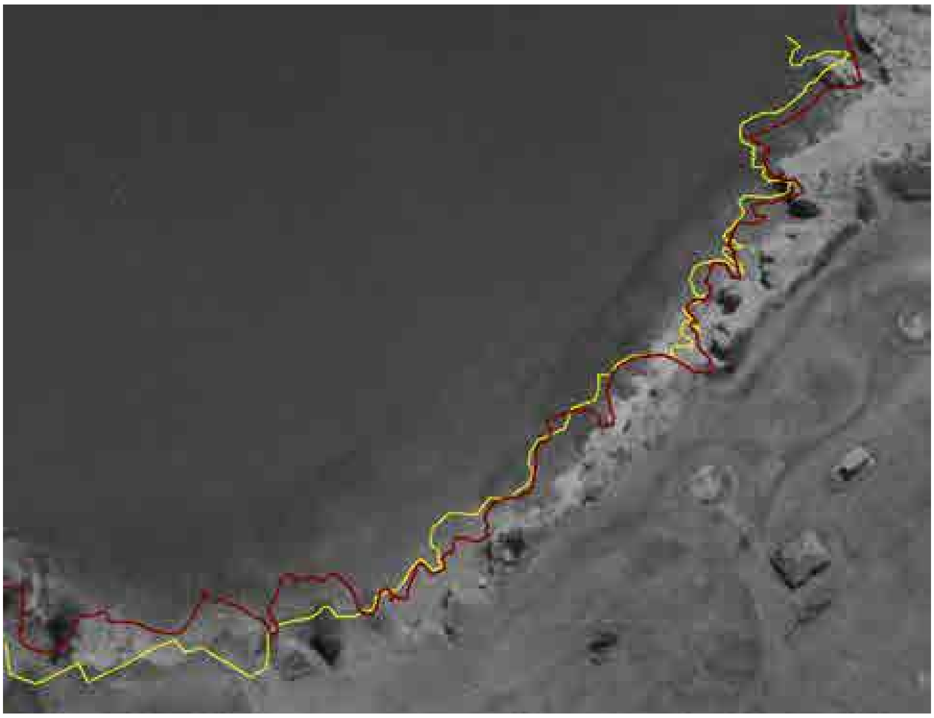


Figure 2-25 1974 Photography with 1945 and 2011 Cliff Lines

It is understood that the geo-referencing is difficult in areas where fixed features (buildings and roads) are not in place historically to allow the photography to be adjusted to suit. This is evidenced in Figure 2-25 where the cliff lines from various dates are clearly divergent. Therefore other more developed areas such as Mindil and Vesteys Beaches and Nightcliff will produce better results because of the larger amount of historical infrastructure which can assist geo-referencing. Also if detailed project specific photogrammety is designed and implemented to investigate local areas i.e. improved geo-referencing and cliff definition, then a reasonable result is expected.

For example the photography of Mindil Beach Figure 2-26 and Vesteys Beach Figure 2-27 shows good correlation between 1945 and 2011 and also indicates little or no change between these dates. In fact the MIdil Beach vegetation line has moved seaward. A check of beach elevations with the survey of Bromley in 1997 also show no significant change.

A total of 70 cross-sections from Mindil Beach to Rapid Creek, using the existing photogrammetry, is given in Appendix A.

Figure 2-26 Mindil Beach 2011 Showing 1945 Shoreline and Stormwater Drainage

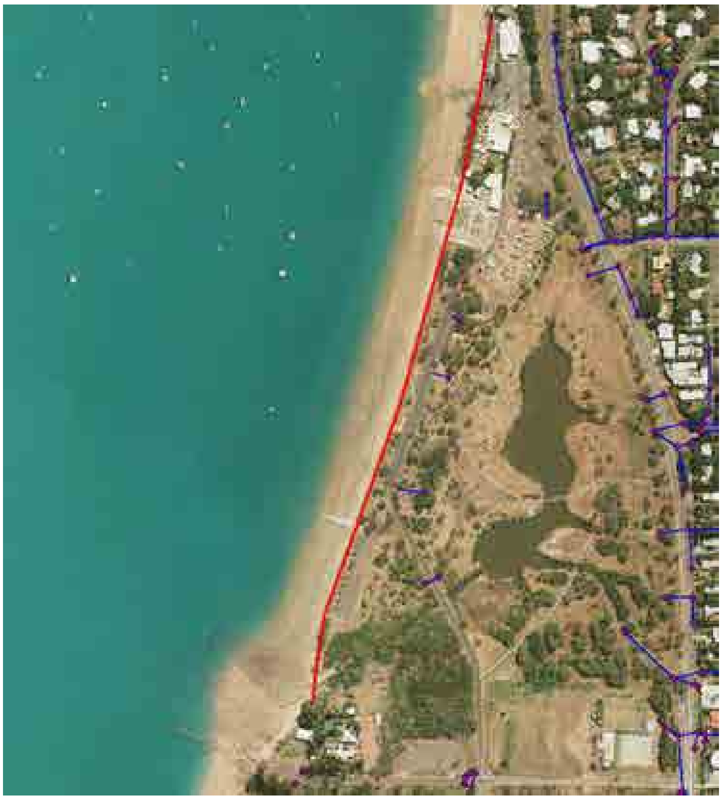


Figure 2-27 Vesteys Beach 2011 Showing 1945 Shoreline and Stormwater Drainage

2.1.3.9 Drainage

As noted during the site visit in early 2012, the drainage of stormwater over the cliffs is one of the primary causes of accelerated cliff erosion. The scientific background to the solubility of the cliffs is given in the documents referenced in Section 2.1.1.

It was noted that many of the major stormwater outfalls are located in deeply recessed fissures.

2.1.4 Review of Existing Management Options

Two site visits have been carried out. Initially Malcolm Andrews and Matt Eliot visited during the week of 16th January 2012 and met with David Cash and Shelly Franklin from City of Darwin and the Northern Territory Department of Lands and Planning officers regarding proposed photogrammetry.

Subsequently, Darwin experienced a severe monsoonal storm on the 25th January 2012, during spring tides, when some damage and overtopping was experienced along the coastline. Malcolm Andrews again inspected the entire coastline during the week of 8th February 2012 to document the physical impact of the storm and assess the success of existing shoreline management works.

During visits to the coastal areas, a significant volume of larger sized rock (including porcelanite) was observed in what appeared to be Council land (car parks, parks, beaches etc.) and some areas where sand could be extracted by land without affecting the littoral processes. It is likely that some proportion of this material could be used for remedial and protection works with a very low cost.

This weather event provided an indication of rock sizes that were moved by the storm. The stable rock seemed to be in the order of 1 tonne in size (approximately 900mm in diameter).

Generally the "plastic" dune crossings did not accept the scarp resulting from the storm very well with a loss of access amenity. The crossings need to be of flexible design with board and chain being an inexpensive and highly recommended option.

A summary of previous reports and preliminary observations taken during the two site visits is given below.

2.1.4.1 Mindil Beach

Mindil Beach is one of Darwin's most heavily used foreshore precincts, with Sky City Casino and the seasonal beach markets both having intense use close to the shore. The beach has had an active history of management, easily identified between the southern (Casino) and northern (Market) areas. Figure 2-28 shows aerial imagery from 1945 and 2007, which clearly illustrates the changes in foreshore infrastructure and infilling of coastal lagoon areas.

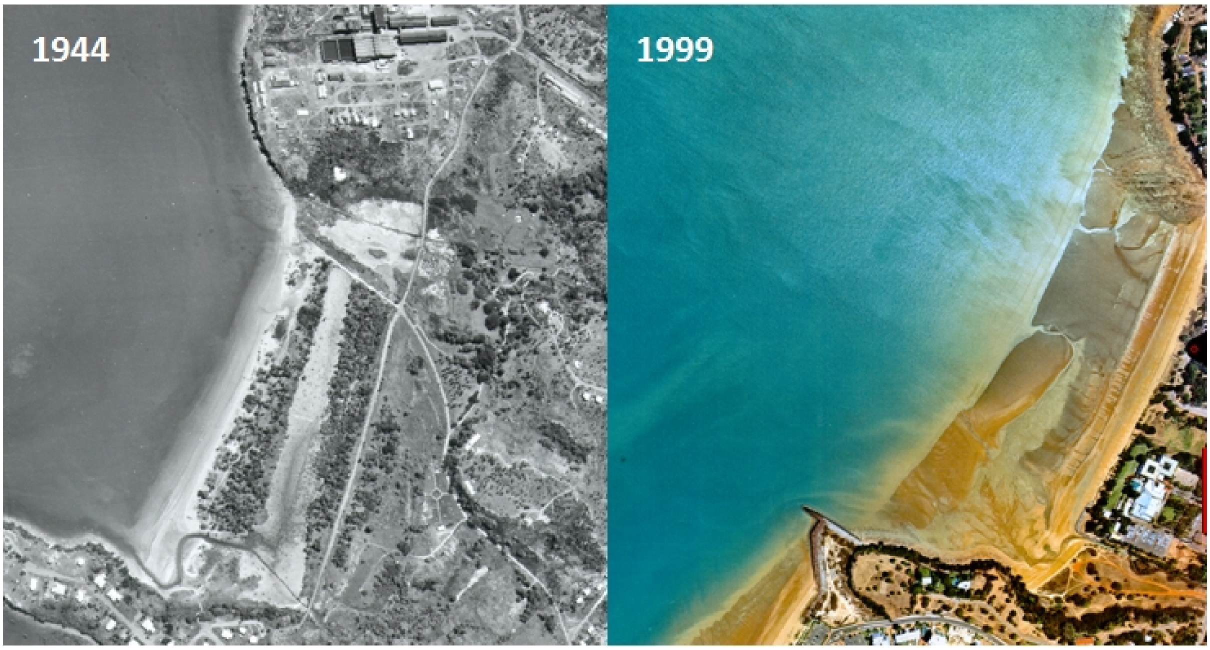


Figure 2-28 Mindil Beach Historical Aerial Imagery (From Goad 2001)

<u>Casino Foreshore</u>

For the southern section of foreshore, hard engineering techniques have been applied with the aim of protecting infrastructure at the Casino site. Revetment walling was constructed in the 1970's, causing beach instability, which gradually subsided, but initiated a progressive sequence of defensive foreshore works.

Presently there are three main types of revetment present along the southern foreshore, including two rock revetments ('large' and 'small' respectively) and one constructed from geosynthetic sand containers (large sand bags). Performance of the three different revetments suggests that the 'large' rock revetment is performing adequately in the active environmental conditions. The other two

revetments are damaged, but the relative tolerance of such walling to stresses above their design threshold has allowed them to continue to retain land. However, damage occurring under moderate conditions implies that there is a potentially unacceptable risk of failure under an extreme event.

Due to the ultra-dissipative structure of the intertidal beach and subtidal terrace, wave loading on the revetments is strongly affected by water levels. This occurs through both reduced frictional dissipation and increased frequency with which waves reach the revetment, presently exposed at an approximate level of +3m AHD. The implications for structural performance are:

- The design wave height will approximately rise with sea level, i.e. for a nominal design event of 1.0m significant wave height (Hs), under a 0.5m sea level rise, an event of equal frequency is likely to be roughly 1.4m Hs; and
- The increased frequency of wave action on the wall will place greater stress on the internal stability including inadequate geofabric and filter layers.

Market Foreshore

The northern section of Mindil Beach has been managed through the use of soft engineering techniques, including beach scraping and dune reconstruction. This is an active program of management, required on an annual basis.

The foreshore dune is presently built to a level of approximately +5.5m AHD, which gives a 1.5m freeboard at highest astronomical tide or approximately 1.0m above the present day 100 year average recurrence interval (ARI) still water level. Whilst this represents a fair level of protection against direct inundation due to wave dissipation across the beach terrace, the dune is only about 30m wide and is subject to erosion under severe wave conditions or sea level rise. A geometric extension of the existing beach slope suggests that the buffer could be eroded through a 2m vertical extension of the beach face, which may be generated by a sustained 10+ year ARI wave event coincident with high tide. The requirement for a storm to be coincident with high tide, along with the relatively short duration of such extreme conditions, determines that the recurrence interval for a ‘complete’ dune erosion event exceeds 50 years. Under sea level rise scenarios, this likelihood increases.

Creek Systems

The two creek systems located at the northern and southern ends of Mindil Beach provide an additional source of coastal dynamics. Under present conditions, there is limited evidence of their ability to interrupt sediment movement along the shore. However, such small catchment coastal creeks in the tropics have the potential to cause short-term reduction of beach material following channel scour during an extreme flood event. The smaller northern creek has a limited capacity for scour both due to its small scale and the presence of underlying rock platform on the beach face.

Beach Movements

Measurements of Mindil Beach profiles from 1996 to 2001 indicate that it is subject to oscillatory behaviour, with both erosion-recovery cycles and direction-switching of beach rotation (Gray 2002). These patterns obscure detection of a coastal trend, but in comparison with the management efforts required in the 1970’s through to the 1980’s (Coaldrake 1976; Wilkinson 1976; Richards & Fogarty 1979; Brown 1985; Letts & Kraatz 1989; Comley 1996) suggest that Mindil Beach has largely adjusted to the imposed coastal engineering and is dynamically stable under current conditions.

Present Day Management Focus

The existing management pattern of beach scraping and dune reconstruction provides a functional means of ensuring a protective buffer against moderate storm events. Effectiveness of the program should be assessed by means of beach profile measurement, with the historic record (Comley 1997; Gray 2002) providing a useful baseline for comparison.

The most challenging area for present-day management is the interface between the hard and soft engineering treatments. In its present configuration, the alignment of the revetment does not match that of the dune, which causes the end of the wall to protrude further on to the beach face than is desirable.

During the site visits the following observation on the effectiveness of current management practices and possible sources of construction material was made:

- A porcelanite cliff exists to the south of the carpark adjacent to Sky City Casino. There is a considerable amount of recently fallen rock (talus) which could be used as construction or aesthetic material to mitigate the visual impact of construction with other quarry material i.e. Acacia Bluestone. Note that porcelanite is suitable as a decorative rather than structural use because of its low strength. A significant volume of sand is evident in the lower part of Mindil Creek that could be retrieved for local beach nourishment.
- The ‘big’ rock wall in front of Sky City Casino is undamaged and the rock size and crest height indicate suitable design values.
- The ‘smaller’ rock wall to the north has suffered rock displacement, possible slumping and has been overtopped. This wall should be rebuilt or topped up to match the storm capacity of the ‘big’ rock seawall.
- The geofabric wall shows signs of its design condition being exceeded during the 25th January 2012 event with much of the lower layer displaced and some bags damaged (empty). Also, some of the 2nd and 3rd rows have slumped resulting in the need for the wall to be substantially rebuilt if it is going to provide future protection against similar events. The ‘end effect’ is to be expected as the seawalls are seaward of the overall beach alignment and as such reduce the ability of the beach to naturally realign with the prevalent wave direction. If the seawalls remain in their current location then the eroded ‘end effect’ will need to be maintained after storms.
- The sand scraping for the northern component of the beach was in progress during the second visit and was efficient and achieving a good result.
- The GIS layer provided by Council shows three stormwater drains but only one was visible.

2.1.4.2 Vesteys Beach

Darwin Sailing Club and a public boat launching ramp are located on Vesteys Beach, making it an important coastal interface for Darwin's boating community. The beach, which was originally comprised of a sand veneer overlying a beach rock ramp, has been extensively modified on the landward side, including construction of revetments in front of the club, fill for foreshore parking and limited armouring of a small storm scarp (refer Figure 2-29).

Figure 2-29 Vesteys Beach Historical Aerial Imagery (From Goad 2001)

Measurement of the beach system from 1996 to 2001 showed cyclic erosion and recovery (Gray 2002). Although this behaviour was not clearly demonstrated as driven by singular storm events, it is a typical characteristic of low-energy tropical beach systems.

Armouring of a previous storm scarp has restricted the planar extension of the beach face during high energy events. Consequently, wave impact results in wave runup and overtopping of the scarp. During moderate events, this process deposits sand across the pathway and the volume of overtopping water drains across the pavement area. For more energetic conditions, or under a sea level rise scenario, the volume of overtopping water may be sufficient to cause local flooding, or structural destabilisation of the path and pavement.

During the site visits the following observation on the effectiveness of current management practices and possible sources of construction material was made:

- There is a significant volume of porcelanite talus in front of the High School (area not managed by City of Darwin).

- The path in front of the Water Ski Club that had sand swept onto it in the January 2012 storm is low. Some local works could possibly build up this area (around the 'Doyle' memorial rock) Alternatively, since the storm was a relatively rare event then reactive maintenance of the walking and riding paths after major events may be less expensive.
- Dune scarps on the beach are minimal and no evidence of dune overtopping was observed during site visits.
- Again there is porcelanite talus at the northern end of the beach under the cliffs.

2.1.4.3 East Point

East Point Reserve is a popular recreation areas for both locals and visitors of Darwin. The reserve is readily accessible and it is by far the largest park area in the city with almost 200 hectares of land including 30 hectares of natural forest. Its management and control was passed to the City of Darwin in 1984. East Point Reserve includes a military history that goes back to 1932 and a range of community and tourist facilities including the Royal Australian Artillery Association Museum and the gun turret precinct.

The beaches on East Point Reserve are an important recreation resource with Fannie Bay Beach and its adjacent parkland and barbecue area being one of the most popular family beaches in Darwin.

During the site visits the following observation on the effectiveness of current management practices and possible sources of construction material was made:

- There is damage to a low revetment at the northern end of Fannie Bay Beach which appears to have been caused by freshwater runoff undermining the foundations and then further damage by wave action. This could be maintained with the inclusion of freshwater runoff drainage features.
- The area in front of the military reserve is exposed with no offshore reef area for protection (refer Figure 2-31 on page 2-36) This results in an accelerated erosion rate.
- Hotspots related to drainage concentrations (natural and manmade) exist throughout the area. In principle it would be preferable to divert the stormwater drainage to adjacent areas or to the base of the cliff to reduce impacts. The worst-affected section of cliff appears to be directly linked to a linear channel that drains the grassed area to the south.
- A large amount of stormwater was noted to be coming through the natural cracks and fissures in the rock. There may be a possibility of locating these by radar and filling – but this is invasive and is likely to transfer the problem to nearby locations. A full stormwater management study including the war time infrastructure may be able to devise means to divert stormwater from the more sensitive historical areas.
- Large rocks (~ 5t) at the heads of these chasms are likely to reduce wave erosion at the base of the cliffs. However they will be less effective at higher water levels (tide and/or surge) when wave conditions will be greatest.
- Another option would be to plan ahead and relocate/retreat as necessary. This would require a long lead time for the historic buildings to allow for consultation and planning.

2.1.4.4 Nightcliff

The Nightcliff foreshore was the site of Royal Australian Air Force camps with spotlights and large guns used to defend Darwin from bombing during World War II. During 1941, a naval outpost including a large concrete artillery outpost bunker was established on the headland. Various other defence facilities were constructed inland as large numbers of military personnel moved into the area. The 2/14 Field Regiment A.I.F. (Australian Infantry Force) was given the task of planning and constructing a hutted camp which became known as "Night Cliff's Camp". After the War, increasing pressure for suburban development caused the Nomenclature Committee of the N.T. to officially name the area on 29 October 1948. The conjoint version of the name, "Nightcliff" was adopted.

After TC Tracy the area was used as a relocation area for waste building materials.

In recent times, a path along the foreshore of Nightcliff is used for walking and cycling, particularly in the evenings after work. The footpath joins Nightcliff Jetty, Nightcliff Beach and Nightcliff Swimming Pool and a variety of user infrastructure such as BBQs, seating and exercise equipment.

During the site visits the following observation on the effectiveness of current management practices and possible sources of construction material was made:

- Along Casuarina Drive between Kurrajong Crescent and Kiranou Place, long period storm waves (swell) are overtopping low areas and sand is pushed onto the bike path. This is the natural way of increasing dune height to prevent overtopping but the sand is being redistributed (swept away). The mangroves barrier is thin at this point and is having little effect on reducing swell penetration at high water levels.
- This phenomenon will be exacerbated by increased sea level. If the predicted rise of 0.8m in 100 years is realised then significant management of this area will be required.
- The fissures north of the jetty cause the waves to shoal strongly because of the ‘funnelling effect’ of the fissure shape and the steeply rising beaches. The waves are likely to be shoaling to 2 or more times their offshore height with the result that rocks can be cast over the low cliffs into the adjacent parks (refer Figure 2-32).
- Large rocks (~5t) placed in the fissures may reduce wave height and reduce the rate of erosion. However, this may not reduce the wave heights at higher tide levels and caution will need to be exercised in storms.
- West of Nightcliff Beach, opposite Cedar Street, has significant cliff erosion at the platform level and also erosion caused by a stormwater outlet. There are existing seawalls (two of sloping loose rock and one grouted vertical revetment) which are intact although the vertical wall is degraded. The adjacent sloping seawalls indicate that a stable rock size appears to be about 1-2t.
- Observation of offshore bathymetry indicated that this area is the least protected by offshore rocky reefs and this natural exposure combined with freshwater runoff and drainage is resulting in accelerated recession. It was noted that the rock platform is located approximately at Highest Astronomical Tide level with variations in elevation due to the presence of variations in the morphology.

- Observations indicate that vegetation (strangler figs) will not provide any long term reduction in cliff rcession.
- The area immediately in front of the swimming pool is protected by offshore reefs but there is a localised area of cliff recession immediately to the north at a drainage outfall.
- There is a significant reduction in buffer between the cliff and the road to the east of the pool carpark, opposite Willow Way. There appears to be regional recession as well as localised acceleration due to a stormwater drainage outlet. Possible alternate stormwater discharge locations are located nearby.

2.2 Key Coastal Management Issues & Coastal Processes

Along the coastal areas considered for this plan, the basic morphology influences the processes driving the key coastal management issues. A simple example is for wave action, which produces gradual run-up on a sandy coast but may result in overtopping and wave cast on a rocky coast. Wave cast is the capacity for breaking waves, particularly at a rocky shore, to throw debris such as cobbles or boulders upward and landward. Under extreme conditions, a storm may build a rampart at the upper level of the rock cliff or platform (a tempestite).

The management issues are presented below for each of the three coast types.

2.2.1 Soft Rock Cliffs

Access along the soft rock cliffs runs a fine balance between discouraging general access across hazardous areas and providing a safe pathway for egress, particularly during emergencies. Existing access management includes provision of fencing along the majority of cliffs, with a small buffer to make allowance for potential erosion. A number of informal access paths have been created by less hazard-averse members of the public, highlighting the difficulty of the management trade-off.

2.2.1.1 Cliff Erosion

Erosion of the soft rock cliffs occurs at three levels, being the shore platform, the surface of the cliff exposed to wave action and the crest area, including processes from the adjacent land as shown in Figure 2-30.

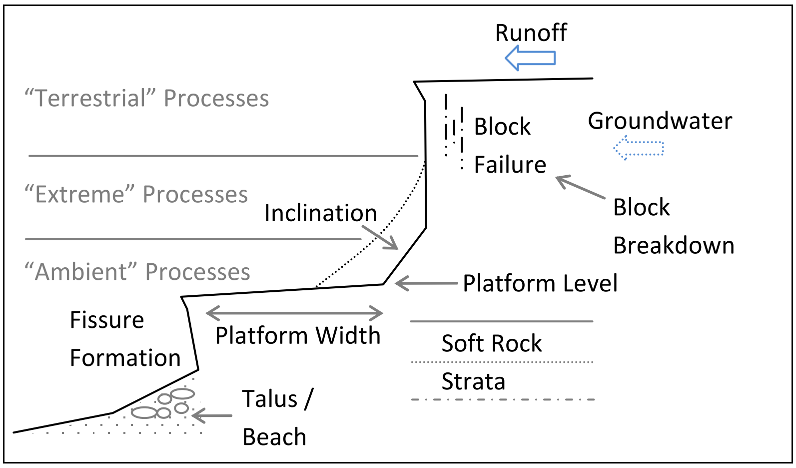


Figure 2-30 Schematic Diagram of Key Erosion Processes for Darwin Soft Rock Cliffs

The shore platform is exposed to ambient processes of tides and waves. In the critical areas of East Point and Nightcliff the absence of sloping offshore platform increases wave exposure (refer Figure 2-31).

Figure 2-31 East Point showing Lack of Offshore Platform at Military Area

The rate at which the platform is eroded is largely determined by the hardness of exposed strata and the presence of fissures caused by rainfall runoff. The latter produces a distinct saw-tooth structure along the edge of the platform for much of the coast, with indentations corresponding to local drainage runoff points. On shallow sections of cliff, breakdown of the platform may form blocks of material (talus) that sites in front of the cliff and slows wave-induced erosion. This occurs mainly where the lower sandstone is exposed, along parts of East Point, whereas the breakdown of siltstone or saprolite at higher levels, such as along Nightcliff, produces smaller blocks which are rapidly washed away.

The upper surface of the shore platform is washed by wave action, the occurrence of which varies significantly according to the platform level. Erosion of the material above the platform is determined by the material stability (composition and slope) along with the width and level of the platform, which determined effective wave loads at the cliff face. Funnelling of wave action through fissures or notches in the cliff face causes enhanced local erosion, which is significant along drainage lines.

The upper part of the cliff face is only exposed to wave action under extreme water level and wave conditions. These produce conditions conducive to block failure, including negative wave pressures and saturation. The corresponding erosion rates are generally more rapid for softer material (especially saprolite), although erosion of harder material (sandstone or siltstone) may occur in more obvious blocks. Indentations in the cliff face cause locally enhanced erosion rates, as on the lower part of the cliff.

The significant influence of channels caused by freshwater runoff on both the platform and the cliff face determine that management of groundwater and surface runoff is essential. As identified previously, focal points of runoff significantly enhance local erosion rates along the soft rock cliffs. Management of the drainage at a local scale is required to minimise the rate of erosion response.

2.2.1.2 Cliff Overwash

Overwash occurs on a number of sections of the soft rock cliffs, despite high crest levels. Subsequent to the 25th January 2012 storm sand and rock was observed on top of the cliffs just to the east of the Nightcliff jetty (Figure 2-32). When rock is thrown from the seabed up onto the cliff it is termed wave cast rock.

The capacity for overwash is more strongly determined by the wave conditions than the water level. Energetic wave conditions are required to mobilise any available material from the seabed or platform to towards the water surface, with transfer over the crest by wave runup. The effect is significantly enhanced by local notches in the platform and cliff face, which are acknowledged to increase water level amplification due to funnelling and effective runup by as much as 50% (Goda 1989).

Figure 2-32 Overwash Deposits Near Nightcliff Jetty
(a) Sand; (b) Wave Cast Rock Fragments

2.2.2 Sandy Beaches

Mindil and Vesteys beaches are both subject to considerable dynamics in response to variation of environmental conditions (Comley 1996; Gray 2002). Seasonal high waves and water levels during the Wet season commonly produce a beach scarp on the existing dune profile, and under more severe storms, denude the vegetation that has been planted to enhance dune stability. On Mindil Beach, this variation is artificially managed through beach scraping and dune reconstruction. For Vesteys Beach, the storm scarp has been armoured with riprap adjacent to infrastructure, or left as a near vertical face otherwise.

2.2.2.1 Beach Erosion

The beaches of Mindil and Vesteys show a normal reaction to wind and waves but appaear to be dynamically stable in the longer term. This means that the sand may move offshore and onshore seasonally or in response to storms but in the long term the beach remains in the same location. Revetment works on both beaches are used to protect built environment close to the shoreline. As indicated, Mindil beach is scraped each year to move sand from the inter-tidal zone to the dunes. This replicates the natural process although at an accelerated rate. Vesteys Beach has an indurated layer in the inter-tidal zone with beach sand perched above this. Again the existing condition of the beach appears stable in the long term.

However, if predicted sea level rise is realised, then the dunes at both beaches will need to be raised to continue to provide inundation protection.

2.2.2.2 Beach Access

Access to Mindil Beach requires regular management to accommodate dune erosion and rebuilding. The existing approach is to use flexible matting that is tied to fencing posts. This is generally considered an appropriate management technique under such circumstances. One minor modification that could be made is to align the paths at an angle to the beach, which marginally reduces the capacity for blowouts or breaching under energetic storm events (Ranwell and Boar 1986; Oma et. al. 1996; Kidd 2001).

At both Mindil and Vesteys beaches, low lying land behind the coast has been identified as subject to the threat of inundation during extreme water levels (SEA 2010). The frontal dune provides a measure of protection, up to an approximate level of 5-6mAHD. Whilst this represents a high level of inundation, the dune is directly affected by wave action. It is therefore subject to both erosion and breaching through low points such as access paths.

2.2.3 Mangrove Coast

The areas of mangrove coast managed by City of Darwin occur on the northern side of East Point and southwest side of Nightcliff. In both locations low lying land behind the coast (approximately 5-6mAHD) has been identified as subject to the threat of inundation during extreme water levels (SEA 2010).

2.2.3.1 Shoreline Access

Access requirements for these sections of coast, including the provision of a popular dual-use path located slightly shoreward of a low scarp, are primarily for recreational purposes as shown in Figure 2-33. Access constraints are largely determined by the potential for inundation under extreme events, and the more frequent issue of sand deposited by overwash.

Figure 2-33 Overwash near Casuarina Drive - January 2012

2.2.3.2 Shoreline Overwash

Overwash occurs on the scarp or revetment behind the mangroves during high storm conditions, causing sand deposits landward of the crest. This process is a natural coastal response to raised high water levels, and is likely to be enhanced under scenarios of sea level rise. In the area adjacent to Casuarina Drive the width of mangrove forest is thin allowing larger waves to propagate to the shoreline and cause overwash. Opportunities for management lie within the difference between the still water level and the limit of wave runup (Figure 2-34). These include raising the crest, flattening the scarp, increasing percolation losses, redirecting overwash flow, raising or relocating the path.

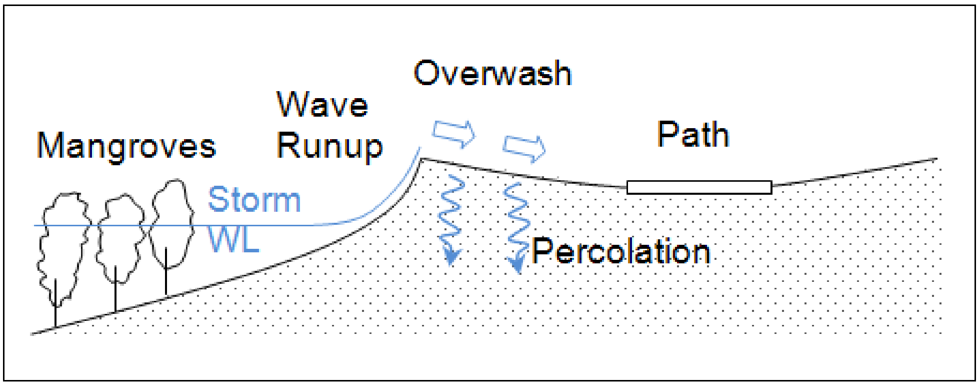


Figure 2-34 Schematic Diagram for Overwash Management

2.2.3.3 Shoreline Erosion

Erosion behind the mangroves has occurred very gradually. The mangroves generally provide a measure of protection against direct wave action. However, they also restrict the capacity for coastal recovery after an erosion event, consequently leading to gradual progressive erosion. Along part of Nightcliff coast, this has been managed through the installation of formal or informal revetments, constructed using rock armour or concrete rubble.

3 RISK IDENTIFICATION

In the assessment section of this report the areas of City of Darwin shoreline at risk from coastal hazards were identified and the causes and level of risk detailed. The areas of concern are shown in

Figure 3-1 and described below.

For shoreline recession, the areas of highest concern are cliff areas at Nightcliff and East Point where paths, roads and historical military infrastructure are at risk. The causes of recession at these locations have been identified as dissolution of the cliff at localised points on a regular basis by uncontrolled stormwater flow and cliff undercutting on an irregular basis by abrasion due to wave action during high wind events. Estimations of the rate of recession were not able to be clarified further than that reported in the Southern Cross University (2008) report, which estimated an average of 0.3m/year. Although historical photogrammetry was trialled, there were deficiencies in the “off the shelf” photogrammetric analysis (refer Section 2.1.2.7) which meant that the cliff face was not well defined.

Using regional average erosion rates does not provide sufficient information for effective management as localised areas of accelerated erosion, due to geological or other variations, will put these areas of infrastructure at greater risk to damage than the “average”. To enable prioritisation of management effort an accurate assessment of local erosion rates is necessary.

The beaches of Mindil and Vesteys appear to be relatively stable over the long term (refer Figure 2-26, Figure 2-27, Figure 2-28 and Figure 2-29).

Berm overwash is currently occurring during storm conditions at high tide along Casuarina Drive, between Kurrajong Crescent and Kiranou Place, at Nightcliff. Currently this is resulting in sand deposition on pathways and some damage to pathways (refer Figure 2-33).

Inundation, due to elevated water levels during cyclones as well as the predictions of future sea level rise, is also a threat to the low lying areas of Mindil Beach, Vesteys Beach and Nightcliff (refer Figure 3-2). This assessment is preliminary and based on the reports by SEA (2006 and 2010) by plotting a regional contour of 4.7mAHD for the existing 2010 storm tide level as well as a predicted 2100 level, including a sea level rise of 0.8m, of 5.5mAHD. These contours represent an indication of risk potential only at this stage. Future planning for predicted storm tide inundation should be the subject of future studies focussed on the interpretation and application of the reports by SEA (2006 and 2010).

Predicted climate change impacts, particularly sea level rise and storm intensity increases, are likely to have adverse effects in all areas. For cliff areas increased water levels and storm intensities will increase wave attack at the cliff face with a resulting increase in cliff recession. For beach areas the increased water levels will require the dune heights to be increased to maintain the current level of protection. For overwash areas the increased water levels and storm intensity will increase wave overtopping and associated inundation and damage.

The risk assessment in this section is based on current conditions and level of protection. It is not certain whether climate change predictions will be realised or how the natural environment will adapt

to the changes. However, it is not expected that the overall risk ranking will be affected by climate change and the discussion of mitigation options includes the implications of possible climate change impacts and methods to maintain or improve the level of risk mitigation.

Title:
Shoreline Areas of Concern

Figure: 3-1

Rev: A

BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

N

0 1 2km

Approx. Scale

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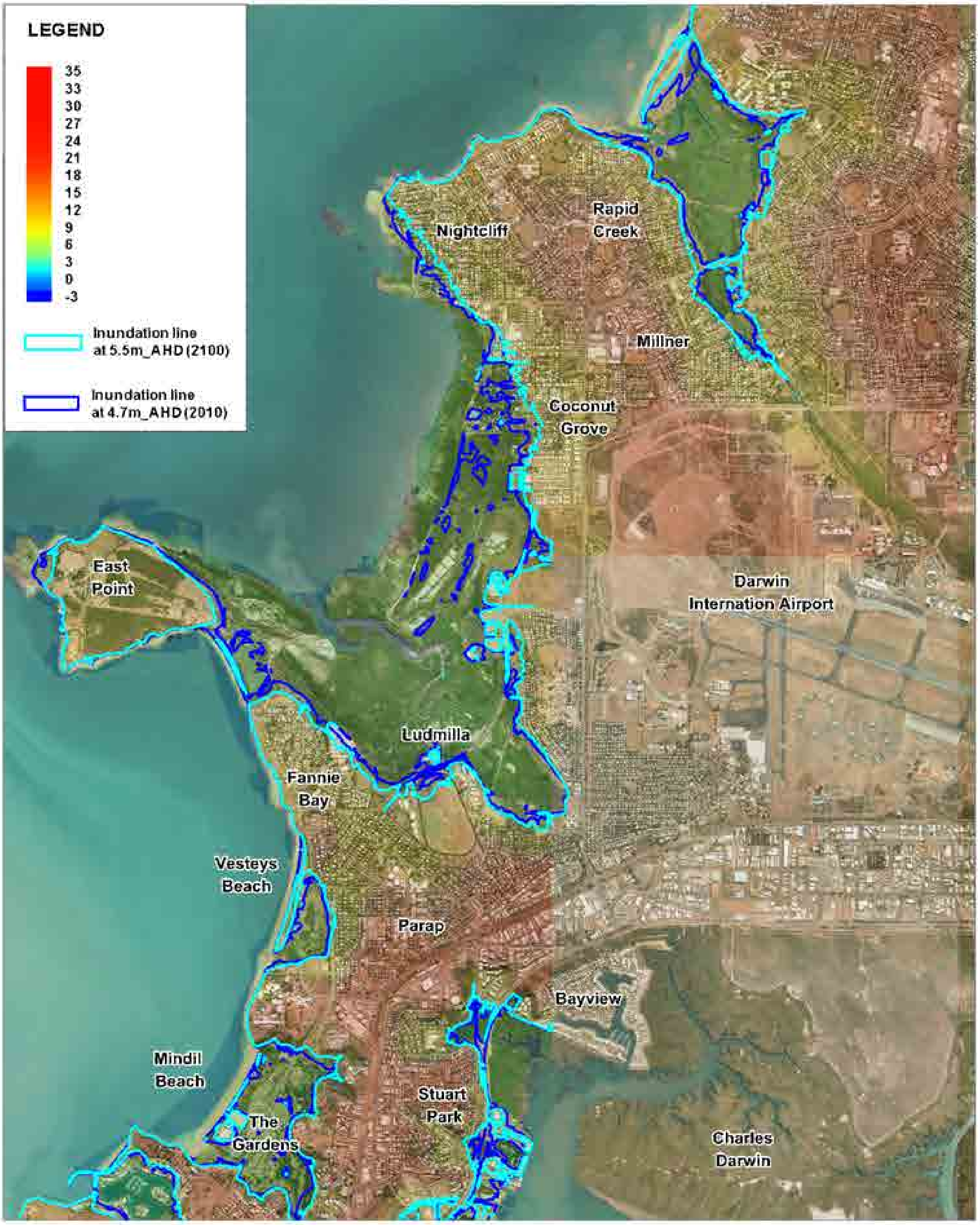

Title:

Storm Tide Inundation Lines 2010 and 2100
(Countours from Photogrammetric DEM 2011)

Figure: 3-2

Rev: A

N

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www.bmtwbm.com.au

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3.1 Likelihood and Consequence

The level of risk is the combination of the likelihood of an event occurring and the level of consequence (damage) of the event. This combination is displayed through a risk matrix, from which the level of risk is analysed. Scales for consequence and likelihood and their combination in a risk matrix form the risk criteria.

The likelihood is a combination of the physical processes involved, the regularity of occurrence and the rate of progress to a critical, unacceptable consequence. Estimates of the likelihood risk have been made in this study based on the knowledge of the driving forces (wind, waves, currents, rainfall runoff etc) and the resistive forces (rock characteristics, mitigation measures etc).

Coastal hazards incorporating shoreline erosion and recession and inundation can have varying impacts on coastal lands depending on how that land is valued. For example, the inundation of a private residence may have a short term reversible impact, while cliff failure or recession has a permanent and thus greater consequence.

For management purposes, the consequence of coastal hazards (e.g. from ‘high’ to ‘low’ impacts) will be largely dependent upon the values of private and public assets. The assigning of consequence shall also incorporate the permanency and type of hazards impact, for example short term repairable impacts from inundation, permanent impacts from erosion or undermining of cliffs and seawalls, resulting in risk to life from cliff falls etc.

3.2 Likelihood Ranking

For this study the likelihood of a hazard occurring is based on the analysis in Section 2 of this report and is summarised in Table 3-1.

3.3 Consequence Ranking

The consequence ranking based on the assessments in this report and discussions with Council officers are shown in Table 3-2.

Table 3-1 Likelihood Ranking

Site	Risk Causal Factor	Level of Hazard	Infrastructure at Risk	Rank	Comment
1a) Mindil Beach South	Seawall failure	High	Buildings and pathways	2	Small rock and geofabric seawalls failing
1b) Mindil Beach North	Beach erosion	Low	Park facilities	1	No observable long-term shoreline recession
2a) Vesteys Beach South	Minor inundation	Low	Pathway	2	Minor pathway inundation
2b) Vesteys Beach North	Seawall	Low	Maritime infrastructure	1	Seawall design appears adequate
3a) East Point South	Shoreline recession	Low	Parks and roads	2	Low retaining wall damage and minor shoreline recession
3b) East Point North	Cliff recession	High	Heritage buildings and roads	2	Slow regional cliff recession with fresh groundwater flows causing local increases
4) Nightcliff South	Berm overwash	High	Roads and pathways	3	Berm height unable to increase naturally
5) Nightcliff Central	Cliff recession	High	Roads and pathways	3	Slow regional cliff recession with stormwater discharge causing local increases
6) Nightcliff North	Cliff recession	High	Roads and pathways	3	Slow regional cliff recession with stormwater discharge causing local increases

Note: ranking is based on likelihood of occurrence with 3 = very likely; 2 = likely and 1 = not likely.

Table 3-2 Consequence Ranking

Site	Risk Causal Factor	Level of Hazard	Infrastructure at Risk	Rank	Comment
1a) Mindil Beach South	Seawall failure	High	Buildings and pathways	3	Significant buildings and infrastructure at risk
1b) Mindil Beach North	Beach erosion	Low	Park facilities	2	Roads, infrastructure and heritage risk
2a) Vesteys Beach South	Minor inundation	Low	Pathway	1	Minor pathway inundation risk
2b) Vesteys Beach North	Seawall	Low	Maritime infrastructure	3	Significant buildings and infrastructure at risk
3a) East Point South	Shoreline recession	Low	Parks and roads	1	Parks and minor facilities at risk
3b) East Point North	Cliff recession	High	Heritage buildings and roads	2	Cultural buildings at risk
4) Nightcliff South	Berm overwash	High	Roads and pathways	2	Pathway amenity at risk
5) Nightcliff Central	Cliff recession	High	Roads and pathways	3	Roads and infrastructure at immediate risk
6) Nightcliff North	Cliff recession	High	Roads and pathways	3	Roads and infrastructure at immediate risk

Note: ranking is based on consequence of occurrence with 3 = high; 2 = medium and 1 = low.

3.4 Risk Evaluation

The level of risk is determined by combining the consequence of a coastal risk with its likelihood, such as given in the risk matrix in Table 3-3. Likelihood and consequence ranking of coastal hazards will be combined to determine overall level of risk for each coastal hazard. The two variables are combined to produce a third variable being "risk" (high, medium, low).

Table 3-3 Risk Evaluation

		CONSEQUENCE		
		Low 1	Medium 2	High 3
LIKELIHOOD	Very Likely 3	Medium	High	High
	Likely 2	Low	Medium	High
	Not Likely 1	Low	Low	Medium

Based on the Likelhood Table (Table 3-1) the Consequence Table (Table 3-2) and the Risk Evaluation Table (Table 3-3) it is now possible to derive a Risk Ranking Table (Table 3-4).

Table 3-4 Risk Ranking

Site	Risk Causal Factor	Level of Hazard	Infrastructure at Risk	Rank	Comment
1a) Mindil Beach South	Seawall failure	High	Gardens and pathways	Medium	Immediate action required
1b) Mindil Beach North	Beach erosion	Low	Park facilities	Medium	Continue current practice and monitor
2a) Vesteys Beach South	Minor inundation	Low	Pathway	Low	Routine maintenance pathway relocation
2b) Vesteys Beach North	Seawall	Low	Maritime infrastructure	Medium	Monitor
3a) East Point South	Shoreline recession	Minor	Parks and roads	Low	Routine repair of retaining wall
3b) East Point North	Cliff recession	High	Heritage buildings and roads	Medium	Action required to protect historical infrastructure
4) Nightcliff South	Berm overwash	High	Roads and pathways	High	Routine maintenance raise berm
5) Nightcliff Central	Cliff recession	High	Roads and pathways	High	Immediate action required
6) Nightcliff North	Cliff recession	High	Roads and pathways	High	Immediate action required

3.5 Piority Ranking

The risk ranking provides a priority clustering of the work that needs to be done to mitigate the risk in each the categories (high, medium, low). A priority ranking will include the order of actions based on the value of infrastructure at risk. It is recommended that the priority ranking be as shown in Table 3-5.

In Section 4 of this report a range of actions to mitigate these risks will be developed.

Table 3-5 Priority Ranking

Location	Risk	Priority Ranking
Nightcliff North	Cliff recession – path and road	High
Nightcliff Central	Cliff recession – path and road	High
East Point North	Cliff recession - heritage structures and roads	High
Mindil Beach South	Seawall failure – significant infrastructure	Medium
Mindil Beach North	Beach erosion – road and park facilities	Medium
Nightcliff South	Berm overwash – path amenity	Medium
Vesteys Beach South	Berm overwash – path amenity	Low
Vesteys Beach North	Seawall failure – maritime infrastructure	Low
East Point South	Low retaining wall failure – park and facilities	Low

Note: The order in each ranking group does not indicate level of risk.

4 CONCEPT ACTIONS

Several sites have been identified where high erosion or inundation risks are present. Management options need to be developed to avoid the risk, minimise the risk, or mitigate the risk. Individual options may reduce the likelihood or the consequence, but more likely, a combination of options will provide the best outcomes. Consideration of the timeframes and triggers for implementing the options will also be of key importance for addressing uncertainty and managing future risks for existing development. The suite of options target both short-term and long-term issues associated with coastal risk along the City of Darwin coastline.

In this case, the focus of the management study is to manage risks from coastal hazards to public safety, public and private property, and community and environmental assets. There may also be a need for consideration of social, recreational, aesthetic and ecological values and issues in the development of management options but these were not included in the brief for this study. Many of the value assessments in this study have been made after consideration of anecdotal information and discussion with Council officers as detailed values data were not made available for this study.

4.1 Level of Design

Where structural actions are recommended the design has been progressed as far as possible with the information available. The design information available mainly consists of regional physical processes data e.g. wind, waves, tides and surge. There is little site specific data including onshore and offshore survey, design wave conditions and bed material characteristics including foundation capacity. This data is essential for preliminary engineering design.

Where seawall and breakwater rock sizes have been estimated, these have been assessed using the standard formula of Hudson (1974). An explanation of the design elements and calculation procedure is given in Appendix C along with a method of inferring the weight of rocks from their size.

4.2 Construction Costs

The assessments described below are based on conceptual design arrangements noting the construction practices and materials that are industry standard and have proven successful in adjacent areas and nationally. These options have been costed by a local cost consultant (Rider Levett Bucknall) and their full report is given in Appendix A. Generally the costs have included materials, labour, ancillary works and mobilisation/demobilisation costs and exclude GST, Consultants Fees and escalation after 2014.

4.3 Cliff Recession

Cliff recession is the coastal hazard that appears in all four high risk areas in the priority ranking. There are a larrge number of geomorphological reports indicating that cliff recession can occur in three vertical zones. At the lowest level the harder sandstone shore platform is exposed to the ambient processes of tides and waves and the rate at which the platform is eroded is largely determined by the hardness of exposed strata and the presence of fissures caused by freshwater flow. On shallow sections of cliff, breakdown of the platform may form blocks of material (talus) that

sites in front of the cliff and slows wave-induced erosion. This occurs mainly where the lower sandstone is exposed, along parts of East Point, whereas the breakdown of siltstone, such as along Nightcliff, produces smaller blocks which rapidly deteriorate and are washed away. The upper surface of the shore platform is washed by wave action, the occurrence of which varies significantly according to the platform level.

Erosion of the material above the platform (porcelanite and saprolite) is determined by the material stability (composition and slope) along with the width and elevation of the platform, which determine effective wave loads at the cliff face. Funnelling and amplification of wave action through fissures in the cliff face causes enhanced local erosion.

The upper part of the cliff face is only exposed to wave action under extreme water level and wave conditions. It should be noted that under storm conditions the amount of energy in the wave attack is proportional to the water depth i.e. wave energy will be highest at high tide. These conditions are conducive to block failure, including negative wave pressures and saturation. The corresponding erosion rates are generally more rapid for softer material (especially saprolite), although erosion of harder material (sandstone or siltstone) may occur in more irregular but dramatic failures of large blocks.

The significant influence of channels caused by freshwater runoff, stormwater discharge and groundwater percolation/movement on both the platform and the cliff face is well documented and runoff discharge points significantly enhance local erosion rates along the soft rock cliffs. Management of the drainage at a local scale is required to minimise the rate of erosion response.

In summary, there are two areas of significance (East Point and Nightcliff) where the long term rate of cliff recession due to coastal processes of waves and tides is of interest. The erosion rate in these locations has previously been assessed (SCU 2011) with the report indicating a regional average rate of 0.3m/a, but with significant variation both locally and regionally. Observation of the cliff recesson at sites where war time material was dumped offshore over 50 years ago generally verifies this average rate.

However, there is significant regional and local variation associated with varying rock levels, hardness and the discharge of stormwater at the cliff face and percolation through the cliff in some places. It is likely that management of stormwater discharge will reduce the accelerated erosion resulting in the longer term average of about 0.3m/a being re-established for wider areas.

This long term recession rate is related to regional processes such as day-to-day wind and wave conditions and therefore requires offshore wave energy dissipation or terminal shoreline protection to mitigate. It should be noted that higher water levels and more intense storms predicted with climate change will increase the rate of erosion.

4.3.1 Management Options

Considering the discussion above the management of cliff erosion will utilise the following mitigation strategies:

- Reduction of cliff face stormwater discharge to reduce localised accelerated erosion due to formation of fissures which concentrate wave energy;

- Consideration of offshore structures to reduce wave energy approaching the site;
- Use of terminal structures (e.g. seawalls) to reduce erosion at the shoreline; and
- Combinations of the above.

4.3.1.1 Long Term Regional Recession

The long term regional recession is driven at sea level and can only be mitigated by reducing the energy approaching the coastline. In this case the following management options are considered:

- Reducing wave energy offshore before reaching the shoreline (e.g. offshore breakwater);
- Absorbing the energy at the shoreline (e.g. seawall); and
- Mitigating localised erosion at low water levels (adding rocks to reinforce the platform).

In considering these concepts there are several physical relationships to consider:

- The range of water levels is in excess of 6m when including tide plus storm surge. Wave energy is reduced as the waves propagate to shore by bed friction. Therefore, greater wave heights will occur at higher water levels and where the offshore bathymetry is deeper i.e. reduced gently sloping offshore zone;
- The literature reviewed has indicated that offshore wave heights range up to 5-6m giving an nearshore wave height of around 3-4m depending on the offshore bathymetry and water level (local wave height can be up to 0.7 times the water depth); and
- The effectiveness of management options is closely tied to the ability to reduce the wave energy at the shoreline or to defend the shoreline against wave energy.

Reduction of wave energy before it reaches the shoreline requires some offshore formation to reduce the energy of the waves as they approach the site. Naturally this occurs in the form of gently sloping offshore platforms that dissipate energy through bed friction losses. These can be seen in several areas throughout the City of Darwin e.g. adjacent to the eroded area at East Point (refer Figure 2-31) and are very effective in reducing wave energy due to bottom friction and wave breaking.

<u>Offshore Breakwater</u>

Energy can also be dissipated through turbulence by using offshore structures to induce wave breaking. The design of these structures requires the crest to protrude into the top 80% of the water profile so that the wave orbitals are intercepted. At lower levels the waves pass over the structure with significantly less energy loss. For example at mid-tide the structure would need to be 2-3m high to be effective but at high tide in storm conditions this structure would be ineffective. For a storm occurring at high tide a structure 4-6m high is required. By default this structure would absorb all wave energy at lower tide levels. The logical approach is to design for the worst condition and hence the structure height adopted for this study is 5m (80% of water depth taken as 6m). To resist the wave condition of around 4m (up to 70% of water depth taken as 6m offshore) would need an armour rock size in the order of 5 tonne. Therefore a 5m high structure with a crest width of 3m with side slopes of 1:1.5 would have a corresponding volume of 50 cum per m of structure length.

Therefore, a likely cost to construct would be in the order of $60,000/m given that the structure will be submerged most of the time, subject to wave and tidal forces during construction, and would need to

be built using barges or bunded to gain access from shore. A typical offshore breakwater is shown in Figure 4-1.

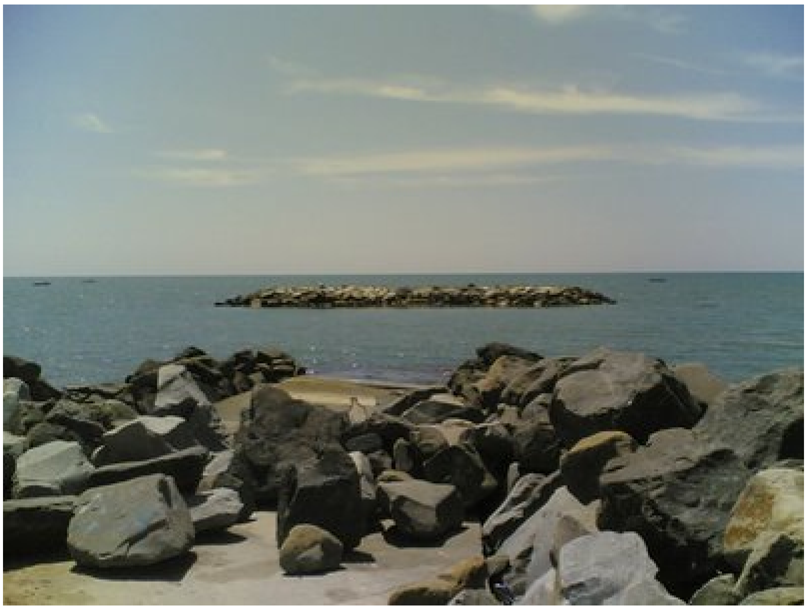

Figure 4-1 Typical Arrangement of an Offshore Breakwater

Seawall

The second defence mechanism is a seawall at the shoreline as has been used successfully in many parts of the Darwin shoreline. The intention in this case is to absorb the wave energy at the shoreline without damage to the cliff. The design would need to take into account the local conditions of waves and tides as well as the foreshore conditions such as current cliff face slope and foundation condition.. Currently, seawalls are being used effectively at Nightcliff with some including stormwater discharges within the seawall to improve the overall benefit.

The area between the two existing walls at Nightcliff will be slightly more exposed than the adjacent seawalls due to the eroded offshore condition. The area of interest at East Point is also very exposed. To be effective under the storm conditions indicated previously these seawalls will need to have site specific designs and have foundations keyed into the existing bed. To resist the design wave condition of around 3.5m (water depth taken as 5m at the shoreline) the seawalls would need a crest to foundation height of 6m and an armour rock size in the order of 3 tonne, with a face slope of 1:1.5 and will have a rock volume of 15 cum per m of structure length (2 layers of armour, filter layer underlain by geotextile). Therefore, a likely cost to construct would be in the order of $8,500/m. A typical seawall design for a sandy beach is shown in Figure 4-2.

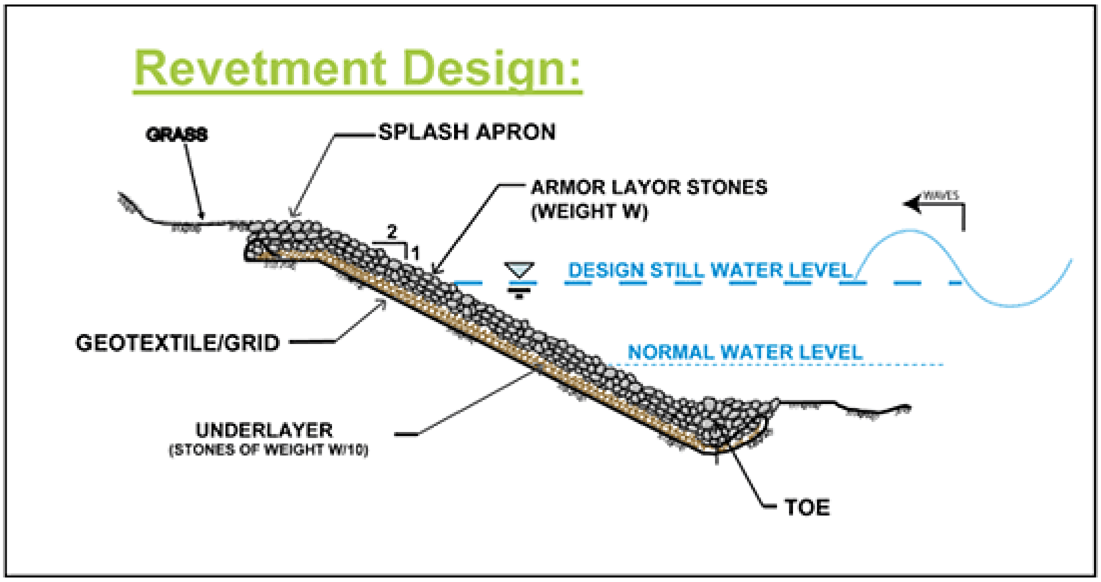

Figure 4-2 Typical Arrangement of an Seawall

Loose Rocks in Fissures

Lastly, localised protection could be provided by dumping large rock into erosion zones (e.g. Nightcliff near Cedar Street refer Figure 4-3). This is likely to have some benefit at lower tide levels although the 6m tidal range may reduce the benefit at higher water levels when wave energy is high. With rock in the platform under 6m of water is likely to provide only minor resistance to the large vertical forces experienced under 3.5m waves. A negative aspect of this option is that if the vertical forces result in the rock being displaced then the subsequent rock movement under wave action may cause extensive damage to the adjacent shelf material which is relatively weak. Experimentation with this option is possible with the recommendation that rock size be about 3 tonne to resist the vertical forces under storm waves. The cost of this option is likely to be in the order of $30,000 per site.

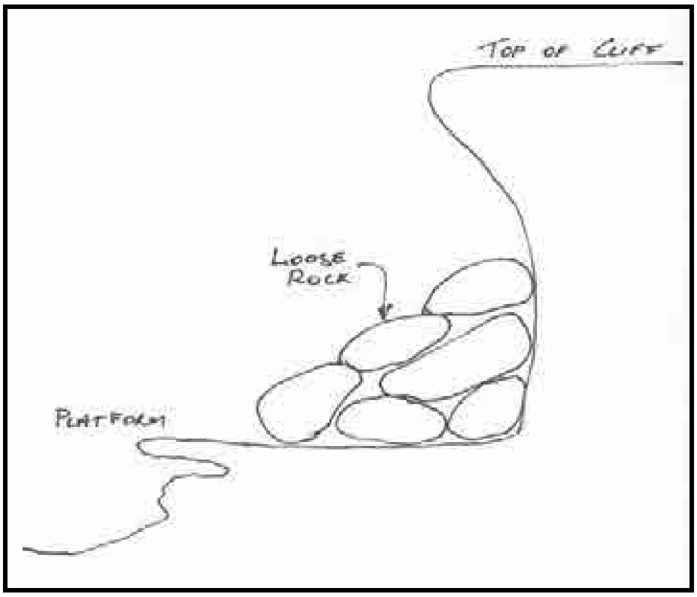

Figure 4-3 Typical Arrangement of Loose Rock Placement

4.3.1.2 Freshwater Drainage

Freshwater runoff can exacerbate cliff erosion when it is concentrated in drainage pipes or natural fissures and discharges at upper levels in the cliff face. Some flows can also follow natural surface contours to discharge through or over the cliffs. It has been established in many previous studies that this accelerates erosion by dissolution of the rock allowing fissures to form. These then concentrate wave forces exacerbating the accelerated erosion. Management of the freshwater drainage will reduce the sawtooth nature of the existing shoreline and the accelerated erosion at the discharge points. The intention of managing stormwater is to reduce cliff erosion to the general regional rate caused by wave and tide action.

Management of the flow of freshwater over the cliffs can be achieved in two ways. Firstly the piped stormwater system can be rationalised to reduce the number of discharge points and preferably to locate these in non-critical areas i.e. where the adjacent land is parkland or where there is a significant buffer to infrastructure. Costs for this option will be dependent on the feasibility at each location and capacity factors but on a basic assumption of two month’s work at day labour costs would indicate costs in the order of $300,000 per site.

Secondly, where discharge over the cliff is unavoidable then it is recommended that the discharge structures be redesigned to deliver water to the bottom of the cliff e.g. have the pipework extend to the base of the cliffs or protect the face of the cliffs behind the outfalls with shot-crete or similar material. However it should be noted that the higher parts of the cliff face are very weak and considerable effort may be required to stabilise e.g. rock bolting. Costs for this option will be dependent on the feasibility of the option and design factors but on a basic assumption of one month’s work at day labour costs would indicate in the order of $100,000 per site.

Thirdly, local runoff should be directed away from critical areas by the use of table drains associated with roads and paths and/or trench drains (refer Figure 4-4). Costs for this option will be dependent on the feasibility of the option and design factors but on a basic assumption of one month’s work at day labour costs would indicate in the order of $100,000 per site.

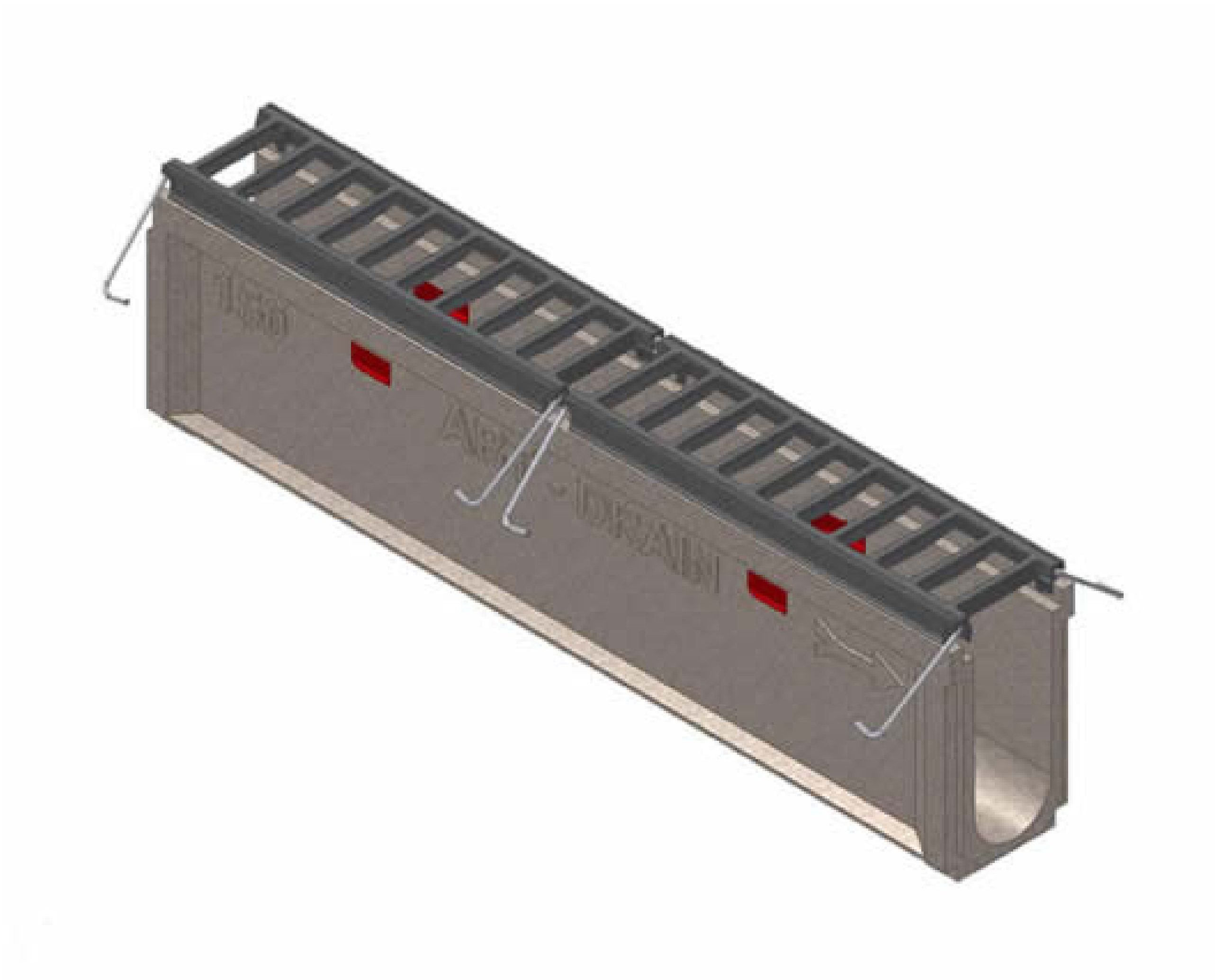

Figure 4-4 Typical Arrangement of a Trench Drain

4.3.1.3 Climate Change

The predictions for climate change which impact on the above options are a seal level rise of 0.8m in 100 years and a possible increase in storm intensity. For the structures considered this is likely to have little impact on rock size but may require the structure crests in be increased in elevation over time if the sea level rise is realised. This would be achieved through topping up of the structures.

4.3.2 Action Assessments

There are three locations where cliff recession is a high risk including:

- East Point North – Military Heritage area;
- Nightcliff South – South of Pool; and
- Nightcliff North - North of Pool.

Concept actions for each of these areas are described below.

4.3.2.1 East Point North

At this location the cliff recession is occurring by undercutting the cliffs near low water level (refer Figure 2-30). This action is greatly accelerated where freshwater discharge is occurring from formalised drainage including historical drains associated with the military infrastructure and natural fissures formed by dissolution of rock. The resulting fissures allow the wave attack to penetrate deeper into the shoreline and also amplifies the wave energy by funnelling and amplifying it to a narrow point at the head of the feature.

The mitigation actions considered for this site include the following:

- Dump large rock in the fissures to reduce wave energy penetrating into the shoreline (refer Figure 4-5). The rock must be in the order of 5 tonnes to resist the forces involved when storms with high winds and waves occur at high tide and the waves are amplified due to funnelling into the fissures. There is one main site causing road relocations and several secondary sites and it is estimated that around 20 rocks per site will be required to provide this protection with an inplace cost of $38,500 per site. Ultimately, if this option is successful it is likely that 6 sites will receive rocks giving an estimated cost of $231,000. As previously noted rock on the seabed under a 6m tide is likely to provide major resistance to the large vertical forces experienced under 3.5m waves. Therefore, the downside to this option is that if the vertical forces caused the rock to be displaced then the movement of these rocks under waves is likely to cause damage to the adjacent cliff material which is relatively weak. A low-cost option would be to trial the main site and monitor results.
- Rationalise the freshwater drainage to divert freshwater flow away from the cliff face. This will involve redirecting overland freshwater flows to adjacent areas by table drains associated with roads and/or trench drains. As well, identifying underground drainage (recent and historical) and directing freshwater flow to the base of the cliff if possible. Underground drainage through natural piping (dissolution of rock) may be difficult to locate and rectify. It is estimated that this option could involve 3 month's work at day labour costs giving a cost in the order of $311,000 for the site.
- Build a rubble mound seawall to resist erosion and include stormwater discharge at beach level in the seawall design (refer Figure 4-6). To resist the design wave condition of around 3m the seawalls would need a crest to foundation height of 6m and an armour rock size in the order of 3 tonne. With and face slopes of 1:1.5 this would have a corresponding volume of 15 cum per m of structure length. Therefore, a likely cost to construct would be in the order of $9,000/m. If this seawall were built across the critical section of 170m then it would have an estimated cost of $1,450,000.
- Construct an offshore breakwater to reduce the wave energy reaching the shoreline (refer Figure 4-7). The most efficient approach is to design for the peak condition and hence the structure height adopted for this study is 5m. To resist the wave condition of around 4m (up to 70% of water depth taken as 6m offshore) would need an armour rock size in the order of 5 tonne and with side slopes of 1:1.5 would have a corresponding volume of 50 cum per m of structure length. Therefore, a likely cost to construct would be in the order of $65,000/m given that the structure will be submerged most of the time, subject to wave and tidal forces, and would need to be built using barges or bunded to gain access from shore. If this breakwater were built to protect the entire site, estimated 170m in length, then it would have a cost of $10,800,000.

The options for East Point North are summarised in Table 4-1.

Figure 4-5 East Point North 2011 Showing Possible Areas of Loose Rock Placement

Figure 4-6 East Point North 2011 Showing Possible Seawall

Figure 4-7 East Point North 2011 Showing Possible Offshore Breakwater

Table 4-1 Summary of Options for East Point North

Option	Cost	Effectiveness	Timing	Comment
1. Dump rock in fissures	Trial $38,500 Total $231,000	Limited due to tidal range	Trial immediately Total option over 2 years	Limitations to option include ineffectiveness at high tide and loss of aesthetics due to use of imported rock. Possible interference to dumped ferric war material.
2. Rationalise drainage	$311,000	Effective for surface water – possibly limited regarding existing natural piping through rock	Immediate	The intention is to divert as much freshwater as possible away from the cliffs
3. Seawall	$1,450,000	Very effective	After trial of Option 1	A seawall will be effective in stopping erosion but will reduce the natural aesthetics of the area. It will also let the natural flow of freshwater over/through the cliffs occur.
4. Offshore breakwater	$10,800,000	Very effective	After trial of Option 1	An offshore breakwater will be effective in reducing wave energy and erosion. It will also allow the natural flow of freshwater through/over the cliffs to occur. However, it will reduce the natural aesthetics associated with a sea view.

It is considered that the best response will be achieved by a combination of options. Drainage rationalisation is an essential component but at best can only achieve the regional estimated erosion rate of 0.3m per year if a total reduction of the accelerated erosion due to freshwater drainage is achieved. Therefore, valuable assets will need further protection from the longer term recession due to waves and tides. In this regard a trial of large loose rock in the main fissure (refer Figure 4-5) is the next recommended option. If this trial is unsuccessful then seawall construction will be required (refer Figure 4-6). An offshore breakwater remains a long term option but may have significant distractions in terms of cost and aesthetics (refer Figure 4-7).

Therefore it is recommended that the following actions be undertaken:

- Immediate action to divert as much stormwater discharge as possible from the cliff face. The stormwater may be able to be redirected to adjacent shorelines. Expected cost over 1 year $311,000.
- Trial loose rock within the main platform fissure only at an estimated cost of $38,500 over 1 year. If successful continue placement across the sire with estimated final cost of $231,000.
- In the longer term prepare for the construction of a seawall at a cost of $1,450,000 with timing dictated by results of rock trial and re-analysis of the long term cliff recession rate.

4.3.2.2 Nightcliff Central

At this location it appears that the recession is occurring by erosion of the platform near low water level as well as accelerated erosion of the cliff face due to freshwater flows. Seawalls have been successfully built on either side of this site and include stormwater discharge at platform/beach level in the design. These structures are very successful with the only detraction being that they are built from imported rock. There is also an aged verical revetment made from grouted rock. This appears to be functional at this time but close inspection indicates that long term performance may be in doubt due to deterioration of the foundations causing significant cracking of the grouting between rocks.

The mitigation actions considered at this site include the following:

- Dump large rock in the fissure in the platform to reduce wave energy penetrating into the shoreline (refer Figure 4-8). The rock must be in the order of 3 tonne to resist the forces involved when storms with high winds occur at high tide. There is one main site and it is estimated that around 20 rocks will be required to provide this protection with an inplace cost of $25,000. As previously noted rock on the seabed under a 6m tide is likely to provide only minor resistance to the large vertical forces experienced under 3.5m waves. Therefore, the downside to this option is that if the vertical forces caused the rock to be displaced then the movement under waves is likely to cause extensive damage to the adjacent material which is relatively weak. For aesthetical reasons similar rock to that in the platform could be considered however it is understood that this is not commercially available.
- Rationalise the freshwater drainage to divert freshwater flow away from the cliff face. This will involve redirecting overland freshwater flows to adjacent areas by table and trench drains and also diverting stormwater to adjacent outlets located low on the beach in the seawalls adjacent to the site. It is noted that stormwater design limitations may constrain this option. Less optimal options would be to protect the cliff face using shot-crete or to modify the existing outfall to

discharge at beach level. It is estimated that this could involve 3 month's work (including pipe network re-design) at day labour costs giving a cost in the order of $286,000 for the site.

- Build a rubble mound seawall, similar to adjacent seawalls, to resist erosion and include stormwater discharge at beach level in the seawall design (refer Figure 4-9). To resist the design wave condition of around 3.5m the seawalls would need a crest to foundation height of 6m with an armour rock size of 3 tonne and with a 1:1.5 face slope would have a rock volume of 15 cum per m of structure length. Therefore, a likely cost to construct would be in the order of $10,000/m. To protect the critical area of erosion this seawall would be 50m in length and have an estimated cost of $504,000.
- Construct an offshore breakwater to reduce the wave energy reaching the shoreline (refer Figure 4-10). The most efficient approach is to design for the peak condition and hence the structure height adopted for this study is 5m. To resist the wave condition of around 4m (up to 70% of water depth taken as 6m offshore) would need an armour rock size in the order of 5 tonne and with side slopes of 1:1.5 would have a corresponding volume of 50 cum per m of structure length. Therefore, a likely cost to construct would be in the order of $61,000/m given that the structure will be submerged most of the time, subject to wave and tidal forces, and would need to be built using barges or bunded to gain access from shore. If this breakwater were built to protect the entire site, estimated 100m in length, then it would have a cost of $6,100,000.

The options for Nightcliff Central are summarised in Table 4-2.

Figure 4-8 Possible Loose Rock Protection at Nightcliff Central

Figure 4-9 Possible Seawall at Nightcliff Central

Figure 4-10 Possible Offshore Breakwater at Nightcliff Central

Table 4-2 Summary of Options for Nightcliff Central

Option	Cost	Effectiveness	Timing	Comment
1. Dump rock in depleted platform	$25,000	Limited due to tidal range	Trial immediately	Limitations to option include ineffectiveness at high tide and loss of aesthetics due to use of imported rock
2. Rationalise drainage	$285,000	Effective if stormwater can be diverted or discharged at platform level	Immediately	The intention is to divert as much stormwater as possible into adjacent networks and reduce overland flow
3. Seawall	$504,000	Very effective	After trial of Option 1	A seawall will be effective in stopping erosion. Will reduce the natural aesthetics of the area.
4. Offshore breakwater	$6,100,000	Very effective	After trial of Option 1	An offshore breakwater will be effective in reducing wave energy and erosion. Will reduce the natural aesthetics associated with a sea view.

It is considered that the best response will be achieved by a combination of options. As such it is considered that drainage rationalisation is an essential component but at best can only achieve the regional estimated erosion rate of 0.3m per year (i.e. a total reduction of the accelerated erosion due to freshwater drainage). Therefore, valuable assets will need further protection from the longer term recession due to waves and tides. In this regard a trial of large loose rock in the main fissure (refer Figure 4-8) is the next recommended option. If this trial is unsuccessful then seawall construction will be required (refer Figure 4-9). Offshore breakwaters remain a long term option but they have significant distractions in terms of cost and aesthetics (refer Figure 4-10).

Therefore it is recommended that the following actions be undertaken:

- Trial loose 3 tonne rock within the platform fissure only at an estimated cost of $25,000 over 1 year.
- Investigate diverting as much stormwater discharge from the cliff face as possible. The stormwater may be able to be redirected to adjacent shorelines. Expected cost over 1 year $286,000.
- In the longer term prepare for the construction of a seawall at a cost of $504,000 with timing dictated by results of drainage works and the loose rock trial.

4.3.2.3 Nightcliff North

At this location it appears that the recession is occurring by erosion of the platform near low water level as well as some accelerated erosion of the cliff face due to freshwater flow. The platform erosion is assumed to be at the regional rate of 0.3m/a. Currently there is an inadequate buffer requiring consideration of immediate defencive actions such as a seawall or offshore breakwater. Also of importance is the accelerated erosion caused by stormwater discharges near the top adjacent to the Nightcliff Pool carpark and in the cliff face near Cedar Street.

In order of cost the mitigation actions could include:

- Rationalise the freshwater drainage to divert freshwater flow away from the cliff face (refer Figure 4-11). This will involve redirecting overland freshwater flows to adjacent areas by table and trench drains and modifying stormwater to outlets to discharge low on the beach. It is noted that corridor width limitations may constrain this option. Less optimal options would be to protect the cliff face using shot-crete although the weak nature of the rock is likly to require technigues such as rock bolting to achieve success. It is estimated that the stormwater resolution could involve 3 month's work (including pipe network re-design) at day labour costs giving a cost in the order of $286,000 for the site.
- Build a rubble mound seawall, similar to other Nightcliff seawalls, to resist erosion and include stormwater discharge at beach level in the seawall design (refer Figure 4-12). To resist the design wave condition of around 3.5m the seawalls would need a crest to foundation height of 6m with an armour rock size of 3 tonne and with 1:1.5 face slope would have a rock volume of 15 cum per m of structure length. Therefore, a likely cost to construct would be in the order of $10,000/m. To protect the critical area of erosion this seawall would be 110m in length and have an estimated cost of $1,100,000.

- Construct an offshore breakwater to reduce the wave energy reaching the shoreline. The most efficient approach is to design for the peak condition and hence the structure height adopted for this study is 5m (refer Figure 4-13). To resist the wave condition of around 4m (up to 70% of water depth taken as 6m offshore) would need an armour rock size in the order of 5 tonne and with side slopes of 1:1.5 would have a corresponding volume of 50 cum per m of structure length. Therefore, a likely cost to construct would be in the order of $65,000/m given that the structure will be submerged most of the time, subject to wave and tidal forces, and would need to be built using barges or bunded to gain access from shore. To be effective this breakwater would need to be about 150m in length and would have a cost of $9,500,000.

The options for Nightcliff North are summarised in Table 4-3.

Figure 4-11 Possible Stormwater Drainage Changes at Nightcliff North

Figure 4-12 Possible Seawall Location at Nightcliff North

Figure 4-13 Possible Breakwater Location at Nightcliff North

Table 4-3 Summary of Options for Nightcliff North

Option	Cost	Effectiveness	Timing	Comment
1. Rationalise drainage	$286,000	Effective if stormwater can be diverted and overland flow reduced	Immediate	The intention is to divert as much stormwater as possible into adjacent networks and reduce overland flow
2. Seawall	$1,100,000	Very effective	After trial of Option 1	A seawall will be effective in stopping erosion. Will reduce the natural aesthetics of the area.
3. Offshore breakwater	$9,500,000	Very effective	After trial of Option 1	An offshore breakwater will be effective in reducing wave energy and erosion. Will reduce the natural aesthetics associated with a sea view.

It is considered that the best response will be achieved by a combination of options. Drainage rationalisation is an essential component but at best can only achieve the regional estimated erosion rate of 0.3m per year if a total reduction of the accelerated erosion due to freshwater drainage is achieved. Therefore, valuable assets will need further protection from the longer term recession due to waves and tides. In this regard a seawall construction will be required. Offshore breakwaters remain a long term option but they have significant distractions in terms of cost and aesthetics.

Therefore it is recommended that the following actions be undertaken:

- Investigations be initiated to relocate as much stormwater discharge from the cliff face as possible. The stormwater should be redirected to adjacent shorelines with an expected cost over 1 year of $286,000.
- At the same time prepare for the construction of a seawall at a cost of $1,100,000 with timing dictated by results of drainage works and reassessed rates of cliff recession.

4.3.2.4 Nightcliff Pool Area

The Nightcliff swimming pool was constructed on a headland in Central Nightcliff in the mid-1960s. The headland and the adjacent shorelines (refer Figure 4-14) show symptoms similar to other cliff areas with a significant talus visible indicating active erosion. The erosion appears linked to natural platform variations combined with wave erosion to the south (i.e. from extreme northerly waves) and stormwater erosion resulting from overland flow concentrations to the west and piped discharges on adjacent sides.

The pool has a good buffer to the edge of the cliff in most areas with the least distance being to the south where there is about 25m clearance. The annual average cliff erosion rate is around 0.3m/yr and with localised accelerated erosion due to focussed stormwater runoff increasing this to around 0.5m/yr. Therefore, it is estimated that protection works would not be required for around 50 years.

Any future planning for infrastructure in this area would need to consider:

- Maintaining a prudent buffer to edge of cliff edge (~25m);
- Architectural and construction elements to maintain the buffer;
- Protection (seawall) if construction is likely to be closer to edge;
- Stormwater implications as runoff concentrations will accelerate erosion i.e. consider berm to divert stormwater to less critical areas;
- Geotechnical advice focused on maintaining uncompromised foundations (i.e. protection from wave erosion and freshwater dissolution).

Figure 4-14 Headland near Nightcliff Pool

4.4 Sandy Beaches

It has been established that the sandy beaches of Mindil Beach and Vesteys Beach are both subject to considerable dynamics in response to variation of environmental conditions but appear to be dynamically stable in the longer term. Dynamic stability means that the beach sand is moved offshore during storm conditions and back onshore during ambient condition with no net change to beach volume or shoreline location. Seasonal high waves and water levels during the wet season commonly produce a beach scarp on the existing dune profile, and under more severe storms, denude the vegetation that has been planted to enhance dune stability. There are indications that the sand has moved into the intertidal zone. On Mindil Beach, this variation is artificially managed through beach scraping and dune reconstruction. Whilst this aids in maintaining the dunes it is sand budget neutral i.e. no extra sand is added to the system nor is it induced to arrive from elsewhere. For Vesteys Beach, the beach and storm scarp are perched above an indurated layer in the inter-tidal zone and the beach scarp has been armoured with riprap adjacent to infrastructure, and left as a near vertical face elsewhere.

Access to Mindil Beach is by fenced tracks which require regular management to accommodate dune erosion and rebuilding. The existing approach is to use flexible matting that is tied to fencing posts and is generally considered an appropriate management technique under such circumstances. One minor modification that could be made is to align the paths at an angle to the beach, which marginally reduces the capacity for blowouts or breaching under energetic storm events. The angle of inclination needs to be sufficient to reduce aeolian drift and commonly 15-30 degrees is used by will be subject to experimentation.

In both locations, low lying land behind the coast has been identified as subject to the threat of inundation during extreme water levels (refer Figure 3-2). The frontal dune provides a measure of

protection, up to an approximate level of 5-6mAHD. Whilst this represents a high level of inundation, the dune is directly affected by wave action and is therefore subject to both erosion and breaching through low points such as access paths.

4.4.1 Management Options

The beaches are currently dynamically stable in the long term and are of sufficient height to prevent significant inundation during present day storms. Therefore, existing management action in the form of routine maintenance and monitoring will continue to allow these beaches to provide a high value, low cost amenity to the City.

For Mindil Beach the routine maintenance includes annual scraping after the monsoon season to move sand from the intertidal zone to the dune and maintenance of walkways across the dune after storms. For Vesteys Beach very little maintenance is required.

In the longer term if predicted sea level rise occurs then the dunes at both beaches may need to be raised in some areas to continue to form a barrier against overwash and inundation. The height required will be dependent on the realisation of predicted sea level rise and as such could be up to 0.8m in the next 100 years. If sea level is realised there may be many secondary effects including more intense storms and cyclones and an overall change in wind and wave directions. The science regarding these scenarios is not definitive and most Council's adopt a Climate Change Adaptation strategy to plan for possible changes.

4.4.2 Recommended Actions

As the beaches of Mindil and Vesteys are dynamically stable it is recommended that the current management practices be continued.

4.5 Seawalls

Seawalls currently exist at several locations along the City of Darwin shoreline. These include Mindil Beach South, Vesteys Beach North and Nightcliff. There is also a low grouted rock revetment at East Point South but this is not of an engineering design standard.

The seawall that exists on Mindil Beach South has been built over several stages primarily as protection of adjacent infrastructure. During the site visit after the January 2012 storm it was noted that the recently installed rock seawall (approximately 75m in length) was undamaged with no rock displacement and no signs of overtopping and therefore incorporates reasonable design values. The next rock seawall to the north (approximately 150m in length installed during Casino construction) has suffered rock displacement and possible slumping and has been overtopped indicating that the current seawall is now inadequate and requires maintenance. Adjacent to the north is a geofabric "soft rock" seawall extending a further 70m in which many of the lower row of geofabric bags are displaced and some bags damaged (empty). Also, some of the 2nd and 3rd rows have slumped. At this stage it is unlikely that this wall will provide future protection against significant storms and will need to be monitored to ensure its structural integrity is maintained. If it deteriorates then it will need to be maintained or replaced with a rock seawall. An 'end effect' where accelerated erosion occurs adjacent to the wall is present and is expected. This area will need to be maintained after storms.

The rock revetments that exist at Vesteys Beach North appear to be undamaged and are adequately protecting the infrastructure located behind them. The seawalls have boat ramps and pedestrian access built into them to provide adequate access to the beach.

The seawalls at Nightcliff (near Mimosa Street and Walker Street) appear to be sound and providing adequate protection to infrastructure at these locations. They are flanked by pathways to the beach providing amenity to residents and both seawalls also have stormwater discharges built into them that discharge at beach level reducing the degree of cliff eroision.

The aged low grouted rock revetment at East Point South has been undercut by overland stormwater flow and is not currently providing shoreline erosion protection.

4.5.1 Management Options

The management options for slumped and damaged seawalls protecting infrastructure is to maintain or upgrade the seawalls. This design also needs to contain basic elements such as foundation and toe design, geofabric filters and adequate armour rock size. These can be taken from the adjacent undamaged seawall or reassessed by a qualified designer. The smaller rock seawall and the geofabric seawall at Mindil Beach South need to be maintained or rebuilt to higher specifications to provide long term protection to infrastructure.

The low grouted rock revetment at East Point South should be replaced with a similar grouted wall with adequate provision for drainage of overland stormwater flow.

All other rock seawalls along the Darwin City shoreline appear to be performing adequately.

The predictions for climate change which impact on the above options are a seal level rise of 0.8m in 100 years and a possible increase in storm intensity. For the existing structures this is likely to have little impact on rock size but may require the structure crests in be increased in elevation. This would be achieved through topping up of the structures with equivalent sized rock or possible larger rock depending on the current design standard.

4.5.2 Recommended Actions

It is recommended that the small rock seawall and geofabric “soft rock” seawall on Mindil Beach South be upgraded to the same standard as undamaged section. This will likely include topping up with larger rock (approx. 1 tonne) at a cost of around $120,000 ($1,000/m) and monitoring of the geofabric “soft rock” seawall to ensure deterioration does not occur. It its structural performance is further compromised then maintenance of the geofabic “soft rock” seawall to the original plans at a maintenance cost of $70,000 ($1,000/m) is recommended.

The low grouted rock revetment at East Point South should be replaced with a similar grouted wall with adequate provision for drainage of overland stormwater flow at an estimated cost of $542,000.

4.6 Foreshore Overwash

Low-lying land behind areas of mangrove coast has been identified as subject to the threat of inundation during extreme water levels (refer Figure 3-2). In populated areas such as Nightcliff South (Causarina Drive) popular paths are located slightly shoreward of a low beach berm. After storms at

high tides access is restricted by sand deposited on the paths by overwash and some damage to the path has occurred.

Overwash occurs on the scarp or revetment behind the mangroves during storm conditions at high tide. The longer period waves cause sand to be carried over the berm and deposited landward of the existing crest. This process is a natural coastal response to raised high water levels, and in undeveloped areas would result in an increase in elevation of the berm until overwash is eliminated. The overwash is likely to be exacerbated with increases in sea level rise. The mangroves generally provide a measure of protection against direct wave action except when winds are from the west and longer period waves are created. However, they also restrict the capacity for coastal recovery after an erosion event, consequently leading to gradual progressive erosion. Along part of Nightcliff coast, this has been managed through the installation of formal or informal revetments, constructed using armour rock or concrete rubble.

A lesser problem of overwash occurs at Vesteys Beach South near the Water Ski Club where overwash deposits sand on the walking path in severe storms.

4.6.1 Management Options

Opportunities for management of overwash at Nightcliff South lie within the zone between the still water level and the limit of wave runup. These options for consideration include:

- Raising the berm crest by 300mm to runup height by the addition of sand or rock;
- Reducing shore slope to reduce runup (constrained by mangroves);
- Increasing percolation losses to reduce runup (provide coarser material e.g. 10mm gravel layer on the beach face – reduction of amenity);
- Re-directing the overwash flow to deliver the sand elsewhere e.g. overwash drainage between the berm and the pathway; and
- Raising or relocating the pathway to above the runup level (i.e. rebuild pathway at higher level).

Many of these options are constrained by the width available to carry out works. It is considered that raising the berm crest level is the only viable option with a high likelihood of success. It also has little disruption to existing usage and the natural environment. An increase in the use of rock revetments to reduce overwash will reduce the amenity of these beach areas. Therefore, the recommendation is to include the placement of beach sand along the top of the berm where overwash is occurring to replicate the natural process (refer Figure 4-15). To raise the berm by 300mm the quantity of sand required will be about 1.5m^3/m of overwash length of 100m i.e. about 150m^3 at a cost of $16,000.

At Vesteys Beach South where the path is adjacent to the shoreline there is no opportunity to modify the berm level. Therefore, the path must be relocated, raised or proactive maintenance after storms will be needed. An allowance of $48,000 has been estimated for path relocation or raising.

In the longer term, if predicted sea level rise is realised then the management option will need to be changed as the space available to berm raising by sand is limited due to the natural repose angle of sand (about 1:5). When sea level rise exceeds 0.3m (predicted in 50 years) then the option for path protection will need to change to a steeper option i.e. rock. It should be noted that inundation of adjacent properties will also become an important issue at this time.

Figure 4-15 Area of Berm to be raised at Nightcliff South

4.6.2 Recommended Action

After consideration of the alternatives it is recommended that the berm overwash and sand deposition problem at Nightcliff be overcome by raising the berm crest level. This option would include the placement of beach sand along the top of the berm where overwash is occurring to replicate the natural berm building process. To raise the berm by 300mm the quantity of sand required will be in the order of $1m^3/m$ of overwash length i.e. about $150m^3$.

At Vesteys Beach South where the path is adjacent to the shoreline it is recommended that the path must be relocated or raised. An allowance of $48,000 has been estimated for path relocation or raising.

In the longer term, if predicted sea level rise is realised then the management option will need to be changed as the space available to berm raising by sand is limited. When sea level rise exceeds 0.3m (predicted in 50 years) then the option for path protection will need to change to an option which can be placed at steeper slopes i.e. rock.

5 RECOMMENDED ACTIONS

The actions recommended in this study are summarised below and in Table 5-1 and indicated in Figure 5-1, Figure 5-2, Figure 5-3 and Figure 5-4.

5.1 Cliff Recession

There are two main locations where the risks associated with ongoing cliff erosion are high. These are East Point and Nightcliff. In both areas the cliff recession appears to exceed the regional average as there is no offshore sloping platform to reduce wave action i.e. in front of the historical military area and either side of the aquatic complex at Nightcliff. However, the cliff recession rate is also locally accelerated by the overflow of stormwater at or near the top of the cliff. The dissolution of the rock by freshwater causes fissures in the cliffs which then concentrate wave energy and exacerbates erosion.

In consideation of the discussion above the recommendations for management of cliff erosion has considered the following mitigation strategies:

- Consideration of offshore structures to reduce wave energy approaching the site;
- Use of terminal structures (e.g. seawalls) to reduce erosion at the shoreline;
- Reduction of cliff face stormwater discharge to reduce localised accelerated erosion due to formation of fissures which concentrate wave energy; and
- Combinations of the above.

The long term regional recession is driven at sea level and can only be mitigated by reducing the energy approaching the coastline. In this case the following management options were considered:

- Reducing wave energy offshore before reaching the shoreline (e.g. offshore breakwater);
- Absorbing the energy at the shoreline (e.g. seawall); and
- Mitigating localised erosion at low water levels (adding rocks to reinforce the platform).

Based on these considerations the following actions have been recommended.

5.1.1 East Point North

At East Point North it is recommended that the following actions be undertaken (refer Figure 5-3):

- Immediate action to divert as much stormwater discharge as possible from the cliff face. The stormwater may be able to be redirected to adjacent shorelines. Expected cost over 1 year $311,000.
- Trial loose rock within the main platform fissure only at an estimated cost of $38,500 over 1 year. If successful continue placement across the site with estimated final cost of $231,000.
- In the longer term prepare for the construction of a seawall at a cost of $1,450,000 with timing dictated by results of rock trial and re-analysis of the long term cliff recession rate.

5.1.2 Nightcliff Central

At Nightcliff Central it is recommended that the following actions be undertaken (refer Figure 5-4):

- Trial loose 3 tonne rock within the platform fissure only at an estimated cost of $25,000 over 1 year.
- Investigate diverting as much stormwater discharge from the cliff face as possible. The stormwater may be able to be redirected to adjacent shorelines. Expected cost over 1 year $286,000.
- In the longer term prepare for the construction of a seawall at a cost of $504,000 with timing dictated by results of drainage works and the loose rock trial.

5.1.3 Nightcliff North

At Nightcliff North it is recommended that the following actions be undertaken (refer Figure 5-4):

Therefore it is recommended that the following actions be undertaken:

- Investigations be initiated to relocate as much stormwater discharge from the cliff face as possible. The stormwater should be redirected to adjacent shorelines with an expected cost over 1 year of $286,000.
- At the same time prepare for the construction of a seawall at a cost of $1,100,000 with timing dictated by results of drainage works and reassessed rates of cliff recession.

5.1.4 High Cost Alternative

In all three locations the option of an offshore breakwater was considered. Although considered effective these have associated costs in the order of $10,000,000 and will be difficult to construct in a macro-tidal environment. There are also aesthetic and possibly environmental implications.

5.1.5 Sea Level Rise

The predictions for climate change which impact on the above options are a seal level rise of 0.8m in 100 years and a possible increase in storm intensity. For the structure considered this is likely to have minor impact on rock size but may require the structure crests in be increased in elevation. This would be achieved through topping up of the structures over time.

5.2 Sandy Beaches

As the beaches are dynamically stable it is recommended that the current management options be continued (refer Figure 5-2). In the longer term, if climate change induced sea level rise is realised then the dunes may need to be raised to suit. The predicted future sea level rise is 0.8m in 100 years.

5.3 Seawalls

It is recommended that maintenance of the small rock and geofabric seawalls on Mindil Beach South be undertaken to return them to original specification. The design should take into account predicted future sea level rise of 0.8m in 100 years.

The low grouted rock revetment at East Point South should be replaced with a similar grouted wall with adequate provision for drainage of overland stormwater flow though the lower section of the wall (similar to retaining wall design) at a cost of $542,000.

All other rock seawalls along the Darwin City shoreline appear to be performing adequately.

5.4 Overwash

It is considered that raising the berm crest level is the option with a high likelihood of success and with least disruption to existing usage and the natural environment. This option would include the placement of beach sand along the top of the berm to raise the level by 300mm where overwash is occurring to replicate the natural process at a cost of $16,000 (refer Figure 5-4).

In the longer term, if predicted sea level rise is realised then the management option will need to be changed as the space available to berm raising by sand is limited. When sea level rise exceeds 0.3m (predicted in 50 years) then the option for path protection will need to change to an option which can be placed at steeper slopes i.e. rock.

At Vesteys Beach South where the path is adjacent to the shoreline it is recommended that the path must be relocated or raised. An allowance of $48,000 has been estimated for path relocation or raising.

5.5 Monitoring

Although the use of phogrammetry has not be successful in this study it is considered that this can be a very useful tool for Council to use to monitor beach and cliff movement in the future. The usefulness of the photogrammetry can be greatly increased by defining limited study areas (e.g. Mindil and Vesteys Beaches, East Point military area and Nightcliff) and requesting a high resolution historical analysis by the Department of Lands and Planning. It is expected that this analysis, using quality low level photography (say over the last 40 years) will be able to produce a better indication of recession rates than the existing 0.3m/a average from the existing analyses.

Monitoring will be useful into the future to assess the implications of sea level rise and other climate change processes on the Darwin shoreline.

Table 5-1 Recommended Action Plan

Location	Mindil South	Mindil North	Vesteys South	Vesteys North	East Point South	East Point North	Nightcliff South	Nightcliff Central	Nightcliff North	All Locations	All Locations
The Problem	Seawall structures damaged.	Nil	Inundation threat to pathway.	Nil	Damage to low revetment.	Cliff recession.	Inundation threat to pathway.	Cliff recession.	Cliff recession.	Project management to ensure satisfactory completion of management actions.	Maintenance of protection structures and beaches including monitoring by photogrammetry.
Proposed Action	Maintain slumped rock seawalls and rebuild geofabric seawall.	Continue beach scraping after each wet season.	Upgrade pathway to above inundation level.	Maintain rock seawalls.	Repair revetment including freshwater runoff drainage.	Redirect freshwater drainage where possible. Trial loose boulders. If unsuccessful build seawall.	Raise beach berm.	Redirect freshwater drainage where possible. Trial loose boulders. If unsuccessful build seawall.	Redirect freshwater drainage where possible. Build seawall.	Project Management.	Ongoing maintenance and monitoring.
The Outcome	Protection of public and private assets.	Protection of public assets.	Protection of public assets.	Protection of public and private assets.	Protection of public assets.	Protection of heritage assets	Reduction of inundation damage to path and loss of amenity.	Protection of public land, assets and preservation of amenity.	Protection of public land, assets and preservation of amenity.	Scheduled tasks completed on schedule and on budget to the satisfaction of the Council and community.	Protection of public land and, assets, preservation of amenity and understanding of cliff recession rates and beach changes.
Cost Estimates (based on 2012 costing, future years need to allow CPI increases)	Maintenance costs Slumped rock seawall $120k Maintenance costs Geofabric seawall $70k	Beach scraping and dune restoration (current practice)	Capital cost $48K	N/A (routine maintenance budget)	Capital cost $542k	Drainage $311k Loose Rock Trial $38.5k Full loose rock $240k Seawall $1.45M	Sand placed on berm $16k	Drainage $286k Loose Rock Trial $25k Seawall $0.5M	Drainage $286k Seawall $1.1M	5 year Project Management Program $250K/yr	Ongoing program at $250K/yr
Timing	0-1 year	Ongoing	0-1 year	Ongoing	0-1 year	2-5 years	0-1 year	2-5 years	2-5 years	0 – 5 years	Ongoing
Sea Level Rise (0.8m in 100 years)	Raise seawalls progressively 0.8m over 100 years	Raise dunes progressively 0.8m over 100 years	Raise berm progressively 0.8m over 100 years	Raise seawalls progressively 0.8m over 100 years	Raise seawalls progressively 0.8m over 100 years	Raise seawalls progressively 0.8m over 100 years	Raise berm progressively 0.8m over 100 years – change to rock	Raise seawalls progressively 0.8m over 100 years	Raise seawalls progressively 0.8m over 100 years	-	-

Title:
Action Key Sheet

Figure: 5-1

Rev: A

BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

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Filepath : I:\B18413_I_MJA_Darwin\DRG\Action_Keysheet.wor

Title:

Mindil and Vesteys Beaches - Recommended Actions

Figure: 5-2

Rev: A

N

0 400 800m

Approx. Scale

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Filepath : I:\B18413_I_MJA_Darwin\DRG\Mindil_and_Vesteys_Recommended_Actions.wor

Title:

East Point - Recommended Actions

Figure: 5-3

Rev: A

N

0 250 500m

Approx. Scale

BMT WBM

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Filepath : I:\B18413_I_MJA_Darwin\DRG\EastPoint_Recommended_Actions.wor

Title:
NightCliff - Recommended Actions

Figure: 5-4

Rev: A

N

0 400 800m

Approx. Scale

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Filepath : I:\B18413_I_MJA_Darwin\DRG\NightCliff_Recommended_Actions.wor

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APPENDIX A: CONSTRUCTION COSTS

RLB | Rider Levett Bucknall

DARWIN SEA DEFENCES

Preliminary Cost Estimate Report No. 1
Rev 2

5 February 2013

Prepared for: BMT WBM
Level 8, 200 Creek Street
Brisbane 4000

Prepared by: Rider Levett Bucknall NT Pty Ltd
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Our reference: J15044

RLB | Rider Levett Bucknall

DARWIN SEA DEFENCES

	COST $
1 EAST POINT NORTH	
Option 1	
Dump Rock in Fissures (per site)	38,445
Dump Rock in Fissures (all 6No sites)	230,670
Option 2	
Rationalise Drainage	310,893
Option 3	
Rubble Mound Seawall	1,449,597
Option 4	
Offshore Breakwater	10,782,662
2 NIGHTCLIFF CENTRAL	
Option 1	
Dump Rock in Depleted Platform	24,019
Option 2	
Rationalise Drainage	286,176
Option 3	
Rubble Mound Seawall	503,519
Option 4	
Offshore Breakwater	6,108,380
3 NIGHTCLIFF NORTH	
Option 1	
Rationalise Drainage	286,176
Option 2	
Rubble Mound Seawall	1,107,611
Option 3	
Offshore Breakwater	9,524,936
4 MINDIL BEACH SOUTH	
Small Rock Seawall	119,284
5 NIGHTCLIFF SOUTH	
Raising Berm Crest	16,158
6 VESTEYS BEACH SOUTH	
Raising Existing Path	48,013
7 EAST POINT SOUTH	
Low Grouted Rock Revetment	542,006

EXCLUSIONS

Goods and Services Tax
Professional Consultants Fees
Escalation after 30 June 2014
Works Other Than Noted
Finance Fees & Charges

DARWIN SEA DEFENCES - EAST POINT NORTH

Flag	Ref	Description	Unit	Qty	Rate	LAB. Cost $	Rate	PLANT Cost $	Rate	MAT. Cost $	Composite Rate		Composite Cost $
1		**EAST POINT NORTH**											
		Option 1											
		Dump Rock in Fissures (per site)											
	1	Supply 5T rocks	T	100					128.50	12,850			
	2	Transport Rocks	T	100			25.80	2,580					
	3	Place rocks in fissures	Hr	120	85.00	10,200	50.00	6,000					
	4	Miscellaneous (ie fencing)	Item	1	800.00	800	250.00	250	750.00	750			
	5	Mobilisation, demob & oncosts	Item	1	3,259.75	3,260	1,253.75	1,254	501.50	502			
		TOTAL	**T**	**100**							**384.45**	**$**	**38,445**
		Dump Rock in Fissures (all 6No sites)											
	1	Supply 5T rocks	T	600					128.50	77,100			
	2	Transport Rocks	T	600			25.80	15,480					
	3	Place rocks in fissures	Hr	720	85.00	61,200	50.00	36,000					
	4	Miscellaneous (ie fencing)	Item	6	800.00	4,800	250.00	1,500	750.00	4,500			
	5	Mobilisation, demob & oncosts	Item	1	19,558.50	19,559	7,522.50	7,523	3,009.00	3,009			
		TOTAL	**T**	**100**							**2,306.70**	**$**	**230,670**
		Option 2											
		Rationalise Drainage											
	1	Supply & Install trench drains	m	170	170.00	28,900	127.09	21,605	113.40	19,278			
	2	Rationalise piped stormwater system	Hr	1280	85.00	108,800	35.00	44,800	40.00	51,200			
	3	Miscellaneous (ie fencing)	Item	1	1,000.00	1,000	1,000.00	1,000	1,000.00	1,000			
	4	Mobilisation, demob & oncosts	Item	1	21,651.50	21,652	8,327.50	8,328	3,331.00	3,331			
		TOTAL	**Item**	**1**							**310,892.88**	**$**	**310,893**

Flag	Ref	Description	Unit	Qty	Rate	LAB. Cost $	Rate	PLANT Cost $	Rate	MAT. Cost $	Composite Rate	Composite Cost $
		Option 3										
		Rubble Mound Seawall										
	1	Supply 3T rocks	T	2678					89.80	240,440		
	2	Transport Rocks	T	2678			25.80	69,080				
	3	Place rocks in seawall	Hr	6800	85.00	578,000	47.50	323,000				
	4	Miscellaneous (ie geotextile)	Item	1	15,000.00	15,000	10,000.00	10,000	25,000.00	25,000		
	5	Mobilisation, demob & oncosts	Item	1	122,900.70	122,901	47,269.50	47,270	18,907.80	18,908		
		TOTAL	**m**	**170**							**8,527.04**	**$ 1,449,597**
		Option 4										
		Offshore Breakwater										
	1	Supply 5T rocks	T	8925					128.50	1,146,863		
	2	Transport Rocks	T	8925			29.80	265,965				
	3	Barge rocks out to site	Day	182			22,500.00	4,095,000				
	4	Place rocks in breakwater	Hr	20400	115.00	2,346,000	65.00	1,326,000				
	5	Preparation of seabed	Hr	320	115.00	36,800	255.00	81,600				
	6	Miscellaneous (ie geotextile)	Item	1	25,000.00	25,000	18,000.00	18,000	35,000.00	35,000		
	7	Mobilisation, demob & oncosts	Item	1	914,182.10	914,182	351,608.50	351,609	140,643.40	140,643		
		TOTAL	**m**	**170**							**63,427.42**	**$ 10,782,662**

Note: Above costs are exclusive of GST

Flag	Ref	Description	Unit	Qty	Rate	LAB. Cost $	Rate	PLANT Cost $	Rate	MAT. Cost $	Composite Rate		Composite Cost $
2		**NIGHTCLIFF CENTRAL**											
		Option 1											
		Dump Rock in Depleted Platform											
	1	Supply 3T rocks	T	60					89.80	5,388			
	2	Transport Rocks	T	60			25.80	1,548					
	3	Place rocks in depleted area	Hr	90	85.00	7,650	50.00	4,500					
	4	Miscellaneous (ie fencing)	Item	1	800.00	800	250.00	250	750.00	750			
	5	Mobilisation, demob & oncosts	Item	1	2,036.45	2,036	783.25	783	313.30	313			
		TOTAL	**T**	**100**							**240.19**	**$**	**24,019**
		Option 2											
		Rationalise Drainage											
	1	Supply & Install trench drains	m	100	170.00	17,000	127.09	12,709	113.40	11,340			
	2	Rationalise piped stormwater system	Hr	1280	85.00	108,800	35.00	44,800	40.00	51,200			
	3	Miscellaneous (ie fencing)	Item	1	1,000.00	1,000	1,000.00	1,000	1,000.00	1,000			
	4	Mobilisation, demob & oncosts	Item	1	24,262.55	24,263	9,331.75	9,332	3,732.70	3,733			
		TOTAL	**Item**	**1**							**286,175.75**	**$**	**286,176**
		Option 3											
		Rubble Mound Seawall											
	1	Supply 3T rocks	T	788					89.80	70,762			
	2	Transport Rocks	T	788			25.80	20,330					
	3	Place rocks in seawall	Hr	2500	85.00	212,500	47.50	118,750					
	4	Miscellaneous (ie geotextile)	Item	1	5,000.00	5,000	3,000.00	3,000	7,500.00	7,500			
	5	Mobilisation, demob & oncosts	Item	1	42,689.40	42,689	16,419.00	16,419	6,567.60	6,568			
		TOTAL	**m**	**50**							**10,070.38**	**$**	**503,519**

Flag	Ref	Description	Unit	Qty	Rate	LAB. Cost $	Rate	PLANT Cost $	Rate	MAT. Cost $	Composite Rate	Composite Cost $

Option 4

Offshore Breakwater

Flag	Ref	Description	Unit	Qty	Rate	LAB. Cost $	Rate	PLANT Cost $	Rate	MAT. Cost $	Composite Rate	Composite Cost $
	1	Supply 5T rocks	T	5250					128.50	674,625		
	2	Transport Rocks	T	5250			29.80	156,450				
	3	Barge rocks out to site	Day	98			22,500.00	2,205,000				
	4	Place rocks in breakwater	Hr	12000	115.00	1,380,000	65.00	780,000				
	5	Preparation of seabed	Hr	188	115.00	21,620	255.00	47,940				
	6	Miscellaneous (ie geotextile)	Item	1	15,000.00	15,000	10,500.00	10,500	20,500.00	20,500		
	7	Mobilisation, demob & oncosts	Item	1	517,884.25	517,884	199,186.25	199,186	79,674.50	79,675		
		TOTAL	**m**	**100**							**61,083.80**	**$ 6,108,380**

Note: Above costs are exclusive of GST

Flag	Ref	Description	Unit	Qty	Rate	LAB. Cost $	Rate	PLANT Cost $	Rate	MAT. Cost $	Composite Rate	Composite Cost $
	3	**NIGHTCLIFF NORTH**										
		Option 1										
		Rationalise Drainage										
	1	Supply & Install trench drains	m	100	170.00	17,000	127.09	12,709	113.40	11,340		
	2	Rationalise piped stormwater system	Hr	1280	85.00	108,800	35.00	44,800	40.00	51,200		
	3	Miscellaneous (ie fencing)	Item	1	1,000.00	1,000	1,000.00	1,000	1,000.00	1,000		
	4	Mobilisation, demob & oncosts	Item	1	24,262.55	24,263	9,331.75	9,332	3,732.70	3,733		
		TOTAL	**Item**	**1**							**286,175.75**	**$ 286,176**
		Option 2										
		Rubble Mound Seawall										
	1	Supply 3T rocks	T	1733					89.80	155,579		
	2	Transport Rocks	T	1733			25.80	44,711				
	3	Place rocks in seawall	Hr	5500	85.00	467,500	47.50	261,250				
	4	Miscellaneous (ie geotextile)	Item	1	11,000.00	11,000	6,600.00	6,600	16,500.00	16,500		
	5	Mobilisation, demob & oncosts	Item	1	93,906.15	93,906	36,117.75	36,118	14,447.10	14,447		
		TOTAL	**m**	**110**							**10,069.19**	**$ 1,107,611**

Flag	Ref	Description	Unit	Qty	Rate	LAB. Cost $	Rate	PLANT Cost $	Rate	MAT. Cost $	Composite Rate	Composite Cost $
		Option 3										
		Offshore Breakwater										
	1	Supply 5T rocks	T	7875					128.50	1,011,938		
	2	Transport Rocks	T	7875			29.80	234,675				
	3	Barge rocks out to site	Day	161			22,500.00	3,622,500				
	4	Place rocks in breakwater	Hr	18000	115.00	2,070,000	65.00	1,170,000				
	5	Preparation of seabed	Hr	282	115.00	32,430	255.00	71,910				
	6	Miscellaneous (ie geotextile)	Item	1	22,500.00	22,500	15,800.00	15,800	30,800.00	30,800		
	7	Mobilisation, demob & oncosts	Item	1	807,548.95	807,549	310,595.75	310,596	124,238.30	124,238		
		TOTAL	**m**	**150**							**63,499.57**	**$ 9,524,936**

Note: Above costs are exclusive of GST

DARWIN SEA DEFENCES - VARIOUS LOCATIONS

Flag	Ref	Description	Unit	Qty	Rate	LAB. Cost $	Rate	PLANT Cost $	Rate	MAT. Cost $	Composite Rate		Composite Cost $
4	**MINDIL BEACH SOUTH**												
	Small Rock Seawall												
	1	Supply 1T rocks (top up only)	T	263					67.50	17,753			
	2	Transport Rocks	T	263			25.80	6,785					
	3	Place rocks in existing seawall	Hr	375	85.00	31,875	47.50	17,813					
	4	Miscellaneous (ie geotextile)	Item	1	8,000.00	8,000	6,600.00	6,500	15,000.00	15,000			
	5	Mobilisation, demob & oncosts	Item	1	10,113.35	10,113	3,889.75	3,890	1,555.90	1,556			
		TOTAL	**m**	**150**							**795.23**	**$**	**119,284**
5	**NIGHTCLIFF SOUTH**												
	Raising Berm Crest												
	1	Grade existing berm	m	100	10.00	1,000	7.00	700					
	2	Supply & install sand	m3	150	17.00	2,550	18.00	2,700					
	4	Miscellaneous (ie fencing)	Item	1	200.00	200	100.00	6,500	400.00	400			
	5	Mobilisation, demob & oncosts	Item	1	1,370.20	1,370	527.00	527	210.80	211			
		TOTAL	**m**	**100**							**161.58**	**$**	**16,158**
6	**VESTEYS BEACH SOUTH**												
	Raising Existing Path												
	1	Remove existing path	m	150	60.50	9,075	47.00	7,050					
	2	Fill to raise existing path area	m3	162	68.00	11,016	146.00	23,652					
	3	Supply & install new path	m2	225	48.00	10,800	29.00	6,525	79.00	17,775			
	4	Miscellaneous (ie fencing)	Item	1	800.00	800	500.00	6,500	1,000.00	1,000			
	5	Mobilisation, demob & oncosts	Item	1	4,070.95	4,071	1,565.75	1,566	626.30	626			
		TOTAL	**m**	**150**							**320.09**	**$**	**48,013**

Flag	Ref	Description	Unit	Qty	Rate	LAB. Cost $	Rate	PLANT Cost $	Rate	MAT. Cost $	Composite Rate	Composite Cost $
7	**EAST POINT SOUTH**											
	Low Grouted Rock Revetment											
	1	Remove existing revetment	m	100	765.00	76,500	190.00	19,000				
	2	Supply & install Geotextile	m2	800	17.00	13,600			43.50	34,800		
	3	Supply & install sand bed	m2	800	1.00	800			0.50	400		
	4	Supply <1T Rocks	T	1200					67.50	81,000		
	6	Transport Rocks	T	1200			25.80	30,960				
	7	Place rocks in existing seawall	Hr	1500	85.00	127,500	47.50	71,250				
	8	Miscellaneous (ie fencing)	Item	1	3,000.00	3,000	2,500.00	6,500	6,000.00	6,000		
	9	Mobilisation, demob & oncosts	Item	1	45,952.40	45,952	17,674.00	17,674	7,069.60	7,070		
		TOTAL	**m**	**100**							**5,420.06**	**$ 542,006**

Note: Above costs are exclusive of GST

APPENDIX B: 1997 BEACH CROSS SECTIONS (COMLEY)

Beach Erosion Survey

The following results of the April 1997 beach survey are part of an ongoing beach monitoring program being undertaken by the Land Resource Conservation Branch of the DLPE. It involves the study of Darwin's coastline through the interpretation of both level surveying and static photography along three of Darwin's beaches: Mindil, Vesteys and Casuarina (Figure 2).

The surveys and photography has been based on permanent bench marks (Figure 1) established along these beaches at regular intervals (Figures 3 and 4). Surveys are conducted and photography taken twice a year, recording seasonal changes in the beach profiles.

A TOPCON GTS-3B10 instrument with a FC2 computer notebook was used to record the survey data. Readings were taken at each change of slope, or at around 50 metre intervals along each transect. The data was processed through the CIVILCAD 5.3 software package and beach cross-sections were constructed. Refer to *Survey and Civilcad 5.3 Operating Procedures*.

Photographs were taken of the beach in the direction of the bench mark from approximately 50 paces, using a PENTAX SLR camera with a 50mm lens.

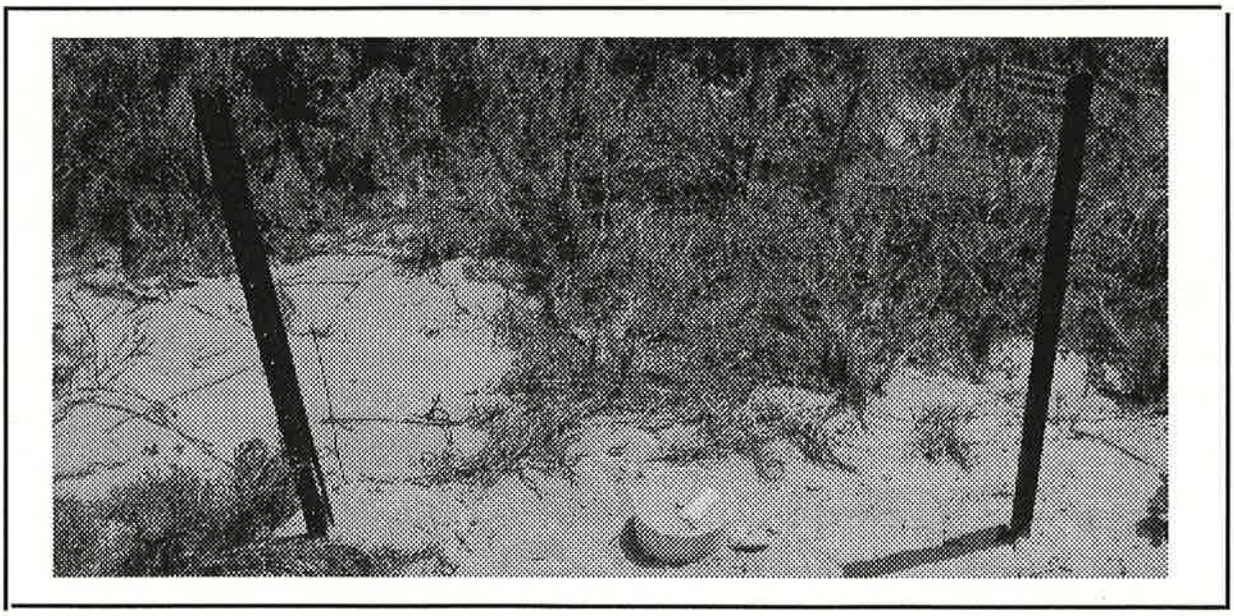


Figure 1: A Bench mark located on Casuarina Beach.

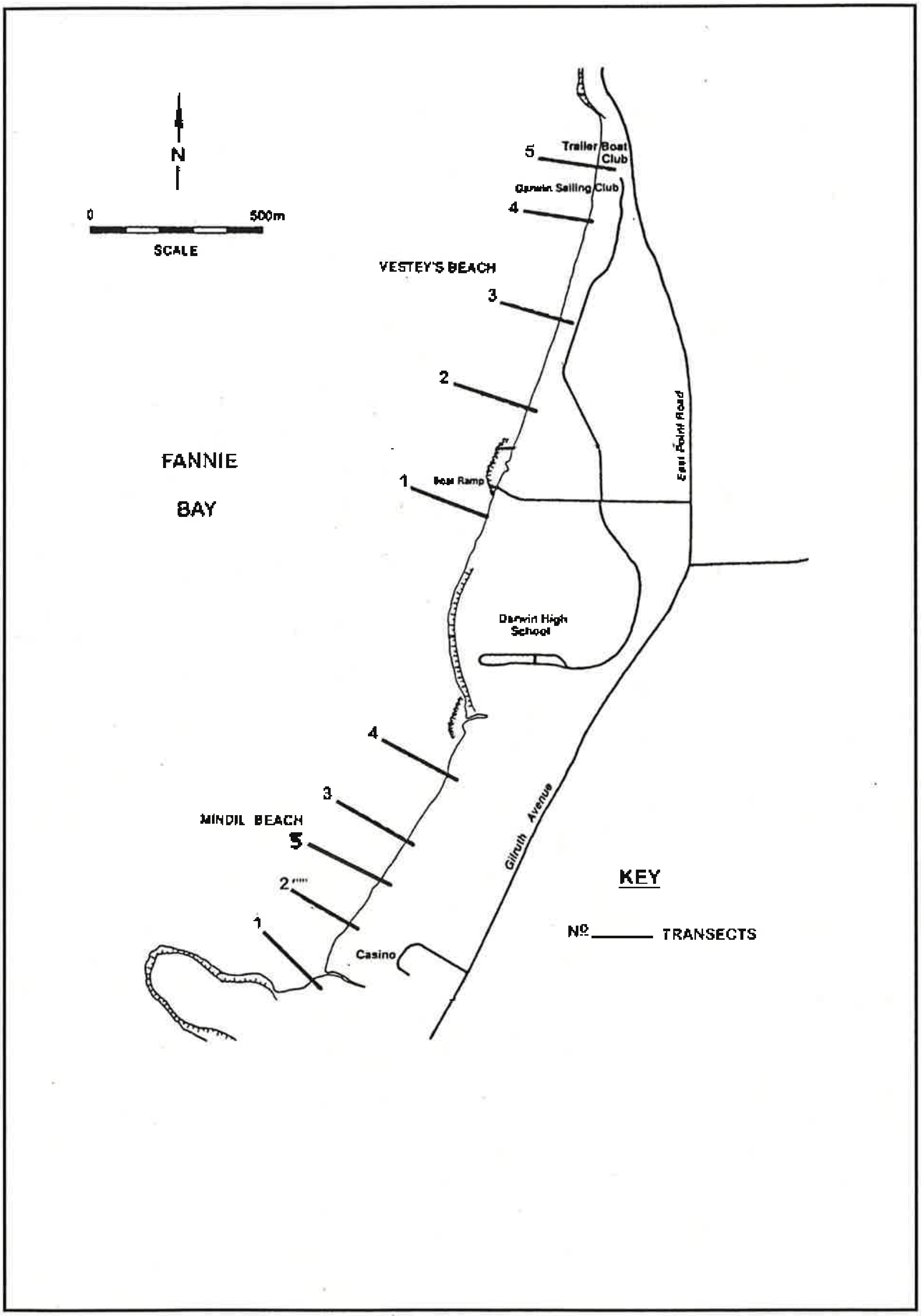


Figure 3: Position of Transects along Mindil and Vesteys beaches.

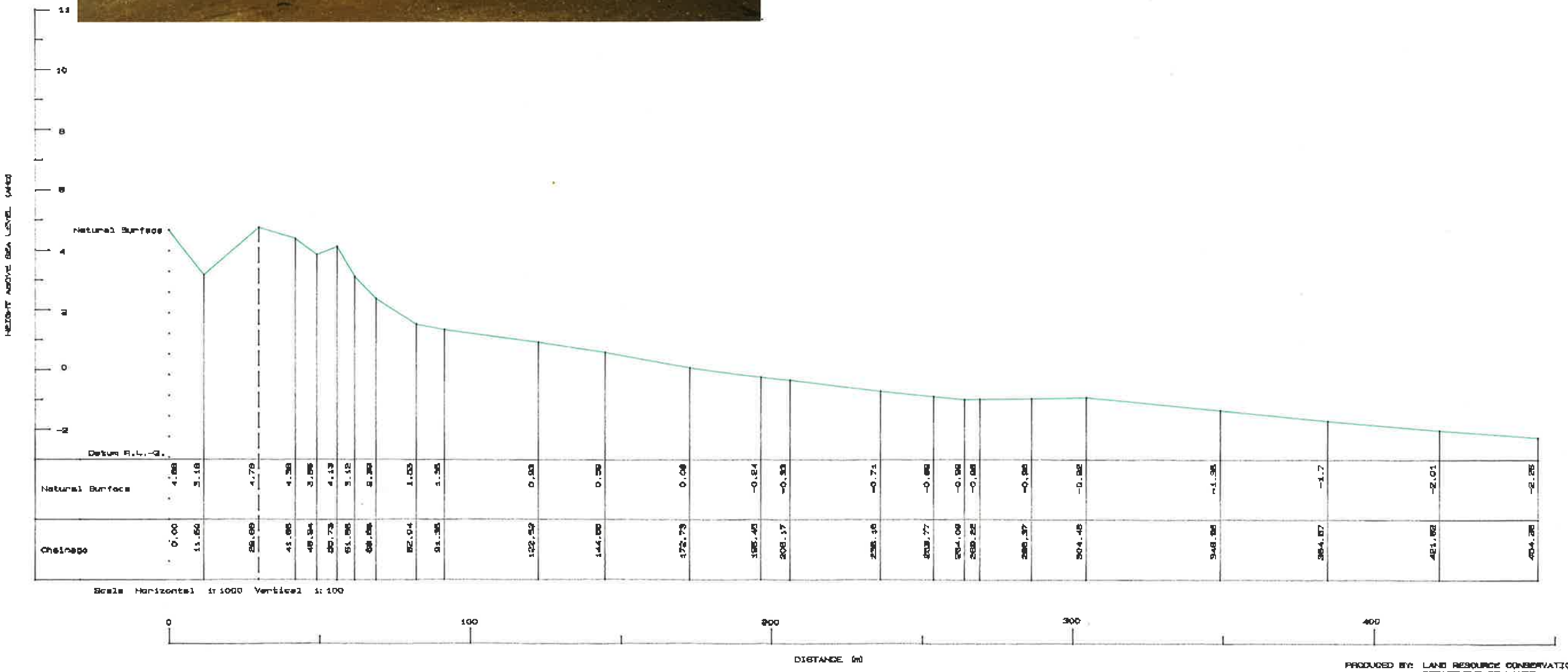
South along Mindil Beach from the walking track.

North along Mindil Beach from Myilly Point.

MINDIL BEACH
TRANSECT 1

. . . BENCH MARK

— — - OCCUPIED SITE



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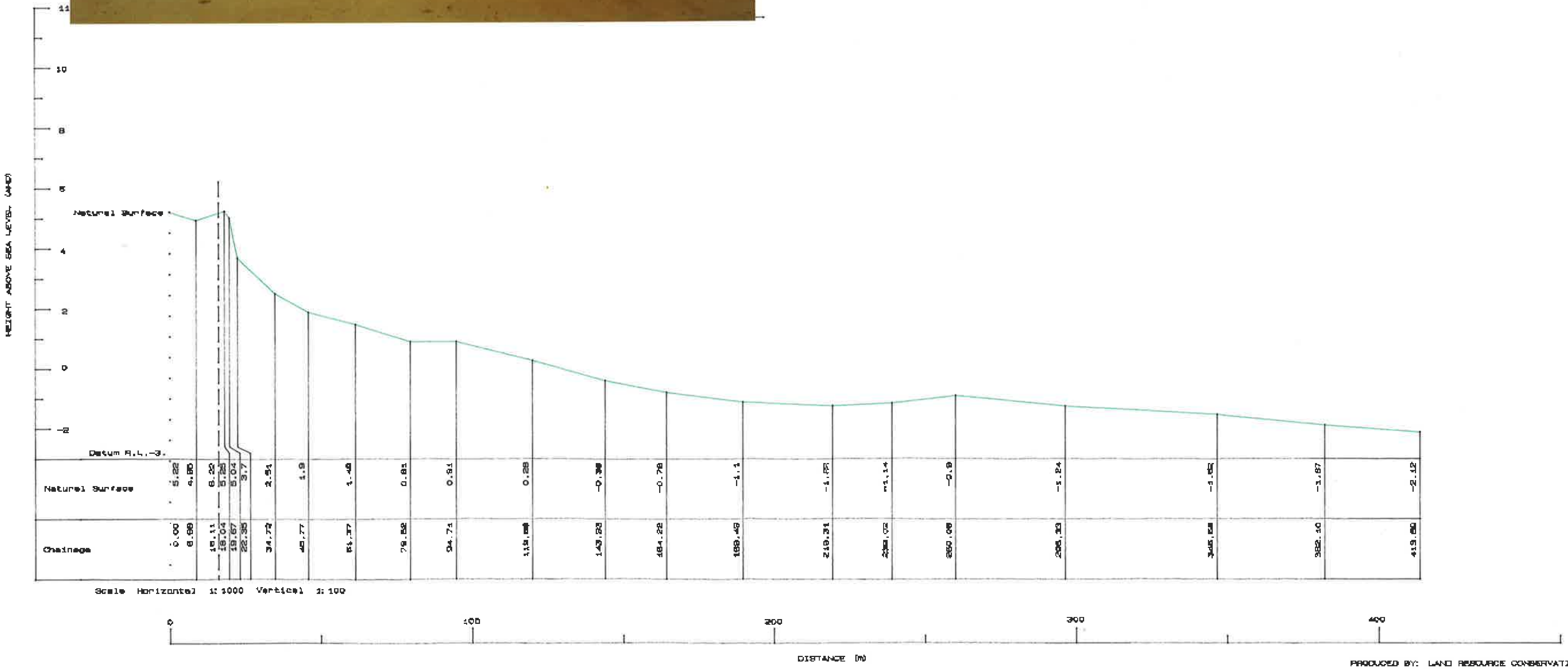
MINDIL BEACH
TRANSECT 2

. . . BENCH MARK

— — - OCCUPIED SITE



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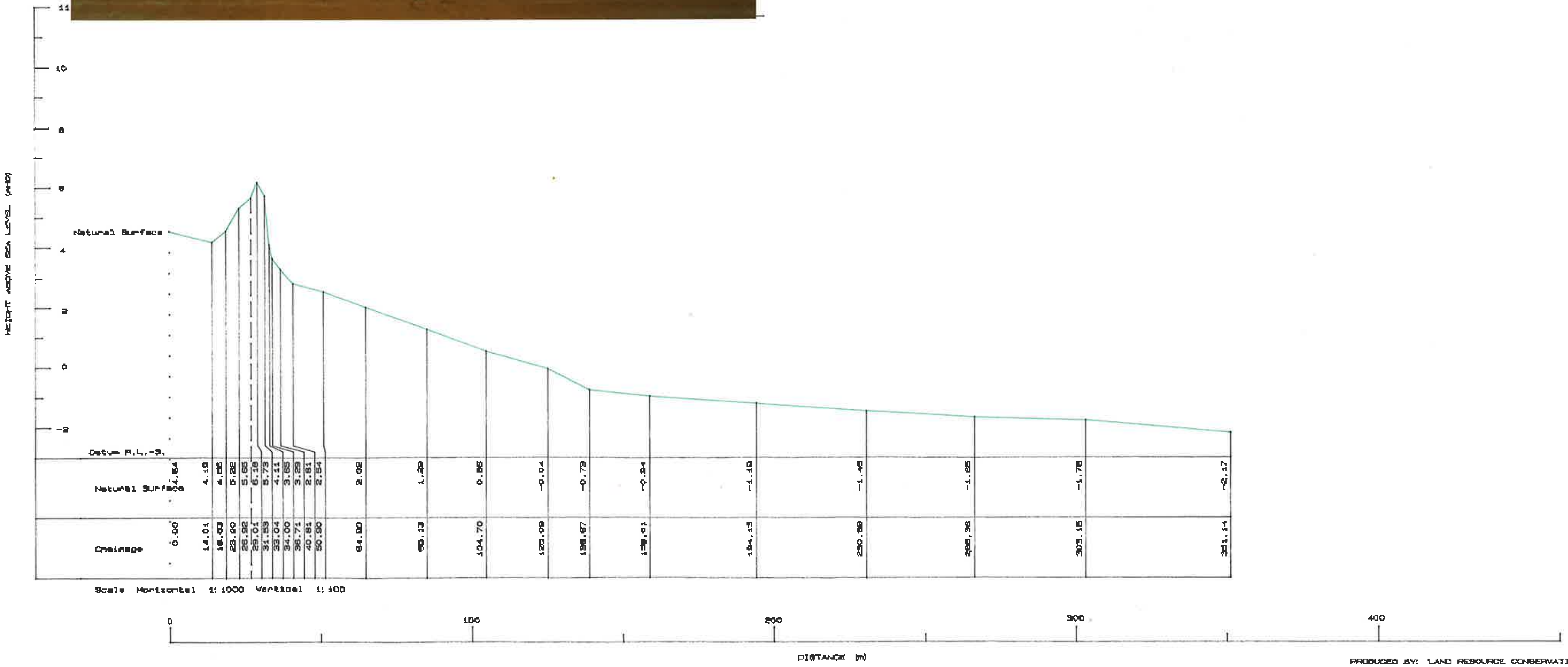
MINDIL BEACH TRANSECT 3

. . . BENCH MARK

- - - OCCUPIED SITES



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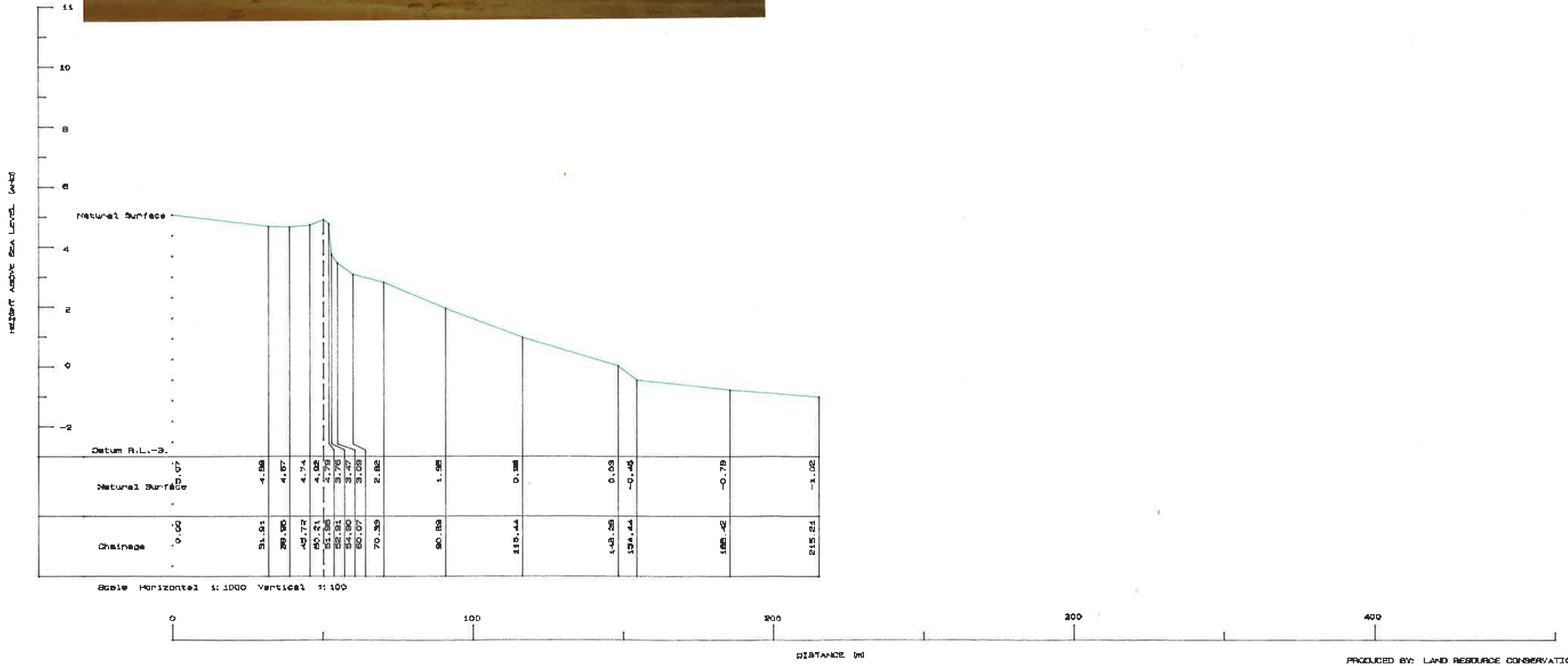
South from middle of Mindil Beach - Transect 3

North from middle of Mindil Beach - Transect 3

MINDIL BEACH
TRANSECT 4

. . . BENCH MARK

— — - OCCUPIED SITE



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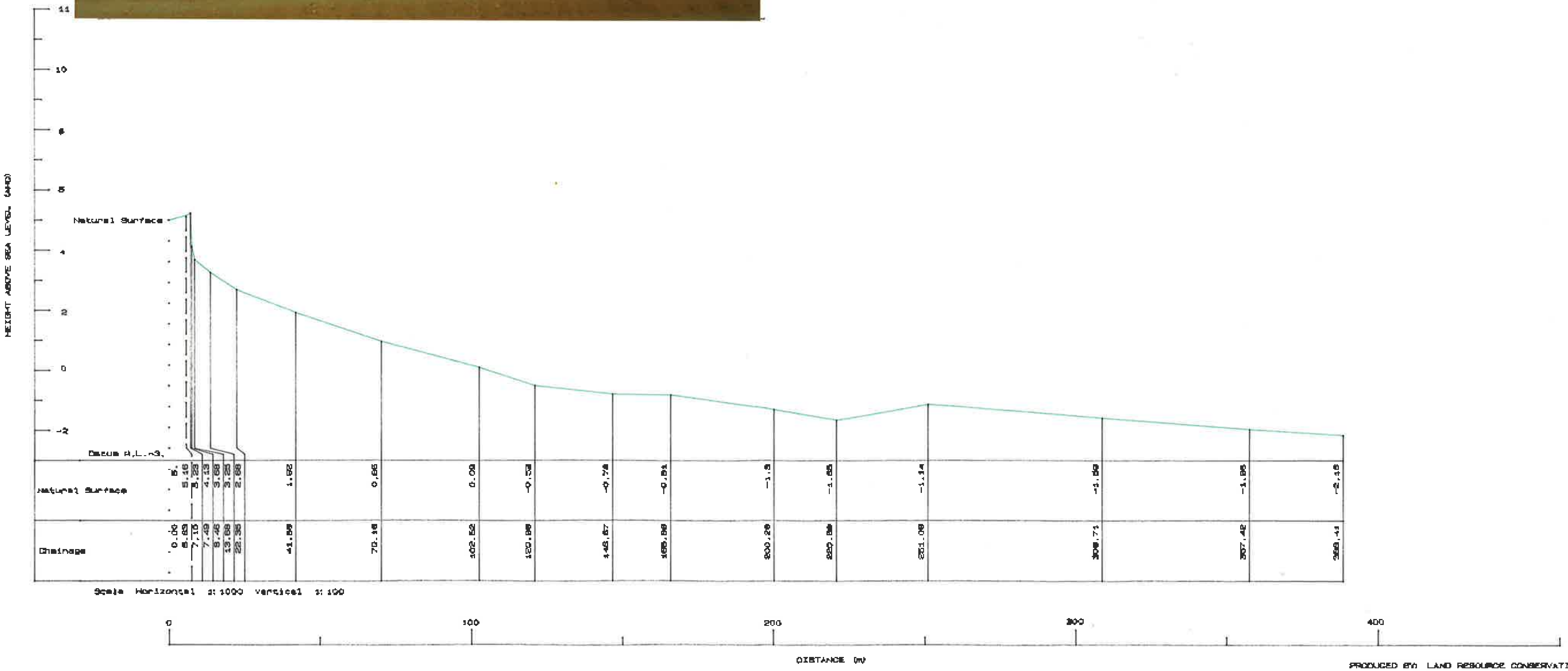
South along Mindil Beach - From transect 4

North from Mindil Beach - From transect 4

MINDIL BEACH
TRANSECT 5

. . . . BENCH MARK

— — - OCCUPIED SITE



Natural Surface	5.	5.16	5.23	4.13	3.68	3.25	2.68	1.92	0.66	0.09	-0.53	-0.78	-0.81	-1.3	-1.65	-1.14	-1.59	-1.85	-2.16
Chainage	0.00	6.63	7.15	7.49	8.46	13.68	22.36	41.89	70.16	102.52	120.86	146.67	165.86	200.26	220.89	251.06	308.71	357.42	389.41

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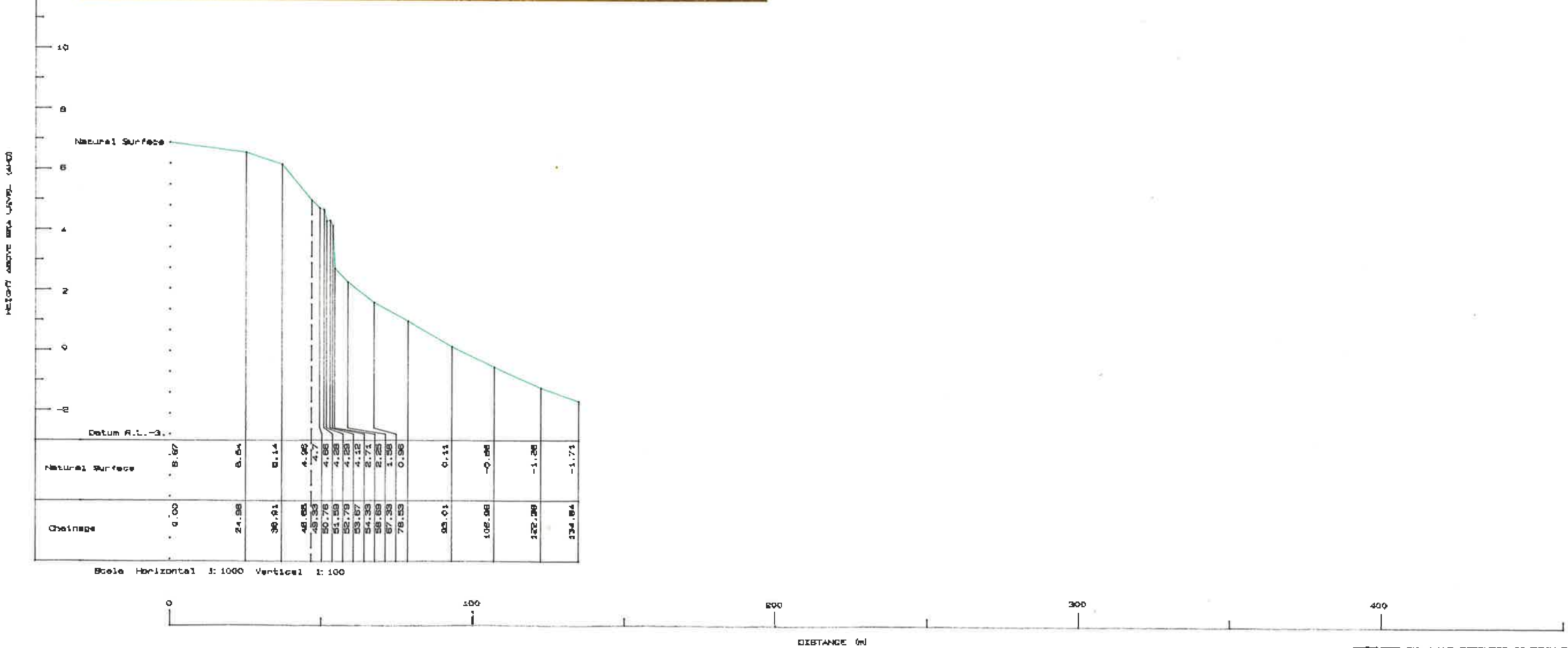
South along Mindil Beach - From transect 5

North from Mindil Beach - From transect 5

VESTEYS BEACH
TRANSECT 1

. . . BENCH MARK

— — - OCCUPIED SITE



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DRAWN BY: BRADLEY COMLEY.

SURVEYED: 24/04/97 BY BRADLEY COMLEY AND ROHAN FISHER.

Southern side of Vesteys Beach Ski Club Boat Ramp.

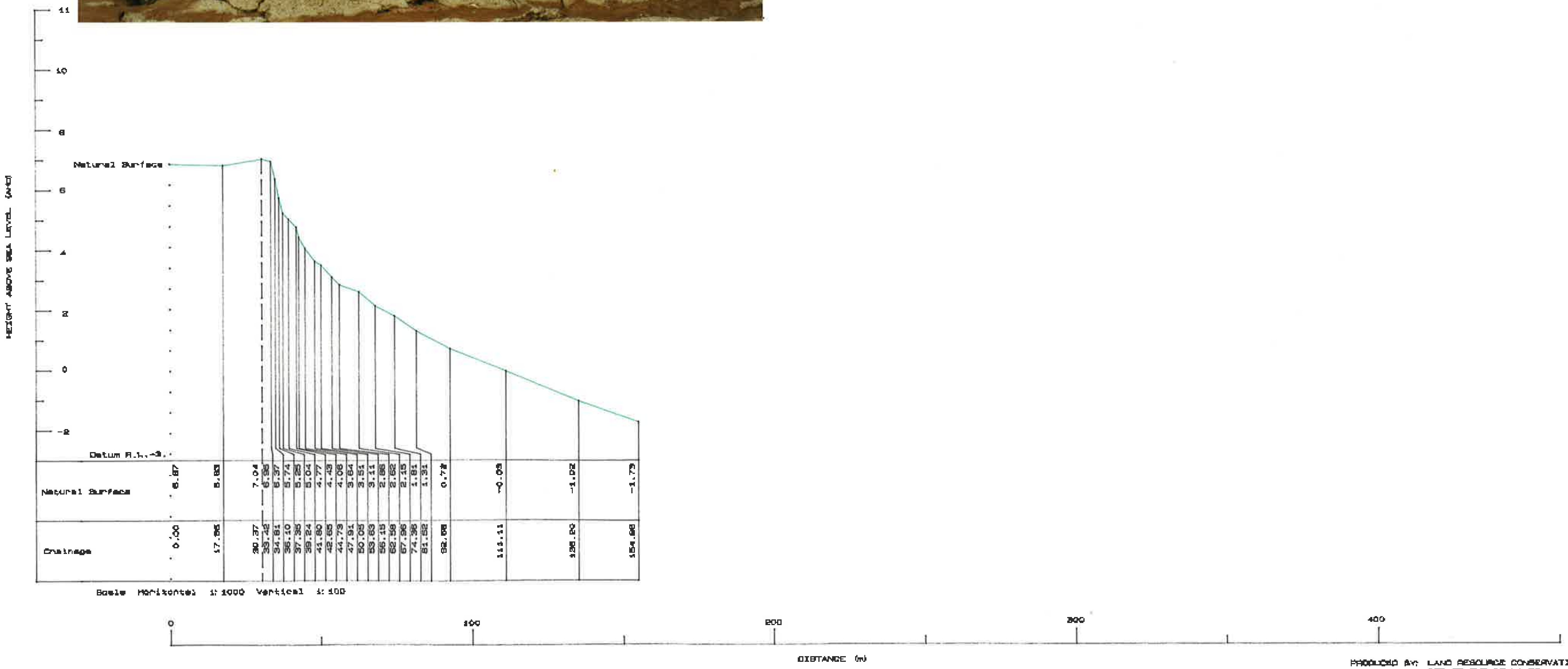
Northern side of Vesteys Beach Ski Club Boat Ramp.

North along seaward side of Vesteys Beach Ski Club, from Vesteys Beach Ski Club Boat Ramp.

VESTEYS BEACH
TRANSECT 2

. . . BENCH MARK

— — - OCCUPIED SITE



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SURVEYED: 24/04/97 BY BRADLEY COWLEY AND ROHAN FISHER.

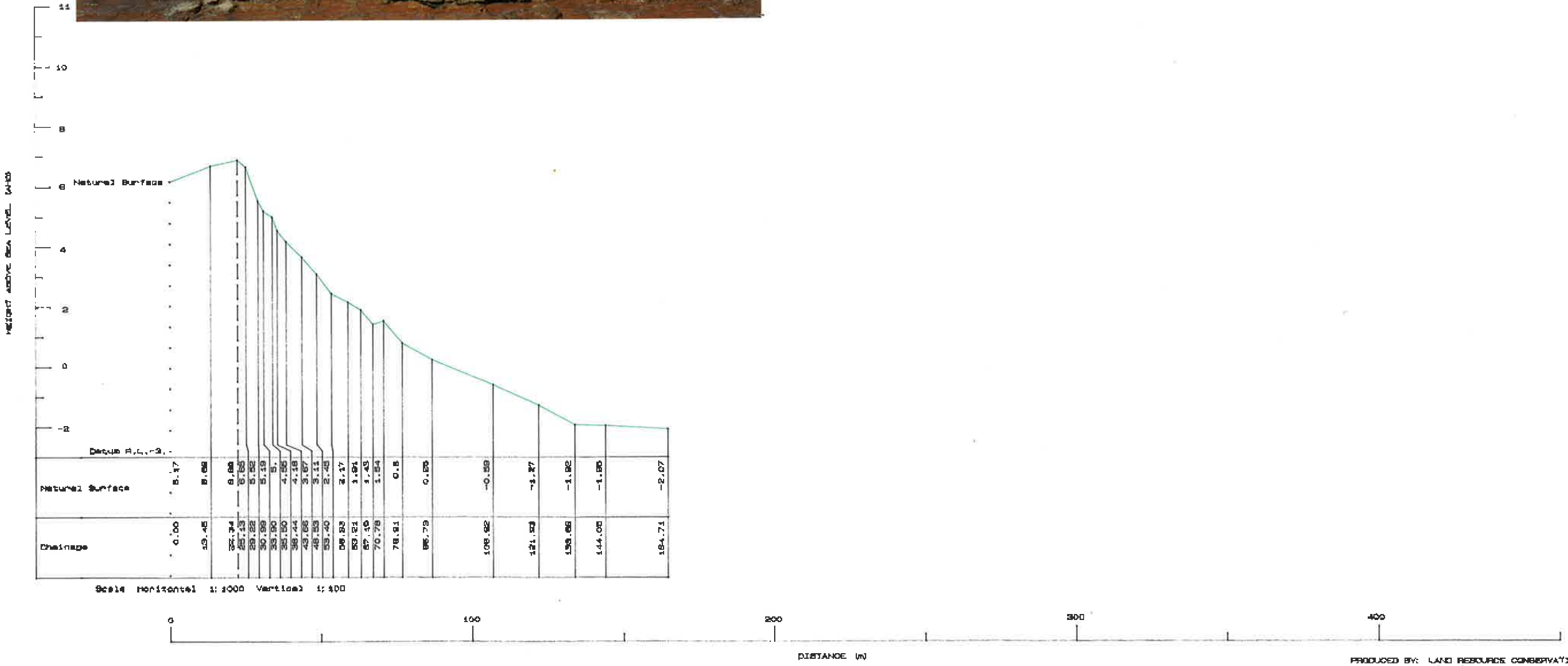
Southern side of Vesteys Beach Boat Ramp, between VB2 and VB3.

Northern side of Vesteys Beach Boat Ramp, between VB2 and VB3.

VESTEYS BEACH
TRANSECT 3

. . . BENCH MARK

— — - OCCUPIED SITE



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South from Transect 3, Vesteys Beach.

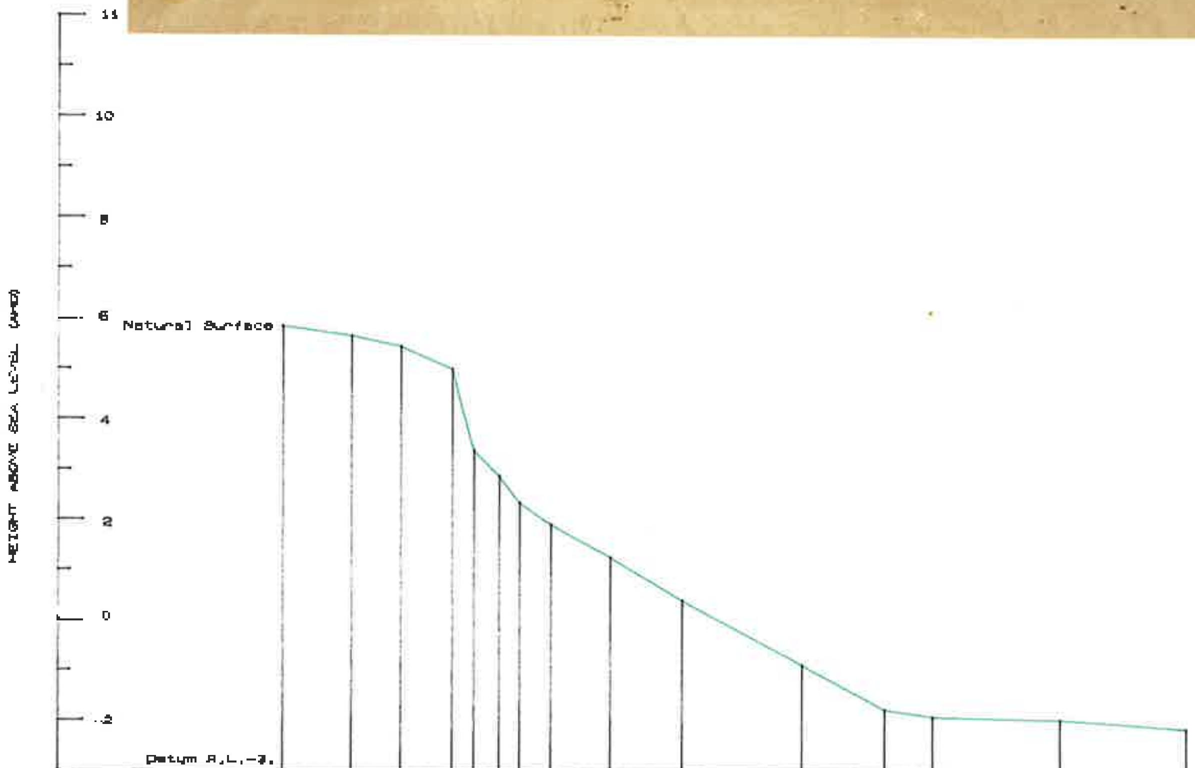
North from Transect 3, Vesteys Beach.

Vesteys Beach Yacht Club, boat ramp with revetment walls

VESTEYS BEACH
TRANSECT 4

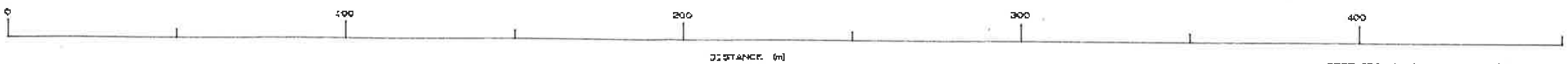
. . . BENCH MARK

— — - OCCUPIED SITE



Natural Surface	5.84	5.65	5.44	5.	3.35	2.88	2.34	1.83	1.26	0.41	-0.57	-1.76	-1.88	-1.92	-2.11
Chainage	0.00	13.54	28.32	33.50	37.80	42.52	45.87	49.40	64.68	79.14	102.88	119.77	128.76	153.33	178.59

Scale Horizontal 1:1000 Vertical 1:100



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Southern side of the Trailer Boat Club, boat ramp, Vesteys Beach between VB4 and VB5

Northern side of the Trailer Boat Club, boat ramp, Vesteys Beach between VB4 and VB5

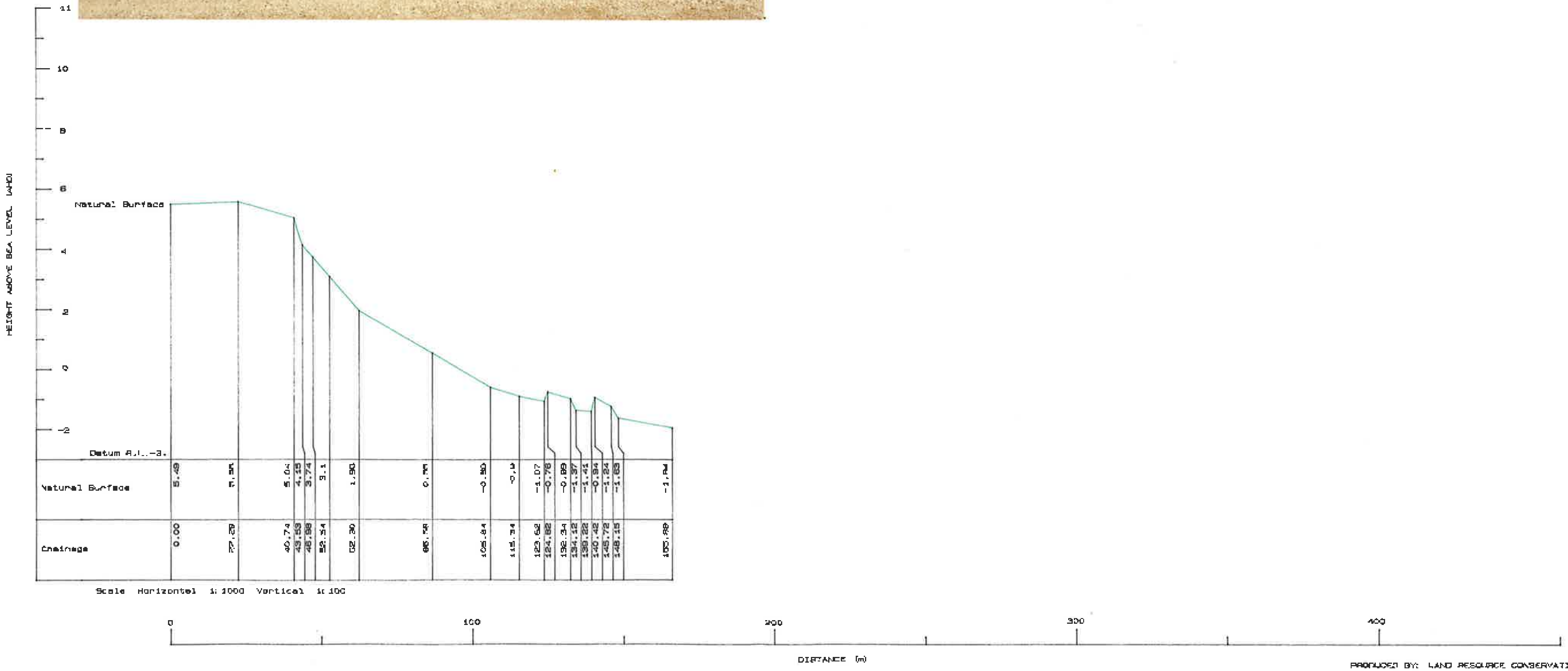
South from Top of Trailer Boat Club, Boat Ramp.

North from Top of Trailer Boat Club, Boat Ramp.

VESTEYS BEACH
TRANSECT 5

. . . BENCH MARK

- - - - OCCUPIED SITE



Natural Surface	5.49	5.55	5.04	4.15	3.74	3.1	1.90	0.55	-0.50	-0.8	-1.07	-0.78	-0.89	-1.37	-1.41	-0.94	-1.24	-1.63	-1.84
Chainage	0.00	22.28	40.74	43.53	45.98	52.54	62.30	86.58	105.84	115.34	123.52	124.82	132.34	134.12	138.22	140.42	145.72	148.15	165.88

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North from Transect 5, Vesteys Beach.

APPENDIX C: HUDSON'S FORMULA AND ROCK GRADING

Hudson's Formula is a preliminary design tool to assess the rock weight required to resist wave forces. Subsequent to this a basic too for assessing rock size to achieve the nominated weight is presented.

Table VI-5-22
Rock, Two-Layer Armored Non-Overtopped Slopes (Hudson 1974)

Irregular, head-on waves

$$\frac{H}{\Delta D_{n50}} = (K_D \cot\alpha)^{1/3} \quad \text{or} \quad M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad (\text{VI}-5-67)$$

where

H	Characteristic wave height (H_s or $H_{1/10}$)
D_{n50}	Equivalent cube length of median rock
M_{50}	Medium mass of rocks, $M_{50} = \rho_s D_{n50}^3$
ρ_s	Mass density of rocks
ρ_w	Mass density of water
Δ	$(\rho_s/\rho_w) - 1$
α	Slope angle
K_D	Stability coefficient

K_D-values by SPM 1977, $H = H_s$, for slope angles $1.5 \le \cot\alpha \le 3.0$. (Based entirely on regular wave tests.)

Stone shape	Placement	Damage, D [4]			
		0-5%		5-10%	10-15%
		Breaking waves [1]	Nonbreaking waves [2]	Nonbreaking waves	Nonbreaking waves
Smooth, rounded	Random	2.1	2.4	3.0	3.6
Rough angular	Random	3.5	4.0	4.9	6.6
Rough angular	Special [3]	4.8	5.5		

K_D-values by SPM 1984, $H = H_{1/10}$.

Stone shape	Placement	Damage, D [4] = 0-5%	
		Breaking waves [1]	Nonbreaking waves [2]
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special [3]	5.8	7.0

[1] Breaking waves means depth-limited waves, i.e., wave breaking takes place in front of the armor slope. (Critical case for shallow-water structures.)

[2] No depth-limited wave breaking takes place in front of the armor slope.

[3] Special placement with long axis of stone placed perpendicular to the slope face.

[4] D is defined according to SPM 1984 as follows: The percent damage is based on the volume of armor units displaced from the breakwater zone of active armor unit removal for a specific wave height. This zone extends from the middle of the breakwater crest down the seaward face to a depth equivalent to the wave height causing *zero* damage below still-water level.

7.4.4 Grading of Quarry Stone

When quarry stone is purchased from a commercial block stone quarry, gradation is usually according to national standards. For W.Europe, one is referred to data in the CUR/CIRIA Manual (Anonymous [1991]).

Converting mass into diameter is done on the basis of the well-known D_n (nominal diameter) method:

$$D_n = \sqrt[3]{M/\rho} \qquad (7.23)$$

In the Netherlands, such standardised gradings are presented in Table 7-8.

Mass	D_n
10-60 kg	0.16-0.30 m
10-200 kg	0.16-0.43 m
60-300 kg	0.30-0.49 m
300-1000 kg	0.49-0.72 m
1000-3000 kg	0.72-1.04 m
3000-6000 kg	1.04-1.31 m
6000-10 000 kg	1.31-1.55 m

Table 7-8, Dutch Standard Grading

If the stone is classified according to sieve diameter, one can determine D_s. Although sieving is not a practical method for the larger stones, one can establish a general relation:

$$D_n = 0.8\, D_s \qquad (7.24)$$

The grading of a stone class is often defined as D_{85}/D_{15}. Common values are:

Type of grading	D_{85}/D_{15}
Narrow	<1.5
Wide	1.5 – 2.5
Very wide (quarry run or riprap)	2.5 – 5 and more

Stability is usually investigated for normal wide grades. Very wide grades will result in slightly more damage than narrow and wide gradings. The very wide gradings can, however, easily lead to demixing or segregation, so that it is difficult to effectively control the quality of stone delivered.

APPENDIX D: DATA ANALYSIS

Analysis of Historic Variability of Forcing

The time series of Darwin coastal change has been evaluated within the context of historic variation of coastal forcing mechanisms.

	Extreme Winds	Sustained Winds	Tropical Cyclones	Mean Sea Level	Tides
	Examined wind speed %iles (non-directional)	Examined wind-drift plots (decadal)	Looked at TC proximity to Darwin (within 100km)	Peak > 0.2m above average	Inter-annual tidal cycle peaks
1941-1950	Strong	Anomalous 3pm (weak N)	(2) 1948 (gales)	No Data	No Data
1951-1960	Strong	Stronger W 3pm 1954-1955	(2)	No Data	No Data
1961-1970			(5) 1968 Bertha		1963, 1967
1971-1980			(4) 1974 Tracy	1974-1977	1972, [1976]
1981-1990			(7) 1981 Max 1981/4 1985 Gretel	1989	1981, 1985, 1989
1991-2000	Very Weak	Anomalous 9am (1991-1996 strong SE; 1997-2000 weak SE) 3pm Strong N, weak W	(2) 1999 Vance	1997, 1999-2000	[1994], 1998
2001-2011		Stronger E 9am	(2) 2006 Monica	2001, 2006, 2008-2009, 2011*	2003, 2007, 2012, (2013)

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